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JIJI BLDG, TOKYO

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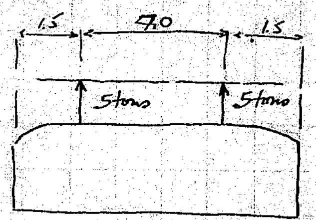
(上記田辺む)

CALCULATIONS FOR

Katase-Enoshima Suspended Elec. Ry. Line

Data:



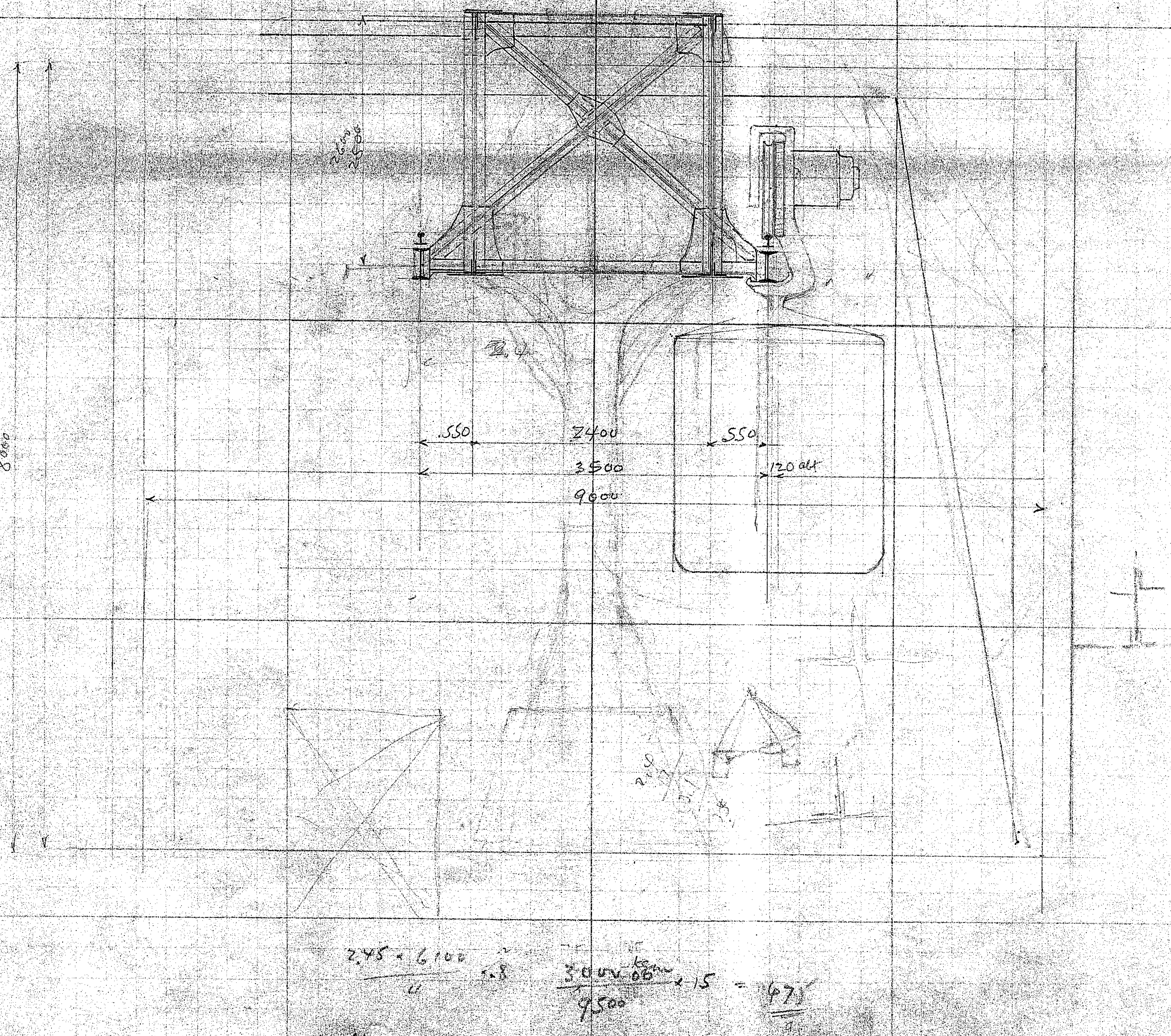
Dead weight of Car = 8.00
Live Load = 2.00
10.00 tons ÷ 2 = 5000 kg. on one Suspender.

Impact allowance $I = S \cdot \frac{45}{45+l} \times \frac{15}{60}$

max. speed of car assumed 15 miles per hr.
I = impact allowance.
S = Stress of the member.
l = loaded length in meter = say ~~5.5~~ 7.0 m

$I = S \cdot \frac{45}{45+7} \times \frac{15}{60} = 0.216\ S$

Concentration of Electric car 5000
21.6% impact = 1080
6,080 call this 6,100 kg for 1 suspender.



$\frac{2.45 \times 6100}{4}$ 6.8 $\frac{3000000\ kgcm}{9500} \times 15 = 473$

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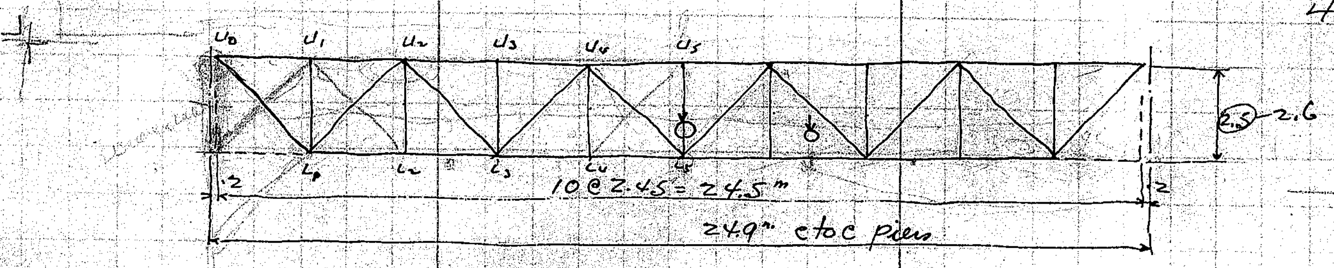
CALCULATIONS FOR

Katase – Enoshima Suspended Elec. Ry Line

Design of Truss. span length 24.5 meters   2.40m c to c of trusses   depth of truss 2.5 meters.

41 spans @ 24.9 = 1020.9m = 3,350 ft.



Dead Load assumed 400 kg/m for one truss.

Dead Load moment $= \frac{1}{8} \times 400 \times 24.5^2 = 30,000$ kgm

Dead Load Shear $= \frac{1}{2} \times 400 \times 24.5 = 4,900$ kg

Live Load. Electric car concentration with impact = 6100 kg for 1 suspender.

Live Load ~~moment~~ Reaction
6100 × 12.25 = 74700
6100 × 8.25 = 50300
125000 ÷ 24.5 = 5100

Live Load moment = 5100 × 12.25 = 62,500 kgm

Shear at end panel.
6100 × 22.05 = 134500
6100 × 18.05 = 110000
244500 ÷ 24.5 = 10,000 kg

Summary of Moments + Shears.

	moment	End Shear	Shear at 2nd panel	Shear at 3rd panel
Dead Load	30000	4900	2940	1960
Live Load	62500	10,000	8770	7550
	92500 kgm	14900 kg	11,710 kg	9510 kg

Chord stress $= \frac{92500}{2.5} = 37000$ kg T or C (35500, 41,000)

net area req'd $= \frac{37000}{1200} = 30.8$ cm² net. (35500; 29.6)

use 2Ls 125 × 90 × 10 = 41.0 cm² gr − 4 × 2.5 = 31.0 cm² net o/c.

least radius of gyration = 3.93 cm $l/r = \frac{245}{3.93} = 62$

allowable unit comp. $= 1200 - 5\frac{l}{r} = 1200 - 5 \times 62 = 890$ kg/cm²

gross area req'd $= \frac{37000}{890} = 41.5$ cm² gr. (35500; 39.9)   truss height = 2.6 = ~~2.5~~ ok

Diagonal stresses   length of diagonal member $= \sqrt{2.45^2 + 2.6^2} = 3.57$ (~~3.83~~)

Coeff $= \frac{3.57}{2.6} = 1.37$

			net section req'd.	gross area req'd
Stress of U0-L1	14900 × 1.37 =	20400 kg T	17.0 cm² net	
〃 〃 L1-U2	11710 × 〃 =	16030 〃 C	~~10.9~~ 〃	22.4 cm² gr
〃 〃 U2-L3	9510 × 〃 =	13030 〃 T	10.9 〃	

U0-L1 use same section as for chord.

L1-U2 + U2-L3 } use 2Ls 100 × 75 × 10 = 33.0 cm² gr − 4 × 2.5 × 1 = 23.0 cm² net

r = 3.1 cm   l = 3.57 − .57 = 300 cm assumed $l/r = \frac{300}{3.1} = 96.8$ < 100

allowable unit comp. = 1200 − 5 × 96.8 = 716 kg/cm² C

gross area req'd for L1-U2 $= \frac{16030}{716} = 22.4$ cm² gr o/c

for diagonals L3-U4 + U4-L5,

use 2Ls 90 × 75 × 9 = 28.08 cm² gr − 4 × 2.5 × 9 = 19.08 cm² net

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CALCULATIONS FOR

Katase-Enoshima Suspended Elec. Ry Line.

Approximate weight of main truss

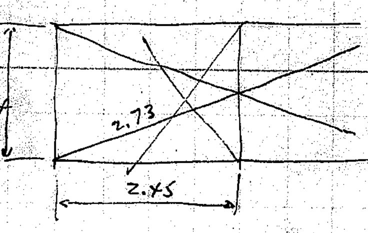
Chords. top	~~2~~Ls	125×90×10 (13)	@ 16.09 (20.61)	× 24.8	=	798	1020
〃 bott.	2Ls	〃 (13)	@ 〃	× 19.6	=	631	807
Diagonals U_0-L_1	4Ls	125×90×10 (13)	@ 〃	× 3.6	=	232	297
〃 (L_1-U_2, U_2-L_3)	8Ls	100×75×10	@ 12.95	× 3.4	=	352	
〃 (L_3-U_4, U_4-L_5)	8Ls	90×75×9	@ 11.02	× 3.4	=	300	
verticals	18Ls	80×80×9	@ 10.66	× 2.7	=	518	
						2831	3294
details including rivet heads. say 35%					=	999	1146
						3830	4440
Shoes etc say						170	
						4,000 kg for one truss.	

Lower Lateral Bracings.

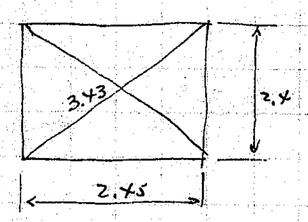
Exposed area assumed 0.6 $^{\square m}$ per lin meter of span on each side or 1.2 $^{\square m}$ in total.
wind load = 1.2 × 300 = 360 kg per lin m. of span
End shear = 360 × 24.9 ÷ 2 = 4480 kg for no live load.
when the car in span
wind load on truss = 1.2 × 200 = 240 kg per lin meter of span
End shear = 240 × 24.9 ÷ 2 = 2,990 kg.
wind load on car
exposed area = 7.0 × 2.5 = 17.5 $^{\square m}$ × 2 = 35.0 $^{\square m}$ for 2 cars
End shear = 35 × 200 = 7000 kg

$7000 \times \frac{22.4}{24.9} = 6300$ kg
2990
9290 kg

~~diagonal length = $\sqrt{2.4^2+2.45^2} = 3.43^m$~~ $\sqrt{2.45^2+1.2^2} = 2.73$
Coeff = ~~3.43 ÷ 2.4 = 1.43~~ 2.73 ÷ 1.2 = 2.27
diagonal stress at end panel = 9290 × ~~1.43~~ 2.27 = ~~13280~~ kg = 21,100
Stress for 1 member = ~~13280~~ 21,100 ÷ 2 = ~~6640~~ kg 10550 kg T or C



Diagonal length = $\sqrt{2.4^2+2.45^2} = 3.43^m$ Coeff = 3.43 ÷ 2.4 = 1.43
use 1L ~~75×75×9~~ 90×90×10 = ~~12.69~~ 17.00 cm² − ~~4.5~~ 5.0 = ~~8.19~~ 12.00 cm² net.
diagonal stress = 9290 × 1.43 = 13280 ±
net area reqd = $\frac{13280}{1200 \times 1.25}$ = ~~11.07~~ 8.90 cm² net o/c.



$l/r = \frac{343}{2.71} = 127 < 200$ o/c.

for end 2 panels use 1L 90×90×10
〃 intermediate panels. 1L 80×80×9.

approx. weight of lower lateral Bracing

diagonals.	8Ls	90×90×10	@ 13.34	× 3.0	=	320	
〃	12Ls	80×80×9	@ 10.66	× 3.0	=	384	
Struts.	18Ls	~~125~~ 100×90×10	@ ~~16.09~~ 14.13	× 2.35	=	~~688~~ 598	
Center conn.	10	@		1.0	=	10.	
End conn.	22	@ ~~12.15~~ 12			=	~~264~~ 330	264
Strut end conn.	18	@	10		=	180	
						~~1904~~	1756
Rivet heads + variations say						~~96~~	74
						~~2000~~ kg for one span.	1,830

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CALCULATIONS FOR

Katase-Enoshima Suspended Elec Ry Line

Approximate weight of upper lateral Bracing.

Diagonals	20 Ls	~~80×80~~ 75×75 ×9	@ ~~10.66~~ 9.96	× 3.0	= ~~640~~ 598
Struts.	18 Ls	100×90×10	@ 13.34	× 2.4	= 576
〃 end.	8 Ls	~~125×90×10~~ 100×90×10	@ ~~16.09~~ 14.13	× 2.4	= ~~309~~ 272
〃	2 P/s	~~400~~ 300 ×9	@ ~~28.26~~ 25.4	× 2.4	= ~~136~~ 122
Conn.	22 ·		@ 15		= 330
					~~1991~~ 1898
	Rivet heads & variations			say	~~109~~ 102
					~~2100~~ 2000 kg for one span.

Sway Bracing and brackets.

Diagonals	4 Ls	~~80×80~~ 75×75 ×9	@ 10.66 9.96	× ~~3.5~~ 3.4	= ~~149~~ 135
Conn. top.	2		@ 15		= 30
〃 center	1		@ 17 11		= ~~17~~ 11
〃 bott.	2		@ 35		= 70
Arm	4 Ls	125×90×10	@ 16.09	× 0.4	= 26
Conn to I beam	8 Ls	100×90×10	@ 14.13	× 0.25	= 28
					320
	Rivet heads &c.			say	10
					~~330~~ 310 kg for one panel pt.

revised

2 Ls 80×80×9 @ 10.66 × 3.0 = 64.
26
5
100
26
28
246
Stiff Ls &c 70
310

I-Beams under rails.

300×15 @ 48.34 × 24.9 = 2405
2 Is ~~12×6 @ 44.0 or 65.6~~ × 24.9 = ~~3270~~
details say ~~230~~ 225
~~3500~~ 2630 kg for one span.

Approximate weight of structural steel for one span super structure

Main trusses.	2 @ ~~4000~~ 4440	= ~~8000~~ 8880	8.2
lower lateral Bracing	1 @ ~~2000~~ 1830	= ~~2000~~ 1830	1.5 3.6
upper 〃 〃	1 @ ~~2100~~ 2000	= ~~2100~~ 2000	1.5
Sway Bracing & bracket	11 @ ~~330~~ 310	= ~~3630~~ 3410	
Prolonging the lower chord & Br. &c.	2 @ 300	= 600	
I-beam under rails		= ~~3500~~ 2630	
misc.	say	~~170~~ +340	
		~~20000~~ ~~18600~~ kg.	
		or ~~20.0~~ kg tons for one span.	
		~~18.6~~	
		19.8	

End conn. of truss 16×16 = 256
4×15 60
316
15
331

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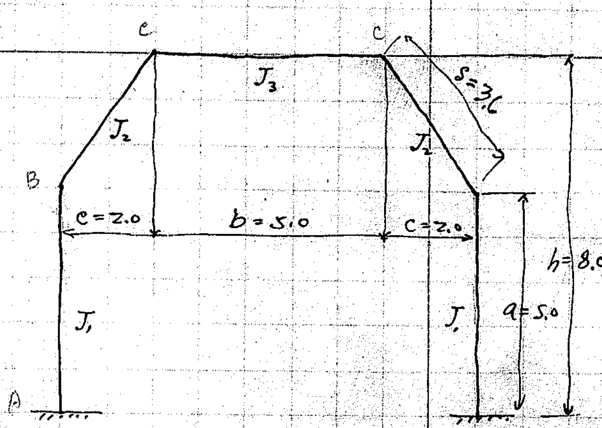
CALCULATIONS FOR

Katase-Enoshima Suspended Elec. Ry. Line.

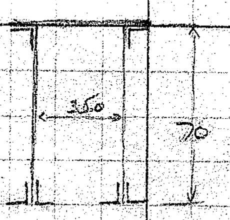
Design of pier bent.

max. load on pier say $6100 \times 2 = 12{,}200$ kg for one truss.
dead load. $400 \times 24.9 = 10{,}000$
Superimposed load = 22,200 kg for one truss.

Dead load on frame = 450 kg per lin. meter of frame.



Top Beam c-C



1 cov. pl. 600×12 =	72.00
2 Ls 100×100×10 =	38.00
4 Ls 100×100×13 =	97.20
2 pls 700×9 =	126.00
2 pls 490×9 =	88.20
	421.4 cm^2 gr.

approx. $r = 0.4 \times 70 = 28^{cm}$

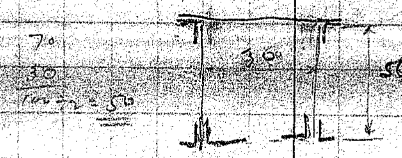
approx. $J_2 = J_3 = 421.4 \times 28^2 = 330{,}000$ cm^4

Column B-C

Same section as above

Column B-A

Average



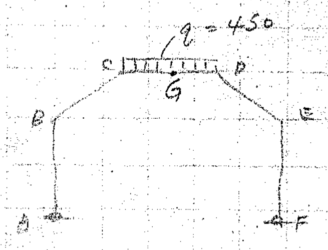
2 Ls 100×100×13	48.60
1 cov. pl. 550×12 =	66.00
4 Ls 100×100×13 =	97.20
2 web pls 500×9 =	45.00 / 256.8
	263.20 cm^2 gr.

approx. $r = 0.4 \times 50 = 20^{cm}$

$J_1 = 263.2 \times 20^2 = 105{,}400$ cm^4

$\frac{J_2}{J_1} = \frac{330{,}000}{105{,}400} = 3.13$

$\frac{J_2}{J_3} = \frac{330{,}000}{330{,}000} = 1.00$



$k_1 = \frac{J_2}{J_1} \cdot \frac{a}{S} = 3.13 \times \frac{5.0}{3.6} = 4.35$, $k_2 = \frac{J_2}{J_3} \cdot \frac{b}{S} = \frac{5.0}{3.6} = 1.39$

$V_A = V_F = \frac{qb}{2} = \frac{450 \times 5}{2} = 1125$ kg

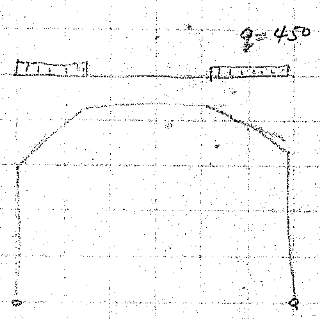
$H = \frac{qb}{4} \cdot \frac{2c(a+2h)+hk_2(l+4c)}{2a^2(k_1+1)+2ha+h^2(2+3k_2)} = \frac{450 \times 5}{4} \times \frac{4 \times 21 + 8 \times 1.39 \times 17}{50 \times 5.35 + 80 + 64 \times 6.17} = 207^{kg}$

(50 · 84 · 189 · 293 · 267.2 · 80 · 394.5 · 741.7)

$M_B = M_E = -Ha = -207 \times 5 = -1035$ kgm

$M_C = M_D = -Hh + \frac{qb}{2}c = -207 \times 8 + \frac{450 \times 5}{2} \times 2 = +595$ kgm

$M_G = \frac{qb^2}{8} + M_c = \frac{450 \times 5^2}{8} + 595 = +2000$ kgm



$V_A = V_F = qc = 450 \times 2 = 900$ kg

$H = \frac{qc^2}{4} \cdot \frac{6hk_2+5h+3a}{2a^2(k_1+1)+2ha+h^2(2+3k_2)} = \frac{450 \times 4}{4} \cdot \frac{48 \times 1.39 + 40 + 15}{741.7} = 74$ kg

(66.7 · 121.7)

$M_B = M_E = -Ha = -74 \times 5 = -370$ kgm

$M_C = M_D = -Hh + \frac{qc^2}{2} = -74 \times 8 + \frac{450 \times 4}{2} = +310$ kgm

$M_G = +310$ kgm.

Summary of DL m. due to wt. of frame

M_A	M_B	M_C	M_G	H	V
0	−1035	+595	+2000	207	1125
0	−370	+310	+310	74	900
0	−1405	+905	+2310	281	2025

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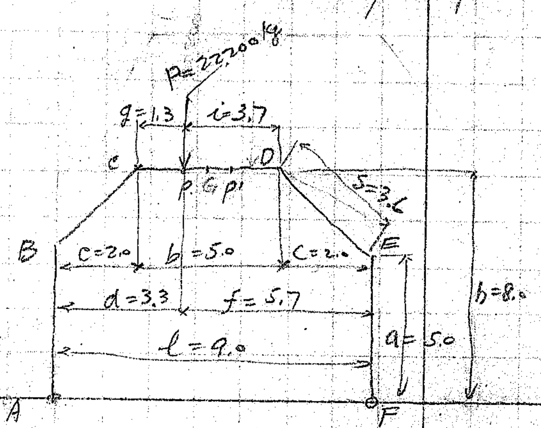
Katase – Enoshima Suspended Elec. Ry Line.

Moment due to superimposed Dead + Live load.



Stresses for one side load.

$V_A = \frac{Pf}{l} = \frac{22200 \times 5.7}{9} = 14070$ kg

$V_F = \frac{Pd}{l} = \frac{22200 \times 3.3}{9} = 8130$ kg

$H = \frac{Pc}{2} \cdot \frac{h(2+3k_2)+a}{2a^2(k_1+1)+2ha+h^2(2+3k_2)} = \frac{22200 \times 2}{2} \cdot \frac{8 \times 6.17+5}{50 \times 5.35+80+64 \times 6.17}$ (544 / 741.7)

$= 1630$ kg

$M_B = M_E = -Ha = -1630 \times 5 = -8150$ kgm

$M_D = -Hh + V_F c = -1630 \times 8 + 8130 \times 2 = +3230$ kgm

$M_C = -Hh + V_A c = -1630 \times 8 + 14070 \times 2 = +15100$ kgm

$M_P = -Hh + V_A d = -1630 \times 8 + 14070 \times 3.3 = +33400$ kgm

$M_{P'} = -Hh + V_F d = -1630 \times 8 + 8130 \times 3.3 = +13820$ kgm

Total stresses for 2 loads.

	M_A	M_B	M_C	M_G (= M_P)	H	V_A	V_F
due to load P	0	−8150	+15100	+33400	1630	14070	8130
〃 〃 P'	0	−8150	+3230	+13820	1630	8130	14070
		−16300	+18330	+47220	3260	22200	22200

Summary

wt. of frame	0	−1405	+905	+2310	281	2025	2025
Superimp. loads.	0	−16300	+18330	+47220	3260	22200	22200
	0	−17700 kgm	+19230 kgm	+49530 kgm	3540 kg	24220 kg	24220 kg

Approx. stresses on members.

Top beam C-D. $m = +49530$ kgm

unit stress $= \frac{49530 \times 100}{330000} \times 33 = 495$ kg/cm² C

〃 $= \frac{49530 \times 100}{330000} \times 38 = 570$ kg/cm² T

allowable unit comp. on flange $= 1100 - 15\frac{l}{b} = 1100 - 15 \times \frac{500}{60} = 975$ kg/cm² o/c

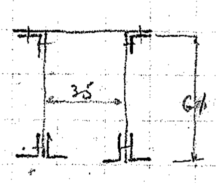
at B. approx.

unit stress $= \frac{17700 \times 100}{105400} \times 28 = 470$ kg/cm² C

allowable Comp. $= 1100 - 15 \times \frac{500}{50} = 950$ kg/cm² o/c

Sections.

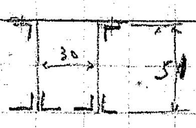
member C-D.



1 cov. pl.	600 × 12	=	72.0
2 Ls	100 × 100 × 10	=	38.0
4 Ls	100 × 100 × 13	=	97.20
2 pls	600 × 9	=	108.0
2 pls.	400 × 10	=	72.0
			387.20 cm² gr.

B C. same as for CD.

A B.



1 cov. pl.	55 × 10	=	55.0
2 Ls	100 × 100 × 10	=	38.0
4 Ls	100 × 100 × 10	=	76.0
2 pls.	500 × 9	=	90.0
			259.0 cm² gr.

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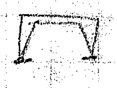
CALCULATIONS FOR

Katase-Enoshima Suspended Elec. Ry. Line.

Estimate of Structural steel for pier bent

Top beam C-D	387.2 @ .785 × 5.0	=	1,520
Column BC + DE	387.2 @ .785 × 3.6×2	=	2,188
" A-B + EF	259.0 @ .785 × 5.0×2	=	2032
			5,740
Splice say 2 meters.			608
" 2 @ 100		=	200
Shoes & anchor bolts 2 @ 330		=	660
Stiffeners diaphragm, tie pl., lacing etc say		=	800
Rivet head say			292
			8,300 kg for one pier

Summary of ~~Structural~~ steel

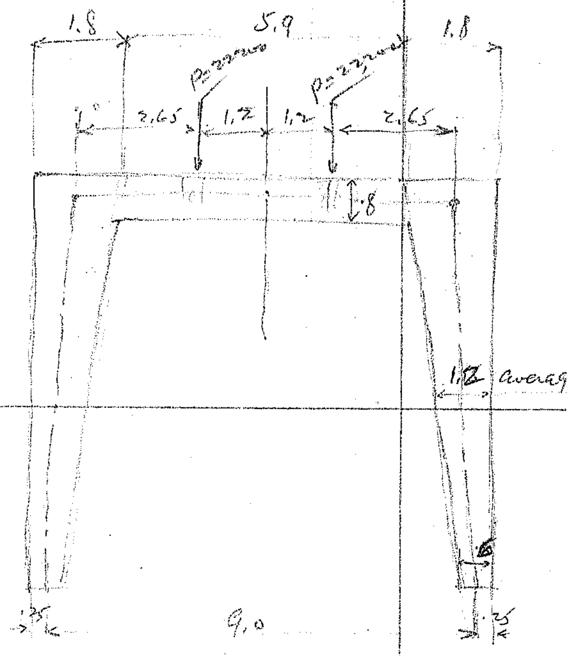
第二案 

第三案 

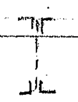
			第二案	第三案
Truss spans	41 @ ~~18.6~~ 19.4	= ~~762.0~~ 795.8 kg tons.	41 @ 20 = 820.	820
Pier	41 @ 8.3	= 340.8	41 @ 6.0 = 246	41@4.6 = 189
		~~1102.0~~ kg tons. 1,135.	1066 kg tons.	1009

69

12 ? 8

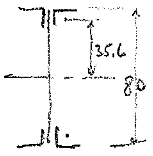


4Ls 150×150×19 = 106.8 − 19.0 = 87.8 cm² net.
1Pl 80×9

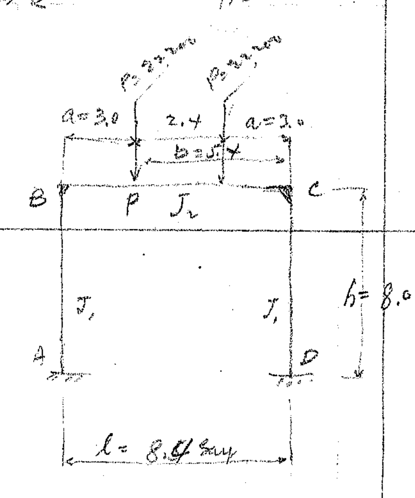


4Ls 150×150×15 = 85.5 − 15 = 70.5 cm² net

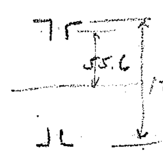
moment of inertia J_2, beam



$4 \times 1098 + 53.39 \times 4 \times 35.6^2 = 4400 + 271,000 = 275,400$
$\frac{.9 \times 79^3}{12}$ 37,000
312,000 cm³



moment of inertia J_1 col.



$4 \times 900 + 171 \times 55.6^2 = 3600 + 528,000 = 535,000$
$\frac{.9 \times 119^3}{12}$ 126,000
661,000

$$k = \frac{J_2}{J_1} \cdot \frac{h}{l} = \frac{312000}{661000} \times \frac{8.0}{8.4} = .45$$

$V = P = 22200$ kg.

$$H = \frac{3Pab}{hl(k+2)} = \frac{3 \times 22200 \times 3.0 \times 5.4}{8.0 \times 8.4 \times 2.45} = 6550 \text{ kg}$$

$M_A = M_D = + H \cdot \frac{h}{3} = 6550 \times \frac{8}{3} = +17450$ kgm.

$M_B = M_C = - H \cdot \frac{2h}{3} = \quad -34900$ kgm

$M_P = Pa - M_B = 22200 \times 3 - 34900 = +31,700$ kgm

DL beam. $4 \times 41.91 = 167.6$
56.5
$224.1 \times 1.35 = 300$ kg/m.

reaction $300 \times 4.2 = 1260 \times$
$1260 \times 3 = 3780$
$\frac{300 \times 3^2}{2} = -1350$
2430 kgm. Mp.

	M_A	M_B	M_P	V.
Summary of moments Superimp load	+17450	−34900	+31,700	22200
beam	say 1550	say 3900	2400	3800
	+19000	−38000	+34200	26000 kg

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CALCULATIONS FOR

12' 高 9.

flange stress $= \frac{38000}{0.7} = 54{,}300$ kg for beam

net area reqd for top beam $= \frac{54300}{1200} = 45.2$ cm² net

2Ls 150×150×19 $= 106.8 - 19 = 87.8$

$\frac{1}{8}$ web $= \frac{80 \times 9}{8} = \frac{9}{96.8}$ cm² net ok

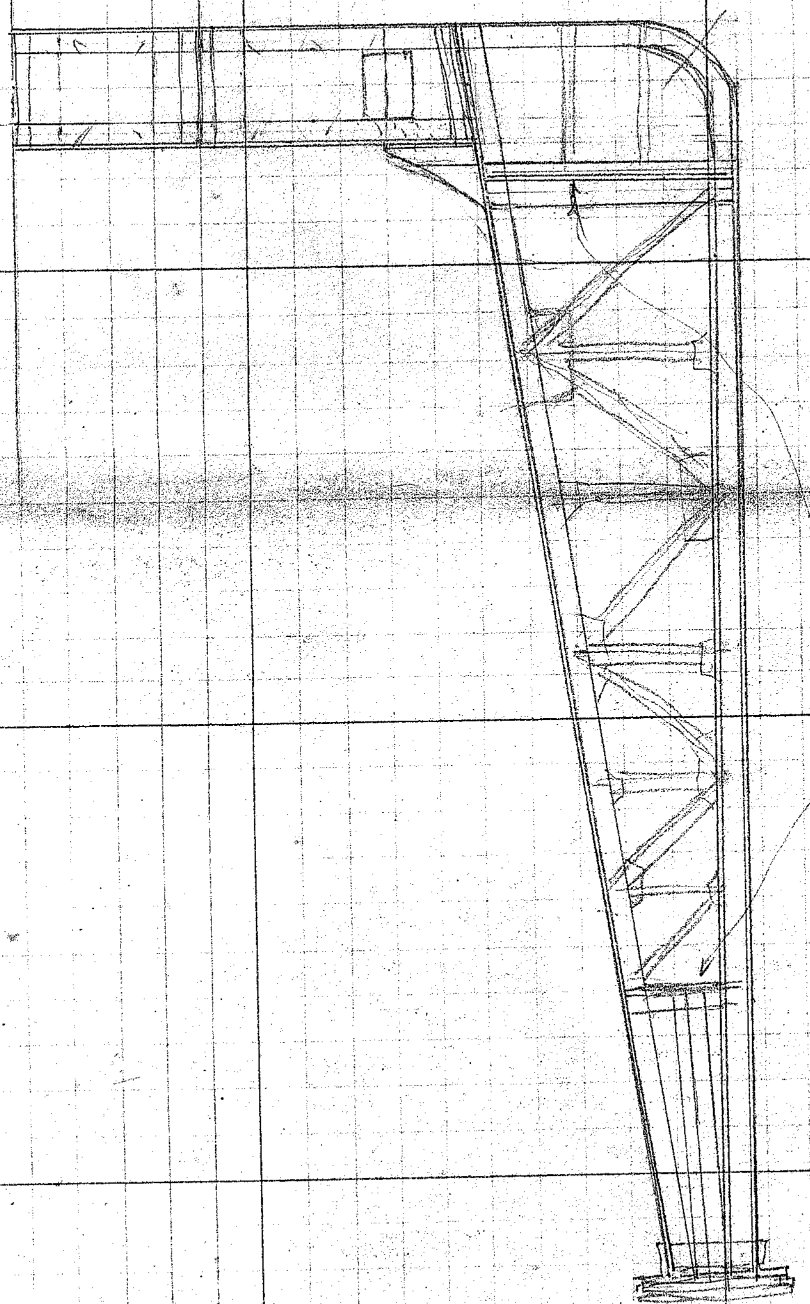
flange stress for column $= \frac{38000}{1.6} = 23{,}800$ kg

direct comp. on col. $= 26000 \div 2 = 13000$

36800 kg c

$1100 - 15\frac{l}{b} = 1100 - \frac{15 \times 800}{31} = 712$ (388)

Gross section reqd $= \frac{36800}{712} = 51.7$ cm² gr. — 2Ls 150×150×15 $= 85.5$ cm² gr. ok



beam	2Ls.	150×150×19	@ 41.91 × 4.5 =	377
〃	2Ls	〃	@ 〃 × 3.0 =	251
〃	1 Pl	800×9	@ 56.52 × 4.8 =	271
〃	10 Ls	125×75×10	@ 12.95 × .76 =	99
〃	5 Pls.	150×19	@ 22.37 × .5 =	56
〃	4 Pls.	300×9	@ 21.2 × .5 =	44
〃	4 Ls	130×130×12	@ 23.36 × .76 =	71
〃	4 Pls	125×19	@ 18.64 × .5 =	37
columns	8 Ls (stiff)	125×75×10	@ 12.95 × 1.0 =	104
〃	4 Ls	150×150×15	@ 33.55 × 8.4 =	1127
	2 Pls.	1200×9	@ 85. × 2.2 =	374
	2 Ls.	75×75×9	@ 9.96 × 11.0 =	219
	10 Pls	350×9	@ 24.73 × .6 =	148
	12 Pls	150×9	@ 10.6 × .35 =	45
	2 Pls	800×9	@ 56.52 × 2.0 =	226
	8 Ls	90×90×10	@ 13.34 × 2.0 =	215
E.T.	4 Ls	90×90×10	@ 13.34 × 2.5 =	133
Splice say		2 @ 630	=	1260
Shoes		2 @ 350	=	700
rivet heads etc				243
				6,000 kg.

260

19.8
0.2
20.0

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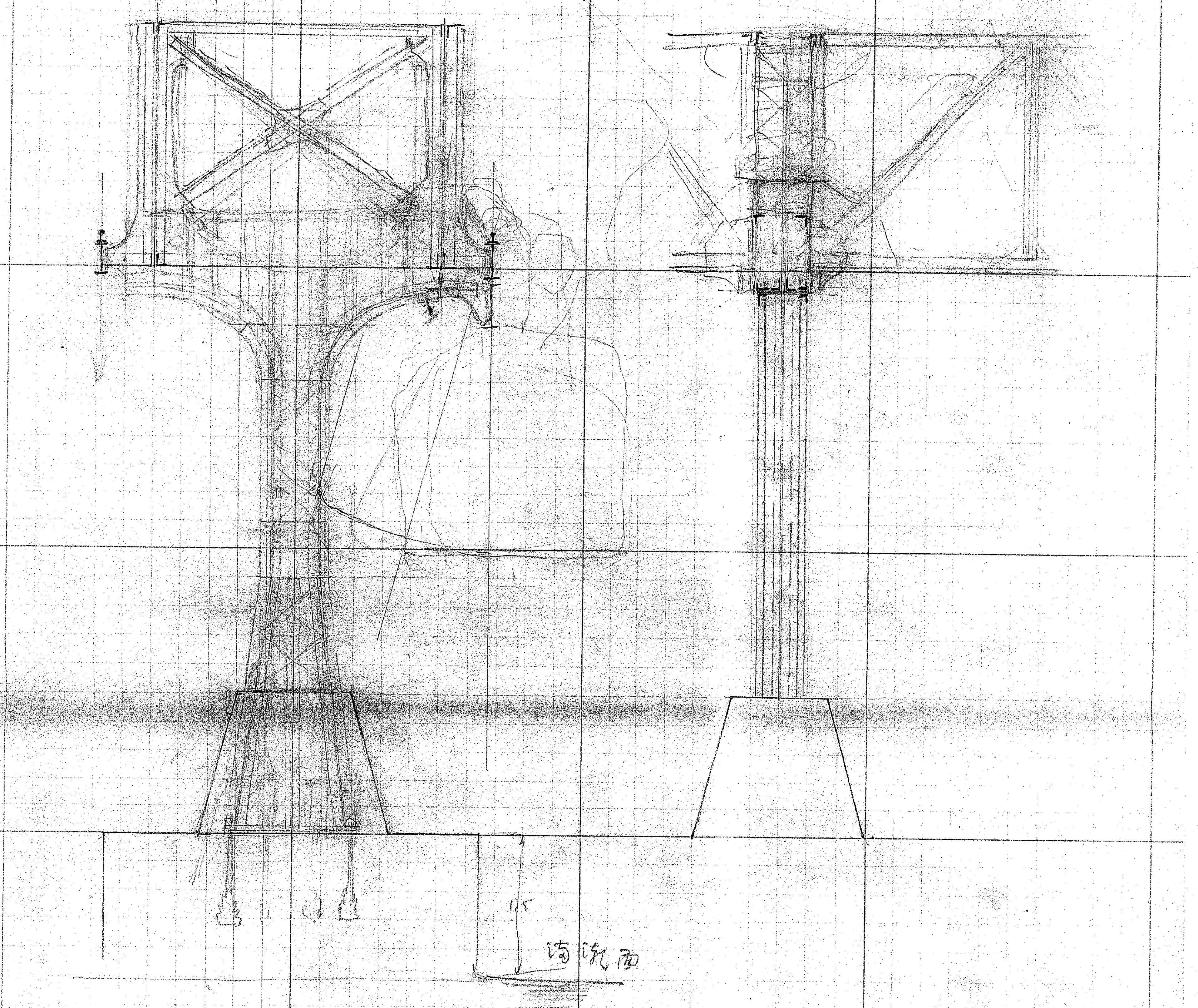
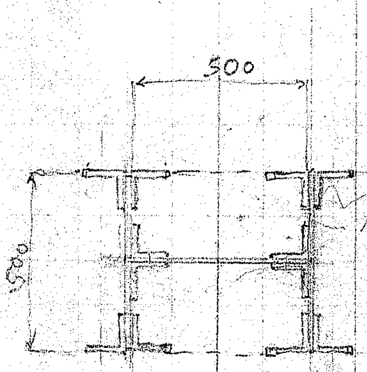
CALCULATIONS FOR

片瀬江ノ島懸垂電車路





8Ls 100×75×10 = 132.0
4Ls 90×90×10 = 68.0
2Pls 500×9 = 90.0
1Pl 500×10 = 50.0
340.0 cm² gr.

Superimposed DL + LL = 44000 kg
Pier say 6000
50000 kg

Gross area req'd = $\frac{50000}{900}$ = 55.5 × 2 = 111. sq

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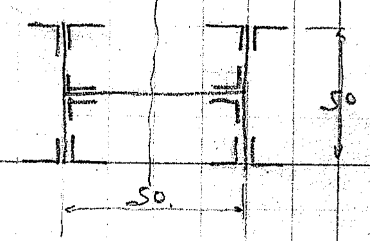
CALCULATIONS FOR

片瀬江ノ島懸空電車路

Wind load on one pier = 9300 kg see page 3.
wind load moment on column say 9300 × 4 = 37,200 kgm = ½ [illegible]
max. load on column = 50,000 kg

moment of inertia of col.



8Ls	100×75×10	160×8 +	132×25² =	83700
4Ls	90×90×10	125×4 +	88×20² =	27700
2ps.			90×25² =	56,200
1 Pl.		$\frac{10\times50^3}{12}$	=	104,200
				272,000 cm⁴

Bending stress = $\frac{37200 \times 100 \times 36}{272000}$ = 493.
direct stress = $\frac{50000}{340}$ = 147
640 kg/cm²

— ½ [illegible]

wind load. 2990 + 3150 = 6,140 kg

vert load on pier Car 2@6100 = 12,200
D.L. truss 20,000
32,200 kg
Pier say 5,800
38,000 kg

moment on column
wind moment 6140 × 4.0 = 24600
Car unbalance 12200 × 2.0 = 24400
49000 kgm

Bending stress = $\frac{49000 \times 100 \times 36}{272000}$ = 648
direct Comp = $\frac{38000}{340}$ = 112
760 kg/cm² O.K.

Approximate weight of structural steel for one pier

8Ls	100×75×10	@	12.95	×	6.6	=	683
4Ls	90×90×10	@	13.34	×	6.6	=	352
2 Pl/s	500×9	@	35.33	×	6.6	=	467
1 Pl.	700×10	@	54.95	×	4.0	=	220
2 Pl.	1200×10	@	94.20	×	4.0	=	752
4 Pl/s	~~500~~ 300 ×10	@	~~39.25~~ 23.55	×	2.0	=	~~157~~ 190
2Ls	125×90×10	@	16.09	×	3.0	=	97
20 Ls	90×75×9	@	11.02	×	1.0	=	220
2 Pl/s	600×9	@	42.39	×	1.2	=	102
2 "	600×9	@	42.39	×	0.7	=	59
20 Ls lac.	65×65×8	@	7.66	×	.65	=	100
Shoes. 2 Pl/s.	1300×19	@	193.90	×	1.5	=	582
2Ls	150×150×15	@	33.55	×	1.4	=	94
2Ls	"	@	"	×	1.2	=	82
splices + bolts, say							300
Rivet heads say							130
							4430 kg
miscellaneous details conn. to truss &c							170
							4,600 kg.

500×9

Summary of structural steel

Truss 41 @ 20 = 820
Pier 41 @ 4.6 = 189
1009 kg/tons

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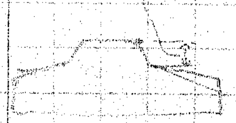
CALCULATIONS FOR

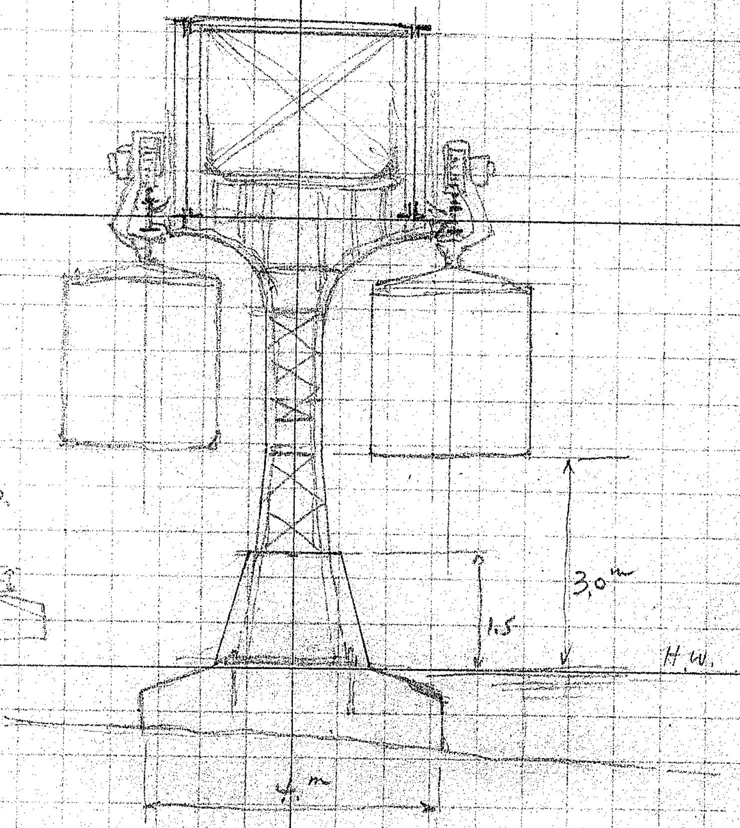
片瀬江ノ島懸垂電車路

P2

F附近

G I J
第ハ 鍛ス





volume of concrete

1.6 × 1.6 × 1.5 = 3.85
4.0 × 4.0 × 1.0 = 16.0
19.85 call this 20. Cub m

Car on one side

direct load on column 32,200 kg
wt. of pier steel 4,200
wt. of concrete base 20@2400 = 48,000
84,400 kg

Moment

Car unblance 12200 × 2.0 = 24,400 kgm
wind moment 6140 × 7.5 = 46,000
70,400 kgm

ecc. = 70400 / 84400 = 0.83 m

Cars on both sides

direct comp. on col. } 50000 kg
wt. of pier steel }
Concrete base 48,000
98,000 kg

Moment

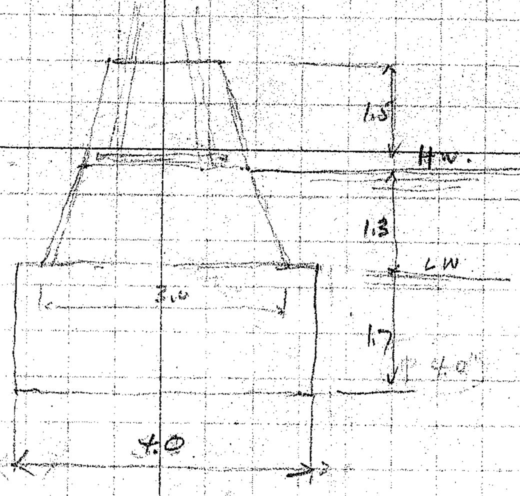
wind moment = 9300 × 7.6 = 70,000 kgm

ecc. = 70000 / 98000 = 0.71 m

max toe pres. = 84400 × 2 / 4 × 4 × 16 = 10.5 ton/m²

P3

I, J ノ間



wt. of conc.

2.1 × 2.1 × 2.8 = 12.3
4.0 × 4.0 × 1.7 = 27.2
39.5 cub m @ 2200 = 87,000 kg

Load.

pier 36,400
base 87,000
123,400 kg

moment. unblance 24,400
wind 6140 × 10.5 = 64,500
89,000 kgm

ecc = 89000 / 123400 = .72 m

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CALCULATIONS FOR

佐渡12 島縣 [illegible]

Estimate of cost

第一案 BM1 から J まで 1448 meters. 58 spans @ 25 m.

Structural steel	truss	58 @ 20 = 1160 kg tons		
	piers	59 @ 4.6 = 271		
		1431 kg tons	@ 250 =	358,000 円
Piers	P1 well	27 @ 1900	= 51,300	
	P2	18 @ 800	= 14,400	
	P3	14 @ 1500	= 21,000	
				86,700
				444,700 円
				70,000
				514,700

21000
87
108000

第2案 BM1 から K まで 延長 1566 m 63 spans @ 25 m

Structural steel	truss	63 @ 20 = 1260 kg tons		
	piers	64 @ 4.6 = 294		
		1554 kg tons	@ 250 =	388,000
Piers	P1	27 @ 1900	= 51300	
	P2	21 @ 800	= 16,800	
	P3	16 @ 1500	= 24,000	
				92100
				480,100 円
				80
				560,100

22000
9000
11000

第三案 BM1 から G. 41 span

Structural steel	truss	41 @ 20 = 820		
	pier	42 @ 4.6 = 193		
		246	1013 @ 250 =	253,500
Piers	P1	26 @ 1900	= 49400	
	P2	6 @ 800	= 4800	
	P3	10 @ 1500	= 15000	
			69,200	69,200
				322,700 円

560100
390000
170000

Loop + Stat. ニ 125
50
steel – 175 @ 250 =
Pier – 15 @ 1500 =

43,800
22,500
66,300

322700
66000
388700

$388700 \times \frac{63}{41} = 596000$ 60 tons @ 250 = 15000
69
84000

(41)

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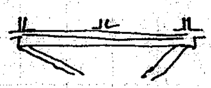
CALCULATIONS FOR

Katase - Enoshima Suspended Elec. Ry. Line
Estimate of Structural steel for the Design proposed by Yokogawa Bridge Co.

Structural steel for Truss.
Horizontal upper truss



Chords	4 Ls	130×130×15	@ 28.84 kg	× 20.4 m	=	2353
diagonals	24 Ls	75×75×9	@ 9.96	× 1.8	=	430
Struts	14 Ls	125×90×10	@ 14.13	× 2.25	=	445
〃	28 Ls	~~75~~ 100×75×9	@ 9.96	× 0.9	=	251
						3479
Details say 36%					=	1251
						4730 kg

Vertical truss

Chords top	~~4~~ 2 Ls	130×130×15	@ 28.84	× 19.6	=	1,130
〃 bottom	2 Ls	〃	〃 @ 〃	× 15.5	=	894
diagonals	8 Ls	130×130×12	@ 23.36	× 3.5	=	655
〃	8 Ls	100×100×10	@ 14.91	× 3.5	=	418
bottom strut	~~4~~ Ls	80×80×9	@ 10.66	× 2.45	=	105
cov. p/s. (chord)	2 P/s.	270×9	@ 19.08	× 5.5	=	210
verticals	14 Ls	80×80×9	@ 10.66	× 2.5	=	373
						3785
Details say 25%					=	945
						4730 kg

Lower Horizontal truss

Chords	2 Is	12"×6"	@ 44.02# or ~~20.0~~ 66. kg	× 20.0	=	~~800~~ 2640
diagonals	8 ~~4~~ Ls	90×90×10	@ 13.34	× ~~4.3~~ 1.9	=	203
〃	8 ~~4~~ Ls	80×80×9	@ 10.66	× ~~4.3~~ 1.9	=	162
〃	8 ~~4~~ Ls	75×75×9	@ 9.96	× ~~4.3~~ 1.9	=	152
Struts	6 Ls	75×75×9	@ 〃	× 3.5	=	209
〃	7 Ls	90×90×10	@ 13.34	× 3.5	=	327
Stays	28 Ls	100×75×~~9~~ 10	@ 12.95	× 3.0	=	1088
Sway bracings	4 Ls	80×80×9	@ 10.66	× 2.8	=	120
〃	8 Ls	〃 〃	@ 10.66	× 1.25	=	107
〃	2 Ls	75×75×9	@ 9.96	× 3.5	=	70
						5078
Details say 25%					=	1272
						6350 kg

Bracing at pier

verticals	4 Ls	90×90×10	@ 13.34	× ~~0.85~~ 1.5	=	~~45~~ 81
diagonals	4 Ls	125×90×10	@ 16.09	× 2.3	=	148
〃	4 Ls	90×90×10	@ 13.34	× 0.85	=	45
Struts.	1 I	200×70	@ 21.12	× 3.5	=	74
Details say						402
						750 kg

Summary of structural steel for one span

upper horizontal truss	4730
vertical truss	4730
lower horizontal truss	6350
bracing at pier	750
	16,560 kg

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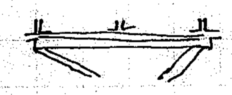
CALCULATIONS FOR

Katase - Enoshima Suspended Elec. Ry. Line
Estimate of structural steel for the Design proposed by Yokogawa Bridge Co.

Structural steel for Truss.

Horizontal upper truss.



Chords	4 Ls	130×130×15 @ 28.84 kg	× 20.4 m	=	2353
diagonals	24 Ls	75×75×9 @ 9.96	× 1.8	=	430
Struts	14 Ls	125×90×10 @ 14.13	× 2.25	=	445
〃	28 Ls	~~75~~ 100×75×9 @ 9.96	× 0.9	=	251

3479

Details say 36% = 1251

4730 kg.

Vertical truss

Chords top	~~4~~ 2 Ls	130×130×15 @ 28.84	× 19.6	=	1,130
〃 bottom	2 Ls	〃 〃 @ 〃	× 15.5	=	894
diagonals	8 Ls	130×130×12 @ 23.36	× 3.5	=	655
〃	8 Ls	100×100×10 @ 14.91	× 3.5	=	418
bottom strut	4 Ls	80×80×9 @ 10.66	× 2.45	=	105
cov. p/s. (chord)	2 P/s.	270×9 @ 19.08	× 5.5	=	210
verticals	14 Ls	80×80×9 @ 10.66	× 2.5	=	373

3785

Details say 25% = 945

4,730 kg

Lower Horizontal truss

Chords	2 Is	12"×6" @ 44 lb/ft or ~~20.0~~ 66. kg	× 20.0	=	~~800~~ 2640
diagonals	8 ~~4~~ Ls	90×90×10 @	13.34 × ~~4~~ 3.19	=	203
〃	8 ~~4~~ Ls	80×80×9 @	10.66 × ~~4~~ 3.19	=	162
〃	8 ~~4~~ Ls	75×75×9 @	9.96 × ~~4~~ 3.19	=	152
Struts	6 Ls	75×75×9 @	〃 × 3.5	=	209
〃	7 Ls	90×90×10 @	13.34 × 3.5	=	327
Stays	28 Ls	100×75×~~9~~ 10 @	12.95 × 3.0	=	1088
Sway bracings	4 Ls	80×80×9 @	10.66 × 2.8	=	120
〃	8 Ls	〃 〃 @	10.66 × 1.25	=	107
〃	2 Ls	75×75×9 @	9.96 × 3.5	=	70

5078

Details say 25% = 1272

6350 kg

Bracing at Pier

verticals	4 Ls	90×90×10 @	13.34 × ~~0.85~~ 1.5	=	~~45~~ 81
diagonals	4 Ls	125×90×10 @	16.09 × 2.3	=	148
〃	4 Ls	90×90×10 @	13.34 × 0.85	=	45
Struts	1 I	200×70 @	21.12 × 3.5	=	74
Details say					402

750 kg

Summary of structural steel for one span

upper horizontal truss	4730
vertical truss	4730
lower horizontal truss	6350
bracing at pier	750
	16,560 kg

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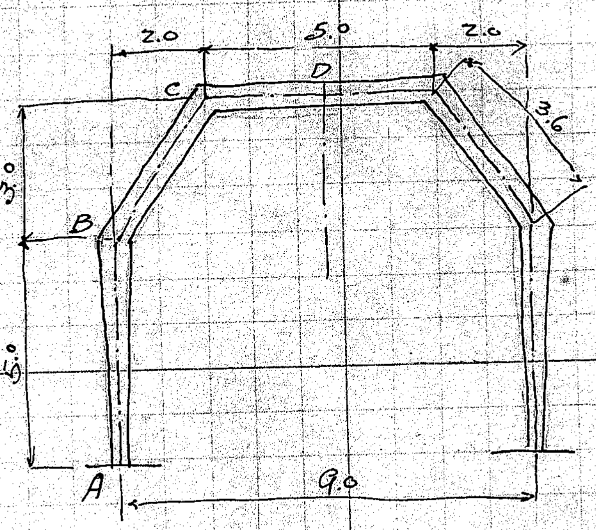
CALCULATIONS FOR

Katase–Enoshima Suspended Elec. Ry. Line

Structural steel for Pier Bent.



Section	A	245.68 cm² qr	@ .785 kg/m =	193 kg/m
〃	B	395.68 〃 〃	@ 〃 =	310.5 〃
〃	C	395.68	@ 〃 =	310.5 〃
〃	D	509.08 〃 〃	@ 〃 =	400. 〃

weight of main section
Between A~B.
~~243~~ 193 × 4 = 772
311

average unit wt. 1083 ÷ 5 = 216.6 kg/m

216.6 × 5 = 1,083 kg × 2 = 2,166 kg
B.C. 310.5 × 3.6 = 1118 × 2 = 2236
C.D. 400 × 2.5 = 1000 × 2 = 2000
6,402 kg

Splice say 2 meters @ 400 = 800
Shoes & anchor bolts 2 @ ~~800~~ 750 375 = ~~1500~~ 750 ~~1600~~
Stiffeners, diaphgrams tigh pl. lacing &c. say 800 ~~1000~~
9802

Rivet heads &c. say 3.5% = 348
~~10150~~ kg
~~9850~~
9,100

Total steel for one span
Super structure = 16.56
Sub structure = ~~10.15~~ ~~9.85~~ 9.10
~~26.71~~ kg tons.
~~26.41~~ 25.66

Total length of structure = 3350 ft = 1,020 meters
no of spans reqd = 1020 ÷ 20 = 51 spans.

Total steel for the whole structure = ~~26.41~~ 25.66 × 51 = ~~1347~~ 1310 kg tons.

BM1 to J 1448 73 @ 20 m
Steel 73 @ 25.66 = 1,880 kg tons

BM1 to K 1566 m 78 @ 20
Steel 78 @ 25.66 = 2010 kg tons

1310
-44
1266

1,310 積り −44 = 1,266
$1160 \times \frac{2240}{2206} =$ 1,180 [illegible]
No.1 1135 } [illegible]
No.2 1066
No.3 1009

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CALCULATIONS FOR 片瀬江ノ島電車 空中

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For two trusses system 第一案

Truss — 19,455
Column — 3,337.5
Rivet heads .800
23,593

TRUSS 2 Req'd

Top & Bott. Chords	4	Ls	125×90×13	24.500	20.61	2019.8
Fills	40	Fills	150×10	.150	11.775	70.7
Diagonal	4	Ls	100×100×10	3.200	14.91	190.8
〃	8	〃	90×90×10	3.200	13.34	341.5
	8	〃	75×75×9	3.200	9.96	255.0
filler	8	Fills	150×10	210	11.775	19.8
〃	16	〃	〃	190	〃	35.8
〃	16	〃	〃	160	〃	30.1
Verticals	18	Ls	75×75×9	2.680	9.96	480.5
Vertical・ST Conn. L	9	〃	〃	700	〃	62.7
filler	36	Fills	150×10	160	11.775	67.8
End Post	4	Ls	100×75×10	2.680	12.95	138.8
Gusset Pls	11	Pl.	400×10	750	31.40	259.1
〃 〃	9	〃	300×10	400	23.55	84.8
〃 at Lo	4	Fills	80×13	400	8.164	13.1
〃 〃 〃	4	Pl	200×10	400	15.700	25.1
						4095.4
						×2
						8,190.8

TOP LATERALS

Diagonals	10	Ls	75×75×9	3.100	9.96	308.8
〃	20	〃	〃	1.500	〃	298.8
gusset	10	Pls	300×9	.550	21.195	116.6
〃	4	〃	400×9	450	28.26	50.9
〃	18	〃	450×9	650	31.793	372.0
Struts	11	Ls	90×90×10	2.200	13.34	322.8
						1,469.9

BOTTOM LATERALS

Diagonals	20	Ls	75×75×9	3.030	9.96	603.6
〃	40	〃	〃	1.450	〃	577.7
Gusset	10	Pls	400×9	650	28.26	183.7
〃	4	〃	600×9	600	42.39	101.7
〃	10	〃	420×9	630	29.673	186.9
〃	8	〃	520×9	950	36.738	279.2
Struts	20	Ls	100×90×10	4.560	14.13	1288.7
Side bracings	12		75×75×9	2.300	9.96	274.9
~~Gusset~~	~~12~~		~~230×9~~	~~320~~	~~16.25~~	~~62.4~~
	27	Washer	60φ×10		@ 0.222	6.0
Gusset	6	Pls	380×9	650	26.847	96.7
〃	12	〃	300×9	350	21.195	89.0
	4	〃	200×9	350	14.13	19.8
						3,707.9

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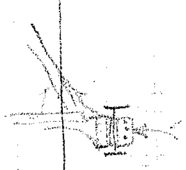
CALCULATIONS FOR

	16	Lac Ls 75×75×9	600	9.96	95.6
	4	〃 〃	1,000	〃	39.8
Side	2	Ls 150×100×9	800	17.02	27.2
	2	〃 〃	490	〃	16.7
	2	Fills 150×10	290	11.775	6.8
	4	Ls 90×90×10	210	13.34	11.2
	2	〃 150×100×9	1,050	17.02	35.7
脚部	2	〃 〃	490	〃	16.7
	2	Fill 150×10	290	11.775	6.8
	2	Ls 150×100×9	760	17.02	25.9
	2	Fills 150×10	600	11.775	14.1
	1	Pl. 750×19	1,050	111.863	117.5
Stiffener for Column	16	Ls 75×75×9	570	9.96	81.3
Anchor	4	bolts 32φ	1.100	@ 7.726	30.9
	4	Washer 150×9	150	10.598	6.4
					3337.5
					33375
		I Beam under Rail			
	2	Is 300×150 @48.34	25,000		2417.0
 Conn. Ls	20	L 100×100×10	240	14.91	71.7
					2488.7

22,649k
.55

22793.5
.55
2334

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CALCULATIONS FOR 片瀬江ノ島空中電車

[illegible]

truss — 19,251.9
Column — 3177.7
Rust heads — 780
23,209.6

for One truss System

Horizontal truss						
Chords	4	Ls	130×130×12	24,250	23.36	2265.9
Diagonal	8	〃	90×90×10	2,250	13.34	240.1
〃	24	〃	75×75×9	2,200	9.96	525.9
End conn.	2	〃	125×90×10	800	16.09	25.7
〃	4	〃	〃	230	〃	14.8
〃	2	Pls	800×9	850	56.52	96.1
Gusset	4	〃	400×9	1,050	28.26	118.7
〃	6	〃	380×9	1,050	26.847	169.1
	2	〃	650×9	850	45.923	78.1
	2	〃	550×9	750	38.858	58.3
tension Pl.	5	〃	75×9	600	5.299	15.9
	8	〃	150×9	400	10.598	33.9
Fill.	40	Fills	150×9	270	〃	114.5
〃	8	〃	150×9	190	〃	16.1
〃	24	〃	150×9	160	〃	40.7
						3,813.8
Vertical truss.						
Chord	2	Ls	150×150×15	24,500	33.55	1,644.0
〃	2	〃	〃	20,200	〃	1,355.4
Diagonal	8	〃	130×130×12	3,150	23.36	588.7
〃	12	〃	100×100×10	3,150	14.91	563.6
Vertical	18	〃	75×75×9	2,720	9.96	487.6
〃	4	〃	100×100×10	2,720	14.91	162.2
lag Ls for diagonals	8	〃	130×130×12	1,600	23.36	186.9
Gusset	2	Pls	750×13	900	76.538	137.8
〃	4	Fills	80×12	500	7.536	15.1
〃	4	Pls	185×12	500	17.427	34.9
〃	2	Pls	700×13	1,200	71.435	171.4
〃	2	〃	600×13	1,000	61.23	122.5
〃	5	〃	500×13	1,000	51.025	255.1
Filler	36	Fills	150×13	150	15.308	82.7
〃	12	〃	150×13	270	〃	49.6
〃	18	〃	150×13	210	〃	57.9
〃	27	〃	150×13	160	〃	66.1
Gusset	9	Pl.	300×13	300	30.615	82.7
Brackets	4	Ls	100×100×10	2,800 3,200	14.91	167.0 191.0
〃	2	Fill	100×13	200	10.205	4.1
~~〃~~	~~4~~	~~Pls~~	~~230×13~~	~~430~~	~~23.472~~	~~40.4~~
~~〃~~	~~8~~	~~Ls~~	~~150×100×9~~	~~180~~	~~17.02~~	~~24.5~~
						6,300.2
						6,259.3
Between truss to truss						
	4	Ls	100×100×10	2450	14.91	~~23.9~~ 146.
	2	Pl.	400×9	500	28.26	~~14.1~~ 28.2
	6	Bars	80×9	700	8.164 5.652	~~29.4~~ 23.7
	2	Ls	100×100×10	490	14.91	14.6
	2	Fills	100×10	290	7.85	4.6
						217.1

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CALCULATIONS FOR 片瀬江ノ島空中電車

第二案

truss — 19,251.9
Column — 3177.7
Rivet heads — 780
23,209.6

For One truss System

Horizontal truss

Chords	4	Ls	130×130×12	24,250	23.36	2265.9
Diagonal	8	〃	90×90×10	2,250	13.34	240.1
〃	24	〃	75×75×9	2,200	9.96	525.9
End conn.	2	〃	125×90×10	800	16.09	25.7
〃	4	〃	〃	230	〃	14.8
〃	2	Pls	800×9	850	56.52	96.1
Gusset	4	〃	400×9	1,050	28.26	118.7
〃	6	〃	380×9	1,050	26.847	169.1
	2	〃	650×9	850	45.923	78.1
	2	〃	550×9	750	38.858	58.3
tension Pl.	5	〃	75×9	600	5.299	15.9
	8	〃	150×9	400	10.598	33.9
Fill	40	Fills	150×9	270	〃	114.5
〃	8	〃	150×9	190	〃	16.1
〃	24	〃	150×9	160	〃	40.7
						3,813.8

Vertical truss.

Chord	2	Ls	150×150×15	24,500	33.55	1,644.0
〃	2	〃	〃	20,200	〃	1,355.2
Diagonal	8	〃	130×130×12	3,150	23.36	588.7
〃	12	〃	100×100×10	3,150	14.91	563.6
Vertical	18	〃	75×75×9	2,720	9.96	487.6
〃	4	〃	100×100×10	2,720	14.91	162.2
lag Ls for diagonals	8	〃	130×130×12	1,000	23.36	186.9
Gusset	2	Pls	750×13	900	76.538	137.8
〃	4	Fills	80×12	500	7.536	15.1
〃	4	Pls	185×12	500	17.427	34.9
〃	2	Pls	700×13	1,200	71.435	171.4
〃	2	〃	600×13	1,000	61.23	122.5
〃	5	〃	500×13	1,000	51.025	255.1
Filler	36	Fills	150×13	150	15.308	82.7
〃	12	〃	150×13	270	〃	49.6
〃	18	〃	150×13	210	〃	57.9
〃	27	〃	150×13	160	〃	66.1
Gusset	9	Pl.	300×13	300	30.615	82.7
Brackets	4	Ls	100×100×10	2,800 3,200	14.91	167.0 191.0
〃	2	Fill	100×13	200	10.205	4.1
~~〃~~	~~4~~	~~Pls~~	~~230×13~~	~~430~~	~~23.472~~	~~40.4~~
〃	~~8~~	~~Ls~~	~~150×100×9~~	~~180~~	~~17.02~~	~~24.5~~
						6,300.2
						6,259.3

Between truss to truss

4	Ls	100×100×10	~~2,450~~	14.91	~~43.9~~	146.
2	Pl.	400×9	500	28.26	~~74.1~~	28.2
6	Bars	80×9	700	8.164 5,652	~~29.4~~	23.7
2	Ls	100×100×10	490	14.91	14.6	
2	Fills	100×10	290	7.85	4.6	
					217.1	

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CALCULATIONS FOR

	Bottom lateral					
Strut	20	Ls	100×90×10	4,560	14.13	1288.7
Diagonal	40 ~~20~~	〃	75×75×9	3,100	9.96	~~617.5~~ 1235.0
	~~40~~	~~〃~~	~~〃~~	~~1,400~~	~~〃~~	~~557.8~~
	~~20~~	~~Pls~~	~~300×9~~	~~550~~	~~21.195~~	~~[illegible]~~
Strut	4	Ls	75×75×9	2,180	9.96	86.9
	2	Pls	400×9	700	28.26	39.6
	4	〃	400×9	700	〃	79.1
	18	〃	380×9	620	26.847	299.6
	2	〃	780×9	900	55.107	99.2
	7	〃	680×9	740	48.042	248.9
	40	Fill	70φ×9		@ 0.335	13.4
	20	〃	75×9	150	5.299	15.9
						3406.3
	Wing strut.					
	36	Ls	100×75×10	3,250	12.95	1515.2
	18	Pls	400×9	980	28.26	498.5
	18	Ls	75×75×9	1,000	9.96	179.3
	18	〃	〃	500	〃	89.6
	18	Pls	350×9	600	24.728	267.1
	36	Ls	100×100×10	240	14.91	128.8
	54	Fills	70φ×9		.335	18.1
						2696.6
	Wing strut on column.					
	8 ~~4~~	Ls	100×75×10	3,150	12.95	~~168.4~~ 326.
	6	Fills	150×9	160	10.598	~~2.0~~ 10.2
	1	Pl.	600×9	800	42.39	~~34.7~~ 33.9
						~~225.1~~ 370.1
						16763.2
						2488.7
						19251.9
	Column					
	4	Ls	100×100×10	6,600	14.91	393.6
	8 ~~4~~	〃	〃	5,500	〃	~~328.0~~ 656.0
	~~4~~	~~〃~~	~~〃~~	~~4,350~~	~~〃~~	~~259.4~~
Web	1	Pl.	480×9	~~4,350~~ 5,500	33.912	~~147.5~~ 187.0
	2	〃	500×9	~~6,600~~ 5,500	35.325	~~466.3~~ 388.0
	2	Ls	100×100×10	~~3,900~~ 3,700	14.91	~~116.3~~ 110.0
	8	〃	〃	240	〃	28.6
	4	〃	75×75×9	~~990~~ 600	9.96	~~39.4~~ 23.8
	2	Pls	600 ~~900~~×9	~~1,500~~ 1,100	63.585	~~190.8~~ 140.0
	4 ~~2~~	〃	550 ~~1,100~~×9	~~4,800~~ 1,050	77.715	~~746.1~~ 327.0
	2	〃	700 ~~1,650~~×9	~~1,650~~ 900	11.304	~~37.3~~ 20.4
diaphragm	4	Ls	90×90×10	~~700~~ 550	13.34	~~37.4~~ 29.4
	1	Pl.	500×9	700	35.325	24.7
	2	Ls	90×90×10	480	13.34	12.8
Lacing	16	〃	75×75×9	600	9.96	95.6
	2	Pls	530×9	1,100	37.445	82.4
	2	Ls	150×100×12	1,100	22.41	49.3
	2	〃	〃	480	〃	21.5

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CALCULATIONS FOR

	2	Fills	150×10	290	11.775	6.8
	4	Ls	90×90×10	210	13.34	11.2
脚部	2	〃	150×100×9	710	17.02	24.2
	2	〃	〃	480	〃	16.3
	2	Fills	150×10	290	11.775	6.8
	2	Ls	150×100×9	470	17.02	16.0
	2	Fills	150×10	290	11.775	6.8
	1	Pl.	710×19	710	105.897	74.6
Stiffener	16	L	75×75×9	510	9.96	81.3
	4	Bolts	32φ	1,100		30.9
	4	Washers	150×9	150		6.4
鬼ボルト	4	anch. bolt		700	@9.kg	36.0
上部レーシング	16	Lac. Bars	70×9	500	4.946	39.6
	4	Pls	220×9	250	15.543	15.6
	4	Lac. Ls	75×75×9	700	9.96	37.9
						3,000.5

317.3

I Beam under Rail.

2	Is	300×150 @ 48.34	25,000	2417.0
20	L	100×100×10	240	71.7
				2488.7

~~3875.0~~

Lacing Bar and Tie Pls for Column.

1	Pl.	470×9	4,500	33.206	150.0
8	Lac. Bars	80×9	600	5.652	27.2
					177.2

22,252.4
177.2
22,429.6

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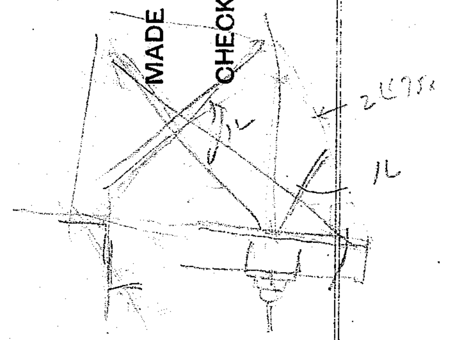
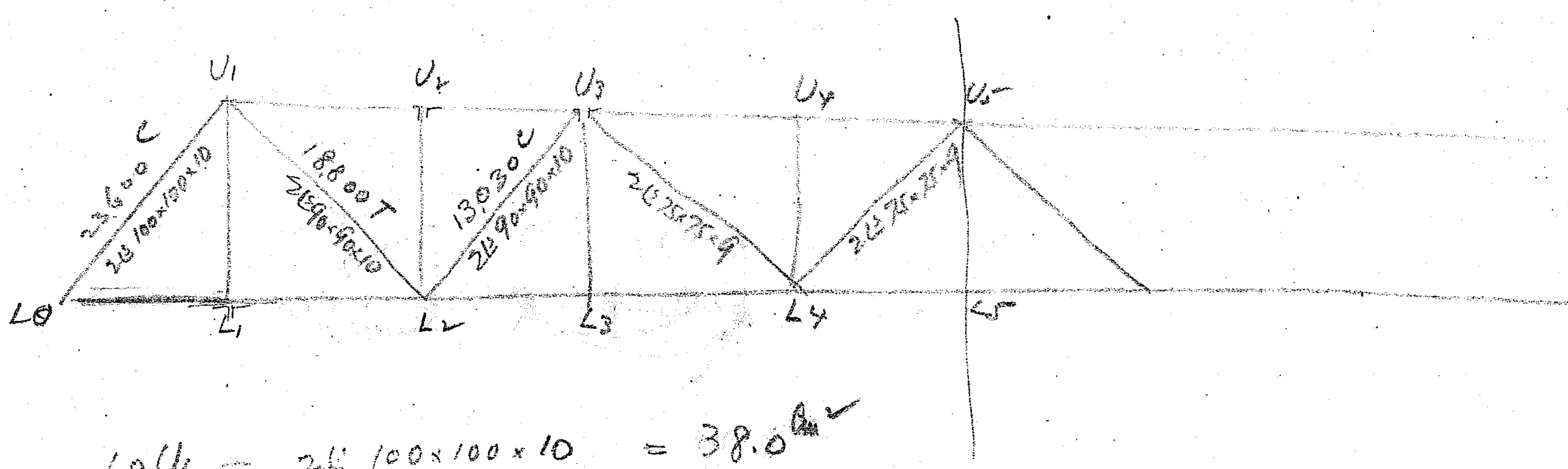


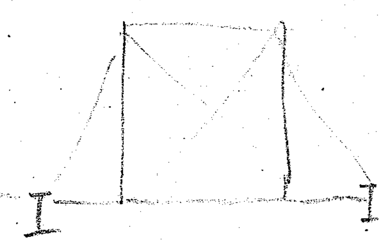
(1)

Vertical truss – for 2 trusses



$\frac{6100}{24.9}\ \cancel{20.9} = \cancel{63800}$

63500



L_0U_1 – 2L 100×100×10 = 38.0 cm²

$r = 3.96$ cm $l = 300$ cm assumed $l/r = \frac{300}{3.96} = 76 < 100$ O.K.

$1200 - 5\,l/r = 821$

$\frac{23{,}600}{821} = 27.6$ cm² O.K.

U_1L_2 – 2L 90×90×10 = 34.0

$34. - 4.4 = 29.6$ net

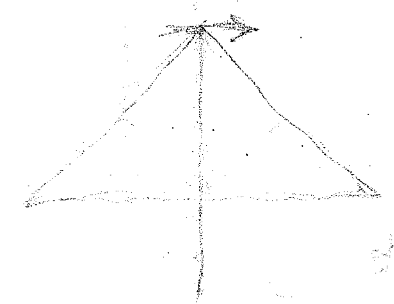
$\frac{18{,}800}{1200} = 15.67$

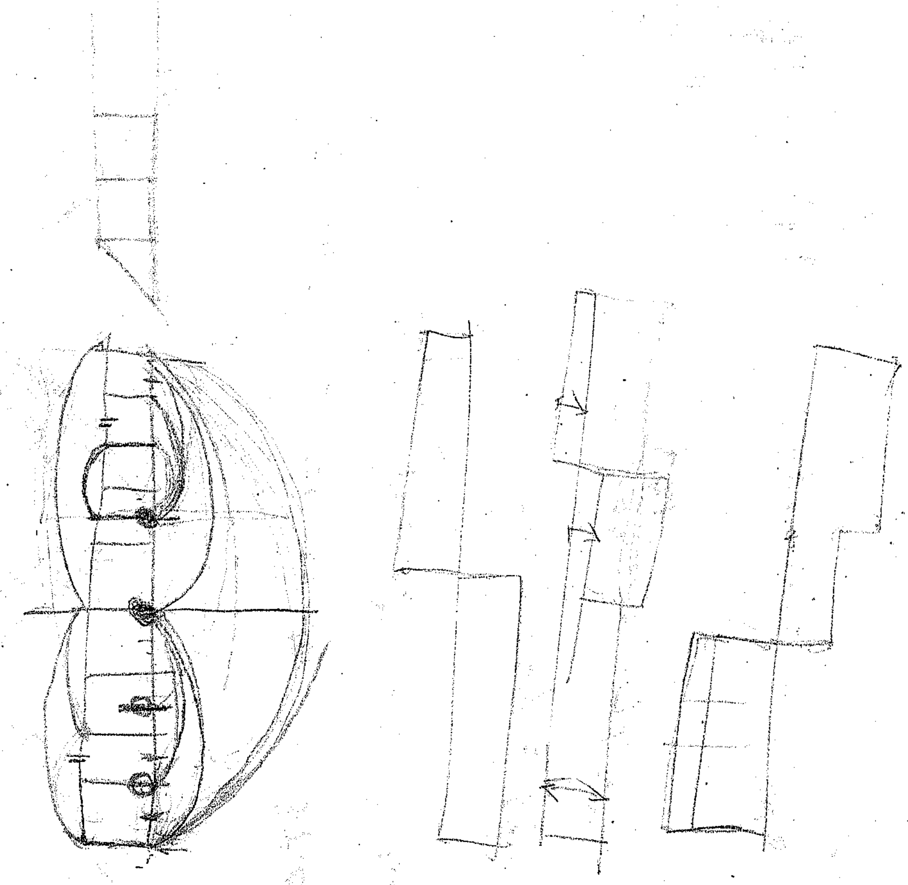
L_2U_3 – 2L 90×90×10 = 34.0 cm²

$r = 3.43$ cm $l = 300$ cm $l/r = \frac{300}{3.43} = 87.5 < 100$

$1200 - 5\,l/r = 762$

$\frac{13{,}030}{762} = 17.1$ cm²





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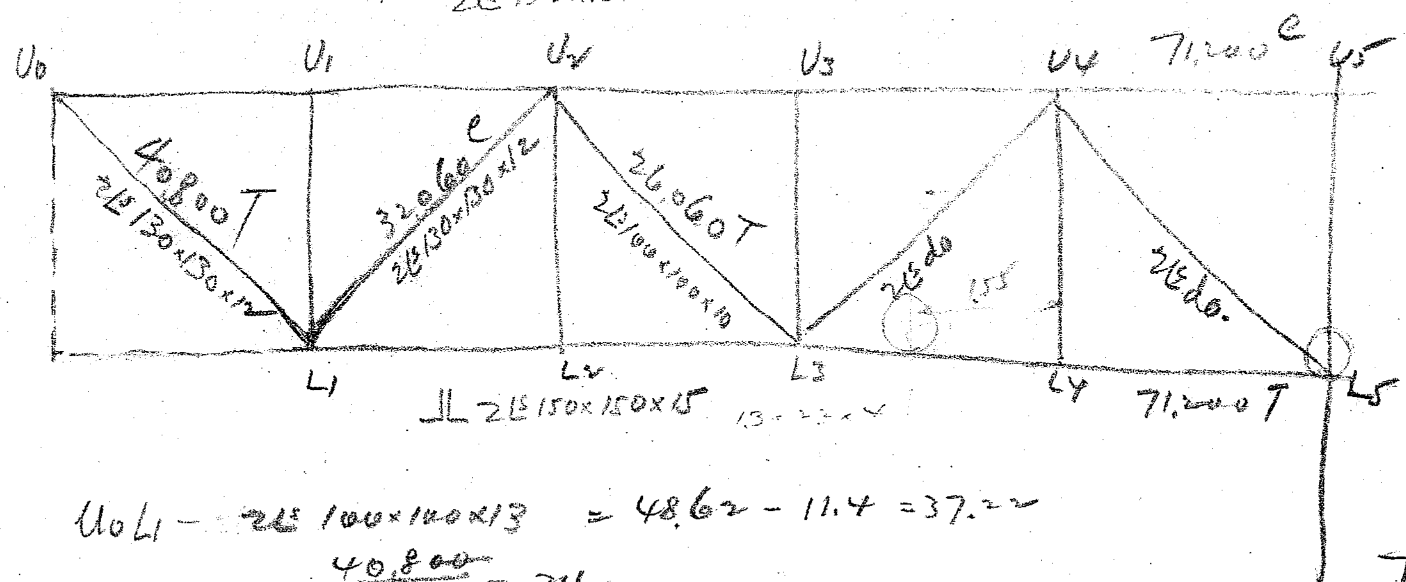
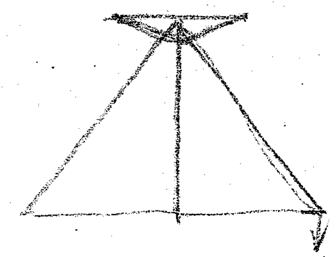
2

Vertical truss for 1 truss



$\frac{92,500}{2.6} \times 2 = 71,200$



	3675
	3675
	7350
	24.31
	97.8

U0L1 — ~~2L~~ 100x100x13 = 48.62 − 11.4 = 37.22

$\frac{40,800}{1200} = 34.0$

L1U2 — 2L 130x130x12 = 59.44 cm²

$r = 4.95$ cm $l = 300$ $l/r = \frac{300}{4.95} = 60.7 < 100$ OK

$1200 - 5\frac{l}{r} = 830$ $\frac{32060}{830} = 38.7$

49.4

100x100x13 = 48.62

$l/r = \frac{300}{389} = 77.3 < 100$ $1200 - 5 l/r = 814$

$\frac{32,060}{814} = 39.4$ cm²

Top chord — 2L 150x150x15 = 85.5 cm²

~~$l/r = \frac{240}{5.91} = 40.7 < 100$~~

~~$1200 - 5 l/r = 970$~~

$\frac{71200}{970} = 73.4$ cm²

$l/r = \frac{240}{4.17} = 57.6$

$1200 - 5 l/r = 912$

$\frac{71200}{912} = 78$

Bottom chord 2L 150x150x15 = 85.5 cm² − 13.2 = 72.3

$\frac{71,200}{1200} = 59.2$

~~$l/r = \frac{240}{4.24} = 56.6$~~

~~$1200 - 5 l/r = 917$~~

~~$\frac{71200}{917} = 77.8$~~

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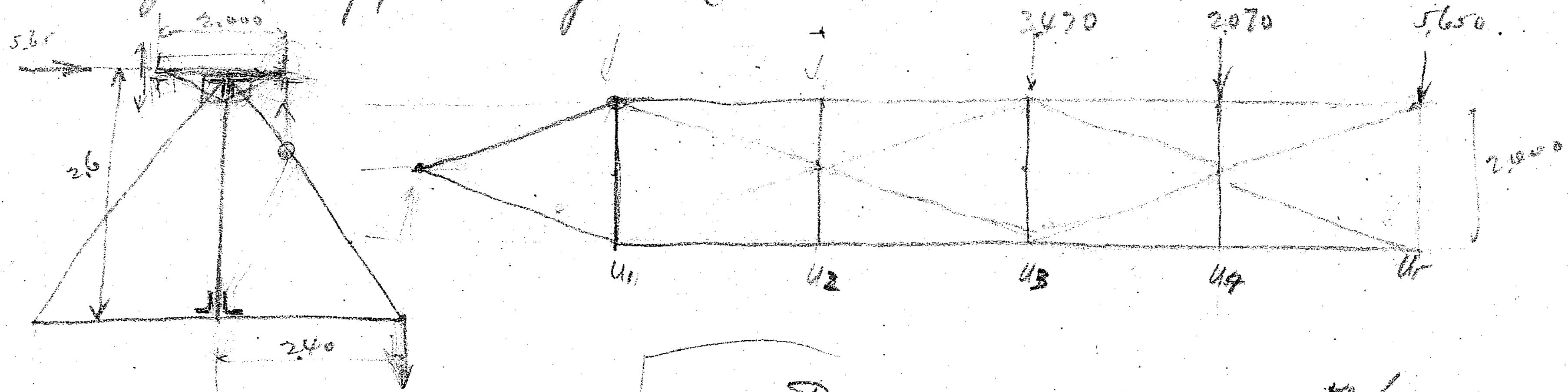
Design of upper horizontal truss



LL - 5,610
WL - 1,020
2.60
6,91

Live load 6,100

6100 × 2.4 = 14,650

$\frac{14,650}{2.6}$ = 5,650

$\frac{6,100}{24.5}$ × 9 = 2,240

$\frac{2,240 \times 2.4}{2.6}$ = 2070

6,100
2,340
3,760

$\frac{3760 \times 2.4}{2.6}$ = 3,470

Dead load assumed 450 kg/m

~~45 × 2.45 × 1100 ÷ 2 = 550~~

~~$\frac{550 \times 2.4}{2.6}$ = 570~~

$\frac{450 \times 2.4}{2.6}$ = 416

D.L mom. — $\frac{1}{8}$ × 416 × 24.5² = 25,000

" shear — $\frac{1}{2}$ × 416 × 24.5 = 5,100

maximum end shear for live load.

2070 × 22.05 = 45,700
3,470 × 19.60 = 68,000
113,700 ÷ 24.5 = 4,640
5,650
10,290

Reaction

5,650 × 12.25 = 69,300
2,070 × 9.80 = 20,270
3,470 × 7.35 = 25,500
115,070 ÷ 24.5 = 4,700

5650 × 12.25 = 69,300
2070 × 14.70 = 30,400
3470 × 17.15 = 59,500
159,200 ÷ 24.5
= 6,500

max. mom.
4,700 × 12.25 = 57,600 kgm

live load shear

5,650 × 24.05 = 137,000
2070 × 19.6 = 40,600
3470 × 17.15 = 59,500
237,100 ÷ 24.5 = 9,700

12.25 = 69,300
14.7 = 30,400
17.15 = 59,500
159,200 ÷ 24.5 = 6,600

Summary of moment and shear

	moment	End shear (panel)	2nd	3rd
Dead load	25,000	5,100	3050	2040
Live load	57,600	9,700	7,950	6,950
	82,600	14,800		
Wind load	7,900	1,290	772	515
	90,500	16,090	11,770	9,510

$\frac{11,770 \times 24.5}{1.85}$ = 15,550

3

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SEIYU BLDG, TOKIO

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CALCULATIONS FOR

Chord stress

$\frac{90500}{2.0} = 45{,}250$ C or T

Compression Chord

2Ls 100×100×13 . = 48.62

$l/r = \frac{240}{3.89} = 61.8 < 100$

$1200 - 5\,l/r = 891$

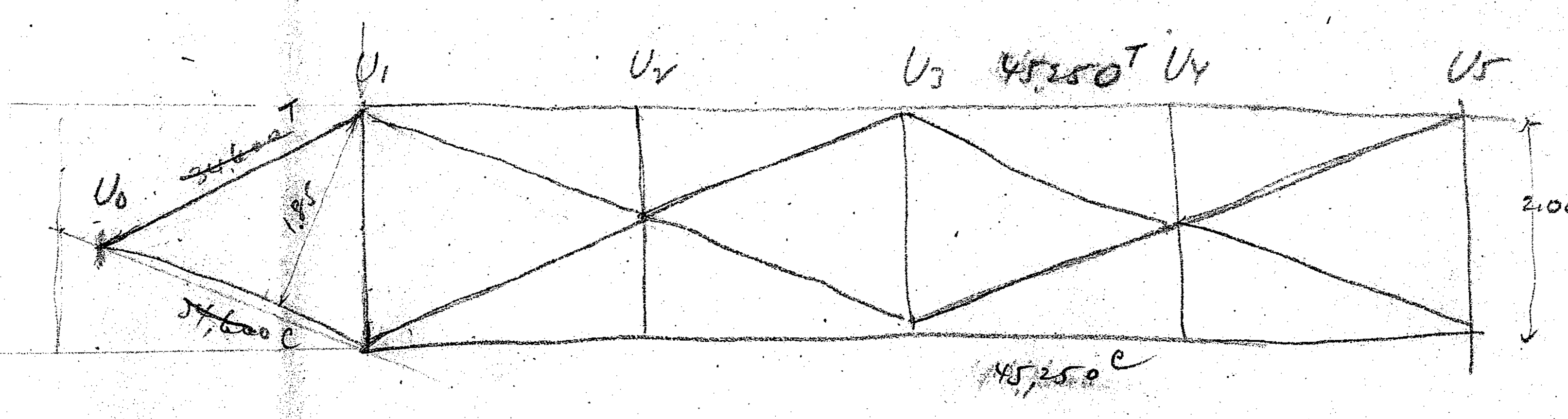
$\frac{44{,}600}{891} = 50.2$ cm²

2Ls 130×130×12 = 59.52

$l/r = \frac{240}{4.95} = 48.5 < 100$

$1200 - 5\,l/r = 958$

$\frac{45{,}250}{958} = 47.2$



$\frac{240}{343} = 81.5$

$\frac{240}{396} = 60.6$

$1200 - 5\frac{l}{r} =$ 730 55.6

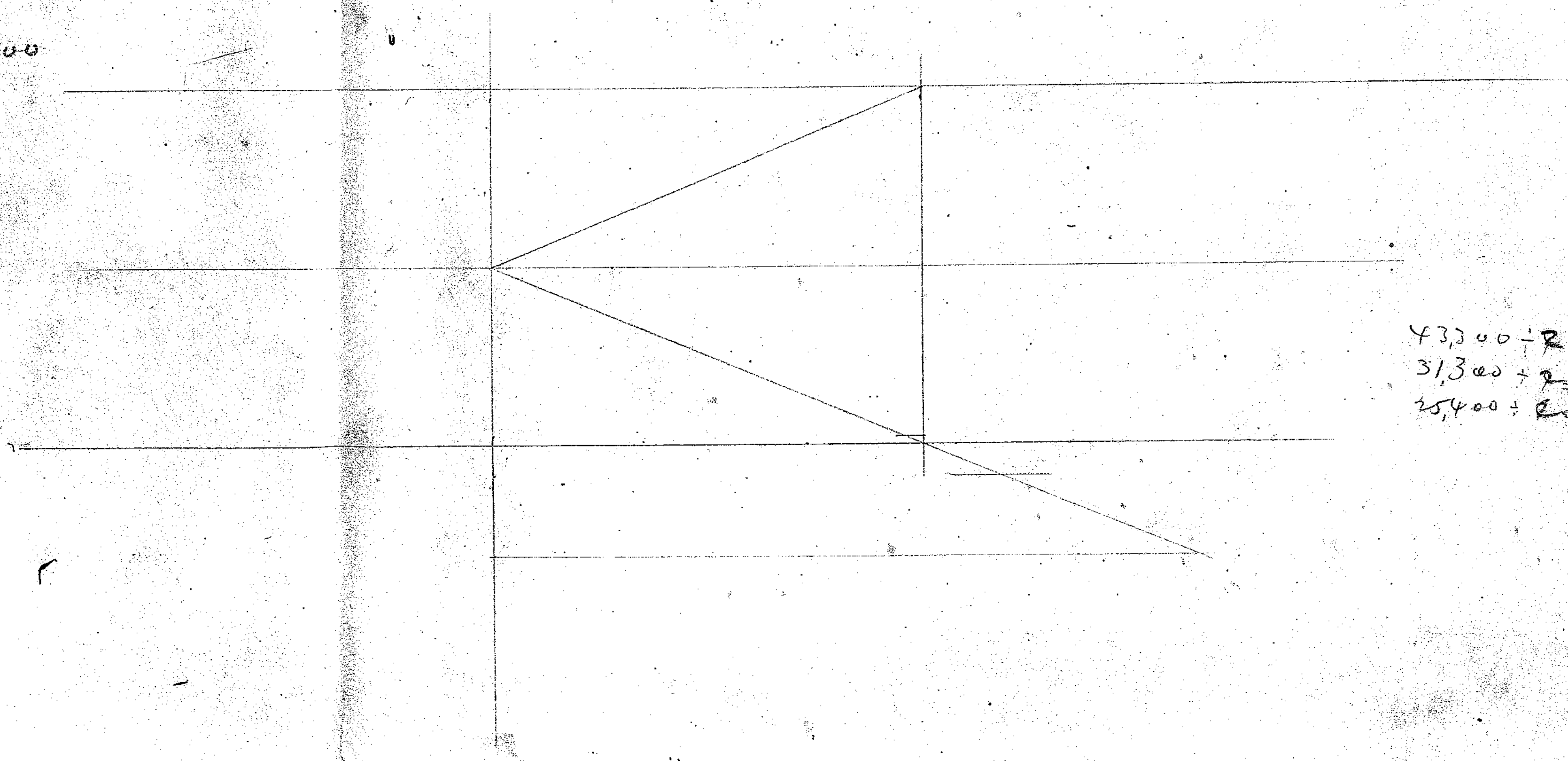
897 4.63

stress in U0 U1

Maximum shear at end panel = 16,090

moment at U1 $16{,}090 \times 2.45 = \cancel{64{,}000}$ 39,500

stress in U0 U1 $\frac{\cancel{64{,}000}\ 39{,}500}{1.85} = \cancel{34{,}600}\ 21{,}350$ T or C



43,300 ÷ 2 = 21,650

31,300 ÷ 2 = 15,650

25,400 ÷ 2 = 12,700

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CALCULATIONS FOR

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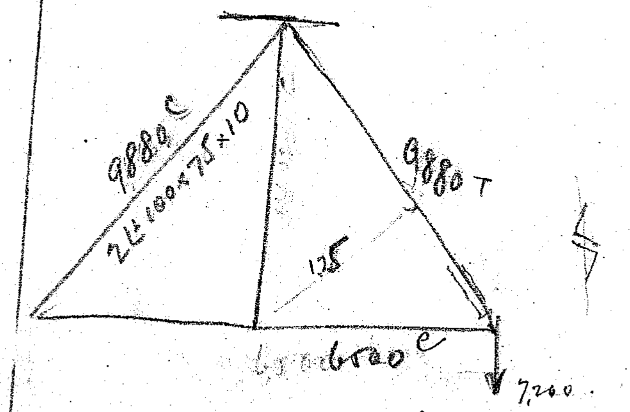
Wind load

$200^{kg} \div .7 = 140 \times 0.75 = 105.$

wind load assumed $105\ ^{kg}/_{m}$

Max mom. $= \frac{1}{8} \times 105 \times 24.5^2 = 7900$ mkg

max shear $= \frac{1}{2} \times 105 \times 24.5 = 1{,}290$



$\frac{7{,}200 \times 2.4}{2.60} = 6{,}650$

$\frac{7{,}200 \times 2.4}{1.8} =$

Dead load — $450 \times 2.45 = 1{,}100$
live load — $6{,}100$
$7{,}200$

$\frac{7{,}200 \times 2.4}{1.78} = 9{,}880$

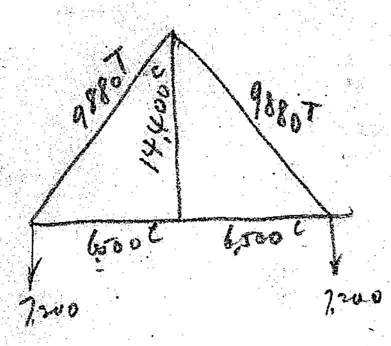
$\frac{7{,}200 \times 2.4}{2.66} = 6{,}500$

2∟ 100×75×10 $= 33.0$ cm²

$l/r = \frac{310}{3.20} = 97. < 100$

$1200 - 5\,l/r = 715$

$\frac{9{,}880}{715} = 13.8$ cm²



Vertical member

2∟ 75×75×9 $= 25.28$

$l/r = \frac{230}{2.94} = 78.3 < 100$

$1200 - 5\,l/r = 809$

$\frac{14{,}400}{809} = 17.8$ O.K.

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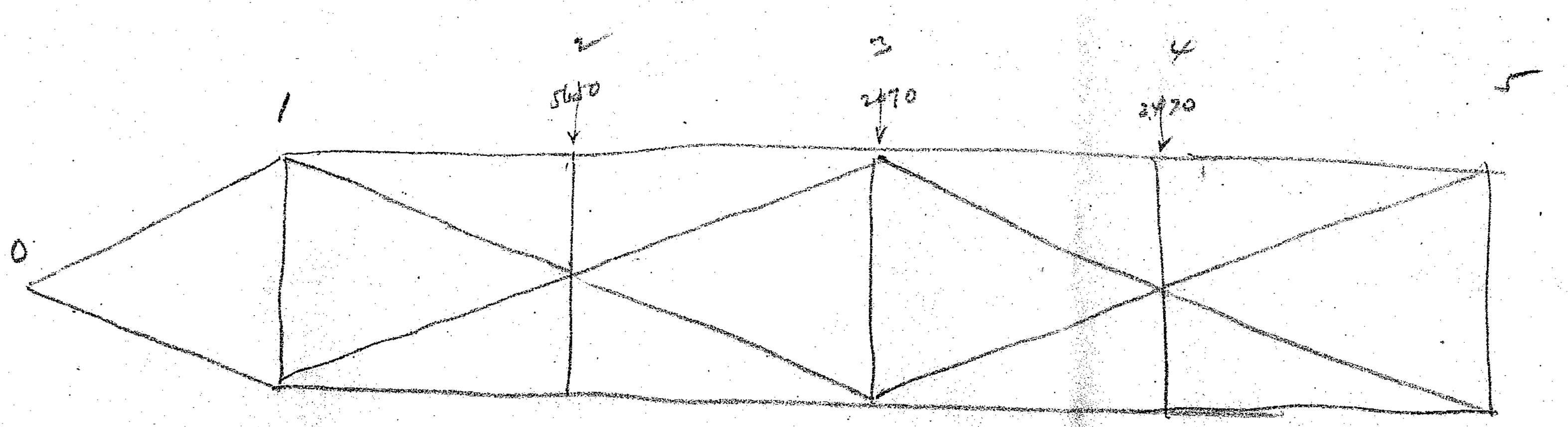

Live load shear

Shear at 2nd panel

5650 × 19.6 = 117,000
2070 × 17.15 = 26,500
3470 × 14.7 = 51,000
194,500 ÷ 24.5 = 7,950

Shear at 3rd panel

5650 × 17.15 = 97,000
2070 × 14.7 = 30,500
3470 × 12.25 = 42,500
170,000 ÷ 24.5 = 6950

Dead load shear

2nd Panel 416 kg/m

416 × 7.35 = 3,050

3rd Panel

416 × 4.9 = 2,040

Wind load shear 105 kg/m

2nd panel

105 × 7.35 = 772

3rd

105 × 4.9 = 515

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CALCULATIONS FOR

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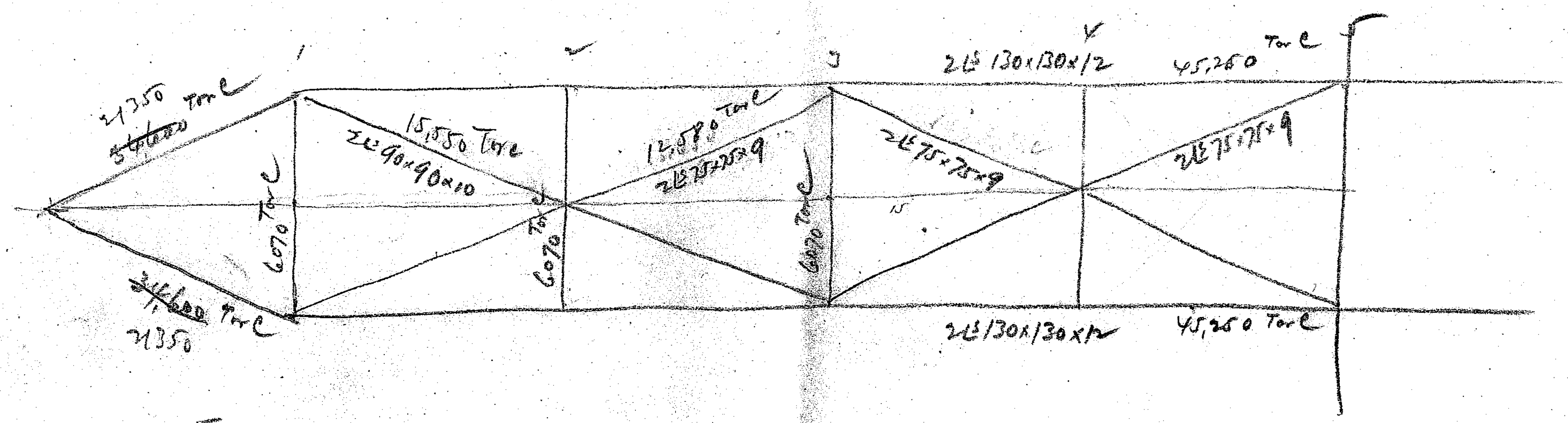


$$\frac{21{,}650^{T}}{1{,}200} = 18.5$$

3−4

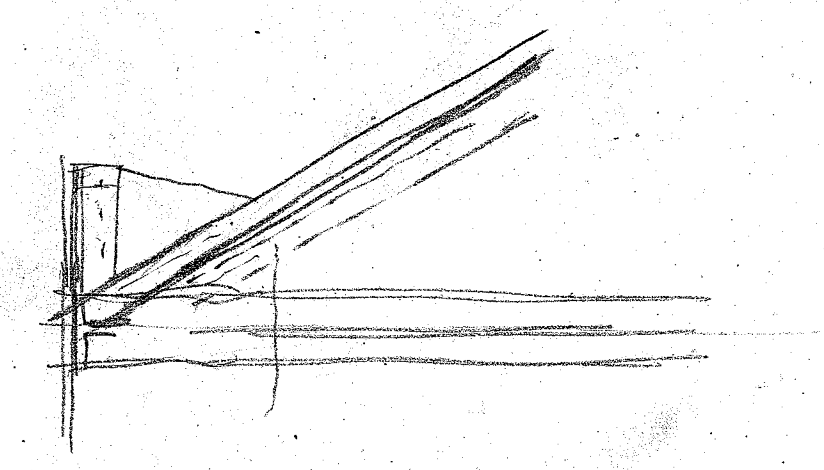
2L 75×75×9 = 25.38

$$l/r = \frac{250}{2.31} = 108 < 120$$

$$\frac{21000000}{3}\left(\frac{r}{l}\right)^2 = \frac{21{,}000{,}000}{3} \times \left(\frac{2.31}{250}\right)^2 = 598.$$

$$\frac{12{,}700}{598} = 21.2$$

$\frac{700}{2100}$ .00925 .0000855



1−2

2L 90×90×10 = 34

$$l/r = \frac{250}{2.72} = 92.0 < 120$$

$$\frac{21{,}000{,}000}{3} \times \left(\frac{2.72}{250}\right)^2 = 830$$

$$\frac{21{,}650}{830} = 26.1$$

$\overline{15{,}550}$ 26.

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（独立行政法人 土木研究所蔵）

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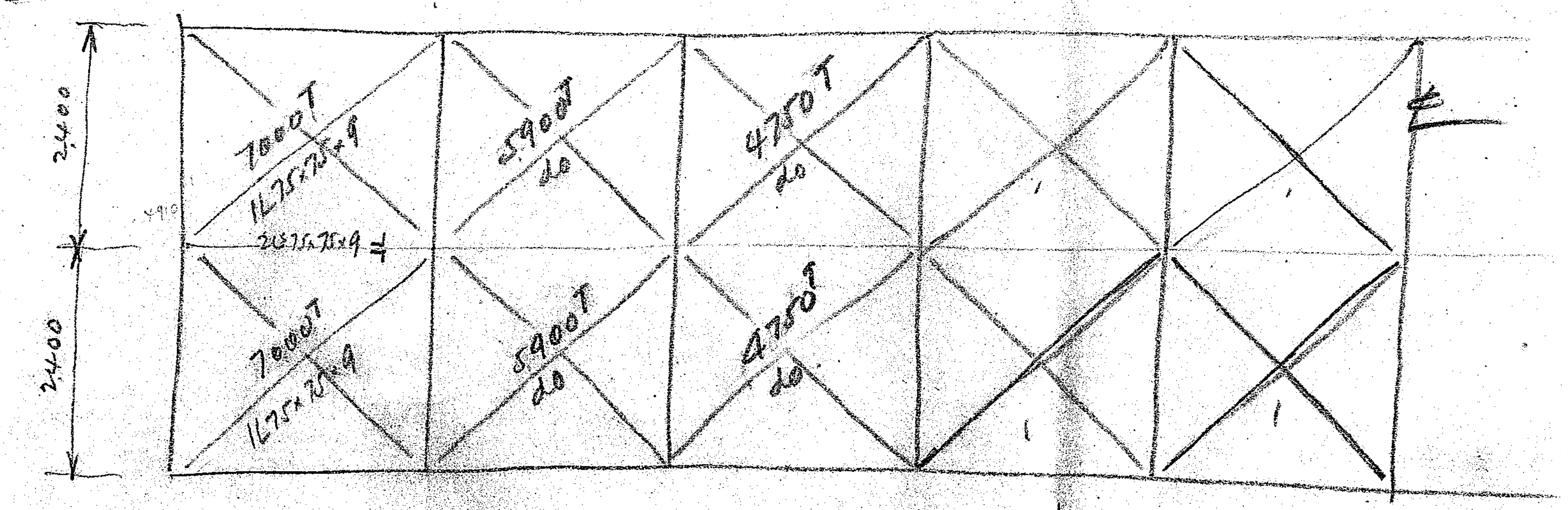
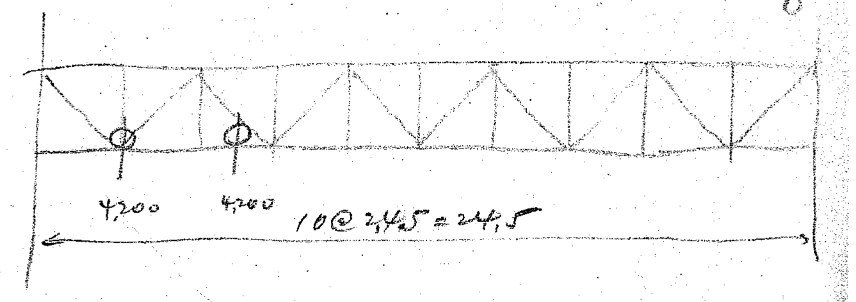
Lower lateral bracing. @ 2.45





電車ノ通過セザル場合

300 × 1.5 = 450 kg per meter.

Maximum mom. $\frac{1}{8} \times 450 \times 24.5^2 = 33,800$

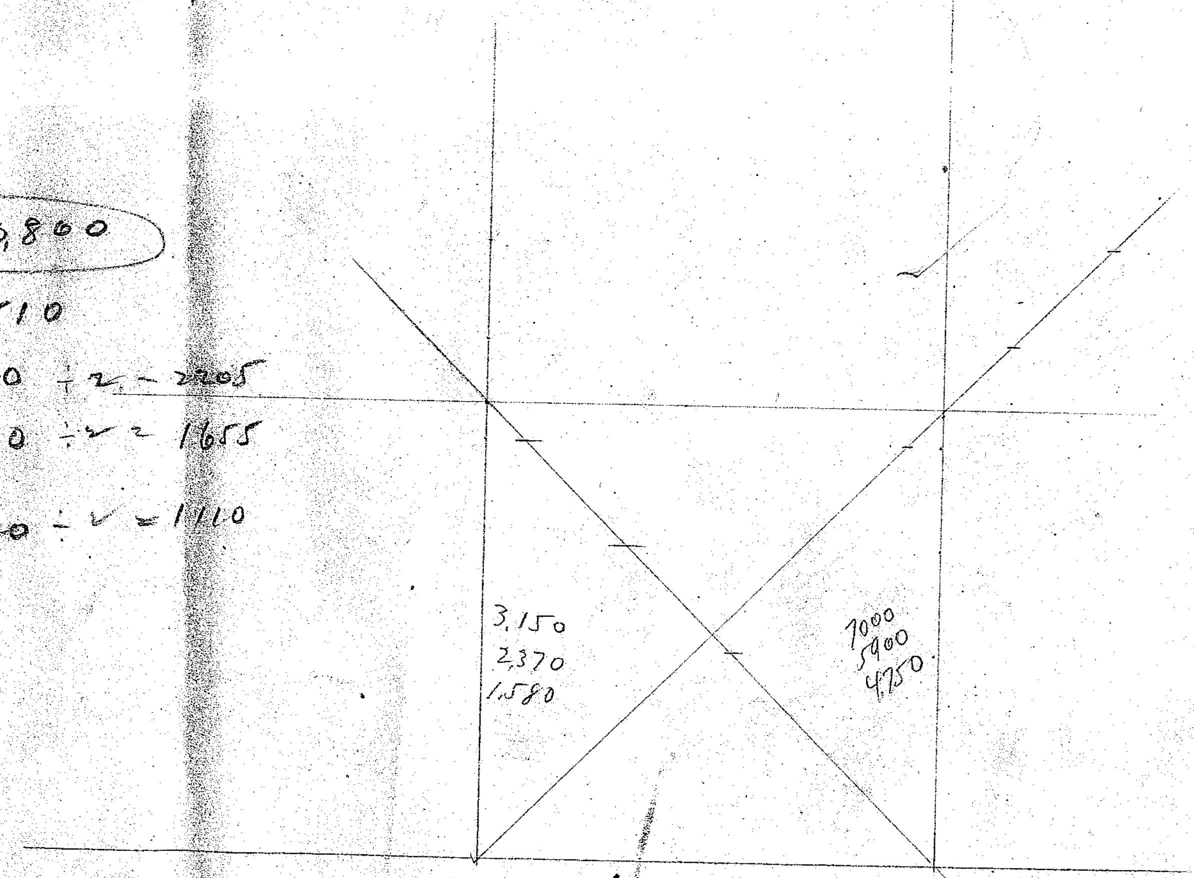
" Shear $\frac{1}{2} \times 450 \times 24.5 = 5,510$

Shear of end Panel $450 \times 9.8 = 4,410 \div 2 = 2205$

" 2nd $450 \times 7.35 = 3,310 \div 2 = 1655$

3rd $450 \times 4.90 = 2,220 \div 2 = 1110$

7000 e



130×130×9 $\frac{280}{2.42} = 116 < 120$

280

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CALCULATIONS FOR

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電車1通過せし場合

$200 \times 1.5 = 300\ \mathrm{kg/m}$

Shear of end Panel $300 \times 9.8 = 2{,}940$
$300 \times 7.35 = 2{,}220$
$300 \times 4.9 = 1{,}470$

$7.00 \times 600 = 4{,}200$

Shear of end panel

$4{,}200 \times 22.05 = 92{,}700$
$4.200 \times 18.05 = \underline{75{,}800}$
$168{,}500 \div 24.5 = 6{,}880$

" of 2nd panel

$4{,}200 \times 19.5 = 8{,}200$
$4{,}200 \times 15.6 = \underline{6{,}560}$
$14{,}760 \div 24.5 = 6{,}020$

" 3rd

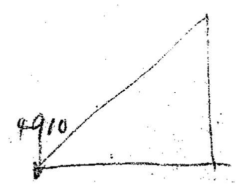
$4{,}200 \times 17.15 = 7{,}200$
$4{,}200 \times 13.15 = \underline{5{,}530}$
$12{,}730 \div 24.5 = 5{,}200$

Summary of Shear

	end	2nd	3rd
	2,940	2,220	1,470
	6,880	6,020	5,200
	9,820	8,240	6,670
½	4,910	4,120	3,335

Use 1L75×75×9 $= 12.69 - 1.98 = 10.71$

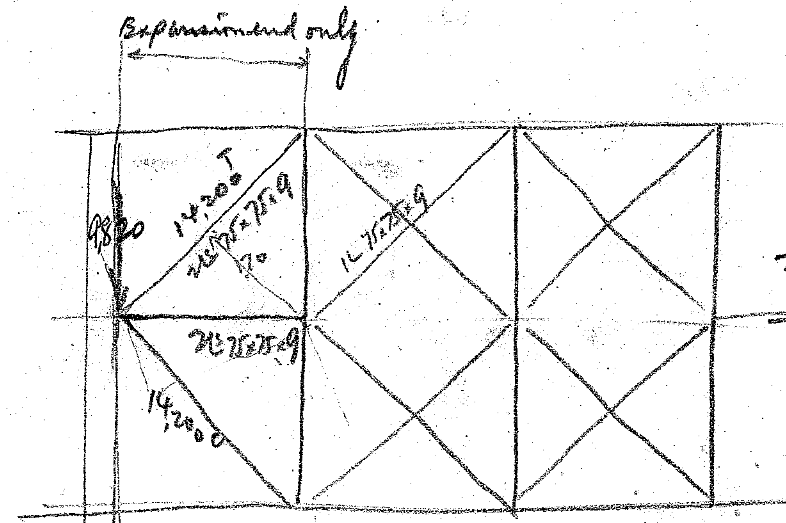
$\frac{7000}{1200} = 5.84$



$\frac{4910 \times 24.5}{24.0} = 5020$

2L75×75×9 $\frac{24}{2.8} =$

3



$\frac{9820 \times 24.5}{17} = 14{,}200$

2L75×75×9 $= 25.38$

$\frac{300}{2.8} = 107 < 120$

$\frac{21{,}000{,}000}{3} \times \left(\frac{2.8}{300}\right)^2 = 610$

$\frac{14200}{610} = 23.3$

700 .0000873

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CALCULATIONS FOR

for Column

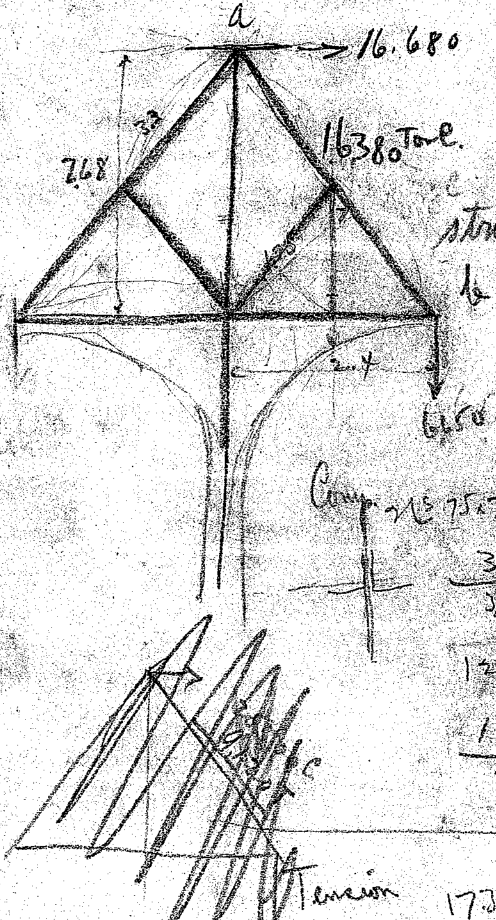
for (upper) horzontal truss

Maximum end shear for live load

$2070 \times 22.05 = 45{,}700$
$3470 \times 19.60 = \underline{68{,}000}$
$113{,}700 \div 24.5 = 4640$
$\underline{+5650}$
$10{,}290$

Maximum end shear for horzontal truss

Live load	10,290
Dead load	5,100
Wind load	1,290
	16,680



½ Dead load	550
live load	6,100
	6,650

stress in a-b $= \dfrac{16.680 \times 2.68}{1.75} = 25{,}500$

$\dfrac{6650 \times 2.4}{1.75} = 9{,}120$

$25{,}500 - 9{,}120 = 16{,}380$ Ton C.

Comp. 2L 75×75×9 = 25.38

$\dfrac{330}{3.58} = 92.3 < 100$

$1200 - 5\dfrac{l}{r} = 739$

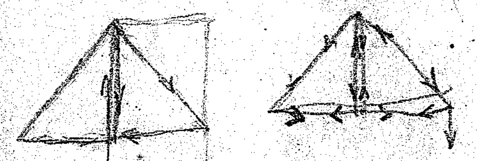
$\dfrac{17310}{739} = 23.4$

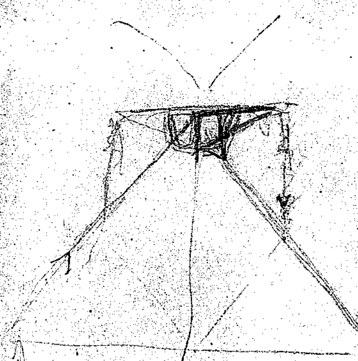
Tension $\dfrac{17310}{1200} = 14.4$

Wieght of upper horzontal truss and top chord of Vertical truss $= 300$ kg/m

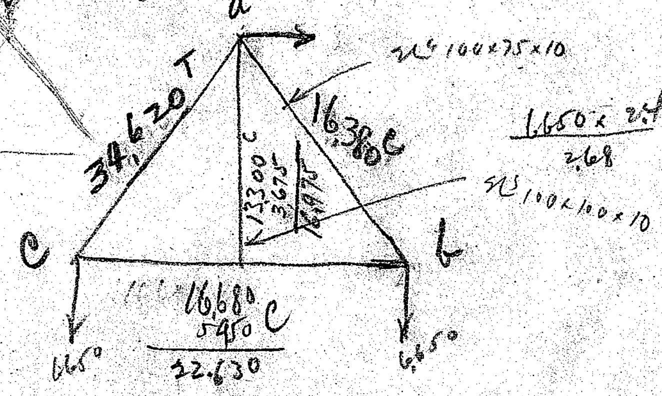
$300 \times 24.5 = 7{,}350$

Reaction $\dfrac{7350}{2} = 3675$





on column.



$\dfrac{6650 \times 2.4}{2.68} = 5{,}950$

a-c member

$\dfrac{34620\,T}{1200} = 28.8\ cm^2$ net

a-b member 2L 100×75×10 = 33.0

$\dfrac{16380}{715} = 23.\ cm^2$