EXERCISE for DESIGN OF CAST-IN PLACE RC PILE

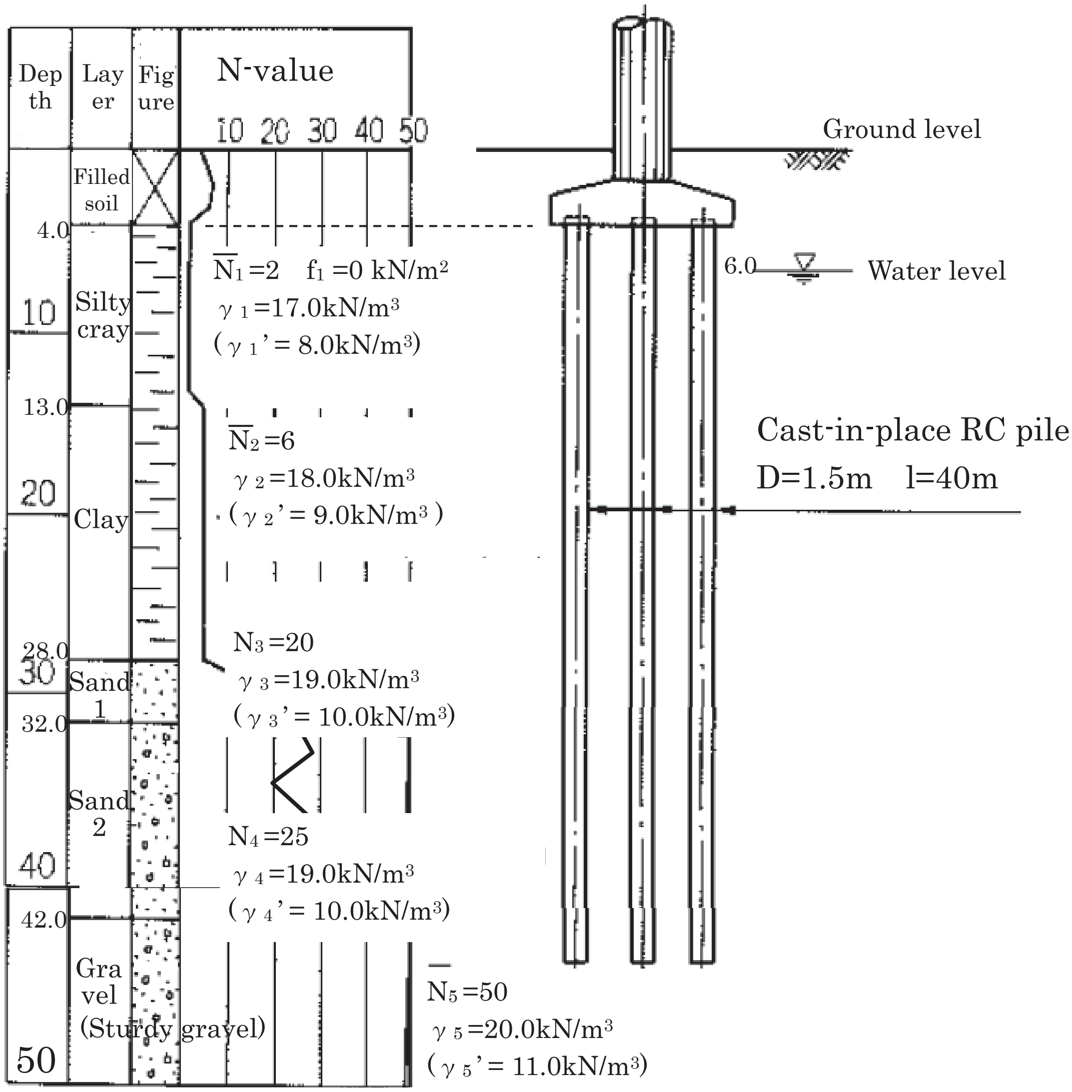
DESIGN CONDITION

(1) Outline of Foundation

Cast-in-place RC pile

- Pile diameter : D =1.5m
- Pile length: L =40.0m
- Unit weight of RC concrete : 24.5 kN/m^3

(2) Geological Condition



Note 1)
Shaft resistance intensity f_1 for silty clay layer is evaluated as 0 kN/m^2.
Note 2)
Liquefaction are not estimated in the ground.

QUESTION. 1

Calculate following pile capacities by use of empirical bearing capacity estimation formula.

Each shall be obtained for Ordinary case and Seismic case, respectively.

- Allowable bearing capacity of pile
- Allowable pull-out capacity of pile

・End bearing capacity intensity (Cast-in-placed RC pile)

Ground Type	Ultimate Bearing Capacity End Bearing Intensity (kN/m^2)
Gravelly Layer and Sandy Layer ($N \geq 30$)	3,000
Sturdy Gravelly Layer ($N \geq 50$)	5,000
Hard Cohesive Soil Layer	$3q_u$

Notes) q_u : unconfined compressive strength (kN/m^2),
N : N value from the Standard Penetration Test (SPT)

・Maximum shaft resistance intensity

Pile Installation Method \ Ground Type	Sandy Soil	Cohesive Soil
Driven Pile Method (including Vibro-hammer Method)	$2N (\leq 100)$	c or $10N (\leq 150)$
Cast-in-place RC Pile Method	$5N (\leq 200)$	c or $10N (\leq 150)$
Bored Pile Method	$2N (\leq 100)$	$0.8c$ or $8N (\leq 100)$
Pre bored Pile Method	$5N (\leq 150)$	c or $10N (\leq 100)$
Steel Pipe Soil Cement Pile Method	$10N (\leq 200)$	c or $10N (\leq 200)$

(Note) c: cohesion of ground (kN/m^2), N: N value from SPT.

ANSWER. 1

(1) Allowable bearing capacity

$$R_a = \frac{\gamma}{n}(R_u - W_s) + W_s - W$$

γ : 1.0

n : for ordinary case : 3, for seisimic case : 2

Ru = $q_d A + U \cdot \Sigma l_i f_i$

qd = 5,000 kN/m2

A = $\pi \times 0.75 \times 0.75$ =1.766 m2

U = $\pi \times D = \pi \times 1.5$ =4.71 m

i	Depth (m)	Thickness l_i (m)	Soil type	Averaged N-value	Shaft resistance intensity f_i (kN/m^2)	$l_i \cdot f_i$ (kN/m)	Unit weight γ_i (kN/m^3)	Effective weight γ_i' (kN/m^3)
1	4.0-6.0	2.0	Silty clay	2	0	0	17.0	17.0
2	6.0-13.0	7.0	Silty clay	2	0	0	17.0	8.0
3	13.0-28.0	15.0	Clay	6	60	900	18.0	9.0
4	28.0-32.0	4.0	Sand1	20	100	400	19.0	10.0
5	32.0-42.0	10.0	Sand2	25	125	1250	19.0	10.0
6	42.0-42.5	0.5	Gravel	50	200	100	20.0	11.0
6’	42.5-44.0	1.5	Gravel	50	200	(300)	20.0	11.0
Total		40.0	-	-	-	2,650 (2,950)	-	-

Note: ( ) is applied for allowable pull-out force.

Ru = 5,000 $\times$ 1.766 + 4.71 $\times$ 2,650 = 21,311.5 kN

Ws : $\pi \times 0.75 \times 0.75 \times (2.0 \times 17.0 + 7.0 \times 8.0 + 15.0 \times 9.0 + 4.0 \times 10.0$

+10.0×10.0 +0.5×11.0 +1.5×11.0) =683.5kN

W : π ×0.75×0.75×{38×(24.5－10.0) +2×24.5} = 1059.8kN

i) For ordinary case

Ra = 1/3×(21,311.5 – 683.5) + 683.5 – 1059.8 = <u>6,500 kN/pile</u>

ii) For seismic case

Ra = 1/2×(21,311.5 – 683.5) + 683.5 – 1059.8 = <u>9,938 kN /pile</u>

(2) Allowable pull-out force

$$P_a = \frac{1}{n} \cdot P_u + W$$

n : for ordinary case : 6, for seisimic case : 3

Pu =4.71×(2,950) = 13,894.5 kN

W : π ×0.75×0.75×{38×(24.5－10.0) +2×24.5} = 1059.8kN

i) For ordinary case

Pa = 1/6×13,894.5 + 1059.8 = <u>3,376 kN/pile</u>

ii) For seismic case

Pa = 1/3×13,894.5 + 1059.8 = <u>5,691 kN/pile</u>

QUESTION. 2

(1) Calculate a coefficient of horizontal subgrade reaction

(2) Determine the value of β. In addition, confirm the pile length within semi-infinite length of pile ($\beta \cdot L \geqq 3$)

(3) Calculate axial spring constant of pile

(4) Calculate radial spring constants of pile

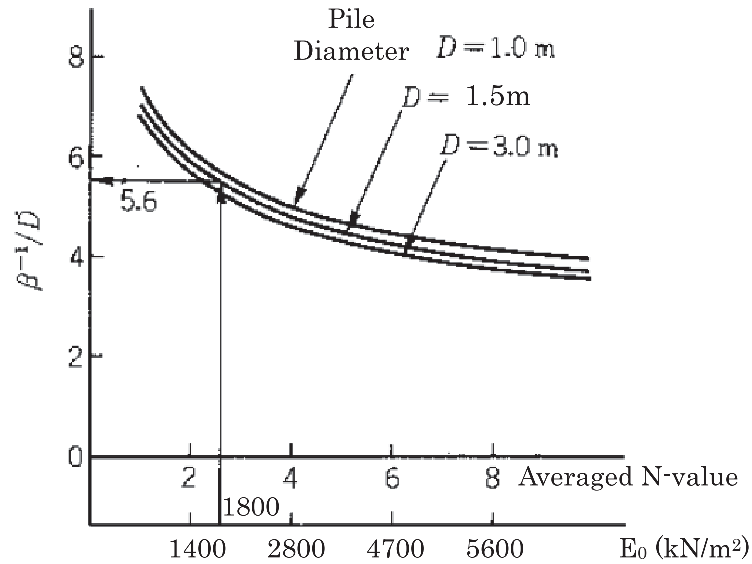
(5) Calculate stiffness matrix of pile for bridge longitudinal direction by use of displacement method

Each shall be calculated for both of Ordinary case and Seismic case.

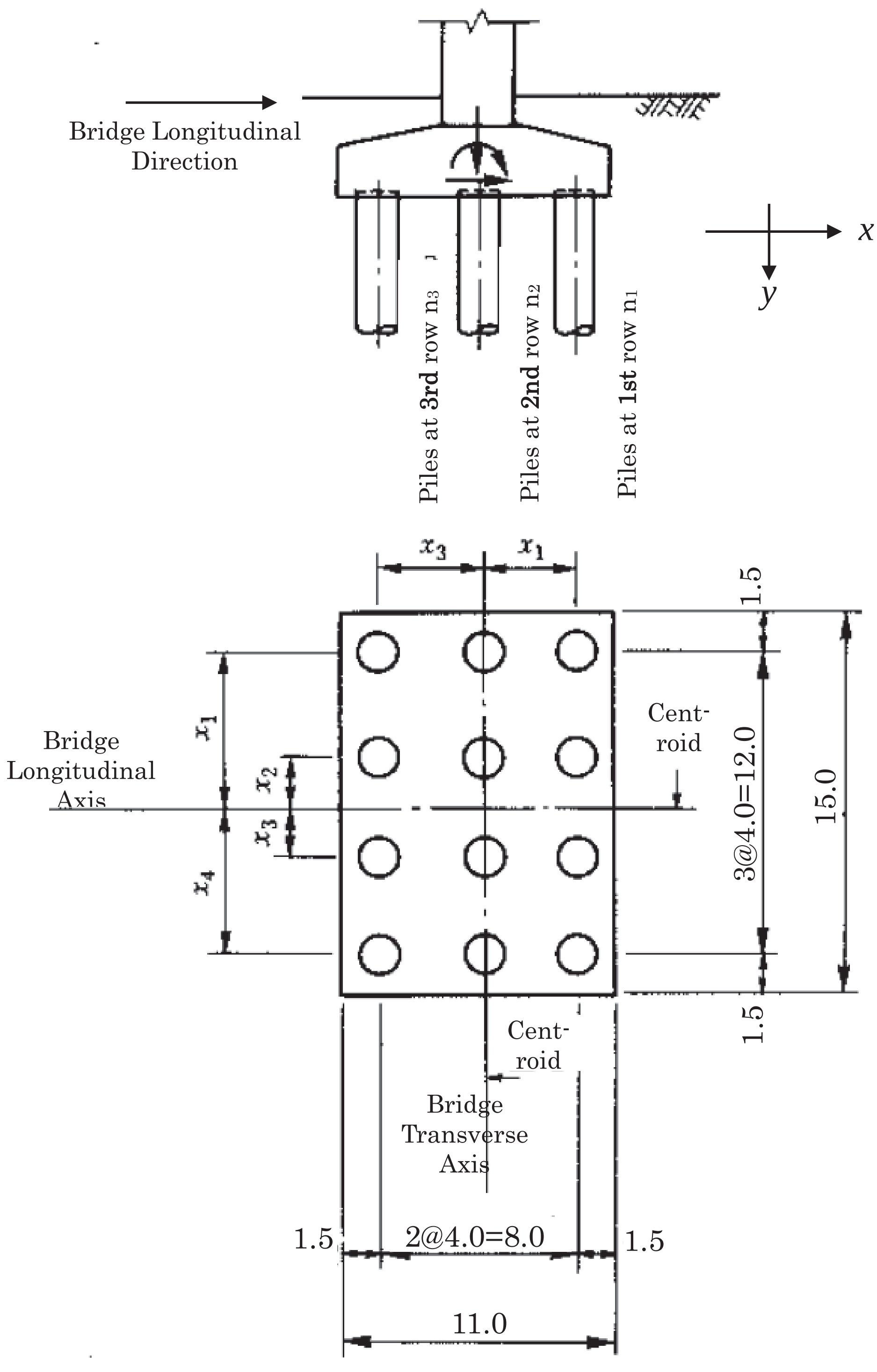
Supplemental Design Condition

・Modulus of deformation of ground (E_0) for silty clay layer was evaluated by unconfined compression test result ; E_0 = 1,800 kN /m^2

・Use the Figure below for tentative value of β^{-1}/ D in order to obtain the coefficient of horizontal subgrade reaction for ordinary case.



・Pile arrangement



・Displacement at the origin point can be obtained by following equations ;

$$A_{xx} \bullet \delta_x + A_{xy} \bullet \delta_y + A_{x\alpha} \bullet \alpha = H_0$$
$$A_{yx} \bullet \delta_x + A_{yy} \bullet \delta_y + A_{y\alpha} \bullet \alpha = V_0$$
$$A_{\alpha x} \bullet \delta_x + A_{\alpha y} \bullet \delta_y + A_{\alpha\alpha} \bullet \alpha = M_0$$

・Each coefficient can be obtained by following equations ;

$$A_{xx} = \sum\left(K_1 \bullet \cos^2\theta_i + K_V \bullet \sin^2\theta_i\right)$$
$$A_{xy} = A_{yx} = \sum\left(K_V - K_1\right) \bullet \sin\theta_i \bullet \cos\theta_i$$
$$A_{x\alpha} = A_{\alpha x} = \sum\left\{\left(K_V - K_1\right)x_i \bullet \sin\theta_i \bullet \cos\theta_i - K_2 \bullet \cos\theta_i\right\}$$
$$A_{yy} = \sum\left(K_V \bullet \cos^2\theta_i + K_1 \bullet \sin^2\theta_i\right)$$
$$A_{y\alpha} = A_{\alpha y} = \sum\left\{\left(K_V \bullet \cos^2\theta_i + K_1 \bullet \sin^2\theta_i\right)x_i + K_2 \bullet \sin\theta_i\right\}$$
$$A_{\alpha\alpha} = \sum\left\{\left(K_V \bullet \cos^2\theta_i + K_1 \bullet \sin^2\theta_i\right)x_i^2 + \left(K_2 + K_3\right)x_i \bullet \sin\theta_i + K_4\right\}$$

where

H_0: lateral loads acting above a footing bottom (kN)

V_0: vertical loads acting above a footing bottom (kN)

M_0: moment of external forces around the origin O (kN·m)

δ_x: lateral displacement at the origin O (m)

δ_y: vertical displacement at the origin O (m)

α: rotational angle of the footing (rad)

x_i: x coordinate of the i-th pile head (m)

θ_i: angle of a vertical axis from the i-th pile axis (degree). Signs to be in accordance with Fig.-C. 12.7.2.

ANSWER. 2

(1) Coefficient of horizontal subgrade reaction

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

$B_H = \sqrt{(D/\beta)} = \sqrt{(\beta^{-1}/D \cdot D2)} = \sqrt{(\beta^{-1}/D)} \cdot D = \sqrt{5.6} \times 1.5 = 3.55\mathrm{m}$

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

For ordinary ; $K_H = 1/0.3 \times 4 \times 1{,}800 \times (3.55/0.3)^{-3/4} \fallingdotseq 3{,}768$ kN/m^3

For seismic ; $K_H = 2 \times 3{,}768 \fallingdotseq 7{,}536$ kN/m^3

(2) Determination of β and confirmation of the pile length

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

D = 1.5m

E = 2.5 x 10^7 (kN/m^2)

I = $\pi \cdot D^4/64 = \pi/64 \times 1.5^4 = 0.2484$ m^4

For ordinary ; $\beta = 0.12282$ m^{-1}

For seismic ; $\beta = 0.14606$ m^{-1}

$\beta \cdot L = 0.12282 \times 40 = 4.91 \geqq 3.0$

Therefore, the pile can be designed as semi-infinite length of pile.

(3) Axial spring constant of pile

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

a =0.031× L/D－0.15

=0.031× (40.0/1.5) －0.15 =0.677

Ap =1/4 × π × D^2 =1/4× π ×1.5^2 =1.766m^2

Kv=0.677× (1.766×2.5×10^7) /40 =747,239 kN/m

(4) Radial spring constant of pile

a) Ordinary Case

K_1 =4EI β^3 = 4×2.5×10^7×0.2484×0.12282^3 =46,021 kN/m
K_2 (=K_3) =2EI β^2 = 2×2.5×10^7×0.2484×0.12282^2 =187,353 kN/rad
K_4 =2EI β = 2×2.5×10^7×0.2484×0.12282 =1,525,424 kN・m/rad

b) Seismic Case
K_1 =4EI β^3 = 4×2.5×10^7×0.2484×0.14606^3 =77,401 kN /m
K_2 (=K_3) =2EI β^2 = 2×2.5×10^7×0.2484×0.14606^2 =264,962 kN /rad
K_4 =2EI β = 2×2.5×10^7×0.2484×0.14606 =1,814,065 kN・m/rad

(5) Stiffness matrix of pile (For Longitudinal direction)
a) Ordinary Case
$A_{xx} = \Sigma (K_1 \cos^2\theta_i + K_v \sin^2\theta_i)$ =(46,021×1.0+0)×12 =552,252 kN /m
$A_{xy} = A_{yx} = \Sigma (K_v - K_1) \sin\theta_i \cdot \cos\theta_i$ = 0 kN /m
$A_{xa} = A_{ax} = \Sigma \{(K_v - K_1) x_i \cdot \sin\theta_i \cdot \cos\theta_i - K_2 \cos\theta_i\}$
= (0－187,353)×12 =－2,248,236 kN
$A_{yy} = \Sigma (K_v \cos^2\theta_i + K_1 \sin^2\theta_i)$ =(747,239×1.0+0)×12 =8,966,868 kN /m
$A_{ya} = A_{ay} = \Sigma \{(K_v \cos^2\theta_i + K_1 \sin^2\theta_i) xi + K_2 \sin\theta_i\}$
={(747,239×1.0 + 0)×4.0}×4 +{(747,239×1.0 +0)×0.0}×4
+{(747,239×1.0 + 0)×(－4.0)}×4 = 0 kN
$A_{aa} = \Sigma \{(K_v \cos^2\theta_i + K_1 \sin^2\theta_i) x_i^2 + (K_2+K_3) x_i \cdot \sin\theta_i + K_4\}$
$= \Sigma \{K_v x_i^2 + K_4\}$
={(747,239×4.0^2+1,525,424}×4 +{(747,239×0^2 +1,525,424}×4
+{(747,239×$(-4.0)^2$ +1,525,424)×4 = 113,951,680 kN・m/rad

b) Seismic Case
A_{xx} = (77,401×1.0+0)×12 =928,812 kN /m
$A_{xy} = A_{yx}$ = 0 kN /m
$A_{xa} = A_{ax}$ = (0-264,962)×12 =－3,179,544 kN
A_{yy} = 747,239×12 =8,966,868 kN /m
$A_{ya} = A_{ay}$ = 0 kN
A_{aa} ={(747,239×4.0^2+1,814,065}×4 +{747,239×0^2 +1,814,065}×4
+{(747,239×$(-4.0)^2$ +1,814,065)×4
= 117,415,372 kN・m/rad

QUESTION. 3

External force at the center of footing bottom is summarized in table below.

With the condition, calculate followings for bridge longitudinal direction;

(1) Displacement at pile head

(2) Axial force, radial force, moment acting on the pile head

(3) Verify the structural stability of the piles

Load case (Ordinary /Seismic)	Vertical force V (kN)	Horizontal force H (kN)	Bending moment M (kN.m)
Ordinary	43,605	1,030	4,738
Seismic	38,005	11,309	39,259

Note: Above are external forces when bridge longitudinal direction is considered.

Supplemental Explanation

With the condition of all piles arranged in vertical and with rigid connection to the footing, the equations for displacement and each forces acting on the pile head can be simplified by equations below;

$$\delta_x = \frac{H_0 \cdot A_{\alpha\alpha} - M_0 \cdot A_{x\alpha}}{A_{xx} \cdot A_{\alpha\alpha} - A_{x\alpha} \cdot A_{\alpha x}}$$

$$\delta_y = \frac{V_0}{A_{yy}}$$

$$\alpha = \frac{-H_0 \cdot A_{\alpha x} + M_0 \cdot A_{xx}}{A_{xx} \cdot A_{\alpha\alpha} - A_{x\alpha} \cdot A_{\alpha x}}$$

$$P_{Ni} = K_V (\delta_y + \alpha x_i)$$

$$P_{Hi} = K_1 \cdot \delta_x - K_2 \alpha$$

$$M_{ti} = -K_3 \cdot \delta_x + K_4 \alpha$$

ANSWER. 3

(1) Displacement at pile head

a) Ordinary Case

$$\delta_x = \frac{H_0 \cdot A_{\alpha\alpha} - M_0 \cdot A_{x\alpha}}{A_{xx} \cdot A_{\alpha\alpha} - A_{x\alpha} \cdot A_{\alpha x}}$$

={1,030×113,951,680－4,738 (－2,248,236)}／{552,252×113,951,680 – (－2,248,236)・(－2,248,236)}

= 2.21 mm

$$\delta_y = \frac{V_0}{A_{yy}}$$

= 43,605／8,966,868 =4.86 mm

$$\alpha = \frac{-H_0 \cdot A_{\alpha x} + M_0 \cdot A_{xx}}{A_{xx} \cdot A_{\alpha\alpha} - A_{x\alpha} \cdot A_{\alpha x}}$$

={－1,030×(－2,248,236) +4,738×552,252}／{552,252×113,951,680 – (－2,248,236)・(－2,248,236)}

= 0.0000852 rad

b) Seismic Case

δ_x ={11,309 × 117,415,372 –39,259 (－ 3,179,544)} ／ {928,812 × 117,415,372 – (－3,179,544)・(－3,179,544)}

= 14.68 mm

δ_y = 38,005／8,966,868 =4.24 mm

α ={ － 11,309 × (－ 3,179,544) +39,259 × 928,812} ／ {928,812 × 117,415,372 – (－3,179,544) ・ (－3,179,544)}

= 0.0007319 rad

(2) Force and moment acting on the pile head

$$P_{Ni} = K_V(\delta_y + \alpha x_i)$$
$$P_{Hi} = K_1 \cdot \delta_x - K_2\alpha$$
$$M_{ti} = -K_3 \cdot \delta_x + K_4\alpha$$

a) Ordinary Case

・ 1st row (i =1)

P_{N1} = 747,239×(0.00486 + 0.0000852×4.000) = 3,886.2 kN /pile

P_{H1} = 46,021×0.00221 – 187,353×0.0000852 = 85.7 kN /pile

M_{t1} =－187,353×0.00221 + 1,525,424×0.0000852 =－284.1 kN・m /pile

・ 2nd row (i =2)

P_{N2} = 747,239×(0.00486 + 0.0000852×0) = 3,631.6 kN /pile

P_{H2} = 46,021×0.00221 – 187,353×0.0000852 = 85.7 kN /pile

M_{t2} =－187,353×0.00221 + 1,525,424×0.0000852 =－284.1 kN・m /pile

・ 3rd row (i =3)

P_{N3} = 747,239×(0.00486 - 0.0000852×4.000) =3,376.9 kN /pile

P_{H3} = 46,021×0.00221 – 187,353×0.0000852 = 85.7 kN /pile

M_{t3} =－187,353×0.00221 + 1,525,424×0.0000852 =－284.1 kN・m /pile

b) Seismic Case

・ 1st row (i =1)

P_{N1} = 747,239×(0.00424 + 0.0007319×4.000) = 5,355.9 kN /pile

P_{H1} = 77,401×0.01468 – 264,962×0.0007319 = 942.3 kN /pile

M_{t1} =－264,962×0.01468 + 1,814,065×0.0007319 =－2,561.9 kN・m /pile

・ 2nd row (i =2)

P_{N2} = 747,239×(0.00424 + 0.0007319×0) = 3,168.3 kN /pile

P_{H1} = 77,401×0.01468 – 264,962×0.0007319 = 942.3 kN /pile

M_{t1} =－264,962×0.01468 + 1,814,065×0.0007319 =－2,561.9 kN・m /pile

・ 3rd row (i =3)

$P_{N3} = 747{,}239 \times (0.00424 - 0.0007319 \times 4.000) = 980.7$ kN /pile

$P_{H1} = 77{,}401 \times 0.01468 - 264{,}962 \times 0.0007319 = 942.3$ kN /pile

$M_{t1} = -264{,}962 \times 0.01468 + 1{,}814{,}065 \times 0.0007319 = -2{,}561.9$ kN・m /pile

(3) Verification of structural stability

δ_{xmax} (Ordinary) $= 2.21$ mm $<$ $\delta_a' = 15.0$ mm

δ_{xmax} (Seismic) $= 14.68$ mm $<$ $\delta_a' = 15.0$ mm

P_{Nmax} (Ordinary) : 3,886.2kN $< R_a = 6{,}500$kN

P_{Nmin} (Ordinary) : 3,376.9kN $> P_a = 0$ kN

P_{Nmax} (Seismic) : 5,355.9kN $< R_a' = 9{,}938$kN

P_{Nmin} (Seismic) : 980.7kN $> P_a' = -5{,}691$ kN

QUESTION. 4

(1) With refer the calculation formula for a pile of semi-infinite length by Hayashi-Chang, express the following formulas in relation with depth x(m).

a) Horizontal displacement for pile head Rigid connection
b) Bending moment for pile head Rigid connection
c) Shear force for pile head Rigid connection
d) Bending moment for pile head Hinged connection

Above shall be calculated for Ordinary case and Seismic case in direction of the bridge longitudinal.

(2) Calculate Maximum bending moment at underground

i) Maximum bending moment at underground with Rigid pile head
ii) Maximum bending moment at underground with Hinged pile head

(3) Express the formula of above a) to d) by tables and graphs in relation of depth x(m). For the graphs, indicate the maximum forces and its depths.

ANSWER. 4

(1) Calculate sectional forces

A. For Ordinary Case

a) Horizontal displacement for pile head rigid

$$y=\frac{H}{2EI\beta^3}e^{-\beta x}\{(1+\beta h_0)\cos\beta x-\beta h_0\sin\beta x\}$$

$= 85.7 / (2\times2.5\times10^7\times0.2484\times0.12282^3)\times e^{-0.12282x}\times\{(1+0.12282\times(-284.1)/85.7)\times\cos(0.12282x) - 0.12282\times(-284.1)/85.7\times\sin(0.12282x)\}$

$=0.00372\exp(-0.12282x)\cdot\{0.5929\cos(0.12282x)+0.4071\sin(0.12282x)\}$

b) Bending moment for pile head rigid

$$M=-\frac{H}{\beta}e^{-\beta x}\{\beta h_0\cos\beta x+(1+\beta h_0)\sin\beta x\}$$

$=-85.7/0.12282\times\exp(-0.12282x)\times\{0.12282\times(-284.1)/85.7\cos(0.12282x)+(1+0.12282\times(-284.1)/85.7\sin(0.12282x))$

$=-697.8\exp(-0.12282x)\cdot(-0.4071\cos(0.12282x)$
$+0.5929\sin(0.12282x))$

c) Shear force for pile head rigid

$$S=-He^{-\beta x}\{\cos\beta x-(1+2\beta h_0)\sin\beta x\}$$

$=-85.7\times\exp(-0.12282x)\times\{\cos(0.12282x)-(1+2\times0.12282\times(-284.1/85.7)\times\sin(0.12282x)\}$

$=-85.7\exp(-0.12282x)\cdot\{\cos(0.12282x)-0.1857\sin(0.12282x)\}$

d) Bending moment for pile head hinged

$$M=-\frac{H}{\beta}e^{-\beta x}\sin\beta x$$

$=-85.7/0.12282\times\exp(-0.12282x)\times\sin(0.12282x)$

$=-697.8\times\exp(-0.12282x)\times\sin(0.12282x)$

B. For Seismic Case

e) Horizontal displacement for pile head rigid

$$y=\frac{H}{2EI\beta^3}e^{-\beta x}\{(1+\beta h_0)\cos\beta x-\beta h_0\sin\beta x\}$$

$= 942.3 / (2\times2.5\times10^7\times0.2484\times0.14606^3)\times e^{-0.14606x}\times\{(1+0.14606\times(-2561.9)/942.3)\times\cos(0.14606x) - 0.14606\times(-2561.9)/942.3\times\sin(0.14606x)\}$

$=0.02435\exp(-0.14606x)\cdot\{0.6029\cos(0.14606x)+0.3971\sin(0.14606x)\}$

f) Bending moment for pile head rigid

$$M=-\frac{H}{\beta}e^{-\beta x}\{\beta h_0\cos\beta x+(1+\beta h_0)\sin\beta x\}$$

$=-942.3/0.14606\times\exp(-0.14606x)\times\{0.14606\times(-2561.9)/942.3\cos(0.14606x)+(1+0.14606\times(-2561.9)/942.3\sin(0.14606x))$

$=-6451.5\exp(-0.14606x)\cdot(-0.3971\cos(0.14606x)$
$+0.6029\sin(0.14606x))$

g) Shear force for pile head rigid

$$S=-He^{-\beta x}\{\cos\beta x-(1+2\beta h_0)\sin\beta x\}$$

$=-942.3\times\exp(-0.14606x)\times\{\cos(0.14606x) - (1+2\times0.14606\times(-2561.9)/942.3)\times\sin(0.14606x)\}$

$=-942.3\exp(-0.14606x)\cdot\{\cos(0.14606x)-0.2057\sin(0.14606x)\}$

h) Bending moment for pile head hinged

$$M=-\frac{H}{\beta}e^{-\beta x}\sin\beta x$$

$=-942.3/0.14606\times\exp(-0.14606x)\times\sin(0.14606x)$

$=-6451.5\times\exp(-0.14606x)\times\sin(0.14606x)$

(2) Maximum bending moment at underground

A. For Ordinary Case

(i) Pile head Rigid

$$l_m=\frac{1}{\beta}\tan^{-1}\frac{1}{1+2\beta h_0}$$

$= 1/0.12282\times\tan^{-1}\{1/(1+2\times0.12282\times(-284.1/85.7))\}$

$=11.294\text{m}$

$$M_m=-\frac{H}{2\beta}\sqrt{(1+2\beta h_0)^2+1}\cdot\exp(-\beta l_m)$$

$=-85.7/(2\times0.12282)\times\sqrt{\ }\{(1+2\times0.12282\times(-284.1)/85.7)^2+1\}$

$\times\exp(-0.12282\times11.294)$

$=-88.6\ \text{kN}\cdot\text{m}$

(ii) Pile head Hinged

$l_m=\dfrac{\pi}{4\beta}$ $=\pi/(4\times0.12282)=6.391\ \text{m}$

$M_m=-0.3224\dfrac{H}{\beta}$ $=-0.3224\times85.7/0.12282=-225.0\ \text{kN}\cdot\text{m}$

B. For Seismic Case

(iii) Pile head Rigid

$$l_m=\frac{1}{\beta}\tan^{-1}\frac{1}{1+2\beta h_0}$$

$= 1/0.14606\times\tan^{-1}\{1/(1+2\times0.14606\times(-2561.9/942.3))\}$

$=9.365\text{m}$

$$M_m=-\frac{H}{2\beta}\sqrt{(1+2\beta h_0)^2+1}\cdot\exp(-\beta l_m)$$

$=-942.3/(2\times0.14606)\times\sqrt{\ }\{(1+2\times0.14606\times(-2561.9)/942.3)^2$

$+1\}\times\exp(-0.14606\times9.365)$

$=-838.6\ \text{kN}\cdot\text{m}$

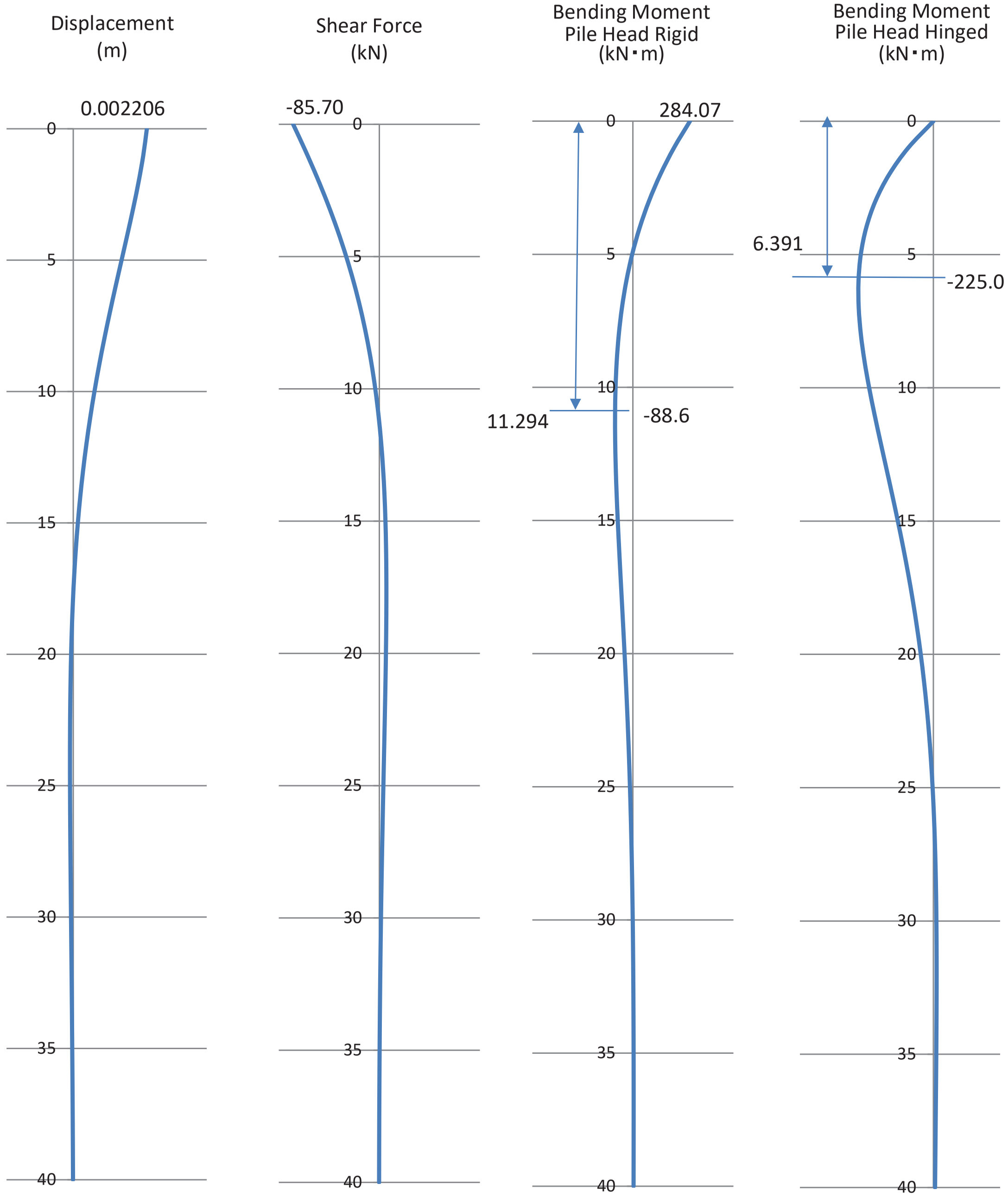
(iv) Pile head Hinged

$l_m=\dfrac{\pi}{4\beta}$ $=\pi/(4\times0.14606)=5.375\ \text{m}$

$M_m=-0.3224\dfrac{H}{\beta}$ $=-0.3224\times942.3/0.14606=-2080.0\ \text{kN}\cdot\text{m}$

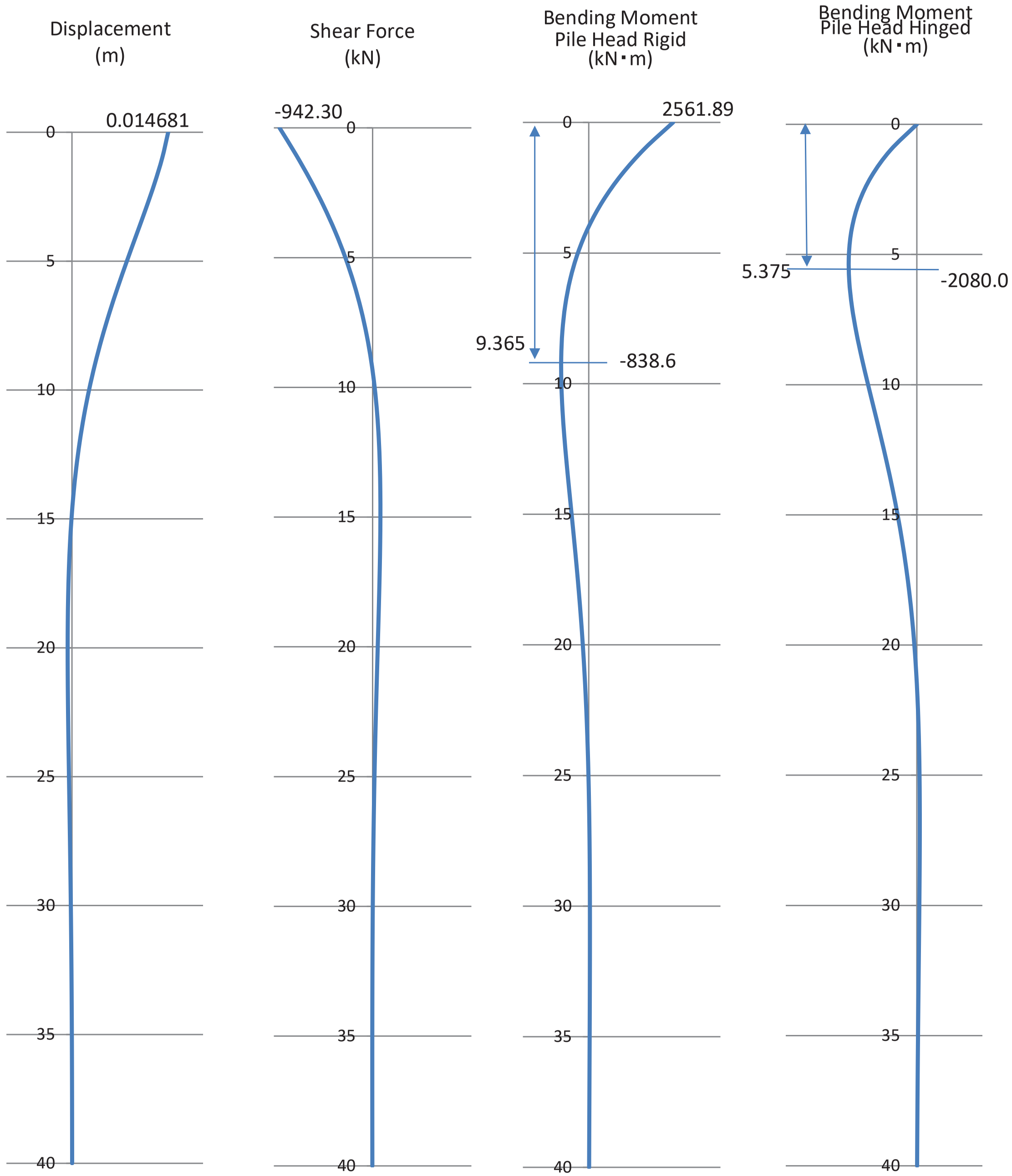
For Ordinary Case

	Pile Head Rigid			Pile Head Higed
Depth (m)	Horizontal displacement y (m)	Shear force (kN)	Bending Moment M (kN・m)	Bending Moment M (kN・m)
0	0.002206	-85.70	284.07	0.00
1	0.002100	-73.50	204.52	-75.61
2	0.001961	-62.00	136.84	-132.73
3	0.001801	-51.34	80.24	-173.87
4	0.001627	-41.64	33.83	-201.41
5	0.001448	-32.94	-3.37	-217.58
6	0.001269	-25.25	-32.38	-224.42
7	0.001095	-18.57	-54.21	-223.79
8	0.000930	-12.85	-69.84	-217.32
9	0.000776	-8.03	-80.20	-206.46
10	0.000635	-4.04	-86.16	-192.45
11	0.000507	-0.82	-88.53	-176.37
12	0.000394	1.73	-88.02	-159.08
13	0.000295	3.67	-85.27	-141.31
14	0.000210	5.09	-80.84	-123.64
15	0.000137	6.07	-75.23	-106.52
16	0.000077	6.67	-68.82	-90.28
17	0.000028	6.97	-61.98	-75.18
18	-0.000011	7.01	-54.97	-61.35
19	-0.000042	6.86	-48.02	-48.91
20	-0.000064	6.55	-41.31	-37.86
21	-0.000080	6.14	-34.95	-28.21
22	-0.000091	5.66	-29.05	-19.92
23	-0.000096	5.12	-23.66	-12.89
24	-0.000098	4.57	-18.81	-7.05
25	-0.000097	4.02	-14.51	-2.30
26	-0.000094	3.48	-10.76	1.48
27	-0.000088	2.96	-7.55	4.40
28	-0.000082	2.48	-4.83	6.56
29	-0.000075	2.04	-2.57	8.08
30	-0.000067	1.64	-0.74	9.05
31	-0.000059	1.28	0.71	9.57
32	-0.000052	0.96	1.83	9.72
33	-0.000044	0.69	2.66	9.58
34	-0.000037	0.46	3.23	9.21
35	-0.000031	0.27	3.60	8.68
36	-0.000025	0.11	3.78	8.03
37	-0.000020	-0.01	3.83	7.31
38	-0.000015	-0.11	3.76	6.55
39	-0.000011	-0.19	3.61	5.78
40	-0.000008	-0.24	3.40	5.03



For Seismic Case

	Pile Head Rigid			Pile Head Higed
Depth (m)	Horizontal displacement y (m)	Shear force (kN)	Bending Moment M (kN・m)	Bending Moment M (kN・m)
0	0.014681	-942.30	2561.89	0.00
1	0.013767	-781.20	1701.00	-811.36
2	0.012577	-632.11	995.48	-1387.27
3	0.011224	-497.48	431.98	-1766.15
4	0.009801	-378.61	-4.69	-1983.93
5	0.008376	-275.90	-330.58	-2073.39
6	0.007004	-189.03	-561.73	-2063.75
7	0.005721	-117.20	-713.61	-1980.47
8	0.004552	-59.25	-800.70	-1845.26
9	0.003512	-13.79	-836.21	-1676.27
10	0.002605	20.66	-831.90	-1488.29
11	0.001832	45.63	-798.01	-1293.06
12	0.001188	62.58	-743.28	-1099.61
13	0.000663	72.94	-675.01	-914.62
14	0.000246	77.98	-599.15	-742.77
15	-0.000074	78.87	-520.41	-587.07
16	-0.000310	76.62	-442.44	-449.17
17	-0.000475	72.12	-367.90	-329.64
18	-0.000581	66.10	-298.68	-228.24
19	-0.000638	59.17	-235.99	-144.11
20	-0.000658	51.82	-180.47	-75.98
21	-0.000648	44.41	-132.37	-22.30
22	-0.000617	37.25	-91.57	18.60
23	-0.000571	30.52	-57.73	48.45
24	-0.000515	24.38	-30.33	68.95
25	-0.000455	18.89	-8.76	81.72
26	-0.000393	14.09	7.67	88.24
27	-0.000332	10.00	19.66	89.86
28	-0.000275	6.57	27.89	87.75
29	-0.000222	3.76	33.00	82.94
30	-0.000174	1.53	35.60	76.29
31	-0.000132	-0.19	36.23	68.52
32	-0.000095	-1.47	35.37	60.18
33	-0.000065	-2.37	33.42	51.74
34	-0.000040	-2.96	30.73	43.53
35	-0.000019	-3.28	27.59	35.80
36	-0.000003	-3.41	24.23	28.70
37	0.000009	-3.37	20.82	22.34
38	0.000017	-3.23	17.51	16.77
39	0.000023	-3.00	14.40	12.00
40	0.000027	-2.71	11.54	7.99



Detailed Design on Bago River Bridge Construction Project

Design Technology Transfer
Lecture : Substructure and Foundation Design

16th November 2017: Group A&B

Contents of Lecture (4)

- Design of SPSP -

1. Construction Methodology of SPSP in New Thaketa Bridge
2. Design of SPSP

Contents of Lecture (4)

- Design of SPSP -

1. Construction Methodology of SPSP in New Thaketa Bridge
    1) Temporary Bridge Staging
    2) SPSP Installation
    3) Support Beam
    4) Footing
    5) Pier
    6) Removal Work



Work Flow of SPSP

Temporary Bridge Staging
- Temporary Bridge Installation
- Temporary Platform Installation
- Pile Rack Installation

SPSP Installation
- Hauling and Stockpiling SPSP
- Guide Pile Installation
- Guide Frame Installation
- Initial Driving of SPSP
- Final Driving of SPSP
- Rebound Testing
- Rapid Loading Test for SPSP
- Installation of Measurement Instrument
- Excavation inside SPSP
- Concrete Casting of SPSP
- Mortar Filling of Steel Pipe Joint

Support Beam
- Dewatering
- Bracket Installation for Support
- Cutting of SPSP (Bulkhead)

Support Beam
- Wale Installation
- Strut Installation
- Bottom Formwork Installation for Gap Filling Concrete between SPSP
- Gap Filling Concrete between SPSP

Footing
- Underwater Excavation inside Cofferdam
- Transport of Excavated Sand
- Soil Scraping
- Laying Gravel Materials
- Non-segregate Underwater Concreting of Bottom Slab
- Removal of Laitance generated on the surface of Bottom Slab Concrete
- Leveling Concrete
- Marking of Welding Place
- Polishing of Welding Place
- Stud Welding
- Pile Head Connection
- Footing Re-bar Installation

Footing
- Enclosed Welding of Re-bar
- Ultrasonic Flaw Detection Test of Enclosed Welding
- Footing Concrete Casting

Pier
- Scaffolding Installation for Pier
- Pier Re-bar Installation
- Enclosed Welding of Pier Re-bar
- Ultrasonic Flaw Detection Test of Enclosed Welding
- Formwork Installation for Pier
- Pouring of Pier Concrete

Removal Work
- Backfilling inside cofferdam
- Removal of Struts and Wales
- Mortar Filling Work
- Cutting of SPSP
- Removal and Hauling of SPSP
- Demobilization of Crawler Crane

1) Temporary Bridge and Platform Installation




Temporary Bridge Installation

↓

Temporary Platform Installation

↓

Pile Rack Installation

↓

Hauling and Stockpiling SPSP

01. Temporary Bridge Installation

Maximum load：approx.100t
（90t Crawler Crane + VibroHammer + H beam + Vibration force ）

200t Crawler Crane must not across
Fabrication/Demobilization of 200t crawler crane must be done on the Temporary Platform

Width：6m

02. Temporary Platform Installation

Maximum load ：approx.250t
（200t Crawler Crane + VibroHammer + SPSP + Vibration force ）

Width：8m
（200t Crawler Crane ：7.5m）

Finish of Installation

03. Pile Rack Installation

Make space for SPSP stockpiling

Hauling by a barge which has Max 20 piles of SPSP

Weight of SPSP：0.5t/m

※P1：48m/pile×20pieces×0.5t/m＝480t

※P2：40m/pile×20pieces×0.5t/m＝400t

04. Hauling and Stockpiling SPSP

Impossible to carry by land transport
Inadequate welding technique and validation test in Myanmar

⇒Steel manufactured in Japan
SPSP fabrication, welding and validation test in Vietnam
No site welding work

2) SPSP Driving and Concrete Filling

Initial Driving of SPSP（Vibro Hammer）

Guide pile
Guide frame

↓

Final Driving of SPSP（Hydraulic Hammer）

Rebounding test
Rapid loading test

↓

Excavation inside SPSP

↓

Concrete Casting inside SPSP

↓

Mortar Filling of Steel Pipe Joint

05. Guide Pile and Guide Frame Installation

Guide frame is also used as 1st wale
Bending process of guide frame was done in Japan




Stopper of SPSP when driving



Guide frame-fixed



SPSP-welded and fixed

06. Driving of SPSP by Vibro Hammer and Hydraulic Hammer

Driving speed of Hydraulic hammer ：1.5 times/sec

The Number of blow times
P1：approx.3,000 times/pile
P2：Max 20,000 times/pile

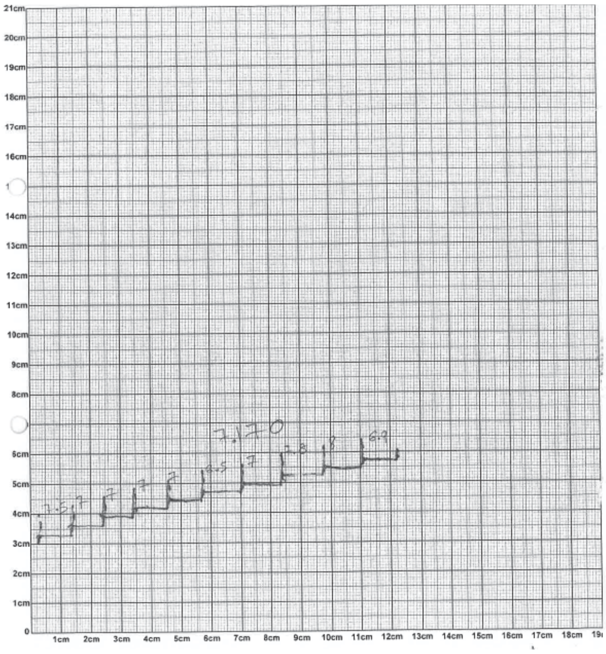
06. Driving of SPSP by Vibro Hammer and Hydraulic Hammer

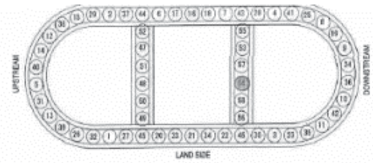
Key points about SPSP driving

※Verticality ➡Level Measurement by 2 transits from different direction,
Using level guide, Visual observation
※Position ➡Adjustment of error range from the reference position
※Buckling ➡Adjustment of blow energy
※Skin friction ➡Don't stop SPSP driving for a long time
※Vibration ➡Monitoring of existing bridge

07. Rebound Testing

Measure each pile rebound of 10 blows when remaining 1m
to the final setting depth





Reading from Pile (depth of pile) (m)	Pile Tip Elevation (m)	Time of blows (hh:mm:ss)	Cumulative Time of blows (hh:mm:ss)	No. of blows	Total no. of blows	Average rebound of 10 blows (mm)	REMARKS
0.000			0:29:44				Hydraulic Vibro Hammer
26.600	-22.255	0:00:00	0:00:00		0		Hydraulic Hammer
27.000	-22.655	0:00:40	0:00:40	18	18		
28.000	-23.655	0:01:43	0:02:23	47	74		
29.000	-24.655	0:01:17	0:03:40	65	139		
30.000	-25.655	0:01:25	0:05:05	72	211		
31.000	-26.655	0:01:46	0:06:51	85	296		

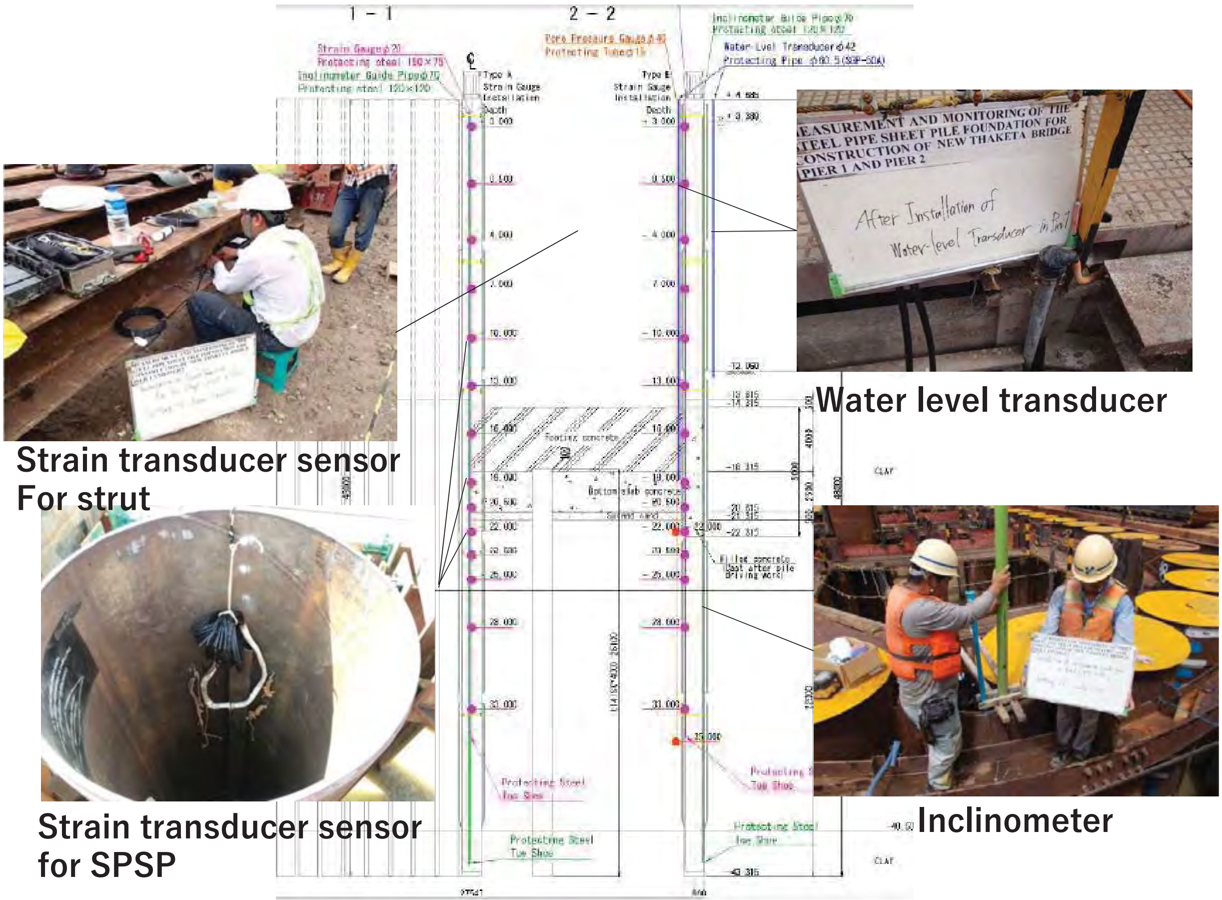
Prepared & Submitted by: Concurred & Approved by:

08. Rapid loading test for SPSP

Load bearing capacity test for SPSP
This test machine can’t be brought from Japan now
because this one was already sold to a company in Malaysia

※Installation：3 week
Test：1 day
Remove：1 week

09. Installation of Measurement Instrument



10. Excavation and Concrete Casting inside SPSP

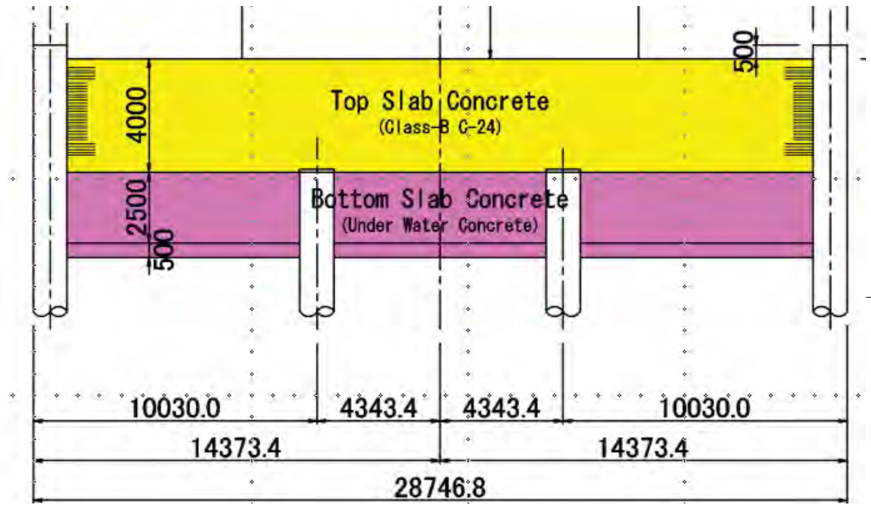
Excavate 10m deeper from the riverbed at P1
Excavate 16m deeper from the riverbed at P2
（Top slab concrete height 4m ×2 = 8m）

Renewing water inside pipes depends on the water quality
to prevent rising of the concrete level inside pipes which is caused by a chemical reaction of the cement with the suspended material in water
※In case No water in SPSP, pay attention to a heaving by underground water.

Riverbed

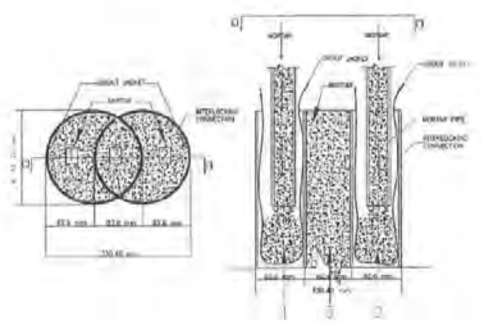
Excavation inside SPSP

Concrete Filling inside SPSP (8000mm)

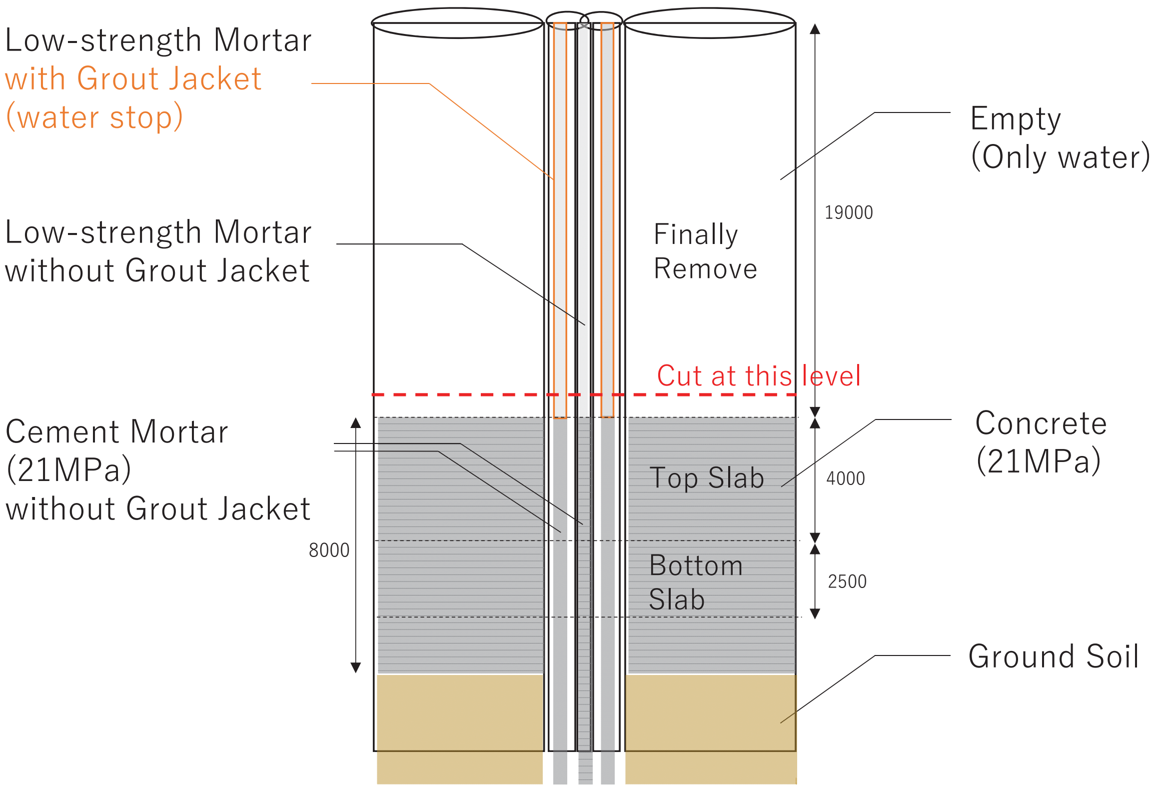


11. Mortar Filling of Steel Pipe Joint

Mortar filling up to the top of Top Slab
Low-strength mortar filling into the temporary SPSP with Grout Jacket



Mortar Filling of Steel Pipe Joint & Concrete Filling of SPSP interior



3) Support Beam Setting and Dewatering inside Cofferdam

Dewatering（Lower the water level)

↓

Cutting of SPSP (Bulkhead)

↓

Bracket Installation for Support Beam

↓

Wale Installation

↓

Strut Installation

Gap Filling
Concrete

12. Dewatering (Lower the water level)

21

Draining inside the cofferdam
(Lower the water level)
down to -1m water level from the position of strut

13. Bracket Installation for Support Beam

14. Cutting of SPSP (Bulkhead)

Gas Cutting
(Cutting machine is not used)

15. Wale Installation

22

Prepare the wale which has extra length to conform to the shape of the cofferdam

16. Strut Installation

Install struts down to the 4^{th} stage with dewatering in repeating process from no.12. to 18.

5^{th} ~ 7^{th} stage will be installed after bottom slab concrete casting
※Bottom slab concrete works as role of footing beam

17. Formwork Installation for gap filling concrete between SPSP

※Prevent a load concentration to the wale

18. Gap Filling Concrete

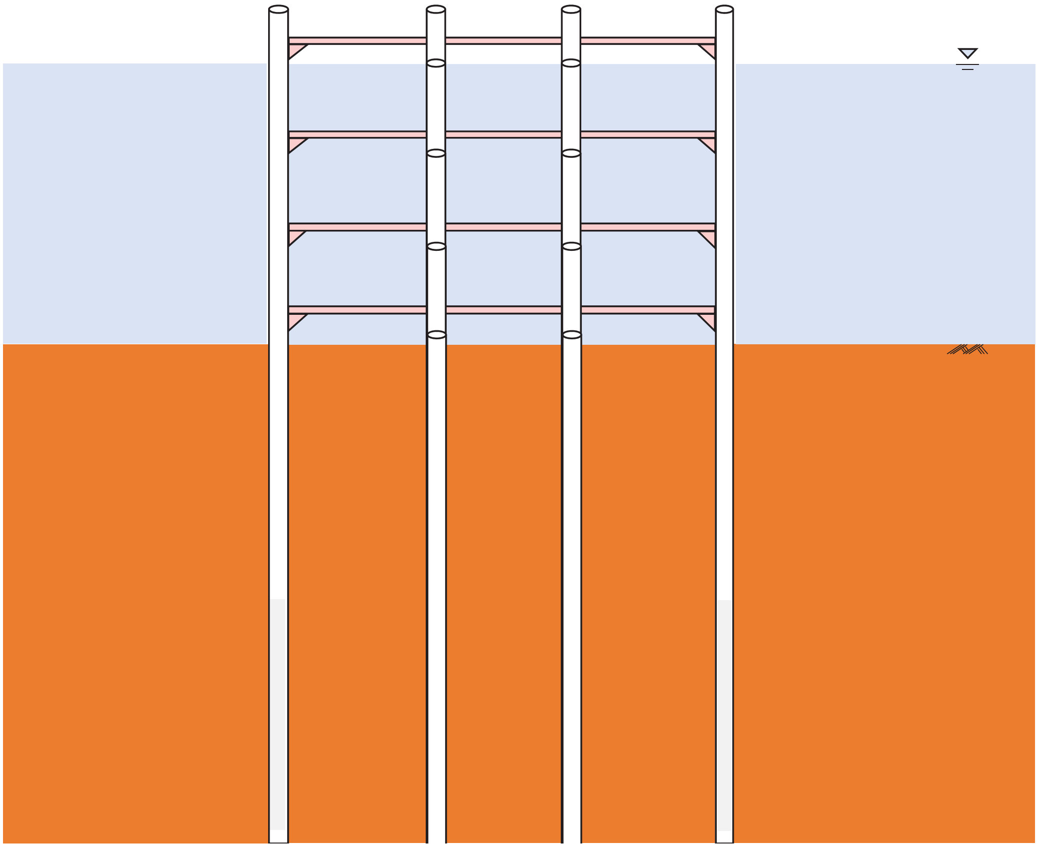
Consider the removal work when casting concrete

・Install a lifting piece for crane lifting up
・Filling concrete is covered in a vinyl

12. Dewatering

Draining inside the cofferdam
(Lower the water level)
up to -1m from the position of strut

Flow of Support Beam Installation



4-1) Excavation inside Cofferdam and Bottom Slab Concreting

Underwater Excavation inside Cofferdam

↓ — Transport of Excavated Soil

Soil Scraping

↓

Laying Gravel Materials

↓

Underwater Concreting of Bottom Slab

↓ — 5th ~ 7th stage: Bracket Installation for Support Beam → Wale Installation → Strut Installation

Laitance Removal & Leveling Concreting

19. Underwater Excavation inside Cofferdam

20. Soil Scraping

21. Laying Gravel Materials

Lay Gravel mat to decrease siltation (500mm)

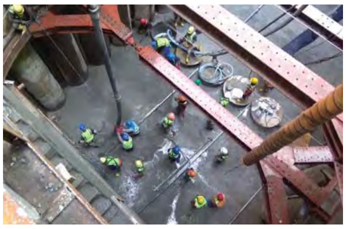
Leveling test by a diver

22. Underwater Concreting of Bottom Slab

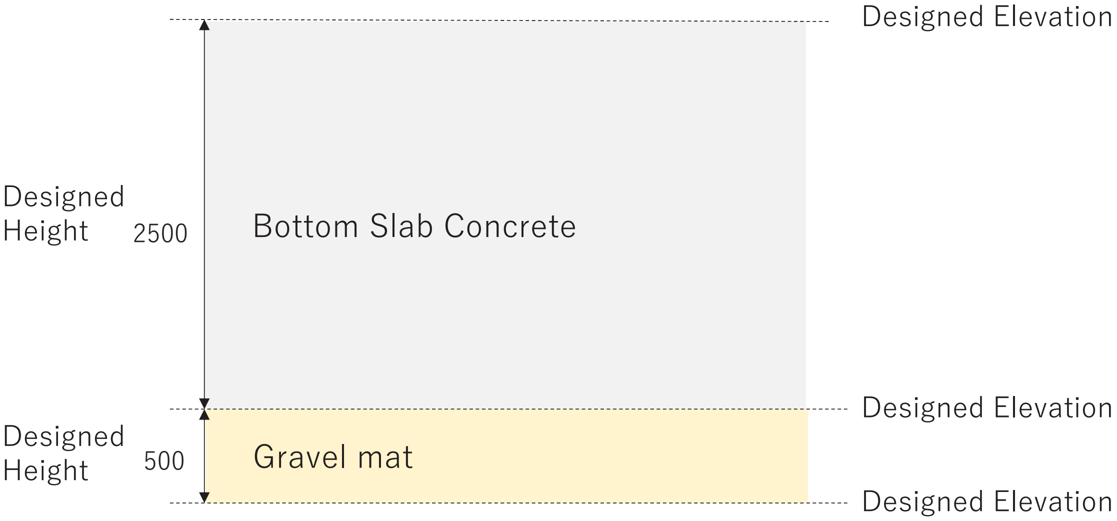
Underwater-inseparable admixtures made in Japan was used.
※Mix and Test of the underwater concrete was repeated several times with this admixtures.
※ A diver checks the finishing condition of concreting (Concrete level and filling in the gap of SPSP

23. Removal of Laitance generated on the surface of Bottom Slab Concrete

24. Leveling Concrete Casting

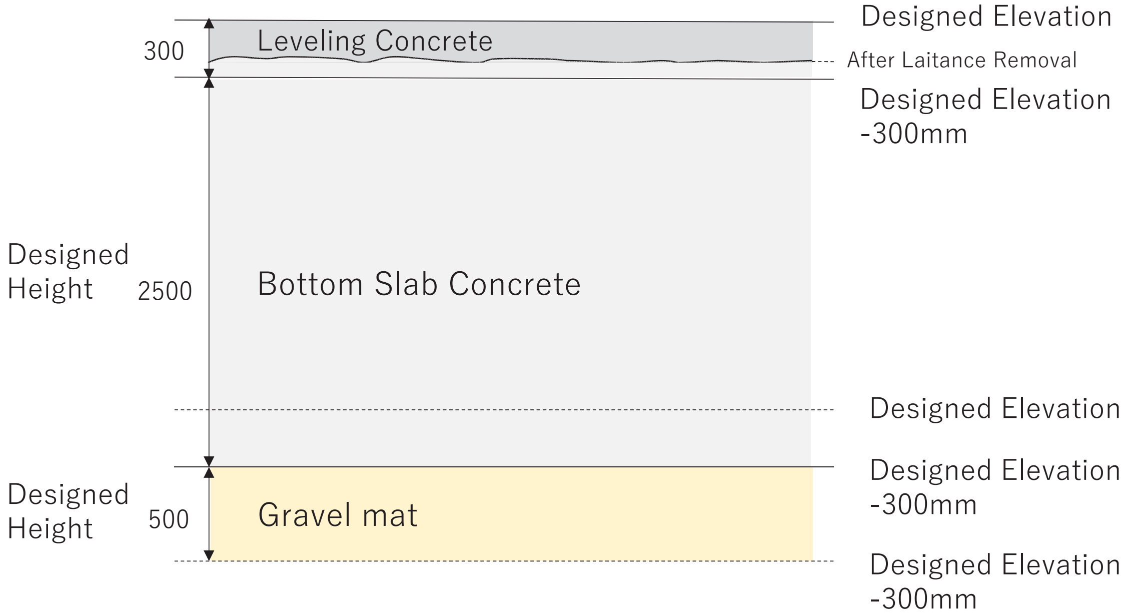


Designed Elevation of Bottom Slab Concrete & Gravel mat



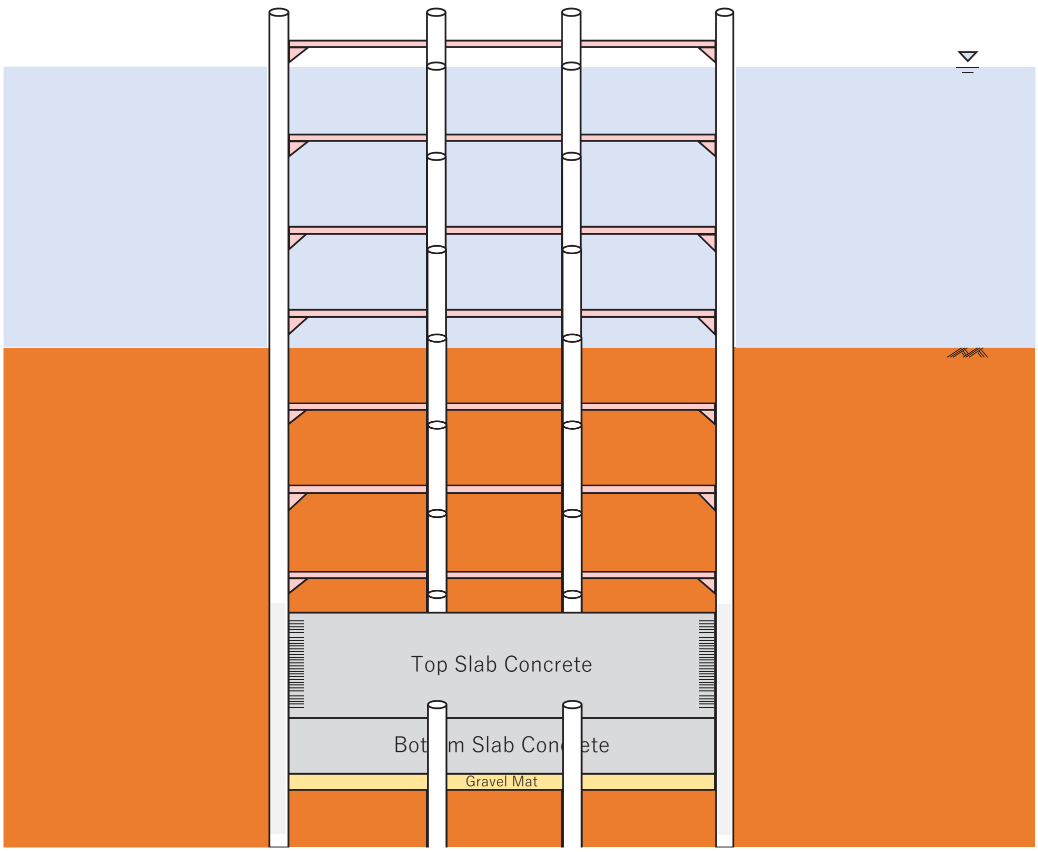
29

Construction Elevation of Bottom Slab Concrete & Gravel mat



30

Flow of Support Beam Installation



31

4-2) Footing Construction

Stud Welding

Pile Head Connection

Footing Re-bar Installation

Enclosed Welding of Re-bar

Ultrasonic Flaw Detection Test

Pier Re-bar Installation(Footing part)

Casting work of Footing Concrete

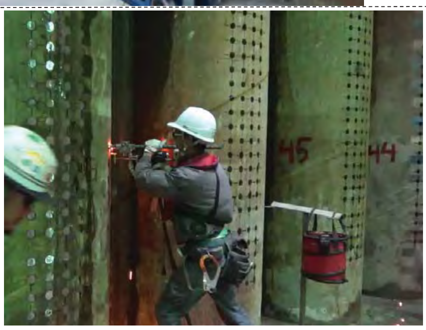
25. Marking of Welding Place

32

Mark the welding points

26. Polishing of Welding Place

Remove rust



27. Stud Welding

Welding speed：approx.1,000 pieces/day

Quantitative quality control in real time
Monitoring of ampere, arc time, Welding penetration depth and so on.

28. Pile Head Connection

Strengthen a connection between SPSP and top slab concrete

33

・Stud Welding

29. Footing Re-bar Installation

34

Be careful for lifting down into the cofferdam using a crawler crane

Need a drainage of leakage from the river

30. Enclosed Welding of Re-bar

Welding Speed：Max 200 numbers/day

Bottom rebar(D29) welded by enclosed welding to keep rebar spacing
※ For less than D19, lap joint was used.

31. Ultrasonic Flaw Detection Test of Enclosed Welding

By Ultrasonic Flaw Detection Test, welded rebar can be evaluated soon at a site OK or NG

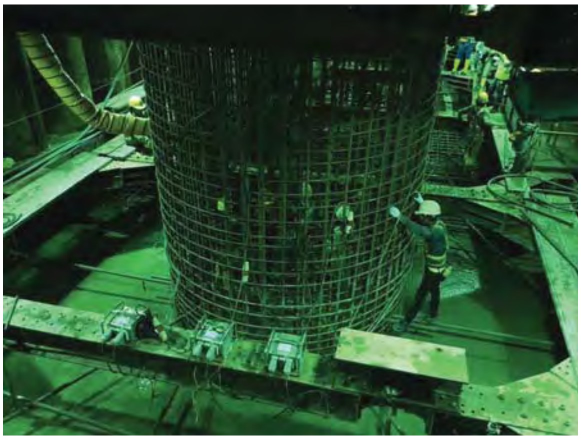
Pier Rebar in Top Slab Concrete

Ultrasonic Flaw Detection Test

32. Footing Concrete Casting

Concrete Casting Speed :
Max 30m^3/h in Thaketa site under quality control

Pay attention to :
- Vibrating under strut (※60mm Vibration machine)
- Hot weather concrete



5) Pier Construction

Pier Re-bar Installation

↓

Enclosed Welding of Pier Re-bar

↓ Ultrasonic Flaw Detection Test

Formwork Installation for Pier

↓

Pier Concrete Casting

↓

Curing of Concrete

33. Scaffolding and Re-bar Installation for Pier

33. Pier Re-bar Installation

34. Enclosed Welding of Pier Re-bar

Welding Speed：Max 140 numbers/day

35. Ultrasonic Flaw Detection Test of Enclosed Welding

36～37. Formwork Installation and Concrete Casting for Pier

36. Formwork Installation for Pier

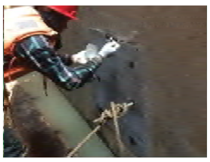
37. Pier Concrete Casting

To prevent a temperature crack,
Use cooling water when concrete mixing
(For footing concrete also)

38. Curing of Concrete

After removal of formwork, cover with sheets for temperature crack prevention

6) Removal works



Backfilling inside the cofferdam

↓

Removal of Struts and Wales

↓

Mortar Filling Work

Dewatering (upper the water level)

↓

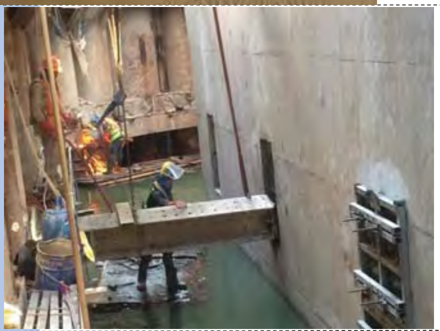
Cutting and Removal of SPSP

Hauling of SPSP

↓

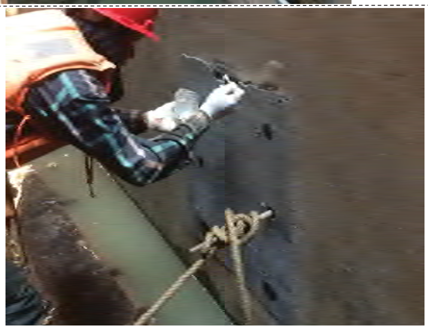
Demobilization of Crawler Crane

39. Backfilling inside the cofferdam



40. Removal of Struts and Wales

Gas cutting



41. Mortar Filling Work & Curing

41～42. Cutting and Lifting of SPSP

42. Cutting and Removal of SPSP

45

Cut SPSP at specified length to carry by trailer

Cutting and Removal took 2 hour per 1 pile

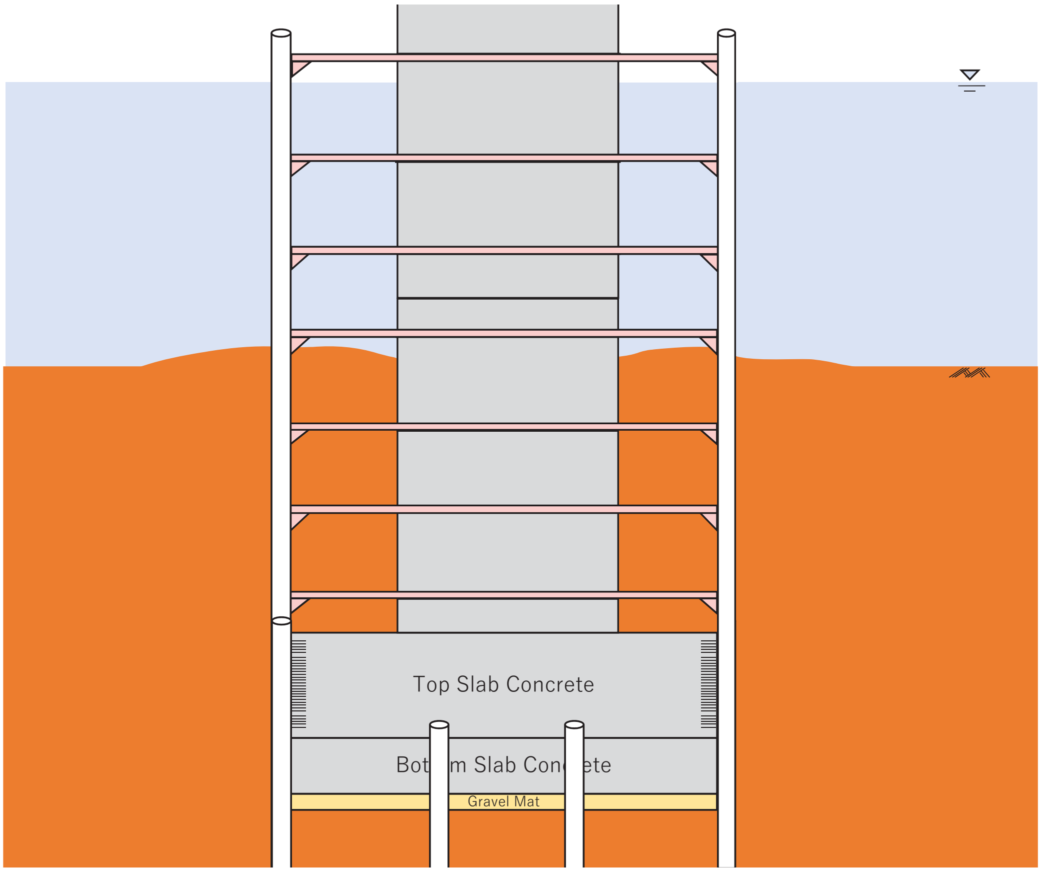
43. Hauling of SPSP

※Cutting and Removal and Hauling
P1： 7 days
P2：10 days

44. Demobilization of Crawler Crane

Flow of Support Beam Removal work

46



Pier after Removal of Cofferdam and Scaffolding

Thank you

You can watch movies of substructure work

Temporary Bridge Staging

- Temporary Bridge Installation
- Temporary Platform Installation
- Pile Rack Installation

SPSP Installation

- Hauling and Stockpiling SPSP
- Guide Pile Installation
- Guide Frame Installation
- Initial Driving of SPSP
- Final Driving of SPSP
- Rebound Testing
- Rapid Loading Test for SPSP
- Installation of Measurement Instrument
- Excavation inside SPSP
- Concrete Casting of SPSP
- Mortar Filling of Steel Pipe Joint

Support Beam

- Dewatering
- Bracket Installation for Support
- Cutting of SPSP (Bulkhead)

Support Beam

- Wale Installation
- Strut Installation
- Bottom Formwork Installation for Gap Filling Concrete between SPSP
- Gap Filling Concrete between SPSP

Footing

- Underwater Excavation inside Cofferdam
- Transport of Excavated Sand
- Soil Scraping
- Laying Gravel Materials
- Non-segregate Underwater Concreting of Bottom Slab
- Removal of Laitance generated on the surface of Bottom Slab Concrete
- Leveling Concrete
- Marking of Welding Place
- Polishing of Welding Place
- Stud Welding
- Pile Head Connection
- Footing Re-bar Installation

Footing

- Enclosed Welding of Re-bar
- Ultrasonic Flaw Detection Test of Enclosed Welding
- Footing Concrete Casting

Pier

- Scaffolding Installation for Pier
- Pier Re-bar Installation
- Enclosed Welding of Pier Re-bar
- Ultrasonic Flaw Detection Test of Enclosed Welding
- Formwork Installation for Pier
- Pouring of Pier Concrete

Removal Work

- Backfilling inside cofferdam
- Removal of Struts and Wales
- Mortar Filling Work
- Cutting of SPSP
- Removal and Hauling of SPSP
- Demobilization of Crawler Crane

Detailed Design on Bago River Bridge Construction Project

JICA Study Team

Design Technology Transfer
Lecture : Substructure and Foundation Design

16th November 2017: Group A&B

Contents of Lecture (4)

- Design of SPSP -

1. Construction Methodology of SPSP in New Thaketa Bridge
2. Design of SPSP

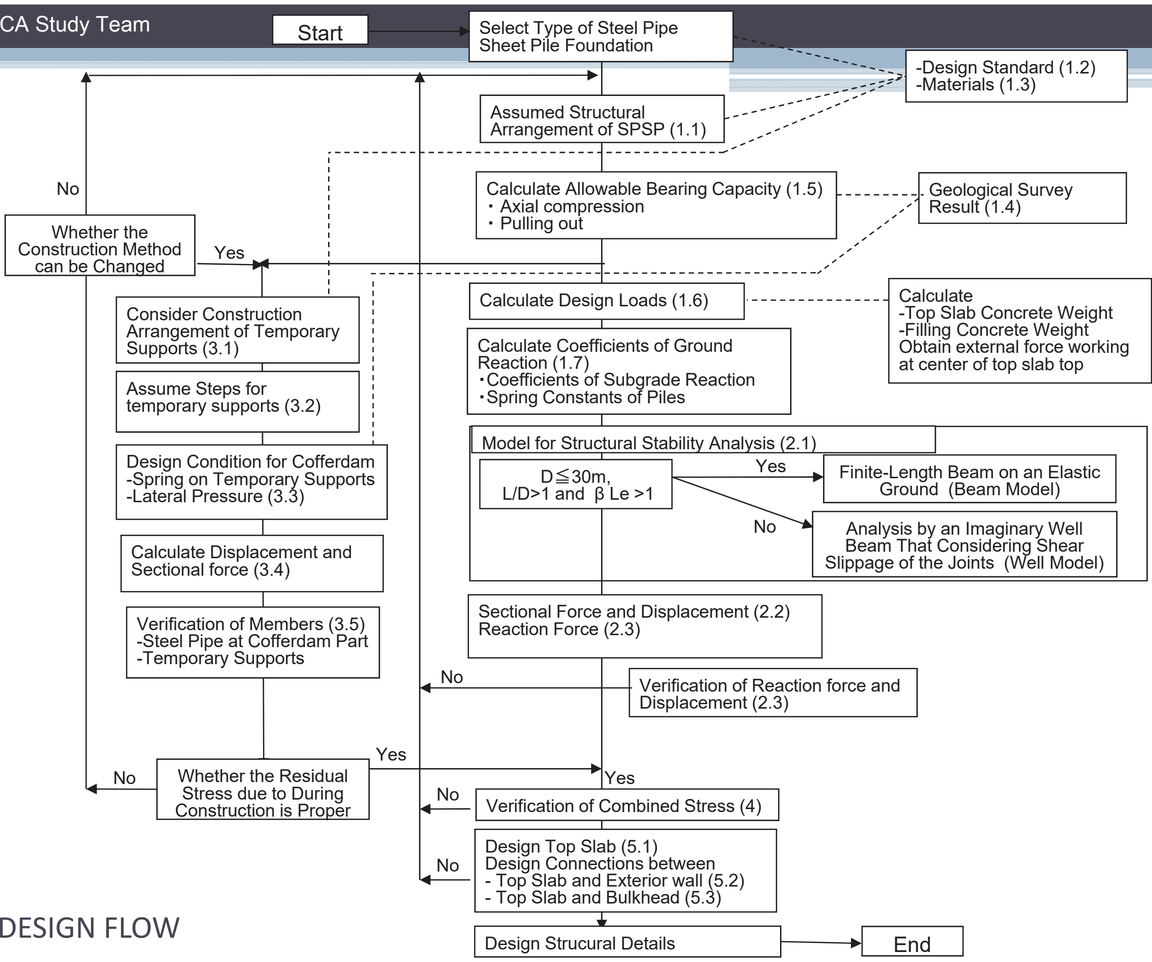
Contents of Lecture (4)

- Design of SPSP -

2. Design of SPSP

1. Design Condition
2. Structural Stability
3. Cofferdam and Temporary Supports
4. Verification of Combined Stress
5. Design of Top Slab and Connection

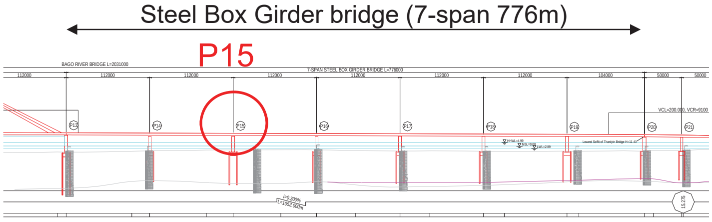


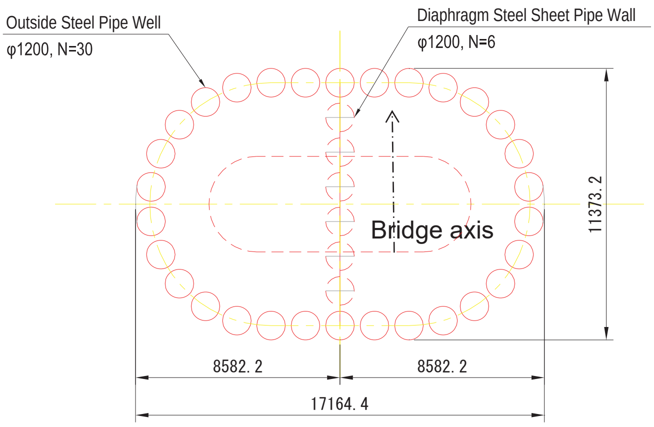


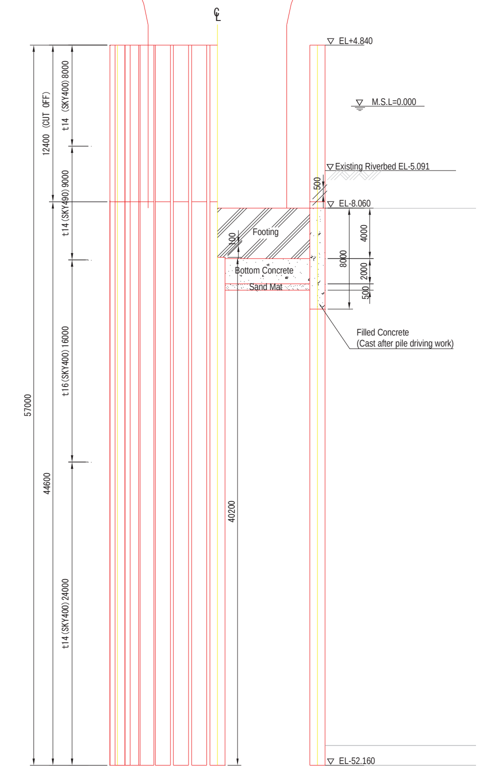
DESIGN FLOW

1.Design Condition

1.1 STRUCTURAL ARRANGEMENT OF SPSP







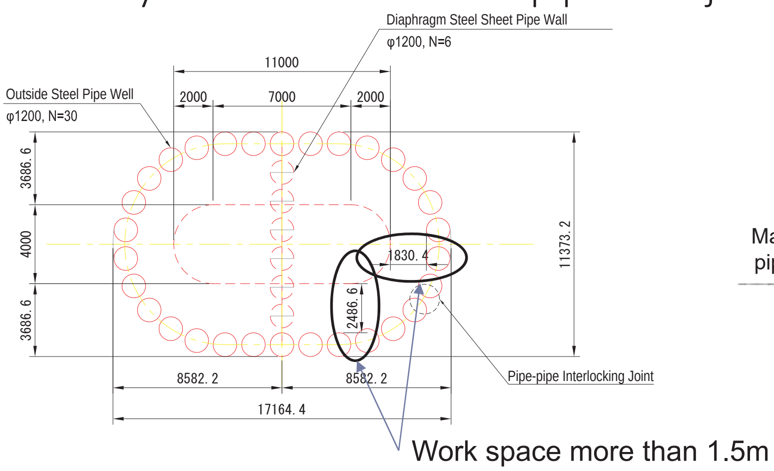
1.1 STRUCTURAL ARRANGEMENT OF SPSP

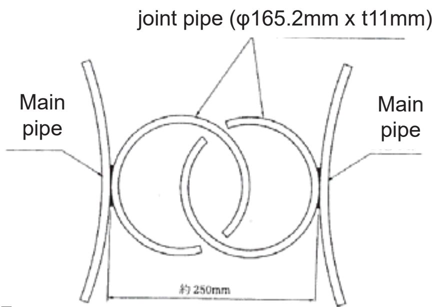
Dimension of well

・Well size is to be decided by taking account of a pier dimension and necessary space for pier construction.

・In aspect of minimization of river flow blockage, oval shape are recommended.

・The well dimension requires to match with geometrical size formed by combination of main pipes and joint pipes.





1.1 STRUCTURAL ARRANGEMENT OF S.P.S.P.

Pile length

・The bottom of SPSP is to be embedded in a good bearing layer in general.

・Upper part of SPSP is cofferdam portion in purpose for closure of water. Hence, top elevation of SPSP is determined by high water level.

Pile diameter

・Pile diameter is selected from available size for steel pipe sheet pile. Generally, 0.8m, 1.0m, 1.2m is well used.

・Due to function as cofferdam, larger pile size has more advantage. whereas, it requires more large machine for driving piles.

・With above conditions, the diameter is to be selected.

1.2 Design standard

SPECIFICATIONS FOR HIGHWAY BRIDGES (JSHB)

Published by JAPAN ROAD ASSOCIATION.

It is composed from five separate parts as below.

Foundation design is described in Part IV: substructures.

- -PART I : COMMON
- -PART II : STEEL BRIDGES
- -PART III : CONCRETE BRIDGES
- -PART IV : SUBSTRUCTURES
- -PART V : SEISMIC DESIGN

In JSHB, the design verification for substructures is based on Allowable stress method.

1.3 Material and Allowable Stress

- The allowable stresses used for the design is determined by type or strength of the material.
- The allowable stresses shall be multiplied by the respective increase coefficients stipulated in each load cases as table.

-Material Type : SKY400

No	Load case	Increase coefficient	Allowable bending compressive stress σsa’	Allowable bending tensile stress σsa
1	Dead + Live	1.00	140.00	140.00
2	Dead + Live + Temperature	1.15	160.00	160.00
3	Seismic Effect	1.50	210.00	210.00

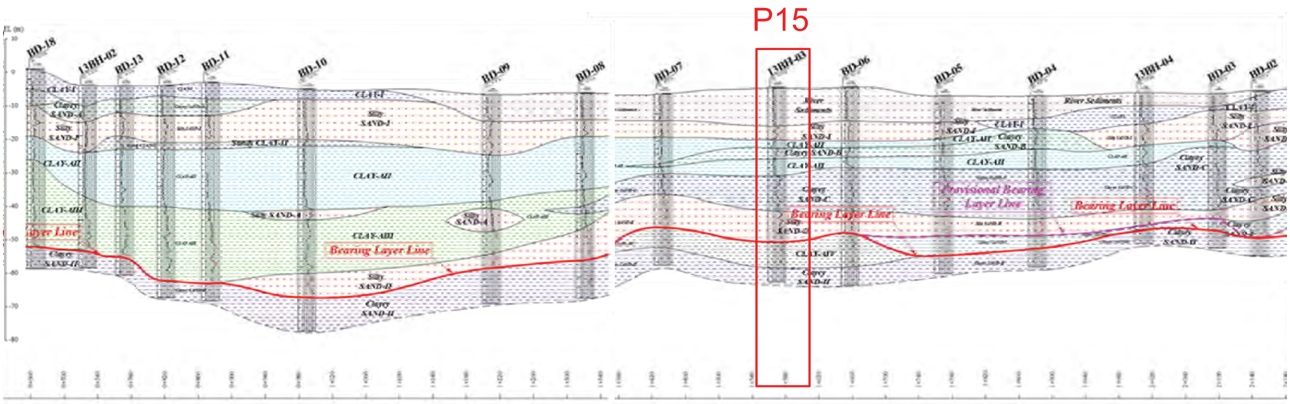
- Material Type : SKY490

No	Load case	Increase coefficient	Allowable bending compressive stress σsa’	Allowable bending tensile stress σsa
1	Dead + Live	1.00	185.00	185.00
2	Dead + Live + Temperature	1.15	215.00	215.00
3	Seismic Effect	1.50	280.00	280.00

1.4 Geological Condition

(1) Geological Profile

・By the observation of core condition, standard penetration test and all other result obtained by the geological survey, the bearing layer for foundations was set up as CLAY-AIV which is stiff to hard clay layer.



Soil Profile with Bearing Layer

1.4 Geological Condition

(2) Design Soil Parameters

- Soil parameters are to be obtained by Geotechnical consideration.

Soil design parameters for foundation design(river section)

No.	Soil Name	Representative N Value	Unit Weight γt (kN/m³)	Unit Weight γsat (kN/m³)	Unit Weight γ' (kN/m³)	Internal Friction Angle φ (°)	Cohesive Strength c (kN/m²)	Deformation Modulus E50 (kN/m²)
1	Silty SAND-River Sediments	3	17.0	18.0	8.0	29	-	1200
2	CLAY-I	1	17.5	17.5	7.5	-	10	900
3	Clayey SAND-A	3	17.5	18.5	8.5	28	-	1200
4	Silty SAND-I	13	17.0	18.0	8.0	33	-	5200
5	Sandy CLAY-II	9	17.5 Same values as CLAY-AII	17.5	7.5	-	54	6300
6	CLAY-AII	7	17.5	17.5	7.5	-	42	4900
7	Clayey SAND-B	13	17.0	18.0	8.0	32	-	9100
8	Silty SAND-A	25	17.0	18.0	8.0	33	-	17500
9	CLAY-AIII	18	18.0	18.0	8.0	-	108	12600
10	Clayey SAND-C	20	17.0	18.0	8.0	33	-	14000
11	Silty SAND-II	30	17.0	18.0	8.0	34	-	21000
12	Clayey SAND-I	35	19.0	20.0	10.0	34	-	24500
13	CLAY-AIV	30	18.0	18.0	8.0	-	180	21000
14	Clayey SAND-II	50	19.0	20.0	10.0	35	-	35000

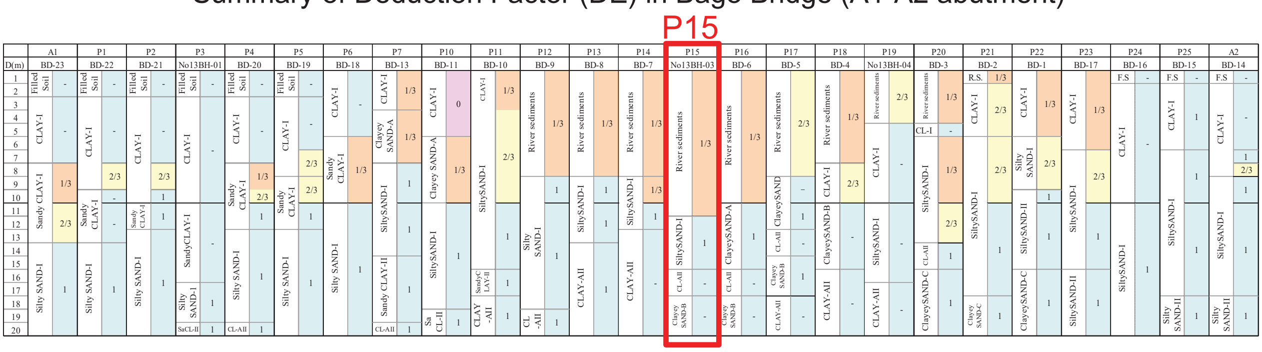
1.4 Geological Condition

(3) Consideration of Liquefaction

Deduction Factor (DE) on Soil Modulus

FL	Depth from an earth surface x(m)	R (The dynamic shear strength ratio) $R \leqq 0.3$	$0.3 < R$
$F_L \leqq 1/3$	$0 \leqq x \leqq 10$	0	1/6
	$10 < x \leqq 20$	1/3	1/3
$1/3 < F_L \leqq 2/3$	$0 \leqq x \leqq 10$	1/3	2/3
	$10 < x \leqq 20$	2/3	2/3
$2/3 < F_L \leqq 1$	$0 \leqq x \leqq 10$	2/3	1
	$10 < x \leqq 20$	1	1

Summary of Deduction Factor (DE) in Bago Bridge (A1-A2 abutment)



1.5 Allowable Bearing Capacity

(1) Axial compressive allowable bearing capacity

(1) Axila compressive allowable bearing capacity

Axial compressive allowable bearing capacity of a single pile is determined by equation below;

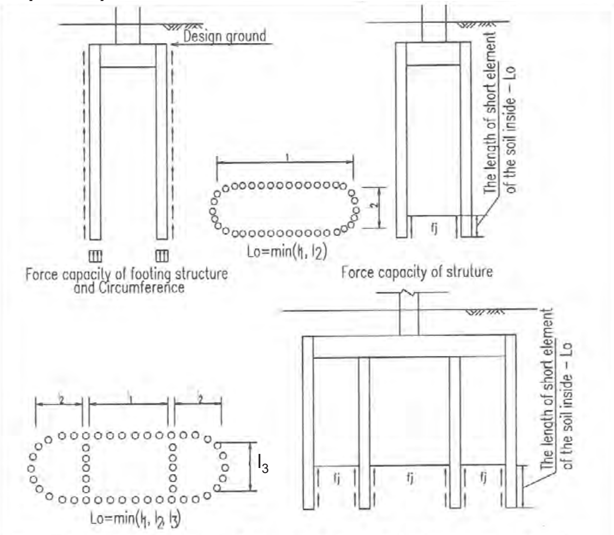


Figure 13. Range considering of skin friction force at the inner peripheral surface of the foundation

$$Ra = \frac{1}{n} \cdot Ru$$

n : safety factor 3.0 (ordinary)

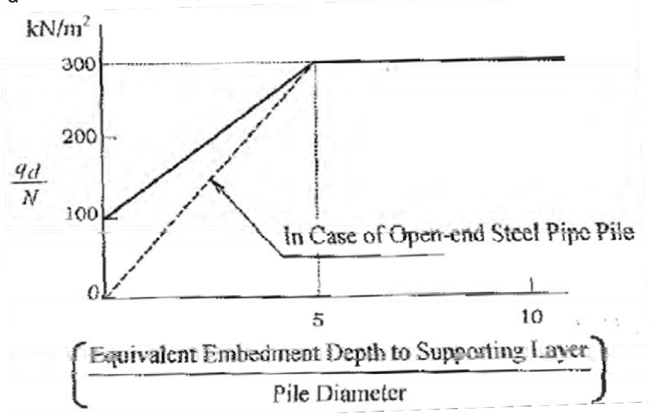
2.0 (seismic)

R_u : ultimate bearing capacity of steel pipe sheet pile (kN/pile)

$$Ru = \underbrace{qd \cdot A1}_{\text{End bearing}} + \frac{1}{n1+n2} \times \underbrace{\left\{ \underbrace{U1 \cdot \Sigma (Li \cdot fi)}_{\text{Outer peripheral surface}} + \underbrace{U2 \cdot \Sigma (Lj \cdot fj)}_{\text{Inner peripheral surface}} \right\}}_{\text{skin friction force}}$$

A_1 : enclosed area of the steel pipe sheet pile (m^2)
q_d : ultimate bearing capacity per unit area to be borne at pile (kN/m^2)
n_1 : number of sheet piles in exterior wall
n_2 : number of sheet piles in bulkhead
U_1 : circumference envelop length of exterior wall (m)
U_2 : circumference envelop length of interior wall and bulkhead (m)
L_i : length of each layer considering side friction for exterior wall (m)
f_i : maximum unit skin friction force at outer peripheral surface (kN/m^2)
L_j : length of each layer considering side friction for interior wall (m)
Consider only the region from the bottom face to the short-side length (L_0) of the inner soil.
f_j : maximum unit skin friction force at inter peripheral surface (kN/m^2)

- End Bearing Capacity Intensity q_d (For driven pile)



N: SPT value of the ground at the pile tip ≤ 40

- Maximum unit skin friction force

Pile Installation Method \ Ground Type	Sandy Soil	Cohesive Soil
Driven Pile Method (including Vibro-hammer Method)	$2N (\leq 100)$	c or $10\,N (\leq 150)$
Cast-in-place RC Pile Method	$5N (\leq 200)$	c or $10\,N (\leq 150)$
Bored Pile Method	$2N (\leq 100)$	$0.8c$ or $8\,N (\leq 100)$
Pre bored Pile Method	$5N (\leq 150)$	c or $10\,N (\leq 100)$
Steel Pipe Soil Cement Pile Method	$10N (\leq 200)$	c or $10\,N (\leq 200)$

(Note) c: cohesion of ground (kN/m^2), N: N value from SPT.

- ✓ If soft soil with N value < 5, maximum unit skin friction force ($f_{i,j}$) will be neglected, namely 0.0.
- ✓ In the ground which becomes unstable at the time of earthquake (liquefaction), the maximum unit skin friction force is multiplied by the reduction coefficient DE value.

The shaft resistance in each soil layer is to be summated.

1.5 Allowable Bearing Capacity

(2) Pull-out allowable bearing capacity

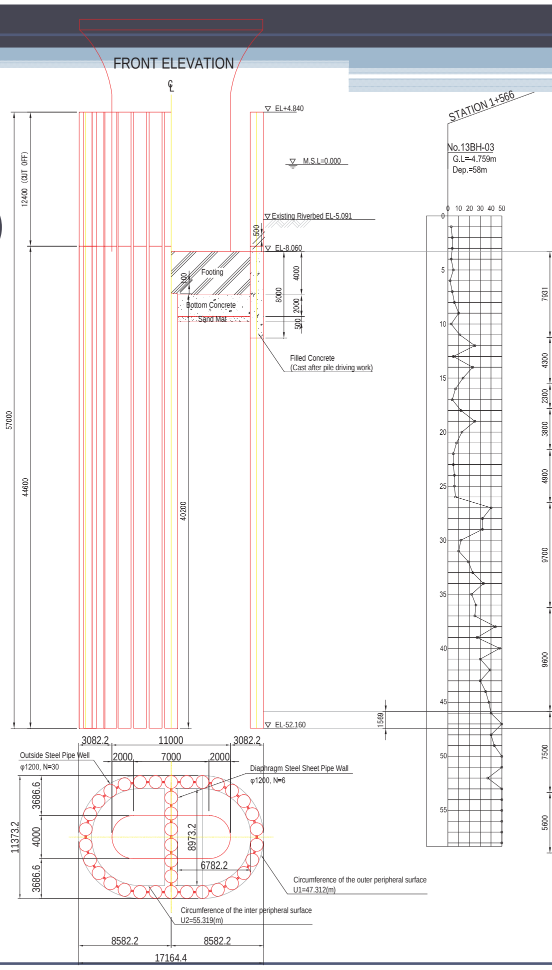
The pull-out allowable bearing capacity of a single pile can be obtained from equation below;

$$Pa = \frac{1}{n} \cdot Pu + W$$

$$Pu = \frac{1}{n1+n2} \times \left\{ U1 \cdot \Sigma (Li \cdot fi) + U2 \cdot \Sigma (Lj \cdot fj) \right\}$$

Pu : ultimate pull-out force of a steel pipe sheet pile (kN/pile)
n : safety factor 6.0 (ordinary), 3.0 (seismic)
W : effective weight of steel pipe sheet pile W (= w1+w2+w3+w4)
w1 :Weight of steel pipe w2 :Weight of joint
w3: Weight of soil in pipe w4: Weight of filling concrete

Exercise1: Calculation of Allowable Bearing Capacity (Case P15 Foundation)



	Soil Name	Average N-Value	Unit Weight γ_{sat} (kN/m³)	Cohesion C_u (kN/m²)	Friction Angle φ (°)	Modulus of Elasticity E_0 (kN/m²)	Liquefaction Coefficient D_E
7931	Silty Sand River Sediments	3	18	0	29	1,200	Liquefaction D_E = 1/3
4300	Silty Sand-I	13	18.0	0	33	5,200	Liquefaction D_E =1
2300	Clay-AII	7	17.5	42	0	4,900	-
3800	Clayey Sand-B	13	18.0	0	32	9,100	-
4900	Clay-AII	7	17.5	42	0	4,900	-
9700	Clayey Sand-C	20	18.0	0	33	14,000	-
9600	Silty Sand-II	30	18.0	0	34	21,000	-
7500	Clayey-AIV	30	18.0	180	0	21,000	-
5600	Clayey Sand-II	50	20.0	0	34	35,000	-

A_1 : 1.131(m²)
n_1 : 30
n_2 : 6
U_1 : 47.312(m)
U_2 : 55.319(m)
L_0 : 6.782(m)

Exercise1: Calculation of Allowable Bearing Capacity

1. Axial Compressive Allowable Bearing Capacity

$$Ra = \frac{1}{n} \cdot Ru$$

$$Ru = qd \cdot A1 + \frac{1}{n1+n2} \times \{ U1 \cdot \Sigma (Li \cdot fi) + U2 \cdot \Sigma (Lj \cdot fj) \}$$

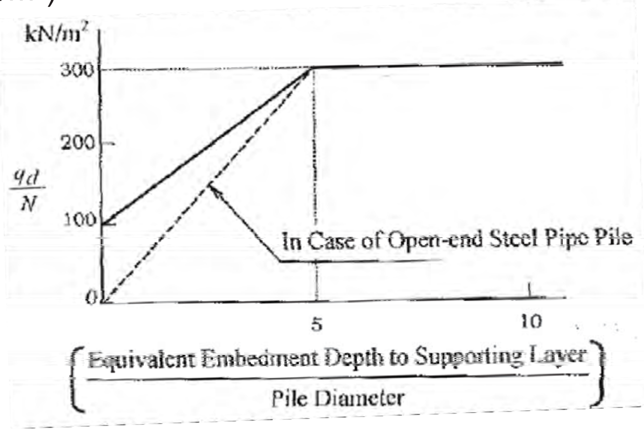
1) End Bearing Capacity

$$\frac{\text{Equivalent embedment depth to supporting layer}}{\text{Pile diameter}} = \frac{\boxed{}}{1.2\text{m}} = \boxed{}$$

qd / N = $\boxed{}$ N = $\boxed{}$ A1 = 1.131 (m²)

=> qd = $\boxed{}$ * $\boxed{}$ = $\boxed{}$ (kN/m²)

∴ qd x A1= 2,663kN



Exercise1: Calculation of Allowable Bearing Capacity

2) Skin Friction Capacity

Skin friction force at the outer peripheral surface

Layer No	Soil type	Average N-value	Depth Li (m)	fi (kN/m²) Ordinary	DEi	Li・fi (kN/m) Ordinary	Li・fi (kN/m) Earthquake
1	Sandy	3.0	7.931	0.0	1/3	0.0	0.0
2	Sandy	13.0	4.300	26.0	1	111.8	111.8
3	Cohesive	7.0	2.300	70.0	1	161.0	161.0
4	Sandy	13.0	3.800	26.0	1	98.8	98.8
5	Cohesive	7.0	4.900	70.0	1	343.0	343.0
6	Sandy	20.0	9.700	40.0	1	388.0	388.0
7	Sandy	30.0	$\boxed{}$	60.0	1	$\boxed{}$	$\boxed{}$
8	Cohesive	30.0	$\boxed{}$	150.0	1	$\boxed{}$	$\boxed{}$
Sum			44.1			1913.9	1913.9

Skin friction force at the inner peripheral surface

Short side length : Lo = 6.782 (m)

Layer No	Soil type	Average N-value	Depth Li (m)	fi (kN/m²) Ordinary	DEi	Li・fi (kN/m) Ordinary	Li・fi (kN/m) Earthquake
1	Sandy	30.0	5.213	60.0	1	312.8	312.8
2	Cohesive	30.0	1.569	150.0	1	235.4	235.4
Sum			6.782			548.1	548.1

$$\therefore \frac{1}{n1+n2} \times \left\{ U1 \cdot \Sigma(Li \cdot fi) + U2 \cdot \Sigma(Lj \cdot fj) \right\}$$

$= 1/(\boxed{}+\boxed{}) \times (\boxed{} \times \boxed{} + \boxed{} \times \boxed{})$

= 3358 (kN/pile) in ordinary, seismic condition

Exercise1: Calculation of Allowable Bearing Capacity

3) Ultimate bearing capacity of pile

$Ru = \boxed{} + \boxed{} =$ 6021 (kN/pile)

4) Allowable bearing capacity of pile

Ordinary Ra = (1 / 3) ・ 6021 = 2,007 (kN/pile)

Seismic Ra = (1 / 2) ・ 6021 = 3,011(kN/pile)

2. Pull-out Allowable Bearing Capacity

1) Ultimate bearing capacity of pile

$$Pa = \frac{1}{n} \cdot Pu + W$$

$$Pu = \frac{1}{n1+n2} \times \left\{ U1 \cdot \Sigma(Li \cdot fi) + U2 \cdot \Sigma(Lj \cdot fj) \right\}$$

Pu=3,358kN/pile in ordinary, seismic condition

2) Allowable bearing capacity of pile

W : effective weight of steel pipe sheet pile W (= w1+w2+w3+w4)

w1 :Weight of steel pipe: 139.8kN　　w2 :Weight of joint: neglect

w3: Weight of soil in pipe:307.0kN　　w4: Weight of filling concrete: neglect

Ordinary Ra = (1 / 6) ・ 3358 + 447 = 1,006 (kN/pile)

Seismic Ra = (1 / 3) ・ 3358 + 447 = 1,566 (kN/pile)

1.6 Design Force

Image of external forces

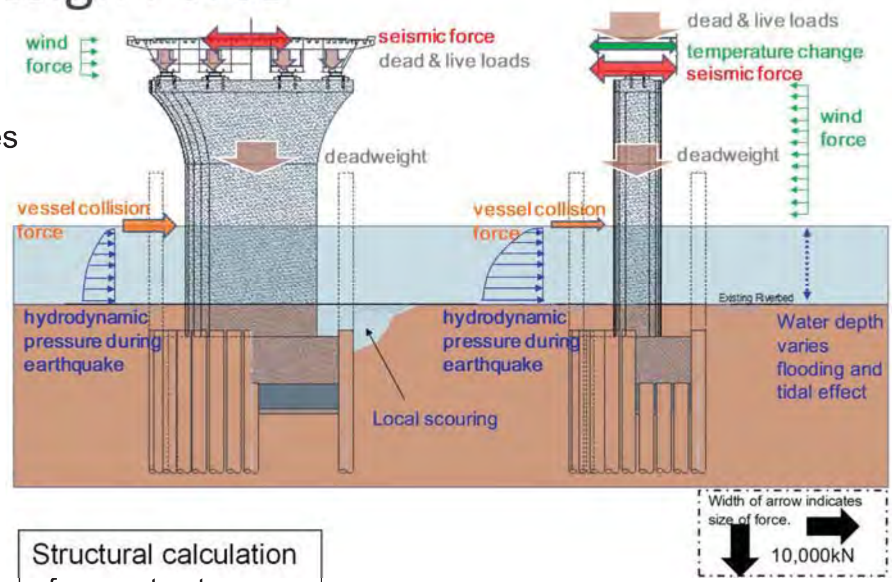


Image of transmitting design force

Structural calculation of superstructure

By using force from super-structure

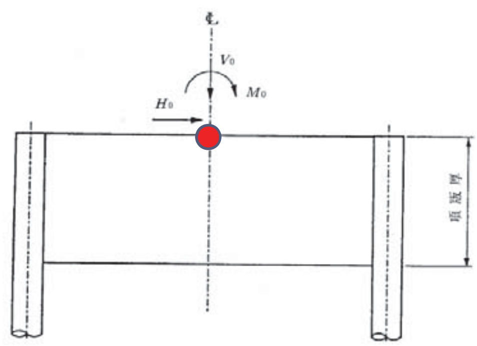
Structural calculation of sub-structure

By using force from sub-structure

Structural calculation of foundation

1.6 Design Force

- For design calculation of SPSP, design force (V, H, M) worked at the center of top slab is needed.
- It is obtained through structural calculation for super-structure and sub-structure.
- Design force for SPSP requires to add the weight of top slab, filling concrete and overburdened soil onto the external force acting at top slab at top.



- In foundation designing, the stability and member stresses should be verified with consideration of the most adverse conditions of the load combination. The water level is selected from L.W.L. to H.W.L. to cover most adverse condition against foundation.
- The design verification shall be performed in both the longitudinal and transverse directions.

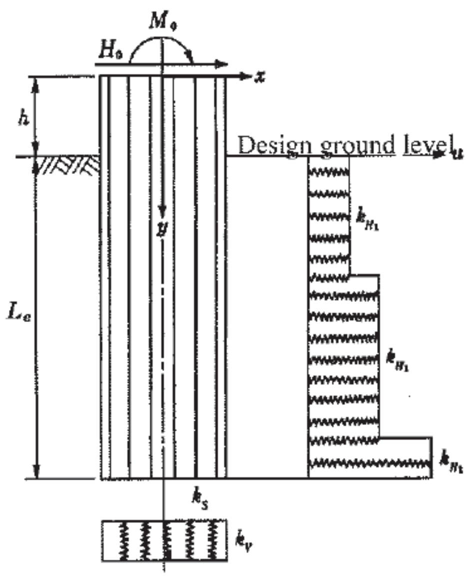
Example of P15 Foundation (longitudinal direction)

No	Load case	Vo (kN)	Ho (kN)	Mo (kN.m)
1	Dead + Live with L.W.L.	62600	0	0
2	Dead + Live with H.W.L.	60400	0	0
3	Dead + Live + Temperature with L.W.L.	62600	3100	67000
4	Dead + Live + Temperature with H.W.L.	60400	3100	67000
5	Extreme wind situation with effect of temperature change	59600	3200	68800
6	Vessel Collison	60600	3200	36500
7	Seismic effect with M.W.L.	54800	14900	237500

1.7 Coefficients of Ground Reaction

(1)General

A coefficient of ground reaction is one of the basic design constants necessary for finding displacement and subgrade reaction of a foundation.



Coefficient of ground includes;

- Coefficient of horizontal ground reaction at a bottom : K_{H1}
- Coefficient of vertical ground reaction at a front side: K_V
- Coefficient of horizontal shear ground reaction at the bottom K_S

1.7 Spring Constants of Pile

(2) Coefficient of Horizontal Ground Reaction

A coefficient of horizontal ground reaction can be calculated by;

$$kH1 = (1 + \alpha H) \cdot kH \cdot \left(\frac{y}{yo}\right)^{-1/2}$$

$$kH = \frac{1}{0.3} \cdot \alpha \cdot Eo \cdot \left(\frac{BH}{0.3}\right)^{-3/4}$$

Where,

kH1: coefficient of horizontal ground reaction considering strain dependency (kN/m^3)

kH: coefficient of horizontal ground reaction (kN/m^3)

αH: Increment factor shared by the horizontal shear ground reaction at the circumferential surface of well part (1.00 may be used)

y: horizontal displacement of foundation at design level (m)

y0: base displacement (m) *in general, 1% of the foundation width ≤5.0cm

BH: equivalent loading width of foundation in front (m)

$$BH = \sqrt{B/\beta} \leqq \sqrt{B \cdot Le}$$

B: loading width orthogonal to loading direction (m)

Le: effective embedment depth of foundation (m)

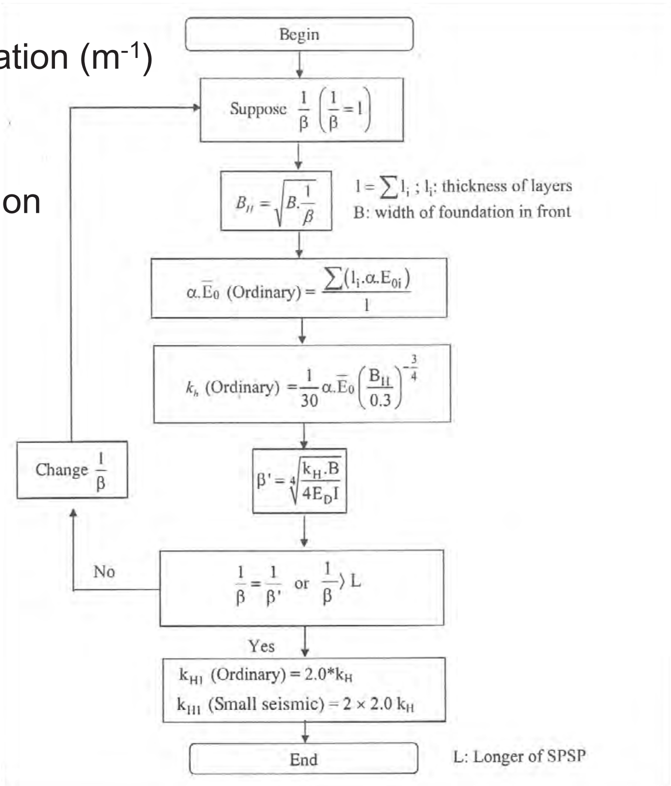
1/β : soil depth relating to lateral resistance, and is less than foundation length (m)

β : characteristic value of foundation (m^{-1})

$$\beta = \sqrt[4]{\frac{kH \cdot B}{4 \cdot E \cdot I}}$$

E : Young's modulus of foundation = 2.00×10^8 (kN/m^2)

I : moment of inertia of foundation (m^4)



Example of P15 Foundation

BH: equivalent loading width of foundation in front (m)

	Bridge Axis Direction		Bridge Axis Perp. Direction	
	Ordinary	Seismic	Ordinary	Seismic
I (cm4)	1.7976E+009		3.0544E+009	
B (cm)	1716.44		1137.32	
β(cm-1)	0.000420	0.000420	0.000362	0.000362
1/β(cm)	2379.4	2379.4	2760.6	2760.6
Average α・Eo (N/cm2)	1842.8	1842.8	2361.6	2361.6
$BH < \sqrt{B \cdot Le}$	2020.9 < 2751.3	2020.9 < 2751.3	1771.9 < 2239.6	1771.9 < 2239.6

kH1: coefficient of horizontal ground reaction considering strain dependency (kN/m³)

Layer No	Layer Thickness (m)		α・Eo(kN/m2)		Bridge Axis kH1(kN/m3)		Bridge Axis Perp kH1(kN/m3)	
	Ordinary	Seismic	Ordinary	Seismic	Ordinary	Seismic	Ordinary	Seismic
1	7.931	7.931	4800	9600	1361	906	1502	1000
2	4.300	4.300	20800	41600	5897	11795	6509	13017
3	2.300	2.300	19600	39200	5557	11114	6133	12266
4	3.800	3.800	36400	72800	10320	20640	11390	22780
5	4.900	4.900	19600	39200	5557	11114	6133	12266
6	9.700	9.700	56000	112000	15877	31754	17523	35046
7	9.600	9.600	84000	168000	23816	47632	26284	52569
8	1.569	1.569	84000	168000	23816	47632	26284	52569

(3) Coefficient of Vertical Ground Reaction

$$kv = \frac{1}{0.3} \cdot \alpha \cdot Eo \cdot \left(\frac{Bv}{0.3}\right)^{-3/4}$$

Where, kv : vertical ground reaction coefficient (kN/m³)

α・Eo : modulus of deformation of ground (kN/m²)

Bv : equivalent loading width of foundation (m) = Diameter of steel pipes: D_0

(4) Coefficient of Horizontal Shear Ground Reaction

ks = 0.3・kv

where, ks : coefficient of horizontal shear ground reaction at well bottom (kN/m³)

Example of P15 Foundation

Bv = D_0 = 1.2m

α・Eo (ordinary) = 84000, α・Eo (seismic) = 168000

	Ordinary	Seismic
Coefficient of vertical ground reaction Kv (kN/m3)	98995	197990
Coefficient of horizontal shear ground reaction Ks (kN/m3)	29698	59397

(5) Spring Constants at Well Bottom

Spring constants at well bottom can be defined as;

a) Vertical spring constant

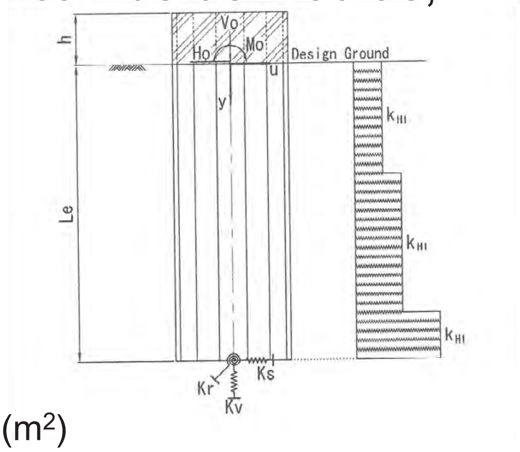
$$Kv = \sum_{i=1}^{3} (ni \cdot kvi \cdot A1i) \quad (kN/m)$$

b) Horizontal spring constant

$$Ks = \sum_{i=1}^{2} (ni \cdot ksi \cdot A1i) \quad (kN/m)$$

c) Rotational spring constant

$$Kr = \sum_{i=1}^{2} (kvi \cdot A1i \cdot IBi) \quad (kN.m/rad)$$



Where,

Ali : closed area of steel pipe sheet piles (m^2)

IB : Integration of square of the distance from centroid to neutral axis of well part in horizontal section for steel pipe sheet pile (m^2)

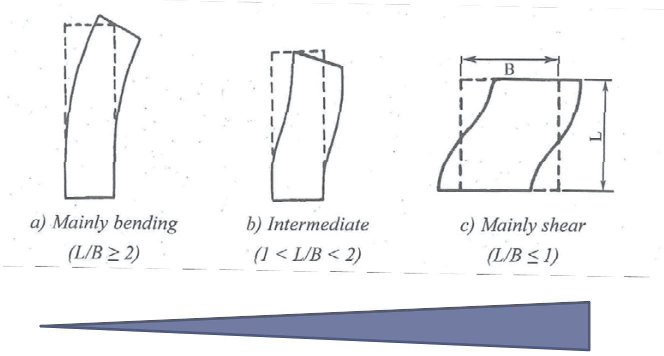
Example of P15 Foundation

		Ordinary	Seismic
Vertical spring constant Kv(kN/m)		4.0306E+006	8.0612E+006
Horizontal spring constant Ks (kN/m)		1.2092E+006	2.4184E+006
Rotational spring constant Kr (kN.m/rad)	Longitudinal Direction	5.9178E+00	1.1836E+008
	Transverse Direction	1.0121E+008	2.0242E+008

2. Structural Stability

2.1 Model for Structural Stability Analysis

- In SPSP foundation, a slippage deformation occurs at the joins, and it behaves intermediate characteristic between pile and caisson foundation.
- Three types deformation modes depend on ratio of foundation length (L) and foundation width (B).

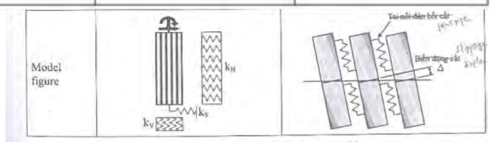


Small shear slippage deformation Large

•Applied Method of foundation design depends on the width of the foundation (B), the length and width ratio (L/B), and multiplication of the characteristic values of the foundation with the effective penetration length of the foundation (βLe).

- For ordinary structure and scale, an influence of slippage deformation is evaluated by composite efficiency.
- If outside the scope of the ordinary structure and scale, shear stiffness of joints is taken account.

		Verification for ordinary conditions, storm and Level 1 earthquake conditions	
		$B \leqq 30m$, $L/B > 1$ and $\beta L_e > 1$	$B > 30m$, $L/B \leqq 1$ or $\beta L_e \leqq 1$
Design model		Finite-length beam on an elastic floor (Beam Model)	Analysis by an imaginary well beam that considering shear slippage of the joints (Well Model)
Foundation body	Steel pipe sheet pile	Linear	
	Shear resistance of joint	Evaluation by composite efficiency and moment distribution factor	Bilinear
Ground resistance element	Horizontal ground resistance at the foundation front face	Linear considering strain dependency	
	Horizontal shear ground resistance at the foundation peripheral faces	Included in the horizontal resistance of the front ground	
	Vertical shear ground resistance at the foundation outer and inner peripheral faces	Included in the bearing capacity of the steel pipe sheet pile	
	Vertical ground resistance at the foundation bottom face	Linear	Linear
	Horizontal shear ground resistance at the foundation bottom faces	Linear	Linear



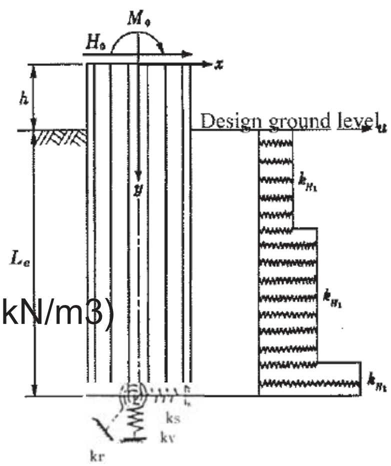
2. Structural Stability

2.2 Sectional Force and Displacement

The sectional force and displacement can be calculated by modeling SPSP as an finite length beam on an elastic floor (ground).

It may be derived from equation of equilibrium;

$$E_S I_z \frac{d^4 u_0}{dy^4} + k_{H1} D u_0 = 0$$



Where;

U0 : Displacement of foundation body (m)

y : Depth from design ground level (m)

KH1 : Coefficient of horizontal ground reaction (kN/m3)

D : width of foundation in front direction (m)

E_sI_z is a flexural rigidity of SPSP and can be derived from ;

$$E_S I_z = E_S \left(\sum_{i=1}^{n_1+n_2} I_{0i} + \mu \sum_{i=1}^{n_1+n_2} A_{0i} x_i^2 \right)$$

where

- E_SI_z: Flexural rigidity of steel pipe sheet pile foundation ($kN \cdot m^2$)
- E_S: Young's modulus of steel pipe sheet pile foundation (kN/m^2)
- I_z: Second moment of area of steel pipe sheet pile foundation (m^4)
- A_{0i}: Net cross-sectional area of i-th steel pipe body (m^2)
- I_{0i}: Second moment of area of i-th steel pipe sheet pile and inner single pile (m^4)
- μ: Composite efficiency (= 0.75)
- n_1: Number of steel pipe sheet piles at periphery of well part (pile)
- n_2: Number of steel pipe sheet piles in bulkhead (pile)
- x_i: Distance from centroid of i-th steel pipe sheet pile and inner single pile to neutral axis in horizontal section of foundation (m)

Example of P15 Foundation

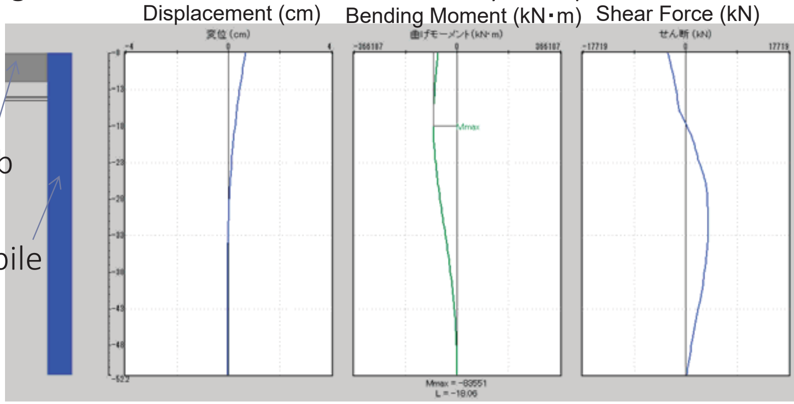
Sectional force and displacement generated on SPSP are to be obtained with model of "an finite length of beam on an elastic floor" by computation.

Displacement (cm) Bending Moment (kN･m) Shear Force (kN)

- Ordinary case (Dead+Live+Temperature)

Top slab

Sheet pile

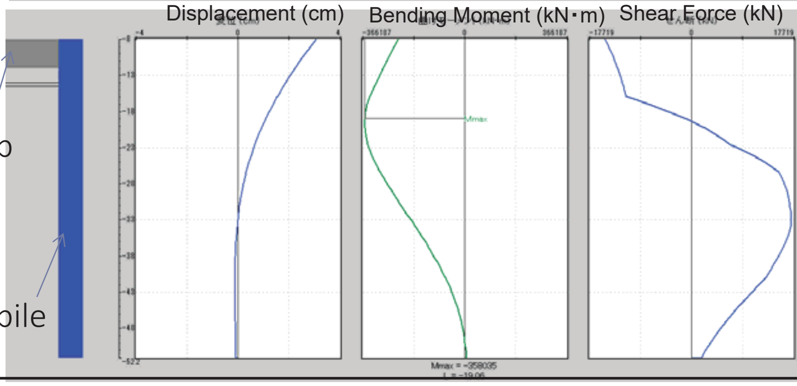


- Seismic case

Displacement (cm) Bending Moment (kN･m) Shear Force (kN)

Top slab

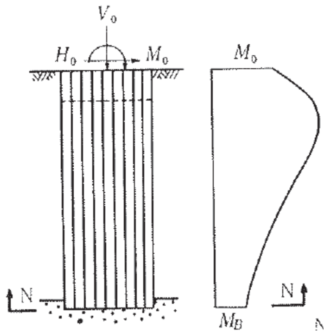
Sheet pile



2. Structural Stability

2.3 Reaction Force

The vertical reaction force of each SPSP at foundation bottom can be derived from ;



$$R = \frac{V_o \times A_{0i}}{n_1 \times A_{o1} + n_2 \times A_{o2}} \pm \frac{(MB \times A_{0i})}{\sum^{n_1}(x_i^2 \times A_{01}) + \sum^{n_2}(x_i^2 \times A_{02})} x_i$$

R: Vertical Reaction (kN/pile)

Vo: Vertical Force (kN)

MB: Bending Moment at foundation bottom (kN.m)

Aoi: net cross sectional area of pile to be verified (m_2)

$Ao_{1,2}$: net cross sectional area (m^2) (A_{01}: outer, A_{02}:bulkhead)

$n_{1,2}$: number of steel pipe sheet piles (nos.) (n_1:outer, n_2:bulkhead)

xi: Distance from centroid of each steel pipe sheet pile to neutral axis in horizontal section of foundation (m)

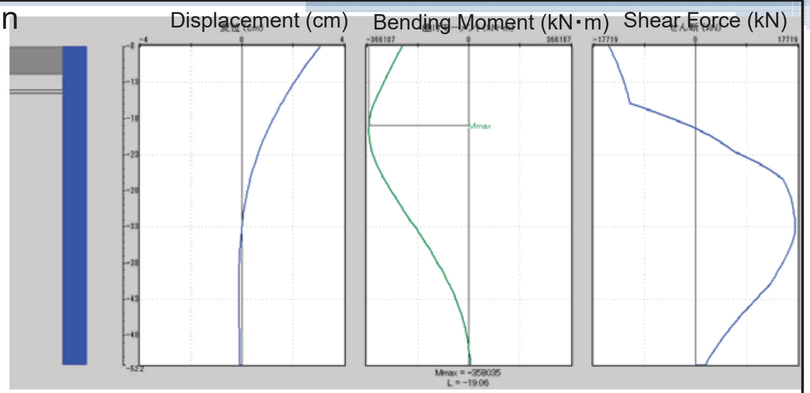
Example of P15 Foundation load case (Seismic)

- External force

  Vo = 54,800 (kN)

  Ho = 14,900 (kN)

  Mo = 237,500 (kN.m)



- Bending Moment and Vertical Reaction

  Maximum bending moment = -358,035 (kN.m)

  Bending moment at bottom = 6,345 (kN.m)

  Maximum vertical reaction R_{max} = 1,581 (kN)

  Minimum vertical reaction R_{min} = 1,459(kN)

- Verification of axial force and displacement

  R_{max} = 1,581 (kN) ≦ Ra = 3,011 (kN) : OK (compression)

  R_{min} = 1,459 (kN) ≧ Ra = -1,566 (kN) : OK (pull-out)

  δ = 3.05 (cm) ≦ δa = 5.0 (cm) : OK

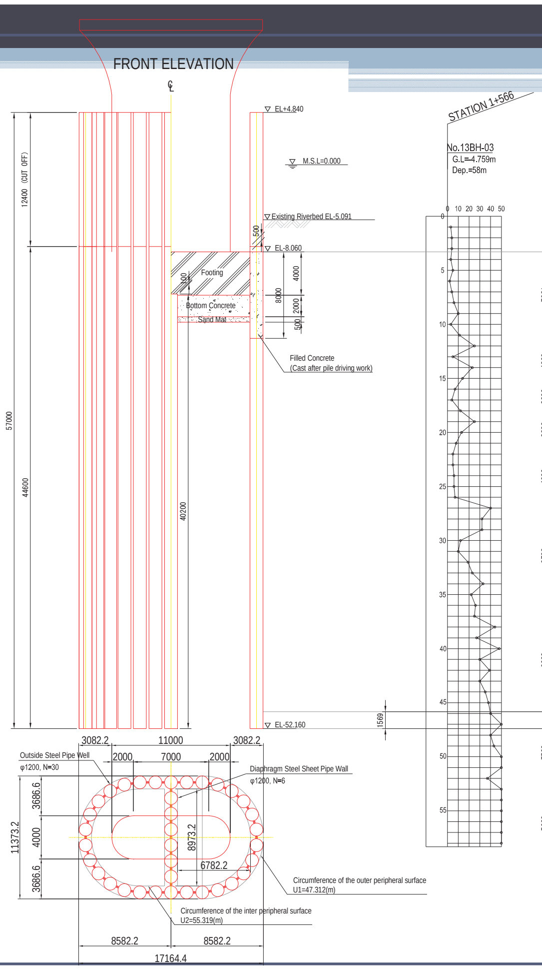
Contents of Next Lecture (5)

- Design of SPSP -

2. Design of SPSP

1. Design Condition
2. Structural Stability
3. Cofferdam and Temporary Supports
4. Verification of Combined Stress
5. Design of Top Slab and Connection

Exercise1: Calculation of Allowable Bearing Capacity



Soil Name	Average N-Value	Unit Weight γ_{sat} (kN/m³)	Cohesion C_u (kN/m²)	Friction Angle φ (°)	Modulus of Elasticity E_0 (kN/m²)	Liquefaction Coefficient D_E
Silty Sand River Sediments	3	18	0	29	1,200	Liquefaction D_E = 1/3
Silty Sand-I	13	18.0	0	33	5,200	Liquefaction D_E =1
Clay-AII	7	17.5	42	0	4,900	-
Clayey Sand-B	13	18.0	0	32	9,100	-
Clay-AII	7	17.5	42	0	4,900	-
Clayey Sand-C	20	18.0	0	33	14,000	-
Silty Sand-II	30	18.0	0	34	21,000	-
Clayey-AIV	30	18.0	180	0	21,000	-
Clayey Sand-II	50	20.0	0	34	35,000	-

A_1 : 1.131(m^2)
n_1 : 30
n_2 : 6
U_1 : 47.312(m)
U_2 : 55.319(m)
L_0 : 6.782(m)

Exercise1: Calculation of Allowable Bearing Capacity

1. Axial Compressive Allowable Bearing Capacity

$$Ra = \frac{1}{n} \cdot Ru$$

$$Ru = qd \cdot A1 + \frac{1}{n1+n2} \times \{ U1 \cdot \Sigma (Li \cdot fi) + U2 \cdot \Sigma (Lj \cdot fj) \}$$

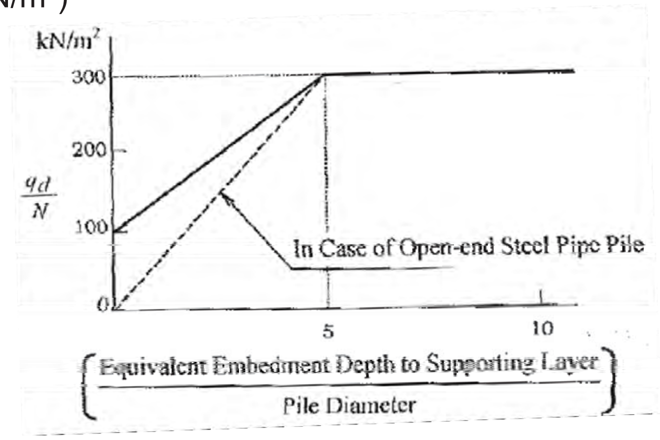
1) End Bearing Capacity

$$\frac{\text{Equivalent embedment depth to supporting layer}}{\text{Pile diameter}} = \frac{1.569\text{m}}{1.2\text{m}} = 1.31$$

qd / N = 78 N = 30 A1 = 1.131 (m^2)

=> qd = 78 * 30 = 2355 (kN/m^2)

∴ qd x A1= 2,663kN



Exercise1: Calculation of Allowable Bearing Capacity

2) Skin Friction Capacity

Skin friction force at the outer peripheral surface

Layer No	Soil type	Average N-value	Depth Li (m)	fi (kN/m^2)	DEi	Li・fi (kN/m)	
				Ordinary		Ordinary	Earthquake
1	Sandy	3.0	7.931	0.0	1/3	0.0	0.0
2	Sandy	13.0	4.300	26.0	1	111.8	111.8
3	Cohesive	7.0	2.300	70.0	1	161.0	161.0
4	Sandy	13.0	3.800	26.0	1	98.8	98.8
5	Cohesive	7.0	4.900	70.0	1	343.0	343.0
6	Sandy	20.0	9.700	40.0	1	388.0	388.0
7	Sandy	30.0	9.600	60.0	1	576.0	576.0
8	Cohesive	30.0	1.569	150.0	1	235.4	235.4
Sum			44.1			1913.9	1913.9

Skin friction force at the inner peripheral surface

Short side length : Lo = 6.782 (m)

Layer No	Soil type	Average N-value	Depth Li (m)	fi (kN/m^2)	DEi	Li・fi (kN/m)	
				Ordinary		Ordinary	Earthquake
1	Sandy	30.0	5.213	60.0	1	312.8	312.8
2	Cohesive	30.0	1.569	150.0	1	235.4	235.4
Sum			6.782			548.1	548.1

$$\therefore \frac{1}{n1+n2} \times \{ U1 \cdot \Sigma (Li \cdot fi) + U2 \cdot \Sigma (Lj \cdot fj) \}$$

= 1/(30+6) x (47.312 x 1913.9 + 55.319 x 548.1)

= 3358 (kN/pile) in ordinary, seismic condition

Exercise1: Calculation of Allowable Bearing Capacity

3) Ultimate bearing capacity of pile

Ru= 2633 + 3358 = 6021 (kN/pile)

4) Allowable bearing capacity of pile

Ordinary Ra = (1 / 3) · 6021 = 2,007 (kN/pile)

Seismic Ra = (1 / 2) · 6021 = 3,011(kN/pile)

2. Pull-out Allowable Bearing Capacity

1) Ultimate bearing capacity of pile

$$Pa = \frac{1}{n} \cdot Pu + W$$

$$Pu = \frac{1}{n1+n2} \times \left\{ U1 \cdot \Sigma (Li \cdot fi) + U2 \cdot \Sigma (Lj \cdot fj) \right\}$$

Pu=3,358kN/pile in ordinary, seismic condition

2) Allowable bearing capacity of pile

W : effective weight of steel pipe sheet pile W (= w1+w2+w3+w4)

w1 :Weight of steel pipe: 139.8kN   w2 :Weight of joint: neglect

w3: Weight of soil in pipe:307.0kN   w4: Weight of filling concrete: neglect

Ordinary Ra = (1 / 6) · 3358 + 447 = 1,006 (kN/pile)

Seismic Ra = (1 / 3) · 3358 + 447 = 1,566 (kN/pile)

Detailed Design on Bago River Bridge Construction Project

JICA Study Team

Design Technology Transfer
Lecture : Substructure and Foundation Design

30th November 2017: Group A
1st December 2017: Group B

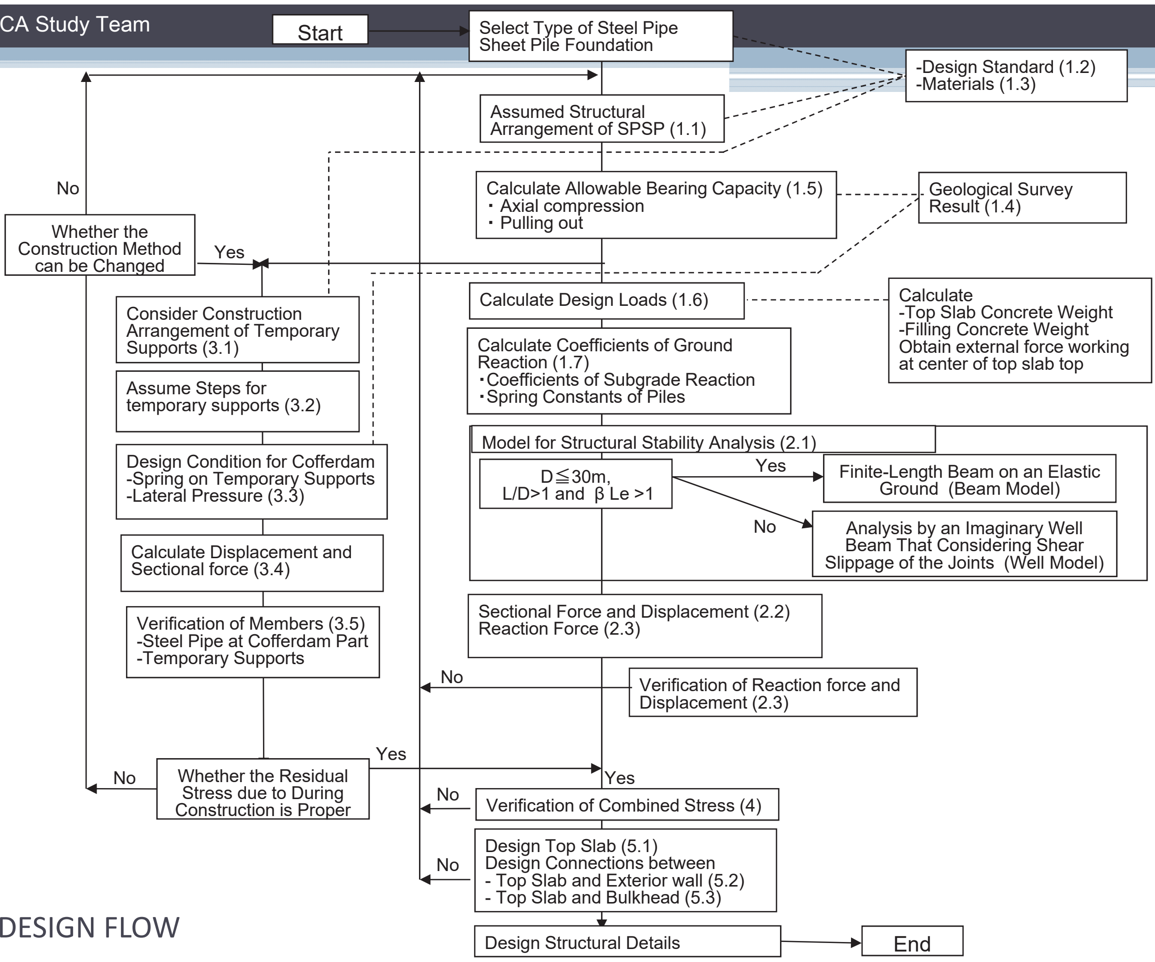
Contents of Lecture (5)

- Design of SPSP -

2. Design of SPSP

1. Design Condition
2. Structural Stability
3. Cofferdam and Temporary Supports
4. Verification of Combined Stress
5. Design of Top Slab and Connection





DESIGN FLOW

1.5 Allowable Bearing Capacity (review)

(1) Axial compressive allowable bearing capacity

$$Ra = \frac{1}{n} \cdot Ru$$

n : safety factor 3.0 (ordinary)
2.0 (seismic)

R_u : ultimate bearing capacity of steel pipe sheet pile (kN/pile)

$$Ru = \underbrace{qd \cdot A1}_{\text{End bearing}} + \frac{1}{n1+n2} \times \underbrace{\left\{ \underbrace{U1 \cdot \Sigma (Li \cdot fi)}_{\text{Outer peripheral surface}} + \underbrace{U2 \cdot \Sigma (Lj \cdot fj)}_{\text{Inner peripheral surface}} \right\}}_{\text{skin friction force}}$$

(2) Pull-out allowable bearing capacity

$$Pa = \frac{1}{n} \cdot Pu + W$$

n : safety factor 6.0 (ordinary), 3.0 (seismic)

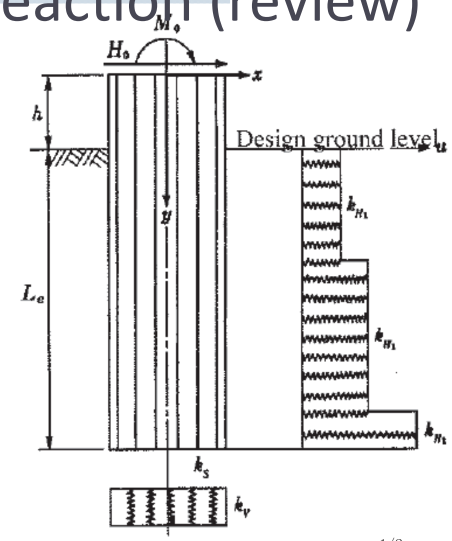
Pu : ultimate pull-out force of a steel pipe sheet pile (kN/pile)

$$Pu = \frac{1}{n1+n2} \times \left\{ U1 \cdot \Sigma (Li \cdot fi) + U2 \cdot \Sigma (Lj \cdot fj) \right\}$$

1.7 Coefficients of Ground Reaction (review)

(1)General

A coefficient of ground reaction is one of the basic design constants necessary for finding displacement and subgrade reaction of a foundation.



Coefficient of ground includes;

- Coefficient of horizontal ground reaction at a front side: K_{H1}
- Coefficient of vertical ground reaction at the bottom : K_V
- Coefficient of horizontal shear ground reaction at the bottom K_S

$$kH1 = (1 + \alpha H) \cdot kH \cdot \left(\frac{y}{yo}\right)^{-1/2}$$

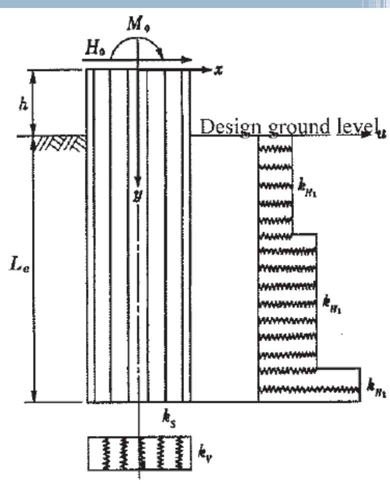
$$kv = \frac{1}{0.3} \cdot \alpha \cdot Eo \cdot \left(\frac{Bv}{0.3}\right)^{-3/4}$$

$$ks = 0.3 \cdot kv$$

2. Structural Stability (review)

2.2 Sectional Force and Displacement

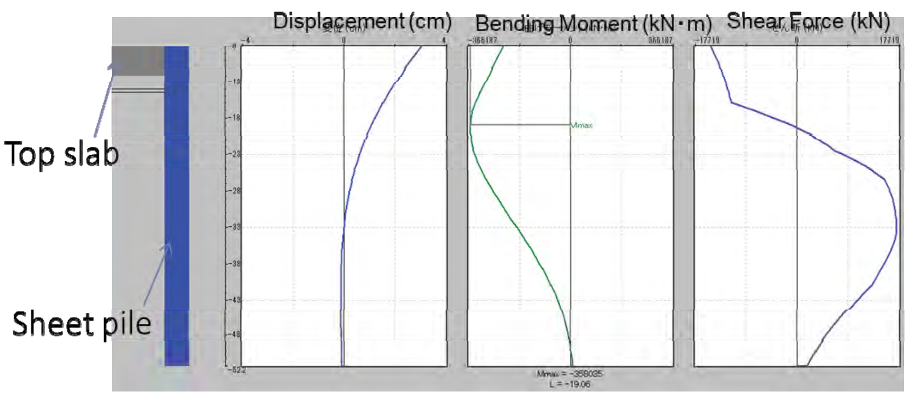
$$E_S I_z \frac{d^4 u_0}{dy^4} + k_{H1} D u_0 = 0$$



Where;

U0 : Displacement of foundation body (m)
y : Depth from design ground level (m)
KH1 : Coefficient of horizontal ground reaction (kN/m3)
D : width of foundation in front direction (m)

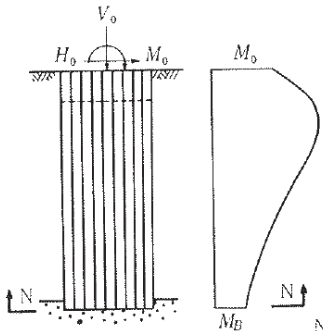
- Seismic case



2. Structural Stability (review)

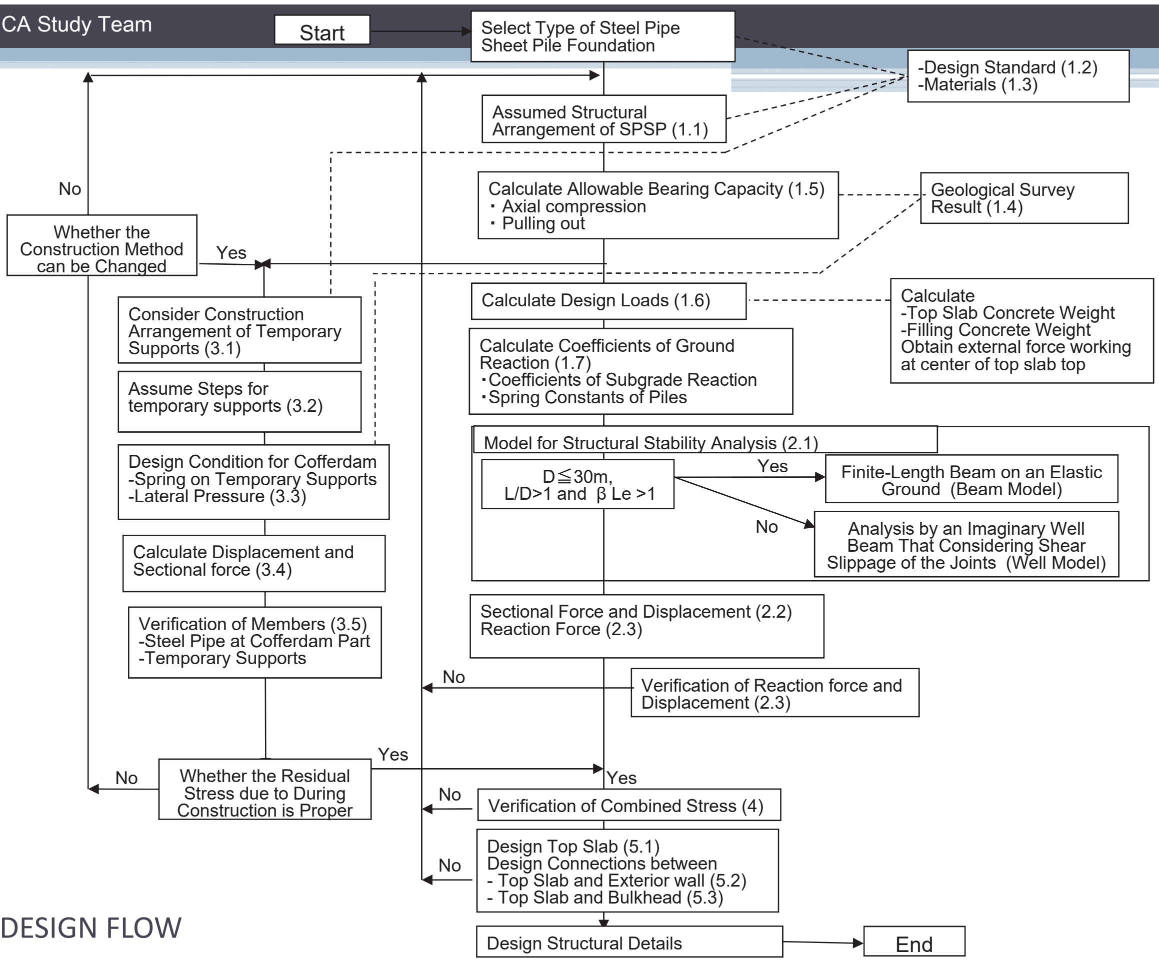
2.3 Reaction Force

$$R = \frac{V_o \times A_{0i}}{n_1 \times A_{o1} + n_2 \times A_{o2}} \pm \frac{(MB \times A_{0i})}{\sum^{n_1}(x_i^2 \times A_{01}) + \sum^{n_2}(x_i^2 \times A_{02})} x_i$$



- Verification of axial force and displacement

$R_{max} \leqq Ra$ (compression)
$R_{min} \geqq Ra$ (pull-out)
$\delta_{\text{at design ground level}} \leqq \delta a = 5.0$ (cm)

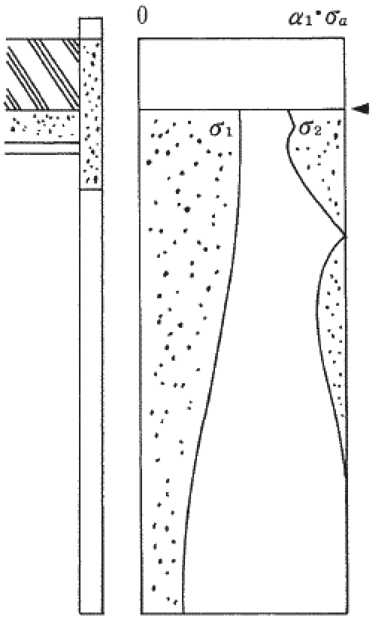


DESIGN FLOW

4 Verification of Combined Stress

4.1 Stress of SPSP due to loads during operation

Permanent portion of pipes shall be verified against the sectional forces caused by combination of loads during construction and after completion.



◄Bottom of Footing (top slab)

$\sigma_1 + \sigma_2 = \sigma_{max} \leqq \sigma_a$

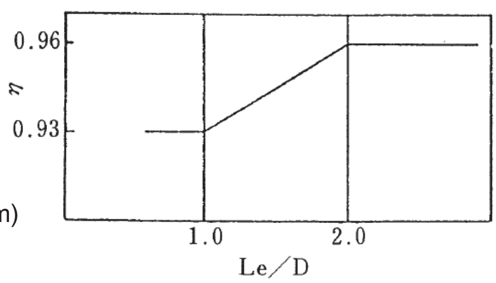
- σ_1 : stress due to after completion loads
- σ_2 : residual stress due to during construction
- σ_{max} : combined stress ($\sigma_1 + \sigma_2$)
- σ_a : allowable stress in steel pipe sheet pile

4 Verification of Combined Stress

4.1 Axial Stress of SPSP during operation

$$\sigma_{yL,i} = \frac{V_o}{n_1 \times A_{o1} + n_2 \times A_{o2}} \pm M_y \left[\frac{\eta}{\sum^{n_1}(x_i^2 \times A_{01}) + \sum^{n_2}(x_i^2 \times A_{02})} x_i + \frac{1-\eta}{n_1 \times Z_{01} + n_2 \times Z_{02}} \right]$$

$\sigma_{yL,i}$: Axial stress in i-th SPSP at depth y from design ground level (kN/m^2)
Vo: Vertical Force (kN)
My: Bending Moment occurring in SPSP at depth y from design ground level (kN.m)
$Ao_{1,2}$: net cross sectional area (m^2) (A_{01}: outer, A_{02}:bulkhead)
$n_{1,2}$: number of steel pipe sheet piles (nos.) (n_1:outer, n_2:bulkhead)
$Zo_{1,2}$: modulus of section per steel pipe sheet pile (m^3) (Z_{01}: outer, Z_{02}:bulkhead)
xi: Distance from centroid of each steel pipe sheet pile to neutral axis in horizontal section of foundation (m)
η: Distribution ratio of bending moment (0.93～0.96)



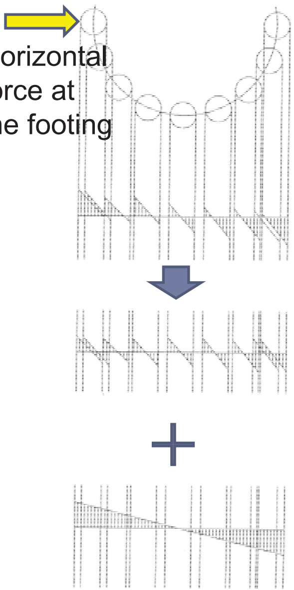
D: loading width orthogonal to loading direction (m)
Le: effective embedment depth of foundation (m)

4 Verification of Combined Stress

4.1 Axial Stress of SPSP during operation

$$\sigma_{yL,i} = \frac{V_o}{n_1 \times A_{o1} + n_2 \times A_{o2}} \pm M_y \left\{ \underbrace{\frac{\eta}{\sum^{n_1}(x_i^2 \times A_{01}) + \sum^{n_2}(x_i^2 \times A_{02})} x_i}_{\text{Bending moment due to the well body}} + \underbrace{\frac{1-\eta}{n_1 \times Z_{01} + n_2 \times Z_{02}}}_{\text{Bending moment due to the each pile}} \right\}$$

η: Distribution ratio of bending moment

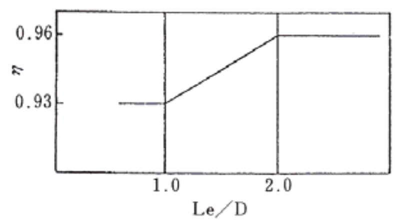


Horizontal force at the footing

Actual distribution of deformation — η:100%

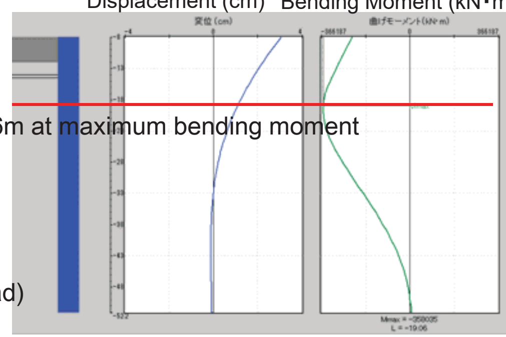
Distribution of deformation of each steel pile — η:4%～7%

Distribution of deformation of the well foundation — η:93%～96%



D: loading width orthogonal to loading direction (m)
Le: effective embedment depth of foundation (m)

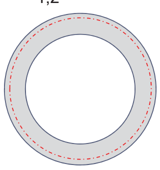
Exercise1: Calculation of Axial Stress in SPSP



Example of P15 Foundation
load case (Seismic)

- External force
  - Vo = 54,704 (kN)
  - Ho = 14,900 (kN)
  - Mo = 237,500 (kN.m)
- Maximum bending moment = -358,035 (kN.m)

- $Ao_{1,2}$: net cross sectional area (m^2) (A_{01}: outer, A_{02}:bulkhead)



Do=1.2m, thickness=16mm, contingency for corrosion in future=2mm

$Ao_1 = Ao_2 =$ [   ] m^2

$Zo_{1,2}$: modulus of section per steel pipe sheet pile (m^3) (Z_{01}: outer, Z_{02}:bulkhead)

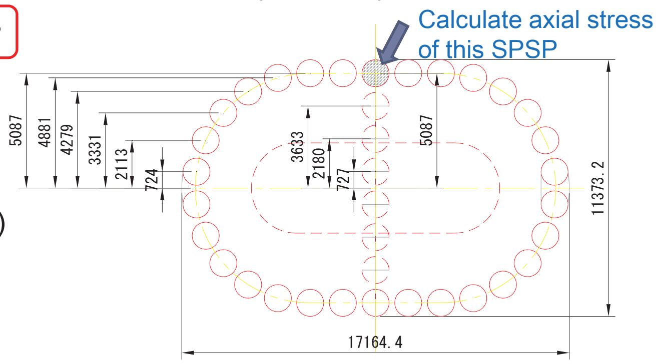
$Zo_1 = Zo_2 =$ [   ] m^3

- $\sum^{n_1} x_i^2$: for outer pipes = [   ] m^2

  $\sum^{n_2} x_i^2$: for bulkhead pipes = [   ] m^2

- $n_{1,2}$: number of steel pipe sheet piles (nos.) (n_1:outer: 30, n_2:bulkhead 6)
- η: Distribution ratio of bending moment=

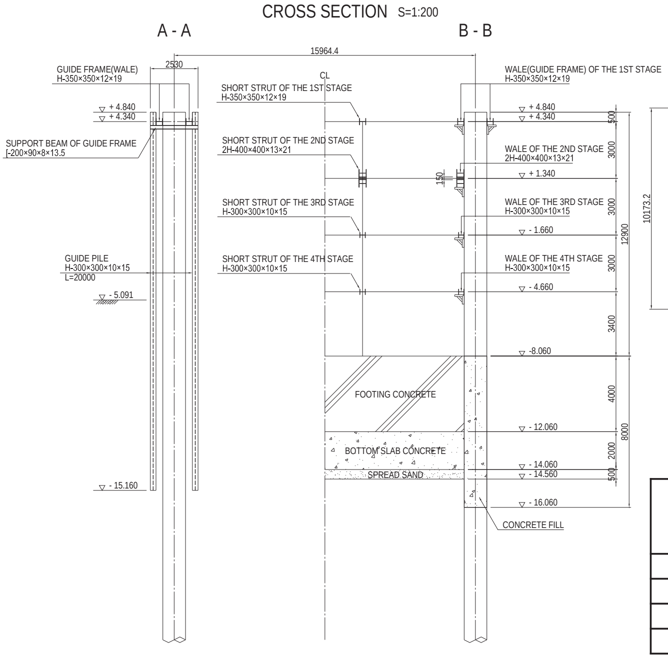
[   ] (D: 17.1644(m), Le: 44.10 (m))

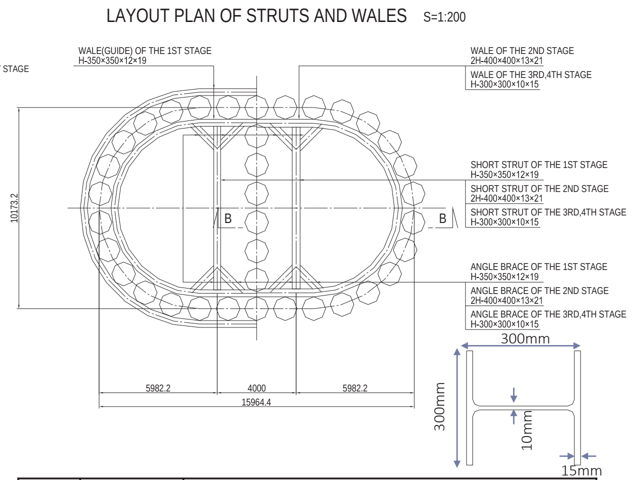


3. Design of Temporary Cofferdam

3.1 ARRANGEMENT OF TEMPORARY SUPPORTS

The cofferdam shall be verified to be safe against the loads acting during temporary works.



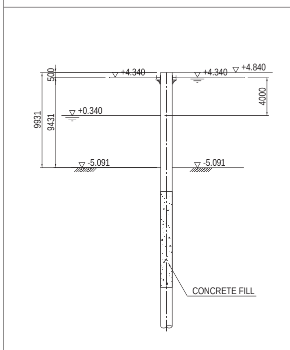


No.	Elevation (m)	H-section steel	
		Wale circular and straight part	Strut and Angle Brace
1	+4.340	H-350*350*12*19	H-350*350*12*19
2	+1.340	2 x H-400*400*13*21	H-400*400*13*21
3	-1.660	H-300*300*10*15	H-300*300*10*15
4	-4.660	H-300*300*10*15	H-300*300*10*15

Example of temporary supports arrangement (P15)

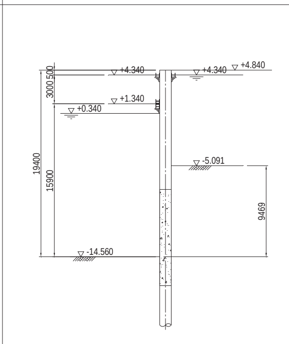
3.2 Construction Step

STEP 1



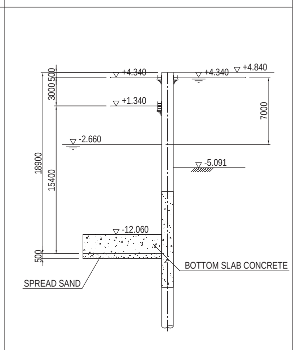
Excavate inside of exterior sheet pilles and filled with concrete as shown.
draining the inside of cofferdam up to +0.340m level.
the 1st support Installation.

STEP 2



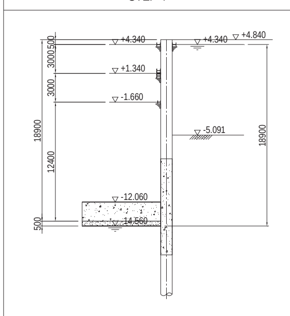
The 2nd support Installation.
underwater excavation up to -14.560m level.

STEP 3



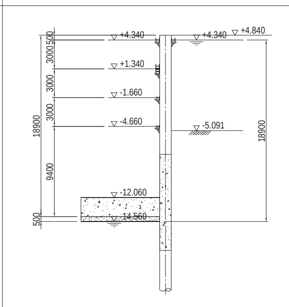
Draining the inside of cofferdam up to -2.660m level.
Placement of spread sand followed by
Casting undeawater bottom slab concrete.

STEP 4



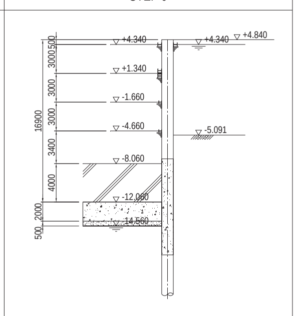
The 3rd support Installation.
Dry up inside the cofferdam.

STEP 5



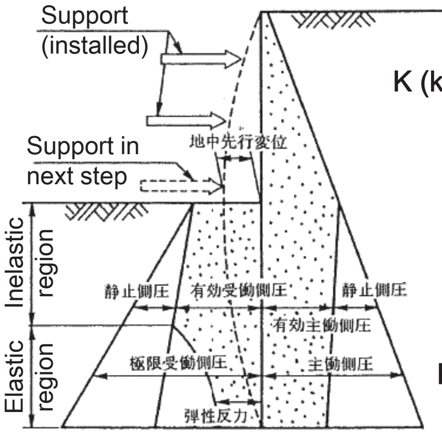
The 4th support Installation.

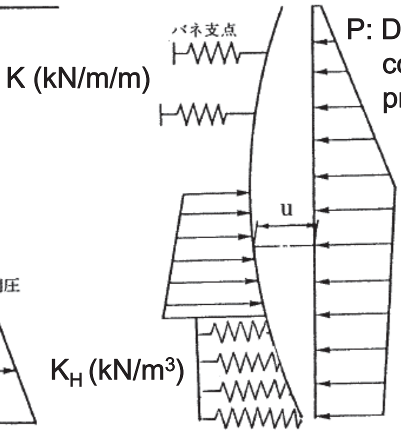
STEP 6



Casting of footing concrete.

3.3 Design Conditions for Temporary Cofferdam Design





P: Distributed Loads (lateral pressure consisting earth pressure and water pressure) (kN/m)

Analysis Method (Basic Equation)

Over the excavation surface and the inelastic region:

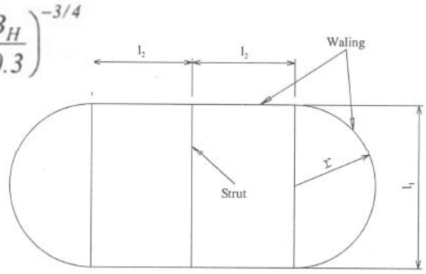
$$E_s I \frac{d^4 u}{dy^4} + p = 0$$

Elastic region:

$$E_s I \frac{d^4 u}{dy^4} + k_H D' u + p = 0$$

Subgrade reaction modulus(K_H)

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



Spring constant of the support (K)

1) Arc section

$$K = \frac{E_s A_1}{r^2} \ (kN/m/m)$$

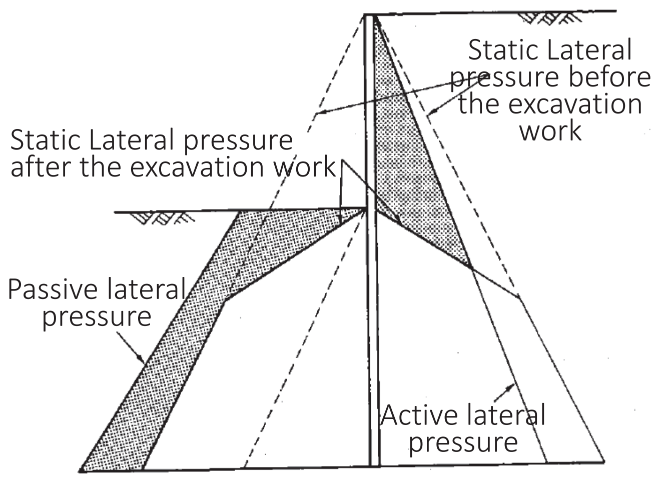
2) Liner section

$$K = \frac{E_s A_2}{l_1/2} \cdot \frac{1}{l_2} = \frac{2E_s A_2}{l_1 l_2} \cdot (kN/m/m)$$

3) The spring constant of the bottom slab concrete

$$K = \alpha \frac{E_c A_c}{B/2} \ (kN/m)$$

3.3 Design Conditions for Temporary Cofferdam Design



(i) Above the excavation base, “active lateral pressure” acts from the back side.

(ii) Below the excavation base, from the back side, “<u>effective active lateral pressure</u>” acts. From the excavation side, “<u>effective passive lateral pressure</u>” acts on the inelastic region. Also, “elastic reaction” which is proportional to the deformation of the wall acts on the elastic region.

<u>Effective active lateral pressure</u> :

「Active lateral pressure from the back side」 ― 「Static Lateral pressure from the excavation side」

<u>Effective passive lateral pressure</u> :

「Passive lateral pressure from the excavation side」 ― 「Static Lateral pressure from excavation side」

3.3 Design Conditions for Temporary Cofferdam Design

For Reference

(1) Active Lateral Pressure

Sandy soil

$$Pa = Ka(\gamma \cdot h - pw1 + q) - 2c \cdot \sqrt{Ka} + pw1$$

Cohesive soil

h≦H $\quad Pa = Ka1(\gamma \cdot h + q)$

h＞H $\quad Pa = Ka1(\gamma \cdot H + q) + Ka2 \cdot \gamma(h - H)$

Ka: coefficient of active earth pressure for sandy soil

ka1, ka2 : coefficient of active earth pressure for cohesive soil

pw1, pw2 : hydrostatic pressure at depth h (kN/m^2)

c : cohesion of soil (kN/m^2)

(2) Passive Lateral Pressure

Sandy soil

$$Pp = Kp(\gamma \cdot h - pw2 + q) + 2c \cdot \sqrt{Kp} + pw2$$

Cohesive soil

$$Pp = Kp(\gamma \cdot h + q) + 2c \cdot \sqrt{Kp}$$

kp : coefficient of passive earth pressure

(3) Static Lateral Pressure

1) Before excavation work

Sandy soil

$$Po = Ko(\gamma \cdot h - pw1 + q) + pw1$$

Cohesive soil

$$Po = ko(\gamma \cdot h + q)$$

2) After excavation work

Sandy soil

$$Po' = Ko(\gamma \cdot h' - pw2 + q) + Ko \cdot \frac{f \cdot h'}{B} + pw2$$

Cohesive soil

$$Po' = Ko(\gamma \cdot h' + q) + Ko \cdot \frac{f \cdot h'}{B}$$

ko : coefficient of lateral pressure at rest

B : effective range of the friction (m) =5.0m

f : friction between steel pipe sheet pile and ground (kN/m^2)

3.4 Displacement and Sectional Force

Displacement and sectional forces worked on steel pipe sheet piles, and reaction force on each temporary supports are obtained.

Example of P15

δMax : 9.8 cm

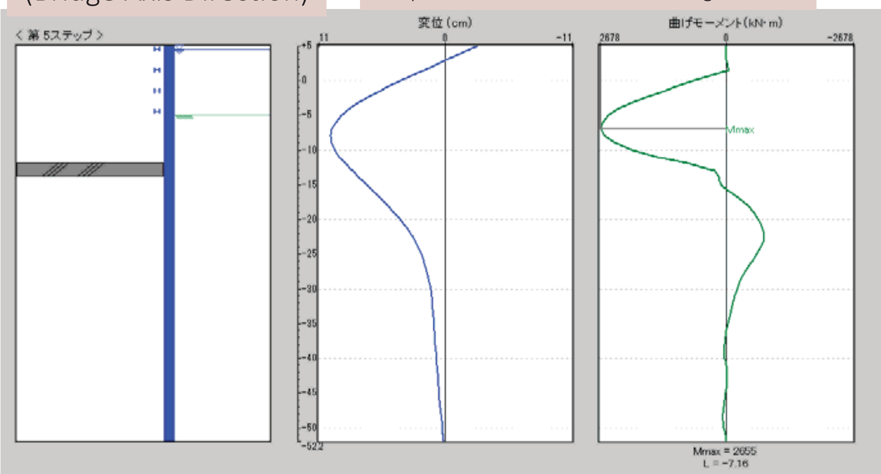
Mmax at cofferdam part : 2655 (kN.m)

Mmax at well part : -800.0 (kN.m)

Reaction force on support

5th Step
(Bridge Axis Direction)

Displacement bending moment



Stage	Reaction force (kN/m)
1	0
2	560
3	82
4	0
Bottom slab concrete	1213

3.5 Verification of Members of Steel Pipe at Cofferdam Part and Temporary Supports

1) Members of Steel Pipe at Cofferdam Part

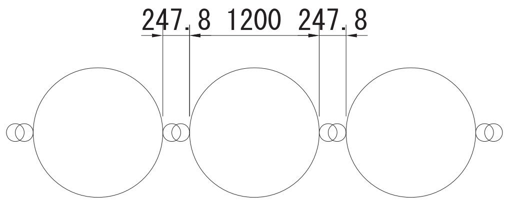
Exercise2: Calculation of Axial Stress in SPSP

M_{max} at cofferdam part : 2655 (kN.m/m)

σ = M/I' x y =M/Z'

Z': Modulus of section per meter (mm^3/m) = $\pi/32D\ (D^4-d^4)\ /\ (D+w)$

W: width of joint pipes, standard 247.8mm



Do=1.2m, thickness=14mm, no consideration of corrosion in future

Material SKY490 (σa=185N/mm^2 x 1.5 times of increase coefficient of allowable stress)

Check σ < σa ?

2) Members of Temporary Supports

(1) Design of the Support at Circular Region

✓ Stress check of wale at circular region

$$\sigma = \frac{N}{A_l} + \frac{M}{Z}$$

where,

N:Axial force (kN), which can be obtained from the equation (12.27),

$$N = Rr \qquad (12.27)$$

M: Bending moment (kN·m), which can be obtained from the equation (12.28),

$$M = R.r^2.\Delta \qquad (12.28)$$

R :Reaction of the support (kN/m),

Δ : Ovalization ratio (standard value: 2%),

r : Radius to the circular support center (m),

A_l:Cross-section area of the circular support (m^2),

Z :Section modulus of the support (m^3).

✓ Check safety against the buckling for wale at circular region

$$R_a = \frac{1}{F}\frac{3E_sI}{r^3}$$

where:

R_a :Allowable support reaction (kN/m),

E_s :Young's modulus of the circular support (kN/m^2),

I : Moment of inertia of the circular support (m^4),

F : Safety factor (1.5 may be used.).

2) Members of Temporary Supports

(2) Design of the Supports at Straight Region

Verification for members of wales and strut at straight portion

a) Buckling (1) $$\frac{\sigma c}{\sigma caz} + \frac{\sigma bcy}{\sigma bagy \cdot \left(1 - \frac{\sigma c}{\sigma eay}\right)} \leqq 1.0$$

$$\sigma c = \frac{N}{A}, \quad \sigma bcy = \frac{M}{Z}$$

$$M = \frac{R2max \cdot L1^2}{8} \qquad N = R1max \cdot r + \Delta N$$

b) Buckling (2) $$\sigma c + \frac{\sigma bcy}{1 - \frac{\sigma c}{\sigma eay}} \leqq \sigma cal$$

c) Shear force $$\tau s = \frac{Smax}{Aw} \leqq \tau sa \qquad Smax = \frac{R2max \cdot L1}{2}$$

σ_{bcy} : bending compressive stress due to bending moment about strong axis (N/mm^2)
σ_{caz} : allowable axial compressive stress about weak axis (N/mm^2)
σ_{bagy} : allowable bending compressive stress without local buckling effect about strong axis (N/mm^2)
σ_{eay} : Euler buckling stress about strong axis (N/mm^2)
σ_{cal} : allowable compressive stress for local buckling of outstanding plate (N/mm^2)
$R1_{max}$,$R2_{max}$: Maximum reaction force on temporary supports (N/mm)
⊿N : Stress induced by temporary increased (kN)

4. Verification of combined stress

Exercise3: Verification of Combined Stress

Calculate combined stresses along the depth and verify with allowable stress of material SKY400. If not satisfied, consider to apply material SKY490 in required length.

After completion load

depth (m)	σ1 for Seismic (N/mm2)
9	180.2
10	175.5
11	167.7
12	156.1
13	144.3
14	132.9
15	120.6
16	110.8
17	100.3
18	90.5
19	80.4
20	70.4
21	60.2
22	50.7
23	45.1
24	38.0
25	34.2
26	33.1

During construction

depth (m)	σ2 (N/mm2)
9	3.2
10	43.2
11	56.2
12	60.2
13	49.2
14	8.2
15	26.2
16	42.2
17	59.2
18	79.2
19	87.2
20	89.2
21	86.2
22	73.2
23	48.2
24	25.2
25	7.2
26	3.2

Combined Stress

depth (m)	σmax (N/mm2)	Material SKY400 or SKY490?
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		
21		
22		
23		
24		
25		
26		

	SKY400 (N/mm^2)	SKY490 (N/mm^2)
Allowable stress (Ordinary case)	140	185
Allowable stress (Seismic case)	210	280

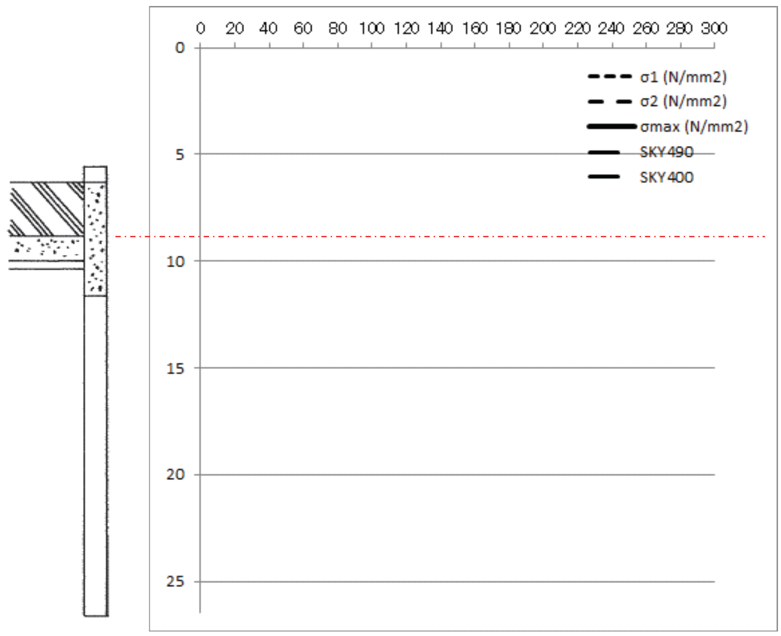
$\sigma_1 + \sigma_2 = \sigma_{max} \leqq \sigma_a$

σ_1 : stress due to after completion loads
σ_2 : residual stress due to during construction
σ_{max} : combined stress ($\sigma_1 + \sigma_2$)
σ_a : allowable stress in steel pipe sheet pile

4. Verification of combined stress

Exercise3: Verification of Combined Stress

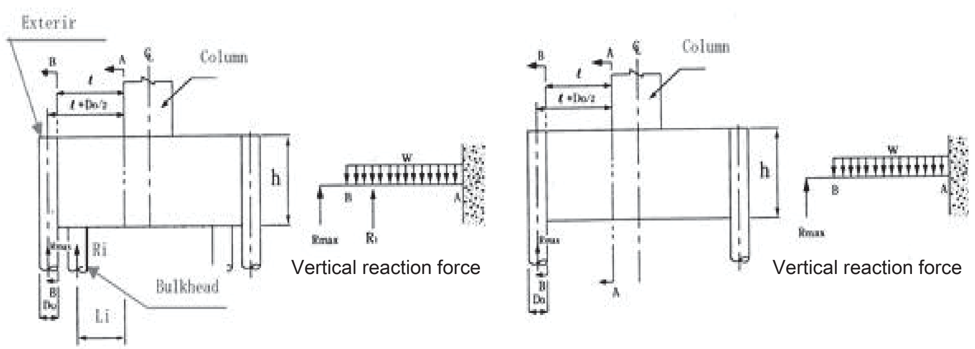
Draw stress distribution line of σ1,σ2,σmax (combined stress) along the depth and also allowable stress of material SKY400 and SKY490.



5. Design of Top Slab and Connection

5.1 TOP SLAB

A verification of the sections of footing will be made at the section A-A for bending moment and section B-B for shear force.
Sectional forces will be calculated per unit width at the position of the steel pipe sheet pile that produces the maximum vertical reaction force.



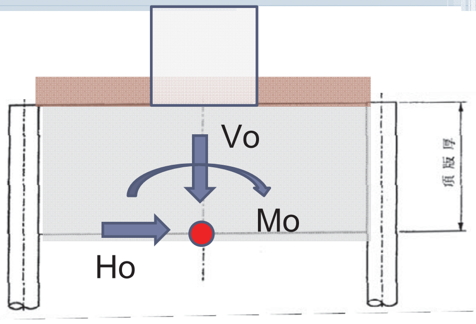
(Bridge Axis Direction) (Bridge Axis Perpendicular Direction)

Cantilever Beam Design

5. Design of Top Slab and Connection

5.1 TOP SLAB

1) Vertical Reaction Force



$$R_j = \frac{V_0 A_{01}}{n_1 A_{01} + n_2 A_{02}} \pm \frac{M_0 A_{01}}{\sum^{n1} x_i^2 A_{01} + \sum^{n2} x_i^2 A_{02}} x_j$$

where:

R_j is the vertical reaction force of i-th steel pipe sheet pile (kN);

V_0 is the vertical load acting on bottom face of top slab (kN) (including a footing and soil backfill);

M_0 is the bending moment at the base of the footing (kN.m);

x_i is the distance from centric of each steel pipe sheet pile and inner single pile to neutral axis in horizontal section of foundation (m);

n_1 is the number of the outer steel pipe sheet pile (pile);

n_2 is the number of steel pipe sheet pile of Bulkhead part (pile);

A_{01} is the section area of steel pipe sheet pile combined outside round of foundation (m^2);

A_{02} is the sectional area of steel pipe sheet pile of Bulkhead part (m2);

5. Design of Top Slab and Connection

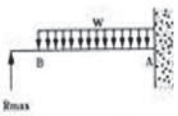
5.1 TOP SLAB

2) Sectional Force

Foundation without bulkhead piles

$$M_A = \frac{R_{max}(1 + D_0/2)}{D_0'} - \frac{1}{2} w l^2$$

$$Q_B = \frac{R_{max}}{D_0'}$$

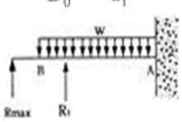


Foundation with bulkhead piles

$$M_A = \frac{R_{max}(1 + D_0/2)}{D_0'} + \sum \frac{R_i x l_i}{a_i} - \frac{1}{2} w l^2$$

$$Q_b = \frac{R_{max}}{D_0'}$$

$$Q_{Bi} = \frac{R_{max}}{D_0'} + \frac{R_i}{a_i} - w(l - h/2)$$



where:

M_A: Bending moment occurring in A-A section (kN.m/m);

Q_B: Shear force occurring in B-B section (kN/m);

Q_{Bi}: Shear force at the location from ½ thickness of footing to foundation bed (kN/m);

R_{max}: Maximum vertical reaction or minimum vertical reaction obtained in a pipe pile. (kN/pile);

R_i: vertical reaction of pipe pile combined a retaining wall, a single pile inside that is effected by cantilever beam when maximum vertical reaction or minimum occur in a pipe pile (kN/pile);

w: Dead weight of top slab and overburden load (kN/m^2);

D_0': Center-to-center interval between adjacent steel pipe sheet piles of well part (m);

D_0: Diameter of a steel pipe sheet pile (m);

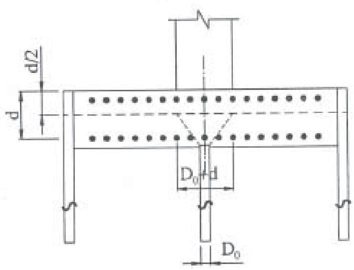
l: Distance from outer edge of lower end of body to side face of foundation (m);

l_i: Distance from outer edge of lower end of body to a center of steel pipe sheet pile of Bulkhead pile;

h: Thickness of footing (m);

$a_i = D + d$ (m);

d: Effective height of footing (m)



5. Design of Top Slab and Connection

5.1 TOP SLAB

3) Design against Bending Moment

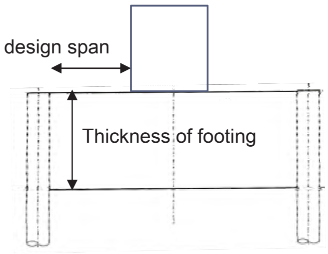
If the thickness of the footing is bigger than ½ design span, it is considered as a structure of thick beam, and the required reinforcing bar is calculated as below:

$$T = \frac{48}{27}\frac{M}{bh}$$

$$A_S = \frac{T}{\sigma sa}$$

where:

T is the tension force caused by the bending moment at the foundation (kN);
M is the bending moment (kN.m);
b is the width of the footing (1m);
h is the thichness of the footing (m);
A_s is the required reinforcement area (m2/m);
σsa is the allowable stress of steel bar (kN/m^2)



<Verification>

As ≤ Arranged reinforcement area (mm^2/m) ➔OK

5. Design of Top Slab and Connection

5.1 TOP SLAB

4) Design against Shear Force

$$\tau_m = \frac{Q}{b \times d}$$

τ_m : average shear stress

Q: shear force (kN)
b: width of footing (1m)
d: effective height of footing (m)

τ'_{a1} : Corrected Allowable Shear Stress (N/mm^2) $= \tau_{a1} \times C_{dc} \times C_e \times C_{pt}$

(N/mm^2)

Concrete Strength	21	24	27	30
τ_{a1}	0.22	0.23	0.24	0.25

Correction Factor on Allowable Shear Stress

Scale ratio of shear force of concrete related to shear span ration (a/d: design span/effective height) (C_{dc})

a/d	0,5	1,0	1,5	2,0	2,5
C_{dc}	6,4	4,0	2,5	1,6	1,0

Complement factor related to effective height (C_e)

Effective height (m)	< 1	3	5	> 10
C_e	1,0	0,7	0,6	0,5

Additional coefficient related to the tensile reinforcement ratio (Cp_t)

Ratio off main tension p_t	0,2	0,3	0,5	> 1,0
c_{pt}	0,9	1,0	1,2	1,5

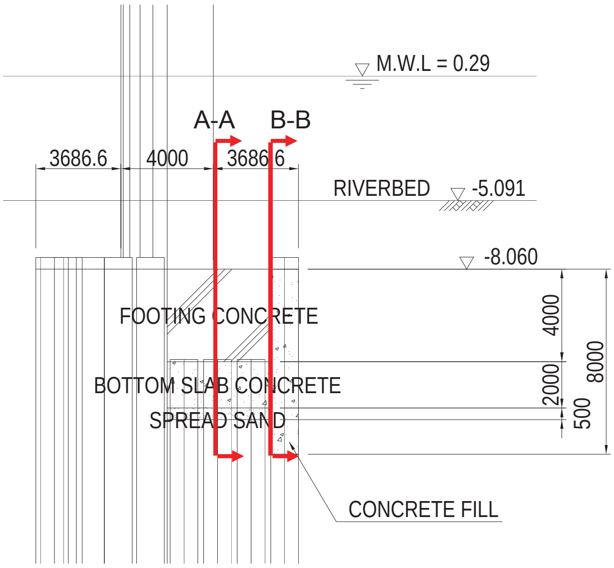
<Verification>

$\tau_m \le \tau'_{a1}$ ➔OK

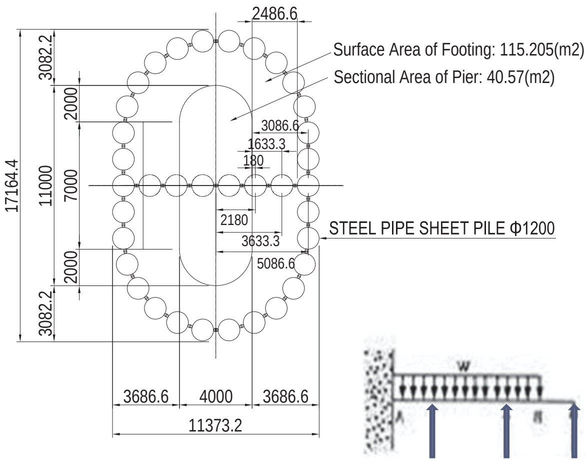
Exercise4: Design of Top Slab

Calculate following items in Seismic Condition and in Longitudinal direction:

1) External Force (V0, M0) acting on the bottom face of footing (kN)
2) Vertical Reaction Force of outer pile and bulkhead pile (kN/pile)
3) Sectional Force, Bending Moment (kN.m/m) at A-A and Shear Force (kN/m) at Section B-B
4) Required area of reinforcing bar (mm^2/m)
5) Average shear stress (N/mm^2)



Longitudinal direction



Exercise4: Design of Top Slab

1) External Force (V0, M0) acting on the bottom face of footing (kN)

Weight	Vo (kN)	Ho (kN)	M (kN.m)	M'=Ho x height of footing (kN.m)	ΣM= Mo(kN.m)
superstructure and pier column	44,300	14,900	237,500	59,600	297,100
footing		-	-	-	-
backfill soil		-	-	-	-
buoyancy on Pier column		-	-	-	-
Total		**14,900**	**-**	**-**	**297,100**

Given:

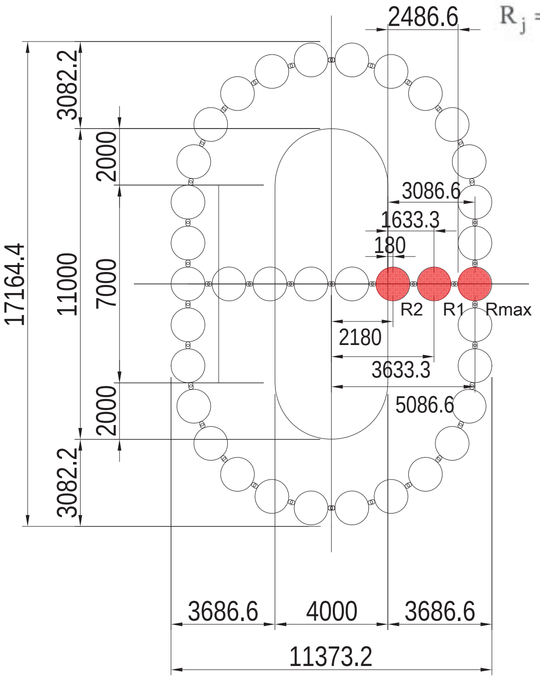
γ_c: unit weight of footing concrete $24.5kN/m^3$

γ_w: unit weight of water $10kN/m^3$

γ_{sat}: saturated unit weight of backfill soil $19kN/m^3$

Exercise4: Design of Top Slab

2) Vertical Reaction Force of outer pile and bulkhead pile (kN/pile)



$$R_j = \frac{V_o A_{01}}{n_1 A_{01} + n_2 A_{02}} \pm \frac{M_0 A_{01}}{\sum^{n1} x_i^2 A_{01} + \sum^{n2} x_i^2 A_{02}} x_j$$

Pile	Vertical Reaction Force (kN/pile)
R_{max}	
R1	
R2	

Given:

	Nos.	Net cross-sectional area of SPSP A_{01} (m^2/pile)	$\sum^{n1,2} xi^2$ (m^2)
Outer part	30	0.04464	491.6
Bulkhead part	6	0.04464	36.96

Exercise4: Design of Top Slab

3) Sectional Force, Bending Moment (kN.m/m) at A-A and Shear Force (kN/m) at Section B-B

$$M_A = \frac{R_{max}(1 + D_0/2)}{D'_0} + \sum \frac{R_i x l_i}{a_i} - \frac{1}{2} w l^2$$

$$Q_b = \frac{R_{max}}{D'_0}$$

Do:	1.2(m)	
D'o:	1.4478 (m)	Width of joint pipe 0.2478m
Rmax:	(kN)	
R1:	(kN)	
R2:	(kN)	
l1	(m)	
l2	(m)	
a1,a2	4.8338m	Effective height of footing d=3.6338m
w	(kN/m2)	
l	(m)	
M_A	(kN.m/m)	
Q_b	(kN/m)	

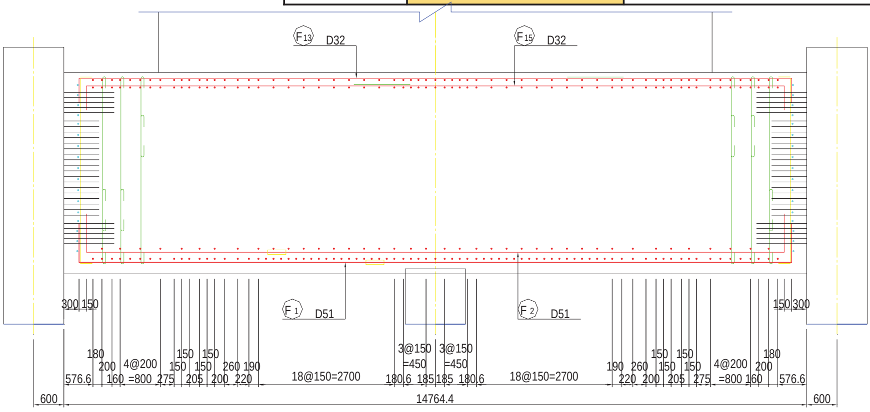
Exercise4: Design of Top Slab

4) Required area of reinforcing bar (mm^2/m) for tension force at Section A-A

$$T = \frac{48}{27}\frac{M}{bh}$$

$$A_s = \frac{T}{\sigma sa}$$

M:	(KN.m/ m)	M_A
b:	1.0(m)	unit width
h:	4.0(m)	
σsa:	300(N/mm^2)	members under water 160N/mm2 for ordinary case 200N/mm2 x increase factor 1.5 for seismic case
T:	(kN)	
As:	(cm2)	



Lower Rebar (tension side)
Rebar φ51mm
Area: 2027mm^2/rebar

1st: φ51 x 81 nos.
2nd:φ51 x 40 nos.

Total area: 2452.7cm^2
per 100cm=
2452.7/1476.44 x 100cm=
166cm^2

Exercise4: Design of Top Slab

5) Average shear stress at Section B-B

$$\tau_m = \frac{Q}{b \times d}$$

Q:	(kN)	Q_b
b:	1.0(m)	unit width
d:	3.6338(m)	
τ_m :	(N/mm^2)	
τ_{a1} :	(N/mm^2)	Concrete Strength 24N/mm2
C_{dc}	5.515	
C_e	0.668	
C_{pt}	1.156	pt=0.456
τ'_{al}	(N/mm^2)	

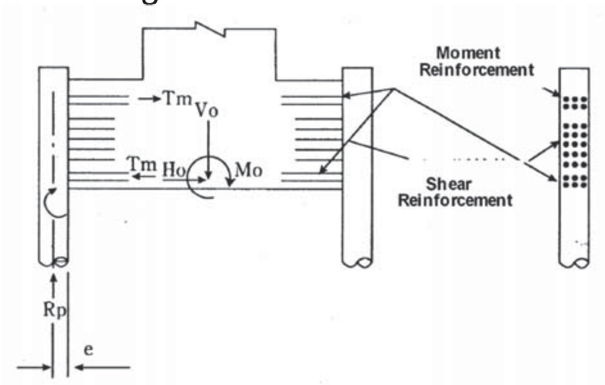
τ'_{al} : Corrected Allowable Shear Stress (N/mm^2)

$= \tau_{a1} \times C_{dc} \times C_e \times C_{pt}$

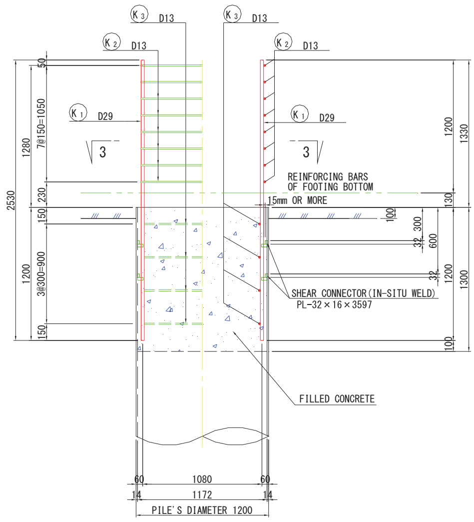
5. Design of Top Slab and Connection

continue to next time

5.2 Design of Connection between Top Slab and Exterior wall



5.3 Design of Connection between Top Slab and Bulkhead



Detailed Design on Bago River Bridge Construction Project

JICA Study Team

Design Technology Transfer
Lecture : Substructure and Foundation Design

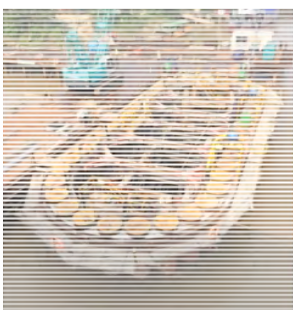
6th December 2017: Group B
7th December 2017: Group A

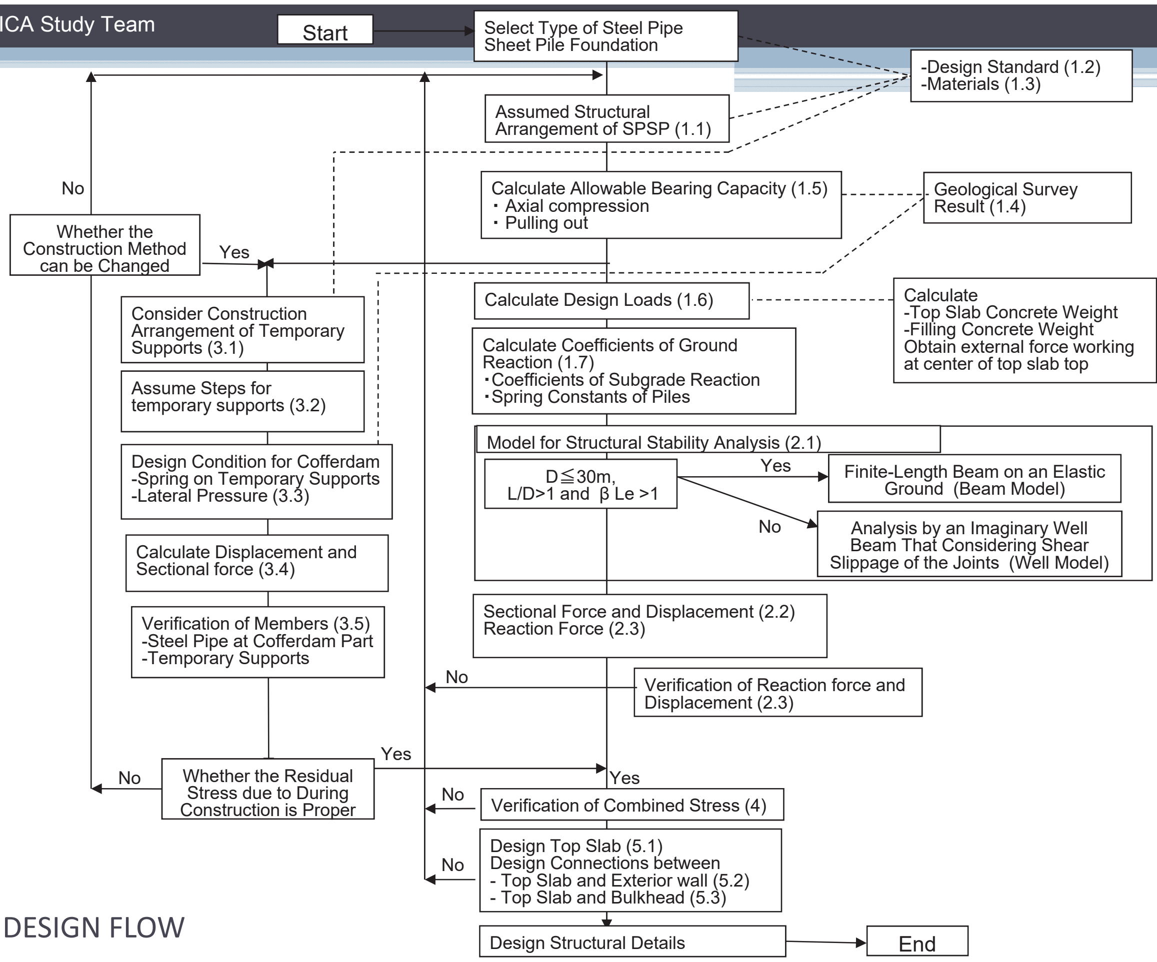
Contents of Lecture (6)

- Design of SPSP -

2. Design of SPSP

1. Design Condition
2. Structural Stability
3. Cofferdam and Temporary Supports
4. Verification of Combined Stress
5. Design of Top Slab and Connection

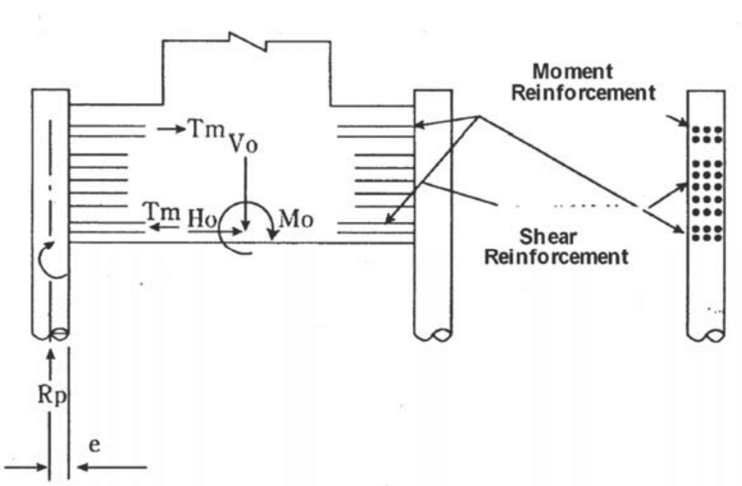




DESIGN FLOW

5. Design of Top Slab and Connection

5.2 Design of Connection between Top Slab and Exterior wall



The load is transmitted by moment reinforcement located on the upper and lower surfaces of the footing, and by a shear reinforcement located between these moment reinforcement.

(1) Design of moment reinforcement

(a) Design moment

Maximum one between M_e and M_{fix}.

$$Me = Rp \cdot e \qquad MFix = \sigma sa \cdot Zo$$

Where, Me : moment due to eccentricity of reaction force (kN.m/pile)

M_{Fix} : restraint moment (kN.m/pile)

Rp : vertical reaction force of a sheet pile (kN/pile)

e : eccentricity = radius of the steel pile D0/2 (m)

σ_{sa} : allowable stress of sheet pile(kN/m^2)

Zo : section modulus of sheet pile body (m^3)

(b) Tensile force due to moment acting on the moment reinforcement (Tm (kN/pile))

$$Tm = \frac{M}{h}$$

Where h : center-to-center interval of moment reinforcement (m)

(c) Tensile force due to horizontal force acting on the footing (Ts (kN/pile))

$$Ts = \frac{Ho}{n1}$$

Where n1 : number of outer piles (nos.)

d) Tensile stress of the moment reinforcement caused by moment (σs1 kN/m^2)

$$\sigma s1 = \frac{Tm}{nb \cdot Ab}$$

nb : number of moment reinforcement (nos.)

Ab : cross-sectional area of one moment reinforcement (m^2)

e) Tensile stress of the moment reinforcement due to horizontal force (σs2 kN/m^2)

$$\sigma s2 = \frac{Ts}{2 \cdot nb \cdot Ab}$$

f) Verification

The sum of both of above tensile stress $\sigma s = \sigma s1 + \sigma s2 \leqq \sigma_{sa}$

where, σ_{sa} : allowable tensile stress of moment reinforcement (N/mm^2)

(2) Design of shear reinforcement

$$\tau s = \frac{Rp}{ns \cdot As} \leqq \tau sa$$

Where, τs : shear stress of shear reinforcement (kN/m^2)

Rp : vertical reaction force of a sheet pile (kN/pile)

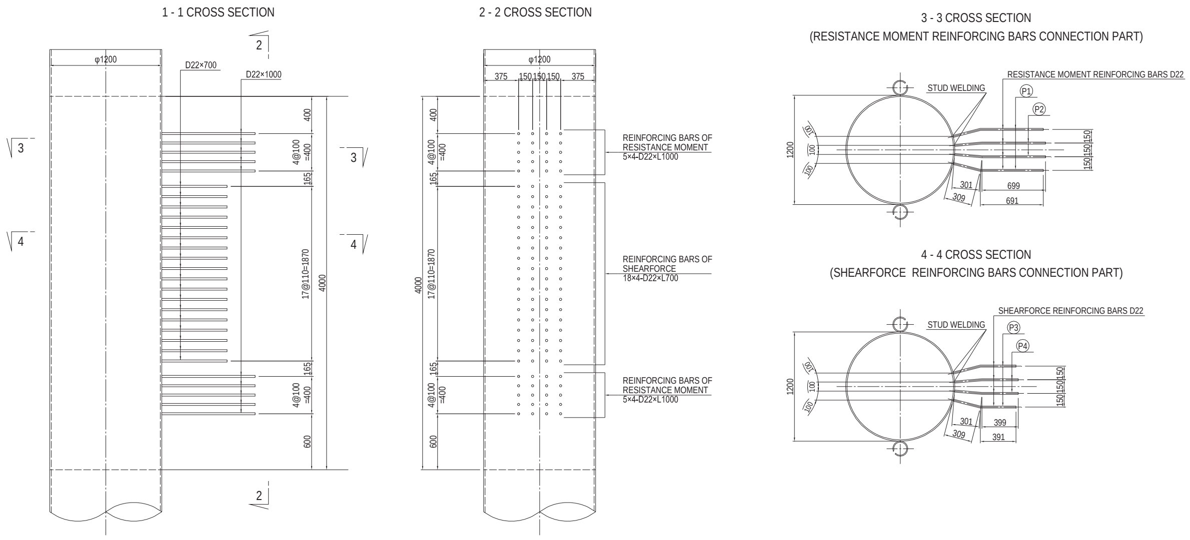
ns : number of shear reinforcement (nos.)

As : cross-sectional area of shear reinforcement (m^2)

τsa : allowable shear stress of shear reinforcement =0.6 x σ_{sa} (kN/m^2)

(3) Arrangement of Reinforcement Studs

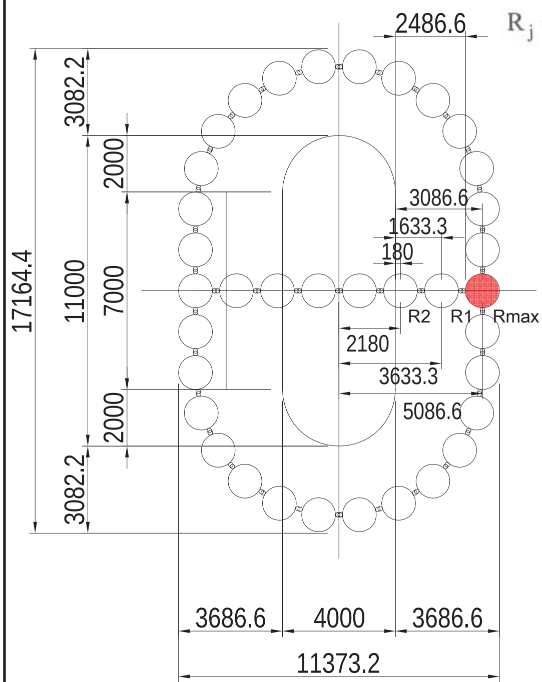
DETAIL OF CONNECTION BETWEEN STEEL PIPE SHEET PILE AND FOOTING



- ✓ The reinforcement studs are installed toward the center of SPSP.
- ✓ The reinforcement stud driving pitch is 100mm or more in the vertical direction, and d (d:diameter of the reinforcing bar) +30mm or more in the circumferential direction of SPSP.
- ✓ Minimum length of moment reinforcement is 1m.

Exercise1: Design of Connection between the footing and SPSP

1) Vertical Reaction Force of outer pile and bulkhead pile (kN/pile)



$$R_j = \frac{V_o A_{01}}{n_1 A_{01} + n_2 A_{02}} \pm \frac{M_0 A_{01}}{\sum^{n1} x_i^2 A_{01} + \sum^{n2} x_i^2 A_{02}} x_j$$

Pile	Vertical Reaction Force (kN/pile)
R_{max}	4237

Given:

	Nos.	Net cross-sectional area of SPSP A_{01} (m^2/pile)	$\Sigma^{n1,2} xi^2$ (m^2)
Outer part	30	0.04464	491.6
Bulkhead part	6	0.04464	36.96

Example of P15, in longitudinal direction, for seismic case

2) Design of Moment Reinforcement

・M=max(Me : M_{Fix})

Me =Rp x e

Rp= kN, e= m Me = x = kN.m

M_{Fix}= σsa x Z_0

σsa =185 N/mm^2, Z_0 = 13081.0cm^3 (Dia1.2m, t14mm, 2mm corrosion)

M_{Fix}= 185,000 x 1.5 x 0.013081 =3,630kN.m

・Tm= M/h h= mm

Tm= / = kN

σ_{s1} = / = N/mm^2

・Ts = / = kN

σ_{s2} = / = N/mm^2

・Combined Stress $\sigma_s = \sigma_{s1} + \sigma_{s2}$ = ≦ σ_{sa} =300 (N/mm^2)

3) Design of Shear Reinforcement

・τs=Rp / (ns・As)

Rp = kN, ns= , As=387.1 mm^2 (dia.22mm)

τs = / = N/mm^2 ≦ τsa = N/mm^2

5.3 Design of Connection between Top Slab and Bulkhead Pile

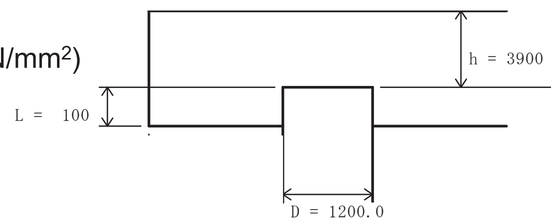
(1) Vertical bearing stress of top slab concrete

$$\sigma cv = \frac{PNmax}{\pi \cdot D^2/4} \leqq \sigma ca$$

Where, P_{Nmax} : maximum pushing force in axial direction (N)

D : diameter of pile (mm)

σ_{ca} : allowable bearing stress of concrete (N/mm^2)



(2) Punching shear stress of top slab concrete

$$\tau v = \frac{PNmax}{\pi \cdot (D + h) \cdot h} \leqq \tau a$$

Where, h : effective depth of top slab which resist to punching shear force in vertical direction (mm)

τa : allowable punching shear stress of concrete (N/mm^2)

(3) Horizontal bearing stress of top slab concrete

$$\sigma ch = \frac{PHmax}{D \cdot I} \leqq \sigma ca$$

Where, PHmax : force in horizontal direction (N)

L : embedded length of pile (mm)

σca : allowable bearing stress of concrete (N/mm^2)

5.3 Design of Connection between Top Slab and Bulkhead Pile

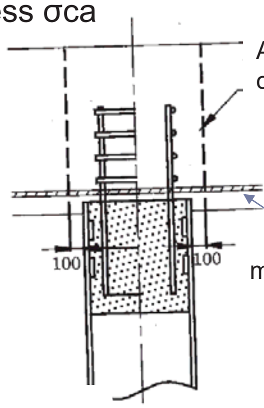
(4) Stresses in the concrete and reinforcing bars in the footing is checked by assuming a virtual RC pile section in the footing.

・Tensile stress ：$\sigma s \leqq$ Allowable tensile stress σsa

・Compressive stress: $\sigma c \leqq$ Allowable compressive stress σca

Diameter of assumed reinforced concrete section

Do = D＋0.25D+100 (mm)



(5) Anchoring length bars into footing inside shall be more than L from the center of the lower main reinforcing bar in footing.

$$L \geqq Lo + 10 \cdot \varphi \qquad Lo = \frac{\sigma sa}{4 \tau oa} \cdot \phi$$

Where,

L ：necessary length of bars in footing inside (mm)

Lo ：necessary anchoring length of bars (mm)

σsa ：allowable tensile stress of bars (N/mm^2)

τoa ：allowable bond stress of concrete (N/mm^2)

φ ：bar diameter(mm)

Detailed Design on Bago River Bridge Construction Project

JICA Study Team

Design Technology Transfer
Lecture : Substructure and Foundation Design

6th December 2017: Group B
7th December 2017: Group A

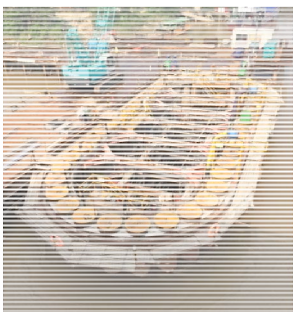
Contents of Lecture (6)

- Design of Substructure -

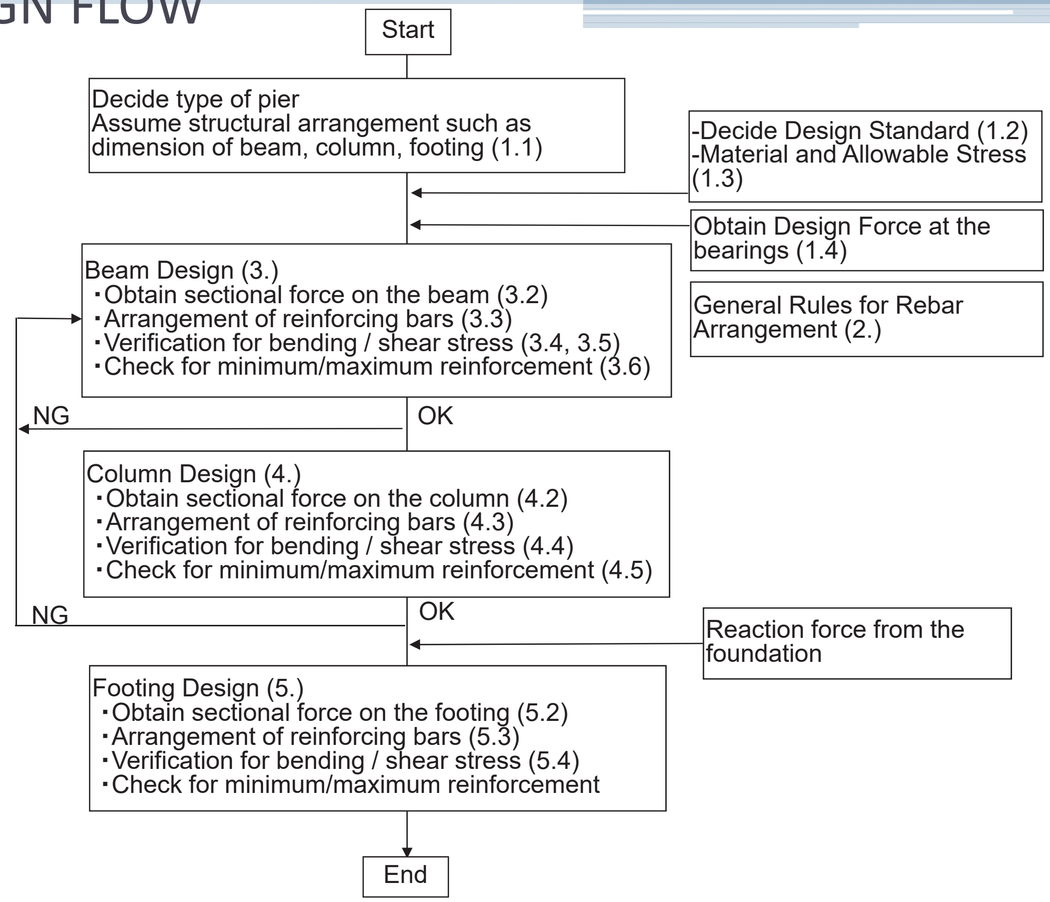
1. Design Condition
2. General Rules for Rebar Arrangement
3. Beam Design
4. Column Design

Reference

A. Behavior of RC Member
B. Basis of RC section calculation

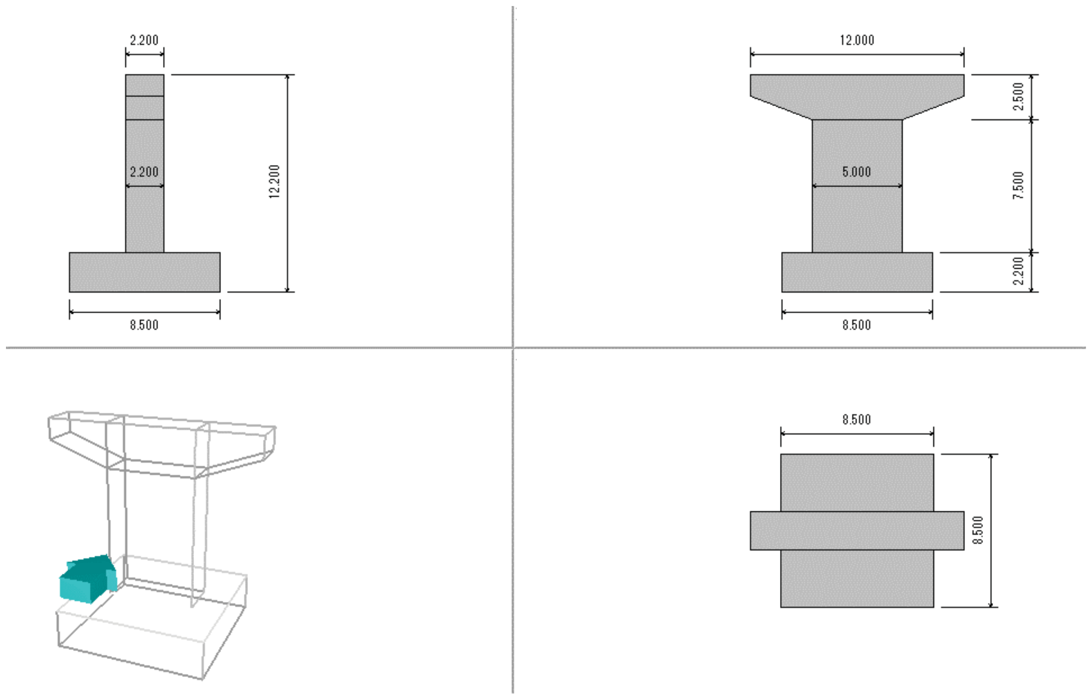


DESIGN FLOW



1.Design Condition

1.1 STRUCURAL ARRANGEMENT OF PIER



1.2 Design standard

SPECIFICATIONS FOR HIGHWAY BRIDGES (JSHB)
Published by JAPAN ROAD ASSOCIATION.
It is composed from five separate parts as below.
Sub-structure design is described in Part IV: substructures.

- -PART I : COMMON
- -PART II : STEEO BRIDGES
- -PART III : CONCRETE BRIDGES
- -PART IV : SUBSTRUCTURES
- -PART V : SEISMIC DESIGN

In JSHB, the design verification for substructures is based on Allowable stress method.

1.3 Material and Allowable Stress

- The allowable stresses used for the design is determined by type or strength of the material.
- The allowable stresses shall be multiplied by the respective increase coefficients stipulated in each load cases as table.

-Design Strength of Concrete for pier: σ=24N/mm^2

No	Load case	Increase coefficient	Allowable bending compressive stress σ_{ca}	Allowable shear stress	
				τ_{a1}*	τ_{a2}*
1	Dead + Live	1.00	8.0 N/mm^2	0.23 N/mm^2	1.70 N/mm^2
2	Dead + Live + Temperature	1.15	9.2 N/mm^2	0.264 N/mm^2	1.955 N/mm^2
3	Seismic Effect	1.50	12.0 N/mm^2	0.350 N/mm^2	2.550 N/mm^2

*τ_{a1}: allowable shear stress if only concrete resists against shear force
*τ_{a2}: allowable shear stress if both concrete and shear reinforcement resist against shear force

-Reinforcing bar Material type for pier: SD345

No	Load case	Increase coefficient	Allowable bending compressive stress σ_{sa}'	Allowable bending tensile stress σ_{sa}
1	Dead + Live	1.00	200.0 N/mm^2	180.0 N/mm^2 **
2	Dead + Live + Temperature	1.15	230.0 N/mm^2	207.0 N/mm^2
3	Seismic Effect	1.50	300.0 N/mm^2	300.0 N/mm^2

**160.0 N/mm^2 for underwater

1.4 Design force

For design calculation of the pier which is not rigid with girders, reaction force (V, H, M) worked at bearing is utilized.

・ For designing the column, weight of beam is added on working force.

・For designing the beam, reaction forces at position of bearings is applied.

Image of design force transmission

Structural calculation of super structure

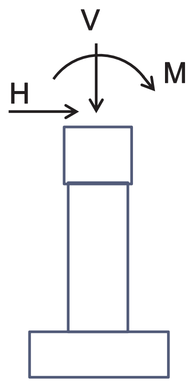
Reaction force from super-structure

Structural calculation of the beam

Structural calculation of the column

Structural calculation of the footing

Reaction force at bearing position



2. General rules for rebar arrangement

2.1 MINIMUM & MAXIMUM REINFORCEMENT

(1)Minimum amount of reinforcement

1) In order to prevent the decrease of member's strength by concrete cracks developing and breaking abruptly, axial tensile reinforcement shall be placed as either of requirement below ;

・The maximum resisting bending moment of the member (Mu) is greater than the cracking bending moment (Mc).

$$Mc \leqq Mu$$

・ 1.7 times of the bending moment M is less than the cracking bending moment.

$$1.7\,M \leqq Mc$$

$$M_c = Z_c(\sigma_{bt} + \frac{N}{A_c})$$

σ_{ck}: Design strength of concrete (N/mm^2)

N: Axial force (N)

A_c: Cross-sectional area of concrete (mm^2)

Where,

M_c: Cracking bending moment (N·mm)

Z_c: Modulus of section of concrete member (mm^3)

σ_{bt}: Bending tensile strength of concrete (N/mm^2)

$\sigma_{bt} = 0.23\ \sigma_{ck}^{2/3}$

2.1 Minimum & maximum reinforcement

(1) Minimum amount of reinforcement

2) In a member in which axial force is dominant, axial reinforcement shall be placed in such manner that no brittle fracture will occur even if an eccentric load equal to or greater than the value assumed in the design, or even if there is a local weak spot in the concrete.

・Hence, amount of axial reinforcement in the member subjected to axial forces is taken at 0.8 percent or more of the calculated necessary concrete cross-sectional area (Ac') for those axial forces.

As≧ Ac' x 0.8%

$$A'_c = N_a / (0.008\, \sigma_{sa} + \sigma_{ca})$$

N_a: Axial compressive force (N)

σ_{sa}: Allowable compressive stress of reinforcement bar (N/mm^2)

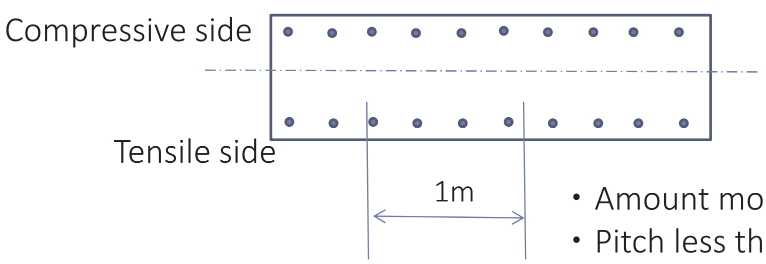
σ_{ca}: Allowable compressive stress of concrete (N/mm^2)

2.1 Minimum & maximum reinforcement

(1) Minimum amount of reinforcement

3) Reinforcement shall be placed in such manner that no harmful cracking will occur from drying shrinkage, temperature gradient, etc.

・Reinforcement shall be placed with satisfying cross-sectional area of more than 500mm^2 /m along the member surface and at a pitch of 300mm or less on centers.



・Amount more than 500mm^2 /m

・Pitch less than 300mm

(2) Maximum amount of reinforcement

1) In a member subjected to bending, axial tensile reinforcement shall be placed in such a manner that no brittle fracture will occur in which concrete fracture precedes the yield of the reinforcement.

   In general, a content of axil tensile reinforcement is arranged to be less than 2% of the effective cross-sectional area of that member.

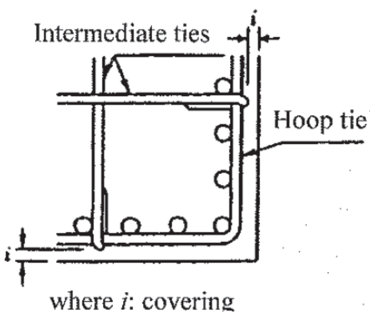
2) When reinforcement is placed too densely, the workability of concrete placing will lower. Therefore, it is desirable that axial reinforcement is a member subjected to axial forces is taken at about 6 percent or less of the total cross-sectional area of that member.

2.2 REINFORCEMENT COVER

・A necessary cover shall be secured to ensure the bond between concrete and reinforcement, prevent reinforcement corrosion, and protect the reinforcement from water current and fire.

・For a member existing under water or ground, a necessary cover shall be secured, considering the difficulty of maintenance as well.

・It is to be taken more than the value below;



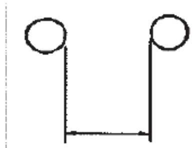
Environmental conditions \ Member type	Beam	Column	Footing
In the air	35	40	–
In the water or earth	–	70	70

(mm)

2.3 CLEAR DISTANCE BETWEEN REINFORCEMENTS

- A clear distance shall be set up between reinforcement bars so that concrete can spread fully around the bars and make it possible to compact the concrete firmly, also for adequate bond between concrete and reinforcement and for acting as a single body.

1) The spacing of main reinforcement bars as figure shall be equal to or lager than 40mm and 4/3 times the maximum size of coarse aggregate.
2) The spacing of reinforcement bars shall lager than 1.5 times the bar diameter in addition to condition 1).



Spacing of reinforcement bars

2.4 JOINT OF REINFORCEMENT BARS

(1) When reinforcement bars are to be joined, the joint shall not form a weak spot in the member.

(2) When designing in accordance with the provision stated below, (1) may be deemed to be satisfied:

1) The joint positions of reinforcement bars shall not concentrate on one section in principle. At the positions of large stress may occur, it is not desirable to joint reinforcement bars.

2) When a lap joint is to be used for tension bars, they shall be lapped over a length larger than the lap joint length l_a calculated by Eq. 7.8.1 and also larger than 20 times the bar diameter. At the lap joint, it is recommended to reinforce by placing transverse reinforcement having a cross-sectional area larger than 1/3 of the cross-sectional area of a jointed bar.

2.4 JOINT OF REINFORCEMENT BARS

$$l_a = \frac{\sigma_{sa}}{4\tau_{0a}}\phi \quad \cdots\cdots (7.8.1)$$

where

l_a: Lap joint length calculated from bonding stress (mm)

σ_{sa}: Allowable tensile stress of reinforcement (N/mm^2)

τ_{0a}: Allowable bonding stress of concrete (N/mm^2)

ϕ: Diameter of reinforcement (mm)

3) When a lap joint is to be used for compression bars, they shall be lapped over a length larger than 80% of l_a calculated by Eq. 7.8.1 and also larger than 20 times the bar diameter.

2.5 HOOK AND BENT SHAPE OF REINFORCEMENT

(1) The bent shape of reinforcement shall take a shape that can be easily worked and will do no damage to the reinforcement material.

(2) The bent shape of reinforcement shall take a shape that will not cause a large bearing stress in the concrete.

(3) When designing in accordance with the provisions stated below, (1) and (2) may be deemed to be satisfied.

1) Hooks of reinforcement bar
Hooks of reinforcement bar shall conform to the following prescriptions.

i) To hook a round bar, it shall be bent in semicircular form.

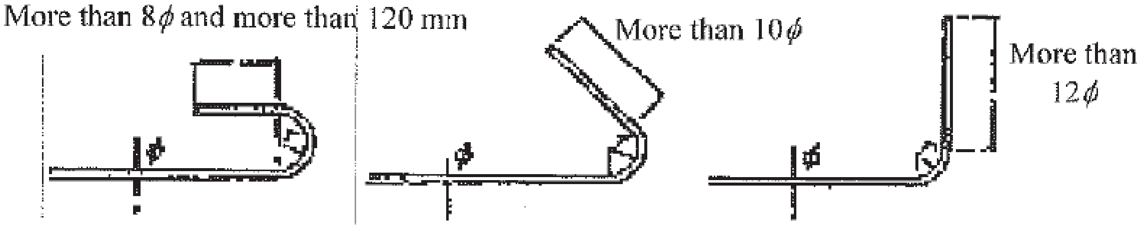
ii) To hook a deformed steel bar, it shall be bent in semicircular form or to right or acute angle.

iii) The hooked part of a reinforcement bar shall extend straight over a length longer than the value given below from the end of the bent part according to Fig.-7.7.1. The radius of curvature of a hook shall be as prescribed in 2).

2.5 HOOK AND BENT SHAPE OF REINFORCEMENT

iii) The hooked part of a reinforcement bar shall extend straight over a length longer than the value given below from the end of the bent part according to Fig.-7.7.1. The radius of curvature of a hook shall be as prescribed in 2).

(a) Semicircular hook: the larger of 8 times the bar diameter and 120 mm
(b) Acute-angled hook: 10 times the bar diameter
(c) Right-angled hook: 12 times the bar diameter



(a) Semicircular hook (b) Acute-angled hook (c) Right-angled hook

where ϕ: Diameter of reinforcement bar (mm)
r: Radius of curvature of reinforcement bar (mm)

r=2.5φ (SD345)
r=3.0φ (SD390)
r=3.5φ (SD490)

2.6 ANCHORAGE OF REINFORCEMENT BAR

1) An end of a bar shall be more than enough anchored to the concrete by one of the methods prescribed below. When round bars are to be used for tension bars, it shall be anchored as a hook.

i) embed the bar in concrete and anchor it by the bonding between it and the concrete.

ii) embed the bar in concrete and anchor it by means of a hook.

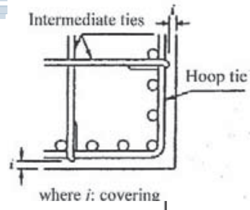
iii) anchor the bar mechanically follow the attach an anchor plate.

2) When a bar is to be anchored by the bonding between it and concrete, the bond length shall be larger than the lap joint length of reinforcement prescribed in (2) 2) and 3) of Sec. 7.8.

$$l_a = \frac{\sigma_{sa}}{4\tau_{0a}}\phi \quad \cdots\cdots (7.8.1)$$

3) When a bar is to be hooked for anchorage, the bonding length shall be larger than 2/3 of the bonding length prescribed in 2). When a compression bar is to be hooked for anchorage, the bonding length shall be as prescribed in 2) and the effect of the hook shall not be taken into account.

2.7 Shear Reinforcement (hoop tie, Intermediate tie)



1) The hoop tie shall be a deformed steel bar 13 mm in diameter or more.

2) The hoop tie shall be placed over the entire length of the columnar member, and its spacing shall be 300 mm or less.

3) The hoop tie shall be placed to enclose the axial reinforcement and shall be hooked at the ends and anchored to the concrete in the inner part of a section as standard. A lap joint without a hook shall not be used, as a general rule. When hoop ties are to be joined, the joints shall be placed in a vertically staggered arrangement.

4) When the hoop tie spacing is to be varied with height, the spacing variations shall be gradual and not steep.

5) When the axial reinforcement in a pier column is to be thinned out, the hoop tie spacing shall be 150 mm or less in the section area equivalent to 1.5 times the shorter-side length or diameter of the pier section above and below the thinning-out position.

DESIGN FLOW

Start

Decide type of pier
Assume structural arrangement such as dimension of beam, column, footing (1.1)

-Decide Design Standard (1.2)
-Material and Allowable Stress (1.3)

Obtain Design Force at the bearings (1.4)

General Rules for Rebar Arrangement (2.)

Beam Design (3.)
・Obtain sectional force on the beam (3.2)
・Arrangement of reinforcing bars (3.3)
・Verification for bending / shear stress (3.4, 3.5)
・Check for minimum/maximum reinforcement (3.6)

NG OK

Column Design (4.)
・Obtain sectional force on the column (4.2)
・Arrangement of reinforcing bars (4.3)
・Verification for bending / shear stress (4.4)
・Check for minimum/maximum reinforcement (4.5)

NG OK

Reaction force from the foundation

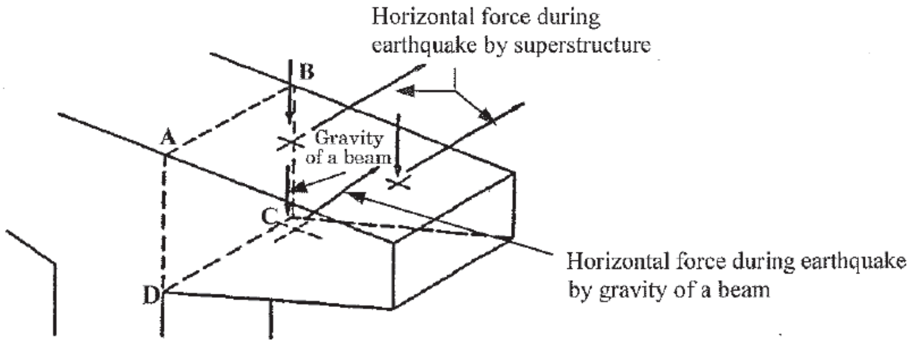
Footing Design (5.)
・Obtain sectional force on the footing (5.2)
・Arrangement of reinforcing bars (5.3)
・Verification for bending / shear stress (5.4)
・Check for minimum/maximum reinforcement

End

3. Beam design

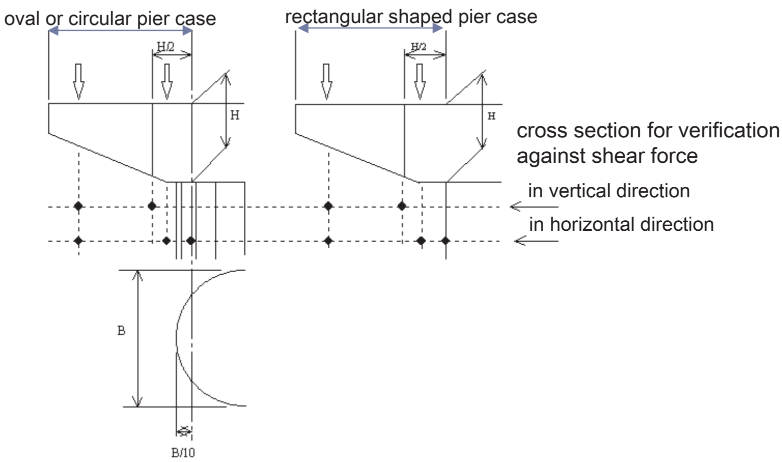
3.1 DESIGN MODEL FOR THE BEAM

1) The overhanging beams of T-shaped piers are designed as cantilever beam.
2) The position of external force is considered at position of each bearings at beam.
3) The tensile stress should be checked.
4) The beams shall be designed not only for vertical forces but also for horizontal forces in the longitudinal direction of the bridge in seismic condition.



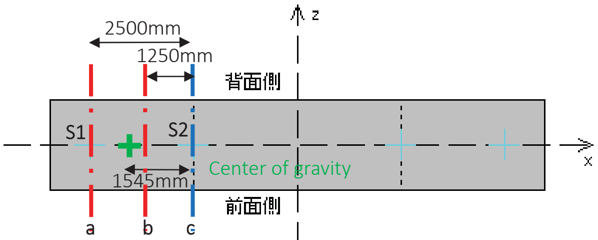
3.1 DESIGN MODEL FOR THE BEAM

5) If the shape of column is circular, the overhanging length of the cantilever beam is defined as the length from the position one tenth of the column diameter inward from the front of the column to the beam end.
6) For bending moment in vertical and horizontal direction, the verification shall be considered for the section defined in 5).
7) For shear force in vertical direction, the verification shall be done at H/2 from the design cross section, and position of bearings which are installed at outer H/2 section. For horizontal direction, at the end of cantilever and at bearings



3.2 SECTIONAL FORCE

Sectional Force



Design section for Shear Force

Design section for Moment in both directions
Shear Force (only horizontal direction)

Reaction force at Bearings

Bearing	Dead Load Rd (kN)	Live Load RL (kN)	Horizontal Force H (kN)
S1	1765	647	316
S2	1471	578	316

*Volume of overhanging beam = 14.245m^{3}x24.5kN/m^3 =349.00 kN

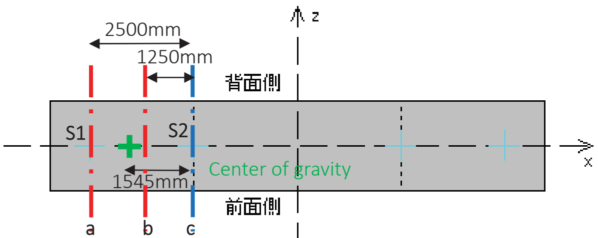
*Horizontal seismic coefficient kh=0.25

Sectional force working at design section (c-c) for **bending moment and shear force**

		For Vertical direction			For Horizontal direction		
		V (kN)	*Arm length* (m)	M (kN.m)	H (kN)	*Arm length* (m)	M (kN.m)
Dead Load	Self-weight	349	1.545	539.22	-	-	-
	Reaction force	1765 1471	2.5 0.0	4412.50 0.0	-	-	-
	Total	**3585**	-	**4951.72**	-	-	-
Dead + Live Load	Self-weight	349	1.545	539.22	-	-	-
	Reaction force	2412 2049	2.5 0.0	6030 0	-	-	-
	Total	**4810**	-	**6569.22**	-	-	-
Seismic Case	Self-weight	-	-	-	349x0.25	1.545	134.81
	Reaction force	-	-	-	316 316	2.5 0.0	790 0.0
	Total	-	-	-	**719.25**	-	**924.81**

3.2 SECTIONAL FORCE

Sectional Force



Design section for Shear Force

Design section for Moment in both directions
Shear Force (only horizontal direction)

Reaction force at bearings

Bearing	Dead Load Rd (kN)	Live Load RL (kN)	Horizontal Force H (kN)
S1	1765	647	316
S2	1471	578	316

*Volume of overhanging beam = 14.245m^{3}x24.5kN/m^3 =349.00 kN

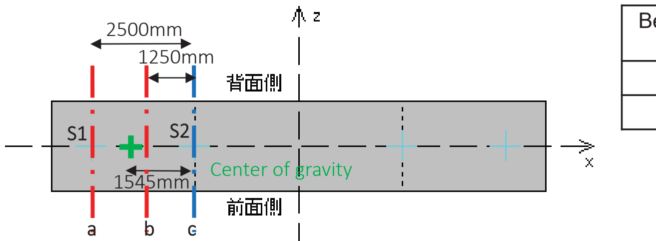
*Horizontal seismic coefficient kh=0.25

Sectional force working at design section (a-a) for **shear force**

		For Vertical direction			For Horizontal direction		
		V (kN)	*Arm length* (m)	M (kN.m)	H (kN)	*Arm length* (m)	M (kN.m)
Dead Load	Self-weight	349	1.545	539.22	-	-	-
	Reaction force	1765 1471	2.5 0.0	4412.50 0.0	-	-	-
	Total	**3585**	-	**4951.72**	-	-	-
Dead + Live Load	Self-weight	349	1.545	539.22	-	-	-
	Reaction force	2412 2049	2.5 0.0	6030 0	-	-	-
	Total	**4810**	-	**6569.22**	-	-	-
Seismic Case	Self-weight	-	-	-	74.68x0.25	0.477	8.92
	Reaction force	-	-	-	316	-	-
	Total	-	-	-	**334.67**	-	**8.92**

3.2 SECTIONAL FORCE

Sectional Force



Reaction force at bearings

Bearing	Dead Load Rd (kN)	Live Load RL (kN)	Horizontal Force H (kN)
S1	1765	647	316
S2	1471	578	316

*Volume of overhanging beam = $14.245m^3 x 24.5kN/m^3$ =349.00 kN

*Horizontal seismic coefficient kh=0.25

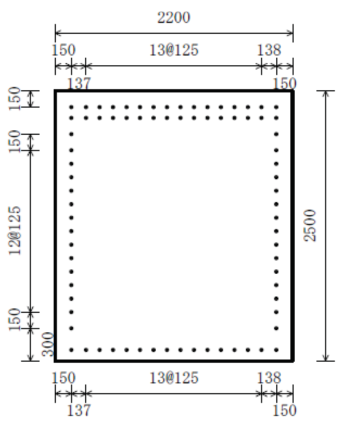
Sectional force working at design section (b-b) for **shear force**

		For Vertical direction			For Horizontal direction		
		V (kN)	*Arm length* (m)	M (kN.m)	H (kN)	*Arm length* (m)	M (kN.m)
Dead Load	Self-weight	349	1.545	539.22	-	-	-
	Reaction force	1765 1471	2.5 0.0	4412.50 0.0	-	-	-
	Total	**3585**	**-**	**4951.72**	**-**	**-**	**-**
Dead + Live Load	Self-weight	349	1.545	539.22	-	-	-
	Reaction force	2412 2049	2.5 0.0	6030 0	-	-	-
	Total	**4810**	**-**	**6569.22**	**-**	**-**	**-**
Seismic Case	Self-weight	-	-	-	-	-	-
	Reaction force	-	-	-	-	-	-
	Total	**-**	**-**	**-**	**-**	**-**	**-**

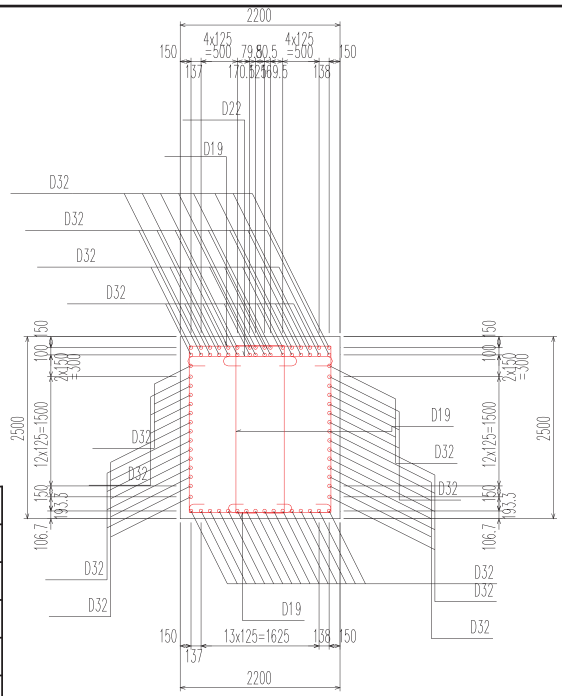
3.3 Rebar arrangement on beam

Rebar shall be arranged taking into account of stress verification and general guidelines of rebar arrangement.

Example Cross section at C-C



Location	Cover (mm)	Dia. (m)	Arrangement
Upper	150	D32	137+13@125+138
	250	D32	137+13@125+138
Lower	100	D32	137+13@125+138
Side	150	D32	150+12@125+150
Stirrup	-	D19	150mm pitch



3.4 Verification against bending moment on beam

Example of Verification against bending moment at C-C Section

(1)Moment in Vertical direction

Item	Unit	Dead Load	Dead + Live
Bending Moment M	kN.m	4951.72	6569.22
Compressive edge～neutral axis x	mm	736	736
Compressive stress σc Tensile stress σs	N/mm2 N/mm2	2.97 97.79	3.94 129.74
Increased coefficient α Allowable compressive stress σca Allowable tensile stress σsa	— N/mm2 N/mm2	1.00 8.00 100.00	1.00 8.00 180.00

(2) Moment in Horizontal direction

Item	Unit	Seismic
Bending Moment M	kN.m	924.81
Compressive edge～neutral axis x	mm	513
Compressive stress σc Tensile stress σs	N/mm2 N/mm2	0.77 34.43
Increased coefficient α Allowable compressive stress σca Allowable tensile sterss σsa	— N/mm2 N/mm2	1.50 12.00 300.00

3.4 Verification against bending moment on beam

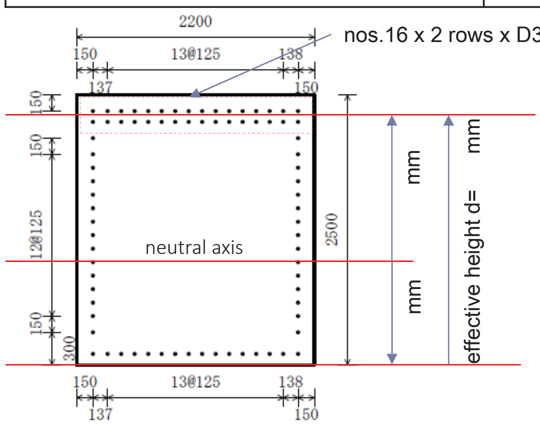
Example of Verification against bending moment at C-C Section

Calculation of σc and σs

Item	Unit	Dead Load
Bending Moment in vertical direction M	kN.m	4951.72
Compressive edge～neutral axis x	mm	736
Compressive stress σc Tensile stress σs	N/mm2 N/mm2	2.97 97.79
Increased coefficient α Allowable compressive stress σca Allowable tensile stress σsa	— N/mm2 N/mm2	1.00 8.00 100.00

*Ratio of Modulus of Elasticity Es/Ec=15

nos.16 x 2 rows x D32 (SD345, A=794.2mm^2/rebar)

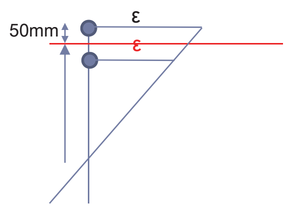


1) Compressive edge～neutral axis x(mm)

2) Moment of Inertia of Converted Concrete Section Area: In (mm^2)

3) σc (N/mm^2)

4) σs (N/mm^2)



3.5 Verification against shear force on beam

1)When only the concrete carries the shear forces, the mean shear stresses τ_m calculated from 3) shall be smaller than the allowable shear stresses τ_{a1}.

2)When the concrete and diagonal tension reinforcement jointly carry the shear forces, the mean shear stresses τ_m shall not exceed the allowable shear stresses τ_{a2}.

3)The mean shear stress of the concrete generated in a section of an RC member shall be calculated by ;

$$\tau_m = \frac{S_h}{b\,d}$$

τ_m : mean concrete shear stress generated in a member section (N/mm²)

S_h : shear force considering the variation in effective depth of the member (N), calculated from $S_h = S - \frac{M}{d}(\tan\beta + \tan\gamma)$

3.5 Verification for shear force on beam

When only the concrete bears the shear forces, the allowable shear stress τ_{a1} should be modified in consideration of the following effects;

- Modification coefficient “Ce” for the effective depth d, at the member section
- Modification coefficient “Cpt” for the longitudinal tension bars ratio pt
- Modification coefficient “CN” for the axial compressive forces

Effective Depth d (mm)	Less than 300	1,000	3,000	5,000	More than 10,000
c_e	1.4	1.0	0.7	0.6	0.5

Longitudinal Tensile Reinforcement Ratio p_t (%)	0.1	0.2	0.3	0.5	1.0 or more
c_{pt}	0.7	0.9	1.0	1.2	1.5

$$CN = 1 + \frac{Mo}{M} \quad (1.0 \leqq CN \leqq 2.0)$$

$$Mo = \frac{N}{Ac} \cdot \frac{Ic}{y}$$

Ac : Area of section

Ic : Moment of inertia of section

y = Distance from center of section to the edge of tensile side

3.5 Verification for shear force on beam

・When the mean shear stress in the concrete exceeds the allowable shear stress τ_{a1}, diagonal tension bars with a cross-sectional area more than calculated the equation below shall be placed.

$$Awreq = \frac{1.15 \cdot Sh' \cdot s}{\sigma sa \cdot d}$$

Aw (mm^2) : Amount of diagonal tension bars at section where is verified

s (mm) :Pitch of diagonal tension bars

Awreq (mm^2) : Required amount of diagonal tension bars

Sh' : Shear strength resisted by diagonal tension bars

$Sh' = S - Sca$ (kN)

Sca : Shear strength resisted by concrete

$Sca = \tau a1 \cdot b \cdot d$ (kN)

Example

・Section (a-a) : at outer bearing

Item	Unit	Dead Load	Dead+Live	Seismic (horizontal)
S	kN	1839.69	2486.69	334.67
M	kN.m	35.68	35.68	8.92
a	mm	2500	2500	2500
d/1.15	mm	1193	1193	——
d	mm	1371	1371	2050
Sh	kN	1830.03	2477.03	334.67
α	——	1.00	1.00	1.50
pt	%	0.842	0.842	0.296
ce	——	0.944	0.944	0.842
cpt	——	1.405	1.405	0.996
τm	N/mm^2	0.607	0.821	0.104
τa_1	N/mm^2	0.305	0.305	0.294
τa_2	N/mm^2	1.700	1.700	2.550
σsa	N/mm^2	180.00	180.00	——
s	mm	150	150	——
Sca	kN	920.92	920.92	——
Sh'	kN	909.10	1556.10	——
AwReq	mm^2	635.28	1087.40	——
Aw	mm^2	1146.00	1146.00	——

・Section (b-b) : H/2 from pier front

Dead Load	Dead+Live
1961.21	2608.21
2407.98	3216.73
2500	2500
1596	1596
1836	1836
1473.98	1957.34
1.00	1.00
0.629	0.629
0.875	0.875
1.278	1.278
0.365	0.485
0.257	0.257
1.700	1.700
180.00	180.00
150	150
1037.94	1037.94
436.05	919.41
227.64	479.98
1146.00	1146.00

example of "Dead+Live Load case at a-a section"

1) $Sh = S - M/d \times (\tan\beta + \tan\gamma)$ (kN)

2) $\tau_m = S_h/bd$ (N/mm^2)

3) $\tau_{a1} = 0.23 \times \alpha \times c_e \times c_{pt} = 0.23 \times 1.0 \times 0.944 \times 1.405 = 0.305\ N/mm^2$

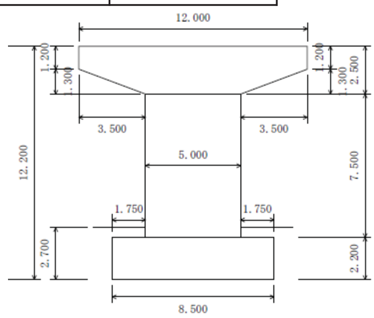
Hence, τ_m > or < τ_{a1}

5) $S_{ca} = \tau_{a1} \times b \times d$ (kN)

$S_{h'} = S_h - S_{ca}$ (kN)

$A_{wreq} = 1.15 \times S_h' \times s / (\sigma_{sa} \times d')$ (if a<d/1.15, d'=1.15a, if not d'=d)

mm^2 > or < A_w: 1146.0 mm^2 ・・・ok or NG



3.6 Minimum & maximum reinforcement

Example of at C-C Section (at front of pier)

(1) Minimum reinforcement

i) Mc = Z_c x σ_{bt} = Z_c x 0.23x $\sigma_{ck}{}^{2/3}$ (Cracking bending moment)

Mu = C x y1 + T x y2 (Resisting bending moment)

C=0.85σck x 0.8X x b= (N)

T=As x σsy = (N)

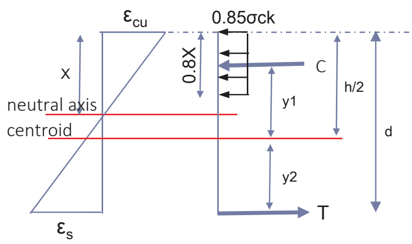
C=T ➔ X= (mm)

y1=h/2-0.4X (mm)

y2=h/2-(h-d) (mm)

Mu: (kN.m)

check Mc, Mu OK or NG



ii) Not applied (beam member)

iii) As=32 x 794.2=25414.4mm^2 ≧500 mm2/m x2.2m=1100mm2

Pitch@125mm < 300mm

3.6 Minimum & maximum reinforcement

Example of at C-C Section (at front of pier)

(2) Maximum reinforcement

As =32 x 794.2 =25414 mm^2

Asb: ultimate equilibrium area of tension bars

Pb=Asb/bd

$=\{0.68 \times \varepsilon_{cu}/(\varepsilon_{cu}+\varepsilon_{sy})\} \times \sigma_{ck}/\sigma_{sy} + As'/bd \times \sigma_s'/\sigma_{sy}$

Asb = Pb x bd

check As and Asb

4. Column design

4.1 DESIGN MODEL FOR THE COLUMN

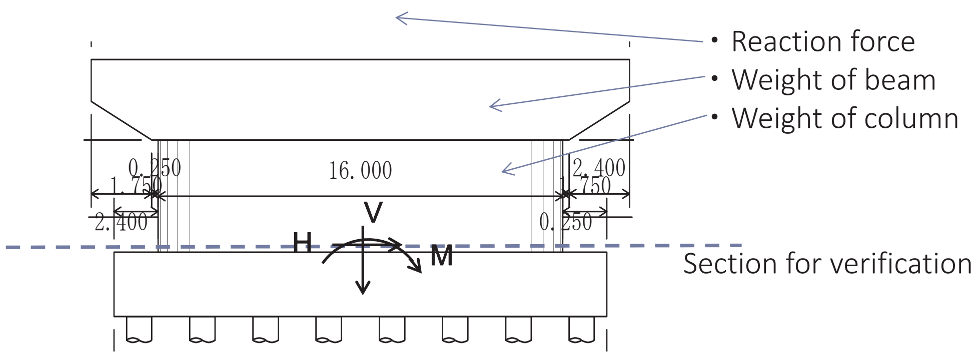
The columns of T-shaped piers can be designed as cantilevers with fixed ends at the section connected to the footings. In the design process, the most adverse combination of axial forces and bending moments shall be applied.

In general, it may be checked with respect to the maximum bending moment by combining the maximum and minimum axial force.

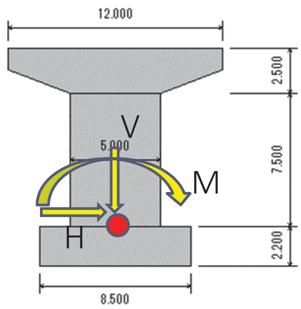
4. Column design

4.2 SECTIONAL FORCE

- The column shall be verified for the section where the maximum bending moment is generated such as at bottom of the column.
- Sectional force is calculated with reaction force from superstructure, dead weight of beam and column.



Example at Sectional force for column design

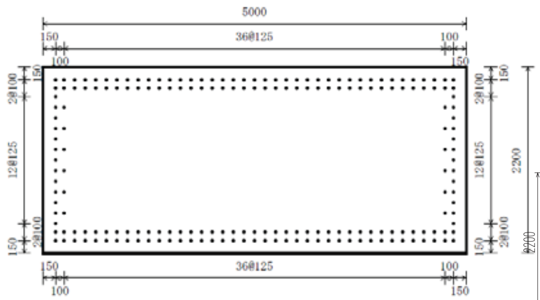


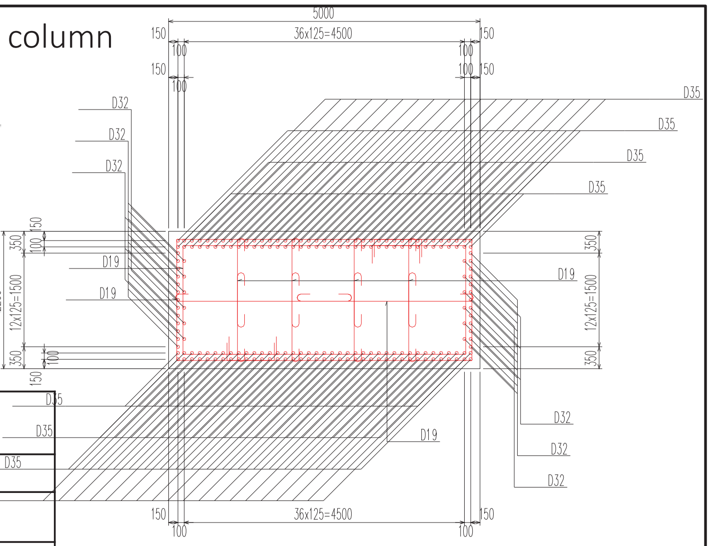
	Force	Vertical Force(kN)	Horizontal Force (kN)	Working heigth (m)	Eccentric moment (kN.m)	Bending moment (kN.m)
Dead+Live	・Reaction Force	9519.73	0.00	10.000	0.00	0.00
	・Weight of Pier	3393.01	0.00	5.830	0.00	0.00
	Total	12912.74	0.00	-	-	0.00
Seismic In Longitudinal	・Reaction Force	6970.00	1577.44	10.000	0.00	15774.40
	・Weight of Pier	3393.01	848.25	5.830	0.00	4945.71
	Total	10363.01	2425.69	-	-	20720.11
Seismic In perpendicular	・Reaction Force	6970.00	986.56	10.000	0.00	9865.60
	・Weight of Pier	3393.01	848.25	5.830	0.00	4945.71
	Total	10363.01	1834.81	-	-	14811.31

4.3 Rebar arrangement on column

Rebar arrangement shall be designed by taking account of stress verification and general rules of reinforcement.

Example of Rebar arrangement of the column

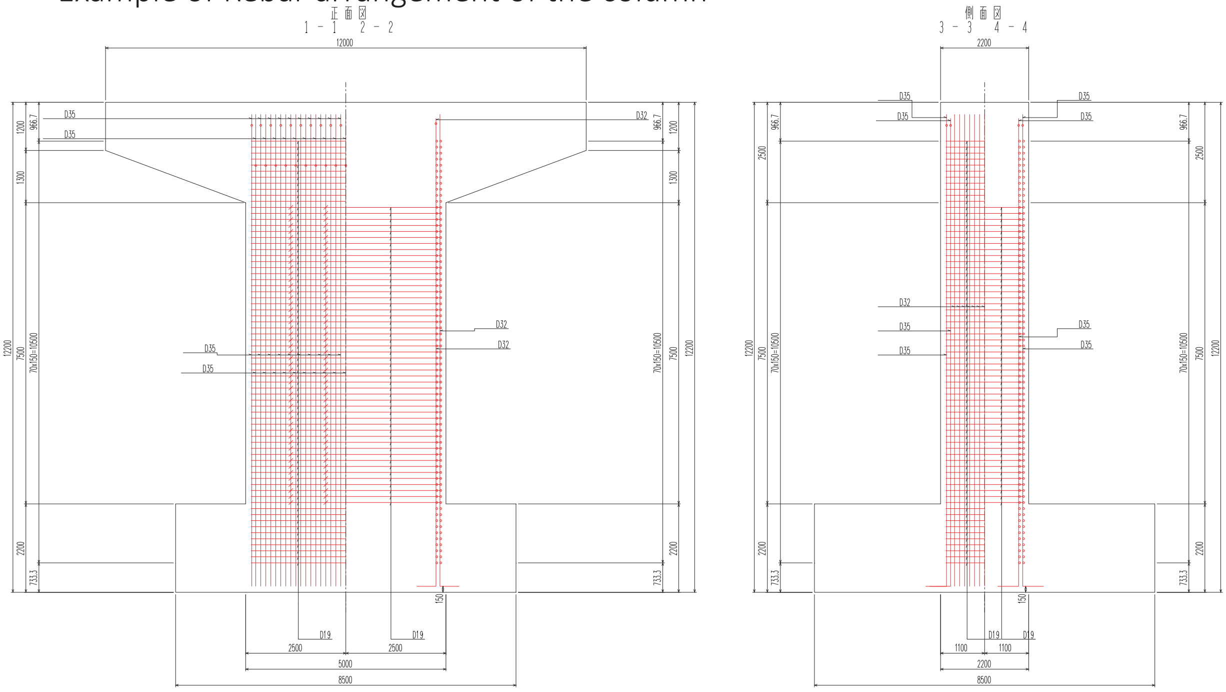




Location	Cover (mm)	Dia. (m)	Arrangement
Front and Back	150	D35	100+36@125+100
	250	D35	100+36@125+100
Left and Right	100	D32	100+100+12@125+100+100
	150	D32	100+225+5@250+225+100
Hoop tie	-	D19	150mm pitch
Intermediate tie	-	D19	Nos.4 in longitudinal Nos.1 in perpendicular

4.3 Rebar arrangement on column

Example of Rebar arrangement of the column



4.4 stress verification of column

- The column is a member which bending moment and axial force is worked.

Example of Verification against bending moment (in longitudinal direction)

Item	Unit	Dead+Live	Seismic
Axial Force N Bending Moment M	kN kN.m	12912.74 0.00	10363.01 20720.11
Compressive edge～neutral axis x	mm	---	927
Compressive stress σc Tensile stress σs	N/mm2 N/mm2	0.98 -14.63	5.46 99.26
Increased coefficient α Allowable compressive stress σca Allowable tensile stress σsa	— N/mm2 N/mm2	1.00 8.00 -200.00	1.50 12.00 300.00

Example of Verification against shear force (in longitudinal direction)

Item	Unit	Seismic
S N M	kN kN kN.m	2425.69 10363.01 20720.11
α pt ce cpt CN	— % — — —	1.50 0.897 0.850 1.438 1.183
τm τa1 τa2	N/mm2 N/mm2 N/mm2	0.243 0.506 2.550

4.5 Maximum & minimum reinforcement

Example of Verification of Max&Min Reinforcement for bridge longitudinal direction

Item	Unit	Dead+Live	Seismic
Cracking bending moment : Mc	kN.m	12453.17	11518.27
Yield bending moment : My0	kN.m	55018.85	53059.83
Maximum resisting bending moment : Mu	kN.m	60342.06	58096.40
Minimum amount as bending member	——	1.7M≦Mc	Mc≦Mu
Minimum amount as compressive member	mm2	12753.3	6823.4
0.008 Ac’	mm2	12753.3	6823.4
As ≧ Asmin (0.0008Ac’)	——	OK	OK
Myo≦Mu	——	OK	OK

(1) Minimum reinforcement

i) Mc =12453 kN.m, M = 0 kN.m,

Hence, 1.7M ≦ Mc

ii) 0.008xAc’= 12,753mm2 < As :179,409mm2

iii) As=179,409 ≧500 mm2/m x 14.4m=7,200mm2

(2) Maximum reinforcement

Ac x 6% =11,000,000 mm2 > As : 179,409mm2

4.5 Minimum & maximum reinforcement

Example of Verification of Max&Min Reinforcement for bridge longitudinal direction in Seismic Condition

(1) Minimum reinforcement

i) $Mc = Z_c \times (\sigma_{bt} + N/Ac) = Z_c \times (0.23 \times \sigma_{ck}{}^{2/3} + N/Ac)$ (Cracking bending moment)

= (5000x2200^2)/6 x (0.23x24^(2/3) + 10363.01 x 10^3/(5000x2200))

= 11518kN.m

1.7xM=35224.2kN.m ------ Mc < 1.7xM ➔ Need to check Mu and Mc

Mu = C x y1 + T x y2 (Resisting bending moment)

C=0.85σck x 0.8X x b=0.68x24x5000x X=81600X (N)

T=As x σsy =149229.6/2 x 345 =25742106 (N) *no count of side rebers

N=C-T ➔ X=315.60mm

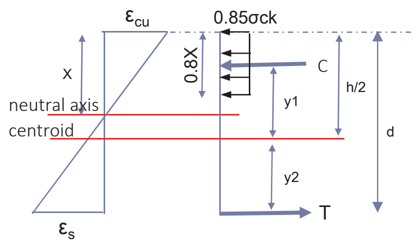
y1=h/2-0.4X=973.76mm

y2=h/2-(h-d)=900mm

Mu=(81600 x 315.60) x 973.76 + 25742106x900

=48245.1kN.m

Hence, Mc ≦ Mu ----OK



ii) A’=Na/(0.008σsa + σca)

= 10363.01x10^3/(0.008x200x1.5+6.5x1.5)=852.92mm2

A’ x 0.8% = 6823mm2 < As=179409.2 ---- OK

4.5 Minimum & maximum reinforcement

continue

iii) As=179409.2mm^2 ≧500 mm2/m x14.4m=7200mm2

Pitch@125mm < 300mm

(2) Maximum reinforcement

i) As =74614.8mm^2 (no count of side rebar, only tension side)

Asb: ultimate equilibrium area of tension bars

Pb=Asb/bd

$=\{0.68 \times \varepsilon_{cu}/(\varepsilon_{cu}+\varepsilon_{sy})-N/(bd\sigma_{ck})\}x\ \sigma_{ck}/\sigma_{sy} + As'/bd \times \sigma_s'/\sigma_{sy}$

={0.68 x 0.0035/(0.0035+345/2x10^5) - 10363.01x10^3/(5000x2000x24)}x 24/345 + 149227.6/2/(5000x2000)x345/345

=0.02868+0.00746

=0.03614

Asb = Pb x bd

= 0.03614 x 5000 x 2000

=361400mm^2

Hence, As ≦ Asb

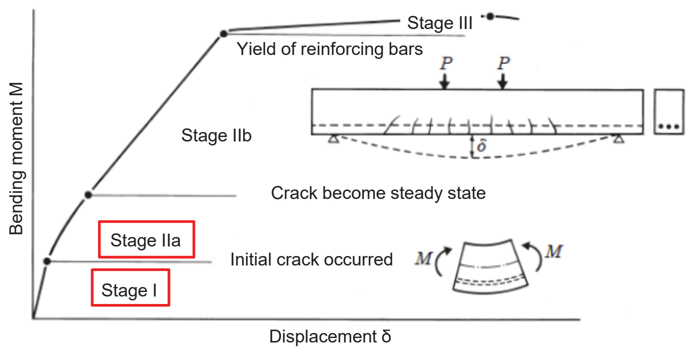
ii) 6% of A=0.06 x 5000 x 2200=660000mm2 > total rebar area 179409.2mm2

Reference

2.Behaivior of RC members

2.1 Behavior of RC beam under bending

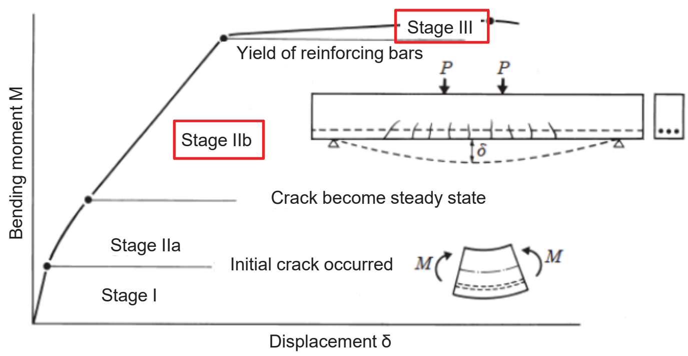
- Stage I： Within initial load, flexural crack is not occurred. The load-displacement curve indicates almost linear.
- Stage II-a：As load increased, flexural crack generated at span center is lengthened and widen gradually. The condition is transferred to that tension force is received by reinforcing bars only.



2.Behaivior of RC members

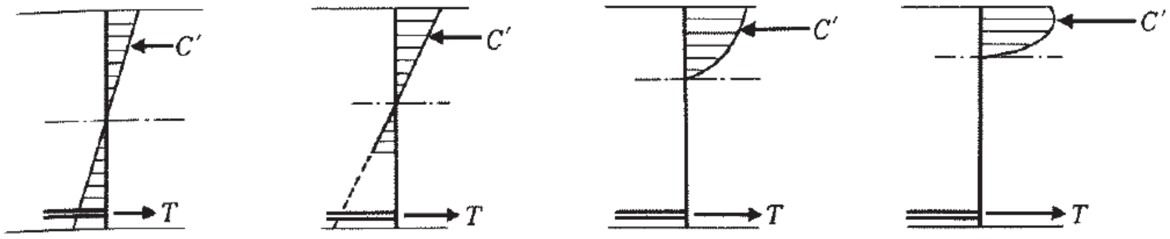
2.1 Behavior of RC beam under bending

- Stage II-b：As load increased, crack is lengthened more. Load-displacement curve is quite linear. The condition continues till yield of reinforcing bars.
- Stage III：Increase of load is quite limited. The fracture occurred either by yielding of reinforcing bars at tension side or crushing of concrete at compressive side.

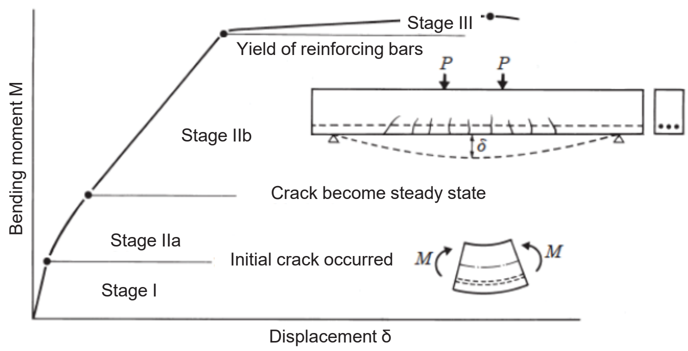


2.Behaivior of RC members

2.1 Behavior of RC beam under bending



Stage I Stage IIa Stage IIb Stage III



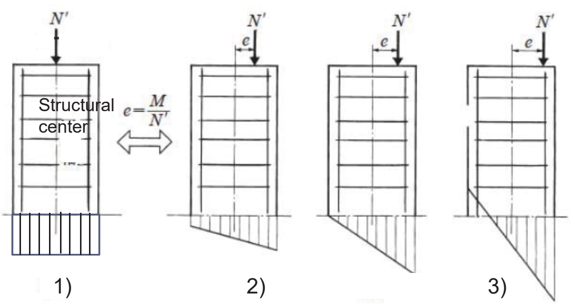
2.Behaivior of RC members

2.2 Behavior of RC column under bending and axial force

1)With axial force worked at structural center, RC section shows all compression condition.

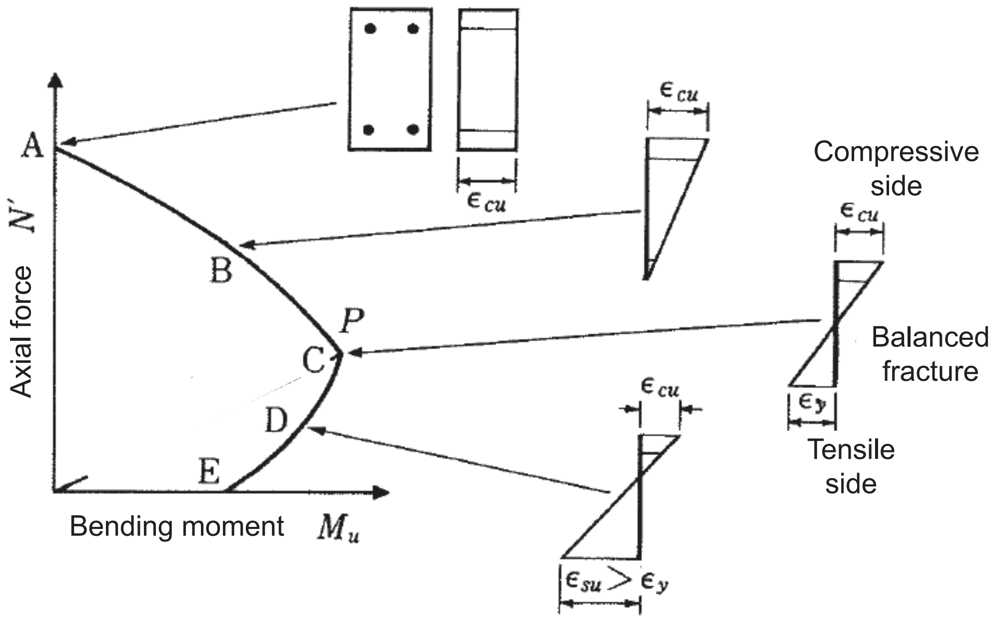
2)With axial force and some bending moment, RC section at lower side keeps compressive condition.

3)As bending moment larger, the condition is transferred to that RC section at lower side is tensile condition.



2.Behaivior of RC members

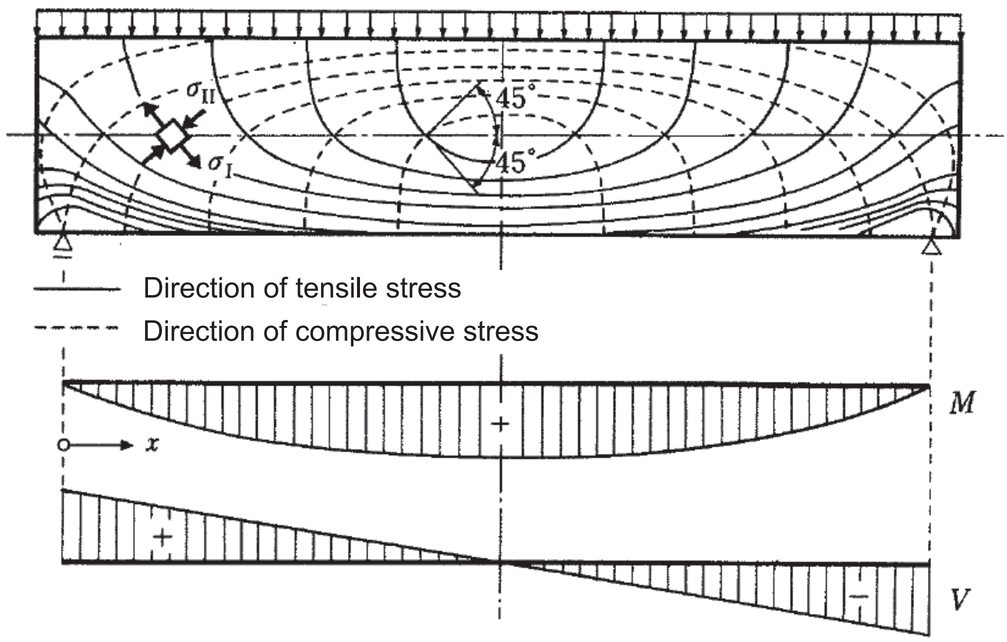
2.2 Behavior of RC column under bending and axial force



2.Behaivior of RC members

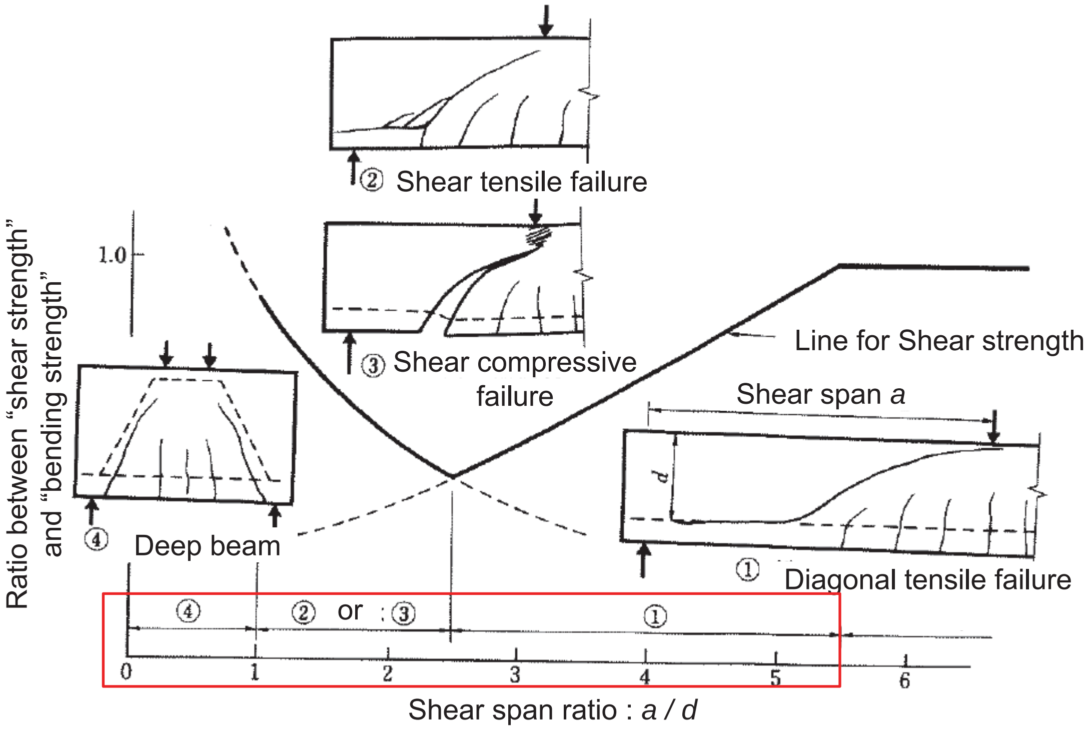
2.3 Behavior of RC beam under shear force

The beam which distributed load is worked.



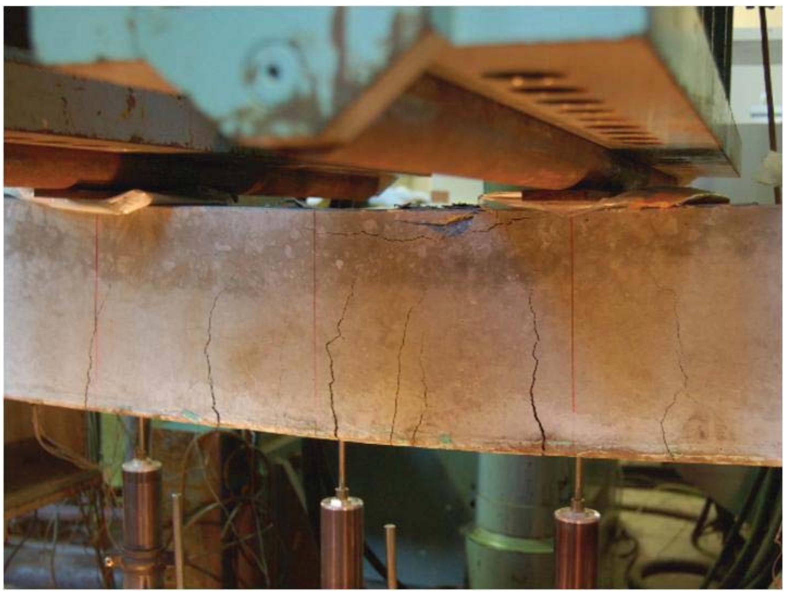
2.3 Behavior of RC beam under shear force

Type of shear failure for each shear span ratio



2.Behaivior of RC members

RC beam with bending cracks



http://arch-env.kanto-gakuin.ac.jp/engineering/2014/12/05/rc造梁の曲げ及びせん断実験<建築構造実験>/

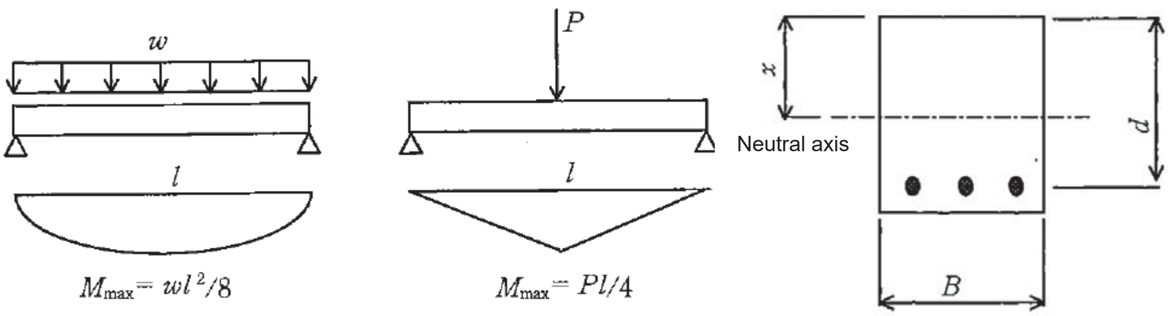
2.Behaivior of RC members

RC beam with shear cracks

http://shimazaki.arch.kanagawa-u.ac.jp/class/Exp/test.html

3.Basis of RC section calculation

3.1 Stress for single reinforced section under bending moment M



Hypothesis for the RC section calculation ;

1) Fiber strain shall be proportional to the distance from the neutral axis.

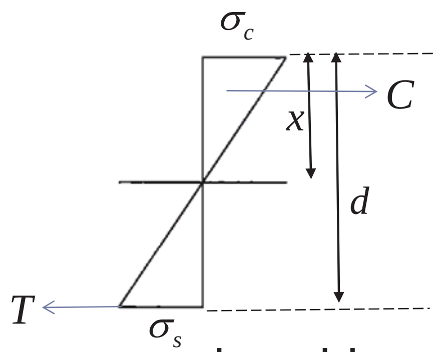
2) The tensile strength of concrete shall be ignored.

3) The Young's modulus ratio of reinforcement to concrete shall be taken to be 15.

3.Basis of RC section calculation

- Fiber strain ε shall be proportional to the distance from neutral axis. (Bernoulli-Euler theory)

$$\frac{\varepsilon_c}{x} = \frac{\varepsilon_s}{d-x}$$



- Resultant force for compression side can be calculated as, $C = \dfrac{\sigma_c B x}{2}$
- Resultant force for tension side can be calculated as, $T = \sigma_s A_s$

3.Basis of RC section calculation

- If axial force does not exist, it can be formulated: $C = T$
- Equilibrium of moment

$$M = CZ' = TZ' \qquad Z' = d - x/3$$

$$\frac{\sigma_c}{E_c}(d-x) = \frac{\sigma_s}{E_s}x \qquad n\sigma_c(d-x) = \sigma_s x$$

$$x = \frac{2A_s}{B}\frac{\sigma_s}{\sigma_c} = \frac{2A_s}{B}\frac{n(d-x)}{x}$$

$$x^2 + 2n\frac{A_s}{B}x - 2n\frac{A_s}{B}d = 0$$

- Position of neutral axis is; $x = \dfrac{nA_s}{B}\left(-1 + \sqrt{1 + \dfrac{2Bd}{nA_s}}\right)$

3.Basis of RC section calculation

- Hence,

$$M = \frac{\sigma_c Bx}{2}(d - \frac{x}{3})$$

- Compressive stress of concrete

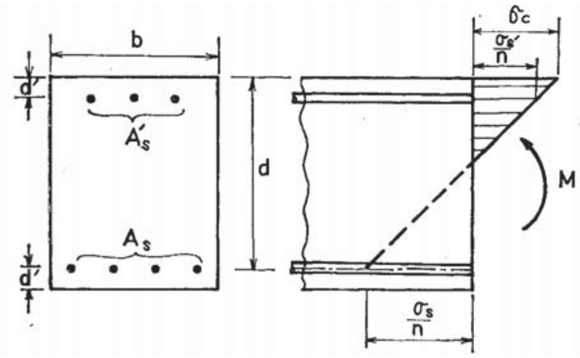
$$\sigma_c = \frac{2M}{Bx(d - \frac{x}{3})}$$

- Tensile stress of reinforcement

$$\sigma_s = \frac{M}{A_s(d - \frac{x}{3})} \quad \text{or} \quad \sigma_s = n \cdot \sigma_c \frac{d - x}{x}$$

3.Basis of RC section calculation

3.2 Stress for doubly reinforced section under bending moment M



$$C = \frac{d}{\sqrt{\frac{M}{b}}}, \quad p = \frac{A_s}{bd}, \quad \alpha = \frac{A_s'}{A_s}$$

$$\sigma_c = \frac{2M}{\left\{k\left(1 - \frac{k}{3}\right) + 2np'(k - f)(1 - f)\frac{1}{k}\right\}bd^2}$$

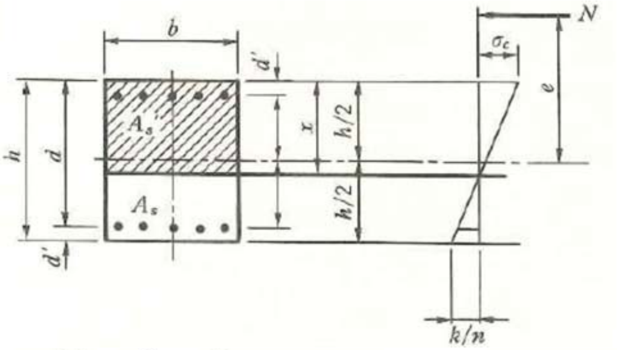
$$\sigma_s = n\sigma_c \frac{1 - k}{k}$$

$$\sigma_s' = n\sigma_c \frac{k - f}{k}$$

$$k = -n(p + p') + \sqrt{n^2(p + p')^2 + 2n(p + p'f)} \qquad f = \frac{d'}{d}$$

3.Basis of RC section calculation

3.3 Stress for doubly reinforced section under bending moment M and axial force N



$$x^3-\left(\frac{h}{2}-e\right)x^2\cdot\frac{6n}{b}\{A_s(e+c)+A_s'(e-c')\}x-\frac{6n}{b}\left\{A_s\left(c+\frac{h}{2}\right)(e+c)+A_s'\left(\frac{h}{2}-c'\right)(e+c')\right\}=0$$

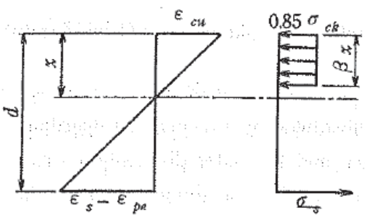
$$\sigma_c=\frac{M}{\frac{b\cdot x}{2}\left(\frac{h}{2}-\frac{x}{3}\right)+\frac{n\cdot A_s'}{x}\cdot c'\left(c'-\frac{h}{2}+x\right)+\frac{n\cdot A_s}{x}\cdot c\left(c+\frac{h}{2}-x\right)}$$

$$\sigma_s=\frac{n\cdot\sigma_c}{x}\left(c+\frac{h}{2}-x\right)$$

Resisting bending moment to failure (Mu)

$N=C-T$ ………… $M_u=C\cdot y_1+T\cdot y_2$

$$\frac{x}{\varepsilon_{cu}}=\frac{d-x}{\varepsilon_s-\varepsilon_{pe}}$$

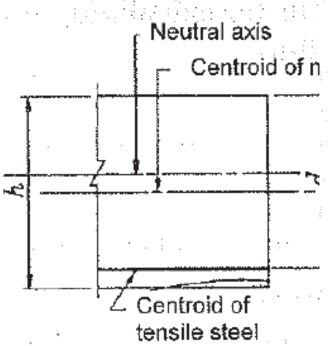


Strain distribution Stress distribution

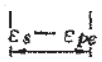
$\beta=0.8$ for $\sigma_{ck}\leqq 50$ N/mm^2

$\beta=0.72$ for $\sigma_{ck}\geqq 60$ N/mm^2

Linear interpolation for $50<\sigma_{ck}<60$ N/mm^2



where
- ε_{cu}: Ultimate strain of concrete
- ε_s: Strain of steel
- σ_{ck}: Design standard strength of concrete (N/mm^2)
- ε_{pe}: Strain of PC tendon due to effective prestressing force
- σ_s: Stress in steel (N/mm^2)
- d: Effective height of member section (mm)
- x: Distance from compression edge to neutral axis (mm)



3.4 Verification against bending moment on beam

Example of Verification against bending moment at C-C Section

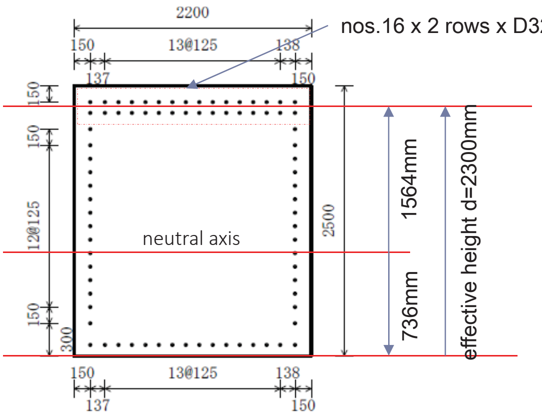
Calculation of σc and σs

Answer Sheet

Item	Unit	Dead Load
Bending Moment in vertical direction M	kN.m	4951.72
Compressive edge～neutral axis x	mm	736
Compressive stress σc Tensile stress σs	N/mm2 N/mm2	2.97 97.79
Increased coefficient α Allowable compressive stress σca Allowable tensile stress σsa	— N/mm2 N/mm2	1.00 8.00 100.00

*Ratio of Modulus of Elasticity Es/Ec=15

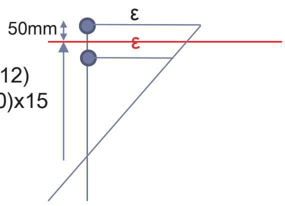
nos.16 x 2 rows x D32 (SD345, A=794.2mm²/rebar)



Moment of Inertia of Converted Concrete Section Area
In=2200 x 736^3/12 + 2200x736x(736/2)^2 + 15x32x794.2x1564^2
= 1.2249 x 10^12 mm²

σc=M/In x y = 4951.72 x 10^6/(1.2249 x 10^12) x736
=2.97 N/mm²

σs=M/In x (y+50) x 15
= 4951.72 x 10^6/(1.2249 x 10^12)
x (1564+50)x15
= 97.79 N/mm²



Example

Answer Sheet

- Section (a-a) : at outer bearing

Item	Unit	Dead Load	Dead+Live	Seismic (horizontal)
S	kN	1839.69	2486.69	334.67
M	kN.m	35.68	35.68	8.92
a	mm	2500	2500	2500
d/1.15	mm	1193	1193	—
d	mm	1371	1371	2050
Sh	kN	1830.03	2477.03	334.67
α	—	1.00	1.00	1.50
pt	%	0.842	0.842	0.296
ce	—	0.944	0.944	0.842
cpt	—	1.405	1.405	0.996
τm	N/mm²	0.607	0.821	0.104
$τa_1$	N/mm²	0.305	0.305	0.294
$τa_2$	N/mm²	1.700	1.700	2.550
σsa	N/mm²	180.00	180.00	—
s	mm	150	150	—
Sca	kN	920.92	920.92	—
Sh'	kN	909.10	1556.10	—
AwReq	mm²	635.28	1087.40	—
Aw	mm²	1146.00	1146.00	—

- Section (b-b) : H/2 from pier front

Dead Load	Dead+Live
1961.21	2608.21
2407.98	3216.73
2500	2500
1596	1596
1836	1836
1473.98	1957.34
1.00	1.00
0.629	0.629
0.875	0.875
1.278	1.278
0.365	0.485
0.257	0.257
1.700	1.700
180.00	180.00
150	150
1037.94	1037.94
436.05	919.41
227.64	479.98
1146.00	1146.00

example of "Dead+Live Load case at a-a section"

Sh=S-M/d x (tanβ+tanγ)=2486.69 – 35.68/1.371 x (1.3/3.5)=2477.03kN

$τ_m$ =S_h/bd =2477.03/(2.2x1.371) =821kN/m² => 0.821N/ mm²

$τ_{a1}$=0.23x α x c_e x c_{pt} = 0.23 x 1.0 x 0.944 x 1.405 =0.305 N/mm²

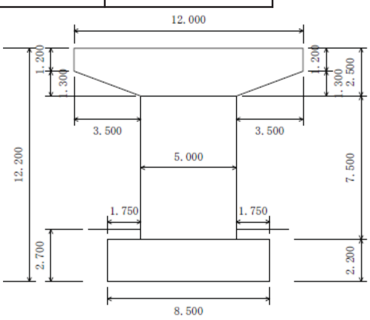
Hence, $τ_m$ > $τ_{a1}$

S_{ca}=$τ_{a1}$ x b x d = 0.305x2200x1371 => 920.92kN

$S_{h'}$ =S_h – S_{ca} =2477.03 – 929.92 =1556.10kN

A_{wreq} = 1.15 x S_h' x s / ($σ_{sa}$x d') (if a<d/1.15, d'=1.15a, if not d'=d)

= 1.15x1556.10x10^3x150/(180x1371) =1087 mm² < A_w: 1146.0 mm²



3.6 Minimum & maximum reinforcement

Example of at C-C Section (at front of pier)

(1) Minimum reinforcement

Answer Sheet

i) Mc = Z_c x σ_{bt} = Z_c x 0.23x σ_{ck} $^{2/3}$ (Cracking bending moment)

= (2200x2500^2)/6 x 0.23x24^(2/3)

= 4385kN.m

Mu = C x y1 + T x y2 (Resisting bending moment)

C=0.85σck x 0.8X x b=0.68x24x2200x X=35904X (N)

T=As x σsy =32x794.2 x 345 =8767968 (N)

C=T ➔ X=244.2mm

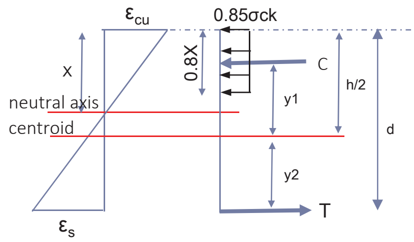
y1=h/2-0.4X=1152.32mm

y2=h/2-(h-d)=1050mm

Mu=(35904 x 244.2) x 1152.32 + 8767968 x 1050

=19310 kN.m

Hence, Mc ≦ Mu



ii) Not applied (beam member)

iii) As=32 x 794.2=25414.4mm^2 ≧500 mm2/m x2.2m=1100mm2

Pitch@125mm < 300mm

3.6 Minimum & maximum reinforcement

Example of at C-C Section (at front of pier)

(2) Maximum reinforcement

Answer Sheet

As =32 x 794.2 =25414 mm^2

Asb: ultimate equilibrium area of tension bars

Pb=Asb/bd

={0.68 x ε_{cu}/(ε_{cu}+ε_{sy})}x σ_{ck}/σ_{sy} + As'/bd x σ_s'/σ_{sy}

=(0.68 x 0.0035(0.0035+345/2x10^5)x 24/345 + 16x794.2/(2200x2300)x345/345

=0.0317+0.0025

=0.0342

Asb = Pb x bd

= 0.0342 x 2200 x 2300

=173052mm^2

Hence, As ≦ Asb