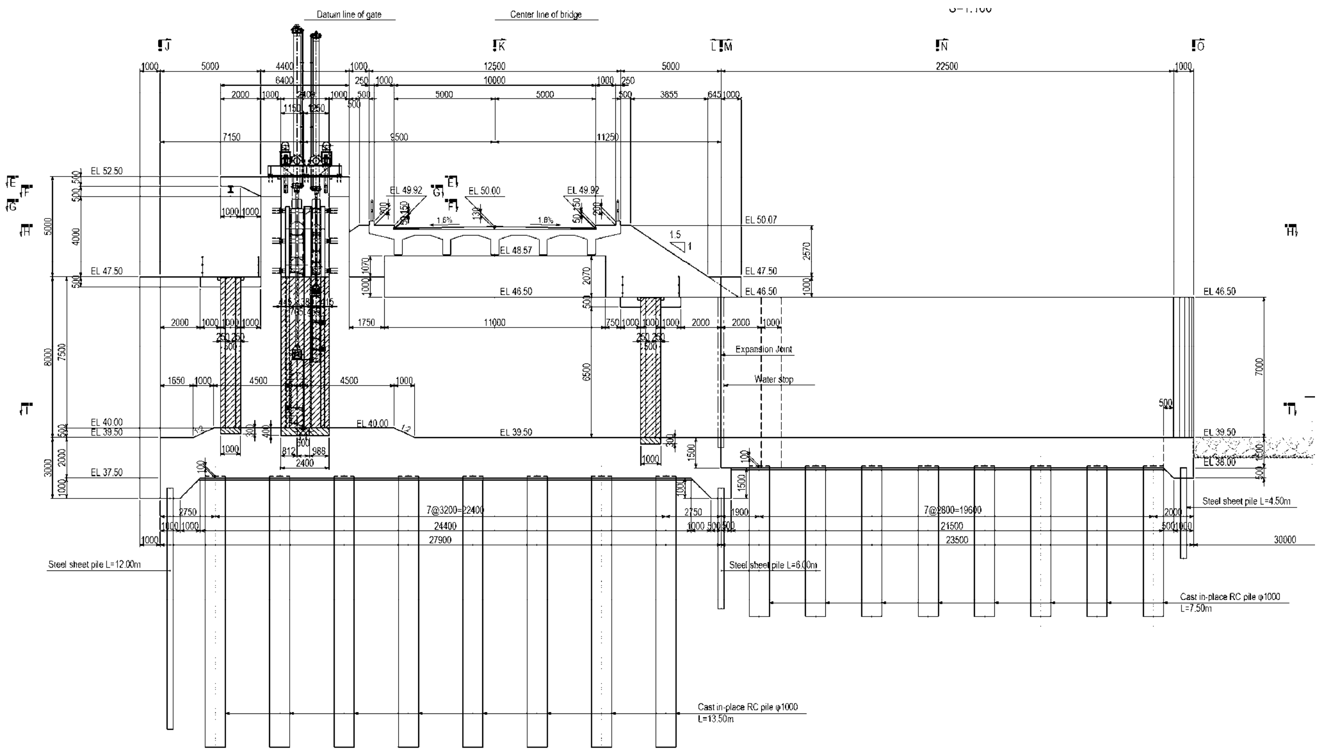
# APPENDIX A-7

## Structural Calculation of Pile Foundation

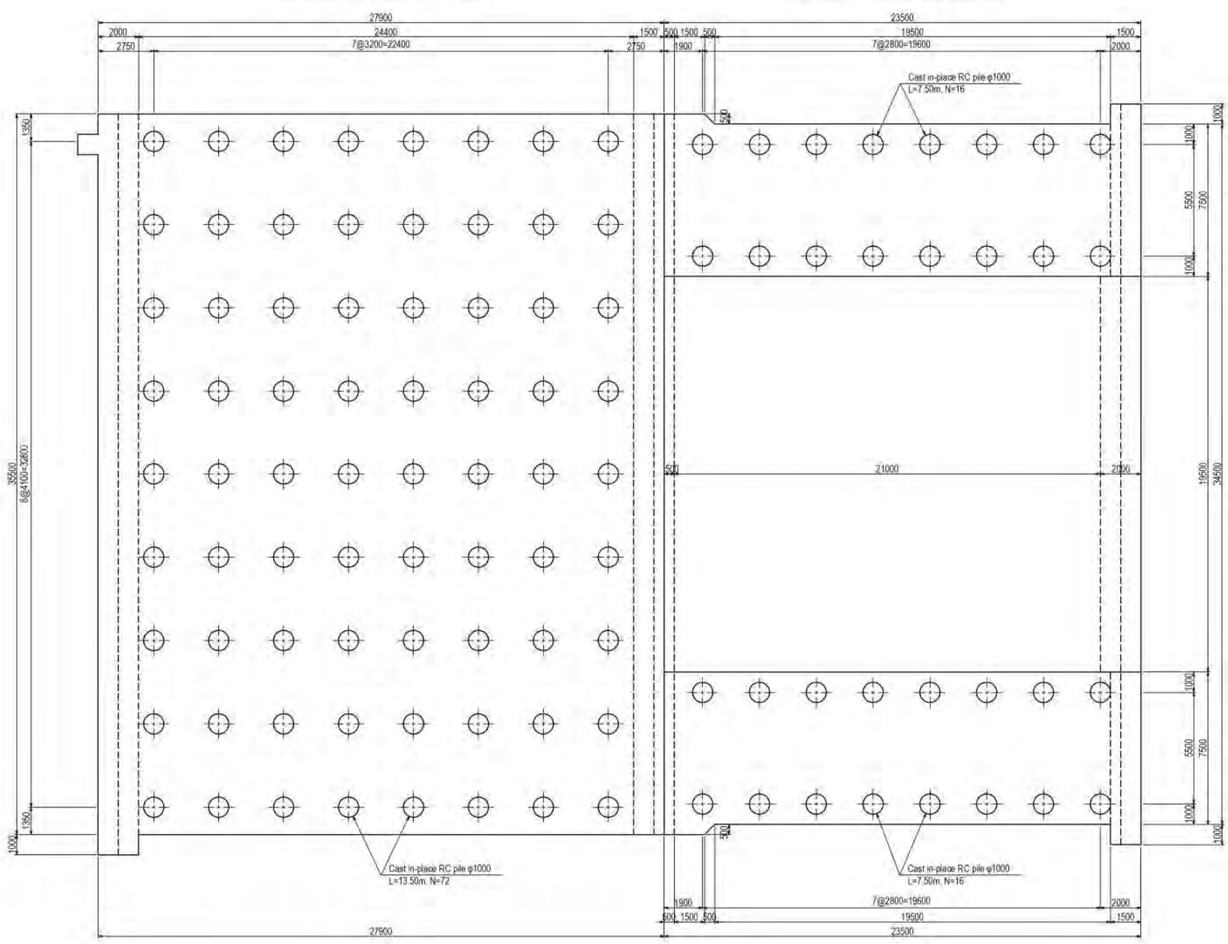
## A-7-1. Structural calculation of pile foundation for Bahr Yusef regulator

| Structure | Nos. of piles | dia. | Length of piles | note |
|---|---|---|---|---|
| Bahr Yusef reg. | 8x9=72 | 1,000mm | 13.5m | Cast in-place concrete pile |




Pile layout of Regulator

Pile layout of Ltype Wall



(Bahr Yusef Regulator)

1. Load data

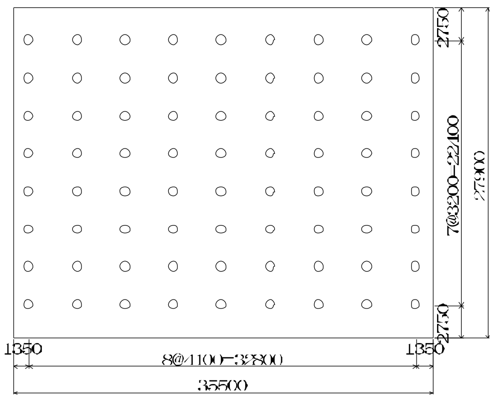
(1) Weir axis direction

| No | Load case | Extra coefficient | Design condition | Vertical force V (kN) | Horizontal force H (kN) | Bending moment M (kN·m) |
|---|---|---|---|---|---|---|
| 1 | Normal | 1.00 | Normal | 90408.54 | 724.42 | 8136.77 |
| 2 | Normal | 1.00 | Normal | 88729.08 | 1777.68 | 17745.82 |
| 3 | Normal | 1.00 | Normal | 134758.39 | 2506.86 | 16171.01 |
| 4 | Seismic | 1.15 | Seismic | 87525.54 | 14202.59 | 68269.92 |
| 5 | Seismic | 1.15 | Seismic | 85846.08 | 11091.17 | 52366.11 |
| 6 | Seismic | 1.15 | Seismic | 131875.39 | 10550.03 | 50112.65 |

(2) Weir axis right angle direction

| No | Load case | Extra coefficient | Design condition | Vertical force V (kN) | Horizontal force H (kN) | Bending moment M (kN·m) |
|---|---|---|---|---|---|---|
| 1 | Normal | 1.00 | Normal | 91358.69 | 883.93 | -43852.17 |
| 2 | Normal | 1.00 | Normal | 88729.08 | 14388.83 | -53237.45 |
| 3 | Normal | 1.00 | Normal | 134758.39 | 987.46 | -40427.52 |
| 4 | Seismic | 1.15 | Seismic | 85846.08 | 25761.42 | -12876.91 |
| 5 | Seismic | 1.15 | Seismic | 131875.39 | 10550.03 | 0.00 |
| 6 | Seismic | 1.15 | Seismic | 89965.54 | 10909.75 | -6297.59 |
| 7 | Normal | 1.00 | Normal | 80093.13 | 2811.86 | 79292.20 |

2. Pile arrangement




3.Ground spring

3-1.Pile spring constant for pile axis direction

$$K_V = a \cdot \frac{A_p \cdot E_p}{L}$$

$$= 0.265 \times \frac{0.785398 \times 25000000}{13.400}$$

$$= 388889.3 \text{ (kN/m)}$$

$K_V$ : Pile spring constant for pile axis direction (kN/m)

a : Correction coefficient

※ Inplace pile

a = 0.031 (L/D) - 0.150

= 0.031 × ( 13.900 / 1.000 ) - 0.150 = 0.281

$A_p$ : Vertical sectional area of the pile ($m^2$)

$E_p$ : Young's modulus of the pile ($kN/m^2$)

L : Insertion length of the pile (m)

D : Pile diameter (m)

3-2.Ground resistance coefficient for horizontal direction

Spring constant for pile axis right angle direction is calculated as following formula with considering multilayered ground:

(1) Calculation of the ground resistance coefficient for horizontal direction

$$k_H = k_{H0}\left(\frac{B_H}{0.3}\right)^{-3/4}$$

$k_H$ : Ground resistance coefficient for horizontal direction ($kN/m^3$)
$k_{H0}$ : Ground resistance coefficient for horizontal direction, which is equivalent to the plate lord test value by the 0.3m diameter rigid disc ($kN/m^3$)

$$k_{H0} = \frac{1}{0.3}\cdot\alpha\cdot E_0$$

$B_H$ : Conversion load width for the foundation (m)

$$B_H = \sqrt{\frac{D}{\beta}}$$

D : Pile diameter (m)
β : Characteristic value of the foundation ($m^{-1}$)

$$\beta = \sqrt[4]{\frac{k_H\cdot D}{4EI}}$$

α : Coefficient for the estimation of ground resistance coefficient
Normal $\alpha = 1$
$E_0$ : Coefficient for the ground deformation ($kN/m^2$)
EI : Bending rigidity of the foundation ($kN\cdot m^2$)

(2) Calculation of the conversion road width $B_H$

Key point on the calculation of $B_H$
・$k_H$ should be normal value when the calculating of $B_H$
・$k_H$ should be average value from the design ground surface to 1/β depth when the calculating of $B_H$ even if the layer shift dipper. Calculation of $k_H$ of each layer should be used $B_H$.

| | | |
|---|---|---|
| Pile diameter | D | 1.000 (m) |
| Young's modules | E | 25000000 ($kN/m^2$) |
| Section secondary moment | I | 0.04908739 ($m^4$) |

In assumption of 1/β = 3.636 (m) (β = 0.27505 ($m^{-1}$))
→Average $E_0$ = 33737.5 ($kN/m^2$)

$$B_H = \sqrt{\frac{1.000}{0.27505}} = 1.907 \text{ (m)}$$

$$k_H = \frac{1}{0.3}\times 1.0\times 33737.5\times\left(\frac{1.907}{0.3}\right)^{-3/4} = 28093.9 \text{ (kN/m}^3\text{)}$$

Calculation result of β is as follows, and this matches to the assumed β.

$$\beta = \sqrt[4]{\frac{28093.9\times 1.000}{4\times 25000000\times 0.04908739}} = 0.27505 \text{ (m}^{-1}\text{)}$$

Form the above, the conversion load width $B_H$ = 1.907 (m)

(3) Calculation of the ground resistance coefficient of each layer for horizontal direction

| | | |
|---|---|---|
| Pile diameter | D | 1.000 (m) |
| Young's mo | E | 25000000 (kN/m$^2$) |
| Section secondary moment | I | 0.04908739 (m$^4$) |

1) Normal

| No | Layer thickness (m) | $E_0$ (kN/m$^2$) | $k_{H0}$ (kN/m$^3$) | $k_H$ (kN/m$^3$) |
|---|---|---|---|---|
| 1 | 0.200 | 19600.0 | 65333.3 | 16321.3 |
| 2 | 1.000 | 16800.0 | 56000.0 | 13989.7 |
| 3 | 1.000 | 36400.0 | 121333.3 | 30311.0 |
| 4 | 1.000 | 44800.0 | 149333.3 | 37305.8 |
| 5 | 1.000 | 47600.0 | 158666.7 | 39637.4 |
| 6 | 1.000 | 36400.0 | 121333.3 | 30311.0 |
| 7 | 1.000 | 19600.0 | 65333.3 | 16321.3 |
| 8 | 1.000 | 30800.0 | 102666.7 | 25647.7 |
| 9 | 1.000 | 30800.0 | 102666.7 | 25647.7 |
| 10 | 1.000 | 25200.0 | 84000.0 | 20984.5 |
| 11 | 1.000 | 70000.0 | 233333.3 | 58290.3 |
| 12 | 1.000 | 81200.0 | 270666.7 | 67616.8 |
| 13 | 1.000 | 117600.0 | 392000.0 | 97927.8 |
| 14 | 1.000 | 120400.0 | 401333.3 | 100259.4 |
| 15 | 0.200 | 168000.0 | 560000.0 | 139896.8 |

Characteristic β : 0.27505 (m$^{-1}$)

2) Seismic

| No | Layer thickness (m) | $E_0$ (kN/m$^2$) | $k_{H0}$ (kN/m$^3$) | $k_H$ (kN/m$^3$) |
|---|---|---|---|---|
| 1 | 0.200 | 19600.0 | 130666.7 | 32642.6 |
| 2 | 1.000 | 16800.0 | 112000.0 | 27979.4 |
| 3 | 1.000 | 36400.0 | 242666.7 | 60621.9 |
| 4 | 1.000 | 44800.0 | 298666.7 | 74611.6 |
| 5 | 1.000 | 47600.0 | 317333.3 | 79274.9 |
| 6 | 1.000 | 36400.0 | 242666.7 | 60621.9 |
| 7 | 1.000 | 19600.0 | 130666.7 | 32642.6 |
| 8 | 1.000 | 30800.0 | 205333.3 | 51295.5 |
| 9 | 1.000 | 30800.0 | 205333.3 | 51295.5 |
| 10 | 1.000 | 25200.0 | 168000.0 | 41969.0 |
| 11 | 1.000 | 70000.0 | 466666.7 | 116580.7 |
| 12 | 1.000 | 81200.0 | 541333.3 | 135233.6 |
| 13 | 1.000 | 117600.0 | 784000.0 | 195855.5 |
| 14 | 1.000 | 120400.0 | 802666.7 | 200518.8 |
| 15 | 0.200 | 168000.0 | 1120000.0 | 279793.6 |

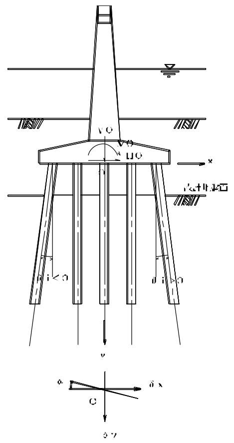
Characteristic β : 0.27505 (m$^{-1}$)

4. Calculation of the displacement and the resistance at pile head

4-1.Calculation method

(1) Calculation formula

Calculation method by the displacement way is to set coordinates as following figure and to set the center of the piles group as origin point 0. The external force acting to point 0 is set as following figure.



In this regard, displacement of the origin 0 is calculated from following equation with three unknowns.

$Axx \cdot \delta_x + Axy \cdot \delta_y + Axa \cdot \alpha = H_0$
$Ayx \cdot \delta_x + Ayy \cdot \delta_y + Aya \cdot \alpha = V_0$
$Aax \cdot \delta_x + Aay \cdot \delta_y + Aaa \cdot \alpha = M_0$

Each coefficient is calculated with the assumption of the footing bottom as flat.

$Axx = \sum (K_1 \cdot \cos^2\theta_i + K_V \cdot \sin^2\theta_i)$
$Axy = Ayx = \sum (K_V - K_1) \cdot \sin\theta_i \cdot \cos\theta_i$
$Axa = Aax = \sum \{(K_V - K_1) \cdot x_i \cdot \sin\theta_i \cdot \cos\theta_i - K_2 \cdot \cos\theta_i\}$
$Ayy = \sum (K_V \cdot \cos^2\theta_i + K_1 \cdot \sin^2\theta_i)$
$Aya = Aay = \sum \{(K_V \cdot \cos^2\theta_i + K_1 \cdot \sin^2\theta_i) \cdot xi + K_2 \cdot \sin\theta_i\}$
$Aaa = \sum \{(K_V \cdot \cos^2\theta_i + K_1 \cdot \sin^2\theta_i) \cdot xi^2 + (K_2 + K_3) \cdot x_i \cdot \sin\theta_i + K_4\}$

$H_0$ : Horizontal load acting to the upper portion of footing bottom (kN)
$V_0$ : Vertical load acting to the upper portion of the footing bottom (kN)
$M_0$ : Moment of the external force around the origin 0 (kN·m)
$\delta_x$ : Displacement of origin 0 for horizontal direction (m)
$\delta_y$ : Displacement of origin 0 for vertical direction (m)
$\alpha$ : Rotation angle of the footing (rad)
$x_i$ : X coordinate of the pile head at i(th) pile (m)
$\theta_i$ : Angle between pile axis and vertical axis at i(th) pile (degree)

From the result of displacement at footing origin point ($\delta_x$, $\delta_y$, $\alpha$), the force for pile axis direction $P_{Ni}$ at pile head, the force for pil eaxis right angle direction $P_{Hi}$ and moment $M_{ti}$ are formulated as following formula;

$P_{Ni} = K_V \cdot \delta_{yi}'$
$P_{Hi} = K_1 \cdot \delta_{xi}' - K_2 \cdot \alpha$
$M_{ti} = - K_3 \cdot \delta_{xi}' + K_4 \cdot \alpha$

$\delta_{xi}' = \delta_x \cdot \cos\theta_i - (\delta_y + \alpha \cdot x_i) \cdot \sin\theta_i$
$\delta_{yi}' = \delta_x \cdot \sin\theta_i + (\delta_y + \alpha \cdot x_i) \cdot \cos\theta_i$

$\delta_{xi}'$ : Displacement of pile head at i(th) pile for pile axis right angle direction (m)
$\delta_{yi}'$ : Displacement of pile head at i(th) pile for pile axis direction (m)
$K_V$ : Pile axis direction force occurring the displacement for pile axis direction at pile head(kN)
(Pile sprig coefficient for pile axis direction)
$K_1$、$K_2$、$K_3$、$K_4$ : Pile spring coefficient for pile axis right angle direction
$x_i$ : x coordinate at i(th) pile head(m)
$\theta_i$ : Angle degree between pile axis and vertical axis at i(th) pile (degree)
$P_{Ni}$ : Pile axis direction force at i(th) pile (kN)
$P_{Hi}$ : Pile axis right angle direction force at i(th) pile (kN)
$M_{ti}$ : Moment acting to the pile head at i(th) pile (kN·m)

Vertical resistance at pile head ($V_i$) and horizontal resistance at pile head ($H_i$) are calculated as following formula;

$V_i = P_{Ni} \cdot \cos\theta_i - P_{Hi} \cdot \sin\theta_i$
$H_i = P_{Ni} \cdot \sin\theta_i + P_{Hi} \cdot \cos\theta_i$

(2) Calculation of the pile spring coefficient

1) Normal

a) Pile conditions

Cast in-place concrete pile φ1000.0 (mm) L = 13.400 (m)
$K_V$ = 388889.3 (kN/m)

b) Pile spring coefficient for pile axis right angle direction

Pile head fixed connection

| No | $K_1$ (kN/m) | $K_2$ (kN/rad) | $K_3$ (kN·m/m) | $K_4$ (kN·m/rad) |
|---|---|---|---|---|
| 1 | 95155.0 | 191901.4 | 191901.4 | 693550.4 |

Pile head hinge

| No | $K_1$ (kN/m) | $K_2$ (kN/rad) | $K_3$ (kN·m/m) | $K_4$ (kN·m/rad) |
|---|---|---|---|---|
| 1 | 42057.0 | 0.0 | 0.0 | 0.0 |

2) Seismic

a) Pile conditions

Cast in-place concrete pile φ1000.0 (mm) L = 13.400 (m)
$K_V$ = 388889.3 (kN/m)

b) Pile spring coefficient for pile axis right angle direction

Pile head fixed connection

Pile head fixed connection

| No | $K_1$ (kN/m) | $K_2$ (kN/rad) | $K_3$ (kN·m/m) | $K_4$ (kN·m/rad) |
|---|---|---|---|---|
| 1 | 153954.8 | 268258.2 | 268258.2 | 827146.1 |

Pile head hinge

| No | $K_1$ (kN/m) | $K_2$ (kN/rad) | $K_3$ (kN·m/m) | $K_4$ (kN·m/rad) |
|---|---|---|---|---|
| 1 | 66953.9 | 0.0 | 0.0 | 0.0 |

$K_1$、$K_3$ : Pile axis right angle direction force (kN/m) and bending moment (kN·m/m), that should be acted to the pile head in order to displace the pile head for unit quantity without rotation.
$K_2$、$K_4$ : Pile axis right angle direction force (kN/m) and bending moment (kN·m/m), that should be acted to the pile head in order to rotate the pile head for unit quantity without displacement.

(3) Rigidity matrix (fixed)

1) Weir axis direction

a) Normal

Pile head rigid connection

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 6851161.0 & 0.0 & -13816898.0 \\ 0.0 & 28000030.0 & 5.9 \\ -13816898.0 & 5.9 & 3187805952.0 \end{bmatrix}$$

Pile head hinge

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 3028106.3 & 0.0 & 0.0 \\ 0.0 & 28000030.0 & 5.9 \\ 0.0 & 5.9 & 3137870336.0 \end{bmatrix}$$

b) Seismic

Pile head rigid connection

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 11084745.0 & 0.0 & -19314588.0 \\ 0.0 & 28000030.0 & 5.9 \\ -19314588.0 & 5.9 & 3197421640.0 \end{bmatrix}$$

Pile head hinge

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 1820680.5 & 0.0 & 0.0 \\ 0.0 & 28000030.0 & 5.9 \\ 0.0 & 5.9 & 3137870336.0 \end{bmatrix}$$

2) Weir axis right angle direction

a) Normal

Pile head rigid connection

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 6851161.0 & 0.0 & -13816898.0 \\ 0.0 & 28000030.0 & 6.7 \\ -13816898.0 & 6.7 & 1555217280.0 \end{bmatrix}$$

Pile head hinge

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 3028106.3 & 0.0 & 0.0 \\ 0.0 & 28000030.0 & 6.7 \\ 0.0 & 6.7 & 1505281664.0 \end{bmatrix}$$

b) Seismic

Pile head rigid connection

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 11084745.0 & 0.0 & -19314588.0 \\ 0.0 & 28000030.0 & 6.7 \\ -19314588.0 & 6.7 & 1564836224.0 \end{bmatrix}$$

Pile head hinge

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 4820680.5 & 0.0 & 0.0 \\ 0.0 & 28000030.0 & 6.7 \\ 0.0 & 6.7 & 1505281661.0 \end{bmatrix}$$

4-2.Pile head displacement and pile head resistance

(1) Weir axis direction

1) Normal

a) Pile head rigid connection

・Acting force for center of pile group
V0 = 90408.54 (kN)
H0 = 724.42 (kN)
M0 = 8136.77 (kN･m)

・Calculation of displacement
$\delta x$ = 0.112 (mm)
$\delta y$ = 3.229 (mm)
$\alpha$ = $0.00304 \times 10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_N$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN･m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -16.400 | 8 | -0.0 | 1236.30 | 10.06 | -19.36 | 1236.30 | 10.06 |
| 2 | -12.300 | 8 | -0.0 | 1241.15 | 10.06 | -19.36 | 1241.15 | 10.06 |
| 3 | -8.200 | 8 | -0.0 | 1245.99 | 10.06 | -19.36 | 1245.99 | 10.06 |
| 4 | -4.100 | 8 | -0.0 | 1250.83 | 10.06 | -19.36 | 1250.83 | 10.06 |
| 5 | 0.000 | 8 | -0.0 | 1255.67 | 10.06 | -19.36 | 1255.67 | 10.06 |
| 6 | 4.100 | 8 | -0.0 | 1260.52 | 10.06 | -19.36 | 1260.52 | 10.06 |
| 7 | 8.200 | 8 | -0.0 | 1265.36 | 10.06 | -19.36 | 1265.36 | 10.06 |
| 8 | 12.300 | 8 | -0.0 | 1270.20 | 10.06 | -19.36 | 1270.20 | 10.06 |
| 9 | 16.400 | 8 | -0.0 | 1275.05 | 10.06 | -19.36 | 1275.05 | 10.06 |

$\delta$ max = 0.112 (mm) < $\delta$ a = 15.00 (mm) OK
PNmax = 1275.05 (kN) < Ra = 1993.31 (kN) OK
PNmin = 1236.30 (kN) > Pa = -920.98 (kN) OK

2) Normal

a) Pile head rigid connection

・Acting force for center of pile group
V0 = 88729.08 (kN)
H0 = 1777.68 (kN)
M0 = 17745.82 (kN･m)

・Calculation of displacement
$\delta x$ = 0.273 (mm)
$\delta y$ = 3.169 (mm)
$\alpha$ = $0.00675 \times 10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_N$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN･m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | 16.400 | 8 | 0.0 | 1189.30 | 24.69 | 47.72 | 1189.30 | 24.69 |
| 2 | -12.300 | 8 | -0.0 | 1200.06 | 24.69 | -47.72 | 1200.06 | 24.69 |
| 3 | -8.200 | 8 | -0.0 | 1210.82 | 24.69 | -47.72 | 1210.82 | 24.69 |
| 4 | -4.100 | 8 | -0.0 | 1221.59 | 24.69 | -47.72 | 1221.59 | 24.69 |
| 5 | 0.000 | 8 | -0.0 | 1232.35 | 24.69 | -47.72 | 1232.35 | 24.69 |
| 6 | 4.100 | 8 | -0.0 | 1243.11 | 24.69 | -47.72 | 1243.11 | 24.69 |
| 7 | 8.200 | 8 | -0.0 | 1253.87 | 24.69 | -47.72 | 1253.87 | 24.69 |
| 8 | 12.300 | 8 | -0.0 | 1264.64 | 24.69 | -47.72 | 1264.64 | 24.69 |
| 9 | 16.400 | 8 | -0.0 | 1275.40 | 24.69 | -47.72 | 1275.40 | 24.69 |

$\delta_{max}$ = 0.273 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1275.40 (kN) < $R_a$ = 1993.31 (kN) OK
$P_{Nmin}$ = 1189.30 (kN) > $P_a$ = -920.98 (kN) OK

3) Normal

a) Pile head rigid connection

・Acting force for center of pile group
V0 = 134758.39 (kN)
H0 = 2506.86 (kN)
M0 = 16171.01 (kN･m)

・Calculation of displacement
$\delta x$ = 0.379 (mm)
$\delta y$ = 4.813 (mm)
$\alpha$ = $0.00672 \times 10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_N$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN･m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -16.400 | 8 | -0.0 | 1189.30 | 24.69 | -47.72 | 1189.30 | 24.69 |
| 2 | -12.300 | 8 | -0.0 | 1200.06 | 24.69 | -47.72 | 1200.06 | 24.69 |
| 3 | -8.200 | 8 | -0.0 | 1210.82 | 24.69 | -47.72 | 1210.82 | 24.69 |
| 4 | -4.100 | 8 | -0.0 | 1221.59 | 24.69 | -47.72 | 1221.59 | 24.69 |
| 5 | 0.000 | 8 | -0.0 | 1232.35 | 24.69 | -47.72 | 1232.35 | 24.69 |
| 6 | 4.100 | 8 | -0.0 | 1243.11 | 24.69 | -47.72 | 1243.11 | 24.69 |
| 7 | 8.200 | 8 | -0.0 | 1253.87 | 24.69 | -47.72 | 1253.87 | 24.69 |
| 8 | 12.300 | 8 | -0.0 | 1264.64 | 24.69 | -47.72 | 1264.64 | 24.69 |
| 9 | 16.400 | 8 | -0.0 | 1275.40 | 24.69 | -47.72 | 1275.40 | 24.69 |

$\delta_{max}$ = 0.379 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1914.49 (kN) < $R_a$ = 1993.31 (kN) OK
$P_{Nmin}$ = 1828.80 (kN) > $P_a$ = -920.98 (kN) OK

4) Seismic

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 87525.54 (kN)
$H_0$ = 14202.59 (kN)
$M_0$ = 68269.92 (kN·m)

・Calculation of displacement
$\delta_x$ = 1.497 (mm)
$\delta_y$ = 3.517 (mm)
$\alpha$ = $0.03197\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -16.400 | 8 | -0.0 | 1028.12 | 197.26 | -333.14 | 1028.12 | 197.26 |
| 2 | -12.300 | 8 | -0.0 | 1075.00 | 197.26 | -333.14 | 1075.00 | 197.26 |
| 3 | -8.200 | 8 | -0.0 | 1121.88 | 197.26 | -333.14 | 1121.88 | 197.26 |
| 4 | -4.100 | 8 | -0.0 | 1168.75 | 197.26 | -333.14 | 1168.75 | 197.26 |
| 5 | 0.000 | 8 | -0.0 | 1215.63 | 197.26 | -333.14 | 1215.63 | 197.26 |
| 6 | 4.100 | 8 | -0.0 | 1262.51 | 197.26 | -333.14 | 1262.51 | 197.26 |
| 7 | 8.200 | 8 | -0.0 | 1309.39 | 197.26 | -333.14 | 1309.39 | 197.26 |
| 8 | 12.300 | 8 | -0.0 | 1356.27 | 197.26 | -333.14 | 1356.27 | 197.26 |
| 9 | 16.400 | 8 | -0.0 | 1403.14 | 197.26 | -333.14 | 1403.14 | 197.26 |

$\delta_{max}$ = 1.333 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1403.14 (kN) < $R_a$ = 3018.01 (kN) OK
$P_{Nmin}$ = 1028.12 (kN) > $P_a$ = -1676.01 (kN) OK

5) Seismic

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 85846.08 (kN)
$H_0$ = 11091.17 (kN)
$M_0$ = 52366.11 (kN·m)

・Calculation of displacement
$\delta_x$ = 1.040 (mm)
$\delta_y$ = 3.066 (mm)
$\alpha$ = $0.02266\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -16.400 | 8 | -0.0 | 1047.78 | 154.04 | -260.26 | 1047.78 | 154.04 |
| 2 | -12.300 | 8 | -0.0 | 1083.91 | 154.04 | -260.26 | 1083.91 | 154.04 |
| 3 | -8.200 | 8 | -0.0 | 1120.05 | 154.04 | -260.26 | 1120.05 | 154.04 |
| 4 | -4.100 | 8 | -0.0 | 1156.18 | 154.04 | -260.26 | 1156.18 | 154.04 |
| 5 | 0.000 | 8 | -0.0 | 1192.31 | 154.04 | -260.26 | 1192.31 | 154.04 |
| 6 | 4.100 | 8 | -0.0 | 1228.44 | 154.04 | -260.26 | 1228.44 | 154.04 |
| 7 | 8.200 | 8 | -0.0 | 1264.57 | 154.04 | -260.26 | 1264.57 | 154.04 |
| 8 | 12.300 | 8 | -0.0 | 1300.70 | 154.04 | -260.26 | 1300.70 | 154.04 |
| 9 | 16.400 | 8 | -0.0 | 1336.83 | 154.04 | -260.26 | 1336.83 | 154.04 |

$\delta_{max}$ = 1.040 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1336.83 (kN) < $R_a$ = 3018.01 (kN) OK
$P_{Nmin}$ = 1047.78 (kN) > $P_a$ = -1676.01 (kN) OK

6) Seismic

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 131875.39 (kN)
$H_0$ = 10550.03 (kN)
$M_0$ = 50112.65 (kN·m)

・Calculation of displacement
$\delta_x$ = 0.989 (mm)
$\delta_y$ = 4.710 (mm)
$\alpha$ = $0.02165\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -16.400 | 8 | -0.0 | 1693.52 | 146.53 | -247.53 | 1693.52 | 146.53 |
| 2 | -12.300 | 8 | -0.0 | 1728.04 | 146.53 | -247.53 | 1728.04 | 146.53 |
| 3 | -8.200 | 8 | -0.0 | 1762.56 | 146.53 | -247.53 | 1762.56 | 146.53 |
| 4 | -4.100 | 8 | -0.0 | 1797.08 | 146.53 | -247.53 | 1797.08 | 146.53 |
| 5 | 0.000 | 8 | -0.0 | 1831.60 | 146.53 | -247.53 | 1831.60 | 146.53 |
| 6 | 4.100 | 8 | -0.0 | 1866.12 | 146.53 | -247.53 | 1866.12 | 146.53 |
| 7 | 8.200 | 8 | -0.0 | 1900.64 | 146.53 | -247.53 | 1900.64 | 146.53 |
| 8 | 12.300 | 8 | -0.0 | 1935.16 | 146.53 | -247.53 | 1935.16 | 146.53 |
| 9 | 16.400 | 8 | -0.0 | 1969.68 | 146.53 | -247.53 | 1969.68 | 146.53 |

$\delta_{max}$ = 0.989 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1969.68 (kN) < $R_a$ = 3018.01 (kN) OK
$P_{Nmin}$ = 1693.52 (kN) > $P_a$ = -1676.01 (kN) OK

(2) Weir axis right angle direction

1) Normal

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 91358.69 (kN)
$H_0$ = 883.93 (kN)
$M_0$ = -43852.17 (kN・m)

・Calculation of displacement
$\delta_x$ = 0.073 (mm)
$\delta_y$ = 3.263 (mm)
$\alpha$ = $-0.02754\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN・m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -11.200 | 9 | -0.0 | 1388.84 | 12.28 | -33.20 | 1388.84 | 12.28 |
| 2 | -8.000 | 9 | -0.0 | 1354.56 | 12.28 | -33.20 | 1354.56 | 12.28 |
| 3 | -4.800 | 9 | -0.0 | 1320.29 | 12.28 | -33.20 | 1320.29 | 12.28 |
| 4 | -1.600 | 9 | -0.0 | 1286.01 | 12.28 | -33.20 | 1286.01 | 12.28 |
| 5 | 1.600 | 9 | -0.0 | 1251.73 | 12.28 | -33.20 | 1251.73 | 12.28 |
| 6 | 4.800 | 9 | -0.0 | 1217.45 | 12.28 | -33.20 | 1217.45 | 12.28 |
| 7 | 8.000 | 9 | -0.0 | 1183.18 | 12.28 | -33.20 | 1183.18 | 12.28 |
| 8 | 11.200 | 9 | -0.0 | 1148.90 | 12.28 | -33.20 | 1148.90 | 12.28 |

$\delta_{max}$ = 0.073 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1388.84 (kN) < $R_a$ = 1993.31 (kN) OK
$P_{Nmin}$ = 1148.90 (kN) > $P_a$ = -920.98 (kN) OK

2) Normal

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 88729.08 (kN)
$H_0$ = 14388.83 (kN)
$M_0$ = -53237.45 (kN・m)

・Calculation of displacement
$\delta_x$ = 2.068 (mm)
$\delta_y$ = 3.169 (mm)
$\alpha$ = $-0.01586\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN・m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -11.200 | 9 | -0.0 | 1301.41 | 199.84 | -407.89 | 1301.41 | 199.84 |
| 2 | -8.000 | 9 | -0.0 | 1281.68 | 199.84 | -407.89 | 1281.68 | 199.84 |
| 3 | -4.800 | 9 | -0.0 | 1261.95 | 199.84 | -407.89 | 1261.95 | 199.84 |
| 4 | -1.600 | 9 | -0.0 | 1242.21 | 199.84 | -407.89 | 1242.21 | 199.84 |
| 5 | 1.600 | 9 | -0.0 | 1222.48 | 199.84 | -407.89 | 1222.48 | 199.84 |
| 6 | 4.800 | 9 | -0.0 | 1202.75 | 199.84 | -407.89 | 1202.75 | 199.84 |
| 7 | 8.000 | 9 | -0.0 | 1183.02 | 199.84 | -407.89 | 1183.02 | 199.84 |
| 8 | 11.200 | 9 | -0.0 | 1163.28 | 199.84 | -407.89 | 1163.28 | 199.84 |

$\delta_{max}$ = 2.068 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1301.41 (kN) < $R_a$ = 1993.31 (kN) OK
$P_{Nmin}$ = 1163.28 (kN) > $P_a$ = -920.98 (kN) OK

3) Normal

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 134758.39 (kN)
$H_0$ = 987.46 (kN)
$M_0$ = -40427.52 (kN・m)

・Calculation of displacement (Stability analysis case3)
$\delta_x$ = 0.093 (mm)
$\delta_y$ = 4.813 (mm)
$\alpha$ = $-0.02517\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN・m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -11.200 | 9 | -0.0 | 1981.25 | 13.71 | -35.37 | 1981.25 | 13.71 |
| 2 | -8.000 | 9 | -0.0 | 1949.94 | 13.71 | -35.37 | 1949.94 | 13.71 |
| 3 | -4.800 | 9 | -0.0 | 1918.62 | 13.71 | -35.37 | 1918.62 | 13.71 |
| 4 | -1.600 | 9 | -0.0 | 1887.30 | 13.71 | -35.37 | 1887.30 | 13.71 |
| 5 | 1.600 | 9 | -0.0 | 1855.99 | 13.71 | -35.37 | 1855.99 | 13.71 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 6 | 4.800 | 9 | -0.0 | 1824.67 | 13.71 | -35.37 | 1824.67 | 13.71 |
| 7 | 8.000 | 9 | -0.0 | 1793.35 | 13.71 | -35.37 | 1793.35 | 13.71 |
| 8 | 11.200 | 9 | -0.0 | 1762.04 | 13.71 | -35.37 | 1762.04 | 13.71 |

$\delta_{max}$ = 0.093 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1981.25 (kN) < $R_a$ = 1993.31 (kN) OK
$P_{Nmin}$ = 1762.04 (kN) > $P_a$ = -920.98 (kN) OK

4) Seismic

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 85846.08 (kN)
$H_0$ = 25761.42 (kN)
$M_0$ = -12876.91 (kN·m)

・Calculation of displacement
$\delta_x$ = 2.360 (mm)
$\delta_y$ = 3.066 (mm)
$\alpha$ = $0.02091\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -11.200 | 9 | -0.0 | 1101.25 | 357.80 | -615.92 | 1101.25 | 357.80 |
| 2 | -8.000 | 9 | -0.0 | 1127.27 | 357.80 | -615.92 | 1127.27 | 357.80 |
| 3 | -4.800 | 9 | -0.0 | 1153.28 | 357.80 | -615.92 | 1153.28 | 357.80 |
| 4 | -1.600 | 9 | -0.0 | 1179.30 | 357.80 | -615.92 | 1179.30 | 357.80 |
| 5 | 1.600 | 9 | -0.0 | 1205.31 | 357.80 | -615.92 | 1205.31 | 357.80 |
| 6 | 4.800 | 9 | -0.0 | 1231.33 | 357.80 | -615.92 | 1231.33 | 357.80 |
| 7 | 8.000 | 9 | -0.0 | 1257.35 | 357.80 | -615.92 | 1257.35 | 357.80 |
| 8 | 11.200 | 9 | -0.0 | 1283.36 | 357.80 | -615.92 | 1283.36 | 357.80 |

$\delta_{max}$ = 2.360 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1283.36 (kN) < $R_a$ = 3018.01 (kN) OK
$P_{Nmin}$ = 1101.25 (kN) > $P_a$ = -1676.01 (kN) OK

5) Seismic

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 131875.39 (kN)
$H_0$ = 10550.03 (kN)
$M_0$ = 0.00 (kN·m)

・Calculation of displacement
$\delta_x$ = 0.973 (mm)
$\delta_y$ = 4.710 (mm)
$\alpha$ = $0.01201\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -11.200 | 9 | -0.0 | 1779.31 | 146.53 | -251.00 | 1779.31 | 146.53 |
| 2 | -8.000 | 9 | -0.0 | 1794.25 | 146.53 | -251.00 | 1794.25 | 146.53 |
| 3 | -4.800 | 9 | -0.0 | 1809.19 | 146.53 | -251.00 | 1809.19 | 146.53 |
| 4 | -1.600 | 9 | -0.0 | 1824.13 | 146.53 | -251.00 | 1824.13 | 146.53 |
| 5 | 1.600 | 9 | -0.0 | 1839.07 | 146.53 | -251.00 | 1839.07 | 146.53 |
| 6 | 4.800 | 9 | -0.0 | 1854.01 | 146.53 | -251.00 | 1854.01 | 146.53 |
| 7 | 8.000 | 9 | -0.0 | 1868.95 | 146.53 | -251.00 | 1868.95 | 146.53 |
| 8 | 11.200 | 9 | -0.0 | 1883.89 | 146.53 | -251.00 | 1883.89 | 146.53 |

$\delta_{max}$ = 0.973 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1883.89 (kN) < $R_a$ = 3018.01 (kN) OK
$P_{Nmin}$ = 1779.31 (kN) > $P_a$ = -1676.01 (kN) OK

6) Seismic

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 89965.54 (kN)
$H_0$ = 10909.75 (kN)
$M_0$ = -6297.59 (kN·m)

・Calculation of displacement
$\delta_x$ = 0.999 (mm)
$\delta_y$ = 3.213 (mm)
$\alpha$ = $0.00830\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -11.200 | 9 | -0.0 | 1213.36 | 151.52 | -261.04 | 1213.36 | 151.52 |
| 2 | -8.000 | 9 | -0.0 | 1223.69 | 151.52 | -261.04 | 1223.69 | 151.52 |
| 3 | -4.800 | 9 | -0.0 | 1234.02 | 151.52 | -261.04 | 1234.02 | 151.52 |
| 4 | -1.600 | 9 | -0.0 | 1244.36 | 151.52 | -261.04 | 1244.36 | 151.52 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| 5 | 1.600 | 9 | -0.0 | 1254.69 | 151.52 | -261.04 | 1254.69 | 151.52 |
| 6 | 4.800 | 9 | -0.0 | 1265.02 | 151.52 | -261.04 | 1265.02 | 151.52 |
| 7 | 8.000 | 9 | -0.0 | 1275.35 | 151.52 | -261.04 | 1275.35 | 151.52 |
| 8 | 11.200 | 9 | -0.0 | 1285.68 | 151.52 | -261.04 | 1285.68 | 151.52 |

$\delta_{max}$ = 0.999 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1285.68 (kN) < $R_a$ = 3018.01 (kN) OK
$P_{Nmin}$ = 1213.36 (kN) > $P_a$ = -1676.01 (kN) OK

7) Normal

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 80093.13 (kN)
$H_0$ = 2811.86 (kN)
$M_0$ = 79292.20 (kN・m)

・Calculation of displacement
$\delta_x$ = 0.523 (mm)
$\delta_y$ = 2.860 (mm)
$\alpha$ = $0.05563 \times 10^{-3}$ (rad)

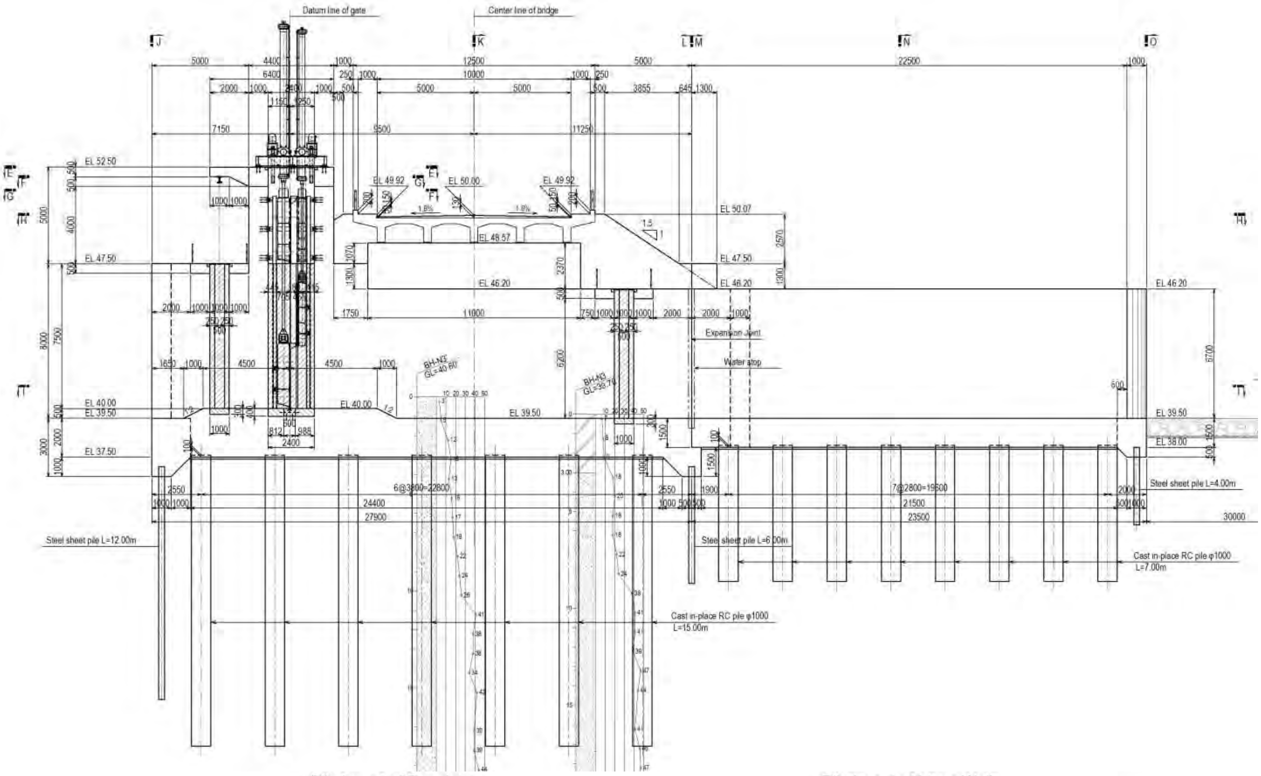
・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN・m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -11.200 | 9 | -0.0 | 870.12 | 39.05 | -61.71 | 870.12 | 39.05 |
| 2 | -8.000 | 9 | -0.0 | 939.34 | 39.05 | -61.71 | 939.34 | 39.05 |
| 3 | -4.800 | 9 | -0.0 | 1008.57 | 39.05 | -61.71 | 1008.57 | 39.05 |
| 4 | -1.600 | 9 | -0.0 | 1077.79 | 39.05 | -61.71 | 1077.79 | 39.05 |
| 5 | 1.600 | 9 | -0.0 | 1147.02 | 39.05 | -61.71 | 1147.02 | 39.05 |
| 6 | 4.800 | 9 | -0.0 | 1216.24 | 39.05 | -61.71 | 1216.24 | 39.05 |
| 7 | 8.000 | 9 | -0.0 | 1285.47 | 39.05 | -61.71 | 1285.47 | 39.05 |
| 8 | 11.200 | 9 | -0.0 | 1354.69 | 39.05 | -61.71 | 1354.69 | 39.05 |

$\delta_{max}$ = 0.523 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1354.69 (kN) < $R_a$ = 1993.31 (kN) OK
$P_{Nmin}$ = 870.12 (kN) > $P_a$ = -920.98 (kN) OK

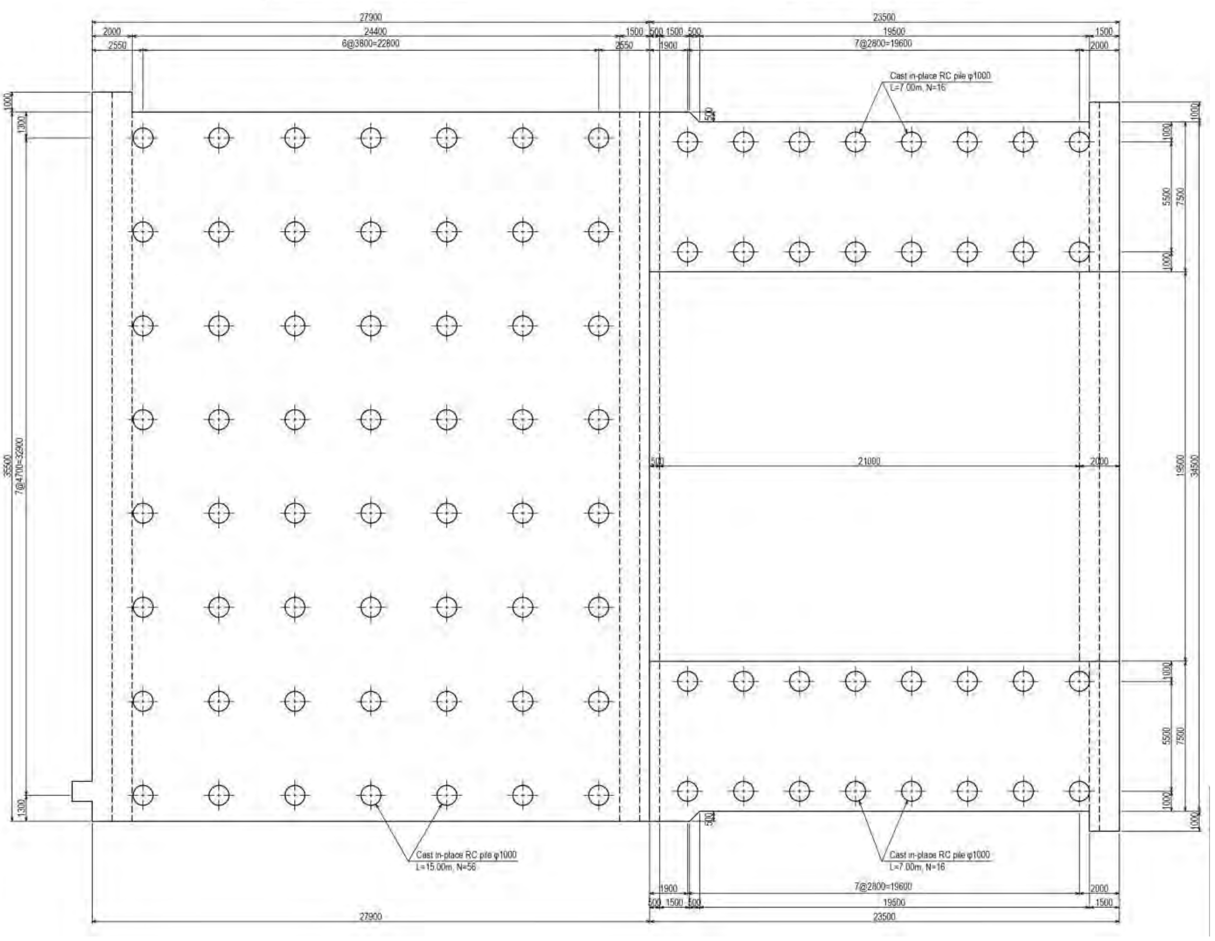
## A-7-2. Structural calculation of pile foundation for Iburahimia regulator

| Structure | Nos. of piles | dia. | Length of piles | note |
|---|---|---|---|---|
| Ibrahimia reg. | 7x8=56 | 1,000mm | 15.0m | Cast in-place concrete pile |



Pile layout of Regulator

Pile layout of Ltype Wall



(Ibrahimia Regulator)

1. Load data

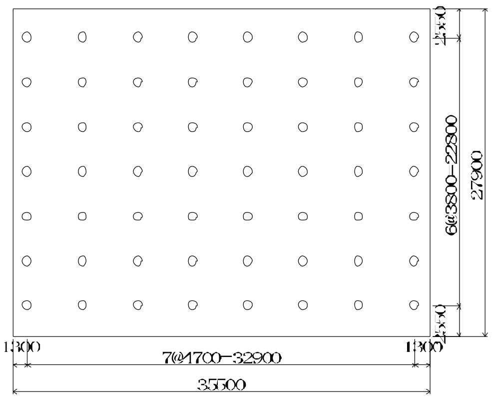
(1) Weir axis direction

| No | Load case | Extra coefficient | Design condition | Vertical force V (kN) | Horizontal force H (kN) | Bending moment M (kN·m) |
|---|---|---|---|---|---|---|
| 1 | Normal | 1.00 | Normal | 90408.54 | 724.42 | 8136.77 |
| 2 | Normal | 1.00 | Normal | 88729.08 | 1777.68 | 17745.82 |
| 3 | Normal | 1.00 | Normal | 134758.39 | 2506.86 | 16171.01 |
| 4 | Seismic | 1.15 | Seismic | 87525.54 | 14202.59 | 68269.92 |
| 5 | Seismic | 1.15 | Seismic | 85846.08 | 11091.17 | 52366.11 |
| 6 | Seismic | 1.15 | Seismic | 131875.39 | 10550.03 | 50112.65 |

(2) Weir axis right angle direction

| No | Load case | Extra coefficient | Design condition | Vertical force V (kN) | Horizontal force H (kN) | Bending moment M (kN·m) |
|---|---|---|---|---|---|---|
| 1 | Normal | 1.00 | Normal | 91358.69 | 883.93 | -43852.17 |
| 2 | Normal | 1.00 | Normal | 88729.08 | 14388.83 | -53237.45 |
| 3 | Normal | 1.00 | Normal | 134758.39 | 987.46 | -40427.52 |
| 4 | Seismic | 1.15 | Seismic | 85846.08 | 25761.42 | -12876.91 |
| 5 | Seismic | 1.15 | Seismic | 131875.39 | 10550.03 | 0.00 |
| 6 | Seismic | 1.15 | Seismic | 89965.54 | 10909.75 | -6297.59 |
| 7 | Normal | 1.00 | Normal | 80819.68 | 4765.33 | 71121.32 |

2. Pile arrangement




3.Ground spring

3-1.Pile spring constant for pile axis direction

$$K_V = a \cdot \frac{A_p \cdot E_p}{L}$$

$$= 0.312 \times \frac{0.785398 \times 25000000}{14.900}$$

$$= 411016.3 \ (kN/m)$$

ここに、

$K_V$ : Pile spring constant for pile axis direction (kN/m)

a : Correction coefficient

※ In-place pile

$a = 0.031 \ (L/D) - 0.150$

$= 0.031 \times ( 14.900 / 1.000 ) - 0.150 = 0.312$

$A_p$ : Vertical sectional area of the pile ($m^2$)

$E_p$ : Young's modulus of the pile ($kN/m^2$)

L : Insertion length of the pile (m)

D : Pile diameter (m)

3-2.Ground resistance coefficient for horizontal direction

Spring constant for pile axis right angle direction is calculated as following formula with considering multilayered ground.

(1) Calculation of the ground resistance coefficient for horizontal direction

$$k_H = k_{H0} \left( \frac{B_H}{0.3} \right)^{-3/4}$$

$k_H$ : Ground resistance coefficient for horizontal direction (kN/m$^3$)
$k_{H0}$ : Ground resistance coefficient for horizontal direction, which is equivalent to the plate lord test value by the 0.3m diameter rigid disc (kN/m$^3$)

$$k_{H0} = \frac{1}{0.3} \cdot \alpha \cdot E_0$$

$B_H$ : Conversion load width for the foundation (m)

$$B_H = \sqrt{\frac{D}{\beta}}$$

D : Pile diameter (m)
β : Characteristic value of the foundation (m$^{-1}$)

$$\beta = \sqrt[4]{\frac{k_H \cdot D}{4EI}}$$

α : Coefficient for the estimation of ground resistance coefficient
Normal α = 1  Seismic α = 2
$E_0$ : Coefficient for the ground deformation (kN/m$^2$)
EI : Bending rigidity of the foundation (kN・m$^2$)

(2) Calculation of the conversion road width $B_H$

Key point on the calculation of $B_H$.
・$k_H$ should be normal value when the calculating of $B_H$.
・$k_H$ should be average value from the design ground surface to 1/β depth when the calculating of $B_H$ even if the layer shift dipper. Calculation of $k_H$ of each layer should be used $B_H$.

| | | |
|---|---|---|
| Pile diameter | D | 1.000 (m) |
| Young's modules | E | 25000000 (kN/m$^2$) |
| Section secondary moment | I | 0.04908739 (m$^4$) |

In assumption of 1/β = 3.443 (m) (β = 0.29043 (m$^{-1}$)) )
→Average $E_0$ = 41094.3 (kN/m$^2$)

$$B_H = \sqrt{\frac{1.000}{0.29043}} = 1.856 \ (m)$$

$$k_H = \frac{1}{0.3} \times 1.0 \times 41094.3 \times \left( \frac{1.856}{0.3} \right)^{-3/4} = 34925.5 \ (kN/m^3)$$

Calculation result of β is as follows, and this matches to the assumed β.

$$\beta = \sqrt[4]{\frac{34925.5 \times 1.000}{4 \times 25000000 \times 0.04908739}} = 0.29043 \ (m^{-1})$$

Form the above, the conversion load width $B_H$ = 1.856 (m)

(3) Calculation of the ground resistance coefficient of each layer for horizontal direction

| | | |
|---|---|---|
| Pile diameter | D | 1.000 (m) |
| Young's mo | E | 25000000 (kN/m$^2$) |
| Section secondary moment | I | 0.04908739 (m$^4$) |

1) Normal

| No | Layer thickness (m) | $E_0$ (kN/m$^2$) | $k_{H0}$ (kN/m$^3$) | $k_H$ (kN/m$^3$) |
|---|---|---|---|---|
| 1 | 0.200 | 33600.0 | 112000.0 | 28556.2 |
| 2 | 1.000 | 42000.0 | 140000.0 | 35695.2 |
| 3 | 1.000 | 36400.0 | 121333.3 | 30935.9 |
| 4 | 1.000 | 44800.0 | 149333.3 | 38074.9 |
| 5 | 1.000 | 47600.0 | 158666.7 | 40454.6 |
| 6 | 1.000 | 50400.0 | 168000.0 | 42834.3 |
| 7 | 1.000 | 61600.0 | 205333.3 | 52353.0 |
| 8 | 1.000 | 67200.0 | 224000.0 | 57112.4 |
| 9 | 1.000 | 72800.0 | 242666.7 | 61871.7 |
| 10 | 1.000 | 114800.0 | 382666.7 | 97566.9 |
| 11 | 1.000 | 106400.0 | 354666.7 | 90427.9 |
| 12 | 1.000 | 106400.0 | 354666.7 | 90427.9 |
| 13 | 1.000 | 95200.0 | 317333.3 | 80909.2 |
| 14 | 2.000 | 117600.0 | 392000.0 | 99946.6 |
| 15 | 0.700 | 109200.0 | 364000.0 | 92807.6 |

Characteristic β : 0.29043 (m$^{-1}$)

2) Seismic

| No | Layer thickness (m) | $E_0$ (kN/m$^2$) | $k_{H0}$ (kN/m$^3$) | $k_H$ (kN/m$^3$) |
|---|---|---|---|---|
| 1 | 0.200 | 33600.0 | 224000.0 | 57112.4 |
| 2 | 1.000 | 42000.0 | 280000.0 | 71390.4 |
| 3 | 1.000 | 36400.0 | 242666.7 | 61871.7 |
| 4 | 1.000 | 44800.0 | 298666.7 | 76149.8 |
| 5 | 1.000 | 47600.0 | 317333.3 | 80909.2 |
| 6 | 1.000 | 50400.0 | 336000.0 | 85668.5 |
| 7 | 1.000 | 61600.0 | 410666.7 | 104706.0 |
| 8 | 1.000 | 67200.0 | 448000.0 | 114224.7 |
| 9 | 1.000 | 72800.0 | 485333.3 | 123743.4 |
| 10 | 1.000 | 114800.0 | 765333.3 | 195133.9 |
| 11 | 1.000 | 106400.0 | 709333.3 | 180855.8 |
| 12 | 1.000 | 106400.0 | 709333.3 | 180855.8 |
| 13 | 1.000 | 95200.0 | 634666.7 | 161818.3 |
| 14 | 2.000 | 117600.0 | 784000.0 | 199893.2 |
| 15 | 0.700 | 109200.0 | 728000.0 | 185615.2 |

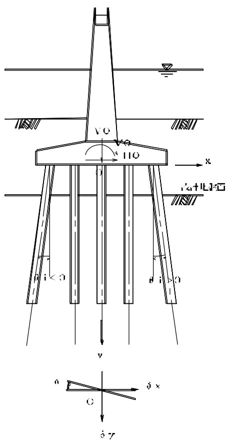
Characteristic β : 0.29043 (m$^{-1}$)

4. Calculation of the displacement and the resistance at pile head

4-1.Calculation method

(1) Calculation formula

Calculation method by the displacement way is to set coordinates as following figure and to set the center of the piles group as origin point 0. The external force acting to point 0 is set as following figure.



In this regard, displacement of the origin 0 is calculated from following equation with three unknowns.

$Axx \cdot \delta_x + Axy \cdot \delta_y + Axa \cdot \alpha = H_0$
$Ayx \cdot \delta_x + Ayy \cdot \delta_y + Aya \cdot \alpha = V_0$
$Aax \cdot \delta_x + Aay \cdot \delta_y + Aaa \cdot \alpha = M_0$

Each coefficient is calculated with the assumption of the footing bottom as flat

$Axx = \sum (K_1 \cdot \cos^2\theta_i + K_V \cdot \sin^2\theta_i)$
$Axy = Ayx = \sum (K_V - K_1) \cdot \sin\theta_i \cdot \cos\theta_i$
$Axa = Aax = \sum\{(K_V - K_1) \cdot x_i \cdot \sin\theta_i \cdot \cos\theta_i - K_2 \cdot \cos\theta_i\}$
$Ayy = \sum (K_V \cdot \cos^2\theta_i + K_1 \cdot \sin^2\theta_i)$
$Aya = Aay = \sum\{(K_V \cdot \cos^2\theta_i + K_1 \cdot \sin^2\theta_i) \cdot xi + K_2 \cdot \sin\theta_i\}$
$Aaa = \sum\{(K_V \cdot \cos^2\theta_i + K_1 \cdot \sin^2\theta_i) \cdot xi^2 + (K_2 + K_3) \cdot x_i \cdot \sin\theta_i + K_4\}$

$H_0$ : Horizontal load acting to the upper portion of footing bottom (kN)
$V_0$ : Vertical load acting to the upper portion of the footing bottom (kN)
$M_0$ : Moment of the external force around the origin 0 (kN・m)
$\delta_x$ : Displacement of origin 0 for horizontal direction (m)
$\delta_y$ : Displacement of origin 0 for vertical direction (m)
$\alpha$ : Rotation angle of the footing (rad)
$x_i$ : X coordinate of the pile head at i(th) pile (m)
$\theta_i$ : Angle between pile axis and vertical axis at i(th) pile (degree)

From the result of displacement at footing origin point ($\delta_x$, $\delta_y$, $\alpha$), the force for pile axis direction $P_{Ni}$ at pile head, the force for pile axis right angle direction $P_{Hi}$ and moment $M_{ti}$ are formulated as following formula;

$P_{Ni} = K_V \cdot \delta_{yi}'$
$P_{Hi} = K_1 \cdot \delta_{xi}' - K_2 \cdot \alpha$
$M_{ti} = - K_3 \cdot \delta_{xi}' + K_4 \cdot \alpha$

$\delta_{xi}' = \delta_x \cdot \cos\theta_i - (\delta_y + \alpha \cdot x_i) \cdot \sin\theta_i$
$\delta_{yi}' = \delta_x \cdot \sin\theta_i + (\delta_y + \alpha \cdot x_i) \cdot \cos\theta_i$

$\delta_{xi}'$ : Displacement of pile head at i(th) pile for pile axis right angle direction (m)
$\delta_{yi}'$ : Displacement of pile head at i(th) pile for pile axis direction (m)
$K_V$ : Pile axis direction force occurring the displacement for pile axis direction at pile head(kN)
(Pile sprig coefficient for pile axis direction)
$K_1$、$K_2$、$K_3$、$K_4$ : Pile spring coefficient for pile axis right angle direction
$x_i$ : x coordinate at i(th) pile head(m)
$\theta_i$ : Angle degree between pile axis and vertical axis at i(th) pile (degree)
$P_{Ni}$ : Pile axis direction force at i(th) pile (kN)
$P_{Hi}$ : Pile axis right angle direction force at i(th) pile (kN)
$M_{ti}$ : Moment acting to the pile head at i(th) pile (kN・m)

Vertical resistance at pile head ($V_i$) and horizontal resistance at pile head ($H_i$) are calculated as following formula;

$V_i = P_{Ni} \cdot \cos\theta_i - P_{Hi} \cdot \sin\theta_i$
$H_i = P_{Ni} \cdot \sin\theta_i + P_{Hi} \cdot \cos\theta_i$

(2) Calculation of the pile spring coefficient

1) Normal

a) Pile conditions

Cast in-place concrete pile φ1000.0 (mm) L = 14.900 (m)
$K_V$ = 411016.2 (kN/m)

b) Pile spring coefficient for pile axis right angle direction

Pile head fixed connection

| No | $K_1$ (kN/m) | $K_2$ (kN/rad) | $K_3$ (kN・m/m) | $K_4$ (kN・m/rad) |
|---|---|---|---|---|
| 1 | 121014.0 | 213267.5 | 213267.5 | 736070.8 |

Pile head hinge

| No | $K_1$ (kN/m) | $K_2$ (kN/rad) | $K_3$ (kN・m/m) | $K_4$ (kN・m/rad) |
|---|---|---|---|---|
| 1 | 59222.4 | 0.0 | 0.0 | 0.0 |

2) Seismic

a) Pile conditions

Cast in-place concrete pile φ1000.0 (mm) L = 14.900 (m)
$K_V$ = 411016.2 (kN/m)

b) Pile spring coefficient for pile axis right angle direction

Pile head fixed connection

Pile head fixed connection

| No | $K_1$ (kN/m) | $K_2$ (kN/rad) | $K_3$ (kN・m/m) | $K_4$ (kN・m/rad) |
|---|---|---|---|---|
| 1 | 200719.2 | 296552.6 | 296552.6 | 863979.9 |

Pile head hinge

| No | $K_1$ (kN/m) | $K_2$ (kN/rad) | $K_3$ (kN·m/m) | $K_4$ (kN·m/rad) |
|---|---|---|---|---|
| 1 | 98930.4 | 0.0 | 0.0 | 0.0 |

$K_1$、$K_3$ : Pile axis right angle direction force (kN/m) and bending moment (kN·m/m), that should be acted to the pile head in order to displace the pile head for unit quantity without rotation.
$K_2$、$K_4$ : Pile axis right angle direction force (kN/m) and bending moment (kN·m/m), that should be acted to the pile head in order to rotate the pile head for unit quantity without displacement.

(3) Rigidity matrix (fixed)

1) Weir axis direction

a) Normal

Pile head rigid connection

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 6776784.5 & 0.0 & -11942980.0 \\ 0.0 & 23016910.0 & -2.7 \\ -11942980.0 & -2.7 & 2710548736.0 \end{bmatrix}$$

Pile head hinge

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 3316452.8 & 0.0 & 0.0 \\ 0.0 & 23016910.0 & -2.7 \\ 0.0 & -2.7 & 2669328640.0 \end{bmatrix}$$

b) Seismic

Pile head rigid connection

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 11240275.0 & 0.0 & -16606945.0 \\ 0.0 & 23016910.0 & -2.7 \\ -16606945.0 & -2.7 & 2717711616.0 \end{bmatrix}$$

Pile head hinge

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 5540104.5 & 0.0 & 0.0 \\ 0.0 & 23016910.0 & -2.7 \\ 0.0 & -2.7 & 2669328640.0 \end{bmatrix}$$

2) Weir axis right angle direction

a) Normal

Pile head rigid connection

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 6776784.5 & 0.0 & -11942980.0 \\ 0.0 & 23016910.0 & 1.6 \\ -11942980.0 & 1.6 & 1370676736.0 \end{bmatrix}$$

Pile head hinge

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 3316452.8 & 0.0 & 0.0 \\ 0.0 & 23016910.0 & 1.6 \\ 0.0 & 1.6 & 1329456768.0 \end{bmatrix}$$

b) Seismic

Pile head rigid connection

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 11240275.0 & 0.0 & -16606945.0 \\ 0.0 & 23016910.0 & 1.6 \\ -16606945.0 & 1.6 & 1377839616.0 \end{bmatrix}$$

Pile head hinge

$$\begin{bmatrix} Axx & Axy & Axa \\ Ayx & Ayy & Aya \\ Aax & Aay & Aaa \end{bmatrix} = \begin{bmatrix} 5540104.5 & 0.0 & 0.0 \\ 0.0 & 23016910.0 & 1.6 \\ 0.0 & 1.6 & 1329456768.0 \end{bmatrix}$$

4-2.Pile head displacement and pile head resistance

(1) Weir axis direction

1) Normal

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 90408.54 (kN)
$H_0$ = 724.42 (kN)
$M_0$ = 8136.77 (kN·m)

・Calculation of displacement
$\delta_x$ = 0.113 (mm)
$\delta_y$ = 3.928 (mm)
$\alpha$ = $0.00350\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -16.450 | 7 | -0.0 | 1590.77 | 12.94 | -21.54 | 1590.77 | 12.94 |
| 2 | -11.750 | 7 | -0.0 | 1597.53 | 12.94 | -21.54 | 1597.53 | 12.94 |
| 3 | -7.050 | 7 | -0.0 | 1604.30 | 12.94 | -21.54 | 1604.30 | 12.94 |
| 4 | -2.350 | 7 | -0.0 | 1611.06 | 12.94 | -21.54 | 1611.06 | 12.94 |
| 5 | 2.350 | 7 | -0.0 | 1617.82 | 12.94 | -21.54 | 1617.82 | 12.94 |
| 6 | 7.050 | 7 | -0.0 | 1624.58 | 12.94 | -21.54 | 1624.58 | 12.94 |
| 7 | 11.750 | 7 | -0.0 | 1631.34 | 12.94 | -21.54 | 1631.34 | 12.94 |
| 8 | 16.450 | 7 | -0.0 | 1638.10 | 12.94 | -21.54 | 1638.10 | 12.94 |

$\delta_{max}$ = 0.113 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1638.10 (kN) < $R_a$ = 2633.03 (kN) OK
$P_{Nmin}$ = 1590.77 (kN) > $P_a$ = -1258.25 (kN) OK

2) Normal

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 88729.08 (kN)
$H_0$ = 1777.68 (kN)
$M_0$ = 17745.82 (kN·m)

・Calculation of displacement
$\delta_x$ = 0.276 (mm)
$\delta_y$ = 3.855 (mm)
$\alpha$ = $0.00776\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -16.450 | 7 | -0.0 | 1531.96 | 31.74 | -53.15 | 1531.96 | 31.74 |
| 2 | -11.750 | 7 | -0.0 | 1546.96 | 31.74 | -53.15 | 1546.96 | 31.74 |
| 3 | -7.050 | 7 | -0.0 | 1561.95 | 31.74 | -53.15 | 1561.95 | 31.74 |
| 4 | -2.350 | 7 | -0.0 | 1576.95 | 31.74 | -53.15 | 1576.95 | 31.74 |
| 5 | 2.350 | 7 | -0.0 | 1591.95 | 31.74 | -53.15 | 1591.95 | 31.74 |
| 6 | 7.050 | 7 | -0.0 | 1606.94 | 31.74 | -53.15 | 1606.94 | 31.74 |
| 7 | 11.750 | 7 | -0.0 | 1621.94 | 31.74 | -53.15 | 1621.94 | 31.74 |
| 8 | 16.450 | 7 | -0.0 | 1636.94 | 31.74 | -53.15 | 1636.94 | 31.74 |

$\delta_{max}$ = 0.276 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1636.94 (kN) < $R_a$ = 2633.03 (kN) OK
$P_{Nmin}$ = 1531.96 (kN) > $P_a$ = -1258.25 (kN) OK

3) Normal

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 134758.39 (kN)
$H_0$ = 2506.86 (kN)
$M_0$ = 16171.01 (kN·m)

・Calculation of displacement
$\delta_x$ = 0.383 (mm)
$\delta_y$ = 5.855 (mm)
$\alpha$ = $0.00766\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -16.450 | 7 | -0.0 | 2354.64 | 44.77 | -76.13 | 2354.64 | 44.77 |
| 2 | -11.750 | 7 | -0.0 | 2369.43 | 44.77 | -76.13 | 2369.43 | 44.77 |
| 3 | -7.050 | 7 | -0.0 | 2384.22 | 44.77 | -76.13 | 2384.22 | 44.77 |
| 4 | -2.350 | 7 | -0.0 | 2399.01 | 44.77 | -76.13 | 2399.01 | 44.77 |
| 5 | 2.350 | 7 | -0.0 | 2413.79 | 44.77 | -76.13 | 2413.79 | 44.77 |
| 6 | 7.050 | 7 | -0.0 | 2428.58 | 44.77 | -76.13 | 2428.58 | 44.77 |
| 7 | 11.750 | 7 | -0.0 | 2443.37 | 44.77 | -76.13 | 2443.37 | 44.77 |
| 8 | 16.450 | 7 | -0.0 | 2458.16 | 44.77 | -76.13 | 2458.16 | 44.77 |

$\delta_{max}$ = 0.383 (mm) < $\delta_a$ = 15.00 (mm) OK

$P_{Nmax}$ = 2458.16 (kN) < $R_a$ = 2633.03 (kN) OK
$P_{Nmin}$ = 2354.64 (kN) > $P_a$ = -1258.25 (kN) OK

4) Seismic

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 87525.54 (kN)
$H_0$ = 14202.59 (kN)
$M_0$ = 68269.92 (kN·m)

・Calculation of displacement
$\delta_x$ = 1.313 (mm)
$\delta_y$ = 3.803 (mm)
$\alpha$ = $0.03314\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -16.450 | 7 | -0.0 | 1338.89 | 253.62 | -360.59 | 1338.89 | 253.62 |
| 2 | -11.750 | 7 | -0.0 | 1402.91 | 253.62 | -360.59 | 1402.91 | 253.62 |
| 3 | -7.050 | 7 | -0.0 | 1466.93 | 253.62 | -360.59 | 1466.93 | 253.62 |
| 4 | -2.350 | 7 | -0.0 | 1530.95 | 253.62 | -360.59 | 1530.95 | 253.62 |
| 5 | 2.350 | 7 | -0.0 | 1594.97 | 253.62 | -360.59 | 1594.97 | 253.62 |
| 6 | 7.050 | 7 | -0.0 | 1658.99 | 253.62 | -360.59 | 1658.99 | 253.62 |
| 7 | 11.750 | 7 | -0.0 | 1723.01 | 253.62 | -360.59 | 1723.01 | 253.62 |
| 8 | 16.450 | 7 | -0.0 | 1787.03 | 253.62 | -360.59 | 1787.03 | 253.62 |

$\delta_{max}$ = 1.313 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1787.03 (kN) < $R_a$ = 3976.55 (kN) OK
$P_{Nmin}$ = 1338.89 (kN) > $P_a$ = -2333.46 (kN) OK

5) Seismic

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 85846.08 (kN)
$H_0$ = 11091.17 (kN)
$M_0$ = 52366.11 (kN·m)

・Calculation of displacement
$\delta_x$ = 1.024 (mm)
$\delta_y$ = 3.730 (mm)
$\alpha$ = $0.02553\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -16.450 | 7 | -0.0 | 1360.36 | 198.06 | -281.75 | 1360.36 | 198.06 |
| 2 | -11.750 | 7 | -0.0 | 1409.68 | 198.06 | -281.75 | 1409.68 | 198.06 |
| 3 | -7.050 | 7 | -0.0 | 1458.99 | 198.06 | -281.75 | 1458.99 | 198.06 |
| 4 | -2.350 | 7 | -0.0 | 1508.31 | 198.06 | -281.75 | 1508.31 | 198.06 |
| 5 | 2.350 | 7 | -0.0 | 1557.62 | 198.06 | -281.75 | 1557.62 | 198.06 |
| 6 | 7.050 | 7 | -0.0 | 1606.94 | 198.06 | -281.75 | 1606.94 | 198.06 |
| 7 | 11.750 | 7 | -0.0 | 1656.25 | 198.06 | -281.75 | 1656.25 | 198.06 |
| 8 | 16.450 | 7 | -0.0 | 1705.57 | 198.06 | -281.75 | 1705.57 | 198.06 |

$\delta_{max}$ = 1.024 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1705.57 (kN) < $R_a$ = 3976.55 (kN) OK
$P_{Nmin}$ = 1360.36 (kN) > $P_a$ = -2333.46 (kN) OK

6) Seismic

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 131875.39 (kN)
$H_0$ = 10550.03 (kN)
$M_0$ = 50112.65 (kN·m)

・Calculation of displacement
$\delta_x$ = 0.975 (mm)
$\delta_y$ = 5.730 (mm)
$\alpha$ = $0.02439\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -16.450 | 7 | -0.0 | 2189.98 | 188.39 | -267.95 | 2189.98 | 188.39 |
| 2 | -11.750 | 7 | -0.0 | 2237.10 | 188.39 | -267.95 | 2237.10 | 188.39 |
| 3 | -7.050 | 7 | -0.0 | 2284.23 | 188.39 | -267.95 | 2284.23 | 188.39 |
| 4 | -2.350 | 7 | -0.0 | 2331.35 | 188.39 | -267.95 | 2331.35 | 188.39 |
| 5 | 2.350 | 7 | -0.0 | 2378.48 | 188.39 | -267.95 | 2378.48 | 188.39 |
| 6 | 7.050 | 7 | -0.0 | 2425.61 | 188.39 | -267.95 | 2425.61 | 188.39 |
| 7 | 11.750 | 7 | -0.0 | 2472.73 | 188.39 | -267.95 | 2472.73 | 188.39 |
| 8 | 16.450 | 7 | -0.0 | 2519.86 | 188.39 | -267.95 | 2519.86 | 188.39 |

$\delta_{max}$ = 0.975 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 2519.86 (kN) < $R_a$ = 3976.55 (kN) OK
$P_{Nmin}$ = 2189.98 (kN) > $P_a$ = -2333.46 (kN) OK

(2) Weir axis right angle direction

1) Normal

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 91358.69 (kN)
$H_0$ = 883.93 (kN)
$M_0$ = -43852.17 (kN·m)

・Calculation of displacement
$\delta_x$ = 0.075 (mm)
$\delta_y$ = 3.969 (mm)
$\alpha$ = $-0.03134\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -11.400 | 8 | -0.0 | 1778.24 | 15.78 | -39.11 | 1778.24 | 15.78 |
| 2 | -7.600 | 8 | -0.0 | 1729.30 | 15.78 | -39.11 | 1729.30 | 15.78 |
| 3 | -3.800 | 8 | -0.0 | 1680.35 | 15.78 | -39.11 | 1680.35 | 15.78 |
| 4 | 0.000 | 8 | -0.0 | 1631.41 | 15.78 | -39.11 | 1631.41 | 15.78 |
| 5 | 3.800 | 8 | -0.0 | 1582.46 | 15.78 | -39.11 | 1582.46 | 15.78 |
| 6 | 7.600 | 8 | -0.0 | 1533.51 | 15.78 | -39.11 | 1533.51 | 15.78 |
| 7 | 11.400 | 8 | -0.0 | 1484.57 | 15.78 | -39.11 | 1484.57 | 15.78 |

$\delta_{max}$ = 0.075 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1778.24 (kN) < $R_a$ = 2633.03 (kN) OK
$P_{Nmin}$ = 1484.57 (kN) > $P_a$ = -1258.25 (kN) OK

2) Normal

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 88729.08 (kN)
$H_0$ = 14388.83 (kN)
$M_0$ = -53237.45 (kN·m)

・Calculation of displacement
$\delta_x$ = 2.087 (mm)
$\delta_y$ = 3.855 (mm)
$\alpha$ = $-0.02066\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -11.400 | 8 | -0.0 | 1681.24 | 256.94 | -460.26 | 1681.24 | 256.94 |
| 2 | -7.600 | 8 | -0.0 | 1648.97 | 256.94 | -460.26 | 1648.97 | 256.94 |
| 3 | -3.800 | 8 | -0.0 | 1616.71 | 256.94 | -460.26 | 1616.71 | 256.94 |
| 4 | 0.000 | 8 | -0.0 | 1584.45 | 256.94 | -460.26 | 1584.45 | 256.94 |
| 5 | 3.800 | 8 | -0.0 | 1552.18 | 256.94 | -460.26 | 1552.18 | 256.94 |
| 6 | 7.600 | 8 | -0.0 | 1519.92 | 256.94 | -460.26 | 1519.92 | 256.94 |
| 7 | 11.400 | 8 | -0.0 | 1487.66 | 256.94 | -460.26 | 1487.66 | 256.94 |

$\delta_{max}$ = 2.087 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1681.24 (kN) < $R_a$ = 2633.03 (kN) OK
$P_{Nmin}$ = 1487.66 (kN) > $P_a$ = -1258.25 (kN) OK

3) Normal

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 134758.39 (kN)
$H_0$ = 987.46 (kN)
$M_0$ = -40427.52 (kN·m)

・Calculation of displacement (Stability analysis case3)
$\delta_x$ = 0.095 (mm)
$\delta_y$ = 5.855 (mm)
$\alpha$ = $-0.02867\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) | |
|---|---|---|---|---|---|---|---|---|---|
| 1 | -11.400 | 8 | -0.0 | 2540.71 | 17.63 | -41.40 | 2540.71 | 17.63 | ※Critical case |
| 2 | -7.600 | 8 | -0.0 | 2495.94 | 17.63 | -41.40 | 2495.94 | 17.63 | |
| 3 | -3.800 | 8 | -0.0 | 2451.17 | 17.63 | -41.40 | 2451.17 | 17.63 | |
| 4 | 0.000 | 8 | -0.0 | 2406.40 | 17.63 | -41.40 | 2406.40 | 17.63 | |
| 5 | 3.800 | 8 | -0.0 | 2361.63 | 17.63 | -41.40 | 2361.63 | 17.63 | |
| 6 | 7.600 | 8 | -0.0 | 2316.86 | 17.63 | -41.40 | 2316.86 | 17.63 | |
| 7 | 11.400 | 8 | -0.0 | 2272.09 | 17.63 | -41.40 | 2272.09 | 17.63 | |

$\delta_{max}$ = 0.095 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 2540.71 (kN) < $R_a$ = 2633.03 (kN) OK
$P_{Nmin}$ = 2272.09 (kN) > $P_a$ = -1258.25 (kN) OK

4) Seismic

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 85846.08 (kN)
$H_0$ = 25761.42 (kN)
$M_0$ = -12876.91 (kN·m)

・Calculation of displacement
$\delta_x$ = 2.319 (mm)
$\delta_y$ = 3.730 (mm)
$\alpha$ = 0.01861×$10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -11.400 | 8 | -0.0 | 1445.77 | 460.03 | -671.74 | 1445.77 | 460.03 |
| 2 | -7.600 | 8 | -0.0 | 1474.83 | 460.03 | -671.74 | 1474.83 | 460.03 |
| 3 | -3.800 | 8 | -0.0 | 1503.90 | 460.03 | -671.74 | 1503.90 | 460.03 |
| 4 | 0.000 | 8 | -0.0 | 1532.97 | 460.03 | -671.74 | 1532.97 | 460.03 |
| 5 | 3.800 | 8 | -0.0 | 1562.03 | 460.03 | -671.74 | 1562.03 | 460.03 |
| 6 | 7.600 | 8 | -0.0 | 1591.10 | 460.03 | -671.74 | 1591.10 | 460.03 |
| 7 | 11.400 | 8 | -0.0 | 1620.16 | 460.03 | -671.74 | 1620.16 | 460.03 |

$\delta_{max}$ = 2.319 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1620.16 (kN) < $R_a$ = 3976.55 (kN) OK
$P_{Nmin}$ = 1445.77 (kN) > $P_a$ = -2333.46 (kN) OK

5) Seismic

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 131875.39 (kN)
$H_0$ = 10550.03 (kN)
$M_0$ = 0.00 (kN·m)

・Calculation of displacement
$\delta_x$ = 0.956 (mm)
$\delta_y$ = 5.730 (mm)
$\alpha$ = 0.01152×$10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -11.400 | 8 | -0.0 | 2300.95 | 188.39 | -273.44 | 2300.95 | 188.39 |
| 2 | -7.600 | 8 | -0.0 | 2318.94 | 188.39 | -273.44 | 2318.94 | 188.39 |
| 3 | -3.800 | 8 | -0.0 | 2336.93 | 188.39 | -273.44 | 2336.93 | 188.39 |
| 4 | 0.000 | 8 | -0.0 | 2354.92 | 188.39 | -273.44 | 2354.92 | 188.39 |
| 5 | 3.800 | 8 | -0.0 | 2372.91 | 188.39 | -273.44 | 2372.91 | 188.39 |
| 6 | 7.600 | 8 | -0.0 | 2390.90 | 188.39 | -273.44 | 2390.90 | 188.39 |
| 7 | 11.400 | 8 | -0.0 | 2408.89 | 188.39 | -273.44 | 2408.89 | 188.39 |

$\delta_{max}$ = 0.956 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 2408.89 (kN) < $R_a$ = 3976.55 (kN) OK
$P_{Nmin}$ = 2300.95 (kN) > $P_a$ = -2333.46 (kN) OK

6) Seismic

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 89965.54 (kN)
$H_0$ = 10909.75 (kN)
$M_0$ = -6297.59 (kN·m)

・Calculation of displacement
$\delta_x$ = 0.981 (mm)
$\delta_y$ = 3.909 (mm)
$\alpha$ = 0.00726×$10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -11.400 | 8 | -0.0 | 1572.52 | 194.82 | -284.74 | 1572.52 | 194.82 |
| 2 | -7.600 | 8 | -0.0 | 1583.86 | 194.82 | -284.74 | 1583.86 | 194.82 |
| 3 | -3.800 | 8 | -0.0 | 1595.19 | 194.82 | -284.74 | 1595.19 | 194.82 |
| 4 | 0.000 | 8 | -0.0 | 1606.53 | 194.82 | -284.74 | 1606.53 | 194.82 |
| 5 | 3.800 | 8 | -0.0 | 1617.86 | 194.82 | -284.74 | 1617.86 | 194.82 |
| 6 | 7.600 | 8 | -0.0 | 1629.20 | 194.82 | -284.74 | 1629.20 | 194.82 |
| 7 | 11.400 | 8 | -0.0 | 1640.53 | 194.82 | -284.74 | 1640.53 | 194.82 |

$\delta_{max}$ = 0.981 (mm) < $\delta_a$ = 15.00 (mm) OK
$P_{Nmax}$ = 1640.53 (kN) < $R_a$ = 3976.55 (kN) OK
$P_{Nmin}$ = 1572.52 (kN) > $P_a$ = -2333.46 (kN) OK

7) Normal

a) Pile head rigid connection

・Acting force for center of pile group
$V_0$ = 80819.68 (kN)

・Calculation of displacement
$\delta_x$ = 0.807 (mm)

$H_0$ = 4765.33 (kN) $\delta_y$ = 3.511 (mm)

$M_0$ = 71121.32 (kN·m) $\alpha$ = $0.05892\times10^{-3}$ (rad)

・Acting force at pile head

| Pile No. | $x_i$ (m) | Quantities | $\theta_i$ (degree) | $P_{Ni}$ (kN) | $P_{Hi}$ (kN) | $M_{ti}$ (kN·m) | $V_i$ (kN) | $H_i$ (kN) |
|---|---|---|---|---|---|---|---|---|
| 1 | -11.400 | 8 | -0.0 | 1167.14 | 85.10 | -128.74 | 1167.14 | 85.10 |
| 2 | -7.600 | 8 | -0.0 | 1259.16 | 85.10 | -128.74 | 1259.16 | 85.10 |
| 3 | -3.800 | 8 | -0.0 | 1351.18 | 85.10 | -128.74 | 1351.18 | 85.10 |
| 4 | 0.000 | 8 | -0.0 | 1443.21 | 85.10 | -128.74 | 1443.21 | 85.10 |
| 5 | 3.800 | 8 | -0.0 | 1535.23 | 85.10 | -128.74 | 1535.23 | 85.10 |
| 6 | 7.600 | 8 | -0.0 | 1627.26 | 85.10 | -128.74 | 1627.26 | 85.10 |
| 7 | 11.400 | 8 | -0.0 | 1719.28 | 85.10 | -128.74 | 1719.28 | 85.10 |

$\delta_{max}$ = 0.807 (mm) < $\delta_a$ = 15.00 (mm) OK

$P_{Nmax}$ = 1719.28 (kN) < $R_a$ = 2633.03 (kN) OK

$P_{Nmin}$ = 1167.14 (kN) > $P_a$ = -1258.25 (kN) OK

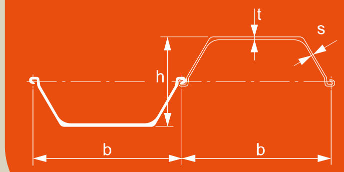
# APPENDIX A-8

## Structural Calculation of Steel Sheet Pile Works

Dimensions and sectional performance of supposed steel products

# U-Sections

## Steel Sheet Pile




The advantages of U-sections are multiple:

- a wide range of sections forming several series with various geometrical characteristics, allowing a technically and economically optimum choice for each specific project;
- combination of great profile depth with large flange thickness giving excellent statical properties;
- the symmetrical form of the single element has made these sheet piles particularly convenient for re-use;
- the possibility of assembling and crimping the piles into pairs at the mill improves installation quality and performance;
- easy fixing of tie-rods and swivelling attachments, even under water;
- good corrosion resistance, the steel being thickest at the critical corrosion points.

| Section | Width | Height | Thickness | | Sectional area | Mass | | Moment of inertia | Elastic section modulus | Static moment | Plastic section modulus | Class[1] | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | b mm | h mm | t mm | s mm | $cm^2/m$ | single pile kg/m | wall $kg/m^2$ | $cm^4/m$ | $cm^3/m$ | $cm^3/m$ | $cm^3/m$ | S 240 GP | S 270 GP | S 320 GP | S 355 GP | S 390 GP | S 430 GP | S 460 AP |
| **AU™ sections** | | | | | | | | | | | | | | | | | | |
| AU 14 | 750 | 408 | 10.0 | 8.3 | 132 | 77.9 | **104** | 28680 | **1405** | 820 | 1663 | 2 | 2 | 3 | 3 | 3 | 3 | 3 |
| AU 16 | 750 | 411 | 11.5 | 9.3 | 147 | 86.3 | **115** | 32850 | **1600** | 935 | 1891 | 2 | 2 | 2 | 2 | 2 | 3 | 3 |
| AU 18 | 750 | 441 | 10.5 | 9.1 | 150 | 88.5 | **118** | 39300 | **1780** | 1030 | 2082 | 2 | 3 | 3 | 3 | 3 | 3 | 3 |
| AU 20 | 750 | 444 | 12.0 | 10.0 | 165 | 96.9 | **129** | 44440 | **2000** | 1155 | 2339 | 2 | 2 | 2 | 3 | 3 | 3 | 3 |
| AU 23 | 750 | 447 | 13.0 | 9.5 | 173 | 102.1 | **136** | 50700 | **2270** | 1285 | 2600 | 2 | 2 | 2 | 3 | 3 | 3 | 3 |
| AU 25 | 750 | 450 | 14.5 | 10.2 | 188 | 110.4 | **147** | 56240 | **2500** | 1420 | 2866 | 2 | 2 | 2 | 2 | 2 | 3 | 3 |
| **PU® sections** | | | | | | | | | | | | | | | | | | |
| PU 12 | 600 | 360 | 9.8 | 9.0 | 140 | 66.1 | **110** | 21600 | **1200** | 715 | 1457 | 2 | 2 | 2 | 2 | 2 | 2 | 3 |
| PU 12-10/10 | 600 | 360 | 10.0 | 10.0 | 148 | 69.6 | **116** | 22580 | **1255** | 755 | 1535 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| PU 18$^{-1}$ | 600 | 430 | 10.2 | 8.4 | 154 | 72.6 | **121** | 35950 | **1670** | 980 | 1988 | 2 | 2 | 2 | 2 | 2 | 3 | 3 |
| PU 18 | 600 | 430 | 11.2 | 9.0 | 163 | 76.9 | **128** | 38650 | **1800** | 1055 | 2134 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| PU 18$^{+1}$ | 600 | 430 | 12.2 | 9.5 | 172 | 81.1 | **135** | 41320 | **1920** | 1125 | 2280 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| PU 22$^{-1}$ | 600 | 450 | 11.1 | 9.0 | 174 | 81.9 | **137** | 46380 | **2060** | 1195 | 2422 | 2 | 2 | 2 | 2 | 2 | 3 | 3 |
| PU 22 | 600 | 450 | 12.1 | 9.5 | 183 | 86.1 | **144** | 49460 | **2200** | 1275 | 2580 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| PU 22$^{+1}$ | 600 | 450 | 13.1 | 10.0 | 192 | 90.4 | **151** | 52510 | **2335** | 1355 | 2735 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| PU 28$^{-1}$ | 600 | 452 | 14.2 | 9.7 | 207 | 97.4 | **162** | 60580 | **2680** | 1525 | 3087 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| PU 28 | 600 | 454 | 15.2 | 10.1 | 216 | 101.8 | **170** | 64460 | **2840** | 1620 | 3269 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| PU 28$^{+1}$ | 600 | 456 | 16.2 | 10.5 | 226 | 106.2 | **177** | 68380 | **3000** | 1710 | 3450 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| PU 32$^{-1}$ | 600 | 452 | 18.5 | 10.6 | 233 | 109.9 | **183** | 69210 | **3065** | 1745 | 3525 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| PU 32 | 600 | 452 | 19.5 | 11.0 | 242 | 114.1 | **190** | 72320 | **3200** | 1825 | 3687 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| PU 32$^{+1}$ | 600 | 452 | 20.5 | 11.4 | 251 | 118.4 | **197** | 75410 | **3340** | 1905 | 3845 | 2 | 2 | 2 | 2 | 2 | 2 | 2 |
| **GU® sections** | | | | | | | | | | | | | | | | | | |
| GU 6N | 600 | 309 | 6.0 | 6.0 | 89 | 41.9 | **70** | 9670 | **625** | 375 | 765 | 3 | 3 | 3 | 4 | 4 | 4 | 4 |
| GU 7N | 600 | 310 | 6.5 | 6.4 | 94 | 44.1 | **74** | 10450 | **675** | 400 | 825 | 3 | 3 | 3 | 3 | 3 | 4 | 4 |
| GU 7S | 600 | 311 | 7.2 | 6.9 | 100 | 46.3 | **77** | 11540 | **740** | 440 | 900 | 2 | 2 | 3 | 3 | 3 | 3 | 3 |
| GU 7HWS | 600 | 312 | 7.3 | 6.9 | 101 | 47.4 | **79** | 11620 | **745** | 445 | 910 | 2 | 2 | 3 | 3 | 3 | 3 | 3 |
| GU 8N | 600 | 312 | 7.5 | 7.1 | 103 | 48.5 | **81** | 12010 | **770** | 460 | 935 | 2 | 2 | 3 | 3 | 3 | 3 | 3 |
| GU 8S | 600 | 313 | 8.0 | 7.5 | 108 | 50.8 | **85** | 12800 | **820** | 490 | 995 | 2 | 2 | 2 | 3 | 3 | 3 | 3 |

Wale

# Fers U normaux européens

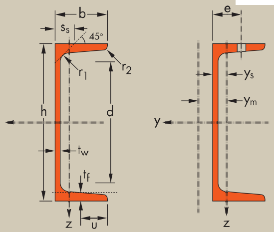
Dimensions: DIN 1026-1: 2009, NF A 45-202: 1986
Tolérances: EN 10279: 2000
Etat de surface: conforme à EN 10163-3: 2004, classe C, sous-classe 1

# European standard channels

Dimensions: DIN 1026-1: 2009, NF A 45-202: 1986
Tolerances: EN 10279: 2000
Surface condition: according to EN 10163-3: 2004, class C, subclass 1

# Europäische U-Stahl-Normalprofile

Abmessungen: DIN 1026-1: 2009, NF A 45-202: 1986
Toleranzen: EN 10279: 2000
Oberflächenbeschaffenheit: Gemäß EN 10163-3: 2004, Klasse C, Untergruppe 1




# UPN

A-307

| Désignation Designation Bezeichnung | G | h | b | $t_w$ | $t_f$ | $r_1$ | $r_2$ | A | d | Ø | $e_{min}$ | $e_{max}$ | $A_L$ | $A_G$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Dimensions Abmessungen | | | | | | | Dimensions de construction Dimensions for detailing Konstruktionsmaße | | | | Surface Oberfläche | |
| | kg/m | mm | mm | mm | mm | mm | mm | mm² x10² | mm | | mm | mm | m²/m | m²/t |
| UPN 50* | 5,59 | 50 | 38 | 5,0 | 7,0 | 7,0 | 3,5 | 7,12 | 21 | - | - | - | 0,232 | 42,22 |
| UPN 65* | 7,09 | 65 | 42 | 5,5 | 7,5 | 7,5 | 4,0 | 9,03 | 34 | - | - | - | 0,273 | 39,57 |
| UPN 80* | 8,64 | 80 | 45 | 6,0 | 8,0 | 8,0 | 4,0 | 11,0 | 47 | - | - | - | 0,312 | 37,10 |
| UPN 100* | 10,6 | 100 | 50 | 6,0 | 8,5 | 8,5 | 4,5 | 13,5 | 64 | - | - | - | 0,372 | 35,10 |
| UPN 120 | 13,4 | 120 | 55 | 7,0 | 9,0 | 9,0 | 4,5 | 17,0 | 82 | - | - | - | 0,434 | 32,52 |
| UPN 140 | 16,0 | 140 | 60 | 7,0 | 10,0 | 10,0 | 5,0 | 20,4 | 98 | M 12 | 33 | 37 | 0,489 | 30,54 |
| UPN 160 | 18,8 | 160 | 65 | 7,5 | 10,5 | 10,5 | 5,5 | 24,0 | 115 | M 12 | 34 | 42 | 0,546 | 28,98 |
| UPN 180 | 22,0 | 180 | 70 | 8,0 | 11,0 | 11,0 | 5,5 | 28,0 | 133 | M 16 | 38 | 41 | 0,611 | 27,80 |
| UPN 200 | 25,3 | 200 | 75 | 8,5 | 11,5 | 11,5 | 6,0 | 32,2 | 151 | M 16 | 39 | 46 | 0,661 | 26,15 |
| UPN 220 | 29,4 | 220 | 80 | 9,0 | 12,5 | 12,5 | 6,5 | 37,4 | 167 | M 16 | 40 | 51 | 0,718 | 24,46 |
| UPN 240 | 33,2 | 240 | 85 | 9,5 | 13,0 | 13,0 | 6,5 | 42,3 | 184 | M 20 | 46 | 50 | 0,775 | 23,34 |
| UPN 260 | 37,9 | 260 | 90 | 10,0 | 14,0 | 14,0 | 7,0 | 48,3 | 200 | M 22 | 50 | 52 | 0,834 | 22,00 |
| UPN 280 | 41,8 | 280 | 95 | 10,0 | 15,0 | 15,0 | 7,5 | 53,3 | 216 | M 22 | 52 | 57 | 0,890 | 21,27 |
| UPN 300 | 46,2 | 300 | 100 | 10,0 | 16,0 | 16,0 | 8,0 | 58,8 | 232 | M 24 | 55 | 59 | 0,950 | 20,58 |
| UPN 320* | 59,5 | 320 | 100 | 14,0 | 17,5 | 17,5 | 8,8 | 75,8 | 246 | M 22 | 58 | 62 | 0,982 | 16,50 |
| UPN 350 | 60,6 | 350 | 100 | 14,0 | 16,0 | 16,0 | 8,0 | 77,3 | 282 | M 22 | 56 | 62 | 1,05 | 17,25 |
| UPN 380* | 63,1 | 380 | 102 | 13,5 | 16,0 | 16,0 | 8,0 | 80,4 | 313 | M 24 | 59 | 60 | 1,11 | 17,59 |
| UPN 400* | 71,8 | 400 | 110 | 14,0 | 18,0 | 18,0 | 9,0 | 91,5 | 324 | M 27 | 61 | 62 | 1,18 | 16,46 |

| | h ≤ 300 | h > 300 |
|---|---|---|
| u | $\frac{b}{2}$ | $\frac{b - t_w}{2}$ |
| Inclinaison des ailes Flange slope Flanschneigung | 8% | 5% |

* Tonnage minimum et conditions de livraison nécessitent un accord préalable.
* Minimum tonnage and delivery conditions upon agreement.
* Mindestbestellmenge und Lieferbedingungen nach Vereinbarung.

Notations pages 219-223 / Bezeichnungen Seiten 219-223

| Désignation Designation Bezeichnung | G | $I_y$ | $W_{el.y}$ | $W_{pl.y}$■ | $i_y$ | $A_{vz}$ | $I_z$ | $W_{el.z}$ | $W_{pl.z}$ | $i_z$ | $s_s$ | $I_t$ | $I_w$ | $y_s$ | $y_m$ | S235 | S355 | S460 | S235 | S355 | S460 | EN 10025-2: 2004 | EN 10025-4: 2004 | EN 10225:2009 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Valeurs statiques / Section properties / Statische Kennwerte | | | | | | | | | | | | | | Classification EN 1993-1-1: 2005 | | | | | | | | |
| | | axe fort y-y strong axis y-y starke Achse y-y | | | | | axe faible z-z weak axis z-z schwache Achse z-z | | | | | | | | | Pure bending y-y | | | Pure compression | | | | | |
| | kg/m | mm⁴ x10⁴ | mm³ x10³ | mm³ x10³ | mm x10 | mm² x10² | mm⁴ x10⁴ | mm³ x10³ | mm³ x10³ | mm x10 | mm | mm⁴ x10⁴ | mm⁶ x10⁹ | mm x10 | mm x10 | | | | | | | | | |
| UPN 50 | 5,59 | 26,4 | 10,6 | 13,1 | 1,92 | 2,77 | 9,12 | 3,75 | 6,78 | 1,13 | 16,7 | 1,12 | 0,03 | 1,37 | 2,47 | 1 | 1 | - | 1 | 1 | - | ✓ | ✓ | ✓ |
| UPN 65 | 7,09 | 57,5 | 17,7 | 21,7 | 2,52 | 3,68 | 14,1 | 5,07 | 9,38 | 1,25 | 18,0 | 1,61 | 0,08 | 1,42 | 2,60 | 1 | 1 | - | 1 | 1 | - | ✓ | ✓ | ✓ |
| UPN 80 | 8,64 | 106 | 26,5 | 32,3 | 3,10 | 4,90 | 19,4 | 6,36 | 11,9 | 1,33 | 19,4 | 2,20 | 0,18 | 1,45 | 2,67 | 1 | 1 | 1 | 1 | 1 | 1 | ✓ | ✓ | ✓ |
| UPN 100 | 10,6 | 206 | 41,2 | 49,0 | 3,91 | 6,46 | 29,3 | 8,49 | 16,2 | 1,47 | 20,3 | 2,81 | 0,41 | 1,55 | 2,93 | 1 | 1 | 1 | 1 | 1 | 1 | ✓ | ✓ | ✓ |
| UPN 120 | 13,4 | 364 | 60,7 | 72,6 | 4,62 | 8,80 | 43,2 | 11,1 | 21,2 | 1,59 | 22,2 | 4,15 | 0,90 | 1,60 | 3,03 | 1 | 1 | 1 | 1 | 1 | 1 | ✓ | ✓ | ✓ |
| UPN 140 | 16,0 | 605 | 86,4 | 103 | 5,45 | 10,4 | 62,7 | 14,8 | 28,3 | 1,75 | 23,9 | 5,68 | 1,80 | 1,75 | 3,37 | 1 | 1 | 1 | 1 | 1 | 1 | ✓ | ✓ | ✓ |
| UPN 160 | 18,8 | 925 | 116 | 138 | 6,21 | 12,6 | 85,3 | 18,3 | 35,2 | 1,89 | 25,3 | 7,39 | 3,26 | 1,84 | 3,56 | 1 | 1 | 1 | 1 | 1 | 1 | ✓ | ✓ | ✓ |
| UPN 180 | 22,0 | 1350 | 150 | 179 | 6,95 | 15,1 | 114 | 22,4 | 42,9 | 2,02 | 26,7 | 9,55 | 5,57 | 1,92 | 3,75 | 1 | 1 | 1 | 1 | 1 | 1 | ✓ | ✓ | ✓ |
| UPN 200 | 25,3 | 1910 | 191 | 228 | 7,70 | 17,7 | 148 | 27,0 | 51,8 | 2,14 | 28,1 | 11,9 | 9,07 | 2,01 | 3,94 | 1 | 1 | 1 | 1 | 1 | 1 | ✓ | ✓ | ✓ |
| UPN 220 | 29,4 | 2690 | 245 | 292 | 8,48 | 20,6 | 197 | 33,6 | 64,1 | 2,30 | 30,3 | 16,0 | 14,6 | 2,14 | 4,20 | 1 | 1 | 1 | 1 | 1 | 1 | ✓ | ✓ | ✓ |
| UPN 240 | 33,2 | 3600 | 300 | 358 | 9,22 | 23,7 | 248 | 39,6 | 75,7 | 2,42 | 31,7 | 19,7 | 22,1 | 2,23 | 4,39 | 1 | 1 | 1 | 1 | 1 | 1 | ✓ | ✓ | ✓ |
| UPN 260 | 37,9 | 4820 | 371 | 442 | 9,99 | 27,1 | 317 | 47,7 | 91,6 | 2,56 | 33,9 | 25,5 | 33,3 | 2,36 | 4,66 | 1 | 1 | 1 | 1 | 1 | 1 | ✓ | ✓ | ✓ |
| UPN 280 | 41,8 | 6280 | 448 | 532 | 10,9 | 29,3 | 399 | 57,2 | 109 | 2,74 | 35,6 | 31,0 | 48,5 | 2,53 | 5,02 | 1 | 1 | 1 | 1 | 1 | 1 | ✓ | ✓ | ✓ |
| UPN 300 | 46,2 | 8030 | 535 | 632 | 11,7 | 31,8 | 495 | 67,8 | 130 | 2,90 | 37,3 | 37,4 | 69,1 | 2,70 | 5,41 | 1 | 1 | 1 | 1 | 1 | 1 | ✓ | ✓ | ✓ |
| UPN 320 | 59,5 | 10870 | 679 | 826 | 12,1 | 47,1 | 597 | 80,6 | 152 | 2,81 | 43,0 | 66,7 | 96,1 | 2,60 | 4,82 | 1 | 1 | - | 1 | 1 | - | ✓ | ✓ | ✓ |
| UPN 350 | 60,6 | 12840 | 734 | 918 | 12,9 | 50,8 | 570 | 75,0 | 143 | 2,72 | 40,7 | 61,2 | 114 | 2,40 | 4,45 | 1 | 1 | - | 1 | 1 | - | ✓ | ✓ | ✓ |
| UPN 380 | 63,1 | 15760 | 829 | 1010 | 14,0 | 53,2 | 615 | 78,7 | 148 | 2,77 | 40,3 | 59,1 | 146 | 2,38 | 4,58 | 1 | 1 | - | 1 | 1 | - | ✓ | ✓ | ✓ |
| UPN 400 | 71,8 | 20350 | 1020 | 1240 | 14,9 | 58,6 | 846 | 102 | 190 | 3,04 | 44,0 | 81,6 | 221 | 2,65 | 5,11 | 1 | 1 | - | 1 | 1 | - | ✓ | ✓ | ✓ |

■ $W_{pl.y}$ est calculé selon l'hypothèse d'un diagramme de contraintes bi-rectangulaire et n'est applicable que si deux ou plusieurs fers U sont associés de facon à constituer une section doublement symétrique pour laquelle un moment de flexion agissant dans le plan du centre de gravité n'engendre pas de torsion.

■ $W_{pl.y}$ is determined assuming a bi-rectangular stress block distribution. Thus, the given value applies only if two or more channels are combined in such a way to form a double symmetric cross-section so that the bending moment acting in the plane of the centre of gravity will not lead to torsion.

■ Für die Berechnung von $W_{pl.y}$ wurde eine doppelrechteckige Spannungsverteilung angenommen. Der angegebene Wert ist daher nur anwendbar, wenn zwei oder mehr U-Profile so miteinander kombiniert sind, dass sie einen doppelsymmetrischen Querschnitt bilden, womit ein Biegemoment, das in der Schwerpunktebene angreift, keine Torsion hervorruft.

# 規格 鋼矢板施工指針等)

## 機械的性質 Mechanical features

| 鋼種 | 種別記号 (Symbol) | 引張強さ (N/mm²) (Tensile strength) | 降伏点 (N/mm²) (yield point) | 伸び (%) | 鋼矢板施工指針該当鋼種 |
|---|---|---|---|---|---|
| 高張力鋼 (セミハイテン) High tension steel | **HT690 (KST-3)** | 690以上 | 440以上 | 19以上 | 3種 |
| | **HT740 (KST-4)** | 740以上 | 540以上 | 17以上 | 4種 |
| 普通鋼 | **SS400 (JIS G 3101)** | 400~510 | φ40以下 235以上 | φ25以下 20以上 | 1種 |
| | | | φ40超 215以上 | φ25超 22以上 | |
| | **SS490 (JIS G 3101)** | 490~610 | φ40以下 275以上 | φ25以下 18以上 | 2種 |
| | | | φ40超 255以上 | φ25超 20以上 | |

ただし、JIS Z 2241 金属材料引張試験片の14A号試験片によります。

## 許容引張応力度 Allowable stress

タイロッドの許容引張応力度については、港湾構造物設計基準および鋼矢板施工指針に基づき、地震を考えない場合は使用材質の保証降伏点の40%、耐震設計の場合は保証降伏点の60%とされています。これをもとにしてタイロッドの許容応力度を算出しますと次のようになります。

Allowable stress

| 鋼種 | 種別記号 | | タイロッドの許容応力度 Normal condition (N/mm²) | Seismic condition (N/mm²) |
|---|---|---|---|---|
| 高張力鋼 (セミハイテン) | **HT690 (KST-3)** | | 176 | 264 |
| | **HT740 (KST-4)** | | 216 | 324 |
| 普通鋼 | **SS400** | φ40以下 | 94 | 141 |
| | | φ40超 | 86 | 129 |
| | **SS490** | φ40以下 | 110 | 165 |
| | | φ40超 | 102 | 153 |

A-308

神鋼セミハイテンタイロッド



# 本体の許容張力表 Allowable tensile force

| タイロッド呼び径 (mm) Diameter | タイロッド断面積 (mm²) Sectional area | Normal condition 常時(kN) HT690 (KST-3) | HT740 (KST-4) | SS400 | SS490 | Seismic condition 地震時(kN) HT690 (KST-3) | HT740 (KST-4) | SS400 | SS490 |
|---|---|---|---|---|---|---|---|---|---|
| φ25 | 490.9 | 86 | 106 | 46 | 53 | 129 | 159 | 69 | 80 |
| φ28 | 615.8 | 108 | 133 | 57 | 67 | 162 | 199 | 86 | 101 |
| φ32 | 804.2 | 141 | 173 | 75 | 88 | 212 | 260 | 113 | 132 |
| φ36 | 1018 | 179 | 219 | 95 | 111 | 268 | 329 | 143 | 167 |
| φ38 | 1134 | 199 | 244 | 106 | 124 | 299 | 367 | 159 | 187 |
| φ42 | 1385 | 243 | 299 | 119 | 141 | 365 | 448 | 178 | 211 |
| φ44 | 1521 | 267 | 328 | 130 | 155 | 401 | 492 | 196 | 232 |
| φ46 | 1662 | 292 | 358 | 142 | 169 | 438 | 538 | 214 | 254 |
| φ48 | 1810 | 318 | 390 | 155 | 184 | 477 | 586 | 233 | 276 |
| φ50 | 1963 | 345 | 424 | 168 | 200 | 518 | 636 | 253 | 300 |
| φ52 | 2124 | 373 | 458 | 182 | 216 | 560 | 688 | 273 | 324 |
| φ55 | 2376 | 418 | 513 | 204 | 242 | 627 | 769 | 306 | 363 |
| φ60 | 2827 | 497 | 610 | 243 | 288 | 746 | 915 | 364 | 432 |
| φ65 | 3318 | 583 | 716 | 285 | 338 | 875 | 1075 | 428 | 507 |
| φ70 | 3848 | 677 | 831 | 330 | 392 | 1015 | 1246 | 496 | 588 |
| φ75 | 4418 | 777 | 954 | 379 | 450 | 1166 | 1431 | 569 | 675 |
| φ80 | 5027 | 884 | 1085 | 432 | 512 | 1327 | 1628 | 648 | 769 |
| φ85 | 5675 | 998 | 1225 | 488 | 578 | 1498 | 1838 | 732 | 868 |
| φ90 | 6362 | 1119 | 1374 | 547 | 648 | 1679 | 2061 | 820 | 973 |
| ※許容応力度 (N/mm²) | | 176 | 216 | 94 (86) | 110 (102) | 264 | 324 | 141 (129) | 165 (153) |

※許容応力度の( )内数字はTR径が40mmを超え100mm以下の場合の値を示す。
※常時許容張力=使用材質の降伏点×0.4
※地震時許容張力=使用材質の降伏点×0.6
※腐食代は加味されておりません。

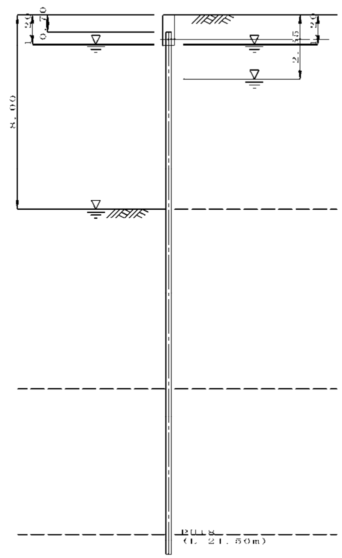
神鋼セミハイテンタイロッド

A-8-1 Structural calculation of steel sheet pile works (Type A)

<Bracing style sheet pile wall>

1Design conditions

1-1 Structural figure




| Depth (m) | Soil type | γ (kN/m3) | Internal friction angle (degree) | C (kN/m2) | N-value |
|---|---|---|---|---|---|
| 8.00 | Sandy soil | 19.0 | 30.0 | 0.0 | |
| 15.37 | Sandy soil | 19.0 | 25.0 | 0.0 | |
| 21.37 | Cohesive soil | 18.0 | 0.0 | 79.0 | |
| | Sandy soil | 19.0 | 38.0 | 0.0 | |

## 1-2 Structural dimensions

Design height $H$ = 8.00 m
Top of sheet pile $H_{lt}$ = 0.70 m
Inside water level $L_{wa}$ = 2.65 m(Normal condition)
$L_{wa}'$ = 1.20 m(Seismic condition)
Outside water level $L_{wp}$ = 8.00 m(Normal condition)
$L_{wp}'$ = 1.20 m(Seismic condition)

Tie-rod installation position $H_t$ = 1.00 m
Tie-rod horizontal intervals $l$ = 2.40 m
Tie-rod installation angle $\theta$ = 0.00000 degree

Method of calculation; Free earth support method

Virtual supporting point; Design front ground level

## 1-3 Design constant parameters

Unit weight of water $\gamma_w$ = 10.0 kN/m$^3$

Type of water pressure; Trapezoid shape

Design Seismic coefficient (in the air) $k$ = 0.080
(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma\gamma_t \cdot h_i + \Sigma\gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma\gamma_t \cdot h_i + \Sigma(\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake; Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma\gamma h + 2Q}{2C}} \cdot \tan\theta$$

Safety factor of penetration depth; 1.50 (Normal condition)
1.20 (Seismic condition)



## 1-4 Coefficient of horizontal subgrade reaction

Formula:

$$K_h = 6910 \times N'^{0.406}$$

where, N: Average N-value

Result of boring survey

| No | Depth m | N-value | No | Depth m | N-value |
|---|---|---|---|---|---|
| 1 | 1.00 | 8 | 11 | 10.87 | 10 |
| 2 | 2.00 | 8 | 12 | 11.87 | 8 |
| 3 | 3.00 | 8 | 13 | 12.87 | 8 |
| 4 | 4.00 | 8 | 14 | 13.87 | 9 |
| 5 | 5.00 | 8 | | | |
| 6 | 6.00 | 8 | | | |
| 7 | 7.00 | 8 | | | |
| 8 | 8.00 | 8 | | | |
| 9 | 8.87 | 11 | | | |
| 10 | 9.87 | 5 | | | |

## 1-5 Overburden pressure

Active pressure side
$Q_a$ = 10.0 kN/m$^2$ (Normal condition)
$Q_a'$ = 5.0 kN/m$^2$ (Seismic condition)

Passive pressure side
$Q_p$ = 0.0 kN/m$^2$ (Normal condition)
$Q_p'$ = 0.0 kN/m$^2$ (Seismic condition)

## 1-6 Soil constant

| No | Depth m | type | N-value | $\gamma$ kN/m$^3$ | $\gamma'$ kN/m$^3$ | $\phi$ Deg. | C kN/m$^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 8.00 | S | 8.0 | 19.00 | 10.00 | 30.0 | 0.0 | 0.0 |
| 2 | 15.37 | S | 9.0 | 19.00 | 10.00 | 25.0 | 0.0 | 0.0 |
| 3 | 21.37 | C | 13.0 | 18.00 | 9.00 | 0.0 | 79.0 | 0.0 |
| 4 | 30.00 | S | 44.0 | 19.00 | 10.00 | 38.0 | 0.0 | 0.0 |

Depth : Distance from the top of sheet pile C : Adhesive force
type : Sandy soil(S)、Cohesive soil(C)、Medium soil(M) a : Angle of adhesive force

N value : average value
$\gamma$ : unit weight of humid soil
$\gamma'$ : unit weight of soil under the water
$\phi$ : internal friction angle

## 1-7 Sheet pile

Type PU18
Young modulus $E$ = 200000 N/mm$^2$
Second moment of area $I_0$ = 38650 cm$^4$ (before corrosion)
Modulus of section $Z_0$ = 1800 cm$^3$ (before corrosion)

Corrosion allowance $t_1$=0.00 mm (front side) $t_2$=0.00 mm (back side)
Corrosion ratio (against $Zo$) $\eta$=1.00
Joint efficiency(against $Zo$) $\mu$=1.00
Allowable stress $\sigma_a$ =165 N/mm$^2$ (Normal condition)
$\sigma_a'$=190 N/mm$^2$ (Seismic condition)

1-8 Tie-rod

Diameter of tie-rod Calculated

Corrosion Considered (0.00mm)

Allowable stress $\sigma_a$ =176 N/mm$^2$ (Normal condition)
$\sigma_a'$=264 N/mm$^2$ (Seismic condition)

1-9 Wale

Type 2 x UPN-220x80

Modulus of section; 490 cm$^3$

Corrosion ; considered (0.00mm)
Corrosion ratio; $\eta$ =1.00

Allowable stress $\sigma_a$ =140 N/mm$^2$ (Normal condition)
$\sigma_a'$=161 N/mm$^2$ (Seismic condition)

1-10 Bracing sheet pile

Calculation method Chang's formula

Crest elevation of bracing works 0.30 m (distance from tie-rod)

Materials PU12
Young's modulus E = 200000 N/mm$^2$
Second moment of area $I_0$= 21600 cm$^4$ (before corrosion)
Modulus of section $Z_0$= 1200 cm$^3$ (before corrosion)

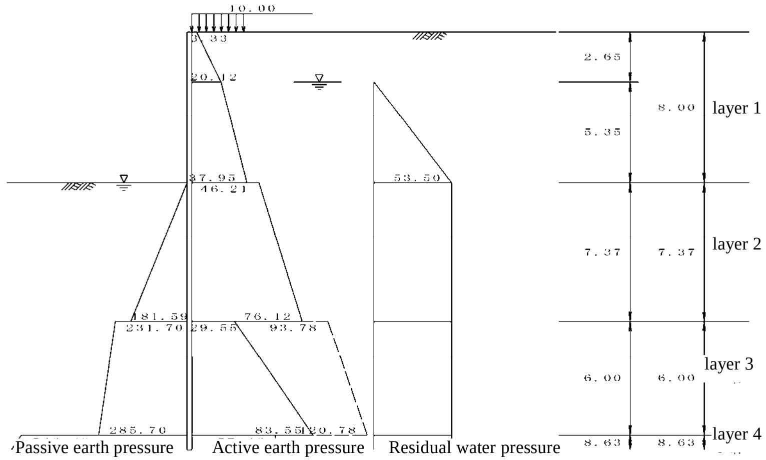
Corrosion allowance $t_1$=0.00 mm (front side) $t_2$=0.00 mm (back side)
Corrosion ratio (against $I_0$) $\eta$ =1.00
Corrosion ratio (against $Z_0$) $\eta$ =1.00
Joint efficiency (against $I_0$) $\mu$ =1.00
Joint efficiency(against $Z_0$) $\mu$ =1.00

Allowable stress $\sigma_a$ =165 N/mm$^2$ (Normal condition)
$\sigma_a'$=190 N/mm$^2$ (Seismic condition)

Allowable displacement $\delta_a$ = 50.0 mm (Normal condition)
$\delta_a'$= 75.0 mm (Seismic condition)

2 Calculation of lateral pressure

2-1 Normal condition




2-1-1 Soil constant of passive side

| | Depth m | Soil type | $\gamma$ kN/m$^3$ | $\phi$ degree | C kN/m$^2$ | $\Sigma\gamma$ h+Qa kN/m$^2$ | Ka | Ka ×cos δ |
|---|---|---|---|---|---|---|---|---|
| 1 | 0.00～ 2.65 | sandy soil | 19.0 | 30.0 | ――― ――― | 10.000 60.350 | 0.33333 0.33333 | 0.33333 0.33333 |
| 2 | 2.65～ 8.00 | sandy soil | 10.0 | 30.0 | ――― ――― | 60.350 113.850 | 0.33333 0.33333 | 0.33333 0.33333 |
| 3 | 8.00～ 15.37 | sandy soil | 10.0 | 25.0 | ――― ――― | 113.850 187.550 | 0.40586 0.40586 | 0.40586 0.40586 |
| 4 | 15.37～ 21.37 | cohesive soil | 9.0 | ―― | 79.0 79.0 | 187.550 241.550 | ――― ――― | ――― ――― |
| 5 | 21.37～ 30.00 | sandy soil | 10.0 | 38.0 | ――― ――― | 241.550 327.850 | 0.23788 0.23788 | 0.23788 0.23788 |

・Coefficient of passive earth pressure ($K_a$) in sandy soil is obtained by following formula
$\delta$ =0.00degree、$\beta$ =0.00degree、$\theta$ =0.00degree

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

### 2-1-2 Soil constant of passive side

| No | Depth m | Soil type | $\gamma$ kN/m$^3$ | $\phi$ degree | C kN/m$^2$ | $\Sigma\gamma h+Qp$ kN/m$^2$ | Kp | Kp $\times\cos\delta$ |
|---|---|---|---|---|---|---|---|---|
| 3 | 8.00～ 15.37 | sandy soil | 10.0 | 25.0 | ——— ——— | 0.000 73.700 | 2.46391 2.46391 | 2.46391 2.46391 |
| 4 | 15.37～ 21.37 | cohesive soil | 9.0 | 0.0 | 79.0 79.0 | 73.700 127.700 | ——— ——— | ——— ——— |
| 5 | 21.37～ 30.00 | sandy soil | 10.0 | 38.0 | ——— ——— | 127.700 214.000 | 4.20375 4.20375 | 4.20375 4.20375 |

・Coefficient of passive earth pressure ($K_p$) in sandy soil is obtained by following formula
$\delta$ =0.00degree、$\beta$ =0.00degree、$\theta$ =0.00degree

$$K_p = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta-\theta)\cdot\left[1-\sqrt{\dfrac{\sin(\phi-\delta)\cdot\sin(\phi+\beta-\theta)}{\cos(\delta-\theta)\cdot\cos(-\beta)}}\right]^2}$$

### 2-1-3 Lateral pressure

| No | Depth m | Passive side | | | residual water pressure | Passive side |
|---|---|---|---|---|---|---|
| | | Pa1 kN/m$^2$ | Pa2 kN/m$^2$ | Pa kN/m$^2$ | Pw kN/m$^2$ | Pp kN/m$^2$ |
| 1 | 0.00～ 2.65 | 3.33 20.12 | ——— ——— | 3.33 20.12 | ——— ——— | ——— ——— |
| 2 | 2.65～ 8.00 | 20.12 37.95 | ——— ——— | 20.12 37.95 | 0.00 53.50 | ——— ——— |
| 3 | 8.00～ 15.37 | 46.21 76.12 | ——— ——— | 46.21 76.12 | 53.50 53.50 | 0.00 181.59 |
| 4 | 15.37～ 21.37 | 29.55 83.55 | 93.78 120.78 | 93.78 120.78 | 53.50 53.50 | 231.70 285.70 |
| 5 | 21.37～ 30.00 | 57.46 77.99 | ——— ——— | 57.46 77.99 | 53.50 53.50 | 536.82 899.60 |

・Calculation formula of passive earth pressure

Sandy soil $P_{a1}=K_a\cdot[\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}]\cdot\cos\delta$

Cohesive $P_{a1}=\Sigma\gamma h+Q-2C$
$P_{a2}=K_c\cdot(\Sigma\gamma h+Q)$

Medium soil $P_{a1}=[K_a(\Sigma\gamma h+Q)-2C\sqrt{K_a}]\cdot\cos\delta$

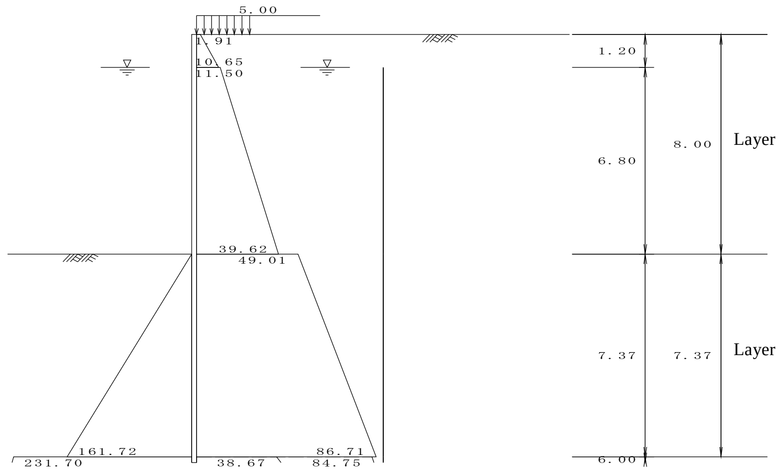
・Calculation formula of passive earth pressure

Sandy soil $P_p=K_p\cdot[\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}]\cdot\cos\delta$

Cohesive soil $P_p=\Sigma\gamma h+Q+2C$

Medium soil $P_p=[K_p(\Sigma\gamma h+Q)+2C\sqrt{K_p}]\cdot\cos\delta$

### 2-2 Seismic condition



Passive earth pressure　Active earth pressure　Residual water pressure

### 2-2-1 Passive side soil constant

| No | Depth m | Soil type | $\gamma$ kN/m$^3$ | $\phi$ degree | C kN/m$^2$ | $\Sigma\gamma h+Q$ kN/m$^2$ | $\gamma w h w$ kN/m$^2$ | k (k') | $\theta$ degree | Ka | Ka $\times\cos\delta$ | $\zeta$ degree |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.00～ 1.20 | sandy soil | 19.0 | 30.0 | ——— ——— | 5.00 27.80 | 0.00 0.00 | 0.080 0.080 | 4.57 4.57 | 0.38296 0.38296 | 0.38296 0.38296 | ——— ——— |
| 2 | 1.20～ 8.00 | sandy soil | 10.0 | 30.0 | ——— ——— | 27.80 95.80 | 0.00 68.00 | 0.124 0.124 | 7.07 7.07 | 0.41358 0.41358 | 0.41358 0.41358 | ——— ——— |
| 3 | 8.00～ 15.37 | sandy soil | 10.0 | 25.0 | ——— ——— | 95.80 169.50 | 68.00 141.70 | 0.143 0.143 | 8.15 8.15 | 0.51157 0.51157 | 0.51157 0.51157 | ——— ——— |
| 4 | 15.37～ 21.37 | cohesive soil | 9.0 | ——— | 79.0 79.0 | 169.50 223.50 | 141.70 201.70 | 0.150 0.150 | 8.53 8.53 | ——— ——— | ——— ——— | 42.41 41.51 |
| 5 | 21.37～ 30.00 | sandy soil | 10.0 | 38.0 | ——— ——— | 223.50 309.80 | 201.70 288.00 | 0.153 0.153 | 8.72 8.72 | 0.32337 0.32337 | 0.32337 0.32337 | ——— ——— |

・Coefficient of sandy soil passive earth pressure ($K_a$) is obtained by following formula
$\delta$ =0.00degree、$\beta$ =0.00degree
compound angle $\theta=\tan^{-1}k$

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

$$\zeta=\tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}\cdot\tan\theta}$$

2-2-2 Passive side soil constant

| No | Depth m | Soil type | $\gamma$ kN/m$^3$ | $\phi$ degree | C kN/m$^2$ | $\Sigma\gamma h+Qp$ kN/m$^2$ | $\gamma w h w$ kN/m$^2$ | k (k') | $\theta$ degree | Kp | Kp $\times\cos\delta$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 3 | 8.00～ 15.37 | sandy soil | 10.00 | 25.0 | ――― ――― | 0.000 73.700 | 0.00 73.70 | 0.160 0.160 | 9.09 9.09 | 2.19428 2.19428 | 2.19428 2.19428 |
| 4 | 15.37～ 21.37 | cohesive soil | 9.00 | 0.0 | 79.0 79.0 | 73.700 127.700 | 73.70 133.70 | 0.162 0.162 | 9.22 9.22 | ――― ――― | ――― ――― |
| 5 | 21.37～ 30.00 | sandy soil | 10.00 | 38.0 | ――― ――― | 127.700 214.000 | 133.70 220.00 | 0.163 0.163 | 9.25 9.25 | 3.85788 3.85788 | 3.85788 3.85788 |

・Coefficient of sandy soil passive earth pressure（$K_p$） is obtained by following formula

$\delta$ =0.00degree、 $\beta$ =0.00degree

compound angle $\theta=\tan^{-1}k$

$$K_p = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta-\theta)\cdot\left[1-\sqrt{\dfrac{\sin(\phi-\delta)\cdot\sin(\phi+\beta-\theta)}{\cos(\delta-\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-2-3 Lateral earth pressure

| No | Depth m | Passive side Pa1 kN/m$^2$ | Passive side Pa2 kN/m$^2$ | Passive side Pa kN/m$^2$ | Residual water pressure Pw kN/m$^2$ | Passive side Pp kN/m$^2$ |
|---|---|---|---|---|---|---|
| 1 | 0.00～ 1.20 | 1.91 10.65 | ――― ――― | 1.91 10.65 | ――― ――― | ――― ――― |
| 2 | 1.20～ 8.00 | 11.50 39.62 | ――― ――― | 11.50 39.62 | 0.00 0.00 | ――― ――― |
| 3 | 8.00～ 15.37 | 49.01 86.71 | ――― ――― | 49.01 86.71 | 0.00 0.00 | 0.00 161.72 |
| 4 | 15.37～ 21.37 | 38.67 102.18 | 84.75 111.75 | 84.75 111.75 | 0.00 0.00 | 231.70 285.70 |
| 5 | 21.37～ 30.00 | 72.27 100.18 | ――― ――― | 72.27 100.18 | 0.00 0.00 | 492.65 825.59 |

・Calculation formula of passive earth pressure

Sandy soil $P_{a1}=K_a\cdot[\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}]\cdot\cos\delta$

Cohesive soil $P_{a1}=\dfrac{(\Sigma\gamma h+Q)\cdot\sin(\theta+\zeta)}{\cos\theta\cdot\sin\zeta} - \dfrac{C}{\cos\zeta\cdot\sin\zeta}$

Medium soil $P_{a1}=[K_a(\Sigma\gamma h+Q)-2C\sqrt{K_a}]\cdot\cos\delta$

・Calculation formula of passive earth pressure

Sandy soil $P_p=K_p\cdot[\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}]\cdot\cos\delta$

Cohesive soil $P_p=\Sigma\gamma h+Q+2C$

Medium soil $P_p=[K_p(\Sigma\gamma h+Q)+2C\sqrt{K_p}]\cdot\cos\delta$

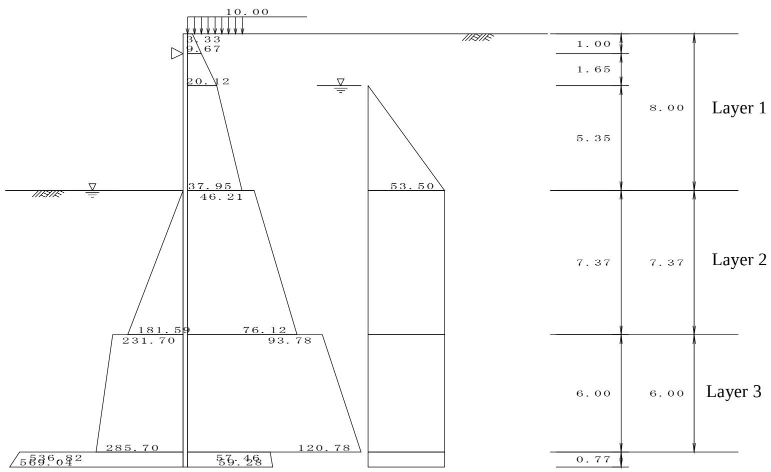
3 Calculation of penetration depth

Penetration depth of sheet pile wall is the depth from the design basement where $F_s\times(\Sigma M_a+\Sigma M_w+\Sigma M_{dw}+\Sigma M_t)\leqq\Sigma M_p$ is established.

where,

$\Sigma M_a$ : Moment by active earth pressure
$\Sigma M_w$ : Moment by residual water pressure
$\Sigma M_p$ : Moment by passive earth pressure
$\Sigma M_{dw}$ : Moment by dynamic water pressure during earthquake
$\Sigma M_t$ : Moment by arbitrary load
$F_s$ : Safety factor（Normal condition=1.50、Seismic condition=1.20）

3-1 Normal condition




Passive earth pressure　Active earth pressure　Residual water pressure

Moment around the tie-rod point is calculated supposing that penetration depth is 14.14 m

3-1-1 Moment by active earth pressure

| No | Soil type | Depth m | Pa kN/m$^2$ | Sa kN/m | Y m | Ma kN・m/m |
|---|---|---|---|---|---|---|
| 1 | sandy soil | 0.00～ 1.00 | 3.33 9.67 | 1.67 4.83 | -0.67 -0.33 | -1.11 -1.61 |
| 2 | sandy soil | 1.00～ 2.65 | 9.67 20.12 | 7.98 16.60 | 0.55 1.10 | 4.39 18.26 |
| 3 | sandy soil | 2.65～ 8.00 | 20.12 37.95 | 53.81 101.52 | 3.43 5.22 | 184.75 529.58 |
| 4 | sandy soil | 8.00～ 15.37 | 46.21 76.12 | 170.27 280.50 | 9.46 11.91 | 1610.21 3341.66 |
| 5 | cohesive soil | 15.37～ 21.37 | 93.78 120.78 | 281.33 362.33 | 16.37 18.37 | 4605.29 6655.91 |
| 6 | sandy soil | 21.37～ 22.14 | 57.46 59.28 | 22.02 22.72 | 20.63 20.88 | 454.26 474.48 |
| Total moment by active earth pressure | | | | | $\Sigma$Ma= | 17876.07 |

$P_a$ : Active earth pressure
$S_a$ : Horizontal force by active earth pressure（$S_a = P_a \times$ Thickness of layer／2）
Y : Arm length from tie-rod
$M_a$ : Moment by active earth pressure（$M_a = S_a \times Y$）

### 3-1-2 Moment by residual water pressure

| No | Soil type | Depth m | Pw kN/m² | Sw kN/m | Y m | Mw kN･m/m |
|---|---|---|---|---|---|---|
| 3 | sandy soil | 2.65～ 8.00 | 0.00 53.50 | 0.00 143.11 | 3.43 5.22 | 0.00 746.57 |
| 4 | sandy soil | 8.00～ 15.37 | 53.50 53.50 | 197.15 197.15 | 9.46 11.91 | 1864.36 2348.68 |
| 5 | cohesive soil | 15.37～ 21.37 | 53.50 53.50 | 160.50 160.50 | 16.37 18.37 | 2627.39 2948.39 |
| 6 | sandy soil | 21.37～ 22.14 | 53.50 53.50 | 20.51 20.51 | 20.63 20.88 | 422.95 428.19 |
| Total moment by residual water pressure | | | | | ΣMw= | 11386.52 |

$P_w$ : Residual water pressure
$S_w$ : Horizontal force by residual water pressure（$S_w = P_w \times$ Thickness of layer／2）
Y : Arm length from tie-rod
$M_w$ : Moment by residual water pressure$M_w = S_w \times Y$）

### 3-1-3 Moment by passive earth pressure

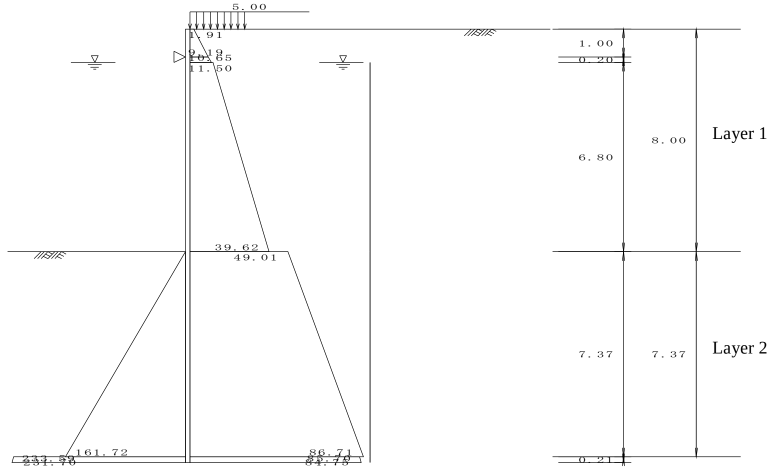
| No | Soil type | Depth m | Pp kN/m² | Sp kN/m | Y m | Mp kN･m/m |
|---|---|---|---|---|---|---|
| 4 | sandy soil | 8.00～ 15.37 | 0.00 181.59 | 0.00 669.16 | 9.46 11.91 | 0.00 7971.93 |
| 5 | cohesive soil | 15.37～ 21.37 | 231.70 285.70 | 695.10 857.10 | 16.37 18.37 | 11378.79 15744.93 |
| 6 | sandy soil | 21.37～ 22.14 | 536.82 569.04 | 205.76 218.11 | 20.63 20.88 | 4243.87 4554.37 |
| Total moment by passive earth pressure | | | | | ΣMp= | 43893.89 |
| Sum of passive earth pressure | | | | | ΣSp= | 2645.23 |

$P_p$ : Passive earth pressure
$S_p$ : Horizontal force by passive earth pressure（$S_p = P_p \times$ Thickness of layer／2）
Y : Arm length from tie-rod
$M_p$ : Moment by passive earth pressure（$M_p = S_p \times Y$）

### 3-1-4 Penetration depth

$F_s \times (\Sigma M_a + \Sigma M_w) - \Sigma M_p = 1.50 \times (17876.07 + 11386.52) - 43893.89 = 0$
Penetration depth D = 14.14 m

### 3-2 Seismic condition




Moment around the tie-rod point is calculated supposing that penetration depth is 7.58 m

### 3-2-1 Moment by active earth pressure

| No | Soil type | Depth m | Pa kN/m² | Sa kN/m | Y m | Ma kN･m/m |
|---|---|---|---|---|---|---|
| 1 | sandy soil | 0.00～ 1.00 | 1.91 9.19 | 0.96 4.60 | -0.67 -0.33 | -0.64 -1.53 |
| 2 | sandy soil | 1.00～ 1.20 | 9.19 10.65 | 0.92 1.06 | 0.07 0.13 | 0.06 0.14 |
| 3 | sandy soil | 1.20～ 8.00 | 11.50 39.62 | 39.09 134.71 | 2.47 4.73 | 96.43 637.63 |
| 4 | sandy soil | 8.00～ 15.37 | 49.01 86.71 | 180.59 319.53 | 9.46 11.91 | 1707.83 3806.65 |
| 5 | cohesive soil | 15.37～ 15.58 | 84.75 85.70 | 8.90 9.00 | 14.44 14.51 | 128.50 130.56 |
| Total moment by active earth pressure | | | | | ΣMa= | 6505.63 |

$P_a$ : Active earth pressure
$S_a$ : Horizontal force by active soil pressure（$S_a = P_a \times$ Thickness of layer／2）
Y : Arm length from tie-rod
$M_a$ : Moment by active earth pressure（$M_a = S_a \times Y$）

### 3-2-2 Moment by residual water pressure

| No | Soil type | Depth m | Pw kN/m² | Sw kN/m | Y m | Mw kN･m/m |
|---|---|---|---|---|---|---|
| 3 | sandy soil | 1.20～ 8.00 | 0.00 0.00 | 0.00 0.00 | 2.47 4.73 | 0.00 0.00 |

| No | Soil type | Depth m | Pw $kN/m^2$ | Sw kN/m | Y m | Mw kN・m/m |
|---|---|---|---|---|---|---|
| 4 | sandy soil | 8.00～ 15.37 | 0.00 0.00 | 0.00 0.00 | 9.46 11.91 | 0.00 0.00 |
| 5 | cohesive soil | 15.37～ 15.58 | 0.00 0.00 | 0.00 0.00 | 14.44 14.51 | 0.00 0.00 |
| Total moment by residual water pressure | | | | ΣMw＝ | | 0.00 |

$P_w$ : Residual water pressure
$S_w$ : Horizontal force by residual water pressure（$S_w＝P_w$×Thickness of layer／２）
Y : Arm length from tie-rod
$M_w$ : Moment by residual water pressure（$M_w＝S_w×Y$）

### 3-2-3 Moment by passive earth pressure

| No | Soil type | Depth m | Pp $kN/m^2$ | Sp kN/m | Y m | Mp kN・m/m |
|---|---|---|---|---|---|---|
| 4 | sandy soil | 8.00～ 15.37 | 0.00 161.72 | 0.00 595.93 | 9.46 11.91 | 0.00 7099.55 |
| 5 | cohesive soil | 15.37～ 15.58 | 231.70 233.59 | 24.33 24.53 | 14.44 14.51 | 351.31 355.90 |
| Total moment by passive earth pressure | | | | ΣMp＝ | | 7806.76 |
| Sum of passive earth pressure | | | | ΣSp＝ | | 644.79 |

$P_p$ : Passive earth pressure
$S_p$ : Horizontal force by passive earth pressure（$S_p＝P_p$×Thickness of layer／２）
Y : Arm length from tie-rod
$M_p$ : Moment by passive earth pressure（$M_p＝S_p×Y$）

### 3-2-4 Penetration depth

$F_s×(ΣM_a＋ΣM_w)－ΣM_p＝1.20×(6505.63＋0.00)－7806.76＝0$
Penetration depth D ＝ 7.58 m

## 3-3 Total sheet pile length

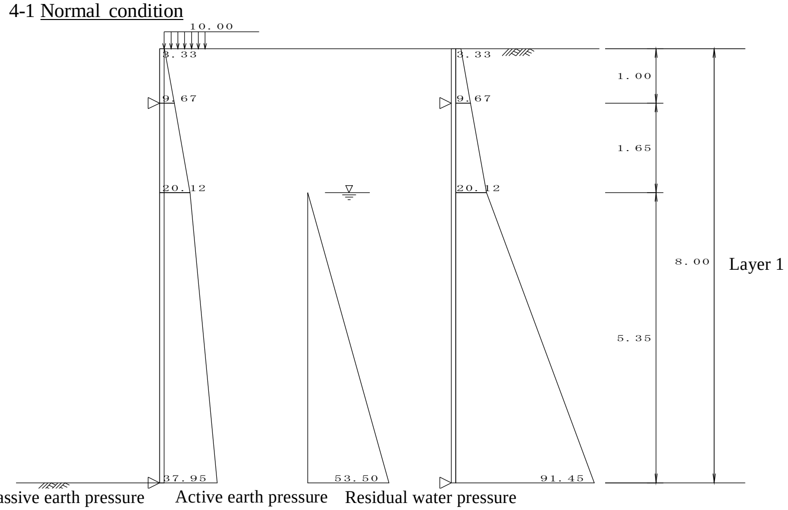
Penetration depth（D）is
Normal condition ; D ＝ 14.14 m
Seismic condition ; D ＝ 7.58 m
L ＝ (8.00－0.70)＋14.14 ＝ 21.44 m
Total sheet pile length= 21.50 m



## 4 Calculation of section force

Section force is calculated as the simple beam between tie-rod and design base level.

### 4-1 Normal condition




Passive earth pressure　Active earth pressure　Residual water pressure

#### 4-1-1 Lateral pressure

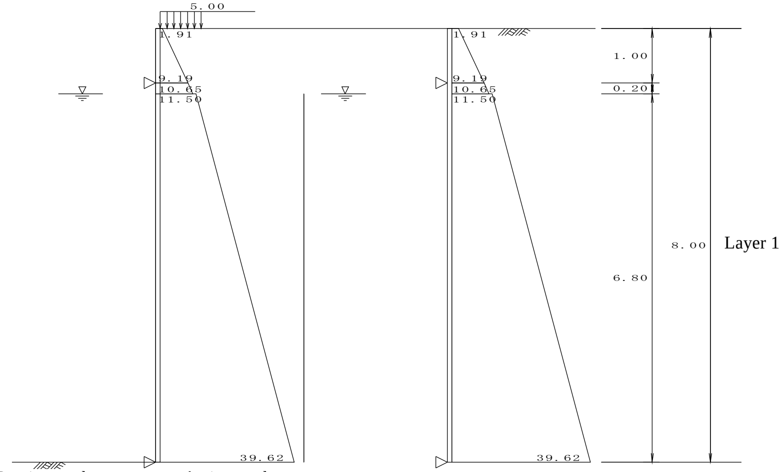
| No | Soil type | Depth m | Pa $kN/m^2$ | Pw $kN/m^2$ | Pp $kN/m^2$ | Ps $kN/m^2$ | S kN/m | Y m | M kN・m/m |
|---|---|---|---|---|---|---|---|---|---|
| 1 | sandy soil | 0.00～ 1.00 | 3.33 9.67 | ——— ——— | ——— ——— | 3.33 9.67 | 1.67 4.83 | -0.67 -0.33 | -1.11 -1.61 |
| 2 | sandy soil | 1.00～ 2.65 | 9.67 20.12 | ——— ——— | ——— ——— | 9.67 20.12 | 7.98 16.60 | 0.55 1.10 | 4.39 18.26 |
| 3 | sandy soil | 2.65～ 8.00 | 20.12 37.95 | 0.00 53.50 | ——— ——— | 20.12 91.45 | 53.81 244.63 | 3.43 5.22 | 184.75 1276.15 |
| | | | | | | ΣS＝ | 329.51 | ΣM＝ | 1480.82 |

$P_a$ : Active earth pressure
$P_w$ : Residual water pressure
$P_p$ : Passive earth pressure
$P_s$ : Lateral pressure intensity（$P_s＝P_a＋P_w－P_p≧0$）
S : Lateral pressure（$S＝P_s$×Thickness of layer／２）
Y : Depth from tie-rod
M : Moment by lateral pressure（M＝S×Y）

#### 4-1-2 Support reaction and maximum bending moment

$R_b$＝211.55 kN, $R_a$=117.97kN
$M_{max}$=283.19 kNm/m

### 4-2 Seismic condition




Passive earth pressure　Active earth pressure　Residual water pressure

#### 4-2-1 Lateral pressure

| No | Soil type | Depth m | Pa $kN/m^2$ | Pw $kN/m^2$ | Pp $kN/m^2$ | Ps $kN/m^2$ | S kN/m | Y m | M kN・m/m |
|---|---|---|---|---|---|---|---|---|---|
| 1 | sandy soil | 0.00～ 1.00 | 1.91<br>9.19 | ―――<br>――― | ―――<br>――― | 1.91<br>9.19 | 0.96<br>4.60 | -0.67<br>-0.33 | -0.64<br>-1.53 |
| 2 | sandy soil | 1.00～ 1.20 | 9.19<br>10.65 | ―――<br>――― | ―――<br>――― | 9.19<br>10.65 | 0.92<br>1.06 | 0.07<br>0.13 | 0.06<br>0.14 |
| 3 | sandy soil | 1.20～ 8.00 | 11.50<br>39.62 | 0.00<br>0.00 | ―――<br>――― | 11.50<br>39.62 | 39.09<br>134.71 | 2.47<br>4.73 | 96.43<br>637.63 |
| | | | | | | ΣS＝ | 181.34 | ΣM＝ | 732.09 |

$P_a$ : Active earth pressure
$P_w$ : Residual water pressure intensity
$P_p$ : Passive earth pressure
$P_s$ : Lateral pressure intensity（$P_s = P_a + P_w - P_p \geqq 0$）
S : Lateral pressure（S＝$P_s$×Thickness of layer／２）
Y : Depth from tie-rod
M : Moment by lateral pressure（M＝S×Y）

#### 4-2-2 Support reaction and maximum bending moment

$R_b$＝104.58 kN, $R_a$=76.76kN
$M_{max}$=154.42 kNm/m

## 5 Calculation of stress intensity

| | |
|---|---|
| Material | PU18 |
| Corrosion allowance | $t_1$=0.00 mm (front side)　$t_2$=0.00 mm (back side) |
| Corrosion ratio | $\eta$ =1.00 |
| Joint efficiency | $\mu$ =1.00 |
| Modulus of section | $Z_0$= 1800 $cm^3$（before reduction） |
| | Z = 1800 $cm^3$（after reduction considering corrosion and joint） |

### 5-1 Normal condition

$$\sigma = \frac{M_{max}}{Z} = \frac{283.19\times10^6}{1800\times10^3} = 157\ N/mm^2 \leqq \sigma_a = 165\ N/mm^2$$

### 5-2 Seismic condition

$$\sigma = \frac{M_{max}}{Z} = \frac{154.42\times10^6}{1800\times10^3} = 86\ N/mm^2 \leqq \sigma_a = 190\ N/mm^2$$

## 6 Calculation of Tie-rod

| | |
|---|---|
| Horizontal intervals of tie-rod | $\ell$ = 2.40 m |
| Horizontal angle of tie-rod | $\theta$ = 0.00000 degree |
| Corrosion allowance | $t_c$= 0.00 mm |

### 6-1 Tie-rod tensile force and necessary diameter

#### 6-1-1 Normal condition

Tie-rod tensile force（$T_p$）

$$T_p = R_a \cdot l \cdot \sec\theta$$
$$= 117.97\times2.40\times\sec0.00000^\circ = 283.12\ kN$$

Necessary diameter（$d_r$）

$$d_r = \sqrt{\frac{4T_p}{\sigma_a\pi}} + 2t_c = \sqrt{\frac{4\cdot283.12\times10^3}{176\cdot\pi}} + 2\cdot0.00 = 45.3\ mm$$

#### 6-1-2 Seismic condition

Tie-rod tensile force（$T_p$）

$$T_p = R_a \cdot l \cdot \sec\theta$$
$$= 76.76\times2.40\times\sec0.00000^\circ = 184.21\ kN$$

Necessary diameter（$d_r$）は、

$$d_r = \sqrt{\frac{4T_p}{\sigma_a\pi}} + 2t_c = \sqrt{\frac{4\cdot184.21\times10^3}{264\cdot\pi}} + 2\cdot0.00 = 29.8\ mm$$

### 6-2 Calculation of stress intensity

Tie-rod diameter $\phi$46mm
Tie-rod section area after corrosion（$A_t$）

$$A_t = \frac{\pi}{4}(46-2\cdot0.00)^2 = 1662\ mm^2$$

#### 6-2-1 Normal condition

$$\sigma = \frac{T_p}{A_t} = \frac{283.12\times10^3}{1662} = 170\ N/mm^2 \leqq \sigma_a = 176\ N/mm^2$$

#### 6-2-2 Seismic condition

$$\sigma = \frac{T_p}{A_t} = \frac{184.21\times10^3}{1662} = 111\ N/mm^2 \leqq \sigma_a = 264\ N/mm^2$$

## 7 Wale

Horizontal intervals of tie-rod $\ell =$ 2.40 m

### 7-1 Maximum bending moment

#### 7-1-1 Normal condition

$$M_{max} = \frac{1}{10} R_a \cdot l^2 = \frac{1}{10} 117.97 \times 2.40^2 = 67.95 \text{ kN·m}$$

#### 7-1-2 Seismic condition

$$M_{max} = \frac{1}{10} R_a \cdot l^2 = \frac{1}{10} 76.76 \times 2.40^2 = 44.21 \text{ kN·m}$$

### 7-2 Calculation of stress intensity

Material 2 x UPN-220x80
Corrosion allowance $t_c$=0.00mm
Corrosion ratio $\eta$ =1.00
Modulus of section $Z_0$= 490 cm$^3$ (before reduction)
$Z$ = 490 cm$^3$ (after reduction by corrosion allowance and joint)

#### 7-2-1 Normal condition

$$\sigma = \frac{M_{max}}{Z} = \frac{67.95 \times 10^6}{490.0 \times 10^3} = 139 \text{ N/mm}^2 \leqq \sigma_a = 140 \text{ N/mm}^2$$

#### 7-2-2 Seismic condition

$$\sigma = \frac{M_{max}}{Z} = \frac{44.21 \times 10^6}{490.0 \times 10^3} = 90 \text{ N/mm}^2 \leqq \sigma_a = 161 \text{ N/mm}^2$$

## 8 Horizontal coefficient of subgrade reaction

### 8-1 Horizontal coefficient of subgrade reaction

Horizontal coefficient of subgrade reaction

$$K_h = 6910 \times N'^{0.406}$$

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

Material PU12
Unit length B =1.00 m
Corrosion allowance $t_1$=0.00 mm (front side) $t_2$=0.00 mm (back side)
Corrosion ratio $\eta$ =1.00
Joint efficiency $\mu$ =1.00
Young's modulus E = 200000 N/mm$^2$
Second section moment $I_0$= 21600 cm$^4$ (before reduction)
I = 21600 cm$^4$ (after reduction by corrosion allowance and joint)
$EI = 200000 \times 10^3 \times 21600 \times 10^{-8} = 4.320 \times 10^4$

| No | Depth m | N-Value | No | Depth m | N-Value |
|---|---|---|---|---|---|
| 1 | 1.00 | 8 | 11 | 10.87 | 10 |
| 2 | 2.00 | 8 | 12 | 11.87 | 8 |
| 3 | 3.00 | 8 | 13 | 12.87 | 8 |
| 4 | 4.00 | 8 | 14 | 13.87 | 9 |
| 5 | 5.00 | 8 | | | |
| 6 | 6.00 | 8 | | | |
| 7 | 7.00 | 8 | | | |
| 8 | 8.00 | 8 | | | |
| 9 | 8.87 | 11 | | | |
| 10 | 9.87 | 5 | | | |

### 8-2 Normal condition

Assuming that $K_h$ = 16074 kN/m$^3$

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}} = 0.552 \text{ m}^{-1}$$

$$L = \frac{1}{\beta} = 1.81 \text{ m}$$

| No | Depth Z m | Thickness of layer h m | N-value top | N-value under | Area A |
|---|---|---|---|---|---|
| 1 | 1.00 | 1.00 | 8.0 | 8.0 | 8.00 |
| 2 | 2.00 | 0.81 | 8.0 | 8.0 | 6.49 |
| L = Σ h = 1.81 | | | ΣA = | | 14.49 |

Average N- value =8.00

$$K_h = 6910 \times N'^{0.406} = 6910 \times 8.00^{0.406} = 16074 \text{ kN/m}^3$$

Horizontal coefficient of subgrade reaction $K_h$ (Normal condition) = 16074 kN/m$^3$

## 8-3 Seismic condition

Assuming that $K_h$ = 16074 kN/m$^3$

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}} = 0.552\ \mathrm{m}^{-1}$$

$$L = \frac{1}{\beta} = 1.81\ \mathrm{m}$$

| No | Depth Z m | Thickness of layer h m | N-value top | N-value under | Area A |
|---|---|---|---|---|---|
| 1 | 1.00 | 1.00 | 8.0 | 8.0 | 8.00 |
| 2 | 2.00 | 0.81 | 8.0 | 8.0 | 6.29 |
| | $L = \Sigma h =$ 1.81 | | | $\Sigma A =$ | 14.49 |

Average N- value =8.00

$$K_h = 6910 \times N'^{0.406} = 6910 \times 8.00^{0.406} = 16074\ \mathrm{kN/m^3}$$

Horizontal coefficient of subgrade reaction $K_h$ (Seismic condition) = 16074 kN/m$^3$

## 9 Bracing sheet pile

Bracing sheet pile is calculated by Chang's formula

Type PU12
Unit length $B$ =1.0000 m
Corrosion allowance $t_1$=0.00 mm (front side) $t_2$=0.00 mm (back side)
Corrosion ratio (against $I_0$) $\eta$ =1.00
Corrosion ratio (against $Z_0$) $\eta$ =1.00
Joint efficiency (against $I_0$) $\mu$ =1.00
Joint efficiency(against $Z_0$) $\mu$ =1.00
Young's modulus $E$ = 200000 N/mm$^2$
Second moment of area $I_0$= 21600 cm$^4$ (before reduction)
$I$ = 21600 cm$^4$ (after reduction by corrosion allowance and joint)
Modulus of section $Z_0$= 1200 cm$^3$ (before reduction)
$Z$ = 1200 cm$^3$ (after reduction by corrosion allowance and joint)
$EI = 200000 \times 10^3 \times 21600 \times 10^{-8} = 4.320 \times 10^4$

### 9-1 Design horizontal force

Normal condition $T = R_a =$ 117.97 kN/m
Seismic condition $T = R_a =$ 76.75 kN/m

### 9-2 Maximum bending moment and displacement ( Chang's method)

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

$$M_{max} = -0.3224\ \frac{T}{\beta}$$

$$\delta = \frac{T}{2EI\beta^3}$$

#### 9-2-1 Normal condition

$$\beta = \sqrt[4]{\frac{16074 \times 1.0000}{4 \times 2.00 \times 10^8 \times 21600 \times 10^{-8}}} = 0.552\ \mathrm{m}^{-1}$$

$$M_{max} = -0.3224 \times \frac{117.97}{0.552} = -68.87\ \mathrm{kN \cdot m}$$

$$\delta = \frac{117.97}{2 \times 2.00 \times 10^8 \times 21600 \times 10^{-8} \times 0.552^3}$$

$$= 0.0081\ \mathrm{m} = 8.1\ \mathrm{mm} \leqq \delta_a = 50.0\ \mathrm{mm}$$

#### 9-2-2 Seismic condition

$$\beta = \sqrt[4]{\frac{16074 \times 1.0000}{4 \times 2.00 \times 10^8 \times 21600 \times 10^{-8}}} = 0.552\ \mathrm{m}^{-1}$$

$$M_{max} = -0.3224 \times \frac{76.75}{0.552} = -44.81\ \mathrm{kN \cdot m}$$

$$\delta = \frac{76.75}{2 \times 2.00 \times 10^8 \times 21600 \times 10^{-8} \times 0.552^3}$$

$$= 0.0053\ \mathrm{m} = 5.3\ \mathrm{mm} \leqq \delta_a = 75.0\ \mathrm{mm}$$

9-3 Calculation of stress intensity

9-3-1 Normal condition

$$\sigma = \frac{M_{max}}{Z} = \frac{68.87 \times 10^6}{1200 \times 10^3} = 57\ N/mm^2 \leqq \sigma_a = 165\ N/mm^2$$

9-3-2 Seismic condition

$$\sigma = \frac{M_{max}}{Z} = \frac{44.81 \times 10^6}{1200 \times 10^3} = 37\ N/mm^2 \leqq \sigma_a = 190\ N/mm^2$$

9-4 Penetration depth (Chang' formula)

9-4-1 Normal condition

$$\beta = \sqrt[4]{\frac{16074 \times 1.0000}{4 \times 2.00 \times 10^8 \times 21600 \times 10^{-8}}} = 0.552\ m^{-1}$$

$D = 3 / \beta = 3 / 0.552 = 5.43\ m$ (Penetration depth)

$l_{m1} = \pi / \beta = \pi / 0.552 = 5.69\ m$ (Depth where bending moment=0 )

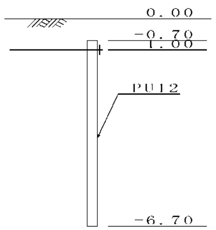
9-4-2 Seismic condition

$$\beta = \sqrt[4]{\frac{16074 \times 1.0000}{4 \times 2.00 \times 10^8 \times 21600 \times 10^{-8}}} = 0.$$

$D = 3 / \beta = 3 / 0.552 = 5.43\ m$ (Penetration depth)

$l_{x1} = \pi / \beta = \pi / 0.552 = 5.69\ m$ (Depth where bending moment=0 )

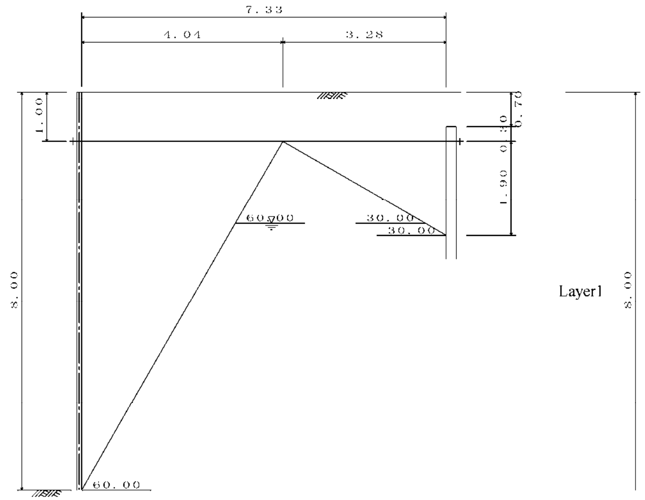
$L = H_u + D = 0.30 + 5.43 = 5.73\ m$
Total length of Bracing works= 6.00 m






10 Installation location of bracing works

10-1 Normal condition




10-1-1 Active collapse angle

| No | Depth m | Soil type | $\phi$ degree | $\delta$ degree | C $kN/m^2$ | $\Sigma\gamma h$ $kN/m^2$ | $\gamma w h w$ $kN/m^2$ | k (k') | $\theta$ degree | $\zeta$ degree |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 8.00～ 2.65 | sandy soil | 30.0 | 0.00 | ――― | ――― | ――― | ――― | ――― | 60.00 |
| 2 | 2.65～ 1.00 | sandy soil | 30.0 | 0.00 | ――― | ――― | ――― | ――― | ――― | 60.00 |

・sandy soil :Active collapse angle

$$\zeta = 90 - \tan^{-1} \frac{-\sin(\phi+\delta) + \sqrt{\dfrac{\cos(\delta+\theta)\cdot\sin(\phi+\delta)}{\sin(\phi-\theta)}}}{\cos(\phi+\delta)}$$

・cohesive soil :Active collapse angle

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma\gamma h + 2Q}{2C} \cdot \tan\theta}$$

where,

- $\zeta$ : Active collapse angle (degree) ( $\zeta \geqq 10.00°$ )
- $\phi$ : Internal friction angle (degree)
- $\delta$ : Angle of wall friction (degree)
- $\theta$ : Compound angle during earthquake (degree)
  $\theta = \tan^{-1} k$ 、or $\theta = \tan^{-1} k'$
- $\gamma$ : Unit weight of soil ($kN/m^3$)
- h : Thickness of layer (m)
- Q : Overburden load ($kN/m^2$)
- C : Adhesion of soil ($kN/m^2$)

### 10-1-2 Passive collapse angle

| No | Depth m | Soil type | $\phi$ degree | $\delta$ degree | C $kN/m^2$ | $\Sigma\gamma h$ $kN/m^2$ | $\gamma w h w$ $kN/m^2$ | k (k') | $\theta$ degree | $\zeta$ degree |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 2.90～ 2.65 | sandy soil | 30.0 | 0.00 | ——— | ——— | ——— | ——— | ——— | 30.00 |
| 2 | 2.65～ 1.00 | sandy soil | 30.0 | 0.00 | ——— | ——— | ——— | ——— | ——— | 30.00 |

・sandy soil: Passive collapse angle

$$\zeta = 90 - \tan^{-1} \frac{\sin(\phi-\delta)+\sqrt{\dfrac{\cos(\delta-\theta)\cdot\sin(\phi-\delta)}{\sin(\phi-\theta)}}}{\cos(\phi-\delta)}$$

・cohesive soil: Passive collapse angle
Passive collapse angle is 45degree.

where

- $\zeta$ : Passive collapse angle (degree) (sandy soil $\zeta \geqq 10.00°$ )
- $\phi$ : Internal friction angle (degree)
- $\delta$ : Angle of wall friction (degree)
- $\theta$ : Compound angle during earthquake (degree)
  $\theta = \tan^{-1} k$ 、or、 $\theta = \tan^{-1} k'$
- $\gamma$ : Unit weight of soil ($kN/m^3$)
- h : Thickness of layer (m)
- Q : Overburden load ($kN/m^2$)
- C : Adhesion of soil ($kN/m^2$)

### 10-1-3 Review of Installation position

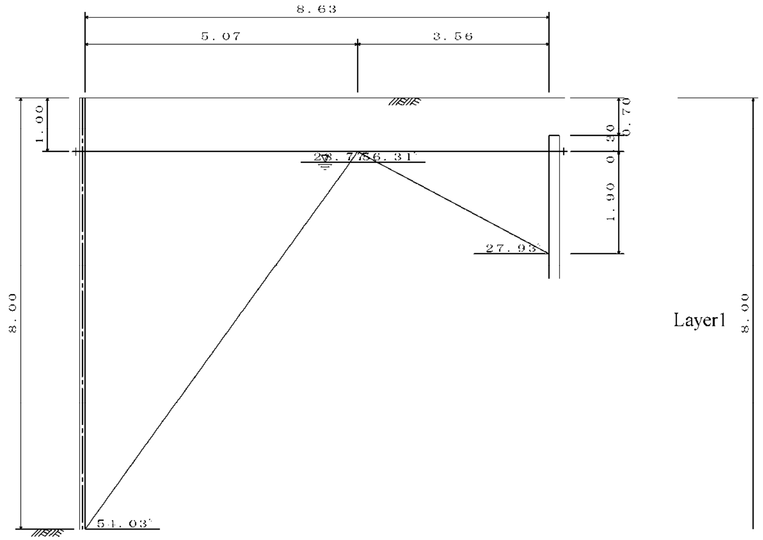
| No | Depth | height | Thickness of layer | Passive collapse line | | | Passive collapse line | | |
|---|---|---|---|---|---|---|---|---|---|
| | (m) | Y (m) | Z (m) | $\zeta$a (degree) | d X (m) | Σ X (m) | $\zeta$p (degree) | d X (m) | Σ X (m) |
| 1 | 1.00～ 2.65 | 7.00 | 1.65 | 60.00 | 0.95 | 4.04 | 30.00 | 2.86 | 4.04 |
| 2 | 2.65～ 2.90 | 5.35 | 0.25 | 60.00 | 0.14 | 3.09 | 30.00 | 0.43 | 6.90 |
| 3 | 2.90～ 8.00 | 5.10 | 5.10 | 60.00 | 2.95 | 2.95 | ——— | ——— | 7.33 |
| 4 | 8.00 | 0.00 | ——— | ——— | ——— | 0.00 | ——— | ——— | ——— |

d X : Width of collapse line（d X＝Z・cot $\zeta$）
Σ X : Distance from front sheet pile

Installation location　　d＝6.33 m or more is necessary（Normal condition）



## 10-2 Seismic condition




### 10-2-1 Active collapse angle

| No | Depth m | Soil type | $\phi$ degree | $\delta$ degree | C $kN/m^2$ | $\Sigma\gamma h$ $kN/m^2$ | $\gamma w h w$ $kN/m^2$ | k (k') | $\theta$ degree | $\zeta$ degree |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 8.00～ 1.20 | sandy soil | 30.0 | 0.00 | ——— | ——— | 68.00 | 0.124 | 7.07 | 54.03 |
| 2 | 1.20～ 1.00 | sandy soil | 30.0 | 0.00 | ——— | ——— | ——— | 0.080 | 4.57 | 56.31 |

・sandy soil: Active collapse angle

$$\zeta = 90 - \tan^{-1} \frac{-\sin(\phi+\delta)+\sqrt{\dfrac{\cos(\delta+\theta)\cdot\sin(\phi+\delta)}{\sin(\phi-\theta)}}}{\cos(\phi+\delta)}$$

・cohesive soil :Active collapse angle

$$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}}\cdot\tan\theta$$

where

- $\zeta$ : Active collapse angle (degree) ( $\zeta \geqq 10.00°$ )
- $\phi$ : Internal friction angle (degree)
- $\delta$ : Angle of wall friction (degree)
- $\theta$ : Compound angle during earthquake (degree)
  $\theta = \tan^{-1} k$ or $\theta = \tan^{-1} k'$
- $\gamma$ : Unit weight of soil ($kN/m^3$)
- h : Thickness of layer (m)
- Q : Overburden load ($kN/m^2$)
- C : Adhesion of soil ($kN/m^2$)

10-2-2 Passive collapse angle

| No | Depth m | Soil type | $\phi$ degree | $\delta$ degree | C $kN/m^2$ | $\Sigma\gamma h$ $kN/m^2$ | $\gamma w h w$ $kN/m^2$ | k (k') | $\theta$ degree | $\zeta$ degree |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 2.90～ 1.20 | sandy soil | 30.0 | 0.00 | ——— | ——— | 16.96 | 0.128 | 7.29 | 27.93 |
| 2 | 1.20～ 1.00 | sandy soil | 30.0 | 0.00 | ——— | ——— | ——— | 0.080 | 4.57 | 28.77 |

・sandy soil :Passive collapse angle

$$\zeta = 90 - \tan^{-1} \frac{\sin(\phi-\delta)+\sqrt{\dfrac{\cos(\delta-\theta)\cdot\sin(\phi-\delta)}{\sin(\phi-\theta)}}}{\cos(\phi-\delta)}$$

・cohesive soil :Passive collapse angle
Passive collapse angle is 45degree.

where
- $\zeta$ : Passive collapse angle (degree) (sandy soil $\zeta \geqq 10.00°$ )
- $\phi$ : Internal friction angle (degree)
- $\delta$ : Angle of wall friction (degree)
- $\theta$ : Compound angle during earthquake (degree)
  $\theta = \tan^{-1} k$ 、or $\theta = \tan^{-1} k'$
- $\gamma$ : Unit weight of soil ($kN/m^3$)
- h : Thickness of layer (m)
- Q : Overburden load ($kN/m^2$)
- C : Adhesion of soil ($kN/m^2$)

10-2-3 Review of Installation position

| No | Depth (m) | Height Y (m) | Thickness of layer Z (m) | Passive collapse line $\zeta a$ (degree) | Passive collapse line dX (m) | Passive collapse line $\Sigma X$ (m) | Passive collapse line $\zeta p$ (degree) | Passive collapse line dX (m) | Passive collapse line $\Sigma X$ (m) |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 1.00～ 1.20 | 7.00 | 0.20 | 56.31 | 0.13 | 5.07 | 28.77 | 0.36 | 5.07 |
| 2 | 1.20～ 2.90 | 6.80 | 1.37 | 54.03 | 1.23 | 4.94 | 27.93 | 3.20 | 5.43 |
| 3 | 2.90～ 8.00 | 5.10 | 5.43 | 54.03 | 3.70 | 3.70 | ——— | ——— | 8.63 |
| 4 | 8.00 | 0.00 | ——— | ——— | ——— | 0.00 | ——— | ——— | ——— |

dX : Width of collapse line（$dX = Z \cdot \cot\zeta$）
$\Sigma X$ : Distance from front sheet pile

Installation location d =8.63 m or more is necessary（Seismic condition）

10-3 Installation location

Installation position of Bracing works（d）is,
Normal condition d = 7.33 m
Seismic condition d = 8.63 m

---> d =8.63 m

11 Calculation results

Front sheet pile PU18

| | | | Normal condition | Seismic condition |
|---|---|---|---|---|
| Second moment of area | I ($cm^4$) | 38650 | | |
| Modulus of section | Z ($cm^3$) | 1800 | | |
| Max. bending moment | Mmax(kN・m/m) | | 283.19 | 154.42 |
| Stress intensity | $\sigma$ ($N/mm^2$) | | 157 ( 165) | 86 ( 190) |
| Penetration depth | D (m) | | 14.14 | 7.58 |
| Total length | L (m) | 21.50 | | |

tie-rod $\phi$ 46mm

| | | | Normal condition | Seismic condition |
|---|---|---|---|---|
| Tensile force | Tp (kN) | | 283.12 | 184.21 |
| Stress intensity | $\sigma$ ($N/mm^2$) | | 170 ( 176) | 111 ( 264) |

Wale 2 x UPN-220x80

| | | | Normal condition | Seismic condition |
|---|---|---|---|---|
| Modulus of section | Z ($cm^3$) | 490 | | |
| Max. bending moment | Mmax(kN・m) | | 67.95 | 44.21 |
| Stress intensity | $\sigma$ ($N/mm^2$) | | 139 ( 140) | 90 ( 210) |

Bracing sheet pile PU12

| | | | Normal condition | Seismic condition |
|---|---|---|---|---|
| Second moment of area | I ($cm^4$) | 21600 | | |
| Modulus of section | Z ($cm^3$) | 1200 | | |
| Max. bending moment | Mmax(kN・m) | | 68.87 | 44.81 |
| stress intensity | $\sigma$ ($N/mm^2$) | | 57 ( 165) | 37 ( 190) |
| Horizontal displacement | $\delta$ (mm) | | 8.11 ( 50.00) | 5.27 ( 75.00) |
| penetration depth | D (m) | | 5.43 | 5.43 |
| Total length | L (m) | 6.00 | | |

Bracing works Installation position

| | | | Normal condition | Seismic condition |
|---|---|---|---|---|
| Bracing works Installation position | d (m) | | 7.33 | 8.63 |

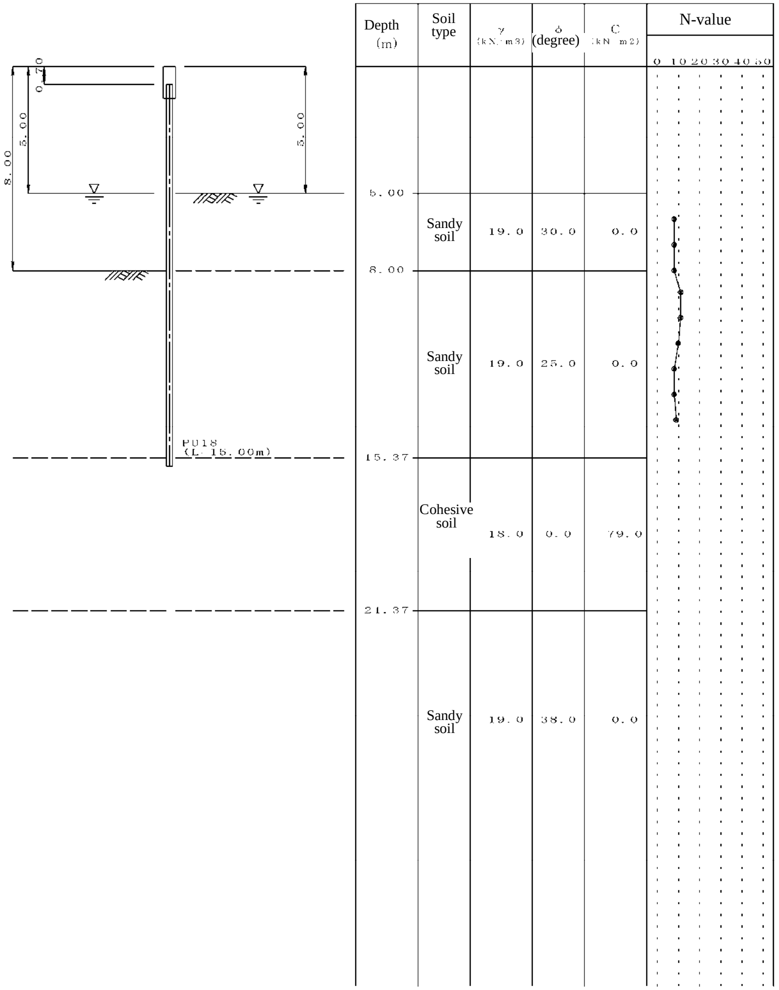
Structural calculation of steel sheet pile works (Type A)

**－ During construction －**

<Cantilevered sheet pile wall>

1Design conditions

1-1 Structural figure



1-2 Structural dimensions

Design base level H = 8.00 m
Overhang length Ho = 5.00m
Top of sheet pile $H_{lt}$ = 0.70 m
Inside water level $L_{wa}$ = 5.00 m(Normal condition)
Outside water level $L_{wp}$ = 5.00 m(Normal condition)

1-3 Calculation method

Calculation method Chang's formula

Penetration depth $L = \frac{\pi}{\beta}$

1-4 Design constants

Unit weight of water $\gamma_w$= 10.0 kN/m$^3$

Type of water pressure; Trapezoid shape

1-5 Coefficient of horizontal subgrade reaction

Formula

$$K_h = 6910 \times N'^{0.406}$$

1-6 Overburden load

Active pressure side
$Q_a$ = 10.0 kN/m$^2$ (Normal condition

Passive pressure side
$Q_p$ = 0.0 kN/m$^2$ (Normal condition)

1-7 Soil constants

Soil constants

| No | Depth m | Layer thickness m |
|---|---|---|
| 1 | 8.00 | 3.00 |
| 2 | 15.37 | 7.37 |
| 3 | 21.37 | 6.00 |
| 4 | 30.00 | 8.63 |

| No | Depth m | Soil type | N-value | γ kN/m$^3$ | γ' kN/m$^3$ | φ Deg, | C kN/m$^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 8.00 | S | 8.0 | 19.00 | 10.00 | 30.0 | 0.0 | 0.0 |
| 2 | 15.37 | S | 9.0 | 19.00 | 10.00 | 25.0 | 0.0 | 0.0 |
| 3 | 21.37 | C | 13.0 | 18.00 | 9.00 | 0.0 | 79.0 | 0.0 |
| 4 | 30.00 | S | 44.0 | 19.00 | 10.00 | 38.0 | 0.0 | 0.0 |

Depth : Distance from the top of sheet pile C : Adhesive force
type : Sandy soil(S)、Cohesive soil(C)、Medium soil(M) a : Angle of adhesive force

N value : average value
γ : unit weight of humid soil
γ' : unit weight of soil under the water
φ : internal friction angle

1-8 Steel sheet pile

Material PU18
Young's modulus E = 200000 N/mm$^2$
Second moment of area $I_0$= 38650 cm$^4$ (before corrosion)
Section modulus $Z_0$= 1800 cm$^3$ (before corrosion)

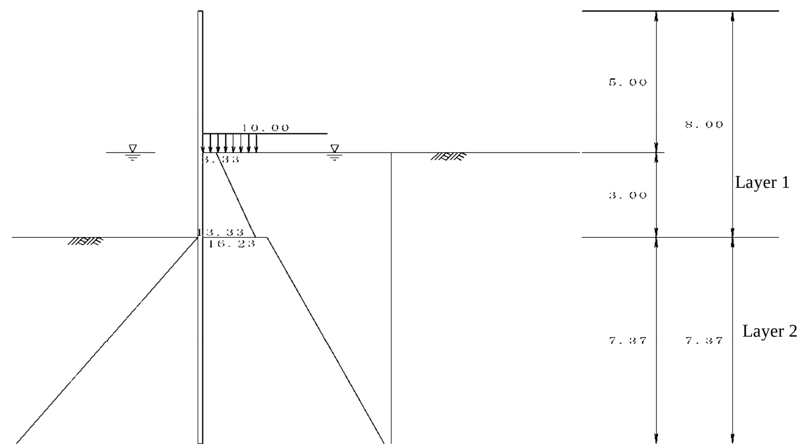
Corrosion allowance $t_1$=0.00 mm (Front side) $t_2$=0.00 mm (Back side)
Corrosion ratio(against $I_0$) η=1.00
Corrosion ratio(against $Z_0$) η=1.00
Efficiency of joint(against $I_0$) μ=1.00
Efficiency of joint(against $Z_0$) μ=1.00

Allowable stress intensity $\sigma_a$ =165 N/mm$^2$ (Normal condition)

Allowable Displacement $\delta_a$ = 50.0 mm (Normal condition)

2 Calculation of lateral pressure

2-1 Normal condition




Passive earth pressure Active earth pressure Residual water pressure

2-1-1 Active side Soil constants

| | Depth m | Soil type | γ kN/m3 | φ Deg | C kN/m² | Σγh+Qa kN/m² | Ka | Ka ×cosδ |
|---|---|---|---|---|---|---|---|---|
| 1 | 0.00～<br>5.00 | — | — | — | —<br>— | —<br>— | —<br>— | —<br>— |
| 2 | 5.00～<br>8.00 | Sandy soil | 10.0 | 30.0 | —<br>— | 10.000<br>40.000 | 0.33333<br>0.33333 | 0.33333<br>0.33333 |
| 3 | 8.00～<br>15.37 | Cohesive soil | 10.0 | 25.0 | —<br>— | 40.000<br>113.700 | 0.40586<br>0.40586 | 0.40586<br>0.40586 |
| 4 | 15.37～<br>20.29 | Cohesive soil | 9.0 | — | 79.0<br>79.0 | 113.700<br>158.000 | —<br>— | —<br>— |
| 5 | 20.29～<br>21.37 | Cohesive soil | 9.0 | — | 79.0<br>79.0 | 158.000<br>167.700 | —<br>— | —<br>— |
| 6 | 21.37～<br>30.00 | Sandy soil | 10.0 | 38.0 | —<br>— | 167.700<br>254.000 | 0.23788<br>0.23788 | 0.23788<br>0.23788 |

・Coefficient of active soil pressure of Sandy soil （$K_a$） is,
δ＝0.00、β＝0.00、θ＝0.00

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

2-1-2 Soil constants of passive side

| No | Depth m | Soil type | γ kN/m³ | φ Deg | C kN/m² | Σγh+Qp kN/m² | Kp | Kp ×cosδ |
|---|---|---|---|---|---|---|---|---|
| 3 | 8.00～<br>15.37 | Cohesive soil | 10.0 | 25.0 | —<br>— | 0.000<br>73.700 | 2.46391<br>2.46391 | 2.46391<br>2.46391 |
| 4 | 15.37～<br>20.29 | Cohesive soil | 9.0 | — | 79.0<br>79.0 | 73.700<br>118.000 | —<br>— | —<br>— |
| 5 | 20.29～<br>21.37 | Cohesive soil | 9.0 | — | 79.0<br>79.0 | 118.000<br>127.700 | —<br>— | —<br>— |
| 6 | 21.37～<br>30.00 | Sandy soil | 10.0 | 38.0 | —<br>— | 127.700<br>214.000 | 4.20375<br>4.20375 | 4.20375<br>4.20375 |

・Coefficient of passive soil pressure of Sandy soil （$K_p$） is,
δ＝0.00、β＝0.00、θ＝0.00

$$K_p = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta-\theta)\cdot\left[1-\sqrt{\dfrac{\sin(\phi-\delta)\cdot\sin(\phi+\beta-\theta)}{\cos(\delta-\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-1-3 Lateral pressure intensity

| No | Depth m | Active side Pa1 kN/m² | Active side Pa2 kN/m² | Active side Pa kN/m² | Residual water pressure Pw kN/m² | Passive side Pp kN/m² |
|---|---|---|---|---|---|---|
| 1 | 0.00～<br>5.00 | —<br>— | —<br>— | —<br>— | —<br>— | —<br>— |
| 2 | 5.00～<br>8.00 | 3.33<br>13.33 | —<br>— | 3.33<br>13.33 | 0.00<br>0.00 | —<br>— |
| 3 | 8.00～<br>15.37 | 16.23<br>46.15 | —<br>— | 16.23<br>46.15 | 0.00<br>0.00 | 0.00<br>181.59 |
| 4 | 15.37～<br>20.29 | -44.30<br>0.00 | 56.85<br>79.00 | 56.85<br>79.00 | 0.00<br>0.00 | 231.70<br>276.00 |
| 5 | 20.29～<br>21.37 | 0.00<br>9.70 | 79.00<br>83.85 | 79.00<br>83.85 | 0.00<br>0.00 | 276.00<br>285.70 |
| 6 | 21.37～<br>30.00 | 39.89<br>60.42 | —<br>— | 39.89<br>60.42 | 0.00<br>0.00 | 536.82<br>899.60 |

・Calculation formula of passive earth pressure

Sandy soil $P_{a1} = K_a\cdot\left[\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}\right]\cdot\cos\delta$

Cohesive soil $P_{a1} = \Sigma\gamma h + Q - 2C$

$P_{a2} = K_c\cdot(\Sigma\gamma h + Q)$

Medium soil $P_{a1} = \left[K_a(\Sigma\gamma h + Q) - 2C\sqrt{K_a}\right]\cdot\cos\delta$

・Calculation formula of passive earth pressure

Sandy soil $P_p = K_p\cdot\left[\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}\right]\cdot\cos\delta$

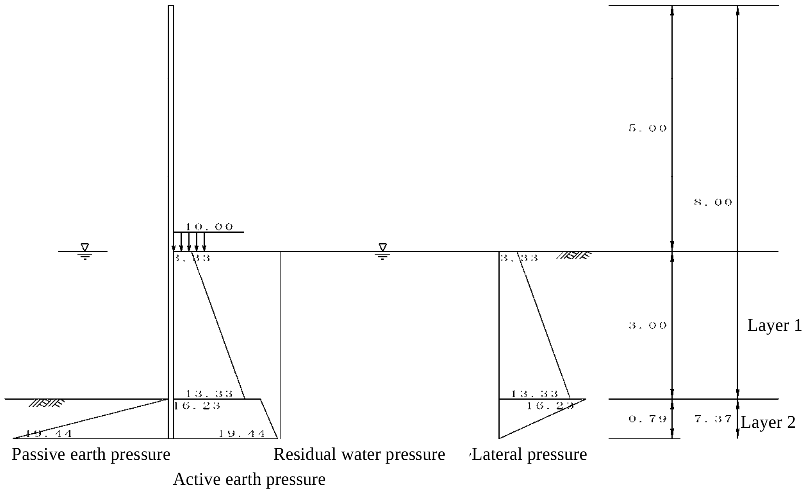
Cohesive soil $P_p = \Sigma\gamma h + Q + 2C$

Medium soil $P_p = \left[K_p(\Sigma\gamma h + Q) + 2C\sqrt{K_p}\right]\cdot\cos\delta$

## 3 Virtual ground level

Virtual ground level $L_k$ is the depth which sum of active soil pressure and Residual water pressure and Passive soil pressure are balanced.

### 3-1 Normal condition




| No | Depth m | Pa kN/m² | Pw kN/m² | Pp kN/m² | Ps kN/m² |
|---|---|---|---|---|---|
| 1 | 0.00～ 5.00 | ―― ―― | ―― ―― | ―― ―― | ―― ―― |
| 2 | 5.00～ 8.00 | 3.33 13.33 | 0.00 0.00 | ―― ―― | 3.33 13.33 |
| 3 | 8.00～ 8.79 | 16.23 19.44 | 0.00 0.00 | 0.00 19.44 | 16.23 0.00 |
| | 8.79～ 15.37 | 19.44 46.15 | 0.00 0.00 | 19.44 181.59 | 0.00 -135.44 |

$P_a$ : Active soil pressure
$P_w$ : Residual water pressure
$P_p$ : Passive soil pressure
$P_s$ : Lateral pressure intensity $\qquad P_s = P_a + P_w - P_p$

Virtual ground level $L_k$ = 0.79 m (GL-3.79 m)

## 4 Penetration depth

| | |
|---|---|
| Material | PU18 |
| Unit width | B ＝1.0000 m |
| Corrosion allowance | not considered |
| Corrosion ratio | $\eta$ ＝1.00 |
| Efficiency of joint | $\mu$ ＝1.00 |
| Young's modulus | E ＝ 200000 N/mm² |
| Second moment of area | $I_0$＝ 38650 cm⁴（before reduction） |
| | I ＝ 38650 cm⁴（after reduction by corrosion allowance and joint） |

$EI = 200000\times10^3\times38650\times10^{-8} = 7.730\times10^4$

### 4-1 Penetration depth and total pile length（Chang）

$$D = L_k + \frac{\pi}{\beta}$$
$$L = H - H_{1t} + D$$
$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$
$$K_h = 6910 \times N^{,0.406}$$

$$x = (h_i \times \beta_i) + (h_{i+1} \times \beta_{i+1}) + (h_{i+2} \times \beta_{i+2}) + \cdot\cdot\cdot + (h_n \times \beta_n)$$

where

$x : L = x/\beta,\ x = \pi$

#### 4-1-1 Normal condition

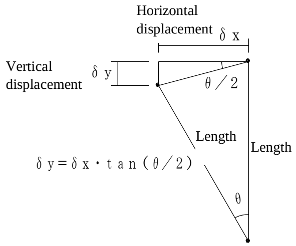
| No | Depth Z m | Layer thickness h m | N-value | αE0 kN/m² | Kh kN/m³ | β m⁻¹ | h×β (-) |
|---|---|---|---|---|---|---|---|
| 1 | 8.79～15.29 | 6.50 | 9.0 | ―― | 16862 | 0.48324 | 3.14159 |
| | | | | | | Σ(h×β)＝ | 3.14159 |

$$D = 0.79 + \frac{\pi}{\beta} = 7.29 \text{ m}$$
$$L = 8.00 - 0.70 + 7.29 = 14.59 \text{ m}$$

Total length of sheet pile ＝15.00m

5 Calculation of section force and Displacement

Top : free（horizontal, vertical and rotation :free）
Bottom : pin（vertical ,horizontal: fix、rotation: free）




5-1 Result of frame calculation

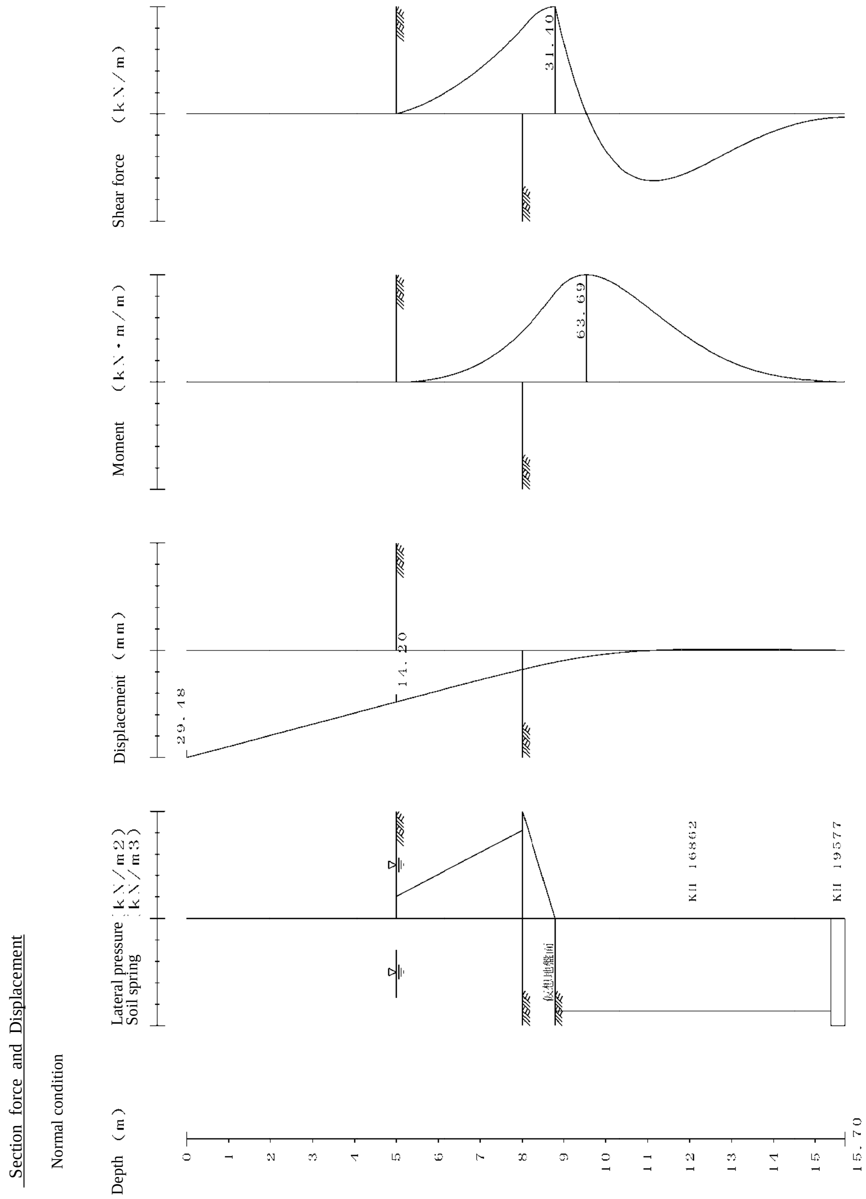
5-1-1 Normal condition

Maximum bending Moment $M_{max}$

$M_{max}(+)$ ＝ 0.00 kN・m/m
$M_{max}(-)$ ＝ -63.69 kN・m/m

Horizontal displacement at the ground ＝ 14.20 ≦ 50.00 mm

| Depth (m) | Distance from top(m) | Bending Moment (kNm/m) | Shearing force (kN/m) | Horizontal displacement (mm) | Deflection angle (rad/10$^3$) | Vertical displacement (mm) |
|---|---|---|---|---|---|---|
| 0.00 | -0.70 | 0.00 | 0.00 | 29.48 | -3.0566 | 0.0423 |
| 0.70 | 0.00 | 0.00 | 0.00 | 27.34 | -3.0566 | 0.0391 |
| 1.20 | 0.50 | 0.00 | 0.00 | 25.82 | -3.0566 | 0.0367 |
| 1.70 | 1.00 | 0.00 | 0.00 | 24.29 | -3.0566 | 0.0344 |
| 2.20 | 1.50 | 0.00 | 0.00 | 22.76 | -3.0566 | 0.0321 |
| 2.70 | 2.00 | 0.00 | 0.00 | 21.23 | -3.0566 | 0.0297 |
| 3.20 | 2.50 | 0.00 | 0.00 | 19.70 | -3.0566 | 0.0274 |
| 3.70 | 3.00 | 0.00 | 0.00 | 18.17 | -3.0566 | 0.0251 |
| 4.20 | 3.50 | 0.00 | 0.00 | 16.65 | -3.0566 | 0.0227 |
| 4.70 | 4.00 | 0.00 | 0.00 | 15.12 | -3.0566 | 0.0204 |
| 5.00 | 4.30 | 0.00 | 0.00 | 14.20 | -3.0566 | 0.0190 |
| 5.20 | 4.50 | -0.07 | -0.73 | 13.59 | -3.0565 | 0.0181 |
| 5.70 | 5.00 | -1.01 | -3.15 | 12.06 | -3.0537 | 0.0157 |
| 6.20 | 5.50 | -3.36 | -6.40 | 10.54 | -3.0405 | 0.0134 |
| 6.70 | 6.00 | -7.55 | -10.48 | 9.03 | -3.0063 | 0.0111 |
| 7.20 | 6.50 | -13.98 | -15.40 | 7.54 | -2.9380 | 0.0089 |
| 7.70 | 7.00 | -23.09 | -21.15 | 6.10 | -2.8197 | 0.0067 |
| 8.00 | 7.30 | -30.00 | -25.00 | 5.26 | -2.7170 | 0.0056 |
| 8.20 | 7.50 | -35.30 | -27.84 | 4.73 | -2.6327 | 0.0048 |
| 8.70 | 8.00 | -50.30 | -31.32 | 3.48 | -2.3568 | 0.0032 |
| 8.79 | 8.09 | -53.09 | -31.40 | 3.27 | -2.2974 | 0.0030 |
| 9.20 | 8.50 | -61.77 | -11.86 | 2.39 | -1.9883 | 0.0019 |
| 9.54 | 8.84 | -63.69 | 0.02 | 1.76 | -1.7098 | 0.0013 |
| 9.70 | 9.00 | -63.33 | 4.37 | 1.50 | -1.5794 | 0.0011 |
| 10.20 | 9.50 | -58.51 | 13.94 | 0.81 | -1.1827 | 0.0005 |
| 10.70 | 10.00 | -50.23 | 18.50 | 0.31 | -0.8298 | 0.0002 |
| 11.20 | 10.50 | -40.60 | 19.55 | -0.03 | -0.5358 | 0.0001 |
| 11.70 | 11.00 | -31.06 | 18.31 | -0.24 | -0.3044 | 0.0000 |
| 12.20 | 11.50 | -22.50 | 15.78 | -0.35 | -0.1319 | 0.0000 |
| 12.70 | 12.00 | -15.38 | 12.66 | -0.38 | -0.0102 | 0.0000 |
| 13.20 | 12.50 | -9.84 | 9.49 | -0.36 | 0.0705 | 0.0000 |
| 13.70 | 13.00 | -5.83 | 6.61 | -0.32 | 0.1204 | 0.0000 |
| 14.20 | 13.50 | -3.15 | 4.23 | -0.25 | 0.1488 | 0.0000 |
| 14.70 | 14.00 | -1.50 | 2.47 | -0.17 | 0.1634 | 0.0000 |
| 15.20 | 14.50 | -0.57 | 1.39 | -0.09 | 0.1698 | 0.0000 |
| 15.37 | 14.67 | -0.35 | 1.19 | -0.06 | 0.1708 | 0.0000 |
| 15.70 | 15.00 | 0.00 | 1.00 | 0.00 | 0.1715 | 0.0000 |



## 5-2 Calculation of Stress intensity

Material PU18

Corrosion allowance $t_1$=0.00 mm (Front side) $t_2$=0.00 mm (Back side)

Corrosion ratio $\eta$ =1.00

Efficiency of joint $\mu$ =1.00

Section modulus $Z_0$= 1800 $cm^3$ (before reduction)

$Z$ = 1800 $cm^3$ (after reduction by Corrosion allowance and joint)

### 5-2-1 Normal condition

$$\sigma = \frac{M_{max}}{Z} = \frac{63.69\times10^6}{1800\times10^3} = 35\ N/mm^2 \leqq \sigma_a = 165\ N/mm^2$$

## 6 Calculation results

Steel sheet pile PU18

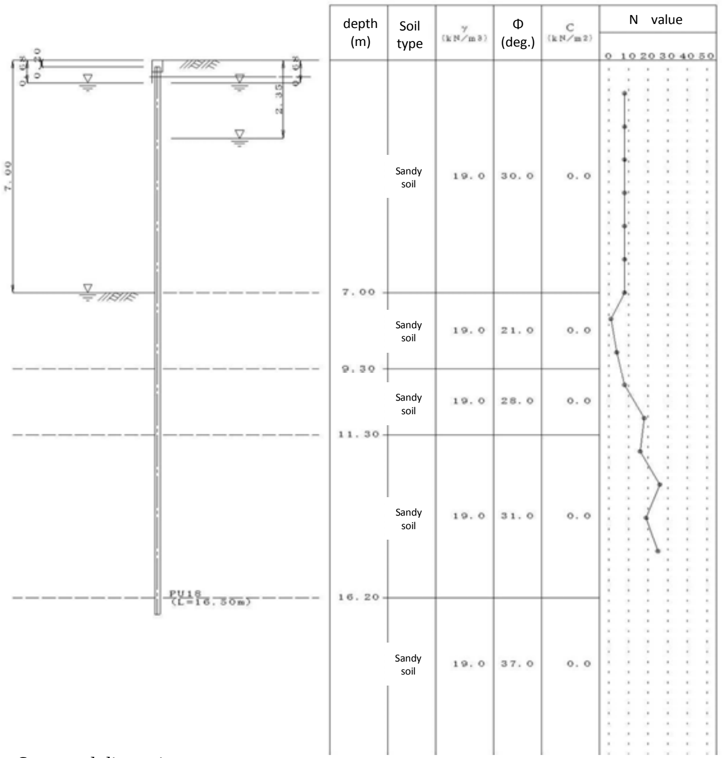
| | | | Normal condition |
|---|---|---|---|
| Second moment of area | I ($cm^4$) | 38650 | |
| Section modulus | Z ($cm^3$) | 1800 | |
| Max. bending moment | Mmax(kN・m/m) | | 63.69 |
| | σ ($N/mm^2$) | | 35 ( 165) |
| Horizontal displacement | δ (mm) | | 14.20 ( 50.0) |
| Penetration depth | D (m) | | 7.29 |
| Total length of sheet pile | L (m) | 15.0 | |

A-8-2 Structural calculation of steel sheet pile works (Type BL-1)

<Bracing style sheet pile wall>

## 1 Design conditions

### 1-1 Structural figure




### 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design height | H= | 8.00 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 2.65 | m (Regular condition) |
| | Lwa'= | 1.20 | m (Seismic condition) |
| Outside water level | Lwp= | 8.00 | m (Regular condition) |
| | Lwp'= | 1.20 | m (Seismic condition) |
| Tie-rod installation position | Ht= | 1.00 | m |
| Tie-rod horizontal intervals | l= | 2.40 | m |
| Tie-rod installation angle | θ = | 0.0000 | degree |

Method of calculation; Free earth support method

Virtual supporting point; Design front ground level

1-3 Design constant parameters

Unit weight of water $\gamma w=$ 10.0 $kN/m^3$

Type of water pressure; trapezoid

Design Seismic coefficient (in the air) k= 0.080

(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma\gamma_t \cdot h_i + \Sigma\gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma\gamma_t \cdot h_i + \Sigma(\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake; Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma\gamma h + 2Q}{2C} \cdot \tan\theta}$$

Safety factor of footing depth; 1.50 (Regular condition)
1.20 (Seismic condition)

1-4 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N': Average N-value

Result of boring survey

| No | Depth m | N-value |
|---|---|---|
| 1 | 1.00 | 8 |
| 2 | 2.00 | 8 |
| 3 | 3.00 | 8 |
| 4 | 4.00 | 8 |
| 5 | 5.00 | 8 |
| 6 | 6.00 | 8 |
| 7 | 7.00 | 8 |
| 8 | 7.80 | 1 |
| 9 | 8.80 | 4 |
| 10 | 9.80 | 8 |

| No | Depth m | N-value |
|---|---|---|
| 11 | 10.80 | 18 |
| 12 | 11.80 | 16 |
| 13 | 12.80 | 26 |
| 14 | 13.80 | 19 |
| 15 | 14.80 | 25 |

1-5 Overburden pressure

Active pressure side Qa= 10.0 $kN/m^2$ (Regular condition)
Qa'= 5.0 $kN/m^2$ (Seismic condition)

Passive pressure side Qp= 0.0 $kN/m^2$ (Regular condition)
Qp'= 0.0 $kN/m^2$ (Seismic condition)

1-6 Soil constant

| No | Depth m | type | N-value | γ $kN/m^3$ | γ' $kN/m^3$ | φ Deg. | C $kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 7.00 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 9.30 | S | 3 | 19.0 | 10.0 | 21.0 | 0.0 | 0.0 |
| 3 | 11.30 | S | 14 | 19.0 | 10.0 | 28.0 | 0.0 | 0.0 |
| 4 | 16.20 | S | 22 | 19.0 | 10.0 | 31.0 | 0.0 | 0.0 |
| 5 | 20.20 | S | 42 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

1-7 Sheet pile

Type PU18 (S270GP)
Young modulus E = 200,000 $N/mm^2$
Second moment of area I0= 38,650 $cm^4$ (before corrosion)
Modulus of section Z0= 1,800 $cm^3$ (before corrosion)

Corrosion allowance t1= 0.00 mm (front side) t2= 0.00 mm (back side)

Allowable stress σa = 165 $N/mm^2$ (Regular condition)
σa' = 190 $N/mm^2$ (Seismic condition)

1-8 Tie-rod

Diameter of tie-rod calculated

Corrosion considered (0.00mm)

Allowable stress σa = 176 $N/mm^2$ (Regular condition)
σa' = 264 $N/mm^2$ (Seismic condition)

1-9 Waling

Type 2×UPN-200×75

Modulus of section; 382 cm3

Corrosion ; considered (0.00mm)
Corrosion ratio; η = 1.00

Allowable stress σa = 140 $N/mm^2$ (Regular condition)
σa' = 161 $N/mm^2$ (Seismic condition)

## 1-10 Bracing sheet pile

Calculation method    Chang's formula

| | | | |
|---|---|---|---|
| Crest elevation of bracing works | | 0.30 | m (distance from tie-rod) |
| Materials | PU12 (S270GP) | | |
| Young's modulus | E = | 200,000 | $N/mm^2$ |
| Second moment of area | I0= | 21,600 | $cm^4$ (before corrosion) |
| Modulus of section | Z0= | 1,200 | $cm^3$ (before corrosion) |
| Corrosion allowance | t1= | 0.00 | mm (front side)    t2= 0.00 mm (back side) |
| Corrosion ratio (againstI0) | η= | 1.00 | |
| Corrosion ratio (against Z0) | η= | 1.00 | |
| Joint efficiency (against I0) | μ= | 1.00 | |
| Joint efficiency(againstZ0) | μ= | 1.00 | |
| Allowable stress | σa = | 165 | $N/mm^2$ (Regular condition) |
| | σa' = | 190 | $N/mm^2$ (Seismic condition) |
| Allowable displacement | δa = | 50.0 | mm (Regular condition) |
| | δa'= | 75.0 | mm (Seismic condition) |

## 2 Calculation results

Front sheet pile    PU18 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 38650 | | |
| Modulus of section | Z | ($cm^3$) | 1800 | | |
| Max. bending moment | Mmax | (kN・m/m) | | 204.59 | 108.64 |
| Stress intensity | σ | ($N/mm^2$) | | 114 (165) | 60 (190) |
| Footing depth | D | (m) | | 9.38 | 5.51 |
| Total length | L | (m) | 16.5 | | |

Tie-rod    φ 42 mm

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Tensile force | Tp | (kN) | | 207.15 | 127.65 |
| Stress intensity | σ | ($N/mm^2$) | | 150 (176) | 92 (264) |

Waling    2×UPN-200×75

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Modulus of section | Z | (cm3) | 382 | | |
| Max. bending moment | Mmax | (kN・m) | | 49.72 | 30.63 |
| Stress intensity | σ | ($N/mm^2$) | | 130 (140) | 80 (161) |

Bracing sheet pile    PU12 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 21,600 | | |
| Modulus of section | Z | ($cm^3$) | 1,200 | | |
| Max. bending moment | Mmax | (kN・m) | | 50.39 | 31.05 |
| Stress intensity | σ | ($N/mm^2$) | | 42 (165) | 26 (190) |
| Horizontal displacement | δ | (mm) | | 5.93 (50.0) | 3.65 (75.0) |
| Footing depth | D | (m) | | 5.43 | 5.43 |
| Total length | L | (m) | 6.00 | | |

Bracing works Installation position

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Bracing works Installation position | d | (m) | | 7.04 | 8.36 |

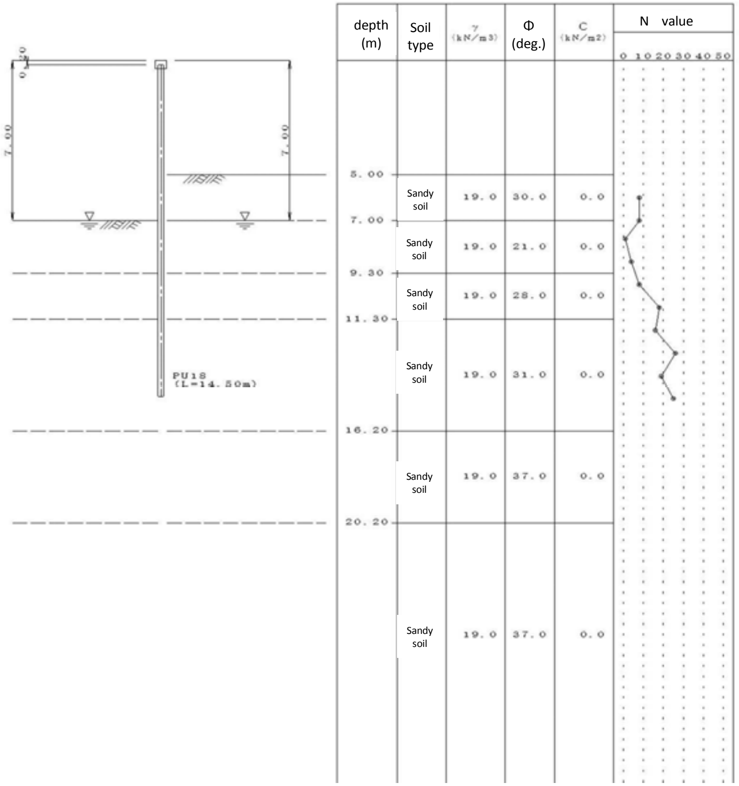
Structural calculation of steel sheet pile works (Type BL-1)

**— During construction —**

<Cantilevered sheet pile wall>

# 1 Design conditions

## 1-1 Structural figure




## 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 7.00 | m |
| Protruction lemgth | H0= | 5.00 | |
| Top of sheet pile | Hlt= | 0.20 | m |
| Inside water level | Lwa= | 7.00 | m (Regular condition) |
| Outside water level | Lwp= | 7.00 | m (Regular condition) |

1-3 Calculation method

Calculation method Chang's formula

Footing depth $L = \frac{\pi}{\beta}$

1-4 Design constant parameters

Unit weight of water $\gamma w =$ 10.0 $kN/m^3$

Type of water pressure; trapezoid

1-5 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N': Average N-value

1-6 Overburden pressure

Active pressure side Qa= 10.0 $kN/m^2$ (Regular condition)

Passive pressure side Qp= 0.0 $kN/m^2$ (Regular condition)

1-7 Soil constant

| No. | Depth<br>m | Layer thickness<br>m |
|---|---|---|
| 1 | 7.00 | 2.00 |
| 2 | 9.30 | 2.30 |
| 3 | 11.30 | 2.00 |
| 4 | 16.20 | 4.90 |
| 5 | 20.20 | 4.00 |
| 6 | 30.00 | 9.80 |

| No | Depth<br>m | type | N-value | γ<br>$kN/m^3$ | γ'<br>$kN/m^3$ | φ<br>Deg. | C<br>$kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 7.00 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 9.30 | S | 3 | 19.0 | 10.0 | 21.0 | 0.0 | 0.0 |
| 3 | 11.30 | S | 14 | 19.0 | 10.0 | 28.0 | 0.0 | 0.0 |
| 4 | 16.20 | S | 22 | 19.0 | 10.0 | 31.0 | 0.0 | 0.0 |
| 5 | 20.20 | S | 42 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |
| 6 | 30.00 | S | 41 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

1-8 Sheet pile

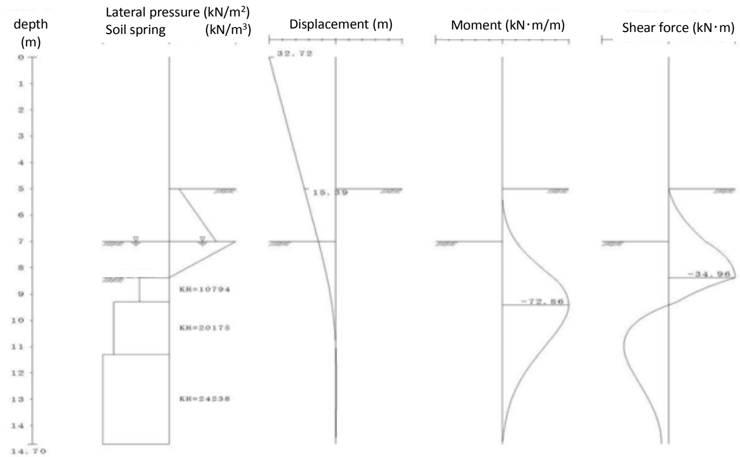
| | | | |
|---|---|---|---|
| Type | PU18 (S270GP) | | |
| Young modulus | E ＝ | 200,000 | $N/mm^2$ |
| Second moment of area | I0＝ | 38650 | $cm^4$（before corrosion） |
| Modulus of section | Z0＝ | 1800 | $cm^3$（before corrosion） |

Corrosion allowance t1＝ 0.00 mm (front side) t2＝ 0.00 mm (back side)
Corrosion ratio (againstI0) η＝ 1.00
Corrosion ratio (against Z0) η＝ 1.00
Joint efficiency (against I0) μ＝ 1.00
Joint efficiency(againstZ0) μ＝ 1.00

Allowable stress σa = 165 $N/mm^2$（Regular condition）

Allowable displacement δa ＝ 50.0 mm（Regular condition）

2 Section force and Displacement




3 Calculation results

Front sheet pile PU18 (S270GP)

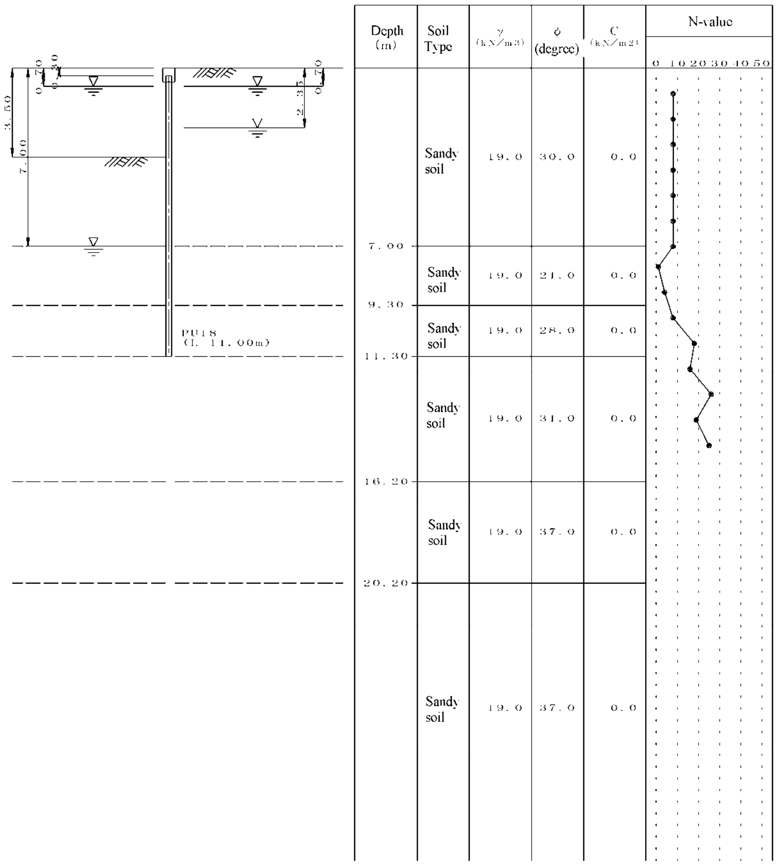
| | | | Regular condition | |
|---|---|---|---|---|
| Second moment of area | I (cm⁴) | 38650 | | |
| Modulus of section | Z (cm³) | 1800 | | |
| Max. bending moment | Mmax (kN･m/m) | | 72.86 | |
| Stress intensity | σ (N/mm²) | | 40 (165) | |
| Horizontal displacement | δ (mm) | | 15.39 (50.0) | |
| Footing depth | D (m) | | 7.57 | |
| Total length | L (m) | 14.50 | | |

A-8-3 Structural calculation of steel sheet pile works (Type BL-2)

<Cantilevered sheet pile wall>

1 Design conditions

1-1 Structural figure



1-2 Structural dimensions

Design base level $H = 3.50$ m
Top of sheet pile $H_{lt} = 0.30$ m
Inside water level $L_{wa} = 2.35$ m(Normal condition)
$L_{wa}' = 0.70$ m(Seismic condition)
Outside water level $L_{wp} = 7.00$ m(Normal condition)
$L_{wp}' = 0.70$ m(Seismic condition)

1-3 Calculation method

Calculation method Chang's formula

Footing depth $L = \dfrac{\pi}{\beta}$

1-4 Design constants

Unit weight of water $\gamma_w = 10.0$ kN/m$^3$

Type of water pressure; trapezoid

Design seismic coefficient (in the air) $k = 0.080$
(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma\gamma_t \cdot h_i + \Sigma\gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma\gamma_t \cdot h_i + \Sigma(\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake; Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma\gamma h + 2Q}{2C} \cdot \tan\theta}$$

$K_c = 0.50$

1-5 Coefficient of horizontal subgrade reaction

Formula
$$K_h = 6910 \times N'^{0.406}$$

1-6 Overburden load

Active pressure side
$Q_a = 10.0$ kN/m$^2$ (normal condition
$Q_a' = 5.0$ kN/m$^2$ (seismic condition)

Passive pressure side
$Q_p = 0.0$ kN/m$^2$ (normal condition)
$Q_p' = 0.0$ kN/m$^2$ (seismic condition)

1-7 Soil constants

Soil constants

| No | Depth m | Layer thickness m |
|---|---|---|
| 1 | 7.00 | 7.00 |
| 2 | 9.30 | 2.30 |
| 3 | 11.30 | 2.00 |
| 4 | 16.20 | 4.90 |
| 5 | 20.20 | 4.00 |
| 6 | 30.00 | 9.80 |

| No | Depth m | Soil type | N-value | γ kN/m$^3$ | γ' kN/m$^3$ | φ Deg, | C kN/m$^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 7.00 | S | 8.0 | 19.00 | 10.00 | 30.0 | 0.0 | 0.0 |
| 2 | 9.30 | S | 3.0 | 19.00 | 10.00 | 21.0 | 0.0 | 0.0 |
| 3 | 11.30 | S | 14.0 | 19.00 | 10.00 | 28.0 | 0.0 | 0.0 |
| 4 | 16.20 | S | 22.0 | 19.00 | 10.00 | 31.0 | 0.0 | 0.0 |
| 5 | 20.20 | S | 42.0 | 19.00 | 10.00 | 37.0 | 0.0 | 0.0 |
| 6 | 30.00 | S | 41.0 | 19.00 | 10.00 | 37.0 | 0.0 | 0.0 |

Depth : Distance from the top of sheet pile C : Adhesive force
type : Sandy soil(S)、Cohesive soil(C)、Medium soil(M) a : Angle of adhesive force

N value : average value
γ : unit weight of humid soil
γ' : unit weight of soil under the water
φ : internal friction angle

1-8 Steel sheet pile

Material PU18
Young's modulus $E = 200000$ N/mm$^2$
Second moment of area $I_0 = 38650$ cm$^4$ (before corrosion)
Section modulus $Z_0 = 1800$ cm$^3$ (before corrosion)

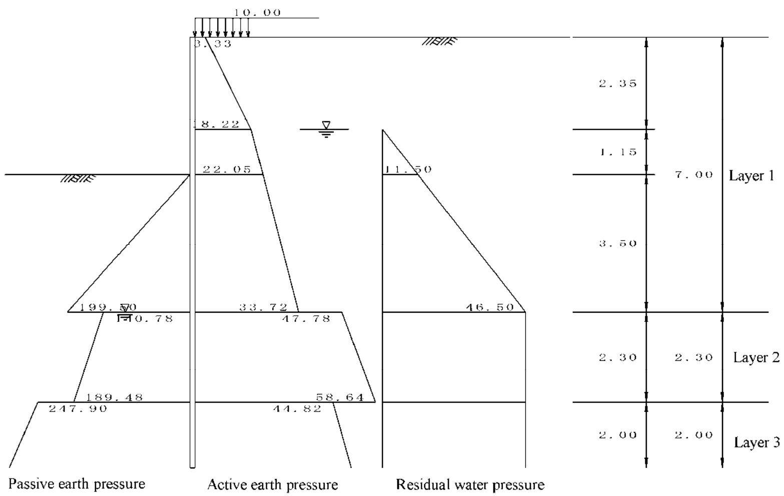
Corrosion allowance $t_1 = 0.00$ mm (Front side) $t_2 = 0.00$ mm (Back side)
Corrosion ratio(against $I_0$) $\eta = 1.00$
Corrosion ratio(against $Z_0$) $\eta = 1.00$
Efficiency of joint(against $I_0$) $\mu = 1.00$
Efficiency of joint(against $Z_0$) $\mu = 1.00$

Allowable stress intensity $\sigma_a = 165$ N/mm$^2$ (Normal condition)
$\sigma_a' = 190$ N/mm$^2$ (Seismic condition)

Allowable Displacement $\delta_a = 50.0$ mm (Normal condition)
$\delta_a' = 75.0$ mm (Seismic condition)

## 2 Calculation of lateral pressure

### 2-1 Normal condition




#### 2-1-1 Active side Soil constants

| | Depth m | Soil Type | γ kN/m³ | φ degree | C kN/m² | Σγh+Qa kN/m² | Ka | Ka ×cos δ |
|---|---|---|---|---|---|---|---|---|
| 1 | 0.00～ 2.35 | Sandy soil | 19.0 | 30.0 | ——— ——— | 10.000 54.650 | 0.33333 0.33333 | 0.33333 0.33333 |
| 2 | 2.35～ 3.50 | Sandy soil | 10.0 | 30.0 | ——— ——— | 54.650 66.150 | 0.33333 0.33333 | 0.33333 0.33333 |
| 3 | 3.50～ 7.00 | Sandy soil | 10.0 | 30.0 | ——— ——— | 66.150 101.150 | 0.33333 0.33333 | 0.33333 0.33333 |
| 4 | 7.00～ 9.30 | Sandy soil | 10.0 | 21.0 | ——— ——— | 101.150 124.150 | 0.47236 0.47236 | 0.47236 0.47236 |
| 5 | 9.30～ 11.30 | Sandy soil | 10.0 | 28.0 | ——— ——— | 124.150 144.150 | 0.36103 0.36103 | 0.36103 0.36103 |
| 6 | 11.30～ 16.20 | Sandy soil | 10.0 | 31.0 | ——— ——— | 144.150 193.150 | 0.32010 0.32010 | 0.32010 0.32010 |
| 7 | 16.20～ 20.20 | Sandy soil | 10.0 | 37.0 | ——— ——— | 193.150 233.150 | 0.24858 0.24858 | 0.24858 0.24858 |
| 8 | 20.20～ 30.00 | Sandy soil | 10.0 | 37.0 | ——— ——— | 233.150 331.150 | 0.24858 0.24858 | 0.24858 0.24858 |

・Coefficient of active soil pressure of Sandy soil （$K_a$） is,
δ＝0.00、β＝0.00、θ＝0.00

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

#### 2-1-2 Soil constants of passive side

| No | Depth m | Soil Type | γ kN/m³ | φ degree | C kN/m² | Σγh+Qp kN/m² | Kp | Kp ×cos δ |
|---|---|---|---|---|---|---|---|---|
| 3 | 3.50～ 7.00 | Sandy soil | 19.0 | 30.0 | ——— ——— | 0.000 66.500 | 3.00000 3.00000 | 3.00000 3.00000 |
| 4 | 7.00～ 9.30 | Sandy soil | 10.0 | 21.0 | ——— ——— | 66.500 89.500 | 2.11705 2.11705 | 2.11705 2.11705 |
| 5 | 9.30～ 11.30 | Sandy soil | 10.0 | 28.0 | ——— ——— | 89.500 109.500 | 2.76983 2.76983 | 2.76983 2.76983 |
| 6 | 11.30～ 16.20 | Sandy soil | 10.0 | 31.0 | ——— ——— | 109.500 158.500 | 3.12404 3.12404 | 3.12404 3.12404 |
| 7 | 16.20～ 20.20 | Sandy soil | 10.0 | 37.0 | ——— ——— | 158.500 198.500 | 4.02279 4.02279 | 4.02279 4.02279 |
| 8 | 20.20～ 30.00 | Sandy soil | 10.0 | 37.0 | ——— ——— | 198.500 296.500 | 4.02279 4.02279 | 4.02279 4.02279 |

・Coefficient of passive soil pressure of Sandy soil （$K_p$） is,
δ＝0.00、β＝0.00、θ＝0.00

$$K_p = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta-\theta)\cdot\left[1-\sqrt{\dfrac{\sin(\phi-\delta)\cdot\sin(\phi+\beta-\theta)}{\cos(\delta-\theta)\cdot\cos(-\beta)}}\right]^2}$$

#### 2-1-3 Lateral pressure intensity

| No | Depth | Active side | | | Residual water pressure | Passive side |
|---|---|---|---|---|---|---|
| | m | Pa1 kN/m² | Pa2 kN/m² | Pa kN/m² | Pw kN/m² | Pp kN/m² |
| 1 | 0.00～ 2.35 | 3.33 18.22 | ——— ——— | 3.33 18.22 | ——— ——— | ——— ——— |
| 2 | 2.35～ 3.50 | 18.22 22.05 | ——— ——— | 18.22 22.05 | 0.00 11.50 | ——— ——— |
| 3 | 3.50～ 7.00 | 22.05 33.72 | ——— ——— | 22.05 33.72 | 11.50 46.50 | 0.00 199.50 |
| 4 | 7.00～ 9.30 | 47.78 58.64 | ——— ——— | 47.78 58.64 | 46.50 46.50 | 140.78 189.48 |
| 5 | 9.30～ 11.30 | 44.82 52.04 | ——— ——— | 44.82 52.04 | 46.50 46.50 | 247.90 303.30 |
| 6 | 11.30～ 16.20 | 46.14 61.83 | ——— ——— | 46.14 61.83 | 46.50 46.50 | 342.08 495.16 |

| No | Depth m | Active side | | | Residual water pressure | Passive side |
|---|---|---|---|---|---|---|
| | | Pa1 kN/m² | Pa2 kN/m² | Pa kN/m² | Pw kN/m² | Pp kN/m² |
| 7 | 16.20～<br>20.20 | 48.01<br>57.96 | ――――<br>―――― | 48.01<br>57.96 | 46.50<br>46.50 | 637.61<br>798.52 |
| 8 | 20.20～<br>30.00 | 57.96<br>82.32 | ――――<br>―――― | 57.96<br>82.32 | 46.50<br>46.50 | 798.52<br>1192.76 |

・Calculation formula of passive earth pressure

Sandy soil $P_{a1} = K_a \cdot [\Sigma \gamma h + \dfrac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Cohesive soil $P_{a1} = \Sigma \gamma h + Q - 2C$
$P_{a2} = K_c \cdot (\Sigma \gamma h + Q)$

Medium soil $P_{a1} = [\ K_a(\Sigma \gamma h + Q) - 2C\sqrt{K_a}\ ] \cdot \cos\delta$

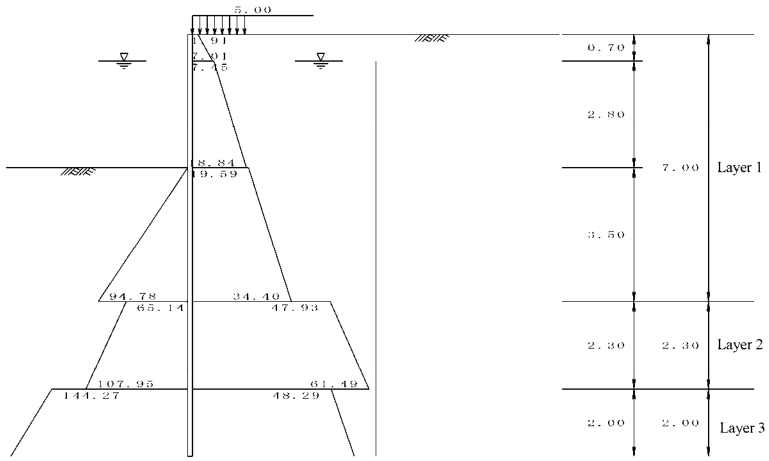
・Calculation formula of passive earth pressure

Sandy soil $P_p = K_p \cdot [\Sigma \gamma h + \dfrac{Q}{\cos(-\beta)}] \cdot \cos\delta$

Cohesive soil $P_p = \Sigma \gamma h + Q + 2C$

Medium soil $P_p = [\ K_p(\Sigma \gamma h + Q) + 2C\sqrt{K_p}\ ] \cdot \cos\delta$

2-2 Seismic condition




2-2-1 Active side Soil constants

| No | Depth m | Soil Type | γ kN/m³ | φ degree | C kN/m² | Σγh+Q kN/m² | γwhw kN/m² | k (k') | θ degree | Ka | Ka ×cosδ | ζ degree |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.00～<br>0.70 | Sandy soil | 19.0 | 30.0 | ――<br>―― | 5.00<br>18.30 | 0.00<br>0.00 | 0.080<br>0.080 | 4.57<br>4.57 | 0.38296<br>0.38296 | 0.38296<br>0.38296 | ――<br>―― |
| 2 | 0.70～<br>3.50 | Sandy soil | 10.0 | 30.0 | ――<br>―― | 18.30<br>46.30 | 0.00<br>28.00 | 0.115<br>0.115 | 6.54<br>6.54 | 0.40686<br>0.40686 | 0.40686<br>0.40686 | ――<br>―― |
| 3 | 3.50～<br>7.00 | Sandy soil | 10.0 | 30.0 | ――<br>―― | 46.30<br>81.30 | 28.00<br>63.00 | 0.137<br>0.137 | 7.80<br>7.80 | 0.42317<br>0.42317 | 0.42317<br>0.42317 | ――<br>―― |
| 4 | 7.00～<br>9.30 | Sandy soil | 10.0 | 21.0 | ――<br>―― | 81.30<br>104.30 | 63.00<br>86.00 | 0.144<br>0.144 | 8.21<br>8.21 | 0.58959<br>0.58959 | 0.58959<br>0.58959 | ――<br>―― |
| 5 | 9.30～<br>11.30 | Sandy soil | 10.0 | 28.0 | ――<br>―― | 104.30<br>124.30 | 86.00<br>106.00 | 0.147<br>0.147 | 8.37<br>8.37 | 0.46295<br>0.46295 | 0.46295<br>0.46295 | ――<br>―― |
| 6 | 11.30～<br>16.20 | Sandy soil | 10.0 | 31.0 | ――<br>―― | 124.30<br>173.30 | 106.00<br>155.00 | 0.150<br>0.150 | 8.54<br>8.54 | 0.41763<br>0.41763 | 0.41763<br>0.41763 | ――<br>―― |
| 7 | 16.20～<br>20.20 | Sandy soil | 10.0 | 37.0 | ――<br>―― | 173.30<br>213.30 | 155.00<br>195.00 | 0.152<br>0.152 | 8.67<br>8.67 | 0.33535<br>0.33535 | 0.33535<br>0.33535 | ――<br>―― |
| 8 | 20.20～<br>30.00 | Sandy soil | 10.0 | 37.0 | ――<br>―― | 213.30<br>311.30 | 195.00<br>293.00 | 0.154<br>0.154 | 8.78<br>8.78 | 0.33665<br>0.33665 | 0.33665<br>0.33665 | ――<br>―― |

・Coefficient of sandy soil passive earth pressure ($K_a$) is obtained by following formula
$\delta = 0.00$degree、$\beta = 0.00$degree
compound angle $\theta = \tan^{-1} k$

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

$$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C}}\cdot\tan\theta$$

2-2-2 Soil constants of Passive side

| No | Depth m | Soil Type | γ kN/m³ | φ degree | C kN/m² | Σγh+Qp kN/m² | γwhw kN/m² | k (k') | θ degree | Kp | Kp ×cosδ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 3 | 3.50～<br>7.00 | Sandy soil | 10.00 | 30.0 | ――<br>―― | 0.000<br>35.000 | 0.00<br>35.00 | 0.160<br>0.160 | 9.09<br>9.09 | 2.70792<br>2.70792 | 2.70792<br>2.70792 |
| 4 | 7.00～<br>9.30 | Sandy soil | 10.00 | 21.0 | ――<br>―― | 35.000<br>58.000 | 35.00<br>58.00 | 0.160<br>0.160 | 9.09<br>9.09 | 1.86128<br>1.86128 | 1.86128<br>1.86128 |
| 5 | 9.30～<br>11.30 | Sandy soil | 10.00 | 28.0 | ――<br>―― | 58.000<br>78.000 | 58.00<br>78.00 | 0.160<br>0.160 | 9.09<br>9.09 | 2.48737<br>2.48737 | 2.48737<br>2.48737 |
| 6 | 11.30～<br>16.20 | Sandy soil | 10.00 | 31.0 | ――<br>―― | 78.000<br>127.000 | 78.00<br>127.00 | 0.160<br>0.160 | 9.09<br>9.09 | 2.82682<br>2.82682 | 2.82682<br>2.82682 |
| 7 | 16.20～<br>20.20 | Sandy soil | 10.00 | 37.0 | ――<br>―― | 127.000<br>167.000 | 127.00<br>167.00 | 0.160<br>0.160 | 9.09<br>9.09 | 3.68994<br>3.68994 | 3.68994<br>3.68994 |
| 8 | 20.20～<br>30.00 | Sandy soil | 10.00 | 37.0 | ――<br>―― | 167.000<br>265.000 | 167.00<br>265.00 | 0.160<br>0.160 | 9.09<br>9.09 | 3.68994<br>3.68994 | 3.68994<br>3.68994 |

・Coefficient of sandy soil passive earth pressure（$K_p$）is obtained by following formula
δ=0.00degree、β=0.00degree
compound angle θ=$\tan^{-1}$k

$$K_p = \frac{\cos^2(\phi-\theta)}{\cos\theta\cdot\cos(\delta-\theta)\cdot\left[1-\sqrt{\dfrac{\sin(\phi-\delta)\cdot\sin(\phi+\beta-\theta)}{\cos(\delta-\theta)\cdot\cos(-\beta)}}\right]^2}$$

2-2-3 Lateral pressure intensity

| No | Depth m | Active side Pa kN/m² | Residual water pressure Pw kN/m² | Passive side Pp kN/m² |
|---|---|---|---|---|
| 1 | 0.00～<br>0.70 | 1.91<br>7.01 | ――<br>―― | ――<br>―― |
| 2 | 0.70～<br>3.50 | 7.45<br>18.84 | 0.00<br>0.00 | ――<br>―― |
| 3 | 3.50～<br>7.00 | 19.59<br>34.40 | 0.00<br>0.00 | 0.00<br>94.78 |
| 4 | 7.00～<br>9.30 | 47.93<br>61.49 | 0.00<br>0.00 | 65.14<br>107.95 |
| 5 | 9.30～<br>11.30 | 48.29<br>57.55 | 0.00<br>0.00 | 144.27<br>194.01 |
| 6 | 11.30～<br>16.20 | 51.91<br>72.37 | 0.00<br>0.00 | 220.49<br>359.01 |
| 7 | 16.20～<br>20.20 | 58.12<br>71.53 | 0.00<br>0.00 | 468.62<br>616.22 |
| 8 | 20.20～<br>30.00 | 71.81<br>104.80 | 0.00<br>0.00 | 616.22<br>977.83 |

・Calculation formula of passive earth pressure

Sandy soil $P_{ai} = K_a\cdot[\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}]\cdot\cos\delta$

Cohesive soil $P_{ai} = \dfrac{(\Sigma\gamma h+Q)\cdot\sin(\theta+\zeta)}{\cos\theta\cdot\sin\zeta} - \dfrac{C}{\cos\zeta\cdot\sin\zeta}$

Medium soil $P_{ai} = [K_a(\Sigma\gamma h+Q)-2C\sqrt{K_a}]\cdot\cos\delta$

・Calculation formula of passive earth pressure

Sandy soil $P_p = K_p\cdot[\Sigma\gamma h + \dfrac{Q}{\cos(-\beta)}]\cdot\cos\delta$

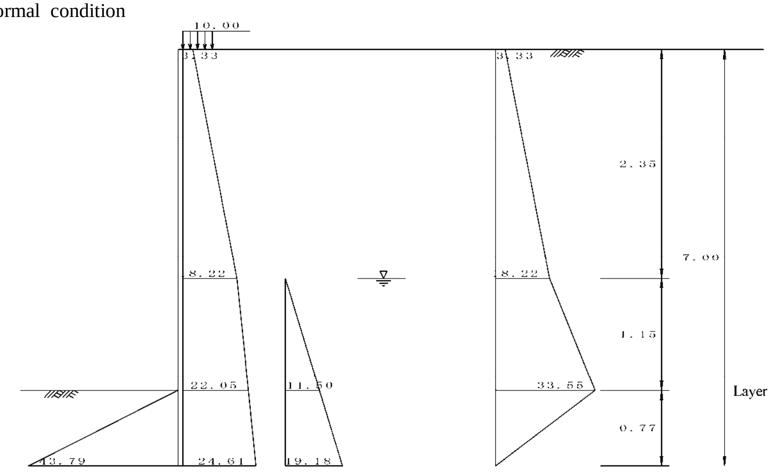
Cohesive soil $P_p = \Sigma\gamma h+Q+2C$

Medium soil $P_p = [K_p(\Sigma\gamma h+Q)+2C\sqrt{K_p}]\cdot\cos\delta$

## 3 Virtual ground level

Virtual ground level $L_k$ is the depth which sum of active soil pressure and Residual water pressure and Passive soil pressure are balanced.

### 3-1 Normal condition

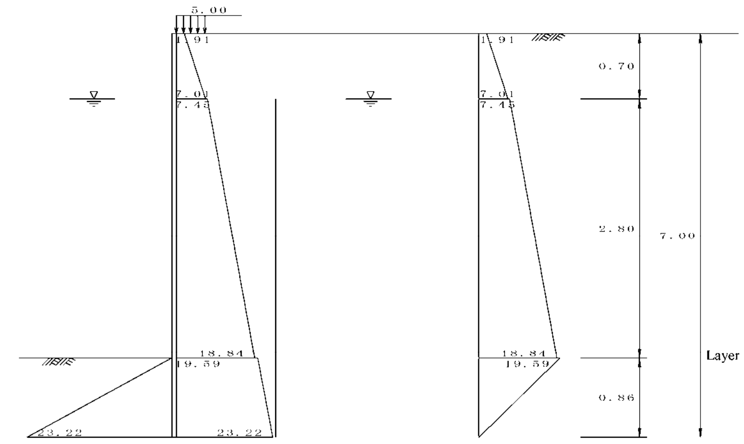



| No | Depth m | Pa kN/㎡ | Pw kN/㎡ | Pp kN/㎡ | Ps kN/㎡ |
|---|---|---|---|---|---|
| 1 | 0.00～ 2.35 | 3.33 18.22 | ―― ―― | ―― ―― | 3.33 18.22 |
| 2 | 2.35～ 3.50 | 18.22 22.05 | 0.00 11.50 | ―― ―― | 18.22 33.55 |
| 3 | 3.50～ 4.27 | 22.05 24.61 | 11.50 19.18 | 0.00 43.79 | 33.55 0.00 |
| 4 | 4.27～ 7.00 | 24.61 33.72 | 19.18 46.50 | 43.79 199.50 | 0.00 -119.28 |

$P_a$ : Active soil pressure
$P_w$ : Residual water pressure
$P_p$ : Passive soil pressure
$P_s$ : Lateral pressure intensity $P_s = P_a + P_w - P_p$

Virtual ground level $L_k$ = 0.77 m (GL-4.27 m)

### 3-2 Seismic condition




| No | Depth m | Pa kN/㎡ | Pw kN/㎡ | Pp kN/㎡ | Ps kN/㎡ |
|---|---|---|---|---|---|
| 1 | 0.00～ 0.70 | 1.91 7.01 | ―― ―― | ―― ―― | 1.91 7.01 |
| 2 | 0.70～ 3.50 | 7.45 18.84 | 0.00 0.00 | ―― ―― | 7.45 18.84 |
| 3 | 3.50～ 4.36 | 19.59 23.22 | 0.00 0.00 | 0.00 23.22 | 19.59 0.00 |
| 4 | 4.36～ 7.00 | 23.22 34.40 | 0.00 0.00 | 23.22 94.78 | 0.00 -60.37 |

$P_a$ : Active soil pressure
$P_w$ : Residual water pressure
$P_p$ : Passive soil pressure
$P_s$ : Lateral pressure intensity $P_s = P_a + P_w - P_p$

Virtual ground level $L_k$ = 0.86 m (GL-4.36 m)

## 4 Footing depth

| | |
|---|---|
| Material | PU18 |
| Unit width | B ＝1.0000 m |
| Corrosion allowance | not considered |
| Corrosion ratio | $\eta$ ＝1.00 |
| Efficiency of joint | $\mu$ ＝1.00 |
| Young's modulus | E ＝ 200000 N/mm$^2$ |
| Second moment of area | $I_0$＝ 38650 cm$^4$（before reduction） |
| | I ＝ 38650 cm$^4$（after reduction by corrosion allowance and joint） |

$EI = 200000\times10^3\times38650\times10^{-8} = 7.730\times10^4$

### 4-1 Footing depth and total pile length（Chang）

$$D = L_k + \frac{\pi}{\beta}$$

$$L = H - H_{1t} + D$$

$$\beta = \sqrt[4]{\frac{K_h \cdot B}{4EI}}$$

$$K_h = 6910\times N'^{\,0.406}$$

$$x = (h_i\times\beta_i) + (h_{i+1}\times\beta_{i+1}) + (h_{i+2}\times\beta_{i+2}) + \cdot\cdot\cdot + (h_n\times\beta_n)$$

where

x : $L = x/\beta$ , $x = \pi$

#### 4-1-1 Normal condition

| No | Depth Z m | Layer thickness h m | N-value | αE0 kN/m$^2$ | Kh kN/m$^3$ | β m$^{-1}$ | h×β (-) |
|---|---|---|---|---|---|---|---|
| 1 | 4.27～ 7.00 | 2.73 | 8.0 | —— | 16074 | 0.47750 | 1.30438 |
| 2 | 7.00～ 9.30 | 2.30 | 3.0 | —— | 10794 | 0.43225 | 0.99418 |
| 3 | 9.30～ 10.97 | 1.67 | 14.0 | —— | 20175 | 0.50541 | 0.84304 |
| | | | | | | Σ(h×β)＝ | 3.14159 |

$$D = 0.77 + \frac{\pi}{\beta} = 7.47 \text{ m}$$

$$L = 3.50 - 0.30 + 7.47 = 10.67 \text{ m}$$

#### 4-1-2 Seismic condition

| No | Depth Z m | Layer thickness h m | N-value | αE0 kN/m$^2$ | Kh kN/m$^3$ | β m$^{-1}$ | h×β (-) |
|---|---|---|---|---|---|---|---|
| 1 | 4.36～ 7.00 | 2.64 | 8.0 | —— | 16074 | 0.47750 | 1.26178 |
| 2 | 7.00～ 9.30 | 2.30 | 3.0 | —— | 10794 | 0.43225 | 0.99418 |
| 3 | 9.30～ 11.05 | 1.75 | 14.0 | —— | 20175 | 0.50541 | 0.88564 |
| | | | | | | Σ(h×β)＝ | 3.14159 |

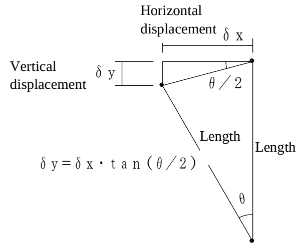
$$D = 0.86 + \frac{\pi}{\beta} = 7.55 \text{ m}$$

$$L = 3.50 - 0.30 + 7.55 = 10.75 \text{ m}$$

Total length of sheet pile ＝11.00 m

## 5 Calculation of section force and Displacement

Top : free（horizontal, vertical and rotation :free）
Bottom : pin（vertical ,horizontal: fix、rotation: free）




### 5-1 Result of frame calculation

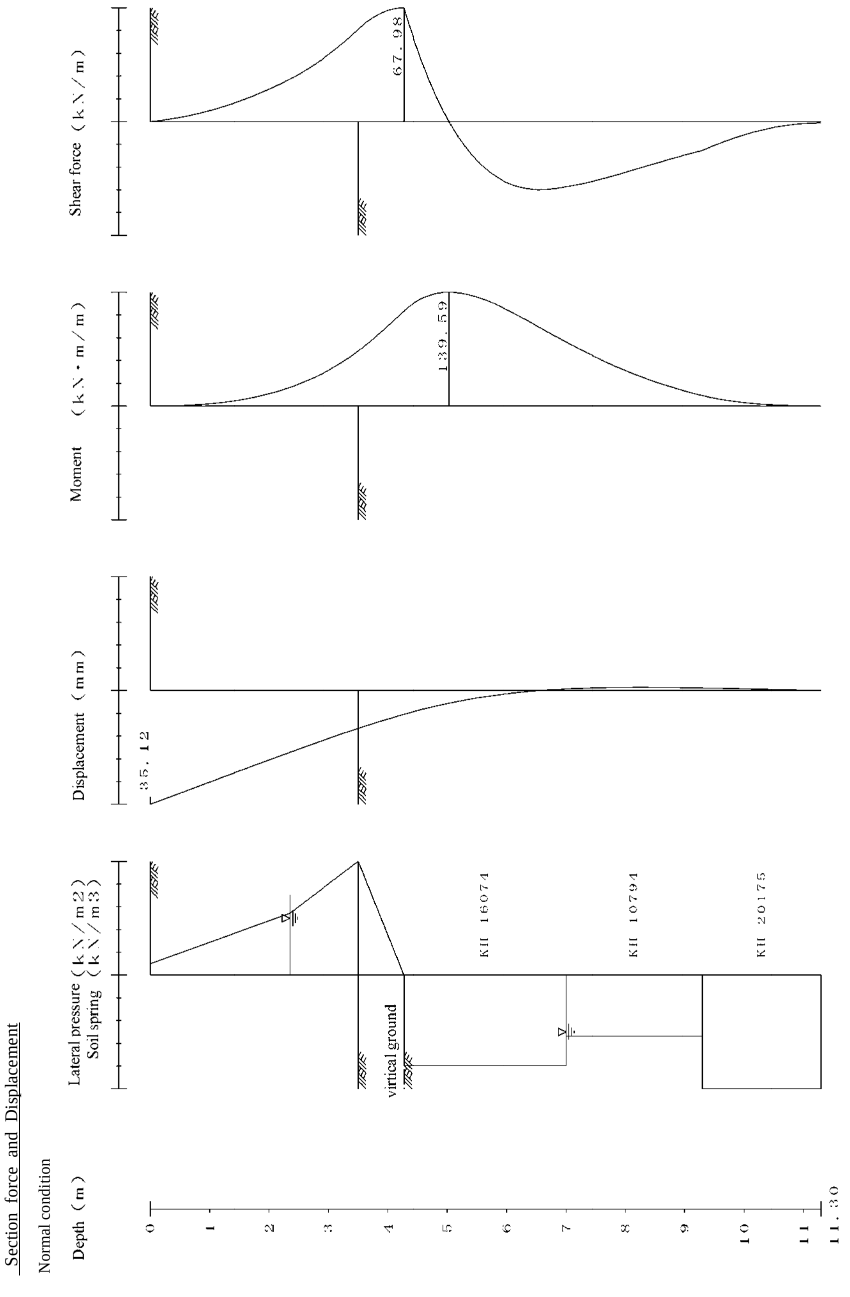
#### 5-1-1 Normal condition

Maximum bending Moment $M_{max}$

$M_{max}(+)$ ＝ 0.00 kN･m/m
$M_{max}(-)$ ＝ −139.59 kN･m/m

Horizontal displacement at the ground ＝ 35.12 ≦ 50.00 mm

| Depth (m) | Distance from top(m) | Bending moment (kN･m/m) | Shearing force (kN/m) | Horizontal displacement (mm) | Deflection angle (rad/10$^3$) | Vertical displacement (mm) |
|---|---|---|---|---|---|---|
| 0.00 | −0.30 | 0.00 | 0.00 | 35.12 | −6.8765 | 0.1069 |
| 0.30 | 0.00 | −0.18 | −1.29 | 33.06 | −6.8763 | 0.0998 |
| 0.80 | 0.50 | −1.61 | −4.69 | 29.62 | −6.8714 | 0.0880 |
| 1.30 | 1.00 | −5.14 | −9.69 | 26.19 | −6.8510 | 0.0762 |
| 1.80 | 1.50 | −11.56 | −16.26 | 22.78 | −6.7988 | 0.0645 |
| 2.30 | 2.00 | −21.66 | −24.42 | 19.40 | −6.6935 | 0.0530 |
| 2.35 | 2.05 | −22.90 | −25.32 | 19.07 | −6.6791 | 0.0519 |
| 2.80 | 2.50 | −36.34 | −34.87 | 16.10 | −6.5088 | 0.0420 |
| 3.30 | 3.00 | −57.08 | −48.64 | 12.91 | −6.2103 | 0.0316 |
| 3.50 | 3.20 | −67.45 | −55.09 | 11.69 | −6.0495 | 0.0278 |
| 3.80 | 3.50 | −85.29 | −63.19 | 9.91 | −5.7539 | 0.0225 |
| 4.27 | 3.97 | −116.37 | −67.98 | 7.35 | −5.1441 | 0.0151 |
| 4.30 | 4.00 | −118.47 | −64.27 | 7.19 | −5.0960 | 0.0147 |
| 4.80 | 4.50 | −137.79 | −16.17 | 4.85 | −4.2543 | 0.0087 |
| 5.03 | 4.73 | −139.59 | 0.02 | 3.92 | −3.8402 | 0.0067 |
| 5.30 | 5.00 | −137.49 | 14.85 | 2.95 | −3.3556 | 0.0049 |
| 5.80 | 5.50 | −125.19 | 32.38 | 1.49 | −2.5014 | 0.0024 |
| 6.30 | 6.00 | −106.79 | 39.81 | 0.43 | −1.7491 | 0.0011 |
| 6.80 | 6.50 | −86.55 | 40.18 | −0.28 | −1.1237 | 0.0005 |
| 7.00 | 6.70 | −78.63 | 38.92 | −0.49 | −0.9101 | 0.0003 |
| 7.30 | 7.00 | −67.23 | 36.95 | −0.72 | −0.6272 | 0.0002 |
| 7.80 | 7.50 | −49.85 | 32.41 | −0.93 | −0.2498 | 0.0002 |
| 8.30 | 8.00 | −34.94 | 27.18 | −0.98 | 0.0230 | 0.0002 |
| 8.80 | 8.50 | −22.66 | 21.99 | −0.92 | 0.2079 | 0.0002 |
| 9.30 | 9.00 | −12.86 | 17.34 | −0.79 | 0.3215 | 0.0002 |
| 9.80 | 9.50 | −6.03 | 10.25 | −0.61 | 0.3807 | 0.0001 |
| 10.30 | 10.00 | −2.28 | 5.07 | −0.41 | 0.4062 | 0.0001 |
| 10.80 | 10.50 | −0.62 | 1.94 | −0.21 | 0.4147 | 0.0000 |
| 11.30 | 11.00 | 0.00 | 0.89 | 0.00 | 0.4164 | 0.0000 |






5-1-2 Seismic condition

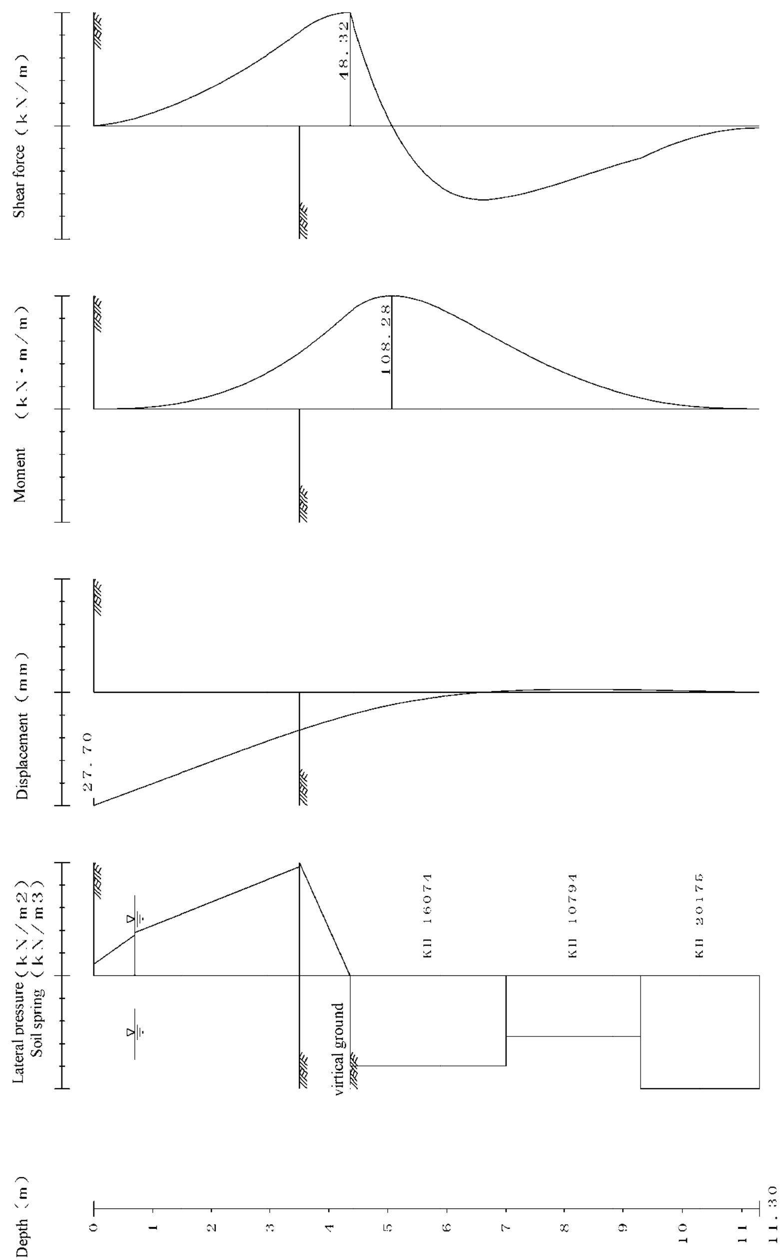
Maximum bending Moment$M_{max}$

$M_{max}(+)$ = 0.00 kN・m/m

$M_{max}(-)$ = -108.28 kN・m/m

Horizontal displacement at the ground = 27.70 ≦ 75.00 mm

| Depth (m) | Distance from top(m) | Bending moment (kN・m/m) | Shearing force (kN/m) | Horizontal displacement (mm) | Deflection angle (rad/$10^3$) | Vertical displacement (mm) |
|---|---|---|---|---|---|---|
| 0.00 | -0.30 | 0.00 | 0.00 | 27.70 | -5.4175 | 0.0662 |
| 0.30 | 0.00 | -0.12 | -0.90 | 26.07 | -5.4173 | 0.0618 |
| 0.70 | 0.40 | -0.89 | -3.12 | 23.90 | -5.4151 | 0.0559 |
| 0.80 | 0.50 | -1.24 | -3.89 | 23.36 | -5.4138 | 0.0544 |
| 1.30 | 1.00 | -4.25 | -8.32 | 20.66 | -5.3972 | 0.0471 |
| 1.80 | 1.50 | -9.73 | -13.77 | 17.97 | -5.3535 | 0.0399 |
| 2.30 | 2.00 | -18.19 | -20.24 | 15.31 | -5.2650 | 0.0328 |
| 2.80 | 2.50 | -30.14 | -27.73 | 12.72 | -5.1107 | 0.0259 |
| 3.30 | 3.00 | -46.09 | -36.23 | 10.22 | -4.8664 | 0.0195 |
| 3.50 | 3.20 | 53.70 | 39.92 | 9.26 | 4.7375 | 0.0172 |
| 3.80 | 3.50 | -66.46 | -44.77 | 7.87 | -4.5048 | 0.0139 |
| 4.30 | 4.00 | -89.96 | -48.28 | 5.74 | -3.9999 | 0.0091 |
| 4.36 | 4.06 | -92.74 | -48.32 | 5.51 | -3.9319 | 0.0087 |
| 4.80 | 4.50 | -106.33 | -15.04 | 3.89 | -3.3551 | 0.0055 |
| 5.07 | 4.77 | -108.28 | 0.04 | 3.03 | -2.9772 | 0.0040 |
| 5.30 | 5.00 | -107.09 | 9.98 | 2.39 | -2.6581 | 0.0031 |
| 5.80 | 5.50 | -98.13 | 24.30 | 1.23 | -1.9906 | 0.0015 |
| 6.30 | 6.00 | -84.13 | 30.60 | 0.39 | -1.3994 | 0.0007 |
| 6.80 | 6.50 | -68.47 | 31.24 | -0.19 | -0.9057 | 0.0003 |
| 7.00 | 6.70 | -62.30 | 30.37 | -0.35 | -0.7366 | 0.0002 |
| 7.30 | 7.00 | -53.40 | 28.92 | -0.54 | -0.5122 | 0.0002 |
| 7.80 | 7.50 | -39.75 | 25.48 | -0.71 | -0.2119 | 0.0001 |
| 8.30 | 8.00 | -28.01 | 21.45 | -0.76 | 0.0062 | 0.0001 |
| 8.80 | 8.50 | -18.30 | 17.43 | -0.72 | 0.1549 | 0.0001 |
| 9.30 | 9.00 | -10.52 | 13.80 | -0.62 | 0.2471 | 0.0001 |
| 9.80 | 9.50 | -5.06 | 8.26 | -0.48 | 0.2960 | 0.0001 |
| 10.30 | 10.00 | -2.01 | 4.20 | -0.32 | 0.3178 | 0.0001 |
| 10.80 | 10.50 | -0.59 | 1.74 | -0.16 | 0.3255 | 0.0000 |
| 11.30 | 11.00 | 0.00 | 0.91 | 0.00 | 0.3272 | 0.0000 |




Section force and Displacement

Seismic condition

5-2 Calculation of Stress intensity

Material PU18
Corrosion allowance $t_1$=0.00 mm (Front side) $t_2$=0.00 mm (Back side)
Corrosion ratio $\eta$ =1.00

$$\sigma = \frac{M_{max}}{Z} = \frac{139.59\times10^6}{1800\times10^3} = 78\ N/mm^2 \leqq \sigma_a = 165\ N/mm^2$$

$$\sigma = \frac{M_{max}}{Z} = \frac{139.59\times10^6}{1800\times10^3} = 78\ N/mm^2 \leqq \sigma_a = 165\ N/mm^2$$

5-2-2 Seismic condition

$$\sigma = \frac{M_{max}}{Z} = \frac{108.28\times10^6}{1800\times10^3} = 60\ N/mm^2 \leqq \sigma_a = 190\ N/mm^2$$

6 Calculation results

Steel sheet pile PU18

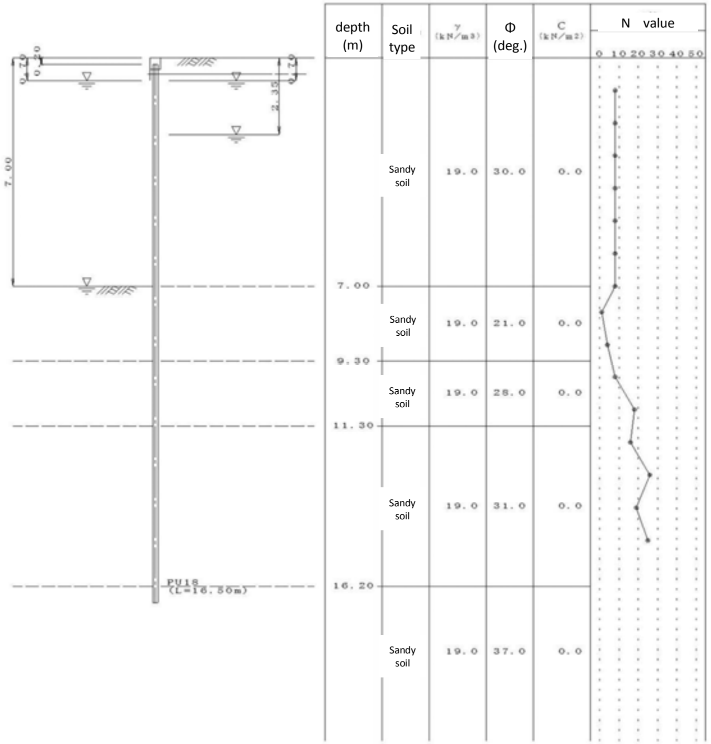
| | | | Normal condition | Seismic condition |
|---|---|---|---|---|
| Second moment of area | I (cm$^4$) | 38650 | | |
| Section modulus | Z (cm$^3$) | 1800 | | |
| Max.bending moment | Mmax(kN・m/m) | | 139.59 | 108.28 |
| | $\sigma$ (N/mm$^2$) | | 78 ( 165) | 60 (190) |
| Horizontal displacement | $\delta$ (mm) | | 35.12 ( 50.0) | 27.70 ( 75.0) |
| Footing depth | D (m) | | 7.47 | 7.55 |
| Total length of sheet pile | L (m) | 11.00 | | |

A-8-3 Structural calculation of steel sheet pile works (Type BR-1)

<Bracing style sheet pile wall>

1 Design conditions

1-1 Structural figure




1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design height | H= | 7.00 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.20 | m |
| Inside water level | Lwa= | 2.35 | m (Regular condition) |
| | Lwa'= | 0.70 | m (Seismic condition) |
| Outside water level | Lwp= | 7.00 | m (Regular condition) |
| | Lwp'= | 0.70 | m (Seismic condition) |
| Tie-rod installation position | Ht= | 0.50 | m |
| Tie-rod horizontal intervals | l= | 2.40 | m |
| Tie-rod installation angle | θ = | 0.0000 | degree |

Method of calculation; Free earth support method

Virtual supporting point; Design front ground level

## 1-3 Design constant parameters

Unit weight of water γw＝ 10.0 kN/m$^3$

Type of water pressure; trapezoid

Design Seismic coefficient (in the air) k= 0.080

(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma \gamma_t \cdot h_i + \Sigma \gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma \gamma_t \cdot h_i + \Sigma (\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake; Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma \gamma h + 2Q}{2C} \cdot \tan\theta}$$

Safety factor of footing depth; 1.50 （Regular condition）
1.20 （Seismic condition）

## 1-4 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N’: Average N-value

Result of boring survey

| No | Depth m | N-value |
|---|---|---|
| 1 | 1.00 | 8 |
| 2 | 2.00 | 8 |
| 3 | 3.00 | 8 |
| 4 | 4.00 | 8 |
| 5 | 5.00 | 8 |
| 6 | 6.00 | 8 |
| 7 | 7.00 | 8 |
| 8 | 7.80 | 1 |
| 9 | 8.80 | 4 |
| 10 | 9.80 | 8 |

| No | Depth m | N-value |
|---|---|---|
| 11 | 10.80 | 18 |
| 12 | 11.80 | 16 |
| 13 | 12.80 | 26 |
| 14 | 13.80 | 19 |
| 15 | 14.80 | 25 |

## 1-5 Overburden pressure

Active pressure side Qa= 10.0 kN/m$^2$ (Regular condition)
Qa'= 5.0 kN/m$^2$ (Seismic condition)

Passive pressure side Qp= 0.0 kN/m$^2$ (Regular condition)
Qp'= 0.0 kN/m$^2$ (Seismic condition)

## 1-6 Soil constant

| No | Depth m | type | N-value | γ kN/m$^3$ | γ' kN/m$^3$ | φ Deg. | C kN/m$^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 7.00 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 9.30 | S | 3 | 19.0 | 10.0 | 21.0 | 0.0 | 0.0 |
| 3 | 11.30 | S | 14 | 19.0 | 10.0 | 28.0 | 0.0 | 0.0 |
| 4 | 16.20 | S | 22 | 19.0 | 10.0 | 31.0 | 0.0 | 0.0 |
| 5 | 20.20 | S | 42 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

## 1-7 Sheet pile

Type PU18 (S270GP)
Young modulus E＝ 200,000 N/mm$^2$
Second moment of area I0＝ 38,650 cm$^4$（before corrosion）
Modulus of section Z0＝ 1,800 cm$^3$（before corrosion）

Corrosion allowance t1＝ 0.00 mm (front side) t2＝ 0.00 mm (back side)

Allowable stress σa = 165 N/mm$^2$（Regular condition）
σa' = 190 N/mm$^2$（Seismic condition）

## 1-8 Tie-rod

Diameter of tie-rod calculated

Corrosion considered（0.00mm）

Allowable stress σa = 176 N/mm$^2$（Regular condition）
σa' = 264 N/mm$^2$（Seismic condition）

## 1-9 Waling

Type 2×UPN-200×75

Modulus of section; 382 cm3

Corrosion ; considered（0.00mm）
Corrosion ratio; η＝ 1.00

Allowable stress σa = 140 N/mm$^2$（Regular condition）
σa' = 161 N/mm$^2$（Seismic condition）

1-10 Bracing sheet pile

Calculation method Chang's formula

Crest elevation of bracing works 0.30 m (distance from tie-rod)

| | | | |
|---|---|---|---|
| Materials | PU12 (S270GP) | | |
| Young's modulus | E = | 200,000 | N/mm$^2$ |
| Second moment of area | I0= | 21,600 | cm$^4$ (before corrosion) |
| Modulus of section | Z0= | 1,200 | cm$^3$ (before corrosion) |

| | | | | | | |
|---|---|---|---|---|---|---|
| Corrosion allowance | t1= | 0.00 | mm (front side) | t2= | 0.00 | mm (back side) |
| Corrosion ratio (againstI0) | η= | 1.00 | | | | |
| Corrosion ratio (against Z0) | η= | 1.00 | | | | |
| Joint efficiency (against I0) | μ= | 1.00 | | | | |
| Joint efficiency(againstZ0) | μ= | 1.00 | | | | |

| | | | |
|---|---|---|---|
| Allowable stress | σa = | 165 | N/mm$^2$ (Regular condition) |
| | σa' = | 190 | N/mm$^2$ (Seismic condition) |
| Allowable displacement | δa = | 50.0 | mm (Regular condition) |
| | δa'= | 75.0 | mm (Seismic condition) |



2 Calculation results

Front sheet pile PU18 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | (cm$^4$) | 38650 | | |
| Modulus of section | Z | (cm$^3$) | 1800 | | |
| Max. bending moment | Mmax | (kN·m/m) | | 204.59 | 108.64 |
| Stress intensity | σ | (N/mm$^2$) | | 114 (165) | 61 (190) |
| Footing depth | D | (m) | | 9.38 | 5.52 |
| Total length | L | (m) | 16.5 | | |

Tie-rod φ 42 mm

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Tensile force | Tp | (kN) | | 207.15 | 127.65 |
| Stress intensity | σ | (N/mm$^2$) | | 150 (176) | 92 (264) |

Waling 2×UPN-200×75

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Modulus of section | Z | (cm3) | 382 | | |
| Max. bending moment | Mmax | (kN·m) | | 49.72 | 30.63 |
| Stress intensity | σ | (N/mm$^2$) | | 130 (140) | 80 (161) |

Bracing sheet pile PU12 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | (cm$^4$) | 21,600 | | |
| Modulus of section | Z | (cm$^3$) | 1,200 | | |
| Max. bending moment | Mmax | (kN·m) | | 50.39 | 31.05 |
| Stress intensity | σ | (N/mm$^2$) | | 42 (165) | 26 (190) |
| Horizontal displacement | δ | (mm) | | 5.93 (50.0) | 3.67 (75.0) |
| Footing depth | D | (m) | | 5.43 | 5.43 |
| Total length | L | (m) | 6.00 | | |

Bracing works Installation position

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Bracing works Installation position | d | (m) | | 7.04 | 8.35 |

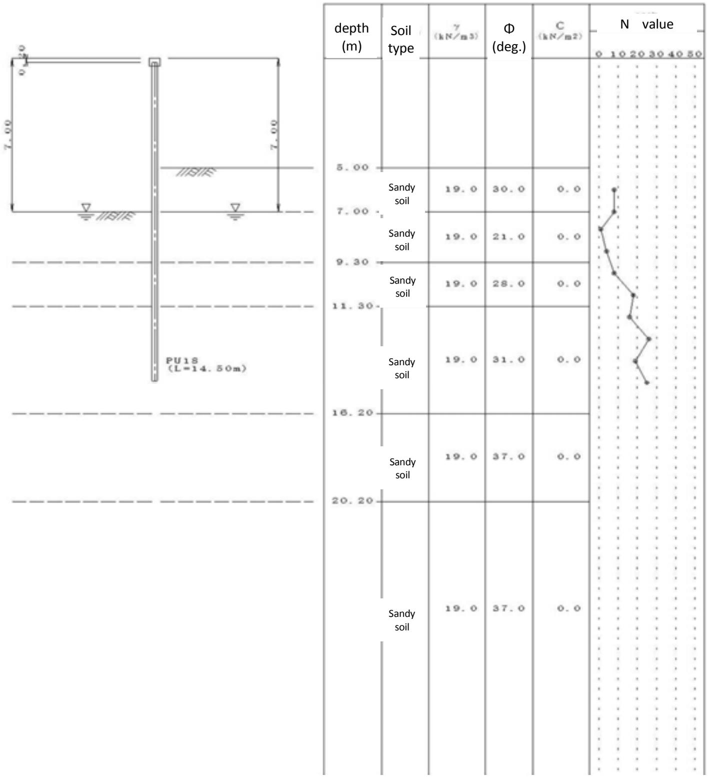
Structural calculation of steel sheet pile works (Type BR-1)

— **During construction** —

<Cantilevered sheet pile wall>

1 Design conditions

1-1 Structural figure




1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 7.00 | m |
| Protruction lemgth | H0= | 5.00 | |
| Top of sheet pile | Hlt= | 0.20 | m |
| Inside water level | Lwa= | 7.00 | m (Regular condition) |
| Outside water level | Lwp= | 7.00 | m (Regular condition) |

1-3 Calculation method

Calculation method    Chang's formula

Footing depth    $L = \frac{\pi}{\beta}$

1-4 Design constant parameters

Unit weight of water    $\gamma w =$    10.0 kN/m$^3$

Type of water pressure;    trapezoid

1-5 Coefficient of horizontal subgrade reaction

Formula:    $K_h = 6910 \times N'^{0.406}$

where,  N': Average N-value

1-6 Overburden pressure

Active pressure side    Qa=    10.0 kN/m$^2$ (Regular condition)

Passive pressure side    Qp=    0.0 kN/m$^2$ (Regular condition)

1-7 Soil constant

| No. | Depth m | Layer thickness m |
|---|---|---|
| 1 | 7.00 | 2.00 |
| 2 | 9.30 | 2.30 |
| 3 | 11.30 | 2.00 |
| 4 | 16.20 | 4.90 |
| 5 | 20.20 | 4.00 |
| 6 | 30.00 | 9.80 |

| No | Depth m | type | N-value | γ kN/m$^3$ | γ' kN/m$^3$ | φ Deg. | C kN/m$^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 7.00 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 9.30 | S | 3 | 19.0 | 10.0 | 21.0 | 0.0 | 0.0 |
| 3 | 11.30 | S | 14 | 19.0 | 10.0 | 28.0 | 0.0 | 0.0 |
| 4 | 16.20 | S | 22 | 19.0 | 10.0 | 31.0 | 0.0 | 0.0 |
| 5 | 20.20 | S | 42 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |
| 6 | 30.00 | S | 41 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile    C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)  a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle



1-8 Sheet pile

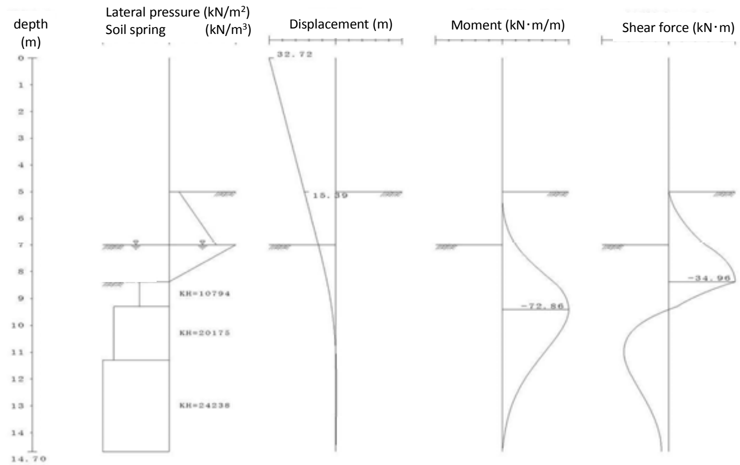
| | | | |
|---|---|---|---|
| Type | PU18 (S270GP) | | |
| Young modulus | E ＝ | 200,000 | N/mm$^2$ |
| Second moment of area | I0＝ | 38650 | cm$^4$（before corrosion） |
| Modulus of section | Z0＝ | 1800 | cm$^3$（before corrosion） |

Corrosion allowance    t1＝    0.00 mm (front side)    t2＝    0.00 mm (back side)
Corrosion ratio (againstI0)    η＝    1.00
Corrosion ratio (against Z0)    η＝    1.00
Joint efficiency (against I0)    μ＝    1.00
Joint efficiency(againstZ0)    μ＝    1.00

Allowable stress    σa =    165 N/mm$^2$（Regular condition）

Allowable displacement    δa ＝    50.0 mm（Regular condition）

2 Section force and Displacement




3 Calculation results

Front sheet pile    PU18 (S270GP)

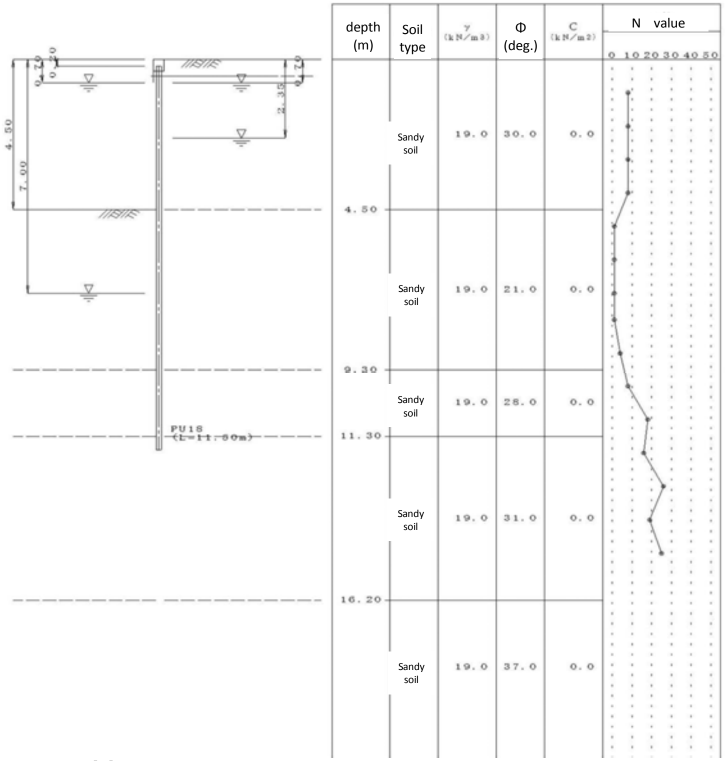
| | | | Regular condition | |
|---|---|---|---|---|
| Second moment of area | I (cm$^4$) | 38650 | | |
| Modulus of section | Z (cm$^3$) | 1800 | | |
| Max. bending moment | Mmax (kN･m/m) | | 72.86 | |
| Stress intensity | σ (N/mm$^2$) | | 40 (165) | |
| Horizontal displacement | δ (mm) | | 15.39 (50.0) | |
| Footing depth | D (m) | | 7.57 | |
| Total length | L (m) | 14.50 | | |

A-8-5 Structural calculation of steel sheet pile works (Type BR-2)

<Bracing style sheet pile wall>

## 1 Design conditions

### 1-1 Structural figure




### 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design height | H= | 4.50 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.20 | m |
| Inside water level | Lwa= | 2.35 | m (Regular condition) |
| | Lwa'= | 0.70 | m (Seismic condition) |
| Outside water level | Lwp= | 7.00 | m (Regular condition) |
| | Lwp'= | 0.70 | m (Seismic condition) |
| | | | |
| Tie-rod installation position | Ht= | 0.50 | m |
| Tie-rod horizontal intervals | l= | 2.40 | m |
| Tie-rod installation angle | θ = | 0.0000 | degree |

Method of calculation; Free earth support method

Virtual supporting point; Design front ground level

1-3 <u>Design constant parameters</u>

Unit weight of water γw＝ 10.0 kN/m$^3$

Type of water pressure; trapezoid

Design Seismic coefficient (in the air) k= 0.080

(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma \gamma_t \cdot h_i + \Sigma \gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma \gamma_t \cdot h_i + \Sigma (\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake; Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma \gamma h + 2Q}{2C} \cdot \tan\theta}$$

Safety factor of footing depth; 1.50 （Regular condition）

1.20 （Seismic condition）

1-4 <u>Coefficient of horizontal subgrade reaction</u>

Formula: $K_h = 6910 \times N'^{0.406}$

where, N’: Average N-value

Result of boring survey

| No | Depth m | N-value |
|---|---|---|
| 1 | 1.00 | 8 |
| 2 | 2.00 | 8 |
| 3 | 3.00 | 8 |
| 4 | 4.00 | 8 |
| 5 | 5.00 | 8 |
| 6 | 6.00 | 8 |
| 7 | 7.00 | 8 |
| 8 | 7.80 | 1 |
| 9 | 8.80 | 4 |
| 10 | 9.80 | 8 |

| No | Depth m | N-value |
|---|---|---|
| 11 | 10.80 | 18 |
| 12 | 11.80 | 16 |
| 13 | 12.80 | 26 |
| 14 | 13.80 | 19 |
| 15 | 14.80 | 25 |

1-5 <u>Overburden pressure</u>

Active pressure side Qa= 10.0 kN/m$^2$ (Regular condition)

Qa'= 5.0 kN/m$^2$ (Seismic condition)

Passive pressure side Qp= 0.0 kN/m$^2$ (Regular condition)

Qp'= 0.0 kN/m$^2$ (Seismic condition)

1-6 <u>Soil constant</u>

| No | Depth m | type | N-value | γ kN/m$^3$ | γ' kN/m$^3$ | φ Deg. | C kN/m$^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 7.00 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 9.30 | S | 3 | 19.0 | 10.0 | 21.0 | 0.0 | 0.0 |
| 3 | 11.30 | S | 14 | 19.0 | 10.0 | 28.0 | 0.0 | 0.0 |
| 4 | 16.20 | S | 22 | 19.0 | 10.0 | 31.0 | 0.0 | 0.0 |
| 5 | 20.20 | S | 42 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force

type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M) a ：Angle of adhesive force

Ｎvalue：average value

γ ：unit weight of humid soil

γ'：unit weight of soil under the water

φ ：internal friction angle

1-7 <u>Sheet pile</u>

Type PU18 (S270GP)

Young modulus E ＝ 200,000 N/mm$^2$

Second moment of area I0＝ 38,650 cm$^4$（before corrosion）

Modulus of section Z0＝ 1,800 cm$^3$（before corrosion）

Corrosion allowance t1＝ 0.00 mm (front side) t2＝ 0.00 mm (back side)

Allowable stress σa = 165 N/mm$^2$（Regular condition）

σa' = 190 N/mm$^2$（Seismic condition）

1-8 <u>Tie-rod</u>

Diameter of tie-rod calculated

Corrosion considered（0.00mm）

Allowable stress σa = 176 N/mm$^2$（Regular condition）

σa' = 264 N/mm$^2$（Seismic condition）

1-9 <u>Waling</u>

Type 2×UPN-180×70

Modulus of section; 300 cm3

Corrosion ; considered（0.00mm）

Corrosion ratio; η ＝ 1.00

Allowable stress σa = 140 N/mm$^2$（Regular condition）

σa' = 161 N/mm$^2$（Seismic condition）

## 1-10 Bracing sheet pile

Calculation method    Chang's formula

Crest elevation of bracing works    0.30 m (distance from tie-rod)

| | | | |
|---|---|---|---|
| Materials | PU12 (S270GP) | | |
| Young's modulus | E = | 200,000 | $N/mm^2$ |
| Second moment of area | I0= | 21,600 | $cm^4$ (before corrosion) |
| Modulus of section | Z0= | 1,200 | $cm^3$ (before corrosion) |

| | | | | | | |
|---|---|---|---|---|---|---|
| Corrosion allowance | t1= | 0.00 | mm (front side) | t2= | 0.00 | mm (back side) |
| Corrosion ratio (againstI0) | η= | 1.00 | | | | |
| Corrosion ratio (against Z0) | η= | 1.00 | | | | |
| Joint efficiency (against I0) | μ= | 1.00 | | | | |
| Joint efficiency(againstZ0) | μ= | 1.00 | | | | |

| | | | |
|---|---|---|---|
| Allowable stress | σa = | 165 | $N/mm^2$ (Regular condition) |
| | σa' = | 190 | $N/mm^2$ (Seismic condition) |
| Allowable displacement | δa = | 50.0 | mm (Regular condition) |
| | δa'= | 75.0 | mm (Seismic condition) |



## 2 Calculation results

Front sheet pile    PU18 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 38650 | | |
| Modulus of section | Z | ($cm^3$) | 1800 | | |
| Max. bending moment | Mmax | (kN・m/m) | | 45.04 | 29.92 |
| Stress intensity | σ | ($N/mm^2$) | | 25 (165) | 17 (190) |
| Footing depth | D | (m) | | 6.80 | 5.49 |
| Total length | L | (m) | 11.5 | | |

Tie-rod    φ 25 mm

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Tensile force | Tp | (kN) | | 84.92 | 62.82 |
| Stress intensity | σ | ($N/mm^2$) | | 173 (176) | 128 (264) |

Waling    2×UPN-180×70

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Modulus of section | Z | (cm3) | 300 | | |
| Max. bending moment | Mmax | (kN・m) | | 20.38 | 15.08 |
| Stress intensity | σ | ($N/mm^2$) | | 68 (140) | 50 (161) |

Bracing sheet pile    PU12 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 21,600 | | |
| Modulus of section | Z | ($cm^3$) | 1,200 | | |
| Max. bending moment | Mmax | (kN・m) | | 20.66 | 15.28 |
| Stress intensity | σ | ($N/mm^2$) | | 17 (165) | 13 (190) |
| Horizontal displacement | δ | (mm) | | 2.43 (50.0) | 1.8 (75.0) |
| Footing depth | D | (m) | | 5.43 | 5.43 |
| Total length | L | (m) | 6.00 | | |

Bracing works Installation position

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Bracing works Installation position | d | (m) | | 5.59 | 6.43 |

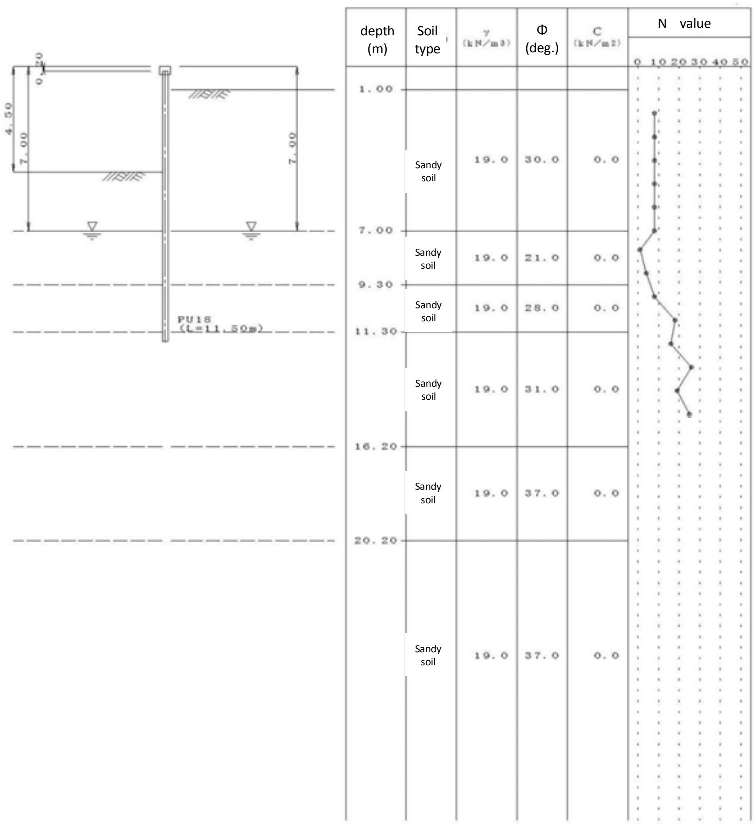
Structural calculation of steel sheet pile works (Type BR-2)

**― During construction ―**

<Cantilevered sheet pile wall>

# 1 Design conditions

## 1-1 Structural figure




## 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 4.50 | m |
| Protruction lemgth | H0= | 1.00 | |
| Top of sheet pile | Hlt= | 0.20 | m |
| Inside water level | Lwa= | 7.00 | m (Regular condition) |
| Outside water level | Lwp= | 7.00 | m (Regular condition) |

1-3 Calculation method

Calculation method    Chang's formula

Footing depth    $L = \frac{\pi}{\beta}$

1-4 Design constant parameters

Unit weight of water    $\gamma w =$    10.0 $kN/m^3$

Type of water pressure;    trapezoid

1-5 Coefficient of horizontal subgrade reaction

Formula:    $K_h = 6910 \times N'^{0.406}$

where,    N': Average N-value

1-6 Overburden pressure

Active pressure side    Qa=    10.0 $kN/m^2$ (Regular condition)

Passive pressure side    Qp=    0.0 $kN/m^2$ (Regular condition)

1-7 Soil constant

| No. | Depth<br>m | Layer thickness<br>m |
|---|---|---|
| 1 | 7.00 | 2.00 |
| 2 | 9.30 | 2.30 |
| 3 | 11.30 | 2.00 |
| 4 | 16.20 | 4.90 |
| 5 | 20.20 | 4.00 |
| 6 | 30.00 | 9.80 |

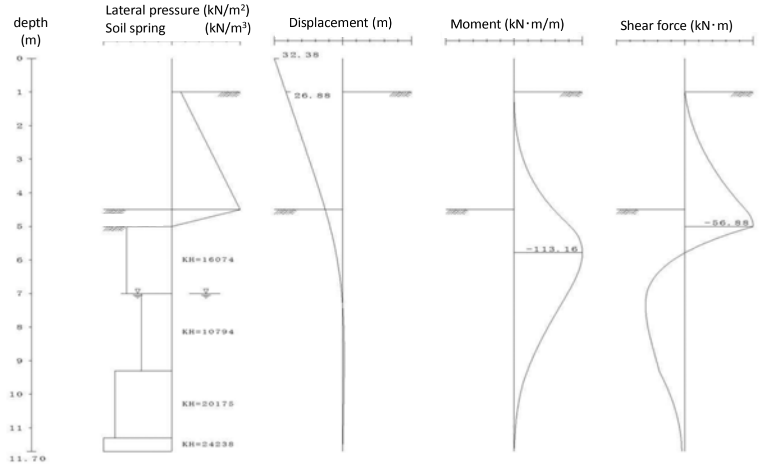
| No | Depth<br>m | type | N-value | γ<br>$kN/m^3$ | γ'<br>$kN/m^3$ | φ<br>Deg. | C<br>$kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 7.00 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 9.30 | S | 3 | 19.0 | 10.0 | 21.0 | 0.0 | 0.0 |
| 3 | 11.30 | S | 14 | 19.0 | 10.0 | 28.0 | 0.0 | 0.0 |
| 4 | 16.20 | S | 22 | 19.0 | 10.0 | 31.0 | 0.0 | 0.0 |
| 5 | 20.20 | S | 42 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |
| 6 | 30.00 | S | 41 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile    C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)  a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

1-8 Sheet pile

| | | | | |
|---|---|---|---|---|
| Type | PU18 (S270GP) | | | |
| Young modulus | E ＝ | 200,000 | $N/mm^2$ | |
| Second moment of area | I0＝ | 38650 | $cm^4$（before corrosion） | |
| Modulus of section | Z0＝ | 1800 | $cm^3$（before corrosion） | |
| Corrosion allowance | t1＝ | 0.00 | mm (front side) | t2＝ 0.00 mm (back side) |
| Corrosion ratio (againstI0) | η＝ | 1.00 | | |
| Corrosion ratio (against Z0) | η＝ | 1.00 | | |
| Joint efficiency (against I0) | μ＝ | 1.00 | | |
| Joint efficiency(againstZ0) | μ＝ | 1.00 | | |
| Allowable stress | σa = | 165 | $N/mm^2$（Regular condition） | |
| Allowable displacement | δa ＝ | 50.0 | mm（Regular condition） | |

2 Section force and Displacement




3 Calculation results

Front sheet pile    PU18 (S270GP)

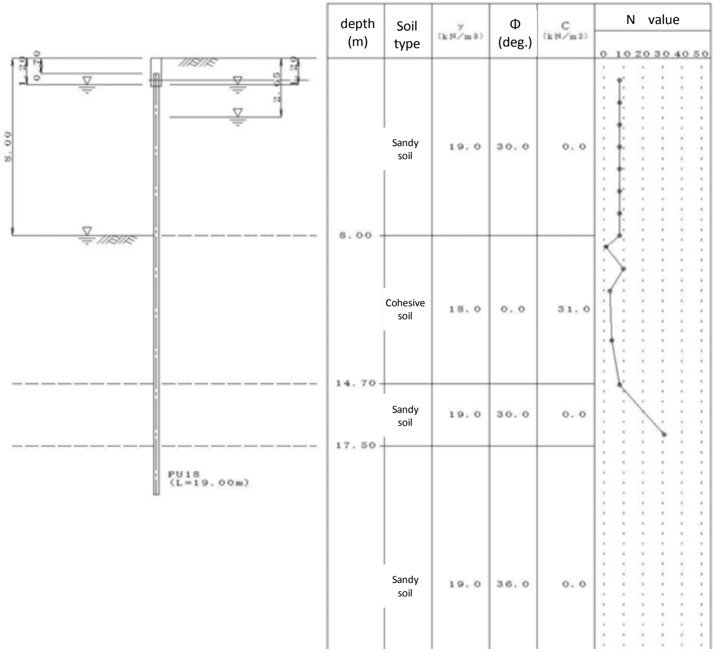
| | | | | Regular condition | |
|---|---|---|---|---|---|
| Second moment of area | I | $(cm^4)$ | 38650 | | |
| Modulus of section | Z | $(cm^3)$ | 1800 | | |
| Max. bending moment | Mmax | (kN･m/m) | | 113.16 | |
| Stress intensity | σ | $(N/mm^2)$ | | 63 (165) | |
| Horizontal displacement | δ | (mm) | | 26.88 (50.0) | |
| Footing depth | D | (m) | | 7.15 | |
| Total length | L | (m) | 11.50 | | |

A-8-6 Structural calculation of steel sheet pile works (Type C-1)

<Bracing style sheet pile wall>

## 1 Design conditions

### 1-1 Structural figure




### 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design height | H= | 8.00 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 2.65 | m (Regular condition) |
| | Lwa'= | 1.20 | m (Seismic condition) |
| Outside water level | Lwp= | 8.00 | m (Regular condition) |
| | Lwp'= | 1.20 | m (Seismic condition) |
| Tie-rod installation position | Ht= | 1.00 | m |
| Tie-rod horizontal intervals | l= | 2.40 | m |
| Tie-rod installation angle | θ = | 0.0000 | degree |

Method of calculation; Free earth support method

Virtual supporting point; Design front ground level

## 1-3 Design constant parameters

Unit weight of water　γw＝ 10.0 $kN/m^3$

Type of water pressure;　trapezoid

Design Seismic coefficient (in the air)　k= 0.080

(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma\gamma_t \cdot h_i + \Sigma\gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma\gamma_t \cdot h_i + \Sigma(\gamma-\gamma_w) \cdot h_j + Q) + (\gamma-\gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake;　Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h+2Q}{2C} \cdot \tan\theta}$$

Safety factor of footing depth;　1.50 （Regular condition）

1.20 （Seismic condition）

## 1-4 Coefficient of horizontal subgrade reaction

Formula:　$K_h = 6910 \times N'^{0.406}$

where,　N': Average N-value

Result of boring survey

| No | Depth m | N-value |
|---|---|---|
| 1 | 1.00 | 8 |
| 2 | 2.00 | 8 |
| 3 | 3.00 | 8 |
| 4 | 4.00 | 8 |
| 5 | 5.00 | 8 |
| 6 | 6.00 | 8 |
| 7 | 7.00 | 8 |
| 8 | 8.00 | 8 |
| 9 | 8.50 | 1 |
| 10 | 9.50 | 10 |

| No | Depth m | N-value |
|---|---|---|
| 11 | 10.50 | 3 |
| 12 | 12.75 | 4 |
| 13 | 14.75 | 8 |
| 14 | 17.00 | 31 |

## 1-5 Overburden pressure

Active pressure side　Qa= 10.0 $kN/m^2$ (Regular condition)

Qa'= 5.0 $kN/m^2$ (Seismic condition)

Passive pressure side　Qp= 0.0 $kN/m^2$ (Regular condition)

Qp'= 0.0 $kN/m^2$ (Seismic condition)

## 1-6 Soil constant

| No | Depth m | type | N-value | γ $kN/m^3$ | γ' $kN/m^3$ | φ Deg. | C $kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 8.00 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 14.70 | C | 5 | 18.0 | 9.0 | 0.0 | 31.0 | 0.0 |
| 3 | 17.50 | S | 20 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 4 | 30.00 | S | 37 | 19.0 | 10.0 | 36.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force

type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force

Ｎvalue：average value

γ ：unit weight of humid soil

γ'：unit weight of soil under the water

φ ：internal friction angle

## 1-7 Sheet pile

Type　PU18 (S270GP)

Young modulus　E ＝ 200,000 $N/mm^2$

Second moment of area　I0＝ 38,650 $cm^4$（before corrosion）

Modulus of section　Z0＝ 1,800 $cm^3$（before corrosion）

Corrosion allowance　t1＝ 0.00 mm (front side)　t2＝ 0.00 mm (back side)

Allowable stress　σa = 165 $N/mm^2$（Regular condition）

σa' = 190 $N/mm^2$（Seismic condition）

## 1-8 Tie-rod

Diameter of tie-rod　calculated

Corrosion　considered（0.00mm）

Allowable stress　σa = 176 $N/mm^2$（Regular condition）

σa' = 264 $N/mm^2$（Seismic condition）

## 1-9 Waling

Type　2×UPN-220×80

Modulus of section;　490 cm3

Corrosion ;　considered（0.00mm）

Corrosion ratio;　η ＝ 1.00

Allowable stress　σa = 140 $N/mm^2$（Regular condition）

σa' = 161 $N/mm^2$（Seismic condition）

## 1-10 Bracing sheet pile

Calculation method Chang's formula

| | | | |
|---|---|---|---|
| Crest elevation of bracing works | | 0.30 | m (distance from tie-rod) |
| Materials | PU12 (S270GP) | | |
| Young's modulus | E = | 200,000 | $N/mm^2$ |
| Second moment of area | I0= | 21,600 | $cm^4$ (before corrosion) |
| Modulus of section | Z0= | 1,200 | $cm^3$ (before corrosion) |
| Corrosion allowance | t1= | 0.00 | mm (front side) t2= 0.00 mm (back side) |
| Corrosion ratio (againstI0) | η= | 1.00 | |
| Corrosion ratio (against Z0) | η= | 1.00 | |
| Joint efficiency (against I0) | μ= | 1.00 | |
| Joint efficiency(againstZ0) | μ= | 1.00 | |
| Allowable stress | σa = | 165 | $N/mm^2$ (Regular condition) |
| | σa' = | 190 | $N/mm^2$ (Seismic condition) |
| Allowable displacement | δa = | 50.0 | mm (Regular condition) |
| | δa'= | 75.0 | mm (Seismic condition) |

## 2 Calculation results

Front sheet pile PU18 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 38650 | | |
| Modulus of section | Z | ($cm^3$) | 1800 | | |
| Max. bending moment | Mmax | (kN・m/m) | | 283.19 | 154.42 |
| Stress intensity | σ | ($N/mm^2$) | | 157 (165) | 86 (190) |
| Footing depth | D | (m) | | 11.58 | 7.92 |
| Total length | L | (m) | 19.00 | | |

Tie-rod φ 46 mm

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Tensile force | Tp | (kN) | | 283.12 | 184.21 |
| Stress intensity | σ | ($N/mm^2$) | | 170 (176) | 111 (264) |

Waling 2×UPN-220×80

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Modulus of section | Z | (cm3) | 490 | | |
| Max. bending moment | Mmax | (kN・m) | | 67.95 | 44.21 |
| Stress intensity | σ | ($N/mm^2$) | | 139 (140) | 90 (161) |

Bracing sheet pile PU12 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 21,600 | | |
| Modulus of section | Z | ($cm^3$) | 1,200 | | |
| Max. bending moment | Mmax | (kN・m) | | 68.87 | 44.81 |
| Stress intensity | σ | ($N/mm^2$) | | 57 (165) | 37 (190) |
| Horizontal displacement | δ | (mm) | | 8.11 (50.0) | 5.27 (75.0) |
| Footing depth | D | (m) | | 5.43 | 5.43 |
| Total length | L | (m) | 6.00 | | |

Bracing works Installation position

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Bracing works Installation position | d | (m) | | 7.33 | 8.63 |

Structural calculation of steel sheet pile works (Type C-1)

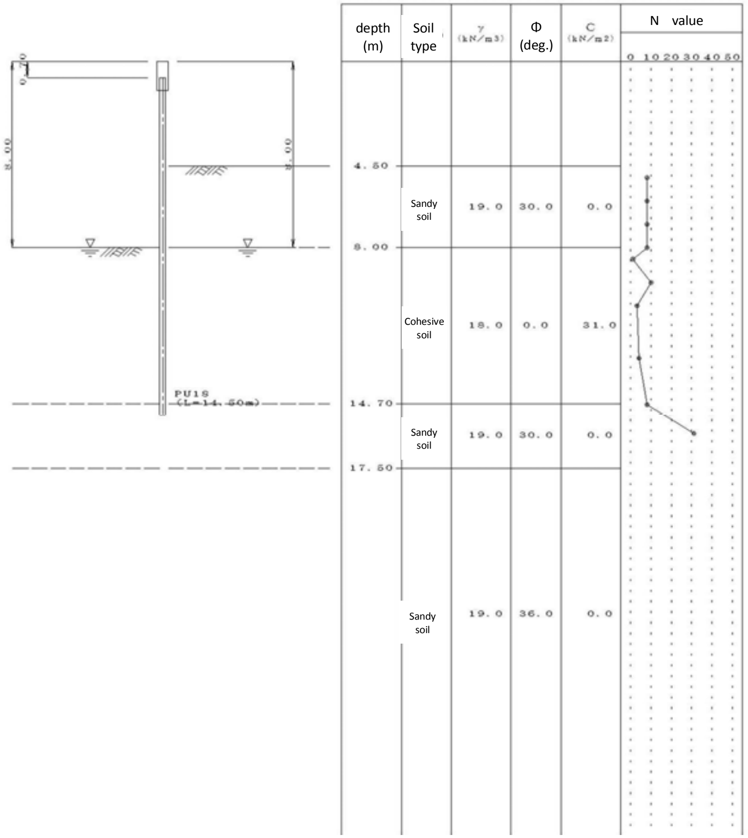
**－ During construction －**

<Cantilevered sheet pile wall>



1 Design conditions

1-1 Structural figure




1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 8.00 | m |
| Protruction lemgth | H0= | 4.50 | |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 8.00 | m (Regular condition) |
| Outside water level | Lwp= | 8.00 | m (Regular condition) |

1-3 Calculation method

Calculation method    Chang's formula

Footing depth    $L = \frac{\pi}{\beta}$

1-4 Design constant parameters

Unit weight of water    $\gamma w =$    10.0 $kN/m^3$

Type of water pressure;    trapezoid

1-5 Coefficient of horizontal subgrade reaction

Formula:    $K_h = 6910 \times N'^{0.406}$

where,    N': Average N-value

1-6 Overburden pressure

Active pressure side    Qa=    10.0 $kN/m^2$ (Regular condition)

Passive pressure side    Qp=    0.0 $kN/m^2$ (Regular condition)

1-7 Soil constant

| No. | Depth<br>m | Layer thickness<br>m |
|---|---|---|
| 1 | 8.00 | 3.50 |
| 2 | 14.70 | 6.70 |
| 3 | 17.50 | 2.80 |
| 4 | 30.00 | 12.50 |

| No | Depth<br>m | type | N-value | γ<br>$kN/m^3$ | γ'<br>$kN/m^3$ | φ<br>Deg. | C<br>$kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 8.00 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 14.70 | C | 5 | 18.0 | 9.0 | 0.0 | 31.0 | 0.0 |
| 3 | 17.50 | S | 20 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 4 | 30.00 | S | 37 | 19.0 | 10.0 | 36.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile    C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)  a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

1-8 Sheet pile

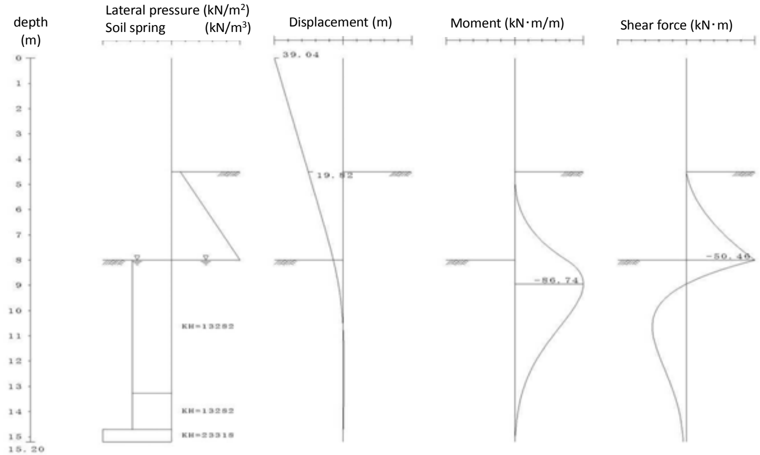
| | | | |
|---|---|---|---|
| Type | PU18 (S270GP) | | |
| Young modulus | E ＝ | 200,000 | $N/mm^2$ |
| Second moment of area | I0＝ | 38650 | $cm^4$（before corrosion） |
| Modulus of section | Z0＝ | 1800 | $cm^3$（before corrosion） |

Corrosion allowance    t1＝    0.00 mm (front side)    t2＝    0.00 mm (back side)
Corrosion ratio (againstI0)    η＝    1.00
Corrosion ratio (against Z0)    η＝    1.00
Joint efficiency (against I0)    μ＝    1.00
Joint efficiency(againstZ0)    μ＝    1.00

Allowable stress    σa =    165 $N/mm^2$（Regular condition）

Allowable displacement    δa ＝    50.0 mm（Regular condition）

2 Section force and Displacement




3 Calculation results

Front sheet pile    PU18 (S270GP)

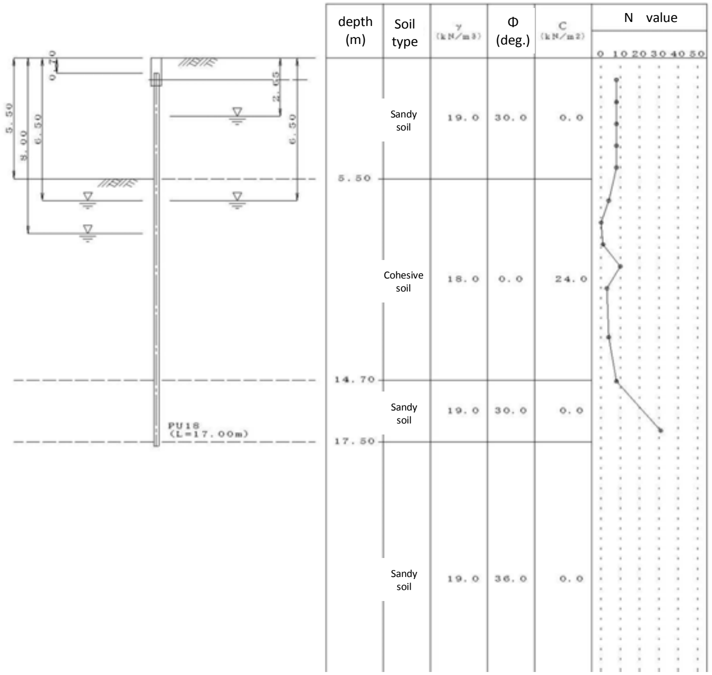
| | | | Regular condition | |
|---|---|---|---|---|
| Second moment of area | I ($cm^4$) | 38650 | | |
| Modulus of section | Z ($cm^3$) | 1800 | | |
| Max. bending moment | Mmax (kN･m/m) | | 86.74 | |
| Stress intensity | σ ($N/mm^2$) | | 48 (165) | |
| Horizontal displacement | δ (mm) | | 19.82 (50.0) | |
| Footing depth | D (m) | | 6.87 | |
| Total length | L (m) | 14.50 | | |

A-8-7 Structural calculation of steel sheet pile works (Type C-2)

<Bracing style sheet pile wall>

1 Design conditions

1-1 Structural figure




1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design height | H= | 5.50 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 2.65 | m (Regular condition) |
| | Lwa'= | 6.50 | m (Seismic condition) |
| Outside water level | Lwp= | 8.00 | m (Regular condition) |
| | Lwp'= | 6.50 | m (Seismic condition) |
| Tie-rod installation position | Ht= | 1.00 | m |
| Tie-rod horizontal intervals | l= | 2.40 | m |
| Tie-rod installation angle | θ = | 0.0000 | degree |

Method of calculation; Free earth support method

Virtual supporting point; Design front ground level

## 1-3 Design constant parameters

Unit weight of water  $\gamma w =$  10.0 $kN/m^3$

Type of water pressure;  trapezoid

Design Seismic coefficient (in the air)  k=  0.080

(under water)  Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma\gamma_t \cdot h_i + \Sigma\gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma\gamma_t \cdot h_i + \Sigma(\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake;  Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma\gamma h + 2Q}{2C} \cdot \tan\theta}$$

Safety factor of footing depth;  1.50 (Regular condition)

1.20 (Seismic condition)

## 1-4 Coefficient of horizontal subgrade reaction

Formula:  $K_h = 6910 \times N'^{0.406}$

where,  N': Average N-value

Result of boring survey

| No | Depth m | N-value |
|---|---|---|
| 1 | 1.00 | 8 |
| 2 | 2.00 | 8 |
| 3 | 3.00 | 8 |
| 4 | 4.00 | 8 |
| 5 | 5.00 | 8 |
| 6 | 6.50 | 4 |
| 7 | 7.50 | 0 |
| 8 | 8.50 | 1 |
| 9 | 9.50 | 10 |
| 10 | 10.50 | 3 |

| No | Depth m | N-value |
|---|---|---|
| 11 | 12.75 | 4 |
| 12 | 14.75 | 8 |
| 13 | 17.00 | 31 |

## 1-5 Overburden pressure

Active pressure side  Qa=  10.0 $kN/m^2$ (Regular condition)

Qa'=  5.0 $kN/m^2$ (Seismic condition)

Passive pressure side  Qp=  0.0 $kN/m^2$ (Regular condition)

Qp'=  0.0 $kN/m^2$ (Seismic condition)

## 1-6 Soil constant

| No | Depth m | type | N-value | γ $kN/m^3$ | γ' $kN/m^3$ | φ Deg. | C $kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 5.50 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 14.70 | C | 4 | 18.0 | 9.0 | 0.0 | 24.0 | 0.0 |
| 3 | 17.50 | S | 20 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 4 | 30.00 | S | 37 | 19.0 | 10.0 | 36.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force

type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force

Ｎvalue：average value

γ ：unit weight of humid soil

γ'：unit weight of soil under the water

φ ：internal friction angle

## 1-7 Sheet pile

Type  PU18 (S270GP)

Young modulus  E ＝  200,000 $N/mm^2$

Second moment of area  I0＝  38,650 $cm^4$ (before corrosion)

Modulus of section  Z0＝  1,800 $cm^3$ (before corrosion)

Corrosion allowance  t1＝  0.00 mm (front side)  t2＝  0.00 mm (back side)

Allowable stress  σa =  165 $N/mm^2$ (Regular condition)

σa' =  190 $N/mm^2$ (Seismic condition)

## 1-8 Tie-rod

Diameter of tie-rod  calculated

Corrosion  considered (0.00mm)

Allowable stress  σa =  176 $N/mm^2$ (Regular condition)

σa' =  264 $N/mm^2$ (Seismic condition)

## 1-9 Waling

Type  2×UPN-180×70

Modulus of section;  300 cm3

Corrosion ;  considered (0.00mm)

Corrosion ratio;  η ＝  1.00

Allowable stress  σa =  140 $N/mm^2$ (Regular condition)

σa' =  161 $N/mm^2$ (Seismic condition)

## 1-10 Bracing sheet pile

Calculation method    Chang's formula

Crest elevation of bracing works    0.30 m (distance from tie-rod)

| | | | |
|---|---|---|---|
| Materials | PU12 (S270GP) | | |
| Young's modulus | E = | 200,000 | $N/mm^2$ |
| Second moment of area | I0= | 21,600 | $cm^4$ (before corrosion) |
| Modulus of section | Z0= | 1,200 | $cm^3$ (before corrosion) |

| | | | | | | |
|---|---|---|---|---|---|---|
| Corrosion allowance | t1= | 0.00 | mm (front side) | t2= | 0.00 | mm (back side) |
| Corrosion ratio (againstI0) | η= | 1.00 | | | | |
| Corrosion ratio (against Z0) | η= | 1.00 | | | | |
| Joint efficiency (against I0) | μ= | 1.00 | | | | |
| Joint efficiency(againstZ0) | μ= | 1.00 | | | | |

| | | | |
|---|---|---|---|
| Allowable stress | σa = | 165 | $N/mm^2$ (Regular condition) |
| | σa' = | 190 | $N/mm^2$ (Seismic condition) |
| Allowable displacement | δa = | 50.0 | mm (Regular condition) |
| | δa'= | 75.0 | mm (Seismic condition) |

## 2 Calculation results

Front sheet pile    PU18 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | $(cm^4)$ | 38650 | | |
| Modulus of section | Z | $(cm^3)$ | 1800 | | |
| Max. bending moment | Mmax | (kN･m/m) | | 74.35 | 64.46 |
| Stress intensity | σ | $(N/mm^2)$ | | 41 (165) | 36 (190) |
| Footing depth | D | (m) | | 12.10 | 11.31 |
| Total length | L | (m) | 17.00 | | |

Tie-rod    φ 32 mm

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Tensile force | Tp | (kN) | | 134.96 | 123.05 |
| Stress intensity | σ | $(N/mm^2)$ | | 168 (176) | 153 (264) |

Waling    2×UPN-180×70

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Modulus of section | Z | (cm3) | 300 | | |
| Max. bending moment | Mmax | (kN･m) | | 32.39 | 29.53 |
| Stress intensity | σ | $(N/mm^2)$ | | 108 (140) | 98 (161) |

Bracing sheet pile    PU12 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | $(cm^4)$ | 21,600 | | |
| Modulus of section | Z | $(cm^3)$ | 1,200 | | |
| Max. bending moment | Mmax | (kN･m) | | 32.83 | 29.93 |
| Stress intensity | σ | $(N/mm^2)$ | | 27 (165) | 25 (190) |
| Horizontal displacement | δ | (mm) | | 3.86 (50.0) | 3.52 (75.0) |
| Footing depth | D | (m) | | 5.43 | 5.43 |
| Total length | L | (m) | 6.00 | | |

Bracing works Installation position

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Bracing works Installation position | d | (m) | | 5.88 | 6.45 |

Structural calculation of steel sheet pile works (Type C-2)

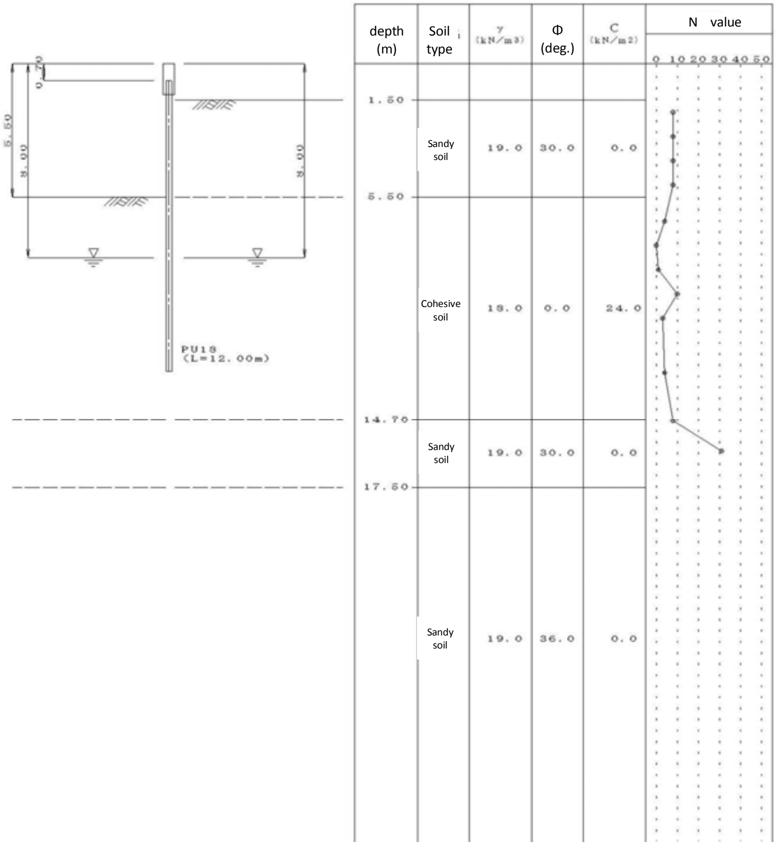
— **During construction** —

<Cantilevered sheet pile wall>



1 Design conditions

1-1 Structural figure




1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 5.50 | m |
| Protruction lemgth | H0= | 1.50 | |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 8.00 | m (Regular condition) |
| Outside water level | Lwp= | 8.00 | m (Regular condition) |

1-3 <u>Calculation method</u>

Calculation method　　Chang's formula

Footing depth　　$L = \frac{\pi}{\beta}$

1-4 <u>Design constant parameters</u>

Unit weight of water　　$\gamma w =$　　10.0 $kN/m^3$

Type of water pressure;　　trapezoid

1-5 <u>Coefficient of horizontal subgrade reaction</u>

Formula:　　$K_h = 6910 \times N'^{0.406}$

where,　N': Average N-value

1-6 <u>Overburden pressure</u>

Active pressure side　　Qa=　　10.0 $kN/m^2$ (Regular condition)

Passive pressure side　　Qp=　　0.0 $kN/m^2$ (Regular condition)

1-7 <u>Soil constant</u>

| No. | Depth m | Layer thickness m |
|---|---|---|
| 1 | 5.50 | 4.00 |
| 2 | 14.70 | 9.20 |
| 3 | 17.50 | 2.80 |
| 4 | 30.00 | 12.50 |

| No | Depth m | type | N-value | γ $kN/m^3$ | γ' $kN/m^3$ | φ Deg. | C $kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 5.50 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 14.70 | C | 4 | 18.0 | 9.0 | 0.0 | 24.0 | 0.0 |
| 3 | 17.50 | S | 20 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 4 | 30.00 | S | 37 | 19.0 | 10.0 | 36.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　　C ：Adhesive force

type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force

Ｎvalue：average value

γ ：unit weight of humid soil

γ'：unit weight of soil under the water

φ ：internal friction angle

1-8 <u>Sheet pile</u>

| | | | |
|---|---|---|---|
| Type | PU18 (S270GP) | | |
| Young modulus | E＝ | 200,000 | $N/mm^2$ |
| Second moment of area | I0＝ | 38650 | $cm^4$（before corrosion） |
| Modulus of section | Z0＝ | 1800 | $cm^3$（before corrosion） |

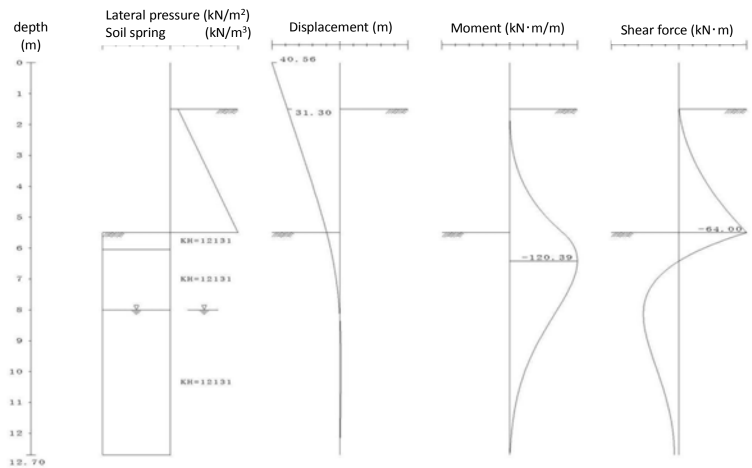
Corrosion allowance　　t1＝　0.00 mm (front side)　　t2＝　0.00 mm (back side)

| | | |
|---|---|---|
| Corrosion ratio (againstI0) | η＝ | 1.00 |
| Corrosion ratio (against Z0) | η＝ | 1.00 |
| Joint efficiency (against I0) | μ＝ | 1.00 |
| Joint efficiency(againstZ0) | μ＝ | 1.00 |

Allowable stress　　σa =　　165 $N/mm^2$（Regular condition）

Allowable displacement　　δa ＝　　50.0 mm（Regular condition）

2 <u>Section force and Displacement</u>




3 <u>Calculation results</u>

Front sheet pile　　PU18 (S270GP)

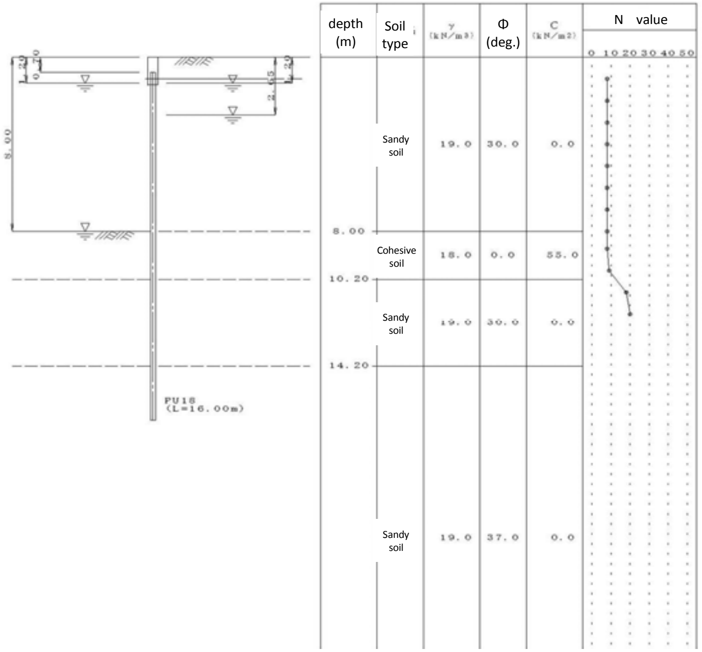
| | | | | Regular condition | |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 38650 | | |
| Modulus of section | Z | ($cm^3$) | 1800 | | |
| Max. bending moment | Mmax | (kN･m/m) | | 120.39 | |
| Stress intensity | σ | ($N/mm^2$) | | 67 (165) | |
| Horizontal displacement | δ | (mm) | | 31.3 (50.0) | |
| Footing depth | D | (m) | | 7.06 | |
| Total length | L | (m) | 12.00 | | |

## A-8-8 Structural calculation of steel sheet pile works (Type D-1)

<Bracing style sheet pile wall>

1 Design conditions

1-1 Structural figure




1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design height | H= | 8.00 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 2.65 | m (Regular condition) |
| | Lwa'= | 1.20 | m (Seismic condition) |
| Outside water level | Lwp= | 8.00 | m (Regular condition) |
| | Lwp'= | 1.20 | m (Seismic condition) |
| Tie-rod installation position | Ht= | 1.00 | m |
| Tie-rod horizontal intervals | l= | 2.40 | m |
| Tie-rod installation angle | θ = | 0.0000 | degree |

Method of calculation; Free earth support method

Virtual supporting point; Design front ground level

## 1-3 Design constant parameters

Unit weight of water $\gamma w=$ 10.0 $kN/m^3$

Type of water pressure; trapezoid

Design Seismic coefficient (in the air) k= 0.080

(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma\gamma_t \cdot h_i + \Sigma\gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma\gamma_t \cdot h_i + \Sigma(\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake; Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma\gamma h + 2Q}{2C} \cdot \tan\theta}$$

Safety factor of footing depth; 1.50 (Regular condition)
1.20 (Seismic condition)

## 1-4 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N': Average N-value

Result of boring survey

| No | Depth m | N-value |
|---|---|---|
| 1 | 1.00 | 8 |
| 2 | 2.00 | 8 |
| 3 | 3.00 | 8 |
| 4 | 4.00 | 8 |
| 5 | 5.00 | 8 |
| 6 | 6.00 | 8 |
| 7 | 7.00 | 8 |
| 8 | 8.00 | 8 |
| 9 | 8.80 | 8 |
| 10 | 9.80 | 9 |

| No | Depth m | N-value |
|---|---|---|
| 11 | 10.80 | 18 |
| 12 | 11.80 | 20 |

## 1-5 Overburden pressure

Active pressure side Qa= 10.0 $kN/m^2$ (Regular condition)
Qa'= 5.0 $kN/m^2$ (Seismic condition)

Passive pressure side Qp= 0.0 $kN/m^2$ (Regular condition)
Qp'= 0.0 $kN/m^2$ (Seismic condition)

## 1-6 Soil constant

| No | Depth m | type | N-value | γ $kN/m^3$ | γ' $kN/m^3$ | φ Deg. | C $kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 8.00 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 10.20 | C | 9 | 18.0 | 9.0 | 0.0 | 55.0 | 0.0 |
| 3 | 14.20 | S | 19 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 4 | 30.00 | S | 40 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

## 1-7 Sheet pile

Type PU18 (S270GP)
Young modulus E = 200,000 $N/mm^2$
Second moment of area I0= 38,650 $cm^4$ (before corrosion)
Modulus of section Z0= 1,800 $cm^3$ (before corrosion)

Corrosion allowance t1= 0.00 mm (front side) t2= 0.00 mm (back side)

Allowable stress σa = 165 $N/mm^2$ (Regular condition)
σa' = 190 $N/mm^2$ (Seismic condition)

## 1-8 Tie-rod

Diameter of tie-rod calculated

Corrosion considered (0.00mm)

Allowable stress σa = 176 $N/mm^2$ (Regular condition)
σa' = 264 $N/mm^2$ (Seismic condition)

## 1-9 Waling

Type 2×UPN-220×80

Modulus of section; 490 cm3

Corrosion ; considered (0.00mm)
Corrosion ratio; η = 1.00

Allowable stress σa = 140 $N/mm^2$ (Regular condition)
σa' = 161 $N/mm^2$ (Seismic condition)

## 1-10 Bracing sheet pile

Calculation method Chang's formula

Crest elevation of bracing works 0.30 m (distance from tie-rod)

| | | | |
|---|---|---|---|
| Materials | PU12 (S270GP) | | |
| Young's modulus | E = | 200,000 | $N/mm^2$ |
| Second moment of area | I0= | 21,600 | $cm^4$ (before corrosion) |
| Modulus of section | Z0= | 1,200 | $cm^3$ (before corrosion) |
| Corrosion allowance | t1= | 0.00 | mm (front side) t2= 0.00 mm (back side) |
| Corrosion ratio (againstI0) | η= | 1.00 | |
| Corrosion ratio (against Z0) | η= | 1.00 | |
| Joint efficiency (against I0) | μ= | 1.00 | |
| Joint efficiency(againstZ0) | μ= | 1.00 | |
| Allowable stress | σa = | 165 | $N/mm^2$ (Regular condition) |
| | σa' = | 190 | $N/mm^2$ (Seismic condition) |
| Allowable displacement | δa = | 50.0 | mm (Regular condition) |
| | δa'= | 75.0 | mm (Seismic condition) |

## 2 Calculation results

Front sheet pile PU18 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 38650 | | |
| Modulus of section | Z | ($cm^3$) | 1800 | | |
| Max. bending moment | Mmax | (kN･m/m) | | 283.1 | 154.42 |
| Stress intensity | σ | ($N/mm^2$) | | 157 (165) | 88 (190) |
| Footing depth | D | (m) | | 8.55 | 1.96 |
| Total length | L | (m) | 16.00 | | |

Tie-rod φ 46 mm

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Tensile force | Tp | (kN) | | 283.12 | 184.21 |
| Stress intensity | σ | ($N/mm^2$) | | 170 (176) | 111 (264) |

Waling 2×UPN-220×80

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Modulus of section | Z | (cm3) | 490 | | |
| Max. bending moment | Mmax | (kN･m) | | 67.95 | 44.21 |
| Stress intensity | σ | ($N/mm^2$) | | 139 (140) | 90 (161) |

Bracing sheet pile PU12 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 21,600 | | |
| Modulus of section | Z | ($cm^3$) | 1,200 | | |
| Max. bending moment | Mmax | (kN･m) | | 68.87 | 44.81 |
| Stress intensity | σ | ($N/mm^2$) | | 57 (165) | 37 (190) |
| Horizontal displacement | δ | (mm) | | 8.11 (50.0) | 5.27 (75.0) |
| Footing depth | D | (m) | | 5.43 | 5.43 |
| Total length | L | (m) | 6.00 | | |

Bracing works Installation position

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Bracing works Installation position | d | (m) | | 7.33 | 8.63 |

Structural calculation of steel sheet pile works (Type D-1)

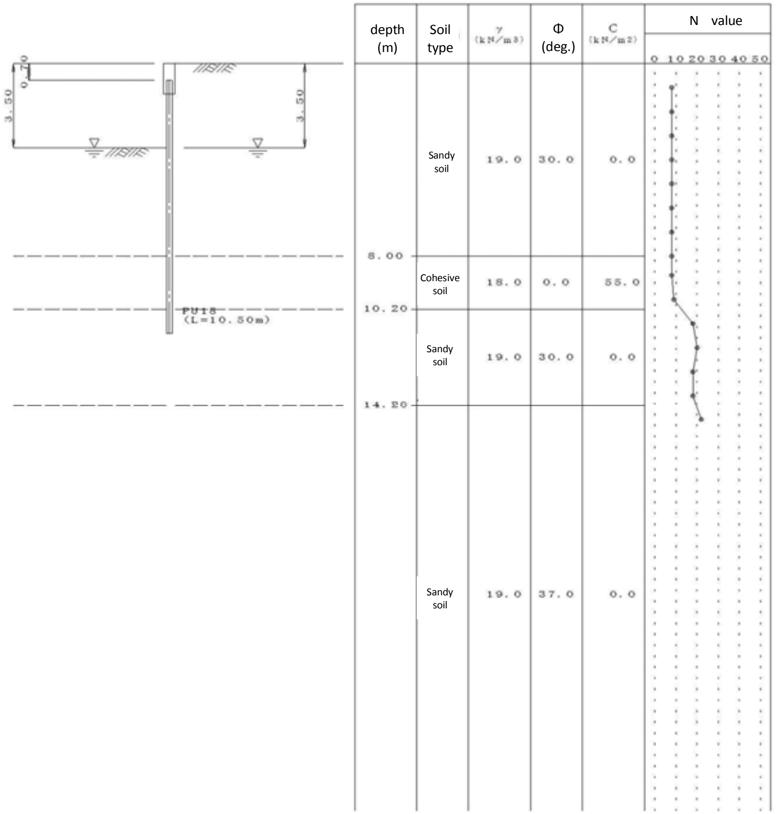
－ **During construction** －

<Cantilevered sheet pile wall>



# 1 Design conditions

## 1-1 Structural figure




## 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 3.50 | m |
| Protruction lemgth | H0= | 0.00 | |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 3.50 | m (Regular condition) |
| Outside water level | Lwp= | 3.50 | m (Regular condition) |

## 1-3 Calculation method

Calculation method: Chang's formula

Footing depth: $L = \dfrac{\pi}{\beta}$

## 1-4 Design constant parameters

Unit weight of water: $\gamma w =$ 10.0 kN/m³

Type of water pressure; trapezoid

## 1-5 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N': Average N-value

## 1-6 Overburden pressure

Active pressure side: Qa= 10.0 kN/m² (Regular condition)

Passive pressure side: Qp= 0.0 kN/m² (Regular condition)

## 1-7 Soil constant

| No. | Depth<br>m | Layer thickness<br>m |
|---|---|---|
| 1 | 8.00 | 8.00 |
| 2 | 10.20 | 2.20 |
| 3 | 14.20 | 4.00 |
| 4 | 30.00 | 15.80 |

| No | Depth<br>m | type | N-value | γ<br>kN/m³ | γ'<br>kN/m³ | φ<br>Deg. | C<br>kN/m² | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 8.00 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 10.20 | C | 9 | 18.0 | 9.0 | 0.0 | 55.0 | 0.0 |
| 3 | 14.20 | S | 19 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 4 | 30.00 | S | 40 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

## 1-8 Sheet pile

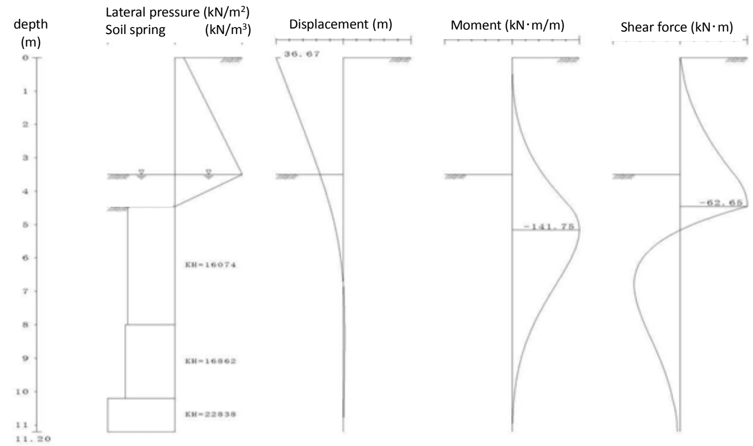
| | | | |
|---|---|---|---|
| Type | PU18 (S270GP) | | |
| Young modulus | E ＝ | 200,000 | N/mm² |
| Second moment of area | I0＝ | 38650 | cm⁴ (before corrosion) |
| Modulus of section | Z0＝ | 1800 | cm³ (before corrosion) |

Corrosion allowance: t1＝ 0.00 mm (front side)　t2＝ 0.00 mm (back side)
Corrosion ratio (againstI0): η＝ 1.00
Corrosion ratio (against Z0): η＝ 1.00
Joint efficiency (against I0): μ＝ 1.00
Joint efficiency(againstZ0): μ＝ 1.00

Allowable stress: σa = 165 N/mm² (Regular condition)

Allowable displacement: δa ＝ 50.0 mm (Regular condition)

## 2 Section force and Displacement




## 3 Calculation results

Front sheet pile: PU18 (S270GP)

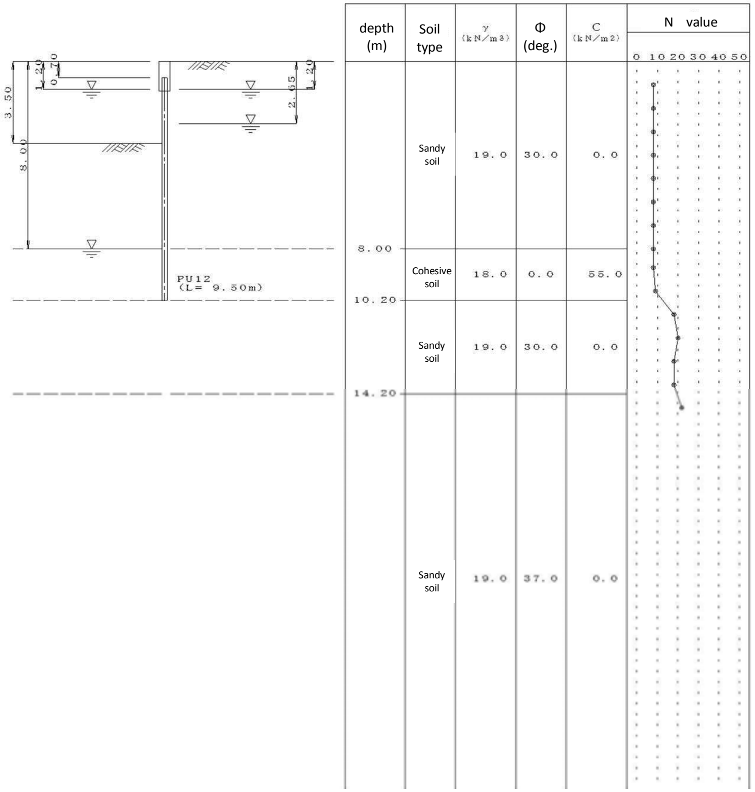
| | | | Regular condition | |
|---|---|---|---|---|
| Second moment of area | I (cm⁴) | 38650 | | |
| Modulus of section | Z (cm³) | 1800 | | |
| Max. bending moment | Mmax (kN·m/m) | | 141.75 | |
| Stress intensity | σ (N/mm²) | | 79 (165) | |
| Horizontal displacement | δ (mm) | | 36.67 (50.0) | |
| Footing depth | D (m) | | 7.44 | |
| Total length | L (m) | 10.50 | | |

A-8-9 Structural calculation of steel sheet pile works (Type D-2)

<Cantilevered sheet pile wall>

# 1 Design conditions

## 1-1 Structural figure




## 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 3.50 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 2.65 | m (Regular condition) |
| | Lwa'= | 1.20 | m (Seismic condition) |
| Outside water level | Lwp= | 8.00 | m (Regular condition) |
| | Lwp'= | 1.20 | m (Seismic condition) |

1-3 Calculation method

Calculation method Chang's formula

Footing depth $L = \frac{\pi}{\beta}$

1-4 Design constant parameters

Unit weight of water $\gamma w =$ 10.0 kN/m$^3$

Type of water pressure; trapezoid

Design Seismic coefficient (in the air) k= 0.080

(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma \gamma_t \cdot h_i + \Sigma \gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma \gamma_t \cdot h_i + \Sigma (\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake; Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma \gamma h + 2Q}{2C} \cdot \tan\theta}$$

$K_c =$ 0.50

1-5 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N': Average N-value

1-6 Overburden pressure

Active pressure side Qa= 10.0 kN/m$^2$ (Regular condition)
Qa'= 5.0 kN/m$^2$ (Seismic condition)

Passive pressure side Qp= 0.0 kN/m$^2$ (Regular condition)
Qp'= 0.0 kN/m$^2$ (Seismic condition)

1-7 Soil constant

| No. | Depth<br>m | Layer thickness<br>m |
|---|---|---|
| 1 | 8.00 | 8.00 |
| 2 | 10.20 | 2.20 |
| 3 | 14.20 | 4.00 |
| 4 | 30.00 | 15.80 |

| No | Depth<br>m | type | N-value | γ<br>kN/m$^3$ | γ'<br>kN/m$^3$ | φ<br>Deg. | C<br>kN/m$^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 8.00 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 10.20 | C | 9 | 18.0 | 9.0 | 0.0 | 55.0 | 0.0 |
| 3 | 14.20 | S | 19 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 4 | 30.00 | S | 40 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

1-8 Sheet pile

Type PU12 (S270GP)
Young modulus E ＝ 200,000 N/mm$^2$
Second moment of area I0＝ 21600 cm$^4$（before corrosion）
Modulus of section Z0＝ 1200 cm$^3$（before corrosion）

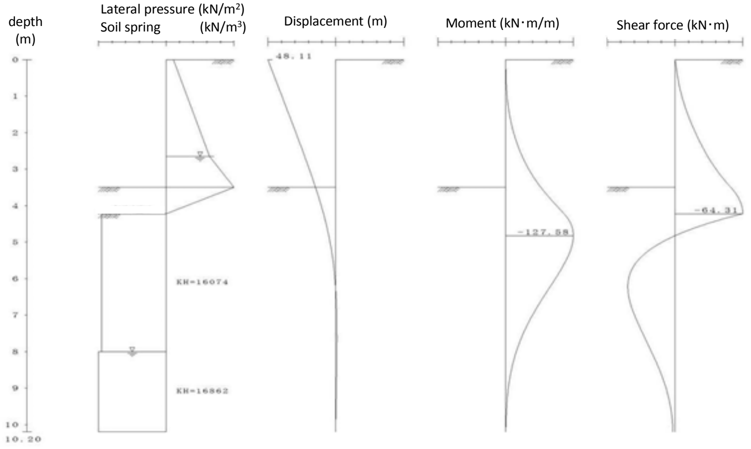
Corrosion allowance t1＝ 0.00 mm (front side) t2＝ 0.00 mm (back side)
Corrosion ratio (againstI0) η＝ 1.00
Corrosion ratio (against Z0) η＝ 1.00
Joint efficiency (against I0) μ＝ 1.00
Joint efficiency(againstZ0) μ＝ 1.00

Allowable stress σa = 165 N/mm$^2$（Regular condition）
σa' = 190 N/mm$^2$（Seismic condition）

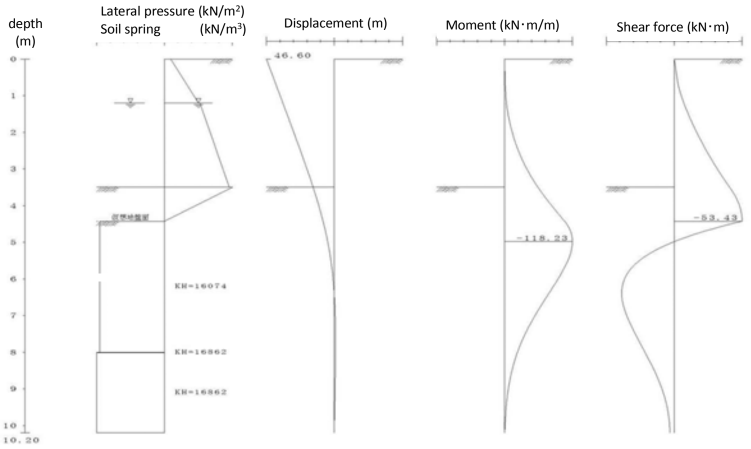
Allowable displacement δa ＝ 50.0 mm（Regular condition）
δa'＝ 75.0 mm（Seismic condition）

2 Section force and Displacement

(Regular condition)




(Seismic condition)




3 Calculation results

Front sheet pile PU12 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 21600 | | |
| Modulus of section | Z | ($cm^3$) | 1200 | | |
| Max. bending moment | Mmax | (kN·m/m) | | 127.58 | 118.23 |
| Stress intensity | σ | ($N/mm^2$) | | 106 (165) | 99 (190) |
| Horizontal displacement | δ | (mm) | | 48.11 (50.0) | 46.6 (75.0) |
| Footing depth | D | (m) | | 6.39 | 6.59 |
| Total length | L | (m) | 9.50 | | |

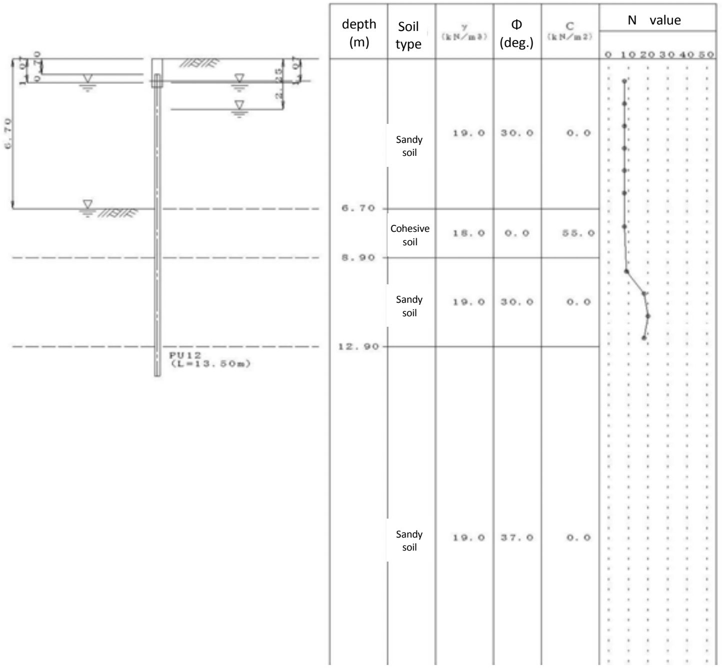
A-8-10 Structural calculation of steel sheet pile works (Type E-1)

<Bracing style sheet pile wall>



# 1 Design conditions

## 1-1 Structural figure




## 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design height | H= | 6.70 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 2.25 | m (Regular condition) |
| | Lwa'= | 1.07 | m (Seismic condition) |
| Outside water level | Lwp= | 6.70 | m (Regular condition) |
| | Lwp'= | 1.07 | m (Seismic condition) |
| Tie-rod installation position | Ht= | 1.00 | m |
| Tie-rod horizontal intervals | l= | 2.40 | m |
| Tie-rod installation angle | θ = | 0.0000 | degree |

Method of calculation; Free earth support method

Virtual supporting point; Design front ground level

### 1-3 Design constant parameters

Unit weight of water $\gamma w=$ 10.0 $kN/m^3$

Type of water pressure; trapezoid

Design Seismic coefficient (in the air) k= 0.080

(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma\gamma_t \cdot h_i + \Sigma\gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma\gamma_t \cdot h_i + \Sigma(\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake; Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma\gamma h + 2Q}{2C} \cdot \tan\theta}$$

Safety factor of footing depth; 1.50 (Regular condition)
1.20 (Seismic condition)

### 1-4 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N': Average N-value

Result of boring survey

| No | Depth m | N-value |
|---|---|---|
| 1 | 1.00 | 8 |
| 2 | 2.00 | 8 |
| 3 | 3.00 | 8 |
| 4 | 4.00 | 8 |
| 5 | 5.00 | 8 |
| 6 | 6.00 | 8 |
| 7 | 7.50 | 8 |
| 8 | 9.50 | 9 |
| 9 | 10.50 | 18 |
| 10 | 11.50 | 20 |

| No | Depth m | N-value |
|---|---|---|
| 11 | 12.50 | 18 |

### 1-5 Overburden pressure

Active pressure side Qa= 10.0 $kN/m^2$ (Regular condition)
Qa'= 5.0 $kN/m^2$ (Seismic condition)

Passive pressure side Qp= 0.0 $kN/m^2$ (Regular condition)
Qp'= 0.0 $kN/m^2$ (Seismic condition)

### 1-6 Soil constant

| No | Depth m | type | N-value | γ $kN/m^3$ | γ' $kN/m^3$ | φ Deg. | C $kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 6.70 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 8.90 | C | 9 | 18.0 | 9.0 | 0.0 | 55.0 | 0.0 |
| 3 | 12.90 | S | 19 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 4 | 30.00 | S | 40 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

### 1-7 Sheet pile

Type PU12 (S270GP)
Young modulus E ＝ 200,000 $N/mm^2$
Second moment of area I0＝ 21,600 $cm^4$ (before corrosion)
Modulus of section Z0＝ 1,200 $cm^3$ (before corrosion)

Corrosion allowance t1＝ 0.00 mm (front side) t2＝ 0.00 mm (back side)

Allowable stress σa = 165 $N/mm^2$ (Regular condition)
σa' = 190 $N/mm^2$ (Seismic condition)

### 1-8 Tie-rod

Diameter of tie-rod calculated

Corrosion considered (0.00mm)

Allowable stress σa = 176 $N/mm^2$ (Regular condition)
σa' = 264 $N/mm^2$ (Seismic condition)

### 1-9 Waling

Type 2×UPN-200×75

Modulus of section; 382 cm3

Corrosion ; considered (0.00mm)
Corrosion ratio; η ＝ 1.00

Allowable stress σa = 140 $N/mm^2$ (Regular condition)
σa' = 161 $N/mm^2$ (Seismic condition)

## 1-10 Bracing sheet pile

Calculation method Chang's formula

Crest elevation of bracing works 0.30 m (distance from tie-rod)

| | | | |
|---|---|---|---|
| Materials | PU12 (S270GP) | | |
| Young's modulus | E = | 200,000 | $N/mm^2$ |
| Second moment of area | I0= | 21,600 | $cm^4$ (before corrosion) |
| Modulus of section | Z0= | 1,200 | $cm^3$ (before corrosion) |

| | | | | | |
|---|---|---|---|---|---|
| Corrosion allowance | t1= | 0.00 mm (front side) | t2= | 0.00 mm (back side) |
| Corrosion ratio (againstI0) | η= | 1.00 | | |
| Corrosion ratio (against Z0) | η= | 1.00 | | |
| Joint efficiency (against I0) | μ= | 1.00 | | |
| Joint efficiency(againstZ0) | μ= | 1.00 | | |

| | | | |
|---|---|---|---|
| Allowable stress | σa = | 165 | $N/mm^2$ (Regular condition) |
| | σa' = | 190 | $N/mm^2$ (Seismic condition) |
| Allowable displacement | δa = | 50.0 | mm (Regular condition) |
| | δa'= | 75.0 | mm (Seismic condition) |



## 2 Calculation results

Front sheet pile PU12 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 21600 | | |
| Modulus of section | Z | ($cm^3$) | 1200 | | |
| Max. bending moment | Mmax | (kN·m/m) | | 161.85 | 88.65 |
| Stress intensity | σ | ($N/mm^2$) | | 135 (165) | 74 (190) |
| Footing depth | D | (m) | | 7.44 | 1.25 |
| Total length | L | (m) | 13.50 | | |

Tie-rod φ 42 mm

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Tensile force | Tp | (kN) | | 208.46 | 137.11 |
| Stress intensity | σ | ($N/mm^2$) | | 150 (176) | 99 (264) |

Waling 2×UPN-200×75

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Modulus of section | Z | (cm3) | 382 | | |
| Max. bending moment | Mmax | (kN·m) | | 50.03 | 32.91 |
| Stress intensity | σ | ($N/mm^2$) | | 131 (140) | 86 (161) |

Bracing sheet pile PU12 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 21,600 | | |
| Modulus of section | Z | ($cm^3$) | 1,200 | | |
| Max. bending moment | Mmax | (kN·m) | | 50.71 | 33.35 |
| Stress intensity | σ | ($N/mm^2$) | | 42 (165) | 28 (190) |
| Horizontal displacement | δ | (mm) | | 5.97 (50.0) | 3.93 (75.0) |
| Footing depth | D | (m) | | 5.43 | 5.43 |
| Total length | L | (m) | 6.00 | | |

Bracing works Installation position

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Bracing works Installation position | d | (m) | | 5.57 | 7.69 |

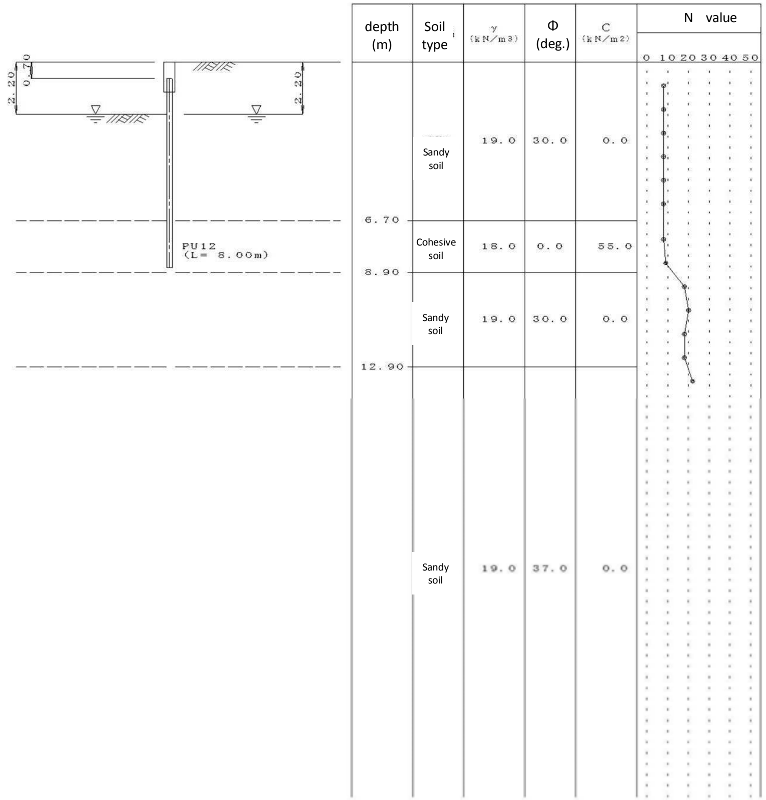
Structural calculation of steel sheet pile works (Type E-1)

－ **During construction** －

<Cantilevered sheet pile wall>

# 1 Design conditions

## 1-1 Structural figure




## 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 2.20 | m |
| Protruction lemgth | H0= | 0.00 | |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 2.20 | m (Regular condition) |
| Outside water level | Lwp= | 2.20 | m (Regular condition) |

## 1-3 Calculation method

Calculation method Chang's formula

Footing depth $L = \frac{\pi}{\beta}$

## 1-4 Design constant parameters

Unit weight of water $\gamma w =$ 10.0 $kN/m^3$

Type of water pressure; trapezoid

## 1-5 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N': Average N-value

## 1-6 Overburden pressure

Active pressure side Qa= 10.0 $kN/m^2$ (Regular condition)

Passive pressure side Qp= 0.0 $kN/m^2$ (Regular condition)

## 1-7 Soil constant

| No. | Depth m | Layer thickness m |
|---|---|---|
| 1 | 6.70 | 6.70 |
| 2 | 8.90 | 2.20 |
| 3 | 12.90 | 4.00 |
| 4 | 30.00 | 17.10 |

| No | Depth m | type | N-value | γ $kN/m^3$ | γ' $kN/m^3$ | φ Deg. | C $kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 6.70 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 8.90 | C | 9 | 18.0 | 9.0 | 0.0 | 55.0 | 0.0 |
| 3 | 12.90 | S | 19 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 4 | 30.00 | S | 40 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M) a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

## 1-8 Sheet pile

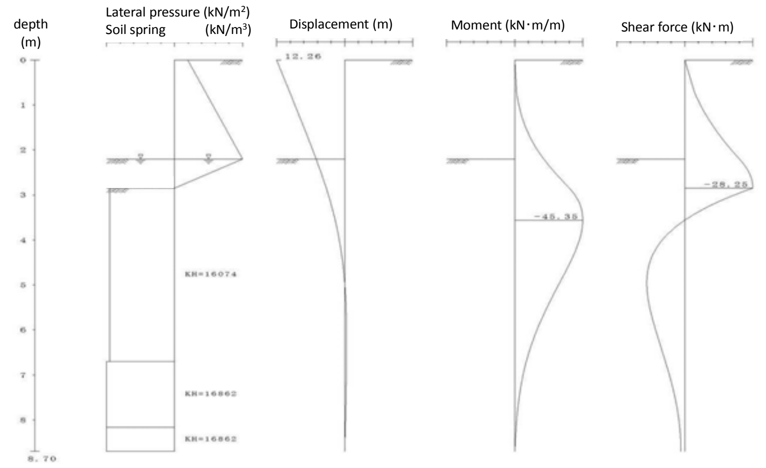
Type PU12 (S270GP)
Young modulus E ＝ 200,000 $N/mm^2$
Second moment of area I0＝ 21600 $cm^4$（before corrosion）
Modulus of section Z0＝ 1200 $cm^3$（before corrosion）

Corrosion allowance t1＝ 0.00 mm (front side) t2＝ 0.00 mm (back side)
Corrosion ratio (againstI0) η＝ 1.00
Corrosion ratio (against Z0) η＝ 1.00
Joint efficiency (against I0) μ＝ 1.00
Joint efficiency(againstZ0) μ＝ 1.00

Allowable stress σa = 165 $N/mm^2$（Regular condition）

Allowable displacement δa ＝ 50.0 mm（Regular condition）

## 2 Section force and Displacement




## 3 Calculation results

Front sheet pile PU12 (S270GP)

| | | | Regular condition | |
|---|---|---|---|---|
| Second moment of area | I $(cm^4)$ | 21600 | | |
| Modulus of section | Z $(cm^3)$ | 1200 | | |
| Max. bending moment | Mmax (kN･m/m) | | 45.35 | |
| Stress intensity | σ $(N/mm^2)$ | | 38 (165) | |
| Horizontal displacement | δ (mm) | | 12.26 (50.0) | |
| Footing depth | D (m) | | 6.31 | |
| Total length | L (m) | 8.00 | | |

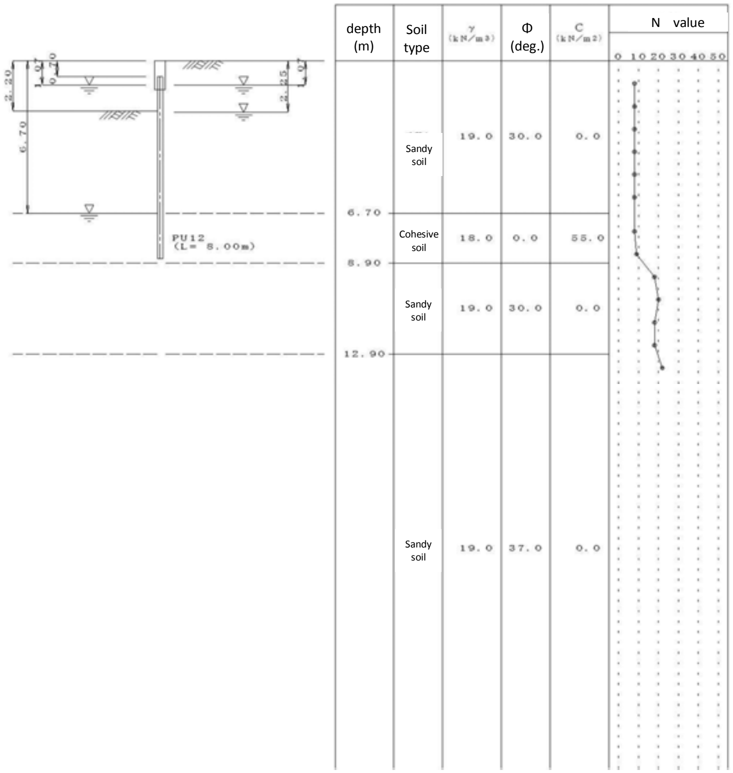
A-8-11 Structural calculation of steel sheet pile works (Type E-2)

<Cantilevered sheet pile wall>



1 Design conditions

1-1 Structural figure




1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 2.20 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 2.25 | m (Regular condition) |
| | Lwa'= | 1.07 | m (Seismic condition) |
| Outside water level | Lwp= | 6.70 | m (Regular condition) |
| | Lwp'= | 1.07 | m (Seismic condition) |

## 1-3 Calculation method

Calculation method　　Chang's formula

Footing depth　　$L = \frac{\pi}{\beta}$

## 1-4 Design constant parameters

Unit weight of water　　$\gamma w =$　　10.0　$kN/m^3$

Type of water pressure;　　trapezoid

Design Seismic coefficient (in the air)　　k=　　0.080

(under water)　Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma \gamma_t \cdot h_i + \Sigma \gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma \gamma_t \cdot h_i + \Sigma(\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake;　　Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma \gamma h + 2Q}{2C} \cdot \tan\theta}$$

$K_c =$　　0.50

## 1-5 Coefficient of horizontal subgrade reaction

Formula:　　$K_h = 6910 \times N'^{0.406}$

where,　N': Average N-value

## 1-6 Overburden pressure

| | | | | |
|---|---|---|---|---|
| Active pressure side | Qa= | 10.0 | $kN/m^2$ | (Regular condition) |
| | Qa'= | 5.0 | $kN/m^2$ | (Seismic condition) |
| Passive pressure side | Qp= | 0.0 | $kN/m^2$ | (Regular condition) |
| | Qp'= | 0.0 | $kN/m^2$ | (Seismic condition) |

## 1-7 Soil constant

| No. | Depth<br>m | Layer thickness<br>m |
|---|---|---|
| 1 | 6.70 | 6.70 |
| 2 | 8.90 | 2.20 |
| 3 | 12.90 | 4.00 |
| 4 | 30.00 | 17.10 |

| No | Depth<br>m | type | N-value | γ<br>$kN/m^3$ | γ'<br>$kN/m^3$ | φ<br>Deg. | C<br>$kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 6.70 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 8.90 | C | 9 | 18.0 | 9.0 | 0.0 | 55.0 | 0.0 |
| 3 | 12.90 | S | 19 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 4 | 30.00 | S | 40 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　　C ：Adhesive force

type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force

Ｎvalue：average value

γ ：unit weight of humid soil

γ'：unit weight of soil under the water

φ ：internal friction angle

## 1-8 Sheet pile

| | | | |
|---|---|---|---|
| Type | PU12 (S270GP) | | |
| Young modulus | E ＝ | 200,000 | $N/mm^2$ |
| Second moment of area | I0＝ | 21600 | $cm^4$（before corrosion） |
| Modulus of section | Z0＝ | 1200 | $cm^3$（before corrosion） |

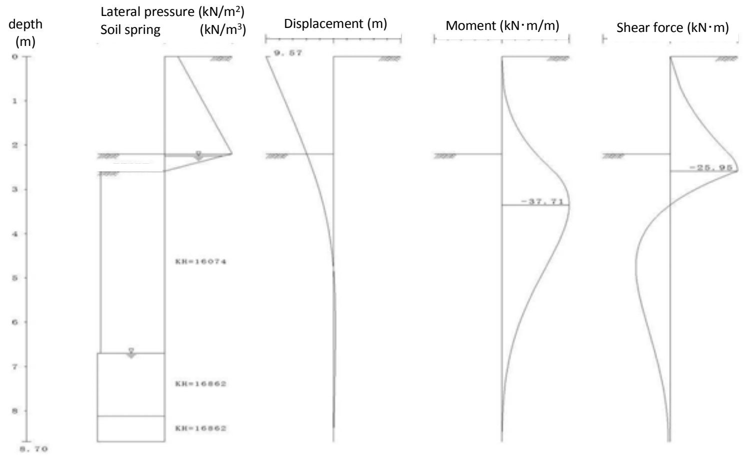
Corrosion allowance　　t1＝　　0.00 mm (front side)　　t2＝　　0.00 mm (back side)

| | | |
|---|---|---|
| Corrosion ratio (againstI0) | η＝ | 1.00 |
| Corrosion ratio (against Z0) | η＝ | 1.00 |
| Joint efficiency (against I0) | μ＝ | 1.00 |
| Joint efficiency(againstZ0) | μ＝ | 1.00 |

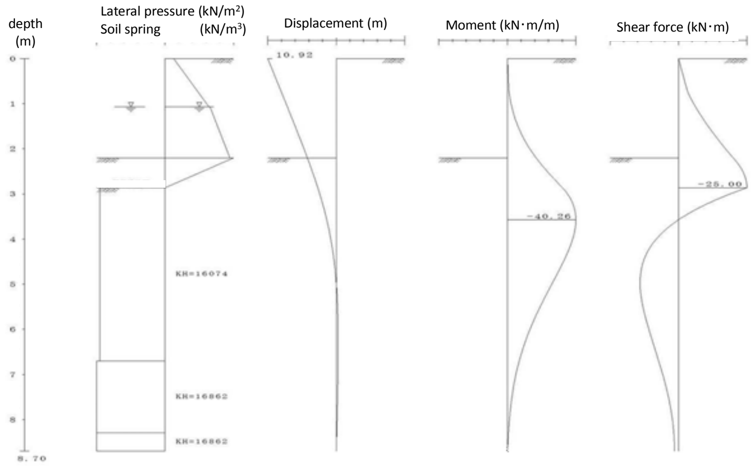
| | | | |
|---|---|---|---|
| Allowable stress | σa = | 165 | $N/mm^2$（Regular condition） |
| | σa' = | 190 | $N/mm^2$（Seismic condition） |
| Allowable displacement | δa ＝ | 50.0 | mm（Regular condition） |
| | δa'＝ | 75.0 | mm（Seismic condition） |

## 2 Section force and Displacement

(Regular condition)




(Seismic condition)




## 3 Calculation results

Front sheet pile PU12 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | (cm$^4$) | 21600 | | |
| Modulus of section | Z | (cm$^3$) | 1200 | | |
| Max. bending moment | Mmax | (kN・m/m) | | 37.71 | 40.26 |
| Stress intensity | σ | (N/mm$^2$) | | 31 (165) | 34 (190) |
| Horizontal displacement | δ | (mm) | | 9.57 (50.0) | 10.92 (75.0) |
| Footing depth | D | (m) | | 6.06 | 6.33 |
| Total length | L | (m) | 8.00 | | |

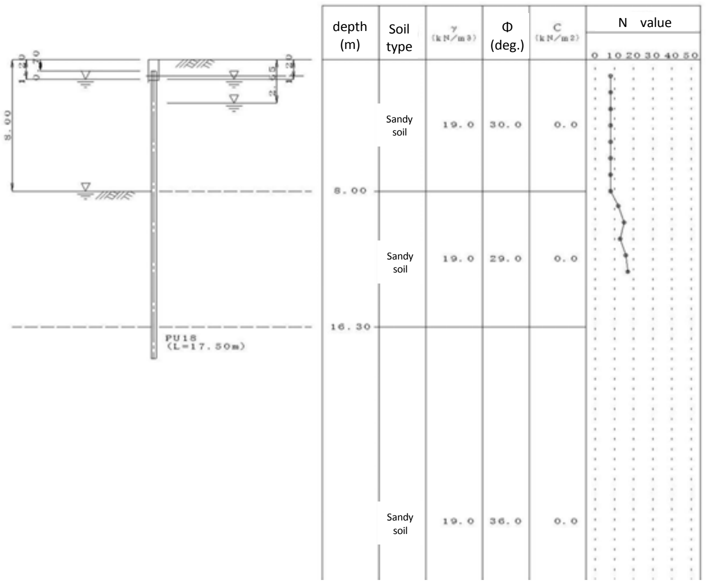
A-8-12 Structural calculation of steel sheet pile works (Type F)

<Bracing style sheet pile wall>



## 1 Design conditions

### 1-1 Structural figure




### 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design height | H= | 8.00 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 2.65 | m (Regular condition) |
| | Lwa'= | 1.20 | m (Seismic condition) |
| Outside water level | Lwp= | 8.00 | m (Regular condition) |
| | Lwp'= | 1.20 | m (Seismic condition) |
| | | | |
| Tie-rod installation position | Ht= | 1.00 | m |
| Tie-rod horizontal intervals | l= | 2.40 | m |
| Tie-rod installation angle | θ = | 0.0000 | degree |

Method of calculation; Free earth support method

Virtual supporting point; Design front ground level

## 1-3 Design constant parameters

Unit weight of water $\gamma w =$ 10.0 $kN/m^3$

Type of water pressure; trapezoid

Design Seismic coefficient (in the air) k= 0.080

(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma \gamma_t \cdot h_i + \Sigma \gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma \gamma_t \cdot h_i + \Sigma (\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake; Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1} \sqrt{1 - \frac{\Sigma \gamma h + 2Q}{2C} \cdot \tan \theta}$$

Safety factor of footing depth; 1.50 (Regular condition)
1.20 (Seismic condition)

## 1-4 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N': Average N-value

Result of boring survey

| No | Depth m | N-value |
|---|---|---|
| 1 | 1.00 | 8 |
| 2 | 2.00 | 8 |
| 3 | 3.00 | 8 |
| 4 | 4.00 | 8 |
| 5 | 5.00 | 8 |
| 6 | 6.00 | 8 |
| 7 | 7.00 | 8 |
| 8 | 8.00 | 8 |
| 9 | 8.90 | 12 |
| 10 | 9.90 | 15 |

| No | Depth m | N-value |
|---|---|---|
| 11 | 10.90 | 13 |
| 12 | 11.90 | 16 |
| 13 | 12.90 | 17 |

## 1-5 Overburden pressure

Active pressure side Qa= 10.0 $kN/m^2$ (Regular condition)
Qa'= 5.0 $kN/m^2$ (Seismic condition)

Passive pressure side Qp= 0.0 $kN/m^2$ (Regular condition)
Qp'= 0.0 $kN/m^2$ (Seismic condition)

## 1-6 Soil constant

| No | Depth m | type | N-value | γ $kN/m^3$ | γ' $kN/m^3$ | φ Deg. | C $kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 8.00 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 16.30 | S | 17 | 19.0 | 10.0 | 29.0 | 0.0 | 0.0 |
| 3 | 40.00 | S | 36 | 19.0 | 10.0 | 36.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M) a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

## 1-7 Sheet pile

Type PU18 (S270GP)
Young modulus E ＝ 200,000 $N/mm^2$
Second moment of area I0＝ 38,650 $cm^4$ (before corrosion)
Modulus of section Z0＝ 1,800 $cm^3$ (before corrosion)

Corrosion allowance t1＝ 0.00 mm (front side) t2＝ 0.00 mm (back side)

Allowable stress σa = 165 $N/mm^2$ (Regular condition)
σa' = 190 $N/mm^2$ (Seismic condition)

## 1-8 Tie-rod

Diameter of tie-rod calculated

Corrosion considered（0.00mm）

Allowable stress σa = 176 $N/mm^2$ (Regular condition)
σa' = 264 $N/mm^2$ (Seismic condition)

## 1-9 Waling

Type 2×UPN-220×80

Modulus of section; 490 cm3

Corrosion ; considered（0.00mm）
Corrosion ratio; η ＝ 1.00

Allowable stress σa = 140 $N/mm^2$ (Regular condition)
σa' = 161 $N/mm^2$ (Seismic condition)

## 1-10 Bracing sheet pile

Calculation method Chang's formula

Crest elevation of bracing works 0.30 m (distance from tie-rod)

| | | | |
|---|---|---|---|
| Materials | PU12 (S270GP) | | |
| Young's modulus | E = | 200,000 | $N/mm^2$ |
| Second moment of area | I0= | 21,600 | $cm^4$ (before corrosion) |
| Modulus of section | Z0= | 1,200 | $cm^3$ (before corrosion) |
| Corrosion allowance | t1= | 0.00 | mm (front side) t2= 0.00 mm (back side) |
| Corrosion ratio (againstI0) | η= | 1.00 | |
| Corrosion ratio (against Z0) | η= | 1.00 | |
| Joint efficiency (against I0) | μ= | 1.00 | |
| Joint efficiency(againstZ0) | μ= | 1.00 | |
| Allowable stress | σa = | 165 | $N/mm^2$ (Regular condition) |
| | σa' = | 190 | $N/mm^2$ (Seismic condition) |
| Allowable displacement | δa = | 50.0 | mm (Regular condition) |
| | δa'= | 75.0 | mm (Seismic condition) |



## 2 Calculation results

Front sheet pile PU18 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 38650 | | |
| Modulus of section | Z | ($cm^3$) | 1800 | | |
| Max. bending moment | Mmax | (kN･m/m) | | 283.19 | 154.42 |
| Stress intensity | σ | ($N/mm^2$) | | 157 (165) | 86 (190) |
| Footing depth | D | (m) | | 9.90 | 5.85 |
| Total length | L | (m) | 17.50 | | |

Tie-rod φ 46 mm

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Tensile force | Tp | (kN) | | 283.12 | 184.21 |
| Stress intensity | σ | ($N/mm^2$) | | 170 (176) | 111 (264) |

Waling 2×UPN-220×80

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Modulus of section | Z | (cm3) | 490 | | |
| Max. bending moment | Mmax | (kN･m) | | 67.95 | 44.21 |
| Stress intensity | σ | ($N/mm^2$) | | 139 (140) | 90 (161) |

Bracing sheet pile PU12 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 21,600 | | |
| Modulus of section | Z | ($cm^3$) | 1,200 | | |
| Max. bending moment | Mmax | (kN･m) | | 68.87 | 44.81 |
| Stress intensity | σ | ($N/mm^2$) | | 57 (165) | 37 (190) |
| Horizontal displacement | δ | (mm) | | 8.11 (50.0) | 5.27 (75.0) |
| Footing depth | D | (m) | | 5.43 | 5.43 |
| Total length | L | (m) | 6.00 | | |

Bracing works Installation position

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Bracing works Installation position | d | (m) | | 7.33 | 8.63 |

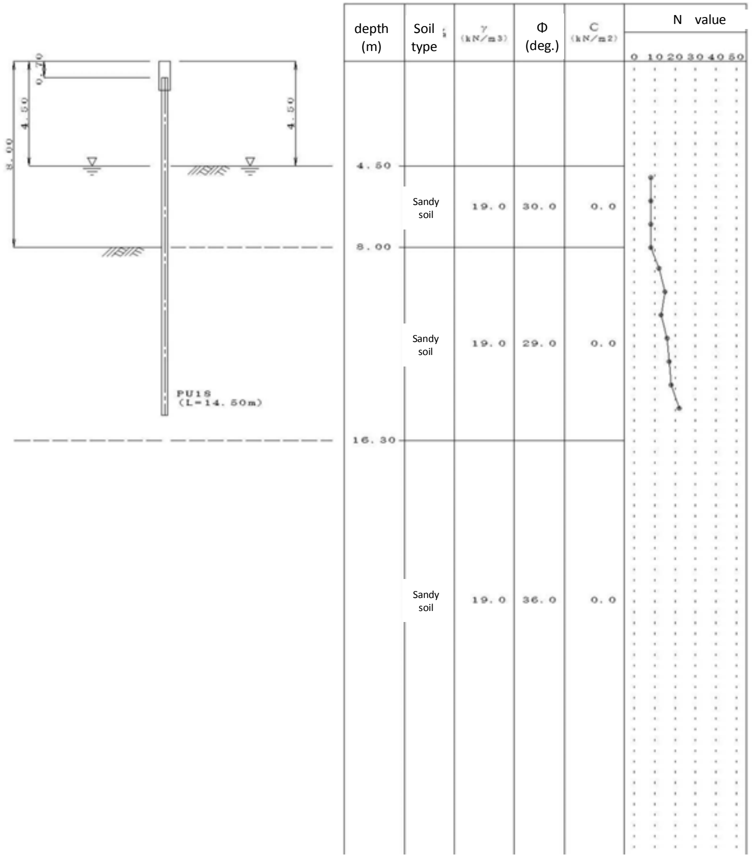
Structural calculation of steel sheet pile works (Type F)

**－ During construction －**

<Cantilevered sheet pile wall>

1　Design conditions

1-1　Structural figure




1-2　Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 8.00 | m |
| Protruction lemgth | H0= | 4.50 | |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 4.50 | m (Regular condition) |
| Outside water level | Lwp= | 4.50 | m (Regular condition) |

1-3 Calculation method

Calculation method Chang's formula

Footing depth $L = \frac{\pi}{\beta}$

1-4 Design constant parameters

Unit weight of water $\gamma w =$ 10.0 $kN/m^3$

Type of water pressure; trapezoid

1-5 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N': Average N-value

1-6 Overburden pressure

Active pressure side Qa= 10.0 $kN/m^2$ (Regular condition)

Passive pressure side Qp= 0.0 $kN/m^2$ (Regular condition)

1-7 Soil constant

| No. | Depth<br>m | Layer thickness<br>m |
|---|---|---|
| 1 | 8.00 | 3.50 |
| 2 | 16.30 | 8.30 |
| 3 | 30.00 | 13.70 |

| No | Depth<br>m | type | N-value | γ<br>$kN/m^3$ | γ'<br>$kN/m^3$ | φ<br>Deg. | C<br>$kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 8.00 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 16.30 | S | 17 | 19.0 | 10.0 | 29.0 | 0.0 | 0.0 |
| 3 | 30.00 | S | 36 | 19.0 | 10.0 | 36.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

1-8 Sheet pile

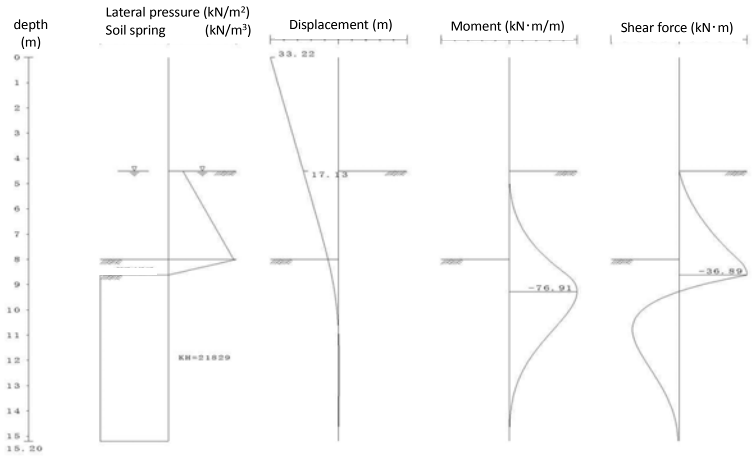
Type PU18 (S270GP)
Young modulus E＝ 200,000 $N/mm^2$
Second moment of area I0＝ 38650 $cm^4$ (before corrosion)
Modulus of section Z0＝ 1800 $cm^3$ (before corrosion)

Corrosion allowance t1＝ 0.00 mm (front side) t2＝ 0.00 mm (back side)
Corrosion ratio (againstI0) η＝ 1.00
Corrosion ratio (against Z0) η＝ 1.00
Joint efficiency (against I0) μ＝ 1.00
Joint efficiency(againstZ0) μ＝ 1.00

Allowable stress σa = 165 $N/mm^2$ (Regular condition)

Allowable displacement δa ＝ 50.0 mm (Regular condition)

2 Section force and Displacement




3 Calculation results

Front sheet pile PU18 (S270GP)

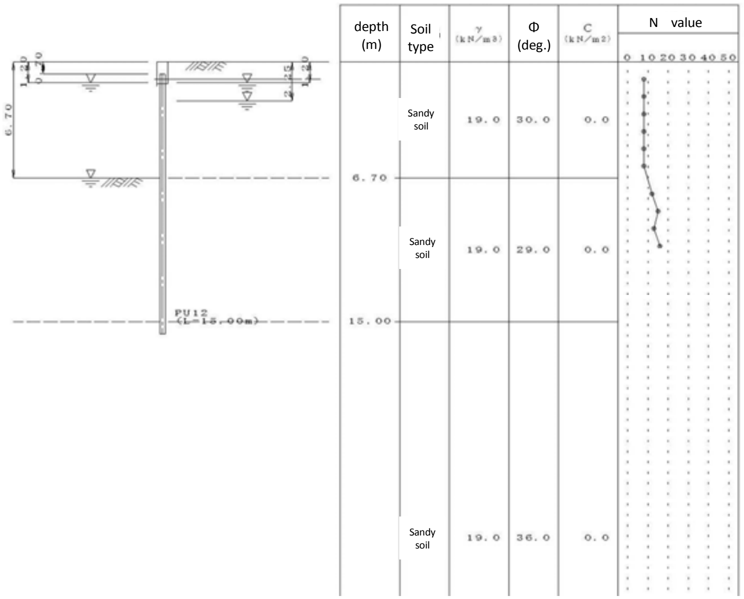
| | | | | Regular condition | |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 38650 | | |
| Modulus of section | Z | ($cm^3$) | 1800 | | |
| Max. bending moment | Mmax | (kN･m/m) | | 76.91 | |
| Stress intensity | σ | ($N/mm^2$) | | 43 (165) | |
| Horizontal displacement | δ | (mm) | | 17.13 (50.0) | |
| Footing depth | D | (m) | | 6.71 | |
| Total length | L | (m) | 14.50 | | |

A-8-13 Structural calculation of steel sheet pile works (Type G-1)

<Bracing style sheet pile wall>

1 <u>Design conditions</u>

1-1 <u>Structural figure</u>




1-2 <u>Structural dimensions</u>

| | | | |
|---|---|---|---|
| Design height | H= | 6.70 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 2.25 | m (Regular condition) |
| | Lwa'= | 1.20 | m (Seismic condition) |
| Outside water level | Lwp= | 6.70 | m (Regular condition) |
| | Lwp'= | 1.20 | m (Seismic condition) |
| Tie-rod installation position | Ht= | 1.00 | m |
| Tie-rod horizontal intervals | l= | 2.40 | m |
| Tie-rod installation angle | θ = | 0.0000 | degree |

Method of calculation; Free earth support method

Virtual supporting point; Design front ground level

## 1-3 Design constant parameters

Unit weight of water $\gamma w =$ 10.0 $kN/m^3$

Type of water pressure; trapezoid

Design Seismic coefficient (in the air) k= 0.080

(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma\gamma_t \cdot h_i + \Sigma\gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma\gamma_t \cdot h_i + \Sigma(\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake; Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma\gamma h + 2Q}{2C} \cdot \tan\theta}$$

Safety factor of footing depth; 1.50 （Regular condition）
1.20 （Seismic condition）

## 1-4 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N’: Average N-value

Result of boring survey

| No | Depth m | N-value |
|---|---|---|
| 1 | 1.00 | 8 |
| 2 | 2.00 | 8 |
| 3 | 3.00 | 8 |
| 4 | 4.00 | 8 |
| 5 | 5.00 | 8 |
| 6 | 6.00 | 8 |
| 7 | 7.60 | 12 |
| 8 | 8.60 | 15 |
| 9 | 9.60 | 13 |
| 10 | 10.60 | 16 |

| No | Depth m | N-value |
|---|---|---|
| | | |

## 1-5 Overburden pressure

Active pressure side Qa= 10.0 $kN/m^2$ (Regular condition)
Qa'= 5.0 $kN/m^2$ (Seismic condition)

Passive pressure side Qp= 0.0 $kN/m^2$ (Regular condition)
Qp'= 0.0 $kN/m^2$ (Seismic condition)

## 1-6 Soil constant

| No | Depth m | type | N-value | γ $kN/m^3$ | γ' $kN/m^3$ | φ Deg. | C $kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 6.70 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 15.00 | S | 17 | 19.0 | 10.0 | 29.0 | 0.0 | 0.0 |
| 3 | 40.00 | S | 36 | 19.0 | 10.0 | 36.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　 C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M) a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

## 1-7 Sheet pile

Type PU12 (S270GP)
Young modulus E ＝ 200,000 $N/mm^2$
Second moment of area I0＝ 21,600 $cm^4$（before corrosion）
Modulus of section Z0＝ 1,200 $cm^3$（before corrosion）

Corrosion allowance t1＝ 0.00 mm (front side) t2＝ 0.00 mm (back side)

Allowable stress σa = 165 $N/mm^2$（Regular condition）
σa' = 190 $N/mm^2$（Seismic condition）

## 1-8 Tie-rod

Diameter of tie-rod calculated

Corrosion considered（0.00mm）

Allowable stress σa = 176 $N/mm^2$（Regular condition）
σa' = 264 $N/mm^2$（Seismic condition）

## 1-9 Waling

Type 2×UPN-200×75

Modulus of section; 382 cm3

Corrosion ; considered（0.00mm）
Corrosion ratio; η ＝ 1.00

Allowable stress σa = 140 $N/mm^2$（Regular condition）
σa' = 161 $N/mm^2$（Seismic condition）

## 1-10 Bracing sheet pile

Calculation method Chang's formula

Crest elevation of bracing works 0.30 m (distance from tie-rod)

| | | | |
|---|---|---|---|
| Materials | PU12 (S270GP) | | |
| Young's modulus | E = | 200,000 | $N/mm^2$ |
| Second moment of area | I0= | 21,600 | $cm^4$ (before corrosion) |
| Modulus of section | Z0= | 1,200 | $cm^3$ (before corrosion) |

| | | | | | |
|---|---|---|---|---|---|
| Corrosion allowance | t1= | 0.00 mm (front side) | t2= | 0.00 | mm (back side) |
| Corrosion ratio (againstI0) | η= | 1.00 | | | |
| Corrosion ratio (against Z0) | η= | 1.00 | | | |
| Joint efficiency (against I0) | μ= | 1.00 | | | |
| Joint efficiency(againstZ0) | μ= | 1.00 | | | |

| | | | |
|---|---|---|---|
| Allowable stress | σa = | 165 | $N/mm^2$ (Regular condition) |
| | σa' = | 190 | $N/mm^2$ (Seismic condition) |
| Allowable displacement | δa = | 50.0 | mm (Regular condition) |
| | δa'= | 75.0 | mm (Seismic condition) |

## 2 Calculation results

Front sheet pile PU12 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 21600 | | |
| Modulus of section | Z | ($cm^3$) | 1200 | | |
| Max. bending moment | Mmax | (kN･m/m) | | 161.85 | 90.22 |
| Stress intensity | σ | ($N/mm^2$) | | 135 (165) | 75 (190) |
| Footing depth | D | (m) | | 8.92 | 4.97 |
| Total length | L | (m) | 15.00 | | |

Tie-rod φ 42 mm

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Tensile force | Tp | (kN) | | 208.46 | 139.53 |
| Stress intensity | σ | ($N/mm^2$) | | 150 (176) | 101 (264) |

Waling 2×UPN-200×75

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Modulus of section | Z | (cm3) | 382 | | |
| Max. bending moment | Mmax | (kN･m) | | 50.03 | 33.49 |
| Stress intensity | σ | ($N/mm^2$) | | 131 (140) | 88 (161) |

Bracing sheet pile PU12 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 21,600 | | |
| Modulus of section | Z | ($cm^3$) | 1,200 | | |
| Max. bending moment | Mmax | (kN･m) | | 50.71 | 33.94 |
| Stress intensity | σ | ($N/mm^2$) | | 42 (165) | 28 (190) |
| Horizontal displacement | δ | (mm) | | 5.97 (50.0) | 3.99 (75.0) |
| Footing depth | D | (m) | | 5.43 | 5.43 |
| Total length | L | (m) | 6.00 | | |

Bracing works Installation position

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Bracing works Installation position | d | (m) | | 6.57 | 7.65 |

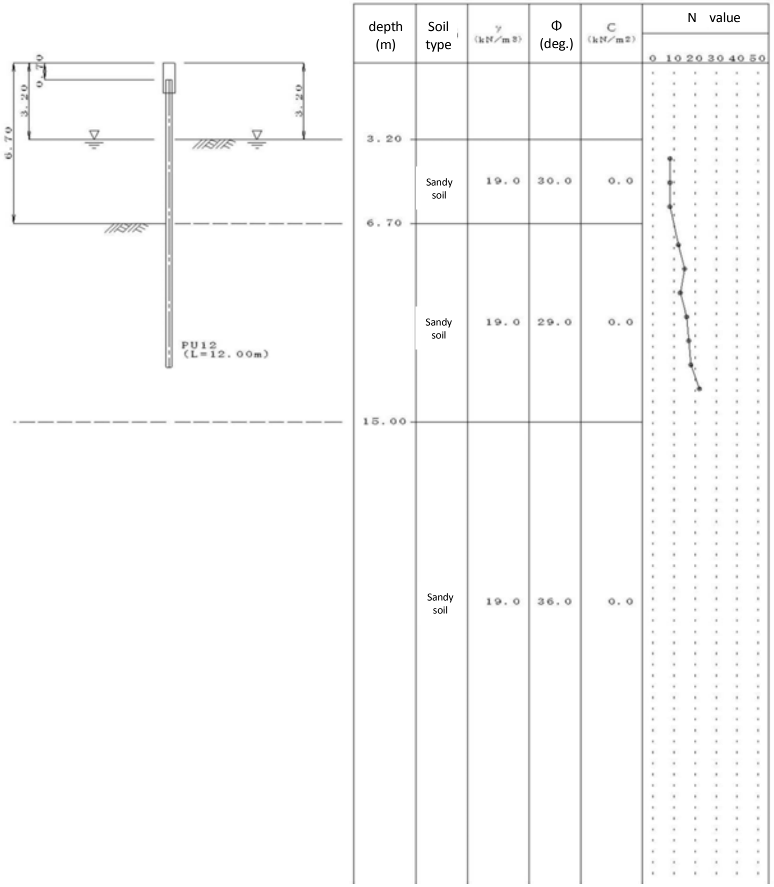
Structural calculation of steel sheet pile works (Type G-1)

**－ During construction －**

<Cantilevered sheet pile wall>

1　Design conditions

1-1　Structural figure




1-2　Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 6.70 | m |
| Protruction lemgth | H0= | 3.20 | |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 3.20 | m (Regular condition) |
| Outside water level | Lwp= | 3.20 | m (Regular condition) |

1-3 Calculation method

Calculation method Chang's formula

Footing depth $L = \frac{\pi}{\beta}$

1-4 Design constant parameters

Unit weight of water γw＝ 10.0 kN/m$^3$

Type of water pressure; trapezoid

1-5 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N': Average N-value

1-6 Overburden pressure

Active pressure side Qa= 10.0 kN/m$^2$ (Regular condition)

Passive pressure side Qp= 0.0 kN/m$^2$ (Regular condition)

1-7 Soil constant

| No. | Depth m | Layer thickness m |
|---|---|---|
| 1 | 6.70 | 3.50 |
| 2 | 15.00 | 8.30 |
| 3 | 30.00 | 15.00 |

| No | Depth m | type | N-value | γ kN/m$^3$ | γ' kN/m$^3$ | φ Deg. | C kN/m$^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 6.70 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 15.00 | S | 17 | 19.0 | 10.0 | 29.0 | 0.0 | 0.0 |
| 3 | 30.00 | S | 36 | 19.0 | 10.0 | 36.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M) a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

1-8 Sheet pile

| | | | |
|---|---|---|---|
| Type | PU12 (S270GP) | | |
| Young modulus | E ＝ | 200,000 | N/mm$^2$ |
| Second moment of area | I0＝ | 21600 | cm$^4$（before corrosion） |
| Modulus of section | Z0＝ | 1200 | cm$^3$（before corrosion） |

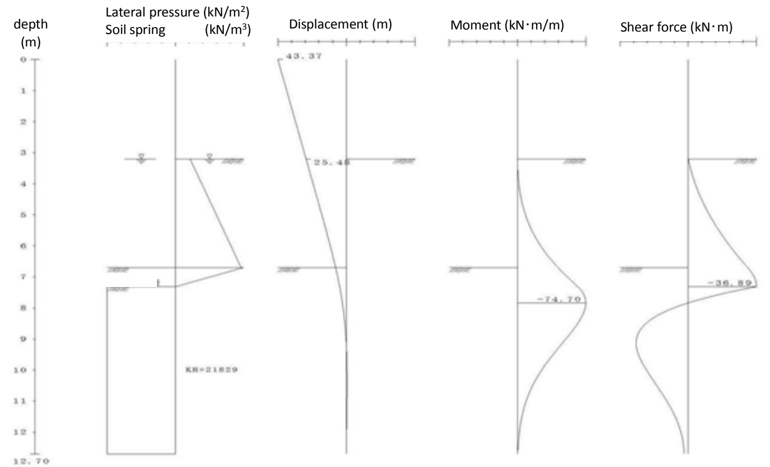
Corrosion allowance t1＝ 0.00 mm (front side) t2＝ 0.00 mm (back side)

| | | |
|---|---|---|
| Corrosion ratio (againstI0) | η＝ | 1.00 |
| Corrosion ratio (against Z0) | η＝ | 1.00 |
| Joint efficiency (against I0) | μ＝ | 1.00 |
| Joint efficiency(againstZ0) | μ＝ | 1.00 |

Allowable stress σa = 165 N/mm$^2$（Regular condition）

Allowable displacement δa ＝ 50.0 mm（Regular condition）

2 Section force and Displacement




3 Calculation results

Front sheet pile PU12 (S270GP)

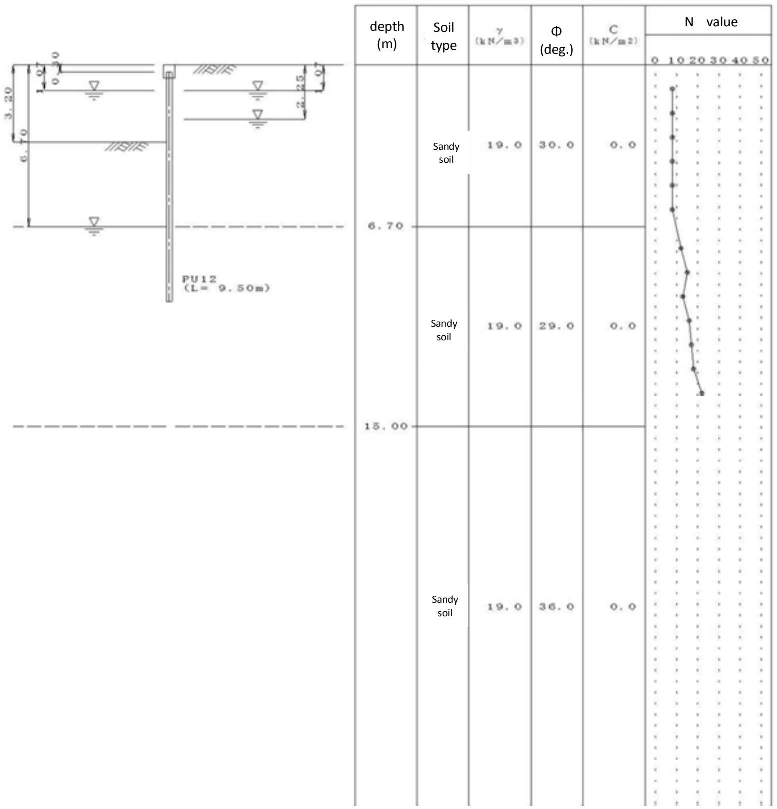
| | | | Regular condition | |
|---|---|---|---|---|
| Second moment of area | I (cm$^4$) | 21600 | | |
| Modulus of section | Z (cm$^3$) | 1200 | | |
| Max. bending moment | Mmax (kN･m/m) | | 74.7 | |
| Stress intensity | σ (N/mm$^2$) | | 62 (165) | |
| Horizontal displacement | δ (mm) | | 25.48 (50.0) | |
| Footing depth | D (m) | | 5.89 | |
| Total length | L (m) | 12.00 | | |

# A-8-14 Structural calculation of steel sheet pile works (Type G-2)

## <Cantilevered sheet pile wall>

1 Design conditions

1-1 Structural figure




1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 3.20 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.30 | m |
| Inside water level | Lwa= | 2.25 | m (Regular condition) |
| | Lwa'= | 1.07 | m (Seismic condition) |
| Outside water level | Lwp= | 6.70 | m (Regular condition) |
| | Lwp'= | 1.07 | m (Seismic condition) |

## 1-3 Calculation method

Calculation method Chang's formula

Footing depth $L = \dfrac{\pi}{\beta}$

## 1-4 Design constant parameters

Unit weight of water $\gamma w =$ 10.0 kN/m$^3$

Type of water pressure; trapezoid

Design Seismic coefficient (in the air) k= 0.080

(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma \gamma_t \cdot h_i + \Sigma \gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma \gamma_t \cdot h_i + \Sigma(\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake; Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma \gamma h + 2Q}{2C} \cdot \tan\theta}$$

$K_c =$ 0.50

## 1-5 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N': Average N-value

## 1-6 Overburden pressure

| | | | |
|---|---|---|---|
| Active pressure side | Qa= | 10.0 | kN/m$^2$ (Regular condition) |
| | Qa'= | 5.0 | kN/m$^2$ (Seismic condition) |
| Passive pressure side | Qp= | 0.0 | kN/m$^2$ (Regular condition) |
| | Qp'= | 0.0 | kN/m$^2$ (Seismic condition) |

## 1-7 Soil constant

| No. | Depth m | Layer thickness m |
|---|---|---|
| 1 | 6.70 | 6.70 |
| 2 | 15.00 | 8.30 |
| 3 | 30.00 | 15.00 |

| No | Depth m | type | N-value | γ kN/m$^3$ | γ' kN/m$^3$ | φ Deg. | C kN/m$^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 6.70 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 15.00 | S | 17 | 19.0 | 10.0 | 29.0 | 0.0 | 0.0 |
| 3 | 30.00 | S | 36 | 19.0 | 10.0 | 36.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M) a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

## 1-8 Sheet pile

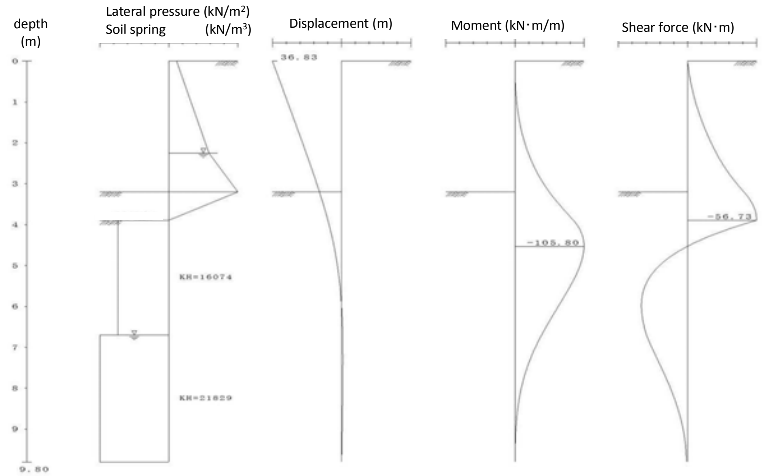
| | | | |
|---|---|---|---|
| Type | PU12 (S270GP) | | |
| Young modulus | E = | 200,000 | N/mm$^2$ |
| Second moment of area | I0= | 21600 | cm$^4$ (before corrosion) |
| Modulus of section | Z0= | 1200 | cm$^3$ (before corrosion) |

Corrosion allowance t1＝ 0.00 mm (front side) t2＝ 0.00 mm (back side)

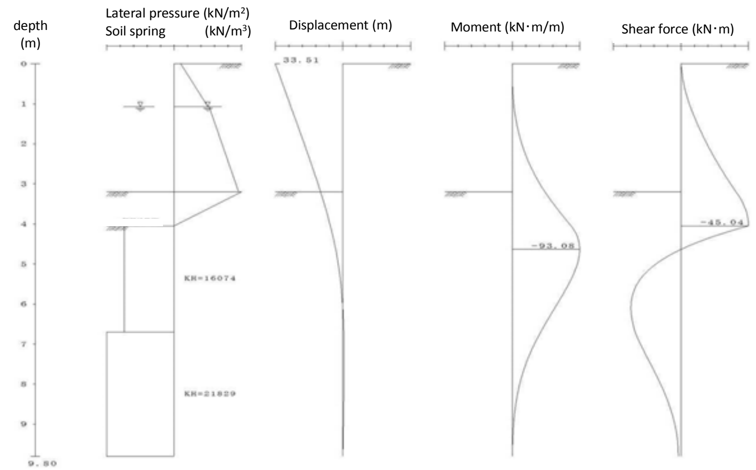
| | | | |
|---|---|---|---|
| Corrosion ratio (againstI0) | η= | 1.00 | |
| Corrosion ratio (against Z0) | η= | 1.00 | |
| Joint efficiency (against I0) | μ= | 1.00 | |
| Joint efficiency(againstZ0) | μ= | 1.00 | |
| Allowable stress | σa = | 165 | N/mm$^2$ (Regular condition) |
| | σa' = | 190 | N/mm$^2$ (Seismic condition) |
| Allowable displacement | δa = | 50.0 | mm (Regular condition) |
| | δa'= | 75.0 | mm (Seismic condition) |

2 Section force and Displacement

(Regular condition)




(Seismic condition)




3 Calculation results

Front sheet pile PU12 (S270GP)

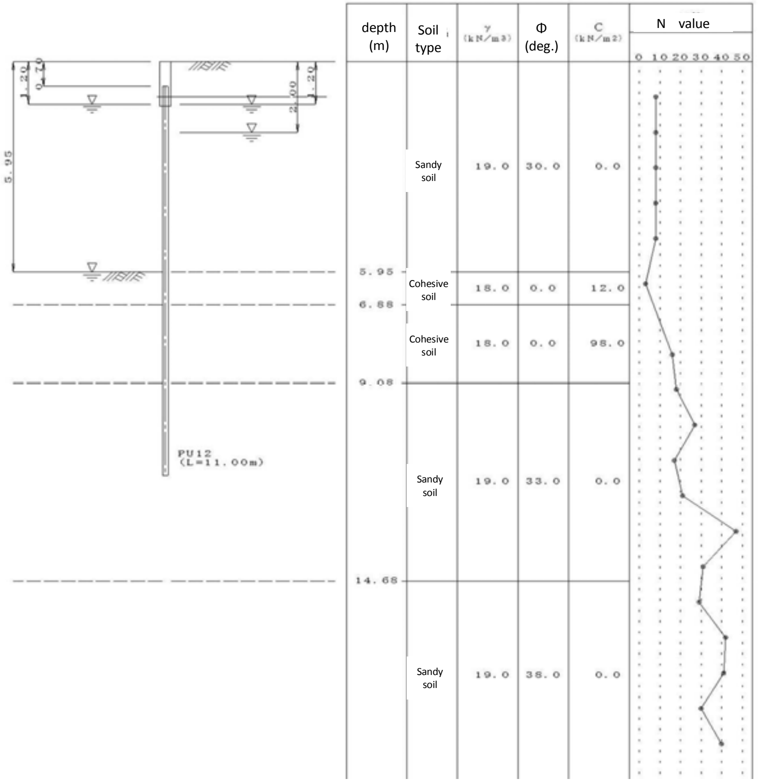
| | | | Regular condition | Seismic condition |
|---|---|---|---|---|
| Second moment of area | I ($cm^4$) | 21600 | | |
| Modulus of section | Z ($cm^3$) | 1200 | | |
| Max. bending moment | Mmax (kN·m/m) | | 106.8 | 93.08 |
| Stress intensity | σ ($N/mm^2$) | | 88 (165) | 78 (190) |
| Horizontal displacement | δ (mm) | | 36.83 (50.0) | 33.51 (75.0) |
| Footing depth | D (m) | | 6.17 | 6.31 |
| Total length | L (m) | 9.50 | | |

# A-8-15 Structural calculation of steel sheet pile works (Type H)

## <Bracing style sheet pile wall>

## 1 Design conditions

### 1-1 Structural figure



## 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design height | H= | 5.95 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 2.00 | m (Regular condition) |
| | Lwa'= | 1.20 | m (Seismic condition) |
| Outside water level | Lwp= | 5.95 | m (Regular condition) |
| | Lwp'= | 1.20 | m (Seismic condition) |
| Tie-rod installation position | Ht= | 1.00 | m |
| Tie-rod horizontal intervals | l= | 2.40 | m |
| Tie-rod installation angle | θ = | 0.0000 | degree |

Method of calculation; Free earth support method

Virtual supporting point; Design front ground level

## 1-3 Design constant parameters

Unit weight of water γw＝ 10.0 $kN/m^3$

Type of water pressure; trapezoid

Design Seismic coefficient (in the air) k= 0.080

(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma\gamma_t \cdot h_i + \Sigma\gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma\gamma_t \cdot h_i + \Sigma(\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake; Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma\gamma h + 2Q}{2C} \cdot \tan\theta}$$

Safety factor of footing depth; 1.50 (Regular condition)

1.20 (Seismic condition)

## 1-4 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N': Average N-value

Result of boring survey

| No | Depth m | N-value |
|---|---|---|
| 1 | 1.00 | 8 |
| 2 | 2.00 | 8 |
| 3 | 3.00 | 8 |
| 4 | 4.00 | 8 |
| 5 | 5.00 | 8 |
| 6 | 6.28 | 3 |
| 7 | 8.28 | 16 |
| 8 | 9.28 | 18 |
| 9 | 10.28 | 27 |
| 10 | 11.28 | 17 |

| No | Depth m | N-value |
|---|---|---|
| 11 | 12.28 | 21 |
| 12 | 13.28 | 47 |
| 13 | 14.28 | 31 |
| 14 | 15.28 | 29 |
| 15 | 16.28 | 42 |
| 16 | 17.28 | 41 |
| 17 | 18.28 | 30 |
| 18 | 19.28 | 40 |
| 19 | 20.28 | 40 |

## 1-5 Overburden pressure

Active pressure side Qa= 10.0 $kN/m^2$ (Regular condition)

Qa'= 5.0 $kN/m^2$ (Seismic condition)

Passive pressure side Qp= 0.0 $kN/m^2$ (Regular condition)

Qp'= 0.0 $kN/m^2$ (Seismic condition)

## 1-6 Soil constant

| No | Depth m | type | N-value | γ $kN/m^3$ | γ' $kN/m^3$ | φ Deg. | C $kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 5.95 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 6.88 | C | 2 | 18.0 | 9.0 | 0.0 | 12.0 | 0.0 |
| 3 | 9.08 | C | 16 | 18.0 | 9.0 | 0.0 | 98.0 | 0.0 |
| 4 | 14.68 | S | 27 | 19.0 | 10.0 | 33.0 | 0.0 | 0.0 |
| 5 | 20.00 | S | 44 | 19.0 | 10.0 | 38.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force

type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force

Ｎvalue：average value

γ ：unit weight of humid soil

γ'：unit weight of soil under the water

φ ：internal friction angle

## 1-7 Sheet pile

| | | | |
|---|---|---|---|
| Type | PU12 (S270GP) | | |
| Young modulus | Ｅ＝ | 200,000 | $N/mm^2$ |
| Second moment of area | I0＝ | 21,600 | $cm^4$ (before corrosion) |
| Modulus of section | Z0＝ | 1,200 | $cm^3$ (before corrosion) |

Corrosion allowance t1＝ 0.00 mm (front side) t2＝ 0.00 mm (back side)

Allowable stress σa = 165 $N/mm^2$ (Regular condition)

σa' = 190 $N/mm^2$ (Seismic condition)

## 1-8 Tie-rod

Diameter of tie-rod calculated

Corrosion considered (0.00mm)

Allowable stress σa = 176 $N/mm^2$ (Regular condition)

σa' = 264 $N/mm^2$ (Seismic condition)

1-9 Waling

Type 2×UPN-180×70

Modulus of section; 300 cm3

Corrosion ; considered（0.00mm）
Corrosion ratio; η ＝ 1.00

Allowable stress σa = 140 N/mm$^2$（Regular condition）
σa' = 161 N/mm$^2$（Seismic condition）

1-10 Bracing sheet pile

Calculation method Chang's formula

Crest elevation of bracing works 0.30 m (distance from tie-rod）

Materials PU12 (S270GP)
Young's modulus E ＝ 200,000 N/mm$^2$
Second moment of area I0＝ 21,600 cm$^4$（before corrosion）
Modulus of section Z0＝ 1,200 cm$^3$（before corrosion）

Corrosion allowance t1＝ 0.00 mm (front side) t2＝ 0.00 mm (back side)
Corrosion ratio (againstI0) η＝ 1.00
Corrosion ratio (against Z0) η＝ 1.00
Joint efficiency (against I0) μ＝ 1.00
Joint efficiency(againstZ0) μ＝ 1.00

Allowable stress σa = 165 N/mm$^2$（Regular condition）
σa' = 190 N/mm$^2$（Seismic condition）

Allowable displacement δa ＝ 50.0 mm（Regular condition）
δa'＝ 75.0 mm（Seismic condition）

2 Calculation results

Front sheet pile PU12 (S270GP)

| | | | Regular condition | Seismic condition |
|---|---|---|---|---|
| Second moment of area | I (cm$^4$) | 21600 | | |
| Modulus of section | Z (cm$^3$) | 1200 | | |
| Max. bending moment | Mmax (kN･m/m) | | 111.06 | 62.65 |
| Stress intensity | σ (N/mm$^2$) | | 93 (165) | 52 (190) |
| Footing depth | D (m) | | 5.29 | 1.58 |
| Total length | L (m) | 11.00 | | |

Tie-rod φ 42 mm

| | | | Regular condition | Seismic condition |
|---|---|---|---|---|
| Tensile force | Tp (kN) | | 171.27 | 116.52 |
| Stress intensity | σ (N/mm$^2$) | | 151 (176) | 103 (264) |

Waling 2×UPN-180×70

| | | | Regular condition | Seismic condition |
|---|---|---|---|---|
| Modulus of section | Z (cm3) | 300 | | |
| Max. bending moment | Mmax (kN･m) | | 41.11 | 27.96 |
| Stress intensity | σ (N/mm$^2$) | | 137 (140) | 93 (161) |

Bracing sheet pile PU12 (S270GP)

| | | | Regular condition | Seismic condition |
|---|---|---|---|---|
| Second moment of area | I (cm$^4$) | 21,600 | | |
| Modulus of section | Z (cm$^3$) | 1,200 | | |
| Max. bending moment | Mmax (kN･m) | | 41.66 | 28.34 |
| Stress intensity | σ (N/mm$^2$) | | 35 (165) | 24 (190) |
| Horizontal displacement | δ (mm) | | 4.9 (50.0) | 3.34 (75.0) |
| Footing depth | D (m) | | 5.43 | 5.43 |
| Total length | L (m) | 6.00 | | |

Bracing works Installation position

| | | | Regular condition | Seismic condition |
|---|---|---|---|---|
| Bracing works Installation position | d (m) | | 6.14 | 7.08 |

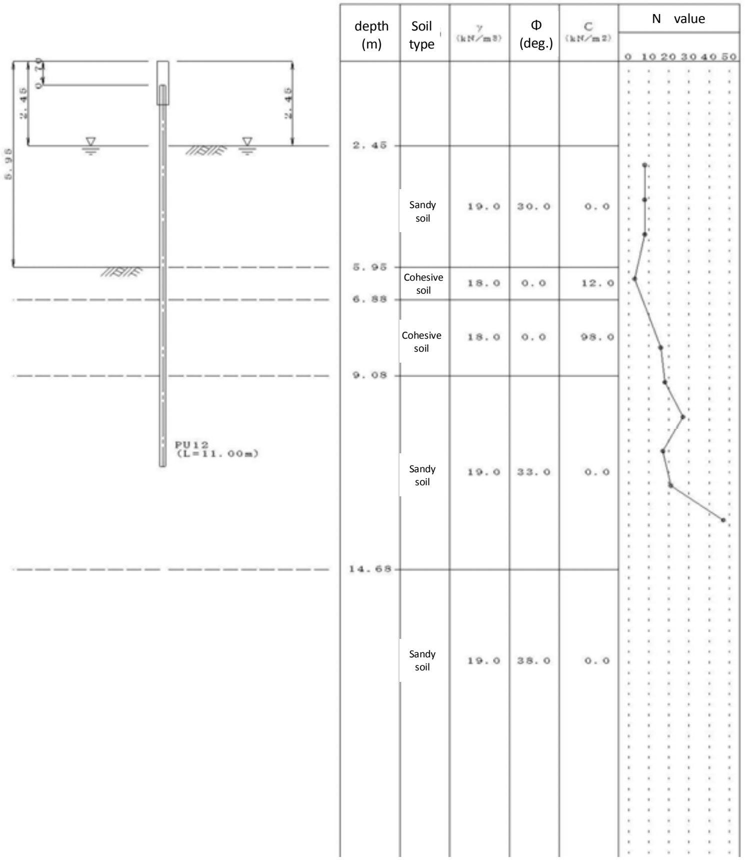
Structural calculation of steel sheet pile works (Type H)

**－ During construction －**

<Cantilevered sheet pile wall>

# 1 Design conditions

## 1-1 Structural figure




## 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 5.95 | m |
| Protruction lemgth | H0= | 2.45 | |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 2.45 | m (Regular condition) |
| Outside water level | Lwp= | 2.45 | m (Regular condition) |

1-3 <u>Calculation method</u>

Calculation method Chang's formula

Footing depth $L = \frac{\pi}{\beta}$

1-4 <u>Design constant parameters</u>

Unit weight of water γw＝ 10.0 kN/m$^3$

Type of water pressure; trapezoid

1-5 <u>Coefficient of horizontal subgrade reaction</u>

Formula: $K_h = 6910 \times N'^{0.406}$

where, N': Average N-value

1-6 <u>Overburden pressure</u>

Active pressure side Qa= 10.0 kN/m$^2$ (Regular condition)

Passive pressure side Qp= 0.0 kN/m$^2$ (Regular condition)

1-7 <u>Soil constant</u>

| No. | Depth<br>m | Layer thickness<br>m |
|---|---|---|
| 1 | 5.95 | 2.00 |
| 2 | 6.88 | 0.93 |
| 3 | 9.08 | 2.20 |
| 4 | 14.68 | 5.60 |
| 5 | 20.00 | 5.32 |

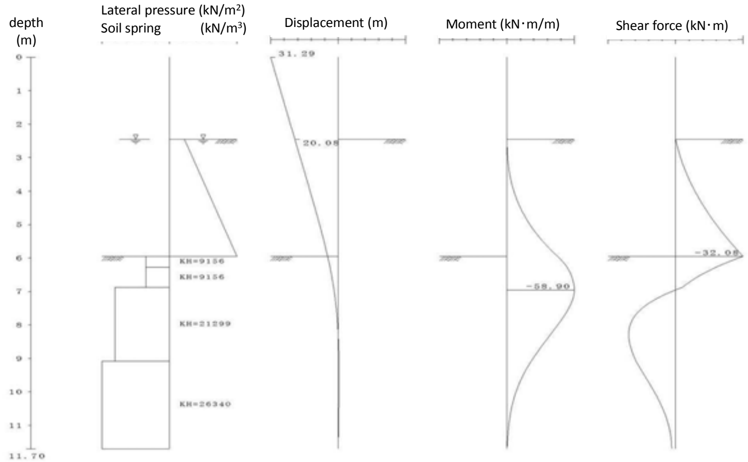
| No | Depth<br>m | type | N-value | γ<br>kN/m$^3$ | γ'<br>kN/m$^3$ | φ<br>Deg. | C<br>kN/m$^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 5.95 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 6.88 | C | 2 | 18.0 | 9.0 | 0.0 | 12.0 | 0.0 |
| 3 | 9.08 | C | 16 | 18.0 | 9.0 | 0.0 | 98.0 | 0.0 |
| 4 | 14.68 | S | 27 | 19.0 | 10.0 | 33.0 | 0.0 | 0.0 |
| 5 | 20.00 | S | 44 | 19.0 | 10.0 | 38.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

1-8 <u>Sheet pile</u>

| | | | |
|---|---|---|---|
| Type | PU12 (S270GP) | | |
| Young modulus | E ＝ | 200,000 | N/mm$^2$ |
| Second moment of area | I0＝ | 21600 | cm$^4$（before corrosion） |
| Modulus of section | Z0＝ | 1200 | cm$^3$（before corrosion） |
| Corrosion allowance | t1＝ | 0.00 | mm (front side)　t2＝ 0.00 mm (back side) |
| Corrosion ratio (againstI0) | η＝ | 1.00 | |
| Corrosion ratio (against Z0) | η＝ | 1.00 | |
| Joint efficiency (against I0) | μ＝ | 1.00 | |
| Joint efficiency(againstZ0) | μ＝ | 1.00 | |
| Allowable stress | σa = | 165 | N/mm$^2$（Regular condition） |
| Allowable displacement | δa ＝ | 50.0 | mm（Regular condition） |

2 <u>Section force and Displacement</u>




3 <u>Calculation results</u>

Front sheet pile PU12 (S270GP)

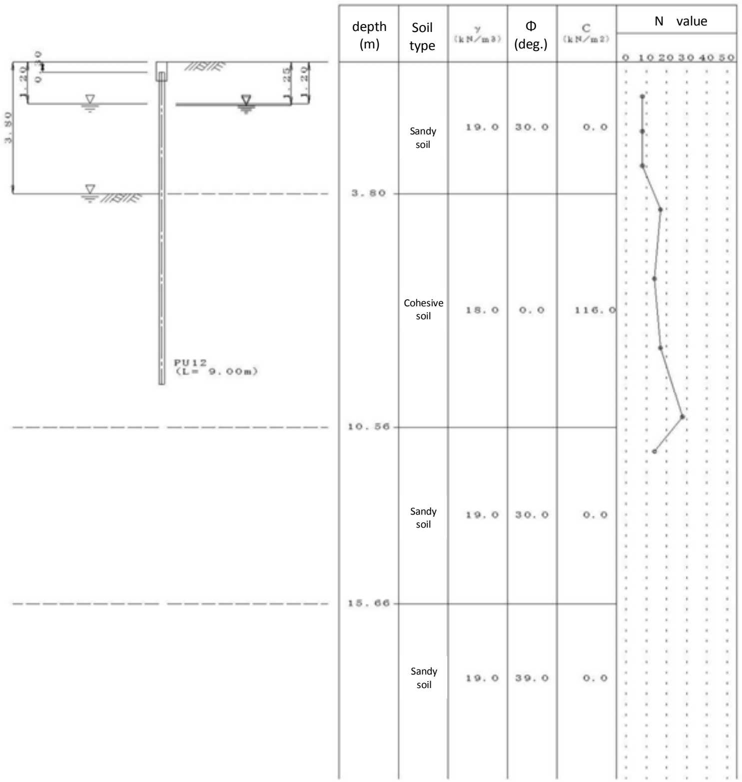
| | | | | Regular condition | |
|---|---|---|---|---|---|
| Second moment of area | I | (cm$^4$) | 21600 | | |
| Modulus of section | Z | (cm$^3$) | 1200 | | |
| Max. bending moment | Mmax | (kN･m/m) | | 58.9 | |
| Stress intensity | σ | (N/mm$^2$) | | 49 (165) | |
| Horizontal displacement | δ | (mm) | | 20.08 (50.0) | |
| Footing depth | D | (m) | | 5.36 | |
| Total length | L | (m) | 11.00 | | |

# A-8-16 Structural calculation of steel sheet pile works (Type I)

## <Cantilevered sheet pile wall>

## 1 Design conditions

### 1-1 Structural figure




### 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 3.80 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.30 | m |
| Inside water level | Lwa= | 1.25 | m (Regular condition) |
| | Lwa'= | 1.20 | m (Seismic condition) |
| Outside water level | Lwp= | 3.80 | m (Regular condition) |
| | Lwp'= | 1.20 | m (Seismic condition) |

## 1-3 Calculation method

Calculation method    Chang's formula

Footing depth    $L = \frac{\pi}{\beta}$

## 1-4 Design constant parameters

Unit weight of water    γw＝    10.0 kN/m$^3$

Type of water pressure;    trapezoid

Design Seismic coefficient (in the air)    k=    0.080
(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma\gamma_t\cdot h_i + \Sigma\gamma\cdot h_j + Q) + \gamma\cdot h}{2(\Sigma\gamma_t\cdot h_i + \Sigma(\gamma-\gamma_w)\cdot h_j + Q) + (\gamma-\gamma_w)\cdot h}\cdot k$$

Dynamic water pressure during earthquake;    Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma\gamma h + 2Q}{2C}\cdot\tan\theta}$$

$K_c$＝    0.50

## 1-5 Coefficient of horizontal subgrade reaction

Formula:    $K_h = 6910 \times N'^{0.406}$
where,    N': Average N-value

## 1-6 Overburden pressure

Active pressure side    Qa=    10.0 kN/m$^2$ (Regular condition)
Qa'=    5.0 kN/m$^2$ (Seismic condition)

Passive pressure side    Qp=    0.0 kN/m$^2$ (Regular condition)
Qp'=    0.0 kN/m$^2$ (Seismic condition)

## 1-7 Soil constant

| No. | Depth<br>m | Layer thickness<br>m |
|---|---|---|
| 1 | 3.80 | 3.80 |
| 2 | 10.56 | 6.76 |
| 3 | 15.66 | 5.10 |
| 4 | 20.00 | 4.34 |

| No | Depth<br>m | type | N-value | γ<br>kN/m$^3$ | γ'<br>kN/m$^3$ | φ<br>Deg. | C<br>kN/m$^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 3.80 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 10.56 | C | 19 | 18.0 | 9.0 | 0.0 | 116.0 | 0.0 |
| 3 | 15.66 | S | 20 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 4 | 20.00 | S | 48 | 19.0 | 10.0 | 39.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile    C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)  a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

## 1-8 Sheet pile

| | | | |
|---|---|---|---|
| Type | PU12 (S270GP) | | |
| Young modulus | E ＝ | 200,000 | N/mm$^2$ |
| Second moment of area | I0＝ | 21600 | cm$^4$ (before corrosion) |
| Modulus of section | Z0＝ | 1200 | cm$^3$ (before corrosion) |

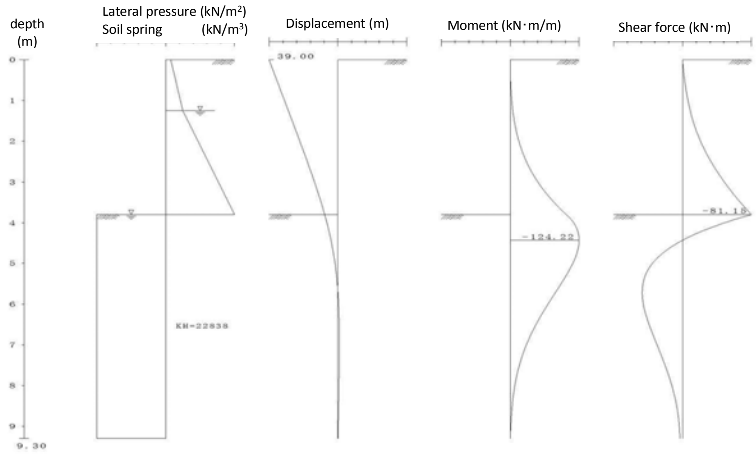
Corrosion allowance    t1＝    0.00 mm (front side)    t2＝    0.00 mm (back side)
Corrosion ratio (againstI0)    η＝    1.00
Corrosion ratio (against Z0)    η＝    1.00
Joint efficiency (against I0)    μ＝    1.00
Joint efficiency(againstZ0)    μ＝    1.00

Allowable stress    σa =    165 N/mm$^2$ (Regular condition)
σa' =    190 N/mm$^2$ (Seismic condition)

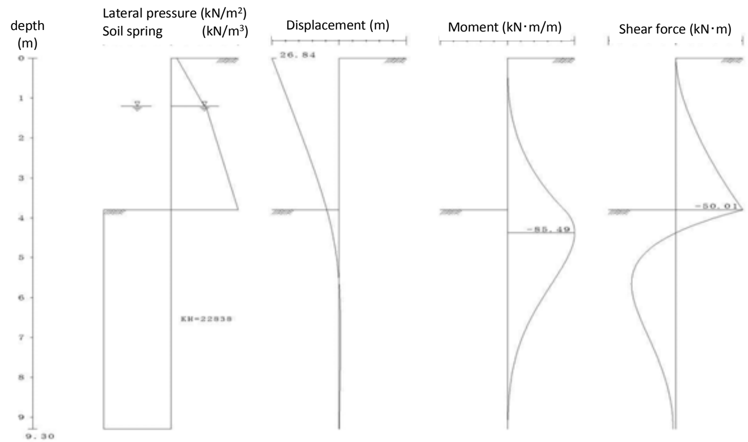
Allowable displacement    δa ＝    50.0 mm (Regular condition)
δa'＝    75.0 mm (Seismic condition)

## 2 Section force and Displacement

(Regular condition)




(Seismic condition)




## 3 Calculation results

Front sheet pile PU12 (S270GP)

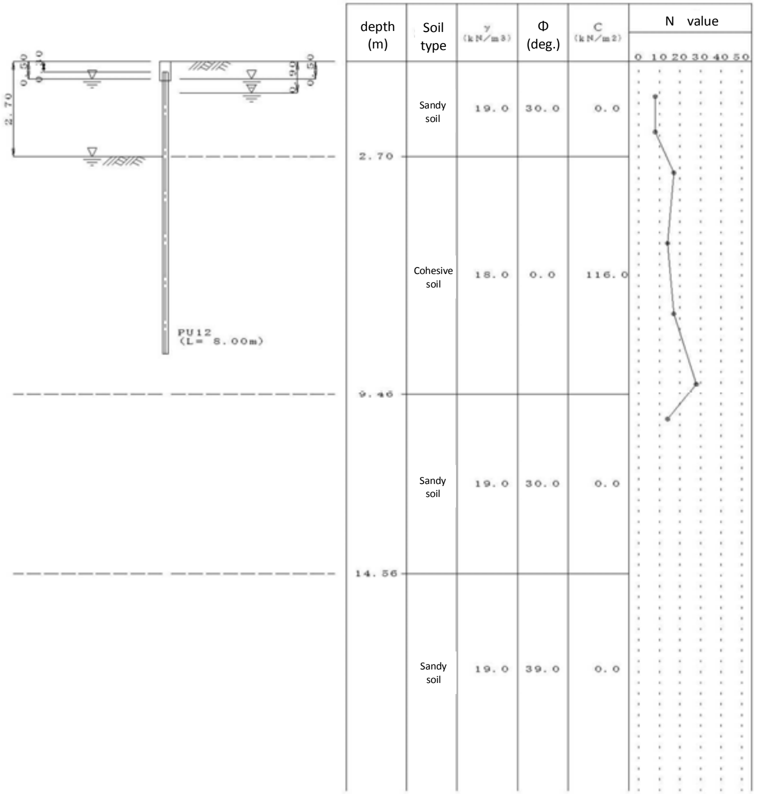
| | | | Regular condition | Seismic condition |
|---|---|---|---|---|
| Second moment of area | I (cm$^4$) | 21600 | | |
| Modulus of section | Z (cm$^3$) | 1200 | | |
| Max. bending moment | Mmax (kN·m/m) | | 124.22 | 85.49 |
| Stress intensity | σ (N/mm$^2$) | | 104 (165) | 71 (190) |
| Horizontal displacement | δ (mm) | | 39 (50.0) | 26.84 (75.0) |
| Footing depth | D (m) | | 5.21 | 5.21 |
| Total length | L (m) | 9.00 | | |

A-8-17 Structural calculation of steel sheet pile works (Type J)

<Cantilevered sheet pile wall>

1 Design conditions

1-1 Structural figure




1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 2.70 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.30 | m |
| Inside water level | Lwa= | 0.90 | m (Regular condition) |
| | Lwa'= | 0.50 | m (Seismic condition) |
| Outside water level | Lwp= | 2.70 | m (Regular condition) |
| | Lwp'= | 0.50 | m (Seismic condition) |

## 1-3 Calculation method

Calculation method Chang's formula

Footing depth $L = \frac{\pi}{\beta}$

## 1-4 Design constant parameters

Unit weight of water $\gamma w =$ 10.0 $kN/m^3$

Type of water pressure; trapezoid

Design Seismic coefficient (in the air) k= 0.080
(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma\gamma_t\cdot h_i + \Sigma\gamma\cdot h_j + Q) + \gamma\cdot h}{2(\Sigma\gamma_t\cdot h_i + \Sigma(\gamma-\gamma_w)\cdot h_j + Q) + (\gamma-\gamma_w)\cdot h}\cdot k$$

Dynamic water pressure during earthquake; Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1-\frac{\Sigma\gamma h + 2Q}{2C}\cdot\tan\theta}$$

$K_c =$ 0.50

## 1-5 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910\times N'^{0.406}$
where, N': Average N-value

## 1-6 Overburden pressure

Active pressure side Qa= 10.0 $kN/m^2$ (Regular condition)
Qa'= 5.0 $kN/m^2$ (Seismic condition)

Passive pressure side Qp= 0.0 $kN/m^2$ (Regular condition)
Qp'= 0.0 $kN/m^2$ (Seismic condition)

## 1-7 Soil constant

| No. | Depth m | Layer thickness m |
|---|---|---|
| 1 | 2.70 | 2.70 |
| 2 | 9.46 | 6.76 |
| 3 | 14.56 | 5.10 |
| 4 | 20.00 | 5.44 |

| No | Depth m | type | N-value | γ $kN/m^3$ | γ' $kN/m^3$ | φ Deg. | C $kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 2.70 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 9.46 | C | 19 | 18.0 | 9.0 | 0.0 | 116.0 | 0.0 |
| 3 | 14.56 | S | 20 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 4 | 20.00 | S | 48 | 19.0 | 10.0 | 39.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M) a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

## 1-8 Sheet pile

| | | | |
|---|---|---|---|
| Type | PU12 (S270GP) | | |
| Young modulus | E ＝ | 200,000 | $N/mm^2$ |
| Second moment of area | I0＝ | 21600 | $cm^4$（before corrosion） |
| Modulus of section | Z0＝ | 1200 | $cm^3$（before corrosion） |

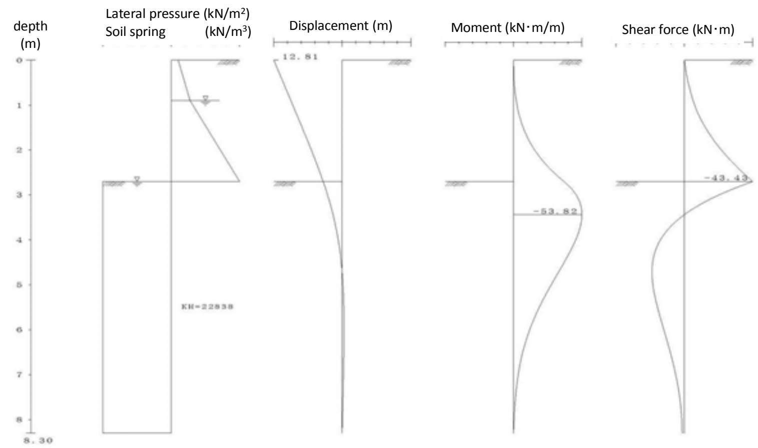
Corrosion allowance t1＝ 0.00 mm (front side) t2＝ 0.00 mm (back side)
Corrosion ratio (againstI0) η＝ 1.00
Corrosion ratio (against Z0) η＝ 1.00
Joint efficiency (against I0) μ＝ 1.00
Joint efficiency(againstZ0) μ＝ 1.00

Allowable stress σa = 165 $N/mm^2$（Regular condition）
σa' = 190 $N/mm^2$（Seismic condition）

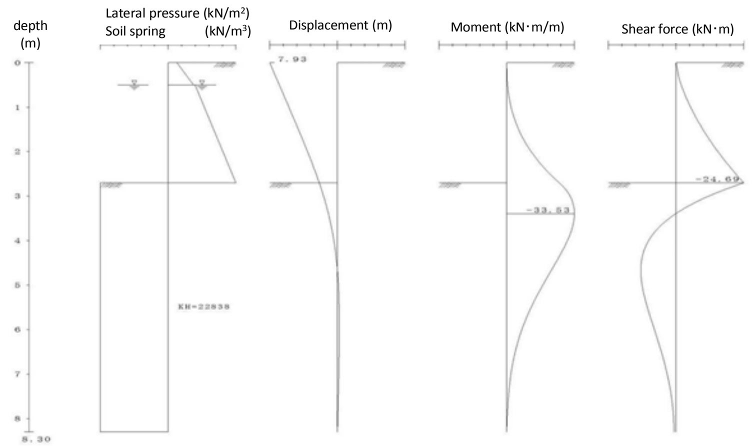
Allowable displacement δa ＝ 50.0 mm（Regular condition）
δa'＝ 75.0 mm（Seismic condition）

2 Section force and Displacement

(Regular condition)




(Seismic condition)




3 Calculation results

Front sheet pile PU12 (S270GP)

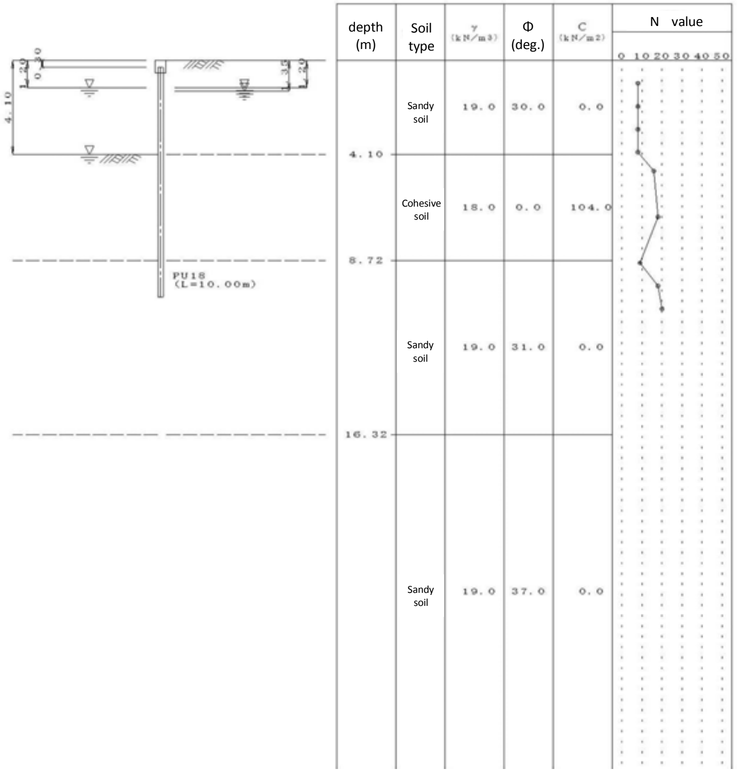
| | | | Regular condition | Seismic condition |
|---|---|---|---|---|
| Second moment of area | I ($cm^4$) | 21600 | | |
| Modulus of section | Z ($cm^3$) | 1200 | | |
| Max. bending moment | Mmax (kN·m/m) | | 53.82 | 33.53 |
| Stress intensity | σ ($N/mm^2$) | | 45 (165) | 28 (190) |
| Horizontal displacement | δ (mm) | | 12.81 (50.0) | 7.93 (75.0) |
| Footing depth | D (m) | | 5.21 | 5.21 |
| Total length | L (m) | 8.00 | | |

A-8-18 Structural calculation of steel sheet pile works (Type K)

<Cantilevered sheet pile wall>

## 1 Design conditions

### 1-1 Structural figure




### 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 4.10 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.30 | m |
| Inside water level | Lwa= | 1.35 | m (Regular condition) |
| | Lwa'= | 1.20 | m (Seismic condition) |
| Outside water level | Lwp= | 4.10 | m (Regular condition) |
| | Lwp'= | 1.20 | m (Seismic condition) |

1-3 Calculation method

Calculation method　　Chang's formula

Footing depth　　$L = \dfrac{\pi}{\beta}$

1-4 Design constant parameters

Unit weight of water　　$\gamma w =$　　10.0　$kN/m^3$

Type of water pressure;　　trapezoid

Design Seismic coefficient (in the air)　　k=　　0.080

(under water)　Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma\gamma_t \cdot h_i + \Sigma\gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma\gamma_t \cdot h_i + \Sigma(\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake;　　Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma\gamma h + 2Q}{2C} \cdot \tan\theta}$$

$K_c =$　　0.50

1-5 Coefficient of horizontal subgrade reaction

Formula:　　$K_h = 6910 \times N'^{0.406}$

where,　N': Average N-value

1-6 Overburden pressure

| | | | |
|---|---|---|---|
| Active pressure side | Qa= | 10.0 | $kN/m^2$ (Regular condition) |
| | Qa'= | 5.0 | $kN/m^2$ (Seismic condition) |
| Passive pressure side | Qp= | 0.0 | $kN/m^2$ (Regular condition) |
| | Qp'= | 0.0 | $kN/m^2$ (Seismic condition) |

1-7 Soil constant

| No. | Depth<br>m | Layer thickness<br>m |
|---|---|---|
| 1 | 4.10 | 4.10 |
| 2 | 8.72 | 4.62 |
| 3 | 16.32 | 7.60 |
| 4 | 30.00 | 13.68 |

| No | Depth<br>m | type | N-value | γ<br>$kN/m^3$ | γ'<br>$kN/m^3$ | φ<br>Deg. | C<br>$kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 4.10 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 8.72 | C | 17 | 18.0 | 9.0 | 0.0 | 104.0 | 0.0 |
| 3 | 16.32 | S | 22 | 19.0 | 10.0 | 31.0 | 0.0 | 0.0 |
| 4 | 30.00 | S | 41 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　　C ：Adhesive force

type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force

Ｎvalue：average value

γ ：unit weight of humid soil

γ'：unit weight of soil under the water

φ ：internal friction angle

1-8 Sheet pile

| | | | |
|---|---|---|---|
| Type | PU18 (S270GP) | | |
| Young modulus | E＝ | 200,000 | $N/mm^2$ |
| Second moment of area | I0＝ | 38650 | $cm^4$ (before corrosion) |
| Modulus of section | Z0＝ | 1800 | $cm^3$ (before corrosion) |

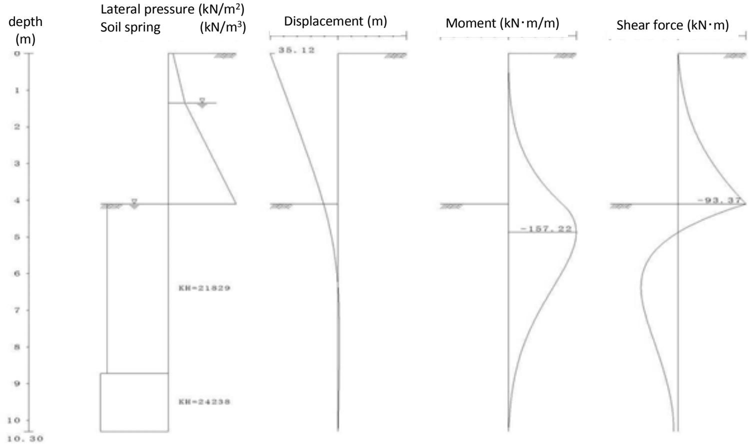
Corrosion allowance　　t1＝　　0.00　mm (front side)　　t2＝　　0.00　mm (back side)

| | | |
|---|---|---|
| Corrosion ratio (againstI0) | η＝ | 1.00 |
| Corrosion ratio (against Z0) | η＝ | 1.00 |
| Joint efficiency (against I0) | μ＝ | 1.00 |
| Joint efficiency(againstZ0) | μ＝ | 1.00 |

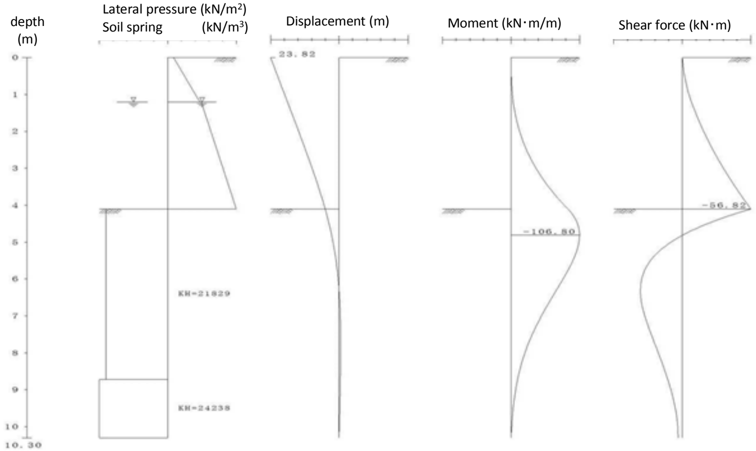
| | | | |
|---|---|---|---|
| Allowable stress | σa = | 165 | $N/mm^2$ (Regular condition) |
| | σa' = | 190 | $N/mm^2$ (Seismic condition) |
| Allowable displacement | δa ＝ | 50.0 | mm (Regular condition) |
| | δa'＝ | 75.0 | mm (Seismic condition) |

2 Section force and Displacement

(Regular condition)




(Seismic condition)




3 Calculation results

Front sheet pile PU18 (S270GP)

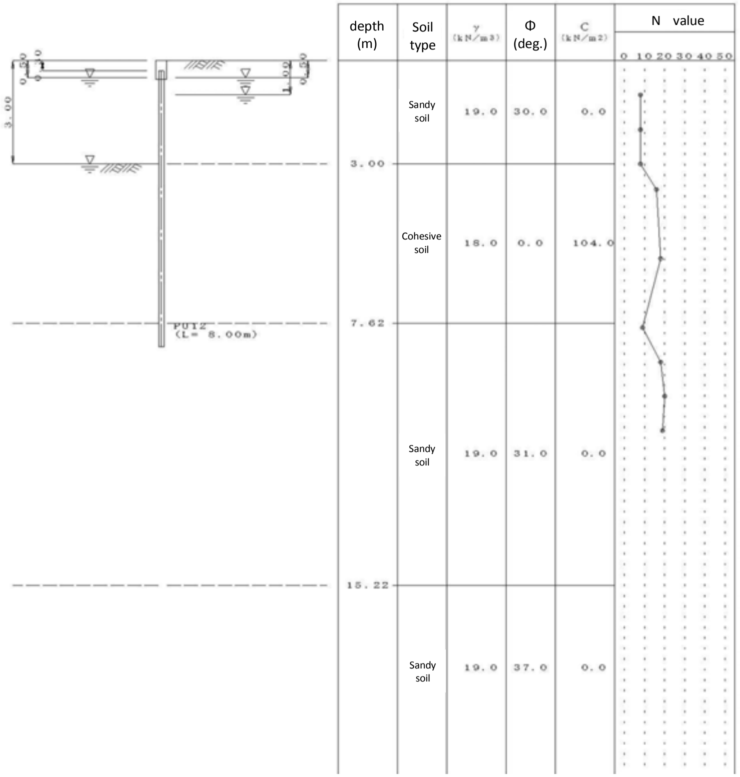
| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | ($cm^4$) | 38650 | | |
| Modulus of section | Z | ($cm^3$) | 1800 | | |
| Max. bending moment | Mmax | (kN·m/m) | | 157.22 | 106.8 |
| Stress intensity | σ | ($N/mm^2$) | | 87 (165) | 59 (190) |
| Horizontal displacement | δ | (mm) | | 35.12 (50.0) | 23.82 (75.0) |
| Footing depth | D | (m) | | 6.06 | 6.06 |
| Total length | L | (m) | 10.00 | | |

# A-8-19 Structural calculation of steel sheet pile works (Type L)

## <Cantilevered sheet pile wall>

1 Design conditions

1-1 Structural figure




1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 3.00 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.30 | m |
| Inside water level | Lwa= | 1.00 | m (Regular condition) |
| | Lwa'= | 0.50 | m (Seismic condition) |
| Outside water level | Lwp= | 3.00 | m (Regular condition) |
| | Lwp'= | 0.50 | m (Seismic condition) |

1-3 Calculation method

Calculation method Chang's formula

Footing depth $L = \frac{\pi}{\beta}$

1-4 Design constant parameters

Unit weight of water $\gamma w =$ 10.0 $kN/m^3$

Type of water pressure; trapezoid

Design Seismic coefficient (in the air) k= 0.080

(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma \gamma_t \cdot h_i + \Sigma \gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma \gamma_t \cdot h_i + \Sigma(\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake; Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma \gamma h + 2Q}{2C} \cdot \tan\theta}$$

$K_c =$ 0.50

1-5 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N': Average N-value

1-6 Overburden pressure

| | | | |
|---|---|---|---|
| Active pressure side | Qa= | 10.0 | $kN/m^2$ (Regular condition) |
| | Qa'= | 5.0 | $kN/m^2$ (Seismic condition) |
| Passive pressure side | Qp= | 0.0 | $kN/m^2$ (Regular condition) |
| | Qp'= | 0.0 | $kN/m^2$ (Seismic condition) |

1-7 Soil constant

| No. | Depth m | Layer thickness m |
|---|---|---|
| 1 | 3.00 | 3.00 |
| 2 | 7.62 | 4.62 |
| 3 | 15.22 | 7.60 |
| 4 | 20.00 | 4.78 |

| No | Depth m | type | N-value | γ $kN/m^3$ | γ' $kN/m^3$ | φ Deg. | C $kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 3.00 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 7.62 | C | 17 | 18.0 | 9.0 | 0.0 | 104.0 | 0.0 |
| 3 | 15.22 | S | 22 | 19.0 | 10.0 | 31.0 | 0.0 | 0.0 |
| 4 | 20.00 | S | 41 | 19.0 | 10.0 | 37.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force

type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force

Ｎvalue：average value

γ ：unit weight of humid soil

γ'：unit weight of soil under the water

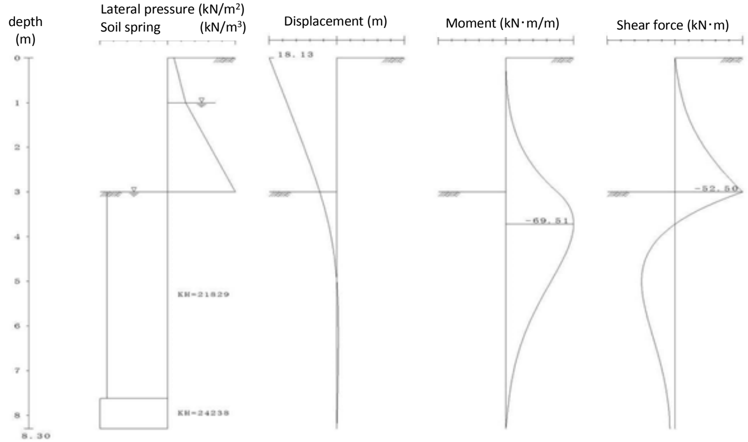
φ ：internal friction angle

1-8 Sheet pile

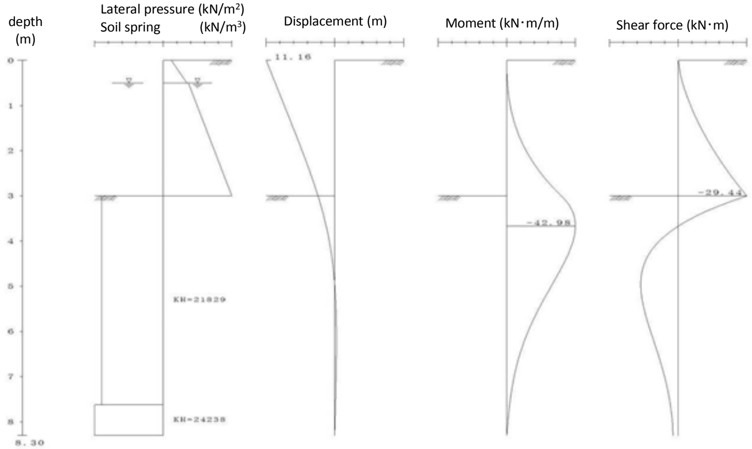
| | | | |
|---|---|---|---|
| Type | PU12 (S270GP) | | |
| Young modulus | E ＝ | 200,000 | $N/mm^2$ |
| Second moment of area | I0＝ | 21600 | $cm^4$ (before corrosion) |
| Modulus of section | Z0＝ | 1200 | $cm^3$ (before corrosion) |
| Corrosion allowance | t1＝ | 0.00 | mm (front side) t2＝ 0.00 mm (back side) |
| Corrosion ratio (againstI0) | η＝ | 1.00 | |
| Corrosion ratio (against Z0) | η＝ | 1.00 | |
| Joint efficiency (against I0) | μ＝ | 1.00 | |
| Joint efficiency(againstZ0) | μ＝ | 1.00 | |
| Allowable stress | σa = | 165 | $N/mm^2$ (Regular condition) |
| | σa' = | 190 | $N/mm^2$ (Seismic condition) |
| Allowable displacement | δa ＝ | 50.0 | mm (Regular condition) |
| | δa'＝ | 75.0 | mm (Seismic condition) |

2 <u>Section force and Displacement</u>

(Regular condition)




(Seismic condition)




3 <u>Calculation results</u>

Front sheet pile PU12 (S270GP)

| | | | Regular condition | Seismic condition |
|---|---|---|---|---|
| Second moment of area | I $(cm^4)$ | 21600 | | |
| Modulus of section | Z $(cm^3)$ | 1200 | | |
| Max. bending moment | Mmax (kN·m/m) | | 69.51 | 42.98 |
| Stress intensity | σ $(N/mm^2)$ | | 58 (165) | 36 (190) |
| Horizontal displacement | δ (mm) | | 18.13 (50.0) | 11.16 (75.0) |
| Footing depth | D (m) | | 5.25 | 5.25 |
| Total length | L (m) | 8.00 | | |

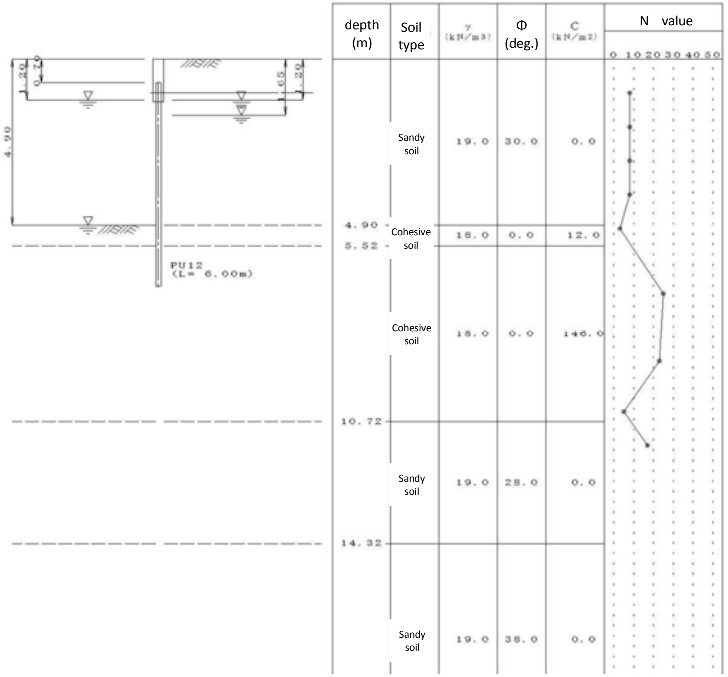
A-8-20 Structural calculation of steel sheet pile works (Type M)

<Bracing style sheet pile wall>



## 1 Design conditions

### 1-1 Structural figure




### 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design height | H= | 4.90 | m |
| Protruction lemgth | H0= | 0.00 | m |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 1.65 | m (Regular condition) |
| | Lwa'= | 1.20 | m (Seismic condition) |
| Outside water level | Lwp= | 4.90 | m (Regular condition) |
| | Lwp'= | 1.20 | m (Seismic condition) |
| | | | |
| Tie-rod installation position | Ht= | 1.00 | m |
| Tie-rod horizontal intervals | l= | 2.40 | m |
| Tie-rod installation angle | θ = | 0.0000 | degree |

Method of calculation; Free earth support method

Virtual supporting point; Design front ground level

## 1-3 Design constant parameters

Unit weight of water γw＝ 10.0 kN/m$^3$

Type of water pressure; trapezoid

Design Seismic coefficient (in the air) k= 0.080

(under water) Following formula proposed by Arai and Yokoi

$$k' = \frac{2(\Sigma\gamma_t \cdot h_i + \Sigma\gamma \cdot h_j + Q) + \gamma \cdot h}{2(\Sigma\gamma_t \cdot h_i + \Sigma(\gamma - \gamma_w) \cdot h_j + Q) + (\gamma - \gamma_w) \cdot h} \cdot k$$

Dynamic water pressure during earthquake; Not considered

Cohesive soil collapse angle during earthquake;

$$\zeta = \tan^{-1}\sqrt{1 - \frac{\Sigma\gamma h + 2Q}{2C} \cdot \tan\theta}$$

Safety factor of footing depth; 1.50 （Regular condition）
1.20 （Seismic condition）

## 1-4 Coefficient of horizontal subgrade reaction

Formula: $K_h = 6910 \times N'^{0.406}$

where, N’: Average N-value

Result of boring survey

| No | Depth m | N-value |
|---|---|---|
| 1 | 1.00 | 8 |
| 2 | 2.00 | 8 |
| 3 | 3.00 | 8 |
| 4 | 4.00 | 8 |
| 5 | 5.00 | 3 |
| 6 | 6.92 | 25 |
| 7 | 8.92 | 23 |
| 8 | 10.42 | 5 |
| 9 | 11.42 | 17 |

| No | Depth m | N-value |
|---|---|---|
| | | |

## 1-5 Overburden pressure

Active pressure side Qa= 10.0 kN/m$^2$ (Regular condition)
Qa'= 5.0 kN/m$^2$ (Seismic condition)

Passive pressure side Qp= 0.0 kN/m$^2$ (Regular condition)
Qp'= 0.0 kN/m$^2$ (Seismic condition)

## 1-6 Soil constant

| No | Depth m | type | N-value | γ kN/m$^3$ | γ' kN/m$^3$ | φ Deg. | C kN/m$^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 4.90 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 5.52 | C | 2 | 18.0 | 9.0 | 0.0 | 12.0 | 0.0 |
| 3 | 10.72 | C | 24 | 18.0 | 9.0 | 0.0 | 146.0 | 0.0 |
| 4 | 14.32 | S | 15 | 19.0 | 10.0 | 28.0 | 0.0 | 0.0 |
| 5 | 20.00 | S | 45 | 19.0 | 10.0 | 38.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile　C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)　a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

## 1-7 Sheet pile

Type PU12 (S270GP)
Young modulus E ＝ 200,000 N/mm$^2$
Second moment of area I0＝ 21,600 cm$^4$（before corrosion）
Modulus of section Z0＝ 1,200 cm$^3$（before corrosion）

Corrosion allowance t1＝ 0.00 mm (front side) t2＝ 0.00 mm (back side)

Allowable stress σa = 165 N/mm$^2$（Regular condition）
σa' = 190 N/mm$^2$（Seismic condition）

## 1-8 Tie-rod

Diameter of tie-rod calculated

Corrosion considered（0.00mm）

Allowable stress σa = 176 N/mm$^2$（Regular condition）
σa' = 264 N/mm$^2$（Seismic condition）

## 1-9 Waling

Type 2×UPN-160×65

Modulus of section; 232 cm3

Corrosion ; considered（0.00mm）
Corrosion ratio; η ＝ 1.00

Allowable stress σa = 140 N/mm$^2$（Regular condition）
σa' = 161 N/mm$^2$（Seismic condition）

## 1-10 Bracing sheet pile

Calculation method　Chang's formula

Crest elevation of bracing works　0.30 m (distance from tie-rod)

| | | | |
|---|---|---|---|
| Materials | PU12 (S270GP) | | |
| Young's modulus | E = | 200,000 | N/mm$^2$ |
| Second moment of area | I0 = | 21,600 | cm$^4$ (before corrosion) |
| Modulus of section | Z0 = | 1,200 | cm$^3$ (before corrosion) |

| | | | | | | |
|---|---|---|---|---|---|---|
| Corrosion allowance | t1 = | 0.00 | mm (front side) | t2 = | 0.00 | mm (back side) |
| Corrosion ratio (againstI0) | η = | 1.00 | | | | |
| Corrosion ratio (against Z0) | η = | 1.00 | | | | |
| Joint efficiency (against I0) | μ = | 1.00 | | | | |
| Joint efficiency(againstZ0) | μ = | 1.00 | | | | |

| | | | |
|---|---|---|---|
| Allowable stress | σa = | 165 | N/mm$^2$ (Regular condition) |
| | σa' = | 190 | N/mm$^2$ (Seismic condition) |
| Allowable displacement | δa = | 50.0 | mm (Regular condition) |
| | δa' = | 75.0 | mm (Seismic condition) |



## 2 Calculation results

Front sheet pile　PU12 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | (cm$^4$) | 21600 | | |
| Modulus of section | Z | (cm$^3$) | 1200 | | |
| Max. bending moment | Mmax | (kN·m/m) | | 59.23 | 34.05 |
| Stress intensity | σ | (N/mm$^2$) | | 49 (165) | 28 (190) |
| Footing depth | D | (m) | | 1.39 | 0.86 |
| Total length | L | (m) | 6.00 | | |

Tie-rod　φ 32 mm

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Tensile force | Tp | (kN) | | 125.69 | 87.75 |
| Stress intensity | σ | (N/mm$^2$) | | 156 (176) | 109 (264) |

Waling　2×UPN-160×65

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Modulus of section | Z | (cm3) | 232 | | |
| Max. bending moment | Mmax | (kN·m) | | 30.17 | 21.06 |
| Stress intensity | σ | (N/mm$^2$) | | 130 (140) | 91 (161) |

Bracing sheet pile　PU12 (S270GP)

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Second moment of area | I | (cm$^4$) | 21,600 | | |
| Modulus of section | Z | (cm$^3$) | 1,200 | | |
| Max. bending moment | Mmax | (kN·m) | | 30.57 | 21.35 |
| Stress intensity | σ | (N/mm$^2$) | | 25 (165) | 18 (190) |
| Horizontal displacement | δ | (mm) | | 3.6 (50.0) | 2.51 (75.0) |
| Footing depth | D | (m) | | 5.43 | 5.43 |
| Total length | L | (m) | 6.00 | | |

Bracing works Installation position

| | | | | Regular condition | Seismic condition |
|---|---|---|---|---|---|
| Bracing works Installation position | d | (m) | | 5.54 | 6.29 |

Structural calculation of steel sheet pile works (Type M)

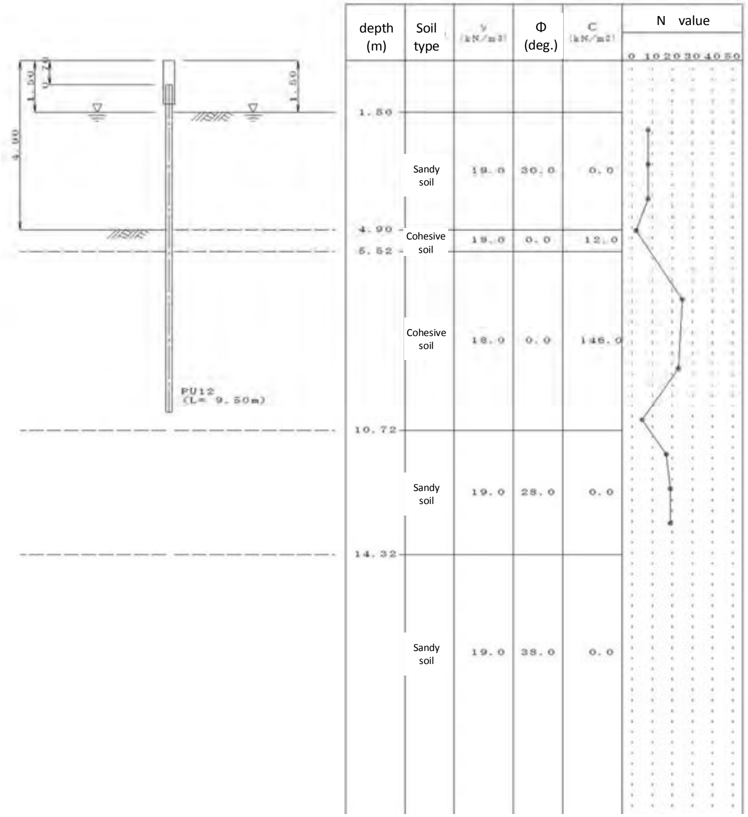
－ **During construction** －

<Cantilevered sheet pile wall>



## 1 Design conditions

### 1-1 Structural figure




### 1-2 Structural dimensions

| | | | |
|---|---|---|---|
| Design base level | H= | 4.90 | m |
| Protruction lemgth | H0= | 1.50 | |
| Top of sheet pile | Hlt= | 0.70 | m |
| Inside water level | Lwa= | 1.50 | m (Regular condition) |
| Outside water level | Lwp= | 1.50 | m (Regular condition) |

## 1-3 Calculation method

Calculation method    Chang's formula

Footing depth    $L = \frac{\pi}{\beta}$

## 1-4 Design constant parameters

Unit weight of water    γw＝    10.0  $kN/m^3$

Type of water pressure;    trapezoid

## 1-5 Coefficient of horizontal subgrade reaction

Formula:    $K_h = 6910 \times N'^{0.406}$

where,  N': Average N-value

## 1-6 Overburden pressure

Active pressure side    Qa=    10.0  $kN/m^2$ (Regular condition)

Passive pressure side    Qp=    0.0  $kN/m^2$ (Regular condition)

## 1-7 Soil constant

| No. | Depth<br>m | Layer thickness<br>m |
|---|---|---|
| 1 | 4.90 | 3.40 |
| 2 | 5.52 | 0.62 |
| 3 | 10.72 | 5.20 |
| 4 | 14.32 | 3.60 |
| 5 | 20.00 | 5.68 |

| No | Depth<br>m | type | N-value | γ<br>$kN/m^3$ | γ'<br>$kN/m^3$ | φ<br>Deg. | C<br>$kN/m^2$ | a |
|---|---|---|---|---|---|---|---|---|
| 1 | 4.90 | S | 8 | 19.0 | 10.0 | 30.0 | 0.0 | 0.0 |
| 2 | 5.52 | C | 2 | 18.0 | 9.0 | 0.0 | 12.0 | 0.0 |
| 3 | 10.72 | C | 24 | 18.0 | 9.0 | 0.0 | 146.0 | 0.0 |
| 4 | 14.32 | S | 15 | 19.0 | 10.0 | 28.0 | 0.0 | 0.0 |
| 5 | 20.00 | S | 45 | 19.0 | 10.0 | 38.0 | 0.0 | 0.0 |

Depth：Distance from the top of sheet pile    C ：Adhesive force
type：Sandy soil(S)、Cohesive soil(C)、Medium soil(M)  a ：Angle of adhesive force
Ｎvalue：average value
γ ：unit weight of humid soil
γ'：unit weight of soil under the water
φ ：internal friction angle

## 1-8 Sheet pile

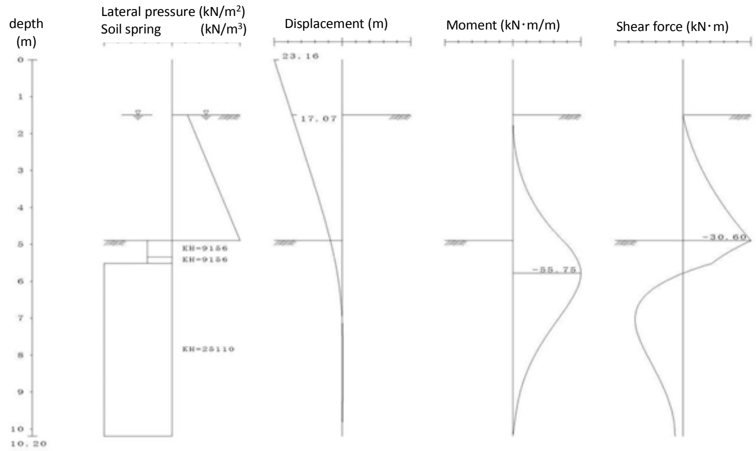
| | | | |
|---|---|---|---|
| Type | PU12 (S270GP) | | |
| Young modulus | E ＝ | 200,000 | $N/mm^2$ |
| Second moment of area | I0＝ | 21600 | $cm^4$ (before corrosion) |
| Modulus of section | Z0＝ | 1200 | $cm^3$ (before corrosion) |

Corrosion allowance    t1＝    0.00 mm (front side)    t2＝    0.00 mm (back side)
Corrosion ratio (againstI0)    η＝    1.00
Corrosion ratio (against Z0)    η＝    1.00
Joint efficiency (against I0)    μ＝    1.00
Joint efficiency(againstZ0)    μ＝    1.00

Allowable stress    σa =    165 $N/mm^2$ (Regular condition)

Allowable displacement    δa ＝    50.0 mm (Regular condition)

## 2 Section force and Displacement




## 3 Calculation results

Front sheet pile    PU12 (S270GP)

| | | | Regular condition | |
|---|---|---|---|---|
| Second moment of area | I ($cm^4$) | 21600 | | |
| Modulus of section | Z ($cm^3$) | 1200 | | |
| Max. bending moment | Mmax (kN·m/m) | | 55.75 | |
| Stress intensity | σ ($N/mm^2$) | | 46 (165) | |
| Horizontal displacement | δ (mm) | | 17.07 (50.0) | |
| Footing depth | D (m) | | 5.23 | |
| Total length | L (m) | 9.50 | | |

# APPENDIX A-9

## Structural Calculation of Ancillary Bridge

# A-9-1. Structural calculation of ancillary bridge for Bahr Yusef and Ibrahimia

A-415

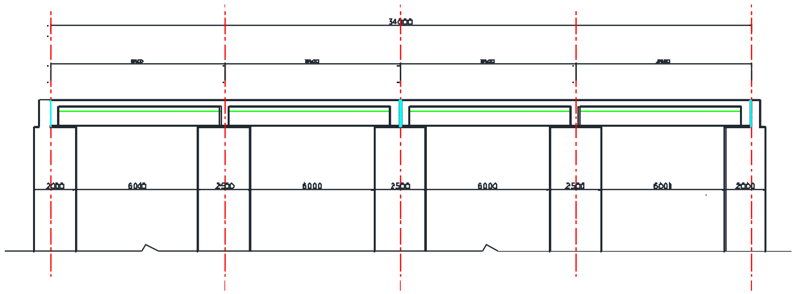
## 1. GEOMETRY OF THE BRIDGE:

- The deck bridge consists of four spans each of them 6.0 m clear span and 8.50 m length (from center lines of supports), the total length of the bridge is 34.00 m. the deck has a constant depth along the whole length and the longitudinal axis is straight and horizontal.
- The upper level of the pavement layer over the bridge is (50.00).
- The deck is made up of a symmetric five girder reinforced concrete cross section. The depth of the main girders is 1.25 m including the slab thickness. The distance between center lines of main girders is 2.40 m.
- Slabs are continuous reinforced concrete supported on the parallel girders 2.40 m spacing from center line to center line of girders. The clear spans of slabs are 2.0 m. the slab depth varies from 0.50 m over the girders to 0.30 at the central point and 0.40m at the free end.
- The width of the bridge is as follows:

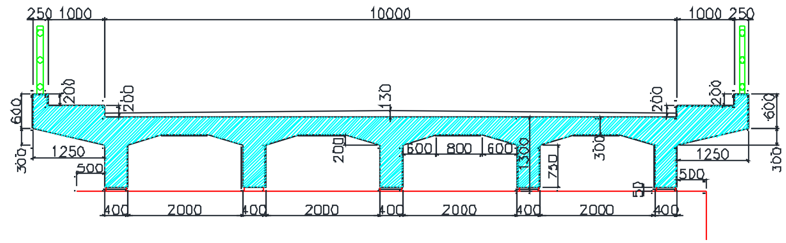
| | | | |
|---|---|---|---|
| For Vehicles | 3.00 m x 3 Lanes | = | 9.0 m |
| For Shoulder | 1.00 m x 1 | = | 1.0 m |
| For Side-walk | 1.00 m x 2 | = | 2.0 m |
| For Guardrail | 0.25 m x 2 | = | 0.5 m |

The total width of the bridge = 9.0 + 1.0 + 2.0 + 0.50 = 12.50 m.

-



Longitudinal elevation



Typical cross- section

- There is a pavement layer with 1.60 % symmetrical super elevation, varies from 0.13 m at the center of the bridge to 0.05 m at the sides of the road way.

- Geometry of the Substructure

  The deck bridge supported on three intermediate reinforced concrete piers 2.50 m width for each one and two reinforced concrete abutment 2.0 m width for each one.

## 2. REFERENCE CODES:

The design process depends mainly in the following Codes

1) The Egyptian code for planning, design & construction of bridges and elevated intersections, part (4) loads and forces on bridges. Code No. (207/4), Ministerial decree No. 233 – 2015, Ministry of housing, utilities and urban communities, 2015.
2) The Egyptian code for planning, design & construction of bridges and elevated intersections, part (5) analysis and design of concrete bridges. Code No. (207/5), Ministerial decree No. 233 – 2015, Ministry of housing, utilities and urban communities, 2015.
3) The Egyptian code for planning, design & construction of bridges and elevated intersections, part (6) analysis and design of steel bridges. Code No. (207/6), Ministerial decree No. 233 – 2015, Ministry of housing, utilities and urban communities, 2015.
4) The Egyptian code for design and construction of concrete structures, Code No. (203), Ministerial decree No. 44 – 2007, Ministry of housing, utilities and urban communities, 2007.
5) The Egyptian code for loads and forces structures, Code No. (203), Ministerial decree No. 44 – 2007, Ministry of housing, utilities and urban communities, 2007.
6) Eurocode 1 - Actions on structures. Part 2: Traffic loads on bridges. EN 1991-2: 2003. European Committee for Standardization.
7) Eurocode 2 – Design of concrete structures. Part 2: Concrete bridges. Design and detailing rules. EN 1992-2: 2005. European Committee for Standardization.
8) AASHTO: Standard Specifications for Highway Bridges, American Association of State Highway and Transportation Officials

*** The last three codes used only for verification and for items not included in the Egyptian codes

## 3. MATERIALS:

**Concrete**

- Concrete class is used as the following table

| Portion to be used | Cube Design Strength N/mm$^2$ | Unit Weight kN/m$^3$ |
|---|---|---|
| Reinforced Concrete | 25 | 25 |
| Plain Concrete | 20 | 23 |
| Leveling Concrete | 20 | 23 |

**[Clause 2-3-1-1/203]**

- Modulus of Elasticity $E_c$

$$E_c = 4400\sqrt{F'_{cu}} \quad N/mm^2 = 4400\sqrt{25} = 22000 \quad N/mm^2 = 2.2\text{x}10^5 \quad \text{kg/cm}^2$$

**[Eq. 2.1/203]**

- For the working stress design.

Allowable compressive strength for bending = 9.50 N/mm$^2$ (95 kg/cm$^2$).
Allowable shearing strength by concrete only = 0.70 N/mm$^2$ (7 kg/cm$^2$).
Allowable shearing strength by concrete and diagonal tension bars = 1.90 N/mm$^2$ (19 kg/cm$^2$).

**[Table. 5-1/203]**

- For limit state design

$\gamma_c$ = 1.50 **[Eq. 3-15-a/203]**

**Reinforcing Steel**

- Steel type 360/520 is used for all diameters of reinforcing bars (main, secondary reinforcing, side bars and stirrups) in the bridge under study.
- Mass Density = 7.85 ton/m$^3$
- Modulus of Elasticity $E_s$

$E_s = 2.0\text{x}10^6$ kg/cm$^2$ **[Fig. 4-1/203]**

- For the working stress design.

Allowable stress for tension and compression = 200 N/mm$^2$ (2000 kg/cm$^2$). [Table. 5-1/203]

For limit state design

Yield stress for tension and compression = 360 N/mm$^2$ (3600 kg/cm$^2$)

$\gamma_s$ = 1.15 [Eq. 3-15-b/203]



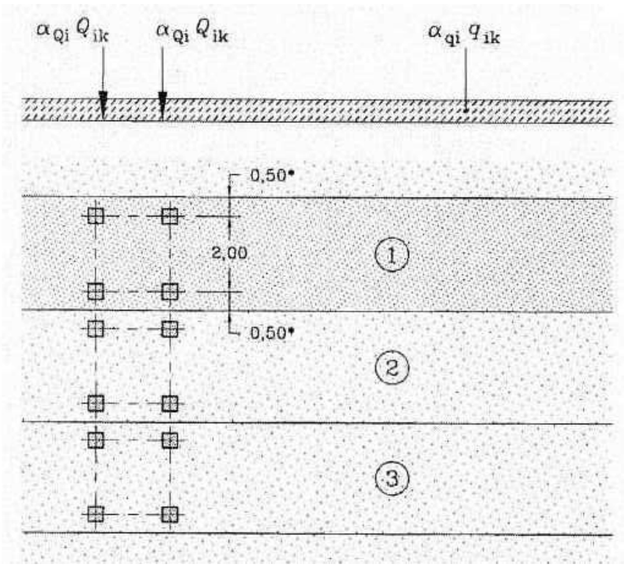
**4. TRAFFIC LOADS: (according to the Egyptian code)**

Vertical loads are represented by the following three models: [Clause 4-5-3/207]

a) Load Model 1 (LM1): Concentrated and uniformly distributed loads, which cover most of the effects of the traffic of lorries and cars. This model should be used for general and local verifications.
b) Load Model 2 (LM2): A single axle load applied on specific tyre contact areas which cover the dynamic effects of the normal traffic short structural members (slabs).
c) Load Model 3 (LM3): A crowd loading intended only for general verifications.

**Load Model 1**

Consists of concentrated loads (tandem system: TS) and uniformly distributed loads (UDL system) affecting in three notional lanes as shown




Load Model 1 (LM1)

Concentrated and Uniform Loads on Traffic Lanes

| Location | Tandem system TS Axle loads Q (kN) | UDL system q $(kN/m^2)$ |
|---|---|---|
| Lane Number 1 | 300 | 9.0 |
| Lane Number 2 | 200 | 2.5 |
| Lane Number 3 | 100 | 2.5 |
| Other lanes | 0 | 2.5 |
| Remaining area (q) | 0 | 2.5 |

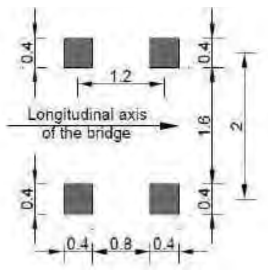
For lane No. 1: Q = 300 kN (30 ton), q = 9.0 $kN/m^2$ (0.900 $ton/m^2$).

For lane No. 2: Q = 200 kN (20 ton), q = 2.5 $kN/m^2$ (0.250 $ton/m^2$).

For lane No. 3: Q = 100 kN (10 ton), q = 2.5 $kN/m^2$ (0.250 $ton/m^2$).

For remaining area: q = 2.5 $kN/m^2$ (0.250 $ton/m^2$).

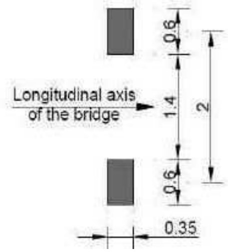
- All concentrated and uniformly distributed loads in the three notional lanes of Model 1 are including the dynamic effect.
- No more than one tandem system should be taken into account per notional lane.
-




Tandem System of LM1

**Load Model 2**

Consists of a single axle load equal to 400 kN , dynamic amplification included, each wheel load is 200 kN and the distance between them is 2.00 m.




**Tandem System of LM2**

**Load Model 3**

Load model 3 represents crowd loading consisting of uniformly distributed loads equal to 5 kN/m2 including the dynamic effect. This load model should be applied on the relevant parts of the length and width of the road bridge.

## 5. DESIGN OF SLABS

**Dead Loads**

Assume the thickness of R. C. slab ($t_s$) = 30 cm
Assume the average thickness of flooring layer ($t_f$) = 12 cm

Own weight of slab = 0.30x2500 = 750 kg / m$^2$
Weight of flooring layer = 0.12x2300 = 276 kg / m$^2$

The total dead load ($w_{D.L.}$) = 750+276 = 1026 kg / m$^2$

$M_{D.L.(cont.)} = w\, L^2/10 = 1026 * 2.4^2 /10 = 591$ kg.m

$Q_{D.L\,(cont.)} = 0.60\, w*L = 0.60*1026*2.40 = 1477$ kg

**Live Loads**

I = 0.3 (1- D/6) =0.30 (1- 0.4/6) = 0.28
Using load model 2 (LM2)
P = 200 kN = 20 ton

$M_{L.L.(simple)}$ = ( S+0.60 ) * $P$ * (1+I) /10 [Eq. 5-4-15/207]

= ( 2.0 + 0.60 ) * *20000* * (1.28) /10 = 6656 kg.m

$M_{L.L.\,(cont.)} = 0.8\, M_{simple}$

$M_{L.L.\,(cont.)} = 0.8\, x\, 6656 = 5325\ kg.m$

$M_{design} = M_{D.L.} + M_{L.L.} = 591 + 5325 = 5916\ kg.m$

**Design of slab section [working stress design]**

$F_{cu}$ = 250 kg/cm$^2$ --------------- $F_c$ = 95 kg/cm$^2$

Steel type 360/520 -------------- $F_s$ = 2000 x0.93 kg/cm$^2$

$F_s$ = 1800 kg/cm$^2$

$M_{design} = 5916\ kg.m$

$d = t - cover = 30 - 5 = 25.0\ cm$

$$d = k_1 \sqrt{\frac{M}{b}}$$

$$d = 25 = k_1 \sqrt{\frac{5916*100}{100}} \quad \ldots\ldots\ldots\ldots\ldots \quad k_1 = 0.325$$

$$k_2 = 1600$$

$$A_s = \frac{M}{k_2\, d} = \frac{5916*100}{1600*25} = 14.79\ cm^2/m$$

*use* $8\,\phi 16\ mm/m$

*i.e.* $16mm@125mm \qquad A_{s\,actual} = 16.08\ cm^2/m$

$$\mu_{actual} = \frac{A_s}{b\,d} = \frac{16.08}{100*25}x100 = 0.6432\ \%$$

$$\mu_{\min} = 0.225\frac{\sqrt{F_{cu}}}{f_y} = 0.225\frac{\sqrt{25}}{360}x100 = 0.3125\%$$

$$\geq \frac{1.1}{f_y} = \frac{1.1}{360}x100 = 0.306\%$$

$\geq 0.15\%$ for high tensile steel

[Clause 4-2-1-2-h/203]

$\mu_{\min} = 0.3125\%$

$\mu_{\max} = 5x10^{-4}\, xF_{cu} = 5x10^{-4}\, x25x100 = 1.25\%$

$$\mu_{\max} = 1.25\% \succ \mu_{actual} = 0.643\% \succ \mu_{\min} = 0.3125\%$$

- Secondary reinforcement steel perpendicular to the main steel can be taken as a percentage of the main steel according to the next formula

$$\mu_t = \frac{120}{\sqrt{S}} \leq 67\%$$ [Eq. 5-4-16/207]

S = 2.00 m

$$\mu_t = \frac{120}{\sqrt{2.0}} \leq 84.85\% \Rightarrow \mu_t = 67\%$$

$$A_s' = 0.67x16.08 = 10.80\, cm^2/m$$

$$use\quad 5.4\,\phi 16\; mm/m$$
$$i.e.\quad 16mm@180mm \qquad A_{s\,actual} = 11.16\,cm^2/m$$

**Design of slab section [Limit State Design]**

$F_{cu}$= 250 kg/cm$^2$ -------------- $\gamma_c$ = 1.50

Steel type 360/520 -------------- $F_y$ = $3600*0.93 \approx 3300$ kg/cm$^2$

$\gamma_s$ = 1.15

$M_{U\,design}$ = 1.40 $M_{D.L.}$ + 1.60 $M_{L.L.}$

$$M_{U\,design} = 1.40*591+1.60*5325 = 9350\,\text{kg.m}$$
$$d = t - cover = 30-5 = 25.0\; cm$$

$$d = C_1\sqrt{\frac{M_U}{F_{cu}\, b}}$$

$$d = 25 = C_1\sqrt{\frac{9350*100}{250*100}} \;\ldots\ldots\ldots\ldots\; C_1 = 4.09$$

$$c/d = 0.180$$

$$(c/d)_{min.} = 0.125 \qquad (c/d)_{max.} = 0.440 \qquad j = 0.807$$



$$A_s = \frac{M_U}{F_y\, J\, d} = \frac{9350*100}{3300*0.807*25} = 14.04\,cm^2/m$$

$$use\quad 8\,\phi 16\, mm/m$$
$$i.e.\quad 16mm@125mm \qquad A_{s\,actual} = 16.08\,cm^2/m$$

- Secondary reinforcement steel perpendicular to the main steel can be taken as a percentage of the main steel according to the next formula

$$\mu_t = \frac{120}{\sqrt{S}} \leq 67\%$$

S = 2.00 m

$$\mu_t = \frac{120}{\sqrt{2.0}} \leq 84.85\% \Rightarrow \mu_t = 67\%$$

$$A_s' = 0.67x16.08 = 10.80\, cm^2/m$$

$$use\quad 5.4\,\phi 16\; mm/m$$
$$i.e.\quad 16mm@180mm \qquad A_{s\,actual} = 11.16\,cm^2/m$$

## 6. DESIGN OF GIRDERS

### Dead Loads

Assume the breadth of the main girder (b) = 40 cm
Assume the depth of the main girder (t) = 125 cm
Own weight of the main girder = 0.40 x 0.95 x 2500 + 2x0.5x 0.6 x0.2x2500
= 1250 kg / m′

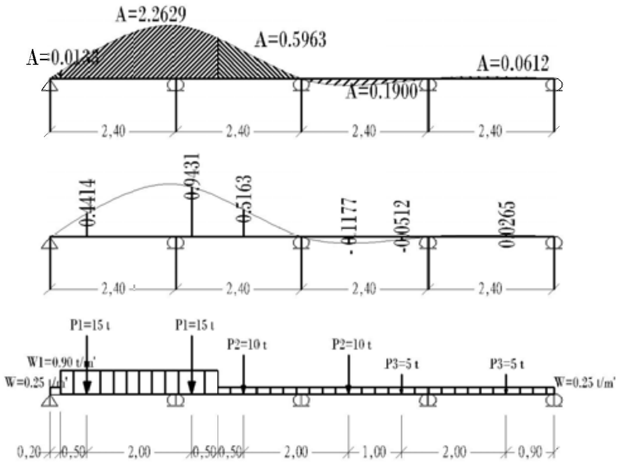
The total dead load of slab (D.L.) = 1026 kg / m$^2$

The weight from slab = 1026 x 2.40 =2462 kg / m′

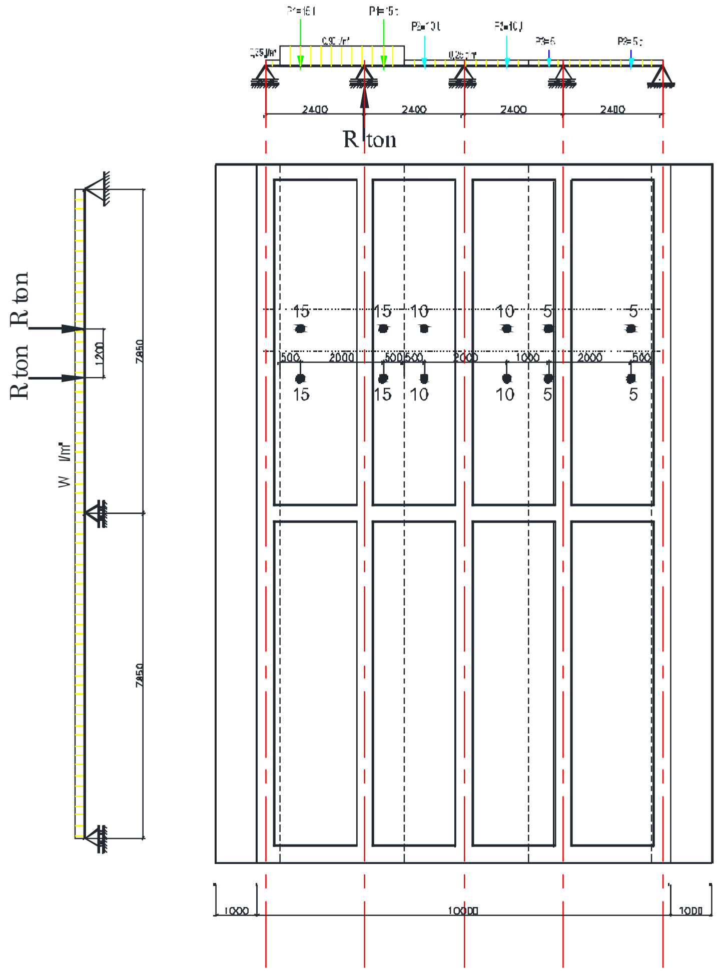
Total dead load ( $w_{D.L.}$)= 1250+2462 = 3712 kg / m

### Live Loads

First, move the lane loads in the direction perpendicular to the traffic direction to get the maximum concentrated reactions ( R ) and the maximum uniform reaction (w) on the intermediate beams.




Maximum Reactions from slabs to beams




Lane Loads on the Bridge (LM1)

R = 15x0.4414+15x0.9431+10x0.5163-10x0.1177-5x0.0512+5x0.0265

= 24.63 ton

$w$ = 0.25x$\Lambda_1$+0.9x$\Lambda_2$+0.25x$\Lambda_3$-0.25x$\Lambda_4$+0.25x$\Lambda_5$

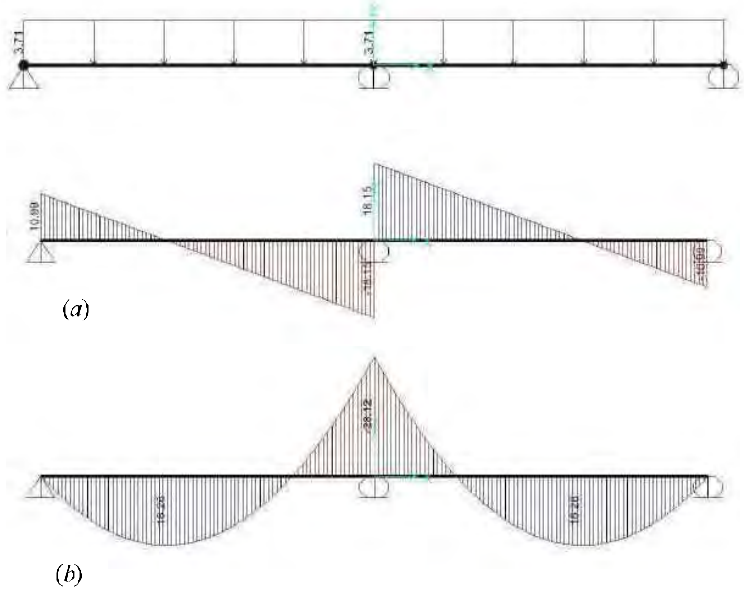
=0.25x0.0133+0.9x2.2629−0.25x0.5963-0.25x0.1900+0.25x0.0612

=2.1568 t/m'

Choice no. (1)

**Assume Two-span Continuous Girder**

<u>Straining actions due to dead load</u>

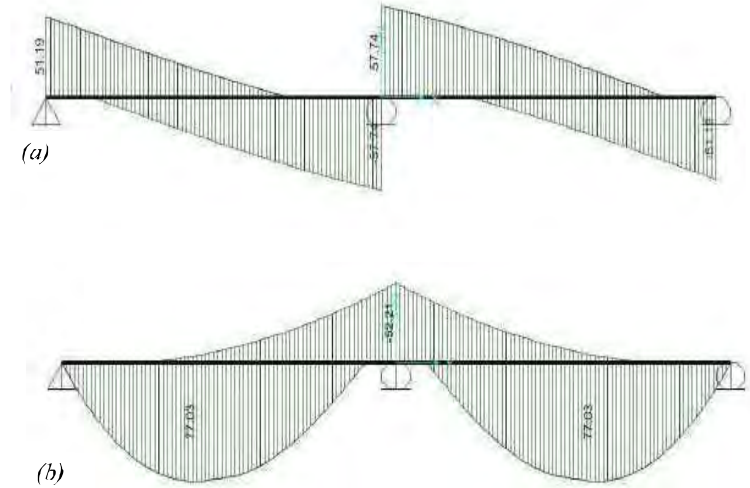



Straining actions due to dead load

*(a) Shear Force Diagram due to Dead Load*

*(b) Bending Moment Diagram due to Dead Load*

<u>Straining actions due to Live load</u>

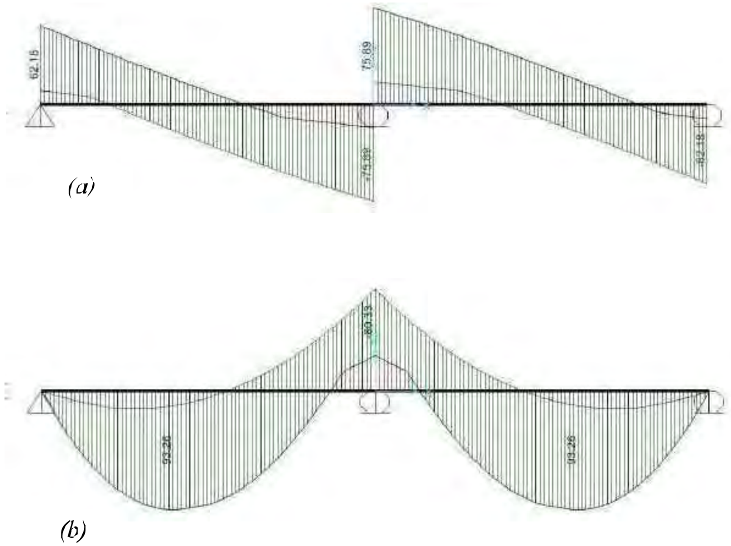



Straining actions due to Live Load

*(a) Maximum Shear Force Diagram due to Live Load*

*(b) Maximum Bending Moment Diagram due to Live Load*

**Design staining actions**




Design straining actions

*(a) Maximum Shear Force Diagram due to Dead & Live Loads*

*(b) Maximum Bending Moment Diagram due to Dead & Live Loads*

$Q_{max}$ (at end supports) = 62.18 ton

$Q_{max}$ (right & left of middle support) = 75.89 ton

$M_{max}$ (positive) =93.26 m.t. (at 3.08 m from the end supports)

$M_{max}$ (negative) =80.33 m.t. (at the middle support)

**7. Design of sections**

**Section (1):** (at 3.08 m from the end supports)

$M_{max}$ positive = 93.26 m.t. , T- sec.

$F_{cu}$ = 250 kg/cm$^2$ --------------- $F_c$ = 95 kg/cm$^2$

Steel type 360/520 -------------- $F_s$ = 2000 kg/cm$^2$

The effective breadth of the section ($b_e$) equal to the least of the following:

- $b_e = 12\,t_s + b_w = 12x30 + 40 = 400$ cm
- $b_e$ = from center to center of girders = 240 cm
- $b_e = L_1/4 = (0.80\,L)/4 = 0.8x785/4 = 157$ cm

$b_e = 157$ cm **[Clause 5-3-3-6-1 /207]**

$M_{design} = 93.26\ t.m.$

$d = t - cover = 125 - 10 = 115\ cm$

$$d = k_1 \sqrt{\frac{M}{b_e}}$$

$$d = 115 = k_1 \sqrt{\frac{93.26 * 10^5}{157}} \ \ldots\ldots\ldots\ldots\ k_1 = 0.471$$

$$\Rightarrow\ F_C = 40\ \text{kg/cm}^2 < F_{C\,allowable} = (2/3)x95 = 63\ \text{kg/cm}^2$$

$$\Rightarrow\ k_2 = 1830$$

**[Clause 5-3-2-6 /203]**

$$A_s = \frac{M}{k_2\, d} = \frac{93.26 * 10^5}{1830 * 115} = 44.31\ cm^2$$

$use\ \ 10\ \phi\ 25\ mm$

$A_{s\ actual} = 49.00\ cm^2$

$$\mu_{actual} = \frac{A_s}{b\,d} = \frac{49.0}{40*115} x100 = 1.065\%$$

$$\mu_{min} = 0.225\frac{\sqrt{F_{cu}}}{f_y} = 0.225\frac{\sqrt{25}}{360} x100 = 0.3125\%$$

$$\geq \frac{1.1}{f_y} = \frac{1.1}{360} x100 = 0.306\%$$

$\geq 0.15\%$ for high tensile steel

$$\mu_{min} = 0.3125\%$$

$$\mu_{max} = 5x10^{-4} xF_{cu} = 5x10^{-4} x25x100 = 1.25\%$$

$$\mu_{max} = 1.25\% \succ \mu_{actual} = 1.065\% \succ \mu_{min} = 0.3125\%$$

**Section (2):** (at the middle support)

$M_{max}$ negative =80.33 m.t., Rectangular sec.

$F_{cu}$= 250 kg/cm$^2$ -------------- $F_c$ = 95 kg/cm$^2$.

Steel type 360/520 -------------- $F_s$ = 2000 kg/cm$^2$.

$b = 40 \quad cm$

$M_{design} = 80.33 \ t.m.$

$d = t - cover = 125 - 10 = 115\,cm$

$$d = k_1 \sqrt{\frac{M}{b}}$$

$$d = 115 = k_1 \sqrt{\frac{80.33*10^5}{40}} \quad ............... \quad k_1 = 0.256$$

$$F_C = 88\,\text{kg/cm}^2 \prec F_{C\,allowable} = 95\,\text{kg/cm}^2$$

$$k_2 = 1735$$

$$A_s = \frac{M}{k_2\,d} = \frac{80.33*10^5}{1735*115} = 40.26\,cm^2$$

$use \quad 10\,\phi\,25\,mm$

$A_{s\,actual} = 49.00\,cm^2$

$$\mu_{min} = 0.3125\%$$

$$\mu_{max} = 5x10^{-4} xF_{cu} = 5x10^{-4} x25x100 = 1.25\%$$

$$\mu_{max} = 1.25\% \succ \mu_{actual} = 1.065\% \succ \mu_{min} = 0.3125\%$$

**Design of beam sections [Limit State Design]**

**Section (1):** (at 3.08 m from the end supports)

T- section

The effective breadth of the section ($b_e$) equal to:

$b_e = 157$ cm

$F_{cu}$= 250 kg/cm$^2$ -------------- $\gamma_c$ = 1.50

Steel type 360/520 -------------- $F_y$ = 3600 kg/cm$^2$

$\gamma_s$ = 1.15

$$M_{U\,design} = 1.40\ M_{D.L.} + 1.60\ M_{L.L.}$$

$$M_{U\,design} = 1.40 * 16.28 + 1.60 * 77.03 = 146.04\ \text{kg.m}$$

$$d = t - cover = 120 - 5 = 115\,cm$$

$$d = C_1 \sqrt{\frac{M_U}{F_{cu}\,b}}$$

$$d = 115 = C_1 \sqrt{\frac{146.04*100000}{250*157}} \quad ............... \quad C_1 = 5.96$$

$$c/d = 0.125$$

$$(c/d)_{min.} = 0.125 \qquad (c/d)_{max.} = 0.440 \qquad j = 0.826$$

$$A_s = \frac{M_U}{F_y\ J\ d} = \frac{146.04*100000}{3600*0.826*115} = 42.71\ cm^2/m$$

$$use \quad 10\,\phi\,25\ mm$$

$$A_{s\ actual} = 49.00\ cm^2$$

**Section (2):** (at the middle support)

Rectangular sec.

$F_{cu}$= 250 kg/cm² -------------- $F_c$ = 95 kg/cm².

Steel type 360/520 **--------------** $F_s$ = 2000 kg/cm².

$b = 40 \quad cm$

$d = t - cover = 125 - 10 = 115\,cm$

$F_{cu}$= 250 kg/cm² **--------------** $\gamma_c$ = 1.50

Steel type 360/520 **--------------** $F_y$ = 3600 kg/cm²

$\gamma_s$ = 1.15

$M_{U\ design} = 1.40\ M_{D.L.} + 1.60\ M_{L.L.}$

$M_{U\ design} = 1.40 * 28.12 + 1.60 * 52.21 = 122.91$ kg.m

$d = t - cover = 120 - 5 = 115\,cm$

$$d = C_1 \sqrt{\frac{M_U}{F_{cu}\ b}}$$

$$d = 115 = C_1 \sqrt{\frac{122.91*100000}{250*40}} \quad ............... \quad C_1 = 3.28$$

$$c/d = 0.30$$

$$(c/d)_{min.} = 0.125 \qquad (c/d)_{max.} = 0.440 \qquad j = 0.765$$

$$A_s = \frac{M_U}{F_y\ J\ d} = \frac{122.91*100000}{3600*0.765*115} = 38.81\ cm^2/m$$

$$use \quad 10\,\phi\,25\ mm$$

$$A_{s\ actual} = 49.00\ cm^2$$

**Check of shear**

$Q_{max} = 75.89 \quad ton$

The critical section for shear stress is the sections at the edge of the cross beam that at a distance equal to 0.20m from the middle support.

$Q_{design} = 73.92 \quad ton$

The cross section dimensions $b$ = 40 cm, $t$ = 125 cm

$d_{sh}$ = t – cover = 120 – 5

$d_{sh}$ = 120 cm

$$q = \frac{Q}{b\ d_{sh}} = \frac{73.92x1000}{40x120} = 15.40\ kg/cm^2$$ **[Eq. 5-6 /203]**

$$F_{cu} = 250\ kg/cm^2 \Rightarrow q_c = 7.0 \quad kg/cm^2$$

$$q_2 = 19.0 \quad kg/cm^2$$

$$q_2 = 19.0 \quad kg/cm^2 \succ q = 15.40\ kg/cm^2 \succ q_c = 7.0 \quad kg/cm^2$$

web reinforcement is required

$$q_s = q - 0.5\ q_c$$ **[Eq. 5-8 /203]**

$$q_s = 15.40 - 0.5x7.0 = 11.90 \quad kg/cm^2$$

Assume four branches vertical stirrups 10mm diameter each 125 mm.

$$q_{st} = \frac{A_{st}\ x\ F_s}{b\ x\ s}$$ **[Eq. 5-9 /203]**

$A_{st} = 4x(0.785) = 3.14\ cm^2$

$Steel\quad 360/520 \Rightarrow F_s = 2000\quad kg/cm^2$

$s = 125\ mm$

$$q_{st} = \frac{3.14\ x\ 2000}{40\ x\ 12.5} = 12.56\ kg/cm^2$$

$$\succ q_s = 11.90\ \ kg/cm^2$$

Use stirrups 10mm@125mm 4-branches

We assume the previous stirrups used at the distance equal to 1.75 m from the cross beam (i.e. 1.95 m from the supports) and increase the distance between stirrups at the remaining length of the beam.

$Q_{design} = 55.01\quad ton$ at a distance 1.95 m from the support

The cross section dimensions $b = 40$ cm, $d_{sh} = 120$ cm

$$q = \frac{Q}{b\ d_{sh}} = \frac{55.01x1000}{40x120} = 11.46\ kg/cm^2$$

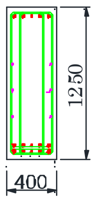
$$q_2 = 19.0\quad kg/cm^2 \succ q = 11.46\ \ kg/cm^2 \succ q_c = 7.0\quad kg/cm^2$$

web reinforcement is required

$$q_s = q - 0.5\ q_c$$

$$q_s = 11.46 - 0.5x7.0 = 7.96\quad kg/cm^2$$

Assume four branches vertical stirrups 10 mm diameter each 180 mm.




stirrups 10mm@140 mm 4-branches

$$q_{st} = \frac{A_{st}\ x\ F_s}{b\ x\ s}$$

$A_{st} = 4x(0.785) = 3.14\ cm^2$

$Steel\quad 360/520 \Rightarrow F_s = 2000\quad kg/cm^2$

$s = 180\ mm$

$$q_{st} = \frac{3.14\ x\ 2000}{40\ x\ 18} = 8.72\ kg/cm^2$$

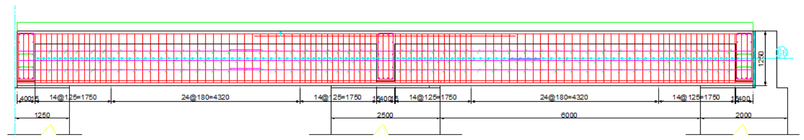
$$\succ q_s = 7.96\ \ kg/cm^2$$

Minimum web reinforcement must be not less than the following

$$\mu_{min} = \frac{A_{st\,min}}{b.s} = \frac{0.4}{f_y} = \frac{0.4}{360} = 0.111\%$$ **[Eq. 5-28 /203]**

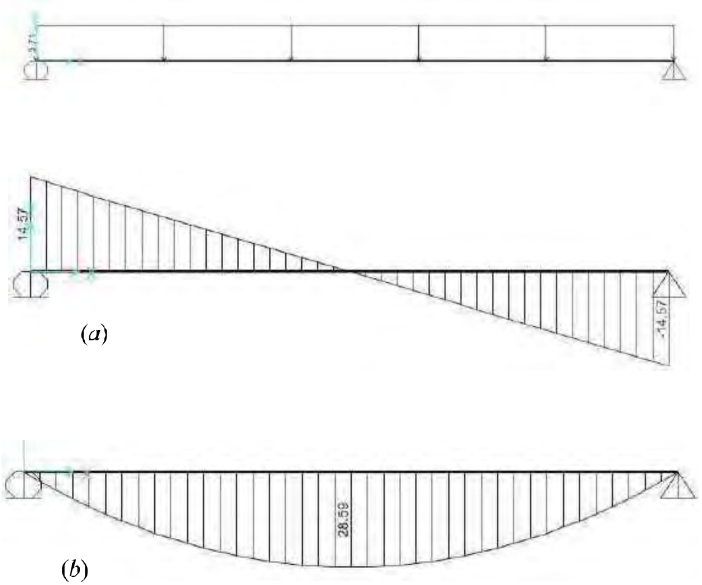
$$A_{st\,min} = \frac{0.4}{360}x40x18 = 0.8\ \ cm^2$$

$$A_{st\,actual} = 3.14\ cm^2 \succ A_{st\,min} = 0.8\ \ cm^2$$



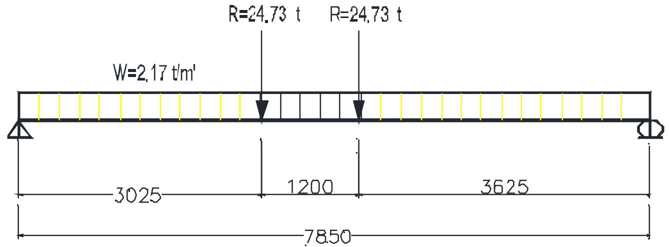
## 8. Assume Simply Supported Girder

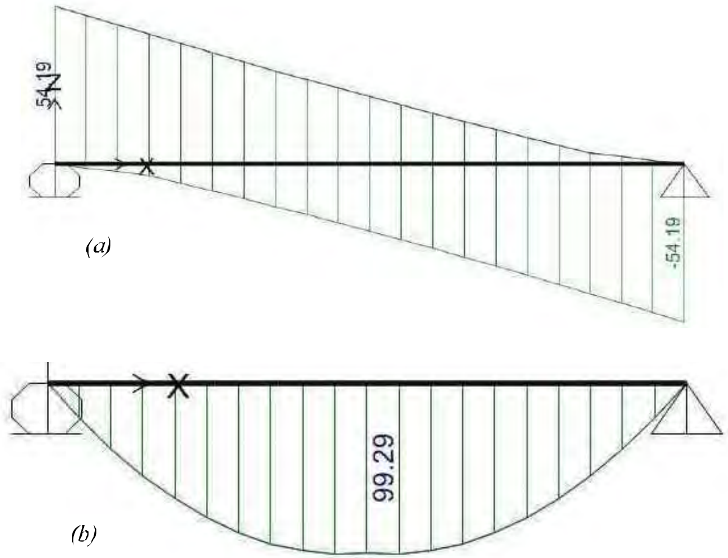
Straining actions due to dead load




Straining actions due to dead load
*(a) Shear Force Diagram due to Dead Load*
*(b) Bending Moment Diagram due to Dead Load*

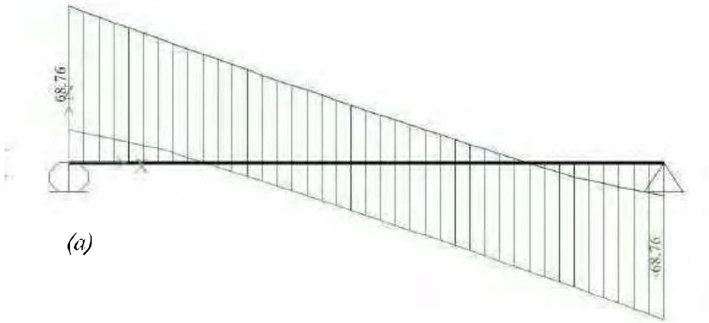
Straining actions due to Live load



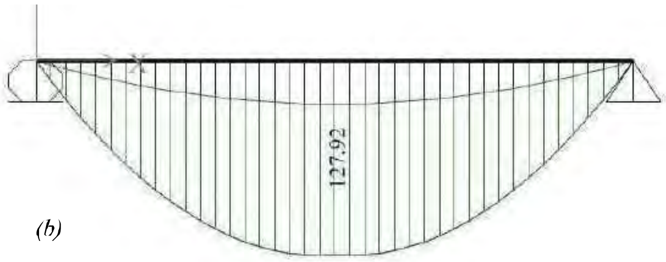




Straining actions due to Live Load
*(a) Maximum Shear Force Diagram due to Live Load*
*(b) Maximum Bending Moment Diagram due to Live Load*

**Design staining actions**




*(a)*




*(b)*

Design straining actions
*(c) Maximum Shear Force Diagram due to Dead & Live Loads*
*(d) Maximum Bending Moment Diagram due to Dead & Live Loads*

$Q_{max}$ = 68.76 ton

$M_{max}$ = 127.92 m.t.

**9. Design of sections**

**Section (1):** (at Mid-point)

$M_{max}$ positive =127.92 m.t. , T- sec.

$F_{cu}$= 250 kg/cm$^2$ --------------- $F_c$ = 95 kg/cm$^2$

Steel type 360/520 --------------- $F_s$ = 2000 kg/cm$^2$

The effective breadth of the section ($b_e$) equal to the least of the following:

- $b_e = 12\,t_s + b_w = 12x30 + 40 = 400$ cm
- $b_e$ = from center to center of girders = 240 cm
- $b_e = L_1/4 = (\,L)/4 = 785/4 = 196$ cm

$b_e = 196$ cm

$M_{design} = 127.92\ t.m.$

$d = t - cover = 125 - 10 = 115\ cm$

$$d = k_1\sqrt{\frac{M}{b_e}}$$

$$d = 115 = k_1\sqrt{\frac{127.92*10^5}{196}} \quad ............... \quad k_1 = 0.450$$

$F_C = 45$ kg/cm$^2$ $\prec$ $F_{C\,allowable} = (2/3)x95 = 63$ kg/cm$^2$

$k_2 = 1830$

$$A_s = \frac{M}{k_2\ d} = \frac{127.92*10^5}{1830*115} = 60.78\ cm^2$$

$use \quad 13\ \phi\ 25\ mm$

$A_{s\,actual} = 63.70\ cm^2$

$$\mu_{actual} = \frac{A_s}{b\ d} = \frac{63.70}{40*115}x100 = 1.385\%$$

$$\mu_{min} = 0.225\frac{\sqrt{F_{cu}}}{f_y} = 0.225\frac{\sqrt{25}}{360}x100 = 0.3125\%$$

$$\geq \frac{1.1}{f_y} = \frac{1.1}{360}x100 = 0.306\%$$

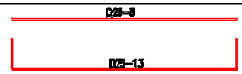
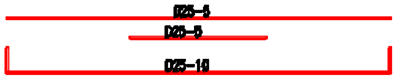
$$\geq 0.15\% \text{ for high tensile steel}$$

$$\mu_{min} = 0.3125\%$$

$$\mu_{max} = 5x10^{-4}xF_{cu} = 5x10^{-4}x25x100 = 1.25\%$$

$$\mu_{actual} = 1.385\% \succ \mu_{max} = 1.25\%$$

Comparison between simply and continuous girders for the bridge

| condition | Simply supported girders | 2 spans continuous girders |
|---|---|---|
| Concrete cross section | b = 40 cm<br>t = 125 cm | b = 40 cm<br>t = 125 cm |
| Concrete cost | 1.0 | 1.0 |
| Steel reinforcement |  |  |
| Reinforcement cost | 1.17 | 1.0 |
| Expansion joint | Three expansion joint | One expansion joint |
| Assessment | Good | Very good<br>(Recommended) |

**Limit states of cracking** **[Clause 4-3-2 /203]**

$$w_k = \beta \cdot s_{rm} \cdot \varepsilon_{sm}$$

$$s_{rm} = [50 + 0.25\, k_1\, k_2\, \frac{\phi}{\rho_r}]$$

$$\varepsilon_{sm} = \frac{f_s}{E_s}\left[1 - \beta_1 \cdot \beta_2 \left(\frac{f_{sr}}{f_s}\right)^2\right]$$

$$\beta = 1.70$$

$$k_1 = 0.80 \qquad k_2 = 0.50$$

$$\phi = 25\ mm$$

$$\rho_r = \frac{A_s}{A_{cef}}$$

$$A_s = tension\ steel = 49.0\,\mathrm{cm}^2 = 4900\,\mathrm{mm}^2$$

$$A_{cef} = t_{cef}.b = 2.5\,(t - d).b$$
$$= 2.5\,(125 - 117)x40$$
$$= 800\ cm^2$$

$$\rho_r = \frac{49}{800} = 0.06125$$

$$s_{rm} = [50 + 0.25x0.8x0.5x\ \frac{25}{0.06125}] = 90.82\ mm$$

$$f_s = 2000\ \ kg/cm^2$$

$$\beta_1 = 0.80 \qquad \beta_2 = 1.0$$

$$E_s = 2.1x10^6\ \ kg/cm^2$$

$$f_{sr} = n.f_{ctr}\ \frac{t/2 - c}{t/2}$$

$$f_{ctr} - 0.6\sqrt{F_{cu}} - 0.6\sqrt{25} - 3.0\ N/mm^2 - 30\ kg/cm^2$$

$$f_{sr} = 10x30\ \frac{62.5 - 8}{62.5} = 261.60\ kg/cm^2$$

$$\varepsilon_{sm} = \frac{2000}{2.1x10^6}\left[1 - 0.80x1.0x\left(\frac{261.6}{2000}\right)^2\right]$$
$$= 9.40x10^{-4}$$

$$w_k = 1.70x90.82x9.4x10^{-4}$$

$$w_k = 0.145 \prec w_k\ (permissible) = 0.20$$

### Development Length

The development length $L_d$ for the steel bars subjected to tension or compression can be determined from **[Clause 4-2-5-1 /203]**

$$L_d = \frac{\alpha \cdot \beta \cdot \eta \left(\frac{f_y}{\gamma_S}\right)}{4\, f_{bu}} \cdot \phi$$

$\alpha = 1.0,\qquad \beta = 0.75,$

$\eta = 1.0$ for lower reinforcement

$\eta = 1.3$ for upper reinforcement

$F_y = 360\ N/mm^2 \qquad \gamma_S = 1.15$

$$f_{bu} = 0.30\sqrt{\frac{F_{cu}}{\gamma_c}}\quad N/mm^2$$

$F_{cu} = 25\ N/mm^2 \qquad \gamma_c = 1.50$

$$f_{bu} = 0.30\sqrt{\frac{25}{1.50}} = 1.225\quad N/mm^2$$

$$L_d = \frac{1.0x0.75x1.0\left(\frac{360}{1.15}\right)}{4\,x1.225} \cdot \phi \quad = \quad 47.91\,\phi \quad \approx \quad 48\,\phi$$

**10. CHECK FOR DEFLECTION** **[Clause 4-3-1-1 /203]**

Total deflection = immediate deflection + long-term deflection

First, we calculate the effective moment of inertia for the sections subjected to negative and positive bending moments.

The deflection of girders under the effect of the frequent loads must be less than the allowable deflection

Frequent live load = 0.75 TS + 0.4 UDL


Maximum B.M. due to frequent live load

**Section (1):**

$M_{max}$ positive =54.46 m.t. , T- sec.

The effective breadth of the section ( $b_e$) equal to the least of the following:

- $b_e = 16\,t_s - b_w = 16x30 + 40 = 520$ cm
- $b_e$ = from center to center of girders = 240 cm
- $b_e = L_1 /4 = (0.80\,L)/4 = 0.8x785/4 = 157$ cm

$b_e = 157$ cm

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

$I_g = 11426756\ cm^4$

$M_a = 54.46$ m.t.

$$z = k_z\sqrt{\frac{M}{b_e}} = 0.110\sqrt{\frac{54.46x100000}{157}} = \quad 20.49\,cm$$

$$I_{cr} = 6942800 \text{ cm}^4$$

$$M_{cr} = \frac{f_{ctr} \cdot I_g}{y_t}$$

$$f_{ctr} = 0.60\sqrt{F_{cu}} \quad N/mm^2 = 0.6\sqrt{25} = 3.0 \ N/mm^2$$
$$= 30.0 \quad \text{kg/cm}^2$$

$$y_t = 82.0 \text{ cm}$$

$$M_{cr} = \frac{30x11426756}{82x10^5} = 41.80 \text{ m.t.}$$

$$I_e = \left(\frac{41.80}{54.46}\right)^3 x\,11426756 + \left[1 - \left(\frac{41.80}{54.46}\right)^3\right] x\,6942800$$
$$= 8970283 \quad cm^4$$

**Section (2):**

$M_{max}$ negative = 33.41 m.t., Rec.- sec.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

$$I_g = 40x125^3/12 = 6510416 \text{ cm}^4$$

$$M_a = 33.41 \text{ m.t.}$$

$$z = k_z\sqrt{\frac{M}{b}} = 0.110\sqrt{\frac{33.41x100000}{40}} = \quad 31.80\ cm$$

$$I_{cr} = 4849225 \text{ cm}^4$$

$$M_{cr} = \frac{f_{ctr} \cdot I_g}{y_t}$$

$$f_{ctr} = 0.60\sqrt{F_{cu}} \quad N/mm^2 = 0.6\sqrt{25} = 3.0 \ N/mm^2$$
$$= 30.0 \quad \text{kg/cm}^2$$

$$y_t = 62.50 \text{ cm}$$

$$M_{cr} = \frac{30x6510416}{62.50x10^5} = 31.25 \text{ m.t.}$$

$$I_e = \left(\frac{31.25}{33.41}\right)^3 x\,6510416 + \left[1 - \left(\frac{31.25}{33.41}\right)^3\right] x\,4849225$$
$$= 6208603 \quad cm^4$$

$$I_e = \frac{I_{e1} + I_{e2}}{2} = \frac{8970283 + 6208603}{2}$$
$$= 7589443 \quad cm^4$$

**Immediate deflection** [Clause 4-3-1-1-1 /203]

The modulus of elasticity of R.C. can be determined from the following equation

$$E_C = 4400\sqrt{F_{cu}} \quad N/mm^2 = 4400\sqrt{25} = 22000 \ N/mm^2$$
$$= 220000 \quad \text{kg/cm}^2$$

By using the elastic methods, the immediate deflection is determined

$$\delta_1 = 0.25 \quad cm$$

**Long-term deflection** [Clause 4-3-1-1-2 /203]

The long term deflection due to shrinkage and creeping of reinforced concrete members can be estimated by the following equations

$$\delta_2 = \alpha\ \delta_1$$

Where,

$$\alpha = 2 - 1.2\left(\frac{A_S'}{A_S}\right) \geq 0.6$$

$\alpha = 2 - 1.2 \ x 0.5 = 1.40$

$\delta_2 = 1.4 x 0.25 = 0.35 cm$

**Total deflection** [Clause 4-3-1-1-3 /203]

The total deflection $\delta$ will be

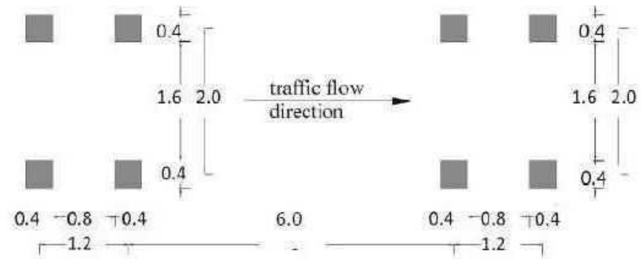
$\delta = 0.25 + 0.35 = 0.60 \quad cm \quad \prec \quad L/600 = 785/600$
$= 1.30 \text{ cm}$



## 11. CHECK FOR FATIGUE

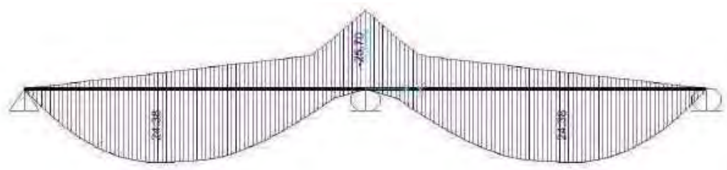
**Fatigue load model for road way bridges (single vehicle model) according to the Egyptian code** [Clause 5-9 /207]

This model consists of four axes, each of them having two identical wheels. The geometry is shown in the following figure. The weight of each axle is equal to 120 kN (12 ton).




Fatigue load model for roadway bridges

The stress range resulting from the fatigue model load must be not more than 125 N/mm$^2$ (1250 kg/cm$^2$). [Clause 5-2-2-5-c /203]



Maximum and minimum bending moment
(Fatigue model)

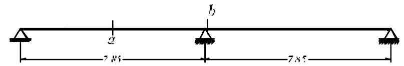
$\Delta M_{max.} = M_{max}^{+ve} - M_{max}^{-ve}$
$= 24.38 - (-5.23) = 29.61 \quad m.t$

$z = 46.63 \ cm$

$$F_{sr\,(actual)} = \frac{\Delta M_{max.}}{A_s(d - z/3)} = \frac{29.61x10^5}{49x(115 - 46.63/3)} = 608\ \text{kg/cm}^2$$

$$< F_{sr\,(allowable)} = 1250\ kg/cm^2$$

**Summary table of the bridge calculation**

| | | | | | |
|---|---|---|---|---|---|
| | condition | <br>b = 40 cm t = 125 cm | | | |
| Main Girder | Concrete Compressive stress Concrete cost | Sec. a | Fc = 40 kg / cm$^2$ < | Fc all. = 63 kg / cm$^2$ | Ok. |
| | | Sec. b | Fc = 88 kg / cm$^2$ < | Fc all. = 95 kg / cm$^2$ | Ok. |
| | Steel reinforcement | Sec. a | D25-10<br>As actual= 49.0 cm$^2$ > | As req.= 44.31 cm$^2$ | Ok. |
| | | Sec. b | D25-10<br>As actual= 49.0 cm$^2$ > | As req.= 40.26 cm$^2$ | Ok. |
| | Shear reinforcement | Sec. a | Stirrups- 4 branches<br>10mm@140 mm<br>qst = 12.56 kg/cm$^2$ > | qs =11.90 kg/cm$^2$ | Ok. |
| | | Sec. b | Stirrups- 4 branches<br>10mm@125 mm<br>qst = 8.72 kg/cm$^2$ > | qs =7.96 kg/cm$^2$ | Ok. |
| | Check of cracking | | Wk = 0.145 < | Wk(permissible) = 0.20 | Ok. |
| | Check of deflection | | δ = 0.60 cm | L/600 = 1.30 cm | Ok. |
| | Check of fatigue | | Fsr actual = 608 kg / cm$^2$ < | Fsr all. = 1250 kg /cm$^2$ | Ok. |

## A-9-2. Structural calculation of ancillary bridge for Badraman

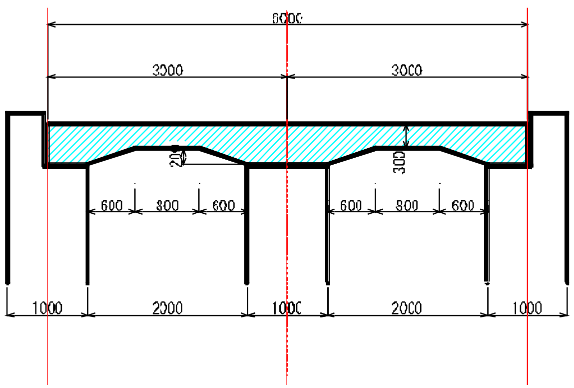
## 1. GEOMETRY OF THE BRIDGE:

- The deck bridge consists of two spans each of them 2.00 m clear span and 3.00 m length (from center lines of supports), the total length of the bridge is 6.00 m. the deck has a constant depth along the whole length and the longitudinal axis is straight and horizontal.
- The upper level of the pavement layer over the bridge is (50.00).
- The deck is made up of a symmetric reinforced concrete slabs (slab type).
- Slabs are continuous reinforced concrete supported on one pier and two abutments 3.00 m spacing from center line to center line of supports. The clear spans of slabs are 2.0 m. the slab depth varies from 0.50 m over the supports to 0.30 at the central point.
- The width of the bridge is as follows:

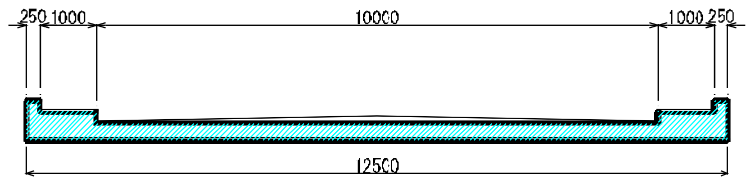
| | | |
|---|---|---|
| For Vehicles | 3.00 m x 3 Lanes | = 9.0 m |
| For Shoulder | 1.00 m x 1 | = 1.0 m |
| For Side-walk | 1.00 m x 2 | = 2.0 m |
| For Guardrail | 0.25 m x 2 | = 0.5 m |

The total width of the bridge = 9.0 + 1.0 + 2.0 + 0.50 = 12.50 m.

-




Longitudinal elevation




Typical cross- section

- There is a pavement layer with 1.60 % symmetrical super elevation, varies from 0.13 m at the center of the bridge to 0.05 m at the sides of the road way.

- Geometry of the Substructure

  The deck bridge supported on one intermediate reinforced concrete pier 1.00 m width and two reinforced concrete abutments 1.00 m width for each one.

**2. REFERENCE CODES:**

The design process depends mainly in the following Codes

1) The Egyptian code for planning, design & construction of bridges and elevated intersections, part (4) loads and forces on bridges. Code No. (207/4), Ministerial decree No. 233 – 2015, Ministry of housing, utilities and urban communities, 2015.
2) The Egyptian code for planning, design & construction of bridges and elevated intersections, part (5) analysis and design of concrete bridges. Code No. (207/5), Ministerial decree No. 233 – 2015, Ministry of housing, utilities and urban communities, 2015.
3) The Egyptian code for planning, design & construction of bridges and elevated intersections, part (6) analysis and design of steel bridges. Code No. (207/6), Ministerial decree No. 233 – 2015, Ministry of housing, utilities and urban communities, 2015.
4) The Egyptian code for design and construction of concrete structures, Code No. (203), Ministerial decree No. 44 – 2007, Ministry of housing, utilities and urban communities, 2007.
5) The Egyptian code for loads and forces structures, Code No. (203), Ministerial decree No. 44 – 2007, Ministry of housing, utilities and urban communities, 2007.
6) Eurocode 1 - Actions on structures. Part 2: Traffic loads on bridges. EN 1991-2: 2003. European Committee for Standardization.
7) Eurocode 2 – Design of concrete structures. Part 2: Concrete bridges. Design and detailing rules. EN 1992-2: 2005. European Committee for Standardization.
8) AASHTO: Standard Specifications for Highway Bridges, American Association of State Highway and Transportation Officials

*** The last three codes used only for verification and for items not included in the Egyptian codes

**3. MATERIALS:**

**Concrete**

- Concrete class is used as the following table

| Portion to be used | Cube Design Strength N/mm$^2$ | Unit Weight kN/m$^3$ |
|---|---|---|
| Reinforced Concrete | 25 | 25 |
| Plain Concrete | 20 | 23 |
| Leveling Concrete | 20 | 23 |

**[Clause 2-3-1-1/203]**

- Modulus of Elasticity $E_c$

$$E_c = 4400\sqrt{F_{cu}} \quad N/mm^2 = 4400\sqrt{25} = 22000 \quad N/mm^2 = 2.2\text{x}10^5 \quad \text{kg/cm}^2$$

**[Eq. 2.1/203]**

- For the working stress design,

Allowable compressive strength for bending = 9.50 N/mm$^2$ (95 kg/cm$^2$).
Allowable shearing strength by concrete only = 0.70 N/mm$^2$ (7 kg/cm$^2$).
Allowable shearing strength by concrete and diagonal tension bars =
1.90 N/mm$^2$ (19 kg/cm$^2$).

**[Table. 5-1/203]**

- For limit state design

$\gamma_c = 1.50$ **[Eq. 3-15-a/203]**

**Reinforcing Steel**

- Steel type 360/520 is used for all diameters of reinforcing bars (main, secondary reinforcing, side bars and stirrups) in the bridge under study.
- Mass Density = 7.85 ton/m$^3$
- Modulus of Elasticity $E_s$

$E_s = 2.0\text{x}10^6$ kg/cm$^2$ **[Fig. 4-1/203]**

- For the working stress design.

Allowable stress for tension and compression = 200 N/mm$^2$ (2000 kg/cm$^2$).
[Table. 5-1/203]

For limit state design

Yield stress for tension and compression = 360 N/mm$^2$ (3600 kg/cm$^2$)

$\gamma_s$ = 1.15 [Eq. 3-15-b/203]



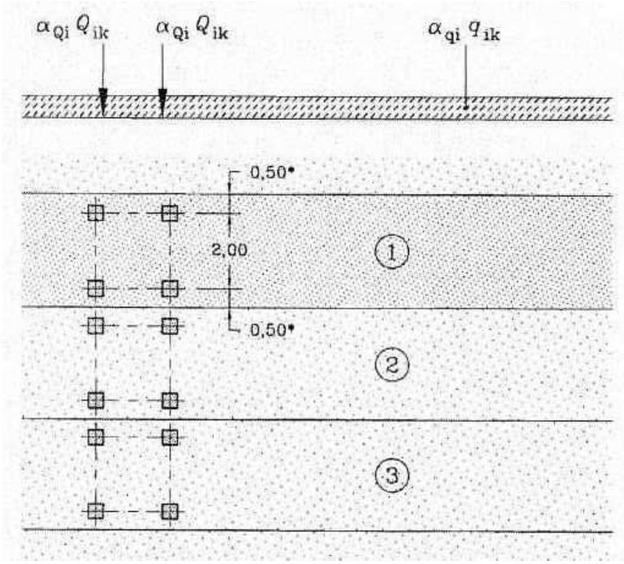
**4. TRAFFIC LOADS: (according to the Egyptian code)**

Vertical loads are represented by the following three models: [Clause 4-5-3/207]

a) Load Model 1 (LM1): Concentrated and uniformly distributed loads, which cover most of the effects of the traffic of lorries and cars. This model should be used for general and local verifications.
b) Load Model 2 (LM2): A single axle load applied on specific tyre contact areas which cover the dynamic effects of the normal traffic short structural members (slabs).
c) Load Model 3 (LM3): A crowd loading intended only for general verifications.

**Load Model 1**

Consists of concentrated loads (tandem system: TS) and uniformly distributed loads (UDL system) affecting in three notional lanes as shown




Load Model 1 (LM1)

Concentrated and Uniform Loads on Traffic Lanes

| **Location** | **Tandem system TS Axle loads Q (kN)** | **UDL system q (kN/m$^2$)** |
|---|---|---|
| Lane Number 1 | 300 | 9.0 |
| Lane Number 2 | 200 | 2.5 |
| Lane Number 3 | 100 | 2.5 |
| Other lanes | 0 | 2.5 |
| Remaining area (q) | 0 | 2.5 |

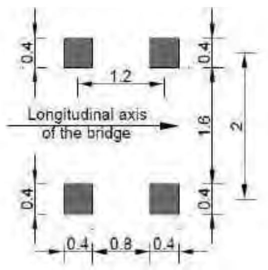
For lane No. 1: Q = 300 kN (30 ton), q = 9.0 kN/m$^2$ (0.900 ton/m$^2$).

For lane No. 2: Q = 200 kN (20 ton), q = 2.5 kN/m$^2$ (0.250 ton/m$^2$).

For lane No. 3: Q = 100 kN (10 ton), q = 2.5 kN/m$^2$ (0.250 ton/m$^2$).

For remaining area: q = 2.5 kN/m$^2$ (0.250 ton/m$^2$).

- All concentrated and uniformly distributed loads in the three notional lanes of Model 1 are including the dynamic effect.
- No more than one tandem system should be taken into account per notional lane.
-

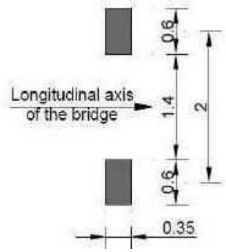



Tandem System of LM1



**Load Model 2**

Consists of a single axle load equal to 400 kN , dynamic amplification included, each wheel load is 200 kN and the distance between them is 2.00 m.




**Tandem System of LM2**

**Load Model 3**

Load model 3 represents crowd loading consisting of uniformly distributed loads equal to 5 kN/m2 including the dynamic effect. This load model should be applied on the relevant parts of the length and width of the road bridge.

## 5. DESIGN OF SLABS

**Dead Loads**

Assume the thickness of R. C. slab ($t_s$) = 30 cm
Assume the average thickness of flooring layer ($t_f$) = 12 cm

Own weight of slab = 0.30x2500 = 750 kg / m$^2$
Weight of flooring layer = 0.12x2300 = 276 kg / m$^2$

The total dead load ($w_{D.L.}$) = 750+276 = 1026 kg / m$^2$

$M_{D.L.(cont.)} = w\, L^2/10 = 1026 * 2.4^2 /10 = 591$ kg.m

$Q_{D.L.(cont.)} = 0.60\, w*L = 0.60*1026*2.40 = 1477$ kg

**Live Loads**

I = 0.3 (1- D/6) =0.30 (1- 0.4/6) = 0.28
Using load model 2 (LM2)
P = 200 kN = 20 ton

$M_{L.L.(simple)} = (S+0.60) * P * (1+I) /10$ [Eq. 5-4-15/207]

$= (2.0 + 0.60) * 20000 * (1.28) /10 = 6656$ kg.m

$M_{L.L.(cont.)} = 0.8\, M_{simple}$

$M_{L.L.(cont.)} = 0.8\, x 6656 = 5325\ kg.m$

$M_{design} = M_{D.L.} + M_{L.L.} = 591 + 5325 = 5916\ kg.m$

**Design of slab section [working stress design]**

$F_{cu}$ = 250 kg/cm$^2$ ------------- $F_c$ = 95 kg/cm$^2$
Steel type 360/520 ------------- $F_s$ = 2000 x0.93 kg/cm$^2$
$F_s$ = 1800 kg/cm$^2$

$M_{design} = 5916\ kg.m$

$d = t - cover = 30 - 5 = 25.0\ cm$

$$d = k_1 \sqrt{\frac{M}{b}}$$

$$d = 25 = k_1 \sqrt{\frac{5916*100}{100}} \quad ............... \quad k_1 = 0.325$$

$$k_2 = 1600$$

$$A_s = \frac{M}{k_2\, d} = \frac{5916*100}{1600*25} = 14.79\ cm^2/m$$

use $8\,\phi 16\ mm/m$

i.e. $16mm@125mm$ $A_{s\,actual} = 16.08\ cm^2/m$

$$\mu_{actual} = \frac{A_s}{b\,d} = \frac{16.08}{100*25} x100 = 0.6432\ \%$$

$$\mu_{min} = 0.225\frac{\sqrt{F_{cu}}}{f_y} = 0.225\frac{\sqrt{25}}{360} x100 = 0.3125\%$$

$$\geq \frac{1.1}{f_y} = \frac{1.1}{360} x100 = 0.306\%$$

$\geq 0.15\%$ for high tensile steel

[Clause 4-2-1-2-h/203]

$\mu_{min} = 0.3125\%$

$\mu_{max} = 5x10^{-4} xF_{cu} = 5x10^{-4} x25x100 = 1.25\%$

$\mu_{max} = 1.25\% \succ \mu_{actual} = 0.643\% \succ \mu_{min} = 0.3125\%$

- Secondary reinforcement steel perpendicular to the main steel can be taken as a percentage of the main steel according to the next formula

$$\mu_t = \frac{120}{\sqrt{S}} \leq \quad 67\%$$ [Eq. 5-4-16/207]

S = 2.00 m

$$\mu_t = \frac{120}{\sqrt{2.0}} \leq \quad 84.85\% \quad \Rightarrow \quad \mu_t = 67\%$$

$$A_s' = 0.67x16.08 = 10.80\, cm^2 / m$$

use $5.4\,\phi 16\ mm/m$

i.e. $16mm@180mm \qquad A_{s\ actual} = 11.16\, cm^2 / m$

**Design of slab section [Limit State Design]**

$F_{cu}$= 250 kg/cm² -------------- $\gamma_c$ = 1.50

Steel type 360/520 ------------- $F_y$ = $3600 * 0.93 \approx 3300$ kg/cm²

$\gamma_s$ = 1.15

$$M_{U\ design} = 1.40\ M_{D.L.} + 1.60\ M_{L.L.}$$

$$M_{U\ design} = 1.40 * 591 + 1.60 * 5325 = 9350\,kg.m$$

$$d = t - cover = 30 - 5 = 25.0\ cm$$

$$d = C_1 \sqrt{\frac{M_U}{F_{cu}\ b}}$$

$$d = 25 = C_1 \sqrt{\frac{9350*100}{250*100}} \quad \ldots\ldots\ldots\ldots\ldots \quad C_1 = 4.09$$

$$c/d = 0.180$$

$$(c/d)_{min.} = 0.125 \qquad (c/d)_{max.} = 0.440 \qquad j = 0.807$$

$$A_s = \frac{M_U}{F_y\ J\ d} = \frac{9350*100}{3300*0.807*25} = 14.04\ cm^2 / m$$

use $8\,\phi 16\ mm/m$

i.e. $16mm@125mm \qquad A_{s\ actual} = 16.08\ cm^2 / m$

- Secondary reinforcement steel perpendicular to the main steel can be taken as a percentage of the main steel according to the next formula

$$\mu_t = \frac{120}{\sqrt{S}} \leq \quad 67\%$$

S = 2.00 m

$$\mu_t = \frac{120}{\sqrt{2.0}} \leq \quad 84.85\% \quad \Rightarrow \quad \mu_t = 67\%$$

$$A_s' = 0.67x16.08 = 10.80\, cm^2 / m$$

use $5.4\,\phi 16\ mm/m$

i.e. $16mm@180mm \qquad A_{s\ actual} = 11.16\, cm^2 / m$

## A-9-3. Structural calculation of ancillary bridge for Diroutiah

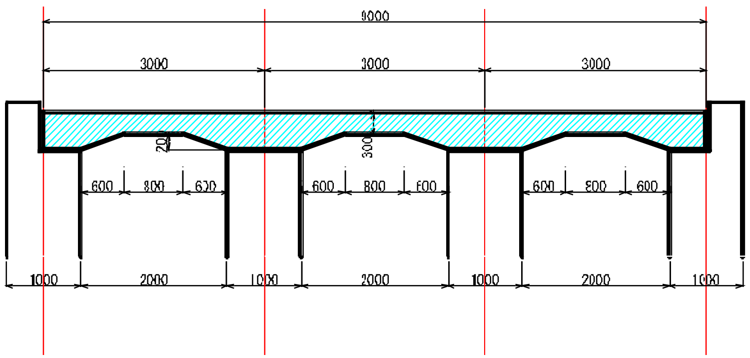
## 1. GEOMETRY OF THE BRIDGE:

- The deck bridge consists of three spans each of them 2.00 m clear span and 3.00 m length (from center lines of supports), the total length of the bridge is 9.00 m. the deck has a constant depth along the whole length and the longitudinal axis is straight and horizontal.
- The upper level of the pavement layer over the bridge is (50.00).
- The deck is made up of a symmetric reinforced concrete slabs (slab type).
- Slabs are continuous reinforced concrete supported on the piers and abutments 3.00 m spacing from center line to center line of supports. The clear spans of slabs are 2.0 m. the slab depth varies from 0.50 m over the supports to 0.30 at the central point.
- The width of the bridge is as follows:

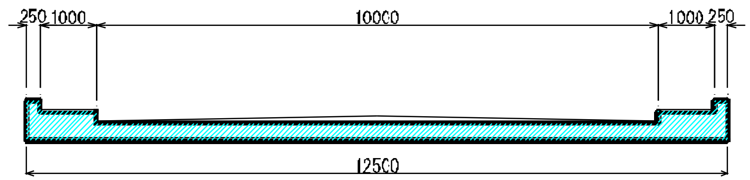
| | | |
|---|---|---|
| For Vehicles | 3.00 m x 3 Lanes | = 9.0 m |
| For Shoulder | 1.00 m x 1 | = 1.0 m |
| For Side-walk | 1.00 m x 2 | = 2.0 m |
| For Guardrail | 0.25 m x 2 | = 0.5 m |

The total width of the bridge = 9.0 + 1.0 + 2.0 + 0.50 = 12.50 m.

-




Longitudinal elevation




Typical cross- section

- There is a pavement layer with 1.60 % symmetrical super elevation, varies from 0.13 m at the center of the bridge to 0.05 m at the sides of the road way.

- Geometry of the Substructure

  The deck bridge supported on two intermediate reinforced concrete piers 1.00 m width for each one and two reinforced concrete abutments 1.00 m width for each one.

## 2. REFERENCE CODES:

The design process depends mainly in the following Codes

1) The Egyptian code for planning, design & construction of bridges and elevated intersections, part (4) loads and forces on bridges. Code No. (207/4), Ministerial decree No. 233 – 2015, Ministry of housing, utilities and urban communities, 2015.
2) The Egyptian code for planning, design & construction of bridges and elevated intersections, part (5) analysis and design of concrete bridges. Code No. (207/5), Ministerial decree No. 233 – 2015, Ministry of housing, utilities and urban communities, 2015.
3) The Egyptian code for planning, design & construction of bridges and elevated intersections, part (6) analysis and design of steel bridges. Code No. (207/6), Ministerial decree No. 233 – 2015, Ministry of housing, utilities and urban communities, 2015.
4) The Egyptian code for design and construction of concrete structures, Code No. (203), Ministerial decree No. 44 – 2007, Ministry of housing, utilities and urban communities, 2007.
5) The Egyptian code for loads and forces structures, Code No. (203), Ministerial decree No. 44 – 2007, Ministry of housing, utilities and urban communities, 2007.
6) Eurocode 1 - Actions on structures. Part 2: Traffic loads on bridges. EN 1991-2: 2003. European Committee for Standardization.
7) Eurocode 2 – Design of concrete structures. Part 2: Concrete bridges. Design and detailing rules. EN 1992-2: 2005. European Committee for Standardization.
8) AASHTO: Standard Specifications for Highway Bridges, American Association of State Highway and Transportation Officials

*** The last three codes used only for verification and for items not included in the Egyptian codes

## 3. MATERIALS:

**Concrete**

- Concrete class is used as the following table

| Portion to be used | Cube Design Strength $N/mm^2$ | Unit Weight $kN/m^3$ |
|---|---|---|
| Reinforced Concrete | 25 | 25 |
| Plain Concrete | 20 | 23 |
| Leveling Concrete | 20 | 23 |

[Clause 2-3-1-1/203]

- Modulus of Elasticity $E_c$

$$E_c = 4400\sqrt{F_{cu}} \quad N/mm^2 = 4400\sqrt{25} = 22000 \quad N/mm^2 = 2.2x10^5 \quad kg/cm^2$$

[Eq. 2.1/203]

- For the working stress design.

Allowable compressive strength for bending = 9.50 $N/mm^2$ (95 $kg/cm^2$).
Allowable shearing strength by concrete only = 0.70 $N/mm^2$ (7 $kg/cm^2$).
Allowable shearing strength by concrete and diagonal tension bars =
1.90 $N/mm^2$ (19 $kg/cm^2$).

[Table. 5-1/203]

- For limit state design

$\gamma_c = 1.50$ [Eq. 3-15-a/203]

**Reinforcing Steel**

- Steel type 360/520 is used for all diameters of reinforcing bars (main, secondary reinforcing, side bars and stirrups) in the bridge under study.
- Mass Density = 7.85 $ton/m^3$
- Modulus of Elasticity $E_s$

$E_s = 2.0x10^6 \quad kg/cm^2$ [Fig. 4-1/203]

- For the working stress design.

Allowable stress for tension and compression = 200 $N/mm^2$ (2000 $kg/cm^2$).

[Table. 5-1/203]

For limit state design

Yield stress for tension and compression = 360 $N/mm^2$ (3600 $kg/cm^2$)

$\gamma_s$ = 1.15 [Eq. 3-15-b/203]

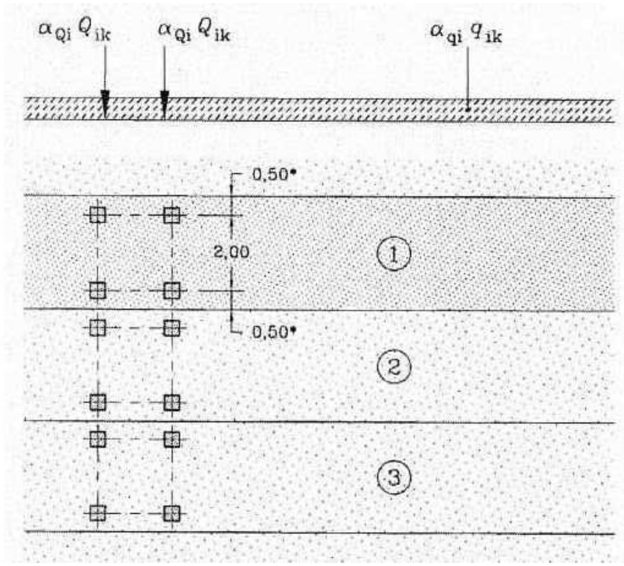
**4. TRAFFIC LOADS: (according to the Egyptian code)**

Vertical loads are represented by the following three models: [Clause 4-5-3/207]

a) Load Model 1 (LM1): Concentrated and uniformly distributed loads, which cover most of the effects of the traffic of lorries and cars. This model should be used for general and local verifications.
b) Load Model 2 (LM2): A single axle load applied on specific tyre contact areas which cover the dynamic effects of the normal traffic short structural members (slabs).
c) Load Model 3 (LM3): A crowd loading intended only for general verifications.

**Load Model 1**

Consists of concentrated loads (tandem system: TS) and uniformly distributed loads (UDL system) affecting in three notional lanes as shown




Load Model 1 (LM1)

Concentrated and Uniform Loads on Traffic Lanes

| **Location** | **Tandem system TS Axle loads Q (kN)** | **UDL system q ($kN/m^2$)** |
|---|---|---|
| Lane Number 1 | 300 | 9.0 |
| Lane Number 2 | 200 | 2.5 |
| Lane Number 3 | 100 | 2.5 |
| Other lanes | 0 | 2.5 |
| Remaining area (q) | 0 | 2.5 |

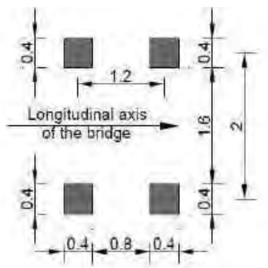
For lane No. 1: Q = 300 kN (30 ton), q = 9.0 $kN/m^2$ (0.900 $ton/m^2$).

For lane No. 2: Q = 200 kN (20 ton), q = 2.5 $kN/m^2$ (0.250 $ton/m^2$).

For lane No. 3: Q = 100 kN (10 ton), q = 2.5 $kN/m^2$ (0.250 $ton/m^2$).

For remaining area: q = 2.5 $kN/m^2$ (0.250 $ton/m^2$).

- All concentrated and uniformly distributed loads in the three notional lanes of Model 1 are including the dynamic effect.
- No more than one tandem system should be taken into account per notional lane.
-

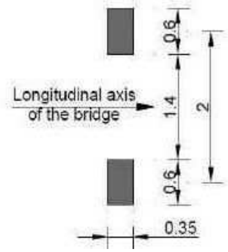



Tandem System of LM1



**Load Model 2**

Consists of a single axle load equal to 400 kN , dynamic amplification included, each wheel load is 200 kN and the distance between them is 2.00 m.




**Tandem System of LM2**

**Load Model 3**

Load model 3 represents crowd loading consisting of uniformly distributed loads equal to 5 kN/m2 including the dynamic effect. This load model should be applied on the relevant parts of the length and width of the road bridge.

## 5. DESIGN OF SLABS

**Dead Loads**

Assume the thickness of R. C. slab ($t_s$) = 30 cm
Assume the average thickness of flooring layer ($t_f$) = 12 cm

Own weight of slab = 0.30x2500 = 750 kg / m$^2$
Weight of flooring layer = 0.12x2300 = 276 kg / m$^2$

The total dead load ($w_{D.L.}$) = 750+276 = 1026 kg / m$^2$

$M_{D.L.(cont.)} = w\,L^2/10 = 1026 * 2.4^2/10 = 591$ kg.m

$Q_{D.L.(cont.)} = 0.60\,w*L = 0.60*1026*2.40 = 1477$ kg

**Live Loads**

I = 0.3 (1- D/6) =0.30 (1- 0.4/6) = 0.28
Using load model 2 (LM2)
P = 200 kN = 20 ton

$M_{L.L.(simple)} = (S+0.60) * P * (1+I)/10$ **[Eq. 5-4-15/207]**

$= (2.0 + 0.60) * 20000 * (1.28)/10 = 6656$ kg.m

$M_{L.L.(cont.)} = 0.8\,M_{simple}$

$M_{L.L.(cont.)} = 0.8\,x\,6656 = 5325\ kg.m$

$M_{design} = M_{D.L.} + M_{L.L.} = 591 + 5325 = 5916\ kg.m$

**Design of slab section [working stress design]**

$F_{cu}$ = 250 kg/cm$^2$ ------------- $F_c$ = 95 kg/cm$^2$
Steel type 360/520 ------------- $F_s$ = 2000 x0.93 kg/cm$^2$
$F_s$ = 1800 kg/cm$^2$

$M_{design} = 5916\ kg.m$
$d = t - cover = 30 - 5 = 25.0\ cm$

$$d = k_1\sqrt{\frac{M}{b}}$$

$$d = 25 = k_1\sqrt{\frac{5916*100}{100}} \quad ............... \quad k_1 = 0.325$$

$$k_2 = 1600$$

$$A_s = \frac{M}{k_2\,d} = \frac{5916*100}{1600*25} = 14.79\ cm^2/m$$

use $8\phi 16\ mm/m$
i.e. $16mm@125mm$ $\quad A_{s\,actual} = 16.08\ cm^2/m$

$$\mu_{actual} = \frac{A_s}{b\,d} = \frac{16.08}{100*25}x100 = 0.6432\ \%$$

$$\mu_{min} = 0.225\frac{\sqrt{F_{cu}}}{f_y} = 0.225\frac{\sqrt{25}}{360}x100 = 0.3125\%$$

$$\geq \frac{1.1}{f_y} = \frac{1.1}{360}x100 = 0.306\%$$

$\geq 0.15\%$ for high tensile steel

**[Clause 4-2-1-2-h/203]**

$\mu_{min} = 0.3125\%$

$\mu_{max} = 5x10^{-4}\,xF_{cu} = 5x10^{-4}\,x25x100 = 1.25\%$

$$\mu_{max} = 1.25\% \succ \mu_{actual} = 0.643\% \succ \mu_{min} = 0.3125\%$$

- Secondary reinforcement steel perpendicular to the main steel can be taken as a percentage of the main steel according to the next formula

$$\mu_t = \frac{120}{\sqrt{S}} \leq \quad 67\%$$ [Eq. 5-4-16/207]

S = 2.00 m

$$\mu_t = \frac{120}{\sqrt{2.0}} \leq \quad 84.85\% \quad \Rightarrow \quad \mu_t = 67\%$$

$$A_s' = 0.67x16.08 = 10.80\,cm^2/m$$

$$use \quad 5.4\,\phi 16\;mm/m$$
$$i.e. \quad 16mm@180mm \qquad A_{s\;actual} = 11.16\,cm^2/m$$

**Design of slab section [Limit State Design]**

$F_{cu}$= 250 kg/cm² -------------- $\gamma_c$ = 1.50

Steel type 360/520 -------------- $F_y$ = $3600 * 0.93 \approx 3300$ kg/cm²

$\gamma_s$ = 1.15

$$M_{U\;design} = 1.40\ M_{D.L.} + 1.60\ M_{L.L.}$$

$$M_{U\;design} = 1.40 * 591 + 1.60 * 5325 = 9350\,kg.m$$
$$d = t - cover = 30 - 5 = 25.0\ cm$$

$$d = C_1\sqrt{\frac{M_U}{F_{cu}\ b}}$$

$$d = 25 = C_1\sqrt{\frac{9350*100}{250*100}} \quad \ldots\ldots\ldots\ldots\ldots \quad C_1 = 4.09$$

$$c/d = 0.180$$

$$(c/d)_{min.} = 0.125 \qquad (c/d)_{max.} = 0.440 \qquad j = 0.807$$

$$A_s = \frac{M_U}{F_y\ J\ d} = \frac{9350*100}{3300*0.807*25} = 14.04\,cm^2/m$$

$$use \quad 8\,\phi 16\,mm/m$$
$$i.e. \quad 16mm@125mm \qquad A_{s\;actual} = 16.08\,cm^2/m$$

- Secondary reinforcement steel perpendicular to the main steel can be taken as a percentage of the main steel according to the next formula

$$\mu_t = \frac{120}{\sqrt{S}} \leq \quad 67\%$$

S = 2.00 m

$$\mu_t = \frac{120}{\sqrt{2.0}} \leq \quad 84.85\% \quad \Rightarrow \quad \mu_t = 67\%$$

$$A_s' = 0.67x16.08 = 10.80\,cm^2/m$$

$$use \quad 5.4\,\phi 16\;mm/m$$
$$i.e. \quad 16mm@180mm \qquad A_{s\;actual} = 11.16\,cm^2/m$$

# APPENDIX A-10

## Design of Architecture

## A-10-1. Design of control house

## 1. REFERENCE CODES:

The design process depends mainly in the following Codes

1) The Egyptian code for design and construction of concrete structures, Code No. (203), Ministerial decree No. 44 – 2007, Ministry of housing, utilities and urban communities, 2007.
2) The Egyptian code for loads and forces structures, Code No. (201), Ministerial decree No. 431 – 2011, Ministry of housing, utilities and urban communities, 2012.

## 2. MATERIALS:

**Concrete**

- Concrete class is used as the following table

| Portion to be used | Cube Design Strength $N/mm^2$ | Unit Weight $kN/m^3$ |
|---|---|---|
| Reinforced Concrete | 25 | 25 |
| Plain Concrete | 20 | 23 |
| Leveling Concrete | 20 | 23 |

**[Clause 2-3-1-1/203]**

- Modulus of Elasticity $E_c$

$$E_C = 4400\sqrt{F_{cu}} \quad N/mm^2 = 4400\sqrt{25} = 22000 \quad N/mm^2 = 2.2x10^5 \quad kg/cm^2$$

**[Eq. 2.1/203]**

- For the working stress design.

Allowable compressive strength for bending = 9.50 $N/mm^2$ (95 $kg/cm^2$).
Allowable shearing strength by concrete only (beams)
= 0.70 $N/mm^2$ (7 $kg/cm^2$).
Allowable shearing strength by concrete only (slabs & footings)
= 0.90 $N/mm^2$ (7 $kg/cm^2$).
Allowable shearing strength by concrete and diagonal tension bars =
1.90 $N/mm^2$ (19 $kg/cm^2$).

**[Table. 5-1/203]**

- For limit state design

$\gamma_c = 1.50$ **[Eq. 3-15-a/203]**

**Reinforcing Steel**

- Steel type 360/520 is used for all diameters of reinforcing bars (main, secondary reinforcing, side bars and stirrups) in the bridge under study.
- Mass Density = 7.85 $ton/m^3$
- Modulus of Elasticity $E_s$

$E_s = 2.0 x 10^6 \quad kg/cm^2$ **[Fig. 4-1/203]**

- For the working stress design.

Allowable stress for tension and compression = 200 $N/mm^2$ (2000 $kg/cm^2$).

**[Table. 5-1/203]**

For limit state design

Yield stress for tension and compression = 360 $N/mm^2$ (3600 $kg/cm^2$)

$\gamma_s = 1.15$ **[Eq. 3-15-b/203]**

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### 3. LIVE LOADS: (according to the Egyptian code)

Vertical loads are represented by uniformly distributed loads (UDL system):

[Table 4-1 /201]

a) UDL = 300 kg/m2 for office rooms.
b) UDL = 400 kg/m2 for stairs.
c) UDL = 400 kg/m2 for balconies.

### 4. DEAD LOADS

**Slab Loads**

- Assume the weight of flooring layer = 200 kg / m$^2$.

- If the thickness of R. C. slab ( $t_s$ ) = 12 cm
  Own weight of slab = 0.12x2500 = 300 kg / m$^2$

The total dead load ($w_{D.L.}$) = 300+200 = 500 kg / m$^2$

- If the thickness of R. C. slab ( $t_s$ ) = 14 cm
  Own weight of slab = 0.14x2500 = 350 kg / m$^2$

The total dead load ($w_{D.L.}$) = 300+200 = 550 kg / m$^2$

- If the thickness of R. C. slab ( $t_s$ ) = 15 cm
  Own weight of slab = 0.15x2500 = 375 kg / m$^2$

The total dead load ($w_{D.L.}$) = 300+200 = 575 kg / m$^2$

- If the thickness of R. C. slab ( $t_s$ ) = 16 cm
  Own weight of slab = 0.16x2500 = 400 kg / m$^2$

The total dead load ($w_{D.L.}$) = 300+200 = 600 kg / m$^2$

**Own weight of beams**

- For beam section 25x60
  Own weight = 0.25*(0.60-0.12)*2500 = 300 kg/m\

- For beam section 30x80
  Own weight = 0.30*(0.80-0.12)*2500 = 510 kg/m\

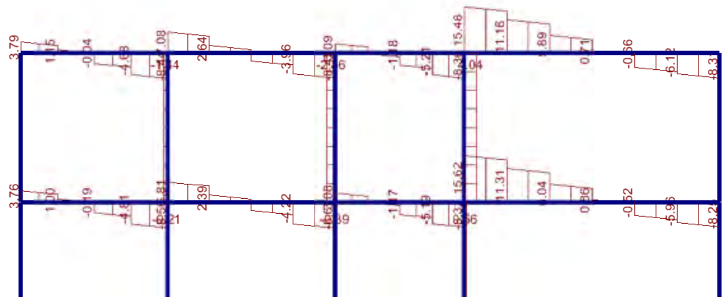
**Own weight of walls**

- For walls 12 cm thickness
  Weight of wall = 220 kg/m$^2$

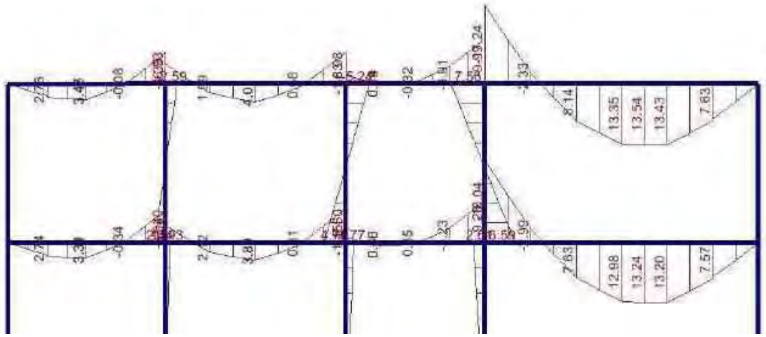
- For walls 25 cm thickness
  Weight of wall = 375 kg/m$^2$

## 5. STRAINING ACTIONS ON BEAMS

Beam at axis 1-1

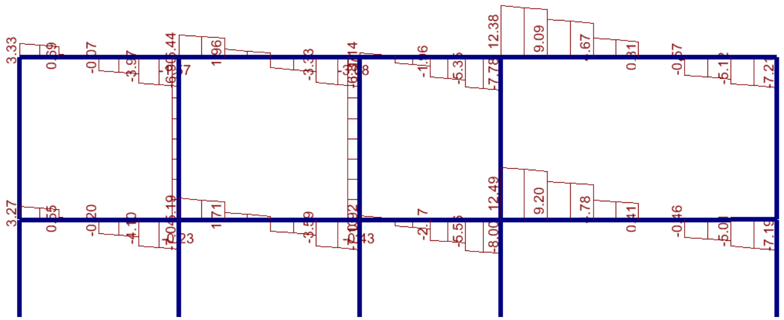


Shear force diagram

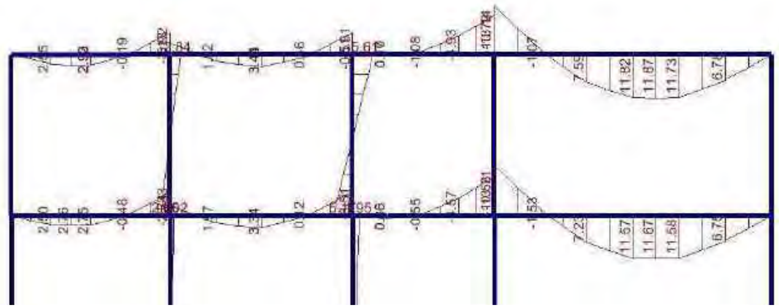


Bending moment diagram

Beam at axis 2-2

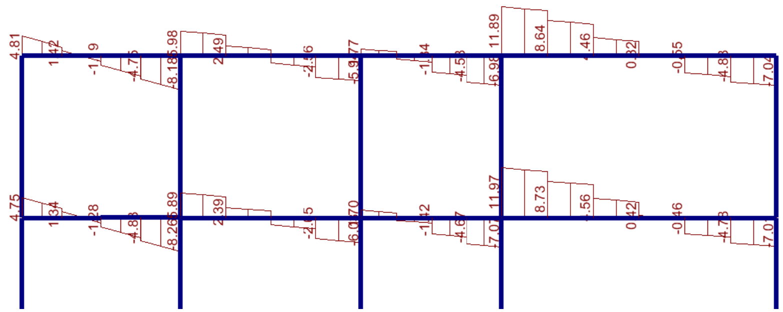


Shear force diagram

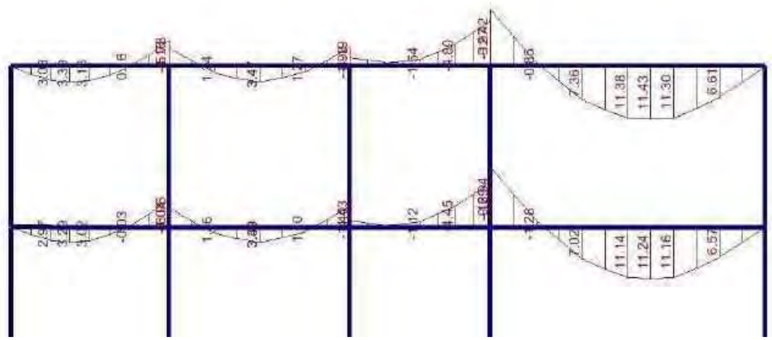


Bending moment diagram

Beam at axis 3-3

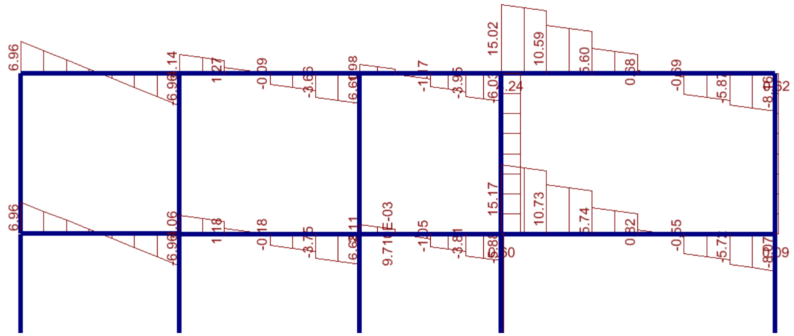


Shear force diagram

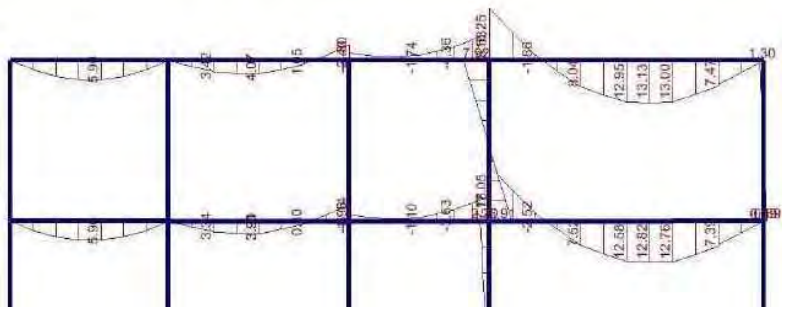


Bending moment diagram

Beam at axis 4-4

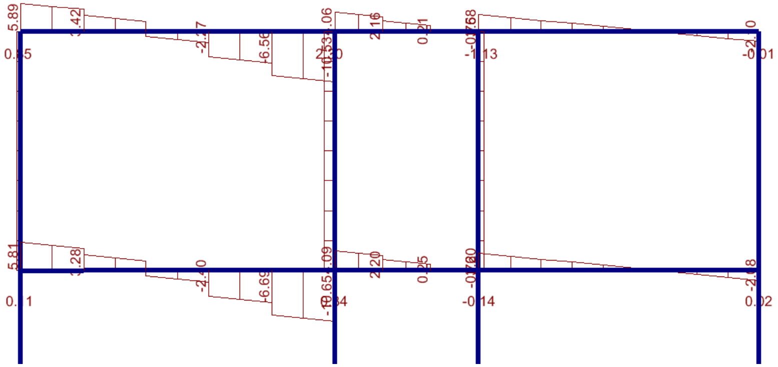


Shear force diagram

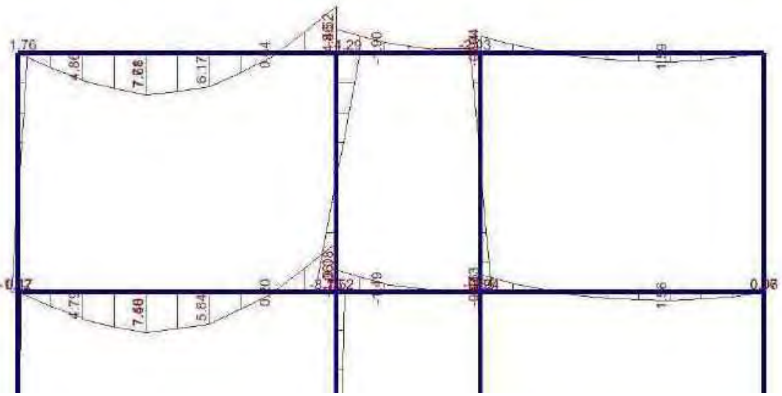


Bending moment diagram

Beam at axis A-A

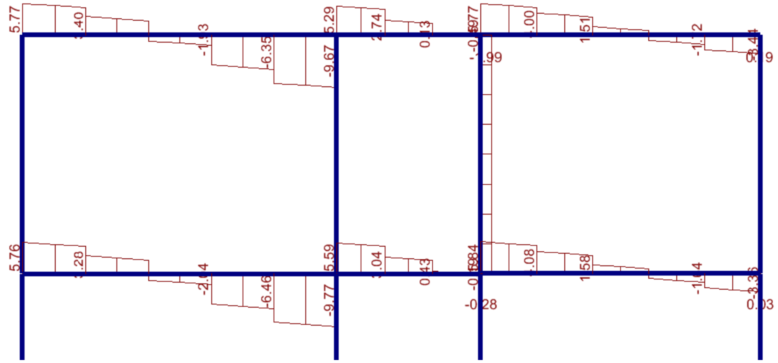


Shear force diagram

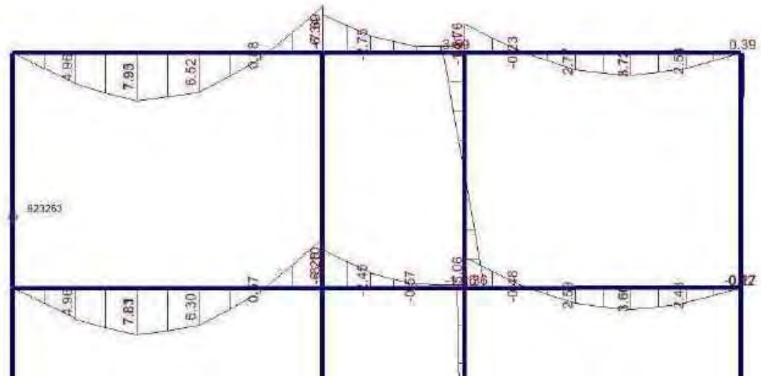


Bending moment diagram

Beam at axis B-B

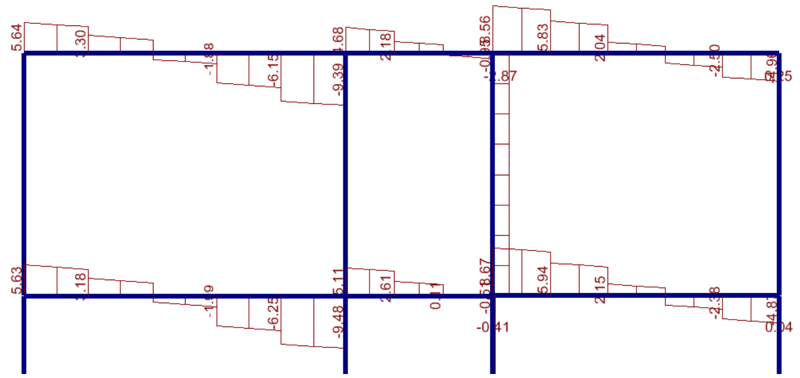


Shear force diagram

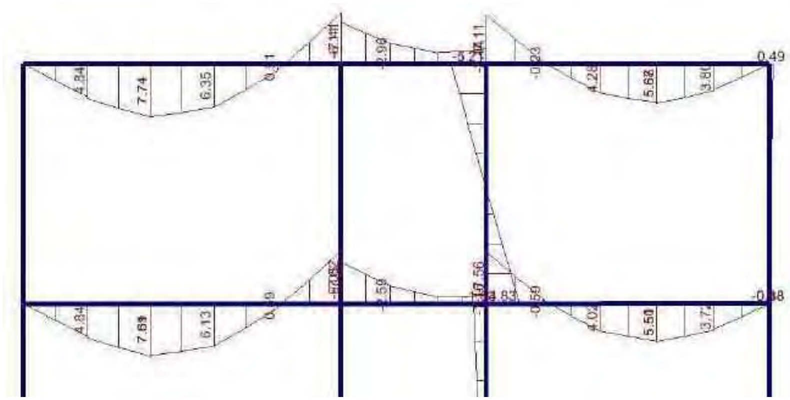


Bending moment diagram

Beam at axis C-C

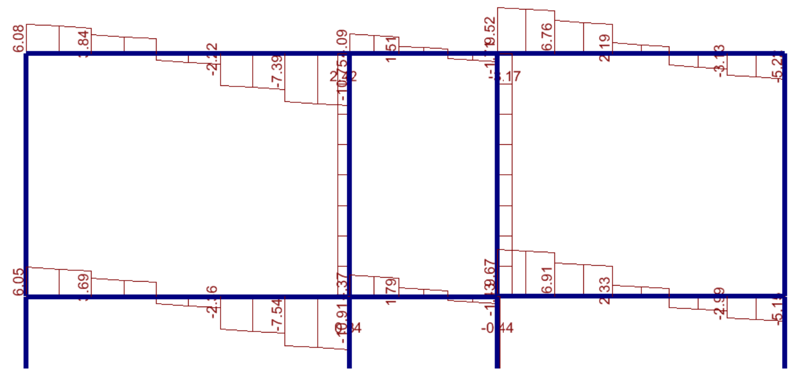


Shear force diagram

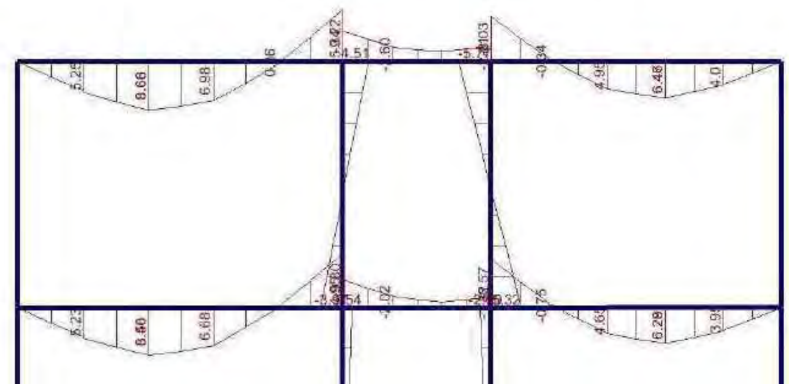


Bending moment diagram

Beam at axis D-D

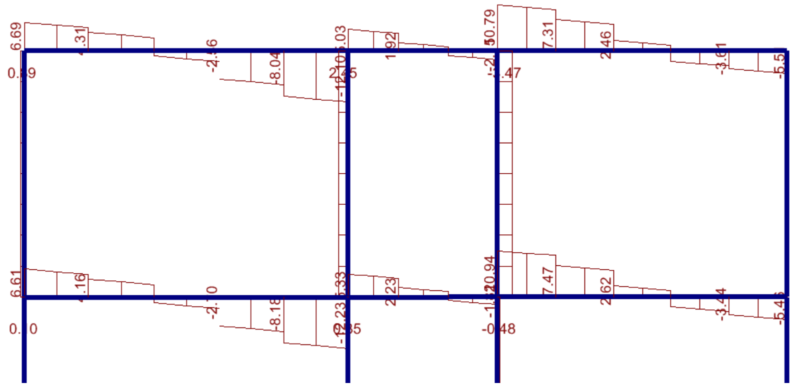


Shear force diagram

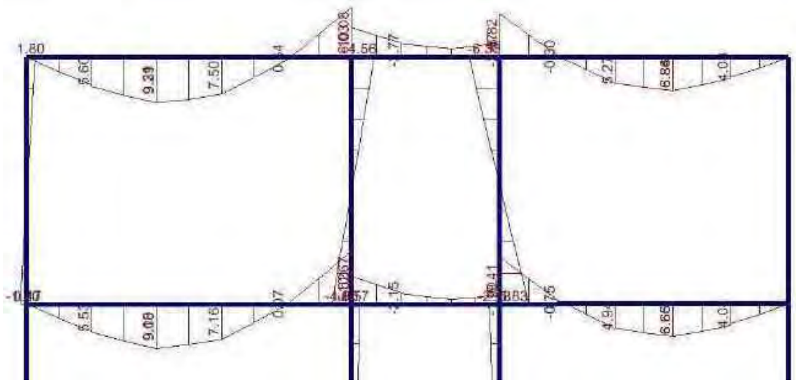


Bending moment diagram

Beam at axis E-E



Shear force diagram



Bending moment diagram



**6. DESIGN OF SECTIONS**

**Section (1):**

$M_{max}$ positive = 14.00 m.t. , Rec.- sec.

$F_{cu}$= 250 kg/cm$^2$ --------------- $F_c$ = 95 kg/cm$^2$

Steel type 360/520 **---------------** $F_s$ = 2000 kg/cm$^2$

$M_{design} = 14.00 \ t.m.$

$t = 70\,cm, \qquad b = 25\,cm$

$d = t - cover = 70 - 5 = 65\,cm$

$$d = k_1 \sqrt{\frac{M}{b}}$$

$$d = 65 = k_1 \sqrt{\frac{14.00 * 10^5}{25}} \ \ldots\ldots\ldots\ldots\ldots \ k_1 = 0.275$$

$$\rightarrow \ F_C = 80 \text{ kg/cm}^2 \prec F_{Callowable} = 95 \text{ kg/cm}^2$$

$$\Rightarrow \ k_2 = 1750$$

$$A_s = \frac{M}{k_2\, d} = \frac{14.00 * 10^5}{1750 * 65} = 12.31\, cm^2$$

$use \quad 7\, \phi\, 16\, mm$

$A_{s\,actual} = 14.07\, cm^2$

$$\mu_{actual} = \frac{A_s}{b\, d} = \frac{14.07}{25 * 65} x100 = 0.866\%$$

$$\mu_{min} = 0.225\frac{\sqrt{F_{cu}}}{f_y} = 0.225\frac{\sqrt{25}}{360}x100 = 0.3125\%$$

$$\geq \frac{1.1}{f_y} = \frac{1.1}{360}x100 = 0.306\%$$

$$\geq 0.15\% \text{ for high tensile steel}$$

$$\mu_{min} = 0.3125\%$$

$$\mu_{max} = 5x10^{-4}xF_{cu} = 5x10^{-4}x25x100 = 1.25\%$$

$$\mu_{max} = 1.25\% \succ \mu_{actual} = 0.866\% \succ \mu_{min} = 0.3125\%$$

**Section (2):**

$M_{max}$ positive = 10.00 m.t. , Rec.- sec.

$F_{cu}$= 250 kg/cm² -------------- $F_c$ = 95 kg/cm²

Steel type 360/520 -------------- $F_s$ = 2000 kg/cm²

$$M_{design} = 10.00 \ t.m.$$

$$t = 70\ cm, \qquad b = 25\ cm$$

$$d = t - cover = 70 - 5 = 65\ cm$$

$$d = k_1\sqrt{\frac{M}{b}}$$

$$d = 65 = k_1\sqrt{\frac{10.00*10^5}{25}} \ \ldots\ldots\ldots\ldots \ k_1 = 0.325$$

$$\Rightarrow F_C = 65 \text{ kg/cm}^2 \prec F_{C\ allowable} = 95 \text{ kg/cm}^2$$

$$\Rightarrow k_2 = 1782$$

$$A_s = \frac{M}{k_2\ d} = \frac{10.00*10^5}{1782*65} = 8.63\ cm^2$$

$$use \quad 5\,\phi\,16\ mm$$

$$A_{s\ actual} = 10.05\ cm^2$$

$$\mu_{actual} = \frac{A_s}{b\ d} = \frac{10.05}{25*65}x100 = 0.618\%$$

$$\mu_{min} = 0.225\frac{\sqrt{F_{cu}}}{f_y} = 0.225\frac{\sqrt{25}}{360}x100 = 0.3125\%$$

$$\geq \frac{1.1}{f_y} = \frac{1.1}{360}x100 = 0.306\%$$

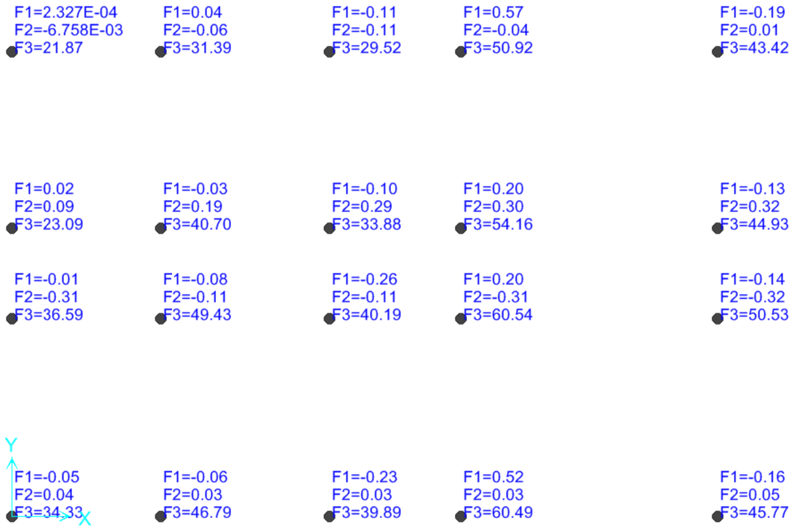
$$\geq 0.15\% \text{ for high tensile steel}$$

$$\mu_{min} = 0.3125\%$$

$$\mu_{max} = 5x10^{-4}xF_{cu} = 5x10^{-4}x25x100 = 1.25\%$$

$$\mu_{max} = 1.25\% \succ \mu_{actual} = 0.618\% \succ \mu_{min} = 0.3125\%$$

### 7. Loads on columns




Loads on columns

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## DESIGN OF COLUMNS

$$P = f_{c0} * A_c + 0.44\, f_y * A_s$$

| | | | | |
|---|---|---|---|---|
| C | 250 | $f_{c0}$ = | 60 | Kg /cm2 |
| st | 52 | $f_y$ = | 3600 | Kg /cm2 |

Assume As = 1 % Ac

P = 60 Ac + 0.44 * 3600 * 0.010 Ac

P = 60 Ac + 15.84 Ac

P = 75.84 Ac

take $\boxed{P = 75.84 \text{ Ac}}$

| Ac | Pw |
|---|---|
| 30 x 50 | 113 ton |

## 8. DESIGN OF FOUNDATIONS

**Design Of Isolated Footings**

**Subject to (Axial Load) By Working Method**

**(Rectangular Footings)/( Rectangular Stress Distribution )**

**[F1]**

| | | | |
|---|---|---|---|
| Bearing Capacity for Soil | $q_{soil}$ = | 6 | $t/m^2$ |
| Tensile Steel Stress | Fs= | 2000 | $Kg/Cm^2$ |
| Compressive Concrete Stress | Fc= | 95 | $Kg/Cm^2$ |
| Allowed Punching Stress | T p= | 90 | $t/m^2$ |
| Allowed Shear Stress | T s= | 90 | $t/m^2$ |
| Allowed Bond Stress | T b= | 12 | $Kg/Cm^2$ |

**Input Data**

| | | | |
|---|---|---|---|
| **Column Working Load** | **$P_w$ =** | **40** | **ton** |
| Column Dimension **(a > b)** | a = | 0.50 | m. |
| | b = | 0.30 | m. |
| Plain Concrete Depth | t = | 0.40 | m. |
| Plain Concrete Extension. | x = | 0.40 | m. |

Pt = 1.1 x P = 44 ton

Pt / (A x B)= 7.33 A x B= 7.33 ...(1)

(A-a) / 2 = (B-b) / 2 A= B+(a-b) ...(2)

$B^2$ + 0.20 x B = 7.33

| | | | | | |
|---|---|---|---|---|---|
| A= | 2.90 | m | B= | 2.70 | m |
| A1= | 2.10 | m | B1= | 1.90 | m |

**Check**

A x B = 7.83 $m^2$ ........>= 7.33 $m^2$ ...O.k.

$q_n$ = Pt/(A1*B1) = 11.03 $t/m^2$ **qn = (30-40)$t/m^2$**

**Dimension of R.C.**

$K_1$ = 0.242 $K_2$ = 1723

| | |
|---|---|
| $M_{1-1}$=qn * B1*(A1-a)$^2$/8 | $M_{2-2}$=$q_n$ * A1*(B1-b)$^2$/8 |
| $M_{1-1}$ = 6.70 m.t. | $M_{2-2}$ = 7.41 m.t. |
| d =$K_1$ (M1/B1)^0.5 | d =$K_1$(M2/A1)^0.5 |
| d= 15.0 Cm. | d= 15.0 Cm. |

Let d = 55 Cm. .... t1 = 60 Cm.

$K_1$ (New)= 0.926 ......$K_2$ = 1916

| | |
|---|---|
| As1= $M_{1-1}$/($K_2$*d) | As2= $M_{2-2}$/($K_2$*d) |
| As1= 6.36 $Cm^2$ | As2= 7.03 $Cm^2$ |

As min = μ %Ac= μ %(B1*df)= 12.38 $Cm^2$

| | |
|---|---|
| As = 16 ϕ 12 | As= 16 ϕ 12 |

**Choose**

| | | | | |
|---|---|---|---|---|
| As = | 16 | ϕ | 12 | in **Short** Direction(B) |
| As = | 16 | ϕ | 12 | in **Long** Direction(A) |

**1-Check Of Shear**

| | |
|---|---|
| Ts1 = Qsh1 /(B1*d) | Ts2 = Qsh2 /(A1*d) |
| Qsh1= qn*B1*((A1-a)/2-d) | Qsh2= qn*A1*((B1-b)/2-d) |
| Qsh1= 5.24 ton | Qsh2= 5.79 ton |
| Ts1 = 5.01 $t/m^2$ | Ts2 = 5.01 $t/m^2$ |
| Ts1 <90 ....O.k. | Ts2 <90 ....O.k. |

**2-Check Of Punching**

Qp= Pt - qn(a+d)*(b+d) = 34.16 ton.

Ap= 2((a+d)+(b+d))d = 2.09 $m^2$

Tp = Qp / Ap = 16.34 $t/m^2$

Tp<90 .....O.K.

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### 3-Check Of Bond

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Tb1 = | Qbh1 /(Eo*d) | | | Tb2 = | Qbh2 /(Eo*d) | | |
| | Qb1= | $q_n$*A1*(B1-b)/2 | | | Qb2= | $q_n$*B1*(A1-a)/2 | |
| | Qb1= | 18.53 | ton | | Qb2= | 16.76 | ton |
| | Tb1 = | 5.59 | $Kg/Cm^2$ | | Tb2 = | 5.06 | $Kg/Cm^2$ |
| Tb1 <12 | ....O.k. | | | Tb2 <12 | ....O.k. | | |

### Output Data

| | | | | | |
|---|---|---|---|---|---|
| a= | 0.50 | m. | b= | 0.30 | m. |
| A1= | 2.10 | m. | B1= | 1.90 | m. |
| A= | 2.90 | m. | B= | 2.70 | m. |

| | | | | |
|---|---|---|---|---|
| Thickness Of R.C | = | 0.60 | m. | |
| Thickness Of Plain Concrete | = | 0.40 | m. | |
| Reinforcement Of Dir. (B) Short | = | 16 | ο | 12 |
| Reinforcement Of Dir. (A) Long | = | 16 | ο | 12 |
| Volume Of R.C. | = | 2.39 | $m^3$ | |

## Design Of Isolated Footings
## Subject to (Axial Load) By Working Method
### (Rectangular Footings)/( Rectangular Stress Distribution )
### [F2]

| | | | |
|---|---|---|---|
| Bearing Capacity for Soil | $q_{soil}$ = | 6 | $t/m^2$ |
| Tensile Steel Stress | Fs= | 2000 | $Kg/Cm^2$ |
| Compressive Concrete Stress | Fc= | 95 | $Kg/Cm^2$ |
| Allowed Punching Stress | T p= | 90 | $t/m^2$ |
| Allowed Shear Stress | T s= | 90 | $t/m^2$ |
| Allowed Bond Stress | T b= | 12 | $Kg/Cm^2$ |

### Input Data

| | | | |
|---|---|---|---|
| **Column Working Load** | $P_w$ = | **48** | **ton** |
| Column Dimension **(a > b)** | a = | 0.50 | m. |
| | b = | 0.30 | m. |
| Plain Concrete Depth | t = | 0.40 | m. |
| Plain Concrete Extension. | x = | 0.40 | m. |

Pt = 1.1 x P = 52.80 ton

Pt / (A x B)= 8.80 A x B= 8.80 ...(1)

(A-a) / 2 = (B-b) / 2 A= B+(a-b) ...(2)

$B^2$ + 0.20 x B = 8.80

| | | | | | |
|---|---|---|---|---|---|
| A= | 3.10 | m | B= | 2.90 | m |
| A1= | 2.30 | m | B1= | 2.10 | m |

**Check**

A x B = 8.99 $m^2$ .........>= 8.80 $m^2$ ...O.k.

$q_r$ = Pt/(A1*B1) = 10.93 $t/m^2$ **qn = (30-40)$t/m^2$**

**Dimension of R.C.**

| | | | |
|---|---|---|---|
| $K_1$ = | 0.242 | $K_2$ = | 1723 |

| | |
|---|---|
| $M_{1-1}$=qn * B1*(A1-a)$^2$/8 | $M_{2-2}$=$q_n$ * A1*(B1-b)$^2$/8 |
| $M_{1-1}$ = 9.30 m.t. | $M_{2-2}$ = 10.18 m.t. |
| d =$K_1$ (M1/B1)^0.5 | d =$K_1$(M2/A1)^0.5 |
| d= 16.10 Cm. | d= 16.10 Cm. |

| | | | | | |
|---|---|---|---|---|---|
| Let d = | 55 | Cm. | .... t1 = | 60 | Cm. |
| $K_1$ (New)= | 0.827 | | ......$K_2$ = | 1907 | |

| | |
|---|---|
| As1= $M_{1-1}$/($K_2$*d) | As2= $M_{2-2}$/($K_2$*d) |
| As1= 8.86 $Cm^2$ | As2= 9.71 $Cm^2$ |

As min = μ %Ac= μ %(B1*df)= 18.98 $Cm^2$

| | | | | | |
|---|---|---|---|---|---|
| As = | 17 | ϕ 12 | As= | 17 | ϕ 12 |

**Choose**

| | | | | |
|---|---|---|---|---|
| As = | 17 | ϕ | 12 | in **Short** Direction(B) |
| As = | 17 | ϕ | 12 | in **Long** Direction(A) |

**1-Check Of Shear**

| | |
|---|---|
| Ts1 = Qsh1 /(B1*d) | Ts2 = Qsh2 /(A1*d) |
| Qsh1= qn*B1*((A1-a)/2-d) | Qsh2= qn*A1*((B1-b)/2-d) |
| Qsh1= 8.03 ton | Qsh2= 8.80 ton |
| Ts1 = 6.96 t/$m^2$ | Ts2 = 6.96 t/$m^2$ |
| Ts1 <90 ....O.k. | Ts2 <90 ....O.k. |

**2-Check Of Punching**

Qp= Pt - qn(a+d)*(b+d) = 43.04 ton.

Ap= 2((a+d)+(b+d))d = 2.09 $m^2$

Tp = Qp / Ap = 20.59 t/$m^2$

Tp<90 .....O.K.

**3-Check Of Bond**

| | |
|---|---|
| Tb1 = Qbh1 /(Eo*d) | Tb2 = Qbh2 /(Eo*d) |
| Qb1= $q_n$*A1*(B1-b)/2 | Qb2= $q_n$*B1*(A1-a)/2 |
| Qb1= 22.63 ton | Qb2= 20.66 ton |
| Tb1 = 6.42 Kg/$Cm^2$ | Tb2 = 5.86 Kg/$Cm^2$ |
| Tb1 <12 ....O.k. | Tb2 <12 ....O.k. |

**Output Data**

| | | | | | |
|---|---|---|---|---|---|
| a= | 0.50 | m. | b= | 0.30 | m. |
| A1= | 2.30 | m. | B1= | 2.10 | m. |
| A= | 3.10 | m. | B= | 2.90 | m. |

| | | | | |
|---|---|---|---|---|
| Thickness Of R.C | = | 0.60 | m. | |
| Thickness Of Plain Concrete | = | 0.40 | m. | |
| Reinforcement Of Dir. (B) Short | = | 17 | ϕ | 12 |
| Reinforcement Of Dir. (A) Long | = | 17 | ϕ | 12 |
| Volume Of R.C. | = | 2.90 | $m^3$ | |

# Design Of Isolated Footings
## Subject to (Axial Load) By Working Method
### (Rectangular Footings)/( Rectangular Stress Distribution )

**[F3]**

| | | | |
|---|---|---|---|
| Bearing Capacity for Soil | $q_{soil}$ = | 6 | $t/m^2$ |
| Tensile Steel Stress | Fs= | 2000 | $Kg/Cm^2$ |
| Compressive Concrete Stress | Fc= | 95 | $Kg/Cm^2$ |
| Allowed Punching Stress | T p= | 90 | $t/m^2$ |
| Allowed Shear Stress | T s= | 90 | $t/m^2$ |
| Allowed Bond Stress | T b= | 12 | $Kg/Cm^2$ |

**Input Data**

| | | | |
|---|---|---|---|
| **Column Working Load** | $P_w$ = | **62** | **ton** |
| Column Dimension **(a > b)** | a = | 0.50 | m. |
| | b = | 0.30 | m. |
| Plain Concrete Depth | t = | 0.40 | m. |
| Plain Concrete Extension. | x = | 0.40 | m. |

Pt = 1.1 x P = 68.20 ton

Pt / (A x B)= 11.37 A x B= 11.37 ...(1)

(A-a) / 2 = (B-b) / 2 A= B+(a-b) ...(2)

$B^2$ + 0.20 x B = 11.37

| | | | | | |
|---|---|---|---|---|---|
| A= | 3.50 | m | B= | 3.30 | m |
| A1= | 2.70 | m | B1= | 2.50 | m |

**Check**

A x B = 11.55 $m^2$ .........>= 11.37 $m^2$ ...O.k.

$q_n$ = Pt/(A1*B1) = 10.10 $t/m^2$ **qn = (30-40)t/m²**

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**Dimension of R.C.**

| | | | |
|---|---|---|---|
| $K_1$ = | 0.242 | $K_2$ = | 1723 |

| | | | | | |
|---|---|---|---|---|---|
| $M_{1-1}$=qn * B1*(A1-a)²/8 | | | $M_{2-2}$=$q_n$ * A1*(B1-b)²/8 | | |
| $M_{1-1}$ = | 15.28 | m.t. | $M_{2-2}$ = | 16.50 | m.t. |
| d =$K_1$ (M1/B1)^0.5 | | | d =$K_1$(M2/A1)^0.5 | | |
| d= | 19.00 | Cm. | d= | 19.00 | Cm. |

| | | | | | |
|---|---|---|---|---|---|
| Let d = | 55 | Cm. | .... t1 = | 60 | Cm. |
| $K_1$ (New)= | 0.703 | | ......$K_2$ = | 1891 | |

| | | | | | |
|---|---|---|---|---|---|
| As1= | $M_{1-1}$/($K_2$*d) | | As2= | $M_{2-2}$/($K_2$*d) | |
| As1= | 14.69 | $Cm^2$ | As2= | 15.87 | $Cm^2$ |

As min = μ %Ac= μ %(B1*df)= 18.98 $Cm^2$

| | | | | | |
|---|---|---|---|---|---|
| As = | 19 | ϕ 12 | As= | 20 | ϕ 12 |

**Choose**

| | | | | |
|---|---|---|---|---|
| As = | 20 | ϕ | 12 | in **Short** Direction(B) |
| As = | 19 | ϕ | 12 | in **Long** Direction(A) |

**1-Check Of Shear**

| | | | | | |
|---|---|---|---|---|---|
| Ts1 = | Qsh1 /(B1*d) | | Ts2 = | Qsh2 /(A1*d) | |
| Qsh1= | qn*B1*((A1-a)/2-d) | | Qsh2= | qn*A1*((B1-b)/2-d) | |
| Qsh1= | 13.89 | ton | Qsh2= | 15.00 | ton |
| Ts1 = | 10.10 | $t/m^2$ | Ts2 = | 10.10 | $t/m^2$ |
| Ts1 <90 | ....O.k. | | Ts2 <90 | ....O.k. | |

**2-Check Of Punching**

Qp= Pt - qn(a+d)*(b+d) = 59.18 ton.

Ap= 2((a+d)+(b+d))d = 2.09 $m^2$

Tp = Qp / Ap = 28.32 $t/m^2$

Tp<90 .....O.K.

### 3-Check Of Bond

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Tb1 = | Qbh1 /(Eo*d) | | | Tb2 = | Qbh2 /(Eo*d) | | |
| Qb1= | $q_n$*A1*(B1-b)/2 | | | Qb2= | $q_n$*B1*(A1-a)/2 | | |
| Qb1= | 30.01 | ton | | Qb2= | 27.79 | ton | |
| Tb1 = | 7.24 | $Kg/Cm^2$ | | Tb2 = | 7.06 | $Kg/Cm^2$ | |
| Tb1 <12 | ....O.k. | | | Tb2 <12 | ....O.k. | | |

**Output Data**

| | | | | | |
|---|---|---|---|---|---|
| a= | 0.50 | m. | b= | 0.30 | m. |
| A1= | 2.70 | m. | B1= | 2.50 | m. |
| A= | 3.50 | m. | B= | 3.30 | m. |
| Thickness Of R.C | = | 0.60 | m. | | |
| Thickness Of Plain Concrete | = | 0.40 | m. | | |
| Reinforcement Of Dir. (B) Short | = | 20 | ø | 12 | |
| Reinforcement Of Dir. (A) Long | = | 19 | ø | 12 | |
| Volume Of R.C. | = | 4.05 | $m^3$ | | |



# Design Of Isolated Footings
## Subject to (Axial Load) By Working Method
### (Rectangular Footings)/( Rectangular Stress Distribution )
**[F4]**

| | | | |
|---|---|---|---|
| Bearing Capacity for Soil | $q_{soil}$ = | 6 | $t/m^2$ |
| Tensile Steel Stress | Fs= | 2000 | $Kg/Cm^2$ |
| Compressive Concrete Stress | Fc= | 95 | $Kg/Cm^2$ |
| Allowed Punching Stress | T p= | 90 | $t/m^2$ |
| Allowed Shear Stress | T s= | 90 | $t/m^2$ |
| Allowed Bond Stress | T b= | 12 | $Kg/Cm^2$ |

**Input Data**

**Exterior Column**

| | | | |
|---|---|---|---|
| Column Load | P1= | 41 | ton |
| Column Dimension **(a>b)** | a1= | 0.50 | m. |
| | b1= | 0.30 | m. |

**Interior Column**

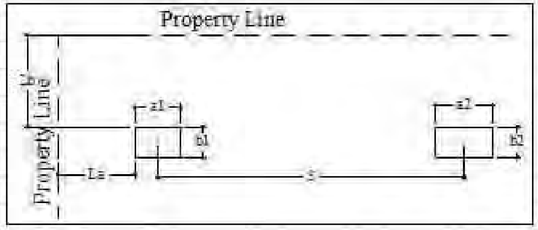
| | | | |
|---|---|---|---|
| Column Load..( **P2 > P1** ) | P2= | 50 | ton |
| Column Dimension **(a>b)** | a2= | 0.30 | m. |
| | b2= | 0.50 | m. |
| Plain Concrete Depth | t1= | 0.40 | m. |
| Plain Concrete **Ext.** | x1= | 0.40 | m. |
| Distance Between The **C.G** of the Columns (S) = | | 2.40 | m. |
| Distance Bet. Property Line & exterior Col. (La) = | | 2.00 | m. |
| Distance Bet. Property Line & exterior Col. (Lb) = | | 2.00 | m. |

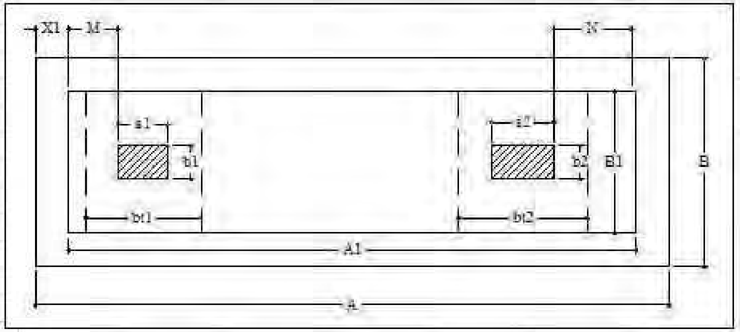
R = P1+P2 = 91.00 ton

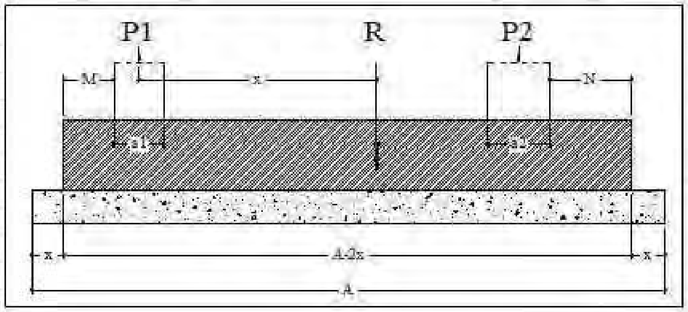
Rt =1.1xR = 100.10 ton

In order to have uniform pressure on the soil.
the **Resultant** must coincide on the center of gravity of the P.C.
the **Resultant** found at distance **x** from the exterior Column **CL**=(P2*S)/R

x = 1.32 m.




| | | | | |
|---|---|---|---|---|
| Length of P.C. (A) **not exceed** | 6.34 | m. | ...O.k. | |
| Length of P.C. (A) **not least** | 3.14 | m. | ...O.k. | |
| Breadth of P.C. (B) **not exceed** | 3.90 | m. | ...O.k. | |

| | | | | | | |
|---|---|---|---|---|---|---|
| | A (P.C.)= | Rt/q | .......>> | A (P.C.)= | 16.68 | $m^2$. |
| | A= | 5.60 | | B= | 3.00 | |
| | A1= | 4.80 | | B1= | 2.20 | |
| **Check** | A1 x B 1= | 16.80 | ..........>= 16.68 | | | ...O.k. |
| | qn = R/(A1xB1)= | 8.62 | $t/m^2$ | **qn = (30-40)$t/m^2$** | | ...O.k. |

**Structural Analysis**

| | | | |
|---|---|---|---|
| W=qnxB1 | W= | 18.96 | t/m` |

**Shearing Force**

| | | |
|---|---|---|
| q1=W*M= | 15.76 | ton. |
| q2=W*(M+a1)-P1= | 15.76 | ton. |
| q3=P2-W*(N+a2)= | 22.16 | ton. |
| q4=-W*N= | 22.16 | ton. |

**Max B.M. @ Zero Shear**

**x1=** P1/W1= 2.16 m.

$Mmax(-)=P1(x1-M-.5*a1)-W*x1^2*0.5=$

| | | |
|---|---|---|
| Mmax(+)ve = | 0.00 | m.t. |
| $M1=W*.5*M^2=$ | 6.55 | |
| $M2=W*.5*(M+a1)^2-P1*a1/2=$ | 6.55 | |
| $M3=W*.5*(N+a2)^2-P2*.5*a2=$ | 12.95 | m.t. |
| $M4=W*N^2/2=$ | 12.95 | m.t. |
| max (-)ve B.M. = | 12.95 | m.t. |
| M=0.5*A1-x-0.5*a1 M= | **0.83** | m.t. |
| N=0.5*A1-(S-x)-0.5*a2 N= | **1.17** | m.t. |

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**Design of Foundation**

.B1 = 2.20 m        Md = 12.95 m.t.

K1 = 0.484        K2 = 1847

d =$K_1$ (M1/B1)^0.5

d = 37.20 cm

let d= 55 cm    t = 60 cm.

K1 = 0.717        K2 = 1893

As min = μ %Ac = μ % (B1*df) = 18.15 $Cm^2$

Choose

As= 13 ϕ 16 Upper Steel

## 1-Check Of Shear

| | For Column (A) | | | For Column (B) | | |
|---|---|---|---|---|---|---|
| | Ts1 = | Qsh1 /(B1*d) | | Ts2 = | Qsh2 /(B1*d) | |
| | Qsh1= | qn*B1*(M+a1+df)-P1 | | Qsh2= | P2-qn*B1*(N+a2+df) | |
| | Qsh1= | 1.06 | ton | Qsh2= | 11.73 | ton |
| | Ts1 = | 0.88 | $t/m^2$ | Ts2 = | 9.69 | $t/m^2$ |
| | Ts1 <60 | ....O.k. | | Ts2 <60 | ....O.k. | |

## 2-Check Of Punch

For Column (B)

| | | | |
|---|---|---|---|
| Qp= | P2-qn(a2+df)*(b2+df) = | 42.31 | ton. |
| Ap= | d*2((a2+df)+(b2+df)) = | 2.09 | $m^2$ |
| | Tp= Qp/Ap= | 20.24 | $t/m^2$ |
| | | Tp<80 | .....O.K. |

For Column (A)

| | | | |
|---|---|---|---|
| Qp= | P1-qn(a1+df)*(b1+df) = | 33.31 | ton. |
| Ap= | d*2((a1+df)+(b1+df)) = | 2.09 | $m^2$ |
| | Tp= Qp/Ap= | 15.94 | $t/m^2$ |
| | | Tp<80 | .....O.K. |

## Reinforcement of Main Longitudinal Direction

As required to resist **M1=** **12.95** m.t.
As required to resist **M2=** **6.55** m.t.

| | | | | | |
|---|---|---|---|---|---|
| As1= | M1/(K2xd) | | As2= | M2/(K2xd) | |
| As1= | 12.44 | $Cm^2$ | As2= | 6.29 | $Cm^2$ |
| As min = | μ %Ac= | μ %(B1*df)= | 18.15 | $Cm^2$ | |
| As= | 7 | ϕ 16 | As= | 4 | ϕ 16 |
| As= | 5 | ϕ 18 | As= | 3 | ϕ 18 |
| As= | 4 | ϕ 22 | As= | 2 | ϕ 22 |

**Choose**

| | | | | |
|---|---|---|---|---|
| As= | 13 | ϕ | 16 | For **M1** |
| As= | 13 | ϕ | 16 | For **M2** |

## Check Of Bond

| For Reif. Resisting M1 | | | For Reif. Resisting M2 | | |
|---|---|---|---|---|---|
| Tb1 = | Qbh1 /(Eo*d2) | | Tb2= | Qbh2 /(Eo*d2) | |
| Qb1= | qn*B1*N | | Qb2= | qn*B1*M | |
| Qb1= | 22.16 | ton | Qb2= | 15.76 | ton |
| Tb1 = | 7.09 | $Kg/Cm^2$ | Tb2 = | 5.04 | $Kg/Cm^2$ |
| Tb1 <10 | ....O.k. | | Tb2 <10 | ....O.k. | |
| Tb = 12 $Kg/Cm^2$ For U-Shape | | | Tb = 10 $Kg/Cm^2$ For L-Shape | | |

**Summary table of the control house calculations**

| | Condition | | | |
|---|---|---|---|---|
| Slabs | One way slabs & Two way slabs | ts = 12 & 14 & 15 cm<br>steel reinforcement d10 mm, d12 mm | | |
| | | $Fc = 60\ kg/cm^2$ < | $Fc_{all.} = 75\ kg/cm^2$ | Ok. |
| | | $Fs = 1600\ kg/cm^2$ < | $Fc_{all.} = 2000\ kg/cm^2$ | Ok. |
| Main beams | Projected Rectangular Sections | b = 12 cm, 25 cm<br>t = 60 cm, 70 cm<br>steel reinforcement d16 mm, d12 mm | | |
| | | $Fc = 80\ kg/cm^2$ < | $Fc_{all.} = 95\ kg/cm^2$ | Ok. |
| | | $Fs = 1800\ kg/cm^2$ < | $Fc_{all.} = 2000\ kg/cm^2$ | Ok. |
| Columns | Rectangular Sections | b = 30 cm<br>t = 50 cm<br>steel reinforcement 10 d16 mm | | |
| | | $Fc = 35\ kg/cm^2$ < | $Fc_{all.} = 60\ kg/cm^2$ | Ok. |
| | | $Fs = 1100\ kg/cm^2$ < | $Fc_{all.} = 2000\ kg/cm^2$ | Ok. |
| Foundations | Isolated & Combined Rectangular Footings | P. C. thickness = 40 cm<br>R. C. thickness = 60 cm<br>steel reinforcement d16 mm, d12 mm | | |
| | | Fc actual < | $Fc_{all.} = 60\ kg/cm^2$ | Ok. |
| | | Fs actual < | $Fc_{all.} = 2000\ kg/cm^2$ | Ok. |
| | | Actual Shear Stress << | $90\ kg/cm^2$ | Ok. |
| | | Actual Punching Stress < | $90\ kg/cm^2$ | Ok. |
| | | Actual Bond Stress < | $12\ kg/cm^2$ | Ok. |

## A-10-2. Design of local control house

## 1. REFERENCE CODES:

The design process depends mainly in the following Codes

1) The Egyptian code for design and construction of concrete structures, Code No. (203), Ministerial decree No. 44 – 2007, Ministry of housing, utilities and urban communities, 2007.
2) The Egyptian code for loads and forces structures, Code No. (203), Ministerial decree No. 44 – 2007, Ministry of housing, utilities and urban communities, 2007.

## 2. MATERIALS:

**Concrete**

- Concrete class is used as the following table

| Portion to be used | Cube Design Strength $N/mm^2$ | Unit Weight $kN/m^3$ |
|---|---|---|
| Reinforced Concrete | 25 | 25 |
| Plain Concrete | 20 | 23 |
| Leveling Concrete | 20 | 23 |

[Clause 2-3-1-1/203]

- Modulus of Elasticity $E_c$

$$E_C = 4400\sqrt{F_{cu}} \quad N/mm^2 = 4400\sqrt{25} = 22000 \quad N/mm^2 = 2.2\text{x}10^5 \quad \text{kg/cm}^2$$

[Eq. 2.1/203]

- For the working stress design.

Allowable compressive strength for bending = 9.50 $N/mm^2$ (95 $kg/cm^2$).
Allowable shearing strength by concrete only (beams)
= 0.70 $N/mm^2$ (7 $kg/cm^2$).
Allowable shearing strength by concrete only (slabs & footings)
= 0.90 $N/mm^2$ (7 $kg/cm^2$).
Allowable shearing strength by concrete and diagonal tension bars =
1.90 $N/mm^2$ (19 $kg/cm^2$).
[Table. 5-1/203]

- For limit state design

$\gamma_c$ = 1.50 [Eq. 3-15-a/203]

**Reinforcing Steel**

- Steel type 360/520 is used for all diameters of reinforcing bars (main, secondary reinforcing, side bars and stirrups) in the bridge under study.
- Mass Density = 7.85 $ton/m^3$
- Modulus of Elasticity $E_s$

$E_s = 2.0\text{x}10^6$ $kg/cm^2$ [Fig. 4-1/203]

- For the working stress design.

Allowable stress for tension and compression = 200 $N/mm^2$ (2000 $kg/cm^2$).
[Table. 5-1/203]

For limit state design

Yield stress for tension and compression = 360 $N/mm^2$ (3600 $kg/cm^2$)
$\gamma_s$ = 1.15 [Eq. 3-15-b/203]

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### 3. LIVE LOADS: (according to the Egyptian code)

Vertical loads are represented by uniformly distributed loads (UDL system):

[Table 4-1 /201]

a) UDL = 300 kg/m2 for office rooms.
b) UDL = 400 kg/m2 for stairs.
c) UDL = 400 kg/m2 for balconies.

### 4. DEAD LOADS

**Slab Loads**

- Assume the weight of flooring layer = 200 kg / m$^2$.

- If the thickness of R. C. slab ( $t_s$ ) = 12 cm
  Own weight of slab = 0.12x2500 = 300 kg / m$^2$

The total dead load ($w_{D.L.}$) = 300+200 = 500 kg / m$^2$

- If the thickness of R. C. slab ( $t_s$ ) = 14 cm
  Own weight of slab = 0.14x2500 = 350 kg / m$^2$

The total dead load ($w_{D.L.}$) = 300+200 = 550 kg / m$^2$

- If the thickness of R. C. slab ( $t_s$ ) = 15 cm
  Own weight of slab = 0.15x2500 = 375 kg / m$^2$

The total dead load ($w_{D.L.}$) = 300+200 = 575 kg / m$^2$

- If the thickness of R. C. slab ( $t_s$ ) = 16 cm
  Own weight of slab = 0.16x2500 = 400 kg / m$^2$

The total dead load ($w_{D.L.}$) = 300+200 = 600 kg / m$^2$

**Own weight of beams**

- For beam section 25x60
  Own weight = 0.25*(0.60-0.12)*2500 = 300 kg/m\

- For beam section 30x80
  Own weight = 0.30*(0.80-0.12)*2500 = 510 kg/m\

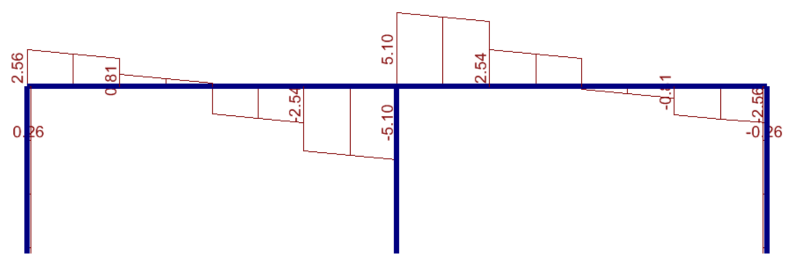
**Own weight of walls**

- For walls 12 cm thickness
  Weight of wall = 220 kg/m$^2$

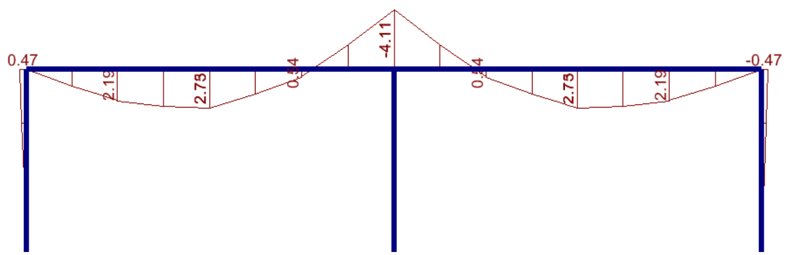
- For walls 25 cm thickness
  Weight of wall = 375 kg/m$^2$

## 5. STRAINING ACTIONS ON BEAMS

Beam at axis 1-1 & Beam at axis 3-3

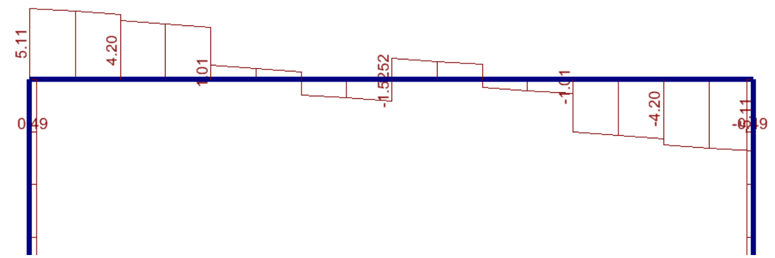



Shear force diagram

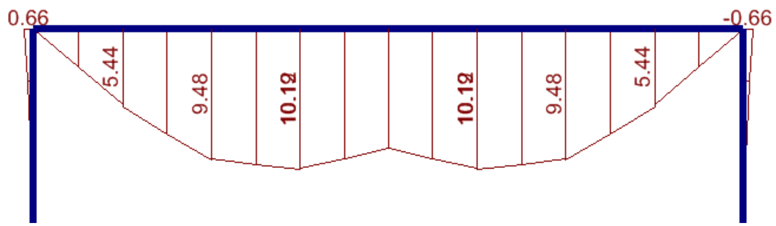



Bending moment diagram

Beam at axis 2-2

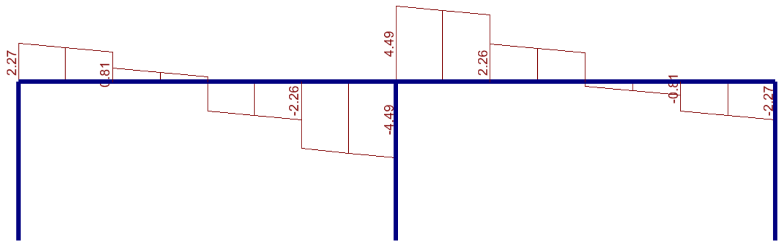



Shear force diagram

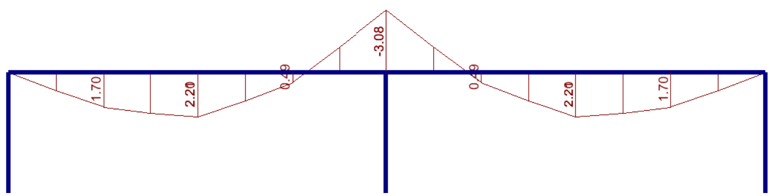



Bending moment diagram

Beam at axis A-A & Beam at axis C-C

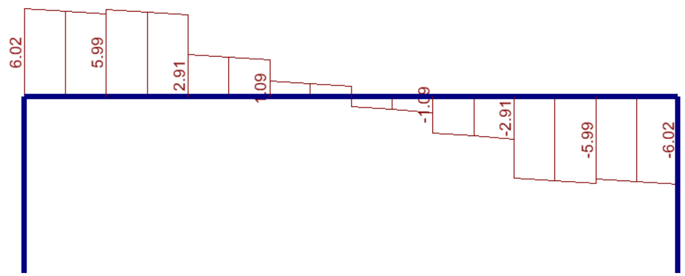



Shear force diagram

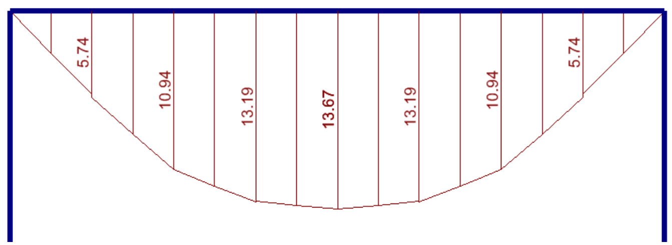



Bending moment diagram

Beam at axis B-B




Shear force diagram




Bending moment diagram

## 6. DESIGN OF SECTIONS

**Section (1):**

$M_{max}$ positive = 10.20 m.t. , T- sec.

$F_{cu}$= 250 kg/cm$^2$ --------------- $F_c$ = 95 kg/cm$^2$

Steel type 360/520 --------------- $F_s$ = 2000 kg/cm$^2$

The effective breadth of the section ($b_e$) equal to the least of the following:

- $b_e = 16\,t_s + b_w = 16x12 + 30 = 222$ cm
- $b_e$ = from center to center of girders = 337 cm
- $b_e = L_1/5 + b_w = (675)/5 + 30 = 165$ cm

$b_e = 165$ cm [Clause 5-3-3-6-1 /203]

$$M_{design} = 10.20\ t.m.$$

$$d = t - cover = 60 - 5 = 55\,cm$$

$$d = k\sqrt{\frac{M}{b_e}}$$

$$d = 55 = k_1\sqrt{\frac{10.20*10^5}{165}} \ \ \ldots\ldots\ldots\ldots\ \ k_1 = 0.700$$

$$\Rightarrow F_C = 30\ \text{kg/cm}^2 \prec F_{C\,allowable} = (2/3)x95 = 63\ \text{kg/cm}^2$$

$$\Rightarrow k_2 = 1885$$

[Clause 5-3-2-6 /203]

$$A_s = \frac{M}{k_2\,d} = \frac{10.20*10^5}{1885*55} = 9.84\,cm^2$$

$$use \quad 5\,\phi\,16\,mm$$

$$A_{s\,actual} = 10.05\,cm^2$$

$$\mu_{actual} = \frac{A_s}{b\,d} = \frac{10.05}{30*55}x100 = 0.609\%$$

$$\mu_{min} = 0.225\frac{\sqrt{F_{cu}}}{f_y} = 0.225\frac{\sqrt{25}}{360}x100 = 0.3125\%$$

$$\geq \frac{1.1}{f_y} = \frac{1.1}{360}x100 = 0.306\%$$

$$\geq 0.15\%\ \text{for high tensile steel}$$

$$\mu_{min} = 0.3125\%$$

$$\mu_{max} = 5x10^{-4}xF_{cu} = 5x10^{-4}x25x100 = 1.25\%$$

$$\mu_{max} = 1.25\% \succ \mu_{actual} = 0.609\% \succ \mu_{min} = 0.3125\%$$

**Section (2):**

$M_{max}$ positive = 13.70 m.t. , T- sec.

$F_{cu}$= 250 kg/cm$^2$ --------------- $F_c$ = 95 kg/cm$^2$

Steel type 360/520 --------------- $F_s$ = 2000 kg/cm$^2$

The effective breadth of the section ($b_e$) equal to the least of the following:

- $b_e = 16\,t_s + b_w = 16x12 + 30 = 222$ cm
- $b_e$ = from center to center of girders = 337 cm
- $b_e = L_1/5 + b_w = (775)/5 + 30 = 185$ cm

$b_e = 185$ cm [Clause 5-3-3-6-1 /203]

$M_{design} = 13.70 \ t.m.$

$d = t - cover = 60 - 5 = 55 \, cm$

$$d = k_1 \sqrt{\frac{M}{b_e}}$$

$$d = 55 = k_1 \sqrt{\frac{13.70 * 10^5}{185}} \ .............. \ k_1 = 0.639$$

$$\Rightarrow \ F_C = 30 \, kg/cm^2 \prec F_{C allowable} = (2/3)x95 = 63 \, kg/cm^2$$

$$\Rightarrow \ k_2 = 1880$$

**[Clause 5-3-2-6 /203]**

$$A_s = \frac{M}{k_2 \, d} = \frac{13.70 * 10^5}{1880 * 55} = 13.25 \, cm^2$$

$use \quad 7 \, \phi 16 \, mm$

$A_{s \, actual} = 14.07 \, cm^2$

$$\mu_{actual} = \frac{A_s}{b \, d} = \frac{14.07}{30 * 55} x100 = 0.853\%$$

$$\mu_{min} = 0.225 \frac{\sqrt{F_{cu}}}{f_y} = 0.225 \frac{\sqrt{25}}{360} x100 = 0.3125\%$$

$$> \frac{1.1}{f_y} = \frac{1.1}{360} x100 = 0.306\%$$

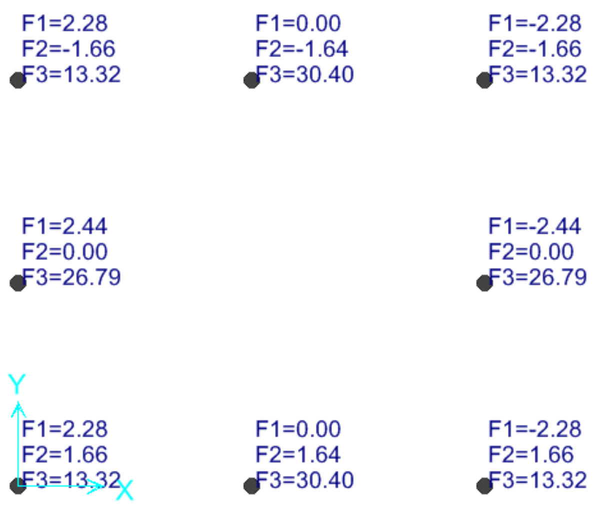
$\geq 0.15\%$ for high tensile steel

$\mu_{min} = 0.3125\%$

$\mu_{max} = 5x10^{-4} x F_{cu} = 5x10^{-4} x25x100 = 1.25\%$

$\mu_{max} = 1.25\% \succ \ \mu_{actual} = 0.853\% \ \succ \ \mu_{min} = 0.3125\%$

## 7. Loads on columns




Loads on columns

## DESIGN OF COLUMNS

$$P = f_{c0} * A_c + 0.44 f_y * A_s$$

C 250 $f_{c0}$ = 60 Kg /cm2

st 52 $f_y$ = 3600 Kg /cm2

Assume As = 1 % Ac

P = 60 Ac + 0.44 * 3600 * 0.010 Ac

P = 60 Ac + 15.84 Ac

P = 75.84 Ac

take **P = 75.84 Ac**

| Ac | Pw |
|---|---|
| 40 x 40 | 121 ton |

## 8. DESIGN OF FOUNDATIONS

**Design Of Isolated Footings**

**Subject to (Axial Load) By Working Method**

**(Rectangular Footings)/( Rectangular Stress Distribution )**

**[F2]**

| | | | |
|---|---|---|---|
| Bearing Capacity for Soil | $q_{soil}$ = | 5 | t/m$^2$ |
| Tensile Steel Stress | Fs= | 2000 | Kg/Cm$^2$ |
| Compressive Concrete Stress | Fc= | 95 | Kg/Cm$^2$ |
| Allowed Punching Stress | T p= | 90 | t/m$^2$ |
| Allowed Shear Stress | T s= | 90 | t/m$^2$ |
| Allowed Bond Stress | T b= | 12 | Kg/Cm$^2$ |

**Input Data**

| | | | |
|---|---|---|---|
| **Column Working Load** | $P_w$ = | **20** | **ton** |
| Column Dimension **(a > b)** | a = | 0.40 | m. |
| | b = | 0.40 | m. |
| Plain Concrete Depth | t = | 0.40 | m. |
| Plain Concrete Extension. | x = | 0.40 | m. |

Pt = 1.1 x P = 22.00 ton

Pt / (A x B)= 4.40 A x B= 4.40 ...(1)

(A-a) / 2 = (B-b) / 2 A= B+(a-b) ...(2)

$B^2$ + 0.00 x B = 4.40 m.

| | | | | | |
|---|---|---|---|---|---|
| A= | 2.30 | m | B= | 2.30 | m |
| A1= | 1.50 | m | B1= | 1.50 | m |

**Check**

A x B = 4.40 m$^2$ .........>= 4.40 m$^2$ ...O.k.

$q_r$ = Pt/(A1*B1) = 9.78 t/m$^2$ **qn = (30-40)t/m$^2$**

**Dimension of R.C.**

| | | | |
|---|---|---|---|
| $K_1$ = | 0.346 | $K_2$ = | 1794 |

| | | | | | |
|---|---|---|---|---|---|
| $M_{1-1}$=qn * B1*(A1-a)$^2$/8 | | | $M_{2-2}$=$q_n$ * A1*(B1-b)$^2$/8 | | |
| $M_{1-1}$ = | 2.22 | m.t. | $M_{2-2}$ = | 2.22 | m.t. |
| d =$K_1$ (M1/B1)^0.5 | | | d =$K_1$(M2/A1)^0.5 | | |
| d= | 15.0 | Cm. | d= | 15.0 | Cm. |

| | | | | | |
|---|---|---|---|---|---|
| Let d = | 55 | Cm. | .... t1 = | 60 | Cm. |
| $K_1$ (New)= | 1.43 | | ......$K_2$ = | 1945 | |

| | | | | | |
|---|---|---|---|---|---|
| As1= | $M_{1-1}$/($K_2$*d) | | As2= | $M_{2-2}$/($K_2$*d) | |
| As1= | 2.10 | $Cm^2$ | As2= | 2.10 | $Cm^2$ |

As min = µ %Ac= µ %(B1*df)= 12.38 $Cm^2$

| | | | | | |
|---|---|---|---|---|---|
| As = | 12 | φ 12 | As= | 12 | φ 12 |

**Choose**

| | | | | |
|---|---|---|---|---|
| As = | 12 | φ | 12 | in **Short** Direction(B) |
| As = | 12 | φ | 12 | in **Long** Direction(A) |

**1-Check Of Shear**

| | | | | | |
|---|---|---|---|---|---|
| Ts1 = | Qsh1 /(B1*d) | | Ts2 = | Qsh2 /(A1*d) | |
| Qsh1= | qn*B1*((A1-a)/2-d) | | Qsh2= | qn*A1*((B1-b)/2-d) | |
| Qsh1= | 0 | ton | Qsh2= | 0 | ton |
| Ts1 = | 0 | $t/m^2$ | Ts2 = | 0 | $t/m^2$ |
| Ts1 <90 | ....O.k. | | Ts2 <90 | ....O.k. | |

**2-Check Of Punching**

| | | | | |
|---|---|---|---|---|
| Qp= | Pt - qn(a+d)*(b+d) | = | 13.1822.69 | ton. |
| Ap= | 2((a+d)+(b+d))d | = | 2.09 | $m^2$ |
| Tp | = Qp / Ap | = | 6.30 | $t/m^2$ |
| | | Tp<90 | | .....O.K. |

**3-Check Of Bond**

| | | | | | |
|---|---|---|---|---|---|
| Tb1 = | Qbh1 /(Eo*d) | | Tb2 = | Qbh2 /(Eo*d) | |
| Qb1= | $q_n$*A1*(B1-b)/2 | | Qb2= | $q_n$*B1*(A1-a)/2 | |
| Qb1= | 8.07 | ton | Qb2= | 8.07 | ton |
| Tb1 = | 3.24 | $Kg/Cm^2$ | Tb2 = | 3.24 | $Kg/Cm^2$ |
| Tb1 <12 | ....O.k. | | Tb2 <12 | ....O.k. | |

**Output Data**

| | | | | | |
|---|---|---|---|---|---|
| a= | 0.40 | m. | b= | 0.40 | m. |
| A1= | 1.50 | m. | B1= | 1.50 | m. |
| A= | 2.30 | m. | B= | 2.30 | m. |

| | | | | |
|---|---|---|---|---|
| Thickness Of R.C | = | 0.60 | m. | |
| Thickness Of Plain Concrete | = | 0.40 | m. | |
| Reinforcement Of Dir. (B) Short | = | 12 | φ | 12 |
| Reinforcement Of Dir. (A) Long | = | 12 | φ | 12 |
| Volume Of R.C. | = | 1.35 | $m^3$ | |

# Design Of Isolated Footings
## Subject to (Axial Load) By Working Method
### (Rectangular Footings)/( Rectangular Stress Distribution )

**[F3]**

| | | | |
|---|---|---|---|
| Bearing Capacity for Soil | $q_{soil}$ = | 5 | $t/m^2$ |
| Tensile Steel Stress | Fs= | 2000 | $Kg/Cm^2$ |
| Compressive Concrete Stress | Fc= | 95 | $Kg/Cm^2$ |
| Allowed Punching Stress | T p= | 90 | $t/m^2$ |
| Allowed Shear Stress | T s= | 90 | $t/m^2$ |
| Allowed Bond Stress | T b= | 12 | $Kg/Cm^2$ |

**Input Data**

| | | | |
|---|---|---|---|
| **Column Working Load** | **$P_w$ =** | **35** | **ton** |
| Column Dimension (a > b) | a = | 0.40 | m. |
| | b = | 0.40 | m. |
| Plain Concrete Depth | t = | 0.40 | m. |
| Plain Concrete Extension. | x = | 0.40 | m. |

Pt = 1.1 x P = 38.50 ton

Pt / (A x B)= 7.70 A x B= 7.70 ...(1)

(A-a) / 2 = (B-b) / 2 A= B+(a-b) ...(2)

$B^2$ + 0.00 x B = 7.70 m.

| | | | | | |
|---|---|---|---|---|---|
| A= | 2.80 | m | B= | 2.80 | m |
| A1= | 2.00 | m | B1= | 2.00 | m |

**Check**

A x B = 7.84 $m^2$ .........>= 7.70 $m^2$ ...O.k.

$q_n$ = Pt/(A1*B1) = 9.63 $t/m^2$ **qn = (30-40)$t/m^2$**

**Dimension of R.C.**

$K_1$ = 0.346 $K_2$ = 1794

| | |
|---|---|
| $M_{1-1}$=qn * B1*(A1-a)$^2$/8 | $M_{2-2}$=$q_n$ * A1*(B1-b)$^2$/8 |
| $M_{1-1}$ = 6.16 m.t. | $M_{2-2}$ = 6.16 m.t. |
| d =$K_1$ (M1/B1)^0.5 | d =$K_1$(M2/A1)^0.5 |
| d= 15 Cm. | d= 15 Cm. |

Let d = 55 Cm. .... t1 = 60 Cm.

$K_1$ (New)= 0.991 ......$K_2$ = 1922

| | |
|---|---|
| As1= $M_{1-1}$/($K_2$*d) | As2= $M_{2-2}$/($K_2$*d) |
| As1= 5.83 $Cm^2$ | As2= 5.83 $Cm^2$ |

As min = μ %Ac= μ %(B1*df)= 16.50 $Cm^2$

| | |
|---|---|
| As = 15 ϕ 12 | As= 15 ϕ 12 |

**Choose**

| | | | | |
|---|---|---|---|---|
| As = | 15 | ϕ | 12 | in **Short** Direction(B) |
| As = | 15 | ϕ | 12 | in **Long** Direction(A) |

**1-Check Of Shear**

| | |
|---|---|
| Ts1 = Qsh1 /(B1*d) | Ts2 = Qsh2 /(A1*d) |
| Qsh1= qn*B1*((A1-a)/2-d) | Qsh2= qn*A1*((B1-b)/2-d) |
| Qsh1= 4.81 ton | Qsh2= 4.81 ton |
| Ts1 = 4.38 $t/m^2$ | Ts2 = 4.38 $t/m^2$ |
| Ts1 <90 ....O.k. | Ts2 <90 ....O.k. |

**2-Check Of Punching**

Qp= Pt - qn(a+d)*(b+d) = 29.81 ton.

Ap= 2((a+d)+(b+d))d = 2.09 $m^2$

Tp = Qp / Ap = 14.26 $t/m^2$

Tp<90 .....O.K.

## 3-Check Of Bond

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Tb1 = | | Qbh1 /(Eo*d) | | Tb2 = | | Qbh2 /(Eo*d) | |
| | Qb1= | $q_n$*A1*(B1-b)/2 | | | Qb2= | $q_n$*B1*(A1-a)/2 | |
| | Qb1= | 15.40 | ton | | Qb2= | 15.40 | ton |
| | Tb1 = | 4.95 | Kg/Cm$^2$ | | Tb2 = | 4.95 | Kg/Cm$^2$ |
| Tb1 <12 | | ....O.k. | | Tb2 <12 | | ....O.k. | |

## Output Data

| | | | | | |
|---|---|---|---|---|---|
| a= | 0.40 | m. | b= | 0.40 | m. |
| A1= | 2.00 | m. | B1= | 2.00 | m. |
| A= | 2.80 | m. | B= | 2.80 | m. |
| Thickness Of R.C = | 0.60 | m. | | | |
| Thickness Of Plain Concrete = | 0.40 | m. | | | |
| Reinforcement Of Dir. (B) Short = | 15 | o | 12 | | |
| Reinforcement Of Dir. (A) Long = | 15 | o | 12 | | |
| Volume Of R.C. = | 2.40 | m$^3$ | | | |

## A-10-3. Design of stop log house

## 1. REFERENCE CODES:

The design process depends mainly in the following Codes

1) The Egyptian code for design and construction of concrete structures, Code No. (203), Ministerial decree No. 44 – 2007, Ministry of housing, utilities and urban communities, 2007.
2) The Egyptian code for loads and forces structures, Code No. (203), Ministerial decree No. 44 – 2007, Ministry of housing, utilities and urban communities, 2007.

## 2. MATERIALS:

**Concrete**

- Concrete class is used as the following table

| Portion to be used | Cube Design Strength $N/mm^2$ | Unit Weight $kN/m^3$ |
|---|---|---|
| Reinforced Concrete | 25 | 25 |
| Plain Concrete | 20 | 23 |
| Leveling Concrete | 20 | 23 |

**[Clause 2-3-1-1/203]**

- Modulus of Elasticity $E_c$

$$E_C = 4400\sqrt{F_{cu}} \quad N/mm^2 = 4400\sqrt{25} = 22000 \quad N/mm^2 = 2.2x10^5 \quad kg/cm^2$$

**[Eq. 2.1/203]**

- For the working stress design.

Allowable compressive strength for bending = 9.50 $N/mm^2$ (95 $kg/cm^2$).
Allowable shearing strength by concrete only (beams)
= 0.70 $N/mm^2$ (7 $kg/cm^2$).
Allowable shearing strength by concrete only (slabs & footings)
= 0.90 $N/mm^2$ (7 $kg/cm^2$).
Allowable shearing strength by concrete and diagonal tension bars =
1.90 $N/mm^2$ (19 $kg/cm^2$).
**[Table. 5-1/203]**

- For limit state design

$\gamma_c = 1.50$ **[Eq. 3-15-a/203]**

**Reinforcing Steel**

- Steel type 360/520 is used for all diameters of reinforcing bars (main, secondary reinforcing, side bars and stirrups) in the bridge under study.
- Mass Density = 7.85 $ton/m^3$
- Modulus of Elasticity $E_s$

$E_s = 2.0 x 10^6 \quad kg/cm^2$ **[Fig. 4-1/203]**

- For the working stress design.

Allowable stress for tension and compression = 200 $N/mm^2$ (2000 $kg/cm^2$).
**[Table. 5-1/203]**

For limit state design

Yield stress for tension and compression = 360 $N/mm^2$ (3600 $kg/cm^2$)
$\gamma_s = 1.15$ **[Eq. 3-15-b/203]**

### 3. LIVE LOADS: (according to the Egyptian code)

Vertical loads are represented by uniformly distributed loads (UDL system):

[Table 4-1 /201]

a) UDL = 300 kg/m2 for office rooms.
b) UDL = 400 kg/m2 for stairs.
c) UDL = 400 kg/m2 for balconies.

### 4. DEAD LOADS

**Slab Loads**

- Assume the weight of flooring layer = 200 kg / $m^2$.

- If the thickness of R. C. slab <u>( $t_s$ ) = 12 cm</u>
  Own weight of slab = 0.12x2500 = 300 kg / $m^2$

The total dead load ($w_{D.L.}$) = 300+200 = <u>500 kg / $m^2$</u>

- If the thickness of R. C. slab <u>( $t_s$ ) = 14 cm</u>
  Own weight of slab = 0.14x2500 = 350 kg / $m^2$

The total dead load ($w_{D.L.}$) = 300+200 = <u>550 kg / $m^2$</u>

- If the thickness of R. C. slab <u>( $t_s$ ) = 15 cm</u>
  Own weight of slab = 0.15x2500 = 375 kg / $m^2$

The total dead load ($w_{D.L.}$) = 300+200 = <u>575 kg / $m^2$</u>

- If the thickness of R. C. slab <u>( $t_s$ ) = 16 cm</u>
  Own weight of slab = 0.16x2500 = 400 kg / $m^2$

The total dead load ($w_{D.L.}$) = 300+200 = <u>600 kg / $m^2$</u>

**<u>Own weight of beams</u>**

- For beam section <u>25x60</u>
  Own weight = 0.25*(0.60-0.12)*2500 = <u>300 kg/m\</u>

- For beam section <u>30x80</u>
  Own weight = 0.30*(0.80-0.12)*2500 = <u>510 kg/m\</u>

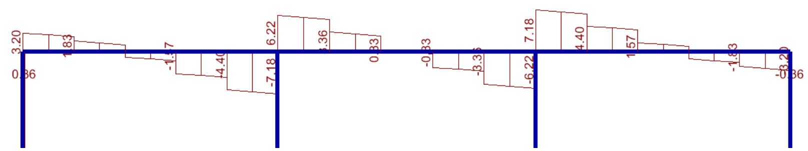
**<u>Own weight of walls</u>**

- For <u>walls 12 cm</u> thickness
  Weight of wall = <u>220 kg/$m^2$</u>

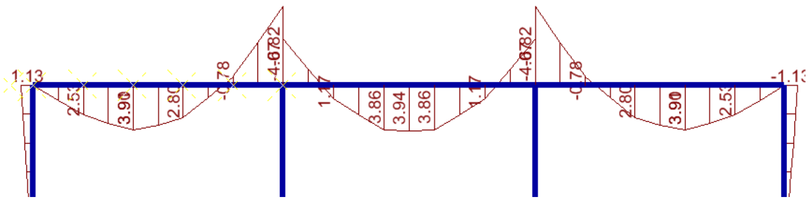
- For <u>walls 25 cm</u> thickness
  Weight of wall = <u>375 kg/$m^2$</u>

## 5. STRAINING ACTIONS ON BEAMS

Beam at axis 1-1 & Beam at axis 4-4

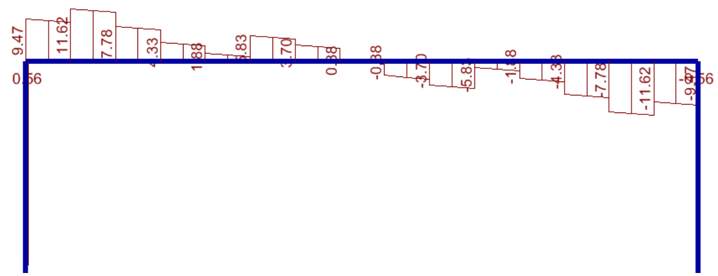


Shear force diagram

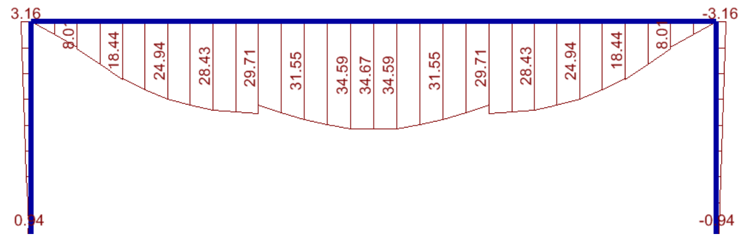


Bending moment diagram

Beam at axis 2-2 & Beam at axis 3-3

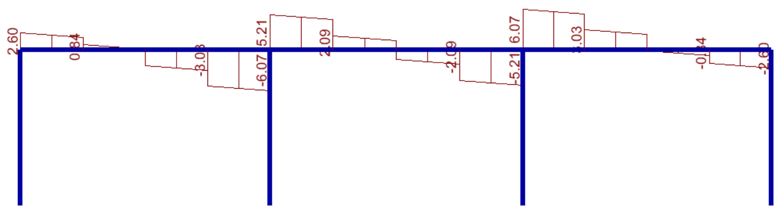


Shear force diagram

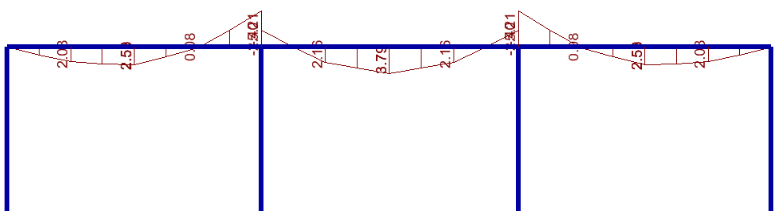



Bending moment diagram

Beam at axis A-A & Beam at axis D-D

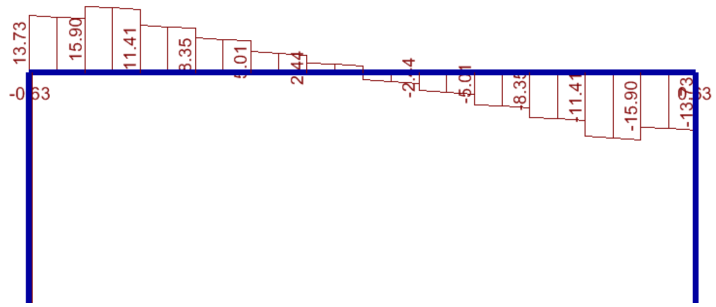



Shear force diagram

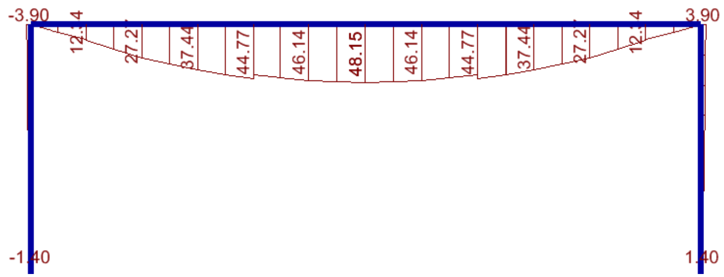



Bending moment diagram

Beam at axis B-B & Beam at axis C-C




Shear force diagram




Bending moment diagram

## 6. DESIGN OF SECTIONS

**Section (1):**

$M_{max}$ positive = 34.70 m.t. , T- sec.

$F_{cu}$= 250 kg/cm$^2$ -------------- $F_c$ = 95 kg/cm$^2$

Steel type 360/520 -------------- $F_s$ = 2000 kg/cm$^2$

The effective breadth of the section ($b_e$) equal to the least of the following:

- $b_e = 16\,t_s + b_w = 16x12 + 30 = 222$ cm
- $b_e$ = from center to center of girders = 370 cm
- $b_e = L_1/5 + b_w = (1300)/5 + 30 = 290$ cm

$b_e = 222$ cm [Clause 5-3-3-6-1 /203]

$M_{design} = 34.70\ t.m.$

$d = t - cover = 80 - 5 = 75\,cm$

$$d = k\sqrt{\frac{M}{b_e}}$$

$$d = 75 = k_1\sqrt{\frac{34.70*10^5}{222}} \quad \ldots\ldots\ldots\ldots \quad k_1 = 0.600$$

$$\Rightarrow \quad F_C = 35\,kg/cm^2 \prec F_{Cal.owable} = (2/3)x95 = 63\,kg/cm^2$$

$$\Rightarrow \quad k_2 = 1862$$

[Clause 5-3-2-6 /203]

$$A_s = \frac{M}{k_2\,d} = \frac{34.70*10^5}{1862*75} = 24.85\,cm^2$$

$use \quad 7\,\phi\,22\,mm$

$A_{s\,actual} = 26.60\,cm^2$

$$\mu_{actual} = \frac{A_s}{b\,d} = \frac{26.60}{30*75}x100 = 1.182\%$$

$$\mu_{min} = 0.225\frac{\sqrt{F_{cu}}}{f_y} = 0.225\frac{\sqrt{25}}{360}x100 = 0.3125\%$$

$$\geq \frac{1.1}{f_y} = \frac{1.1}{360}x100 = 0.306\%$$

$$\geq 0.15\% \text{ for high tensile steel}$$

$\mu_{min} = 0.3125\%$

$\mu_{max} = 5x10^{-4}xF_{cu} = 5x10^{-4}x25x100 = 1.25\%$

$\mu_{max} = 1.25\% \succ \mu_{actual} = 1.182\% \succ \mu_{min} = 0.3125\%$

**Section (2):**

$M_{max}$ positive = 48.20 m.t. , T- sec.

$F_{cu}$= 250 kg/cm$^2$ -------------- $F_c$ = 95 kg/cm$^2$

Steel type 360/520 -------------- $F_s$ = 2000 kg/cm$^2$

The effective breadth of the section ($b_e$) equal to the least of the following:

- $b_e = 16\,t_s + b_w = 16x12 - 30 = 222$ cm
- $b_e$ = from center to center of girders = 440 cm
- $b_e = L_1/5 + b_w = (1100)/5 + 30 = 250$ cm

$b_e = 222$ cm [Clause 5-3-3-6-1 /203]

$M_{design} = 48.20\ t.m.$

$$d = t - \text{cover} = 80 - 5 = 75\,cm$$

$$d = k_1 \sqrt{\frac{M}{b_e}}$$

$$d = 75 = k_1 \sqrt{\frac{48.20 * 10^5}{222}} \;\;\ldots\ldots\ldots\ldots\ldots \;\; k_1 = 0.509$$

$$\Rightarrow \quad F_C = 38\,\text{kg/cm}^2 \prec F_{C\,allowable} = (2/3)x95 = 63\,\text{kg/cm}^2$$

$$\Rightarrow \quad k_2 = 1850$$

**[Clause 5-3-2-6 /203]**

$$A_s = \frac{M}{k_2\, d} = \frac{48.20 * 10^5}{1850 * 75} = 34.74\,cm^2$$

$use \quad 10\,\phi\,22\;mm$

$$A_{s\,actual} = 38.00\,cm^2$$

$$\mu_{actual} = \frac{A_s}{b\,d} = \frac{38.00}{30 * 75} x100 = 1.689\%$$

$$\mu_{min} = 0.225 \frac{\sqrt{F_{cu}}}{f_y} = 0.225 \frac{\sqrt{25}}{360} x100 = 0.3125\%$$

$$\geq \frac{1.1}{f_y} = \frac{1.1}{360} x100 = 0.306\%$$

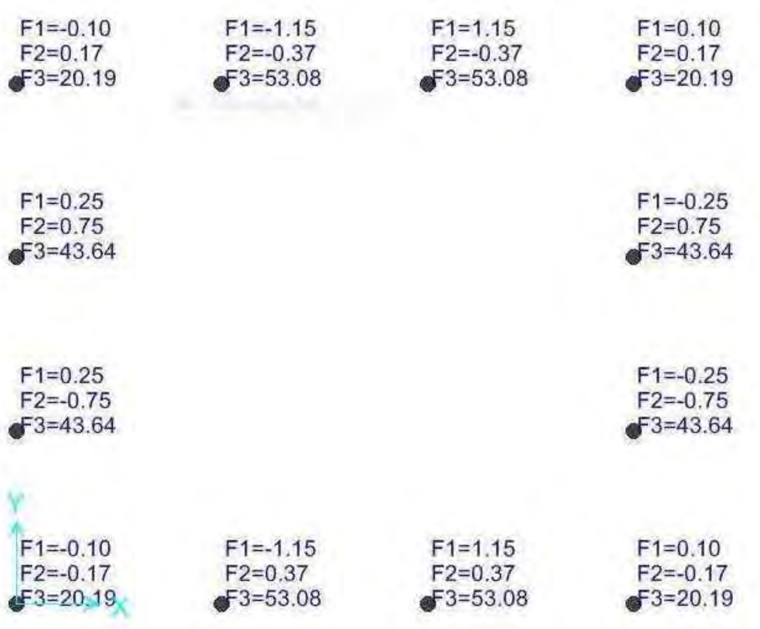
$\geq 0.15\%$ for high tensile steel

$$\mu_{min} = 0.3125\%$$

$$\mu_{max} = 5x10^{-4}\, xF_{cu} = 5x10^{-4}\, x25x100 = 1.25\%$$

$\mu_{max} = 1.25\% \prec \mu_{actual} = 1.182\%$ use compression steel

## 7. Loads on columns




Loads on columns

## DESIGN OF COLUMNS

**$P = f_{c0} * A_c + 0.44 f_y * A_s$**

C 250 $f_{c0}$ = 60 Kg /cm2

st 52 $f_y$ = 3600 Kg /cm2

Assume As = 1 % Ac

P = 60 Ac + 0.44 * 3600 * 0.010 Ac

P = 60 Ac + 15.84 Ac

P = 75.84 Ac

take | P = | 75.84 Ac |

| Ac | | | Pw | |
|---|---|---|---|---|
| 40 | x | 40 | 121 | ton |
| 40 | x | 50 | 152 | ton |

## 8. DESIGN OF FOUNDATIONS

**Design Of Isolated Footings**

**Subject to (Axial Load) By Working Method**

**(Rectangular Footings)/( Rectangular Stress Distribution )**

**[F2]**

| | | | |
|---|---|---|---|
| Bearing Capacity for Soil | $q_{soil}$ = | 6 | t/m$^2$ |
| Tensile Steel Stress | Fs= | 2000 | Kg/Cm$^2$ |
| Compressive Concrete Stress | Fc= | 95 | Kg/Cm$^2$ |
| Allowed Punching Stress | T p= | 90 | t/m$^2$ |
| Allowed Shear Stress | T s= | 90 | t/m$^2$ |
| Allowed Bond Stress | T b= | 12 | Kg/Cm$^2$ |

**Input Data**

| | | | |
|---|---|---|---|
| **Column Working Load** | $P_w$ = | **25** | **ton** |
| Column Dimension **(a > b)** | a = | 0.40 | m. |
| | b = | 0.40 | m. |
| Plain Concrete Depth | t = | 0.40 | m. |
| Plain Concrete Extension. | x = | 0.40 | m. |

Pt = 1.1 x P = 33.00 ton

Pt / (A x B)= 5.50 A x B= 5.50 ...(1)

(A-a) / 2 = (B-b) / 2 A= B+(a-b) ...(2)

$B^2$ + 0.00 x B = 5.50 m.

| | | | | | |
|---|---|---|---|---|---|
| A= | 2.50 | m | B= | 2.50 | m |
| A1= | 1.70 | m | B1= | 1.70 | m |

**Check**

A x B = 6.25 m$^2$ .........>= 5.50 m$^2$ ...O.k.

$q_r$ = Pt/(A1*B1) = 11.42 t/m$^2$ **qn = (30-40)t/m$^2$**

**Dimension of R.C.**

| | | | |
|---|---|---|---|
| $K_1$ = | 0.346 | $K_2$ = | 1794 |

| | |
|---|---|
| $M_{1-1}$=qn * B1*(A1-a)$^2$/8 | $M_{2-2}$=$q_n$ * A1*(B1-b)$^2$/8 |
| $M_{1-1}$ = 4.10 m.t. | $M_{2-2}$ = 4.10 m.t. |
| d =$K_1$ (M1/B1)^0.5 | d =$K_1$(M2/A1)^0.5 |
| d= 17.0 Cm. | d= 17.0 Cm. |

| | | | | | |
|---|---|---|---|---|---|
| Let d = | 55 | Cm. | .... t1 = | 60 | Cm. |
| $K_1$ (New)= | 1.120 | | ......$K_2$ = | 1930 | |

| | |
|---|---|
| As1= $M_{1-1}$/($K_2$*d) | As2= $M_{2-2}$/($K_2$*d) |
| As1= 3.86 $Cm^2$ | As2= 3.86 $Cm^2$ |

As min = µ %Ac= µ %(B1*df)= 14.03 $Cm^2$

| | | | | | |
|---|---|---|---|---|---|
| As = | 14 | ϕ 12 | As= | 14 | ϕ 12 |

**Choose**

| | | | | |
|---|---|---|---|---|
| As = | 14 | ϕ | 12 | in **Short** Direction(B) |
| As = | 14 | ϕ | 12 | in **Long** Direction(A) |

**1-Check Of Shear**

| | |
|---|---|
| Ts1 = Qsh1 /(B1*d) | Ts2 = Qsh2 /(A1*d) |
| Qsh1= qn*B1*((A1-a)/2-d) | Qsh2= qn*A1*((B1-b)/2-d) |
| Qsh1= 1.94 ton | Qsh2= 1.94 ton |
| Ts1 = 2.08 t/$m^2$ | Ts2 = 2.08 t/$m^2$ |
| Ts1 <90 ....O.k. | Ts2 <90 ....O.k. |

**2-Check Of Punching**

| | | | | |
|---|---|---|---|---|
| Qp= | Pt - qn(a+d)*(b+d) | = | 22.69 | ton. |
| Ap= | 2((a+d)+(b+d))d | = | 2.09 | $m^2$ |
| Tp | = Qp / Ap | = | 10.86 | t/$m^2$ |
| | | | Tp<90 | .....O.K. |

**3-Check Of Bond**

| | |
|---|---|
| Tb1 = Qbh1 /(Eo*d) | Tb2 = Qbh2 /(Eo*d) |
| Qb1= $q_n$*A1*(B1-b)/2 | Qb2= $q_n$*B1*(A1-a)/2 |
| Qb1= 12.62 ton | Qb2= 12.62 ton |
| Tb1 = 4.35 Kg/$Cm^2$ | Tb2 = 4.35 Kg/$Cm^2$ |
| Tb1 <12 ....O.k. | Tb2 <12 ....O.k. |

**Output Data**

| | | | | | |
|---|---|---|---|---|---|
| a= | 0.40 | m. | b= | 0.40 | m. |
| A1= | 1.70 | m. | B1= | 1.70 | m. |
| A= | 2.50 | m. | B= | 2.50 | m. |

| | | | | |
|---|---|---|---|---|
| Thickness Of R.C | = | 0.60 | m. | |
| Thickness Of Plain Concrete | = | 0.40 | m. | |
| Reinforcement Of Dir. (B) Short | = | 14 | ϕ | 12 |
| Reinforcement Of Dir. (A) Long | = | 14 | ϕ | 12 |
| Volume Of R.C. | = | 1.73 | $m^3$ | |

# Design Of Isolated Footings

## Subject to (Axial Load) By Working Method

### (Rectangular Footings)/( Rectangular Stress Distribution )

**[F3]**

| | | | |
|---|---|---|---|
| Bearing Capacity for Soil | $q_{soil}$ = | 6 | t/m$^2$ |
| Tensile Steel Stress | Fs= | 2000 | Kg/Cm$^2$ |
| Compressive Concrete Stress | Fc= | 95 | Kg/Cm$^2$ |
| Allowed Punching Stress | T p= | 90 | t/m$^2$ |
| Allowed Shear Stress | T s= | 90 | t/m$^2$ |
| Allowed Bond Stress | T b= | 12 | Kg/Cm$^2$ |

**Input Data**

| | | | |
|---|---|---|---|
| **Column Working Load** | **$P_w$ =** | **55** | **ton** |
| Column Dimension (a > b) | a = | 0.50 | m. |
| | b = | 0.40 | m. |
| Plain Concrete Depth | t = | 0.40 | m. |
| Plain Concrete Extension. | x = | 0.40 | m. |

Pt = 1.1 x P = 60.50 ton

Pt / (A x B)= 10.08 A x B= 10.08 ...(1)

(A-a) / 2 = (B-b) / 2 A= B+(a-b) ...(2)

$B^2$ + 0.10 x B = 10.08 m.

| | | | | | |
|---|---|---|---|---|---|
| A= | 3.30 | m | B= | 3.20 | m |
| A1= | 2.50 | m | B1= | 2.40 | m |

**Check**

A x B = 10.56 m$^2$ .........>= 10.08 m$^2$ ...O.k.

$q_n$ = Pt/(A1*B1) = 10.08 t/m$^2$ **qn = (30-40)t/m$^2$**

**Dimension of R.C.**

$K_1$ = 0.346 $K_2$ = 1794

| | |
|---|---|
| $M_{1-1}$=qn * B1*(A1-a)$^2$/8 | $M_{2-2}$=$q_n$ * A1*(B1-b)$^2$/8 |
| $M_{1-1}$ = 12.10 m.t. | $M_{2-2}$ = 12.60 m.t. |
| d =$K_1$ (M1/B1)^0.5 | d =$K_1$(M2/A1)^0.5 |
| d= 24.6 Cm. | d= 24.6 Cm. |

Let d = 55 Cm. .... t1 = 60 Cm.

$K_1$ (New)= 0.775 ......$K_2$ = 1901

| | |
|---|---|
| As1= $M_{1-1}$/($K_2$*d) | As2= $M_{2-2}$/($K_2$*d) |
| As1= 11.57 Cm$^2$ | As2= 12.06 Cm$^2$ |

As min = μ %Ac= μ %(B1*df)= 20.63 Cm$^2$

| | |
|---|---|
| As = 19 ϕ 12 | As= 19 ϕ 12 |

**Choose**

| | | | | |
|---|---|---|---|---|
| As = | 19 | ϕ | 12 | in **Short** Direction(B) |
| As = | 19 | ϕ | 12 | in **Long** Direction(A) |

**1-Check Of Shear**

| | |
|---|---|
| Ts1 = Qsh1 /(B1*d) | Ts2 = Qsh2 /(A1*d) |
| Qsh1= qn*B1*((A1-a)/2-d) | Qsh2= qn*A1*((B1-b)/2-d) |
| Qsh1= 10.89 ton | Qsh2= 11.34 ton |
| Ts1 = 8.25 t/m$^2$ | Ts2 = 8.25 t/m$^2$ |
| Ts1 <90 ....O.k. | Ts2 <90 ....O.k. |

**2-Check Of Punching**

Qp= Pt - qn(a+d)*(b+d) = 50.44 ton.

Ap= 2((a+d)+(b+d))d = 2.20 m$^2$

Tp = Qp / Ap = 22.93 t/m$^2$

Tp<90 .....O.K.

**3-Check Of Bond**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Tb1 = | | Qbh1 /(Eo*d) | | Tb2 = | | Qbh2 /(Eo*d) | |
| | Qb1= | $q_n$*A1*(B1-b)/2 | | | Qb2= | $q_n$*B1*(A1-a)/2 | |
| | Qb1= | 25.21 | ton | | Qb2= | 24.20 | ton |
| | Tb1 = | 6.40 | $Kg/Cm^2$ | | Tb2 = | 6.15 | $Kg/Cm^2$ |
| Tb1 <12 | | ....O.k. | | Tb2 <12 | | ....O.k. | |

**Output Data**

| | | | | | |
|---|---|---|---|---|---|
| a= | 0.50 | m. | b= | 0.40 | m. |
| A1= | 2.50 | m. | B1= | 2.40 | m. |
| A= | 3.30 | m. | B= | 3.20 | m. |

| | | | | |
|---|---|---|---|---|
| Thickness Of R.C | = | 0.60 | m. | |
| Thickness Of Plain Concrete | = | 0.40 | m. | |
| Reinforcement Of Dir. (B) Short | = | 19 | ø | 12 |
| Reinforcement Of Dir. (A) Long | = | 19 | ø | 12 |
| Volume Of R.C. | = | 3.60 | $m^3$ | |