昭和八年六月

國道拾貳號線

岐阜縣木曽川橋設計々算書

（下部工事）

JIUN MASUDA
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~~JIJI~~ SHOWA BLDG, TOKYO

MADE BY J. Masuda DATE 8-5-2 FILE NO
CHECKED BY M. Jinda DATE 8-5-15 PAGE NO 1

CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu-Ken

The location of the new bridge selected 150 meters below the present bridge after careful study of general traffic and local condition and also approaches thereto. The total length of bridge is 462.4 meters between faces of parapet walls of both abutments or 460.0 meters about between front faces of the said abutments. The bridge is divided into 7 equal spans of 65.1 meters each after study of economic layout. The low steel of bridge at both abutments is 1.5 meters high above assumed highwater level and cambered .60 meter toward the center of bridge, and the bridge floor corresponding to the above.

The width of roadway 9.0 meters clear between curb lines and the whole width paved with granolithic concrete on reinforced concrete slab.

The handrails throughout the bridge are made of cast iron and the pedistals at entrance over both abutments are of cut stone and ornamental design.

From findings of borings all piers sunk to the depth 20. meters about below low water level where the firm soil of sand and gravel can be found.

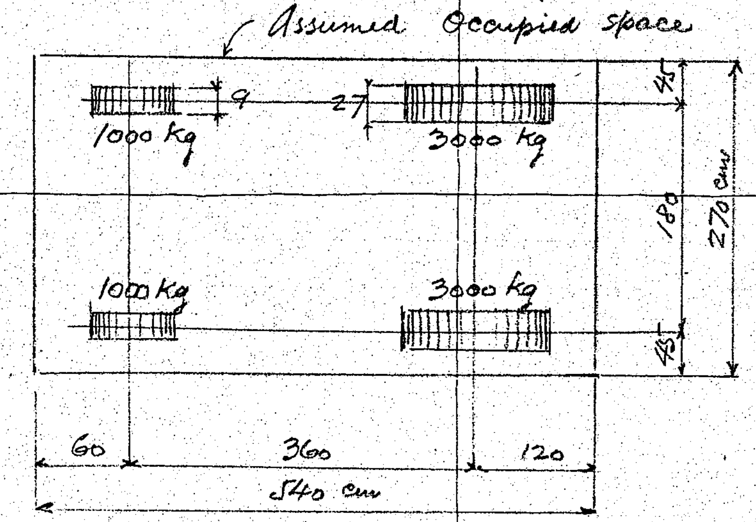
Both abutments built on sand soil in the elevation of river bed with timber piling foundation and part sheet piling to protect foundation against scoring.

Assumed loadings

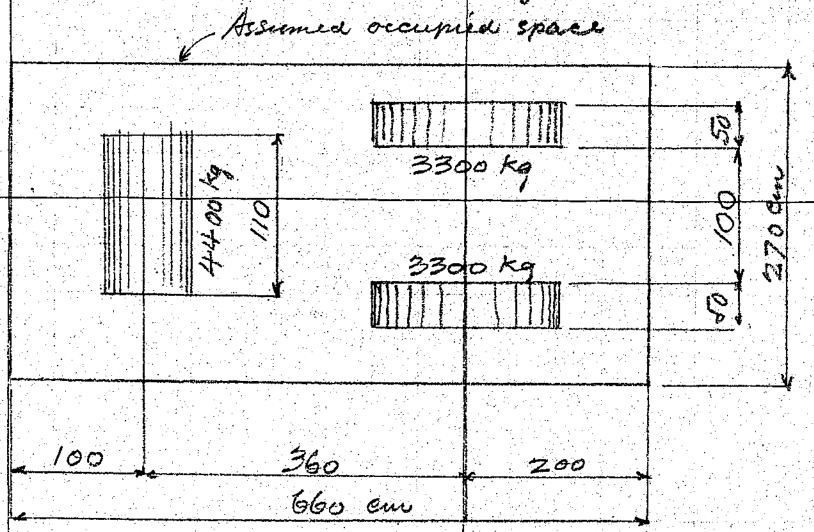
Uniform load on roadway $w = \frac{100.000}{170+l} \leq 500$ kgs/m²

where w = uniform load in kg per sq. meters
l = span length in meters

8 ton motor truck loading



11 ton Road Roller Loading



3 Lines of motor truck on roadway with occupied width of 270 cm each; unoccupied space around the motor trucks 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 meters
max impact 30%

No impact for roadroller and uniform live load.

Allowable Working Strength

Concrete

	for 1:2:4 Concrete		Assumed strength for 1:2½:5 Concrete
Direct compression	35	Kg/cm²	31.5
Fibre stress due to bending	45	"	40.5
Combined stresses due to direct and bending	35	"	31.5
Punching shear.	9	"	8.1
plain shear	4	"	3.6

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CALCULATIONS FOR

Design of Kiso-gawa Bashi for Gifu-Ken

Bearing value ---------- 45 kg/cm² 40.5 kg/cm²
Bond stress ---------- 6 " 5.4 "

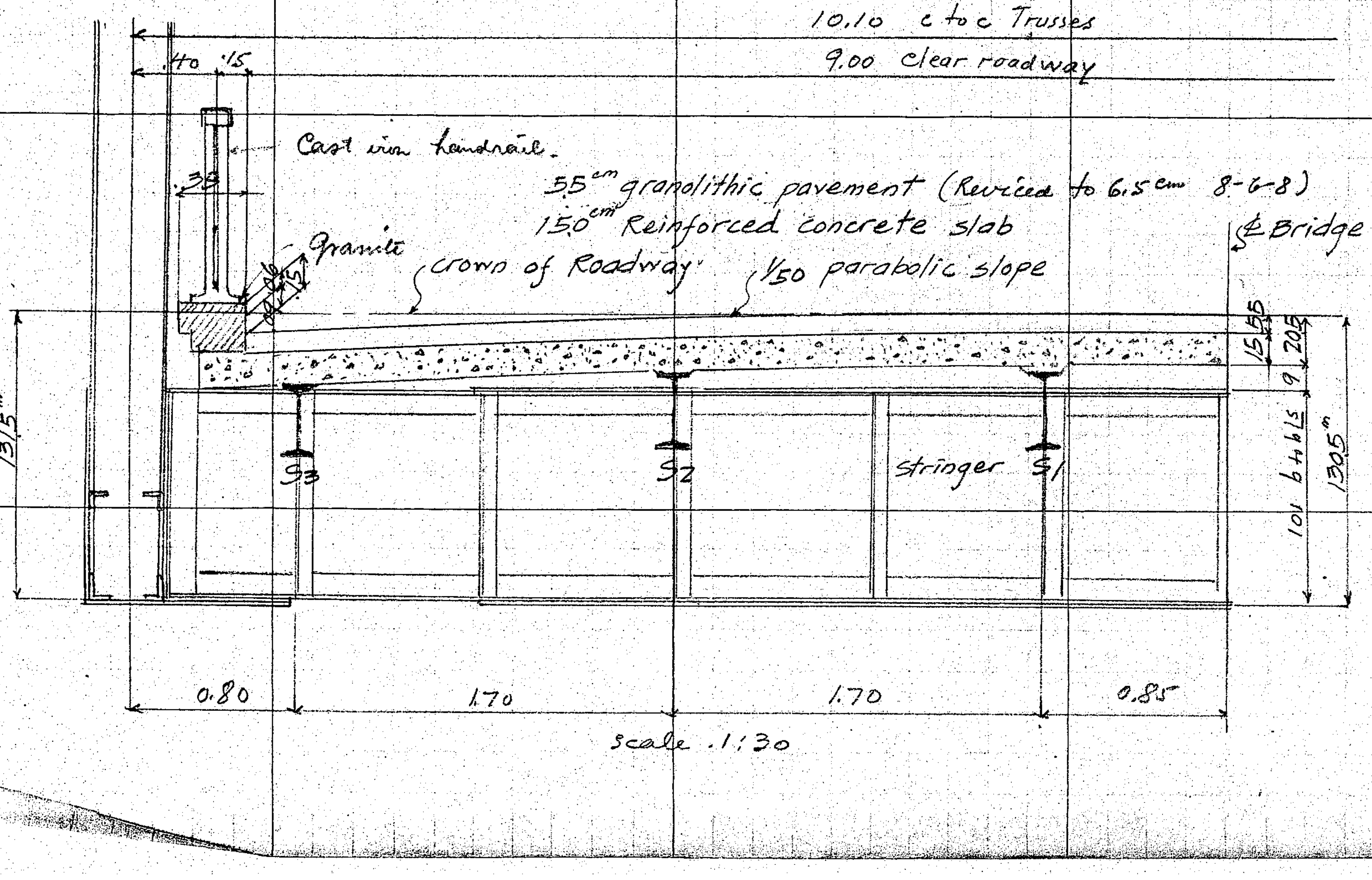
Reinforcing bars
Tension or compression ---------- 1200 kg/cm²
shearing strength ---------- 900 "

Structural steel
Tension net ---------- 1200 kg/cm²
Extreme fibre stress net ---------- 1200 "
Shear of web gross section ---------- 900 "
Compression member
$1500(1 - 0.0055\, l/r)$ not over ---------- 1000 "
where l = length of member in cm
r = least radius of gyration in cm
Compression flange of girder
$1200(1 - 0.012\, l/b)$ not over ---------- 1100 kg/cm²
where l = Unsupported length of flange in cm
b = width of flange in cm
Shear on shop driven rivets (machine driven) ---------- 850 kg/cm²
" " field " " and turned bolts (machine driven) -- 750 "
Shear on pin ---------- 900 kg/cm²
Bearing on shop driven rivets (machine) ---------- 1700 "
" " field " " ---------- 1500 "
" " pin ---------- 1800 "

Roller 45d kg/cm where d = diameter of roller 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 earthquake, the working strength increased 60%
Seismic acceleration for this locality assumed 3000 mm/sec² or k = 0.3

Cross section of bridge assumed as shown on sketch below.



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu ken.

Design of Piers.

Superimposed loads on pier.

Dead Load:-

Floor and pavement	9.00^m	@	500^{kg}	=	4,500	
Copings	2	@	220 "	=	440	
Handrails say	2	@	85 "	=	170	
Stringers	6	@	62 "	=	372	
Floor beams	2300^{kg} ÷		4.65	=	495	
Miscellaneous fittings say					13	
						5990

Top lateral bracings say				130	
Sway bracings "	4	@	2000 = 8000		
Struts "	5	@	800 = 4000		
Portal bracings "	2	@	1800 = 3600		
			15600 ÷ 65.1 =	240	
Bottom lateral bracings:				120	
					490
main trusses say	172000 ÷		65.1 =		2650
					9130 kg per lin meter

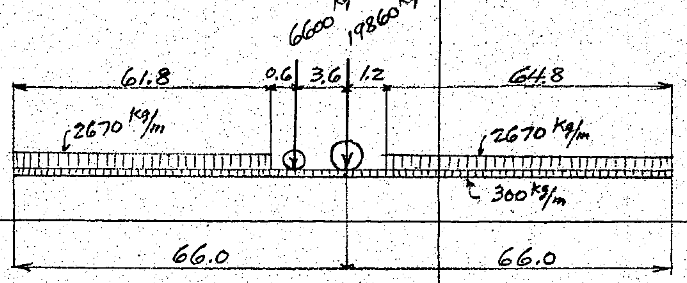
Dead Load on pier say

66.0 @ 9130 = 602000

Shoes + expansion joint etc say 5,000

607,000 kg on one pier.

Live Load:-



8 ton motor truck rear wheel concentration 3000

Impact coef. $= \frac{20}{60+132} = 10.4\%$ 310

3310 kg

Front wheel with impact say 3310 ÷ 3 = 1100 kg

Uniform live load on roadway

$w = \frac{100,000}{170+132} =$ 330 kg per sq. meter.

Unif. load on front and rear of truck

$= 9.0^m$ @ 330 = 2970 kg per lin meter of bridge

Unif. load on sides of truck

$= 9.0 - 3 \times 2.7 = 0.90^m$ @ 330 = 300 " "

Difference between above two unif. loads = 2670 kg/m

Wheel concentrations

Rear wheels 6 @ 3310 = 19860 kg

Front " 6 @ 1100 = 6600 "

Max. Live Load on pier.

Uniform load. $\frac{2670 \times 61.8^2}{2 \times 66.0} =$ 77200

" $\frac{2670 \times 64.8^2}{2 \times 66.0} =$ 85000

" $300 \times 66.0 =$ 19800

Rear wheels 19860

Front " $\frac{6600 \times 62.4}{66.0} =$ 6240

208,100 kg Call this 208,000 kg

Summary of superimposed loads on pier

Dead Load 607,000

Live Load 208,000

815,000 kg on one pier

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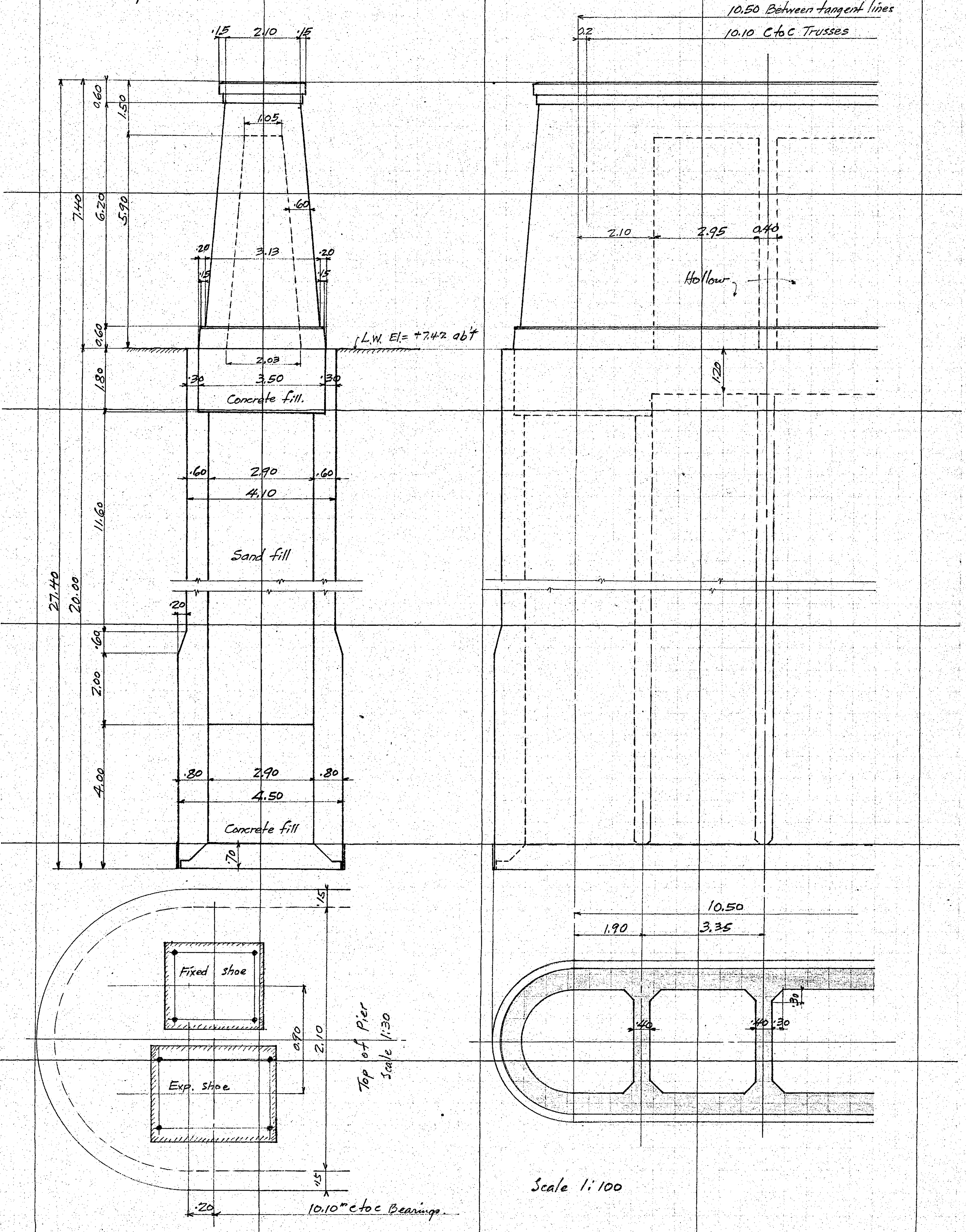
CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu ken.

Sketch of Pier no 3.



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CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu Ken.

Approximate weight and center of gravity of pier shaft.

	Dimensions		Volume	Lever arm	Moment abt bottom of shaft.
Coping	$2.40^{\phi} \times 0.35$	=	1.59	× 7.22 =	11.48
"	$2.40 \times 0.35 \times 10.5$	=	8.82	"	63.70
"	$2.26^{\phi} \times 0.25$	=	1.00	6.92	6.92
"	$2.26 \times 0.25 \times 10.5$	=	5.93	"	41.05
Shaft	$2.62^{\phi} \times 6.2$	=	33.45	3.31	110.75
"	$2.62 \times 6.2 \times 10.5$	=	170.60	3.50	597.00
"	$3.48^{\phi} \times 0.6$	=	5.71	0.30	1.71
"	$3.48 \times 0.6 \times 10.5$	=	21.92	0.30	6.58
Hollow, less	$1.54 \times 5.9 \times 5.9$	=	−53.60	2.66	−142.60
			195.42 m³	3.57 m	696.59

Weight of shaft = 195.42 @ 2400 = 469,000 kg

Approximate weight and center of gravity of well.

Top 1.8 m

	$4.10^{\phi} \times 1.8$ =	23.80	@ 2400 =	57,100	× 0.90 =	51,400
	$4.10 \times 1.8 \times 10.5$ =	77.50	" =	186,000	× 0.90 =	167,400
Hollow, less	$2.95 \times 2.9 \times 2 \times 0.6$ = −	10.26	" = −	24,600	× 0.30 =	− 7,400
fillets, add	$0.3 \times 0.3 \times 4 \times 0.6$ =	0.22	" =	500	0.30 =	200
Sand filling		10.04	@ 1700 =	17,100	0.30 =	5,100
				236,100 kg	0.92 m	216,700

next 11.6 m

Cross sectional area

$4.10^{\phi} - 2.90^{\phi}$ = 6.60
10.50×4.10 = 43.05
2.90×9.30 = − 26.97
$0.30 \times 0.30 \times 6$ = 0.54
23.22 sq. meter net.

weight per lin meter
Shell 23.22 @ 2400 = 55,700 kg
Sand fill 33.03 @ 1700 = 56,100 "
111,800 kg

weight of well 11.6 m @ 111,800 = 1,297,000 kg arm 5.8 m

next 2.6 m

$4.50^{\phi} - 2.90^{\phi}$ = 9.30
10.50×4.50 = 47.25
2.90×9.30 = − 26.97
$0.30 \times 0.30 \times 6$ = 0.54
30.12 sq. meter net

weight per lin meter
Shell 30.12 @ 2400 = 72,300
Sand fill 33.03 @ 1700 = 56100
128,400 kg

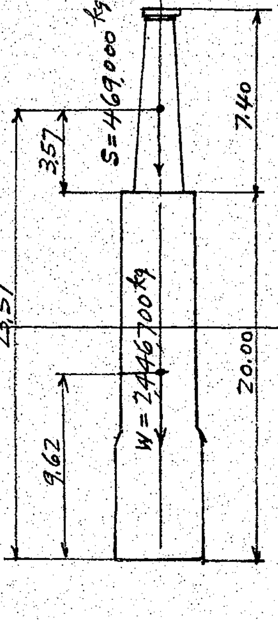
weight of well 2.6 m @ 128,400 = 334,000 kg arm say 1.3 m

Bottom 4.0 m

4.50^{ϕ} = 15.90
10.50×4.50 = 47.25
63.15 sq. meter net

weight per lin meter
Shell 30.12 @ 2400 = 72,300
Concrete fill 33.03 @ 2200 = 72,600
144,900 kg

weight of well 4.0 m @ 144900 = 579,600 kg arm 2.0 m



Weight and center of gravity of the whole well.

	weight		arm		
Top 1.8 m	236,100	×	19.12	=	4,520,000
next 11.6 m	1,297,000	×	12.40	=	16,080,000
" 2.6 m	334,000	×	5.30	=	1,770,000
Bottom 4.0 m	579,600	×	2.00	=	1,159,000
	2,446,700 kg		9.62 m		23,529,000

Total volume of concrete

Shaft			195.42
well	1.8		91.26
	11.6 @ 23.22	=	269.20
	2.6 @ 30.12	=	78.10
	4.0 @ 63.14	=	252.70
			886.68 cub. meters

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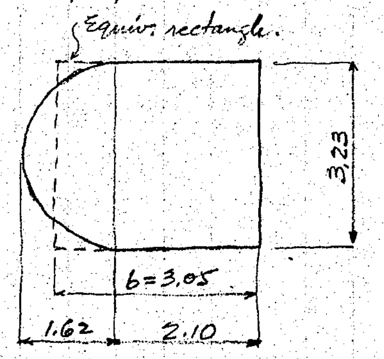
CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu ken.

Stresses of shaft at bottom section :-



Equivalent rectangular section of the same moment of inertia

Moment of inertia of the assumed section.

semicircle	$0.0491 \times 3.23^4 \div 2$	=	2.67
rectangle	$2.10 \times 3.23^3 \div 12$	=	5.90
			$8.57\ m^4$

Length of equivalent rectangle = b.

$$b \times 3.23^3 \div 12 = 8.57$$

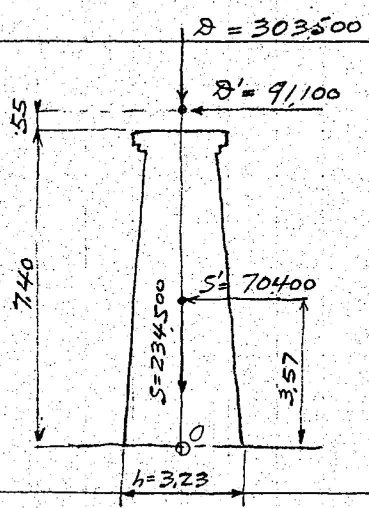
$$b = \frac{8.57 \times 12}{3.23^3} = 3.05 \text{ meters}$$

Dead Load on one half of Pier	=	303500 kg
Live Load " " "	=	104000
		407500
weight of shaft for " "	=	234500
		642000

$$\text{Average unit bearing pressure} = \frac{642000}{323 \times 305} = 6.5\ kg/cm^2$$

Seismic stresses. k assumed 0.300.

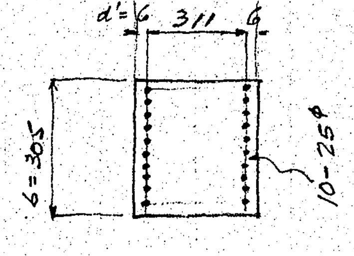
Taking moment about the centroid of the bottom area. 0



Loads	Hor. forces	Vert. forces		Lev. arms		Moments abt. 0
D		303500	×	0	=	0
D'	91100		×	7.95	=	724000
S		234500	×	0	=	0
S'	70400		×	3.57	=	251200
	161,500 kg	538,000 kg		ε=1.81 m		M=975,200 kgm

Try reinforcements 10- 25φ bars = 49.1 cm²

for both sides $A_0 = 2A_s = 2 \times 49.1 = 98.2\ cm^2$



$$\text{Steel ratio } p_0 = 2p = \frac{98.2}{305 \times 323} = 0.0010,$$

$$d'/h = 6/323 = 0.019, \quad \varepsilon/h = 181/323 = 0.560$$

From the prepaired diagrams of combined stresses, we have.

$$k = 0.280 \qquad L = 0.076$$

$$f_c = \frac{M}{Lbh^2} = \frac{975200 \times 100}{0.076 \times 305 \times 323^2} = 40.3\ kg/cm^2 \qquad < 35 \times 1.6 = 56\ kg/cm^2$$

$$f_s = nf_c\left(\frac{d}{kh} - 1\right) = 15 \times 40.3\left(\overset{2.51}{\frac{317}{.28 \times 323}} - 1\right) = 1515\ kg/cm^2 < 1200 \times 1.6 = 1920\ kg/cm^2$$

$$\text{Unit shear} = \frac{161500}{305 \times \frac{7}{8} \times 317} = 1.91\ kg/cm^2$$

$$\text{Unit bond} = \frac{161500}{10 \times 7.85 \times \frac{7}{8} \times 317} = 7.42 \text{ "} \qquad < 6 \times 1.6 = 9.6\ kg/cm^2$$

Assumed section is ample.

Use 22φ bars for the upper half of the shaft at the same spacing.

" 19φ bars for the curtain walls between both columns.

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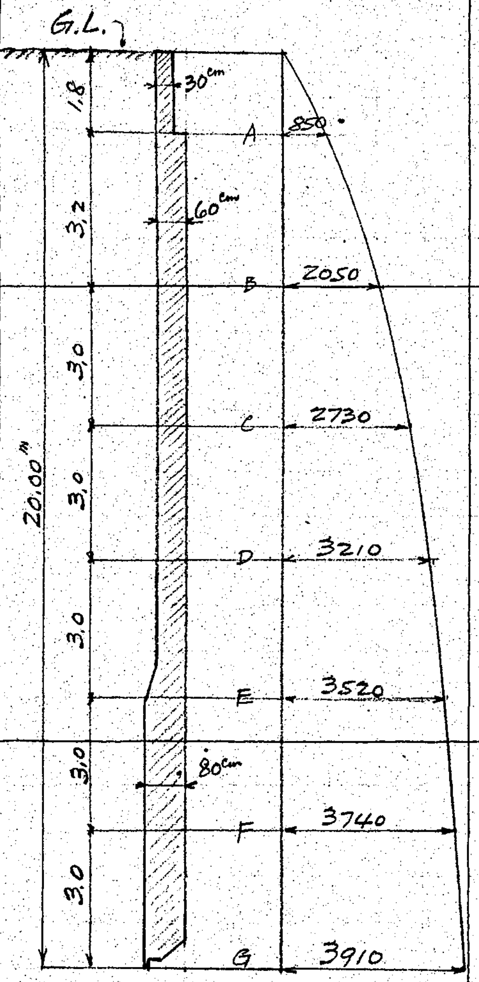
CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu ken.

Design of well.

Temporary earth pressure on well during well sinking execution. (Referred to Ketchum's Walls, Bins, and Grain Elevators, on page 120 and 121)



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

where V = Vertical unit pressure in kg/m² at depth y
L = Lateral 〃 〃 〃 〃 〃
w = weight of earth in kg/m³
$\mu = \tan\phi$ = Coefficient of friction of earth on earth
ϕ = angle of repose of earth.
b = the distance in meters that the earth breaks around the well
k = a constant $\doteqdot {}^{1-\sin\phi}/_{1+\sin\phi}$

Let us assume $w = 1600$ kg/m³, $b = 3.0$ meters, and $k = 1/3$

From the prepared diagram, we have the temporary pressures as follows:

Depth of earth y in meters	Temporary pressure on well in kg/m²
1.8	850
5.0	2050
8.0	2730
11.0	3210
14.0	3520
17.0	3740
20.0	3910

Bending moments on wall at several sections.

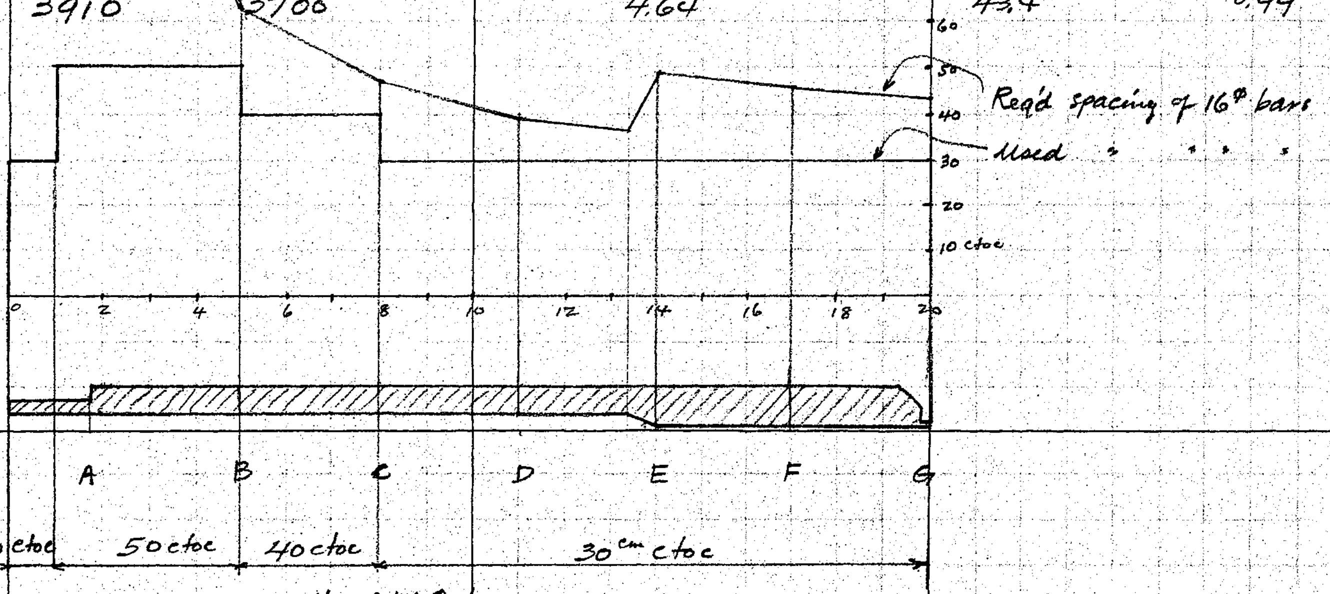
Span length of wall assumed 3.35 meters for straight walls, diameters of end rings 3.5 + 3.7 m

moment assumed $\pm\frac{1}{12}Ll^2 = \pm\frac{1}{12}\times L\times 3.35^2 = 0.935L$ for straight walls.

and $\pm\frac{1}{16}LD^2 = \pm\frac{1}{16}\times L\times 3.50^2 = 0.766L$ for end rings

or $\pm\frac{1}{16}LD^2 = \pm\frac{1}{16}\times L\times 3.70^2 = 0.856L$ 〃

		Earth pressure L	moment 0.935L	eff. depth of wall	steel area required	size of bars + spacings 12φ	16φ	Unit shear
At section	A	850 kg/cm²	795 kgm	56 cm	1.35 cm²	84.0	149.0	0.29 kg/cm²
	B	2050	1920	〃	3.26	34.7	61.7	0.70
	C	2730	2550	〃	4.34	26.1	46.4	0.93
	D	3210	3000	〃	5.10	22.2	39.5	1.10
	E	3520	3290	76	4.12		48.8	0.89
	F	3740	3500	〃	4.39		45.8	0.94
	G	3910	3700	〃	4.64		43.4	0.99



Spacing diagram of Horizontal reinforcements.

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CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu ken.

Stability of Pier.

At normal state:

Superimposed dead load = 607,000
" live load = 208000
816,000

Weight of shaft = 469,000
" " well = 2446,700
2915,700

Total load on bottom section = 3730,700 kg

Area of skin friction. Frictional resistance assumed at 1200 kg/m²

4.1^ϕ = 12.88
10.5×2 = 21.00
33.88 × 14.0 = 474.0

4.5^ϕ = 14.14
10.5×2 = 21.00
35.14 × 6.0 = 211.0
685.0 sq. meters

Frictional resistance = 685.0 @ 1200 = 822,000 kg

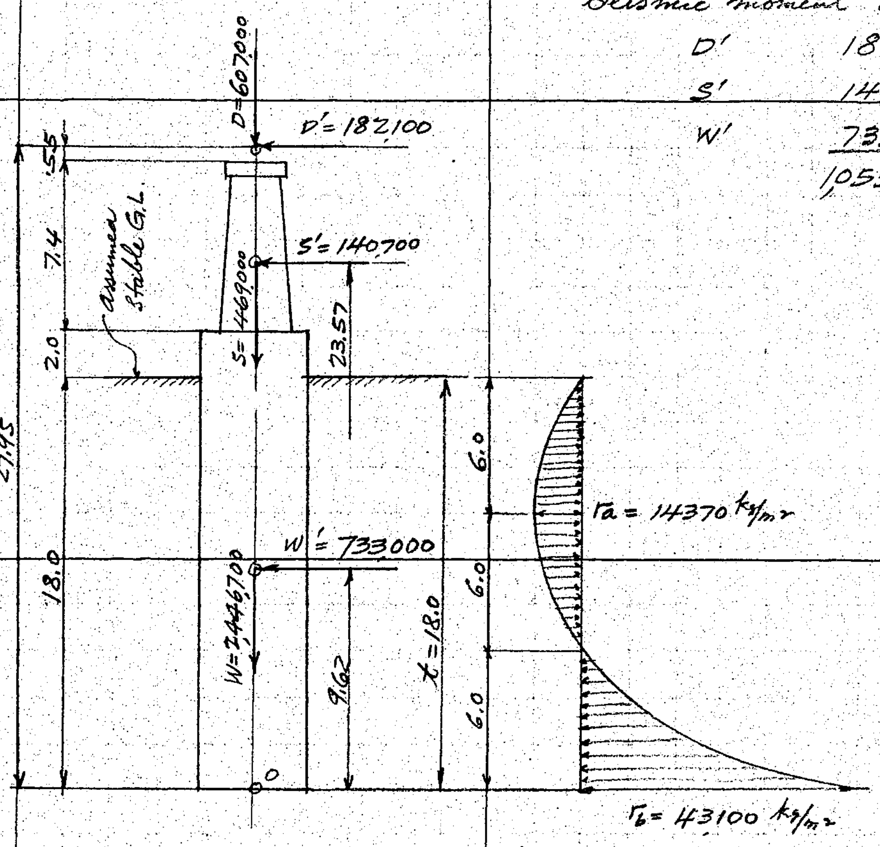
Total net load on bottom area 3730,700
− 822000
2,908,700 kg

Unit bearing pressure $= \frac{2,908,700}{63.15} =$ 46,100 kg/m² or (4.2 tons/□′)

Stability during earthquake. k assumed at 0.300

Seismic moment at center of bottom area. vert. load.

D′ 182,100 × 27.95 = 5,090,000 D 607,000
S′ 140,700 × 23.57 = 3,310,000 S 469,000
W′ 733,000 × 9.62 = 7,060,000 W 2446,700
1,055,800 kg M = 15460000 kgm 3522700 kg



Average width of well assumed 13.3 meters.

$r_b = \frac{12M}{t^2} = \frac{12 \times 15460000}{18^2} = 573000$ kg

for 1 meter strip

$r_b = \frac{573000}{13.3} = 43,100$ kg/m²

$r_a = \frac{r_b}{3} = \frac{43100}{3} = 14,370$ "

14370 × 13.3 = 191,000 kg

Passive pressure of earth.

$P = wx \frac{1+\sin\phi'}{1-\sin\phi'}$ where $\phi' = \phi - \tan^{-1} k$

ϕ = angle of repose of earth at normal state, assumed 30°

$\phi' = 30° - \tan^{-1} 0.300 = 13°20'$, $\sin\phi' = \sin 13°20' = 0.231$

$\frac{1+\sin\phi'}{1-\sin\phi'} = \frac{1.231}{0.769} = 1.60$

Passive earth pressure at r_a, $P_a = 1600 \times 6.0 \times 1.60 =$ 15370 kg/m² > 14370

" " r_b, $P_b = 1600 \times 18.0 \times 1.60 =$ 46,100 kg/m² > 43,100

unit bearing pressure on soil $= \frac{3522700}{63.15} =$ 55900 kg/m² (5.1 tons/□′)

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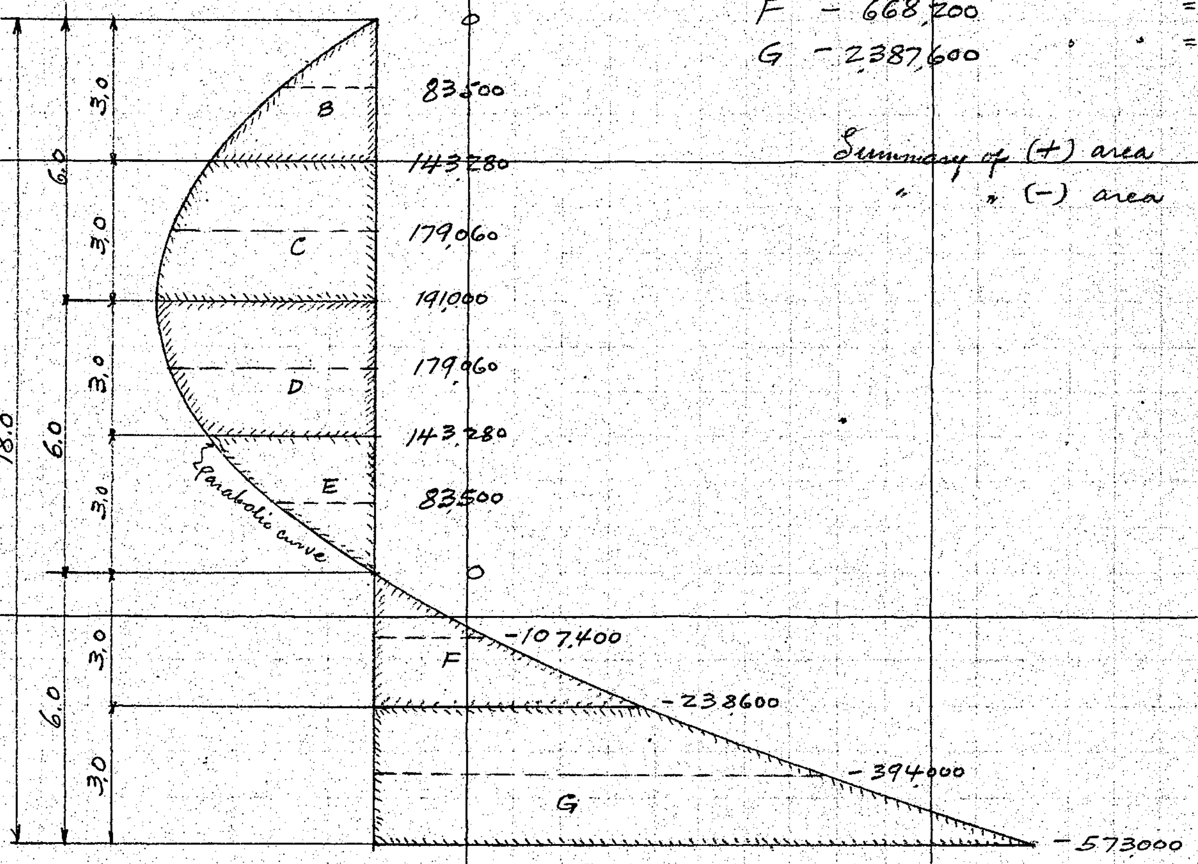
Design of Kisogawa Bashi for Gifu Ken.

Earth pressure on each section of well by Simpson's formula.

						C.G.	Arm from bottom
B.	477,280	×1.50÷3	=	238600	kg	1.00	16.00 m
C	1050,520	" "	=	525300	"	1.43	13.43
D	1050,520	" "	=	525300	"	1.57	10.57
E	477,280	" "	=	238600	"	2.00	8.00
F	−668,200	" "	=	−334000	"	1.00	4.00
G	−2387,600	" "	=	−1193800	"	1.30	1.30

Summary of (+) area 1527800 (check $\frac{2}{3} \times 12 \times 191000 = 1528000$) ✓

" " (−) area 1527800



Weight and seismic loads on each section. arm

Superimposed dead load D = 607000 kg, Seismic load D' = 182100 kg 7.95 m

weight of shaft S = 469000 " " " S' = 140700 " 3.57

Section A. P_A = 236100 " " P_A' = 70900 " 0.92

Section B. $P_B = 3.2 \times 111800$ = 358000 " " " P_B' = 107400 " 1.60

Section C $P_C = 3.0 \times 111800$ = 335400 " " " P_C' = 100600 " 1.50

Section D P_D = " = " " " P_D' = " " "

Section E P_E = " = " " " P_E' = " " "

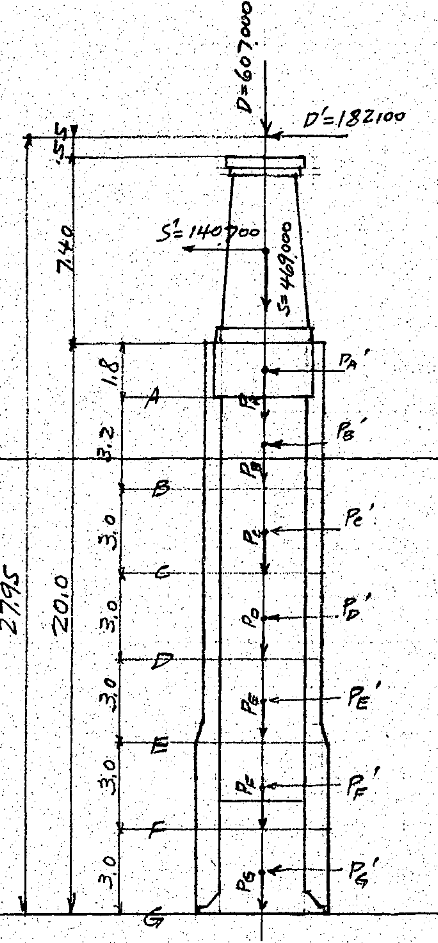
Section F P_F = 401700 " " " P_F' = 120500 " 1.46

$2.0 \times 128400 = 256800 \times 2.0 = 513600$

$1.0 \times 144900 = \underline{144900} \times \underline{0.5} = \underline{72500}$

401,700 1.46 586100

Section G $P_G = 3.0 \times 144900$ = 434700 kg " " P_G' = 130500 " 1.50



Seismic earth pressures on each section arms

Section A E_A = 0

Section B E_B = 238600 kg 1.00 m

" C E_C = 525300 1.43

" D E_D = 525300 1.57

" E E_E = 238600 2.00

" F E_F = −334000 1.00

" G E_G = −1193800 1.30

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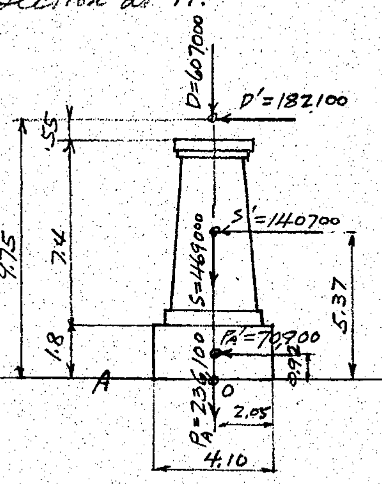
CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu Ken.

Moments at several sections of well during earthquake.

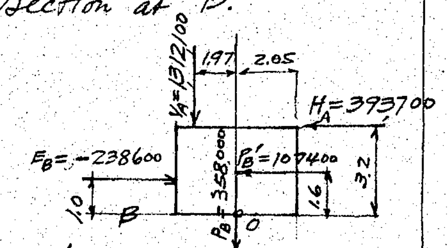
Section at A.



Taking moments about point O.

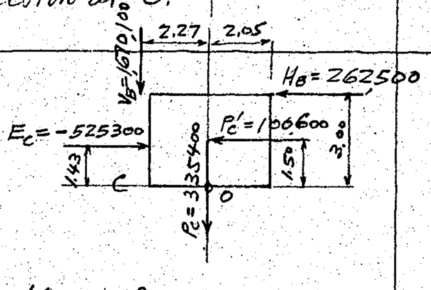
Loads	Horizontal forces	Vertical forces		Lever arms		Moments.
D		607,000	×	0	=	0
D'	182,100		×	9.75	=	1,775,000
S		469,000	×	0	=	0
S'	140,700		×	5.37	=	755,000
P_A		236,100	×	0	=	0
P_A'	70,900		×	0.92	=	65,200
	H_A = 393,700 kg	V_A = 1,312,100 kg		1.97 m		M_A = 2595200 kgm

Section at B.



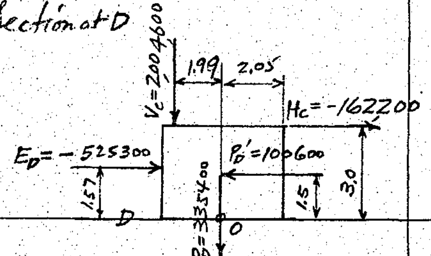
V_A		1312100	×	1.97	=	2,595,200
H_A	393,700		×	3.20	=	1,260,000
P_B		358,000	×	0	=	0
P_B'	107,400		×	1.60	=	171,800
E_b	−238600		×	1.00	=	−238600
	H_B = 262,500 kg	V_B = 1,670,100 kg		2.27 m		M_B = 3,788,400 kgm

Section at C.



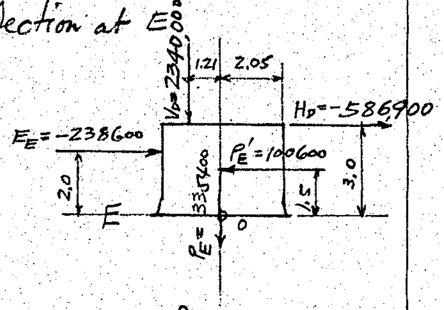
V_B		1,670,100	×	2.27	=	3,788,400
H_B	262,500		×	3.00	=	787,000
P_C		334,500	×	0	=	0
P_C'	100,600		×	1.50	=	150,900
E_C	−525,300		×	1.43	=	−751,000
	H_C = −162,200	V_C = 2,004,600		1.99		M_C = 3,975,300

Section at D



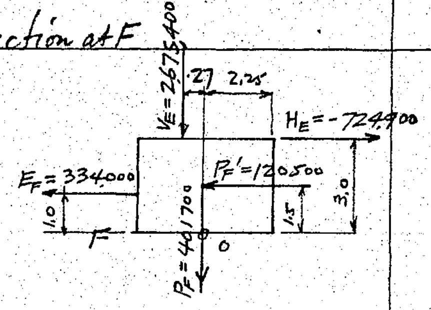
V_C		2,004,600	×	1.99	=	3,975,300
H_C	−162,200		×	3.00	=	−487,000
P_D		335,400	×	0	=	0
P_D'	100,600		×	1.50	=	150,900
E_D	−525,300		×	1.57	=	−824,000
	H_D = −586,900	V_D = 2,340,000		1.21		M_D = 2,815,200

Section at E



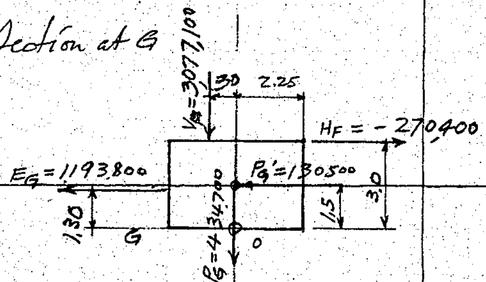
V_D		2,340,000	×	1.21	=	2,815,200
H_D	−586,900		×	3.00	=	−1,760,000
P_E		335,400	×	0	=	0
P_E'	100600		×	1.50	=	150,900
E_E	−238600		×	2.00	=	−477,200
	H_E = −724,900	V_E = 2,675,400		0.27		M_E = 728,900

Section at F



V_E		2,675,400	×	0.27	=	728,900
H_E	−724,900		×	3.00	=	−2,170,500
P_F		401,700	×	0	=	0
P_F'	120,500		×	1.50	=	180,800
E_F	334,000		×	1.00	=	334,000
	H_F = −270,400	V_F = 3,077,100		0.30		M_F = −926,800

Section at G



V_F		3,077,100	×	0.30	=	−926,800
H_F	−270,400		×	3.00	=	−810,000
P_G		434,700	×	0	=	0
P_G'	130,500		×	1.50	=	195,800
E_G	1,193,800		×	1.30	=	1,540,000
	H_G = 1,053,900	V_G = 3,511,800				M_G = −1,000 ≒ 0

M_G must theoritically be zero.

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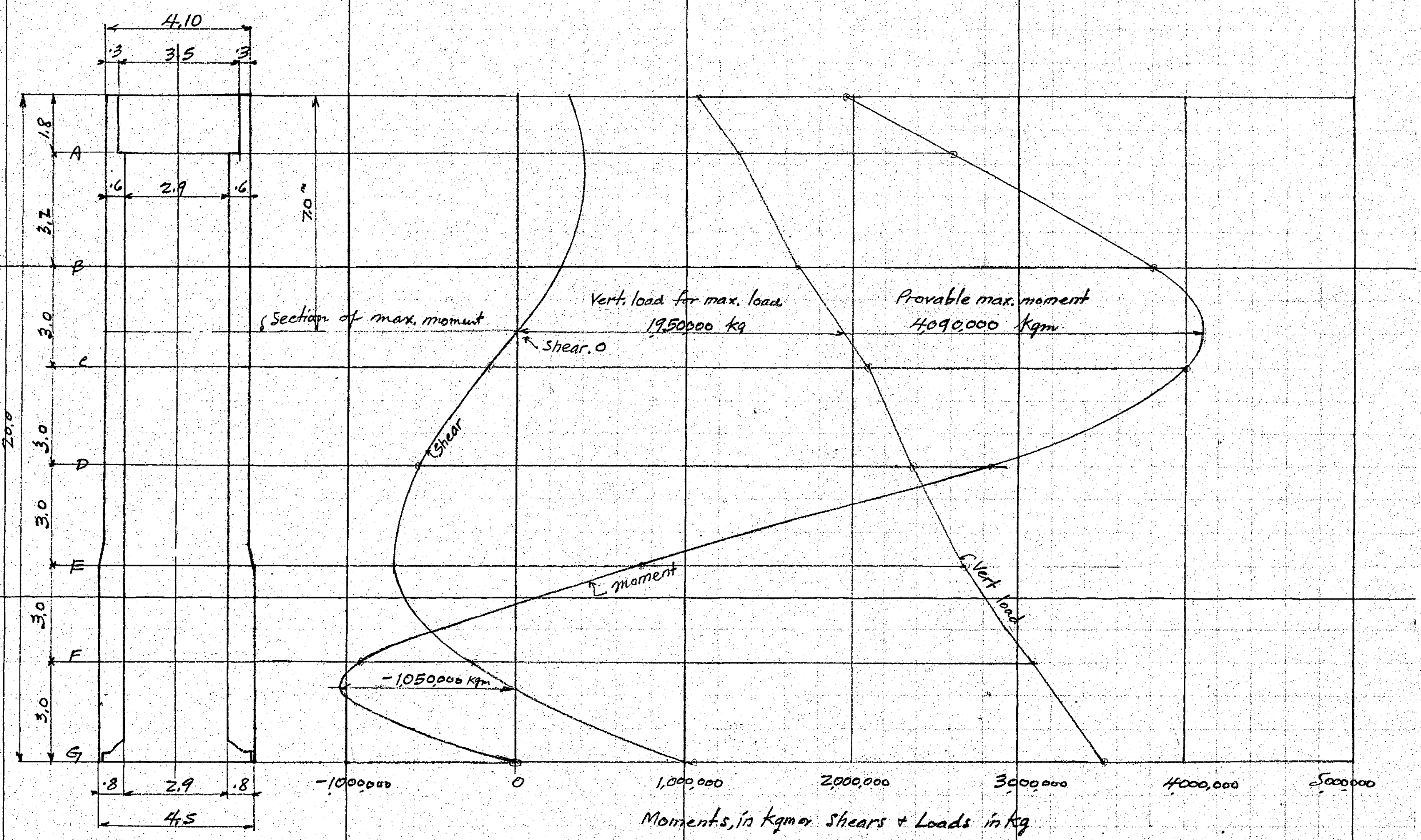
CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu Ken.

Moments, shears, and direct load diagram of well.



Vertical reinforcements in well.

max. moment in well scaled in the above diagram = 4090,000 kgm
Corresponding direct load on well = 1950,000 kg
sectional area of well = 23.22 sq. meter (see page 5)

Direct stress on concrete = $\frac{1950000}{23.22 \times 100^2}$ = 8.40. kg/cm² C

Bending stress. Lever arm 3.50m, Eff. width of well say = 13.3 meters

Stress on wall = $\frac{4090000}{3.5 \times 13.3}$ = 88,000 kg C or T per lin meter of side walls.

unit stress = $\frac{88000}{100 \times 60}$ = 14.67 kg/cm² T or C.

Combined stresses

Direct stress	8.40 C	or	8.40 C
Moment stress	14.67 C	or	14.67 T
	23.07 kg/cm² C	or	6.27 kg/cm² T

max. compressive stress on reinforcement = 15 × 23.07 = 346 kg/cm²

Vertical reinforcements required = $\frac{6.27 \times 60 \times 100}{1200 \times 1.6}$ = 19.60 cm² per meter of wall.

Use 25mmφ bars at 45 cm ctoc on both sides of wall. = 4.909 ÷ 45 = 2180 cm²

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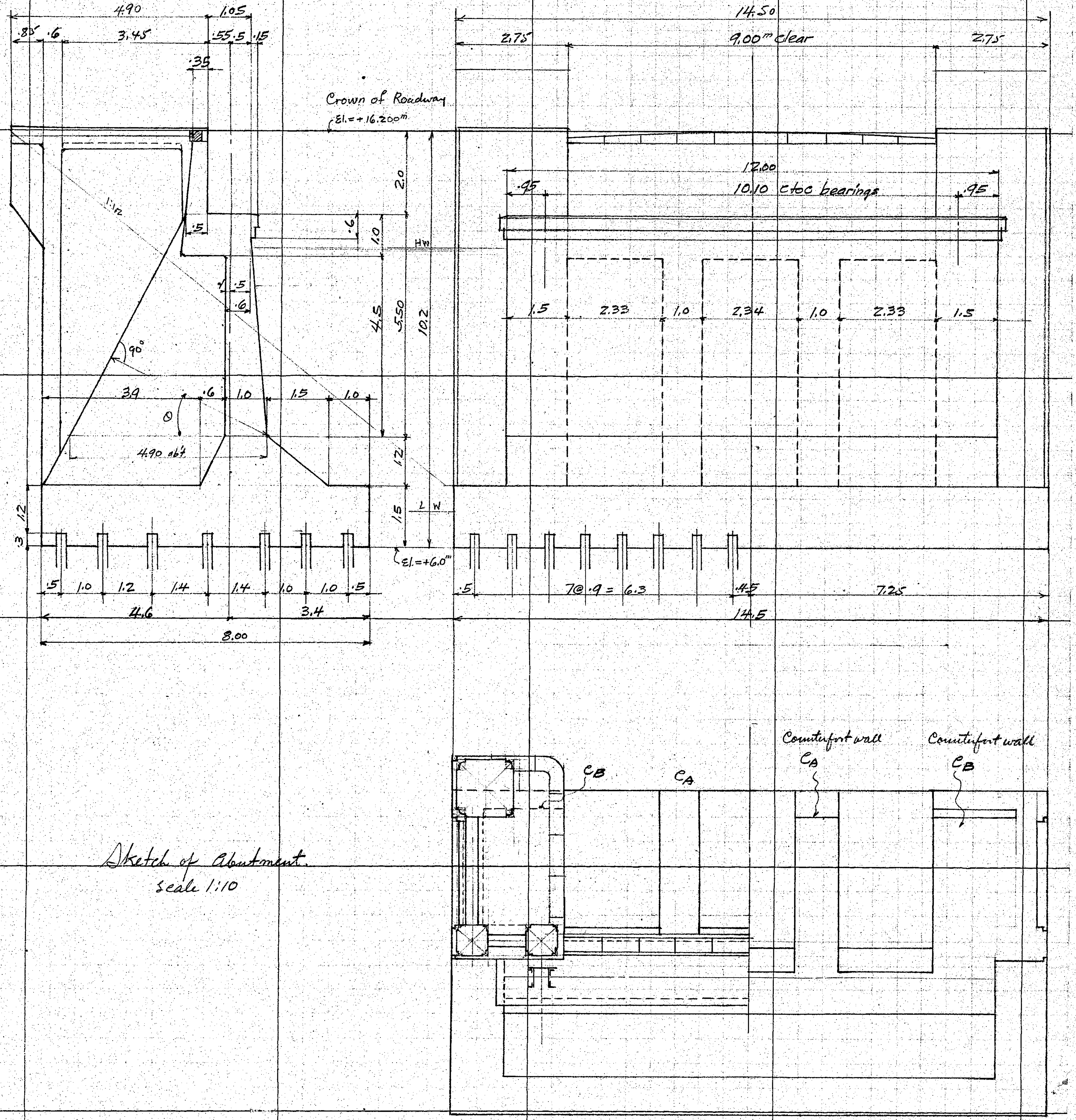
CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu ken.

Design of Abutment.

General construction of abutment assumed as shown on sketch below;



Sketch of Abutment.
scale 1:10

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Design of Kisogawa Bashi for Gifu-ken.

Superimposed Loads on Abutment.

Dead Load:- Same as for pier (see on page 3)

Dead Load per lin meter of bridge = 9130 kg.

Dead Load on abutment say 33.0 @ 9130 = 301000

Shoes + exp. jt. say 3000

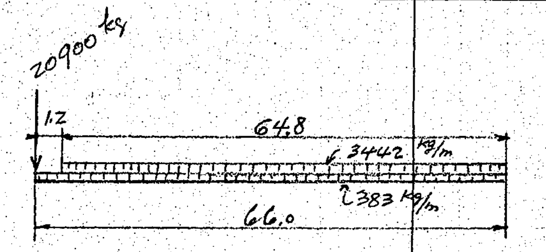
D = 304000 kg.

Seismic load for k = 0.300 D' = 304000 × .3 = 91200 "

Live Load :- 8 ton motor truck rear wheel concentration = 3000

Impact coef. $= \frac{20}{60+66} = 15.9\%$ = 480

3480 kg



Front wheel with impact say ⅓ × 3480 = 1160 "

Wheel concentrations

Rear wheels 6 @ 3480 = 20900 kg

Front " 6 @ 1160 = 6960 "

Uniform load on roadway

$w = \frac{100000}{170+66} = 425$ kg per sq. meter.

Unif. load on rear of truck

= 9.0 @ 425 = 3825 kg per lin meter

Unif. load on sides of trucks

= 9.0 − 3 @ 2.7 = 0.90 m @ 425 = 383 " "

Difference between above two unif. loads = 3442 kg/lin m.

Max. load on abutment.

Uniform load. $\frac{3442 \times 64.8^2}{2 \times 66}$ = 109500

" 383 × 66 ÷ 2 = 12600

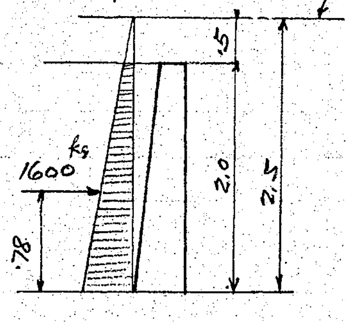
Rear wheels of trucks = 20900

L = 143000 kg

Summary of superimposed loads on abutment.

		for one abutment	for one-half of abutment
Dead Load	ΣD =	304000	D = 152000
Live Load	ΣL =	143000	L = 71500
Total load.	ΣP =	447,000 kg	P = 223500 kg

Design of Parapet wall



Assumed surcharge due to live load.

Earth pressure at normal state

⅓ × 1600 × .5 = 267

⅓ × 1600 × 2.5 = 1333

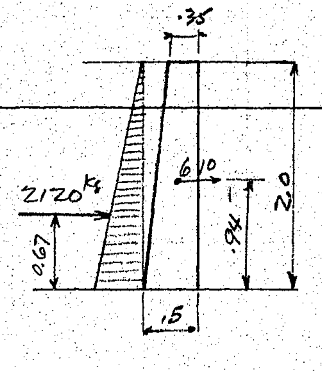
1600 ÷ 2 = 800 kg/m² average

800 × 2.0 = 1600 kg per meter strip of wall.

Moment at bottom of wall.

1600 × 0.78 = 1,250 kgm per meter strip

Shear 1,600 kg "



Earth pressure during earthquake. k = 0.300

$0.662 \times \frac{1600 \times 2^2}{2}$ = 2120 kg per meter strip

Weight of wall

$\frac{.35+.5}{2} \times 2.0 = 0.85 @ 2400 =$ 2040 kg Seismic force = 2040 × .3 = 610 kg

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Design of Kisogawa Bashi for Gifu ken.

Moment during earthquake.

Earth pressure $2120 \times 0.67 = 1420$

wall $610 \times 0.94 = \underline{574}$

1.994 kgm per meter strip ($\div 1.6 = 1245$)

Shear $2120 + 610 = 2730$ kg.

moment at normal state governs the section.

Effective depth required for $f_c = 40.5$ kg/cm² and $f_s = 1200$ kg/cm²

$d = \sqrt{\frac{M}{bR}}$ where R = a constant $= 6.04$, $b = 100$ cm

$d = \sqrt{\frac{1250 \times 100}{100 \times 6.04}} = 14.4$ cm Use 47 cm eff. depth with 3 cm insulation

Steel area required $= \frac{1250 \times 100}{1200 \times \frac{7}{8} \times 47} = 2.54$ cm² per meter strip.

Unit shear at normal state $= \frac{1600}{100 \times \frac{7}{8} \times 47} = 0.39$ kg/cm²

" during earthquake $= \frac{2730}{100 \times \frac{7}{8} \times 47} = 0.66$ "

Use 12 mm⌀ bars at 30 cm c to c on rear side
12 " " " 60 " " front side

Design of wingwall. Span length 3.00 m assumed

Section at 3 m below top.

Earth pressure at normal state. surcharge of LL assumed 50 cm of earth.

$\frac{1}{3} \times 1600 \times 3.5 = 1870$ kg/m²

moment $= \frac{1}{10} \times 3.00^2 \times 1870 = 1,680$ kgm per meter strip

Shear $= \frac{1}{2} \times 3.00 \times 1870 = 2,800$ kg "

Earth pressure during earthquake

$0.662 \times 1600 \times 3.0 =$ 3180

weight of wall

$0.4 @ 2400 = 960$ kg seismic force $960 \times 0.3 = \underline{290}$ kg/m²

3,470 "

moment $= \frac{1}{10} \times 3.00^2 \times 3,470 = 3,120$ kgm per meter strip

Shear $= \frac{1}{2} \times 3.00 \times 3470 = 5,200$ kg " "

Seismic stresses govern the section of wall.

Eff. depth required $= \sqrt{\frac{3120 \times 100}{100 \times 6.04 \times 1.60}} = 18.0$ cm

Use 37 cm eff. depth with 3 cm insulation

Steel area required $= \frac{3120 \times 100}{1200 \times 1.6 \times \frac{7}{8} \times 37} = 5.02$ cm² per meter strip

Use 16 mm⌀ bars at 40 cm c to c = 5.03 cm² " on both sides.

End column of wing wall. Height of column say 8.0 meters.

Dead Load on column.

Light pedestal

	$1.4 \times 1.4 \times 1.5 =$	2.94	(arm × .75)				
	$1.1 \times 1.1 \times 1.6 =$	1.94	(" 2.30)		1.50 m arm + 8.0 = 9.5 m		
	$.5 \times .5 \times 1.2 =$	$\underline{.30}$	(" 3.70 average)				
		5.18 @ 2200	=	11400 kg	arm × 9.50 =	108300
Handrail	$.3 \times .9 \times 1.3 =$	0.35 @ 2600	=	$\underline{900}$	× 8.45	7600
wall	$.4 \times 1.5 \times 8.0 =$	4.80 @ 2400	=	11520	× 4.00	46100
"	$.6 \times 2.0 \times 8.0 =$	9.60 @ 2400	=	23000	× 4.00	92000
"	$.85 \times 1.7 \times 2.5 =$	3.61 @ 2400	=	8670	× 6.80	58900
slab	$2.5 \times 2.75 \times 0.4 =$	$\underline{2.75}$ @ 石井 2500	=	$\underline{6870}$	× 7.80	$\underline{53600}$
				62360	5.88 m	366500
		call this		62400 kg		

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CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu Ken

Earth pressure on wall. (Earth pressure on the outside of wall neglected)

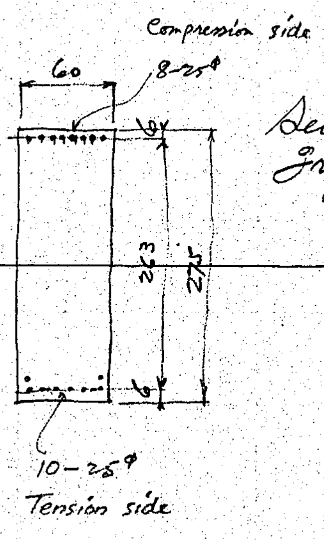
Depth of earth	Earth pressure at normal state 1/3×1600×h×width				Earth pressure during earth quake 0.662×1600×h×width				Lever arm say (approx)	
0	0	× 2.85m	=	0	0	× 2.85	=	0		
1	530	〃	=	1510 kg	1060	〃		3020 kg	× 7	= 21,100
2	1070	〃	=	3050	2120	〃		6040	6	36,200
3	1600	1.8	=	2880	3180	1.8		5720	5	28,600
4	2130	1.5	=	3200	4240	1.5		6360	4	25,400
5	2670	1.3	=	3470	5300	1.3		6890	3	20,700
6	3200	1.0	=	3200	6360	1.0		6360	2	12,700
7	3730	0.8	=	2980	7420	0.8		5930	1	5,900
8	4270	× 0.5×½	=	½×2140	8480	0.5×½	=	½×4240	0	
				21360 kg				42440 kg	3.55 m	150600

Moment at normal state = 21360 × 3.55 = 75800 kgm   eccentricity $\varepsilon = 1.22^m$
vertical load = 62400 kg
Shear = 21360 kg

moment during earth quake = 42440 × 3.55 = 150600   eccentricity $\varepsilon = 4.18^m$
Own weight 62400 × 0.3 × 5.88 = 110,000
260,600 kgm

vertical load. 62400 kg

Shear earth pressure 42400
own weight. 62400 × .3 = 18700
61,100 kg



Seismic stresses govern the section. (By inspection).
Try reinforcements 8-25mmφ bars on both sides = 78.5 cm²

$$p_0 = 2p = \frac{78.5}{60 \times 275} = 0.00475, \quad d'/h = 6/275 = 0.022, \quad \varepsilon/h = 418/275 = 1.52$$

From the prepared diagrams of combined stresses.
K = 0.268, L = 0.1162

d'/h	K	L
.022	.268	.1162
.05	.265	.1095
.10	.260	.0975

$$f_c = \frac{M}{Lbh^2} = \frac{260600 \times 100}{0.1162 \times 60 \times 275^2} = 49.5 \text{ kg/cm}^2 < 31.5 \times 1.6 = 50.5$$

$$f_s = nf_c\left(\frac{d}{Kh} - 1\right) = 15 \times 49.5 \left(\frac{269}{.268 \times 275} - 1\right) = 1967 \text{ kg/cm}^2$$ (2.65) Over stress.

add 2-25φ bars near the tension side edge. Total steel area on tension side 10 @ 4.909 = 49.09 cm²
Approximate steel stress

$$f_s = 1967 \times \frac{8}{10} = 1573 \text{ kg/cm}^2 < 1200 \times 1.6 = 1920$$

$$\text{Unit shear} = \frac{61100}{60 \times \frac{7}{8} \times 269} = 4.33 \text{ kg/cm}^2 < 3.6 \times 1.6 = 5.75$$

$$\text{Unit bond} = \frac{61100}{78.54 \times \frac{7}{8} \times 269} = 3.31 \text{ kg/cm}^2 < 5.4 \times 1.6 = 8.65$$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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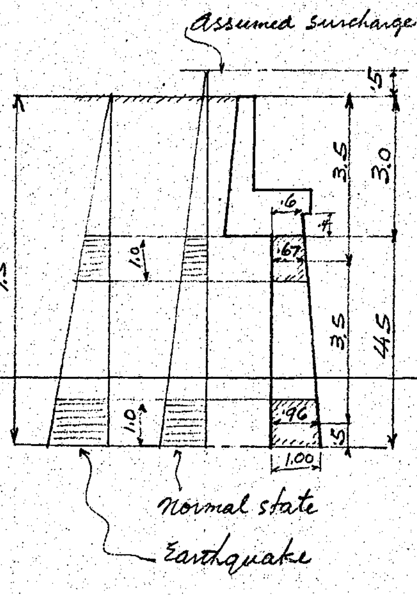
CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu Ken.

Design of Curtain wall.



Span length assumed 3.34 meters.

Stresses at normal state.

Earth pressure on top 1 meter strip

$1/3 \times 1600 \times 4.0 = 2135$ kg/m²

moment $= 1/10 \times 2135 \times 3.34^2 = 2370$ kgm

shear $= 1/2 \times 2135 \times 3.34 = 3565$ kg

Earthpressure on bottom 1 meter strip.

$1/3 \times 1600 \times 7.5 = 4000$ kg/m²

moment $= 1/10 \times 4000 \times 3.34^2 = 4460$ kgm

shear $= 1/2 \times 4000 \times 3.34 = 6680$ kg

Stresses during earthquake.

Earth pressure on top 1 meter strip

$0.662 \times 1600 \times 3.5 = 3710$

Seismic force on top 1 meter strip.

$0.67 @ 2400 \times 0.30 = \underline{480}$

4190 kg/m²

moment $= 1/10 \times 4190 \times 3.34^2 = 4680$ kgm

shear $= 1/2 \times 4190 \times 3.34 = 7000$ kg

Earth pressure on bottom 1m strip

$0.662 \times 1600 \times 7.0 = 7410$

seismic force on bottom 1m strip

$0.96 \times 2400 \times 0.30 = \underline{690}$

8100 kg/m²

moment $= 1/10 \times 8100 \times 3.34^2 = 9030$ kgm

shear $= 1/2 \times 8100 \times 3.34 = 13530$ kg

Seismic stresses govern the section of the curtain wall.

Sections.

Top 1 meter strip.

Eff. depth required $= \sqrt{\frac{4680 \times 100}{100 \times 6.04 \times 1.6}} = 22.0$ cm

Use 63cm eff. depth with 4cm insulation

Steel area reqd. $= \frac{4680 \times 100}{1920 \times \frac{7}{8} \times 63} = 4.42$ cm²

Use 16φ bars 40cm ctoc = 5.03 cm²

Unit shear $= \frac{7000}{100 \times \frac{7}{8} \times 63} = 1.27$ kg/cm²

Bottom 1 meter strip.

Eff. depth required $= \sqrt{\frac{9030 \times 100}{100 \times 6.04 \times 1.6}} = 30.6$ cm

Use 92cm eff. depth with 4cm insulation.

Steel area required $= \frac{9030 \times 100}{1920 \times \frac{7}{8} \times 92} = 5.85$ cm²

Use 16φ bars 30cm ctoc = 6.70 cm²

Unit shear $= \frac{13530}{100 \times \frac{7}{8} \times 92} = 1.68$ kg/cm²

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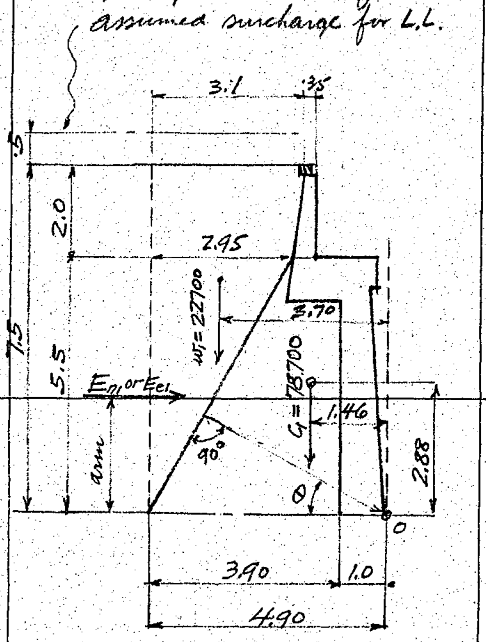
CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu-Ken

Design of Counterfort wall near center. CA



weight and center of gravity of counterfortwall CA, width 3.34 meters.

				arm	Vert m.	arm	Hor. m.
Granite	0.30×0.30×3.34 =	0.30 @ 2600 =	780 kg	7.35	5,730	1.70	1,330
parapet wall	0.43×1.70×3.34 =	2.44 @ 2400 =	5850	6.30	36850	1.67	9760
top strut	1.00×1.60×3.34 =	5.34 @ 〃 =	12,820	4.99	63,900	1.20	15380
Coping	0.12×0.60×3.34 =	0.24 @ 〃 =	580	5.25	3,050	0.34	200
Curtain wall	0.82×4.50×3.34 =	12.32 @ 〃 =	29,550	2.08	61,500	0.58	17,150
Counterfort	2.70×1.00×4.50 =	12.15 @ 〃 =	29,150	1.91	55620	2.44	71100
	granite	0.30	78,730 kg	2.88 m	226650	1.46 m	114920

Concrete 32.49 m³ Call this $C_1=78,700$ kg

weight and center of gravity of earth on counterfort wall.

3.03×1.0×2.00 =	6.06 @ 1600 =	9,700	3.38	32800
1.48×1.0×5.50 =	8.14 @ 〃 =	13,000	3.92	51000
	w_1 =	22,700 kg	3.70 m	83800

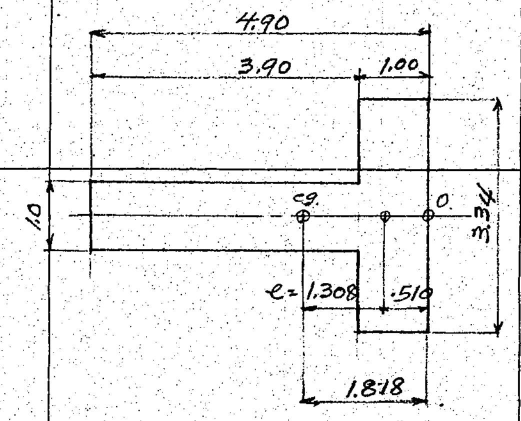
Earth pressure at normal state.

1/3×1600×0.50 = 267
1/3×1600×8.00 = 4267
4534÷2 = 2267 kg/m² average.

Earth pressure E_{n1} = 2267×7.5×3.34 = 56,800 kg arm 2.59 m
Earth pressure on frontside neglected.

Earthpressure during earthquake.
$0.662\times1600\times7.5^2\div2\times3.34=E_{e1}=$ 99,500 kg arm 2.50 m

Stresses at normal state.



Taking moment about toe 0.

Loads	Hor. forces	Vert. forces		Lever arms		Moment about 0.
C1		78,700	×	1.46	=	114,900
w1		22,700	×	3.70	=	84,000
E_{n1}	−56800		×	2.59	=	−147,200
	−56,800 kg	101,400 kg		0.510 m		51,700

Center of gravity of bottom section.

1.00×3.34 = 3.34 × 0.50 = 1.67
1.00×3.90 = 3.90 × 2.95 = 11.50
7.24 m² 1.818 m 13.17

Eccentricity e = 1.818 − 0.510 = 1.308 m
moment at bottom section = 101,400×1.308 = 132,700 kgm
Shear = 56,800 kg

Stresses during earthquake.

Taking moment about toe 0. Seismic force toward river side.

Loads	Hor. forces	Vert. forces		Lev. arms		moment abt. 0
C1		78,700	×	1.46	=	114,900
C1′	−23600		×	2.88	=	−68,000
w1		22700	×	3.70	=	84000
E_{e1}	−99500		×	2.50	=	−248500
	−123,100 kg	101,400 kg		−1.16		−117,600

Eccentricity e = 1.16 + 1.818 = 2.978 m
moment at bottom section = 101,400×2.978 = 302,000 kgm
Shear = 123,100 kg

Seismic force toward shore side.

Loads	Hor. forces	Vert. forces		Lev. arms		moment abt. 0
C_1		78700	×	1.46	=	114900
C_1'	23600		×	2.88	=	68000
w_1		22700	×	3.70	=	84000
	23600 kg	101,400 kg		2.63 m		266900

Eccentricity ε = 1.818 − 2.63 = −0.812 m
moment = 101400×0.812 = −82,300 kgm, Shear = 23,600 kg

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CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu Ken.

Governing stresses.

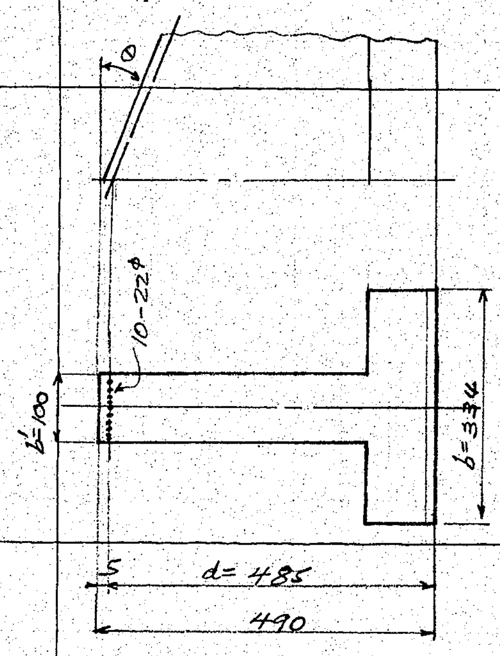
moment. + 302000 kgm during earthquake
" − 82300 " "
shear 123100 kg "

Effective depth assumed
$d = 490 - 5 = 485$ cm.
$\cos\theta = \frac{5.50}{6.24} = 0.882$
$d\cos\theta = 485 \times .882 = 4.28$ m

Steel area required for moment $\doteqdot \frac{302000 \times 100}{1920 \times \frac{7}{8} \times 428} = 42.00$ cm²

Use 10 − 22ᶲ bars = 38.01 cm²

Steel ratio $p = \frac{38.01}{334 \times 485} = 0.000235$, $t/d = \frac{100}{485} = 0.206$



Neutral axis in the flange.

$k = \sqrt{2pn + (pn)^2} - pn = \sqrt{2 \times 0.000235 \times 15 + (0.000235 \times 15)^2} - 0.000235 \times 15 = 0.0805$

$j = 1 - \frac{k}{3} = 1.000 - \frac{0.0805}{3} = 0.973$

$f_s = \frac{302000 \times 100}{38.01 \times 0.973 \times 428} = 1910$ kg/cm² < 1920.

This stress will be much reduced by direct compression.

$f_c = \frac{f_s k}{n(1-k)} = \frac{1910 \times 0.0805}{15(1-0.0805)} = 11.2$

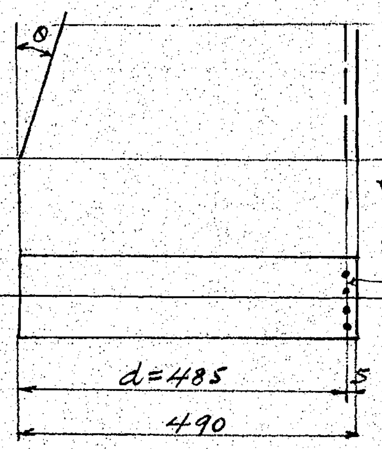
Direct compression $= \frac{101400}{724 \times 10000} = \frac{1.4}{12.6}$ kg/cm²

Unit shear $= \frac{V_1}{b'jd}$ where $V_1 = V - \frac{M}{d}\tan\theta$ $\tan\theta = \frac{2.95}{5.50} = 0.536$

$V_1 = 123100 - \frac{\overset{330}{302000}}{485} \times 0.536 = 122770$ kg

Unit shear $= \frac{122770}{100 \times 0.973 \times 485} = 2.61$ kg/cm²

Unit bond $= \frac{122770}{6.91 \times 10 \times 0.973 \times 485} = 3.77$ "



For negative moment.

Steel area required $= \frac{M}{f_s jd} = \frac{82300 \times 100}{1920 \times \frac{7}{8} \times 485} = 10.1$ cm²

Use 4 − 19ᶲ bars = 11.35 cm²

$p = \frac{11.35}{100 \times 485} = 0.000234$, $p_1 = \frac{p}{\cos^2\theta} = \frac{0.000234}{0.882^2} = 0.00030$, $j = 0.958$
$k = 0.125$

$f_s = \frac{M}{A_s jd} = \frac{82300 \times 100}{11.35 \times 0.958 \times 485} = 1560$ kg/cm² < 1920

This stress will be somewhat reduced by direct compression.

$f_c = \frac{2 f_s p_1}{k} = \frac{2 \times 1560 \times 0.0003}{0.125} = 7.5$

Direct compression $= \frac{101400}{724 \times 100} = \frac{1.4}{8.9}$ kg/cm²

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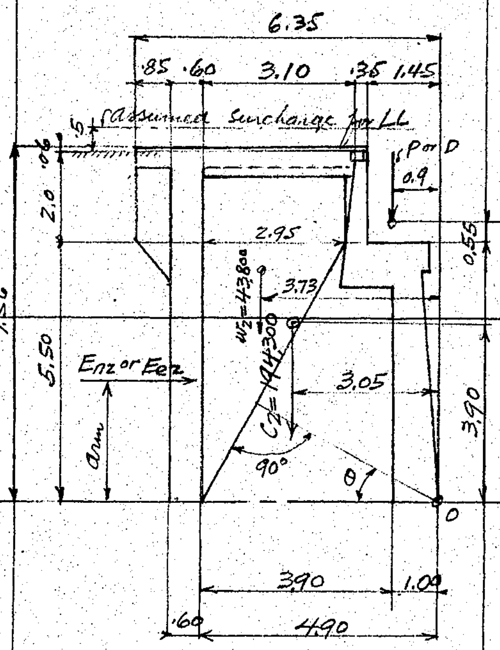
CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu ken.

Design of counterfort wall at end. CB. Total width of wall = 3.91 meters.



Weight and center of gravity of counterfort wall CB.

				arm	Vert. m.	arm	Hor. m.
Light pedestal	Granite + concrete	5.18^{m^3} @ 2400 =	12430^{kg}	9.06	113000	5.55	69000
granite posts.	0.75 × ·75 × 1.10 =	0.62 @ 2600 =	1,610	8.11	13100	1.93	3100
handrail	0.3 × 0.9 × 2.60 =	0.70 @ 2600 =	1820	8.00	14600	3.60	6600
〃	0.3 × 0.9 × 0.90 =	0.24 @ 2600 =	620	8.00	5000	1.93	1200
Slab + curb &c.	0.4 × 2.75 × 4.90 =	5.39 @ 2500 =	13480	7.36	99300	3.90	52600
wing wall (side)	0.4 × 3.45 × 7.16 =	9.88 @ 2400 =	23700	3.58	84900	3.18	75400
〃 (front)	0.85 × 2.10 × 1.66 =	2.96 @ 〃 =	7100	6.33	44900	1.78	12700
〃 (rear)	0.85 × 1.70 × 2.10 =	3.04 @ 〃 =	7300	6.10	44500	5.90	43100
〃 Column	2.75 × 0.60 × 7.16 =	11.81 @ 〃 =	28400	3.58	101,700	5.20	147700
Parapet wall (To)	0.25 × 0.3 × 1.16 =	0.09 @ 2600 =	230	7.33	1,700	1.70	400
〃	0.43 × 1.65 × 1.16 =	0.82 @ 2400 =	1970	6.28	12,400	1.67	3300
top strut	1.00 × 1.60 × 2.66 =	4.25 @ 〃 =	10200	4.99	50900	1.20	12,200
Curtain wall	0.82 × 4.50 × 2.66 =	9.81 @ 〃 =	23600	2.08	49100	0.58	13,700
Coping	0.12 × 0.60 × 2.66 =	0.19 @ 〃 =	460	5.25	2,400	0.34	200
〃	0.12 × 0.60 × 1.20 =	0.09 @ 〃 =	220	5.25	1,200	0.85	200
Counterfort	2.70 × 2.10 × 4.50 =	25.51 @ 〃 =	61200	1.91	117000	2.44	149400
	Granite — —	7.15^{m^3}	194340	3.90^{m}	755700	3.05^{m}	590800

Concrete — — 73.43 Call this C2=194300

Weight and center of gravity of earth on counterfort wall.

2.95 × 1.66 × 2.1 =	10.28 @ 1600 =	16500	× 3.42	56400
2.95 × 5.5 × 2.1 ÷ 2 =	17.05 @ 1600 =	27300	× 3.92	107000
		$w_2 = 43800^{kg}$	3.73^{m}	163400

Earth pressure at normal state.

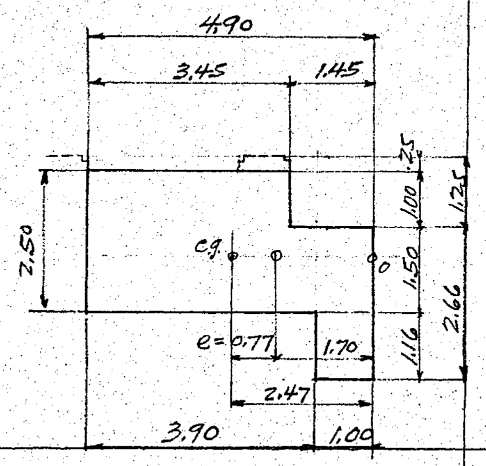
1/3 × 1600 × 0.50 = 267
1/3 × 1600 × 8.00 = 4267
4534 ÷ 2 = 2267 kg/m² average.

Earth pressure E_{n2} = 2267 × 7.5 × 3.91 = 66500 kg arm 2.59^{m}

Earth pressure during earthquake
$E_{e2} = 0.662 \times 1600 \times 7.5^2 \div 2 \times 3.91$ = 116500 kg arm 2.50^{m}
Earth pressure on front side neglected.

Stresses at normal state. Taking moment about point O.

Loads	Hor. forces	Vert. forces		Lev. arms		moment about O.
P		223500	×	0.90	=	201000
C2		194300	×	3.05	=	593000
w_2		43,800	×	3.73	=	163500
E_{n2}	−66500		×	2.59	=	−172200
	−66,500 kg	461,600 kg		1.70 m		785300



Center of gravity of bottom section.

1.16 × 1.00	=	1.16	× 0.50	=	0.58
1.50 × 4.90	=	7.35	× 2.45	=	18.00
1.00 × 3.45	=	3.45	× 3.18	=	10.97
		11.96^{m^2}	2.47^{m}		29.55

Eccentricity ε = 2.47 − 1.70 = 0.77 meter.
moment at bottom section = 461600 × 0.77 = 355500 kgm
Shear = 66500 kg

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CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu Ken.

Stresses during earthquake. Seismic forces toward river side.

Loads	Hor. forces	Vert. forces	Lev. arms	Moment about point O.
D		152,000 ×	0.90 =	136,800
D′	− 45,600	×	6.05 =	− 276,000
C2		194,300 ×	3.05 =	593,000
C2′	− 58,300	×	3.90 =	− 227,500
w_2		43,800 ×	3.73 =	163,500
Ee2	− 116,500	×	2.50 =	− 291,200
	− 220,400 kg	390,100 kg	0.25 m	98,600

Eccentricity ε = 2.47 − 0.25 = 2.22 m

moment at bottom section = 390,100 × 2.22 = 866,000 kgm
shear = 220,400 kg

Seismic forces toward shore side.

Loads	Hor. forces	Vert. forces	Lev. arms	moment about point O.
D		152,000 ×	0.90 =	136,800
D′	45,600	×	6.05 =	276,000
C2		194,300 ×	3.05 =	593,000
C2′	58,300	×	3.90 =	227,500
w_2		43,800 ×	3.73 =	163,500
	103,900 kg	390,100 kg	3.58 m	1,396,800

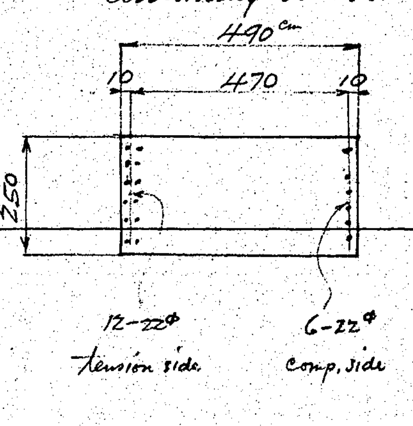
Eccentricity ε = −3.58 + 2.47 = −1.11 m
moment at bottom section = −390,100 × 1.11 = −433,000 kgm
shear 103,900 kg

Governing stresses.

Positive moment. 866000 kgm during earthquake Eccentricity ε = 2.22 meters
Corresponding shear 220400 kg 〃
〃 vertical load 390100 kg 〃

Negative moment −433,000 kgm 〃 Eccentricity ε = 1.11 meters.
Corresponding shear 103,900 kg 〃
〃 vertical load 390100 kg 〃

Assuming the bottom section as a rectangle of 2.50 × 4.90 m, effective depth = 480 cm



Try reinforcements 12-22φ bars = 45.61 on rear side
6-22φ 〃 = 22.81 on front side
68.42 cm²

$$p_0 = \frac{68.42}{490\times250} = 0.00056,\quad \frac{\varepsilon}{h} = 222/490 = 0.453,\quad d'/h = 10/490 = 0.020$$

From the prepared diagrams of combined stresses, we have as follows:
$k = 0.295$, $L = 0.065$

$$f_c = \frac{M}{Lbh^2} = \frac{866000\times100}{0.065\times250\times490^2} = 22.2 \text{ kg/cm}^2$$

$$f_s = nf_c\left(\frac{d}{kh}-1\right) = 15\times22.2\left(\overset{2.32}{\frac{480}{0.295\times490}}-1\right) = 772 \text{ kg/cm}^2$$

$$\text{Unit shear} = \frac{220400}{250\times\frac{7}{8}\times480} = 2.1 \text{ kg/cm}^2$$

$$\text{Unit bond} = \frac{220400}{6.91\times12\times\frac{7}{8}\times480} = 6.3 \text{ 〃} < 5.4\times1.6 = 8.65$$

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CALCULATIONS FOR

Design of Kissogawa Bashi for Gifu ken.

Stability of Abutment.

Superimposed loads on abutment. Seismic force

Dead Load ΣD = 304000 × 0.3 = 91200 kg = ΣD′

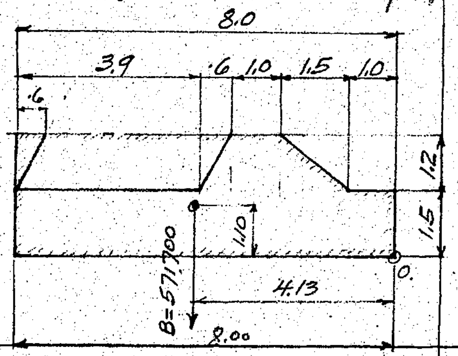
Live Load ΣL = 143000

ΣP = 447000 kg

Weight and center of gravity of shaft.

Counterfort	weight	vert arm	moment	hor. arm	moment.
2 C_A	157400	2.88	453300	1.46	229800
2 C_B	388600	3.90	1511400	3.05	1181600
	C = 546000 kg	3.60 m 2.70 6.30	1964700	2.59 m 2.50 5.09	1411400

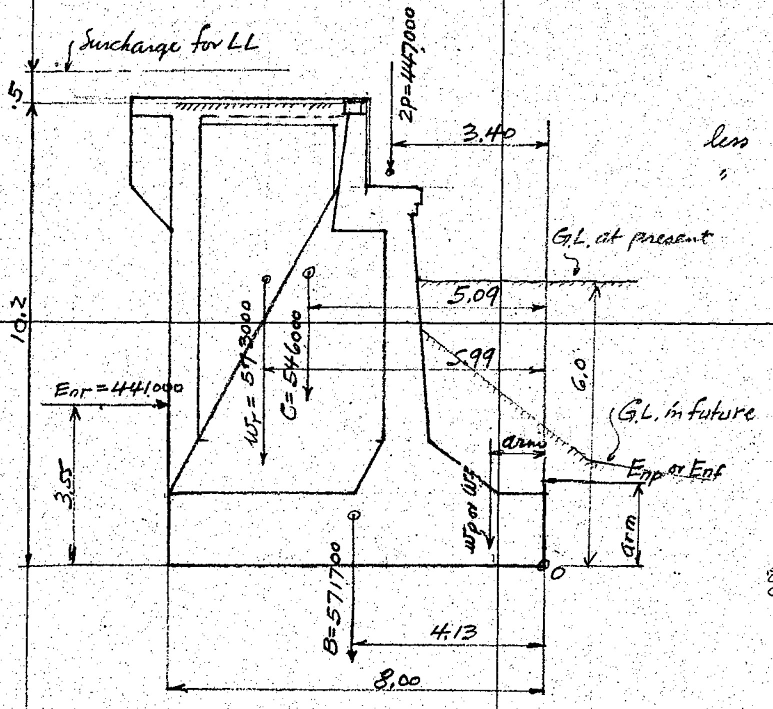
Weight and center of gravity of base.



Taking moment about toe O.

					vert arm	m	hor arm	m
Base	1.5 × 8.0 × 14.5	= 174.00	@ 2400 =	417500	0.75	313000	4.00	1670000
wall base	2.05 × 1.2 × 12.0	= 29.50	〃 〃	70800	2.00	141600	2.80	198300
Counterfort	7.0 × 1.2 × 3.9	= 32.75	〃 〃	78600	2.10	165000	5.75	452000
▽	0.6 × 1.2 × 2.75	= 1.98	〃 〃	4800	2.30	11000	7.80	37500
		238.23 m³	B =	571700 kg	1.10 m	630600	4.13 m	2357800

Stability at normal state.



Earth on rear footing.

				vert arm	m.	hor. arm	m.
	3.60 × 8.70 × 13.2	=	413.5	5.85	2420	6.20	2565
	0.90 × 5.70 × 13.2	=	67.7	4.35	294	3.95	268
less	0.60 × 4.20 × 8.0	= −	20.2	6.20	−125	7.70	−156
〃	2.90 × 5.70 × 6.2	= −	102.5	3.90	−400	5.15	−528
			358.5 m³	6.10 m	2189	5.99 m	2149

W_r = 358.5 @ 1600 = 573000 kg

Earth on front footing after future excavation for East abutment.

2.0 × 2.5 × 14.5 = 72.5 m³

W_f = 72.5 @ 1600 = 116000 kg hor. arm 1.25 m.

Earth pressure on rear side. Surcharge assumed 0.5 m for L.L.

1/3 × 1600 × 0.5 = 267

1/3 × 1600 × 10.7 = 5700.

5967 ÷ 2 = 2984 kg/m² average.

Earth pressure E_{nr} = 10.2 × 14.5 @ 2984 = 441,000 kg arm 3.55 m

Earth pressure on front side after future excav.

1/6 × 1600 × 3.5² × 14.5 = E_{nf} = 37400 kg arm 1.17 m

Earth on front footing for present G.L.

4.5 × 2.5 × 14.5 = 163 m³

W_p = 163 @ 1600 = 261000 kg.

Earth pressure on front side for present G.L.

1/6 × 1600 × 6.0² × 14.5 = E_{np} = 139000 kg arm 2.00 m

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CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu Ken.

Stability against present G.L.

Taking moment about point O.

Loads	Hor. forces	Vert. forces		Lev. arms		moment about O.
2P		447000	×	3.40	=	1520000
C		546000	×	5.09	=	2780000
w_r		573000	×	5.99	=	3430000
w_p		261000	×	1.25	=	326000
B		571700	×	4.13	=	2360000
E_{nr}	−441000		×	3.55	=	−1565000
E_{np}	139000		×	2.00	=	278000
	−302000 kg	2398700		3.80 m		9129000

$H/V = 0.126$

Eccentricity $\varepsilon = 4.00 - 3.80 = 0.20$ meter

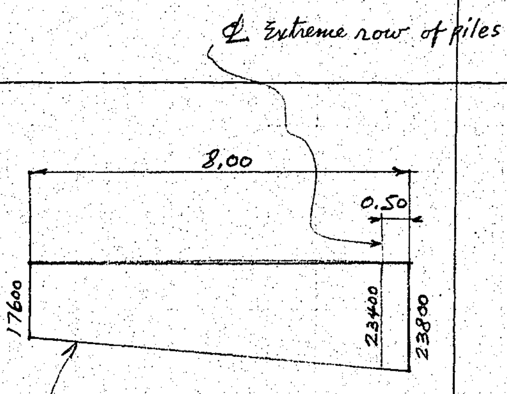
Resultant force within middle third. .15

max. toe pressure $= \dfrac{2398700}{14.5 \times 8.0}\left(1 \pm \dfrac{6 \times 0.20}{8.0}\right) =$ 23800 kg/m² (2.18 tons/○')
or 17600.

Stability against future G.L.

2P		447000	×	3.40	=	1520000
C		546000	×	5.09	=	2780000
w_r		573000	×	5.99	=	3430000
w_f		116000	×	1.25	=	145000
B		571700	×	4.13	=	2360000
E_{nr}	−441000		×	3.55	=	−1565000
E_{nf}	37400		×	1.17	=	44000
	−403600 kg	2253700 kg		3.87 m		8714000

$H/V = 0.179$



pressure diagram for Present G.L.

Eccentricity $\varepsilon = 4.00 - 3.87 = 0.13$ meter

It is not necessary to study this stress more.

max. load on one pile, neglecting bearing pressure of foundation.
$1.0 \times 0.90 \times 23400 = 21100$ kg or 21.1 kgtons

If 10 kgtons/○m be allowed on sand foundation for bearing,
Load on one pile $= 21.1 - 1.0 \times 0.9 \times 10 = 12.1$ kgtons.

Stability during earthquake.

Earth pressure on rear side
$E_{er} = \frac{1}{2} \times 0.662 \times 1600 \times 10.2^2 \times 14.5 =$ 799000 kg arm 3.40 m

Earth pressure on front side.
for Present G.L.
$E_{ep} = \frac{1}{2} \times 0.662 \times 1600 \times 6.00^2 \times 14.5 =$ 276000 kg arm 2.00 m

front future G.L.
$E_{ef} = \frac{1}{2} \times 0.662 \times 1600 \times 3.5^2 \times 14.5 =$ 94000 kg arm 1.17 m

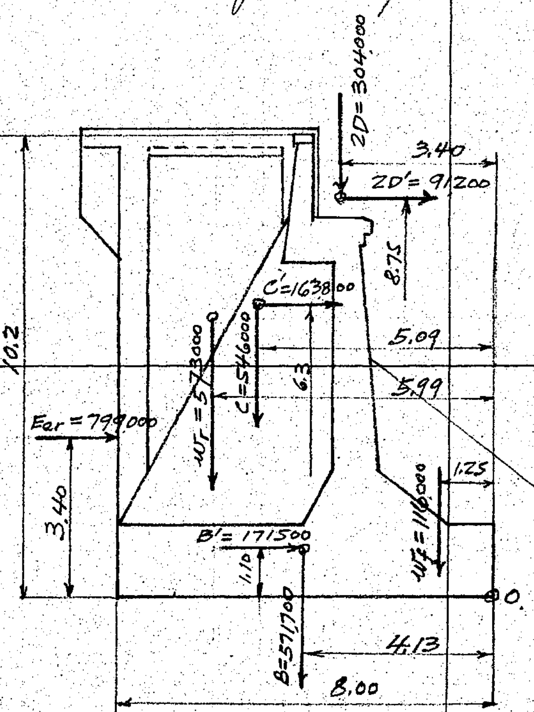
Superimposed dead load 2D = 304000 " " 3.40 m

Seismic forces due to

Dead Load	$2D' = 304000 \times .3 =$	91200	kg	"	8.75 m
Shaft	$C' = 546000 \times .3 =$	163800	"	"	6.30 m
Base	$B' = 571700 \times .3 =$	171500	"	"	1.10 m

Design of Kisogawa Bashi for Gifu Ken.

Seismic force toward river side.
For future ground line.



Taking moment about toe O.

Loads	Hor. forces.	Vert. forces.	Lev. arms.	moments abt. O.
ZD		304,000	× 3.40	= 1,034,000
ZD'	-91,200		× 8.75	= - 798,000
C		546,000	× 5.09	= 2,780,000
C'	-163,800		× 6.30	= -1,032,000
B		571,700	× 4.13	= 2,360,000
B'	-171,500		× 1.10	= - 189,000
w_r		573,000	× 5.99	= 3,430,000
w_f		116,000	× 1.25	= 145,000
Eer	-799,000		× 3.40	= -2,715,000
	-1,225,500 kg	2,110,700 kg	2.38 m	5,015,000

H/V = 0.58

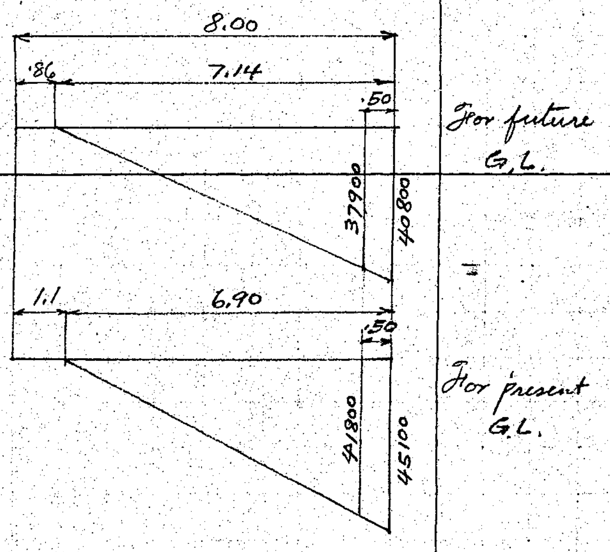
Eccentricity Σ = 4.00 - 2.38 = 1.62 m
Resultant force outside of middle third. Neglecting tension.
Pressure area = 2.38 × 14.5 × 3 = 103.5 square meters.
max. toe pressure = $\frac{2,110,700 \times 2}{103.5}$ = 40,800 kg/m² or (3.73 tons/□')

For Present ground line.

ZD		304000	× 3.40	= 1,034,000
ZD'	- 91,200		× 8.75	= - 798,000
C		546,000	× 5.09	= 2,780,000
C'	- 163,800		× 6.30	= -1,032,000
B		571,700	× 4.13	= 2,360,000
B'	- 171,500		× 1.10	= - 189,000
w_r		573,000	× 5.99	= 3,430,000
w_p		261,000	× 1.25	= 326,000
Eer	- 799,000		× 3.40	= -2,715,000
	-1225,500	2255700	2.30 m	5,196,000

H/V = 0.544

Eccentricity Σ = 4.00 - 2.30 = 1.70 m
Resultant force outside of middle third. Neglecting tension.
Pressure area = 2.30 × 3 × 14.5 = 100.0 sq. meters.
max. Toe pressure = $\frac{2255.700 \times 2}{100}$ = 45100 kg/m² or (4.13 tons/□')



max. load on one pile = 1.0 × 0.9 × 41800 = 37.6 kg tons
If. 16 tons/□m be allowed on sand foundation for bearing
load on one pile = 37.6 - 16 × 0.9 = 23.2 kg tons.

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CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu ken

Seismic force toward shore side.

For future ground line. Taking moment about toe O.

Loads	Hor. forces	Vert. forces		Lev. arms		moment about O.
2D		304,000	×	3.40	=	1,034,000
2D'	91,200		×	8.75	=	798,000
C		546,000	×	5.09	=	2,780,000
C'	163,800		×	6.30	=	1,032,000
B		571,700	×	4.13	=	2,360,000
B'	171,500		×	1.10	=	189,000
w_r		573,000	×	5.99	=	3,430,000
w_f		116,000	×	1.25	=	145,000
E_{ef}	94,000		×	1.17	=	110,000
	520,500 kg	2,110,700 kg		5.62 m		11,878,000

H/V = 0.247 $8.0-5.62=2.38$

Eccentricity $\varepsilon = 4.00-5.62 = 1.62$ meters

Resultant force outside of middle third; neglecting tension on heel,

pressure area $= 2.38 \times 3 \times 14.5 = 103.5$ sq. meters.

max. toe pressure $= \frac{2110700 \times 2}{103.5} = 40,800$ kg/m² or (3.73 tons/□')

For present ground line.

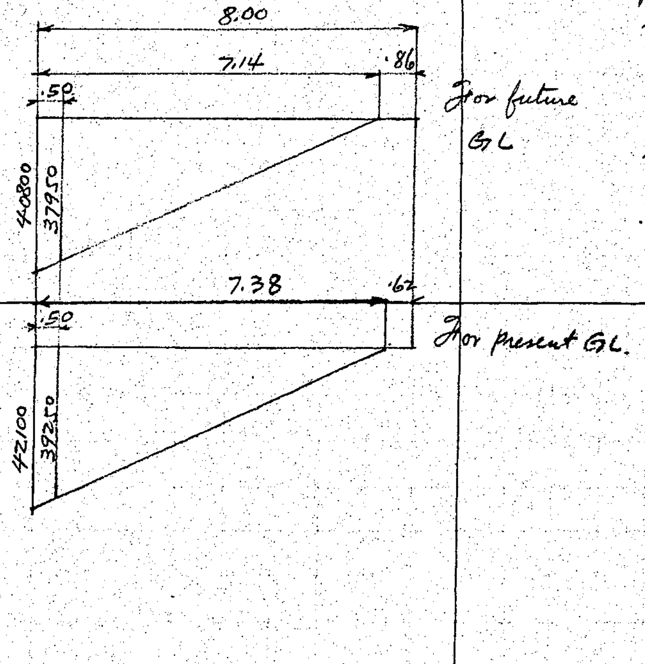
2D		304,000	×	3.40	=	1,034,000
2D'	91,200		×	8.75	=	798,000
C		546,000	×	5.09	=	2,780,000
C'	163,800		×	6.30	=	1,032,000
B		571,700	×	4.13	=	2,360,000
B'	171,500		×	1.10	=	189,000
w_r		573,000	×	5.99	=	3,430,000
w_p		261,000	×	1.25	=	326,000
E_{ep}	276,000		×	2.00	=	552,000
	702,500 kg	2,255,700 kg		5.54 m		12,501,000

H/V = 0.311 $8.0-5.54=2.46$

Eccentricity $\varepsilon = 4.00-5.54 = -1.54$ meters

pressure area $= 2.46 \times 3 \times 14.5 = 107.0$ sq. meters

max. toe pressure $= \frac{2255700 \times 2}{107.0} = 42100$ kg/m² or (3.85 tons/□')



max. load on one pile $= 1.0 \times 0.9 \times 39250 = 35.3$ kg tons.

If 16 kg tons/□m be allowed on sand foundation for bearing

load on one pile $= 35.3 - 16 \times 0.9 = 20.9$ kg tons.

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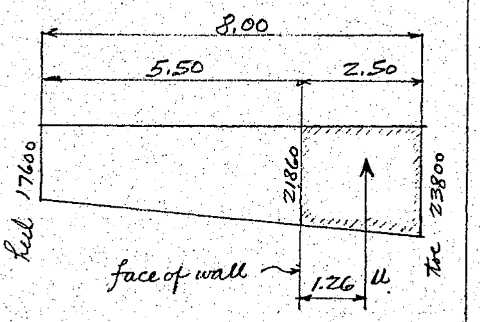
CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu ken.

Design of Cantilever footing at toe.

moment at normal state



Upward pressure on footing 23800
21860
45660 ÷ 2 = 22,830 kg/m² average.

Total pressure u = 22830 × 2.5 = 57000 kg per meter strip of footing.

Downward pressure on footing.

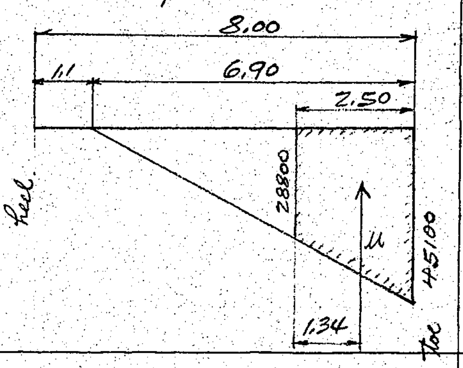
Concrete footing
1.5 × 2.5 = 3.75 @ 2400 = 9000 × 1.25 = 11250
1.5 × 1.2 ÷ 2 = 0.90 @ 2400 = 2200 × 0.50 = 1100
4.65 11,200 kg 1.10ᵐ 12350 kgm

Earth on footing
4.5 × 2.5 = 11.25 @ 1600 = 18000 × 1.25 = 22500
less 1.5 × 1.2 ÷ 2 = - 0.90 @ 1600 = - 1400 × 0.50 = - 700
10.35 16600 kg 1.31ᵐ 21,800 kgm

moments and shears.
Upward pressure 57000 × 1.26 = 71800
Downward "
Concrete footing - 11200 × 1.10 = - 12350
earth on footing - 16600 × 1.31 = - 21800
29200 kg 37650 kgm

Moment during earthquake.

max. positive moment.

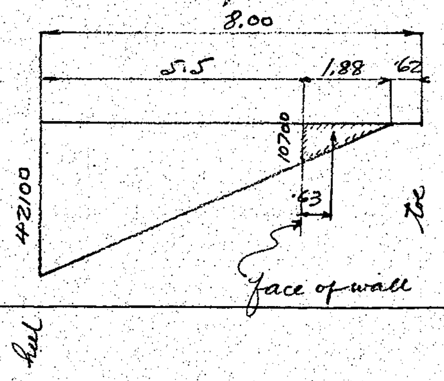


Upward pressure on footing 45100
28800
73900 ÷ 2 = 36950

Total pressure u = 36950 × 2.5 = 92400 kg per meter strip of footing.

moment + shear on footing.
Upward pressure 92400 × 1.34 = 123800
Downward "
Concrete footing - 11200 × 1.10 = - 12,350
earth on " - 16600 × 1.31 = - 21,800
64,600 kg 89650 kgm.

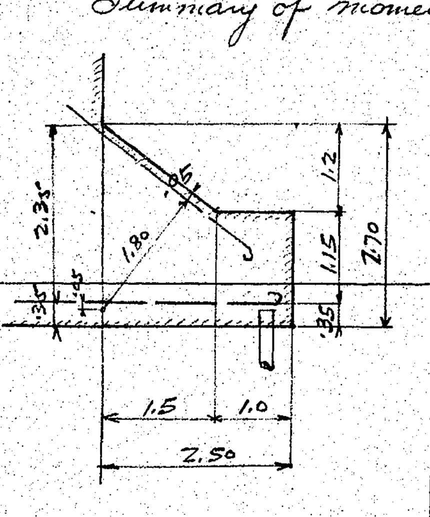
max. negative moment.



Upward pressure on footing
u = ½ × 10700 × 1.88 = 10,100 kg per meter strip.

moment and shear on footing
Upward pressure 10100 × 0.63 = 6360
Downward "
Concrete footing - 11200 × 1.10 = - 12350
Earth on " - 16600 × 1.31 = - 21800
- 17,700 kg - 27790 kgm

Summary of moments and shears on footing.



	moment	shear
Normal state	37650 kgm	29200 kg
During earthquake	89650 "	64600 "
"	- 27790 "	- 17700 "

Seismic stresses control the section of footing.

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CALCULATIONS FOR Design of Kisogawa-Bashi for Gifu-ken

Bottom reinforcements for positive moment.

$$\text{Steel area required} = \frac{89650 \times 100}{1920 \times \frac{7}{8} \times 235} = 22.7 \text{ cm}^2 \text{ per meter strip.}$$

Use 22^{ϕ} bars at 30^{cm} ctoc = 12.68 perimeter for bond $3.333 @ 6.91 = 23.05$
19^{ϕ} " " 30 " " = $\underline{9.45}$ $3.333 @ 5.97 = \underline{19.90}$
22.13 cm² 42.95 cm

Steel ratio $p = 22.13/100 \times 235 = 0.00094$
From the prepared table we have;
$k = 0.155$, $j = 0.948$

$$f_s = \frac{89650 \times 100}{22.13 \times 0.948 \times 235} = 1820 \text{ kg/cm}^2 < 1920 ✓$$

$$f_c = \frac{1820 \times 0.155}{15(1-0.155)} = 22.3 \text{ kg/cm}^2 < 405 \times 1.6 = 648$$

$$\text{Unit shear} = \frac{\overset{.845}{64600}}{100 \times 0.948 \times 235} = 2.9 \text{ kg/cm}^2 < 5.75$$

$$\text{Unit bond} = \frac{64600}{42.95 \times 0.948 \times 235} = 6.8 \text{ kg/cm}^2 < 8.65$$

Top reinforcements for negative moment.

$$\text{Steel area required} = \frac{27790 \times 100}{1920 \times \frac{7}{8} \times 180} = 9.18 \text{ cm}^2 \text{ per meter strip of footing.}$$

Use 19^{ϕ} bars at 30^{cm} ctoc = 9.45 cm² perimeter 19.90 cm
Steel ratio $p = 9.45/100 \times 180 = 0.00053$

$$k = \sqrt{2 \times 15 \times 0.00053 + (15 \times 0.00053)^2} - 15 \times 0.00053 = 0.118,$$

$$j = 1 - \tfrac{1}{3} k = 1 - \frac{0.118}{3} = 0.961$$

$$f_s = \frac{27790 \times 100}{9.45 \times 0.961 \times 180} = 1700 \text{ kg/cm}^2 < 1920$$

$$f_c = \frac{1700 \times 0.118}{15(1-0.118)} = 15.2 \text{ kg/cm}^2$$

$$\text{Unit shear} = \frac{\overset{.882}{17700}}{100 \times .961 \times 180} = 1.0 \text{ "}$$

$$\text{Unit bond} = \frac{17700}{19.90 \times .961 \times 180} = 5.1 \text{ "}$$

Design of Rear footing. Span length 3.34 meters.

max. upward pressure.		Extreme one meter strip	One meter strip of inside edge.
upward pressure		39250	22700
downward "	Earth 8.7 @ 1600 =	− 13920	− 13920
	footing 1.5 @ 2400 =	− 3600	− 3600
		21730 kg/m²	5180 kg/m²

moment $1/10 \times 3.34^2 \times 21730 = \pm 24200$ kgm, $1/10 \times 3.34^2 \times 5180 = \pm 5800$ kgm
Shear $1/2 \times 3.34 \times 21730 = 36300$ kg, $1/2 \times 3.34 \times 5180 = 8700$ kg.

max. downward pressure.		Extreme one meter strip	One meter strip of inside edge.
upward pressure		0	14800
downward "	Earth	− 13920	− 13920
	footing	− 3600	− 3600
		− 17520 kg/m²	− 2720 kg/m²

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CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu Ken.

Reinforcements in rear footing.

Steel area required $= \dfrac{24200 \times 100}{1920 \times \frac{7}{8} \times 115} = 12.5\ \text{cm}^2$ for extreme one meter strip.

Use 19^{ϕ} bars 23 cm ctoc $= 12.34\ \text{cm}^2$

steel ratio $p = \dfrac{12.34}{100 \times 115} = 0.0011$

$k = 0.164 \qquad j = 0.946$

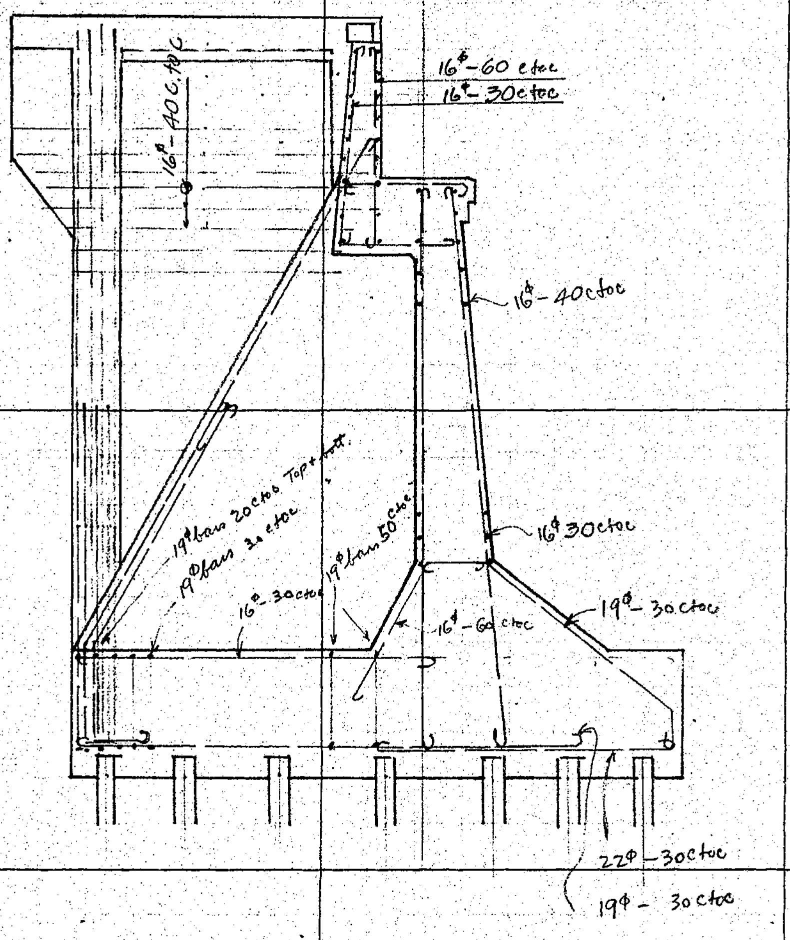
$f_s = \dfrac{24200 \times 100}{1234 \times .946 \times 115} = 1803\ \text{kg/cm}^2 < 1920$

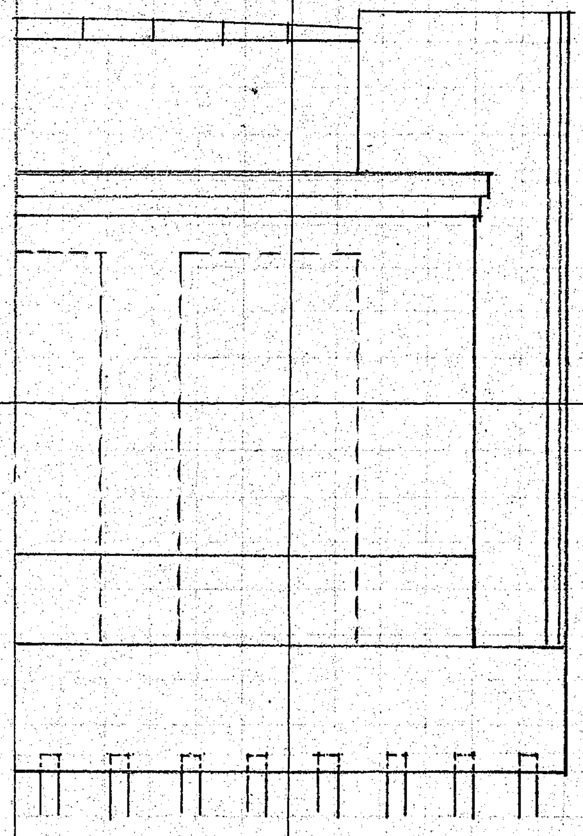
$f_c = \dfrac{1803 \times 0.164}{15(1-.164)} = 23.6$ "

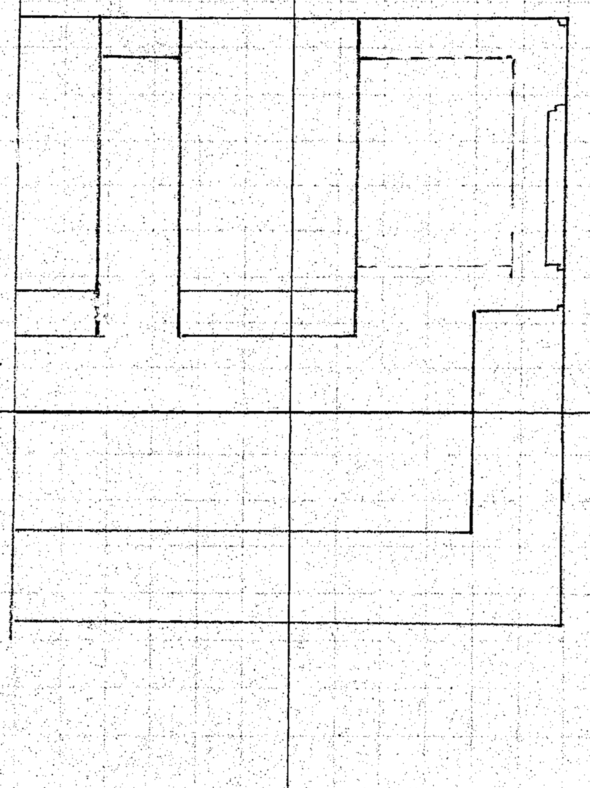
Unit shear $= \dfrac{36300}{100 \times .946 \times 115} = 3.34\ \text{kg/cm}^2$

For negative moment at counterfort, use the same reinforcements as above.

Sketch of reinforcements in abutment.







Note, approximate volume of concrete

Counterfort wall C_A $2 \times 32.49 = 64.98$

Counterfort wall C_B $2 \times 73.43 = 146.86$

Base 238.23

450.07 cub meters

昭和八年六月

國道拾貳號線

岐阜縣木曾川橋材料調書

（下部工事）

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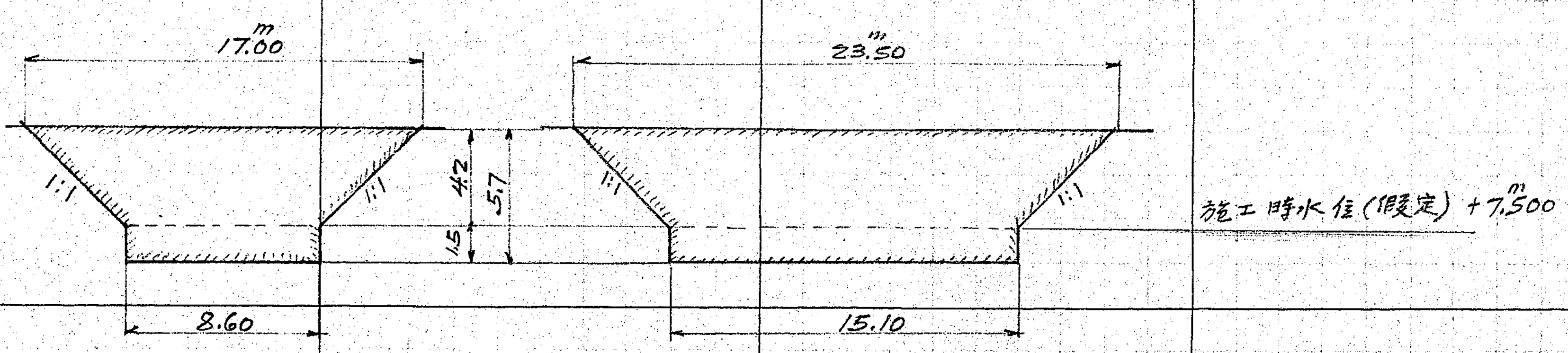
CALCULATIONS FOR

Materials of Kisogawa-Bashi for Gifu ken.

Materials of East and West abutments.



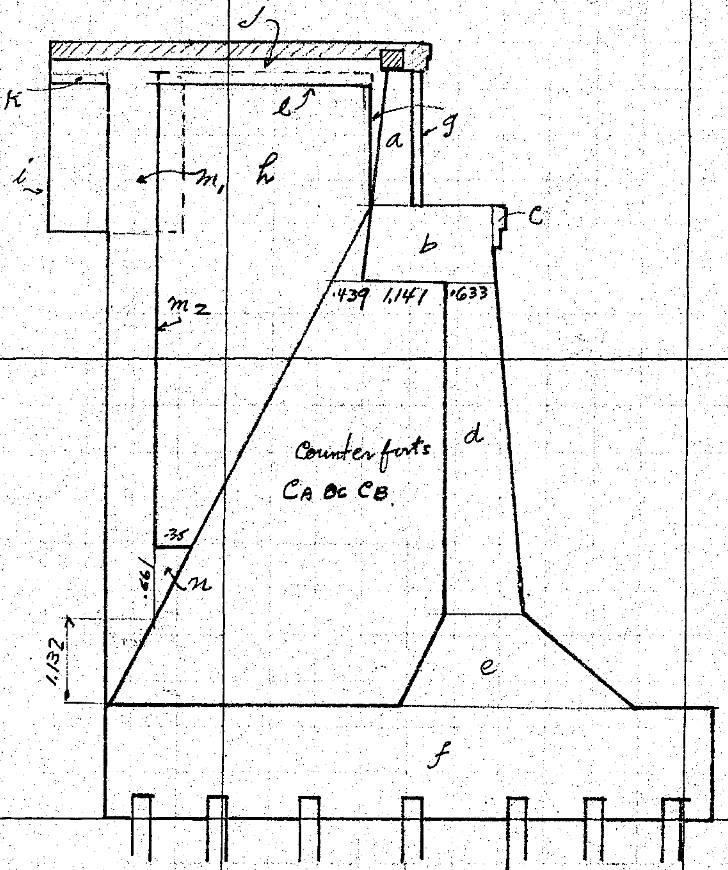
Excavation above ground water
12.80 × 19.30 × 4.20 = 1037.6 Cub. meters.

Excavation under ground water.
8.60 × 15.10 × 1.50 = 194.8 〃 〃

Foundation piles. 生松丸太 21 cmφ at tip × 9.00 m long.
16 rows @ 7 piles = 112 piles.

Concrete 1:2.5:5 mixture.

		Width thickness length		Volume.
Parapet wall	a	1.647 × 0.425 × 9.000	=	6.300
Strut	b.	1.704 × 1.000 × 12.000	=	20.448
coping	c	0.600 × 0.117 × 14.400	=	1.011
front wall	d	4.500 × 0.817 × 12.000	=	44.118
base	e	2.050 × 1.200 × 12.000	=	29.520
〃	f	8.000 × 1.500 × 14.500	=	174.000
Pile heads less		0.280φ × 0.300 × 112	= (−)	2.070
Counterfort	CA.	0.220 × 1.000 × 1.000 × 2	=	0.440
〃	〃	3.090 × 1.000 × 5.700 × 2	=	35.226
〃	〃 less	0.300 × 1.200 × 1.000 × 2	= (−)	0.720
〃	CB.	0.220 × 1.000 × 2.100 × 2	=	0.924
〃	〃	3.090 × 1.500 × 5.700 × 2	=	52.839
〃	〃	2.650 × 0.600 × 5.700 × 2	=	18.126
〃	〃 less	0.300 × 1.200 × 1.500 × 2	= (−)	1.080
Wing wall, front	g	1.730 × 0.600 × 2.100 × 2	=	4.360
〃	h	4.150 × 0.400 × 8.430 × 2	=	27.988
Column top	i	1.800 × 0.250 × 2.640 × 2	=	2.376
〃 〃	〃	1.350 × 0.850 × 2.680 × 2	=	6.151
deck slab	j	2.300 × 0.150 × 4.600 × 2	=	3.174
〃 〃	k	1.150 × 0.400 × 0.850 × 1½ say	=	0.587
〃 beam	l	0.400 × 0.300 × 2.950 × 2	=	0.708
〃 〃 fillets say		0.100 × 0.050 × 9.500 × 2	=	0.095
Column	m₁	2.100 × 0.600 × 2.680 × 2	=	6.754
〃	m₂	2.100 × 0.600 × 5.187 × 2	=	13.071
〃	〃	0.600 × 0.250 × 5.757 × 2	=	1.727
〃 fillet	n	0.175 × 2.100 × 0.661 × 2	=	0.486



Summary of concrete for abutment = 446.559
Call this 446.56 cub. meters.

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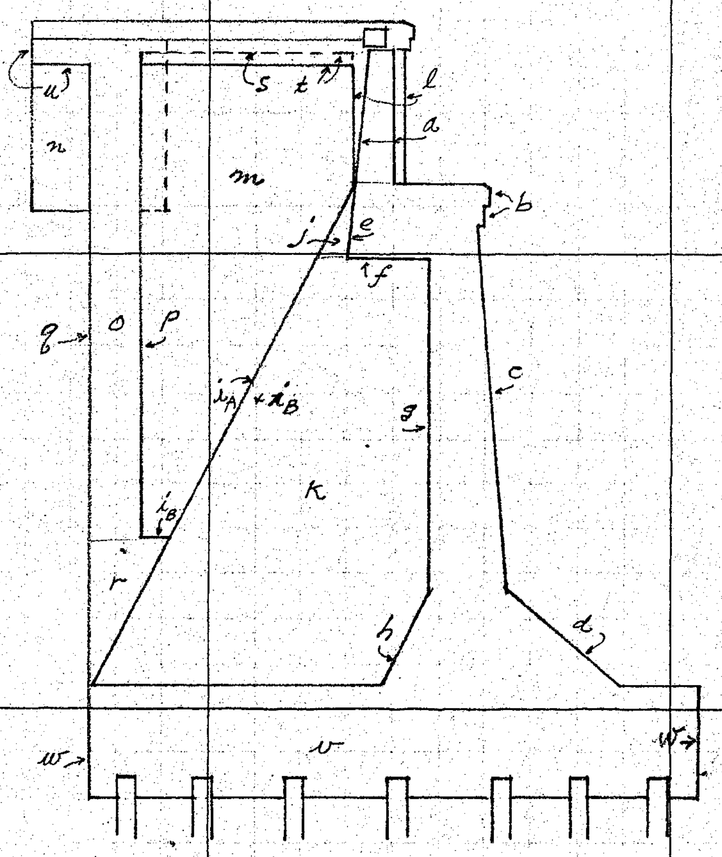
CALCULATIONS FOR

Materials of Kisogawa-Bashi for Gifu-Ken.

Reinforcements. plain bars.
See Drawing ---------- 11.524 kg tons.

Forms. 杉板 厚五三粍

		width		length		req'd.no		
Parapet wall	a.	1.647	×	9.000	×	2	=	29.65
Coping	b	0.750	×	14.400	×	1	=	10.80
front wall, front.	c	4.900	×	12.000	×	1	=	58.80
〃 sides	〃	1.250	×	4.900	×	2	=	12.25
〃 front	d	1.921	×	12.000	×	1	=	23.05
〃 sides	〃	2.200	×	1.200	×	2	=	5.28
〃 rear	e	1.000	×	7.000	×	1	=	7.00
〃 〃	f	1.141	×	7.000	×	1	=	7.99
〃 〃	g	4.500	×	7.000	×	1	=	31.50
〃 〃	h	1.342	×	7.000	×	1	=	9.39
Counterforts, rear.	i_A	1.000	×	7.580	×	2	=	15.16
〃 〃	i_B	2.100	×	5.543	×	2	=	23.28
〃 〃	〃	0.350	×	2.100	×	2	=	1.47
〃 sides	j	0.220	×	1.000	×	6	=	1.32
〃 〃	k	3.090	×	5.700	×	6	=	105.68
〃 〃	less	0.300	×	1.200	×	6	=	(−) 2.16
wing wall. front	l	1.730	×	2.600	×	2	=	9.00
〃 rear	l	1.730	×	2.180	×	2	=	7.54
〃 front	l	1.000	×	6.700	×	2	=	13.40
〃 outside	m	4.150	×	8.400	×	2	=	69.72
〃 〃	〃	0.250	×	8.750	×	2	=	4.38
〃 〃	〃 bott.	0.85	×	1.600	×	2	=	2.72
〃 Projection	n both sides	0.850	×	4.930	×	2	=	8.38
〃 inside	m	2.950	×	1.730	×	2	=	10.21
〃 〃	m	1.650	×	4.900	×	2	=	16.17
Column, face	o	0.600	×	6.637	×	2	=	7.96
〃 side	p	2.100	×	6.637	×	2	=	27.88
〃 〃	q	2.500	×	8.560	×	2	=	42.80
〃 〃	r	0.475	×	1.800	×	2	=	1.71
deck bottom	s	1.800	×	2.950	×	2	=	10.62
〃 beam	t	1.200	×	2.950	×	2	=	7.08
〃	u	1.150	×	0.850	×	say 1½	=	1.47
〃	〃	0.500	×	3.000	×	2	=	3.00
base	v	1.500	×	8.000	×	2	=	24.00
〃	w	1.500	×	14.50	×	2	=	43.50



Summary of forms for west abutment 652.00 sq. meters.
less sheet pile area 1.000 × 22.500 × 1 = (−) 22.50

Summary of forms for east abutment 629.50 sq. meters.

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CALCULATIONS FOR

Kisogawa-Bashi for Gifu ken.

Granite blocks. 踏掛石.	width thickness length		volume.	
	4@ 0.34 × 0.30 × 0.85	=	0.347	
	2@ 0.33 × 0.30 × 0.85	=	0.168	
	2@ 0.31 × 0.30 × 0.95	=	0.177	
	2@ 0.29 × 0.30 × 1.00	=	0.174	
			0.866	Call this 0.87 cub. meter.
Curb stones	2@ 0.22 × 0.20 × 1.24	=	0.109	
	4@ " " × 0.96	=	0.169	
	2@ " " × 0.72	=	0.063	
Circular corner	2@ 0.30 × 0.20 × 0.90	=	0.108	
" "	2@ 0.30 × 0.20 × 0.80	=	0.096	
Under pedestal	4@ 0.22 × 0.355 × 0.90	=	0.281	
" "	2@ " " × 1.36	=	0.212	
" "	2@ " " × 0.53	=	0.083	
			1.121	call this 1.12 cub. meters
Coping stones	6@ 0.40 × 0.355 × 0.973	=	0.829	
	2@ " " × 0.40	=	0.114	
	4@ " " × 0.96	=	0.545	
			1.488	call this 1.49 cub. meters } 4.53 Cub meters
Slab stones	18@ 0.48 × 0.15 × 0.88	=	1.140	
	14@ " " × 0.44	=	0.444	
	2@ " " × 0.98	=	0.141	
	2@ " " × 0.54	=	0.078	
	2@ 0.42 × 0.15 × 0.90	=	0.113	
			1.916	Call this 1.92 cub. meters.
人造洗出仕上				
Rear of post	0.25 × 1.60 × 2	=	0.80	
Outside of wing	2.33 × 5.00 × 2	=	23.30	
Front side of wing	0.25 × 1.80 × 2	=	0.90	
"	2.50 × 1.70 × 2	=	8.50	
"	1.00 × 2.80 × 2	=	5.60	
Coping . front	0.90 × 12.30 × 1	=	11.07	
" . both ends.	0.90 × 1.20 × 2	=	2.16	
Front wall. front	2.20 × 12.00 × 1	=	26.40	
" both ends	1.16 × 2.20 × 2	=	510	
			8383	sq. meters.
Cement mortar finish	1:2 mortar.			
	1.15 × 9.00	=	10.35	
	1.05 × 1.50 × 2	=	3.15	
			13.50	sq. meters.

Gas pipes for Electric wiring.

4 – 2"φ gas pipes × 2.20 meters long.

増田淳氏関係資料（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

Materials of Kisogawa-Bashi for Gifu-Ken.

Sheet piling for East abutment only.

Sheet piles	松材. 厚 5cm 長 4.00m 延長 22.50 meters.	
wales	松材 4 @ 12cm × 15cm × 4.20m = 4 @ 0.076 =	0.304 cub. meters
	〃 6 @ 12 × 15 × 5.30 = 6 @ 0.095 =	0.570 〃
Bolts	20 - 19mmφ × 35 cm bolts with washers @ 1.20 kg =	24.00 kgs
	8 - 19mmφ × 20 〃 〃 〃 @ 0.85 =	6.80 〃

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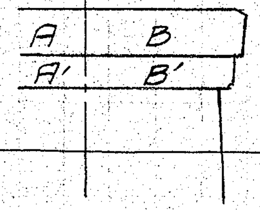
CALCULATIONS FOR

Materials of Kisogawa-Bashi for Gifu-ken

materials of Pier Shafts P1, P2 and P3
Shafts for P4, P5 and P6 are identical to those for P3, P2 and P1 respectively

Concrete for shaft 1 : 2 : 4 mixture
Coping

	Section	length	Req'd no.	Volume	Remark
Rectangle	$\overset{m}{2.39} \times \overset{m}{.35}$	$\overset{m}{10.50}$	1	8.783	A
〃	2.23 × .25	10.50.	1	5.854	A′
circular end	2.39$^{\phi}$	.35	1	1.570	B
〃	2.23$^{\phi}$	.25	1	.976	B′
				17.183 cub. meters	



Shaft
Top section
Rectangle $\overset{m}{2.10} \times \overset{m}{10.50}$ 1 22.05
circular end 2.10$^{\phi}$ 1 3.46
25.51 sq. meters

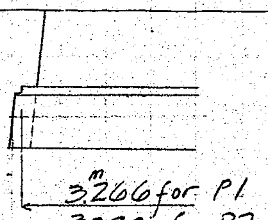
Middle Section
Rect. portion cir. ends
P1 26.33 × 1.05 = 27.65 + 5.44 = 33.09 sq.m
P2 26.49 × 1.05 = 27.81 + 5.51 = 33.32
P3 26.58 × 1.05 = 27.91 + 5.55 = 33.46

Bottom section

	Rectangular portion				Circular end		
P1	$\overset{m}{3.166} \times \overset{m}{10.50}$	=	33.24	+	7.87	=	41.11 sq.m
P2	3.198 × 10.50	=	33.58	+	8.03	=	41.61
P3	3.215 × 10.50	=	33.76	+	8.12	=	41.88

Volume
P1 1/6 × 6.393 × (25.51 + 41.11 + 4 × 33.09) = 212.013 cub. meters
P2 1/6 × 6.589 × (25.51 + 41.61 + 4 × 33.32) = 220.073 〃 〃
P3 1/6 × 6.688 × (25.51 + 41.88 + 4 × 33.46) = 224.304 〃 〃

Collar
P1 .15^{m} × .60^{m} × (21.00 + 10.26) = 2.813 cub. meters
P2 .15 × .60 × (21.00 + 10.36) = 2.822 〃 〃
P3 .15 × .60 × (21.00 + 10.41) = 2.827 〃 〃



$\overset{m}{3.266}$ for P1
3.298 for P2
3.315 for P3

Hollow
Top section
P1 $\overset{m}{1.05} \times \overset{m}{2.95}$ = 3.10 sq.m
P2 1.05 × 2.95 = 3.10 〃
P3 1.05 × 2.95 = 3.10 〃

Bottom section
1.966 × $\overset{m}{2.95}$ = 5.80 sq.m.
1.998 × 2.95 = 5.89 〃 〃
2.015 × 2.95 = 5.94 〃 〃

Volume of hollow
P1 1/2 × 5.493 × (3.10 + 5.80) = 24.444 × 2 = 48.888 cub. meters
P2 1/2 × 5.689 × (3.10 + 5.89) = 25.572 × 2 = 51.144 〃
P3 1/2 × 5.788 × (3.10 + 5.94) = 26.162 × 2 = 52.324 〃

Volume of Bottom Slab
Rectangle $\overset{m}{3.50} \times \overset{m}{1.20} \times \overset{m}{6.30}$ = 26.460
〃 $\overset{m}{3.50} \times \overset{m}{1.80} \times 4.20$ = 26.460
circular end 3.50$^{\phi}$ × 1.80 = 17.318
dowels 4 @ 1.20 × 1.20 × .20 = 1.152
71.390 cub. meters

Total volume of concrete

Pier	Coping	Shaft	Collar	Slab	hollow		Call these
P1	17.183	+ 212.013	+ 2.813	+ 71.390	− 48.888	= 254.511 cub. m	254.51 cub. m
P2	17.183	+ 220.073	+ 2.822	+ 71.390	− 51.144	= 260.324 〃	260.32 〃
P3	17.183	+ 224.304	+ 2.827	+ 71.390	− 52.324	= 263.380 〃	263.38 〃

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MADE BY M. Jinda DATE 8-5-10 **FILE NO**
CHECKED BY K. Inaba DATE 8-5-19 **PAGE NO** M6

CALCULATIONS FOR

Materials of Kisogawa-Bashi for Gifu-Ken

Forms for shaft
Coping
Side 2 c .75^m × 10.50^m = 15.75
circular ends 1 c .75^m × 2.40$^\phi$ = 5.65
21.40 sq. meters

Shaft, Total length of perimeter at top
Side 2 c 10.50^m = 21.00^m
circular end 1 c 2.10$^\phi$ = 660
27.60 meters

Perimeter of collar

	dia	cir. end		rectangle	
P1	3.416	10.73^m +		21.00^m =	31.73^m
P2	3.448	10.83 +		21.00 =	31.83
P3	3.465	10.89 +		21.00 =	31.89

Total length of perimeter at bottom

	dia	circular end		Straight portion		mean perimeter
P1	30.66	9.63	+	21.00^m	= 30.63^m	29.115^m
P2	30.98	9.73	+	21.00	= 30.73	29.165
P3	31.15	9.79	+	21.00	= 30.79	29.195

Top length around hollow
4 c 1.05^m + 4 × 2.95^m = 16.00^m

Bottom length around hollow

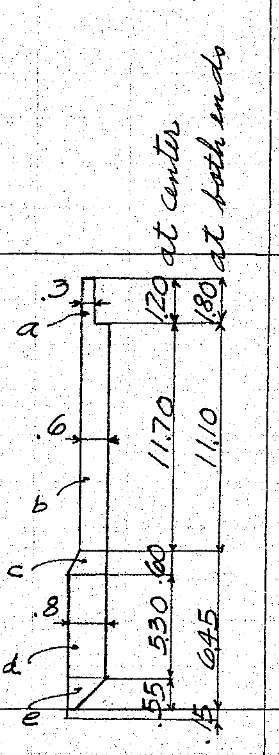
			mean perimeter
P1	4 c 1.966 + 4 × 2.95	= 19.66^m	17.83^m
P2	4 c 1.998 + 4 × 2.95	= 19.79	17.90
P3	4 c 2.015 + 4 × 2.95	= 19.86	17.93

Total area of forms

Pier	Coping	Shaft	hollow	collar	Total
P1	21.40,	29.115 × 57.93 = 168.66,	17.83 × 54.93 = 97.94,	31.73 × .60 = 19.04	307.04 sq.m
P2	21.40,	29.165 × 59.89 = 174.67,	17.90 × 56.89 = 101.83,	31.83 × .60 = 19.10	317.00 〃
P3	21.40,	29.195 × 60.88 = 177.74,	17.93 × 57.88 = 103.78,	31.89 × .60 = 19.13	322.05 〃

埋込型枠 Top of hollow 2 c 1.05 × 2.95 = 6.20 sq meters
Reinforcements, plain bars
P1 59.58 kg tons P2 61.20 kg tons P3 61.49 kg tons
人造仕上 straight portion 2 c .90 × 10.50 = 18.90
circular ends .90 × 2.25$^\phi$ = 6.36
25.26 sq. meters
mortar finish, Straight portion 2.10 × 10.50 = 22.05
circular ends 2.10$^\phi$ = 3.46
25.51 sq. meters

Materials of 19.5 meters well
Concrete 1 : 2 : 4 mixture
Lot a (upper 1.2^m) 4.10^m × 10.50^m = 43.05
4.10$^\phi$ = 13.20
56.25
hollow 3.50^m × 10.50^m = 36.75
3.50$^\phi$ = 9.62
− 46.37
Sectional area 9.88 sq. m
depth × 1.20
volume 11.856 cub. meters



Lot a (lower .60^m) 9.88
add 2 c .30^m × 6.30^m = 3.78
partion wall .40 × 2.90 = 1.16
fillets 2 c .30 × .30 = .18
Sectional area 15.00 sq. m net

Materials of Kisogawa-Bashi for Gifu-Ken

Sectional area 15.00
depth × .60
volume 9.00 cub. meters

Lot b gross sectional area 56.25 sq. m
hollow less 2.90 × 10.50 = −30.45
" 2.90$^{\phi}$ = − 6.61 } − 37.06
part. wall add 3 @ .40 × 2.90 = 3.48
fillets add 6 @ .30 × .30 = .54
− 33.04

net Sectional area 23.21 sq. m
depth × 11.10
volume 257.631 cub. meters

Sectional area of lot d
4.50 × 10.50 = 47.25
4.50$^{\phi}$ = 15.90
63.15 gross
hollow less same amount as b − 33.04
30.11 sq. m net

Lot c Sectional area of b lot = 23.21
" " " d " = 30.11
mean sectional area 2 | 53.32 = 26.66
depth × .60
volume 15.996 cub. meters

Lot d Sectional area, net 30.11 sq. m
depth × 5.30
Volume 159.583 cub. meters

Lot e Sectional area, gross 63.15
hollow, top area −37.06
hollow, bottom area
4.182 × 10.50 = −43.91
4.182$^{\phi}$ = −13.74
−57.65 } −47.36 mean
15.79
depth × .55
volume 8.685 cub. meters

Total volume of concrete for Shell 462.751 call this 462.75 cub. m

Concrete fillings
Top filling 1 : 4 : 8 mixture
Rectangle 2.90 × 9.30 = 26.97
circule 2.90$^{\phi}$ = 6.61
fillet less 2 @ .30 × .30 = −.18
33.40
depth × .60
20.040
dowels less 4 @ .12 × .12 × .02 = −1.152
18.888 cub. meters
Intermediate filling 1 : 4 : 8 mixture
Sectional area of filling 33.04
depth × .60
19.824 cub. meters

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MADE BY M. Jinda DATE 8-5-11 FILE NO
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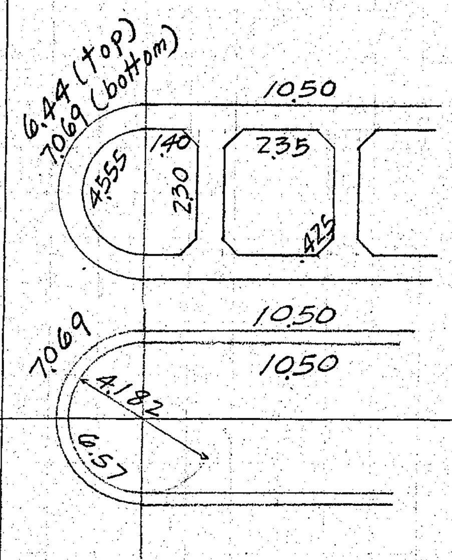
CALCULATIONS FOR

materials of kisogawa-Bashi for Gifu-Ken

Total volume of 1 : 4 : 8 concrete fillings 38.712 call this 38.71 cub. m

Bottom filling 1 : 2 : 4 mixture
Sectional area of filling 33.04
depth × 3.30
109.032
47.36 × .55 = 26.048
63.15 × .15 = 9.473
144.553 call this 144.55 cub. m.

Forms
Outside 2 c 10.50 × 13.20 = 277.20
" 2 c 6.44 × 13.20 = 170.02
" 2 c 10.50 × 5.80 = 121.80
" 2 c 7.069 × 5.80 = 82.00
Inside top 1.2m 2 c 10.50 × 1.20 = 25.20
" " 2 c 5.50 × 1.20 = 13.20
" next 0.6m 2 c 13.25 × .60 = 15.90
" " 2 c 5.50 × .60 = 6.60
" Int. part. 2 c 11.00 × 17.00 = 374.00
" End part. 2 c 10.505 × 17.00 = 357.17
" 2 c 10.50 × .85 = 17.85
" 2 c 6.57 × .85 = 11.17
1472.11 sq. meters.



Reinforcements, plain bars 22.251 Kg tons

curb shoe 3.364 Kg tons

Sand filling
Lot a 2.90 × 5.90 = 17.11
fillet less 4 c .3 × .3 = - 0.36
16.75 × 0.60 = 10.1
Lot b, c, & d 33.04 × 12.50 = 413.0
423.1 cub. m.

Excavation
Bottom area of well 63.15 m²
Depth of sinking say 19.5 m
Excavation 63.15 × 19.5 = 1,231.4 cub. meters

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CALCULATIONS FOR

MADE BY M. Jinda DATE 8-5-11 FILE NO
CHECKED BY K. Inaba DATE 8-5-19 PAGE NO M9

Materials of Kisogawa - Bashi for Gifu-Ken

Materials of 20.0 meter well

Concrete 1:2:4 mixture for shell

same part as for 19.5 meter well 462.751 cub. m.
for extra portion, add 23.21 m^2 × .50 = 11.605
474.356 call this 474.36 cub. meters

Concrete fillings, 1:4:8 mixture, same as for 19.5 m well 38.71 cub. meters
1:2:4 mixture, " " " 144.55 cub. meters

Forms

same part as for 19.5 meter well 1472.11
Inside, add 2 c 11.000 × .50 = 11.00
2 c 10.505 × .50 = 10.51
Outside, add 2 c 10.50 × .50 = 10.50
2 c 6.44 × .50 = 6.44
1,510.56 sq. m.

Reinforcements, plain bars 22.719 Kg tons

Curb shoe 3.364 Kg tons

Sand filling

same part as for 19.5 m well 423.1
for extra portion, add 33.04 × 0.5 = 16.5
439.6 cub. meters

Excavation

63.15 × 20.0 = 1,263.0 cub. meters

昭和五年九月

岐阜縣

揖斐長良川橋比較設計書

公道橋。電車橋及併用橋

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MADE BY K. Inaba DATE 5-9-14 FILE NO

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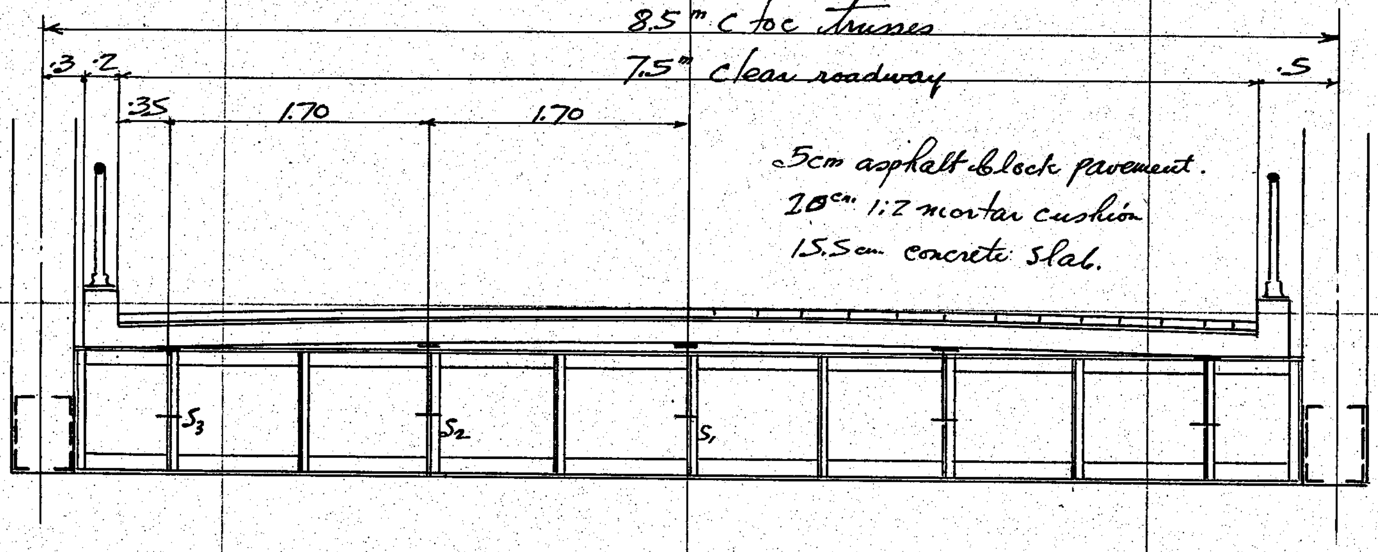
CALCULATIONS FOR

Ibi and Nagara Highway Bridges for Gifu ken.

WE have decided as follows.

Effective width of Roadway = 7.50 meters between curb lines.
Span length = 64.00 meters c to c bearings.
Panel length 10 panels @ 6.40 "
Center to center of trusses
Loading 2nd Class loadings.
Seismic acceleration 3000 $^{mm}/_{sec^2}$

Cross section of Bridge is as shown on sketch below.



Design of Floor slab. span length 1.70 meters.

Dead Load.

5cm	asphalt block pav.	@ 21 kg	=	105
2cm	cement mortar cushion	@ 17	=	34
15.5cm	Concrete slab	@ 24	=	372
	miscellaneous concrete say			9
				520 kg/m²

Dead Load moment $= \frac{1}{10} \times 520 \times 1.7^2 = 150$ kgm
Dead Load shear $= \frac{1}{2} \times 520 \times 1.7 = 442$ kg.

Live Load

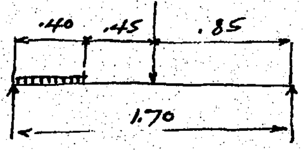
motor truck rear wheel concentration	3,000
30% impact	900
	3,900 kg.

Distribution of wheel concentration on slab.

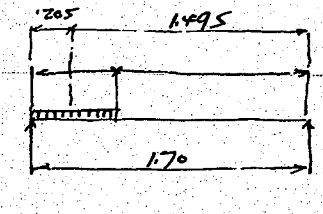
~~Longitudinal~~ tranoverse distribution $b = 27 + 2 \times 7 = 41$ cm
Effective width $\Sigma = \frac{2}{3} l + \underset{a}{\cancel{b}} = \frac{2}{3} \times 1.70 + .34 = 1.47$ m

Longitudinal distribution
contact 20
Distribution $2 \times 7 =$ 14
.34 cm

Load per meter strip of slab $= 3900 \div 1.47 = 2650$ kg
Unif. live load 500 kg per sq. m.



motor truck loading $\frac{2650}{2} \times .85 = 1125$
unif. load $\frac{500 \times .4^2}{2 \times 1.7} = 23.5 \times .85 = 20$
1145



For continuity of slab, moment $= 0.8 \times 1145 = 915$ kgm.

End shear $= 2650 \times \frac{1.495}{1.7} = 2330$ kg

Summary of moments and shears.

	moments	shears
Dead Load	150	442
Live Load	915	2330
	1,065 kgm	2772 kg

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CALCULATIONS FOR

Gifu Ken Ibi and Nagara Highway Bridges.

Effective depth required $= \sqrt{\frac{1065 \times 100}{100 \times 7.18}} = 12.2$ cm

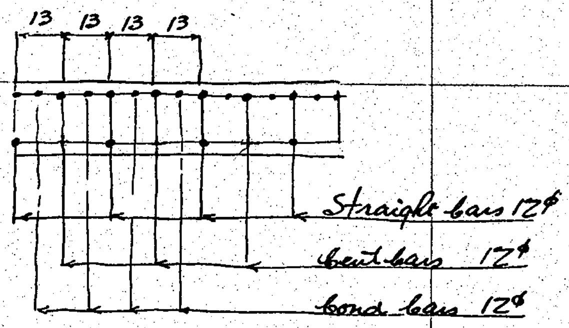
use effective depth of 12.5 cm with 3 cm insulation on tension side total depth = 15.5 cm

Steel area required $= \frac{1065 \times 100}{1200 \times \frac{7}{8} \times 12.5} = 8.12$ cm² per lin meter of slab.

use 12 mmφ bars at 13 cm c to c = 8.70 cm² per meter strip.

steel ratio $p = \frac{8.70}{100 \times 12.5} = .0071$, $k = .367$, $j = .878$

unit shear $= \frac{2772}{100 \times .878 \times 12.5} = 2.53$ kg/cm² ok.



Total perimeter of bars.

main reinforcements $7.70 \times 3.77 = 29.0$

bond bars $3.85 \times 3.77 = \underline{14.5}$

43.5 cm

unit bond $= \frac{2772}{43.5 \times .878 \times 12.5} = 5.8$ kg/cm² ok.

Design of stringers S_1 and S_2. Span length 6.40 meters, spacing 1.70 meters.

Dead Load

floor slab and pavement $520 \times 1.70 = 885$

beam assumed $\underline{95}$

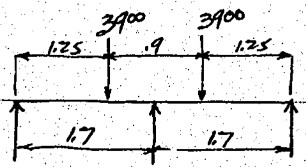
980 kg per lin meter.

Dead Load moment $= \frac{1}{8} \times 980 \times 6.40^2 = 5020$ kgm.

Dead Load end shear $= \frac{1}{2} \times 980 \times 6.40 = 3,135$ kg

Live Load

motor truck loading rear wheel $2 \times 3900 \times \frac{1.25}{1.70} = 5740$ kg.

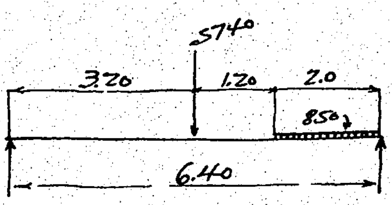


Uniform live load $= 500 \times 1.70 = 850$ kg per lin m.

motor truck rear wheel $5740 \div 2 = 2870$

unif. load $\frac{850 \times 2.0^2}{2 \times 6.4} = \underline{265}$

3135



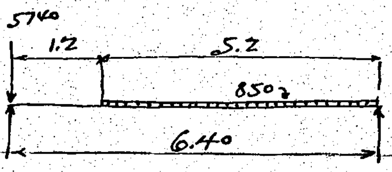
Moment $= 3135 \times 3.20 = 10,030$ kgm.

End shear

unif load $\frac{850 \times 5.2^2}{2 \times 6.4} = 1,795$

rear wheel $\underline{5740}$

7535 kg



Summary of moments and shears.

	moments	end shears
Dead Load	5,020	3135
Live Load	10030	7535
	15,050 kgm	10670 kg

Section modulus required $= \frac{15050 \times 100}{1100} = 1368$ cm³

Use I beam 450 × 175 @ 91.66 kg

S.m. = 1743 cm³ ok.

unit shear on web

$= \frac{10670}{45 \times 1.1} = 21.55$ kg/cm² ok.

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3

CALCULATIONS FOR

Gifu Ken Ibi and Nagara Highway Bridges.

Design of Stringers
Dead Load.

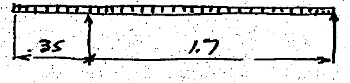
S_3 Span length = 6.40 meters, spacing between $S_2 + S_3$ = 1.70 meters.

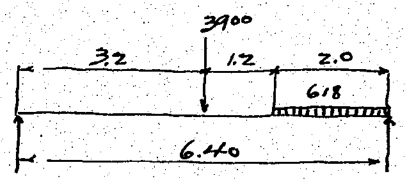
floor and pavement between $S_2 + S_3$	$520 \times .85$ =	442
overhanging slab pav. coping + handrail say		400
beam assumed		75
Extra load due to overhanging moment = $\frac{130}{1.7}$	=	77
		994 kg per lin m.

Dead Load moment = $\frac{1}{8} \times 994 \times 6.40^2 =$ 5080 kgm.
Dead Load shear = $\frac{1}{2} \times 994 \times 6.40 =$ 3,180 kg

Live Load.

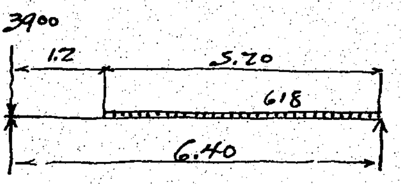
One rear wheel of motor truck directly on stringer assumed.
uniform live load
$\frac{500 \times 2.05^2}{2 \times 1.7} =$ 618 kg per lin m.





uniform load $\frac{618 \times 2.0^2}{2 \times 6.40} =$ 193
rear wheel 3900 ÷ 2 1950
2,143 kg

moment = 2143 × 3.20 = 6860 kgm.



End shear.
unif. load $\frac{618 \times 5.2^2}{2 \times 6.4} =$ 1,305
rear wheel = 3900
5205 kg

Summary of moments and shears.

	moments	Shears.
Dead Load	5080	3,180
Live Load.	6860	5205
	11,940 kgm	8,385 kg

Section modulus required = $\frac{11940 \times 100}{1100} = 1,085\ cm^3$

Use I beam 400 × 150 @ 72.01 kg
S_m = 1,199 cm^3 ok

unit shear on web = $\frac{8385}{40 \times 1} = 210\ kg/cm^2$ ok

Design of Intermediate floor beam. Span length 8.50 meters, spacing = 6.40 meters.
Dead Load.

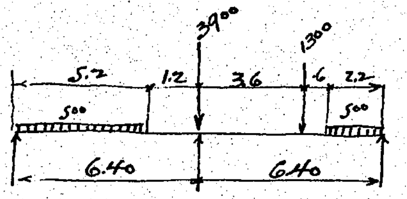
floor slab and pavement	6.40×520 =	3,330
Stringers	$435 \times 6.4 \div 8.5$ =	328
floor beam assumed		200
		3858

Call this 3860 kg per lin meter.

Dead Load moment = $\frac{1}{8} \times 3860 \times 8.5^2 =$ 34900 kgm
Dead Load shear = $\frac{1}{2} \times 3860 \times 8.5 =$ 16,400 kg.

Live Load.

wheel load concentration on floor beam (approximately.)
1300 × 2.8 ÷ 6.4 = 570
3900
4,470 kg



unif. load
$\frac{500 \times 2.2^2}{2 \times 6.4} =$ 189
$\frac{500 \times 5.2^2}{2 \times 6.4} =$ 1057
1246 call this 1250 kg per lin m.

unif. load on side of motor trucks 7.50 − 5.40 = 2.10
500 × 6.4 = 3200 kg per lin m
3200 − 1250 = 1950

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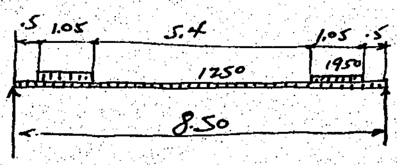
CALCULATIONS FOR

Gifu Ken Ibi and Nagara Highway Bridges

moment due to uniform live load.



$1950 \times 1.05 = 2045$

moment.

$2045 \times 4.25 = 8690$

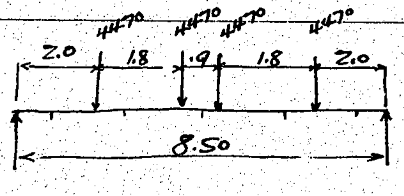
$2045 \times 3.225 = -6600$

2090

$1/8 \times 1250 \times 8.5^2 = 11,290$

$13,380$ kgm.

moment due to motor truck concentrations



Reaction $4470 \times 2 = 8940$ kg

moment

$8940 \times \overset{3.80}{\cancel{4.35}} = 33,950$

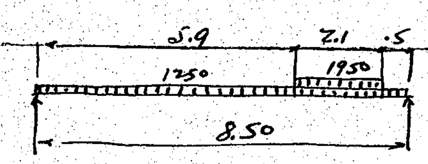
$4470 \times 1.80 = -8050$

$25,900$ kgm.

13380

Total L.L. moment $= 39,280$ kgm

End shear.

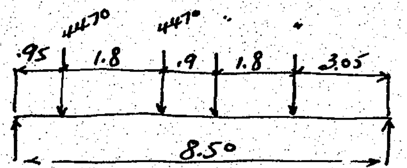


unif load.

$\frac{1950 \times 2.1 \times 1.55}{8.5} = 746$

$1250 \times 4.25 = 5310$

6056 kg. call this 6060 kg.



motor truck.

$4 \times 4470 = 17880$ kg

$17,880 \times \frac{5.3}{8.5} = 11,150$ kg.

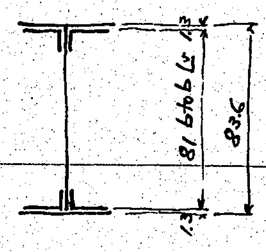
6060

Total L.L. shear $= 17,210$ kg

Summary of Dead Load and Live Load moments and shears.

	moments	end shears.
Dead Load	34,900	16,400
Live Load	39,280	17,210
	74,180 kgm	33,610 kg.

Assumed section



Try web plate 800×9 $1/8$ web area $= 9.0\ \text{cm}^2$ back to back of Ls $= 81.0$ cm

effective depth say $83.6 - 2 \times 2.15 = 79.3$ cm.

Flange stress $= \frac{74180 \times 100}{79.3} = 93500$ kg T or C

Flange area required for tension $= \frac{93500}{1200} = 77.90\ \text{cm}^2$ net

$1/8$ web area $- 9.00$

$68.90\ \text{cm}^2$ net.

Use 2 Ls $150 \times 100 \times 9 = 43.38 - 9.0 = 34.38$

1 Pl. $340 \times 12 = 40.80 - 6.0 = 34.80$

$69.18\ \text{cm}^2$ net ok.

Shear on web $= \frac{33610}{72.0} = 467\ \text{kg/cm}^2$

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CALCULATIONS FOR

Gifu Ken, Ibi and Nagara Highway Bridges.

Approximate weight of floor beam.

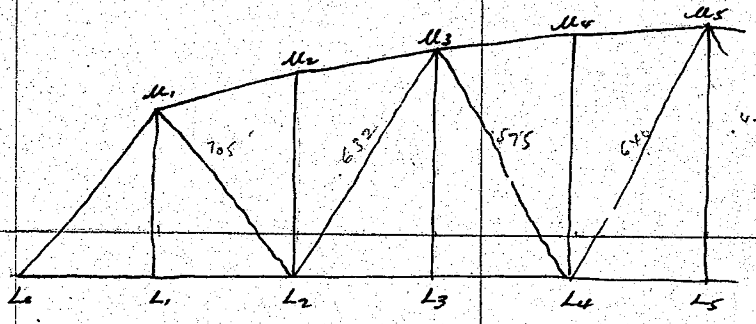
1 web pl.	800 × 9	@ 56.52 kg	× 8.10 =	458
4 flg Ls	150 × 100 × 9	@ 17.02	× 8.10 =	552
2 cov. pls	340 × 12	@ 32.03	× 6.50 =	415
				1425
	Details say 30%		=	425
				1,850 kg

For End floor-beam, weight assumed 1700 kg.

Approximate Dead panel load on truss.

floor slab and pavement between S_3.	520 × 6.8 =	3540
Overhanging slab, coping + handrail.	2 × 400 =	800
Stringers $S_1 + S_2$	3 × 95 =	285
〃 S_3	2 × 75 =	150
floor beam	1850 ÷ 6.4 =	390
bottom lateral bracings say		120
top lateral bracings with portals + sways. say		315
main trusses say	128000 ÷ 64 =	2000
		7,600 kg per lin meter.

Panel concentration = 7600 × 6.40 = 48,600 kg for 2 trusses
24300 〃 〃 one truss.



Dead Load stresses of Truss members.

Members	Dead panel load	Stress due to unity	D.L. Stress	Live panel load	L.L. Stress unif. load	Single con.	Total stress.
L_0-U_1	24300 kg	5.89 ✓	143,200	10300	60,700	8,100	212,000 C
U_1-U_2	〃	5.78 ✓	140,500	〃	59,500	8,000	208,000 C
U_2-U_3	〃	5.75	139,700	〃	59,200	8000	206,900 C
U_3-U_4	〃	7.16 ✓	174,000	〃	73,700	9,900	257,600 C
U_4-U_5	〃	7.11.	172,700	〃	73,300	9900	255,900 C
L_0-L_1	〃	3.79 ✓	92,100	〃	39,000	5,400	136,500 T
L_1-L_2	〃	〃	〃	〃	〃	5,400	〃 T
L_2-L_3	〃	6.62 ✓	161,000	〃	68,200	9100	238,300 T
L_3-L_4	〃	〃	〃	〃	〃	9100	〃 T
L_4-L_5	〃	7.28 ✓	177,000	〃	75,000	10,000	262,000 T
U_1-L_1	〃			〃			
U_2-L_2	〃			〃			
U_3-L_3	〃			〃			
U_4-L_4	〃			〃			
U_5-L_5	〃			〃			
U_1-L_2	〃	2.870 2.87	69,700	〃 3.17	32,700	4900	107,300 T
L_2-U_3	〃	1.850 1.85	45,000	〃 2.54	26,200	4400	75,600 C
U_3-L_4	〃	.990 .99	24,100	〃 2.01	20,700	4000	48,800 T
U_4-U_5	〃	.310 .31	7,500	〃 $\begin{cases}1.59\\-1.07\end{cases}$	$\begin{cases}16,400\\-11,000\end{cases}$	$\begin{cases}4400\\-3,000\end{cases}$	$\begin{cases}28,300\ C\\6,500\ T\end{cases}$

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CALCULATIONS FOR

Gifu Ken, Ibi and Nagara Highway Bridges.

Live panel load.

Uniform live load $= \frac{100000}{170+64} = 428$ call this 430 kg/m²

panel load due to full unif. load $= 3.75 \times 430 = 1610$ kg/m for one truss.

$1610 \times 6.4 = \underline{10300}$ kg

Extra single concentration due to rear wheel of motor truck

$17210 - 10300 = 6910$ call this $\underline{\underline{6900}}$ kg

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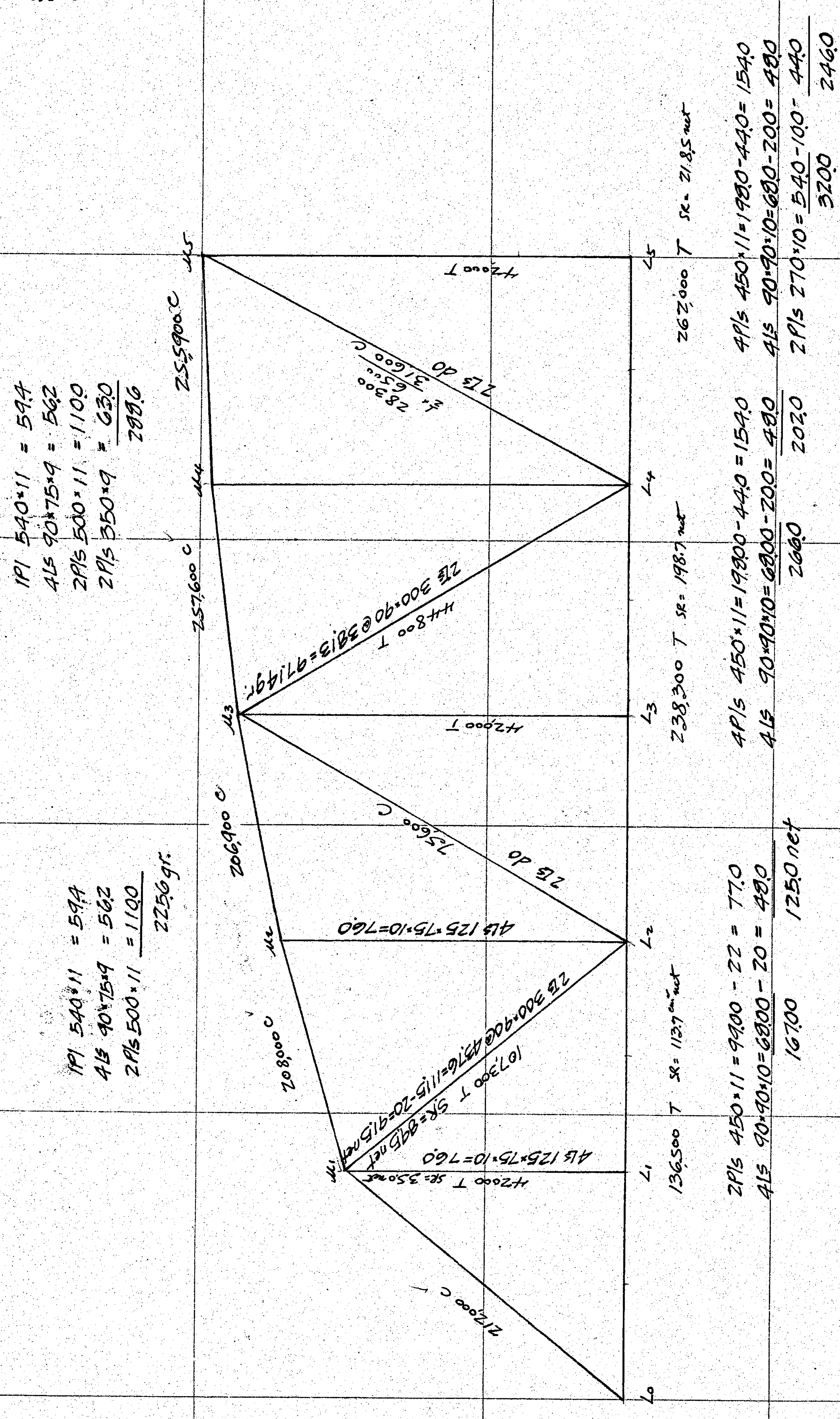
(7)

CALCULATIONS FOR

Gifu Ken Ibi and Nagara Highway Bridges.

Stress sheet



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CALCULATIONS FOR

Ibi and Nagara Bashi for Gifu-Ken, Highway Design.

Materials of slab

Cross sectional area of slab

Coping	2@.20 × .375 = .150
slab	.155 × 7.50 = 1.160
fillet	.015 × .165 = .002
	1.312 m^2

Slab length .... 65.00 meters

1.312 × 65.00 = 85.3 cub. meters

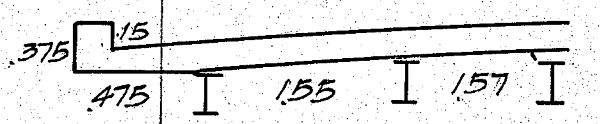
Forms width .... 8.24 meters

8.24 × 65.00 = 537.0 sq.m

less top of floor beam

20@.21 × .475 = -1.99

535.01 sq.m



人造洗出仕上

2@.75 × 65.0 = 97.5 sq.m

Reinforcements 25 kgs/m^2 assumed

25 × 7.9 × 65.0 = 12,800 kgs or 12.8 kg.tons

Construction joint 3/8" carey elastite 3ヶ所

Drain - 12ヶ 30 kgs for one drain

Handrails 85 kgs per lin. meter assumed

2@85 × 65.00 = 11,000 kgs or 11.0 kg tons

asphalt block pavement with 1:2 cement mortar cushion

7.5 × 65.00 = 487 sq.m

Materials for one span

Concrete	85.3 cub. m
Forms	535.01 sq.m
asphalt block pavement	487.0 sq.m
人造洗出仕上	97.5 sq.m
Reinforcements	12.8 kg tons
Handrails	11.0 kg tons
Construction joints	3 ヶ所
Drain	12ヶ ------- 12@30 kgs

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CALCULATIONS FOR

Gifu Ken Ibi and Nagara Bashi

Approximate weight of Truss.

members.	gross area	unit wt.	Length	weight
L_0-U_1	288.6	.785	9.94	2250
U_1-U_3	225.6	,	13.05	2315
U_3-U_5	288.6	,	12.82	2900
L_0-L_2	167.0	,	12.80	1680
L_2-L_4	266.0	,	12.80	2670
L_4-L_5	320.0	,	6.40	1610
$U_1-L_2+L_2-U_3$	111.5	,	21.94	1920
$U_3-L_4+L_4-U_5$	97.14	,	24.74	1890
verticals	76.00	,	43.12	2575

19810 × 2 = 39620

Details say 45% = 17830

57450 kg

Call this 57.500 kg.

Approximate weight of steel in one span.

Stringers 3 @ 95 = 285
" 2 @ 75 = 150
435 × 65.0 = 28,300

Lower lateral bracings 120
Upper lateral bracings sway + portal 350
470 × 64.0 = 30,100

Floor beams. 9 @ 1850 = 16,650
" 2 @ 1700 = 3400

trusses 2 @ 57.500 = 115,000

193450

Shoes and expansion joint say 4550

198,000 kg

or 198.00 kg tons.

Estimate of Cost for Superstructure.

Structural steel	198.00 kg tons.	@ 190.00	160	37,600	31,700
floor concrete 1:2:4 mix.	85.3 m³	@ 14.00	11.5	1,195	980
Forms	535 m²	@ 18.00	1.5	962	800
pavement asphalt block 5cm	487 "	@ 44.00	4.3	2,335	2090
人造 石	98 "	@ 44.00	3.3	392	320
Reinforcements plain bars	12.8 kg tons	@ 100.00		11,280	
handrails cast iron + pipe	11.0 "	@ 280.00		3,080	
Construction joints	3 "	@ 20.00		60	
drains cast iron	12 "	@ 8.00		96	
Electric wiring	65	@ 7.00		455	
				47,455	call this 47,500

Highway Design. 6 @ 47,500 = 285,000
7 @ 47,500 = 333,000

Super 40,860 × 7 = 286,000
Sub 140,000
426,000
34,000
460,000

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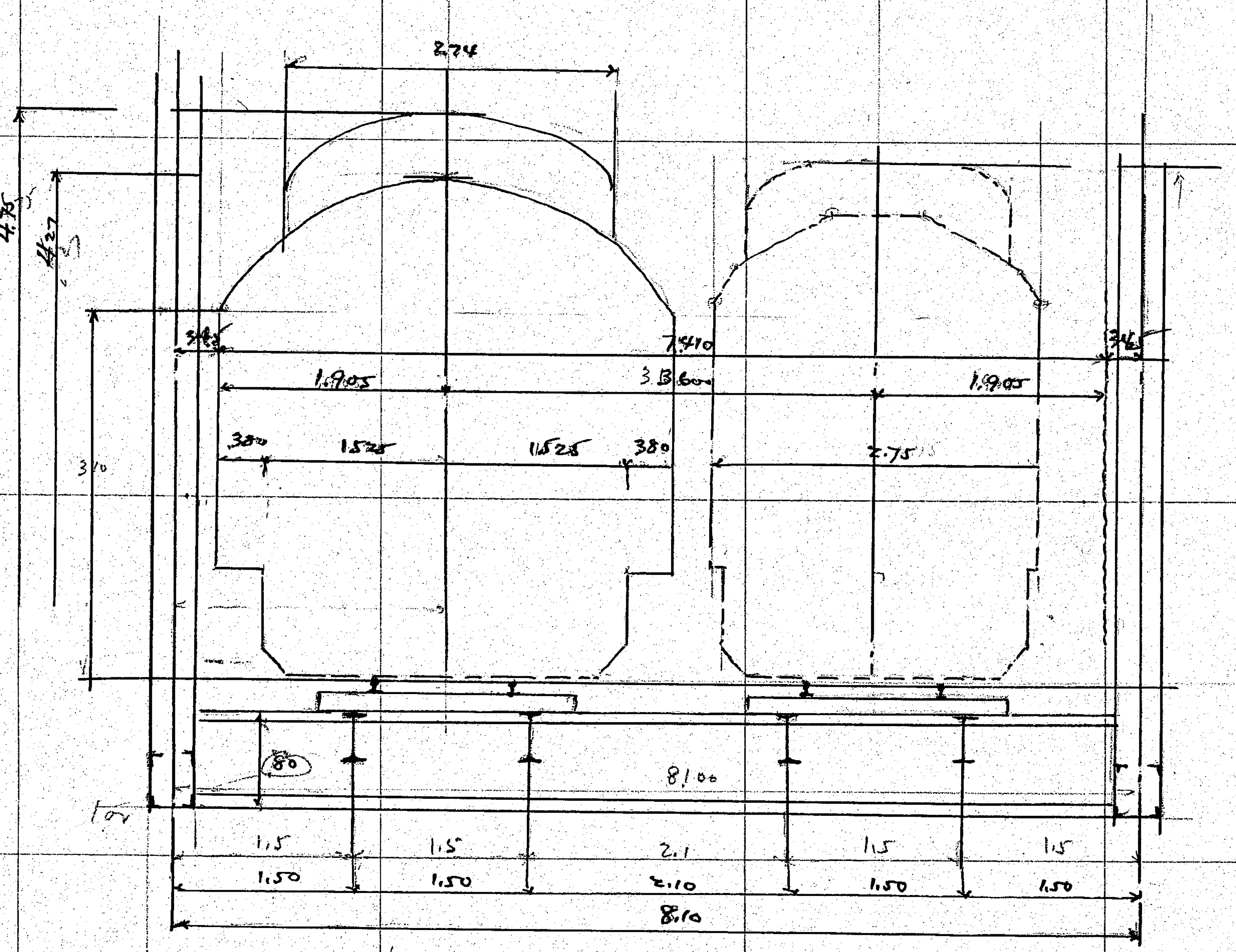
CALCULATIONS FOR

Gifu-Ken Ibi-nagaragawa Bashi Railway Design.

Loading K12 double track span length of bridge 64.0 meters between end pins.

Cross section of bridge as shown below.



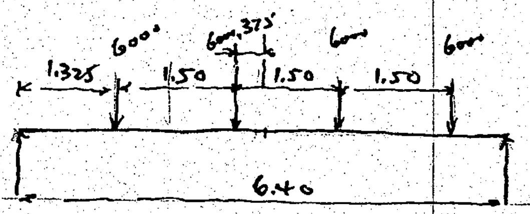
7.410
.700
8.110

8.600
7.410
.690

3' 6½"

1.16 m

Stringer span length 6.4 meters



Reaction $4 \times 6000 \times \frac{2.825}{6.400} = 10600$

moment $= 10600 \times 2.825 = 30.000$

$6000 \times 1.5 = \underline{-9.000}$

21.000 kg meters

impact $I = \frac{45}{45 + 6.4} = 87.5\%$ $\underline{18400}$

39400 kgm

Dead Load ~~400~~ kg per lin. meter of track

Including ~~stringers~~ bracings say 250 kg per meter of one rail.

Dead Load moment $= \frac{1}{8} \times 250 \times 6.4^2 = 1280$ kgm

Live Load moment $\underline{39400}$

40680

Section modulus reqd approx $= \frac{4068000}{1100} = 3700$

web assumed $800 \times 9 = \underline{72.}$ 1/8 web $= 9.00$ cm back to back of angles 81.0 cm

flange assumed 2L 150 × 100 × 9 = ~~43.38~~ − 9.0 = 34.38 net

Effective depth $81.0 - 4.6 = .764$

flange stress $\frac{40680}{.764} = 53300.$

Section reqd $53300 \div 1200 = 44.5$

$\underline{9.0}$

35.5 net

Gifu-Ken Hi-nagara gawa Bashi Railway Design

approximate weight of stringers one track
one stringer

1 web	800 × 9	@	56.52	×	6.4	=	362.0
4 Ls	150 × 100 × 9	@	17.02	×	6.4	=	435.0
4 Ls	150 × 100 × 9	@	17.02	×	0.8	=	35.0
4 Pls.	150 × 9	@	10.60	×	0.5	=	21.0
8 Ls	125 × 90 × 10	@	16.09	×	0.8	=	103.0
							956.0
	3½% variation						34.0
							990.0 kg

Lateral bracings

6 Ls	90 × 90 × 10	@	13.34	×	1.50	=	120.0
1 L	90 × 90 × 10	@	13.34	×	1.30	=	17.0
connections	8 @ 10					=	80.0
	misc say						40.0
							317.0

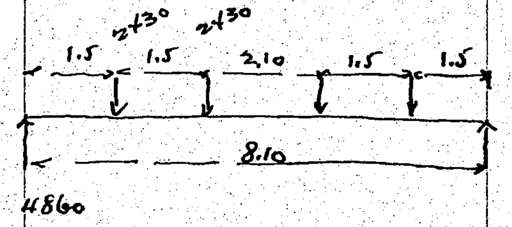
Summary

2 stringers 2 @ 990 = 1980
Laterals 317
2297 kg

2297 ÷ 6.4 = 360 kg per lin. meter

For 2 tracks 2 × 360 = 720 kg per lin. meter

Design of floor beam span length 8.10 meters spacing 6.4 meters
Dead Load.



track per rail 200
steel - 180
380 × 6.4 = 2430

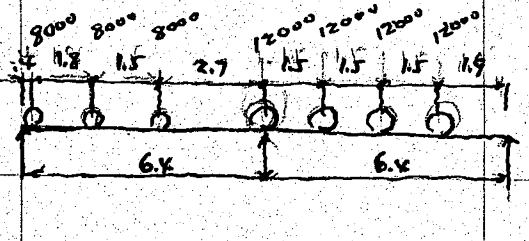
Girder assumed 250 kg per lin meter

moment due to concentration 4860 × 3.0 = 14600
2430 × 1.5 = - 3600
9000 kgm

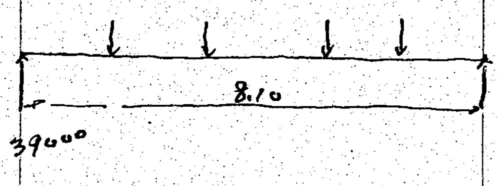
weight of girder $\frac{1}{8} \cdot 250 \cdot 8.10^2$ = 2040
11,040 "

Live Load

Load on floor beam per track
12000 × 16.6 = 199,000
8000 × 6.3 = 50400
249400 ÷ 6.4 = 39000 kg.



Impact = $\frac{45}{45+12.8}$ = 78 %



moment 39000 × 3.0 = 117000
$\frac{39000}{2}$ × 1.5 = - 29200
87800 kgm
Impact 78% 68400
Total live load moment 156200 kgm
Dead load moment 11040
167240 kg. m

Depth of beam assumed 1000 × 9 = 90.0 □cm $\frac{1}{8}$ web = 11.25 □
back to back of Ls 101.0 cm

2 Ls 150 × 150 × 19 = 106.78 - 19.0 = 87.78
1 Pl 350 × 9 = 31.50 - 4.5 = 27.00
1 Pl. 350 × 9 = 31.50 - 4.5 = 27.00
169.78 141.78

Effective depth say .95 flange stress = $\frac{167240}{.95}$ = 176500
Section reqd = 176500000 ÷ 1200 = 147.0
11.0
136.0 net

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CALCULATIONS FOR

Gifu-Ken Hi-nagara Gawa Bashi Railway Design

approximate weight of floor beam

4 Ls	150 × 150 × 19	@	41.91	× 7.7	=	1290
1 Pl.	1000 × 9	@	70.65	× 7.7	=	543
2 Pl	350 × 9	@	24.73	× 6.0	= say	300
2 Pl	350 × 9	@	24.73	× 4.0	= say	200
4 Ls	150 × 100 × 12	@	22.41	× 1.0	=	90
4 Pls.	150 × 19	@	22.37	× 0.7	=	62
18 Ls	125 × 90 × 10	@	16.09	× 1.0	=	290
8 Pls.	say					160
						2935

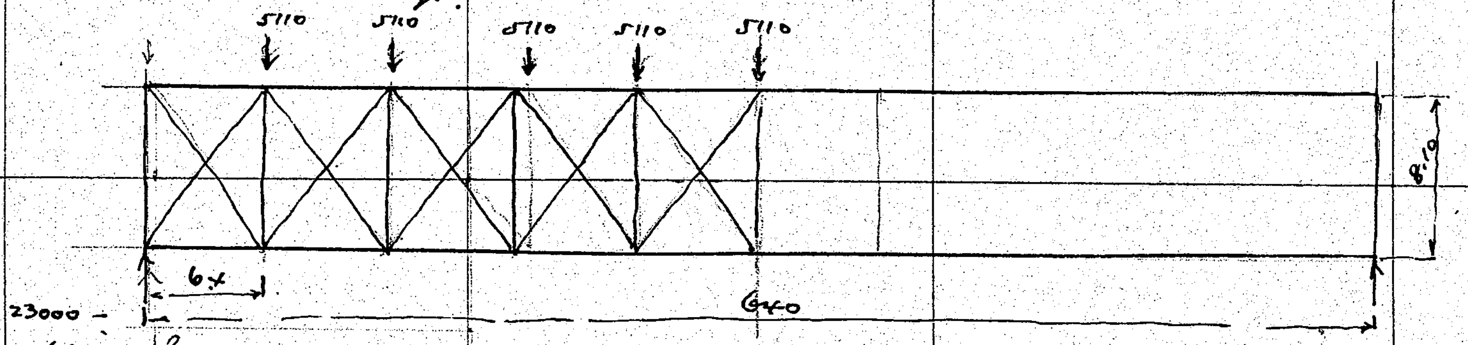
call this ~~2950~~ 3000 kg.

3000 ÷ 7.7 = 390 kg per lin. meter of girder

3000 ÷ 6.4 = 470 kg per lin. meter of span

End floor beam assumed – 2500 kg.

Lower Lateral Bracings.



$8.10^2 = 65.5$
$6.40^2 = 41.0$
$106.5 - 10.3$

$\frac{10.3}{8.10} = 1.275$

Wind load

200
600
800 × 6.4 = 5110

stress = 23000 × 1.275 = 29300

for one member 29300 ÷ 2 = 14650 kg.

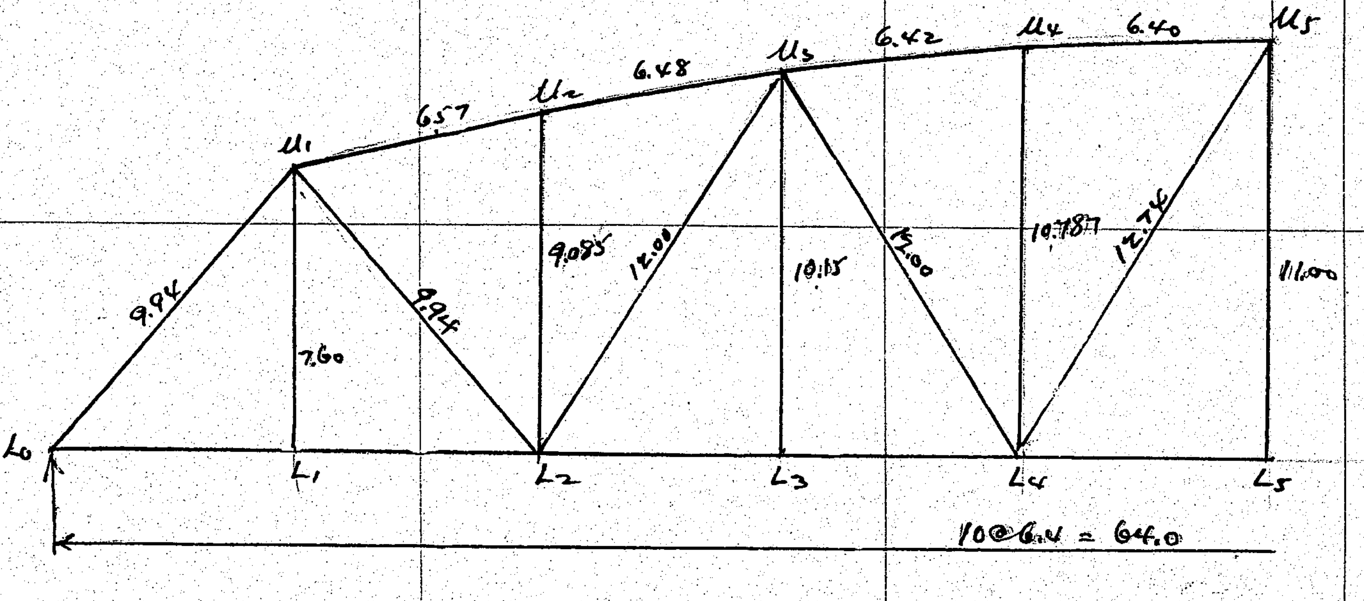
approximate weight of lower lateral bracings 110 kg per lin. meter.

Top Lateral Bracing assumed 350 kg per lin. meter.

approximate of steel

stringers	720
floor beam	470
Bottom Laterals.	110
Top Laterals.	350
trusses assumed	3300
	4950 kg per lin. meter. call this 5000 kg
Deck construction	800
	5800

Panel Concentration for one truss. $\frac{5800}{2} \times 6.4 =$ 18550 kg. say 18600 kg.



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CALCULATIONS FOR

13

Gifu-Ken Ibi and Nagara Bashi Railway Design

Dead Load stresses

L_0-U_1	5.89 ✓	@ 18600	=	109400	C
U_1-U_2	5.78 ✓		=	107200	C
U_2-U_3	~~6.70~~ 5.75		=	107000	C
$U_3 U_4$	7.16		=	133000	C
$U_4 U_5$	7.11		=	132000	C
$L_0-L_1-L_2$	3.79		=	70500	T
L_2-~~L_3~~-4	6.62		=	123000	T
L_4-L_5	7.28		=	135000	T
U_1-L_2	2.87		=	53500	T
L_2-U_3	1.85		=	34400	C
U_3-L_4	0.99		=	18400	T
L_4-U_5	0.31		=	5760	C

Live Load Stresses.

K12. loading. Equivalent Uniform live load
moment at 1/4 point 1779.600 kg meter

Equivalent Uniform load $w = \frac{32}{3} \cdot \frac{1779000}{64^2} = 4625$ for one track

Impact $\frac{45}{45+64} = 41.2\ \%.$ $\frac{1900}{6525}$ kg

Panel concentration for one truss $6525 \cdot 6.4 = 41700$ kg.

Live Load stresses

L_0-U_1	5.89	@ 41700	=	245000	C
U_1-U_2	5.78			240.000	C
U_2-U_3	5.75			239.000	C
U_3-U_4	7.16			298.000	C
U_4-U_5	7.11			296.000	C
$L_0-L_1-L_2$	3.79			158.000	T
L_2-3-L_4	6.62			276.000	T
L_0-L_5	7.28			303.000	T
U_1-L_2	3.17			132.000	T
L_2-U_3	2.54			106.000	C
U_3-L_4	2.01			83500	T
L_4-U_5	1.59			66000	C
U_5-L_6'	1.07			44500	C
L_4-U_3	.57			23800	

Dead Load reaction 2 trusses $5800 \cdot 32.0 = 185500$
Live Load $13050 \cdot 32.0 = 418000$
603500

Design load say 610.0 tons
on one pier $610 \cdot 2 = 1220$ tons

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CALCULATIONS FOR

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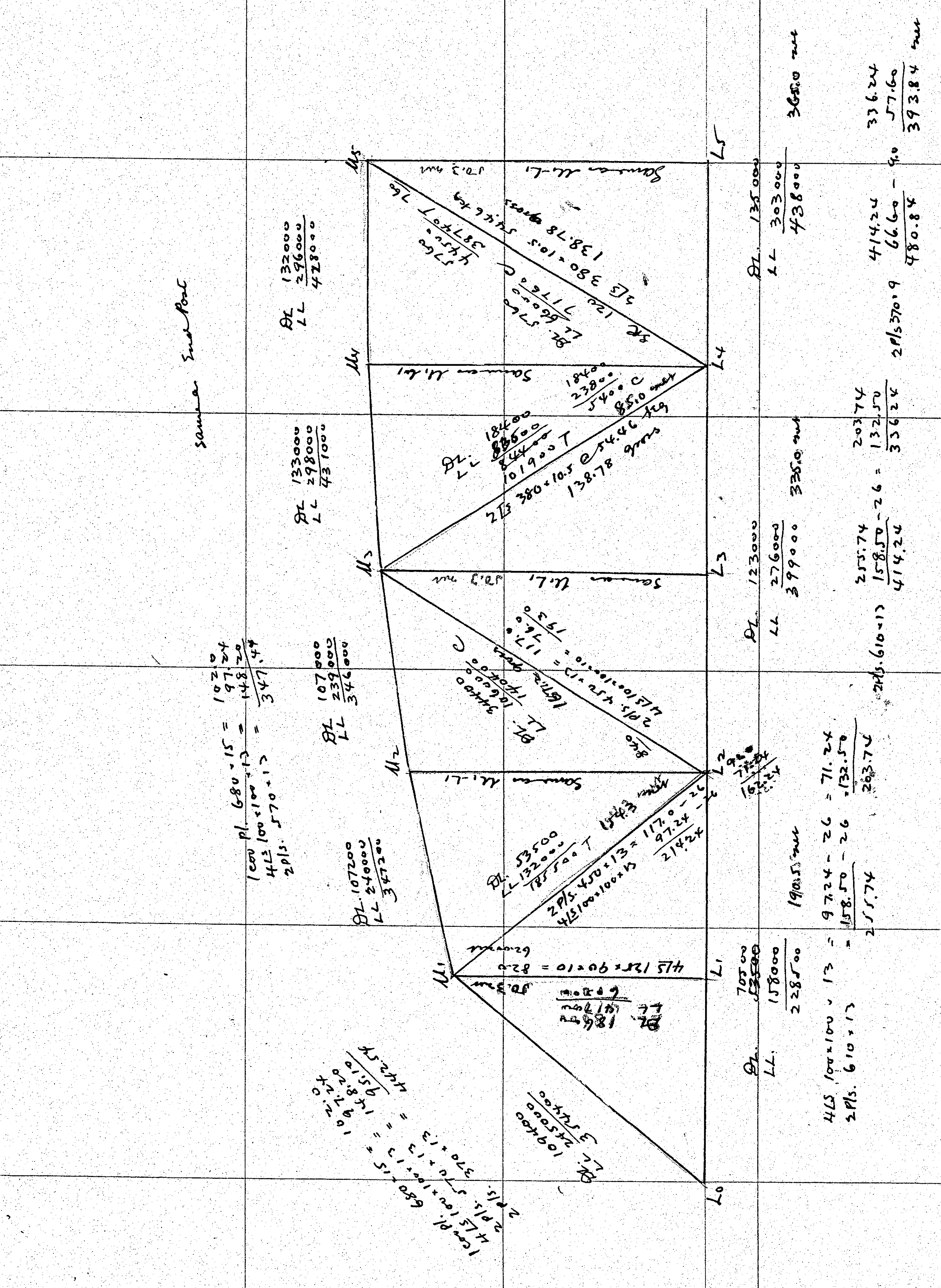
Gifu-Ken Ibi and Nagara Bashi Railway Design

Stresses and sections of truss.



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CALCULATIONS FOR

Gifu-Ken Hi and Nagara Bashi Railway Design

approximate weight of truss.

L_0-U_1	442.54	@ .785 ×	9.94	=	3440
$U_1-U_2-U_3$	347.44	×	13.05	=	3560
$U_3-U_4-U_5$	442.54	×	12.82	=	4450
L_0-L_2	255.74	×	12.80	=	2570
L_2-L_4	414.24	×	12.80	=	4150
L_4-L_5	480.84	×	6.40	=	2420
U_1-L_2	214.24	×	9.94	=	1670
L_2-U_3	193.00	×	12.00	=	1820
U_3-L_4	138.78	×	12.00	=	1305
L_4-U_5	138.78	×	12.74	=	1390
vert	82.00	×	43.11	=	2780

29555 × 2 = 59110

Details ~~45~~ % ~~26600~~

85710

171400
2 @ 85700 = 171400
171400 ÷ 64 = 2700 kg per lin. meter.

Structural Steel in one span

stringers	720 × 65.0	=	46800
floor beam Int.	9 @ 3000	=	27.000
" " End	2 @ 2500		5.000
Bottom lateral			7700
Top laterals & Sway bc			22800
trusses			171400
shoes.			5500
misc			1500
			287700

call this 290 tons per span

Hi qawa	215 ft	6 spans	@ 290	= 1740	tons
Nagara	256 ft	7 spans	@ 290	= 2030	tons

Estimate of cost of Deck construction.

~~Hi~~ nagara qawa 29.5 ton per track 59.0 tons for double track @ 110^{00} = 6490
1500 sleepers. @ 8^{50} 12200
19190
say 19200^{00}

~~nagara~~ Hi qawa double track 508 tons @ 110^{00} = 5588
sleepers 1300 @ 8^{50} 11000
16588 say 16600^{00}

Estimate of cost superstructure

Hi qawa - Steel 1740 tons @ 190^{00} = 330600
Deck construction 16600
347200^{00}

Nagara qawa Steel 2030 @ 190^{00} = 385700
Deck construction 19200
404900^{00}

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CALCULATIONS FOR

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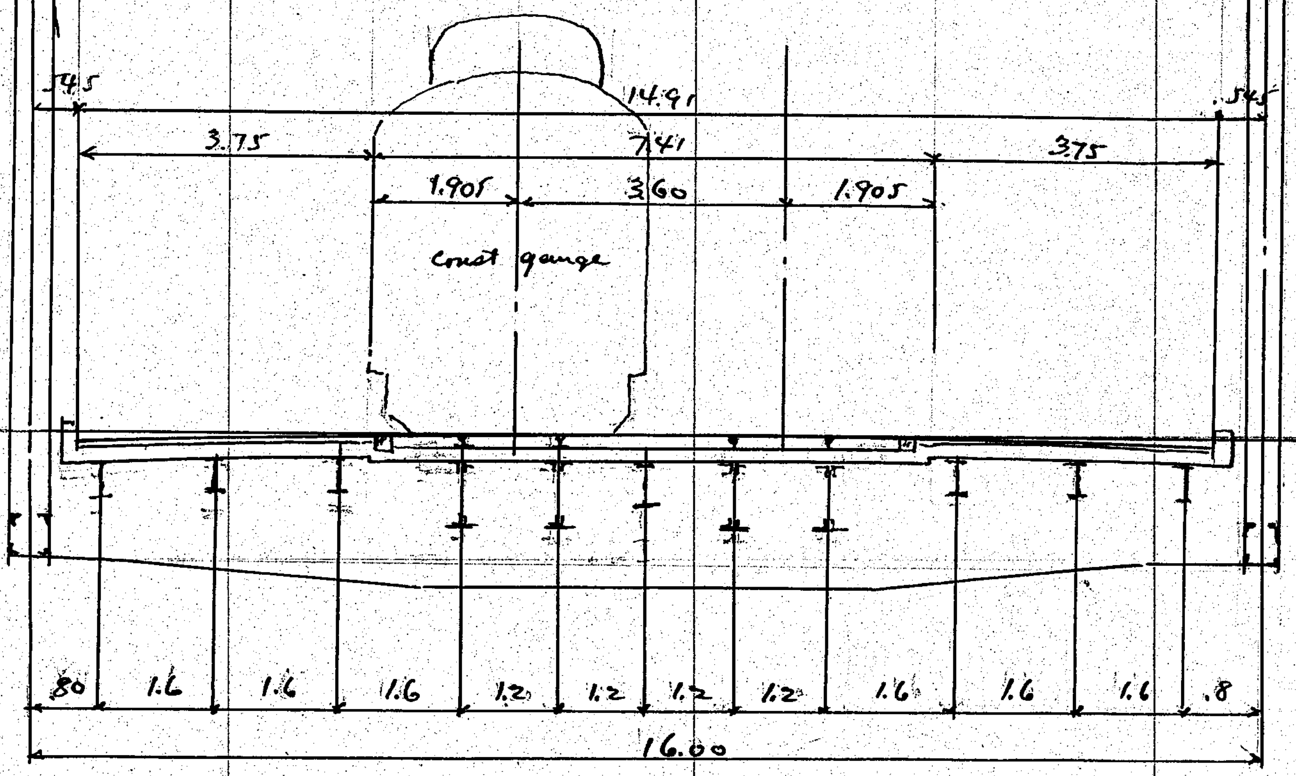
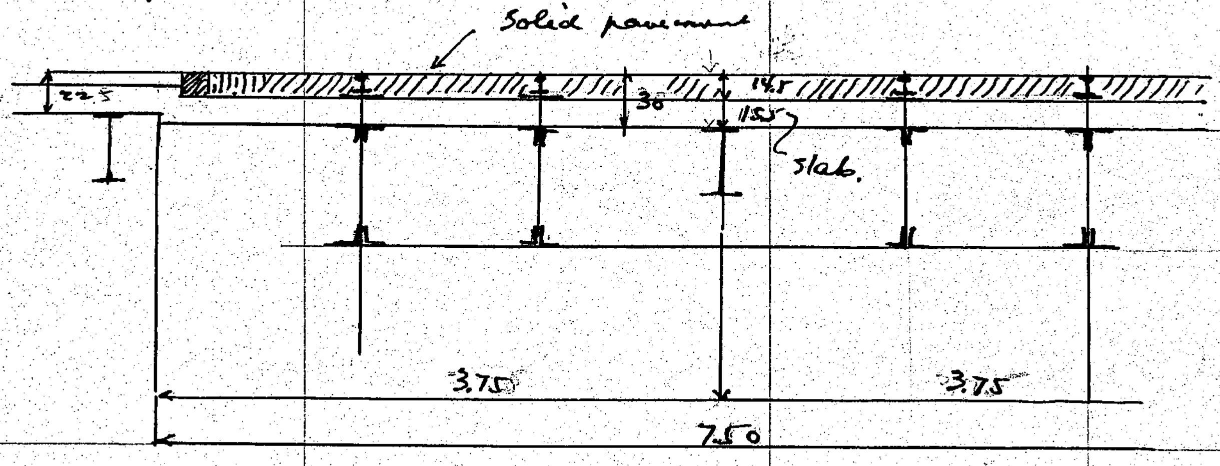
Gifu-Ken Ibi and Nagara Bashi Combined Design

Cross Section of Combined Design



Slab detail around Railway Tracks.



Stringers Highway portion same as Highway design 400 × 150 @ 72.01 kg.
Railway portion assumed same as railway design. 450 × 175 @ 91.66.

Highway portion. 2 @ 75 = 150
4 @ 95 = 380

530

Railway Portion
including center stringer 750

1280 kg per meter

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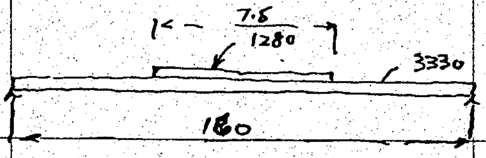
CALCULATIONS FOR

Gifu-Ken Hi ana Nagara Bashi Combined Design

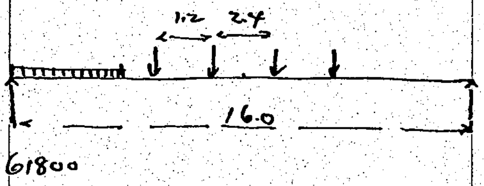
Floor Beam span length 16.0 meters spacing 6.4 meters
Highway flooring 520 kg per square meter
520 × 6.4 = 3330 kg
Extra load at track 8cm @ 24 = say 200 kg per sq meter
200 × 6.4 = 1280 kg 7.5 meter wide at center.

Dead Load

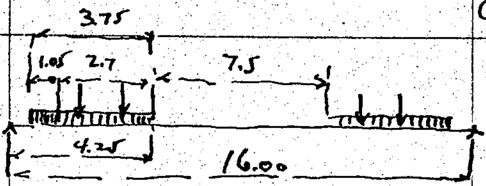


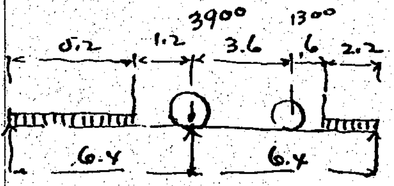
Dead Load moment $\frac{1}{8}$ × 3330 × 16.0^2 = 106500 kgm
Extra load for track $\frac{1}{8}$ × 1280 × 16.0^2 = 41000 kgm
Dead Load beam $\frac{1}{8}$ × 400 × 16.0^2 = 12800 kgm
160300

Live Load



Electric Car loading. 39000 kg per track
impact 78% × .75 / 58.5% 22800
61800 kg per track
moment = 61800 × 6.8 = 420000
$\frac{61800}{2}$ × 1.2 = − 37000
383000 kgm.





Highway loading approximate
front wheel 1300 × 2.8 ÷ 6.4 = 570
rear wheel 3900
4470 kg.
Uniform load at rear and front 1250 kg per meter
Uniform load on sides of motor truck Extra load 2045 kg.

motor truck 2 × 4470 × 2.9 = 26000 kgm
Uniform load
1250 × 3.75 = 4690
moment = 4690 × 2.38 = 11200
37200 kgm

Summary for moments

	Highway	Railway	Sum
Dead Load	112900	47400	160300
Live Load	37200	383000	420200
	150100	430400	580500
	26%	74%	

web assumed 2000 × 13 = 260. $\frac{1}{8}$ web = 32.50 cm
Effective depth say 1.90
flange stress = $\frac{580500}{1.9}$ = 306000 kg
SR = $\frac{30600000}{1200}$ = 255.0
− 32.5
222.5 □ net

2Ls	150 × 150 × 19	=	106.78	− 19.0	=	87.78
2 Side pls	300 × 13	=	78.00	− 13.0	=	65.00
1 Pl	350 × 13	=	45.50	− 6.5	=	39.00
1 Pl	350 × 13	=	45.50	− 6.5	=	39.00
flange			275.78			230.78 net
"			275.78			
web.			260.00			

811.56 @ .785 = 635 kg
Details say 25% 160
795 call this 800 kg. per meter

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18

CALCULATIONS FOR

Gifu-Ken Ibi and Nagara Gawa Bashi Combined Design

approximate 800 × 15.4 = 12,300 kg. 12300 ÷ 6.4 = 1920 kg per meter

Highway 1920 × 26 % = 500 kg per meter
Railway 1920 × 74 % = 1420 " per meter

Lower Lateral Bracing say 230 kg per lin. meter of truss.

Top Lateral Bracing say 800 kg per lin. meter of truss.

End floor beam Highway 1700 kg. 1850 kg including shoe.
Railway 2500 kg. 2650 kg "
4200
Shoes. say 300
4500 —

structural steel. for Highway + Railway

Stringers	1280
Inter floor beam	1920
lower laterals	230
upper laterals Sway etc	800
trusses assumed	4000
	8730

8230
9160
17390 ÷ 2 = 8700 kg. truss

Dead Load floor. slab. 520 × 14.91 = 7750
copings 2@ 250 = 500
H Rails 2@ 80 = 160
Extra. 100 × 7.5 = 750
9160

Panel Concentration 8700 × 6.4 = 55700 kg.

Live Load K12. Equiv. Uniform load 4625 kg.
Impact 41.2 × .75 = 30.9 % 1425
6050 kg per track

Panel Concentration 6050 × 6.4 = 38800 kg.

Dead Load stresses.

L_0-U_1	5.89	@ 55700	=	328000	C
U_1-U_2	5.78		=	322000	C
U_2-U_3	5.75		=	320,000	C
U_3-U_4	7.16		=	399,000	C
U_4-U_5	7.11		=	396,000	C
L_0-L_2	3.79		=	211,000	T
L_2-L_4	6.62		=	366,000	T
L_4-L_5	7.28		=	405,000	T
U_1-L_2	2.87		=	160,000	T
L_2-U_3	1.85		=	103,000	C
U_3-L_4	0.99		=	55,000	T
L_4-U_5	0.31		=	17300	C

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CALCULATIONS FOR

19

Gifu-Ken Ibi and Nagara gawa Bashi Combined Design

Live load stresses
Railway Loading.

Highway Loading motor truck & Unif.

L_0-U_1	5.89	@	38800	=	228.000	C	68800
U_1-U_2	5.78			=	224.000	C	67500
U_2-U_3	5.75			=	223.000	C	67200
U_3-U_4	7.16			=	278.000	C	83600
U_4-U_5	7.11			=	276.000	C	83200
L_0-L_2	3.79			=	147.000	T	44400
L_2-L_4	6.62			=	257.000	T	77300
L_4-L_5	7.28			=	282.000	T	85000
U_1-L_2	3.17			=	123.000	T	37600
L_2-U_3	2.54			=	98500	C	30600
U_3-L_4	2.01			=	78000	T	24700
L_4-U_5	1.59			=	61600	C	20800
U_5-L_4	1.07			=	41500	₵	13000
L_4-U_3	.57			=	22000	Ŧ	

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CALCULATIONS FOR

Gifu-Ken Ibi and Nagara Bashi Combined Design

Stresses + Sections

1 cov Pl. 800 × 17 = 136.0
2 Pls. 700 × 19 = 266.0
4 Ls 150×150×19 = 213.56
615.56

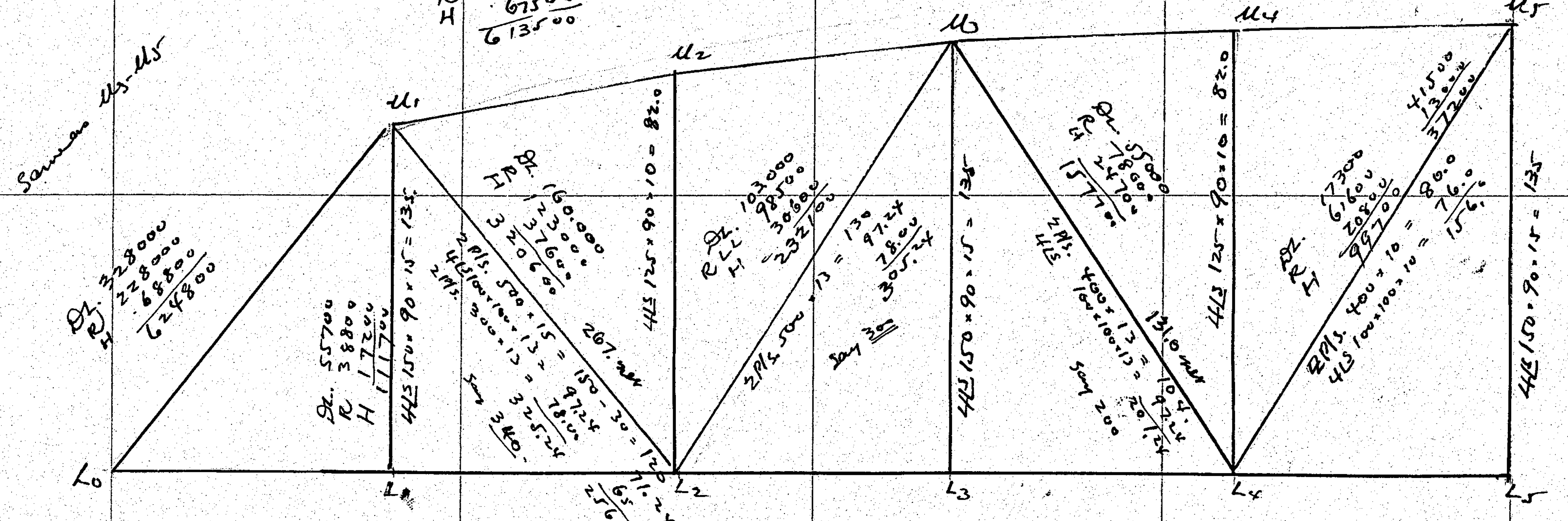
add 2 Pls. 400 × 19 = 615.56
152.0
767.56

Dz. 322000
R 224000
H .67500
613500

Dz. 320.000
R 223000
H .67200
610200

Dz. 399000
R 278000
H. 83600
760600

Dz. 396.000
R 276.000
H .83200
755200



Dz. 211.000
R 147000
H .44400
402400 336.0 mm

2 Pls 640 × 19 = 243. − 38 = 205.
2 Ls 150×150×11 = 63.58 − 11 = 52.58
2 Pls. 490 × 11 = 108.00 − 16.5 = 91.50
414.58 349.05

Dz. 366000
R 257000
H 77300
700300 580.0 mm

4 Pls 640 × 19 = 486.0 − 76 = 410
2 Pls. 490 × 13 = ~~88.2~~ 127.0 − 13.5 = ~~74.7~~ 107.5
2 Ls 150×150×19 = 106.78 − 19.0 = 87.78
719.78 ~~552.48~~ 605.28

Say 700.00

Dz. 405000
R 282000
H .85000
772000 645.0 mm

4 Pls 640 × 19 = 486.0 − 76 = 410.0
2 Pls. 490 × 19 = 186.0 − 28.5 = 157.5
2 Ls 150×150×19 = 106.78 − 19.0 = 87.78
778.78 655.28

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CALCULATIONS FOR

Gifu-Ken Ibi and Nagara Bashi Combine Design.

Materials of floor system

Concrete 1:2:4 mixture

cross sectional area

coping 2 @ .20 × .375 = .150

Slab .155 × 14.91 = 2.312

2.462 sq.m

Span length …… 65.00 meters

2.462 × 65.00 = 160.0 cub. meters

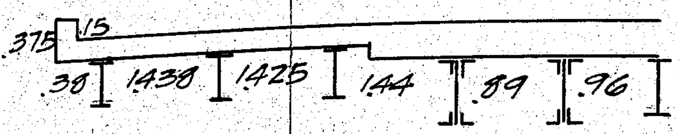
Extra concrete under track

.075 × 7.50 × 65.00 = 36.55 cub. meters

Forms width …… 14.116 meters

14.116 × 65.00 = 920 sq.m

Top of floor beam neglected



Pavements

14.55 × 65.00 = 946 sq.m

境界石 2 @ .145 × .18 × 65.00 = 3.4 cub. m

Reinforcements 25 Kgs/m² assumed

25 × 15.31 × 65.00 = 24,900 Kgs or 24.9 Kg tons

人造洗出仕上

2 @ .75 × 65.00 = 97.5 sq.m

Construction joint 3/8" carey elastite 3ヶ所

Drain 12ヶ 30 Kgs for one drain

Handrails 85 Kgs per lin. meter assumed

2 @ 85 × 65.00 = 11,000 Kgs or 11.0 Kg tons

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22

CALCULATIONS FOR
Gifu-Ken Ibi and Nagara-Bashi Combined Design.

approximate weight of truss

Lo-U1	767.56	@ .785	×	9.94	=	6000
U1-U3	615.56		×	13.05	=	6300
U3-U5	767.56		×	12.82	=	9700
Lo-L2	414.58		×	12.80	=	4150
L2-L4	700.00		×	12.80	=	7000
L4-L5	779.00		×	6.40	=	3900
U1-L2	340.		×	9.94	=	2670
L2-U3	300		×	12.00	=	2820
U3-L4	200		×	12.00	=	1890
L4-U5	156		×	12.74	=	1560
vert	82		×	19.87	=	1290
Hanger	135		×	23.25	=	2470 ✓

47750 × 2 = 95500
42% 40000
135500

for 2 trusses 2 × 135500 = 271.000 kg.

271.000 ÷ 64 = 4230 kg per lin. meter

Structural Steel in one span

stringers	1280 × 65.0	=	83200
F.B. intermediate	9 × 12300	=	110.800
End floor beam	2 @ 4500	=	9000
Bottom laterals			14700
Top laterals &c			51200
trusses			271000
shoes. say			6000
Expansion			1000
			546900

call this 550 tons.

Estimate of cost one span

structural steel	550 tons	@	190	=	104500
slab concrete 1:2:4	196.55	@	14^{00}	=	2750
forms	920	@	1.80	=	1660
asphalt pavement	946	@	4.80	=	4550
人造	98	@	4.00	=	392
Reinforcing Bars	24.9 ton	@	100^{00}	=	2490
Handrails	11 tons	@	280^{00}	=	3080
Construction Joints	3	@	40^{00}	=	120
Drains	12	@	8^{00}		96
Electric wiring	65 m	@	7.00		455
[illegible]	3.4 m^3	@	180^{00}		610.
					120803.

say 120700^{00}

12070
10400
1620

Combined Design	6 spans @	120700	=	724200^{00}	1.145	632.000^{00}
	7 spans @	120700	=	844900^{00}	1.145	737.000^{00}
Highway Design	6 spans @			285.000^{00}		
	7 spans @			333.000^{00}		
Railway Design	6 spans.			347.200^{00}		
	7 spans			404900^{00}		

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CALCULATIONS FOR

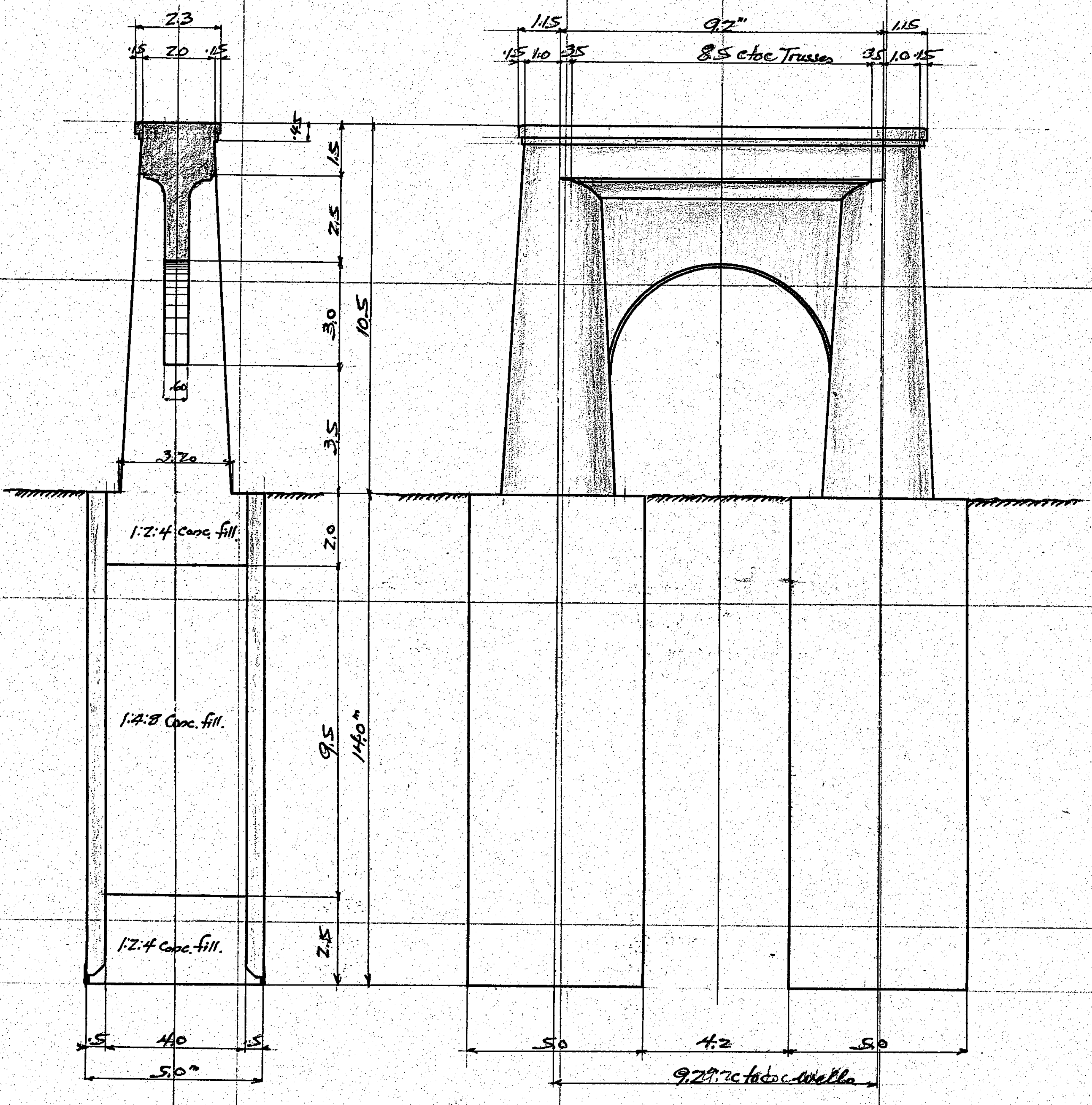
Gifu Ken Ibi and Nagara Bashi. Highway Design.

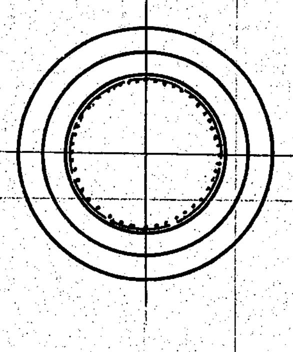
Max. Load on shoes.

Dead Load.	32 m @ 7600 =	243000
Live Load.	32 @ 3220 =	103000
		346000
misc. load.		4
		350.00 × 2 = 700.00 kg tons. on one pier

Design of Piers.

General dimensions are assumed as shown on sketch below





General Sketch of Pier.
Scale 1:150

MADE BY K. Inaba DATE 5-9-14 FILE NO

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CALCULATIONS FOR

Gifu Ken Ibi and Nagara Bashi. Highway Design.

Approximate volume of Concrete and weight of pier.

Coping	2.3 × .45 × 9.20	=	9.52		
"	2.3^{ϕ} × .45	=	1.87		
Top strut	2.05 × 1.3 × 9.20	=	24.52		
"	2.05^{ϕ} × 1.3 ×	=	4.29		
Shaft.	2.63^{ϕ} × 2 × 8.75	=	95.20		
Curtain wall	0.60 × 3.0 × 7.00	=	12.60		
				148.00 cub.m	1:2:4 mix
well. shell.	$(5.0^{\phi} - 4.0^{\phi})$ × 14.0 × 2	=	198.00		
top filling	4.0^{ϕ} × 2.0 × 2	=	50.25		
bottom "	4.0^{ϕ} × 2.5 × 2	=	62.75		
				311.00 "	1:2:4 mix
intermediate fill	4.0^{ϕ} × 9.5 × 2	=	239.00		
				239.00 "	1:4:8 mix
				698.00 m^3 @ 2300 =	1,605,000 kg

Total Load on pier.

Superimposed Load = 700,000 kg
weight of Pier = 1,605,000
2,305,000 kg.

Skin friction of well assumed 1700 kg/sq. meter.
Circumference of well = 2 × 5.0^{ϕ} = $\overset{31.42}{\cancel{39.3}}$ m for 2 wells.
Effective depth for skin friction assumed 14.0 − 4.0 = 10.0 meters.

total skin friction = 1700 × $\overset{31.42}{\cancel{39.3}}$ × 10.0 = 535,000 kg
bearing area at bottom = 2 × 5^{ϕ} = 39.3 sq.m.

total load on pier 2,305,000
less frictional resistance − 535,000
1,770,000

unit bearing pressure = $\frac{1{,}770{,}000}{39.3}$ = 45,000 kg/m² or (4.11 tons/□')

Estimate of cost for one pier.

Concrete	1:2:4 mix	459.0 m^3	@	14.00	=	6,420
"	1:4:8 "	239.0 "	@	9.20	=	2,200
reinforcements plain bars.		15.0 kg tons	@	100.00	=	1,500
forms.		1,100.0 m^2	@	2.20	=	2,420
Excavation for well sinking		600.0 m^3	@	5.50	=	3,300
curb shoes	2 @ 1.5 =	3.00 kg tons	@	160.00	=	480
test load and misc. expence say						1180
						17,500 円

築島費ヲ除ク.

築島費ハ二ヶ所 各 3000円宛.

橋台費ヲ橋脚費ト約半額ト見テ

				築島		
6 spans @	17500 =	105,000	+	6000	=	111,000 円
7 spans @	17500 =	122,500	+	6000	=	128,500

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CALCULATIONS FOR

Gifu Ken Ibi and Nagara Bashi. Railway Design.

Design of pier.
Use same details of shaft as for highway design page 9.
Use 5.6 meter dia. wells.

Approximate volume of concrete and weight of pier.

Shaft same as highway bridge. = 148.00 m³ 1:2:4 mixture

well shell. $(5.6^{\phi} - 4.5^{\phi}) \times 14.0 \times 2$ = 244.5
top filling $4.5^{\phi} \times 2.0 \times 2$ = 63.9
bottom " $4.5^{\phi} \times 2.5 \times 2$ = 79.6
388.00 " 1:2:4 mix.

intermediate fill. $4.5^{\phi} \times 9.5 \times 2$ = 302.00 " 1:4:8 mix.
838.0 @ 2300 = 1,927,000 kg.

Total load on pier.
Superimposed Load = 1,220,000
weight of pier = 1,927,000
3,147,000

Skin friction assumed 1700 kg/m²
~~A~~ Circumference of 2 wells. = $2 \times 5.6^{\phi}$ = ~~49.3~~ 35.2 meters
Effective depth of well for skin friction say 10. m
total skin friction = 1700 × 35.2 × 10 = 599,000 kg

total load on pier 3,147,000
less skin friction 599,000
2,548,000 kg
unit bearing pressure = $\frac{2,548,000}{49.3}$ = 51,600 kg/m² or (4.72 tons/□')

Estimate of cost for one pier.

Concrete 1:2:4 mix 536.0 m³ @ 14.00 = 7510.
" 1:4:8 " 302.0 " @ 9.20 = 2,780
reinforcements plain bars. 17.00 kg tons @ 100.– = 1,700
forms. 1180.0 m² @ 2.20 = 2,600
Excavation for well sinking 750. m³ @ 5.50 = 4,120
Curb shoes 2 @ 1.75 = 3.5 kg tons @ 160.– = 560
test load and miscellaneous expence say 1230
20,500 円 (築島費ヲ除ク)
築島費ハ二ケ所 各 3500 円トス

橋台費ヲ橋脚費ノ半額位ト見テ

6 spans. @ 20,500 = 123,000 + 7000 (築島) = 130,000 円
7 spans @ 20,500 = 143,500 + 7000 = 150,500 円

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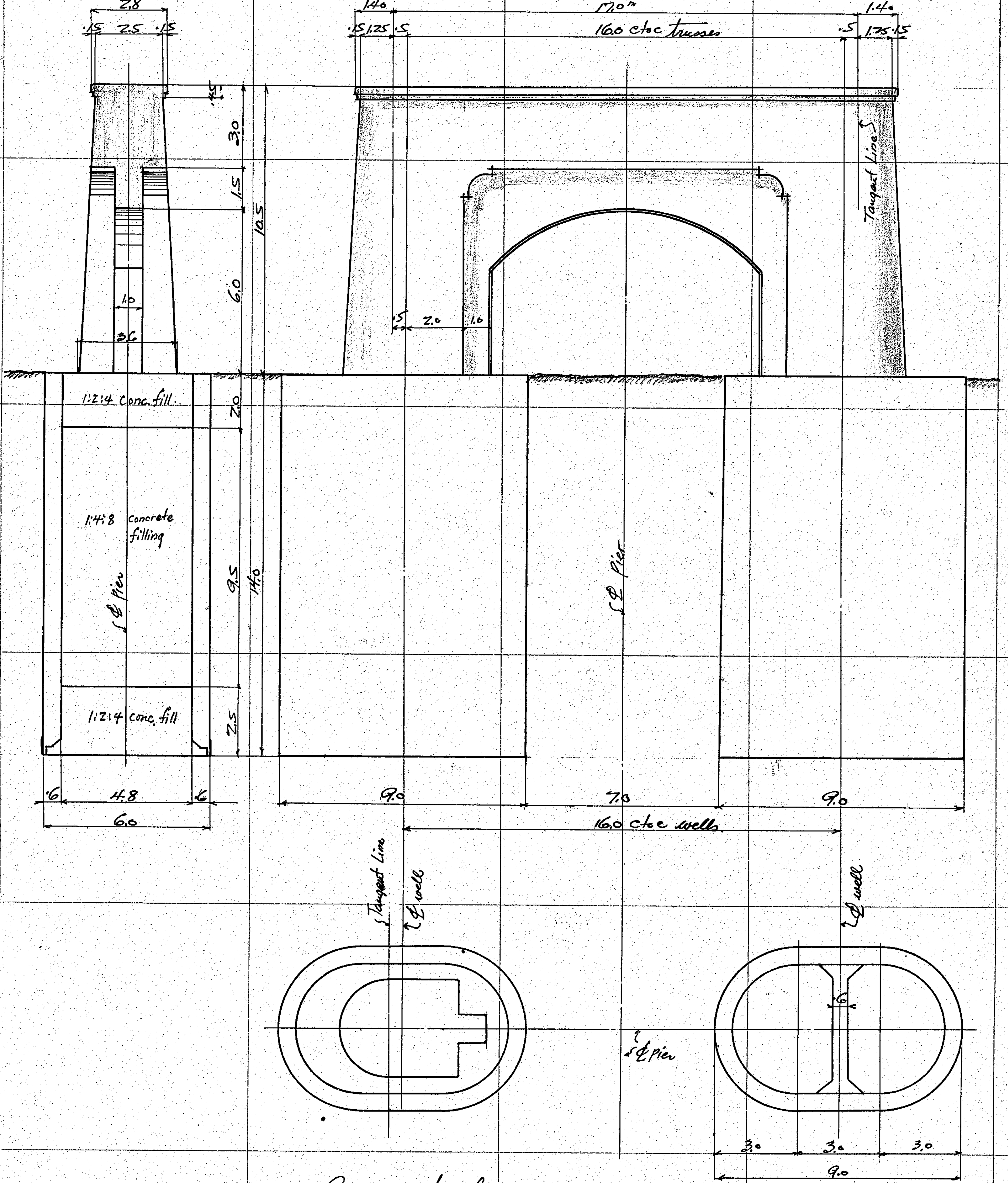
CALCULATIONS FOR

Gifu Ken Ibi and Nagara Bashi; Combined Design.

Design of pier.

General dimensions are assumed as shown on sketch below.



General sketch of Pier.
Scale 1: 150.

MADE BY K. Inaba DATE 5-9-15 FILE NO

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CALCULATIONS FOR

Gifu-Ken Ibi and Nagara Bashi Combined Design.

Approximate volume of concrete and weight of pier.

Coping 2.80 × .45 × 17.0 = 21.40
" 2.80^{ϕ} × .45 = 2.77
Top strut. 2.63 × 2.55 × 17.0 = 114.00
" 2.63^{ϕ} × 2.55 = 13.85
Shaft. 3.18 × 2.50 × 7.5 × 2 = 119.20
Curtain wall. 1.0 × 1.0 × 7.5 × 2 = 15.00
" 1.0 × 2.5 × 10.0 = 25.00

311.22 m³ 1:2:4 mix

well. shell. 3.0 × .6 × 14.0 × 4 = 100.80
" 16.97 × .6 × 14.0 × 2 = 285.00
partition wall. 4.8 × .6 × 12.0 × 2 = 69.10
fillets .5 × .5 × 14.0 × 4 = 14.00
top filling 4.8 × 3.0 × 2.0 × 2 = 57.60
" " 4.8^{ϕ} × 2.0 × 2 = 72.40
bottom fill 2.4 × 4.8 × 2.5 × 2 = 57.60
" " 4.8^{ϕ} × 2.5 × 2 = 90.50

747.00 1:2:4 mix

Intermediate filling 2.4 × 4.8 × 9.5 × 2 = 219.00
" " 4.8^{ϕ} × 9.5 × 2 = 344.00

563.00 1:4:8 mix
1621.00 in round no.

total wt. of pier = 1621 @ 2300 = 3,720,000 Kg.

Loads on Pier.
Superimposed Dead Load = 1,110,000
" Live Load = 985,000
2,095,000
weight of pier = 3,720,000
5,815,000 Kg.

Skin friction on well assumed 1700 kg/m²
Circumferential area of 2 wells = 2 × 6.0^{ϕ} + 12.0 = 49.7 × 10 = 497.
total skin friction = 497 × 1700 = 845,000 kg

Net load on bottom = 5,815,000
− 845,000
4,970,000 kg

bottom area of well.
3.0 × 6.0 = 18.0
6.0^{ϕ} = 28.3
46.3 × 2 = 92.6 sq. meters.

Unit bearing pressure = $\frac{4,970,000}{92.6}$ = 53700 kg/m² or (4.89 tons/□')

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CALCULATIONS FOR

Gifu Ken Ibi and Nagara Bashi. Combined Design.

Estimate of Cost for one pier.

Concrete 1:2:4 mix.	1,058.0 m³	@	14.00	=	14,830
〃 1:4:8 〃	563.0 〃	@	9.20	=	5,180
reinforcements	25.0 kg tons	@	100.00	=	2,500
forms.	1900.0 m²	@	2.20	=	4180
Excavation for well sinking	1400.0 m³	@	5.50	=	7,700
curb shoe. 2 @ 2.3 =	4.6 kg tons	@	160. 〃	=	740
test load and misc. expense. say				=	1,870
					37,000 円 (築島費ヲ除ク.)

築島費ハ二ケ所 各 4500円宛トス

橋台費ヲ橋脚費ノ半額ト見テ [illegible]

				築島			
6 spans	@	37000 =	222,000	+ 9,000 =	231,000 円	0.96	241,000 円
7 spans	@	37000 =	259,000	+ 9.000 =	268,000 円	0.96	279,000 円

工費比較表.

	揖斐川				長良川			
	Super structure	Sub structure	Total cost	Percentage	Super structure	Sub structure	Total cost	percentage
Highway Design	285,000	111,000	396,000 円	45.4%	333,000	128,500	461,500 円	45.4%
Railway Design	347,200	130,000	477,200	54.6%	404,900	150,500	555,400	54.6%
Summary	632,200	241,000	873,200	100.0%	737,900	279,000	1,016,900	100.0%
Combined design	724,200	231,000	955,200	109.5%	844,900	268,000	1,112,900	109.5%
			865,000	99% 103			1007,900	99%

備考 最後ノ行ニ記載セル数字ハ電車ヲ片側ニ寄セテ電車軌道敷ニハ床ヲ張ラズシテ電車ト反対側ノトラス外側ニ歩道巾2米ノモノヲ設置シタル場合ノ概算工費ナリ 此設計最モ有利ノ如シ

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CONSULTING ENGINEER
~~JIJI~~ BLDG, TOKYO
SHOWA

CALCULATIONS FOR

MADE BY________ DATE________ FILE NO________
CHECKED BY________ DATE________ PAGE NO________

昭和八年八月

國道拾貳號線

岐阜縣木曽川橋設計々算書

（上部工事）

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

MADE BY J. Masuda DATE 8-8-20 FILE NO
CHECKED BY M. Jinda DATE 8-6-26 PAGE NO 1

CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

The location of the new bridge selected 150 meters below the present bridge after careful study of general traffic and local condition and also approaches thereto. The total length of the bridge is 462.4 meters between faces of parapet walls of both abutments or 460.0 meters about between front faces of the said abutments. The bridge is divided into 7 equal spans of 65.1 meters each after the study of economic layout. The low steel of bridge at both abutments is 1.5 meters high above assumed highwater level and cambered .60 meter toward the center of bridge, and the bridge floor corresponding to the above.

The width of roadway 9.0 meters clear between curb lines and the whole width paved with granolithic concrete on reinforced Concrete slabs.

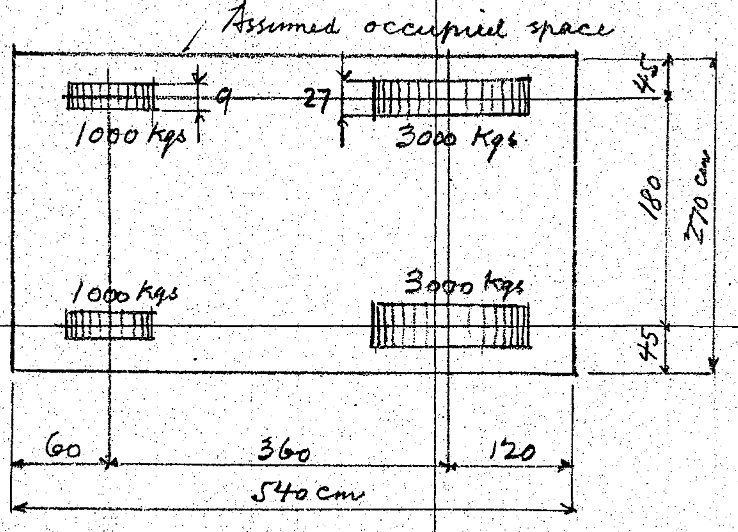
The handrails throughout the bridge are made of cast iron and the pedestals at entrance over both abutments are of cut stone and ornamental design.

Assumed Loadings

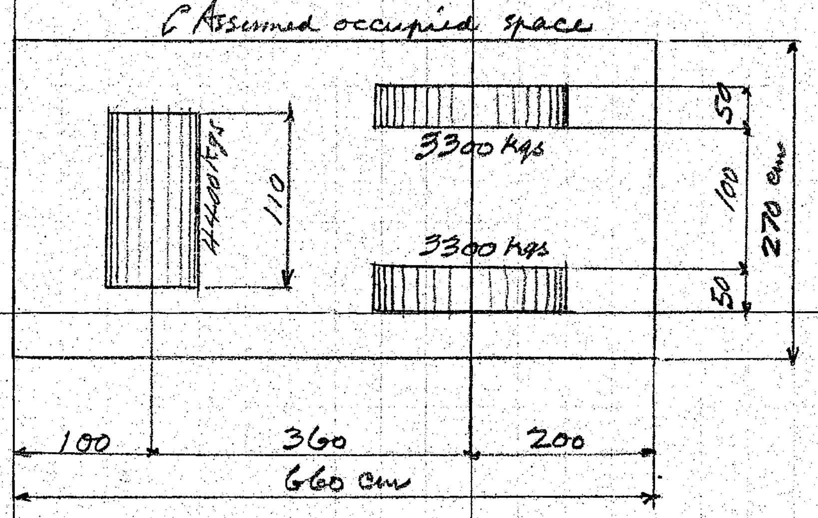
Uniform load on roadway $w = \frac{100{,}000}{170 + l} \leq 500$ kgs/m²

where w = uniform load in kg per sq. meter
l = Span length in meter.

8 ton motor truck loading



11 ton Road Roller Loading



3 lines of motor truck traffic on roadway with occupied width of 270 cm each; unoccupied space around the motor trucks 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 meter max impact 30%

No impact for roadroller and uniform live load.

Allowable working strength

Concrete 1:2:4 mixture

Direct compression	35	kgs/cm²
Fibre stress due to bending	45	"
Combined stresses due to direct and bending	35	"
Punching shear	9	"
Plain shear	4	"
Bearing value	45	"
Bond stress	6	"

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~~JIJI~~ SHOWA BLDG, TOKYO

MADE BY J. Masuda DATE 8-8-20 FILE NO
CHECKED BY M. Jinda DATE 8-6-26 PAGE NO 2

CALCULATIONS FOR

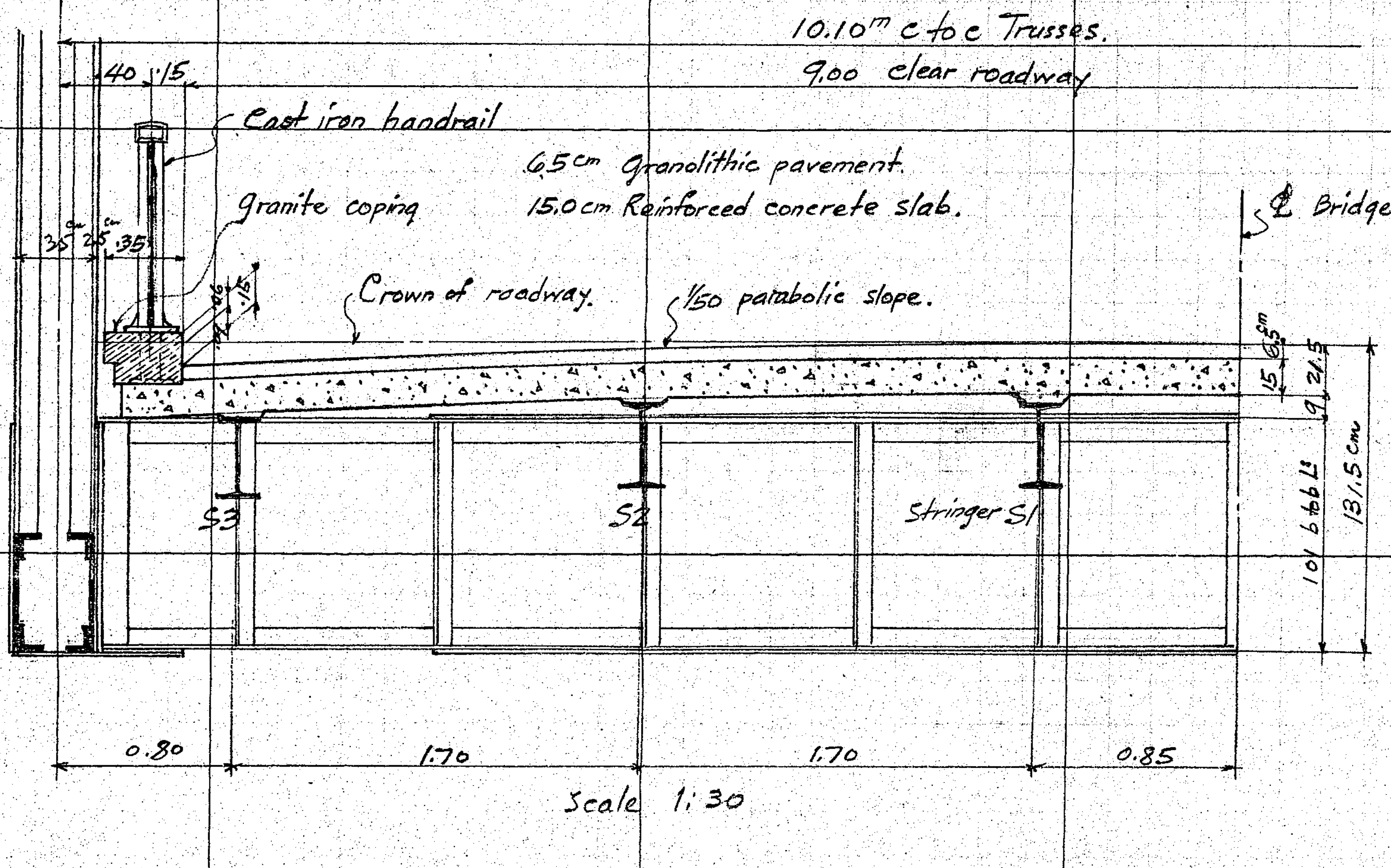
Design of Kisogawa-Bashi for Gifu-Ken

Reinforcing bars	
Tension or compression	1200 kgs/cm²
Shearing strength	900 "
Structural Steel	
Tension net	1200 kgs/cm²
Extreme fibre stress net	1200 "
Shear of web gross section	900 "
Compression member $1500(1 - 0.0055\ l/r)$ not over	1000 "
where l = length of member in cm r = least radius of gyration in cm	
Compression flange of girder $1200(1 - 0.012\ l/b)$ 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)	1700 "
" " field " " "	1500 "
" " pin	1800 "

Roller 45d kgs/cm where d = diameter of roller 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 earthquake, the working strength shall be increased 60%.
Seismic acceleration for this locality assumed 3000 mm/sec² or k = 0.3

Cross section of bridge assumed as shown on sketch below.



Scale 1:30

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MADE BY K. Inaba DATE 8-6-8 FILE NO
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CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu-ken

Design of Floor slab. Span length = 1.700 meters.

Dead Load:

6.5 cm Granolithic pavement @ 22 kg = 143
15.0 cm Concrete slab @ 24 " = 360
Fillet &c say = 7
510 kg per sq. meter.

Dead Load moment = $1/10 \times 510 \times 1.70^2$ = 147 kgm.
Dead Load shear = $1/2 \times 510 \times 1.70$ = 433 kg.

Live Load: Motor truck rear wheel concentration = 3000
30% impact = 900
Max. wheel concentration on slab = 3900 kg

Distribution of wheel concentration on slab

Longitudinal distribution a. Contact between wheel & pavement 20
Distribution 2@ 6.5 = 13
a = 33 cm

Transverse distribution b = 27 + 2@6.5 = 40 cm.

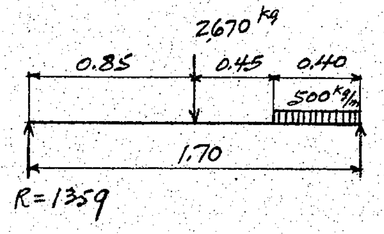
Effective width of slab against wheel concentration

$E = 2/3 \cdot l + a = 2/3 \times 1.70 + 0.33 = 1.463$ meters.

Load per meter strip of slab = 3900 ÷ 1.463 = 2670 kg

Uniform live load assumed 500 kg per sq. meter.

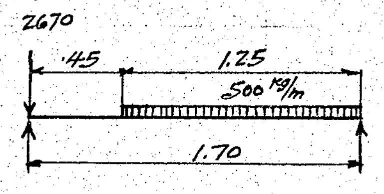
Max. moment:



Reaction $\frac{500 \times 0.40^2}{2 \times 1.70}$ = 24
2670 ÷ 2 = 1335
1359 kg.

Moment as a simple beam 1359 × 0.85 = 1155
For continuity of slab, moment may be taken as 8/10 of above value.
max. moment = 8/10 × 1155 = 924 kgm

Max. end shear.



$\frac{500 \times 1.25^2}{2 \times 1.70}$ = 230
2670
max. end shear = 2900 kg

Summary of moments and shears.

	moment	shear
Dead Load	147	433
Live Load	924	2900
	1071 kgm	3333 kg

Effective depth required for $f_s = 1200$, $f_c = 45$ kg/cm²

$d = \sqrt{\frac{M}{bk}}$ where b = width of slab = 100 cm
k = a constant = 7.13

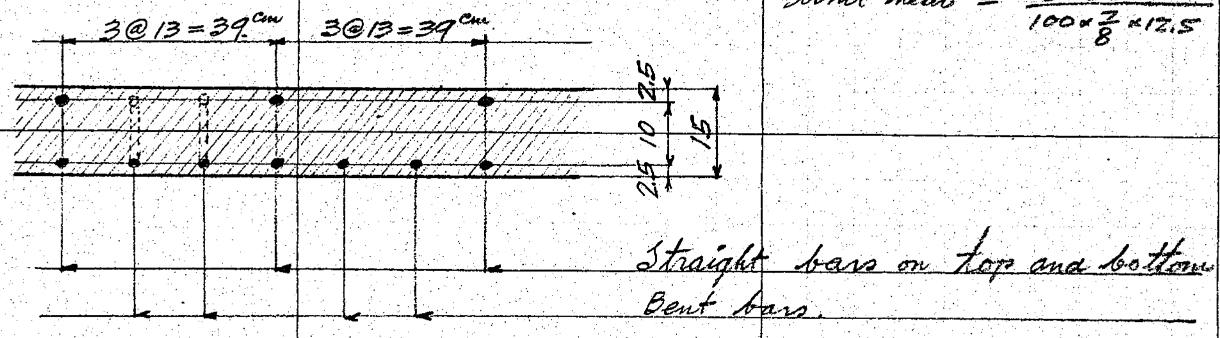
$d = \sqrt{\frac{1071 \times 100}{100 \times 7.13}} = 12.25$ cm

Use 12.5 cm effective depth with 2.5 cm insulation or 15.0 cm overall depth.

Steel area required = $\frac{1071 \times 100}{1200 \times \frac{7}{8} \times 12.5}$ = 8.17 cm²/m. strip

Use 12 mmφ bars 13 cm c to c = 8.70 cm² per meter strip.

Unit shear = $\frac{3333}{100 \times \frac{7}{8} \times 12.5}$ = 3.05 kg/cm²



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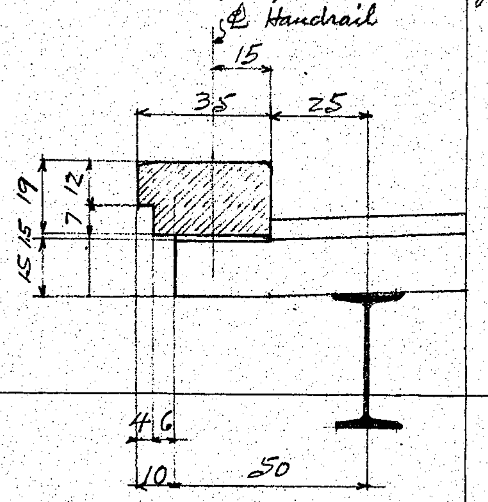
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-ken.

Overhanging slab beyond stringer S3.



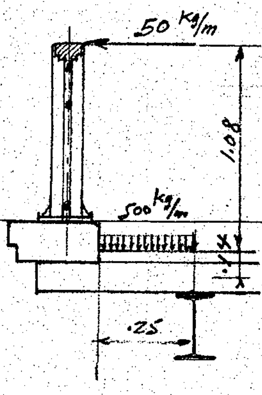
Dead Load.

Coping stone .12×.35 @ 2600 = 109 × 0.425 = 46
" " .07×.31 @ 2600 = 56 × 0.405 = 23
" concrete .15×.25 @ 2400 = 90 × 0.375 = 34
255 kg

Slab and pavement. 0.25 @ 510 = 128 × 0.125 = 16
handrail assumed 120 × 0.400 = 48
503 kg 167 kgm

Dead Load moment = 167 kgm
Dead Load shear = 503 kg.

Live Load:



Uniform live load on roadway = 500 kg/m² assumed
Horizontal thrust on top of handrail = 50 kg per lin meter of top rail.

moment on slab. 500 × 0.25 × 0.125 = 16
50 × 1.22 = 61
Live Load moment = 77 kgm.
Live Load shear 500 × 0.25 = 125 kg.

Summary of moments and shears.

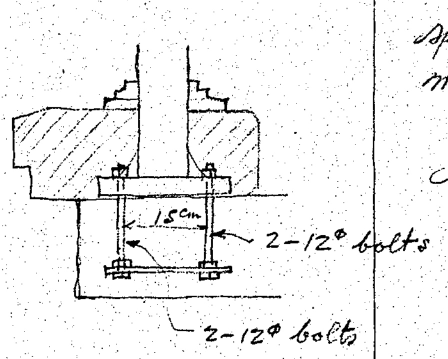
	moment	shear
Dead Load	167	503
Live Load	77	125
	244 kgm	628 kg.

Steel area required

$$= \frac{244 \times 100}{1200 \times \frac{7}{8} \times 12.5} = 1.86 \text{ cm}^2/\text{m strip}$$

Use 12 mmφ bars 39 cm ctoc = 2.90 " "

Anchor bolts for Handrail posts. Spacing 4.65 ÷ 3 = 1.55 meter ctoc.
max. moment on one post
= 61 × 1.55 = 95 kgm.



spacing of bolts assumed 15 cm. ctoc.
max. load on bolts.
= 95 ÷ 0.15 = 633 kg T.
Use 2-12 mmφ bolts on each side

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MADE BY K. Inaba DATE 8-6-12 **FILE NO**

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu ken.

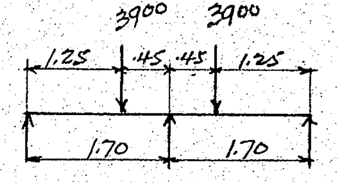
Design of I-beam stringer S1. Span length 4.65 meters, spacing 1.70 meters.

Dead Load: Floor and pavement $510 \times 1.70 =$ 867
I-beam assumed 62
929 kg per lin meter

Dead Load moment $= 1/8 \times 929 \times 4.65^2 =$ 2,510 kgm
Dead Load shear $= 1/2 \times 929 \times 4.65 =$ 2160 kg.

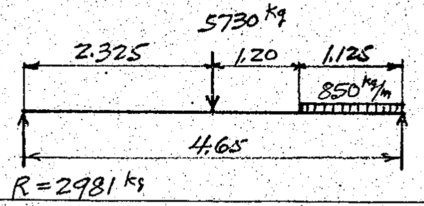
Live Load: motor truck rear wheel concentration with impact = 3,900 kg
" " front " " " " = 1,300 "

Max. concentration on stringer.



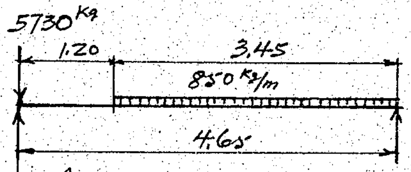
Rear wheel $2 \times 3900 \times \frac{1.25}{1.70} =$ 5730 kg
front " $2 \times 1300 \times \frac{1.25}{1.70} =$ 1910 "

Uniform live load $= 500 \times 1.70 = 850$ kg per lin. meter.



Max. moment.
Reaction $850 \times \frac{1.125^2}{2 \times 4.65} =$ 116
$5730 \div 2 =$ 2865
2981 kg

moment $= 2981 \times 2.325 =$ 6,930 kgm



Max. end shear.
$850 \times \frac{3.45^2}{2 \times 4.65} =$ 1,087
5730
6817 kg

Summary of moments and end shears.

	moment	shear
Dead Load	2,510	2160
Live Load	6930	6817
	9,440 kgm	8,977 kg

Section modulus required $= \frac{9440 \times 100}{1,100} = 858$ cm³

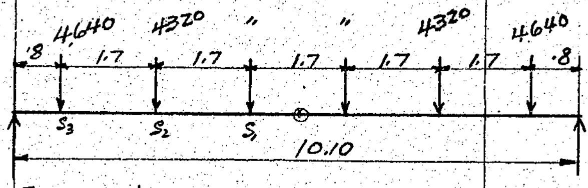
Use I-beam 350×150 @ 58.5 kg. Sm = 870.6 cm³

Unit shear on web $= \frac{3}{2} \times \frac{8977}{350 \times 0.9} = 428$ kg/cm²

For stringers S2 and S3, use the same section as for stringer S1.

Design of intermediate floor beam. Span length 10.10 meters, spacing 4.65 meters c to c.

Dead Load:- Stringer concentrations on floor beam.
from stringers S1 and S2.
$929 \times 4.65 =$ 4,320 kg
from stringer S3.
Coping (see on page 4) 255
Slab and pavement $1.10 @ 510 =$ 561
Stringer assumed 62
handrail " 120
998 kg/m



Concentration $= 998 \times 4.65 =$ 4,640 kg
Reaction. $4320 \times 2 =$ 8640
4640
13,280 kg

moment
$13,280 \times 5.05 =$ 67,000
$4640 \times 4.25 = -$ 19,700
$4320 \times 1.70 \times 2 = -$ 14,680
32,620
weight of beam. $1/8 \times 240 \times 10.10^2 =$ 3060
35,680 kgm

End shear. Stringer concentration : 13,280
beam $1/2 \times 240 \times 10.10 =$ 1210
14,490 kg

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CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu-Ken.

Live Load:-

3 - 8 ton motor trucks on roadway with uniform load around them.

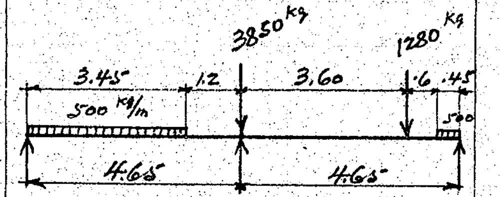
Rear wheel concentration = 3,000

Impact coef. = $\frac{20}{60+10.1}$ = 28.5% = 850

3,850 kg

front wheel with impact say 3850 ÷ 3 = 1,280 "

Uniform live load on roadway = 500 kg per sq. meter.



Max. load on floor beam, wheel loads being assumed directly on floor beam.

Unif. load on front and rear of truck.

$\frac{500 \times 0.45^2}{2 \times 4.65}$ = 10

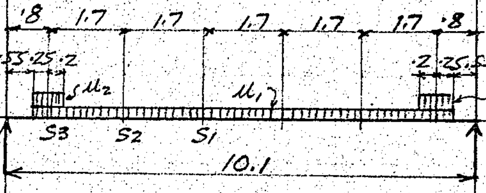
$\frac{500 \times 3.45^2}{2 \times 4.65}$ = 640

650 kg/m. = U_1

Unif. load on sides of trucks.

500 × 4.65 = 2325 "

Difference = 1,675 " = U_2.



Stringer concentration due to uniform load.

On stringer S1. 650 × 1.70 = 1105 kg

" " S2. $\frac{1675 \times 0.2^2}{2 \times 1.70}$ = 20

650 × 1.70 = 1105

1,125 kg

" " S3. $\frac{1675 \times 0.2 \times 1.6}{1.7}$ = 315

$\frac{1675 \times 0.25 \times 0.675}{0.8}$ = 353

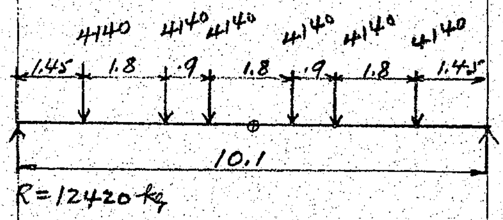
650 × 0.85 = 552

$\frac{650 \times 0.25 \times 0.675}{0.8}$ = 137

1,357 kg call this 1,360 kg

max. moments.

Wheel loads.



max. wheel concentration on floor beam.

front wheel $\frac{1280 \times 1.05}{4.65}$ = 290

rear " 3850

4140 kg.

Reaction 3 @ 4140 = 12,420 "

moment at center of span.

12420 × 5.05 = 62700

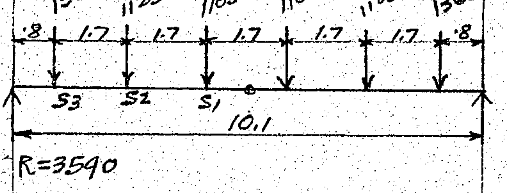
4140 × 3.60 = - 14900

4140 × 1.80 = - 7450

4140 × 0.90 = - 3730

36620 kgm

Uniform load.



Reaction 1360 + 1125 + 1105 = 3,590 kg

moment at center of span.

3590 × 5.05 = 18,130

1360 × 4.25 = - 5,780

1125 × 2.55 = - 2870

1105 × 0.85 = - 940

8,540 kgm.

max. end shear

wheel loads.

$\frac{4140 \times 6 \times 5.50}{10.1}$ = 13,520 kg

Uniform load.

$\frac{1675 \times 0.90 \times 1.00}{10.1}$ = 150

650 × 4.50 = 2930

3,080 kg

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CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu Ken.

Summary of Live Load moments and shears.

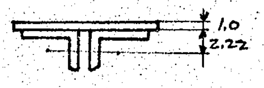
	moments	End shears
Wheel loads	36620	13520
Unif. loads.	8540	3080
	45160 kgm	16600 kg

Summary of Dead Load and Live Load moments and shears.

	moments	End shears
Dead Load	35680	14490
Live Load	45160	16600
	80,840 kgm	31,090 kg

Try web. 1000 × 9 = 90.0 cm^2, 1/8 web = 11.25 cm^2
Effective depth say 101. + 2 − 2@2.07 = 98.86 cm
Flange stress = $\frac{80840}{0.9886}$ = 81,800 kg

Flange area reqd. = $\frac{81800}{1200}$ = 68.20
1/8 web area = − 11.25
56.95 cm^2 net



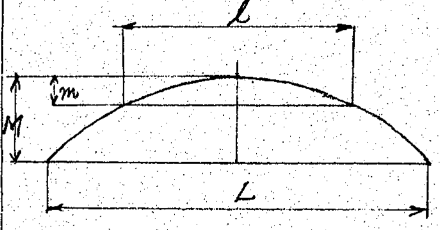
41.00 × 3.22 = 132.00
30.00 × 0.50 = 15.00
71.00 2.07 cm 147.00

Effective depth of girder
= 101.00 + 2.00 = 103.00
2 × 2.07 = − 4.14
98.86 cm.

Use
2Ls 125 × 90 × 10 = 41.00 − 8.80 = 32.20
1 cov. pl. 300 × 10 = 30.00 − 4.40 = 25.60
71.00 cm^2 gr 57.80 cm^2 net

Unit shear on web gross section = $\frac{31090}{90}$ = 346 kg/cm^2 average.

Required Length of cover plate.



M = moment at center of floor beam = 80840 kgm
m = moment carried by cover plate = 80840 × $\frac{25.60}{57.80+11.25}$ = 29950 kgm
l = required length of cover plate
L = Span length of floor beam. = 10.10 meters
Assuming parabolic change of moment.
$l^2 = L^2 \frac{m}{M} = 10.10^2 \times \frac{29950}{80840} = 37.8$
$l = \sqrt{37.8} = 6.15$ meters
Length required to develope cover plate stress assumed 32.5 cm on each end.
Total length of cover plate = 6.15 + 2@32.5 = 6.80 meters.

Approximate weight of intermediate floor beam.

Flange	4Ls	125 × 90	× 10	@	16.10	×	9.73	=	626
〃	2 cov. pls.	300	× 10	@	23.55	×	6.80	=	320
web	1 web pl.	1000	× 9	@	70.65	×	9.72	=	687
Stiffeners crimped	10Ls	90 × 90	× 10	@	13.30	×	1.01	=	134
〃 〃	12Ls	125 × 90	× 9	@	14.60	×	1.01	=	177
fills. for stringer conn.	12 pls	90	× 10	@	7.07	×	0.25	=	21
Connection Ls	4Ls	150 × 100	× 12	@	22.40	×	0.99	=	89
〃 fills	4 pls	215	× 10	@	16.88	×	0.82	=	55
web splice	2 pls	410	× 10	@	32.19	×	0.82	=	53
	Rivet heads and variations say 3½ %							=	76
									2238 kg
					Call this				2240 kg

weight per lin meter = 2240 ÷ 9.73 = 230 kg.

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CALCULATIONS FOR

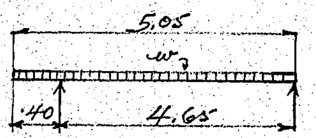
Design of Kisogawa Bashi for Gifu-ken.

Design of End floor beam. Span length = 10.10 meters.

Overhang assumed 0.40 meter.

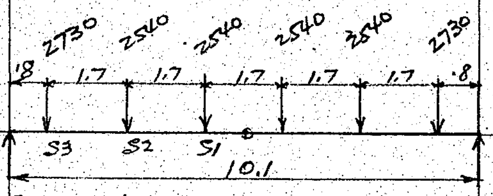
Dead Load:

Stringer concentration on floor beam.



$$\frac{w \times 5.05^2}{2 \times 4.65} = 2.74 w$$

On S1 and S2 2.74 × 929 = 2540 kg
on S3 2.74 × 998 = 2730 "
Reaction 2540 × 2 = 5080
2730
7810 kg

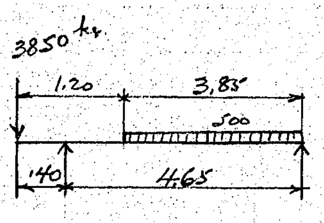


R=7810 kg

weight of beam assumed 220 kg per lin meter
Reaction = 220 × 5.05 = 1110 kg
End shear = 7810 + 1110 = 8920 kg
moment at center of beam.
Stringer concentration
7810 × 5.05 = 39400
2730 × 4.25 = − 11600
2540 × 3.40 = − 8640
19160
weight of beam 1/8 × 220 × 10.1^2 = 2810
21970 kgm.

Live Load:

max. load on end floor beam.
Wheel load



Rear wheel concentration with impact 3850 kg (see page 6)

$$3850 \times \frac{5.05}{4.65} = 4180 \text{ kg}$$

Unif. load on rear of trucks

$$\frac{500 \times 3.85^2}{2 \times 4.65} = 800 \text{ kg per lin m of span} = u_1$$

Unif. load on sides of trucks
2.74 × 500 = 1370 " " "
difference $u_1 - u_2$ = 570 " " " = u_2

max. moments.
wheel loads.

By proportion. load ratio = $\frac{4180}{4140}$ = 1.01
moment = 36620 × 1.01 = 37000 kgm (see on page 6)

Stringer concentration due to uniform loads. (Refer to figures on page 6)
On stringer S1. 800 × 1.70 = 1360 kg
" " S2 $\frac{570 \times 0.2^2}{2 \times 1.70}$ = 7
800 × 1.70 = 1360
1367 kg
" " S3. $\frac{570 \times 0.2 \times 1.6}{1.7}$ = 107
$\frac{570 \times 0.25 \times 0.675}{0.8}$ = 120
800 × 0.85 = 680
$\frac{800 \times 0.25 \times 0.675}{0.8}$ = 169
1076 kg

uniform load.

Reaction = 3803 kg
moment at center of span.
3803 × 5.05 = 19200
1076 × 4.25 = − 4570
1367 × 2.55 = − 3480
1360 × 0.85 = − 1160
9990 kgm

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-ken

max. end shear.

Wheel load. $\frac{4180 \times 6 \times 5.50}{10.1} = 13650$ kg

Unif. load. $\frac{570 \times 0.90 \times 9.10}{10.1} = 50$

$800 \times 4.50 = \underline{3600}$

$\underline{3650}$

17300 kg

Summary of moments and end shears.

	moment	End shear
Live Load. wheel load	37000	13650
〃 unif. load	9990	3650
Summary for Live Load	46990	17300
Dead Load	21970	8920
	68960 kgm	26220 kg

Try web $1000 \times 9 = 90.0$ cm², 1/8 web area = 11.25 cm²

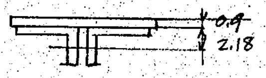
Effective depth say $102.80 - 2@2.02 = 98.76$ cm

Flange stress = $\frac{68960}{0.9876} = 69800$ kg

Flange area reqd. = $\frac{69800}{1200} = 58.20$

1/8 web areas $= \underline{-11.25}$

46.95 cm² net



$37.08 \times 3.08 = 114.20$
$\underline{25.20 \times 0.45 =} \quad \underline{11.30}$
62.28 2.02 cm 125.50

Use 2Ls $125 \times 90 \times 9 = 37.08 - 7.92 = 29.16$

1 cov. pl. $280 \times 9 = \underline{25.20} - 3.96 = \underline{21.24}$

62.28 gr 50.40 cm² net

Unit shear on web gross section

$= \frac{26220}{90} = 292$ kg/cm² average.

Required length of cover plate.

$m = \frac{68960 \times 21.24}{50.40 + 11.25} = 23700$ kgm

$l^2 = 10.10^2 \times \frac{23700}{68960} = 35.15$

$l = \sqrt{35.15} = 5.93$

add $2@.335 = \underline{.67}$

Total length of cov. pl. = 6.60 meters

Approximate weight of End floor beam.

Flange	4Ls	125 × 90 × 9	@	14.60 ×	9.73	=	568
〃	2 cov. pls	280 × 9	@	19.78 ×	6.60	=	261
web	1 web pl.	1000 × 9	@	70.65 ×	9.72	=	687
Stiffeners (crimped)	10 Ls	90 × 90 × 10	@	13.30 ×	1.01	=	134
〃 〃	12Ls	125 × 90 × 9	@	14.60 ×	1.01	=	177
fills. for stringer conn.	12 Pls	90 - × 9	@	6.36 ×	0.25	=	19
Connection Ls	4Ls	150 × 100 × 12	@	22.40 ×	0.99	=	89
〃 fills	4 Pls	215 × 9	@	15.19 ×	0.82	=	50
web splice	2 pls	410 × 9	@	28.97 ×	0.82	=	48
	Rivet heads and variations say 3½%					=	71
							2104 kg

Call this 2105 kg

Weight of beam per lin meter = $2105 \div 9.73 = 217$ kg.

Design of Kisogawa-Bashi for Gifu-Ken.

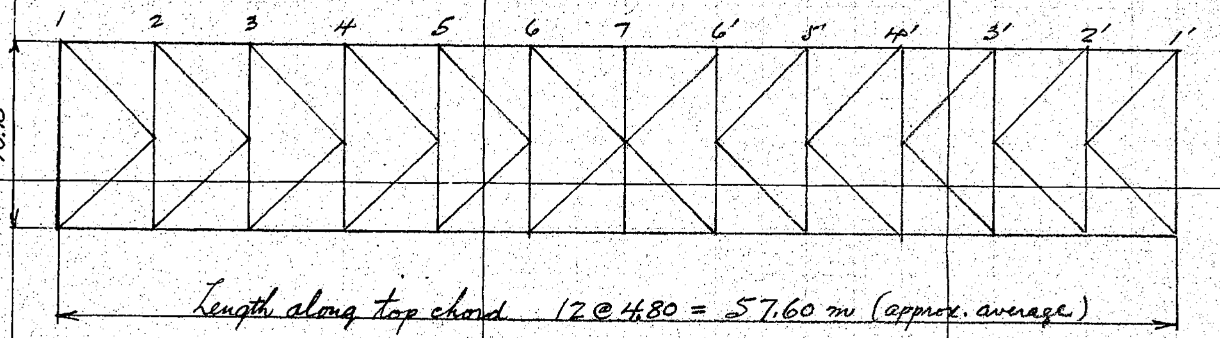
Design of Upper lateral bracings.

Wind load on upper lateral $w = 200 + 2@15 = 230$ kg per lin meter
Panel load $4.65 @ 230^{kg} = 1,070$ kg.

Seismic load on upper lateral. Seismic coef assumed $k = 0.300$ (moving load assumed)

upper lateral bracings assumed			= 110
2 portal bracings	@ 2,500	= 5,000	
5 sway "	@ 2000	= 10,000	
6 struts	@ 800	= 4800	
		19,800 ÷ 65.1 =	304
main truss assumed ½×172,000 ÷ 65.1			= 1322
misc. say			34
			1770 kg per lin meter

Seismic panel load = $1770 \times 4.65 \times 0.30 = 2470$ kg. Seismic stresses govern.

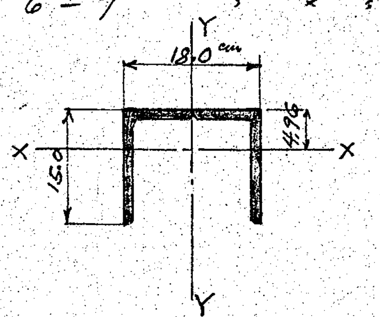


Average diagonal length
$= \sqrt{5.05^2 + 4.8^2} = 6.97$ meters
Coefficients $= \frac{6.97}{5.05} = 1.38$

Least radius of gyration required for diagonal members $= \frac{697}{150} = 4.65$ cm

Seismic stresses in diagonal members,

members		Shear		Stress for one member	$f = 1920^{kg/cm^2}$ Sectional area reqd for tension	$r = 3400^{kg}$ 19φ rivet no.	$f = 465^{kg/cm^2}$ sectional area reqd for compression see below
1 - 2	$2470 \times \frac{4.8}{57.6} \times 66 =$	13,600	$\times \frac{1.38}{2} =$	9390 kg T or C	4.89 cm² net	2.8	20.2 cm² gross
2 - 3	" × " × 55 =	11,320	=	7810	4.07	2.3	16.8
3 - 4	" × " × 45 =	9270	=	6400	3.34	1.9	13.8
4 - 5	" × " × 36 =	7410	=	5110	2.66	1.5	11.0
5 - 6	" × " × 28 =	5760	=	3980	2.07	1.2	8.6
6 - 7	" × " × 21 =	4320	=	2980	1.55	0.9	6.4



Use 2∠ $150 \times 90 \times 9 = 41.88 - 3.96 = 37.92$ cm² net.
moment of inertia about Y-Y axis
$2 @ 129 + 41.88 \times 7.02^2 = 2320$ cm⁴
radius of gyration $r_y = \sqrt{\frac{2320}{41.88}} = 7.44$ cm
" " " $r_x = 4.75$ cm

max. slenderness ratio $= \frac{697}{4.75} = 146.7 < 150$

Allowable unit compression $= 1500(1 - \overset{.194}{0.0055 \times 146.7}) = 291$ kg/cm²
In case of earthquake $f = 291 \times 1.60 = 465$ kg/cm²

This section is good for the seismic stresses;
Tension of $37.92 \times 1920 = 72,800$ kg
Compression of $41.88 \times 465 = 19,480$ kg

The above section is ample.

CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

Approximate weight of upper lateral bracings for one panel. (average)

4Ls 150 × 90 × 9 @ 16.40 × 6.30 = 413
10 tie pls @ 2.00 = 20
connection to sway + strut say = 30
Rivet heads and variation - say = 17
480 kg

Total weight. 12 panels @ 480 = 5,760 kg
weight per lin meter of bridge = $\frac{5760}{4.65 \times 12}$ = 103 kg.

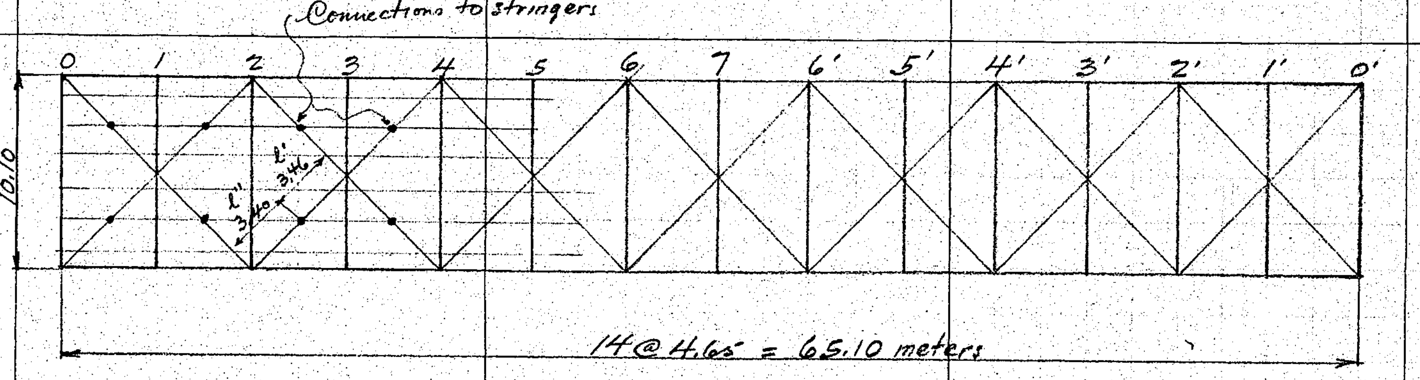
Design of lower lateral bracings. (moving load assumed)
Wind load on lower lateral w = 400 + 2 @ 15 = 430 kg per lin meter
panel load = 430 × 4.65 = 2,000 kg

Seismic load on lower lateral. (moving load assumed)
Floor and pavement 9.00 @ 510 = 4,590
copings 2 @ 255 = 510
handrails assumed 2 @ 120 = 240
stringers 6 @ 62 = 372
floor beams 2240 ÷ 4.65 = 482
lower lateral bracing assumed 120
main trusses assumed ½ × 172000 ÷ 65.1 = 1322
electric wiring cables and other fittings say 54
7690 kg per lin meter.

Seismic panel load = 7690 × 4.65 × 0.30 = 10,730 kg.

Seismic load on panel point 1 from upper lateral bracings.

		Load on panel point 1		Load on panel point 1'
For full load (1'-1)	$2470 \times \frac{48}{57.6} \times 72'$ =	14800 kg	$2470 \times \frac{48}{57.6} \times 72'$ =	14800 kg
load on 1'~2	〃 〃 66 =	13600	〃 〃 72' =	14,800
〃 1'~3	〃 〃 55 =	11320	〃 〃 71 =	14,620
〃 1'~4	〃 〃 45 =	9270	〃 〃 69 =	14,200
〃 1'~5	〃 〃 36 =	7410	〃 〃 66 =	13,580
〃 1'~6	〃 〃 28 =	5760	〃 〃 62 =	12760
〃 1'~7	〃 〃 21 =	4320	〃 〃 57 =	11,740



Diagonal length = $\sqrt{5.05^2 + 4.65^2}$ = 6.86 meters
Coefficient = 6.86 ÷ 5.05 = 1.36

Each diagonal to be hanged vertically from stringers S2, and so the unsupported length with respect to horizontal axis will be devided into following two parts

$l' = 6.86 \times \frac{2.55}{5.05} = 3.46$ meters
$l'' = 6.86 \times \frac{2.50}{5.05} = 3.40$ 〃
$l = l' + l'' = 6.86$ 〃

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Design of Kisogawa-Bashi for Gifu-Ken.

Seismic stresses on bottom lateral members.

Stresses due to seismic loads on bottom lateral only

Members		Shear	stress for one member
0-1	10,730 × $\frac{4.65}{65.10}$ × 91 =	69,800 × $\frac{1.36}{2}$ =	47,500 kg T or C.
1-2	〃 〃 × 78 =	59,800 〃 =	40,700
2-3	〃 〃 × 66 =	50,600 〃 =	34,400
3-4	〃 〃 × 55 =	42,200 〃 =	28,700
4-5	〃 〃 × 45 =	34,500 〃 =	23,500
5-6	〃 〃 × 36 =	27,600 〃 =	18,800
6-7	〃 〃 × 28 =	21,500 〃 =	14,600

Stresses due to horizontal thrusts on panel points 1 and 1' transmitted from upper lateral through portals.
Loads corresponding to above stresses for each member.

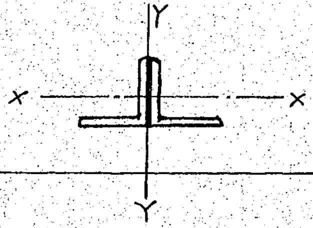
members.					Shear	
0-1	13/14 × 14800 =	13,750 + 1/14 × 14800 =	1,060	− 0	14,810 × $\frac{1.36}{2}$ =	10,070 kg T or C
1-2	〃 × 13600 =	12630 + 〃 × 14800 =	1060	− 13,600	90 〃	60
2-3	〃 × 11320 =	10520 + 〃 × 14620 =	1040	− 11320	240 〃	160
3-4	〃 × 9270 =	8610 + 〃 × 14200 =	1010	− 9270	350 〃	240
4-5	〃 × 7410 =	6880 + 〃 × 13580 =	970	− 7410	440 〃	300
5-6	〃 × 5760 =	5350 + 〃 × 12760 =	910	− 5760	500 〃	340
6-7	〃 × 4320 =	4010 + 〃 × 11740 =	840	− 4320	530 〃	360

Summary of seismic stresses in lateral members.

Members	loads on bottom lateral	loads from upper lateral	Total stress	SR. for tension f = 1920 kg/cm²	22ᶲ rivet no reqd. r = 4560 kg	
0-1	47500	10,070	57,570 kg T or C	30.00 cm² net	12.6	use 18
1-2	40700	60	40,760	21.25	8.9	12
2-3	34400	160	34,560	18.00	7.6	12
3-4	28700	240	28,940	15.08	6.4	10
4-5	23500	300	23,800	12.40	5.2	8
5-6	18800	340	19,140	9.96	4.2	8
6-7	14600	360	14,960	7.79	3.3	8

Sections used.

members 0-1.



2Ls 150 × 150 × 11 = 64.00 − 11.00 = 53.00 cm² net

moment of inertia I_y 2@ 663.3 + 64.00 × 4.07² = 2387 cm⁴

radius of gyration r_y = $\sqrt{\frac{2387}{64}}$ = 6.11 cm

Slenderness ratio l/r_y = 686/6.11 = 112.2

radius of gyration r_x = 4.57 cm

Slenderness ratio l'/r_x = 346/4.57 = 75.8

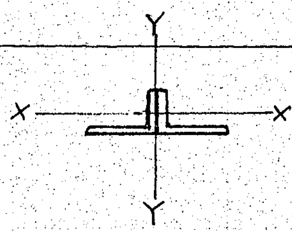
allowable unit compression $f = 1500(1 - 0.0033 \times \overset{.383}{112.2}) = 574 \times 1.6 = 920$ kg/cm²

This section is good for the seismic stresses

Tension of 53.00 @ 1920 = 101,800 kg
Compression of 64.00 @ 920 = 58,900 〃

members 1-2 and 2-3.

2Ls 150 × 100 × 12 = 57.12 − 12.00 = 45.12 cm² net



I_y 2@ 640.2 + 57.12 × 4.88² = 2640 cm⁴

r_y = $\sqrt{\frac{2640}{57.12}}$ = 6.80 cm l/r_y = 686/6.80 = 101.

r_x = 2.83 〃 l'/r_x = 346/2.83 = 122.2

allowable unit comp. $f = 1500(1 - 0.0055 \times \overset{.328}{122.2}) = 492 \times 1.6 = 787$ kg/cm²

Good for Tension of 45.12 @ 1920 = 86,600 kg
〃 comp. of 57.12 @ 787 = 45,000 〃

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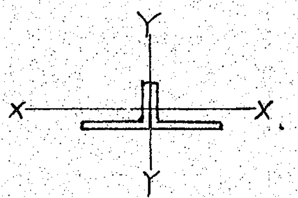
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

member 3-4



2Ls 150 × 100 × 9 = 43.68 − 9 = 34.68 cm² net

$I_y = 2 @ 486.4 + 43.68 \times 4.77^2 = 1967$ cm⁴

$r_y = \sqrt{\frac{1967}{43.68}} = 6.71$ cm $\quad l/r_y = 686/6.71 = 102.2$

$r_x = 2.84$ 〃 $\quad l'/r_x = 346/2.84 = 121.9$

Allowable unit. comp. $f = 1500(1 - \overset{.329}{0.0055 \times 121.9}) = 493 \times 1.6 = 789$ kg/cm²

This section good for
Tension of 34.68 @ 1920 = 66500 kg
Compression of 43.68 @ 789 = 34450 〃

member 4-5, 5-6, + 6-7.

2Ls 125 × 90 × 9 = 37.08 − 9 = 28.08 cm² net

$I_y = 2 @ 287.4 + 37.08 \times 3.91^2 = 1142$ cm⁴

$r_y = \sqrt{\frac{1142}{37.08}} = 5.55$ cm $\quad l/r_y = 686/5.55 = 123.6$

$r_x = 2.60$ 〃 $\quad l'/r_x = 346/2.60 = 133.2$

Allowable unit comp. $f = 1500(1 - \overset{.268}{0.0055 \times 133.2}) = 402 \times 1.6 = 643$ kg/cm²

This section good for
Tension of 28.08 @ 1920 = 53900 kg
Compression of 37.08 @ 643 = 23850 〃

Approximate weight of lower lateral bracings.

members.						
0 − 1	8Ls	150 × 150 × 11	@ 25.10 kg	× average 6.25 m	=	1,255
1 − 2 and 2 − 3	16Ls	150 × 100 × 12	@ 22.40	× 6.25	=	2240
3 − 4	8Ls	150 × 100 × 9	@ 17.10	× 6.25	=	855
4-5, 5-6, and 6-7	24Ls	125 × 90 × 9	@ 14.60	× 6.25	=	2190
Center connections	2		@ 80.00		=	160
〃 〃	2		@ 65.00		=	130
〃 〃	3		@ 50.00		=	150
hangers from stringers	28		@ 20.00		=	560
Rivet heads and variations say 3½ %					=	264
						7804 Call this 7810 kg

Weight of lower lateral per lin meter of bridge = 7810 ÷ 65.10 = 120 kg.

Wind and seismic stresses in tie as chord members of lower lateral bracing.

Seismic stresses.

Members.	moment due to load on lower lateral	Due to load from upper lateral	Total moment	Seismic stress in tie
		14800 × 4.650 =		
0 − 2	69,800 × 4.650 = 325,000	+ 69,000	= 394,000 kgm ÷ 10.10 =	39,000 kg T or C
2 − 4	69,800 × 4.650 × 3 = 975,000 10730 × 4.650 × 3 = −150,000 825,000	+ 69,000	= 894,000 ÷ 10.10 =	88,500 〃
4 − 6	69800 × 4.650 × 5 = 1,623,000 10730 × 4.650 × 10 = −499,000 1,124,000	+ 69000	= 1,193,000 ÷ 10.10 =	118,200 〃
6 − 7	69800 × 4.650 × 7 = 2272,000 10730 × 4.650 × 21 = −1048000 1,224,000	+ 69,000	= 1293,000 ÷ 10.10 =	128,000 〃

Wind stresses. (By proportions of panel loads)

		$69000 \times \frac{1070}{2470} =$	Total moment	Wind stress in tie
0 − 2	$325,000 \times \frac{2000}{10730} = 60500$	+ 29900	= 90400 kgm ÷ 10.10 =	9,000 kg T or C
2 − 4	825,000 × 〃 = 153700	+ 〃	= 183600 ÷ 〃 =	18,200 〃
4 − C	1,124,000 × 〃 = 209500	+ 〃	= 239400 ÷ 〃 =	23700 〃
6 − 7	1224,000 × 〃 = 228,000	+ 〃	= 257,900 ÷ 〃 =	25,500 〃

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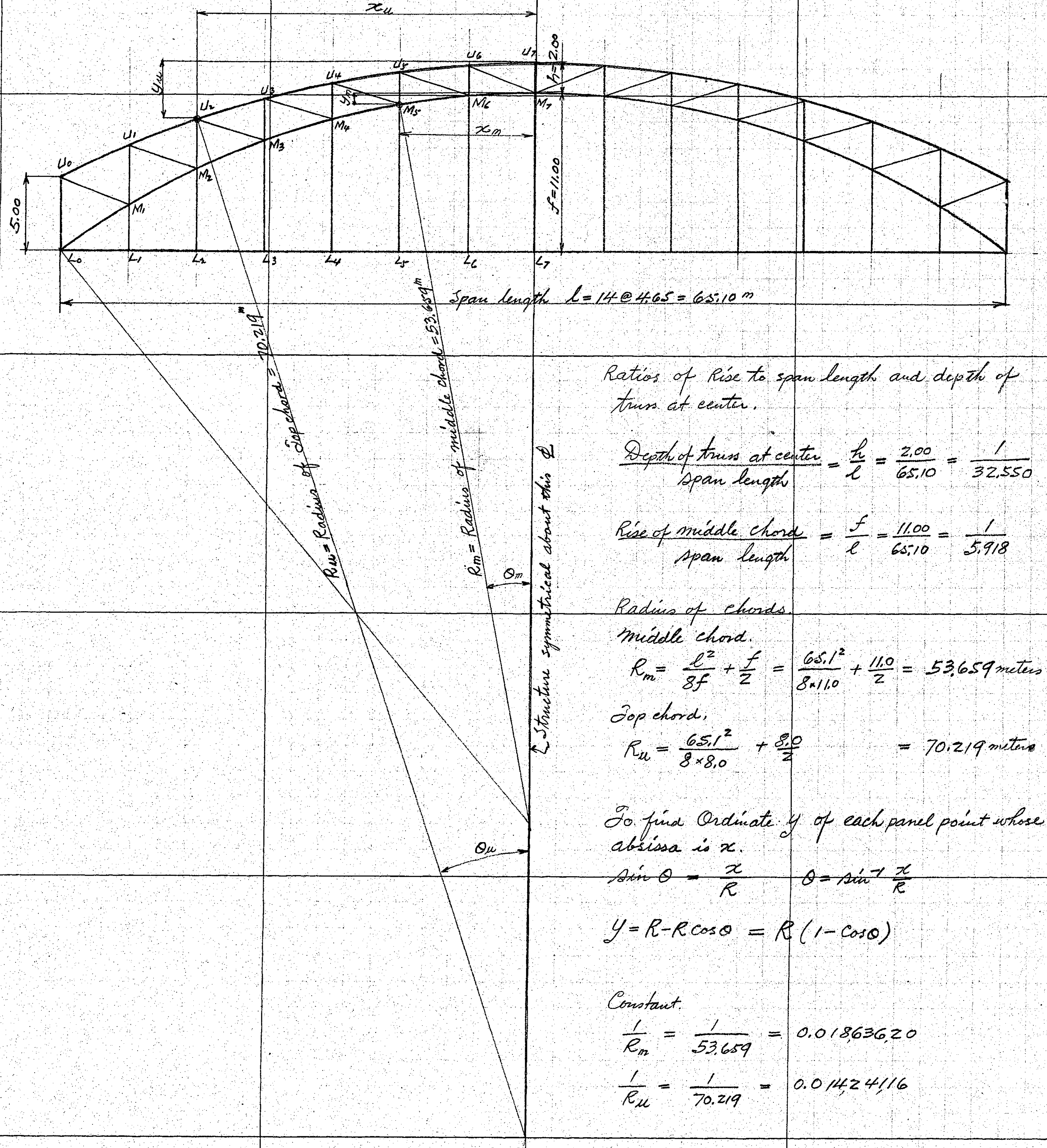
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu Ken.

Design of main truss.

Top and middle chords circular curves.

General dimensions of truss as shown on sketch below.



Ratios of Rise to span length and depth of truss at center.

$$\frac{\text{Depth of truss at center}}{\text{span length}} = \frac{h}{l} = \frac{2.00}{65.10} = \frac{1}{32.550}$$

$$\frac{\text{Rise of middle chord}}{\text{span length}} = \frac{f}{l} = \frac{11.00}{65.10} = \frac{1}{5.918}$$

Radius of chords.

middle chord.

$$R_m = \frac{l^2}{8f} + \frac{f}{2} = \frac{65.1^2}{8 \times 11.0} + \frac{11.0}{2} = 53.659 \text{ meters}$$

Top chord.

$$R_u = \frac{65.1^2}{8 \times 8.0} + \frac{8.0}{2} = 70.219 \text{ meters}$$

To find Ordinate y of each panel point whose absissa is x.

$$\sin\theta = \frac{x}{R} \qquad \theta = \sin^{-1}\frac{x}{R}$$

$$y = R - R\cos\theta = R(1-\cos\theta)$$

Constant.

$$\frac{1}{R_m} = \frac{1}{53.659} = 0.018636{,}20$$

$$\frac{1}{R_u} = \frac{1}{70.219} = 0.014{,}24{,}116$$

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CALCULATIONS FOR
Design of Kisogawa-Bashi for Gifu-Ken.

Upper chord.
Summary of Ordinates from appex.

Panel points	x in meter	$\sin\theta_u = x_u/R_u$	Angle θ_u	$\cos\theta_u$	$1-\cos\theta_u$	$y_u = R_u(1-\cos\theta_u)$
7	0.000	0.000,000,00	0° - 0.000′	1.000 000 0	0.000 000 0	0.0000 m
6	4.650	0.0662214	3°-47.819′	0.997 805 0	0.002 195 0	0.1541
5	9.300	0.1324428	7°- 36.646′	0.991 190 6	0.008 8094	0.6186
4	13.950	0.1986642	11°- 27.531′	0.980 067 6	0.019 9324	1.3996
3	18.600	0.2648856	15°- 21.609′	0.964 2799	0.0357201	2.5082
2	23.250	0.3311070	19°- 20.159′	0.943 5932	0.056 4068	3.9608
1	27.900	0.3973284	23°- 24.676′	0.917 6765	0.082 3235	5.7807
0	32.550	0.4635498	27°- 36.984′	0.886 0710	0.113 9290	8.0000

Middle chord
Summary of Ordinates from appex.

Panel Points	x in meter	$\sin\theta_m = x_m/R_m$	Angle θ_m	$\cos\theta_m$	$1-\cos\theta_m$	$y_m = R_m(1-\cos\theta_m)$
7	0.000	0.000 000 0	0°- 0.000′	1.000 000 0	0.000 000 0	0.0000 m
6	4.650	0.086 6583	4°- 58.283′	0.996 2381	0.003 7619	0.2019
5	9.300	0.173 3167	9°- 58.843′	0.984 8661	0.015 1339	0.8121
4	13.950	0.259 9750	15°- 4.115′	0.965 6153	0.034 3847	1.8450
3	18.600	0.346 6333	20°- 16.892′	0.938 0006	0.061 9994	3.3268
2	23.250	0.433 2917	25°- 40.598′	0.901 2538	0.098 7462	5.2986
1	27.900	0.519 9500	31°- 19.734′	0.854 1967	0.145 8033	7.8237
0	32.550	0.606 6083	37°- 20.680′	0.795 0009	0.204 9991	11.0000

Lengths of Truss members.
Vertical members S_m

Panel Points	Middle chord Ordinates (+)		Upper chord Ordinates (−)		Depth of truss at center h.(+)		Length of verticals. S_m		
7	0.0000	−	0.0000	+	2.0000	=	2.0000 m	or	2.000
6	0.2019		0.1541		〃		2.0478		2.048
5	0.8121		0.6186		〃		2.1935		2.194
4	1.8450		1.3996		〃		2.4454		2.445
3	3.3268		2.5082		〃		2.8186		2.819
2	5.2986		3.9608		〃		3.3378		3.338
1	7.8237		5.7807		〃		4.0430		4.043
0	11.0000		8.0000		〃		5.0000		5.000

Hangers r_m

Panel points	f		y_m				r_m
7	11.0000	−	0.0000	=	11.0000 m	or	11.000
6	〃		0.2019		10.7981		10.798
5	〃		0.8121		10.1879		10.188
4	〃		1.8450		9.1550		9.155
3	〃		3.3268		7.6732		7.673
2	〃		5.2986		5.7014		5.701
1	〃		7.8237		3.1763		3.176
0	〃		11.0000		0.0000		0.000

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-ken.

Upper chord members U_m

panel point	Diff. of height	A	Panel length B	A^2+B^2	$\sqrt{A^2+B^2} = U_m$	
7	0.1541 - 0.0000 =	0.1541	4.650	21.64625	4.653	(6-7)
6	0.6186 - 0.1541 =	0.4645	〃	21.83826	4.673	
5	1.3996 - 0.6186 =	0.7810	〃	22.23246	4.715	
4	2.5082 - 1.3996 =	1.1086	〃	22.85149	4.780	
3	3.9608 - 2.5082 =	1.4526	〃	23.73255	4.872	
2	5.7807 - 3.9608 =	1.8199	〃	24.93454	4.993	
1	8.0000 - 5.7807 =	2.2193	〃	26.54779	5.152	(0-1)
0						

Middle chord members b_m

panel point	Diff. of height	A	Panel length B	A^2+B^2	$\sqrt{A^2+B^2} = b_m$	
7						
6	0.2019 - 0.0000 =	0.2019	4.650	21.66326	4.654	(6-7)
5	0.8121 - 0.2019 =	0.6102	〃	21.99484	4.690	
4	1.8450 - 0.8121 =	1.0329	〃	22.68938	4.763	
3	3.3268 - 1.8450 =	1.4818	〃	23.81823	4.880	
2	5.2986 - 3.3268 =	1.9718	〃	25.51050	5.051	
1	7.8237 - 5.2986 =	2.5251	〃	27.99863	5.291	
0	11.0000 - 7.8237 =	3.1763	〃	31.71138	5.631	(0-1)

Diagonal members. d_m

panel point	Diff. of height	A	Panel length B	A^2+B^2	$\sqrt{A^2+B^2} = d_m$	
7	12.8459 - 11.0000 =	1.8459	4.650	25.02985	5.003	(6-7)
6	12.3814 - 10.7981 =	1.5833	〃	24.12934	4.912	
5	11.6004 - 10.1879 =	1.4125	〃	23.61766	4.860	
4	10.4918 - 9.1550 =	1.3368	〃	23.40953	4.838	
3	9.0392 - 7.6732 =	1.3660	〃	23.48846	4.846	
2	7.2193 - 5.7014 =	1.5179	〃	23.92652	4.891	
1	5.0000 - 3.1763 =	1.8237	〃	24.94838	4.995	(0-1)
0						

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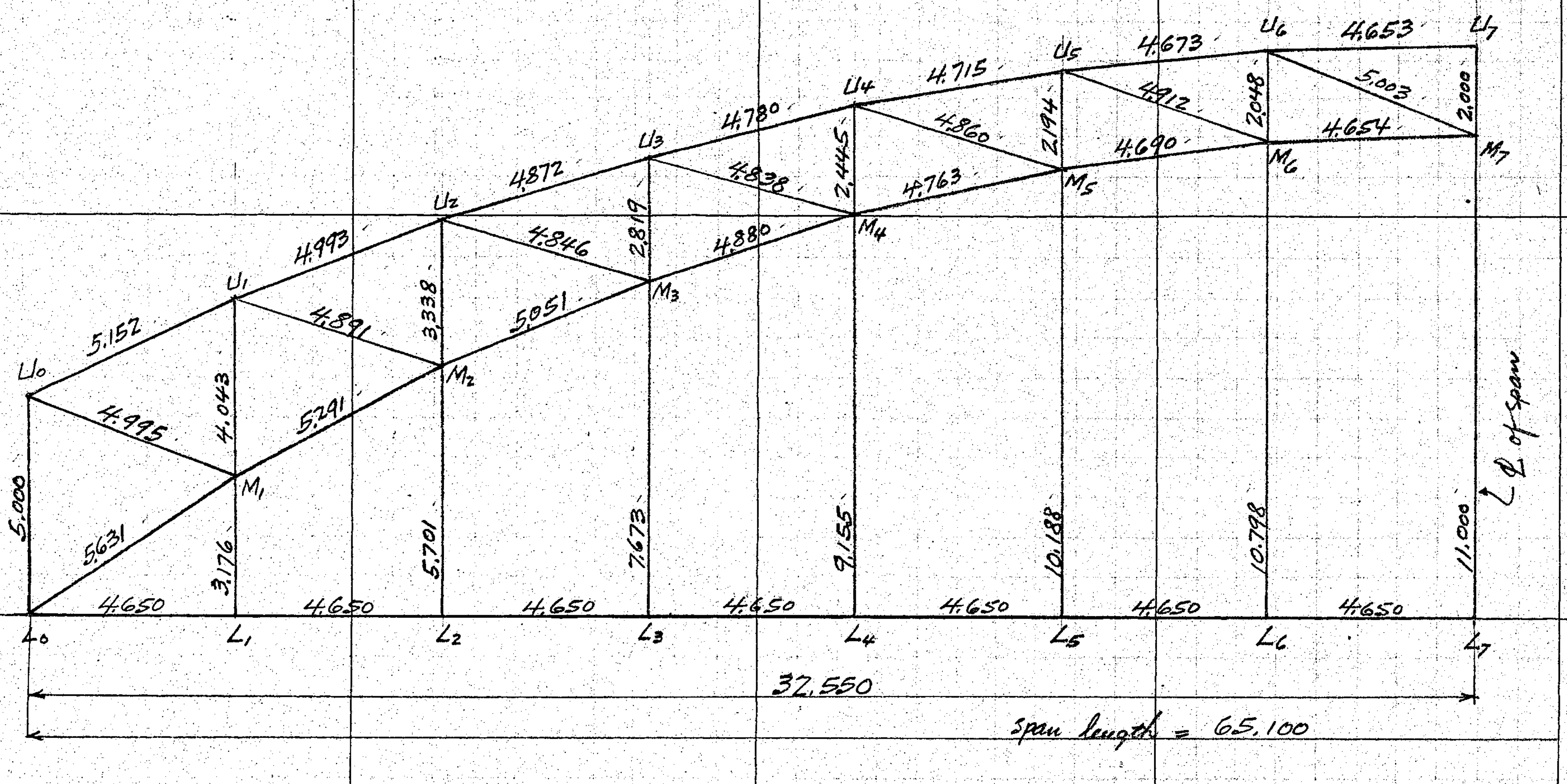
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-ken

Summary for length of members in meter.

members	Upper chords Um	Middle chords bm	Verticals Sm	Diagonals dm	Hangers. rm
7	4.653	—	2.000	5.003	11.000
6	4.673	4.654	2.048	4.912	10.798
5	4.715	4.690	2.194	4.860	10.188
4	4.780	4.763	2.445	4.838	9.155
3	4.872	4.880	2.819	4.846	7.673
2	4.993	5.051	3.338	4.891	5.701
1	5.152	5.291	4.043	4.995	3.176
0	—	5.631	5.000	—	0.000



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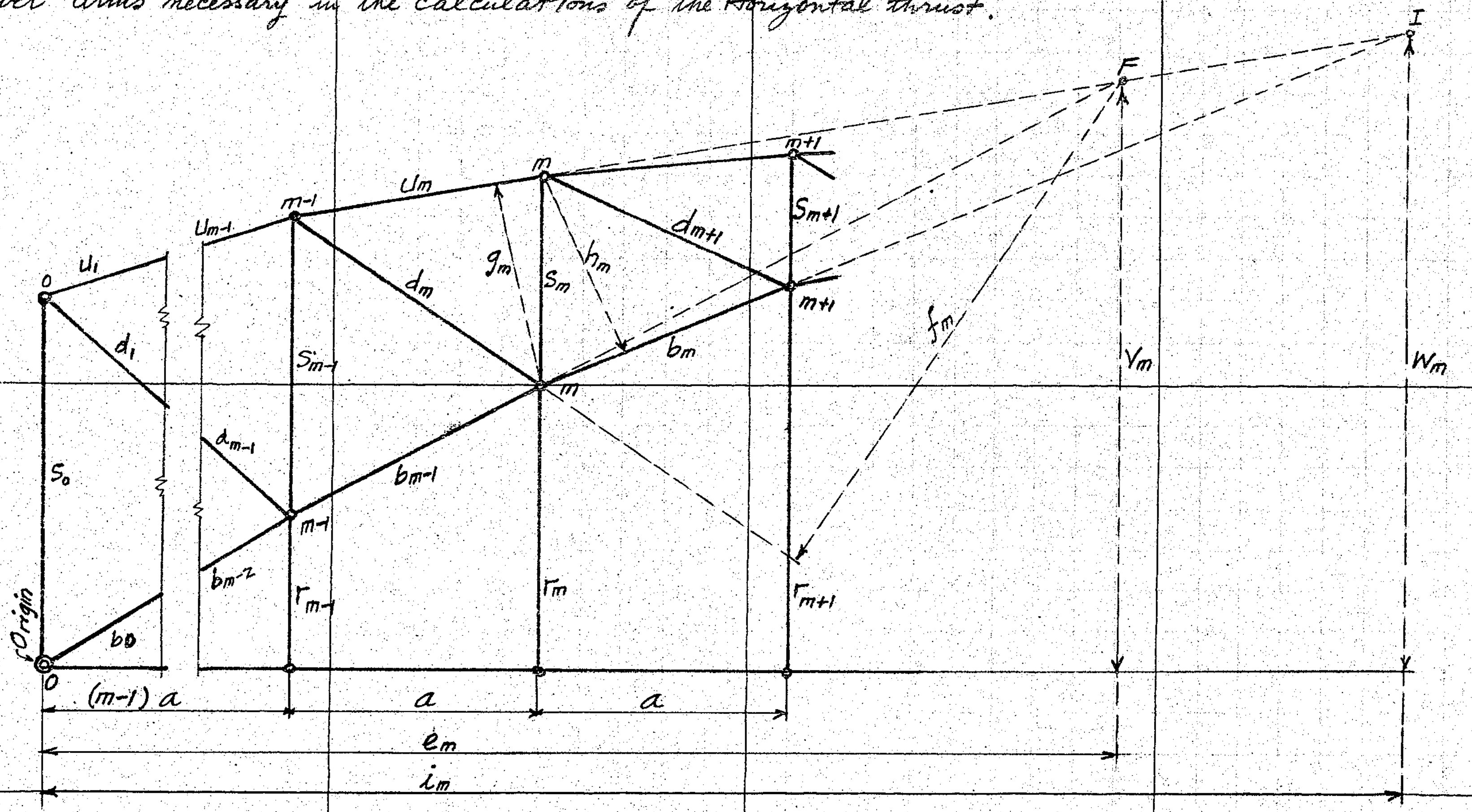
CALCULATIONS FOR

Design of Kiso-gawa Bashi for Gifu-Ken

Lever arms necessary in the calculations of the Horizontal thrust.



Notation.

S_m = Length of verticals;
d_m = length of diagonals;
U_m = length of upper chords;
b_m = length of middle chords;
r_m = length of hangers;
a = panel length;
g_m = perpendicular distance from lower panel point m to upper chord U_m;
h_m = perpendicular distance from upper panel point m to middle chord b_m;
V_m = vertical distance from intersection point F of chords U_m and b_{m-1} to origin 0;
f_m = perpendicular distance from F to diagonal d_m;
W_m = vertical distance from intersection point I of chords U_m and b_m to origin 0;
e_m = horizontal distance from intersection point F to origin 0;
i_m = horizontal distance from intersection point I to origin 0.

Then we have in general as follows:

$$g_m = \frac{a}{U_m} S_m ; \checkmark$$

$$h_m = \frac{a}{b_m} S_m ; \checkmark$$

$$V_m = \frac{S_{m-1}}{S_{m-1} - S_m} (r_m - r_{m-1}) + r_{m-1} ; \checkmark$$

$$f_m = \frac{S_{m-1}}{S_{m-1} - S_m} \cdot \frac{a}{d_m} \cdot S_m ; \checkmark$$

$$e_m = \frac{S_{m-1}}{S_{m-1} - S_m} \cdot a + (m-1)a ; \checkmark$$

$$i_m = \frac{S_m}{(r_{m+1} - r_m) - (y_m - y_{m-1})} \cdot a + ma ; \checkmark$$

$$W_m = \frac{S_m}{(r_{m+1} - r_m) - (y_m - y_{m-1})} \cdot (r_{m+1} - r_m) + r_m . \checkmark$$

where

$$y_m = r_m + S_m \checkmark$$

$$y_{m-1} = r_{m-1} + S_{m-1} . \checkmark$$

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Find $g_m = \frac{a}{U_m} \cdot S_m$ where $a = 4.650$

Panel point no. m	a	U_m	S_m	$a\,S_m$	g_m
1	4.650	5.152	4.043	18.8000	3.649
2	〃	4.993	3.338	15.5217	3.109
3	〃	4.872	2.819	13.1084	2.691
4	〃	4.780	2.445	11.3693	2.379
5	〃	4.715	2.194	10.2021	2.164
6	〃	4.673	2.048	9.5232	2.038
7		4.653	2.000	9.3000	1.999

Find $h_m = \frac{a}{b_m} \cdot S_m$

Panel point no. m.	a	b_m	S_m	$a.S_m$	h_m
0	4.650	5.631	5.000	23.2500	4.129
1	〃	5.291	4.043	18.8000	3.553
2	〃	5.051	3.338	15.5217	3.073
3	〃	4.880	2.819	13.1084	2.686
4	〃	4.763	2.445	11.3693	2.387
5	〃	4.690	2.194	10.2021	2.175
6	〃	4.654	2.048	9.5232	2.046

Find $V_m = \frac{S_{m-1}}{S_{m-1} - S_m}(r_m - r_{m-1}) + r_{m-1}$.

m.	S_{m-1}	S_m	$S_{m-1} - S_m$	r_m	r_{m-1}	$r_m - r_{m-1}$	$S_{m-1}(r_m - r_{m-1})$	$\frac{S_{m-1}}{S_{m-1}-S_m}(r_m - r_{m-1})$	V_m
1	5.000	4.043	0.957	3.176	0.000	3.176	15.8800	16.594	16.594
2	4.043	3.338	0.705	5.701	3.176	2.525	10.2086	14.480	17.656
3	3.338	2.819	0.519	7.673	5.701	1.972	6.5825	12.683	18.384
4	2.819	2.445	0.374	9.155	7.673	1.482	4.1778	11.171	18.844
5	2.445	2.194	0.251	10.188	9.155	1.033	2.5257	10.063	19.218
6	2.194	2.048	0.146	10.798	10.188	0.610	1.3383	9.166	19.354
7	2.048	2.000	0.048	11.000	10.798	0.202	0.4137	8.619	19.417

Find $f_m = \frac{S_{m-1}}{S_{m-1} - S_m} \cdot \frac{a}{d_m} \cdot S_m$. $a = 4.650$

m	S_{m-1}	$a\,S_{m-1}$	S_m	$S_{m-1}\,a.S_m$	$S_{m-1} - S_m$	d_m	$(S_{m-1} - S_m)d_m$	f_m
1	5.000	23.2500	4.043	93.9998	0.957	4.995	4.7802	19.664
2	4.043	18.8000	3.338	62.7544	0.705	4.891	3.4482	18.199
3	3.338	15.5217	2.819	43.7557	0.519	4.846	2.5151	17.397
4	2.819	13.1084	2.445	32.0500	0.374	4.838	1.8094	17.713
5	2.445	11.3693	2.194	24.9442	0.251	4.860	1.2199	20.448
6	2.194	10.2021	2.048	20.8939	0.146	4.912	0.7172	29.133
7	2.048	9.5232	2.000	19.0464	0.048	5.003	0.2401	79.327

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Find $e_m = \frac{s_{m-1}}{s_{m-1}-s_m} \cdot a + (m-1)a$.

m	$s_{m-1} \cdot a$	$s_{m-1}-s_m$	$\frac{s_{m-1}}{s_{m-1}-s_m} \cdot a$	$(m-1)a$	e_m
1	23.2500	0.957	24.295	0.000	24.295
2	18.8000	0.705	26.667	4.650	31.317
3	15.5217	0.519	29.907	9.300	39.207
4	13.1084	0.374	35.049	13.950	48.999
5	11.3693	0.251	45.296	18.600	63.896
6	10.2021	0.146	69.877	23.250	93.127
7	9.5232	0.048	198.400	27.900	226.300

Find $i_m = \frac{s_m}{(r_{m+1}-r_m)-(y_m-y_{m-1})} \cdot a + ma = i_m' + ma$ where $y_m - y_{m-1} = (r_m - r_{m-1}) - (s_{m-1} - s_m)$

m	$s_m \cdot a$	$r_{m+1}-r_m$	$y_m - y_{m-1}$	$(r_{m+1}-r_m)-(y_m-y_{m-1})$	i_m'	ma	i_m
0	23.2500	3.176	—	3.176	—	—	—
1	18.8000	2.525	2.219	0.306	61.438	4.650	66.088
2	15.5217	1.972	1.820	0.152	102.116	9.300	111.416
3	13.1084	1.482	1.453	0.029	452.014	13.950	465.964
4	11.3693	1.033	1.108	−0.075	−151.591	18.600	−132.991
5	10.2021	0.610	0.782	−0.172	−59.315	23.250	−36.065
6	9.5232	0.202	0.464	−0.262	−36.348	27.900	−8.448

Find $W_m = \frac{s_m}{(r_{m+1}-r_m)-(y_m-y_{m-1})}(r_{m+1}-r_m) + r_m = W_m' + r_m$

m	s_m	$r_{m+1}-r_m$	$s_m(r_{m+1}-r_m)$	$(r_{m+1}-r_m)-(y_m-y_{m-1})$	W_m'	r_m	W_m
1	4.043	2.525	10.2086	0.306	33.361	3.176	36.537
2	3.338	1.972	6.5825	0.152	43.306	5.701	49.007
3	2.819	1.482	4.1778	0.029	144.062	7.673	151.735
4	2.445	1.033	2.5257	−0.075	−33.676	9.155	−24.521
5	2.194	0.610	1.3383	−0.172	−7.781	10.188	2.407
6	2.048	0.202	0.4137	−0.262	−1.579	10.798	9.219

Summary.

m	U_m	b_m	s_m	d_m	r_m	g_m	h_m	V_m	f_m	e_m	i_m	W_m	$i_m - ma$	m
0	—	5.631	5.000	—	0.000	—	4.129	—	—	—	—	—	—	0
1	5.152	5.291	4.043	4.995	3.176	3.649	3.553	16.594	19.664	24.295	66.088	36.537	61.438	1
2	4.993	5.051	3.338	4.891	5.701	3.109	3.073	17.656	18.199	31.317	111.416	49.007	102.116	2
3	4.872	4.880	2.819	4.846	7.673	2.691	2.686	18.384	17.397	39.207	465.964	151.735	452.014	3
4	4.780	4.763	2.445	4.838	9.155	2.379	2.387	18.844	17.713	48.999	−132.991	−24.521	−151.591	4
5	4.715	4.690	2.194	4.860	10.188	2.164	2.175	19.218	20.448	63.896	−36.065	2.407	−59.315	5
6	4.673	4.654	2.048	4.912	10.798	2.038	2.046	19.354	29.133	93.127	−8.448	9.219	−36.348	6
7	4.653	—	2.000	5.003	11.000	1.999	—	19.417	79.327	226.300	—	—	—	7

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CALCULATIONS FOR

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Calculation of Horizontal Thrust H:

General formula,

$$H = \frac{\Sigma S_0 S_a \frac{l}{A}}{\Sigma S_a^2 \frac{l}{A}} \quad \text{for unit load.}$$

Where H = Horizontal thrust due to unity;
S_0 = Stress of each member due to unit load applied and redundancy removed;
S_a = Stress of each member when $H = -1$ kg applied;
l = length of each member in centimeters;
A = gross sectional area of each member in square centimeters.

Stresses of each member for load unity and redundancy removed. (S_0)

Upper chord members.

$m=1$ (U_0-U_1) $S_{0.1} = -\frac{13}{14}\cdot\frac{a}{g_1} = -\frac{13}{14}\cdot\frac{4650}{3.649} = -13\times 0.0910 = 13 S_{0.1}'$

2 (U_1-U_2) $S_{0.2} = -\frac{12}{14}\cdot\frac{2a}{g_2} = -\frac{12}{14}\cdot\frac{2\times4650}{3.109} = -12\times 0.2137 = 12 S_{0.1}'$

$S_{0.1} = -\frac{1}{14}\cdot\frac{12a}{g_2} = -\frac{1}{14}\cdot\frac{12\times4650}{3.109} = -1\times 1.2820$

3 (U_2-U_3) $S_{0.3} = -\frac{11}{14}\cdot\frac{3a}{g_3} = -\frac{11}{14}\cdot\frac{3\times4.650}{2.691} = -11\times 0.3703 = 11.S_{0.1}'$

$S_{0.2} = -\frac{2}{14}\cdot\frac{11a}{g_3} = -\frac{2}{14}\cdot\frac{11\times4.650}{2.691} = -2\times 1.3577 = 2.S_{0.1}$

4 (U_3-U_4) $S_{0.4} = -\frac{10}{14}\cdot\frac{4a}{g_4} = -\frac{10}{14}\cdot\frac{4\times4.650}{2.379} = -10\times 0.5585 = 10.S_{0.1}'$

$S_{0.3} = -\frac{3}{14}\cdot\frac{10a}{g_4} = \frac{3}{14}\cdot\frac{10\times4650}{2.379} = -3\times 1.3961 = 3.S_{0.1}$

5. (U_4-U_5) $S_{0.5} = -\frac{9}{14}\cdot\frac{5a}{g_5} = -\frac{9}{14}\cdot\frac{5\times4.650}{2.164} = -9\times 0.7674 = 9.S_{0.1}'$

$S_{0.4} = -\frac{4}{14}\cdot\frac{9a}{g_5} = -\frac{4}{14}\cdot\frac{9\times4.650}{2.164} = -4\times 1.3814 = 4.S_{0.1}$

6. (U_5-U_6) $S_{0.6} = -\frac{8}{14}\cdot\frac{6a}{g_6} = -\frac{8}{14}\cdot\frac{6\times4.650}{2.038} = -8\times 0.9778 = 8.S_{0.1}'$

$S_{0.5} = \frac{5}{14}\cdot\frac{8a}{g_6} = -\frac{5}{14}\cdot\frac{8\times4.650}{2.038} = -5\times 1.3038 = 5.S_{0.1}$

7. (U_6-U_7) $S_{0.7} = -\frac{7}{14}\cdot\frac{7a}{g_7} = -\frac{7}{14}\cdot\frac{7\times4.650}{1.999} = -7\times 1.1631 = 7.S_{0.1}'$

$S_{0.6} = -\frac{6}{14}\cdot\frac{7a}{g_7} = -\frac{6}{14}\cdot\frac{7\times4.650}{1.999} = -6\times 1.1631 = 6.S_{0.1}.$

Middle chord members.

$m=0$ (L_0-M_1) $S_{0.1} = S_{0.2} = S_{0.3}$ - - - - - - - $= S_{0.2}' = S_{0.1}' = 0.000$

1 (M_1-M_2) $S_{0.1} = \frac{13}{14}\cdot\frac{a}{h_1} = \frac{13}{14}\cdot\frac{4650}{3.553} = 13\times 0.0935 = 13 S_{0.1}'$

2 (M_2-M_3) $S_{0.2} = \frac{12}{14}\cdot\frac{2a}{h_2} = \frac{12}{14}\cdot\frac{2\times4650}{3.073} = 12\times 0.2162 = 12.S_{0.1}'$

$S_{0.1} = \frac{1}{14}\cdot\frac{12a}{h_2} = \frac{1}{14}\cdot\frac{12\times4650}{3.073} = 1\times 1.2970$

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3. (M_3-M_4)

$$S_{0.3} = \frac{11}{14}\cdot\frac{3a}{h_3} = \frac{11}{14}\cdot\frac{3\times4.650}{2.686} = 11\times0.3710 = 11\cdot S_{0.1}'$$

$$S_{0.2} = \frac{2}{14}\cdot\frac{11a}{h_3} = \frac{2}{14}\cdot\frac{11\times4.650}{2.686} = 2\times1.3602 = 2.S_{0.1}$$

4. (M_4-M_5)

$$S_{0.4} = \frac{10}{14}\cdot\frac{4a}{h_4} = \frac{10}{14}\cdot\frac{4\times4.650}{2.387} = 10\times0.5566 = 10\cdot S_{0.1}'$$

$$S_{0.3} = \frac{3}{14}\cdot\frac{10a}{h_4} = \frac{3}{14}\cdot\frac{10\times4650}{2.387} = 3\times1.3915 = 3.S_{0.1}$$

5 (M_5-M_6).

$$S_{0.5} = \frac{9}{14}\cdot\frac{5a}{h_5} = \frac{9}{14}\cdot\frac{5\times4.650}{2.175} = 9\times0.7635 = 9.S_{0.1}'$$

$$S_{0.4} = \frac{4}{14}\cdot\frac{9a}{h_5} = \frac{4}{14}\cdot\frac{9\times4.650}{2.175} = 4\times1.3744 = 4.S_{0.1}$$

6 (M_6-M_7)

$$S_{0.6} = \frac{8}{14}\cdot\frac{6a}{h_6} = \frac{8}{14}\cdot\frac{6\times4.650}{2.046} = 8\times0.9740 = 8.S_{0.1}'$$

$$S_{0.5} = \frac{5}{14}\cdot\frac{8a}{h_6} = \frac{5}{14}\cdot\frac{8\times4.650}{2.046} = 5\times1.2987 = 5.S_{0.1}$$

Diagonal members
$m = 1$ (U_0-M_1)

$$S_{0.1} = \frac{13}{14}\times\frac{e_1}{f_1} = \frac{13}{14}\cdot\frac{24.295}{19.664} = 13\times0.0883 = 13.S_{0.1}'$$

2 (U_1-M_2)

$$S_{0.2} = \frac{12}{14}\times\frac{e_2}{f_2} = \frac{12}{14}\cdot\frac{31.317}{18.199} = 12\times0.1229 = 12.S_{0.1}'$$

$$S_{0.1} = \frac{13}{14}\times\frac{e_2}{f_2} - \frac{e_2-a}{f_2} = \frac{1}{f_2}\cdot\left(a-\frac{e_2}{14}\right)$$

$$= \frac{1}{18.199}\left(4.650-\frac{31.317}{14}\right) = 0.1325$$

3. (U_2-M_3)

$$S_{0.3} = \frac{11}{14}\times\frac{e_3}{f_3} = \frac{11}{14}\cdot\frac{39.207}{17.397} = 11\times0.1610 = 11.S_{0.1}'$$

$$S_{0.2} = \frac{2}{14}\cdot\frac{l-e_3}{f_3} = \frac{2}{14}\cdot\frac{65.1-39.207}{17.397} = 2\times0.1063 = 2.S_{0.1}$$

4. (U_3-M_4)

$$S_{0.4} = \frac{10}{14}\times\frac{e_4}{f_4} = \frac{10}{14}\cdot\frac{48.999}{17.713} = 10\times0.1976 = 10.S_{0.1}'$$

$$S_{0.3} = \frac{3}{14}\cdot\frac{l-e_4}{f_3} = \frac{3}{14}\cdot\frac{65.1-48.999}{17.713} = 3\times0.0649 = 3.S_{0.1}$$

5 (U_4-M_5)

$$S_{0.5} = \frac{9}{14}\times\frac{e_5}{f_5} = \frac{9}{14}\cdot\frac{63.896}{20.448} = 9\cdot0.2232 = 9.S_{0.1}'$$

$$S_{0.4} = \frac{4}{14}\times\frac{l-e_5}{f_5} = \frac{4}{14}\cdot\frac{65.1-63.896}{20.448} = 4\times0.0042 = 4.S_{0.1}$$

6. (U_5-M_6)

$$S_{0.6} = \frac{8}{14}\cdot\frac{e_6}{f_6} = \frac{8}{14}\cdot\frac{93.127}{29.133} = 8\times0.2283 = 8.S_{0.1}'$$

$$S_{0.5} = \frac{5}{14}\cdot\frac{l-e_6}{f_6} = \frac{5}{14}\cdot\frac{65.1-93.127}{29.133} = -5\times0.0687 = 5.S_{0.1}$$

7. (U_6-M_7)

$$S_{0.7} = \frac{7}{14}\times\frac{e_7}{f_7} = \frac{7}{14}\cdot\frac{226.300}{79.327} = 7\times0.2038 = 7.S_{0.1}'$$

$$S_{0.6} = \frac{6}{14}\times\frac{l-e_7}{f_7} = \frac{6}{14}\cdot\frac{65.1-226.300}{79.327} = -6\times0.1451 = 6\cdot S_{0.1}$$

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Vertical members.

$m = 0 \quad (U_0-L_0)$

$$S_{0.1} = -\frac{13}{14} = -13\times0.0714 = 13\cdot S_{0.1}'$$

$1 \quad (U_1-M_1)$

$$S_{0.2} = -\frac{12}{14}\cdot\frac{i_1}{i_1-a} = -\frac{12}{14}\cdot\frac{66.088}{61.438} = -12\times0.0768 = 12\cdot S_{0.1}'$$

$$S_{0.1} = -\frac{1}{14}\cdot\frac{l-i_1}{i_1-a} = -\frac{1}{14}\cdot\frac{-0.988}{61.438} = 1\times0.0011 = 1\cdot S_{0.1}$$

$2. \quad (U_2-M_2)$

$$S_{0.3} = -\frac{11}{14}\cdot\frac{i_2}{i_2-2a} = -\frac{11}{14}\cdot\frac{111.416}{102.116} = -11\times0.0779 = 11.S_{0.1}'$$

$$S_{0.2} = -\frac{2}{14}\cdot\frac{l-i_2}{i_2-2a} = -\frac{2}{14}\cdot\frac{-46.316}{102.116} = 2\times0.0324 = 2.S_{0.1}$$

$3 \quad (U_3-M_3)$

$$S_{0.4} = -\frac{10}{14}\cdot\frac{i_3}{i_3-3a} = -\frac{10}{14}\cdot\frac{465.964}{452.014} = -10\times0.0736 = 10.S_{0.1}'$$

$$S_{0.3} = -\frac{3}{14}\cdot\frac{l-i_3}{i_3-3a} = -\frac{3}{14}\cdot\frac{-400.864}{452.014} = 3\times0.0633 = 3\cdot S_{0.1}$$

$4. \quad (U_4-M_5)$

$$S_{0.5} = -\frac{9}{14}\cdot\frac{i_4}{i_4-4a} = -\frac{9}{14}\cdot\frac{-132.991}{-151.591} = -9\times0.0627 = 9\cdot S_{0.1}'$$

$$S_{0.4} = -\frac{4}{14}\cdot\frac{l-i_4}{i_4-4a} = -\frac{4}{14}\cdot\frac{198.091}{-151.591} = 4\times0.0933 = 4.S_{0.1}$$

$5. \quad (U_5-M_5)$

$$S_{0.6} = -\frac{8}{14}\cdot\frac{i_5}{i_5-5a} = -\frac{8}{14}\cdot\frac{-36.065}{-59.315} = -8\times0.0434 = 8.S_{0.1}'$$

$$S_{0.5} = -\frac{5}{14}\cdot\frac{l-i_5}{i_5-5a} = -\frac{5}{14}\cdot\frac{101.165}{-59.315} = 5\times0.1218 = 5.S_{0.1}$$

$6 \quad (U_6-M_6)$

$$S_{0.7} = -\frac{7}{14}\cdot\frac{i_6}{i_6-6a} = -\frac{7}{14}\cdot\frac{-8.448}{-36.348} = -7\times0.0166 = 7.S_{0.1}'$$

$$S_{0.6} = -\frac{6}{14}\cdot\frac{l-i_6}{i_6-6a} = -\frac{6}{14}\cdot\frac{73.548}{-36.348} = 6\times0.1445 \doteq 6.S_{0.1}$$

$7. \quad (U_7-M_7)$

$$S_{0.7} = -2(U_6-U_7)\frac{0.154}{4.653} = 7\times1.1631\times\frac{0.154}{4.653}\times2 = 7\times0.0770 = 7.S_{0.1}'$$

Summary of stress So for all members.

Upper chord So.

members

Load unity on panel point

m =		1'	2'	3'	4'	5'	6'	7
1	(U_0-U_1)	-0.091	-0.182	-0.273	-0.364	-0.455	-0.546	-0.637
2		-0.214	-0.427	-0.641	-0.855	-1.069	-1.282	-1.496
3		-0.370	-0.741	-1.111	-1.481	-1.852	-2.222	-2.592
4		-0.559	-1.117	-1.676	-2.234	-2.793	-3.351	-3.910
5		-0.767	-1.535	-2.302	-3.070	-3.837	-4.604	-5.372
6		-0.978	-1.956	-2.933	-3.911	-4.889	-5.867	-6.845
7	(U_6-U_7)	-1.163	-2.326	-3.489	-4.652	-5.816	-6.979	-8.142
7'	$(U_6'-U_7)$	-1.163	-2.326	-3.489	-4.652	-5.816	-6.979	-8.142
6'		-1.304	-2.608	-3.911	-5.215	-6.519	-7.822	-6.845
5'		-1.381	-2.763	-4.144	-5.526	-6.907	-6.139	-5.372
4'		-1.396	-2.792	-4.188	-5.585	-5.027	-4.468	-3.910
3'		-1.358	-2.715	-4.073	-3.703	-3.333	-2.962	-2.592
2'		-1.282	-2.564	-2.351	-2.137	-1.923	-1.710	-1.496
1'	$(U_0'-U_1')$	-1.183	-1.092	-1.001	-0.910	-0.819	-0.728	-0.637

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

Middle chord So.

members.	Load unity on panel point.						
m=	1'	2'	3'	4'	5'	6'	7
0	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	0.094	0.187	0.281	0.374	0.468	0.561	0.655
2	0.216	0.432	0.649	0.865	1.081	1.297	1.513
3	0.371	0.742	1.113	1.484	1.855	2.226	2.597
4	0.557	1.113	1.670	2.226	2.783	3.340	3.896
5	0.764	1.527	2.291	3.054	3.818	4.581	5.345
6	0.974	1.948	2.922	3.896	4.870	5.844	6.818
6'	1.299	2.597	3.896	5.195	6494	7.792	6.818
5'	1.374	2.749	4.123	5.498	6.872	6.108	5.345
4'	1.392	2.783	4.175	5.566	5.009	4.453	3.896
3'	1.360	2.720	4.081	3.710	3.339	2.968	2.597
2'	1.297	2.594	2.378	2.162	1.946	1.730	1.513
1'	1.216	1.122	1.029	0.935	0.842	0.748	0.655
0'	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Diagonal member So.

members	Load unity on panel point						
m=	1'	2'	3'	4'	5'	6'	7
1	0.088	0.177	0.265	0.353	0.442	0.530	0.618
2	0.123	0.246	0.369	0.492	0.615	0.737	0.860
3	0.161	0.322	0.483	0.644	0.805	0.966	1.127
4	0.198	0.395	0.593	0.790	0.988	1.186	1.383
5	0.223	0.446	0.670	0.893	1.116	1.339	1.562
6	0.228	0.457	0.685	0.913	1.142	1.370	1.598
7	0.204	0.407	0.611	0.815	1.019	1.222	1.426
7'	-0.145	-0.290	-0.436	-0.581	-0.726	-0.871	1.426
6'	-0.069	-0.137	-0.206	-0.275	-0.344	1.826	1.598
5'	0.004	0.008	0.013	0.017	2.009	1.786	1.562
4'	0.065	0.130	0.195	1.976	1.778	1.581	1.383
3'	0.106	0.213	1.771	1.610	1.449	1.288	1.127
2'	0.133	1.475	1.352	1.229	1.106	0.983	0.860
1'	1.148	1.060	0.971	0.883	0.795	0.706	0.618

Vertical members So.

members	Load unity on panel point						
m=	1'	2'	3'	4'	5'	6'	7
0	-0.071	-0.143	-0.214	-0.286	-0.357	-0.428	-0.500
1	-0.077	-0.154	-0.230	-0.307	-0.384	-0.461	-0.538
2	-0.078	-0.156	-0.234	-0.312	-0.390	-0.467	-0.545
3	-0.074	-0.147	-0.221	-0.294	-0.368	-0.442	-0.515
4	-0.063	-0.125	-0.188	-0.251	-0.314	-0.376	-0.439
5	-0.043	-0.087	-0.130	-0.174	-0.217	-0.260	-0.304
6	-0.017	-0.033	-0.050	-0.066	-0.083	-0.100	-0.116
7	0.077	0.154	0.231	0.308	0.385	0.462	0.539
6'	0.145	0.289	0.434	0.578	0.723	0.867	-0.116
5'	0.122	0.244	0.365	0.487	0.609	-0.347	-0.304
4'	0.093	0.187	0.280	0.373	-0.564	-0.502	-0.439
3'	0.063	0.127	0.190	-0.736	-0.662	-0.589	-0.515
2'	0.032	0.065	-0.857	-0.779	-0.701	-0.623	-0.545
1'	0.001	-0.922	-0.845	-0.768	-0.691	-0.614	-0.538
0'	-0.928	-0.857	-0.785	-0.714	-0.643	-0.571	-0.500

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken

Summary of Stress Sa for all members.

Members	Upper chords			Middle chords			Diagonals			Verticals		
m	r_m	g_m	$S_a=-\frac{r_m}{g_m}$	S_m+r_m	h_m	$S_a=\frac{S_m+r_m}{h_m}$	V_m	f_m	$S_a=\frac{V_m}{f_m}$	W_m	i_m-ma	$S_a=-\frac{W_m}{i_m-ma}$
0 + 0′	0.000	—	—	5.000	4.129	1.211	—	—	—	$-\frac{3.176}{4.650}$	=	-0.683
1 + 1′	3.176	3.649	-0.870	7.219	3.553	2.032	16.594	19.664	0.844	36.537	61.438	-0.595
2 + 2′	5.701	3.109	-1.834	9.039	3.073	2.941	17.656	18.199	0.970	49.007	102.116	-0.480
3 + 3′	7.673	2.691	-2.851	10.492	2.686	3.906	18.384	17.397	1.057	151.735	452.014	-0.336
4 + 4′	9.155	2.379	-3.848	11.600	2.387	4.860	18.844	17.713	1.064	-24.521	-151.591	-0.162
5 + 5′	10.188	2.164	-4.708	12.382	2.175	5.693	19.218	20.448	0.940	2.407	-59.315	+0.041
6 + 6′	10.798	2.038	-5.298	12.846	2.046	6.279	19.354	29.133	0.664	9.219	-36.348	+0.254
7 + 7′	11.000	1.999	-5.503	13.000	—	—	19.417	79.327	0.245	$5.503\times\frac{0.154}{4.653}\times 2=$		+0.364

Stress on tie $S_a = -1.000$ througthout the tie.

Assumed gross sectional area and slenderness ratio l/A of truss members.

Members.	Upper chords			Middle chords			Diagonals			Verticals		
m =	l cm	A cm²gr	l/A	l cm	A cm²gr	l/A	l cm	A cm²gr	l/A	l cm	A cm²gr	l/A
0	—	—	—	563.1	330.0	1.706	—	—	—	500.0	240.0	2.083
1	515.2	240.0	2.147	529.1	〃	1.603	499.5	85.0	5.876	404.3	115.0	3.516
2	499.3	〃	2.080	505.1	240.0	2.105	489.1	〃	5.754	333.8	95.0	3.514
3	487.2	〃	2.030	488.0	〃	2.033	484.6	110.0	4.405	281.9	〃	2.967
4	478.0	〃	1.992	476.3	180.0	2.646	483.8	115.0	4.207	244.5	〃	2.574
5	471.5	300.0	1.572	469.0	〃	2.606	486.0	〃	4.226	219.4	〃	2.309
6	467.3	〃	1.558	465.4	〃	2.586	491.2	〃	4.271	204.8	〃	2.156
7	465.3	〃	1.551	—	—	—	500.3	110.0	4.548	200.0	〃	2.105

Total length of tie = 14@465.0 = 65.100
gross area of tie assumed 280.00 cm²

For tie $l/A = \frac{6510}{280.0} = 23.250$

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken

Upper chord $S_{0.1}'S_a\, l/A$ and $S_a^2\, l/A$

marks	members	$S_{0.1}'$	S_a	$S_{0.1}'S_a$	S_a^2	l/A	$S_{0.1}'S_a\, l/A$	$S_a^2\, l/A$
A	1	-0.091	-0.870	0.0792	0.7569	2.147	0.1700	1.6251
B	2	-0.214	-1.834	0.3925	3.3636	2.080	0.8164	6.9963
C	3	-0.370	-2.851	1.0549	8.1282	2.030	2.1414	16.5002
D	4	-0.559	-3.848	2.1510	14.8071	1.992	4.2848	29.4957
E	5	-0.767	-4.708	3.6110	22.1653	1.572	5.6765	34.8439
F	6	-0.978	-5.298	5.1814	28.0688	1.558	8.0726	43.7312
G	7	-1.163	-5.503	6.4000	30.2830	1.551	9.9264	46.9689
H	7′	-1.163	-5.503	6.4000	30.2830	1.551	9.9264	46.9689
I	6′	-1.304	-5.298	6.9086	28.0688	1.558	10.7636	43.7312
J	5′	-1.381	-4.708	6.5017	22.1653	1.572	10.2207	34.8439
K	4′	-1.396	-3.848	5.3718	14.8071	1.992	10.7006	29.4957
L	3′	-1.358	-2.851	3.8717	8.1282	2.030	7.8596	16.5002
M	2′	-1.282	-1.834	2.3512	3.3636	2.080	4.8905	6.9963
N.	1′	-1.183	-0.870	1.0292	0.7569	2.147	2.2097	1.6251

$\Sigma = 360.3226$

Middle chord. $S_{0.1}'S_a\, l/A$ and $S_a^2\, l/A$

A	0	0.000	1.211	0.0000	1.4665	1.706	0.0000	2.5018
B	1	0.094	2.032	0.1910	4.1290	1.603	0.3062	6.6188
C	2	0.216	2.941	0.6353	8.6495	2.105	1.3373	18.2072
D	3	0.371	3.906	1.4491	15.2568	2.033	2.9460	31.0171
E	4	0.557	4.860	2.7070	23.6196	2.646	7.1627	62.4975
F	5	0.764	5.693	4.3495	32.4102	2.606	11.3348	84.4610
G	6	0.974	6.279	6.1157	39.4258	2.586	15.8152	101.9551
H	6′	1.299	6.279	8.1564	39.4258	2.586	21.0925	101.9551
I	5′	1.374	5.693	7.8222	32.4102	2.606	20.3847	84.4610
J	4′	1.392	4.860	6.7651	23.6196	2.646	17.9005	62.4975
K	3′	1.360	3.906	5.3122	15.2568	2.033	10.7997	31.0171
L	2′	1.297	2.941	3.8145	8.6495	2.105	8.0295	18.2072
M	1′	1.216	2.032	2.4709	4.1290	1.603	3.9609	6.6188
N	0′	0.000	1.211	0.0000	1.4665	1.706	0.0000	2.5018

$\Sigma = 614.5170$

Diagonals $S_{0.1}'S_a\, l/A$ and $S_a^2\, l/A$

A	1	0.088	0.844	0.0743	0.7123	5.876	0.4366	4.1855
B	2	0.123	0.970	0.1193	0.9409	5.754	0.6865	5.4139
C	3	0.161	1.057	0.1702	1.1172	4.405	0.7497	4.9213
D	4	0.198	1.064	0.2107	1.1321	4.207	0.8864	4.7627
E	5	0.223	0.940	0.2096	0.8836	4.226	0.8858	3.7341
F	6	0.228	0.664	0.1514	0.4409	4.271	0.6466	1.8831
G	7	0.204	0.245	0.0500	0.0600	4.548	0.2274	0.2729
H	7′	-0.145	0.245	-0.0355	0.0600	4.548	-0.1615	0.2729
I	6′	-0.069	0.664	-0.0458	0.4409	4.271	-0.1956	1.8831
J	5′	0.004	0.940	0.0038	0.8836	4.226	0.0161	3.7341
K	4′	0.065	1.064	0.0692	1.1321	4.207	0.2911	4.7627
L	3′	0.106	1.057	0.1120	1.1172	4.405	0.4934	4.9213
M	2′	0.133	0.970	0.1290	0.9409	5.754	0.7423	5.4139
N	1′	1.148	0.844	0.9689	0.7123	5.876	5.6933	4.1855

$\Sigma = 50.3470$ ✓

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu Ken.

Verticals. $S_{0,1}'S_a\, l/A$ and $S_a^2\, l/A$.

marks	members	$S_{0,1}'$	S_a	$S_{0,1}'S_a$	S_a^2	l/A	$S_{0,1}'S_a\, l/A$	$S_a^2\, l/A$
A	0	−0.071	−0.683	0.0485	0.4665	2.083	0.1010	0.9717
B	1	−0.077	−0.595	0.0458	0.3540	3.516	0.1610	1.2447
C	2	−0.078	−0.480	0.0374	0.2304	3.514	0.1314	0.8096
D	3	−0.074	−0.336	0.0249	0.1129	2.967	0.0739	0.3350
E	4	−0.063	−0.162	0.0102	0.0262	2.574	0.0263	0.0674
F	5	−0.043	0.041	−0.0018	0.0017	2309	−0.0042	0.0039
G	6	−0.017	0.254	−0.0043	0.0645	2.156	−0.0093	0.1391
G′	7	0.077	0.364	0.0280	0.1325	2.105	0.0589	0.2789
H	6′	0.145	0.254	0.0368	0.0645	2.156	0.0793	0.1391
I	5′	0.122	0.041	0.0050	0.0017	2.309	0.0115	0.0039
J	4′	0.093	−0.162	−0.0151	0.0262	2.574	−0.0389	0.0674
K	3′	0.063	−0.336	−0.0212	0.1129	2.967	−0.0629	0.3350
L	2′	0.032	−0.480	−0.0154	0.2304	3.514	−0.0541	0.8096
M	1′	0.001	−0.595	−0.0006	0.3540	3.516	−0.0021	1.2447
N	0′	−0.928	−0.683	0.6338	0.4665	2.083	1.3202	0.9717
							Σ=	7.4217

Tie $S_{0,1}'S_a\, l/A$ and $S_a^2\, l/A$.

member	$S_{0,1}'$	S_a	$S_{0,1}'S_a$	S_a^2	l/A	$S_{0,1}'S_a\, l/A$	$S_a^2\, l/A$
L_0-L_0'	—	−1.000	—	1.000	23.250	—	23.2500
						Σ =	23.2500

Summary of $S_a^2\, l/A$ for the entire span.

Upper chord members	$\Sigma S_a^2\, l/A$	=	360.3226
middle chord members	〃	=	614.5170
diagonal members	〃	=	50.3470
vertical members	〃	=	7.4217
tie	〃	=	23.2500
	Summary	=	1055.8583

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Design of Kisogawa-Bashi for Gifu-Ken

Summary of $S_{0.1}' S_a \frac{l}{A}$ for the entire span.

marks	Upper chord	Middle chord	Diagonals	Verticals	Summary	Second summary	Remarks	
A	0.1700	0.0000	0.4366	0.1010	0.7076	0.7076	A	1
B	0.8164	0.3062	0.6865	0.1610	1.9701	2.6777	A ~ B	2
C	2.1414	1.3373	0.7497	0.1314	4.3598	7.0375	A ~ C	3
D	4.2848	2.9460	0.8864	0.0739	8.1911	15.2286	A ~ D	4
E	5.6765	7.1627	0.8858	0.0263	13.7513	28.9799	A ~ E	5
F	8.0726	11.3348	0.6466	− 0.0042	20.0498	49.0297	A ~ F	6
G	9.9264	15.8152	0.2274	− 0.0093	25.9597	(74.9894)		
〃	—	—	—	0.0589	0.0589	75.0483	A ~ G	7
H	9.9264	21.0925	− 0.1615	0.0793	30.9367	105.9850	A ~ H	7′
I	10.7636	20.3847	− 0.1956	0.0115	30.9642	136.9492	A ~ I	6′
J	10.2207	17.9005	0.0161	− 0.0389	28.0984	165.0476	A ~ J	5′
K	10.7006	10.7997	0.2911	− 0.0629	21.7285	186.7761	A ~ K	4′
L	7.8596	8.0295	0.4934	− 0.0541	16.3284	203.1045	A ~ L	3′
M	4.8905	3.9609	0.7423	− 0.0021	9.5916	212.6961	A ~ M	2′
N	2.2097	0.0000	5.6933	1.3202	9.2232	221.9193	A ~ N	1′

Influence surface of Horizontal Thrust

Load unity on point					$\Sigma S_0 S_a \frac{l}{A}$		$\Sigma S_a^2 \frac{l}{A}$		Thrust H
1	221.9193	+	13 × 0.0000	=	221.9193	÷	1055.8583	=	0.210 = H_1
2	2 × 203.1045	+	12 × 2.6777	=	438.3414	÷	〃	=	0.415 = H_2
3	3 × 186.7761	+	11 × 7.0375	=	637.7408	÷	〃	=	0.604 = H_3
4	4 × 165.0476	+	10 × 15.2286	=	812.4764	÷	〃	=	0.769 = H_4
5	5 × 136.9492	+	9 × 28.9799	=	945.5651	÷	〃	=	0.896 = H_5
6	6 × 105.9850	+	8 × 49.0297	=	1028.1476	÷	〃	=	0.974 = H_6
7	7 × 75.0483	+	7 × 74.9894	=	1050.2639	÷	〃	=	0.995 = H_7

Summary of H for the entire span = 8.731

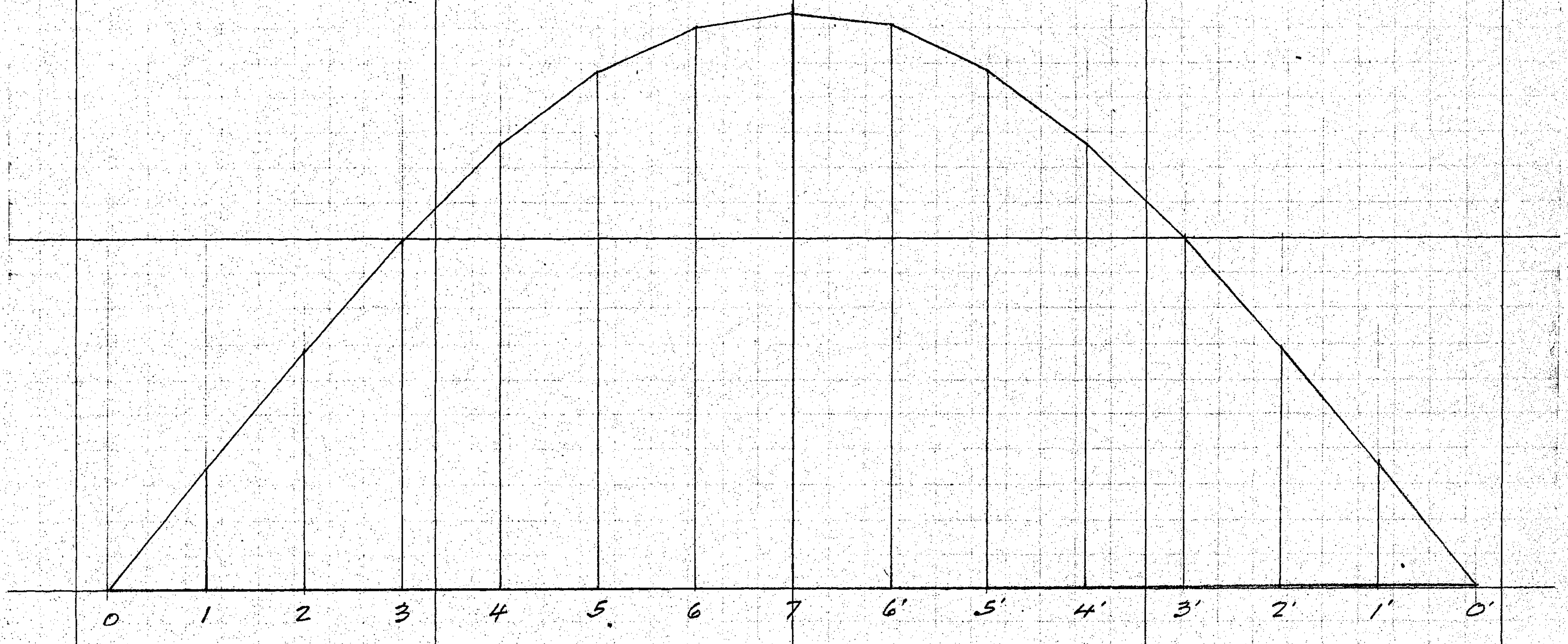

Diagram of H-surface.

Scale of thrust 1/10 meter = 1.000 kg, (Point of application of H = 13.000 × 1.00372
= 13.04833 m)

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

Influence surface of stresses for truss members.

General formula for member stress. $S = S_0 - S_a H$ for unit load.

$S_a H$ for Upper chord members.

		Load unity on panel point						
		1	2	3	4	5	6	7
members	S_a	H= 0.210	0.415	0.604	0.769	0.896	0.974	0.995
U_0-U_1	-0.870	-0.183	-0.361	-0.525	-0.669	-0.780	-0.847	-0.866
U_1-U_2	-1.834	-0.385	-0.761	-1.108	-1.410	-1.643	-1.786	-1.826
U_2-U_3	-2.851	-0.599	-1.183	-1.722	-2.192	-2.554	-2.777	-2.837
U_3-U_4	-3.848	-0.808	-1.597	-2.324	-2.958	-3.448	-3.748	-3.829
U_4-U_5	-4.708	-0.989	-1.954	-2.844	-3.620	-4.218	-4.586	-4.684
U_5-U_6	-5.298	-1.113	-2.199	-3.200	-4.073	-4.747	-5.160	-5.272
U_6-U_7	-5.503	-1.156	-2.284	-3.324	-4.231	-4.931	-5.360	-5.475

$S_a H$ for Middle chord members.

L_0-M_1	1.211	0.254	0.503	0.731	0.931	1.085	1.180	1.205
M_1-M_2	2.032	0.427	0.843	1.227	1.563	1.821	1.979	2.022
M_2-M_3	2.941	0.618	1.221	1.776	2.262	2.635	2.865	2.926
M_3-M_4	3.906	0.820	1.621	2.359	3.004	3.500	3.804	3.886
M_4-M_5	4.860	1.021	2.017	2.935	3.737	4.355	4.734	4.836
M_5-M_6	5.693	1.196	2.363	3.439	4.378	5.101	5.545	5.665
M_6-M_7	6.279	1.319	2.606	3.793	4.829	5.626	6.116	6.248

$S_a H$ for Diagonal members.

U_0-M_1	0.844	0.177	0.350	0.510	0.649	0.756	0.822	0.840
U_1-M_2	0.970	0.204	0.403	0.586	0.746	0.869	0.945	0.965
U_2-M_3	1.057	0.222	0.439	0.638	0.813	0.947	1.030	1.052
U_3-M_4	1.064	0.223	0.442	0.643	0.818	0.953	1.036	1.059
U_4-M_5	0.940	0.197	0.390	0.568	0.723	0.842	0.916	0.935
U_5-M_6	0.664	0.139	0.276	0.401	0.511	0.595	0.647	0.661
U_6-M_7	0.245	0.051	0.102	0.148	0.188	0.220	0.239	0.244

$S_a H$ for Vertical members

U_0-L_0	-0.683	-0.143	-0.283	-0.413	-0.525	-0.612	-0.665	-0.680
U_1-M_1	-0.595	-0.125	-0.247	-0.359	-0.458	-0.533	-0.580	-0.592
U_2-M_2	-0.480	-0.101	-0.199	-0.290	-0.369	-0.430	-0.468	-0.478
U_3-M_3	-0.336	-0.071	-0.139	-0.203	-0.258	-0.301	-0.327	-0.334
U_4-M_4	-0.162	-0.034	-0.067	-0.098	-0.125	-0.145	-0.158	-0.161
U_5-M_5	0.041	0.009	0.017	0.025	0.032	0.037	0.040	0.041
U_6-M_6	0.254	0.053	0.105	0.153	0.195	0.228	0.247	0.253
U_7-M_7	0.364	0.076	0.151	0.220	0.280	0.326	0.355	0.362

$S_a H$ for Tie

L_0-L_0'	-1.000	-0.210	-0.415	-0.604	-0.769	-0.896	-0.974	-0.995

✓

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CONSULTING ENGINEER
~~HIII~~ SHOWA BLDG, TOKYO

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken

Influence surface of Top chord members. Stress $S = S_0 - S_aH$ for unit load.

Load on	U_0-U_1			U_1-U_2			U_2-U_3		
	S_0	S_aH	S	S_0	S_aH	S	S_0	S_aH	S
1′	−0.091	−0.183	0.092	−0.214	−0.385	0.171	−0.370	−0.599	0.229
2′	−0.182	−0.361	0.179	−0.427	−0.761	0.334	−0.741	−1.183	0.442
3′	−0.273	−0.525	0.252	−0.641	−1.108	0.467	−1.111	−1.722	0.611
4′	−0.364	−0.669	0.305	−0.855	−1.410	0.555	−1.481	−2.192	0.711
5′	−0.455	−0.780	0.325	−1.069	−1.643	0.574	−1.852	−2.554	0.702
6′	−0.546	−0.847	0.301	−1.282	−1.786	0.504	−2.222	−2.777	0.555
7	−0.637	−0.866	0.229	−1.496	−1.825	0.329	−2.592	−2.837	0.245
6	−0.728	−0.847	0.119	−1.710	−1.786	0.076	−2.962	−2.777	−0.185
5	−0.819	−0.780	−0.039	−1.923	−1.643	−0.280	−3.333	−2.554	−0.779
4	−0.910	−0.669	−0.241	−2.137	−1.410	−0.727	−3.703	−2.192	−1.511
3	−1.001	−0.525	−0.476	−2.351	−1.108	−1.243	−4.073	−1.722	−2.351
2	−1.092	−0.361	−0.731	−2.564	−0.761	−1.803	−2.715	−1.183	−1.532
1	−1.183	−0.183	−1.000	−1.282	−0.385	−0.897	−1.358	−0.599	−0.759
	Σ Plus stresses		1.802			3.010			3.495
	Σ minus 〃		−2.487			−4.950			−7.117
	Summary		−0.685			−1.940			−3.622

Load on	U_3-U_4			U_4-U_5			U_5-U_6		
	S_0	S_aH	S	S_0	S_aH	S	S_0	S_aH	S
1′	−0.559	−0.808	0.249	−0.767	−0.989	0.222	−0.978	−1.113	0.135
2′	−1.117	−1.597	0.480	−1.535	−1.954	0.419	−1.956	−2.199	0.243
3′	−1.676	−2.324	0.648	−2.302	−2.844	0.542	−2.933	−3.200	0.267
4′	−2.234	−2.958	0.724	−3.070	−3.620	0.550	−3.911	−4.073	0.162
5′	−2.793	−3.448	0.655	−3.837	−4.218	0.381	−4.889	−4.747	−0.142
6′	−3.351	−3.748	0.397	−4.604	−4.586	−0.018	−5.867	−5.160	−0.707
7	−3.910	−3.829	−0.081	−5.372	−4.684	−0.688	−6.845	−5.272	−1.573
6	−4.468	−3.748	−0.720	−6.139	−4.586	−1.553	−7.822	−5.160	−2.662
5	−5.027	−3.448	−1.579	−6.907	−4.218	−2.689	−6.519	−4.747	−1.772
4	−5.585	−2.958	−2.627	−5.526	−3.620	−1.906	−5.215	−4.073	−1.142
3	−4.188	−2.324	−1.864	−4.144	−2.844	−1.300	−3.911	−3.200	−0.711
2	−2.792	−1.597	−1.195	−2.763	−1.954	−0.809	−2.608	−2.199	−0.409
1	−1.396	−0.808	−0.588	−1.381	−0.989	−0.392	−1.304	−1.113	−0.191
	Σ Plus stresses		3.153			2.114			0.807
	Σ minus stresses		−8.654			−9.355			−9.309
	Summary		−5.501			−7.241			−8.502

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken

Load on	U_6-U_7		
	S_o	S_aH	S
1'	- 1.163	- 1.156	- 0.007
2'	- 2.326	- 2.284	- 0.042
3'	- 3.489	- 3.324	- 0.165
4'	- 4.652	- 4.231	- 0.421
5'	- 5.816	- 4.931	- 0.885
6'	- 6.979	- 5.360	- 1.619
7	- 8.142	- 5.475	- 2.667
6	- 6.979	- 5.360	- 1.619
5	- 5.816	- 4.931	- 0.885
4	- 4.652	- 4.231	- 0.421
3	- 3.489	- 3.324	- 0.165
2	- 2.326	- 2.284	- 0.042
1	- 1.163	- 1.156	- 0.007
	Σ Plus stresses		0.000
	Σ Minus "		- 8.945
	Summary		- 8.945

Influence surface of middle chord members.

Load on	L_0-M_1			M_1-M_2			M_2-M_3		
	S_o	S_aH	S	S_o	S_aH	S	S_o	S_aH	S
1'	0.000	0.254	- 0.254	0.094	0.427	- 0.333	0.216	0.618	- 0.402
2'	0.000	0.503	- 0.503	0.187	0.843	- 0.656	0.432	1.221	- 0.789
3'	0.000	0.731	- 0.731	0.281	1.227	- 0.946	0.649	1.776	- 1.127
4'	0.000	0.931	- 0.931	0.374	1.563	- 1.189	0.865	2.262	- 1.397
5'	0.000	1.085	- 1.085	0.468	1.821	- 1.353	1.081	2.635	- 1.554
6'	0.000	1.180	- 1.180	0.561	1.979	- 1.418	1.297	2.865	- 1.568
7	0.000	1.205	- 1.205	0.655	2.022	- 1.367	1.513	2.926	- 1.413
6	0.000	1.180	- 1.180	0.748	1.979	- 1.231	1.730	2.865	- 1.135
5	0.000	1.085	- 1.085	0.842	1.821	- 0.979	1.946	2.635	- 0.689
4	0.000	0.931	- 0.931	0.935	1.563	- 0.628	2.162	2.262	- 0.100
3	0.000	0.731	- 0.731	1.029	1.227	- 0.198	2.378	1.776	0.602
2	0.000	0.503	- 0.503	1.122	0.843	0.279	2.594	1.221	1.373
1	0.000	0.254	- 0.254	1.216	0.427	0.789	1.297	0.618	0.679
	Σ Plus stresses		0.000			1.068			2.654
	Σ minus "		- 10.573			- 10.298			- 10.174
	Summary		- 10.573			- 9.230			- 7.520

Design of Kisogawa-Bashi for Gifu-Ken.

	M_3-M_4			M_4-M_5			M_5-M_6		
Load on	S_0	S_{aH}	S	S_0	S_{aH}	S	S_0	S_{aH}	S
1'	0.371	0.820	-0.449	0.557	1.021	-0.464	0.764	1.196	-0.432
2'	0.742	1.621	-0.879	1.113	2.017	-0.904	1.527	2.363	-0.836
3'	1.113	2.359	-1.246	1.670	2.935	-1.265	2.291	3.439	-1.148
4'	1.484	3.004	-1.520	2.226	3.737	-1.511	3.054	4.378	-1.324
5'	1.855	3.500	-1.645	2.783	4.355	-1.572	3.818	5.101	-1.283
6'	2.226	3.804	-1.578	3.340	4.734	-1.394	4.581	5.545	-0.964
7	2.597	3.886	-1.289	3.896	4.836	-0.940	5.345	5.665	-0.320
6	2.968	3.804	-0.836	4.453	4.734	-0.281	6.108	5.545	0.563
5	3.339	3.500	-0.161	5.009	4.355	0.654	6.872	5.101	1.771
4	3.710	3.004	0.706	5.566	3.737	1.829	5.498	4.378	1.120
3	4.081	2.359	1.722	4.175	2.935	1.240	4.123	3.439	0.684
2	2.720	1.621	1.099	2.783	2.017	0.766	2.749	2.363	0.386
1	1.360	0.820	0.540	1.392	1.021	0.371	1.374	1.196	0.178
	Σ plus stresses		4.067			4.860			4.702
	Σ minus "		-9.603			-8.331			-6.307
	Summary		-5.536			-3.471			-1.605

	M_6-M_7		
Load on	S_0	S_{aH}	S
1'	0.974	1.319	-0.345
2'	1.948	2.606	-0.658
3'	2.922	3.793	-0.871
4'	3.896	4.829	-0.933
5'	4.870	5.626	-0.756
6'	5.844	6.116	-0.272
7	6.818	6.248	0.570
6	7.792	6.116	1.676
5	6.494	5.626	0.868
4	5.195	4.829	0.366
3	3.896	3.793	0.103
2	2.597	2.606	-0.009
1	1.299	1.319	-0.020
	Σ Plus stresses		3.583
	Σ minus "		-3.864
	Summary		-0.281

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

Influence surface of Diagonal members.

		U_0-M_1			U_1-M_2			U_2-M_3	
Load on	S_o	S_aH	S	S_o	S_aH	S	S_o	S_aH	S
1′	0.088	0.177	−0.089	0.123	0.204	−0.081	0.161	0.222	−0.061
2′	0.177	0.350	−0.173	0.246	0.403	−0.157	0.322	0.439	−0.117
3′	0.265	0.510	−0.245	0.369	0.586	−0.217	0.483	0.638	−0.155
4′	0.353	0.649	−0.296	0.492	0.746	−0.254	0.644	0.813	−0.169
5′	0.442	0.756	−0.314	0.615	0.869	−0.254	0.805	0.947	−0.142
6′	0.530	0.822	−0.292	0.737	0.945	−0.208	0.966	1.030	−0.064
7	0.618	0.840	−0.222	0.860	0.965	−0.105	1.127	1.052	0.075
6	0.706	0.822	−0.116	0.983	0.945	0.038	1.288	1.030	0.258
5	0.795	0.756	0.039	1.106	0.869	0.237	1.449	0.947	0.502
4	0.883	0.649	0.234	1.229	0.746	0.483	1.610	0.813	0.797
3	0.971	0.510	0.461	1.352	0.586	0.766	1.771	0.638	1.133
2	1.060	0.350	0.710	1.475	0.403	1.072	0.213	0.439	−0.226
1	1.148	0.177	0.971	0.133	0.204	−0.071	0.106	0.222	−0.116
	Σ plus stresses		2.415			2.596			2.765
	Σ minus "		−1.747			−1.347			−1.050
	Summary		0.668			1.249			1.715

		U_3-M_4			U_4-M_5			U_5-M_6	
Load on	S_o	S_aH	S	S_o	S_aH	S	S_o	S_aH	S
1′	0.198	0.223	−0.025	0.223	0.197	0.026	0.228	0.139	0.089
2′	0.395	0.442	−0.047	0.446	0.390	0.056	0.457	0.276	0.181
3′	0.593	0.643	−0.050	0.670	0.568	0.102	0.685	0.401	0.284
4′	0.790	0.818	−0.028	0.893	0.723	0.170	0.913	0.511	0.402
5′	0.988	0.953	0.035	1.116	0.842	0.274	1.142	0.595	0.547
6′	1.186	1.036	0.150	1.339	0.916	0.423	1.370	0.647	0.723
7	1.383	1.059	0.324	1.562	0.935	0.627	1.598	0.661	0.937
6	1.581	1.036	0.545	1.786	0.916	0.870	1.826	0.647	1.179
5	1.778	0.953	0.825	2.009	0.842	1.167	−0.344	0.595	−0.939
4	1.976	0.818	1.158	0.017	0.723	−0.706	−0.275	0.511	−0.786
3	0.195	0.643	−0.448	0.013	0.668	−0.555	−0.206	0.401	−0.607
2	0.130	0.442	−0.312	0.008	0.390	−0.382	−0.137	0.276	−0.413
1	0.065	0.223	−0.158	0.004	0.197	−0.193	−0.069	0.139	−0.208
	Σ Plus stresses		3.037			3.715			4.342
	Σ Minus "		−1.068			−1.836			−2.953
	Summary		1.969			1.879			1.389

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CALCULATIONS FOR

Design of Kisogawa Bashi for Gifu Ken.

		U_6-M_7	
Load on	S_o	S_{aH}	S
1′	0.204	0.051	0.153
2′	0.407	0.102	0.305
3′	0.611	0.148	0.463
4′	0.815	0.188	0.627
5′	1.019	0.220	0.799
6′	1.222	0.239	0.983
7.	1.426	0.244	1.182
6	−0.871	0.239	−1.110
5	−0.726	0.220	−0.946
4	−0.581	0.188	−0.769
3	−0.436	0.148	−0.584
2	−0.290	0.102	−0.392
1	−0.145	0.051	−0.196
	Σ plus stresses		4.512
	Σ Minus 〃		−3.997
	Summary		0.515

Influence surface of Vertical members.

		U_0-L_0			U_1-M_1			U_2-M_2	
Load on	S_o	S_{aH}	S	S_o	S_{aH}	S	S_o	S_{aH}	S
1′	−0.071	−0.143	0.072	−0.077	−0.125	0.048	−0.078	−0.101	0.023
2′	−0.143	−0.283	0.140	−0.154	−0.247	0.093	−0.156	−0.199	0.043
3′	−0.214	−0.413	0.199	−0.230	−0.359	0.129	−0.234	−0.290	0.056
4′	−0.286	−0.525	0.239	−0.307	−0.458	0.151	−0.312	−0.369	0.057
5′	−0.357	−0.612	0.255	−0.384	−0.533	0.149	−0.390	−0.430	0.040
6′	−0.428	−0.665	0.237	−0.461	−0.580	0.119	−0.467	−0.468	0.001
7	−0.500	−0.680	0.180	−0.538	−0.592	0.054	−0.545	−0.478	−0.067
6	−0.571	−0.665	0.094	−0.614	−0.580	−0.034	−0.623	−0.468	−0.155
5	−0.643	−0.612	−0.031	−0.691	−0.533	−0.158	−0.701	−0.430	−0.271
4	−0.714	−0.525	−0.189	−0.768	−0.458	−0.310	−0.779	−0.369	−0.410
3	−0.785	−0.413	−0.372	−0.845	−0.359	−0.486	−0.857	−0.290	−0.567
2	−0.857	−0.283	−0.574	−0.922	−0.247	−0.675	0.065	−0.199	0.264
1	−0.928	−0.143	−0.785	0.001	−0.125	0.126	0.032	−0.101	0.133
	Σ Plus stresses		1.416			0.869			0.617
	Σ minus 〃		−1.951			−1.663			−1.470
	Summary		−0.535			−0.794			−0.853

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

	U_3-M_3			U_4-M_4			U_5-M_5		
Load on	S_o	S_aH	S	S_o	S_aH	S	S_o	S_aH	S
1'	-0.074	-0.071	-0.003	-0.063	-0.034	-0.029	-0.043	0.009	-0.052
2'	-0.147	-0.139	-0.008	-0.125	-0.067	-0.058	-0.087	0.017	-0.104
3'	-0.221	-0.203	-0.018	-0.188	-0.098	-0.090	-0.130	0.025	-0.155
4'	-0.294	-0.258	-0.036	-0.251	-0.125	-0.126	-0.174	0.032	-0.206
5'	-0.368	-0.301	-0.067	-0.314	-0.145	-0.169	-0.217	0.037	-0.254
6'	-0.442	-0.327	-0.115	-0.376	-0.158	-0.218	-0.260	0.040	-0.300
7	-0.515	-0.334	-0.181	-0.439	-0.161	-0.278	-0.304	0.041	-0.345
6	-0.589	-0.327	-0.262	-0.502	-0.158	-0.344	-0.347	0.040	-0.387
5	-0.662	-0.301	-0.361	-0.564	-0.145	-0.419	0.609	0.037	0.572
4	-0.736	-0.258	-0.478	0.373	-0.125	0.498	0.487	0.032	0.455
3	0.190	-0.203	0.393	0.280	-0.098	0.378	0.365	0.025	0.340
2	0.127	-0.139	0.266	0.187	-0.067	0.254	0.244	0.017	0.227
1	0.063	-0.071	0.134	0.093	-0.034	0.127	0.122	0.009	0.113
	Σ Plus stresses		0.793			1.257			1.707
	Σ minus 〃		-1.529			-1.731			-1.803
	Summary		-0.736			-0.474			-0.096

	U_6-M_6			U_7-M_7			Influence surface of Tie L_o-L_o'		
Load on	S_o	S_aH	S	S_o	S_aH	S	S_o	S_aH	S
1'	-0.017	0.053	-0.070	0.077	0.076	0.001	0.000	-0.210	0.210
2'	-0.033	0.105	-0.138	0.154	0.151	0.003	〃	-0.415	0.415
3'	-0.050	0.153	-0.203	0.231	0.220	0.011	〃	-0.604	0.604
4'	-0.066	0.195	-0.261	0.308	0.280	0.028	〃	-0.769	0.769
5'	-0.083	0.228	-0.311	0.385	0.326	0.059	〃	-0.896	0.896
6'	-0.100	0.247	-0.347	0.462	0.355	0.107	〃	-0.974	0.974
7	-0.116	0.253	-0.369	0.539	0.362	0.177	〃	-0.995	0.995
6	0.867	0.247	0.620	0.462	0.355	0.107	〃	-0.974	0.974
5	0.723	0.228	0.495	0.385	0.326	0.059	〃	-0.896	0.896
4	0.578	0.195	0.383	0.308	0.280	0.028	〃	-0.769	0.769
3	0.434	0.153	0.281	0.231	0.220	0.011	〃	-0.604	0.604
2	0.289	0.105	0.184	0.154	0.151	0.003	〃	-0.415	0.415
1	0.145	0.053	0.092	0.077	0.076	0.001	〃	-0.210	0.210
	Σ plus stresses		2.055			0.595			8.731
	Σ minus 〃		-1.699			-0.000			-0.000
	Summary		0.356			0.595			8.731

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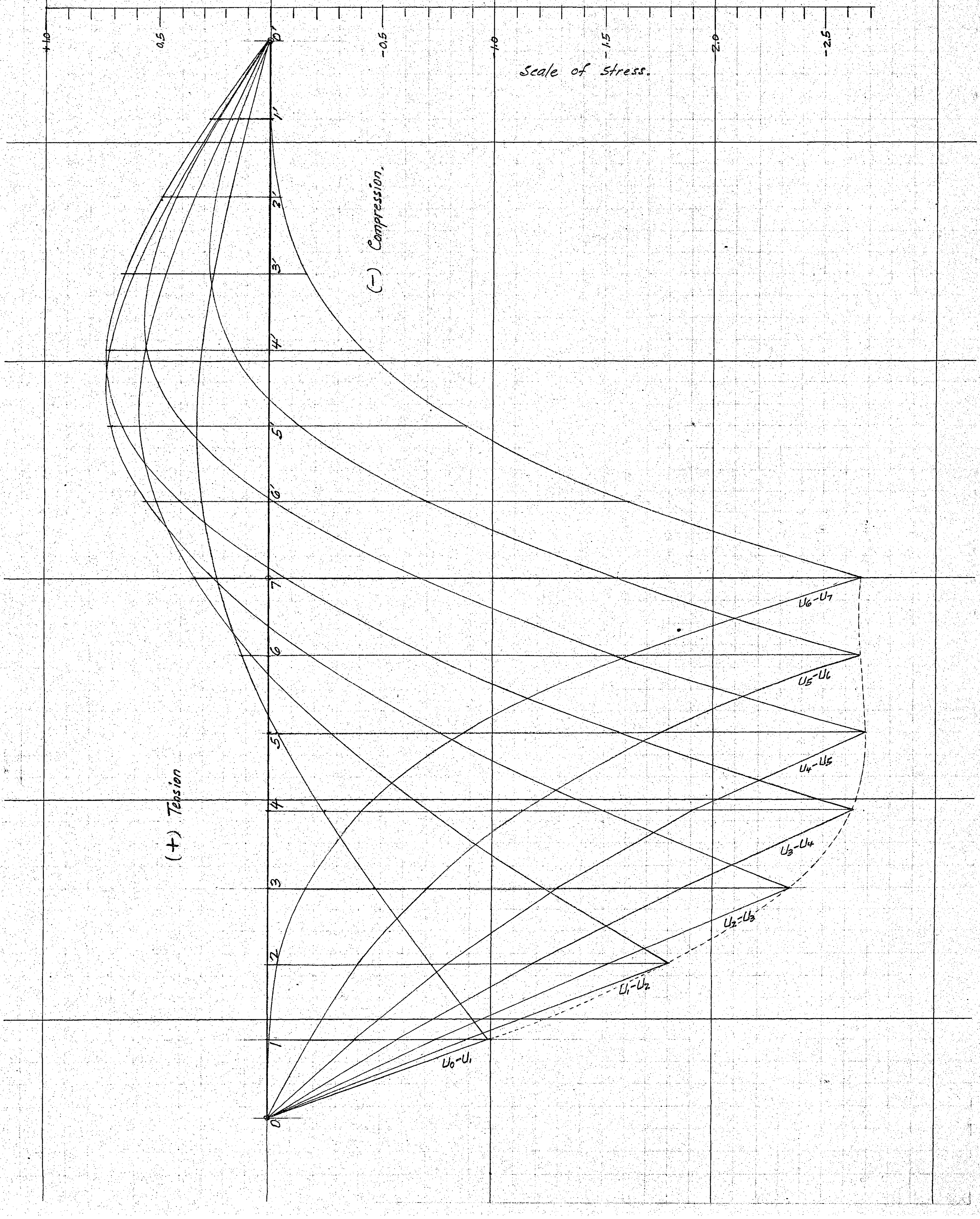
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

Diagrams of Influence surfaces.

Influence surface of Top chords.



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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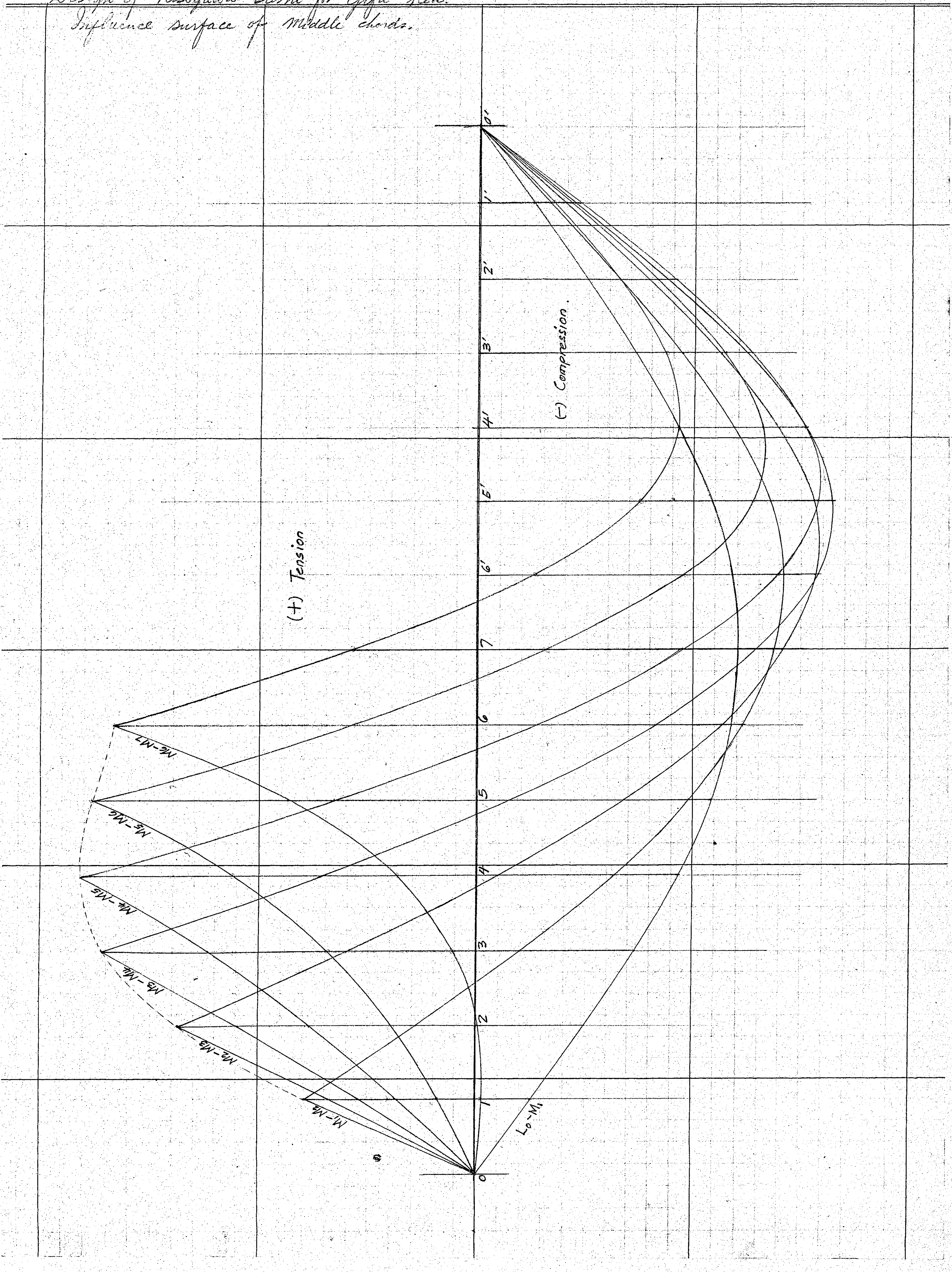
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-ken.
Influence surface of middle chords.



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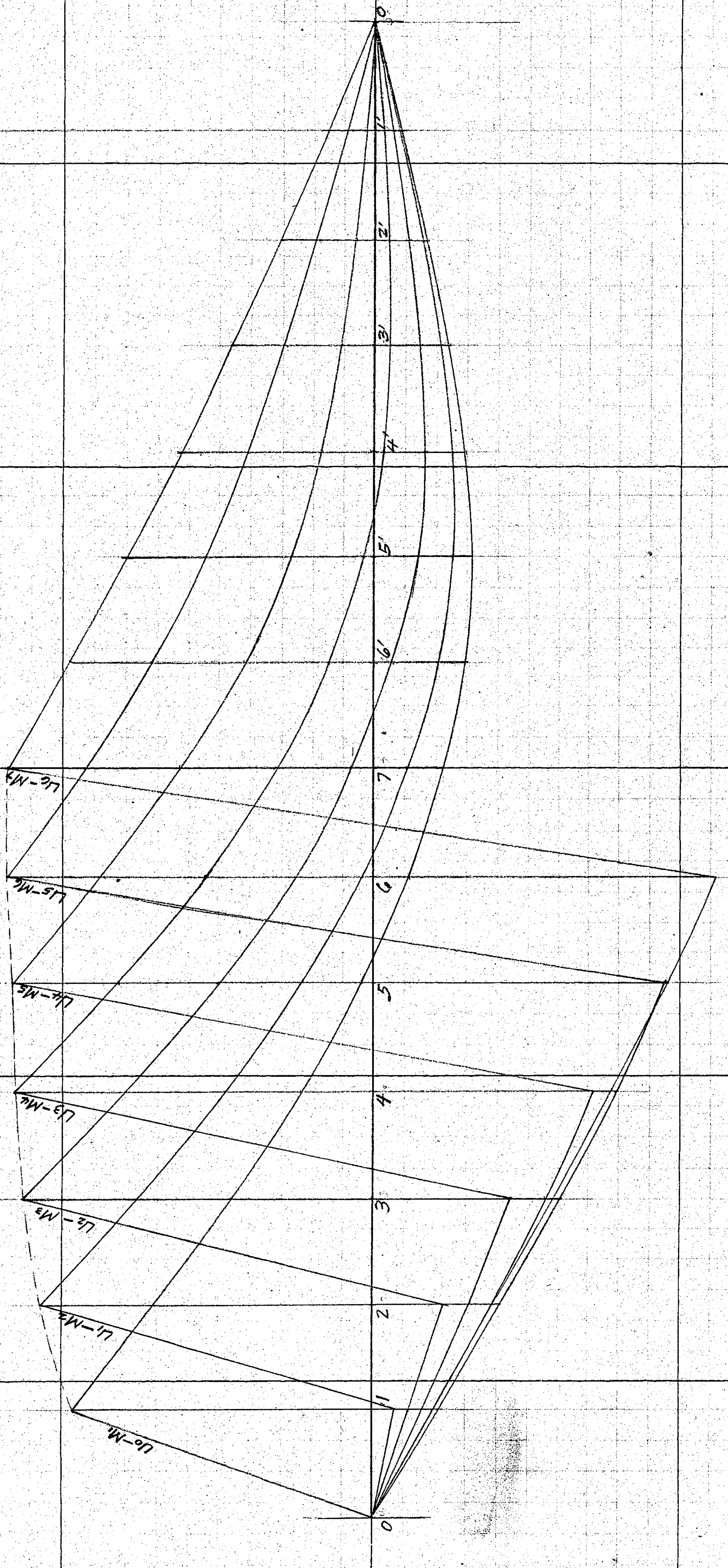
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-ken.

Influence surface of Diagonal members.



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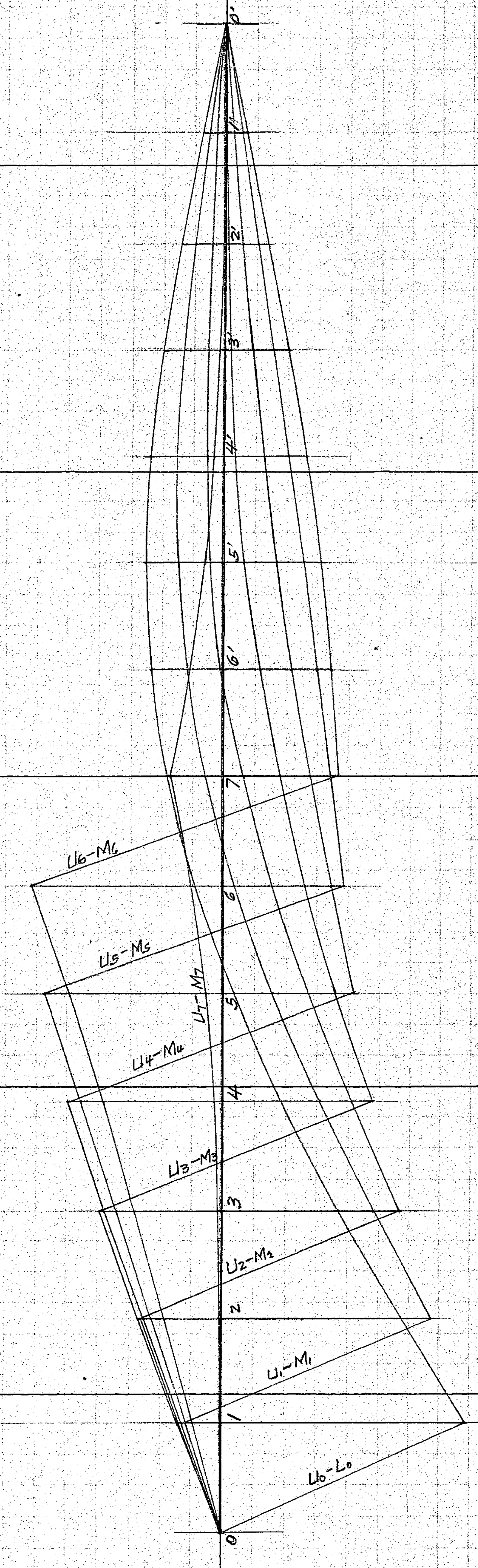
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken
Influence surface of Vertical members.



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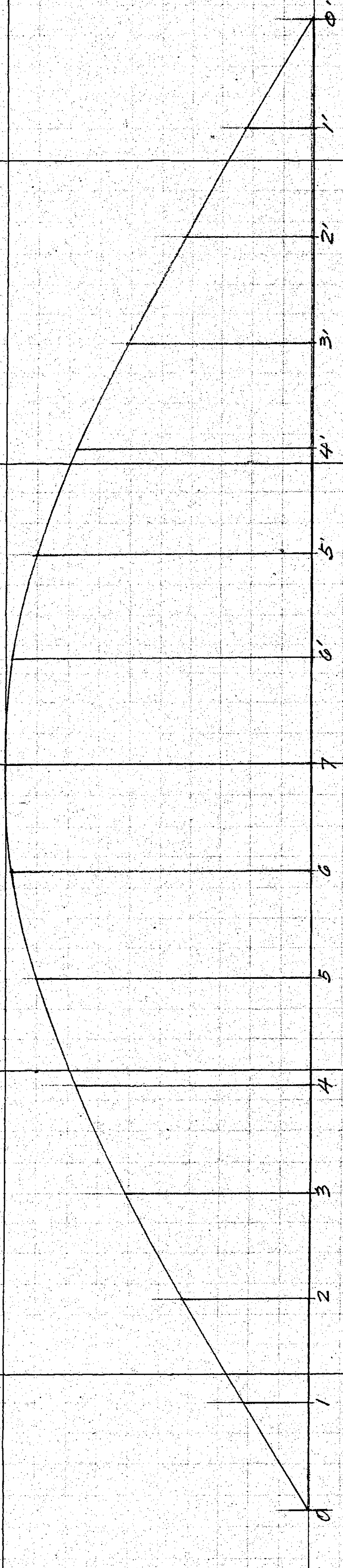
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

Influence line for stresses of Tie



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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

Design of Main truss. Span length 14 panels @ 4.65 = 65.100 meters

Dead panel load:

Floor and pavement	9.00ᵐ	@	510 kg	=	4590
Copings	2	@	255	=	510
handrails assumed	2	@	120	=	240
Stringers	6	@	62	=	372
floor beams	2240	÷	4.65	=	482
miscellaneous fittings say				=	26
					6220

Lower lateral bracings			120
upper " "	5760 ÷ 65.1	=	89
Sway, portal bracings + struts	19800 ÷ 65.1	=	304
main trusses assumed	172000 ÷ 65.1	=	2643
electric wiring cables and other fittings say			44
			3200

Total Dead Load of bridge per lin. meter of span = 9420 kg

Dead panel load on one truss.

$$\frac{9420}{2} \times 4.65 = 21,900 \text{ kg}$$

Live panel load:

Uniform live load on roadway $w = \frac{100,000}{170+65.1} = 425$ kg per sq. meter.

motor truck rear wheel concentration = 3,000

impact coefficient $= \frac{20}{60+65.1} = 16.0\%$ 480

3480 kg for one wheel.

front wheel with impact say 1/3 × 3480 = 1,160 " "

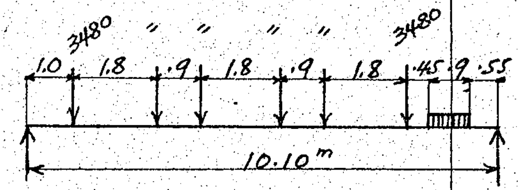
Full uniform load on truss = 4.50 × 425 = 1,910 kg per lin. meter

Uniform load on side of trucks

0.90 × 425 = 383

$383 \times \frac{1.00}{10.10}$ = 38 " " = U_1

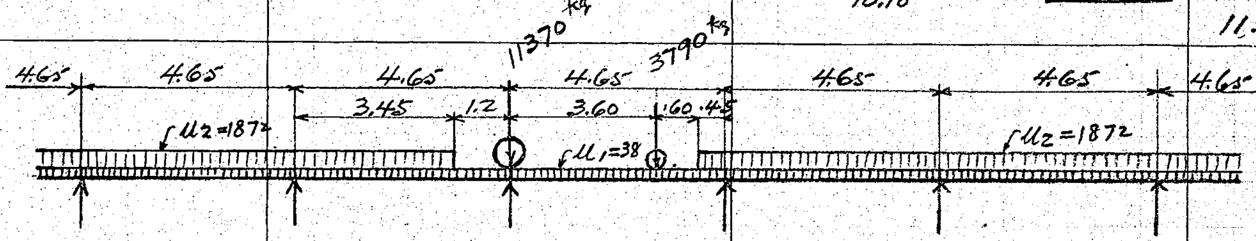
Difference = 1872 " " = U_2



Rear wheel 3 @ 3480 = 10440

$20880 \times \frac{0.45}{10.10}$ = 930

11,370 kg



front wheels say 1/3 × 11370 = 3790 kg

Rear wheels			11,370			
front "			855	2935		
Unif. load U2		4055 -----	2,400			
" "			40 -----	800		
" "	8700 ---	4350		4350 ----	8700 ----	8700
" U1	177	177	177	177	177	177
	8,877	8,582 (−295)	14,842 (+5,965)	8,262 (−615)	8,877	8,877

For the sake of convenience of stress calculations, let us assume the panel loads as follows.

Panel loads 8880 kg throughout and extra single concentration of 5800 kg at panel pt. giving the maximum stress for the member in question.

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-ken.

Dead and Live Load stresses of Truss members.

Members	Summary of Influence ordinates			Maximum	D.L. Stresses	L.L.	Stresses	Total L.L.	Maximum
	(+) Ordinates	(−) Ordinates	Summary	Ordinates	21900^{kg}	8880^{kg}	5800^{kg}	Stresses	Stresses.
Upper Chord members.									
U_0-U_1	1.802	−2.487	−0.685	−1.000	−15.000	−22,080	−5800	−27,880	−42,880
U_1-U_2	3,010	−4.950	−1.940	−1.803	−42,450	−43,950	−10450	−54400	−96,850
U_2-U_3	3.495	−7.117	−3.622	−2.351	−79,300	−63200	−13640	−76,840	−156,140
U_3-U_4	3,153	−8.654	−5.501	−2.627	−120,500	−76,800	−15,230	−92,030	−212,530
U_4-U_5	2,114	−9.355	−7.241	−2.689	−158,600	−83,100	−15,600	−98700	−257,300
U_5-U_6	0.807	−9.309	−8.502	−2.662	−186,200	−82,700	−15,450	−98,150	−284,350
U_6-U_7	0.000	−8.945	−8.945	−2.667	−195,800	−79,400	−15,470	−94,870	−290,670
Middle chord members.									
L_0-M_1	0.000	−10.573	−10.573	−1.205	−231,500	−93800	−7,000	−100,800	−332,300
M_1-M_2	1.068	−10.298	−9.230	−1.418	−202,000	−91,400	−8220	−99,620	−301,620
M_2-M_3	2.654	−10.174	−7.520	−1.568	−164,700	−90,300	−9100	−99,400	−264,100
M_3-M_4	4.067	−9.603	−5.536	−1.645	−121,200	−85,300	−9,540	−94,840	−216,040
M_4-M_5	4.680	−8.331	−3.471	−1.572 1.829	−76,000 $-\frac{2}{3}\times76{,}000$	−74,000 41,540	−9,120 10,600	−83,120 52,140	−159,120 1,470
M_5-M_6	4.702	−6.307	−1.605	−1.324 1.771	−35,150 $-\frac{2}{3}\times35{,}150$	−56,000 41,750	−7,680 10,270	−63,680 52,020	−98,830 28,590
M_6-M_7	3.583	−3.864	−0.281	−0.933 1.676	−6,150 $-\frac{2}{3}\times6{,}150$	−34,300 31,800	−5410 9720	−39,710 41,520	−45,860 37,420
Diagonal members.									
U_0-M_1	2.415	−1.747	0.668	0.971 −0.314	14,630 $\frac{2}{3}\times14{,}630$	21,450 −15,500	5630 −1820	27,080 −17,320	41,710 −7,560
U_1-M_2	2.596	−1.347	1.249	1.072	27,350	23,050	6220	29,270	56,620
U_2-M_3	2.765	−1.050	1.715	1.133	37,540	24,550	6,580	31,130	68,670
U_3-M_4	3.037	−1.068	1.969	1.158	43,100	26,970	6720	33,690	76,790
U_4-M_5	3.715	−1.836	1.879	1.167	41,100	33,000	6770	39,770	80,870
U_5-M_6	4.342	−2.953	1.389	1.179 −0.939	30,400 $\frac{2}{3}\times30{,}400$	38,550 −26,220	6840 −5450	45,390 −31,670	75,790 −11,420
U_6-M_7	4.512	−3.997	0.515	1.182 −1.110	11,280 $\frac{2}{3}\times11{,}280$	40,100 −35,500	6850 −6440	46,950 −41,940	58,230 −34,410

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

members	Summary of Influence Ord's. (+) Ordinates	(−) Ordinates	Summary	Maximum Ordinates.	D.L. Stresses 21,900 kg	L.L. Stresses 8880 kg	, 5800	Total L.L. Stresses	Maximum Stresses.
Vertical members:									
$U_0 - L_0$	1.416	−1.951	−0.535	−0.785 0.255	−11,720 −⅔×11720	−17,330 12,570	−4,560 1,480	−21,890 14,050	−33,610 6,240
$U_1 - M_1$	0.869	−1.663	−0.794	−0.675	−17,400	−14,770	−3,920	−18,690	−36,090
$U_2 - M_2$	0.617	−1.470	−0.853	−0.567	−18,680	−13,050	−3,290	−16,340	−35,020
$U_3 - M_3$	0.793	−1.529	−0.736	−0.478	−16,100	−13,570	−2,770	−16,340	−32,440
$U_4 - M_4$	1.257	−1.731	−0.474	−0.419 0.498	−10,380 −⅔×10380	−15,370 11,160	−2,430 2,890	−17,800 14,050	−28,180 7,130
$U_5 - M_5$	1.707	−1.803	−0.096	0.572 −0.387	−⅔×2100 −2100	15,150 −16,020	3,320 −2,250	18,470 −18,270	17,070 −20,370
$U_6 - M_6$	2.055	−1.699	0.356	0.620 −0.369	7800 ⅔×7,800	18,250 −15,080	3,600 −2,140	21,850 −17,220	29,650 −12,020
$U_7 - M_7$	0.595	−0.000	0.595	0.177	13,030	5,280	1,030	6,310	19,340
Ties									
$L_0 - L_0'$	8.731	−0,000	8.731	0.995	191,200	77,500	5,770	83,270	274,470

max. stress in hangers.
Dead Load ——— 21,900 kg

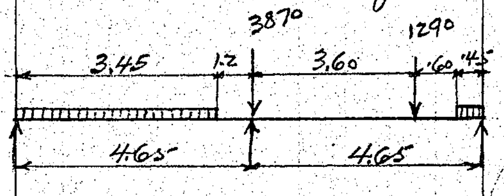
Live Load.

motor truck rear wheel concentration = 3000

impact coef. $= \frac{20}{60+9.0} = 29.0\%$ = 870

3870 kg

front wheel with impact say 3870÷3 = 1290



max. concentration on floor beam.

Front wheels $1290 \times \frac{1.05}{4.65} =$ 290

Rear 〃 3870

4160 kg

Uniform load on floor beam (front + rear of trucks)

$\frac{500 \times 0.45^2}{2 \times 4.65} =$ 11

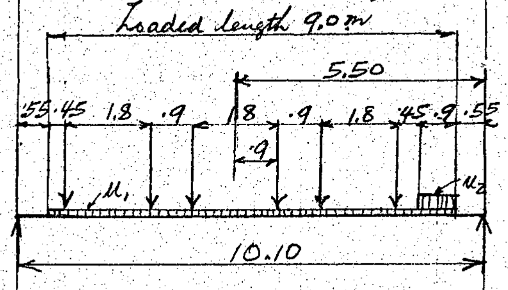
$\frac{500 \times 3.45^2}{2 \times 4.60} =$ 640

651 kg per lin meter of floor beam. u_1

〃 sides of trucks

$500 \times 4.65 =$ 2325 〃 〃

1674 u_2



max. stress in hanger.

Concentrations $6 @ 4160 \times \frac{5.50}{10.10} =$ 13600

Unif. load u_1 $4.5^m @ 651$ = 2930

〃 u_2 $1674 \times 0.9 \times \frac{1.0}{10.1}$ = 150

Total L.L. stress = 16,680 kg

D.L. stress = 21,900

Summary = 38,580 kg.

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

Max. load on shoe.

Dead Load

7 @ 21,900 = 153,300 kg

Live Load

7 @ 8,880 = 62160
5800
67,960
221,260 kg.

weight of shoe and exp. jt. say 1140
222,400 kg on one shoe.

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CALCULATIONS FOR

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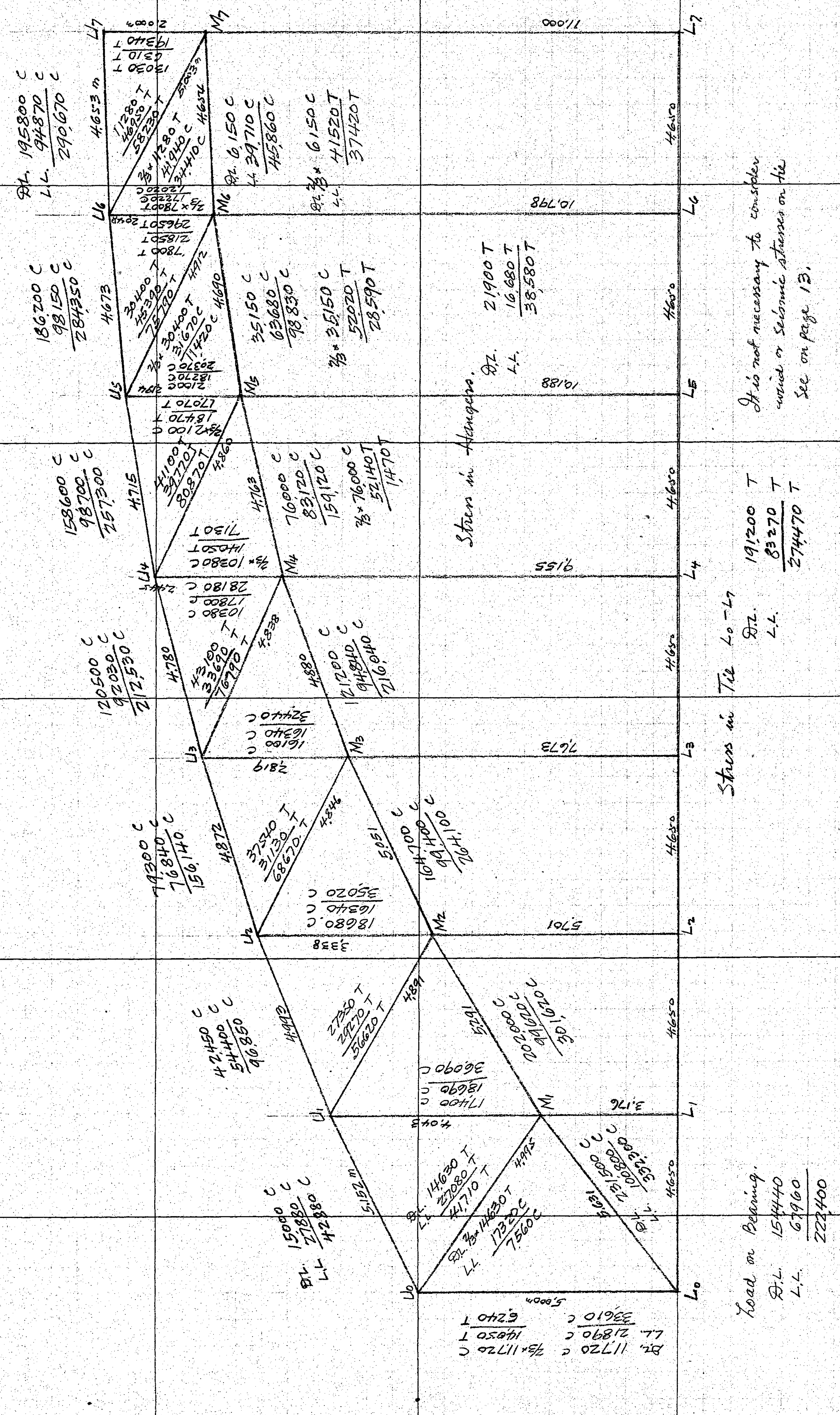
Design of Kisogawa-Bashi for Gifu-Ken

Stress diagram.



Stress in Hangers.
D.L. 21900 T
L.L. 16680 T
38580 T

Stress in Tie L0-L7
D.L. 191200 T
L.L. 83270 T
274470 T

It is not necessary to consider wind or seismic stresses on tie.
See on page 13.

Load on Bearing
D.L. 154440
L.L. 67960
222400

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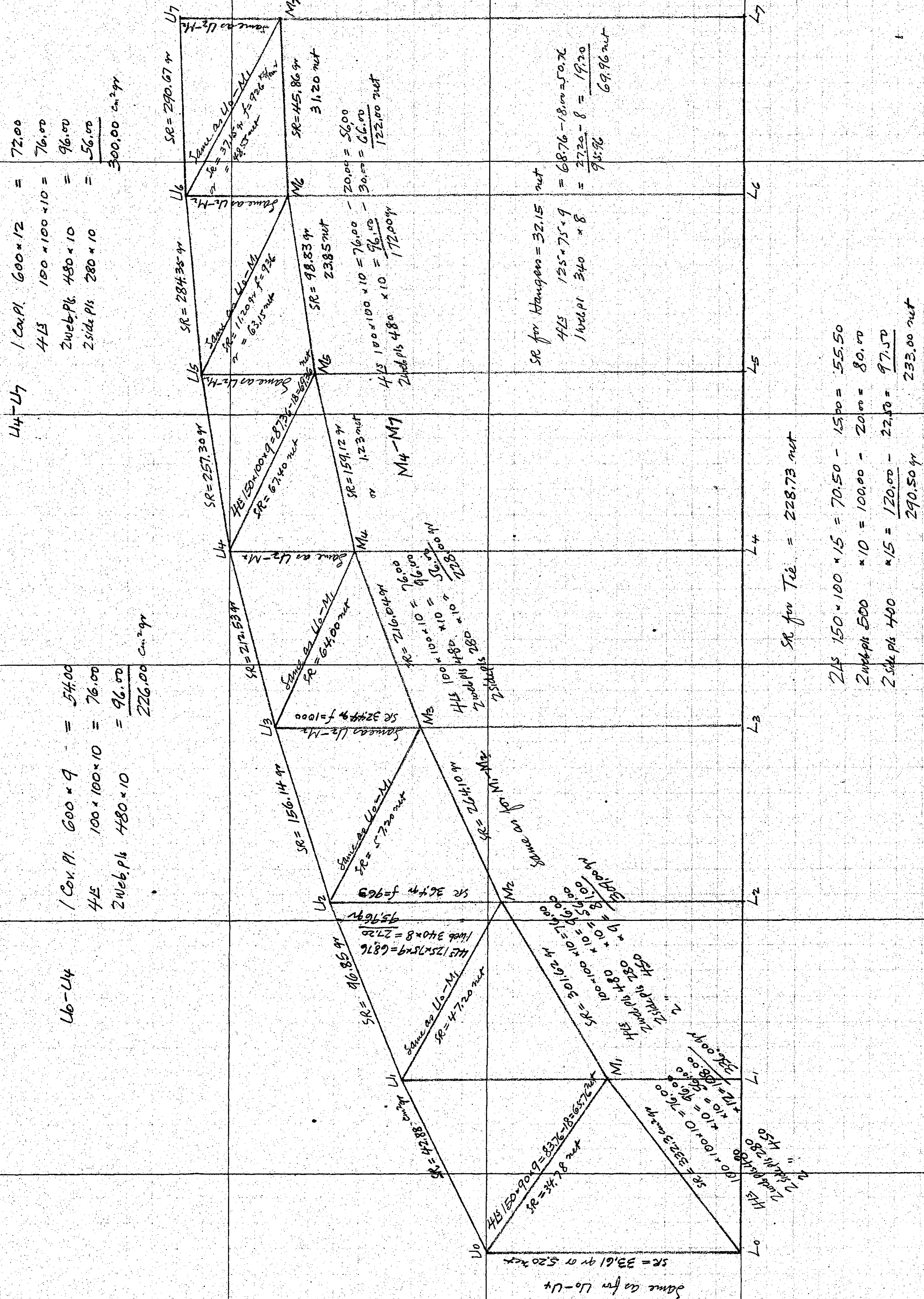
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken
Sections of truss members.



U4-U7
1 Cov. Pl. 600 × 12 = 72.00
4LS 100 × 100 × 10 = 76.00
2 Web. Pls. 480 × 10 = 96.00
2 side Pls 280 × 10 = 56.00
300.00 cm² gr

M4-M7
4LS 100×100×10 = 76.00 - 20.00 = 56.00
2 web pls 480 ×10 = 96.00 - 30.00 = 66.00
172.00 gr 122.00 net

SR for Hangers = 32.15 net
4LS 125×75×9 = 68.76 - 18 × 2.5 × 0.X
1 web Pl 340 × 8 = 27.20 - 8 = 19.20
95.96 69.96 net

SR for Tie = 228.73 net
2LS 150 × 100 × 15 = 70.50 - 15.00 = 55.50
2 web pls 500 × 10 = 100.00 - 20.00 = 80.00
2 side pls 400 × 15 = 120.00 - 22.50 = 97.50
290.50 gr 233.00 net

U0-U4
1 Cov. Pl. 600 × 9 = 54.00
4LS 100 × 100 × 10 = 76.00
2 Web. pls 480 × 10 = 96.00
226.00 cm² gr

CALCULATIONS FOR

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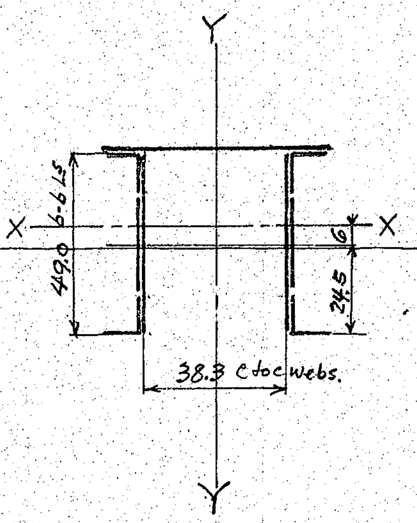
Design of Kisogawa-Bashi for Gifu-ken.

Top chord sections.

U_6-U_7	290,670 c
U_5-U_6	284,350 c
U_4-U_5	257,300 c



Center of gravity of section

			area		arm		moment
1 Cov. pl.	600 × 12	=	72.00	×	49.60	=	3,570
4 Ls	100 × 100 × 10	=	76.00	×	24.50	=	1,863
2 Web Pls.	480 × 10	=	96.00	×	24.50	=	2,352
2 Side Pls.	280 × 10	=	56.00	×	24.50	=	1,372
			300.00 gr.		30.5 cm		9,154

Ecc. = 30.5 − 24.50 = 6.00 cm.

Moment of inertia

X-X axis.

1 Cov. Pl. 600 × 12 = $72.00 \times 19.10^2 + 60 \times 1.2^3 \div 12$ = 26250 + 9 = 26259

2 Ls 100 × 100 × 10 = $38.00 \times 15.69^2 + 174.5 \times 2$ = 9360 + 349 = 9709

2 Ls 100 × 100 × 10 = $38.00 \times 27.69^2 + 174.5 \times 2$ = 29130 + 349 = 29,479

2 web pls 480 × 10 = $96.00 \times 6.00^2 + 2 \times 48^3 \div 12$ = 3455 + 18440 = 21,895

2 side pls 280 × 10 = $56.00 \times 6.00^2 + 2 \times 28^3 \div 12$ = 2015 + 3660 = 5675

300.00 cm² gr. I_x = 93,017 cm⁴

Radius of gyration $r_x = \sqrt{\frac{93017}{300}} = 17.60$ cm $\quad \frac{l}{r_x} = \frac{471.5}{17.6} = 26.8$

Y-Y axis.

1 Cov. Pl. 600 × 12 = 72.00 $1.2 \times 60^3 \div 12$ = = 21600

4 Ls 100 × 100 × 10 = $76.00 \times 22.46^2 + 174.5 \times 4$ = 38,360 + 698 = 39058

2 web Pls. 480 × 10 = $96.00 \times 19.15^2 + \frac{48 \times 1^3}{12} \times 2$ = 35180 + 8 = 35188

2 side pls 280 × 10 = $56.00 \times 20.15^2 + \frac{28 \times 1^3}{12} \times 2$ = 22720 + 5 = 22725

300.00 118,571 cm⁴

Radius of gyration $r_y = \sqrt{\frac{118571}{300}} = 19.88$ cm $\quad \frac{l}{r_y} = \frac{471.5}{19.88} = 23.7$

Allowable unit compression = $1500(1 - \overset{.8525}{0.0055 \times 26.8}) = 1280$ kg/cm² Use 1000 kg/cm²

Section required for

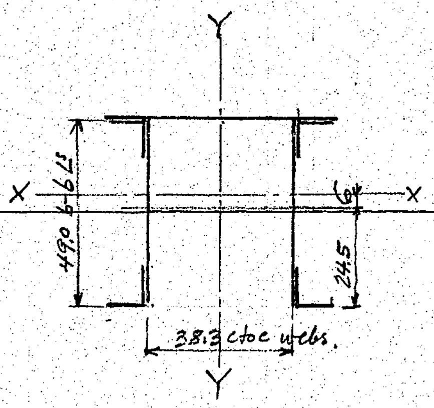
U_6-U_7 290670 ÷ 1000 = 290.67 cm² gross.

U_5-U_6 284350 ÷ 1000 = 284.35 " "

U_4-U_5 257300 ÷ 1000 = 257.30 " "

Top chord U_3-U_4, U_2-U_3, U_1-U_2, U_0-U_1, and U_0-L_0 max. stress = 212530 kg c

Center of gravity of section.



			area		arm		moment
1 Cov. pl.	600 × 9	=	54.00	×	49.45	=	2670
4 Ls	100 × 100 × 10	=	76.00	×	24.50	=	1863
2 web pls	480 × 10	=	96.00	×	24.50	=	2352
			226.00 cm² gr		30.5		6885

Ecc. = 30.5 − 24.5 = 6.00 cm

Moment of inertia

X-X axis.

1 Cov. pl. 600 × 9 = 54.00×18.95^2 19,380

2 Ls 100 × 100 × 10 = $38.00 \times 15.69^2 + 174.5 \times 2$ = 9,360 + 349 = 9709

2 Ls 100 × 100 × 10 = $38.00 \times 27.69^2 + 174.5 \times 2$ = 29130 + 349 = 29479

2 web pls 480 × 10 = $96.00 \times 6.0^2 + 2 \times 48^3 \div 12$ = 3,455 + 18440 = 21895

226.00 80,463 cm⁴

$r_x = \sqrt{\frac{80463}{226}} = 18.85$ cm, $\quad \frac{l}{r_x} = \frac{515.2}{18.85} = 27.4$

CALCULATIONS FOR

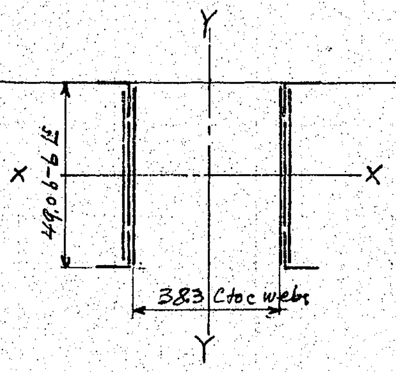
Design of Kisogawa-Bashi for Gifu-ken.

Y-Y axis
1 cov. pl. 600 × 9 = 54.00 $0.9 \times 60^3 \div 12$ = 16,200 = 16,200
4 Ls 100 × 100 × 10 = 76.00 × 22.46^2 + 174.5 × 4 = 38360 + 698 = 39,058
2 web pls 480 × 10 = 96.00 × 19.15^2 = 35180 = 35,180
226.00 cm² 90,438 cm⁴

$r_y = \sqrt{\frac{90438}{226.00}} = 20.0$ cm $l/r_y = \frac{515.2}{20.0} = 25.76$

Allowable unit compression = 1500 (1 − 0.0055 × 27.4) = 1275 kg/cm², use 1,000 kg/cm² (.8490)
Section required for
U₃−U₄ 212530 ÷ 1000 = 212.53 cm² gross
U₂−U₃ 156,140 ÷ 〃 = 156.14 〃
U₁−U₂ 96850 ÷ 〃 = 96.85 〃
U₀−U₁ 42,880 ÷ 〃 = 42.88 〃
U₀−L₀ 33,610 ÷ 〃 = 33.61 〃

Middle chord sections.
L₀−M₁ 332,300 C



x-x axis
4 Ls 100 × 100 × 10 = 76.00 × 21.69^2 + 174.5 × 4 = 35750 + 698 = 36,448
2 web pls 480 × 10 = 96.00 $2 \times 48.0^3 \div 12$ = 18,440
2 side pls. 280 × 10 = 56.00 $2 \times 28.0^3 \div 12$ = 3660
2 〃 450 × 12 = 108.00 $2 \times 1.2 \times 45.0^3 \div 12$ = 18240
336.00 gr 76,788 cm⁴

$r_x = \sqrt{\frac{76788}{336.0}} = 15.12$ cm $l/r_x = \frac{563.1}{15.12} = 37.3$

Y-Y axis
4 Ls 100 × 100 × 10 = 76.00 × 22.46^2 + 174.5 × 4 = 38,360 + 698 = 39058
2 web pls. 480 × 10 = 96.00 × 19.15^2 = 35,180
2 side pls. 280 × 10 = 56.00 × 20.15^2 = 22,720
2 〃 450 × 12 = 108.00 × 21.25^2 = 48750
336.00 gr 145,708 cm⁴

$r_y = \sqrt{\frac{145708}{336.00}} = 20.82$ cm $l/r_y = \frac{563.1}{20.82} = 27.1$

Allowable unit compression = 1500 (1 − 0.0055 × 37.3) = 1192 kg/cm² use 1000 kg/cm² (.795)
Section req'd. for L₀−M₁ = 332300 ÷ 1000 = 332.30 cm² gross.

M₁−M₂ 301,620 C
M₂−M₃ 264,100 C

x-x axis.
4 Ls 100 × 100 × 10 = 76.00 = 36,448
2 web pls. 480 × 10 = 96.00 = 18,440
2 side pls 280 × 10 = 56.00 = 3660
2 〃 450 × 9 = 81.00 $2 \times 0.9 \times 45.0^3 \div 12$ = 13680
309.00 cm² gr 72228 cm⁴

$r_x = \sqrt{\frac{72228}{309.0}} = 15.3$ cm, $l/r_x = \frac{529.1}{15.3} = 34.6$

Y-Y axis.
4 Ls 100 × 100 × 10 = 76.00 = 39,058
2 web pls. 480 × 10 = 96.00 = 35,180
2 side pls 280 × 10 = 56.00 = 22,720
2 〃 450 × 9 = 81.00 × 21.10^2 = 36050
309.00 gr 133,008 cm⁴

$r_y = \sqrt{\frac{133008}{309.00}} = 20.75$ cm, $l/r_y = \frac{529.1}{20.75} = 25.5$

Allowable unit compression = 1500 (1 − 0.0055 × 34.6) = 1215 kg/cm² Use 1,000 kg/cm² (.810)
Section required for
M₁−M₂ 301,620 ÷ 1000 = 301.62 cm² gross
M₂−M₃ 264,100 ÷ 1000 = 264.10 〃

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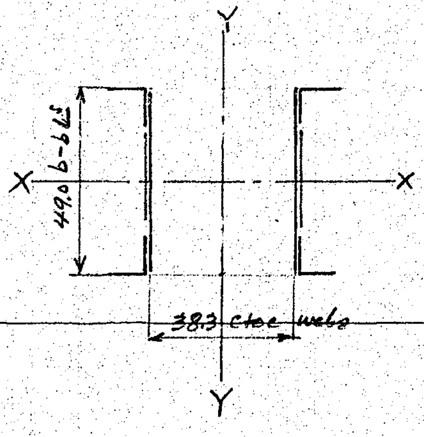
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-ken.

M3-M4 216040 c



x-x axis.

4Ls 100 × 100 × 10 = 76.00 = 36448
2web pls 480 × 10 = 96.00 = 18,440
2side pls 280 × 10 = 56.00 = 3660
228.00 cm² gr. 58,548 cm⁴

$r_x = \sqrt{\frac{58548}{228.0}} = 16.03$ cm . $l/r_x = \frac{488.0}{16.03} = 30.4$

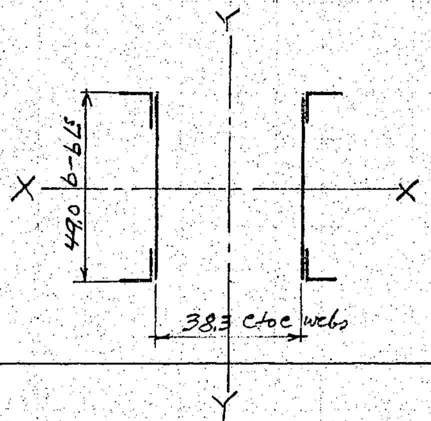
Y-Y axis

4Ls 100 × 100 × 10 = 76.00 = 39,058
2web pls. 480 × 10 = 96.00 = 35180
2side pls 280 × 10 = 56.00 = 22,720
228.00 gr. 96,958 cm⁴

$r_y = \sqrt{\frac{96958}{228.00}} = 20.62$ cm , $l/r_y = \frac{488.0}{20.62} = 23.7$

Allowable unit compression = $1500(1 - \overset{.833}{0.0033} \times 30.4) = 1250$ kg/cm², use 1000 kg/cm²
Section required for M3-M4 = 216040 ÷ 1000 = 216.04 cm² gross.

M4 - M5 159,120 c or 1470 T
M5 - M6 98830 c or 28590 T
M6 - M7 45,860 c or 37420 T



X-X axis.

4Ls 100 × 100 × 10 = 76.00 = 36448
2web pls 480 × 10 = 96.00 = 18440
172.00 cm² gr 54888 cm⁴

$r_x = \sqrt{\frac{54888}{172.00}} = 17.9$ cm , $l/r_x = \frac{476.3}{17.9} = 26.6$

Y-Y axis.

4Ls 100 × 100 × 10 = 76.00 = 39,058
2web pls. 480 × 10 = 96.00 = 35180
172.00 74238 cm⁴

$r_y = \sqrt{\frac{74238}{172.00}} = 20.78$ cm , $l/r_y = \frac{476.3}{20.78} = 23.0$

Allowable unit compression = $1500(1 - \overset{.834}{0.0033} \times 26.6) = 1280$ kg/cm², use 1000 kg/cm²
Section required for
M4-M5 159,120 ÷ 1000 = 159.120 gross
or 1470 ÷ 1200 = 1.23 net

M5-M6 98830 ÷ 1000 = 9883 gross
or 28590 ÷ 1200 = 23.80 net

M6-M7 45860 ÷ 1000 = 45.86 gross
or 37,420 ÷ 1200 = 31.20 net

net area of the section

4Ls 100 × 100 × 10 = 76.00 − 20.00 = 56.00
2web pls 480 × 10 = 96.00 − 30.00 = 66.00
172.00 gr 122.00 cm² net

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CONSULTING ENGINEER
~~[illegible]~~ BLDG, TOKYO
SHOWA

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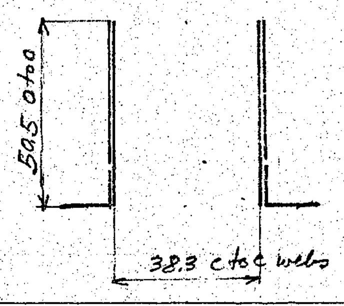
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken

Tie section

Stress 274,470 T Section required = $\frac{274470}{1200}$ = 228.73 cm² net.

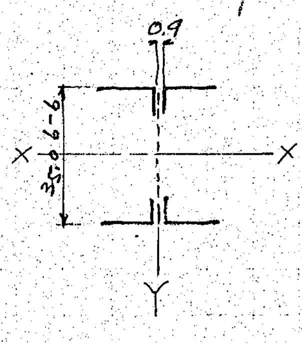


2Ls 150×100×15 = 70.50 − 15.00 = 55.50
2 web pls. 500 ×10 = 100.00 − 20.00 = 80.00
2 side pls. 400 ×15 = 120.00 − 22.50 = 97.50
290.50 gr   233.00 net

Diagonal sections.

U0−M1	41,710 T or 7,560 C	÷1200	= 34.78 cm² net	4Ls 150×90×9 = 83.76 − 18.00 = 65.76 net
U1−M2	56,620 T	÷1200	= 47.20 〃	〃 〃 〃
U2−M3	68,670 T	÷1200	= 57.20 〃	〃 〃 〃
U3−M4	76,790 T	÷1200	= 64.00 〃	〃 〃 〃
U4−M5	80,870 T	÷1200	= 67.40 〃	4Ls 150×100×9 = 87.36 − 18.00 = 69.36 net 32
U5−M6	75,790 T or 11,420 C	÷1200	= 63.15 〃	4Ls 150×90×9 = 83.76 − 18.00 = 65.76 net 28
U6−M7	58,230 T or 34,410 C	÷1200	= 48.55 〃	〃 〃 〃

moment of inertia



Y−Y axis

4Ls 150×90×9 = 83.76 × 5.41² + 468.2×4 = 2450 + 1870 = 4320 cm⁴

$r_y = \sqrt{\frac{4320}{83.76}}$ = 7.18 cms

for U6−M7 $l/r_y = \frac{500.3}{7.18}$ = 69.7

Allowable unit comp. = 1500 (1 − $\overset{.617}{0.0055 \times 69.7}$) = 926 kg/cm²

gross area required = 34410 ÷ 926 = 37.15 gross.

for U5−M6 $l/r_y = \frac{491.2}{7.18}$ = 68.5

allowable unit comp. = 1500 (1 − $\overset{.6235}{0.0055 \times 68.5}$) = 936 kg/cm²

~~gross area~~ required = 11420 ÷ 936 = 12.20 gross

Hangers and verticals.

All hangers 38580 T

Section required = 38580 ÷ 1200 = 32.15 net

Use

4Ls 125×75×9 = 68.76 − 18 = 50.76
1 web pl. 340 ×8 = 27.20 − 8 = 19.20
95.96   69.96 cm² net.

for verticals (except U1−M1)

4Ls 125×75×9 = 68.76 − 18 = 50.76 net
1 web 340 ×8 = 27.20 − 10 = 17.20
95.96   67.96 net

moment of inertia I_y

68.76×4.59² + 269.1×4 = 2525

$r_y = \sqrt{\frac{2525}{95.96}}$ = 5.13 cm

$l/r_y = \frac{333.8}{5.13}$ = 65.1

f = 1500 (1 − $\overset{.642}{0.0055 \times 65.1}$) = 963 kg/cm² C

verticals. (except U1−M1)

35020 C   SR = 35020 ÷ 963 = 36.4 gross
29650 T   SR = 29650 ÷ 1200 = 24.7 net

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken

Final calculation of Horizontal thrust H. (Refer to page 25-28).
Used gross sectional area and slenderness ratio l/A of truss members.

members	Upper chords			Middle chords			Diagonals			Verticals		
m	l cm	A cm² gr	l/A	l cm	A cm²	l/A	l cm	A cm²	l/A	l cm	A cm²	l/A
0	—	—	—	563.1	336.0	1.676	—	—	—	500.0	226.00	2.212
1	515.2	226.0	2.280	529.1	309.0	1.712	499.5	83.76	5.963	404.3	115.00	3.516
2	499.3	〃	2.209	505.1	〃	1.635	489.1	〃	5.839	333.8	95.96	3.479
3	487.2	〃	2.156	488.0	228.0	2.140	484.6	〃	5.786	281.9	〃	2.938
4	478.0	〃	2.115	476.3	172.0	2.769	483.8	〃	5.776	244.5	〃	2.548
5	471.5	300.0	1.572	469.0	〃	2.727	486.0	87.36	5.563	219.4	〃	2.286
6	467.3	〃	1.558	465.4	〃	2.706	491.2	83.76	5.864	204.8	〃	2.134
7	465.3	〃	1.551	—	—	—	500.3	〃	5.973	200.0	〃	2.084

Total length of tie 14 @ 4.650 = 65.100 meters
used gross area of tie = 290.50 cm²

For tie $l/A = \frac{6510}{290.5} = 22.410$

Upper chord $S_{0,1}' S_a l/A$ and $S_a^2 l/A$. ($S_{0,1}' S_a$ and S_a^2 same as for preliminary calculation on P.26)

Marks	members	$S_{0,1}' S_a$	S_a^2	l/A	$S_{0,1}' S_a l/A$	$S_a^2 l/A$
A	1	0.0792	0.7569	2.280	0.1806	1.7257
B	2	0.3925	3.3636	2.209	0.8670	7.4302
C	3	1.0549	8.1282	2.156	2.2744	17.5244
D	4	2.1510	14.8071	2.115	4.5494	31.3170
E	5	3.6110	22.1653	1.572	5.6765	34.8439
F	6	5.1814	28.0688	1.558	8.0726	43.7312
G	7	6.4000	30.2830	1.551	9.9264	46.9689
H	7'	6.4000	30.2830	1.551	9.9264	46.9689
I	6'	6.9086	28.0688	1.558	10.7636	43.7312
J	5'	6.5017	22.1653	1.572	10.2207	34.8439
K	4'	5.3718	14.8071	2.115	11.3614	31.3170
L	3'	3.8717	8.1282	2.156	8.3474	17.5244
M	2'	2.3512	3.3636	2.209	5.1938	7.4302
N	1'	1.0292	0.7569	2.280	2.3466	1.7257
						367.0826 $= \Sigma S_a^2 l/A$

Middle chord $S_{0,1}' S_a l/A$ and $S_a^2 l/A$.

A	0	0.0000	1.4665	1.676	0.0000	2.4579
B	1	0.1910	4.1290	1.712	0.3270	7.0688
C	2	0.6353	8.6495	1.635	1.0387	14.1419
D	3	1.4491	15.2568	2.140	3.1011	32.6496
E	4	2.7070	23.6196	2.769	7.4957	65.4027
F	5	4.3495	32.4102	2.727	11.8611	88.3826
G	6	6.1157	39.4258	2.706	16.5491	106.6862
H	6'	8.1564	39.4258	2.706	22.0712	106.6862
I	5'	7.8222	32.4102	2.727	21.3311	88.3826
J	4'	6.7651	23.6196	2.769	18.7326	65.4027
K	3'	5.3122	15.2568	2.140	11.3681	32.6496
L	2'	3.8145	8.6495	1.635	6.2367	14.1419
M	1'	2.4709	4.1290	1.712	4.2302	7.0688
N	0'	0.0000	1.4665	1.676	0.0000	2.4579
						633.5794 $= \Sigma S_a^2 l/A$

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Design of Kisogawa-Bashi for Gifu-Ken.

Diagonals $S_{o.1}' S_a \, l/A$ and $S_a^2 \, l/A$.

Marks	members	$S_{o.1}' S_a$	S_a^2	l/A	$S_{o.1}' S_a \, l/A$	$S_a^2 \, l/A$
A	1	0.0743	0.7123	5.963	0.4431	4.2474
B	2	0.1193	0.9409	5.839	0.6966	5.4939
C	3	0.1702	1.1172	5.786	0.9848	6.4641
D	4	0.2107	1.1321	5.776	1.2170	6.5390
E	5	0.2096	0.8836	5.563	1.1660	4.9155
F	6	0.1514	0.4409	5.864	0.8878	2.5854
G	7	0.0500	0.0600	5.973	0.2987	0.3584
H	7'	− 0.0355	0.0600	5.973	− 0.2120	0.3584
I	6'	− 0.0458	0.4409	5.864	− 0.2686	2.5854
J	5'	0.0038	0.8836	5.563	0.0211	4.9155
K	4'	0.0692	1.1321	5.776	0.3997	6.5390
L	3'	0.1120	1.1172	5.786	0.6480	6.4641
M	2'	0.1290	0.9409	5.839	0.7532	5.4939
N	1'	0.9689	0.7123	5.963	5.7776	4.2474
						61.2074 = $\Sigma S_a^2 \, l/A$

Verticals $S_{o.1}' S_a \, l/A$ and $S_a^2 \, l/A$

A	0	0.0485	0.4665	2.212	0.1073	1.0319
B	1	0.0458	0.3540	3.516	0.1610	1.2447
C	2	0.0374	0.2304	3.479	0.1301	0.8016
D	3	0.0249	0.1129	2.938	0.0732	0.3317
E	4	0.0102	0.0262	2.548	0.0260	0.0668
F	5	− 0.0018	0.0017	2.286	− 0.0041	0.0039
G	6	− 0.0043	0.0645	2.134	− 0.0092	0.1376
G	7	0.0280	0.1325	2.084	0.0584	0.2761
H	6'	0.0368	0.0645	2.134	0.0785	0.1376
I	5'	0.0050	0.0017	2.286	0.0114	0.0039
J	4'	− 0.0151	0.0262	2.548	− 0.0385	0.0668
K	3'	− 0.0212	0.1129	2.938	− 0.0623	0.3317
L	2'	− 0.0154	0.2304	3.479	− 0.0536	0.8016
M	1'	− 0.0006	0.3540	3.516	− 0.0021	1.2447
N	0'	0.6338	0.4665	2.212	1.4020	1.0319
						7.5125 = $\Sigma S_a^2 \, l/A$

Tie $S_{o.1}' S_a \, l/A$ and $S_a^2 \, l/A$.

	$L_0 - L_0'$	—	1.0000	22.410	—	22.4100
						22.4100 = $\Sigma S_a^2 \, l/A$

Summary of $S_a^2 \, l/A$ for the entire span.

Upper chord members.	$\Sigma S_a^2 \, l/A$	=	367.0826
middle chord members	"	=	633.5794
diagonal members	"	=	61.2074
vertical members	"	=	7.5125
tie	"	=	22.4100
	Summary	=	1091.7919

Design of Kisogawa-Bashi for Gifu-Ken

Summary of S_{o1}, $S_a \frac{l}{A}$ for the entire span.

Marks	Upper chord	Middle chord	Diagonals	Verticals	Summary	Second Summary	Remarks	
A	0.1806	0.0000	0.4431	0.1073	0.7310	0.7310	A	1
B	0.8670	0.3270	0.6966	0.1610	2.0516	2.7826	A ~ B	2
C	2.2744	1.0387	0.9848	0.1301	4.4280	7.2106	A ~ C	3
D	4.5494	3.1011	1.2170	0.0732	8.9407	16.1513	A ~ D	4
E	5.6765	7.4957	1.1660	0.0260	14.3642	30.5155	A ~ E	5
F	8.0726	11.8611	0.8878	−0.0041	20.8174	51.3329	A ~ F	6
G	9.9264	16.5491	0.2987	−0.0092	26.7650	78.0979		
〃	—	—	—	0.0584	0.0584	78.1563	A ~ G	7
H	9.9264	22.0712	−0.2120	0.0785	31.8641	110.0204	A ~ H	7′
I	10.7636	21.3311	−0.2686	0.0114	31.8375	141.8579	A ~ I	6′
J	10.2207	18.7326	0.0211	−0.0385	28.9359	170.7938	A ~ J	5′
K	11.3614	11.3681	0.3997	−0.0623	23.0669	193.8607	A ~ K	4′
L	8.3474	6.2367	0.6480	−0.0536	15.1785	209.0392	A ~ L	3′
M	5.1938	4.2302	0.7532	−0.0021	10.1751	219.2143	A ~ M	2′
N	2.3466	0.0000	5.7776	1.4020	9.5262	228.7405	A ~ N	1′

Influence surface of Horizontal Thrust.
Load unity on points

						$\Sigma S_o S_a \frac{l}{A}$		$\Sigma S_a^2 \frac{l}{A}$		Thrust H.	
1	228.7405	+	13 × 0.0000	=		228.7405	÷	1091.7919	=	0.210	H_1
2	2 × 209.0392	+	12 × 2.7826	=		451.4696	÷	〃	=	0.414	H_2
3	3 × 193.8607	+	11 × 7.2106	=		660.8987	÷	〃	=	0.605	H_3
4	4 × 170.7938	+	10 × 16.1513	=		844.6882	÷	〃	=	0.774	H_4
5	5 × 141.8579	+	9 × 30.5155	=		983.9290	÷	〃	=	0.901	H_5
6	6 × 110.0204	+	8 × 51.3329	=		1070.7856	÷	〃	=	0.981	H_6
7	7 × 78.1563	+	7 × 78.0979	=		1093.7794	÷	〃	=	1.002	H_7

Summary of H for the entire span = 8.772
(Point of application of H = 0.999 × 13.0)

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

Final calculations for Influence surfaces of truss members.

General formula for member stresses

$S = S_0 - S_aH.$ for unit load.

SaH for Upper chord members.

		Load unity on panel point						
		1	2	3	4	5	6	7
members	Sa	H=0.210	0.414	0.605	0.774	0.901	0.981	1.002
U_0-U_1	− 0.870	− 0.183	− 0.360	− 0.526	− 0.673	−0.784	− 0.853	− 0.872
U_1-U_2	− 1.834	− 0.385	− 0.759	− 1.110	− 1.420	− 1.652	− 1.799	− 1.838
U_2-U_3	− 2.851	− 0.599	− 1.180	− 1.725	− 2.207	− 2.569	− 2.797	− 2.857
U_3-U_4	− 3.848	− 0.808	− 1.593	− 2.328	− 2.978	−3.467	− 3.775	− 3.856
U_4-U_5	− 4.708	− 0.989	− 1.949	− 2.848	− 3.644	−4.242	− 4.619	− 4.717
U_5-U_6	− 5.298	− 1.113	− 2.193	− 3.205	−4.101	−4.773	− 5.197	− 5.309
U_6-U_7	− 5.503	− 1.156	− 2.278	− 3.329	−4.259	−4.958	− 5.398	− 5.614

SaH for Middle chord members.

L_0-M_1	1.211	0.254	0.501	0.733	0.937	1.091	1.188	1.213
M_1-M_2	2.032	0.427	0.841	1.229	1.573	1.831	1.993	2.036
M_2-M_3	2.941	0.618	1.218	1.779	2.276	2.650	2.885	2.947
M_3-M_4	3.906	0.820	1.617	2.363	3.023	3.519	3.832	3.914
M_4-M_5	4.860	1.021	2.012	2.940	3.762	4.379	4.768	4.870
M_5-M_6	5.693	1.196	2.357	3.444	4.406	5.129	5.585	5.704
M_6-M_7	6.279	1.319	2.600	3.799	4.860	5.657	6.160	6.292

SaH for Diagonal members.

U_0-M_1	0.844	0.177	0.349	0.511	0.653	0.760	0.828	0.846
U_1-M_2	0.970	0.204	0.402	0.587	0.751	0.874	0.952	0.972
U_2-M_3	1.057	0.222	0.438	0.639	0.818	0.952	1.037	1.059
U_3-M_4	1.064	0.223	0.440	0.644	0.824	0.959	1.044	1.066
U_4-M_5	0.940	0.197	0.389	0.569	0.728	0.847	0.922	0.942
U_5-M_6	0.664	0.139	0.275	0.402	0.514	0.598	0.651	0.665
U_6-M_7	0.245	0.051	0.101	0.148	0.190	0.221	0.240	0.245

SaH for Vertical members

U_0-L_0	− 0.683	− 0.143	−0.283	−0.413	−0.529	−0.615	− 0.670	− 0.684
U_1-M_1	− 0.595	− 0.125	−0.246	−0.360	− 0.461	−0.536	− 0.584	− 0.596
U_2-M_2	− 0.480	− 0.101	− 0.199	−0.290	− 0.372	− 0.432	− 0.471	−0.481
U_3-M_3	− 0.336	− 0.071	− 0.139	− 0.203	− 0.260	− 0.303	− 0.330	− 0.337
U_4-M_4	− 0.162	− 0.034	− 0.067	− 0.098	− 0.125	−0.146	− 0.159	− 0.162
U_5-M_5	0.041	0.009	0.017	0.025	0.032	0.037	0.040	0.041
U_6-M_6	0.254	0.053	0.105	0.154	0.197	0.229	0.249	0.255
U_7-M_7	0.364	0.076	0.151	0.220	0.282	0.328	0.357	0.365

SaH for Tie

L_0-L_0'	− 1.000	− 0.210	− 0.414	− 0.605	− 0.774	−0.901	− 0.981	− 1.002

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-ken

Final Influence surface of Upper chord members.

Stress of each member $S = S_0 - S_aH$

	U_0-U_1			U_1-U_2			U_2-U_3		
Load on	S_0	S_aH	S	S_0	S_aH	S	S_0	S_aH	S
1'	− 0.091	− 0.183	0.092	− 0.214	− 0.385	0.171	− 0.370	− 0.599	0.229
2'	− 0.182	− 0.360	0.178	− 0.427	− 0.759	0.332	− 0.741	− 1.180	0.439
3'	− 0.273	− 0.526	0.253	− 0.641	− 1.110	0.469	− 1.111	− 1.725	0.614
4'	− 0.364	− 0.673	0.309	− 0.855	− 1.420	0.565	− 1.481	− 2.207	* 0.726
5'	− 0.455	− 0.784	* 0.329	− 1.069	− 1.652	* 0.583	− 1.852	− 2.569	0.717
6'	− 0.546	− 0.853	0.307	− 1.282	− 1.799	0.517	− 2.222	− 2.797	0.575
7	− 0.637	− 0.872	0.235	− 1.496	− 1.838	0.342	− 2.592	− 2.857	0.265
6	− 0.728	− 0.853	0.125	− 1.710	− 1.799	0.089	− 2.962	− 2.797	− 0.165
5	− 0.819	− 0.784	− 0.035	− 1.923	− 1.652	− 0.271	− 3.333	− 2.569	− 0.764
4	− 0.910	− 0.673	− 0.237	− 2.137	− 1.420	− 0.717	− 3.703	− 2.207	− 1.496
3	− 1.001	− 0.526	− 0.475	− 2.351	− 1.110	− 1.241	− 4.073	− 1.725	* − 2.348
2	− 1.092	− 0.360	− 0.732	− 2.564	− 0.759	* − 1.805	− 2.715	− 1.180	− 1.535
1	− 1.183	− 0.183	* − 1.000	− 1.282	− 0.385	− 0.897	− 1.358	− 0.599	− 0.759
	Σ plus stresses		1.828			3.068			3.565
	Σ minus 〃		− 2.479			− 4.931			− 7.067
	Summary		− 0.651			− 1.863			− 3.502

	U_3-U_4			U_4-U_5			U_5-U_6		
Load on	S_0	S_aH	S	S_0	S_aH	S	S_0	S_aH	S
1'	− 0.559	− 0.808	0.249	− 0.767	− 0.989	0.222	− 0.978	− 1.113	0.135
2'	− 1.117	− 1.593	0.476	− 1.535	− 1.949	0.414	− 1.956	− 2.193	0.237
3'	− 1.676	− 2.328	0.652	− 2.302	− 2.848	0.546	− 2.933	− 3.205	* 0.272
4'	− 2.234	− 2.978	* 0.744	− 3.070	− 3.644	* 0.574	− 3.911	− 4.101	0.190
5'	− 2.793	− 3.467	0.674	− 3.837	− 4.242	0.405	− 4.889	− 4.773	− 0.116
6'	− 3.351	− 3.775	0.424	− 4.604	− 4.619	0.015	− 5.867	− 5.197	− 0.670
7	− 3.910	− 3.856	− 0.054	− 5.372	− 4.717	− 0.655	− 6.845	− 5.309	− 1.536
6	− 4.468	− 3.775	− 0.693	− 6.139	− 4.619	− 1.520	− 7.822	− 5.197	* − 2.625
5	− 5.027	− 3.467	− 1.560	− 6.907	− 4.242	* − 2.665	− 6.519	− 4.773	− 1.746
4	− 5.585	− 2.978	* − 2.607	− 5.526	− 3.644	− 1.882	− 5.215	− 4.101	− 1.114
3	− 4.188	− 2.328	− 1.860	− 4.144	− 2.848	− 1.296	− 3.911	− 3.205	− 0.706
2	− 2.792	− 1.593	− 1.199	− 2.763	− 1.949	− 0.814	− 2.608	− 2.193	− 0.415
1	− 1.396	− 0.808	− 0.588	− 1.381	− 0.989	− 0.392	− 1.304	− 1.113	− 0.191
	Σ Plus stresses		3.219			2.176			0.834
	Σ Minus 〃		− 8.561			− 9.224			− 9.119
	Summary		− 5.342			− 7.048			− 8.285

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-ken

Load on	S_o	U_6-U_7 S_aH	S
1'	-1.163	-1.156	-0.007
2'	-2.326	-2.278	-0.048
3'	-3.489	-3.329	-0.160
4'	-4.652	-4.259	-0.393
5'	-5.816	-4.958	-0.858
6'	-6.979	-5.398	-1.581
7	-8.142	-5.514	* -2.628
6	-6.979	-5.398	-1.581
5	-5.816	-4.958	-0.858
4	-4.652	-4.259	-0.393
3	-3.489	-3.329	-0.160
2	-2.326	-2.278	-0.048
1	-1.163	-1.156	-0.007
	Σ Plus stresses		0.000
	Σ Minus "		-8.722
	Summary		-8.722

Final Influence surface of Middle chord members.

Load on	L_o-M_1 S_o	 S_aH	 S	M_1-M_2 S_o	 S_aH	 S	M_2-M_3 S_o	 S_aH	 S
1'	0.000	0.254	-0.254	0.094	0.427	-0.333	0.216	0.618	-0.402
2'	0.000	0.501	-0.501	0.187	0.841	-0.654	0.432	1.218	-0.786
3'	0.000	0.733	-0.733	0.281	1.229	-0.948	0.649	1.779	-1.130
4'	0.000	0.937	-0.937	0.374	1.573	-1.199	0.865	2.276	-1.411
5'	0.000	1.091	-1.091	0.468	1.831	-1.363	1.081	2.650	-1.569
6'	0.000	1.188	-1.188	0.561	1.993	* -1.432	1.297	2.885	* -1.588
7	0.000	1.213	* -1.213	0.655	2.036	-1.381	1.513	2.947	-1.434
6	0.000	1.188	-1.188	0.748	1.993	-1.245	1.730	2.885	-1.155
5	0.000	1.091	-1.091	0.842	1.831	-0.989	1.946	2.650	-0.704
4	0.000	0.937	-0.937	0.935	1.573	-0.638	2.162	2.276	-0.114
3	0.000	0.733	-0.733	1.029	1.229	-0.200	2.378	1.779	0.599
2	0.000	0.501	-0.501	1.122	0.844	0.281	2.594	1.218	* 1.376
1	0.000	0.254	-0.254	1.216	0.427	* 0.789	1.297	0.618	0.679
	Σ Plus stresses		0.000			1.070			2.654
	Σ Minus "		-10.621			-10.382			-10.293
	Summary		-10.621			-9.312			-7.639

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

	M3-M4			M4-M5			M5-M6		
Load on	S_o	S_{aH}	S	S_o	S_{aH}	S	S_o	S_{aH}	S
1'	0.371	0.820	-0.449	0.557	1.021	-0.464	0.764	1.196	-0.432
2'	0.742	1.617	-0.875	1.113	2.012	-0.899	1.527	2.357	-0.830
3'	1.113	2.363	-1.250	1.670	2.940	-1.270	2.291	3.444	-1.153
4'	1.484	3.023	-1.539	2.226	3.762	-1.536	3.054	4.406	* -1.352
5'	1.855	3.519	* -1.664	2.783	4.379	* -1.596	3.818	5.129	-1.311
6'	2.226	3.832	-1.606	3.340	4.768	-1.428	4.581	5.585	-1.004
7	2.597	3.914	-1.317	3.896	4.870	-0.974	5.345	5.704	-0.359
6	2.968	3.832	-0.864	4.453	4.768	-0.315	6.108	5.585	0.523
5	3.339	3.519	-0.180	5.009	4.379	0.630	6.872	5.129	* 1.743
4	3.710	3.023	0.687	5.566	3.762	* 1.804	5.498	4.406	1.092
3	4.081	2.363	* 1.718	4.175	2.940	1.235	4.123	3.444	0.679
2	2.720	1.617	1.103	2.783	2.012	0.771	2.749	2.357	0.392
1	1.360	0.820	0.540	1.392	1.021	0.371	1.374	1.196	0.178
	Σ Plus stresses		4.048			4.811			4.607
	Σ minus "		-9.744			-8.482			-6.441
	Summary		-5.696			-3.671			-1.834

	M6-M7		
Load on	S_o	S_{aH}	S
1'	0.974	1.319	-0.345
2'	1.948	2.600	-0.652
3'	2.922	3.799	-0.877
4'	3.896	4.860	* -0.964
5'	4.870	5.657	-0.787
6'	5.844	6.160	-0.316
7	6.818	6.292	0.526
6	7.792	6.160	* 1.632
5	6.494	5.657	0.837
4	5.195	4.860	0.335
3	3.896	3.799	0.097
2	2.597	2.600	-0.003
1	1.299	1.319	-0.020
	Σ Plus stresses		3.427
	Σ minus "		-3.964
	Summary		-0.537

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CALCULATIONS FOR

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Design of Kisogawa-Bashi for Gifu-ken.

Final Influence surface of Diagonal members.

Load on	U_0-M_1			U_1-M_2			U_2-M_3		
	S_o	SaH	S	S_o	SaH	S	S_o	SaH	S
1'	0.088	0.177	- 0.089	0.123	0.204	- 0.081	0.161	0.222	-0.061
2'	0.177	0.349	- 0.172	0.246	0.402	- 0.156	0.322	0.438	- 0.116
3'	0.265	0.511	- 0.246	0.369	0.587	- 0.218	0.483	0.639	- 0.156
4'	0.353	0.653	- 0.300	0.492	0.751	* - 0.259	0.644	0.818	- 0.174
5'	0.442	0.760	* - 0.318	0.615	0.874	* - 0.259	0.805	0.952	- 0.147
6'	0.530	0.828	- 0.298	0.737	0.952	- 0.215	0.966	1.037	- 0.071
7	0.618	0.846	- 0.228	0.860	0.972	- 0.112	1.127	1.059	0.068
6	0.706	0.828	- 0.122	0.983	0.952	0.031	1.288	1.037	0.251
5	0.795	0.760	0.035	1.106	0.874	0.232	1.449	0.952	0.497
4	0.883	0.653	0.230	1.229	0.751	0.478	1.610	0.818	0.792
3	0.971	0.511	0.460	1.352	0.587	0.765	1.771	0.639	* 1.132
2	1.060	0.349	0.711	1.475	0.402	* 1.073	0.213	0.438	* - 0.225
1	1.148	0.177	* 0.971	0.133	0.204	- 0.071	0.106	0.222	- 0.116
	Σ Plus stresses		2.407			2.579			2.740
	Σ minus "		- 1.773			- 1.371			-1.066
	Summary		0.634			1.208			1.674

Load on	U_3-M_4			U_4-M_5			U_5-M_6		
	S_o	SaH	S	S_o	SaH	S	S_o	SaH	S
1'	0.198	0.223	- 0.025	0.223	0.197	0.026	0.228	0.139	0.089
2'	0.395	0.440	- 0.045	0.446	0.389	0.057	0.457	0.275	0.182
3'	0.593	0.644	- 0.051	0.670	0.569	0.101	0.685	0.402	0.283
4'	0.790	0.824	- 0.034	0.893	0.728	0.165	0.913	0.514	0.399
5'	0.988	0.959	0.029	1.116	0.847	0.269	1.142	0.598	0.544
6'	1.186	1.044	0.142	1.339	0.922	0.417	1.370	0.651	0.719
7	1.383	1.066	0.317	1.562	0.942	0.620	1.598	0.665	0.933
6	1.581	1.044	0.537	1.786	0.922	0.864	1.826	0.651	* 1.175
5	1.778	0.959	0.819	2.009	0.847	* 1.162	-0.344	0.598	* -0.942
4	1.976	0.824	* 1.152	0.017	0.728	* - 0.711	-0.275	0.514	-0.789
3	0.195	0.644	* - 0.449	0.013	0.569	- 0.556	-0.206	0.402	-0.608
2	0.130	0.440	- 0.310	0.008	0.389	- 0.381	-0.137	0.275	-0.412
1	0.065	0.223	- 0.158	0.004	0.197	- 0.193	-0.069	0.139	-0.208
	Σ Plus stresses		2.996			3.681			4.324
	Σ minus "		- 1.072			- 1.841			- 2.959
	Summary		1.924			1.840			1.365

Design of Kisogawa-Bashi for Gifu-Ken.

		U6-M7	
Load on	So	SaH	S
1'	0.204	0.051	0.153
2'	0.407	0.101	0.306
3'	0.611	0.148	0.463
4'	0.815	0.190	0.625
5'	1.019	0.221	0.798
6'	1.222	0.240	0.982
7	1.426	0.245	* 1.181
6	-0.871	0.240	* - 1.111
5	-0.726	0.221	- 0.947
4	-0.581	0.190	- 0.771
3	-0.436	0.148	- 0.584
2	-0.290	0.101	- 0.391
1	-0.145	0.051	- 0.196
	Σ plus stresses		4.508
	Σ minus "		- 4.000
	Summary		0.508

Final Influence surface of Vertical members.

		U0-L0			U1-M1			U2-M2	
Load on	So	SaH	S	So	SaH	S	So	SaH	S
1'	-0.071	-0.143	0.072	-0.077	-0.125	0.048	-0.078	-0.101	0.023
2'	-0.143	-0.283	0.140	-0.154	-0.246	0.092	-0.156	-0.199	0.043
3'	-0.214	-0.413	0.199	-0.230	-0.360	0.130	-0.234	-0.290	0.056
4'	-0.286	-0.529	0.243	-0.307	-0.461	* 0.154	-0.312	-0.372	0.060
5'	-0.357	-0.615	* 0.258	-0.384	-0.536	0.152	-0.390	-0.432	0.042
6'	-0.428	-0.670	0.242	-0.461	-0.584	0.123	-0.467	-0.471	0.004
7	-0.500	-0.684	0.184	-0.538	-0.596	0.058	-0.545	-0.481	-0.064
6	-0.571	-0.670	0.099	-0.614	-0.584	- 0.030	-0.623	-0.471	-0.152
5	-0.643	-0.615	- 0.028	-0.691	-0.536	- 0.155	-0.701	-0.432	-0.269
4	-0.714	-0.529	-0.185	-0.768	-0.461	- 0.307	-0.779	-0.372	-0.407
3	-0.785	-0.413	-0.372	-0.845	-0.360	- 0.485	-0.857	-0.290	* - 0.567
2	-0.857	-0.283	-0.574	-0.922	-0.246	* - 0.676	0.065	-0.199	* 0.264
1	-0.928	-0.143	* - 0.785	0.001	-0.125	0.126	0.032	-0.101	0.133
	Σ Plus stresses		1.437			0.883			0.625
	Σ Minus "		- 1.944			-1.653			- 1.459
	Summary		- 0.507			-0.770			- 0.834

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Design of Kisogawa-Bashi for Gifu-ken.

	U3-M3			U4-M4			U5-M5		
Load on	S_o	S_aH	S	S_o	S_aH	S	S_o	S_aH	S
1'	-0.074	-0.071	-0.003	-0.063	-0.034	-0.029	-0.043	0.009	-0.052
2'	-0.147	-0.139	-0.008	-0.125	-0.067	-0.058	-0.087	0.017	-0.104
3'	-0.221	-0.203	-0.018	-0.188	-0.098	-0.090	-0.130	0.025	-0.155
4'	-0.294	-0.260	-0.034	-0.251	-0.125	-0.126	-0.174	0.032	-0.206
5'	-0.368	-0.303	-0.065	-0.314	-0.146	-0.168	-0.217	0.037	-0.254
6'	-0.442	-0.330	-0.112	-0.376	-0.159	-0.217	-0.260	0.040	-0.300
7	-0.515	-0.337	-0.178	-0.439	-0.162	-0.277	-0.304	0.041	-0.345
6	-0.589	-0.330	-0.259	-0.502	-0.159	-0.343	-0.347	0.040	* -0.387
5	-0.662	-0.303	-0.359	-0.564	-0.146	* -0.418	0.609	0.037	* 0.572
4	-0.736	-0.260	* -0.476	0.373	-0.125	* 0.498	0.487	0.032	0.455
3	0.190	-0.203	* 0.393	0.280	-0.098	0.378	0.365	0.025	0.340
2	0.127	-0.139	0.266	0.187	-0.067	0.254	0.244	0.017	0.227
1	0.063	-0.071	0.134	0.093	-0.034	0.127	0.122	0.009	0.113
	Σ Plus stresses		0.793			1.257			1.707
	Σ minus 〃		-1.512			-1.726			-1.803
	Summary		-0.719			-0.469			-0.096

	U6-M6			U7-M7			Final Influence surface of Tie Lo-Lo'		
Load on	S_o	S_aH	S	S_o	S_aH	S	S_o	S_aH	S
1'	-0.017	0.053	-0.070	0.077	0.076	0.001	0.000	-0.210	0.210
2'	-0.033	0.105	-0.138	0.154	0.151	0.003	〃	-0.414	0.414
3'	-0.050	0.154	-0.204	0.231	0.220	0.011	〃	-0.605	0.605
4'	-0.066	0.197	-0.263	0.308	0.282	0.026	〃	-0.774	0.774
5'	-0.083	0.229	-0.312	0.385	0.328	0.057	〃	-0.901	0.901
6'	-0.100	0.249	-0.349	0.462	0.357	0.105	〃	-0.981	0.981
7	-0.116	0.255	* -0.371	0.539	0.365	* 0.174	〃	-1.002	* 1.002
6	0.867	0.249	* 0.618	0.462	0.357	0.105	〃	-0.981	0.981
5	0.723	0.229	0.494	0.385	0.328	0.057	〃	-0.901	0.901
4	0.578	0.197	0.381	0.308	0.282	0.026	〃	-0.774	0.774
3	0.434	0.154	0.280	0.231	0.220	0.011	〃	-0.605	0.605
2	0.289	0.105	0.184	0.154	0.151	0.003	〃	-0.414	0.414
1	0.145	0.053	0.092	0.077	0.076	0.001	〃	-0.210	0.210
	Σ Plus stresses		2.049			0.580			8.772
	Σ minus 〃		-1.707			-0.000			-0.000
	Summary		0.342			0.580			8.772

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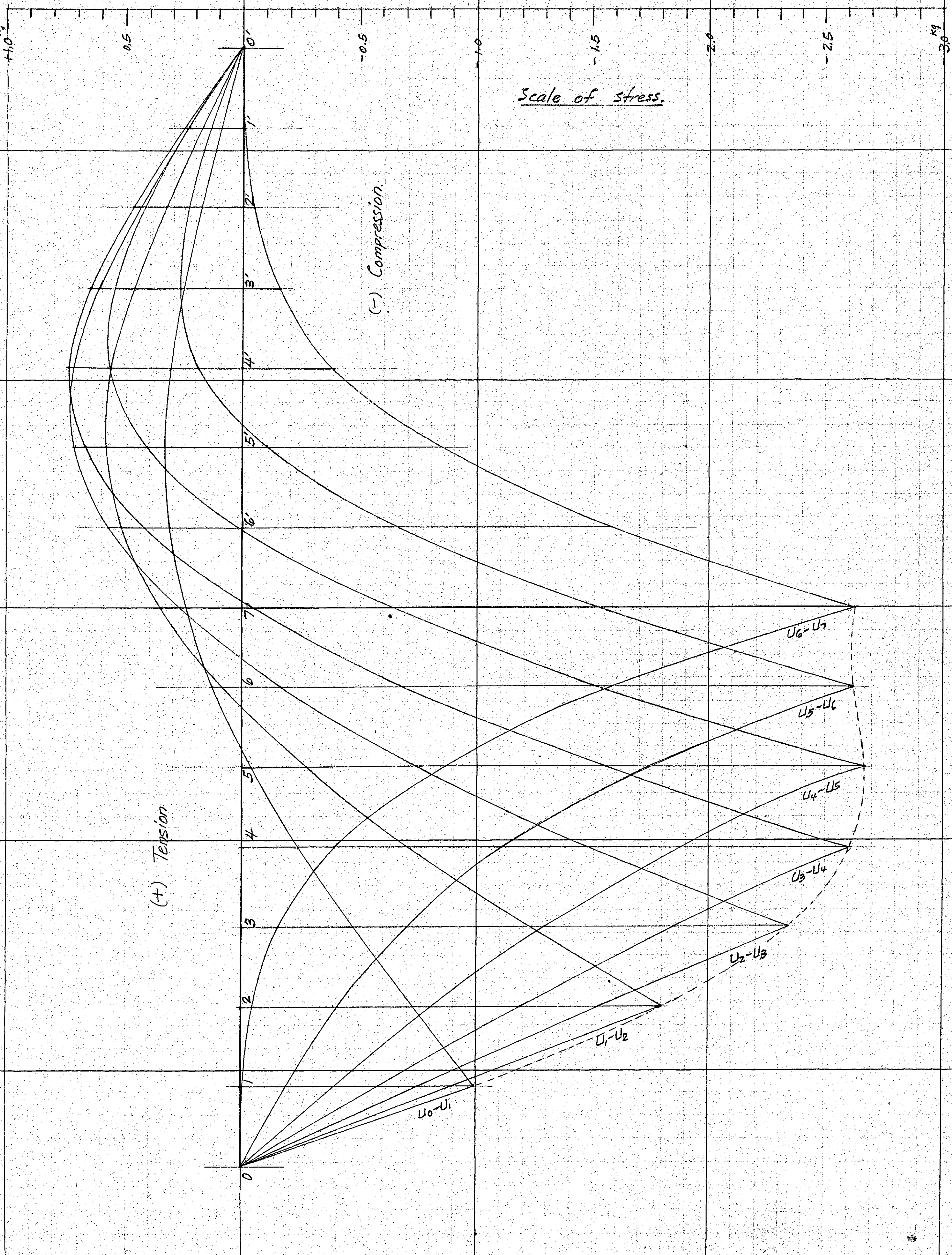
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.
Diagrams of Final Influence surfaces.
Final Influence Diagram of Top chords.



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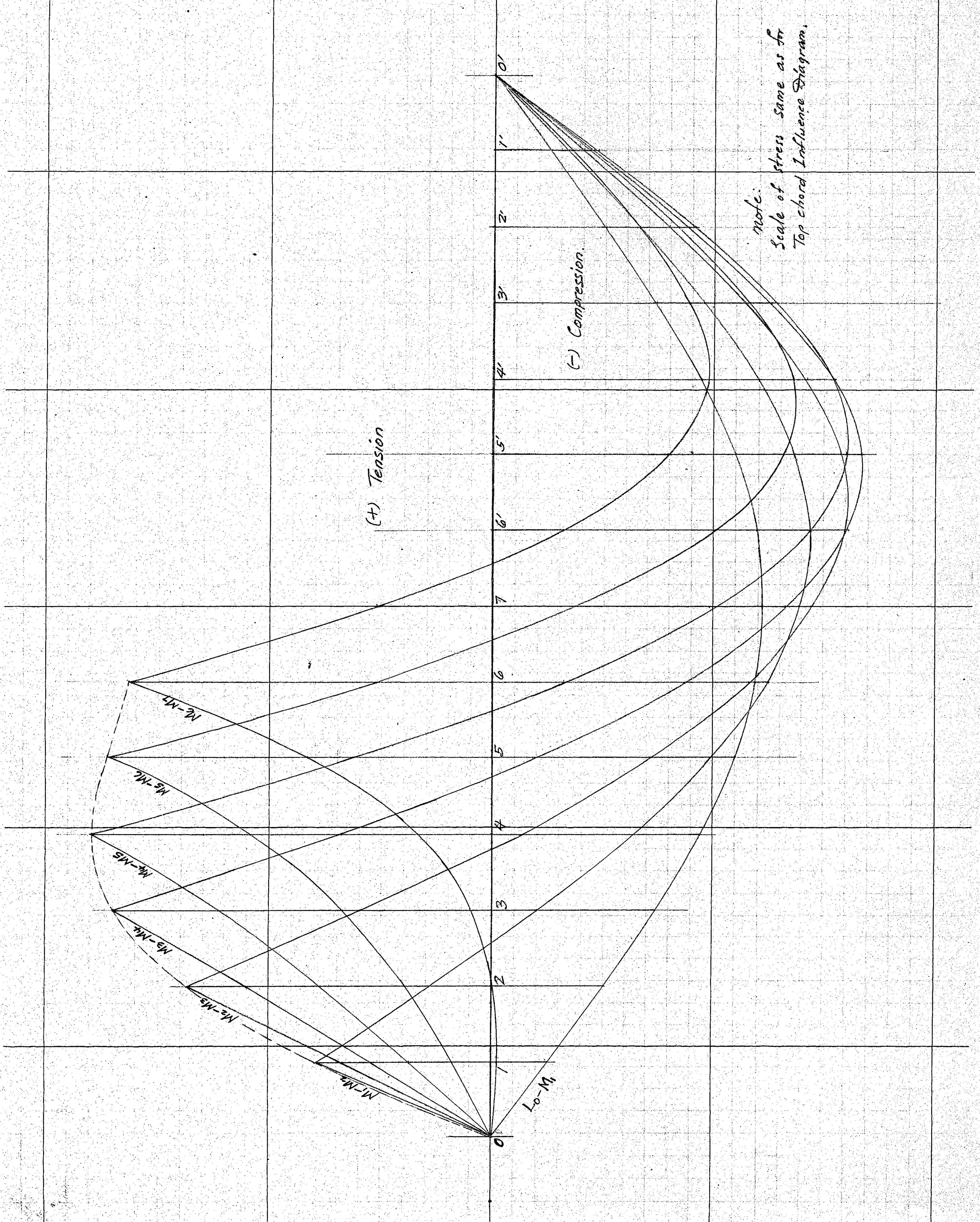
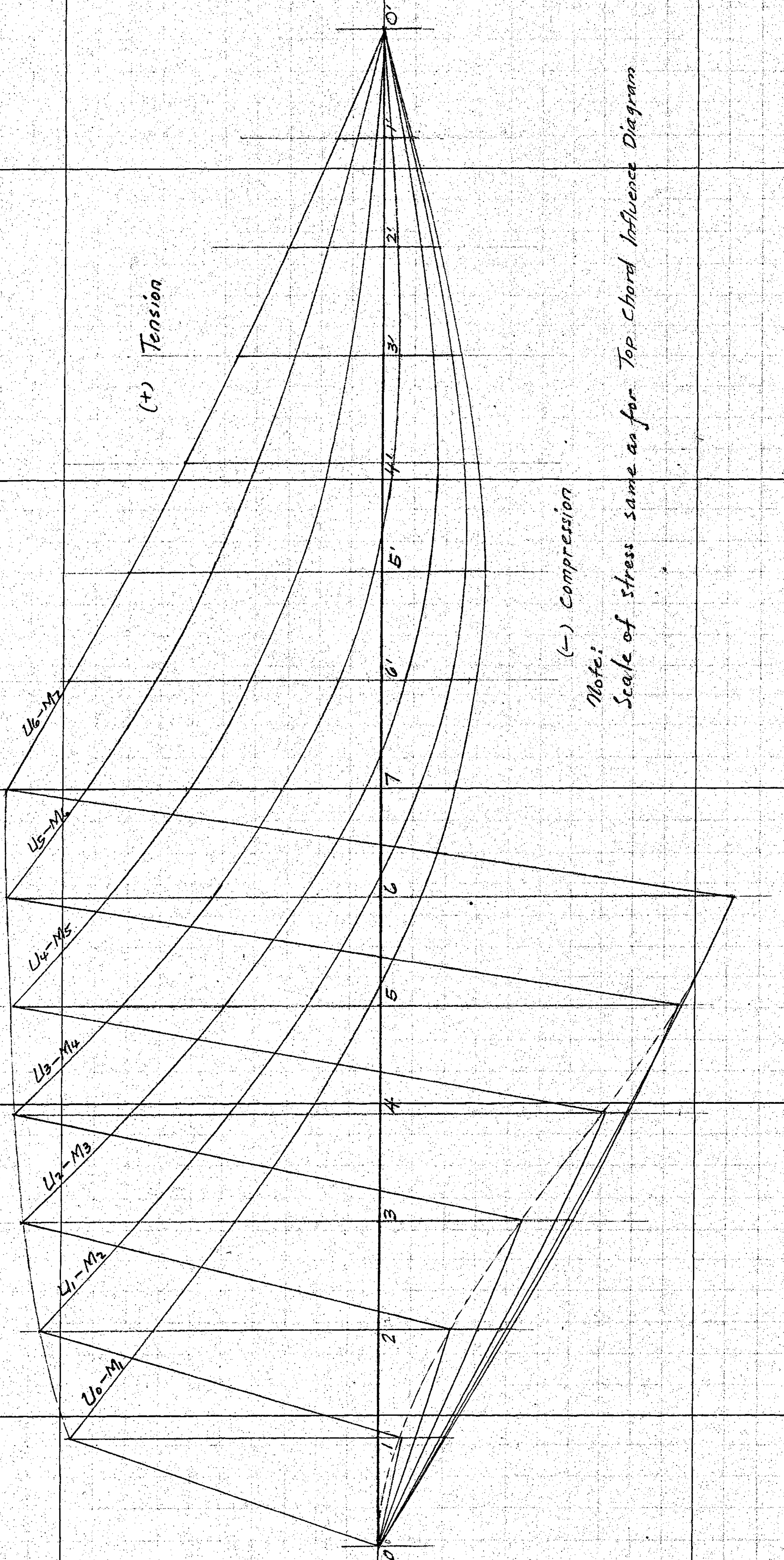
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-ken.
Final Influence Diagram of Middle chords.



Design of Kisogawa-Bashi for Gifu-ken.

Final Influence Diagram of Diagonals.



(+) Tension

(−) Compression

Note:
Scale of stress same as for Top Chord Influence Diagram

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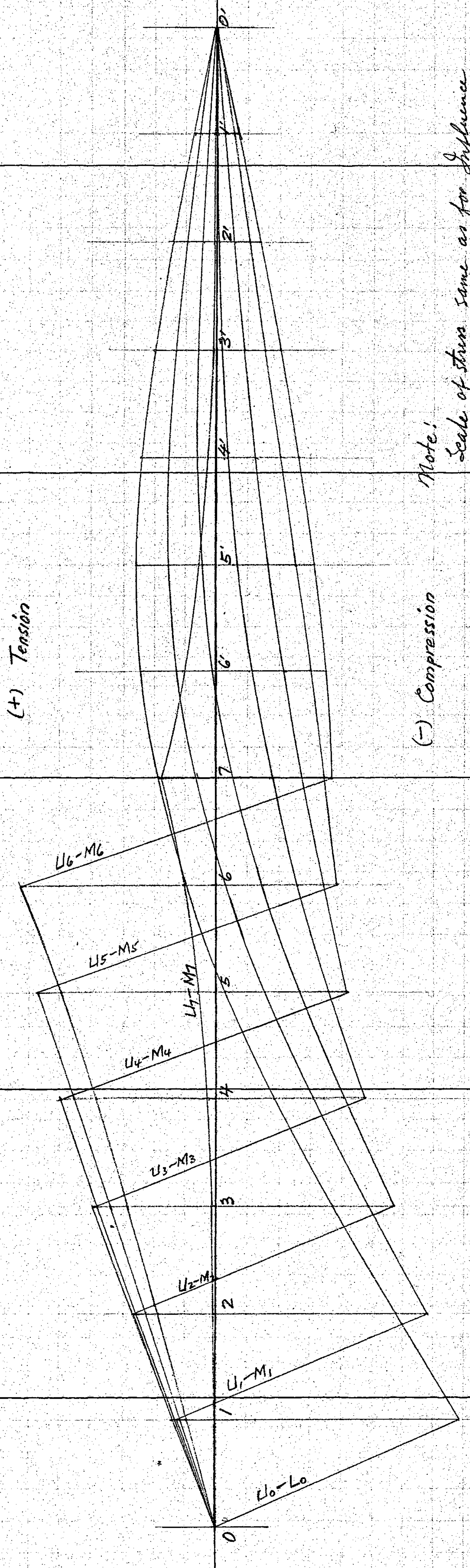
Design of Kisogawa-Bashi for Gifu-Ken

Final Influence Diagram of Vertical members.



Note!
Scale of stress same as for Influence Diagram of Top chords.

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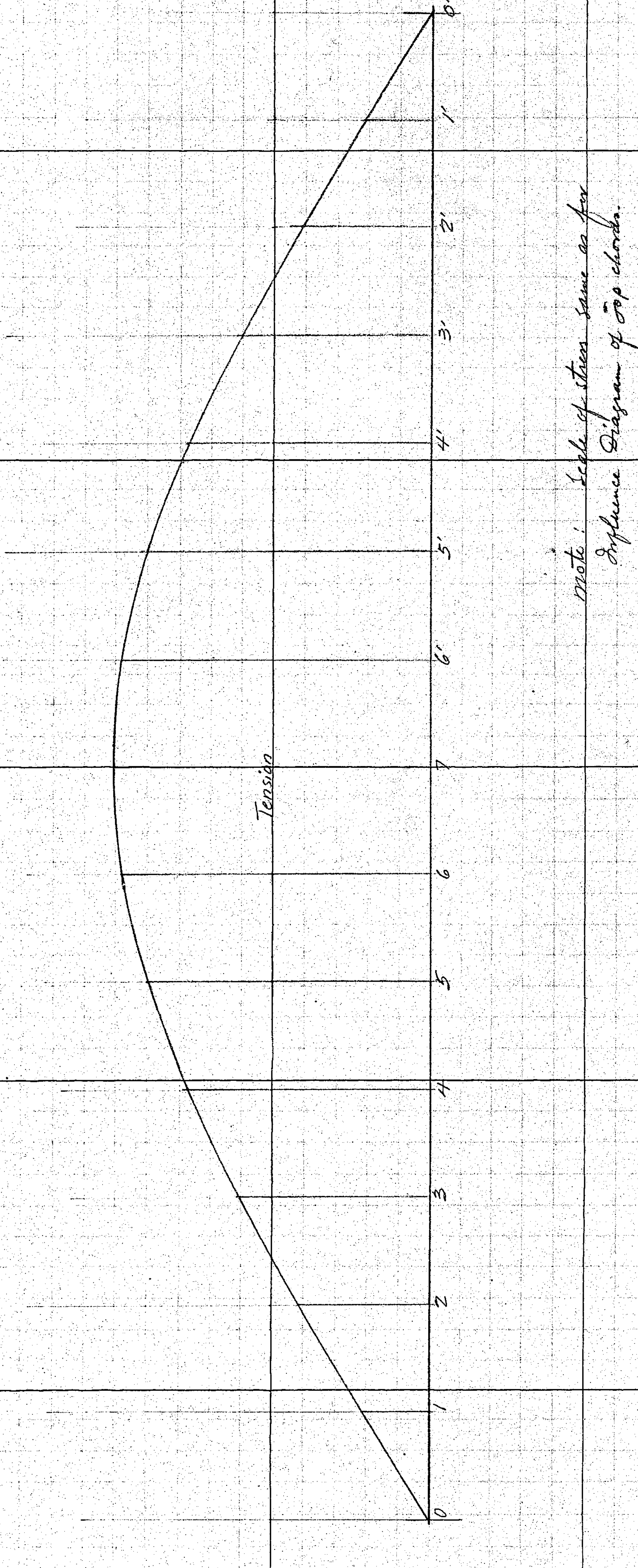
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-ken.

Final Influence line of stress in Tie.



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CALCULATIONS FOR

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Design of Kisogawa-Bashi for Gifu-Ken.

Dead Load and Live Load stresses of Truss members.

Members	Summary of Influence Ordinates (+) Ordinates	(−) Ordinates	Summary	Maximum Ordinates	D.L. Stresses 21900 kg	L.L. Stresses 8880 kg	5800 kg	Total L.L. Stresses	Maximum Stresses
Upper Chord members.									
U0-U1	1.828	-2.479	-0.651	-1.000	-14,250	-22,000	-5,800	-27,800	-42,050
U1-U2	3.068	-4.931	-1.863	-1.805	-40,800	-43,800	-10,470	-54,270	-95,070
U2-U3	3.565	-7.067	-3.502	-2.348	-76,700	-62,750	-13,620	-76,370	-153,070
U3-U4	3.219	-8.561	-5.342	-2.607	-117,100	-76,000	-15,120	-91,120	-208,220
U4-U5	2.176	-9.224	-7.048	-2.665	-154,400	-81,900	-15,450	-97,350	-251,750
U5-U6	0.834	-9.119	-8.285	-2.625	-181,500	-81,000	-15,230	-96,230	-277,730
U6-U7	0.000	-8.722	-8.722	-2.628	-191,000	-77,500	-15,240	-92,740	-283,740
Middle Chord members.									
L0-M1	0.000	-10.621	-10.621	-1.213	-232,500	-94,300	-7,040	-101,340	-333,840
M1-M2	1.070	-10.382	-9.312	-1.432	-204,000	-92,200	-8,310	-100,510	-304,510
M2-M3	2.654	-10.293	-7.639	-1.588	-167,300	-91,400	-9,220	-100,620	-267,920
M3-M4	4.048	-9.744	-5.696	-1.664	-124,800	-86,500	-9,650	-96,150	-220,950
M4-M5	4.811	-8.482	-3.671	-1.596	-80,400	-75,350	-9,260	-84,610	-165,010
M5-M6	4.607	-6.441	-1.834	-1.352 1.743	-40,150 -⅓×40150	-57,200 40900	-7,840 10,110	-65,040 51,010	-105,190 24,240
M6-M7	3.427	-3.964	-0.537	-0.964 1.632	-11,770 -⅓×11,770	-35,200 30,430	-5,590 9,460	-40,790 39,890	-52,560 32,040
Diagonal members.									
U0-M1	2.407	-1.773	0.634	0.971 -0.318	13,880 ⅔×13,880	21380 -15,750	5,630 -1,850	27,010 -17,600	40,890 - 8350
U1-M2	2.579	-1.371	1.208	1.073	26,460	22,900	6,230	29,130	55,590
U2-M3	2.740	-1.066	1.674	1.132	36,640	24,340	6,560	30,900	67,540
U3-M4	2.996	-1.072	1.924	1.152	42,120	26,600	6,680	33,280	75,400
U4-M5	3.681	-1.841	1.840	1.162	40,280	32,700	6,740	39,440	79,720
U5-M6	4.324	-2.959	1.365	1.175 -0.942	29,890 ⅔×29,890	38,400 -26,260	6,810 -5,460	45,210 -31,720	75,100 - 11,790
U6-M7	4.508	-4.000	0.508	1.181 -1.111	11,130 ⅔×11,130	40,030 -35,520	6,850 -6,450	46,880 -41,970	58,010 - 34,550

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-ken.

Members	Summary of Influence ordinates			maximum	D.L. Stresses	L.L. Stresses		Total L.L.	Maximum
	(+) Ordinates	(-) Ordinates	Summary	Ordinates	21900 kg	8880 kg	5800 kg	Stresses	Stresses
Vertical members									
U₀-L₀	1.437	-1.944	-0.507	-0.785	-11,100	-17,270	-4,550	-21,820	-32,920
				0.258	-2/3×11,100	12,770	1,500	14,270	6,870
U₁-M₁	0.883	-1.653	-0.770	-0.676	-16,860	-14,680	-3,920	-18,600	-35,460
U₂-M₂	0.625	-1.459	-0.834	-0.567	-18,250	-12,950	-3,290	-16,240	-34,490
U₃-M₃	0.793	-1.512	-0.719	-0.476	-15,750	-13,420	-2,760	-16,180	-31,930
U₄-M₄	1.257	-1.726	-0.469	-0.418	-10,270	-15,330	-2,430	-17,760	-28,030
				0.498	-2/3×10,270	11,160	2,890	14,050	7,200
U₅-M₅	1.707	-1.803	-0.096	-0.387	-2,100	-16,020	-2,250	-18,270	-20,370
				0.572	-2/3×2,100	15,160	3,320	18,480	17,080
U₆-M₆	2.049	-1.707	0.342	0.618	7,490	18,200	3,580	21,780	29,270
				-0.371	2/3×7,490	-15,150	-2,150	-17,300	-12,310
U₇-M₇	0.580	-0.000	0.580	0.174	12,700	5,150	1,010	6,160	18,860
Ties									
L₀-L₀′	8.772	-0.000	8.772	1.002	192,100	77,900	5,810	83,710	275,810

max. stress in Hangers. Same as for Preliminary design (see on page 43).

Dead Load	21,900
Live Load	16,680
	38,580 kg

max. load on shoe.

Dead Load	153,300	
weight of shoe + exp. jt say	1140	
		154,440
Live Load		67,960
		222,400 kg on one shoe.

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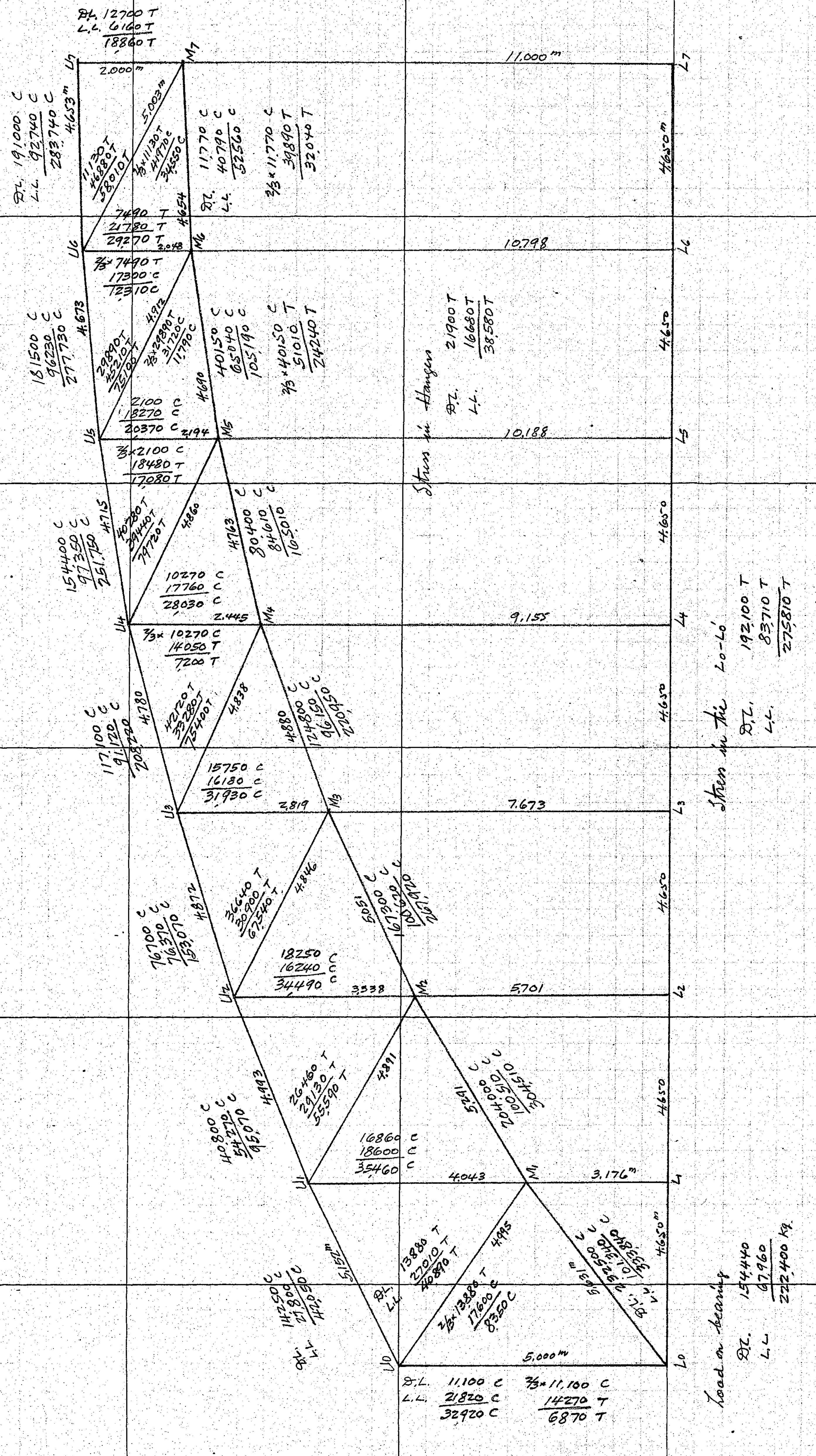
Design of Kisogawa-Bashi for Gifu-ken.

Stress Diagram.



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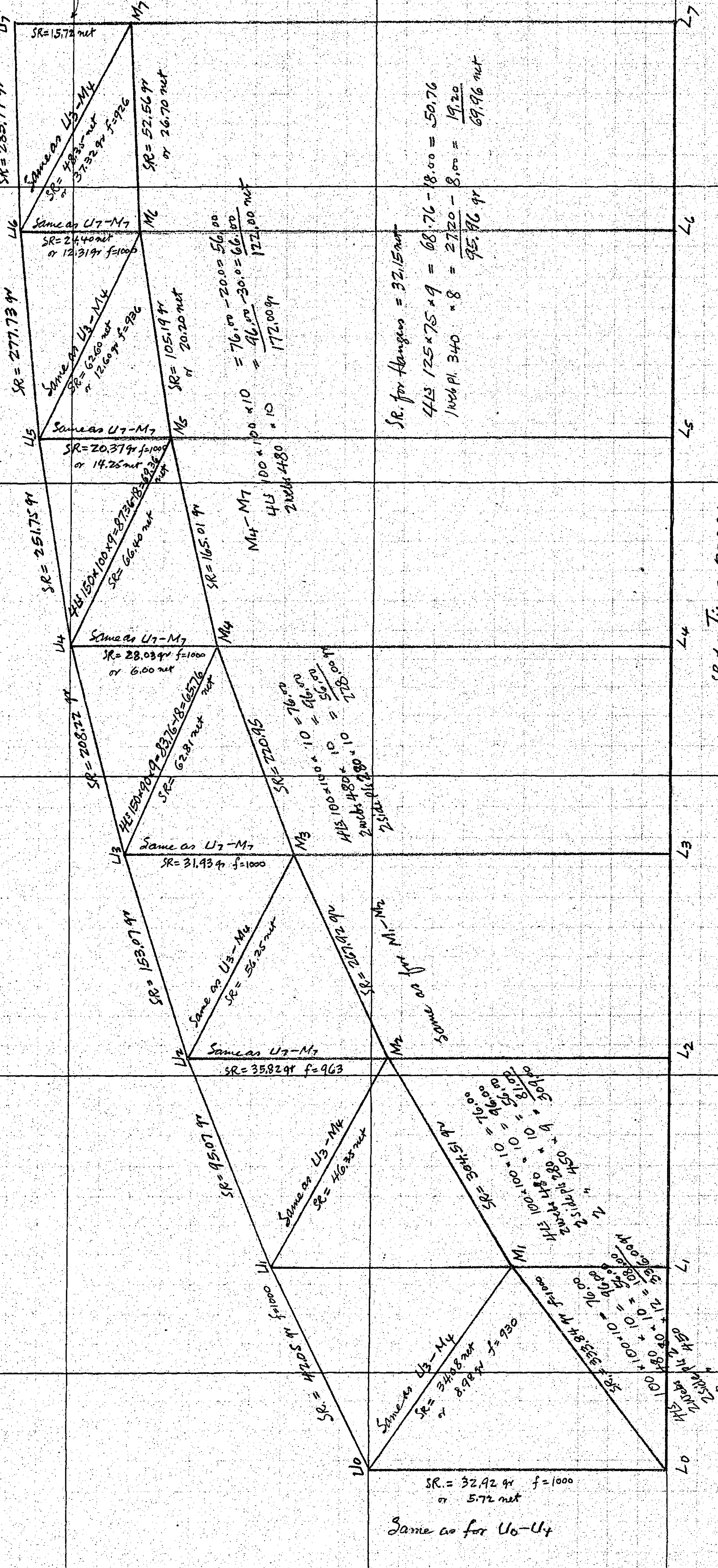
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

Sections of truss members.

4Ls 125×75×9 = 68.76 − 18.00 = 50.76
1 web 340 × 8 = 27.20 − 8.00 = 19.20
95.96 gr 69.96 net



U4-U7.
1 Cov. Pl. 600×12 = 72.00
4Ls 100×100×10 = 76.00
2 web Pls. 480×10 = 96.00
2 side Pls. 280×10 = 56.00
300.00 cm² gr.

U0-U4.
1 Cov. Pl. 600×9 = 54.00
4Ls 100×100×10 = 76.00
2 web Pls. 480×10 = 96.00
226.00 gross

Note: Revised to 600×12

M4-M7
4Ls 100×100×10 = 76.00 − 20.00 = 56.00
2 webs 480 × 10 = 96.00 − 30.0 = 66.00
172.00 gr 122.00 net

M2-M4
4Ls 100×100×10 = 76.00
2 webs 480×10 = 96.00
2 side Pls 280×10 = 56.00
228.00 gr

M1-M2
4Ls 100×100×10 = 76.00
2 webs 480 × 10 = 96.00
2 side pls 280 × 9 = 81.00
450 " 2 "
309.00

M0-M1
4Ls 100×100×10 = 76.00
2 webs 480×10 = 96.00
2 side pls 480 × 12 = 108.00
450 " 2 "
280.00

SR. for Hangers = 32.15 net
4Ls 125×75×9 = 68.76 − 18.00 = 50.76
1 web pl. 340 × 8 = 27.20 − 8.00 = 19.20
95.96 gr 69.96 net

SR. for Tie = 229.84 net
2Ls 150×100×15 = 70.50 − 15.00 = 55.50
2 webs 500 × 10 = 100.00 − 20.00 = 80.00
2 side Pls 400 × 15 = 120.00 − 22.50 = 97.50
290.50 gr 233.00 net

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CHECKED BY M. Iida DATE 8-8-4 **PAGE NO** 70

CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

Approximate weight of main truss.

members	gross sectional area in cm^2	length in m.	req'd. no.	volume in cm^2 m.	remarks.
U_0-U_4	226.00	19.797	2	8,950	Upper chord.
U_4-U_7	300.00	14.041	2	8,420	〃
L_0-M_1	336.00	5.631	2	3,785	Middle chord
M_1-M_3	309.00	10.342	2	6,390	〃
M_3-M_4	228.00	4.880	2	2,225	〃
M_4-M_7	172.00	14.107	2	4,855	〃
L_0-L_0'	290.50	65.100	1	18,920	Tie
$U_7-M_7-L_7$	95.96	13.000	1	1,248	Verticals and Hangers
$U_6-M_6-L_6$	〃	12.846	2	2,465	〃 〃
$U_5-M_5-L_5$	〃	12.382	2	2,376	〃 〃
$U_4-M_4-L_4$	〃	11.600	2	2,226	〃 〃
$U_3-M_3-L_3$	〃	10.492	2	2,015	〃 〃
$U_2-M_2-L_2$	〃	9.039	2	1,735	〃 〃
$U_1-M_1-L_1$	say 115.00	7.219	2	1,660	〃 〃
U_0-L_0	226.00	5.000	2	2,260	〃 〃
U_0-M_1	83.76	4.995	2	837	Diagonals
U_1-M_2	〃	4.891	2	819	〃
U_2-M_3	〃	4.846	2	812	〃
U_3-M_4	〃	4.838	2	810	〃
U_4-M_5	87.36	4.860	2	849	〃
U_5-M_6	83.76	4.912	2	823	〃
U_6-M_7	〃	5.003	2	838	〃

75,318 @ 0.785 = 59,100
Details say 45 % = 26600
85,700 kg for one truss

Approximate weight of structural steel in one span.

Stringers		$6 @ 62^{kg} \times$	65.90	=	24,500
Floor beams,	intermediate	13 @	$2,240^{kg}$	=	29,100
〃	ends.	2 @	2,105	=	4,210
Lower lateral bracings		65.1 @	120	=	7,810
Upper lateral bracings		55.8 @	103	=	5,760
Sway bracings		5 @	2,200	=	11,000
Sway struts		6 @	700	=	4,200
Portal bracings		2 @	2,300	=	4,600
Main trusses		2 @	85,700	=	171,400
Expansion joint say				=	1,000
Shoes		4 @	1,000	=	4,000
					267,580 kg.
				call this	268.0 kg tons

Summary of structural weight for the entire bridge
7 spans @ 268.00 = 1876 kg tons.

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

Deflection of Main truss at center of span.

General formula of deflection

$$\Delta = \Sigma \frac{S_1 L}{EA} S = \frac{1}{E} \Sigma \frac{S_1 L}{A} S. \text{ for } E \text{ constant.}$$

where

Δ = Deflection at any panel point in cm;

S = Stress in each member in kg;

S_1 = Stress in each member due to unit load on the panel point at which deflection is desired in the direction of the deflection in kg;

L = Length of each member in cm;

A = Gross sectional area of each member in cm²;

E = modulus of elasticity of structural steel in kg/cm² = 2,100,000.

members	Length L	Area A	Unity S_1	$\frac{S_1 L}{A}$	D.L. Stress S_D	L.L. Stress		Total L.L. Stress S_L	$\frac{S_1 L}{A} S_D$	$\frac{S_1 L}{A} S_L$
						8880	5800			
U_0-U_1	515.2	226.00	0.235	0.536	-14,250	-5,780	1,360	-4,420	-7,640	-2,370
U_1-U_2	499.3	〃	0.342	0.756	-40,800	-16,550	1,980	-14,570	-30,840	-11,020
U_2-U_3	487.2	〃	0.265	0.571	-76,700	-31,100	1,540	-29,560	-43,800	-16,880
U_3-U_4	478.0	〃	-0.054	-0.114	-117,100	-47,450	-310	-47,760	13,350	5,440
U_4-U_5	471.5	300.00	-0.655	-1,030	-154,400	-62,600	-3,800	-66,400	159,000	68,400
U_5-U_6	467.3	〃	-1,536	-2.392	-181,500	-73,600	-8,910	-82,510	434,000	197,300
U_6-U_7	465.3	〃	-2,628	-4.078	-191,000	-77,410	-15,240	-92,650	779,000	377,500
L_0-M_1	563.1	336.00	-1.213	-2.035	-232,500	-94,300	-7,040	-101,340	473,000	206,200
M_1-M_2	529.1	309.00	-1.381	-2.365	-204,000	-82,700	-8,010	-90,710	482,500	214,500
M_2-M_3	505.1	〃	-1.434	-2.345	-167,300	-67,800	-8,320	-76,120	392,000	178,500
M_3-M_4	488.0	228.00	-1,317	-2.818	-124,800	-50,600	-7,640	-58,240	352,000	164,100
M_4-M_5	476.3	172.00	-0.974	-2.698	-80,400	-32,600	-5,650	-38,250	217,000	103,300
M_5-M_6	469.0	〃	-0.359	-0.979	-40,150	-16,280	-2,080	-18,360	39,300	17,970
M_6-M_7	465.4	〃	0.526	1.423	-11,770	-4,770	3,050	-1,720	-16,760	-2,450
U_0-M_1	499.5	83.76	-0.228	-1.360	13,880	5,630	-1,320	4,310	-18,880	-5,860
U_1-M_2	489.1	〃	-0.112	-0.654	26,460	10,730	-650	10,080	-17,300	-6,600
U_2-M_3	484.6	〃	0.068	0.394	36,640	14,850	390	15,240	14,440	6,000
U_3-M_4	483.8	〃	0.317	1.832	42,120	17,080	1,840	18,920	77,150	34,650
U_4-M_5	486.0	87.36	0.620	3.450	40,280	16,330	3,600	19,930	139,000	68,800
U_5-M_6	491.2	83.76	0.933	5.472	29,890	12,120	5,410	17,530	163,600	96,000
U_6-M_7	500.3	〃	1.181	7.060	11,130	4,510	6,850	11,360	78,600	80,200
U_0-L_0	500.0	226.00	0.184	0.407	-11,100	-4,500	1,070	-3,430	-4,520	-1,400
U_1-M_1	404.3	115.00	0.058	0.204	-16,860	-6,840	340	-6,500	-3,440	-1,330
U_2-M_2	333.8	95.96	-0.064	-0.223	-18,250	-7,400	-370	-7,770	4,070	1,730
U_3-M_3	281.9	〃	-0.178	-0.523	-15,750	-6,380	-1,030	-7,410	8,240	3,880
U_4-M_4	244.5	〃	-0.277	-0.706	-10,270	-4,160	-1,610	-5,770	7,250	4,080
U_5-M_5	219.4	〃	-0.345	-0.789	-2,100	-850	-2,000	-2,850	1,660	2,250
U_6-M_6	204.8	〃	-0.371	-0.791	7,490	3,040	-2,150	890	-5,930	-700
U_7-M_7	200.0	$\frac{1}{2}\times 95.96$	$\frac{1}{2}\times 0.174$	0.363	$\frac{1}{2}\times 12,700$	$\frac{1}{2}\times 5,150$	$\frac{1}{2}\times 1,010$	3,080	2,310	1,120
L_0-L_7	3,255.0	290.5	1.002	11.225	192,100	77,870	5,810	83,680	2,157,000	939,500
M_7-L_7	1100.0	$\frac{1}{2}\times 95.96$	$\frac{1}{2}\times 1.000$	11.460	$\frac{1}{2}\times 21,900$	$\frac{1}{2}\times 8,880$	$\frac{1}{2}\times 5,800$	7,340	125,500	84,100

$\Sigma \frac{S_1 L}{A} S$ = 5,970,860 2,806,910

Summary for the entire span = 11,941,720 5,613,820

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CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

Dead Load Deflection $= \frac{11941720}{2100000} = 5.69$

Live Load Deflection $= \frac{5613820}{2100000} = 2.68$

maximum Deflection $= 8.37$ cm   Deflection ratio $= \frac{8.37}{6510} = \frac{1}{780}$

A camber of $5.69 + \frac{2.68}{2} = 7.0$ cm to be given to truss

Deflection increments due to several parts of Dead Load.

Structural steel $5.69 \div 9420 \times 4080 = 2.46$ cm
floor slab and copings 〃 ÷ 〃 × 3810 = 2.30 〃
pavement 〃 ÷ 〃 × 1290 = 0.78 〃
handrails 〃 ÷ 〃 × 240 = 0.15 〃
5.69 〃

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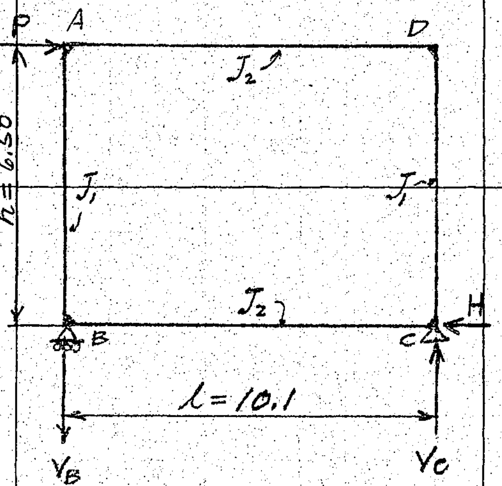
CALCULATIONS FOR

Design of Kisogawa-Bashi for Gifu-Ken.

Design of Portal Bracing.

Let us design the portal bracing as a Rahmen constructed by portal bracing, verticals, and floor beam.



Formulas for moments and reactions. (Referred to Kleinlogels' "Rahmen-formeln" on page 288)

moments.

$$M_A = M_C = Ph\left[\frac{3K+1}{4(3K+1)}\right] = \frac{Ph}{4}$$

$$M_B = M_D = -Ph\left[\frac{3K+1}{4(3K+1)}\right] = -\frac{Ph}{4}$$

Reactions. $V_B = V_C = \frac{Ph}{l}$

$H = P$

where $K = \frac{J_2}{J_1}\cdot\frac{h}{l}$.

J_2 = moments of inertia of portal bracing and floor beam being assumed to be equal to each other.

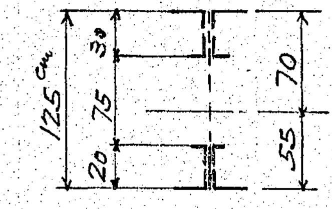
J_1 = moment of inertia of vertical member.

h assumed 6.50 meters

l = span length = 10.10 meters.

Assumed sections and moments of inertia.

Portal bracing



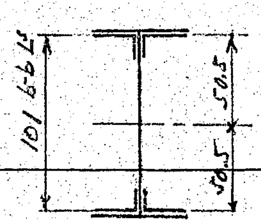
Center of gravity

4Ls 150×90×9 =	83.76	× 62.5 =	5235
4Ls 90×75×9 =	56.16	× 67.5 =	3790
1Pl. 190 ×9 =	17.10	× 115.0 =	1965
	157.02 cm²	70.0 cm	10990

moment of inertia.

2Ls 150×90×9 =	$41.88 \times 68.02^2 + 2\times129$ =	194000
2Ls 〃 =	$41.88 \times 53.02^2 + 2\times129$ =	118000
2Ls 90×75×9 =	$28.08 \times 42.74^2 + 2\times109$ =	51500
2Ls 〃 =	$28.08 \times 37.74^2 + 2\times109$ =	40200
1Pl. 190 ×9 =	$17.10 \times 45.00^2 + \frac{0.9\times19^3}{12}$ =	35100
		438800 cm⁴

Floor beam.



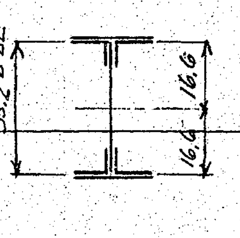
moment of inertia.

4Ls 125×90×10 =	$82.00 \times 48.28^2 + 4\times138$ =	191600
1web 1000 ×9	$\frac{0.9\times100^3}{12}$ =	75000
2Cov.pls 300 ×10 =	60.00×51.0^2 =	156000
		422600 cm⁴

Above two moments of inertia are very nearly equal.
Use average of the two as the value of J_2.

$$\begin{array}{r} 438800 \\ \underline{422600} \\ 861400 \end{array} \div 2 = 430700 \text{ cm}^4 = J_2$$

Verticals.



4Ls 125×90×9 =	74.16	$\times 14.42^2 + 125\times4$ =	15900
1Web 322 ×9 =	28.98	$0.9\times32.2^3\div12$ =	2500
2Cov.pls 265 ×9 =	47.70	$\times 17.05^2$ =	13850
	150.84		J_1 = 32250 cm⁴

Constant $K = \frac{J_2}{J_1}\cdot\frac{h}{l} = \frac{430700}{32250}\cdot\frac{650}{1010} = 8.60$

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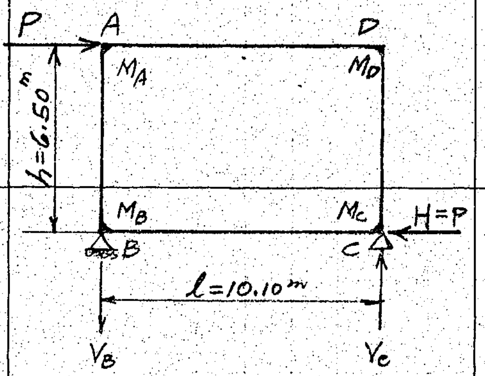
Design of Kisogawa-Bashi for Gifu-Ken

Stresses due to Horizontal thrusts.

Horizontal thrust on portal bracing.

Wind thrust = 6@1070 = 6420 kg see page 10

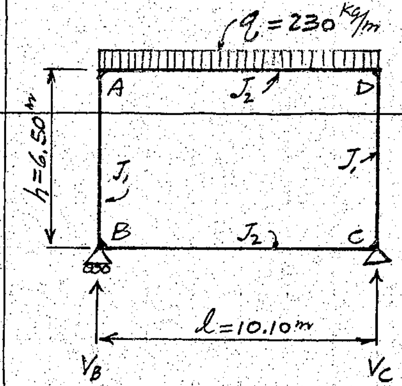
Seismic " = 6@2470 = 14,820 kg "



		Wind stress $P=6420$ kg	Seismic stress $P=14820$ kg
$M_A = M_C = \frac{1}{4}Ph = 1.625P$		10,430 kgm	24,080 kgm
$M_B = M_D = -\frac{1}{4}Ph = -1.625P$		-10,430 "	-24,080 "
$V_B = V_C = \frac{Ph}{l} = 0.643P$		4130 kg	9520 kg
$H = P$		6420 "	14,820 "

Stresses due to own weight of portal bracing.

weight of bracing per lin m assumed 230 kg.



$$M_A = -\frac{ql^2(2K+3)}{12(K^2+4K+3)} = M_D = -\frac{230\times10.1^2\times20.20}{12\times111.36} = -355 \text{ kgm}$$

$$M_B = \frac{ql^2K}{12(K^2+4K+3)} = M_C = \frac{230\times10.1^2\times8.60}{12\times111.36} = 150 \text{ "}$$

$$V_B = V_C = \frac{ql}{2} = \frac{230\times10.1}{2} = 1160 \text{ kg}$$

where $K = 8.60$

$2K+3 = 20.20$

$K^2+4K+3 = 111.36$

Stresses due to dead and live loads on floor beam.

max moment at center of floor beam

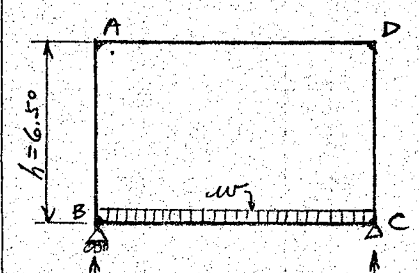
Dead load moment = 35680 kgm

Live load " = 45160 " $m = \frac{wl^2}{8}$ ∴ $w = \frac{8m}{l^2}$

Equivalent uniform loads

Dead load $w_D = \frac{8m}{l^2} = \frac{8\times35680}{10.1^2} = 2800$ kg/lin m.

Live load $w_L = \frac{8m}{l^2} = \frac{8\times45160}{10.1^2} = 3540$ " $\frac{3540}{2800} = 1.265$



$$M_A = -\frac{wl^2K}{12(K^2+4K+3)} = M_D,$$

$$V_B = V_C = \frac{wl}{2}$$

$$M_B = \frac{wl^2(2K+3)}{12(K^2+4K+3)} = M_C,$$

Dead load stress

$$M_A = M_D = -\frac{2800\times10.10^2\times8.60}{12\times111.36} = -1840 \text{ kgm}$$

$$M_B = M_C = \frac{2800\times10.10^2\times20.20}{12\times111.36} = 4320 \text{ "}$$

$$V_B = V_C = \frac{2800\times10.10}{2} = 14,140 \text{ kg}$$

Live load stresses.

$M_A = M_D = -1.265\times1840 = -2,330$ kgm

$M_B = M_C = 1.265\times4320 = 5,460$ "

$V_B = V_C = 1.265\times14140 = 17,900$ kg

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Design of Kisogawa-Bashi for Gifu-Ken.

Summary of stresses in portal bracing.

	M_A	M_B	M_C	M_D	V_B	V_C
Own weight	- 355	150	150	- 355	1160	1160
D.L. on floor beam.	- 1840	4320	4320	-1840	14140	14140
Summary of D.L. stress	- 2195	4470	4470	- 2195	15,300	15,300
Live Load stress	- 2330	5460	5460	-2330	17900	17900
wind load stress	10430	-10430	10430	- 10430	-4130	4130
Seismic "	24080	-24080	24080	- 24080	-9520	9520
Summary for D.L. + L.L.	4525 kgm	9930 kgm	9930 kgm	4525 kgm	33,200 kg	33200 kg
D.L. + W.L.	8235	-5960	14900	- 12625	11170	19430
D.L. + S.L.	21,885	-19,610	28550	- 26275	5780	24820

Direct stresses on portal bracing
wind stress $6420 \div 2 = 3210$ kg C
seismic " $14820 \div 2 = 7410$ " C

max. stress in top chord of portal.

$\frac{26275}{2.30} = 11420$ kg T $\div 1920 = 5.95$ cm² net

$\frac{21885}{2.30} = 9520$ kg C
7410
16930 $\div 896 = 18.90$ cm² gr

least radius of gyr. = 4.43 cm
$l/r = 505/4.43 = 114$
$f = 1500(1-0.0035 \times 114) = 560$
$560 \times 1.6 = 896$ kg/cm²

Use
2L $150 \times 90 \times 9 = 41.88 - 4.5 = 37.38$
2L $90 \times 75 \times 9 = 28.08 - 4.5 = 23.58$
69.96 gr 60.96 net

max. stress in bottom chord of portal.

$\frac{21885}{1.75} = 12,500$ kg T $\div 1920 = 6.51$ cm² net

$\frac{26275}{1.75} = 15030$
$7410 \times 1.15 = 8540$
23570 kg C $\div 896 =$ (approx.) 26.30 cm² gr.

Use
2L $150 \times 90 \times 9 = 41.88 - 4.5 = 37.38$
2L $90 \times 75 \times 9 = 28.08 - 4.5 = 23.58$
1Pl. $190 \times 9. = 17.10 - 4.5 = 12.60$
87.06 gr 73.56 net

max. stress in vertical.

$M_C = 28550$ kgm

Stress $= \frac{28550 \times 100 \times 17.5}{32250} = 1550$ kg/cm² T $< 1200 \times 1.6 = 1920$

Direct comp. $= \frac{9520}{150.84} = 63$

Bending stress $= 1550$
1613 kg/cm² C

Allowable unit comp. $= 1200(1-0.012 \times \frac{280}{26.5}) = 1048 \times 1.6 = 1675$ kg/cm² C ✓

昭和八年八月

國道拾貳号線

岐阜縣木曽川橋材料調書

（上部工事）

JIUN MASUDA
CONSULTING ENGINEER
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SHOWA

MADE BY M. Jinda DATE 8-8-13 FILE NO M
CHECKED BY H.K DATE 8-8-17 PAGE NO 1

CALCULATIONS FOR

Materials of Kisogawa Bashi for Gifu-Ken

Main truss

End Post $L_0-U_0{}_L^R$ 4-Required (For one span)

Main Section	1	Cov. Pl	600 × 12 c	56.520	× 4,330	=	244.73	
〃 〃	2	∟s	100 × 100 × 10 c	14.9	× 4,845	=	144.38	
〃 〃	2	〃	〃 c	〃	× 4,590	=	136.78	
〃 〃	2	Web Pls	500 × 10 c	39.250	× 4,305	=	337.94	
Tie plate	2	Pls	430 × 9 c	30.380	× 595	=		36.15
Lac. bars	8	〃	70 × 12 c	6.594	× 815	=		42.99
washers	2	Wash.	70φ c	3.02	× 12	=		.72
Gusset	2	Pls	1,020 × 10 c	80.070	× 1,390	=		222.59
Splice	1	Pls	600 × 12 c	56.520	× 475	=		26.85
〃	2	∟s	100 × 100 × 10 c	14.9	× 1,010	=		30.10
〃	2	Pls	95 × 9 c	6.712	× 525	=		7.05
〃	2	〃	765 × 9 c	54.047	× 1,225	=		132.42
〃	2	〃	630 × 10 c	49.455	× 1,260	=		124.63

863.83 + 623.50 = 1,487.33

Top chord $U_0-U_1{}_L^R$ 4-Required

Main Section	1	Cov. Pl	600 × 12 c	56.520	× 5,285	=	298.71	
〃 〃	2	∟s	100 × 100 × 10 c	14.9	× 6,140	=	182.97	
〃 〃	2	〃	〃 c	〃	× 5,880	=	175.22	
〃 〃	2	Web Pls	480 × 10 c	37.680	× 5,280	=	397.90	
Tie plate	2	Pls	500 × 9 c	35.325	× 595	=		42.04
Lac. bars	10	〃	70 × 12 c	6.594	× 815	=		53.74
washer	2	Wash.	70φ c	3.02	× 12	=		.72
Splice	1	Pls	450 × 12 c	56.520	× 600	=		33.91
〃	2	〃	795 × 10 c	23.158	× 640	=		29.64
〃	4	∟s	90 × 90 × 13 c	17.0	× 580	=		39.44
〃	2	Pls	95 × 9 c	6.712	× 370	=		4.97
Gusset	2	〃	870 × 10 c	68.295	× 1,360	=		185.76
〃	1	Pl	445 × 9 c	31.439	× 765	=		24.05
Filler	1	〃	80 × 9 c	5.652	× 310	=		1.75

1,054.80 + 416.02 = 1,470.82

$U_1-U_2{}_L^R$ 4-Required

Main Section	1	Cov. Pl	600 × 12 c	56.520	× 5,020	=	283.73	
〃 〃	2	∟s	100 × 100 × 10 c	14.9	× 5,130	=	152.87	
〃 〃	2	〃	〃 c	〃	× 5,110	=	152.28	
〃 〃	2	Web Pls	480 × 10 c	37.680	× 5,015	=	377.93	
Tie plate	2	Pls	570 × 9 c	40.271	× 595	=		47.92
Lac. bars	8	〃	70 × 12 c	6.594	× 815	=		42.99
washer	2	Wash.	70φ c	3.02	× 12	=		.72
Splice	1	Pl	450 × 12 c	56.520	× 600	=		33.91
〃	2	Pls	295 × 10 c	23.158	× 560	=		25.94
〃	4	∟s	90 × 90 × 13 c	17.0	× 580	=		39.44
〃	2	Pls	95 × 9 c	6.712	× 370	=		4.97
Gusset	2	〃	860 × 10 c	67.510	× 1,445	=		195.10
〃	1	Pl	420 × 9 c	29.673	× 735	=		21.81
Filler	1	〃	80 × 9 c	5.652	× 280	=		1.58

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CALCULATIONS FOR

Materials of Kisogawa Bashi for Gifu-Ken

966.81 + 414.38 = 1,381.19

$U_2 - U_{3\,L}^{\,R}$ 4-Required

Main Section	1	Cov. Pl	600 × 12 c	56.520 ×	4,890 =	276.38	
" "	2	Ls	100 × 100 × 10 c	14.9 ×	5,020 =	149.60	
" "	2	"	" c	" ×	4,990 =	148.70	
" "	2	Web Pls	480 × 10 c	37.680 ×	4,890 =	368.51	
Tie plate	1	Pl	500 × 9 c	35.325 ×	595 =		21.02
" "	1	"	430 × 9 c	30.380 ×	595 =		18.08
Lac. bar	8	Pls	70 × 12 c	6.594 ×	815 =		42.99
Washer	2	Wash.	70ϕ c	3.02 ×	12 =		0.72
Splice	1	Pl	450 × 12 c	56.520 ×	600 =		33.91
"	2	Pls	295 × 10 c	23.158 ×	510 =		23.62
"	4	Ls	90 × 90 × 13 c	17.0 ×	580 =		39.44
"	2	Pls	95 × 9 c	6.712 ×	370 =		4.97
Gusset	1	Pl	445 × 9 c	31.439 ×	735 =		23.11
"	2	Pls	840 × 10 c	65.940 ×	1,535 =		202.44
Filler	1	Pl	80 × 9 c	5.652 ×	280 =		1.58

943.19 + 411.88 = 1,355.07

$U_3 - U_{4\,L}^{\,R}$ 4-Required

Main section	1	Cov. Pl.	600 × 12 c	56.520 ×	4,810 =	271.86	
" "	2	Ls	100 × 100 × 10 c	14.9 ×	4,945 =	147.36	
" "	2	"	" c	" ×	4,920 =	146.62	
" "	2	Web Pls	480 × 10 c	37.680 ×	4,800 =	361.73	
Tie plate	1	Pl.	595 × 9 c	42.037 ×	630 =		26.48
" "	1	"	570 × 9 c	40.271 ×	595 =		23.96
Lac. bar	6	Pls	70 × 12 c	6.594 ×	815 =		32.24
Washer	2	Wash.	70ϕ c	3.02 ×	12 =		0.72
Splice	1	Pl.	590 × 12 c	55.578 ×	600 =		33.35
"	2	Pls	95 × 9 c	6.712 ×	370 =		4.97
"	2	"	280 × 10 c	21.980 ×	495 =		21.76
"	2	"	470 × 10 c	36.895 ×	775 =		57.19
"	4	Ls	90 × 90 × 13 c	17.0 ×	580 =		39.44
Gusset	2	Pls	840 × 10 c	65.940 ×	1,670 =		220.24
"	1	Pl.	420 × 9 c	29.673 ×	805 =		23.89
"	1	Fill.	80 × 12 c	7.536 ×	210 =		1.58

927.57 + 485.82 = 1,413.39

$U_4 - U_{5\,L}^{\,R}$ 4-Required

Main Section	1	Cov. Pl.	600 × 12 c	56.520 ×	4,735 =	267.62	
" "	2	Ls	100 × 100 × 10 c	14.9 ×	4,765 =	142.00	
" "	2	"	" c	" ×	4,740 =	141.25	
" "	2	Web Pls	480 × 10 c	37.680 ×	4,730 =	356.45	
" "	2	Pls	280 × 10 c	21.980 ×	4,730 =	207.93	

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CALCULATIONS FOR

Materials of Kisogawa Bashi for Gifu-ken

Tie plate	2	P/s	500 × 9 c	35.325 ×	595 =		42.04
Lac. bar	6	〃	70 × 12 c	6.594 ×	815 =		32.24
Washer	2	Wash.	70ᶲ c	30.2 ×	12 =		0.72
Splice	1	Pl	590 × 12 c	55.578 ×	600 =		33.35
〃	2	P/s	465 × 10 c	36.503 ×	750 =		54.75
〃	2	〃	95 × 9 c	6.712 ×	370 =		4.97
〃	4	Ls	90×90×13 c	17.0 ×	580 =		39.44
Gusset	2	P/s	810 × 10 c	63.585 ×	1,670 =		212.37
〃	1	Pl.	445 × 9 c	31.439 ×	805 =		25.31
〃	1	Fill.	80 × 12 c	7.536 ×	210 =		1.58

1,115.25 + 446.77 = 1,562.02

$U_5 - U_{6L}^{R}$ 4-Required

Main Section	1	Cov. Pl	600 × 12 c	56.520 ×	4,695 =	265.36	
〃 〃	2	Ls	100×100×10 c	14.9 ×	4,740 =	141.25	
〃 〃	2	〃	〃 c	〃 ×	4,720 =	140.66	
〃 〃	2	Web P/s	480 × 10 c	37.680 ×	4,690 =	353.44	
〃 〃	2	P/s	280 × 10 c	21.980 ×	4,685 =	205.95	
〃 〃	2	〃	〃 c	〃 ×	1,320 =	58.03	
Tie plate	1	Pl	500 × 9 c	35.325 ×	595 =		21.02
〃 〃	1	〃	430 × 9 c	30.380 ×	595 =		18.08
Lac. bar	6	P/s	70 × 12 c	6.594 ×	815 =		32.24
washer	2	Wash.	70ᶲ c	30.2 ×	12 =		0.72
Splice	1	Pl	590 × 12 c	55.578 ×	600 =		33.35
〃	2	P/s	465 × 10 c	36.503 ×	735 =		53.66
〃	2	〃	95 × 9 c	6.712 ×	370 =		4.97
〃	4	Ls	90×90×13 c	17.0 ×	595 =		40.46
〃	2	P/s	280 × 10 c	21.980 ×	445 =		19.56
Gusset	2	〃	810 × 10 c	63.585 ×	1,690 =		214.92
〃	1	Pl.	420 × 9 c	29.673 ×	805 =		23.89
〃	1	Fill.	80 × 12 c	7.536 ×	210 =		1.58

1,164.69 + 464.45 = 1,629.14

$U_6 - U_7 - U_6$ 2-Required

Main Section	1	Cov Pl	600 × 12 c	56.520 ×	9,350 =	528.46	
〃 〃	2	Ls	100×100×10 c	14.9 ×	6,710 =	199.96	
〃 〃	2	〃	〃 c	〃 ×	6,680 =	199.06	
〃 〃	4	Web P/s	480 × 10 c	37.680 ×	4,675 =	704.62	
〃 〃	4	P/s	280 × 10 c	21.980 ×	3,360 =	295.41	
Tie plate	2	〃	500 × 9 c	35.325 ×	595 =		42.04
〃 〃	2	〃	430 × 9 c	30.380 ×	595 =		36.15
Lac. bar	12	〃	70 × 12 c	6.594 ×	815 =		64.49
Washer	4	Wash.	70ᶲ c	30.2 ×	12 =		1.45
Splice	2	P/s	460 × 10 c	36.110 ×	730 =		52.72
Gusset	2	〃	790 × 10 c	62.015 ×	730 =		90.54
〃	1	Pl	390 × 9 c	27.554 ×	590 =		16.26
〃	1	Fill.	80 × 12 c	7.536 ×	590 =		4.45

1,927.51 + 308.10 = 2,235.61

Materials of Kisogawa Bashi for Gifu-ken

		Diaphragm DM1		4-Required			
	4	Ls	100×75×10 c	13.0 ×	990 =		51.48
	1	Pl	345 × 9 c	24.374 ×	990 =		24.13
	2	Ls	75×75×9 c	9.96 ×	353 =		7.03
							82.64
		Diaphragm DM5		64-Required			
	2	Ls	90×75×9 c	11.0 ×	485 =		10.67
	1	Pl	365 × 9 c	25.787 ×	485 =		12.51
	1	L	75×75×9 c	9.96	365 =		3.64
							26.82
		Middle chord	$L_0 - M_1$	4-Required			
Main Section	2	Ls	100×100×10 c	14.9 ×	6,185 =	184.31	
〃 〃	2	〃	〃 c	〃 ×	5,910 =	176.12	
〃 〃	2	Web Pls	480 × 10 c	37.680 ×	4,470 =	336.86	
〃 〃	2	Pls	280 × 10 c	21.980 ×	5,225 =	229.69	
〃 〃	2	〃	450 × 12 c	42.390 ×	4,775 =	404.82	
Tie plate	2	〃	595 × 9 c	42.037 ×	710 =		59.69
〃 〃	1	Pl	570 × 9 c	40.271 ×	595 =		23.96
〃 〃	1	〃	500 × 9 c	35.325 ×	595 =		21.02
Lac. bar	16	Pls	70 × 12 c	6.594 ×	815 =		85.99
washer	4	Wash.	70φ c	30.2 ×	12 =		1.45
Gusset	2	Pls	925 × 10 c	72.613 ×	1,460 =		212.03
Splice	2	〃	490 × 12 c	46.158 ×	1,460 =		134.78
〃	2	〃	475 × 12 c	44.745 ×	870 =		77.86
〃	4	Ls	90×90×13 c	17.0 ×	580 =		39.44
〃	2	Fills	290 × 3 c	6.830 ×	450 =		6.15
						1,331.80 + 662.37 = 1,994.17	
		$M_1 - M_2$		4-Required			
Main Section	2	Ls	100×100×10 c	14.9 ×	5,225 =	153.71	
〃 〃	2	〃	〃 c	〃 ×	5,190 =	154.66	
〃 〃	2	Web Pls	480 × 10 c	37.680 ×	5,325 =	401.29	
〃 〃	2	Pls	280 × 10 c	21.980 ×	5,315 =	233.65	
〃 〃	2	〃	450 × 9 c	31.793 ×	5,320 =	338.28	
Tie plate	1	Pl	595 × 9 c	42.037 ×	735 =		30.90
〃 〃	1	〃	〃 c	〃 ×	780 =		32.79
〃 〃	1	〃	510 × 9 c	36.032 ×	595 =		21.44
〃 〃	1	〃	570 × 9 c	40.271 ×	595 =		23.96
Lac. bar	20	Pls	70 × 12 c	6.594 ×	815 =		107.48
washer	4	Wash.	70φ c	30.2 ×	12 =		1.45
Gusset	2	Pls	915 × 10 c	71.828 ×	1,335 =		191.78
Splice	2	〃	485 × 9 c	34.265 ×	1,250 =		85.66
〃	2	〃	470 × 9 c	33.206 ×	800 =		53.13
〃	4	Ls	90×90×13 c	17.0 ×	580 =		39.44
						1,281.59 + 588.03 = 1,869.62	

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CALCULATIONS FOR

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M2-M3 4-Required

Main Section	2	Ls 100×100×10 c	14.9 × 5,040 =	150.19	
〃 〃	2	〃 〃 c	〃 × 5,000 =	149.00	
〃 〃	2	Web P/s 480 ×10 c	37.680 × 5,080 =	382.83	
〃 〃	2	P/s 280 ×10 c	21.980 × 5,070 =	222.88	
〃 〃	2	〃 450 ×9 c	31.793 × 4,291 =	272.85	
Tie plate	1	P/ 595 ×9 c	42.037 × 735 =		30.90
〃 〃	1	〃 〃 c	〃 × 710 =		29.85
〃 〃	1	〃 〃 c	〃 × 760 =		31.95
〃 〃	1	〃 〃 c	〃 × 640 =		26.90
Lac. bar	20	P/s 70 ×12 c	6.594 × 815 =		107.48
washer	4	Wash. 70ϕ c	3.02 × 12 =		1.45
Gusset	2	P/s 910 ×10 c	71.435 × 1,320 =		188.59
Splice	2	〃 490 ×9 c	34.619 × 1,180 =		81.70
〃	2	〃 480 ×9 c	33.912 × 950 =		64.43
〃	4	Ls 90×90×13 c	17.0 × 580 =		39.44

1,177.75 + 602.69 = 1,780.44

M3-M4 4-Required

Main Section	2	Ls 100×100×10 c	14.9 × 4,755 =	141.70	
〃 〃	2	〃 〃 c	〃 × 4,720 =	140.66	
〃 〃	2	Web P/s 480 ×10 c	37.680 × 4,910 =	370.02	
〃 〃	2	P/s 280 ×10 c	21.980 × 4,900 =	215.40	
Tie plate	1	P/ 595 ×9 c	42.037 × 735 =		30.90
〃 〃	1	〃 〃 c	〃 × 640 =		26.90
〃 〃	1	〃 〃 c	〃 × 620 =		26.06
〃 〃	1	〃 500 ×9 c	35.325 × 595 =		21.02
Lac. bar	18	P/s 70 ×12 c	6.594 × 815 =		96.73
washer	4	Wash. 70ϕ c	3.02 × 12 =		1.45
Gusset	2	P/s 900 ×10 c	70.650 × 1,390 =		196.41
Splice					
〃	4	Ls 90×90×13 c	17.0 × 585 =		39.78
〃	2	P/s 280 ×10 c	21.980 × 410 =		18.02

867.78 + 457.27 = 1,325.05

M4-M5 4-Required

Main Section	2	Ls 100×100×10 c	14.9 × 4,080 =	121.58	
〃 〃	2	〃 〃 c	〃 × 4,060 =	120.99	
〃 〃	2	Web P/s 480 ×10 c	37.680 × 4,790 =	360.97	
Tie plate	1	P/ 595 ×9 c	42.037 × 620 =		26.06
〃 〃	1	〃 570 ×9 c	40.271 × 595 =		23.96
〃 〃	1	〃 550 ×9 c	38.858 × 595 =		23.12
〃 〃	1	〃 430 ×9 c	30.380 × 595 =		18.08
Lac. bar	18	P/s 70 ×12 c	6.594 × 815 =		96.73
washer	4	wash. 70ϕ c	3.02 × 12 =		1.45
Gusset	2	P/s 900 ×10 c	70.650 × 1420 =		200.65
Splice	2	〃 295 ×10 c	23.158 × 515 =		23.85
〃	2	〃 95 ×9 c	6.712 × 340 =		4.56
〃	2	Ls 90×90×13 c	17.0 × 780 =		26.52
〃	2	P/s 95 ×9 c	6.712 × 390 =		5.24

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Splice	2	L³	90×90×13 c	17.0 × 745 =		25.33

603.54 + 475.55 = 1,079.09

M5 - M6 4-Required

Main Section	2	L³	100×100×10 c	14.9 × 4,710 =	140.36	
〃 〃	2	〃	〃 c	〃 × 4,700 =	140.06	
〃 〃	2	Web P/s	480 × 10 c	37.680 × 4,710 =	354.95	
Tie plate	2	P/s	570 × 9 c	40.271 × 595 =		47.92
〃 〃	1	〃	500 × 9 c	35.325 × 595 =		21.02
〃 〃	1	〃	430 × 9 c	30.380 × 595 =		18.08
Lac. bar	18	〃	70 × 12 c	6.594 × 815 =		96.73
washer	4	Wash.	70ᶲ c	3.02 × 12 =		1.45
Gusset	2	P/s	890 × 10 c	69.865 × 1,470 =		205.40
Splice	2	〃	295 × 10 c	23.158 × 465 =		21.54
〃	2	L³	90×90×13 c	17.0 × 730 =		24.82
〃	2	〃	〃 c	〃 × 760 =		25.84
〃	2	P/s	95 × 9 c	6.712 × 340 =		4.56
〃	2	〃	〃 c	〃 × 380 =		5.10

635.37 + 472.46 = 1,107.83

M6-M7 4-Required

Main Section	2	L³	100×100×10 c	14.9 × 4,690 =	139.76	
〃 〃	2	〃	〃 c	〃 × 4,640 =	138.27	
〃 〃	2	Web P/s	480 × 10 c	37.680 × 4,680 =	352.68	
Tie plate	2	P/s	570 × 9 c	40.271 × 595 =		47.92
〃 〃	2	〃	430 × 9 c	30.380 × 595 =		36.15
Lac. bar	18	〃	70 × 12 c	6.594 × 815 =		96.73
washer	4	Wash.	70ᶲ c	3.02 × 12 =		1.45

630.71 + 182.25 = 812.96

Splice at M7 2-Required

Gusset	2	P/s	910 × 10 c	71.435 × 2,260 =		322.89
Splice	2	〃	290 × 10 c	22.765 × 440 =		20.03
〃	2	L³	90×90×13 c	17.0 × 710 =		24.14
〃	2	〃	〃 c	〃 × 740 =		25.16
〃	2	P/s	95 × 9 c	6.712 × 360 =		4.83
〃	2	〃	〃 c	〃 × 340 =		4.56

401.61

Diaphragm DM4 56-Required

	2	L³	90×75×9 c	11.0 × 480 =		10.56
	1	P/	365 × 9 c	25.787 × 480 =		12.38
	1	L	75×75×9 c	9.96 × 365 =		3.64

26.58

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CALCULATIONS FOR

Materials of Kisogawa Bashi for Gifu-Ken

Tie T1 R/L 4-Required

Main section	2	Web P/s	500 × 10 c	39.250 × 5,875 =	461.19		
〃 〃	2	P/s	400 × 15 c	47.100 × 7,220 =	680.12		
〃 〃	2	Ls	150 × 100 × 15 c	27.7 × 6,690 =	370.63		
Gusset	2	P/s	1,670 × 10 c	131.095 × 1,805 =		473.25	
〃	2	〃	1,395 × 10 c	109.508 × 1,470 =		321.95	
〃	2	〃	525 × 10 c	41.213 × 885 =		72.95	
〃	2	Fil/s	480 × 7 c	26.376 × 525 =		27.69	
Pin plate	2	P/s	525 × 10 c	41.213 × 710 =		58.52	
〃 〃	2	〃	310 × 15 c	36.503 × 525 =		38.33	
Splice at Lo	2	〃	315 × 10 c	24.728 × 800 =		39.56	
〃	1	Pl	315 × 15 c	37.091 × 505 =		18.73	
Diaphragm	2	Ls	100×75 × 10 c	13.0 × 810 =		21.06	
〃	1	Pl	345 × 9 c	24.374 × 810 =		19.74	
Tie plate	4	P/s	360 × 9 c	25.434 × 695 =		70.71	
Gusset	1	Pl	500 × 9 c	35.325 × 1,060 =		37.44	
〃	1	Fill	260 × 10 c	20.410 × 500 =		10.21	
〃	1	Pl	690 × 9 c	48.749 × 910 =		44.36	
〃	1	Fill	215 × 10 c	16.878 × 260 =		4.39	
Gusset connection	1	L	125 × 90 × 9 c	14.6 × 600 =		8.76	
〃 〃	1	Fill	85 × 10 c	6.673 × 425 =		2.84	
〃 〃	1	〃	85 × 15 c	10.009 × 425 =		4.25	
Splice SP1	2	P/s	500 × 10 c	39.250 × 900 =		70.65	
〃 〃	2	〃	480 × 15 c	56.520 × 900 =		101.74	
〃 〃	2	〃	115 × 9 c	8.125 × 830 =		13.49	
〃 〃	1	Pl	695 × 9 c	49.102 × 830 =		40.75	
Filler	2	P/s	500 × 10 c	39.250 × 690 =		54.17	
					1,511.94 + 1,555.54 = 3,067.48		

T2 R/L 4-Required

Main section	2	Web P/s	500 × 10 c	39.250 × 8,095 =	635.46		
〃 〃	2	P/s	400 × 15 c	47.100 × 8,095 =	762.55		
〃 〃	2	Ls	150 × 100 × 15 c	27.7 × 8,095 =	448.46		
Tie plate	4	P/s	360 × 9 c	25.434 × 695 =		70.71	
Gusset	1	Pl	990 × 9 c	69.944 × 1,340 =		93.72	
〃	1	Fill	260 × 10 c	20.410 × 295 =		6.02	
〃	1	Pl	690 × 9 c	48.749 × 910 =		44.36	
〃	1	Fill	215 × 10 c	16.878 × 260 =		4.39	
Splice SP2	2	P/s	500 × 10 c	39.250 × 865 =		67.90	217.64
〃	2	〃	480 × 15 c	56.520 × 865 =		97.78	
〃	2	〃	115 × 9 c	8.125 × 795 =		12.92	
〃	1	Pl	695 × 9 c	49.102 × 795 =		39.04	
Filler	4	P/s	500 × 10 c	39.250 × 690 =		108.33	
					1,846.47 + 545.17 = 2,391.64		

T3 R/L 4-Required

Main section	2	Web P/s	500 × 10 c	39.250 × 6,990 =	548.72		
〃 〃	2	P/s	400 × 15 c	47.100 × 6,990 =	658.46		
〃 〃	2	Ls	150 × 100 × 15 c	27.7 × 6,990 =	387.25		
Tie plate	4	P/s	360 × 9 c	25.434 × 695 =		70.71	
Gusset	1	Pl	945 × 9 c	66.764 × 1,265 =		84.46	

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Gusset	1	Fill	250 × 10 c	19.625 × 260 =		5.10
Splice SP2				=		217.64
Filler	2	P/s	500 × 10	39.250 × 690 =		54.17
						1,594.43 + 432.08 = 2,026.51

T4^R_L 4-Required

Main section	2	Web P/s	500 × 10 c	39.250 × 6,950 =	545.58	
〃 〃	2	P/s	400 × 15 c	47.100 × 6,950 =	654.69	
〃 〃	2	Ls	150 × 100 × 15 c	27.7 × 6,950 =	385.03	
Tie plate	3	P/s	360 × 9 c	25.434 × 695 =		53.03
Gusset	1	Pl	690 × 9 c	48.749 × 910 =		44.36
〃	1	Fill	215 × 10 c	16.878 × 260 =		4.39
〃	1	Pl	945 × 9 c	66.764 × 1,290 =		86.13
〃	1	Fill	250 × 10 c	19.625 × 260 =		5.10
Splice SP2				=		217.64
Filler	4	P/s	500 × 10 c	39.250 × 690 =		108.33
						1,585.30 + 518.98 = 2,104.28

T5 2-Required

Main section	2	Web P/s	500 × 10 c	39.250 × 6,990 =	548.72	
〃 〃	2	P/s	400 × 15 c	47.100 × 6,990 =	658.46	
〃 〃	2	Ls	150 × 100 × 15 c	27.7 × 6,990 =	387.25	
Tie plate	4	P/s	360 × 9 c	25.434 × 695 =		70.71
Gusset	1	Pl	690 × 9 c	48.749 × 910 =		44.36
〃	1	Fill	215 × 10 c	16.878 × 260 =		4.39
Filler	2	P/s	500 × 10 c	39.250 × 690 =		54.17
						1,594.43 + 173.63 = 1,768.06

Diaphragm DM2 24-Required

	1	Pl	365 × 9 c	25.787 × 500 =	12.89	
	4	Ls	90 × 75 × 9 c	11.0 × 505 =	22.22	
	2	〃	〃 c	〃 × 195 =	4.29	
						39.40

Diaphragm DM3 4-Required

	1	Pl	345 × 9 c	24.374 × 500 =	12.19	
	4	Ls	90 × 75 × 9 c	11.0 × 505 =	22.22	
	2	〃	〃 c	〃 × 195 =	4.29	
						38.70

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CALCULATIONS FOR

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		Diagonal D1			4-Required		
Main Section	2	L͡s	150×90×9	c	16.4 × 4,675 =	153.34	
〃 〃	2	〃	〃	c	〃 × 4,760 =	156.13	
Tie plate	2	P/s	340 × 9	c	24.021 × 500 =		24.02
Lac. bar	14	〃	75 × 9	c	5.299 × 366 =		27.15
						309.47 + 51.17 =	360.64
		D2			4-Required		
Main Section	2	L͡s	150×90×9	c	16.4 × 4,340 =	142.35	
〃 〃	2	〃	〃	c	〃 × 4,360 =	143.01	
Tie plate	2	P/s	340 × 9	c	24.021 × 500 =		24.02
Lac. bar	13	〃	75 × 9	c	5.299 × 366 =		25.21
						285.36 + 49.23 =	334.59
		D3			4-Required		
Main section	2	L͡s	150×90×9	c	16.4 × 4,200 =	137.76	
〃 〃	2	〃	〃	c	〃 × 4,240 =	139.07	
Tie plate	2	P/s	340 × 9	c	24.021 × 430 =		20.66
Lac. bar	13	〃	75 × 9	c	5.299 × 366 =		25.21
						276.83 + 45.87 =	322.70
		D4			4-Required		
Main section	2	L͡s	150×90×9	c	16.4 × 4,075 =	133.66	
〃 〃	2	〃	〃	c	〃 × 4,125 =	135.30	
Tie plate	2	P/s	340 × 9	c	24.021 × 430 =		20.66
Lac. bar	12	〃	75 × 9	c	5.299 × 366 =		23.27
						268.96 + 43.93 =	312.89
		D5			4-Required		
Main section	2	L͡s	150×100×9	c	17.1 × 3,790 =	129.62	
〃 〃	2	〃	〃	c	〃 × 3,770 =	128.93	
Tie plate	2	P/s	340 × 9	c	24.021 × 500 =		24.02
Lac. bar	10	〃	75 × 9	c	5.299 × 355 =		18.81
						258.55 + 42.83 =	301.38
		D6			4-Required		
Main section	2	L͡s	150×90×9	c	16.4 × 3,720 =	122.02	
〃 〃	2	〃	〃	c	〃 × 3,735 =	122.51	
Tie plate	2	P/s	340 × 9	c	24.021 × 500 =		24.02
Lac. bar	10	〃	75 × 9	c	5.299 × 366 =		19.39
						244.53 + 43.41 =	287.94
		D7			4-Required		
Main section	2	L͡s	150×90×9	c	16.4 × 3,770 =	123.66	
〃 〃	2	〃	〃	c	〃 × 3,765 =	123.49	
Tie plate	2	P/s	340 × 9	c	24.021 × 500 =		24.02
Lac. bar	11	〃	75 × 9	c	5.299 × 366 =		21.33
						247.15 + 45.35 =	292.50

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CALCULATIONS FOR

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Hanger $H1^R_L$ 4- Required

Main section	2	P/s	265 × 9 c	18.722 × 3,670 =	137.42	
〃 〃	4	Ls	125 × 90 × 9 c	14.6 × 3,670 =	214.33	
〃 〃	1	P/	325 × 9 c	22.961 × 3,670 =		84.27
Splice	2	P/s	300 × 9 c	21.195 × 725 =		30.73
〃	2	〃	145 × 9 c	10.244 × 725 =		14.85
					351.75 + 129.85 = 481.60	

$H2^R_L$ 4- Required

Main section	4	Ls	125 × 75 × 9 c	13.5 × 6,160 =	332.64	
〃 〃	1	P/	340 × 8 c	21.352 × 6,160 =	131.53	
Splice	2	P/s	190 × 8 c	11.932 × 365 =		8.71
					464.17 + 8.71 = 472.88	

$H3^R_L$ 4- Required

Main section	4	Ls	125 × 75 × 9 c	13.5 × 8,110 =	437.94	
〃 〃	1	P/	340 × 8 c	21.352 × 8,110 =	173.16	
Splice	2	P/s	190 × 8 c	11.932 × 365 =		8.71
					611.10 + 8.71 = 619.81	

$H4^R_L$ 4- Required

Main section	4	Ls	125 × 75 × 9 c	13.5 × 9,565 =	516.51	
〃 〃	1	P/	340 × 8 c	21.352 × 9,565 =	204.23	
Splice	2	P/s	190 × 8 c	11.932 × 365 =		8.71
					720.74 + 8.71 = 729.45	

$H5^R_L$ 4- Required

Main section	4	Ls	125 × 75 × 9 c	13.5 × 10,590 =	571.86	
〃 〃	1	P/	340 × 8 c	21.352 × 5,580 =	119.14	
〃 〃	1	〃	〃 c	〃 × 5,005 =	106.87	
Splice	4	P/s	190 × 8 c	11.932 × 365 =		17.42
					797.87 + 17.42 = 815.29	

$H6^R_L$ 4- Required

Main section	4	Ls	125 × 75 × 9 c	13.5 × 11,190 =	604.26	
〃 〃	1	P/	340 × 8 c	21.352 × 5,580 =	119.14	
〃 〃	1	〃	〃 c	〃 × 5,605 =	119.68	
Splice	4	P/s	190 × 8 c	11.932 × 365 =		17.42
					843.08 + 17.42 = 860.50	

H7 2 - Required

Main section	4	Ls	125 × 75 × 9 c	13.5 × 11,390 =	615.06	
〃 〃	1	P/	340 × 8 c	21.352 × 5,580 =	119.14	
〃 〃	1	〃	〃 c	〃 × 5,805 =	123.95	
Splice	4	P/s	190 × 8 c	11.932 × 365 =		17.42
					858.15 + 17.42 = 875.57	

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CALCULATIONS FOR

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		Vertical $V1^{R}_{L}$		4-Required			
main section	2	Pl's	265 × 9 c	18.722 × 3.850	= 144.16		
〃 〃	2	Ls	125 × 90 × 9 c	14.6 × 3.775	= 110.23		
〃 〃	2	〃	〃 c	〃 × 3.850	= 112.42		
〃 〃	1	Pl	325 × 9 c	22.961 × 3.775	= 86.68		
						= 453.49	
		$V2^{R}_{L}$		4-Required			
main section	2	Ls	125 × 75 × 9 c	13.5 × 3.115	= 84.11		
〃 〃	2	〃	〃 c	〃 × 3.165	= 85.46		
〃 〃	1	Pl	340 × 8 c	21.352 × 3.115	= 66.51		
						= 236.08	
		$V3^{R}_{L}$		4-Required			
main section	2	Ls	125 × 75 × 9 c	13.5 × 2.605	= 70.34		
〃 〃	2	〃	〃 c	〃 × 2.635	= 71.15		
〃 〃	1	Pl	340 × 8 c	21.352 × 2.605	= 55.62		
						= 197.11	
		$V4^{R}_{L}$		4-Required			
main section	2	Ls	125 × 75 × 9 c	13.5 × 2.250	= 60.75		
〃 〃	2	〃	〃 c	〃 × 2.270	= 61.29		
〃 〃	1	Pl	340 × 8 c	21.352 × 2.250	= 48.04		
						= 170.08	
		$V5^{R}_{L}$		4-Required			
main section	2	Ls	125 × 75 × 9 c	13.5 × 1.995	= 53.87		
〃 〃	2	〃	〃 c	〃 × 2.020	= 54.54		
〃 〃	1	Pl	340 × 8 c	21.352 × 1.995	= 42.60		
	1	〃	210 × 10 c	16.485 × 260	=	4.29	
					151.01 +	4.29	= 155.30
		$V6^{R}_{L}$		4-Required			
main section	4	Ls	125 × 75 × 9 c	13.5 × 1.870	= 100.98		
〃 〃	1	Pl	340 × 8 c	21.352 × 1.870	= 39.93		
						= 140.91	
		V 7		2-Required			
main section	4	Ls	125 × 75 × 9 c	13.5 × 1.815	= 98.01		
〃 〃	1	Pl	340 × 8 c	21.352 × 1.815	= 38.75		
	1	〃	210 × 10 c	16.485 × 260	=	4.29	
					136.76 +	4.29	= 141.05

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CALCULATIONS FOR

Materials of Kisogawa-Bashi for Gifu-Ken

Summary of main truss for one span

End Post	$L_0 - U_0{}^R_L$	4 @	1,487.33	=	5,949.32	
Top chord	$U_0 - U_1{}^R_L$	4 @	1,470.82	=	5,883.28	
〃	$U_1 - U_2{}^R_L$	4 @	1,381.19	=	5,524.76	
〃	$U_2 - U_3{}^R_L$	4 @	1,355.07	=	5,420.28	
〃	$U_3 - U_4{}^R_L$	4 @	1,413.39	=	5,653.56	
〃	$U_4 - U_5{}^R_L$	4 @	1,562.02	=	6,248.08	
〃	$U_5 - U_6{}^R_L$	4 @	1,629.14	=	6,516.56	
〃	$U_6 - U_7 - U_6'$	2 @	2,235.61	=	4,471.22	
Diaphragm	DM1	4 @	82.64	=	330.56	
〃	DM5	64 @	26.82		1,716.48	
						47,714.10
Middle chord	$L_0 - M_1$	4 @	1,994.17	=	7,976.68	
〃	$M_1 - M_2$	4 @	1,869.62	=	7,478.48	
〃	$M_2 - M_3$	4 @	1,780.44	=	7,121.76	
〃	$M_3 - M_4$	4 @	1,325.05	=	5,300.20	
〃	$M_4 - M_5$	4 @	1,079.09	=	4,316.36	
〃	$M_5 - M_6$	4 @	1,107.83	=	4,431.32	
〃	$M_6 - M_7$	4 @	812.96	=	3,251.84	
Splice at M_7		2 @	401.61	=	803.22	
Diaphragm	DM4	56 @	26.58	=	1,488.48	
						42,168.34
Tie	$T1{}^R_L$	4 @	3,067.48	=	12,269.92	
〃	$T2{}^R_L$	4 @	2,391.64	=	9,566.56	
〃	$T3{}^R_L$	4 @	2,026.51	=	8,106.04	
〃	$T4{}^R_L$	4 @	2,104.28	=	8,417.12	
〃	T5	2 @	1,768.06	=	3,536.12	
Diaphragm	DM2	24 @	39.40	=	945.60	
〃	DM3	4 @	38.70	=	154.80	
						42,996.16
Diagonal	D1	4 @	360.64	=	1,442.56	
〃	D2	4 @	334.59	=	1,338.36	
〃	D3	4 @	322.70	=	1,290.80	
〃	D4	4 @	312.89	=	1,251.56	
〃	D5	4 @	301.38	=	1,205.52	
〃	D6	4 @	287.94	=	1,151.76	
〃	D7	4 @	292.50	=	1,170.00	
						8,850.56
Hanger	$H1{}^R_L$	4 @	481.60	=	1,926.40	
〃	$H2{}^R_L$	4 @	472.88	=	1,891.52	
〃	$H3{}^R_L$	4 @	619.81	=	2,479.24	
〃	$H4{}^R_L$	4 @	729.45	=	2,917.80	
〃	$H5{}^R_L$	4 @	815.29	=	3,261.16	
〃	$H6{}^R_L$	4 @	860.50	=	3,442.00	
〃	H7	2 @	875.57	=	1,751.14	
Vertical	$V1{}^R_L$	4 @	453.49	=	1,813.96	
〃	$V2{}^R_L$	4 @	236.08	=	944.32	
〃	$V3{}^R_L$	4 @	197.11	=	788.44	
〃	$V4{}^R_L$	4 @	170.08	=	680.32	
〃	$V5{}^R_L$	4 @	155.30	=	621.20	
〃	$V6{}^R_L$	4 @	140.91	=	563.64	
〃	V7	2 @	141.05	=	282.10	
						23,363.24
						165,092.40

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CALCULATIONS FOR

Materials of Kisogawa Bashi for Gifu-ken

Portal Bracing PB1 2- Required

Top rail	4	Ls	150×90×9	c	16.4	×	4,800	=	314.88
〃 〃	4	〃	90×75×9	c	11.0	×	4,800	=	211.20
Bottom rail	4	〃	〃	c	〃	×	5,200	=	228.80
〃 〃	4	〃	150×90×9	c	16.4	×	5,450	=	357.52
Diagonal	4	〃	125×75×9	c	13.5	×	1,380	=	74.52
〃	4	〃	〃	c	〃	×	1,030	=	55.62
〃	4	〃	〃	c	〃	×	990	=	53.46
〃	4	〃	〃	c	〃	×	1,000	=	54.00
Vertical	4	〃	75×75×9	c	9.96	×	1,880	=	74.90
〃	4	〃	〃	c	〃	×	1,760	=	70.12
〃	4	〃	〃	c	〃	×	1,130	=	45.02
〃	4	〃	〃	c	〃	×	1,070	=	42.63
〃	4	〃	〃	c	〃	×	830	=	33.07
〃	4	〃	〃	c	〃	×	810	=	32.27
〃	8	〃	〃	c	〃	×	770	=	61.35
〃	4	〃	〃	c	〃	×	750	=	29.88
Gusset & web	2	Pls	540×9	c	38.151	×	610	=	46.54
〃	2	〃	〃	c	〃	×	630	=	48.07
〃	2	〃	550×9	c	38.858	×	620	=	48.18
〃	2	〃	530×9	c	37.445	×	620	=	46.43
〃	1	Pl	465×9	c	32.852	×	570	=	18.73
〃	2	Pls	570×9	c	40.271	×	1,100	=	88.60
〃	2	〃	300×9	c	21.195	×	970	=	41.12
〃	2	〃	570×9	c	40.271	×	630	=	50.74
〃	2	〃	210×9	c	14.837	×	550	=	16.32
〃	2	〃	460×9	c	32.499	×	640	=	41.60
〃	2	〃	210×9	c	14.837	×	490	=	14.54
〃	2	〃	420×9	c	29.673	×	650	=	38.57
〃	2	〃	210×9	c	14.837	×	470	=	13.95
〃	1	Pl	410×9	c	28.967	×	740	=	21.44
End connection	4	Ls	125×90×9	c	14.6	×	520	=	30.37
〃 〃	4	〃	〃	c	〃	×	980	=	57.23
Splice	2	Pls	310×9	c	21.902	×	630	=	27.60
〃	2	〃	70×9	c	4.946	×	150	=	1.48
〃	2	〃	280×9	c	19.782	×	570	=	22.55
〃	2	Fills	120×9	c	8.478	×	570	=	9.66
〃	2	Pls	180×9	c	12.717	×	740	=	18.82
	2	〃	250×9	c	17.663	×	310	=	10.95
	16	Bar	50×9	c	3.533	×	300	=	16.96
	10	〃	70×9	c	4.946	×	300	=	14.84
									2,484.53

Sway Bracing SB3 1- Required

Top rail	4	Ls	150×90×9	c	16.4	×	4,740	=	310.94
〃 〃	2	Pls	265×9	c	18.722	×	4,080	=	152.77
Diagonal	4	Ls	90×75×9	c	11.0	×	1,110	=	48.84
〃	4	〃	〃	c	〃	×	1,120	=	49.28
〃	4	〃	〃	c	〃	×	1,170	=	51.48
Gusset	2	Pls	570×9	c	40.271	×	750	=	60.41
〃	2	〃	390×9	c	27.554	×	465	=	25.63
〃	2	〃	360×9	c	25.434	×	465	=	23.65

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CALCULATIONS FOR

Materials of Kisogawa Bashi for Gifu-Ken

Gusset	1	Pl	350 × 9	c	24.728 ×	570 =	14.09	} 945.99
Vertical	4	Ls	125×75×9	c	13.5 ×	1,080 =	58.32	
〃	4	〃	〃	c	〃 ×	1,020 =	55.08	
〃	4	〃	75×75×9	c	9.96 ×	1,000 =	39.84	
Splice	2	Pls	170 × 9	c	12.011 ×	770 =	18.50	
〃	2	〃	125 × 11	c	10.794 ×	570 =	12.31	
〃	4	〃	150 × 11	c	12.953 ×	185 =	9.59	
〃	1	Pl	310 × 11	c	26.769 ×	570 =	15.26	
Bottom rail	4	Ls	150×150×11	c	25.1 ×	4,980 =	499.99	
Gusset	2	Pls	415 × 9	c	29.320 ×	465 =	27.27	
〃	2	〃	565 × 9	c	39.917 ×	585 =	46.70	
〃	1	Pl	700 × 10	c	54.950 ×	830 =	45.61	
Diagonal	4	Ls	90×75×9	c	11.0 ×	1,365 =	60.06	
Vertical	4	〃	125×75×9	c	13.5 ×	1,290 =	69.66	
〃	4	〃	75×75×9	c	9.96 ×	1,860 =	74.10	
〃	4	〃	〃	c	〃 ×	1,760 =	70.12	
End connection	4	〃	125×90×9	c	14.6 ×	475 =	27.74	
〃 〃	4	〃	〃	c	〃 ×	500 =	29.20	
								1,896.44

SB2 2- Required

Same part for SB3						=	945.99	
Bottom rail	4	Ls	150×150×11	c	25.1 ×	5,100 =	512.04	
Gusset	1	Pl	510 × 10	c	40.035 ×	830 =	33.23	
〃	2	Pls	440 × 9	c	31.086 ×	465 =	28.91	
〃	2	〃	570 × 9	c	40.271 ×	585 =	47.12	
Vertical	4	Ls	125×75×9	c	13.5 ×	1,300 =	70.20	
〃	4	〃	75×75×9	c	9.96 ×	2,000 =	79.68	
〃	4	〃	〃	c	〃 ×	1,880 =	74.90	
End connection	4	〃	125×90×9	c	14.6 ×	490 =	28.62	
〃 〃	4	〃	〃	c	〃 ×	500 =	29.20	
Diagonal	4	〃	90×75×9	c	11.0 ×	1,375 =	60.50	
								1,910.39

SB1 2- Required

Same part for SB3						=	945.99	
Bottom rail	4	Ls	150×150×11	c	25.1 ×	5,200 =	522.08	
Gusset	1	Pl	510 × 10	c	40.035 ×	830 =	33.23	
〃	2	Pls	450 × 9	c	31.793 ×	465 =	29.57	
〃	2	〃	600 × 9	c	42.390 ×	680 =	57.65	
Vertical	4	Ls	75×75×9	c	9.96 ×	2,220 =	88.44	
〃	4	〃	〃	c	〃 ×	2,070 =	82.41	
〃	4	〃	125×75×9	c	13.5 ×	1,370 =	73.98	
Diagonal	4	〃	90×75×9	c	11.0 ×	1,420 =	62.48	
End connection	4	〃	125×90×9	c	14.6 ×	500 =	29.20	
〃 〃	4	〃	〃	c	〃 ×	560 =	32.70	
	2	Pls	180 × 9	c	12.717 ×	310 =	7.88	
								1,965.67

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CALCULATIONS FOR

Materials of Kisogawa Bashi for Gifu-ken

Strut ST1 2-Required

Rail	4	Ls	125×75×9 @	13.5 × 9,480 =	511.92		
Tie plate	1	Pl	440 × 9 @	31.086 × 490 =	15.23		
〃	2	Fills	70 × 9 @	4.946 × 490 =	4.85		
〃	4	Pls	310 × 9 @	21.902 × 440 =	38.55		
〃	8	Fills	70 × 9 @	4.946 × 310 =	12.27		
〃	2	Pls	280 × 9 @	19.782 × 440 =	17.41	872.39	
〃	4	Fills	70 × 9 @	4.946 × 280 =	5.54		
Lac. bar	28	Pls	70 × 9 @	〃 × 605 =	83.79		
washer	56	Wash.	70φ @	30.2 × 9 =	15.22		
Gusset	1	Pl	460 × 9 @	32.499 × 750 =	24.37		
〃	2	Pls	450 × 9 @	31.793 × 500 =	31.79		
〃	8	Fills	70 × 4 @	2.198 × 350 =	6.15		
End connection	4	Ls	125×75×9 @	13.5 × 1,950 =	105.30		
〃 〃	4	〃	125×90×9 @	14.6 × 530 =	30.95		
washer	20	Wash	70φ @	30.2 × 9 =	5.44		
							908.78

ST 2 2-Required

Same part for ST1				=	872.39	
End connection	4	Ls	125×90×9 @	14.6 × 490 =	28.62	
washer	20	Wash	70φ @	30.2 × 9 =	5.44	
						906.45

ST 3 2-Required

Same part for ST1				=	872.39	
End connection	4	Ls	125×90×9 @	14.6 × 475 =	27.74	
washer	20	Wash.	70φ @	30.2 × 9 =	5.44	
						905.57

Longitudinal strut LS1 2-Required

2	Ls	125×90×9 @	14.6 × 4,720	137.82	
2	〃	90×90×10 @	13.3 × 270	7.18	
1	Pl	290 × 9	20.489 × 480	9.83	
					154.83

Summary of Portal bracing, Sway bracings and strut for one span

Portal bracings	PB1	2	@ 2,484.53	=	4,969.06	
Sway 〃	SB3	1	@ 1,896.44	=	1,896.44	
〃 〃	SB2	2	@ 1,910.39	=	3,820.78	
〃 〃	SB1	2	@ 1,965.67	=	3,931.34	
Strut	ST1	2	@ 908.78	=	1,817.56	
〃	ST2	2	@ 906.45	=	1,812.90	
〃	ST3	2	@ 905.57	=	1,811.14	
〃	LS1	2	@ 154.83	=	309.66	
						20,368.88

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CALCULATIONS FOR

Materials of Kisogawa-Bashi for Gifu-Ken.

Stringers.

$S1^{R}_{L}$, S2, $S3^{R}_{L}$, $S4^{R}_{L}$, $S5^{R}_{L}$, S6 & $S7^{R}_{L}$ 1 Required for each span.

Beams	84 I^{s}	350 × 150	@	58.50	× 4.610	=	22653.54	
Conn. L^{s}	56 L^{s}	100×90 × 10	@	14.10	× 0.280	=	221.09	
〃 〃	112 L^{s}	100×90 × 10	@	〃	× 0.250	=	394.80	
								23,269.43 kg

Brackets.

$BR1^{R}_{L}$		2 Required for each end span.						
Top flange	2 L^{s}	75×75×9	@	9.96	× 0.415	=	8.27	
bottom 〃	1 L	75×75×9	@	〃	× 0.470	=	4.68	
web	1 Pl	445 × 9	@	31.44	× 0.525	=	16.51	
tension pl.	1 Pl.	160 × 9	@	11.30	× 0.530	=	5.99	
Conn. L	1 L	100×90×10	@	14.10	× 0.280	=	3.95	
								39.40
$BR2^{R}_{L}$		4 Required for each end span						
	2L^{s}	75×75×9	@	9.96	× 0.405	=	8.07	
	1L	75×75×9	@	〃	× 0.470	=	4.68	
	1Pl.	445 × 9	@	31.44	× 0.525	=	16.51	
	1Pl.	160 × 9	@	11.30	× 0.550	=	6.22	
	1L	100×90×10	@	14.10	× 0.250	=	3.53	
								39.01
$BR3^{R}_{L}$		2 and 4 required for each end and int. span respectively						
	2L^{s}	75×75×9	@	9.96	× 0.265	=	5.28	
	1L	75×75×9	@	〃	× 0.350	=	3.49	
	1Pl	375 × 9	@	26.49	× 0.445	=	11.79	
	1Pl.	160 × 9	@	11.30	× 0.530	=	5.99	
	1L	100×90×10	@	14.10	× 0.280	=	3.95	
								30.50
$BR4^{R}_{L}$		4 and 8 req'd. for each end and int. span respectively.						
	2L^{s}	75×75×9	@	9.96	× 0.255	=	5.08	
	1L	75×75×9	@	〃	× 0.350	=	3.49	
	1Pl	375 × 9	@	26.49	× 0.445	=	11.79	
	1Pl.	160 × 9	@	11.30	× 0.550	=	6.22	
	1L	100×90×10	@	14.10	× 0.250	=	3.53	
								30.11

Summary of Stringers and brackets for one span.

End span.

Stringers							23,269.43
brackets	$BR1^{R}_{L}$	2	@	39.40	=	78.80	
	$BR2^{R}_{L}$	4	@	39.01	=	156.04	
	$BR3^{R}_{L}$	2	@	30.50	=	61.00	
	$BR4^{R}_{L}$	4	@	30.11	=	120.44	
							416.28
							23,685.71 kg

Intermediate span.

Stringers							23,269.43
brackets	$BR3^{R}_{L}$	4	@	30.50	=	122.00	
	$BR4^{R}_{L}$	8	@	30.11	=	240.88	
							362.88
							23632.31 kg.

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CALCULATIONS FOR

Materials of Kisogawa-Bashi for Gifu-Ken.

Floor Beams.

FB1 2 Required.

Flange	4Ls	125×90×9	@	14.60 × 9.656	=	563.91	
〃	2 Cov. Pls.	280 ×9	@	19.78 × 6.560	=	259.51	
web	1 web pl.	1000 ×9	@	70.65 × 4.815	=	340.18	
〃	1 〃	1000 ×9	@	70.65 × 4.835	=	341.59	
							1505.19
end connections	4Ls	150×100×12	@	22.40 × 0.992	=	88.88	
〃 〃	4 Fills	215 ×9	@	15.19 × 0.820	=	49.82	
Stiffeners	12Ls	125×90×9	@	14.60 × 1.010	=	176.95	
〃	8Ls	90×90×10	@	13.30 × 1.010	=	107.46	
〃	2Ls	90×90×10	@	13.30 × 0.992	=	26.39	
Splice	2Pls	410 ×9	@	28.97 × 0.820	=	47.51	
Stringer connections	4 Fills	90 ×9	@	6.36 × 0.290	=	7.38	
〃 〃	8 〃	90 ×9	@	6.36 × 0.240	=	12.21	
							516.60 (34.3%)
							2021.79 kg.

FB2, FB3, FB4, FB5, FB6 & FB7 13 Required

Flange	2Ls	125×90×10	@	16.10 × 9.750	=	313.95	
〃	2Ls	125×90×10	@	16.10 × 9.685	=	311.86	
〃	2 Cov. pls.	300 ×10	@	23.55 × 6.930	=	326.40	
web	1 web pl.	1000 ×9	@	70.65 × 4.860	=	343.36	
〃	1 〃	1000 ×9	@	70.65 × 4.880	=	344.77	
							1640.34
end connections	4Ls	150×100×12	@	22.40 × 0.500	=	44.80	
〃 〃	4Ls	150×100×12	@	22.40 × 0.480	=	43.01	
〃 〃	4 fills	215 ×10	@	16.88 × 0.415	=	28.02	
〃 〃	4 〃	215 ×10	@	16.88 × 0.395	=	26.67	
stiffeners	12Ls	125×90×9	@	14.60 × 1.010	=	176.95	
〃	8Ls	90×90×10	@	13.30 × 1.010	=	107.46	
〃	2Ls	90×90×10	@	13.30 × 0.990	=	26.33	
splice	2Pls	410 ×10	@	32.19 × 0.820	=	52.79	
stringer connections	4 fills	90 ×10	@	7.07 × 0.290	=	8.20	
〃 〃	8 〃	90 ×10	@	7.07 × 0.240	=	13.57	
							527.80 (32.2%)
							2168.14 kg

Lateral plates

LP1.	2 Pls	980 ×9	@	69.24 × 1.210	=	167.56	
LP2	2 Pls	870 ×9	@	61.47 × 1.080	=	132.78	
LP3	3 Pls	770 ×9	@	54.40 × 0.960	=	156.67	
							457.01

Summary for Floor Beams and Lateral plates for one span.

Floor Beams	FB1	2	@	2,021.79	=	4043.58
〃 〃	FB2, FB3, FB4, FB5, FB6 & FB7.	13	@	2,168.14	=	28185.82
						32229.40
Lateral plates						457.01
						32686.41 kg.

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CALCULATIONS FOR

Materials of Kisogawa-Bashi for Gifu-Ken.

Bottom lateral bracings.

BL1^R_L 4 required.

1L 150-150×11 @ 25.10 × 5.965 = 149.72
1L 150×150×11 @ 25.10 × 6.125 = 153.74
1L 75×75×9 @ 9.96 × 0.650 = 6.47
1L 150×100×9 @ 17.10 × 0.192 = 3.28
1Pl 145 ×9 @ 10.24 × 0.165 = 1.69
314.90 kg

BL2^R_L 8 Required

1L 150×100×12 @ 22.40 × 5.965 = 133.62
1L 150×100×12 @ 22.40 × 6.125 = 137.20
1L 75×75×9 @ 9.96 × 0.680 = 6.77
1L 150×100×9 @ 17.10 × 0.192 = 3.28
1Pl. 145 ×9 @ 10.24 × 0.145 = 1.48
282.35

BL3^R_L 4 Req'd

1L 150×100×9 @ 17.10 × 5.965 = 102.00
1L 150×100×9 @ 17.10 × 6.125 = 104.74
1L 75×75×9 @ 9.96 × 0.680 = 6.77
1L 150×100×9 @ 17.10 × 0.192 = 3.28
1Pl. 145 ×9 @ 10.24 × 0.145 = 1.48
218.27

BL4^R_L 12 Required

1L 125×90×9 @ 14.60 × 5.975 = 87.24
1L 125×90×9 @ 14.60 × 6.140 = 89.64
1L 75×75×9 @ 9.96 × 0.685 = 6.82
1L 150×100×9 @ 17.10 × 0.192 = 3.28
1Pl. 140× 9 @ 9.89 × 0.145 = 1.43
188.41

Summary of Bottom lateral bracings for one span.

BL1^R_L 4 @ 314.90 = 1,259.60
BL2^R_L 8 @ 282.35 = 2,258.80
BL3^R_L 4 @ 218.27 = 873.08
BL4^R_L 12 @ 188.41 = 2,260.92
6,652.40 kg

Top lateral bracings.

TL1^R_L 4@2 = 8Ls 150×90×9 @ 16.40 × 6,380 = 837.06
TL2^R_L 4@2 = 8Ls 〃 @ 〃 × 6,290 = 825.25
TL3^R_L 4@2 = 8Ls 〃 @ 〃 × 6.220 = 816.06
TL4^R_L 4@2 = 8Ls 〃 @ 〃 × 6.175 = 810.16
TL5^R_L 4@2 = 8Ls 〃 @ 〃 × 6.135 = 804.91
TL6^R_L 4@2 = 8Ls 〃 @ 〃 × 6.135 = 804.91
Summary for one span = 4,898.35 kg

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CALCULATIONS FOR

Materials of Kisogawa Bashi for Gifu-Ken

Rivet head for one span

Main truss

22ϕ shop rivet 75,630 @ 0.074 = 5,596.62
" field rivet 40,710 @ 0.074 = 3,012.54
19ϕ field rivet 240 @ 0.048 = 11.52
8620.68

Portal bracings, Sway bracings, struts, and top lateral bracings

19ϕ shop rivet 6,180 @ 0.048 = 296.64
" field rivet 3,970 @ 0.048 = 190.56
487.20

Bottom lateral bracings

22ϕ field rivet 1,220 @ 0.074 = 90.28
19ϕ shop rivet 1,360 @ 0.048 = 65.28
" field rivet 120 @ 0.048 = 5.76
161.32

Stringer and floor beam

22ϕ field rivet 1,350 @ 0.074 = 99.90
19ϕ shop rivet 23,150 @ 0.048 = 1,111.20
" field rivet 3,220 @ 0.048 = 154.56
1,365.66

Bracket for End span

19ϕ shop rivet 330 @ 0.048 = 15.84
" field rivet 100 @ 0.048 = 4.80
20.64

Bracket for Intermediate span

19ϕ shop rivet 270 @ 0.048 = 12.96
" field rivet 100 @ 0.048 = 4.80
17.76

Summary of rivet heads

For end span	10,655.50 Kg.
For intermediate span	10,652.62 Kg.

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CALCULATIONS FOR

Materials of Kisogawa Bashi for Gifu-Ken

Roller shoe (one set) 2-Required

Top casting	RS1		1 @	400.00 =	400.00	
Bed plate	BP1		1 @	327.00 =	327.00	
Roller	RN1		5 @	85.20 =	426.00	
Pin	PN1		1 @	84.00 =	84.00	
plate		2 @ 80 × 12 @ 7.536 ×		.620 =	9.34	
Dust Guard	DG1	2 @ 163 × 9 @ 11.516 ×		.926 =	21.33	
〃 〃	DG2		2 @	32.10 =	64.20	
Pin	P1		6 @	0.15 =	.90	
〃	P2		4 @	0.16 =	.64	
Tapped bolt	6φ × 25		10 @	0.01 =	.10	
〃 〃	22φ × 50		4 @	0.28 =	1.12	
〃 〃	12φ × 40		4 @	0.06 =	.24	
anchor bolt	42φ × 800		4 @	10.80 =	43.20	
washer		4 @ 100 × 10 @ 7.850 ×		100 =	3.14	
						1,381.21

Fixed shoe (one set) 2-Required

Shoe	FS1		1 @	545.00 =	545.00	
Pin	PN1		1 @	84.00 =	84.00	
anchor bolt	42φ × 800		4 @	10.80 =	43.20	
washer		4 @ 100 × 10 @ 7.850 ×		100 =	3.14	
						675.34

Summary of shoe

Roller shoe	2 @	1,381.21 =	2762.42	
Fixed shoe	2 @	675.34 =	1350.68	
				4,113.10

Expansion Joint Required for all span

EJ 1 6-Required

1	L	75 × 75 × 9	@ 9.96 ×	10.010 =	99.70	
1	CH. Pl	255 × 9	@ 18.016 ×	10.000 =	180.16	
1	Pl	275 × 9	@ 19.429 ×	9.000 =	174.86	
2	Pls	335 × 9	@ 23.668 ×	.350 =	16.57	
6	Ls	75 × 75 × 9	@ 9.96 ×	.210 =	12.55	
6	Wash.	70 × 10	@ 5.495 ×	.160 =	5.28	
5	Bolt	16φ × 500		@ 0.89 =	4.45	
12	〃	12φ × 45		@ 0.11 =	1.32	
						494.89

EJ 2 6-Required

1	L	150 × 90 × 9	@ 16.4 ×	10.010 =	164.16	
1	Pl	275 × 9	@ 19.429 ×	9.000 =	174.86	
1	〃	50 × 10	@ 3.925 ×	10.010 =	392.89	
2	Pls	335 × 9	@ 23.668 ×	.390 =	18.46	
6	Ls	75 × 75 × 9	@ 9.96 ×	.210 =	12.55	
5	Bolt	16φ × 500		@ 0.89 =	4.45	
12	〃	12φ × 45		@ 0.11 =	1.32	
6	Wash.	70 × 10	@ 5.495 ×	.160 =	5.28	
						773.97

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CALCULATIONS FOR

Materials of Kisogawa Bashi for Gifu-Ken

EJ 3 1-Required

1	L	75 × 75 × 9 @	9.96 × 9,290 =	92.53
1	CH.Pl	245 × 9 @	17.309 × 9,270 =	160.45
1	Pl	275 × 9 @	19.429 × 9,000 =	174.86
6	Ls	75 × 75 × 9 @	9.96 × 210 =	12.55
6	Wash.	70 × 10 @	5.495 × 160 =	5.28
5	Bolt	16φ × 500	@ 0.89 =	4.45
12	〃	12φ × 45	@ 0.11 =	1.32
				451.44

EJ 4 1-Required

1	L	125 × 75 × 9 @	13.5 × 9,310 =	125.69
1	〃	75 × 75 × 9 @	9.96 × 9,020 =	89.84
1	Pl	335 × 9 @	23.668 × 9,020 =	213.49
2	Ls	75 × 75 × 9 @	9.96 × 226 =	4.50
2	〃	〃 @	〃 × 247 =	4.92
2	〃	〃 @	〃 × 274 =	5.46
2	〃	〃 @	〃 × 292 =	5.82
2	〃	〃 @	〃 × 305 =	6.08
2	〃	〃 @	〃 × 311 =	6.20
6	Wash.	70 × 10 @	5.495 × 160 =	5.28
6	Is	150 × 125 @	36.2 × 300 =	65.16
12	Bolt	12φ × 45	@ 0.11 =	1.32
5	〃	16φ × 340	@ 0.63 =	3.15
5	Wash	70 × 9 @	4.946 × 70 =	1.73
				538.64

Expansion metal EM1 2-Required (cast iron)
weight = 15.00

Summary for all spans

EJ1	6 @ 494.89	=	2,969.34
EJ2	6 @ 773.97	=	4,643.82
EJ3	1 @ 451.44	=	451.44
EJ4	1 @ 538.64	=	538.64
EM1	2 @ 15.00	=	30.00
			8,633.24

19φ Shop rivet	4,780 @ 0.048	=	229.44
12φ 〃 〃	1,170 @ 0.016	=	18.72
			248.16
			8,881.40

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CALCULATIONS FOR

Materials of Kisogawa Bashi for Gifu-Ken

Summary of structural steel in one span

For End span

Main trusses	165,092.40		
Portals, sway & struts	20,368.88		
stringers & brackets	23,685.71		
Floor beams	32,686.41		
Bottom laterals	6,652.40		
Top laterals	4,898.35		
Rivet heads	10,655.50		(4.04%)
		264,039.65	
shoes		4,113.10	
			268,152.75

For Intermediate span

Main trusses	165,092.40		
Portals, sway & struts	20,368.88		
stringers & brackets	23,632.31		
Floor beams	32,686.41		
Bottom laterals	6,652.40		
Top laterals	4,898.35		
Rivet heads	10,652.62		
		263,983.37	
Shoes		4,113.10	
			268,096.47

Grand summary for the entire bridge.

For structural steel

End spans	2	c	264,039.65	=	528,079.30
Intermediate Spans	5	c	263,983.37	=	1,319,916.85
					1847,996.15
Expansion metals, including rivet heads.					8,881.40
					1856,877.55 Kgs
					or 1856.878 Kg tons
For Cast steel shoes and forged steel pins and rollers.	7	c	4,113.10	=	28,791.70 Kgs
					or 28.792 Kg tons
Total					1,885.670 Kg tons

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CALCULATIONS FOR

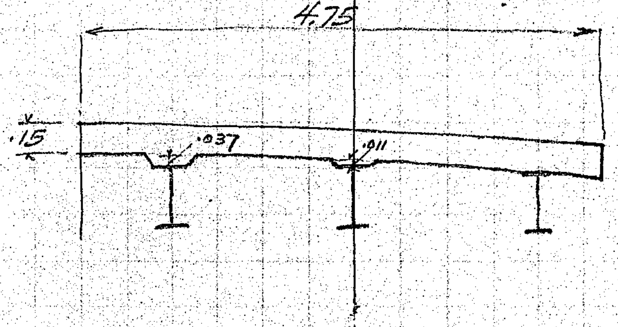
Materials of Kisogawa-Bashi for Gifu-Ken.

Deck construction.

Concrete 1:2:4 mixture.

Cross section of slab

Slab	0.150 × 9.500	= 1.425
fillet	0.190 × 0.037 × 2	= 0.014
〃	0.160 × 0.011 × 2	= 0.004
		1.443 sq. meters.



Fillets on floor beams.

average 0.350 × 0.08 × $\frac{2}{3}$ × 9.0 = 0.168 cub. m. on one beam.

Drain holes concrete to be reduced.

0.18 × 0.25 × 0.15 = 0.007 cub. m. for one drain.

Total length of slab.

Slab no. 1.	1	@ 14.26	=	14.26
〃 no. 2	21	@ 13.94	=	292.74
〃 no. 3	6	@ 10.11	=	60.66
〃 no. 4	6	@ 13.91	=	83.46
〃 no. 5	1	@ 10.24	=	10.24
				461.36 meters.

Total volume of concrete.

461.36 × 1.443 = 665.74
0.168 × 105 = 17.64
−0.007 × 84 = (−) 0.59
682.79 cub. meters.

Reinforcements, plain bars (see drawing) 61.172 kg tons.

Forms.

Total width of form.

Bottom of slab.			9.50
Ends 〃 〃	2 @ 0.15	=	0.30
fillets	4 @ .037	=	0.15
〃	4 @ .011	=	0.04
Top of stringers less	6 @ .150	=	−0.90
			9.09 meters

Total length of form.

Total length of slab		=	461.36
top of floor beams	105 @ .30	=	− 31.50
fillets on 〃 〃	210 @ .05	=	10.50
			440.36 meters

Total area of form.

440.36 × 9.09 = 4002.87
drain hole 84 × 0.86 × 0.15 = 10.84
4013.71 sq. meters.

Pavement. 6.5 cm granolithic.

Total length of pavement 461.36
28 @ .01 = 0.28
461.64 meters

Total area of pavement 461.64 × 9.00 = 4154.76
drain holes 84 × .255 × .355 = − 7.60
4147.16 sq. meters

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CALCULATIONS FOR

Materials of Kisogawa-Bashi for Gifu-ken

Cast iron drains with steel trough, 84 sets required.

Construction joints 28 joints - 9^{mm} Elastite fillings.

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CALCULATIONS FOR

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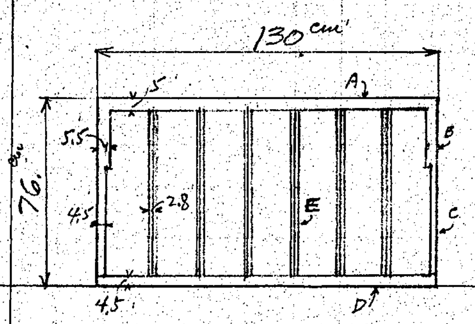
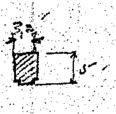
Materials of Kisogawa Bashi for Gifu-Ken.

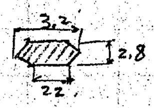
Handrail.

Cast iron grate.
HG1. & HG2.

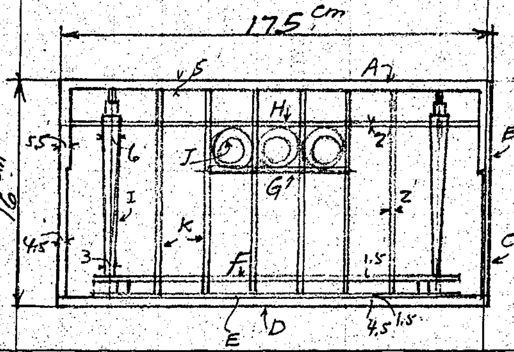


Part A. 5.0 × 3.2 × 130.0 = 2080
B. 5.5 × 3.2 × 25.0 × 2 = 880
C 4.5 × 3.2 × 41.5 × 2 = 1195
D 4.5 × 5.0 × 130.0 = 2925
E 2.8 × 2.7 × 66.5 × 6 = 3016
Corners less 0.1 × 0.1 × 750.0 = − 8
10088 cm^3 @ 0.00725 = 73.14 kg



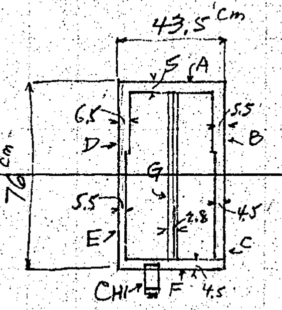


HG3 & HG4

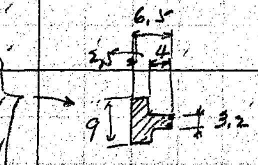


Part A 5.0 × 3.2 × 175.0 = 2800
B 5.5 × 3.2 × 25.0 × 2 = 880
C 4.5 × 3.2 × 41.5 × 2 = 1195
D 4.5 × 5.0 × 175.0 = 3938
E 3.0 × 1.5 × 137.0 = 617
F 3.0 × 1.5 × 137.0 = 617
G 3.0 × 2.0 × 15.0 × 3 = 270
H 3.0 × 2.0 × 152.0 = 912
I 4.5 × 3.0 × 58.0 × 2 = 1566
J 3.0 × 2.0 × 40.0 × 3 = 720
K 3.0 × 2.0 × 61.5 × 6 = 2214
corners less 0.1 × 0.1 × 930.0 = − 9
15720 cm^3 @ 0.00725 = 113.97 kg

HG5

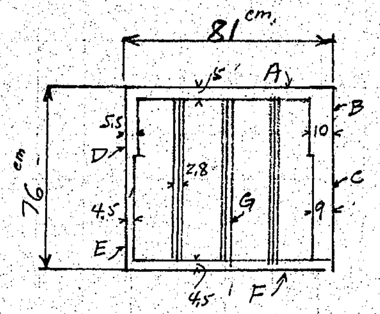


Part A 5.0 × 3.2 × 43.5 = 696.
B 5.5 × 3.2 × 25.0 = 440
C 4.5 × 3.2 × 41.5 = 598
D 4.0 × 3.2 × 25.0 = 320
E 3.0 × 3.2 × 41.5 = 398
D 9.0 × 2.5 × 25.0 = 563
E 9.0 × 2.5 × 41.5 = 934
F 4.5 × 5.0 × 43.5 = 979
G 2.8 × 2.7 × 66.5 = 503
Corners less 0.1 × 0.1 × 200.0 = − 2
5429 cm^3 @ 0.00725 = 39.36 kg



Chair CH1. 10.0 × 5.0 × 16.0 = 800. @ 0.00725 = 5.80 kg
45.16

HG6



Part A 5.0 × 3.2 × 81.0 = 1296.
B 10.0 × 3.2 × 25.0 = 800
C 9.0 × 3.2 × 41.5 = 1195
D 5.5 × 3.2 × 25.0 = 440
E 4.5 × 3.2 × 41.5 = 598
F 4.5 × 5.0 × 81.0 = 1823
G 2.8 × 2.7 × 66.5 × 3 = 1508
7660 cm^3 @ 0.00725 = 55.54 kg

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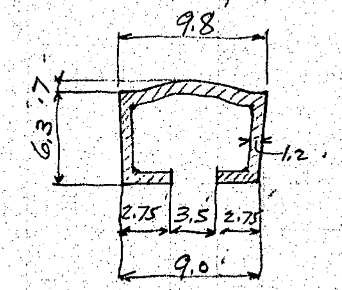
CALCULATIONS FOR

Materials of Kisogawa-Bashi for Gifu-Ken

Cast iron top rails.



Cross section
10.0 × 1.2 = 12.00
2.75 × 1.2 × 2 = 6.60
3.90 × 1.2 × 2 = 9.36
27.96 cm²

tie 10.0 × 1.0 × 6.6 = 66.00 cm³

TR1.
27.96 × 140 = 3914
tie 2 @ 66 = 132
4046 cm³ @ 0.00725 = 29.33 kg

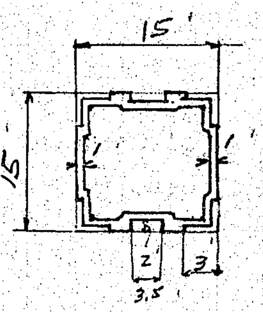
TR2.
27.96 × 185 = 5173
tie 3 @ 66 = 198
5371 cm³ @ 0.00725 = 38.94 kg

TR3
27.96 × 190 = 5312
tie 3 @ 66 = 198
5510 @ 0.00725 = 39.95 kg

TR4
27.96 × 227.5 = 6361
tie 4 @ 66 = 264
6625 @ 0.00725 = 48.03 kg

Cast iron posts,
HP1 & HP2.

Cross section



15.0 × 15.0 = 225.0
less 7.0 × 0.6 × 2 = − 8.4
〃 11.8 × 3.15 × 2 = − 74.3
〃 5.5 × 9.0 = − 49.5 } 132.2
〃 5.4 × 0.6 × 4 = − 13.0
〃 3.5 × 2.0 × 2 = − 14.0
65.8 cm²

post 65.8 × 85.0 = 5593.
notch less 10.0 × 10.0 × 1 × 2 = − 200
footing 22.0 × 22.0 × 1.2 = 581
less 132.2 × 1.2 = − 159
say 80.0 × 7.5 × 1.0 = 600
Cap. 16.5 × 19.0 × 1.0 = 314
〃 67.0 × 6.0 × 1.0 = 402
7131 cm³

grout hole
3.5 × 10 × 1.0 = 35 cm³
Exp. hole.
3.5 × 75 × 1.0 = 263 cm³
298

For HP1 7131 − 35 × 2 = 7061 @ 0.00725 = 51.19 kg
〃 HP2 7131 − 298 = 6833 @ 0.00725 = 49.54 〃

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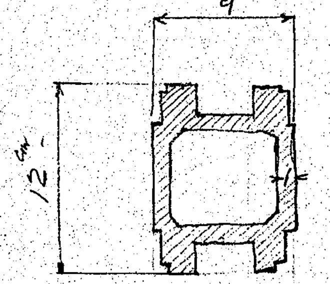
CALCULATIONS FOR

Materials of Kisogawa-Bashi for Gifu-Ken.

Cast iron post
HP3.



Cross section
12.0 × 9.0 = 108.00
less. 3.3 × 0.6 × 4 = − 7.92
〃 3.5 × 2.0 × 2 = − 14.00
〃 6.0 × 7.0 = − 42.00
44.08 cm²

Post. 44.08 × 79.0 = 3482
footing 18.0 × 15.0 × 1.2 = 324
less 6.0 × 7.0 × 1.2 = − 50
say 62.0 × 7.5 × 1.0 = 465
4,221 cm³
less 10.0 × 3.5 × 1.0 = − 35
4186 @ 0.00725 = 30.35 kg.

Anchor casting
AC1 & AC2.

8.0 × 11.0 = 88.0
less 6.0 × 9.0 = 54.0
34.0 × 36.0 = 1224
18.0 × 15.0 × 1.5 = 405
less 6.0 × 9.0 × 1.5 = − 81
324
1548 cm³

for AC1 1548 @ 0.00725 = 11.22 kg.

〃 AC2 4.5 × 10.0 × 1 = −45
1548
1503 @ .00725 = 10.90 kg

AC3.
5.0 × 6.0 = 30.0
3.0 × 4.0 = −12.0
18.0 cm²
18.0 × 36.0 = 648
18.0 × 15.0 × 1.5 = 405
3.0 × 4.0 × 1.5 = − 18
1035 cm³
1035 × 0.00725 = 7.50 kg

Anchor bolts

4 bolts (12 mm φ × 0.15) @ 0.24 = 0.96 (double nuts)
2 bars 60 × 9 × 0.15 @ 4.24 = 1.27
2.23 kg

Summary of Handrail castings.

		Reqd. no	Piece wt.	Total weight
Top rail	TR1	376	29.33	11,028.08
〃	TR2	196	38.94	7,632.24
〃	TR3	4	39.95	159.80
〃	TR4	12	48.03	576.36
Posts	HP1	198	51.19	10,135.62
〃	HP2	12	49.54	594.48
〃	HP3	392	30.35	11,897.20
Grates	HG1	208	73.14	15,213.12
〃	HG2	184	〃	13,457.76
〃	HG3	104	113.97	11,852.88
〃	HG4	92	〃	10,485.24
〃	HG5	4	45.16	180.64
〃	HG6	12	55.54	666.48
Anchor casting	AC1	198	11.22	2,221.56
〃	AC2	12	10.90	130.80
〃	AC3	392	7.50	2,940.00
Anchor bolts + washers		602 sets	2.23	1,342.46
				100,514.72 kg or 100.515 kg tons.

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CALCULATIONS FOR

Materials of Kisogawa-Bashi for Gifu-Ken

Coping stones Granite blocks

	Reqd. no	Section	length	vol./piece	Total volume
C1	1176	0.35m × 0.19m	× 0.47	0.0313	36.81
C2	784	〃 〃	× 0.46	0.0306	23.99
C3	2	〃 〃	× 0.55	0.0366	0.07
C4	2	〃 〃	× 0.50	0.0333	0.07
C5	24	〃 〃	× 0.39	0.0259	0.62
					61.56 cub meters

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CALCULATIONS FOR

Materials of Kisogawa-Bashi for Gifu-Ken.

Light pedestal and handrail on Wing.
Granite 北木島産

Light pedestal.	Req'd. no	Dimensions	vol./block	Total volume.	
Base layer	2	0.15 × 0.22 × 0.80	0.0264	0.053	
"	4	0.22 × 0.30 × 0.70	0.0462	0.185	
Layers A. & B	12	0.24 × 0.385 × 0.46	0.0425	0.510	
" "	12	0.15 × 0.385 × 0.46	0.0266	0.319	
" "	6	0.15 × 0.385 × 0.48	0.0277	0.166	
" C	4	0.125 × 0.30 × 0.46	0.0173	0.692	
" "	4	0.125 × 0.30 × 0.40	0.0150	0.060	
" "	2	0.125 × 0.30 × 0.48	0.0180	0.036	
" D.	4	0.25 × 0.25 × 0.50	0.0313	0.125	
" E & F	28	0.20 × 0.385 × 0.40	0.0308	0.862	
" "	14	0.15 × 0.385 × 0.40	0.0231	0.323	
Top layer	4	0.20 × 0.40 × 0.455	0.0364	0.146	
" "	2	0.15 × 0.40 × 0.455	0.0273	0.055	
					3.532
End post.					
Base layer	2	0.15 × 0.22 × 0.40	0.0132	0.026	
" "	4	0.20 × 0.22 × 0.40	0.0176	0.070	
Layers A & B.	12	0.20 × 0.335 × 0.40	0.0268	0.322	
" "	6	0.15 × 0.335 × 0.40	0.0201	0.121	
Top layer	6	0.125 × 0.20 × 0.40	0.0100	0.060	
					0.599
Handrail:					
Top rail	1	0.39 × 0.45 × 0.74	0.1299	0.130	
"	2	0.39 × 0.45 × 0.68	0.1193	0.239	
"	1	0.39 × 0.45 × 0.685	0.1202	0.120	
"	1	0.39 × 0.45 × 0.515	0.0904	0.090	
"	1	0.39 × 0.45 × 0.57	0.1000	0.100	
Bottom rail	1	0.26 × 0.45 × 0.73	0.0854	0.085	
"	2	0.26 × 0.45 × 0.68	0.0796	0.159	
"	1	0.26 × 0.45 × 0.685	0.0801	0.080	
"	1	0.26 × 0.45 × 0.515	0.0602	0.060	
"	1	0.26 × 0.45 × 0.56	0.0655	0.066	
Top rail corner	1	0.39 × 0.45 × 0.45	0.0790	0.079	
Bottom " "	1	0.26 × 0.45 × 0.45	0.0527	0.053	
Posts.	2	0.26 × 0.36 × 0.67	0.0627	0.125	
"	1	0.36 × 0.42 × 0.67	0.1013	0.101	
"	1	0.62 × 0.62 × 0.67	0.2575	0.258	
					1.745

Summary for 1 set = 5.876 cub. meters
with 7 lead dove-dails

Concrete 1:2:4 mixture.
Total volume of light pedestal.
1.40 × 1.40 × 1.50 = 2.940
0.80 × 0.80 × 3.65 = 2.336
5.276

" " " End post
0.80 × 0.80 × 1.35 = 0.864
6.140

Volume of concrete
6.140
Granite for light pedestal, less −3.532
" " end post " −0.599
2.009 call this 2.01 cub. meters

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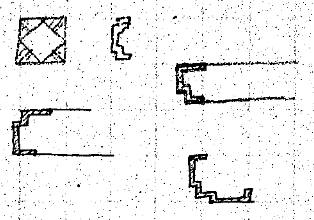
CALCULATIONS FOR

Materials for Kisogawa Bashi for Gifu-Ken

Ornamental bronze. 1-set.

Ornaments 4 - 40cm × 0.35 × 40. = 2240
top 4 - 15 × 0.2 × 90. = 1.080
bottom 4 - 15 × 0.2 × 90 = 1080
posts 4 - 19 × 0.2 × 38 = 578
4978 cm³



weight of bronze = 4978 @ 0.00815 = 40.57 kg
Specify at 40 kg.

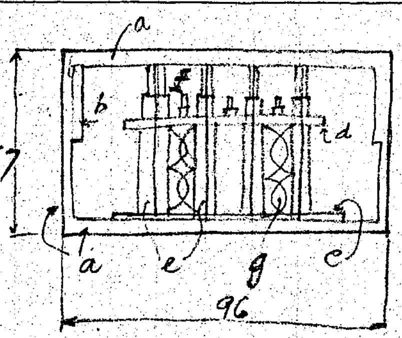
Bronze name plate.
1 - 18cm × 0.35 × 50cm = 315 cm³
weight of bronze = 315 @ 0.00815 = 2.57 kg.
Specify at 2.5 kg.

Reinforcements
8 - 16φ × 5.40m @ 1.58 = 68.26
11 - 9φ × 2.10 @ 0.499 = 11.53
79.79 kg or 0.080 kg ton.

Gas pipe for wiring.
for Light pedestal 1½"φ main pipe with 4 - 1"φ branch pipes.
″ end post 1½"φ main pipe.

Cast iron grate

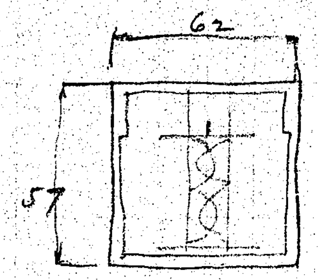
AG1.



Part a. 9.0 × 4.0 × 290.0 = 10,440
b 9.0 × 1.0 × 22.0 × 2 = 396
c 8.4 × 1.5 × 67.0 = 844
d 8.0 × 2.0 × 63.0 = 1,008
e. 7.0 × 5.0 × 30.0 × 4 = 4200
″ 6.0 × 4.0 × 6.5 × 4 = 624
″ 5.0 × 3.0 × 9.0 × 4 = 540
f 4.0 × 2.0 × 6.5 × 3 = 156
g 0.5 × 3.0 × 76.0 × 3 = 342
″ 1.5 × 3.0 × 80.0 × 2 = 720
19270 cm³

weight of grate AG1 19270 @ 0.00725 = 139.71 kg

AG2.



Part a 9.0 × 4.0 × 222.0 = 7,992
b 9.0 × 1.0 × 22.0 × 2 = 396
c 8.4 × 1.5 × 34.0 = 428
d 8.0 × 2.0 × 30.0 = 480
e 7.0 × 5.0 × 30.0 × 2 = 2,100
″ 6.0 × 4.0 × 6.5 × 2 = 312
″ 5.0 × 3.0 × 9.0 × 2 = 270
f 4.0 × 2.0 × 6.5 × 1 = 52
g 0.5 × 3.0 × 76.0 = 114
″ 1.5 × 3.0 × 80.0 = 360
12,504 cm³

weight of grate AG2 12,504 @ 0.00725 = 90.65 kg.

Summary.
2 - AG1 @ 139.71 = 279.42
1 - AG2 = 90.65
370.07 kg or 0.370 kg ton.

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CALCULATIONS FOR

Materials of Kisogawa-Bashi for Gifu-ken

Electric lighting.
Lamp on light pedestal.
Top lamp. 1 reqd. for 1 pedestal.
weight of bronze.

Top. 8 × 28.0 × 31.0 ÷ 2 " = 3472 cm^2
" 8 × 10.0 × 28.0 " = 2240 "
" 8 × 4.0 × 20.0 " = 640 "
post 8 × 20.0 × 34.0 " = 5440 "
bottom 8 × 17.0 × 30.0 " = 4080 "
misc say 128 "
16000 " cm^2

Average thickness assumed 0.2 cm
volume 16000 × 0.2 " = 3200 " cm^3
weight of bronze 3200 × 0.00815 = 26.08 kg
specify at <u>26</u> kg

note:
Head lamp over end post same as for this lamp.

8 – 5 mm 摺硝子 (両面)
1 – 60 watt lamp.

Side lamps. 4 required for one pedestal.

Top 1 × 12.0 × 35.0 " = 420 "
" 1 × 20.0 × 8.0 " = 160 "
" 1 × 25.0 × 1.5 " = 38 "
" 1 × 50.0 × 9.0 " = 450 "
" 1 × 75.0 × 6.0 " = 450 "
" 4 × 12.0 × 20.0 " = 960 "
posts 4 × 16.0 × 60.0 " = 3840 "
bottom 4 × 6.0 × 20.0 " = 480 "
" 1 × 60.0 × 8.0 " = 480 "
" 8 × 5.0 × 12.0 " = 480 "
" 8 × 5.0 × 35.0 " = 1400 "
" 8 × 11.0 × 12.0 " = 1056 "
misc detail 236 "
10450 " cm^2

Average thickness assumed 0.20 cm.
volume = 10450 × 0.2 " = 2090 " cm^3
Total weight of bronze = 2090 " @ 0.00815 " = 17.03 kg
Specify at <u>17</u> kg

4 – 3 mm curved glass (両面摺硝子)
1 – 60 watt lamp.

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CALCULATIONS FOR

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Materials of Kisogawa Bashi for Gifu-ken

Lamps over spans. 42 required (= 7 spans @ 6 lamps).

Top.	1 × 16 × 40.0	=	640 cm²
〃	1 × 20 × 11.0	=	220
〃	1 × 36 × 1.5	=	54
〃	1 × 65 × 10.0	=	650
〃	1 × 95 × 6.0	=	570
〃	4 × 12 × 25.0	=	1200
Posts	4 × 20 × 75.0	=	6000
bottom	4 × 8 × 25.0	=	800
〃	1 × 80 × 11.0	=	880
〃	1 × 40 × 15.0	=	600
〃	1 × 30 × 85.0	=	2550
〃	1 × 60 × 12.0	=	720
〃	1 × 30 × 12.0	=	360
〃	1 × 10 × 10.0	=	100
〃	1 × 40 × 100.0	=	4000
misc			156
			19,500 cm²

average thickness 0.20 cm
volume 19500 × 0.2 = 3900 cm³
Total weight of bronze = 3900 @ 0.00815 = 31.8 kg
Specify at 32 kg.

4 – 3mm curved glass ([illegible])
1 – 60 watt lamps.

増田淳氏関係資料
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