6. STUDY ON WALING AND STRUTS

6-1 STUDY ON WALING

6-1-1 Section force

(1) Axial force

Axial force is calculated with the following formula.

$$N = R \cdot B + Nt$$

where,

R : Support reaction (refer section 5-3-2)

B : Assignment width of axial force (= 1.43 m)

Nt : Effect of temperature (= 0.0)

(2) Bending moment

Bending moment is calculated with the following formula.

$$M = \frac{R \cdot L^2}{8}$$

where,

L : Calculation span (= 4.0 m)

R : Support reaction (refer section 5-3-2)

(3) Shearing force

Shearing force is calculated with the following formula.

$$S = \frac{R \cdot L}{2}$$

6-1-2 Check on stress

(1) Axial compressive stress

Axial stress is calculated with the following formula.

$$\sigma c = \frac{N}{A}$$

where,

N : Axial force (N)

A : Section area of wale (mm^2)

(2) Bending stress

Bending stress is calculated with the following formula.

$$\sigma bcy = \frac{M}{Z}$$

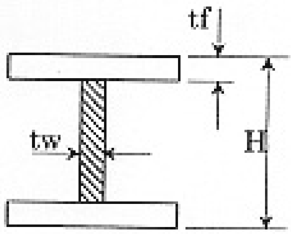
where,

M : Bending moment (N mm)

Z : Section modulus of wale (mm^3)

(3) Shearing stress

Shearing stress is calculated with the following formula.



$$\tau = \frac{S}{Aw}$$

where,

S : Shearing force (N)

Aw : Section area of wale for shearing (mm^2)

$= (H - 2 \cdot tf) \cdot tw$

(4) Check on Buckling

Bucking is checked with the following formula.

$$\frac{\sigma c}{\sigma caz} + \frac{\sigma bcy}{\sigma bagy \cdot (1 - \sigma c / \sigma eay)} \le 1.0 \quad \text{---------- formula 1}$$

$$\sigma c + \frac{\sigma bcy}{(1 - \sigma c / \sigma eay)} \le \sigma cal \quad \text{------------------------ formula 2}$$

where,

σc : Axial compressive stress (N / mm^2)

σbc : Bending stress (N / mm^2)

σcaz : Allowable axial compressive stress (N / mm^2)

$l / rz \leqq 18$ $\sigma caz = 210$

$18 < l / rz \leqq 92$ $\sigma caz = 140 - 0.82 (l / r - 18) \times 1.5$

$92 < l / rz$ $\sigma caz = \dfrac{1{,}200{,}000}{6{,}700 + (l / r)^2} \times 1.5$

where, l : Buckling length (mm)

rz : Radius of gyration of z-section (mm)

$\sigma bagy$: Allowable axial compressive stress (N / mm^2)

$lb / b \leqq 4.5$ $\sigma bagy = 210$

$4.5 < lb / r \leqq 30$ $\sigma bagy = 140 - 2.4 (lb / b - 4.5) \times 1.5$

where, lb : Fixed distance of flange (mm)

r : Width of flange (mm)

σeay : Euler's buckling stress (N / mm^2)

$$\sigma eay = \frac{1{,}200{,}000}{(ly / ry)^2}$$

where, ly : Buckling length (mm)

ry : Radius of gyration of y-section (mm)

σcal : Allowable local buckling stress (N / mm^2)

6-1-3 Checking results

(1) Position and support reaction of wale

Wale position		R (kN/m)
Level 1 wale	GL – 1.50 m	123.7
Level 2 wale	GL – 4.50 m	153.1
Level 3 wale	GL – 7.50 m	345.4
Level 4 wale	GL – 9.80 m	217.1
Level 5 wale	GL – 11.80 m	474.7
Level 6 wale	GL – 14.20 m	430.8
Level 7 wale	GL – 16.40 m	472.4
Level 8 wale	GL – 18.60 m	156.2

(2) Section force

No.	R (kN/m)	B (m)	Nt (kN/m^2)	N (kN)	L (m)	M (kN m)	S (kN)
1	123.7	1.43	0.0	177	4.0	247	247
2	153.1	1.43	0.0	219	4.0	306	306
3	345.4	1.43	0.0	253	4.0	354	354
4	217.1	1.43	0.0	310	4.0	434	434
5	474.7	1.43	0.0	339	4.0	475	475
6	430.8	1.43	0.0	308	4.0	330	330
7	472.4	1.43	0.0	338	4.0	472	472
8	156.2	1.43	0.0	223	4.0	312	312

(3) Stress

No.	Specification of wale	A (cm^2)	Z (cm^3)	Aw (cm^2)	σc (N/mm^2)	σbcy (N/mm^2)	τ < 120 (N/mm^2)
1	H-350x350x12x19	172	2280	37.4	10	109	66 OK
2	H-350x350x12x19	172	2280	37.4	13	134	82 OK
3	2H-350x350x12x19	172	2280	37.4	15	155	95 OK
4	H-400x400x13x21	219	1.43	15.0	14	130	93 OK
5	2H-400x400x13x21	219	1.43	15.0	16	143	102 OK
6	2H-400x400x13x21	219	1.43	15.0	14	99	81 OK
7	2H-400x400x13x21	219	1.43	15.0	15	142	102 OK
8	H-350x350x12x19	172	1.43	15.0	13	137	83 OK

(4) Check on buckling

No.	σc N/mm^2	σbcy N/mm^2	L/rz	σcaz N/mm^2	Lb/r	$\sigma bagy$ N/mm^2	Ly/ry	σeay N/mm^2	σcal N/mm^2	formula 1	formula 2	
1	10	109	45	177	11.4	185	26.3	1733	210	0.65	119	OK
2	13	134	45	177	11.4	185	26.3	1733	210	0.80	148	OK
3	15	155	45	177	11.4	185	26.3	1733	210	0.93	172	OK
4	14	130	40	183	10.0	190	22.9	2297	210	0.77	145	OK
5	16	143	40	183	10.0	190	22.9	2297	210	0.84	159	OK
6	14	99	40	183	10.0	190	22.9	2297	210	0.60	114	OK
7	15	142	40	183	10.0	190	22.9	2297	210	0.84	158	OK
8	13	137	45	177	11.4	185	26.3	1733	210	0.82	151	OK
										<1.0	$<\sigma cal$	

6-2 STUDY ON STRUTS

6-2-1 Section force

(1) Axial force

Axial force is calculated with the following formula.

$$N = R \cdot B + Nt$$

where,

R : Support reaction (refer section 5-3-2)

B : Assignment width of axial force (= 5.0 m)

Nt : Effect of temperature (= 150 kN/m^2 ,refe section 2-3-5 ;T_1)

(2) Bending moment

Bending moment is calculated with the following formula.

$$M = \frac{w \cdot L^2}{8}$$

where,

L : Calculation span (= 4.15 m)

w : Vertical load on strut (= 5.0 kN/m^2 , refer section 2-3-5; T_2)

6-2-2 Check on stress

(1) Axial compressive stress

Axial stress is calculated with the following formula.

$$\sigma c = \frac{N}{A}$$

where,

N : Axial force (N)

A : Section area of strut (mm^2)

(2) Bending stress

Bending stress is calculated with the following formula.

$$\sigma bcy = \frac{M}{Z}$$

where,

M : Bending moment (N mm)

Z : Section modulus of strut (mm^3)

(3) Check on Buckling

Bucking is checked with the following formula.

$$\frac{\sigma c}{\sigma caz} + \frac{\sigma bcy}{\sigma bagy \cdot (1 - \sigma c / \sigma eay)} \leq 1.0 \quad \text{---------- formula 1}$$

$$f c Ⓓ \frac{\sigma bcy}{(1 - \sigma c / \sigma eay)} \leq \sigma cal \quad \text{------------------------ formula 2}$$

where,

σc : Axial compressive stress (N / mm^2)

σbc : Bending stress (N / mm^2)

σcaz : Allowable axial compressive stress (N / mm^2)

$l / rz \leqq 18$	$\sigma caz = 210$
$18 < l / rz \leqq 92$	$\sigma caz = 140 - 0.82\ (l / r - 18) \times 1.5$
$92 < l / rz$	$\sigma caz = \dfrac{1{,}200{,}000}{6{,}700 + (l / r)^2} \times 1.5$

where, l : Buckling length (mm)

rz : Radius of gyration of z-section (mm)

$\sigma bagy$: Allowable axial compressive stress (N / mm^2)

$lb / b \leqq 4.5$	$\sigma bagy = 210$
$4.5 < lb / r \leqq 30$	$\sigma bagy = 140 - 2.4\ (lb / b - 4.5) \times 1.5$

where, lb : Fixed distance of flange (mm)

r : Width of flange (mm)

σeay : Euler's buckling stress (N / mm^2)

$$\sigma eay = \frac{1{,}200{,}000}{(ly / ry)^2}$$

where, ly : Buckling length (mm)

ry : Radius of gyration of y-section (mm)

σcal : Allowable local buckling stress (N / mm^2)

6-2-3 Checking results

(1) Position and support reaction of strut

Strut position		R (kN/m)
Level 1 strut	GL – 1.50 m	123.7
Level 2 strut	GL – 4.50 m	153.1
Level 3 strut	GL – 7.50 m	345.4
Level 4 strut	GL – 9.80 m	217.1
Level 5 strut	GL – 11.80 m	474.7
Level 6 strut	GL – 14.20 m	430.8
Level 7 strut	GL – 16.40 m	472.4
Level 8 strut	GL – 18.60 m	156.2

(2) Section force

No.	R (kN/m)	B (m)	Nt (kN/m²)	N (kN)	L (m)	M (kN m)
1	123.7	5.00	150	769	4.15	10.8
2	153.1	5.00	150	915	4.15	10.8
3	345.4	5.00	150	1036	4.15	10.8
4	217.1	5.00	150	1235	4.15	10.8
5	474.7	5.00	150	1337	4.15	10.8
6	430.8	5.00	150	1227	4.15	10.8
7	472.4	5.00	150	1331	4.15	10.8
8	156.2	5.00	150	931	4.15	10.8

(3) Stress

No.	Specification of wale	A (cm²)	Z (cm³)	σc (N/mm²)	obey (N/mm²)
1	H-300x300x10x15	118	1350	65	8
2	H-300x300x10x15	118	1350	77	8
3	2H-300x300x10x15	118	1350	88	8
4	H-300x300x10x15	118	1350	104	8
5	2H-300x300x10x15	118	1350	113	8
6	2H-300x300x10x15	118	1350	104	8
7	2H-300x300x10x15	118	1350	112	8
8	H-300x300x10x15	118	1350	79	8

(4) Check on buckling

No.	σc N/mm²	σbcy N/mm²	L/rz	σcaz N/mm²	Lb/r	σbagy N/mm²	Ly/ry	σeay N/mm²	σcal N/mm²	formula 1	formula 2	
1	65	8	55	165	13.8	176	31.7	1196	210	0.44	73	OK
2	77	8	55	165	13.8	176	31.7	1196	210	0.52	86	OK
3	88	8	55	165	13.8	176	31.7	1196	210	0.58	96	OK
4	104	8	55	165	13.8	176	31.7	1196	210	0.68	113	OK
5	113	8	55	165	13.8	176	31.7	1196	210	0.74	122	OK
6	104	8	55	165	13.8	176	31.7	1196	210	0.68	112	OK
7	112	8	55	165	13.8	176	31.7	1196	210	0.73	121	OK
8	79	8	55	165	13.8	176	31.7	1196	210	0.53	87	OK
										<1.0	<σcal	

6-3 STUDY ON BRACE

6-3-1 Section force

(1) Axial force

Axial force is calculated with the following formula.

$N = 1/2 \cdot (L1+L2) \cdot R / \cos\theta + Nt$

where,

R : Support reaction (maximum = 237 kN/m, No.5 level, refer section 5-3-2)

L1,L2 : Assignment width of axial force (L1 = 1.0 m, L2 = 3.0 m)

θ : Installation angle (= 45°)

Nt : Effect of temperature (= 150 kN/m^2 ,refe section 2-3-5 ;T_1)

$= 1/2 \cdot (1.0+3.0) \times 237 / \cos 45 + 150$

$= 821$ kN

(2) Bending moment

Bending moment is calculated with the following formula.

$$M = \frac{w \cdot L^2}{8}$$

where,

L : Calculation span (= 4.15 m)

w : Vertical load on strut (= 5.0 kN/m^2 , refer section 2-3-5; T_2)

$$= \frac{5.0 \times 4.15^2}{8}$$

$= 1.24$ kN m

6-3-2 Check on stress

(1) Axial compressive stress

Axial stress is calculated with the following formula.

$$\sigma c = \frac{N}{A}$$

where,

N : Axial force (= 821 kN)

A : Section area of strut (=11,840 mm^2 ; H-300x300x10x15)

$$= \frac{821 \times 10^3}{11{,}840}$$

$= 69$ N/mm^2 $\leq$ 210 N/mm^2 OK

(2) Bending stress

Bending stress is calculated with the following formula.

$$\sigma bcy = \frac{M}{Z}$$

where,

M : Bending moment (=1.24 kN m)

Z : Section modulus of strut (1,350,000 mm^3)

$$= \frac{1.24 \times 10^6}{1,350,000}$$

$$= 1 \ N/mm^2$$

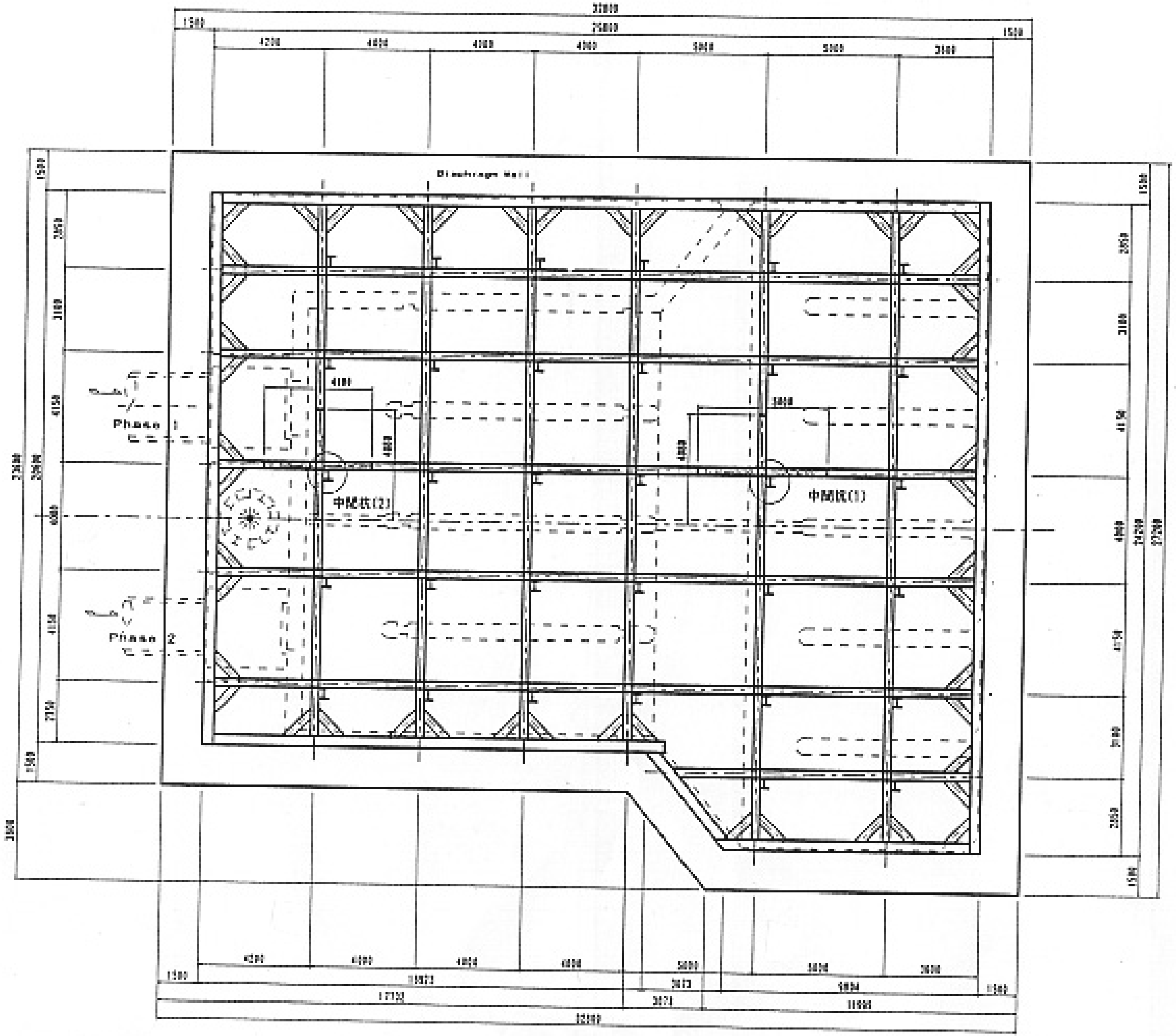
(3) Composition stress

$\sigma c + \sigma bcy \quad < 210 \ N/mm^2$

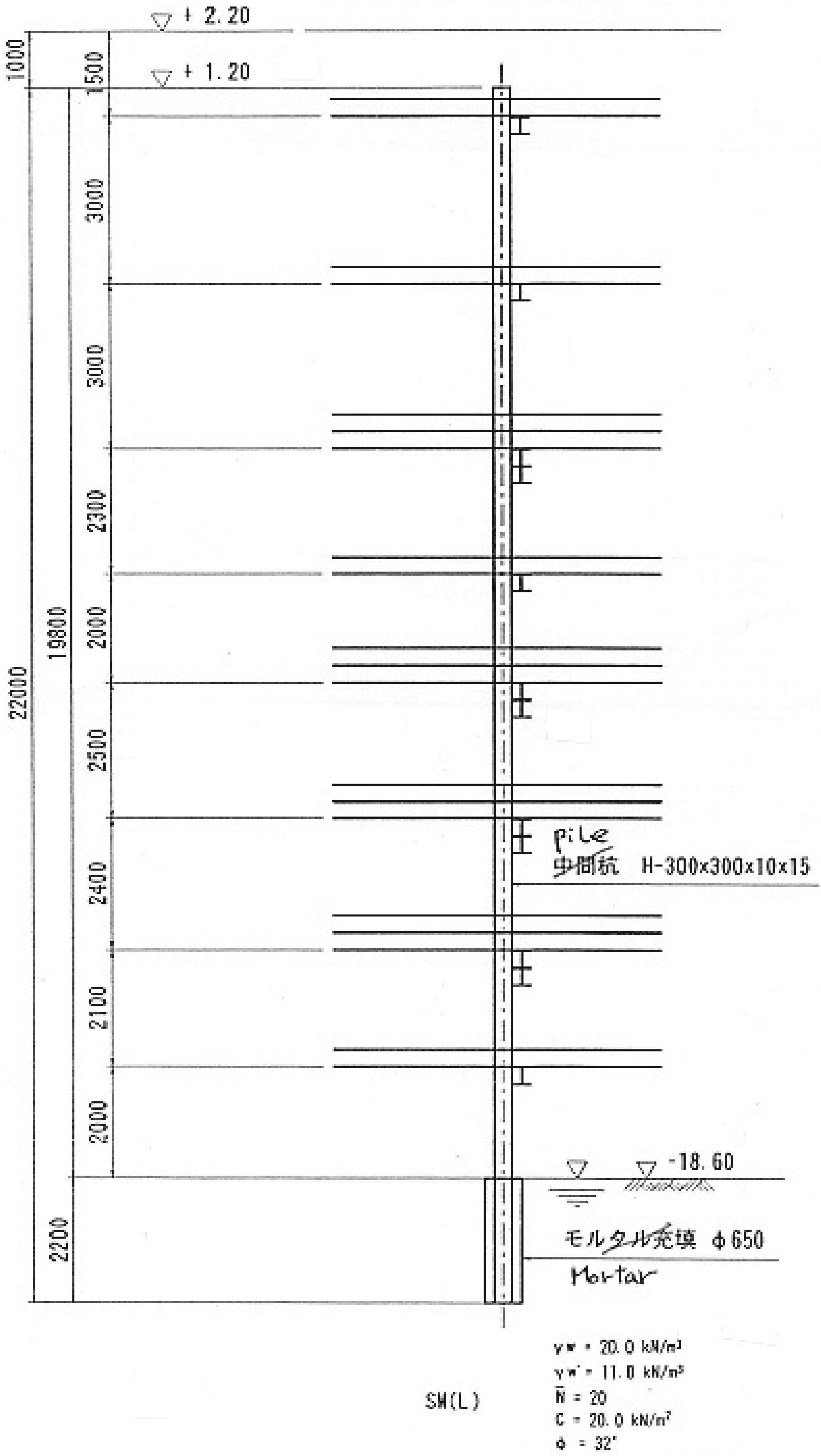
$69.0 + 1.0 \quad = 70 \ N/mm^2 \quad < \quad 210 \ N/mm^2$ OK

7. CHECK ON INTERMEDIATE PILES

7-1 SHAPE AND DIMENSIONS

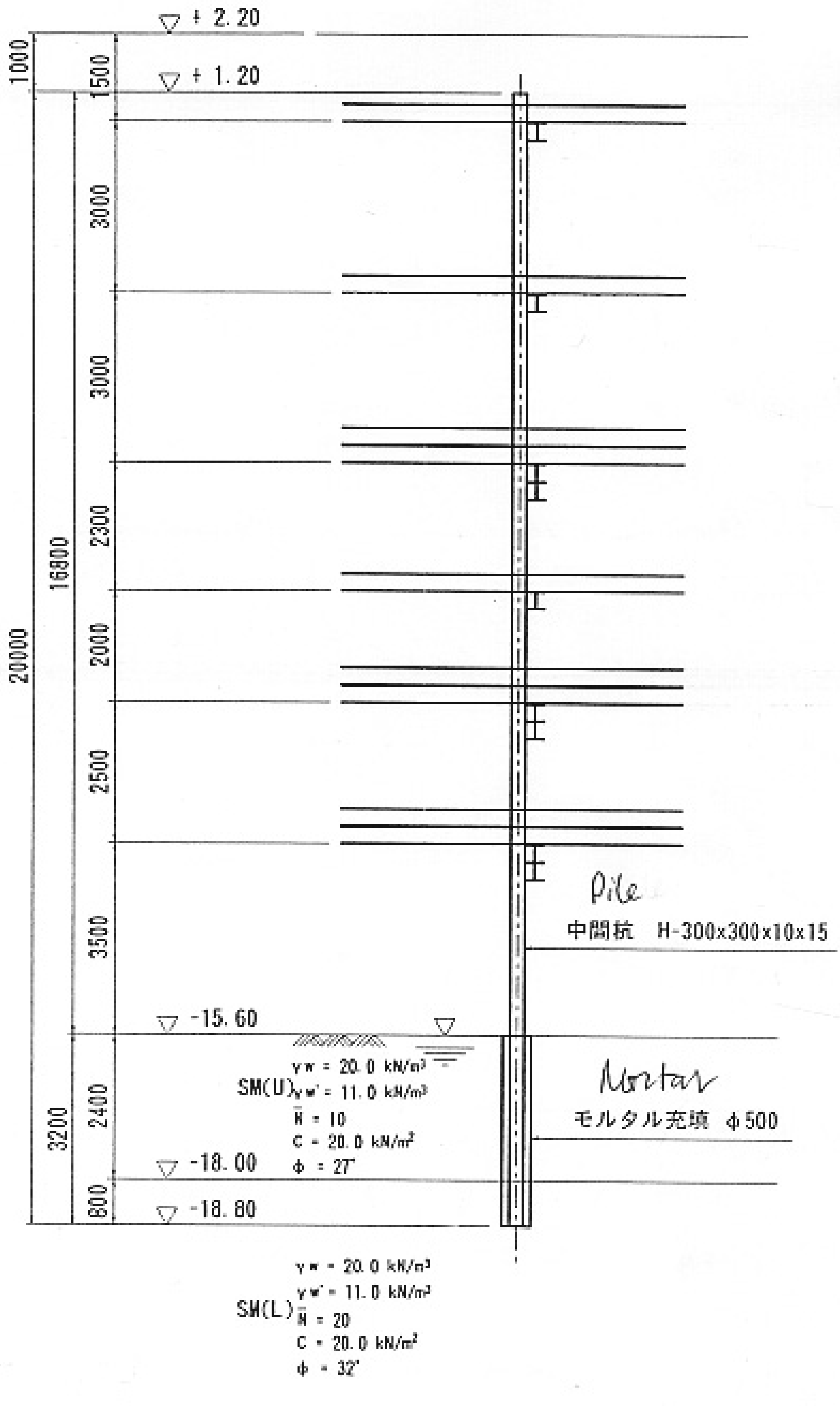


(1) No.1 pile



+ 2.20
+ 1.20
1000
1500
3000
3000
2300
2000
2500
2400
2100
2000
19800
22000
2200
pile
中間杭 H-300x300x10x15
-18.60
モルタル充填 φ650
Mortar
SM(L)
γw = 20.0 kN/m³
γw' = 11.0 kN/m³
N̄ = 20
C = 20.0 kN/m²
φ = 32°

(2) No.2 pile



▽ + 2.20
▽ + 1.20
1000
1500
3000
3000
2300
2000
2500
3500
16800
20000
Pile
中間杭 H-300x300x10x15
▽ -15.60
SM(U)
γw = 20.0 kN/m³
γw' = 11.0 kN/m³
N̄ = 10
C = 20.0 kN/m²
φ = 27°
Mortar
モルタル充填 φ500
3200
2400
▽ -18.00
800
▽ -18.80
SM(L)
γw = 20.0 kN/m³
γw' = 11.0 kN/m³
N̄ = 20
C = 20.0 kN/m²
φ = 32°

7-2 STUDY ON No.1 PILE

7-2-1 Loading

(1) Vertical load by buckling

Vertical load by buckling is calculated with the following formula.

$N1 = 1/50 \cdot \Sigma(R \cdot L1 + Nt + R \cdot L2 + Nt)$

where, R : Support reaction (refer, section 5-3-2)

L1, L2 : Assignment width of axial force of both directions

L1 = 5.0 m, L2 = 4.08 m

Nt : Effect of temperature (=150 kN/m^2, refer section 2-3-5; T_1)

Strut No.	R (kN/m)	L1 (m)	L2 (m)	Nt (kN)	Σ(R·L1+Nt+R·L2+Nt) (kN)
1	124	5.0	4.08	150	1,423
2	153	5.0	4.08	150	1,690
3	354	5.0	4.08	150	3,518
4	217	5.0	4.08	150	2,271
5	475	5.0	4.08	150	4,610
6	430	5.0	4.08	150	4,212
7	472	5.0	4.08	150	4,590
8	156	5.0	4.08	150	1,718
Total					24,032

N1 = 1 / 50 x 24,032 = 481 kN

(2) Vertical load by self-weight of strut

Vertical load by self-weight of strut is calculated with the following formula.

$N2 = \Sigma(w \cdot L1 + w \cdot L2)$

where, w : Self-weight of strut (=5.0 kN/m^2, refer section 2-3-5; T_2)

L1, L2 : Assignment width of axial force of both directions

L1 = 5.0 m, L2 = 4.08 m

N2 = (5.0 x 5.0 + 5.0 x 4.08) x 8 = 363 kN

(3) Vertical load by self-weight of pile

Vertical load by self-weight of pile is calculated with the following formula.

$N3 = W \cdot Lp$

where, W : Unit weight of strut (=0.91 kN/m, H-300x300x10x15)

Lp : Length of pile (= 22.0 m)

N3 =0.91 x 22.0 = 20 kN

(4) Total vertical load

N = N1 + N2 + N3

= 481 + 363 +20 =864 kN

7-2-2 Check on stress

(1) Axial compressive stress

Axial stress is calculated with the following formula.

$$\sigma c = \frac{N}{A}$$

where,

N : Axial force (= 864,000 N)

A : Section area of wale (11,840 mm^2)

$$= \frac{864,000}{11,840}$$

$= 73\ N/mm^2$

(2) Allowable axial compressive stress

Allowable axial compressive stress is calculated with the following formula.

$l / rz \leqq 18$	$\sigma caz = 210$
$18 < l / rz \leqq 92$	$\sigma caz = 140 - 0.82\ (l / r - 18)\ x1.5$
$92 < l / rz$	$\sigma caz = \dfrac{1,200,000}{6,700 + (l / r)^2} \times 1.5$

where, l : Buckling length (= 3,000 mm)

rz : Radius of gyration of z-section (75.5 mm; H-300x300x10x15)

$l / rz = 3,000 / 75.5$

$= 39.7$

therefor,

$\sigma caz = 140 - 0.82\ (l / r - 18)\ x1.5$

$= 140 - 0.82\ x\ (39.7 - 18)\ x\ 1.5$

$= 183\ N/mm^2 \quad > \quad \sigma c = 73\ N/mm^2$ OK

7-2-3 Check on bearing capacity

Allowable bearing capacity of pile is calculated with the following formula.

$Ra = Ru / Fs$

where,

Ru : Ultimate bearing capacity (kN)

$= \alpha \cdot N \cdot Ap$

α : Coefficient by construction method (= 300; preboring and mortar method)

N : N-value of ground (= 20)

Ap : Section area of pipe

$= \pi / 4 \cdot D^2 = \pi / 4 \times 0.65^2 = 0.332\ m^2$

$= 300 \times 20 \times 0.332$

$= 1991$ kN

Fs : Safety factor (= 2.0)

$= 1991 / 2$

$= 996$ kN $>$ $N = 864$ kN OK

7-3 STUDY ON No.2 PILE

7-3-1 Loading

(1) Vertical load by buckling

Vertical load by buckling is calculated with the following formula.

$N1 = 1 / 50 \cdot \Sigma(R \cdot L1 + Nt + R \cdot L2 + Nt)$

where, R : Support reaction (refer, section 5-3-2)

L1, L2 : Assignment width of axial force of both directions

$L1 = 4.10$ m, $L2 = 4.08$ m

Nt : Effect of temperature (=150 kN/m^2, refer section 2-3-5; T_1)

Strut No.	R (kN/m)	L1 (m)	L2 (m)	Nt (kN)	Σ(R·L1+Nt+R·L2+Nt) (kN)
1	124	4.10	4.08	150	1,312
2	153	4.10	4.08	150	1,552
3	354	4.10	4.08	150	3,199
4	217	4.10	4.08	150	2,076
5	475	4.10	4.08	150	4,183
6	430	4.10	4.08	150	3,824
Total					16,146

$N1 = 1 / 50 \times 16{,}146 \quad = 323$ kN

(2) Vertical load by self-weight of strut

Vertical load by self-weight of strut is calculated with the following formula.

$N2 = \Sigma(w \cdot L1 + w \cdot L2)$

where, w : Self-weight of strut (=5.0 kN/m^2, refer section 2-3-5; T_2)

L1, L2 : Assignment width of axial force of both directions

$L1 = 4.10$ m, $L2 = 4.08$ m

$N2 = (4.10 \times 5.0 + 5.0 \times 4.08) \times 6 \quad = 245$ kN

(3) Vertical load by self-weight of pile

Vertical load by self-weight of pile is calculated with the following formula.

$N3 = W \cdot Lp$

where, W : Unit weight of strut (=0.91 kN/m, H-300x300x10x15)

Lp : Length of pile (= 20.0 m)

$N3 = 0.91 \times 20.0 \quad = 18.2$ kN

(4) Total vertical load

$N = N1 + N2 + N3$

$= 323 + 245 + 18.2$

$= 464$ kN

7-3-2 Check on stress

(2) Axial compressive stress

Axial stress is calculated with the following formula.

$$\sigma c = \frac{N}{A}$$

where,

N : Axial force (= 464,000 N)

A : Section area of wale (11,840 mm^2)

$$= \frac{464,000}{11,840}$$

$= 39$ N/mm^2

(2) Allowable axial compressive stress

Allowable axial compressive stress is calculated with the following formula.

$l / rz \leqq 18$	$\sigma caz = 210$
$18 < l / rz \leqq 92$	$\sigma caz = 140 - 0.82\,(l / r - 18)\,x1.5$
$92 < l / rz$	$\sigma caz = \dfrac{1,200,000}{6,700 + (l/r)^2} \times 1.5$

where, l : Buckling length (= 3,000 mm)

rz : Radius of gyration of z-section (75.5 mm; H-300x300x10x15)

$l / rz = 3,000 / 75.5$

$= 39.7$

therefor,

$\sigma caz = 140 - 0.82\,(l / r - 18)\,x1.5$

$= 140 - 0.82 \times (39.7 - 18) \times 1.5$

$= 183$ N/mm^2 $>$ $\sigma c = 39$ N/mm^2 OK

7-3-3 Check on bearing capacity

Allowable bearing capacity of pile is calculated with the following formula.

$Ra = Ru / Fs$

where,

Ru : Ultimate bearing capacity (kN)

$= \alpha \cdot N \cdot Ap$

α : Coefficient by construction method (= 300; preboring and mortar method)

N : N-value of ground (= 20)

Ap : Section area of pipe

$= \pi / 4 \cdot D^2 = \pi / 4 \times 0.50^2 = 0.196\ m^2$

$= 300 \times 20 \times 0.196$

$= 1176$ kN

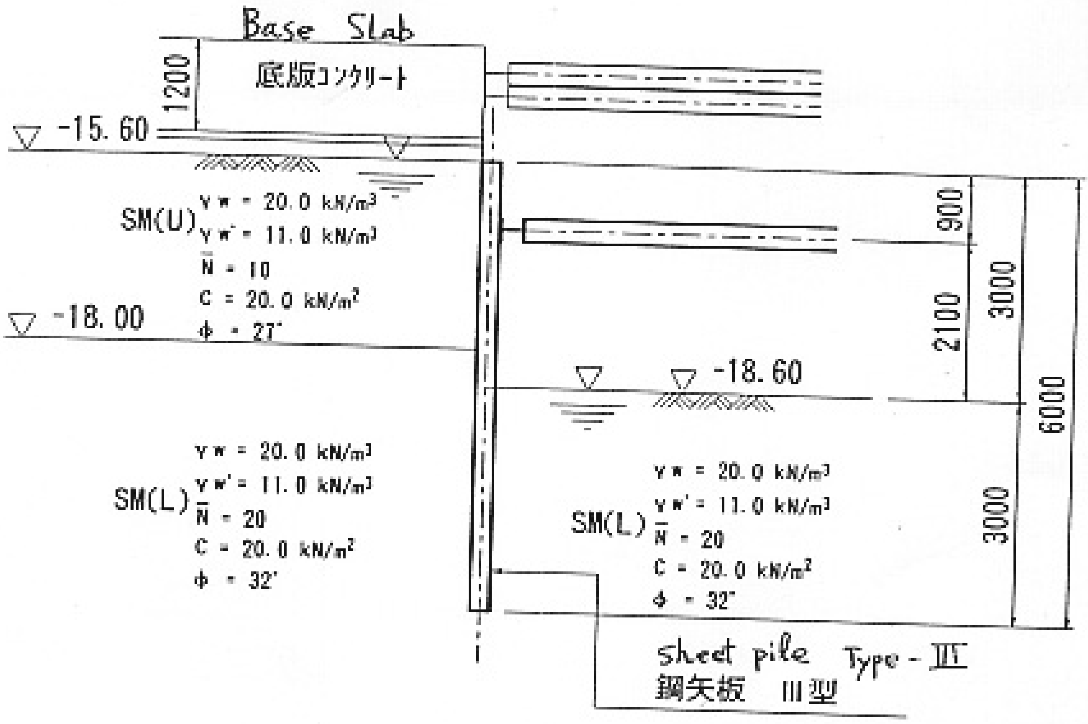
Fs : Safety factor (= 2.0)

$= 1176 / 2$

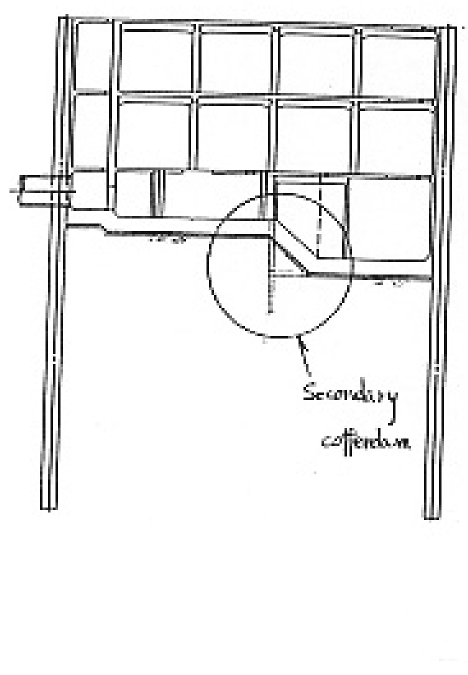
$= 588$ kN $>$ N = 464 kN OK

8. STUDY ON SCONDARY COFFERDAM

8-1 SHAPE AND DIMENSIONS



KEY MAP



8-2 STUDY ON PENETRATION LENGTH BY EARTH PRESSURE BALANCE

8-2-1 Loading

The required penetration length decided from a balance of moment by the soil pressure of bottom from the strut. The ground water pressure is assumed zero at the tip of wall.

The pressure that acted on the wall was calculated with the formula of the section 2-3-4. Excavation level of this case is GL – 1.4 m.

(1) Active side

Active earth pressure is calculated with the following formula.

$$P_H = (P_V + q)\cdot k_{oa} - 2\cdot c\cdot\sqrt{k_{oa}}$$

$$k_{oa} = \tan^2(45 - \phi/2)$$

where, ϕ : Effective internal friction angle

q : Surcharge load (Take self-weight of slab into consideration.)

=1.2 x 24.5 = 29.4 kN/m²

depth thickness of layer

No	深さ z G.L. -m	層厚 h m	γ kN/m³	φ 度	c kN/m²	Σγh+q kN/m²	Ka	pa kN/m²
1	0.00 0.90	0.90	11.0	27.0	20.0 20.0	29.40 39.30	0.37552	0.00 0.00
2	0.90 2.40	1.50	11.0	27.0	20.0 20.0	39.30 55.80	0.37552	0.00 0.00
3	2.40 3.00	0.60	11.0	32.0	20.0 20.0	55.80 62.40	0.30726	0.00 0.00
4	3.00 3.36	0.36	11.0	32.0	20.0 20.0	62.40 66.36	0.30726	0.00 1.17

depth thickness of layer horizontal force length of arm bending moment

Moment by active earth pressure

No	深さ z G.L. -m	層厚 h m	pa kN/m²	水平力 Pa kN	アーム長 y m	モーメント Ma kN・m
2	0.90 2.40	1.50	0.00 0.00	0.00 0.00	0.50 1.00	0.00 0.00
3	2.40 3.00	0.60	0.00 0.00	0.00 0.00	1.70 1.90	0.00 0.00
4	3.00 3.36	0.36	0.00 1.17	0.00 0.21	2.22 2.34	0.00 0.49
Σ				0.21		0.49

Ground water pressure load is calculated with the following formula.

$$P_w = \gamma_w \cdot z_w$$

Where:

γ_w ... unit weight of water (=10.0 kN/m^3)

z_w ... depth below water table for the load case considered

depth — thickness of layer — horizontal force — length of arm — bending moment

No	深さ z G. L. -m	層厚 h m	pw kN/m²	水平力 Pw kN	アーム長 y m	モーメント Mw kN・m
1	0. 90 3. 00	2. 10	9. 00 30. 00	9. 45 31. 50	0. 70 1. 40	6. 62 44. 10
2	3. 00 3. 36	0. 36	30. 00 0. 00	5. 40 0. 00	2. 22 2. 34	11. 99 0. 00
Σ				46. 35		62. 70

Total horizontal pressure at active side Pa = 0.21 + 46.4 = 46.6 kN

Total moment at active side Ma = 0.49 + 62.7 = 63.2 kN m

(2) Passive pressure

$$P_H = (P_V + q) \cdot k_{oa} + 2 \cdot c \cdot \sqrt{k_{oa}}$$

$k_{oa} = \tan^2 (45 + \phi/2)$

where, ϕ : Effective internal friction angle

q : Surcharge load (= 0.0 kN/m^2)

pp＝（q＋Σγh）Kp＋２c√（Kp），Kp＝tan²（45＋φ／2）

depth — thickness of layer

No	深さ z G. L. -m	層厚 h m	γ kN/m³	φ 度	c kN/m²	Σγh＋q kN/m²	Kp	pp kN/m²
4	3. 00 3. 36	0. 36	11. 0	32. 0	20. 0 20. 0	0. 00 3. 96	3. 25459	72. 16 85. 05

Moment by passive pressure

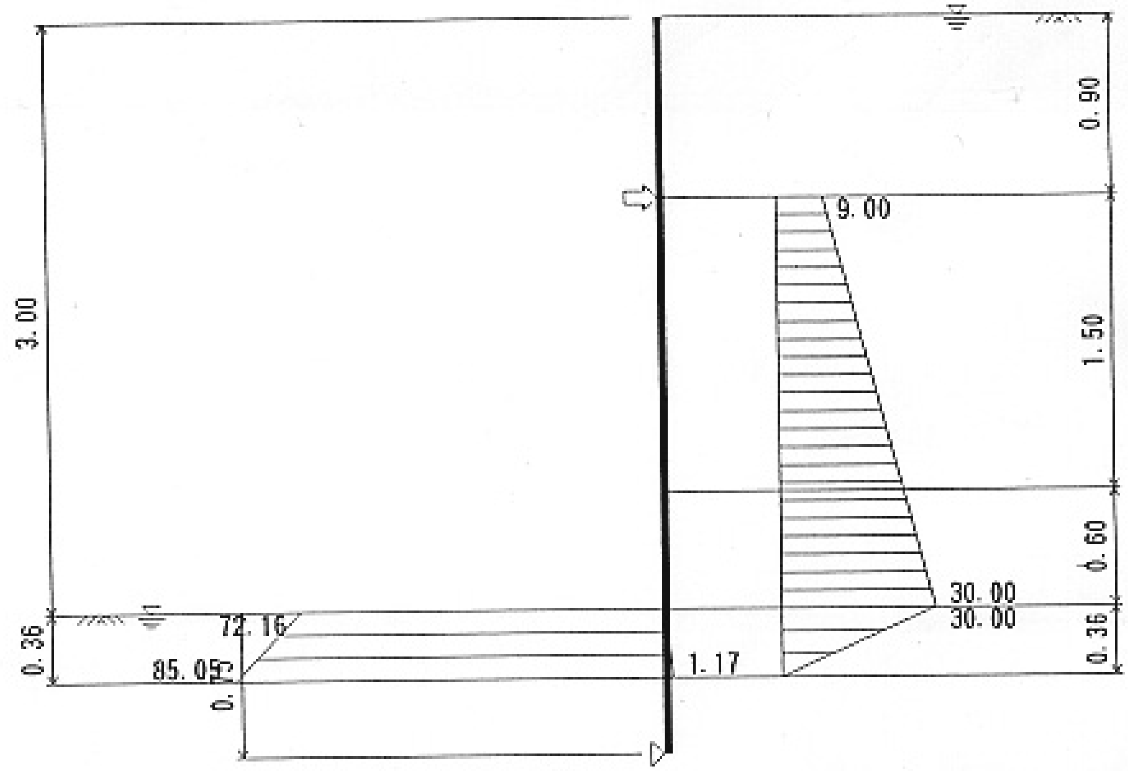
depth — thickness of layer — horizontal force — length of arm — bending moment

No	深さ z G. L. -m	層厚 h m	pp kN/m²	水平力 Pp kN	アーム長 y m	モーメント Mp kN・m
4	3. 00 3. 36	0. 36	72. 16 85. 05	12. 99 15. 31	2. 22 2. 34	28. 84 35. 82
Σ				28. 30		64. 66

Total horizontal pressure at passive side Pp = 28.3 kN

Total moment at passive side Mp = 64.7 kN m

(3) Show a load figure in the following.



8-2-2 Balance of moment

$Mp / Ma = 64.7 / 63.2 = 1.0$ OK!

Balanced depth (Z) is 0.36 m

Therefor,

Penetration length $D = Z \times 1.2 = 0.36 \times 1.2 = 0.43$ m → 3.0 m

8-2-3 Depth of virtual supporting point

$$Y = \frac{Mp}{Pp} - L0$$

Where, L0 : Length between excavation level and strut

= 3.00 m – 0.90 m = 2.10 m

$$= \frac{64.7}{28.3} - 2.1$$

= 0.18 m   (GL-3.18 m)   < 0.75 m → 0.75 m

8-3 STUDY ON BOILING

Boiling is checked with the following formula.

$$Fs = \frac{2 \cdot \gamma' \cdot Ld}{\gamma_w \cdot h_w}$$

where,

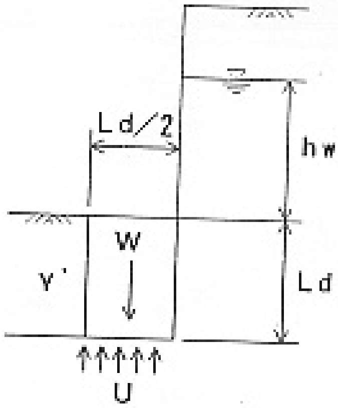
γ_b ... Bulk unit weight of soil (given in section 8-1)

γ' ... buoyant unit weight ($= \gamma_b - \gamma_w = 10.0$ kN/m^3)

γ_w ... unit weight of water (= 10.0 kN/m^3)

Ld ... penetration length (assumed = 3.0 m)

h_w ... depth below water table (= 3.0 m)



$$Fs = \frac{2 \times 10 \times 3.0}{10 \times 3.0}$$

$= 2.0 \geqq$ Fsa = 1.2 OK!

8-4 SUMMARY OF PENETRATION LENGTH

Case	Excavation Depth (m)	Penetration Depth (m)	Length of wall (m)
Penetration length by earth pressure balance	3.00	3.00	6.00
Check on boiling	3.00	3.00	6.00

Therefore, Temporary wall length is 6.0 m.

8-5 STUDY ON SECONDARY COFFERDAM

8-5-1 Calculation length

$L = D + Y - Ds$

where, D : Excavation depth (= 3.0 m)

Y : Depth of virtual supporting point (= 0.75 m; refer, 8-2-3)

Ds : Depth between surface and strut (= 0.9 m)

= 3.0 + 0.75 – 0.9

= 2.85 m

8-5-2 Loading

Active earth pressure is calculated with the following formula.

$$P_a = \left(\sum \gamma \cdot h + q\right) \cdot k$$

where, γ: Average unit weight (= 20.0 kN/m^3)

q : Surcharge load (Take self-weight of slab into consideration.)

=1.2 x 24.5 = 29.4 kN/m^2

k : Coefficient of earth pressure

= 0.60 – 0.02·Z = 0.60 – 0.02 x 3.0

= 0.54

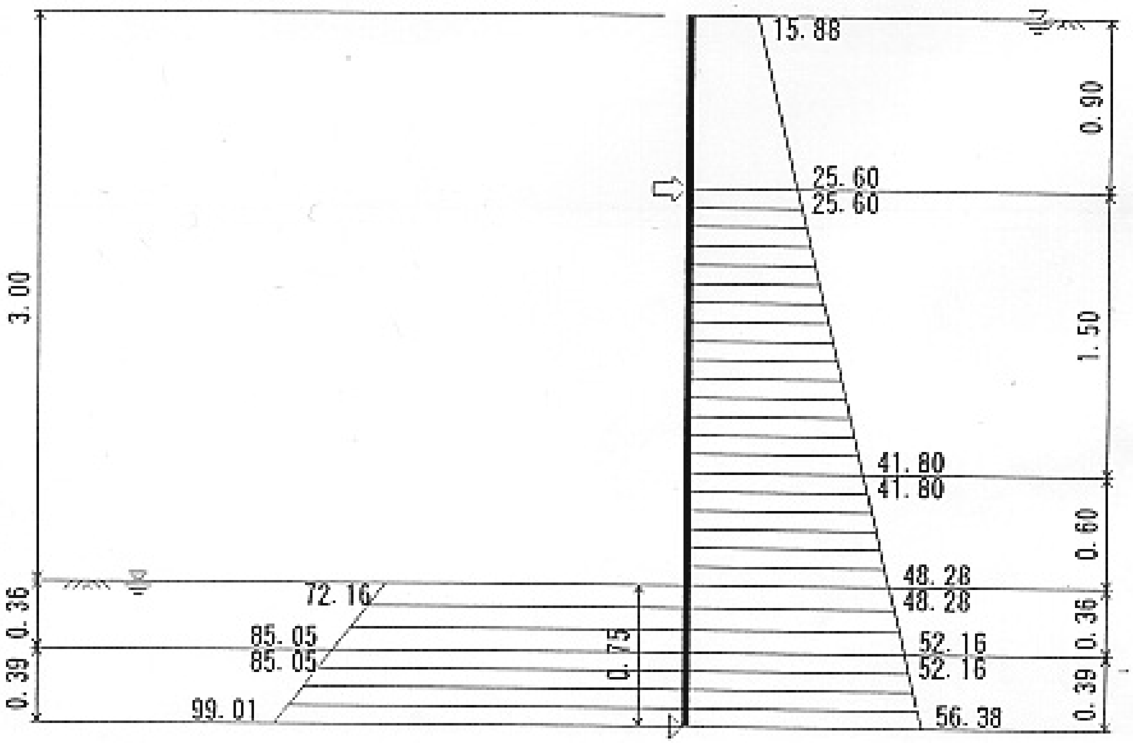
Passive pressure

$$P_p = \left(P_V + q\right) \cdot k_{oa} + 2 \cdot c \cdot \sqrt{k_{oa}}$$

$k_{oa} = \tan^2 (45 + \phi/2)$

where, ϕ : Effective internal friction angle (= 32.0 deg)

q : Surcharge load (= 0.0 kN/m^2)


15.88
0.90
25.60
25.60
3.00
1.50
41.80
41.80
0.60
72.16
48.28
48.28
0.36
0.36
85.05
85.05
0.75
52.16
52.16
0.39
0.39
99.01
56.38

8-5-3 Section force

Bending moment	M_{max} = 34.5 kN m
Shearing force	S_{max} = 46.1 kN
Support reaction	Ra = 46.1 kN
	Rb = 31.5 kN

8-5-4 Specification of temporary wall

Use the Sheet pile 3-type

Elastic modulus	Z = 1340 cm^3
Coefficient	$\alpha = 0.6$

8-5-4 Check on stress

Bending stress is checked with the following formula.

$$\sigma = \frac{M}{\alpha \cdot Z}$$

$$= \frac{34.5 \times 10^6}{0.6 \times 1{,}340{,}000}$$

$$= 43\ N/mm^2 \quad \leqq \quad 270\ N/mm^2 \qquad \text{OK}$$

9. THE CALCULATION OF THE DIAPHRAGM WALL IN THE PERMANENT CONDITION

Calculation model of diaphragm wall in the permanent condition is a spring support beam model.

9-1 INPUT DATA

9-1-1 Specifications of diaphragm wall

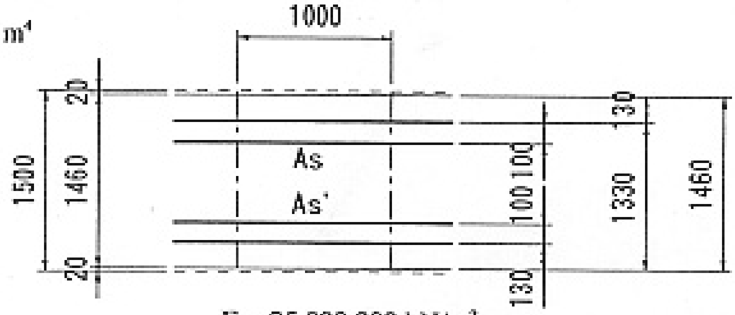
(1) Length $L = 36.5$ m

(2) Moment of inertia $I = 0.1556$ m^4

$I = 1/12 \times b \times d^3 \times 0.6$

$= 1/12 \times 1.00 \times 1.46^3 \times 0.6$

$= 0.1556$ m^4



(3) Elastic modulus $E = 25{,}000{,}000$ kN/m^2

9-1-2 Spring constant of slab

Spring constant of slab is calculated with the following formula.

$$Kc = \frac{2 \cdot E \cdot A}{L \cdot (1 + \phi_c)} \cdot \frac{I}{b \cdot (1 - \varepsilon_c)} \cdot \beta$$

where,

E : Elastic modulus of concrete ($= 2.5 \times 10^7$ kN/m^2)

A : Section area of slab

Type	Area (m^2)
Middle Slab	0.30
Base Slab	1.20

L : Length of slab (= 29.6 m)

b : Horizontal interval of strut (= 1.0 m)

ϕ_c : Creep coefficient of concrete (= 1.2)

ε_c : Drying shrinkage strain of concrete ($= 180 \times 10^{-6}$)

β : Coefficient of opening (= (L1 - L2) / L1)

L1 : Internal width (m)

L2 : Opening width (m)

Type	L1	L2	β
Middle Slab	24.0	11.55	0.519
Base Slab	24.0	0.00	1.00

Middle slab

$$Kc = \frac{2 \times 2.5 \times 10^7 \times 0.3}{29.6 \times (1 + 1.2)} \times \frac{1}{1.0 \times (1 - 180 \times 10^{-6})} \times 0.519$$

$= 119{,}527$ kN/m²

Base slab

$$Kc = \frac{2 \times 2.5 \times 10^7 \times 1.2}{29.6 \times (1 + 1.2)} \times \frac{1}{1.0 \times (1 - 180 \times 10^{-6})} \times 1.00$$

$= 921{,}210$ kN/m²

9-1-3 Coefficient of horizontal subgrade reaction

Coefficient of horizontal subgrade reaction is calculated with the following formula.

$$Kh = \frac{1}{0.3} \cdot \alpha \cdot E_0 \cdot \left(\frac{B}{0.3}\right)^{-\frac{3}{4}}$$

where,

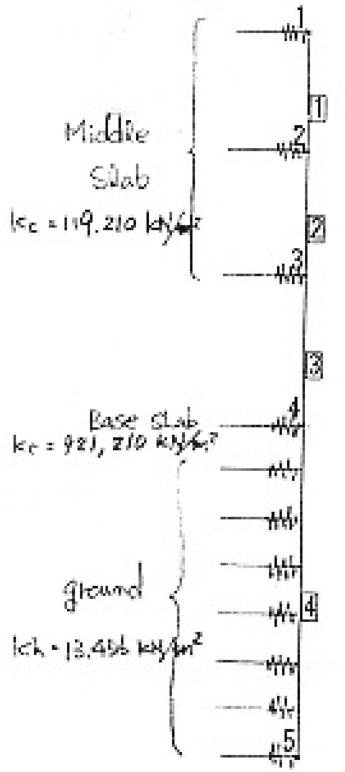
E_0 : Modulus of deformation of ground (kN/m²)

B : Loading width (= 10.0 m)

α : Coefficient (= 4)

Layer	E_0 (kN/m²)	α	B (m)	Kh (kN/m³)
Made ground	840	4	10.0	807
OH layer	700	4	10.0	673
SC layer	3,500	4	10.0	3,363
SM(u) layer	7,000	4	10.0	6,728
SM(l) layer	14,000	4	10.0	13,456
CH layer	33,600	4	10.0	32,294

9-1-4 Calculation model



9-1-5 Laoding

(1) Live load

A uniform traffic surcharge of 10 kN/m^2 is applied.

(2) Earth pressure and water pressure of active side

Earth pressure(Ps) of active side is applied with formula of section 2-3-4(2) a), and water pressure(Pw) of active side is applied with formula of section 2-3-3.

a) EL+2.20m

$Ps1 = 0.5 \times (18.0 \times 0.15 + 10.0)$

$= 6.35\ kN/m^2$

b) EL+0.20m

$Ps2 = 0.5 \times (18.0 \times 0.15 + 18.0 \times 1.85 + 10.0)$

$= 23.0\ kN/m^2$

c) EL-0.10m

$Ps3 = 0.5 \times (18.0 \times 0.15 + 18.0 \times 1.85 + 9.0 \times 0.3 + 10.0)$

$= 24.35\ kN/m^2$

$Pw1 = 10.0 \times 0.3$

$= 3.0\ kN/m^2$

d) EL-2.10m

$Ps4 = 0.5 \times (18.0 \times 0.15 + 18.0 \times 1.85 + 9.0 \times 0.3 + 5.0 \times 2.5 + 10.0)$

$= 24.35\ kN/m^2$

$Pw2 = 10.0 \times 2.30$

$= 23.0\ kN/m^2$

e) EL-3.95m

$Ps5 = 0.5 \times (18.0 \times 0.15 + 18.0 \times 1.85 + 9.0 \times 0.3 + 5.0 \times 2.5 + 11.0 \times 1.85 + 10.0)$

$= 39.5\ kN/m^2$

$Pw3 = 10.0 \times 4.15$

$= 41.5\ kN/m^2$

f) EL-7.10m

$Ps6 = 0.5 \times (18.0 \times 0.15 + 18.0 \times 1.85 + 9.0 \times 0.3 + 5.0 \times 2.5 + 11.0 \times 1.85 + 11.0 \times 3.15$
$+ 10.0)$

$= 61.9\ kN/m^2$

$Pw4 = 10.0 \times 7.30$

$= 73.0\ kN/m^2$

g) EL-10.25m

$Ps7 = 0.5 \times (18.0 \times 0.15 + 18.0 \times 1.85 + 9.0 \times 0.3 + 5.0 \times 2.5 + 11.0 \times 1.85 + 11.0 \times 3.15$
$+ 11.0 \times 3.15 + 10.0)$

$= 79.2\ kN/m^2$

Pw5 = 10.0 x 10.45

= 104.5 kN/m^2

h) EL-17.70m

Ps8 = 0.5 x (18.0 x 0.15 + 18.0 x 1.85 + 9.0 x 0.3 + 5.0 x 2.5 + 11.0 x 1.85 + 11.0 x 3.15

+ 11.0 x 3.15 + 11.0 x 7.45 + 10.0)

= 120 kN/m^2

Pw6 = 10.0 x 17.9

= 179 kN/m^2

i) EL-18.00m

Ps9 = 0.5 x (18.0 x 0.15 + 18.0 x 1.85 + 9.0 x 0.3 + 5.0 x 2.5 + 11.0 x 1.85 + 11.0 x 3.15

+ 11.0 x 3.15 + 11.0 x 7.45 + 11.0 x 0.3 + 10.0)

= 122 kN/m^2

Pw7 = 10.0 x 18.2

= 182 kN/m^2

j) EL-34.30m

Ps10 = 0.5 x (18.0 x 0.15 + 18.0 x 1.85 + 9.0 x 0.3 + 5.0 x 2.5 + 11.0 x 1.85 + 11.0 x 3.15

+ 11.0 x 3.15 + 11.0 x 7.45 + 11.0 x 0.3 + 11.0 x 16.3 + 10.0)

= 211 kN/m^2

Pw8 = 10.0 x 34.5

= 345 kN/m^2

(3) Earth pressure of passive side

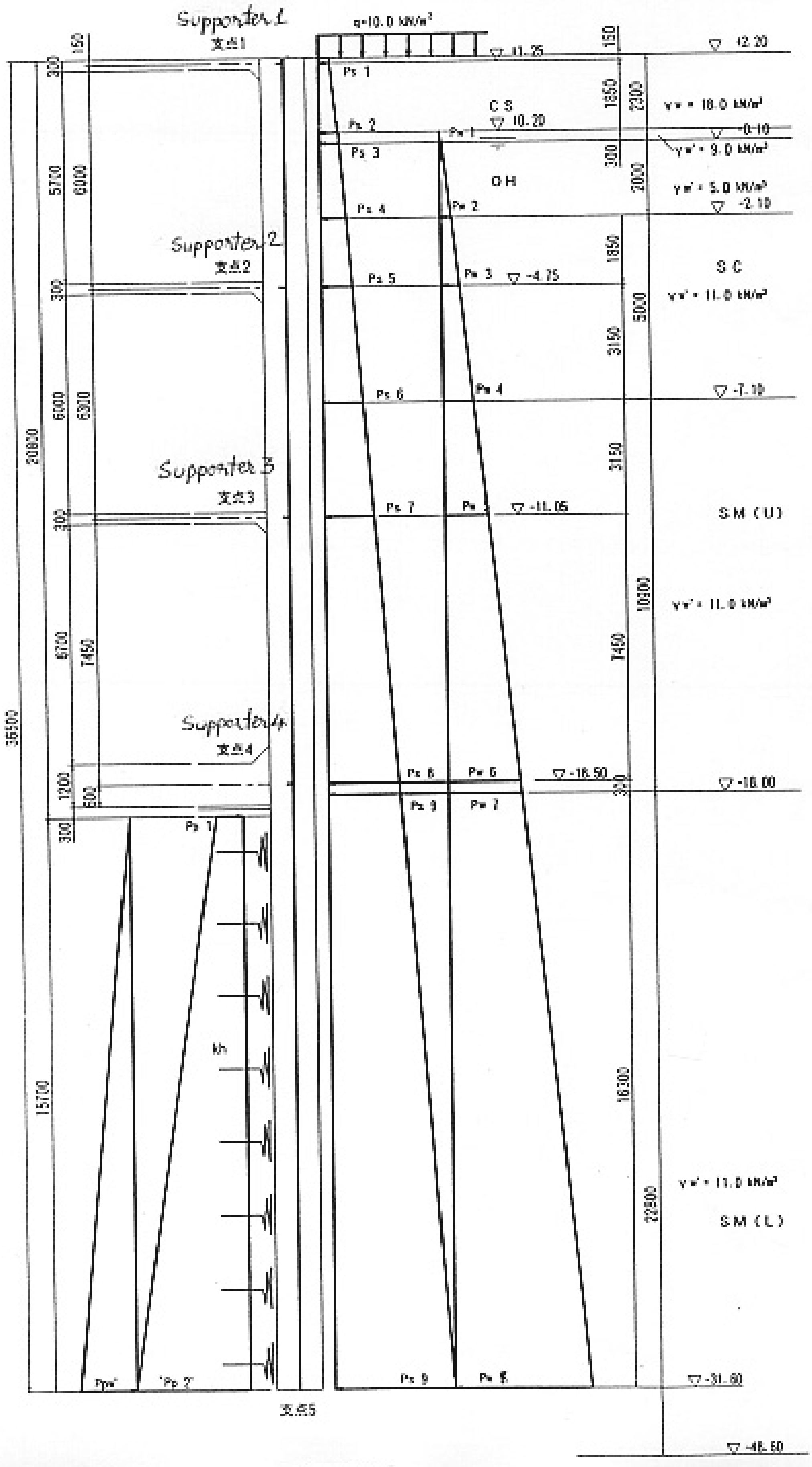
Pp1 = 72.2 kN/m^2

Pp2 = 928 kN/m^2

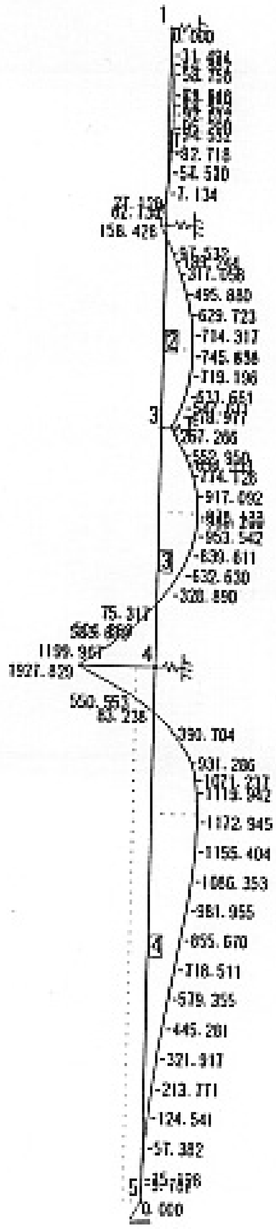
Refer, Passive earth pressure at section 5-3-1 step 9.

9-1-6 Loading diagram

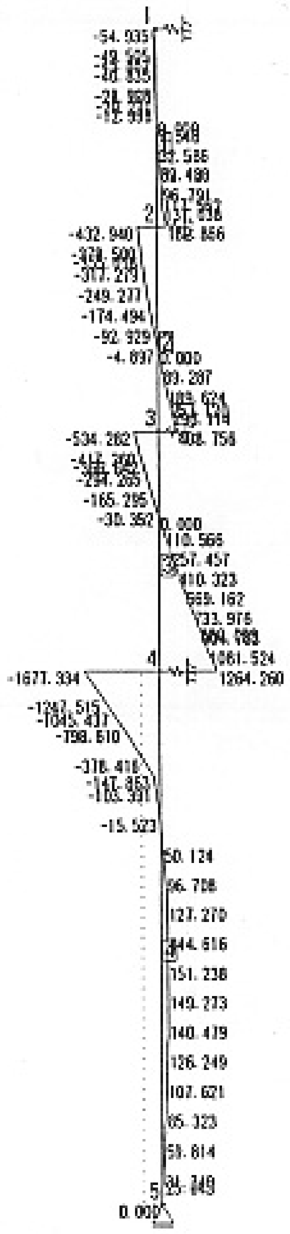
9-1-6 Loading diagram



9-2 SECTION FORCE

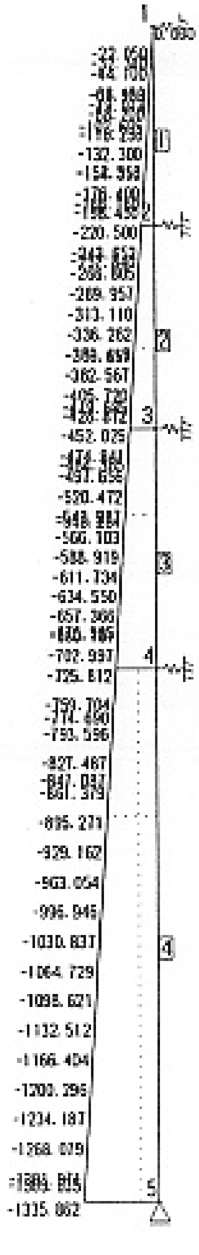


Bending moment diagram



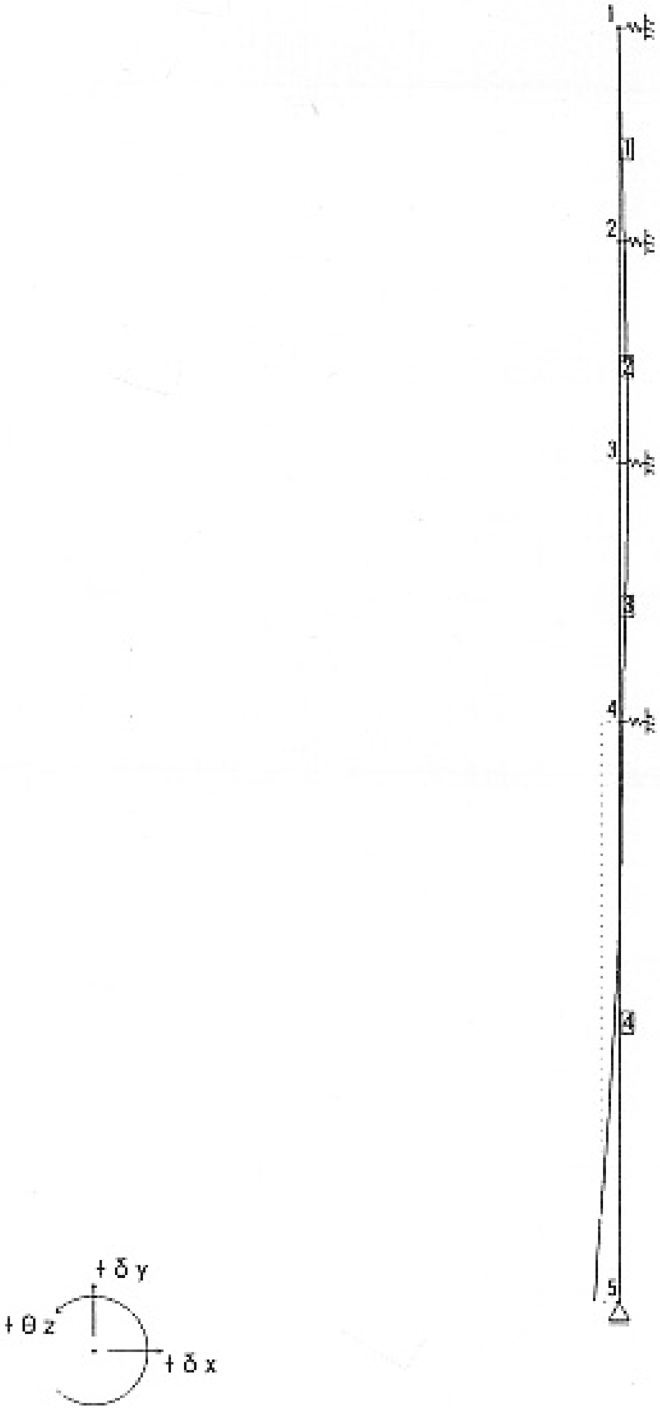
+S ↑ i———j ↓ +S

Shearing force diagram

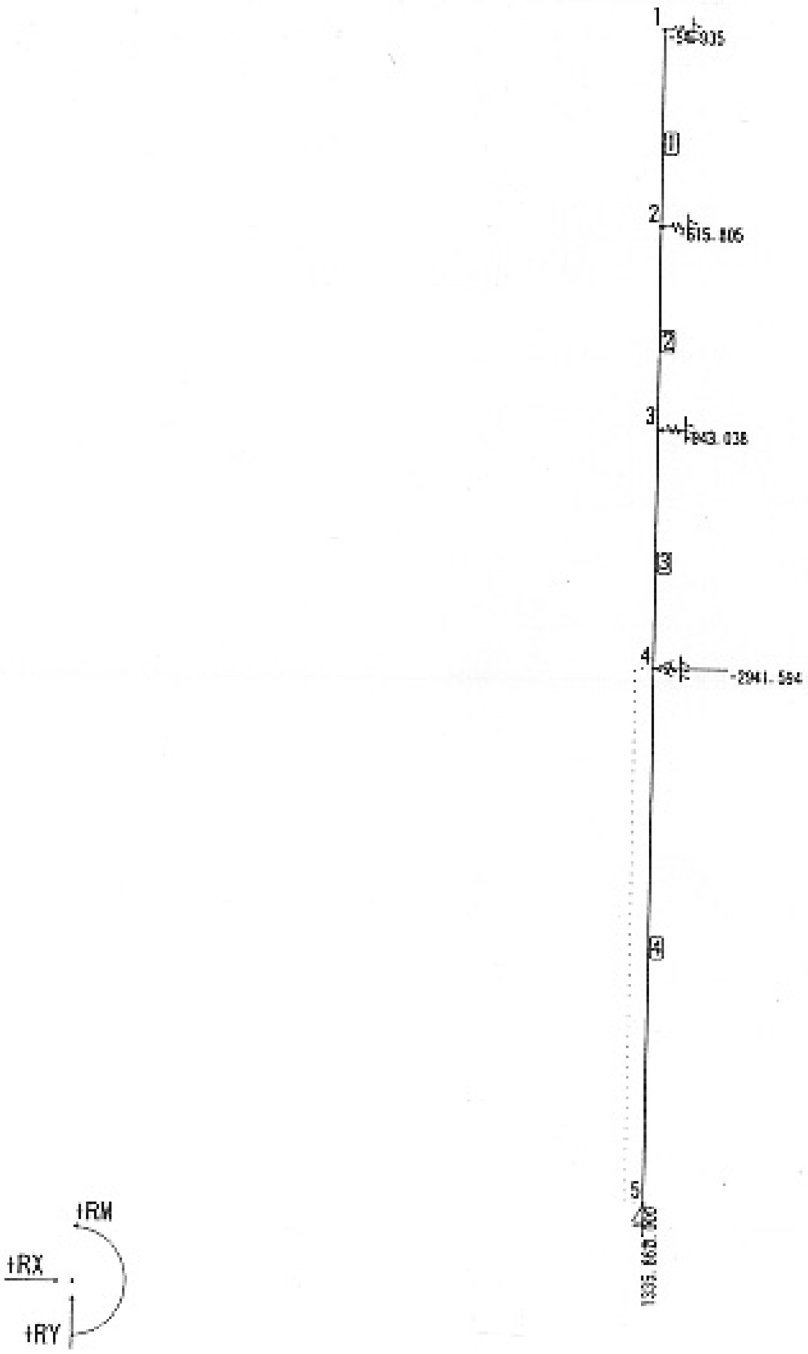


+N — i——j → +N

Axial force diagram



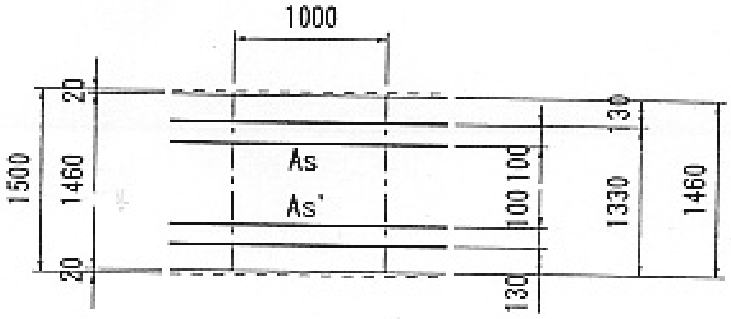
Displacement diagram



Support reaction diagram

9-3 CHECK ON STRESS

9-3-1 Arrangement of reinforcement bar



9-3-2 Resisting moment

Specification of reinforced concrete

- Concrete grade — f'ck = 24 N/mm^2 (in slurry)
  σca = 8 N/mm^2
- Reinforcement bar grade — SD295A,B
  σsa = 160 N/mm^2
- Minimum pitch of re-bars — @150
- Minimum amount of re-bars — 0.002 bd = 0.002 x 133 x 100 = 26.6 cm^2
  (As > D25 @ 150)

9-3-3 Bending moment

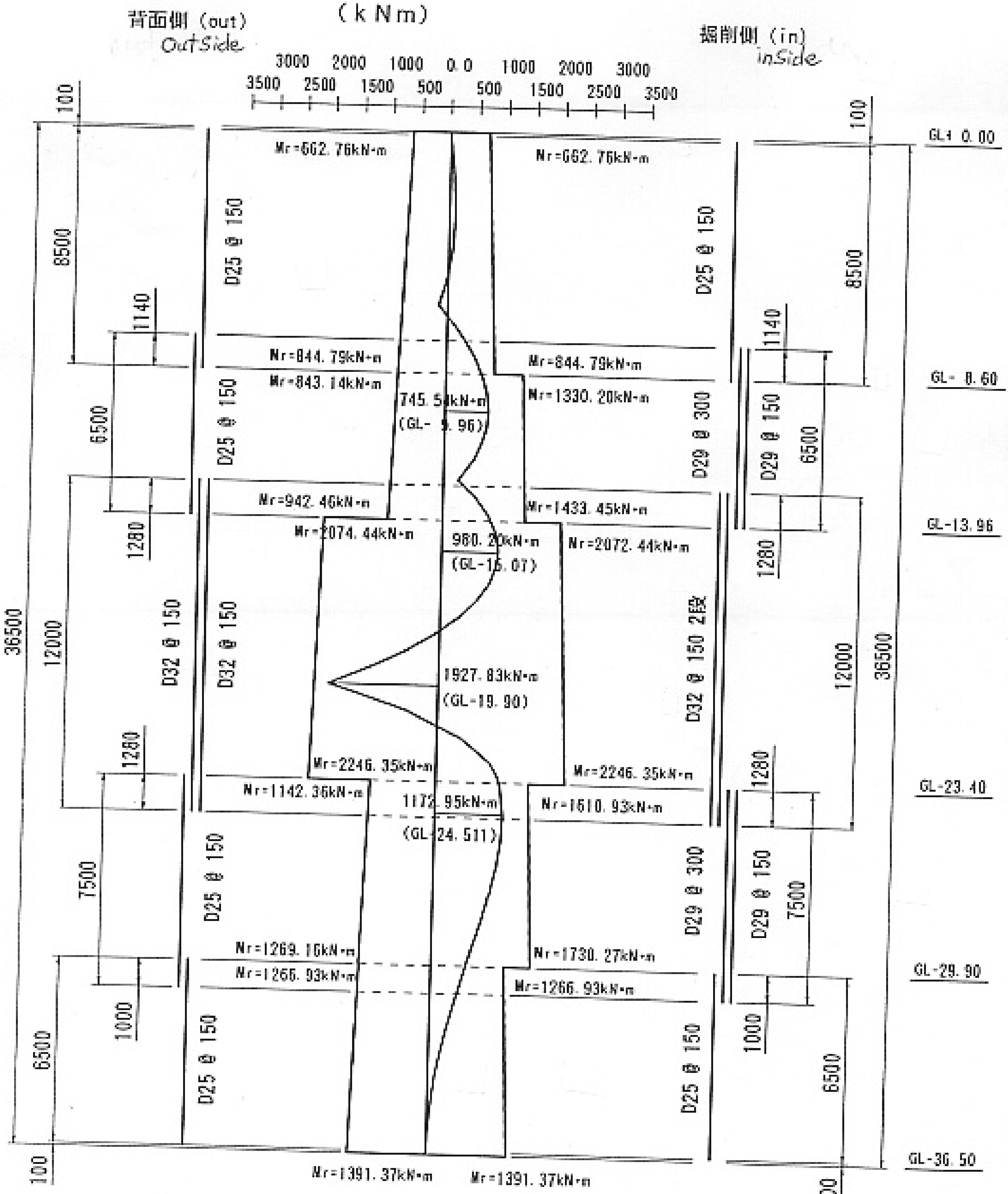
(1) Calculation results of resisting moment

・完成時の抵抗モーメント Permanent Resisting moment Axial force

配置位置 Depth / Position	配　筋 Bar arrangement		掘削側(in)Mr inside (kN·m)	背面側(out)Mr Outside (kN·m)	自重による軸力 (kN)
GL− 0.00	D25@150 1段	in	662.76	662.76	0.00
	D25@150 1段	out			
GL− 8.60(U)	D25@150 1段	in	844.79	844.79	316.05
	D25@150 1段	out			
GL− 8.60(L)	D29@150 1.5段	in	1330.20	843.14	316.05
	D25@150 1段	out			
GL−13.96(U)	D29@150 1.5段	in	1433.45	942.46	513.03
	D25@150 1段	out			
GL−13.96(L)	D32@150 2段	in	2072.44	2072.44	513.03
	D32@150 2段	out			
GL−23.40(U)	D32@150 2段	in	2246.35	2246.35	859.95
	D32@150 2段	out			
GL−23.40(L)	D29@150 1.5段	in	1610.93	1142.36	859.95
	D25@150 1段	out			
GL−29.90(U)	D29@150 1.5段	in	1730.27	1269.16	1098.83
	D25@150 1段	out			
GL−29.90(L)	D25@150 1段	in	1266.93	1266.93	1098.83
	D25@150 1段	out			
GL−36.50	D25@150 1段	in	1675.15	1675.15	1341.38
	D25@150 1段	out			

Figure of moment (permanent)

抵抗モーメント図（完成時）



Moment calculation

■抵抗モーメントの計算 Rectangular

ケース番号 number タイトル title		No. 1 [矩 形] GL-0.00	No. 2 [矩 形] GL-8.60U(in, out)	No. 3 [矩 形] GL-8.60L(in)	No. 4 [矩 形] GL-8.60L(out)
断面寸法 B1 H1 (m) B2 H2 Dimension B3 H3		1.000 1.460 0.000 0.000 ——— 0.000	1.000 1.460 0.000 0.000 ——— 0.000	1.000 1.460 0.000 0.000 ——— 0.000	1.000 1.460 0.000 0.000 ——— 0.000
moment 抵抗モーメントMr' kN·m 軸 力 Nr kN Axial force		662.762 0.000	844.786 316.050	1330.197 316.050	843.142 316.050
鉄筋量 Bar arrangement	d1 /As1 (m) (cm2)	1.330 33.782 (6.667-D25)	1.330 33.782 (6.667-D25)	1.330 42.829 (6.667-D29)	1.330 33.782 (6.667-D25)
	d2 /As2 (m) (cm2)	0.130 33.782 (6.667-D25)	0.130 33.782 (6.667-D25)	1.230 21.411 (3.333-D29)	0.230 21.411 (3.333-D29)
	d3 /As3 (m) (cm2)	——— ——— ———	——— ——— ———	0.130 33.782 (6.667-D25)	0.130 42.829 (6.667-D29)
Total	合計 cm2	67.564	67.564	98.022	98.022
Check of Stress 応力度 σc σca N/mm2 σs σsa		3.1 < 8.0 160.0 = 160.0	4.0 < 8.0 160.0 < 160.0	5.2 < 8.0 160.0 = 160.0	3.8 < 8.0 160.0 < 160.0
Neutral axis 中立軸 x m		0.2964	0.3629	0.4342	0.3490
Modulus of Elasticity ヤング係数比 Ratio		n = 15.00	n = 15.00	n = 15.00	n = 15.00

Moment calculation

■抵抗モーメントの計算 Rectangular

ケース番号 Number タイトル title		No. 5 [矩 形] GL-13.96U(in)	No. 6 [矩 形] GL-13.96U(out)	No. 7 [矩 形] GL-13.96L(in, out)	No. 8 [矩 形] GL-23.40U(in, out)
断面寸法 B1 H1 (m) B2 H2 Dimension B3 H3		1.000 1.460 0.000 0.000 ——— 0.000	1.000 1.460 0.000 0.000 ——— 0.000	1.000 1.460 0.000 0.000 ——— 0.000	1.000 1.460 0.000 0.000 ——— 0.000
Moment 抵抗モーメントMr' kN·m 軸 力 Nr kN Axial force		1433.447 513.030	942.455 513.030	2072.443 513.030	2246.350 859.950
鉄筋量 Bar arrangement	d1 /As1 (m) (cm2)	1.330 42.829 (6.667-D29)	1.330 33.782 (6.667-D25)	1.330 52.949 (6.667-D32)	1.330 52.949 (6.667-D32)
	d2 /As2 (m) (cm2)	1.230 21.411 (3.333-D29)	0.230 21.411 (3.333-D29)	1.230 52.949 (6.667-D32)	1.230 52.949 (6.667-D32)
	d3 /As3 (m) (cm2)	0.130 33.782 (6.667-D25)	0.130 42.829	0.230 52.949 (6.667-D32)	0.230 52.949 (6.667-D32)
	d4 /As4 (m) (cm2)	——— ——— ———	——— ——— ———	0.130 52.949 (6.667-D32)	0.130 52.949 (6.667-D32)
Total	合計 cm2	98.022	98.022	211.796	211.796
Check of Stress 応力度 σc σca N/mm2 σs σsa		5.6 < 8.0 160.0 = 160.0	4.8 < 8.0 160.0 = 160.0	6.1 < 8.0 160.0 = 160.0	6.8 < 8.0 160.0 < 160.0
Neutral axis 中立軸 x m		0.4599	0.4119	0.4060	0.5165
Modulus of Elasticity ヤング係数比 Ratio		n = 15.00	n = 15.00	n = 15.00	n = 15.00

Moment calculation

■抵抗モーメントの計算 Rectangular

ケース番号 Number / タイトル Title		No. 9 [矩 形] GL-23.40L(in)	No. 10 [矩 形] GL-23.40L(out)	No. 11 [矩 形] GL-29.90U(in)	No. 12 [矩 形] GL-29.90U(out)
断面寸法 B1 H1 / (m) B2 H2 / Dimension B3 H3		1.000 1.460 / 0.000 0.000 / ——— 0.000	1.000 1.460 / 0.000 0.000 / ——— 0.000	1.000 1.460 / 0.000 0.000 / ——— 0.000	1.000 1.460 / 0.000 0.000 / ——— 0.000
抵抗モーメントMr' kN·m / 軸 力 Axial force N kN		1610.930 / 859.950	1142.358 / 859.950	1730.267 / 1098.830	1269.164 / 1098.830
鉄筋量 Bar arrangement	d1 /As1 (m) (cm2)	1.330 42.829 (6.667-D29)	1.330 33.782 (6.667-D25)	1.330 42.829 (6.667-D29)	1.330 33.782 (6.667-D25)
	d2 /As2 (m) (cm2)	1.230 21.411 (3.333-D29)	0.230 21.411 (3.333-D29)	1.230 21.411 (3.333-D29)	0.230 21.411 (3.333-D29)
	d3 /As3 (m) (cm2)	0.130 33.782 (6.667-D25)	0.130 42.829 (6.667-D29)	0.130 33.782 (6.667-D25)	0.130 42.829 (6.667-D29)
Total	合計 cm2	98.022	98.022	98.022	98.022
Check of stress 応力度 σc σca / N/mm2 σs σsa		6.4 < 8.0 / 160.0 < 160.0	5.1 < 8.0 / 160.0 < 160.0	6.9 < 8.0 / 160.0 < 160.0	5.6 < 8.0 / 160.0 = 160.0
Neutral axis 中立軸 x m		0.4999	0.4300	0.5243	0.4590
Modulus of Elasticity Ratio ヤング係数比		n = 15.00	n = 15.00	n = 15.00	n = 15.00

Moment calculation

■抵抗モーメントの計算 Rectangular

ケース番号 Number / タイトル Title		No. 13 [矩 形] GL-29.90L(in, out)	No. 14 [矩 形] GL-36.50U(in, out)
断面寸法 B1 H1 / (m) B2 H2 / Dimension B3 H3		1.000 1.460 / 0.000 0.000 / ——— 0.000	1.000 1.460 / 0.000 0.000 / ——— 0.000
抵抗モーメントMr' kN·m / 軸 力 Axial force N kN		1266.931 / 1098.830	1391.368 / 1341.380
鉄筋量 Bar arrangement	d1 /As1 (m) (cm2)	1.330 33.782 (6.667-D25)	1.330 33.782 (6.667-D25)
	d2 /As2 (m) (cm2)	0.130 33.782 (6.667-D25)	0.130 33.782 (6.667-D25)
Total	合計 cm2	67.564	67.564
Check of stress 応力度 σc σca / N/mm2 σs σsa		6.0 < 8.0 / 160.0 = 160.0	6.5 < 8.0 / 160.0 = 160.0
Neutral axis 中立軸 x m		0.4766	0.5037
Modulus of Elasticity Ratio ヤング係数比		n = 15.00	n = 15.00

(2) Check on bending stress

Moment calculation

・曲げ応力度の計算 Rectangular

ケース番号 Number タイトル title		No. 1 [矩 形] GL-9.96	No. 2 [矩 形] GL-15.07	No. 3 [矩 形] GL-19.90	No. 4 [矩 形] GL-24.51
断面寸法 (m) Dimension	B1 H1 B2 H2 B3 H3	1.000 1.460 0.000 0.000 ------- 0.000	1.000 1.460 0.000 0.000 ------- 0.000	1.000 1.460 0.000 0.000 ------- 0.000	1.000 1.460 0.000 0.000 ------- 0.000
断面力 Force	M kN-m N kN	745.540 366.030	980.200 553.820	1927.830 731.330	1172.950 900.740
鉄筋量 Bar arrangement	d1 /As1 (m) (cm2)	1.330 42.829 (6.667-D29)	1.330 52.949 (6.667-D32)	1.330 52.949 (6.667-D32)	1.330 42.829 (6.667-D29)
	d2 /As2 (m) (cm2)	1.230 21.411 (3.333-D29)	1.230 52.949 (6.667-D32)	1.230 52.949 (6.667-D32)	1.230 21.411 (3.333-D29)
	d3 /As3 (m) (cm2)	0.130 33.782 (6.667-D25)	0.230 52.949 (6.667-D32)	0.230 52.949 (6.667-D32)	0.130 33.782 (6.667-D25)
	d4 /As4 (m) (cm2)	------- ------- -------	0.130 52.949 (6.667-D32)	0.130 52.949 (6.667-D32)	------- ------- -------
Total	合計 cm2	98.022	211.796	211.796	98.022
Check of stress 応力度 N/mm2	σc σca σs σsa	3.0 < 8.0 76.2 < 160.0	3.0 < 8.0 62.2 < 160.0	5.8 < 8.0 137.6 < 160.0	4.7 < 8.0 98.3 < 160.0
Neutral axis 中立軸	x m	0.4899	0.5601	0.5157	0.5580
Modulus of elasticity ratio ヤング係数比		n = 15.00	n = 15.00	n = 15.00	n = 15.00

9-3-4 Shearing force

(1) Calculation of resisting shearing force

a) The allowable shearing force that it can be borne only with concrete

The allowable shearing force that it can be borne only with concrete is calculated with the following formula.

$Scr = \tau a \cdot b \cdot d$

where, τa : Allowable shearing stress of concrete

(= 0.39 N/mm², f'ck = 24 N/mm² (in slurry))

b : Unit width (= 100 cm)

d : Effective depth (= 133 cm)

$Scr = (0.39 \times 100) \times 100 \times 133$

$= 518{,}700$ N

$= 518.7$ kN

b) The allowable shearing force in consideration of stirrup

The allowable shearing force in consideration of stirrup is calculated with the following formula.

$Sr = Sh + Sc$

$Sh = \sigma sa \cdot d \cdot As / (1.15 \cdot a)$

where, σsa : Allowable tensile stress of concrete (= 160 N/mm², SD295A,B)

d : Effective depth (= 133 cm)

As : Section area of stirrup (cm²/ m)

a : Pitch of stirrup (cm)

Diameter of stirrup	As		σsa	d	a	Sh (kN)
D13	4.223	@300	160	133	30.0	260
D16	6.619	@300	160	133	30.0	408
D19	9.549	@300	160	133	30.0	589
D22	12.902	@300	160	133	30.0	796

$Sc = 1 / 2 \cdot \tau a \cdot b \cdot d = 1 / 2 \cdot Scr$

$= 519 / 2 = 259$ kN

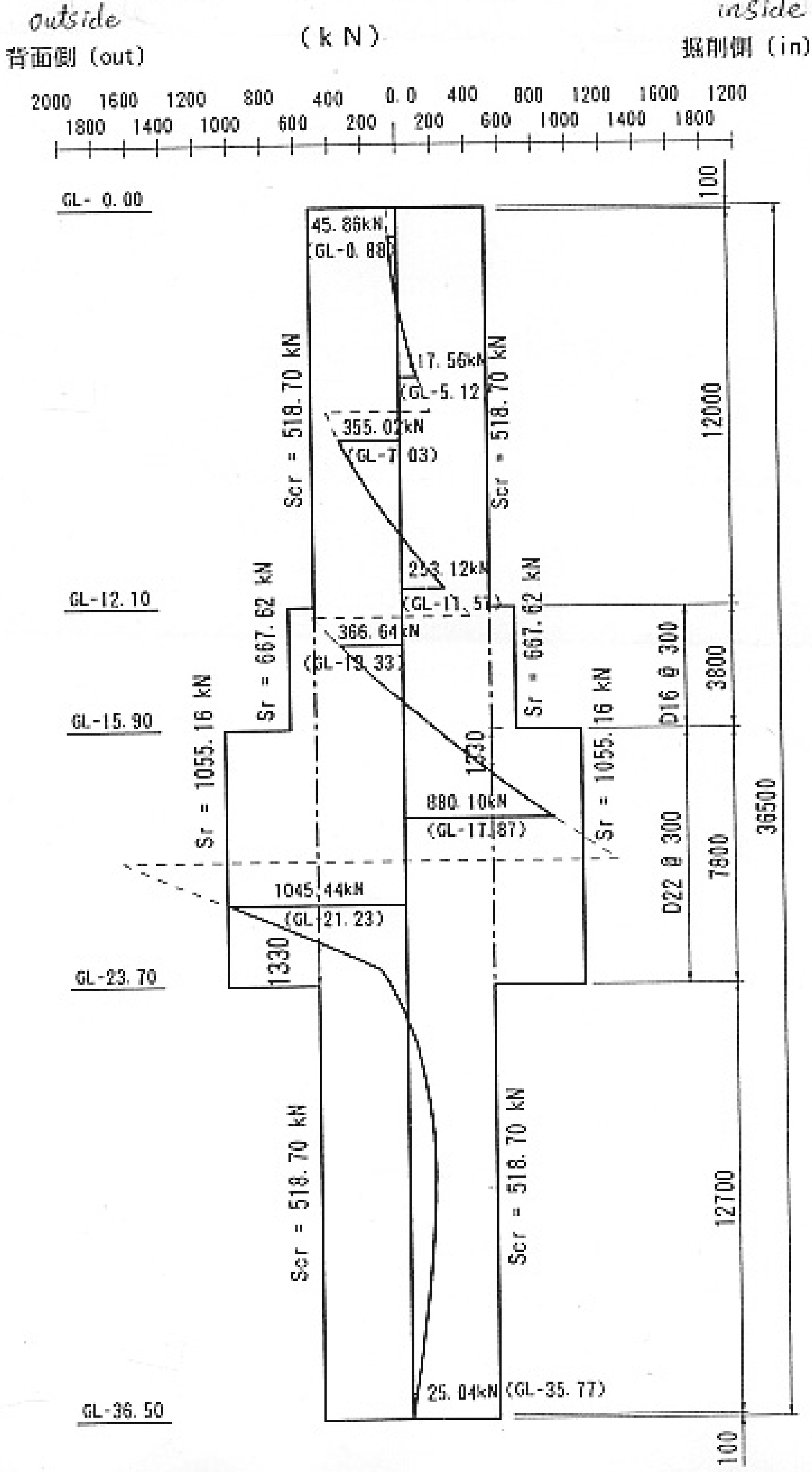
Therefor,

Diameter of stirrup	Sh (kN)	Sc (kN)	Sr = Sh + Sc (kN)
D13	260	259	520
D16	408	259	668
D19	589	259	848
D22	796	259	1,055

(2) Calculation results of resisting shearing force

Figure of Sharing force (Permanent)

抵抗せん断力図(完成時)



Comparison of Temporary Wall

Pump well part of Intermediate Wastewater Pumping Station is 24.20m at width, 29.80m at length and 17.80m to 20.80m at depth.
Pump well is deep structure.
Soil condition is shown below.

Layer	Elevation of layer		Unit weight (t/m3)	C(t/m2)	N
	Upper	Lower			
OH(Organic clay)	+ 1.40	- 2.10	1.40	0.60	1
SC(Clayer sand)	- 2.10	- 7.10	2.00	2.20	5
SM(Silty sand)	- 7.10	- 40.80	2.00	$\phi = 30$	8 ~ 28
CH(Clay)	- 40.80		2.00	7.00	44 ~ 51

In comparison of temporary wall for deep structure, some factors should be take into consideration.
(a) High rigidity is required for temporary wall, because big earth pressure and water pressure affect to temporary wall.
(b) Temporary wall should have reliability for construction and many results.
(c) Temporary wall should have high reliability for watertight.
(d) Construction of temporary wall do not cause any influence on surrounding inhabitant, such as noise and vibration.

From above factors, 3 temporary wall methods are selected.
(a) Sheet Pile Method
(b) Soil Mixing wall method
(c) Diaphragm Wall Method

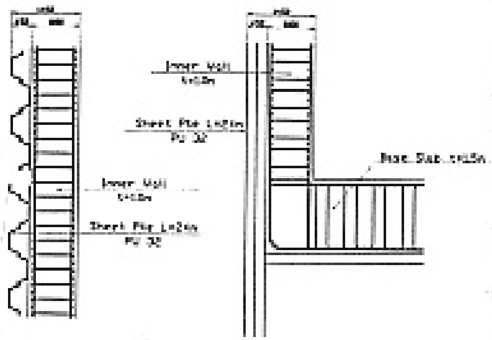
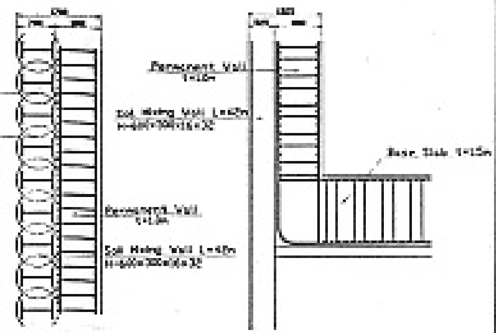
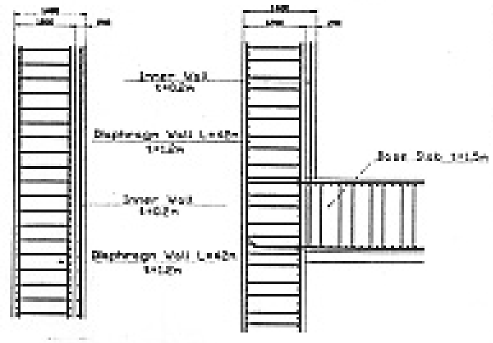
From the result of comparison, sheet pile method was excluded because of the reasons shown below.

- Rigidity of sheet pile is the lowest one in comparison with other 2 methods.
- So soil improvement under the base slab is necessary to prevent a occurrence of strong stress for sheet pile.
- It is necessary to construct a permanent wall, because sheet pile is only use for temporary wall.
- The cost is most expensive compared with other 2 methods.

Soil mixing wall method is also more expensive than diaphragm wall method.
Because soil mixing wall is only use for temporary wall and it is necessary to construct a permanent wall.
Diaphragm wall method has most high rigidity and soil improvement is not necessary.
It is not necessary to construct a permanent wall because diaphragm wall can use for permanent wall.

The cost of diaphragm wall is most economic.
From above reasons, diaphragm wall method was selected for temporary wall method of Intermediate Wastewater Pumping Station.

Comparative Table of Earth Retaining Wall

	Sheet Pile Method	Soil Mixing Wall Method	Diaphragm Wall Method
General Figure			
Outline of Method	Driving a sheet pile into the soil continuously by splice each sheetpile to make a watertight wall. Sheetpile is temporary wall. After excavation inside, construct a structure and take off the sheetpile. Maximum excavation depth is arround 25 m. Allowable capacity for earth pressure is small compared with another method. Most suitable driving method is jacked pile method by using a earth auger. This method is suitable for soft silt soil, sand soil and clay soil.	Excavate a soil by earth auger (D = 900mm) with pouring a cement mortar. Construct a watertight wall by mixing a cement mortar and existing soil. To make a core, H section steel (600x300x16x32) is put into a wall. After finishing of excavation, it is necessary to construct a structure inside. Because this wall is temporary wall. Maximum excavation depth is arround 50 m. Allowable capacity for earth pressure is bigger than sheetpile.	Excavate a soil (width 1.20m) by using a bentonite liquid to stabilize a surface of soil. After excavation, insert a reinforcement and replacement a bentonite liquid with concrete to make a watertight wall. This wall use for permanent structure. It is necessary to attach a thin wall (t=200mm) after excavation of inside. Maximum excavation depth is arround 60 m. But it is possible to excavate a 100 m depth. Allowable capacity for earth pressure is bigger than other method.
Adaptability to Ground Condition	Excavation depth is to deep and the soil arround a bottom of excavation is not hard, strong stress should occur to sheetpile. It is necessary to take measures to stop the occurrence of strong stress.	Some measurement is necessary to stabilize a soil arround bottom of excavation soil. Suitable method for existing soil condition, because allowable capacity for earth pressure is big.	Some measurement is necessary to stabilize a soil arround bottom of excavation soil. Suitable method for existing soil condition, because allowable capacity for earth pressure is big. Bentonite liquid is not leak to soil, because coefficient of permeability of sand is very low.
Supplemental Methods	To stabilize a soil arround bottom of excavation and to stop the occurrence of strong stress for sheetpile, some measurement are necessary. Such as lowering of groundwater level method or soil improvement method. It is difficult to take a method of lowering of groundwater level. Because coefficient of permeability of sand is very low. Soil improvement method is suitable method.	To stabilize a bottom soil, the tip point elevation of wall should reach to good soil (CH layer, GL - 40m). Other measure is not necessary.	To stabilize a bottom soil, the tip point elevation of wall should reach to good soil (CH layer, GL - 40m). Other measure is not necessary.
Cost	Sheetpiles construction cost Soil improvement cost Inner wall construction cost (t = 1000mm) 1.00	Soil Mixing Wall construction cost Inner Wall construction cost (t = 1000mm) 0.94	Diaphragm Wall construction cost Inner Wall constructioncost 0.81
Assesment	× Soil improvement is necessary	△ Cost is expensive than Diaphragm Wall Method	○ Cost is cheap and structure is strong