JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

CALCULATIONS FOR

MADE BY _____ DATE _____ FILE NO

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No.1057

昭和十一年十二月

東京市第二水道擴張計畫

檜村橋設計計算書

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR

橋村橋

For transportation of construction materials direct to Ogōchi dam site which is planned to increase capacity of water supply to growing city of Tokyo, a private road to the dam site shall be built from one kilometer west of Kinawa sta., Ōtaki, Tokyo Prefecture. The bridge is to be built as first bridge on the route across stream of well known Tamagawa and such type has been adopted to suit the circumstances of site. The main span is 72.6 meters in length of spandrel braced arch type with 11 meter approach girder spans on both sides, and the total length of bridge proper is 95.6 meters to the returning ends of girder abutments. Clear roadway 6.0 meter wide between curb lines paved with 3cm mortar on reinforced concrete floor slab.

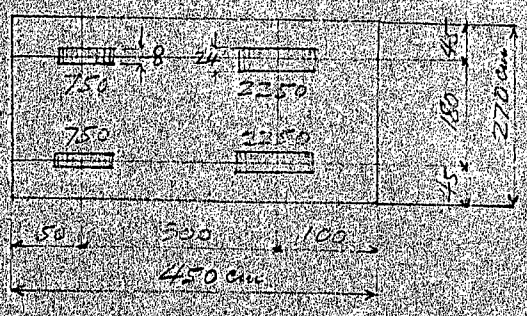
The arch span will have a rise of 11.5 meter at center to give the good harmony to deep valley and skilful banks on both sides. Arch pier built on rock foundation and girder abutments on sound soil. Railings made of cast iron and entrance pairs of artificial stone. Both approaches 7.0 meter clear gravel road. Here are the general features of this bridge.

Assumed Loadings:

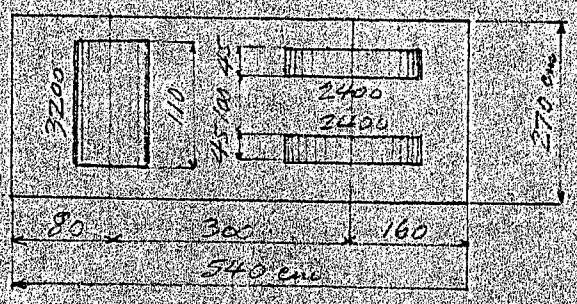
Uniform load $w = \frac{100.000}{170+L} \leqq 500$ kg/m²

where w = uniform load in kg/m²
L = span length in meter.

6 ton motor truck loading



8 ton road roller.



Two rows of motor truck traffic on roadway with occupied width of 270 cm, unoccupied space around the motor truck shall be filled with uniform load specified above, one road roller on one span.

Impact for motor truck loading $\text{Coef} = \frac{20}{60+L}$ where L = loaded length in meter
max impact 30%

Seismic acceleration assumed 3000 mm/sec² Temperature change ± 30° C

Allowable working Unit

Structural steel & reinforcing bars		Concrete 1:2:4 mixture	
Tension net	1200 kg/cm²	Direct compression	35 kg/cm²
Extreme fibre stress net	1200 "	Fibre stress due to bending	45 "
Shear at web gross	900 "	Combined stress direct and bending	35 "
Compression member		Punching shear	9 "
$1500\left(1-0.00555\frac{l}{r}\right)$ not over 1000	"	Shear of plain concrete	4 "
l = length of member in cm		Bearing	45 "
r = least radius of gyration			
Compression flange of girder		For combined wind or temperature stress with dead, live and impact stresses, the allowable working stress shall be increased 25%.	
$1200\left(1-0.017\frac{l}{b}\right)$ not over 1100 kg/cm²			
Shear on shop rivet 850 kg/cm²		Increase of earthquake increase unit stress by 60%.	
" " field rivet 750 "			
Bearing on shop rivet 1700 "			
" " field rivet 1500 "			
" " pin 1800 "			

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

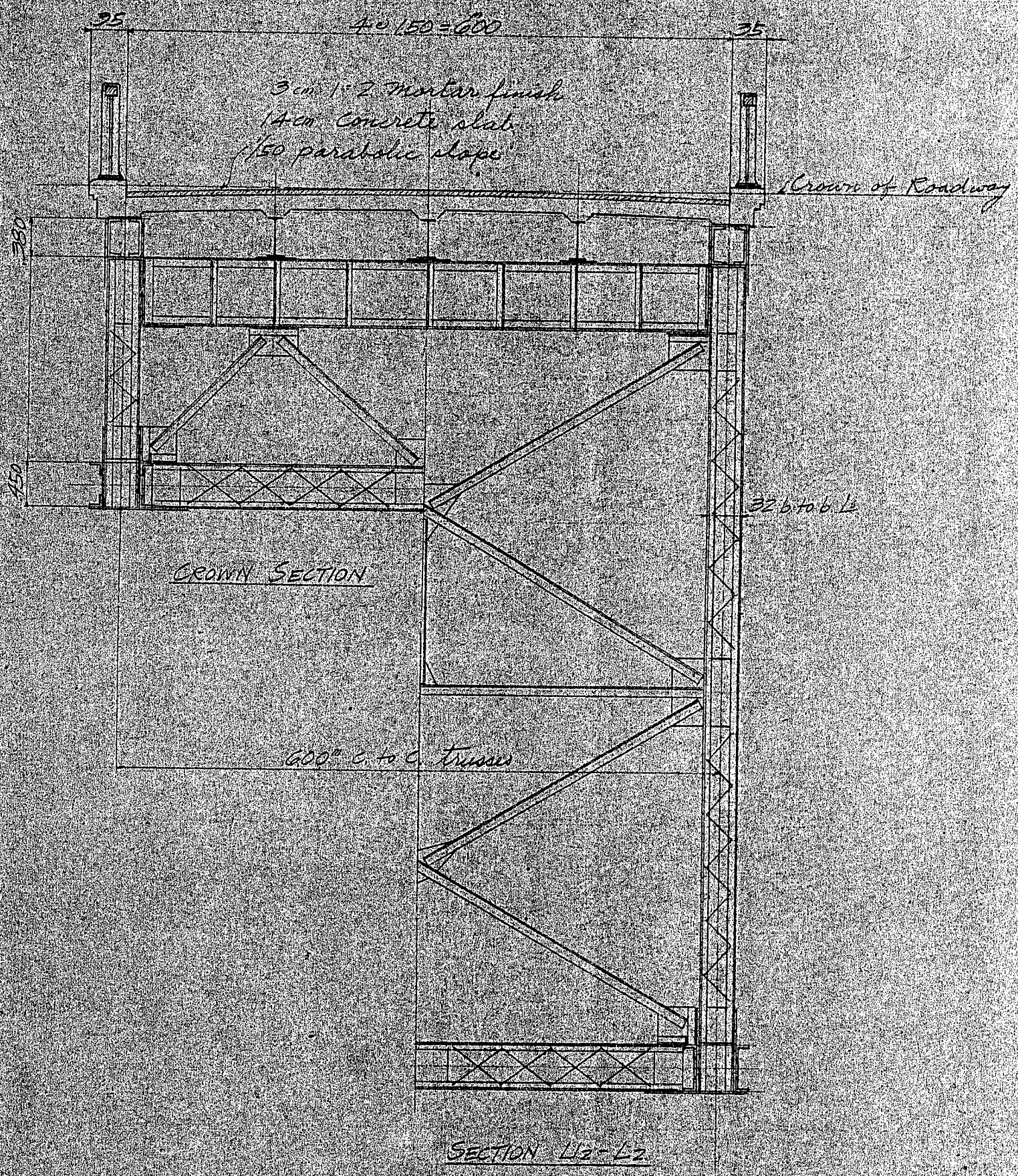
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CONSULTING ENGINEER
GOTANDA, TOKYO

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CALCULATIONS FOR

檜村橋

Cross section of the bridge



Scale 1:50

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CONSULTING ENGINEER
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CALCULATIONS FOR

増村橋

Floor slab Span length 150 meters

Dead load

3 cm mortar finishment c 17 = 51
14 cm concrete slab c 24 = 336
edge 8
395 kg/m²

Dead load moment = 1/10 × 395 × 150² = 89 t-m
Dead load shear = 1/2 × 395 × 150 = 296 kg

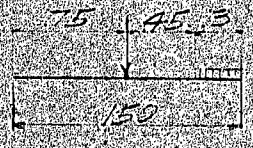
Live load motor truck rear wheel concentration 2250
impact 30% 675
3,925 kg

Distribution of wheel concentration on slab

longitudinal distribution a contact between wheel + pavement 20
distribution 2 : 3 6
26 cm

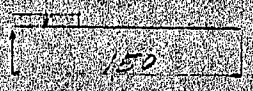
transverse distribution b = 24 + 6 = 30 cm
effective width e = 2/3 l + a
= 2/3 × 150 + 26 = 126 m

Moment



load per meter strip 2975 ÷ 126 = 2320 kg
Uniform load 500 kg per square meter
moment as a simple beam 2320/2 × 50/2 = 870 kg-m
50 × 0.3² / 2 × 150 × 0.75 = 11
881 kg-m
as continuous beam 881 × 0.8 = 705 kg-m

Max end shear



reaction 2320 × 135/150 = 2090 kg

Summary of moment and shear

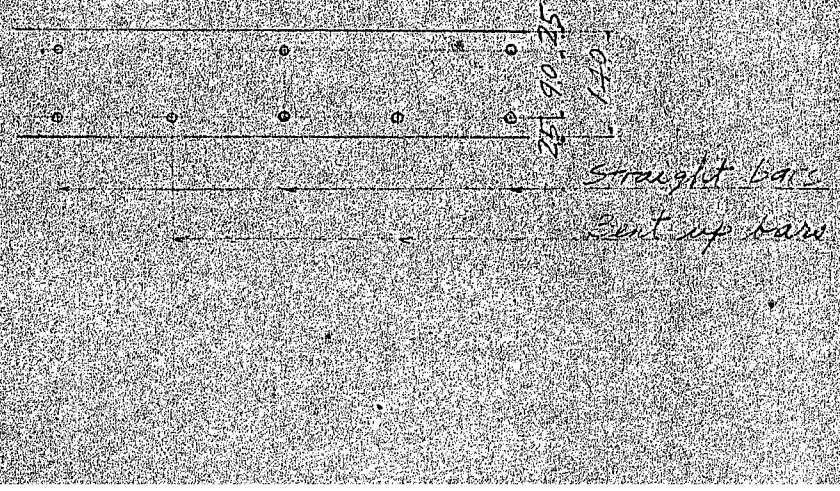
	moment	shear
Dead load	89	296
Live load	705	2090
	794 kg-m	2386 kg

Effective depth required for slab f_s = 1200 kg/cm²
f'_c = 45 "
$R = \frac{M}{bd^2}$ $d = \sqrt{\frac{M}{bR}}$ where R = 7.13
$d = \sqrt{\frac{794 \times 100}{100 \times 7.13}} = 10.6$ cm

use 12 cm slab; Insulation at bottom 2.5 cm Effective depth 11.5 cm
steel area required $\frac{794 \times 100}{7/8 \times 1200 \times 11.5} = 6.6$ cm² per meter strip

use 12 mm bars 15 cm spacing = 7.54 cm² per meter strip

unit shear $\frac{2386}{100 \times 7/8 \times 11.5} = 24$ kg/cm²



増田淳氏関係資料
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JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO

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CALCULATIONS FOR

檜村橋

Design of I beam stringer: span length 520 meters spacing 150 meters

Dead load

Floor and pavement 395 · 150 = 592
I-beam assumed 60
652 kg per meter

Dead load moment 1/8 · 652 · 520² = 2205 t-m
Dead load shear 1/2 · 652 · 520 = 1695 kg

Live load:

Motor truck rear wheel concentration with impact 2925 kg
Unit " " 975 "

Max. load on stringer

105 + 105 + 150 + 150

2925 · 2 · 105/150 = 4100 kg
Uniform live load 500 · 150 = 750 kg/m

Reaction

Max. wheel concentration 1/2 · 4100 = 2050
Uniform load 750 · 160²/(2 · 520) = 185
2235 kg

Moment 2235 · 260 = 5810 kg-m

Max. end shear

10 + 420 + 750 + 520

750 × 420²/(2 · 520) = 1270 ... uniform load
4100 ... motor truck
5370 kg

Summary of maximum end shear

	Moment	Shear
Dead load	2205	1695
Live load	550	5370
	2015 t-m	7065 kg

Steel members

Section modulus req'd = 2015 × 100/1100 = 729 cm³
use I beam 350 · 150 · 585 kg
section modulus 870.6 cm³

Floor members

Section modulus assumed as uniform load 50
Dead load moment 220
10
930 kg per m

Dead load moment as a simple beam 1/8 · 930 · 520² = 3145 kg-m

Live load

45 + 105 + 150 + 150 + 10 + 160 + 375 + 520

Reaction wheel concentration on floor member 2925 · 105/150 = 2050 kg
Uniform load 500 · 075 = 375 kg/m

Reaction due to uniform load 375 · 160²/(2 · 520) = 92 kg
moment due to motor truck 1/4 · 2050 · 520 = 2665
92 · 260 = 240
2905 kg-m

Summary of moment

	moment as a simple beam	3/4 moment
Dead load	3145	2360 kg-m
Live load	2905	
	6050 kg-m	4540 kg-m

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JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO

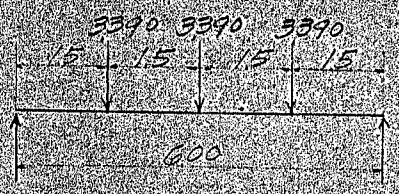
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CALCULATIONS FOR

檜村橋

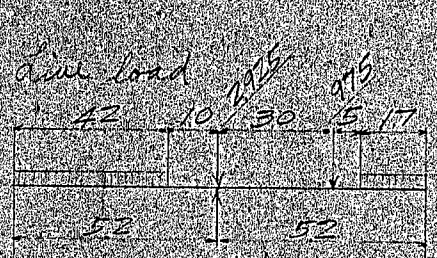
Beam of intermediate floor beam, span length 600 meters, spacing 5.20 meters

Dead load



Concentration from stringer 6.52 × 5.20 = 33.90 t
Moment at center 1.5 × 33.90 × 3.00 = 152.50
33.90 × 1.50 = 50.80
101.70 t-m
Beam assumed as 1.35 t/m $\frac{1.35 \times 6.0^2}{8}$ = 6.10
107.80 t-m

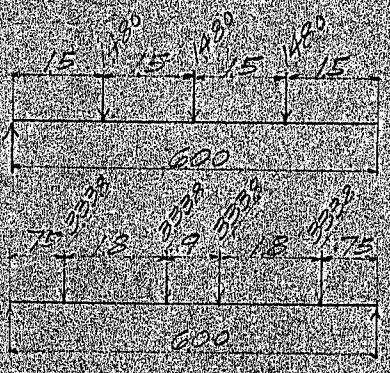
Dead load



Stringer concentration 1.5 × 33.90 = 50.85
Beam weight 12 × 1.35 × 6.0 = 4.05
54.90 t

Live load

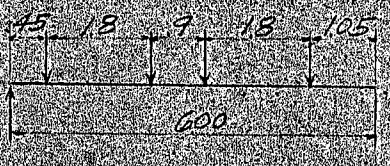
2 motor truck on roadway, front and rear filled with uniform load
Motor truck rear wheel concentration with impact 29.25 t
" front " " " " 9.75
Uniform load on axles of motor truck neglected
motor truck loading rear wheel 29.25
" front " 9.75 × $\frac{2.2}{5.2}$ = 4.13
33.38 t



Uniform load $\frac{5.00 \times 1.72}{2 \times 5.20}$ = 1.39
$\frac{5.00 \times 4.2^2}{2 \times 5.20}$ = 8.48
9.87 t per meter
Concentration on stringer 9.87 × 1.50 = 14.80 t
Moment due to uniform load 1.5 × 14.80 × 3.00 = 66.60
14.80 × 1.50 = -22.20
44.40 t-m
Moment due to wheel concentration 20 × 33.38 × 3.00 = 200.30
33.38 × 2.25 = -75.00
33.38 × 0.45 = -15.00
110.30 t-m

Summary of live load moment
Uniform load 14.40
wheel concentration 110.30
125.70 t-m

Shear end values



Uniform load 1.5 × 14.80 = 22.20
motor truck wheel concentration 4 × 33.38 × $\frac{3.32}{6.00}$ = 73.40
= 33.38 × $\frac{14.5}{5.0}$ = 73.40
169.00 t

Summary of moments & shears

	Moment	Shear
Dead load	107.80	54.90
Live load	125.70	72.20
	262.50 t-m	127.10 t

Web assumed 650 × 9 = 585 cm²; 1/3 web = 73 cm²
Back to back of L's 660 mm
Effective depth 660 − 2 × 17.1 = 625.8 cm
flange stress $\frac{262.50}{0.626}$ = 419.50 t
Required section 419.50 ÷ 1.200 = 350 cm²
1/3 web area 73
277 cm² net

Use 2 L's 125 × 75 × 9 = 3438 − 396 = 3042 cm²

Approximate weight of intermediate floor beam

web plate	1 P	650 × 9	45.92 × 5.68	= 261
flange angles	4 L's	125 × 75 × 9	135 × 5.68	= 307
stiffeners	14 L's	90 × 75 × 9	11.0 × 6.6	= 102
end stiffeners	4 L's	150 × 100 × 9	17.1 × 6.4	= 44
filler	4 Pls	145 × 9	10.24 × 5.0	= 20
	3 Pls	220 × 9	20.49 × 4.0	= 35
Rivet heads and misc.				21
				750 kg

789 ÷ 5.68 = 137 kg per linear meter

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（独立行政法人 土木研究所蔵）

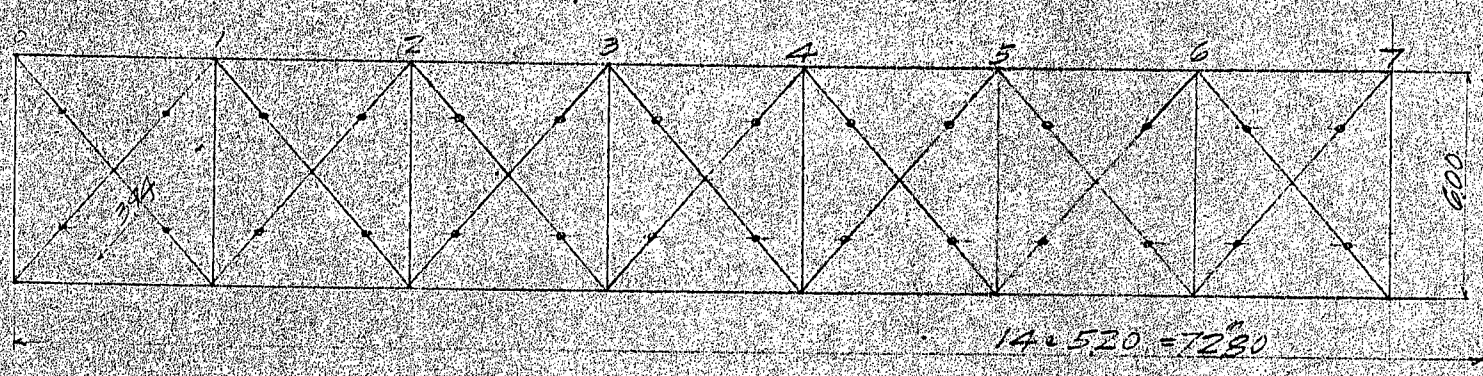
JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO

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CALCULATIONS FOR

増村橋

Design of Top Lateral bracing



Diagonal length
$600^2 = 3600$
$520^2 = 2704$
6304
7.940m

$\sec\theta = \frac{794}{600} = 1323$ $\tan\theta = \frac{520}{600} = 0.867$

Wind load assumed 445 kg per linear meter, Panel load $445 \cdot 520 = 2,315$ kg

Diagonal stress

Panel				shear				stresses			
0-1	$\frac{2135}{14}$	· 91	=	13880	· 1.323	=	18370	÷ 1200	=	153	cm² net
1-2	"	· 78	=	11900			15750			131.2	
2-3	"	· 66	=	10070			13320			111.0	
3-4	"	· 55	=	8390			11,100			92.5	
4-5	"	· 45	=	6860			9080			75.6	
5-6	"	· 36	=	5490			7,260			60.5	
6-7	"	· 28	=	4270			5,650			47.1	

Stresses during earthquake, Acceleration assumed 3000 mm/sec² or $k = 0.30$

Dead load

Floor and pavement	600	⊂ 395	= 2,370
Coping	2	⊂ 369	= 538
Handrail	2	⊂ 85	= 170
Fairing	2	⊂ 50	= 100
Stringer	3	⊂ 60	= 180
Floor beam	780	: 520	= 150
Top lateral bracing	say		87
bottom lateral bracing	say		100
strut, sway and portal bracings			220
main truss assumed			1515
			5430 kg per lin. meter

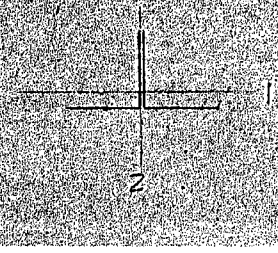
Dead panel load on top chord assumed as $\frac{2}{3} \cdot 5430 \cdot 520 = 18820$ kg
" " bottom chord " " $\frac{1}{3} \cdot 5430 \cdot 520 = 9410$ kg

Seismic force $18820 \cdot 0.3 = 5640$ kg for top chord
$9410 \cdot 0.3 = 2820$ kg for bottom chord

Diagonal stress tension & compression

Panel			shear			stress		Section required		used 19φ rivet	
0-1	$\frac{5640}{14}$	· 91	= 36670	× $\frac{1323}{2}$	=	24200	÷ 1230	= 126 cm² net	197 cm² gross	8	
1-2		78	= 31420			20800		108	169	8	
2-3		66	= 26600			17600		92	143	6	
3-4		55	= 22200			14690		77	119	6	
4-5		45	= 18130			12000		63	97	6	
5-6		36	= 14500			9595		50	78	6	
6-7		28	= 11280			7465		39	61	6	

Section for diagonal



$\angle L$ 75·75·9 = 2538 ÷ 792 = 174.6 cm² net

$\Gamma = 2.25$ $\lambda/i = \frac{197}{2.25} = 87.6$ allowable stress 770 × 1.6 = 1232 kg/cm²

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
(OTSUKA, TOKYO)

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CALCULATIONS FOR

檜村橋

Steel stress due to earthquake

Panel

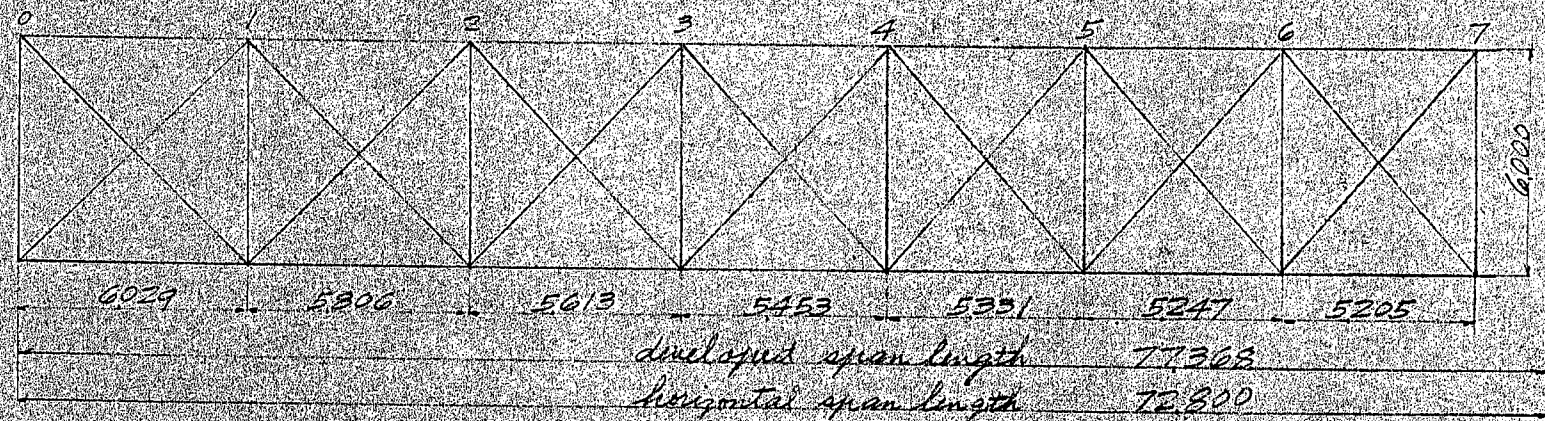
0-1 5640 × 65 × 0.867 = 31,800 kg
1-2 " × 120 " = 58,700
2-3 " × 165 " = 80,700
3-4 " × 200 " = 97,500
4-5 " × 225 " = 110,000
5-6 " × 240 " = 117,300
6-7 " × 245 " = 119,800

Approximate weight of top lateral bracing.

Dead load 28½ × 75 × 75 × 7 c 9.96 kg ÷ 7250 = 2020
" 56½ " " " " 3350 = 1870
Details and rivet heads 480
4370 kg

4370 ÷ 73.80 = 60 kg per linear meter

Design of bottom lateral bracing.



Panel	Panel length 6000			diagonal length	Coefficient	summary of lever arm
0-1	6025	= 3635 + 3600	= 7235	851 ÷ 600	= 1.418	502.892
1-2	5806	= 3371 "	= 6971	835	= 1392	431.553
2-3	5613	= 3151 "	= 6751	822	= 1370	336.020
3-4	5453	= 2974 "	= 6574	811	= 1352	306.100
4-5	5331	= 2842 "	= 6442	803	= 1338	251.633
5-6	5247	= 2753 "	= 6353	797	= 1328	202.497
6-7	5205	= 2709 "	= 6309	794	= 1323	158.608

Wind load assumed 245 kg per linear metre, panel load 245 × 5.20 = 1275 kg

Diagonal stress

Panel			shear	coef.	diagonal stress		
0-1	1275 ×	502.892 = 77368	= 8290 ×	1.418	= 11,760 kg	÷ 12.00 =	980 cm² net
1-2	" ×	431.553 "	= 7,120 ×	1392	= 9910		826
2-3	" ×	366020 "	= 6,040 ×	1,370	= 8270		689
3-4	" ×	306.100 "	= 5050 ×	1352	= 6830		569
4-5	" ×	251.633 "	= 4150 ×	1338	= 5550		463
5-6	" ×	202.497 "	= 3340 ×	1328	= 4440		370
6-7	" ×	158.608 "	= 2620 ×	1323	= 3470		289

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO

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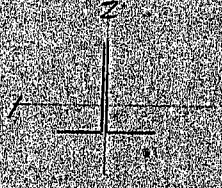
CALCULATIONS FOR

檜村橋

Seismic force ; 2820 kg for each panel

diagonal stresses

panel			shear		diagonal stress				19φ rivet use
0-1	2820 × 502892 ÷ 77368	=	18350 × 1418 ÷ 26020	=	1920	=	1355 kg/cm² net	7.7	8
1-2	" × 431553 "	=	15740 × 1392 ÷ 21900				1141	5.4	8
2-3	" × 366020 "	=	13350 × 1370 ÷ 18300				953	5.4	5
3-4	" × 306100 "	=	11170 × 1352 ÷ 15100				787	4.4	6
4-5	" × 251633 "	=	9170 × 1338 ÷ 12280				649	3.6	6
5-6	" × 202497 "	=	7380 × 1328 ÷ 9800				512	2.9	6
6-7	" × 158608 "	=	5790 × 1323 ÷ 7660				399	2.3	6



use 2L 150×90×9 = 4198 − 792 = 3396 cm²

r_1 = 4.75 max. value of $\ell/r_1 = \frac{251}{4.75} = 179$

r_2 = 3.18 " $\ell/r_2 = \frac{4255}{3.18} = 134$

Bottom chord stresses due to earthquake

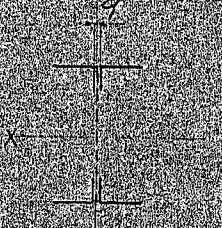
Panel			Panel load		stress
0-1	6.5 × 6029	=	39.2 × 2820 ÷ 6000	=	18420 kg
1-2	" × 11335 − 5806	=	71.1 " "	=	33420
2-3	" × 17448 − 17032	=	9.4 " "	=	45300
3-4	" × 22901 − 33391	=	11.55 " "	=	54250
4-5	" × 28232 − 54715	=	12.90 " "	=	60600
5-6	" × 33479 − 80950	=	13.66 " "	=	64200
6-7	" × 38684 − 112180	=	13.92 " "	=	65400

Approximate weight of bottom lateral bracing.

Diagonals	56 ls	150×90×9	= 16.4 kg × 350	=	6750
Center Conn.	14 p/s		= 80	=	1120
Rivet heads				=	80
					7950 kgs

7950 ÷ 7280 = 1.09 kg per linear meter

Bottom strut 4ls 90×75×9 for all panel points



Least radius of gyration required 600 ÷ 150 = 4.0 cm

Radius of gyration about Y axis = 4.23 cm

Approximate weight of strut

4ls 90×75×9 = 11.0 kg × 5.30 = 233

Details say 97

330 kg

330 ÷ 520 = 64 kg per linear meter

Sway bracing.

Panel 7



use 1L 90×90×10 least radius of gyration 1.71 cm

ℓ/r = 192 ÷ 171 = 1.12

Approximate weight of sway bracing 7

4ls 90×90×10 = 13.3 kg × 1850 = 93

details 107

200 kg

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JIUN MASUDA
CONSULTING ENGINEER
GOBANDA, TOKYO

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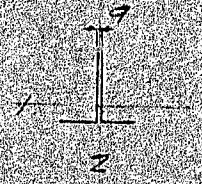
CALCULATIONS FOR

楡村橋

Panel 6

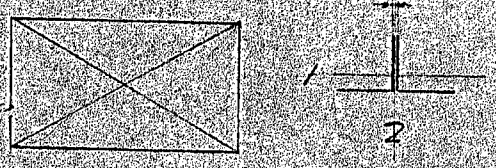
use 1L 90·90·10
$\ell/r = 209 \div 1.71 = 122$
Approximate weight of sway bracing 6
4 L 90·90·10 @ 13.3 kg · 1.87 = 99
details + rivet heads 121
220 kg

Panel 5



use 2 L 90·75·9 $r_1 = 2.78$, $r_2 = 3.30$
$\ell/r = 263 \div 2.78 = 95$
Approximate weight of sway bracing 5
8 L 90·75·9 @ 11.0 kg · 2.400 = 211
details + rivet heads 109
320 kg

Panel 4



use 2 L 90·90·10 $r_1 = 2.71$, $r_2 = 4.05$
$\ell/r_2 = 652 \div 4.05 = 161$
$\ell/r_1 = 326 \div 2.71 = 120$
Approximate weight of sway bracing 4
4 L 90·90·10 @ 13.3 kg · 6.10 = 325
details + rivet heads 115
440 kg

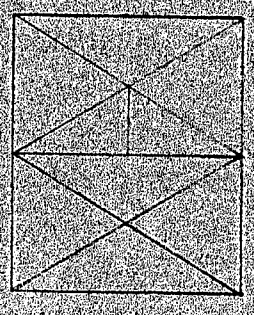
Panel 3

use 2 L 90·90·10
$\ell/r_2 = 748 \div 4.05 = 185$, $\ell/r_1 = 374 \div 2.71 = 138$
Approximate weight of sway bracing 3
4 L 90·90·10 @ 13.3 kg · 7.20 = 383
details + rivet heads 107
490 kg

Panel 2

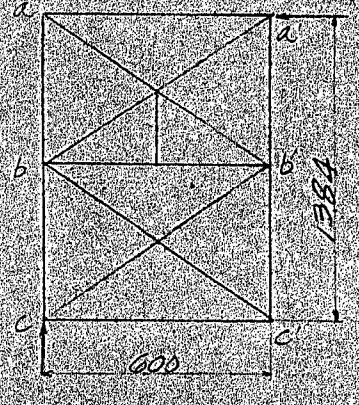
use 2 L 125·90·9
$\ell/r_2 = 899 \div 5.87 = 153$, $\ell/r_1 = 450 \div 2.6 = 173$
Approximate weight of sway bracing 2
2 L 125·90·9 @ 14.6 kg · 8.80 = 257
4 L 125·90·9 @ 14.6 · 4.35 = 254
details + rivet heads 139
650 kg

Panel 1



use 2 L 125·90·9
$\ell/r_2 = 737 \div 5.87 = 126$ $\ell/r_1 = 369 \div 2.6 = 142$
Approximate weight of sway bracing 1
8 L 125·90·9 @ 14.6 kg · 7.70 = 830
2 L 125·90·9 @ 14.6 · 5.50 = 161
2 L 75·75·9 @ 9.96 · 2.20 = 44
details + rivet heads 205
1240 kg

Portal bracing



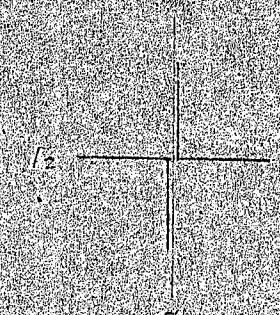
Transverse thrust on portal bracing due to
wind load on truss span $\frac{1}{2}$ @ 445 · 72.80 = 16200
girder span $\frac{1}{2}$ · 600 · 10.65 = 3200
19400 kg
Seismic load on truss span 7 @ 5,640 = 39480
girder span (assumed) 6,600
46080 kg
diagonal length
$600^2 = 36000$
$692^2 = 47886$
$83886 - 916^m$

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
OCHANOMIZ, TOKYO

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CALCULATIONS FOR 增村橋



Reaction Rc 46080 × 1394 ÷ 600 = 106300 kg
Stress a-b' 46080 × 916 ÷ 600 = 70300 kg T
" b-b' 46080 " C
" b'-c 70300 " T
Use 2L 130×130×9 = 4548 - 792 = 3756 cm²
Radius of gyration I1 = 5.71 cm, I2 = 5.29 cm
ℓ/r_1 = 835 ÷ 5.71 = 146
ℓ/r_2 = 552 ÷ 5.71 = 97
Required sectional area of a-b' & b'-c
70300 ÷ 1920 = 36.6 cm² net
Required sectional area of b-b', allowable stress 775 kg/cm²
46080 ÷ (693×16) = 4.16 cm²
Approximate weight of portal bracing

8 L 130×130×9	179 kg	7.85	= 1,125
2 L 130×130×9	179 "	5.35	= 192
2 L 75×75×9	9.96 "	2.75	= 55
Details & rivet heads			978
			2350 kg

Total weight of sway and portal bracing

Panel point			
7	1 @	200	= 200
6	2 @	220	= 440
5	2 @	320	= 640
4	2 @	440	= 880
3	2 @	490	= 980
2	2 @	650	= 1300
1	2 @	1,240	= 2480
0	2 @	2350	= 4700
			11,620 kg

11,620 ÷ 72.80 = 160 kg per linear meter

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
OTANDA, TOKYO.

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CALCULATIONS FOR

檜村橋

Design of main Truss

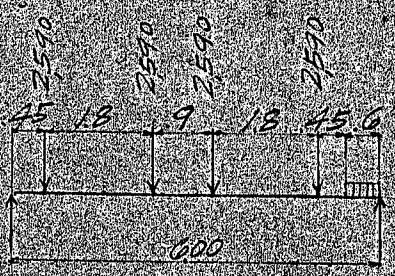
Dead load

Floor and pavement	6.00 × 395 =	2370
Coping	2 c 209 =	538
handrail	2 c 85 =	170
wering	2 c 50 =	100
stringer	3 c 60 =	180
floor beam	730 ÷ 520 =	150
Top lateral bracing		60
bottom lateral bracing		107
struts		64
sway and portal bracing		160
main truss assumed		1809
		5510 kg per linear meter

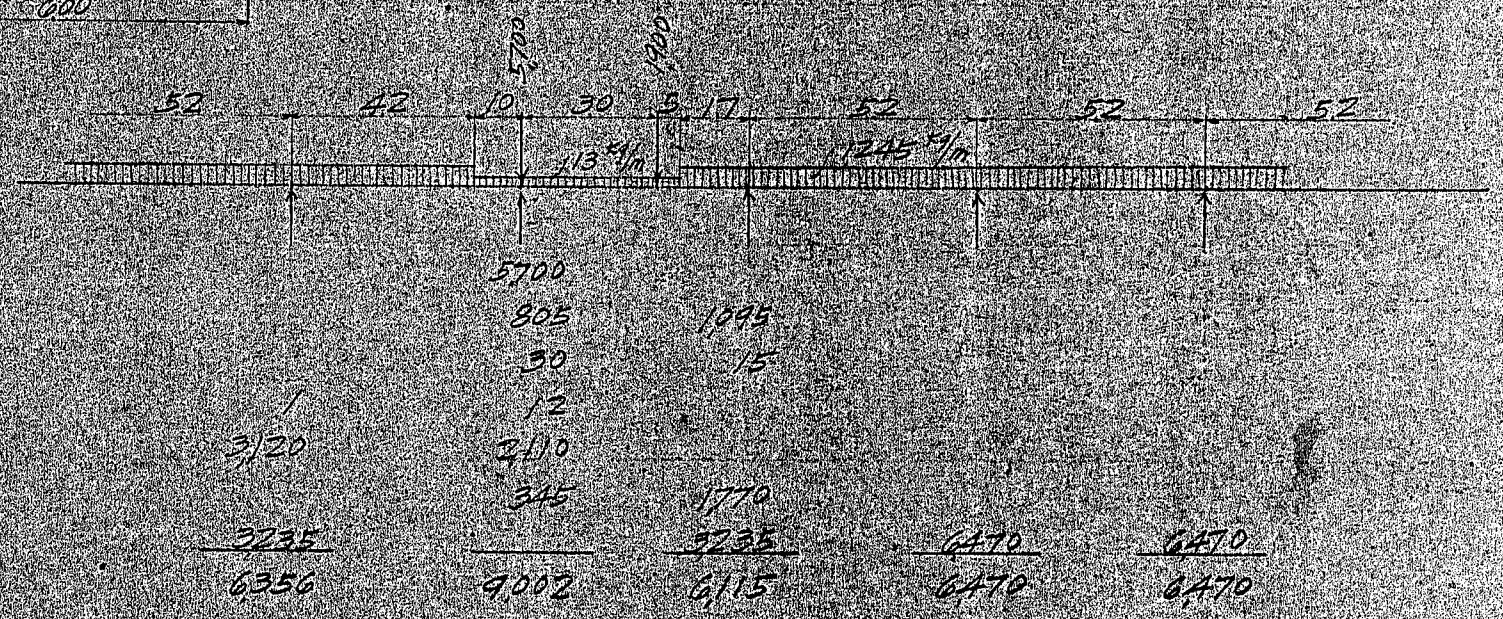
for one truss 5510 ÷ 2 = 2755 kg per linear meter
Dead panel load 2755 × 5.20 = 14320 kg for one truss

Live Load

uniform load on roadway w = 400000 / 170+728 = 412 kg/m² call 415 kg per sq. m.
motor truck rear wheel 2250
Impact coefficient 20 / 60+728 = 15.1% 340
2590 kg
Front wheel concentration with impact 2590 ÷ 3 = 860 kg
Live load concentration on main truss
full uniform load 300 × 415 = 1245 kg per linear meter
unif. load on side of truck 60×415 = 250 " " "



Rear wheel concentration 4 c 2590 × 330/600 = 5700 kg
Front wheel concentration 1/3 × 5700 = 1900 kg
uniform load on side of truck 250 × 30/600 = 13 kg per lin. meter



Let us assume the above live panel load as follows;
6470 kg throughout and
2500 kg extra concentration at the panel point giving the max. for member in question.

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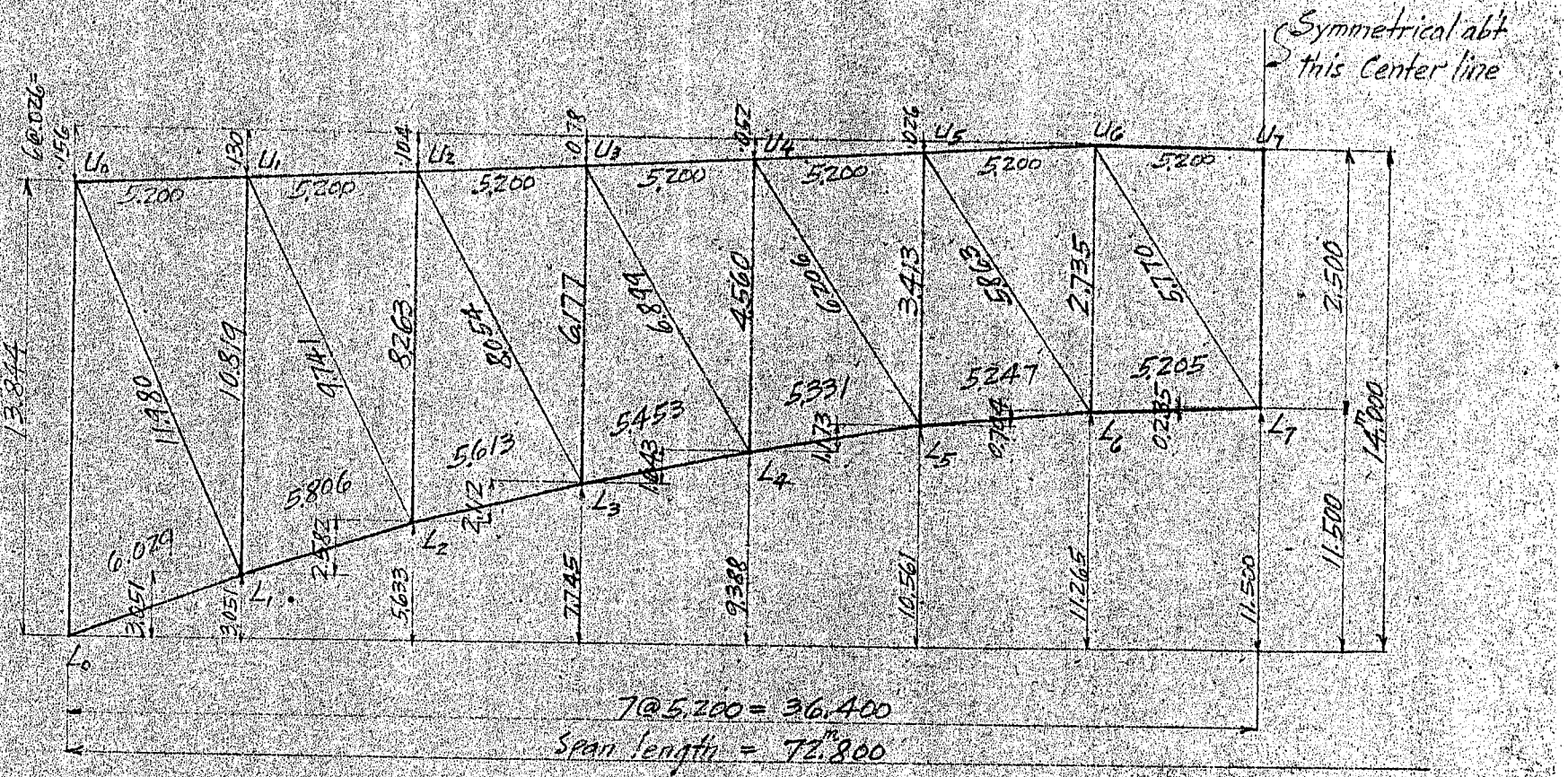
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CONSULTING ENGINEER
AOTANDA, TOKYO.

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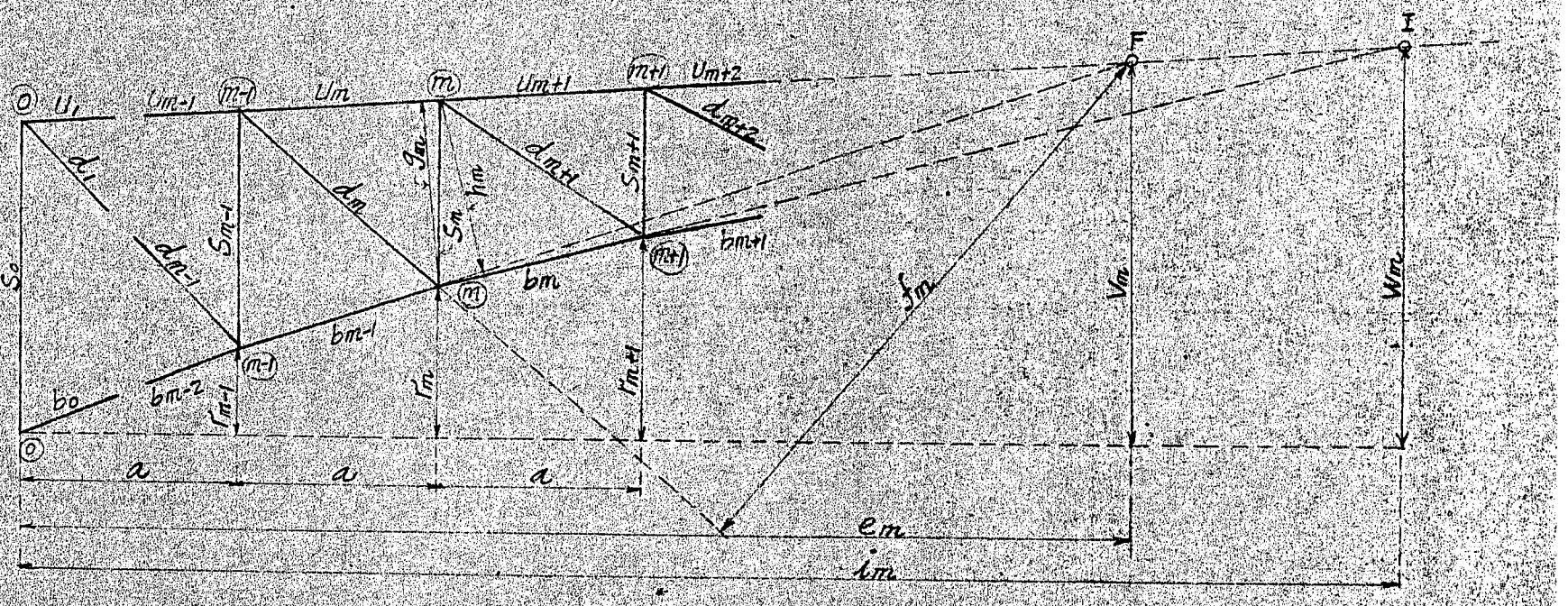
CALCULATIONS FOR

增村橋

Dimensions of the Arch Truss.



Moment arm for each member.



General Equation of moment arms:

$$g_m = \frac{a}{d_m} \cdot s_m \qquad e_m = \frac{s_{m-1}}{s_{m-1} - s_m} \cdot a + (m-1)a$$

$$h_m = \frac{a}{b_m} \cdot s_m \qquad i_m = \frac{s_m}{(r_{m+1} - r_m) - (y_m - y_{m-1})} \cdot a + ma$$

$$V_m = \frac{s_{m-1}}{s_{m-1} - s_m} (r_m - r_{m-1}) + r_{m-1} \qquad W_m = \frac{s_m}{(r_{m+1} - r_m) - (y_m - y_{m-1})} (r_{m+1} - r_m) + r_m$$

$$f_m = \frac{s_{m-1}}{s_{m-1} - s_m} \cdot \frac{a}{d_m} \cdot s_m \qquad \text{where } y_m = r_m + s_m$$

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR
檜村橋

Numerical calculation of force arm.

$$g_m = \frac{a}{d_m} \cdot S_m$$

m	a	d_m	S_m	g_m
1	5.200	5.200	10.819	10.819 m
2	"	"	8.263	8.263
3	"	"	6.177	6.177
4	"	"	4.560	4.560
5	"	"	3.413	3.413
6	"	"	2.735	2.735
7	"	"	2.500	2.500

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

m	a	b_m	S_m	$a S_m$	h_m
0	5.200	6.029	13.844	71.9888	11.940 m
1	"	5.806	10.819	56.2588	9.690
2	"	5.613	8.263	42.9676	7.655
3	"	5.453	6.177	32.1204	5.890
4	"	5.331	4.560	23.7120	4.448
5	"	5.247	3.413	17.7476	3.382
6	"	5.205	2.735	14.2220	2.732

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

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	13.844	10.819	3.025	3.051	0	3.051	42.2280	13.962	13.962 m
2	10.819	8.263	2.556	5.633	3.051	2.582	27.9347	10.929	13.980
3	8.263	6.177	2.086	7.745	5.633	2.112	17.4515	8.366	13.977
4	6.177	4.560	1.617	9.388	7.745	1.643	10.1488	6.276	14.021
5	4.560	3.413	1.147	10.561	9.388	1.173	5.3489	4.663	14.051
6	3.413	2.735	.678	11.265	10.561	.704	2.4028	3.544	14.105
7	2.735	2.500	.235	11.500	11.265	.235	.6427	2.735	14.000

$$f_m = \frac{S_{m-1}}{S_{m-1} - S_m} \cdot \frac{a}{d_m} \cdot S_m \quad \text{where } a = 5.200$$

m	S_{m-1}	$a S_{m-1}$	S_m	$a S_{m-1} S_m$	$S_{m-1} - S_m$	d_m	$(S_{m-1} - S_m) d_m$	f_m
1	13.844	71.9888	10.819	778.8468	3.025	11.980	36.2395	21.492 m
2	10.819	56.2588	8.263	464.8665	2.556	9.741	24.8970	18.672
3	8.263	42.9676	6.177	265.4109	2.086	8.054	16.8006	15.798
4	6.177	32.1204	4.560	146.4690	1.617	6.899	11.1557	13.130
5	4.560	23.7120	3.413	80.9291	1.147	6.206	7.1183	11.369
6	3.413	17.7476	2.735	48.5397	0.678	5.863	3.9751	12.211
7	2.735	14.2220	2.500	35.5550	0.235	5.770	1.3560	26.221

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

m	S_{m-1}	$a S_{m-1}$	$S_{m-1} - S_m$	$\frac{S_{m-1}}{S_{m-1} - S_m} a$	$(m-1) a$	e_m
1	13.844	71.9888	3.025	23.798	0	23.798 m
2	10.819	56.2588	2.556	22.010	5.200	27.210
3	8.263	42.9676	2.086	20.598	10.400	30.998
4	6.177	32.1204	1.617	19.864	15.600	35.464
5	4.560	23.7120	1.147	20.673	20.800	41.473
6	3.413	17.7476	0.678	26.176	26.000	52.176
7	2.735	14.2220	0.235	60.519	31.200	91.719

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
GOKANDA, TOKYO.

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CALCULATIONS FOR
檜村橋

$$i_m' = \frac{S_m}{(\bar{r}_{m+1}-\bar{r}_m)-(y_m-y_{m-1})} \cdot a + ma = i_m' + ma$$

m.	Sm.a	$\bar{r}_{m+1}-\bar{r}_m$	y_m-y_{m-1}	$(\bar{r}_{m+1}-\bar{r}_m)-(y_m-y_{m-1})$	i_m'	m.a	i_m
1	56.2598	2.582	0.026	2.556	22.010	5.200	27.210
2	42.9676	2.112	0.026	2.086	20.598	10.400	30.998
3	32.1204	1.643	0.026	1.617	19.864	15.600	35.464
4	23.7120	1.173	0.026	1.147	20.673	20.800	41.473
5	17.7476	0.704	0.026	0.678	26.176	26.000	52.176
6	14.2220	0.235	0.026	0.209	68.048	31.200	99.248

$$W_m = \frac{S_m}{(\bar{r}_{m+1}-\bar{r}_m)-(y_m-y_{m-1})}(\bar{r}_{m+1}-\bar{r}_m) + \bar{r}_m = W_m' + \bar{r}_m$$

m	S_m	$\bar{r}_{m+1}-\bar{r}_m$	$S_m(\bar{r}_{m+1}-\bar{r}_m)$	$(\bar{r}_{m+1}-\bar{r}_m)-(y_m-y_{m-1})$	W_m'	$\bar{r}_m$	W_m
1	10.819	2.582	27.9347	2.556	10.929	3.051	13.980
2	8.263	2.112	17.4515	2.086	8.366	5.633	13.999
3	6.177	1.643	10.1488	1.617	6.276	7.745	14.021
4	4.560	1.173	5.3489	1.147	4.663	9.388	14.051
5	3.413	0.704	2.4028	0.678	3.544	10.561	14.105
6	2.735	0.235	0.6427	0.209	3.076	11.265	14.341

Summary of known quantities.

m	W_m	b_m	S_m	d_m	$\bar{r}_m$	y_m	h_m
0	—	6.029'	13.844'	—	—	—	11.940'
1	5.200'	5.806'	10.819'	11.980'	3.051'	10.819'	9.690'
2	5.200'	5.613'	8.263'	9.741'	5.633'	8.263'	7.635'
3	5.200'	5.453'	6.177'	8.084'	7.745'	6.177'	5.890'
4	5.200'	5.331'	4.560'	6.899'	9.388'	4.560'	4.448'
5	5.200'	5.247'	3.413'	6.206'	10.561'	3.413'	3.382'
6	5.200'	5.205'	2.735'	5.863'	11.265'	2.735'	2.732'
7	5.200'	—	2.500'	5.770'	11.500'	2.500'	—

m	V_m	f_m	e_m	i_m	W_m	$i_m-m.a$
0	—	—	—	—	—	—
1	13.962'	21.492'	23.798'	27.210'	13.980'	22.010'
2	13.980'	18.672'	27.210'	30.998'	13.999'	20.598'
3	13.999'	15.798'	30.998'	35.464'	14.021'	19.864'
4	14.021'	13.130'	35.464'	41.473'	14.051'	20.673'
5	14.051'	11.369'	41.473'	52.176'	14.105'	26.176'
6	14.105'	12.211'	52.176'	99.248'	14.341'	68.048'
7	14.000'	26.221'	91.719'	—	—	—

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR

檜村橋

Calculation of Horizontal thrust H

General formula,

$$H = \frac{\Sigma S_o S_a \ell/A}{\Sigma S_a^2 \ell/A} \quad \text{for unit load}$$

Where H = Horizontal thrust due to unity;
S_o = 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 centimeter;

Top chord stress S_o, redundancy removed and unit load applied.

Member m	Load on panel point 1'	2'	3'	4'	5'	6'	7
1 (U_0-U_1)	-0.034	-0.069	-0.103	-0.137	-0.172	-0206	-0.240
2	-0.090	-0.180	-0.270	-0360	-0450	-0539	-0.629
3	-0.180	-0.361	-0.541	-0.722	-0902	-1.082	-1.263
4	-0.326	-0.652	-0.977	-1.303	-1.629	-1.955	-2.281
5	-0.544	-1.088	-1.632	-2.176	-2721	-3265	-3809
6	-0.815	-1.630	-2444	-3259	-4074	-4889	-5704
7 (U_6-U_7)	-1.040	-2.080	-3120	-4.160	-5200	-6240	-7280
7' (U_7-U_6)	-1.040	-2.080	-3120	-4.160	-5200	-6240	-7280
6'	-1.086	-2.173	-3259	-4346	-5432	-6518	-5704
5'	-0.979	-1.959	-2938	-3918	-4897	-4353	-3809
4'	-0.815	-1.629	-2444	-3258	-2932	-2606	-2281
3'	-0.661	-1323	-1984	-1804	-1624	-1443	-1263
2'	-0.539	-1.079	-0989	-0899	-0809	-0719	-0629
1' (U_1-U_0)	-0446	-0412	-0377	-0343	-0309	-0274	-0240

Bottom chord stress So, redundancy removed and unit load applied.

Member m	Load on panel point 1'	2'	3'	4'	5'	6'	7
0 (L_0-L_1)	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	0.038	0.077	0.115	0.153	0.192	0230	0268
2	0.097	0.194	0291	0388	0485	0582	0679
3	0.189	0378	0568	0757	0946	1135	1324
4	0334	0668	1002	1336	1670	2004	2338
5	0549	1098	1647	2196	2746	3295	3844
6 (L_6-L_7)	0816	1631	2447	3263	4079	4894	5710
6' (L_7-L_6)	1088	2175	3263	4350	5438	6526	5710
5'	0988	1977	2965	3954	4942	4393	3844
4'	0835	1670	2505	3340	3006	2672	2338
3'	0694	1387	2081	1892	1703	1514	1324
2'	0582	1164	1067	0970	0873	0776	0679
1'	0498	0460	0421	0383	0345	0306	0268
0' (L_1-L_6)	0000	0000	0000	0000	0000	0000	0000

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JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR

檜村橋

Diagonal stress So, redundancy removed and unit load applied.

Member m	Load on panel point 1′	2′	3′	4′	5′	6′	7
1 (U_0-L_1)	0.079	0.158	0.237	0.316	0.396	0.475	0.554
2′	0.104	0.208	0.312	0.416	0.521	0.625	0.729
3	0.140	0.280	0.421	0.561	0.701	0.841	0.981
4	0.193	0.386	0.579	0.772	0.965	1.157	1.350
5	0.261	0.521	0.782	1.042	1.303	1.564	1.824
6	0.305	0.610	0.916	1.221	1.526	1.831	2.136
7 (U_6-L_7)	0.250	0.500	0.750	1.000	1.250	1.499	1.749
7′ (L_7-U_6)	-0.052	-0.103	-0.155	-0.206	-0.258	-0.309	1.749
6′	0.121	0.241	0.362	0.482	0.603	2.442	2.136
5′	0.197	0.344	0.590	0.787	2.345	2.085	1.824
4′	0.203	0.406	0.609	1.929	1.736	1.543	1.350
3′	0.189	0.378	1.542	1.402	1.262	1.122	0.981
2′	0.174	1.249	1.145	1.041	0.937	0.833	0.729
1′ (L_1-U_0)	1.028	0.949	0.870	0.791	0.712	0.633	0.554

Vertical stress So, redundancy removed and unit load applied.

Member m	1′	2′	3′	4′	5′	6′	7
0 (U_0-L_0)	-0.071	-0.143	-0.214	-0.286	-0.357	-0.428	-0.500
1	-0.088	-0.177	-0.265	-0.353	-0.442	-0.530	-0.618
2	-0.108	-0.215	-0.323	-0.430	-0.538	-0.645	-0.753
3	-0.128	-0.255	-0.383	-0.510	-0.638	-0.765	-0.893
4	-0.143	-0.287	-0.430	-0.573	-0.717	-0.860	-1.003
5	-0.142	-0.285	-0.427	-0.570	-0.712	-0.854	-0.997
6 (U_6-L_6)	-0.104	-0.208	-0.313	-0.417	-0.521	-0.625	-0.729
6′ (U_6-L_6)	+0.028	+0.056	+0.083	+0.111	+0.139	+0.167	-0.729
5′	-0.056	-0.113	-0.169	-0.225	-0.282	-1.139	-0.997
4′	-0.108	-0.216	-0.325	-0.433	-1.290	-1.146	-1.003
3′	-0.134	-0.269	-0.403	-1.275	-1.148	-1.020	-0.893
2′	-0.145	-0.290	-1.183	-1.075	-0.968	-0.860	-0.753
1′	-0.148	-1.060	-0.971	-0.883	-0.795	-0.706	-0.618
0′ (U_0-L_0)	-0.928	-0.857	-0.785	-0.714	-0.643	-0.571	-0.500

Summary of stress Sa for all members, H=-1 applied.

Member	Top chord			Bottom chord			Diagonal			Vertical		
m	Γ_m	g_m	$Sa=-\frac{\Gamma_m}{g_m}$	$S_m+\Gamma_m$	h_m	$Sa=\frac{S_m+\Gamma_m}{h_m}$	V_m	f_m	$Sa=\frac{V_m}{f_m}$	W_m	l_m-M_a	$Sa=\frac{W_m}{l_m-M_a}$
0·0′	0.000	0.000	—	13.844	11.940	1.159	—	—	—	$-\frac{3051}{5200}$	—	-0.587
1·1′	3051	10.819	-0.282	13.870	9.690	1.431	13.962	21.492	0.650	13.980	22.010	-0.635
2·2′	5.633	8.263	-0.682	13.896	7.655	1.815	13.980	18.672	0.749	13.999	20.598	-0.680
3·3′	7.745	6.177	-1.254	13.922	5.890	2.364	13.999	15.798	0.886	14.021	19.864	-0.706
4·4′	9.388	4.560	-2.059	13.948	4.448	3.136	14.021	13.130	1.068	14.051	20.673	-0.680
5·5′	10.561	3.413	-3.094	13.974	3.382	4.132	14.051	11.369	1.236	14.105	26.176	-0.539
6·6′	11.265	2.735	-4.119	14.000	2.732	5.124	14.105	12.211	1.155	14.341	68.048	-0.211
7·7′	11.500	2.500	-4.600	14.000	—	—	14.000	26.221	0.534	—	—	—

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MADE BY M. J DATE 11-10-9 FILE NO.
CHECKED BY M. J DATE PAGE NO 17

CALCULATIONS FOR

檜村橋

Assumed gross sectional area and slenderness ratio ℓ/A of truss members.

Member m	Top chord ℓ cm	Top chord A cm²	Top chord ℓ/A	Bottom chord ℓ cm	Bottom chord A cm²	Bottom chord ℓ/A	Diagonal ℓ cm	Diagonal A cm²	Diagonal ℓ/A	Vertical ℓ cm	Vertical A cm²	Vertical ℓ/A
0	—	—	—	6029	241	2502	—	—	—	13844	243	5697
1	5200	167	3114	5806	231	2513	11980	87	13770	10819	97	11154
2	〃	〃	〃	5613	〃	2430	9741	74	13164	8263	87	9498
3	〃	〃	〃	5453	198	2754	8054	〃	10884	6177	74	8347
4	〃	〃	〃	5331	170	3136	6899	〃	9323	4560	〃	6162
5	〃	〃	〃	5247	〃	3087	6206	〃	8386	3413	〃	4612
6	〃	221	2353	5205	〃	3062	5863	〃	7923	2735	〃	3696
7	〃	〃	〃	—	—	—	5770	〃	7797	2500	〃	3378

Top chord $Sa \cdot \ell/A$ and $Sa^2 \cdot \ell/A$

Mark	Member	Sa'	Sa	Sa'·Sa	Sa²	ℓ/A	Sa'·Sa·ℓ/A	Sa²·ℓ/A
A	1	-0.034	-0.282	00096	00795	3114	00299	02476
B	2	-0.090	-0.682	00614	04651	〃	01912	14483
C	3	-0.180	-1.254	02257	15725	〃	07028	48968
D	4	-0.326	-2.059	06712	42395	〃	20901	132018
E	5	-0.544	-3.094	16831	95728	〃	52412	298097
F	6	-0.815	-4.119	33570	169662	2353	78990	399215
G	7	-1.040	-4.600	47840	211600	〃	112568	497895
H	7'	-1.040	-4.600	47840	211600	〃	112568	497895
I	6'	-1.086	-4.119	44732	169662	〃	105254	399215
J	5'	-0.979	-3.094	30290	95728	3114	94323	298097
K	4'	-0.815	-2.059	16781	42395	〃	52256	132018
L	3'	-0.661	-1.254	08289	15725	〃	25812	48968
M	2'	-0.539	-0.682	03676	04651	〃	11447	14483
N	1'	-0.446	-0.282	01258	00795	〃	03917	02476

Σ = 2786304

Bottom chord $Sa' \cdot Sa \cdot \ell/A$ and $Sa^2 \cdot \ell/A$

Mark	Member	Sa'	Sa	Sa'·Sa	Sa²	ℓ/A	Sa'·Sa·ℓ/A	Sa²·ℓ/A
A	0	0.000	1.159	0.0000	1.3433	2502	0.0000	33609
B	1	0.038	1431	00544	20478	2513	01367	51461
C	2	0.097	1.815	01761	32942	2430	04279	80049
D	3	0189	2364	04468	55885	2754	12305	153907
E	4	0334	3136	10474	98345	3136	32846	308410
F	5	0549	4132	22685	170734	3087	70029	527056
G	6	0.816	5124	41812	262554	3062	128028	803940
H	6'	1088	5124	55749	262554	3062	170703	803940
I	5'	0988	4132	40824	170734	3087	126024	527056
J	4'	0835	3136	26186	98345	3136	82119	308410
K	3'	0694	2364	16406	55885	2754	45182	153907
L	2'	0582	1.815	10563	32942	2430	25668	80049
M	1'	0498	1431	07126	20478	2513	17908	51461
N	0'	0.000	1.159	0.0000	1.3433	2502	0.0000	33609

Σ = 3916864

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MADE BY M. J DATE 11-10-9 FILE NO
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CALCULATIONS FOR

檜村橋

Diagonals $So.l.\ Sa\ l/A$ and $Sa^2\ l/A$

Mark	Member	So.l.	Sa	So.l.Sa	Sa^2	l/A	So.l.Sa l/A	Sa^2 l/A
A	1	0.079	0.650	0.0514	0.4225	13.770	0.7078	5.8178
B	2	0.104	0.749	0.0779	0.5610	13.164	1.0255	7.3850
C	3	0.140	0.886	0.1240	0.7850	10.884	1.3496	8.5439
D	4	0.193	1.068	0.2061	1.1406	9.323	1.9215	10.6338
E	5	0.261	1.236	0.3226	1.5277	8.386	2.7053	12.8113
F	6	0.305	1.155	0.3523	1.3340	7.923	2.7913	10.5693
G	7	0.250	0.534	0.1335	0.2852	7.797	1.0409	2.2237
H	7'	-0.052	0.534	-0.0278	0.2852	7.797	-0.2168	2.2237
I	6'	0.121	1.155	0.1398	1.3340	7.923	1.1076	10.5693
J	5'	0.197	1.236	0.2435	1.5277	8.386	2.0420	12.8113
K	4'	0.203	1.068	0.2168	1.1406	9.323	2.0212	10.6338
L	3'	0.189	0.886	0.1675	0.7850	10.884	1.8231	8.5439
M	2'	0.174	0.749	0.1303	0.5610	13.164	1.7153	7.3850
N	1'	1.028	0.650	0.6682	0.4225	13.770	9.2011	5.8178
								Σ = 115.9696

Verticals $So.l.\ Sa\ l/A$ and $Sa^2\ l/A$

Mark	Member	So.l.	Sa	So.l.Sa	Sa^2	l/A	So.l.Sa l/A	Sa^2 l/A
A	0	-0.071	-0.587	0.0417	0.3446	5.697	0.2376	1.9632
B	1	-0.088	-0.635	0.0559	0.4032	11.154	0.6235	4.4973
C	2	-0.108	-0.680	0.0734	0.4624	9.498	0.6972	4.3919
D	3	-0.128	-0.706	0.0904	0.4984	8.347	0.7546	4.1601
E	4	-0.143	-0.680	0.0972	0.4624	6.162	0.5989	2.8493
F	5	-0.142	-0.539	0.0765	0.2905	4.612	0.3528	1.3398
G	6	-0.104	-0.211	0.0219	0.0445	3.696	0.0809	0.1645
H	6'	+0.028	-0.211	-0.0059	0.0445	3.696	-0.0218	0.1645
I	5'	-0.056	-0.539	0.0302	0.2905	4.612	0.1393	1.3398
J	4'	-0.108	-0.680	0.0734	0.4624	6.162	0.4523	2.8493
K	3'	-0.134	-0.706	0.0946	0.4984	8.347	0.7896	4.1601
L	2'	-0.145	-0.680	0.0986	0.4624	9.498	0.9365	4.3919
M	1'	-0.148	-0.635	0.0940	0.4032	11.154	1.0486	4.4973
N	0'	-0.928	-0.587	0.5447	0.3446	5.697	3.1032	1.9632
								Σ = 38.732

Summary of $Sa^2\ l/A$ for the entire span.

Top chord members	Σ $Sa^2\ l/A$	=	27.86304
Bottom chord members	"	=	39.16864
Diagonal members	"	=	11.59696
Vertical members	"	=	3.87322
	Summary	=	82.50186

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CALCULATIONS FOR

檜村橋

Summary of Sa1 Sa1/A for the entire span

Mark	Top chord	Bottom chord	Diagonals	verticals	summary	second summary	Remarks	
A	0.0299	0.0000	0.7078	0.2376	0.9753	0.9753	A	1
B	0.1912	0.1367	1.0255	0.6235	1.9769	2.9522	A~B	2
C	0.7028	0.4279	1.3496	0.6972	3.1775	6.1297	A~C	3
D	2.0901	1.2305	1.9215	0.7546	5.9967	12.1264	A~D	4
E	5.2412	3.2846	2.7053	0.5989	11.8300	23.9564	A~E	5
F	7.8990	7.0029	2.7913	0.3528	18.0460	42.0024	A~F	6
G	11.2568	12.8028	1.0409	0.0809	25.1814	67.1838	A~G	7
H	11.2568	17.0703	-0.2168	-0.0218	28.0885	95.2723	A~H	7'
I	10.5254	12.6024	1.1076	0.1393	24.3747	119.6470	A~I	6'
J	9.4323	8.2119	2.0420	0.4523	20.1385	139.7855	A~J	5'
K	5.2256	4.5182	2.0212	0.7896	12.5546	152.3401	A~K	4'
L	2.5812	2.5668	1.8230	0.9365	7.9075	160.2476	A~L	3'
M	1.1447	1.7908	1.7153	1.0485	5.6993	165.9469	A~M	2'
N	0.3917	0.0000	0.2011	0.1032	1.2696	167.6429	A~N	1'

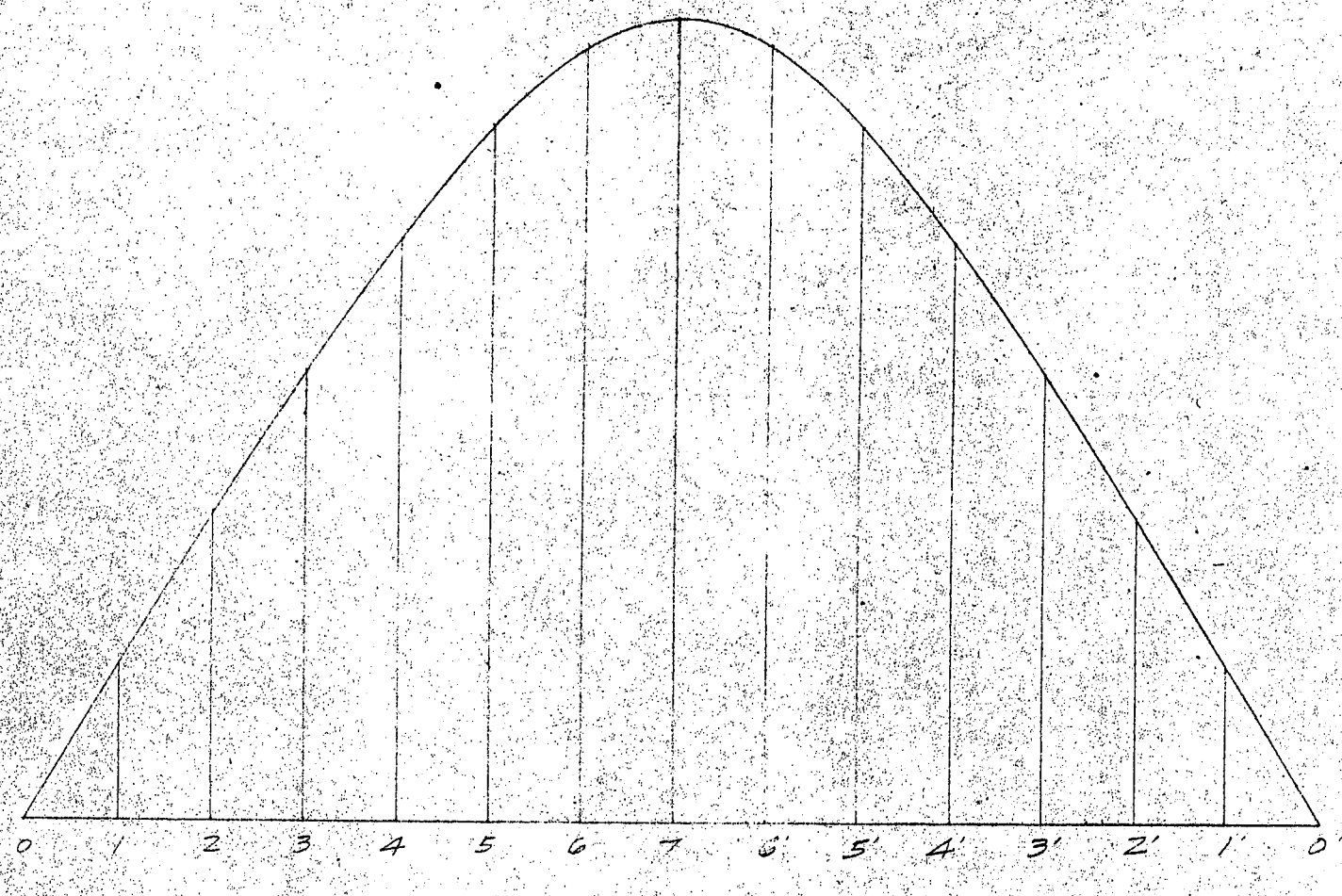
Influence surface of horizontal thrust

load unity on point

1	1 × 1.786429	+	13 × 0.0000	=	1.786429	÷ 8.250186	=	0.217	= H1
2	2 × 1.602476	+	12 × 2.9522	=	3.559216	"	=	0.431	= H2
3	3 × 1.523401	+	11 × 6.1297	=	5.244470	"	=	0.636	= H3
4	4 × 1.397855	+	10 × 12.1264	=	6.804060	"	=	0.825	= H4
5	5 × 1.196470	+	9 × 23.9564	=	8.138426	"	=	0.986	= H5
6	6 × 0.952723	+	8 × 42.0024	=	9.076530	"	=	1.100	= H6
7	7 × 0.671838	+	7 × 67.1838	=	9.405732	"	=	1.140	= H7

Summary of H for the entire span 9.530

Diagram for H-surface



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CALCULATIONS FOR

檜村橋

Influence surface of stresses for truss members
General formula for member stress $S = S_0 - S_a H$ for unit load
Find $S_a H$
For Top chord members

		Load unity on panel point						
Member	Sa	1 H=0217	2 0431	3 0636	4 0825	5 0986	6 0.1100	7 0.1140
U_0-U_1	-0282	-0061	-0.122	-0.179	-0232	-0278	-0.310	-0321
U_1-U_2	-0682	-0148	-0294	-0434	-0563	-0672	-0750	-0777
U_2-U_3	-1254	-0272	-0540	-0798	-1035	-1236	-1379	-1430
U_3-U_4	-2059	-0447	-0887	-1310	-1699	-2030	-2265	-2347
U_4-U_5	-3094	-0671	-1334	-1968	-2553	-3050	-3403	-3527
U_5-U_6	-4119	-0894	-1775	-2620	-3398	-4061	-4531	-4690
U_6-U_7	-4600	-0998	-1983	-2926	-3795	-4536	-5060	-5244

For Bottom Chord members

Member	Sa	1	2	3	4	5	6	7
L_0-L_1	1159	0252	0500	0737	0956	1143	1275	1321
L_1-L_2	1431	0311	0617	0910	1181	1411	1574	1631
L_2-L_3	1815	0394	0782	1154	1497	1790	1997	2069
L_3-L_4	2364	0513	1019	1504	1950	2331	2600	2695
L_4-L_5	3136	0681	1352	1994	2587	3092	3450	3575
L_5-L_6	4132	0897	1781	2628	3409	4074	4545	4710
L_6-L_7	5124	1112	2208	3258	4227	5052	5636	5841

For Diagonal members

Member	Sa	1	2	3	4	5	6	7
U_0-L_1	0650	0141	0280	0413	0536	0641	0715	0741
U_1-L_2	0749	0163	0323	0476	0618	0739	0824	0554
U_2-L_3	0886	0192	0382	0563	0731	0874	0975	1010
U_3-L_4	1068	0232	0460	0679	0881	1053	1175	1218
U_4-L_5	1236	0268	0533	0786	1020	1219	1360	1409
U_5-L_6	1155	0251	0498	0735	0953	1139	1271	1317
U_6-L_7	0534	0116	0230	0340	0441	0527	0587	0609

For Vertical members

Member	Sa	1	2	3	4	5	6	7
U_0-L_0	-0587	-0127	-0253	-0373	-0484	-0579	-0646	-0669
U_1-L_1	-0635	-0138	-0274	-0404	-0524	-0626	-0699	-0724
U_2-L_2	-0680	-0148	-0293	-0432	-0561	-0670	-0748	-0775
U_3-L_3	-0706	-0153	-0304	-0449	-0582	-0696	-0777	-0805
U_4-L_4	-0680	-0148	-0293	-0432	-0561	-0670	-0748	-0775
U_5-L_5	-0539	-0117	-0232	-0343	-0445	-0531	-0593	-0614
U_6-L_6	-0211	-0046	-0091	-0134	-0174	-0208	-0232	-0241

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CALCULATIONS FOR

檜村橋

Influence surface of Top chord members.

Load on	$U_0 - U_1$ S_0	S_aH	S	$U_1 - U_2$ S_0	S_aH	S	$U_2 - U_3$ S_0	S_aH	S
1	-0.034	-0.061	0.027	-0.090	-0.148	0.058	-0.180	-0.272	0.092
2	-0.069	-0.122	0.053	-0.180	-0.294	0.114	-0.361	-0.540	0.179
3	-0.103	-0.179	0.076	-0.270	-0.343	0.164	-0.541	-0.798	0257
4	-0.137	-0.232	0.095	-0.360	-0.563	0203	-0.722	-1.035	0313
5	-0.172	-0.278	0.106	-0.450	-0.672	0222	-0.902	-1.236	0334
6	-0.206	-0.310	0.104	-0.539	-0.750	0211	-1.082	-1.379	0297
7	-0.240	-0.321	0.081	-0.629	-0.777	0.148	-1.263	-1.430	0.167
6	-0.274	-0.310	0.036	-0.719	-0.750	0.031	-1.443	-1.379	-0.064
5	-0.309	-0.278	-0.031	-0.809	-0.672	-0.137	-1.624	-1.236	-0.388
4	-0.343	-0.232	-0.111	-0.899	-0.563	-0.336	-1.804	-1.035	-0.769
3	-0.377	-0.179	-0.198	-0.989	-0.434	-0.555	-1.984	-0.798	-1.186
2	-0.412	-0.122	-0.290	-1.079	-0.294	-0.785	-1.323	-0.540	-0.783
1	-0.446	-0.061	-0.385	-0.539	-0.148	-0.391	-0.661	-0.272	-0.389
	Σ plus stresses		+ 0.578			+ 1.151			+ 1.639
	Σ Minus "		- 1.015			- 2.204			- 3.579
	Summary		- 0.437			- 1.053			- 1.940

Load on	$U_3 - U_4$ S_0	S_aH	S	$U_4 - U_5$ S_0	S_aH	S	$U_5 - U_6$ S_0	S_aH	S
1	-0.326	-0.447	0.121	-0.544	-0.671	0.127	-0.815	-0.894	0.079
2	-0.652	-0.887	0.235	-1.088	-1.334	0246	-1.360	-1.775	0.145
3	-0.977	-1.310	0333	-1.632	-1.968	0.336	-2.444	-2.620	0.176
4	-1.303	-1.699	0396	-2.176	-2.553	0.377	-3.259	-3.398	0.139
5	-1.629	-2.030	0401	-2.721	-3.050	0.329	-4.074	-4.061	-0.013
6	-1.955	-2.265	0310	-3.265	-3.403	0.138	-4.889	-4.531	-0.358
7	-2.281	-2.347	0.066	-3.809	-3.527	-0.282	-5.704	-4.696	-1.008
6	-2.606	-2.265	-0.341	-4.353	-3.403	-0.950	-6.518	-4.531	-1.987
5	-2.932	-2.030	-0.902	-4.897	-3.050	-1.847	-5.432	-4.061	-1.371
4	-3.258	-1.699	-1.559	-3.918	-2.553	-1.365	-4.346	-3.398	-0.948
3	-2.444	-1.310	-1.134	-2.938	-1.968	-0.970	-3.259	-2.620	-0.639
2	-1.629	-0.887	-0.742	-1.959	-1.334	-0.625	-2.173	-1.775	-0.398
1	-0.815	-0.447	-0.368	-0.979	-0.671	-0.308	-1.086	-0.894	-0.192
	Σ plus stresses		+ 1.862			+ 1.553			+ 0.539
	Σ Minus "		- 5.046			- 6.347			- 6.914
	Summary		- 3.184			- 4.794			- 6.375

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CALCULATIONS FOR

檜村橋

load on	U_6-U_7 So	SaH	
1'	-1.040	-0.998	-0.042
2'	-2.080	-1.983	-0.097
3'	-3.120	-2.926	-0.194
4'	-4.160	-3.795	-0.365
5'	-5.200	-4.536	-0.664
6'	-6.240	-5.060	-1.180
7	-7.280	-5.244	-2.036
6	-6.240	-5.060	-1.180
5	-5.200	-4.536	-0.664
4	-4.160	-3.795	-0.365
3	-3.120	-2.926	-0.194
2	-2.080	-1.983	-0.097
1	-1.040	-0.998	-0.042
	Σ plus stresses		+0.000
	Σ minus		-7.120
	summary		-7.120

Influence surface of Bottom chord members

load on	L_0-L_1 So	SaH	S	L_1-L_2 So	SaH	S	L_2-L_3 So	SaH	S
1'	0.000	0.252	-0.252	0.038	0.311	-0.273	0.097	0.394	-0.297
2'	"	0.500	-0.500	0.077	0.617	-0.540	0.194	0.782	-0.588
3'	"	0.737	-0.737	0.115	0.910	-0.795	0.291	1.154	-0.863
4'	"	0.956	-0.956	0.153	1.181	-1.028	0.388	1.497	-1.109
5'	"	1.143	-1.143	0.192	1.411	-1.219	0.485	1.790	-1.305
6'	"	1.275	-1.275	0.230	1.574	-1.344	0.582	1.997	-1.415
7	"	1.321	-1.321	0.268	1.631	-1.363	0.679	2.069	-1.390
6	"	1.275	-1.275	0.306	1.574	-1.268	0.776	1.997	-1.221
5	"	1.143	-1.143	0.345	1.411	-1.066	0.873	1.790	-0.917
4	"	0.956	-0.956	0.383	1.181	-0.798	0.970	1.497	-0.527
3	"	0.737	-0.737	0.421	0.910	-0.489	1.067	1.154	-0.087
2	"	0.500	-0.500	0.460	0.617	-0.157	1.164	0.782	+0.382
1	"	0.252	-0.252	0.498	0.311	+0.187	0.582	0.394	+0.188
	Σ plus stresses		+0.000			+0.187			+0.570
	Σ minus stresses		-1.047			-10.340			-9.719
	Summary		-1.047			-10.153			-9.149

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CALCULATIONS FOR

檜村橋

Load on	L_3-L_4 S_0	$S_a H$	S
1'	0.189	0513	-0324
2'	0378	1.019	-0641
3'	0568	1504	-0936
4'	0757	1950	-1193
5'	0946	2331	-1385
6'	1.135	2600	-1465
7	1324	2695	-1371
6	1.514	2600	-1086
5	1.703	2331	-0628
4	1892	1950	-0058
3	2081	1504	+0577
2	1387	1.019	+0368
1	0.694	0513	+0181
		Σ plus stresses	+1.126
		Σ minus "	-9087
		Summary	-7.961

	L_4-L_5 S_0	$S_a H$	S
	0334	0681	-0347
	0668	1325	-0657
	1002	1994	-0992
	1336	2587	-1251
	1670	3092	-1422
	2004	3450	-1446
	2338	3575	-1237
	2672	3450	-0778
	3006	3092	-0086
	3340	2587	+0753
	2505	1994	+0511
	1670	1325	+0345
	0835	0681	+0154
			+1.763
			-8216
			-6453

	L_5-L_6 S_0	$S_a H$	S
	0549	0897	-0348
	1.098	1781	-0683
	1647	2628	-0981
	2196	3409	-1213
	2756	4074	-1318
	3295	4545	-1250
	3844	4710	-0866
	4393	4545	-0152
	4942	4074	+0868
	3954	3409	+0545
	2965	2628	+0337
	1977	1781	+0196
	0988	0897	+0091
			+2037
			-6811
			-4774

Load on	L_6-L_7 S_0	$S_a H$	S
1'	0816	1112	-0296
2'	1631	2208	-0577
3'	2447	3258	-0811
4'	3263	4227	-0964
5'	4079	5052	-0973
6'	4894	5636	-0742
7	5710	5841	-0131
6	6526	5636	+0890
5	5438	5052	+0386
4	4350	4227	+0123
3	3263	3258	+0005
2	2175	2208	-0033
1	1088	1112	-0024
		Σ plus stresses	+1404
		Σ minus "	-4551
		Summary	-3147

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CALCULATIONS FOR

檜村橋

Influence surface of Diagonal members

	U_0-L_1			U_1-L_2			U_2-L_3		
Load on	S_0	S_aH	S	S_0	S_aH	S	S_0	S_aH	S
1'	0.079	0.141	-0.062	0.104	0.163	-0.059	0.140	0.192	-0.052
2'	0.158	0.280	-0.122	0.208	0.323	-0.115	0.280	0.382	-0.102
3'	0.237	0.413	-0.176	0.312	0.476	-0.164	0.421	0.563	-0.142
4'	0.316	0.536	-0.220	0.416	0.618	-0.202	0.561	0.731	-0.170
5'	0.396	0.641	-0.245	0.521	0.739	-0.218	0.701	0.874	-0.173
6'	0.475	0.715	-0.240	0.625	0.824	-0.199	0.841	0.975	-0.134
7	0.554	0.741	-0.187	0.729	0.854	-0.125	0.981	1.010	-0.029
6	0.633	0.715	-0.082	0.833	0.824	0.009	1.122	0.975	0.148
5	0.712	0.641	0.071	0.937	0.739	0.198	1.262	0.874	0.388
4	0.791	0.536	0.255	1.041	0.618	0.423	1.402	0.731	0.671
3	0.870	0.413	0.457	1.145	0.476	0.669	1.542	0.563	0.979
2	0.949	0.280	0.669	1.249	0.323	0.926	0.378	0.382	-0.004
1	1.028	0.141	0.887	0.174	0.163	0.011	0.189	0.192	-0.003
Σ plus stresses			+2.339			+2.236			+2.185
Σ minus "			-1.334			-1.082			-0.809
Summary			+1.005			+1.154			+1.376

	U_3-L_4			U_4-L_5			U_5-L_6		
Load on	S_0	S_aH	S	S_0	S_aH	S	S_0	S_aH	S
1'	0.193	0.232	-0.039	0.261	0.268	-0.007	0.305	0.251	0.054
2'	0.386	0.460	-0.074	0.521	0.533	-0.012	0.610	0.498	0.112
3'	0.579	0.679	-0.100	0.782	0.786	-0.004	0.916	0.735	0.181
4'	0.772	0.881	-0.109	1.042	1.020	0.022	1.221	0.953	0.268
5'	0.965	1.053	-0.088	1.303	1.219	0.084	1.526	1.139	0.387
6'	1.157	1.175	-0.018	1.564	1.360	0.204	1.831	1.271	0.560
7	1.350	1.218	0.132	1.824	1.409	0.415	2.136	1.317	0.819
6	1.543	1.175	0.368	2.085	1.360	0.725	2.442	1.271	1.171
5	1.736	1.053	0.683	2.345	1.219	1.126	0.603	1.139	-0.536
4	1.929	0.881	1.048	0.787	1.020	-0.233	0.482	0.953	-0.471
3	0.609	0.679	-0.070	0.590	0.786	-0.196	0.362	0.735	-0.373
2	0.406	0.460	-0.054	0.394	0.533	-0.139	0.241	0.498	-0.257
1	0.203	0.232	-0.029	0.197	0.268	-0.071	0.121	0.251	-0.130
Σ plus stresses			+2.231			+2.576			+3.552
Σ minus "			-0.581			-0.662			-1.767
Summary			-1.650			+1.914			+1.785

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CALCULATIONS FOR

檜村橋

		$U_6 - L_7$	
Load on	S_o	$S_a H$	S
1'	0250	0116	+0134
2'	0500	0230	+0270
3	0750	0340	+0410
4	1000	0441	+0559
5	1250	0527	+0723
6	1499	0578	+0921
7	1749	0609	+1140
6	-0309	0578	-0887
5	-0258	0527	-0785
4	-0206	0441	-0647
3	-0155	0340	-0495
2	-0103	0230	-0333
1	-0052	0116	-0168
	Σ plus stresses		+4157
	Σ minus "		-3315
	Summary		+0842

Influence surface of Vertical members

	$U_0 - L_0$			$U_1 - L_1$			$U_2 - L_2$		
Load on	S_o	$S_a H$	S	S_o	$S_a H$	S	S_o	$S_a H$	S
1'	-0071	-0127	+0056	-0088	-0138	0050	-0108	-0148	0040
2'	-0143	-0253	+0110	-0177	-0274	0097	-0215	-0293	0078
3'	-0214	-0373	+0159	-0265	-0404	0139	-0323	-0432	0109
4'	-0286	-0484	+0198	-0353	-0524	0171	-0430	-0561	0131
5'	-0357	-0579	+0222	-0442	-0626	0194	-0538	-0670	0132
6'	-0428	-0646	+0218	-0530	-0699	0169	-0645	-0748	0103
7	-0500	-0669	+0169	-0618	-0724	0106	-0753	-0775	0022
6	-0571	-0646	+0075	-0706	-0699	-0007	-0860	-0748	-0112
5	-0643	-0579	-0064	-0795	-0626	-0169	-0968	-0670	-0298
4	-0714	-0484	-0230	-0883	-0524	-0359	-1075	-0561	-0514
3	-0785	-0373	-0412	-0971	-0404	-0567	-1183	-0432	-0751
2	-0857	-0253	-0604	-1060	-0274	-0786	-0290	-0293	0003
1	-0928	-0127	-0810	-0148	-0138	-0010	-0145	-0148	0003
	Σ plus stresses		+1207			+0926			+0621
	Σ minus "		-2111			-1898			-1675
	Summary		-0904			-0972			-1055

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CALCULATIONS FOR

檜村橋

Load on	U_3-L_3 S_0	S_aH	S	U_4-L_4 S_0	S_aH	S	U_5-L_5 S_0	S_aH	S
1'	-0.128	-0.153	0.025	-0.143	-0.148	0.005	-0.142	-0.117	-0.025
2'	-0.255	-0.304	0.049	-0.287	-0.293	0.006	-0.285	-0.232	-0.053
3'	-0.383	-0.449	0.066	-0.430	-0.432	0.002	-0.427	-0.343	-0.084
4'	-0.510	-0.582	0.072	-0.573	-0.561	-0.012	-0.570	-0.445	-0.125
5'	-0.638	-0.696	0.058	-0.717	-0.670	-0.047	-0.712	-0.531	-0.181
6'	-0.765	-0.777	0.012	-0.860	-0.748	-0.112	-0.854	-0.593	-0.261
7	-0.893	-0.805	-0.088	-1.003	-0.775	-0.228	-0.997	-0.614	-0.383
6	-1.020	-0.777	-0.243	-1.146	-0.748	-0.398	-1.139	-0.593	-0.546
5	-1.148	-0.696	-0.452	-1.290	-0.670	-0.620	-0.282	-0.531	0.249
4	-1.275	-0.582	-0.693	-0.433	-0.561	0.128	-0.225	-0.445	0.220
3	-0.403	-0.449	0.046	-0.325	-0.432	0.107	-0.169	-0.343	0.174
2	-0.269	-0.304	0.035	-0.216	-0.293	0.077	-0.113	-0.232	0.119
1	-0.134	-0.153	0.019	-0.108	-0.148	0.040	-0.056	-0.117	0.061
	Σ plus stresses		+0.382			+0.365			+0.823
	Σ minus "		-1.476			-1.417			-1.658
	Summary		-1.094			-1.052			-0.835

Load on	U_6-L_6 S_0	S_aH	S
1'	-0.104	-0.0046	-0.0058
2'	-0.208	-0.091	-0.117
3'	-0.313	-0.134	-0.179
4'	-0.417	-0.174	-0.243
5'	-0.521	-0.208	-0.313
6'	-0.625	-0.232	-0.393
7	-0.729	-0.241	-0.488
6	0.167	-0.232	0.399
5	0.139	-0.208	0.347
4	0.111	-0.174	0.285
3	0.083	-0.134	0.217
2	0.056	-0.091	0.147
1	0.028	-0.046	0.074
	Σ plus stresses		+1.469
	Σ minus "		-1.791
	summary		-0.322

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GOTANDA, TOKYO.

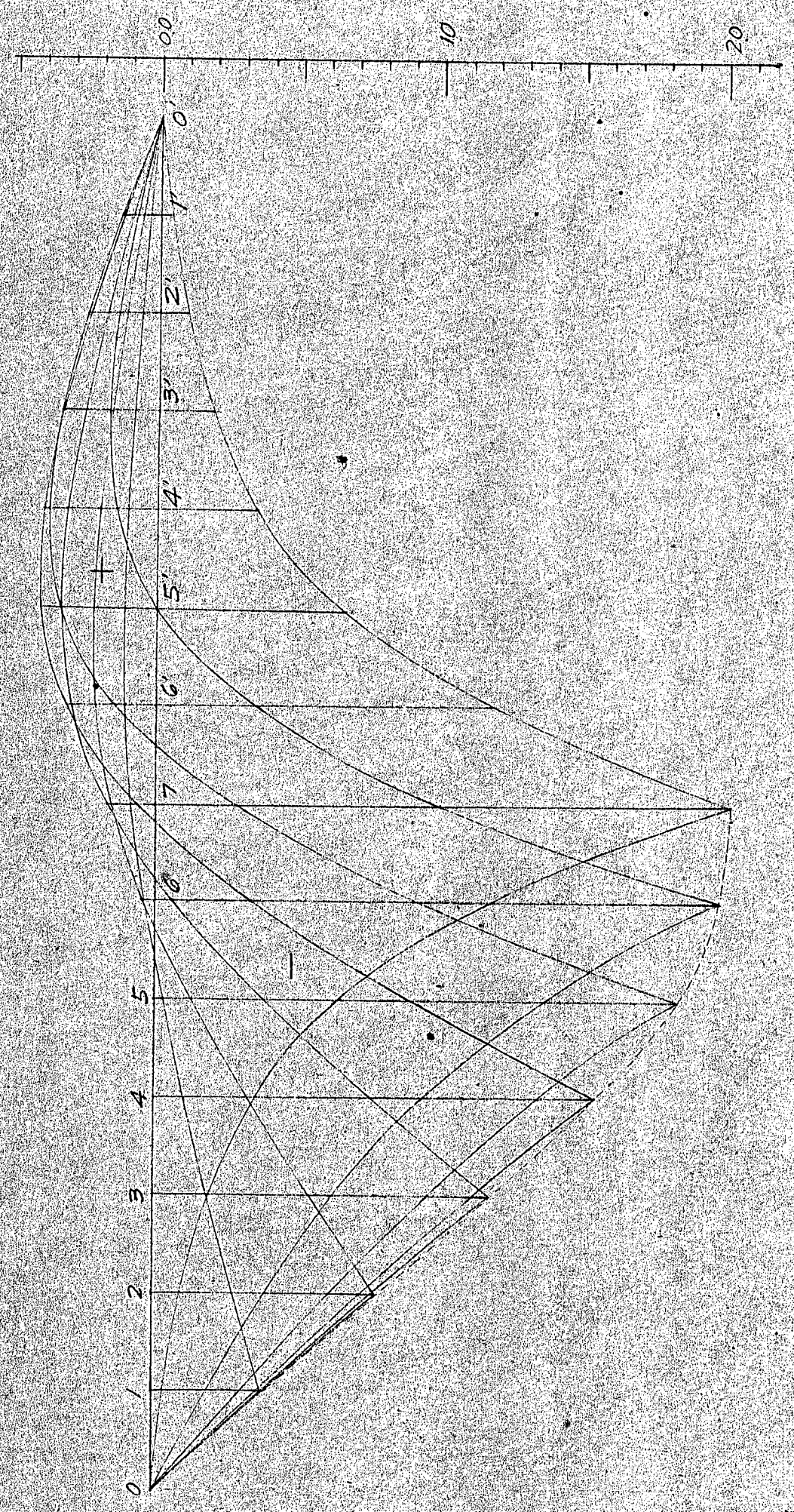
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CALCULATIONS FOR

檜村橋

Influence Surface of Top chord members



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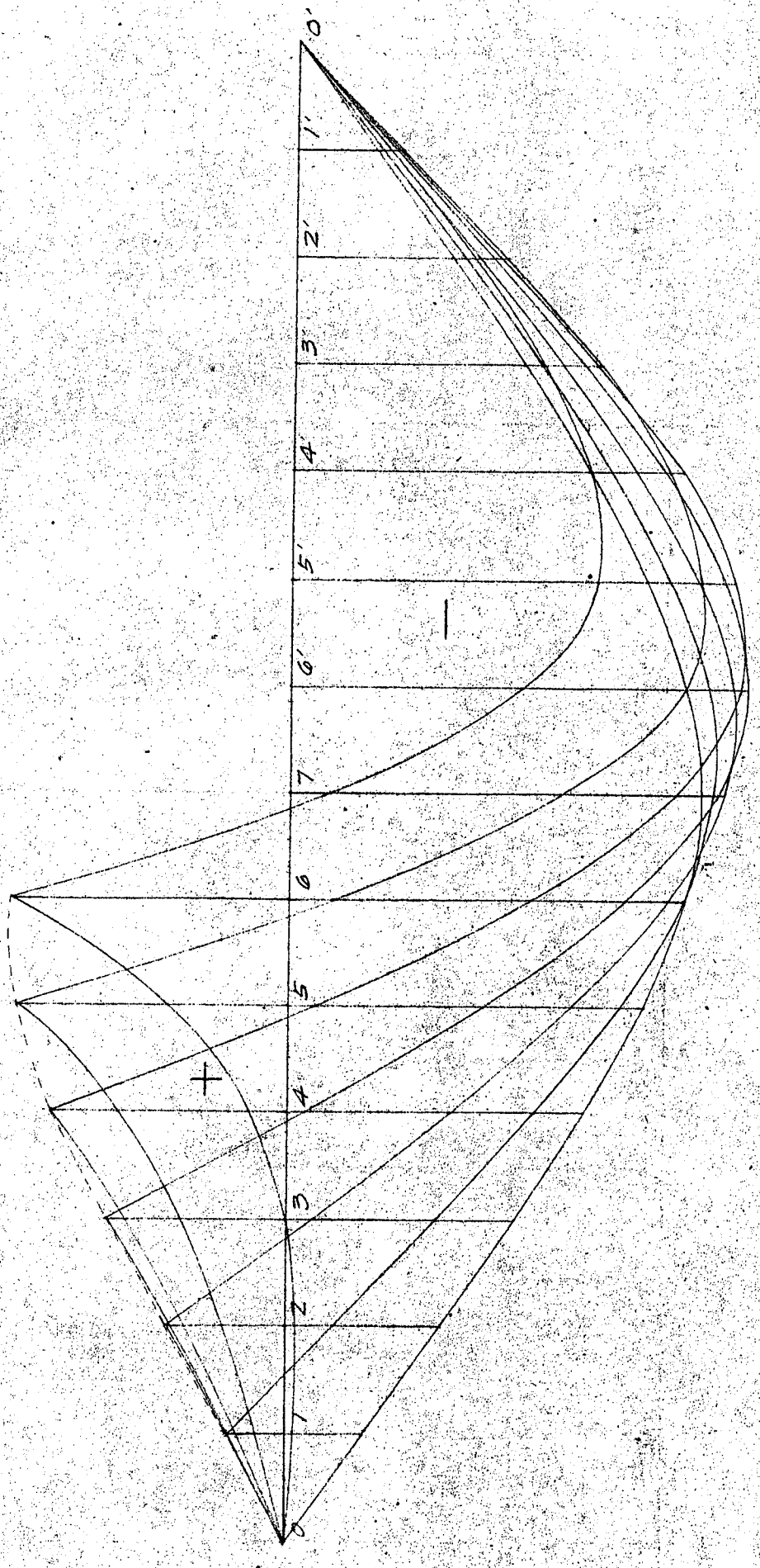
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CALCULATIONS FOR

檜村橋

Influence Surface of Bottom chord members.



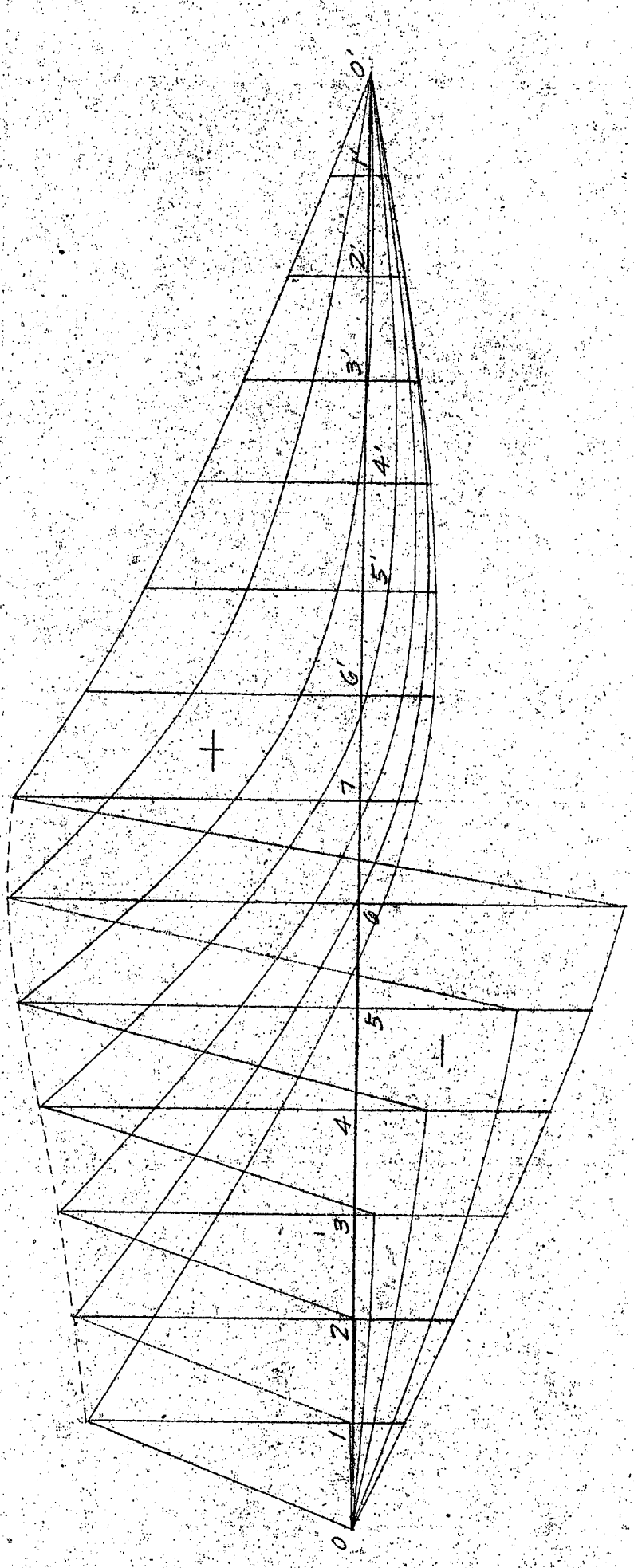
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CALCULATIONS FOR

檜村橋

Influence surface of Diagonal members



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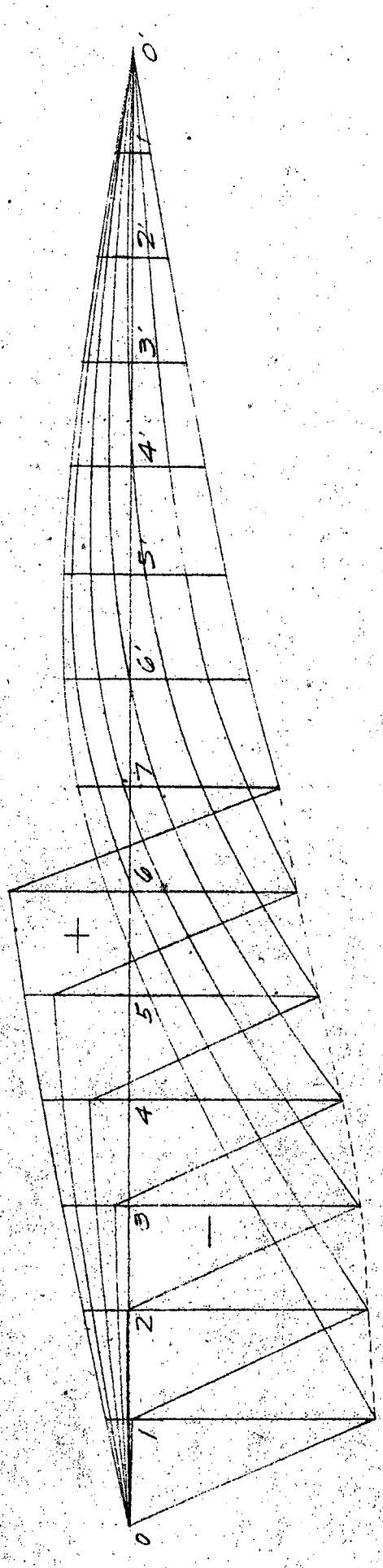
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CALCULATIONS FOR

檜村橋

Influence Surface of Vertical members



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CALCULATIONS FOR

檜村橋

Stresses in each members
Panel load

Dead load = 14320 kg. for one truss
Live load = 6470 "
Extra single concentration = 2,500 "

Dead Load			Live Load									
Member	Σ ordinate	D.L. stresses	Σ+ordinate	Σ-ordinate	+ stress	- stress	Max+ord.	Max-ord.	+ stress	- stress	Total LL stress	
U_0-U_1	-0437	- 6260	0.578	-1015	3,740	- 6,570	0.106	-0385	270	- 960	4,010	- 7,530
U_1-U_2	-1053	-15080	1,151	-2204	7,450	-14260	0222	- 0785	550	-1,960	8,000	-16220
U_2-U_3	-1,940	-27780	1639	-3579	10,600	-23160	0334	-1,186	830	-2960	11430	-26,120
U_3-U_4	-3184	-45590	1862	-5046	12050	-32650	0401	-1,159	1,000	-2890	13050	-35540
U_4-U_5	-4794	-68650	1553	-6347	10,050	-41,070	0377	-1847	940	-4620	10,990	-45690
U_5-U_6	-6375	-91,290	0539	-6914	3490	-44730	0.176	-1987	440	-4970	3930	-44700
U_6-U_7	-7120	-101,960	0.000	-7120	0	-46070	0.000	-2036	0	-5090	0	-51,160
L_0-L_1	-11047	-158190	0.000	-11047	0	-71470	0.000	-1321	0	-3300	0	-74770
L_1-L_2	-10153	-145390	0187	-10340	1210	-66900	0187	-1363	470	-3410	1,680	-70310
L_2-L_3	-9149	-131,010	0570	- 9719	3690	-62,880	0382	- 1415	950	-3540	4640	-66420
L_3-L_4	-7961	-114000	1126	- 9087	7290	-58790	0577	-1465	1440	-3660	8730	-62450
L_4-L_5	-6453	- 92410	1763	- 8216	11410	-53,160	0753	-1446	1880	-3620	13290	-56780
L_5-L_6	-4774	-68360	2037	- 6811	13180	-44070	0868	-1318	2170	-3300	15350	-47370
L_6-L_7	-3147	-45070	1404	- 4551	9080	-29440	0890	- 0973	2230	-2430	11310	-31870
U_0-L_1	1005	14390	2339	-1334	15130	- 8630	0887	-0245	2220	- 610	17350	-9240
U_1-L_2	1154	16530	2236	-1082	14470	- 7000	0926	-0218	2320	- 540	16790	-7540
U_2-L_3	1376	19700	2185	-0809	14140	- 5230	0979	-0173	2450	-430	16590	-5660
U_3-L_4	1650	23630	2231	-0581	14430	- 3760	1048	-0109	2620	-270	17050	-4030
U_4-L_5	1914	27410	2576	-0662	16,670	-4280	1126	-0233	2820	- 580	19490	-4860
U_5-L_6	1785	25560	3552	-1767	22980	-11430	1171	-0536	2930	-1340	25910	-12770
U_6-L_7	0842	12060	4157	-3315	26900	-21450	1140	-0887	2850	-2220	29750	-23670
U_0-L_0	-0904	-12950	1207	-2111	7810	-13660	0222	-0801	560	-2000	8370	-15660
U_1-L_1	-0972	-13920	0926	-1898	5990	-12280	0194	-0786	490	-1970	6480	-14250
U_2-L_2	-1055	-15110	0621	-1675	4020	-10840	0132	-0751	330	-1880	4350	-12720
U_3-L_3	-1094	-15670	0382	-1476	2470	- 9550	0078	-0693	180	-1730	2650	-11280
U_4-L_4	-1052	-15060	0365	-1417	2360	- 9170	0128	-0620	320	-1550	2680	-10720
U_5-L_5	-0835	-11960	0823	-1658	5320	-10730	0249	-0546	620	-1370	5940	-12100
U_6-L_6	-0322	- 4610	1469	-1791	9500	-11590	0399	-0488	1000	-1220	10500	-12810
U_7-L_7	-1000	-14320	0000	-1000	0	- 6470	0.000	-1000	0	-2500	0	- 8970

U_0-L_0

Dead load Stress

		- 12950
from girder span	2030 × 1065	= - 10,800
panel load at 0	1/2 · 14320	= - 7160
		30,910 kg

live load Stress

	1500 × 1065	= - 8,000
	2	
from girder span		- 15,660
panel load at 0	1/2 · 6470	- 3240
		26900 kg

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CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR

檜村橋

Calculation for Temperature stresses;
Horizontal thrust due to temperature change

$$Hx = \frac{\Sigma E t_0 Sa\, l}{\Sigma Sa^2 l / EA}$$ where Hx = horizontal thrust due to temperature change.

E = Coefficient of expansion for steel due to temperature change of 1°C. = 0.000012.

t_0 = temperature change in degree C = ± 30°.

l = length of each members.

Sa = stresses of each members due to redundancy of H = -1 applied.

E = modulus of elasticity of steel = 2100000 t/cm².

A = gross area of each member in cm².

$E t_0 = 0.000012 \times 30 = 0.00036$

$\Sigma Sa^2 l / EA = 8250186 \div 2100000 = 0.0003929$

$$Hx = \frac{0.00036 \Sigma Sa\, l}{0.0003929} = \pm 0.916 \Sigma Sa\, l$$

Find $\Sigma Sa\, l$

Member	Top chord Sa	l	Sal	Bottom chord Sa	l	Sal	Diagonal Sa	l	Sal	Vertical Sa	l	Sal
0	—	—	—	1159	6029	6988	—	—	—	-0587	13844	8126
1	-0282	5200	-1466	1431	5806	8308	0650	11980	7787	-0635	10819	6870
2	-0682	"	-3546	1815	5613	10188	0749	9741	7296	-0680	8263	5619
3	-1254	"	-6521	2364	5453	12891	0886	8054	7136	-0706	6177	4361
4	-2059	"	-10707	3136	5331	16718	1068	6899	7368	-0680	4560	3101
5	-3094	"	-16089	4132	5247	21681	1236	6206	7671	-0539	3413	1840
6	-4119	"	-21419	5124	5205	26670	1155	5863	6772	-0211	2735	577
7	-4600	"	-23920	—	—	—	0534	5770	3081	—	—	—
	Summary		-167336			206888			94222			-60988

$\Sigma Sa\, l = -167336 + 206888 + 94222 - 60988 = 72786$

$Hx = -0.916 \times 72786 = -6660$ kg for 30°C rise

Temperature stresses of each members - Sa ⊙ for rise of temperature

Top chord

Member	Sa	Hx	S
U_0-U_1	-0282	-6660	1880
U_1-U_2	-0682	"	4540
U_2-U_3	-1254	"	8350
U_3-U_4	-2059	"	13710
U_4-U_5	-3094	"	20610
U_5-U_6	-4119	"	27430
U_6-U_7	-4600	"	30640

Bottom chord

Member	Sa	Hx	S
L_0-L_1	1159	-6660	-7720
L_1-L_2	1431	"	-9530
L_2-L_3	1815	"	-12090
L_3-L_4	2364	"	-15740
L_4-L_5	3136	"	-20890
L_5-L_6	4132	"	-27520
L_6-L_7	5124	"	-34130

Diagonals

Member	Sa	Hx	S
U_0-L_1	0650	-6660	-4330
U_1-L_2	0749	"	-5000
U_2-L_3	0886	"	-5900
U_3-L_4	1068	"	-7110
U_4-L_5	1236	"	-8230
U_5-L_6	1155	"	-7690
U_6-L_7	0534	"	-3560

Verticals

Member	Sa	Hx	S
U_0-L_0	-0587	-6660	3910
U_1-L_1	-0635	"	4230
U_2-L_2	-0680	"	4530
U_3-L_3	-0706	"	4700
U_4-L_4	-0680	"	4530
U_5-L_5	-0539	"	3590
U_6-L_6	-0211	"	1410

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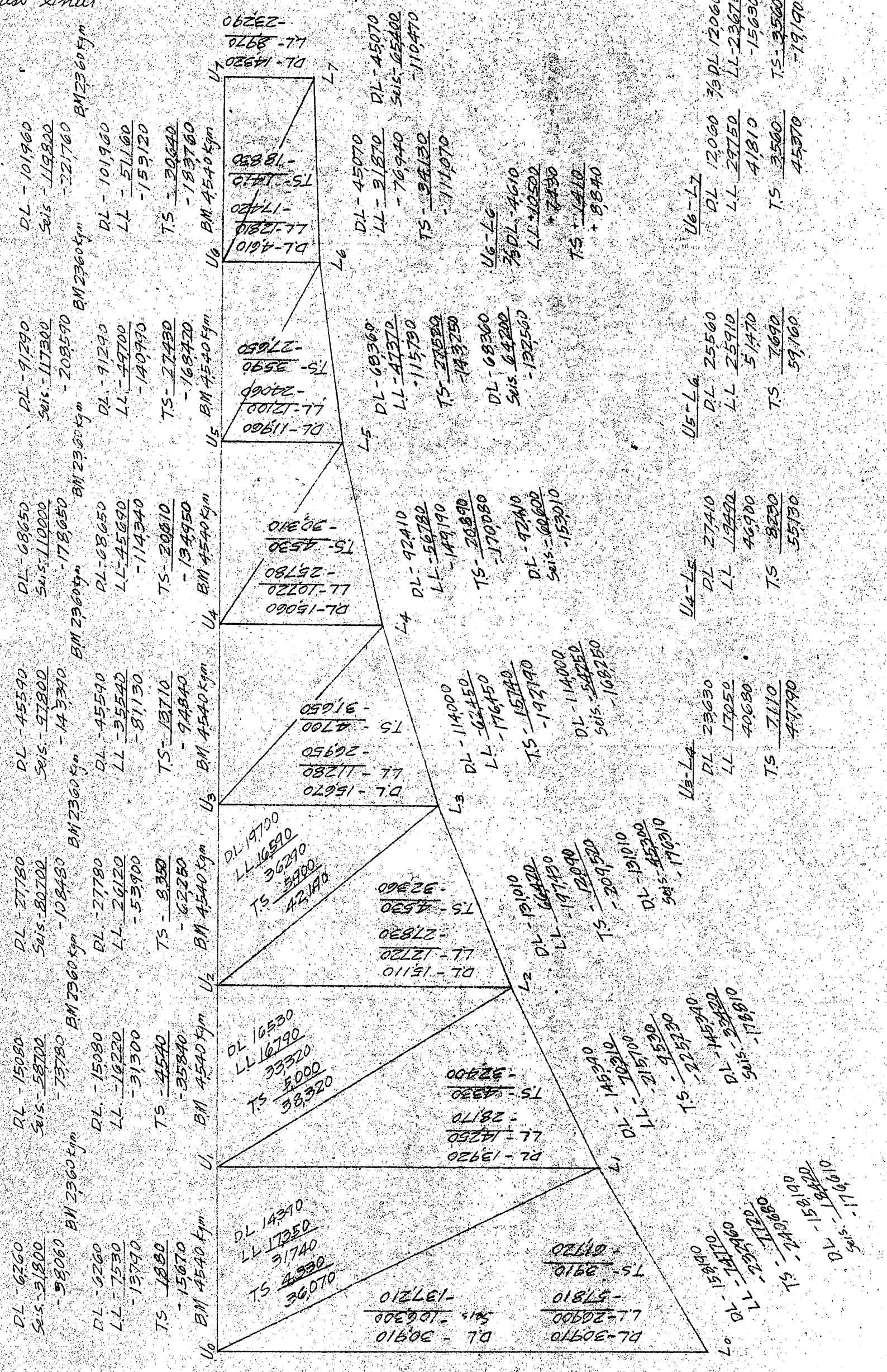
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CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR

檜村橋

Stress Sheet



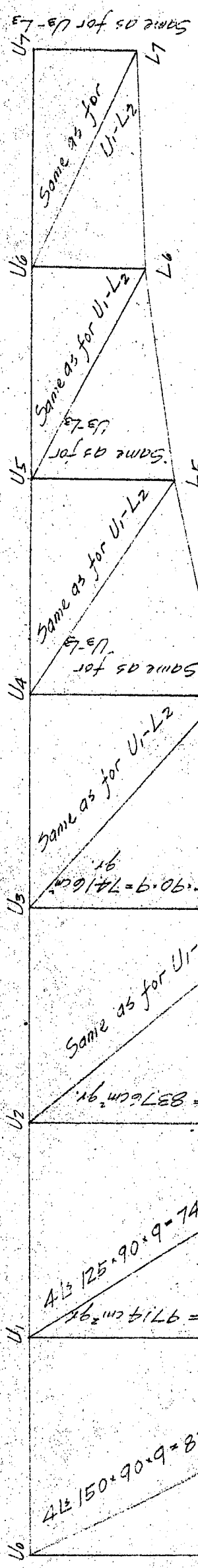
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CALCULATIONS FOR

檜村橋

Cross Section of truss members



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CALCULATIONS FOR

檜村橋

Max. Load on shoe.

Max. horizontal thrust

			Corresponding vertical load	
Dead load thrust	14320 · 9530 =	136470	14320 · 7.0 =	100240
Live load thrust	6470 · 9530 =	61660	6470 · 7.0 =	45290
" extra concentration	2500 · 1.140 =	2850	2500 · 0.5 =	1250
		200980 kg		146780 kg
Temperature change		6660		10800
		207640 kg		157580 kg

Dead load from girder span 1/2 · 4060 · 5325 = 10800 kg

Min. horizontal thrust

			Corresponding vertical load	
Dead load thrust	14320 · 9530 =	136470	14320 · 7.0 =	100240
Temperature change		− 6660		10800
		129810 kg		8000
				119040 kg

Live load from girder span 500 · 300 · 5325 = 8000 kg

Load on shoe during Earthquake

Longitudinal earthquake

			Corresponding vertical load	
Dead load thrust		= 136470		100240
Seismic thrust	111040 · 0.3	= ± 33300		10800
		169770 kg		111040 kg
		or 103170 kg		

Transverse earthquake

			Corresponding vertical load	
Dead load thrust		= —		100240
Seismic thrust	111040 · 0.3	= 33300 kg		10800
				111040 kg
with a longitudinal thrust of 136470 kg				± 131300
				242340 kg
				or − 20260 "

Vertical load due to overturning moment on top lateral ± 106300

" " " " on bottom " 6.5 × 2820 · 820/6 = ± 25000

± 131300 kg.

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GOTANDA, TOKYO.

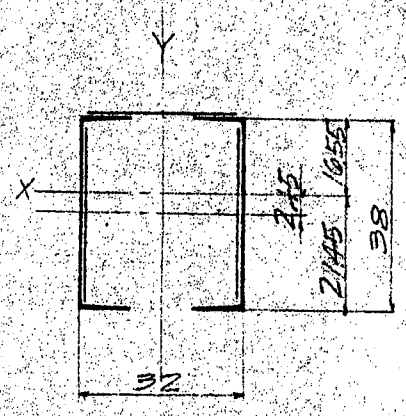
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CALCULATIONS FOR

檜村橋

Sections of Top Chord members

$U_5 - U_7$



Max. stress 153420 kg C for D.L+L.L
" 183760 " " D.L+L.L+T.S.
" 221760 " " D.L + Seis.
Bending moment 4540 kgm " D.L+L.L
" 2360 " " D.L

1 Cov. Pl 310 × 9 = 2790 × 1945 = 5405
2 I's 380×100c 545 kg = 13878
2 Side P/s 300 × 9 = 5400
22068 cm² × 245 = 5405

Moment of inertia about X-X axis
1 Cov. Pl. 2790 × 1700² = 8,060
2 I's 13878 × 245² + 2c 14481 = 29800
2 P/s 5400 × 245² + 18×509/12 = 4370
I_x = 42230 cm⁴

Moment of inertia about Y-Y axis
1 cov. Pl. 9×310³/12 = 2230
2 I's 13878 × 1359² + 2 × 567 = 26760
2 P/s 5400 × 1450² = 11360
I_y = 40350 cm⁴

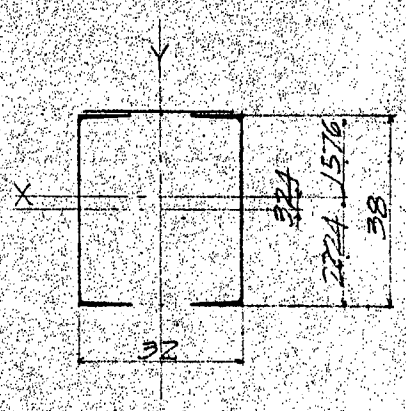
Radius of gyration $r_y = \sqrt{40350/22068} = 1353$ cm, $\ell/r = 520/1353 = 384$
Allowable unit compression = 1500 (1 - 0.0055 × 384) = 1184 kg/cm² use 1000 kg/cm²

D.L + L.L Direct stress = 153120/22068 = 694
Bending stress = 4540×100×1745/42230 = 188
882 kg/cm²

D.L+L.L+T.S Direct stress = 183760/22068 = 832
Bending stress = 188
1020 kg/cm² < 1250 kg/cm²

D.L + Seis. Direct stress = 221760/22068 = 1005
Bending stress = 2360×100×1745/42230 = 98
1103 kg/cm² < 1600 kg/cm²

$U_0 - U_5$



Max. stress 114340 kg C for D.L + L.L
" 134950 " for D.L + L.L + T.S ÷ 125 = 108000 kg C
" 178650 " for D.L + Seis. ÷ 160 = 111700 "
Bending moment 4540 kgm " D.L + L.L
" 2360 " " D.L

1 Cov. Pl 310 × 9 = 2790 × 1945 = 5405
2 I's 380×100c 545 kg = 13878
16668 cm² × 324 = 5405

Moment of inertia about X-X axis
1 Cov. Pl 2790 × 1621² = 7330
2 I's 13878 × 324² + 2c 14481 = 30420
37750 cm⁴

Moment of inertia about Y-Y axis
1 Cov Pl = 2230
2 I's = 26760
28990 cm⁴

Radius of gyration $r_y = \sqrt{28990/16668} = 132$ cm, $\ell/r = 520/132 = 39.4$
Allowable unit compression use 1000 kg/cm²

D.L + L.L Direct stress = 114340/16668 = 686
Bending stress = 4540×100×1666/37750 = 200
886 kg/cm²

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JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

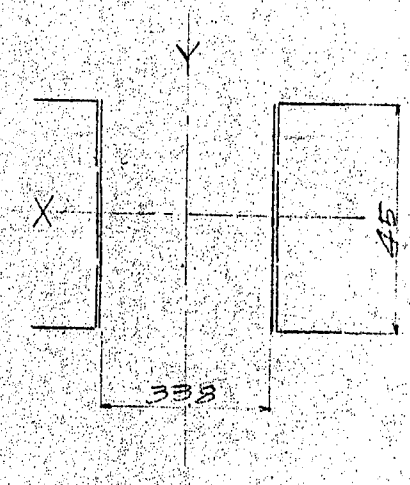
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CALCULATIONS FOR

檜村橋

Sections of Bottom chord members

$L_0 - L_1$



Max. stress 232960 kg c for DL+LL SR = 233.0 cm²
" 240,680 " for DL+LL+T.S " 192.5 "
" 176,610 " for DL+Seis. " 110.4 "

4 L³ 130·130·12 = 119.04 × 2142² + 4c466 = 44320
2 Web P/s 440·9 = 7920 = 12,780
2 side P/s 180·12 = 4320 2·9·24³/12 + 2·12·18³/12 = 1165
24144 cm² I_x = 58265 cm⁴

Moment of inertia about Y-Y axis
4 L³ 119.04 × 2142² + 4c466 = 56520
2 Web P/s 7920 × 1735² = 23840
2 side P/s 4320 × 1840² = 14620
I_y = 94980 cm⁴

Radius of gyration $r_x = \sqrt{\frac{58265}{24144}} = 15.5$ cm $l/r_x = \frac{6029}{155} = 38.9$

Allowable unit compression use 1,000 kg/cm²
unit compression = $\frac{232960}{24144} = 964$ kg/cm²

$L_1 - L_3$

Max. stress 215,700 kg c for DL+LL SR = 215.7 cm²
" 225230 " " DL+LL+T.S 180.3 "
" 178810 " " DL+Seis. 111.7 "

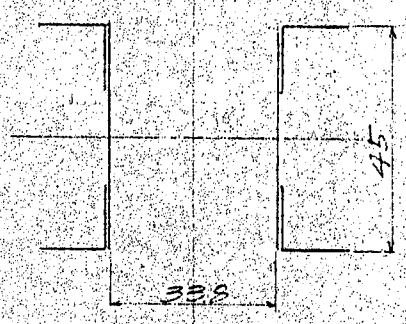
Use cross sectional area + moment of inertia about X-X axis
4 L³ 130·130·12 = 119.04 = 44320
2 Web P/s 440·9 = 7920 = 12,780
2 side P/s 180·9 = 3240 2·9·18³/12 = 870
23064 cm² I_x = 57,970 cm⁴

Moment of inertia about Y-Y axis
4 L³ = 56520
2 Web P/s = 23840
2 side P/s 3240 · 1825² = 10800
I_y = 91,160 cm⁴

Radius of gyration $r_x = \sqrt{\frac{57970}{23064}} = 15.85$ cm $l/r_x = \frac{5806}{15.85} = 36.6$

allowable unit compression use 1,000 kg/cm²
unit compression = $\frac{215700}{23064} = 935$ kg/cm²

$L_3 - L_4$



Max. stress 176450 kg c for DL+LL SR = 176.5 cm²
" 192,190 " " DL+LL+T.S " 153.8 "
" 163250 " " DL+Seis. " 105.2 "

Use Cross sectional area + moment of inertia about X-X axis
4 L³ 130×130×12 = 11904 = 44320
2 Web P/s 440×9 = 7920 = 12,780
19824 cm² I_x = 57,100 cm⁴
Moment of inertia about Y-Y axis I_y = 80,360 cm⁴

Radius of gyration $r_x = \sqrt{\frac{57100}{19824}} = 17.0$ cm, $l/r_x = \frac{5453}{17.0} = 32.1$

Allowable unit compression use 1,000 kg/cm²

unit compression = $\frac{176450}{19824} = 890$ kg/cm²

$L_4 - L_7$

Max. stress 149,190 kg c for DL+LL SR = 149.2 cm²
" 170,080 " " DL+LL+T.S " 136.0 "
" 153,010 " " DL+Seis. " 95.7 "

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR

檜村橋

Use cross sectional area + moment of inertia about x-x axis

4L's 130×130×9 = 9096 × 18.99² + 4×355 = 33940
2 Web P/s 440×9 = 7920 × 2×9×44³/12 = 12780
17016 cm² I_x = 46720 cm⁴

Moment of inertia about y-y axis

4L's 9096 × 2.13² + 4×355 = 42740
2 Web P/s 2200
I_y = 44940 cm⁴

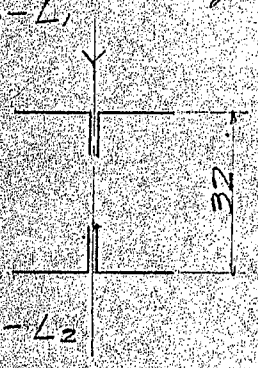
Radius of gyration $r_x = \sqrt{\frac{46720}{17016}} = 16.6$ cm $\quad l/r_x = \frac{533.1}{16.6} = 32.1$

Allowable unit compression use 1000 kg/cm²

Unit compression = $\frac{149190}{17016} = 8.77$ kg/cm²

Sections of diagonal members

U_0-L_1



Max. stress 31,740 kg T for D.L + L.L SR = 265 cm²
" 36,070 " " D.L + L.L + T.S " 240 "

use 4L's 150×90×9 = 83.76 − 18.0 = 65.76 cm² net
r_y = 7.20 cm
$l/r_y = \frac{1178.0}{7.20} = 166 < 200$

U_1-L_2

Max. stress 33320 kg T for D.L + L.L SR = 27.8 cm²
" 38,320 " " D.L + L.L + T.S " 25.6 "

use 4L's 125×90×9 = 74.16 − 18.0 = 56.16 cm² net
r_y = 5.87 cm, $l/r_y = \frac{974.1}{5.87} = 166 < 200$

U_2-L_3, U_3-L_4, U_4-L_5, U_5-L_6

U_2-L_3	Max. stress	36290 kg T	for	D.L + L.L	SR = 30.2 cm²
"	"	42190 "	"	D.L + L.L + T.S	" 28.1 "
U_3-L_4	"	40,680 "	"	D.L + L.L	" 33.9 "
"	"	47,790 "	"	D.L + L.L + T.S	" 31.9 "
U_4-L_5	"	46900 "	"	D.L + L.L	" 39.1 "
"	"	55,130 "	"	D.L + L.L − T.S	" 36.8 "
U_5-L_6	"	51,470 "	"	D.L + L.L	" 42.9 "
"	"	59,160 "	"	D.L + L.L + T.S	" 37.4 "

use 4L's 125×90×9 = 74.16 − 18.0 = 56.16 cm² net
r_y = 5.87 cm, $l/r_y = \frac{805.4}{5.87} = 137$

U_6-L_7

Max. stress	41,810 kg T	for	D.L + L.L	SR = 34.9 cm²
"	15,630 kg C	"	"	" 22.7 "
"	45,370 kg T	"	D.L + L.L − T.S	" 30.3 "
"	19,190 kg C	"	"	" 22.3 "

use 4L's 125×90×9 = 74.16 − 18.0 = 56.16 cm² net
r_y = 5.87 cm, $l/r_y = \frac{577.0}{5.87} = 98.3 < 120$

Allowable unit compression 1500(1 − 0.0055 × 98.3) = 690 kg/cm²

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

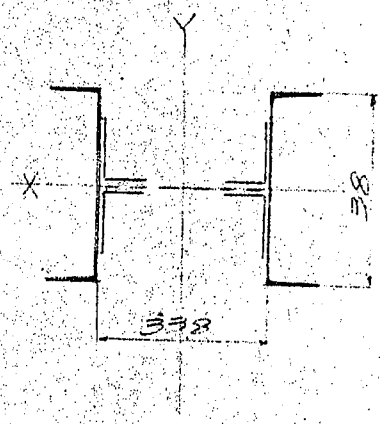
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CALCULATIONS FOR

楢村橋

Sections for Vertical members

U_0-L_0



Max. stress 57810 kg C for D.L + L.L , SR = 102.5 cm²
" 61,720 " " D.L + L.L + T.S " 87.6 cm²
" 137,210 " " D.L + Sets. " 152.1 cm²

Use cross sectional area + moment of inertia about X-X axis

2 ⊟ 380 × 100 − 545 kg = 138.78 I_x = 14,481 = 28,960

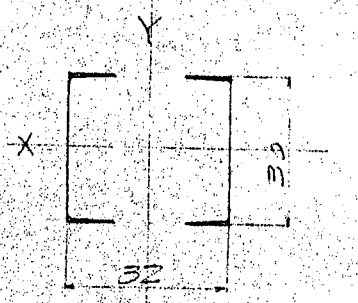
4 L 125 × 90 × 9 = 74.16 × 436² + 4 × 287 = 2560

212.74 cm² I_x = 31,520 cm⁴

Radius of gyration $r_x = \sqrt{\frac{31520}{212.64}}$ = 12.2 cm , $\ell/r_x = \frac{13844}{12.2}$ = 113.5

Allowable unit compression 1500(1 − 0.0055 × 113.5) = 564 kg/cm²

U_1-L_1



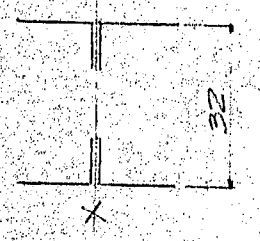
Max. stress 28170 kg C for D.L + L.L SR = 38.9 cm²
" 32,400 " " D.L + L.L + T.S " 35.8 "

use 2 ⊟ 300 × 90 − 38.1 kg = 97.14 cm²

I_x = 12,870 cm⁴ , r_x = 11.51 cm , $\ell/r_x = \frac{1081.9}{11.51}$ = 94.0

Allowable unit compression 1500(1 − 0.0055 × 94.0) = 724 kg/cm²

U_2-L_2



Max. stress 27830 kg C for D.L + L.L SR = 50.4 cm²
" 32,360 " " D.L + L.L + T.S " 46.9 "

use 4 L 150 × 90 × 9 = 83.76 cm²

r_x = 7.20 cm , $\ell/r_x = \frac{82.63}{7.20}$ = 115

Allowable unit compression 1500(1 − 0.0055 × 115) = 552 kg/cm²

U_3-L_3, U_4-L_4, U_5-L_5, U_6-L_6, U_7-L_7

U_3-L_3	Max. stress	26950 kg C	for D.L + L.L	SR = 42.6 cm²	gross
"	"	31,050 "	" D.L + L.L + T.S	" 40.0 "	"
U_4-L_4	"	25780 "	" D.L + L.L	" 30.0 "	"
"	"	30,310 "	" D.L + L.L + T.S	" 28.2 "	"
U_5-L_5	"	24060 "	" D.L + L.L	" 24.1 "	"
"	"	27,650 "	" D.L + L.L + T.S	" 22.1 "	"
U_6-L_6	"	17,420 "	" D.L + L.L	" 17.4 "	"
"	"	18,830 "	" D.L + L.L + T.S	" 15.1 "	"
"	"	7430 kg T	" D.L + L.L	" 6.2 "	net
"	"	8840 "	" D.L + L.L + T.S	" 7.4 "	"
U_7-L_7	"	23,240 kg C	" D.L + L.L	" 23.3 "	gross

use 4 L 125 × 90 × 9 = 74.16 cm² − 18.0 = 56.16 cm² net

r_x = 5.87 cm

	ℓ	r_x	ℓ/r_x	allowable unit compression
U_3-L_3	617.7 ÷	5.87 =	105	633 kg/cm²
U_4-L_4	456.0 "		77.6	860 "
U_5-L_5	341.3 "		58.1	1,000 "
U_6-L_6	273.5 "		46.6	1,000 "
U_7-L_7	250.0 "		42.6	1,000 "

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CALCULATIONS FOR

檜村橋

Approximate weight of main truss.

Member	No.	Sectional area	unit wt.	length	weight
U_0-U_1	2	16668	c 0.785 ×	5200 =	1360
U_1-U_2	"	"	"	"	1360
U_2-U_3	"	"	"	"	1360
U_3-U_4	"	"	"	"	1360
U_4-U_5	"	"	"	"	1360
U_5-U_6	"	22068	"	"	1800
U_6-U_7	"	"	"	"	1800
					10400
L_0-L_1	2	24144	c 0.785 ×	6029 =	2285
L_1-L_2	"	23064	"	5806	2102
L_2-L_3	"	"	"	5613	2032
L_3-L_4	"	19824	"	5453	1697
L_4-L_5	"	17016	"	5331	1424
L_5-L_6	"	"	"	5247	1402
L_6-L_7	"	"	"	5205	1390
					12332
U_0-L_1	2	8376	c 0.785 ×	11.480 =	1575
U_1-L_2	"	7416	"	9741	1134
U_2-L_3	"	"	"	8054	937
U_3-L_4	"	"	"	6899	803
U_4-L_5	"	"	"	6206	722
U_5-L_6	"	"	"	5863	682
U_6-L_7	"	"	"	5770	671
					6524
U_0-L_0	2	21294	c 0.785 ×	13844 =	4625
U_1-L_1	"	9714	"	10819	1649
U_2-L_2	"	8376	"	8263	1087
U_3-L_3	"	7416	"	6177	719
U_4-L_4	"	"	"	4560	531
U_5-L_5	"	"	"	3413	397
U_6-L_6	"	"	"	2735	318
U_7-L_7	1	"	"	2500	145
					9471
			for one truss		38727
			Details 45%		17433
					56160 kg

for one span 2c 56,160 = 112,320 kg

Summary of Structural steel for truss span

Stringers	3 c 600 = 180	·	728 =	13,100
floor beam	15 c		780 =	11,700
top lateral bracing			=	4370
bottom lateral bracings			=	7950
bottom struts	15 c		330 =	4950
sway and portal bracing			=	11,620
trusses			=	112,320
expansion joints say			=	1,200
shoe say	4 c		1,500 =	6000
				173,210 kg

Call this 1730 kg ton

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CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR

檜村橋

Vertical deflection of truss at crown

General equation of deflection $\Delta = \Sigma \frac{SL}{EA} T$ or $\frac{1}{E} \Sigma \frac{SL}{A} T$

Where Δ = Deflection at any point in cm.
S = Stresses of each members.
T = Stresses of each members due to a unit load on the panel point at which deflection is desired in the direction of the deflection.
L = Length of each member in cm.
A = Modulus of elasticity in kg/cm² = 2,100,000

Dead load stress Panel load 14,320 kg			L.L. stress due to full load, panel load 6470 kg		L.L. stress due to single conc. panel load 2500 kg		Total live load stress
Member	Σ ordinate	D.L. stress	Σ ordinate	stress	Crown ord.	stress	
U_0-U_1	-0437	-6260	-0437	-2830	0081	200	-2630
U_1-U_2	-1053	-15080	-1053	-6810	0148	370	-6440
U_2-U_3	-1940	-27780	-1940	-12550	0167	420	-12130
U_3-U_4	-3184	-45590	-3184	-20600	0066	170	-20430
U_4-U_5	-4794	-68650	-4794	-31020	-0282	-710	-31730
U_5-U_6	-6375	-91290	-6375	-41250	-1008	-2520	-43770
U_6-U_7	-7120	-101960	-7120	-46070	-2036	-5090	-51160
L_0-L_1	-11047	-158190	-11047	-7470	-1321	-3300	-74770
L_1-L_2	-10153	-145390	-10153	-65690	-1363	-3410	-69100
L_2-L_3	-9149	-131010	-9149	-59190	-1390	-3480	-62670
L_3-L_4	-7961	-114000	-7961	-51510	-1371	-3430	-54940
L_4-L_5	-6453	-92410	-6453	-41750	-1237	-3090	-44840
L_5-L_6	-4774	-68360	-4774	-30890	-0866	-2170	-33060
L_6-L_7	-3147	-45070	-3147	-20360	-0131	-330	-20690
U_0-L_1	1005	14390	1005	6500	-0187	-470	6030
U_1-L_2	1154	16530	1154	7470	-0125	-310	7160
U_2-L_3	1376	19700	1376	8900	-0029	-70	8830
U_3-L_4	1650	23630	1650	10680	0132	330	11010
U_4-L_5	1914	27410	1914	12380	0415	1040	13420
U_5-L_6	1785	25560	1785	11550	0819	2050	13600
U_6-L_7	0842	12060	0842	5450	1140	2850	8300
U_0-L_0	-0904	-12950	-0904	-5850	0169	420	-5430
U_1-L_1	-0972	-13920	-0972	-6290	0106	270	-6020
U_2-L_2	-1055	-15110	-1055	-6830	0022	60	-6770
U_3-L_3	-1094	-15670	-1094	-7080	-0088	-220	-7300
U_4-L_4	-1052	-15060	-1052	-6810	-0228	-570	-7380
U_5-L_5	-0835	-11960	-0835	-5400	-0383	-960	-6360
U_6-L_6	-0322	-4160	-0322	-2080	-0488	-1220	-3300
U_7-L_7	-1000	-14320	-1000	-6470	-1000	-2500	-8970

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GOTANDA, TOKYO.

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CALCULATIONS FOR

檜村橋

Vertical deflection of truss at crown

Member	L/A	T	LT/A	S_D	$S_D LT/A$	S_L	$S_L LT/A$	S_x	$S_x LT/A$
U_0-U_1	3.114	0.081	0.252	-6,260	-1,580	-2630	-660	1,880	470
U_1-U_2	"	0.148	0461	-15080	-6950	-6440	-2970	4,540	2090
U_2-U_3	"	0.167	0.520	-27780	-14450	-12130	-6310	8,350	4340
U_3-U_4	"	0.066	0.206	-45590	-9390	-20,430	-4210	13,710	2820
U_4-U_5	"	-0282	-0.878	-68,650	60270	-31730	27,860	20,610	-18,100
U_5-U_6	2.353	-1,008	-2372	-91,290	216540	-43770	103,820	27,430	-65060
U_6-U_7	"	-2,036	-4791	-101,960	488490	-51,160	245,110	30,640	-146800
L_0-L_1	2502	-1321	-3305	-158,190	522,820	-74770	247,110	-7720	25510
L_1-L_2	2513	-1363	-3425	-145390	497960	-69100	236670	-9530	32,640
L_2-L_3	2430	-1390	-3378	-131,010	442,600	-62670	211,700	-12,090	40,840
L_3-L_4	2754	-1371	-3776	-114,000	430,460	-54940	207,450	-15,740	59,430
L_4-L_5	3.136	-1237	-3878	-92,410	358,460	-44840	173930	-20,890	81,010
L_5-L_6	3087	-0.866	-2673	-68360	182,730	-33060	88370	-27,520	73,560
L_6-L_7	3062	-0.131	-0401	-45,070	18,070	-20,690	8,300	-34,130	13,690
U_0-L_1	13,770	-0.187	-2575	14,390	-37,050	6030	-15530	-4330	11450
U_1-L_2	13164	-0.125	-1646	16,530	-27,210	7,160	-11,790	-5000	8230
U_2-L_3	10.884	-0029	-0316	19,700	-6230	8830	-2790	-5900	1860
U_3-L_4	9323	0.132	1231	23630	29090	11,010	13,550	-7,110	-8750
U_4-L_5	8386	0415	3480	27,410	95390	13420	46700	-8230	-28,640
U_5-L_6	7923	0.819	6489	25,560	165860	13600	88250	-7,690	-49900
U_6-L_7	7797	1,140	8889	12,060	107,200	8300	73780	-3560	-31,640
U_0-L_0	5697	0.169	0963	-12,950	-12470	-5430	-5230	3910	3770
U_1-L_1	11.154	0.106	1182	-13920	-16450	-6020	-7,120	4230	5000
U_2-L_2	9498	0.022	0209	-15,110	-3160	-6770	-1410	4,530	950
U_3-L_3	8347	-0088	-0735	-15,670	11,520	-7300	5370	4,700	-3,450
U_4-L_4	6162	-0228	-1405	-15060	21,160	-7380	10,370	4,530	-6360
U_5-L_5	4612	-0383	-1766	-11,960	21,120	-6360	11,230	3,590	-6340
U_6-L_6	3696	-0488	-1804	-4610	8320	-3300	5950	1,410	-2540
U_7-L_7	3378	-1000	-3378	-14320	48370	-8470	30300		
					+3726430		+1835820		+367660
					-134940		-58020		-367580
					+3591490		+1777800		+80

Dead load deflection $\Delta_d = \frac{2 \cdot 3591490}{2100000} = 3.42$ cm

Live load deflection $\Delta_l = \frac{2 \cdot 1777800}{2100000} = 1.69$ cm

Temperature deflection 30° C rise

$\Delta_t = \frac{+2 \cdot 80}{2100000} = 0.00001$ cm neglected

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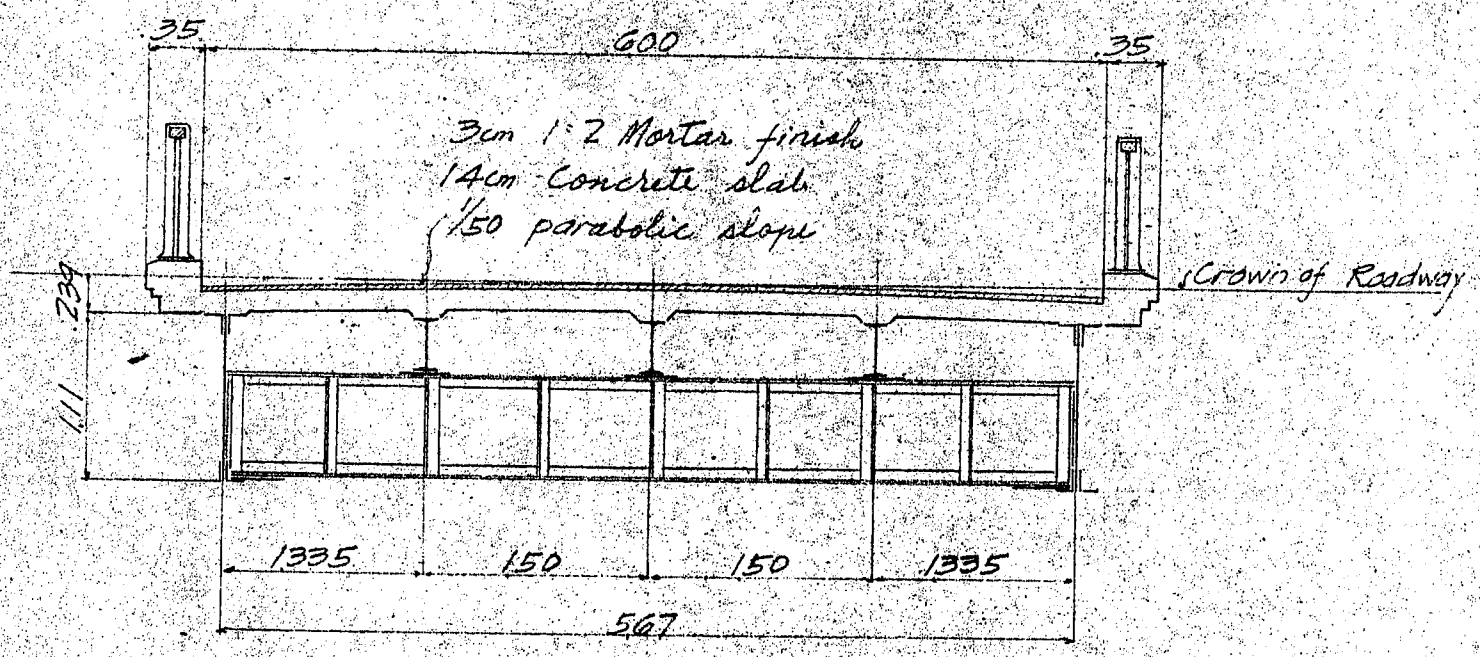
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GOTANDA, TOKYO.

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CHECKED BY J. M. DATE 11-12-5 PAGE NO 43

CALCULATIONS FOR

檜村橋

Cross section of the girder span



Scale 1:50

Floor slab

used same as for arch span

Over hanging slab

Dead load moment

Floor and pavement	395 · 0.165 = 65	· 0.083 =	5
coping say	269	· 0.35 =	94
handrail	85	· 0.36 =	31

shear = 419 kg moment = 130 kgm

Live load

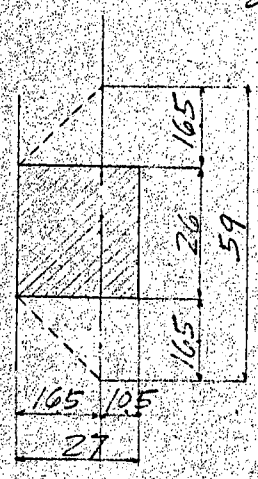
Motor trucks rear wheel concentration with impact 2925 kg

Outside of tyre in contact with curb line assumed and distribution assumed as follows,

Longitudinal distribution $a = 26$ cm

distribution of slab $2 \times 0.165 = 33$

59 cm



Transverse distribution $b = 24 + 3 = 27$ cm

Load per meter strip $2925 \times \frac{165}{27} \times \frac{100}{59} = 3030$ kg

moment $3030 \times \frac{0.165}{2} = 250$ kgm

shear 3030 kg

Summary of moment and shear

	moment	shear
Dead load	130	419
Live load	250	3030
	380 kg	3449 kgm

Effective depth required for slab

$f_s = 1200$ kg/cm²

$f_c = 45$

$R = \frac{M}{bd^2}$, $d = \sqrt{\frac{M}{bR}}$ where $R = 7.13$

$d = \sqrt{\frac{380 \cdot 100}{100 \cdot 7.13}} = 7.3$ cm

use 14 cm slab, insulation at bottom 2.5 cm, effective depth 11.5 cm

use 12 mm bars 15 cm spacing

unit shear $\frac{3449}{100 \cdot 7/8 \cdot 115} = 3.4$ kg/cm²

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CALCULATIONS FOR

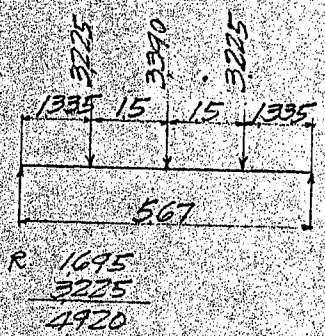
檜村橋

Stringer

used same as for arch span.

Design of intermediate floor beam, span length 5.67 meters spacing 5.20 meters

Dead load



Concentration from center stringer = 3390 kg
Concentration from outside stringer
Floor and pavement 395 · 1.418 = 560
I-beam assumed 60
620 kg per m.

620 · 5.20 = 3225 kg

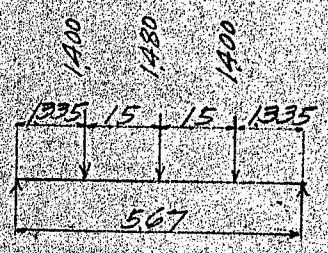
Moment at center
4920 · 2.835 = 13950
3225 · 1.50 = − 4840
9110 kgm
Beam assumed as 135 kg/m. 1/8 · 135 · 5.67² = 540
9650 kgm

End shear from concentration 4920
" beam weight 1/2 · 135 · 5.67 = 380
5300 kg

Live load

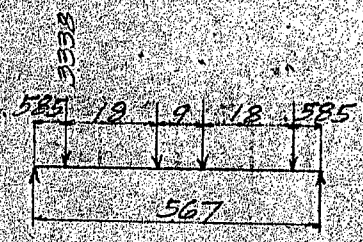
2 motor trucks on roadway, front and rear filled with uniform load
motor truck rear wheel concentration with impact 2925 kg
" " front " " 975 "
motor truck loading rear wheel 2925
" front " 975 · 22/5.72 = 413
3338 kg

uniform load $\frac{500 \cdot 1.7^2}{2 \cdot 5.20}$ = 139
$\frac{500 \cdot 4.20^2}{2 \cdot 5.20}$ = 848
987 kg per meter

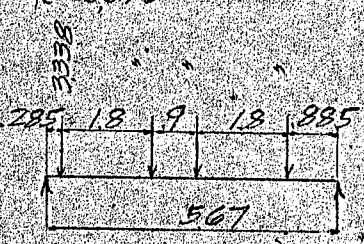


concentration of center stringer 987 · 1.50 = 1,480 kg
" outside " 987 · 1.418 = 1,400 "

moment due to uniform load
2,140 · 2.835 = 6060
1400 · 1.500 = − 2100
3960 kgm



Moment due to wheel concentration 6676 · 2.385 = 15920
3338 · 1.80 = − 6000
9920 kgm
Summary of live load moment 13880 kgm



Max. end shear
uniform load 2140
motor truck wheel concentration 4 · 3338 · 3.135/5.67 = 7380
3338 · 1.05/1.335 = − 2625
6895 kg

Summary of moments & shears

	moment	shear
Dead load	9650	5300
Live load	13880	6895
	23530 kgm	12195 kg

Web assumed 650 · 9 = 585 cm²; 1/8" web = 7.3 cm²
back to back of L's 660 mm
Effective depth 660 − 2 · 1.9 = 62.2 cm
Flange stress $\frac{23530}{0.622}$ = 37850 kg

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CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR

檜村橋

Required section 37,850 ÷ 1,200 = 31.6 cm²
1/8 web area 73
243 cm² net
use 2L 100·75·10 = 33.00 - 4.40 = 28.6 cm²

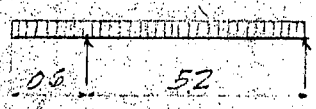
Approximate weight of intermediate floor beam

web plate	1Pl. 650 · 9	= 45.92	× 5.65	= 260
flange	4L 100·75·10	= 13.0	× 5.65	= 294
stiffeners	14L 90·75·9	= 11.0	· 0.66	= 102
end connection	4L 150·100·9	= 17.1	· 0.64	= 44
filler	4Fills 145 · 10	= 11.38	· 0.50	= 23
	3 Pls 290 · 9	= 20.49	· 0.40	= 25
				748 kg
Rivet heads + misc.				17
				765 kg

765 ÷ 5.67 = 135 kg per linear meter

Design of end floor beam, span length 5.67 meter
Dead load

Concentration from center stringer 652 × 5.80 × 290/520 = 2,110 kg
outside " 620 × 5.80 × 290/520 = 2,010 "

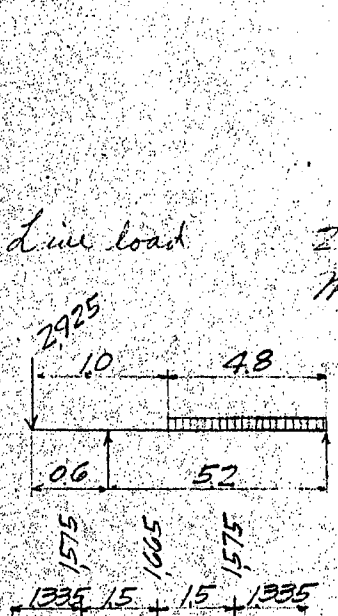


Reaction R = 1055 + 2,010 = 3,065 kg

Moment at center 3,065 · 2.835 = 8690
2,010 · 1.50 = - 3020
5,670 kgm
due to beam weight 540
6210 kgm

End shear from concentration 3,065
beam weight 380
3,445 kg

Live load 2 motor trucks on roadway, front filled with uniform load
Motor truck rear wheel concentration with impact 2,925 kg
Motor truck loading on floor beam 2925 × 5.8/5.2 = 3,260 kg



Uniform load on floor beam 500 × 4.8 × 24/5.2 = 1,110 kg

Concentration of center stringer 1,110 × 1.50 = 1,665 kg
" outside " 1,110 × 1.418 = 1,575 "

Moment due to uniform load 2,408 · 2.835 = 6820
1,575 · 1.50 = - 2360
4460 kgm

Moment due to wheel concentration 6,520 · 2.385 = 15,550
3,260 · 1.80 = - 5870
9,680 kgm

Summary of live load moment 14,140 kgm

Max. end shear uniform load 2,408
motor truck wheel concentration 4 × 3,260 × 3.135/5.67 = 7,210
3,260 × .105/1.335 = - 2,563
7,055 kg

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JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

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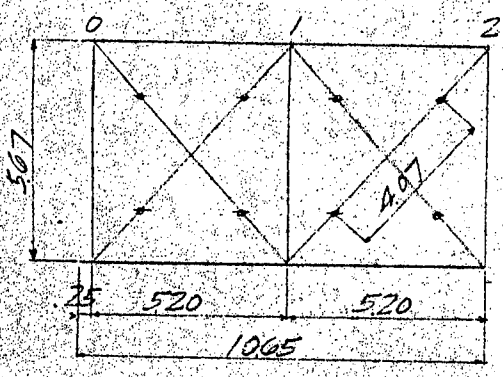
CALCULATIONS FOR

檜村橋

Summary of moment and shear

	moment	shear	
Dead load	6210	3445	used same as for intermediate floor beam
Live load	14140	7055	
	20350 kgm	10500 kg	

Design of lateral bracing.



Diagonal length $5.67^2 = 321489$
$520^2 = 270400$
$591889 - 7693$

Coefficient $= \frac{769.3}{567} = 1358$

Wind load assumed 400 kg per linear meter of bridge.
Panel load on 0 $400 \cdot 2.85 = 1140$ kg
" 1 $400 \cdot 520 = 2080$ kg
End shear

$1140 \cdot \frac{1040}{1065} = 1110$

$2080 \cdot \frac{520}{1065} = 1015$

2125 kg

Shear at panel 0-1 2125 − 1140 = 985 kg diagonal stress $985 \cdot 1358 = 1340$ kg T
1-2 3220 − 2125 = 1095 " " $1095 \cdot$ " $= 1490$ " T

Stresses during earthquake. acceleration 3000 mm/sec² or k = 0.3
Dead load

Floor and pavement	6.00 × 395	= 2370
Coping	2 × 269	= 538
handrail	2 × 85	= 170
wireing	2 × 50	= 100
stringer	3 × 60	= 180
floor beam	765 ÷ 520	= 147
lateral bracing assumed		80
main girder assumed		480
		4065 kg per lin. meter

Dead panel load on 0 $4065 \cdot 285 = 11580$ kg
" 1 $4065 \cdot 520 = 21150$ "

Seismic force on 0 $0.3 \cdot 11580 = 3475$ kg
" 1 $0.3 \cdot 21150 = 6340$ "

End shear at 2 $3475 \cdot \frac{25}{1065} = 80$

$6340 \cdot \frac{545}{1065} = 3245$

3325 kg

diagonal stress
$3325 \cdot 1358 = 4515$ kg
Required net section
$4515 \div 1920 = 2.35$ cm² net

use 1 L 75·75·9 = 12.69 − 1.98 = 10.71 cm² net

max value of l/r 407 ÷ 225 = 181

増田淳氏関係資料
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JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO

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CALCULATIONS FOR

檜村橋

Approximate weight of latical bracings

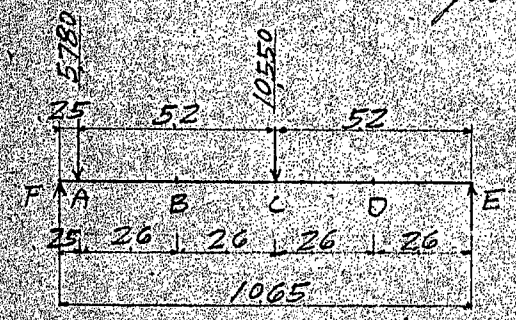
Diagonals	4 L's	75·75·9	= 9.96 kg · 700 =	279
Gusset plate	6 P/s	460 · 9	= 3250 · 0.70 =	137
Center connection	2 P/s	280 · 9	= 1978 · 0.61 =	24
hangers	8		= 130 =	104
rivet heads say				16
				560 kg

560 ÷ 1065 = 53kg per linear meter of girder

Design of main girder, Span length 1065 meter

Dead load

Floor and pavement	600 · 395	= 2370
coping	2 c	269 = 538
handrail	2 c	85 = 170
wiring	2 c	50 = 100
stringer	3 c	60 = 180
floor beam	765 ÷ 520	= 147
latral bracings		53
main girder assumed		502
		4060 kg / linear meter



Panel load on A 2030 · 2.85 = 5,780 kg
" C 2030 · 5.20 = 10,550 "

Reaction at E $10550 \cdot \frac{545}{1065} = 5400$ at F 16,330

$5780 \cdot \frac{25}{1065} = 135$ 5535

5,535 kg 10,795 kg

Dead load moment at D

5535 · 26 = 14,380 kgm load on shear $2030 \cdot \frac{1125}{2 \cdot 1065} = 1206$ kg

Dead load moment at C 5535 · 52 = 28,760 "

Dead load moment at B 5535 · 78 = 43,150
10,550 · 26 = - 27,430
15,720 "

Dead load moment at A 10,795 · 25 = 2678 "

Dead load shear at F 10,795 kg

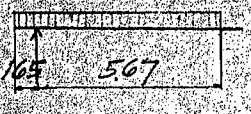
Live load

uniform load on roadway $w = \frac{100000}{170 + 1065} = 552$ kg/m² use 500 kg/m²

motor truck rear wheel 2,250
impact coefficient $\frac{20}{60 + 1065} = 28.3\%$ 640
2890 kg

Front wheel concentration with impact 2,890 ÷ 3 = 965 kg

uniform load on girder $\frac{500 \cdot 5.835^2}{2 \cdot 5.67} = 1,500$ kg/m



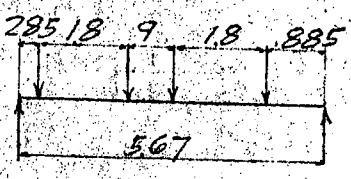
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JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR

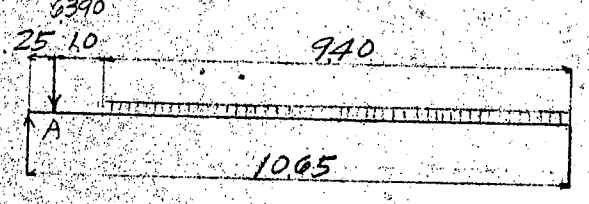
檜村橋



Rear wheel concentration on girder $2{,}890 \times 4 \cdot \frac{3135}{5.67} = 6{,}390$ kg

Front wheel concentration " $1/3 \cdot 6{,}390 = 2{,}130$ kg

Live load moment at A



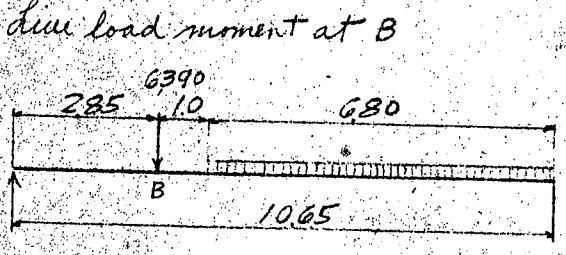
Reaction $6390 \cdot \frac{10.4}{10.65} = 6{,}240$

$1500 \cdot \frac{9.40^2}{2 \cdot 10.65} = 6220$

$12{,}460$ kg

moment $12{,}460 \times 0.25 = 3{,}110$ kgm

Live load moment at B



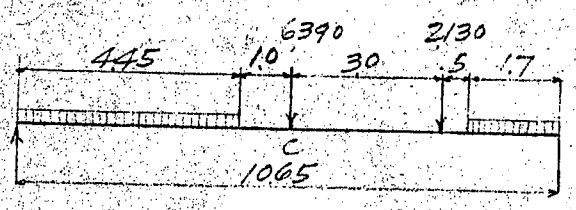
Reaction $6390 \cdot \frac{7.8}{10.65} = 4{,}680$

$1{,}500 \cdot \frac{6.80^2}{2 \cdot 10.65} = 3260$

$7{,}940$ kg

Moment $7{,}940 \times 2.85 = 22{,}620$ kgm

Live load moment at C



Reaction $6390 \cdot \frac{5.2}{10.65} = 3120$

$2130 \cdot \frac{2.2}{10.65} = 440$

$1500 \cdot \frac{1.7^2}{2 \cdot 10.65} = 200$

$1500 \cdot 4.45 \cdot \frac{8.425}{10.65} = 5280$

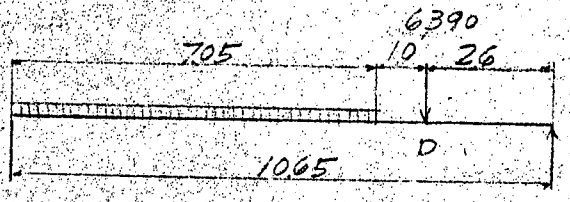
$9{,}040$ kg

Moment $9{,}040 \times 5.45 = 49300$

$1500 \times 4.45 \times 3.225 = -21500$

$27{,}800$ kgm

Live load moment at D



Reaction $6390 \times \frac{8.05}{10.65} = 4830$

$1500 \times \frac{7.05^2}{2 \cdot 10.65} = 3500$

8330 kg

Moment $8330 \times 2.60 = 21650$ kgm

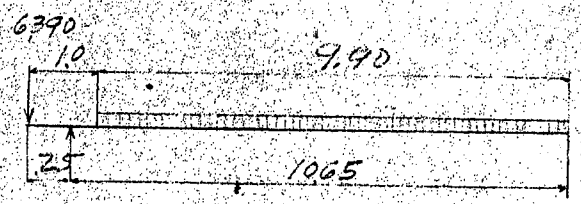
Max end shear

$1500 \cdot \frac{9.65^2}{2 \cdot 10.65} = 6560$

6390

$12{,}950$ kg

Max load on shoe



$1500 \cdot \frac{9.40^2}{2 \cdot 10.65} = 6400$

$6390 \cdot \frac{10.40}{10.65} = 6540$

$12{,}940$ kg

Summary of moment, shear and load on shoe

	Moment at A	B	C	D	shear	load on shoe
Dead load	2670	15720	28760	14380	10795	12060
Live load	3110	22620	27800	21650	12950	13440
	5780 kgm	38340 kgm	56560 kgm	36030 kgm	23745 kg	25500 kg

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

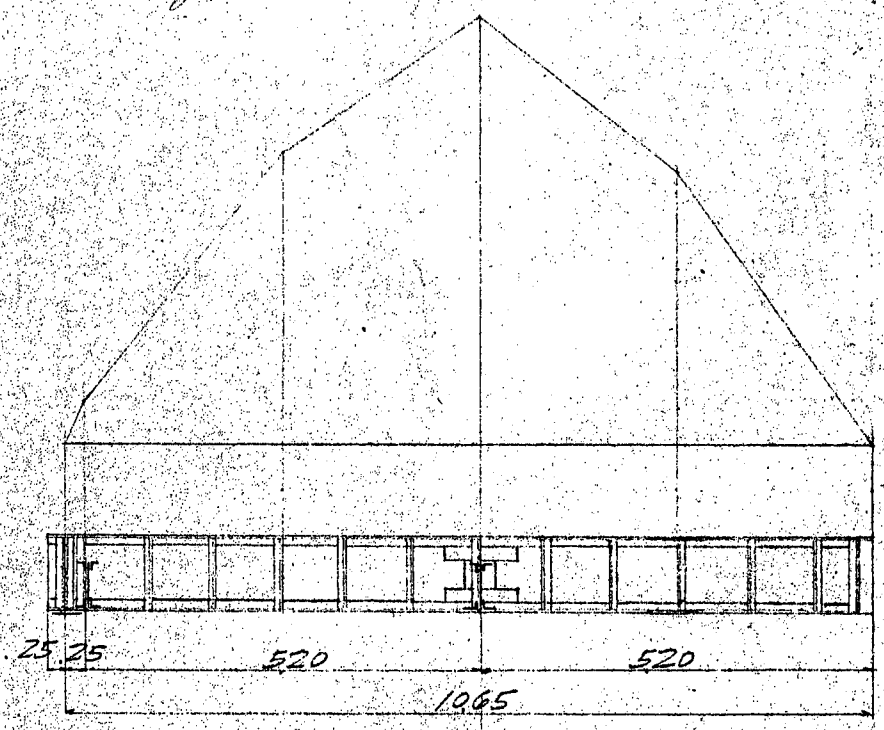
JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR

楠村橋

Moment diagram



Scale of span 1:100
Scale of moment 1cm = 10,000 kgm

Section of main girder
web plate assumed 1030×9 = 927 cm²
1/8 e web = 116 cm²
back to back of angle 1040 mm
Effective depth 104 - 2e 229 = 99.42 cm
Flange stress 56560 ÷ 0.9942 = 57000 kg
Required section 57000 ÷ 1200 = 47.5 cm²
1/8 web area 11.6
35.9 cm² net

use 2L 150×100×9 - 43.68 - 7.42 = 35.76 cm² net

unit shear $\frac{23745}{103.\times 9} = 256$ kg/cm²

Weight of one girder with details = 2200 about
Rivet heads assumed 5% 110
2310 kgs

2310 ÷ 10.9 = 212 kg per linear metre of girder

Approximate weight of girder span

Stringers	3 @ 60	. 10.90	=	1960
Floor beams	2 × 765		=	1530
Lateral bracing say				560
2 girders @ 2310				4620
Misc. say				600
				9270 kg

For 2 spans 2 × 9.27 = 18.54 tons.

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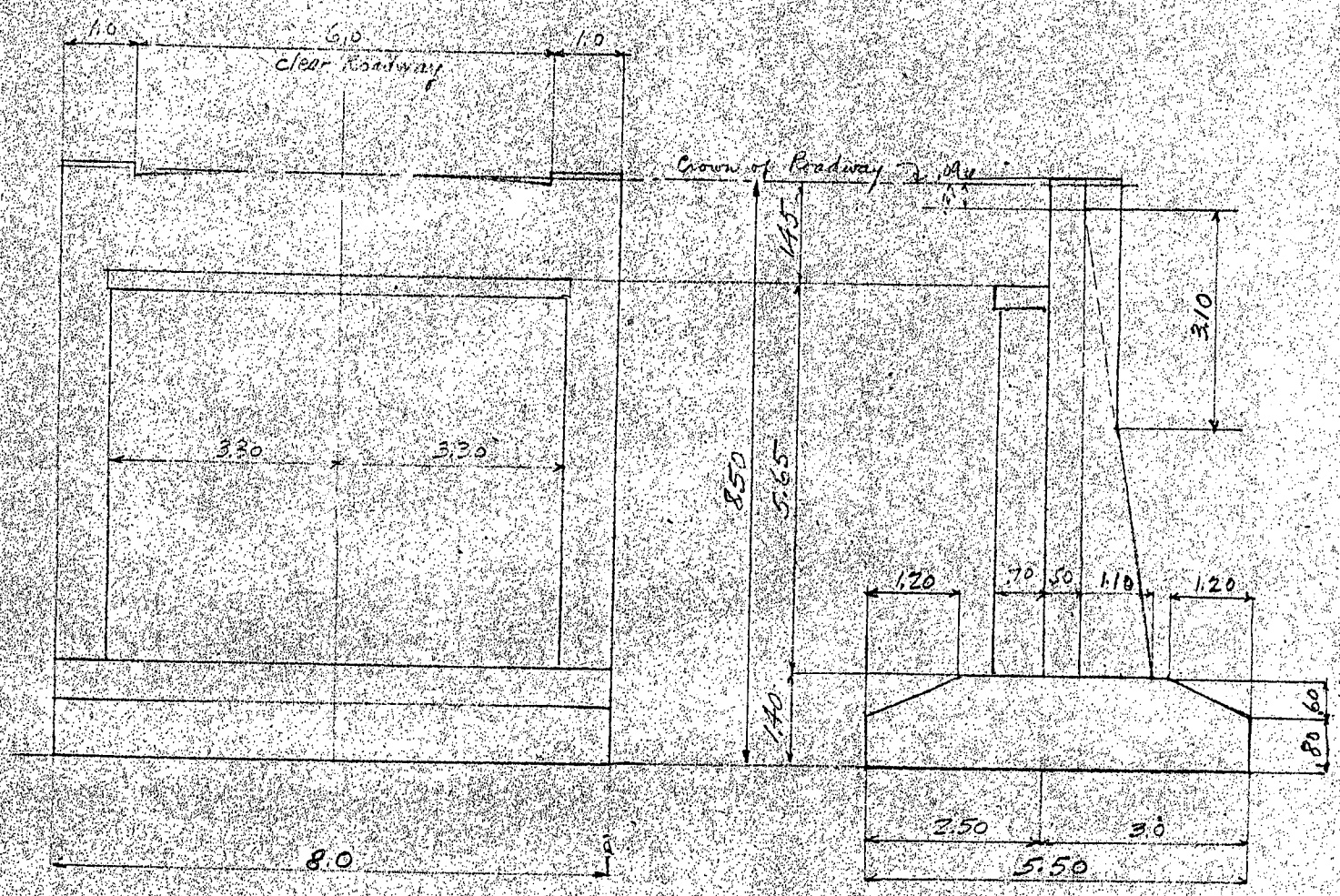
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CALCULATIONS FOR

檜村橋

Design of Girder abutment.
West abutment. Height crown of roadway to bottom of base 8.5 meters, cross section and elevation as shown below in sketch.



Stability of Abutment

To find Center of gravity of wall and Earth fill.

Concrete body

			volume m³		arm from heel		moment
1	Entrance Posts	2 @ 1.0 × 1.0 × 1.5	= 3.00	×	2.50	=	7.50
2	Corners below Posts	2 @ .36 × 1.0 × 1.0	= .72	×	2.50	=	1.80
3	" " "	2 @ .50 × 3.10/2 × 1.0	= 1.55	×	2.17	=	3.36
4	Parapet wall vert.	0.50 × 7.10 × 8.0	= 28.40	×	2.75	=	78.10
5	" " batter	1.10/2 × 6.83 × 8.0	= 30.00	×	2.13	=	63.90
6	girder seat coping	0.30 × .80 × 6.80	= 1.63	×	3.40	=	5.54
7	girder seat body	0.70 × 6.60 × 5.35	= 24.70	×	3.35	=	82.70
8	Base	6.98 × 8.0	= 55.85	×	2.75	=	153.50
			145.85		2.72		396.40

Hor. Centr of gravity from heel = 2.72

Vertical Centr of gravity from bottom of base.

	vol.	arm from bottom		moment
1	3.00	× 9.25	=	27.75
2	.72	× 8.41	=	6.05
3	1.55	× 7.20	=	11.15
4	28.40	× 4.95	=	140.50
5	30.00	× 3.68	=	110.40
6	1.63	× 6.90	=	11.25
7	24.70	× 4.23	=	104.50
8	55.85	× 0.69	=	38.50
	145.85	3.09		450.10

Weight of concrete 2200 kg/m³ assumed

Total load and moments

load 145.85 @ 2200 = 321000 kg

Hor. moment 321000 × 2.72 = 872000 kg-m

Vertical moment " × 3.09 = 992000

Horizontal thrust during earthquake

321000 × 0.3 = 96300 kg

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO

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CALCULATIONS FOR

橋柱木裔

Earth filling at rear ; Front earth fill neglected on safe side of design

				m³	Arm	moment
1	At crown	2.50 × .27 × 6.00	=	4.05	× 1.25	= 5.06
2	middle filling front batter	$\frac{1.10}{2}$ × 6.83 × 8.00	=	30.100	× 1.77	= 53.10
3	do rear.		=	1.55	× 2.17	= 3.36
4	middle filling rear	1.40 × 7.10 × 8.00	=	79.60	× 0.70	= 55.70
5	at base	$\frac{.60}{2}$ × 1.20 × 8.00	=	2.88	× 0.40	= 1.15
				114.98	1.013	111.65

Hor. centre of gravity $\frac{111.65}{114.98}$ = 1.013 from heel of base.

Vertical centre of gravity of Earth fill

	m³		Arm		Moment
1	4.05	×	8.37	=	33.90
2	30.100	×	5.95	=	198.50
3	1.55	×	7.20	=	11.15
4	79.60	×	4.95	=	394.00
5	2.88	×	1.20	=	3.46
	114.98	×	5.20	=	598.71

Weight of Earth fill say 1700 kg/m³ assumed, gravel and broken stone being mixed for rear filling

weight 114.98 × 1700 = 195500 kg

Hor. moment 195500 × 1.013 = 198.000 kgm

Vert. " " × 5.20 = 101500 kgm

Centre of gravity of resultant load of concrete body and earth fill.

		weight kg	Arm m	moment kgm
Horizontal	Concrete body	324000	2.72	873000
	Earth fill	195500	1.013	198000
		516500	2.07	1070000
Vertical				
	Concrete body	321000	3.09	992000
	Earth fill	195500		1015000
		516500	3.88	2007000

Superimposed Dead Load from girder & parapet = 14000 kg

" Live load " " " = 28.000 "

Horizontal dead load thrust = 24000 × 0.3 = 7200 kg

Earth Pressure

Normal Condition Angle of repose assumed 35° Coefficient for earth pressure taken as 0.27

Surcharge for live load assumed 0.5 meter.

Earth Pressure at top 0.27 × 1700 × 0.50 = 230 kg/m²

" bottom 0.27 × 1700 × 9.00 = 4130 "

3900 "

Total pressure on wall and moment about bottom of base.

Pressure		Arm		Moment kgm
230 × 8.50 × 8.0	= 15630	× 4.25	=	66400
3900 × $\frac{8.50}{2}$ × 8.0	= 132500	× 2.83	=	375000
	148130	2.98		441400

During Earthquake Angle of repose assumed 35°. Coefficient for this comp. 0.45 for K = 0.3 Vertical component taken 60% of horizontal thrust.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO

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CALCULATIONS FOR

検討橋

Horizontal thrust during earthquake depth 8.5 meters

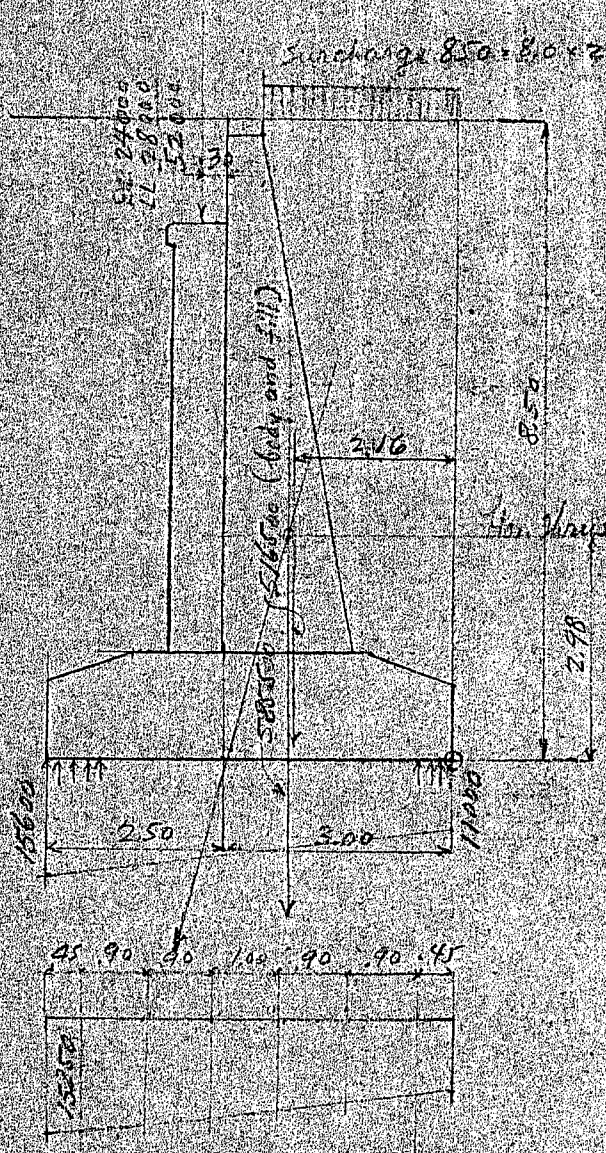
0.45 × 1700 × 8.5 = 6500 kg/m²

Total thrust $\frac{650 + 0}{2} \times 8.5 = 27600$ kg per m.

for 8.0 m width 27600 × 8 = 220800 kg point of application $\frac{8.5}{3} = 2.83$

Vertical thrust 220800 × 0.69 = 152300

Stability of abutment in normal state
loads as shown sketch below



Moment about O

Body & fill	576500 × 2.07	=	1070000
Superimposed load	52000 × 3.30	=	171600
Surcharge	17000 × 1.25	=	21200
	585500 2.16		1262800

All due to Hor. thrust 198130 × 2.98 = 441400

1704200

Hor. thrust 148130 kg

Point of application of total resultant at bottom of base

1704200 : 585500 = 2.91 m

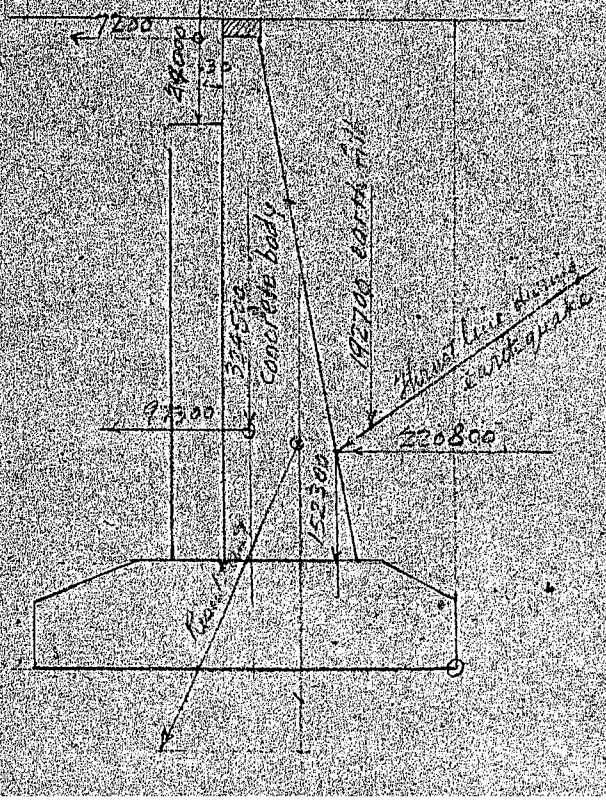
Eccentricity 2.91 − 2.75 = 0.16 m

Pressure at bottom of base

$\frac{585500}{5.5 \times 8.0} \left(1 \pm \frac{6 \times 0.16}{5.5}\right) = 15600 \times 11000$ kg/m²
(13780) 1.175 .825 1.40 or .82 tons/o²

max load on pile (approximate) if used
15250 × .90 × .90 = 12350 kg per pile

Stability of abutment during Earthquake.
Forward stability



Moment about O due to vertical loads

Superimposed load	24000 × 3.30	=	79200
Concrete body	221000 × 2.72	=	872000
Earth fill	195500 × 1.013	=	198000
Ver. comp. earth pressure	152300 × 1.55 (about)	=	236000
	692800 2.00		1385200

Moment about O due to Hor. load

Superimposed load	7200 × say 8.20	=	59000
Concrete body	96300 × 3.09	=	298000
Hor. comp. earth pressure	220800 × 2.83	=	625000
	324300 3.03		982000

Total moment 1385200 + 982000 = 2307200

Point of application
2307200 ÷ 692800 = 3.43 m

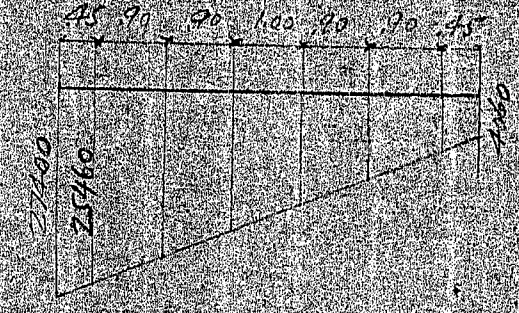
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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO

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CALCULATIONS FOR

柳橋木橋



Eccentricity 3.43 − 2.75 = 0.68

Pressure at bottom of base

$\frac{692800}{5.5\times8.0}\left(1+\frac{6\times0.68}{5.5}\right)$ = 27400 or 4060 kg/m²

(157.0) 1942 2.58 2.55 tm/m²

Max load on pile (approximate) if used

25460 × 0.81 = 20600 kg for pile.

Backward Stability

Moment taken at heel of base

Moments about heel due to vertical loads

Superimposed dead load	24000	× 3.3	= 79200
Concrete body	321000	× 2.72	= 872000
Earth fill	195500	× 1.013	= 198000
	540500	2.13	1149200

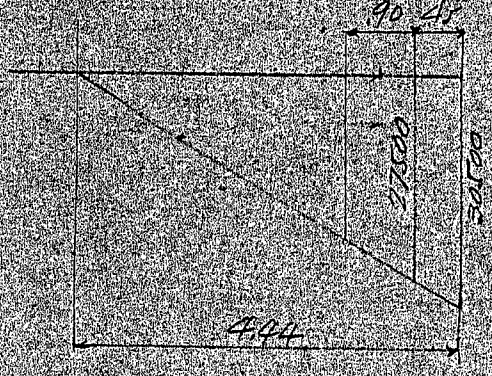
Moments about heel due to horizontal thrusts

Superimposed dead load	7200	× 8.20 about	= 59000
Concrete body	96300	× 3.09	= 298000
	103500	3.45	357000

Total moment 1149200 − 357000 = 792200

Point of application at bottom 792200 ÷ 540500 = 1.48 m

Resultant fall outside of middle third pressure area 3.48 − 1.48 = 4.44



Max soil pressure = $\frac{540500\times2}{444\times8.0}$ = 30500 kg/m²

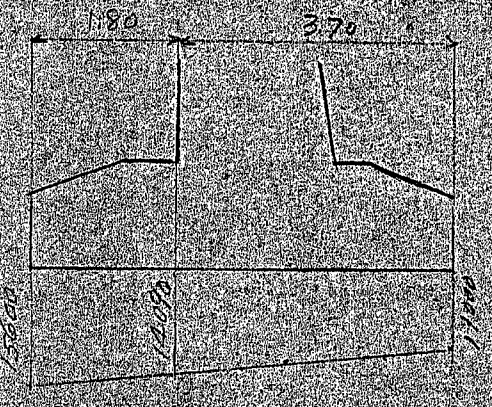
This assumption neglecting all horizontal thrust during earth quake prefer. Hence safe could be proofed to take such earth thrust against abutment body (perhaps ½ of thrust. Hence this assumption or so are safe side for design of abutment.

Max load on pile = 27500 × 0.81 = 22200 kg if used

Design of base of abutment

Case 1

Normal condition



Upward moment due to soil pressure

			Arm	moment
14070	× 1.8	= 25400	90	22860
(2.10)	× 1.2/2	= 1360	120	1630
		26760		24490

Downward moment

weight of concrete and soil on the footing assumed

Concrete footing	1.4 × 2200	=	3080
Earth fill	0.6 × 1700	=	1020
			4100 kg

Moment 4100 × 1.8/2 = say 6600 kgm

Summary moment 24490 − 6600 = 17890 kgm

Required fill area = $\frac{1789000}{7/8\times130\times1200}$ = 13.1 cm²/cm

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

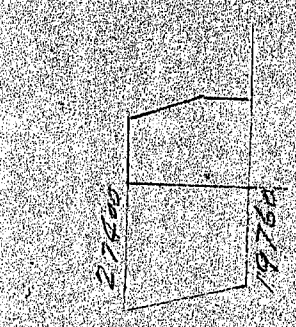
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CALCULATIONS FOR

檜村橋

Levee earthquake

Upward pressure 27400 and 19760 as shown



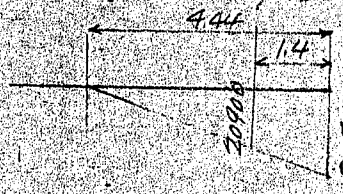
Upward moment

		arm	Moment
19760 × 1.8	= 35600	0.9	= 32000
7640 × .9	= 6880	⅓	= 8250
			40250 kgm
Downward moment			− 6600
			33650 kgm

Mint Stress of steel bar 1200 × 1.6 = 1920 kg/cm²

Steel area required = $\frac{3365000}{\frac{7}{8} \times 130 \times 1920}$ = 15.4 cm²/m

Use 25 mm bars @ 4.909 30 cm c to c = 16.35 cm²/m



Steel:—

Downward Load.

Footing 2200 × 1.4 = 3080

Fill 1700 × 7.6 = 12920

16000 kg/m²

Downward Moment = 16000 × $\frac{1.4^2}{2}$ = 15670 Kgm

Steel area required = $\frac{1567000}{\frac{7}{8} \times 130 \times 1200}$ = 11.5 cm²/m

Upward moment

20900 × $\frac{1.4^2}{2}$ = 20500

9600 × $\frac{1.4^2}{3}$ = 6280

26780

say − 15670

11110

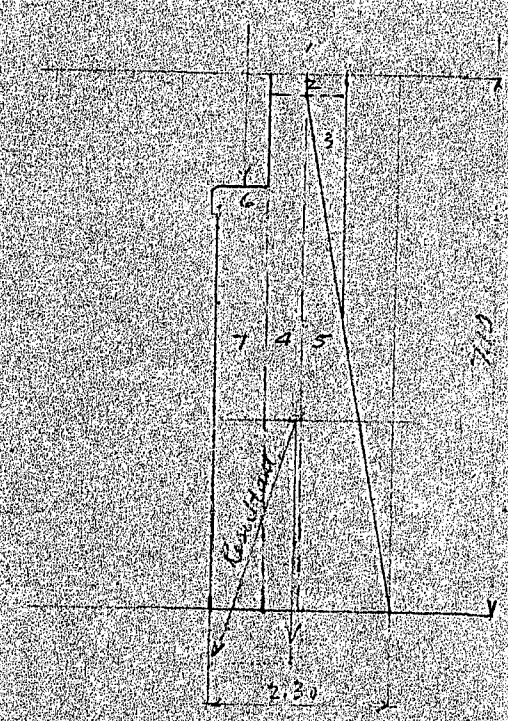
Total area = $\frac{1111000}{\frac{7}{8} \times 130 \times 1920}$ = 5.1 cm²/m

Use 22 mm bars @ 3.8 cm² 30 cm c to c. 12.65 cm²/m at top

25 mm bars @ 4.909 cm² " " 16.35 cm²/m at bottom

Moment about at top of base.

Normal condition.



Concrete Load

	vol m³	arm m	Moment
1	3.00	1.10	3.30
2	.72	1.10	.79
3	3.10	0.77	2.38 ← approximate only neglected error.
4	28.40	1.35	38.30
5	30.00	0.73	21.90
6	1.63	2.00	3.26
7	24.70	1.95	48.20
	91.55	1.29	118.13

Weight 91.55 × 2200 = 201,000 kg

Rear fill. (approximate)

vol = $\frac{1.10}{2}$ × 7.10 × 8.0 = 31.2 m³

weight = 31.2 × 1700 = 53000 kg

Center of gravity of Concrete and fill

Concrete	201.000	× 1.29	=	259.000
Earth	53.000	× 0.37	=	19600
	254000	1.095		278600

Load from girder span Total 52000 kg

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JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR
大潤村木橋

Surcharge say $0.5 \times 1700 \times 1.1 \times 8.0 = 7500$ kg

Centre of gravity

Concrete and Earth fill	254000	× 1.095 =	278600
Superimposed load	52000	× 1.90 =	98800
Surcharge say	7500	× 0.55 =	4100
	313500	1.22	381500

Horizontal earth pressure Surcharge 0.5 metre

Earth pressure at top $0.27 \times 1700 \times 0.50 = 230$
" " " bottom $0.27 \times 1700 \times 7.60 = 3490$
3260 kg/m²

Total pressure on wall and point of application.

	Total pressure	arm	Moment
$230 \times 7.10 \times 8.0 =$	13050	3.55	46300
$3260 \times \frac{7.10}{2} \times 8.0 =$	92600	2.37	219500
	105650	2.51	265800

Moment about Top of base (neutral axis) due to vertical and horizontal forces.

Eccentricity $1.22 - 1.15 = 0.07$

Abutment and Earth $313500 \times 0.07 = 21900$
Earth pressure 265800
287700 kgm

Point of resultant = .92

Vertical load 1 metre strip 39100 kg
Moment 1 metre strip 36000 kgm

Fibre Stresses $\frac{39100}{100 \times 230} \pm \frac{6 \times 36000.00}{100 \times 230^2} = 5.78$ kg/cm² Compression
1.7 or 2.38 " Tension.

Use reinforcing bars in the joint

Rectangular Section in concrete $f_c = \frac{360000}{0.0822 \times 100 \times 225^2} = 8.65$ kg/cm²

16 mm dia 30 cm c to c assumed Net area $2.015 \times 3.33 = 6.70$ cm² on one side
Both sides Steel % = 0.058 % $K = 0.41$

$f_s = 15 \times 8.65 \times \left(\frac{225}{.41 \times 230} - 1\right) = 180$ kg/cm²

16 mm dia bars 30 cm c to c shall be enough

Steel in parapet wall.

Depth 1.45 metre Surcharge say 0.5 m

Horizontal Earth pressure $0.5 \times 1700 \times 0.27 = 230$
$1.95 \times 1700 \times 0.27 = 895$
665 kg/m²

Horizontal moment $230 \times \frac{1.45^2}{2} = 242$
$665 \times \frac{1.45^2}{3} = 466$
708 kgm

Neglecting direct stress, the required steel area for moment only is

Steel area = $\frac{70800}{\frac{7}{8} \times 60 \times 1200} = 1.12$ cm² per metre

Use 12 mm dia bars 30 cm c to c. for parapet wall.

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CONSULTING ENGINEER
GOTANDA, TOKYO.

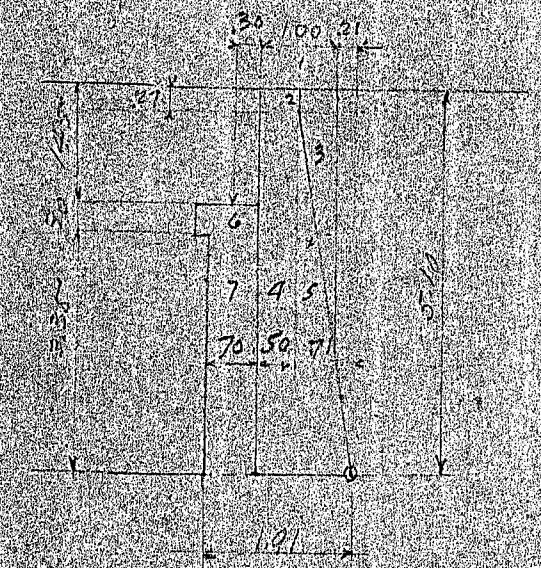
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CALCULATIONS FOR

橋台橋

Section of abutment body 2.0 m above top of base

width of section 19 l m



Concrete body

	Vol				
1	3.00	×	.71	=	2.13
½	.72	×	.71	=	.51
3	1.55	×	.38	=	.59
4	19.30	×	.96	=	18.54
5	13.70	×	.48	=	6.57
6	2.63	×	1.61	=	2.62
7	15.50	×	1.56	=	24.20
	55.40		.99		55.16

weight 55.40 @ 2200 = 122000 kg

Rear fill approximately

.71 × 5.1 × 8.0 = 14.5 m³

weight 14.5 × 1700 = 24700 kg

Centre of gravity

	Concrete	122000	× .99	=	121000
	fill	24700	× 1.37	=	5850
		146700	.863		126850

Surcharge say 0.5 × 1700 × 0.7 × 8.0 = 4770 kg

Centre of gravity due to vertical loads

Concrete and rear fill	146700	× .863	=	126850
Equivalent point load	52000	× 1.51	=	78500
Surcharge say	4770	× .36	=	1720
	203470	1.018		207070

Secondary = .063

1. Horizontal Earth pressure surcharge 0.5 mtr. Angle of repose = 35°

Earth pressure at 4.5 m 0.27 × 1700 × 0.50 = 230

" " at bottom 0.27 × 1700 × 5.60 = 2570

2340 kg/m²

Total earth pressure on wall and point of application.

230	. 5.1 × 8.0	=	9400	. 2.55	=	23900
2340	. 5.1 × 8.0	=	47700	. 1.70	=	81100
			57100	1.84		105000

Point of application of total forces

	Vertical	Moment
	203470	207070
for		105000
	1.53	312070

Without effect neutral axis due to vertical and for loads.

Eccentricity in mtrs	203470	× 0.063	=	12800
Earth pressure				105000
	Eccentricity =	.579		117800

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JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

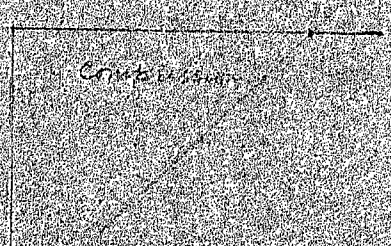
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CALCULATIONS FOR
不静定計算

Vertical load / metre strip = 25400 Kg
Horizontal " " " = 14700 Kg

Vertical thrust = $\frac{25400}{100 \times 191} = \frac{6 \times 14700.00}{100 \times 191}$

= 1.33 + 2.42 = 3.75 or - 1.09 kg/cm²



Reinforcing steel used mainly for tying base to the concrete above, the steel area 6.70 cm² per metre strip in one side. Steel ratio = 0.579 $\phi/4$ = 0.303 K = 0.63 about

$f_c = \frac{1470000}{0.0955 \times 100 \times 191^2} = 4.22$ kg/cm²

$f_s = 15 \times 4.22 \times \left(\frac{185}{0.63 \times 191} - 1\right) = 35.0$ kg/cm²

Need very slight reinforcement in this section.

Stability against Sliding.

Normal state: Total load — say 585500 Kg
Total horizontal thrust — say 148000 " — 25.3% of vertical load

During earthquake:
Total load — say 693000 Kg
Horz. thrust — say 324000 " — 46.8% of vertical load

Sliding friction between earth and concrete supposed about 50% of vertical load. In both cases of normal and earthquake to secure ample safety against sliding, use projection below base at both toe and heel using certain amount of reinforcing bars.

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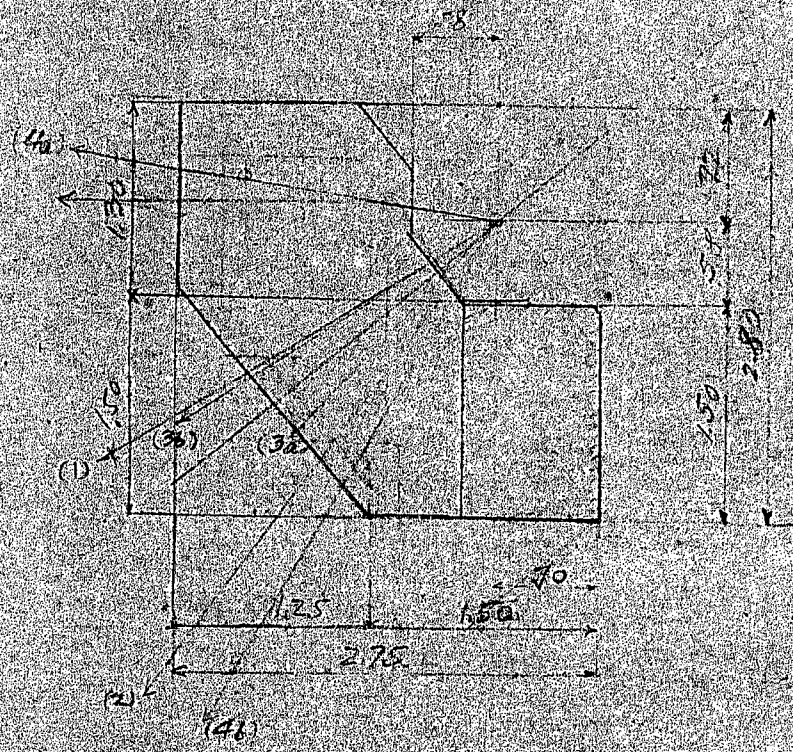
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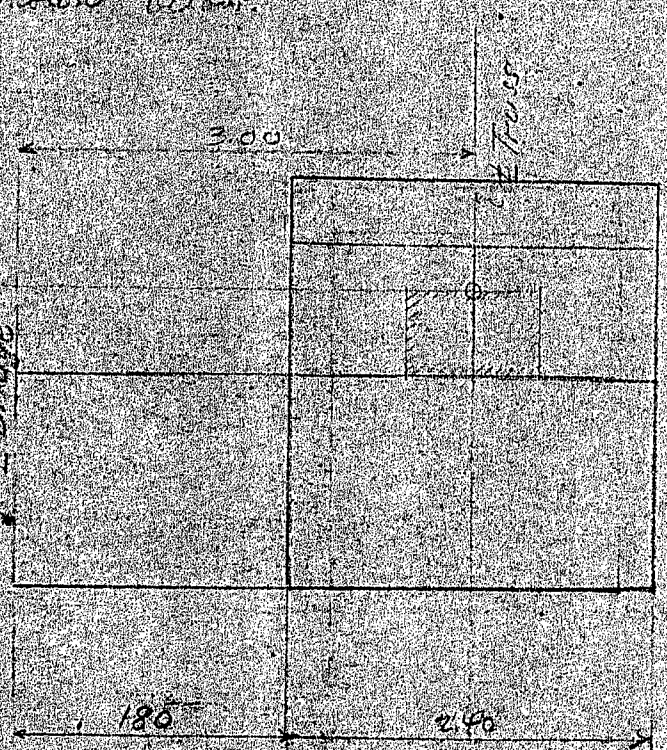
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CALCULATIONS FOR

神納村橋

Weight of Each Abutment.
East abutment (upstream) West abutment as above (Plan)





(1) Max load on shoe dead, live and temperature 158000 kg + live load on garden area
Corresponding thrust 208,000 "

(2) Max. hor. thrust 124200 kg
Corresponding vertical load 119200 kg + dead load temperature of each span + live dead loads of guide bridge

(3) Longitudinal quake K = 0.3
Longitudinal quake thrust 69720 to 103.170 kg
Corresponding vertical load 111.840 kg

(4) Transverse quake Transverse thrust 33300 kg each shoe
Longitudinal thrust 136470 "
Corresponding vert. load 242340 kg ∞ 220,260 kg

Volume of Concrete (Approximate only)

1.50 × 1.50 × 2.40	= 5.40	× 0.75	= 4.05
1.25 × 1.50 × 2.40	= 2.25	× 1.92	= 4.31
1.50 × 1.72 × 2.40	= 4.58	× 2.01	= 9.21
	12.23	4.68	17.57

12.23 @ 2200 = 26900 kg

Case 1a Resultant $\sqrt{136470^2 + 202680^2}$ = 137800 final resultant by graphic 136000
Resultant cut 55 mm. from edge pressure area = 3 × .62 × 2.24 = 446

Max. pressure = $\frac{136000 \times 2}{446}$ = 610 kg/cm² or 568 ton/□'

Case 1. Resultant Thrust by graphic weight of abutment and earth 300,000 kg
Projected bottom area 3.85 m² 208 = .30

Pressure at bottom = $\frac{300,000}{3.85 \times 2.24}\left(1 \pm \frac{61.82}{3.85}\right)$ = 4700 or 17300 kg/m²
(32500) (1468 4.932) 444 ton/□'

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CONSULTING ENGINEER
GOTANDA, TOKYO.

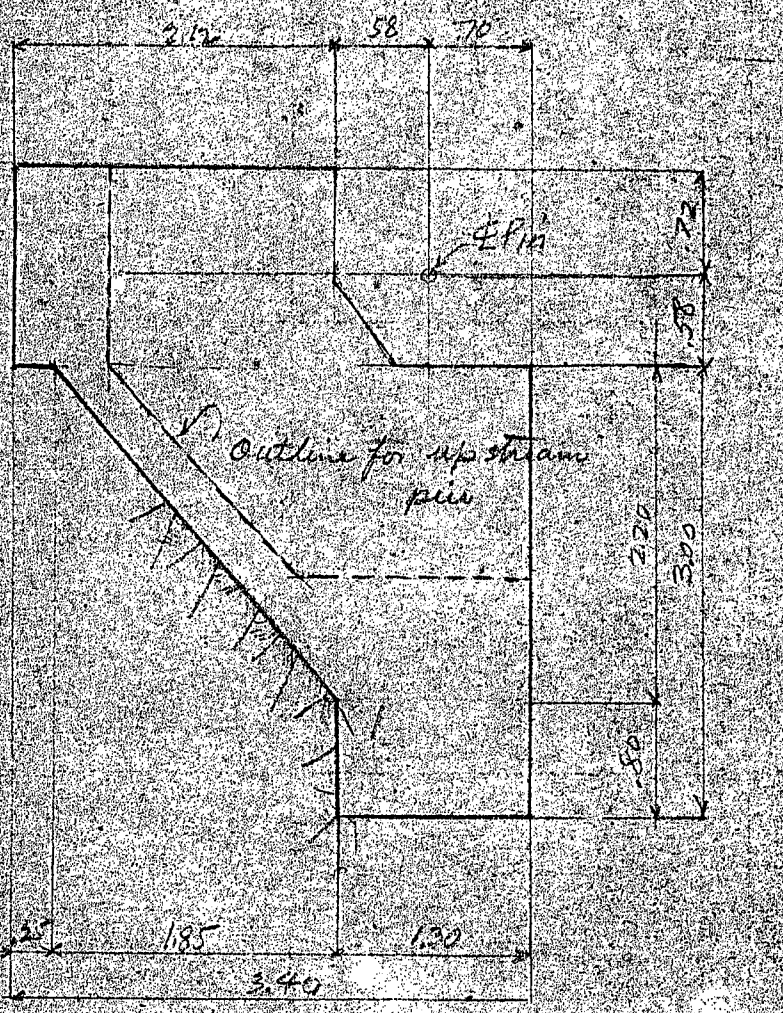
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CALCULATIONS FOR

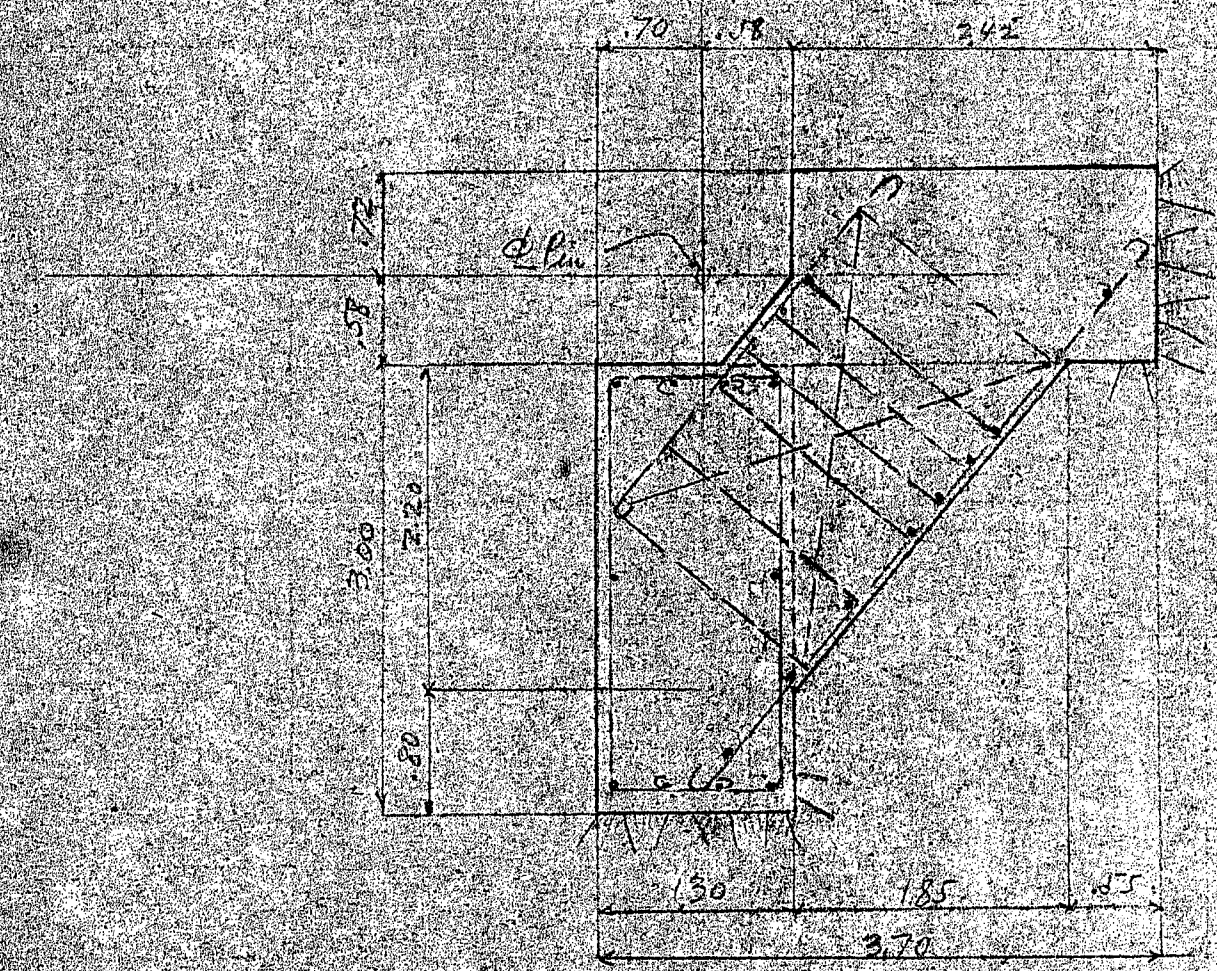
橋村橋

East abutment

(Down Stream) Sketch shown below. All dimensions in abutment unit mm.



West abutment (left and downstream).



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JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

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檜村橋材料調書

昭和十二年十二月

東京市第二水道擴張計畫

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR

檜村橋材料表

Structural steel for arch span

			Top chord	U_0-U_2	1-Required				
Main section	1	Cov. Pl	310	× 9	c 21902	kg/s	× 9.670 m	2118	
" "	2	I-B	380	× 100	c 545		× 9670	10540	
Splice at 2	1	Pl	310	× 19	c 46237		× 1140		527
"	1	"	310	× 9	c 21902		× 860		188
"	2	Pls	370	× 12	c 34854		× 720		502
Tie plates	3	"	310	× 9	c 21902		× 400		263
Lacing bars	61	"	65	× 9	c 4592		× 298		835
washers	3	Wash.	65φ		c 260		× 009		7
Lateral connection	1	Pl	300	× 9	c 21195		× 615		130
"	1	L	125×75×9		c 135		× 615		83
"	1	Pl	300	× 9	c 21195		× 930		197
"	1	L	125×75×9		c 135		× 930		126
"	1	Fill	105	× 10	c 8243		× 290		24
"	1	"	105	× 9	c 7418		× 205		15
Gusset at 1	2	Pls	790	× 9	c 55814		× 955		1066

12658 + 3963 = 16621
4 c 16621 = 66484

			U_2-U_3		4-Required				
Main section	1	Cov. Pl	310	× 9	c 21902		× 7330	1605	
"	2	I-B	380	× 100	c 545		× 7330	7990	
Splice at 3	1	Pl	310	× 19	c 46237		× 1140		527
"	1	Pl	310	× 9	c 21902		× 860		188
"	2	Pls	370	× 12	c 34854		× 720		502
Tie plates	2	"	310	× 9	c 21902		× 380		166
Lacing bars	35	"	65	× 9	c 4592		× 298		479
washers	2	Wash.	65φ		c 260		× 009		5
Lateral connection	1	Pl	300	× 9	c 21195		× 885		188
"	1	L	125×75×9		c 135		× 885		119
"	1	Fill	105	× 9	c 7418		× 205		15
"	1	Pl	255	× 9	c 18016		× 850		153
"	1	L	125×75×9		c 135		× 850		115
"	1	Fill	105	× 9	c 7418		× 105		12
Gusset at 2	2	Pls	875	× 9	c 61819		× 885		1094
" 3	2	"	885	× 9	c 62525		× 1015		1269

9595 + 4832 = 14427
4 c 14427 = 57708

			U_3-U_5		4-Required				
Main section	1	Cov. Pl	310	× 9	c 21902		× 8290	1816	
"	2	I-B	380	× 100	c 545		× 8290	9036	
Splice at 5	1	Pl	310	× 19	c 46237		× 1140		527
"	1	"	310	× 9	c 21902		× 860		188
"	2	Pls	370	× 12	c 34854		× 720		502
Tie plates	2	"	310	× 9	c 21902		× 380		166
Lacing bars	55	"	65	× 9	c 4592		× 298		753
Washers	3	Wash.	65φ		c 260		× 009		7
Lateral connection	1	L	125×75×9		c 135		× 850		115
"	1	Pl	255	× 9	c 18016		× 850		153
"	1	Fill	105	× 9	c 7418		× 105		12
Gusset	2	Pls	815	× 9	c 57580		× 1185		1365

10352 + 3788 = 14640
4 c 14640 = 58560

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR

檜村橋材料表

			U5 – U6			4-Required			
Main section	1	Cov. Pl	310	× 9	c	21.902	× 7.650	167.6	
"	2	L	380	× 100	c	545	× 7.650	8339	
"	2	Pls	300	× 9	c	21.195	× 7005	2969	
Splice at 6	1	Pl	310	× 19	c	46237	× 1.170		54.1
"	1	"	310	× 9	c	21902	× .890		195
"	2	Pls	370	× 12	c	34854	× .730		50.9
"	2	"	300	× 9	c	21195	× .440		18.7
Tie plates	3	"	310	× 9	c	21902	× .380		250
Lacing bars	35	"	65	× 9	c	4592	× .298		47.9
Washers	2	Wash.	65φ		c	260	× .009		.5
Lateral connection	2	L	125	× 75 × 9	c	135	× .850		23.0
"	2	Pls	255	× 9	c	18016	× .850		30.6
"	2	Fills	105	× 9	c	7418	× .105		2.4
Gusset	2	Pls	815	× 9	c	57580	× 1.400		161.2
"	2	"	815	× 9	c	"	× 1.350		155.5

12984 + 5893 = 18877
4 c 18877 = 75508

			U6 – U6			2-Required			
Main section	1	Cov. Pl	310	× 9	c	21902	× 7350	1610	
"	2	L	380	× 100	c	545	× 7350	8012	
"	2	Pls	300	× 9	c	21195	× 7350	3116	
Tie plates	2	"	310	× 9	c	21902	× .380		16.6
Lacing bars	46	"	65	× 9	c	4592	× .298		62.9
Washers	4	Wash.	65φ		c	260	× .009		.9
Lateral connection	1	L	125	× 75 × 9	c	135	× .850		11.5
"	1	Pl	255	× 9	c	18016	× .850		15.3
"	2	Fills	105	× 9	c	7418	× .105		2.4
Gusset	2	Pls	510	× 9	c	36032	× .815		58.7

12738 + 1683 = 14421
2 c 14421 = 28842

		Diaphragm DM9				20-Required			
	4	L	100 × 75 × 10		c	130	× .300		15.6
	1	Pl	290	× 9	c	20489	× .300		6.1
									21.7

20 c 21.7 = 4340

		Diaphragm DM10				10-Required			
	4	L	100 × 75 × 10		c	130	× .300		15.6
	1	Pl	270	× 9	c	14076	× .300		5.7
									21.3

10 c 21.3 = 2130

Summary of structural steel for top chord 29357.2 kgs

		Bottom chord		L0 – L1		4-Required			
Main section	2	L	130 × 130 × 12		c	234	× 6040	2827	
"	2	"	130 × 130 × 12		c	"	× 6025	2820	
"	2	Web Pls	440	× 9	c	31086	× 6040	3755	
"	2	Pls	180	× 12	c	16956	× 6035	2047	
Gusset	2	"	825	× 12	c	77715	× .930		144.5
"	2	"	1,100	× 12	c	10362	× 1.125		233.1
Pin plate	2	"	410	× 12	c	38622	× .655		50.6
"	2	"	410	× 12	c	"	× .510		39.4

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR

檜村橋材料表

Tie plates	2	P/s	500 · 9 ·	c	35325	×	.605	427	
Gusset	1	P/	760 · 9	c	53694	×	.860	46.2	
Lacing bars	14	P/s	70 · 12	c	6594	×	.755	69.7	
"	20	"	70 · 12	c	"	×	.737	97.2	
Washers	4	Wash.	70ϕ	c	30.2	×	.012	.14	
								11449+7248 = 18697	
								4 c 18697 = 74788	

L1 - L2 4-Required

Main section	2	L's	130·130·12	c	234	×	5830	2728	
"	2	"	130·130·12	c	"	×	5800	2714	
"	2	Web P/s	440 · 9	c	31086	×	5830	3625	
"	2	P/s	180 · 9	c	12717	×	5820	1480	
Gusset	2	"	1,170 · 9	c	82661	×	1.185	195.9	
"	1	P/	865 · 9	c	61112	×	1150	70.3	
Splice	2	P/s	435 · 12	c	40977	×	1015	83.2	
"	2	Fill's	180 · 3	c	4239	×	.540	4.6	
"	2	P/s	125 · 12	c	11775	×	.800	18.8	
Tie plates	2	"	500 · 9	c	35325	×	.605	42.7	
Lacing bars	14	"	70 · 12	c	6594	×	.769	71.0	
"	20	"	70 · 12	c	"	×	.737	97.2	
washers	4	Wash.	70ϕ	c	30.2	×	.012	.14	
								10547 + 5851 = 16398	
								4 c 16398 = 65592	

L2 - L3 4-Required

Main section	2	L's	130·130·12	c	234	×	5640	2640	
"	2	"	130·130·12	c	"	×	5610	2625	
"	2	Web P/s	440 · 9	c	31086	×	5640	3507	
"	2	P/s	180 · 9	c	12717	×	5625	143.1	
Gusset	2	"	1,100 · 9	c	77.72	×	1160	180.3	
"	1	P/	860 · 9	c	60759	×	1100	66.8	
Splice	2	P/s	435 · 12	c	40977	×	1005	82.4	
"	2	Fill's	205 · 3	c	4828	×	.985	9.5	
"	2	P/s	125 · 12	c	11775	×	.800	18.8	
Tie plates	2	"	500 · 9	c	35325	×	.605	42.7	
Lacing bars	14	"	70 · 12	c	6594	×	.755	69.7	
"	18	"	70 · 12	c	"	×	.769	91.3	
washers	4	Wash.	70ϕ	c	30.2	×	.012	.14	
								10203 + 5629 = 15832	
								4 c 15832 = 63328	

L3 - L4 4-Required

Main section	2	L's	130·130·12	c	234	×	5475	2562	
"	2	"	130·130·12	c	"	×	5445	254.8	
"	2	Web P/s	440 · 9	c	31086	×	5475	3404	
Gusset	2	P/s	1,030 · 9	c	72770	×	1145	166.6	
"	1	P/	815 · 9	c	57580	×	1020	58.7	
Splice	2	P/s	430 · 12	c	40506	×	.995	80.6	
"	2	Fill's	180 · 9	c	12717	×	.515	13.1	
"	2	P/s	125 · 12	c	11775	×	.800	18.8	
Tie plate	1	P/	570 · 9	c	40271	×	.605	24.4	
"	1	"	500 · 9	c	35325	×	.605	21.4	
Lacing bars	12	P/s	70 · 12	c	6594	×	.769	60.8	
"	18	"	70 · 12	c	"	×	.755	89.6	
washers	4	Wash.	70ϕ	c	30.2	×	.012	.14	
								8514 + 5354 = 13868	
								4 c 13868 = 55472	

増田淳氏関係資料
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CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR

檜村橋材料表

			L4 - L5		4 - Required				
Main section	2	L3	130·130·9	c	17.9	×	5355	1917	
〃	2	〃	130·130·9	c	〃	×	5325	1906	
〃	2	Web Pl3	440 · 9	c	31.086	×	5355	3329	
Gusset	2	Pl3	1000 · 9	c	70650	×	1230		1738
〃	1	Pl	810 · 9	c	57227	×	990		567
Splice	2	Pl3	435 · 9	c	30733	×	975		599
〃	2	Fill3	205 · 12	c	14483	×	965		280
〃	2	〃	110 · 3	c	2591	×	500		26
〃	2	〃	110 · 3	c	〃	×	485		25
〃	2	Pl3	125 · 9	c	8831	×	560		99
Tie plates	2	〃	500 · 9	c	35325	×	605		427
Lacing bars	12	〃	70 · 12	c	6594	×	755		597
〃	18	〃	70 · 12	c	〃	×	744		883
Washers	4	Wash.	70ϕ	c	302	×	012		14

7152 + 5255 = 12407
4 c 12407 = 49628

			L5 - L6		4 - Required				
Main section	2	L3	130·130·9	c	17.9	×	5270	1887	
〃	2	〃	130·130·9	c	〃	×	5240	1876	
〃	2	Web Pl3	440 · 9	c	31.086	×	5270	3276	
Gusset	2	Pl3	945 · 9	c	66764	×	1225		1636
〃	1	Pl	810 · 9	c	57227	×	980		561
Splice	2	Pl3	425 · 9	c	30026	×	685		411
〃	2	Fill3	195 · 9	c	13777	×	675		185
〃	2	Pl3	125 · 9	c	8831	×	560		99
Tie plates	2	〃	500 · 9	c	35325	×	605		427
Lacing bars	12	〃	70 · 12	c	6594	×	737		583
〃	18	〃	70 · 12	c	〃	×	744		883
washers	4	Wash.	70ϕ	c	302	×	012		14

7039 + 4800 = 11839
4 c 11839 = 47356

			L6 - L7 - L6'		2 - Required				
Main section U6-U7	2	L3	130·130·9	c	17.9	×	5230	1872	
〃	2	L3	130·130·9	c	〃	×	5200	1862	
〃	2	Web Pl3	440 · 9	c	31086	×	5230	3252	
〃 U7-U6'	2	L3	130·130·9	c	17.9	×	5280	1890	
〃	2	〃	130·130·9	c	〃	×	5250	1880	
〃	2	Web Pl3	440 · 9	c	31086	×	5280	3283	
Gusset	4	Pl3	890 · 9	c	62879	×	1420		3572
〃	2	〃	880 · 9	c	62172	×	2120		2636
〃	3	〃	815 · 9	c	57580	×	980		1693
Splice	4	〃	425 · 9	c	30026	×	675		811
〃	4	Fill3	195 · 9	c	13777	×	665		366
〃	6	Pl3	125 · 9	c	8831	×	560		297
〃	2	〃	430 · 9	c	30380	×	680		413
〃	2	Fill3	195 · 9	c	13777	×	680		187
Tie plates	4	Pl3	430 · 9	c	30380	×	605		735
Lacing bars	60	〃	70 · 12	c	6594	×	737		2916
washers	8	Wash.	70ϕ	c	302	×	012		29

14039 + 13655 = 27694
2 c 27694 = 55388

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CONSULTING ENGINEER
GOTANDA, TOKYO.

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CALCULATIONS FOR

檜村橋材料表

Diaphragm DM1 4-Required

4	L	100·75·10	@	130	·	440	229
2	"	100·75·10	@	"	·	130	34
1	Pl	280 · 9	@	19782	·	440	87
							350

4 @ 350 = 1400

Diaphragm DM2 4-Required

2	L	125·75·9	@	135	·	440	119
2	"	100·75·10	@	130	·	440	114
2	"	100·75·10	@	"	·	160	42
1	Pl	310 · 9	@	21902	·	440	96
							371

4 @ 371 = 1484

Diaphragm DM3 4-Required

2	L	125·75·9	@	135	·	430	116
2	"	100·75·10	@	130	·	430	112
2	"	100·75·10	@	"	·	160	42
1	Pl	310 · 9	@	21902	·	430	94
							364

4 @ 364 = 1456

Diaphragm DM4 4-Required

4	L	100·75·10	@	130	·	420	218
2	"	100·75·10	@	"	·	160	42
1	Pl	310 · 9	@	21902	·	420	92
							352

4 @ 352 = 1408

Diaphragm DM5 4-Required

4	L	100·75·10	@	130	·	415	216
2	"	100·75·10	@	"	·	160	42
1	Pl	310 · 9	@	21902	·	415	91
							349

4 @ 349 = 1396

Diaphragm DM6 4-Required

4	L	100·75·10	@	130	·	400	208
2	"	100·75·10	@	"	·	160	42
1	Pl	310 · 9	@	21902	·	400	88
							338

4 @ 338 = 1352

Diaphragm DM7 4-Required

4	L	100·75·10	@	130	·	395	205
2	"	100·75·10	@	"	·	160	42
1	Pl	310 · 9	@	21902	·	395	87
							334

4 @ 334 = 1336

Diaphragm DM8 2-Required

4	L	100·75·10	@	130	·	390	203
2	"	100·75·10	@	"	·	160	42
1	Pl	310 · 9	@	21902	·	390	85
							330

2 @ 330 = 660

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GOTANDA, TOKYO.

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CALCULATIONS FOR

檜村橋材料表

Summary of structural stil for Bottom chord 422044 kgs

		Verticals	$U_0 - L_0$		4 - Required				
Main section	2	⊏	380 × 100	c	545	×	7533	821.1	
〃	1	L	150·100 × 9	c	17.1	×	7019	120.0	
〃	1	Fill	145 × 9	c	10244	×	6436	65.9	
〃	2	⊏	380 × 100	c	545	×	6281	684.6	
〃	1	L	150·100 × 9	c	17.1	×	5896	100.8	
〃	1	Fill	145 × 9	c	10244	×	4777	48.9	
Splice	2	Pls	530 × 9	c	37445	×	860		64.4
〃	2	〃	370 × 9	c	26141	×	720		37.6
〃	1	Pl	145 × 9	c	10244	×	1240		12.7
〃	2	Pls	290 × 9	c	20489	×	305		12.5
Gusset	2	〃	745 × 9	c	52634	×	1140		120.0
Tie plate	1	Pl	530 × 9	c	37445	×	570		21.3
〃	1	〃	530 × 9	c	〃	×	830		31.1
〃	1	〃	530 × 9	c	〃	×	780		29.2
〃	1	〃	540 × 9	c	38151	×	1510		57.6
Lacing bars	84	Pls	70 × 12	c	6594	×	747		413.8
washers	8	Wash.	70ϕ	c	302	×	012		2.9
Diaphragm	2	L	100·75·10	c	130	×	570		14.8
〃	1	Pl	280 × 9	c	19782	×	570		11.3
〃	2	Fills	95 × 12	c	8949	×	210		3.8
〃	2	L	100·75·10	c	130	×	910		23.7
〃	1	Pl	330 × 9	c	23315	×	910		21.2
〃	2	L	100·75·10	c	130	×	1165		30.3
〃	1	Pl	310 × 9	c	21902	×	1165		25.5
〃	2	Fills	95 × 9	c	6712	×	415		5.6

1,841.3 + 939.3 = 2,780.6
4 c 2,780.6 = 11,122.4

			$U_1 - L_1$		4 - Required				
Main section	2	⊏	300 × 90	c	38.1	×	10280	783.3	
Tie plate	4	Pls	310 × 9	c	21902	×	420		36.8
Lacing bars	88	〃	65 × 9	c	4592	×	318		128.5
Filler	1	Pl	290 × 9	c	20489	×	440		9.0
Diaphragm	4	L	125·75·9	c	135	×	420		22.7
〃	1	Pl	290 × 9	c	20489	×	420		8.6
〃	4	L	125·75·9	c	135	×	1,000		54.0
〃	1	Pl	290 × 9	c	20489	×	1,000		20.5

783.3 + 280.1 = 1,063.4
4 c 1,063.4 = 4,253.6

			$U_2 - L_2$		4 - Required				
Main section	4	L	150·90·9	c	16.4	×	7.735	507.4	
Tie plate	1	Pl	310 × 9	c	21902	×	445		9.7
〃	1	〃	310 × 9	c	〃	×	980		21.5
Lacing bars	29	Pls	65 × 9	c	4592	×	318		42.3
Filler	1	Pl	300 × 9	c	21195	×	580		12.3

507.4 + 85.8 = 593.2
4 c 593.2 = 2,372.8

			$U_3 - L_3$		4 - Required				
Main section	4	L	125·90·9	c	14.6	×	5.670	331.1	
Tie plate	1	Pl	310 × 9	c	21902	×	370		8.1
〃	1	〃	310 × 9	c	〃	×	980		21.5

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
OHOWA BLDG. TOKYO

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CALCULATIONS FOR

檜村橋材料表

Lacing bars	20	Pl⅜	65	9	c	4592	318	292	
Filler	1	Pl	250	9	c	17663	510	90	
								3311 + 678 = 3989	
								4 c 3989 = 15956	

			U4 - L4			A-Required			
Main section	4	L⅜	125	90	9	146	4005	2374	
Tie plate	1	Pl	310	9	c	21902	390	85	
"	1	"	310	9	c	"	980	215	
Lacing bars	12	Pl⅜	65	9	c	4592	318	175	
Filler	1	Pl	250	9	c	17663	580	102	
								2374 + 577 = 2951	
								4 c 2951 = 11804	

			U5 - L5			A-Required			
Main section	4	L⅜	125	90	9	146	2930	1711	
Tie plate	1	Pl	310	9	c	21902	370	81	
"	1	"	310	9	c	"	700	153	
Lacing bars	8	Pl⅜	65	9	c	4592	318	117	
Filler	1	Pl	250	9	c	17663	280	49	
								1711 + 400 = 2111	
								4 c 2111 = 8444	

			U6 - L6			A-Required			
Main section	4	L⅜	125	90	9	146	2255	1317	
Tie plate	1	Pl	310	9	c	21902	370	81	
"	1	"	310	9	c	"	700	153	
Lacing bars	5	Pl⅜	65	9	c	4592	318	73	
Filler	1	Pl	250	9	c	17663	280	49	
								1317 + 356 = 1673	
								4 c 1673 = 6692	

			U7 - L7			2-Required			
Main section	4	L⅜	125	90	9	146	2025	1183	
Tie plate	1	Pl	310	9	c	21902	370	81	
"	1	"	310	9	c	"	700	153	
Lacing bars	4	Pl⅜	65	9	c	4592	318	58	
Filler	1	Pl	250	9	c	17663	280	49	
								1183 + 341 = 1524	
								2 c 1524 = 3048	

Summary of structural steel for vertical members 223493 kgs

			Diagonal U0 - L1			4-Required			
Main section	4	L⅜	150	90	9	164	10900	7150	
Tie plate	2	Pl⅜	310	9	c	21902	390	171	
Lacing bars	47	"	65	9	c	4592	318	686	
								7150 + 857 = 8007	
								4 c 8007 = 32028	

			U1 - L2			4-Required			
Main section	4	L⅜	125	90	9	146	8970	5238	
Tie plate	2	Pl⅜	310	9	c	21902	390	171	
Lacing bars	37	"	65	9	c	4592	318	540	
								5238 + 711 = 5949	
								4 c 5949 = 23796	

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
[illegible] BLDG, TOKYO.

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CALCULATIONS FOR

檜村橋材料表

		U2 - L3			4 - Required				
Main section	2	L	125×90×9	c	146	×	7350	2140	
〃	2	〃	125×90×9	c	〃	×	7405	2162	
Tie plate	2	P/3	310 × 9	c	21902	×	390		171
Lacing bars	30	〃	65 × 9	c	4592	×	318		438
									4308 + 609 = 4917
									4 c 4917 = 19668
		U3 - L4			4 - Required				
Main section	2	L	125×90×9	c	146	×	6150	1796	
〃	2	〃	125×90×9	c	〃	×	6205	1812	
Tie plate	2	P/3	310 × 9	c	21902	×	390		171
Lacing bars	24	〃	65 × 9	c	4592	×	318		350
									3608 + 521 = 4129
									4 c 4129 = 16516
		U4 - L5			4 - Required				
Main section	2	L	125×90×9	c	146	×	5260	1536	
〃	2	〃	125×90×9	c	〃	×	5370	1568	
Tie plate	2	P/3	310 × 9	c	21902	×	390		171
Lacing bars	20	〃	65 × 9	c	4592	×	318		292
									3104 + 463 = 3567
									4 c 3567 = 14268
		U5 - L6			4 - Required				
Main section	2	L	125×90×9	c	146	×	4735	1383	
〃	2	〃	125×90×9	c	〃	×	4845	1415	
Tie plate	2	P/3	310 × 9	c	21902	×	390		171
Lacing bars	17	〃	65 × 9	c	4592	×	318		248
									2798 + 419 = 3217
									4 c 3217 = 12868
		U6 - L7, L7 - U6'			2 - Required				
Main section (U6-L7)	4	L	125×90×9	c	146	×	4570	2669	
〃 (L7-U6)	4	〃	125×90×9	c	〃	×	4620	2698	
Tie plate	4	P/3	310 × 9	c	21902	×	390		342
Lacing bars	32	〃	65 × 9	c	4592	×	318		467
									5367 + 809 = 6176
									2 c 6170 = 12352

Summary of structural steel for Diagonal members 134446 kgs

Summary of structural steel of main trusses 1070544 kgs

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CONSULTING ENGINEER
[illegible] BLDG. TOKYO

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CALCULATIONS FOR

檜村橋材料表

Stringers S1, S2, S1A & S2A 1- Required

39	I⁵	350 · 150	c	585	·	5190	11841.0
3	"	350 · 150	c	"	·	5240	919.6
90	Pl⁵	160 · 9	c	11304	·	280	284.9
420	Bolts	12ϕ · 200	c	0253			106.3
420	Wash	60 · 8	c	3768	·	060	95.0
							13246.8

Floor Beam FB1 2- Required

4	Flg L⁵	125 · 75 · 9	c	135	·	5638	304.5
1	Pl	650 · 9	c	45923	·	5630	258.5
4	L⁵	150 · 100 · 9	c	17.1	·	642	43.9
4	Fills	145 · 9	c	10244	·	500	20.5
14	L⁵	90 · 75 · 9	c	11.0	·	660	101.6
3	Pl⁵	255 · 9	c	18016	·	290	15.7
							744.7

2 c 744.7 = 1489.4

Floor Beams FB2,3,4,5,6,7 & 8 13- Required

4	Flg L⁵	125 · 75 · 9	c	135	·	5660	305.6
1	Pl	650 · 9	c	45923	·	5650	259.5
Part Same as for FB1							181.7
							746.8

13 c 746.8 = 9708.4

11,197.8

Summary of structural steel of Floor Beams

Bottom Struts BS1 2- Required

3	L⁵	90 · 75 · 9	c	11.0	·	5370	177.2
2	Pl⁵	405 · 9	c	28613	·	440	25.2
1	Pl	310 · 9	c	21902	·	440	9.6
4	L⁵	100 · 75 · 10	c	13.0	·	410	21.3
20	Pl⁵	60 · 9	c	4239	·	588	49.9
							283.2

2 c 283.2 = 566.4

Bottom Struts BS2,3,4,5,6,7 & 8 1- Required

52	L⁵	90 · 75 · 9	c	11.0	·	5370	307.6
4	"	125 · 75 · 9	c	13.5	·	420	22.7
4	"	100 · 75 · 10	c	13.0	·	420	21.8
4	"	125 · 75 · 9	c	13.5	·	410	22.1
4	"	100 · 75 · 10	c	13.0	·	410	21.3
8	"	100 · 75 · 10	c	"	·	400	41.6
8	"	100 · 75 · 10	c	"	·	390	40.6
8	"	100 · 75 · 10	c	"	·	385	40.0
8	"	100 · 75 · 10	c	"	·	380	39.5
4	"	100 · 75 · 10	c	"	·	375	19.5
4	Pl⁵	420 · 9	c	24675	·	470	55.3
4	"	420 · 9	c	"	·	460	54.6
4	"	420 · 9	c	"	·	455	54.0
4	"	425 · 9	c	30026	·	450	54.0
10	"	425 · 9	c	30026	·	445	133.6
10	"	190 · 9	c	13424	·	460	61.8
10	"	190 · 9	c	"	·	455	61.1
10	"	190 · 9	c	"	·	450	60.4
30	"	190 · 9	c	"	·	445	179.2
5	"	310 · 9	c	21902	·	445	48.7
							1039.7

Summary of structural steel of Bottom Struts

4670.3

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
[illegible] BLDG. TOKYO
SHOWA

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CALCULATIONS FOR

檜村橋材料表

Top Lateral Bracings TL 1,2,3,3A,4,5,6 & 6A 1-Required

4	L	75·75·9	@	996	=	7195	2866
22	"	75·75·9	"	"	=	7310	15799
2	"	75·75·9	"	"	=	7280	1450
4	"	75·75·9	"	"	=	3290	1311
50	"	75·75·9	"	"	=	3305	16459
2	"	75·75·9	"	"	=	3375	672
4	Pls	320·9	@	22608	=	795	719
10	"	320·9	"	"	=	905	1503
56	"	100·9	@	7065	=	1160	593
							41372

Bottom Lateral Bracings BL 1,2,3,4,5,6,7 & 7A 1-Required

4	L	150·90·9	@	164	=	7360	4828
4	"	150·90·9	"	"	=	7370	4835
4	"	150·90·9	"	"	=	7270	4769
4	"	150·90·9	"	"	=	7170	4704
4	"	150·90·9	"	"	=	7110	4664
4	"	150·90·9	"	"	=	7060	4631
2	"	150·90·9	"	"	=	7030	2306
2	"	150·90·9	"	"	=	7100	2329
4	"	150·90·9	"	"	=	3610	2368
4	"	150·90·9	"	"	=	3550	2329
4	"	150·90·9	"	"	=	3615	2371
4	"	150·90·9	"	"	=	3555	2332
4	"	150·90·9	"	"	=	3560	2335
4	"	150·90·9	"	"	=	3500	2296
4	"	150·90·9	"	"	=	3510	2303
4	"	150·90·9	"	"	=	3450	2263
4	"	150·90·9	"	"	=	3475	2280
4	"	150·90·9	"	"	=	3415	2240
4	"	150·90·9	"	"	=	3450	2263
4	"	150·90·9	"	"	=	3390	2224
3	"	150·90·9	"	"	=	3455	1690
3	"	150·90·9	"	"	=	3375	1661
1	L	150·90·9	"	"	=	3505	575
1	"	150·90·9	"	"	=	3445	565
4	Pls	370·9	@	26141	=	700	732
4	"	370·9	"	"	=	590	617
6	"	370·9	"	"	=	600	941
							67451

Sway Bracings at Panel 0 1-Required

4	L	130·130·9	@	179	=	7960	5999
4	"	130·130·9	"	"	=	3965	2767
4	"	130·130·9	"	"	=	3755	2710
24	"	130·130·9	"	"	=	430	1847
2	"	130·130·9	"	"	=	5328	1907
2	"	75·75·9	@	996	=	2750	548
1	Pl	725·12	@	68295	=	825	563
1	"	725·12	"	"	=	775	529
1	"	190·12	@	17898	=	455	81
28	Pls	170·12	"	16014	=	860	1166

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CALCULATIONS FOR

檜村橋材料表

2	L's	90 × 75 × 9	c	110	×	5428	1194
2	P/s	290 × 9	c	20489	×	370	152
5	"	190 × 9	c	13424	×	290	195
20	Wash.	70'	c	302	×	012	72
4	L's	90 × 90 × 10	c	133	×	370	197
4	"	100 × 100 × 10	c	149	×	510	304
2	P/s	470 × 12	c	44274	×	510	452
4	L's	150 × 100 × 9	c	171	×	920	629
2	P/s	470 × 12	c	44274	×	920	815
4	L's	100 × 100 × 10	c	149	×	520	310
2	P/s	470 × 12	c	44274	×	520	460
							22597
							2 c 22597 = 45194

Sway Bracings at panel 1 2-Required

4	L's	125 × 90 × 9	c	146	× 7125	416.1
4	"	125 × 90 × 9	c	"	× 3485	203.5
4	"	125 × 90 × 9	c	"	× 3520	205.6
2	"	75 × 75 × 9	c	996	× 2190	43.6
2	"	125 × 90 × 9	c	146	× 5490	160.3
1	P/s	395 × 9	c	27907	× 465	13.0
1	"	350 × 9	c	24728	× 395	9.8
1	"	190 × 9	c	13424	× 285	3.8
61	Wash	70'	c	302	× 009	16.6
4	L's	90 × 75 × 9	c	110	× 250	11.0
4	"	125 × 75 × 9	c	135	× 285	15.4
2	P/s	285 × 9	c	20135	× 325	13.1
4	L's	125 × 90 × 9	c	146	× 430	25.1
2	P/s	285 × 9	c	20135	× 430	17.3
4	L's	125 × 75 × 9	c	135	× 320	17.3
4	L's	90 × 75 × 9	c	110	× 190	8.4
2	P/s	330 × 9	c	23315	× 350	16.3
						11962
						2 c 11962 = 23924

Sway Bracings at Panel 2 2-Required

2	L's	125 × 90 × 9	c	146	× 8720	254.6
4	"	125 × 90 × 9	c	"	× 4290	250.0
1	P/s	360 × 9	c	25434	× 445	11.3
4	L's	125 × 90 × 9	c	146	× 355	20.7
4	"	90 × 75 × 9	c	110	× 250	11.0
2	P/s	340 × 9	c	24021	× 355	17.1
4	L's	150 × 90 × 9	c	164	× 350	23.0
4	"	90 × 75 × 9	c	110	× 190	8.4
2	P/s	330 × 9	c	23315	× 375	17.5
32	Wash.	70'	c	302	× 009	8.7
						6223
						2 c 6223 = 12446

Sway Bracings at Panel 3 2-Required

2	L's	90 × 90 × 10	c	188	× 7230	192.3
4	"	90 × 90 × 10	c	"	× 3550	188.9
1	P/s	355 × 9	c	25081	× 395	9.9
4	L's	125 × 75 × 9	c	135	× 285	15.4
4	"	90 × 75 × 9	c	110	× 250	11.0
2	P/s	285 × 9	c	20135	× 325	13.1

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CALCULATIONS FOR

檜村橋材料表

4	L's	125·75·9	@	13.5	×	.270		146
4	"	90·75·9	"	11.0	"	.190		84
2	Pls	285·9	"	20.135	"	.330		133
24	Wash.	70'	"	302	"	.009		65
								4734
							2 @ 4734 =	9468

Sway Bracings at Panel 4 2-Required

2	L's	90·90·10	@	133	×	6180		1644
4	"	90·90·10	"	"	"	3000		1596
1	Pl	320·9	"	22608	"	.485		110
4	L's	125·75·9	"	135	"	.285		154
4	"	90·75·9	"	110	"	.310		136
2	Pls	285·9	"	20135	"	.385		155
4	L's	125·75·9	"	135	"	.265		143
4	"	90·75·9	"	110	"	.250		110
2	Pl.	280·9	"	19782	"	.390		154
20	Wash.	70'	"	302	"	.009		54
								4256
							2 @ 4256 =	8512

Sway Bracings at Panel 5 2-Required

4	L's	90·75·9	@	110	×	2375		1045
4	"	90·75·9	"	"	"	2445		1076
4	"	90·75·9	"	"	"	335		147
2	Pls	275·9	"	19429	"	335		130
4	L's	125·75·9	"	135	"	305		197
4	"	90·75·9	"	110	"	190		84
2	Pls	330·9	"	23315	"	380		177
2	L's	90·75·9	"	110	"	350		77
1	Pl	290·9	"	20499	"	350		72
12	Wash.	70'	"	302	"	.009		33
								3038
							2 @ 3038 =	6076

Sway Bracings at Panel 6 2-Required

2	L's	90·90·10	@	133	×	1870		497
2	"	90·90·10	"	"	"	1920		511
4	"	90·75·9	"	110	"	415		183
2	Pls	265·9	"	18722	"	415		155
4	L's	125·75·9	"	135	"	365		197
4	"	90·75·9	"	110	"	190		84
2	Pls	330·9	"	23315	"	380		177
2	L	90·75·9	"	110	"	375		83
1	Pl	240·9	"	16956	"	375		64
								1951
							2 @ 1951 =	3902

Sway Bracings at Panel 7 1-Required

2	L's	90·90·10	@	133	×	1690		450
2	"	90·90·10	"	"	"	1725		459
4	"	90·75·9	"	110	"	475		209
2	Pls	266·9	"	18969	"	475		175
4	L's	125·75·9	"	135	"	360		194
4	"	90·75·9	"	110	"	190		84

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CALCULATIONS FOR

檜村橋材料表

2	Pls	330 × 9	=	2595	× .975	175
2	L	90 · 75 · 9	=	110	× 410	90
1	R	250 × 9	=	1025	× 410	67
						1903

Summary of Structural steel of Sway Bracings 11,425

Summary of Structural Steel of arch Span

Main Trusses	107,054#
Stringers	13248
Floor Beams	11,978
Bottom Struts	4670.3
Top Lateral Bracings	4137.2
Bottom Lateral Bracings	6745.1
Sway Bracings	11,425
	156,194.1 kgs

Shoe CS1 PN1 & AB1 1-Required

Cast steel Shoe	CS1	1 c	10050	=	10050
Pin	PN1	1 c	1098	=	1098
Anchor bolts	AB1	36ϕ × 700	4 c	673	= 269
					11417
				4 c 1141.7	= 45668 kgs

Setting apparatus of Shoe 1-Required

4	L	125 · 90 · 13	c	206	× 560	461
12	Set screw	22ϕ · 110			× 447	54
8	Anchor bolts	22ϕ · 200			× 828	66
8	Wash.	80 · 8	c	5024	× .080	32
8	Stud bolts	22ϕ ·			× .882	7.1
8	Sheet plate	100 · 10 ·		7850	× .100	63
4		100 · 10 ·	c	7850	× .180	57
						804

Grand summary of structural steel of arch span 1625813 kgs

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CALCULATIONS FOR

楢村橋材料表

	Main Girder G1 R L			4- Required				
Main section	4	Flg Ls	150×100×9	c	17.1	× 10701	7319	
"	1	Web Pl	1030 × 9	c	72770	× 5700	4148	
"	1	"	1030 × 9	"	"	× 4995	3635	
Stiffeners	4	Ls	125×90×9	c	14.6	× 1022		597
"	2	Fills	170 × 9	c	12011	× 830		199
"	4	Ls	125×75×9	c	135	× 1022		552
"	2	Fills	70 × 9	c	4946	× 830		82
"	1	Fill	70 × 9	c	"	× 545		27
"	16	Ls	125×75×9	c	135	× 1040		2246
End connection	2	"	150×100×12	"	224	× 1022		458
"	2	Fills	215 × 9	c	15190	× 830		252
Splice	4	Pls	190 × 9	c	13424	× 540		290
"	2	"	300 × 9	c	21195	× 440		187
End connection	1	Pl	470 × 9	c	33206	× 734		244
"	1	Fill	310 × 9	c	21902	× 340		74
Gusset	1	Pl	410 × 9	c	28967	× 500		145
"	1	"	500 × 9	c	35325	× 640		226
"	1	"	585 × 9	c	41330	× 640		265
Sole plate	1	"	360 × 25	c	70650	× 370		261
								15102 + 6105 = 21207
								4 c 21207 = 84828
	Floor Beam FB9			4- Required				
	1	Flg L	100×75×10	c	13.0	× 5630		732
	1	"	100×75×10	c	"	× 5380		699
	2	Flg Ls	100×75×10	c	"	× 5350		1391
	1	Web Pl	650 × 9	c	45923	× 5630		2585
	14	Ls	90×75×9	c	110	× 660		1016
	3	Pls	110 × 9	c	7772	× 290		68
	2	Ls	125×75×9	c	135	× 570		154
								6645
								4 c 6645 = 26580
	Stringers S3 R L, S4, S5 R L & S6			1- Required				
	6	Is	350 × 150	c	585	× 5695		19989
	6	"	350 × 150	c	"	× 5190		18217
	12	Pls	160 × 9	c	11304	× 290		380
	126	Bolts	12" × 200	"	0253			319
	126	Wash	60 × 8	c	3768	× 060		285
								39190
	Lateral Bracings LB1 & LB2			1- Required				
	4	Ls	75×75×9	c	996	× 7170		2857
	3	"	75×75×9	c	"	× 3535		2817
	4	Pls	270 × 9	c	19076	× 480		366
	16	Ls	75×75×9	c	996	× 660		1052
	16	"	150×90×9	c	164	× 192		504
	16	Pls	140 × 9	c	9891	× 185		229
								7825
	Summary of structural steel of Girder span							158423 Kgs

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CALCULATIONS FOR

楠村橋材料表

Shoe C52 & AB2 1- Required

Cast iron shoe	C52			4 @	129.0 =	516.0
Cap plate	8	Pls	80 · 45 · 2826 ·	.160	=	36.2
	16	Bolts	22φ · 115	@ 0.456	=	7.3
Anchor bolts	16		25φ · 600	@ 2.70	=	43.2
	16	Wash.	100 · 9 · 7065 ·	.100	=	11.3
						614.0

Grand Summary of structural steel of Girder span 16,1563 kgs

Expansion Joint EJ 1 2- Required

1	L	150 · 90 · 9	@	164	× 6.330	103.8
1	Pl	230 · 9	@	16.250	× 6.000	97.5
1	Bar	40 · 10	@	3.140	× 6.310	19.8
5	Ls	75 · 75 · 9	@	996	× .230	11.5
5	Wash.	60 · 10	@	4.710	× .140	3.3
5	Bolts	12φ · 530	@	.51		2.6
5	Wash.	60 · 5	@	2.355	× .060	.7
5	Is	150 · 125	@	362	× .300	54.3
10	Bolts	12φ · 60	@	.092		.9
10	Wash.	60 · 8	@	3.768	× .060	23
						296.7
						2 · 2967 = 593.4

Expansion Joint EJ 2 2- Required

1	L	75 · 75 · 9	@	996	× 6.330	63.0
1	Pl	230 · 9	@	16.250	× 6.000	97.5
1	Checkd Pl	260 · 9	@	18.369	× 6.310	115.9
5	Bolts	12φ · 500	@	.493		2.5
5	Ls	75 · 75 · 9	@	996	× .230	11.5
5	Wash.	60 · 10	@	4.710	× .060	1.4
2	Bolts	12φ · 50	@	.083		.2
3	"	12φ · 65	@	.096		.3
3	Wash.	60 · 8	@	3.768	× .060	.7
						293.0
						2 · 2930 = 586.0

Summary of structural steel of Expansion joint 1,1794 kgs

Grand summary of structural steel

Arch span	162,843
Girder span	16,563
Expansion joints	1,794
	180,370 kgs

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CALCULATIONS FOR

猪村橋材料表
鋼材種類別内譯表
構拱徑間

部材		所要数	鋼鈑	等辺山形鋼	不等辺山形鋼	溝形鋼	工形鋼	ボールト	總重量
主構	U_0-U_2	4	23488		836	42160			66484
〃	U_2-U_3	〃	24912		736	31760			57708
〃	U_3-U_5	〃	21956		460	36144			58560
〃	U_5-U_6	〃	41232		720	33356			75508
〃	U_6-U_6	2	12588		230	10024			28842
〃	DM4.10	30	1790		4680				6470
			125866		8062	159644			293572
主構	L_0-L_1	4	52200	22588					74788
〃	L_1-L_2	〃	43824	21768					65592
〃	L_2-L_3	〃	42268	21060					63328
〃	L_3-L_4	〃	35032	20440					55472
〃	L_4-L_5	〃	34336	15292					49628
〃	L_5-L_6	〃	32304	15052					47356
〃	$L_6-L_7-L_6$	2	40380	15008					55388
〃	DM1234567&8	30	2710		7782				10492
			283054	131208	7782				422044
主構	U_0-L_0	4	39412		11584	60228			111224
〃	U_1-L_1	〃	8136		3068	31332			42536
〃	U_2-L_2	〃	3432		20296				23728
〃	U_3-L_3	〃	2712		13244				15956
〃	U_4-L_4	〃	2308		9496				11804
〃	U_5-L_5	〃	1600		6844				8444
〃	U_6-L_6	〃	1424		5268				6692
〃	U_7-L_7	2	682		2366				3048
			59726		72166	91560			223432
主構	U_0-L_1	4	3428		29600				32028
〃	U_1-L_2	〃	2844		20952				23796
〃	U_2-L_3	〃	2436		17232				19668
〃	U_3-L_4	〃	2084		14432				16516
〃	U_4-L_5	〃	1852		12416				14268
〃	U_5-L_6	〃	1676		11192				12868
〃	$U_6-L_7-U_6$	2	1618		10734				12352
			15938		115558				131496
			484564	131208	203568	251204			1070544
縦桁		1	3799				127608	1063	132468
横桁		15	44335		67643				111978
支材		15	9326		37377				46703
上部綴構		1	2815	38557					41372
下部綴構		1	2290		65161				67451

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
SHOWA BLDG. TOKYO

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CALCULATIONS FOR

檜村橋材料表

部材		所要数	鋼鈑	等辺山形鋼	不等辺山形鋼	溝形鋼	工形鋼	ボールト	総重量
対傾綾構	0	2	897.0	325.78	30.46				45.194
〃	1	〃	179.8	8.72	212.54				23.924
〃	2	〃	109.2		113.54				12.446
〃	3	〃	85.6	76.24	9.88				7.468
〃	4	〃	94.6	64.80	10.86				8.512
〃	5	〃	92.4		52.52				6.076
〃	6	〃	79.2	20.16	10.94				3.902
〃	7	1	41.7	9.09	5.77				1.903
			156.95	504.79	452.51				114.425
			562.24	220.44	419.00	251.204	127.606	10.63	1581.941

部材		所要数	鋼鈑	鋳鋼	不等辺山形鋼	アンカーボールト及ボールト	錨及ナット	総重量
鋼橋付属物	沓	4		40.200		10.76	43.92	45.668
		1	1.52		4.61	1.91		8.04
			1.52	40.200	4.61	12.67	43.92	46.472

鋼鈑	56.298
等辺山形鋼	22.044
不等辺山形鋼	41.946
工形鋼	12.761
溝形鋼	25.120
ボールト及アンカーボールト	2.33
錨及ナット	4.39
鋳鋼	40.20
	158.149

163.841 Kgs

鉄桁 部材		所要数	鋼鈑	等辺山形鋼	不等辺山形鋼	工形鋼	アンカーボールト及ボールト	鋳鋼	総重量
間詰	桁	4	40.140		44.688				84.828
	主桁	4	10.612		15.968				26.580
	縦桁	1	6.05			32.206	3.19		37.190
	横桁	〃	5.95	6.726	5.04				7.825
	承	4	4.75				5.05	51.60	6.140
			52.487	6.726	51.60	32.206	8.24	51.60	164.563
伸縮金物	石工	2	24.72	2.30	20.76	10.86	0.70		59.34
	鉄	1	4.310	1.490			0.60		5.860
			6.782	1.720	20.76	10.86	1.30		11.794
			59.269	8.446	63.236	32.292	9.54	51.60	176.357
			59.27	8.45	63.24	32.29	0.95	5.16	176.36 Kgs

170.25

総計	
鋼鈑	62.225
等辺山形鋼	22.869
不等辺山形鋼	48.270
工形鋼	16.690
溝形鋼	25.120
ボールト及アンカーボールト	3.28
錨及ナット	4.39
鋳鋼	40.20
鉄	5.16
	180.477 Kgs

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（独立行政法人 土木研究所蔵）

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CONSULTING ENGINEER
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CALCULATIONS FOR

増村橋材料表

Materials of floor slab for entire bridge.

Concrete 1:2:4 mixture

Slab No.1	2	6.03 × .014 × 10.69 =	17959
" No.2 & 2A	6	6.00 × .014 × 10.391 =	52371
" No.3	1	6.00 × .014 × 10.782 =	4.057
Copings	2	.295 × .034 × 9.4508 =	18958
Fillets on Stringer (Girder span)			
Outside stringer	4	.225 × .075 × 10.381 =	0734
Inside stringer	2	.240 × .090 × 10.881 =	0470
Fillets on Chord & Stringer (Arch span)			
Chord	4	.332 × .022 × 5.200 =	0.152
"	4	.347 × .037 × 5.191 =	0.267
"	4	.355 × .045 × 5.200 =	0.332
"	4	.355 × .045 × 5.191 =	0.332
"	4	.347 × .037 × 5.200 =	0.267
"	4	.332 × .022 × 5.191 =	0.152
"	2	.336 × .026 × 5.200 =	0.091
Outside Stringer	4	.225 × .075 × 2.600 =	0.176
"	4	.247 × .097 × 5.200 =	0.498
"	4	.262 × .112 × 5.191 =	0.609
"	4	.270 × .120 × 5.200 =	0.674
"	4	.270 × .120 × 5.191 =	0673
"	4	.262 × .112 × 5.200 =	0.610
"	4	.247 × .097 × 5.191 =	0.497
"	2	.251 × .101 × 5.200 =	0264
Inside Stringer	2	.240 × .090 × 2.600 =	0.112
"	2	.262 × .112 × 5.200 =	0305
"	2	.277 × .127 × 5.191 =	0365
"	2	.285 × .135 × 5.200 =	0400
"	2	.285 × .135 × 5.191 =	0399
"	2	.277 × .127 × 5.200 =	0366
"	2	.262 × .112 × 5.191 =	0305
"	1	.266 × .116 × 5.200 =	0.160
			107555 cub. m.



Forms

Girder span	2	6.728 × 10.881 =	14641
Arch span	1	6.840 × 7.2746 =	49758
			64399 cm²

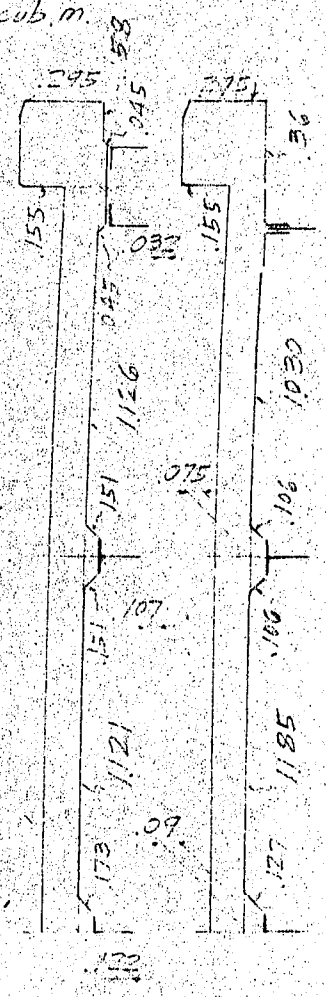
Reinforcements, Plain bars 10.700 k. tons

Pavement, 1:2 mortar finish area 6.00 × 9.4508 = 56.705 m²
Volume 0.03 × 56.705 = 17.012 m³

Construction joint, 9 mm elastite 挿入 8 + ft

Coping, Cement mortar finish 2.5 cm thick
area 2 × 0.505 × 9.4508 = 9.545 m²
Volume 0.025 × 9.545 = 2.386 m³

Drain, 雨水弁 Cast iron 4 × 57.0 = 228 kgs
6" Cast iron flanged pipe 1.45 meter long 一本ヨリ 40 kg 以上モノ 4 + ft



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
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MADE BY M.J DATE 11-11-28 FILE NO
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CALCULATIONS FOR

檜村橋材料表

Materials of handrails

Handrail grate	HR1	4	c 446	= 1784
	HR2345	144	c 490	= 70560
Handrail post		146	c 327	= 47742
Top rail		4	c 550	= 2200
		140	c 378	= 52920
Anchor bolt	16mm² 180mm	8	c 0382	= 3.1
Washer	60·8·130	4	c 0490	= 20
Cap plate	100·9·380	146	c 2685	= 3920

179177 Kg 一米当り 189.2Kg

部類譯表

鑄鉄	175206 Kgs
アンカーボールト	3.1
鋼鈑	394.0

兩水桁据付モルタル 四個所分 配合 1:3

2 c .17·.01·.19 = 0.00065
.24·.01·.23 = 0.00055
.24·.01·.16 = 0.00038
0.00158 c 4 = 0.006 m³

高欄間柱据付用モルタル 146個所分
配合 1:3

146 c 0.416·.001·.021 = 0.128
146 c 0.04·.0012·.008 = 0.006
0.134 m³

増田淳氏関係資料
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CHECKED BY J. M. DATE 11-12-9 PAGE NO 20

CALCULATIONS FOR

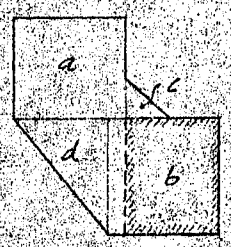
檜村橋材料表

Materials of East arch pier

Concrete, 1:2:4 mixture

上流側

Body, part a 130 · 147 · 240 = 4586 ✓
" " b 150 · 150 · 240 = 5400 ✓
" " c ½ · 044 · 053 · 240 = 0280 ✓
" " d ½ · 125 · 150 · 240 = 2250 ✓



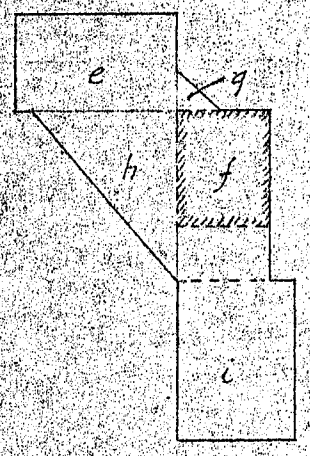
上流側

下流側

Body, part e 130 · 212 · 240 = 6614 ✓
" " f 130 · 220 · 240 = 6864 ✓
" " g ½ · 044 · 053 · 240 = 0280 ✓
" " h ½ · 185 · 220 · 240 = 4884 ✓
" " i 160 · 220 · 270 = 9504 ✓

Strut 130 · 150 · 360 = 7020 ✓
47682 m³ ✓

混凝土填充部分 say ½ · 090 · 080 · 180 = 0468 ✓
48150 m³ ✓



下流側

Forms 岩石ニ切込ニタル部分ハ型枠ヲ用イズ

上流側 part b, front side 030 · 240 = 072 ✓
" a, front side 077 · 240 = 185 ✓
" c " " 069 · 240 = 166 ✓
" a, inside 130 · 141 = 191 ✓
" c, " ½ · 044 · 053 = 012 ✓

下流側 part e, front side 077 · 240 = 185 ✓
" " Both sides ×2 · 130 · 212 = 551 ✓
" g, front side 069 · 240 = 166 ✓
" " Both sides 044 · 053 = 023 ✓
" f, front side 220 · 240 = 528 ✓
" " out side 130 · 220 = 286 ✓
" h, Both sides 185 · 220 = 407 ✓
" i, front side say ½ · 220 · 270 = 297 ✓
" " out side 160 · 220 = 352 ✓

Strut, front side say 150 · 220 = 330 ✓
37.51 m² ✓

Reinforcement, Plain bars 0.634 kg tons See drawings ✓

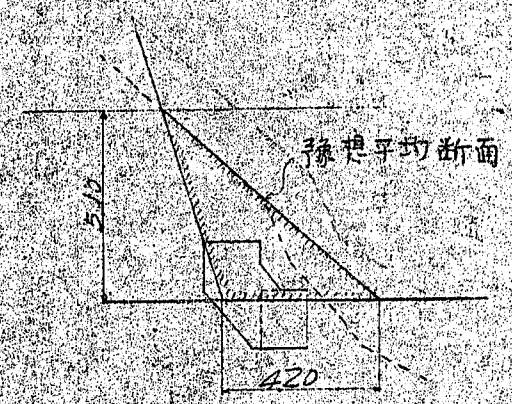
Excavation

掘鑿豫想幅ヲ15米トス

½ · 420 · 510 · 150 = 1607 m³ ✓

土砂ヲ金切取ニ三割トス

掘鑿土砂 482 m³ ✓
掘鑿岩石 1125 m³ ✓



切取表面石積 (控30種)

切取面 350 · 150 = 525 ✓
橋脚間天端 200 · 36 = 72 ✓
" 背面 130 · 36 = 47 ✓
644 m² ✓

裏込混凝土 配合 1:3:6
644 · 02 = 12880 m³

石積用モルタル 配合 1:3
644 · 03 · 02 = 3864 m³

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTING ENGINEER
~~FETE~~ BLDG, TOKYO
~~BHOWA~~

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CHECKED BY J.M DATE 11-12-9 PAGE NO 21

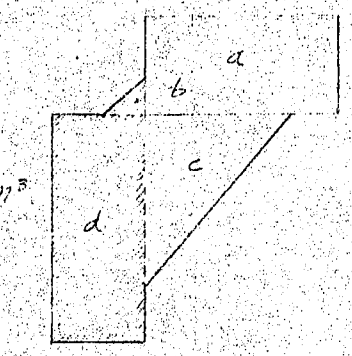
CALCULATIONS FOR

檜村橋材料表

Materials of West arch pier

Concrete, 1:2:4 mixture

Body, part a　2 c 1.30 · 2.52 · 2.40 = 15.35 ✓
" " b　0.44 · 0.53 · 2.40 = 0.560 ✓
" " c　1.85 · 2.20 · 2.40 = 9.768 ✓
Body & strut d　1.30 · 3.00 · 8.40 = 32.60 ✓
58.83 m³



Forms, 岩石ニ切込ミタル部分ハ型枠ヲ用イズ

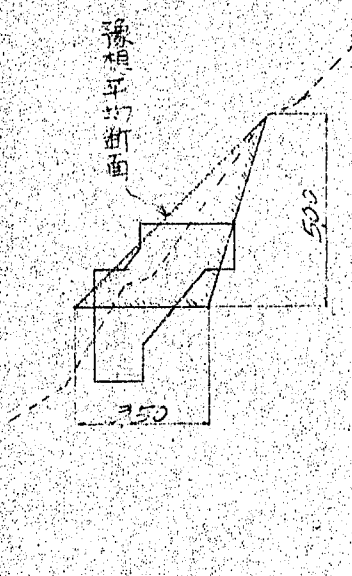
part a, front side　2 c 0.77 · 2.40 = 3.70 ✓
" " both sides　4 c 1.30 · 2.52 = 13.10 ✓
" "b" front side　2 · 0.69 · 2.40 = 3.31
" " both sides　2 c 0.44 · 0.53 = 0.47 ✓
" c, " 2 c 1.85 · 2.20 = 8.14 ✓
" d, out side　2 c 1.30 · 3.00 = 7.80 ✓
Part d + strut, front side　3.00 · 8.40 = 25.20 ✓
67.72 m²

Reinforcements, Plain bars　0.67 ✓ t. tons

Excavation

掘鑿豫想幅ヲ12米トス

1/2 c 3.50 · 5.00 · 12.00 = 105.0 m³ ✓



岩石ヲ全切取、三割トス

掘鑿土砂　735 m³ ✓
掘鑿岩石　315 m³ ✓

切取表面石積　(控 30 糎)

切取面　3.10 · 12.0 = 37.2 ✓
橋脚間天端　1.35 · 3.6 = 6.7 ✓
" 背面　1.30 · 3.6 = 4.7 ✓
4.36 m² ✓

石積用モルタル　配合 1:3

4.86 · 0.3 · 0.2 = 2.916 m³

裏込混凝土　配合 1:3:6

4.86 · 0.2 = 9.720 m³

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
~~ｻﾝｹｲ~~ BLDG, TOKYO
~~SHOWA~~

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CALCULATIONS FOR

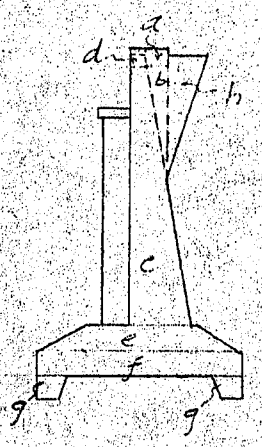
櫓村橋材料表

Materials of girder abutment

Concrete 1:3:6 mixture

Below entrance post	a	2	1.00	1.00	0.36 = 0.720 ✓
" " "	b		0.50	1.00	3.10 = 1.550 ✓
Parapet Wall + body	c		8.00	1.05	6.83 = 57.372 ✓
" top	d		.25	0.50	6.00 = 0.750 ✓
踏掛石 less			.20	0.35	6.00 = − 0.420 ✓
girder seat coping			.80	0.30	6.80 = 1.632 ✓
" body			.70	5.37	6.60 = 24.809 ✓
base "	e		.60	4.30	8.00 = 20.640 ✓
" "	f		.80	5.50	8.00 = 35.200 ✓
" "	g	2	.65	0.60	8.00 = 6.240 ✓
Wing wall "	h		.98	0.30	3.27 = .961 ✓

✓
149.454 m³



蹴込石 化粧石 .35 · .20 · 6.00 = 0.420 m³

蹴込石据付用モルタル
1/4 · .35 · .0075 × .20 = 0.006 ✓
.35 · .001 · 6.00 = .0021 ✓
配合 1:3 0.027 m³ ✓

Forms

Below entrance post, front + side	6	0.27	1.00	1.62 ✓
" " " , rear	2	0.19	1.00	0.38 ✓
Parapet wall, front			1.20	8.00 = 9.60 ✓
Body, front	2	0.30	0.60	= 0.36 ✓
" "	2	0.70	5.37	= 7.52 ✓
Girder seat Coping "			0.40	8.40 = 3.36 ✓
" " body, front			5.37	6.60 = 35.44 ✓
" " " , side	2	0.70	5.37	= 7.52 ✓
Body, side	2	1.05	6.83	= 14.34 ✓
Below entrance post, side	2	0.50	3.10	= 3.10 ✓
" " " , rear	2	0.70	3.27	= 4.58 ✓
Body, rear			6.00	6.96 = 41.76 ✓
" "	2	1.00	3.78	= 7.56 ✓
Base, front + rear	2	1.40	8.00	= 22.40 ✓
" , side	2	0.60	4.30	= 5.16 ✓
" " "	2	0.80	5.50	= 8.80 ✓
" " "	4	0.65	0.60	= 1.56 ✓
Wing wall, side	2	0.98	3.27	= 6.41 ✓
" " , rear	2	0.30	3.41	= 2.05 ✓

✓
183.52 m²

Reinforcements, Plain bars 1816 kg tons ✓

Excavation for East girder abutment

豫想平均掘鑿高 5.50m ✓

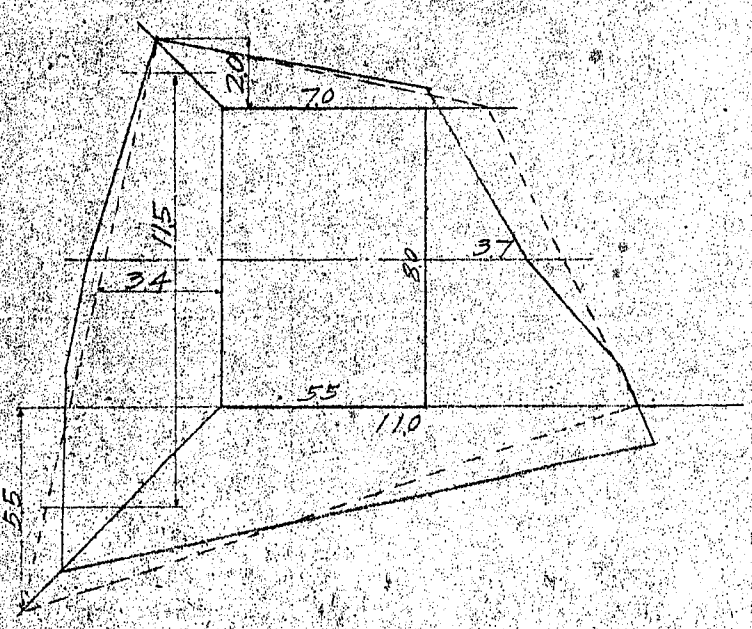
底面積 5.50 · 8.00 = 44.0 m² ✓

天端面積 (圖示ノ通リ假定ス)

	5.50 · 8.00	= 44.0 ✓
½ c	5.50 · 11.00	= 30.3 ✓
	3.40 · 11.50	= 39.1 ✓
½ c	2.00 · 7.00	= 7.0
	3.70 · 8.00	= 29.6
		150.0 m² ✓

平均面積 97.0 m² ✓

掘鑿土砂量 97.0 · 5.5 = 533.5 m³ ✓



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

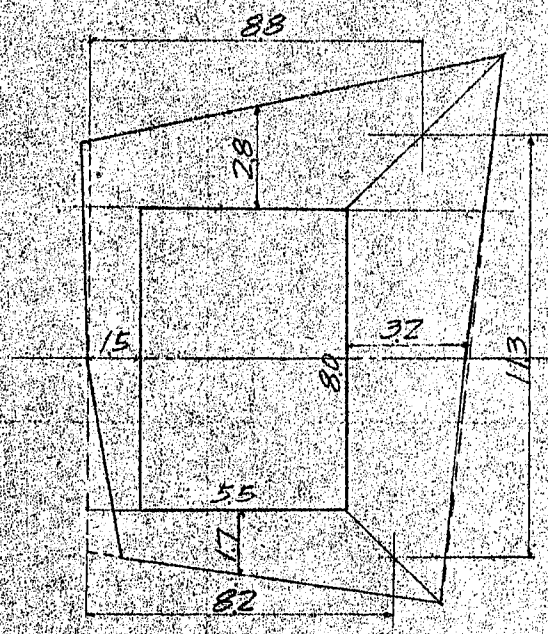
JIUN MASUDA
CONSULTING ENGINEER
BLDG, TOKYO

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CALCULATIONS FOR

檜村橋材料表

Excavation for west girder abutment
豫想平均掘鑿高 5.00m



底面積 5.50 × 8.00 = 44.0 m²
天端面積 (圖示ノ通リ假定ス)
5.50 × 8.00 = 44.0 〃
2.80 × 8.80 = 24.6 〃
3.20 × 11.30 = 36.2 〃
1.70 × 8.20 = 13.9 〃
1.50 × 8.00 = 12.0 〃
130.7 m²
平均面積 87.4 m² 〃

掘鑿土砂量 87.4 × 5.00 = 437.0 m³ 〃

埋戻土 取付道路ニ計上

親柱及袖高欄 一基ニ

Concrete, 1:3:6 mixture
親柱 0.87 × 0.87 × 1.20 = 0.934 〃
袖高欄 0.27 × 0.585 × 1.00 = 0.158 〃
1.142 m³

Forms
親柱 4 ∈ 0.87 × 1.30 = 4.52 〃
〃 0.27 × 0.41 = 0.11 〃
袖高欄 2 ∈ 0.585 × 1.00 = 1.17 〃
〃 0.27 × 0.585 = 0.16 〃
5.74 m² 〃

Reinforcements, Plain bars 0.031 Kg-ton 〃

人造洗出仕上
親柱下部 4 ∈ 0.19 × 1.00 = 0.76 〃
親柱 4 ∈ 0.90 × 1.35 = 4.79 〃
親柱天端 1.00 × 1.00 = 1.00 〃
袖高欄 2 ∈ 0.05 × 0.41 = 0.04 〃
〃 2 ∈ 0.60 × 0.98 = 1.18 〃
〃 0.30 × 1.03 = 0.09 〃
10.86 m² 〃

橋名鈑 親柱一基ニ 3/4 枚ヲ計上ス 〃

伸縮金物据付モルタル 配合 1:3
5 ∈ 0.15 × 1.5 × 0.31 = 0.035 m³

鈑桁鉄筋据付モルタル 配合 1:2
2 ∈ 0.44 × 0.024 × 0.70 = 0.015 m³

横桁鋼筋据付モルタル 配合 1:2
2 ∈ 0.95 × 0.025 × 1.29 = 0.061 m³

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
[redacted] BLDG. TOKYO.

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CALCULATIONS FOR

檜杖橋杭料表

東岸取付道路

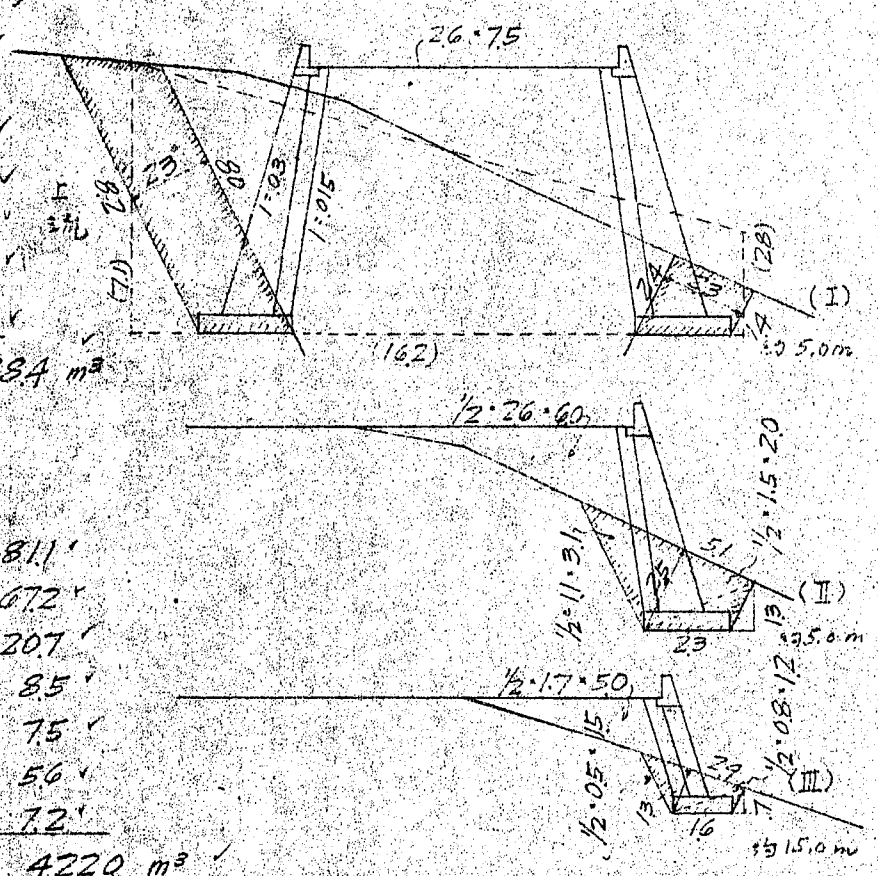
振鑿（杭頭堂拡鑿ヨリ深ク工ル処ヲ要スル部分）

上流側　8.1 · 2.3 · 7.0 = 130.4 ✓

下流側 I　2.3 · 1.9 · 5.0 = 21.9 ✓

II　½ · 5.1 · 2.5 · 5.0 = 31.9 ✓
　　½ · 2.3 · 1.3 · 5.0 = 7.5 ✓

III　½ · 2.9 · 1.3 · 15.0 = 28.3 ✓
　　½ · 1.6 · 0.7 · 15.0 = 8.4 ✓
　　228.4 m³ ✓



埋戻土（橋台振鑿ノ埋戻ヲ含ム）

I　4.95 · 16.2 · 6.0 = 481.1 ✓

石積部分　1.50 · 6.4 · 7.0 = 67.2 ✓
　　〃　1.80 · 2.3 · 5.0 = 20.7 ✓

II　½ · 1.10 · 3.1 · 5.0 = 8.5 ✓
　　½ · 1.50 · 2.0 · 5.0 = 7.5 ✓

III　½ · 0.50 · 1.5 · 15.0 = 5.6 ✓
　　½ · 0.80 · 1.2 · 15.0 = 7.2 ✓
　　422.0 m³ ✓

盛土工

I　2.60 · 7.5 · 5.0 = 97.5 ✓

II　½ · 2.60 · 6.0 · 5.0 = 39.0 ✓

III　½ · 1.70 · 5.0 · 15.0 = 63.8 ✓
　　200.3 m³ ✓

粗石練積工（控30糎）

上流側　6.60 · 6.0 = 39.6 ✓
下流側　6.60 · 5.2 = 34.3 ✓
　　〃　4.90 · 5.0 = 24.5 ✓
　　〃　2.60 · 15.0 = 39.0 ✓
　　137.4 m² ✓

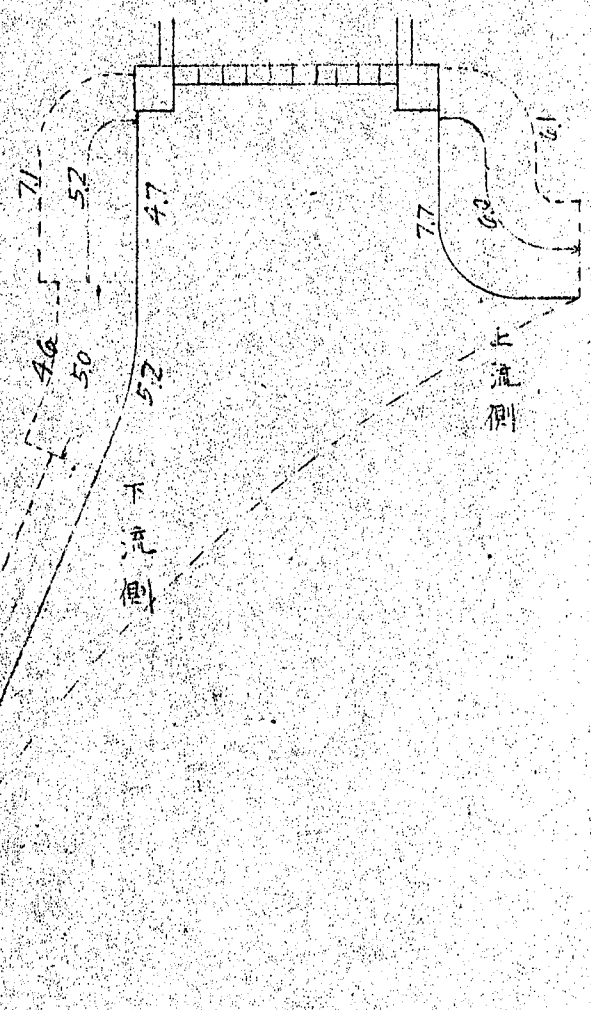
石積用モルタル　配合 1:3
　　137.4 · 0.3 · 0.2 = 8.244 m³

裏込混凝土　混合 1:3:6

上流側　6.6 · 6.8 · 6.0 = 269.28 ✓
下流側　6.6 · 6.8 · 5.2 = 233.38 ✓
　　〃　4.9 · 5.6 · 5.0 = 137.20 ✓
　　〃　2.6 · 2.0 · 15.0 = 78.00 ✓
　　717.86 m³ ✓

練積工基礎混凝土　混合 1:3:6

上流側　25 · 0.5 · 6.1 = 76.25 ✓
下流側　25 · 0.5 · 7.1 = 88.75 ✓
　　〃　23 · 0.5 · 4.6 = 52.90 ✓
　　〃　16 · 0.4 · 15.0 = 96.00 ✓
　　313.90 m³ ✓



表張敷砂利　厚9糎

4.9 · 8.0 = 39.2
½ · 1.2 · 11.0 = 6.6
　　100.8 m² ✓

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
BLDG. TOKYO

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CALCULATIONS FOR

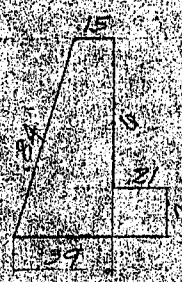
檜村橋材料表

裏石工（練積工裏込用）

上流側　66 × 45 × 60 = 178 ✓
下流側　66 × 45 × 52 = 154 ✓
〃　49 × 45 × 50 = 110 ✓
〃　26 × 45 × 150 = 176 ✓
613 m³ ✓

胴止用混凝土　混合 1:3:6

断面　027 × 090 = 0216 ✓
020 × 021 = 00042 ✓
0258 m³ ✓



上流側　0258 × 77 = 1987 ✓
下流側　0258 × 249 = 6424 ✓
8411 m³ ✓

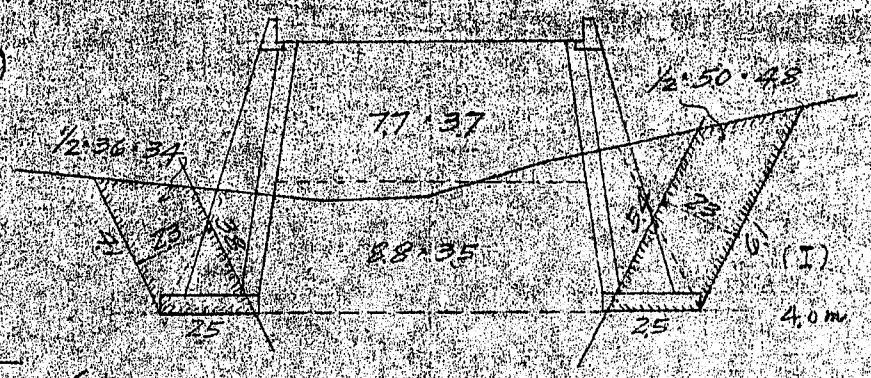
胴止用型枠

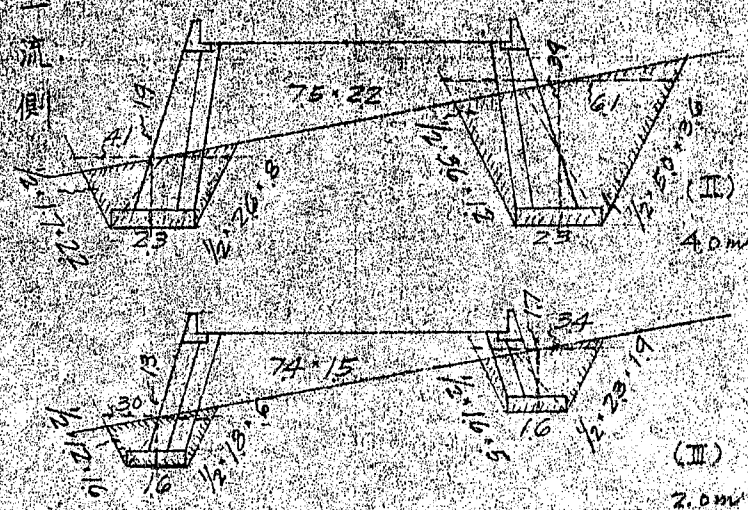
上流側　164 × 77 = 1263 ✓
下流側　164 × 249 = 4084 ✓
5347 m² ✓

西岸取付道路

掘鑿（横断寸法ヨリ求メタル断面ニ依ル部分）

上流側 I　40 × 23 × 40 = 368 ✓
〃 I　32 × 19 × 40 = 243 ✓
〃 II　23 × 13 × 20 = 60 ✓
下流側 I　53 × 23 × 40 = 554 ✓
〃 II　42 × 39 × 40 = 655 ✓
〃 III　25 × 17 × 20 = 85 ✓
1945 m³ ✓





埋戻土（橋台掘鑿ノ埋戻ヲ含ム）

I　½ × 36 · 34 · 40 = 245 ✓
I　88 · 35 · 40 = 1232 ✓
I下　½ × 50 · 48 · 40 = 480 ✓
II上　½ × 17 · 22 · 40 = 75 ✓
II上　½ × 26 · 08 · 40 = 42 ✓
II下　½ × 36 · 12 · 40 = 86 ✓
II下　½ × 50 · 36 · 40 = 360 ✓
III上　½ × 12 · 16 · 20 = 19 ✓
III上　½ × 18 · 06 · 20 = 11 ✓
III下　½ × 16 · 05 · 20 = 08 ✓
III下　½ × 23 · 19 · 20 = 44 ✓
2602 m³ ✓

盛土工

I　77 · 37 · 40 = 1140 ✓
II　75 · 22 · 40 = 660 ✓
III　74 · 15 · 20 = 222 ✓
2022 m³ ✓

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
SHOWA BLDG. TOKYO

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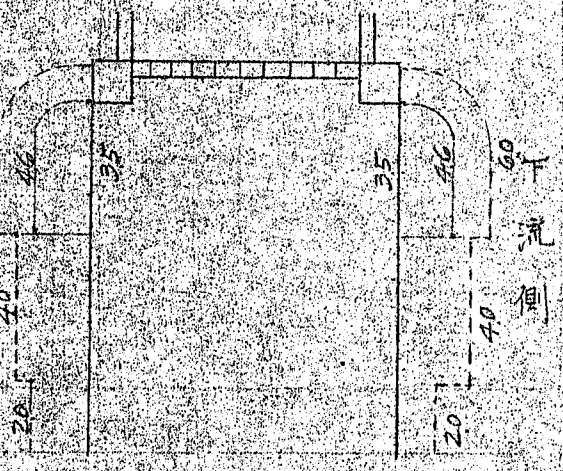
CALCULATIONS FOR

檜村橋材料表

粗石練積工 (控 30 種)

上流側
66 × 46 = 304 ㎡
45 × 40 = 180 〃
31 × 20 = 62 〃

下流側
66 × 46 = 304 〃
45 × 40 = 180 〃
16 × 20 = 32 〃
1062 ㎡ 〃



石積用モルタル 配合 1:3

1062 × 03 × 02 = 6372 m³

裏込混凝土 混合 1:3:6

上流側
66 × 68 × 46 = 20645 〃
45 × 54 × 40 = 9720 〃
31 × 20 × 20 = 1240 〃

下流側
66 × 68 × 46 = 20645 〃
45 × 54 × 40 = 9720 〃
16 × 20 × 20 = 0640 〃
62610 m³ 〃

練積工基礎混凝土 混合 1:3:6

2 × 25 × 05 × 60 = 15000 〃
2 × 22 × 0.5 × 40 = 8800 〃
2 × 16 × 04 × 20 = 2560 〃
26360 m³ 〃

粟石工 (練積工裏込用)

2 × 66 × 045 × 46 = 273 〃
2 × 45 × 045 × 40 = 162 〃
31 × 045 × 20 = 28 〃
16 × 045 × 20 = 14 〃
477 m³ 〃

駒止用混凝土 混合 1:3:6

斯面 0.258 m²

2 × 0258 × 9.5 = 4902 m³ 〃

駒止用型枠

2 × 164 × 9.5 = 3116 m² 〃

表裏敷砂利 厚 9 糎

80 × 100 = 80.0 m²

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTING ENGINEER
~~JI-II~~ BLDG, TOKYO

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CALCULATIONS FOR

檜村橋材料表

セメント袋数内譯明細表

上部工事

名稱	材料	配合	容積	一立米当り袋数	袋数
床版	混凝土	1:2:4	107.555 m³	6.6	709.863
鋪装	モルタル	1:2	17.012 〃	14.4	244.973
地覆仕上	〃	〃	2.356 〃	〃	34.358
雨水桝据付	〃	1:3	0.006 〃	10.8	0.065
高欄間柱据付	〃	〃	0.134 〃	〃	1.447
親柱	混凝土	1:3:6	4.568 〃	4.8	21.926
〃	人造仕上		4.344 m²	0.33	14.335
					1026.967

東橋脚

橋脚	混凝土	1:2:4	48.150	6.6	317.790
石積用	モルタル	1:3	3.864	10.8	41.731
鋼筈据付	〃	1:2	0.061	14.4	0.878
石積裏込	混凝土	1:3:6	12.880	4.8	61.824
					422.223

西橋脚

橋脚	混凝土	1:2:4	58.813	6.6	388.166
石積用	モルタル	1:3	2.916	10.8	31.493
鋼筈据付	〃	1:2	0.061	14.4	0.878
石積裏込	混凝土	1:3:6	9.720	4.8	46.656
					467.193

橋臺

橋臺	混凝土	1:3:6	149.454	4.8	717.379
蹴込石据付	モルタル	1:3	0.027	10.8	0.292
伸縮金物据付	〃	〃	0.035	〃	0.378
鉄筈据付	〃	1:2	0.015	14.4	0.216
					718.265
					2
					1436.530

東岸

取付道路					
石積用	モルタル	1:3	8.244	10.8	89.035
裏込	混凝土	1:3:6	71.786	4.8	344.573
基礎	〃	〃	31.390	〃	150.672
駒止	〃	〃	8.411	〃	40.373
					624.653

西岸

取付道路					
石積用	モルタル	1:3	6.372	10.8	68.818
裏込	混凝土	1:3:6	62.610	4.8	300.528
基礎	〃	〃	26.360	〃	126.528
駒止	〃	〃	4.902	〃	23.530
					519.404

2496.970 袋

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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