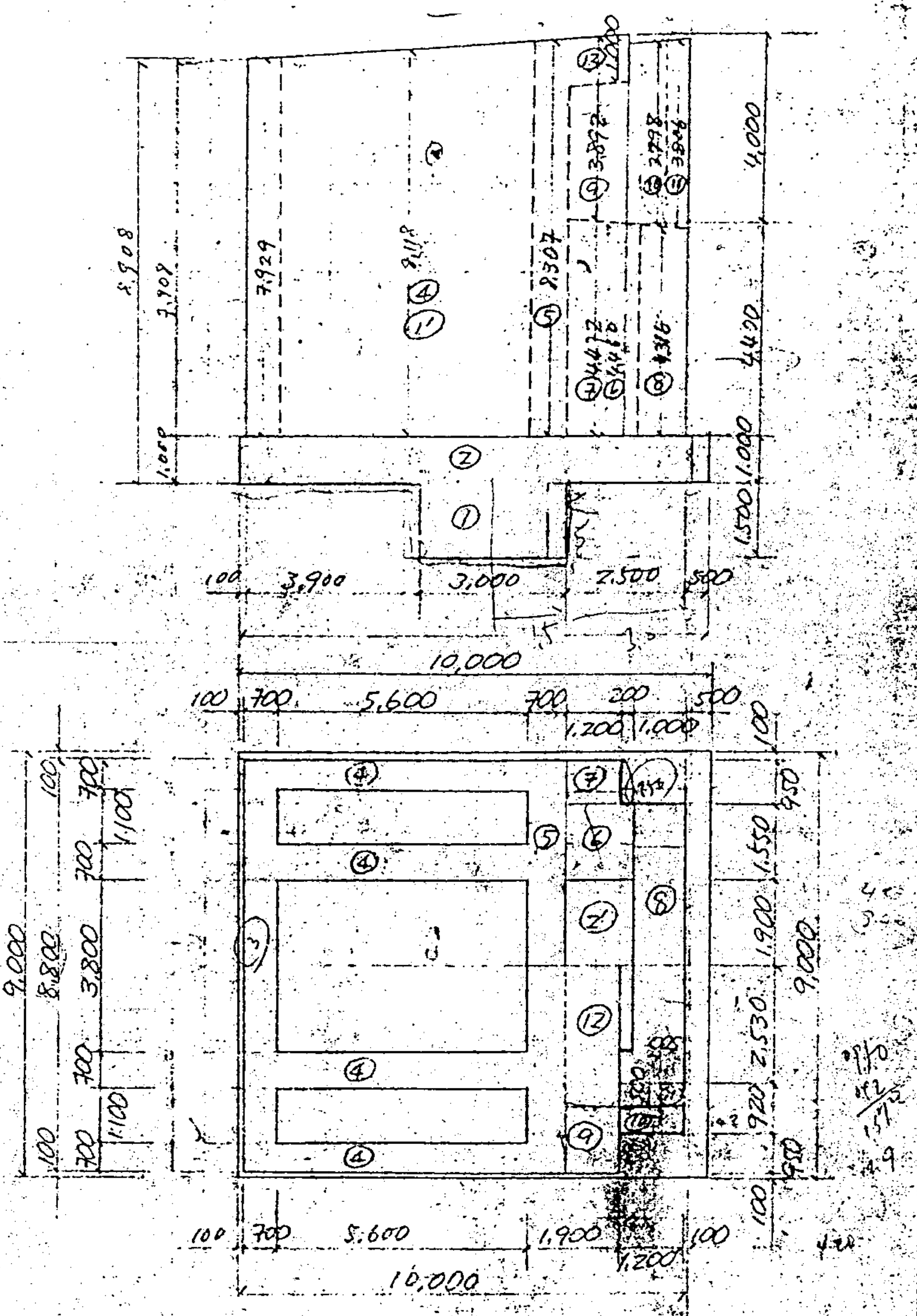
才 3 章

下部構の設計

I. 左岸橋台

§1. 寸法


10,000
100
3,900
3,000
2,500
500
700
5,600
200
1,200
1,000
1,500
1,900
9,000
8,800
3,800
1,100
1,550
2,530
920
950

§2 重量

コンクリート

①	3.000	× 1.500 × 9.000	=	40.5
②	10.000	× 1.000 × 9.000	=	90.0
③	0.700	× 7.929 × 8.800	=	48.8
④	5.600	× 8.118 × 2.800	=	127.3
⑤	0.700	× 8.307 × 8.800	=	51.2
⑥	1.400	× 4.460 × 3.100	=	19.4
⑦	1.200	× 4.472 × 1.900	=	10.2
⑧	1.000	× 4.316 × 6.900	=	29.8
⑨	1.200	× 3.892 × 2.900	=	13.5
⑩	1.200	× 3.798 × 1.000	=	4.6
⑪	0.500	× 3.806 × 1.840	=	3.5
⑫	1.200	× 1.000 × 5.900	=	7.1

$\Sigma V = 445.9\ m^3$

$0.50 \times 3.806 \times 0.94 + 0.24 \times 0.80 \times 0.45$

$W = 445.9 \times 2.5 = 1115\ t$

工事中 ¥1

①'	5.600	× 8.118 × 6.000	=	272.8
②'	1.400	× 4.460 × 3.800	=	23.7

$\Sigma V = 296.5\ m^3$

$W = 296.5 \times 1.6 = 474\ t$

$\Sigma W = 1115 + 474 = 1589\ t$

53 重心位置

コンクリート

①	40.5	×	5.5	=	222.8	×	−0.75	= −30.4
②	90.0	×	5.0	=	450.0	×	0.5	= 45.0
③	48.8	×	0.45	=	22.0	×	4.965	= 242.3
④	127.3	×	3.6	=	458.3	×	5.059	= 644.0
⑤	57.2	×	6.75	=	345.6	×	5.254	= 269.0
⑥	19.4	×	7.8	=	151.3	×	3.23	= 62.7
⑦	10.2	×	7.7	=	78.5	×	3.236	= 33.0
⑧	29.8	×	9.0	=	268.2	×	3.158	= 94.1
⑨	13.5	×	7.7	=	104.0	×	7.418	= 100.1
⑩	4.6	×	7.9	=	40.9	×	7.627	= 35.1
⑪	3.5	×	9.25	=	32.4	×	7.589	= 26.6
⑫	7.1	×	7.7	=	54.7	×	2.864	= 62.9
					2228.7			1584.4

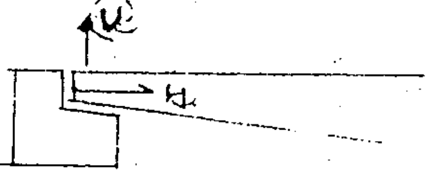
土石力

⑪	272.8	×	3.6	=	922.1	×	5.079	= 1380.1
⑫	23.7	×	7.8	=	184.9	×	7.23	= 76.6
					1167.0			1456.7

$$X = \frac{2228.7 \times 2.5 + 1167.0 \times 1.6}{1589} = 4.62^{m}$$

$$Y = \frac{1584.4 \times 2.5 + 1456.7 \times 1.6}{1589} = 3.96^{m}$$

3.4 橋台に作用する力



i) 設計荷重時

V 自重 −339.1 / −305.1 +78 + (−364.4) = −286.4

静荷重 −87.5 / −60.7 −87.6

活荷重 −140.6 / −112.1 −21.09 −86.5 − 32.7 = −141.4
+66.2 / 58.4 21.7 + 10.3 + 30.7 = 66.3

V_{max} −307.4 / 360.4 −307.8

V_{min} −477.9 / 567.2 −515.5 ⊕ −37.6

ii) 地震時

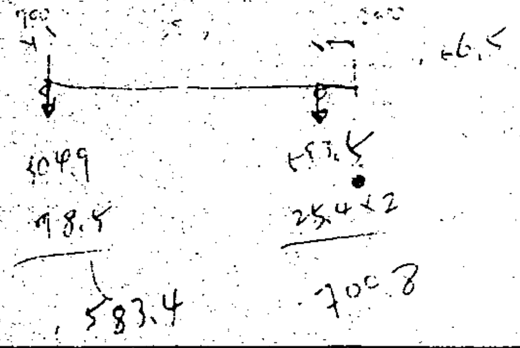
−V = 305.1 + 60.7 = ~~365.8~~ −426.6 t 2730.9

H = (1231.9 + 1144.2 + 100 + 56 + 87.8) × 0.15 − 38.5 = 357.5 + 426.6 = ~~368.1~~ 409.6 t (橋脚より) 415.8 437.6

437 t

iii) 供給向施工直後

V = 22.47.2 / 39.3 = ~~571.8~~ t



§5 常時背面土圧

$h = 8.908^{m} \longrightarrow 8.91^{m}$

過載荷重 $1^{t/m^2} \longrightarrow \Delta h = 0.65^{m}$

$$E_1 = \omega \cdot \frac{1-\sin\phi}{1+\sin\phi} \cdot h \cdot \Delta h \qquad \phi = 30^\circ \quad \sin\phi = 0.5$$

$$= 1.6 \times \frac{1-0.5}{1+0.5} \times 8.91 \times 0.65 = 3.09^{t/m}$$

作用高 $y = \frac{h}{2} = \frac{8.91}{2} = 4.46^{m}$

$$E_2 = \omega \cdot \frac{1-\sin\phi}{1+\sin\phi} \cdot \frac{h^2}{2}$$

$$= 1.6 \times \frac{1-0.5}{1+0.5} \times \frac{8.91^2}{2} = 21.2^{t/m}$$

作用高 $y = \frac{h}{3} = \frac{8.91}{3} = 2.97^{m}$

$$E = E_1 + E_2 = 3.09 + 21.2 = 24.3^{t/m}$$

作用高 $y = \frac{3.09 \times 4.46 + 21.2 \times 2.97}{24.3} = 3.16^{m}$

§6 地震時背面土圧

地震時には土砂の内部摩擦角が $\theta = \tan^{-1}\left[\frac{k_H}{1-k_V}\right]$ だけ減少したものとして考える。

$$\theta = \tan^{-1}\left[\frac{0.15}{1-0.1}\right] = \tan^{-1} 0.167 = 9.6^\circ$$

$$\therefore \phi_e = 30^\circ - 9.6^\circ = 20.4^\circ$$

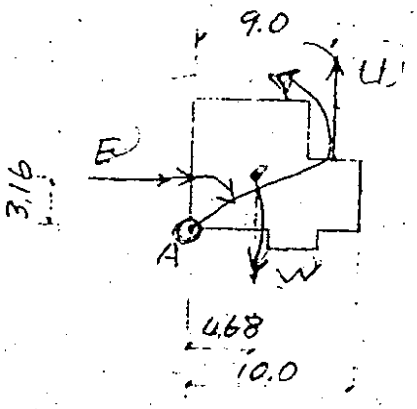
$$E = \omega \cdot \frac{1-\sin\phi_e}{1+\sin\phi_e} \cdot \frac{h^2}{2} \qquad \sin\phi_e = 0.348$$

$$= 1.6 \times \frac{1-0.348}{1+0.348} \times \frac{8.91^2}{2} = 30.8^{t/m}$$

作用高 $y = \frac{h}{3} = \frac{8.91}{3} = 2.97^{m}$

§7 転倒安全率（橋軸方向）

(1) 常時（設計荷重時）



後方転倒モーメント（A点）

U : 478 × 9.0 = 4302 t·m

抵抗モーメント

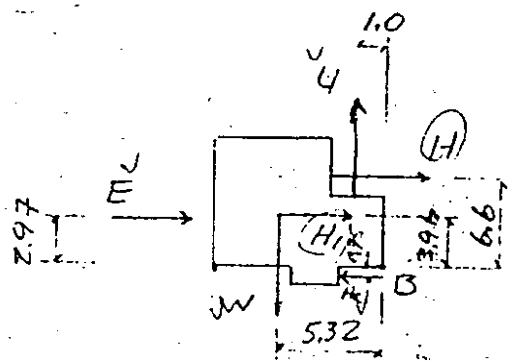
W : 1589 × 4.68 = 7437 t·m

E : 243 × 8.8 × 3.16 = 676 t·m

8113

安全率 $F = \frac{8113}{4302} = 1.8$

(2) 地震時



鉛直震度 +0.1

転倒モーメント

U : 366 × 1.1 × 1.0 = 403 tm
H : 413 × 6.6 = 2726
H1 : 238 × 3.96 = 942
E : 30.8 × 8.8 × 2.97 = 805
JHK : 922 × 0.75 = 692
5568 tm

抵抗モーメント

W : 1589 × 1.1 × 5.32 = 9299 tm

安全率 F = 9299 / 5568 = 1.66

鉛直震度 −0.1

転倒モーメント

U : 366 × 0.9 × 1.0 = 329 tm
H : 413 × 6.6 = 2726
H1 : 238 × 3.96 = 942
E : 30.8 × 8.8 × 2.97 = 805
JHK : 922 × 0.75 = 692
5494 tm

抵抗モーメント

W : 1589 × 0.9 × 5.32 = 7608 tm

安全率 F = 7608 / 5494 = 1.38 > 1.2

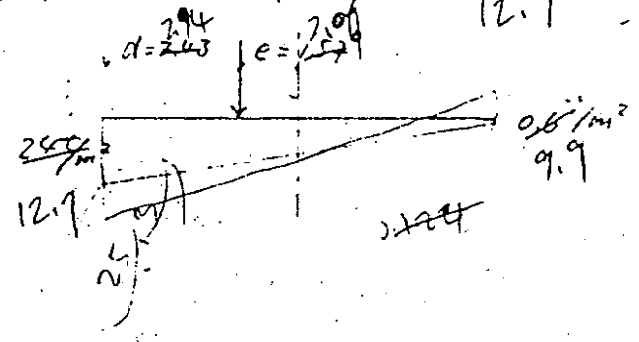
§8. 地盤反力

(1) 常時

M = 8113 − 4302 = 3811 tm
N = 1589 − 478 = 1111 t
d = 3811 / 1111 = 3.43 m
e = 10.0/2 − 3.43 = 1.57 < B/6 = 1.66 m

$\sigma = \frac{N}{A}\left(1 \pm \frac{6e}{B}\right)$ ∴ $\frac{2N}{3dl}$

$= \frac{102}{9\times10}\left(1 \pm \frac{6\times2.06}{10}\right) = 24.0 \quad 0.6\ t/m^2$
$12.7 \quad 9.9$



(2) 地震時

鉛直震度 +0.1

$M = 9299 - 5568 = 3731$

$N = 1589\times1.1 - 366\times1.1 = 1345$
1279

$d = \frac{3731}{1345} = 2.77^m$
$1279 \quad 2.76$

$e = \frac{10.0}{2} - 2.77 = 2.23^m < \frac{B}{3} = 3.33^m$
$2.76 \quad 2.24$

$\sigma = \frac{2N}{3dl} = \frac{2\times1345}{3\times2.77\times9}$
$1279 \quad 2.76$

$= 36.0\ t/m^2$
34.7

鉛直震度 −0.1

$M = 7608 - 5694 = 1914\ t\cdot m$

$N = 1589\times0.9 - 366\times0.9 = 1101^t$
$426.6 \quad 1046$

$d = \frac{152}{1101} = 1.92$
$1046 \quad 1.83$

$e = \frac{10.0}{2} - 1.92 = 3.08^m$
$1.83 \quad < \frac{B}{3} = 3.33^m$

$\sigma = \frac{2\times1046}{3\times1.92\times9} = 42.5\ t/m^2$
$1.83 \quad 42.4$

§9. 滑動安全率

(1) 常時

土圧とコンクリートとの摩擦係数 $\mu = 0.5$ とする

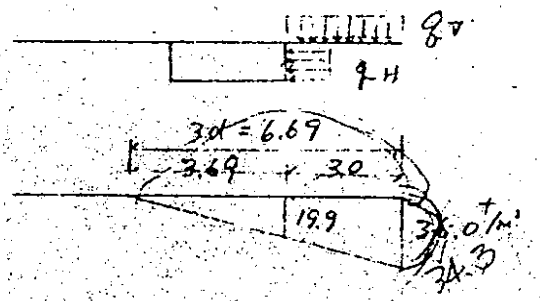
水平力 $H = 24.3 \times 8.8 = 214^{T}$ (土圧)

安全率 $F = \dfrac{N \cdot \mu}{H} = \dfrac{(1589 - 478) \times 0.5}{214} = 2.6 \quad > 2.0$

(2) 地震時

水平力 $H = 922^{t}$

鉛直震度 $+0.1$



$q_H = q_V \cdot \tan^2\left(45^\circ + \dfrac{\phi}{2}\right)$

$= \dfrac{36.0 + 19.9}{2} \times \tan^2(45^\circ + 10.2^\circ)$

$28.0 \times 2.07 = 58.0^{t/m^2}$

抵抗水平力

$H_R = 58.0 \times 1.5 \times 9$

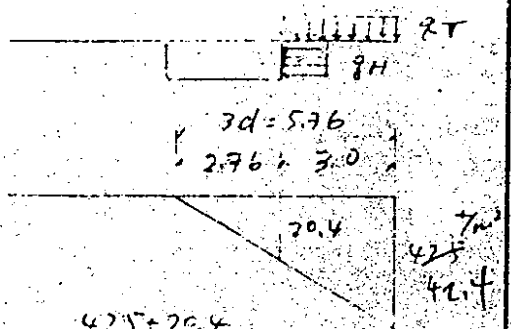
$+ \dfrac{1}{2} \times 19.9 \times 3.69 \times 9 \times 0.5$ (μ)

$= 783 + 165$

$= 948^{T}$

$F = \dfrac{948}{922} = 1.03$

鉛直震度 -0.1



$q_V = \dfrac{42.5 + 20.4}{2} = 36.5^{t/m^2}$

$q_H = 36.5 \times 2.07 = 75.5^{t/m^2}$

抵抗水平力

$H_R = 75.5 \times 1.5 \times 9$

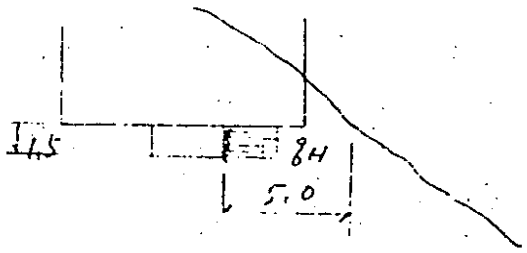
$+ \dfrac{1}{2} \times 20.4 + 2.76 \times 9 \times 0.5$

$= 1019 + 127 = 1146^{t}$

$F = \dfrac{1146}{922} = 1.24$

安全率が低いようであるが実際は地盤は岩盤であるから実証たわで水平力に対するように切り、次の

ような計算となる。



H = 922 t

岩盤に生じる圧縮応力

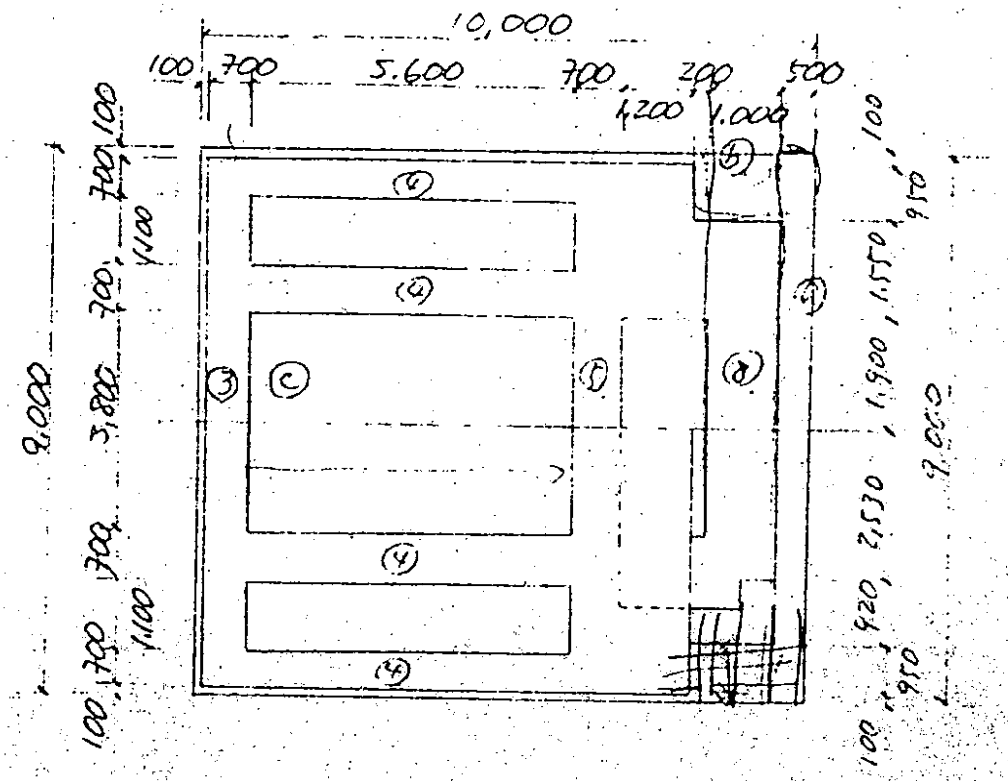
$$\sigma_H = \frac{N}{A} = \frac{922}{1.5 \times 9} = 68.3\ t/m^2$$

剪断応力

$$\tau_H = \frac{N}{A'} = \frac{922}{5.0 \times 9} = 20.5\ t/m^2$$

十分安全である。

§10. 橋台各部の鉄筋量の計算



1. 底版先端Ⓐ部

地震時最大地盤反力 $w = 42.5\ t/m^2$

$$M = \frac{w\ell^2}{2} = \frac{42.5 \times 0.5^2}{2} = 5.3\ t\cdot m/m$$

$$A_s = \frac{M}{\sigma_{sa} \cdot 7/8 \cdot d} = \frac{5.3}{2.7 \times 7/8 \times 0.9} = 2.5\ cm^2/m$$

2. 底版先端Ⓑ部

$$M = \frac{42.5 \times 1.05^2}{2} = 23.5\ t\cdot m/m$$

$$A_s = \frac{23.5}{2.7 \times 7/8 \times 0.9} = 11.0\ cm^2/m$$

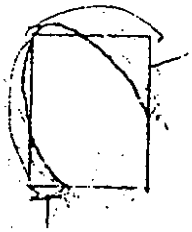
D16 @150 = 13.3 cm^2/m > 11.0 cm^2/m

ここは固定版として解くと

日本建築学会 鉄筋コンクリート構造 計算基準・同解説 477ページから

$\frac{l_y}{l_x} = \frac{1.7}{1.05} = 1.62$ に相当するMx, Myは

対する係数は 0.403, 0.400 である



$M_x = 0.403 \times 42.5 \times 1.05^2 = 18.7$ t·m/m

$M_y = 0.400 \times 42.5 \times 1.05^2 = 18.8$ t·m/m

Mxに対して

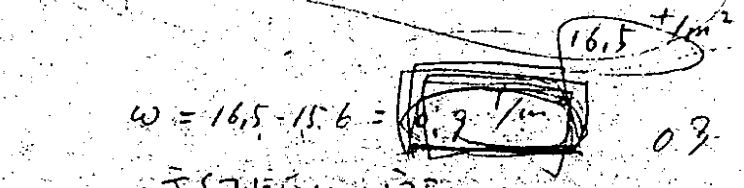
$A_s = \frac{18.9}{2.7 \times 7/8 \times 0.9} = 9.9$ cm²/m

D16@150 = 13.2 cm²/m

面t当. 6.6

3 床版の設計
常時

土砂の重量 8.1×1.6 = 13.0
版自重 1.0 ×2.5 = 2.5
通載荷重 1.0



ω = 16.5 - 15.6 = 0.9

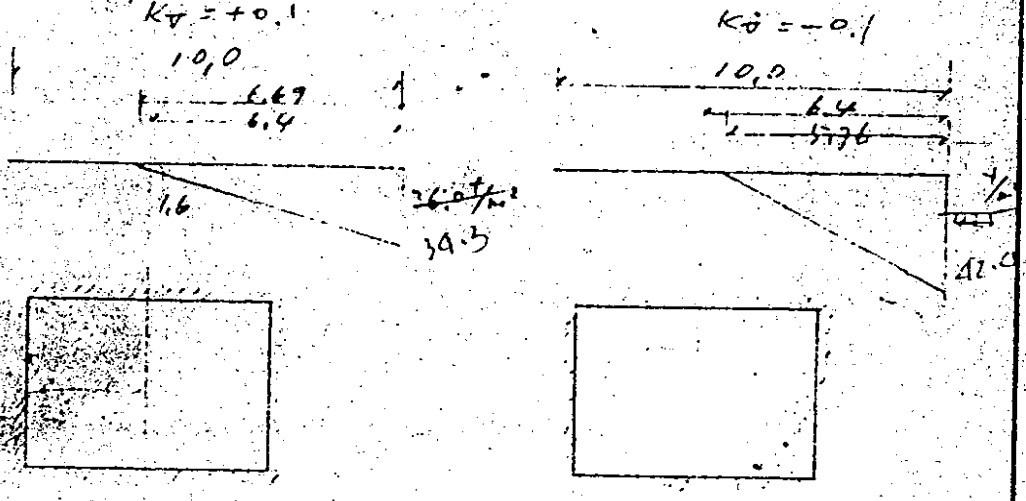
一方向版とし固定

立壁部 $R_1 = \frac{\omega \ell}{12} = \frac{0.9 \times 4.5^2}{12} = 1.52$ t·m/m

$A_s = \frac{1.52}{1.8 \times 7/8 \times 0.9} = 1.07$ cm²/m (一段)

D16 @10.0 = 19.86 cm²/m > 1.07 cm²/m

地震時



土砂の重量 13.0
躯体自重 2.5
15.5 t/m²

$K_v = +0.1$

$\omega = 15.5 \times 1.1 - 1.6 = 15.5$

$K_v = -0.1$

$\omega = 15.5 \times 0.9 = 14.0$

$$M = \frac{\omega \ell^2}{12} = \frac{15.5 \times 4.5^2}{12} = 26.2 \text{ t·m/m}$$

$$A_s = \frac{26.2}{2.7 \times 7/8 \times 0.9} = 12.3 \text{ cm}^2/\text{m} \quad (\text{上側})$$

D16 @150 = 13.2 cm²/m > 12.3 cm²/m

4. 後横壁③

常時は前後の土圧が釣合って曲げモーメントは生じない。

地震時

$$\text{土圧 } \varepsilon = \omega \cdot \frac{1-\sin\phi_e}{1+\sin\phi_e} \cdot h = 1.6 \cdot \frac{1-0.348}{1+0.348} \cdot 7.91$$

$$= 6.13 \text{ t/m}^2$$

壁体自重 $p = 1 \times 0.7 \times 2.5 \times 0.15 = 0.26$ t/m²

反対側からの主働土圧

$$\varepsilon' = \omega \cdot \frac{1-\sin\phi}{1+\sin\phi} \cdot h = 1.6 \cdot \frac{1-0.5}{1+0.5} \cdot 7.91 = 4.21 \text{ t/m}^2$$

$$\omega = 6.13 + 0.26 - 4.21 = 2.18 \text{ t/m}^2$$

$$M = \frac{2.18 \times 4.5^2}{12} = 3.68 \text{ t·m/m}$$

$$A_s = \frac{3.68}{2.7 \times 7/8 \times 0.62} = 2.5 \text{ cm}^2/\text{m} \quad \text{D16 @300}$$

$= 6.6 \text{ cm}^2/\text{m} > 2.5 \text{ cm}^2/\text{m}$

地震が後ろへ作用したとき

受働土圧

$$\varepsilon = 1.6 \times \frac{1+0.348}{1-0.348} \times 7.91 = 26.0 \ t/m^2$$

実際に働く受働土圧は

$$\varepsilon_1 = \frac{1}{1.76} \times 26.0 = 14.8 \ t/m^2$$

反対方向からの主働土圧

$\varepsilon' = 6.13 \ t/m^2$

壁体自重 $0.26 \ t/m^2$

$$\omega = 14.8 - 6.13 - 0.26 = 8.41 \ t/m^2$$

$$M = \frac{8.41 \times 4.5^2}{12} = 14.2 \ t\cdot m/m$$

$$A_s = \frac{14.2}{2.7 \times 7/8 \times 0.62} = 9.7 \ cm^2/m$$

三辺固定版として計算すると

日本建築学会の鉄筋コンクリート構造計算規準・同解説

476 ページから

$\frac{l_y}{l_x} = \frac{7.92}{4.5} = 1.76$ に相当する M_x, M_y に対する係数は同書より 0.044, 0.038

$$M_x = 0.044 \times 8.41 \times 4.5^2 = 7.50 \ t\cdot m/m$$

$$M_y = 0.038 \times 8.41 \times 4.5^2 = 6.48 \ t\cdot m/m$$

M_x について

$$A_s = \frac{7.50}{2.7 \times 7/8 \times 0.62} = 5.12 \ cm^2/m$$

D16 @300 $= 6.6 \ cm^2/m > 5.12 \ cm^2/m$

5 底版 ⑩

$\omega = 2.18\ t/m^2$

$M = \frac{2.18 \times 6.3^2}{12} = 7.21\ t\cdot m/m$

$A_s = \frac{7.21}{2.7 \times 7/8 \times 0.62} = 4.92\ cm^2/m$

D16@300 = 6.6 cm^2/m > 4.92 cm^2/m

6 横壁 ⑨

後壁⑬と同寸

7 前壁 ⑧

常時

土圧 $e = \omega \cdot \frac{1-\sin\phi}{1+\sin\phi} \cdot h = 1.6 \cdot \frac{1-0.5}{1+0.5} \cdot 4.4 = 2.35\ t/m^2$

$M = \frac{2.35 \times 4.5^2}{12} = 3.96\ t\cdot m/m$

$A_s = \frac{3.96}{1.2 \times 7/8 \times 0.92} = 2.74\ cm^2/m$

D16@300 = 6.6 cm^2/m > 2.74 cm^2/m

地震時

土圧 $e = \omega \cdot \frac{1-\sin\phi_e}{1+\sin\phi_e} \cdot h = 1.6 \cdot \frac{1-0.348}{1+0.348} \cdot 4.4 = 3.41\ t/m^2$

躯体慣性 $p = 1 \times 1 \times 1.5 \times 0.15 = 0.38\ t/m^2$

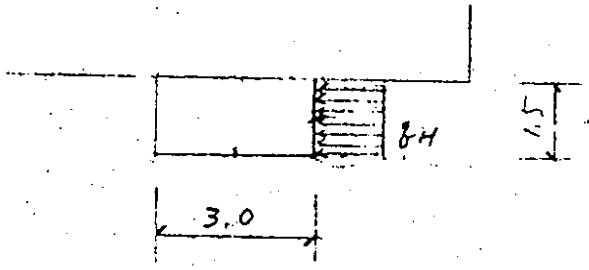
$\omega = 3.41 + 0.38 = 3.79\ t/m^2$

$M = \frac{3.79 \times 4.5^2}{12} = 6.40\ t\cdot m/m$

$A_s = \frac{6.40}{2.7 \times 7/8 \times 0.92} = 2.94\ cm^2/m$

D16@300 = 6.6 cm^2/m > 2.94 cm^2/m

8 突起



$$8H = \frac{H}{A} = \frac{922}{1.5 \times 9} = 68.3\ t/m^2$$

$$M = \frac{68.3 \times 1.5^2}{2} = 76.8\ t\cdot m/m$$

$$As = \frac{76.8}{2.7 \times 7/8 \times 2.9} = 11.2\ cm^2/m$$

地耐力は地震時 120 t/m^2 までとするので突起の高さを1mとすれば

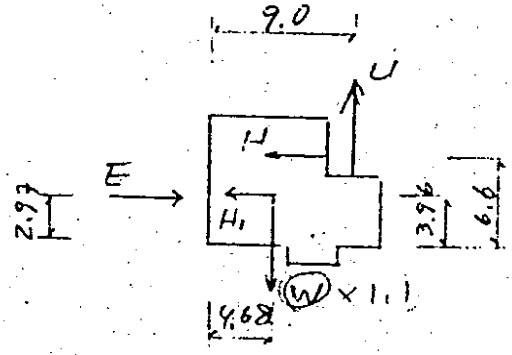
$$8H = \frac{H}{A} = \frac{922}{1.0 \times 9} = 102.4\ t/m^2 < 120\ t/m^2$$

$$M = \frac{102.4 \times 1.0^2}{2} = 51.2\ t\cdot m/m$$

$$As = \frac{51.2}{2.7 \times 7/8 \times 2.9} = 7.47\ cm^2/m$$

D16 @150 = 13.2 cm^2/m

地震時（後方へ地震が作用したとき）



受働土圧

$$\varepsilon = \omega \cdot \frac{1+\sin\phi_e}{1-\sin\phi_e} \cdot h_1 = 1.6 \times \frac{1+0.342}{1-0.342} \times 8.91 = 29.3 \ t/m^2$$

$$E = \frac{\varepsilon}{2} \cdot h \cdot B = \frac{29.3}{2} \times 8.91 \times 8.8 = 1149^t$$

$K_v = +0.1$

転倒モーメント

U	427 366×1.1×9.0	4227 3627
H	434 413×6.6	2864 2726
H_1	238×3.96	942
		7295 t·m
		8033

抵抗モーメント

W	1589×1.1×4.68	8170
E	1149×2.97	3410
		11580 t·m

$$F = \frac{11580}{7295} = 1.59$$

$$8022 \quad 1.44 > 1.2$$

ok

$K_v = -0.1$

U	427 366×0.9×9.0	3459 2961
H	434 413×6.6	2864 2726
H_1	238×3.96	942
		6629 t·m
		7265

抵抗モーメント

W	1589×0.9×4.68	6690
E	1149×2.97	3410
		10100 t·m

$$F = \frac{10100}{6629} = 1.52$$

$$7252 \quad 1.39 > 1.2$$

ok

土圧力

$K_\tau = +0.1$

$M = 11580 - 7295 = 4285$

$N = 1589 \times 1.1 - 366 \times 1.1 = 1345^{t}$

$d = \frac{4285}{1345} = 3.18^{m}$

$e = \frac{10.0}{2} - 3.18 = 1.82^{m} < \frac{B}{3} = 3.33^{m}$

$\sigma = \frac{2 \times 1345}{3 \times 3.18 \times 9} = 31.4^{t/m^2}$

$K_\tau = -0.1$

$M = 10100 - 6629 = 3471$

$N = 1585 \times 0.9 - 366 \times 0.9 = 1101^{t}$

$d = \frac{3471}{1101} = 3.15^{m}$

$e = \frac{10.0}{2} - 3.15 = 1.85^{m} < \frac{B}{3} = 3.33^{m}$

$\sigma = \frac{2 \times 1101}{3 \times 3.15 \times 9} = 25.8^{t/m^2}$

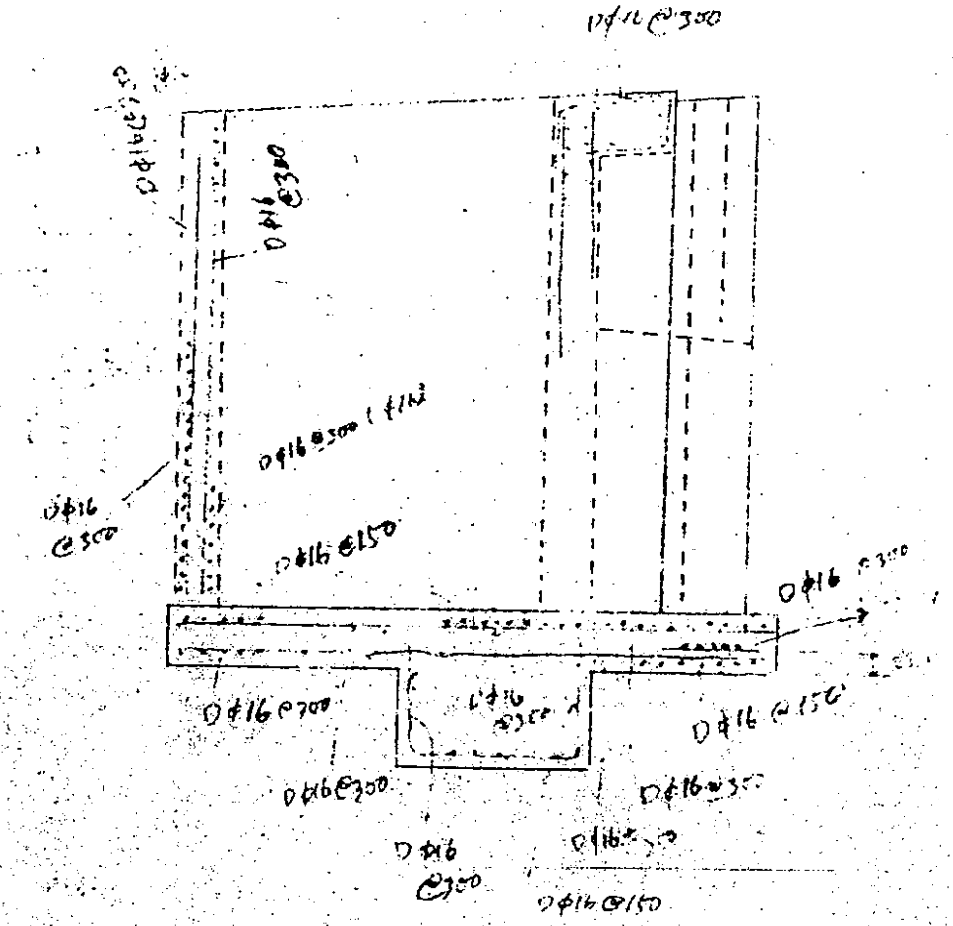
滑動力に対して

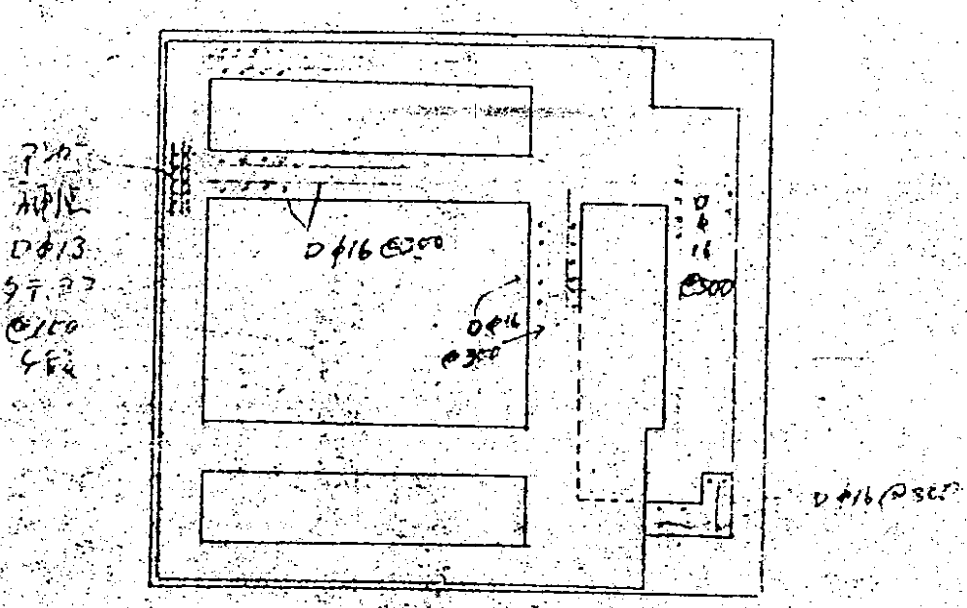
作用する水平力 $H = 413 + 238 = 651^{t}$

抵抗土圧力 $E = 1149^{t}$

$F = \frac{1149}{651} = 1.76$

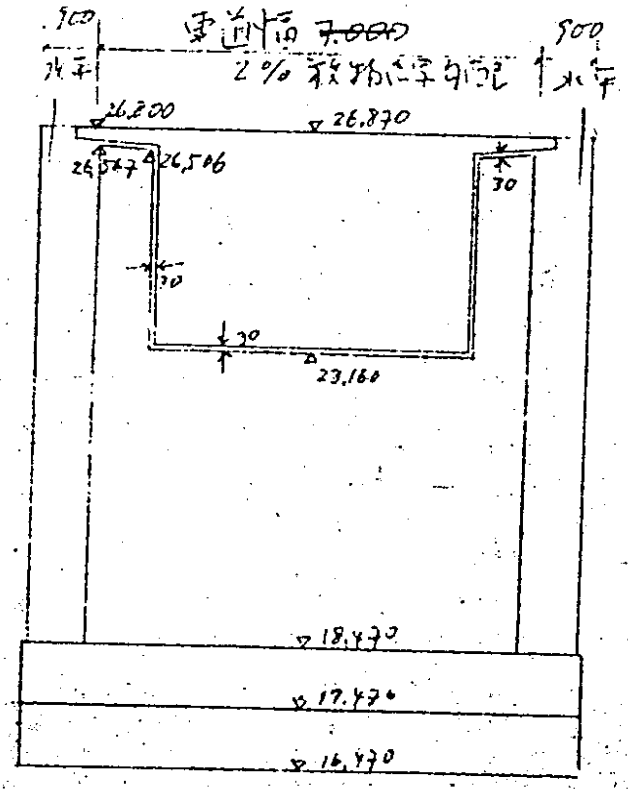
S11 鉄筋配置図

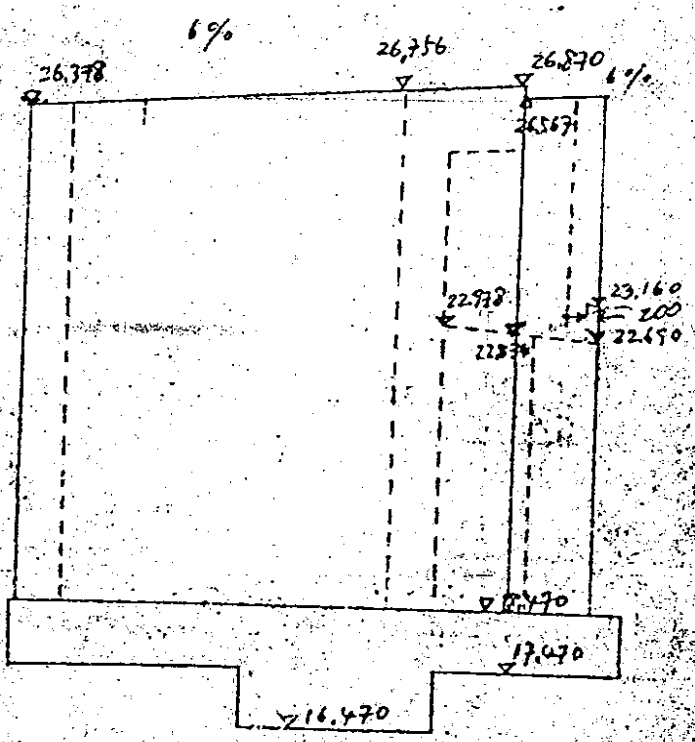

D16@300
D16@300
D16@300
D16@150
D16@300
D16@700
D16@300
D16@150
D16@300
D16@300
D16@150

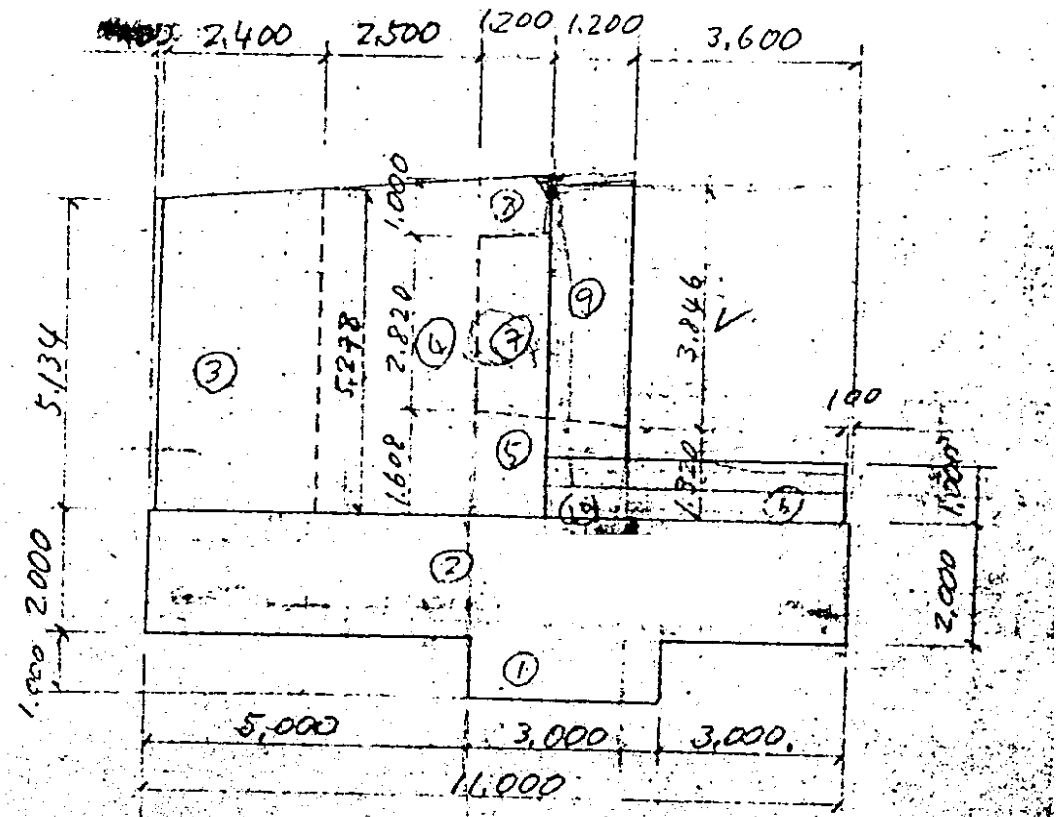

D16@300
D16@300
D16@300

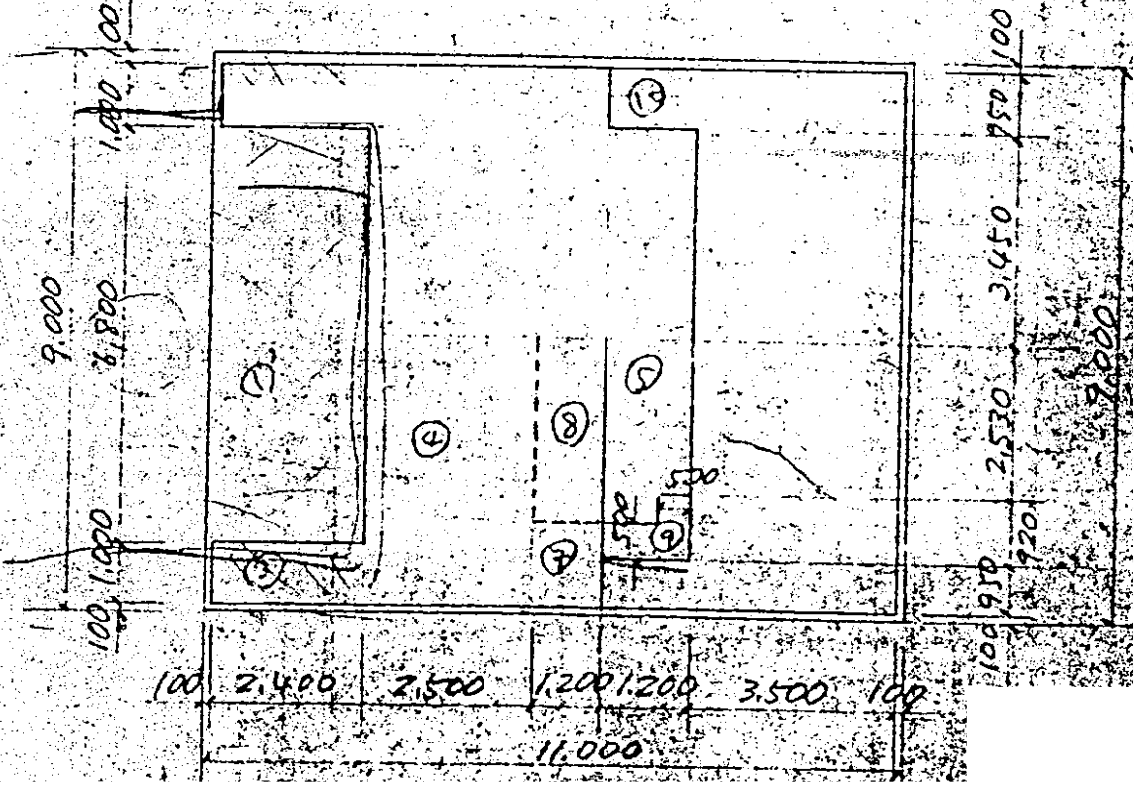
Ⅱ. 右岸橋台.

§1. 寸法.


500
500
26,200
26,870
26,547
26,506
30
30
30
23,160
18,470
17,470
16,470


6%
6%
26,378
26,756
26,870
26,547
22,978
23,160
200
22,960
22,776
18,470
17,470
16,470


2,400
2,500
1,200
1,200
3,600
1,000
5,134
5,278
2,820
3,846
1,608
100
2,000
1,200
2,000
5,000
3,000
3,000
11,000


100
1,000
100
950
3,450
2,530
920
100
100
2,400
2,500
1,200
1,200
3,500
100
11,000

第2 重量

コンクリート

①	3 × 1 × 9	=	27.0
②	11 × 2 × 9	=	198.0
③	2.4 × 5,206 × 1 × 2	=	25.0
④	2.5 × 5,353 × 8.8	=	117.8
⑤	2.4 × 1.464 × 6.9	=	24.2
⑥	3.5 × 1 × 8.8	=	30.8
⑦	1.2 × 3.82 × 1.45 × 2	=	13.3
⑧	1.2 × 1 × 5.9	=	7.1
⑨	(1.2 × 0.5 + 0.5 × 0.42) × 3.846 × 2	=	6.2
⑩	1.2 × 1 × 0.95 × 2	=	2.3

$\Sigma V = 451.7\ m^3$

$W = 451.7 \times 2.5 = 1129\ t$

土砂

① $25. \times 5.206 \times 6.8 = 88.5\ m^3$

$W = 88.5 \times 1.6 = 142\ t$

$\Sigma W = 1129 + 142 = 1271\ t$

§4 常時背面土圧

$h = 7.134^{m}$

遮載荷重 $1^{t}/m^{2}$ ⟶ $\Delta h = 0.65^{m}$

$$E_1 = \omega \frac{1-\sin\phi}{1+\sin\phi} \cdot h \cdot \Delta h \qquad \phi = 30^{\circ} \quad \sin\phi = 0.5$$

$$= 1.6 \times \frac{1-0.5}{1+0.5} \times 7.134 \times 0.65 = 2.47^{t/m}$$

作用高 $y = \frac{h}{2} = \frac{7.134}{2} = 3.567^{m}$

$$E_2 = \omega \cdot \frac{1-\sin\phi}{1+\sin\phi} \cdot \frac{h^2}{2}$$

$$= 1.6 \times \frac{1-0.5}{1+0.5} \cdot \frac{7.134^2}{2} = 13.56^{t/m}$$

作用高 $y = \frac{h}{3} = \frac{7.134}{3} = 2.378$

$$E = E_1 + E_2 = 2.47 + 13.56 = 16.0^{t/m}$$

作用高 $y = \frac{2.47 \times 3.567 + 13.56 \times 2.378}{16.0} = 2.54^{m}$

§5 地震時背面土圧

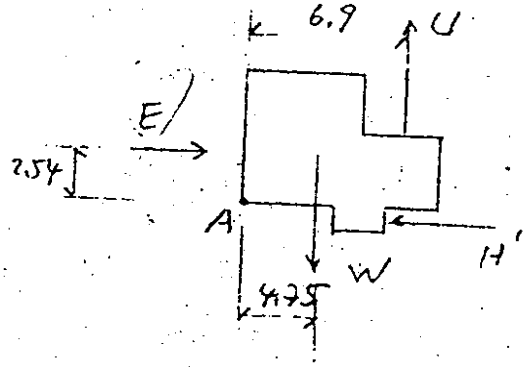
$$E = \omega \cdot \frac{1-\sin\phi_e}{1+\sin\phi_e} \cdot \frac{h^2}{2} \qquad \sin\phi_e = 0.348$$

$$= 1.6 \times \frac{1-0.348}{1+0.348} \cdot \frac{7.134^2}{2} = 19.7^{t/m}$$

作用高 $y = \frac{h}{3} = 2.38^{m}$

6. 転倒安全率

1. 常時



後方転倒モーメント (A矢)

~~568~~ 3920

U: ~~568~~ × 6.9 = ~~3998~~

抵抗モーメント

W: 1271 × 4.75 = 6037

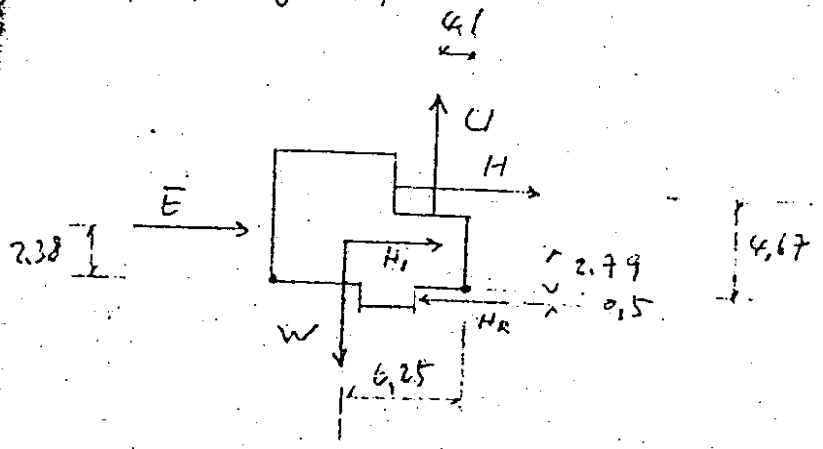
E: 160 × ~~8.8~~ (141) × 2.54 = 358

H': 141 × 0.5 = 70

6465

$F = \frac{6461}{\text{~~3998~~ } 3920} = \text{~~1.9~~ } 1.65 > 1.5$

2、地震時（分方）



$K_V = -0.1$

転倒モーメント

U : 366 × 1.1 × 4.1 = ~~1651~~ 1921

H : ~~433~~ 434 × 4.67 = ~~1929~~ 2030

H1 : 191 × 2.79 = 533

E : $\frac{197 \times 8.8 \times 2.38}{123}$ = 412

HR : 777 × 0.5 = 389

~~4944~~ 5285

抵抗モーメント

W : 1271 × 1.1 × 6.25 = 8738

F = $\frac{8738}{5285}$ = ~~1.7~~ 1.65

$K_V = -0.1$

転倒モーメント

U : 366 × 0.9 × 4.1 = ~~1351~~ 1570

H : 426 ~~1929~~ 2030

H1 : 533

E : 412

HR : 389

抵抗モーメント

W : 1271 × 0.9 × 6.4 = 7149 ~~8934~~

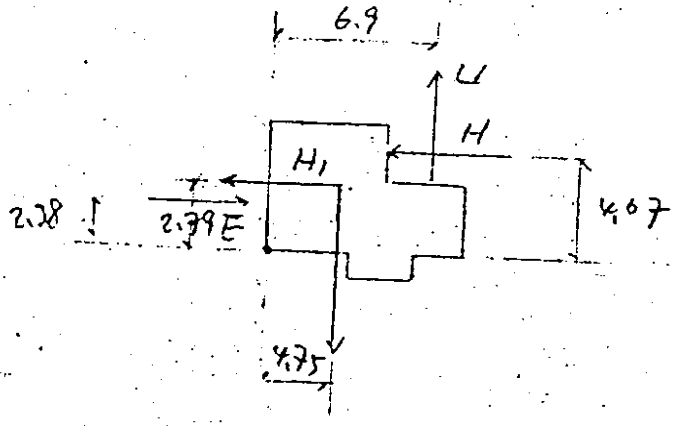
F = $\frac{7149}{4934}$ = 1.5

3、地震時（後方）

背面受働土圧

$E = w \cdot \frac{1+\sin\phi_e}{1-\sin\phi_e} \cdot \frac{h^2}{2}$

$= 1.6 \times \frac{1+0.748}{1-0.348} \cdot \frac{7.174^2}{2} = 84.1$ t/m



Kr = +0.1

転倒モーメント

U : 426.6 × 1.1 × 6.9 = ~~366~~ 3240 ~~2778~~

H : 434 × 4.67 = ~~413~~ 2030 ~~1929~~

H1 : 191 × 2.79 = 533

~~5240~~ 5803

抵抗モーメント

W : 1271 × 1.1 × 4.75 = 6641

E : 24.1 × 3.1 × 2.38 = 1761

8402

F = 8402 / ~~5240~~ 5803 = 1.5 (1.45)

Kr = −0.1

転倒モーメント

U : 426.6 × 0.9 × 6.9 = ~~366~~ 2650 ~~2273~~

H : ~~2030~~ ~~1929~~

H1 : 533

~~4755~~ 5213

抵抗モーメント

W : 1271 × 0.9 × 4.75 = 5432

E : 1761

7195

F = 7195 / ~~4755~~ 5213 = 1.5 = 1.43

7. 地盤反力

1 常時

$M = 6465 - \overset{3920}{\cancel{3298}} = \overset{2545}{\cancel{3167}}$

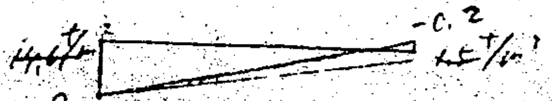
$N = 1271 - \overset{568}{\cancel{478}} = \overset{703}{\cancel{793}}$

$d = \dfrac{\overset{2545}{\cancel{3167}}}{\overset{703}{\cancel{793}}} = \overset{3.62}{\cancel{4.00}}$

$e = \dfrac{11}{2} - \overset{3.62}{\cancel{4.00}} = \overset{1.88}{\cancel{1.50}}\ m$

$\sigma = \dfrac{N}{A}\left(1 \pm \dfrac{6e}{13}\right)$ 1.025

$= \dfrac{\overset{703}{\cancel{793}}}{99}\left(1 \pm \dfrac{6 \times \overset{1.88}{\cancel{1.50}}}{11}\right) = \overset{15.8}{\cancel{14.6}}\ t/m^2,\ \overset{-0.2\ t/m^2}{\cancel{1.5}}\ t/m^2$



2 地震時(前方)

$K_H = +0.15$

$M = 8238 - \overset{5285}{\cancel{4414}} = \overset{3453}{\cancel{3824}}\ t$

$N = 1271 \times 1.1 - \overset{426.6}{\cancel{366}} \times 1.1 = \overset{930}{\cancel{995}}\ t$

$d = \dfrac{\overset{3453}{\cancel{3824}}}{\overset{930}{\cancel{995}}} = \overset{3.71}{\cancel{3.85}}\ m$

$e = \dfrac{11}{2} - \overset{3.71}{\cancel{3.85}} = \overset{1.79}{\cancel{1.65}}\ m$

$< \dfrac{11}{3} = 3.68$

$\sigma = \dfrac{\overset{930}{\cancel{995}}}{99}\left(1 \pm \dfrac{6 \times \overset{1.79}{\cancel{1.65}}}{11}\right)$ 0.98

$= \overset{18.6}{\cancel{19.1}}\ t/m^2,\ \overset{0.2}{\cancel{1.0}}\ t/m^2$

$K_H = -0.14934$

$M = 7149 - \overset{4833}{\cancel{4614}} = \overset{2215}{\cancel{2535}}\ t$

$N = 1271 \times 0.9 - \overset{426.6}{\cancel{366}} \times 0.9 = \overset{761}{\cancel{815}}\ t$

$d = \dfrac{\overset{2215}{\cancel{2535}}}{\overset{761}{\cancel{815}}} = \overset{2.92}{\cancel{3.11}}\ m$

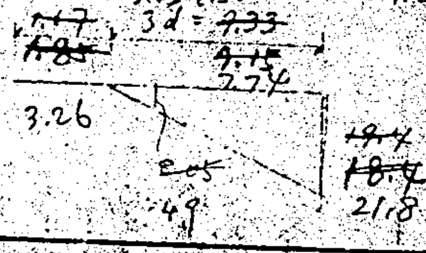
$e = \dfrac{11}{2} - \overset{2.92}{\cancel{3.11}} = \overset{2.58}{\cancel{2.39}}\ m$

$< \dfrac{13}{3} = \dfrac{11}{3} = 3.67$

$\sigma = \dfrac{2N}{3d\ell}$

$= \dfrac{2 \times \overset{759}{\cancel{815}}}{3 \times \overset{2.58}{\cancel{3.11}} \times 9} = \overset{21.8}{\cancel{19.4}}\ t/m^2$ (~~18.4~~)

$3d = \overset{7.74}{\cancel{9.33}}$ (~~9.15~~)



地震時（後方）

$K_v = +0.1$

$M = 8402 - \sout{5240}\ 5803 = \sout{3162}\ 2599$ t·m

$N = 1271 \times 1.1 - \sout{360} 426.6 \times 1.1 = \sout{995}\ 930$ T

$d = \dfrac{\sout{3162}\ 2599}{\sout{995}\ 930} = \sout{3.18}\ 2.80$ m

$e = 5.5 - \sout{3.18}\ 2.80 = \sout{2.32}\ 2.70$ m

$\sigma = \dfrac{2 \times \sout{995}\ 930}{3 \times \sout{3.18}\ 2.80 \times 9} = \sout{23.2}\ 24.7$ t/m²

$3d = \sout{9.54}\ 8.40$ m $\quad \sout{4.46}\ 2.60$

$K_v = -0.1$

$M = 7195 - \sout{4735}\ 5213 = \sout{2460}\ 1982$ t·m

$N = 1171 \times 0.9 - \sout{365} 426.6 \times 0.9 = \sout{815}\ 759$ T

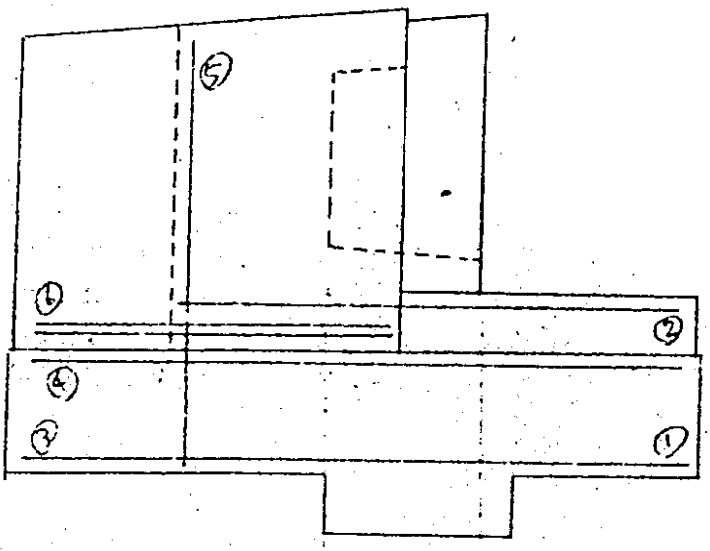
$d = \dfrac{\sout{2460}\ 1982}{\sout{815}\ 759} = \sout{3.02}\ 2.61$ m

$e = 5.5 - \sout{3.02}\ 2.61 = \sout{2.48}\ 2.89$ m

$\sigma = \dfrac{2 \times \sout{815}\ 759}{3 \times \sout{3.02}\ 2.61 \times 9} = \sout{20.0}\ 21.6$ t/m²

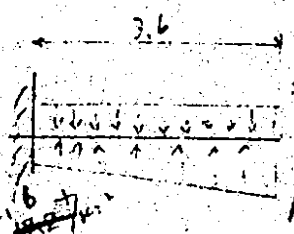
$3d = \sout{9.06}\ 7.83$ $\quad \sout{1.94}\ 3.17$ m

3. 鉄筋量の計算



鉄筋①

A断面

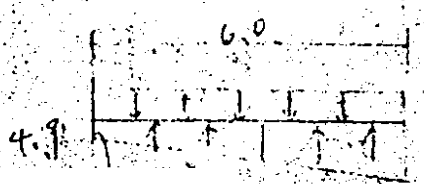


$$M = \frac{12.6}{2} \times 3.6^2 + \frac{6.0}{3} \times 3.6^2 - \frac{7.5 \times 3.6^2}{2} = 59.1 \text{ t-m/m}$$

$$A_s = \frac{59.1}{62.5} = \frac{8.6 \text{ cm}^2/\text{m}}{2.7 \times 7/8 \times 2.9}$$

Dϕ19@150 = 19.1 cm²/m > 8.6 cm²/m

B断面

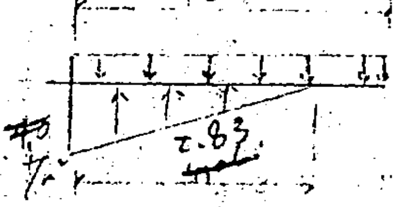


$$M = \frac{8.6}{4.12} \times 6^2 + \frac{16.9}{3} \times 6^2 - \frac{7.5 \times}{2} = 156.3 \text{ t-m/m}$$

$$A_s = \frac{156.3}{2.7 \times 7/8 \times 3.5} = 21.3 \text{ cm}^2/\text{m}$$

Dϕ19@150 = 19.1 cm²/m

鉄筋 ②



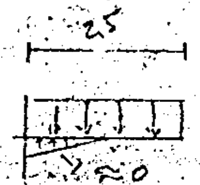
$$M = \frac{7.5 \times 6^2}{2} - \frac{7.8 \times 2.83^2}{6} = 125 \ \text{t·m/m}$$

$$A_s = \frac{125}{2.7 \times 7/8 \times 3.5} = 15.2 \ \text{cm}^2/\text{m}$$

D ϕ19 @150 = 19.1 cm²/m > 15.2 cm²/m

鉄筋 ③ 鉄筋①で十分

鉄筋 ④



$5.134 \times 1.6 + 2 \times 2.5 = 13.2 \ \text{t/m}^2$

$$M = \frac{13.2 \times 2.5^2}{2} = 41.3 \ \text{t·m/m}$$

$$A_s = \frac{41.3}{2.7 \times 7/8 \times 1.9} = 9.2 \ \text{cm}^2/\text{m}$$

D ϕ19 @300 = 9.5 cm²/m > 9.2 cm²/m

鉄筋 ⑤

$$M = 434 \times 1.2 = 521 \ \text{t·m}$$

$$A_s = \frac{521}{2.7 \times 7/8 \times 2.4} = 92 \ \text{cm}^2$$

$$\frac{92}{8.8} = 10.5 \ \text{cm}^2/\text{m}$$

D ϕ19 @150 = 19.1 cm²/m > 10.5 cm²/m

鉄筋 ⑥

土圧(常時)

$$P = 1.6 \times \frac{1}{3} \times 0.65 \times 1.6 \times \frac{1}{2.0} \times 5.134 = 3.09 \ \text{t/m}^2$$

M = [illegible] = 9.7 t·m

$$A_s = \frac{9.7}{2.7 \times 7/8 \times 0.6} = 6.85 \ \text{cm}^2/\text{m}$$

D ϕ16 @300 = 6.6 cm²/m ≒ 6.85 cm²/m

(安全側の計算をしているため)

土圧(地震時)

$$P = 1.6 \times \frac{0.6^2}{1.348} \times 5.134 = 3.97$$

自立水平力 $2.5 \times 0.15 = 0.38$

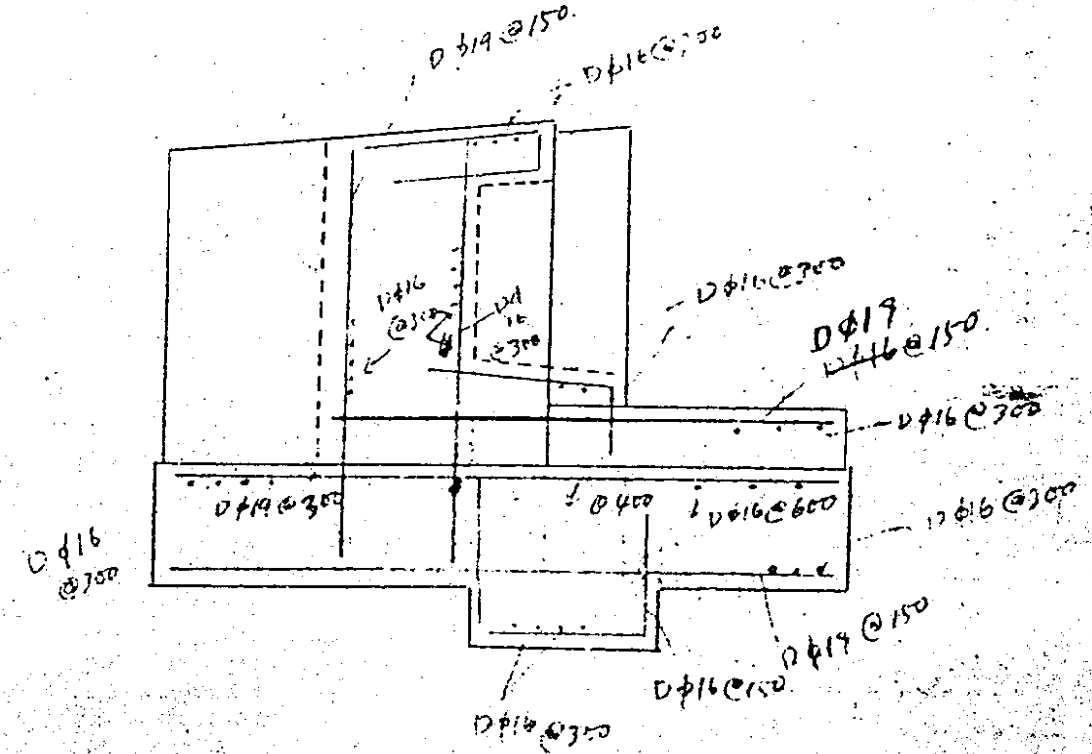
$4.35\ t/m^2$

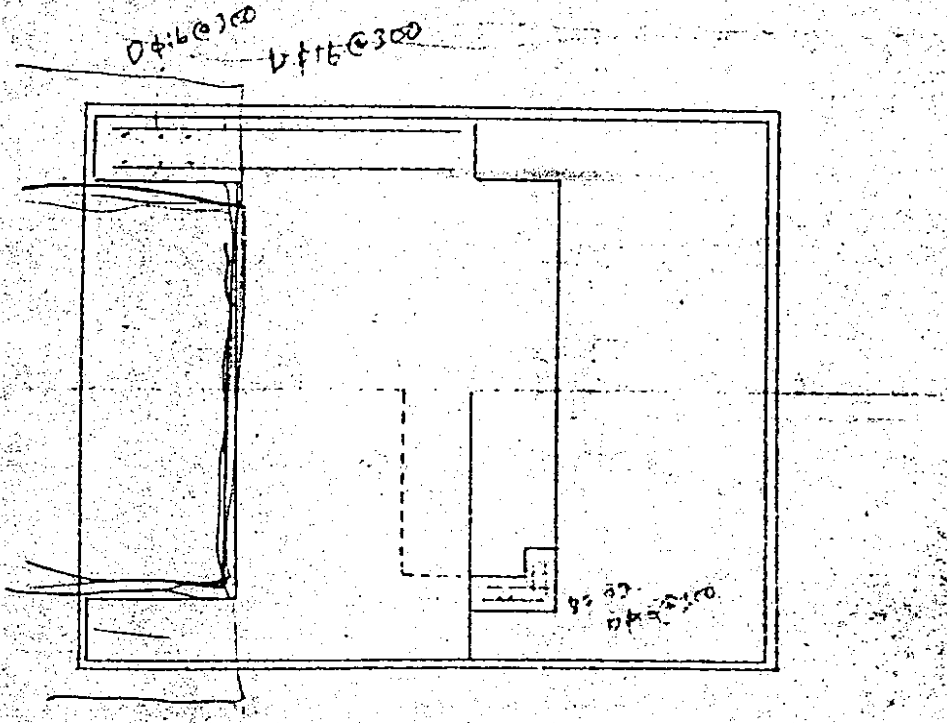
$$M = \frac{4.35 \times 2.5^2}{2} = 13.6\ t\cdot m/m$$

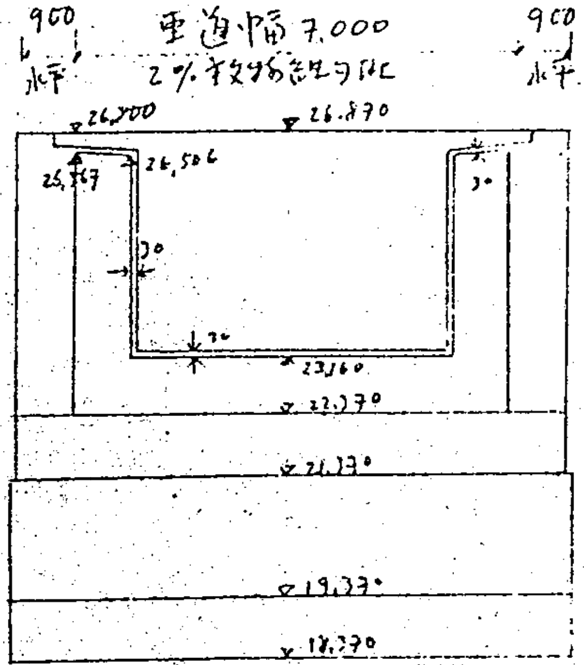
$$As = \frac{13.6}{2.7 \times 7/8 \times 0.9} = 6.40\ cm^2/m$$

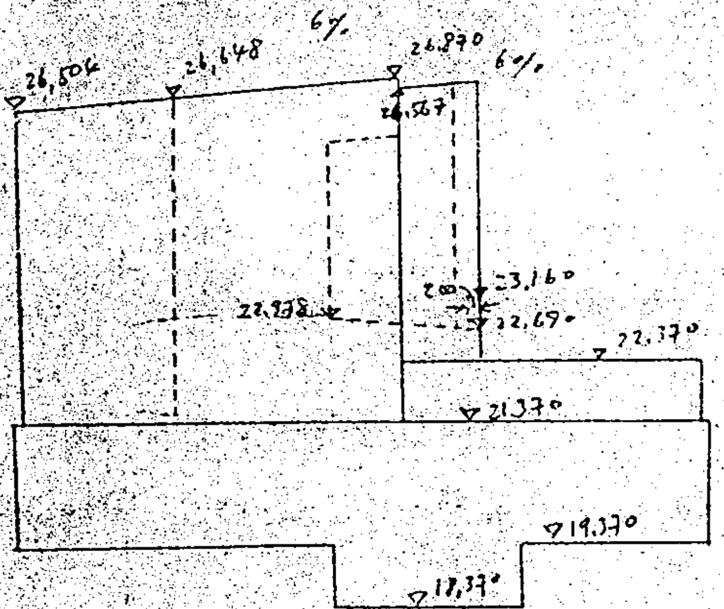
D16@300 = 6.6 cm^2/m > 6.40 cm^2/m

§9. 鉄筋配置図.


D φ19 @150.
D φ16 @300
D φ16 @300
D φ19
D φ16 @150.
D φ16 @300
D φ16 @300
D φ19 @300
D φ16 @600
D φ16 @300
D φ16 @300
D φ19 @150
D φ16 @150
D φ16 @300

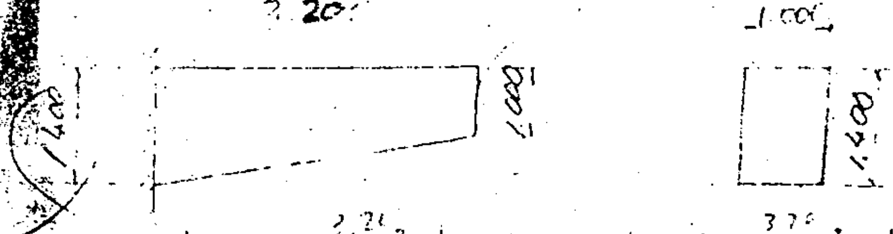

D φ16 @300
D φ16 @300


900
水平
車道幅7,000
900
水平
26,900
26,870
26,506
30
30
30
23,160
22,370
21,370
19,370
18,370


6%
26,870
6%
26,506
26,648
26,567
23,160
200
22,838
22,690
22,370
21,370
19,370
18,370

[illegible]

1) 設計荷重 [illegible]



(自重) $M_g = 1.00 \times 3.20^2 \times 1/2 \times 2.5 + 0.40 \times 1/2 \times 3.20^2 \times 1/3 \times 2.5$

$= 12.8 + 1.7 = 14.5^{t \cdot m}$

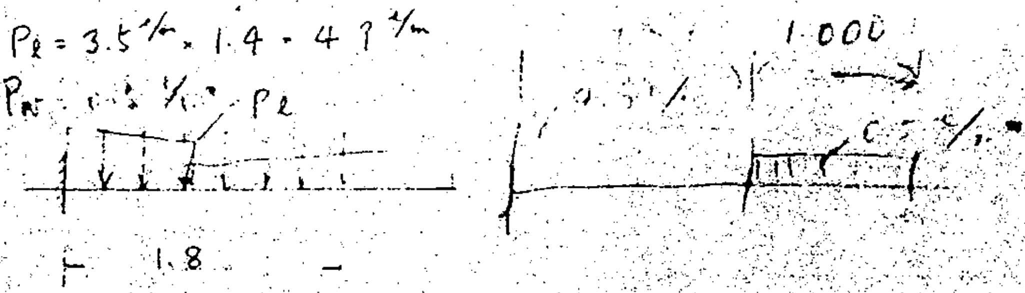
$S_g = 1.00 \times 3.20 \times 2.5 + 0.40 \times 1/2 \times 3.20 \times 2.5$

$= 8.0 + 1.6 = 9.6^{t}$

(活荷重)

$P_l = 3.5^{t/m^2} \times 1.4 = 4.9^{t/m}$

P_W : 0.5 t/m² , P_l



$M_l = 4.9 \times \frac{1.444^2}{2} = 5.4^{t \cdot m}$

$S_l = 4.9 \times 1.444 = 7.3^{t}$

(計)

$M_T = 14.5 + 6.4 = 20.9^{t \cdot m}$

$S_T = 9.6 + 7.8 = 17.4^{t}$

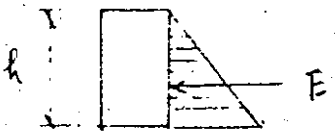
$M_l = 4.9 \times \frac{1.444^2}{2} + $ [illegible]

$= 5.4 + 1.0 = 6.4^{t \cdot m}$

$S_l = 4.9 \times 1.444 + 0.5 \times 1.00 m$

$= 7.3 + 0.5 = 7.8^{t}$

4) 地震時の土圧



$h = \frac{1}{2}(1.00 + 1.40) = 1.20\ m$

ランキンの公式より

$$E = \frac{1}{2} w \frac{1-\sin\phi}{1+\sin\phi} h$$

土の内部マサツ $\phi = 30^\circ \rightarrow \sin\phi = 0.5$

土の単位重量 $w = 1.6\ t/m^3$

$$\frac{1-\sin\phi}{1+\sin\phi} = \frac{1-0.5}{1+0.5} = \frac{1}{3}$$

$$E = \frac{1}{6} w h$$

$$= \frac{1}{6} \times 1.6 \times 1.20 = 0.32\ t/m$$

$$M_E = \frac{1}{2} \times E \times h^2 = \frac{1}{2} \times 0.32 \times 2.7^2 = 1.2\ t\cdot m$$

$$S_E = E \cdot h = 0.32 \times 2.7 = 0.9\ t$$

5) 鉄筋量の算定

a) 設計荷重時

$M_T = \frac{20.5}{18.3}$ t·m

b = 1.000 h = 1.400 d = 1.300

$$A_s = \frac{M_T}{7/8 \times \sigma_{sa} \times d}$$

$$= \frac{20.5}{7/8 \times 18.000 \times 1.30} = 1.40 \times 10^{-3} = 0.0090\ m^2$$

D16 - use $A_{s0} = 0.0001986$

$$n = \frac{A_s}{A_{s0}} = \frac{0.0090}{0.0001986} = \frac{4.5}{5.14} \rightarrow 15\ cm\ ピッチ$$

$S_T = 170$ t

$$\tau = \frac{S_T}{7/8 \times d} = \frac{17.4}{7/8 \times 1.30} = \frac{15}{20}\ t/m^2 < 65\ t/m^2$$

b) 地震時

$M_E = 1.2$ t·m

b = 1.250 h = 1.000 d = 0.850

$$A_s = \frac{1.2}{1.40 \times 7/8 \times 27.000 \times 0.85} = 0.043 \times 10^{-3} = 0.00043\ m^2$$

φ13 - use $A_{s0} = 0.0001267$

φ13 - 20cm ピッチ

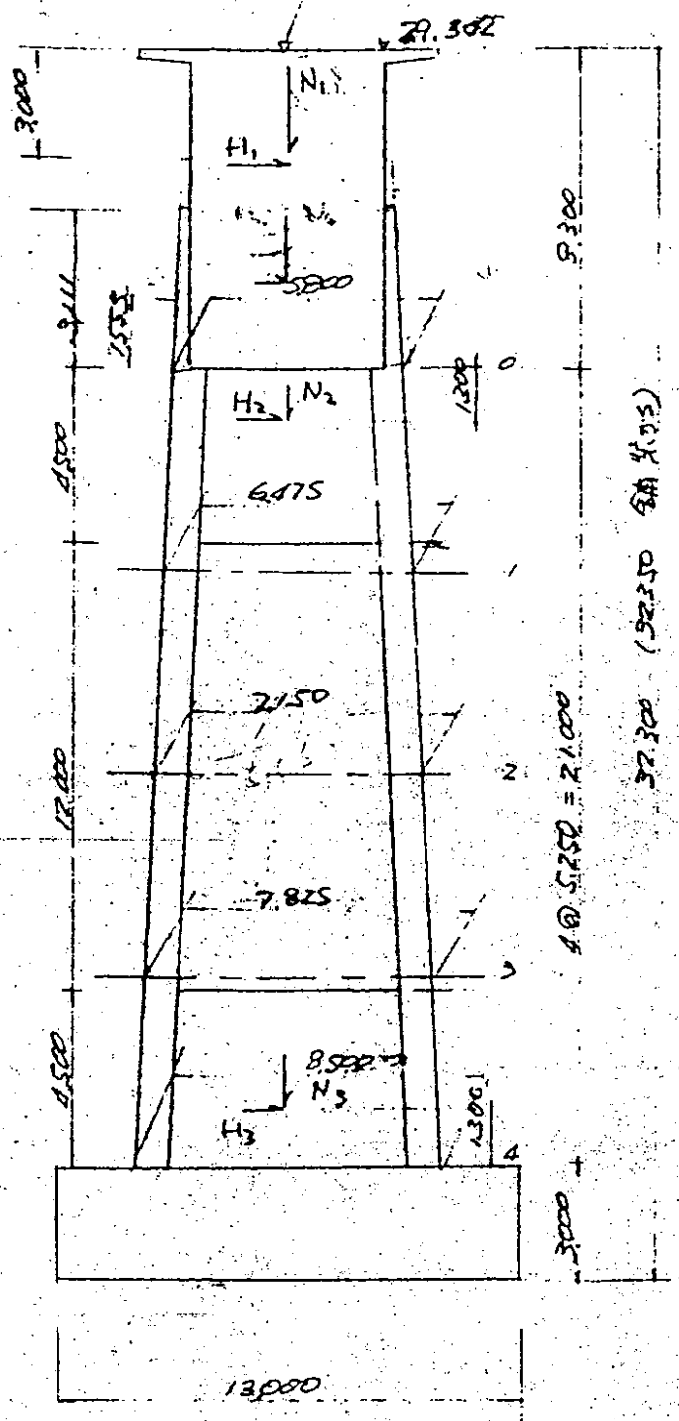
$S_E = 0.9$ t

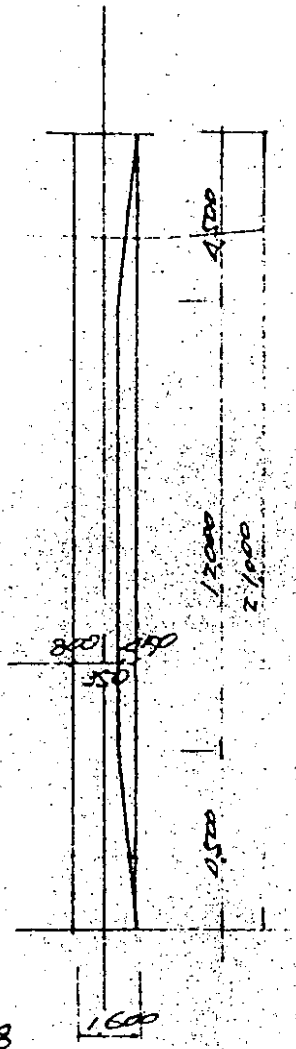
$$\tau = \frac{0.9}{1.4 \times 7/8 \times 0.85} = 0.98\ t/m^2 < 65\ t/m^2 \times 1.5$$

Ⅲ. 橋脚

51. 寸法

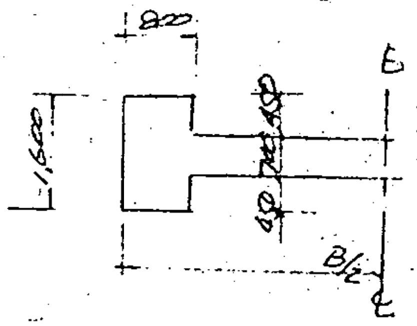
29.320 (鋼桁底面)





§2. 橋軸直角方向の計算

1) 脚柱各段断面断面力



断面積 $A = 1.6B - 0.9(B-1.6)$ （560t）

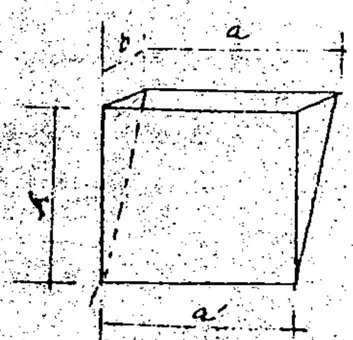
B	1.6B	0.9(B−1.6)	A	2.5W₀	S	2.5W₀…	S′	$M^{t\cdot m}$
0	5.800	9.280	3.780	5.500	37.13 (0)	5.57	(0)	0
					37.13		5.57	
1	6.475	10.360	4.388	5.972	78.39 (74.3)	11.76	(11.5)	29
					115.52		17.33	
2	7.150	11.440	4.995	6.445	85.59 (158.3)	12.84	(23.7)	120
					201.11		30.17	
3	7.825	12.520	5.6025	6.918	90.80 (246.5)	13.62	(37.0)	279
					291.91		43.79	
4	8.500	13.600	6.210	7.390	47.47	7.12		509
					339.38		50.91	

$2.5\ \frac{6c}{12} = 1.09375 \quad a z = 5.250$

S：（　）内の数値は各断面の剪断力

S′：（　）　〃　のせん断力

H_2 : N_2 について



$a = 5.8 - 1.6 = 4.200$

$a' = \frac{8.5-5.8}{21.0} \times 4.5 + 5.8 - 1.6 = 4.779$

$h = 4.500 \qquad b = 0.450$

$N_2 = \frac{4.2+4.779}{2} \times 0.45 \times 4.5 \times \frac{1}{2} \times 2.5 \times 2 = 22.7^t$

$H_2 = 22.7 \times 0.15 = 3.4^t$ $\frac{h}{2}$ を固化とする。

H_3 N_3 について

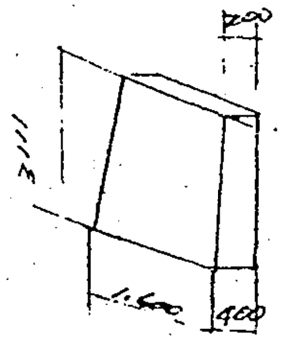
$a = 8.5 - 1.6 = 6.900$

$a' = \frac{9.5-5.8}{21.0} \times (21.0-4.5) + 5.8 - 1.6 = 6.321$

$N_3 = \frac{6.9+6.321}{2} \times 0.45 \times 4.5 \times \frac{1}{2} \times 2.5 \times 2 = 33.5$

$H_3 = 33.5 \times 0.15 = 5.0^t$

H4・N4について



$N_4 = \frac{(0.2+0.4)\times 1.6}{2} \times 13.14 \times 2.5 \times 2 \approx 7.5^t$

$H_4 = 7.5 \times 0.15 = 1.1^t$

各点曲げモーメント

473.6

$M_{H1} = 443.4\,(8.300 - 3.000 + 0Z \cdot H)$

$M_{H2} = 3.4\,(0Z \cdot H - 1.7)$ 　124頁より

$M_{H3} = 1.3 \times 5.0 = 6.5$ 　4.80m

$M_{H4} = 1.1\,(3.555 + 0Z \cdot H)$

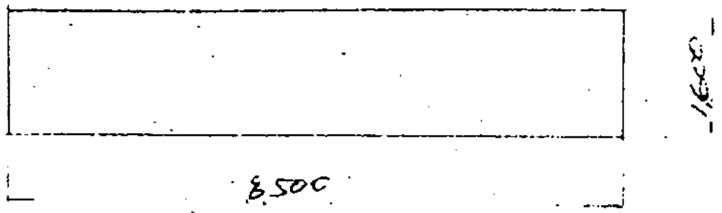
	脚柱	M_{H1}	M_{H2}	M_{H3}	M_{H4}	ΣM t-m
0	0	2510 2350	0	0	2	2512 2352
1	29	5000 4678	13	0	7	5049 4727
2	120	7480 7006	31	0	13	7644 7170
3	279	9960 9334	48	0	19	10,306 9680
4	509	12,450 11661	66	7	25	13,057 12268

各点軸力

	脚柱	N_1	N_2	N_3	N_4	ΣN	1.1	0.9
0	0	3158 2956	0	0	8	3166 2964	3480 3260	2850 2668
1	76	〃 2956	23	0	8	3265 3063	3590 3369	2940 2757
2	158	〃 2956	23	0	8	3347 3145	3680 3460	3010 2831
3	247	2956	23	0	8	3436 3234	3780 3557	3090 2911
4	339	〃 2956	23	34	8	3562 3360	3920 3696	3210 3034

(i) 各点曲げ応力度の検討.

Sect 4 点



$M = 12261^{t\cdot m}$ (13,057)　　$N = 3696^t$ or 3034^t (3720 or 3210)

kv = -0.1 N　　$e = \frac{12268}{3030} = 4.044$ (13,057 / 3210 = 4.068)　　$\frac{e}{h} = \frac{4000}{8500} = 0.476$ (0.479)

$\frac{d'}{h} = \frac{40}{850} = 0.047$

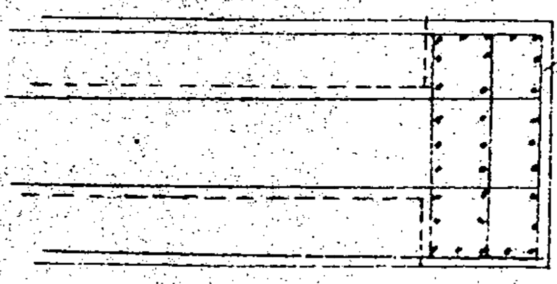
$p = 0.0015$ (0.0017)　$k = 0.43$ (0.46)　$C = 0.205$ (0.21)　$A_s = 204\ cm^2$

$f_c = \frac{N}{h \cdot b (C)} = \frac{3696000}{850 \times 160 \times 0.205} = 109$ (3925000; 0.21; 141; 136; 150 kg/cm²)

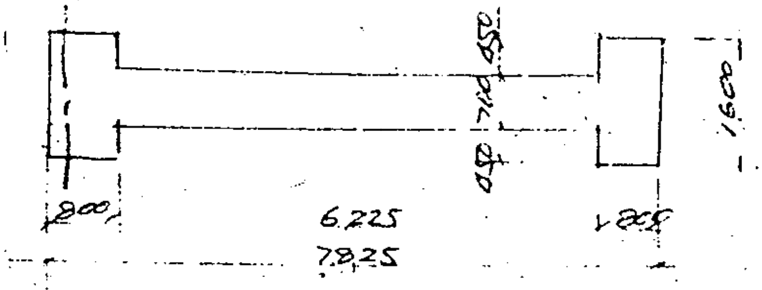
$f_s = n f_c \frac{1 - k - \frac{d'}{h}}{k} = 15 \times 109 \times \frac{1 - 0.43 - 0.05}{0.43}$ (141; 138)

$= 1277\ kg/cm^2 < 2700\ kg/cm^2$　o.k. (2560; 2400)

$A_s = 7.92 \times 26^本 = 206\ cm^2$　　D22 - 26本 (29; 250)



Sect 3 $M = 9660$ $N = 3557$ or 2411^t



$A_s = 206\ cm^2$

$b_0 r^3 + 3[(b-b_0)(a+t)^2 - ba^2 + 2n(A_s f + A_s' f')] r - 2(b-b_0)(a+t)^3 + 2ba^3$

$- 6n(A_s f^2 + A_s' f'^2) = 0$

$d' = 40$ $d = 782.5 - 40 = 742.5$ $e = \frac{9680}{2411} = 332.5$

$a = 332.5 - \frac{782.5}{2} + 40 = -18.75$

$f = h - d' + a = 782.5 - 40 - 18.75 = 723.75$

$f' = a + d' = 40 - 18.75 = 21.25$

$-18.75 - 40$

$70 r^3$

$3[(160-70)(-18.75+80)^2 - 160(-18.75)^2 + 2 \times 15 \times 206(723.75 + 21.25)] r$

337,6.. 56,250.0

$= 14,656.5 \times 10^2 \times r$

$-2(160-70)(-18.75+80)^3 = -41,360.9 \times 10^3$

$2 \times 160 \times (-18.75)^3 = -2109.4 \times 10^3$

$-6 \times 15 \times 206(723.75^2 + 21.25^2) = -9719,884.7 \times 10^3$

$-9763,355.0 \times 10^3$

$r = 388$ $x = 388 + 18.75 = 406.8$

$$\sigma_c = \frac{N \cdot x}{\frac{1}{2}[bx^2 - (b-b_0)(x-t)^2] + nA_s(x-d'+x-d)}$$

$$= \frac{3070 \quad 2911000 \times 406.8}{\frac{1}{2}[160 \times 406.8^2 - (160-20)(406.8-80)^2] + 15 \times 206(406.8 \times 2 - 60 - 742.5)}$$

$$= \frac{\text{圧縮} \quad 1182194.8 \times 10^3}{8432.0 \times 10^3 + 961 \times 10^3} \quad \text{引張} \quad 1390 \text{ kg/cm}^2$$

$$\sigma_s = 289 \times 15 \times \frac{742.5 - 406.8}{406.8} \quad (1720 \text{ kg/cm}^2)$$

$\sigma_{sa} = 2400$

ΔV = 0.1 の時

$e = \frac{9680}{3557} = 272$　　$d' = 40$　　$d = 742.5$

$a = 272 - 391.3 + 40 = -79.3$

$f = 782.5 - 40 - 79.3 = 663.2$

$f' = -79.3 + 40 = -39.3$

70 t².

$3[(160-70)(-79.3+80)^2 - 160(-79.3)^2 + 2\times15\times206(663.2-39.3)]h$

$8.548.7\times10^3$　　$/\alpha\ 6.153$

$-2(160-70)(-79.3+80)^3 = -61$

$2\times160\times(-79.3)^3 = -159,577$

$-6\times15\times206(663.2^2+39.3^2) = -8183.162$

$8342,939\ r\cdot0^3$

$r = 411$　　$x = 411 + 79.3 = 490.3$

$\sigma_c = \frac{3557000\times490.3}{\frac{1}{2}[160\times490.3^2 - (160-70)(490.3-80)^2] + 15\times206(460.3\times2-40-742.5)}$

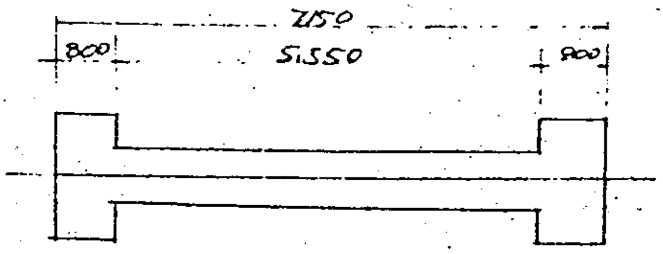
$15171/\alpha$

$= \frac{1743.997\times10^3}{26,807.1\times10^3 + 612\times10^3} = 64\ kg/cm^2$

σ_s は省略する.

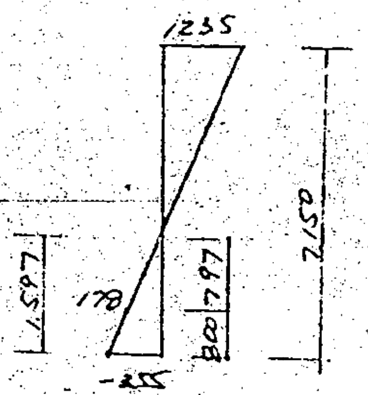
Sect 2

$M = 7.170^{t\cdot m}$ $N = 3060^{t}_{ST}$ 2831



$Z = \frac{1}{6}\left(7.15^2 \times 1.6 - 5.55^2 \times 0.9\right) = 9.014$

$\sigma_c = \frac{2831}{6.445} \pm \frac{7170}{9.014} = 440 \pm 795 = \frac{1235}{355}$



$P = \frac{355+178}{2} \times 1.6 \times 0.8 + \frac{178}{2} \times 0.7 \times 0.797$

$= 341 + 49 = 390^{t}$

$A_s = \frac{390 \times 10^3}{2700} = 146^{cm^2}$

ここで鉄筋量を 125^{cm^2} として応力度の検討をする

$k_v = -0.1$ $d' = 40$ $d = 7150 - 40 = 675$ $e = \frac{7170}{2831} = 253$

$a = 253 - \frac{715}{2} + 40 = -65$ (358)

$f = h - d' + a = 715 - 40 - 65 = 610$

$f' = 40 - 65 = -25$

$70x^2$

$3[(160-70)(-65+80)^2-160(-65)^2+2\times15\times125(610-25)]x=4614\times10^3$

20250　676000

$-2(160-70)(-65+80)^3 = -607.5\times10^3$

$2\times160(-65)^3 = -87880\times10^3$

$-6\times15\times125(610-25)^2 = -4193156\times10^3$

-4281643.5

$x=339$　　$x=339+65=404$

$$\sigma_c=\frac{2871\times10^3\times404}{\frac{1}{2}\left[160\times404^2-(160-70)(404-80)^2\right]+15\times125(2\times404-40-675)}$$

26115　8334

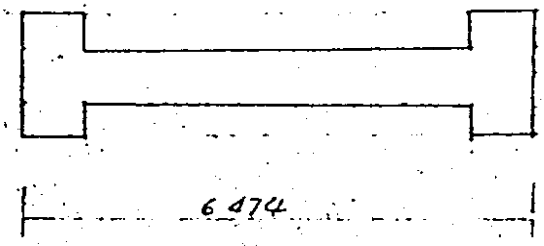
$$=\frac{1160.724\times10^3}{8500\times10^3}=134.4\ \text{kg/cm}^2$$

$$\sigma_s=134.4\times15\times\frac{675-404}{404}=1352\ \text{kg/cm}^2<2700\ \text{kg/cm}^2$$

o.k.

故に $A_s=125\ cm^2$ とする

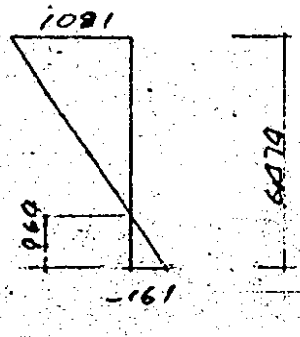
Sect 1　　M=4727　　N=2757



A = 5.972.

$$z = \frac{1}{6}(6.474^2 \times 1.6 - 4.874^2 \times 0.9) = 7.613$$

$$\sigma_c = \frac{2757}{5.972} \pm \frac{4727}{7.613} = 460 \pm 621 \quad \begin{matrix} 1081 \\ -161 \end{matrix}$$ kg/cm²



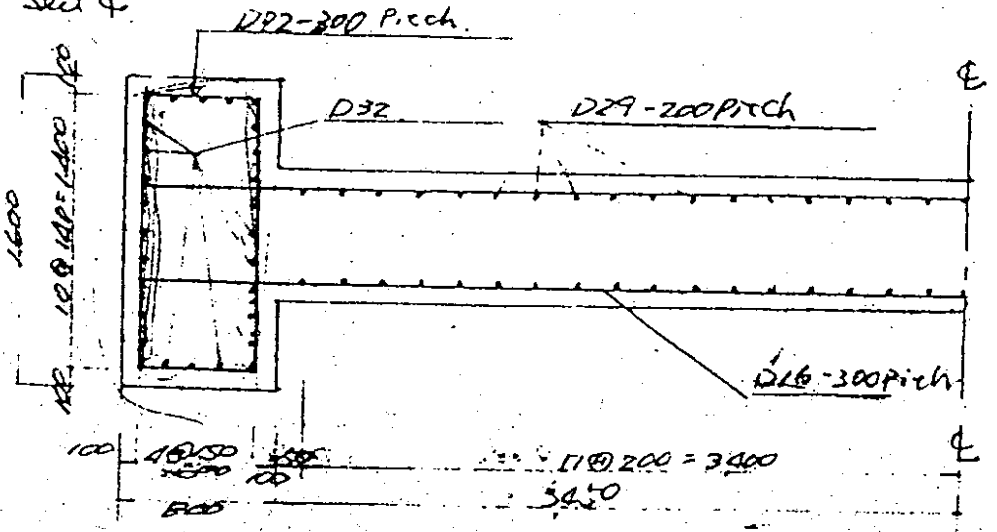
$$P = \frac{161}{2} \times 10.64 \times 10.8 = 62\text{T}$$

$$A_s = \frac{62000}{2700} = 22.9\ \text{cm}^2$$

sect 0 は省略する。

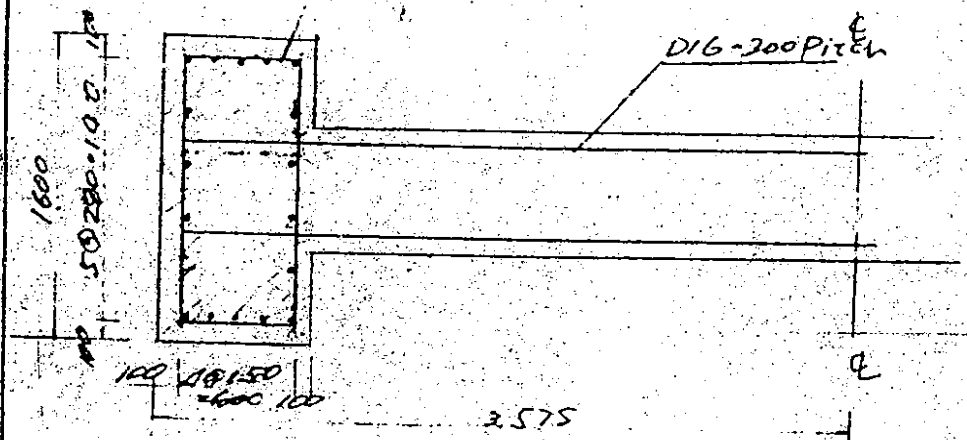
iii). 設計断面 鉄筋配置.

Sect 4



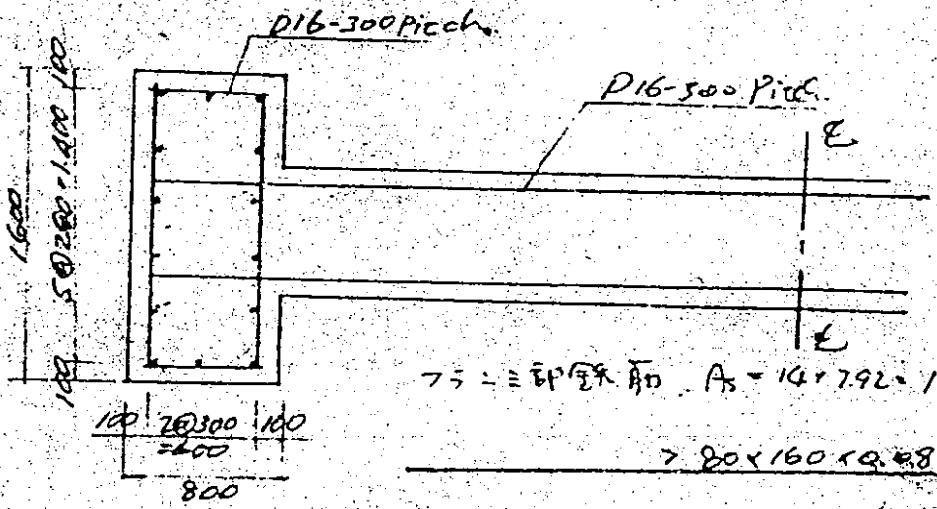
フランジ部鉄筋量 $A_s = 221.76\ cm^2 > 206$

Sect 2



フランジ部鉄筋. $A_s = 102.56 > 250$

Sect 1

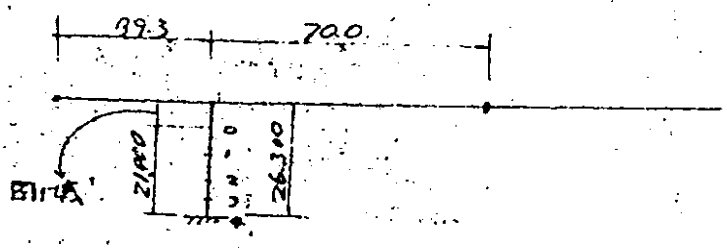


フランジ部鉄筋 $A_s = 14 \times 7.92 = 110.88$

$> 80 \times 160 \times 0.8 = 1024\ cm^2$ OK.

§3. 橋軸方向の検討

1) 橋脚にかかる曲げモーメント、軸力の計算



1) 張出HLP断面性能　$\frac{1.6 \times 1.6^3}{12} = 0.5459$

Set 4　$(8.5-1.6) \times \frac{0.7^3}{12} + 0.5459 = 0.7431$　1.502

Set 3　$(7.825-1.6) \times \frac{0.7^3}{12} + 0.5459 = 0.7238$　1.382

Set 2　$(7.150-1.6) \times \frac{0.7^3}{12} + 0.5459 = 0.7045$　1.419

Set 1　$(7.475-1.6) \times \frac{0.7^3}{12} + 0.5459 = 0.6852$　1.460

Set 0　$(5.8-1.6) \times \frac{0.7^3}{12} + 0.5459 = 0.6659$　1.502

2) 先端に 1t を作用した時のタワミ

	ox.	M	1/I	M/I	W	S	E·δ
0'	5.300	0	0	0	7.25		8,817.8
						493.7	
0	5.250	5.300	1.502	7.961	41.2		6201.5
						452.5	
1	5.250	10.550	1.382	14.580	77.1		3826.0
						375.4	
2	5.250	15.800	1.419	22.420	117.9		1855.3
						257.5	
3	5.250	21.050	1.460	30.733	161.6		503.5
						95.9	
4	5.250	26.300	1.502	39.503	95.9		0

$\frac{\Delta x}{12} = \frac{5.25}{12}$　　ΣS = 500.95

3) 先端に 1t-m を作用した時のタワミ

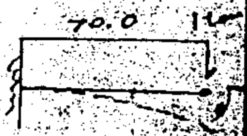
	M/I	W	S	E·δ
0'	0	1.669		497.115
			33.523	
0	1.502	7.176		319.445
			26.347	
1	1.382	7.324		181.123
			19.023	
2	1.419	7.452		81.255
			11.571	
3	1.460	7.666		20.505
			3.906	
4	1.502	3.906		0

ΣS = 35.193

4) ギンリに1tを作用した時のたわみ.

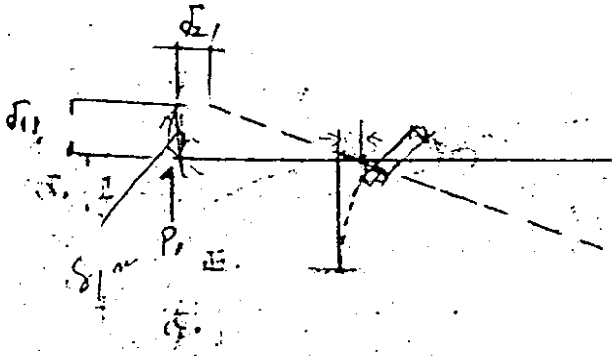
	M	y/I	M/I	W	S	Eδ
0	0	0.2608	0	3.359		3,515.51
					114.338	
1	8.75	0.2296	2.009	16.721		2,515.06
					97.617	
2	17.5	0.1624	2.842	24.077		1,660.91
					73.540	
3	26.25	0.0987	2.591	22.469		1,017.43
					51.071	
4	35.00	0.0589	2.062	18.051		570.56
					33.020	
5	43.75	0.0353	1.544	13.551		281.64
					19.469	
6	52.50	0.0206	1.082	9.596		111.28
					9.870	
7	61.25	0.0130	0.796	7.022		24.92
					2.848	
8	70.00	0.0084	A588	2.848		0

$\frac{\Delta x}{12} = \frac{8.75}{12} = 0.72916\cdots$



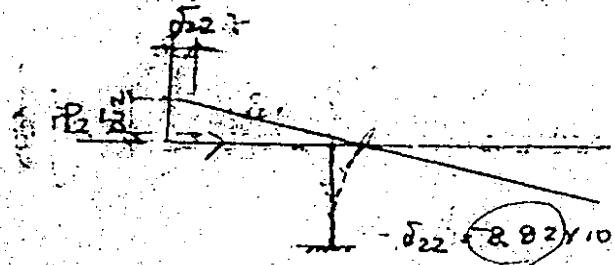
	M	y/I	M/I	W	S	Eδ
9	39.3	0.0084	0.326	1.100		0
					1.100	
10	32.75	0.0108	0.354	2.315		7.21
					3.415	
11	26.20	0.0143	0.375	2.450		29.57
					5.865	
12	19.65	0.0196	0.385	2.508		67.99
					8.373	
13	13.10	0.0282	0.369	2.378		122.83
					10.751	
14	6.55	0.0431	0.282	1.741		193.25
					12.492	
15	0	0.0722	0	-.391		275.07

$\frac{\Delta x}{12} = \frac{6.55}{12} = 0.545833\cdots$



$\delta_{11} = 275.07 + 39.3 \times 35.193 \times 39.3 = 50.63 \times 10^3$

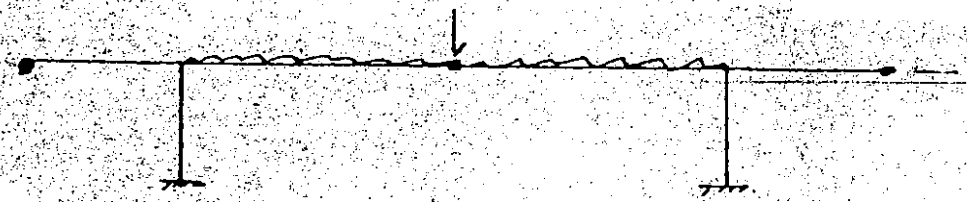
$\delta_{21} = +39.3 \times 497.115 = +19.54 \times 10^3$



$\delta_{22} = 8.82 \times 10^3$

$-\delta_{21} = 50.95 \times 39.3 = 19.69 \times 10^3$

活荷重作用時



求. 上図の載荷状態が脚柱に最大曲げモーメントを生じせしめる。

b) 静定系における脚柱に生ずる曲げモーメントMは

$P = 1.451 \qquad P = 24.2$

$$M = \frac{1.451 \times 70^2}{2} + \frac{24.2 \times 70}{2} = 4412.45 \text{ t·m}$$

$\delta_{2m} = 4.412 \times 497.115 = 2,193.50 \times 10^3$

$\delta_{1m} = 4.412 \times 35.193 \times 39.3 = 6.102.79 \times 10^3$

$$\delta_{11} X_1 + \delta_{12} X_2 = \delta_{1m}$$

$$\delta_{12} X_1 + \delta_{22} X_2 = \delta_{2m}$$

$$X_1 = \frac{\begin{vmatrix} \delta_{1m} & \delta_{12} \\ \delta_{2m} & \delta_{22} \end{vmatrix}}{\begin{vmatrix} \delta_{11} & \delta_{12} \\ \delta_{12} & \delta_{22} \end{vmatrix}} = \frac{\begin{vmatrix} +6102.79\times10^{3} & 19.6\times10^{3} \\ 2.193.50\times10^{3} & +8.81\times10^{3} \end{vmatrix}}{\begin{vmatrix} 58.63\times10^{3} & 19.6\times10^{3} \\ 19.6\times10^{3} & +8.81\times10^{3} \end{vmatrix}}$$

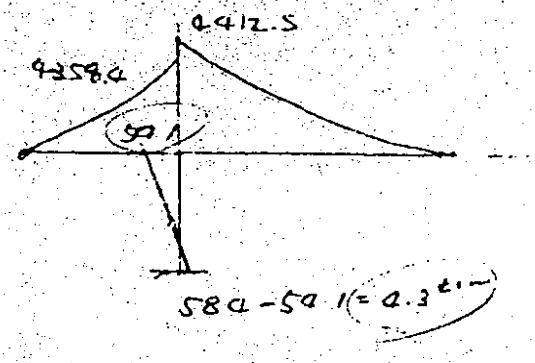
$$= \frac{10.820.6}{97.56} = 110.9$$

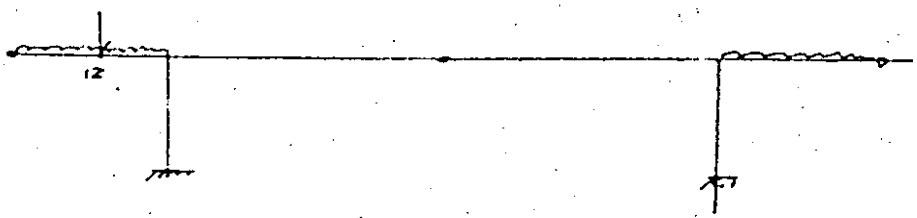
$$X_2 = \frac{\begin{vmatrix} 58.63 & +6.102.79 \\ 19.6 & 2.193.50 \end{vmatrix}}{97.56} = 2.22$$

6) 不静定反力によって脚柱に生ずる曲げモーメント

$M_1 = 110.9 \times 39.3 = 4358.4$

$M_2 = 22.2 \times 26.3 = 58.4$





※ 上図の状態が脚柱に最も悪い影響を生じせしめる。

P = 26.8　　n = 1.604

	$\frac{qx^2}{2}$	Pax	M	1/I	M/I	W	S	Eδ
9	1234	527	1766	0.0084	14.8	46.5		
							46.5	
10	860	351	1211	0.0108	13.1	86.0		
							132.5	
11	551	176	827	0.0143	11.8	74.9		
							207.4	
12	310	0	310	0.0196	6.1	61.9		
							269.3	
13	137.6		138	0.0282	3.9	75.4		
							274.7	
14	34.4		34	0.0431	1.5	10.3		
							285.0	
15	0		0	0.0722	0	1.4	(1195.4)	

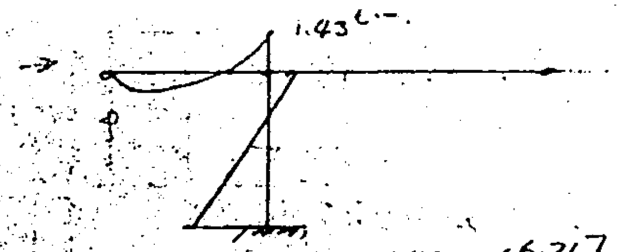
$\frac{\Delta x}{12} = \frac{6.55}{12} = 0.545833\cdots$

$-E\delta_{1m} = 1195.4 \times 6.55 + 1766 \times 35.173 \times 39.3 = 2,450.36 \times 10^3$

$-E\delta_{2m} = 1766 \times 49.715 = 877.9 \times 10^3$

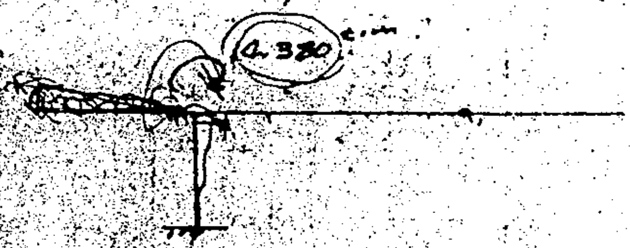
$$X_1 = \frac{\begin{vmatrix} -2450.36 & 19.6 \\ -877.9 & 8.81 \end{vmatrix}}{97.56} = -0.0\cdots$$

$$Y_2 = \frac{\begin{vmatrix} 54.63 & -2450.36 \\ 19.6 & -877.9 \end{vmatrix}}{97.56} = \frac{67.4}{97.56} = 0.69t$$



7) 自重及 静荷重による、

静定系 P による生ずる曲げモーメントの差 M = 3502 − 1116 + 30465 − 22472 = 14.380 t·m

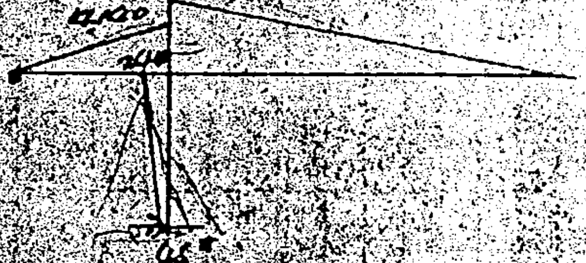


$\delta_{1m} = 14380 \times 35193 \times 39.3 = 19,889 \times 10^3$

$\delta_{2m} = 0.97115 \times 14,380 = 7.149 \times 10^3$

$$X_1 = \frac{\begin{vmatrix} 19.889 & 19.6 \\ 7.149 & 8.01 \end{vmatrix}}{97.56} = 357.8^t$$

$$X_2 = \frac{\begin{vmatrix} 57.63 & 19.889 \\ 19.6 & 7.149 \end{vmatrix}}{97.56} = 7.43^t$$



$-0.03 \times 26.5 + 240 = 65^t$

8) 温度変化、弾性塑性変形によって躯体柱に生ずる曲げモーメント

温度変化　$20 \times 39.3 \times 10^{-5} = 786 \times 10^{-5}$

乾燥収縮　$15 \times 10^{-5} \times 39.3 = 589.5 \times 10^{-5}$

クリープ　$600 \times \frac{40}{3250000} \times 2 = 1476.9 \times 10^{-5}$

2852.4×10^{-5}

ここに　$\Phi = 2.0$　$\sigma_c = 600\ ^{t}/_{m^2}$ とする。

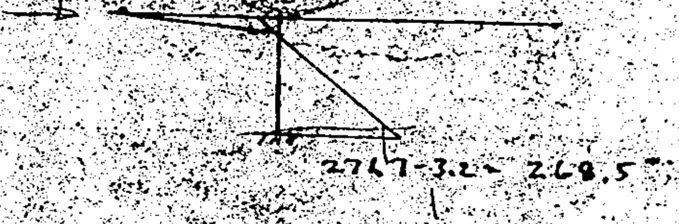
$\therefore E\delta = 2852.4 \times 10^{-5} \times 3250000 = 92.702$

$\delta_{m2} = 92.703$ ✓

$\delta_{m1} = 10.51 K 500.95 \times 39.3 = 206,913.89$ ✓

$$X_1 = \frac{\begin{vmatrix} 206.913 & 19.62 \\ 92.704 & 8.82 \end{vmatrix}}{97.56} = 0.0827$$

$$X_2 = \frac{\begin{vmatrix} 54.63 & 206.91 \\ 19.64 & 92.70 \end{vmatrix}}{97.56} = 12.337$$

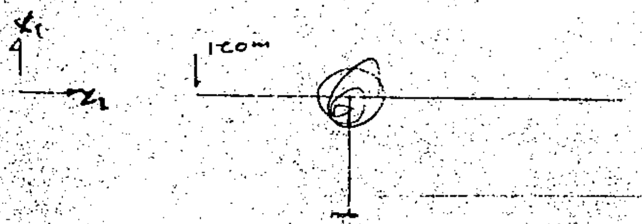


2767-3.2 = 268.5

1) プレストレスによる2次モーメント　$P=49^t$

	e	η	M	1/I	M/I	W	S	E·δ
9	4.322	136	28 802	0.0084	242	740		0
							740	
10	3933	116	22 355	0.0108	241	1682		4 847
							2422	
11	3533	92	29 903	0.0143	428	2582		20.711
							5004	
12	3105	70	10.650	0.0196	209	1471		53 487
							6475	
13	2665	48	6 264	0.0282	177	1148		95.899
							7623	
14	2.253	26	2870	0.0431	124	773		145.829
							8396	
15	0.	0.	0	—	0	135		200,810

$\Delta x = 6\,550 \quad \frac{\Delta x}{12} = 0\,54583 \cdots$

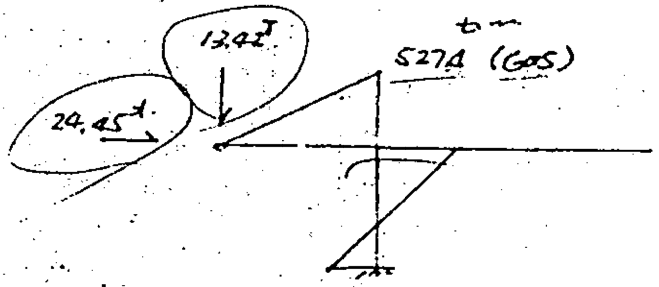


$E \cdot \delta_{11} = 54.63 \times 10^3 \quad E \cdot \delta_{21} = \delta_{12} = 19.61 \times 10^3$

$E \cdot \delta_{1m} = 200.81 \times 10^3 \quad E \cdot \delta_{2m} = 600 \times 39.2 = 23.52 \times 10^3 \; (\sigma_c = 60^{kg}/_{cm^2} とする)$

$$X_1 = \frac{\begin{vmatrix} \delta_{1m} & \delta_{12} \\ \delta_{2m} & \delta_{22} \end{vmatrix}}{\begin{vmatrix} \delta_{11} & \delta_{12} \\ \delta_{12} & \delta_{22} \end{vmatrix}} = \frac{\begin{vmatrix} 200.81 & 19.64 \\ 23.52 & 8.82 \end{vmatrix}}{\begin{vmatrix} 54.63 & 19.6 \\ 19.6 & 8.82 \end{vmatrix}} = \frac{1,309.2}{97.56} = 13.42$$

$$X_2 = \frac{\begin{vmatrix} 54.63 & 200.81 \\ 19.6 & 23.52 \end{vmatrix}}{97.56} = \frac{2651}{97.56} = 24.5^t$$



( )内の値はプレストレス導入直後の値

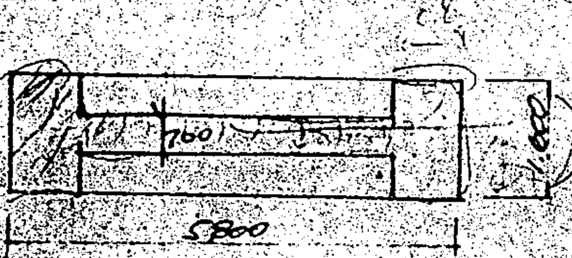
24.45 12.63 − 527.4 = 116 t·m (133)

曲げモーメント総括

	Ml Max	Ml Min	Md	Mu	Mp	Mpe	導入直後 Max	導入直後 Min	Max	Min
0	54	−1	240	3	−605	−527	−317	−366	−230	−285
1										
2										
3										
4	−4	17	95	−269	133	116	124	195	−112	−97

iii) 応力検討

Sect. 0



安全のため Sect 0 は箱形断面であるが I形断面で検討する

$A = 5.5\ m^2$

$Z = \frac{1}{6}(1.6 \times 1.6^2 + 4.2 \times 0.7^2) = 1.03$ [illegible]

$M = 366$

$N = 2964 + 26.8 \times \frac{1}{2} + 1.604 \times \frac{39.12}{2}$

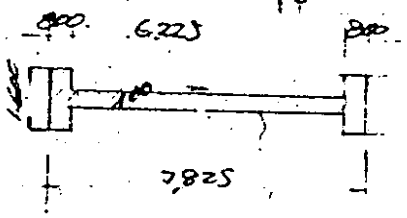
$= 3009$

$\sigma = \frac{3009}{5.5} \pm \frac{366}{1.03} = 547 \pm 355 = 902\ t/m^2 = 90\ kg/cm^2$ [illegible]

その他の断面は省略する

地震時 橋軸方向の検討 (架設時脚柱が立ち上がる時)

Sect 3. M=346 N=288.



$A_1 = 1.6 \times 1.6 = 2.56$

$A_2 = 6.225 \times 0.7 = 4.358$

$I_1 = 0.546$ $Z_1 = 0.683$

$I_2 = 0.179$ $Z_2 = 0.508$

軸力Nは面積に比例するから $N_1 = \frac{2.56}{2.56+4.358} \times 288 = 107^t$

$N_2 = \frac{4.358}{6.918} \times 288 = 181^t$

曲げMは Zに比例するから $M_1 = \frac{0.683}{0.683+0.508} \times 346 = 198$

$M_2 = 346 - 198 = 148^{t\cdot m}$

$M_1 = 198$ $N_1 = 107$ $e = \frac{198}{107} = 1.85$

$A_s = 3.88 \times 10 = 38.8$ $p = \frac{38.8}{160 \times 160} = 0.00152$

$\frac{d'}{h} = \frac{15}{160} = 0.094$ $\frac{e}{h} = \frac{185}{1.6} = 1.156$

$k = 0.24$ $c = 0.07$

$\sigma_c = \frac{107000}{160 \times 160 \times (0.07)} = 59.7^{kg/cm^2}$

$\sigma_s = 59.7 \times 15 \times \frac{1 - 0.24 - 0.07}{0.24} = 2500^{kg/cm^2}$

$M_2 = 168^{t-m}$ $N_2 = 181^{t}$

$e = \frac{168}{181} = 0.819$

P22 - 3本 $A_s = 384 \times 3 = 120.3$

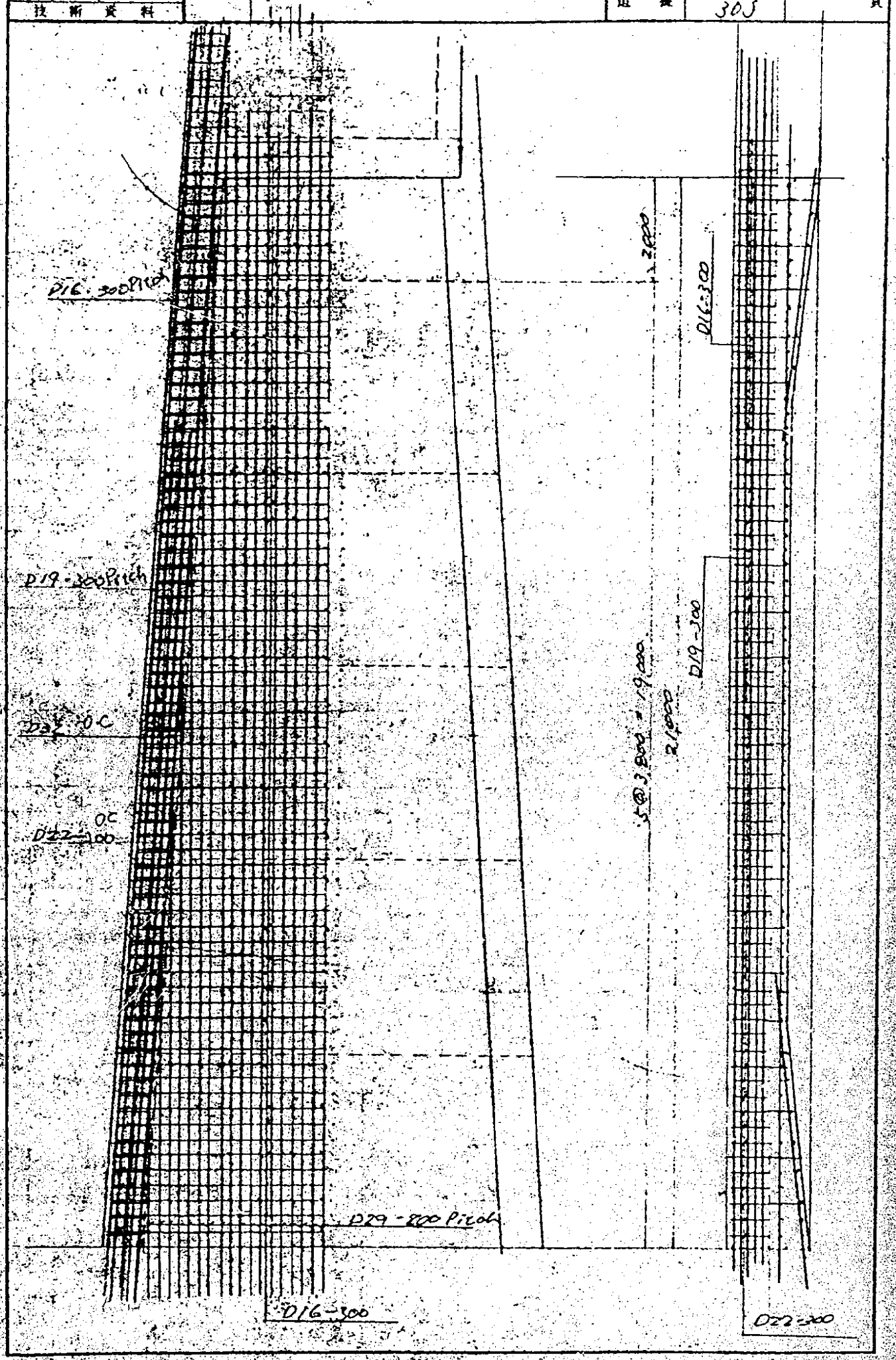
$P = \frac{120.3}{623 \times 70} = 0.0028$

$\frac{d'}{h} = \frac{15}{70} = 0.215$ $\frac{e}{h} = \frac{0.819}{0.7} = 1.169$

$\lambda = 0.275$ $C = 0.068$

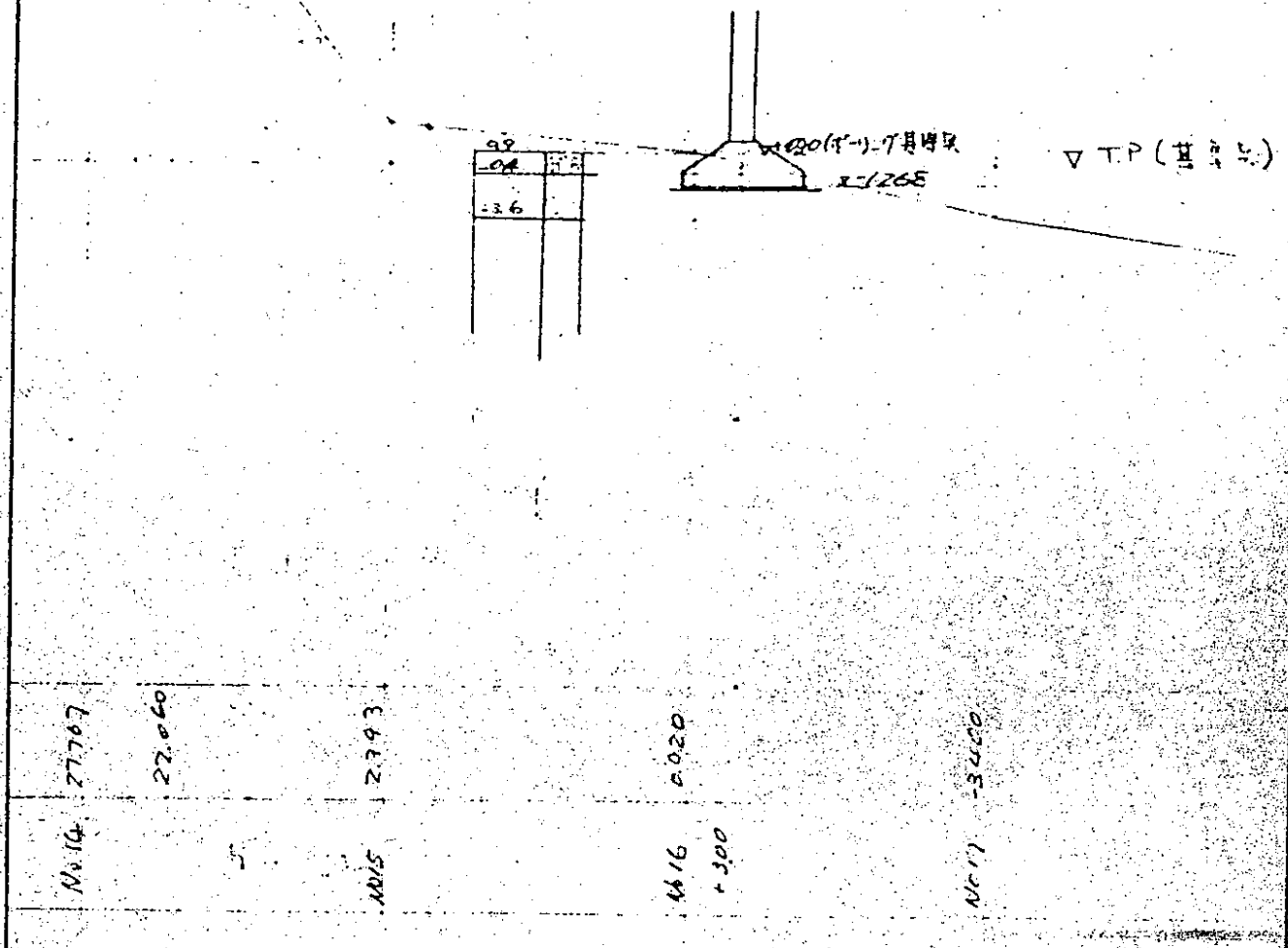
$\sigma_c = \frac{181000}{623 \times 70 \times (0.068)}$ kg/cm²

$\sigma_s = 61 \times 15 \times \frac{1 - 0.275 - 0.215}{0.275} = 1696$ kg/cm²


D16-300Pitch
D19-300Pitch
D22-300
D16-300
D29-300 Pitch
D16-300
D19-300
D22-300

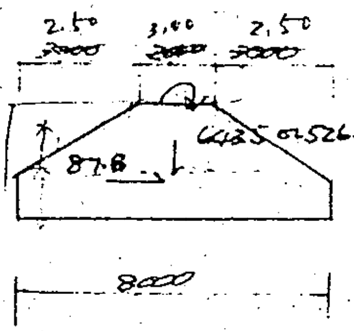
§3. フーチングの設計

1) フーチング断面図 (左岸側)



※. To右岸側は 左岸側と同じEL (-12.68) とする。

2). 荷重項.



$W_c = (80 \times 3.0 - 3.0 \times 2.0) \times 1.0 \times 2.5$

= ~~238~~ 247 × 2.5 = ~~585~~ 618

~~643.5~~ 680 (0.1)
~~526.5~~ (−0.1)
556

H_c ~~87.8~~ 92.7^t

図心.

2.5×2	×(1+0.666)	=	8.333
3×2	×(1+0.666)	=	9.996
3×2 / 2×2	× 2	=	~~8~~ 12
1×8	× 0.5	=	4
~~18~~ 19			~~21.996~~ 24.333

$a_1 = \frac{2.996}{1.8} = 1.222$ 1.281

3) 常時地盤反力.

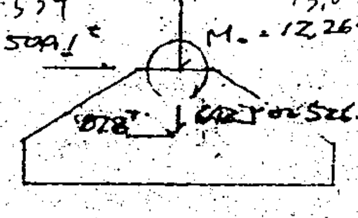
3562 585 + 334

N = ~~3360~~ + 585 = ~~3945~~^{t} 4481

$\sigma_c = \frac{\text{~~3945~~ } 4481}{8 \times 13.0} = 38.6$ 43.1 t/m² ok

$\frac{4687}{8 \times 13} = 45$ t/m²

4) 地震時の安定計算〔橋軸直角方向〕



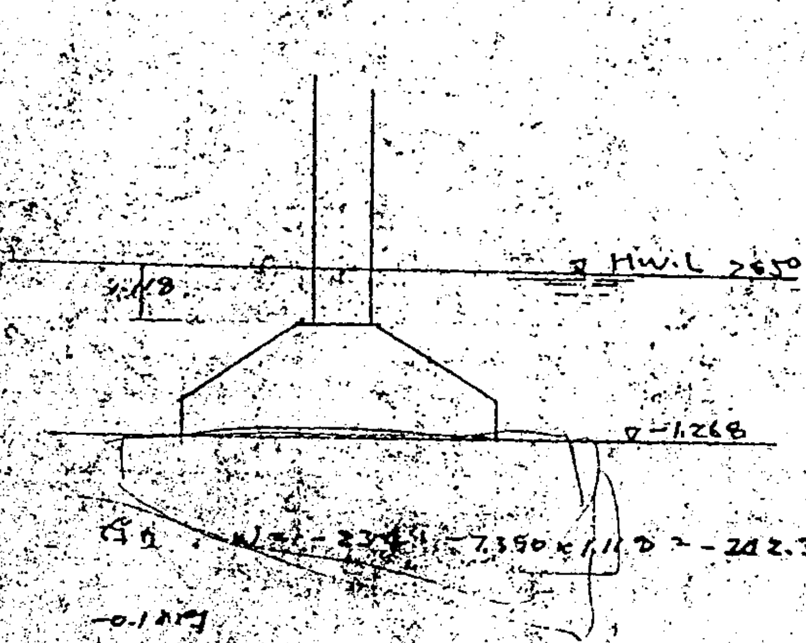
$H_0 = H_1 + H_2 + H_3 + H_0 + H_P$

= 1.1 + 1.3 + 3.4 + 443.4 + 50.9

= 500.1^t

上部 橋脚 フーチング

N = 3492 + 577 + 618 = 4687^t



浮力 $N = -334 - 2.350 \times 1.110 = -242.3$

i) 転倒　$N = 3034 + 524.5 - 242.3 = 3318.2$ (3210 / 3,494.2)

$M_R = 12.267 + 500 \times 3.0 + 878 \times 1.222 = 13,875.6$ (13,457 / 14,664)

$M_S = 3,318.2 \times 6.5 = 21,568$ (3,494.2 / 22,700)

$F_S = \frac{21568}{13876} = 1.5$ ok (22700 / 14664 = 1.55 ok)

ii) 地盤反力　$d = \frac{M_R - M_S}{N} = \frac{7692}{3318} = 2.318$ (8036 / 3494 = 2.30)

$3d = 6.95 = 6.9$

$\sigma = \frac{3318 \times 2}{2.318 \times 3 \times 8} = 119.2$ t/m² < 120 t/m²

126.5 << 120 t/m² out

iii) スベリ　$F_S = \frac{N \tan\theta}{\Sigma H} = \frac{3318.2 \times 0.5}{500 + 87.8} = 2.8 > 1.0$ (3494 / 534 / 621.8 / 2.81)

0.147 の地盤反力

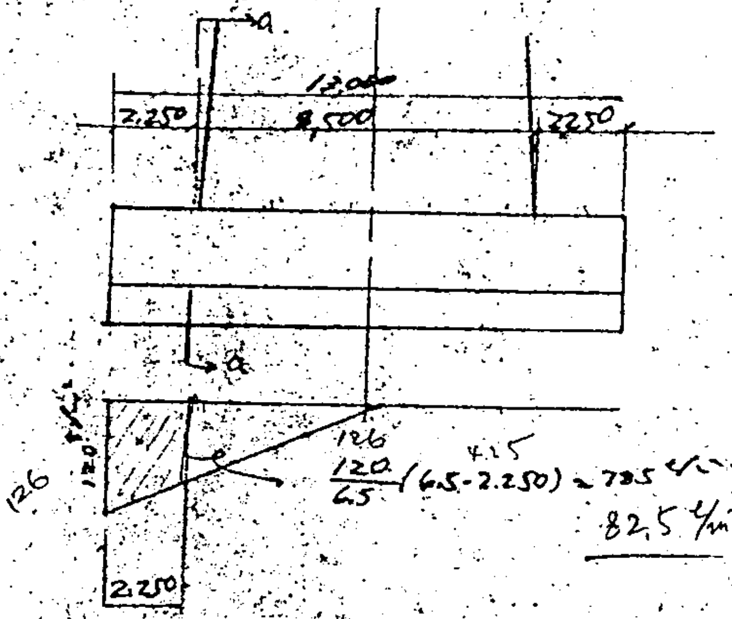
$N = 3696 + 643.5 = 4337.5$ (3920 / 4564)

$\sigma = \frac{2 \cdot N^2}{3(M_R - M_S)\,l} = \frac{2 \times 4339.5^2}{3(4337.5 \times 6.5 - 13,875.6) \times 8.0}$ (4564² / 4,564 / 14,664 / 14,986)

$= \frac{37662.5 \times 10^3}{343.9 \times 10^3} = 109.5$ t/m² < 120 t/m²　116 t/m² < ok

※ 橋軸方向については十分安全であるから省略する。

5) 橋台翼角部の底版検討



$\frac{120}{6.5}(6.5-2.250) = 78.5$ t/m

82.5 t/m²

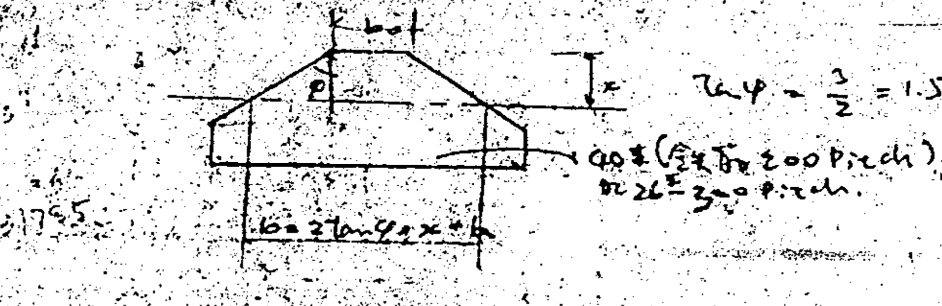
Sect a-a

$M_a = \frac{2.250}{6}(2\times 120 + 78.5)\times 2.250 = 283$ t-m/m

(2×126, 82.5) → 268 t-m/m

$283 \times 8 = 2260$ t-m

$M = 268 \times 8 = 2150$



$\tan\varphi = \frac{3}{2} = 1.5$

$\frac{b + b_0}{2} \cdot x \cdot \frac{x}{3} - n A_s (d - x) = 0$

$\frac{2\tan\varphi \cdot x + b_0 \times 2}{2} \cdot x \cdot \frac{x}{3} - n A_s (d - x) = 0$

上式より x を算出する

$A_s = 39 \times 3.88 = 151.3$ D22

$39 \times 5.07 = 197.7$ D25

349.0

$0.5x^3 + 66.7x^2 + 5235x - 149{,}975 = 0$

$x = 94$

$\sigma_s = \dfrac{215{,}000{,}000}{349\left(285 - \frac{94}{2}\right)}$ = 2720 kg/cm² < 2700 kg/cm² out

(226,00,000 written above the numerator; 238 below the bracket)

$\sigma_c = \dfrac{2589}{15} \times \dfrac{94}{(285-94)} = 85$ kg/cm² < 240/3 × 1.5 o.k.

せん断力の検討は有効高さが大きいので斜引張に注目する
設計断面が存在しない。よって押抜応力の検討をする

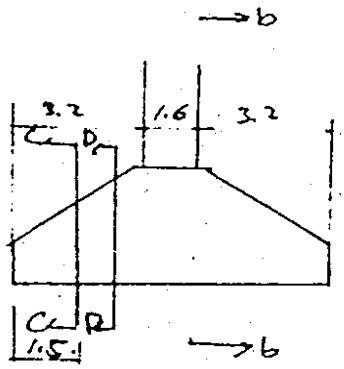
$A_\tau = 8.0 \times 13.0 = 104$ m² (フーチング)

$A_p = 7.39$ m² (柱面積)

$S_p = 3949\left(1 - \dfrac{7.39}{104}\right) = 3665$ t

$\tau_p = \dfrac{3665}{2(1.6 + [illegible]) \times 7.0} = 60$ t/m² = 6 kg/cm²

c) 橋軸方向応力検討



$\sigma = 38\ kg/cm^2$

Sect b-b

$M = \frac{3.2^2 \times 38}{2} = 195\ t\cdot m$

$A_s = \frac{19500000}{1800 \times \frac{7}{8} \times 286} = 43.5\ cm^2$

$A'_s = \frac{43.5}{5} = 8.7 \rightarrow D32 \quad 7.92\ cm^2$

$A_s \quad \frac{1.0}{0.15} \times 6.04 = 42.733 \quad D32 - 150\ Pitch$

$P = \frac{42.733}{285 \times 100} = 0.00150$

$\frac{1}{\ell_s} = 770 \qquad \frac{1}{\ell_c} = 1115$

2.4008

$\sigma_c = \frac{19500000}{100 \times 285^2} \times 11.2 = 27\ kg/cm^2$

$\sigma_s = 2.4008 \times 720 = 1729\ kg/cm^2 \quad o.k.$

Sect c-c.

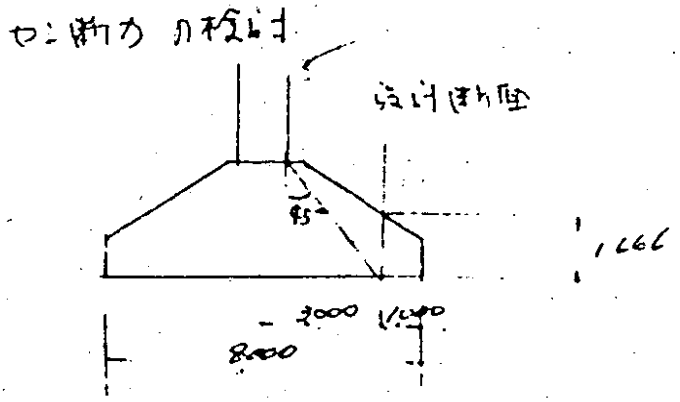
$M = \frac{1.5^2 \times 38}{2} = 42.8\ t\cdot m$

$A_s = \frac{4290000}{1800 \times \frac{7}{8} \times 175} = 16.2\ cm^2$

Sect D-D

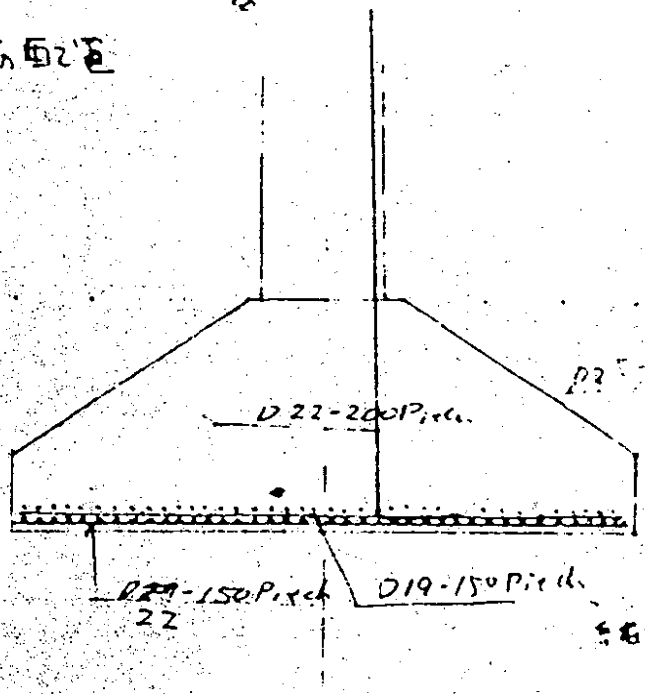
$M = \frac{2.25^2 \times 38}{2} = 96.17\ t\cdot m$

$A_s = \frac{96.17800}{1800 \times \frac{7}{8} \times 235} = 25.9$



$$S = 38 \times 1.0 = 38^{t}$$

$$\tau = \frac{38000}{100 \times \frac{7}{8} \times 152} = 2.9 \text{kg/cm}^2 < 7 \text{kg/cm}^2$$

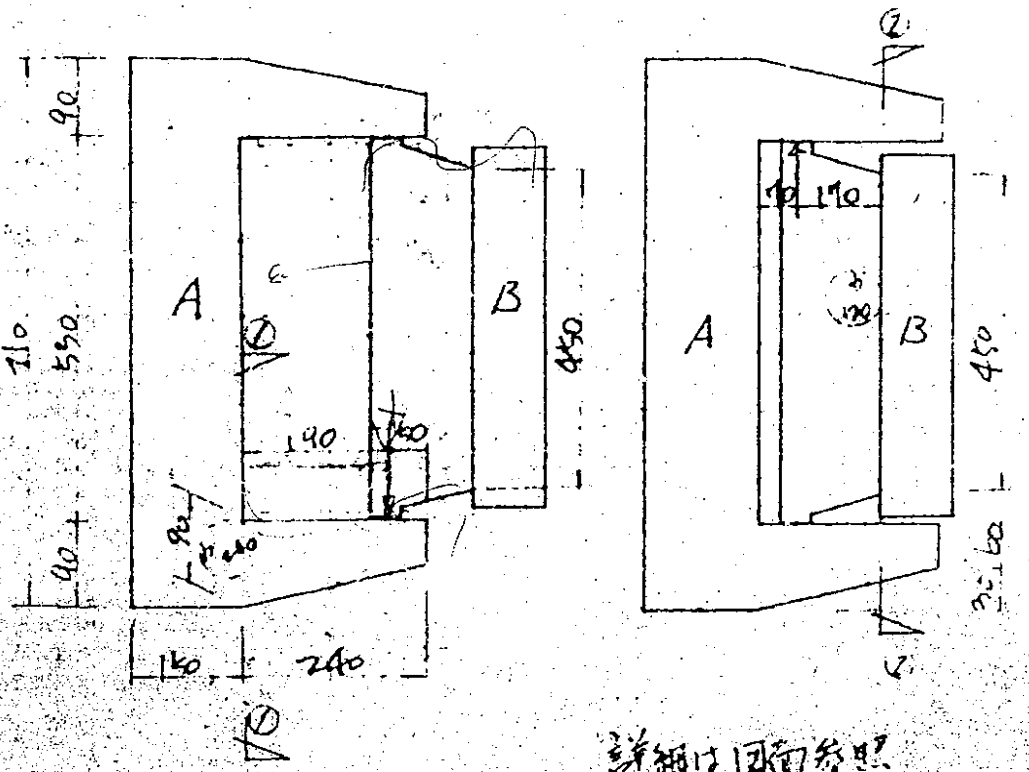


才 4 章

付帯構造の設計

I. 中央ヒンジ梁の計算

1) ヒンジ梁の寸法図



2) 各部応力の検討

i) 金物の A

断面 ①~①

$M = 19 \times \frac{18.5}{2} \times 10^3 \times \frac{18}{2} = 334,500$ kg·cm

$Z = \frac{1}{6} \times 24 \times 9^2 = 324$ cm³

$\sigma = \frac{334,500}{324} = 1032$ kg/cm² < 1,400 kg/cm² ok

$\tau = 1.5 \times \frac{19,500}{24 \times 9} = 135$ kg/cm² < 300 kg/cm² ok

2) ヒンジに作用するせん断力

活荷重によるせん断力 ± ~~32.5~~ 30.0 t

プリープにより生ずるせん断力 ± 5.0 t (仮定) } ~~35.0~~ 37.3

片ヒンジ当は 2 t 使用するから 1 t 当りの最大反力は

~~17.5~~ 18.7 t とする

3) 鋼材の支圧応力度

今 鋳鋼の許容応力を $P_0 = 6,000$ kg/cm² とする

$\frac{R}{\ell} \cdot \frac{~~17,500~~\ 18,700}{10} = ~~1,750~~\ 1,870$ kg/cm

鋼道路橋示方書より $\sigma = 45d$

$d = 47$ cm

$\sigma = 45 \times 47 = 2,115$ kg/cm $> ~~1,750~~\ 1,870$ kg/cm

4) ヒンジ部の変位量

(1) 温度変化による変位

$\delta_1 = \pm 15 \times 10^{-5} \times 140 = \pm 0.021$ m

(2) 乾燥収縮による変位

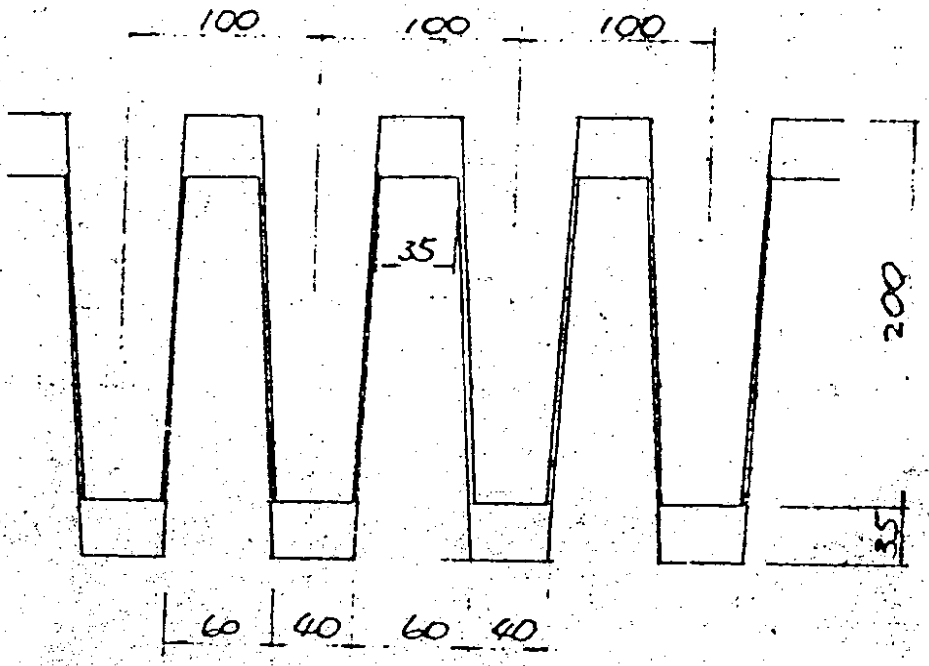
$\delta_2 = 15 \times 10^{-5} \times 140 = -0.021$ m

(3) プリープによる変位 (死荷重作用時平均応力 60 kg/cm² と仮定)

$\delta_3 = \frac{60}{350,000} \times 2 \times 140 = -0.048$ m

(4) 合計

$\delta_{max} = -0.090$ m $\quad \delta_{min} = -0.048$ m



材質は SS 41

許容曲げ応力度 $\sigma = 1,400$ kg/cm²

許容せん断応力度 $\sigma = 800$ kg/cm²

(iii) 荷重

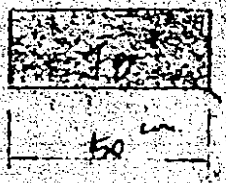
衝撃係数

$$i = \frac{20}{50+\ell} = \frac{20}{50+0.2} = 0.398$$

鋼道路橋示方書より 後輪荷重 5.6t を考慮する

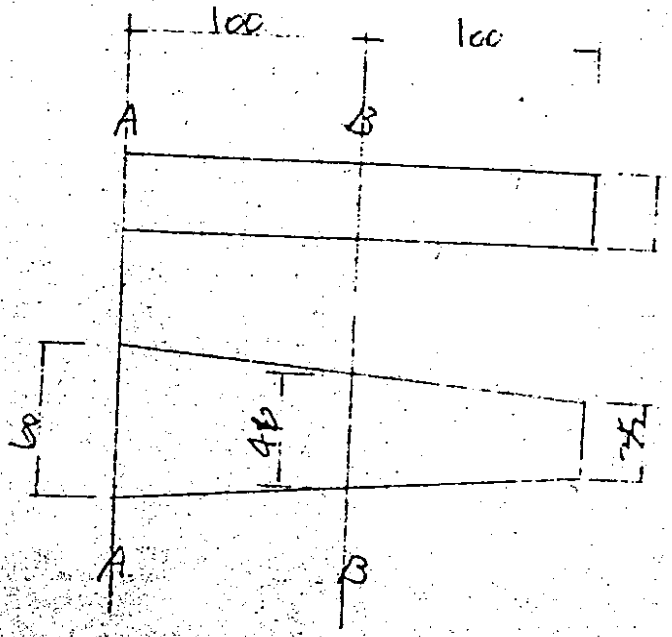
$$5.6(1+0.398) = 7.8\text{t}$$

輪帯巾



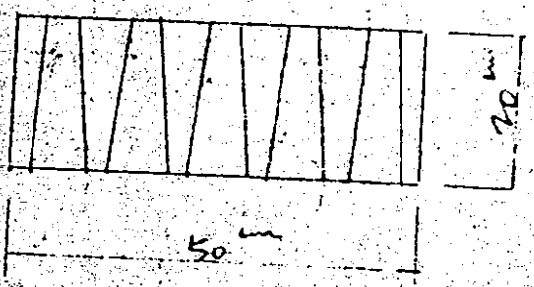
(V) 突出部の検討

次図の断面において検討する。



荷重の分担

輪荷重巾 50cm には下図の如く5本の突出部が含まれる。



突出部 1本当り $\frac{7.8}{5} = 1.56^t$

$$g = \frac{1560}{20} = 78\ kg/cm$$

a) 曲げ応力度の検討

Section A~A

$M = \frac{1}{2} \times 78 \times 20^2 = 15.600$ kg-m

$Z = \frac{1}{6} \times 6 \times 3.5^2 = 12.25$ cm³

$\sigma = \frac{15.600}{12.25} = 1273$ kg/cm² < 1.400 kg/cm² ok

Section B~B

$M = \frac{1}{2} \times 78 \times 10^2 = 3.900$ kg-m

$Z = \frac{1}{6} \times 48 \times 3.5^2 = 9.8$ cm³

$\sigma = \frac{3.900}{9.8} = 398$ kg/cm² < 1.400 kg/cm² ok

b) せん断応力度の検討

Section A~A

$Q = 1.560$ kg

$\tau = 1.5 \times \frac{1.560}{3.5 \times 6} = 110$ kg/cm² < 800 kg/cm² ok

Section B~B

$Q = 780$ kg

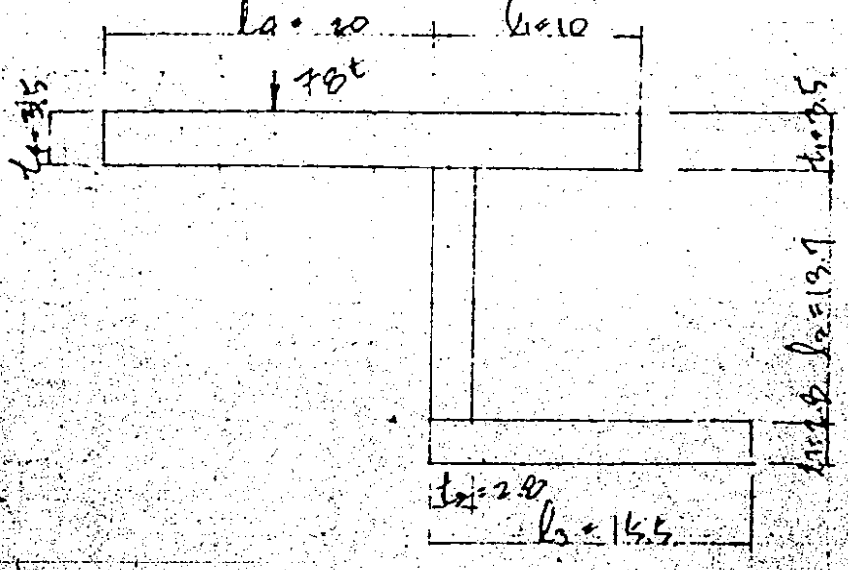
$\tau = 1.5 \times \frac{780}{48 \times 3.5} = 46$ kg/cm² < 800 kg/cm² ok

V) キャンバル桁の検討

拡幅桁には突出部に載荷された後輪荷重により ネジレモーメントが作用する。

その際のネジレせん断応力度はサンブナンの原理より

$$\tau = \frac{3Tt}{(2)l t^3}$$



図心位置 $\bar{x}$ を求める。

$$\bar{x} = \{ \frac{10^2}{2} \times 3.5^2 + 13.7 \times 2.8 \times (10 - \frac{2.8}{2}) + 15.5 \times 2.8 \times (10 - \frac{15.5}{2}) + 20 \times 3.5 \times 20 + 3.5 \times (20+10) + 2.8 \times (13.7 + 15.5) \}$$

$$= \frac{2624}{200} = 13.1 \text{ cm}$$

$$T = 1.8 \times 1000 \times (10 + \frac{20}{2} - 13.1) = 53820 \text{ kg-cm}$$

$$\tau = \frac{3 \times 53820 \times 3.5}{30 \times 3.5^3 + (13.7 + 15.5) \times 2.8^3} = 293 \text{ kg/cm}^2 < 400 \text{ kg/cm}^2$$

Ⅲ. 橋体取付鋼棒の計算

1. 引張り鋼棒に作用する力

桁自重	−305.1 t	339.1	
静荷重	−60.7	87.5	
活荷重	−112.1	140.6	
計	−477.9	→ −478 t	−567 t

2. 引張り鋼棒の本数

ST 95/120 φ33 mm ($A_p = 7.88\ cm^2$) を用い，破断に対して安全率を3とり，設計荷重に対して橋台と桁との相対変位を0ならしめるように鋼棒を緊張する．

鋼棒1本の破断荷重

$120\ kg/mm^2 \times 7.88\ cm^2 = 94.8$ t

鋼棒1本の緊張力

$$P = \frac{1}{3} \times 94.8 = 31.9\ t$$

所要本数は 567

$$N = \frac{478}{31.9} = 15\text{本} \longrightarrow 16\text{本}$$ 用113.

17.84 → 18本

鋼棒1本当り38 t で緊張すると

$$P = 38 \times 16 = 608\ t$$

18 : 685 t

桁自重作用時

$U = -305.1$ t -339.1 t

$T = 608 - 305.1 = 302.9$ t

685 − 339.1 = 345.9

静荷重作用時

$U = -60.7$ t -87.5 t

$T = 302.9 - 60.7 = 242.2$ t

345.9 − 87.5 = 258.4 t

クリープ、乾燥収縮により8%の緊張力が減少するとすれば

54.8t
608 × 0.08 = 48.7 t

鋼材のレラクセーションにより緊張力の減少を3%とすれば：

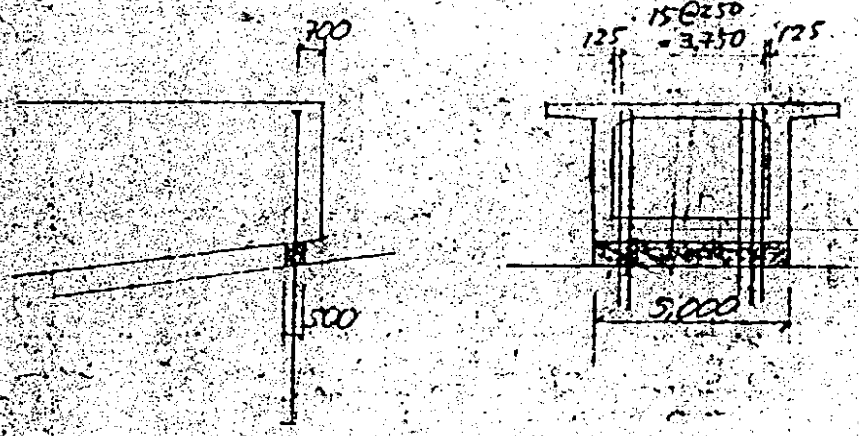
20.5t
608 × 0.03 = 18.3 t

T = 242.2 − 48.7 − 18.3 = 175.2 t
258 t − 54.8 − 20.5 − 183.1

設計荷重作用時

−114.6
U = −112.1

47.5 t > 0
T = 175.2 − 112.1 = 63.1 > 0



コンクリート受台の応力

受台に作用する力

側径間施工直後 R = 571.8 t

$$\sigma_c = \frac{R}{A} = \frac{571.8}{0.5 \times 5} = 229\ t/m^2 = 22.9\ kg/cm^2$$

この他に壁が建てると H = 571.8 × 0.15 = 85.8 t

$$\sigma_c = \frac{H}{A} = \frac{85.8}{2.5} = 34\ t/m^2 = 3.4\ kg/cm^2$$

設計荷重作用時（最大モーメント）

プレストレス　608^{t}

設計荷重　−307.4

300.6^{t}

$$\sigma_c = \frac{300.6}{2.5} = 120^{t/m^2} = 12^{kg/cm^2}$$