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CALCULATIONS FOR

List of Materials for Chorokia-Bache for Kumano-to-kan Revised 14-4-7 19-4-29

Weight of Stringers

Stringers RS1 A1B1C1

web pl	1Pl.	28 . 3/8	@	28.06	.	26'-0"	=	730.0
Top flange	2Ls	5 . 3½ . 3/8	@	10.40	.	26'-0"	=	540.0
Bott. flange	2Ls	5 . 3½ . 3/8	@	10.40	.	23'-6"	=	494.0
do	2Ls	3½ . 3½ . 3/8	@	8.50	.	3'-6"	=	59.4
Stiffs	4Ls	3 . 3 . 3/8	@	7.20	.	1'-3 3/8	=	36.8
fills	2Pls	6 . 3/8	@	7.65	.	0'-9"	=	11.30
End stiffs	2Ls	3½ . 3½ . 3/8	@	8.5	.	1'-9 3/4	=	30.8
fills	2Pls	3½ . 3/8	@	4.46	.	1'-3½	=	11.5
								1913.8
		Rivet Heads → variation 3½%						67.0
								1980.8

8 Stringers RS1 A1B1C1 @ 1980.8 = 15846.4#

Stringers AS1

Beam	1I	20"	@	65.4#	.	26'-0"	=	1700.0
Stiffs	4Ls	3 . 3 . 5/16	@	6.1	.	1'-7"	=	38.2
End Conn	2Ls	4 . 4 . 3/8	@	9.8	.	1'-5"	=	28.4
								17666
		Rivet heads + variations 2%						35.3
								1801.9#

1½ Stringers AS1 @ 1801.9 = 21620.#

Stringers SS1 + SS1E

Beam	1I	15"	@	42.9	.	24'-0"	=	1029.6
"	1I	15"	@	42.9	.	2'-0"	=	85.8
End Conn	4Ls	4 . 4 . 3/8	@	9.8	.	1'-1"	=	42.5
" "	2Ls	4 . 4 . 3/8	@	9.8	.	1'-1"	=	21.3
splice pl.	1Pl.	6 . ½	@	10.2	.	2'-0"	=	20.4
								1199.6
		Rivet heads + variation			2%			24.6
								1224.2#

4 Stringers SS1 @ 1224.2 = 4896.8#

Curb Stringer CS1 L1

web pl.	1Pl	24" . 5/16	@	25.50	.	23'-11"	=	610.0
flange	1L	4 . 4 . ½	@	12.80	.	26'-0"	=	332.8
flange	1L	3 . 3 . 5/16	@	6.10	.	23'-11"	=	306.2
"	1L	4 . 4 . ½	@	12.80	.	22'-6"	=	288.0
End Conn	1L	4 . 4 . 3/8	@	9.8	.	1'-6½	=	15.1
fill	1Pl.	4 . ½	@	6.80	.	1'-2½	=	8.2
Conn	2Ls	3½ . 3½ . 3/8	@	8.5	.	1'-2½	=	20.6
flange	1L	4 . 4 . 3/8	@	9.8	.	1'-11"	=	18.8
web	1Pl.	23 1/4 . 5/16	@	24.71	.	2'-0	=	49.4
End Conn	4Ls	4 . 4 . 3/8	@	9.8	.	1'-0"	=	39.2
	1L	3 . 3 . 5/16	@	6.1	.	1'-11"	=	11.7
End Conn	2Ls	3½ . 3½ . 3/8	@	8.5	.	1'-1½	=	32.6
								1732.6
		River heads + variation say						53.4
								1786.0

4 stringers CS1 L1 @ 1786.0 = 7144.0#

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CALCULATIONS FOR

List of Materials for Chorokubashi for Kumamoto-Ken Revised 14-4-9

Fascia Stringers FS1. $\frac{R}{L}$

web	1Pl	22 × 9/16	Sketch pl.	×	2.6'-0"	=	490.6
flange	1L	3½ × 3½ × 3/8	@ 8.5	×	26'-0"	=	221.1
"	1L	3½ × 3½ × 3/8	@ 8.5	×	26'-3	=	225.3
							937.0 #
		Rivet Heads and Variation			3.5%		32.8
							969.8 #

4 Fascia Stringers @ 969.8 = 3879.2 #

Cy Stringers RSz. A & B $\frac{R}{L}$

web	1Pl	22 × 3/8	@ 2805	×	24'-0"	=	6730
flange	2Ls	5 × 3½ × 3/8	@ 10.40	×	24'-0"	=	499.2
"	2Ls	5 × 3½ × 3/8	@ 10.40	×	24'-0"	=	499.2
End conn	2Ls	3½ × 3½ × 3/8	@ 8.5	×	1'-9¾"	=	30.8
do	2Ls	3½ × 3½ × 3/8	@ 8.5	×	1'-9¾"	=	30.8
fills	2Pls	3½ × 3/8	@ 4.46	×	1'-3½"	=	11.5
"	2Pls	3½ × 3/8	@ 4.46	×	1'-3½"	=	11.5
							1750.0
		Rivet heads + variation			3½%		68.0
							1818.0 #

8 Cy Stringers RSz @ 1818.0 # = 14544.0

Hy Stringers HSz

Beam	1I	20"	@ 65.4	×	24'-0"	=	1569.6
End conn	2Ls	4 × 4 × 3/8	@ 9.8	×	1'-5"	=	17.90
" "	2Ls	4 × 4 × 3/8	@ 9.8	×	1'-5"	=	17.9
							1605.4
		Rivet heads + variation			2%		32.1
							1637.5

12 Hy Stringers HSz @ 1637.5 = 19620.0 #

Sidewalk Stringers SSz

Beam	1I	15"	@ 42.9#	×	24'-0"	=	1029.6
End conn	2Ls	4 × 4 × 3/8	@ 9.8	×	1'-5"	=	21.4
" "	2Ls	4 × 4 × 3/8	@ 9.8	×	1'-5"	=	21.4
							1072.4
		Rivet heads + variation			2%		22.0
							1094.4

4 Sidewalk Stringers @ 1094.4 # = 4377.6 #

Curb Stringers CSz $\frac{R}{L}$

web	1Pl	24 × 9/16	@ 25.5	×	24'-0"	=	611.0
flange	1L	4 × 4 × ½	@ 12.8	×	24'-0	=	307.2
"	1L	3.3 × 9/16	@ 6.1	×	24'-0	=	146.2
"	1L	4 × 4 × ½	@ 12.8	×	24'-0	=	307.2
End conn	4Ls	3½ × 3½ × 3/8	@ 8.5	×	1'-1½	=	38.5
"	~~2Pls~~	~~4 × ½~~	~~@ 6.8~~	~~×~~	~~1'-2½~~	=	~~16.4~~
							1407.1
		Rivet heads + variation			3½%		49.6
							1456.7

4 Curb Stringers @ 1456.7 # = 5826.8 #

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CALCULATIONS FOR

List of materials for Chorokyu-Bashi

Fascia stringers

web	1Pl.	22 . 5/16	@	sketch	.	24-0	=	435.1
flange	1L	3½ . 3½ . 3/8	@	8.5	.	24-0"	=	204.0
flange	1L	3½ . 3½ . 3/8	@	8.5	.	24-2"	=	205.4
								844.5
		Rivet heads + variation				3.5%		29.5
								873.0

4 fascia stringers @ 873.0 = 3492#

Summary for stringers

	Panel I	Panel II
Ry stringers	158464	14544
Ay stringers	21620	19620
Sidewalk stringers	4896.8	4377.6
Curb stringers	7144.0	5826.8
Fascia stringers	3879.2	3492.
	533864#	478604#

Note 1- Materials for stringers for panels 3-4-5 same as for panel 2.

Grand Total for stringers

Panel I 53386.4

Panels 2-3-4-5 4@ 47860.4# = 191441.6

244,828.0# or 109.298 tons

End Floor Beam FB1A and FB1B

web	2Pls	36", 5/16	@	38.25	.	17-2 7/8	=	1318.8
"	1Pl.	36 . 5/16	@	38.25	.	14-8 5/8	=	563.0
flanges	4Ls	3½ . 3½ . 3/8	@	8.5	.	17-2 7/8	=	586.0
"	4Ls	3½ . 3½ . 3/8	@	8.5	.	16-6 3/8	=	562.0
"	4Ls	3½ . 3½ . 3/8	@	8.5	.	14-8 5/8	=	502.0
End conn	4Ls	6 . 4 . 3/8	@	12.8	.	1-10 5/8	=	93.0
End Ls	4Ls	3½ . 3½ . 3/8	@	8.5	.	1-4 3/4	=	47.1
fills	4Pls.	3½ . 3/8	@	4.46	.	1-1½	=	20.2
Stiff. bracket	2Pls.	23¼ . 5/16	@	24.71	.	2-5"	=	119.6
Backer	4Ls	3½ . 3½ . 3/8	@	8.5	.	1-9"	=	59.5
Stiffs	16Ls	3 . 3 . 5/16	@	6.1	.	3-0½	=	297.0
"	8Ls	3 . 3 . 5/16	@	6.1	.	2-11¾	=	145.6
"	8Ls	3 . 3 . 3/8	@	7.2	.	2-11¾	=	171.6
fills.	8Pls	3 . 3/8	@	3.83	.	2-5½	=	75.8
"	4Pls.	6 . 3/8	@	7.66	.	2-5¼	=	74.8
								4636.0

Bearing Pls and so forth for FB1A only (fixed End).

Sole pls	2Pls.	18" . ½	@	30.60	.	1-6"	=	91.8
Bed Pls	2Pls	18 . ½	@	30.60	.	1-6"	=	91.8
fills at stiffs	2Pls.	7/8 . 3/8	@	9.09	.	3-0"	=	54.5
Anchor	8 bolts	1"ø					=	39.2
								277.3#

Bearing pls etc for FB1B only (Roller End).

Bearing	4Pls.	3 . 3/8	@	3.73	.	1-6"	=	23.0
Sole Pl	2Pls.	12 . ½	@	20.40	.	1-1"	=	44.6
Bed Pls	2Pls	18 . ½	@	30.60	.	1-6"	=	91.8
brackets	2Pls.	27⅝ . 3/8	@	35.23	.	3-0"	=	211.4
"	4Ls	3½ . 3½ . 3/8	@	8.5	.	1-8"	=	56.8
Anchor	8 Bolts	1"ø						39.2
								366.8#

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CALCULATIONS FOR

Revised - 14-4-9

List of Materials for Chonokubashi

Summary for 2nd Floor Beam		FB1A	FB1B
girder		4636.0	4636.0
bearings		27.3	366.8
Rivet Heads + var.		169.0	169.0
	5082.3 # / Rgd	5171.8 # / Rgd

Weight of 2nd Cantilever Brackets CB1

web.	1PL.	52" × 5/16	sketch	×	9'-2"	=	316.0
flange	2Ls	3½ × 3½ × 5/16	@ 7.2	×	9'-2 3/8	=	132.0
"	2Ls	3½ × 3½ × 5/16	@ 7.2	×	9'-8"	=	139.0
stiffs	2Ls	3 × 3 × 5/16	@ 6.1	×	1'-10"	=	22.2
fills	2Pls	3 × 5/16	@ 3.19	×	1'-3"	=	8.0
stiffs	2Ls	3 × 3 × 5/16	@ 6.1	×	2'-11"	=	35.6
fills	2Pls.	6 5/8 × 5/16	@ 6.93	×	0'-9"	=	10.4
"	2Pls	3 × 5/16	@ 3.19	×	1'-6"	=	9.6
End Conn	2Ls	4 × 4 × 3/8	@ 9.8	×	3'-9 7/8	=	74.9
fills	2Pls.	4 × 5/16	@ 4.25"	×	3'-3 5/8	=	28.1
around hole	2Ls	2½ × 2½ × ¼	@ 4.1	×	3'-8"	=	30.2
" "	2Ls	2½ × 2½ × ¼	@ 4.1	×	5'-3"	=	43.1
Conn	1L	4 × 3 × 3/8	@ 8.5	×	1'-6"	=	12.8
"	1PL	11 × 3/8	@ 14.03	×	1'-6"	=	21.0
"	1L	4 × 4 × 3/8	@ 9.8	×	0'-6"	=	4.9
							888.1
			Rivet heads + variation say				30.6
							918.7 #

4 Cantilever Brackets CB1 @ 918.7 = 3674.8 #

Intermediate Floor Beams FB2 and FB3

web.	2Pls.	60 × 3/8	@ 76.5	×	23'-2 4/16	=	3554.2
"	2Pls	21½ × 3/8	@ 27.41	×	5'-10¼	=	320.7
Reinf Pls	4Pls.	48¼ × 5/8	@ 102.53	×	5'-2½	=	2136.7
Conn. c	8Ls	6 × 4 × 5/8	@ 20.0	×	7'-4"	=	1172.8
b	4Pls	17¾ × 5/8	@ 37.72	×	2'-9¾	=	424.0
g	8Ls	6 × 4 × 5/8	@ 20.0	×	2'-3½	=	366.4
r	8Ls	6 × 4 × 5/8	@ 20.0	×	2'-0"	=	320.0
h	4Ls	6 × 4 × 5/8	@ 20.0	×	1'-5¾	=	118.4
a	2Pls.	16 × 3/8	@ 20.4	×	8'-4½	=	341.9
c	8Pls	4 × 5/8	@ 8.5	×	2'-0¼	=	137.4
f	8Pls.	5¼ × ⅜	@ 2.23	×	1'-9"	=	31.2
p	4Pls.	3½ × ⅜	@ 1.49	×	0'-9"	=	4.5
Bott. flange	4Ls	6 × 6 × 3/4	@ 28.7	×	26'-4 7/8	=	3031.9
Top flange	4Ls	6 × 6 × 3/4	@ 28.7	×	24'-4"	=	2793.1
Cov Pl.	4Pls.	14 × 5/8	@ 29.75	×	18'-7¾	=	2219.4
"	4Pls	14 × 5/8	@ 29.75	×	12'-2"	=	1448.2
flange opl.	2Pls.	14 × 5/8	@ 29.75	×	8'-7"	=	495.0
"	2Pls.	14 × 5/8	@ 29.75	×	4'-2"	=	247.5
splice	4Ls	6 × 6 × 3/4	@ 28.7	×	4'-2"	=	479.0
web opl.	4Pls.	6 × 3/4	@ 15.3	×	3'-0"	=	183.5
"	2Pls.	12 × 3/8	@ 15.3	×	1'-8½	=	52.3
"	2Pls.	11½ × 3/8	@ 14.66	×	1'-0"	=	19.3
Stiffs	20Ls	5 × 3½ × 3/8	@ 10.4	×	4'-11"	=	1023.4
fills	2Pls	6½ × ½	@ 11.05	×	1'-8"	=	37.1
"	12Pls.	8 × 3/4	@ 20.40	×	1'-0"	=	244.4
"	8Pls.	8½ × 3/4	@ 19.87	×	1'-7"	=	250.8
"	8Pls.	3½ × 3/4	@ 8.93	×	2'-4½	=	168.4
"	12Pls.	3½ × 3/4	@ 8.93	×	2'-11"	=	593.2
							22214.7 #

web splice for FB2 only 2Pls 12 × 3/8 @ 15.3 × 2'-7" = 79.0 #
FB3 only 2Pls. 12 × 3/8 @ 15.3 × 3'-0⅛ = 92.0 #

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CALCULATIONS FOR

List of Materials for Choroku-bashi.

Weight of Intermediate Floor Beams

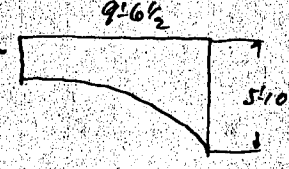
FB2	22214.7	FB3	22214.7
splice	79.0	splice	92.0
Rivet heads	754.0	Rivet heads etc	755.0
	23047.7		23061.7

Summary for Floor Beams-

5	Floor Beams	FB2	@	23047.7	= 115238.5
4	Floor Beams	FB3	@	23061.7	= 92246.8
					207485.3

Intermediate Cantilever Bracket CB2

web.	1Pl.	70. 3/8"	sketch		9'-6 1/2"	= 559.5
flange	2Ls	3 1/2. 3 1/2. 5/16	@	7.2	9'-2 1/2"	= 132.6
"	2Ls	3 1/2. 3 1/2. 5/16	@	7.2	10'-9"	= 154.8
stiffe	2Ls	3. 3. 5/16	@	6.1	1'-10"	= 22.3
fille	2Pls	3. 5/16	@	3.19	1'-3"	= 8.0
stiffe	2Ls	3. 3. 5/16	@	6.1	3'-3 1/2"	= 40.8
fille	2Pls	6 5/8". 5/16	@	6.93	0'-9"	= 10.4
"	2Pls	3. 5/16	@	3.19	1'-11"	= 12.3
cone	2Ls	6.4. 5/8	@	20.0	1'-5 1/2"	= 58.4
Conduit	2Ls	2 1/2. 2 1/2. 1/4	@	4.1	3'-8"	= 30.2
Pipe	2Ls	2 1/2. 2 1/2. 1/4	@	4.1	5'-3"	= 43.1



1072.4
Rivet Heads + variation 28.2
1100.6#

18 intermediate Brackets @ 1100.6 = 19810.8#

Summary for Floor Beams and Cantilever Brackets

End Floor Beams	FB1A			5082.3
" " "	FB1B			5171.8
Cantilever CB.	4			36748
Int. Floor Beams	FB2	5 @ 23047.7	=	115238.5
	FB3	4 @ 23061.7	=	92246.8
Cantilever Brackets	CB2	18 @ 1100.6	=	19810.8
				241225.0# or 107.690 tons

Weight of bottom lateral bracing

Bracing BB1

main section	2Ls	6. 6. 3/8	@	14.9	32'-6 7/8"	= 968.00
stringer conn A.	2Ls	5. 3 1/2. 3/8	@	10.4	0'-11"	= 19.10 } 64.80
" " B	2Ls	5. 3 1/2. 3/8	@	10.4	0'-11"	= 19.10 }
" " "	2Pls	9 1/4. 3/8	@	11.79	11'-3 1/4"	= 26.60 }
						1032.8
		Rivet heads + variation 2%				20.7
						1053.5#

Bracing BB2

main section	2Ls	6. 6. 3/8	@	14.9	32'-6 7/16"	= 968.00
stringer conn						64.80
						1032.8
		Rivet heads + variation				20.7
						1053.5#

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CALCULATIONS FOR

List of materials for chorokutashi

Bracing BB3 main section Stringer comm.	2Ls 6.6.3/8	@ 14.9	. 32'-6 1/32"	=	967.00	
					64.80	
					1031.8	
	Rivet heads and variation 2%				20.6	
						1052.4 #
Bracing BB4 main section Stringer comm	2Ls 6.6.3/8	@ 14.9#	. 32'-6"	=	967.00	
					64.80	
					1031.80	
	Rivet heads + variation 2%				20.60	
						1052.4 #
Connection Plates and angles						
Lo	2Pls. 33.3/8"	@ 42.08	. 2'-9 1/2"	=	235.0	
" for truss	2Ls 6.6.1/2	@ 19.60	. 3'-4"	=	130.5	
" " fill	2Pls. 6.3/4	@ 15.3	. 1'-1 1/2"	=	34.6	
" "	2fills 6.1/2	@ 10.2	. 1'-1 1/2"	=	23.0	
" "	2fills 5 1/2.3/8	@ 7.01	. 1'-1 1/2"	=	15.8	
" "	2 fills 6.3/8	@ 7.65	. 2'-0"	=	30.6	
Lo	2Ls 6.6.3/8	@ 14.9	. 2'-6"	=	74.5	
T4	2Plo 27.3/8	@ 34.43	. 4'-5"	=	305.0	
"	4Ls 6.6.3/8	@ 14.9	. 1'-6"	=	89.4	
"	2Pls 25".3/8	@ 31.88	. 4'-5"	=	282.2	
T8	4Plo 25".3/8	@ 31.88	. 4'-3 1/2"	=	547.0	
"	4Ls 6.6.3/8	@ 14.9	. 1'-6"	=	89.4	
B2	2Plo 21 1/2.3/8	@ 27.41	. 3'-8 1/2"	=	203.5	
"	2Ls 4.4.3/8	@ 9.80	. 3'-8 1/2"	=	72.8	
B6	2Pls. 20 1/2.3/8	@ 26.14	. 3'-6"	=	186.0	
"	2Ls 4.4.3/8	@ 9.80	. 3'-6"	=	68.6	
B10	1Pl. 20 1/2.3/8	@ 26.14	. 3'-6"	=	93.0	
"	1L 4.4.3/8	@ 9.80	. 3'-6"	=	34.3	
					2515.2	
	Rivet heads + variation say				46.8	
						2562.0 #

Summary for weight of bottom lateral Bracings.

BB1	4	@	1053.5	=	4214.0
BB2	4	@	1053.5	=	4214.0
BB3	4	@	1052.4	=	4210.0
BB4	8	@	1052.4	=	8419.2
comm. Pls+Ls	2	@	2562.0	=	5124.0
					26181.2 # or 11.688 tons.

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CALCULATIONS FOR

List of Materials for Chorofusubashi

Summary for Bottom Lateral Bracings and Connections.

2	BB1R	@ 1455.4	=	2910.8
2	BB1L	@ 1455.4	=	2910.8
2	BB2R	@ 1309.3	=	2618.6
2	BB2L	@ 1309.3	=	2618.6
2	BB3R	@ 1309.3	=	2618.6
2	BB3L	@ 1309.3	=	2618.6
4	BB4R	@ 1309.3	=	5237.2
4	BB4L	@ 1309.3	=	5237.2
Connection Pls + Ls			=	5130.5

31900.9# or 14241 tons.

Weight of materials for Top Lateral Bracings.

4	TB1	2Ls 6×4"×⅜	@ 12.3#	× 31'-7½"	=	3112.0
4	TB2	2Ls 6×4×⅜	@ 12.3	× 32'-2½	=	3175.0
4	TB3	2Ls 6×4×⅜	@ 12.3	× 31'-10"	=	3140.0
4	TB4	2Ls 6×4×⅜	@ 12.3	× 31'-11"	=	3142.0
4	TB5	2Ls 6×4×⅜	@ 12.3	× 31'-9"	=	3121.0
						15690.0#
5	gusset pls	39×⅜	@ 49.73	× 3'-9"	=	933.0
2	" "	44×⅜	@ 56.10	× 3'-10"	=	430.0
2	" "	42×⅜	@ 53.55	× 3'-6"	=	375.0
						1738.0
			Total			17428.0#

Weight of materials for Sway Bracings.

upper flange	4Ls	6×6×⅜	@ 14.9	× 24'-4"	=	1447.0	
bottom flange	4Ls	6×6×⅜	@ 14.9	× 25'-2"	=	1498.0	
Diagonals	4Ls	4×3"×5/16	@ 7.2	× 6'-2¼	=	178.0	
"	4Ls	4×3×5/16	@ 7.2	× 5'-11¼	=	171.0	
"	4Ls	4×3×5/16	@ 7.2	× 5'-4½	=	154.4	
"	16Ls	4×3×5/16	@ 7.2	× 5'-4"	=	614.0	
Center Conn L	4Ls	3½×3½×5/16	@ 7.2	× 5'-4"	=	153.3	
" fill	1Pl	7⅜×⅜	@ 9.5	× 3'-1¼"	=	29.4	
" "	2Pls	7⅜×⅜	@ 9.5	× 4'-10¼	=	92.0	
Splice pl.	1Pl.	12½×⅜	@ 15.94	× 3'-0"	=	47.9	
fills	2Pls.	3½×⅜	@ 4.46	× 1'-6"	=	13.4	
End Conn.	4Ls	6×4×⅜	@ 12.3	× 1'-8"	=	82.0	
Gusset pls.	2Pls	18"×⅜	@ 22.95	× 2'-4"	=	107.2	
	2Pls.	23×⅜	@ 29.33	× 2'-3"	=	132.0	
	10Pls	20×⅜	@ 25.50	× 2'-0"	=	510.0	
	1Pl.	16×⅜	@ 20.6	× 1'-5"	=	28.9	
	1Pl.	19"×⅜	@ 24.23	× 2'-2"	=	52.6	
						5311.1	Common to all Sway bracings.
Sway Bracing at Ill0. only							
gusset pls.	2Pls.	19×⅜	@ 24.23	× 2'-0"	=	96.9	
Connection	4Ls	6×4×⅜	@ 12.3	× 2'-0"	=	98.5	
Fills.	4Pls.	6×⅜	@ 7.65	× 1'-4"	=	40.8	
"	2Pls.	12½×¾	@ 31.9	× 1'-0"	=	63.8	
						300.0	

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CALCULATIONS FOR

List of materials for Chorokubashi

Sway bracing at U_8 only

Gusset pls	2Pls.	18 × 3/8	@	22.95	× 2'-6½"	=	116.7
Connectin	4Ls	6.4 × 3/8	@	12.3	× 2'-6½"	=	127.0
fills	4Pls.	6 × 3/8	@	7.65	× 1'-2"	=	35.4
"	2Pls.	(9) 3/4	@	22.95	× 0'-10"	=	38.2
							317.3

Sway Bracing at U_6 only

Gusset pl.	2Pls	18 × 3/8	@	22.95	× 1'-9"	=	80.5
Connectin	4Ls	6 × 4 × 3/8	@	12.30	× 1'-9"	=	86.1
fill	4Pls	6 × 3/8	@	7.65	× 1'-2"	=	35.7
"	2Pls.	12½ × 3/4	@	31.9	× 1'-3½"	=	84.6
							286.9 #

Sway bracing at U_4 and U_2 only

Gusset pl	2Pls.	18 × 3/8	@	22.95	× 1'-9"	=	80.5
Conn	4Ls	6 × 4 × 3/8	@	12.30	× 1'-9"	=	86.1
Fills	4Pls	6 × 3/8	@	7.65	× 1'-2"	=	35.7
							202.3 #

Weight of Sway Bracing.

U_{10}	U_8	U_6	U_4	U_2	
5311.1	5311.1	5311.1	5311.1	5311.1	Common to all
300.0	317.3	286.9	202.3	202.3	Special
196.2	197.0	196.0	194.0	194.0	Rivet heads + variation
5807.3	5825.4	5794.0	5707.4	5707.4	

Summary of Sway Bracings

U_{10}			5807.3
U_8	2 @ 5825.4	=	116508
U_6	2 @ 5794.0	=	11588.0
U_4	2 @ 5707.4	=	11414.8
U_2	2 @ 5707.4	=	11414.8
			51875.7 #

Weight of Portal Bracing

web.	4Pls.	80" × 9/16	@	16'-10⅜"	@ 1090.0 net	=	4036.0
"	2 Pls.	54 × 9/16	@	57.4	× 16'-0"	=	1836.0
flange	4Ls	6 × 4 × 3/8	@	12.3	× 16'-6 3/8	=	816.0
"	2Ls	6 × 4 × 3/8	@	12.3	× 16'-0"	=	393.8
"	4Ls	6 × 4 × 3/8	@	12.3	× 17'-0"	=	836.0
"	2Ls	6 × 4 × 3/8	@	12.3	× 16'-0"	=	3938
cov. Pl.	2Pls.	18 × 5/16	@	19.13	× 17'-6 15/16	=	670.0
"	2Pls.	18 × 5/16	@	19.13	× 13'-5½"	=	256.5
fill	2Pls	18 × 5/16	@	19.13	× 2'-7"	=	99.0
splice	2Pls.	18 × 3/8 5/16	@	19.13	× 4'-1"	=	156.2
Gusset pl.	1Pl.	36 × 3/8	@	45.9	× 5'-6"	=	252.3
splice	2Pls.	18 × 3/8	@	22.95	× 2'-6"	=	114.8
End Conn	4Ls.	3 × 3 × 3/8	@	7.2	× 6'-0"	=	172.7
Stiffs	8Ls	3½ × 3½ × 5/16	@	7.2	× 5'-0"	=	288.0
"	32Ls	do			× 41'-5¾"	=	1032.0
fills.	4Pls.	7 × 3/8	@	8.93	× 4'-0"	=	143.0
"	16 Pls.	7 × 3/8	@	8.93	× 3'-6½"	=	560
Deaph	web 5Pls.	9 × 3/8	@	11.48	× 4'-6"	=	658.0
	20Ls	3½ × 3½ × 5/16	@	7.2	× 4'-6"	=	648.0
							12908.1
							460.0
							13368.1 #

Rivet heads + variation say

2 End Portals @ 13368.1 = 26736.2 #

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CALCULATIONS FOR

Design of Chorotsubashi - List of materials

Weight of Longitudinal Struts.

Longitudinal Strut LS1								
Top Flange	2Ls	5 × 3½ × 5/16	@	8.7	×	23'-1"	=	403.0
Bottom flange	2Ls	5 × 3½ × 5/16	@	8.7	×	23'-5 3/8	=	410.0
Connection Pl	1Pl	23" × 3/8"	@	29.33	×	3'-10"	=	112.2 less ent 39.3 = 72.9
" "	1Pl	30" × 3/8"	@	38.25	×	4'-2"	=	159.5 " " 56.0 = 103.5
Conn. Ls	2Ls	3½ × 3½ × 5/16	@	7.2	×	4'-5 3/4"	=	64.4
"	1 Fill.	7 3/8 × 3/8	@	9.5	×	3'-6½	=	33.6
fill.	1Pl	3½ × 3/8	@	4.46	×	0'-10½	=	3.9
Lacing bars	24 bars	2½ × 3/8	@	3.19	×	2'-6"	=	191.0
								1282.3 net
		Rivet Heads + Variation				3.5%		45.0
								1327.3 #
Longitudinal Strut LS2								
Top flange	2Ls	5" × 3½ × 5/16	@	8.7	×	23'-3"	=	406.0
bott. flange	2Ls	5 × 3½ × 5/16	@	8.7	×	23'-9"	=	413.0
Connection	1Pl.	27 × 3/8	@	34.43	×	4'-9"	=	167.2 - 58.6 = 108.6 net
"	1Pl	30" × 3/8	@	38.25	×	4'-2"	=	159.3 - 55.8 = 103.5 net
Lacing bars	24 bars	2½ × 3/8	@	3.19	×	2'-6"	=	190.8
								1221.9 net
		Rivet Heads + variation						42.5
								1264.4 #
Longitudinal Strut LS3								
Top flange	2Ls	5 × 3½ × 5/16	@	8.7	×	23'-3"	=	405.0
Bott. flange	2Ls	5 × 3½ × 5/16	@	8.7	×	23'-3"	=	405.0
Conn	1Pl.	27 × 3/8	@	34.43	×	14'-9"	=	167.2 - 58.6 = 108.6 net
"	1Pl	25 × 3/8	@	31.88	×	4'-4"	=	138.0 - 48.3 = 89.7 "
Lacing bars	24 bars	2½ × 3/8	@	3.19	×	2'-6"	=	190.8
								1199.1 net
		Rivet Heads + Variation				3.5%		42.0
								1241.1
Longitudinal Strut LS4								
Top flange	2Ls	5 × 3½ × 5/16	@	8.7	×	23'-2"	=	402.5
Bott. flange	2Ls	5 × 3½ × 5/16	@	8.7	×	23'-2"	=	402.5
Conn	1Pl	27 × 3/8	@	34.43	×	4'-8"	=	161.0 - 56.4 = 104.6 net
"	1Pl	23 × 3/8	@	29.33	×	4'-5"	=	129.6 - 45.1 = 84.5 net
Lacing Bars	24	2½ × 3/8	@	3.19	×	2'-6"	=	190.8
								1184.9 net
		Rivet Heads + Variation				3.5%		41.4
								1226.3 #
Longitudinal Strut LS5								
Top flange	2Ls	5 × 3½ × 5/16	@	8.7	×	22'-11"	=	398.0
Bott flange	2Ls	5 × 3½ × 5/16	@	8.7	×	22'-11"	=	398.0
Conn	1Pl	25 × 3/8	@	31.88	×	4'-6"	=	143.6 - 50.3 = 93.3 net
"	1Pl	27 × 3/8	@	34.33	×	4'-5"	=	152.6 - 53.3 = 99.3 net
Lacing Bars	24	2½ × 3/8	@	3.19	×	2'-6"	=	190.8
								1179.4
		Rivet Heads + variation						41.2
								1220.6 #

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CALCULATIONS FOR

List of materials for Chorokubashi.

Summary for Longitudinal Bracing (Strut).

Longitudinal Bracing	LS1	2@	1327.3	=	2654.6
	LS2	2@	1264.4	=	2528.8
	LS3	2@	1241.1	=	2482.2
	LS4	2@	1226.3	=	2452.6
	LS5	2@	1220.6	=	2441.2
					12559.4 #

Summary for top Lateral System.

Top Lateral Bracing	17428.0 #
Sway Bracing	51875.0
End Portal Bracing	26736.0
Longitudinal Struts	12559.0
	108598.0 # ÷ 240 = 452 # per lin ft.
	48.480 tons

Weights of materials for main truss.

Upper chord U_0-U_1-U_2 main section

cov.	1Pl	33 × 3/8	@	42.08	×	24'-0 1/2"	= 1012.0
web.	2Pls.	27 × 5/8	@	57.38	×	24'-10 1/4"	= 2853.0
flange	2Ls	6 × 6 × 3/8	@	14.90	×	24'-11 1/2	= 743.0
"	2Ls	6 × 6 × 3/8	@	14.90	×	24'-11 1/2	= 743.0
							5351.0 #

U_2-U_3-U_4 main section

cov.	1Pl.	33 × 3/8	@	42.08	×	24'-6 3/4	= 1033.0
web	2Pls.	27 × 5/8	@	57.38	×	24'-11 3/4	= 2862.0
flange	2Ls	6 × 6 × 1/2	@	19.6	×	25'-0 3/4	= 984.0
flange	2Ls	6 × 6 × 1/2	@	19.6	×	24'-10 3/4	= 976.0
							5855.0

U_4-U_5-U_6 Main Section

cov.	1Pl	33 × 5/8	@	70.13	×	24'-4 1/2	= 1708.0
web.	2Pls.	27 × 3/4	@	68.9	×	24'-11"	= 3430.0
flange	2Ls	6 × 6 × 1/2	@	19.6	×	12'-8 1/2	= 498.0
"	2Ls	6 × 6 × 1/2	@	19.6	×	12'-7 1/2	= 495.0
"	2Ls	6 × 6 × 5/8	@	24.2	×	12'-1"	= 584.0
"	2Ls	6 × 6 × 5/8	@	24.2	×	12'-0 1/2	= 582.0
SidePls	2Pls	15 × 5/8	@	31.88	×	15'-7 1/2"	= 996.0
							8293.0

U_6-U_7-U_8 main section

cov.	1Pl	33 × 5/8	@	70.13	×	12'-1"	= 847.0
"	1Pl	33 × 5/8	@	70.13	×	24'-3"	= 1700.0
web	2Pls	27 × 3/4	@	68.9	×	23'-7 7/8	= 3248.0
flange	2Ls	6 × 6 × 5/8	@	24.2	×	25'-2"	= 1218.0
"	2Ls	6 × 6 × 5/8	@	24.2	×	25'-1"	= 1213.0
splice	2Pls	15 × 5/8	@	31.88	×	25'-1 1/8	= 1600.0
"	2Pls	25 × 3/8	@	31.88	×	27'-1 1/8	= 1729.0
							11555.0

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CALCULATIONS FOR

Design of Shorokubashi for Kumamoto-Ken. Revised 14-4-9

Upper chord U8 U9 - U10 Main section -

Cov. Pl.	1Pl	33" × 5/8	@	70.13	×	21L7"	=	1522.0
"	1Pl	33 × 5/8	@	70.13	×	24L1"	=	1690.0
Web	2Pl6	27 × 3/4	@	68.9	×	24L08	=	3310.0
flange	2L5	6 × 6 × 3/4	@	28.7	×	21L1"	=	1210.0
"	2L5	6 × 6 × 3/4	@	28.7	×	21L1"	=	1205.0
Side Pl.	2Pl3	15 × 3/4	@	38.3	×	21L08	=	1615.0
"	2Pl6	25 × 3/8	@	31.88	×	12L0"	=	766.0
"	2Pl6	25 × 1/2	@	42.5	×	8'-10"	=	750.6

12.0686#

Summary for top chords for one span

U0-U1-U2	53.51.0
U2-U3-U4	58530
U4-U5-U6	8293.0
U6-U7-U8	115550
U8 U9-U10	120686
	43120.6# × 4 = 172482.4

Lower chord L0-L1-L2 main section

web	2Pl6	27 × 3/4	@	68.9	×	4'-11½"	=	683.0
"	2Pl6	27 × 3/4	@	68.9	×	23'-7"	=	3185.0
flange	2L5	6 × 6 × 3/4	@	28.7	×	24'-9¼	=	1440.0
"	2L5	6" × 6 × 3/4	@	28.7	×	24'-6"	=	1412.0
Side Pl6	2Pl6	15 × 3/4	@	3.83	×	24'-6"	=	1893.0
	2Pl6	25 × 1/2	@	42.5	×	9'-6"	=	807.5
	2Pl6	25 × 3/4	@	638	×	17'-10¼	=	2280.0
	2Pl6	25 × 3/8	@	31.88	×	20'-4"	=	1298.0

13013.5#

Lower chord L2-L3-L4

web	2Pl6	27 × 3/4	@	68.9	×	26'-0½"	=	3588.3
flange	2L5	6 × 6 × 3/4	@	28.7	×	26'-1½"	=	1499.9
"	2L5	6 × 6 × 3/4"	@	28.7	×	25'-11½"	=	1490.6
Side Pl6	2Pl6	15" × 3/4	@	38.3	×	26'-0½"	=	1994.7
"	2Pl6	25" × 3/4	@	638	×	12'-8⅛"	=	1618.0
"	2Pl6	25" × 1/2	@	42.5	×	13'-4½"	=	1137.3

11328.3#

Lower chord L4-L5-L6

web	2Pl6	27" × 3/4	@	68.9	×	24'-11 3/4	=	3442.2
flange	2L5	6 × 6 × 3/4	@	28.7	×	25'-0 3/4	=	1439.0
"	2L5	6 × 6 × 3/4	@	28.7	×	24'-0 3/4	=	1430.0
Side Pl6	2Pl6	15" × 3/8	@	19.13	×	25'-5 3/4	=	975.0
Side Pl6	2Pl6	15" × 3/8	@	19.13	×	12'-11 3/8"	=	496.0

7782.2#

Lower chord L6-L7-L8

web	2Pl6	27 × 5/8	@	57.3	×	24'-1 5/8	=	2768.0
flange	2L5	6 × 6 × 5/8	@	24.2	×	13'-2"	=	637.0
"	2L5	6 × 6 × 5/8	@	24.2	×	13'-1"	=	633.0
	2L5	6 × 6 × 3/8	@	14.9	×	11'-11¼	=	356.0
	2L5	6 × 6 × 3/8	@	14.9	×	11'-10¼	=	353.0

4747.0#

Lower chord L8-L9-L10

web	2Pl6	27 × 5/8	@	57.3	×	23'-3"	=	2668.0
flange	2L5	6 × 6 × 3/8	@	14.9	×	22'-8½"	=	676.0
"	2L5	6 × 6 × 3/8	@	14.9	×	22'-7½"	=	675.0

4019.0#

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CALCULATIONS FOR

Lisp materials for chorokubashi Revised 14-4-9

Summary for lower Chord for one span

L0-L1-L2	129985
L2-L3-L4	113283
L4-L5-L6	77822
L6-L7-L8	47470
L8-L9-L10	4019.0
	40875.0 × 4 = 163500.0# for one span

Ties :-

Tie L0-T2 main section

web	2Pls	24 × 3/4	@ 61.2	"	16'-8" =	2040.0
"	2Pls	24" × 3/4	@ 61.2	"	18'-5" =	2256.0
"	2Pls	24" × 3/4	@ 61.2	"	17'-10" =	2182.0
Side Pls	2Pls	18 × 1/2	@ 30.6	"	17'-6 1/2 =	1073.0
flange	2Ls	6 × 6 × 1/2	@ 19.6	"	17'-6 1/2 =	688.0
						8239.0

Tie T2-T4 main section

web	4Pls	24 × 3/4	@ 61.2	"	24'-0" =	5870.0
"	2Pls	24 × 3/4	@ 61.2	"	20'-0" =	2443.0
Side Pls	2Pls	18 × 1/2	@ 30.6	"	17'-0" =	1040.0
flange	2Ls	6 × 6 × 1/2	@ 19.6	"	17'-0" =	666.0
						10019.0#

Ties T4-T6 ; T6-T8 and T8-T10 same as for T2-T4.

Summary for Ties for one span

L0-T2	8239.0
T2-T4	10019.0
T4-T6	10019.0
T6-T8	10019.0
T8-T10	10019.0
	48315.0 × 4 = 193260# for one span

Verticals

M0-L0 main section

Side pls	2Pls	18 × 1/2	@ 30.60	"	29'-7 1/2 =	1809.2
web.	1Pl	16 1/2 × 3/8	@ 21.04	"	29'-7 1/2 =	6234
diaphragm	4Ls	4 × 4 × 3/8	@ 9.8	"	29'-7 1/2 =	2228.2
Flange	4Ls	4 × 4 × 1/2	@ 12.8	"	29'-7 1/2 =	1517.1
						6182.9

M1-L1 main section

flange	2Ls	6 × 4 × 1/2	@ 16.2	"	23'-10 1/2 =	774.0
"	2Ls	6 × 4 × 1/2	@ 16.2	"	23'-10 1/2 =	774.0
cov.	2Pls	12 1/2 × 3/4	@ 31.9	"	16'-10" =	1075.0
						26230

M2-L2 main section

flange	2Ls	6 × 4 × 3/4	@ 23.6	"	19'-0 11/16 =	900
"	2Ls	6 × 4 × 3/4	@ 23.6	"	19'-0 11/16 =	900
cov.	2Pls	12 1/2 × 3/4	@ 31.9	"	13'-4" =	840
						2640.0

M3-L3 main section

flange	2Ls	6 × 4 × 3/4	@ 23.6	"	17'-10 3/8 =	843
"	2Ls	6 × 4 × 3/4	@ 23.6	"	17'-10 3/8 =	843
						1686.0

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CALCULATIONS FOR

List of materials for Choiokubashi

Verticals -

U_0-L_0	main section								
	flange	2 L's	6×4×½	@	16.2	×	14'-2"	=	459.6
	"	2 L's	6×4×½	@	16.2	×	14'-2"	=	459.6
	cover	2 Pl's	12½×¾	@	31.9	×	8'-7"	=	547.0
									1466.2
U_5-L_5	main section								
	flange	2 L's	6×4×¾	@	23.6	×	13'-9½"	=	650.0
	"	2 L's	6×4×¾	@	23.6	×	13'-9½"	=	650.0
									1300.0
U_6-L_6	main section								
	flange	4 L's	6×4×⅜	@	12.3	×	11'-3¼"	=	553.0
	cover pl	2 Pl's	12½×¾	@	31.9	×	5'-4½"	=	342.0
									895.0
U_7-L_7	main section								
	flange	4 L's	6×4×⅝	@	20.0	×	11'-3½"	=	903.0
U_8-L_8	main section	4 L's	6×4×¾	@	23.6	×	9'-4"	=	880.0
U_9-L_9	" "	4 L's	6×4×½	@	16.2	×	10'-1¾"	=	657.0
U_{10}-L_{10}	" "	4 L's	6×4×⅜	@	12.3	×	8'-9"	=	430.0

Summary for verticals for one span (main section)

U_0-L_0	6682.0
U_1-L_1	2623.0
U_2-L_2	2640.0
U_3-L_3	1686.0
U_4-L_4	1466.2
U_5-L_5	1300.0
U_6-L_6	895.0
U_7-L_7	903.0
U_8-L_8	880.0
U_9-L_9	657.0
½ U_{10}-L_{10}	215.0

19457.6 × 4 = 77808# for one span

Diagonals main section -

U_0-L_1	flange	2 L's	6×4×7/16	@	14.3	×	19'-8"	=	562.0
	"	2 L's	6×4×7/16	@	14.3	×	20'-0"	=	572.0
									1134.0
U_1-L_2	flange	4 L's	6×4×⅜	@	12.3	×	17'-4"	=	853.0
	cover	2 Pl's	12½×½	@	21.25	×	14'-6"	=	617.0
									1470.0
U_2-L_3	flange	2 L's	6×4×9/16	@	18.1	×	16'-3"	=	588.0
	"	2 L's	6×4×9/16	@	18.1	×	16'-0"	=	579.0
									1167.0
U_3-L_4	flange	4 L's	6×4×⅜	@	12.3	×	14'-1½"	=	695.0
	cow	2 Pl's	12½×¾	@	31.9	×	10'-3"	=	654.0
									1349.0
U_4-L_5	flange	4 L's	6×4×⅜	@	12.3	×	12'-9½"	=	628.0
	cover	2 Pl's	12½×½	@	21.25	×	9'-11"	=	422.0
									1050.0
U_5-L_6	flange	4 L's	6×4×½	@	16.2	×	11'-8⅞"	=	760.0
	cover	2 Pl's	12½×¾	@	31.9	×	8'-0"	=	510.0
									1270.0
U_6-L_7	flange	4 L's	6×4×⅜	@	12.3	×	11'-0"	=	542
	cover	2 Pl's	12½×½	@	21.25	×	8'-2"	=	347.2
									889.2

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List of materials for Chorokubashi

Diagonals (main section)

U7-L8	flange	4 L's	6×4×½	@	16.2	×	10'-11"	=	707.0
	cover	2 Pl's	12½×¾	@	31.9	×	7'-7"	=	483.0
									1190.0
U8-L9	flange	4 L's	6×4×⅝	@	20.0	×	11'-3"	=	900.0
U9-L10	flange	4 L's	6×4×7/16	@	14.3	×	10'-10"	=	619.0
	cov.	2 Pl's	12½×½	@	21.25	×	7'-5"	=	315.0
									934.0
				Total					113532

For one apon 4 × 113532 = 45412.8 #

Hangers (main section)

L2-T2	4 L's	6×4×⅝	@	20.0	×	11'-7 7/16	=	826.4
L4-T4	4 L's	6×4×⅝	@	20.0	×	21'-0 5/16	=	1682.4
L6-T6	4 L's	6×4×⅝	@	20.0	×	27'-0⅞	=	2165.6
L8-T8	4 L's	6×4×⅝	@	20.0	×	30'-11 5/32	=	2474.4
								7248.8
U10-T10	4 L's	6×4×⅝	@	20.0	×	32'-2"	=	2573.6

Summary for one apon

4 @	7248.8	=	28995.2
2 @	2573.6	=	5147.2
			34142.4 #

Summary for main section of truss.

Upper chords	172482.4
Lower chords	163500.0
Ties	193260.0
Verticals	77808.4
Diagonals	45412.8
Hangers	34142.4
	686606.0 #

Details of Top chord

Diaphragm E and F

1 web	19×⅜	@	24.23	×	2'-3"	=	54.6		
4 L's	3½×3½×⅜	@	8.5	×	2'-3"	=	76.5		
4 L's	3½×3½×⅜	@	8.5	×	1'-6¾	=	53.0		
1 Tie Pl.	14×⅜	@	17.85	×	2'-9"	=	49.1		
4 fills	3½×⅜	@	4.46	×	1'-0"	=	17.8		
							231.0	20 @ 231.0	= 4620 #
U1. Gusset Pls.	2 Pl's	45½×¾	@	116.1	×	4'-0"	=	928.0	
U2 Gusset pls.	2 Pls.	51×¾	@	130.1	×	4'-1½"	=	1073.0	
fills	2 Pls.	6×⅝"	@	2.55	×	1'-8½	=	8.7	
splice	2 Pls.	15×⅜	@	19.13	×	1'-9"	=	67.5	
"	2 L's	6×6×½	@	19.6	×	2'-11"	=	114.4	
"	2 L's	6×6×½	@	19.6	×	2'-6"	=	98.0	
fills	2 Pls.	6×⅝	@	2.55	×	1'-3"	=	6.4	
"	1 Pl	5×⅜	@	6.38	×	1'-3½	=	8.7	
"	1 Pl	5×⅜	@	6.38	×	1'-2½	=	7.7	
Splice	1 Pl	33×⅜	@	42.08	×	2'-7"	=	108.8	
Gusset	1 Pl.	24×⅜	@	30.60	×	5'-8½	=	175.0	
								1667.7 #	

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List of Materials for Chorokubashi

U3	gusset plo	2Plo.	54 . 3/4	@ 137.7	. 4'3"	= 1170.0#	
U3-U4							
	Sec Pl.	1L	3½ . 3½ . 3/8	@ 8.5	. 1'6"	= 12.7	
		2Ls	3½ . 3½ . 3/8	@ 8.5	. 1'7½	= 27.6	
							40.3#
U4	gusset	2Plo.	55 . 3/4	@ 140.3	. 4'3"	= 1194.0	
	splice	2Ls	6 . 6 . 5/8	@ 24.2	. 2'6	= 121.0	
	"	2Ls	6 . 6 . 5/8	@ 24.2	. 2'6"	= 121.0	
	fillo	4Plo.	5¼ . ¼	@ 4.46	. 1'3"	= 22.2	
	subpl	2plo	15 . 3/8	@ 19.13	. 1'0"	= 38.3	
	fill	2plo.	6 . ⅛	@ 2.55	. 1'3"	= 6.4	
	"	1Pl.	18 . ¼"	@ 15.30	. 2'9"	= 42.1	
	splice	1Pl	33 . 5/8	@ 70.13	. 2'9"	= 192.8	
	gusset	1Pl	16 . 3/8	@ 22.95	. 2'9"	= 63.1	
							1800.9
U5	gusset	2Plo.	57" . 3/4	@ 145.4	. 4'3"	= 1238.0	
	fill	4Plo	5¼ . ⅛	@ 2.23	. 1'3"	= 11.2	
	splice	2Plo.	6 . 5/8	@ 12.75	. 2'6"	= 63.8	
	"	4Plo.	5¼ . 5/8	@ 11.16	. 2'6"	= 111.6	
	"						14246
U6.	gusset	2Plo.	48 . 3/4	@ 122.4	. 5'0½	= 1233.0	
	splice	4Ls	6 . 6 . 5/8	@ 24.2	. 2'6"	= 242.0	
	"	1Pl.	33 . 5/8	@ 70.13	. 3'1½	= 218.2	
	fill	2Plo.	5 . 5/8	@ 10.63	. 0'9"	= 16.0	
	gusset	1Pl.	20 . 3/8	@ 25.50	. 4'9"	= 121.0	
							1830.3
U7	gusset	2Plo.	47 . 3/4	@ 119.9	. 5'6½	=	1327.0
U8	gusset	2Plo	46 . 3/4	@ 117.3	. 5'9½	= 1357.0	
	"	1Pl.	18 . 3/8	@ 22.95	. 2'7"	= 59.3	
	splice	1Pl.	33 . 5/8	@ 70.13	. 5'2"	= 362.0	
	fills	1Pl.	33 . 5/8	@ 70.13	. 2'7"	= 181.0	
	splice "	2Ls	6 . 6 . 3/4	@ 28.70	. 2'9"	= 158.0	
	"	2Plo	6 . 3/4	@ 15.30	. 2'9"	= 84.1	
	fill	2Plo	25 . ½	@ 12.50	. 3'9"	= 318.5	
	"	2Plo	25 . ⅛	@ 10.63	. 1'0"	= 21.3	
	"	2Plo	25 . ⅛	@ 10.63	. 1'6"	= 32.0	
							2573.2
U9	gusset	2Plo	45 . 3/4	@ 114.8	. 5'4"	= 1222	
	fill	2Plo	25 . ⅛	@ 10.63	. 1'0"	= 21.3	
	splice	2Plo.	25 . ½	@ 42.50	. 2'0"	= 170.0	
							1413.3
					Summary U6-U9		18795.3
U10	gusset	2Plo.	40 . 3/4	@ 102.0	. 4'0"	= 816.0	
	splice	2Plo	25 . 3/4	@ 63.8	. 6'0½	= 769.0	
	fill	2Plo.	25 . ½	@ 42.5	. 3'7"	= 304.0	
	sp	2Ls	6 . 6 . 3/4	@ 28.7	. 3'7"	= 206.0	
		1Pl.	33 . 5/8	@ 70.13	. 5'2"	= 362.5	
	gusset	1Pl.	21 . 3/8	@ 26.78	. 5'2"	= 138.3	
	fill.	1Pl.	33 . 5/8	@ 70.13	. 2'7"	= 181.0	
							2776.8

Summary for details of top chord

4 . 18795.3 = 75181.2
2 @ 2776.8 = 5553.6

80734.8#

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CALCULATIONS FOR

List of materials for Chōrokubashi. Revised 14-4-9

Summary for top chord.

main section	172482.4	net	
Details	80734.0		
Rivet heads + variation 3¾%	9439.0	} 89609.0	57.9%*
	262091.4		

Details of Bottom chord.

20.	flange splice	4Ls	6-4-3/8	@ 12.3*	= 4'6"	= 221.2	
	web splice	2Pls.	27"-3/4"	@ 68.9*	= 6'10"	= 940.0	
	Triple	2Pls.	17½-3/8	@ 22.31	= 3'0"	= 134.0	
							1295.2*
L1.	Gusset pl.	2Plo	50-3/4"	@ 127.5	= 4'4"	= 1105.0	
	Splice	2Plo	27-3/4	@ 68.9	= 2'6"	= 344.2	
	"	1Pl	33-½	@ 56.1	= 2'10½	= 161.0	
	fillo	2Plo	25-¼"	@ 21.25	= 1'0"	= 425	
	Splice	2Plo	25-3/8	@ 31.88	= 1'4"	= 110.2	
							1762.9
L2	Gusset	2Plo	58"-3/4	@ 147.9	= 4'3"	= 1255.0	
	Sp.	2Plo	6-3/4	@ 15.3	= 2'9"	= 84.1	
	"	2Plo	6-3/4	@ 15.3	= 2'9"	= 84.1	
	"	2Plo	25-3/8	@ 31.88	= 5'7"	= 356.0	
							1779.2
L3	Gusset	2Plo	52-3/4	@ 132.6	= 4'6"	= 1192.0	
	fillo	2Plo	25-¼	@ 21.25	= 1'0"	= 42.5	
	Splice	2Plo	25-½	@ 21.25	= 2'0"	= 85.0	
	Tie	1Pl.	33-½	@ 56.10	= 3'5"	= 192.0	
							1511.5
L4	Gusset	2Plo	53-3/4	@ 135.2	= 5'0"	= 1352.0	
	Sp.	2Ls	6-6-3/4	@ 28.7	= 3'9½	= 217.3	
	"	2Ls	6-6-3/4	@ 28.7	= 3'8½	= 212.3	
							1781.6
L5	Gusset	2Plo.	48-3/4	@ 122.4	= 4'0"	= 980.0	
	tie	1Pl.	33-½	@ 56.1	= 2'9"	= 154.3	
							1134.3
L6.	Gusset	2Plo.	52-3/4	@ 132.6	= 5'6"	= 1460.0	
	fillo	2Plo	15-⅜	@ 6.38	= 0'6"	= 6.4	
	"	4Plo	5'4-¼	@ 4.46	= 1'0"	= 17.9	
	Splice	2Ls	6-6-5/8	@ 24.72	= 2'9½	= 135.0	
	"	2Ls	6-6-5/8	@ 24.72	= 2'8½	= 131.1	
							1750.4
L7	Gusset	2Plo	48"-3/4	@ 122.4	= 4'10"	= 1183.0	
	fillo	4Plo	5-¼	@ 4.25	= 1'0"	= 17.0	
	Splice	4Ls	6-6-3/8	@ 14.90	= 2'0"	= 119.3	
	Tie	1Pl.	33-½	@ 56.10	= 2'8"	= 150.0	
							1469.3*
L8	Gusset	2Plo	47"-3/4	@ 119.9	= 5'9"	= 1380.0	
	Splice	4Ls	6-6-3/8	@ 14.9	= 1'8"	= 99.5	
							1479.5*
L9	Gusset	2Plo	46-3/4	@ 117.3	= 5'3"	= 1232.0	
	Splice	1Pl.	33-½	@ 56.1	= 2'10½	= 161.0	
							1393.0

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CALCULATIONS FOR

List of materials for Chorokubashi

Bottom chord details

Diaphragm A	web	1Pl	17½ × ⅜	@ 22.31	× 2'-3"	=	50.2
		4Ls	3½ × 3½ × ⅜	@ 8.5	× 2'-3"	=	76.5
		4Ls	3½ × 3½ × ⅜	@ 8.5	× 1'-5¼	=	48.9
	fill	4Pls	3½ × ⅜	@ 4.46	× 0'-11"	=	16.4
	rivet	2Pls	14 × ⅜	@ 17.85	× 2'-9"	=	98.1
					1 Rennia		290.1
Diaphragm B	web	1Pl	19 × ⅜	@ 24.23	× 2'-3"	=	54.6
		4Ls	3½ × 3½ × ⅜	@ 8.5	× 2'-3"	=	76.4
		4Ls	3½ × 3½ × ⅜	@ 8.5	× 1'-6¾	=	52.0
	fill	4Pl	3½ × ⅜	@ 4.46	× 1'-0"	=	17.8
	rivet	2Pls	14 × ⅜	@ 17.85	× 2'-9"	=	98.1
					18 × 298.9	=	5380.2
							21027.2 #
L8	Gusset	2Pls	44 × ¾	@ 112.2	× 6'-1½	=	1378.0
	c/p	2Ls	6.6 × ⅜	@ 14.9	× 2'-7½	=	78.2
		2Ls	6.6 × ⅜	@ 14.9	× 2'-6½	=	75.7
							1531.9 #

Summary of details of Bottom Chord

L0-to L10	4 @ 21027.2	=	84108.8
at L10	2 @ 1531.9	=	3063.8
			87172.6 #

Summary for Bottom chord

main section	163590.0		
Details	87172.6	} 958772.6	58.6% to main section.
Rivet heads + variation	8700.0		
	259372.6		

Details of ties.

Diaphragm C	web	1Pl.	16 × ⅜	@ 20.6	× 1'-9"	=	35.7
		4Ls	3½ × 3½ × ⅜	@ 8.5	× 1'-9"	=	59.6
		2Ls	3½ × 3½ × ⅜	@ 8.5	× 1'-3¾	=	22.2
	fills	2Pls	3½ × ⅜	@ 4.46	× 0'-9½	=	7.0
	rivet	1Pl.	30 × ½	@ 51.00	× 2'-10½	=	146.0
					1 @		270.5
Diaphragm D	web	1Pl.	17½ × ⅜	@ 22.31	× 1'-9"	=	39.0
		4Ls	3½ × 3½ × ⅜	@ 8.5	× 1'-9"	=	59.6
		2Ls	3½ × 3½ × ⅜	@ 8.5	× 1'-5¼	=	24.5
	fills	2Pls	3½ × ⅜	@ 4.46	× 0'-11"	=	8.2
	rivet	1Pl.	14 × ⅜	@ 17.85	× 2'-10½	=	51.3
					14 × 182.6	=	2556.4 #
L0-		1L	60 × ½	@ 19.6	× 2'-1½	=	41.6
	Gusset	2Pls.	70½ × ¾	@ 179.8	× 9'-0¾"	=	3258.0
		2Pls	60¼ × ¾	@ 177.9	× 7'-4½"	=	2608.0
		2Pls	60½ × ¾	@ 177.3	× 5'-10½	=	1042.5
		2Pls	48 × ½	@ 81.6	× 7'-6"	=	1240.3
		2Pls.	49 × ¾	@ 125.0	× 5'-4½	=	1355.8
	C/p.	2Pls.	18½ × ½	@ 31.45	× 3'-2¾	=	203.2
	fills	2Pls	19 × ½	@ 15.3	× 1'-11¼	=	594.0
	fills	2Pls	[illegible]	@ [illegible]	× [illegible]	=	[illegible]
		1Pl	[illegible]	@ [illegible]	× [illegible]	=	[illegible]
							10301.8 #

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CALCULATIONS FOR

List of materials for Choro Bubashi

Details of tie

T_2 splice	No.	W.	L	C	Cl.30
	4 Pls	24 × 3/4	@ 61.2 × 7'-1"	=	1733.2
	2 Pls	18 × 1/2	@ 30.6 × 7'-1"	=	433.0
	2 Ls	6 × 6 × 1/2	@ 19.6 × 7'-1"	=	278.0
	2 Pls	23 × 3/4	@ 58.7 × 10'-1"	=	1177.0
Tie Pl.	2 Pls	26 × 1/2	@ 44.2 × 2'-11"	=	258.1
				4 ×	3879.3 = 15517.2 #

Summary for tie. (Details)

at Lo		10301.8
Diaphragm C		270.5
Diaphragm D		2556.4
Panel splice T_2-T_4-T_6-T_8		15517.2
	28645.9 × 4 =	114583.6
T_{10}	3879.3 × 2 =	7758.6
		122342.2

Summary for weights of tie.

main section	193260.0		
Details	122342.2	} 328422	68.75% to main section.
Rivet heads + Variation	10500.0		
	326102.2 #		

Details of Verticals.

L_0-L_0	gusset pl.	2 Pls	42" × 3/4	@ 107.1 × 5'-2"	=	1110.0
	fills	2 Pls	3" × 3/4	@ 7.65 × 14 10 1/2	=	28.6
	bottom	2 Ls	6 × 4 × 3/8	@ 12.30 × 14 4 1/2	=	33.7
	"	2 Pls	4 × 3/8 ×	@ 5.10 × 0'-8 3/4	=	7.5
	cap.	1 Pl.	27 1/2 × 3/8	@ 22.31 × 2'-9"	=	61.3
	"	2 Ls	6 × 4 × 3/8	@ 12.3 × 14 4 1/4	=	33.2
	"	2 Pls	4 × 3/8	@ 5.10 × 0'-8 3/4	=	7.5
	"	2 Ls	4 × 4 × 3/8	@ 9.8 × 14 6"	=	29.4
						1311.2 #
L_1-L_1	Tie Pl	1 Pl.	17 1/2 × 3/8	@ 22.31 × 3'-6"	=	78.0
	"	1 Pl	17 1/2 × 3/8	@ 22.31 × 3'-0"	=	66.9
	splice	4 Pls.	12 1/2 × 3/4	@ 31.9 × 2'-10"	=	362.0
	Lacing	20 Pls.	2 1/2 × 3/8	@ 3.19 × 14 6 3/8	=	96.2
						603.1
L_2-L_2	side Pls	4 Pls	12 1/2 × 3/4	@ 31.9 × 3'-3"	=	415.0
	tie	1 Pl	17 1/2 × 3/8	@ 22.31 × 2'-9"	=	61.3
	"	1 Pl.	17 1/2 × 3/8	@ 22.31 × 1'-9"	=	39.0
	Lacing	12 Pls	2 1/2 × 3/8	@ 3.19 × 14 6 3/8	=	57.9
						573.2
L_3-L_3	Tie	1 Pl	17 1/2 × 3/8	@ 22.31 × 3'-0"	=	66.9
	"	1 Pl	17 1/2 × 3/8	@ 22.31 × 3'-6"	=	78.0
	Lacing	12 Pls	2 1/2 × 3/8	@ 3.19 × 14 6 3/8	=	57.9
						202.8
L_4-L_4	tie	1 Pl	17 1/2 × 3/8	@ 22.31 × 2'-9"	=	61.3
	side Pls	4 Pls	12 1/2 × 3/4	@ 31.9 × 3'-1"	=	393.0
	tie	1 Pl	17 1/2 × 3/8	@ 22.31 × 1'-9"	=	39.0
	Lacing bars	9 Pls	2 1/2 × 3/8	@ 3.19 × 14 6 3/8	=	43.3
						536.6
L_5-L_5	Tie	2 Pls	17 1/2 × 3/8	@ 22.31 × 3'-0"	=	133.8
	Lacing	9 Pls	2 1/2 × 3/8	@ 3.19 × 14 6 3/8	=	43.3
						177.1

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CALCULATIONS FOR

List of materials for Chorokubashi

Details for Verticals

U_6-L_6		4 Pls	12½ . 3/4	@ 31.9	. 3'1"	= 393.0	
	tie	1 Pl	17½ . 3/8	@ 22.31	. 3'0"	= 66.9	
	"	1 Pl	17½ . 3/8	@ 22.31	. 6'8"	= 149.0	
	lacing	4 Pls	2½ . 3/8	@ 3.19	. 1'6⅛	= 19.2	
							628.1
U_7-L_7	tie	2 Pls	17½ . 3/8	@ 22.31	. 3'0"	= 133.8	
	lacing	6 pls	2½ . 3/8	@ 3.19	. 1'6⅛	= 28.8	
							162.6
U_8-L_8		2 Pls	12½ . 3/4	@ 31.9	. 14'8½	= 109.2	
		2 Pls	12½ . 3/4	@ 31.9	. 14'2½	= 77.3	
	tie	1 Pl	17½ . 3/8	@ 22.31	. 2'9"	= 61.3	
	"	1 Pl	17½ . 3/8	@ 22.31	. 4'10"	= 108.0	
	lacing	4 Pls	2½ . 3/8	@ 3.19	. 1'6⅛	= 19.2	
							375.0
U_9-L_9	tie	2 Pls	17½ . 3/8	@ 22.31	. 3'0"	= 133.8	
	lacing	4 Pls	2½ . 3/8	@ 3.19	. 1'6⅛	= 19.2	
							153.0
U_{10}-L_{10}	tie	1 Pl	17½ . 3/8	@ 22.31	. 2'6"	= 55.8	
		1 Pl	17½ . 3/8	@ 22.31	. 4'2"	= 93.0	
	Lacing	4 Pls	2½ . 3/8	@ 3.19	. 1'6⅛	= 19.2	
							168.0

Summary Details of verticals -

U_6-L_6 to U_9-L_9	4 @	4722.7	= 18,890.8
U_{10}-L_{10}	2 @	168.0	= 336.0
			19226.8 #

Summary for weights of verticals

main section	77808.4		
Details	19226.8		
Rivet heads + variation 35%	32300	} 224568 #	28.9% to main section.
	100265.2 #		

Details of Diagonals

U_0-L_1	tie	2 Pls	17½ . 3/8	@ 22.31	. 2'0"	= 89.2	
	lacing	18 Pls	2½ . 3/8	@ 3.19	. 1'6⅛	= 86.6	
							155.8
U_1-L_2	tie	2 Pls	17½ . 3/8	@ 22.31	. 2'0"	= 89.2	
	sp.	4 Pls	12½ . ½	@ 21.25	. 2'-11"	= 248.0	
	fill	4 Pls	12½ . ¼	@ 10.63	. 1'6"	= 63.8	
	lacing	16 pls	2½ . 3/8	@ 3.19	. 1'6⅛	= 77.0	
							478.0
U_2-L_3	tie	2 Pls	17½ . 3/8	@ 22.31	. 2'0"	= 89.2	
	lacing	14 Pls	2½ . 3/8	@ 3.19	. 1'6⅛	= 67.4	
							156
U_3-L_4	tie	2 Pls	17½ . 3/8	@ 22.31	. 2'0"	= 89.2	
	sp.	4 Pls	12½ . 3/4	@ 31.9	. 3'9½	= 484.0	
	lacing	12 Pls	2½ . 3/8	@ 3.19	. 1'6⅛	= 57.7	
							630.9
U_4-L_5	tie	2 Pls	17½ . 3/8	@ 22.31	. 2'0"	= 89.2	
		4 Pls	12½ . ½	@ 21.25	. 3'0"	= 255.0	
	fills	2 Pls	12½ . ¼	@ 10.63	. 1'6"	= 31.9	
	lacing	10 Pls	2½ . 3/8	@ 3.19	. 1'6⅛	= 48.1	
							424.2
U_5-L_6	tie	2 Pls	17½ . 3/8	@ 22.31	. 2'0"	= 89.2	
	side	4 Pls	12½ . 3/4	@ 31.9	. 3'9½	= 484.0	
	lacing	9 Pls	2½ . 3/8	@ 3.19	. 1'6⅛	= 43.3	
							616.5

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CALCULATIONS FOR

List of Materials for Chorokutashi

U6-L7	tie	2Pls	17½ · ⅜	@ 22.31	· 2'-0" = 89.2	
		4Pls	12½ · ½	@ 21.25	· 2'-11" = 248.0	
	fills	4Pls	12½ · ¼	@ 10.63	· 1'-6" = 63.8	
	lacing	8Pls	2½ · ⅜	@ 3.19	· 116⅛ = 38.5	
						439.5
U7-L8	tie	2Pls	17½ · ⅜	@ 22.31	· 2'-0" = 89.2	
		4Pls	12½ · ¾	@ 31.9	· 4'-6" = 573.0	
	lacing	7Pls	2½ · ⅜	@ 3.19	· 116⅛ = 33.6	
						695.8
U8-L9	tie	2Pls	17½ · ⅜	@ 22.31	· 2'-0" = 89.2	
	lacing	7Pls	2½ · ⅜	@ 3.19	· 116⅛ = 33.6	
						122.8
U9-L10	tie	2Pls	17½ · ⅜	@ 22.31	· 2'-0" = 89.2	
		4Pls	12½ · ½	@ 21.25	· 2'-9" = 234.0	
	fills	4Pls	12½ · ¼	@ 10.63	· 1'-6" = 63.8	
	lacing	7Pls	2½ · ⅜	@ 3.19	· 116⅛ = 33.6	
						420.6

Summary for details

4 @ 4140.7 = 16562.8 #

Summary for weight of diagonals

main section	45412.8		
details	16562.8	} 18732.8	41.3% to main section.
Rivet heads + variation	2170.0		
	64145.6		

Details of Hangers

L2-T2	Tie	1Pl	17½ · ⅜	@ 22.31	· 3'-6" = 78.0	
L4-T4	"	1Pl	17½ · 7/16	@ 26.03	· 2'-6" = 65.0	
	fill	2Pls	9¾ · ⅝	@ 20.72	· 1'-9" = 72.5	
	"	2Pls	16 · ⅛	@ 6.80	· 1'-6" = 20.4	
						235.9
T6-L6	Tie	1Pl	17½ · ⅜	@ 22.31	· 3'-0" = 669	
L8-T8	splice	1Pl	17½ · 7/16	@ 26.03	· 2'-6" = 65.8	
L10 T10	fills	2Pls	9¾ · ⅝	@ 20.72	· 1'-9" = 72.5	
	fills	2Pls	16 · ⅛	@ 6.80	· 1'-6" = 20.4	
	splice	2Pls	12½ · ¾	@ 3.19	· 3'-1½ = 198.0	
						423.6

Lacing bars.

L2T2	8 Pls	2½ · ⅜	@ 3.19	· 116⅛ = 38.5	} 370.9	
L4T4	19 Pls	"	@ "	" = 91.5		
L6 T6	23 Pls	"	@ "	" = 110.8		
L8-T8	27 Pls	"	@ "	" = 130.6		
L10T10	29 Pls	"	@ "	" = 139.7		
						510.6 #

Summary

2-4	@ 235.9	=	1887.2
10	@ 423.6	=	4236.0
lacing 4	@ 370.9	=	1483.6
" 2	@ 139.7	=	279.4
			7886.2 #

Summary for Hangers

main section	34442.4		
Details	7886.2	} 9386.2	27.5% to main section
Rivet Heads + variation say	1500.0		
	43828.6 #		

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CALCULATIONS FOR

List of Materials for Chorotsu bashi

Grand Summary of weight of truss (one span)

		main section	Details	
Upper chord	262091.4	172482.4	89609.0	51.9 %
Lower chord	259372.6	163500	95872.6	58.6
Verticals	100265.2	77808.4	22456.8	28.9
Diagonals	64145.6	45412.8	18732.8	413
Hangers	43528.6	43142.4	9386.2	21.5
Tie	326102	193260.0	132842	68.75
	1055505.6#			
	471.208 tons			

Weight of shoes

Roller shoes — Top Casting as per detail 10374.6 cubic inches or 6.01 cubic ft
weight 6.01 @ 490# = 2940.#
Bearing Pls. 5722 cubic inches = 33.2 cubic ft @ 490# = 1626
Longitudinal guard 2@173# = 346
Transv. guard 2@78.5 = 157
Rollers 8 Rollers @ 314# = 2510
1 Roller 309 = 2819
Roller spacer 4@14.21# = 56.8
Foot bar 2@2.5# = 5.0
7949.8#

Fixed shoes 11.12 cubic ft @ 490# = 5450#
8¼"ϕ pin with standard nuts = 528.8#
Anchor bolts 1¼" × 1'-10½ with head and nut = 9.66#

Summary for shoes pins and Anchor Bolts.

Roller shoes	2 @ 7949.8 =	15899.6
Fixed shoes	2 @ 5450 =	10900.0
Pins	4 @ 528.8 =	2115.0
Anchor	32 @ 9.66 =	309.0
	29223.6# or	13.046 tons

Summary for structural steel in span

Stringers complete	109.298 tons
Floor beams complete	107.690 "
Cotton Lateral	11.688 "
Top Lateral system complete	48.481 "
Trusses complete	471.208 "
Shoes etc	13.046 "
	761.416 tons

Ornamental iron casting for End posts (cast iron)

base	2405
bottom section	3850
top section	1845
Cap.	1435
	9535#
Add 10%	953
	10488

4 @ 10488 = 41952# or 18.729 tons

Grand Total of metal on span

structural steel in truss	761.416
End posts (cast iron)	18.729
	780.140 tons

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CALCULATIONS FOR

List of Materials for Chōroku-bashi for Tsumamoto-ken Revised

Revised

Concrete in slab

Sidewalk	33	× 24.18	× 245.3尺	=	195735	
Long. beams	4@	0.5 × .734	× 7.0	=	10.30	
Roadway slab.		0.5 × 2.135	× 245.3	=	3311.56	
Slab under Elec Ry		0.54 × 18.0	× 245.3	=	2384.32	
Fillet over stringers		2 @ .216	× 245.3	=	106.00	
Joint Cy & Rwy Slab		2 @ .212	× 245.3	=	104.00	
					8406.31 cubic尺 or	38.91 立方

Reinforcing bars in Reinforced Concrete slab
As shown on Sheet # 6 30.216 tons

Lean mortar filling under Electric Ry

3½" thick	0.293	× 18.0	× 245.3尺	=	1290 立方尺	or 5.98 立方
at rail.	4@.25	× .35	× 245.3	=	86.0 " "	0.40
						6.38 "

Asphaltic Concrete Pavement.

30尺 wide 30.0 × 245.3 ÷ 36 = 204.2 両坪

Granite Pavement for Electric Ry tracks.

(18.0 − 1.750(width)×2) × 0.33 × 245.3 = 1342.0才 including joint

Curb stone size of stone 0.5 × 0.7 × 2.5

vol = 2 @ 0.5 × 0.7 × 245.3尺 = 172.0才 including joint

Sidewalk slabs. Asphalt finish

Sidewalks 2 @ 12.25 × 245.3 = 6000.0 平方尺 or 166.7 両坪

人造石洗出仕上

2 @ 1.37 × 245.3 = 672.0 平方尺 or 18.7 両坪

Handrail cast iron.

Typical Handrail

Hor.	2	− 1.250"	@ 3.12"	×	5'-8½	=	44.6
"	1	1.00	@ "	×	5'-6"	=	17.2
Vert.	2	1.25	@ "	×	2'-8½	=	20.9
"	21	1.00	@ "	×	0'-5"	=	27.6
"	16	1.00	@ "	×	2'-0"	=	100.0
Hor.	12	1.00	@ "	×	0'-2"	=	6.4
Panels.	12		12"			=	14.4
							231.1
Top and bottom rails	2 @ 4.5"	× 3.12"	× 5'-8½	=			161.0
							392.1 #

80 panels @ 392.1 = say 31400 #

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List of materials for Chorokubashi

Handrail Posts P_1 Cast iron

above sidewalk upper section 3/4" pipes — weight q'ty 107.6 #
Lower section do 167.6
175.2 #

82 posts @ 175.2 = 14,366. #

Handrail Posts P_2.

above sidewalks slab. weight 107.6
base add 5.5
113.1 #

4 posts @ 113.1 = 452.4

Handrail Posts P_3.

above sidewalks slab & base say 113.8 #
Panel say 184.8
298.6 over this 300 #
4 @ 300 = 1200 #

Anchor bolts 502 @ 5/8" ϕ . 0'-5" @ .66 per piece = 331 #.
Panel F 93.1 # 4 @ 93.1 = 372.4

Summary for Handrail

Typical panels A-B-C-D-E	31400
Posts P_1	14366
" P_2	452.4
P_3	1200.0
Anchor bolts	331.0
Panel F	372.4
	48121.8 #, or 21.48 tons

Forms for slabs.

under slabs and edge 444.73 ㎡

Materials for abutment

Concrete in abutment 1:3:6 Concrete

60 . 17.0 . 83.5 = 8525.0
30 . 14.0 . 83.5 = 3510.0
12035.0 cubic ft 55.72 ㎥

Concrete in abutment 1:2:4 concrete

Parapet wall 1.75 . 28.34 . 73.5 = 3642.0
.65 . 7.99 . 73.5/2 = — 166.0
3476.0

Side column 2@ 4. . 4.5 . 28.34 = 1020.0
" " bottom filler 2@ 2.0 . 9.0/2 . 4.5 = 81.0

Body of abutment with coping Bottom area = 398.0
Top area = 346.
volume = (346 + 398 + √(346 . 398)) . 11.95 . 1/3 = 4450.0
Cases for guide seats 2@ 377 . 30 . 197 = 43.5
Bathen walls for parapet 39.8
Total 9110.3 or 42.26 ㎥

net volume = 42.26 − 3.72 stone & brick = 38.54

Body of abutment below stone facing

top area 2@ 32.245 . 6.37 = 410
bottom area 2@ 33.085 . 6.91 = 457.5
867.5
√Product 432.0
1299.5 . 1/3 . 8.25 = 3580 or 16.55 ㎥

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CALCULATIONS FOR

List of Materials for Abutments Revised

Concrete under Shoe of Truss

7 @ $\frac{5.7+4.2}{2}$ × .415 × .414 = 18.1 コンクリート or 0.09 立坪

Summary for Concrete in Abutment

38.54 + 16.55 + 0.09 = 55.18 立坪

Brick Facing Total 41.00 面坪

Forms for Concrete of Abutment 198.5 面坪

Excavation 16.0 × 17 = 272.0

$\frac{4+11}{2}$ × 15 = 112.5

384.5 × 83.5 ÷ 216 = 148.5 立坪

60% increase = 89.3

237.8 〃

Add this 240.0 立坪

Reinforcing Steel in abutment see sheet no 3 8.700 tons per abutment

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CALCULATIONS FOR

Preliminary Design of Steel Arch Bridge for Kitamoto-ku

During recent flood the old bridge entirely destroyed and the new bridge shall be constructed at the same site, the officials decide to build the bridge of one span without pier on the river channel on account of reducing obstruction during flood. The total span length of the new bridge will be of 240 between end bearings and of the type of the arch to give pleasant effect to the bridge site and surroundings.

The width of roadway 48' between curbs and the 4 double tracks suburban electric railway line to be located at the center of roadway. The pavement would be of Stone block on asphalt, or equivalent weight material of the nearly same weight to reduce the dead load of the bridge. The floor slab shall be of reinforced concrete resting on steel stringers. Sidewalks on the both sides of roadway will be of 9' clear outside of chord of truss and made of reinforced concrete finished with ½" mortar. The handrails will be of ornamental iron, likewise with for fascia girder, to give homogenious good appearance to the metalic arch span.

Assumed loadings

Uniform load on roadway $w = \frac{120.000}{170+l} \fallingdotseq 600\ ^{kg}/_{m^2} \fallingdotseq 123\ ^{\#}/_{0'}$

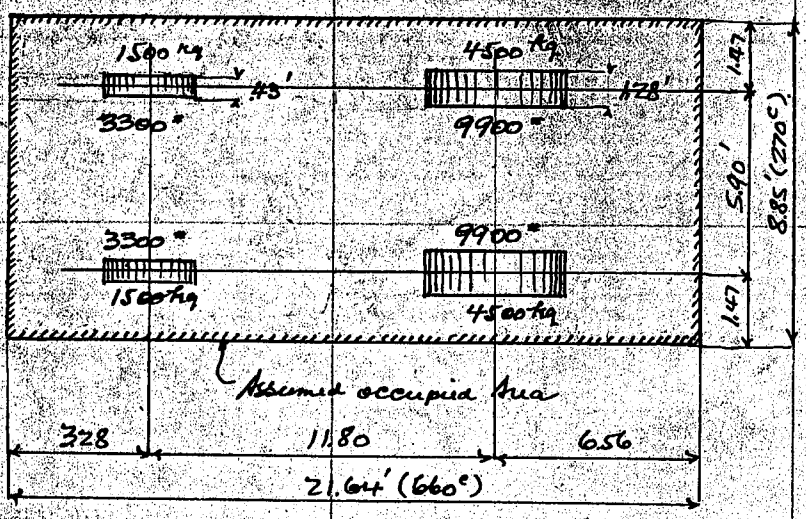
Uniform load on sidewalk $w = \frac{100.000}{170+l} \fallingdotseq 500\ ^{kg}/_{m^2} \fallingdotseq 102\ ^{\#}/_{0'}$

where w = Uniform load in kg per square meter

l = Span length in meter.

For floor system the uniform load of $600\ ^{kg}/_{m^2}$ for roadway and $500\ ^{kg}/_{m^2}$ for sidewalks; For truss members or main girders, the uniform load shall be figured by the above formulas giving l as span length in meter.

Motor trucks loading 12 ton motor trucks (26300#)

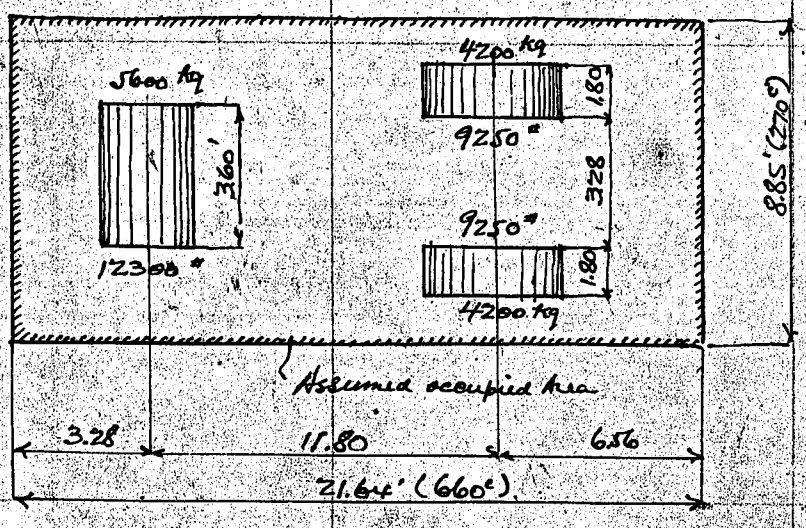


Four rows of motor traffic for roadway with occupied space of 8.85' each Impact into consideration. Combined with electric car line 2 rows of motor traffic assumed. Unoccupied space of motor trucks shall be filled in with specified Uniform load.

Only one motor truck in the direction of bridge assumed; rear and the front of motor trucks to be filled in with the said uniform load.

Impact for motor trucks as specified.

Load Roller 14 ton roadroller (30800#).



Only one motor truck roadroller on the bridge without impact.

Impact for highway loading and also for Electric Ry loading

$$\text{Impact} = \frac{20}{60+l}$$ where l = Loaded length in meter

Max. impact limited to 30% No impact for uniform load and roadroller concentration.

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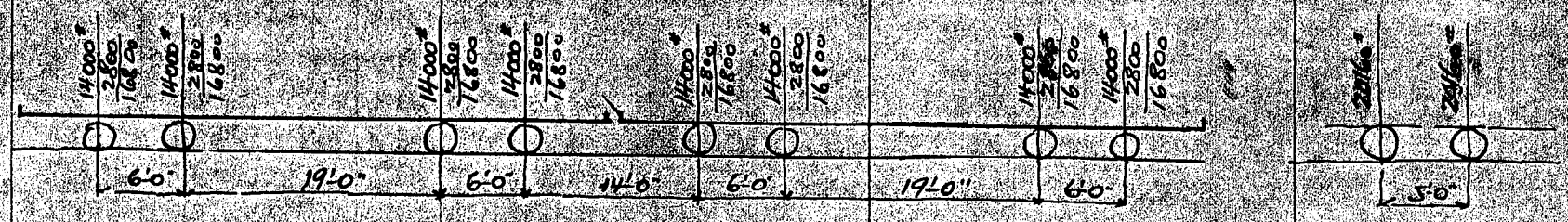
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CALCULATIONS FOR

Preliminary Design of Shoroku bashi for Kumamoto-ken

Electric Railway Loading. 30 ton cars coupled.



Length of car out to out of end baffles = 45'-0"
For design of floor system use #2660 concentration
Impact coefficient as specified on pp/

Total load = 67200 # or 30 tons of 2240 #
or 36 ton car of wheel base of 5'-0"

Assumed weight of materials.

Creosoted wood blocks	900 kg/m³	or	56 # per cubic ft
mortar 1:3 mix	1700 "	"	106 # " " "
Asphalt	2100 "	"	130 " " "
Plain Concrete	2200 "	"	140 " " "
Reinforced Concrete	2400 "	"	150 " " "
Structural Steel	7850 "	"	490 " " "
Masonry Granite	2600 "	"	160 " " "
Cast iron	7250 "	"	452 " " "

Allowable working strength of materials.

Structural Steel or Reinforcing bar

Tension	12000 kg/cm²	or	17000 #/□"
Extreme fibre stress mst	12000 "	"	17000 #/□"
Shearing of web gross section	9000 "	"	12800

Compression members.

$$1500\left(1-0.0055\frac{l}{r}\right) \text{ or not over } 1{,}000 \text{ kg/cm}^2$$

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

Equivalent formula for inch-lbs units.

$$21300\left(1-0.0055\frac{l}{r}\right) \text{ or not over } 14000 \text{ \#/□" gross area}$$

Compression flange of plate girder

$$1200\left(1-0.012\frac{l}{b}\right) \text{ kg} \leqq 1{,}100 \text{ kg/cm}^2$$

where l = unsupported length of flange in centimeter
b = width of flange in centimeter.

Equivalent formula for inch-lbs units.

$$17000\left(1-0.012\frac{l}{b}\right). \leqq 15400 \text{ \#/□"}$$

Shearing on shop driven rivets (machine rivets)		12.000 #/□"
Shearing on field driven rivets and turned bolts (machine)		10.000 #/□"
Extreme fibre of pin		24.000 #/□"
Bearing on pin		24.000 #/□"
Bearing on shop rivets		24000 "
Bearing on field rivets and turned bolts		20.000 "
Expansion roller	45 d kg/cm where d = diameter of roller in centimeter	
or inch-lbs units	610 d # per linear inch where d = dia. of roller in inches.	
Bearing on masonry 1:2:4 concrete	45 kg/cm² = 640 #/□"	

l/r for compression member not over 120 for truss members 140 for wind bracings
l/r for riveted tension members limited to 200

Strength of Concrete 1:2:4 mixture

Direct compression for column	35 kg/cm²	or	500 #/□"
Compressive fibre stress	45 "	"	640 "
Combined stress for columns	35 "	"	500 #/□"

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CALCULATIONS FOR

Preliminary Design of Chōkubashi for Tsumamoto-machi

Combined stress for arches	45 kg/cm²	or	640 #/□"
Punching shear	9 "	"	128 "
Shear plain concrete	4 "	"	58 "
Bond stress for plain bar	6 "	"	85 "
Bond stress for deformed bar stress increased 50%			130 "
Shear for reinforced concrete with web reinforcements assumed			128 "

1/2 for reinforced concrete compression member limited to 50

Considering wind, traction and temperature stresses in addition to dead, live and impact stresses, the allowable working shall be increased 25% and proportioned the section of members. In case of figuring earthquake effect, the working stress shall be increased 80%

Miscellaneous Data

Ratio between moduli of elasticities of steel and concrete = 15
Expansion coefficient of concrete = 0.000011 per 1°c
" " " steel = 0.000012 " 1°c
Temperature change for concrete structure ± 15°c
" " " steel structure ± 30°c
Standard temperature assumed 20°c
Accelleration of earthquake = 2000 mm/sec²

Cross section of structure

scale 1/50

Cross section of bridge at End Floor Beam

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CALCULATIONS FOR

Preliminary Design of Chōrokubashi for Kumamoto-Ken

Concrete Floor Slab for Roadway

Floor slabs under asphalt pavement span length = 4'3" c to c of stringers

2" asphalt @ 131#/12 = 22
6" concrete assumed 75
97# call this 100# per sq ft including filler over stringer

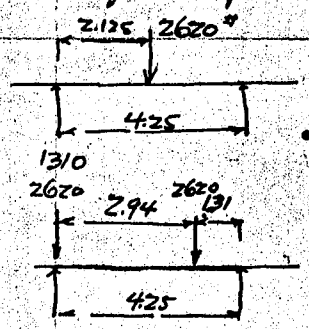
Dead Load Moment = 1/10 · 100 · 4.25² = 181 ft#
Dead Load Shear = 100 · 4.25/2 = 212#

Live Load motor truck rear wheel concentration 9900#
Impact = 20/(60 + 4.25/3.28) = .326 Impact assumed 30% 2970
12870# including impact

Distribution of wheel concentration
Dimension of contact between wheel and pavement assumed 20 centimeter = .66'
Thickness of asphalt pavement 2" = .34
1.00' = a

Effective width = E = 2/3 (l+b) + a where l = span length b = 1.28 + .34 = 1.62 = b
= 2/3 (4.25+1.62) + 1.00 = 4.9'

Load for one ft strip = 12870 ÷ 4.9 = 2620#



Live Load moment = 1310 · 2.125 = 2780 ft# as simple beam
0.8 · 2780 = 2220 ft# as continuous beam

Max End shear 2620 · 1.31/4.25 = 806
2620
Tot. 3426#

Summary for moment and shear

	Moment	Shear
Dead Load	181	212
Live Load	2220	3426
Total	2401 ft#	3638#

Effective depth of slab for steel 17000#/□" concrete 640#/□" d = √(2401/102) = 4.85"

Steel Required = 2401 × 12 / (7/8 × 5 × 17000) = .387 □" per ft make slab 6" thick with 1" insulation

Use ½"ϕ bars 6" spacing 0.39 □" per ft

Unit shear = 3638/(7/8 × 5 × 12) = 69#/□" require web reinforcement

Floor slab under Electric Ry tracks span length = 5'-0"

4" stone block pavement	@ 162#/12	=	54
4½" lean mortar filling	@ 137/12	=	52
Concrete slab assumed			94
			200# per sq ft

Dead Load moment = 1/10 · 200 · 5² = 500 ft#
Dead Load shear = 1/2 · 200 · 5/2 = 500#

Live load motor trucks wheel concentration with impact = 12870#
Distribution of load lengthwise a = .66 + 1.42 = 2.08
crosswise b = 1.28 + 1.42 = 2.70
span length = 5.0'

E = 2/3 (5.00 + 2.70) + 2.08 = 7.22 Assume distribution 6.0 wide

Load per ft = 12870 ÷ 6.0 = 2140#
Live load moment concentration at center 1070 · 2.5 = 2680 ft# as simple beam
2680 · 0.8 = 2140 as continuous beam

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

Preliminary Design of Chorokubashi for Kumamoto-ken

Max End shear $2140 + \frac{286}{550} = 915$

2140
$3055^{\#}$

Summary for moment and shear

	Moment	Shear
Dead Load	500	500
Live Load	2140	3055
	$2640^{\#}$	$3555^{\#}$

Effective Depth of slab for Kat = 17000 #/□". Concrete 640 #/□". $d = \sqrt{\frac{2640}{102}} = .51"$
Make slab 6½" over all with 1" insulation

Steel required $= \frac{2640 \times 12}{\frac{7}{8} \times 5.5 \times 17000} = 0.389$ use ½" bars 6" centers = 0.39 □". ok

Unit shear $= \frac{3555}{\frac{7}{8} \times 5.5 \times 12} = 61.5$ #/□". ok

Sidewalk Slab. span length 5'-10½"

Dead Load Slab.

¾" wearing course @	$\frac{137}{12}$	= 9.0
3½" Reinforced concrete @	$\frac{450}{12}$	= 44.0
		53.0 # per sq. ft.

moment $= \frac{1}{10} \cdot 53 \cdot 5.87^2 = 183^{\#}$
shear $= 53 \cdot \frac{5.87}{2} = 156^{\#}$

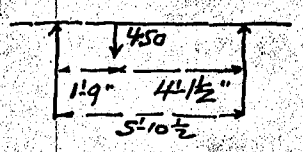
Dead Load from water main

water main assumed 12" dia = 100 # per ft.
volume of water $\frac{113}{144} \cdot 62.5 = \text{Say } \underline{50}$
150 # per lin ft.

Water pipe will be hanged every 6' apart.
Total load = 150 · 6 = 900#

Assume this load distributed over 2' of sidewalk slab. or 450 # per lin ft.

Reaction $= 450 \cdot \frac{4.12}{5.87} = 315$



moment $= 315 \cdot 1.75 = 551^{\#}$

For continuity of beam
moment $= 0.8 \cdot 551 = 440^{\#}$

Live Load on sidewalk 500 kg/m² = 102 #/□'

moment $= \frac{1}{10} \times 102 \cdot 5.87^2 = 352^{\#}$
shear $= 102 \cdot \frac{5.87}{2} = \text{Say } 300$ as simple beam

Summary for moments and shears

	Moment	Shear
Dead Load	183	156
water main	440	315
Live Load	352	300
	975	771

Effective depth required
$d = \sqrt{\frac{975 \times 12}{12 \cdot 102}} = 3.1"$

Use 4" slab with ¾" insulation

Steel Area $= \frac{975 \times 12}{\frac{7}{8} \cdot 325 \cdot 17000} = 0.242$ □" per ft strip
Use ⅜"ϕ bars 4" centers = 0.33 □" per ft strip

Note:—
Add sufficient reinforcement at hanger of water main to spread load over 2' at least.

Design of Stringers under tracks span length = 24'-0" spacing = 5'-0"

Dead Load

4" stone blocks pavement	54.0
4½" lean mortar cushion	52.0
6½" concrete slab.	81.0
	187.0 # per sq ft

weight of one rail 85# ÷ 3 = 28.4
accessories 2.8
31.2 #
call this 30 # per ft additional to flooring.

Dead Load of Floor = 187.5 = 935#
Rail 30
Stringer assumed 85
1050 # per lin ft

moment $= \frac{1}{8} \cdot 1050 \cdot 24.0^2 = 75500^{\#}$
shear $= \frac{1}{2} \cdot 1050 \cdot 24 = 12600^{\#}$

増田淳氏関係資料
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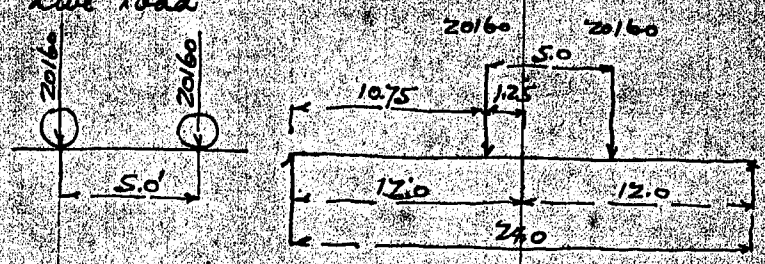
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CALCULATIONS FOR

Preliminary Design of Chorokubashi for Kamamoto-Ken

Design of Stringer for Electric Ry Track (Continued)

Live Load



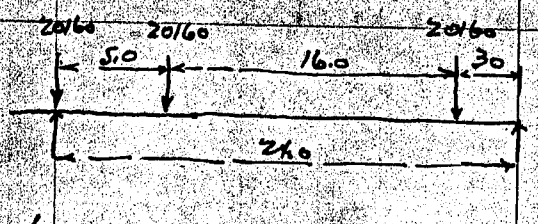
Moment = 40320 × 10.75²/24.0 = 194500 ft

Impact allowance = 20/(60+ 121/328) = 29.7 %

Impact = 194.500 · 29.7% = 58000

Live Load moment with impact = 252500 ft per track
" " " " " = 126250 ft per rail

Live Load shear



20160 · 27.0/24.0 = 18500
20160
38660 #
Impact 29.7 % 11500
50160 # with impact per track
25080 " " " per rail

Summary for moments and shears

	Moment	End shear
Dead Load	75500	12600
Live Load	126250	25080
	201750 ft	37680 #

section modulus required = 20175.0 × 12/17000 = 142.0

~~Try 26 3/8" wb flange 15 6×½" weight per ft = 98.0# good for end shear of 65000#~~
~~Section modulus 258.9 Ok call this 105# per lin ft~~ Use 24" I · 79.9# Sm = 143.9 Ok

Stringer for Roadway Span length 24'0" spacing 4'3"

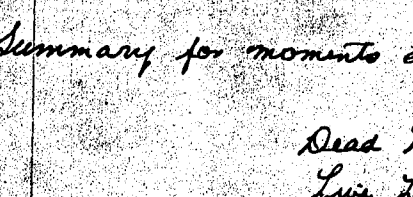
Dead Load

2" Asphalt	22
6" Concrete slab	75
	97 call this 100# @ 4.25 = 425
beam assumed	70
	495 #

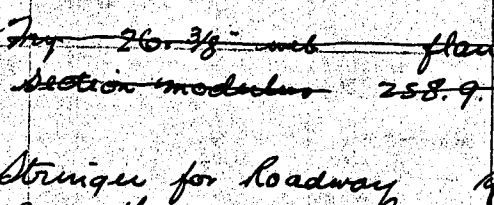
Bending Moment = 1/8 · 495 · 24² = 35700 End shear = ½ · 495 · 24 = 5950#

Live Load Rear wheel of motor trucks 9900 #
Impact = 120/(60+ 24/328) = 29.7% 2940
12840 #
Front wheel concentration 12840 × 1/3 = 4280 #

Max Reaction on Stringer



Concentration on stringer = 12840 · 131/4.25 = 3950
12840
16790
Front wheel 16790 · 1/3 = 5600
22390 #

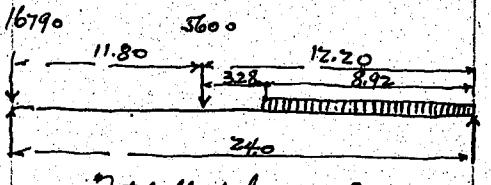


Uniform Live Load = 123# per sq ft @ 4.25 = 522# per ft.

Moment due to motor trucks = 22390 · 10.52²/24.0 = 103000 ft

Reaction R 522 · 3.96 = 2070# for uniform load
2070 · 1.98/24.0 = 171 # Moment due to Unif. load = 171 · 13.48 = 2300 ft
Total 105300 ft

Max End Shear



Total Unif. load = 8.92 · 522 = 4700 #

Motor trucks concentration = 5600 × 12.20/24.0 = 2850
16790
19640
Uniform load 4700 × 4.46/24.0 = say 870
20510 #

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CALCULATIONS FOR

Preliminary Design of Chorokubashi for Kumamoto City

Summary for moments and shears

	Moment	Shear
Dead Load	35700	5950
Live Load	105300	20510
	141000 ft	26460 #

Section modulus = $\frac{141000 \times 12}{17000}$ = 1000

Use 20" I 65.4# Sm = 116.9

Sidewalk Stringer at center of sidewalk. Span length = 24'-0" spacing @ 10½"

Dead Load

¾" wearing course	= 9.0
4" Reinforced concrete slab	= 50.0
	59.0

call this 60# per ft @ 5.87 = 353#

Load from water main 100 × $\frac{4.12}{5.87}$ = 70

Conduit pipes say 40 ± 2 = 20

443 #

Moment = $\frac{1}{8} \cdot \overset{488}{443} \cdot 24^2 = \overset{35200}{34800}$ #

End shear = $\frac{488}{443} \cdot \frac{24}{2} = \overset{5850}{5310}$ #

45 beam assumed

488

Live load Uniform live load 102 × 5.87 = say 600#

Reaction from water main 50 × $\frac{4.12}{5.87}$ = 35

635

Live load moment = $\frac{1}{8} \times 635 \times 24^2 = 45700$ End shear = $635 \cdot \frac{24}{2} = 7620$#

Summary for moments and shears

	Moment	Shear
Dead Load	35200	5850
Live Load	45700	7620
	80900 ft#	13470 #

Section modulus rqd = $\frac{80900 \times 12}{17000}$ = 57.0

Use 15" I 42.9# Sm = 58.9 ok.

Fascia Girder at Sidewalk

Dead Load

Sidewalk slab and wearing course	60 × 3.43 = 206.#
weight of Handrail assumed	60
Dead Load beam assumed	40
Load from conduit pipe.	20
	326#

Bending moment = $\frac{1}{8} \cdot 326 \cdot 24^2 = 23500$ ft# End shear = $326 \cdot \frac{24}{2} = 3910$#

Live Load - 102# × 2.93 = say 300# per lin ft

Bending moment = $\frac{1}{8} \times 300 \times 24^2 = 21600$ ft# End shear = $300 \times \frac{24}{2} = 3600$#

Summary for moments and shears

	Moment	Shear
Dead Load	23500	3910
Live Load	21600	3600
	45100 ft#	7510 #

Try 20", ⅜" web area = 7.50 □" ⅛ web = .940"

Effective Depth = 1.54 stem = 45100 ÷ 1.54 = 30.000#

Section rqd = 30.000 ÷ 17000 = 1.765

.94

.825 □" net

Use 1L 3 × 3 × ⅜" = 2.11 gross − .65 = 1.46 □" net

Approximate weight of fascia girder

1 web pl.	20 × ⅜	255
2L 3 × 3 × ⅜	@ 7.2	144
		39.9 #
add details.		4.1
		44.0# per lin ft.

Stringer at curb.

Sidewalk slab	60# × 2.93 =	176
water main	100 × $\frac{1.75}{5.87}$ =	30
curb stone		65
		271 #

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CALCULATIONS FOR

Preliminary Design of Chōroku bashi for Kumamoto-Ken

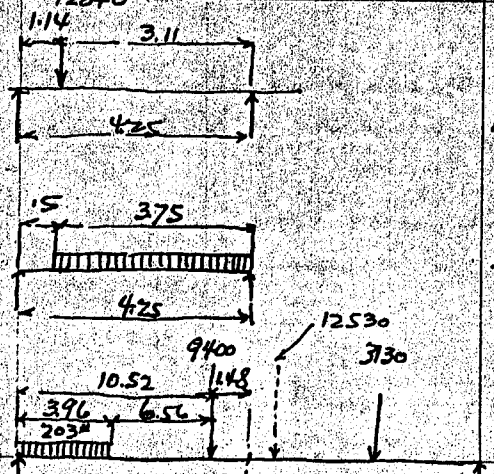
From above 271
Roadway slab say 100 × 4.25/2 = 212
concrete filler say 50
Dead Load of beam assumed 50
583 call this 600# per lin ft

Bending Moment = 1/8 · 600 · 24² = 43200# End Shear = 1/2 · 600 · 24 = 7200#

Live Load.

Uniform live load on sidewalks say 102 · (5.87/2 + 0.5) = 3690#

Motor truck rear wheel concentration 12840# with impact
" " front wheel " 4280 " " "



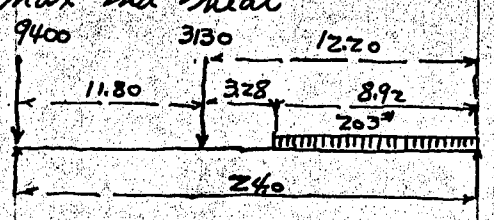
Reaction on Stringer 12840 × 3.11/4.25 = 9400#
front wheel 3130
12530#

Uniform Live Load 123 · 3.75²/2 · 4.25 = 203# per lin ft

moment due to motor trucks = 12530 · 10.52²/240 = 57700 1#

Uniform load 203 × 3.96 = say 800#
Reaction = 800 × 1.98/24.0 = 66.0# moment = 66.0 · 13.48 = 900

Unif. load on sidewalk 1/8 · 360 · 24² = 26000
84600 1#



motor trucks 3130 · 12.20/24.0 = 1590
9400
10990

Uniform load 8.92 × 203# = 1810#
1810 · 4.46/24.0 = say 340
Unif. load on sidewalks 360 · 24/2 = 4320
15650#

Summary for moments and shears

	Moment	Shear
Dead Load	43200	7200
Live Load	84600	15650
	127800 1#	22850#

Try 20 · 3/8" web = 7.50" 1/8 web = .94"
20½" btob of L's Effective depth say 1.54'
stress = 127800 ÷ 1.54 = 83000#
Section required = 83000 ÷ 17000 = 4.87
Void
less .94
3.93" net

Try 24 · 3/8" web = 9.00" 1/8 web = 1.12" 24½" btob of L's Effective depth = 1.86'
stress = 127800 ÷ 1.86 = 68500 Section Reqd = 68500 ÷ 17000 = 4.03
less .94
3.09" net

Use 1L 4 · 4 · 5/16" = 3.75" gross or 3.25" net Ofy

Approximate weights of stringer
2L³ 4 · 4 · 5/16 @ 12.8 = 25.6
1 web 2 4 · 3/8 = 30.6
56.2
Details say 5.8
62.0# per lin ft

Summary for Stringers

Fascia Girder		44
sidewalk Stringer		45
curb Stringer		62
Roadway Stringers	3 @ 70 =	210
Electric Ry Stringers	2 @ 85 =	170
		521# per lin ft of span for ½ width
	or	1042# " " " " "
	Call this	1050# per ft

増田淳氏関係資料
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CALCULATIONS FOR

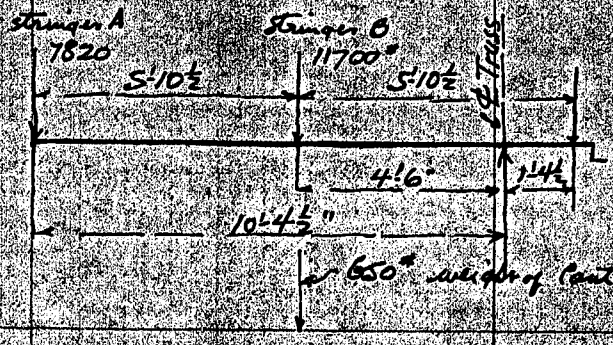
Preliminary Design of Chorotsu bashi for Kumamoto Ken

Design of Cantilever Bracket for Intermediate Floor Beam spacing 24'-0"

Dead Load

Dead Load concentration at A (Truss Girder)
See page 7 326 × 24 = 7820#

Dead Load Concentration at B
488 × 24 = 11700



Bending Moment at ℄ truss

7820 × 10.37 = 81000
11700 × 4.50 = 52600
650 × 4.50 = 2920
136520 lft

Live Load - Uniform Live Load 102 × 24 = 2448# per lin ft
moment at ℄ of truss = 2448 × $\frac{10.37^2}{2}$ = 132000 lft

water main 50 × 24 = 1200#
Moment at ℄ of truss = 1200 × 2.75 = 3300
135300
135300
Total moment 271820 lft

½ effective depth at truss = 40 assumed Str'no = 271820 ÷ 4 = 67800# SR = 67800 ÷ 17000 = 2.81"
web assumed 48" × ⅜" = 18.0" ℄ web = 2.25" 2.81 − 2.25 = 0.560" ok

Use 2L 3.3.⅜" for both top and bottom flanges -

Approximate weight of Cantilever bracket

1 web av. 38 × ⅜	@	48.45
4L 3.3.⅜	@ 7.2	28.80
		77.25
Details 25%		15.50
		92.75# × 10.37 = say 1000# per piece.

Design of Intermediate Floor Beam

Dead Load.

Dead Load of slab for Electric Ry tracks

4" stone blocks pavement	84
4½" mortar filling	52
6½" Concrete slab.	82
	188#
Dead Load beam 105 ÷ 5 = say	22
	210# per sq ft

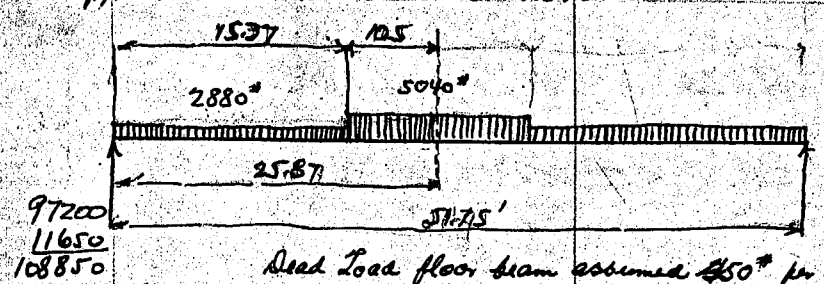
210 × 24 = 5040# per lin ft of span

Dead Load slab for Roadway

2" Asphalt Pavement	22
6" Concrete slab.	75
filler &c say	5
Beams say	18
	120# per sq ft

120 × 24 = 2880# per lin ft of span

Approximate Dead Load Moment



Dead Load floor beam assumed 450# per lin ft
Loading assumed as shown above for convenience's sake.

Moment at center
Reaction 5040 × 10.5 = 52900
2880 × 15.37 = 44300
97200

Moment 97200 × 25.87 = 2520000
52900 × 5.25 = 278000
44300 × 18.19 = 805000
less 1083000
1437,000 lft

Dead load beam ⅞ × 450 × 51.75² = 150500
1,587,500
Less cantilever brackets D.L.M. say 136,500
1,451,000 ft

Live Load Electric Ry loadings

coefficient of impact = $\frac{20}{60 + \frac{48}{27.8}}$ = 0.268

増田淳氏関係資料
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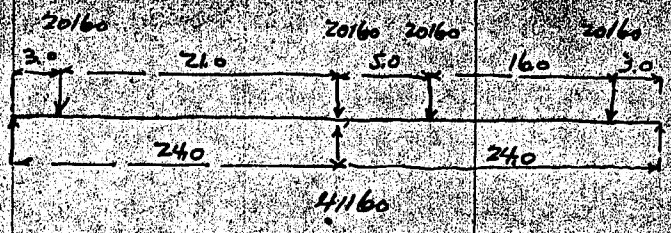
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CALCULATIONS FOR

Preliminary design of Chorofu Bridge for Tsumamoto-Ito

Electric railway loading



Concentration 20160 per truck

20160 × 25/24 = 21000

20160

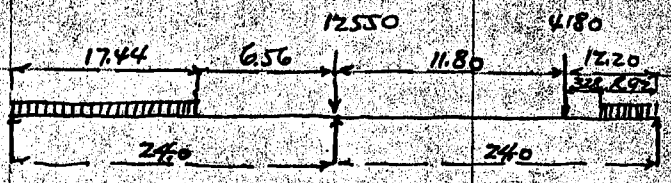
41160 # per truck

Impact 26.8% 11000

52160 #

26080 # per rail with impact

Motor trucks loading



Rear wheel concentration = 9900

Impact 26.8% 2650

12550 #

Front wheel conc 1/3 · 12550 4180 #

Reaction 4180 · 12.20/24.0 = 2120

12550

14670 # per wheel

Uniform load 123# per sq ft

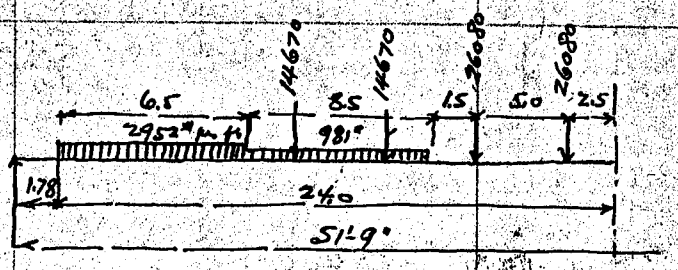
Reaction = 123 · 17.44²/(2 · 24.0) = 777

123 · 8.92²/(2 · 24.0) = 204

981 # per lin ft

123# Uniform load throughout

123 · 24. = 2952 # per lin ft



Moment due to concentration

81500 · 25.78 = 2,100,000

26080 · 10 = 260800

14670 · 26.50 = 388000

648.800

1,451,200 # = 1451.200

81500 conc.
27530 unif
109030

Reaction Uniform load

2952 · 6.5 = 19200

981 · 8.5 = 8330

27530

Moment due to Uniform load

27530 · 25.78 = 709000

8330 · 13.25 = 110,500

19200 · 2075 = 398.000

508500

200500 # 200500 #

1,651,700 #

Max End shear say 120.000# due to live load

Summary for moments and shear

	Moment	Shear
Dead Load	1.451.000	108850
Live Load	1.651.700	120000 assumed
	3.102,700 #	228850 #

web plate assumed 60 × 1/2 = 30.0" 5/8 web = 375" 60 1/2 · b to b of Ls Effective depth say 490'

Stress = 3102700 ÷ 490 = 633000 # R = 633000 ÷ 17000 = 37.20 − 3.75 = 33.45 0" net

Use	2 Ls 6×6 · 3/4	= 16.88 gr	13.88 0" net
	1 Pl. 12 1/2 · 5/8	7.80	6.57
	1 Pl. 12 1/2 · 5/8	7.80	6.57
	2 Pls. 6 × 3/4	9.00	7.50
		41.48 0"	34.52 0" net

Approximate weights of floor beam

Flanges 41.48 × 2 = 82.96

web 30.00

112.96

Details say 22% 24.80

137.76 @ 34 = call this 470# per lin ft

weight of one floor beam = 470 × 51.75 = 24400

weight of cant. brackets 2 @ 1000 = 2000

26400 # or 11.8 tons

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CALCULATIONS FOR

Preliminary Design of Choshubashi for Tamamoto-Ku.

Design of 2nd Floor Beam

Cantilever Bracket see sketch on page 9. concentration at A = $\frac{26.25^2}{2 \cdot 24}$ = 14.4 . 326 = 4700
conc. at B = 1.44 . 488 = 7000

Bending moment at L Truss 4700 . 10.37 = 48800
7000 . 4.50 = 31500
Dead load cantilever 650 . 4.50 = 2900
83200 lbs

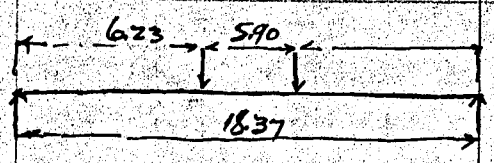
Live load moment $\frac{1440}{24.0} = 0.60$ 135300 . 0.60 = 81100
164300 lbs

Section of cantilever brackets same as for used for intermediate floor beam

Approximate weight of brackets say — 1000 # a piece.

Design of 2nd floor Beam under roadway. span length = 18'-4½"
Approximate Dead Load thru floor assumed 120 # per sq ft
Reaction on floor beam 120 . $\frac{26.25^2}{2 \cdot 24}$ = 1370 # per lin ft.
Dead Load moment as simple beam = ⅛ . 1370 . 18.37^2 = 57600 lbs
Less ½ cant moment 41600
add Dead Load beam → 16000 lbs
16000 lbs per rail

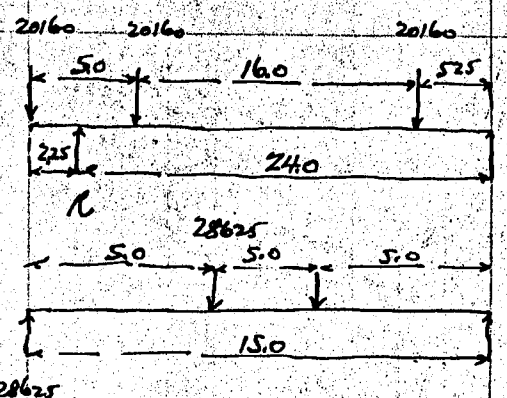
Live Load moment motor trucks loading say 15000 # per wheel including impact assumed



Live Load moment 15000 . 6.23 = 93500
add for unif load 16500
Live Load M. 110000 lbs
Dead Load m 16000
126000 lbs

Design of 2nd floor beam under Electric Ry tracks span length 15'0"
Dead Load. flooring 210 . $\frac{26.25^2}{2 \cdot 24}$ = 3020 # m = ⅛ . 3020 . 15^2 = 84700 lbs
Dead load beam m = ⅛ . 100 . 15^2 = 2300
87000 lbs

Live Load



Reaction R = 20160 . $\frac{52.75}{24.0}$ = 44250 #

Impact = $\frac{20}{60 + \frac{26.25}{3.28}}$ = 29.4% 13000
57250 per track
28625 per rail.

Live load moment = 28625 . 5.0 = 143.000
Dead load moment 87.000
230.000 lbs

Design End Floor beam for moment of 230.000 lbs
web assumed 36 . ⅜" = 13.50" ⅛ web = 1.69" 36½" c to c of flg Effective depth say 2.90'
Stress = 230.000 ÷ 2.90 = 79.200 # Section regd = 79200 ÷ 17000 = 4.66 - 1.69 = 2.97"
use 2 L's 3½ . 3½ . ⅜" = 496.0" gross or 421.0" net
approximate weights of floor beam

1 web pl.	36 . ⅜	45.9
4 L's	3½ . 3½ . ⅜ @ 8.5 =	42.5
		88.4
Details -	22%	20.0
		108.4 #

weight of floor beam 108.4 . 51.75 = 5610
" " cantilever Brackets 2@1000 = 2000
casting + c 1000
8610 #
3.85 tons per piece.

Approximate Bearing on shoe

Dead load from outer girder	3120 . 15/2 =	23400 #
" " " side girder	1470 . 9.20 =	13500
		36900 #
Live Load from outer girder		28625
" " " side girder including Unif load say		17000
		45625 #
		82525 # on cast shoe.

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CALCULATIONS FOR

Preliminary Design of Chorokutashi for Kumamoto-Ken

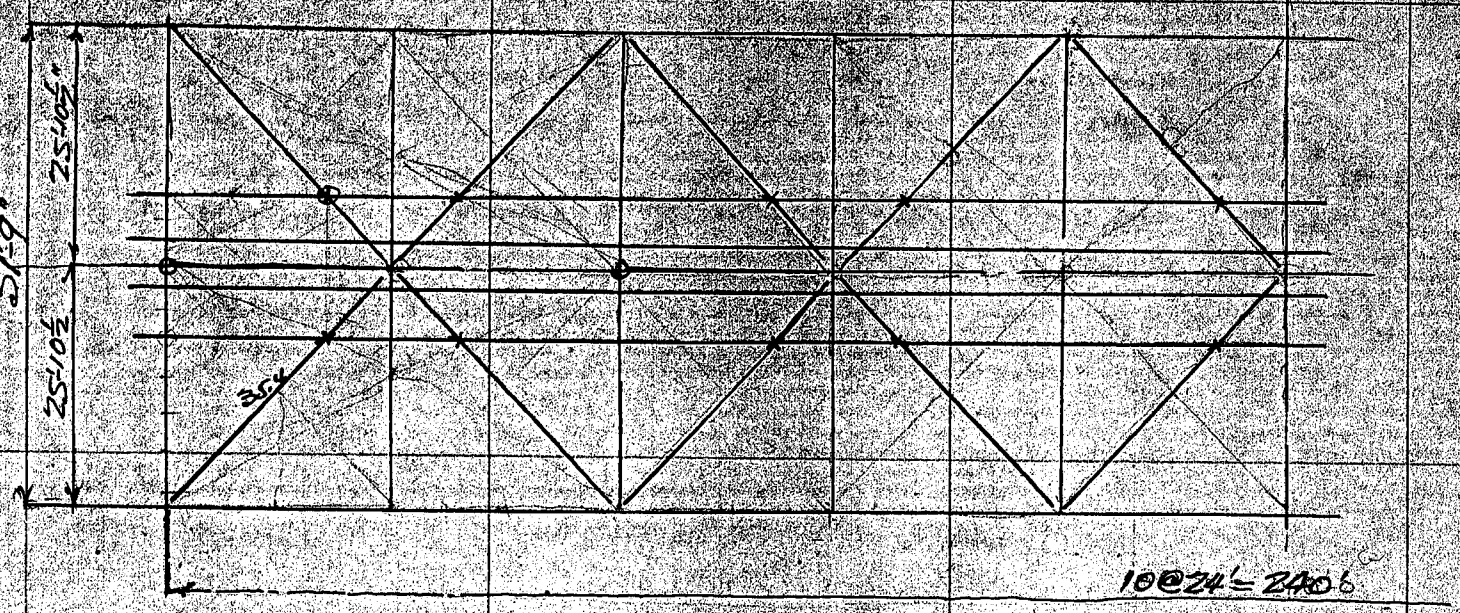
Bearing area required 82525 ÷ 640 = 129 □'

Use 18.18 about = 3240 □' as shown on sketch page 3.

Weight of metal in floor beams

Intermediate floor beam	9 @ 2640	=	23760
End Floor beams say	2 @ 9000	=	18000
			25560 # or 114 tons per span.

Lower Lateral Bracings



wind pressure on lower chord assumed as 300 # per lin ft treated as moving load.

$\sec\theta = \frac{35.4}{24.0} = 1.47$ Panel concentration 300 × 24 = 7200 #

End reaction = 7200 × 4.5 = 32400 #

Stress in diagonal at end panel = $\frac{32400}{2} \times 1.47$ = 23800 # tension or compression.

No of rivets required for end connection = 4.

L/r for wind bracing limited to 140 Unsupported length of member = 25.5 about

radius of gyration required = $\frac{25.5 \times 12}{140} = 2.19$

Use 2 Ls 6 × 4 × 3/8 = 12.3 = 24.6

1 PL. 8 × 3/8 = 10.2

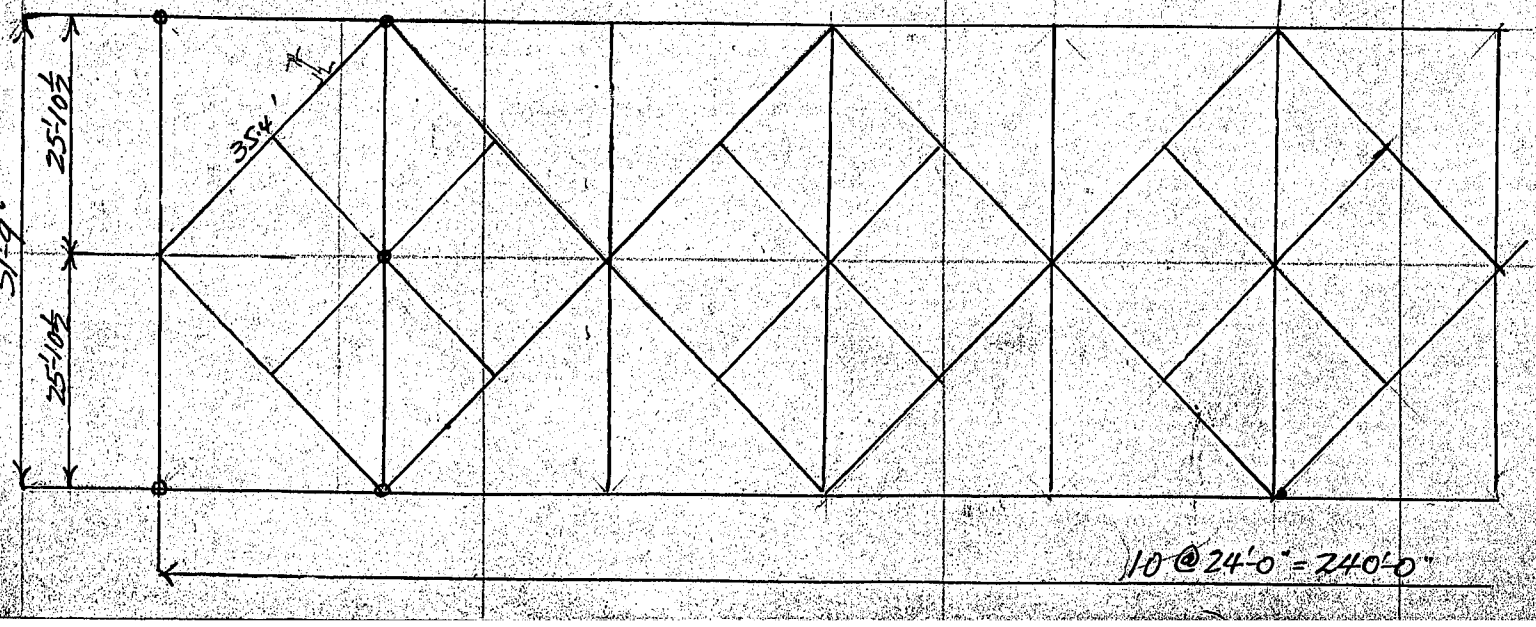
348 # per lin ft

Approximate weight of Lower Lateral Bracing.

Diagonal	34.8 × 33.5	=	1170 #
	20 @ 1170	=	23400
	20 @ 50	=	1000
	10 @ 120	=	1200
For details.			2000
			27600 #

27600 # ÷ 240 = 115 # per lin ft of span

Top Lateral Bracings.



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CALCULATIONS FOR

Preliminary Design of Chonotsubashi for Kumamoto-Ken.

Top Lateral Bracings

Transverse strut between trusses

Unsupported Length 25'-10½" = 310.5 l/r limited to 140 Radius of gyration rqd = 2.22

Use 4 ls 5 × 3½ × 3/8 laced between flanges.
6 × 4

Diagonals Unsupported length 35.4' ÷ 2 = 17.7' or 212" Radius of gyration rqd = 1.51

Use 4 ls 4 × 3 × 3/8 laced

Sub diagonals Unsupported length same as for diagonals

Use 2 ls 6 × 4 × 3/8 - T

Approximate weight of top Laterals.

Transverse strut	4 ls 6 × 4 × 3/8	@ 12.3#	× 500	=	2460#
	Lacing and details say				1000
					3460# per piece
Main diagonals	4 ls 4 × 3 × 3/8	@ 8.5	× 350	=	1200
	Lacing & details say				500
					1700# per piece
Subdiagonals	2 ls 6 × 4 × 3/8	@ 12.3	× 17.5	=	430
	Details say				20
					450# per piece

Summary for weight of top Laterals.

Transverse strut	9 @ 3460	=	31140#
Main diagonals	20 @ 1700	=	34000
Subdiagonals	20 @ 450	=	9000
Details for connection. say			10000
		84140#	350# per lin ft
Sway bracing at centr of span say		10000	
Portal Bracing	2 @ 10000	20000	
		114140#	475# per lin ft
		or 51 tons for one truss	

Summary for structural steel

weight of Stringers	1050 × 244.5	=	257000 - 115.0
weight of Floor beams			255600 - 114.0
Lower Laterals			27600 - 12.3
Upper Laterals & Sways			114140 - 51.0
			2923 tons

Dead Load of Floor (½ width)

stone blocks pavement	4"	@	54# sq ft	× 9.0	=	486
lean mortar filling	4½	@	52	× 9.0	=	469
6½" reinforced concrete slab.			81	× 9.5	=	770
2" asphalt pavement			22	× 13.5	=	297
6" reinforced concrete slab.			75	× 13.5	=	1010
Reinforced concrete gutter			100	× 1.5	=	150
Curb stone						67
Concrete under curb stone say						80
¾" wearing course on sidewalk slab			9	× 12.25	=	110
4" Reinforced concrete slab.			50	× 12.25	=	611
						4050#
water main say						150
conduit pipes say						40
Handrail say						60
						4300# per lin ft

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CALCULATIONS FOR

Preliminary Design of "Chorokubashi" for Kumamoto-Ken

Metal in bridge - 1 width

Stringers 1050# × 24 = 25200
Int. Floor beam 26400
51600 ÷ 24 = 2150#
Lower Laterals 115
Upper Laterals Sway + Portals 475
2740# ÷ 2 = 1370# per lin ft
Dead load floor 4300
trusses assumed 2300#
7970# per lin ft
or call this 8000# per lin ft of one truss

Panel concentration 8000 × 24.0 = 192000#
2300 · 35/14 × 12 = 17300
174700

Note: For conditions of quarter concentration of truss.
In calculation of stresses in members 13300# used, correction should be made in final design.

Live Load on truss.

Uniform load on sidewalks 100.000 / (170 + 240/328) = 411 kg/m² = 84.3 #/o'

Uniform load on Roadway 120.000 / (170 + 240/328) = 493 kg/m² = 101.0 #/o'

Uniform live load on truss — On sidewalks 84.3 · 12.25 = 1035
101.0 · 6.15 = 620
1655#

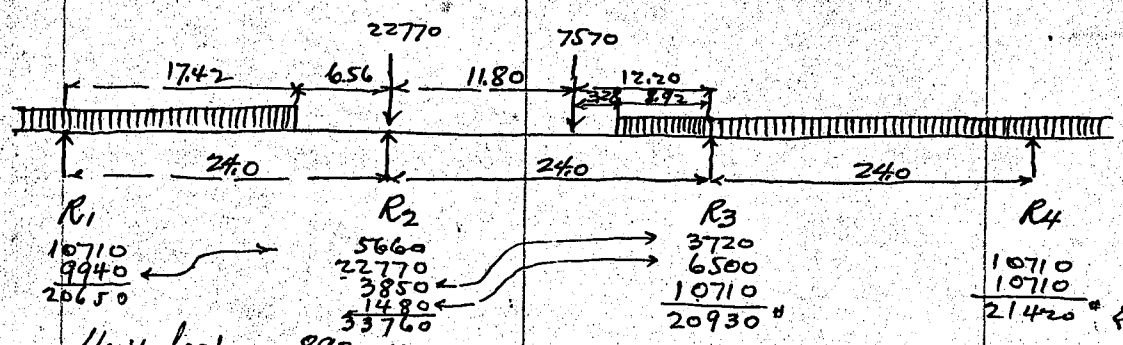
Panel concentration 1655 × 24 = 39700#

Uniform load conjunction with motor trucks loading
Occupied width of motor trucks assumed as 8.85' (270c)
Uniform load at front or rear of motor trucks 885 · 101 = 893#

Motor trucks loading
Impact = 20 / (60 + 240/3.28) = 15%

Rear wheel 9900
1485
11385# × 2 = 22770#
Front wheels 7570#
Total 30340#

Panel concentration

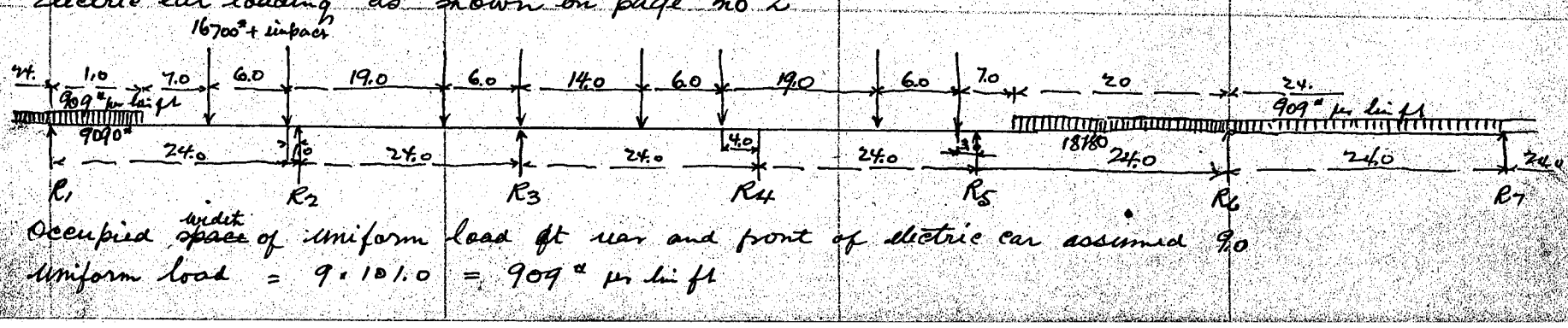


Panel conc in case of full unif. load on span

Unif. load 893 × 12 = 10710
" " 893 × 17.42 = 15600 — 15600 × 8.71/24.0 = 5660# — 15600 − 5660 = 9940
" " 893 × 8.92 = 7980 — 7980 × 4.46/24.0 = 1480 — 7980 − 1480 = 6500
Front wheel Concentration — 7570 × 12.20/24.0 = 3850 — 7570 − 3850 = 3720

For approximation max conc. R_2 as 34000#
and other conc assume as 21000# per panel.

Electric Ry loading
Coupled Electric cars with uniform load at rear and front
Electric car loading as shown on page no 2



Occupied width space of uniform load at rear and front of electric car assumed 9.0
Uniform load = 9 · 101.0 = 909# per lin ft

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CALCULATIONS FOR

Preliminary Design of Chorofutabashi for Kumamoto-Ken

Impact for Electric Car Loading, 15% comes for motor trucks loading,

Axel Concentration 16700 #
15% 2500
19200 #

Reaction R_1

M.L 9090 × 12/24 = 10900
Unif.L. 9090 × 19/24 = 7200
19200 × 8/24 = 6400
24500 #

Reaction R_2

Unif. load 9090 − 7200 = 1890
19200 × 40/24 = 32000
19200 × 6/24 = 4800
38690 #

Reaction R_3

19200 × 18/24 = 14400
= 19200
19200 × 14/24 = 11200
44800 #

Reaction R_4

19200 × 24/24 = 27200
19200 × 12/24 = 9600
36800 #

Reaction R_5

19200 × 36/24 = 28800
Unif load 18180 × 10/240 = 7560
36360 #

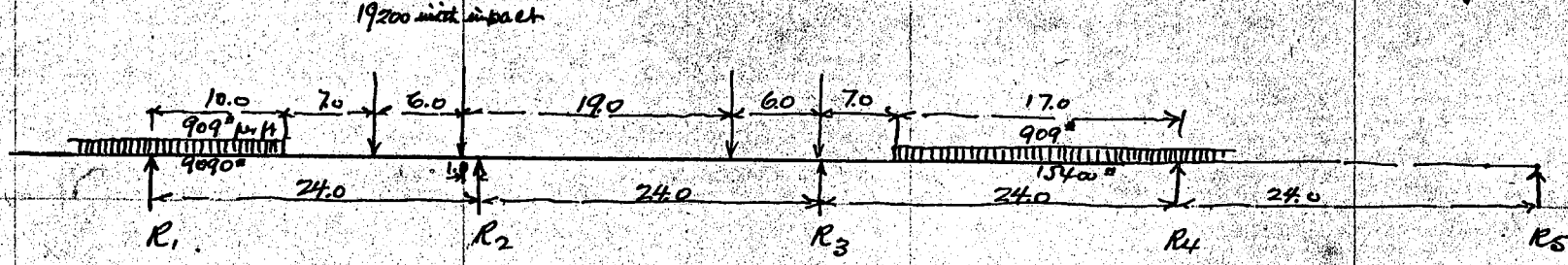
Reaction R_6

Unif. load 18180 − 7560 = 10620
909 × 12 = 10900
21520 #

Reaction R_7

Unif. load 909 × 24 = 21800 #

Single Electric Car loading, with uniform load at front and rear,



Reaction R_1

Unif. load 909 × 12 = 10900
" " 9090 × 19/24 = 7200
19200 × 8/24 = 6400
24500 #

Reaction R_2

Unif. load 9090 − 7200 = 1890
19200 × 40/24 = 32000
19200 × 6/24 = 4800
38690 #

Reaction R_3

19200 × 18/24 = 14400
Cone. 19200
Unif. 15400 × 8.5/24 = 5450
39050 #

R_4 −
Unif. 15400 × 15.5/24.0 = say 10000
Unif. 909 × 12 = 10900
20900 #

R_5 − 909 × 24 = 21800 #

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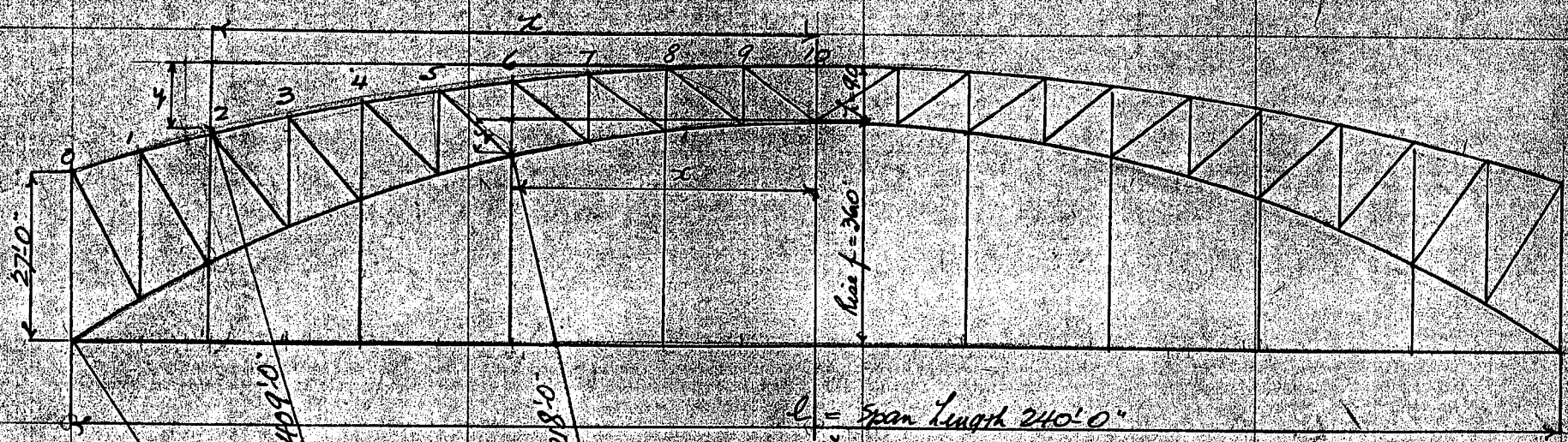
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CALCULATIONS FOR

Preliminary Design of Choshubashi for Kumamoto-Ken

Design of Main truss -

The span length - 240'-0" The floor panel will be divided into 10 panels and the truss members into 20. After preliminary sketch made and approved by several concerned in the bridge the rise of lower chord determined as f = 36' above horizontal line of the at the bearings and the depth of frame at the center line of bridge 9'-0" and 27'-0" at support. From facility of shop work the top and bottom chord members fixed as circular curves. The sketch of truss as shown below.



℄ = Span Length 240'-0"

Proportion for rise to span length and others

$\frac{\text{Height of truss at center}}{\text{span length}} = \frac{9'}{240} = \frac{1}{26.7}$

$\frac{\text{Rise of Lower Chord}}{\text{span length}} = \frac{1}{6.7}$

Radius for upper chord = 409.0'
Radius for lower chord = 218.0'

To find ordinates of panel points, the following is used

$\sin\theta = \frac{x}{R}$

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

Upper chord members
Radius of curvature R = 409'0"

$\frac{1}{R} = 0.00244949876$

$12 \cdot \frac{1}{R} = 0.029340$

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CALCULATIONS FOR

Preliminary Design of Chorokubashi for Kumamoto-Ken

Summary of ordinates from apex

Panel Pt	x in ft.	x:12	sinθ = x/R	Angle θ	Cosθ	1 − Cosθ	y = R(1−Cosθ)	
10	0	0	0.0000000	0° 00.000′	1.0000000	0.0000000	0.000	0
9	12	1	0.0293400	1° 40.878	0.9995695	0.0004305	0.176	0′-2⅛″
8	24	2	0.0586800	3° 21.843	0.9982769	0.0017231	0.705	0′-8 15/32″
7	36	3	0.0880200	5° 02.982	0.9961188	0.0038812	1.587	1′-7/16″
6	48	4	0.1173600	6° 44.386	0.9930894	0.0069106	2.826	2′-9 29/32″
5	60	5	0.1467000	8° 26.144	0.9891810	0.0108190	4.425	4′-5 3/32″
4	72	6	0.1760400	10° 08.352	0.9843830	0.0156170	6.387	6′-4 21/32″
3	84	7	0.2053800	11° 51.105	0.9786823	0.0213177	8.719	8′-8⅞″
2	96	8	0.2347200	13° 34.508	0.9720629	0.0279371	11.426	11′-5⅝″
1	108	9	0.2640600	15° 18.666	0.9645062	0.0354938	14.517	14′-6 3/16″
0	120	10	0.2934000	17° 03.698	0.9559897	0.0440103	18.000	18′-0″

Lower Chord Members
Radius = 218.0′ $\frac{1}{R}$ = 0.004587558 12 × $\frac{1}{R}$ = 0.0550459

Summary of ordinates from apex

Panel Pt	x in ft.	x:12	sinθ = x/R	Angle θ	Cosθ	1 − Cosθ	y = R(1−Cosθ)	
10	0	0	0.0000000	0° 0.000	1.0000000	0.0000000	0.000	0′-0″
9	12	1	0.0550459	3° 9.329	0.9984838	0.0015162	0.330	0′-3 31/32″
8	24	2	0.1100918	6° 19.236	0.9939214	0.0060786	1.325	1′-3 29/32″
7	36	3	0.1651377	9° 30.314	0.9862705	0.0137295	2.993	2′-11 29/32″
6	48	4	0.2201836	12° 43.189	0.9754585	0.0245415	5.350	5′-4 3/16″
5	60	5	0.2752295	15° 58.542	0.9613785	0.0386215	8.419	8′-5 1/32″
4	72	6	0.3302754	19° 17.130	0.9438846	0.0561154	12.233	12′-2 13/16″
3	84	7	0.3853213	22° 39.822	0.9227825	0.0772175	16.833	16′-10″
2	96	8	0.4403672	26° 07.640	0.8978177	0.1021823	22.276	22′-3 5/16″
1	108	9	0.4954131	29° 41.819	0.8686576	0.1313424	28.633	28′-7 19/32″
0	120	10	0.5504590	33° 23.910	0.8348623	0.1651377	36.000	36′-0″

Lengths of members of truss.
Verticals Vm

Panel point	+ Lower chord ordinates	− Upper chord ordinates	+ Height at center	Length of Verticals Vm
10	0′-0″	0′-0″	9′-0″	9′-0
9	0′-3 31/32″	0′-2⅛″	"	9′-1 27/32
8	1′-3 29/32″	0′-8 15/32″	"	9′-7/16″
7	2′-11 29/32″	1′-7/16″	"	10′-4 27/32
6	5′-4 3/16″	2′-9 29/32″	"	11′-6 9/32
5	8′-5 1/32″	4′-5 3/32″	"	12′-11 15/16″
4	12′-2 13/16″	6′-4 21/32″	"	14′-10 5/32
3	16′-10″	8′-8⅝″	"	17′-1⅜″
2	22′-3 5/16″	11′-5⅛″	"	19′-10 3/16
1	28′-7 19/32″	14′-6 3/16″	"	23′-1 13/32
0	36′-0″	18′-0″	"	27′-0″

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CALCULATIONS FOR

Preliminary Design of Choritsubashi for Tsumamoto Line

Lengths of Hangers Hm

Panel Pt	+	−	Hm
0	36'-0"	36'-0"	0'0"
1			
2	36'-0"	22'-3 5/16	13'-8 11/16
3			
4	36'-0"	12'-2 13/16	23'-9 3/16
5			
6	36'-0"	5'-4 3/16	30'-7 13/16
7			
8	36'-0"	1'-3 29/32	34'-8 3/32
9			
10	36'-0"	0'-0"	36'-0"

Lengths of Upper Chords Um

	A	Panel Length B	$\sqrt{A^2+B^2}$ = Um
18'-0"	− 14'-6 3/16	12'-0"	12'-5 15/16
14'-6 3/16	− 11'-5 1/8	"	12'-4 1/16
11'-5 1/8	− 8'-8 7/8	"	12'-3 7/8
8'-8 7/8	− 6'-4 21/32	"	12'-2 1/16
6'-4 21/32	− 4'-5 3/32	"	12'-1 29/32
4'-5 3/32	− 2'-9 29/32	"	12'-1 9/32
2'-9 29/32	− 1'-7 1/16	"	12'-0 3/4"
1'-7 1/16	− 0'-8 15/32	"	12'-0 3/8
0'-8 15/32	− 0'-2 1/8	"	12'-0 1/8
0'-2 1/8	− 0'-0"	"	12'-0"
0'-0"			

Lengths of Lower Chords Lm

Panel Pt	+	−	A	Panel Length B	$\sqrt{A^2+B^2}$ = Lm
0	36'-0"	28'-7 19/32	7'-4 13/32	12'-0"	14'-31/32
1	28'-7 19/32	22'-3 5/16	6'-4 9/32	"	13'-6 31/32
2	22'-3 5/16	16'-10	5'-5 5/16	"	13'-2 1/8
3	16'-10"	12'-2 13/16	4'-7 3/16	"	12'-10 7/32
4	12'-2 13/16	8'-5 1/32	3'-9 25/32	"	12'-7 3/32
5	8'-5 1/32	5'-4 3/16	3'-27/32	"	12'-4 5/8
6	5'-4 3/16	2'-11 29/32	2'-4 9/32	"	12'-2 3/4
7	2'-11 29/32	1'-3 29/32	1'-8"	"	12'-1 3/8
8	1'-3 29/32	0'-3 31/32	0'-11 15/16	"	12'-0 1/2
9	0'-3 31/32	0'-0"	0'-3 31/32	"	12'-1/16
10	0'-0"				

Lengths of Diagonals Dm

Panel Pt	+	−		A	Panel Length B	$\sqrt{A^2+B^2}$ = Dm
0	27'-0"	7'-4 13/32	=	19'-7 19/32	12'-0"	23'-0 1/8
1	23'-1 13/32	6'-4 9/32	=	16'-9 1/4	"	20'-7 3/8
2	19'-10 3/16	5'-5 5/16	=	14'-4 7/8	"	18'-9"
3	17'-1 3/8	4'-7 3/16	=	12'-6 3/16	"	17'-4 1/16
4	14'-10 5/32	3'-9 25/32	=	11'-0 3/8	"	16'-3 19/32
5	12'-11 1/16	3'-27/32	=	9'-11 3/32	"	15'-6 7/8
6	11'-6 9/32	2'-4 9/32	=	9'-2"	"	15'-1 7/32
7	10'-4 27/32	1'-8	=	8'-8 27/32	"	14'-10 1/8
8	9'-7 1/16	0'-11 15/16	=	8'-7 1/32	"	14'-9 11/32
9	9'-1 27/32	0'-3 31/32	=	8'-9 7/8	"	14'-10 23/32
10	9'-0	0'-0				

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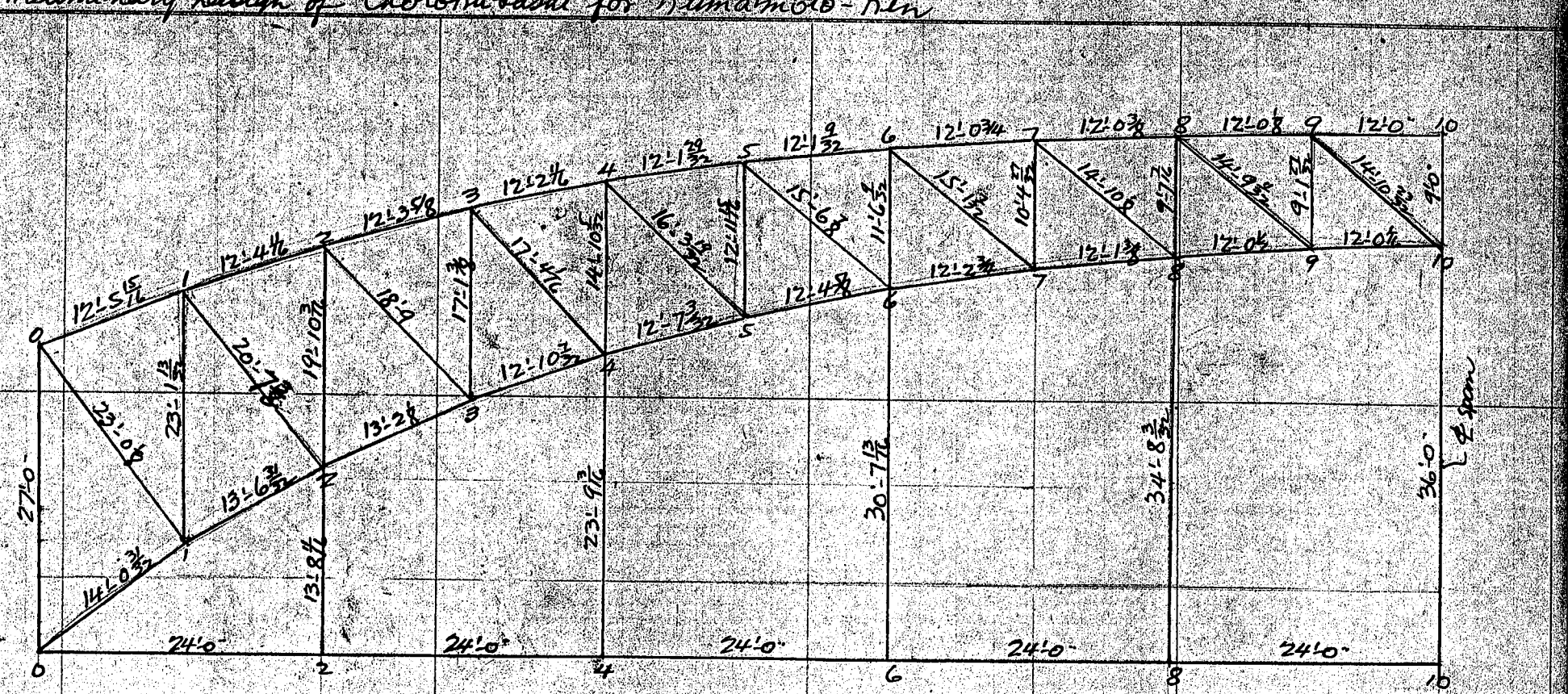
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CALCULATIONS FOR

Preliminary design of Chorokubashi for Kumamoto-Ken



Summary for lengths of members in decimals of ft and their logarithms

member	Upper chords Um.	Lower chords Lm.	Verticals Vm.	Diagonals Dm.	Hangers Hm.
0		14.0807 1.14863	27.0000 1.43136		
1	12.4948' 1.09673	13.5807 1.13292	23.1172 1.36393	23.0104 1.36192	7.3672
2	12.3906' 1.09309	13.1771 1.11982	19.8490 1.29774	20.6224 1.31434	13.7240
3	12.3021' 1.08998	12.8516 1.10896	17.1146 1.23337	18.7500 1.27300	19.1667
4	12.2240' 1.08721	12.5911 1.10007	14.8464 1.17162	17.3385 1.23901	23.7656
5	12.1589' 1.08489	12.3854 1.09291	12.9948 1.11377	16.2995 1.21217	27.5807
6	12.1068' 1.08303	12.2292 1.08740	11.5234 1.06158	15.5729 1.19237	30.6172
7	12.0625' 1.08144	12.1146 1.08331	10.4036 1.01719	15.1016 1.17902	33.0078
8	12.0313' 1.08031	12.0417 1.08069	9.6198 0.98317	14.8438 1.17154	34.6745
9	12.0104' 1.07956	12.0052 1.07937	9.1536 0.96159	14.7786 1.16963	35.6693
10	12.0000' 1.07918		9.0000 0.95424	14.8932 1.17299	36.0000

Notation :-

a = panel length of truss

q_m = perpendicular distance from lower panel point m to upper chord U_{m-1} to U_m

h_m = " " " upper " " m to lower chord L_{m-1} to L_m

f_m = perpendicular distance from F (Intersection of chord members U_{m-1}-U_m and L_{m-1}-L_m) to the diagonal D_m

l_m = horizontal distance from F to left support

i_m = " " from I (Intersection of U_{m-1}-U_m and L_m-L_{m+1}) to the left support

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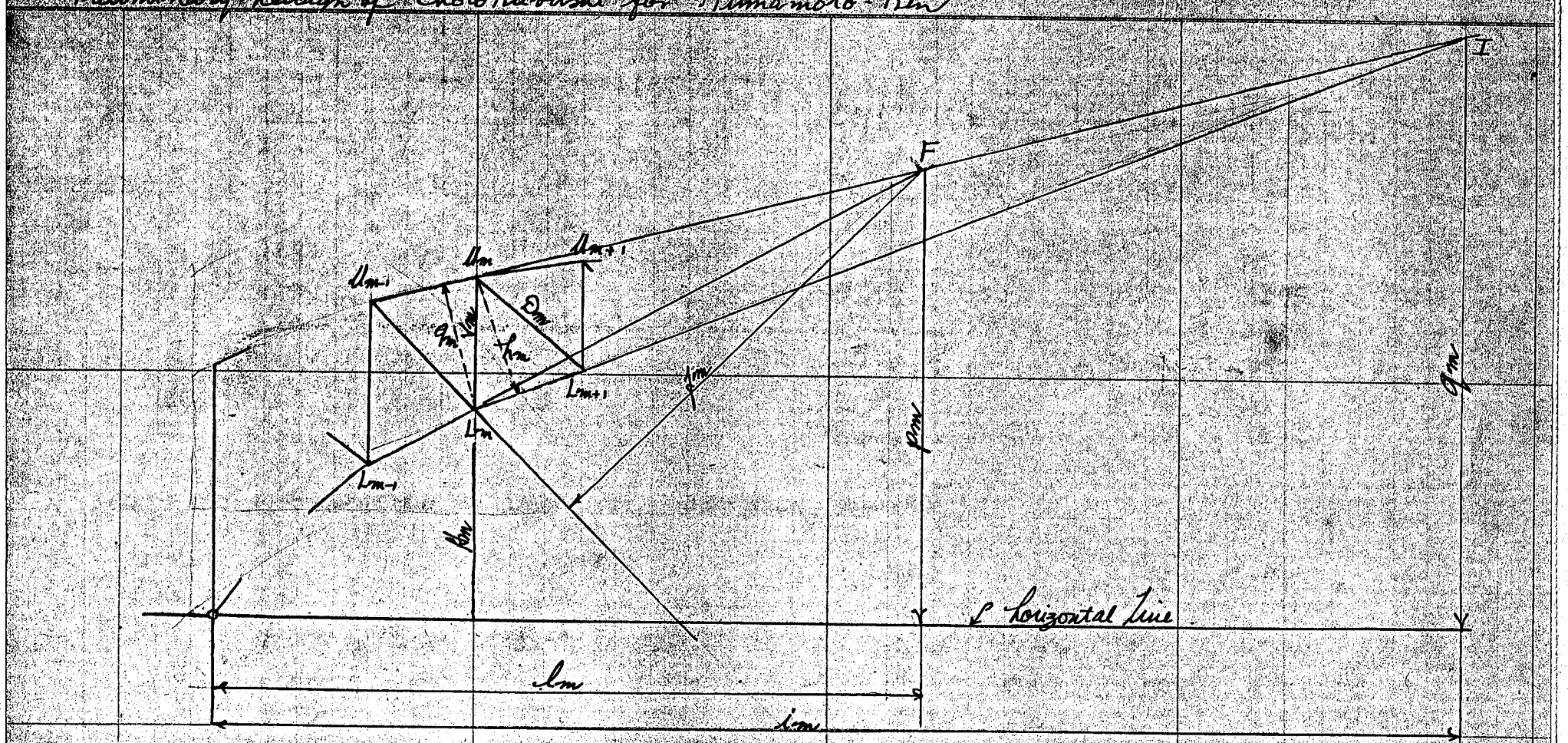
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CALCULATIONS FOR

Preliminary Design of Chorotabashi for Kumamoto-Ken



Lever Arm $q_m = \frac{a}{U_m} \times V_m$

Lever Arm $h_m = \frac{a}{L_m} \times V_m$

moment Arm for diagonal $f_m = \frac{V_{m-1}}{V_{m-1} - V_m} \times \frac{a}{D_m} \times V_m$

Dist. intersection Point F from end support $= l_m = \frac{V_{m-1}}{V_{m-1} - V_m} \times a + (m-1)a$

Dist intersection Point I from end support $i_m = \frac{V_m}{(H_{m+1} - H_m) - (y_m - y_{m-1})} \times a + ma$

where $y_m = H_m + V_m$

Dist. intersection Point F from horizontal tie $p_m = \frac{V_{m-1}}{V_{m-1} - V_m} \times (H_m - H_{m-1}) + H_{m-1}$

Dist. intersection Point I from horizontal tie $q_m = \frac{V_m}{(H_{m+1} - H_m) - (y_m - y_{m-1})} \times (H_{m+1} - H_m) + H_m$

From above formulas the respective values figured and tabulated below:—

m	q_m	h_m	f_m	l_m	i_m	ma	$i_m - ma$
1	22.20′	20.43′	83.83′	83.44′	108.58′	12′	96.58′
2	19.22	18.08	81.70	96.88	125.18	24	101.18
3	16.70	15.98	79.51	111.11	144.63	36	108.63
4	14.58	14.15	77.53	126.55	168.02	48	120.02
5	12.81	12.59	76.71	144.22	200.90	60	140.90
6	11.42	11.31	78.42	165.98	254.48	72	182.48
7	10.35	10.30	85.07	195.49	374.55	84	290.55
8	9.59	9.59	103.22	243.27	1126.90	96	1030.90
9	9.15	9.15	153.39	343.65	−447.00	108	−555.00
10	9.00	—	432.02	822.91	———	120	———

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CALCULATIONS FOR

Preliminary Design of Chorokubashi for Kumamoto-Ken

m	ma÷gm	ma÷hm	lm÷fm	im÷(lm-ma)	Hm+Vm	lm-ma	12÷fm	Pm	gm
1	0.94	0.59	0.99	1.12	30.48	83.44	0.143	51.19	58.34
2	1.25	1.33	1.18	1.24	33.57	84.88	0.147	52.14	59.25
3	2.15	2.25	1.40	1.33	36.28	87.11	0.151	52.87	60.29
4	3.29	3.39	1.63	1.40	38.61	90.55	0155	53.36	61.29
5	4.68	4.76	1.88	1.42	40.58	96.22	0.157	53.72	68.75
6	6.30	6.37	2.12	1.39	42.14	105.98	0.153	53.95	62.88
7	7.40	8.16	2.30	1.29	43.41	123.49	0.141	54.03	72.46
8	10.01	10.01	2.36	1.09	44.29	159.27	0.116	54.27	119.17
9	11.80	11.80	2.24	0.81	44.82	247.65	0.078	54.42	19.46
10	13.33	—	1.90	—	45.00	714.91	0.028	54.46	—

Upper chord stresses due to unit load (redundancy removed)

load on / member	U_1	U_2	U_3	U_4	U_5	U_6	U_7	U_8	U_9	U_{10}
1	0.027	0.062	0.108	0.165	0.234	0.315	0.406	0.501	0.590	0.667
2	0.054	0.125	0.215	0.329	0.469	0.630	0.812	1.002	1.180	1.333
3	0.081	0.187	0.323	0.494	0.703	0.945	1.217	1.502	1.770	2.000
4	0.108	0.250	0.431	0.658	0.937	1.260	1.623	2.003	2.360	2.667
5	0.135	0.312	0.539	0.823	1.171	1.575	2.028	2.504	2.950	3.333
6	0.162	0.375	0.646	0.987	1.405	1.890	2.434	3.005	3.540	4.000
7	0.189	0.437	0.754	1.152	1.639	2.205	2.840	3.505	4.130	4.667
8	0.216	0.500	0.862	1.317	1.874	2.520	3.245	4.006	4.720	5.333
9	0.243	0.562	0.970	1.482	2.107	2.835	3.652	4.506	5.310	6.000
10	0.270	0.624	1.077	1.646	2.342	3.150	4.058	5.007	5.900	6.667
9'	0.297	0.687	1.185	1.810	2.578	3.465	4.464	5.508	6.490	6.000
8'	0.324	0.750	1.293	1.975	2.812	3.780	4.870	6.009	5.769	5.333
7'	0.351	0.812	1.400	2.140	3.040	4.095	5.275	5.258	5.048	4.667
6'	0.378	0.874	1.508	2.304	3.280	4.410	4.522	4.507	4.327	4.000
5'	0.405	0.937	1.616	2.468	3.515	3.675	3.768	3.756	3.606	3.333
4'	0.432	1.000	1.723	2.633	2.812	2.940	3.014	3.005	2.884	2.667
3'	0.459	1.061	1.832	1.985	2.109	2.205	2.261	2.253	2.163	2.000
2'	0.486	1.123	1.221	1.327	1.406	1.470	1.507	1.502	1.442	1.333
1'	0.513	0.562	0.611	0.658	0.703	0.735	0.754	0.751	0.721	0.667

member / load on	1	2	3	4	5	6	7	8	9	10
U_1	0.027	0.054	0.081	0.108	0.135	0.162	0.189	0.216	0.243	0.270
U_2	0.062	0.125	0.187	0.250	0.312	0.375	0.437	0.500	0.562	0.624
U_3	0.108	0.215	0.323	0.431	0.539	0.646	0.754	0.862	0.970	1.077
U_4	0.165	0.329	0.494	0.658	0.823	0.987	1.152	1.317	1.482	1.646
U_5	0.234	0.469	0.703	0.937	1.171	1.405	1.639	1.874	2.107	2.342
U_6	0.315	0.630	0.945	1.260	1.575	1.890	2.205	2.520	2.835	3.150
U_7	0.406	0.812	1.217	1.623	2.028	2.434	2.840	3.245	3.652	4.058
U_8	0.501	1.002	1.502	2.003	2.504	3.005	3.505	4.006	4.506	5.007
U_9	0.590	1.180	1.770	2.360	2.950	3.540	4.130	4.720	5.310	5.900
U_{10} & U_{10}'	0.667	1.333	2.000	2.666	3.333	4.000	4.667	5.333	6.000	6.667
U_9'	0.721	1.442	2.163	2.884	3.606	4.327	5.048	5.769	6.490	5.900
U_8'	0.751	1.502	2.253	3.005	3.756	4.507	5.258	6.009	5.508	5.007
U_7'	0.754	1.507	2.261	3.014	3.768	4.522	5.275	4.870	4.464	4.058
U_6'	0.735	1.470	2.205	2.940	3.675	4.410	4.095	3.780	3.465	3.150
U_5'	0.703	1.406	2.109	2.812	3.515	3.280	3.040	2.812	2.578	2.342
U_4'	0.658	1.327	1.985	2.633	2.468	2.304	2.140	1.975	1.810	1.646
U_3'	0.611	1.221	1.832	1.723	1.616	1.508	1.400	1.293	1.185	1.077
U_2'	0.562	1.123	1.061	1.000	0.937	0.874	0.812	0.750	0.687	0.624
U_1'	0.513	0.486	0.459	0.432	0.405	0.378	0.351	0.324	0.297	0.270

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CALCULATIONS FOR

Preliminary Design of Choro-tu-tashi for Kumamoto-Kin

Lower Chord stresses due to unit load

Member / Load on	L_0	L_1	L_2	L_3	L_4	L_5	L_6	L_7	L_8	L_9
1	0.095	0.029	0.066	0.113	0.170	0.238	0.318	0.408	0.501	0.590
2	0.190	0.059	0.133	0.225	0.339	0.476	0.637	0.816	1.002	1.180
3	0.285	0.088	0.199	0.338	0.509	0.714	0.955	1.223	1.502	1.770
4	0.380	0.117	0.265	0.450	0.678	0.952	1.273	1.632	2.003	2.360
5	0.475	0.147	0.332	0.563	0.848	1.191	1.592	2.038	2.504	2.950
6	0.570	0.176	0.398	0.676	1.018	1.428	1.910	2.447	3.005	3.540
7	0.665	0.206	0.464	0.788	1.187	1.667	2.228	2.854	3.505	4.130
8	0.760	0.235	0.531	0.901	1.357	1.905	2.547	3.262	4.006	4.720
9	0.855	0.264	0.597	1.013	1.527	2.143	2.866	3.670	4.506	5.310
10	0.950	0.294	0.664	1.126	1.697	2.382	3.184	4.076	5.007	5.900
9'	1.045	0.323	0.730	1.238	1.867	2.619	3.502	4.485	5.508	6.490
8'	1.140	0.352	0.798	1.352	2.036	2.858	3.820	4.892	6.009	5.769
7'	1.235	0.382	0.863	1.464	2.205	3.096	4.140	5.300	5.258	5.048
6'	1.330	0.411	0.929	1.577	2.374	3.334	4.456	4.543	4.507	4.327
5'	1.425	0.440	0.995	1.689	2.544	3.572	3.713	3.786	3.756	3.606
4'	1.520	0.470	1.062	1.802	2.714	2.858	2.971	3.028	3.005	2.884
3'	1.615	0.499	1.128	1.915	2.036	2.143	2.228	2.271	2.253	2.163
2'	1.710	0.528	1.194	1.277	1.357	1.429	1.485	1.514	1.502	1.442
1'	1.805	0.558	0.597	0.638	0.679	0.714	1.743	0.757	0.751	0.721

Load on / member	1	2	3	4	5	6	7	8	9	10
L_0	0.095	0.190	0.285	0.380	0.475	0.570	0.665	0.760	0.855	0.950
L_1	0.029	0.059	0.088	0.117	0.147	0.176	0.206	0.235	0.264	0.294
L_2	0.066	0.133	0.199	0.265	0.332	0.398	0.464	0.531	0.597	0.664
L_3	0.113	0.225	0.338	0.450	0.563	0.676	0.788	0.901	1.013	1.126
L_4	0.170	0.339	0.509	0.678	0.848	1.018	1.187	1.357	1.527	1.697
L_5	0.238	0.476	0.714	0.952	1.199	1.428	1.667	1.905	2.143	2.382
L_6	0.318	0.637	0.955	1.273	1.592	1.910	2.228	2.547	2.866	3.184
L_7	0.408	0.816	1.223	1.632	2.038	2.447	2.854	3.262	3.670	4.076
L_8	0.501	1.002	1.502	2.003	2.504	3.005	3.505	4.006	4.506	5.007
L_9	0.590	1.180	1.770	2.360	2.950	3.540	4.130	4.720	5.310	5.900
L_9'	0.721	1.442	2.163	2.884	3.606	4.327	5.048	5.769	6.490	5.900
L_8'	0.751	1.502	2.253	3.005	3.756	4.507	5.258	6.009	5.508	5.007
L_7'	0.757	1.514	2.271	3.028	3.786	4.543	5.300	4.892	4.485	4.076
L_6'	0.743	1.485	2.228	2.971	3.713	4.456	4.140	3.820	3.502	3.184
L_5'	0.714	1.429	2.143	2.858	3.572	3.334	3.096	2.858	2.619	2.382
L_4'	0.679	1.357	2.036	2.714	2.544	2.374	2.205	2.036	1.867	1.697
L_3'	0.638	1.277	1.915	1.802	1.689	1.577	1.464	1.352	1.238	1.126
L_2'	0.597	1.194	1.128	1.062	0.995	0.929	0.863	0.797	0.730	0.664
L_1'	0.558	0.528	0.499	0.470	0.440	0.411	0.382	0.352	0.323	0.294
L_0'	0.805	0.710	1.615	1.526	1.478	0.336	1.235	1.140	1.045	0.952

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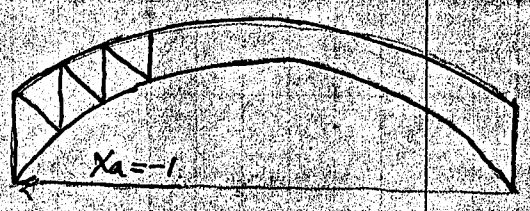
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CALCULATIONS FOR

Preliminary Design of Etorokubashi for Kumamoto-Ken

Upper and Lower chord stresses due to horizontal force Xa = −1 acting at support



$$\text{Upper chord } Um = -\frac{Xa \cdot Hm}{gm} = -\frac{Hm}{gm}$$

$$\text{Lower chord } Lm = \frac{Xa\,ym}{hm} = +\frac{ym}{hm} = +\frac{Hm + Vm}{hm}$$

Panel Pt	Upper chord	Lower chord
0		1.173
1	0.332	1.492
2	0.714	1.857
3	1.147	2.268
4	1.630	2.728
5	1.997	3.220
6	2.580	3.725
7	3.188	4.214
8	3.615	4.618
9	3.900	5.000
10	4.000	

$$Xa = \frac{\Sigma So\,Sa\,L}{\Sigma Sa^2 L + \frac{lt}{Et \cdot At}}$$

where Xa = Horizontal thrust
So = Stress due to vertical load unit, considering the arch as simple span
Sa = Stress due to a horizontal force unit, acting in the line of horizontal tie.
L = Length of member
lt = length of horizontal tie
At = Cross section of tie
Et = modulus of elasticity of steel

Upper chord Stress.

member	Sa	L.	Sa L	So		So Sa L	$\frac{So\,Sa\,L}{0.112}$	$Sa^2 L$
1	0.332	12.49	4.15	0.027	A	0.112	1.00	1.379
2	0.714	12.39	8.86	0.062	B	0.549	4.9	6.326
3	1.147	12.30	14.12	0.108	C	1.525	13.6	16.196
4	1.630	12.22	19.92	0.165	D	3.287	29.4	32.470
5	1.997	12.16	24.30	0.234	E	5.686	50.8	48.529
6	2.580	12.11	31.24	0.315	F	9.841	87.8	80.599
7	3.188	12.06	38.45	0.406	G	15.611	139.4	122.581
8	3.615	12.03	43.49	0.501	H	21.788	191.0	157.216
9	3.900	12.01	46.84	0.590	K	26.636	237.8	182.676
10	4.000	12.00	48.00	0.667	L	32.016	285.8	192.000
10'	———		48.00	0.667	L	32.016	285.8	Σ=839.97 + 2
9'	———		46.84	0.721 11/9	K	33.772	301.5	= 1679.94
8'	———		43.49	0.751 12/8	H	32.661	291.6	
7'	———		38.45	0.754 13/7	G	28.991	258.8	
6'	———		31.24	0.735 14/6	F	22.961	205.0	
5'	———		24.30	0.703 15/5	E	17.083	152.5	
4'	———		19.92	0.658 16/4	D	13.107	117.0	
3'	———		14.12	0.611 17/3	C	8.627	76.8	
2'	———		8.86	0.562 18/2	B	4.979	44.4	
1'	———		4.15	0.513 19	A	2.129	19.0	

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CALCULATIONS FOR

Preliminary Design of Chorokubashi for Kumamoto-Ken

Upper Chord

number	mark	l	Summation	multiple for respective load point 2	3	4	5	6	7	8	9	10
1	A	1.0	1.0	2	3	4	5	6	7	8	9	10
2	B	4.9	5.9	2	3	4	5	6	7	8	9	10
3	C	13.6	19.5	2	3	4	5	6	7	8	9	10
4	D	29.4	48.9	2	3	4	5	6	7	8	9	10
5	E	50.8	99.7	2	3	4	5	6	7	8	9	10
6	F	87.8	187.5	2	3	4	5	6	7	8	9	10
7	G	139.4	326.9	2	3	4	5	6	7	8	9	10
8	H	191.0	517.9	2	3	4	5	6	7	8	9	10
9	K	237.8	755.7	2	3	4	5	6	7	8	9	10
10	L	285.8	1041.5	2	3	4	5	6	7	8	9	10
10'	M	285.8	1327.3	2	3	4	5	6	7	8	9	10
9'	N	301.5	1628.8	2	3	4	5	6	7	8	11K	
8'	O	291.6	1920.4	2	3	4	5	6	7	12H	11H	
7'	P	258.8	2179.2	2	3	4	5	6	13G	12G	11G	
6'	Q	205.0	2384.2	2	3	4	5	14F	13F	12F	11F	
5'	R	152.5	2536.7	2	3	4	15E	14E	13E	12E	11E	
4'	S	117.0	2653.7	2	3	16D	15D	14D	13D	12D	11D	
3'	T	76.8	2730.5	2	17C	16C	15C	14C	13C	12C	11C	
2'	U	44.4	2774.9	18B	17B	16B	15B	14B	13B	12B	11B	
1'	V	19.0	2793.9	18A	17A	16A	15A	14A	13A	12A	11A	10

Load on

							$\frac{\Sigma S_0 S_a L}{0.112}$	$\Sigma S_0 S_a L$	
U1		2793.9	= 2793.9				2793.7	312.917	
U2	2 ×	2730.5	= 5461.0	5.9 ×	18	= 111.2	5572.2	624.086	
U3	3 ×	2653.7	= 6961.1	19.5 ×	17	= 331.5	7292.6	816.771	
U4	4 ×	2536.7	= 10146.8	48.9 ×	16	= 782.4	10929.2	1224.070	
5	5 ×	2384.2	= 11921.0	99.7 ×	15	= 1495.5	13416.5	1502.748	
6	6 ×	2179.2	= 13075.2	187.5 ×	14	= 2625.0	15700.2	1758.422	
7	7 ×	1920.4	= 13442.8	326.9 ×	13	= 4249.7	17692.5	1981.560	
8	8 ×	1628.8	= 13030.4	517.9 ×	12	= 6214.8	19245.2	2156.462	
9	9 ×	1327.3	= 11945.7	755.7 ×	11	= 8312.7	20258.4	2269.053	
10	10 ×	1041.5	= 10415.0	1041.5 ×	10	= 10415.0	20830.0	2332.960	

Lower chord

member	S_a	L	$S_a L$	S_0	$S_a S_0 L$		Summation	$S_a^2 L$
0	1.173	14.08	16.52	0.095	1.570	A	0.43 + 1.140	19.378
1	1.492	13.58	20.26	0.029	0.59	B	1.02 + "	30.228
2	1.857	13.18	24.48	0.066	1.62	C	2.64 + "	45.459
3	2.268	12.85	29.14	0.113	3.29	D	5.93 + "	66.090
4	2.728	12.59	34.35	0.170	7.54	E	13.47 + "	93.707
5	3.220	12.39	39.90	0.238	9.50	F	22.97 + "	128.478
6	3.725	12.23	45.56	0.318	14.49	G	37.46 + "	169.711
7	4.214	12.11	51.03	0.408	20.82	H	58.28 + "	215.040
8	4.618	12.04	55.60	0.501	27.86	K	86.14 + "	256.768
9	5.000	12.00	60.00	0.590	35.40	L	121.54 + "	300.000
9			60.00	0.721	43.26	M	164.80 + "	1324.859 × 2
8			55.60	0.751	41.76	N	206.56 + "	= 2649.718
7			51.03	0.757	38.63	O	245.19 + "	
6			45.56	0.743	33.85	P	279.04 + "	
5			39.90	0.714	28.49	Q	307.53 + "	
4			34.35	0.679	23.32	R	330.85 + "	
3			29.14	0.638	18.59	S	349.44 + "	
2			24.48	0.597	14.61	T	364.05 + "	
1			20.26	0.558	11.31	U	375.36 + "	
0			16.52	0.805	29.82	V	383.57 + 22.750	
					29.82			

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CALCULATIONS FOR

Preliminary design of Chōrokubashi for Kumamoto Ken

Load on						ΣSoSaL	油ヒ
1		383.57				383.57	add 22,750
2	2 × 364.05	= 728.10	1.02 × .18	= 18.36		746.46	+ 24,800
3	3 × 349.44	= 1148.32	2.64 × .17	= 34.88		1083.20	+ 22,800
4	4 × 330.85	= 1323.40	5.93 × .16	= 94.88		1418.28	+ 22,800
5	5 × 307.53	= 1537.65	13.47 × .15	= 202.05		1739.70	+ 22,800
6	6 × 279.04	= 1674.24	22.97 × .14	= 321.58		1995.82	+ 〃
7	7 × 245.19	= 1716.34	37.46 × .13	= 486.98		2203.32	+ 〃
8	8 × 206.56	= 1652.48	58.28 × .12	= 699.36		2351.84	+ 〃
9	9 × 164.80	= 1483.20	86.14 × .11	= 947.54		2430.74	+ 〃
10	10 × 121.54	= 1215.40	121.54 × .10	= 1215.40		2430.80	+ 22,800

Note!! Horizontal thrust should be corrected according to the corrections above; However on account of preliminary design the old figures used to find stress in member

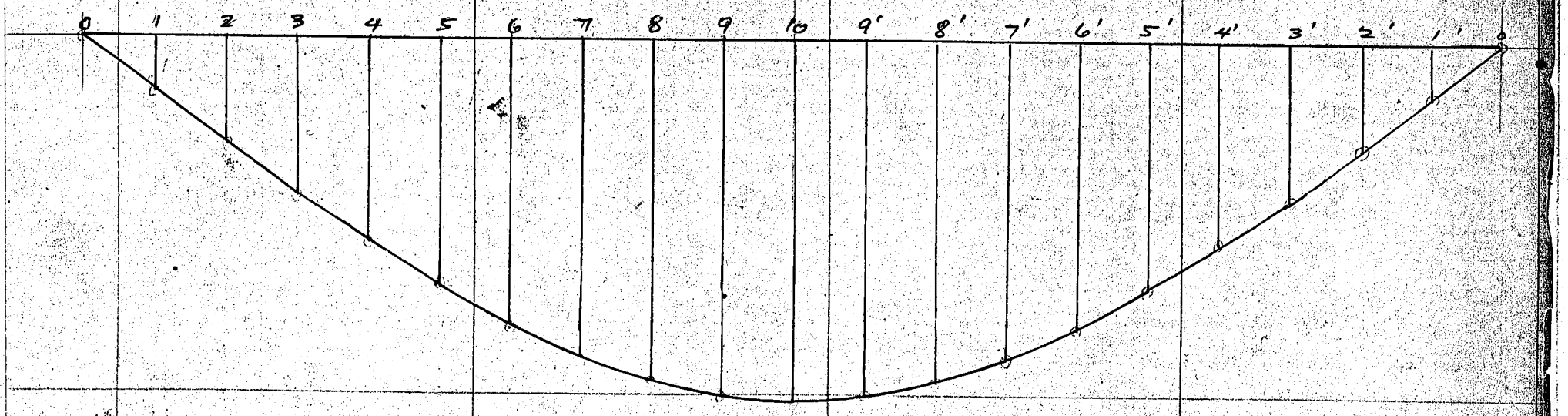
Assumed proportion of chord members and tie in gross section.
Upper chord : lower chord : tie = 1.0 : 0.7 : 1.0

$\Sigma Sa\frac{L}{At} + \frac{lt}{At}$:— For upper chord = 1 — 1679.94
For lower chord 2649.72 ÷ 0.7 = 3785.30
For tie 240 ÷ 1.0 = 240.00
5705.24

ΣSoSaL

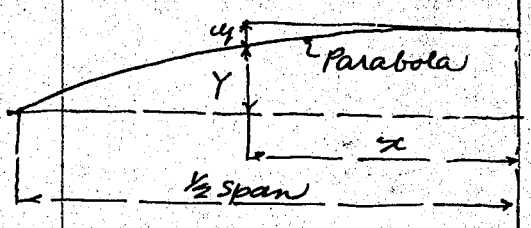
Load on	1	2	3	4	5	6	7	8	9	10
Upper chord: 1.0	312.92	624.09	816.77	1224.07	1502.75	1758.42	1981.56	2156.46	2269.05	2332.96
Lower chord: 0.7	547.96	1066.37	1690.29	2026.13	2485.30	2851.17	3147.60	3359.77	3472.50	3472.57
	860.88	1690.46	2507.06	3250.20	3988.05	4609.59	5129.16	5516.23	5741.55	5805.53
divided by 5705.24	0.151	0.296	0.439	0.570	0.699	0.808	0.899	0.967	1.007	1.017

Influence surface for Xa



Camber of tie 1:10 at ℄ span ; the intermediate panel point assumed on parabola

Panel Pt	dist from ℄ span	y	Y
10	0	0	1.00
9	12	0.01	0.99
8	24	0.04	0.96
7	36	0.09	0.91
6	48	0.16	0.84
5	60	0.25	0.75
4	72	0.36	0.64
3	84	0.49	0.51
2	96	0.64	0.36
1	108	0.81	0.19
0	120	1.00	0.00



$y = ax^2$

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CALCULATIONS FOR

Preliminary Design of Chorokubashi for Kumamoto-Ken

Stress for upper chord $U_m = U_{mo} - X_a \frac{H_m - Y}{g_m}$

Panel P.	H_m	Y.	$H_m - Y$	X_a	g_m	$X_a \frac{H_m-Y}{g_m}$	$\frac{H_m-Y}{g_m}$
1	7.37	0.19	7.18	0.151	22.20	0.05	0.32
2	13.72	0.36	13.36	0.296	19.22	0.21	0.70
3	19.17	0.51	18.66	0.439	16.70	0.49	1.12
4	23.77	0.64	23.13	0.570	14.58	0.90	1.59
5	27.58	0.75	26.83	0.699	12.81	1.47	2.09
6	30.62	0.84	29.78	0.808	11.42	2.11	2.61
7	33.01	0.91	32.10	0.899	10.35	2.79	3.10
8	34.67	0.96	33.71	0.967	9.59	3.40	3.51
9	35.67	0.99	34.76	1.007	9.15	3.83	3.80
10	36.00	1.00	35.00	1.017	9.00	3.96	3.89

Lower chord $L_m = L_{mo} - X_a \frac{H_m - Y}{L_m}$

Panel	H_m	Y	$H_m - Y$	L_m	X_a	$X_a \frac{H_m-Y}{L_m}$	$\frac{H_m-Y}{L_m}$
0	27.00	0.00	27.00	23.01	0.151	0.18	1.17
1	30.48	0.19	30.29	20.43	0.296	0.44	1.48
2	33.57	0.36	33.21	18.08	0.439	0.81	1.83
3	36.28	0.51	35.77	15.98	0.570	1.28	2.24
4	38.61	0.64	37.97	14.15	0.699	1.88	2.68
5	40.58	0.75	39.83	12.59	0.808	2.56	3.16
6	42.14	0.84	41.30	11.31	0.899	3.29	3.65
7	43.41	0.91	42.50	10.30	0.967	3.99	4.13
8	44.29	0.96	43.33	9.59	1.007	4.56	4.52
9	44.82	0.99	43.83	9.15	1.017	4.87	4.79

Upper chord

member	$\frac{H_m-Y}{g_m}$ \ X_a →	1 0.151	2 0.296	3 0.439	4 0.570	5 0.699	6 0.808	7 0.899	8 0.967	9 1.007	10 1.017
1	0.32	0.048	0.095	0.141	0.182	0.223	0.259	0.287	0.309	0.322	0.325
2	0.70	0.106	0.207	0307	0.399	0489	0.566	0.636	0.677	0.704	0.712
3	1.12	0.169	0.332	0.492	0.638	0783	0.905	1.007	1.083	1.127	1.149
4	1.59	0.240	0.471	0.698	0.906	1.112	1.285	1.428	1.537	1.600	1.617
5	2.09	0.316	0.619	0.918	1.192	1.462	1.690	1.878	2.020	2.103	2125
6	2.61	0.394	0.773	1.146	1.487	1.825	2.118	2346	2.523	2.627	2.653
7	3.10	0.468	0.918	1.362	1.767	2166	2505	2787	2.995	3120	3152
8	3.51	0.530	1.038	1.542	2.000	2453	2837	3155	3394	3533	3568
9	3.79	0.572	1.122	1.664	2.160	2.649	3062	3407	3665	3817	3854
10	389	0.588	1.152	1.707	2.217	2720	3144	3497	3760	3920	3960

Lower chord

Panel Point	X_a → / $\frac{y_m-Y}{L_m}$	1 0.151	2 0.296	3 0.439	4 0.570	5 0.699	6 0.808	7 0.899	8 0.967	9 1.007	10 1.017
0	1.17	0.177	0.346	0.514	0.667	0.818	0.945	1.052	1.131	1.178	1.190
1	1.48	0.223	0.438	0.640	0.844	1.034	1.196	1.331	1.431	1.490	1.505
2	1.83	0.276	0.541	0.803	1.043	1.279	1.479	1.645	1.770	1.843	1.861
3	2.24	0.338	0.663	0.983	1.277	1.566	1.810	2014	2.166	2.256	2.278
4	2.68	0.405	0.793	1.177	1.528	1.873	2.165	2409	2.591	2.699	2726
5	316	0477	0935	1.387	1.801	2.209	2.553	2841	3.056	3.180	3.214
6	365	0.551	1.080	1.602	2.081	2551	2949	3281	3530	3676	3712
7	413	0.624	1.222	1813	2354	2887	3337	3713	3994	4158	4200
8	452	0.683	1338	1.979	2.576	3.159	3652	4.063	4371	4.552	4597
9	4.79	0.723	1.418	2.103	2.730	3348	3870	4306	4632	4824	4871

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CALCULATIONS FOR

Preliminary Design of Chorotsubashi for Kumamoto-ken

Upper Chord $U_m = U_{mo} - X_a \frac{t_{bm} - t}{g_m}$

Member \ Load	1	2	3	4	5	6	7	8	9	10
U1	-0.021	-0.041	-0.060	-0.074	-0.088	-0.097	-0.098	-0.093	-0.078	-0.055
2	-0.044	-0.082	-0.120	-0.149	-0.177	-0.191	-0.202	-0.177	-0.142	-0.088
3	-0.061	-0.117	-0.169	-0.207	-0.244	-0.259	-0.253	-0.221	-0.157	-0.072
4	-0.075	-0.142	-0.204	-0.248	-0.289	-0.298	-0.276	-0.220	-0.118	0.029
5	-0.082	-0.150	-0.215	-0.255	-0.291	-0.285	-0.239	-0.146	0.004	0.217
6	-0.079	-0.143	-0.201	-0.227	-0.250	-0.228	-0.141	0.002	0.208	0.497
7	-0.062	-0.106	-0.145	-0.144	-0.138	-0.071	0.053	0.250	0.532	0.906
8	-0.029	-0.036	-0.040	0.003	0.051	0.168	0.350	0.612	0.973	1.439
9	0.018	0.058	0.106	0.200	0.301	0.478	0.723	1.055	1.493	2.046
10	0.079	0.181	0.293	0.449	0.613	0.856	1.170	1.573	2.080	2.707
10'	0.079	0.181	0.293	0.449	0.613	0.856	1.170	1.573	2.080	2.707
9'	0.149	0.320	0.499	0.724	0.957	1.265	1.641	2.104	2.673	2.046
8'	0.221	0.464	0.711	1.005	1.303	1.670	2.103	2.615	1.975	1.439
7'	0.286	0.589	0.899	1.247	1.602	2.017	2.488	1.875	1.344	0.906
6'	0.341	0.697	1.059	1.453	1.850	2.292	1.749	1.257	0.838	0.497
5'	0.387	0.787	1.191	1.620	2.053	1.590	1.162	0.792	0.475	0.217
4'	0.418	0.856	1.287	1.727	1.356	1.019	0.712	0.438	0.210	0.029
3'	0.442	0.889	1.340	1.085	0.833	0.603	0.393	0.210	0.058	-0.072
2'	0.456	0.916	0.754	0.601	0.448	0.308	0.173	0.073	-0.017	-0.088
1'	0.465	0.391	0.318	0.250	0.182	0.119	0.064	0.015	-0.025	-0.055

Influence surface for upper chord

Load \ Member	U_1	U_2	U_3	U_4	U_5	U_6	U_7	U_8	U_9	U_{10}
1	-0.021	-0.044	-0.061	-0.075	0.082	-0.079	-0.062	-0.029	0.018	0.079
2	-0.041	-0.082	-0.117	-0.142	0.150	-0.143	-0.106	-0.036	0.058	0.181
3	-0.060	-0.120	-0.169	-0.204	0.215	-0.201	-0.145	-0.040	0.106	0.293
4	-0.074	-0.149	-0.207	-0.248	0.255	-0.227	-0.144	-0.003	0.200	0.449
5	-0.088	-0.177	-0.244	-0.289	0.291	-0.250	-0.138	0.051	0.301	0.613
6	-0.097	-0.191	-0.259	-0.298	0.285	-0.228	-0.071	0.168	0.478	0.856
7	-0.098	-0.202	-0.253	-0.276	0.239	-0.141	+0.053	0.350	0.723	1.170
8	-0.093	-0.177	-0.221	-0.220	-0.146	-0.002	0.250	0.612	1.055	1.573
9	-0.078	-0.142	-0.157	-0.118	+0.004	0.208	0.532	0.973	1.493	2.080
10	-0.055	-0.088	-0.072	+0.029	0.217	0.497	0.906	1.439	2.046	2.707
9'	-0.025	-0.017	+0.058	0.210	0.475	0.838	1.344	1.975	2.673	2.080
8'	+0.015	+0.073	0.210	0.438	0.792	1.257	1.875	2.615	2.104	1.573
7'	0.064	0.173	0.393	0.712	1.162	1.749	2.488	2.103	1.641	1.170
6'	0.119	0.308	0.603	1.019	1.590	2.292	2.017	1.670	1.265	0856
5'	0.182	0.448	0833	1.356	2.053	1.850	1.602	1.303	0.957	0.613
4'	0.250	0.601	1.085	1.727	1.620	1453	1.247	1.005	0.724	0.449
3'	0.318	0.754	1.340	1.287	1.191	1.059	0.899	0.711	0.499	0.293
2'	0.391	0.916	0.889	0.856	0.787	0.697	0.589	0.464	0.320	0.181
1'	0.465	0.456	0.442	0.418	0.387	0.341	0.286	0.221	0.149	0.079
Odd panel	-0.370	-0.702	-0.884	-0.962	-0.827	-0.671	-0.345	-0.069	0.0	0.0
	+1.029	+1.831	+3.066	+3.983	+5.272	+6.045	+7.204	+7.687	+8.560	+8.470
Even panel	-0.360	-0.687	-0.876	-0.908	-0.836	-0.600	-0.321	-0.036	0.0	0.0
	+.775	+1.898	+2.787	+4.069	+5.006	+6.196	+6.884	+7.976	+8.250	+8.825

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CALCULATIONS FOR

Preliminary design of Chorotsubashi for Fukumoto-san

Lower Chord

member \ Load	1	2	3	4	5	6	7	8	9	10
L0	-0.197	-0.346	-0.514	-0.667	-0.818	-0.945	-1.052	-1.131	-1.178	-1.198
1	-0.194	-0.379	-0.552	-0.727	-0.887	-1.020	-1.125	-1.196	-1.226	-1.211
2	-0.210	-0.408	-0.604	-0.778	-0.947	-1.081	-1.181	-1.239	-1.246	-1.197
3	-0.225	-0.438	-0.645	-0.827	-1.003	-1.134	-1.226	-1.265	-1.243	-1.152
4	-0.235	-0.454	-0.668	-0.850	-1.025	-1.147	-1.222	-1.234	-1.172	-1.029
5	-0.239	-0.459	-0.673	-0.849	-1.010	-1.125	-1.174	-1.151	-1.039	-0.832
6	-0.233	-0.443	-0.647	-0.808	-0.959	-1.039	-1.053	-0.983	-0.810	-0.528
7	-0.216	-0.406	-0.590	-0.722	-0.849	-0.890	-0.859	-0.732	-0.488	-0.124
8	-0.182	-0.336	-0.477	-0.573	-0.655	-0.647	-0.558	-0.365	-0.046	0.410
9	-0.133	-0.238	-0.333	-0.370	-0.398	-0.330	-0.176	0.088	+0.486	1.029
9'	-0.002	+0.024	+0.060	+0.154	+0.258	+0.457	0.742	1.137	1.666	1.029
8'	+0.068	0.164	0.274	0.429	0.597	0.855	1.195	1.638	0.956	0.416
7'	0.133	0.292	0.458	0.674	0.899	1.206	1.587	0.898	+0.327	0.124
6'	0.192	0.405	0.626	0.890	1.162	1.507	0.859	0.290	-0.174	-0.528
5'	0.237	0.494	0.756	1.057	1.363	0.781	0.255	-0.198	-0.563	-0.832
4'	0.274	0.564	0.859	1.186	0.671	0.209	-0.204	-0.555	-0.832	-1.029
3'	0.300	0.614	0.932	0.525	0.123	-0.233	-0.550	-0.814	-1.018	-1.152
2'	0.321	0.653	0.325	-0.019	-0.284	-0.550	-0.782	-0.973	-1.113	-1.197
1'	0.335	0.090	-0.141	-0.374	-0.594	-0.785	-0.949	-1.079	-1.167	-1.211
0'	-0.197	-0.346	-0.514	-0.667	-0.818	-0.945	-1.052	-1.131	-1.178	-1.198

Influence surface for Lower chord

Load \ member	L0	L1	L2	L3	L4	L5	L6	L7	L8	L9
1	-0.197	-0.194	-0.210	-0.225	-0.235	-0.239	-0.233	-0.216	-0.182	-0.133
2	-0.346	-0.379	-0.408	-0.438	-0.454	-0.459	-0.443	-0.406	-0.336	-0.238
3	-0.514	-0.552	-0.604	-0.645	-0.668	-0.673	-0.647	-0.590	-0.477	-0.333
4	-0.667	-0.727	-0.778	-0.827	-0.850	-0.849	-0.808	-0.722	-0.573	-0.370
5	-0.818	-0.887	-0.947	-1.003	-1.025	-1.010	-0.959	-0.849	-0.655	-0.398
6	-0.945	-1.020	-1.081	-1.134	-1.147	-1.125	-1.039	-0.890	-0.647	-0.330
7	-1.052	-1.125	-1.181	-1.226	-1.222	-1.174	-1.053	-0.859	-0.558	-0.176
8	-1.131	-1.196	-1.239	-1.265	-1.234	-1.151	-0.983	-0.732	-0.365	0.088
9	-1.178	-1.226	-1.246	-1.243	-1.172	-1.039	-0.810	-0.488	-0.046	0.486
10	-1.190	-1.211	-1.197	-1.152	-1.029	-0.832	-0.528	-0.124	0.410	1.029
9	-1.178	-1.167	-1.113	-1.018	-0.832	-0.563	-0.174	0.327	0.956	1.666
8	-1.131	-1.079	-0.973	-0.814	-0.555	-0.198	0.290	0.898	1.638	1.137
7	-1.052	-0.949	-0.782	-0.550	-0.204	+0.255	0.859	1.587	1.195	0.742
6	-0.945	-0.785	-0.550	-0.233	0.209	0.781	1.507	1.206	0.855	0.457
5	-0.818	-0.594	-0.284	+0.123	0.671	1.363	1.162	0.899	0.597	0.258
4	-0.667	-0.374	-0.019	0.525	1.186	1.057	0.890	0.674	0.429	0.154
3	-0.514	-0.141	0.325	0.932	0.859	0.756	0.626	0.458	0.274	0.060
2	-0.346	0.090	0.653	0.614	0.564	0.494	0.405	0.292	0.164	0.024
1	-0.197	0.335	0.321	0.300	0.274	0.237	0.192	0.133	0.068	-0.002

Summary

Odd panel	-7.478	-6.835	-6.367	-5.910	-5.358	-4.698	-3.876	-3.002	-1.918	-1.042
	+0.000	+0.335	+0.646	+1.355	+1.804	+2.611	+2.839	+3.404	+3.090	+3.212
Even panel	-7.368	-6.771	-6.245	-5.863	-5.269	-4.614	-3.801	-2.874	-1.921	-0.938
	+0.000	+0.090	+0.653	+1.139	+1.959	+2.332	+3.092	+3.070	+3.496	+2.886

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CALCULATIONS FOR

Preliminary Design of Shorokubashi for Kumamoto-Ken

Diagonals $D_m = \left(\frac{l_m}{f_m}\right) R_A$ $D_m' = \frac{l_m}{f_m}(1-R_A)$ $D_m' = \frac{n_a}{f_m} - \frac{l_m}{f_m} R_A$

Load on / Member $\frac{n_a}{f_m}$	D_1	D_2	D_3	D_4	D_5	D_6	D_7	D_8	D_9	D_{10}
	0.143	0.147	0.151	0.155	0.157	0.153	0141	0.116	0.078	0.028
1	0.050	0059	0070	0.082	0.094	0.106	0.115	0.118	0.112	0.095
2	0.100	0.119	0.140	0.163	0.188	0212	0230	0.236	0224	0.190
3	0149	0.178	0.210	0245	0282	0.317	0345	0353	0.336	0.286
4	0199	0237	0.280	0326	0.376	0.423	0460	0.471	0.448	0.381
5	0249	0.297	0.350	0408	0470	0.529	0575	0.589	0.561	0.476
6	0299	0.356	0.419	0490	0.564	0635	0.689	0.707	0.673	0.571
7	0.349	0.415	0.489	0.571	0.658	0.741	0.804	0.825	0785	0.666
8	0398	0474	0.559	0653	0752	0846	0919	0942	0897	0762
9	0448	0534	0629	0734	0846	0952	1034	1.060	1.009	0.857
10	0498	0.593	0699	0816	0940	1.058	1.149	1.178	1.121	0952
9'	0548	0652	0.769	0898	1.034	1.164	1.264	1.296	1.233	-0.603
8'	0598	0712	0839	0979	1128	1.270	1379	1414	-0272	-0536
7'	0647	0771	0909	1.061	1222	1.375	1494	-0014	-0238	-0469
6'	0697	0830	0979	1142	1316	1481	0156	-0012	-0204	-0402
5'	0747	0890	1049	1224	1410	0235	0130	-0010	-0170	-0335
4'	0797	0949	1118	1306	0252	0188	0104	-0008	-0136	-0268
3'	0847	1008	1188	0219	0189	0141	0078	-0006	-0102	-0201
2'	0896	1067	0160	0146	0126	0094	0052	-0004	-0068	-0134
1'	0946	0088	0080	0073	0063	0047	0026	-0002	-0034	-0067

Verticals $V_{mL} = -\left(\frac{i_m}{i_m - m_a}\right) R_A$ $V_m' = -\left(\frac{i_m}{i_m - m_a}\right)(1-R_A)$ $V_m' = -\left\{\frac{n_a}{i_m - m_a} - \frac{i_m}{i_m - m_a} R_A\right\}$

Load on / $\frac{i_m}{i_m - m_a}$	V_1	V_2	V_3	V_4	V_5	V_6	V_7	V_8	V_9
	1.124	1.237	1.337	1.400	1426	1.394	1.288	1.094	0.805
1	0.056	0062	0.070	0070	0071	0.070	0.0064	0.055	0.040
2	0.112	0.124	0.134	0.140	0.143	0139	0129	0.109	0.080
3	0169	0.186	0201	0210	0214	0209	0.193	0.164	0.121
4	0225	0248	0268	0280	0285	0279	0258	0219	0161
5	0281	0310	0335	0350	0356	0349	0322	0274	0202
6	0337	0371	0401	0420	0428	0418	0386	0328	0242
7	0393	0433	0468	0490	0499	0488	0451	0383	0282
8	0450	0495	0535	0560	0570	0558	0515	0438	0322
9	0506	0557	0602	0630	0642	0627	0580	0492	0363
10	0562	0619	0669	0700	0713	0697	0644	0547	0403
9'	0618	0681	0736	0770	0784	0767	0708	0602	0443
8'	0674	0743	0803	0840	0856	0836	0773	0656	-0488
7'	0731	0805	0870	0910	0927	0906	0837	-0301	-0427
6'	0787	0867	0937	0980	0998	0976	-0138	-0258	-0366
5'	0843	0929	1004	1050	1070	-0020	-0115	-0215	-0305
4'	0899	0990	1070	1120	0056	-0016	-0092	-0172	-0244
3'	0955	1052	1137	0090	0042	-0012	-0069	-0129	-0183
2'	1012	1114	0080	0060	0028	-0008	-0046	-0086	-0122
1'	1068	0057	0040	0030	0014	-0004	-0023	-0043	-0061
$\frac{12}{i_m - m_a}$	0124	0119	0110	0100	0085	0066	0041	0012	-0021

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CALCULATIONS FOR

Preliminary Design of Chigrotsubashi for Kumamoto-shi

Diagonals

				A →	1	2	3	4	5	6	7	8	9	10
member	ℓm	fm	kg/m		0.151	0296	0439	0.570	0.699	0.808	0.899	0.967	1.007	1.017
D1	51.19	83.83	0.61		0.092	0.181	0.268	0.348	0.426	0493	0.548	0590	0.614	0.620
2	52.14	81.70	064		0097	0.189	0.281	0.365	0447	0.517	0.575	0619	0.644	0.651
3	52.87	79.51	067		0.101	0.198	0.294	0.381	0.468	0.541	0.602	0648	0.675	0681
4	53.36	77.53	0.69		0.104	0204	0303	0393	0.482	0.558	0.620	0.667	0.695	0702
5	53.72	76.71	070		0.106	0.207	0307	0399	0.489	0.566	0629	0.677	0705	0712
6	53.95	7842	0.69		0.104	0204	0303	0393	0.482	0558	0.620	0.667	0.695	0.702
7	54.03	85.07	063		0095	0.186	0.277	0359	0.440	0.509	0566	0609	0634	0641
8	54.27	10322	052		0.079	0.154	0228	0296	0363	0420	0467	0503	0524	0529
9	54.42	15339	035		0053	0.104	0.154	0.200	0245	0283	0315	0338	0352	0356
10	54.46	43202	0.13		0020	0.038	0.057	0.074	0091	0105	0117	0126	0131	0132

Verticals

				A →	1	2	3	4	5	6	7	8	9	10
member	g/m	in-ma	g/in-ma		0.151	0296	0439	0.570	0.699	0.808	0.899	0967	1.007	1017
V1	58.34	96.58	0.60		0.091	0.178	0.263	0.342	0.419	0.485	0.539	0.580	0.604	D.610
2	59.25	101.18	0.59		0.089	0.175	0.259	0.336	0.412	0.477	0530	0.570	0594	0.600
3	60.29	108.63	0.55		0.083	0.163	0.241	0.314	0384	0.444	0.494	0532	0554	0559
4	61.29	120.02	0.51		0.077	0.151	0224	0.291	0.356	0.412	0.458	0493	0514	0.519
5	62.88	140.90	0.45		0.068	0.133	0.198	0257	0315	0.364	0.405	0435	0.453	0.458
6	65.62	182.48	0.36		0.054	0.107	0.158	0.205	0.252	0.291	0324	0348	0363	0.366
7	7246	290.55	0.25		0.038	0.074	0.110	0.143	0.175	0.202	0.225	0242	0.252	0254
8	119.17	103.90	0.12		0.018	0.036	0.053	0.068	0.084	0097	0.108	0.116	0.121	0.122
9	19.46	555.00	0.04		-0.006	-0.012	-0.018	-0.023	-0.030	-0.032	-0.036	-0.039	-0.040	-0041

Diagonals removed redundancy $X_a = 1$

member \ Load on →	1	2	3	4	5	6	7	8	9	10
D1	0.050	0.100	0.149	0.199	0249	0.299	0.349	0398	0.448	0.498
2	0.059	0.119	0.178	0.237	0.297	0.356	0.415	0.474	0.534	0.593
3	0.070	0.140	0.210	0.280	0350	0.419	0.489	0.559	0.629	0.699
4	0.082	0.163	0.245	0.326	0.408	0.490	0.571	0.653	0.734	0.816
5	0.094	0.188	0.282	0.376	0.470	0.564	0.658	0752	0.846	0.940
6	0.106	0.212	0317	0423	0529	0635	0.741	0846	0.952	1.058
7	0.115	0.230	0345	0.460	0.575	0.689	0.804	0.919	1.034	1.149
8	0.118	0.236	0353	0.471	0.589	0707	0.825	0942	1.060	1.178
9	0.112	0.224	0336	0448	0.561	0673	0.785	0.897	1.009	1.121
10	0.095	0.190	0.286	0.381	0476	0.571	0.666	0.762	0.857	0952
10'	-0.067	-0134	-0201	-0.268	-0.335	-0402	-0469	-0536	-0603	0952
9	-0.034	-0068	-0.102	-0.136	-0.170	-0.204	-0238	-0272	1.233	1.124
8	-0.002	-0.004	-0006	-0008	-0.010	-0012	-0.014	1.414	1.296	1.178
7	0.026	0.052	0.078	0.104	0.130	0.156	1.494	1379	1.264	1.149
6	0.047	0.094	0.141	0.188	0235	1.481	1.375	1.270	1.164	1.058
5	0.063	0.126	0.189	0252	1.410	1.316	1.222	1.128	1.034	0.940
4	0.073	0.146	0.219	1.306	1.224	1.142	1.061	0979	0898	0.816
3	0.080	0.160	1.188	1.118	1.049	0.979	0.909	0.839	0.769	0.699
2	0.088	1.067	1.008	0.949	0890	0.830	0.771	0712	0652	0.593
1	0.946	0.896	0.847	0.797	0.747	0697	0.647	0.598	0548	0.498

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CALCULATIONS FOR

Preliminary Design of Chōtoku-bashi for Kumamoto-Ken

Diagonals considering redundancy $X_a = 1$

Load \ member	1	2	3	4	5	6	7	8	9	10
D_1	-0.042	-0.081	-0.119	-0.149	0.177	-0.194	-0.199	0.192	-0.166	-0.122
D_2	-0.038	-0.070	-0.103	-0.128	0.150	-0.161	-0.160	0.145	-0.110	+0.058
D_3	-0.031	-0.058	-0.084	-0.101	0.118	-0.122	-0.113	0.089	-0.046	+0.018
4	-0.022	-0.041	-0.058	-0.067	0.074	-0.068	-0.049	0.014	0.039	0.114
5	-0.012	-0.019	-0.025	-0.023	-0.019	-0.002	0.029	0.075	0.141	0.228
6	+0.002	+0.008	+0.014	+0.050	+0.047	0.077	0.121	0.179	0.257	0.356
7	0.020	0.044	0.068	0.101	0.135	0.180	0.238	0.310	0.400	0.504
8	0.039	0.082	0.125	0.175	0.226	0.287	0.358	0.439	0.526	0.649
9	0.059	0.120	0.182	0.248	0.316	0.390	0.470	0.559	0.657	0.765
10	0.075	0.158	0.229	0.305	0.385	0.466	0.549	0.636	0.726	0.820
10′	-0.087	-0.172	-0.258	-0.342	-0.426	-0.507	-0.586	-0.662	-0.734	0.820
9′	-0.087	-0.172	-0.256	-0.336	-0.445	-0.487	-0.552	-0.610	0.881	0.765
8′	-0.081	-0.158	-0.234	-0.304	-0.373	-0.432	-0.481	0.911	0.772	0.649
7′	-0.069	-0.134	-0.199	-0.255	-0.310	-0.353	0.928	0.770	0.630	0.504
6′	-0.057	-0.110	-0.162	-0.205	-0.247	0.823	0.755	0.603	0.459	0.356
5′	-0.043	-0.081	-0.118	-0.147	0.921	0.750	0.593	0.451	0.329	0.228
4′	-0.031	-0.058	-0.084	0.913	0.742	0.584	0.441	0.312	0.203	0.114
3′	-0.021	-0.038	+0.894	0.737	0.581	0.438	0.307	0.181	0.094	0.048
2′	-0.008	0.878	0.728	0.584	0.443	0.313	0.196	0.093	0.008	-0.058
1′	0.854	0.715	0.579	0.449	0.321	0.204	0.099	0.008	-0.066	-0.120

Influence surface for diagonals

Load \ member	D_1	D_2	D_3	D_4	D_5	D_6	D_7	D_8	D_9	D_{10}
1	-0.042	-0.038	-0.031	-0.022	-0.012	-0.002	0.020	0.039	0.059	0.075
2	-0.081	-0.070	-0.058	-0.041	-0.019	-0.008	0.044	0.082	0.120	0.152
3	-0.119	-0.103	-0.084	-0.058	-0.025	-0.014	0.068	0.125	0.182	0.229
4	-0.149	-0.128	-0.101	-0.067	-0.023	-0.030	0.101	0.175	0.248	0.305
5	-0.177	-0.150	-0.118	-0.074	-0.019	0.047	0.135	0.226	0.316	0.385
6	-0.194	-0.161	-0.122	-0.068	-0.002	0.077	0.180	0.287	0.390	0.466
7	-0.199	-0.160	-0.113	-0.049	0.029	0.121	0.238	0.358	0.470	0.549
8	-0.192	-0.145	-0.089	-0.014	0.075	0.179	0.310	0.439	0.559	0.636
9	-0.166	-0.110	0.046	0.039	0.141	0.257	0.400	0.526	0.657	0.726
10	-0.122	-0.058	0.018	0.114	0.228	0.356	0.504	0.649	0.765	0.820
9′	-0.066	0.008	0.094	0.203	0.329	0.459	0.630	0.772	0.881	-0.734
8	0.008	0.093	0.181	0.312	0.451	0.603	0.770	0.911	-0.610	-0.662
7	0.099	0.196	0.307	0.441	0.593	0.755	0.928	-0.481	-0.552	-0.586
6	0.204	0.313	0.438	0.584	0.750	0.923	-0.353	-0.432	-0.487	-0.507
5	0.321	0.443	0.581	0.742	0.921	-0.247	-0.310	-0.373	-0.415	-0.426
4	0.449	0.584	0.737	0.913	-0.147	-0.205	-0.255	-0.304	-0.336	-0.342
3	0.579	0.728	0.894	-0.084	-0.118	-0.162	-0.199	-0.234	-0.256	-0.258
2	0.715	0.878	-0.038	-0.058	-0.081	-0.110	-0.134	-0.158	-0.172	-0.172
1	0.854	-0.008	-0.021	-0.031	-0.043	-0.057	-0.069	-0.081	-0.087	-0.087
Summary										
Odd panel	-0.769	-0.569	-0.443	-0.318	-0.217	-0.466	-0.578	-1.169	-1.310	-2.091
	+1.853	+1.375	+1.876	+1.425	+2.013	+1.655	+2.419	+2.046	+2.565	+1.964
Even panel	-0.738	-0.562	-0.426	-0.248	-0.272	-0.315	-0.742	-0.894	-1.605	-1.683
	+1.376	+1.868	+1.356	+1.923	+1.504	+2.176	+1.909	+2.543	+2.082	+2.379

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CALCULATIONS FOR

Preliminary Design of Chonokubooki for Kumamoto-ken

Verticals Removed redundancy $X_a = 1$

member \ Load→	1	2	3	4	5	6	7	8	9	
V1	0.056	0.112	0.169	0.225	0.281	0.337	0.393	0.450	0.506	0.562
2	0.062	0.124	0.186	0.248	0.310	0.371	0433	0.495	0.557	0.619
3	0.070	0.134	0.201	0.268	0335	0.401	0468	0.535	0.602	0.669
4	0.070	0.140	0.210	0.280	0.350	0420	0490	0.560	0.630	0.700
5	0.071	0.143	0.214	0.285	0.356	0428	0499	0.570	0.642	0713
6	0.070	0.139	0.209	0.279	0349	0418	0488	0.558	0627	0697
7	0.064	0.129	0.193	0.258	0.322	0.386	0.451	0.515	0580	0.644
8	0055	0.109	0.164	0219	0.274	0.328	0383	0438	0.492	0547
9	0.040	0.080	0.121	0.161	0.202	0242	0282	0322	0.363	0.403
9'	-0.061	-0.122	-0.183	-0244	-0305	-0366	-0427	-0.488	0.443	0.403
8	-0.043	-0.086	-0.129	-0172	-0.215	-0258	-0301	0.656	0.602	0547
7	-0.023	-0.046	-0.069	-0092	-0.115	-0.138	+0.837	0773	0.708	0.644
6	-0.004	0.008	-0.012	-0016	-0.020	0.976	0.906	0836	0.767	0.697
5	0.044	0.028	0.042	0.056	1.070	0.998	0.927	0856	0.784	0.713
4	0.030	0.060	0.090	1.120	1.050	0.980	0.910	0.840	0.770	0.700
3	0.040	0.080	1.137	1.070	1.004	0937	0.870	0.803	0.736	0.669
2	0.057	1.114	1.052	0990	0.923	0.867	0805	0.743	0681	0.619
1	1.068	1.012	0955	0.899	0843	0787	0731	0674	0.618	0.562

Verticals considering horizontal thrust X_a

Member	1	2	3	4	5	6	7	8	9	10
1	-0.035	-0.066	-0095	-0.117	-0.136	-0148	-0.146	-0.130	-0.098	-0.048
2	-0.027	-0.051	-0.073	-0.088	-0.102	-0.106	-0.097	-0.076	-0.037	0.019
3	-0.013	-0.029	-0.040	-0.046	-0.049	-0.043	-0.028	0.003	0.048	0.110
4	-0.009	-0.011	-0.014	-0.011	-0.006	0.008	0.032	0.067	0.116	0.181
5	+0.003	+0.010	-0.016	0.028	0.0441	0.064	0.094	0.135	0.189	0.255
6	0.016	0.032	-0.051	0.074	0.097	0.1297	0.164	0.210	0.264	0.331
7	0.026	0.055	0.083	0.085	0.147	0.184	0.226	0.273	0.328	0.390
8	0.038	0.073	0.110	0.151	0.188	0231	0.275	0322	0.371	0.425
9	0.046	0.092	0.139	0.184	0.232	0.274	0.318	0361	0.403	0.444
9'	-0.055	-0.110	-0.165	-0.221	-0.268	-0334	-0391	-0.449	0.483	0.444
8'	-0.061	-0.122	-0.182	-0.240	-0.299	-0.355	-0.409	0.540	0.481	0.425
7'	-0.061	-0.120	-0.179	-0.235	-0.290	-0.346	0.612	0.531	0.456	0.390
6'	-0058	-0.115	-0.170	-0.221	-0.272	0.685	0.582	0.488	0397	0.331
5'	-0.054	-0.105	-0.136	-0.201	0.755	0.634	0.522	0.421	0.331	0.255
4'	-0.047	-0091	-0.134	0.829	0694	0.568	0.452	0347	0.256	0.181
3'	-0.033	-0.083	0.896	0.756	0.620	0.493	0.376	0.271	0.182	0.116
2'	-0.032	0.939	0.793	0.654	0.511	0.390	0.275	0.172	0.087	0.019
1	+0.977	0.834	0.692	0.557	0.424	0.302	0.192	0.094	0.014	-0.048

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CALCULATIONS FOR

Preliminary Design of Chorotsubashi for Kumamoto-shi

Influence surface for Vertical

Load on Number	V_1	V_2	V_3	V_4	V_5	V_6	V_7	V_8	V_9
1	-0.035	-0.027	-0.013	-0.007	-0.003	0.016	0.026	0.038	0.046
2	-0.066	-0.051	-0.029	-0.011	-0.010	0.032	0.055	0.073	0.092
3	-0.095	-0.073	-0.040	-0.014	-0.016	0.051	0083	0.110	0.139
4	-0.117	-0.088	-0.046	-0.011	0.026	0.074	0.085	0.151	0.184
5	-0.136	-0.102	-0.049	-0.006	0.041	0.097	0.147	0.188	0232
6	-0.148	-0.106	-0.043	0008	0064	0.127	0.184	0231	0.274
7	-0.146	-0097	-0028	0.032	0094	0.164	0226	0275	0.318
8	-0.130	-0.076	0.003	0.067	0.135	0.210	0273	0.322	0.361
9	-0.098	-0.037	0048	0.116	0.189	0.264	0328	0371	0.403
10	-0.048	0.019	0.110	0.181	0255	0.331	0390	0425	0.444
9'	0.014	0.087	0.180	0256	0331	0.397	0.456	0.481	0.483
8'	0.094	0.172	0.271	0347	0421	0488	0.531	0540	-0449
7'	0.192	0275	0376	0453	0522	0582	0.612	-0.409	-0.391
6'	0302	0390	0493	0568	0634	0685	-0.346	-0355	-0334
5'	0424	0511	0620	0694	0755	-0272	-0290	-0299	-0268
4'	0557	0654	0756	0829	-0201	-0221	-0235	-0240	-0221
3'	0692	0793	0896	-0134	-0136	-0170	-0179	-0182	-0165
2'	0834	0939	-0083	-0091	-0105	-0115	-0120	-0122	-0110
1'	0977	0032	-0033	-0047	-0054	-0058	-0061	-0061	-0055
Summary									
	-0.510	-0336	-0163	-0208	-0190	-0500	-0530	-0951	-0879
	+22.99	+1.698	+2.172	+1.550	+1.951	+1.571	+1.878	+1.463	+1.621
	-0.509	-0321	-0201	-0113	-0306	-0336	-0701	-0717	-1.114
	+1.787	+2.174	+1.633	+2.000	+1.547	+1.947	+1.518	+1.742	+1.355

Dead load stress in the main panel — Panel one

6.299 × 174700 = 1.100.000

Intermediate panel 6.390 × 6390 = 85.000

1,185.000 #

Live Load stress in the

sidewalk loading	6.299	× 39700	=	259040
motor trucks + unif. load	6.299	× 21000	=	132.300
Extra for conc.	1.017	× 13000	=	13.200

Electric car loading and unif. load at rear + front 205.400

610.000 #

Total Stress 1.795,000 #

Stress in Hangers

Dead Load reaction ---------- 174700

Live Load reaction of floor beam pay 120.000

Live load reaction from sidewalk pay 27.000

147000

321700 call this 330.000 #

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CALCULATIONS FOR

Preliminary Design of Choobetsubashi for Kumamoto-ken

Influence Diagrams for Upper chord.

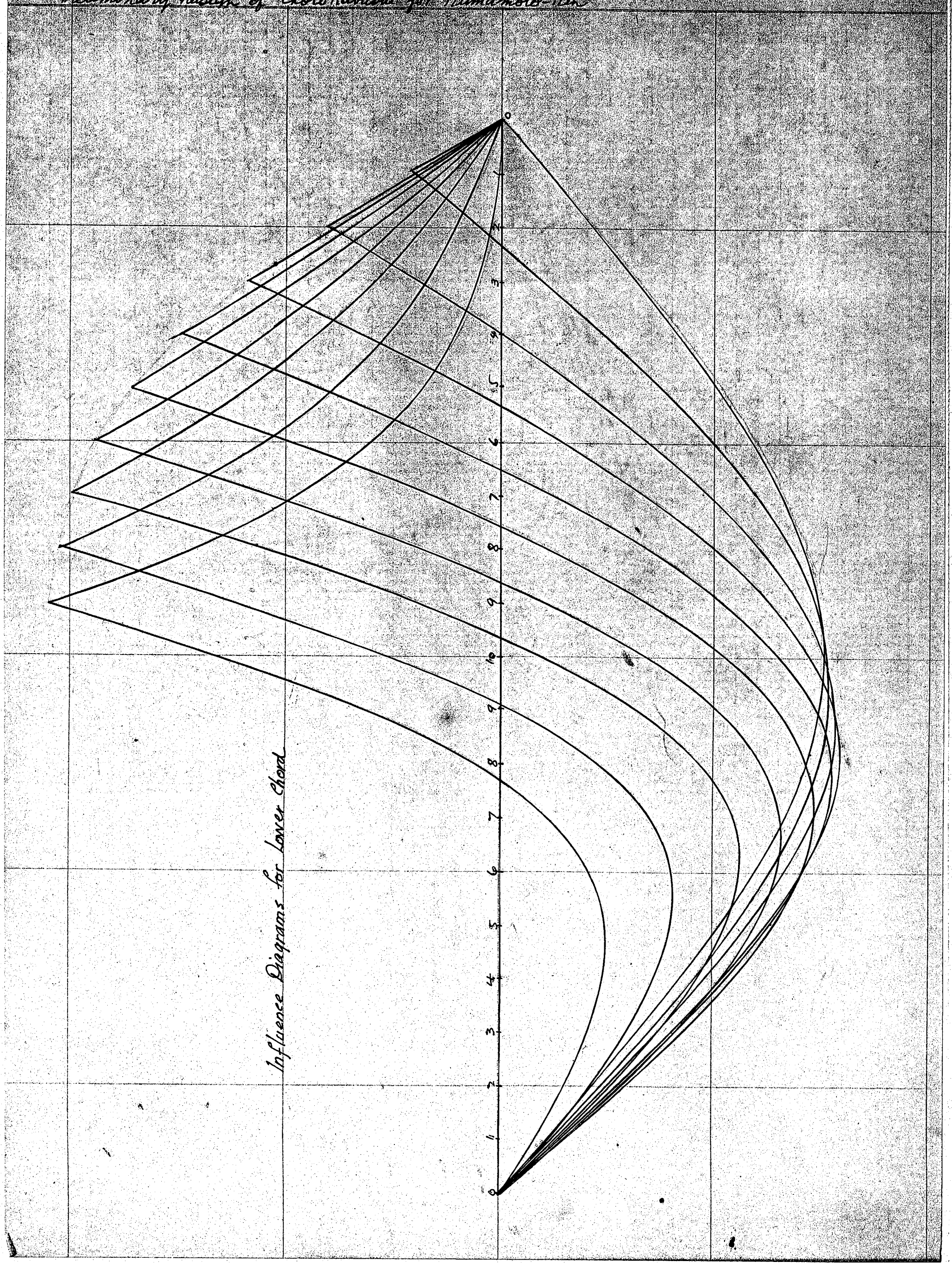
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CALCULATIONS FOR

Preliminary Design of Chorokubashi for Kumamoto-Ken



Influence Diagrams for Lower Chord

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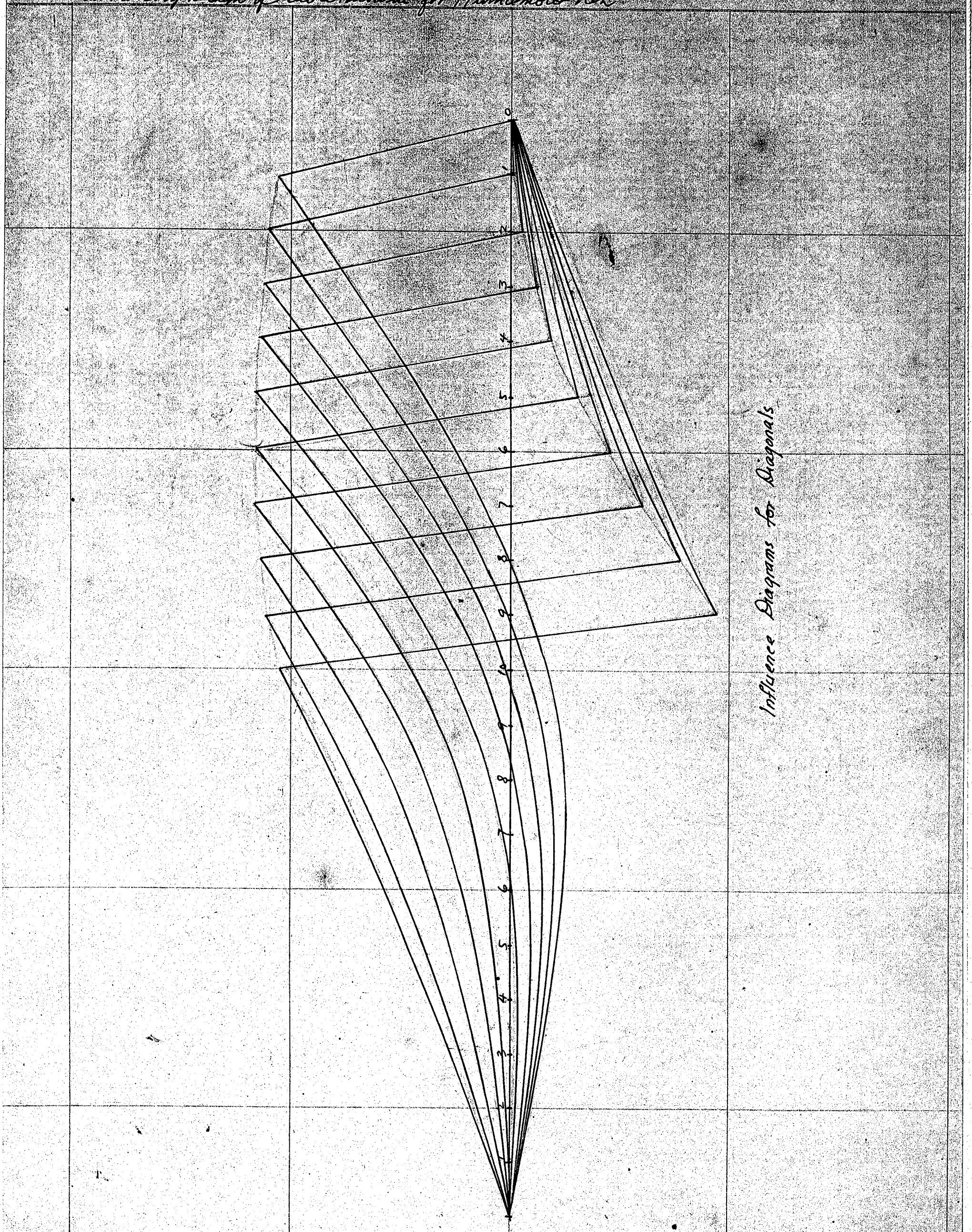
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CALCULATIONS FOR

Preliminary Design of Horokutadai Br Kumamoto Ken



Influence Diagrams for Diagonals

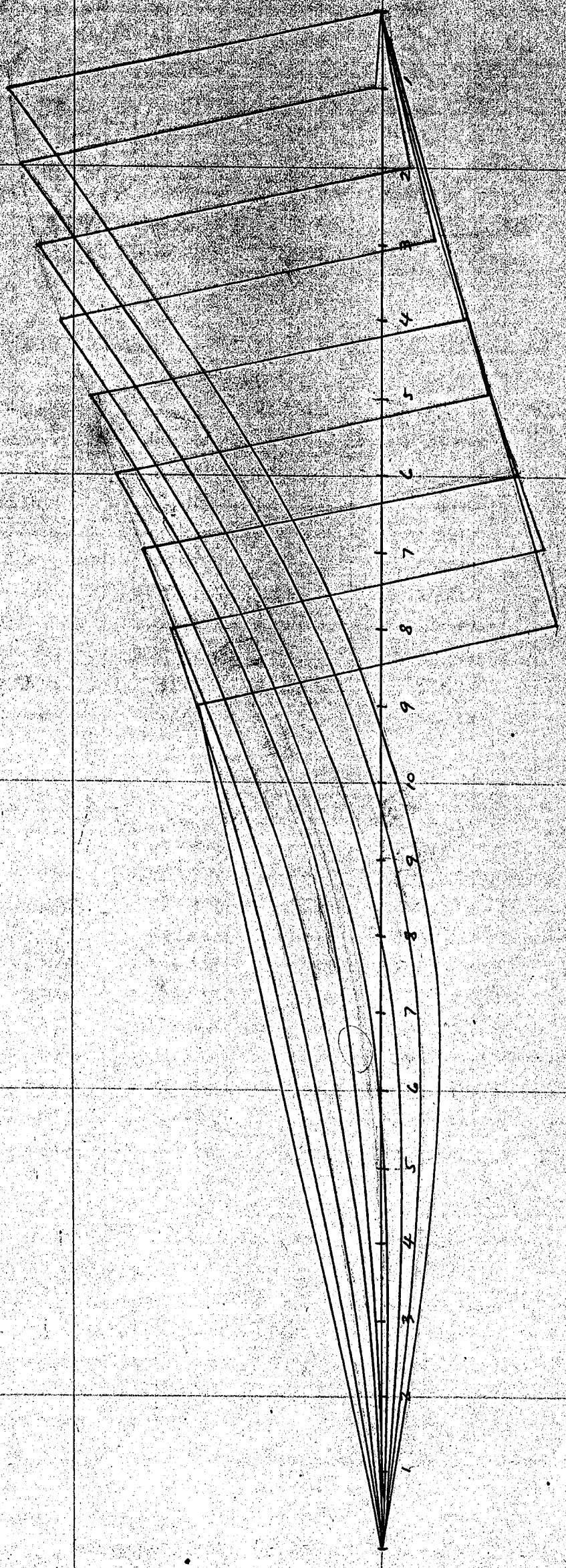
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CALCULATIONS FOR

Preliminary Design of Choyukubashi for Kumamoto-Ken



Influence Diagram for Vertical

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CALCULATIONS FOR

Preliminary Design of Cantilever for Kumamoto-Fen

Upper Chord Dead Load

	main Panel				Inter Panel				
Member	Comp	Tension	Sum	Panel load 174700#	Comp	Tension	Sum	13300#	Summary D.L. Stress
L_0-U_1	0.775	0.360	0.415	72500	1.029	0.370	0.659	8800	80500#
U_1-U_2	1.898	0.687	1.211	211500	1.831	0.702	1.129	15000	226500
U_2-U_3	2.787	0.876	1.911	333800	3.066	0.884	2.182	29000	362800
U_3-U_4	4.069	0.908	3.161	552000	3.983	0.962	3.021	40200	592200
U_4-U_5	5.006	0.836	4.170	728.000	5.272	0.827	4.445	59200	787200
U_5-U_6	6.196	0.600	5.596	978.000	6.045	0.671	5.374	71500	1049500
U_6-U_7	6.884	0.321	6.563	1143000	7.204	0.345	6.859	91200	1234200
U_7-U_8	7.976	0.036	7.940	1387000	7.687	0.069	7.618	101300	1488300
U_8-U_9	8.250		8.250	1441000	8.580		8.580	113800	1554800
U_9-U_{10}	8.825		8.825	1542000	8.470		8.470	113000	1655000

Upper chord Live Load stress

member	max comp	Unif. load conc 39700	motor unif. conc 21000	Trucks Loading + unif. max ord.	max stress Elec Car	Summary L.L. Stress	Tot. Stress
L_0 U_1	0.775	30800	16300	0391 @ 13000 = 5100	31400	83600#	164100
U_1 U_2	1.898	57300	40000	0.916 12000	76100	185400	411900
U_2 U_3	2.787	120300	58500	1.085 14100	112900	305800	668600
U_3 U_4	4.069	161500	85500	1.727 22500	164900	434400	1026600
U_4 U_5	5.006	198700	105000	1.620 21000	198300	523000	1310200
U_5 U_6	6.196	246000	130.000	2.292 29800	235600	641400	1690900
U_6 U_7	6.884	273300	144500	2.017 26200	260300	704300	1938500
U_7 U_8	7.976	316600	167500	2.615 34000	299500	817600	2305900
U_8 U_9	8.250	327500	173300	2.104 27400	299400	827600	2382800
U_9 U_{10}	8.825	350400	185500	2707 35200	319300	890400	2545400

Lower chord

Dead Load stress

	main panel		panel conc. 174700			panel conc 13300	Dead Load stress	max ordinate	
member	+	−		+	−				
L_0-L_1	0.000	7.368	−7.368	−1287.200	0.000	7.478	−7.478	−99.500	−1386.700	−1.190	
L_1-L_2	0.090	6.771	−6.681	−1167.000	0.335	6.835	−6.500	−86.500	−1253200	−1.211	
L_2-L_3	0.653	6.245	−5.592	−977000	0.646	6.367	−5.721	−76200	−1053200	−1.239	
L_3-L_4	1.139	5.863	−4.724	−826000	1.355	5.910	−4.555	−60600	−886600	−1.265	
L_4-L_5	1.959	5.269	−3.310	−578.000	1.804	5.358	−3.554	−47300	−625300	−1.234	
L_5-L_6	2.332	4.614	−2.282	−399000	2.611	4.698	−2.087	−27800	−426800	−1.125	
L_6-L_7	3.092	3.801	−0.709	−124.000	2.839	3.876	−1.037	−13800	−137.800	−1.038	+1.507
L_7-L_8	3.070	2.874	+0.196	+34300	3.404	3.002	+0.402	+5300	+39.600	+0.722	+1.206
L_8-L_9	3.496	1.921	+1.575	+275000	3.090	1.918	+1.172	+15600	+290600	+0.647	+1.638
L_9-L_{10}	2.886	0.938	+1.951	+341000	3.212	1.042	+2.170	+28900	+369900	+0.370	+1.637

Live Load stresses

member	side walk Conc 39700	motor car loading	Conc 13000	Electric car loading	Live Load stress	Total stress
L_0-L_1	−292500	−154700	−15,500	−240700	−703400	−2090,100
L_1-L_2	−268800	−142200	−15700	−230300	−657000	−1910200
L_2-L_3	−247900	−131200	−16100	−218200	−613500	−1666600
L_3-L_4	−232800	−123000	−16400	−208200	−580300	−1466900
L_4-L_5	−209200	−110,700	−16000	−186900	−522800	−1148100
L_5-L_6	−183200	−97000	−14600	−171900	−467300	−893500
L_6-L_7	−150900 (+122800)	−79800 (+65000)	−13500 (+19600	−144300 (122300)	−388700 (329700)	−526300 (+191900)
L_7-L_8	−114100 (+121900)	−60.000 (+64200)	−9400 (+13300)	−116000 (126100)	−299500 (332800)	−259900 (+465100)
L_8-L_9	−76300 (+103800)	−40600 (+73200)	−8400 (+21300)	−75600 (36800)	−200300 (335100)	+625700
L_9-L_{10}	−37200 (+114700)	−19700 (+60600)	−4800 (+21700)	−11800 (113000)	−173500 (310,000)	+679,900

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CALCULATIONS FOR

Preliminary Design of Choriobashi for Kumamoto-ku

Diagonals

Dead Load Stresses

Member	lo	+	Main Panel −	+	Panel cone 174700	+	Intermidiate Panel −	+	Panel Cone 13300	Total D.L. stress
U_0-L_1	1.376		0.738	+0.638	111.500	1.853	0.769	+1.084	14400	125900
U_1 L_2	1.868		0.562	+1.306	228.100	1.375	0.569	+0.806	10700	239800
U_2 L_3	1.356		0.426	+0.930	162.500	1.876	0.413	+1.463	19500	182000
U_3 L_4	1.923		0.248	+1.675	292700	1.425	0.318	+1.107	14700	307400
U_4 L_5	1.504		0.272	+1.232	215200	2.013	0.217	+1.796	23900	239100
U_5 L_6	2.176		0.315	+1.861	325100	1.655	0.466	+1.189	15800	340900
U_6 L_7	1.909		0.742	+1.167	203800	2.419	0.578	+1.841	24500	228300
U_7 L_8	2.543		0.894	+1.649	288200	2.046	1.169	+0.877	11700	299900
U_8 L_9	2.082		1.605	+0.477	83400	2.565	1.310	+1.255	16700	100100
U_9 L_{10}	2.379		1.683	+0.696	121700	0.964	2.091	−0.127	−1700	120000

Live Load Stresses

Member	Unif load on sidewalk, cone 39700		motor car loading, Panel cone 21000		Panel conc. 13000		Electric Car Loading		Tot. D+L.L.	Tot. L.L.
U_0-L_1	+54700	−29300	28900	−15500	+.715 −0.194	9300	−2500	56200	−28200	275000	149000 −75500
U_1-L_2	74200		39300		0.878	11400		74800	−28200	439500	199700
U_2 L_3	53800		28500		0.737	9600		54500		328400	146400
U_3 L_4	76300		40400		0.913	11900		74400		510400	203000
U_4 L_5	59700		31600		0.750	9800		59000		399200	160100
U_5 L_6	96100		45700		0.923	12,000		82300		577000	236100
U_6 L_7	75800		40100		0.770	10,000		72300		426500	198200
U_7 L_8	101000		53200		0.911	11800		94900		559900	260,000
U_8 L_9	82600	−63700	43700	−33700	+0.765 −0.610	10,000	−7900	79400	−60300	315800 −165500	215700 −165600
U_9 L_{10}	94500	−66800	50000	−35300	+0.820 −0.662	10700	−8600	87000	−62000	362400 −52700	242400 −172700

Verticals

Dead Load Stresses

Member	+	−		Panel cone 174700	+	−		Panel cone 13300	Dead Load Stress	max ordinate
U_1 L_1	1.787	0.509	+1.278	223700	2.299	0.510	+1.789	23800	247500	+0.834
U_2 L_2	2.174	0.321	+1.853	324000	1.698	0.336	+1.362	18100	342100	0.937
U_3 L_3	1.633	0.201	+1.432	250400	2.122	0.163	+1.959	26100	276500	0.756
U_4 L_4	2.000	0.113	+1.887	330000	1.550	0.208	+1.342	17900	347900	0.829
U_5 L_5	1.547	0.306	+1.241	216800	1.954	0.190	+1.761	23400	240200	0.634
U_6 L_6	1.949	0.336	+1.611	281600	1.571	0.500	+1.070	14200	295800	0.685
U_7 L_7	1.518	0.701	+0.817	142800	1.878	0.530	+1.348	18000	160800	0.531
U_8 L_8	1.742	0.717	+1.025	179300	1.463	0.951	+0.512	6800	186100	0.540 −0.355
U_9 L_9	1.355	1.114	+0.241	42800	1.621	0.879	+0.742	9900	52100	0.444 −0.449

Live Load Stresses

Member	sidewalk cone 39700		motor car loading 21800		cone 13000		Electric Car Loading		Live Load stress		Total stress	
U_1 L_1	70800		37500		10800		71500		180600		428100	
U_2 L_2	86200		45700		12200		87000		231100		573200	
U_3 L_3	64800		34300		9800		64700		173600		450100	
U_4 L_4	79400		42000		10800		77700		209900		557800	
U_5 L_5	61400		32500		8200		57700		159800		400000	
U_6 L_6	77300		40900		8900		70900		198000		493800	
U_7 L_7	60300		31900		6900		55300		154400		315200	
U_8 L_8	69200	(−29500)	36600	−20000	7000	−4600	62400	−25700	175200	−79800	361300	−
U_9 L_9	58800	(−44200)	28500	−15600	5800	−5800	51300	−41600	144400	−107200	196500	(−55100)

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CALCULATIONS FOR

Preliminary Design of Chord sections for Kumamoto-Fm

Upper chord sections :-

member	Stress	Section Rqd	Used section				
U_0-U_1	1641100 C	117 □"	6350	@3.4"	× 12.5	= 2700	
U_1-U_2	4441900 C	31.5	6350		× 12.4	= 2680	
U_2-U_3	668600 C	47.5	69.14		× 12.3	= 2900	
U_3-U_4	1026600 C	730	7739		× 12.2	= 3200	
U_4-U_5	1310200 C	93.5	9489		× 12.1	= 3910	
U_5-U_6	1690900 C	120.0	11739		× 12.1	= 4840	
U_6-U_7	1938500 C	138.0	138.02		× 12.05	= 5660	
U_7-U_8	2305900 C	165.0	169.28		× 12.05	= 6940	
U_8-U_9	2382400 C	170.3	169.28		× 12.0	= 6960	
U_9-U_{10}	2549400 C	182.0	188.03		× 12.0	= 7660	
							47390 #

U_0-U_1 and U_1-U_2

1 cov pl.	33 × 3/8	=	12.38
2 web plo	27 × 5/8	=	33.76
4 LS	6×6×3/8	=	17.36
			63.50

U_2-U_3

1 cov Pl.	33 × 3/8	=	12.38
2 web plo	27 × 5/8	=	33.76
4 LS	6×6×1/2	=	23.00
			69.14

U_3-U_4

1 cov pl	33 × 5/8	=	20.63
2 web plo	27 × 5/8	=	33.76
4 LS	6×6×1/2	=	23.00
			77.39

U_4-U_5

1 cov Pl	33 × 5/8	=	20.63
2 webs	27 × 3/4	=	40.50
4 LS	6×6×3/4	=	33.76
			94.89

U_5-U_6

1 cov Pl.	33 × 5/8	=	20.63
2 web plo	27 × 3/4	=	40.50
4 LS	6×6×3/4	=	33.76
2 side Plo	15 × 3/4	=	22.50
			117.39 □"

U_6-U_7

2 cov Plo	33 × 5/8	=	41.26
2 webs	27 × 3/4	=	40.50
4 LS	6×6×3/4	=	33.76
2 side Plo	15 × 3/4	=	22.50
			138.02

U_7-U_8 and U_8-U_9

2 cov Plo	33 × 5/8	=	41.26
2 webs	27 × 3/4	=	40.50
4 LS	6×6×3/4	=	33.76
2 Side Plo	15 × 3/4	=	22.50
2 Side Plo	25 × 5/8	=	31.26
			169.28

U_9-U_{10}

2 cov Plo	33 × 5/8	=	41.26
2 webs	27 × 3/4	=	40.50
4 LS	6×6×3/4	=	33.76
2 Side Plo	15 × 3/4	=	22.50
2 Side Plo	25 × 5/8	=	31.26
2 Side Plo	25 × 3/8	=	18.75
			188.03 □"

Lower chord sections :-

member	Stress	Rqd Section	Used section				
L_0-L_1	2090000 C	149.0	156.16	@3.4	× 14.0	= 7430	
L_1-L_2	1910000	136.0	137.40		× 13.5	= 6320	
L_2-L_3	1666600	119.0	121.83		× 13.2	= 5450	
L_3-L_4	1466900	105.0	96.83		× 12.85	= 4220	
L_4-L_5	1148100	82.0	87.77		× 12.66	= 3760	
L_5-L_6	893500	64.0	69.01		× 12.40	= 2920	
L_6-L_7	526300 C	37.7	51.12		× 12.30	= 2140	
L_7-L_8	465100 T	27.4 □" net	51.12		× 12.10	= 2110	
L_8-L_9	625700 T	36.9 "	51.12		× 12.04	= 2090	
L_9-L_{10}	679900 T	40.0 "	51.12		× 12.00	= 2080	
							38520 #

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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MARUNOUCHI, TOKIO

MADE BY J.M. DATE 13-8-9 FILE NO
CHECKED BY DATE PAGE NO 44

CALCULATIONS FOR

Preliminary Design of Chōrotsubashi for Kumamoto-Ken

L9-L10, L8-L9, L7-L8, and L6-L7

2 web plts	27 × 5/8	=	33.76
4 Ls	6×6×3/8	=	17.36
			51.120"

L5-L6

2 web plts	27 × 3/4	=	40.57
4 Ls	6×6×5/8	=	28.44
			69.01

L4-L5

2 web plts	27 × 3/4	=	40.57
4 Ls	6×6×5/8	=	28.44
2 side Plts	15 × 5/8	=	18.76
			87.77

L2-L3 and L3-L4

2 web plts	27×3/4	=	40.57
4 Ls	6×6×3/4	=	33.76
2 side Plts	15×3/4	=	22.50
2 side Plts	25×1/2	=	25.00
			121.83

L1-L2

4 webs	27×3/4	=	81.14
4 Ls	6×6×3/4	=	33.76
2 Plts	15×3/4	=	22.50
			137.40"

L0-L1

4 webs	27×3/4	=	81.14
4 Ls	6×6×3/4	=	33.76
2 Plts	15×3/4	=	22.50
5 Plts	25×3/8	=	18.76
			156.16"

Tie:-

Stress = 1795000 section required = 105.5

6 webs	24×3/4	=	108.00	90.00
2 side Plts	18×1/2	=	18.00	15.00
2 Ls	6×6×1/2	=	11.50	9.50
			137.50	114.50

weight of Tie = 137.5 @3.4 × 120 = 56000 # for 1/2 span

Diagonals

member	Stress	Section reqd	Used Section			
L0-L1	275000 T	16.20" net	21.24	@3.4	* 23.0	= 1660
L1-L2	439500	25.8	33.19		20.6	= 2325
L2-L3	338400	19.3	27.76		18.75	= 1770
L3-L4	510400	30.0	37.75		17.30	= 2220
L4-L5	399200	23.4	41.75 41.75		16.30	= 2320
L5-L6	5[illegible]1000	34.6	41.75		15.60	= 2210 2210
L6-L7	426500	25.1	33.19		15.10	= 1705
L7-L8	559900	32.9	41.75		14.85	= 2110
L8-L9	315800 T 65500C	18.6 Tension only	33.19		14.75	= 1665
L9-L10	362400 T 52700C	21.3 "	33.19		14.85	= 1675
						19660 #

L0-L1 4Ls 6×4×9/16 21.24 - 4.5 = 16.740" net

L1-L2 2 Pls. 12 1/2 × 3/4 = 18.75 15.75
4Ls 6×4×3/8 = 14.44 11.44
33.19 27.190" net

L2-L3 4Ls 6×4×3/4 = 27.76 - 21.760" net

L3-L4 2 Pls 12 1/2 × 3/4 = 18.75 - 15.75
4Ls 6×4×1/2 = 19.00 - 15.00
37.75 30.75

L4-L5 and L6-L7 ; L8-L9 ; L9-L10
2 Plts 12 1/2 × 3/4 = 18.75 15.75
4Ls 6×4×3/8 = 14.44 11.44
33.19 27.19

L5-L6 and L7-L8
2 Plts 12 1/2 × 3/4 = 18.75 - 15.75
4Ls 6×4×5/8 = 23.00 18.00
41.75 33.750" net

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MADE BY J.M DATE 13-8-9 FILE NO
CHECKED BY DATE PAGE NO 42

CALCULATIONS FOR

Preliminary design of Chorokutaski for Kumamoto-Ku

Verticals

Member	Stress	Rqd section	Used section			
U_1-L_1	428100	41.20"	44.13	@ 34#	23.10 =	3460#
U_2-L_2	573200	48.1	48.13		19.80 =	3240
U_3-L_3	450100	37.9	35.47		17.10 =	2060
U_4-L_4	557800	39.8	41.75		14.80 =	2100
U_5-L_5	400000	29.3	33.19		13.00 =	1465
U_6-L_6	493800	35.2	35.47		11.50 =	1390
U_7-L_7	315200	22.5	23.00		10.40 =	810
U_8-L_8	361300	25.8	33.19		9.60 =	1080
U_9-L_9	196500	14.0	23.00		9.10 =	710
					½ U_{10}-L_{10} say =	350
					2nd post U_{10}-L_{10} say	5510
						#2175

U_1-L_1

1 web pl	17×⅜	=	6.38
2 P/s	12½×¾	=	18.75
4 L's	6×4×½	=	19.00
			44.13

U_2-L_2

1 web pl	17×⅜	=	6.38
2 P/s	12½×¾	=	18.75
4 L's	6×4×⅝	=	23.00
			48.13

U_3-L_3 and U_6-L_6

2 P/s	12½×¾	=	18.75
4 L's	6×4×7/16	=	16.72
			35.47

U_4-L_4

2 P/s	12½×¾	=	18.75
4 L's	6×4×⅝	=	23.00
			41.75

U_5-L_5 - U_8-L_8

2 P/s	12½×¾	=	18.75
4 L's	6×4×⅜	=	14.44
			33.19

U_7-L_7 and U_9-L_9 and U_{10}-L_{10}

4 P/s 6×4×⅝ = 23.00"

Hangers

Total stress say 330.000# Section reqd = 330.000 ÷ 17.000 = 19.4 □" net

1 web pl	17×⅜	= 6.38	5.63 □"
4 L's	6×6×⅝	23.00	18.00
		29.38	23.63 □" net

Unit stress = 330.000 ÷ 23.63 = 14000 #/□" for direct stress

Weight of Hangers 29.38 @ 34 × 120.8 = 12080#

Summary for main section of truss

Upper chords	47390
Lower chords	38520
Tie	56000
Diagonals	19660
Verticals	22175
Hangers	12080
	195825
Details say 40%	78330
	274155
	4
	1096620 ÷ 2240 = 489 tons
Ornamental Cast iron post	24
	513 tons

Excluding ornamental Cast iron post 1096620 ÷ 240 = 4560# or 2280# per ft
Including ditto 1150000 ÷ 240 = 4790# or 2395# per 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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MADE BY J.M DATE 13-8-9 FILE NO
CHECKED BY DATE PAGE NO 43

CALCULATIONS FOR

Preliminary Design of Chōfu-bashi for Fukuoka-Fu

Estimate of metal in bridge.

Stringers	115.0	
Floor beams	114.0	
Lower Laterals	12.3	
Upper Lateral Swayetc	51.0	
	292.3 tons	
weight of truss	513.0	
	815.3	
weight of shoes say	24.7	
Grand Total	840 tons	

Load on bearing

Dead Load metal $\frac{840}{4}$ tons × 2240 = 470.000 #

Dead Load flooring 4300 × 122.25 = 525.000

995.000

Live Load

Uniform load 1655 × 122.25 = 202500

" 893 × 122.25 = 109500

Electric car

Continuous 30 tons car Uniformly load with impact 15% 1614 × 122.25 = 198000

523000

1,518,000 #

Load will be carried by cast iron shoes for End floor beam; Let us design shoes for truss for load of 1,500.000 #

Approximate load on bearing for shoes of End floor beam

Dead Load Floor say 46000

" " metal say 10.000

56000

Live Load Electric Ry loading 40.000

Uniform load say 10.000

50.000

Total 106.000 # call this 100.000 #

Bearing area rqd = 100.000 ÷ 640.0#/□" = 164□"

make base of cast iron shoe 18 × 18 = 324□"

Unit bearing = 100.000 ÷ 324 = 309#/□" ok

Shoes for truss

Total load 1,500.000 # Bearing area required = 1,500.000 ÷ 640. = 2340□"

Try 6½" rollers 6.5 × 610 = 3970# per lin inch of roller

1500.000 ÷ 3970# = 378" net use 9 rollers @42" = 378" net

make length of rollers = 44" over all

Size of base = 45 × 56 = 2520 Unit bearing 1,500.000 ÷ 2520 = 596#/□" ok

See next page for Sketch of Roller End shoe.

Bearing area for fixed shoes made same as for Roller End

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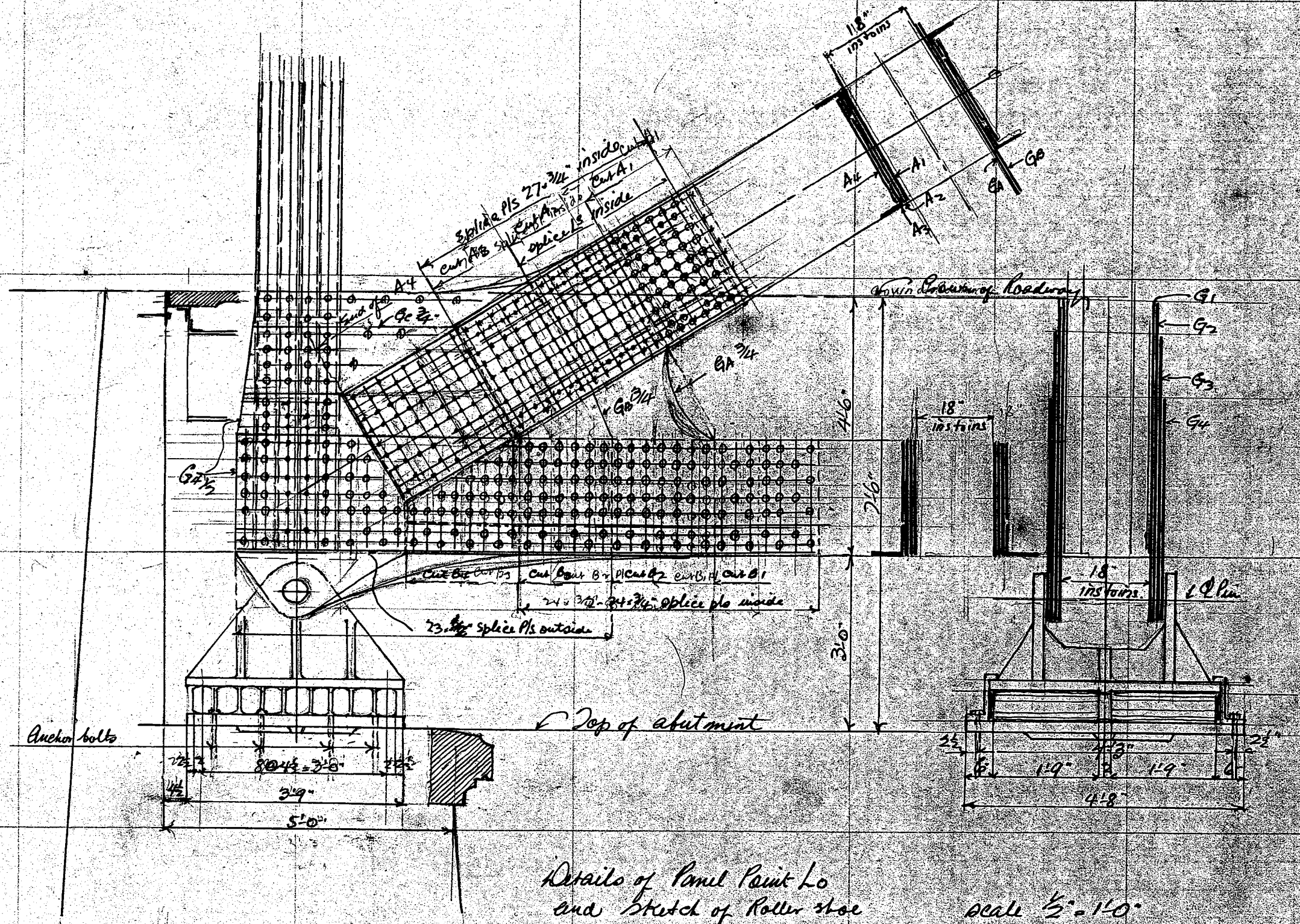
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CALCULATIONS FOR

Eliminary Design of Substructure for Truman Overpass

MADE BY _____ DATE 3-30-67 FILE NO _____

CHECKED BY _____ DATE _____ PAGE NO 44



Details of Panel Point Lo
and Sketch of Roller shoe Scale 1/2" = 1'-0"

Bearing for Fixed shoe same as for Roller shoe shown above

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MUSENDOCHI, TOKYO

CALCULATIONS FOR

Design of abutment for Kumanogo-Ike

MADE BY J. Masuda DATE 1957 FILE NO

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0.5 4.0 0.5 9.0 2.75 0.5 7.40

Elev. 104.35

3"

Elev. 104.35

Expansion Joint 1 Sq. 0.005

Expansion End Floor Beam D+L 108.000

4.25

15.375

10.875

1.75

25'-10½"

18'-4½"

7'-6"

granite

Tile Facing

Tile facing

8.465 1.91 5.00

1:2:4 concrete Reinforced.

Scale 1/60.

1:3:6 concrete

1:3:6 concrete

4.75'

Elev. 104.85

4.0"

Expansion Joint 1 Sq. 0.005

7.3'

2.25'

6.75

Elev. = 97.35

1.5'

19.1

5.0

1.5

2.5

2.5

4.0

1.57

8.4'

3.0

Elev. 100.0

Elev. 77.0

for final design

Elev. 68.0

Elev. 77.0

Elev. 68.0

100

100

100

100

3.0

3.0

6.5

6.0

3.0

3.0

1.75

1.75

1.60

1.70

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MADE BY J Masuda DATE 13-8-17 FILE NO
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CALCULATIONS FOR

Design of Chodzubashi for Kumamoto-Ken.

Weight of Abutment
Sketch drawing of abutment shown on pp 45.0
Approximate Concrete in Abutment (One half).

Wall above coping	1.3' av × 7.5 × 36.75	=	358.0
" below coping	1.5 × 20.35 × 36.75	=	1120.0
Column at wing	4.5 × 4.0 × 28.35	=	510.0
Coping for shaft	5.5 × 1.25 × 31.375	=	216.0
Body of shaft	5.0 × 30.875 = 154.50		
	6.91 × 32.785 = 226.00		
	½ × 38.05 × 19.10	=	3630.0

5834.0 cubic ft
27.0 ℔/ft

Base	3.0 × 13.0 × 41.75	=	1630.0
	6.0 × 17.0 × 41.75	=	4260.0

5890.0 cubic ft
27.30 ℔/ft

Total Volume shaft 2 @ 27.0 = 54.00
base 2 @ 27.30 = 54.60
108.60 ℔/ft

℄ of gravity for vertical load.

			weight	Arm about heel of Base	moment
wall above coping	358.0	@150 =	53700	2.35 =	126.200
" below coping	1120.0	@150 =	168000	2.25	378.000
Column at wing	510.0	@150 =	76500	2.00	153.000
Coping for shaft	216.0	@150 =	32400	6.75	219.000
Body of shaft	3630.0	@150 =	545000	6.98	3810.000
Base	1630.0	@140 =	228000	6.50	1482.000
Base	4260.0	@140 =	596000	8.50	5070.000
			1699600#	6.63	11,238,200 ft#

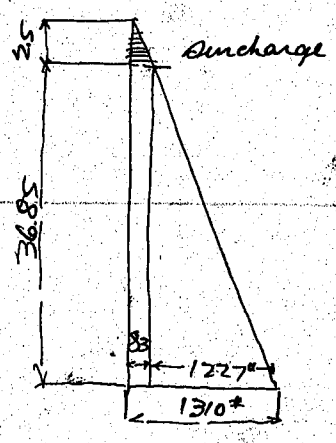
Earth fill at rear

			weight	Arm	moment
3.0 × 30.35 × 36.75 = 3340		@100 =	334000	× 1.5 =	501.000
.5 × 30.35 × 5.0 = 76		@100 =	7600	× 0.25 =	2000
			341600		503.000 ft#

Summary of loads and moments

	weight	arm	moment
Body of Abutment	1699600		11.238.200
Earth fill	341600		503.000
	2041200#	× 5.74 =	11.741200
Dead + Live Load from truss	1600.000	× 6.75 =	10800000
	3641,200	× 6.17 =	22541200 ft#

Horizontal Pressure



2.5 × 100/3 = 83#/0' 39.35 × 100/3 = 1310#

℄ of gravity of Resultant of Horizontal forces.

	Hor force	arm	moment
83 × 36.85 =	3060 ×	18.42 =	56400
1227/2 × 36.85 =	22600 ×	12.28 =	277500
	25660	13.00	333900 ft#

Total force	arm	moment
25660 × 4.25 = 1071,000	× 13.0 =	13,920.000 ft#

増田淳氏関係資料
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MADE BY J. Masuda DATE 3-8-17 FILE NO
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CALCULATIONS FOR

Design of Clorokubashi for Kumamoto-ken

Design of Abutment

Soil pressure due to full live load on span (with hor. Earth Pressure)

Load	Arm	Moment
3641200#	× 6.17 =	22541200
Horizontal moment →		13920000

36461200 # ÷ 3641200 = say 10.00'
8.5
1.50

Soil pressure = $\frac{3641200}{17 \times 41.75}\left(1 \pm \frac{6 \times 1.50}{17}\right)$ = 7840 ℄ or 3.50 tons/0'
2400 ℄ or 1.07 tons/0'

Soil pressure due to full live load on span (without Earth pressure)

Soil pressure = $\frac{3641200}{17 \times 41.75}\left\{1 \pm \frac{6 \times (8.5-6.17)}{17}\right\}$ = 9350 #/0' or 4.17 tons/0' ℄
907 " 0.405 " 0' ℄

Soil pressure due to dead load on span with earth pressure -

Dead Load Reaction truss 995.000 #
" " " 2nd F. Beam 56.000 # } approximate only

				Moment
Weight of abutment	2041200 #		=	11741200
From truss	995000	× 6.75	=	6720.000
" 2nd floor Beam	56000	× 6.75	=	378.000
Tot.	3092200 #			7098.000 #
		Arm = 6.09'		18839.200 #
		Horizontal Moment		13920.000
				32.759.200

Resultant Arm = 32.759.200 ÷ 3092200 = 10.60
8.50
2.10'

Soil pressure = $\frac{3092200}{17 \times 41.75}\left(1 \pm \frac{6 \times 2.10}{17}\right)$ = 7600 #/0' ℄ or 3.40 tons/0'
1124 " or 0.50 "

Soil pressure due to dead load on span without earth pressure.

Soil Pressure = $\frac{3092200}{17 \times 41.75}\left(1 \pm \frac{6 \times 241}{17}\right)$ = 8080 #/0' ℄ or 3.60 tons/0'
645 " 0.29 "

Horizontal ℄ of gravity of body of abutment

	Weight	Arm about Bottom of Base		Moment
wall above coping	53700	× 32.60	=	1.750.000
" below coping	168000	19.17		3220.000
Column at wing	76500	23.17		1.770.000
coping for shaft	32400	28.725		930.000
Body of shaft	545000	17.50 about		9530.000
Base	228000	7.50		1.710.000
Base	596000	3.0		1.788.000
	1699600 #	11.60		19.698.000 #
Earth fill	341600	17.67		6030.000
	2041200	12.64		25728000

Coefficient for Earthquake force k assumed .2 3/4 k = 0.15

Horizontal Earthquake force 204120 × 0.15 = 306200 # × 12.64 = Moment 3860.000 #
Body of abutment

Earth Pressure due to seismic force.

		arm	Moment	
2566o × 1.15	= 29500	× 13.0	= 383.000 #	per ℄ strip
Total	41.75 × 29500 = 1.232.000	× 13.0	= 16.000.000	for 1/2 wedth

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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MADE BY J. Masuda DATE 13-8-17 FILE NO
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CALCULATIONS FOR

Design of Choraku-bashi for Kumamoto-Ken

Case A Body of abutment and back fill subject to earthquake force no superimposed load on abutment $k_h = 0.15$

Body of abutment 2041200 × 0.15 = 306200 × 12.64 = 3.860.000 #
Back fil. 16.000.000
19860.000
Vertical moment of abutment about heel 11741200
31.601,200 #

Resultant Arm = 31601200 ÷ 2041200 = 15.45
17.00 − 15.45 = 1.55

Soil pressure = $\frac{2041200 \times 3}{15.5 \times 3 \times 41.75}$ = 21.000 #/□' or 9.37 tons/□'

Case B. Body of abutment with superimposed dead load; no back filling
Earthquake coefficient $k_h = 0.2$

			Hrush	moment
Earthquake Hrush body of abutment	1699.600	× 0.2 =	339920	× 11.60 = 3940.000 #
" " superimposed Dead load	1051.000	× 0.2 =	210200	× 36.50 = 7.670.000
	2750600 #			11.610.000

Dead Load moment due to vertical loads about heel

body of abutment	1699600	× 6.63 =	11,288,200
superimposed load	1051000	× 6.75 =	7098.000
	2750600		18336200

Total moment for max intensity of toe 29,946,200 #

Resultant Arm = 29946200 ÷ 2750600 = 10.90
8.5
2.40

Soil Pressure at Toe = $\frac{2750600}{17 \times 41.75}\left(1 \pm \frac{6 \times 2.4}{17}\right)$ = 7150 #/□' e or 3.19 tons/□'
592 · c or 0.26 ·

Moment for max intensity of Heel 18.336200
11.610.000
6,726.200 #

Resultant arm = 6.726200 ÷ 2750600 = 2.455

Soil Pressure at Heel = $\frac{2750600 \times 2}{2.455 \times 3 \times 41.75}$ = 17900 #/□' or 7.98 tons/□'
tension neglected

Stability against Sliding
Dead load and normal earth pressure -
body of abutment and back filling 2041200
Dead Load from truss span 1051000
3092200 #

Earth pressure earth at front neglected - 1071.000 #
Ratio against sliding 1071.000 ÷ 3092200 = 34.7 % of

Case A during earth quake

		k	
body of abutment and fill	2041200	× 0.15 =	306.000
Dead load from truss span	1051000	× 0.15 =	158.000
Earth pressure	1071.000	× 1.15 =	1230.000
			1694.000

Ratio against sliding 1694.000 ÷ 3092200 = 55 % ok for during earth quake

Without dead Load from truss span 1536000 ÷ 2041200 = 75 %

増田淳氏関係資料
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MADE BY J. Masuda DATE 12-8-20 FILE NO
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CALCULATIONS FOR

Design of Chorokutashi for Kumamoto-Ken

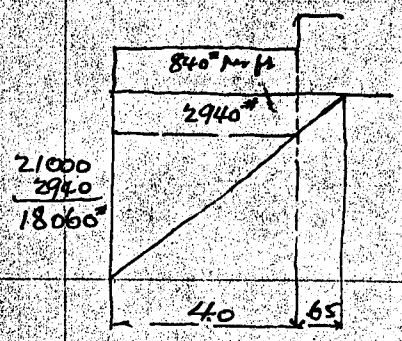
The base of abutment buried 10' to 15' below river bed, fills at front or rear will resist against sliding of body of abutment. Say Ok for sliding during earthquake.

Case B during Earth quake
No Earth pressure assumed

Body of abutment	1699600	× 0.2	=	340000
Superimposed dead load	1051000	× 0.2	=	210000
	2750600#			550.000#

Ratio against sliding = 550.000 ÷ 2750600 = 20% Ok

Details of Design
Strength at toe of base. max bearing intensity during quake = 21000# See pp 48 case A.

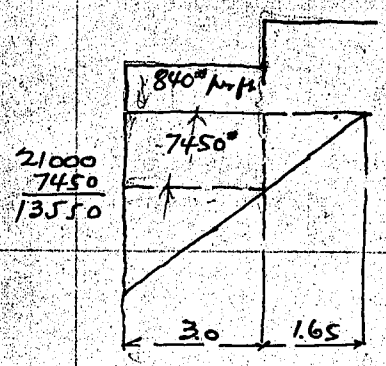


$21000 \times \frac{65}{465} = 2940^{\#}$

		arm		
2940 × 4.0	=	11760 × 2.0	=	23520
$\frac{18060}{2}$ × 4.0	=	36120 × $\frac{4.2}{3}$	=	96000
				119520
Less $840 \times \frac{4^2}{2}$			=	6720
				112800#

Fibre stress = $\frac{6 \cdot 112800 \cdot 12}{12 \cdot 72^2}$ = 130.5 #/□" tension at bottom fiber

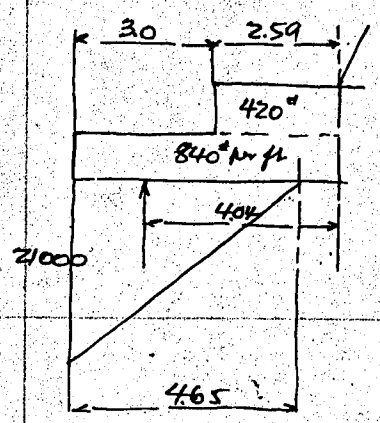
This unit stress is too high; Let us change projection 3.0' instead of 4.0'



$21000 \times \frac{1.65}{465} = 7450^{\#}$

		arm		
7450 × 3.0	=	22350 × 1.5	=	33500
$\frac{13550}{2}$ × 3.0	=	20300 × 2.0	=	40600
				74100
Less $840 \times \frac{3^2}{2}$			=	3800
				70300#

Fibre stress = $\frac{6 \cdot 70300 \cdot 12}{12 \cdot 72^2}$ = 81.5 #/□" Ok.

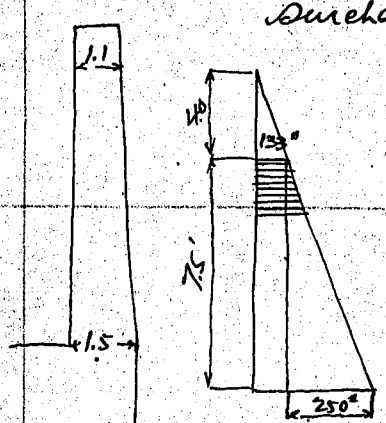


Strength at toe of base; projection 5.59'

$\frac{21000}{2}$ × 4.65	=	48800# × 4.04	=	197.000#
$840 \times \frac{5.59^2}{2}$	=	13100		
$420 \times \frac{2.59^2}{2}$	=	1410	less	14510
				182490#

Fibre stress = $\frac{6 \cdot 182490 \cdot 12}{12 \cdot 108^2}$ = 94.0 #/□" Ok.

Reinforcement in Parapet wall.
Surcharge assumed 4.0 Earth pressure assumed $\frac{100}{3}$
weight of earth = 100# per ft³.



Earth pressure

		arm		
133 × 7.5	=	1000 × $\frac{7.5}{2}$	=	3750
$\frac{250}{2}$ × 7.5	=	937 × $\frac{7.5}{3}$	=	2340
		1937 3.14		6090#

$\frac{M}{bd^2} = R = 102$ for 640#/□" concrete stress and 17000#/□" steel stress

Effective depth = $\sqrt{\frac{6090}{102}}$ = 7.7" depth assumed 16"

Steel req'd = $\frac{6090 \cdot 12}{7 \cdot 16 \cdot 17000}$ = 0.307 ½" ⌀ bars 8" centers.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

Design of Chorofutashi for Kumamoto-Ken

Horizontal Reinforcement in the wing.

75' below top of roadway, surcharge assumed 2.5
clear span length 60' span assumed 70'
Earth pressure = $10 \cdot \frac{100}{3} = 330$ #/0'
moment = $\frac{1}{0} \times 330 \times 7^2 = 1620$ ''

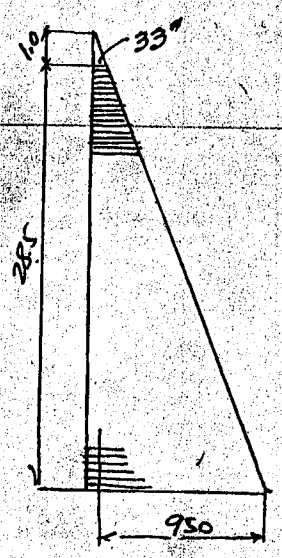
Use ½"ϕ bars 12" centers

27.5' below top of Roadway; surcharge assumed 1.5'
clear span 4.1' assumed span length = 5.0
Earth pressure = $29 \cdot \frac{100}{3} = 956$ #/0'
moment = $\frac{1}{0} \cdot 956 \cdot 5^2 = 2390$ ''

Use ½"ϕ bars 12" centers

Reinforcement in the column of wing.

surcharge assumed 1.0' Depth = 28.5' below crown of roadway



$33 \cdot 28.5 = 940 \times 14.25 = 13400$
$\frac{950}{2} \cdot 28.5 = 13530 \times \frac{28.5}{3} = 128800$
142200

Due to reaction from wing wall, moment per ft = $142200 \times \frac{7.15}{4.5} = 225.000$ ''
Depth assumed 60'
Effective depth required $\sqrt{\frac{225000}{102}} = 47$"

Steel Area required = $\frac{225000 \times 12}{\frac{7}{8} \cdot 70 \cdot 17000} = 2.60$ □"

Use 4 @ 7/8"ϕ bars = 2.44 □" neglecting direct load on column
On compression side use 1-7/8"ϕ per ft strip

surcharge 1.0' Depth = 18.5 below crown of roadway

$33 \cdot 18.5 = 610 \times 9.25 = 5650$
$\frac{616}{2} \cdot 18.5 = 5700 \times \frac{18.5}{3} = 37000$
42.650

Due to reaction from wing wall - moment per ft = $42650 \times \frac{7.4}{4.5} = 70.000$ ''
Effective depth say = $\sqrt{\frac{70000}{102}} = 28$"

Depth assumed 4.0'
Steel Area = $\frac{70.000 \times 12}{\frac{7}{8} \cdot 45 \cdot 17000} = 1.26$ □" Use 2-7/8"ϕ = 1.22 □" per ft

Surcharge 1.0 Depth = 8.5 below crown of roadway

$33 \cdot 8.5 = 280 \cdot 4.2 = 1190$
$\frac{283}{2} \cdot 8.5 = 1200 \cdot \frac{8.5}{3} = 3400$
4590

Due to reaction from wing wall, moment per ft = $4590 \times \frac{7.6}{4.5} = 7730$ ''

Steel Area = $\frac{7730 \cdot 12}{\frac{7}{8} \cdot 45 \cdot 17000} = 0.1385$ Use 1-7/8"ϕ = .601 □" per ft

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

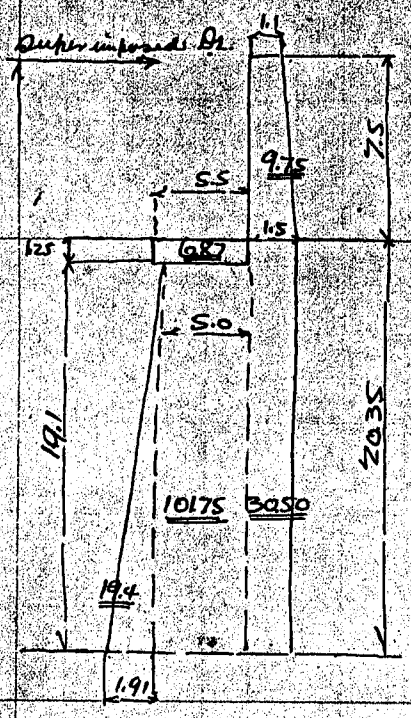
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MADE BY J. Masuda DATE 13-8-20 FILE NO.
CHECKED BY DATE PAGE NO. 51

CALCULATIONS FOR

Design of Chorokubashi for Kumamoto Ken

Reinforcement in body of shaft at top of base
During earthquake, body of abutment and superimposed dead load, no earth pressure considered. Coefficient for earthquake = k = 0.2
Body of abutment per ft strip



	Area	Arm	Moment
	9.75	24.0	= 234.0
	30.50	10.175	= 310.0
	6.87	19.72	= 135.5
	10.175	9.55	= 97.1.0
	19.40	6.36	= 123.5
	168.27	10.55	= 1774.0

weight = 168.27 @ 150 = 25200#
Seismic force = 25200 · 0.2 = 5040#
Moment = 5040 × 10.55 = 53250 ft#
Superimposed load per ft strip assumed 1051.000 / 32.875 = 32,000#
Seismic force = 32000 · 0.2 = 6400
Moment = 6400 × 27.5 = 176.000 ft#
Total Seismic moment = 53250 + 176000 = 229.250 ft#

Direct Load body of abutment 25200
Superimposed dead load 32000
57200 ÷ (8.41 × 144) = 47.3#/□" compression.

Fibre stress = $\frac{6M}{bd^2} = \frac{6 \cdot 229250 \cdot 12}{12 \cdot 100^2}$ = 138# tension or compression.

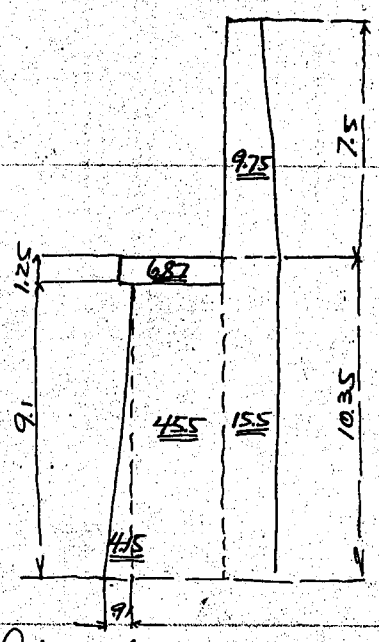
Summary for fibre stress 138 − 47. = 91#/□"
138 + 47 = 185#/□"

Tension stress is under allowable limit during earth quake, however, on account of safety add reinforcing bars in shaft.

For binding moment only Steel Area req'd = $\frac{229250 \times 12}{\frac{7}{8} \cdot 97 \cdot 30600}$ = 1.06 □"

Steel stress = 17000 · 1.8 = 30600#/□" assumed Use 7/8" ϕ bars 6" centers about at bottom of shaft.

Reinforcement in body of shaft 10' above top of base.



	Area	Arm	Moment
	9.75	14.10	= 137.5
	15.50	5.17	= 80.0
	6.87	9.73	= 66.9
	45.50	4.55	= 207.0
	4.15	3.03	= 12.6
	81.77	6.17	504.0

weight = 81.77 · 150 = 12300#
Seismic moment 12300 · 0.2 = 2460 × 6.17 = 15200
superimposed DL. 1051000 ÷ 25 = 42100# per ft assumed
Seismic moment 42100 · 0.2 = 8420 × 17.5 = 147500

Total seismic moment = 15200 + 147500 = 162700 ft#
Depth say 89"

Direct load
Abutment 12300
Superimposed DL. 42100
54400

Unit load = $\frac{54400}{12 \cdot 89}$ = 51#/□"

fibre stress due to bending only = $\frac{6 \cdot 162700 \cdot 12}{12 \cdot 89^2}$ = 123#/□"

Summary for fibre stress 72#/□" tension or 174#/□" comp.

Steel Area req'd = $\frac{162700 \times 12}{\frac{7}{8} \cdot 86 \cdot 30600}$ = 0.85□" neglecting direct compression.

増田淳氏関係資料
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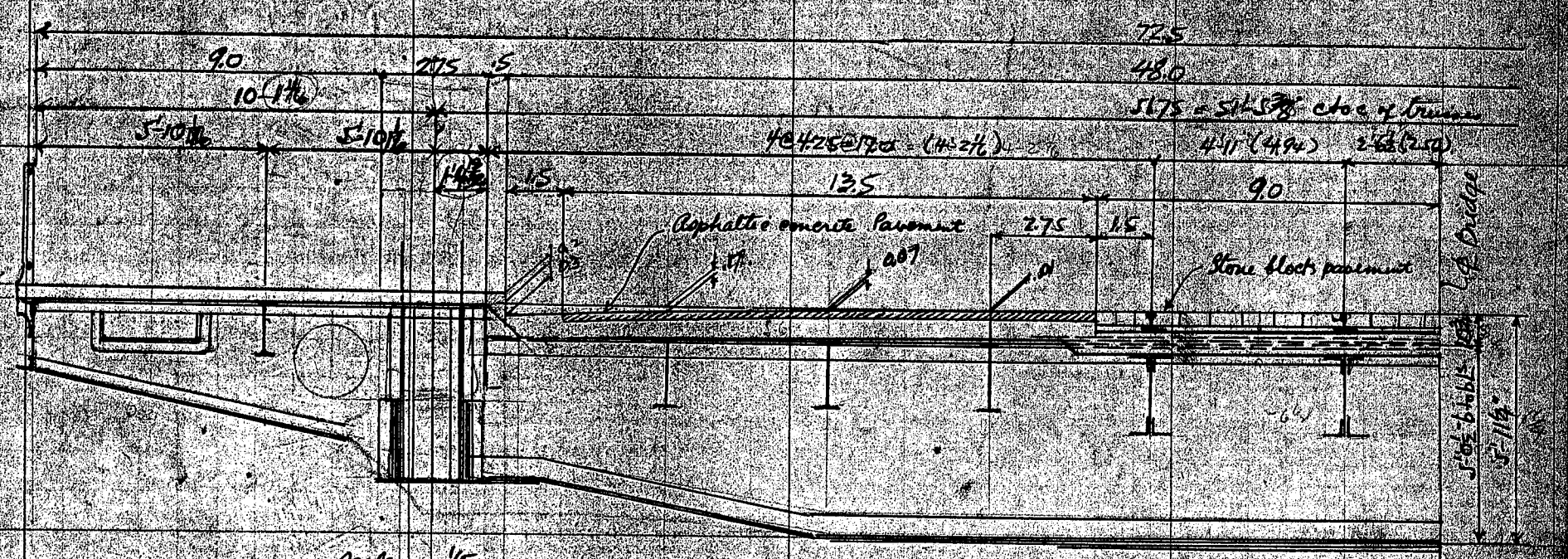
MADE BY J. Masuda DATE 13-5-26 FILE NO.
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CALCULATIONS, BOOK

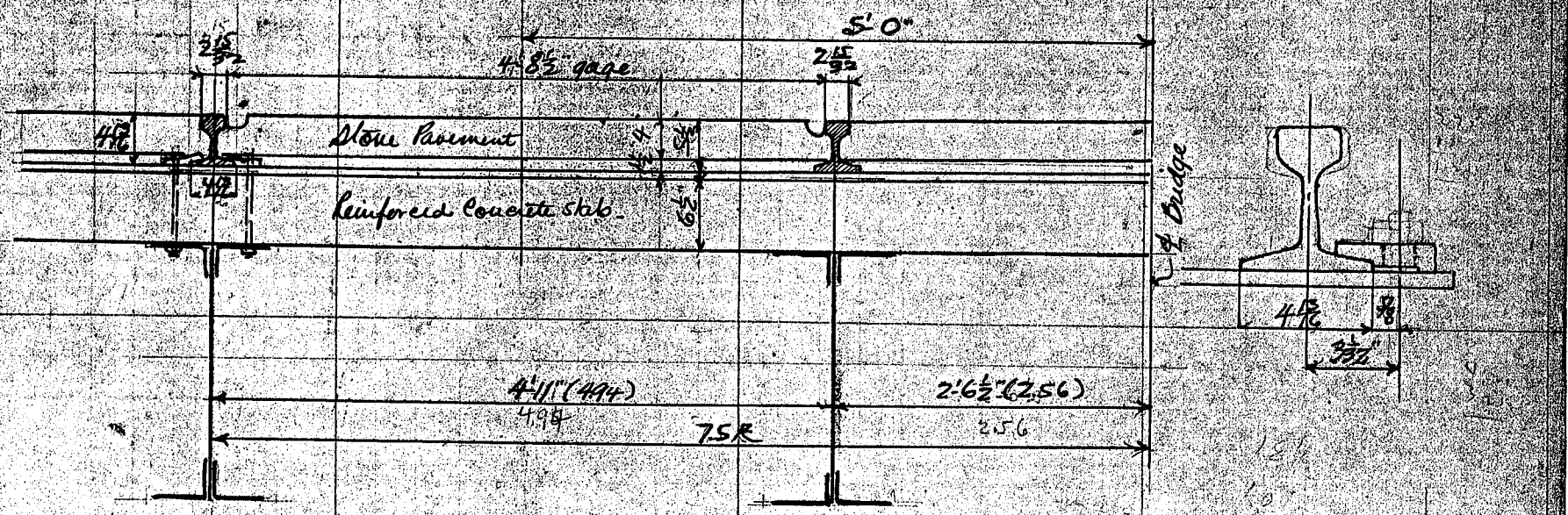
Design of Steelwork for Sumoto Ry.

Design of Truss = 240' Lid deck
Electric Ry Loading coupled 20 ton cars unit wheel load for both trusses and floor system.
See page No 2 for diagram.

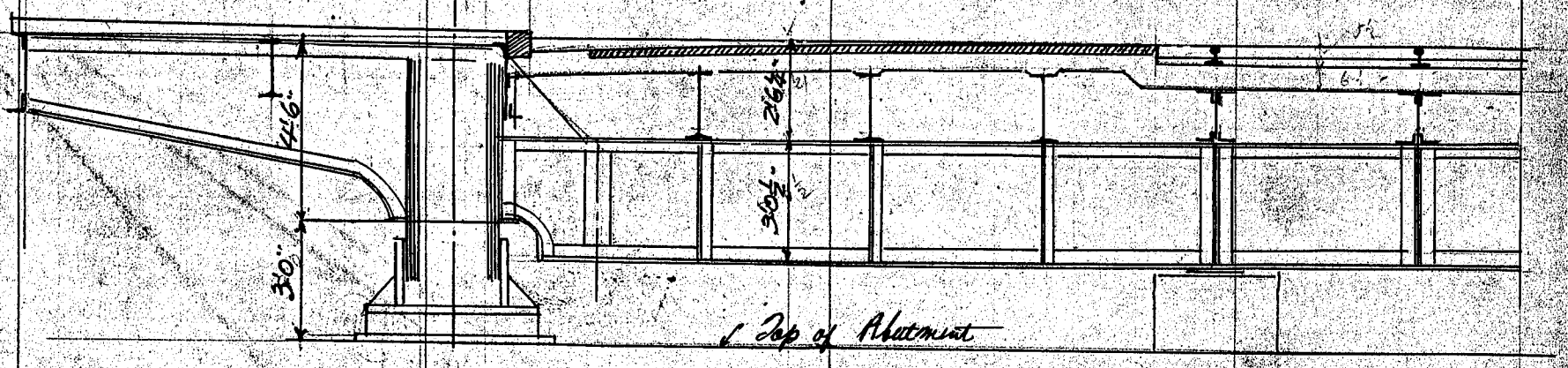
Cross section of Bridge for Final Design.



Cross Section of Bridge (Intermediate Floor Beams)



Details of Electric Ry Tracks Scale = 3/4" = 1'-0"



Scale 1/50 Sketch of End Floor Beam

増田淳氏関係資料
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MADE BY J. Masuda DATE 10-9-17 FILE NO.
CHECKED BY DATE PAGE NO 53

CALCULATIONS FOR

Design of Floor Slab for Fukuzumoto Ten:

Concrete slab under roadway

Floor slab under Asphalt pavement span Length 4'-2½" = 4.22 centers to center of stringers

Dead Load

2" Asphalt @ 151/12 = 22
6" Reinforced Concrete = 75
97# call this 100# per square ft including filler over stringers

Dead Load Moment = 1/10 · 100 · 4.22² = 178 ft
Dead Load Shear = 100 · 4.22/2 = 211#

Live Load motor trucks rear wheel concentration 9900
Impact 30% 2970
12870# including impact

Distribution of wheel concentration

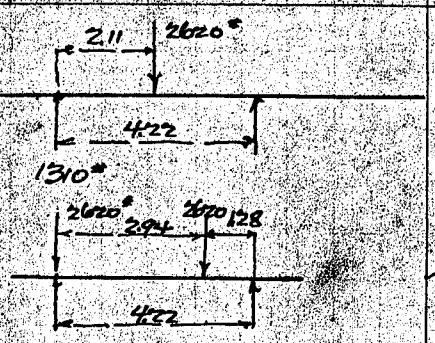
Contact between wheel and pavement assumed 20 centimeter = .66
2" Asphalt pavement = .34
100 = a

b = 1.28 + .34 = 1.62'

Effective width E = 2/3 (l+b) + a where l = span length
= 2/3 (4.25+1.62) + 1.00 = 4.49'

Load for one ft strip = 12870 ÷ 4.49 = 2620#

Live Load moment = 1310 · 2.11 = 2764 ft# as simple beam
2764 · 0.8 = 2210 ft# as continuous beam



Max End shear as simple span
2620 · 1.28/4.22 = 794
2620
3414#

Summary for moments and shears

	Moment	Shear
Dead Load	178	211
Live Load	2210	3414
	2388 ft#	3625 #

Effective depth of slab for steel stress of 17000 #/□" and concrete stress of 640 #/□" d = √(2388/102) = 4.83"
make slab 6" with 1" insulation.

Steel area required = 2388 · 12 / 7/8 · 5 · 17000 = 0.385 □" per ft Use ½"ϕ bars 6" spacing, 0.39 □" per ft

Unit shear = 3625 / 7/8 · 5 · 12 = 69 #/□" require web reinforcement

Floor Slab under Electric Ry Tracks span length 5'-1" max

Dead Load

4" stone blocks pavement	@ 16/12	= 54.0
1½" Lean mortar filling	@ 137/12	= 17.0
Concrete slab assumed		= 81.0
		152.# per sq ft

Dead Load m = 1/10 · 152 · 5.08² = 392 ft#
DL shear = ½ · 152 · 5.08 = 386#

Live Load
motor trucks wheel concentration with impact = 12870#
Distribution of wheel load Lengthwise a = .66 + .92 = 1.58
Crosswise b = 1.28 + .92 = 2.20
span length = 5.08
Effective width = E = 2/3 (5.08 + 2.20) + 1.58 = 6.43 assume distribution 6.0' wide.
Load per ft = 12870 ÷ 6.0 = 2140#

Live Load moment wheel concentration at center 1070 · 5.08/2 = 2720 ft# as simple span
2720 · 0.8 = 2180 as continuous beam

Live Load shear 2140 · 2.14/5.08 = 900
2140
3040#

増田淳氏関係資料
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CALCULATIONS FOR

Design of Choetsutaaki for Kumamoto-Ken

Summary for moments and shears

	Moment	Shear
Dead Load	392	386
Live Load	2180	3040
	2572#	3426#

Unit shear = $\frac{3426}{\frac{7}{8} \cdot 55 \cdot 12}$ = 59.0 #/□". Ok

Effective depth $d = \sqrt{\frac{2572}{102}}$ = 5.02"
Make slab 6½" one all with 1" insulation
Steel area required = $\frac{2572 \times 12}{\frac{7}{8} \cdot 55 \cdot 17000}$ = 0.377 □"
Use ½" dia bars 6" centers = 0.390 □"

Sidewalk Slab

Span length 5'-10⅛" = 5.84'

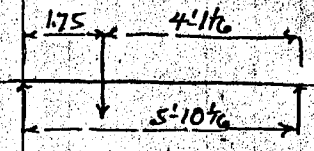
Dead Load

¾" wearing course	@ 137/12 =	9.0
3½" Reinforced concrete	@ 150/12 =	44.0
		53.0# per sq ft

Moment = $\frac{1}{10} \cdot 53 \cdot 5.84^2$ = 181#
Shear = $53 \cdot \frac{5.84}{2}$ = 155#

Dead Load from water main.

10" water main with protection say 100.0
vol of water .54 @ 62.5 = say 35.0
135.0 per lin ft



Reaction = $405 \cdot \frac{4.08}{5.84}$ = 283#

water main will be hanged every 6' apart
Total load = 135 · 6 = 810#
Assume this load distributed over 2' of slab.
or 405# per lin ft.
Moment = 283 · 1.75 = 495# as simple beam
0.8 · 495 = 400# as continuous

Live Load on sidewalk 500 kg/m² = 102 #/□' Moment = $\frac{1}{10} \cdot 102 \cdot 5.84^2$ = 344#
Shear = $102 \cdot \frac{5.84}{2}$ = say 300# as simple span

Summary for moments and shears

	Moment	Shear
Dead Load	181.	155
water main	400	283
Live Load	344	300
	925#	738#

Effective depth required $d = \sqrt{\frac{925 \times 12}{12 \cdot 102}}$ = 3.0"
Use 4" slab with ¾" insulation
steel area = $\frac{925 \times 12}{\frac{7}{8} \cdot 325 \cdot 17000}$ = 0.230 □"
Use ⅜" ϕ bars 6" centers = 0.330 □" per ft strip.

Design of Stringers under tracks

span length 24'-0"

Dead Load

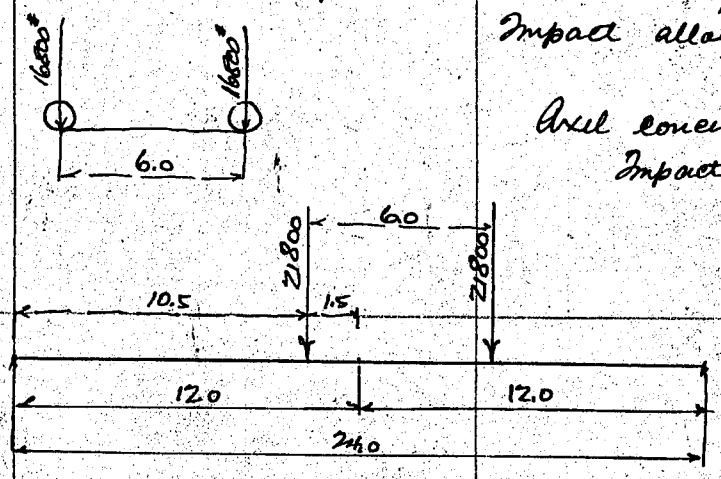
4" stone block pavement	54.0
1½ Lean mortar filling	17.0
concrete slab -	81.0
	152.0

Dead Load of Floor = 152.0 · 5 = 760#
weight of rail say 25
stringer assumed 80
865# per ft

weight of one rail 75 ÷ 3 = 25.0
accessories - 2.5
27.5#
Call this 25# per ft additional to floor load

Moment = $\frac{1}{8} \cdot 865 \cdot 24.0^2$ = 62300#
Shear = ½ · 865 · 24.0 = 10400#

Live Load Coupled 30 ton Electric Ry Cars



Impact allowance = $\frac{20}{60 + \frac{24}{328}}$ = 29.7%

Axel concentration = 16800#
Impact 29.7% = 5000
21800#

moment = $43600 \cdot \frac{10.5^2}{24.0}$ = 200.000#

moment per rail = 100.000#

Live Load shear $21800 \cdot \frac{18}{24}$ = 16350
21800
38150#

38150 ÷ 2 = 19075# per rail.

増田淳氏関係資料
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CALCULATIONS FOR

Design of Chorobubashi for Kumamoto-Ken

Summary for moments and shears

	Moment	Shear
Dead Load	62300	10400
Live Load	100.000	19075
	162300#	29475#

Section modulus required = $\frac{162300 \times 12}{17000} = 115.0$

On account of anchor bolt connection of rails this stringer shall be made built-section of 5"×3½" angles for flanges.

Try 24"×⅜" web = 9.0" 8 web = 112.0" Backs to back of L's = 24½" Effective depth = 1.90'

stress in flange = 162300 ÷ 1.9 = 85500# Section required = 85500 ÷ 17000 = 5020" net

Use 2 L's 5"×3½"×⅜" = 61.0 gross or 5350" net

less ¾" holes .66

4690" net

Max shear = $\frac{29475}{9}$ = 3280#/0" ots

1.12

3900" net

Approximate weight say 80# per lin ft

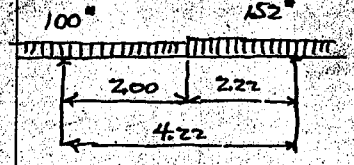
Intermediate Stringer for roadway Span length 24.0 spacing 4'-2⅞" = 4.22'

Dead Load

2" asphalt pavement 22
6" concrete slab. 75
97- call this 100# including fillers etc.

Floor slab for Electric Ry 152# per square ft

Load on stringer



Reaction $\frac{152 \times 2.22^2}{2 \times 4.22} = 88.$

$200 \times \frac{3.22}{4.22} = 153$

$100 \times \frac{4.22}{2} = 211$

452# per ft call this 450#

beam assumed 70

520# per lin ft

Bending moment = ⅛ × 520 × 24² = 37400#

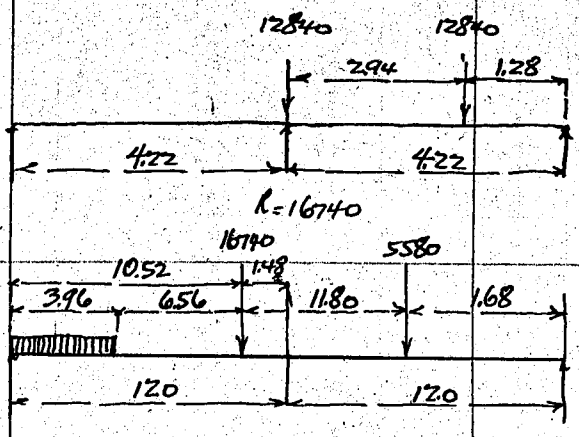
End shear = ½ × 520 × 24 = 6250#

Live Load

Rear wheel of motor trucks including impact = 12840#

Front wheel concentration 12840 × ⅓ = 4280

max Reaction on stringer



Reaction 12840 × $\frac{1.28}{4.22}$ = 3900

12840

Rear wheel 16740# } 22320#

Front wheel ⅓ × 16740 = 5580

Uniform Live Load = 123# × 4.22 = 520# per lin ft

moment due to motor trucks = 22320 × $\frac{10.52^2}{24.0}$ = 103.000#

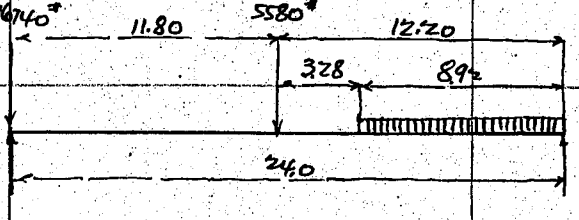
Moment due to Unif. load = 170 × 1348 = 2300

105300#

Reaction 520 × 3.96 = 2060#

2060 × $\frac{1.98}{24.0}$ = 170#

End shear



motor trucks conc. = 5580 × $\frac{12.20}{24.0}$ = 2840

16740

19580#

Uniform load = 4640 × $\frac{4.46}{24}$ = 860

20440#

Uniform load = 520 × 8.92 = 4640#

Summary for moments and shears

	Moment	Shear
Dead Load	37400	6250
Live Load	105300	20440
	142700#	26690#

section mod = $\frac{142700 \times 12}{17000}$ = 101.0

Use 20"I-65.4#

増田淳氏関係資料
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CALCULATIONS FOR

Design of Chorokubashi for Kumamoto-ken

Sidewalk Stringer at Centre of Sidewalk Span length 24'-0" spacing 5'-10⅞" = 5.84'

Dead Load

¾" wearing course	= 90	
4" Reinforced Concrete slab	= 500	
59# call this 60# per ft	×5.84 =	351
Load from water main	$100 \times \frac{409}{5.84}$ =	70
Conduit pipes		20
		441
beam assumed		45
		486# per ft

Dead Load Moment = ⅛ × 486 × 24² = 35100# End shear = $\frac{486}{2} \times 24$ = 5830#

Live Load

Uniform live load	102 × 5.84 =	say	600#
Reaction from water main	$35 \times \frac{409}{584}$ =		25
			625# per ft

Moment = ⅛ × 625 × 24² = 45000#
Shear = ½ × 625 × 24 = 7500#

Summary for shears and moments

	Moment	Shear
Dead Load	35100	5830
Live Load	45000	7500
	80100#	13330#

Section modulus rqd = $\frac{80100 \times 12}{17000}$ = 56.5

Use 15"I. 42.9# Sm = 58.9 ok

Fascia Girder at sidewalk span length 24'-0"

Dead Load

Sidewalk slab and wearing course	60 × 3.31 =	198.
weight of Handrail assumed		60
Load from conduit pipe		20
		278
Dead Load Beam assumed		45
		323# per ft

Bending moment = ⅛ × 323 × 24² = 23300# End shear = $323 \times \frac{24}{2}$ = 3880#

Live Load

Uniform live load = 102 × 29½ = say 300# per ft
Bending Moment = ⅛ × 300 × 24² = 21600# End shear = $300 \times \frac{24}{2}$ = 3600#

Summary for moments and shears

	Moment	Shear
Dead Load	23300	3880
Live Load	21600	3600
	44900#	7480#

Try 18" × 5/16" web area = 5.62" ⅛ web = 0.700"
Effective depth say 1.44 stress = 31200#
section required = 31200 ÷ 17000 = 1.835
70
1.135

Use 1L 3.3 × ⅜" = 2.11 gross − 0.65 = 1.460" net
web can be made 16" at centre of panels.

Approximate weight of fascia girder

1 web pl average	20 × 5/16 =	21.25
2 L 3.3 × ⅜ @ 7.2	=	14.4
details say		6.35
		44.00# per lin ft.

Stringer at curb Line.

Dead Load sidewalk slab 60 × 2.92 = 175
water main $100 \times \frac{1.75}{5.84}$ = 30
curb stone 65
Roadway slab say $100 \times \frac{422}{2}$ = 211
addition misc details 50
Dead Load of beam assumed 50
581 call this 600# per lin ft

Bending Moment = ⅛ × 600 × 24² = 43200 1# End shear = ½ × 600 × 24 = 7200#

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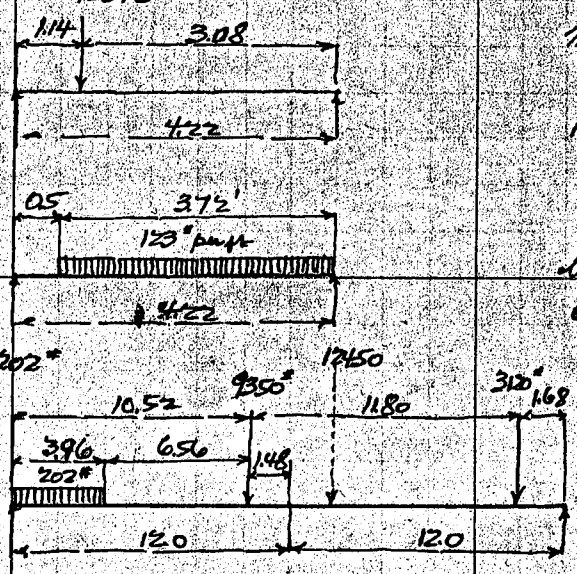
CALCULATIONS FOR

Design of Clove Kubashi for Kumamoto Line

Live Load

Uniform live load on sidewalks say 102# × (5.84/2 + 0.5) = say 350#
Weight of water in water main 10
360# per lin ft

Motor truck rear wheel concentration = 12840# with impact
" " front " " 4280# " "
Reaction on stringer 12840 × 3.08/4.22 = 9350#
front wheel 3120#
12450



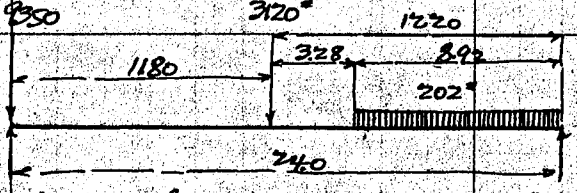
Uniform live load 123 × 3.72² / 2 × 4.22 = 202# per lin ft
on roadway 360 on sidewalks
562# per lin ft

moment due to motor trucks = 12450 × 10.52²/24.0 = 57500 in# 57500
Uniform load 202 × 3.96 = 800#
Reaction = 800 × 1.98/24.0 = .660# moment = 660 × 13.48 = say 900 in#

moment due to unif. load on sidewalks = 1/8 × 360 × 24² = 26000 in#
Total. 84400 in#

End Shear

motor trucks loading
Rear wheel 9350
Front wheel 3120 × 12.20/24.0 = 1590
10940



Uniform load = 202 × 8.92 = 1800#

Unif load on roadway 1800 × 4.46/24.0 = say 340
Unif. load on sidewalks 360 × 24/2 = 4320
Total shear 15600#

Summary for moments and shears

	Moment	Shears
Dead Load	43200	7200
Live Load	84400	15600
	127600 in#	22800#

Try 24 × 7/16 = 7.50# ⅞ web = .94 24½" b to b of L's.
Effective depth = 1.86' Stress = 127600 ÷ 1.86 = 68500#
Section reqd = 68500 ÷ 17000 = 4.03
Less .94
3.09 sq in
Use 1 PL 4 × 4 × ½" = 3.75 sq in gross or 3.250 sq in

Approximate weight of Stringers

2 Ls 4 × 4 × ½	@	12.8#	= 25.6
1 web 24 × 5/16	@	25.5	25.5
1 shelf 1 3 × 3 × 5/16	@	6.1	6.1
			57.2
details + rivets			4.8
			62.0#

Summary for Stringers

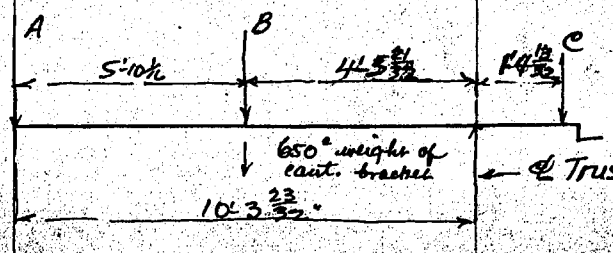
Fascia girder	44.0
Sidewalk stringer	45.0
Curb stringer	62.0
Lwy stringers 3@70 =	210.0
Elec. Ry stringers 2@80 =	160.0
	521.0# for ½ width

For whole width say 1040# per lin ft.

Design of Cantilever Bracket for Intermediate Floor Beam spacing 24.0'

Dead Load

Dead load concentration at A (Fascia girder)
See page 56 conc = 323 × 24 = 7750#
Dead Load conc at B. = 486 × 24 = 11700#
Bending moment at ℄ of truss
7750 × 10.31 = 80.000
11700 × 4.47 = 52300
650 × 4.47 = 2900
135200 in#



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CALCULATIONS FOR

Design of Chorokubashi for Kumamoto Ken

Live Load Uniform live Load 102 × 24 = 2448# per lin ft
moment at ℄ of truss = 2448 × $\frac{10.31^2}{2}$ = 133000 ft
center main 35# × 24 = 840#
moment = 840 × 2.72 = 2300
135300 ft

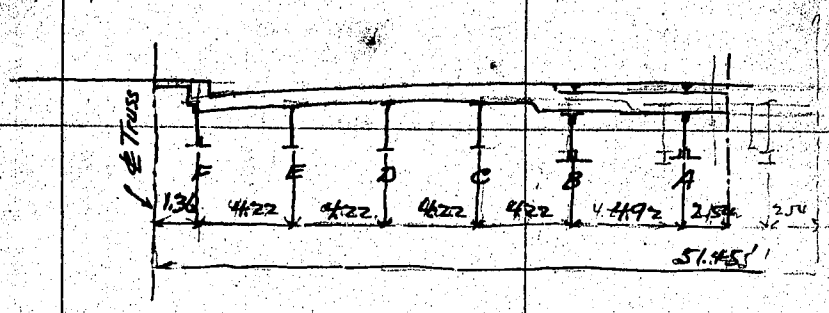
Dead Load moment = 135200
Live Load moment = 135300
Total 270500 ft

Moment at B.
Dead Load moment 7750 × 5.83 = 45200
Live Load moment 2448 × $\frac{5.83^2}{2}$ = 41700
Cantilever Bracket say 800#
87700 ft

Shear DL 7750 + 11700 + 650 = 20100#
LL 2448 × 10.31 = 25200
Total 45300#

Effective depth at truss say 4.0 assumed stress = 270500 ÷ 4 = 67600#
Section required = 67600# ÷ 17000 = 3.98 □" web assumed 48 × 3/8 = 18.00 □" & web = 2.25 □"
net section required = 3.98 − 2.25 = 1.73 □" net
moment at B = 87700 section at B = Effective depth say 3.0 stress = 87700 ÷ 3.0 = 29200#
Use same section as for ℄ truss.
Section for Cantilever Bracket 2 ∠s 3 × 3 × 3/8" = 4.22 □" or 3.47 □" net
Approximate weight of Cantilever Bracket
1 web pl av. 38 × 3/8 @ 48.45
4 ∠s 3 × 3 × 3/8 @ 7.2 28.80
77.25
Details say 25% 15.50
92.75# × 10.31 = say 1000# per piece.

Design of Intermediate Floor Beam
Dead Load
Dead Load Concentration from Stringers.



A 865 × 24 = 20800# page 54
B 152 × 2.46 = 374 from between A+B
200 × $\frac{10}{4.22}$ = 48 " " B+C
152 × 2.22 = 337 @ $\frac{3.11}{4.22}$ = 248 B+C
670 × 24 = 16100#
C 520 × 24 = 12500#
D 100 × 4.22 = 422
Stringer 70
492 × 24 = 11800#
E same as D 11800
F 600 × 24 = 14400#

Dead Load of Beam assumed 450# per lin ft

End shear A = 20800
B = 16100
C = 12500
D = 11800
E = 11800
F = 14400
87400#
Beam 450 × $\frac{51.45}{2}$ = 11600
Total. 99000#

Moment at center of span
87400 × 25.72 = 2250000
A 20800 × 2.54 = 52800
B 16100 × 7.46 = 120000
C 12500 × 11.68 = 146000
D 11800 × 15.90 = 187500
E 11800 × 20.12 = 238000
F 14400 × 24.34 = 351000
1095300
11547.00 ft
Beam = $\frac{1}{8}$ × 450 × 51.45² = 149000
Total Dead Load moment 1,303,700 ft
Less cantilever moment − 135200
Dead Load moment net 1.168500 ft

Live Load
Electric Car loading 2-30 ton coupled cars.
Coefficient of impact = $\frac{20}{60 + \frac{48}{328}}$ = 0.268
Equiv. Elec. Car. 16800
Impact 26.8% 4500
21300#

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（独立行政法人 土木研究所蔵）

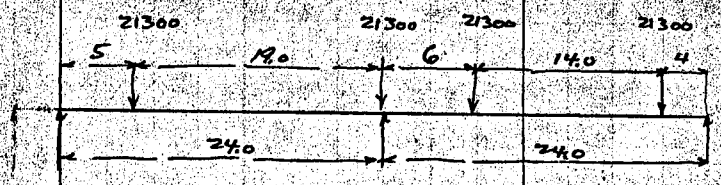
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CALCULATIONS FOR

Design of Horokubashi for Kumamoto-Ken

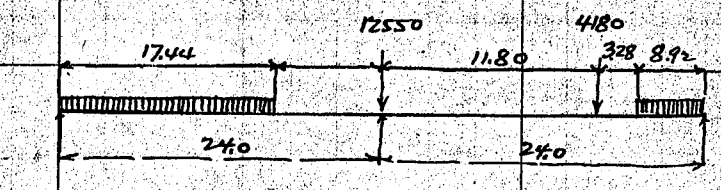
Electric Cy. Loading



Concentration with impact = 21300# per axel

Reaction on Floor Beam = 21300 × 57/24 = 23950
21300
with impact 45250#
22625# per rail

Motor trucks Loading



Rear wheel Concentration = 9900
Imp. 26.8% 2650
12550#

Front wheel conc = 1/3 × 12550 = 4180#

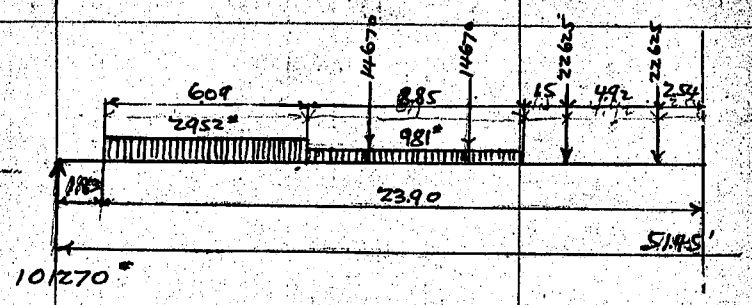
Uniform Live Load 123#/0'

Reaction due to Uniform live load

Reaction = 123 · 17.44²/(2·24) = 777

123 · 8.92²/(2·24) = 204
981# per ft strip

For full Uniform load = 123 · 24 = 2952# per ft strip.



101270#

Reaction 981 · 8.85 = 8680#
2952 · 6.09 = 18000

Unif. load 26680
Conc. 2@14670 = 29340
Conc. 2@22625 = 45250
101270#

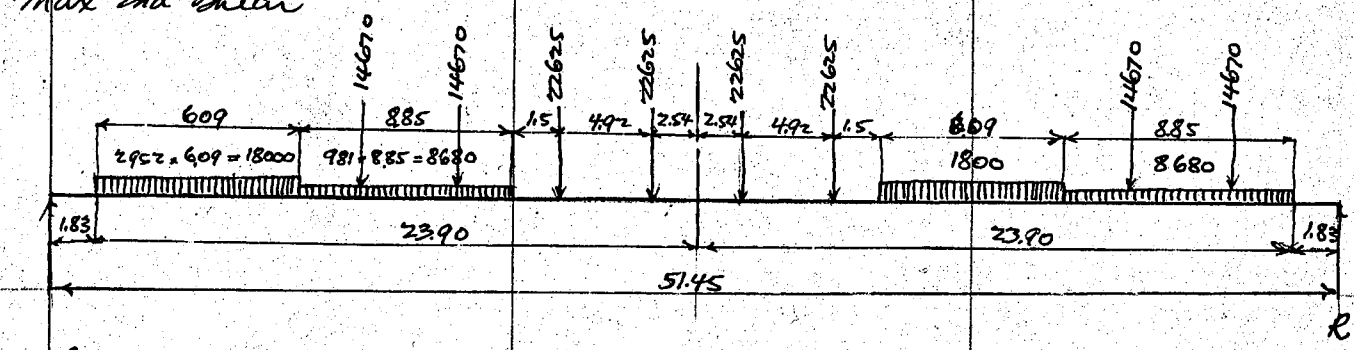
Reaction due to motor trucks loading
Front wheel 4180 · 12.20/24.00 = 2120
Rear wheel 12550
14670#

Moment due to concentration
74590 · 25.72 = 1919000
22625 · 10 = 226250
14670 · 26.76 = 392200
- 618450
1300550# --- 1300550

Moment due to Unif. load
26680 · 25.72 = 686000
8680 · 13.38 = 116200
18000 · 20.85 = 375000
- 491200

194800
Total moment 1495350#

Max End shear



Reaction due to Electric Car loading 2@22625 = 45250
" " " motor trucks 2×14670 · 57.54/51.45 = 32800
Uniform load 8680 · 57.54/51.45 = 9700
" " 18000 × 42.60 ÷ 51.45 = 14900
102650#

Summary for moments and shears

	Moment	Shear
Dead Load	1,168,500	99000
Live Load	1495350	102650
	2,663,850#	201650#

Web plate assumed 60 × 7/16 = 26.25
1/8 web = 3290" b to b of ls 60 1/2"
Effective depth say 4.92
sores = 2663850 ÷ 4.92 = 542000#
Section required = 542000 ÷ 17000 = 31.90
3.29
28.61 □" net

Use 2 ls	6×6×3/4	=	16.88 □"	13.88 □" net	
1 Pl.	14 × 5/8	=	8.75	7.50	39'
1 Pl	14 × 5/8	=	8.75	7.50	28.5'
			34.38 □"	28.88 □" net	

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CALCULATIONS FOR

Design of Chorokubashi for Kumamoto-Ken

Approximate weight of floor beam

1 web pl. 60 - ℔	@	89.3 #	. 51.5 =	4600
4 Ls 6 × 6 × 3/4	@	28.7	. 51.5 =	5910
2 cov. plo 14 × 5/8	@	29.75	. 39.0 =	2360
2 cov plo. 14 × 5/8	@	29.75	. 28.5 =	1700
				14530 #
Details say 25%				3620
				18150 # ÷ 515 = 352 # per lin ft
weight of cantilever Brackets 2@1000 =				2000
				20150 # or 9.0 tons

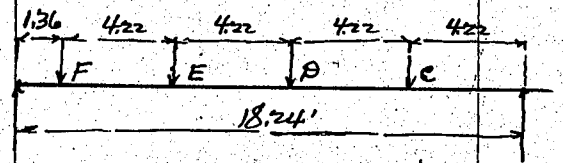
Design of End Floor Beam
Cantilever Bracket see sketch on page 57.0
Overhang say 2.25' Concentration at A = $323 \cdot \frac{26.25^2}{2 \times 24}$ = 4640#

Concentration at B = $486 \cdot \frac{26.25^2}{2 \times 24}$ = 6980#

Bending moment at ℄ of truss = A 4640 . 10.31 = 48000
B 6980 . 4.47 = 31200
650 . 4.47 = 2900
Dead Load m. 82100
Live Load Moment see p. 58 = 13530 . 0.597 = 80900
163000 in#

Use section of this cantilever bracket same as for intermediate floor beam
Approximate weight of bracket say 1000# per piece.

End floor beam between trusses divided into 3 spans, center span 14.92' side spans 18.24' about
Side span under motor way.
Dead Load



Unif. load - 100# per lin ft

at C	0.597 . 12500 =	7450
D	0.597 . 11800 =	7050
E	0.597 . 11800 =	7050
F	0.597 . 14400 =	8600

Reaction R1

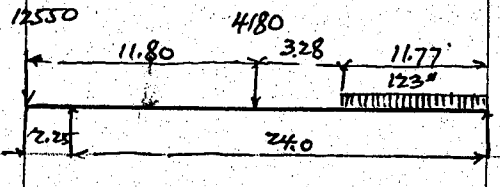
7450	. 4.22 =	31500
7050	. 8.44 =	59500
7050	. 12.66 =	89200
8600	. 16.88 =	145000
30150		325200 in# ÷ 18.24 = 17800#

Reaction R2 = 30150 - 17800 = 12350#

Dead Load moment at D

12350 . 8.44	=	104000
7450 . 4.22	=	31500
		72500 in#
beam = 1/8 . 100 . 18.24²	=	41500
		114000 in#

Live Load Moment



motor trucks loading 12550 . 26.25/24 = 13750

4180 . 14.45/24 = 2520

16270 in#

Uniform load at front of motor trucks
load = 123 . 11.77 = say 1450# R = 1450 . 588/24 = 355#
Uniform load = 123 . 26.25²/2×24 = 1770# per ft strip

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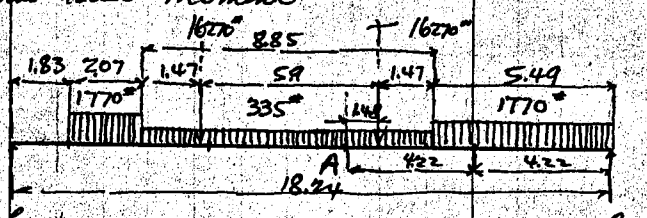
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CALCULATIONS FOR

Design of chorokubashi for Kumamoto-Ken

Live Load Moment



load 1770 × 2.07 = 3660#
1770 × 5.49 = 9700#
355 × 8.85 = 3140#

Reaction R_1

$2 \times 16270 \times \frac{9.91}{18.24} = 17650$

$9700 \times 2.75 \div 18.24 = 1465$
$3140 \times 9.91 \div 18.24 = 1700$
$3660 \times 15.37 \div 18.24 = 3080$
6245#

17650
6245
23895#

moment due to motor trucks loading

17650 × 9.80 = 173000
16270 × 4.42 = 72000
101000#

moment due to Unif. load

6245 × 9.80 = 61200
3660 × 6.93 = 25400
$335 \times \frac{4.4^2}{2}$ = 3240
28640

32560

Summary for Live load Moment

Motor trucks = 101000
Unif. load = 32560
133560#

Live Load shear.

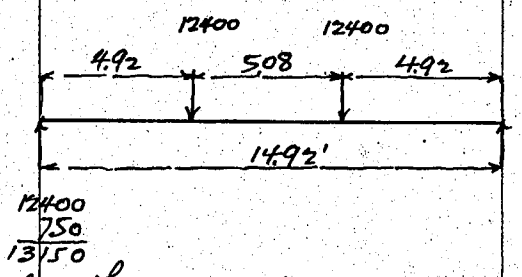
motor trucks loading R_2 $2 \times 16270 \times 13.82 \div 18.24 = 24700$
Unif. load 3140 × 13.82 ÷ 18.24 = 2380
" " 13360 × 5.61 ÷ 18.24 = 4100
31180# call this 32000# for both ends

Summary for moments and shears.

	Moment	Shear
Dead Load	114000	17800
Live Load	133560	32000
	247560#	49800#

End Floor Beam center span

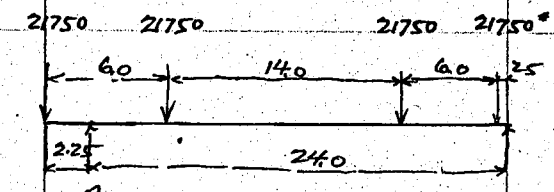
Dead Load



12400
750
13150

Dead Load Concentration 0.597 × 20800 = 12400
Dead Load beam assumed 100# per ft
Dead Load Moment = 12400 × 4.92 = 61000#
$\frac{1}{8} \times 100 \times 14.92^2$ = 27800
88800#

Live Load Electric Car loading



Impact allowance = $\frac{20}{60 + \frac{2625}{328}} = 29.4\%$

Concentration Elec. Car 16800
Imp 29.4% 4950
21750#

Reaction R_1 $21750 \times \frac{53}{24} = 48000\#$ or $24000\#$ per rail.

Live Load Moment = 24000 × 4.92 = 118000# Shear = 24000#

Summary for moments and shears

	Moments	Shear
Dead Load	88800	13150
Live Load	118000	24000
	206800#	37150#

Design End Floor beam for End span moment = 247560# shear = 49800#
web assumed 36 × 9/16 = 11.25 ℄ web = 1.41 0" back to back of Ls = 36½"
Effective depth say 2.87' stress = 247560 ÷ 2.87 = 86200# SR = 86200 ÷ 17000 = 5.07
Use 2Ls 3½ × 3½ × 9/16 = 4.18 0" gr 3.56 0" net
1.41
3.66 0" net

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CALCULATIONS FOR

Design of Chorotsubashi for Kumamoto-shi

Approximate weight of End floor beam

1 web pl 36. 5/16	@	38.25	= 38.25
4 Ls 3½. 3½. 5/16	@	7.2	= 28.80
			67.05 #
Details say 25%			= 16.80
			83.85 # call this 85# per lin ft

Let us try 33. 5/16" web = 1030" ℄ web = 1790" Backs to backs of Ls = 33½" depth say = 2.62
Stress = 247560 ÷ 2.62 = 94500.0# SR = 94500 ÷ 17000 = 5.55
1.29
4.26"

Try 2 Ls 3½. 3½. 3/8 = 496" gr 4.21" net

weights of floor beam

1 web pl 33. 5/16	@	35.06	
4 Ls 3½. 3½. 3/8	@ 8.5	= 34.0	
		69.06	
Details say		16.80	
		85.86 call this 85# per ft	

Use sister design in the course of detailing the end floor beam.

weight of End floor beam
85. 50 = 4250#
brackets 2 @ 1000 = 2000
6250#

Load on shoe.
Dead Load.

From center span	13150 #
From end span. say	12350
Stringer B 0.597 × 16100 =	9600
	35100#

Live Load

From center span	24000
Stringer	24000
From end span	32000
	80000#

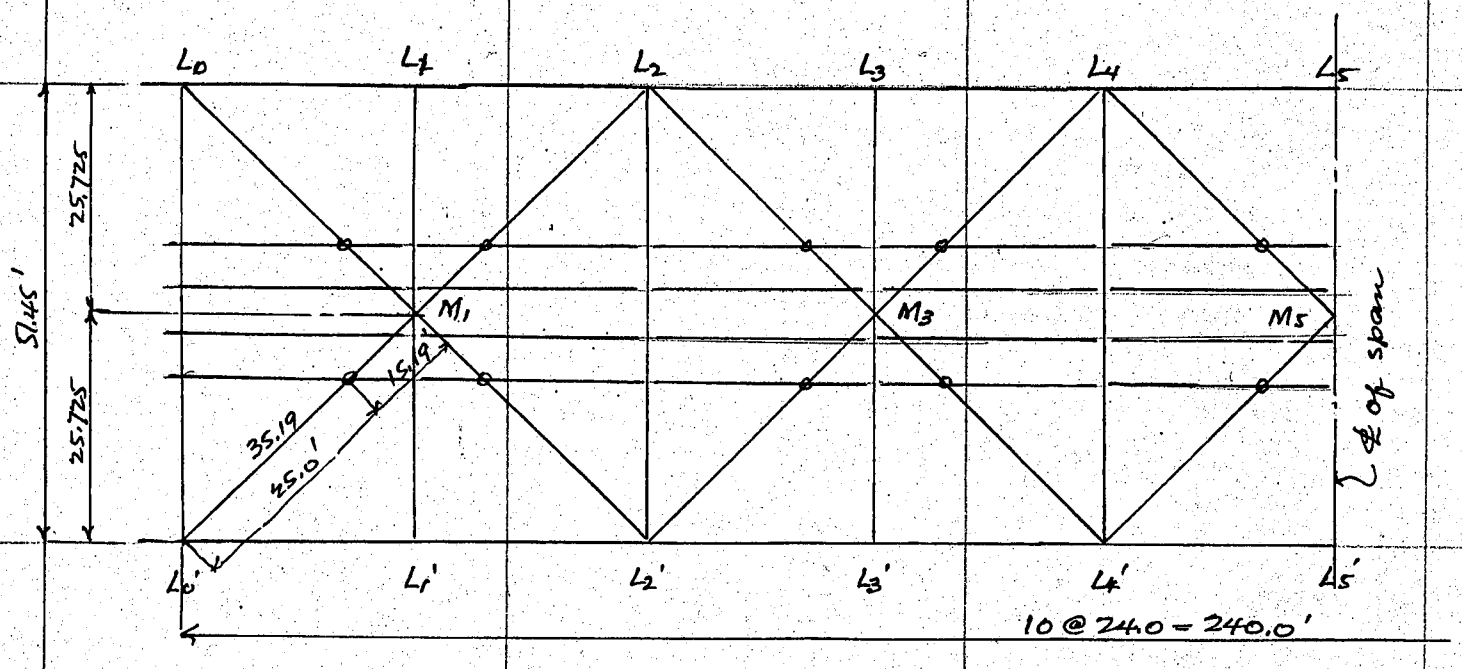
Total load on bearing = 115100#

Bearing area required
= 115100 ÷ 640 = 180"
Use 18" × 18" bed plate under shoe of end floor beam.

Weights of metal in Floor Beams

Intermediate Floor beam	9 @	20150	= 181350
End Floor beam	2 @	6250	= 12500
			193850# 86.4 tons.

Lower Lateral Bracing



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CALCULATIONS FOR

Design of Chōroku bashi for Kumamoto-Ken.

$\sec\theta = \frac{3519}{240} = 1.466$

Wind pressure on lower chord assumed as 300# per lin ft treated as moving load.

Panel concentration = 300 · 24 = 7200# End reaction = 7200 · 45 = 32400#

Stress in diagonal at end panel = $\frac{32400}{2} \cdot 1.466 = 23800$# tension or compression.

Earthquake Effect

Dead Load of floor say	8120
Dead Load metal stringer	1040
Intermediate floor beam	840
Lower lateral Bracing	115
trusses (ties and Hangers.)	1150

Seismic force 11265# · 0.2 = 2250# per lin ft

Panel load = 2250 · 24 = 54000#

Reaction at end 54000 · 4.5 = 243000#

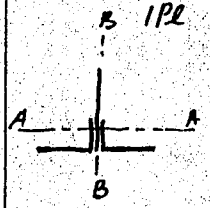
member	stress	Unit stress	Section Rqd	No of Rivets 7/8"ø
L_0-M_1	178000#	30600#	5.81 □" net	18
M_1-L_2	138500	"	4.52	14
L_2-M_3	99000	"	3.23	10
M_3-L_4	59400	"	1.94	6
L_4-M_5	19800	"		

On Compression member l/r for wind bracing limited to 150 $\frac{102}{68}$

Unsupported length of member = 25.0 or 300" $r = \frac{300}{150} = 2.00$

Section assumed

				moment of inertia
2L 6×4×3/8	= 7.22	· 0.94 = 6.79	7.22 · 1.41^2 + 9.8	= 24.1
1Pl 10×3/8	= 3.75	· 5.06 = 19.00	3.75 · 2.71^2 + 31.25	= 58.85
	10.97	2.35 25.79		82.95



AA axis radius of gyration = $\sqrt{\frac{82.95}{10.97}} = 2.75$ $r = 1.88$

BB axis " " " = 2.87 about

$l/r = 300 \div 2.75 = 109$

Unit stress for compression = 21300 (1 − 0.0055 l/r) = 8500#

For seismic stress 8500 · 1.8 = 15300#

L_0M_1	178000 ÷ 15300	= 11.62 □"	7.00	8.7	Use. L_0-M_1 2L 6×4×3/8 = 7.22
M_1-L_2	138500 ÷ 15300	= 9.05	5.45	8.7	1Pl. 10×1/2 = 5.00
L_2-M_3	99000 ÷ 15300	= 6.47	3.94	8.7	12.22 □" @ 34 = 41.6#
M_3-L_4	59400 ÷ 15300	= 3.88	2.30	8.7	For other members 2L 6×4×3/8 = 7.22
L_4-M_5	19800 ÷ 15300	= 1.30			1Pl. 10×3/8 = 3.75
					10.97 □" @ 34 = 37.2#

Approximate weight of lower lateral Bracings.

Diagonals end panel.	2 @ 41.6 · 34.0	= 2830
" other panels.	8 @ 37.2 · 34.0	= 10100

12930# × 2 = 25860 call this 26000#

Diagonals		26000
Connection	5 @ 120	600
Conn.	20 @ 30	600
Details say		800

28000 ÷ 240. = 117# per lin ft.

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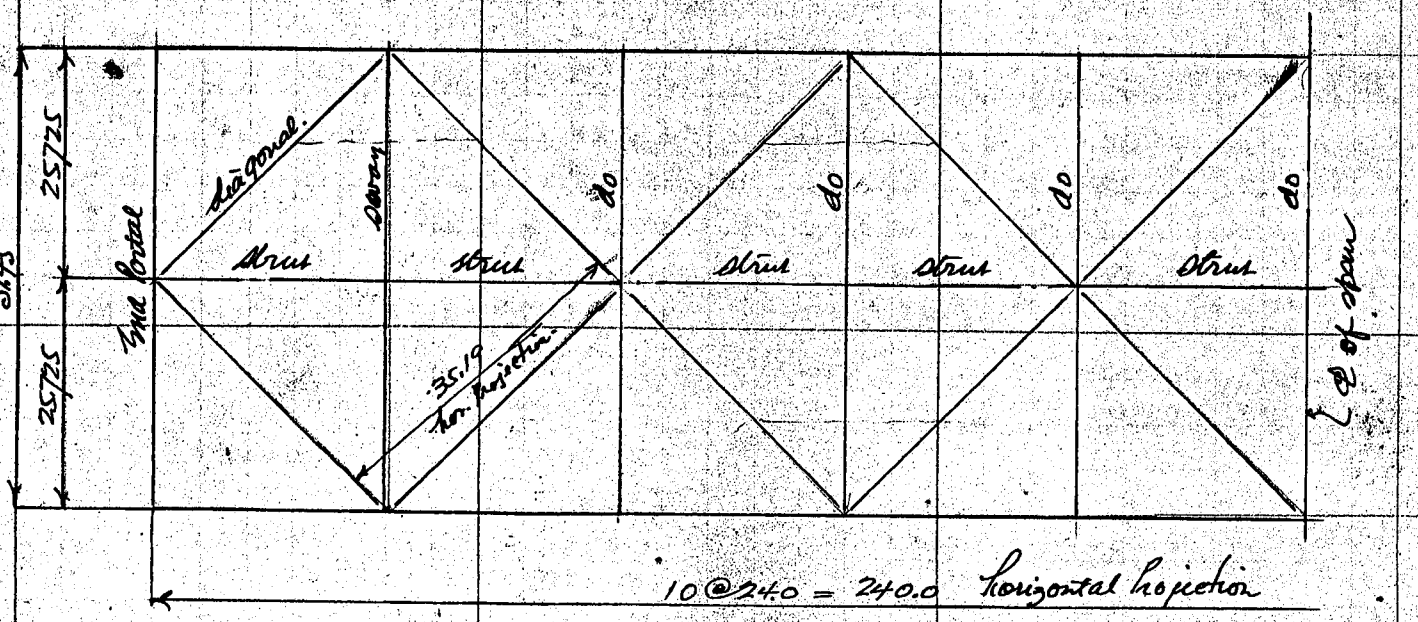
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CALCULATIONS FOR

Design of Chorokubashi for Kumamoto-Ken

Upper Lateral Bracing



Sec 0 = 1.47 wind pressure on upper chord members assumed as 150# per lin ft treated as moving load. Panel concentration = 150 × 24 = 3600# 2nd reaction = 3600 · 4.5 = 16200#
Stress in diagonal (Tension only) 16200 × 1.47 = 23800#
no of rivets 7/8"ϕ = 4 required each connection.

Seismic stress :-
weight of top lateral bracing assumed 450#
trusses assumed 3450
Seismic force 3900 × 0.2 = 780# per lin ft
Panel load 780 × 24.0 = 18700# Reaction = 18700 · 4.5 = 84000#
Stress in diagonal for end panel = 84000 × 1.47 = 123500#
Section required as tension member = 123500 ÷ 30600 = 4.030" net
no of rivets 7/8" = 6.010 × 1.8 = 10800# ; 123500 ÷ 10800 = 11.4 12' Rivets
Stress in second panel. 65400 × 1.47 = 96000# section required = 96000 ÷ 30600 = 3.140"
no of rivets = 96000 ÷ 10800 = 9.0
Stress in 3rd panel = 46700 × 1.47 = 68700. section reqd = 68700 ÷ 30600 = 2.25" net
no of rivets = 6.0

Try 2Ls 6×6×3/8 = 8.72 − 1.5 = 7.220" net
use 2Ls 6×4×3/8 = 7.22 − 1.5 = 5.720" net.

Section of longitudinal strut : span length say 24.0 l/r say 150 required = $\frac{24\times12}{150}$ = 1.92
use ~~4Ls 4×3×7/16" laced~~ 2Ls 6×4×3/8 and 1Pl 10"×3/8" r = 2.75
$l/r = \frac{24\times12}{2.75} = 105.0$

Transverse sway bracing
Use 2Ls 6"×6×3/8" for top and bottom flanges
and 2Ls 3×3×5/16 for diagonal members.
all gusset pls 5/16" thick. depth at ℄ bridge = 4'-6".

End portal Bracings
Depth at center line of bridge = 4'-6"
Section of top flange 1 cov. pl 18×5/16 web 2 webs 5/16" thick.
2Ls 6×4×3/8"
Section of bottom flange 2Ls 6×4×3/8"

Section of Longitudinal strut max length = 24'-10" about 298"
try 2Ls 5×3½×5/16 . backs to backs 3/4" r = 2.54 l/r = 298 ÷ 2.54 = 118. O.K.

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CALCULATIONS FOR

Design of Chorotsubashi for Kumamoto-Ken

Approximate weight of transverse sway bracing

Top angles.	2- 2Ls	6×6×3/8	@ 14.9#	× 24'-4"	= 1450
Bottom angles	2- 2Ls	6×6×3/8	@ 14.9#	× 25'-2"	= 1505
Diagonals	2- 2Ls	4×3×5/16	@ 7.2	× 6'-2½	= 178
"	2- 2Ls	4×3×5/16	@ "	× 5'-11½"	= 171
"	2- 2Ls	4×3×5/16	@ "	× 5'-4½"	= 155
"	2- 8Ls	4×3×5/16	@ "	× 5'-4"	= 616
verticals.	4Ls	3½×3½×5/16	@ 7.2#	× 5'-4"	= 154
fills	2Pls.	7⅜×3/8	@ 9.4	× 4'-10½"	= 91
"	1Pl	7⅜×3/8	@ 9.4	× 3'-1½"	= 29
Gusset pls.	13 @	50#			= 450
"	1 @	25			= 25
Plate angle connection		2@ 80			= 160
do.		2@ 75			= 150
					5 (4)324

Rivet heads and variation say 86
5 (4)220# per piece.

Approximate weight of End Portal.

Main section	2 webs	54×5/16	@ 57.4	=	1148
	4Ls	6×4×3/8	@ 12.3	=	492
	1 cov Pl.	18×9/16	@		19.13
				183.13 × 49.82	= 9110#
			Add for fillet at ends		500
					9610#
Details say	25%				2410
					12020# per piece.

Summary for sway bracings and End portals

Sway bracings	9 @ 4220	= 37980
End Portals	2 @ 12020	= 24040
		62020#

Approximate weight of longitudinal struts

main sect.	4Ls 5×3½×5/16	@ 8.7	× 24.3 av.	=	846.
Lacing.		9.5	× 24.0	=	238
Die Pls. say		2@100		=	200
		Rivet heads &c say			56
					1340#

Approximate weight of top Lateral Bracings.

Diagonals.	4- 2Ls	6×4×3/8	@ 12.3#	× 31'-7½"	= 3120#
"	4- 2Ls	6×4×3/8	@ "	× 32'-2½	= 3165
"	4- 2Ls	6×4×3/8	@ "	× 31'-10"	= 3140
"	4- 2Ls	6×4×3/8	@ "	× 31'-11"	= 3150
"	4- 2Ls	6×4×3/8	@ "	× 31'-9"	= 3120
Gusset pls	5 @ 150				450
" "	4 @ 220				880
	Rivet heads and variations say				340
					17365#

Summary for Top lateral bracings and Longitudinal struts

Top Lateral bracings	17365
Longl struts 10@1340 =	13400
	30765# call this 30800#

Summary for top lateral system

Sways and portals	62020
diagonals + struts	30800
	92820# ÷ 240 = 387# per ft say 400# per ft

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CALCULATIONS FOR

Design of Chorokubashi for Kumamoto-Ken

Dead Load of Floor (½ width)

stone blocks pavement	4"	54 #/sq ft	× 9.0 =	486
Lean mortar filling	1½"	18"	× 9.0 =	162
6½" reinforced concrete slab		81	× 9.5 =	770
2" asphaltic concrete		22	× 13.5 =	297
6" reinforced concrete slab.		75	× 13.5 =	1010
Reinforced concrete gutter		100	× 1.5 =	150
Curb stone				67
Concrete under curb stone say				55
¾" wearing course on sidewalk		9	× 12.25 =	110
4" Reinforced concrete slab.		50	× 12.25 =	611
				3718
water main say				150
Conduit pipes say				40
Rails and accessories				55
Handrails assumed				100
				4063

Call this 4060 # per lin ft of span

Panel Concentration -
metal in bridge

Stringers	10470 × 24 =	24960
Floor beam		20150
	45110 ÷ 24 =	1880#
Lower Lateral Bracing say		120
Upper Lateral system		400
	2400 ÷ 2 =	1200# for one truss
Dead Load Floor complete		4060 " " "
Truss assumed		2340
		7600# " " "

Concentration 7600 × 24 = 182400#
2340 × 2.5/4 × 12 say 17300#
165100#

sub-panel concentration assumed
panel concentration

Live Load on truss

Uniform load on sidewalks $\frac{100.000}{170 + \frac{240}{328}}$ = 411 kg/m² = 84.3 #/o'

Uniform load on Roadway $\frac{120.000}{170 + \frac{240}{328}}$ = 493 kg/m² = 101.0 #/o'

Full uniform Live Load on Truss.

On sidewalks	84.3 × 12.25 =	1035
	101.0 × 6.15 =	620
		1655

Panel concentration = 1655 × 24.0 = 39700#

Unif Live Load conjunction with motor trucks loading
Occupied width of motor trucks assumed as 8.85 (270c).
Uniform load at front or rear of motor trucks 8.85 × 101. = 893.0

motor trucks Loading	Rear wheel	9900	
Impact	15%	1485	
		11385 × 2 =	22770
Front wheel	⅓		7570
	Total		30340#

For panel concentration see preliminary design pp 14.
That is uniform concentration assumed 21000# per panel and 13000# concentration at any panel as extra load will be equivalent to one motor trucks loading with uniform load at rear and front of motor trucks.

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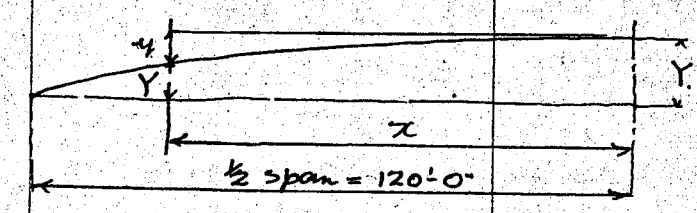
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CALCULATIONS FOR

Design of Chorokubashi for Kumamoto-Ken

Final Design of truss-
Span length = 240'-0" The floor panel divided into 10 panels and truss members into 20. The rise of lower chord 36'-0" above horizontal line of tie at both bearings and the depth of frame at center line of span = 9'-0" and 27'-0" at bearings - Due to height of filling and grade of both approaches and also flood condition; horizontal tie cambered at center of span 2'-0"
Final design of truss made by these data. Form and lengths of members of truss same as for preliminary design except said camber of tie. See sheet 16 to 19 for lengths of members.
Camber of tie 2'-0" at ℄ of span



Panel point	dist.	y	Y	
10	0	0	2.00	2'-0"
9	12	0.02	1.98	
8	24	0.08	1.92	1'-11 1/32"
7	36	0.18	1.82	
6	48	0.32	1.68	1'-8 5/32"
5	60	0.50	1.500	
4	72	0.72	1.28	1'-3 3/8"
3	84	0.98	1.02	
2	96	1.28	0.72	0'-8 5/8"
1	108	1.62	0.38	
0	120	2.00	0.00	0'-0"

Assumed section of members -

Upper chord		Lower chord		Diagonals		Verticals		Hangers	Tie
U_1	64.0 □"	L_0	155.0 □"	D_1	21.0 □"	V_0	60.0 □"	30.0 □"	138.0 □"
U_2	64.0	L_1	135.0	D_2	33.0	V_1	44.0		
U_3	90.0	L_2	120.0	D_3	28.0	V_2	48.0		
U_4	77.0	L_3	96.0	D_4	38.0	V_3	35.0		
U_5	95.0	L_4	85.0	D_5	42.0	V_4	42.0		
U_6	120.0	L_5	65.0	D_6	42.0	V_5	33.0		
U_7	140.0	L_6	50.0	D_7	33.0	V_6	35.0		
U_8	170.0	L_7	50.0	D_8	42.0	V_7	23.0		
U_9	170.0	L_8	50.0	D_9	33.0	V_8	33.0		
U_{10}	190.0	L_9	50.0	D_{10}	33.0	V_9	23.0		
						V_{10}	23.0		

Upper chord members $Sa = \frac{Hm - Y}{gm}$

Panel	Hm	Y	Hm-Y	gm	log (Hm-Y)	log gm	log $\frac{Hm-Y}{gm}$	Sa
1	7.37	0.38	6.99	22.20	0.8445	1.3464	$\bar{1}$.4981	0.315
2	13.72	0.72	13.00	19.22	1.1139	1.2838	$\bar{1}$.8301	0.676
3	19.17	1.02	18.15	16.70	1.2588	1.2226	0.0362	1.086
4	23.77	1.28	22.49	14.58	1.3519	1.1636	0.1883	1.543
5	27.58	1.50	26.08	12.81	1.4163	1.1081	0.3032	2.033
6	30.62	1.68	28.94	11.42	1.4615	1.0577	0.4038	2.534
7	33.01	1.82	31.19	10.35	1.4940	1.0149	0.4791	3.014
8	34.67	1.92	32.75	9.59	1.5152	0.9820	0.5332	3.414
9	35.67	1.98	33.69	9.15	1.5275	0.9612	0.5663	3.684
10	36.00	2.00	34.00	9.00	1.5315	0.9542	0.5773	3.779

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CALCULATIONS FOR

Design of Chorokubashi for Kumamoto-ken

Lower chord members $Da = \frac{Ym - Y}{hm}$

Panel	Ym	Y	Ym-Y	log Ym-Y	log hm	log $\frac{Ym-Y}{hm}$	Sa
0	27.00	0.00	27.00	1.4314	1.3619	0.0615	1.152
1	30.48	0.38	30.10	1.4786	1.3102	0.1684	1.473
2	33.57	0.72	32.85	1.5166	1.2572	0.2594	1.818
3	36.28	1.02	35.26	1.5472	1.2036	0.3436	2.206
4	38.61	1.28	37.33	1.5720	1.1507	0.4213	2.638
5	40.58	1.50	39.08	1.5920	1.1000	0.4920	3.105
6	42.14	1.68	40.46	1.6070	1.0535	0.5535	3.577
7	43.41	1.82	41.59	1.6189	1.0128	0.6061	4.037
8	44.29	1.92	42.37	1.6270	0.9818	0.6452	4.418
9	44.82	1.98	42.84	1.6318	0.9614	0.6704	4.681

Diagonals $Pm' = \frac{Vm-1}{Vm-1 + Vm} (Hm - Hm-1)$.

Panel	Pm'	Hm-1	pm = Pm'+Hm+1	fm	log pm	log fm	log Da	Sa
1	51.23	0	51.23	8383	1.7096	1.9234	$\bar{1}$.7862	0.612
2	44.96	6.99	51.95	81.73	1.7156	1.9124	$\bar{1}$.8032	0.635
3	39.51	13.00	52.51	79.51	1.7203	1.9005	$\bar{1}$.8198	0.660
4	34.70	18.15	52.85	77.53	1.7230	1.8895	$\bar{1}$.8335	0.682
5	30.59	22.49	53.08	76.71	1.7249	1.8849	$\bar{1}$.8400	0.692
6	27.12	26.08	53.20	78.42	1.7259	1.8944	$\bar{1}$.8315	0.678
7	24.25	28.94	53.19	85.07	1.7258	1.9298	$\bar{1}$.7960	0.625
8	22.12	31.19	53.31	103.22	1.7268	2.0138	$\bar{1}$.7130	0.516
9	20.71	32.75	53.46	153.39	1.7280	2.1858	$\bar{1}$.5422	0.349
10	19.70	33.69	53.39	432.02	1.7274	2.6355	$\bar{1}$.0919	0.124

Verticals $qm = \frac{Vm}{(Hm+1 - Hm) - (Ym - Ym-1)}$ when $Ym = Hm + Vm$

Panel	qm'	Hm	qm = qm'+Hm	im-ma	log qm	log im-ma	log Sa	Sa
1	51.16	6.99	58.15	96.58	1.7646	1.9849	$\bar{1}$.7797	0.602
2	45.89	13.00	58.89	101.18	1.7701	2.0051	$\bar{1}$.7650	0.582
3	41.63	18.15	59.78	108.63	1.7766	2.0359	$\bar{1}$.7407	0.550
4	38.16	22.49	60.65	120.02	1.7829	2.0793	$\bar{1}$.7036	0.505
5	36.05	26.08	62.13	140.90	1.7933	2.1487	$\bar{1}$.6446	0.441
6	35.84	28.94	64.78	182.48	1.8114	2.2612	$\bar{1}$.5502	0.355
7	40.36	31.19	71.55	290.55	1.8546	2.4647	$\bar{1}$.3899	0.245
8	85.46	32.75	118.22	1030.90	2.0729	3.0132	1.0595	0.115
9	−15.30	33.69	18.39	−555.00	1.2646	−2.7443	$\bar{2}$.5203	−0.033

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CALCULATIONS FOR

Design of chorokubashi for Kumamoto-Ken

To find S_{o1} S_a ℓ/A.

upper chord

	S_{o1}	S_a	ℓ	A	$\log S_{o1}$	$\log S_a$	$\log \ell$	$\log A$		$S_{o1} S_a \ell/A$	
U_1	0.027	0.315	12.495'	64.0°	$\bar{2}.4314$	$\bar{1}.4981$	1.0967	1.8062	$\bar{3}.2200$	0.0017	A
U_2	0.062	0.676	12.391	64.0	$\bar{2}.7924$	$\bar{1}.8301$	1.0931	1.8062	$\bar{3}.9094$	0.0081	B
U_3	0.108	1.086	12.302	70.0	$\bar{1}.0334$	0.0362	1.0900	1.8451	$\bar{2}.3145$	0.0206	C
U_4	0.165	1.543	12.224	77.0	$\bar{1}.2175$	0.1883	1.0872	1.8865	$\bar{2}.6065$	0.0404	D
U_5	0.234	2.033	12.159	95.0	$\bar{1}.3692$	0.3082	1.0849	1.9777	$\bar{2}.7846$	0.0609	E
U_6	0.315	2.534	12.107	120.0	$\bar{1}.4983$	0.4038	1.0830	2.0792	$\bar{2}.9059$	0.0805	F
U_7	0.406	3.014	12.063	140.0	$\bar{1}.6085$	0.4791	1.0814	2.1461	$\bar{1}.0229$	0.1054	G
U_8	0.501	3.414	12.031	170.0	$\bar{1}.6998$	0.5332	1.0803	2.2304	$\bar{1}.0829$	0.1211	H
U_9	0.590	3.684	12.010	170.0	$\bar{1}.7709$	0.5663	1.0796	2.2304	$\bar{1}.1864$	0.1536	K
U_{10}	0.667	3.779	12.000	190.0	$\bar{1}.8241$	0.5773	1.0792	2.2788	$\bar{1}.2018$	0.1592	L
U_{10}'	0.667	3.779	12.000	190.0	$\bar{1}.8241$	0.5773	1.0792	2.2788	$\bar{1}.2018$	0.1592	M
U_9'	0.721	3.684	12.010	170.0	$\bar{1}.8579$	0.5663	1.0796	2.2304	$\bar{1}.2734$	0.1877	N
U_8'	0.751	3.414	12.031	170.0	$\bar{1}.8756$	0.5332	1.0803	2.2304	$\bar{1}.2587$	0.1814	O
U_7'	0.754	3.014	12.063	140.0	$\bar{1}.8774$	0.4791	1.0814	2.1461	$\bar{1}.2918$	0.1958	P
U_6'	0.735	2.534	12.107	120.0	$\bar{1}.8663$	0.4038	1.0830	2.0792	$\bar{1}.2739$	0.1879	Q
U_5'	0.703	2.033	12.159	95.0	$\bar{1}.8470$	0.3082	1.0849	1.9777	$\bar{1}.2624$	0.1830	R
U_4'	0.658	1.543	12.224	77.0	$\bar{1}.8182$	0.1883	1.0872	1.8865	$\bar{1}.2070$	0.1612	S
U_3'	0.611	1.086	12.302	70.0	$\bar{1}.7860$	0.0362	1.0900	1.8451	$\bar{1}.0671$	0.1167	T
U_2'	0.562	0.676	12.391	64.0	$\bar{1}.7497$	$\bar{1}.8301$	1.0931	1.8062	$\bar{2}.8667$	0.0736	U
U_1'	0.513	0.315	12.495	64.0	$\bar{1}.7101$	$\bar{1}.4981$	1.0967	1.8062	$\bar{2}.4987$	0.0315	V

Lower chord

	S_{o1}	A	$\log S_{o1}$	$\log S_a$	$\log \ell$	$\log A$	$\log \frac{S_{o1} S_a \ell}{A}$	$S_o S_a \frac{\ell}{A}$	
L_0	0.000	155.0°	$-\infty$	0.0615	1.1486	2.1903	$-\infty$	0	A
L_1	0.029	135.0	$\bar{2}.4624$	0.1684	1.1329	2.1303	$\bar{3}.6334$	0.0043	B
L_2	0.066	120.0	$\bar{2}.8195$	0.2594	1.1198	2.0792	$\bar{2}.1195$	0.0132	C
L_3	0.113	96.0	$\bar{1}.0531$	0.3436	1.1090	1.9823	$\bar{2}.5234$	0.0333	D
L_4	0.170	85.0	$\bar{1}.2304$	0.4213	1.1001	1.9294	$\bar{2}.8224$	0.0634	E
L_5	0.238	65.0	$\bar{1}.3766$	0.4920	1.0929	1.8129	$\bar{1}.1486$	0.1408	F
L_6	0.318	50.0	$\bar{1}.5024$	0.5535	1.0874	1.6990	$\bar{1}.4443$	0.2782	G
L_7	0.408	50.0	$\bar{1}.6107$	0.6061	1.0833	1.6990	$\bar{1}.6011$	0.3991	H
L_8	0.501	50.0	$\bar{1}.6998$	0.6452	1.0807	1.6990	$\bar{1}.7267$	0.5329	K
L_9	0.590	50.0	$\bar{1}.7709$	0.6704	1.0794	1.6990	$\bar{1}.8217$	0.6633	L
L_9'	0.721	50.0	$\bar{1}.8579$	0.6704	1.0794	1.6990	$\bar{1}.9087$	0.8108	M
L_8'	0.751	50.0	$\bar{1}.8756$	0.6452	1.0807	1.6990	$\bar{1}.9025$	0.7989	N
L_7'	0.757	50.0	$\bar{1}.8791$	0.6061	1.0833	1.6990	$\bar{1}.8695$	0.7404	O
L_6'	0.743	50.0	$\bar{1}.8710$	0.5535	1.0874	1.6990	$\bar{1}.8129$	0.6500	P
L_5'	0.714	65.0	$\bar{1}.8537$	0.4920	1.0929	1.8129	$\bar{1}.6257$	0.4224	Q
L_4'	0.679	85.0	$\bar{1}.8319$	0.4213	1.1001	1.9294	$\bar{1}.4239$	0.2655	R
L_3'	0.638	96.0	$\bar{1}.8048$	0.3436	1.1090	1.9823	$\bar{1}.2751$	0.1884	S
L_2'	0.597	120.0	$\bar{1}.7760$	0.2594	1.1198	2.0792	$\bar{1}.0760$	0.1191	T
L_1'	0.558	135.0	$\bar{1}.7466$	0.1684	1.1329	2.1303	$\bar{2}.9176$	0.0827	U
L_0'	0.000	155.0	$-\infty$	0.0615	1.1486	2.1903	$-\infty$	0.0000	V

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Design of Horokubashi for Kumamoto-ken

Diagonals.

	S_{01}	A	$\log S_{01}$	$\log S_a$	$\log l$	$\log A$	$\log S_{01}S_a\frac{l}{A}$	$S_{01}S_a\frac{l}{A}$	
D_1	0.050	21.0°	$\bar{2}.6990$	$\bar{1}.7862$	1.3619	1.3222	$\bar{2}.5249$	0.0335	A
D_2	0.059	33.0	$\bar{2}.7709$	$\bar{1}.8032$	1.3143	1.5185	$\bar{2}.3699$	0.0234	B
D_3	0.070	28.0	$\bar{2}.8451$	$\bar{1}.8198$	1.2730	1.4472	$\bar{2}.4907$	0.0310	C
D_4	0.082	38.0	$\bar{2}.9138$	$\bar{1}.8335$	1.2390	1.5798	$\bar{2}.4065$	0.0255	D
D_5	0.094	42.0	$\bar{2}.9731$	$\bar{1}.8400$	1.2122	1.6232	$\bar{2}.4021$	0.0252	E
D_6	0.106	42.0	$\bar{1}.0253$	$\bar{1}.8315$	1.1924	1.6232	$\bar{2}.4260$	0.0267	F
D_7	0.115	33.0	$\bar{1}.0607$	$\bar{1}.7960$	1.1790	1.5185	$\bar{2}.5172$	0.0329	G
D_8	0.118	42.0	$\bar{1}.0719$	$\bar{1}.7130$	1.1715	1.6232	$\bar{2}.7915$	0.0619	H
D_9	0.112	33.0	$\bar{1}.0492$	$\bar{1}.5422$	1.1696	1.5185	$\bar{2}.2425$	0.0175	K
D_{10}	0.095	33.0	$\bar{2}.9777$	$\bar{1}.0919$	1.1730	1.5185	$\bar{3}.7841$	0.0053	L
D_{10}'	−0.067	33.0	$-\bar{2}.8261$	$\bar{1}.0919$	1.1730	1.5185	$-\bar{3}.5725$	−0.00037	M
D_9'	−0.034	33.0	$-\bar{2}.5315$	$\bar{1}.5422$	1.1696	1.5185	$-\bar{3}.7258$	−0.00053	N
D_8'	−0.062	42.0	$-\bar{3}.3010$	$\bar{1}.7130$	1.1715	1.6232	$-\bar{4}.5623$	−0.0004.	O
D_7'	+0.026	33.0	$\bar{2}.4150$	$\bar{1}.7960$	1.1790	1.5185	$\bar{3}.8715$	0.00074	P
D_6'	0.047	42.0	$\bar{2}.6721$	$\bar{1}.8315$	1.1924	1.6232	$\bar{2}.0728$	0.0118	Q
D_5'	0.063	42.0	$\bar{2}.7993$	$\bar{1}.8400$	1.2122	1.6232	$\bar{2}.2283$	0.0169	R
D_4'	0.073	38.0	$\bar{2}.8633$	$\bar{1}.8335$	1.2390	1.5798	$\bar{2}.3560$	0.0227	S
D_3'	0.080	28.0	$\bar{2}.9031$	$\bar{1}.8198$	1.2730	1.4472	$\bar{2}.5487$	0.0354	T
D_2'	0.088	33.0	$\bar{2}.9445$	$\bar{1}.8032$	1.3143	1.5185	$\bar{2}.5435$	0.0350	U
D_1'	0.946	21.0	$\bar{1}.9759$	$\bar{1}.7862$	1.3619	1.3222	$\bar{1}.8018$	0.6336	V

Verticals

	S_{01}	A	$\log S_{01}$	$\log S_a$	$\log l$	$\log A$	$\log S_{01}S_a\frac{l}{A}$	$S_{01}S_a\frac{l}{A}$	
V_0	0.00	60.0°		$-\infty$	1.4314		$-\infty$	0.0	A
V_1	0.056	44.0	$\bar{2}.7482$	$\bar{1}.7797$	1.3639	1.6435	$\bar{2}.2483$	0.0177	B
V_2	0.062	48.0	$\bar{2}.7924$	$\bar{1}.7650$	1.2977	1.6812	$\bar{2}.1739$	0.0149	C
V_3	0.070	35.0	$\bar{2}.8451$	$\bar{1}.7407$	1.2334	1.5441	$\bar{2}.2751$	0.0188	D
V_4	0.070	42.0	$\bar{2}.8451$	$\bar{1}.7036$	1.1716	1.6232	$\bar{2}.0971$	0.0125	E
V_5	0.071	33.0	$\bar{2}.8513$	$\bar{1}.6446$	1.1138	1.5185	$\bar{2}.0912$	0.0123	F
V_6	0.070	35.0	$\bar{2}.8451$	$\bar{1}.5502$	1.0616	1.5441	$\bar{3}.9128$	0.00082	G
V_7	0.064	23.0	$\bar{2}.8062$	$\bar{1}.3899$	1.0172	1.3617	$\bar{3}.8516$	0.00071	H
V_8	0.055	33.0	$\bar{2}.7404$	$\bar{1}.0595$	0.9832	1.5185	$\bar{3}.2646$	0.0018	K
V_9	0.040	23.0	$\bar{2}.6021$	$-\bar{2}.5203$	0.9616	1.3617	$-\bar{4}.7223$	−0.0005	L
V_9'	−0.061	23.0	$-\bar{2}.7853$	$-\bar{2}.5203$	0.9616	1.3617	$+\bar{4}.9055$	0.0008	M
V_8'	−0.043	33.0	$-\bar{2}.6335$	$\bar{1}.0595$	0.9832	1.5185	$-\bar{3}.1577$	−0.00014	N
V_7'	−0.023	23.0	$-\bar{2}.3617$	$\bar{1}.3899$	1.0172	1.3617	$-\bar{3}.4071$	−0.00026	O
V_6'	−0.004	35.0	$-\bar{3}.6021$	$\bar{1}.5502$	1.0616	1.5441	$-\bar{3}.6698$	−0.00047	P
V_5'	0.014	33.0	$\bar{2}.1461$	$\bar{1}.6446$	1.1138	1.5185	$\bar{3}.3860$	0.00024	Q
V_4'	0.030	42.0	$\bar{2}.4771$	$\bar{1}.7036$	1.1716	1.6232	$\bar{3}.7291$	0.00054	R
V_3'	0.040	35.0	$\bar{2}.6021$	$\bar{1}.7407$	1.2334	1.5441	$\bar{2}.0321$	0.0108	S
V_2'	0.057	48.0	$\bar{2}.7559$	$\bar{1}.7650$	1.2977	1.6812	$\bar{2}.1374$	0.0137	T
V_1'	1.068	44.0	0.0285	$\bar{1}.7797$	1.3639	1.6435	$\bar{1}.5286$	0.3378	U
V_0'		60.0		$-\infty$	1.4314		$-\infty$	0.0	N

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Summary Σ Sa So $\frac{l}{A}$.

Sa So.1 $\frac{l}{A}$	Sa So.2 $\frac{l}{A}$	Sa So.3 $\frac{l}{A}$	Sa So.4 $\frac{l}{A}$	Sa So.5 $\frac{l}{A}$	Sa So.6 $\frac{l}{A}$	Sa So.7 $\frac{l}{A}$	Sa So.8 $\frac{l}{A}$	Sa So.9 $\frac{l}{A}$	Sa So.10 $\frac{l}{A}$
A	2A	3A	4A	5A	6A	7A	8A	9A	10A
B	2B	3B	4B	5B	6B	7B	8B	9B	10B
C	2C	3C	4C	5C	6C	7C	8C	9C	10C
D	2D	3D	4D	5D	6D	7D	8D	9D	10D
E	2E	3E	4E	5E	6E	7E	8E	9E	10E
F	2F	3F	4F	5F	6F	7F	8F	9F	10F
G	2G	3G	4G	5G	6G	7G	8G	9G	10G
H	2H	3H	4H	5H	6H	7H	8H	9H	10H
K	2K	3K	4K	5K	6K	7K	8K	9K	10K
L	2L	3L	4L	5L	6L	7L	8L	9L	10L
M	2M	3M	4M	5M	6M	7M	8M	9M	10L
N	2N	3N	4N	5N	6N	7N	8N	11K	10K
O	2O	3O	4O	5O	6O	7O	12H	11H	10H
P	2P	3P	4P	5P	6P	13G	12G	11G	10G
Q	2Q	3Q	4Q	5Q	14F	13F	12F	11F	10F
R	2R	3R	4R	15E	14E	13E	12E	11E	10E
S	2S	3S	16D	15D	14D	13D	12D	11D	10D
T	2T	17C	16C	15C	14C	13C	12C	11C	10C
U	18B	17B	16B	15B	14B	13B	12B	11B	10B
V	18A	17A	16A	15A	14A	13A	12A	11A	10A

Log. $Sa^2\frac{l}{A}$ and $Sa^2\frac{l}{A}$ for truss members

Upper chord			Lower chord			Diagonals			Verticals		
U_1	$\bar{2}.2867$	0.0194	L_0	$\bar{1}.0813$	0.1206	D_1	$\bar{1}.6121$	0.4094	V_1	$\bar{1}.2798$	0.1904
U_2	2.9471	0.0885	L_1	$\bar{1}.3394$	0.2185	D_2	$\bar{1}.4022$	0.2524	V_2	$\bar{1}.1465$	0.1402
U_3	$\bar{1}.3173$	0.2076	L_2	$\bar{1}.5594$	0.3625	D_3	$\bar{1}.4654$	0.2920	V_3	$\bar{1}.1707$	0.1481
U_4	$\bar{1}.5773$	0.3779	L_3	$\bar{1}.8139$	0.6515	D_4	$\bar{1}.3262$	0.2119	V_4	$\bar{2}.9556$	0.0903
U_5	$\bar{1}.7236$	0.5291	L_4	0.0133	1.0311	D_5	$\bar{1}.2690$	0.1858	V_5	$\bar{2}.8845$	0.0767
U_6	$\bar{1}.8114$	0.6477	L_5	0.2640	1.8370	D_6	$\bar{1}.2322$	0.1707	V_6	$\bar{2}.6179$	0.0415
U_7	$\bar{1}.8935$	0.7825	L_6	0.4954	3.1290	D_7	$\bar{1}.2525$	0.1788	V_7	$\bar{2}.4353$	0.0272
U_8	$\bar{1}.9163$	0.8247	L_7	0.5965	3.9490	D_8	$\bar{2}.9743$	0.0943	V_8	$\bar{3}.5837$	0.0038
U_9	1.9818	0.9590	L_8	0.6721	4.7000	D_9	$\bar{2}.7355$	0.0544	V_9	$\bar{4}.6405$	0.0004
U_{10}	1.9550	0.9016	L_9	0.7212	5.2620	D_{10}	$\bar{3}.8383$	0.0069	V_{10}		
	$\Sigma Sa^2\frac{l}{A} \rightarrow$	10.6760			42.5224			3.7131			1.4364

Upper chords 10.6760
lower chords 42.5224
Diagonals 3.7131
Verticals 1.4364
$\Sigma Sa^2\frac{l}{A} \longrightarrow$ 58.3479
$\frac{lt}{At} = 240 \div 138$ 1.7390
60.0869 $= \Sigma Sa\frac{l}{A} + \frac{lt}{At}$

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	Upper chord			Lower chord		Diagonals		Verticals	
	Sa	So.1	Summary	Sa	So.1	Summary	Sa	So.1	Summary
A	0.0017		0.0017	0.0000		0.0000	0.0335	0.0335	0.0000	0.0000
B	0.0081		0.0098	0.0043		0.0043	0.0234	0.0569	0.0177	0.0177
C	0.0206		0.0304	0.0132		0.0175	0.0310	0.0879	0.0149	0.0326
D	0.0404		0.0708	0.0333		0.0508	0.0255	0.1134	0.0188	0.0514
E	0.0609		0.1317	0.0634		0.1142	0.0252	0.1386	0.0125	0.0639
F	0.0805		0.2122	0.1408		0.2550	0.0267	0.1653	0.0123	0.0762
G	0.1054		0.3176	0.2782		0.5332	0.0329	0.1982	0.0082	0.0844
H	0.1211		0.4387	0.3991		0.9323	0.0619	0.2601	0.0071	0.0915
K	0.1536		0.5923	0.5329		1.4652	0.0175	0.2776	0.0018	0.0933
L	0.1592		0.7515	0.6633		2.1285	0.0053	0.2829	−0.0005	0.0928
M	0.1592		0.9107	0.8108		2.9393	−0.0037	0.2792	0.0008	0.0936
N	0.1877		1.0921	0.7989		3.7382	−0.0053	0.2739	−0.0014	0.0922
O	0.1814		1.2735	0.7404		4.4786	−0.0004	0.2735	−0.0026	0.0896
P	0.1958		1.4693	0.6500		5.1286	0.0074	0.2809	−0.0047	0.0849
Q	0.1879		1.6572	0.4224		5.5510	0.0118	0.2927	0.0024	0.0873
R	0.1830		1.8402	0.2655		5.8165	0.0169	0.3096	0.0054	0.0927
S	0.1612		2.0014	0.1884		6.0049	0.0227	0.3323	0.0108	0.1035
T	0.1167		2.1181	0.1191		6.1240	0.0354	0.3677	0.0137	0.1172
U	0.0736		2.1917	0.0827		6.2067	0.0350	0.4027	0.3378	0.4550
V	0.0315		2.2232	0.0000		6.2067	0.6336	1.0363	0.0000	0.4550

Upper chord

Load on	A			B			A+B
1	2.2232						2.2232
2	2. 2.1181	=	4.2362	18. 0.0098	=	0.1764	4.4126
3	3. 2.0014	=	6.0042	17. 0.0304	=	0.5168	6.5210
4	4. 1.8402	=	7.3608	16. 0.0708	=	1.1328	8.4936
5	5. 1.6572	=	8.2860	15. 0.1317	=	1.9755	10.2615
6	6. 1.4693	=	8.8158	14. 0.2122	=	2.5708	11.3866
7	7. 1.2735	=	8.9145	13. 0.3176	=	4.1288	13.0433
8	8. 1.0921	=	8.7368	12. 0.4387	=	5.2644	14.0012
9	9. 0.9107	=	8.1963	11. 0.5923	=	6.5153	14.7116
10	10. 0.7515	=	7.5150	10. 0.7515	=	7.5150	15.0300

Lower chords

Load on	A			B			A+B
1	6.2067						6.2067
2	2. 6.1240	=	12.2480	18. 0.0043	=	0.0774	12.3254
3	3. 6.0049	=	18.0147	17. 0.0175	=	0.2975	18.3122
4	4. 5.8165	=	23.2660	16. 0.0508	=	0.8128	24.0788
5	5. 5.5510	=	27.7550	15. 0.1142	=	1.7130	29.4680
6	6. 5.1286	=	30.7716	14. 0.2550	=	3.5700	34.3416
7	7. 4.4768	=	31.3502	13. 0.5332	=	6.9316	38.2818
8	8. 3.7382	=	29.9056	12. 0.9323	=	11.1876	41.0932
9	9. 2.9393	=	26.4537	11. 1.4652	=	16.1172	42.5709
10	10. 2.1285	=	21.2850	10. 2.1285	=	21.2850	42.5700

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Diagonals

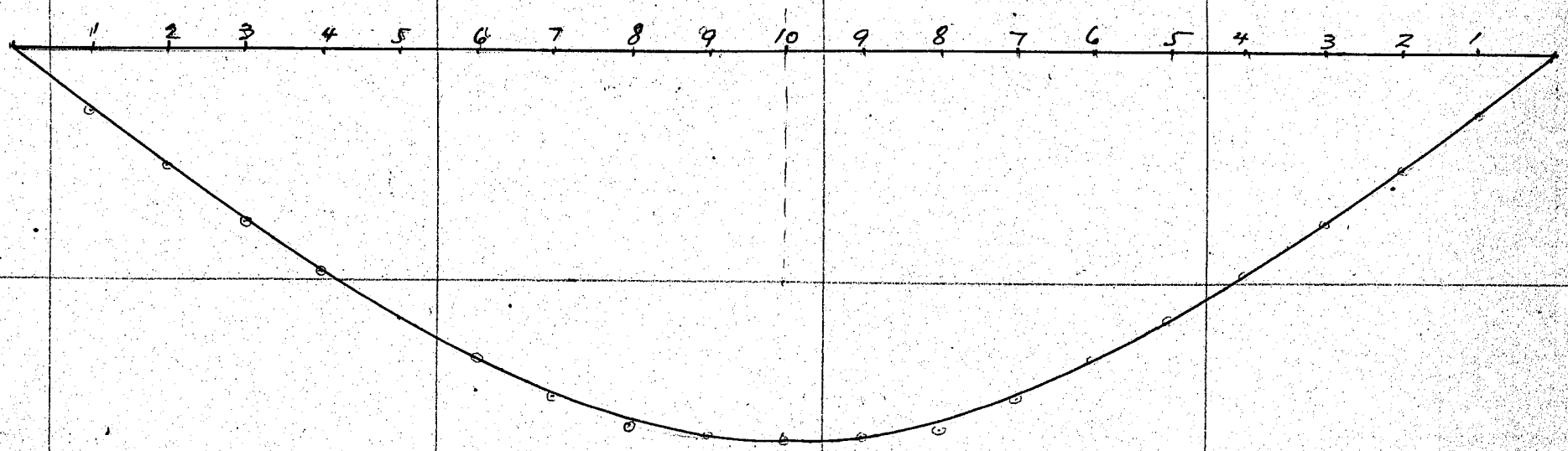
Load on	A			B	A+B
1	1.0363				1.0363
2	2·0.3677 = 0.7354	18·0.0569	=	1.0242	1.7596
3	3·0.3323 = 0.9969	17·0.0879	=	1.4943	2.4912
4	4·0.3096 = 1.2384	16·0.1134	=	1.8144	3.0528
5	5·0.2927 = 1.4635	15·0.1386	=	2.0790	3.5425
6	6·0.2809 = 1.6854	14·0.1653	=	2.3142	3.9996
7	7·0.2735 = 1.9145	13·0.1987	=	2.5766	4.4911
8	8·0.2739 = 2.1912	12·0.2601	=	3.1212	5.3124
9	9·0.2792 = 2.5128	11·0.2776	=	3.0536	5.5664
10	10·0.2829 = 2.8290	10·0.2829	=	2.8290	5.6580

Verticals

Load on	A			B	A+B
1	0.4550				0.4550
2	2·0.1172 = 0.2344	18·0.0177	=	0.3186	0.5530
3	3·0.1035 = 0.3105	17·0.0326	=	0.5542	0.8647
4	4·0.0927 = 0.3708	16·0.0514	=	0.8224	1.1932
5	5·0.0873 = 0.4365	15·0.0639	=	0.9585	1.3950
6	6·0.0849 = 0.5094	14·0.0762	=	1.0668	1.5762
7	7·0.0896 = 0.6272	13·0.0844	=	1.0972	1.7244
8	8·0.0922 = 0.7376	12·0.0915	=	1.0980	1.8356
9	9·0.0936 = 0.8424	11·0.0933	=	1.0263	1.8687
10	10·0.0928 = 0.9280	10·0.0928	=	0.9280	1.8560

Load on	Upper chord	Lower chord	Diagonals	Verticals	Summary	Value of H (60.0869)
1	2.2232	6.2067	1.0363	0.4550	9.9212	0.165
2	4.4126	12.3254	1.7596	0.5530	19.0506	0.318
3	6.5210	18.3122	2.4912	0.8647	28.1891	0.470
4	8.4936	24.0788	3.0528	1.1932	36.8184	0.614
5	10.2615	29.4680	3.5425	1.3950	44.6670	0.744
6	11.3866	34.3416	3.9996	1.5762	51.3040	0.855
7	13.0433	38.2818	4.4911	1.7244	57.5406	0.959
8	14.0012	41.0932	5.3124	1.8656	62.2724	1.038
9	14.7116	42.5709	5.5664	1.8687	64.7175	1.078
10	15.0300	42.5700	5.6580	1.8560	65.1140	1.085

Influence diagram for Horizontal thrust



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CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Design of Chorokubashi for Kumamoto-Ken

Upper chord

member	Sa	Xa →	1	2	3	4	5	6	7	8	9	10
			0.165	0.318	0.490	0.614	0.744	0.855	0.959	1.038	1.078	1.085
1	0.315		0.052	0.100	0.148	0.193	0.234	0.269	0.302	0.327	0.340	0.342
2	0.676		0.112	0.215	0.318	0.415	0.503	0.578	0.648	0.702	0.729	0.733
3	1.086		0.179	0.345	0.510	0.667	0.808	0.929	1.041	1.127	1.170	1.178
4	1.543		0.255	0.491	0.725	0.947	1.148	1.319	1.480	1.602	1.663	1.674
5	2.033		0.335	0.646	0.956	1.248	1.513	1.738	1.950	2.110	2.192	2.206
6	2.534		0.418	0.806	1.191	1.556	1.885	2.167	2.430	2.630	2.732	2.749
7	3.014		0.497	0.958	1.417	1.851	2.242	2.577	2.890	3.129	3.249	3.270
8	3.414		0.563	1.086	1.605	2.096	2.540	2.919	3.274	3.544	3.680	3.704
9	3.684		0.608	1.172	1.731	2.262	2.741	3.150	3.533	3.824	3.972	3.997
10	3.779		0.624	1.202	1.771	2.320	2.812	3.231	3.684	3.923	4.074	4.100

Lower chord

member	Load on		1	2	3	4	5	6	7	8	9	10
	Sa	Xa =	0.165	0.318	0.470	0.614	0.744	0.855	0.959	1.038	1.078	1.085
0	1.152		0.190	0.366	0.541	0.707	0.857	0.985	1.105	1.196	1.242	1.250
1	1.473		0.243	0.468	0.692	0.904	1.096	1.259	1.413	1.529	1.588	1.598
2	1.818		0.300	0.578	0.854	1.116	1.353	1.554	1.743	1.887	1.942	1.973
3	2.206		0.364	0.702	1.037	1.354	1.641	1.886	2.116	2.290	2.378	2.394
4	2.638		0.435	0.839	1.240	1.620	1.963	2.255	2.530	2.738	2.844	2.862
5	3.105		0.512	0.987	1.459	1.906	2.310	2.655	2.978	3.223	3.347	3.369
6	3.577		0.590	1.137	1.681	2.196	2.661	3.058	3.430	3.712	3.856	3.881
7	4.037		0.666	1.284	1.897	2.479	3.004	3.452	3.871	4.190	4.352	4.380
8	4.418		0.729	1.405	2.076	2.713	3.287	3.779	4.237	4.586	4.763	4.794
9	4.681		0.772	1.489	2.200	2.874	3.483	4.002	4.489	4.859	5.046	5.079

Diagonals

member	Load on		1	2	3	4	5	6	7	8	9	10
	Sa	Xa	0.165	0.318	0.490	0.614	0.744	0.855	0.959	1.038	1.078	1.085
1	0.612		0.101	0.195	0.288	0.376	0.455	0.523	0.587	0.636	0.660	0.664
2	0.635		0.105	0.202	0.298	0.390	0.472	0.543	0.606	0.659	0.685	0.689
3	0.660		0.109	0.210	0.310	0.405	0.491	0.564	0.633	0.686	0.712	0.716
4	0.682		0.112	0.217	0.321	0.419	0.508	0.583	0.654	0.708	0.735	0.740
5	0.692		0.114	0.220	0.325	0.425	0.518	0.592	0.664	0.718	0.746	0.751
6	0.678		0.112	0.216	0.319	0.416	0.504	0.580	0.650	0.704	0.731	0.736
7	0.625		0.103	0.199	0.294	0.384	0.465	0.534	0.600	0.649	0.674	0.678
8	0.516		0.085	0.164	0.243	0.317	0.384	0.441	0.495	0.536	0.556	0.560
9	0.349		0.058	0.111	0.164	0.214	0.260	0.298	0.335	0.362	0.376	0.379
10	0.124		0.020	0.039	0.058	0.076	0.092	0.106	0.119	0.129	0.134	0.135

Verticals

member	Load on		1	2	3	4	5	6	7	8	9	10
	Sa	Xa =	0.165	0.318	0.490	0.614	0.744	0.855	0.959	1.038	1.078	1.085
1	0.602		0.099	0.191	0.283	0.370	0.448	0.515	0.577	0.625	0.649	0.653
2	0.582		0.096	0.185	0.274	0.358	0.433	0.498	0.558	0.604	0.628	0.632
3	0.550		0.091	0.175	0.258	0.338	0.409	0.570	0.528	0.571	0.593	0.597
4	0.505		0.083	0.161	0.237	0.310	0.376	0.432	0.484	0.524	0.545	0.548
5	0.441		0.073	0.140	0.207	0.271	0.328	0.377	0.423	0.458	0.476	0.478
6	0.355		0.059	0.113	0.167	0.218	0.264	0.304	0.341	0.368	0.373	0.385
7	0.245		0.040	0.078	0.115	0.151	0.182	0.210	0.235	0.254	0.264	0.266
8	0.115		0.019	0.037	0.054	0.071	0.086	0.098	0.110	0.119	0.124	0.125
9	-0.033		-0.005	-0.011	-0.016	-0.020	-0.025	-0.028	-0.032	-0.034	-0.036	-0.036

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CALCULATIONS FOR

Design of Chōgotsubashi for Kumamoto-Ken

Upper chord $M_m = M_{mo} - x_a \frac{H_m - Y}{q_m}$

Load on → / member	1	2	3	4	5	6	7	8	9	10
M1	-0.025	-0.046	-0.067	-0.085	-0.099	-0.107	-0.113	-0.111	-0.097	-0.072
2	-0.050	-0.090	-0.131	-0.165	-0.185	-0.203	-0.211	-0.202	-0.167	-0.109
3	-0.071	-0.130	-0.187	-0.236	-0.269	-0.283	-0.287	-0.265	-0.200	-0.101
4	-0.090	-0.162	-0.231	-0.289	-0.275	-0.332	-0.328	-0.235	-0.181	-0.028
5	-0.101	-0.177	-0.253	-0.311	-0.342	-0.333	-0.311	-0.236	-0.085	0.136
6	-0.103	-0.176	-0.246	-0.296	-0.310	-0.277	-0.225	-0.110	0.103	0.401
7	-0.091	-0.146	-0.200	-0.228	-0.214	-0.143	-0.050	+0.216	0.403	0.788
8	-0.062	-0.084	-0.103	-0.093	-0.036	0.086	0.231	0.462	0.826	1.303
9	-0.018	0.008	0.039	0.098	+0.209	0.390	0.597	0.896	1.338	1.903
10	0.043	0.131	0.229	0.346	0.521	0.769	1.043	1.410	1.926	2.567
10'	0.043	0.131	0.229	0.346	0.521	0.769	1.043	1.410	1.926	2.567
9'	0.113	0.270	0.432	0.622	0.865	1.177	1.515	2.945	2.518	1.903
8'	0.183	0.416	0.648	0.909	1.216	1.588	1.984	2.465	1.828	1.303
7'	0.257	0.549	0.844	1.163	1.526	1.945	2.385	1.741	1.215	0.788
6'	0.317	0.664	1.014	1.384	1.790	2.243	1.665	1.150	0.733	0.401
5'	0.368	0.760	1.150	1.564	2.002	1.542	1.090	0.702	0.386	0.136
4'	0.403	0.836	1.250	1.686	1.320	0.985	0.660	0.373	0.147	-0.028
3'	0.432	0.876	1.322	1.056	0.808	0.579	0.359	0.166	-0.015	-0.101
2'	0.450	0.908	0.743	0.585	0.434	0.296	0.164	0.048	-0.042	-0.109
1	0.461	0.386	0.311	0.239	0.171	0.109	0.049	-0.003	-0.043	-0.072

Influence surface for upper chord

Load on → / member	M_1	M_2	M_3	M_4	M_5	M_6	M_7	M_8	M_9	M_{10}
1	-0.025	-0.050	-0.071	-0.090	-0.101	-0.103	-0.091	-0.062	-0.018	0.043
2	-0.046	-0.090	-0.130	-0.162	-0.177	-0.176	-0.146	-0.084	0.008	0.131
3	-0.067	-0.131	-0.187	-0.231	-0.253	-0.246	-0.200	-0.103	0.039	0.229
4	-0.085	-0.165	-0.236	-0.289	-0.311	-0.296	-0.228	-0.093	0.098	0.346
5	-0.099	-0.185	-0.269	-0.325	-0.342	-0.310	-0.214	-0.036	0.209	0.521
6	-0.107	-0.203	-0.283	-0.332	-0.333	-0.277	-0.143	0.086	0.390	0.769
7	-0.113	-0.211	-0.287	-0.328	-0.311	-0.225	-0.050	0.231	0.599	1.043
8	-0.111	-0.202	-0.265	-0.285	-0.236	-0.110	0.116	0.462	0.896	1.410
9	-0.097	-0.167	-0.200	-0.181	-0.085	0.103	0.403	0.826	1.338	1.926
10	-0.072	-0.109	-0.101	-0.028	0.136	0.401	0.788	1.303	1.903	2.567
9'	-0.043	-0.042	+0.015	0.147	0.386	0.733	1.215	1.828	2.518	1.926
8'	-0.003	0.048	+0.166	0.373	0.702	1.150	1.741	2.465	1.945	1.410
7'	0.049	0.164	0.359	0.660	1.090	1.665	2.385	1.984	1.515	1.043
6'	0.109	0.296	0.579	0.985	1.542	2.243	1.945	1.588	1.177	0.769
5'	0.171	0.434	0.808	1.320	2.002	1.790	1.526	1.216	0.865	0.521
4'	0.239	0.585	1.056	1.686	1.564	1.384	1.163	0.909	0.622	0.346
3'	0.311	0.743	1.322	1.250	1.150	1.064	0.844	0.648	0.432	0.229
2'	0.386	0.908	0.876	0.836	0.760	0.664	0.549	0.416	0.270	0.131
1'	0.461	0.450	0.432	0.403	0.368	0.317	0.257	0.183	0.113	0.043

Summary

Add	-0.411	-0.786	-1.014	-1.155	-1.092	-0.884	-0.555	-0.201	-0.018	0.000
panel	+0.992	+1.791	+2.936	+3.780	+4.996	+5.920	+6.630	+6.916	+7.626	7.524
Even	-0.424	-0.769	-1.015	-1.096	-1.057	-0.859	-0.517	-0.177	0.000	0.000
Panel	+0.734	+1.837	+2.677	+3.880	+4.704	+5.842	+6.302	+7.229	7.309	7.879

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CALCULATIONS FOR

Design of Chorokubashi for Kumamoto-Ken

Lower Chord

Member \ Loadon	1	2	3	4	5	6	7	8	9	10
Lo	-0.190	-0.366	-0.541	-0.707	-0.857	-0.985	-1.105	-1.196	-1.242	-1.260
1	-0.214	-0.409	-0.604	-0.787	-0.949	-1.083	-1.207	-1.294	-1.324	-1.304
2	-0.234	-0.445	-0.655	-0.851	-1.020	-1.156	-1.279	-1.356	-1.345	-1.309
3	-0.251	-0.477	-0.699	-0.904	-1.078	-1.210	-1.328	-1.389	-1.365	-1.268
4	-0.265	-0.500	-0.731	-0.942	-1.115	-1.237	-1.343	-1.381	-1.317	-1.165
5	-0.274	-0.511	-0.745	-0.954	-1.111	-1.227	-1.311	-1.318	-1.204	-0.987
6	-0.272	-0.500	-0.726	-0.923	-1.069	-1.148	-1.202	-1.165	-0.990	-0.697
7	-0.258	-0.468	-0.674	-0.847	-0.966	-1.005	-1.017	-0.928	-0.682	-0.304
8	-0.228	-0.403	-0.574	-0.710	-0.783	-0.772	-0.732	-0.580	-0.257	0.213
9	-0.188	-0.309	-0.430	-0.514	-0.533	-0.462	-0.359	-0.139	0.264	0.821
9'	-0.051	-0.047	-0.037	0.010	+0.123	0.325	0.558	0.910	1.444	0.854
8'	0.022	0.097	0.177	0.292	0.469	0.730	1.021	1.423	0.745	0.244
7'	0.091	0.230	0.374	0.549	0.782	1.091	1.429	0.702	0.133	-0.276
6'	0.153	0.348	0.547	0.775	1.052	1.398	0.710	0.108	-0.354	-0.672
5'	0.202	0.442	0.694	0.952	1.262	0.679	0.118	-0.365	-0.728	-0.965
4'	0.244	0.518	0.796	1.094	0.581	0.119	-0.325	-0.702	-0.977	-1.147
3'	0.277	0.575	0.878	0.448	0.048	-0.309	-0.652	-0.938	-1.140	-1.252
2'	0.297	0.616	0.274	-0.054	-0.358	-0.625	-0.880	-1.090	-1.212	-1.296
1'	0.315	0.060	-0.183	-0.434	-0.656	-0.848	-1.031	-1.177	-1.265	-1.294
0'	-0.190	-0.366	-0.541	+0.707	-0.857	-0.985	-1.105	-1.196	-1.242	-1.250

Influence surface for Lower chord

Loadon \ member	Lo	L1	L2	L3	L4	L5	L6	L7	L8	L9
1	-0.190	-0.214	-0.234	-0.251	-0.265	-0.274	-0.272	-0.258	-0.228	-0.188
2	-0.366	-0.409	-0.445	-0.477	-0.500	-0.511	-0.500	-0.468	-0.403	-0.309
3	-0.541	-0.604	-0.655	-0.699	-0.731	-0.745	-0.726	-0.674	-0.574	-0.430
4	-0.707	-0.787	-0.851	-0.904	-0.942	-0.954	-0.923	-0.847	-0.710	-0.514
5	-0.857	-0.949	-1.020	-1.078	-1.115	-1.111	-1.069	-0.966	-0.783	-0.533
6	-0.985	-1.083	-1.156	-1.210	-1.237	-1.227	-1.148	-1.005	-0.772	-0.462
7	-1.105	-1.207	-1.279	-1.328	-1.343	-1.311	-1.202	-1.017	-0.732	-0.359
8	-1.196	-1.294	-1.356	-1.389	-1.381	-1.318	-1.165	-0.928	-0.580	-0.139
9	-1.242	-1.324	-1.345	-1.365	-1.317	-1.204	-0.990	-0.682	-0.257	0.264
10	-1.250	-1.304	-1.309	-1.268	-1.165	-0.987	-0.697	-0.304	0.213	0.821
9'	-1.242	-1.265	-1.212	-1.140	-0.977	-0.728	-0.354	0.133	0.745	1.444
8'	-1.196	-1.177	-1.090	-0.938	-0.702	-0.365	0.108	0.702	1.423	0.910
7'	-1.105	-1.031	-0.880	-0.652	-0.325	0.118	0.710	1.429	1.021	0.558
6'	-0.985	-0.848	-0.625	-0.309	0.119	0.679	1.398	1.091	0.730	0.325
5'	-0.857	-0.656	-0.358	0.048	0.581	1.262	1.052	0.782	0.469	0.123
4'	-0.707	-0.434	-0.054	0.448	1.094	0.952	0.775	0.549	0.292	0.010
3'	-0.541	-0.183	0.274	0.878	0.796	0.694	0.547	0.374	0.177	-0.037
2'	-0.366	+0.060	0.616	0.575	0.518	0.442	0.348	0.230	0.097	-0.047
1'	-0.190	+0.315	0.297	0.277	0.244	0.202	0.153	0.091	0.022	-0.051

Summary

Odd	-7.870	-7.433	-6.983	-6.513	-6.073	-5.373	-4.613	-3.597	-2.574	-1.598
panel	+0.000	+0.315	+0.571	+1.203	+1.621	+2.276	+2.462	+2.809	+2.434	+2.389
Even	-7.758	-7.336	-6.886	-6.495	-5.927	-5.362	-4.433	-3.552	-2.465	-1.471
Panel	+0.000	+0.060	+0.616	+1.023	+1.731	+2.073	+2.629	+2.572	+2.755	+2.066

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CALCULATIONS FOR

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Diagonals.

Load on member	1	2	3	4	5	6	7	8	9	10
D_1	-0.051	-0.095	-0.139	-0.177	-0.206	-0.224	-0.238	-0.238	-0.212	-0.156
2	-0.046	-0.083	-0.120	-0.153	-0.175	-0.187	-0.191	-0.185	-0.151	-0.096
3	-0.039	-0.070	-0.100	-0.125	-0.141	-0.145	-0.144	-0.127	-0.083	-0.017
4	-0.030	-0.054	-0.076	-0.093	-0.108	-0.093	-0.086	-0.055	-0.001	+0.056
5	-0.020	-0.032	-0.043	-0.049	-0.045	-0.028	-0.006	0.034	0.100	0.189
6	-0.006	-0.004	-0.002	+0.007	+0.025	+0.055	+0.091	0.142	0.206	0.322
7	+0.017	+0.031	+0.051	0.076	0.110	0.155	0.204	0.270	0.363	0.471
8	0.033	0.072	+0.110	0.154	0.205	0.266	0.330	0.406	0.504	0.618
9	0.054	0.113	0.172	0.234	0.301	0.375	0.450	0.535	0.633	0.742
10	0.075	0.151	0.228	0.305	0.3824	0.465	0.547	0.633	0.723	0.817
10'	-0.087	-0.173	-0.259	-0.344	-0.429	-0.508	-0.588	-0.665	-0.737	0.817
9'	-0.092	-0.179	-0.266	-0.350	-0.430	-0.502	-0.573	-0.634	-0.857	0.742
8'	-0.087	-0.168	-0.249	-0.325	-0.394	-0.453	-0.509	+0878	+0.740	0.618
7'	-0.077	-0.147	-0.216	-0.280	-0.335	-0.378	+0.894	0.730	0.590	0.471
6'	-0.065	-0.122	-0.176	-0.228	-0.269	+0.901	0.725	0.566	0.433	0.322
5'	-0.041	-0.094	-0.136	-0.173	+0.895	0.724	0.559	0.416	0.288	0.189
4'	-0.039	-0.071	-0.102	+0.887	+0.716	0.559	0.407	0.271	0.163	0.056
3'	-0.029	-0.050	+0.978	0.713	0.558	0.415	0.276	0.153	0.057	-0.017
2'	-0.017	+0.865	+0.710	0.559	0.448	0.287	0.165	0.053	-0.033	-0.096
1'	+0.845	+0.701	+0.559	0.421	0.292	0.174	0.060	-0.038	-0.112	-0.156

Influence surface for Diagonals.

Load on member	D_1	D_2	D_3	D_4	D_5	D_6	D_7	D_8	D_9	D_{10}
1	-0.051	-0.046	-0.039	-0.030	-0.020	-0.006	+0.012	0.033	0.054	0.075
2	-0.095	-0.083	-0.070	-0.054	-0.032	-0.004	0.031	0.072	0.113	0.151
3	-0.139	-0.120	-0.100	-0.076	-0.043	-0.002	0.051	0.110	0.172	0.228
4	-0.177	-0.153	-0.125	-0.093	-0.049	0.007	0.076	0.154	0.234	0.305
5	-0.206	-0.175	-0.141	-0.100	-0.045	0.025	0.110	0.205	0.301	0384
6	-0.224	-0.187	-0.145	-0.093	-0.028	0.055	0.155	0.266	0.375	0.465
7	-0.238	-0.191	-0.144	-0.086	-0.006	0.091	0.204	0.330	0.450	0.547
8	-0.238	-0.185	-0.127	-0.055	0.034	0.142	0.270	0.406	0.535	0.633
9	-0.212	-0.151	-0.083	-0.001	0.100	0.206	0.363	0.504	0.633	0.723
10	-0.156	-0.096	-0.017	0.076	0.189	0.322	0.471	0618	0.742	0.817
9'	-0.112	-0.033	+0.057	0.163	0.288	0.433	0.590	0.740	0.857	-0.737
8'	-0.038	0.053	0.153	0.271	0.410	0.566	0.730	0.878	-0.634	-0.665
7'	+0.060	0.165	0.276	0.407	0.558	0.725	0.894	-0.509	-0.573	-0.588
6'	+0.174	0.287	0415	0.559	0.724	0.901	-0.378	-0.453	-0.502	-0.508
5'	0.292	0.418	0.558	0.716	0.895	-0.269	-0.335	-0.394	-0.430	-0.427
4'	0.421	0.559	0.713	0.887	-0.173	-0.228	-0.280	-0.325	-0.350	-0.344
3'	0.559	0.710	0878	-0.102	-0.136	-0.176	-0.216	-0.249	-0.266	-0259
2'	0.701	0.865	-0.050	-0.071	-0.094	-0.122	-0.147	-0.168	-0.179	-0.173
1'	0.845	-0.017	-0.029	-0.039	-0.041	-0.065	-0.077	-0.087	-0.092	-0.087

Summary

Dead	-0.957	-0.733	-0.536	-0434	-0.291	-0.518	-0.628	-1.239	-1.361	-2.098
panel	+1.756	1.293	1.769	1.286	1.841	1.480	2.224	1.922	2.467	1.957
Even	-0.938	-0.704	-0.534	-0.366	-0.376	-0.354	-0.805	-0.946	-1.665	-1.690
panel	1.296	1.764	1.281	1.793	1.357	1.993	1.733	2.394	1.999	2.371

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CALCULATIONS FOR

Design of Chorokubashi for Kumamoto-Ken

Verticals -

Load on member	1	2	3	4	5	6	7	8	9	10
V1	-0.043	-0.079	-0.114	-0145	-0.167	-0.178	-0.184	-0.175	-0.143	-0.091
2	-0.034	-0.061	-0.088	-0.110	-0.123	-0.127	-0.125	-0.109	-0.071	-0.013
3	-0.021	-0.641	-0.057	-0.070	-0074	-0069	-0.060	-0.036	0.009	0.072
4	-0.013	-0.021	-0.027	-0030	-0.026	-0.012	0.006	0.036	0.085	0.152
5	-0.002	+0.003	0.007	0.014	0.024	0.051	0.076	0.112	0.166	0.235
6	0.011	0.026	0.042	0.061	0.085	0.114	0.147	0.189	0.254	0.312
7	0.024	0.051	0.078	0.107	0.140	0.176	0.216	0.261	0.316	0.378
8	0.036	0.072	0.110	0.148	0.188	0.230	0.273	0.319	0368	0.422
9	0.045	0.091	0137	0.182	0.227	0.270	0.314	0.356	0404	0.439
9'	-0.056	-0.111	-0.167	-0.224	-0.280	-0338	-0.395	-0454	0479	0.439
8	-0062	-0.123	-0183	-0243	-0301	-0356	-0411	0537	0479	0.422
7	-0.063	-0124	-0.184	-0.243	-0.297	-0348	0.602	0.519	0.444	0.378
6	-0.063	-0.121	-0.179	-0.234	-0.284	0.672	0565	0.467	0.394	0.312
5	-0.059	-0112	-0165	-0.215	0.742	0621	0.504	0.398	0308	0.235
4	-0.053	-0099	-0.147	0.810	0.674	0.548	0.426	0.316	0225	0.152
3	-0.051	-0.095	0.879	0.732	0.595	0267	0342	0.232	0.143	0.072
2	-0039	+0.929	0.778	0.632	0490	0369	0247	6139	0053	-0.013
1	+0.968	0.821	0.672	0.529	0.395	0272	0.154	0.049	-0.031	-0.091

Influence Surface of Verticals -

Load on / member	V1	V2	V3	V4	V5	V6	V7	V8	V9
1	-0.043	-0034	-0.021	-0.013	-0.002	0.011	0024	0.036	0045
2	-0.079	-0061	-0041	-0.021	0.003	0.026	0.051	0072	0091
3	-0.114	-0.088	-0.057	-0.027	0007	0042	0.078	0.110	0.137
4	-0.145	-0110	-0.070	-0.030	0014	0.061	0.107	0.148	0.182
5	-0.167	-0.123	-0074	-0026	0.024	0.085	0.140	0.188	0.227
6	-0.178	-0127	-0069	-0012	0.051	0.114	0176	0.230	0.270
7	-0184	-0125	-0.060	+0.006	0076	0.147	0216	0.273	0.314
8	-0175	-0.109	-0.036	0036	0112	0.189	0261	0.319	0.356
9	-0143	-0071	0009	0085	0.166	0254	0316	0368	0404
10	-0091	-0013	0072	0152	0235	0312	0378	0422	0439
9'	-0031	0053	0143	0225	0308	0394	0444	0478	0479
8'	0049	0139	0232	0316	0398	0467	0519	0537	-0454
7'	0154	0247	0342	0426	0504	0565	0602	-0411	-0395
6'	0272	0369	0267	0548	0621	0672	-0348	-0356	-0338
5'	0395	0490	0595	0674	0742	-0284	-0297	-0301	-0280
4'	0529	0632	0732	0810	-0215	-0234	-0243	-0243	-0224
3'	0672	0778	0879	-0147	-0165	-0179	-0184	-0183	-0167
2'	0821	0929	-0095	-0099	-0112	-0121	-0124	-0123	-0111
1'	0968	-0039	-0051	-0053	-0059	-0063	-0063	-0062	-0056

Summary

Odd panel	-0.682	-0.480	-0263	-0266	-0220	-0.526	-0544	-0957	-0898
	2.189	1.568	1.968	1.416	1.827	1.498	1.820	1.453	1.606
Even panel	-0.668	-0420	-0311	-0162	-0327	-0355	-0715	-0772	-1127
	1.671	2.069	1.303	1.862	1.434	1.841	1.492	1.728	1.338

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CALCULATIONS FOR

Design of Chorokubashi for Kumamoto-Ken

Dead Load Stress in tie

			Panel conc.		
main panel	6.735	×	165100	=	1.111.900
Intermediate Panel	6.832	×	17300	=	118200
					1230100 *

Live Load Stress in tie

sidewalk loading	6.735	·	39700	=	267000
motor trucks and Unif.	6.735	·	21000	=	141400
Extra conc	1.085	·	13000	=	14100
Electric car					269800
					692300
					1922400 #

Stress in Hangers.

Dead Load Stress

From floor beam	99000
" cantilever	20100
Tie and Hanger say	16100
	135200 #

Live Load Stress.

From floor beam say	102000
From cantilever	25200
	127200
	262400 #

Stresses in Upper chord

Dead Load stress

member	comp.	Ten.	main Panel summary	comp	Ten	intermediate summary	max ord	Dl. main panel 165100	Dl. Int. 17300	Total
U_0-U_1	0.734	0.424	0.310	0.992	0.411	0.581	0.386	51200	10100	61300 C
U_1-U_2	1.837	0.769	1.068	1.791	0.786	1.005	0.908	176300	17400	193700
U_2-U_3	2.677	1.015	1.662	2.936	1.014	1.922	1.056	274300	33300	307600
U_3-U_4	3.880	1.096	2.784	3.780	1.155	2.625	1.686	459.400	45400	504800
U_4-U_5	4.704	1.057	3.647	4.996	1.092	3.904	1.564	682000	67500	669500
U_5-U_6	5.842	-0.859	4.983	5.920	0.884	5.036	2.243	822700	87100	909800
U_6-U_7	6.302	0.517	5.785	6.630	0.555	6.075	1.945	955100	105100	1060200
U_7-U_8	7.229	0.177	7.052	6.916	0.201	6.715	2.465	1164400	116200	1280600
U_8-U_9	7.309		7.309	7.626	0.018	7.608	1.945	1206800	131600	1338300
U_9-U_{10}	7.879		7.879	7.524	0.000	7.524	2.567	1300800	130200	1431000

Live load stresses.

member		motor truck conc. 13000	motor truck unif. 21000	Unif. load 39700	Electric car loading	Total LL.	Total stress	Rigid Area
U_0-U_1	0.734	5000	15400	29200	29400	79000	140300 # C	10.0 □"
U_1-U_2	1.837	11800	38600	72900	73500	196800	390500	28.0
U_2-U_3	2.677	13700	56200	106300	107100	283400	591000	42.2
U_3-U_4	3.880	21900	81500	154000	155200	412600	917400	65.5
U_4-U_5	4.704	20300	98800	186700	188200	494000	1163500	83.0
U_5-U_6	5.842	29200	122700	232000	233700	617600	1527400	109.0
U_6-U_7	6.302	25300	132300	232000	252100	659900	1720100	123.0
U_7-U_8	7.229	32000	151800	287000	289200	760000	2040600	145.6
U_8-U_9	7.309	25300	153500	290200	292500	761500	2099800	150.0
U_9-U_{10}	7.879	33400	165500	312800	315200	826900	2257900	161.2

Notes: For panel concentration of live load see page no 66.
Stresses due to Electric car loading figured as shown on preliminary design on page no 14 and 15 and max stress is nearly equivalent to load of 40.000 # concentration at panel point. The said stresses hereafter figured by concentration of 40.000 # at panel point.

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CALCULATIONS FOR

Design of Chorotsubashi for Kumamoto-Ken.

Lower chord stresses

Member	main panel Comp.	Ten	Sum	2nd. panel Comp	Ten	Sum	Max ord.	Panel conc. 165100	Panel conc 17300	Total Dead Load
L_0-L_1	7.758	0.000	7.758	7.870	0.000	7.870	1.250	1280.800	136200	1417000 C
L_1-L_2	7.336	0.060	7.276	7.433	0.315	7.118	1.304	1201.300	123100	1324400 C
L_2-L_3	6.886	0.616	6.270	6.983	0.571	6.412	1.356	1035200	110900	1146100 C
L_3-L_4	6.495	1.023	5.472	6.513	1.203	5.310	1.389	903400	91900	995300 C
L_4-L_5	5.927	1.731	4.196	6.073	1.621	4.452	1.381	692800	77000	769800 C
L_5-L_6	5.362	2.073	3.289	5.373	2.276	3.097	1.318	543000	53600	596600 C
L_6-L_7	4.433	2.629	1.804	4.613	2.462	2.151	1.165 (1.348)	297800	37200	335000 C
L_7-L_8	3.552	2.572	0.980	3.597	2.809	0.788	1.005 (1.091)	161800	13600	175400 C
L_8-L_9	2.465	2.755	(0.290)	2.574	2.434	0.141	0.772 (1.423)	(47900) T	2400	45500 C
L_9-L_{10}	1.471	2.066	(0.595)	1.598	2.389	(0.791)	0.514 (0.910)	(98200) T	(13700) T	(111900) T

Member	motor trucks conc. 13000	motor trucks unif 21000	Unif. load 34700	Elec. car loading
L_0-L_1	16300 C	162900 C	308000 C	310300 C
L_1-L_2	17000 C	154100 C	291200 C	293400 C
L_2-L_3	17600 C	144600 C	273400 C	275400 C
L_3-L_4	18100 C	136400 C	257900 C	259800 C
L_4-L_5	18000 C	124500 C	235300 C	237100 C
L_5-L_6	17100 C	112600 C	212900 C	214500 C
L_6-L_7	15100 C 18200 T	93100 C 55200 T	176000 C 104400 T	177300 C 105200 T
L_7-L_8	13100 C 14100 T	74600 C 54000 T	141000 C 102100 T	142100 C 102900 T
L_8-L_9	10000 C 18500 T	51800 C 57900 T	97900 C 109400 T	98600 C 110200 T
L_9-L_{10}	6700 C 11800 T	30900 C 43400 T	58400 C 82000 T	58800 C 82600 T

Member	Total Live Load Stress	Total Stress	Adjusted Stress	Reqd section
L_0-L_1	797500 C	2214500 C	2214500 C	158.2
L_1-L_2	755700 C	2080100 C	2080100 C	148.6
L_2-L_3	711000 C	1857100 C	1857100 C	132.7
L_3-L_4	672200 C	1667500 C	1667500 C	119.0
L_4-L_5	614900 C	1384700 C	1384700 C	98.9
L_5-L_6	557100 C	1153700 C	1153700 C	82.4
L_6-L_7	461500 C 283000 T	796500 C	796500 C	56.9
L_7-L_8	370800 C 273100 T	546200 C 97700 T	595100 C 146600 T	42.5 (8.6)
L_8-L_9	258300 C 296000 T	303800 C 250500 T	429100 C 375800 T	30.7 (22.1)
L_9-L_{10}	154800 C 219800 T	42.900 C 331700 T	64400 C 353200 T	4.6 (20.8)

Diagonals.

Member	main panel Ten	comp	Sum	Int. panel Ten	comp	Sum	Max Ord	Dl. Panel conc 165100	Intermediate conc. 17300	Tot. D.L.
U_0-L_1	1.296	0.938	0.358	1.756	0.957	0.799 T	0.845 T	59.100	13800	72900 T
U_1-L_2	1.764	0.704	1.060	1.293	0.733	0.560 T	0.865	175.000	9700	184700 T
U_2-L_3	1.281	0.534	0.747	1.769	0.536	1.233 T	0.713	123.300	21300	144600 T
U_3-L_4	1.793	0.366	1.457	1.286	0.434	0.852 T	0.887	240600	14700	255300 T
U_4-L_5	1.357	0.376	0.981	1.841	0.291	1.550 T	0.724	162.000	26800	188800 T
U_5-L_6	1.993	0.354	1.639	1.480	0.518	0.962 T	0.901	270.600	16600	287200 T
U_6-L_7	1.733	0.805	0.928	2.224	0.628	1.596 T	0.730 T 0.378 C	153.200	27600	180800 T
U_7-L_8	2.394	0.946	1.448	1.922	1.239	0.683 T	0.878 T 0.453 C	239100	11800	250900 T
U_8-L_9	1.999	1.665	0.334	2.467	1.361	1.106 T	0.742 T 0.634 C	55100	19100	74200 T
U_9-L_{10}	2.371	1.690	0.681	1.957	2.098	0.141 C	0.817 T 0.665 C	112400	(2400) C	110000 T

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CALCULATIONS FOR

Design of Chorokubashi for Kumamoto-Ken

Diagonals

Members	motor trucks conc. 13000		motor trucks unif. uniform load Electric loading 100700#		Live Load Summary		Total Stress		Adjusted Stress	Section
U_0-L_1	11.000 T		130500 T		141500 T		2144400 T		214400 T	12.7°
U_1-L_2	11.200 T		177600 T		188800 T		373500 T		373500 T	22.0
U_2-L_3	9300 T		129000 T		138300 T		282900 T		282900 T	16.6
U_3-L_4	11500 T		180600 T		192100 T		447400 T		447400 T	26.3
U_4-L_5	9400 T		136500 T		145900 T		334700 T		334700 T	19.7
U_5-L_6	11700 T		200700 T		212400 T		499600 T		499600 T	29.4
U_6-L_7	9500 T	4900 C	174500 T	81100 C	184000 T		364800 T		364800 T	21.5
U_7-L_8	11400 T	5900 C	241100 T	95300 C	252500 T		503400 T		503400 T	29.6
U_8-L_9	9600 T	8200 C	201300 T	167700 C	210900 T	175900 C	285100 T	101700 C	336000 T	19.8
U_9-L_{10}	10600 T	8600 C	238800 T	170200 C	249400 T	178800 C	359400 T	68800 C	393800 T	23.2

Verticals

Members	main panel Comp	Ten	Sum	Intermediate panel Comp	Ten	Sum	max ord		Dl. main p. 165100	Dl. Int panel 17300	Dl. Total	
U_1-L_1	1.671	0.668	1.003	2.189	0.682	1.507	0.821	(0.178)	165600	26100	191700	C
U_2-L_2	2.069	0.420	1.649	1.568	0.480	1.088	0.929		272200	18800	291000	C
U_3-L_3	1.303	0.311	0.992	1.968	0.263	1.705	0.732		163800	29500	193300	C
U_4-L_4	1.862	0.162	1.700	1.416	0.266	1.150	0.810		280700	19900	300600	C
U_5-L_5	1.434	0.327	1.107	1.827	0.226	1.601	0.621		182800	27700	210500	C
U_6-L_6	1.841	0.355	1.486	1.498	0.526	0.972	0.672		245000	16800	262100	C
U_7-L_7	1.492	0.715	0.777	1.820	0.544	1.276	0.519	(0.348)	128300	22100	150400	C
U_8-L_8	1.728	0.722	1.006	1.453	0.957	0.496	0.537	(0.356)	166100	8600	174700	C
U_9-L_9	1.838	1.127	1.211	1.606	0.898	0.708	0.439	(0.454)	34800	12200	47000	C

Members	motor trucks conc 13000		motor truck unif. unif. + Elec. loading 100700		Live Load Summary		Total Stress		Adjusted Stress		Section	
U_1-L_1	10.700 C		168.300 C		179.000 C		370700 C		370700 C		35.6	
U_2-L_2	12100 C		208300 C		220400 C		511400 C		511400 C		43.0	
U_3-L_3	9500 C		131200 C		140700 C		334000 C		334000 C		25.1	
U_4-L_4	10500 C		187500 C		198000 C		498600 C		498600 C		35.6	
U_5-L_5	8100 C		144400 C		152500 C		363000 C		363000 C		25.9	
U_6-L_6	8700 C		185400 C		194100 C		456200 C		456200 C		32.6	
U_7-L_7	6700 C	4500 T	150200 C	172000 T	156900 C		307300 C		307300 C		22.0	
U_8-L_8	7000 C	4600 T	174000 C	72700 T	181000 C		355700 C		355700 C		25.4	
U_9-L_9	5000 C	5900 T	134700 C	113500 T.	140400 C	119400 T	187400 C	72400 T	223600 C	148600 T	16.0	(8.7)

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

Design of Chorokubashi for Kumamoto-Ken

Upper chord sections

member	stress	section req'd	Used section				weight
U_0-U_1	140.300 C	10.0 □"	63.58 □"	@ 3.4#	× 12.5	=	2700#
U_1-U_2	390.500 C	28.0	63.58		× 12.4	=	2680#
U_2-U_3	591 000 C	42.2	69.14		× 12.3	=	2890
U_3-U_4	917 400 C	65.5	69.14		× 12.2	=	2870
U_4-U_5	1163 500 C	83.0	84.13		× 12.1	=	3460
U_5-U_6	1527 400 C	109.0	108.32		× 12.1	=	4460
U_6-U_7	1720 100 C	123.0	127.07		× 12.05	=	5210
U_7-U_8	2040 600 C	145.6	147.70		× 12.05	=	6060
U_8-U_9	2099 800 C	150.0	156.77		× 12.0	=	6360
U_9-U_{10}	2257 900 C	161.2	163.02		× 12.0	=	6650
							43340#

U_0-U_1 and U_1-U_2

1 cov Pl	33 × 3/8	=	12.38
2 web plo	27 × 5/8	=	33.76
4 Ls	6×6×3/8	=	17.44
			63.58 □"

U_2-U_3 and U_3-U_4

1 cov Pl	33 × 3/8	=	12.38
2 web plo	27 × 5/8	=	33.76
4 Ls	6×6×1/2	=	23.00
			69.14 □"

U_4-U_5

1 cov Pl	33 × 5/8	=	20.63
2 web plo	27 × 3/4	=	40.50
4 Ls	6×6×1/2"	=	23.00
			84.13 □"

U_5-U_6

1 cov Pl	33 × 5/8	=	20.63
2 web plo	27 × 3/4"	=	40.50
4 Ls	6×6×5/8	=	28.44
2 Plo	15 × 5/8	=	18.75
			108.32 □"

U_6-U_7

1 cov Pl	33 × 5/8"	=	20.63
2 webs	27 × 3/4"	=	40.50
4 Ls	6×6×5/8"	=	28.44
2 Plo	15 × 5/8"	=	18.75
2 Plo	25 × 3/8"	=	18.75
			127.07 □"

U_7-U_8

2 cov Plo	33 × 5/8"	=	41.26
2 webs	27 × 3/4"	=	40.50
4 Ls	6×6×5/8"	=	28.44
2 Pls	15 × 5/8"	=	18.75
2 Plo	25 × 3/8"	=	18.75
			147.70 □"

U_8-U_9

2 cov Plo	33 × 5/8	=	41.26
2 webs	27 × 3/4	=	40.50
4 Ls	6×6×3/4	=	33.76
2 Pls	15 × 3/4	=	22.50
2 Pls.	25 × 3/8	=	18.75
			156.77 □"

U_9-U_{10}

2 cov Plo	33 × 5/8	=	41.26
2 webs	27 × 3/4	=	40.50
4 Ls	6×6×3/4	=	33.76
2 Plo	15 × 3/4	=	22.50
2 Plo	25 × 1/2"	=	25.00
			163.02 □"

Lower Chord sections

member	stress	section req'd	Used section				weight
L_0-L_1	2214 500 C	158.2 □"	162.26 □"	@ 3.4#	× 14.0	=	7710
L_1-L_2	2080 100 C	148.6	152.98		× 13.5	=	7020
L_2-L_3	1857 100 C	132.7	134.23		× 13.2	=	6020
L_3-L_4	1667 500 C	119.0	121.83		× 12.85	=	5320
L_4-L_5	1384 000 C	98.9	96.83		× 12.60	=	4150
L_5-L_6	1153 700 C	82.4	85.58		× 12.40	=	3600
L_6-L_7	796 500 C	56.9	57.20		× 12.30	=	2390
L_7-L_8	595 100 C	42.5	51.20		× 12.10	=	2110
L_8-L_9	429 100 C	30.7	51.20		× 12.04	=	2090
L_9-L_{10}	353 200 T	20.8 □" net	51.20		× 12.00	=	2080
							42490#

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CALCULATIONS FOR

Design of chorokubashi for Kumamoto-ken

Lower chord

L9-L10, L8-L9, L7-L8

2 web plo	27 × 5/8	=	33.76
4 Ls	6×6×3/8	=	17.44
			51.20

L6-L7

2 web plo	27 × 5/8	=	33.76
4 Ls	6×6×5/8	=	23.44
			57.20

L6-L5

2 web plo	27 × 3/4	=	40.50
4 Ls	6×6×3/4	=	33.76
2 Plo	15 × 3/8	=	11.25
			85.58

L4-L5

2 web plo	27 × 3/4	=	40.50
4 Ls	6×6×3/4	=	33.76
4 Pls	15 × 3/8	=	22.50
			96.83°

L3-L4

2 web plo	27 × 3/4	=	40.50
4 Ls	6×6×3/4	=	33.76
2 Pls.	15 × 3/4	=	22.50
2 Plo	25 × 1/2	=	25.00
			121.83°

L2-L3

2 web plo.	27 × 3/4	=	40.50
4 Ls	6×6×3/4	=	33.76
2 Pls.	15 × 3/4	=	22.50
2 Pls.	25 × 3/4	=	37.50
			134.23°

L1-L2

2 webs	27 × 3/4	=	40.50
4 Ls	6×6×3/4	=	33.76
2 Pls	15 × 3/4	=	22.50
2 Pls	25 × 3/4	=	37.50
2 Pls.	25 × 3/8	=	18.75
			152.98°

L0-L1

4 webs	27 × 3/4	=	81.00
4 Ls	6×6×3/4	=	33.76
2 Pls.	15 × 3/4	=	22.50
2 Pls	25 × 1/2	=	25.00
			162.26°

Section of Tie -

Total stress = 1922400# section reqd = 113.40° net

6 webs	24 × 3/4	=	108.00 gr.	90.00
2 side plo	18 × 1/2	=	18.00	15.00
2 Ls	6×6×1/2"	=	11.50	9.50
			137.50	114.50° net

weight of tie = 137.5 @ 34# × 120 = 56000 # for 1/2 span

Diagonals.

member	stress	Section Reqd	Used Section			
L0-L1	214400# T	12.70° net	16.72 gr	@34 .	23.0	= 1310#
L1-L2	373500 T	22.0	26.94		20.6	= 1885
L2-L3	282900 T	16.6	21.24		18.75	= 1355
L3-L4	447400 T	26.3	33.19		17.30	= 1960
L4-L5	334700 T	19.7	26.94		16.30	= 1490
L5-L6	499600 T	29.4	37.75		15.60	= 2000
L6-L7	364800 T	21.5	26.94		15.10	= 1390
L7-L8	503400 T	29.6	37.75		14.85	= 1905
L8-L9	336000 T	19.8	23.44		14.75	= 1170
L9-L10	398000 T	23.2	29.22		14.85	= 1475
						15940#

L0-L1

4 Ls 6×4×7/16 = 16.72° gr. 13.22° net

L1-L2 — L4-L5 L6-L7

2 Pls.	12 1/2 × 1/2	=	12.50 gr	10.50° net
4 Ls	6×4×3/8	=	14.44	11.44
			26.94°	21.94°

L2-L3

4 Ls 6×4×9/16 = 21.24° gr. 16.74° net

L3-L4

			gr	net
2 Pls	12 1/2 × 3/4	=	18.75°	15.75°
4 Ls	6×4×3/8	=	14.44	11.44
			33.19	27.19

L5-L6 L7-L8

2 Plo	12 1/2 × 3/4	=	18.75	15.75°
4 Ls	6×4×1/2	=	19.00	15.00
			37.75	30.75° net

L8-L9

4 Ls 6×4×5/8 = 23.44 18.44° net

L9-L10

2 Plo.	12 1/2 × 1/2	=	12.50	10.50
4 Ls	6×4×7/16	=	16.72	13.22
			29.22 gr	23.72° net

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CALCULATIONS FOR

Design of Chorokubashi for Kumamoto-Ken

Verticals

members	Stress	Section Req'd	Section used
U_1-L_1	370700 C	35.60"	37.75
U_2-L_2	511400	43.0	46.51
U_3-L_3	334000	25.1	27.76
U_4-L_4	498600	35.6	37.75
U_5-L_5	363000	25.9	27.76
U_6-L_6	456200	32.6	33.19
U_7-L_7	307300	22.0	23.44
U_8-L_8	355700	25.4	27.76
U_9-L_9	223600	16.0	19.00

U_1-L_1 - U_4-L_4

2 Plo. $12\frac{1}{2} \times \frac{3}{4}$ = 18.75
4 Ls $6 \times 4 \times \frac{1}{2}$ = 19.00
37.75"

U_2-L_2

2 Plo $12\frac{1}{2} \times \frac{3}{4}$ = 18.75
4 Ls $6 \times 4 \times \frac{3}{4}$ = 27.76
46.51

U_3-L_3 U_5-L_5 - U_8-L_8

4 Ls $6 \times 4 \times \frac{3}{4}$ = 27.76

U_6-L_6

2 Plo $12\frac{1}{2} \times \frac{3}{4}$ = 18.75
4 Ls $6 \times 4 \times \frac{3}{8}$ = 14.44
33.19"

U_7-L_7

4 Ls $6 \times 4 \times \frac{5}{8}$ = 23.44"

U_9-L_9

4 Ls $6 \times 4 \times \frac{1}{2}$" = 19.00" gr.

U_{10}-L_{10}

4 Ls $6 \times 4 \times \frac{3}{8}$ = 14.44" gr.

U_0-L_0

2 Plo. $18 \times \frac{1}{2}$ = 18.00
4 Ls $4 \times 4 \times \frac{1}{2}$ = 15.00
1 Web. $16\frac{1}{2} \times \frac{3}{8}$ = 6.19
4 Ls $4 \times 4 \times \frac{3}{8}$ = 11.44
50.63"

weight of $\frac{1}{2}$ $U_{10}L_{10}$. 7.22 @ 3.4 × 9.0 = 220
U_0-L_0 50.63 @ 3.4 × 270 = 4650
4870

@ 3.4	×	23.10	=	2970	
	×	19.80	=	3130	
	×	17.10	=	1615	
	×	14.80	=	1900	
	×	13.00	=	1230	
	×	11.50	=	1300	
	×	10.40	=	830	
	×	9.60	=	910	
	×	9.10	=	590	

End post & center verticals 14445
4870
19315

Hangers

Total stress 262400# ÷ 17000 = 15.45" net
4 Ls $6 \times 4 \times \frac{5}{8}$ = 23.00 or 18.00" net

262400 ÷ 18.0 = 14600 #/"

Weight of Hangers 2300 × 3.4# × 120.8 = 9400 #

Summary for main section of truss

Upper chord	43340
Lower chord	42490
Tie	56000
Diagonals	15940
verticals	19315
Hangers	9400
	186485 #
Details say 40%	74600
	261085 call this 261000

For one span 4 × 261000 = 1044000# or 465 tons

1044400 ÷ 240 = 4350# per ft of span
or 2175# " " of truss

Weight of Truss 465
cast iron post say 25
490 tons

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CALCULATIONS FOR

Design of Chorokubashi for Kumamoto-Ken

Estimate of metal in bridge

Stringers	1040# × 244.5'	=	254300	
Int. Floor beams say	9@22000	=	198000	
End Floor beams "	2@ 5000	=	10000	
Cantilever brackets	22@ 900	=	19800	
				482100
Lower Lateral system				28000
Upper Lateral system say				93000
weight of trusses complete				1044000
Ornamental Cast iron posts say				56000
Cast iron shoes say				56000
				1759100# or 785 tons

Load on bearing for shoe.

Dead Load metal		440000
" " floor	4060 × 122.5 =	497000
		937000

Live Load

Uniform load	1655 × 122.25	=	202500
	893 × 122.25	=	109500

Electric car
Continuous 30 ton car uniformly loaded with impact

	1614 × 122.25 =	198000
		523000
		1510000#

Load will be carried by shoes for end floor beam ; Let us design shoes for truss for load of 1,500,000#

Approximate load on bearing for shoe of End floor beam

Dead load floor say	46000
" " metal say	10000
	56000
Live load Electric by loading say	40.000
Uniform load say	10.000
	50.000
Total	106000#

Bearing area required 106000 ÷ 640 = 166 P"
Make base 18"×18" = 324 P"

Shoes for trusses
Total load 1500.000# Bearing area required = 1500.000 ÷ 640 = 2340 □"
Try 6½" rollers 6.5 × 610 = 3970# per lin. inch of roller
1500.000 ÷ 3970 = 378" net Use 9 Rollers @ 42" = 378" net
make length of Rollers 44" over all.
Size of base 45 × 56 = 2520 □"
Unit bearing 1500.000 ÷ 2520 = 596#/□" OK

Bearing for fixed shoes made same as for roller shoes.

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