JIUN MASUDA
CONSULTING ENGINEER
~~FUJI~~ SHOWA BLDG, TOKYO

MADE BY _____ DATE _____ FILE NO _____
CHECKED BY _____ DATE _____ PAGE NO _____

CALCULATIONS FOR

昭和八年十二月

埼玉縣

南縣道浦和所謹線

木下橋設計書

（木下工事）

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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MADE BY J. M DATE 8-12-2 FILE NO ____
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CALCULATIONS FOR

Design of Akigase-Bashi for Saitama-Ken

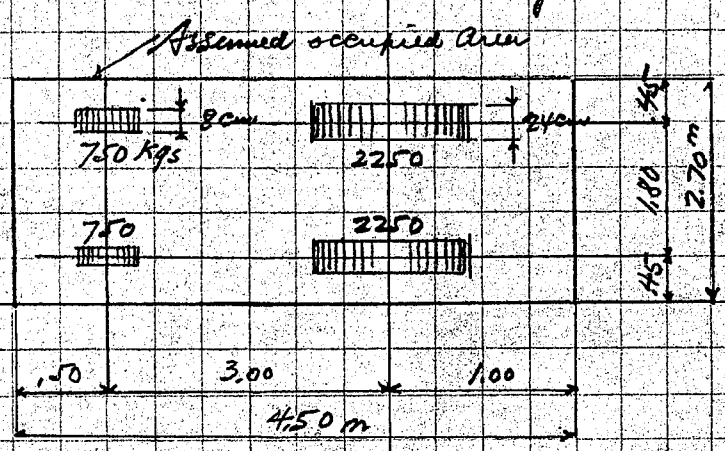
This bridge is to be built over Arakawa on highway line between Urawa and Fukaozawa, Saitama-Ken. The location of the bridge is at the place where the transverse dykes were built by Engineers' corps of Home affairs Department. The length between ends of dykes 780.0 metres more or less, and the length of bridge 755.0 metres about. The bridge is divided into two parts of permanent and temporary structure, the former is to be built over the proposed main channel and the latter over flooded district. The permanent structure is composed of truss spans over stream and plate girder spans for the other parts, the total length of such structure is 748.55 metres and the wooden beam structure over wooden piles for temporary structure which length is 207.0 metre long. Clear roadway of bridge is 5.5 metres wide paved with 5 cm granolithic on reinforced concrete slab for permanent structure and earth covering for temporary structure. Handrails for permanent structure will be pipes and structural steel design, and wooden Handrails for timber structure.

Assumed loading

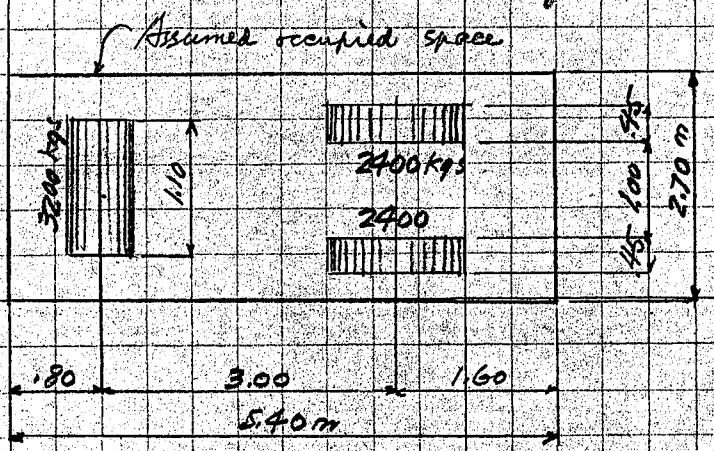
Uniform load $w = \frac{100.000}{170+l} \leqq 500$ kgs/m²

where w = Uniform load in kgs per sq. metres
l = Span length in metre

6 ton motor truck loading



8 ton roadroller loading



2 Rows of motor traffic lines on roadway with occupied width of 2.70 metres each; unoccupied space around motor truck and road roller shall be filled with uniform load specified above. One road roller on one span assumed.

Impact for motor truck loading

Coef. $\frac{20}{60+l}$ where l = Loaded length in metre
max impact 30%

No impact for road roller and uniform live load.

Allowable working strength

Reinforcing bar 1200 kgs/cm² for tension
900 " " shear

Concrete 1:2:4 mixture

Direct compression	35	kgs/cm²
Fibre stress due to bending	45	"
Combined stress direct and bending (Compression member)	35	"
Punching shear of Concrete	9	"
Shear of plain concrete	4	"
Bearing	45	"
Bond stress	6	"

増田淳氏関係資料
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CALCULATIONS FOR

Design of Akigase Bashi for Saitama-Ken

Concrete 1 : 2.5 : 5 mixture

Direct compression	31.5	kg/cm²
Fibre stress due to bending	40.5	"
Combined stress direct and bending	31.5	"
Bearing	40.5	"

Note: Bond stress due to bending moment under 22 mm in diameter of bar shall not be counted.

For combined wind or temperature stress with dead live and impact stresses, the allowable working strength shall be increased 25%. In case of earthquake, increase unit stress by 50%. Seismic acceleration assumed 2500 mm/sec² or k=0.25

This calculation for substructure only, hence the design of superstructure approximated in this calculation.

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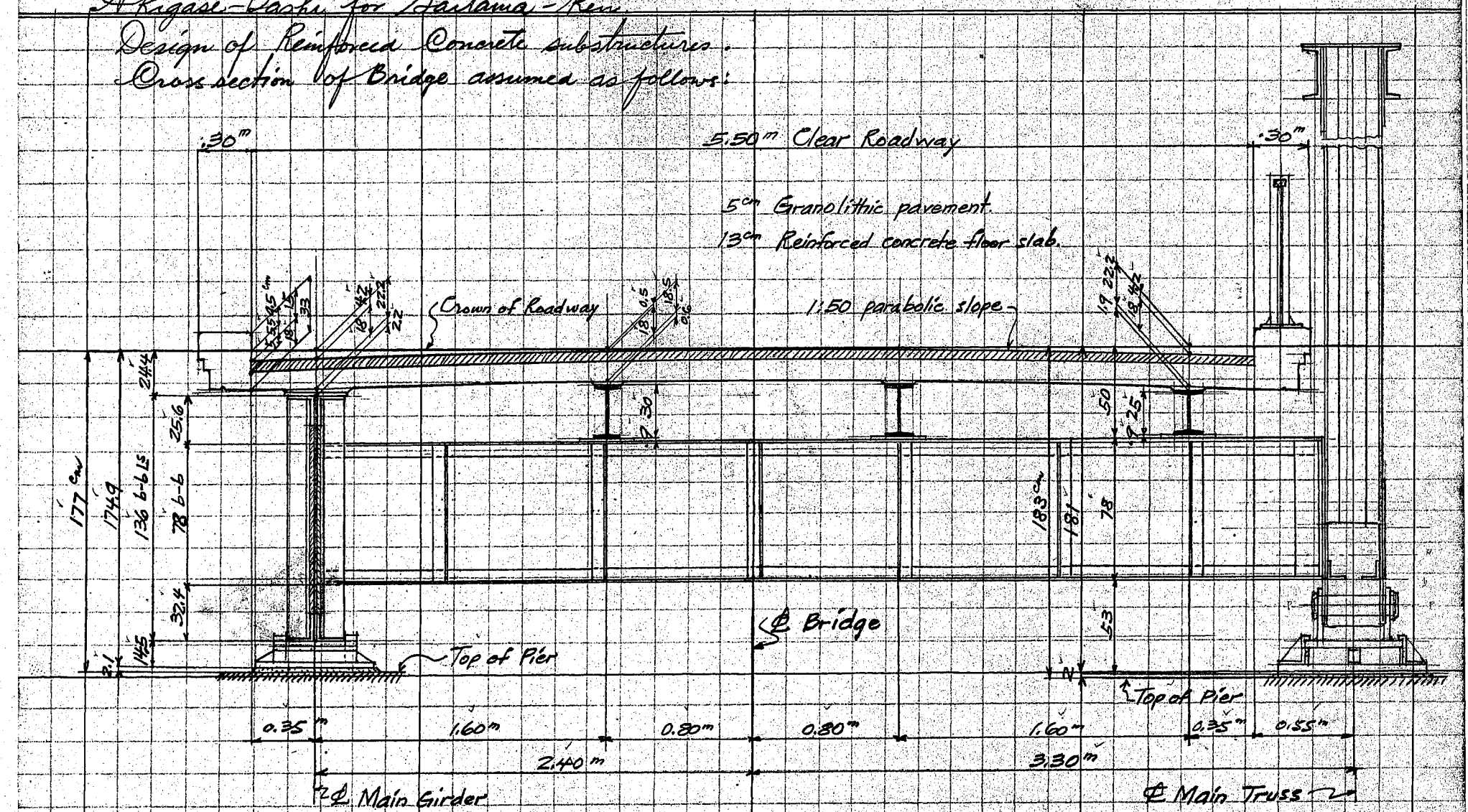
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Design of Reinforced Concrete substructures.

Cross section of Bridge assumed as follows:



GIRDER SPAN TRUSS SPAN
CROSS SECTION OF BRIDGE
Scale 1:30

Approximate Dead Load of Bridge:

Girder Span.

Floor	Granolithic pavement 5cm	@ 22 kg	=	110	
	Concrete floor slab 13"	@ 24 kg	=	312	
	fillets and misc concrete say			8	
				430 kg/m²	
	5.50m	@ 430	=		2,365
Copings say	2	@ 200	=		400
Handrails say	2	@ 75	=		150
					2915
Structural steel say	19150 kg ÷ 20.00		=		955
					3870 kg/lin.m.

Truss Span.

Floor	5.50	@ 430	=		2365
Copings say	2	@ 200	=		400
Handrails "	2	@ 75	=		150
					2916
Structural steel say	59000 kg ÷ 37.50		=		1575
					4490 kg/lin.m.

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Design of Pier for Girder Span.

Superimposed Loads on pier.

Dead Load:

3870 kg per lin. meter) Distance c to c of piers 20.10 meters assumed.
3870 × 20.10 = 77,800 kg
Shoes, Electric wiring etc say 400
78200 kg = D

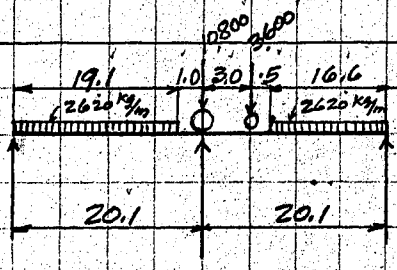
Live Load:

Uniform live load on roadway

$w = \frac{100000}{170+l}$ where l = loaded length in meters

$= \frac{100000}{170+40.2} = 475$ kg per sq. meter

5.50 m @ 475 = 2620 kg per lin. meter of span.



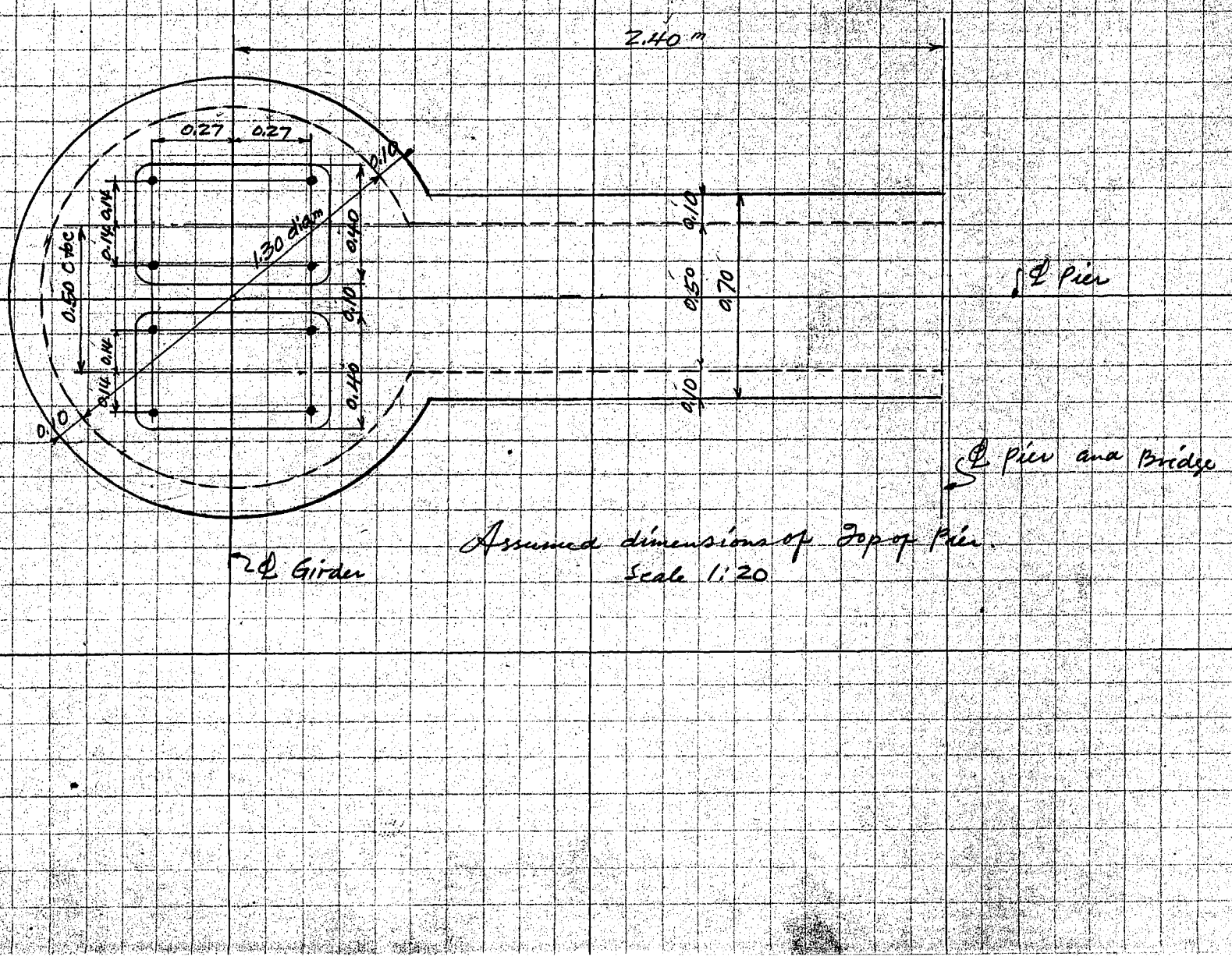
6 ton - motor truck

Rear wheel concentration = 2260
Impact coef. = $\frac{20}{60+40.2} \fallingdotseq 20\%$ = 450
2700 kg

For 2 rows of trucks 4 wheels @ 2700 = 10800 kg
Front wheels with impact say 10800 ÷ 3 = 3600

Superimposed Live load due to,

uniform load $\frac{2620 \times 16.6^2}{2 \times 20.1}$ = 17950
" $\frac{2620 \times 19.1^2}{2 \times 20.1}$ = 23750
rear wheels = 10800
front " $3600 \times \frac{17.1}{20.1}$ = 3060
55560 kg
Call this 55600 kg = L



Assumed dimensions of Top of Pier.
Scale 1:20

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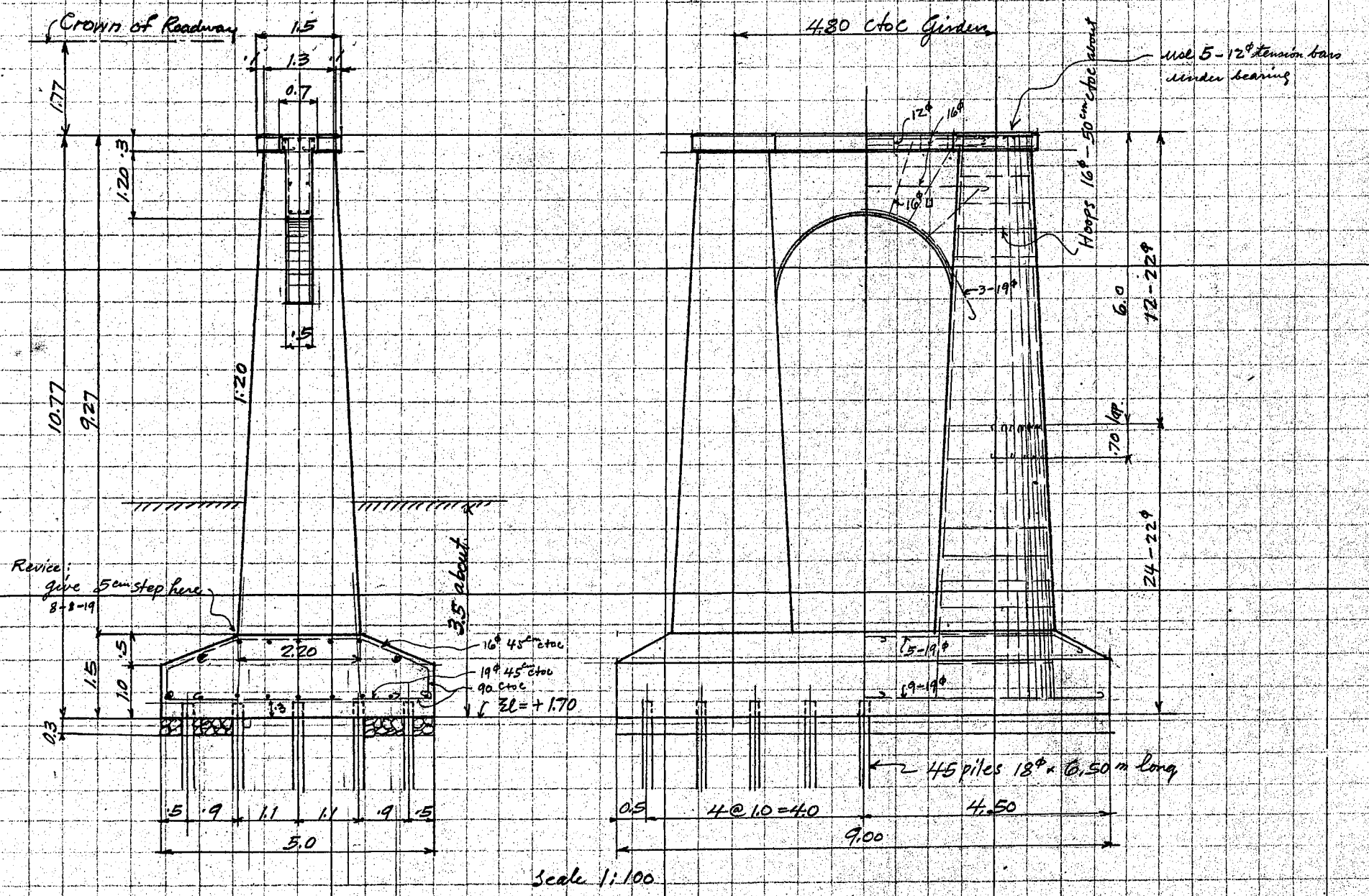
CHECKED BY H.T. DATE 8-10-20 PAGE NO 5

CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

General sketch of Pier for Girder span. (Pier P612)

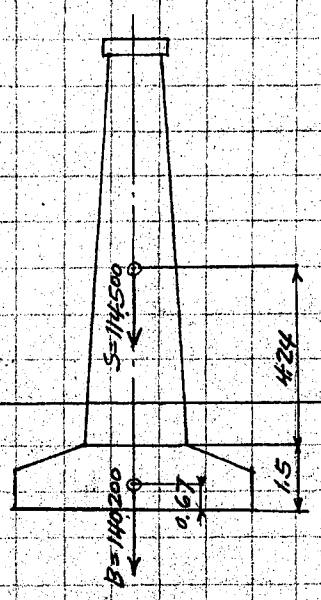


Scale 1:100

Weight and center of gravity of pier.

Pier shaft.

				Vol. of concrete		Lever arm from bott. of shaft		
Coping	1.5φ × 0.30 × 2	=	1.06	×	9.12	=	9.67
"	0.70 × 0.30 × 3.40	=	0.71	×	9.12	=	6.48
Strut	0.50 × 1.60 × 3.50	=	2.80	×	8.17	=	22.88
Shafts.	1.75φ × 8.97 × 2	=	43.10	×	3.78	=	163.00
			47.67		4.24 m		202.03
Base	3.60 × 0.50 × 7.45	=	13.40	×	1.22 (from bott. of base)	=	16.35
"	5.00 × 1.00 × 9.00	=	45.00	×	0.50	=	22.50
			58.40		0.67 m		38.85



Summary of concrete = 106.07 cub. meters for one pier.

weight of pier.

Shaft. 47.67 @ 2400 = 114,500 kg = S
base 58.40 @ 2400 = 140,200 " = B
= 254,700 kg

Seismic forces. acceleration assumed 2500 mm/sec² or k = 0.25

shaft S' = 114,500 × 0.25 = 28,600 kg.
base B' = 140,200 × 0.25 = 35,000 "

Earth on footing. Height 4.0 m above bottom of base assumed.

2.8 × 2.2 × 9.0 = 55.4 @ 1600 = 88,600 kg = E

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CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Stability of Pier.

Case 1. Dead load only:

Superimposed Dead Load	=	78200 kg	see on page 4.
Dead load of pier	=	254700 "	" " 5
Earth on footings	=	88600 "	
		421500 kg	

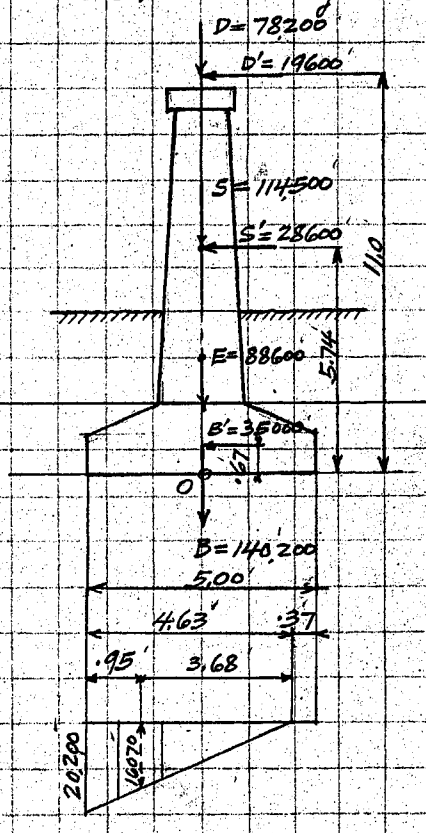
Bearing area = 5.0 × 9.0 = 45.00 square meters.
Unit Bearing pressure on earth
= 421500 ÷ 45 = 9370 kg/m² or (0.86 ton/ft²)
Number of piles assumed 9 rows @5 = 45 assumed
Load on one pile = 421500 ÷ 45 = 9370 kg

Case 2. Fully loaded:

Total Dead load + earth	=	421500	
Live load	=	55600	see on page 4.
		477100 kg	

Unit bearing pressure on earth
= 477100 ÷ 45.00 = 10600 kg/m² (correspond to 0.97 ton/ft²)
Load on one pile = 477100 ÷ 45 = 10600 kg

Case 3. During Earthquake: k assumed 0.250



Taking moment about the center of base area O,

Load mark	Vert. forces	Hor. forces	Lev. arms		moments
D	78200		× 0	=	0
D'		19600	× 11.00	=	215600
S	114500		× 0	=	0
S'		28600	× 5.74	=	164200
E	88600		× 0	=	0
B	140200		× 0	=	0
B'		35000	× 0.67	=	23500
	421500	83200	0.957 m		403300

Eccentricity = 0.957 > 5.00/6
Resultant force outside of middle third, neglecting tension, we have.
pressure area = 2.50 − 0.957 = 1.543 × 3 × 9.00 = 41.70 Sq. meters
Max. toe pressure = 421500 × 2 / 41.70 = 20200 kg/m² (correspond to 1.85 tons/ft²)

Max. load on one pile.
20200 × 3.68 / 4.63 = 16070
20200
36270 ÷ 2 = 18135

Load on one pile = 18135 × 0.95 × 1.00 = 17430 kg
use 45 piles 18 cm φ × 6.50 m long

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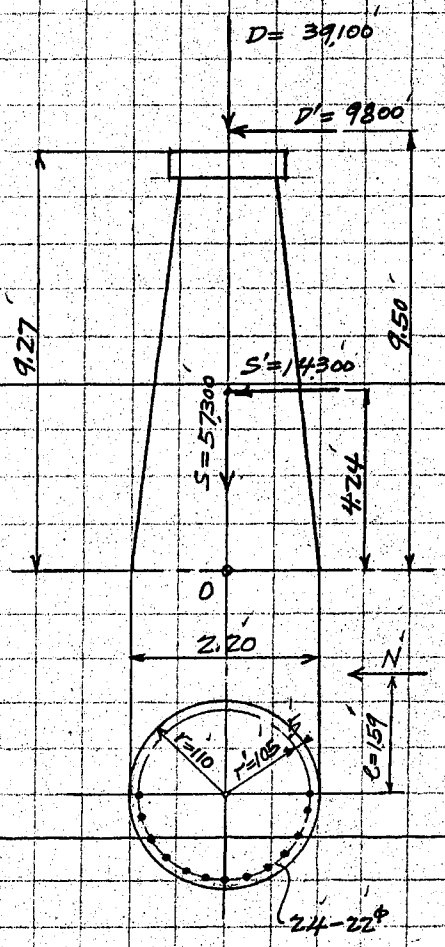
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Seismic stresses in shaft at bottom section.



Loads on one column shaft. Seismic load

Superimposed Dead Load	D =	39100 kg	D' =	9800 kg
Weight of shaft	S =	57300 "	S' =	14300 "

Taking moment about center of bottom area O.

Load marks	Hor. forces	Vert. forces	Lev. arms	moments
D		39100	× 0 =	0
D'	9800		× 9.50 =	93100
S		57300	× 0 =	0
S'	14300		× 4.24 =	60600
	24100	N= 96400	e= 1.59 m	153700 kg.m.

Steel area assumed 24-22φ bars = 91.2 cm²

Steel ratio $p = \frac{91.2}{110^2\pi} = 0.0024$

$r = 110$ cm, $r' = 105$, $r'/r = 105/110 = 0.955$, $e/r = 159/110 = 1.445$

By the prepared diagrams (柴田直光著 ノモグラムに依る鉄筋混凝土の計算参照)

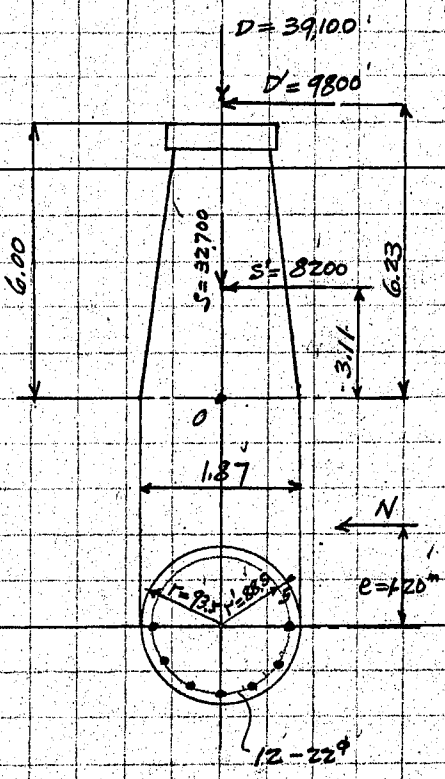
$k = 0.57$, $C = 0.22$

Direct stress $N = 96400$ kg

$$f_c = \frac{N}{r^2 C} = \frac{96400}{110^2 \times 0.22} = 36.2 \text{ kg/cm}^2 < 40.5 \times 1.6 = 64.8 \text{ (for 1:2.5:5 Concrete)}$$

$$f_s = n f_c \frac{1 + \frac{r'}{r} - k}{k} = 15 \times 36.2 \times \frac{1 + 0.955 - 0.57}{0.57} = 1320 \text{ kg/cm}^2 < 1200 \times 1.6 = 1920 \text{ (seismic)}$$

Seismic stresses in shaft at section 6m below top.



Weight and center of gravity.

Coping		1.06 × 5.85 =	6.20	
"		0.71 × 5.85 =	4.15	
Strut		2.80 × 4.90 =	13.72	
Shafts	1.59φ × 5.70 ×2 = 22.65	× 2.67 =	60.50	
	27.22	3.11 "	84.57	

$S = \frac{27.22}{2} \times 2400 = 32700$ kg for one column.

$S' = 32700 \times 0.25 = 8200$ kg " "

Taking moment about O,

Load marks	Hor. forces	Vert. forces	Lev. arms	Moment
D		39100	× 0 =	0
D'	9800		× 6.23 =	61000
S		32700	× 0 =	0
S'	8200		× 3.11 =	25500
	18000 kg	N= 71800 kg	e= 1.20 m	86500

Steel area assumed 12-22φ bars = 45.6 cm²

Steel ratio $p = \frac{45.6}{93.5^2\pi} = 0.00166$

$r'/r = 88.5/93.5 = 0.947$, $e/r = 120/93.5 = 1.285$

$k = 0.55$, $C = 0.229$

$$f_c = \frac{71800}{93.5^2 \times 0.229} = 35.8 < 64.8$$

$$f_s = 15 \times 35.8 \times \frac{1 + 0.947 - 0.55}{0.55} = 1360 \text{ kg/cm}^2 < 1920$$

Assumed sections are ample!

増田淳氏関係資料
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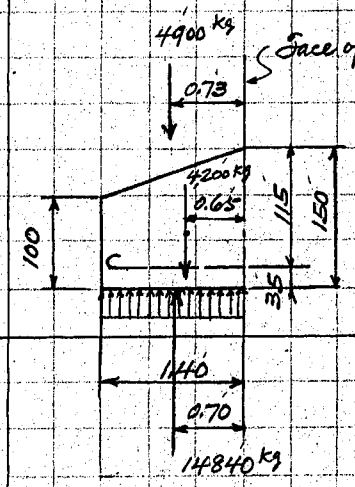
MADE BY K. Inaba DATE 8-10-17 FILE NO
CHECKED BY H T DATE 8-10-23 PAGE NO 8

CALCULATIONS FOR

Akigase – Bashi for Saitama-Ken

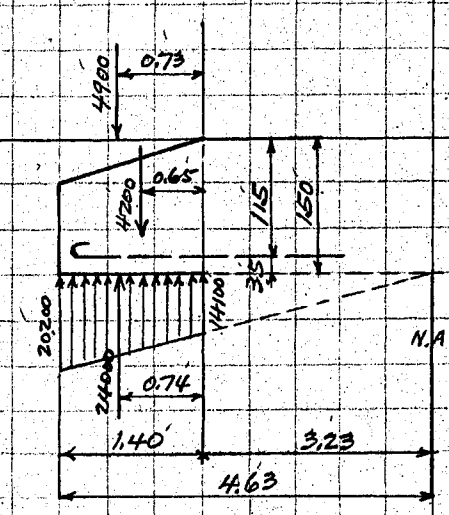
Stresses in footing

Case 1. Stresses at normal state.



Loads on footing per meter strip			arm		moment
Upward force	10600 × 1.40 =	14840 kg	× 0.70	=	10,380
Downward force					
Concrete	1.25×1.40@2400 =	−4200	× 0.65	=	−2,730
Earth	2.2×1.40@1600 =	−4900	× 0.73	=	−3580
		5740 kg			4070 kgm

Case 2. Stresses during Earthquake.



Loads on footing per meter strip.

Upward force.

$$20200 \times \frac{3.23}{4.63} = 14100$$

$$\frac{20200}{34300 \div 2 = 17150 \times 1.4 = 24000 \text{ kg}}$$

Downward forces same as for Case 1.

moment on footing				
Upward force	24000	× 0.74	=	17,750
Downward force Concrete	− 4200	× 0.65	=	− 2730
" earth	− 4900	× 0.73	=	− 3580
	14900 kg			11,440 kgm

Effective depth required

$$d = \sqrt{\frac{M}{bR}}$$ where $R = 6.04$ for 1:2.5:5 concrete.

$f_c = 40.5$, $f_s = 1200$

$$d = \sqrt{\frac{11440 \times 100}{100 \times 6.04 \times 1.6}} = 34.4 \text{ cm}$$ (1.6 = seismic coef.)

Use 115 cm effective depth with 5 cm insulation (底部 30 cm、土含混凝土)

Steel area required $= \frac{11440 \times 100}{1920 \times \frac{7}{8} \times 115} = 5.92 \text{ cm}^2$ per meter strip

use 19 mmϕ bars at 45 cm c to c = 6.30 cm²

Unit shear $= \frac{14900}{100 \times \frac{7}{8} \times 115} = 1.48 \text{ kg/cm}^2$

Longitudinal reinforcements in base:

Upward pressure at normal state.

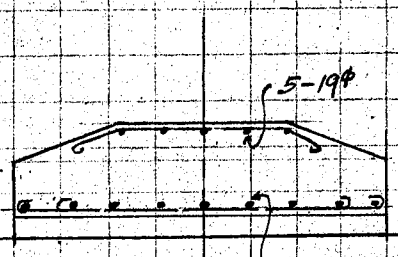
$10600 \times 5.0 = 53000$ kg per lin meter

Downward forces

Concrete say $-140200 \div 9.0 = -15600$

earth $2.2 \times 5 @ 1600 = -17600$

$19,800$



moment on base say

$$m = \frac{19800 \times 3.0^2}{12} = 14900 \text{ kgm}$$

Steel area req'd. $= \frac{14900 \times 100}{1200 \times \frac{7}{8} \times 115} = 12.33 \text{ cm}^2$

use 5-19 mmϕ bars = 14.17 cm² on top and bottom each

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MADE BY K. Inaba DATE 8-10-18 FILE NO
CHECKED BY H.T. DATE 8-10-23 PAGE NO 9

CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken

Design of Pier for Truss span PT2.

Span length 38.1 meters c to c of piers average.

Superimposed Dead load on pier.

4490 × 38.1 = 171,000 kg see on page 3.

Thos electric wiring &c say 800

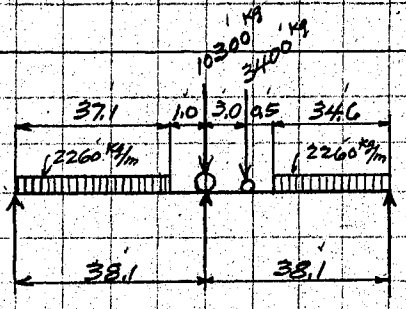
171,800 kg = D.

Superimposed live load on pier

Uniform live load on roadway

$w = \frac{100,000}{170+76.2} = 406$ kg/m² Call this 410 kg/m²

5.50 m @ 410 = 2260 kg per lin. meter of span.



6 ton motor truck

Rear wheel concentration = 2250

Impact coef. = $\frac{20}{60+76.2} = 14.7\%$ = 330

2580 kg

For two rows of trucks, 4 wheels @ 2580 = 10300 kg

Front wheels with impact say 10300 ÷ 3 = 3400 "

Superimposed live load due to

Uniform load $\frac{2260 \times 37.1^2}{2 \times 38.1}$ = 40850

" $\frac{2260 \times 34.6^2}{2 \times 38.1}$ = 35,500

rear wheels 10,300

front " $\frac{3400 \times 35.1}{38.1}$ = 3130

89,780 kg

Call this 89,800 kg = L

Assumed Dimensions of Top of Pier.
Scale 1:20

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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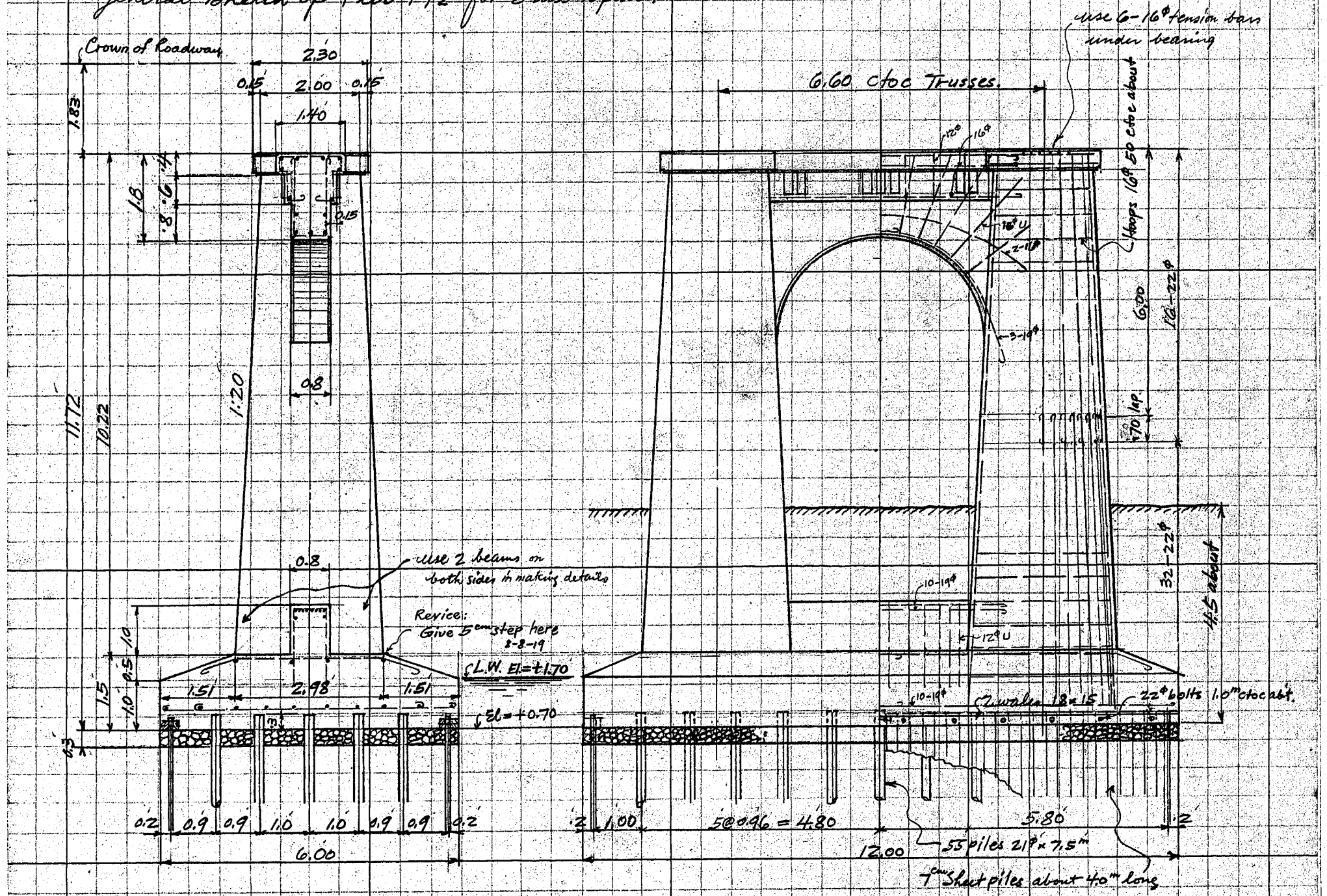
~~SHIBA~~ SHOWA BLDG, TOKYO

MADE BY K. Inaba DATE 8-10-18 FILE NO
CHECKED BY H.T DATE 8-10-23 PAGE NO 10

CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

General Sketch of Pier P$_{T2}$ for Truss span.

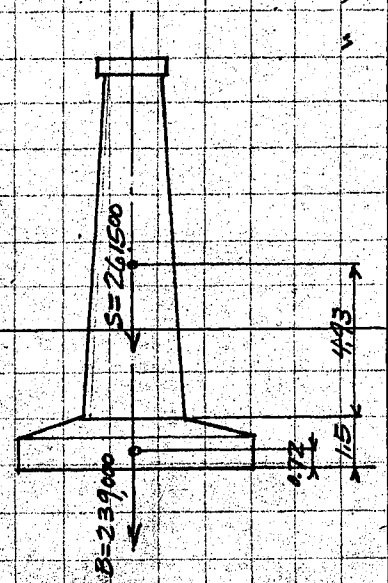


Scale 1:100

Weight and Center of Gravity of Pier.

				Volume of Concrete		Lever arm from Bottom of Shaft		
Pier shaft	Coping	2.30$^{\phi}$	× 0.40′ × 2′	= 3.32′	×	10.02′	=	33.27′
	″	1.40′	× 0.40 × 4.50′	= 2.52′	×	10.02′	=	25.25′
	Strut	1.10′	× 0.60′ × 4.70′	= 3.10′	×	9.52′	=	29.50′
	″	0.80′	× 1.20 × 4.50′	= 4.32′	×	8.62′	=	37.24′
	Shafts	2.49$^{\phi}$	× 9.82′ × 2′	= 95.64′	×	4.30′	=	411.30′
				108.90′		4.93^{m}		536.56′

				arm from bott. of base			
Base	4.49′	× 0.50 × 11.00	=	24.70′	×	1.22′	=	30.13′
″	6.00′	× 1.00 × 12.00	=	72.00′	×	0.50′	=	36.00′
″	0.80′	× 1.00′ × 3.60	=	2.88′	×	2.00′	=	5.76′
				99.58′		0.72^{m}		71.89′



Summing of Concrete = 208.48 cub. meters
weight of pier, Shaft. 108.90 @ 2400 = 261,500′ kg = S′
base 99.58 @ 2400 = 239000′ ″ = B′
500,500′ ″

Seismic forces. acceleration assumed as 2500 cm/sec^2 or K=0.250
Shaft. S′= 261500 × 0.25 = 65400′ kg
B′= 239000 × 0.25 = 59800′ ″

weight of earth on footing
6.0 × 12.0 × 3.0 = 216.0′
less shaft 2.83$^{\phi}$ × 3.0 × 2 = − 37.8′
″ strut. 0.80 × 1.0 × 3.6 = − 2.9′
175.3′ @ 1600 = 280,000 kg = E′

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MADE BY K. Isata DATE 8-10-18 FILE NO
CHECKED BY H. T. DATE 8-10-23 PAGE NO 11

CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Stability of Pier.

Case 1. Dead Load only:

Superimposed Dead Load = 171,800' kg see on page 9.
Dead Load of Pier S+B = 500,500' " " 10'
weight of earth on footing = 280000'
952,300' kg

Bearing area = 6.0 × 12.0 = 72.0 square meters.
Unit bearing pressure on earth
= 952,300' / 72.0 = 13230' kg/m² (Correspond to 1.21' tons/a')

Load on one pile, say 0.95 × 0.96 × 13200 = 12,030' kg

Case 2. Fully loaded:

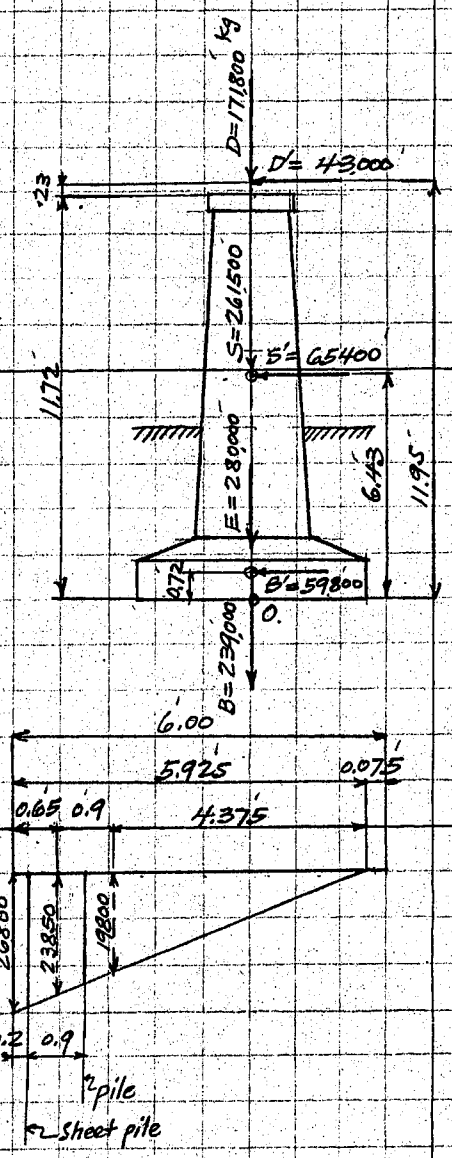
Total Dead Load + Earth load = 952,300'
Live Load = 89,800' see on page 9.
1,042,100' kg

Unit bearing pressure on earth
= 1042100 / 72.0 = 14480' kg/m² (Correspond to 1.32' tons/a')

Case 3. During Earthquake: k assumed as 0.25

Taking moment about center of base area 0.

Load marks	Hor. forces	Vert. forces	Lev. arms	moments
D		171,800' ×	0' =	0'
D'	43,000'	×	11.95' =	514,000'
S		261,500' ×	0' =	0'
S'	65,400'	×	6.43' =	420,000'
E		280,000' ×	0' =	0'
B		239,000' ×	0' =	0'
B'	59800'	×	0.72' =	43,000'
	168,200'	952,300'	1.025 m	977,000'



Eccentricity = 1.025 > 6.0/6
Resultant force outside of middle third, neglecting tension, we have
pressure area = 3.00 − 1.025 = 1.975 × 3 × 12.0 = 71.1' sq meters.
max. toe pressure
= 952,300 × 2 / 71.1 = 26800' kg/m² (Correspond to 2.45' tons/a')

Load on sheet piles and piles.

26800' / 5.925' × 4.375' = 19800' kg/m²

26800' / 5.925' × 5.275' = 23850' "

Load on sheet piles

23850'
26800'
50650 ÷ 2 = 25330' kg/m²

or 25330 × 0.65 = 16450' kg per lin. meter of sheet piles.

Load on one pile next to sheet piles

19800'
23850'
43650 ÷ 2 = 21830' kg/m²

Load on one pile = 21830 × 0.90 × 0.96 = 18850' kg.

Use 7 cm Sheet piles about 4.0 meter long
55 piles 21 cm φ 7.5 meter long

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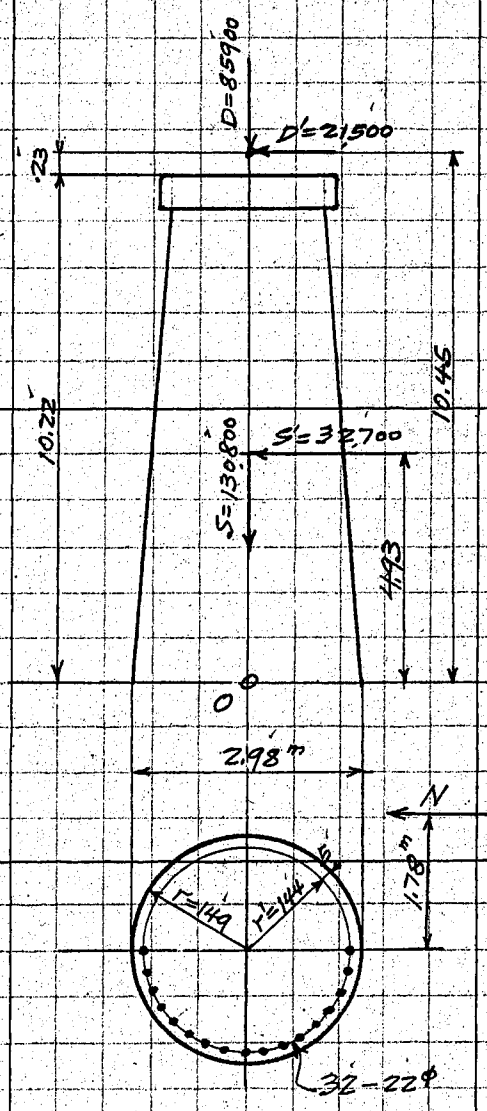
CALCULATIONS FOR

Akigase-Bashi for Saitama Ken.

Seismic stresses in shaft at bottom section.



Loads on one column shaft. Seismic load
Superimposed Dead Load D = 85900 kg, D' = 21500 kg
weight of shaft S = 130800, S' = 32700.

Taking moment about center of bottom area O.

Load marks	Hor. forces	Vert. forces	Lev. arm	moments
D		85900 ×	0 =	0
D'	21500	×	10.45 =	224500
S		130800 ×	0 =	0
S'	32700	×	4.93 =	161300
	54200 kg	N = 216700 kg	e = 1.78 m	385800

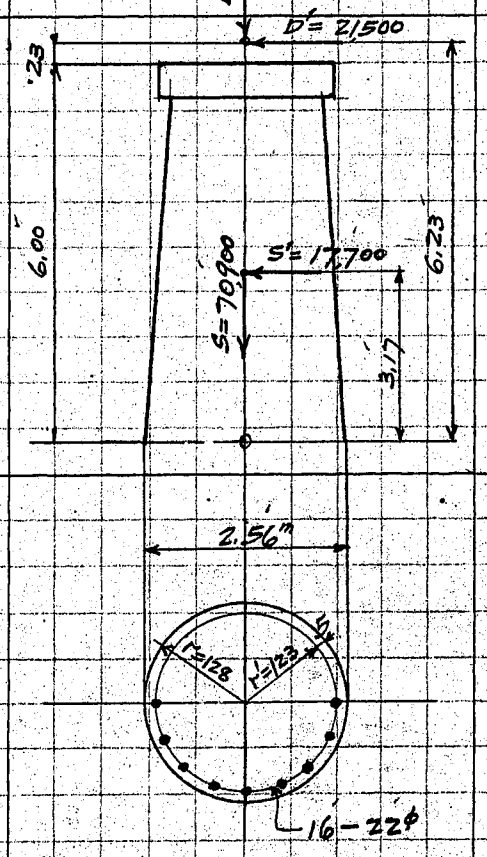
Steel area assumed as 32 - 22φ bars = 121.7 cm² = As
Steel ratio $p = \frac{As}{r^2\pi} = \frac{121.7}{149^2\pi} = 0.001745$

r = 149, r' = 144, $r'/r = 144/149 = 0.966$; $e/r = 178/149 = 1.195$
From the diagrams,
k = 0.58, c = 0.25

$$f_c = \frac{N}{r^2 c} = \frac{216700}{149^2 \times 0.25} = 39.0 \text{ kg/cm}^2 < 40.5 \times 1.6 = 64.8 \text{ (for 1:2.5:5 concrete)}$$

$$f_s = \frac{1+\frac{r'}{r}-k}{k} \cdot n f_c = 15 \times 39.0 \times \frac{1+0.966-0.58}{0.58} = 1398 \text{ kg/cm}^2 < 1200 \times 1.6 = 1920 \text{ (seismic)}$$
(1.386)

Seismic stresses in shaft at section 6 meters below top.



weight and center of gravity

Coping		3.32 ×	5.80 =	19.26
"		2.52 ×	5.80 =	14.63
Strut		3.10 ×	5.30 =	16.43
"		4.32 ×	4.40 =	19.02
Shafts	2.28φ × 5.6 × 2 = 45.75	×	2.58 =	118.00
	59.01		3.17 m	187.34

S = 59.01 ÷ 2 × 2400 = 70900 kg, S' = 70900 × 0.25 = 17700 kg

Taking moments about O.

Load marks	Hor. forces,	Vert. forces	Lev. arms	moments
D		85900 ×	0 =	0
D'	21500	×	6.23 =	134000
S		70900 ×	0 =	0
S'	17700	×	3.17 =	56100
	39200 kg	N = 156800 kg	e = 1.21 m.	190100

Steel area assumed as 16 - 22φ bars = 60.8 cm² = As.
Steel ratio $p = \frac{60.8}{128^2\pi} = 0.00118$

$r'/r = 123/128 = 0.961$, $e/r = 121/128 = 0.945$.
k = 0.63 c = 0.315.

$$f_c = \frac{156800}{128^2 \times 0.315} = 30.4 \text{ kg/cm}^2$$

$$f_s = 15 \times 30.4 \times \frac{1+0.961-0.63}{0.63} = 963 \quad (1.331)$$

Assumed sections are ample.

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CALCULATIONS FOR

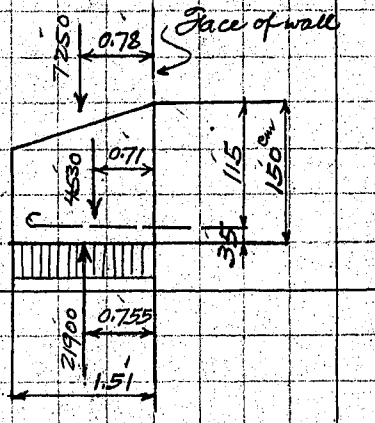
Akigase-Bashi for Saitama-ken.

Stresses in footing

Case 1. Stresses at normal state.

Loads on footing per meter strip.

			arm	moment
Upward force	14480 × 1.51 =	21,900	× 0.755 =	16,530
Downward force				
Concrete	1.25 × 1.51 @2400 =	−4530	× 0.71 =	−3220
earth	3.0 × 1.51 @1600 =	−7250	× 0.78 =	−5650
		10,120 kg		7,660 kgm



Case 2. Stresses during earthquake.

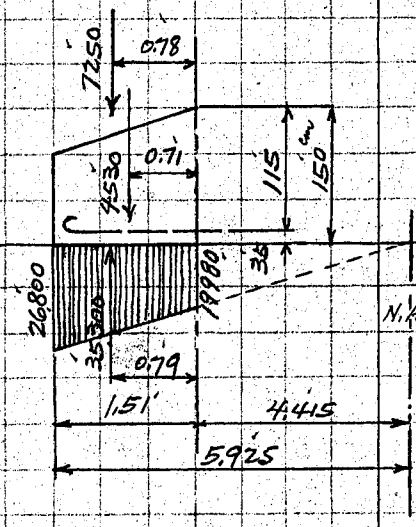
Loads on footing per meter strip.

Upward pressure

$26800 \times \frac{4.415}{5.925} = 19980$

26800

46780 ÷ 2 = 23390 × 1.51 = 35300 kg



moment on footing

upward force	35300 × 0.79 =	27900	
Downward force concrete	− 4530 × 0.71 =	− 3220	
earth	− 7250 × 0.78 =	− 5650	
	23520 kg	19030	kgm

effective depth required $d = \sqrt{\frac{M}{BR}}$ where R = 6.04 for 1:2½:5 Concrete, f_c = 40.5, f_s = 1200

$d = \sqrt{\frac{19030 \times 100}{100 \times 6.04 \times 1.6}} = 44.4$ cm

Use 115 cm effective depth with 5 cm insulation (原予 30 cm ハ 拾貳吋鐵ト)

Steel area required = $\frac{19030 \times 100}{1920 \times \frac{7}{8} \times 115} = 9.85$ cm² per meter strip.

Use 19 mmϕ bars at 25 cm c to c = 11.35 cm²

Unit shear = $\frac{23520}{100 \times \frac{7}{8} \times 115} = 2.34$ kg/cm²

Longitudinal reinforcements in base.

Upward pressure at normal state.

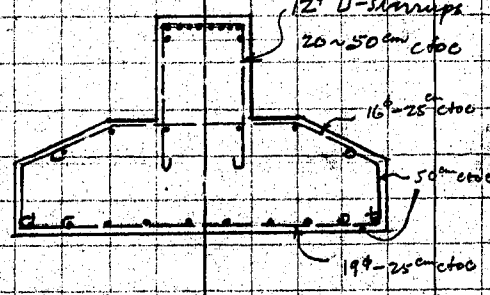
14480 × 6.0 = 86900 kg per lin. meter

Downward force

Concrete say −23900 ÷ 12 = −19900 "

earth " −28000 ÷ 12 = −23400 "

43600 "



moment on base say

$M = \frac{43600 \times 4.00^2}{12} = 58100$ kgm

Steel area req'd. = $\frac{58100 \times 100}{1200 \times \frac{7}{8} \times 215} = 25.75$ cm²

Use 10-19 mmϕ bars = 28.35 cm² on top and bottom each.

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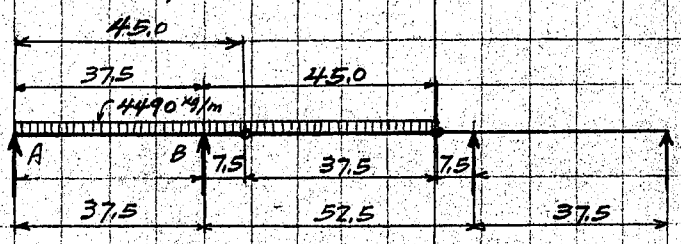
CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Design of Pier P14 for truss span.

Superimposed loads on pier.

Dead Load :- 4490 kg per lin. meter of span. See on page 13.



Concentration on cantilever end

$4490 \times 37.5 \div 2 = 84200$ kg

Taking moment about A-support

$4490 \times 45 \times 22.5 = 4543000$

$84200 \times 45.0 = 3790000$

8333000

Reaction at Support B. $= \frac{8333000}{37.5} = 222300$

Shoes, electric wiring +c say 1200

Dead Load on pier $= 223500$ kg $= D$

Live Load :-

Uniform live load on roadway

$w = \frac{100000}{170+82.5} = 39.6$ call this 400 kg/m²

5.50 m @ 400 = 2200 kg per lin. meter of bridge.

6 ton motor truck rear wheel concentration = 2250

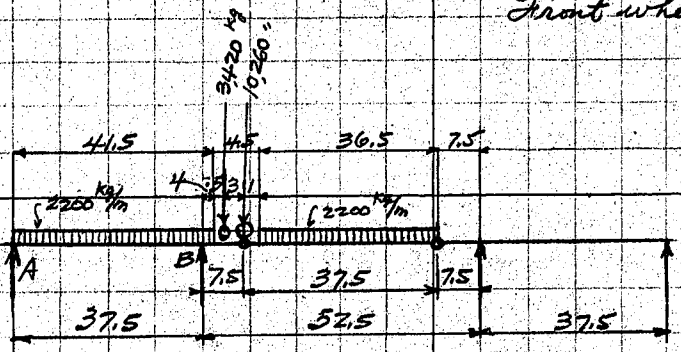
Impact coefficient $= \frac{20}{60+82.5} = 14\%$ $= \frac{315}{2565}$ kg

For two rows of trucks 4 wheels @ 2565 = 10260 kg

Front wheels with impact say $10260 \div 3 = 3420$

Concentration on cantilever end due to uniform load on suspended span.

$\frac{2200 \times 36.5^2}{2 \times 37.5} = 39100$ kg



Taking moment about A-Support

unif load	$2200 \times 41.5 \times 20.75$	=	1893000
front wheels	3420×42.00	=	143700
rear wheels	10260×45.00	=	461800
Suspended span	39100×45.00	=	1760000
			4258500

Reaction at Support B $= \frac{4258500}{37.5} = 113600$ kg $= L$

Summary of Superimposed loads on pier.

	On the whole pier,	On one-half of Pier
Dead Load D=	223500	111800
Live Load L=	113600	56800
	337100 kg	168600 kg
Seismic load D'=	55900 kg	28000 kg

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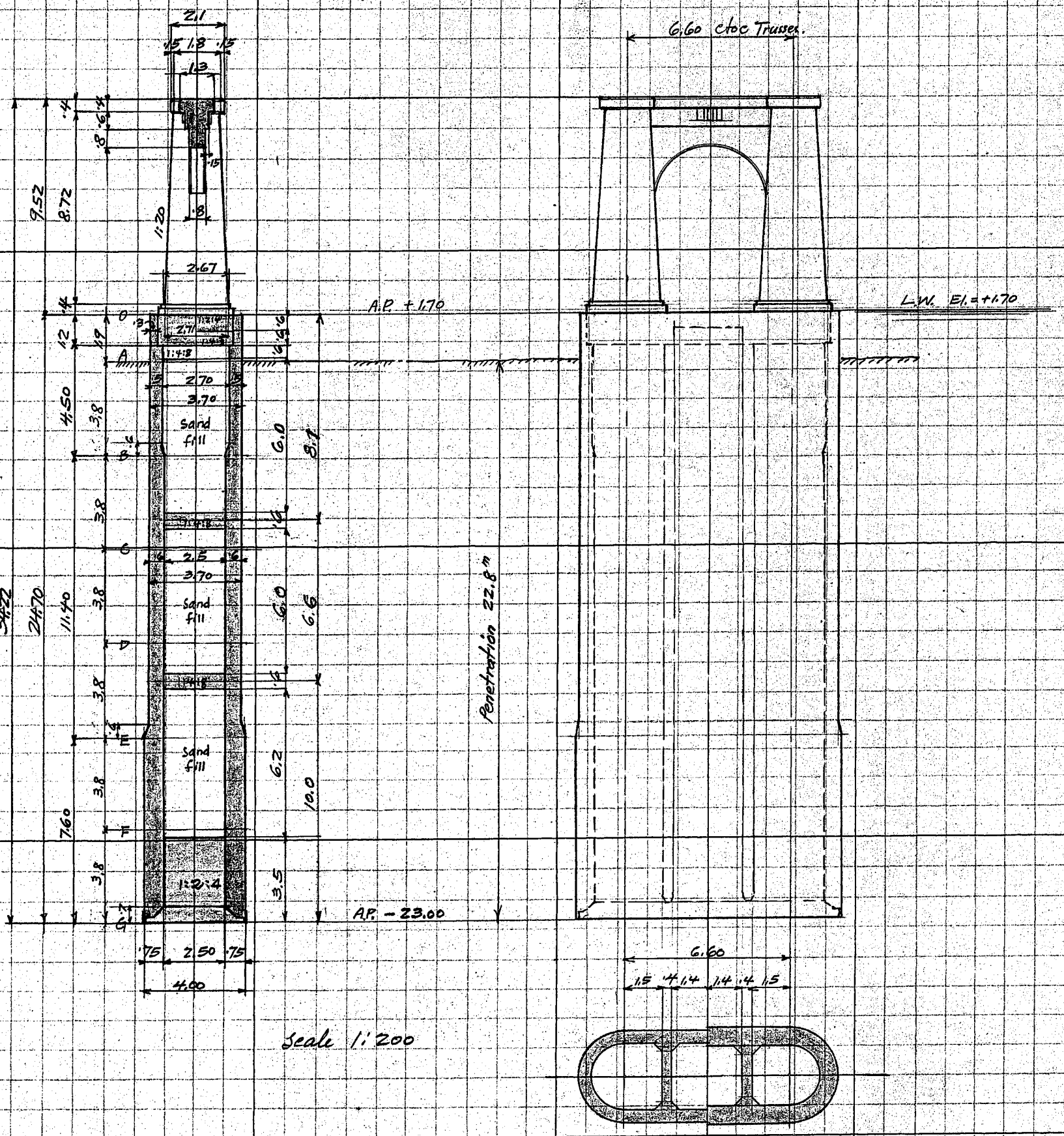
CALCULATIONS FOR

Akigase Bashi for Saitama Ken.

General sketch of Pier P14 for River spans.



Scale 1:200

Weight and center of gravity of Pier shaft.

	Dimensions		Volume	lev. arm	moment abt. bottom of shaft.
Copings	2.10$^{\phi}$ × 0.40	×2 =	2.77	× 9.32 =	25.80
=	1.30 × 0.40 × 4.70	=	2.444	× 9.32 =	22.75
Strut	1.10 × 0.60 × 4.90	=	3.23	× 8.82 =	28.50
=	0.80 × 1.30 × 4.70	=	4.89	× 7.87 =	38.47
Shafts	2.25$^{\phi}$ × 9.12	× 2 =	72.53	× 3.97 =	288.00
Collar footings say	0.20 × 0.40 × 2.89$^{\phi}$	× 2 =	1.45	× 0.20 =	0.29
			87.31 m^3	4.63 m	403.81

weight of shaft 87.31 @ 2400 = 209500 kg = S
Seismic force 209500 × 0.25 = 52400 " = S'

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CALCULATIONS FOR

Akigase-Bashi for Daitama-Area

Weight and center of gravity of several sections of well.

Section O-A height 1.90 meters.

	3.70φ × 1.20	=	12.90	@	2400	=	31,000	×	1.30	= 40,300
	3.70 × 6.60 × 1.20	=	29.30	@	〃	=	70,300	×	1.30	= 91,400
section at A.	For 1:4:8 fill less 7.38×1.6	=	4.43	@	200 (2400−2200)	= −	900	×	1.00	= − 900
Total gross area	3.70φ = 10.75									
	3.70 × 6.60 = 24.42									
	35.17 m²									
End partition areas	2.70φ = 5.73									
〃	2.70 × 1.50 × 2 = 8.10									
fillets, less	0.30 × 0.30 × 2 = −0.18									
	13.65 m²									
Center partition area	2.70 × 2.80 = 7.56									
fillets, less	0.30 × 0.30 × 2 = − 0.18									
	7.38 m²									
Total partition area	= 21.03 m²									
Net area of shell	= 14.14 m²									
shell	14.14 × 0.70	=	9.90	@	2,400	=	23,800	×	0.35	= 8,330
1:4:8 fill	13.65 × 0.60	=	8.19	@	2200	=	18,000	×	0.40	= 7,200
sand fill	13.65 × 0.10	=	1.37	@	1600	=	2,200	×	0.05	= 110
〃	7.38 × 0.70	=	5.16	@	〃	=	8300	×	0.35	= 2,910
					W_A =		152,700 kg		0.98 m	149,350
					W_A' =		38,200 kg			

Section A-B height 3.80 meters

shell	14.14 × 3.80	=	53.70	@	2400	=	137,500	×	1.90	
sand fill	21.03 × 3.80	=	79.90	@	1600	=	127,800	×	1.90	
					W_B =		265,300 kg		1.90 m	
					W_B' =		66,300 kg			

Section B-C height 3.80 meters.

Total gross area	= 35.17 m²									
End partition areas	2.5φ = 4.91									
〃	2.50 × 1.5 × 2 = 7.50									
fillets, less	0.30 × 0.30 × 2 = −0.18									
	12.23 m²									
Center partition area	2.50 × 2.80 = 7.00									
fillets, less	−0.18									
	6.82 m²									
Total partition area	= 19.05 m²									
Net area of shell	= 16.12 m²									
shell	16.12 × 3.80	=	61.20	@	2,400	=	147,000	×	1.90	= 279,200
1:4:8 fill	19.05 × 0.60	=	11.43	@	2,200	=	25200	×	1.40	= 35,300
sand fill	19.05 × 1.10	=	20.95	@	1,600	=	33,500	×	0.55	= 18,450
〃	19.05 × 2.10	=	40.00	@	1,600	=	64000	×	2.75	= 176,000
					W_C =		269,700 kg		1.89 m	508,950
					W_C' =		67,400 kg			

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Section C-D height 3.80 meters.

Shell	16.12	× 3.80 =	61.20	@ 2400 =	147000	× 1.90	
Sand fill	19.05	× 3.80 =	72.40	@ 1600 =	115800	× 1.90	
				W_D =	262800 kg	1.90 m	
				W_D' =	65700 kg		

Section D-E height 3.80 meters.

Shell	16.12	× 3.80 =	61.20	@ 2400 =	147000	× 1.90 =	279200
1:4:8 fill	19.05	× 0.60 =	11.43	@ 2200 =	25200	× 2.40 =	60500
Sand fill	19.05	× 2.10 =	40.00	@ 1600 =	64000	× 1.05 =	67200
"	19.05	× 1.10 =	20.95	@ " =	33500	× 3.25 =	109000
				W_E =	269700 kg	1.91 m	515900
				W_E' =	67400 "		

Section E-F height 3.80 meters

Total gross area	4.00 ϕ	12.57	
"	4.00 × 6.60 =	26.40	
		38.97 m²	
2nd partition areas		12.23	
Centre "		6.82	
Total partition area	=	19.05 m²	
Net area of shell	=	19.92 m²	

Shell	19.92	× 3.80 =	75.70	@ 2400 =	181600	× 1.90	
Sand fill	19.05	× 3.80 =	72.40	@ 1600 =	115800	× 1.90	
				W_F =	297400 kg	1.90	
				W_F' =	74400 kg		

Section F-G height 3.80 meters

Shell	19.92	× 3.80 =	75.70	@ 2400 =	181600	× 1.90 =	345000
1:2:4 fill	19.05	× 3.50 =	66.70	@ 2200 =	146500	× 1.75 =	256500
Sand fill	19.05	× 0.30 =	5.71	@ 1600 =	9100	× 3.65 =	33200
Shoe				=	2500	× 0.20 =	500
				W_G =	339700 kg	1.87 m	635200
				W_G' =	85000 kg		

Summary of weight and centre of gravity.

Sections		Weights	Seismic forces		arm from bott of section	arm from bott of well	moments about bottom of well.
O-A	W_A =	152700	W_A' =	38200	0.98 m	23.78 m	908000
A-B	W_B =	265300	W_B' =	66300	1.90	20.90	1385000
B-C	W_C =	269700	W_C' =	67400	1.89	17.09	1152000
C-D	W_D =	262800	W_D' =	65700	1.90	13.30	874000
D-E	W_E =	269700	W_E' =	67400	1.91	9.51	641000
E-F	W_F =	297400	W_F' =	74400	1.90	5.70	424000
F-G	W_G =	339700	W_G' =	85000	1.87	1.87	159000
	W =	1857300 kg	W' =	464400 kg		11.95 m	5543000

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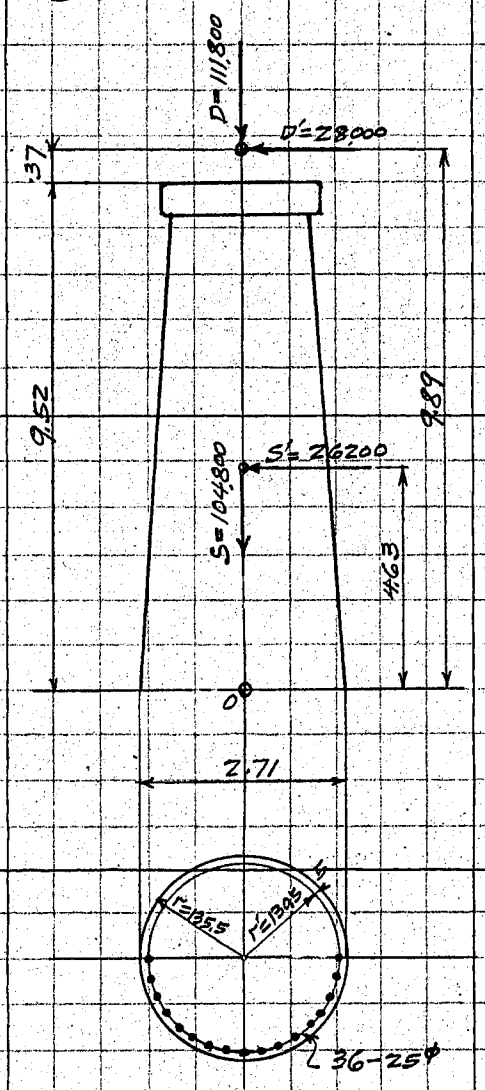
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Seismic stresses in shaft at Bottom section.



Load on one column shaft — Seismic loads.

Superimposed dead load	D = 111,800 kg	D' = 28,000 kg
Weight of shaft	S = 104,800 "	S' = 26,200 "

Taking moment about center of bottom area O.

Load marks	Hor. forces	Vert. forces	Leverarms	Moment abt. O
D		111,800	× 0	= 0
D'	28000		× 9.89	= 277,000
S		104,800	× 0	= 0
S'	26200		× 4.63	= 121,300
	54200 kg	216600 kg	e = 1.84 m	398,300

Steel area assumed as A_s = 36-25φ bars = 176.7 cm²

Steel ratio $p = \frac{176.7}{135.5^2\pi} = 0.00306$

r = 135.5 cm, r' = 130.5 cm, $r'/r = 130.5/135.5 = 0.963$, $e/r = \frac{184}{135.5} = 1.36$

From the prepared diagrams, we have as follows.

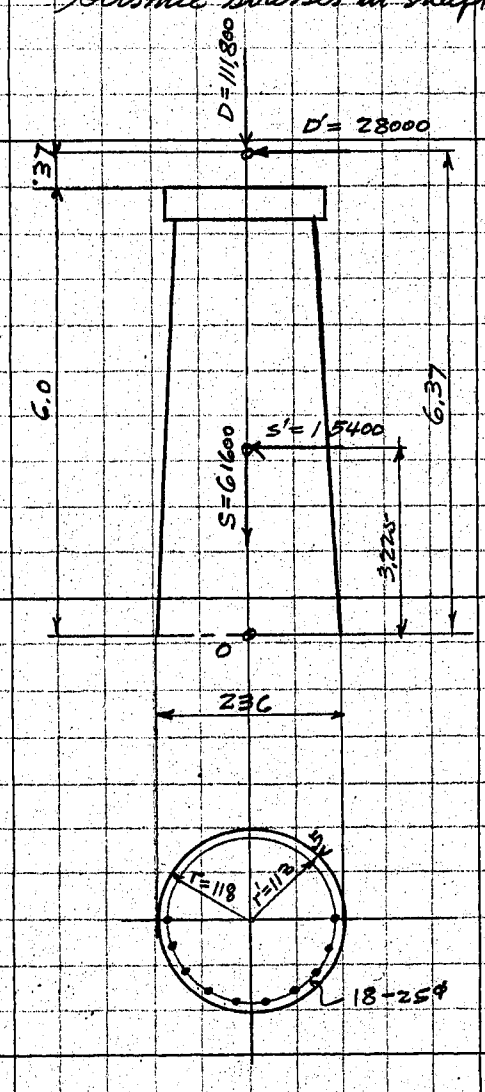
k = 0.64, c = 0.275

$$f_c = \frac{216600}{135.5^2 \times 0.275} = 42.9 \text{ kg/cm}^2 < 315 \times 1.6 \text{ for 1:2.5:5 concrete}$$

$$f_s = 42.9 \times 15 \frac{1 + 0.963 - 0.640}{0.640} = 1330 \text{ kg/cm}^2 < 1200 \times 1.6$$

(1.323)

Seismic stresses in shaft at section 6 m below top.



Weight and center of gravity.

Coping		2.77 × 5.80	=	16.07
"		2.44 × 5.80	=	14.15
Strut		3.23 × 5.30	=	17.12
"		4.89 × 4.35	=	21.28
Shaft	2.08φ × 5.60 × 2 = 38.00	× 2.55	=	97.00
	51.33	3.225 m		165.62

S = ½ × 51.33 @ 2400 = 61600 kg

S' = 61600 × 0.25 = 15400 "

Taking moment about point O.

Load marks	Hor. forces	Vert. forces	Lev. arms	Moments
D		111,800		
D'	28000		× 6.37	= 178,300
S		61600		
S'	15400		× 3.225	= 49600
	43400 kg	173,400 kg	1.314 m	227,900

$e/r = 131.4/118 = 1.113$

Steel area assumed as 18-25φ bars = 88.4 cm² = A_s

Steel ratio $p = \frac{88.4}{118^2\pi} = 0.00202$

$r'/r = 113/118 = 0.958$

k = 0.640, c = 0.30

$$f_c = \frac{173400}{118^2 \times 0.30} = 41.5 \text{ kg/cm}^2$$

$$f_s = 15 \times 41.5 \frac{1 + 0.958 - 0.64}{0.64} = 1280 \text{ kg/cm}^2$$

(1.318)

Assumed sections are ample.

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CHECKED BY H.T. DATE A-11-21 PAGE NO 19R

CALCULATIONS FOR

Ukigase-Bashi for Saitama-ken.

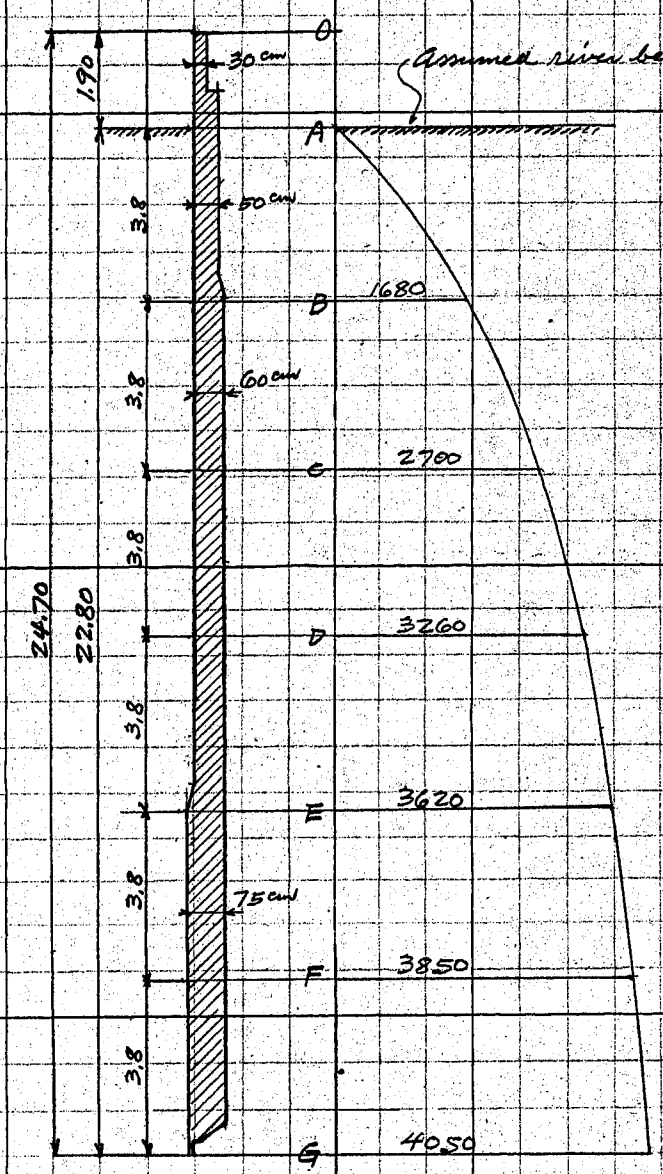
Design of well.

Temporary earth pressure on well during sinking execution. (Refered to Ketchum's "Walls, Bins, and grain elevators" on page 120 and 121)

$$L = \frac{w b}{2\mu}\left(1 - e^{\frac{-2K\mu y}{b}}\right), \quad V = \frac{w b}{2K\mu}\left(1 - e^{\frac{-2K\mu y}{b}}\right).$$

Where V = Vertical unit pressure in kg/m² at depth y meters;
L = Lateral " " " " " " " " ;
w = weight of earth in kg/m³;
μ = $\tan\phi$ = Coefficient of friction of earth on earth;
ϕ = angle of repose of the earth;
b = the distance in meters that the earth breaks around the well;
K = a constant $\fallingdotseq \frac{1-\sin\phi}{1+\sin\phi}$.

Let us assume that, $w = 1600$ kg/m³; $b = 30$ meters; and $K = \frac{1}{3}$.
From the prepared diagram, we have the values of L for different depths.



Sections at	Depth of earth y in meters	Temporary pressure on well in kg/m²
A	0.00	L = 0
B	3.80	1,680
C	7.60	2,700
D	11.40	3,260
E	15.20	3,620
F	19.00	3,850
G	22.80	4,050

Bending moment on wall at several sections.

Span length of straight wall = 3.20 meter.
Diameter of end rings = 3.2, 3.1 and 3.25 meters.

Straight walls: $m = \pm 1/12\, L l^2 = \pm 1/12 \times 3.20^2 L = \pm 0.853 L$
End rings $m = \pm 1/16\, L D^2 = \pm 1/16 \times 3.20^2 L = \pm 0.640 L$ Between A-B
or $\pm 1/16 \times 3.10^2 L = \pm 0.601 L$ " B-E
or $\pm 1/16 \times 3.25 L = \pm 0.660 L$ " E-G

Design all walls for the moment of $\pm 0.853 L$

Sections	Earth pressure L in kg/m²	Moment 0.853L in kgm M	Eff. depth of walls in cm d	Steel area req'd in cm²	Spacings for 16ϕ bars in cm	Unit shear in kg/cm²
A	0	0	46	0	∞	
B	1,680	1,430	46	2.96	68.0	0.7
			56	2.43	82.8	0.5
C	2,700	2,300	56	3.91	51.5	0.9
D	3,260	2,780	56	4.73	42.5	1.1
E	3,620	3,090	56	5.25	38.3	1.2
			71	4.15	48.5	0.9
F	3,850	3,280	71	4.40	45.7	1.0
G	4,050	3,450	71	4.63	43.5	1.0

note:- Steel area required $= \frac{M \times 100}{1200 \times \frac{7}{8} \times d} = \frac{M}{10.5d}$

Unit shear $= \frac{V}{100 \times \frac{7}{8} \times d} = \frac{3.2L}{200 \times \frac{7}{8} \times d} = 0.0183 \frac{L}{d}$

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CALCULATIONS FOR

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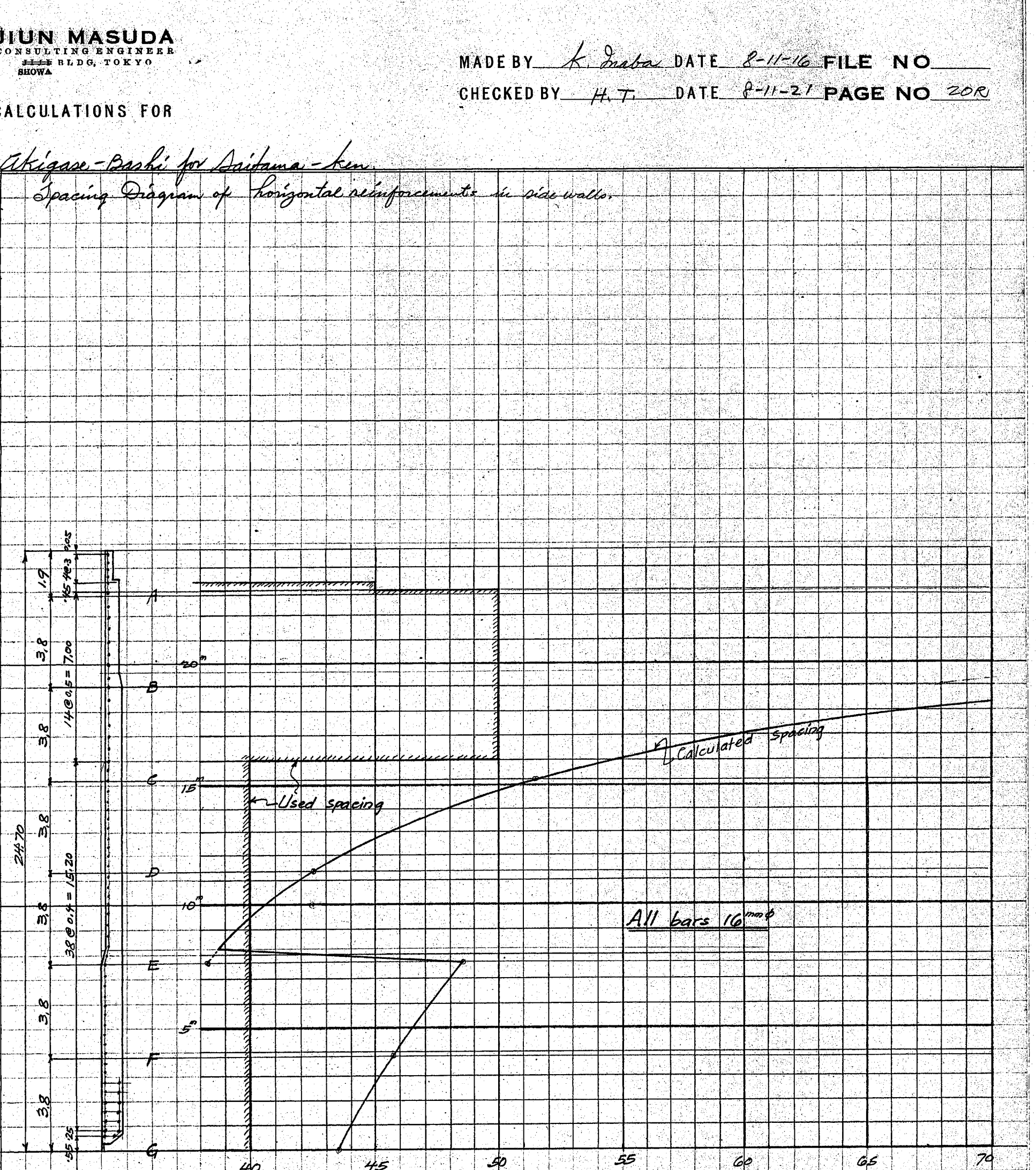
Spacing Diagram of horizontal reinforcements in side walls.



Note. Use same spacings on both sides of wall.

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CALCULATIONS FOR

Ukigase-Bashi for Saitama-Ken.

Stability of Pier

Stability at normal state.

Superimposed dead load on pier D = 223,500 ——— see on page 14R
" live load on pier L = 113600 ——— "
337100

weight of shaft S = 209500 ——— see on page 15R
weight of well W = 1857300 ——— " 17R
2066800
2403900 kg.

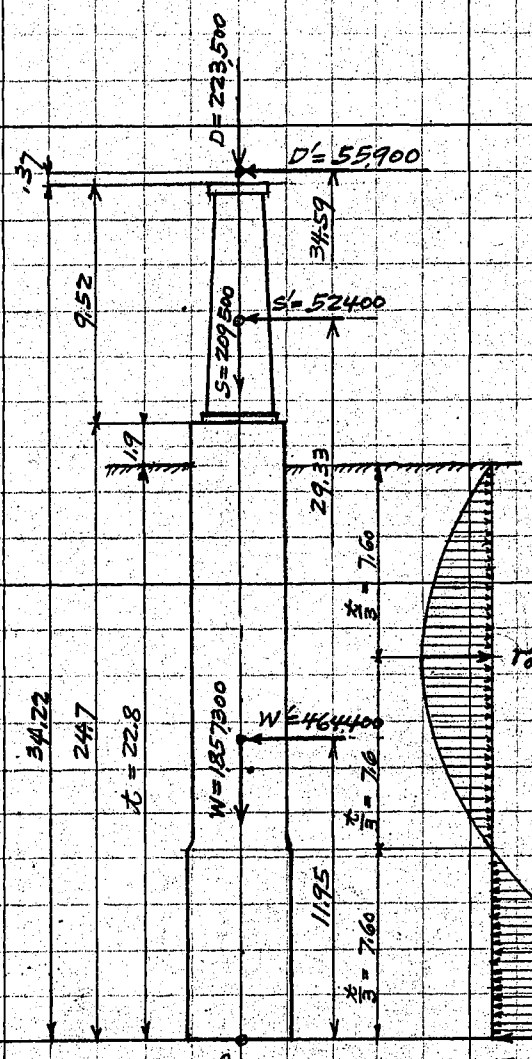
Frictional resistance between well and earth.
Resistance assumed as 1200 kg/m²
Area of skin friction.

3.70 φ = 11.62
6.60 × 2 = 13.20
24.82 × 15.20 = 377

4.00 φ = 12.57
6.60 × 2 = 13.20
25.77 × 7.60 = 196
573 m²

Frictional resistance = 573 @ 1200 = 688000 kg
Total load on bottom area 2403900 − 688000 = 1715900 kg
Base area = 38.97 m² (see page 17R)
Unit bearing pressure on soil = 1715900 / 38.97 = 44000 kg/m² (corresponds to 4.0 tons/ft²)

Stability of Pier during earthquake. Seismic coef. k assumed as 0.250



Seismic moment about center of bottom area O.

Load marks	Hor. forces	Vert. forces	Lev. arm	moment abt. O.
D		223,500		
D'	55900		× 34.59 =	1,933,000
S		209,500		
S'	52,400		× 29.33 =	1,537,000
W		1,857,300		
W'	464,400		× 11.95 =	5550,000
	572,700 kg	2,290,300 kg		M = 9020,000 kgm

Horizontal bearing pressure on earth.

$r_b = \frac{12M}{t^2} = \frac{12 \times 9020000}{22.8^2} = 208200$ kg per meter strip.

$r_a = \frac{r_b}{3} = 208200 \div 3 = 69400$ " " "

Unit bearing side pressures.
For the earthquake along the ℄ of bridge.

$r_b = 208200 \div 10.6 = 19650$ kg/m²
$r_a = 69400 \div 10.3 = 6740$ "

For the earthquake across the ℄ of bridge.

$r_b = 208200 \div 4.0 = 52050$ kg/m²
$r_a = 69400 \div 3.7 = 18750$ "

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Allowable values of bearing pressures ra and rb.

Upward passive earth pressure allowable.

$$P = w x \frac{1+\sin\phi'}{1-\sin\phi'}$$

where P = upward passive pressure allowable in kg/m²,
w = weight of earth in kg per cub. meter,
x = depth of earth in meters,
$\phi' = \phi - \tan^{-1} \kappa = \phi - \tan^{-1} 0.250$,
ϕ = angle of repose of the earth at normal state, assumed as 30°.

$\phi' = 30° - 14° = 16°$

$\sin\phi' = 0.276$

$\frac{1+\sin\phi'}{1-\sin\phi'} = \frac{1.276}{0.724} = 1.765$

for x = 7.60 meters $r_a = 1600 \times 7.60 \times 1.765 = 214.70$ kg/m²

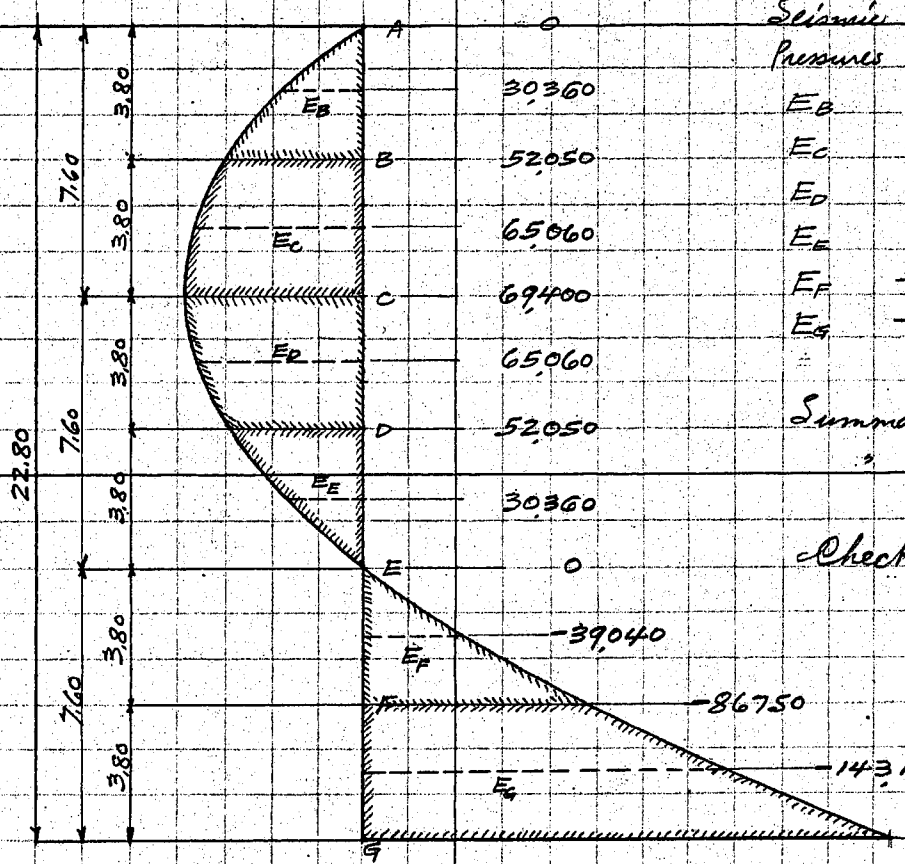
" x = 22.80 " $r_b = 1600 \times 22.80 \times 1.765 = 644.00$ "

The structure is stable during earthquake.

Seismic earth pressure on each section of well.

By Simpson's formula, $\text{area} = \frac{d}{3}[h_1 + 4h_2 + h_3]$ where d = 1.90 meters

$= 0.6333 [h_1 + 4h_2 + h_3]$



	Total pressures per meter strip.	Seismic Pressures	$h_1+4h_2+h_3$	d/3	Pressures	Centre of gravity
A	0					
	30,360	E_B	173,490	× 0.6333 =	109,800 kg	1.27 m
B	52,050	E_C	381,690	× " =	241,700	1.73
	65,060	E_D	381,690	× " =	241,700	2.07
C	69,400	E_E	173,490	× " =	109,800	2.53
	65,060	E_F	− 242,910	× " =	− 153,800	1.26
D	52,050	E_G	− 867,510	× " =	− 549,200	1.66
	30,360					
E	0					
	−39,040					
F	−86,750					
	−143,140					
G	−208,200					

Summary of (+) pressure = 703,000 kg
" " (−) " = − 703,000 "

Check $\frac{2}{3} \times 69,400 \times 15.2 = 703,000$ kg ✓

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CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Moments at several sections of well during earthquake.

Taking moments about center of bottom area O for each sections.

At section A

Load marks.	Hor. forces	Vert. forces		Lever arms		moment abt. O.
D		223,500				
D'	55,900		×	11.79	=	659,000
S		209,500				
S'	52,400		×	6.53	=	342,000
w_A		152,700				
w_A'	38,200		×	0.98	=	37,400
	H_A = 146,500 kg	V_A = 585700 kg		1.775 m		M_A = 1038400 kgm

At section B.

V_A		585,700	×	1.775	=	1,038,400
H_A	146,500		×	3.80	=	556,600
w_B		265,300				
w_B'	66,300		×	1.90	=	126,000
E_B	−109,800		×	1.27	=	−139,500
	H_B = 103,000 kg	V_B = 851,000		1.859 m		M_B = 1,581,500 kgm

At section C.

V_B		851,000	×	1.859	=	1,581,500
H_B	103,000		×	3.80	=	391,500
w_C		269,700			=	
w_C'	67,400		×	1.89	=	127,400
E_C	−241,700		×	1.73	=	−418,000
	H_C = −71,300	V_C = 1,120,700		1.503 m		M_C = 1,682,400 kgm

At section D.

V_C		1,120,700	×	1.503	=	1,682,400
H_C	−71,300		×	3.80	=	−271,000
w_D		262,800				
w_D'	65,700		×	1.90	=	124,800
E_D	−241,700		×	2.07	=	−500,000
	H_D = −247,300	V_D = 1,383,500		0.749		M_D = 1,036,200 kgm

At section E

V_D		1,383,500	×	0.749	=	1,036,200
H_D	−247,300		×	3.80	=	−940,000
w_E		269,700				
w_E'	67,400		×	1.91	=	128,800
E_E	−109,800		×	2.53	=	−278,000
	H_E = −289,700	V_E = 1,653,200		−0.032 m		M_E = −53,000 kgm

At section F

V_E		1,653,200	×	−0.032	=	−53,000
H_E	−289,700		×	3.80	=	−1,101,000
w_F		297,400	×			
w_F'	74,400		×	1.90	=	141,200
E_F	+153,800		×	1.26	=	193,700
	H_F = −61,500	V_F = 1,950,600		−0.420 m		M_F = −819,100 kgm

At section G.

V_F		1,950,600	×	−0.420	=	−819,700
H_F	−61,500		×	3.80	=	−233,800
w_G		339,700				
w_G'	85,000		×	1.87	=	158,900
E_G	549,200		×	1.66	=	911,000
	H_G = 572,700 kg	V_G = 2290,300 kg				M_G = 17,000 kgm

M_G must theoritically be zero.

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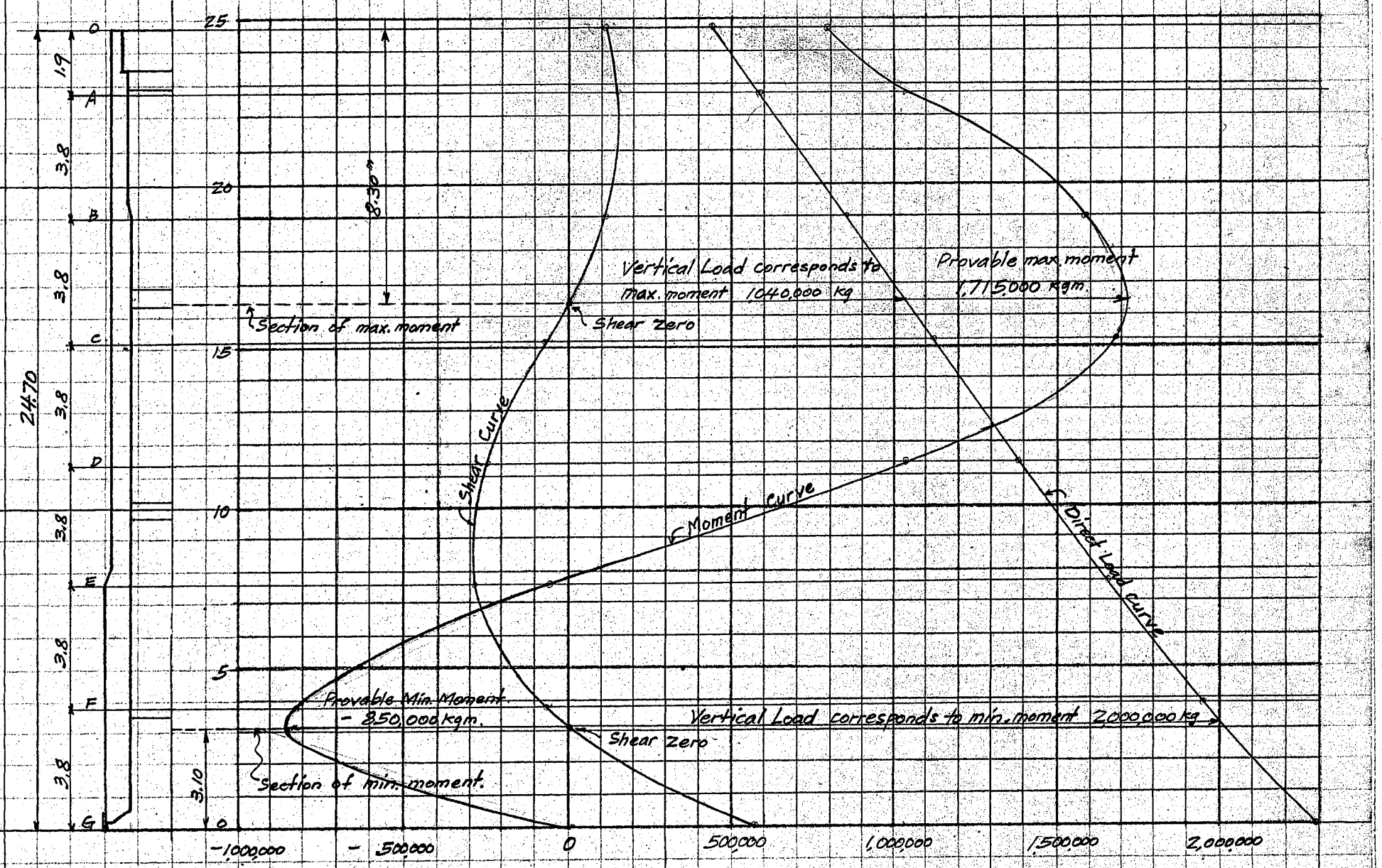
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Moments, shears, and direct load diagram of well. (During earthquake)



Vertical reinforcements in the well.

Maximum moment on well, scaled from the above diagram = 1,715,000 kgm
Corresponding vertical load on well = 1,040,000 kg.

Total net area of well shell = 16.12 m^2 (see on page 16R)

Direct compression on concrete = $\frac{1040000}{16.12 \times 100^2}$ = 6.5 kg/cm² C

Bending stress. Lever arm 2.50+0.60 = 3.10 meters, Effective width of well say 9.0 m

Stress on side walls = $\frac{1715000}{3.10 \times 9.0}$ = 61500 kg C or T per lin meter of side wall.

Unit stress = $\frac{61500}{100 \times 60}$ = 10.3 kg/cm² C or T

Combined stresses

Direct stress	6.5 C	6.5 C
moment stress	10.3 C	10.3 T
	16.8 C or	3.8 T

Vertical reinforcements required = $\frac{3.8 \times 60 \times 100}{1200 \times 1.6}$ = 11.87 cm² per meter of wall.

use 19 mmϕ bars at 40 cm c to c = 2 × 2.835 ÷ 0.40 = 14.18 " " " "

Unit stres in reinforcements.

Tension $\frac{3.8 \times 60 \times 100}{14.18}$ = 1608 kg/cm² < 1200×1.6

Compression 16.8 × 1.5 = 252 " < "

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CALCULATIONS FOR

Akigase - Bashi for Saitama - Ken.

Minimum moment on well scaled from the moment diagram = - 850,000 kgm.
Corresponding vertical load on well = 2,000,000 kg
Assuming the vertical load to be totally carried by the shell,
net area of shell = 19.92 square metres. (see page 17R)

Direct compression on concrete $= \frac{2,000,000}{19.92 \times 100^2} = 10.0$ kg/cm² C

Bending stress. Lever arm = 2.50 + 0.75 = 3.25 metres.
Effective width of well say = 9.20 "

Stress on shell $= \frac{850000}{3.25 \times 9.20} = 28450$ kg C or T per lin meter of side wall.

Unit stress $= \frac{28450}{100 \times 75} = 3.8$ kg/cm² C or T

Combined stresses

Direct compression	10.0 C	10.0 C
moment stress	3.8 C	3.8 T
	13.8 C	6.2 C

For vertical reinforcements, use 19 mmϕ bars at 40 cm c to c on both sides throughout the well.

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MADE BY K. Inaba DATE 8-10-24 FILE NO
CHECKED BY H.T. DATE 8-10-26 PAGE NO 27
(P26R)

CALCULATIONS FOR

Akigase-Bashi for Saitama Ken.

Design of abutment A6 for girder span.

Superimposed loads on abutment:

Dead load:

3870 kg per lin meter, loaded length = 20.1 meters assumed.

3870 × 20.1 ÷ 2 = 38900

Shoes, Electric wiring +o say = 200

39100 kg = D

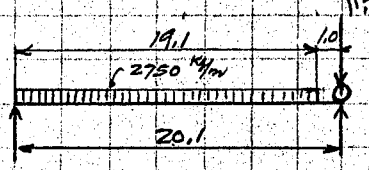
Seismic force 39100 × 0.25 = 9800 kg = D'

Live load:

Uniform live load on roadway

$w = \frac{100,000}{170 + 20.1} = 526$ use 500 kg/m²

5.50 m @ 500 = 2750 kg per lin. meter of span.



6 ton motor truck

Rear wheel concentration = 2250

Impact coef. = $\frac{20}{60+20.1} = 25\%$ 560

2810 kg

For 2 rows of trucks 4 wheels @ 2810 = 11240 kg

Front wheels with imp. say 11240 ÷ 3 = 3750 "

Superimposed live load due to

uniform load $\frac{2750 \times 19.1^2}{2 \times 20.1} = 24960$

rear wheels 11240

36200 kg = L

Summary of superimposed loads on abutment.

At normal state.	For the whole abutment	For one-half of abutment
Dead Load	D = 39100	D = 19600
Live load	L = 36200	L = 18100
	75300 kg	37700 kg
During earthquake.		
Dead Load	D = 39100 kg	D = 19600 kg
Seismic thrust	D' = 9800 kg	D' = 4900 kg

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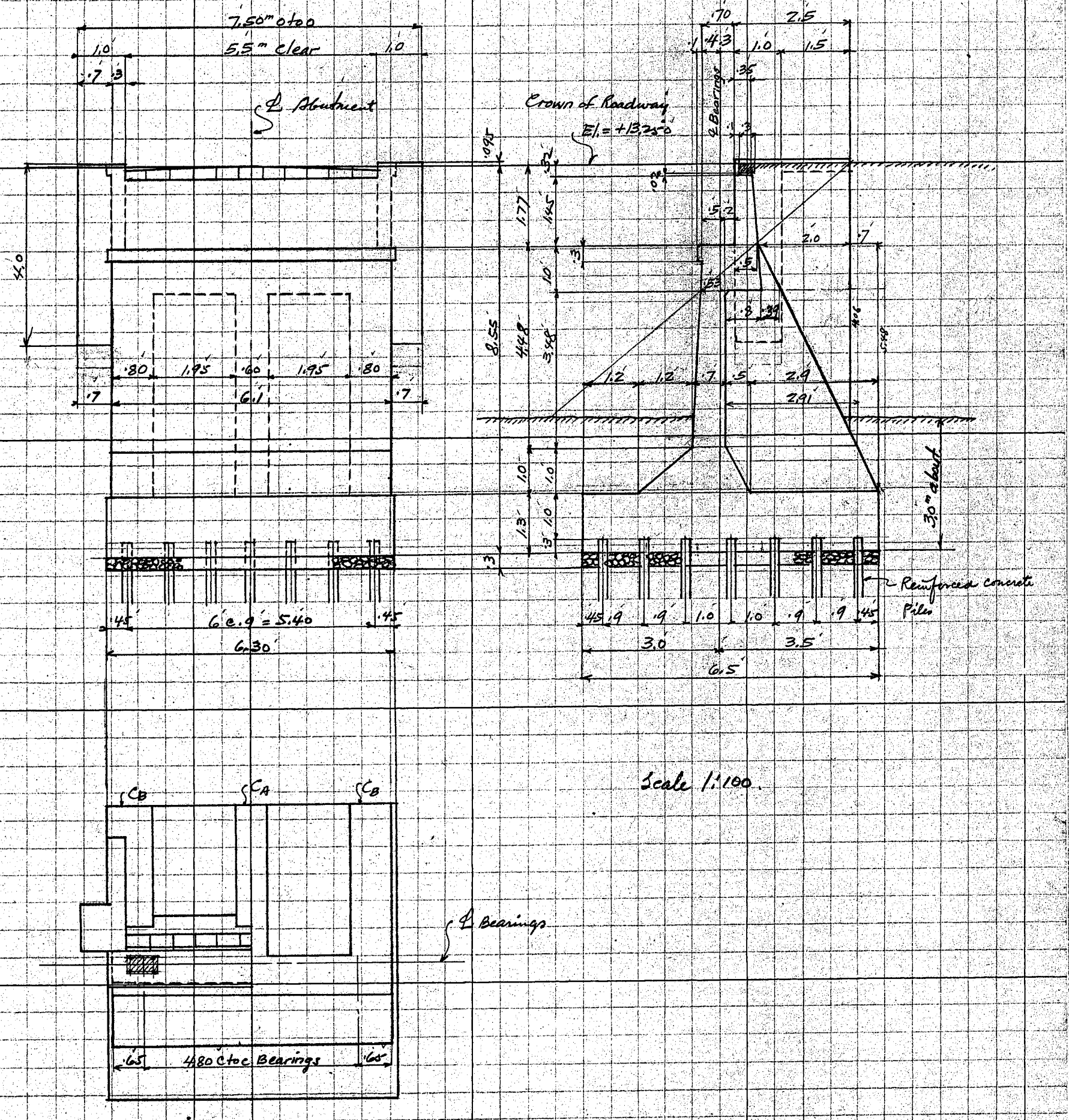
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

General sketch of abutment for girder span. AG.



Scale 1:100.

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MADE BY K. Inaba DATE 8-10-25 FILE NO
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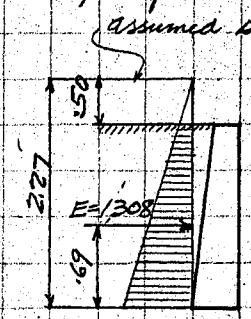
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Design of Parapet wall



Earth pressure at normal state.

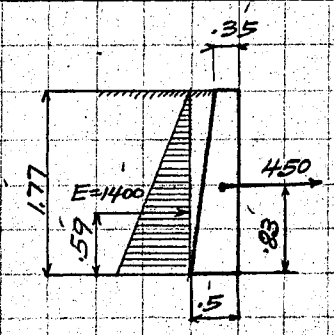
1/3 × 1600 × 0.5 = 267
1/3 × 1600 × 2.27 = 1211
1478 ÷ 2 = 739 kg per sq. meter average.

739 × 1.77 = 1308 kg per meter strip of wall

moment at bottom of wall

1308 × 0.69 = 903 kgm per meter strip of wall.

Shear at bottom of wall = 1308 kg " " "



Earth pressure during earthquake $k = 0.250$,

$E = C \cdot \frac{wh^2}{2}$ From the prepared table $C = 0.659$ for $\phi = 30°$.

$= 0.559 \times \frac{1600 \times 1.77^2}{2} = 1400$ kg. per meter strip of wall.

Seismic force

weight of wall $\frac{.35+.5}{2} \times 1.77 = .752 \text{ @ } 2400 = 1800$ kg

Seismic force = 1800 × .25 = 450 kg per meter of strip.

moment at bottom of wall

1400 × 0.59 = 826
450 × 0.83 = 374
1200 kgm per meter strip of wall.

Shear at bottom of wall

1400 + 450 = 1850 kg " " " "

Stresses at normal state govern the section of wall.

Effective depth required for moment

$d = \sqrt{\frac{M}{bR}}$ where $R = 6.04$ for 1:2.5:5 concrete

$= \sqrt{\frac{903 \times 100}{100 \times 6.04}} = 12.2$ cm. Use 47 cm effective depth with 3 cm insulation.

Steel area required $= \frac{903 \times 100}{1200 \times \frac{7}{8} \times 47} = 1.83$ cm² per meter strip of wall.

Unit shear at normal state $= \frac{1308}{100 \times \frac{7}{8} \times 47} = 0.32$ kg/cm²

Use 12φ bars at 30 cm c to c on rear side.
" 12φ " " 60 cm " " front side.

Design of wing wall.

Cantilever span length = 1.5 meters, total thickness of wall = 30 cm
At depth 2.77 meters from roadway.

Earth pressure on wall at normal state, surcharge being taken as 50 cm.

1/3 × 3.27 × 1600 = 1745 kg/m²

moment on wall = 1/2 × 1745 × 1.5² = 1965 kgm for 1 meter strip at bottom

Shear on wall = 1745 × 1.5 = 2620 kg " " " "

Earth pressure on wall during earthquake

0.559 × 1600 × 3.27 = 2930 kg/m²

Seismic thrust on wall

0.30 × 1.0 @ 2400 × 0.75 = 180 "

3110 "

Seismic moment on wall

= 1/2 × 3110 × 1.5² = 3500 kgm for 1 meter strip at bottom

" Shear " = 3110 × 1.5 = 4660 kg " " "

Seismic stresses govern the section of wall.

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Effective depth required for 1:2.5:5 concrete

$d = \sqrt{\frac{3500 \times 100}{100 \times 6.04 \times 1.6}} = 19$ cm use 27 cm effective depth with 3 cm insulation.

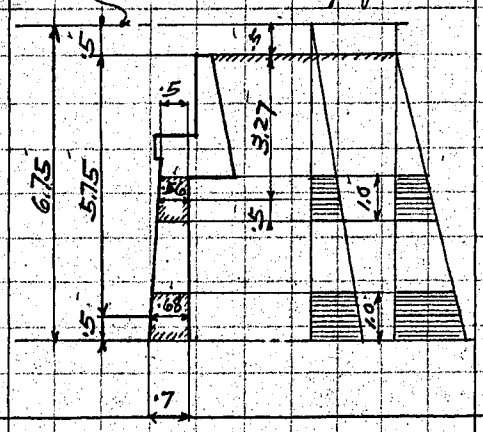
Steel area required $= \frac{3500 \times 100}{1200 \times 1.6 \times \frac{7}{8} \times 27} = 7.72$ cm² for 1 meter strip at bottom.

Use 16 mmφ bars at 25 cm c to c = 8.05 cm² on inside of wall at bottom.
12 mmφ " " 25 " = 4.52 cm² on outside " " "

Design of Curtain wall.

Span length assumed $1.95 + \frac{0.60}{2} + \frac{0.80}{2} = 2.65$ meters.

Stresses at normal state.



Earth pressure on top 1 meter strip

$\frac{1}{3} \times 1600 \times 3.77 = 2010$ kg/m²

Moment $= \frac{1}{10} \times 2010 \times 2.65^2 = 1410$ kgm

Shear $= \frac{1}{2} \times 2010 \times 2.65 = 2660$ kg

Earth pressure on bottom 1 meter strip.

$\frac{1}{3} \times 1600 \times 6.25 = 3335$ kg/m²

Moment $= \frac{1}{10} \times 3335 \times 2.65^2 = 2340$ kgm

Shear $= \frac{1}{2} \times 3335 \times 2.65 = 4420$ kg.

Stresses during earthquake.

Earth pressure and seismic thrust on top 1 meter strip.

0.559 × 1600 × 3.27 = 2930
0.56 × 1.0 C 2400 × 0.25 = 340
3270 kg/m²

Moment $= \frac{1}{10} \times 3270 \times 2.65^2 = 2300$ kgm

Shear $= \frac{1}{2} \times 3270 \times 2.65 = 4330$ kg

Earthpressure and seismic thrust on bottom 1 meter strip.

0.559 × 1600 × 5.75 = 5140
0.68 × 1.0 C 2400 × 0.25 = 410
5550 kg/m²

Moment $= \frac{1}{10} \times 5550 \times 2.65^2 = 3900$ kgm

Shear $= \frac{1}{2} \times 5550 \times 2.65 = 7350$ kg.

Seismic stresses govern the section of curtain wall.

Sections:-

Top 1 meter strip.

Effective depth req'd. $= \sqrt{\frac{2300 \times 100}{100 \times 6.04 \times 1.6}} = 15.4$ cm

use 52.5 cm eff. depth with 3.5 cm insulation.

Steel area required $= \frac{2300 \times 100}{1920 \times \frac{7}{8} \times 52.5} = 2.60$ cm²

use 16 mmφ bars at 70 cm c to c = 2.88 cm².
on both sides.

Unit shear $= \frac{4330}{100 \times \frac{7}{8} \times 52.5} = 0.94$ kg/cm²

Bottom 1 meter strip

Effective depth req'd. $= \sqrt{\frac{3900 \times 100}{100 \times 6.04 \times 1.6}} = 20.1$ cm

use 64.5 cm effective depth with 3.5 cm insulation.

Steel area required $= \frac{3900 \times 100}{1920 \times \frac{7}{8} \times 64.5} = 3.60$ cm²

use 16 mmφ bars at 50 cm c to c = 4.02 cm²

Unit shear $= \frac{7350}{100 \times \frac{7}{8} \times 64.5} = 1.30$ kg/cm²

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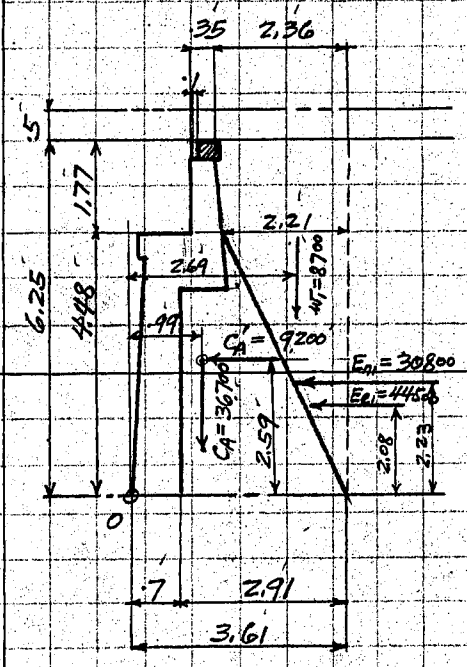
CALCULATIONS FOR

Kigase-Bashi for Saitama-Ken.

Design of Counterfort wall C_A at center.



Weight and center of gravity of counterfort wall C_A, width = 2.55 meters.

		Volume	Weight	arm	Vert. m	arm	Hor. m
Granite	0.30×0.30×2.55 =	0.23 @ 2600 kg =	600 kg	6.10 m	3660	1.15 m	690
parapet wall	0.43×1.47×2.55 =	1.61 c 2400 =	3860	5.17	19960	1.11	4270
top strut	1.27×1.00×2.55 =	3.24 c " =	7780	3.97	30900	0.82	6380
Coping	0.10×0.30×2.55 =	0.08 c " =	190	4.33	820	0.15	30
Curtain wall	0.62×3.48×2.55 =	5.50 c " =	13200	1.66	21900	0.39	5150
Counterfort	0.60×2.10×3.48 =	4.38 c " =	10510	1.51	15870	1.80	18920
" top △	0.20×1.00×0.60 =	0.12 c " =	290	3.81	1110	1.60	460
Misc. concrete say		0.113 c " =	270	2.59	700	0.99	270
Volume of granite		0.23 m³	C_A = 36700 kg	2.59 m	94920	0.99 m	36170
" " concrete		15.04	Seismic force C_A' = 36700×0.25 = 9200 kg				

Weight and center of gravity of earth on counterfort wall.

2.29×1.77×0.60 =	2.43 @ 1600 =	3900	2.46	9600	
1.11×4.48×0.60 =	2.99 c " =	4800	2.87	13800	
	W_1 =	8700	2.69 m	23400	

Earth pressure at normal state on near side.

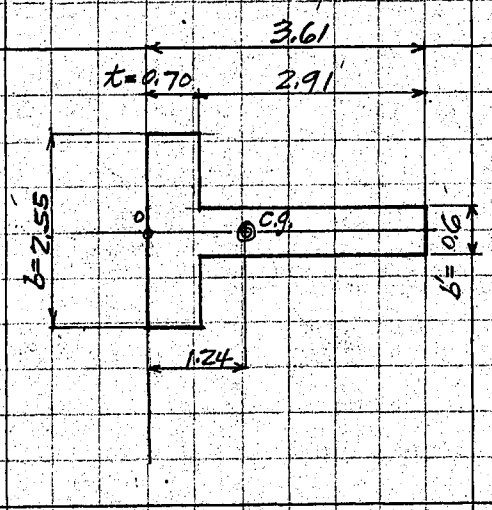
1/3×1600×0.50 = 267
1/3×1600×6.75 = 3600
3867÷2 = 1934 kg/m² average.

Earth pressure E_{n1} = 1934×6.25×2.55 = 30800 kg arm = 2.23 m

Seismic earth pressure on near side.

E_{e1} = 0.559×1600×6.25²÷2×2.55 = 44600 kg arm = 2.08 m

Stresses in counterfort wall C_A at normal state.



Center of gravity of bottom section.

2.55×0.70 = 1.785	× 0.35 =	0.625	
2.91×0.60 = 1.745	× 2.155 =	3.760	
3.530 m²	1.24 m	4.385	

Taking moment about point O.

Load marks	Hor. forces	Vert. forces	Lev. arms	Moment about O.
C_A		36700 ×	−0.99 =	−36300
W_1		8700 ×	−2.69 =	−23400
E_{n1}	30800	×	2.23 =	68700
	30800 kg	45400 kg	0.20 m	9000

Eccentricity = 1.24 + 0.20 = 1.44 meters
Moment on bottom section = 45400×1.44 = 65400 kgm
Shear " " = 30800 kg

Stresses in counterfort wall during earthquake.

Seismic forces toward river side.

Taking moment about point O.

Load marks	Hor. forces	Vert. forces	Lev. arms	Moment abt O
C_A		36700 ×	−0.99 =	−36300
C_A'	9200	×	2.59 =	23800
W_1		8700 ×	−2.69 =	−23400
E_{e1}	44500	×	2.08 =	92500
	53700 kg	45400 kg	1.25 m	56600

Eccentricity = 1.24 + 1.25 = 2.49 m
Moment on bottom section = 45400×2.49 = 113000 kgm
Shear " " = 53700 kg

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CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Seismic forces toward shore side.

Taking moment about point o.

Load marks	Hor. forces	Vert. forces	Rev. arm	Moment abt. o.
C_A		36,700 ×	− 0.99 =	− 36,300
C_A'	− 9200	×	2.59 =	− 23,800
W_1		8700 ×	− 2.69 =	− 23,400
	− 9200 kg	45400 kg	− 1.84	− 83,500

Eccentricity = 1.24 − 1.84 = −0.60 m.

Moment at bottom section = 45400 × (−0.6) = − 27,300 kgm.

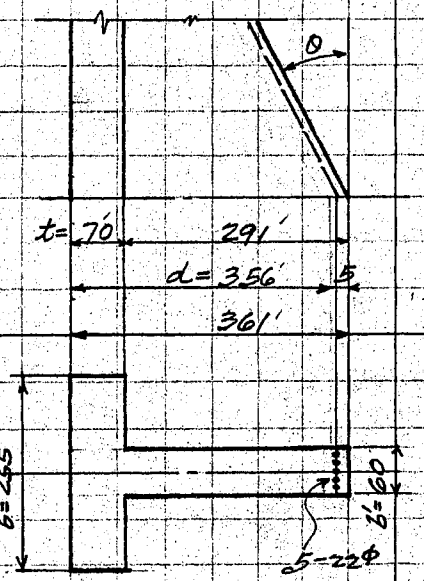
Shear " " = − 9200 kg

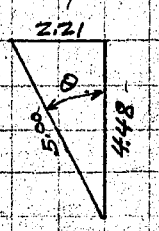
Summary of Governing Stresses:-

			Corresponding vertical load	Corresponding shear.
positive moment	113,000	kgm during earthquake	45,400 kg	53,700 kg
negative moment	− 27,300	" "	45,400 "	− 9,200 "
max. shear	53,700	kg "		

Reinforcements in Counterfort wall C_A.

Positive moment.





Let the angle subtended by the rear face of counterfort with the vertical = θ

$\sqrt{2.21^2+4.48^2} = 5.00$

$\cos\theta = 4.48 \div 5.00 = 0.896$

$\tan\theta = 2.21 \div 4.48 = 0.493$

Effective depth = 361 − 5 = 356 cm = d.

$d\cos\theta = 356 \times 0.896 = 319$ cm

Steel area required for moment $= \frac{M}{f_s j d\cos\theta} = \frac{113000\times100}{1920\times\frac{7}{8}\times319} = 21.09$ cm²

Use 5 − 22 mm φ bars = 19.01 cm² = A_s

Steel ratio $p = \frac{19.01}{255\times356} = 0.00021$, $\frac{t}{d} = \frac{70}{356} = 0.197$

" $p_1 = p\cos\theta = 0.00021\times0.896 = 0.000188$

Neutral axis in the flange.

$k = \sqrt{2p_1 n + (p_1 n)^2} - p_1 n = \sqrt{2\times0.000188\times15 + (0.000188\times15)^2} - 0.000188\times15 = 0.0724$

$j = 1 - \frac{k}{3} = 1 - \frac{0.0724}{3} = 0.976$

$f_s = \frac{M}{A_s j d\cos\theta} = \frac{113000\times100}{19.01\times0.976\times319} = 1910$ kg/cm²

$f_c = \frac{f_s k}{n(1-k)} = \frac{1910\times0.0724}{15(1-0.0724)} = 9.94$

Direct compression $= \frac{45400}{35300} = \frac{1.29}{11.23}$ kg/cm² (45400 − 9276 = 35300)

Unit shear $= \frac{V_1}{b' j d}$ where $V_1 = V - \frac{M}{d}\tan\theta$

$= \frac{38000}{60\times0.976\times356}$ $= 53700 - \frac{113000\times100}{356}\times0.493$ (113000×100 − 15700)

$= 1.82$ kg/cm² $= 38000$ kg

b' = width of stem = 60 cm.

Unit bond $= \frac{V_1}{\Sigma o j d} = \frac{38000}{6.91\times5\times0.976\times356}$

$= 3.16$ kg/cm²

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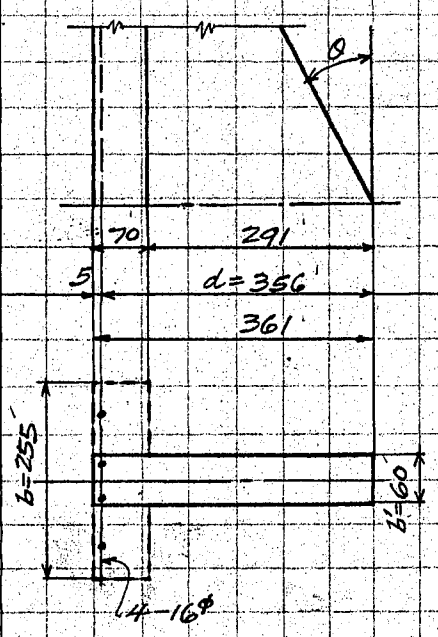
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Negative moment.



Steel area required $= \frac{27300 \times 100}{1920 \times \frac{7}{8} \times 356} = 4.56\ cm^2$

Use 4-16 mmφ bars $= 8.04\ cm^2$

Steel ratio $p = \frac{8.04}{60 \times 356} = 0.000377$

" $p_1 = \frac{p}{\cos^2\theta} = \frac{0.000377}{0.896^2} = 0.00047$

$k = \sqrt{2 \times 0.00047 \times 15 + (0.00047 \times 15)^2} - 0.00047 \times 15 = 0.112$

$j = 1.0 - \frac{0.112}{3} = 0.963$

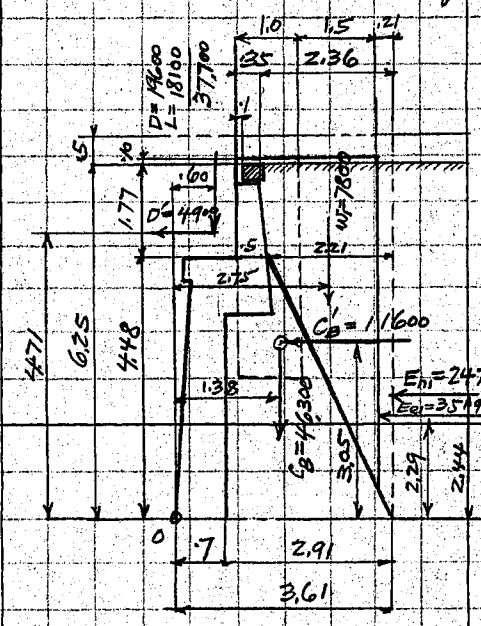
$f_s = \frac{M}{A_s j d} = \frac{27300 \times 100}{8.04 \times 0.963 \times 356} = 990\ kg/cm^2$

$f_c = \frac{f_s k}{n(1-k)} = \frac{990 \times 0.112}{15(1-0.112)} = 8.33$

Direct compression $= \frac{45400}{35300} = 1.29$

$9.62\ kg/cm^2$

Design of Counterfort wall at both ends C_B width = 1.78 meters



		volume		weight	arm	vert m	arm	hor. m
End pedestal	0.80×0.80×1.10 =	0.70	@2200 =	1540	6.90	10630	1.40	2160
granite	0.30×0.30×1.48 =	0.133	@2600 =	350	6.10	2140	1.15	400
parapet wall	0.43×1.47×1.78 =	1.13	@2400 =	2710	5.17	14000	1.11	3010
wing wall	0.30×1.87×2.08 =	1.17	@ " =	2810	5.43	15250	2.36	6630
" "	0.30×1.00×4.06 =	1.22	@ " =	2930	3.13	9170	2.73	8000
" coping	0.30×0.10×1.50 =	0.05	@ " =	120	6.20	740	2.65	320
top strut	1.27×1.00×1.78 =	2.26	@ " =	5420	3.97	21520	0.82	4450
" coping front	0.10×0.30×1.78 =	0.05	@ " =	120	4.33	520	0.15	20
" " side	0.10×0.30×0.80 =	0.02	@ " =	50	4.33	220	0.50	30
Curtain wall	0.62×3.48×1.78 =	3.84	@ " =	9210	1.66	15300	0.38	3500
Counterfort	2.05×3.48×0.80 =	5.71	@ " =	13710	1.50	20560	1.81	24840
"	0.20×1.00×0.80 =	0.16	@ " =	380	3.81	1450	1.60	610
projection	1.00×0.70×4.00 =	2.80	@ " =	6720	4.35	29200	1.40	9410
		0.096	@ "	230	3.05	700	1.38	320
Inside concrete say		0.133	C_B =	46300 kg	3.05 m	141400	1.38 m	63600
Volume of granite			C_B' = 46300×0.25 =			11600 kg		
" " concrete								

Weight and center of gravity of earth on counterfort wall.

	2.29×1.77×0.80 = 3.24	@1600 =	5180	× 2.46	12750
	1.11×4.48×0.80 = 3.98	@ " =	6370	2.87	18300
wing, less	0.30×2.08×1.77 = 1.11	@ " =	-1770	2.36	-4180
" "	0.30×1.00×4.06 = 1.22	@ " =	-1950	2.73	-5320
			7830	2.75 m	21550

Call this 7800 kg = W_1

Earth pressure at normal state on rear side. (See on page 31R)

1934 × 6.25 × 1.78 = 21500 × 2.23 = 48000

on pedestal base. $\frac{1}{3} \times 1600 \times 0.5 = 267$

$\frac{1}{3} \times 1600 \times 3.9 = 2080$

$2347 \div 2 = 1174$

$1174 \times 3.90 \times 0.70 = \frac{3200}{24700\ kg} \times \frac{3.80}{2.44\ m} = \frac{12150}{60150}$

E_{H1} =

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CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Seismic earthpressure on rear side.

E_{e1} $0.559 \times 1600 \times 6.25^2 \div 2 \times 1.78 = 31100 \times 2.08 = 64700$

on pedestal base $0.559 \times 1600 \times 3.9^2 \div 2 \times 0.70 = 4800 \times 3.625 = 17500$

$E_{e1} = 35900^{kg}$ 2.29^{m} 82200

Stresses in Counterfort wall C_B at normal state.

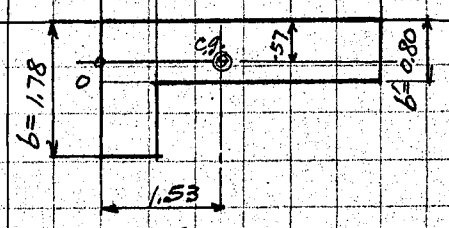
Center of gravity of bottom area

$1.78 \times 0.70 = 1.245 \times 0.35 = 0.436$

$2.91 \times 0.80 = 2.330 \times 2.155 = 5.018$

3.575^{m^2} 1.53^{m} 5.454

or $35750\ cm^2$



Taking moment about point O.

Load marks	Hor. forces	Vert. forces	Lev. arms	moment abt. O.
D+L		37700 ×	−0.60 =	−22600
C_B		46300 ×	−1.38 =	−63900
w_1		7800 ×	−2.75 =	−21450
E_{e1}	24700 ×		2.44 =	60300
	24700 kg	91800 kg	−0.52 m	−47650

Eccentricity = 1.53 − 0.52 = 1.01 meters

moment at bottom section = 91800 × 1.01 = 92800 kgm

Shear " " " = 24700 kg

Stresses during earthquake. Seismic force toward river side.

D		19600 ×	−0.60 =	−11760
D′	4900	×	4.71 =	23100
C_B		46300 ×	−1.38 =	−63900
C_B'	11600	×	3.05 =	35360
w_1		7800 ×	−2.75 =	−21450
E_{e1}	35900 ×		2.29 =	82200
	52400 kg	73700 kg	0.59 m	43550

Eccentricity = 1.53 + 0.59 = 2.12 meters

moment on bottom section = 73700 × 2.12 = 156200 kgm

Shear " " " = 52400 kg

Seismic force toward shore side.

D		19600 ×	−0.60 =	−11760
D′	−4900	×	4.71 =	−23100
C_B		46300 ×	−1.38 =	−63900
C_B'	−11600	×	3.05 =	−35360
w_1		7800 ×	−2.75 =	−21450
	−16500 kg	73700 kg	−2.11 m	−155570

Eccentricity = 1.53 − 2.11 = −0.58 meter.

moment on bottom section = −73700 × 0.58 = −42800 kgm

Shear " " " = −16500 kg

Summary of Governing Stresses:-

		Corresponding vert. load	Corresponding shear
Positive moment	156200 kgm during earthquake	73700 kg	52400 kg
Negative "	−42800 kg " "	73700	−16500
max. shear	52400 kg " "		

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CALCULATIONS FOR

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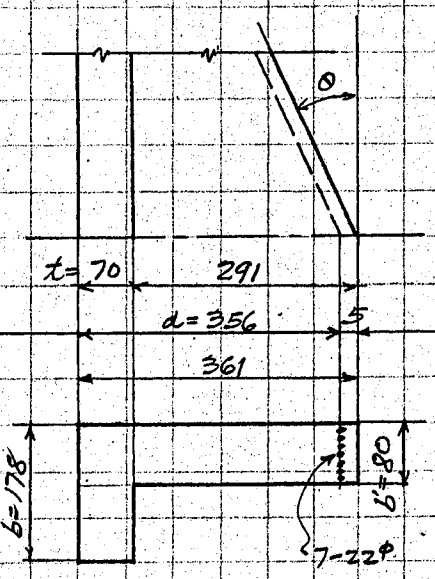
Reinforcements in counterfort wall C_B.

Positive moment.



Angle θ same as for C_A see on page 32R.

$\cos\theta = 0.896$

$\tan\theta = 0.493$

Effective depth $= 361 - 5 = 356\ cm = d$.

$d\cos\theta = 356 \times 0.896 = 319\ cm$

$$\text{Steel area req'd. for moment} = \frac{156200 \times 100}{1920 \times \frac{7}{8} \times 319} = 29.15\ cm^2$$

Use 7-22φ bars $= 26.61\ cm^2$

$$\text{Steel ratio } p = \frac{26.61}{178 \times 356} = 0.00042, \quad t/d = \frac{70}{356} = 0.197$$

$p_1 = p\cos\theta = 0.00042 \times 0.896 = 0.000376$

neutral axis in the flange.

$$k = \sqrt{2 \times 0.000376 \times 15 + (0.000376 \times 15)^2} - 0.000376 \times 15 = 0.101$$

$$j' = 1 - \frac{0.101}{3} = 0.966$$

$$f_s = \frac{156200 \times 100}{26.61 \times 0.966 \times 319} = 1904\ kg/cm^2$$

$$f_c = \frac{1904 \times 0.101}{15(1 - 0.101)} = 14.23$$

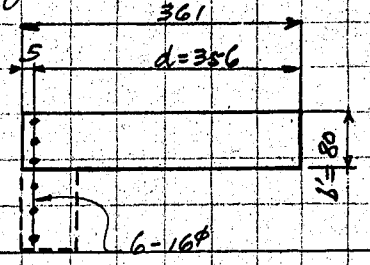
$$\text{Direct compression} = \frac{73700}{35750} = \frac{2.06}{16.31}\ kg/cm^2$$

$$V_1 = 52400 - \frac{156200 \times 100}{356} \times 0.493 = 30800\ kg$$

$$\text{Unit shear } v = \frac{30800}{80 \times 0.966 \times 356} = 1.12\ kg/cm^2$$

$$\text{Unit bond} = \frac{30800}{6.91 \times 7 \times 0.966 \times 356} = 1.85$$

Negative moment.



$$\text{Steel area req'd. for moment} = \frac{42800 \times 100}{1920 \times \frac{7}{8} \times 356} = 7.16\ cm^2$$

Use 6-16φ bars $= 12.07\ cm^2$

$$\text{Steel ratio } p = \frac{12.07}{80 \times 356} = 0.000424, \quad p_1 = \frac{0.000424}{0.896^2} = 0.000528$$

$$k = \sqrt{2 \times 0.000528 \times 15 + (0.000528 \times 15)^2} - 0.000528 \times 15 = 0.118$$

$$j' = 1 - \frac{.118}{3} = 0.961$$

$$f_s = \frac{42800 \times 100}{12.07 \times 0.961 \times 356} = 1037\ kg/cm^2$$

$$f_c = \frac{1037 \times 0.118}{15(1 - 0.118)} = 9.35$$

$$\text{Direct comp.} = \frac{73700}{35750} = \frac{2.06}{11.31}\ kg/cm^2$$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CHECKED BY H. T. DATE 8-11-8 PAGE NO 36R

CALCULATIONS FOR

Akigase Bashi for Daitama-ken

Stability of Abutment.

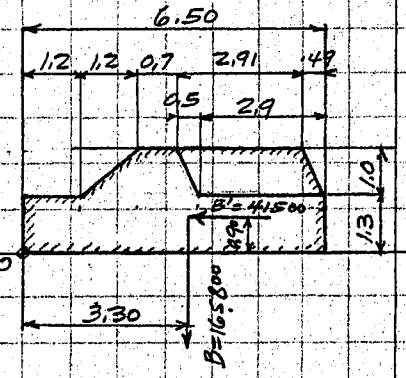
Superimposed loads on abutment:

Dead load D = 39,100 kg
Live load L = 36,200 kg
$D+L$ = 75,300 kg

Seismic force:
D' = 9,800 kg

Weight and center of gravity of shaft.

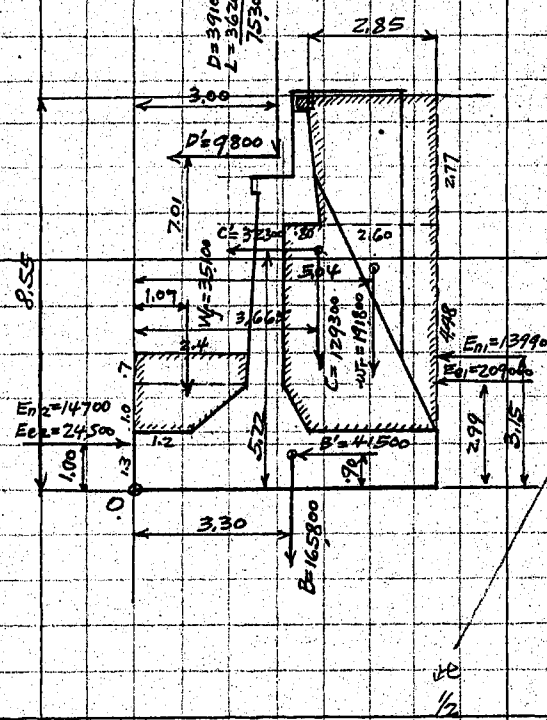
		arm	vert. mo.	arm	hor. mo.
Counterfort C_A	36,700	2.59	94,920	0.99	36,170
$2C_B$	92,600	3.05	282,800	1.38	127,400
C = 129,300 kg		2.92 m	377,720	1.265 m	163,570
(C' = 32,300)		2.30		2.400	
Arms from toe, 0		5.22 m		3.665 m	



Weight and center of gravity of base.

Base	1.30×6.50×6.30 = 53.20	@2400 =	127,700	0.65	82,900	3.25	414,500
wall base	1.55×1.00×6.10 = 9.45	c " =	22,700	1.71	38,800	2.54	57,600
Counterfort	1.00×2.91×2.20 = 6.40	c " =	15,400	1.80	27,700	4.80	73,900
	69.05	B = 165,800 kg		0.90 m	149,400	3.30 m	546,000
		B' = 41,500 kg					

Weight and center of gravity of earth on rear footing.



	2.73×2.77×6.30 = 47.60	@1600 =	76,200	7.18	547,000	5.13	391,000
	3.40×4.48×6.30 = 96.00	@1600 =	153,500	3.54	543,000	4.80	737,000
fillet less ½	0.50×1.00×3.90 = - 1.95	c " = -	3,120	1.63	- 5,100	3.27	- 10,200
wing "	2×0.30×1.87×2.08 = - 2.34	c " = -	3,750	7.73	- 29,000	4.76	- 17,900
" "	2×0.30×1.00×4.06 = - 2.44	c " = -	3,900	5.43	- 21,200	5.13	- 20,000
" "	2×0.30×0.10×1.50 = - 0.09	c " = -	140	8.50	- 1,200	5.05	- 700
" "	2×0.10×1.00×3.90 = - 0.78	c " = -	1,250	6.60	- 8,300	3.80	- 4,800
Counterfort	2.05×3.48×2.20 = - 15.70	c " = -	25,100	3.80	- 95,400	4.19	- 105,200
△	0.20×1.00×2.20 = - 0.44	c " = -	700	6.11	- 4,300	4.00	- 2,800
	119.90		191,740	4.83 m	925,500	5.04 m	966,400

Call this W_r = 191,800 kg

Weight and center of gravity of earth on front footing.

	2.40×0.70×6.30 = 10.58	@1600 =	17,000	2.65	45,000	1.20	20,400
此½	1.80×1.00×6.30 = 11.32	c " =	18,100	1.85	33,500	0.94	17,000
	21.90	W_f = 35,100 kg		2.24 m	78,500	1.07	37,400

付け訂正せず 脱漏セルを誤差修正

Earth pressure at normal state

On rear side. Total depth of earth = 8.55 m with 1.50 m surcharge for L.L.

1/3×1600×0.50 = 267
1/3×1600×9.05 = 4835
5092 ÷ 2 = 2546 kg/m² average

1/3×1600×0.5 = 267
1/3×1600×4.4 = 2347
2614 ÷ 2 = 1307 kg/m² average

Total pressure

2546×8.55×6.10 = 132,800 × 3.00 = 398,000
2×1307×3.90×0.70 = 7100 × 6.07 = 43,000
E_{n1} = 139,900 kg 3.15 m 441,000

On front side, total depth of earth = 3.00 m

$E_{n2} = \frac{1}{6} \times 1600 \times 3.00^2 \times 6.1$ = 14,700 kg arm 1.00 meter

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Seismic earth pressure.

On rear side.

$1/2 \times 0.559 \times 1600 \times 8.55^2 \times 6.10 = 199{,}500 \times 2.85 = 568{,}000$

$1/2 \times 0.559 \times 1600 \times 3.90^2 \times 1.40 = 9500 \times \frac{5.95}{2.99^m} = 56{,}500$

$E_{e1} = 209{,}000\ kg \qquad 624{,}500$

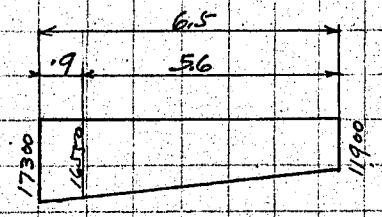
On front side

$1/2 \times 0.559 \times 1600 \times 3.00^2 \times 6.1 = E_{e2} = 24{,}500\ kg$ arm 1.00 meter.

Stability of Abutment at normal state.

Taking moment about toe 0.

Load marks	Hor. forces	Vert. forces	Lev. arms	Moments
D+L		75,300	× − 3.00 =	− 226,000
C		129,300	× − 3.665 =	− 474,000
W_T		191,800	× − 5.04 =	− 966,000
W_f		35,100	× − 1.07 =	− 37,500
E_{e1}	139,900		× 3,15 =	441,000
E_{e2}	− 14,700		× 1.00 =	− 14,700
B		165800	× − 3.30 =	− 547,000
	125,200 kg	597,300 kg	− 3,05 m	− 1,824,200



Eccentricity = 3.25 − 3.05 = 0.20 meter

Resultant force within middle third

max. toe pressure = $\frac{597300}{6.3 \times 6.5}\left(1 \pm \frac{6 \times 0.20}{6.5}\right) = 17{,}300\ kg/m^2$ (1.58 tons/ft²) or 11900

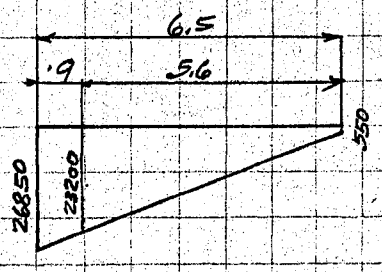
max. load on one pile = 16550 × 0.90 × 0.90 = 13400 kg.

Stability of abutment during earthquake.

Seismic force toward river side.

Taking moment about toe 0.

D		39,100	× − 3.00 =	− 117,300
D'	9800		× 7.01 =	68,500
C		129,300	× − 3.665 =	− 474,000
C'	32,300		× 5.22 =	168,700
W_T		191,800	× − 5.04 =	− 966,000
W_f		35,100	× − 1.07 =	− 37,500
B		165,800	× − 3.30 =	− 547,000
B'	41,500		× 0.90 =	37,400
E_{e1}	209000		× 2.99 =	625000
	292,600 kg	561,100 kg	− 2.21 m	− 1,242,200



Eccentricity = 3.25 − 2.21 = 1.04 meters

Resultant force within middle third .96

max. bearing pressure = $\frac{561100}{6.3 \times 6.5}\left(1 \pm \frac{6 \times 1.04}{6.5}\right) = 26850\ kg/m^2$ (or 2.45 tons/ft²) or 550

max. load on one pile = 23200 × .9 × .9 = 18800 kg

増田淳氏関係資料
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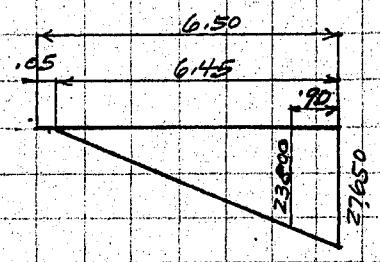
CALCULATIONS FOR

Akigase-Bashi for Saitama-ken

Taking moment about toe o.

Load marks	Hor. forces	Vert. forces		Lev. arms		Moments
D		39,100	×	− 3.00	=	− 117,300
D'	− 9,800		×	7.01	=	− 68,500
C		129,300	×	− 3.665	=	− 474,000
C'	− 32,300		×	5.22	=	− 168,700
W1		191,800	×	− 5.04	=	− 966,000
W2		35,100	×	− 1.07	=	− 37,500
B		165,800	×	− 3.30	=	− 547,000
B'	− 41,500		×	0.90	=	− 37,400
Ee2	− 24,500		×	1.00	=	− 24,500
	− 108,100 kg	561,100 kg		− 4.35 m		− 2,440,900



Eccentricity = 3.25 − 4.35 = 1.10 meter

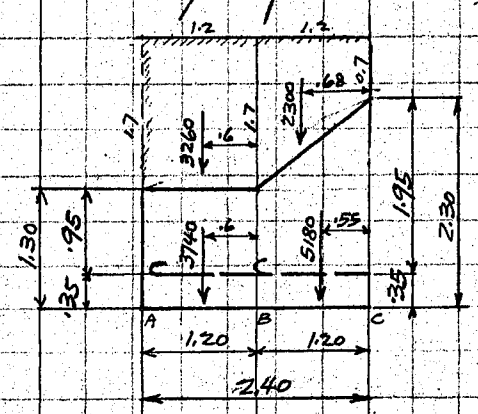
Resultant force outside of middle third. Neglecting tension,

pressure area = (6.50 − 4.35) × 3 = 6.45 × 6.30 = 40.60 m²

max. bearing pressure = $\frac{561100 \times 2}{40.60}$ = 27650 kg/m² (≈ 2.53 tons/m²)

max. load on one pile = 23800 × 0.9 × 0.9 = 19300 kg

Design of Cantilever footing at toe.



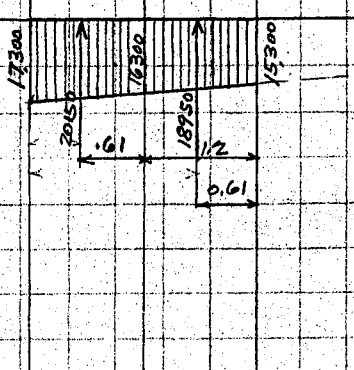
At normal state.

Upward pressure

on AB.	16800 × 1.20 =	20150 kg	× arm	0.61
on BC	15800 × 1.20 =	18950 kg	"	0.61

Downward pressure

on AB. concrete	1.3 × 1.2 @ 2400 =	3740 kg	"	0.60
earth	1.7 × 1.2 @ 1600 =	3260 "	"	0.60
on BC concrete	1.8 × 1.2 @ 2400 =	5180 "	"	0.55
earth	1.2 × 1.2 @ 1600 =	2300 "	"	0.68

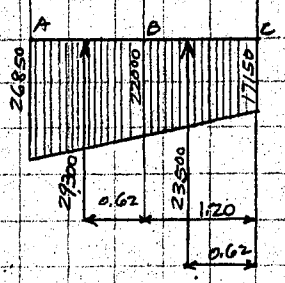


Moment and shear at B.

	20150 × 0.61	=	12300	
− 7000	− 3740 × 0.60	=	− 2250	− 4210
	− 3260 × 0.60	=	− 1960	
	13150 kg		8090 kgm	

Moment and shear at C.

	20150 × 1.81	=	36500	
	18950 × 0.61	=	11550	
	− 3740 × 1.80	=	− 6730	− 17010 Resultant arm 1.175 m
	− 3260 × 1.80	=	− 5860	
− 14480	− 5180 × 0.55	=	− 2850	
	− 2300 × 0.68	=	− 1570	
	24620 kg		31040 kgm	



During earthquake.

Upward pressure

on AB	24430 × 1.2 =	29300 kg	Arm	0.62
on BC	19580 × 1.2 =	23500 "	"	0.62

Moment and shear at B

	29300 × 0.62	=	18160	
− 7000	× 0.60	=	− 4210	
	22300 kg		13950 kgm	

増田淳氏関係資料
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CALCULATIONS FOR

Akigase-Bashi for Saitama-ken

moment and shear at D

29300	× 1.82	=	53300	
23500	× 0.62	=	14560	
−14480	× 1.175	=	−17010	
38320 kg			50850 kgm	

Summary of governing stresses

at point B. positive moment 13950 kgm (seismic)
negative " say −4210 " (")
max. shear 22,300 kg (")

at point C. positive moment 50850 kgm (seismic)
negative " −17010
upward pressure 12350 × .78 = 9630
−7380 kgm (seismic)
max. shear = 24620 kg (normal state)

Steel area required = $\frac{13950 \times 100}{1920 \times \frac{7}{8} \times 95}$ = 8.73 cm² per meter strip at B.

= $\frac{50850 \times 100}{1920 \times \frac{7}{8} \times 195}$ = 15.52 " " " at C

Use 22φ bars at 14.0 cm c to c = 9.50 cm² at B Bottom
22φ " " 20 cm " = 19.00 " at C "

Steel area required for negative moment.

= $\frac{4210 \times 100}{1920 \times \frac{7}{8} \times 95}$ = 2.63 cm² at B

= $\frac{7380 \times 100}{1920 \times \frac{7}{8} \times 195}$ = 2.26 " " C

Use 16φ bars at 60 cm c to c = 3.33 cm² at B and C top.

Unit shear.

at B, $v = \frac{22300}{100 \times \frac{7}{8} \times 95}$ = 2.69 kg/cm²

at C $v = \frac{24620}{100 \times \frac{7}{8} \times 195}$ = 1.44 "

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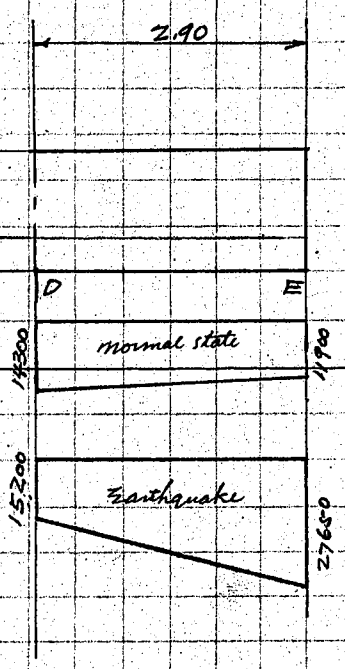
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Design of Rear footing.



Downward pressure

Concrete	1.30 @ 2400 =	3120
earth	7.25 @ 1600 =	11,600
		14720 kg/m

Pressure on rear footing for one meter strip.

	normal state	earthquake
at E.		
Upward pressure	11,900	27650
Downward "	− 14720	− 14720
	− 2,820 kg/m	12930 kg/m
at D.		
Upward pressure	14300	15,200
Downward "	− 14720	− 14720
	− 420 kg/m	+ 480 kg/m

Span length of rear footing assumed as 2.65 meters.

Moment on beam at E for 1 meter width

$m = 1/10 \times 12930 \times 2.65^2 = 9090$ kgm.

$Shear = 1/2 \times 12930 \times 2.65 = 17,150$ kg

$Steel\ required = \frac{9090 \times 100}{1920 \times \frac{7}{8} \times 95} = 5.70$ cm²

for 16φ bars, required spacing = 35 cm c to c. for top and bottom.

$Unit\ shear = \frac{17150}{100 \times \frac{7}{8} \times 95} = 2.06$ kg/cm²

Moment on beam at D for 1 meter width.

$m = 1/10 \times 480 \times 2.65^2 = 340$ kgm

$Steel\ required = \frac{340 \times 100}{1920 \times \frac{7}{8} \times 95} = 0.21$ cm²

for 16φ bars, required spacing = 1000 cm c to c.

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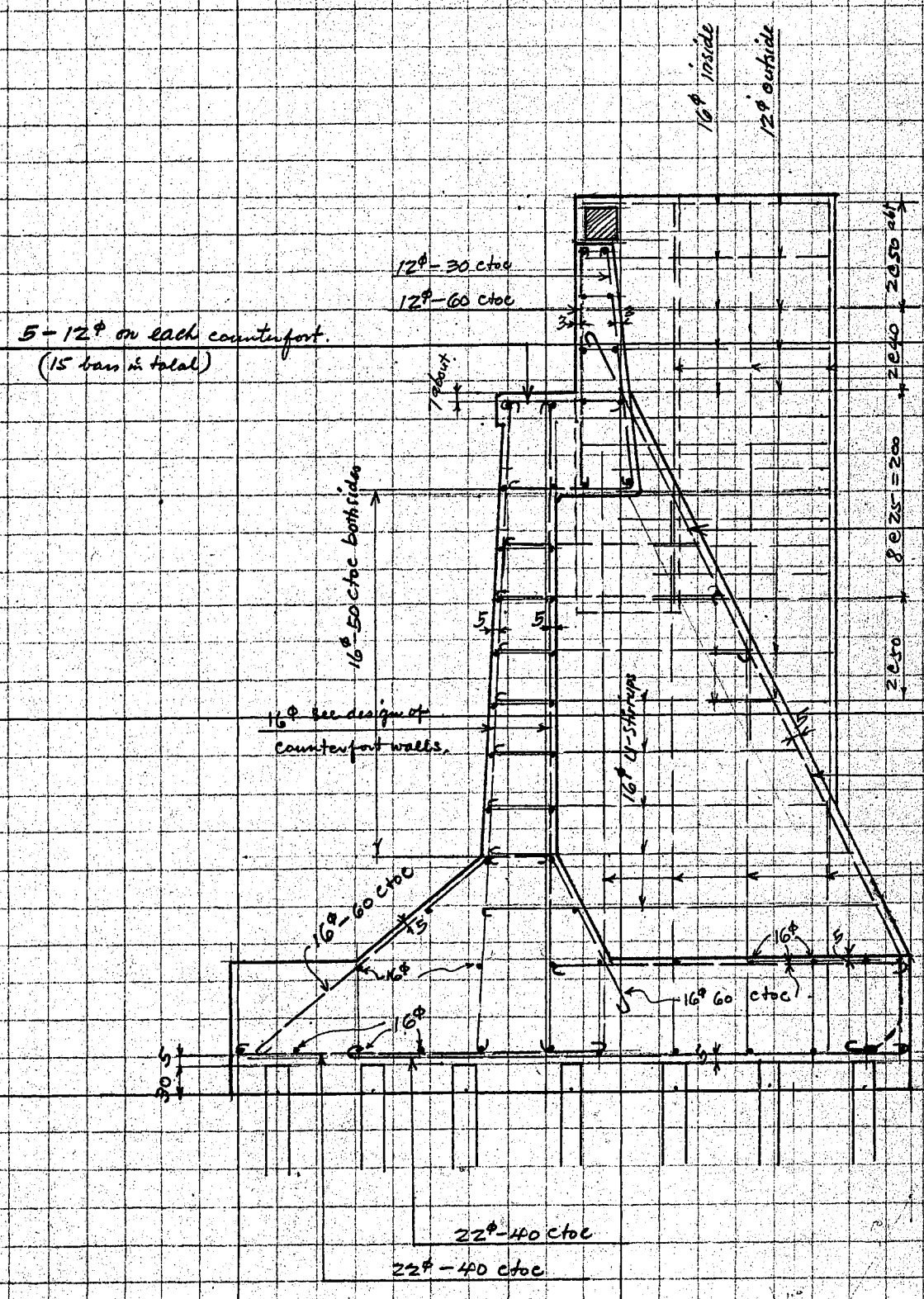
CALCULATIONS FOR

Akigase-Bashi for Saitama-ken

General arrangements of Reinforcements in Abutment.



Scale 1:60

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CALCULATIONS FOR

埼玉県道浦和所沢線
秋ヶ瀬橋下部工事変更設計書

鋼鈑桁橋脚 P60 及 P61
鋼構桁橋脚 P62 P63 及 P64

昭和九年四月

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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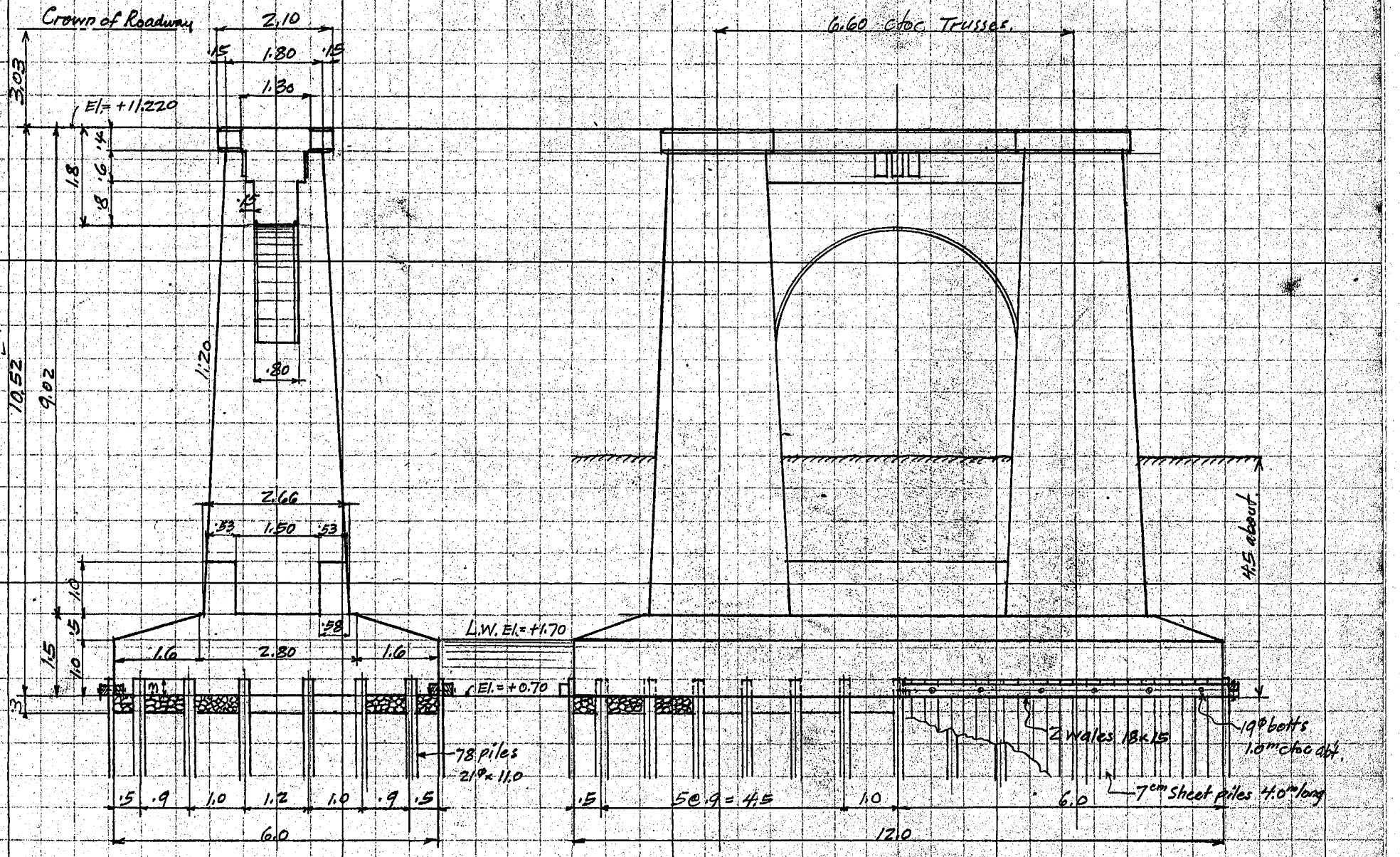
CHECKED BY M. J. DATE 9-2-24 PAGE NO 10R

CALCULATIONS FOR

Akigase - Bashi for Saitama - ken.

General Sketch of Pier P12 for Truss span



Scale 1:100.

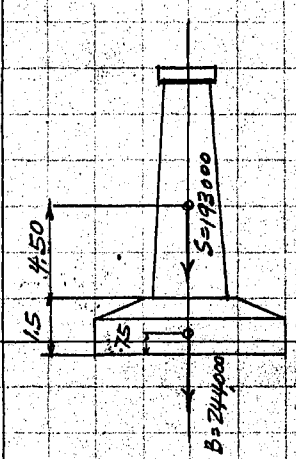
Weight and Center of gravity of Pier:

				Volume of Concrete		Lev. arm from bottom		
Pier shaft.	Coping	2.10φ × 0.40 × 2	=	2.77	×	8.82	=	24.42
"		1.30 × 0.40 × 4.70	=	2.45	×	8.82	=	21.60
	Strut	1.10 × 0.60 × 4.90	=	3.23	×	8.32	=	26.88
"		0.80 × 1.20 × 4.70	=	4.51	×	7.42	=	33.50
	Shafts	2.73φ × 8.62 × 2	=	67.35	×	3.78	=	254.50
				80.31 m³		4.50 m		360.90
Base.	Beam.	0.56 × 1.00 × 5.0×2	=	5.60	×	1.99	=	11.15
	base	4.40 × 0.50 × 11.0	=	24.20	×	1.22	=	29.50
"		6.00 × 1.00 × 12.0	=	72.00	×	0.50	=	36.00
				101.80 m³		0.75 m		76.65

Summary of concrete = 182.11 cub. m

Weight of pier. Shaft S = 80.31 @ 2400 = 193.000 kg
base B = 101.80 @ 2400 = 244.000 "
437.000 "

Seismic thrust Shaft S' = 193000 × 0.25 = 48.200 "
base B' = 244000 × 0.25 = 61.000 "



Weight of earth on footing.

	6.0 × 12.0 × 3.0	=	216.00	
less shaft	2.51φ × 3.0 × 2	= −	29.70	
" beam	0.56 × 1.0 × 5.0 × 2	= −	5.60	
	E =		180.70 @ 1600 =	289000 kg

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MADE BY K. Inaba DATE 9-1-13 FILE NO
CHECKED BY M. J. DATE 9-2-24 PAGE NO 11R

CALCULATIONS FOR

Ukigase-Bashi for Saitama-ken.

Stability of Pier.

Case 1. Dead Load only:-

Superimposed Dead Load D = 223500 kg (see on page 14)
Dead Load of pier S+B = 437000
weight of earth on footing E = 289000
949500 kg

Bearing area = 12.0×6.0 = 72.0 sq. meters.
Unit bearing pressure on earth
= 949500 / 72.0 = 13200 kg/m² (corresponds to 1.27 tons/ft²)

Load on one pile = 949500 ÷ 78 = 12180 kg

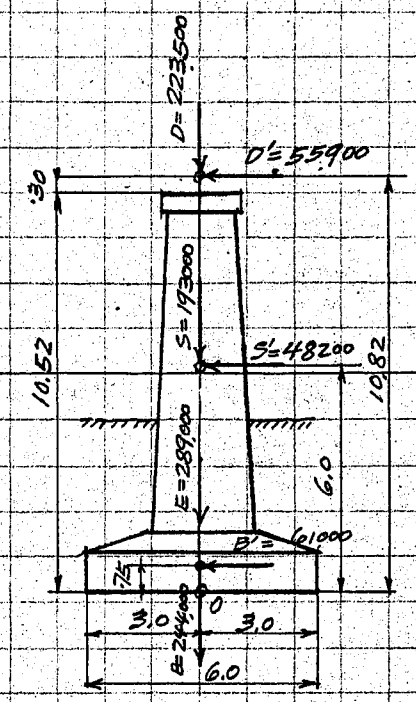
Case 2. Fully loaded:-

Total Dead Load + Earth load = 949500
Live Load on pier L = 113600 (see on page 14)
1063100 kg

unit bearing pressure on earth
= 1063100 / 72.0 = 14780 kg/m² (corresponds to 1.35 tons/ft²)

Case 3. During Earthquake: Acceleration assumed at 2500 mm/sec² or k = 0.25-0

Taking moment about center O of the base area.



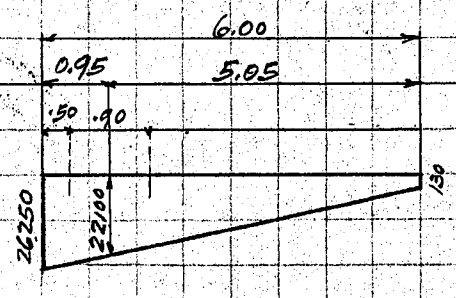
Load marks	Hor. forces	Vert. forces	Lev. arms		moments
D		223,500	× 0	=	0
D'	55,900		× 10.82	=	604,000
S		193,000	× 0	=	0
S'	48200		× 6.00	=	289,000
E		289,000	× 0	=	0
B		244,000	× 0	=	0
B'	61000		× 0.75	=	45800
	165,100 kg	949,500 kg	0.99 m		938,800

Resultant force within middle third.

max. toe pressure = 949.500 / (12.0×6.0) (1 ± 6×0.99 / 6.0) = 26250 kg/m² (2.40 tons/ft²)
or 130 ″

max. load on one pile

26250
22100
48350 ÷2 = 24180
24180 × 0.95 × 0.90 = 20670 kg



use 78 piles of 21 cm φ × 11.00 m as shown on previous sketch.

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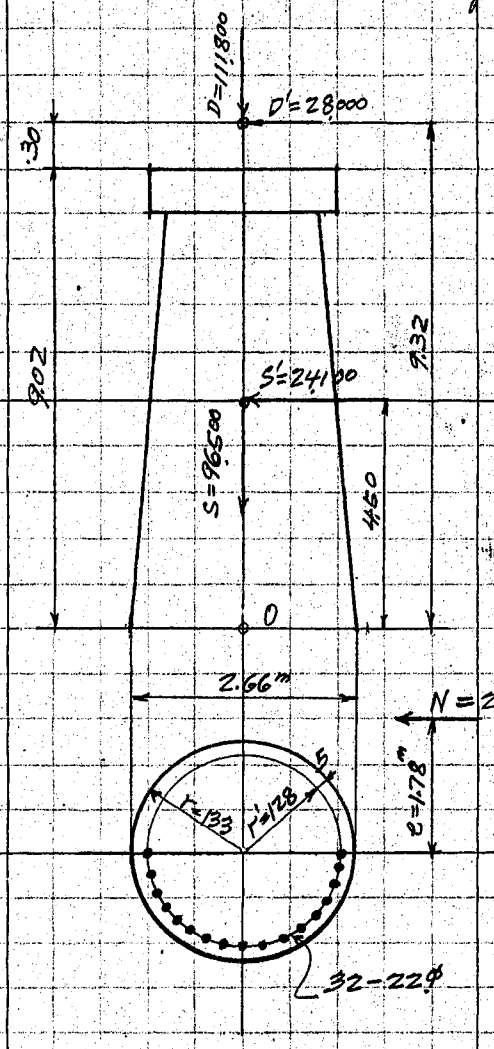
CALCULATIONS FOR

Akigase Bashi for Saitama-Ken

Seismic stresses in shaft at bottom section.



Loads on one column shaft.

Superimposed Dead load = D = 111,800 kg, Seismic thrust D' = 28,000 kg,
weight of shaft . S = 96,500 " " " S' = 24,100 "

Taking moment about center of bottom area O.

Load marks	Hor. forces	Vert. forces	Lev. arms	moments.
D		111,800		
D'	28,000		× 9.32 =	261,000
S		96,500		
S'	24,100		× 4.50 =	108,500
	52,100 kg	N = 208,300 kg	e = 1.78 m	369,500

Steel area assumed as 32 - 22φ bars = 121.7 cm² = As.

Steel ratio $p = \frac{A_s}{r^2\pi} = \frac{121.7}{133^2\pi} = 0.00219$

ratio $r'/r = 128/133 = 0.962$; $e/r = 178/133 = 1.34$

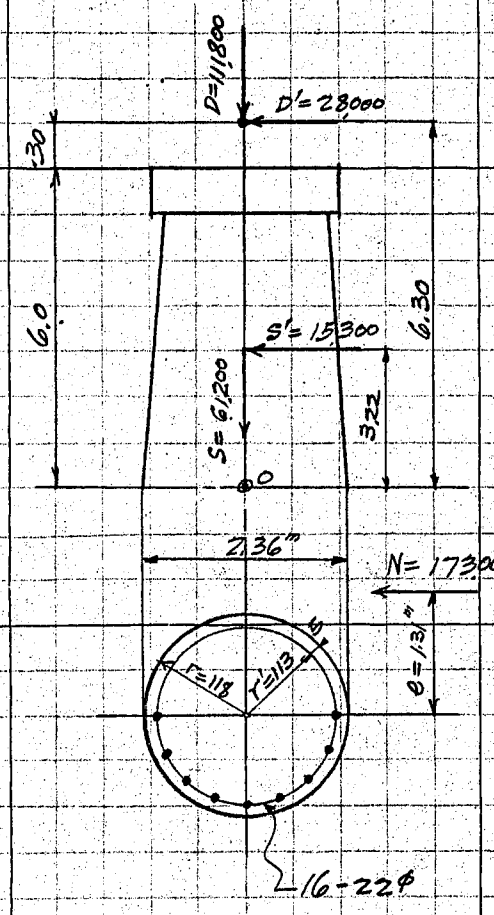
From the diagram.

$k = 0.59$, $C = 0.28$.

$f_c = \frac{N}{r^2 C} = \frac{208300}{133^2 \times 0.28} = 42.1$ kg/cm² < 40.5×1.6 = 64.8 for 1:2:5.5 Concrete

$f_s = \frac{1+\frac{r'}{r}-k}{k} \cdot n f_c = 15 \times 42.1 \times \frac{1+0.962-0.59}{0.59} = 1468$ kg/cm² < 1200×1.6 = 1920 (1372)

Seismic stresses in shaft at section 6.0 meter below top.



Weight and center of gravity.

Coping		2.77 × 5.80 =	16.07
"		2.45 × 5.80 =	14.20
Strut		3.23 × 5.30 =	17.12
"		4.51 × 4.40 =	19.85
Shaft	2.08φ × 5.60 × 2 = 38.65	× 2.55 =	97.00
	51.01	3.22 m	164.24

$S = \frac{51.01}{2} \times 2400 = 61{,}200$ kg ; S' = 61200 × .25 = 15,300 kg

Taking moment about O.

Load marks	Hor. forces	Vert. forces	Lev. arms	Moments.
D		111,800		
D'	28,000		× 6.30 =	176,500
S		61,200		
S'	15,300		× 3.22 =	49,300
	43,300 kg	N = 173,000 kg	e = 1.31 m	225,800

Steel area assumed as 16 - 22φ = 60.8 cm² = As.

Steel ratio $p = \frac{60.8}{118^2\pi} = 0.00144$

$r'/r = 113/118 = 0.958$; $e/r = 131/118 = 1.111$.

$k = 0.58$, $C = 0.26$

$f_c = \frac{173000}{118^2 \times 0.26} = 47.8$ kg/cm² < 64.8

$f_s = 15 \times 47.8 \times \frac{1+0.958-0.58}{0.58} = 1700$ kg/cm² < 1920 (1378)

Assumed sections are ample.

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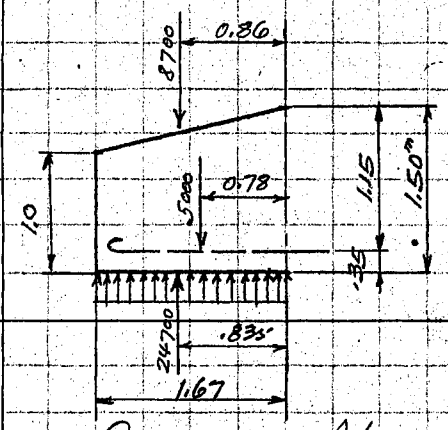
CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Stresses in footing.

Case 1. Stresses at normal state.

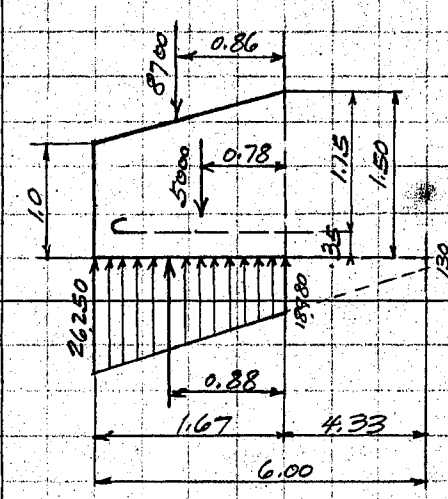
loads on footing per meter strip.



			arm	moment
Upward force	14780 × 1.67 =	24700 kg	× 0.835 =	20600
Downward forces				
Concrete	1.25 × 1.67 @2400 =	−5000	× 0.78 = −	3900
earth	3.25 × 1.67 @1600 =	−8700	× 0.86 = −	7500
		11000 kg		9200 kgm

Case 2. Stresses during earthquake.

loads on footing per meter strip.



26250
18980
45230 ÷ 2 = 22615

Upward pressure	22615 × 1.67 =	37800 kg	× 0.88 =	33300
Downward forces				
Concrete		−5000	× 0.78 = −	3900
earth		−8700	× 0.86 = −	7500
		24100 kg		21900 kgm

Seismic stresses govern the section of footing.

Effective depth required

$d = \sqrt{\frac{M}{bR}}$ where R = 6.04 for 1:2.5:5 concrete, f_c = 40.5 kg/cm², f_s = 1200 kg/cm²

$d = \sqrt{\frac{21900 \times 100}{100 \times 6.04 \times 1.6}} = 47.6$ cm

use 115 cm effective depth with 5 cm insulation. (底版 30 cm 、捨コン)

Steel area required $= \frac{21900 \times 100}{1920 \times \frac{7}{8} \times 115} = 11.33$ cm²

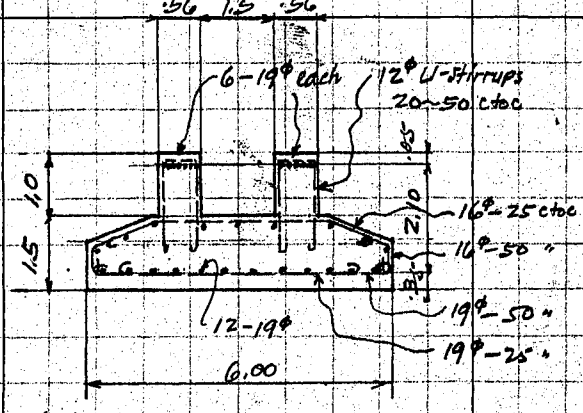
Use 19 mmφ bars at 25 cm spacing = 11.35 cm²

Unit shear $= \frac{24100}{100 \times \frac{7}{8} \times 115} = 2.4$ kg/cm²

Longitudinal reinforcements in base.

upward pressure at normal state

14780 × 6.0 = 88800 kg per lin. meter



Downward force

Concrete say	244000 ÷ 12 =	− 20300	"
earth "	289000 ÷ 12 =	− 24100	"
		44400	"

moment on base say

$m = \frac{244400 \times 4.3^2}{12} = 68400$ kgm shear = 44400 × 2.15 = 95400 kg

Steel area required $= \frac{68400 \times 100}{1200 \times \frac{7}{8} \times 215} = 30.32$ cm²

Use 12 − 19 mmφ bars = 34.08 cm²

Unit shear $= \frac{95400}{112 \times \frac{7}{8} \times 215} = 14.53$ kg/cm² use vertical stirrups

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CALCULATIONS FOR

Akigase - Bashi for Saitama - Ken

Design of truss pier PT4
General design feature of this pier same as for PT2 ; Superimposed load , base area and bottom of base same as PT2 except sheet piling in which steel sheet piles 6.0 metre long has been used ; Hence 72 timber pilings with spacing as shown inside of steel sheet piles.

Design of PT3.
By the data of permit the well sunk to depth of elev. -23.00 which is same as PT4 of original design. Loading on pier somewhat less than original PT4 , Hence used same design as for original PT4 on safe side.

Design of Girder Pier PG0 and PG1.
these two piers spans 15 metre girder instead of 20.0 metre. On account of more safety used similar design as for original PG1 for 20.0 metre girder span without any minute calculations.

設計 PT3 PG0 PG1 等の設計は凡そ當設計 PT4 及鋼杭打木杭の設計と等しきを見る可し

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CALCULATIONS FOR

MADE BY_____DATE_____FILE NO_____
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旧号
和九年四月

埼玉縣

府縣道補和斜譯線

秩父

ケ瀬橋設計ノ義書

(上部工事)

増田淳氏関係資料
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CALCULATIONS FOR

Design of Superstructure of Akigase - Bashi for Saitama-Ken.

General Layout of this bridge has been changed by the opinion of Engineers corp of Home Affair's Department after minute investigation of our plan. Conditions of permit are

1. River spans should be symmetrical about center of truss span and center pier being built on center of stream. This data obligate the shifting of bridge structure 15 meters toward west.
2. 15 meter girder span should be added at the end of 20 meter girder span This makes to lengthen the permanent structure by 15 meters at the east end.
3. Center pier of River span should be sunk to the elevation -23.0 meters and piers at both shore lines on piling foundation, Pier P T 2 enclosed by timber sheet piles and Pier P T 4 by steel sheet piles.

Design and Estimate of Cost of substructure has been changed and the final calculation of superstructure shall be made in this paper

River span ;- Anchor span of 37.5 m with 7.5 meter overhanging arm ; suspended span 37.5 meter. 2 sets of such spans over river,

West approach 6 spans of 20.10 meter girders = 120.60 meters
East approach 12 spans of 20.10 meter girders = 241.20 meters
1 span of 15.10 meter girder = 15.10 "

Wooden beam spans 23 spans of 9.00 meter beam = 207.00 meters

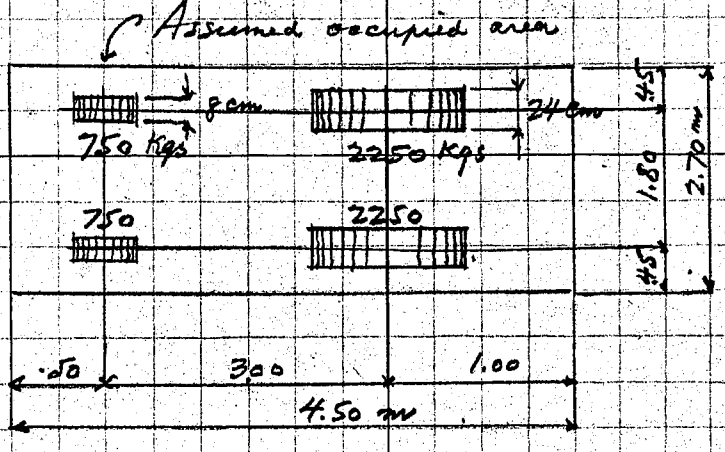
Thus the total length of bridge 770.50 meters. Grade of bridge level grade for truss river spans ; west approach 1:100 about with vertical curve; east approach 1:400 about with vertical curve at connection with river span.
Clear roadway 5.5 meters between curb lines, paved with 5 cm granolithic on reinforced concrete slab for permanent structure and earth covering for temporary structure. Handrails for permanent structure will be pipe and structural design and wooden handrails for temporary timber structure.

Assumed loading

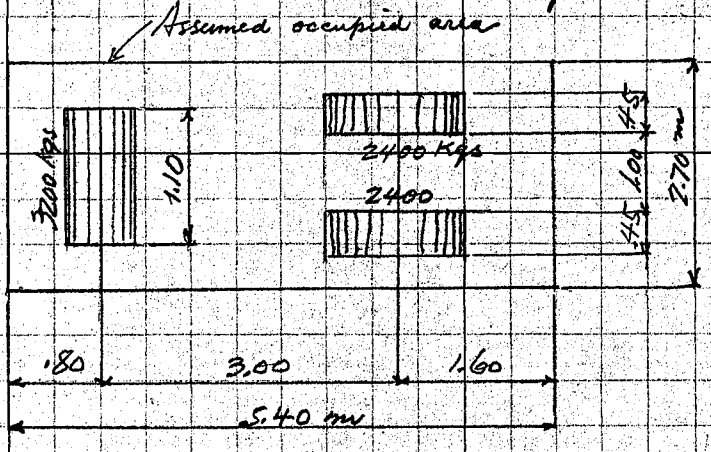
Uniform load $w = \frac{100.000}{170 + l} \leq 500$ kgs/m²

where w = Uniform load in kgs per sq. meter
l = Span length in meter

6 ton motor truck loading



8 ton road roller loading



2 Rows of motor traffic lines on roadway with occupied width of 2.70 metres each. Unoccupied space around motor truck and road roller shall be filled with uniform load specified above. One road roller on one span assumed.

Impact for motor truck loading

$$\text{coef} = \frac{20}{60 + l}$$

where l = loaded length in meter
max impact 30%

No impact for road roller and uniform live load.

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CALCULATIONS FOR

Design of superstructure of Akigase-Bashi for Saitama-Ken.

Allowable working Strength

Reinforcing bars 1200 kgs/cm² for tension
900 " " shear

Concrete 1:2:4 mixture

Direct compression	35	kgs/cm²
Fibre stress due to bending	45	"
Combined stress direct and bending (Compression member)	35	"
Resisting shear of Concrete	9	"
Shear of plain concrete	4	"
Bearing	45	"
Bond stress	6	"

Note Bond stress for bars and 22 mm in diameter shall not be counted.

Structural Steel

Tension net	1200	kgs/cm²
Extreme fibre stress net	1200	"
Shear of web gross section	900	"
Compression member $1500\left(1-0.0055\frac{\ell}{r}\right)$ not over	1000	"

where ℓ = length of member in cm
r = least radius of gyration in cm²

Compression flange of girder $1200\left(1-0.012\frac{\ell}{b}\right)$ not over	1100	"

where ℓ = Unsupported length of flange in cm
b = width of flange in cm

Shear on shop driven rivets (machine driven)	850	"
" " field " " and turned bolts (machine driven)	750	"
Shear on pin	900	"
Bearing on shop driven rivets (machine driven)	1700	"
" " field " "	1500	"
" " pin	1800	"

Roller $45d$ kg where d = diameter in cm.

Considering wind or temperature stress in addition to dead, live and impact stresses the allowable working Strength shall be increased 25%; In case of seismic stress increase by 60%.
Seismic acceleration assumed 2500 mm/sec², or $K = 0.25$

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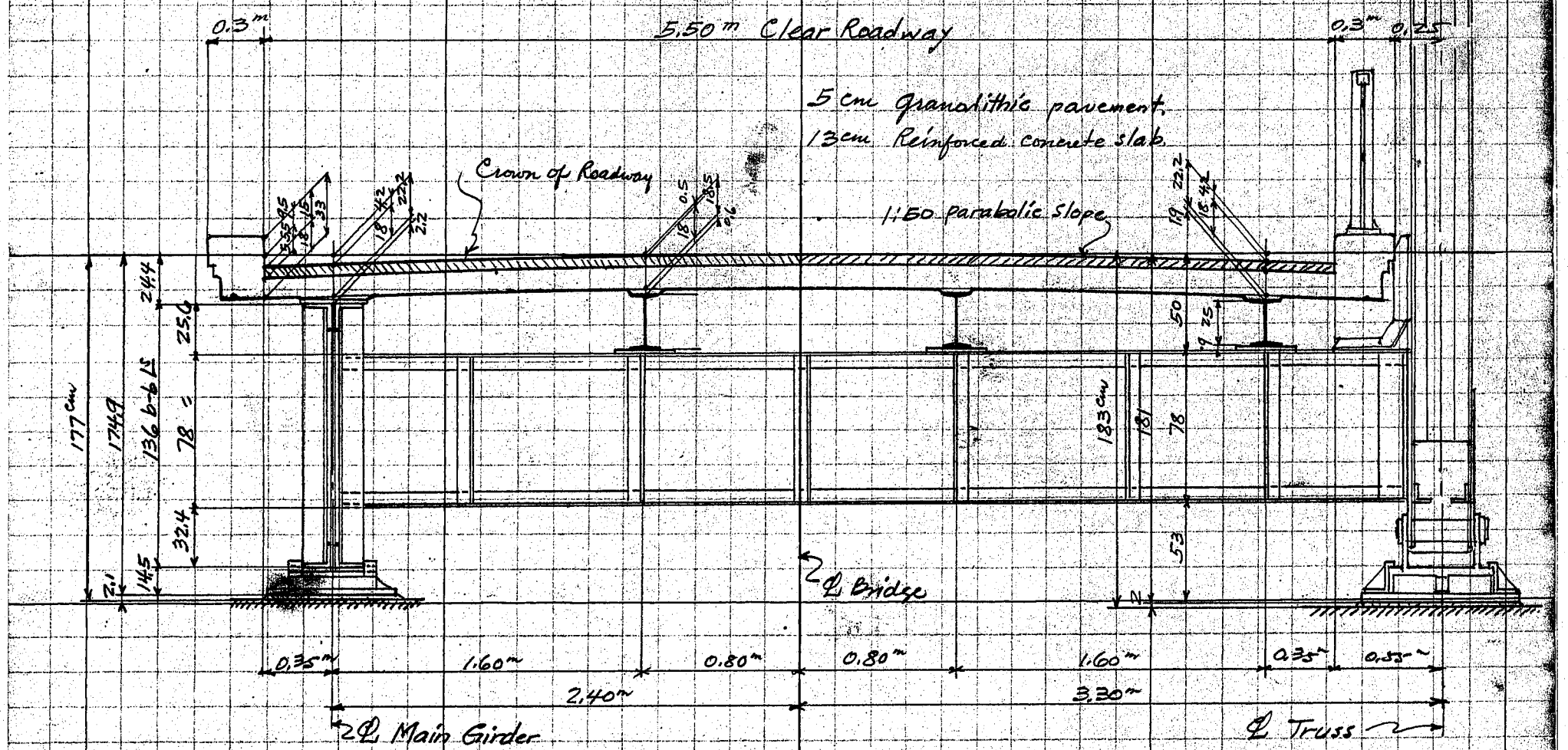
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Design of Floor Construction.

Cross section of bridge assumed as shown on sketch below.



GIRDER SPAN TRUSS SPAN
CROSS SECTION OF BRIDGE
Scale 1:30

Design of Floor slab. Span length = 1.60 metres

Dead Load:

Granolithic pavement	5.0 cm @ 22 kg	=	110.
Concrete slab	13.0 " @ 24 "	=	312
fillets and misc. concrete say		=	8
			430 kg/m²

Dead load moment = 1/10 × 430 × 1.60² = 110 kgm
Dead load shear = 1/2 × 430 × 1.60 = 345 kg

Live Load: 6 ton motor truck loading, Rear wheel concentration = 2250
Impact coefficient = 30% 675
2925 kg

Front wheel with imp. say 2925 ÷ 3 = 975 "

Distribution of wheel concentration:

Contact between wheel and pavement		= 20.0
distributions	2 @ 5.0	= 10.0
Longitudinal distribution		a = 30.0 cm

Transverse distribution b = 24 + 10 = 34.0 cm

Effective width of slab. E = 2/3 · l + a where l = span length in metres
= 2/3 × 1.6 + 0.3 = 1.37 metres

Load per metre strip = 2925 ÷ 1.37 = 2140 kg

Uniform live load on roadway assumed as 500 kg/m²

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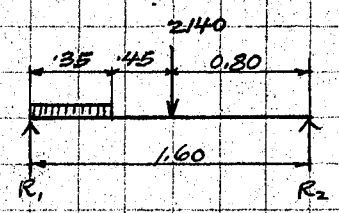
CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Moment at center of span. Reaction R_2



Uniform load $\frac{500 \times 0.35^2}{2 \times 1.6} = 19$

rear wheel $2140 \div 2 = 1070$

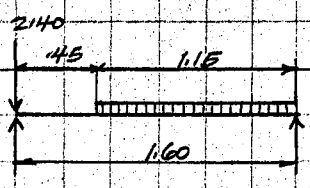
1089 kg

moment $= 1089 \times 0.80 = 871$

For continuity of slab moment may be taken as 80% of the above value.

moment at center $= 0.80 \times 871 = 697$ kgm.

End shear.



uniform load. $\frac{500 \times 1.15^2}{2 \times 1.6} = 207$

rear wheel $= 2140$

2347 kg

Summary of moments and end shears.

	moment	end shear
Dead Load	110	345
Live Load	697	2347
	807 kgm	2692 kg

Effective depth of slab required for 1:2.5:5 concrete, $f_s = 1200$, $f_c = 4.5$ kg/cm², $R = 7.13$

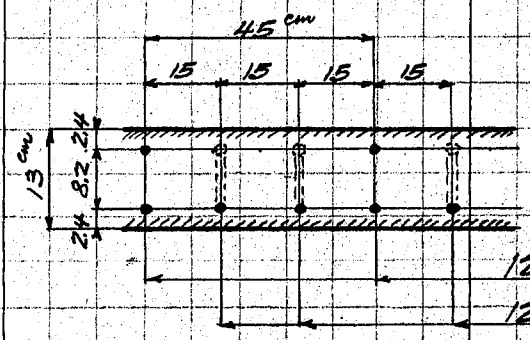
$$d = \sqrt{\frac{M}{bR}} = \sqrt{\frac{807 \times 100}{100 \times 7.13}} = 10.6 \text{ cm}$$

Use 10.6 cm effective depth with 2.4 cm insulation, or 13 cm in total.

Steel area required $= \frac{807 \times 100}{1200 \times \frac{7}{8} \times 10.6} = 7.25$ cm² per meter strip

Use 12 mmϕ bars at 15 cm c to c $= 7.54$ cm² " " "

unit shear $= \frac{2692}{100 \times \frac{7}{8} \times 10.6} = 2.90$ kg/cm²



12ϕ Straight bars on ~~top~~ and bottom.

12ϕ Bent bars (Bent up at each support.)

Overhanging slab beyond main girder or end stringer of truss span.

General Sketch of Handrail and Coping.
Scale 1:20

Note:- Use same details for handrails over Girder and Truss spans.

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Approximate weight of Handrail.

Top rail	1-2 1/2" φ Gas pipe	@ 7.47	× 1.25	=	9.34	
hor. L	1L 65×50×5	@ 4.36	× 1.25	=	5.45	
vertical	5-3/4" φ gas pipes	@ 1.69	× 0.38	=	2.96	
post	1 I 3"×3"	@ 12.65	× 0.75	=	9.49	
" base	2L 65×65×6	@ 5.91	× 0.13	=	1.54	
" base pl.	1Pl 210 ×12	@ 19.78	× 0.22	=	4.35	
cap pl.	1Pl 75 ×6	@ 3.53	× 0.20	=	0.71	
conn L (hor L)	2L 65×65×6	@ 5.91	× 0.06	=	0.71	
Anchoring	4 bolts 12φ × 0.20	@ 0.30		=	1.20	
washer	2 Pls 50 × 6	@ 2.36	× 0.20	=	0.94	
misc. details say					0.81	
					37.50	kg

weight per lin. meter = 37.5 ÷ 1.25 = 30. kg.

Approximate weight of Coping

0.13 × 0.30 =	0.039	× 0.150	=	0.00585
0.05 × 0.27 =	0.014	× 0.135	=	0.00189
0.15 × 0.23 =	0.035	× 0.115	=	0.00403
	0.088	0.134 m		0.01177

weight = 0.088 @ 2400 = 210 kg

Dead Load moment on overhanging arm.

Handrail	30	× 0.48	= 14
Coping	210	× 0.484	= 102
Slab and pavement			
430×0.35 = 150		× 0.175	= 26

Shear = 390 kg Moment = 142 kgm

Live Load

Motor truck rear wheel at 15 cm inside of curb line assumed.
Distribution or effective width on main girder assumed thus.

2 @ 0.35 + 0.20 = 0.90 meter

Transverse distribution 0.24 + 10 = 1.34 meter
Load for one meter strip = 2925 ÷ 0.9 = 3,250 kg
Live load moment = 3250 × 0.20 = 650 kgm
Shear assumed punching shear.
Sheared length along girder = = 0.90
" " across " = 2×0.35 = 0.70
1.60 meters

Shear for one meter = 3250 ÷ 1.6 = 2030 kg

Summary of moments and shears.

	moment	shear
Dead Load	142	390
Live Load	650	2030
	792 kgm	2420 kg

Steel area req'd. = $\frac{792 \times 100}{1200 \times 7/8 \times 10.6}$ = 7.12 cm²
use 12 mm φ bars at 15 cm c to c = 7.54 cm² on top.
Unit shear = $\frac{2420}{100 \times \frac{7}{8} \times 10.6}$ = 2.61 kg/cm²

Temperature bars in slab.
Steel area required = $\frac{3}{1000}$ × 550 × 13 = 21.45 cm² for the whole width of roadway

12 mm φ bars, required no. of bars = $\frac{21.45}{1.13}$ = 19

use more than 20 bars.

増田淳氏関係資料
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MADE BY K. Inaba DATE 8-11-7 FILE NO
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CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Design of Structural steel parts for Girder span.

Design of I-beam stringer :- Span length = 4.65 meters, spacing 1.60 meters.

Dead load :

Floor slab and pavement $430 \times 1.60 = 688$
Stringer assumed as $\frac{50}{738}$ Call this 740 kg per lin. meter.

Dead load moment $= 1/8 \times 740 \times 4.65^2 = 2{,}000$ kgm.
Dead load shear $= 1/2 \times 740 \times 4.65 = 1{,}720$ kg.

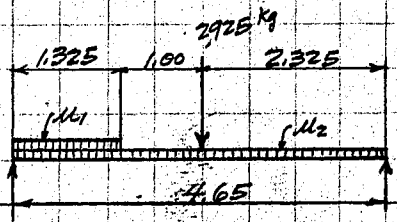
Live load : Motor truck rear wheel concentration with impact = 2,925 kg

Uniform live load on roadway = 500 kg/m²

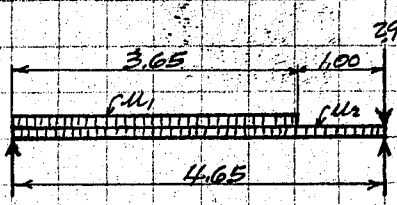
Load on stringer

Side of truck $= \frac{500 \times 1.15^2}{2 \times 1.60} = 207$ kg per lin. meter of span ---- M_2

Front and rear of truck $500 \times 1.6 = 800$ " " " " "
Difference $= 593$ " " " " ---- M_1



Moment,
due to rear wheel $2925 \div 2 \times 2.325 = 3400$
Uniform load M_1 $\frac{593 \times 1.325^2}{2 \times 4.65} \times 2.325 = 260$
" " M_2 $1/8 \times 207 \times 4.65^2 = 560$
4220 kgm



End shear,
due to Uniform load M_1 $\frac{593 \times 3.65^2}{2 \times 4.65} = 850$
" " " M_2 $207 \times 4.65 \div 2 = 480$
" rear wheel $= 2925$
4255 kg

Summary for moments and shears.

	moment	shear
Dead Load	2,000	1,720
Live Load	4220	4255
	6,220 kgm	5975 kg

Section modulus required $= \frac{6220 \times 100}{1100} = 565$ cm³

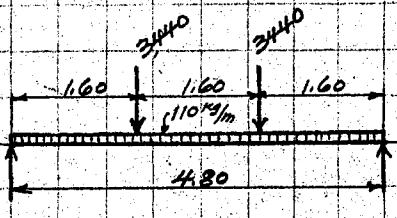
use I beam 300×150 @ 48.3 kg whose $S_m = 633.2$ cm³

unit fibre stress $= \frac{6220 \times 100}{633.2} = 983$ kg/cm² T or C

unit shear on web $= \frac{5975}{30 \times 0.8} = 249$ kg/cm²

Design of Intermediate floor beam :- Span length 4.80 meters, spacing 4.65 meters.

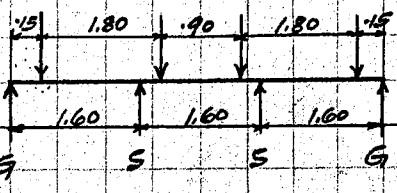
Dead Load :



Stringer concentration on floor beam = 740 × 4.65 = 3440 kg
floor beam assumed as 110 kg per lin meter of span.
Dead load moments
due to stringer concentrations $3440 \times 1.60 = 5500$
due to weight of floor beam $1/8 \times 110 \times 4.80^2 = 317$
5,817 kgm.

Dead load end shear
due to stringer concentrations 3440
due to weight of floor beam $1/2 \times 110 \times 4.80 = 264$
3704 kg

Live load :



motor truck rear wheel concentration with impact = 2,925 kg
" " front " " " " " = 975 "

Load on S $\frac{.15}{1.60} = 0.094$
$\frac{1.000}{1.094}$ @ 2925 = 3,200 kg --- rear wheel
1.094 @ 975 = 1,065 kg --- front "

Uniform live load = 500 × 1.60 = 800 kg per lin m on stringer

増田淳氏関係資料
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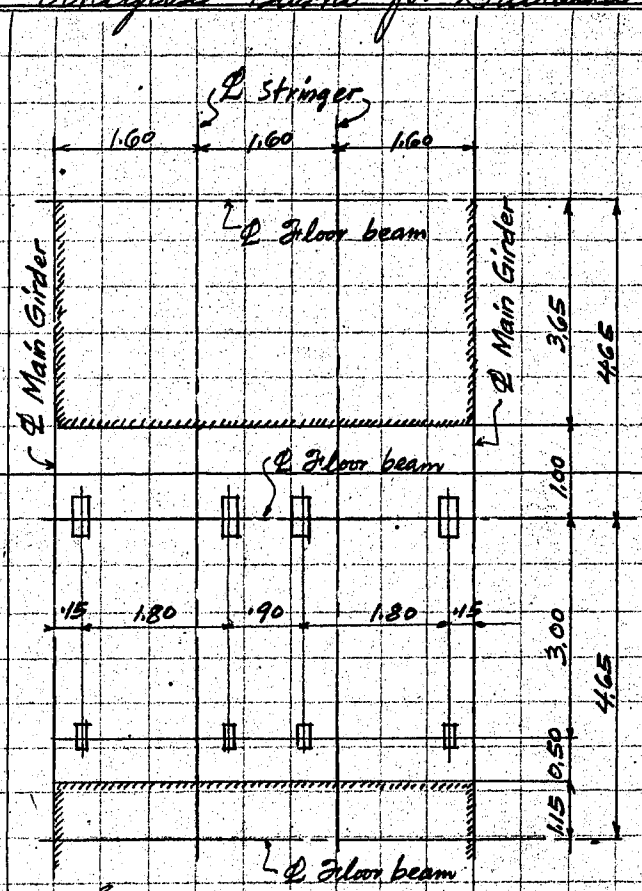
CALCULATIONS FOR

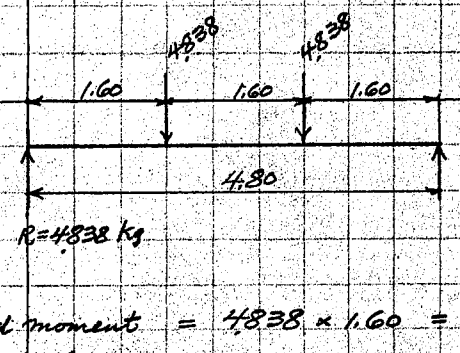
Akigase-Bashi for Saitama-ken.



Concentration on floor beam from stringer

Motor truck loading, rear wheel 8000 × 1.65/4.65 = 3200
front wheel 1065 × 1.65/4.65 = 378
3578 kg

Uniform load 800 × 3.65²/(2×4.65) = 1146
800 × 1.15²/(2×4.65) = 114
1260
4838 kg



Live load moment = 4838 × 1.60 = 7740 kgm
Live load shear = 4838 kg

Summary of moments and end shears.

	moment	shear
Dead Load	5817	3704
Live Load	7740	4838
	13557 kgm	8542 kg

Web is assumed as 770 × 8 = 61.6 cm², 1/8 web area = 7.70 cm²
Depth back to back of flange ℒs 78 cm.
Effective depth say 78.0 − 2@2.15 = 73.7 cm
flange stress = 13557 × 100 / 73.7 = 18400 kg C or T
flange area req'd. = 18400 ÷ 1200 = 15.33
less 1/8 web area = − 7.70
7.63 cm² net
one ℒs 75×75×9 = 25.38 − 3.96 = 21.42 cm² net.

Unit shear on web = 8542 / 61.6 = 139 kg/cm²

For end connection, 22φ rivet no. required = 8542 / 2640 = 3.24 Use 8 or 9 field rivets.

Approximate weight of Intermediate Floor Beam.

Flange	4 ℒs	75×75×9	@	9.96	× 4.61 (average)	=	184
web	1 PL.	770 ×8	@	48.36	× 4.78	=	231
Stiff. ℒs	10 ℒs	75×65×8	@	8.79	× 0.78	=	65
end conn. ℒs	2 ℒs	125×75×9	@	13.50	× 0.62	=	17
Stringer base	2 PLs	290 ×9	@	20.49	× 0.31	=	13
rivet heads and variations say							20
							530 kg

weight of floor beam per lin. meter = 530 / 4.80 = 110 kg

増田淳氏関係資料
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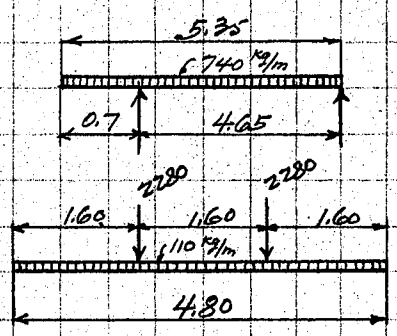
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Design of End floor beam:- Span length 4.80 meters, spacing 4.65 meters.



Stringer concentration on floor beam.

$$\frac{740 \times 5.35^2}{2 \times 4.65} = 2{,}280 \text{ kg}$$

Weight of floor beam assumed as 110 kg per lin meter of span.

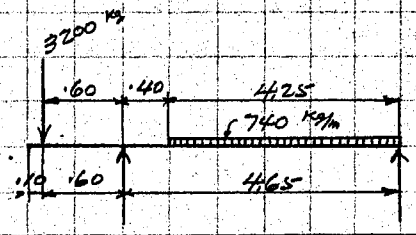
Dead load moment:-

Stringer concentration = 2280 × 1.60 = 3,648
weight of floor beam = 1/8 × 110 × 4.80² = 317
3,965 kgm

End shear

Concentrations of stringer 2280
weight of floor beam 1/2 × 110 × 4.80 = 264
2,544 kg

Live load:-



Stringer concentration due to rear wheel = $\frac{3200 \times 5.25}{4.65}$ = 3610
" " " unif load = $\frac{740 \times 4.25^2}{2 \times 4.65}$ = 1440
5050 kg

moment = 5050 × 1.60 = 8,080 kgm
end shear = 5,050 kg

Summary of moments and shears.

	moment	shear
Dead Load	3,965	2,544
Live Load	8,080	5,050
	12,045 kgm	7,594 kg

Use a similar detail as for intermediate floor beam.
weight of beam say 530 kg.

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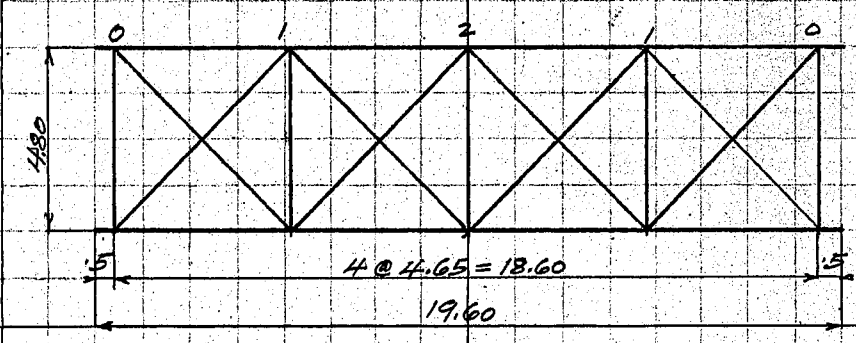
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken

Design of Lateral Bracings:-



Diagonal length
$4.80^2 = 23.04$
$4.65^2 = 21.62$
44.66
$\sqrt{44.66} = 6.68$ meters.
Coefficient $= \frac{6.68}{4.80} = 1.392$

Wind load assumed 400 kg per lin meter of bridge, Panel load = 400 × 4.65 = 1,860 kg
End shear at panel 0-1 = 1860 × 1.5 = 2,790 kg
Stress in diagonal 0-1 = 2790 × 1.392 = 3,880 kg T.

Seismic stresses. Acceleration assumed 2500 mm/sec² or $k = 0.25$-0
Dead Load.

Floor slab and pavement between main girders	4.80 × 430 =	2,065
Handrail, coping, and floor outside of girders	2 @ 390 =	780
		2845
Stringers	2 @ 50 =	100
floor beams	530 ÷ 4.65 =	114
lateral bracings say		50
main girders assumed as		630
		894
		3739
	Call this	3740 kg per lin. m.

Panel load = 3740 × 4.65 = 17,400 kg
Horizontal panel load = 17400 × 0.25 = 4,350 kg

Shear in panel 0-1 = 4350 × 1.5 = 6,520 kg
Stress in diagonal 0-1 = 6520 × 1.392 = 9,080 kg T
Seismic stresses govern the sections of diagonal members.
Sectional area required $= \frac{9080}{1920} = 4.73$ cm² net

19φ rivet no. required $= \frac{9080}{2126 \times 1.6} = 2.67$ use 3-19φ rivets

use 1L 125 × 75 × 9 = 17.19 - 1.98 = 15.21 cm² net

Approximate weight of lateral bracings.

Diagonals	8L	125 × 75 × 9	@	13.50 × 6.30 =	680
Center connections	4		@	11.00 =	44
Conn. to girders	10		@	13.00 =	130
rivet heads say					16
					870 kg

870 ÷ 18.6 = 47 kg per lin. meter

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MADE BY K. Inata DATE 8-11-13 FILE NO
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CALCULATIONS FOR

Gkigase-Bashi for Saitama-Ken

Design of main girders. Span length 19.60 meters c.to.c. Bearings, 20.0 meters c.to.c.

Dead load :-

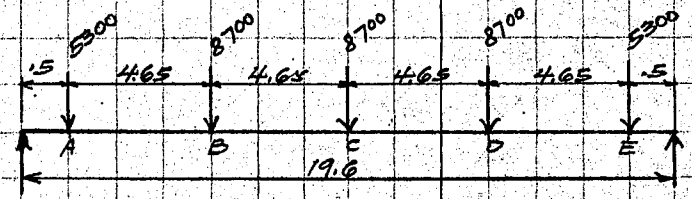
Floor slabs, pavement, handrails + copings	=	2845	See on page 9
Stringers, floor beams, laterals + main girders	=	894	" "
		3739	

Call this 3740 kg per lin. meter.

Dead load for one girder = 3740 ÷ 2 = 1870 " " "

Let us assume the dead load to be concentrated at panel points as follows:

Intermediate panel concentration = 1870 × 4.66 = 8,700 kg for one girder
End panel concentration = 1870 × 2.83 = 5,300 " "



Reaction R

1.5 × 8700 = 13,050
5300
18350 kg

moment at A. 18350 × 0.50 = 9180 kgm

moment at B. 18350 × 5.15 = 94500
5300 × 4.65 = − 24650
69850 kgm

moment at C 18350 × 9.80 = 180000
5300 × 9.30 = − 49300
8700 × 4.65 = − 40500
90,200 kgm

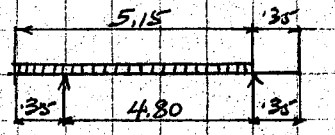
End shear = 18350 kg
Dead load on shoe 1870 × 20.0 ÷ 2 = 18700
weight of shoe say 200
18900 kg on one shoe.

Live load.

Uniform live load = 500 kg per sq. m.
Impact for motor truck loading = 70 / 60+19.6 = 25.1%

motor truck rear wheel concentration = 22.50
25.1% impact = 5.65
28.15 kg

Front wheel concentration with impact = 28.15 / 3 = 938 kg

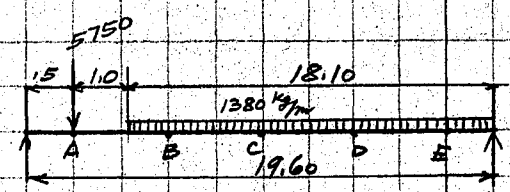


Uniform live load = 500 × 5.15² / 2×4.8 = 1380 kg per lin. meter

Wheel concentrations on main girders assumed as below.

Rear wheel ~~4×~~ 2815 × 2.45 / 24.80 = 5750 kg

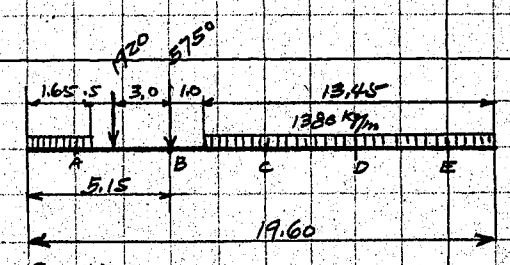
front wheel say 5750 ÷ 3 = 1920 "



Moment at A.

Reaction 5750 × 19.1 / 19.6 = 5600
1380 × 18.1² / 2×19.6 = 11530
17130 kg

moment = 17130 × 0.50 = 8565 kgm



Moment at B.

Reaction 5750 × 14.45 ÷ 19.6 = 4240
1920 × 17.45 ÷ 19.6 = 1710
1380 × 13.45 × 6.725 ÷ 19.6 = 6370
1380 × 1.65 × 18.775 ÷ 19.6 = 2190
14510 kg

moment 14510 − 5.15 = 74800
1920 × 3.00 = − 5760
1380 × 1.65 × 4.325 = − 9850
59190 kgm

増田淳氏関係資料
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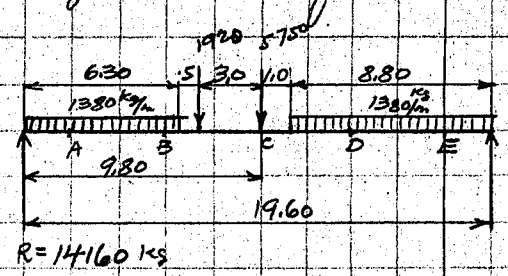
CALCULATIONS FOR

Ikigase-Bashi for Saitama-Ken



moment at C.

Reaction
5750 × 2 = 2875
1920 × 12.8 ÷ 19.60 = 12535
1380 × 8.8 × 4.4 ÷ 19.6 = 2730
1380 × 6.3 × 16.45 ÷ 19.6 = 7300
14160 kg

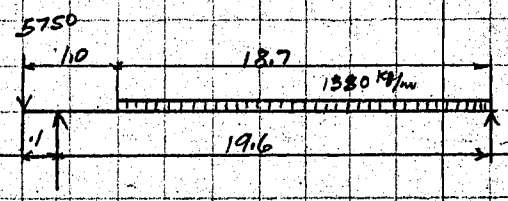
Moment
14160 × 9.8 = 138800
1920 × × 3.0 = − 5760
1380 × 6.3 × 6.65 = − 57800
75240 kgm

max. end shear. Say rear wheels at ℄ Bearings.

uniform load. $\frac{1380 \times 18.6^2}{2 \times 19.6}$ = 12170

rear wheels = 5750
17920 kg

max. load on bearing.

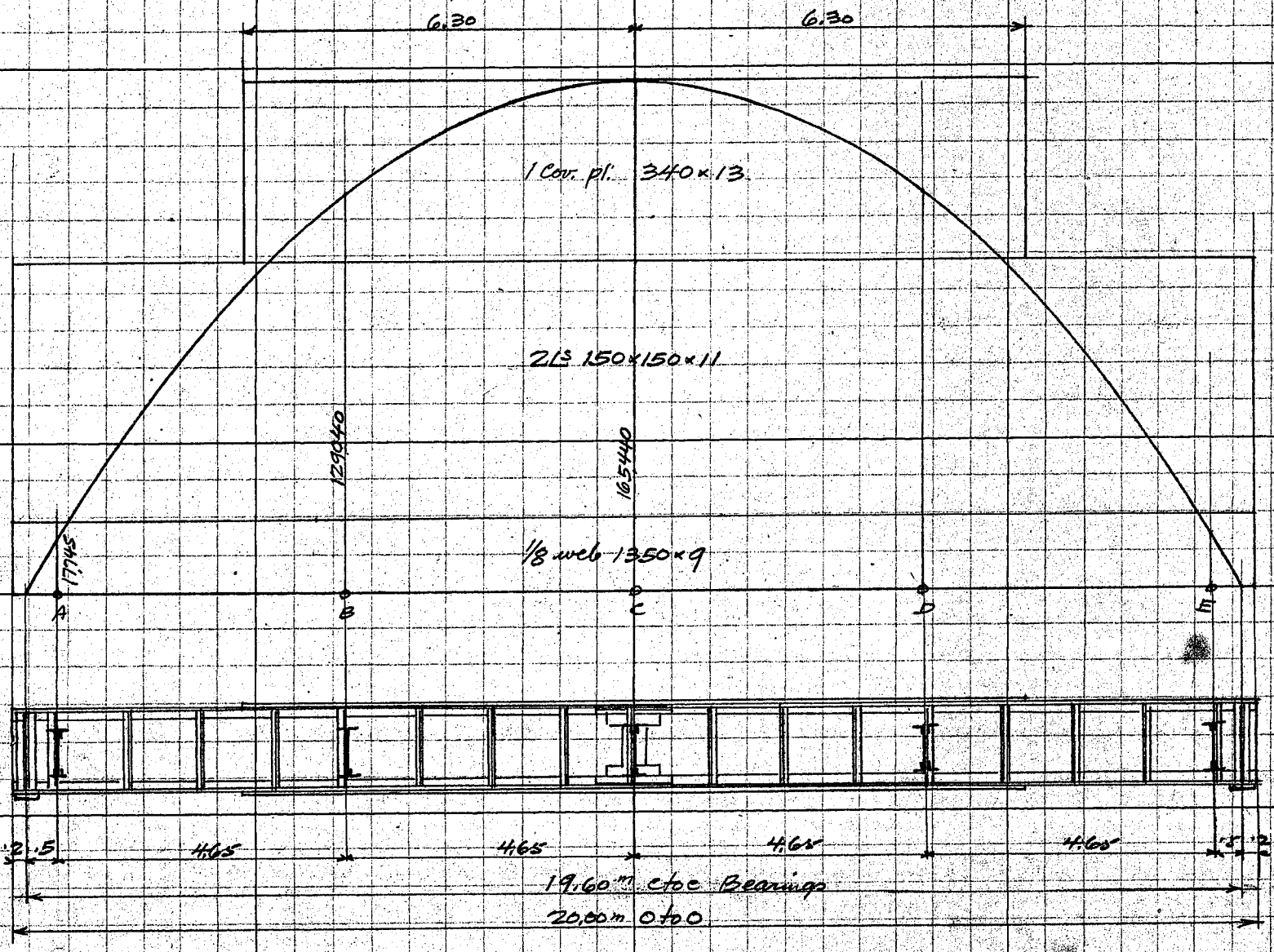


uniform load = $\frac{1380 \times 18.70^2}{2 \times 19.6}$ = 12300

rear wheels 5750
18050 kg

Summary of moment, shears, and loads on bearing.

	moment at A	moment at B	moment at C	End shear	Load on bearing
Dead Load	9180	69850	90200	18350	18900
Live Load	8565	59190	75240	17920	18050
	17745 kgm	129040 kgm	165440 kgm	36270 kg	36950 kg



Moment Diagram,

Scale of space 1:100
Scale of moment 1/20m = 100,000 kgm

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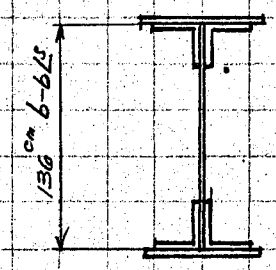
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken

Section of main girder



Dry web pl. 1350 × 9 = 121.5 cm² 1/8 web area = 15.20 cm²
Depth back to back of flange Ls = 136 cm
Effective depth = 136 − 2@2.14 = 131.72 cm
Flange stress = $\frac{165440}{1.3172}$ = 125500 kg T or C

Flange section required

Bottom flange $\frac{125500}{1200}$ = 104.60
less 1/8 web area − 15.20
89.40 cm² net

1 Cov. pl. 340×13 = 44.20 × 0.65 = 28.7
2 Ls 150×150×11 = 64.00 × (−4.07) = − 260.5
108.20 2.14 cm − 231.8

Top flange $\frac{125500}{1100}$ = 114.20
less 1/8 web area = 15.20
99.00 cm² gross.

Use 2 Ls 150×150×11 = 64.00 − 11.00 = 53.00
1 Cov. pl. 340 × 13 = 44.20 − 6.50 = 37.70
108.20 cm² gross 90.70 cm² net

Assume shear on web gross section

$= \frac{36270}{121.5} = 299$ kg/cm²,

Approximate weight of main girder.

Flange	4 Ls	150 × 150 × 11	@ 25.10	× 20.00	=	2,000
3	2 Cov. pls	340 × 13	@ 34.70	× 12.60	=	875
web	1 Pl.	1350 × 9	@ 95.39	× 20.00	=	1,908
End stiffeners	8 Ls	125 × 90 × 10	@ 16.10	× 1.34	=	173
	4 fills	180 × 11	@ 15.54	× 1.06	=	66
Int. "	32 Ls	125 × 75 × 9	@ 13.50	× 1.36	=	587
center "	2 Ls	" " "	@ "	× 1.31	=	35
Conn. Ls	8 Ls	150 × 100 × 9	@ 17.10	× average 3.25	=	34
Sole pl.	2 Pls	400 × 25	@ 78.50	× 0.46	=	72
Center splice say						330
Rivet heads 4% say						220
						6300 kg

6.80

Weight of girder per lin metre = 6300 ÷ 20 = 315 kg.

Summary of structural steel in one span.

Stringers	2 @ 500 × 20.00	=	2,000	
floorbeams	5 @ 530	=	2,650	
lateral bracing complete		=	870	18800 kg
main girders	2 @ 6300	=	12,600	
shoes say	4 @ 170	=	680	
expansion metal say			600	
			19,400 kg	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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MADE BY K. Inaba DATE 8-11-13 FILE NO
CHECKED BY H.T. DATE 8-11-17 PAGE NO 13

CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Design of Structural steel parts for Truss span.

Design of inside stringer. Span length 3.75 meters, spacing 1.60 meters.

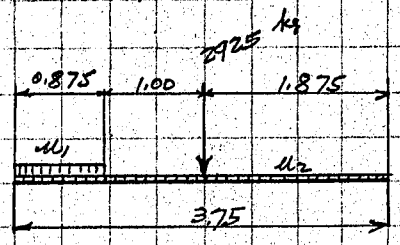
Dead load:-

Floor slab and pavement 430 × 1.60 = 688
Stringer assumed as 50
738 Call this 740 kg per lin meter.

Dead load moment = 1/8 × 740 × 3.75² = 1,300 kgm
Dead load shear = 1/2 × 740 × 3.75 = 1,390 kg

Live load:- Motor truck rear wheel concentration with impact = 2925 kg.

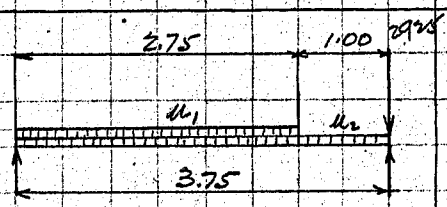
1st case



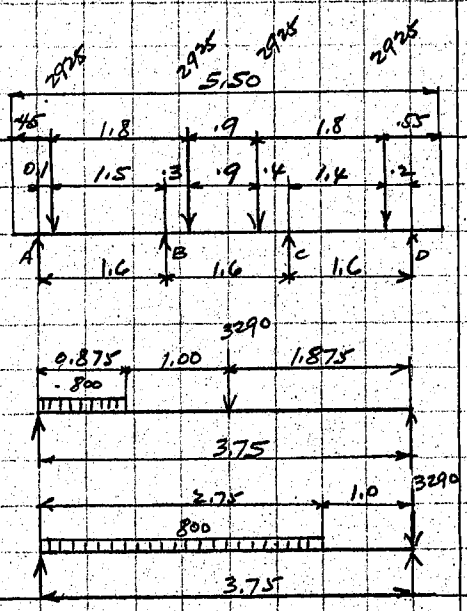
Uniform live load on roadway = 500 kg/m²
Load on stringer.
Side of truck 207 kg per lin meter = u2 see on page 6.
front and rear of truck 800 " " = u1 "
difference 593 " " "

Moment due to
rear wheel 2925 ÷ 2 × 1.875 = 2743
Unif. load u1 593 × 0.875² / 2 × 3.75 × 1.875 = 114
" " u2 1/8 × 207 × 3.75² = 364
3221 kgm

End shear due to
rear wheel 2925
Unif. load u1 593 × 2.75 × 1.375 / 3.75 = 598
" " u2 207 × 3.75 ÷ 2 = 388
3911 kg



2nd. case.



Load on stringer B.

2925 × 0.4 = 1170
" × 1.3 = 3800
" × 0.1 = 293
5263 ÷ 1.6 = 3290 kg

Moment

800 × 0.875² / 2 × 3.75 × 1.875 = 153
3290 ÷ 2 × 1.875 = 3085
3238 kgm

End shear

800 × 2.75² / 2 × 3.75 = 807
3290
4097 kg

Summary of moments and shears.

	Moment	Shear
Dead Load	1,300	1,390
Live Load	3238	4097
	4538 kgm	5,487 kg.

Section modulus required = 4538 × 100 / 1100 = 412.5 cm³

Use I beam 300 × 150 @ 48.3 kg Sm = 633.2 cm³

unit shear on web = 5487 / 30 × 8 = 229 kg/cm²

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CHECKED BY H. T. DATE 8-11-17 PAGE NO 14

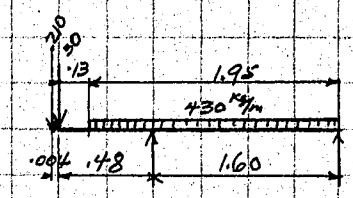
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Design of Outside Stringer. Span length = 3.75 meters, spacing 1.60 meters.

Dead load :-

Load on stringer.



$\frac{430 \times 1.95^2}{2 \times 1.60}$ = 511

30 × 2.08 ÷ 1.6 = 39

210 × 2.084 ÷ 1.6 = 274

824

Stringer assumed 40

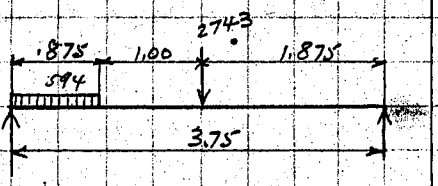
864 kg per lin meter.

Dead load moment = 1/8 × 864 × 3.75² = 1,520 kgms

Dead load shear = 1/2 × 864 × 3.75 = 1,620 kg.

Live load :-

Load on stringer = 2925 × 1.5/1.6 = 2,743 kg

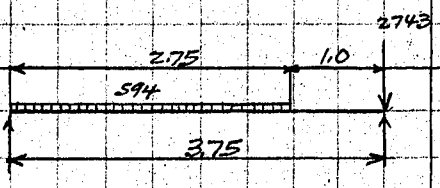


unif. load. $\frac{500 \times 1.95^2}{2 \times 1.60}$ = 594 kg/lin m.

moment. $\frac{594 \times .875^2}{2 \times 3.75} \times 1.875$ = 114

2743 × 3.75 ÷ 4 = 2570

2,684 kgm



Shear

$\frac{594 \times 2.75^2}{2 \times 3.75}$ = 600

2743

3,343 kg

Summary of moments and shear

	moment	shear
Dead Load	1,520	1620
Live Load	2684	3343
	4,204 kgm	4963 kg

Section modulus required = 420400/1100 = 382 cm³

use I beam 250×125@38.3 Sm = 414.9 cm³

Design of Intermediate Floor Beam. Span length 6.60 meters, spacing 3.75 meters.

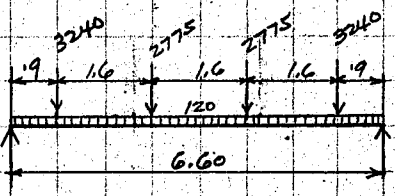
Dead load :

Stringer concentrations on floor beam.

Inside stringers 740 × 3.75 = 2,775 kg

outside " 864 × 3.75 = 3,240 kg.

floor beam assumed as 120 kg per lin meter.



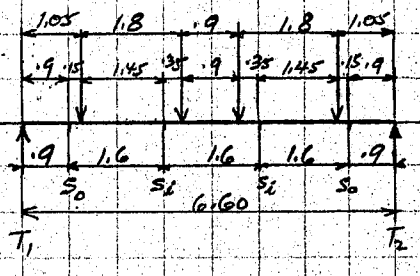
moment

3240			
2775			
6,015	× 2.50	=	15040
3240	× 1.60	= −	5180
1/8 × 120 ×	6.60²	=	650
			10,510 kgm

Shear

120 × 3.30	=	395
		6015
		6410 kg

Live Load :



motor truck rear wheel concentration with impact = 2925 kg

" " front " " " " = 975 "

Load on stringers S_0 = 2925 × 1.45 ÷ 1.6 = 2650 kg — rear wheel

S_i = 2925 × .15 ÷ 1.6 = 275

2925

3200 kg — rear wheel,

S_0 2650 ÷ 3 = 885 kg — front "

S_i 3200 ÷ 3 = 1,068 " — " "

uniform load. S_0 594 kg/lin m.

S_i 800 " "

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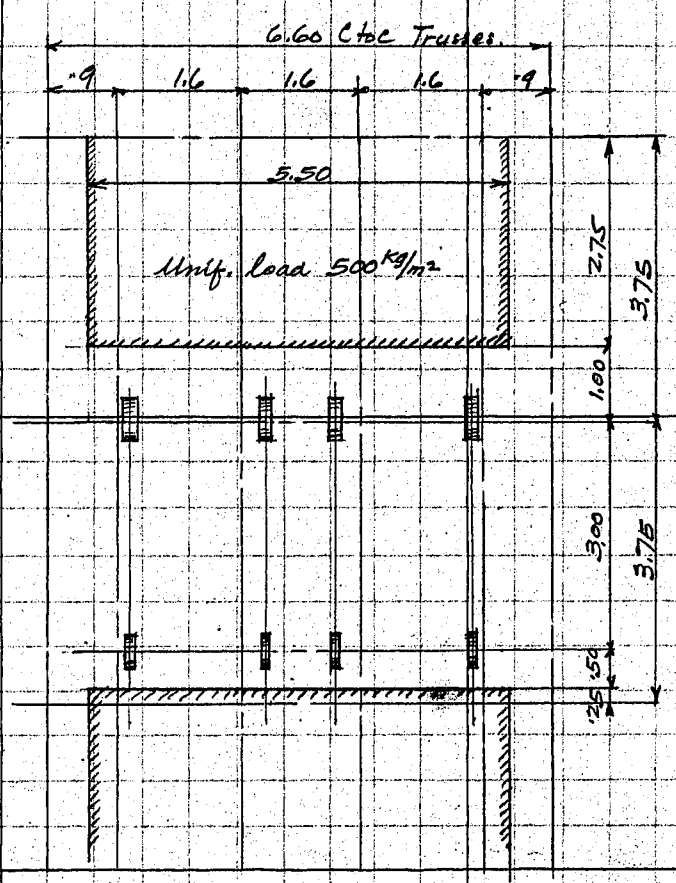
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.



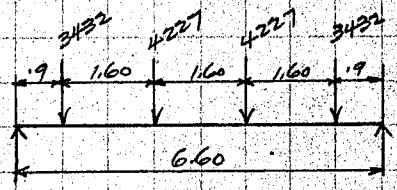
Stringer concentrations on floor beam.

Outside stringer So.

Rear wheel		2650
front "	$885 \times \frac{0.75}{3.75}$ =	177
Unif. load	$\frac{594 \times 2.75^2}{2 \times 3.75}$ =	600
" "	$\frac{594 \times 0.75^2}{2 \times 3.75}$ =	5
		3432 kg

Inside stringer Si

Rear wheel		3200
front "	$1065 \times \frac{0.75}{3.75}$ =	213
Unif. load	$\frac{800 \times 2.75^2}{2 \times 3.75}$ =	807
" "	$\frac{800 \times 0.75^2}{2 \times 3.75}$ =	7
		4227 kg



End shear 3432 + 4227 = 7659 kg

moment 7.659 × 2.50 = 19.150

3432 × 1.60 = − 5490

13,660 kgm

Summary of moments and shears.

	moment	shear
Dead Load	10510	6410
Live Load	13660	7659
	24170 kgm	14069 kg

Web assumed as 770 × 8 = 61.6 cm², 1/8 web area = 7.70 cm²

Depth back to back of flange Ls 78 cm.

Effective depth say 78.0 − 2c2.15 = 73.7 cm

flange stress = $\frac{24170}{.737}$ = 32800 kg T or C

flange area reqd. = $\frac{32800}{1200}$ = 27.35

less 1/8 web area = − 7.70

19.65 cm² net

Use 2Ls 75 × 75 × 9 = 25.38 − 3.96 = 21.42 cm² net

22φ rivet no required for end connection = $\frac{14069}{2992}$ = 4.7 use 8 or 9 shop rivets

Approximate weight of Intermediate floor beam.

2flange	4Ls	75 × 75 × 9	@	9.96	× 6.30	=	251
1web	1Pl	770 × 8	@	48.36	× 6.30	=	305
Stiffeners	14Ls	75 × 65 × 8	@	8.29	× 0.78	=	91
end conn.	4Ls	125 × 90 × 10	@	16.10	× 0.762	=	49
fills	4Pls	125 × 9	@	8.83	× 0.62	=	22
Stringer base	4Pls	290 × 9	@	20.49	× 0.31	=	25
brackets	2		@	40.00		=	80
	Rivet heads and variation say						32
							855 kg

weight per lin. meter = $\frac{855}{6.6}$ = 129.5 kg

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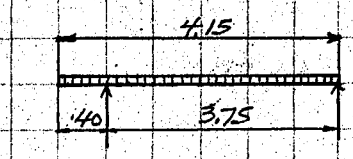
CHECKED BY H. J. DATE 8-11-17 PAGE NO 16

CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Design of End floor beam :- Span length 6.60 metres, spacing 3.75 metres.

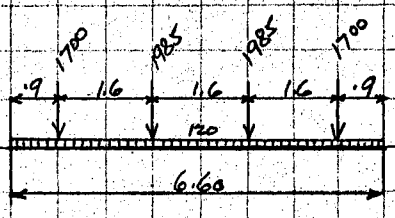
Dead Load :- Stringer concentration on floor beam.



Outside stringer So. $\frac{740 \times 4.15^2}{2 \times 3.75} = 1,700$ kg.

Inside " Si $\frac{864 \times 4.15^2}{2 \times 3.75} = 1985$ "

floor beam assumed as 120 kg per lin. m.



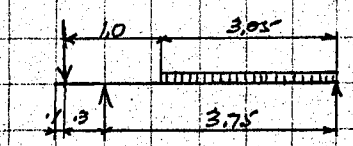
Dead Load moment

	1700	
	1985	
	3685 × 2.50 =	9210
	1,700 × 1.60 =	2,720
1/8 × 120 × 6.60² =		650
		7,140 kgm

Dead Load end shear

1/2 × 120 × 6.60 =		395
		3685
		4080 kg.

Live Load :- Stringer concentrations on floor beam.

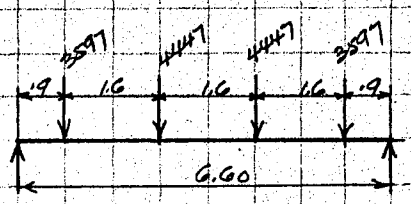


Outside Stringer So.

rear wheel	$\frac{2650 \times 4.05}{3.75}$ =	2860
unif. load	$\frac{894 \times 3.05^2}{2 \times 3.75}$ =	737
		3597 kg.

Inside Stringer Si.

rear wheel	$\frac{3200 \times 4.05}{3.75}$ =	3455
unif. load	$\frac{800 \times 3.05^2}{2 \times 3.75}$ =	992
		4447 kg.



Live Load moment

3597 × 0.90 =		3235
4447 × 2.50 =		11110
8044 kg		14345 kgm

Summary of moments and shears.

	moment	end shear
Dead Load	7140	4080
Live Load	14345	8044
	21485 kgm	12124 kg.

Use same details as for intermediate floor beam.
weight = 850 kg.

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CALCULATIONS FOR

Akigase - Bashi for Saitama - Ken.

Dead load of Bridge:

Floor	Floor and pavement	5.50 @ 430	=	2,365
	Copings	2 @ 210	=	420
	Handrails	2 @ 30	=	60
				2845
Structural steel parts.				
	Stringers	2 @ 50	=	100
		2 @ 40	=	80
	floor beams	855 ÷ 3.75	=	228
	lateral bracings say			70
	main trusses say	2 @ 500	=	1000
				1478
	Electric wiring and other fittings say			17
				4340 kg per lin meter of bridge.

For one truss 4340 ÷ 2 = 2,170 kg per lin meter.

Dead panel load = 2170 × 3,75 = 8,140 kg

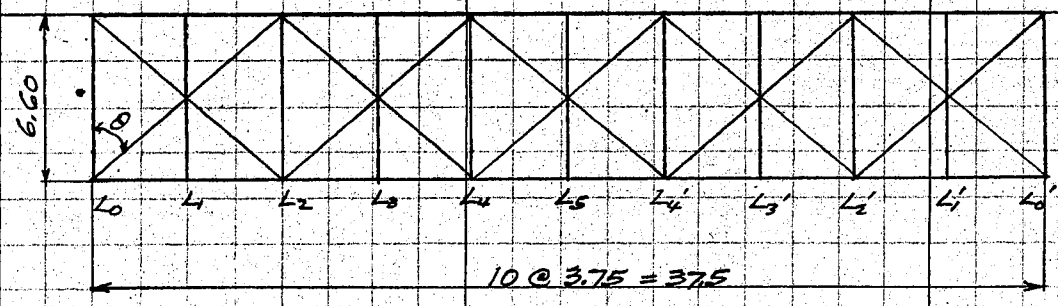
Seismic panel load = 8140 × 2 × 0,250 = 4,070 kg

Design of Lateral bracings.

Loads on lateral bracings:-

Wind load	On loaded chord	400
	on unloaded "	200
		600 kg per lin meter of bridge.
	Panel load	= 600 × 3,75 = 2,250 kg.
Seismic load.	Panel load	= 8140 × 2 × 0.25 = 4,070 kg

Seismic load governs the section of diagonal members.



Diagonal length = $\sqrt{7.50^2+6.60^2}$ = 9.99 m

Sec θ = 9.99 / 6.60 = 1.513

tan θ = 7.50 / 6.60 = 1.136

Diagonal stresses for seismic load. (as moving load.)

Panel			Shear		coef.		stress		Net area	19φ rivet use		
0-1	4070/10	× 45	= 18320	×	1.513	=	27700 kgT	÷1920 =	14.43 cm²	8.1 − 10 (6.5)	2L 125×75×9=3438-792=26[46 net	
1-2	"	× 36	14650				22200		11.57	7.3 − "	"	
2-3	"	× 28	11400				17250		8.98	5.1 − "	"	
3-4	"	× 21	8550				12940		6.74	3.8 − "	"	
4-5	"	× 15	6110				9250		4.82	2.7 − "	"	

Approximate weight of lateral bracing.

Diagonals	4 Ls	125×75×9	@ 13.50	× 4.50	= 2430
5 center connection	5 pls	810 - 9	@ 57.23	× 0.82	= 235
rivet heads say					55
					2720 kg.

Weight of lateral bracing per lin meter = 2720 / 37.5 = 72.5 kg.

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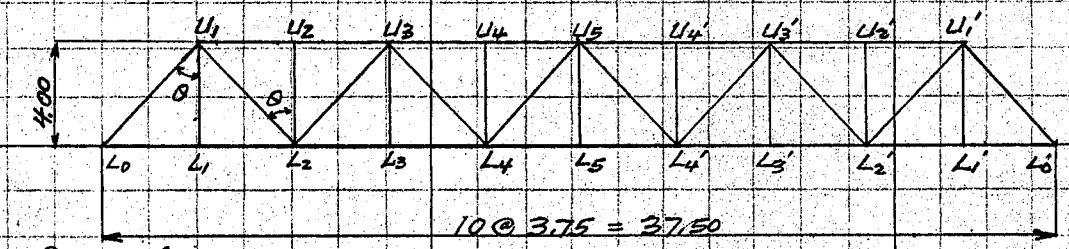
CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Design of Main Truss for simple span and suspended span.
Span length = 10 panels @ 3.75 = 37.50 meters;
Spacing of trusses assumed as 6.60 meters c to c;
Clear width of roadway = 5.50 meters clear;
Depth of truss assumed as 4.00 meters.



Diagonal length
$= \sqrt{4.00^2 + 3.75^2} = 5.483$ meters

$\sec\theta = \frac{5.483}{4.000} = 1.371$

$\tan\theta = \frac{3.750}{4.000} = 0.938$

Dead load stresses:-
Dead panel load $W_D = 8,140$ kg see on page 17
$W_D \sec\theta = 8,140 \times 1.371 = 11,170$ kg
$W_D \tan\theta = 8,140 \times 0.938 = 7,640$ "

Chord stresses:

	$W_D \tan\theta$			
$L_0 - L_2$	$7,640 \times$	4.5	= 34,400 kg	T
$U_1 - U_3$		8.0	= 61,100 "	C
$L_2 - L_4$		10.5	= 80,200 "	T
$U_3 - U_5$		12.0	= 91,700 "	C
$L_4 - L_4'$		12.5	= 95,500 "	T

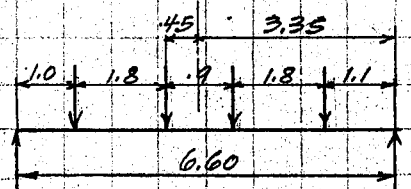
Diagonal stresses:

	$W_D \sec\theta$			
$L_0 - U_1$	$11,170 \times$	4.5	= 50,200 kg	C
$U_1 - L_2$		3.5	= 39,100 "	T
$L_2 - U_3$		2.5	= 27,900 "	C
$U_3 - L_4$		1.5	= 16,800 "	T
$L_4 - U_5$		0.5	= 5,600 "	C

Hangers $U_1 - L_1$, $U_3 - L_3$, and $U_5 - L_5$ 8,140 kg T

Live load stresses:- Uniform load on roadway $w = \frac{100,000}{170 + 37.5} = 482$ kg per square meter.

Motor truck rear wheel
Impact coef. $= \frac{20}{60 + 37.5} = 20.5\% = \frac{2250}{460}$
2,710 kg.
Front wheel with impact say $2710 \div 3$ = 903 "



Max. wheel concentration on truss:
Rear wheel $2710 \times 4 \times 3.35 \div 6.60$ = 5,500 kg
Front wheel $903 \times 4 \times 3.35 \div 6.60$ = 1,830 "
Uniform load 2.75×482 = 1,325 kg per lin. meter of truss.

$1325 \times 2.75 = 3,640$ kg
$1325 \times 0.25 = 330$ "
$1325 \times 3.75 = 4,970$ "

Unif. load	4,970	2,485				
"		2,305	1335	2,485	2,485	
"			10	320	2,485	4,790
Rear wheels			5,500			
Front wheels			365	1,465		
Summary	4,970 kg	4,790	7,210	4,270	4,970	4,970 kg

Now let us assume the panel concentration for live load as 4,970 kg on each panel point throughout and 7210 − 4970 = 2240 kg extra single concentration for motor truck loading.

$W_u = 4,970$ kg, $W_u \sec\theta = 4970 \times 1.371 = 6,810$ kg, $W_u \tan\theta = 4970 \times 0.938 = 4660$ kg
$W_c = 2,240$ ", $W_c \sec\theta = 2240 \times 1.371 = 3,070$ ", $W_c \tan\theta = 2240 \times 0.938 = 2100$ "

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CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Live load stress of truss members:-

Chord stresses:- Full load.

	Wu tan θ				Wc tan θ				Total Live load stresses	
L_0-L_2	4660 × 4.5	=	20,980	kg T	2,100 × 0.90 × 1	=	1,890	kg T	22870	kg T
U_1-U_3	8.0	=	37,250	" C	0.80 × 2	=	3360	" C	40,610	" C
L_2-L_4	10.5	=	48,950	" T	0.70 × 3	=	4440	" T	53360	" T
U_3-U_5	12.0	=	55,900	" C	0.60 × 4	=	5040	" C	60,940	" C
L_4-L_4'	12.5	=	58,300	" T	0.50 × 5	=	5250	" T	63,550	" T

Diagonal stresses:- Partial load.

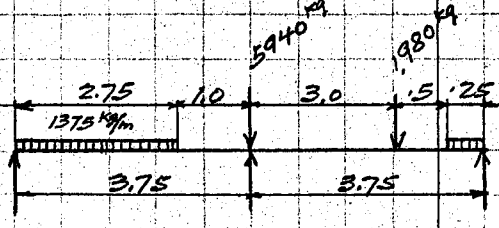
	Wu sec θ				Wc sec θ					
L_0-U_1	6810 × 4.5	=	30,650	kg C	3,070 × 0.90	=	2,760	kg C	33,410	kg C
U_1-L_2	3.6	=	24,520	" T	0.80	=	2,460	" T	26,980	" T
L_2-U_3	2.8	=	19,070	" C	0.70	=	2,150	" C	21,220	" C
U_3-L_4	2.1	=	14,300	" T	0.60	=	1,840	" T	16,140	" T
L_4-U_5	1.5	=	10,220	" C	0.50	=	1,540	" C	11,760	" C

Counter stress in diagonals:- Partial load.

	Wu sec θ				Wc sec θ					
L_0-U_1	6,810 × 0.00	=	0	kg	3,070 × 0.00	=	0	kg	0	kg
U_1-L_2	0.10	=	680	" C	0.10	=	310	" C	990	" C
L_2-U_3	0.30	=	2,040	" T	0.20	=	610	" T	2,650	" T
U_3-L_4	0.60	=	4,090	" C	0.30	=	920	" C	5,010	" C
L_4-U_5	1.00	=	6,810	" T	0.40	=	1,230	" T	8,040	" T

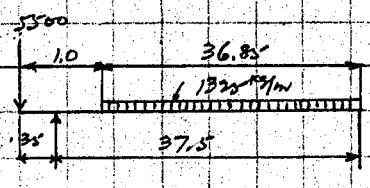
Hangers U_1-L_1, U_3-L_3, and U_5-L_5:-

Uniform load 500 kg/m², 500 × 2.75 = 1,375 kg per lin meter of truss.
Rear wheel of motor truck with impact = 2,925 kg
front " " " " " = 975 "
max. wheel concentration on truss,
Rear wheel 2925 × 4 × 3.35 ÷ 6.60 = 5940 kg.
front " 975 × 4 × 3.35 ÷ 6.60 = 1,980 "



Uniform load $\frac{1375 \times 2.75^2}{2 \times 3.75}$ = 1,390
" $\frac{1375 \times 0.25^2}{2 \times 3.75}$ = 10
Rear wheels = 5940
Front wheels $1980 \times \frac{0.75}{3.75}$ = 400
7,740 kg T

Max. load on shoe.
Dead load 38.3 m @ 2170 ÷ 2 = 41,600
Live load.



$\frac{1325 \times 36.85^2}{2 \times 37.5}$ = 24,000
$5500 \times \frac{37.85}{37.5}$ = 5560
29,560
71,160 kg

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CHECKED BY M. J. DATE 8-12-10 PAGE NO 20

CALCULATIONS FOR

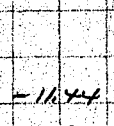
Akigase-Bashi for Saitama-ken.

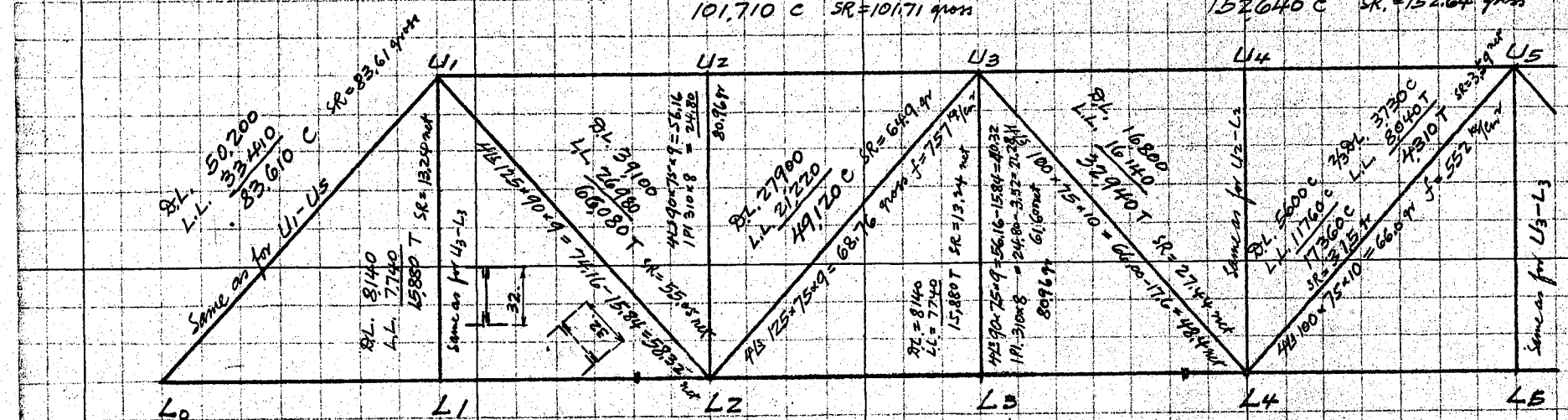
Summary of Dead and live load stresses.

	Members	Dead load	Live load	Total stress	Remarks
Lower chords	L_0-L_2	34,400 T	22,870 T	57,270 T	D.L. + L.L.
	L_2-L_4	80,200 T	53,360 T	133,560 T	"
	L_4-L_4'	95,500 T	63,550 T	159,050 T	"
Upper chords	U_1-U_3	61,100 C	40,610 C	101,710 C	"
	U_3-U_5	91,700 C	60,940 C	152,640 C	"
Diagonals	L_0-U_1	50,200 C	33,410 C	83,610 C	"
			0 T		
	U_1-L_2	39,100 T	26,980 T	66,080 T	"
			990 C	25,080 T	$\frac{2}{3}$ D.L. + L.L.
	L_2-U_3	27,900 C	21,220 C	49,120 C	D.L. + L.L.
			2,650 T	15,950 C	$\frac{2}{3}$ D.L. + L.L.
	U_3-L_4	16,800 T	16,140 T	32,940 T	D.L. + L.L.
			5,010 C	6,190 T	$\frac{2}{3}$ D.L. + L.L.
	L_4-U_5	5,600 C	11,760 C	17,360 C	D.L. + L.L.
			8,040 T	4,310 T	$\frac{2}{3}$ D.L. + L.L.
Hangers	U_1-L_1, U_3-L_3, U_5-L_5	8,140 T	7,740 T	15,880 T	D.L. + L.L.

Top chord U_1-U_5

1 Cov. pl. 540×11 = 59.40
2 L 300×90×9 = 97.14
156.54 cm² gross





D.L. 34,400
L.L. 22,870
57,270 T SR = 47.75 net

D.L. 80,200
L.L. 53,360
133,560 T SR = 111.40 net

D.L. 95,500
L.L. 63,550
159,050 T SR = 132.6 net

D.L. 61,100
L.L. 40,610
101,710 C SR=101.71 gross

D.L. 91,700
L.L. 60,940
152,640 C SR. = 152.64 gross

L_0-L_2
2L 100×100×13 = 48.62 − 11.44 = 37.18
2Pls 300 × 12 = 72.00 − 15.84 = 56.16
120.62 gr 93.34 net

L_2-L_4
2L 100×100×13 = 48.62 − 11.44 = 37.18
2Pls 300 × 12 = 72.00 − 15.84 = 56.16
2Pls 200 × 9 = 36.00 − 7.92 = 28.08
156.62 gr 121.42 net

L_4-L_4'
2L 100×100×13 = 48.62 − 11.44 = 37.18
2Pls 300 ×12 = 72.00 − 15.84 = 56.16
2Pls 200 ×13 = 52.00 − 11.44 = 40.56
172.62 gr 133.90 net

note :— all rivets 19 ϕ.
max. load on shoe = 71,160 kg for one shoe.

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CHECKED BY M. J. DATE 8-12-10 PAGE NO 21

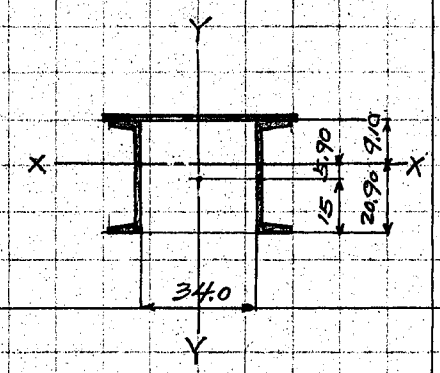
CALCULATIONS FOR

Akigase-Bashi for Saitama-ken

Section of truss members:

Top chord:

max. stress = 152,640 kg C



Assumed section.

1 Cov. pl.	540 × 11	=	59.40
2 [	300 × 90 × 9	=	97.14
			156.54 cm² gross.

Eccentricity $\frac{59.40 \times 15.55}{156.54} = 5.90$ cm

Moment of inertia:

I_x	1 Pl.	59.40×9.65^2	=	5530
	2 [	$97.14 \times 5.90^2 + 6435 \times 2$	=	16250
			I_x =	21,780 cm⁴

I_y	1 Pl.	$1.1 \times 54^3 \div 12$	=	14430
	2 [	$97.14 \times 19.23^2 + 325 \times 2$	=	36570
			I_y =	51,000 cm⁴

Radius of gyration.

$r_x = \sqrt{\frac{21780}{156.54}} = 11.78$ cm, $\frac{l}{r_x} = \frac{375}{11.78} = 31.8$

$r_y = \sqrt{\frac{51000}{156.54}} = 18.05$ " , $\frac{l}{r_y} = \frac{750}{18.05} = 41.6$

Allowable unit stress = $1500(1 - 0.0055 \times 41.6) = 1157$
use 1000 kg/cm² C

Assumed section good for 156,540 kg C

End post. L_0-U_1 83,610 kg C

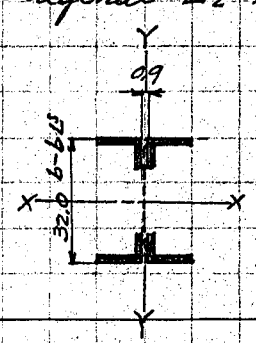
Use the same section as for top chord.

$l/r_x = \frac{548.3}{11.78} = 46.6$, $l/r_y = \frac{548.3}{18.05} = 30.4$

Allowable unit stress = $1500(1 - 0.0055 \times 46.6) = 1115$
use 1000 kg/cm² C

good for 156,540 kg C

Diagonal L_2-U_3 49,120 kg C



2 L's 125 × 75 × 9 = 68.76 cm² gross.

r_y = 6.1 cm, $l/r_y = \frac{548.3}{6.1} = 90.0$

Allowable unit stress = $1500(1 - 0.0055 \times 90) = 757$ kg/cm² C

Above section good for $68.76 \times 757 = 52{,}000$ kg C

L_4-U_5 17,360 kg C

2 L's 100 × 75 × 10 = 66.00 cm² gross

r_y = 4.77 cm $l/r_y = \frac{548.3}{4.77} = 115$

Allowable unit stress = $1500(1 - 0.0055 \times 115) = 552$ kg/cm² C

Above section good for $66 \times 552 = 36{,}400$ kg C.

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Resistance to buckling of top chord.

General formula:-

$$A_{req} = \frac{\psi^2 S^2 l}{4EJ}$$

where A_{req} = lateral stiffness of the cross rahmen, consisting from floor beam and vertical posts (kg/cm);
ψ = factor of safty preferably larger than 5;
S = chord stress in kg = 152,640
l = panel length in cm = 375;
E = modulus of elasticity in kg/cm² = 2,100,000;
J = moment of inertia of top chord section with respect to its vertical axis in cm⁴.

For the truss in hand, we have as follows:

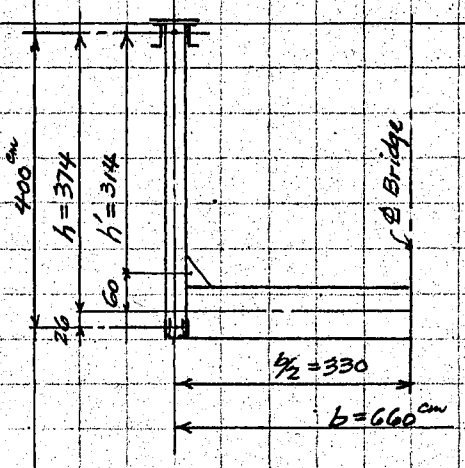
J = 51,000 cm⁴ see on page 21.

$$A_{req} = \frac{5^2 \times 152640^2 \times 375}{4 \times 2{,}100{,}000 \times 51{,}000} = 510 \text{ kg/cm}$$

Stiffness of cross rahmen:

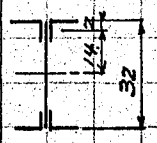
$$A = \frac{1}{\delta} = \frac{1}{\frac{h'^3}{3EJ_v'} + \frac{h^2 b}{2EJ_q}}$$

where h' = 314 cm;
$J_v' \doteqdot J_v$ = moment of inertia of vertical posts;
h = 374 cm;
b = 660 cm;
J_q = moment of inertia of cross beam.



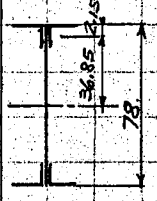
J_v

4Ls 90×75×9	= 56.16	68×4 + 56.16×14²	= 11270
1web 310×8	= 24.80	0.8×31³ ÷12	= 1990
	80.96 cm²		J_v = 13,260 cm⁴



J_q

4Ls 75×75×9	= 50.76	64×4 + 50.76×36.85²	= 69160
1web 770×8	= 61.60	0.8×77³ ÷12	= 30440
	112.36 cm²		J_q = 99,600 cm⁴



$$A = \frac{1}{\frac{314^3}{3 \times 2{,}100{,}000 \times 13260} + \frac{374^2 \times 660}{2 \times 2{,}100{,}000 \times 99600}} = \frac{1}{0.000371 + 0.000221} = \frac{1}{0.000592}$$

$$= 1690 \text{ kg/cm}$$

Actual factor of safty $S = \sqrt{\frac{25A}{510}} = \sqrt{\frac{25 \times 1690}{510}} = 9.10$ ✓

The truss has ample stiffness against lateral buckling.

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CHECKED BY M. J. DATE 8-12-12 PAGE NO. 23.

CALCULATIONS FOR

Ukigase Bashi for Daitama-ken

Deflection of the truss at center of span.

General equation to the deflection:-

$$\Delta = \Sigma \frac{SL}{EA} \cdot T \quad \text{or} \quad \frac{1}{E} \Sigma \frac{SL}{A} \cdot T ,$$

where Δ = Deflection at any point in cm;
S = stress of each member in kg;
T = stress of each member in kg due to a unit load on the panel point at which deflection is desired, in the direction of the deflection;
L = length of each member in cm;
A = gross sectional area of each member in cm^2;
E = modulus of elasticity of Steel in kg/cm^2 = 2,100,000.

Members	L	A	L/A	S_D	T	$\frac{S_D L}{A} T$
$L_0 - L_2$	750	120.62	6.218	34,400	0.4690	100,200
$L_2 - L_4$	"	156.62	4.788	80,200	1.4060	540,000
$L_4 - L_6$	375	172.62	2.174	95,500	2.3450	486500
$U_0 - U_3$	750	156.54	4.790	- 61,100	- 0.9380	274,500
$U_3 - U_5$	"	"	4.790	- 91,700	- 1.8760	824,000
$L_0 - U_1$	548.3	156.54	3.502	- 50,200	- 0.6855	120500
$U_1 - L_2$	"	74.16	7.400	39,100	0.6855	198,500
$L_2 - U_3$	"	68.76	7.980	- 27,900	- 0.6855	152,600
$U_3 - L_4$	"	66.00	8.310	16,800	0.6855	95700
$L_4 - U_5$	"	66.00	8.310	- 5,600	- 0.6855	31,900
$U_5 - L_5$	400 × 1/2	80.96	2.470	8,140	1.0000	20100
						2844500
						5689,000

Summary for the entire span

Dead load Deflection = $\frac{5689,000}{2,100,000}$ = 2.71

Mnif. Live load " = $2.71 \times \frac{4970}{8140}$ = 1.65

4.36 cm

Deflection ratio = $\frac{4.36}{3750} = \frac{1}{860}$

Deflection Increment during erection,

					to round no.
Due to Structural steel	$\frac{2.71}{4340}$	× 1495	= 0.93	cm ≒	0.9 cm
floor slab and copings	"	× 2180	= 1.36	"	1.4
pavement	"	× 605	= 0.38	"	0.4
handrails	"	× 60	= 0.04	"	—
			2.71 cm		2.7

Deflection for D.L. + 1/2 L.L. = 2.71 + 0.83 = 3.54 cm

4.0 cm camber shall be given to the main truss.

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CHECKED BY M. J. DATE 8-12-12 PAGE NO 24

CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Approximate weight of structural steel for main truss.

	no.		Section		Length		volume (m. cm²)
Top chord	1	@	156.54 cm²	×	30.000 m	=	4,695
end post	2	@	〃		5.483		1,718
bottom chord	2	@	120.62		7.500		1,810
〃	2	@	156.62		〃		2350
〃	1	@	172.62		〃		1295
diagonal	2	@	74.16		5.483		814
〃	2	@	68.76		〃		754
〃	4	@	66.00		〃		1447
vertical	9	@	80.96		4.000		2915
							17,798 @ 0.785 = 13970

Details including rivet heads say 35-% = 4890

18860 kg

weight of truss per lin. meter = $\frac{18860}{37.5}$ = 503 kg.

Summary of structural steel for one simple or suspended span.

Stringers	2 @ 50 = 100		
〃	2 @ 40 = 80		
	180 × 38.2 =		6880
floor beams	11 @ 855	=	9400
lateral bracing		=	2720
main trusses	2 @ 18860	=	37720
metals for expansion joint say		=	680
Shoes say	4 @ 450	=	1800
			59200 kg.

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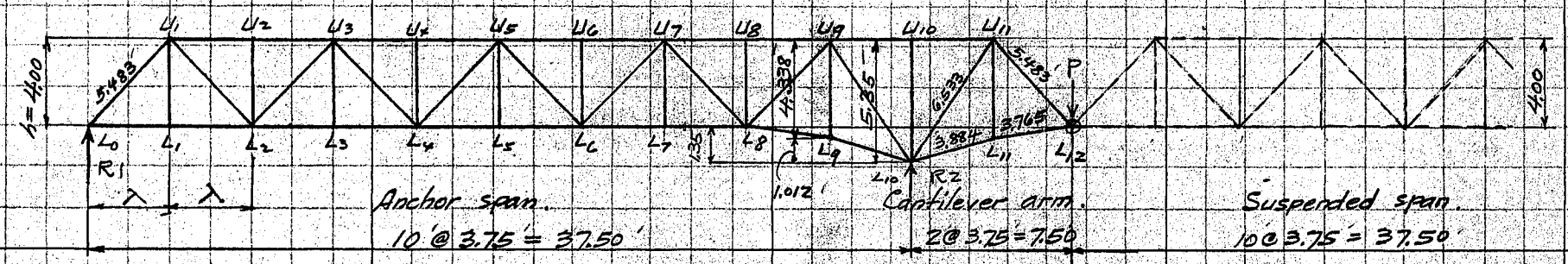
CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Design of Main Truss for Anchor span with a cantilever arm.
General dimensions of truss as follows:



Diagonal length. L_0-U_1 to L_8-U_9 Same as for suspended span = 5.483' meters sec θ = 1.371
Sec θ_1 = 1.371', tan θ_1 = 0.938', see on page 18.

U_9-L_{10} and $L_{10}-U_{11}$ $\sqrt{3.75^2+5.35^2}$ = 6.533' meters

Chord length L_8-L_9 and $L_{11}-L_{12}$ $\sqrt{3.75^2+0.338^2}$ = 3.765'
L_9-L_{10} and $L_{10}-L_{11}$ $\sqrt{3.75^2+1.012^2}$ = 3.884'

Lever arms. for L_8-L_9 and $L_{11}-L_{12}$ $3.750 \times \frac{4.338}{3.765}$ = 4.321' ✓

for L_9-L_{10} and $L_{10}-L_{11}$ $3.750 \times \frac{4.338}{3.884}$ = 4.188'

for L_8-U_9 center of moment $3.750' \times \frac{4.00'}{0.388'}$ = 38.660' from left side of L_8
or $38.660' - 8 \times 3.750'$ = 8.660' " " " " L_0
lever arm $4.000' \times \frac{42.410'}{5.483'}$ = 30.939'

for U_9-L_{10} center of moment $3.750' \times \frac{4.338'}{1.012'}$ = 16.075' from left side of L_9.
or $33.750' - 16.075'$ = 17.675' " right side " L_0
lever arm $5.350' \times \frac{16.075'}{6.533'}$ = 13.164'

for $L_{10}-U_{11}$ refer to U_9-L_{10}

for $U_{11}-L_{12}$ lever arm $4.321' \times \frac{3.765'}{5.483'}$ = 2.967' ✓ center of m. at L_{11}.

Vertical components of L_8-L_9 $\frac{0.338'}{3.765'}$ = 0.090'
" " " L_9-L_{10} $\frac{1.012'}{3.884'}$ = 0.261'

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Stresses of each member for unit load at each panel point.

Top chord members.

M_1-M_3	Load on L_2	$-\frac{8}{10}\cdot\frac{2\lambda}{h}$	$=-\frac{8}{10}\cdot\frac{2\times3.75}{4.00}$	$=$	-8×0.1875
	Load on L_1	$-\frac{1}{10}\cdot\frac{8\lambda}{h}$	$=-\frac{1}{10}\cdot\frac{8\times3.75}{4.00}$	$=$	-1×0.7500
	Load on L_{12}	$\frac{2}{10}\cdot\frac{2\lambda}{h}$	$=\frac{2}{10}\cdot\frac{2\times3.75}{4.00}$	$=$	2×0.1875
M_3-M_5	Load on L_4	$-\frac{6}{10}\cdot\frac{4\lambda}{h}$	$=-\frac{6}{10}\cdot\frac{4\times3.75}{4.00}$	$=$	-6×0.3750
	Load on L_3	$-\frac{3}{10}\cdot\frac{6\lambda}{h}$	$=-\frac{3}{10}\cdot\frac{6\times3.75}{4.00}$	$=$	-3×0.5625
	Load on L_{12}	$\frac{2}{10}\cdot\frac{4\lambda}{h}$	$=\frac{2}{10}\cdot\frac{4\times3.75}{4}$	$=$	2×0.3750
M_5-M_7	Load on L_6	$-\frac{4}{10}\cdot\frac{6\lambda}{h}$	$=-\frac{4}{10}\cdot\frac{6\times3.75}{4.00}$	$=$	-4×0.5625
	Load on L_5	$-\frac{5}{10}\cdot\frac{4\lambda}{h}$	$=-\frac{5}{10}\cdot\frac{4\times3.75}{4.00}$	$=$	-5×0.3750
	Load on L_{12}	$\frac{2}{10}\cdot\frac{6\lambda}{h}$	$=\frac{2}{10}\cdot\frac{6\times3.75}{4.00}$	$=$	2×0.5625
M_7-M_9	Load on L_8	$-\frac{2}{10}\cdot\frac{8\lambda}{h}$	$=-\frac{2}{10}\cdot\frac{8\times3.75}{4.00}$	$=$	-2×0.7500
	Load on L_7	$-\frac{7}{10}\cdot\frac{2\lambda}{h}$	$=-\frac{7}{10}\cdot\frac{2\times3.75}{4.00}$	$=$	-7×0.1875
	Load on L_{12}	$\frac{2}{10}\cdot\frac{8\lambda}{h}$		$=$	2×0.7500
M_9-M_{11}	Load on L_{10}				0.0000
	Load on L_9				0.0000
	Load on L_{12}	$\frac{2}{10}\cdot\frac{10\lambda}{h'}$	$=\frac{2}{10}\cdot\frac{10\times3.75}{5.35}$	$=$	2×0.7009

Bottom chord members.

L_0-L_2	Load on L_1	$\frac{9}{10}\cdot\frac{\lambda}{h}$	$=\frac{9}{10}\cdot\frac{3.75}{4.00}$	$=$	9×0.0938
	Load on L_{12}	$-\frac{2}{10}\cdot\frac{\lambda}{h}=$		$=$	-2×0.0938
L_2-L_4	Load on L_3	$\frac{7}{10}\cdot\frac{3\lambda}{h}$	$=\frac{7}{10}\cdot\frac{3\times3.75}{4.00}$	$=$	7×0.2813
	Load on L_2	$\frac{2}{10}\cdot\frac{7\lambda}{h}$	$=\frac{2}{10}\cdot\frac{7\times3.75}{4.00}$	$=$	2×0.6563
	Load on L_{12}	$-\frac{2}{10}\cdot\frac{3\lambda}{h}=$		$=$	-2×0.2813
L_4-L_6	Load on L_5	$\frac{5}{10}\cdot\frac{5\lambda}{h}$	$=\frac{5}{10}\cdot\frac{5\times3.75}{4.00}$	$=$	5×0.4688
	Load on L_4	$\frac{4}{10}\cdot\frac{5\lambda}{h}$	$=\frac{4}{10}\cdot\frac{5\times3.75}{4.0}$	$=$	4×0.4688
	Load on L_{12}	$-\frac{2}{10}\cdot\frac{5\lambda}{h}$		$=$	-2×0.4688

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CALCULATIONS FOR

Ukegase-Bashi for Saitama-Ken.

L_6-L_8	Load on L_7	$\frac{3}{10}\cdot\frac{7\lambda}{h}$	$=\frac{3}{10}\cdot\frac{7\times3.75}{4.00}$	$=3\times0.6563$
	Load on L_6	$\frac{6}{10}\cdot\frac{3\lambda}{h}$		$=6\times0.2813$
	Load on L_{12}	$-\frac{2}{10}\cdot\frac{7\lambda}{h}$		$=-2\times0.6563$
L_8-L_9	Load on L_9	$\frac{1}{10}\cdot\frac{9\lambda}{arm}$	$=\frac{1}{10}\cdot\frac{9\times3.75}{4.321}$	$=1\times0.7811$
	Load on L_8	$\frac{8}{10}\cdot\frac{\lambda}{arm}$	$=\frac{8}{10}\cdot\frac{3.75}{4.321}$	$=8\times0.0868$
	Load on L_{12}	$-\frac{2}{10}\cdot\frac{9\lambda}{arm}$		$=-2\times0.7811$
L_9-L_{10}	Load on L_9	$\frac{1}{10}\cdot\frac{9\lambda}{arm}$	$=\frac{1}{10}\cdot\frac{9\times3.75}{4.188}$	$=1\times0.8059$
	Load on L_8	$\frac{8}{10}\cdot\frac{\lambda}{arm}$	$=\frac{8}{10}\cdot\frac{3.75}{4.188}$	$=8\times0.0895$
	Load on L_{12}	$-\frac{2}{10}\cdot\frac{9\lambda}{arm}$		$=-2\times0.8059$
$L_{10}-L_{11}$	Load on L_{12}	$-\frac{\lambda}{arm}$	$=-\frac{3.75}{4.188}$	$=-0.8954$
	Load on L_9 or L_{11}			0.0000
$L_{11}-L_{12}$	Load on L_{12}	$-\frac{\lambda}{arm}$	$=-\frac{3.75}{4.321}$	$=-0.8679$
	Load on L_{11}			0.0000

Diagonal members.

L_0-U_1	Load on L_1	$-\frac{9}{10}\times1.371$	$=-9\times0.1371$
	Load on L_{12}	$2/10\times1.371$	$=2\times0.1371$
U_1-L_2	Load on L_2	$8/10\times1.371$	$=8\times0.1371$
	" L_{12}		$=-2\times0.1371$
	" L_1	$-1/10\times1.371$	$=-1\times0.1371$
L_2-U_3	" L_3	$-7/10\times1.371$	$=-7\times0.1371$
	" L_2	$2/10\times$ "	$=2\times0.1371$
	" L_{12}		$=2\times0.1371$
U_3-L_4	" L_4	$6/10\times1.371$	$=6\times0.1371$
	" L_3	$-3/10\times$ "	$=-3\times0.1371$
	" L_{12}	$-2/10\times$ "	$=-2\times0.1371$
L_4-U_5	" L_5		$=-5\times0.1371$
	" L_4		$=4\times0.1371$
	" L_{12}		$=2\times0.1371$
U_5-L_6	" L_6		$=4\times0.1371$
	" L_5		$=-5\times0.1371$
	" L_{12}		$=-2\times0.1371$

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CALCULATIONS FOR

Ukigase-Bashi for Saitama-Ken.

L_6-U_7	Load on	L_7	$-3/10 \times 1.371$	$= -3 \times 0.1371$	
	"	L_6	$6/10 \times$ "	$= 6 \times 0.1371$	
	"	L_{12}	$2/10 \times$ "	$= 2 \times 0.1371$	
U_7-L_8	"	L_8		$= 2 \times 0.1371$	
	"	L_7		$= -7 \times 0.1371$	
	"	L_{12}		$= -2 \times 0.1371$	
L_8-U_9	"	L_9	$-1/10 \times \frac{8.660}{30.939}$	$= -1 \times 0.0280$	
	"	L_8	$8/10 \times \frac{46.160}{30.939}$	$= 8 \times 0.1492$	
	"	L_{12}	$2/10 \times \frac{8.660}{30.939}$	$= 2 \times 0.028$	
U_9-L_{10}	"	L_{10}			0.0000
	"	L_9	$-9/10 \times \frac{19.825}{13.164}$	$= -9 \times 0.1506$	
	"	L_{12}	$2/10 \times \frac{17.675}{13.164}$	$= 2 \times 0.1343$	
$L_{10}-U_{11}$	"	L_{12}	$-1 \times \frac{12.325}{13.164}$	$=$	-0.9363
	"	L_{11}	$-1 \times \frac{16.075}{13.164}$	$=$	-1.2211
$U_{11}-L_{12}$	"	L_{12}	$\frac{3.750}{2.967}$	$=$	1.2639

Vertical members.

U_1-L_1, U_3-L_3, U_5-L_5, and L_7-L_7 $=$ 1.0000

U_9-L_9 Load on L_9

vertical component of L_8-L_9 $-0.7811 \times 0.090 = -0.0703$

" " " L_9-L_{10} $0.8059 \times 0.261 = 0.2103$

Load unity 0.1400

1.0000

1.1400

Load on L_{12}	
1.562×0.090	$= 0.1406$
-1.6118×0.261	$= -0.4207$
	-0.2801

Load on L_8

vertical component of L_8-L_9 $-8 \times 0.0868 \times 0.090 = -8 \times 0.0078$

" " " L_9-L_{10} $8 \times 0.0895 \times 0.261 = 8 \times 0.0234$

8×0.0156

$U_{11}-L_{11}$ Load on L_{12}

vertical comp. of $L_{10}-L_{11}$ $0.8954 \times 0.261 = -0.2337$

" " " $L_{11}-L_{12}$ $0.8679 \times 0.090 = +0.0781$

-0.1556

Load on L_{11}

vertical comp. of $L_{10}-L_{11}$ & $L_{11}-L_{12}$ are zero respectively

$\therefore$ stress on $U_{11}-L_{11}$ $= 1.000$

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CALCULATIONS FOR

Ukigase-Bashi for Saitama-ken

Influence surface of Top chord members.

Load on	M_1-M_3	M_3-M_5	M_5-M_7	M_7-M_9	M_9-M_{11}
1	- 0.7500	-0.5625	- 0.3750	- 0.1875	0.0000
2	- 1.5000	- 1.1250	- 0.7500	- 0.3750	〃
3	- 1.3125	- 1.6875	- 1.1250	- 0.5625	〃
4	- 1.1250	- 2.2500	- 1.5000	- 0.7500	〃
5	- 0.9375	- 1.8750	- 1.8750	- 0.9375	〃
6	- 0.7500	- 1.5000	- 2.2500	- 1.1250	〃
7	- 0.5625	- 1.1250	- 1.6875	- 1.3125	〃
8	- 0.3750	- 0.7500	- 1.1250	- 1.5000	〃
9	- 0.1875	-0.3750	- 0.5625	- 0.7500	〃
10	0.0000	0.0000	0.0000	0.0000	〃
11	0.1875	0.3750	0.5625	0.7500	0.7009
12	(0.3750)	(0.7500)	(1.1250)	(1.5000)	(1.4018)
Load 5.5 on 12	2.0625	4.1250	6.1875	8.2500	7.7099
Σ	- 7.500	- 11.250	- 11.250	- 7.500	- 0.000
Σ	+ 2.250	+ 4.500	+ 6.750	+ 9.000	+ 8.411
Full load	- 5.250	- 6.750	- 4.500	1.500	8.411
	0.375	0.750	1.125	1.500	1.402

Influence surface of Bottom chord members.

Load on	L_0-L_2	L_2-L_4	L_4-L_6	L_6-L_8	L_8-L_9	L_9-L_{10}	$L_{10}-L_{11}$	$L_{11}-L_{12}$
1	0.8442	0.6563	0.4688	0.2813	0.0868	0.0895	0.0000	0.0000
2	0.7504	1.3126	0.9376	0.5626	0.1736	0.1790	〃	〃
3	0.6566	1.9691	1.4064	0.8439	0.2604	0.2685	〃	〃
4	0.5628	1.6878	1.8752	1.1252	0.3472	0.3580	〃	〃
5	0.4690	1.4065	2.3440	1.4065	0.4340	0.4475	〃	〃
6	0.3752	1.1252	1.8752	1.6878	0.5208	0.5370	〃	〃
7	0.2814	0.8439	1.4064	1.9689	0.6076	0.6265	〃	〃
8	0.1876	0.5626	0.9376	1.3126	0.6944	0.7160	〃	〃
9	0.0938	0.2813	0.4688	0.6563	0.7811	0.8059	〃	〃
10	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
11	- 0.0938	- 0.2813	- 0.4688	- 0.6563	- 0.7811	- 0.8059	0.0000	0.0000
12	-(0.1876)	-(0.5626)	-(0.9376)	-(1.3126)	-(1.5622)	-(1.6118)	-(0.8954)	-(0.8679)
Load 5.5 on 12	- 1.0318	- 3.0943	- 5.1568	- 7.2193	- 8.5921	- 8.8649	- 4.9247	- 4.7735
Σ	+ 4.221	+ 9.845	+ 11.720	+ 9.845	+ 3.906	+ 4.0287	+ 0.000	+ 0.000
Σ	- 1.126	- 3.376	- 5.626	- 7.876	- 9.373	- 9.671	- 4.925	- 4.774
Full load	+ 3.095	+ 6.469	+ 6.094	+ 1.969	- 5.467	- 5.643	- 4.925	- 4.774
	- 0.188	- 0.563	- 0.938	- 1.313	- 1.562	- 1.612	- 0.895	- 0.868

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken

Influence Surface of Diagonal members

Load on	L_0-U_1	U_1-L_2	L_2-U_3	U_3-L_4	L_4-U_5	U_5-L_6
1	− 1.2339	− 0.1371	0.1371	− 0.1371	0.1371	− 0.1371
2	− 1.0968	1.0968	0.2742	− 0.2742	0.2742	− 0.2742
3	− 0.9597	0.9597	− 0.9597	− 0.4113	0.4113	− 0.4113
4	− 0.8226	0.8226	− 0.8226	0.8226	0.5484	− 0.5484
5	− 0.6855	0.6855	− 0.6855	0.6855	− 0.6855	− 0.6855
6	− 0.5484	0.5484	− 0.5484	0.5484	− 0.5484	0.5484
7	− 0.4113	0.4113	− 0.4113	0.4113	− 0.4113	0.4113
8	− 0.2742	0.2742	− 0.2742	0.2742	− 0.2742	0.2742
9	− 0.1371	0.1371	− 0.1371	0.1371	− 0.1371	0.1371
10	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
11	0.1371	− 0.1371	0.1371	− 0.1371	0.1371	− 0.1371
12	(0.2742)	−(0.2742)	(0.2742)	− (0.2742)	(0.2742)	−(0.2742)
Load 5.5 on 12	1.5081	− 1.5081	1.5081	− 1.5081	1.5081	− 1.5081
Σ	− 6.170	+ 4.936	− 3.839	+ 2.879	− 2.057	+ 1.371
Σ	+ 1.645	− 1.782	2.057	− 2.468	+ 3.016	− 3.702
Full load	− 4.525	+ 3.154	− 1.782	+ 0.411	+ 0.959	− 2.331
	+ 0.274	− 0.274	0.274	− 0.274	+ 0.274	− 0.274

Load on	L_6-U_7	U_7-L_8	L_8-U_9	U_9-L_{10}	$L_{10}-U_{11}$	$U_{11}-L_{12}$
1	0.1371	− 0.1371	0.1492	− 0.1506	0.0000	0.0000
2	0.2742	− 0.2742	0.2984	− 0.3012	〃	〃
3	0.4113	− 0.4113	0.4476	− 0.4518	〃	〃
4	0.5484	− 0.5484	0.5968	− 0.6024	〃	〃
5	0.6855	− 0.6855	0.7460	− 0.7530	〃	〃
6	0.8226	− 0.8226	0.8952	− 0.9036	〃	〃
7	− 0.4113	− 0.9597	1.0444	− 1.0542	〃	〃
8	− 0.2742	0.2742	1.1936	− 1.2048	〃	〃
9	− 0.1371	0.1371	− 0.0280	− 1.3554	〃	〃
10	0.0000	0.0000	0.0000	0.0000	0.0000	〃
11	0.1371	− 0.1371	0.0280	0.1343	− 1.2211	〃
12	(0.2742)	− (0.2742)	(0.0560)	(0.2686)	−(0.9363)	(1.2639)
Load 5.5 on 12	1.5081	− 1.5081	0.3080	1.4773	− 5.1497	6.9515
Σ	+ 4.524	− 5.484	+ 5.707	− 6.777	+ 0.000	+ 6.952
Σ	− 0.823	+ 0.411	− 0.028	+ 1.612	− 6.371	− 0.000
Full load	+ 3.701	− 5.073	+ 5.679	− 5.165	− 6.371	+ 6.952
	+ 0.274	+ 0.274	+ 0.056	+ 0.269	− 0.936	+ 1.264

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken

Influence surface of Vertical members.

Load on	$M_9 - L_9$	$M_{11} - L_{11}$
1	0.0156	0.0000
2	0.0312	〃
3	0.0468	〃
4	0.0624	〃
5	0.0780	〃
6	0.0936	〃
7	0.1092	〃
8	0.1248	〃
9	1.1400	〃
10	0.0000	〃
11	− 0.1400	1.0000
12	−(0.2801)	−(0.1556)
load 5.5 on 12	− 1.5406	− 0.8558
Σ	+ 1.702	+ 1.000
Σ	− 1.681	− 0.856
Full load	+ 0.021	+ 0.144

Influence surface of Reactions R_1 and R_2.

Load on	R_2	R_1
1	0.1000	0.9000
2	0.2000	0.8000
3	0.3000	0.7000
4	0.4000	0.6000
5	0.5000	0.5000
6	0.6000	0.4000
7	0.7000	0.3000
8	0.8000	0.2000
9	0.9000	0.1000
10	1.0000	0.0000
11	1.1000	− 0.1000
12	(1.2000)	−(0.2000)
load 5.5 on 12	6.6000	− 1.1000
Σ	+ 13.200	+ 4.500
Σ	− 0.000	− 1.200
Full load	+ 13.200	+ 3.300

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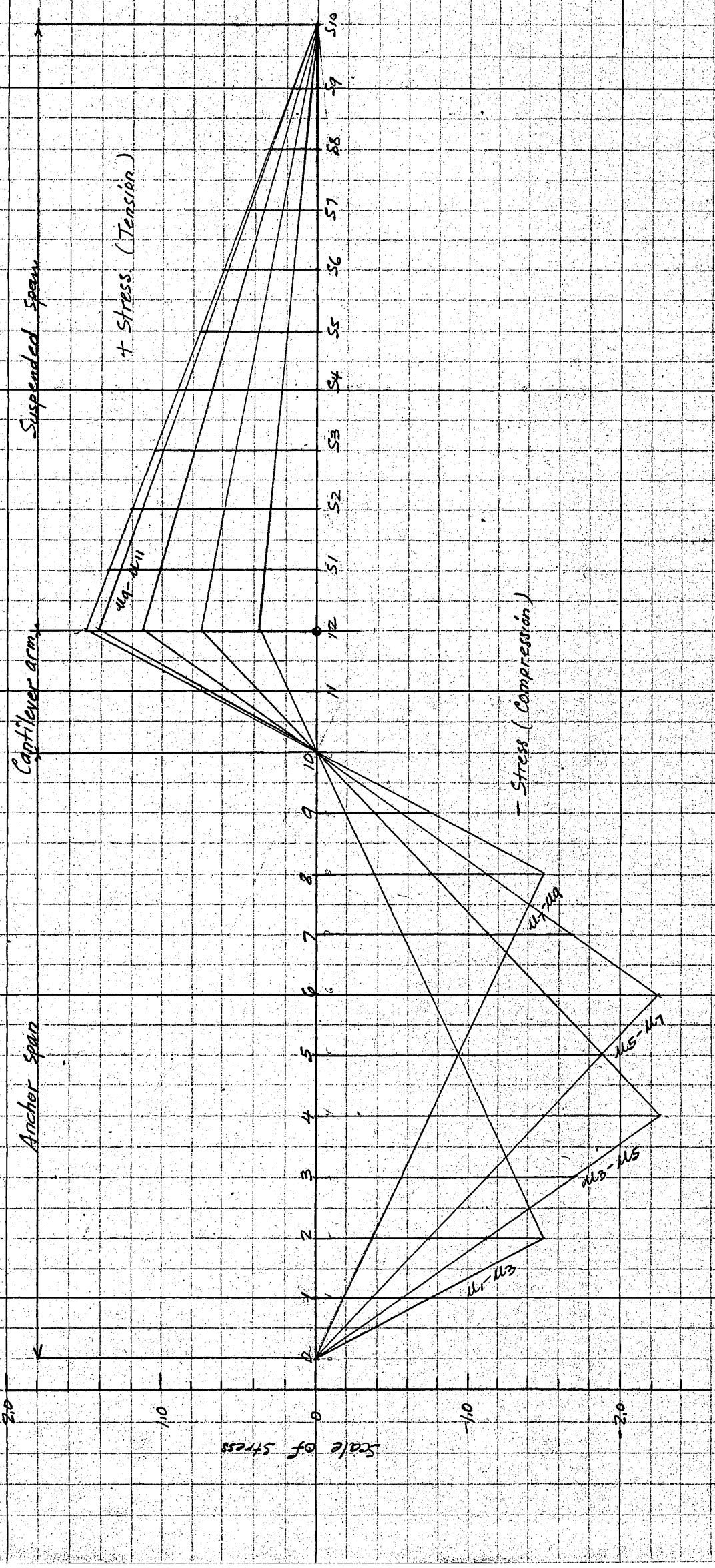
CALCULATIONS FOR

Akigase - Bashi for Saitama - Ken

Influence Diagram of Top chord members.



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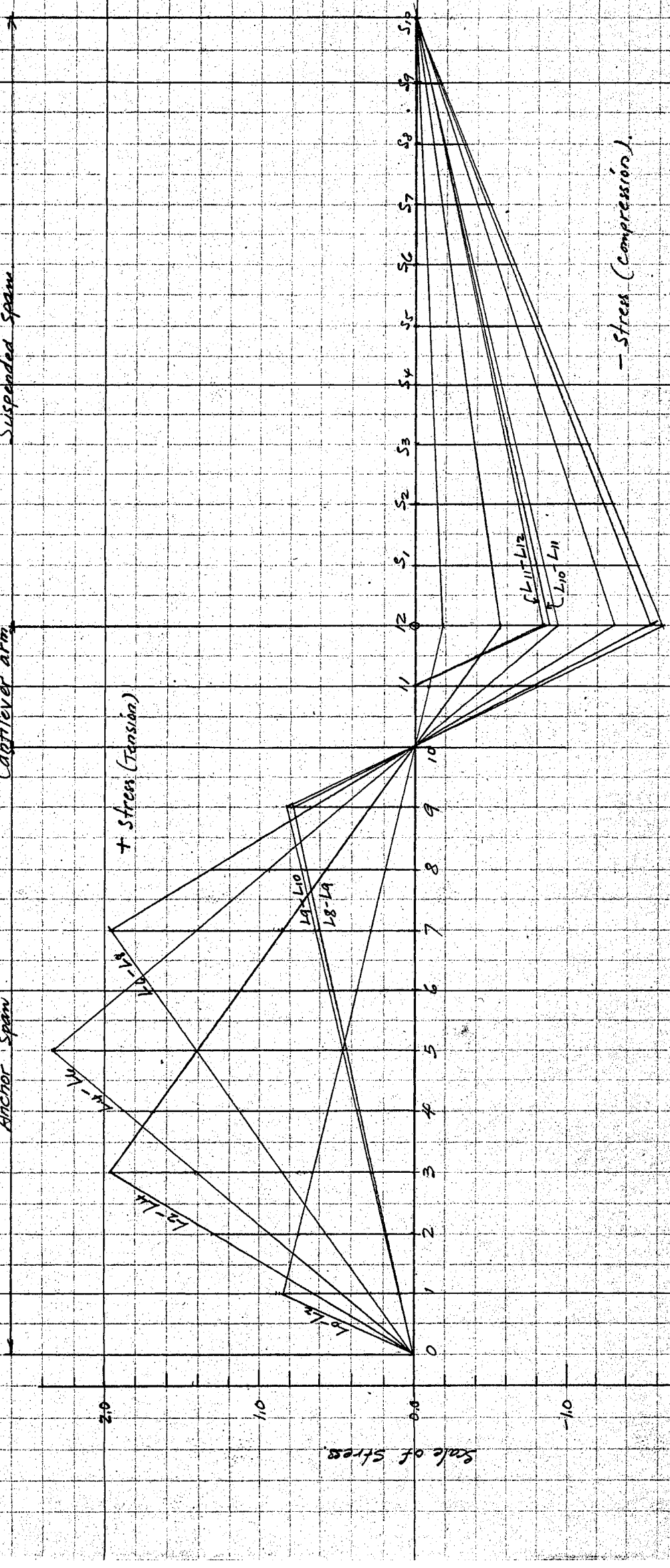
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken

Influence Diagram of Bottom Chord members.



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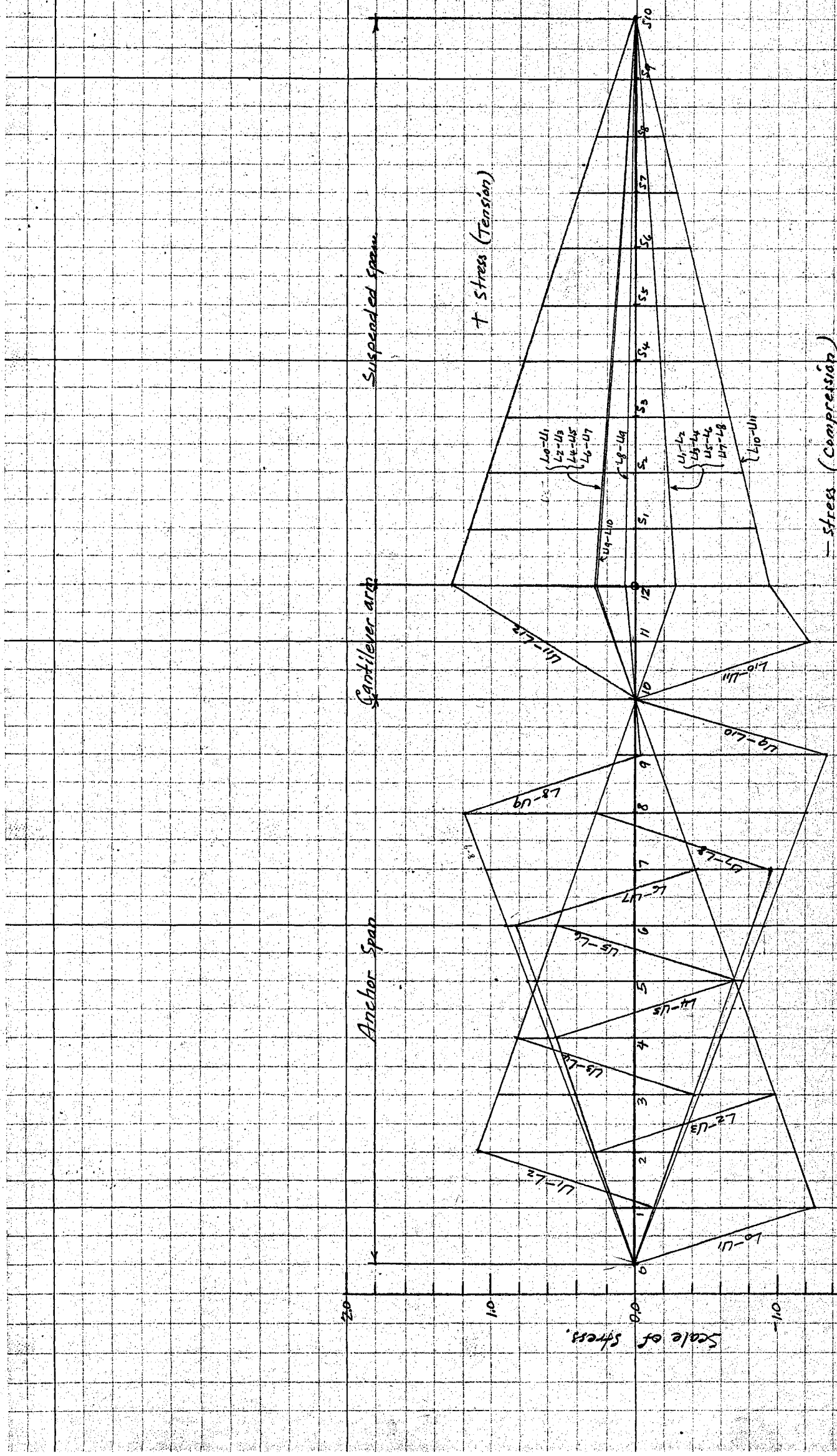
CALCULATIONS FOR

Ushigase-Bashi for Saitama-Ken.

Influence Diagram of Diagonal members.



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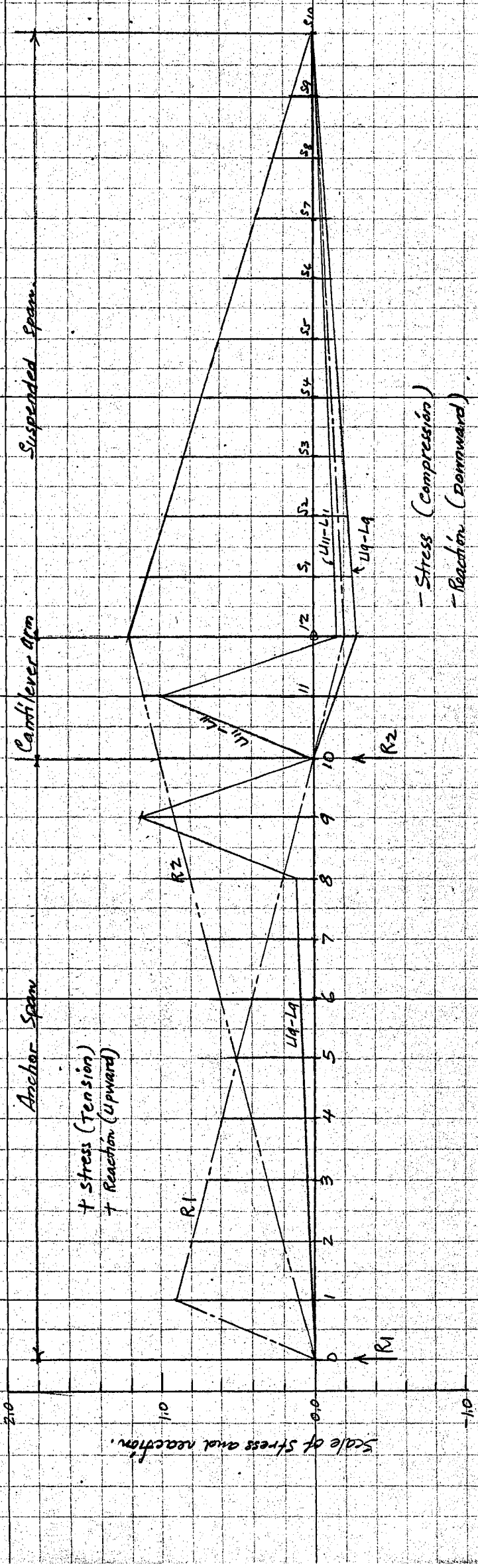
CALCULATIONS FOR

Ukagase-Bashi for Saitama-Ken.

Influence Diagram of Vertical members U_4-L_4 and U_{11}-L_{11} and Reactions R_1 and R_2.



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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Panel loads on one truss.

Dead load :-

Dead panel load = 2,170 × 3.75 = 8,140 kg. see on page 17.

Live load :-

Anchor span fully loaded. Loaded length l = 37.5 metres

uniform load $w = \frac{100000}{170+37.5}$ = 482 kg/m²

or 2.75 @ 482 = 1325 kg per lin meter.

motor truck rear wheel 2,250

imp. coef. $= \frac{20}{60+37.5}$ = 20.5% $\frac{460}{2,710}$

max. concentration on truss 2710 × 4 × 3.35 ÷ 6.6 = 5,500 kg

" " due to front wheel say 5500 ÷ 3 = 1,830 "

Suspended span and cantilever arm fully loaded. Loaded length l = 37.5 + 7.5 = 45.0 metres

uniform load $w = \frac{100000}{170+45}$ = 465 kg/m²

or 2.75 @ 465 = 1,280 kg per lin meter.

motor truck rear wheel 2,250

imp. coef. $= \frac{20}{60+45}$ = 19.1% $\frac{430}{2,680}$

max. concentration on truss 2,680 × 4 × 3.35 ÷ 6.6 = 5,440 kg

" " due to front wheel say 5440 ÷ 3 = 1,810 "

Fully loaded throughout the bridge. Loaded length = 45 + 37.5 = 82.5 metres

uniform load $w = \frac{100000}{170+82.5}$ = 396 kg/m²

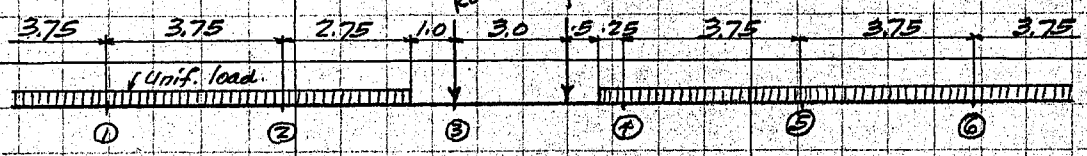
or 2.75 @ 396 = 1,090 kg per lin meter.

motor truck rear wheel 2250

imp. coef. $= \frac{20}{60+82.5}$ = 14.0% $\frac{315}{2565}$

max. concentration on truss 2,565 × 4 × 3.35 ÷ 6.6 = 5210 kg

" " due to front wheel say 5210 ÷ 3 = 1,740 "



(Referred to page 18)

Anchor span fully loaded.

	①	②	③	④	⑤	⑥
Uniform load 1,325 kg/m	4,970	4,790	1,345	2805	4970	4970
wheel loads, 5500 and 1,830			5865	1465		
	4,970	4,790	7,210	4,270	4,970	4,970

assumed loading: 4,970 kg throughout and an extra single concentration of 2,240 kg

By the similar process we have as follows:

Anchor span and cantilever arm fully loaded. 4,800 4,620 7,100 4,160 4,800 4,800

assumed loading 4,800 kg throughout and an extra single concentration of 2,300 kg

Fully loaded throughout the bridge.

4,080 3,940 6,670 3,690 4,080 4,080

Assumed loading 4,080 kg throughout and an extra single concentration of 2,590 kg

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Maximum and Minimum stresses in each member.

Top chords.

Members	Dead Load stress: Panel load 8140 kg. Σ ordinates	Dead Load stress: Stress	Live Load Stresses: Panel load 4970 (−) or 4800 (+) kg. Σ ordinates	Live Load Stresses: Stress	Single con. 2240 (−) or 2300 (+) kg. Max. ordinate	Live Load Stresses: Stress	Total L.L. stress	Combined Stress.	
U_1-U_3	− 5.250	−42,800	− 7.500	−37,300	− 1.500	− 3,400	−40,700	− 83,500	
			2.250	10,800	0.375	900	11,700	− 16,800	--- 2/3 D.L.+L.L.
U_3-U_5	− 6.750	−55,000	− 11.250	−55,900	− 2.250	− 5,000	−60,900	−115,900	
			4.500	21,600	0.750	1,700	23,300	− 13,400	--- 〃
U_5-U_7	− 4.500	−36,600	− 11.250	−55,900	− 2.250	− 5,000	−60,900	−97,500	
			6.750	32,400	1.125	2,600	35,000	10,600	--- 〃
U_7-U_9	1.500	12,200	− 9.000	43,200	1.500	3,500	46,700	58,900	
			− 7.500	−37,300	− 1.500	− 3,400	−40,700	− 32,600	--- 〃
U_9-U_{11}	8.411	68,500	8.411	40,400	1.402	3,200	43,600	112,100	
			− 0.000	—	− 0.000	—	—	—	

Bottom chords.

(+) 4970 (−) 4800. (+) 2240 (−) 2300

L_0-L_2	3,095	25,200	4.221	21,000	0.844	1,900	22,900	48,100	
			− 1.176	− 5,400	−0.188	− 400	− 5,800	1,1,000	--- 2/3 D.L.+L.L.
L_2-L_4	6.469	52,600	9.845	48,900	1.969	4,400	53,300	105,900	
			− 3.376	− 16,200	− 0.563	− 1,300	−17,500	17,600	--- 〃
L_4-L_6	6.094	49,600	11.720	58,200	2.344	5,200	63,400	113,000	
			− 5.626	−27,000	− 0.938	− 2,200	− 29,200	3,900	--- 〃
L_6-L_8	1.969	16,000	9.845	48,900	1.969	4,400	53,300	69,300	
			− 7.876	−37,800	− 1.313	− 3,000	−40,800	− 30,100	--- 〃
L_8-L_9	−5.467	−44,500	− 9,373	− 45,000	− 1.562	− 3,600	−48,600	− 93,100	
			3.906	19,400	0.781	1,800	21,200	− 8,500	--- 〃
L_9-L_{10}	−5.643	−46,000	− 9.671	−46,400	− 1.612	− 3,700	−50,100	− 96,100	
			4.028	20,000	0.806	1,800	21,800	− 8,900	--- 〃
$L_{10}-L_{11}$	−4.925	−40,100	− 4.925	−23,700	− 0.895	− 2,100	− 25,800	− 65,900	
			—	—	—	—	—	—	
$L_{11}-L_{12}$	− 4.774	− 38,900	− 4.774	−22,900	− 0.868	− 2,000	−24,900	− 63,800	
			—	—	—	—	—	—	

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CHECKED BY M. J. DATE 8-12-21 PAGE NO 38

CALCULATIONS FOR

Ukigase-Bashi for Saitama-Ken.

Diagonals.

Members	Dead Load stresses Panel load 8140 kg Σ Ordinates	Stress	Live Load stresses Panel load 4970 or 4300 kg Σ Ordinates	Stress	Single con. 2240 or 2300 kg Max. ordinates	Stress	Total L.L. Stress	Combined Stresses.	
L_0-U_1	− 4.525	−36800	− 6.170	− 30,700	− 1.234	− 2,800	− 33,500	− 70,300	
			1.645	7,900	0.274	600	8,500	− 16,000 --- 2/3 DL+LL	
U_1-L_2	3.154	25,700	4.936	24,500	1.097	2,500	27,000	52,700	
			− 1.782	− 8,600	− 0.274	− 600	− 9,200	7,900 --- "	
L_2-U_3	− 1.782	−14,500	− 3.839	−19,100	− 0.960	− 2,200	−21,300	− 35,800	○
			2.057	9,900	0.274	600	10,500	800 --- "	
U_3-L_4	0.411	3,300	2.879	14,300	0.823	1,800	16,100	19,400	○
			− 2.468	−11,900	− 0.411	− 900	−12,800	− 10,600 --- "	
L_4-U_5	0.959	7,800	3.016	14,500	0.548	1,300	15,800	23,600	○
			− 2.057	−10,200	− 0.686	−1,500	−11,700	− 6,500 --- "	
U_5-L_6	− 2.331	−19,000	− 3.702	−17,800	− 0.686	−1,600	−19,400	− 38,400	
			1.371	6,800	0.548	1,200	8,000	− 4,700 --- "	
L_6-U_7	3.701	30,100	4.524	21,700	0.823	1,900	23,600	53,700	
			− 0.823	− 4,100	− 0.411	− 900	−5,000	15,100 --- "	
U_7-L_8	− 5.073	−41,300	− 5.484	−26,300	− 0.960	− 2,200	−28,500	− 69,800	
			0.411	2,000	0.274	600	2,600	− 24,900 --- "	
L_8-U_9	5.679	46,200	5.707	28,400	1.194	2,700	31,100	77,300	
			− 0.028	− 100	− 0.028	− 100	− 200	30,600 --- "	
U_9-L_{10}	− 5.165	−42,100	− 6.777	−33,700	− 1.355	− 3,000	−36,700	− 78,800	
			1.612	7,700	0.269	600	8,300	− 19,800 --- "	
$L_{10}-U_{11}$	− 6.371	−51,900	− 6.371	−30,600	− 0.936	− 2,200	−32,800	− 84,700	
			—	—	—	—	—	—	
$U_{11}-L_{12}$	6.952	56,600	6.952	33,400	1.264	2,900	36,300	92,900	
			—	—	—	—	—	—	
Verticals.									
U_9-L_9	0.021	200	1.702	8,500	1.140	2,600	11,100	11,300	○
			− 1.681	− 8,100	− 0.280	− 600	− 8,700	− 8,600 --- "	
$U_{11}-L_{11}$	0.1444	1,200	1,000	5,000	1.000	2200	7,200	8400	○
			− 0.856	− 4,100	− 0.156	− 400	− 4,500	− 3,700 --- "	
Reactions.									
R_1	3.300	26900	4,500	22,400	0.900	2000	24400	51,300	
			− 1,200	− 5,800 (4080)	− 0.200	− 500 (2590)	− 6300	11,600 --- "	
R_2.	13,200	107,500	13.200	53,900	1,200	3,100	56800	164,500	
			—		—				

max load on shoe.

at L_0 51300 + ½(8140+4970) = 57900 Call this 58,000 kg

at L_{10} = 164,500

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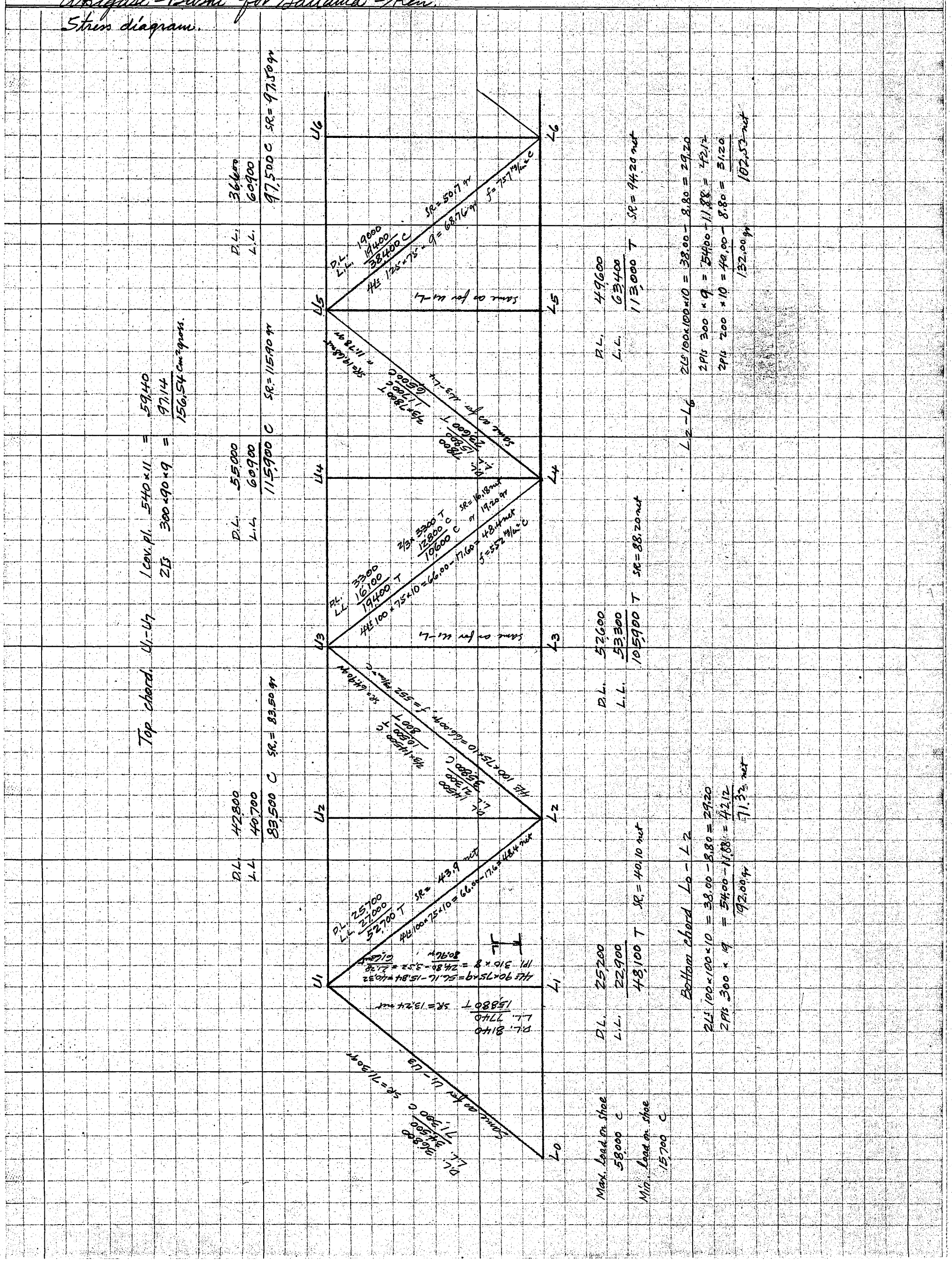
CHECKED BY M. J. DATE 8-12-24 PAGE NO 39

CALCULATIONS FOR

Kigasa-Bashi for Saitama-Ken.

Stress diagram.



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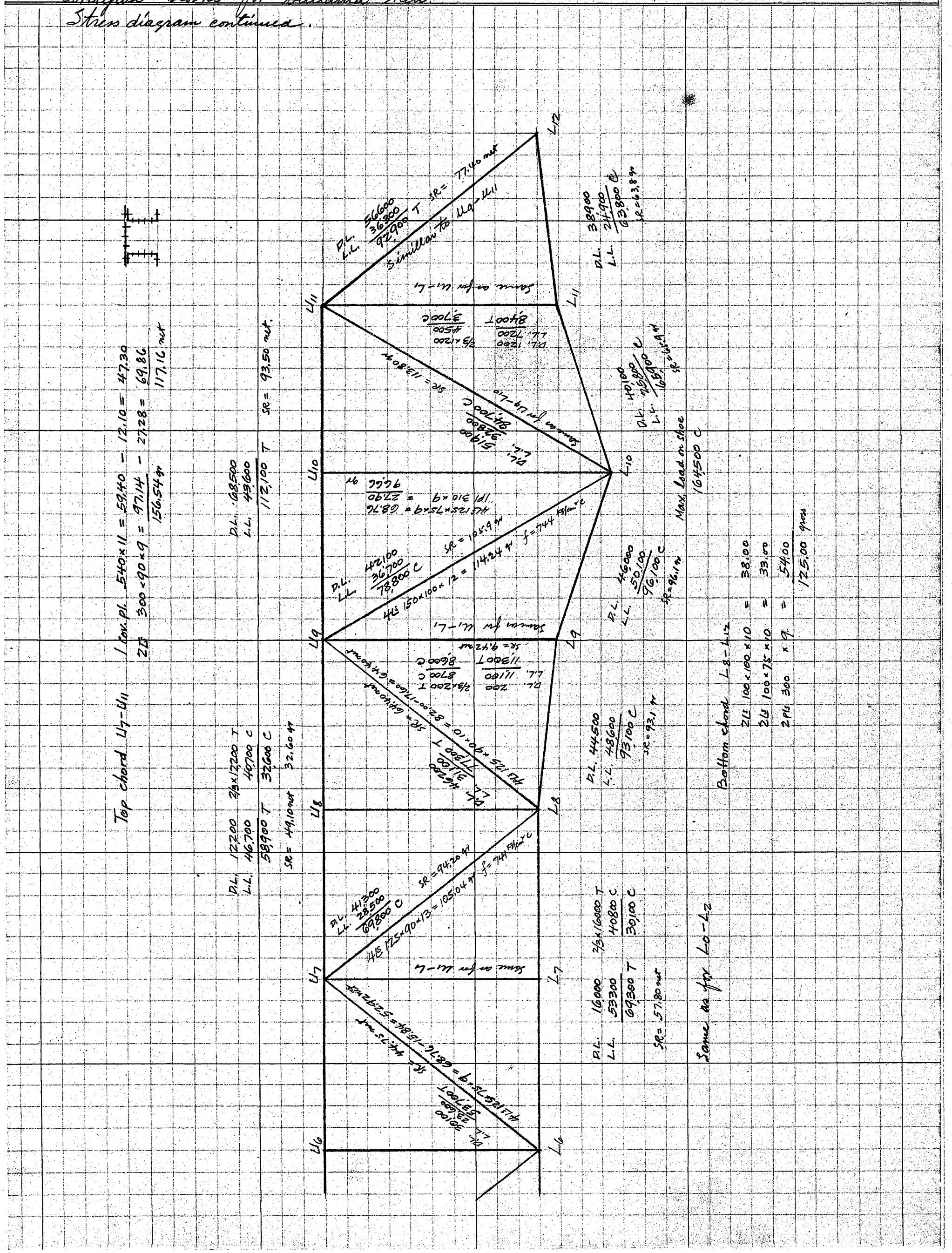
CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Stress diagram continued.



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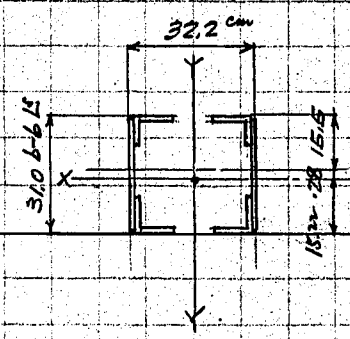
CALCULATIONS FOR

Ukigase-Bashi for Saitama-ken.

Top chord.

use same section as for simple span. See on page 21. $f = 1000$ kg/cm².

Bottom chord. $L_9 - L_{10}$ stress = 96,100 kg C



2Ls 100×100×10	= 38.00	× 12.69	=	482.0
2Ls 100×75×10	= 33.00	× −13.56	=	−447.5
2Pls 300×9	= 54.00	× 0.00	=	0.0
	125.00 cm²	0.28 cm		34.5

Moment of inertia with respect to X-X axis.

2Ls	38.00 × 12.41²	+ 175×2	=	6200
2Ls	33.00 × 13.84²	+ 76×2	=	6470
2Pls	54.00 × 0.28²	+ $\frac{0.9\times2\times30^3}{12}$	=	4050
			I_x =	16720 cm⁴

Radius of gyration $r_x = \sqrt{\frac{16,720}{125}} = 11.55$ cm, $\frac{l}{r_x} = \frac{388.4}{11.48} = 33.6$

Moment of inertia with respect to Y-Y axis.

2Ls	38.00 × 12.39²	+ 175×2	=	6180
2Ls	33.00 × 12.03²	+ 159×2	=	5100
2Pls	54.00 × 15.65²		=	13240
			I_y =	24520 cm⁴

Radius of gyration $r_y = \sqrt{\frac{24520}{125}} = 14.0$ cm, $\frac{l}{r_y} = \frac{388.4}{14.0} = 27.8$

Allowable unit compression = $1500(1 - 0.0055 \times 33.6) = 1220$ → use 1000 kg/cm²

This section is good for 125.0 @ 1000 = 125,000 kg C.

Diagonals.

For $L_2 - U_3$, $U_3 - L_4$, and $U_5 - L_6$ see on page 21.

$U_7 - L_8$ 69,800 kg C.

use 4Ls 125×90×13 = 105.04 cm² gross.

$r_y = 5.97$ cm, $\frac{l}{r_y} = \frac{548.3}{5.97} = 92$

$f = 1500(1 - 0.0055 \times 92) = 741$ kg/cm² C

good for 105.04 @ 741 = 77,800 kg C

$L_{10} - U_{11}$ 84,700 kg C $l = 653.3$ cm

use 4Ls 150×100×12 = 114.24 cm² gross

$r_y = 7.13$ cm $\frac{l}{r_y} = \frac{653.3}{7.13} = 91.6$

$f = 1500(1 - 0.0055 \times 91.6) = 744$ kg/cm² C

good for 114.24 @ 744 = 85,000 kg C

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CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Approximate weight of structural steel for main truss.

	no.		section		length		volume (m.cm²)
Top chords	1	@	156.54 cm²	×	37.50 m	=	5870
End posts	2	@	"	×	5.483	=	1718
bottom chords	2	@	92.00	×	7.500	=	1380
"	1	@	132.00	×	15.000	=	1980
"	2	@	125.00	×	3.765	=	941
"	2	@	125.00	×	3.884	=	971
diagonals	4	@	66.00	×	5.483	=	1448
"	2	@	68.76	×	5.483	=	754
"	1	@	105.04	×	5.483	=	576
"	1	@	82.00	×	5.483	=	450
"	2	@	114.24	×	6.533	=	1493
verticals	8	@	80.96	×	4.000	=	2590
"	2	@	80.96	×	4.338	=	702
"	1	@	96.66	×	5.350	=	517
false members	1	@	156.54	×	7.500	=	1175
"	1	@	80.96	×	4.000	=	324
					22,889 @ .785	=	17950
Details including rivet heads say 35%						=	6280
							24230 kg

weight of truss per lin meter = $\frac{24230}{45.0}$ = 538 kg

Summary of structural steel for one anchor span with a cantilever arm.

Stringers	2@50 = 100		
"	2@40 = 80		
	180 × 45.70 =		8230
floor beams	13@855	=	11110
lateral bracings			3,700 -- see on page 44
main trusses	2@24230	=	48,460
metals for expansion joints say			680
Shoes say	2@450 = 900		
" say	2@700 = 1400		
			2300
Strut at L10 say			1,000
Brackets etc say			200
			75,680 kg

71980

(76000)

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Design of Lateral Bracings.

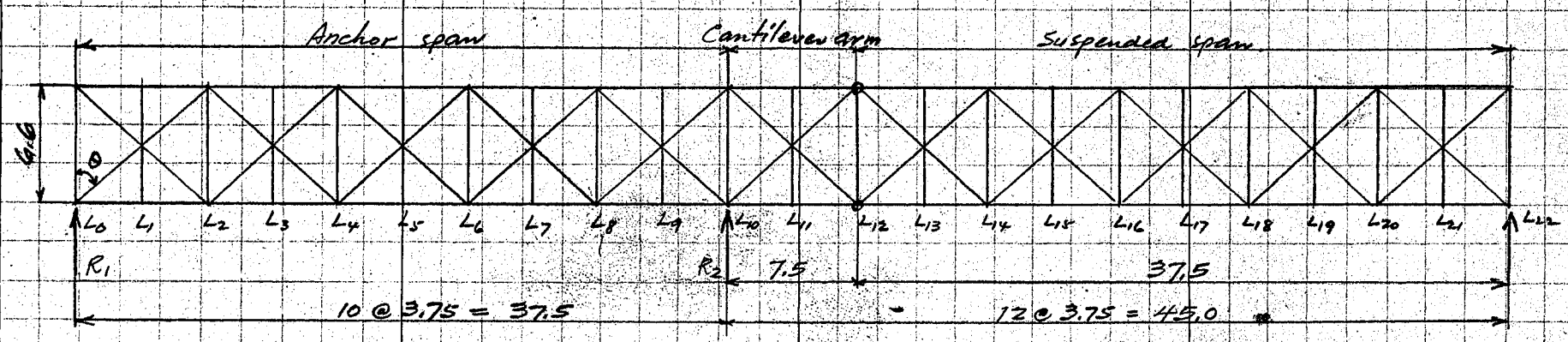
Loads on lateral bracings :-

Wind load. On loaded chord 400
" unloaded " 200
600 kg per lin meter of bridge.

Panel load = 600 × 3.75 = 2250 kg

Seismic load. Panel load = 8140 × 2 × 0.25 = 4070 kg

Seismic load governs the section of diagonal members.



Diagonal length = $\sqrt{7.50^2+6.60^2}$ = 9.99 m

sec θ = 9.99 ÷ 6.60 = 1.513
tan θ = 7.50 ÷ 6.60 = 1.136

Stress in Diagonal members due to seismic loads (as moving load).

Load on anchor span only see on page 17. (S1)

panel			Shear	coef.	Stress S1
0-1 and 9-10	$\frac{4070}{10}$ × 4.5	=	18,320	× 1.513 =	27,700 kg T
1-2 " 8-9	" 3.6	=	14,650	"	22,200 "
2-3 " 7-8	" 2.8	=	11,400	"	17,250 "
3-4 " 6-7	" 2.1	=	8,550	"	12,940 "
4-5 " 5-6	" 1.5	=	6,110	"	9,250 "

Load on Cantilever arm and suspended span only. (S2)

panel			Shear	coef.	Stress S2
L10-L11	4070 × 6.5	=	26450	× 1.513 =	40,000 kg T
L11-L12	" × 5.5	=	22,400	× "	33,900 "

For all diagonals in anchor span.

4,070 × 5.5 × 2 = 44,770
4070 × 1 × 1 = 4070
48840 × $\frac{1.513}{10}$ = 7390 kg T

Summary of Diagonal Stresses.

Members	Stress S1.	Stress S2	Summary	Net area	19mmφ rivet no. reqd. no.	used no.	
0-1	27,700		27,700 kg T ÷ 1920 =	14.43 cm²	8.1	10	use 2L 125×75×9 = 34.38
1-2	22,200		22,200 "	11.57	6.5	10	rivet holes − 7.92
2-3	17,250		17,250 "	8.98	5.1	10	26.46 cm² net
3-4	12,940		12,940 "	6.74	3.8	10	
4-5	9,250		9,250 "	4.82	2.7	10	
5-6	9,250	7,390	16,640 "	8.66	4.9	10	
6-7	12,940	7,390	20,330 "	10.60	6.0	10	
7-8	17,250	7,390	24,640 "	12.84	7.2	10	
8-9	22,200	7,390	29,590 "	15.41	8.7	12	use 2L 130×130×9 = 45.48
9-10	27,700	7,390	35,090 "	18.28	10.3	16	rivet holes − 7.92
10-11	—	40,000	40,000 "	20.85	11.8	16	37.56 cm² net
11-12	—	33,900	33,900 "	17.66	10.0	12	

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken

Approximate weight of lateral bracings.

Diagonals	32	L 125×75×9	@ 13.50 × 4.50 =		1942
"	16	L 130×130×9	@ 17.90 × 4.70 =		1345
Center connection	6		@ 50.0 =		300
rivet heads say					113
					3700 kg

weight of lateral bracing per lin. meter = 3700÷45 = 82 kg

Summary of structural steel for Truss spans.

One simple span			59,200
One Suspended span			59,200
two anchor spans with cantilever arms	2	@ 75680 =	151,360
			269,760 kg

Summary of structural steel for girder spans.

Nineteen girder spans	19	@ 19400 =	368,600 kg
			638,360 "

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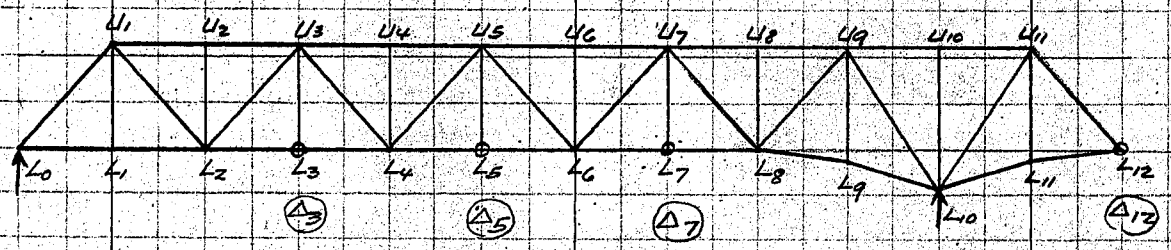
CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Deflection of the truss at several panel points.



General equation to the deflection :- $\Delta = \Sigma \frac{SL}{EA} \cdot T$ or $\frac{1}{E} \Sigma \frac{SL}{A} \cdot T$

Dead load Deflection at panel points L_3, L_5, L_7, and L_{12}.

Member	L	A	L/A	S_D	$\frac{S_D L}{A}$	T_3	$\frac{S_D L}{A} T_3$	T_5	$\frac{S_D L}{A} T_5$	T_7	$\frac{S_D L}{A} T_7$	T_{12}	$\frac{S_D L}{A} \cdot T_{12}$
U_1-U_3	750	156.54	4.790	- 42,800	- 205,000	- 1.3125	269,100	- 0.9375	192,100	- 0.5625	115,300	0.3750	- 76,900
U_3-U_5	〃	〃	〃	- 55,000	- 263,500	- 1.6875	444,000	- 1.8750	493,600	- 1.1250	296,700	0.7500	- 197,500
U_5-U_7	〃	〃	〃	- 36,600	- 175,300	- 1.1250	197,200	- 1.8750	328,700	- 1.6875	295,800	1.1250	- 197,200
U_7-U_9	〃	〃	〃	12,200	58,400	- 0.5625	- 32,900	- 0.9375	- 54,700	- 1.3125	- 76,700	1.5000	87,600
U_9-U_{11}	〃	〃	〃	68,500	328,000	0.0000	—	0.0000	—	0.0000	—	1.4018	459,500
L_0-L_2	750	92.0	8.150	25,200	205,300	0.6566	134,700	0.4690	96,200	0.2814	57,700	- 0.1876	- 38,500
L_2-L_4	〃	132.0	5,680	52,600	299,000	1.9691	588,200	1.4065	420,300	0.8439	252,400	- 0.5626	- 168,200
L_4-L_6	〃	〃	〃	49,600	282,000	1.4064	396,500	2.3440	661,000	1.4064	396,500	- 0.9376	- 264,500
L_6-L_8	〃	92.0	8.150	16,000	130,300	0.8439	110,000	1.4065	183,300	1.9689	256,500	- 1.3126	- 171,200
L_8-L_9	376.5	125.0	3.015	- 44,500	- 134,100	0.2604	- 34,900	0.4340	- 58,200	0.6076	- 81,400	- 1.5622	209,500
L_9-L_{10}	388.4	〃	3.105	- 46,000	- 142,800	0.2685	- 38,400	0.4475	- 63,900	0.6265	- 89,500	- 1.6118	230,200
$L_{10}-L_{11}$	388.4	〃	3.105	- 40,100	- 124,500	0.0000	—	0.0000	—	0.0000	—	- 0.8954	111,500
$L_{11}-L_{12}$	376.5	〃	3.015	- 38,900	- 117,200	0.0000	—	0.0000	—	0.0000	—	- 0.8679	101,700
L_0-U_1	548.3	156.54	3.502	- 36,800	- 129,000	- 0.9597	123,800	- 0.6855	88,400	- 0.4113	53,100	0.2742	- 35,400
U_1-L_2	〃	66.0	8.310	25,700	213,500	0.9597	205,000	0.6855	146,300	0.4113	87,800	- 0.2742	- 58,500
L_2-U_3	〃	〃	〃	- 14,500	- 120,500	- 0.9597	115,600	- 0.6855	82,600	- 0.4113	49,500	0.2742	- 33,100
U_3-L_4	〃	〃	〃	3,300	27,400	- 0.4113	- 11,300	0.6855	18,800	0.4113	11,300	- 0.2742	- 7,500
L_4-U_5	〃	〃	〃	7,800	64,800	0.4113	26,700	- 0.6855	- 44,400	- 0.4113	- 26,700	0.2742	17,800
U_5-L_6	〃	68.76	7.980	- 19,000	- 151,500	- 0.4113	62,300	- 0.6855	103,800	0.4113	- 62,300	- 0.2742	41,500
L_6-U_7	〃	〃	〃	30,100	240,200	0.4113	98,800	0.6855	164,700	- 0.4113	- 98,800	0.2742	65,900
U_7-L_8	〃	105.04	5.220	- 41,300	- 215,600	- 0.4113	88,600	- 0.6855	147,700	- 0.9597	207,000	- 0.2742	59,100
L_8-U_9	〃	82.00	6.690	46,200	309,000	0.4476	138,300	0.7460	230,500	1.0444	322,500	0.0560	17,300
U_9-L_{10}	653.3	114.24	5.720	- 42,100	- 240,800	- 0.4518	108,800	- 0.7530	181,300	- 1.0542	253,800	0.2686	- 64,700
$L_{10}-U_{11}$	653.3	〃	〃	- 51,900	- 297,000	0.0000	—	0.0000	—	0.0000	—	- 0.9363	278,000
$U_{11}-L_{12}$	548.3	156.54	3.502	56,600	198,300	0.0000	—	0.0000	—	0.0000	—	1.2639	250,700
U_9-L_9	433.8	88.96	5.360	200	1,100	0.0468	100	0.0780	100	0.1092	100	- 0.2801	- 300
$U_{11}-L_{11}$	433.8	〃	〃	1,200	6,400	0.0000	—	0.0000	—	0.0000	—	- 0.1556	- 1,000
Hanger	400	80.96	4.940	8,100	40,000	1.0000	40,000	1.0000	40,000	1.0000	40,000	—	—
							3,147,700		3,579,600		2,695,500		1,930,300
							- 117,500		- 221,200		- 435,400		- 1,314,500
							3,030,200		3,358,400		2,260,100		615,800

Summary

Dead Load defl. (Devide by E = 2,100,000) Δ_3 = 1.44 cm Δ_5 = 1.60 cm Δ_7 = 1.08 cm Δ_{12} = 0.29 cm

Live Load 〃 (Full load) coef = $\frac{4080}{8140}$ 0.72 0.80 0.54 0.15

2.16 cm 2.40 cm 1.62 cm 0.44 cm

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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MADE BY K. Inaba DATE 8-12-18 FILE NO
CHECKED BY M. J. DATE 9-1-18 PAGE NO 46

CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Live load deflection for Partial loadings.

Panel loads.

Anchor span loaded, Panel load = 4970 kg, see on page 36.
Suspended span and arm loaded, " = 4800 ", "

Anchor span loaded.

Members	Sum of Ordinates	Stress S	L/A	SL/A	T_3	$\frac{SL}{A}T_3$	T_5	$\frac{SL}{A}T_5$	T_7	$\frac{SL}{A}T_7$	T_{12}	$\frac{SL}{A}T_{12}$
U_1-U_3	−7.500	−37300	4.790	−178700	−1.3125	234500	−0.9375	167500	−0.5625	100500	0.3750	−67000
U_3-U_5	−11.250	−55900	"	−268000	−1.6875	452000	−1.8750	502500	−1.1250	301500	0.7500	−201000
U_5-U_7	−11.250	−55900	"	−268000	−1.1250	301500	−1.8750	502500	−1.6875	452000	1.1250	−301500
U_7-U_9	−7.500	−37300	"	−178700	−0.5625	100500	−0.9375	167500	−1.3125	234500	1.5000	−268000
L_0-L_2	4221	21000	8.150	171100	0.6566	112000	0.4690	80200	0.2814	48100	−0.1876	−32100
L_2-L_4	9.845	49000	5.680	278500	1.9691	548000	1.4065	391500	0.8439	234900	−0.5626	−156500
L_4-L_6	11.720	58200	"	330500	1.4064	465000	2.3440	775000	1.4064	465000	−0.9376	−310000
L_6-L_8	9.845	49000	8.150	399000	0.8439	336500	1.4065	561000	1.9689	785000	−1.3126	−523500
L_8-L_9	3.906	19400	3.015	58500	0.2604	15200	0.4340	25400	0.6076	35500	−1.5622	−91400
L_9-L_{10}	4.028	20000	3.105	62100	0.2685	16700	0.4475	27800	0.6265	38900	−1.6118	−100000
L_0-U_1	−6.170	−30700	3.502	−107500	−0.9597	103100	−0.6855	73600	−0.4113	44200	0.2742	−29500
U_1-L_2	4.799	23900	8.310	198500	0.9597	190500	0.6855	136000	0.4113	81600	−0.2742	−54400
L_2-U_3	−3.428	−17000	"	−141200	−0.9597	135500	−0.6855	96700	−0.4113	58000	0.2742	−38700
U_3-L_4	2.057	10200	"	84800	−0.4113	−34900	0.6855	58100	0.4113	34900	−0.2742	−23300
L_4-U_5	−0.686	−3400	"	−28300	0.4113	−11600	−0.6855	19400	−0.4113	11700	0.2742	−7800
U_5-L_6	−0.686	−3400	7.980	−27100	−0.4113	11200	−0.6855	18600	0.4113	−11200	−0.2742	7400
L_6-U_7	2.057	10200	"	81400	0.4113	33500	0.6855	55800	−0.4113	−33500	0.2742	22300
U_7-L_8	−3.428	−17000	5.220	−88800	−0.4113	36500	−0.6855	60800	−0.9597	85200	−0.2742	24400
L_8-U_9	5.343	26600	6.690	178000	0.4476	79700	0.7460	132800	1.0444	185800	0.0560	10000
U_9-L_{10}	−6.777	−33700	5.720	−192800	−0.4518	87000	−0.7530	145000	−1.0542	203100	0.2686	−51800
U_9-L_9	1.702	8500	5.360	45600	0.0468	2100	0.0780	3600	0.1092	5000	−0.2801	−12800
Hangers	1.000	5000	4.940	24700	1.0000	24700	1.0000	24700	1.0000	24700	—	—
						3285700		4026000		3430100		64100
						−46500		—		−44700		−2269300
						3239200		4026000		3385400		−2205200

Devide by E=2,100,000, Deflection = Δ_3 = 1.54 cm Δ_5 = 1.92 cm Δ_7 = 1.61 cm Δ_{12} = −1.05 cm

Suspended span and Cantilever arm loaded.

		Δ_3	Δ_5	Δ_7	Δ_{12}
For the unif. panel load of 4800 all over the spans.	D.L. Defl. × 4800/8140	0.85	0.94	0.64	0.17
Loads 4800 on anchor span, less	4800/4970	1.49	1.85	1.55	−1.01
Suspended span and arm loaded 4800 throughout		−0.64 cm	−0.91 cm	−0.91 cm	+0.87 cm

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CALCULATIONS FOR

Ikigase Bashi for Saitama-Ken.

Summary of Deflection at several panel points for several load manners.

Suspended span. Deflection at center of span. L5.

Dead load deflection				2.71
Live load " . load on suspended span only				1.65
				4.36 cm
Dead load deflection				2.71
Live load " . load on suspended span and arm.	1.65 × 4800/4970	=		1.59
				4.30 cm
Dead load deflection				2.71
Live load " . Full load on all spans.	1.65 × 4086/4970	=		1.35
				4.06 cm

Anchor span and cantilever arm. L3, L5, L7 and L12.

	Δ3	Δ5	Δ7	Δ12
Dead load deflections	1.44	1.60	1.08	0.29
Live load " . Anchor span loaded	1.54	1.92	1.61	-1.05
	2.98 cm	3.52 cm	2.69 cm	-0.76 cm
Dead load deflection	1.44	1.60	1.08	0.29
Live load " . Suspended span + arm loaded	-0.64	-0.91	-0.91	0.84
	0.80 cm	0.69 cm	0.17 cm	1.13 cm
Dead load deflection	1.44	1.60	1.08	0.29
Live load " . Full load on all spans.	0.85	94	0.64	0.17
	2.29 cm	2.54 cm	1.72 cm	0.46 cm

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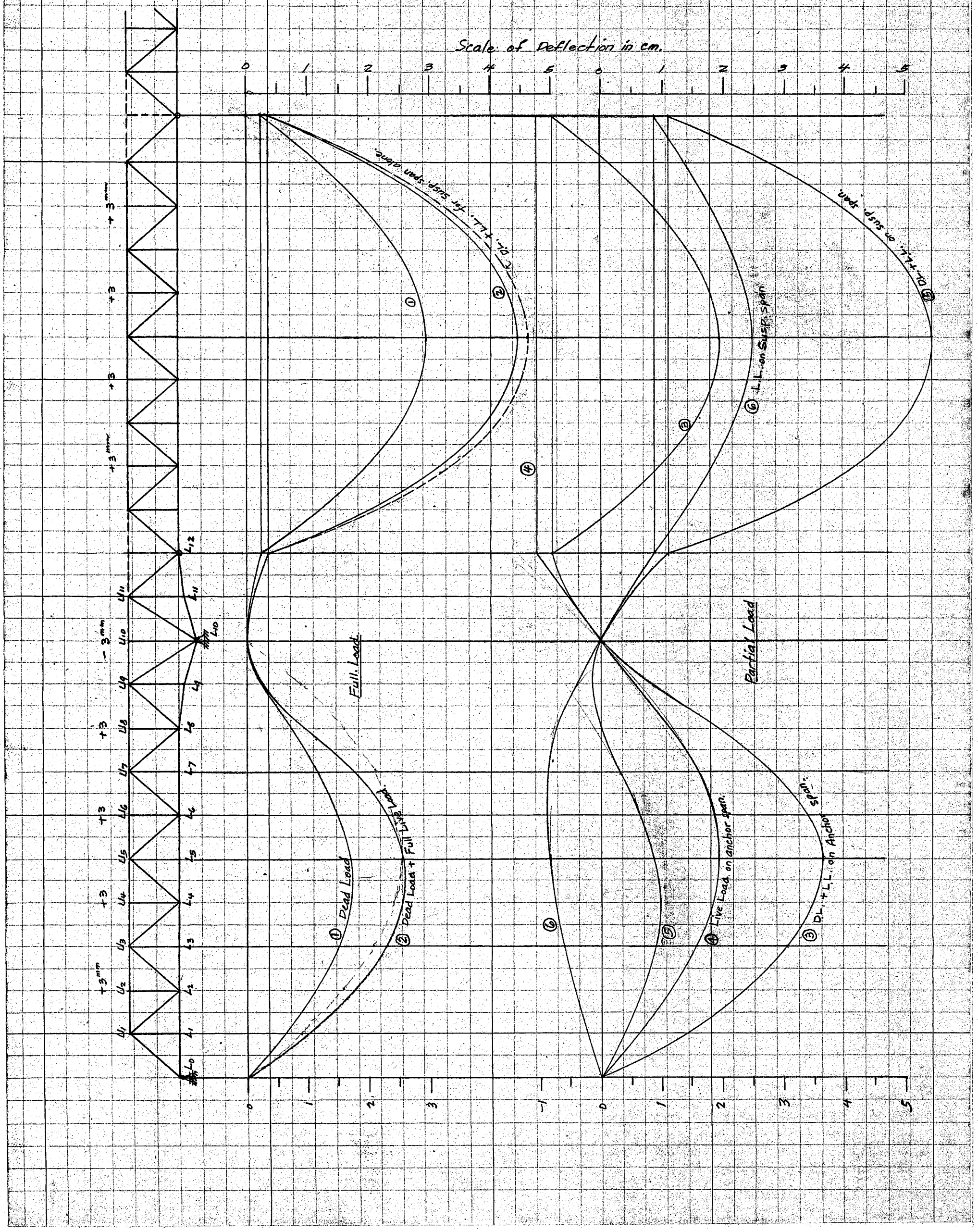
CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Deflection Diagram of several loadings.



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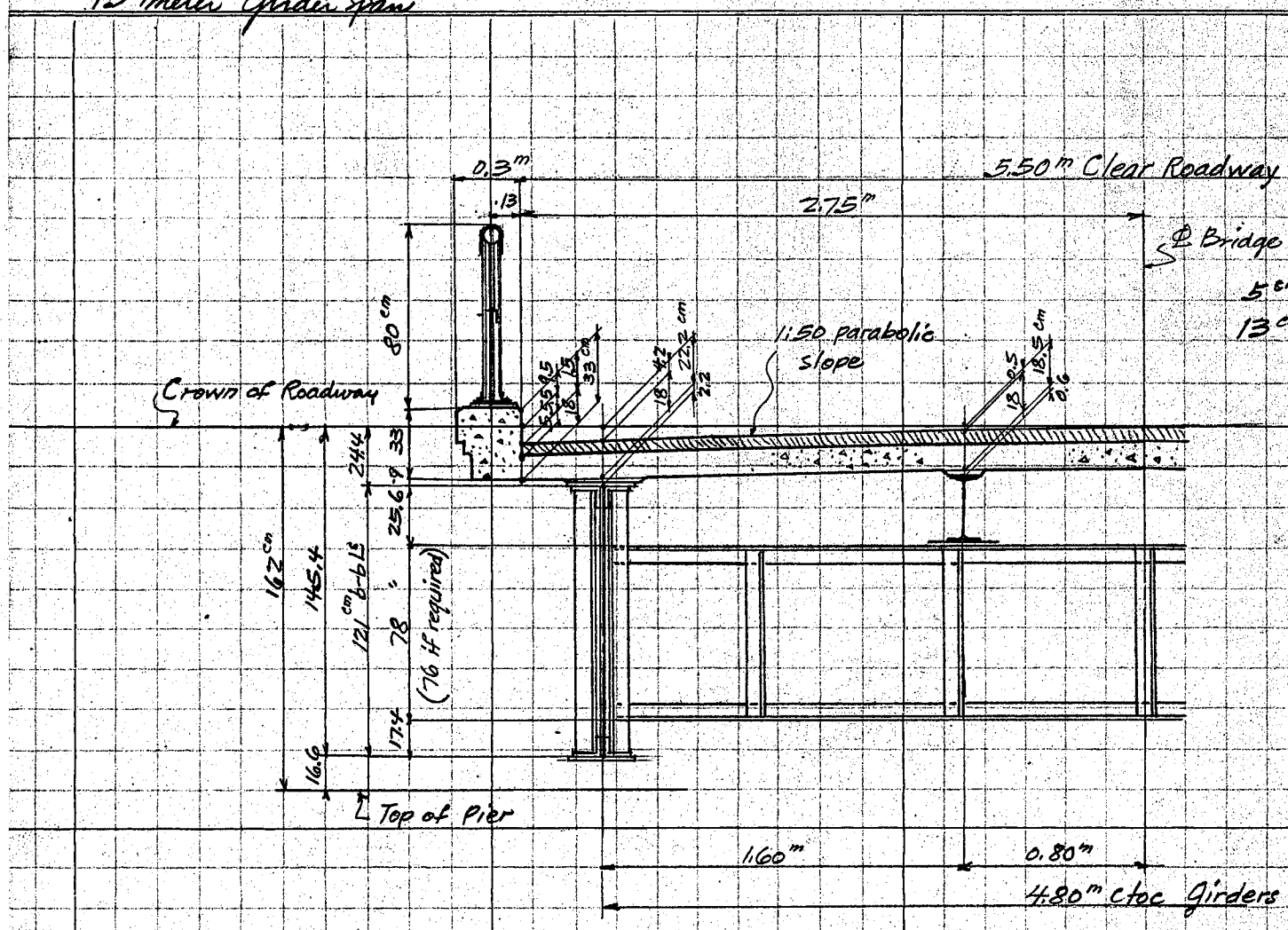
CALCULATIONS FOR

15 meter Girder Span



CROSS SECTION OF BRIDGE.
Scale 1:30

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MADE BY K. Inaba DATE 9-2-22 FILE NO
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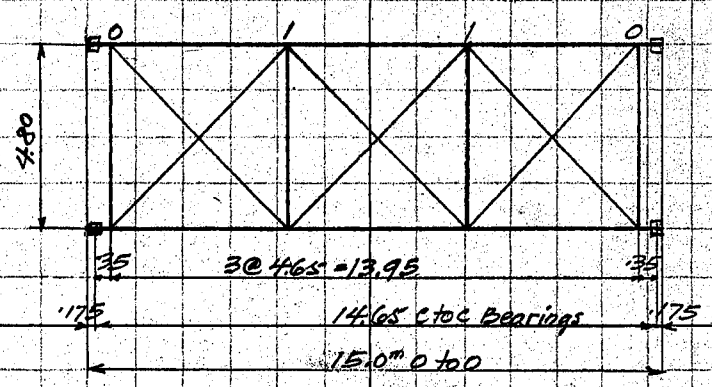
CALCULATIONS FOR

15 meter Girder span

Design of Lateral Bracings :-



Diagonal length

$4.80^2 = 23.04$
$4.65^2 = 21.62$
44.66

$\sqrt{44.66} = 6.68$ meters.

Coefficient $= \frac{6.68}{4.80} = 1.392$

Mind load assumed 400 kg per lin. meter of bridge. panel load = 400 × 4.65 = 1860 kg
Shear between 0 and 1 = 1,860 kg
Stress in diagonal 0-1 = 1860 × 1.392 = 2,590 kg T

Seismic stresses. Acceleration assumed as 2500 mm/sec² or k = 0.250
Dead load of bridge.

Floor slab and pavement between main girders	4.80 × 430	=	2,065
Handrails, copings, floor outside of main girders	2 @ 390	=	780
			2845
Stringers	2 @ 50	=	100
floor beams	530 ÷ 4.65	=	114
lateral bracings assumed			50
main girders assumed	2 @ 255	=	510
			774
			3619
		Call this	3,620 kg per lin m.

Panel load = 3620 × 4.65 = 16850 kg
Horizontal panel load = 16850 × 0.25 = 4210 kg

Stress in diagonal 0-1 = 4210 × 1.392 = 5850 kg T
Seismic stresses govern the section of diagonal.
Sectional area required = $\frac{5850}{1920}$ = 3.05 cm² net

19 mm φ rivet no. required for connection = $\frac{5850}{2126 \times 1.6}$ = 1.72 use 3 rivets.

use L 125 × 75 × 9 = 17.19 − 1.98 15.21 cm² net

Approximate weight of lateral bracings.

Diagonals	6 ls	125 × 75 × 9	@ 13.50 × 6.30	=	510
center connections	3		@ 11.00	=	33
conn. to girders	8		@ 13.00	=	104
rivet heads &c say					13
					660 kg.

weight of bracing per lin meter = 660 ÷ 13.95 = 47 kg

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CHECKED BY M.J DATE 9-4-1 PAGE NO 51

CALCULATIONS FOR

15 meter Girder Span

Design of main girders. Span length 14.65 meters c to c of bearings. Length of girder 15 meters O to O.

Dead load :-

Dead load direct on main girder.

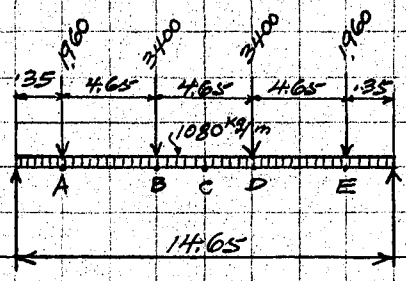
Floor slab and pavement	$\frac{430 \times 1.95^2}{2 \times 1.60}$ =	510	
Coping	210 × 2.084 ÷ 1.60 =	273	
Handrail	30 × 2.08 ÷ 1.60 =	39	
main girder assumed		255	
		1077	
	call this	1080	kg per lin meter

Dead load through stringers + floor beams concentrated at panel points.

Floor slab and pavement.	$\frac{430 \times 1.95 \times 0.625}{1.60}$ =	328	
" " "	430 × 0.80 =	344	
Coping	210 − 273 =	−63	
Handrail	30 − 39 =	− 9	
lateral bracing	50 ÷ 2 =	25	
floor beam	530 ÷ 4.65 ÷ 2 =	57	
Stringers		50	
		732	kg per lin meter

Panel load = 732 × 4.65 = 3400 kg for int. panel pt.
732 × 2.68 = 1960 " " end " "

Dead load moments and shears.



Reaction

1080 × 14.65 ÷ 2 =	7910	
	1960	} 5360
	3400	
R =	13270	kg = 2nd shear.

Moment at point A

Concentrations	5360 × 0.35	=	1876
unif. load	7910 × 0.35	=	2770
"	$-\frac{1080 \times 0.35^2}{2}$	=	− 66
			4580 kgm

Moment at point B.

Concentrations	5360 × 5.00	=	26800
	− 1960 × 4.65	=	− 9110
unif. load	7910 × 5.00	=	39550
"	$-1080 \times \frac{5.00^2}{2}$	=	− 13500
			43740 kgm

moment at point C (center of span)

Concentrations	5360 × 7.325	=	39260
unif. load	1/8 × 1080 × 14.65²	=	29000
less	− 1960 × 6.975	=	− 13670
"	− 3400 × 7.325	=	− 7900
			46690 kgm

Dead load on shoe

Dead load	1080		
	732		
	1812 × 15.0 ÷ 2	=	13600
	(weight of shoe say		200
			13800 kg on one shoe

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CALCULATIONS FOR

15 meter girder span

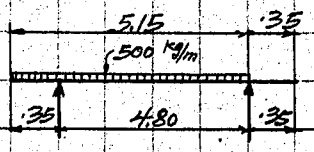
Live load :-

Uniform live load = 500 kg per square meter

Impact for motor truck loading = $\frac{20}{60+14.65}$ = 26.8 %

Motor truck rear wheel concentration = 2250
26.8 % impact = 600
2850 kg

Front wheel with impact 2850÷3 = 950 "

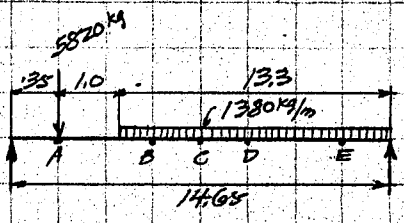


Uniform load on one girder = $\frac{500 \times 5.15^2}{2 \times 4.8}$ = 1380 kg per lin meter.

Wheel concentrations on main girder assumed as follows.

Rear wheels 4 × 2850 × $\frac{2.45}{4.80}$ = 5820 kg

Front wheel 5820÷3 = 1940 kg



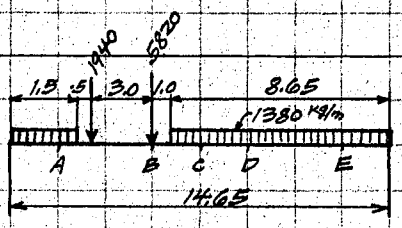
Moment at point A

Reaction 5820 × 14.3 ÷ 14.65 = 5680

$\frac{1380 \times 13.3^2}{2 \times 14.65}$ = 8330

14010 kg

Moment = 14010 × 0.35 = 4900 kgm



Moment at point B.

Reaction 5820 × 9.65 ÷ 14.65 = 3840

1940 × 12.65 ÷ 14.65 = 1675

$\frac{1380 \times 8.65^2}{2 \times 14.65}$ = 3525

1380 × 1.5 × 13.9 ÷ 14.65 = 1960

11000 kg

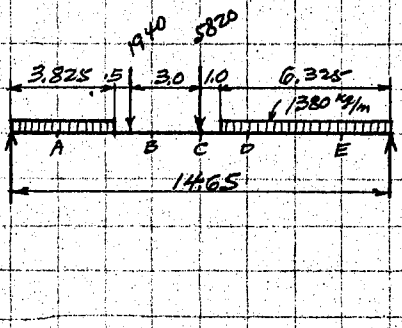
Moment =

11,000 × 5.00 = 55000

− 1940 × 3.00 = − 5820

1380 × 1.5 × 4.25 = − 8800

40380 kgm



Moment at point C

Reaction 5820 × 1/2 = 2910

1940 × 10.325 ÷ 14.65 = 1370

$\frac{1380 \times 6.325^2}{2 \times 14.65}$ = 1885

$\frac{1380 \times 3.825 \times 13.738}{14.65}$ = 4950

11115 kg

Moment. = 11115 × 14.65 ÷ 2 = 81400

− 1940 × 3.00 = − 5820

− 1380 × 3.825 × 5.4125 = − 28600

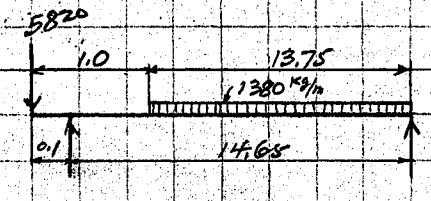
46980 kgm

max. end shear.

$\frac{1380 \times 13.65^2}{2 \times 14.65}$ = 8780

5820

14600 kg



Maximum load on shoe

$\frac{1380 \times 13.75^2}{2 \times 14.65}$ = 8910

5820 × $\frac{14.75}{14.65}$ = 5860

14770 kg

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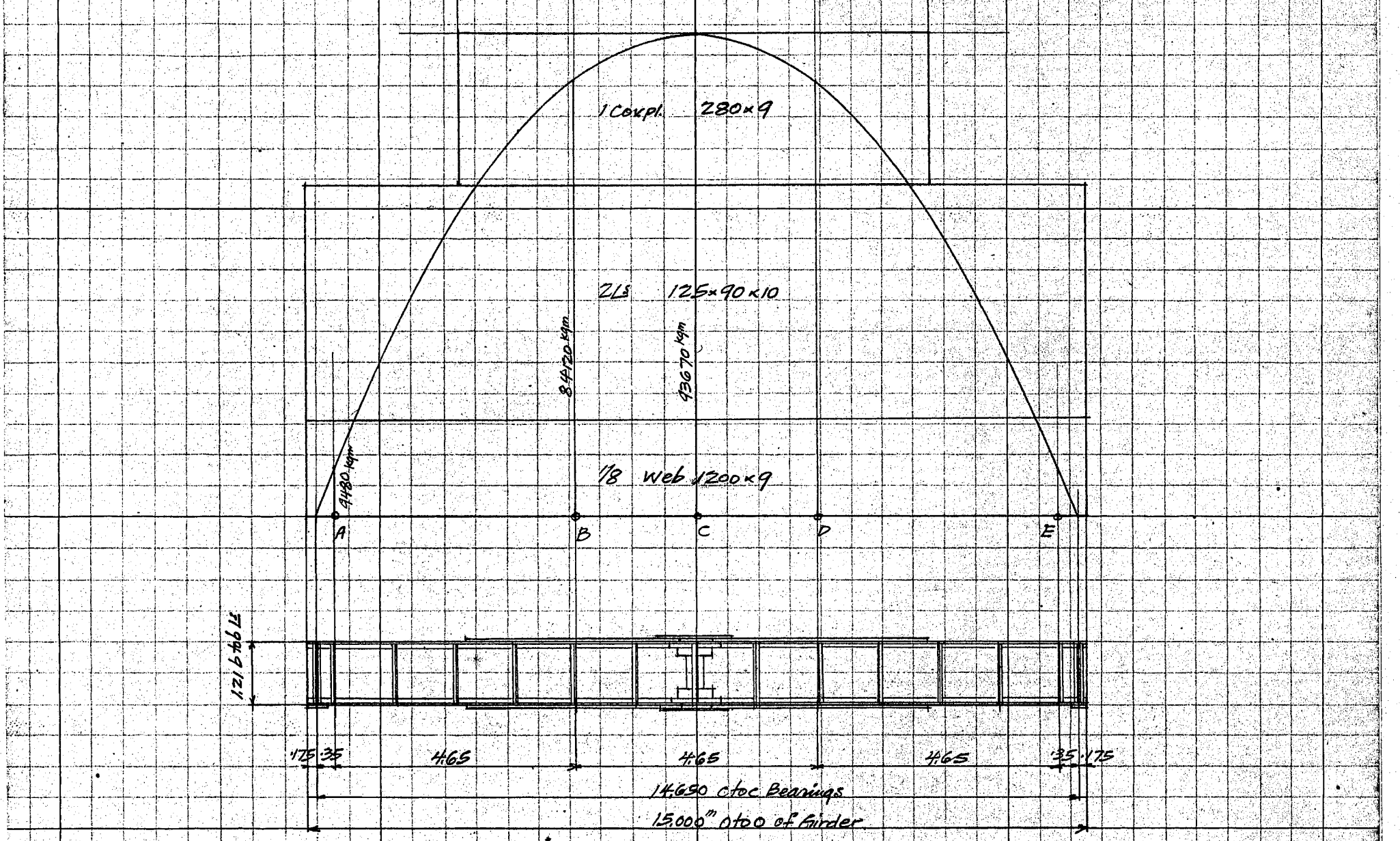
CALCULATIONS FOR

15 meter Girder Span

Summary of moments, end shear and loads on shoe.

	moment at A	moment at B	moment at C	end shear	load on one shoe
Dead Load	4,580	43740	46690	13270	13800
Live Load	4900	40380	46980	14600	14770
	9480 kgm	84120 kgm	93670 kgm	27870 kg	28570 kg



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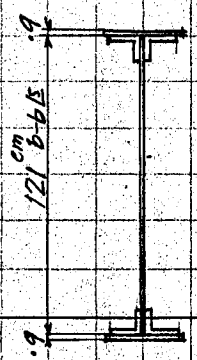
CALCULATIONS FOR

15 Meter Girder Span

Section of main girder.



Dry web pl. 1200 × 9 = 108.00 cm², 1/8 web area = 13.50 cm²
Depth back to back of Ls = 121.0 cm
effective depth = 121.0 − 2@1.20 = 118.60 cm
∴ flange stress = $\frac{93670}{1.1860}$ = 78900 kg T or C

Flange area required = $\frac{78900}{1200}$ = 65.75
1/8 web area = − 13.50
52.25 cm² net

use 19ϕ rivet

1 cov. pl.	280 × 9	= 25.20	− 3.96	= 21.24
2 Ls	125 × 90 × 10	= 41.00	− 8.80	= 32.20
		66.20 cm² gr		53.44 cm² net.

1 cov. pl. 280×9 =	25.20	× (−0.45)	=	− 11.34
2 Ls 125×90×10 =	41.00 / 66.20	× 2.22 / 1.20 cm	=	91.00 / 79.66

unit shear on web = $\frac{27870}{108}$ = 258 kg/cm²

Approximate weight of main girder.

∴ flange = web	4 Ls	125 × 90 × 10	@ 16.10	× 15.00	=	966	
	2 cov. pls	280 × 9	@ 19.78	× 9.00	=	356	
	1 web pl.	1200 × 9	@ 84.78	× 15.00	=	1272	
							2594
end stiffeners	8 Ls	100 × 75 × 10	@ 13.00	× 1.19	=	124	
	4 fills	150 × 10	@ 11.78	× 1.02	=	48	
int. "	24 Ls	90 × 75 × 9	@ 11.00	× 1.21	=	320	
center "	2 Ls	90 × 75 × 9	@ 11.00	× 1.17	=	26	
Conn. Ls say	4 Ls	90 × 90 × 10	@ 13.30	× 0.50	=	27	
Sole pl.	2 pls	350 × 25	@ 68.69	× 0.41	=	56	
Center splice say						250	
Rivet heads +c say						125	
						976	(37.6%)
						3570 kg	

∴ weight of the girder per lin meter = 3570 ÷ 15.0 = 238 kg

Summary of structural steel in one span.

Stringers	2 @ 50 × 15.00	=	1,500
floor beams	4 @ 530	=	2,120
lateral bracing complete		=	660
main girders	2 @ 3570	=	7,140
Shoes say	4 @ 150	=	600
expansion metal +c say		=	600
			12,620 kg.

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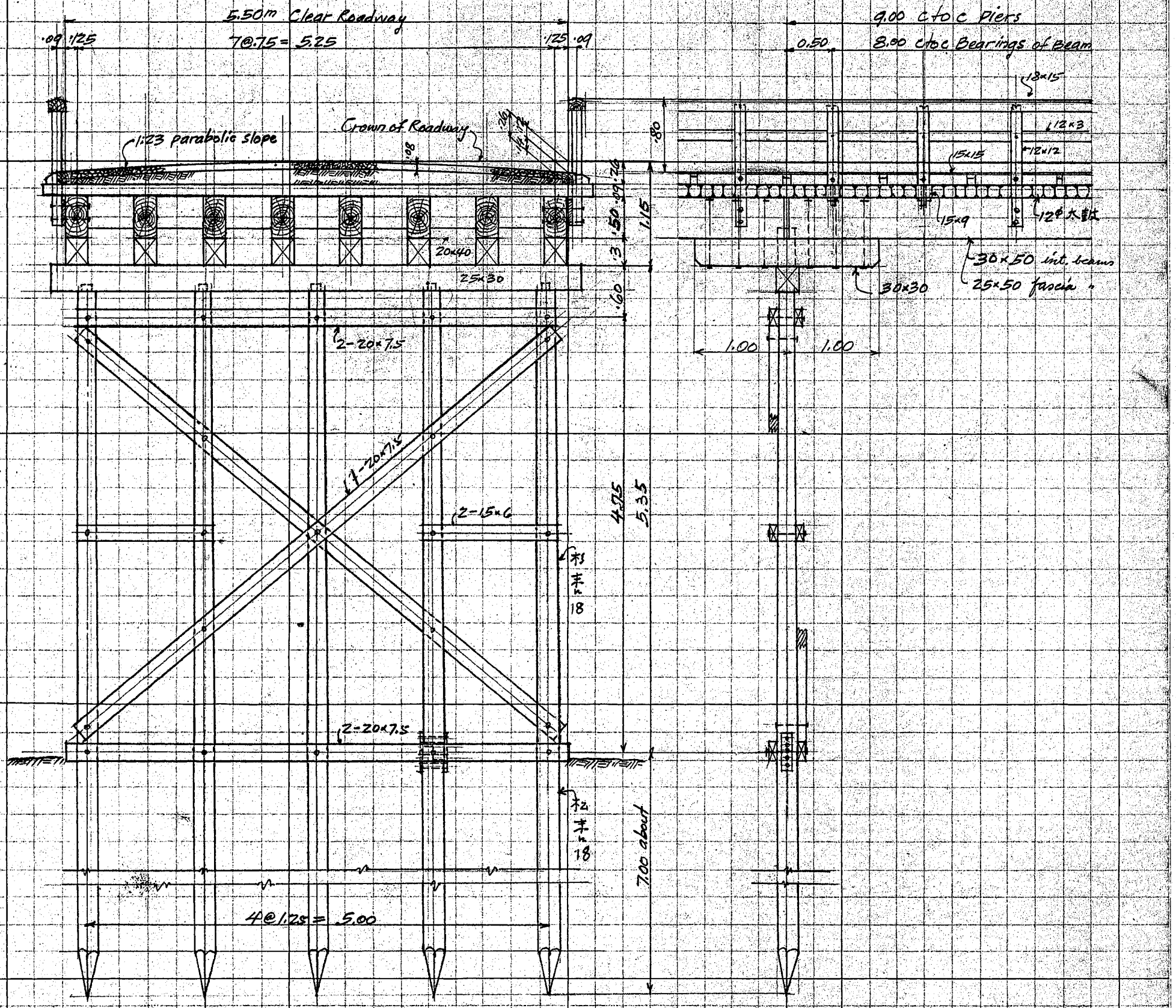
CALCULATIONS FOR

Ukigase-Bashi for Daitama-Ken.

Design of wooden beam span.
Cross section of the bridge assumed as follows:-



Scale 1:50

Design of Timber floor slab. span length 0.75m continuous.
Dead Load:-

Gravel pavement	8cm	@ 17kg	=	136
earth filling	18 "	@ 16 "	=	288
timber slab assumed	9 "	@ 6.5	=	59
misc. say				17
				500 kg/o.m

Dead load moment = 1/10 × 500 × 0.75² = 28 kgm. for one meter strip
Dead load shear = 1/2 × 500 × 0.75 = 188 kg " " "

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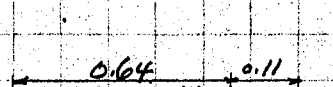
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Live load:- Min. thickness of gravel and earth at centre of span = 20 cm (end span)

6 ton motor truck rear wheel concentration = 2250
30% impact = 675
2925 kg

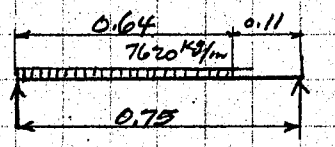
Longitudinal distribution a = 20+2@20 = 60 cm
transverse " b = 24+2@20 = 64 cm
distributed load = $\frac{2925}{0.6 \times 0.64}$ = 7620 kg/m²



reaction = 7620 × 0.64 ÷ 2 = 2440 kg per meter strip ✓

moment
2,440 × 0.375 = 915
− 2440 × 0.160 = − 390
525

For continuity of slab, moment may be taken as
525 × 8/10 = 420 kgm. per meter strip



Shear $\frac{7620 \times 0.64 \times 0.43}{0.75}$ = 2795 kg

Summary of moments and shears.

	Moments	End shears
Dead load	28	188
Live load	420	2795
	448 kgm	2983 kg

Section modulus required = $\frac{448 \times 100}{60}$ = 747 cm³

Depth = $\sqrt{\frac{747 \times 6}{100}}$ = 6.70 cm

use 9 cm timbers.

Design of main beam. Effective span length = 8.00 meters

Dead load:-

Deck construction 0.75 @ 500 = 375
main beam assumed as 0.3×0.5 @ 650 = 98
473 call this 475 kg per lin meter.

Dead load moment = 1/8 × 475 × 8.00² = 3800 kgm
Dead load shear = 1/2 × 475 × 8.00 = 1900 kg

Live load:-

Live load assumed a full uniform load of 500 kg/m²
Load for one beam = 500 × 0.75 = 375 kg per lin meter.
Live load moment = 1/8 × 375 × 8.00² = 3000 kgm
Live load shear = 1/2 × 375 × 8.00 = 1500 kg

Summary of moments and shears.

	Moments	End shears
Dead load	3800	1900
Live load	3000	1500
	6800 kgm	3400 kg

Section modulus required = $\frac{6800 \times 100}{60}$ = 11330 cm³

use 30 cm × 50 cm timber, Sm. = $\frac{30 \times 50^2}{6}$ = 12,600 cm³

unit shear = $\frac{3}{2} \times \frac{3400}{30 \times 50}$ = 3.4 kg/cm² ✓

Cantilever arm supporting main beam. Cantilever arm 0.50 meter.

max. moment = 3400 × 0.50 = 1700 kgm
max. shear = 3400 kg

Sm. req'd. = $\frac{1700 \times 100}{60}$ = 2830 cm³

use 30 cm × 30 cm timber, Sm. = $\frac{30 \times 30^2}{6}$ = 4500 cm³
unit shear = $\frac{3}{2} \times \frac{3400}{30 \times 30}$ = 5.7 kg/cm²

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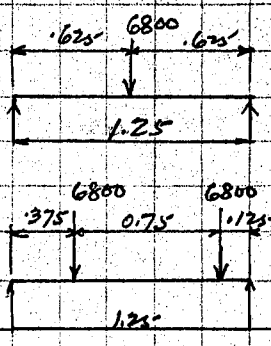
CALCULATIONS FOR

Akigase - Baoki for Saitama Ken

Design of pier.

Top beam.



Main beam concentration = 2@3400 = 6800 kg for one beam.

max. moment

$6800 \times \frac{1.25}{4} = 2125$

For continuity of beam,

moment = 8/10 × 2125 = 1,700 kgm

max. shear.

6800 × 0.375 ÷ 1.25 = 2040
6800 × 1.125 ÷ 1.25 = 6120
8160 kg.

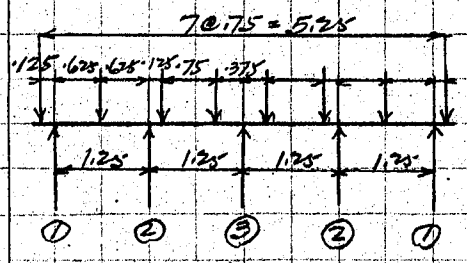
Section modulus required = $\frac{1700 \times 100}{60} = 2833\ cm^3$

Use 25 cm × 30 cm timber $Sm = \frac{25 \times 30^2}{6} = 3750\ cm^3$

unit shear = $\frac{8160}{25 \times 30} \times \frac{3}{2} = 16.3\ kg/cm^2 < 25\ kg/cm^2$

Columns.

Load on columns.



Concentrations. Dead Load 2@1900 = 3,800 kg for each beam assumed
Live Load 2@1500 = 3000 " "

Load on pile ①

Dead Load
3800 × .625 = 2380
3800 × 1.375 = 5220
7600 ÷ 1.25 = 6080 kg

Live Load
6080 × 3000/3800 = 4800 "

Load on pile ②

Dead Load
3800 × 0.625 = 2380
3800 × 0.375 = 1425
3800 × 1.125 = 4275
− 3800 × 0.125 = − 475
7605 ÷ 1.25 = 6090 kg

Live Load
6090 × 3000/3800 = 4800 "

Load on pile ③

Dead Load
3800 × 0.125 = 475
3800 × 0.875 = 3325
3800 × 2 ÷ 1.25 = 6080 kg

Live Load
6080 × 3000/3800 = 4800 "

Dead wt. of pier for one column.

Cantilever arms.	8@.3×.3×2.0 =	1.44
Spacers	7@.2×.4×.45 =	.25
Top beam	.25×.3×5.80 =	.43
battens	2@.075×.2×5.20 =	.16
"	4@.06×.15×1.50 =	.05
diagonals	2@.075×.2×7.00 =	.21
columns	5@.20² × 5.00 =	.79
	3.33@650 =	2,170 kg

For one pile 2170 ÷ 5 = 430 kg.

Load on one column or pile.

Dead Load 6090 + 430 = 6520
Live Load 4800
11,320 kg

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CALCULATIONS FOR

Akigase - Bashi for Saitama - Ken

Unsupported length of column assumed as $l = 500$ cm

Diameter of column at the middle of l $d = 18 + \frac{500}{120} + 2 = 20.1$

Slenderness ratio $l/d = 500/20.1 = 24.9$

allowable unit compression on column $\sigma = 65\left(1 - \frac{l}{60d}\right) = 65\left(1 - \frac{24.9}{60}\right) = 38.1$ kg/cm²

Safe load on one column $P = 38.1 \times \frac{\pi 20.1^2}{4} = 12{,}100$ kg $> 11{,}320$ ✓

For foundation piles use 5 – 18φ × 7.00 m pine piles. Penetration shall be 6.00 m min.

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CALCULATIONS FOR

私ノ講義ノ為ニ計算書ヲ作ルニ不用トナリシ紙ヲ、

15	28
16	30
17	31
18	32
19	33
20	33
21	34
22	35
23	36
24	37
25	38
26	
十三枚	十枚

well 21.7m　　高11.05m

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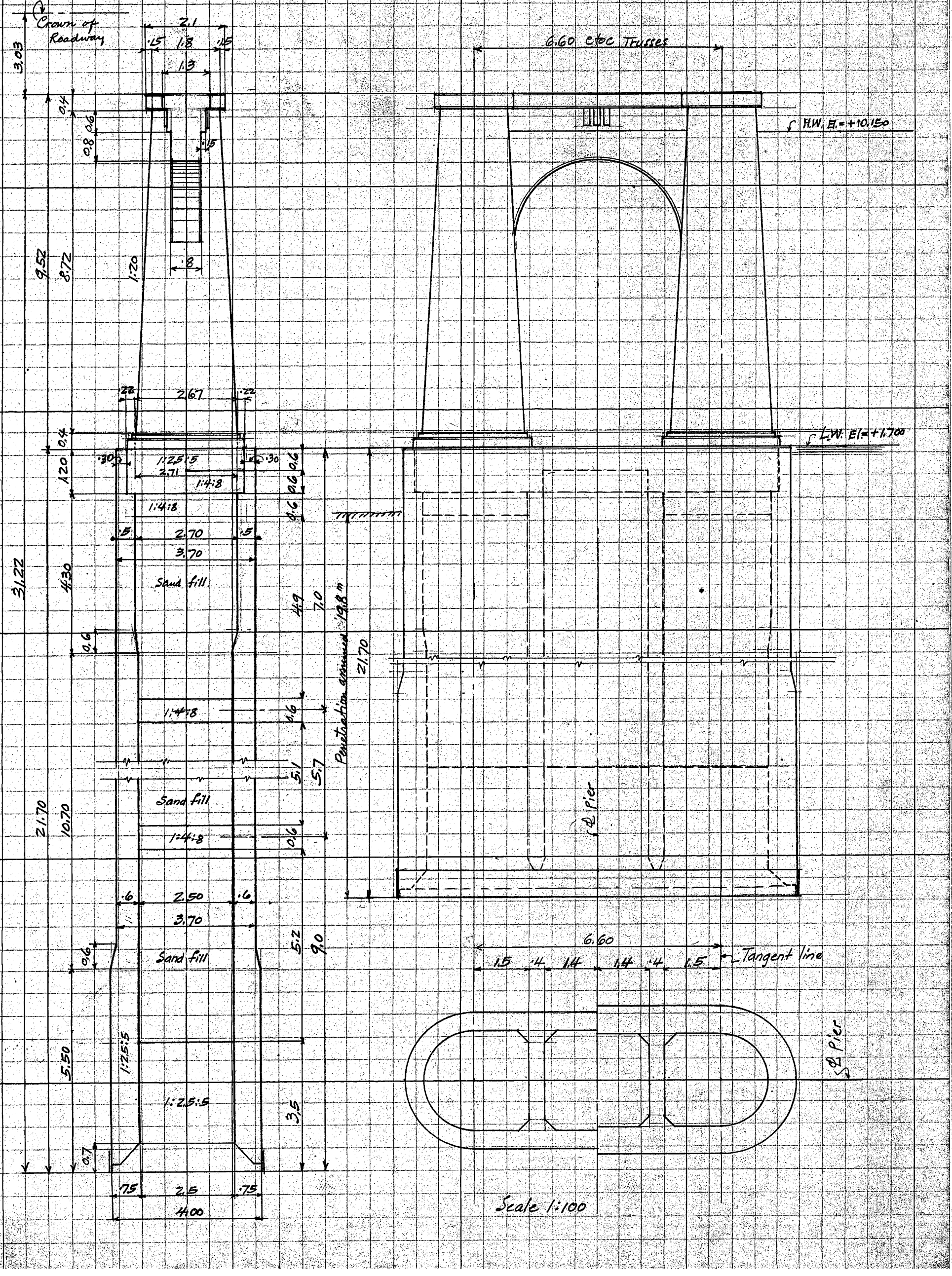
CALCULATIONS FOR

Akigase - Bashi for Saitama - Ken.

General sketch of Pier P_{T3} for River Spans.



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CALCULATIONS FOR

Akigase Bashi for Saitama-ken.

Approximate weight and center of gravity of Pier shaft.

	Dimensions		Volume		Lev. arm.		Moment abt. bott. of Shaft.
Copings	2.10$^{\phi}$ × 0.40 × 2	=	2.77	×	9.32	=	25.80
"	1.30 × 0.40 × 4.70	=	2.44	×	9.32	=	22.75
Strut	1.10 × 0.60 × 4.90	=	3.23	×	8.82	=	28.50
"	0.80 × 1.30 × 4.70	=	4.89	×	7.87	=	38.47
Shafts	2.25$^{\phi}$ × 9.12 × 2	=	72.53	×	3.97	=	288.00
collar footings say	0.20 × 0.40 × 2.89$^{\phi}$ × 2	=	1.45	×	0.20	=	0.29
			87.31 m³		4.63 m		403.81

Weight of shaft = 87.31 @ 2400 = 209,500 kg = S
Seismic force = 209,500 × 0.25 = 52400 " = S'

Approximate weight and center of gravity of well.

Top 1.20 meters.

	3.70$^{\phi}$ × 1.20	=	12.90 @ 2400	=	31,000 × 0.6	=	18,600
	3.70 × 6.60 × 1.20	=	29.30 @ 2400	=	70,300 × 0.6	=	42,200
Less 1:4:8 fill at center	− 3.10 × 0.60 × 2.80	= −	5.21 @ 2400	=	− 12,500 × 0.3	=	− 3,800
add 1:4:8 " "	3.10 × 0.60 × 2.80	=	5.21 @ 2200	=	11,500 × 0.3	=	3,500
					100,300 kg	0.60 m	60,500

Next 4.30 meters. Cross sectional area

Total gross area	3.70$^{\phi}$	=	10.75	
"	3.70 × 6.60	=	24.42	
			35.17 m²	
End partition areas	2.70$^{\phi}$	=	5.73	
"	2.70 × 1.5 × 2	=	8.10	
fillets, less	0.30 × 0.30 × 2	= −	0.18	
			13.65 m²	} 21.03 m²
Center partition area	2.70 × 2.80	=	7.56	
fillet, less.	0.30 × 0.30 × 2	= −	0.18	
			7.38 m²	
Total net area of shell		=	14.14 m²	

weight per lin meter of well for (used in page 23)

Shell and sand filling

14.14	@ 2400 =	33900
21.03	@ 1700 =	35700
		69,600 kg/m

Shell and Concrete filling

14.14	@ 2400 =	33900
21.03	@ 2200 =	46200
		80,100 kg/m

Weight and Center of gravity

Shell	14.14 × 4.30	=	60.80 @ 2,400	=	146,000 × 2.15	=	314,000
1:4:8 conc. fills.	13.65 × 0.60	=	8.19 @ 2,200	=	18,000 × 4.00	=	72,000
Sand fills. ends.	13.65 × 3.70	=	50.50 @ 1,700	=	85,800 × 1.85	=	159,000
" center	7.38 × 4.30	=	31.72 @ "	=	53,900 × 2.15	=	116,000
					303,700 kg	2.18 m	661,000

Next 10.70 meters.

Total gross area 35.17 m²

End partition areas	2.50$^{\phi}$	=	4.91
"	2.50 × 1.5 × 2	=	7.50
fillets, less	0.30 × 0.30 × 2	= −	0.18
			12.23 m² } 19.05 m²
Center partition area	2.50 × 2.80	=	7.00
fillets, less	0.30 × 0.30 × 2	= −	0.18
			6.82 m²
Total net area of shell		=	16.12 m²

weight per lin meter of well for

Shell and sand filling

16.12	@ 2400 =	38,700.
19.05	@ 1700 =	32400
		71,100 kg/m

Shell and concrete filling

16.12	@ 2400 =	38700
19.05	@ 2200 =	41900
		80,600 kg/m

Weight and center of gravity

Shell	16.12 × 10.70	=	172.50 @ 2400	=	414000 × 5.35	=	2,215,000
1:4:8 conc. fills	19.05 × 0.6 × 2	=	22.85 @ 2,200	=	50,300 × 6.35	=	319,000
Sand filling	19.05 × 1.20	=	22.85 @ 1,700	=	38800 × 10.10	=	392,000
"	19.05 × 5.10	=	97.20 @ "	=	165200 × 6.35	=	1048000
"	19.05 × 3.20	=	60.95 @ "	=	103500 × 1.60	=	166000
					771,800 kg	5.37 m	4,140,000

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CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Botton 5.50 meters

Total gross area 4.00$^{\phi}$ = 12.57
" 4.00 × 6.60 = 26.40
38.97 m²

End partition areas 12.23
Center " " 6.82
19.05 m²

Total net area of shell = 19.92 m²

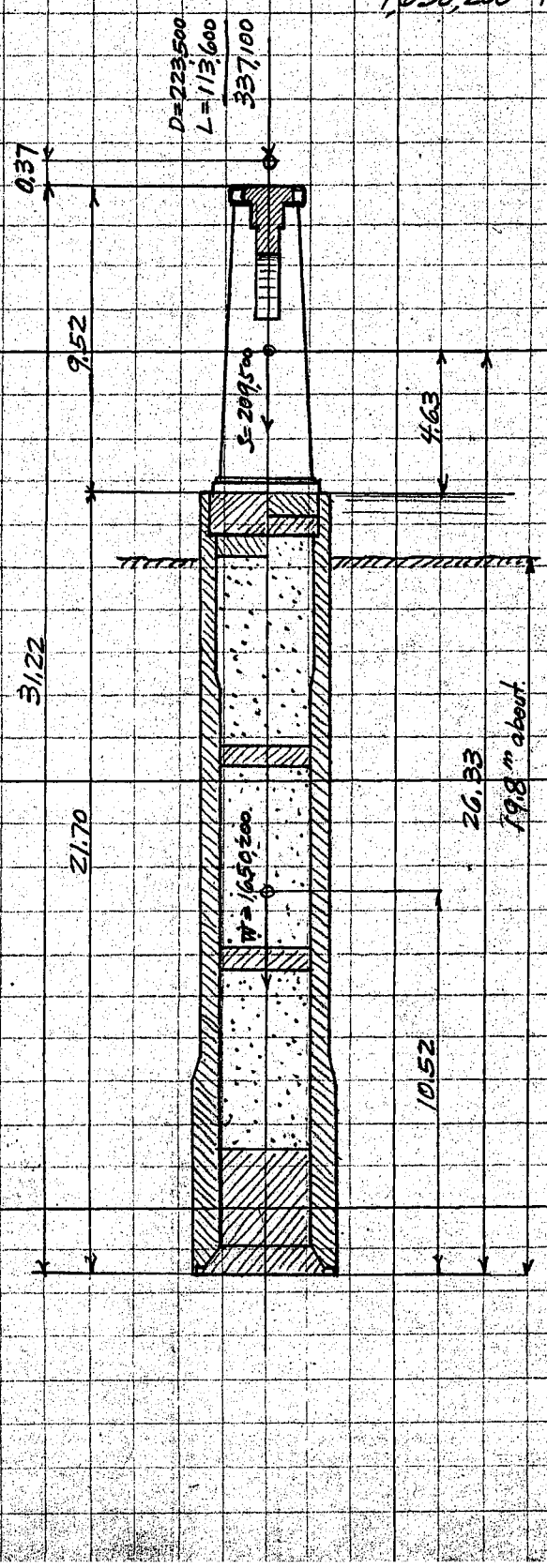
weight per 1m meter of well for
Shell and sand filling
19.92 @ 2400 = 47800
19.05 @ 1700 = 32400
80200 kg/m
Shell and concrete filling
19.92 @ 2400 = 47800
19.05 @ 2200 = 41900
89700 kgm

weight and center of gravity

Shell	19.92	× 5.50 = 109.50	@ 2400	= 263000	× 2.75	=	723000
1:2.5:5 bott. fill	19.05	× 3.50 = 66.67	@ 2200	= 146600	× 1.75	=	257000
Sand filling	19.05	× 2.00 = 38.10	@ 1700	= 64800	× 4.50	=	292000
				474400 kg	2.68 m		1272000

Summary of weight, center of gravity, and volume of concrete + fillings.

	weight	Seismic force	center of gravity	Volume of Concrete 1:2.5:5	1:4:8	Sand filling
Top 1.20 meter	100,300 kg	25,100 kg	0.60 m	36.99 m³	5.21 m³	—
next 4.30 "	303,700	75,900	2.18	60.80	8.19	82.22 m³
" 10.70 "	771,800	193,000	5.37	172.50	22.85	181.00
Botton 5.50 "	474400	118,600	2.68	176.12	—	38.10
	1,650,200 kg	412,600 kg		446.41 m³	36.25 m³	301.32 m³



Center of gravity of well as a whole.

	weight	arm	moment
Top 1.20 meter	100,300	× 21.10 =	2,117,000
next 4.30 "	303,700	× 18.38 =	5,580,000
" 10.70 "	771,800	× 10.87 =	8,380,000
Botton 5.50 "	474400	× 2.68 =	1,272,000
	W = 1,650,200 kg	10.52 m	17,349,000

Seismic force W' = 412,600 kg

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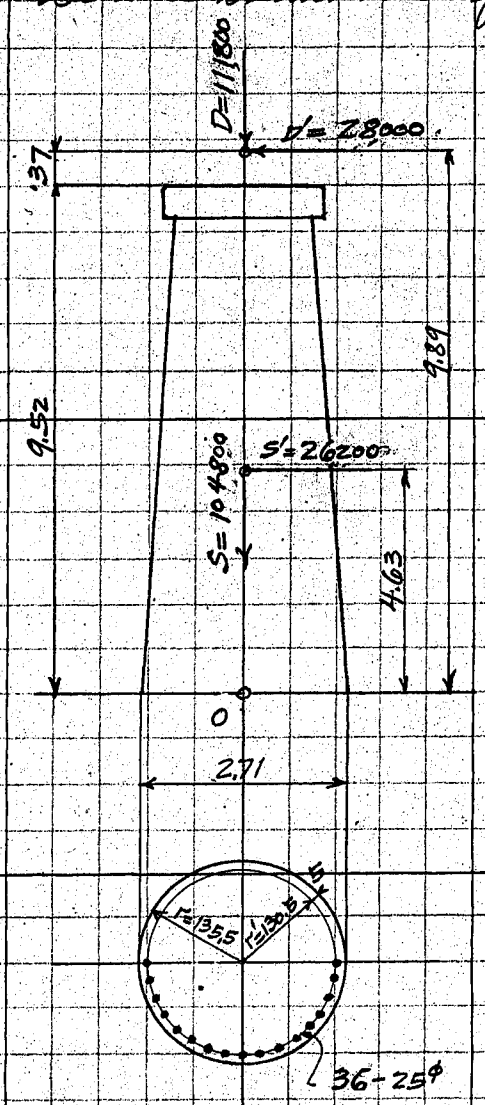
CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Seismic Stresses in Shaft at bottom section.



Loads on one column shaft.

		Seismic loads.
Superimposed Dead Load	D = 11,800,	D' = 28,000
Weight of shaft	S = 11,9800,	S' = 30000

Taking moment about center of bottom area O.

Load marks	Hor. forces	Vert. forces	Lev. arm	moment abt. O
D		11,800 ×	0 =	0
D'	28,000	×	9.89 =	277,000
S		104800 ×	0 =	0
S'	26200	×	4.63 =	121300
	54200 kg	N = 216,600 kg	e = 1.84 m	398300

Steel area assumed as A_s = 36 − 25φ bars = 176.7 cm²

Steel ratio $P = \frac{176.7}{135.5^2\pi} = 0.00306$

$r = 135.5$ cm, $r' = 130.5$ cm; $r'/r = \frac{130.5}{135.5} = 0.963$, $e/r = \frac{184}{135.5} = 1.36$

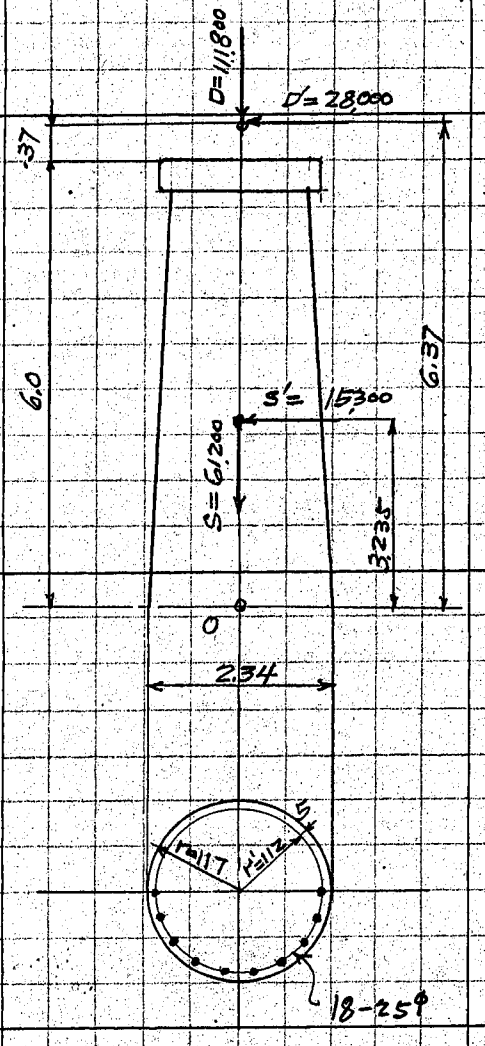
From the diagrams, we have as follows.

$k = 0.640$, $C = 0.275$

$$f_c = \frac{216600}{135.5^2 \times 0.275} = 42.9 \text{ kg/cm}^2 < 40.5 \times 1.6 = 64.8 \text{ (for 1:2.5:5 concrete)}$$

$$f_s = 42.9 \times 15 \frac{1 + 0.963^{1.323} - 0.640}{0.640} = 1330 \text{ kg/cm}^2 < 1200 \times 1.6 = 1920 \text{ (seismic stress)}$$

Seismic stresses in shaft at section 6 meters below top.



Weight and center of gravity.

Coping		2.77 × 5.80	=	16.07
"		2.44 × 5.80	=	14.15
Strut		3.23 × 5.30	=	17.12
"		4.89 × 4.35	=	21.28
Shafts	2.07φ × 5.60 × 2 = 37.70 / 51.03	× 2.56 = 3.235 m		96.50 / 165.12

S = 51.03 ÷ 2 @ 2400 = 61,200 kg

S' = 61200 × 0.250 = 15,300 "

Taking moment about point O.

Load marks	Hor. forces	Vert. forces	Lev. arms	moment
D		11,800 ×	0 =	0
D'	28000	×	6.37 =	178,300
S		61,200 ×	0 =	0
S'	15300	×	3.235' =	49,500
	43,300 kg	N = 173,000 kg	e = 1.317 m	227,800

$e/r = 131.7/117 = 1.125$

Steel area assumed as 18 − 25φ = 88.4 cm² = A_s

Steel ratio $p = \frac{88.4}{117^2\pi} = 0.00206$

$r = 117$, $r' = 112$; $r'/r = 112/117 = 0.958$

$k = 0.64$, $C = 0.30$

$$f_c = \frac{173000}{117^2 \times 0.30} = 42.1 \text{ kg/cm}^2 < 64.8$$

$$f_s = 15 \times 42.1 \frac{1 + 0.958^{1.318} - 0.64}{0.64} = 1300 \text{ "} < 1920$$

Assumed sections are ample.

増田淳氏関係資料
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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

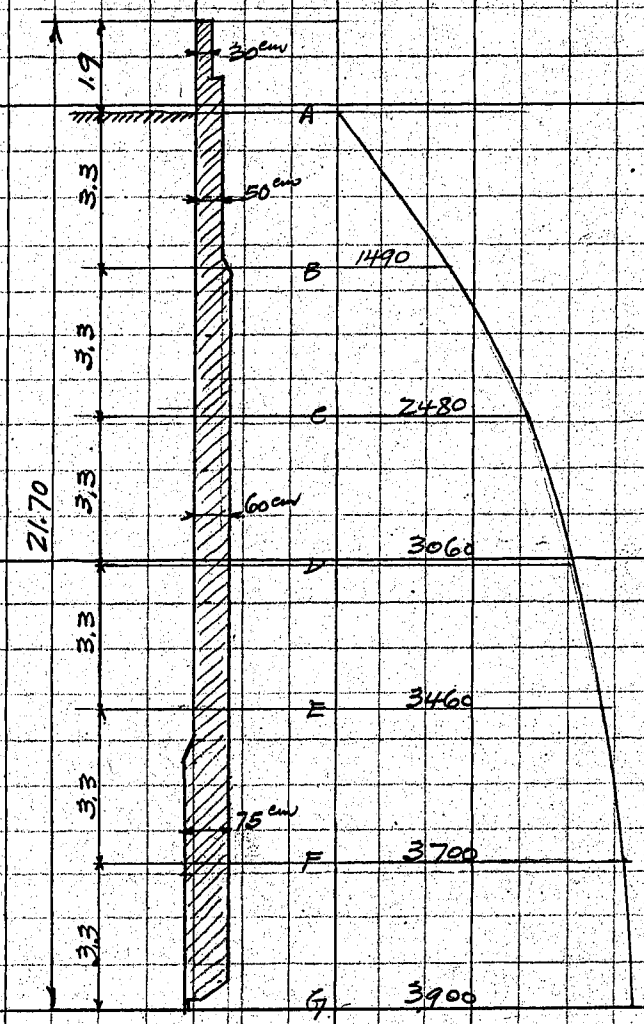
Design of well.

Temporary earth pressure on well during sinking execution. (Referred to Ketchum's "Walls, Bins, and Grain Elevators", on page 120 and 121.)

$$L = \frac{cwb}{2\mu}\left(1 - e^{-\frac{2k\mu y}{b}}\right), \quad V = \frac{cwb}{2k\mu}\left(1 - e^{-\frac{2k\mu y}{b}}\right).$$

where V = Vertical unit pressure in kg/m^2 at depth y meters;
L = Lateral " " " " " " " " ;
cw = weight of earth in kg/m^3;
$\mu = \tan\phi$ = Coefficient of friction of earth on earth,
ϕ = angle of repose of the earth;
b = the distance in meters that the earth breaks around the well;
K = a constant $= \frac{1-\sin\phi}{1+\sin\phi}$;

Let us assume that, $cw = 1600\ kg/m^3$; $B = 3.0$ meters; and $K = 1/3$.
From the prepared diagram, we have the temporary earth pressures as follows:



Sections	Depths of earth y in meters	Temporary pressure on well L in kg/m^2
B	3.30	1,490
C	6.60	2,480
D	9.90	3,060
E	13.20	3,460
F	16.50	3,700
G	19.80	3,900

Bending moment on wall at several sections:
Span length of wall assumed as 3.20 meters for straight walls, Diameter of end rings 3.2, 3.1 and 3.25 m
moments assumed as $\pm 1/12\ Ll^2 = \pm 1/12 L \times 3.20^2 = \pm 0.853L$ for straight walls;
$\pm 1/16\ LD^2 = \pm 1/16 L \times 3.20^2 = \pm 0.640L$ " end rings
or $\pm 1/16 L \times 3.10^2 = \pm 0.601L$ " "
or $\pm 1/16 L \times 3.25^2 = \pm 0.660L$ " "

Design all walls for the moment of $\pm 0.853L$

Sections	Earth pressure L in kg/m^2	moment $0.853L$ in kgm.	Eff. depth of wall in cm	Steel area reqd. in cm^2	Spacings for 16ϕ bars in cm	unit Shear v in kg/cm^2
A	0	0	46	0	∞	
B	1,490	1,270	46	2.63	76.5	0.6
			56	2.16	93.2	0.5
C	2,480	2,120	56	3.61	55.7	0.8
D	3,060	2,610	"	4.44	45.3	1.0
E	3,460	2,950	"	5.02	40.1	1.1
F	3,700	3,160	71	4.24	47.4	1.0
G	3,900	3,330	"	4.47	45.0	1.0

note: Steel area required $= \frac{M \times 100}{1200 \times \frac{7}{8} \times d} = \frac{M}{10.5d}$

unit shear $= \frac{V}{100 \times \frac{7}{8} d} = \frac{1.6L}{100 \times \frac{7}{8} d} = 0.0183 \frac{L}{d}$

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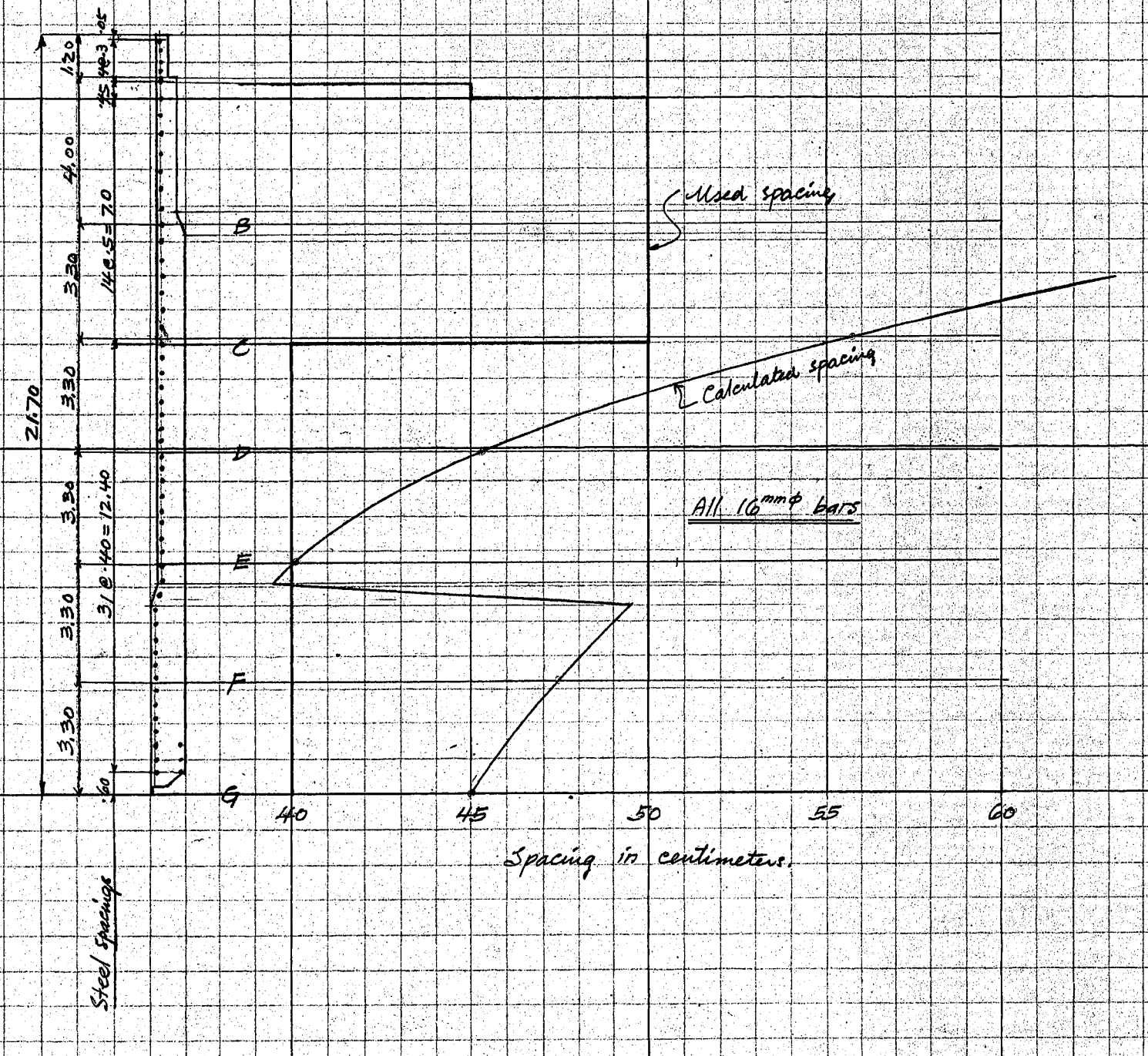
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Spacing diagram of horizontal reinforcements in side walls.



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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Stability of Pier.

Stability at normal state.

Superimposed Dead Load on pier D = 223,500 — see on page 14
Live Load on pier L = 113,600
337,100

weight of shaft S = 239,500 — see on page 16
weight of well W = 1650,200 — " " 17
1889,700

2226,800 kg

Frictional resistance between well and earth.

Resistance assumed as 1200 kg/m²

Area of skin friction

3.70φ = 11.62
6.60×2 = 13.20
24.82 × 14.30 = 355.0

4.00φ = 12.57
6.60×2 = 13.20
25.77 × 5.50 = 141.7
496.7 m²

Frictional resistance = 496.7 @ 1200 = 595,800 kg

Total net load on bottom area 2226,800 − 595,800 = 1631,000 kg

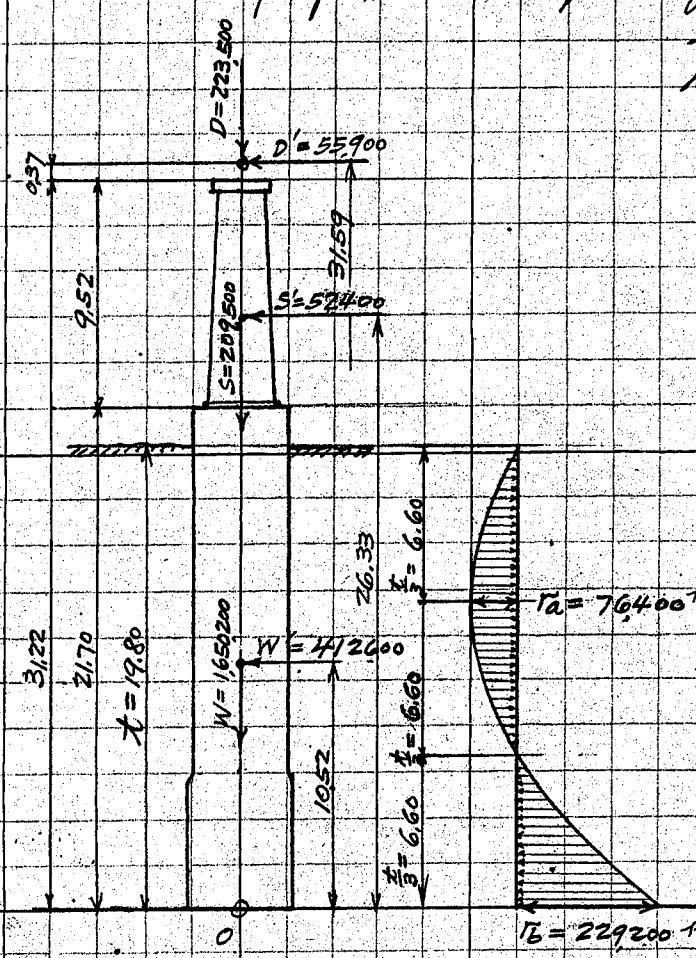
Base area = 38.97 m² — see on page 17

Unit bearing pressure = 1631,000 / 38.97 = 41,800 kg/m² (corresponds to 3.82 kg/cm²)

Stability of Pier during earthquake. k assumed as 0.250

Seismic moment about center of bottom area O.

Load marks	Hor. forces	Vert. forces	Lev. arms	moment about O
D		223,500 ×	0 =	0
D'	55,900	×	31.59 =	1765,000
S		209,500 ×	0 =	0
S'	524.00	×	26.33 =	1380,000
W		1650,200 ×	0 =	0
W'	412,600	×	10.52 =	4340,000
	520,900 kg	2083,200 kg		M = 7,485,000 kgm



Horizontal bearing pressure on earth

$T_b = \frac{12M}{t^2} = \frac{12 \times 7485000}{19.8^2} = 229,200$ kg per meter strip

$T_a = \frac{T_b}{3} = \frac{229200}{3} = 76400$ "

For the earthquake along the ℄ Bridge

Limit bearing side pressure

$T_b = \frac{229200}{10.6} = 21,620$ kg/m²

$T_a = \frac{76400}{10.3} = 7420$ kg/m²

For the earthquake across the ℄ Bridge.

$T_b = \frac{229200}{4.00} = 57,300$ kg/m²

$T_a = \frac{76400}{3.70} = 20650$ "

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken

Allowable values of bearing pressures r_a and r_b.

Earth only.

Upward passive pressure of earth allowable.

$$p = w x \frac{1+\sin\phi'}{1-\sin\phi'}$$ where $\phi' = \phi - \tan^{-1} k = \phi - \tan^{-1} 0.250$

ϕ = angle of repose of filling at normal state, being assumed as 30° for earth.

x = depth of filling in meters.

w = weight of filling arround the well.

$\phi' = 30° - 14° = 16°$

$\sin\phi' = 0.276$

$\frac{1+\sin\phi'}{1-\sin\phi'} = \frac{1.276}{0.724} = 1.765$

for $x = 6.6$ meters $r_a = 1600 \times 6.6 \times 1.765 = 18650$ kg/m²

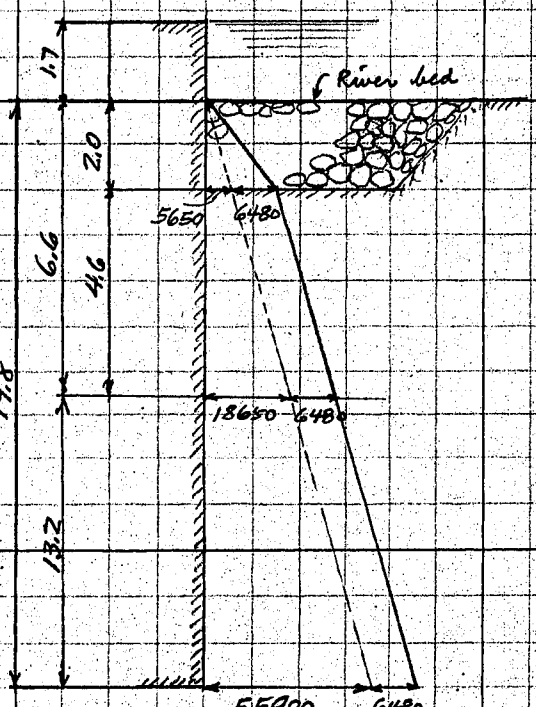
" $x = 19.8$ " $r_b = 1600 \times 19.8 \times 1.765 = 55900$ "

The structure is safe for the earthquake along Q bridge.

Use rubble filling 2 meters deep on upstream and downstream ends of well.

For rubble filling ϕ assumed as 45°

Assuming void of rubble being filled partially with sand, let us take the value of w as 2000 kg/m².



$\phi' = 45° - 14° = 31°$, $\sin\phi' = 0.515$ ∴ $\frac{1+\sin\phi'}{1-\sin\phi'} = \frac{1.515}{0.485} = 3.12$

for $x = 2.0$ meters $p = 2000 \times 2.0 \times 3.12 = 12480$ kg/m²

for earth filling and $x = 2.0$

$p = 1600 \times 2.0 \times 1.765 = 5650$ "

difference = 6480 "

for earth filling and $x = 6.6$

$p = 1600 \times 6.6 \times 1.765 = 18650$ "

" " $x = 19.8$

$p = 1600 \times 19.8 \times 1.765 = 55900$ "

Total pressure allowable

$r_a = 18650 + 6480 = 25130$ kg/m²

$r_b = 55900 + 6480 = 62380$ "

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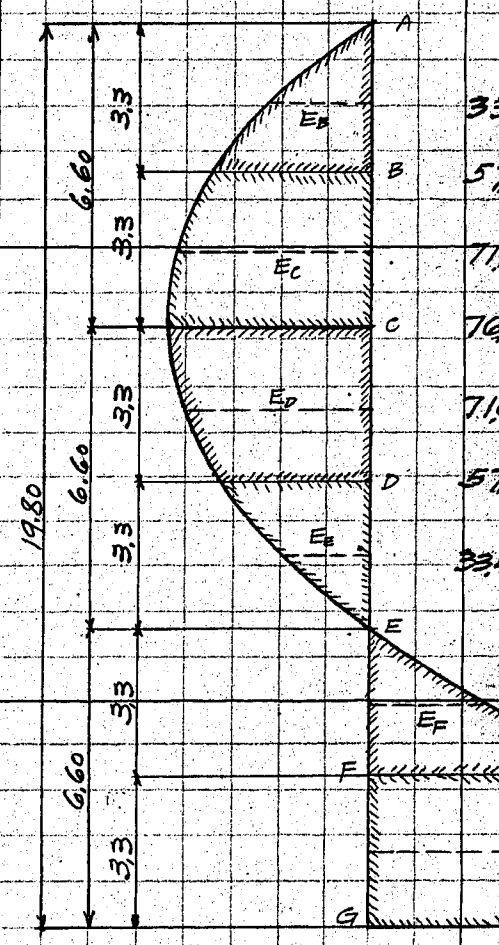
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Seismic earth pressure during earthquake along the ℄ bridge.
By Simpson's formula.

$$\text{area} = \frac{d}{3}[h_1 + 4h_2 + h_3] \quad \text{where } d = 1.65 \text{ meters}$$
$$= 0.55[h_1 + 4h_2 + h_3]$$



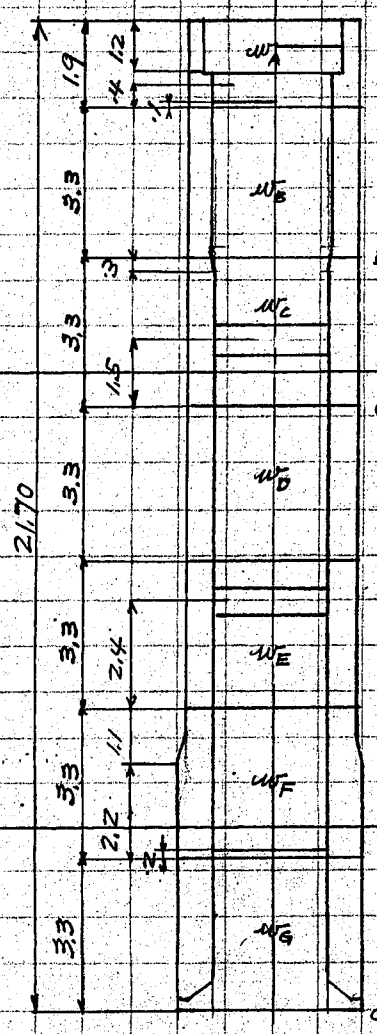
	Total Pressures/m.
A	0
	33,420
B	57,300
	71,680
C	76,400
	71,630
D	57,300
	33,420
E	0
	−42,480
F	−95,500
	−157,580
G	−229,200

Seismic Pressures	$h_1+4h_2+h_3$		$\frac{d}{3}$		Pressures.	C.g.	Lever arms from bottom of well.
E_B	191,000	×	0.55	=	105,100	1.10 m	17.60 m
E_C	420,200	×	"	=	231,100	1.57	14.77
E_D	420,200	×	"	=	231,100	1.73	11.63
E_E	191,000	×	"	=	105,100	2.20	8.80
E_F	267,400	×	"	=	−147,100	1.08	4.38
E_G	955,000	×	"	=	−525,300	1.47 ✓	1.41 ✓

Summary of (+) area = 672400 kg
" " (−) " = 672,400 "

Check: $\frac{2}{3} \times 76400 \times 13.20 = 672,400$ kg ✓

Weight of each section of well. (Refer to page 16 + 17)



W_A

					weight		arm		moment	Seismic forces.
Top 1.20 meters					100,300	×	1.30	=	130,400	
next 0.60 " say	0.60	@	80,100	=	48100	×	0.40	=	19200	
" 0.10 "	0.10	@	69,600	=	7000	×	0.05	=	400	
					W_A = 155,400 kg		0.97 m		150,000	W_A' = 38900 kg

W_B

					arm			
W_B =	3.30	@	69,600	=	229,700 kg	1.65 m		W_B' = 57400

W_C

Top 0.30 meter	0.30	@	69,600	=	20,900	×	3.15	=	65,800	
next 1.20 "	1.20	@	71,100	=	85,300	×	2.40	=	204,500	
next 0.60 "	0.60	@	80,600	=	48,400	×	1.50	=	72,600	
bott. 1.20 "	1.20	@	71,100	=	85,300	×	0.60	=	51,200	
					W_C = 239,900 kg		1.64 m		394,100	W_C' = 59,900

W_D

					arm			
W_D =	3.30	@	71,100	=	234,800 kg	1.65 m		W_D' = 58,700

W_E

Top 0.60 meter	0.60	@	71,100	=	42,700	×	3.00	=	128,000	
next 0.60 "	0.60	@	80,600	=	48,400	×	2.40	=	116,200	
bott. 2.10 "	2.10	@	71,100	=	149,300	×	1.05	=	156,800	
					W_E = 240,400 kg		1.67 m		401,000	W_E' = 60,100

W_F

Top 1.10 meters	1.10	@	71,100	=	78,200	×	2.75	=	215,000	
next 2.00 "	2.00	@	80,200	=	160,400	×	1.20	=	192,500	
bott. 0.20 "	0.20	@	89,700	=	17,900	×	0.10	=	1,800	
					W_F = 256,500 kg		1.60 m		409,300	W_F' = 64,100

W_G

					arm			
W_G =	3.30	@	89,700	=	296,000 kg	1.65 m		W_G' = 74000

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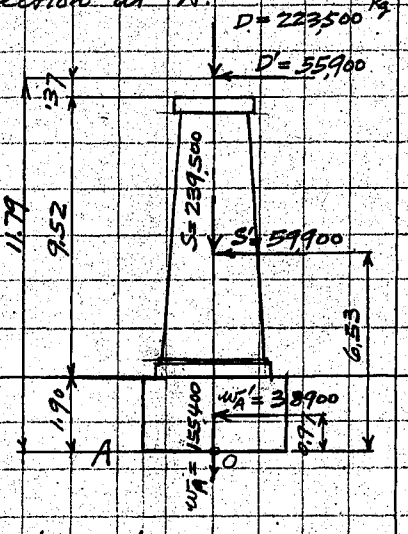
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Moments at several sections of well during earthquake.

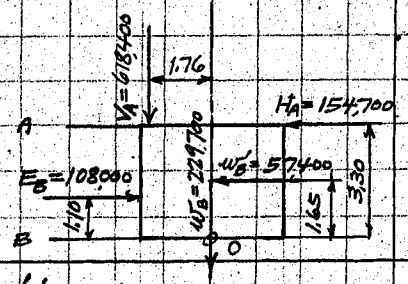
Section at A.



Taking moment about point O.

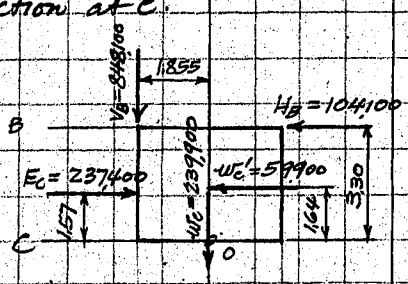
Load marks	Horizontal forces	Vertical forces	Lever arms	Moment about point O.
D		223,500		
D'	55,900		× 11.79 =	659,000
S		239,500		
S'	59,900		× 6.53 =	391,000
W_A		155,400		
W_A'	38,900		× 0.97 =	37,700
	H_A = 154,700	V_A = 618,400	e_A = 1.76 m	M_A = 1,087,700 kgm

Section at B.



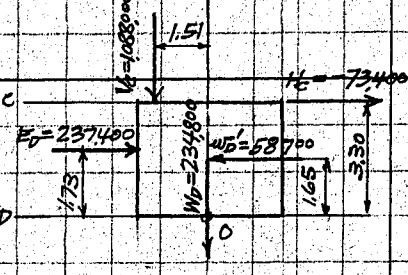
V_A		618,400	× 1.76 =	1,087,700
H_A	154,700		× 3.30 =	510,000
W_B		229,700		
W_B'	57,400		× 1.65 =	94,700
E_B	− 108,000		× 1.10 =	− 118,800
	H_B = 104,100	V_B = 848,100	e_B = 1.855	M_B = 1,573,600

Section at C.



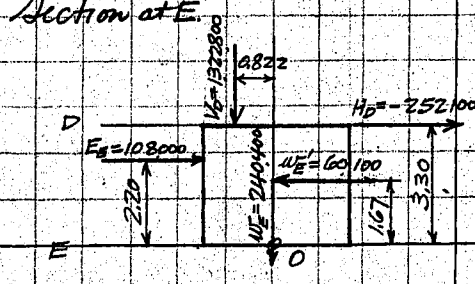
V_B		848,100	× 1.855 =	1,573,600
H_B	104,100		× 3.30 =	343,500
W_C		239,900		
W_C'	59,900		× 1.64 =	98,200
E_C	− 237,400		× 1.57 =	− 372,600
	H_C = − 73,400	V_C = 1,088,000	e_C = 1.51	M_C = 1,642,700

Section at D.



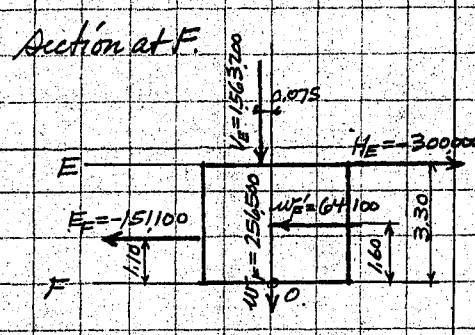
V_C		1,088,000	× 1.51 =	1,642,700
H_C	− 73,400		× 3.30 =	− 242,200
W_D		234,800		
W_D'	58,700		× 1.65 =	96,900
E_D	− 237,400		× 1.73 =	− 410,500
	H_D = − 252,100	V_D = 1,322,800	e_D = 0.822	M_D = 1,086,900

Section at E.



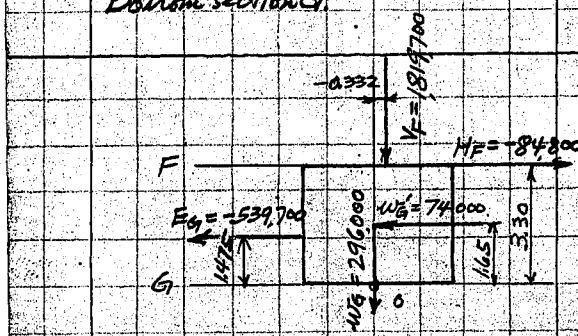
V_D		1,322,800	× 0.822 =	1,086,900
H_D	− 252,100		× 3.30 =	− 832,000
W_E		240,400		
W_E'	60,100		× 1.67 =	100,200
E_E	− 108,000		× 2.20 =	− 237,600
	H_E = − 300,000	V_E = 1,563,200	e_E = 0.075	M_E = 117,500

Section at F.



V_E		1,563,200	× 0.075 =	117,500
H_E	− 300,000		× 3.30 =	− 990,000
W_F		256,500		
W_F'	64,100		× 1.60 =	102,500
E_F	+ 151,100		× 1.10 =	166,200
	H_F = − 84,800	V_F = 1,819,700	e_F = −0.332	M_F = − 603,800

Bottom section G.



V_F		1,819,700	× −0.332 =	− 603,800
H_F	− 84,800		× 3.30 =	− 280,000
W_G		296,000		
W_G'	74,000		× 1.65 =	122,100
E_G	+ 539,700		× 1.415 =	763,500
	H_G = + 528,900	V_G = 2,115,700	e_G = 0.00	M_G = 1,800

M_G must theoretically be zero.

増田淳氏関係資料
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H 3 mm
V 216 mm
M 1630 mm

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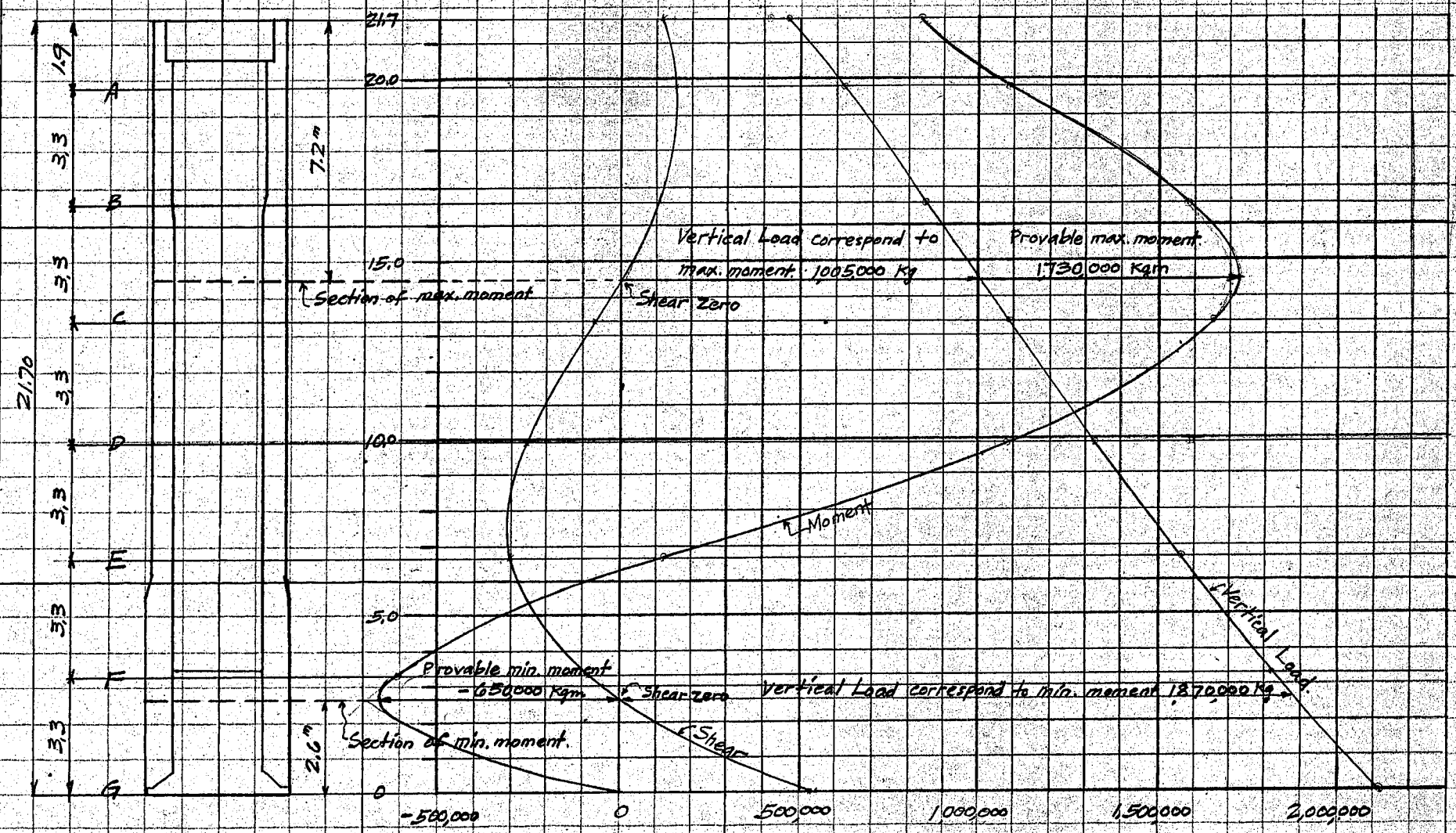
CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Moments, shears, and direct load diagram of well.



Moments in kgm, Shears & Loads in kg.

Vertical Reinforcements in the well.

Maximum moment on well scaled from the above diagram = 1,730,000 kgm
Corresponding vertical load on well = 1,005,000

Total net area of shell = 16.12 m² (see on page 16)

Direct compression on concrete = $\frac{1005,000}{16.12 \times 100^2}$ = 6.2 kg/cm² C

Bending stress. Lever arm 2.50+0.6 = 3.1 m, Effective width of well say = 9.0 meters

Stress on shell = $\frac{1,730,000}{3.1 \times 9.0}$ = 62,000 kg C or T per lin. meter of side walls.

Unit stress = $\frac{62000}{100 \times 60}$ = 10.3 kg/cm² T or C

Combined stress.

Direct stress	6.2 C	6.2 C
moment stress	10.3 C	10.3 T
	16.5 kg/cm² C	4.1 kg/cm² T

Vertical reinforcements required = $\frac{4.1 \times 60 \times 100}{1200 \times 1.6}$ = 12.81 cm² meter of wall.

Use 19 mm⌀ bars at 140 cm c to c = 2 × 2.835 ÷ 0.40 = 14.18 cm² " "

Unit stresses in reinforcements

Tension $\frac{4.1 \times 60 \times 100}{14.18}$ = 1735 kg/cm² < 1200 × 1.6

Compression 16.5 × 15 = 247 " < "

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CALCULATIONS FOR

Akigase - Bashi for Saitama - Ken.

Minimum moment on well scaled from the diagram = - 650,000 kgm
Corresponding vertical load on well = 1,870,000 kg
Assuming the vertical load to be totally carried by the shell.
net area of shell = 19.92 m^2 (See on page 17)

Direct compression on concrete = $\frac{1870000}{19.92 \times 100^2}$ = 9.4 kg/cm^2 C

Bending stress Lever arm = 3.25 m, effective width of well say = 9.2 metres.
Stress on shell = $\frac{650.000}{3.25 \times 9.2}$ = 21,750 kg T or C per lin meter of side wall,

Unit stress = $\frac{21.750}{100 \times 75}$ = 2.9 kg/cm^2 T or C

Combined stress.

Direct stress	9.4 C	9.4 C
moment stress	2.9 C	2.9 T.
	12.3 kg/cm^2 C,	6.5 kg/cm^2 C

For vertical reinforcements, use 199 bars at 40 cm c to c on both sides throughout the well.

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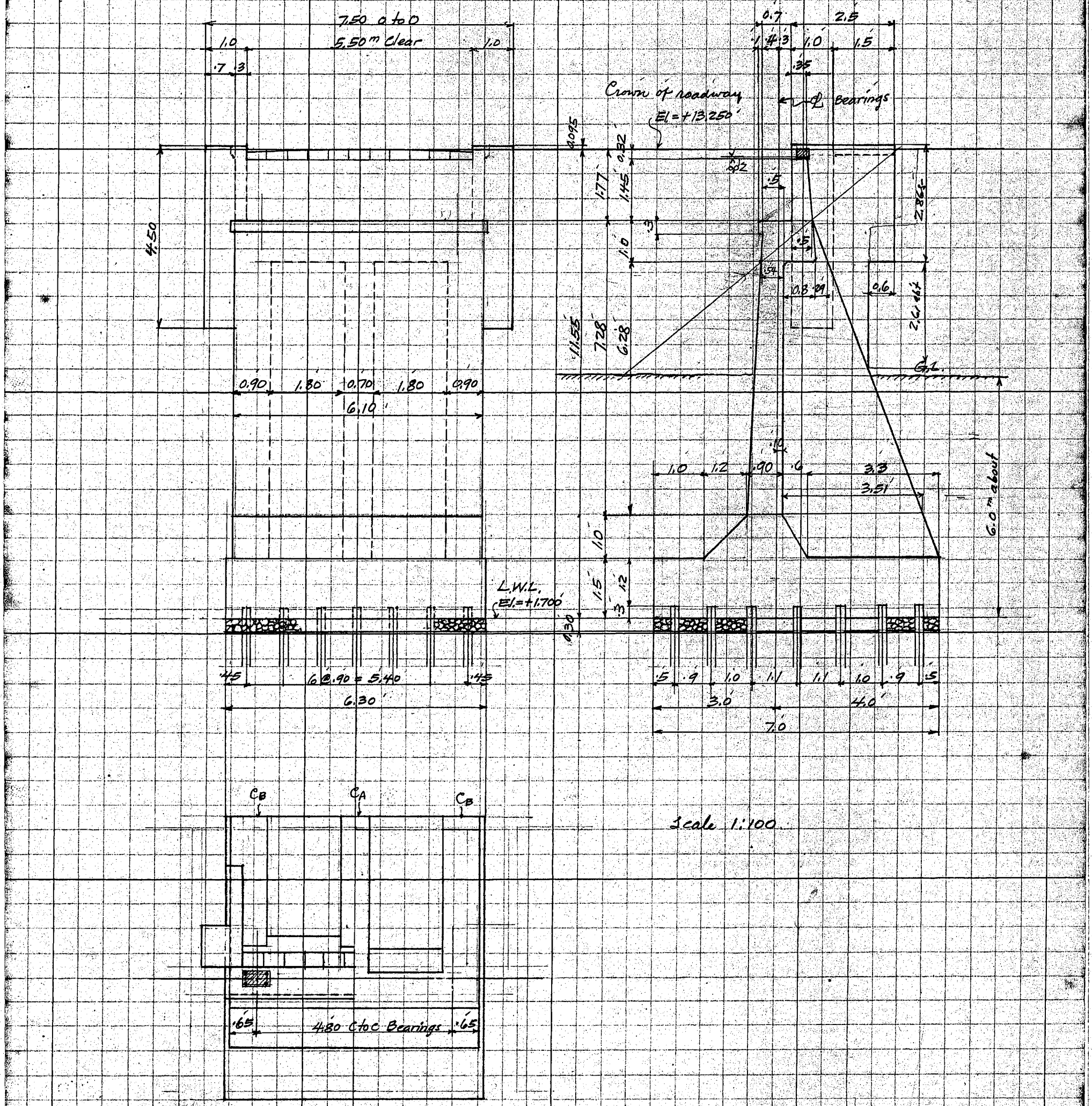
CALCULATIONS FOR

Ikigase-Bashi for Saitama-Ken.

General sketch of abutment for girder span, AG.



Scale 1:100

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CHECKED BY H. T DATE 8-10-26 PAGE NO 30

CALCULATIONS FOR

Ikigase-Bashi for Saitama-Ken.

Effective depth req'd for 1:2.5:5 concrete:

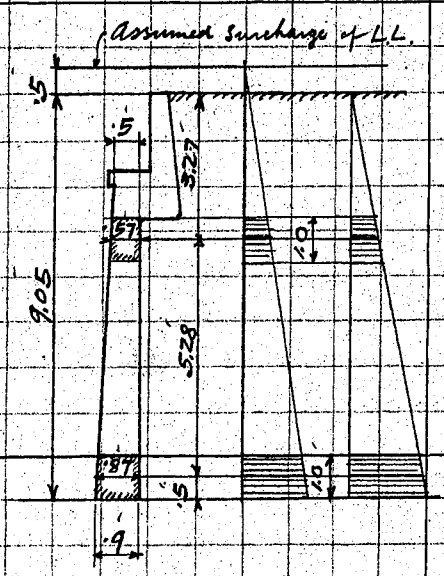
$d = \sqrt{\frac{3500\times100}{100\times6.04\times1.6}} = 19$ cm. use 27 cm eff. depth with 3 cm insulation

Steel area required $= \frac{3500\times100}{1200\times1.6\times\frac{7}{8}\times27} = 7.72$ cm² for 1 meter strip at bottom.

use 16 mmφ bars at 25 cm c to c = 8.05 cm² on inside of wall.
13 mmφ " " 25 cm c to c = 4.52 cm² on outside of wall.

Design of Curtain wall. Span length assumed as $1.80 + \frac{0.7}{2} + \frac{0.9}{2} = 2.60$ meters.

Stresses at normal state.



Earth pressure on top 1 meter strip

$1/3 \times 1600 \times 3.77 = 2,010$ kg/m²

moment $= 1/10 \times 2010 \times 2.60^2 = 1360$ kgm

Shear $= 1/2 \times 2010 \times 2.60 = 2,610$ kg

Earth pressure on bottom 1 meter strip

$1/3 \times 1600 \times 9.05 = 4830$ kg/m²

moment $= 1/10 \times 4830 \times 2.60^2 = 3270$ kgm

Shear $= 1/2 \times 4830 \times 2.60 = 6,280$ kg

Stresses during earthquake.

Earth pressure and seismic thrust on top 1 meter strip

$0.559 \times 1600 \times 3.27 = 2920$

$0.57 \times 1.0 @ 2400 \times 0.25 = 340$

3260 kg/m²

moment $= 1/10 \times 3260 \times 2.60^2 = 2,200$ kgm

Shear $= 1/2 \times 3260 \times 2.60 = 4240$ kg

Earth pressure and seismic thrust on bottom 1 meter strip

$0.559 \times 1600 \times 8.55 = 7650$

$0.87 \times 1.0 @ 2400 \times 0.25 = 520$

8170 kg/m²

moment $= 1/10 \times 8170 \times 2.60^2 = 5560$ kgm

Shear $= 1/2 \times 8170 \times 2.60 = 10620$ kg.

Seismic stresses govern the sections of curtain wall.

Sections:—

Top 1 meter strip

Effective depth required $= \sqrt{\frac{2200\times100}{100\times6.04\times1.6}} = 15.1$ cm

use 53 cm eff. depth with 4 cm insulation.

Steel area required $= \frac{2200\times100}{1920\times\frac{7}{8}\times53} = 2.47$ cm²

use 16 mmφ bars at 70 cm c to c = 2.88 cm² on both sides

unit shear $= \frac{4240}{100\times\frac{7}{8}\times53} = 0.92$ kg/cm²

Bottom 1 meter strip

Eff. depth req'd $= \sqrt{\frac{5560\times100}{100\times6.04\times1.6}} = 24.0$ cm

use 83 cm eff. depth with 4 cm insulation.

Steel area req'd. $= \frac{5560\times100}{1920\times\frac{7}{8}\times83} = 3.99$ cm²

use 16 mmφ bars at 45 cm c to c = 4.47 cm² on both sides

unit shear. $= \frac{10620}{100\times\frac{7}{8}\times83} = 1.46$ kg/cm²

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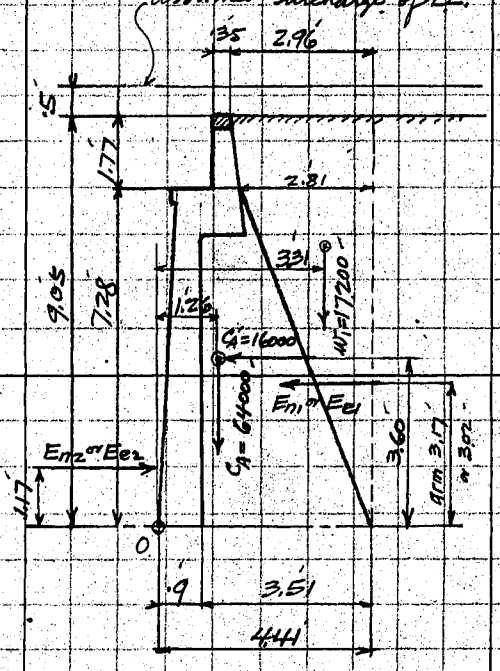
CALCULATIONS FOR

Akigase - Bashi for Saitama - Ken.

Design of Counterfort wall C_A at center.



Weight and center of gravity of counterfort wall C_A, width = 2.50 meters

			weight	arm	Vert. m	arm	Hor. m.
Granite	0.30×0.30×2.50 =	0.23 @ 2600 =	600	8.90	5340	1.35	810
parapet wall	0.43×1.47×2.50 =	1.58 @ 2400 =	3790	7.97	30200	1.31	4970
top strut	1.25×1.00×2.50 =	3.13 @ " =	7500	6.78	50800	1.00	7500
Coping	0.10×0.30×2.50 =	0.08 @ " =	190	7.13	1360	0.35	70
Curtain wall	0.72×2.50×6.28 =	11.30 @ " =	27120	2.86	77600	0.53	14350
Counterfort	0.70×2.30×6.28 =	10.10 @ " =	24240	2.58	62500	2.15	52100
" top △	0.15×0.70×1.00 =	0.11 @ " =	260	6.61	1720	1.77	460
misc concrete say		0.125 @ " =	300	3.60	1080	1.26	380
Volume of granite		0.23 m³	C_A = 64000 kg		230600	1.26 m	80640
" " concrete		26.43 "	Seismic force C_A' = 64000×0.25 =		16000 kg		

Weight and center of gravity of earth on counterfortwall.

2.89×1.77×0.70 =	3.58 @ 1600 =	5730	8.17	46800	2.97	17020
1.41×7.28×0.70 =	7.18 @ 1600 =	11470	4.85	55600	3.47	39800
	10.76	W_1 = 17200 kg	5.96 m	102400	3.31 m	56820

Earth pressure at normal state on rear side.

1/3 × 1600 × 0.50 = 267

1/3 × 1600 × 9.55 = 5090

5357 ÷ 2 = 2679 kg/m² average.

Earth pressure E_{n1} = 2679 × 9.05 × 2.50 = 60600 kg arm 3.17 m

Earth pressure on front side at normal state.

Depth of earth filling taken as 3.50 meters

Earth pressure E_{n2} = 1/6 × 1600 × 3.50² × 2.50 = 8170 kg arm 1.17 m

Seismic earth pressure on rear side

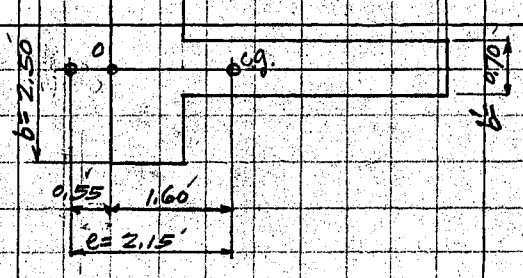
E_{e1} = 0.559 × 1600 × 9.05² ÷ 2 × 2.5 = 91600 kg arm 3.02 m

Do on front side

E_{e2} = 0.559 × 1600 × 3.50² ÷ 2 × 2.5 = 13700 kg arm 1.17 m

note :- E_{e1} and E_{e2} are assumed not to act together, ie, one of them act in each case of earthquake whose direction considers to its direction.

Stresses in counterfortwall C_A at normal state.



Taking moment about toe O.

Load marks	Hor. forces	Vert. forces	Lev. arms	Moment about point O.
C_A		64000	× − 1.26	= − 80600
W_1		17200	× − 3.31	= − 56800
E_{n1}	+ 60600		× + 3.17	= + 192000
E_{n2}	− 8170		" + 1.17	= − 9600
	+ 52430 kg	81200 kg	+ 0.55 m	+ 45000

Center of gravity of bottom section.

2.50 × 0.90 = 2.25 × 0.45 = 1.013

3.51 × 0.70 = 2.46 × 2.66 = 6.542

4.71 m² 1.60 m 7.555

Eccentricity e = 1.60 + 0.55 = 2.15 meters.

moment at bottom section = 81200 × 2.15 = 174500 kgm

Shear " " " = + 52430 kg

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken

Stresses during earthquake.

Seismic force toward river side.

Taking moment about toe O.

Load marks	Hor. forces	Vert. force	Leverarms	moments about O.
C_A		64,000	× −1.26 =	− 80,600
C_A'	16,000		× 3.60 =	57,600
W_1		17,200	× −3.31 =	− 56,900
E_{e1}	91,600		× 3.02 =	276,600
	107,600 kg	81,200 kg	2.42 m	196,700

Eccentricity e = 1.60 + 2.42 = 4.02 m

Moment on bottom section = 81,200 × 4.02 = 326,000 kgm

Shear on " " = 107,600 kg

Seismic force toward shore side.

Taking moment about toe O.

C_A		64,000	× −1.26 =	− 80,600
C_A'	− 16,000		× + 3.60 =	− 57,600
W_1		17,200	× −3.31 =	− 56,900
E_{e2}	− 13,700		× + 1.17 =	− 16,000
	− 29,700 kg	81,200 kg	− 2.60 m	− 211,100

Eccentricity e = 1.60 − 2.60 = −1.00 m

Moment on bottom section = −81,200 × 1.00 = − 81,200 kgm

Shear " " " = − 29,700 kg

Governing stresses:—

			corresponding vertical load.
positive moment	+ 326,000 kgm	during earthquake	81,200 kg
negative moment	− 81,200 "	" "	81,200 "
max shear	107,600 kg	" "	

Reinforcements in counterfort wall.

Positive moment.

Let the angle subtended by the rear face of counterfort wall with the vertical = θ

$\sqrt{3.20^2 + 8.28^2} = 8.88$

$\cos\theta = \frac{8.28}{8.88} = 0.933$

$\tan\theta = \frac{3.20}{8.28} = 0.387$

Effective depth = 441 − 10 = 431 cm, $d\cos\theta = 431 \times 0.933 = 402$ cm

Steel area reqd. for moment = $\frac{M}{f_s j d\cos\theta} = \frac{326000 \times 100}{1920 \times \frac{7}{8} \times 402} = 48.25$ cm²

Use 12 − 22φ bars = 45.60 cm² on rear side.

Steel ratio $p = \frac{45.60}{250 \times 431} = 0.000423$, $t/d = \frac{90}{431} = 0.209$

$p_1 = p\cos\theta = 0.000395$

neutral axis in the flange.

$k = \sqrt{2p_1 n + (p_1 n)^2} - p_1 n = \sqrt{2 \times 0.000395 \times 15 + (0.000395 \times 15)^2} - 0.000395 \times 15$

$= 0.103$

$j = 1 - \frac{k}{3} = 1 - \frac{0.103}{3} = 0.966$

$f_s = \frac{M}{A_s j d\cos\theta} = \frac{326000 \times 100}{45.6 \times 0.966 \times 402} = 1840$ kg/cm² < 1920

This stress will much be reduced by direct compression.

$f_c = \frac{f_s k}{n(1-k)} = \frac{1840 \times 0.103}{15(1-0.103)} = 14.1$

Direct comp. = $\frac{81200}{47100} = \frac{1.7}{15.8}$ kg/cm²

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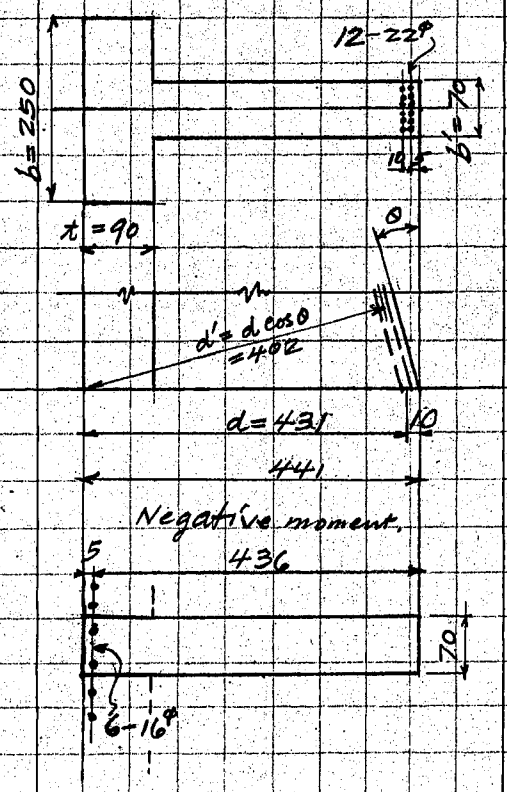
CALCULATIONS FOR

Akagawa Bashi for Saitama-Ken.

positive moment



Unit shear $= \frac{V_1}{b'jd}$ where $V_1 = V - \frac{M}{d}\tan\theta$

$= 107600 - \frac{32600000}{431} \times 0.387$

$= \frac{78300}{70 \times 0.966 \times 431}$ $= 78300$ kg.

$= 2.69$ kg/cm² b' = width of stem = 70 cm

Unit bond $= \frac{78300}{12 \times 6.91 \times 0.966 \times 431} = 2.27$ kg/cm²

For negative moment

Steel area required $= \frac{81200 \times 100}{1920 \times \frac{7}{8} \times 436} = 11.1$ cm² on front side

use 6-16φ bars $= 12.6$ cm²

$p = \frac{12.6}{70 \times 436} = 0.00041$, $p_1 = \frac{p}{\cos^2\theta} = \frac{0.00041}{0.933^2} = 0.00047$

$k = \sqrt{(p_1 n)^2 + 2p_1 n} - p_1 n = \sqrt{(0.00047 \times 15)^2 + 2 \times 0.00047 \times 15} - 0.00047 \times 15$

$= 0.112$

$j = 1 - \frac{k}{3} = 1 - \frac{0.112}{3} = 0.963$

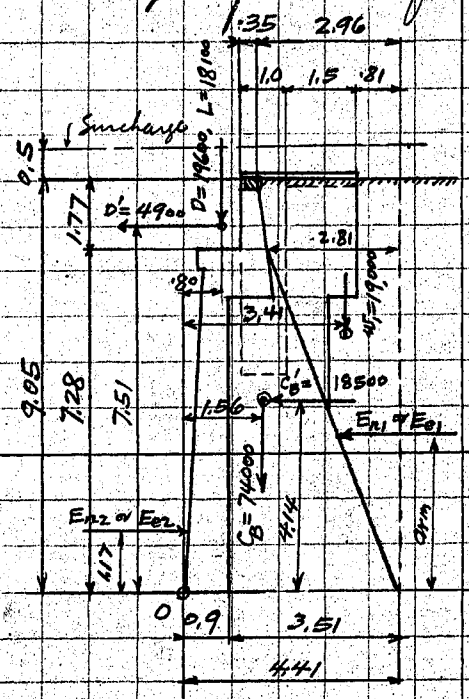
$f_s = \frac{81200 \times 100}{12.6 \times 0.963 \times 436} = 1535$ kg/cm² < 1920

$f_c' = \frac{1535 \times 0.112}{1.5(1-0.112)} = 12.90$

Direct comp. $= \frac{81200}{47100} = \frac{1.73}{14.63}$ kg/cm² < 40.5 × 1.6 = 64.8

Shear for this case = −13,700 kg, very small.

Design of counterfort wall C8 at both ends. width = 1.80 meters



Weight and center of gravity of counterfort wall C8.

Moment about point 0.		Volume			Weight	arm	Vert. m.	arm	hor. m.
End pedestal	0.80×0.80×1.20 =	0.77	@	2200 =	1,700	9.75	16,560	1.60	2,720
granite	0.30×0.30×1.50 =	0.135	@	2600 =	350	8.90	3110	1.35	470
parapet wall	0.43×1.47×1.80 =	1.14	@	2400 =	2,740	7.97	21820	1.31	3590
wing wall	0.30×1.90×2.87 =	1.64	@	" =	3,930	7.80	30650	2.65	10,420
"	0.30×0.51×2.61 =	0.40	@	" =	960	5.41	5190	2.66	2,560
" coping	0.30×0.10×1.50 =	0.05	@	" =	120	9.00	1080	2.85	340
top strut	1.25×1.00×1.80 =	2.25	@	" =	5400	6.78	36600	1.00	5400
" coping (front)	0.10×0.30×1.80 =	0.05	@	" =	120	7.13	860	0.35	40
" (end)	0.10×0.30×0.80 =	0.02	@	" =	50	7.13	360	0.70	40
Curtain wall	0.72×6.28×1.80 =	8.14	@	" =	19,540	2.86	55850	0.53	10350
Counterfort	0.90×2.30×6.28 =	13.00	@	" =	31,200	2.58	80,500	2.15	67050
projection	1.00×0.70×4.50 =	3.15	@	" =	7,560	6.90	52150	1.60	12100
Misc. conc. say		0.14	@	" =	330	4.14	1370	1.56	520
volume of granite		0.135 m³		C_B = 74,000 kg		4.14 m	306,100	1.56 m	115600
" " concrete		30.75 m³		Seismic force	C_B' = 74000×0.25 =		18,500 kg		

Weight and center of gravity of earth on the counterfort wall.

2.89×1.77×0.90 =	4.61	@	1600 =	7370	8.17	60,200	2.97	21,900
1.41×7.28×0.90 =	9.24	@	" =	14,800	4.85	71800	3.47	51300
less 0.30×1.90×2.77 =	−1.58	@	" =	−2530	7.75	−19600	2.65	−6700
" 0.30×0.51×2.61 =	−0.40	@	" =	−640	5.41	−3460	2.66	+1700
				wt = 19,000 kg	5.74 m	108940	3.41 m	64800

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken

Earthpressure at normal state on rear side.

Earth pressure E_{n1} 2,679 × 9.05 × 1.80 = 43,600 × 3.17 = 138,200

1/3 × 1600 × 0.50 = 267

1/3 × 1600 × 4.40 = 2347

2614 ÷ 2 = 1307

1307 × 0.70 × 4.40 = 4000 × 6.26 = 25000

E_{n1} = 47,600 kg 3.43 m 163,200

Earthpressure at normal state on front side.

E_{n2} = 1/6 × 1600 × 3.50² × 1.80 = 5,900 kg arm 1.17 m

Seismic earthpressure on rear side

0.559 × 1600 × 9.05² ÷ 2 × 1.8 = 66000 × 3.02 = 199,400

0.559 × 1600 × 4.40² ÷ 2 × 0.7 = 6,100 × 6.12 = 37,300

E_{e1} = 72,100 kg 3.28 m 236,700

Do. on front side.

E_{e2} = 0.559 × 1600 × 3.50² ÷ 2 × 1.8 = 9900 arm 1.17 m

Superimposed loads on abutment. (for one bearing)

Dead load D = 19,600 kg

Seismic force D' = 4,900 "

Live load L = 18,100 "

L + D = 37,700 "

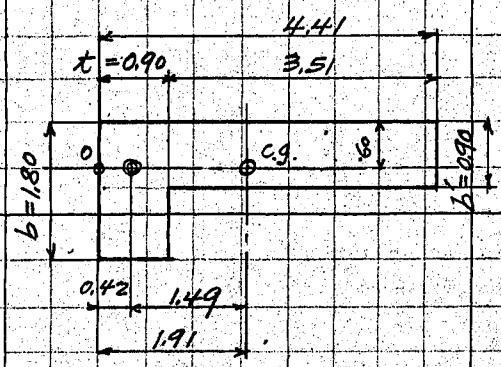
Stresses in counterfort wall. C_B at normal state.

Center of gravity of bottom section.

1.80 × 0.90 = 1.62 × 0.45 = 0.729

3.51 × 0.90 = 3.16 × 2.66 = 8.400

4.78 m² 1.91 m 9.129



Taking moment about point O.

Load marks	Hor. forces	Vert. forces	Lev. arms	Moment about point O.
D+L		37,700	× −0.80 =	− 30,200
C_B		74,000	× −1.56 =	− 115,600
W_1		19,000	× −3.41 =	− 64,800
E_{n1}	47,600		× 3.43 =	163,200
E_{n2}	− 5,900		× 1.17 =	− 6,900
	41,700 kg	130,700 kg	− 0.42 m	− 54,300

Eccentricity = 1.91 − 0.42 = 1.49 m

moment at bottom section = 130,700 × 1.49 = 194,700 kgm

Shear " " " = 41,700 kg.

Stresses during earthquake.

Seismic forces toward river side.

Taking moment about toe O.

D		19,600	× −0.80 =	− 15,700
D'	4,900		× 7.51 =	36,800
C_B		74,000	× −1.56 =	− 115,600
C_B'	18,500		× 4.14 =	76,600
W_1		19,000	× −3.41 =	− 64,800
E_{e1}	72,100		× 3.28 =	236,700
	95,500 kg	112,600 kg	1.367 m	154,000

Eccentricity e = 1.91 + 1.367 = 3.277 m

moment at bottom section = 112,600 × 3.277 = 369,000 kgm

Shear " " " = 95,500 kg.

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CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Seismic forces toward shore side.

Taking moment about toe O.

Load marks	Hor. forces	Vert. forces		Lev. arms		moment abt. O.
D		19,600	×	− 0.80	=	− 15,700
D′	− 4,900		×	7.51	=	− 36,800
C_B		74,000	×	− 1.56	=	− 115,600
C_B'	− 18,500		×	4.14	=	− 76,600
wt.		19,000	×	− 3.41	=	− 64,800
Eez	− 9,900		×	1.17	=	− 11,600
	−33,300 kg	112,600 kg		− 2.85 m		− 321,100

Eccentricity $e = -2.85 + 1.91 = -0.94$ m

moment at bottom section $= -112,600 \times 0.94 = -105,900$ kgm

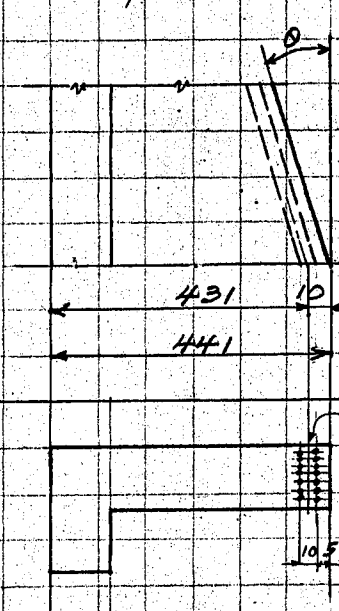
Shear " " " $= -33,300$ kg

Governing stresses :—

		Corresponding vertical load.
positive moment	+ 369,000 kgm during earthquake	112,600 kg
negative "	− 105,900 " " "	112,600 "
maximum shear	+ 95,500 kg " "	

Reinforcements in counterfort wall.

positive moment.



$\cos\theta = 0.933$, $\tan\theta = 0.387$

Effective depth $= 441 - 10 = 431$ cm, $d\cos\theta = 431 \times 0.933 = 402$ cm

Steel area req'd. for moment $= \dfrac{369,000 \times 100}{1920 \times \frac{7}{8} \times 402} = 54.6$ cm²

Use 14-22φ bars $= 53.2$ cm²

Steel ratio $p = \dfrac{53.2}{180 \times 431} = 0.000686$, $p_1 = p\cos\theta = 0.000686 \times 0.933 = 0.00064$

$t/d = 90/431 = 0.209$

Neutral axis in the flange.

$k = \sqrt{2 \times 0.00064 \times 15 + (0.00064 \times 15)^2} - 0.00064 \times 15 = 0.129$

$j = 1 - \dfrac{0.129}{3} = 0.957$

$f_s = \dfrac{369,000 \times 100}{53.2 \times 0.957 \times 402} = 1802$ kg/cm²

$f_c = \dfrac{1802 \times 0.129}{15(1-0.129)} = 17.8$

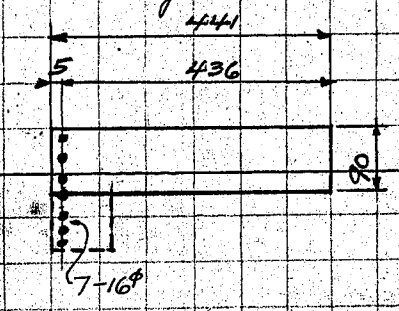
Direct comp. $= \dfrac{112600}{47800} = \dfrac{2.4}{20.2}$ kg/cm²

$V_1 = 95500 - \dfrac{369000 \times 100}{431} \times 0.387 = 62400$ kg

unit shear $= \dfrac{62400}{90 \times 0.957 \times 431} = 1.68$ kg/cm²

unit bond $= \dfrac{62400}{14 \times 6.91 \times 0.957 \times 431} = 1.56$

Negative moment



Steel area req'd. for moment $= \dfrac{105,900 \times 100}{1920 \times \frac{7}{8} \times 436} = 14.45$ cm²

Use 7-16φ bars $= 14.08$ cm²

$p = \dfrac{14.08}{90 \times 436} = 0.00036$ $\quad p_1 = \dfrac{p}{\cos^2\theta} = \dfrac{0.00036}{0.933^2} = 0.00041$

$k = \sqrt{2 \times 0.00041 \times 15 + (0.00041 \times 15)^2} - 0.00041 \times 15 = 0.105$

$j = 1 - \dfrac{0.105}{3} = 0.965$

$f_s = \dfrac{105900 \times 100}{14.08 \times 0.965 \times 436} = 1790$ kg/cm²

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CALCULATIONS FOR

Akigase-Bashi for Saitama-ken

$$f_c = \frac{1790 \times 0.105}{15(1-0.105)} = 14.0$$

$$\text{Direct comp.} = \frac{112600}{47800} = \frac{2.4}{16.4}\ kg/cm^2$$

Stability of Abutment.

Superimposed loads on abutment: seismic force.

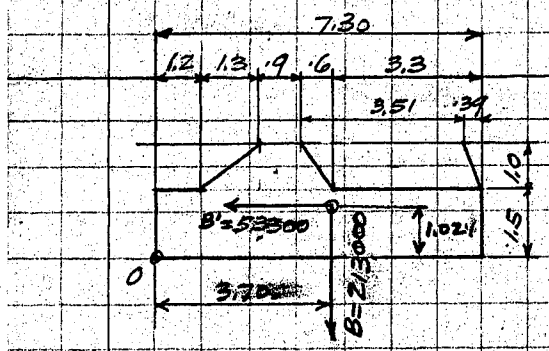
Dead load $D = 39100$, $D' = 9800$ kg

Live load $L = 36200$

$D+L = 75300$ kg

Weight and center of gravity of shaft.

Counterfort	weight	arm	vert m.	arm	hor. m.
C_A	64,000	3.60	230600	1.26	80640
$2\,C_B$	148000	4.14	612,200	1.56	234,200
$C =$	212,000 kg	3.97m	842,800	1.47m	311,840
$(C' =$	53,000)	2.50		2.50	
arms from toe	---	6.47m	---	---	3.97m



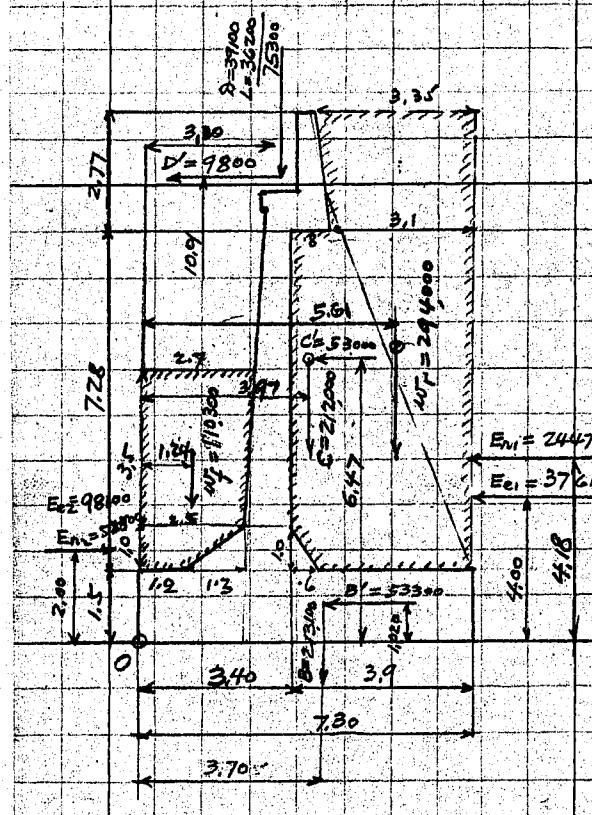
Weight and center of gravity of base.

Taking moment about toe 0.

					arm	vert m.	arm	hor. m.
Base	$1.50 \times 7.30 \times 6.30 =$	69.00	@2,400 =	165,500	0.75	124,000	3.65	604,000
wall base	$1.85 \times 1.00 \times 6.10 =$	11.28	@ " =	27,100	1.915	51,900	2.745	74,400
Counterfort	$3.41 \times 1.00 \times 2.50 =$	8.52	@ " =	20,500	2.00	41000	5.40	110700
		88.80	$B =$	213,100 kg	1.02	216900	3.70	789100
			$B' =$	53300 kg				

Weight and center of gravity of earth on rear footing.

	$3.23 \times 2.77 \times 6.30 =$	56.40	@1,600 =	90,200	10.18	919,000	5.68	512,000
	$3.90 \times 7.28 \times 6.30 =$	179.00	@ " =	286400	5.14	1471,000	5.85	1533,000
fillet less	$0.30 \times 1.00 \times 3.60 =$	- 1.08	@ " =	- 1,700	1.83	- 3,100	3.60	- 6,100
wing "	$2 \times 2.03 \times 0.30 \times 2.77 =$	- 3.37	@ " =	- 5400	10.19	- 55000	5.08	- 27400
" "	$2 \times 0.51 \times 0.30 \times 2.61 =$	- 0.80	@ " =	- 1,300	7.91	- 10300	5.17	- 6,700
" "	$2 \times 0.10 \times 1.00 \times 4.40 =$	- 0.88	@ " =	- 1,400	9.35	- 13,100	4.10	- 5700
Counterfort	$2.50 \times 2.50 \times 7.28 =$	- 45.50	@ " =	- 72,800	4.46	- 324,500	4.80	- 349,500
		183.77	$W_f =$	294,000 kg	6.75m	1984,000	5.61m	1649,600



Weight and center of gravity of earth on front footing.

$2.60 \times 3.50 \times 6.30 =$	57.30	@1,600 =	91,700	1.30	119,200
$1.85 \times 1.00 \times 6.30 =$	11.65	@ " =	18,600	0.96	17900
	68.95	$W_f =$	110300 kg	1.24	137,100

Earth pressure at normal state.

On rear side. total depth of earth = 11.55m with a surcharge of 0.5m for L.L.

$1/3 \times 1600 \times 0.50 =$ 267

$1/3 \times 1600 \times 12.05 =$ 6430

$6,697 \div 2 = 3349$ kg/m²

267

$1/3 \times 1600 \times 4.90^2 =$ 2615

$2882 \div 2 = 1441$ kg/m² ✓

Total pressure = $3349 \times 11.55 \times 6.10 =$ 235,800 × 4.01 = 946,000

$1441 \times 4.40 \times 1.40 =$ 8900 × 8.75 = 78,000

$E_{H1} =$ 244,700 kg, 4.18m, 1024,000

On front side. total depth of earth = 6.0m

Total pressure = $\frac{1}{6} \times 1600 \times 6.0^2 \times 6.1 =$ 58500 kg = E_{H2}, arm = 2.00m

増田淳氏関係資料
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CALCULATIONS FOR

Akigase Bashi for Saitama-ken.

Earth pressure during earthquake.

On rear side E_{e1} -

$\frac{1}{2} \times .559 \times 1600 \times 11.55^2 \times 6.10 = 364000 \times 3.85 = 1402000$

$\frac{1}{2} \times .559 \times 1600 \times 4.40^2 \times 1.40 = 12100 \times 8.62 = 104300$

$E_{e1} = 376100^{kg}$ 4.00^{m} 1506300

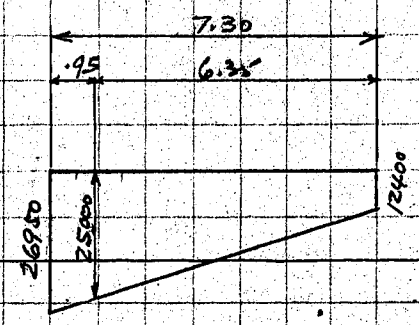
On front side E_{e2}

$\frac{1}{2} \times 0.559 \times 1600 \times 6.00^2 \times 6.10 = 98100^{kg}$ arm $= 2.00^{m}$

Stability at normal state.

Taking moment about toe 0

Load marks	Hor. forces	Vert. forces	Lev. arms	Moments abt. 0
D+L		75300 ×	- 3.30	= - 248500
C		212000 ×	- 3.97	= - 841000
W_F		294000 ×	- 5.61	= - 1650000
W_f		110300 ×	- 1.24	= - 136800
E_{e1}	244700	×	4.18	= 1023000
E_{e2}	- 58500	×	2.00	= - 117000
B		213100 ×	- 3.70	= - 788000
	186200 kg	904700 kg	- 3.05 m	2758300



Eccentricity $e = 3.50 - 3.05 = 0.45^{m}$

Resultant force within middle third

max. Bearing pressures $= \frac{904700}{6.3 \times 7.3}\left(1 \pm \frac{6 \times 0.45}{7.3}\right)$ 370

$= 26950^{kg/m^2}$ (Correspond to 2.46 tons/a.)

or 12400 "

max. load on one pile

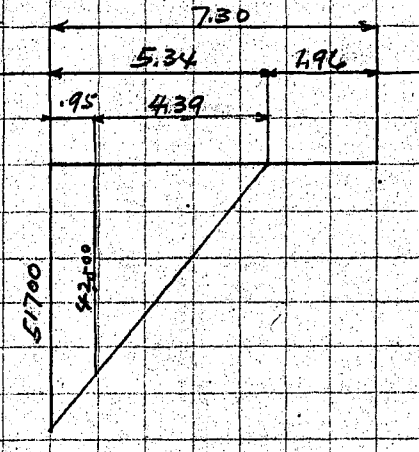
$(26950 + 25000) \div 2 = 25980 \times 0.9 \times 0.95 = 22200^{kg}$

Stability during earthquake

Seismic force toward river side.

Taking moments about toe 0.

Load marks	Hor. forces	Vert. forces	Lev. arms	moment abt. 0.
D		39100 ×	- 3.30	= - 129000
D'	9800	×	10.01	= 98100
C		212000 ×	- 3.97	= - 841000
C'	53000	×	6.47	= 343000
W_F		294000 ×	- 5.61	= - 1650000
W_f		110300 ×	- 1.24	= - 136800
B		213100 ×	- 3.70	= - 788000
B'	53300	×	1.02	= 54300
E_{e1}	376100	×	4.00	= 1505000
	492200 kg	868500 kg	- 1.78 m	- 1544400



Eccentricity $e = 3.65 - 1.78 = 1.87$ m

Resultant force outside of middle third.

Neglecting tension, pressure area $= 1.78 \times 3 \times 6.3 = 33.62\ m^2$

max. toe pressure $= \frac{868500 \times 2}{33.62} = 51700^{kg/m^2}$ (6.72 tons/a.)

max. load on one pile =

$(51700 + 42500) \div 2 = 47100 \times 0.9 \times 0.95 = 40200^{kg}$

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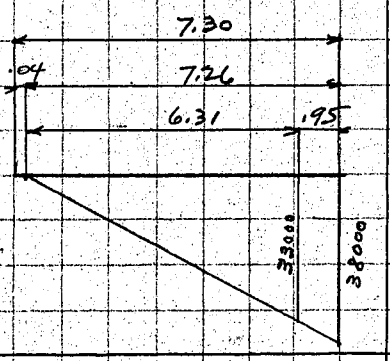
CALCULATIONS FOR

Akigase-Bashi for Saitama-ken.

Seismic force toward shore side.

Taking moment about toe O.

Load marks	Hor. forces	Vert. forces	Lev. arms		moment about O.
D		39,100	× − 3.30	= −	129,000
D′	− 9,800		× 10.01	= −	98,100
C		212,000	× − 3.97	= −	841,000
C′	− 53,000		× 6.47	= −	343,000
W_F		294,000	× − 5.61	= −	1650,000
W_P		110,300	× − 1.24	= −	136,800
B		213,100	× − 3.70	= −	788,000
B′	− 53,300		× 1.02	= −	54,300
E_{e2}	− 98,100		× 2.00	= −	196,200
	− 214,200 kg	868,500 kg	− 4.88 m	−	4236,400



Eccentricity $e = -4.88 + 3.65 = -1.23$ m

Resultant force outside of middle third.

Neglecting tension, pressure area

$3.65 - 1.23 = 2.42 \times 3 = 7.26$ m²

$7.26 \times 6.3 = 45.70$ m²

max. toe pressure $= \frac{868500 \times 2}{45.7} = 38000$ kg/m²

max. load on one pile

$= (38000 + 33000) \div 2 = 35500 \times 0.9 \times 0.95 = 30400$ kg

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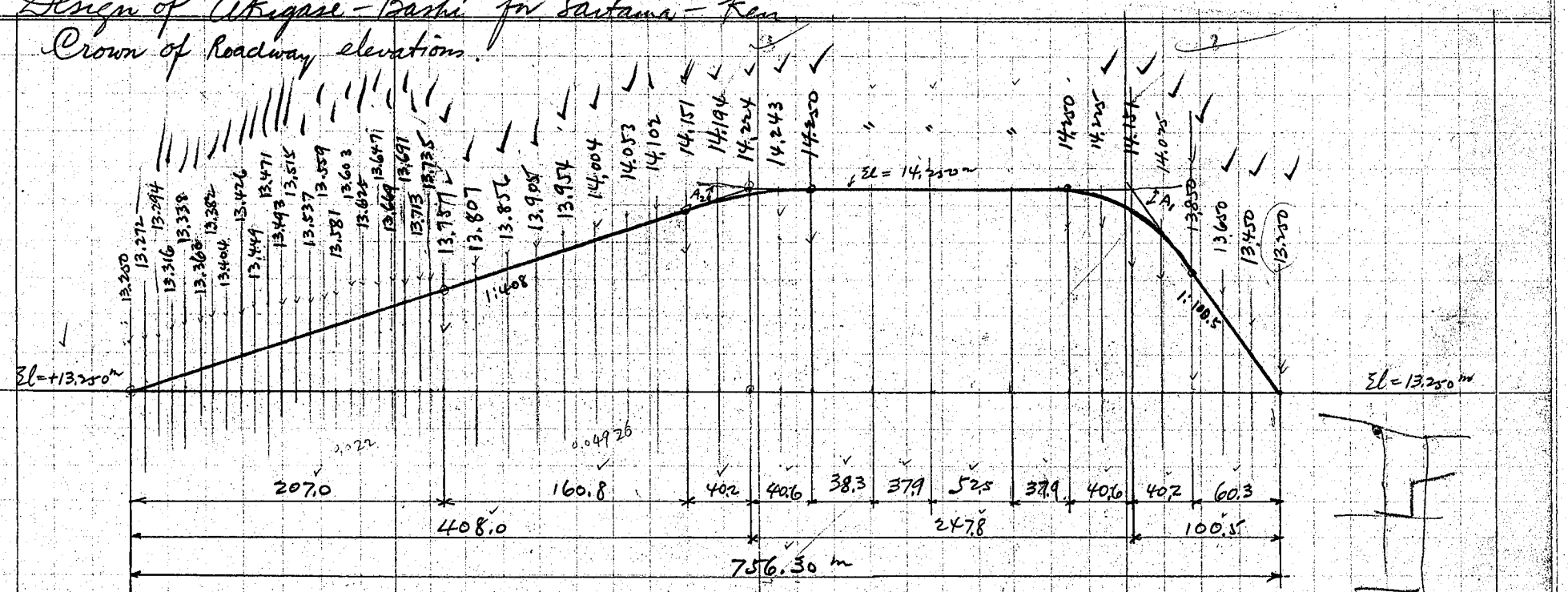
CALCULATIONS FOR

Design of Akigase-Bashi for Saitama-Ken

Crown of Roadway elevations.



Intersection angle A_1.

$A_1 = \tan^{-1} \frac{1}{100.5} = \tan^{-1} 0.00995$ ✓

$A_1 = 0° - 34'$

$R_1 = T_1 \cot \frac{A_1}{2} = 40.2 \times \cot 0° - 17'$

$= 40.2 \times 202.22$

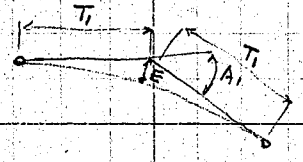
$= 8130$ m

$E_1 = R\left(\sec \frac{A}{2} - 1\right) = 8130 \times 0.000.0122$

$= 0.099$ m or 99 mm ✓

$\frac{1}{4} \times 99 = 25$ mm ✓

0°-34'-12.4
17'-6.2
(402)(198)
= 7960
(7960)(0.0123)
= 0.098
98 mm ✓
25 ✓



Int. L A_2

$\tan^{-1} \frac{1}{408} = \tan^{-1} 0.002451$ ✓

$A_2 = 0° - 8' - 30''$

$R_2 = T_2 \cot \frac{A_2}{2} = 40.2 \cot 0° - 4' - 15''$

$= 40.2 \times 816 = 32800$ m

$E_2 = R_2\left(\sec \frac{A_2}{2} - 1\right) = 32800 \times 0.000.000 8$ ✓

$= 0.026$ m ✓

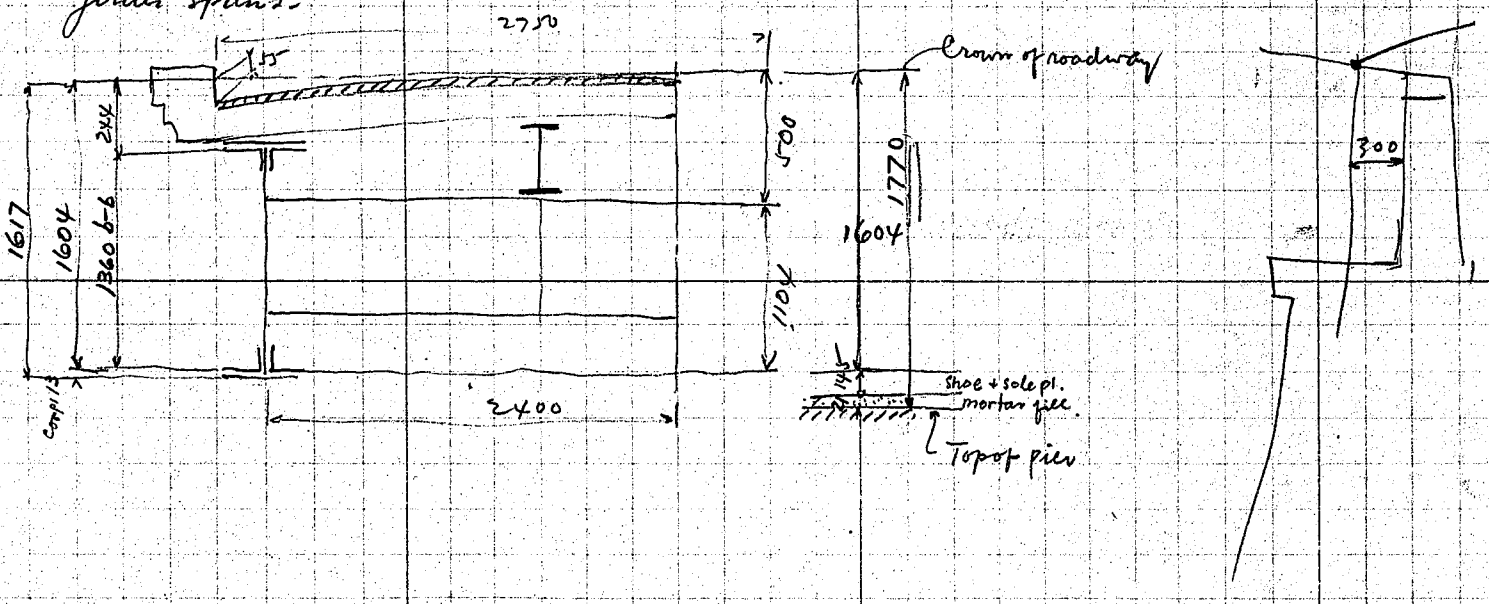
or 26 mm

$\frac{1}{4} \times 26 = 7$ mm ✓

0°-8'-25.6"
4'-12.8
(402)(823)
= 33.085
(33085)(0.08)
= 0.026

Height of Pier

Girder spans.



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CALCULATIONS FOR

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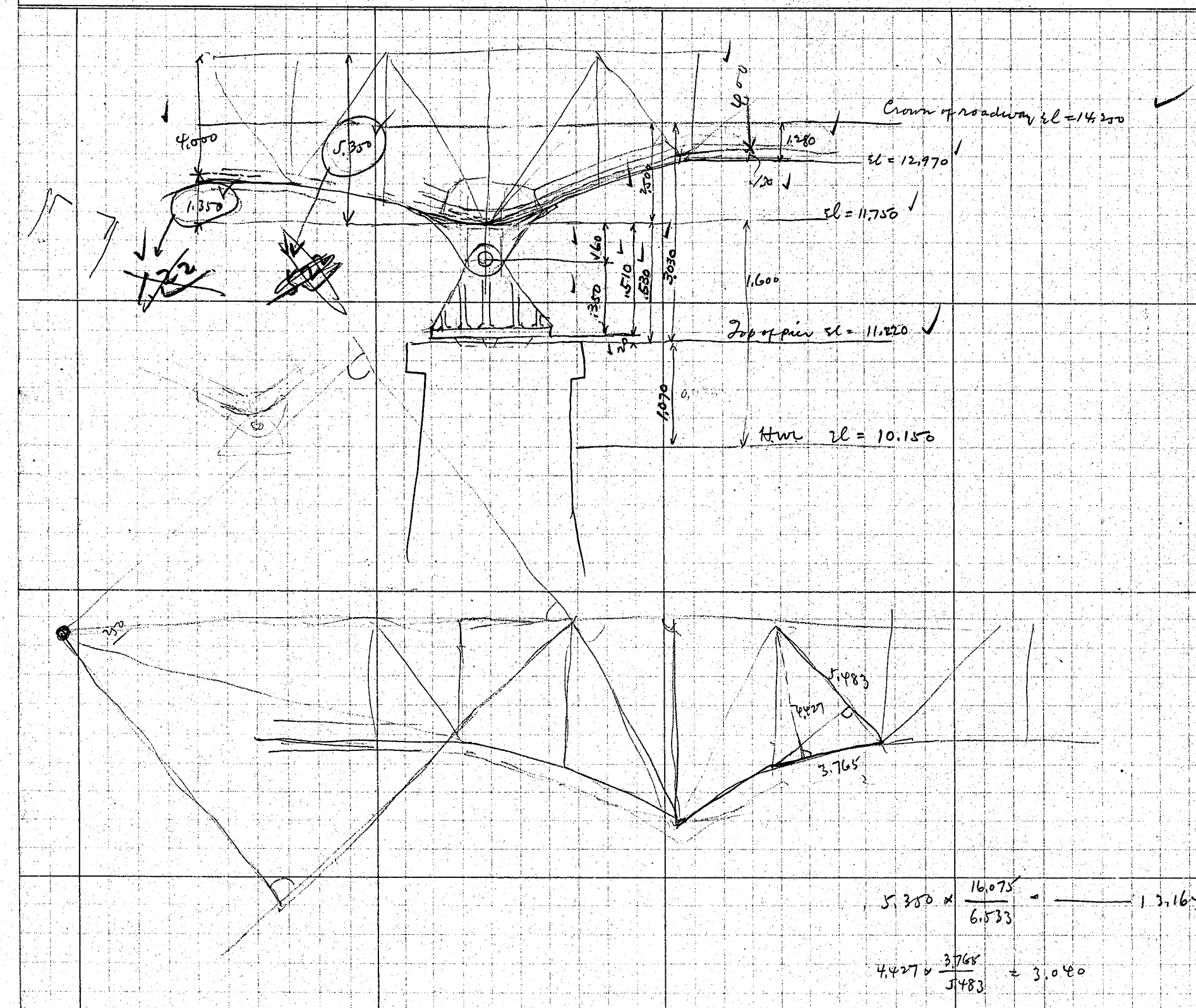
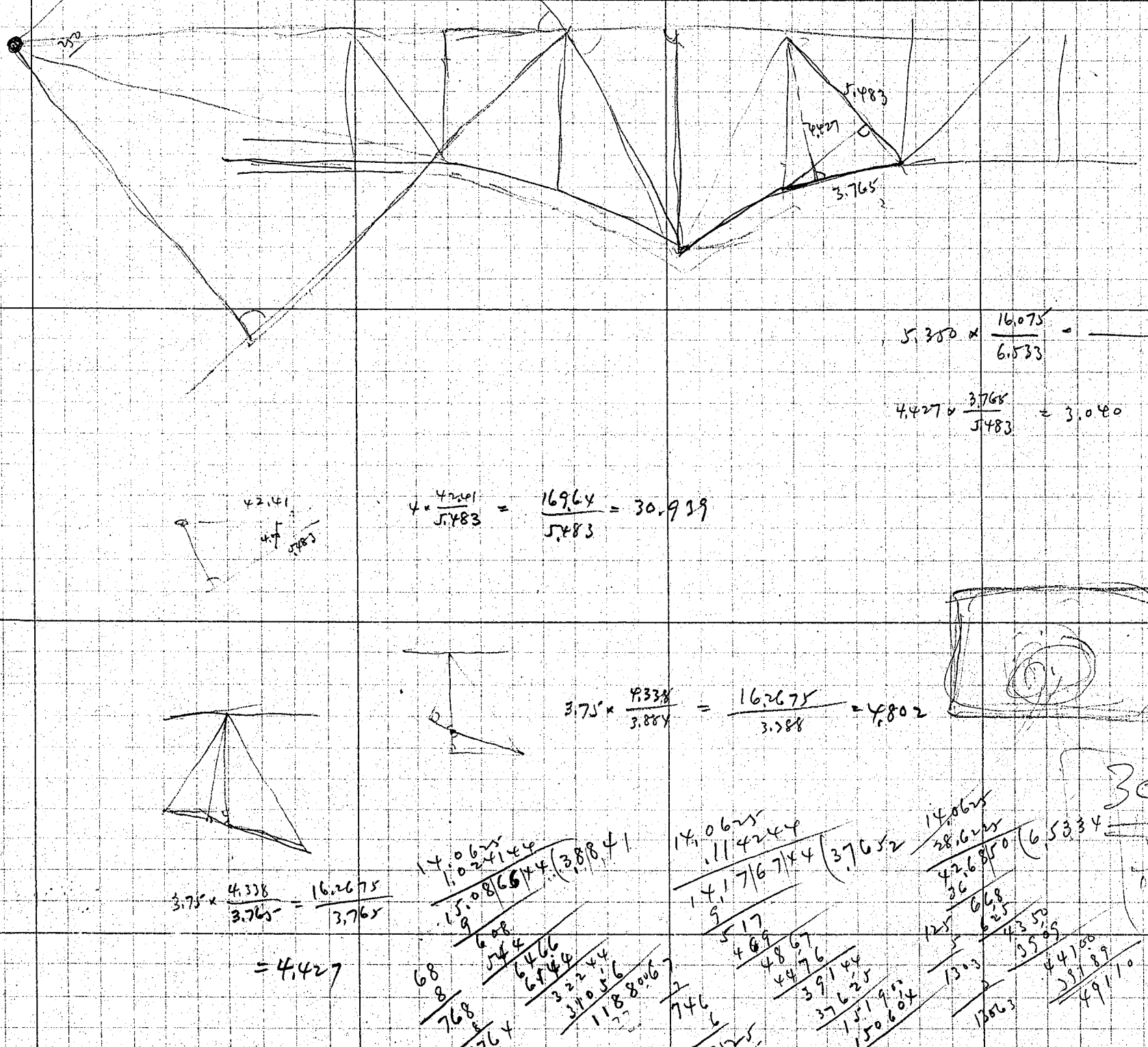


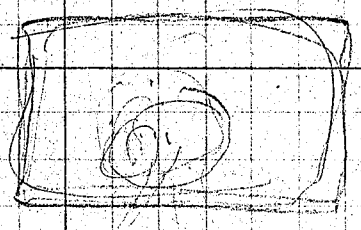


$5.350 \times \frac{16.075}{6.533} = 13.164$

$4.427 \times \frac{3.768}{5.483} = 3.040$

$4 \times \frac{42.41}{5.483} = \frac{169.64}{5.483} = 30.939$

$3.75 \times \frac{4.338}{3.884} = \frac{16.2675}{3.388} = 4.802$



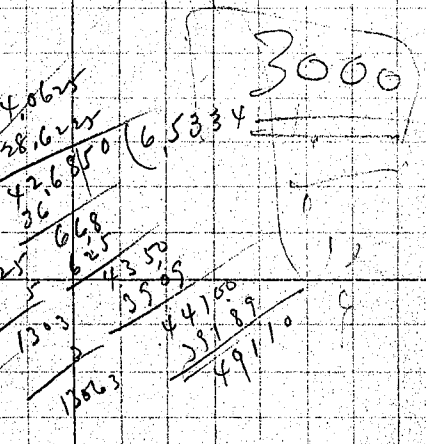
$3.75 \times \frac{4.338}{3.765} = \frac{16.2675}{3.765}$

$= 4.427$

14.0625 / 1.02441 44 (3.8841 / 15.08664 / 9 / 608 / 5444 / 6466 / 6144 / 32244 / 3405.6 / 11880.67 / 7 / 746 / 7532 / 75302

14.0625 / 1.1424 44 (3.765 2 / 14.17676 44 / 9 / 517 / 4669 / 4867 / 4476 / 39144 / 3762.25 / 151905 / 150604

14.0625 / 38.6225 (6.5334 / 42.685 50 / 36 / 668 / 625 / 4350 / 3505 / 41100 / 39189 / 1911.0 / 1303 / 12063



104

増田淳氏関係資料
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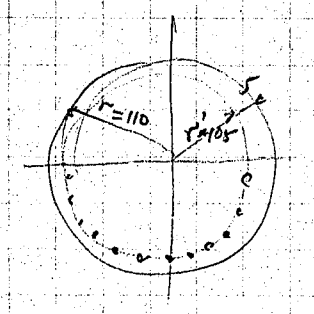
CALCULATIONS FOR



$A_s = 28 \text{ @ } 3.801 = 106.5\ cm^2$ ✓ (28 − 22φ)

$p = \frac{106.5}{110^2\pi} = 0.0028$

$N = 96400\ kg$

$r = 110$, $r' = 105$ $\quad r'/r = 105/110 = 0.955$ $\quad z/r = 159/110 = 1.445$

$k = 0.61$ $\quad c = 0.255$

$f_c = \frac{N}{r^2 c} = \frac{96400}{110^2 \times 0.255} = 31.1\ kg/cm^2$ ✓ $< 40\sigma$

$f_s = m f_c \frac{1 + \frac{r'}{r} - k}{k} = 15 \times 31.1 \times \frac{1 + 0.955 - 0.61}{0.61} = 1028\ kg/cm^2$ ✓

(1.345, r)

$A_s = 24 \text{ @ } 3.801 = 91.2\ cm^2$ ✓ (24 − 22φ)

$p = \frac{91.2}{110^2\pi} = 0.0024$

$N = 96,400$

$k = 0.57$ $\quad c = 0.22$

$f_c = \frac{96400}{110^2 \times 0.22} = 36.2\ kg/cm^2$ ✓ $< 40\sigma \times 1.6 = 64.8$

$f_s = 15 \times 36.2 \times \frac{1 + 0.955 - 0.57}{0.57} = 1320\ kg/cm^2$ ✓ $< 1200 \times 1.6 = 1920$

(1.385)

5m from top

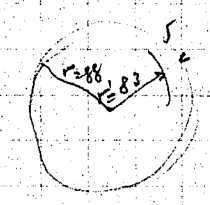
$A_s = 12 \text{ @ } 3.801 = 45.6\ cm^2$ ✓

$p = \frac{45.6}{88^2\pi} = 0.0019$

m	v
9800 = 2700	3915
1.35 × 700 = 900	2990
3600	6900

$z/r = 52/88 = 0.591$

$z_{cc} = 0.52$



$r'/r = 83/88 = 0.943$, (1.35)

$k = 1.35$ $\quad c = 0.92$

$f_c = \frac{690}{88^2 \times 0.92} = 9.7$

$f_s = 15 \times 9.7 \times \frac{1 + 0.943 - 1.35}{1.35} = 80.6$ (.693)

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CALCULATIONS FOR

10017

Pier shaft of P12
Bottom section

Steel ratio, $A_s = 32 @ 22\phi = 121.7\ cm^2$

$$p = \frac{121.7}{149^2\pi} = 0.001745$$

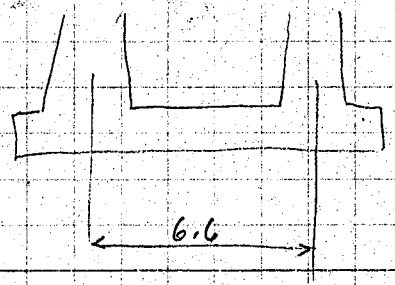
$r = 149$, $r' = 144$; $r'/r = 144/149 = 0.966$ $\quad t/r = 178/149 = 1.195$

$k = 0.58$ $\quad e = 0.35$

$N = 216,700$

$$f_c = \frac{216700}{149^2 \times 0.25} = 39.0$$

$$f_s = 15 \times 39.0 \times \frac{1 + \overset{1.386}{0.966} - 0.58}{0.58} = 1398$$



Moment on base

$$\frac{43600 \times 6.6^2}{10} = 190,000\ kgm$$

$$\text{Steel reqd} = \frac{190,000 \times 100}{1200 \times \frac{7}{8} \times 215} = 84.1$$

• Simple beam.

$$\frac{43600 \times 4.0^2}{8} = 87200$$

$$A_s = \frac{87200 \times 100}{1200 \times \frac{7}{8} \times 215} = 39.6$$

for f_s height of 265

$$A_s = 39.6 \times \frac{215}{265} = 31.3 \qquad 11-19\phi$$
$$\text{or } 9-22\phi$$

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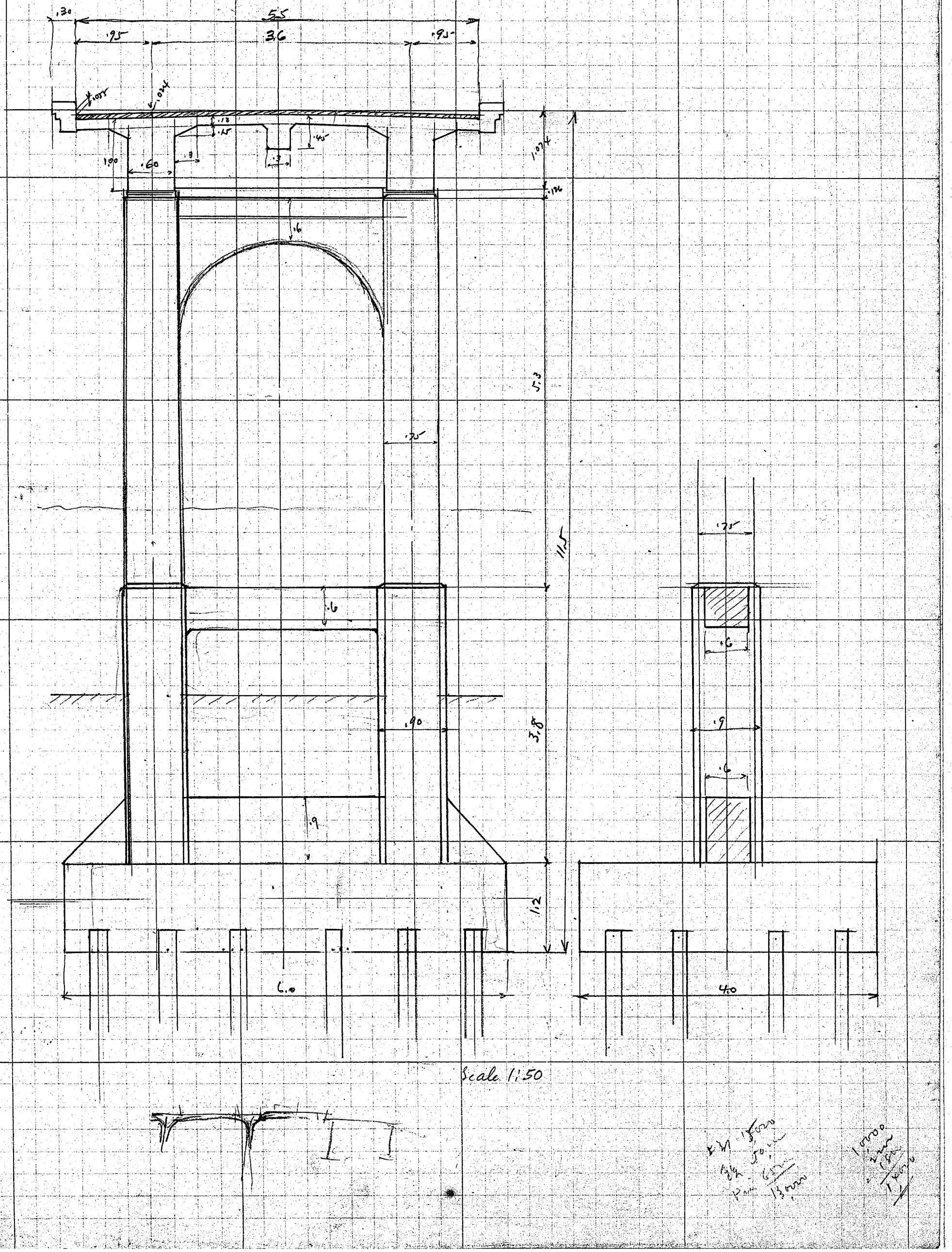
CALCULATIONS FOR

Akigase Bashi for Saitama-Ken.

Approach Concrete girder 5 spans. Span length 12.00 meter c to c Bearings. Ba. 3o S.S. m 78



Scale 1:50

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CALCULATIONS FOR

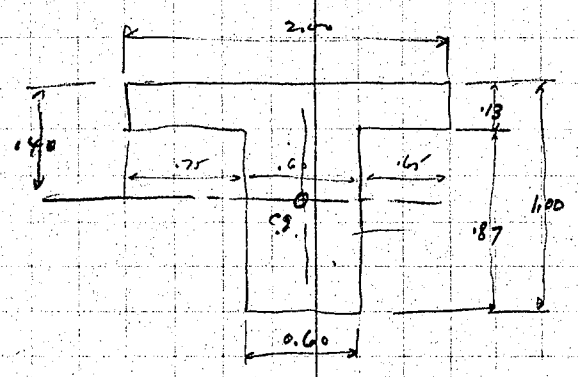
Akigase Bashi Saitama Ken.

Design of main girder

Dead Load

5cm granolithic pavement	@ 22 kg.	=	110
13 cm Reinf. conc. slab	@ 24	=	312
			8
		430 × 5.5 =	2365
@ spring	.25 × .35 @ 24 cm	= 210 × 2 =	420
fillets	.30 × .15 × 2 cm/m	=	216
			6
web of beam	.16 × 1.87 cm/m	= 1252.5 × 2 =	2505
+ 7 stringers	.13 × .32 cm/m	=	230
Cross beam	.30 × .85 × 3.0 cm/m = 1836 ÷ 3.0 m	=	612
			11
			4000
Handrails	1 @ 80	=	160
			6525
			75
			6600 ÷ 2 = 3300 kg/m for one beam

Moment of inertia of beam. I.



2.00 × .13 = .260	× .065	=	.017
.870 × .60 = .522	× .565	=	.295
.782	0.40		.312

.07 / 100

$\frac{2.00 \times .13^3}{12} + .260 \times .335^2 = 0.00074 + 0.0292 = 0.0296$

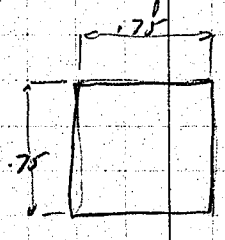
$\frac{.60 \times .87^3}{12} + .522 \times .165^2 = 0.0329 + 0.0142 = 0.0471$

$0.0767\ m^4$

Steel say 3% = 23

$I = 0.0790$

Moment of inertia of Column



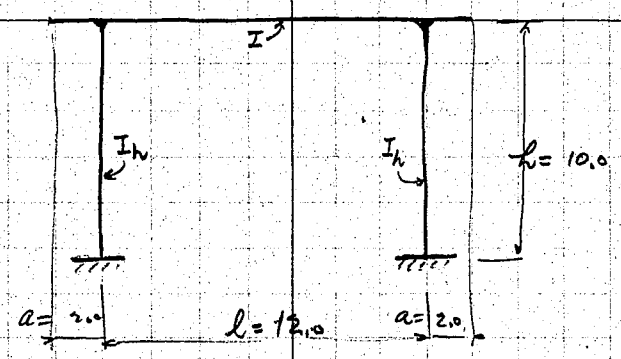
$\frac{.75 \times .70^3}{12} = 0.0264$

Steel say 3% = 8

$I_h = 0.0272\ m^4$

Constants req'd. for calculation.

$v = \frac{I.h}{I_h.l} = \frac{0.0790}{0.0272} \times \frac{10.0}{12.0} = 3.5$



$v+2 = 5.5$, $2+3v = 12.5$, $6v+1 = 22.0$, $6v-1 = 16.5$

$a^2 = 4.0$, $l^2 = 144.0$, $6a^2 = 24.0$, $3a^2 = 12.0$

増田淳氏関係資料
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CALCULATIONS FOR

Akigase Bashi for Sadamura Sen.

Design of Suspended span. Span length = 8.0 m

Dead load moment $= 1/8 \cdot 3300 \cdot 8.0^2 = 26400$ kgm.
" shear $= 1/2 \cdot 3300 \cdot 8.0 = 13200$ kg.

Live load moment $= 1/8 \cdot 1500 \cdot 8.0^2 = 12000$ kgm
" shear $= 1/2 \cdot 1500 \cdot 8.0 = 6000$ kg

1500 × 2.75 = 1375 kg/m (uniformly distributed load)
12 m

Summary	Moment	Shear
D.L.	26400	13200
L.L.	12000	6000
	38400 kgm	19200 kg

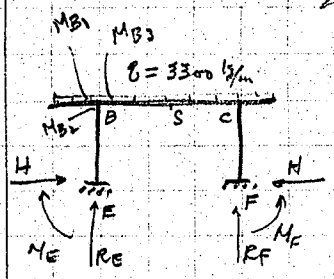
Steel req'd $= \dfrac{3840000}{1200 \times \frac{2}{8} \times \frac{90}{95}} = 40.6$ (+8.8) cm²

Use 12-22φ = 45.6 cm² (49.09)

Unit shear $= \dfrac{19200}{60 \times \frac{2}{8} \times 90} = 4.06$ kg/cm²

Design of Anchor Span.

Dead Load.



$R_E = R_F = 3300 \times 8.0 = +26400$ kg

$H = \dfrac{q(l^2 - 6a^2)}{4h(\nu+2)} = \dfrac{3300(144 - 24)}{40 \times 5.5} = +1800$ kg.

$M_E = M_F = \dfrac{q(l^2 - 6a^2)}{12(\nu+2)} = \dfrac{3300 \times 120}{12 \times 5.5} = +60000$ kgm

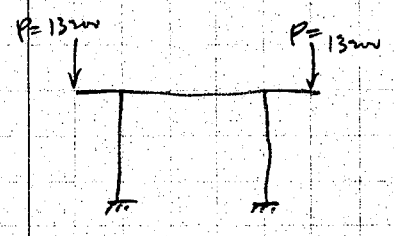
$M_{B2} = M_{C2} = \dfrac{q(6a^2 - l^2)}{6(\nu+2)} = \dfrac{3300 \times (-120)}{6 \times 5.5} = -120000$ kgm

$M_{B3} = M_{C3} = -\dfrac{3qa^2\nu + ql^2}{6(\nu+2)} = -\dfrac{3 \times 3300 \times 4 \times 3.5 + 3300 \times 144}{6 \times 5.5}$ (1385.00, ×7.5.500)

$= -\dfrac{614000}{33} = -18600$ kgm

$M_S = \dfrac{8l^2(2+3\nu) - 12qa^2\nu}{24(\nu+2)} = \dfrac{3300 \times 144 \times 12.5 - 12 \times 3300 \times 4 \times 3.5}{24 \times 5.5}$ (×94.000, ×554.000)

$= \dfrac{5286000}{132} = +40800$ kgm (132.)



$M_{B1} = \dfrac{3300 \times 2.0^2}{2} = -6600$ kgm

$R_E = R_F = +13200$ kg

$H = -\dfrac{Pa}{h} \cdot \dfrac{3}{\nu+2} = -\dfrac{1320 \times 2 \times 3}{10 \times 5.5} = -1440$ kg

$M_E = M_F = -Pa \cdot \dfrac{6\nu+1}{(6\nu+1)(\nu+2)}$ (= $-\frac{Pa}{\nu+2}$) $= -\dfrac{1320 \times 2 \times 22}{22 \times 5.5} = -4800$ kgm

$M_{B2} = +\dfrac{2Pa}{\nu+2} = +9600$ "

$M_{B3} = -Pa\dfrac{\nu}{\nu+2} = -\dfrac{1320 \times 2 \times 3.5}{5.5} = -16800$ "

$M_{B1} = -Pa = -1320 \times 2 = -2640$ "

$M_S = -Pa\dfrac{\nu}{\nu+2} = -16800$

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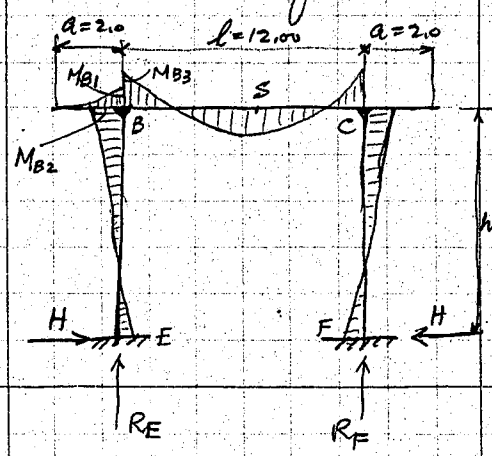
CALCULATIONS FOR

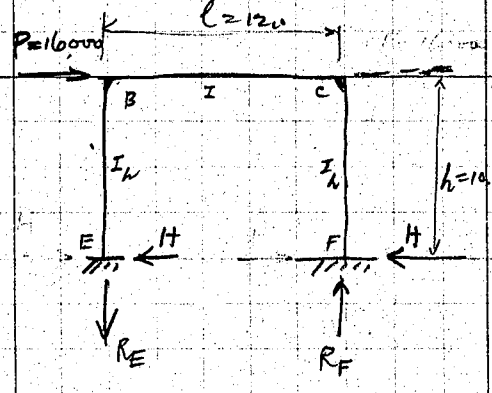
Akigase Bashi for Saitama Ken

Summary for moments.



	Reactions $R_E = R_F$	Thrust H	Moments $M_E=M_F$,	$M_{B2}=M_{C2}$,	$M_{B3}=M_{C3}$,	$M_{B1}=M_{C1}$.	M_S.
D.L. (g)	+ 26,400	+ 1800	+ 6000	-12,000	-18,600	-6,600	+40,800
(P)	+ 13,200	- 1440	- 4800	+ 9600	-16,800	-26,400	- 16,800
	+39,600 kg	+ 360 kg	+1,200 kgm	-2,400 kgm	-35,400 kgm	-33,000 kgm	+24,000 kgm
L.L. (= 52×.453)	+ 12,000	+ 820	+ 2,740	- 5460	- 8460	- 3,000	+18,550
	+ 6,000				- 7640	- 12,000	
	+18,000	+ 820	+ 2740	-5460	-16,100	-15,000	+18550
Summary.	+57,600 kg	+1,180 kg	+3,940 kgm	-7860	-51,500	-48,000	+42,550 kgm

Seismic stress. K assumed 0.200 Seismic load = $0.20 \times 3300 \times 24.25 = 16{,}000$ kg.



$$M_E = -\frac{P}{2}\cdot\frac{h(3v^2+7v+2)}{(v+2)(6v+1)} = -\frac{8000\times 10\times 63.25}{15\times 22.0} = \mp 41{,}800 \text{ kgm}$$

$$M_{B2} = M_{B3} = +\frac{Pv}{2}\cdot\frac{3h}{6v+1} = +\frac{8000\times 3.5\times 30}{121} = \mp 6950 \text{ kgm}$$

$$M_{C2} = M_{C3} = -\frac{Pv}{2}\cdot\frac{3h}{6v+1} = -\frac{8000\times 3.5\times 30}{121} = \mp 6950$$

$$M_F = +\frac{P}{2}\cdot\frac{h(3v^2+7v+2)}{(v+2)(6v+1)} = +\frac{8000\cdot 10\times 63.25}{121} = \pm 41800$$

$$R_E = -R_F = \frac{16000\times 10}{12} = \pm 13{,}340 \text{ kg} \quad 16700$$

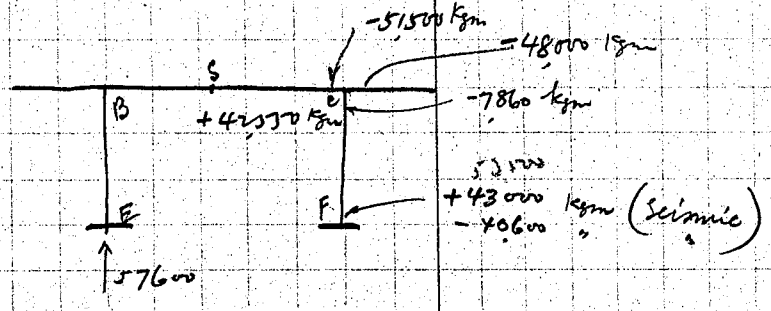
$3v^2 = 36.75$

$3v^2+7v+2 = 63.25$

Summary.

	$R_E=R_F$	Thrust	Moments $M_E=M_F$	$M_{B2}=M_{C2}$	$M_{B3}=M_{C3}$	$M_{B1}=M_{C1}$	M_S
D.L.	+ 39600	+360	+ 1,200	- 2,400	- 35,400	-33,000	+24,000
Earth quake	± 13340 (16700)	± 8000	± 41800 (42500)	± 6950	± 6950		
	+52940 kg (53700)	+8360	+ 43000 (53500)	+ 4550	- 42350		
	+26,260	- 7640	- 40,600	- 9350			

Design moment.



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CALCULATIONS FOR

Akigase Bashi for Saitama Ken.

Beam.

Point S. $M = 42550$ kgm

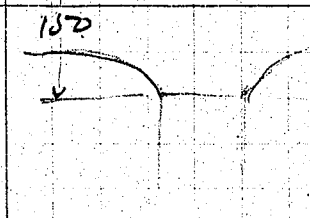
Steel required $= \frac{42550 \times 100}{1200 \times \frac{2}{8} \times 90} = 45.00$ cm² (54.1)

use $12 - 22\phi = 45.6$ cm²
$12 - 25\phi = 58.9$

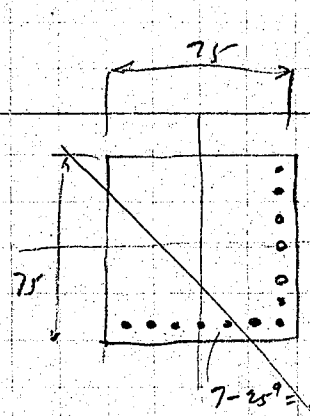
Point C. $M = -51{,}500$ kgm.

Steel req'd $= \frac{51500 \times 100}{1200 \times \frac{2}{8} \times 120} = 35.0$ (39.2) cm²

use $12 - 22\phi = 45.6$ cm² ✓
$10 - 25\phi = 49.09$



Column Point F. Seismic moment $= +43{,}300$ kgm (53,300) —— vert reaction $= +3940$ kg (56.3 m)
or -40600 " —— " $= +26{,}260$ "
-51100



Steel $\frac{43000 \times 100}{120 \times 70} = 51.2$ cm²

Try $7 - 25\phi = 34.3$ cm² on one side

$p_0 = \frac{34.3 \times 2}{75 \times 75} = 0.0122$

$e = \frac{4300000 \times 12}{3940} = 81.3$ cm

$e/h = \frac{81.3}{75} = 1.083$

$d'/h = 5/75 = 0.067$

Void

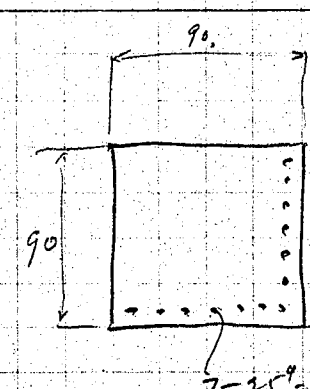
$k = 0.391$ $\quad j = 0.860$

$f_c = \frac{43000 \times 100}{0.86 \times 75 \times 75^2} = 64.0$ kg/cm² $> 35 \times 1.6 = 56$ over stress

$f_s = 15 \times 64.0 \left(\frac{70}{0.391 \times 75} - 1\right) = 1335$ kg/cm² $< 120 \times 1.6 = 192$ ✓

over stress.

2nd Trial.



$p_0 = \frac{68.6}{90 \times 90} = 0.00847$ $\quad e/h = \frac{81.3}{90} = 0.904$

$d'/h = 5/90 = 0.0333$

$k = 138$ $\quad j = 1139$

$f_c = \frac{43000 \times 100}{113 \times 90 \times 90^2} = 45.4$ kg/cm² ✓

$f_s = 15 \times 45.4 \left(\frac{88}{138 \times 90} - 1\right) = 1080$ kg/cm² ✓

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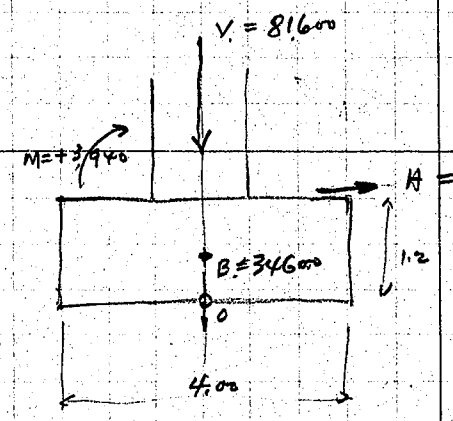
CALCULATIONS FOR

Ketsugase Bashi for Saitama Kem.
Design of column footings.

	M moment	V corresponding load.	H
D.L. + L.L.	+ 3940 kgm	+ 5760 + 2400 (col.) = 8160 kg.	+ 1180 kg
D.L. + Earthquake.	+ 43,000	+ 5294.0 + 2400 = 7694.0 kg	+ 8,360 kg

normal state

Base $1.2 \times 4.0 + \frac{6.0}{2} = 14.4 \text{ @ } 2.4\text{m} = 34600$ kg



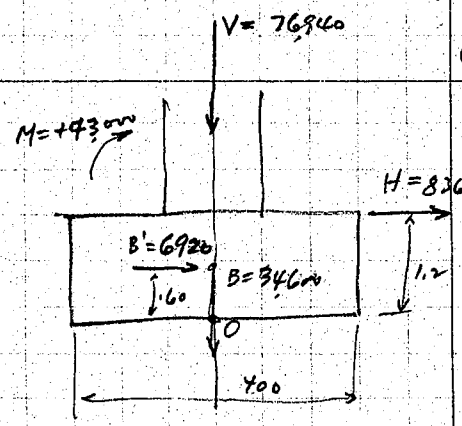
moment abt 0.

	Load	Hor. force	Vert force	Arm	moment
V			81600	× 0 =	0
H	1180			× 1.2 =	1420
B			34600	× 0 =	0
M				=	3940
	1180		116200 kg	0.046	5360 kgm

max. toe pressure $= \frac{116200}{4.0 \times 3.0}\left(1 \pm \frac{6 \times 0.046}{4.0}\right) = 10.350$ kg/m² (0.95 tons/ft²) or 1.9020 "

max. load on one pile $= 0.9 \times 0.9 \times 10.35 = \underline{8.4}$ kg/tons

During earthquake



V			76940	× 0 =	0
H	8360			× 1.2 =	10030
B			34600	× 0 =	0
B'	6920			× 0.6 =	4150
M				=	+ 43000
	15280		111540	0.572	57180

max toe pres. $= \frac{111540}{4.0 \times 3.0}\left(1 \pm \frac{6 \times 0.572}{4.0}\right) = 16.450$ kg/m² (1.50 tons/ft²) or 3.160 "

max. load on one pile $0.9 \times 0.9 \times 16.75 = \underline{13.3}$ tons.

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CALCULATIONS FOR

Akigase Bashi for Gifu-Ken

Materials of Suspended span. span length 8.00m (for 1 span)

Concrete.

slab.	5.5 × 0.13	=	0.715
Copings.	0.25 × 0.35 × 2	=	0.175
stems of beams	0.60 × 1.87 × 2	=	0.864 (1.72) + 0.044
fillets	0.3 × 0.15 × 2	=	0.090
stem of stringer	0.3 × 0.32	=	0.096
fillets	0.1 × 0.1	=	0.010
			2.130 (1.950) m² × 8.00 = 15.60 + 1.05 (2.16 − 2.26) m³
			= 16.65 (17.76) m³ call this 20.00 m³ (18.0)

Cross beams. 0.30 × 0.85 (.80) × 3.0 × 3 = 2.70 (2.70)

Reinforcements 20.0 m³ @ 165 kg = 3.30 kg/tons.
18.0 @ 150 =

Forms. 6.10 + .35×2 + .20×2 + .87 (.72)×4 + .32×2 = 11.32 (10.72)

11.32 × 8.0 = 85.8 (90.5)

Cross beam 3.0 × .85 (.8) × 6 = 14.4 (15.3)

1.95 (1.930) × 2 = 3.9 (4.26)

170.06 (104.1) call this 110.0 m² (105.0)

pavement granolithic 5cm thick

5.5 × 8 = 44.0 m²

45

Drains

Exp. jt

Materials of Concrete Anchor span Total length 12.0 + 2 × 2.0 = 16.0 m

1.95 (1.930) × 16 = 31.2 (30.9)

Cross beam .72 (.765) × 6 = 4.32 (4.6)

35.52 (35.50) call this 39.0 m²

Reinforcements 39.0 @ 165 = 6.44 (5.33) call this 6.5 tons.
35.5 150

Forms. say 105 × 2 = 210 (200.0) m²

pavement. 88.0 m²

Drains 8

exp jt 2 × 2

Shoes. 4

Pier. Concrete.

Columns.	1.75 × .75 × 5.3 × 2	=	5.96
"	.9 × .9 × 3.8 × 2	=	6.15
strut	.8 × .6 × 2.85	=	1.38
"	.6 × .6 × 2.70	=	0.97
"	.6 × .9 × 2.70	=	1.46
"	.6 × .9 × .9	=	0.49
base	4.0 × 6.0 × 1.2	=	28.80
			45.21 call this 45.5 m³

Reinforcements 45.5 m³ @ 50 kg = 2.3 kg/ton.

Forms.

Columns.	1.75 × 4 × 5.3 × 2	=	31.8
"	.9 × 4 × 3.8 × 2	=	27.4
Struts	2.4 × 2.85	=	6.8
	2.4 × 2.70	=	6.5
	2.4 × 3.60	=	8.7
	1.2 × 20.0	=	24.0
			105.2 call this 105 m²

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CALCULATIONS FOR

Akigase Bashi for Saitama Pref.

Approx. weight of pier reinforcements.

Column	vert.	48	@	3.85 × 11	=	2038
	hoop.	80	@	0.888 × 5.5	=	390
Struts		18	@	2.25 × 4.2	=	170
	hoop.	30	@	0.888 × 2.7	=	70
Base		20	@	2.98 × 4.2	=	250
"		10	@	1.58 × 4.2	=	65
"		10	@	1.58 × 6.5	=	102
						3080 call this 3.10 kg tons / pier

Pier 本体 1 基

conc.	0.9×0.9×2.3×2	= 3.73	
	0.6×0.6×2.7	= 0.9×	
		4.67 @ 15.-	= − 70.
reinf.	2423 × 2.9/11	= 500	
	270 ÷ 3	= 80	
		1,585 @ 15.	= − 76
forms.	7.2 × 2.3	= 16.6	
	2.4 × 2.7	= 6.5	
		23. @ 1.5	= − 35
excav.		65 @ 1.8	= − 52
			− 17
			− 250

4.5 / 7.5 8.- 250 / 2.5 / 6.-

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CALCULATIONS FOR

Akigase Bashi for Saitama Ken.

piles 18φ × 7.5 — 24 ㎥
Excavation 8.0 × 10.0 × 3.5 = 280 m³
埋戻 4.5 × 6.5 × 3.5 = 8.8 m³

Concrete 1:2:4

Cement	2 Br	@ 4.50	=	9.00	1.88 Br	@ 4.40	= 8.27
gravel	1.0 m³	@ 3.5	=	3.50	0.90	@ 3.50	= 3.15
Sand	0.5 m³	@ 1.0	=	0.50	0.45	@ 1.00	= 0.45
Labour	2.0 人	@ 1.00	=	2.00	2.5 人	@ 0.70	= 1.75
				15.00 m/m³			13.62 call the 13.70

1:4:8 9.60

Estimate of cost.

Suspended span.

Concrete 1:2:4 mix.
Reinforcements, plain bars
forms
drains
Shoes + exp. jts

18.00 m³	@	14.12 13.70 15.00	=	300.	270 254 247
3.70 3.30 tons	@	130.00	=	430	330
105 110.0 m²	@	1.50 2.00	=	220	160
4 ㏊	@	5.00	=	20	20
				950	1220 19 (1000) 980 990

Anchor span

Concrete 1:2:4 mix
reinforcements
forms
drains
Shoes + exp. jts

39.5 39.00 m³	@	14.12 13.70 15.00	=	585	534 501 487
6.33 6.5 tons	@	130.00	=	845	690
330 220. m²	@	1.50 2.00	=	440	315
8 ㏊	@	5.00	=	40	40
				2060	2160 18 (1780) 1735 1750

Pier 1基2脚 4ヶ所

Concrete 1:2:4
Reinforcements
forms
piles
Excavation
埋戻

45.5 m³	@	14.12 13.70 15.00	=	685	642 623
3.1 2.5 tons	@	130.00	=	405 300	
105.0 m²	@	1.50 2.00	=	157 210	
24 ㏊	@	15.00	=	360	
28.0 m³	@	0.8 1.00	=	224 280	
8.8 m³	@	4.00	=	35	
				1870	18 ✓ 1820

Cost of approach concrete girder Bridge 50 C 12 = 600 m.

Suspendid spans	26 spans	@	1000 980	=	31700	26000 25480
Anchors	24	@	1780 1735	=	51800	42700 41600
granolithic pav.	5.5 × 6.00 = 3300 m²	@	2.5	=	8,300	
Handrails	2 × 6.00 = 1200 m	@	6.7	=	8000	
					99800	85000 83400
Piers	27	@	1870 1800	=	50300	48600
	3	@	2000	=	6000	
	18	@	1600 1500	=	28800	27000
					85300	81600
abutments	2	@	3500	=	7000	

Total cost for approach girder bridge = 192100 (9230) 8860 177300 172000

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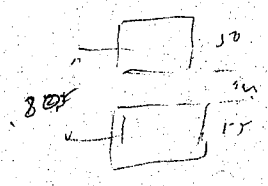

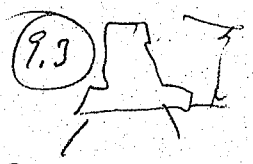

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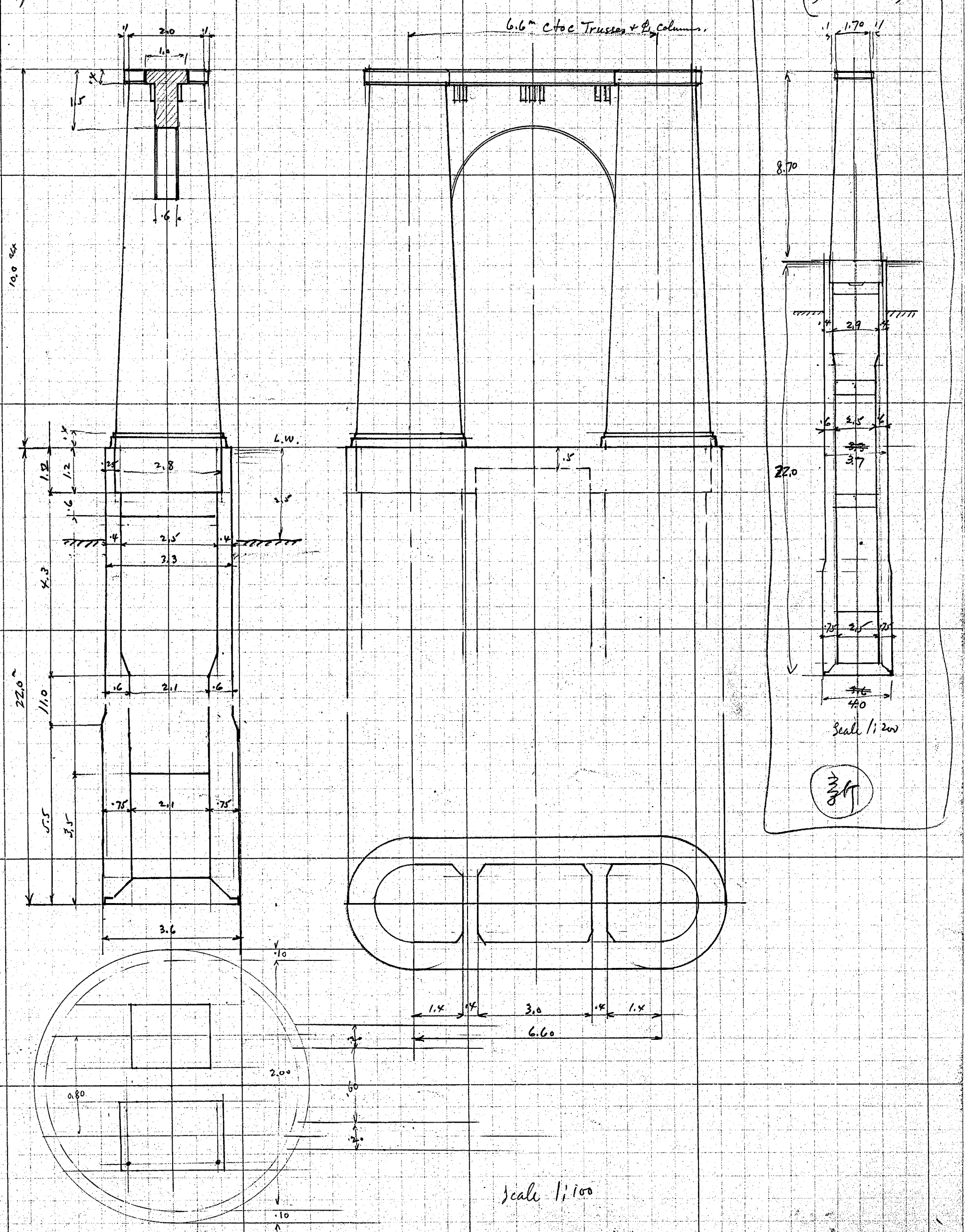
CALCULATIONS FOR

Akigase Bashi for Saitama-Ken.
Design of River spans.
Piers.

(revised)



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CALCULATIONS FOR

Kigase Bashi for Saitama Pref.

Approximate weight of pier

Shaft. Coping 2.2φ × 0.4 × 2 = 3.04
" 1.0 × 0.4 × 4.5 = 1.80
Strut. 0.6 × 1.6 × 4.5 = 4.32
Shafts. 2.4φ × 9.2 × 2 = 83.21
3.2φ × 0.4 × 2 = 6.43
0.20

wt. of shaft. 99.00 m³ @ 2.40 = 238.000 kg

well. 22m long.

3.3φ = 8.55

3.3×6.6 = 21.78
30.33 × 1.2 = 36.40
2.5×6.6×0.7 = − 11.50
24.90

30.33
− 2.5×5.8 = − 14.50
·3×3×4 = + 0.36
− 2.5φ = − 4.91
11.28 × 4.3 = 48.50

30.33
− 2.1×5.80 = − 12.18
·3×3×4 = + 0.36
− 2.1φ = − 3.46
15.05 × 11.0 = 165.55

3.6φ = 10.18
3.6×6.6 = 23.75
33.93
− 12.18
+ 0.36
− 3.46
18.65 × 2.0 = 37.30
33.93 × 3.5 = 118.75
1:2:4 — 395.00 @ 2.40 = 948.000 kg.

1:4:8 filling
2.5×5.8 = 14.50
− ·3×3×4 = − 0.36
2.5φ = 4.91
19.05 × 0.60 = 11.43

Sand filling
2.1×5.8 = 12.18
− 0.36
2.1φ 3.46
15.28 × 0.6 × 2 = 18.32
29.75 @ 2.2 = 66.000 kg
call this 30m

19.05 × 3.70 = 70.50
15.28 × 11.80 = 180.50
251.0
11.5
262.5 @ 1.7 = 446.000 kg
wt of well. = 1.460.000 kg

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CALCULATIONS FOR

Akigase Bashi for Saitama Pref.

Summary of concrete for pier.

	Volume of Concrete 1:2:4	Volume of Concrete 1:4:8	Sand filling	weight
Shaft.	99.00 m³			238,000 kg
well	395.00	30.00	263.00	1460.00
	494.00	30.00	263.00	1,698,000 kg.

Superimposed load on pier

D.L.	182.00
L.L.	100.00
	282.00
Total weight of pier	1698.00
	1980,000 kg.

Bearing pressure on bottom of well.

Total load including weight of pier = 1,980,000 kg
Frictional resistance say 1200 × 24.0 × 1.8 = − 520.00
1,460,000 kg.

Bottom area of well = 33.93 m²

Bearing pressure = 1460.00 / 33.93 = 43,000 kg/m² (3.93 tons/ft²)

Materials of pier

Concrete 1:2:4 Shaft 99.00
well 395.00
494.00 m³

1:4:8 filling 30.00 "

Sand filling 263.00 "

Reinforcements. Shaft 99 m³ @ 35 kg = 3,470 kg.
well 395 m³ @ 50 = 19,500
30
23,000 kg.

Form.

Shaft. 7.5 × 2 × 10 = 1500
2.0 × 5 = 10
1.0 × 5 = 5
1650

well. 24.0 × 22 = 528.
24.0 × 22 = 528
1056

1220 m²

Shoe say 1.200 tons.

Estimate of cost.

Concrete	1:2:4	494.00 m³ @	13.70	=	6,770
	1:4:8	30.00 " @	9.50		285
Sand fill		263.00 " @	0.50		132
Reinforcements		23,000 kg @	130.00		2,990
Forms		1220.0 m² @	1.50		1830
Shoe		1.2 tons @	200.00		240
Excavation	(打込み) -36 m	22.0 m³ @	225.00		4950
150.0 well island	600 + 200	1 @	1300.0		1300
					18497 calculation

3m 深サノ好セン

+ 60.0	@ 14.		840
−			
60.0	@ 5		30
+ 3,000	@ 130		390
150	@ 1.5		225
37.40	3.0 m	@ 450	1350
			2835 好
		Calcula	3000 円

18500 円 17300 采領

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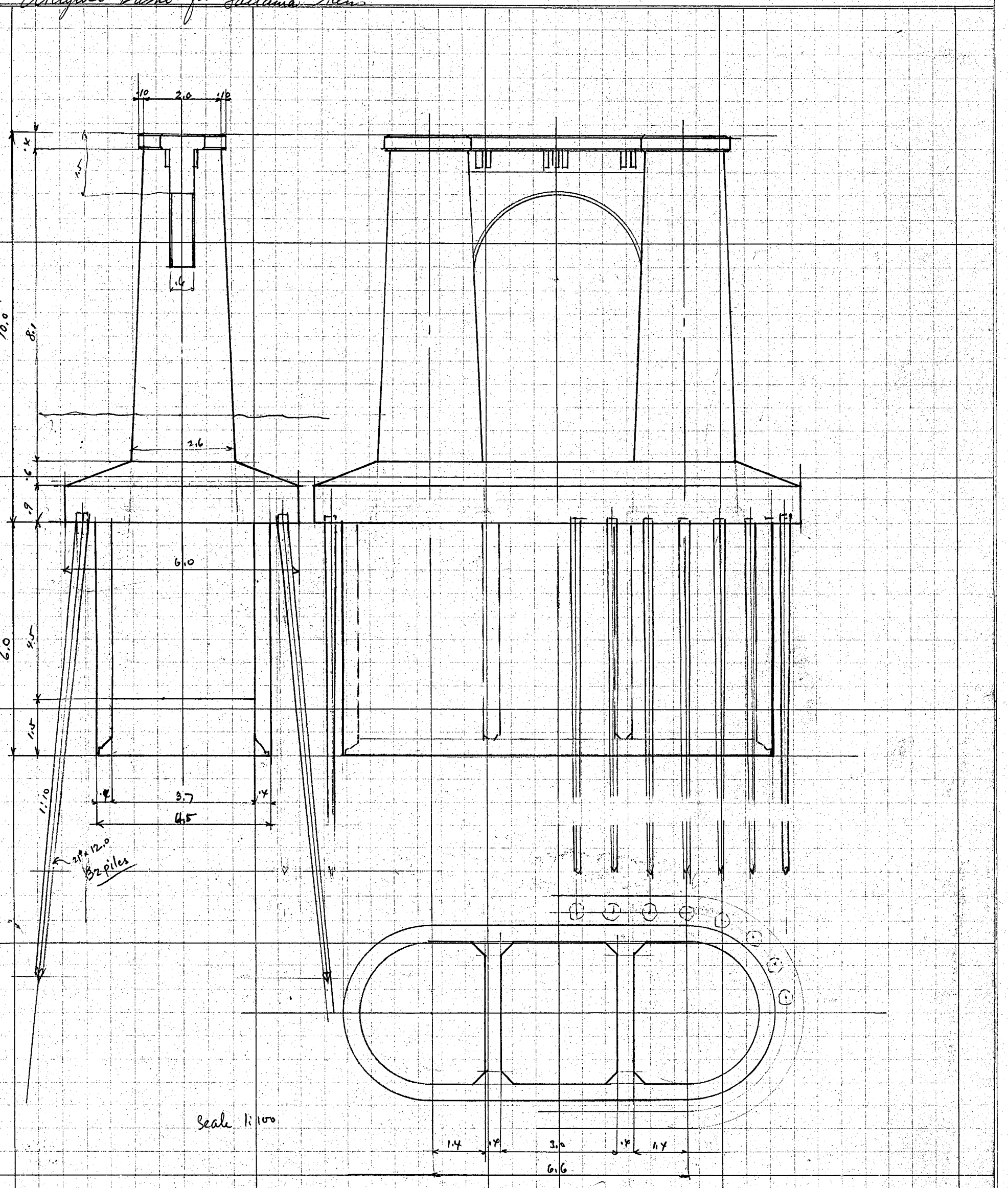
CALCULATIONS FOR

Utsugase Bashi for Saitama Pref.



27.5

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CALCULATIONS FOR

Akigase Bashi for Saitama Ken.

Materials:

- Concrete.

Coping 4.84
Strut 4.32
Shaft 2.3φ × 8.1 × 2 = 67.20
footing 7.98 × 9.6 = 76.60
153.0 m³

2.6
6.0
8.6 7.3 × 1.6 = 29.8
6 × 1.3 = 54.0
7.98

well.
4.5φ = 15.90
4.5 × 6.6 = 29.70
45.60

3.7 × 5.8 = − 21.48
1.3 × 1.3 × 4 = + 1.36
3.7φ = − 10.75
−31.87
13.72 × 6.0 × 2 = 82.4
31.87 × 1.5 = 47.8
130.2
1:2:4 — 283.2 m³

Sand filling 31.87 × 4.5 = 144. m³

Reinforcements Shaft 153.0 @ 80 = 5350
well 130.× @ 50 = 6550
11900 kg.

Forms Shaft 2.3φ × 8.5 × 2 = 123
8.0 × 4.7 = 24
0.9 × 32 = 29
Kutt 0.75 × 30 = 24
200 m²

17 well 4.5φ = 14.14
6.6 × = 13.2
27.6 × 6 = 166 m²

17φ = 11.62
5.8 × 2 = 11.6
3.7 × 4 = 14.80
38.02 × 6 = 228
394 m²
594.0 m²
1.5 tons

Excav. above L.W.
10 × 17 × 3 = 510 m³

Estimate of cost.

Concrete 1:2:4	283.2 m³	@	13.70 =	3880	153 @ 14.12 =	2080
Sand filling	144.0 =	@	0.50 =	72		
reinforcements	11.9 tons	@	130.00 =	1547	535 @ 130 =	695
Forms	594.0 m²	@	1.5 =	891	200 @ 150 =	300
Shores	1.5 tons	@	200.00 =	300		
well sinking (¥3 B 26, 3 19)	6.0 m	@	200.00 =	1200		
excavation open	510 m³	@	1.00 =	510	510 @ 1.0 =	510
piles 21φ × 12.0m	32 ea	@	40.00 =	1280	72 @ 33. =	2160
				9680	32 @ 35 =	1120
				call this	9700 ¥	6765

Call this 6200 ¥

.67
535
130
665

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CALCULATIONS FOR

Akigase Bashi for Saitama Ken.

Estimate of River Spans. 1 Spans., Total length 3@37.5 + 1-25.5 + 8 = 166 m abt.

Structural steel Co.o = 166/38. = 260.m tons @ 200.m = ¥52000 B

Deck construction

	Slab concrete	160 m³ @ 13.70 =	2,190
.86	reinforcements	16 tons @ 130. =	2080
	forms	1050 m² @ 1.50 =	1575
	granolithic	905 @ 2.50 =	2260
	drains	44 ₹ @ 5.0 =	220
	Handrail	332 m @ 6.5 =	2160
	const. jts	18 sy @ 10 =	180
	misc		331
			11000
			63000

Piers

2 @ 18.5 m = 37.0 m
3 @ 9.7 m = 29.1 m

66,100
129,100 B

Summary of cost for the entire bridge

Approach girder bridge including abutments = 172000
River span = 129.1 m
301,100 B

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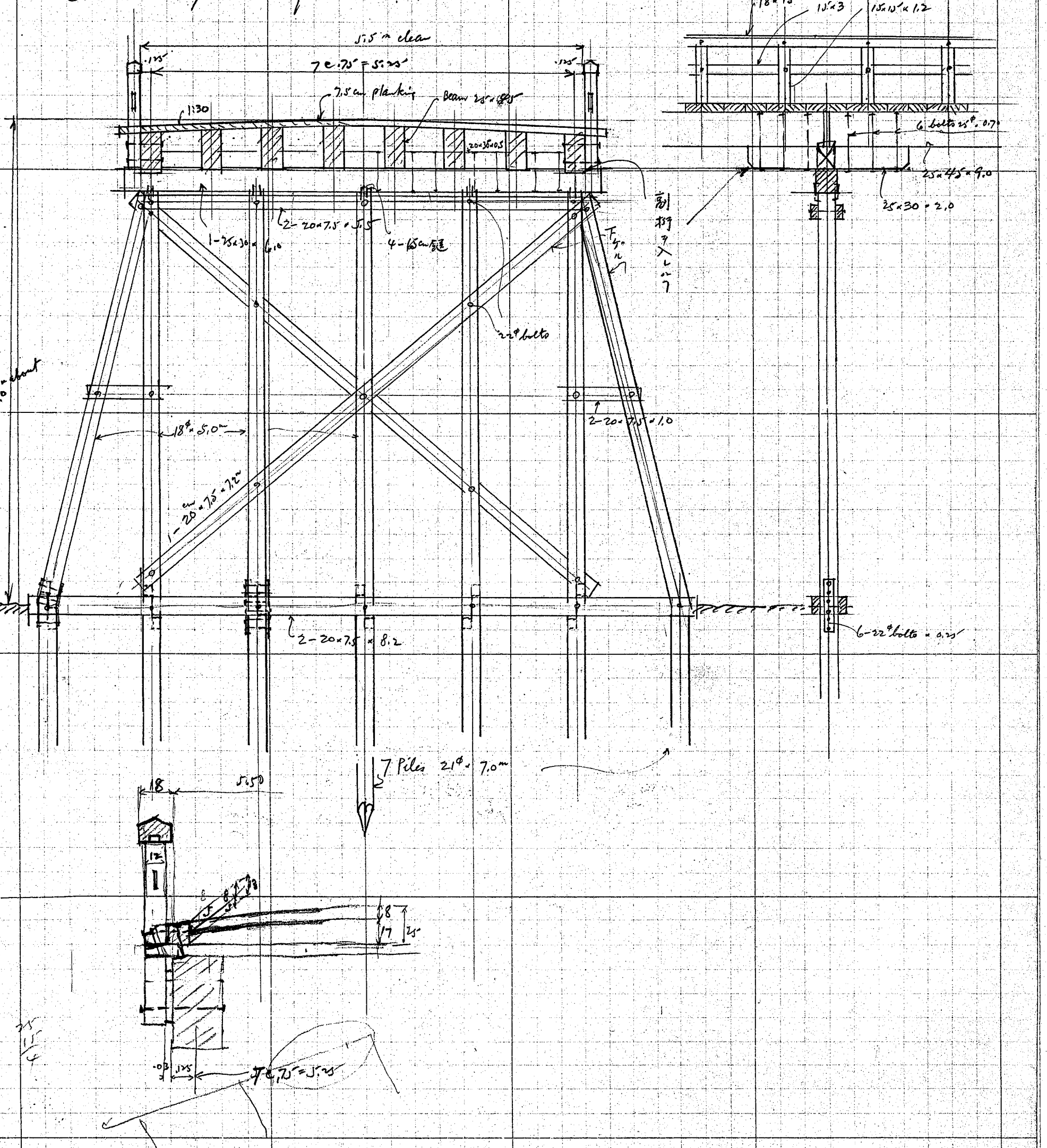
CALCULATIONS FOR

Akigase Bashi for Saitama Ken.

Preliminary estimate of temporary wooden bridge on the approach of Urawa side
Total length of wooden beam bridge = 23 spans @ 9.0m = 207 meters.
Clear width of roadway = 5.5m.



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CALCULATIONS FOR

Utigase Bashi for Saitama Ken.

Design of floor planking + main beam.

planking span length 0.75 m.

D.L. 7.5 cm @ 6.5 = 500

D.L. m = 1/10 × 500 × .75² = 3 kgm.

" shear = 1/2 × 500 × 0.75 = 19 kg

motor truck rear wheel con = 2250
30% imp = 680
2930 kg.

Eff. width E = 2/3 l + a = 2/3 × 75 + 0.24 = 0.50 + 0.24 = 0.74

Concentration per meter strip = 2930/0.74 = 4000 kg.

L.L. moment = 4000 × 0.75 / 4 = 750

for continuity of planking m = 8/10 × 750 = 600 kgm.

shear 4000 kg

Summary of moments + shear

	moment	shear
D.L	3	19
L.L	600	4000
	603 kgm	4019 kg

Sm req'd. for f = 60 kg/cm²

Sm = 603×100/60 = 1005.7 cm³

d = √(1005.7×6/100) = 7.77 cm. b d²/6 = 1093

unit shear = 4019/(100×7.5 × 3/2) = 8.04 kg/cm² ✓

use 7.5 cm planking

Main beam Span length 9.0 m spacing 0.75 m c to c.

D.L. planking 0.75 × 500 = 375
beam assumed 120.0 / 2.5
160.0 kg/lin m

D.L. moment = 1/8 × 9.0² × 160 = 1620 kgm
D.L. shear = 1/2 × 9.0 × 160 = 720 kg

Live load. 500 kg/m² only.
load on beam 0.75 @ 500 = 375 kg/lin m.
L.L. m = 1/8 × 9.0² × 375 = 3800 kgm
L.L. s = 1/2 × 9.0 × 375 = 1690 kg

Summary

	moment	shear
D.L.	1620	720
L.L.	3800	1690
	5420 kgm	2410 kg

Sm req'd. = 5420×100/60 = 9040 cm³

d = √(9040×6/75) = 46.6

use 25 × 48 = 10" × 18" ネガス.

剥材ニテ 8cm ヲ減ズル.

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CALCULATIONS FOR

Ukigase Bashi for Saitama Ken.

Estimate of cost for one span.

Deck construction

planting (植付) 7.5 cm thick	6.0 × 9.0 =		54.00 m² @ 6.00	= 324
main beams (主桁)	8 — 25 cm × 45 cm × 9.0 m	=	8.10	
副桁	8 — 25 × 30 × 2.0	=	1.20	
spacers	7 — 20 × 35 × 0.5	=	0.245	
			9.545 m³ @ 55.00	= 525
Handrail				
笠木	6 — 18 × 15 × 3.1	=	0.502	
要	6 — 15 × 3 × 3.0	=	0.081	
束木	18 — 15 × 15 × 1.2	=	0.486	
			1.069 m³ @ 60	= 65
ボーム	床-桁材 96 — 25φ × 0.70	@ 2.20	= 217	
〃	梁-支材 14 — 22φ × 0.60	@ 2.15	= 30	
〃	高欄 36 — 12φ × 0.40	@ 0.47	= 17	
			264 kg @ 1.25	= 66
				980 ¥
				900

Pier.

梁	1 — 25 × 30 × 6.0	=	0.450	
扶要	2 — 20 × 7.5 × 5.5	=	0.165	
〃	2 — 20 × 7.5 × 8.2	=	0.246	
筋違	2 — 20 × 7.5 × 7.2	=	0.216	
扶要	4 — 20 × 7.5 × 1.0	=	0.060	
			1.137 m³ @ 75	= 85
支柱	57 — 18φ × 5.0	=	1.16	
杭	57 — 21φ × 7.0	=	2.30	
			3.46 m	@ 50 = 175
杭打ち			74	@ 10 = 70
				8
ボーム	27 — 22φ × 0.35 m	@ 1.37	= 37	
〃	42 — 22φ × 0.35	@ 1.06	= 45	
丹ボウ金物	14 — 60 mm × 6 × 0.60	@ 1.70	= 24	
鎖	20 — 15 cm 鎖	@ 0.5	= 10	
			116 kg @ 1.25	= 39
				370

Total cost of wooden bridge (207 m)

Super structure	23 spans	@ 980	=	22550
piers including abutment piers	23 piers	@ 370	=	8520
				31070 ¥

Estimate of cost for [illegible]

Superstructure			
piers including abutment	23 piers	@ 370	=
			31070

Reinforced concrete girder bridge

Superstructure	17 spans	@ 950	=
Abutments	16	@ 1770	=
pavements	[illegible]	@ 2.50	=
handrails	786	@ 6.70	=
Piers	6	@ 1800	=
〃	7	@ [illegible]	=
〃	3	@ 1570	=
abutments	1	@ 3320	=

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CALCULATIONS FOR

Preliminary estimate of Akigase Bashi for Saitama Ken.

Summary of cost for the entire bridge.

第一案 { 木橋 / 混凝土桁 / 鉄構橋 }

Approach wooden beam bridge (23 spans @ 9.0m = 207 meters)

Super structures	23 spans	@ 980.	=	22550
Piers including pier abutment	23 piers	@ 370	=	8520
				31070

Approach concrete girder bridge (33 spans @ 12.0m = 396 meters)

Suspended spans	17 spans	@ 980	=	16650
Anchor "	16 "	@ 1735	=	27750
pavement 5cm granolithic	2162 m²	@ 2.50	=	5410
handrails	786 m	@ 6.70	=	5260
				55070
Piers	26	@ 1800	=	46800
"	3	@ 2000	=	6000
"	3	@ 1500	=	4500
abutment 高木側	1	@ 3300	=	3300
				60800
				115870

River spans. (3@37.5 + 52.5 = 165. meters)

Structural steel	260. tons	@ 200	=	52000
Deck construction complete				11000
				63000
Piers	2	@ 18500	=	37000
"	3	@ 9700	=	29100
				66100
				129100
				276040 円

第二案 { 混凝土桁 / 鉄橋橋 }

Approaches concrete girder spans. (50 spans @ 12.0m = 600 meters)

Suspended spans	26 spans	@ 980.	=	25450
Anchor "	24 "	@ 1735	=	41650
pavement 5cm granolithic	3300 m²	@ 2.5	=	8300
handrails	1200 m	@ 6.70	=	8000
				83400
Piers	27	@ 1800	=	48600
"	3	@ 2000	=	6000
"	18	@ 1500	=	27000
				81600
Abutments	2	@ 3500	=	7000
				172000

River spans. (3@37.5 + 52.5 = 165. meters)

Super structure	63000
Deck "	66100
	129100
	301100

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CALCULATIONS FOR

Estimate of Akigase Bashi for Saitama Ken

River-span-pier see page 9.

Approximate volume of concrete and weight of pier.

Shaft. Coping 1.9φ × 0.4 × 2 = 2.27
" 1.0 × 0.4 × 4.8 = 1.92
Strut 0.6 × 1.6 × 4.8 = 4.61
Shaft. 2.1φ × 7.9 × 2 = 54.70
" 3.0φ × 0.4 × 2 = 5.62
Shaft. 1:2:4 70.00 m³ @ 2400 = 168.000 kg

Well 22 m long.
3.7φ = 10.75
3.7×6.6 = 24.45
35.20 × 1.2 = 42.25
less. 2.9 × 0.7 × 3.0 = - 6.08
36.17

35.20
- 2.9 × 5.8 = - 16.82
- 2.9φ = - 6.61 } -23.07
3 × 3 × 4 = + 0.36

12.13 × 4.3 = 52.20

35.20
- 2.5 × 5.8 = - 14.50
- 2.5φ = - 4.91 } -19.05
= + 0.36

16.15 × 11.0 = 177.70

4.0φ = 12.57
4.0 × 6.6 = 26.40
38.97
- 19.05

19.92 × 2.0 = 39.84
38.97 × 3.5 = 136.45
well 1:2:4 — 442.36 @ 2400 = 1,062,000 kg.

1:4:8 filling 23.07 × 0.6 = 13.85
19.05 × 0.6 × 2 = 22.85
1:4:8 — 36.70 @ 2200 = 81,000 kg.

Sand filling 23.07 × 3.7 = 85.4
19.05 × 11.8 = 224.8
6.1
Sand — 316.3 @ 1.7 = 538,000 kg.

Total wt. of well. = 1,681,000 kg.

Summary of concrete for pier

	Volume of concrete 1:2:4	1:4:8	Sand fill.	weight
Shaft	70.0			168000
well.	443.0	37.0	317.0	1,682m
	513.0			1,850,000 kg.

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CALCULATIONS FOR

Estimate of Akigase Bashi for Saitama Pier

Summary of loads on pier

Superimposed loads

D.L. 182.0m
L.L. 100.0m
282.000

Total wt. of pier = 1850m
2,132.000

Bearing pressure on bottom area of well 38.97 m²

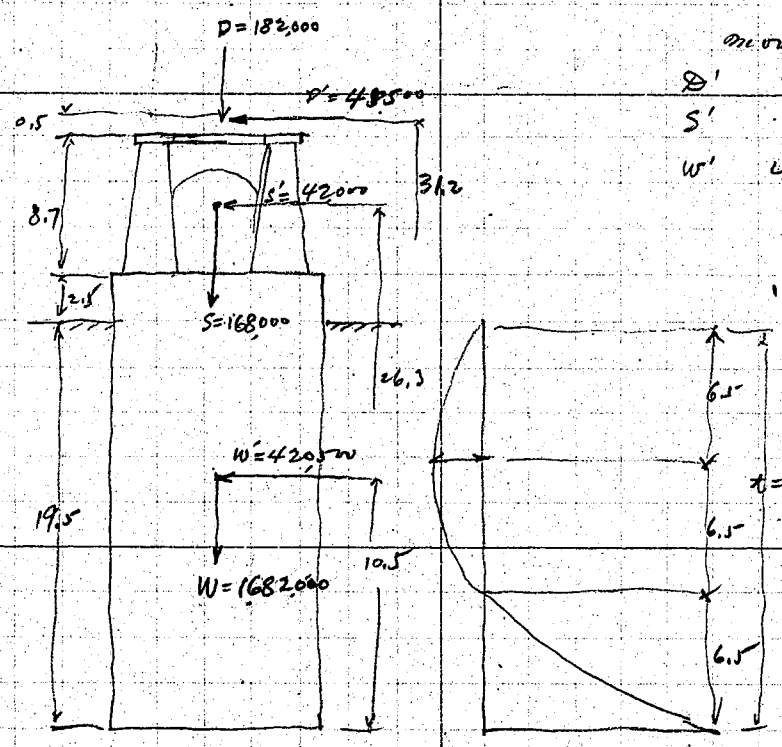
Total load including wt. of pier = 2132.000
Frictional resistance 1200 × 25.45 × 18 = − 550.0m
1,582.0m

Bearing pressure = 1582,000 / 38.97 = 40600 kg/m² or (3.71 ton/0')

12

12.25'
23.2
35.45

Stability of pier during earthquake. k = 0.35



		moment		
D'	45500	× 31.2	=	1,420,000
S'	42,000	× 26.3	=	1105.0m
W'	42050	× 10.5	=	441.5m
		M =		6,940.0m

$r_b = \frac{12M}{t^2} = \frac{12 \times 6940.0m}{19.5^2} = 21900$ kg/

$r_a = \frac{r_b}{3} = \frac{219.0m}{3} = 73.0m$ kg

Unit pressure on earth

$r_b = \frac{219.0m}{4.0} = 54800$ kg/m² or (5.6 ton/0')

$r_a = \frac{73.0m}{3.7} = 19.7m$ or (1.8 ")

Passive earth pressure during earthquake

$p = wx \frac{1+\sin\phi'}{1-\sin\phi'}$ where $\phi' = \phi - \tan^{-1} k$

$\phi' = 30° - \tan^{-1} 0.250 = 30° - 14° = 16°$

$\sin\phi' = 0.2756$

$\frac{1+\sin\phi'}{1-\sin\phi'} = \frac{1.2756}{0.7244} = 1.76$

$p = 1600 \times 1.76\, x = 2815x$

at r_a 2815 × 6.5' = 18,300 kg/m²
at r_b 2815 × 19.5' = 54900

This is 2 m, I think it's enough.

31.4000
~~31.5~~
906.25'
14718.7
216
216
490.625'
690,800
500 1.2 1.1
1600m

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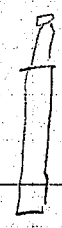
CALCULATIONS FOR

Estimate of Akigase Bashi for Saitama Ken.

1:2:4 鉄筋コンクリート	Cement	1.88	@ 4.42	=	8,310	
	Sand	0.45	@ 0.96	=	0.432	
	gravel	0.90	@ 4.03	=	3.627	
	labour	2.5	@ 0.70	=	1.750	
					14.119	calculate 14.12 ¥/m³
1:4:8 "	Cement	1.05	@ 4.42	=	4.641	
	Sand	0.60	@ 0.96	=	0.432	
	gravel	0.90	@ 4.03	=	3.627	
	labor	1.8	@ 0.70	=	1.26	
					9.96 ¥/m³	

Steel 70 @ 35 = 2450
440 @ 50 = 22,150
24,600 kg.

Estimate of cost for river pier. (22m well, ø1)



Concrete	1:2:4	513.0 m³	@ 14.12	=	7245
"	1:4:8	37.0 "	@ 9.96	=	369
Sand fill		317.0 "	@ 0.40	=	127
Reinforcement		25.0 tons	@ 130.00	=	3,250
Forms		1,250. m²	@ 1.50	=	1875
Shoe		1.2 tons	@ 200.00	=	240
Excavation	38.97×22=857 m³ (Tj 31.19)	22 m	@ 200.00	=	4,400
well island					1,200
ø2 m					294

19000 ¥

Estimate of cost for river pier (6m caisson ø2.21)

Concrete 1:2:4	280. m³	@ 14.12	=	3960
Sand fill	144 "	@ 0.40	=	58
reinforcement	11.9 tons	@ 130.0	=	1548
forms	590. m²	@ 1.50	=	885
Shoes	1.5 tons	@ 200.	=	300
well sinking (Tj 26.13)	6.0 m	@ 200.	=	1200
Excavation above C.W.	510 m³	@ 0.7	=	357
piles 21ø × 12.0m	32 ø	@ 30.0	=	960
				32

9300 ¥

River span Superstructure

Structural steel	260.00 tons	@ 200.00	=	52,000
Deck construction complete				11,000

63000

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CALCULATIONS FOR

Estimate of Akigase Bashi for Daitama Ken

Approach wooden beam bridge (23 span @ 9.0 = 207 meters).

Super structure:	23 spans	@	900	=	20,700
Piers including pier abutment	23 Piers	@	280	=	6,500
					27,200

Approach concrete girder bridges (33 spans @ 12.0 m = 396 meters)

Suspended span	17 spans	@	990.00		16820
Anchor span	16 "	@	1750.00		28000
Pavement 5 cm granolithic	2162 m²	@	2.50	=	5,420
Handrails	786 m	@	6.70	=	5,260
					55500
Piers	26	@	1820	=	47,300
"	3	@	2000	=	6,000
"	3	@	1500	=	4,500
Pier abutment R.C. type	1	@	3500	=	3,500
					61300
					116,800

Steel river spans. (3 @ 37.5 + 5 @ 21.5 = 165 meters)

Structural steel	260. tons	@	2000	=	52,000
Deck construction complete				=	11000
					63000
Piers	2	@	19,000	=	38,000
"	3	@	9,300	=	27,900
					65,900
					128,900
					272,900

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CALCULATIONS FOR

Estimate of Cost for Arigase - Bashi Saitama-Ken

20 metre Span

Dead weight of deck - 19.00 tons. 20

950 kg per lin metre.

Deck construction - 13.5 × 0.5 = 24.2
coping .240
.982 cr400 =

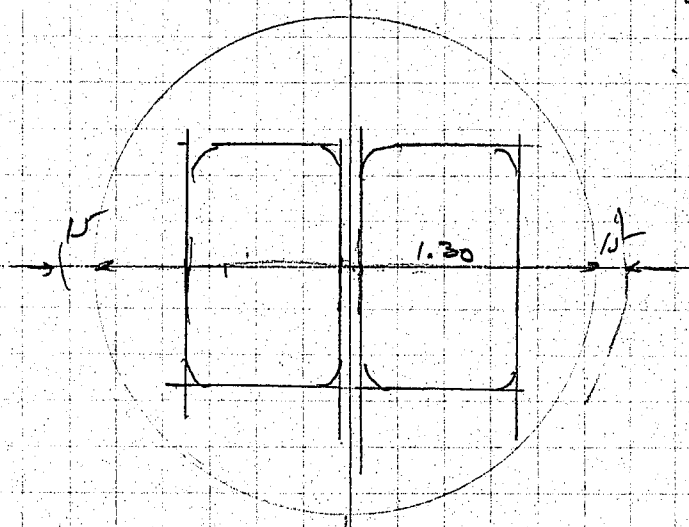
pavement 106 kg × 0.75

2350 -
883.
2933
950
3883 × 20.0 = 77700 kgs.
66,000
143,700 kgs.

110
60
170

Live Load. Say 600 × 5.5 = 3,300 × 20 = 66,000 kgs.
~~600 × 20 = 12,000~~

Base 40 c × 60 c Spacing. ~~4.80~~ m



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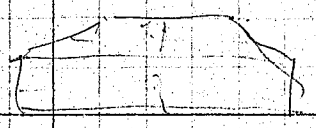
CALCULATIONS FOR

Estimate of Cost for Akigase - Bashi for Saitama-Ken
approximate list of materials

Top area - 1.3 9 1.320
Bottom area - 2.2 3.80
2 × 5.12 / 2 × 8.5 = 43.5 cubic metres

10.
1.5
8.5 — ~~Struts~~ .60 × 1.2 × 3.50 = 2.5

Base - 4.5 × .75 = } 3.0 m² × 2 = 6
3.5 × .75 = }
volume say 2 × 3.0 × 9.30 = 56. ~~28.0~~ (10%) = 74.0 cubic metres.



Reinforcing Bars. say. 40- @ 3.00 × ~~100~~ 120 kg
120.0 24 m
(2.5) 30 m

2.5 tons

forms

Excavation 4.5 × 9.3 × 5 = 210
Real excavation say 400

2 × 4.6 = 9.2
2 × 9.3 = 18.6
27.6
shaft - 2 × 5.6 × 8.5 = 95.0
10.6
133.0 square metres

Piling say 10 × 5 = 50 piles.

Estimate of Cost

Concrete	(74) 10m	@ 12.88	=	952 m
Reinf. Bars.	2.5	@ 120 m	=	300 m
forms.	133.0	@ 1.5	=	200 m
excavation	400	@ 1. m	=	400 m
piling	(50) 15	@ ~~20~~ m	=	750
				2602 m

Call this 2500 m

Total load 74. @ 24 m = 178.000 m
Superimposed load. 144 m
322. m
41.9
36.0

= 9,000 m kgs/m²
= 7.1 m tons/m²
9 m tons/m²
1.25 tons/m²

Base area = 4.5 × 9.3 = 41.9
4. 4.0 × 9.0 = 36
48
4.
36
(9.3)
(4.0)

Seismic stability load
144.000 × .25 = 36.0 m × 10.0 = 360 m
178.000 × .25 = 44.5 m × 5.0 = 222.5 m
322 m 80.0 m 582.5 m
80.5 m × 2 = 160. m tons
422.5 m

Acc = 422.0 m / 322 m = 1.31 g

322. m × 2 / 2.07 × 9.0 = 34.5 m kgs/m² = 3.16 tons/m².

322.0 m × 2 / 2.82 × 9.3 = 24.6 m kgs/m² = 2.25 tons/m².

OK for 250 m acc

2.5 / 1.31 = 1.9
2.82
2.07

.69 = 3.

33.0 / 4.5 = 7.3 = 1.67

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CALCULATIONS FOR

Estimate of Cost Akigase-Bashi Saitama-Ken

River span

Structural steel in truss span 260 t @ 220 = 57200
Deck construction 166.6 × 5.5 = 916.0 @ 12 = 11000
63000

Substructure - 2 @ 1850 = 37000
3 @ 6200 = 18600
55600
118600
-2%
116228

Girder span 20.0 meter out to out. 2980 etc of pier.

Structural steel
19.14 @ 200 = 3828
Deck construction 20.1 × 5.5 = 110 @ 12 = 1332
5160
25%
7660

Substructure one pier

19 span @ 7660 = 145540

River span 116228
Girder span 14550
2000
Beam span 2760
135310

Wooden beam span 9.0 meter span use wood.

~~180~~ 120 @ 23 = 2760
46
276

River span
Girder span
100.19
99 11
14 spans @ 7660 = 107000
11860

River span — 123800
15 @ 7660 — 115000
extra - abutment 2000
32 @ 120 = 3840
279300

Wooden Beam
22
34 span @ 120 = 4080
3
2660
5200
2000
27360
1

60

3 span

abutment

363.66

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CALCULATIONS FOR

Okigase-Bashi for Saitama-Ken

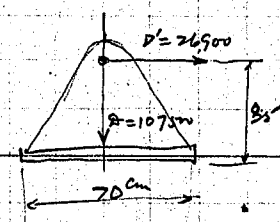
Design of Shoe at L12.

max. load = 164,500 kg c DL = 107,500, LL 56,800, 164,300

Bearing area required = $\frac{164,300}{45}$ = 3,653 cm² 60 × 70 = 4200

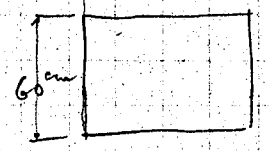
Seismic thrust 107,500 × 0.25 = 26,900 kg.

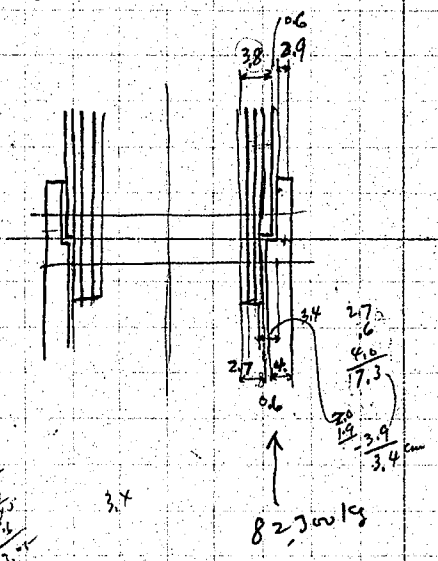
Eccentricity e = $\frac{26900 \times 35}{107,500}$ = 8.75 cm



max. bearing pressure during earthquake

= $\frac{107500}{60 \times 70}\left(1 \pm \frac{6 \times 8.75}{70}\right)$ = 44.75 kg/cm² < 45 × 1.6 or 6.4





(3.75) = 29,000

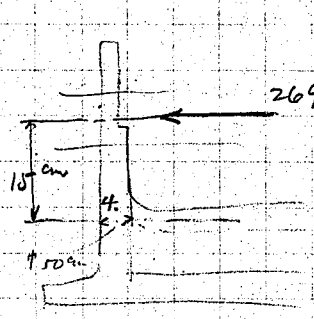
moment on pin 8230 × 3.40 = 28,000 kg cm

Use 120φ pin allowable bending m = 30.4 kg cm

Bearing

$\frac{8230}{180}$ = 45.7 cm²

t = $\frac{457}{12}$ = 3.8 cm



m = 13,500 × 15 = 202,500 kg cm

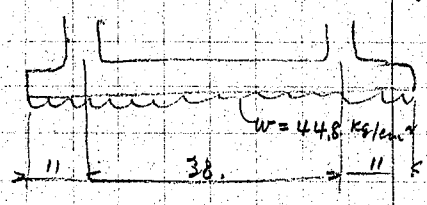
S_m = $\frac{50 \times 4^2}{6}$ = 133.4

fibre stress = ± $\frac{202500}{133.4}$ = 1,520 kg/cm² < 1920

3.5T

Shear $\frac{13500}{50 \times 4}$ = 67.5 × $\frac{3}{2}$ = 101 kg/cm² ✓

8 c.c.



m = $\frac{44.8 \times 38^2}{8}$ = 8080

$\frac{44.8 \times 11^2}{2}$ = − 2710

m = 5,370 kg cm/cm strip

S_m = $\frac{1 \times 4^2}{6}$ = 2.67 cm³

stress = $\frac{5370}{2.67}$ = 2010 kg/cm² > 1920

5% over.

use ribs

5 ribs

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CALCULATIONS FOR

Akigase-Bashi for Saitama-Ken.

Shoe for simple span and anchor span.
Fixed shoe.

max. load on shoe

Simple span.

DL	41600	× 0.25 =	10,400 kg
LL	29560		
	71160		

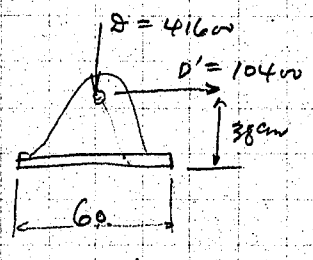
Anchor span

DL	31000	× 0.25 =	7,750 kg
LL	26900		
	57900		

Bearing area reqd = $\frac{71160}{45}$ = 1580 cm²

60.0 × 60.0 = 3600 cm²

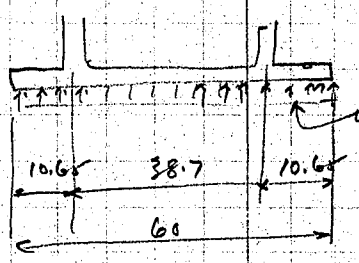
$P = \frac{71160}{3600} = 19.8$ kg/cm² ✓



$e = \frac{10400 \times 38}{41600} = 9.5$ cm

max bearing pressure = $\frac{41600}{60 \times 60}\left(1 \pm \frac{6 \times 9.5}{60}\right) = \frac{22.5}{0.5}$ kg/cm² < c

or



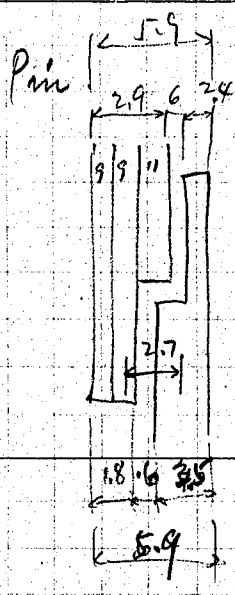
moment

1/8 × 22.5 × 38.7² = 42210 kg cm
1/2 × 22.5 × 10.65² = − 1,280
2930 per cm strip

sm. = $\frac{1 \times 4.5^2}{6}$ = 3.38

$f = \frac{2930}{3.38} = 867$ kg/cm² < c ✓

Pin



Bearing on pin

= $\frac{71160}{10 \times 2.9 \times 2}$ = 1227 < 1800 ✓

Moment on pin

= $\frac{71160}{2} \times 2.7$ = 96100 kg cm

1.75		
1.45		
3.20		
5.90		
2.70	✓	

100⁰ pin m = 176700 ✓

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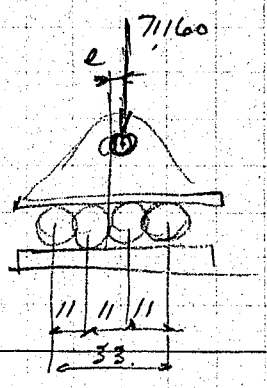
CALCULATIONS FOR

Akigase Bashi for Saitama-Ken.
Roller shoe for Simple span.



Max. load on shoe

DL = 41,600 × 0.25 = 10400 = D'
LL = 29560
71160 kg

4 — 100φ rollers assumed.

Eccentricity

expansion $\frac{3750}{1000} \times \frac{1}{2} = 1.88$ cm

erection error etc $\frac{1.12}{300\text{ cm}} \times \frac{1}{2} = 1.5$ cm = e.

equiv. Bearing pressure. 0.2045 7957

$= \frac{71160}{44 \times 1} \times \left(1 \pm \frac{6 \times 1.5}{44}\right) = 1945$
1285

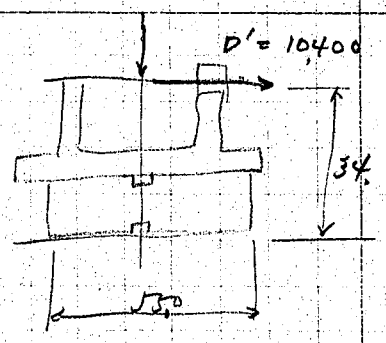
3 2.3 6

max pressure for one roller = 1945 × 11 = 21,400 kg 1×100

allowable comp. on roller = 45 × 10 = 450 kg/cm²

req'd. length of one roller = $\frac{21400}{450} = 47.6$ cm use 50 cm net.

D = 41600



Acc. = $\frac{10400 \times 34}{41600} = 8.5$ cm

DL on one roller = 41600 ÷ 4 = 10,400 kg

max. Bearing pressure on roller 1.027

$= \frac{10400}{55} \left(1 \pm \frac{6 \times 8.5}{55}\right) = 365$ kg/cm² < 450 × 1.6

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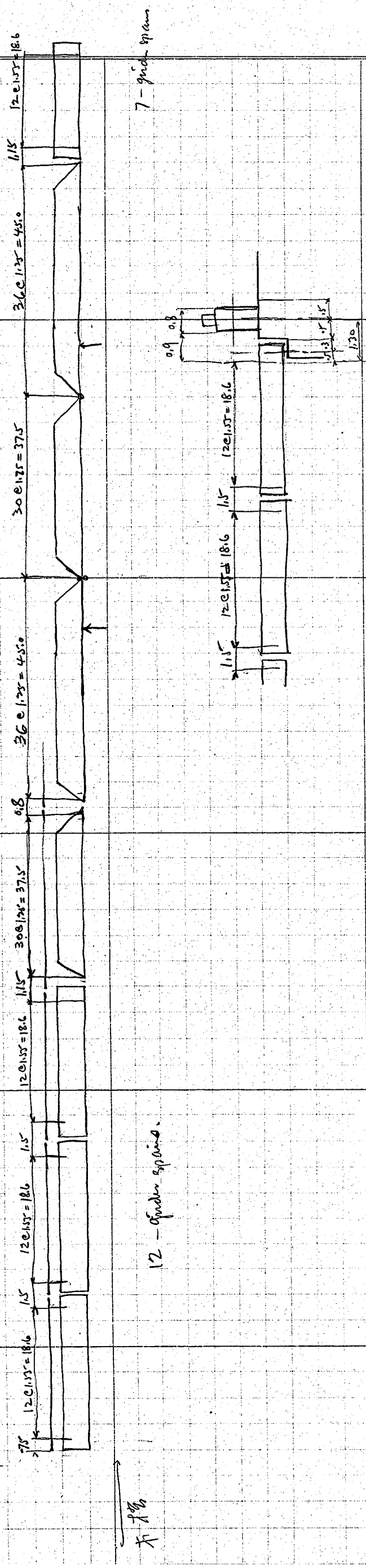
CALCULATIONS FOR

ホルタ橋



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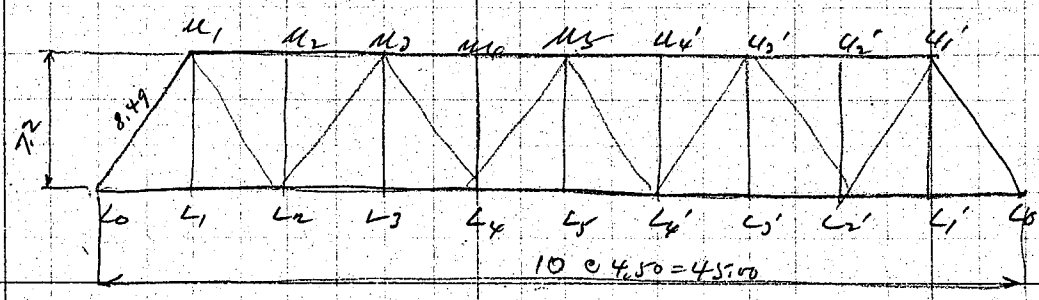
CALCULATIONS FOR

Ukigase-Bashi 45 m truss span.

I. beam stringer — span length 4.5 m
use I beam 300×150 @ 48.3 kg/m — girder span 1 本ずつ

Floor beams. 1/2 ワ ずつ

Main truss.
Dead load. 2170 × 4.5 = 9760 kg for one panel.
Live load. 1280 × 4.5 = 5760
Single con. say 5800



Diagonal length
$= \sqrt{7.20^2 + 4.50^2} = \frac{51.84}{\sqrt{72.09}} = \frac{51.84}{20.75} = 8.49$ m

$\sec\theta = \frac{8.49}{7.20} = 1.18$

$\tan\theta = \frac{4.50}{7.20} = 0.625$

Dead load stress :-
Dead panel load W_D = 9760 kg.
$W_D \sec\theta = 9760 \times 1.18 = 11500$ kg
$W_D \tan\theta = 9760 \times 0.625 = 6100$ "

Chord stress.

	$W_D \tan\theta$			
L_0-L_2	6100 × 4.5	= 27,500 kg	T	
U_1-U_3	" 8.0	48,800	C	
L_2-L_4	" 10.5	64,000	T	
U_3-U_5	" 12.0	73,200	C	
L_4-L_4'	" 12.5	76,300	T	

Live load stress.

	$W_L \tan\theta$			
L_0-L_2	3600 × 4.5	= 16,200	1560 × 0.9 × 1 = 1400	17,600 T
U_1-U_3	" 8.0	= 28,800	" 0.8 × 2 = 2500	31,300
L_2-L_4	" 10.5	= 37,800	0.7 × 3 = 3300	41,100 T
U_3-U_5	" 12.0	43,200	0.6 × 4 = 3700	46,900 C
L_4-L_4'	" 12.5	45,000	0.5 × 5 = 3900	48,900 T

Diagonal.

	$W_D \sec\theta$			$W_L \sec\theta$			
L_0-U_1	6800 × 4.5	30,600	C	2950 × 0.9	= 2700	C	33,300 C
U_1-L_2	" 3.6	24,500	T	" 0.8	2400	T	26,900 T
L_2-U_3	" 2.8	19,000	C	" 0.7	2100	C	21,100 C
U_3-L_4	" 2.1	14,300	T	" 0.6	1800	T	16,100 T
L_4-U_5	" 1.5	10,200	C	" 0.1	1500	C	11,700 C

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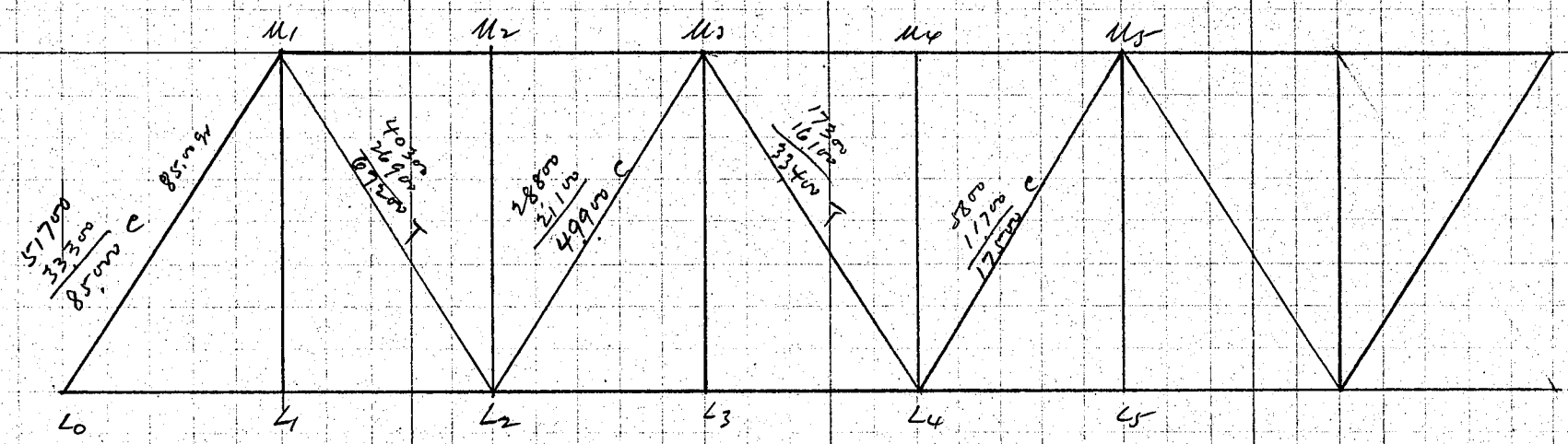
CALCULATIONS FOR

Akigase-Bashi 45 meter span
Stresses and sections of truss member

1 cover pl. 540.11 = 54.40
2 L 300×90×9 = 97.14
156.54 gr

DL. 48800 73200
LL. 31300 46900
80100 C SR 80.1 gr 120,100 C 120.1 gr



DL. 27,500 64000 76,300
LL. 17,600 41,100 48,900
45,100 T SR 37.6 net 105,100 T 87.7 net 125,200 T 104.3 net

2 Pls 100×10 × 10 = 38.00 − 8.8 = 29.20
2 Pls 300 × 10 = 60.00 − 13.20 = 46.80
2 Pls 200 × 10 = 40.00 − 6.60 = 33.4 ∴ 76.00
109.4 net

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