**JIUN MASUDA**
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

三井鑛山會社

神通川橋梁設計々算書

及材料表

増田橋梁事務所

**JIUN MASUDA**
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CALCULATIONS FOR

Design of Jinzu Bashi

This project crossing over Jinzu Gawa is planned by Mitsui Mining Co Limited to facilitate the transportation of mining product ~~across the river~~ of Kamioka-Mine Toyama-Ken across the river and also general traffic on both banks of river. The General plan is given by "Mining Co", however the span length of bridge is changed somewhat from economic stand point and general layout. Center span is made of 68.6 meters instead of original span of 66.0 meter giving ample span during flood over the river channel. The span is composed of a suspended span at center carried by cantilever arms from anchor spans of both sides. The anchor spans are ~~cas~~ erected on steel tower bents and side spans on both ends are rest on tower bent and masonry pier; the span is made of simple span on account of ease of erection.

Loading - Coupled 8 ton electric Engines followed by freight car of 2 tons per lin. meter; This track will occupy 2.5 meters at center of bridge, gage being 0.763 meter wide. On both sides of track provide 2 - 1.5 meter footwalk assumed loading 400 kg per Square meter.

Impact allowance $\quad I = L\left(\frac{60}{90+l}\right)$

where $L$ = Live Load stress of member
$l$ = loaded length in meter.

Wind load On loaded deck 600 kg per meter
Unloaded chord 300 " "

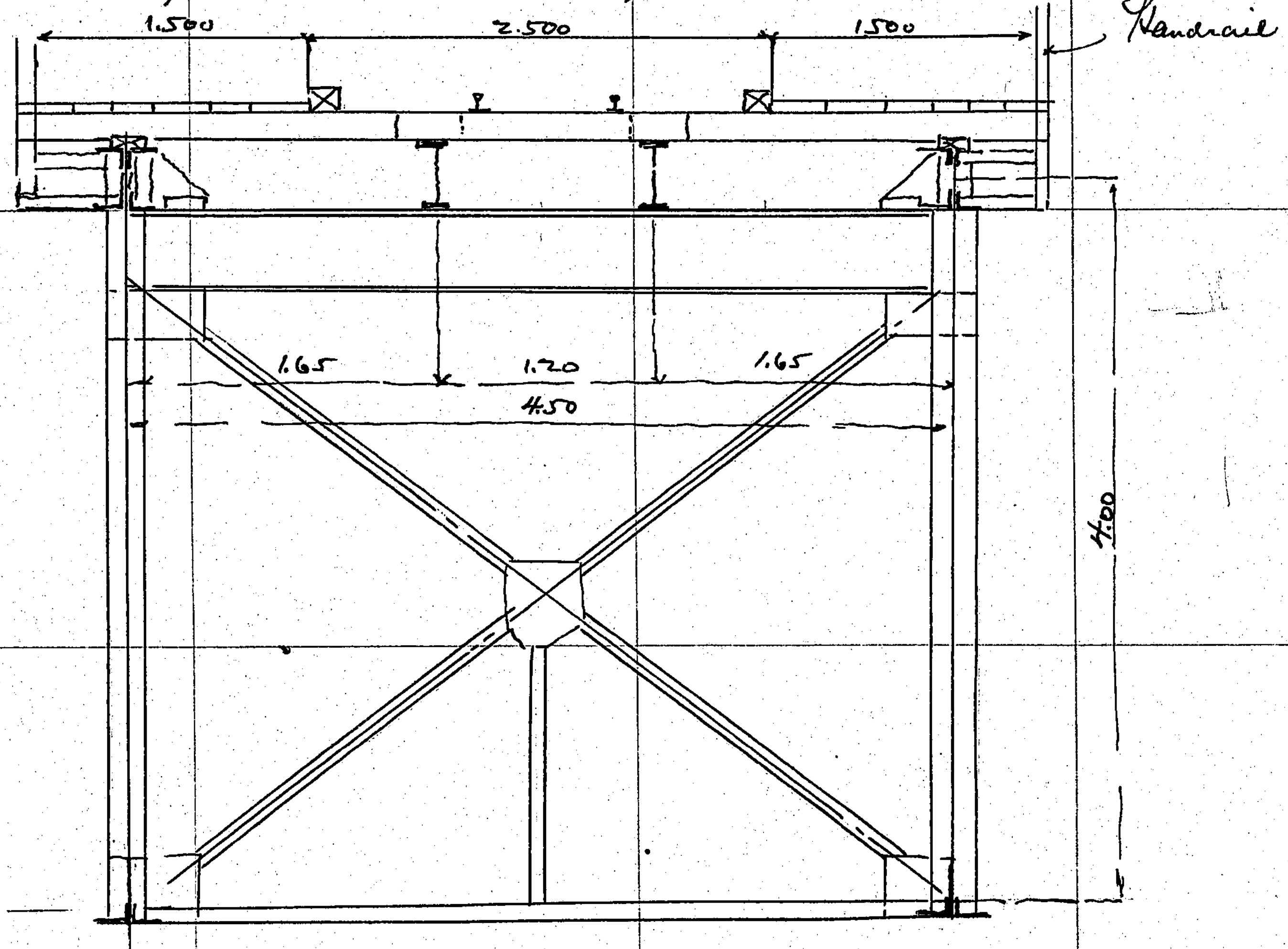
For tower 250 kg/m² for exposed area no live load on deck
150 " " for exposed area in case of with moving live load.

Height of moving live load 1. meter point of application 3/4 meter above base of rail.
Exposed area to be counted increased 50% for leeward truss.

Temperature change 80°F

Allowable strength of material & other specifications - Imperial Railway Standard.

General cross section of bridge is given by sketch below.



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CALCULATIONS FOR

Design of Jinzu-Bashi

Foot walk planking  span length $\frac{4.9}{6} = .815$  call this .82 meter

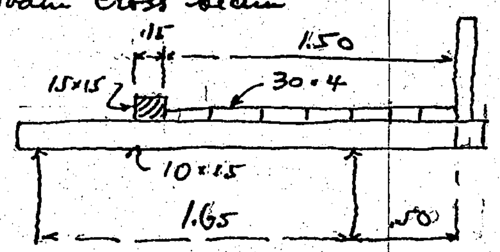
Dead Load  3cm planking @ 8.0 = 24
live load 400
424 per square meter

moment = $\frac{1}{10} \times 424 \times .82^2 = 28.5$ kgmeter

Unit stress assumed 800 # per sq inch = 56 kg/cm²

Effective depth = $\sqrt{\frac{6 \times 2850}{100 \times 56}} = 1.75$ cm  use 4cm planking

Wooden cross beam




Dead Load planking 32 kg/m² × .82 = 26.2
cross beam say 10×15 cm 12.0
Live Load 400 × .82 = 330.0
368.2
call this 370 kg.

Cantilever moment = $\frac{1}{2} \times 370 \times 0.5^2 = 46.3$ kgm

Fibre stress = $\frac{4630 \times 6}{10 \times 15^2} = 12.4$ kg/cm²

Full load on 1.65 span assumed  moment = $\frac{1}{8} \times 370 \times 1.65^2 = 125.6$ kgm

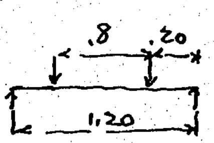
Fibre stress = $\frac{12560 \times 6}{10 \times 15^2} = 33.50$ kg/cm²

Design of Railway Track
approximate weight of Deck construction.

| | | | |
|---|---|---|---|
| 2 Rails | 30# | 2 × 15 = | 30 |
| foot planking | 60cm × 4cm | | 19 |
| timber guard | 2 - 15×15 | | 36 |
| sleeper | $15 \times 15 \times \frac{1.50}{.82} \times 800$ = | | 33 |
| Rail accessories - | | | 10 |
| | | | 128 kg per lin. meter |

By specification weight of Deck construction assumed 200 kg per lin. meter.

Strength of Cross Sleeper.
Concentration from traction Engine. 2,000
impact say ⅔ 1330
3330 kg.




Bending moment = 3330 × .20 = 666. kgmeter
For one tie 333.0 kg meter

Fibre stress = $\frac{\cancel{66600}\ 333 \times 6}{20 \times 15^2} = 44.4$ kg/cm²

Hand rail with connection assumed 50 kg per lin. meter. assumed.

Weight of Foot walk.

| | | | |
|---|---|---|---|
| | 4 × 1.50 @ 8.0 = | 48.0 | |
| | 15×15 cm | 16.0 | |
| Cross beam | $\frac{10 \times 15}{100 \times 100}$ cm × 2.50 m × $\frac{800}{.82}$ | 36.5 | |
| | | 100.5 | call this 100 kg. |

2 @ 100 = 200 kg per lin. meter.

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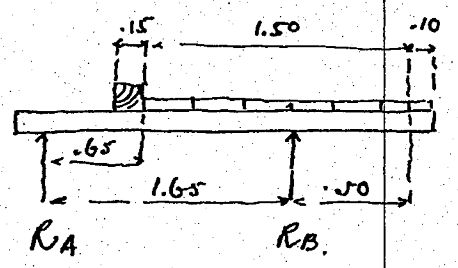
CALCULATIONS FOR

*Design of Jinzū-Bashi.*

Reaction on stringer due to footwalk.




$R_B$ 48.0 × $\frac{1.40}{1.65}$ = 41.0

$R_A$ 48.0 − 41.0 = 7.0

guard $R_B$ 16.0 × $\frac{.575}{1.65}$ = 5.6

$R_A$ 16.0 − 5.6 = 10.4

Cross beam $R_B$ 36.5 × $\frac{1.05}{1.65}$ = 23.2

$R_A$. 36.5 − 23.2 = 13.3

Summary for reactions

| | $R_A$ | $R_B$ |
|---|---|---|
| | 7.0 | 41.0 |
| | 10.4 | 5.6 |
| | 13.3 | 23.2 |
| | 30.7 kg. | 69.8 |
| say | 30.0 | 70.0 |

Live Load reactions 400 × 1.50 = 600 kg.

$R_B$ 600 × $\frac{1.40}{1.65}$ = 510 kg.

$R_A$ 600 − 510 = 90 kg.

Design of Stringer span length 4.90 meters.

Dead Load Deck construction 100
from footwalk 30

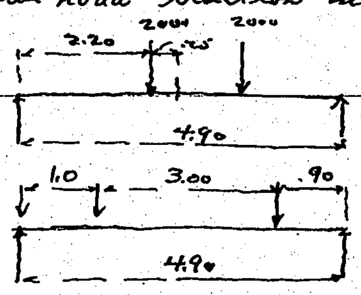
130 kg

Live Load on footwalk 90 kg.
stringer assumed 55 kg.

Dead Load moment $\frac{1}{8}$ × 185 × 4.9² = 555.0 kg meter  End shear = $\frac{185}{2}$ × 4.9 = 453

Live Load Uniform $\frac{1}{8}$ × 90 × 4.9² = 270.0 " "  End shear = $\frac{90}{2}$ × 4.9 = 220

Live Load traction Engine.




moment = 4000 × $\frac{2.2^2}{4.9}$ = 3950 kgm

impact $\frac{60}{90+4.9}$ = 63.3% 2500 "

End shear 2000 × $\frac{4.80}{4.90}$ = 1960
2000
3960 kgm

impact 2510 " "

Summary for moment and shears

| | moment | shear |
|---|---|---|
| Dead Load | 555 | 453 |
| Live Load Unif | 270 | 220 |
| Live Load traction Engine | 3950 | 3960 |
| impact | 2500 | 2510 |
| | 7275 kgm | 7143 kg |

use 350 × 15 @ 58.5 kg per meter Zm = 870.6

unit stress = $\frac{727500}{870.6}$ = 835 kg/cm²

allowable unit stress 1200 kg/cm² net for tension side

1100 − 15 × $\frac{245}{15}$ = 855 kg/cm² for compression flange.
1150  50
905

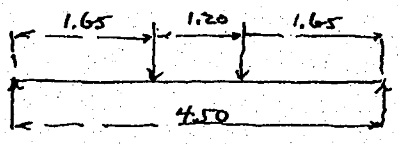
CALCULATIONS FOR

Design of Jinzu-Bashi

Design of Cross Beam — span length 4.50 meters. Spacing 4.90 meters

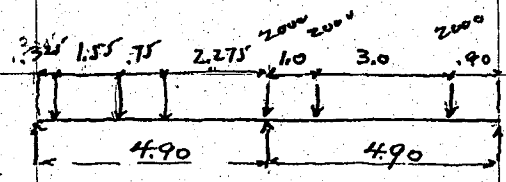
Dead Load — Dead load flooring assumed 200 kg instead of 185 kg

Dead load concentration 200 × 4.90 = 980 kg.




Live load Uniform concentration 90 × 4.90 = 440 kg.

Live load traction Engine. loaded cars $1150 \times \frac{4.825}{4.900} = 1135$




traction Engine 3960

5095 kg

impact allowance $\frac{60}{90+9.8} = 60\%$.

Dead Load stress beam
$m = 980 \times 1.65 = 1620$ kgm
$m = \frac{1}{8} \times 100 \times 4.5^2 = 253$
1873 kgm

End shear = 980
225
1205 kg.

Live Load stress Unif. $m = 440 \times 1.65 = 725$ kgm
End shear 440 kg.

Live Load stress Engine. $m = 5095 \times 1.65 = 8400$ kgm
impact 60% 5040 "
13440

End shear 5095 kg
impact 60% 3060 "
8155

Summary for moments and shears

| | moment | shear |
|---|---|---|
| Dead Load | 1873 | 1205 |
| Live Load Unif. | 725 | 440 |
| Live load Engine. | 13440 | 8155 |
| | 16038 kgm | 9800 kg. |

Try ~~350 × 150~~ 450 × 175 @ ~~58.5~~ 91.7 kg/m Sm = 1743.

Unit stress = $\frac{1603800}{1743} = 915$ kg/cm²

allowable unit stress = ~~1100~~ 1150 $- 15 \times \frac{165}{175} = 959$ kg/cm²

approximate weight of stringers
2 @ 58.5 × 4.90 = 575
2 Pls. 30 × 30 × .9 13
strut 1L 75 × 75 × 9 @ 9.96 = 10
598 call this 600 kg per panel

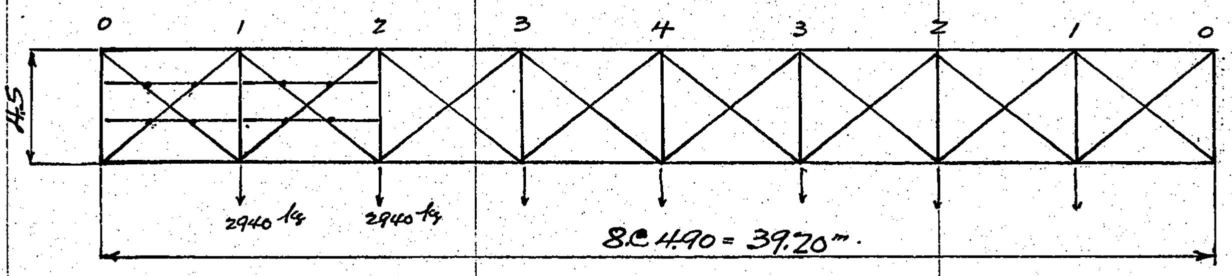
approximate weight of cross beam 91.7 × 4.50 = 413 call this 420 " pieces

Dead Load Deck

track construction 200
Sidewalks 2 @ 100 = 200
Handrails 2 @ 50 = 100
500 kg.

Design of Jinzu Bashi

Design of Top Lateral Bracings.
For Suspended span and simple span.




Diagonal Length
$4.50^2 = 20.25$
$4.90^2 = 24.01$
$44.26$
$\sqrt{44.26} = 6.653$ meters.
Coefficient $= \frac{6.653}{4.50} = 1.478$

Wind Load assumed 600 kg per lin meter
Panel load 4.90 @ 600 = 2940 kg  Reaction 3.5 @ 2940 = 10,300 kg

Diagonal Stresses.

| Panel | Shear | Coef. | Stress | Net section required | 19φ rivet no reqd. | | |
|---|---|---|---|---|---|---|---|
| 0-1 | 10,300 × 1.478 = | | 15220 kg T | 12.68 cm² | 7.15 | use 8 | 1L 125×75×9 = 17.19 - 1.98 = 15.21 cm² net. |
| 1-2 | 7360 | | 10880 | 9.07 | 5.12 | 6 | 1L do |
| 2-3 | 4420 | | 6540 | 5.45 | 3.08 | 4 | 1L 75×75×9 = 12.69 - 1.98 = 10.71 " |
| 3-4 | 1480 | | 2190 | 1.83 | 1.03 | 4 | 1L do |

moving load 变更

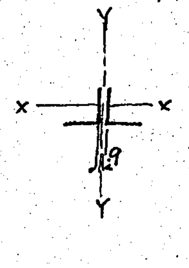
Bottom Lateral Bracings for Suspended spans and simple spans.
Wind Load assumed 300 kg per lin meter.
panel load - 4.90 × 300 = 1470 kg
Stresses of all diagonal members same as ½ of those for upper lateral bracing.
Diagonal stresses

| Panel | Stress | net section required | 19φ rivet no reqd. | | | |
|---|---|---|---|---|---|---|
| 0-1 | 7620 kg T | 6.34 cm² | 3.58 | use 4 | 1L { 125×75×9 / 75×75×9 | continuous member / broken " |
| 1-2 | 5440 | 4.54 | 2.56 | 3 | do | |
| 2-3 | 3270 | 2.73 | 1.84 | 3 | do | |
| 3-4 | 1,095 | .92 | .52 | 3 | do | |

Approximate weight of top lateral bracings.
8Ls 125 × 75 × 9 @ 13.50 × 6.65 = 718
8Ls 75 × 75 × 9 @ 9.96 × 6.65 = 530
8 center connections say @ 25.0 = 200
16 end connections say @ 20.0 = 320
1768 kg ÷ 39.20 = 45.0 kg/lin meter.

Approximate weight of bottom lateral bracings. same as for top lateral = 45.0 "
approximate weight of sway bracings.
2Ls 75×75×9 @ 9.96 × 5.20 = 104
4 corner connections @ 15.0 = 60
bottom strut.
2Ls 90×90×10 @ 13.30 × 4.50 = 120
1L 75×75×9 @ 9.96 × 1.80 = 18
center conn. say 1 @ 16.0 = 16
miscellaneous details say = 25
343 kg ÷ 4.90 = 70.0 kg/lin meter
Summary = ~~3880 kg~~ 160.0 "

Radius of gyration of strut.
2Ls 90×90×10 = 34.0 cm²
Y-Y axis $r_y$ = 4.05 cm
X-X " $r_x$ = 2.71




$l/r_x$ = 450/~~4.05~~ = 111. ok.
$l/r_y$ = 225/2.71 = 83 "

Total weight of Lateral & sway bracings
Top Lateral br. 1768
Bottom " " 1768
Sway bracings 9 @ 343 = 3094
6630 kg
6630 ÷ 39.2 = 169 kg/lin m.

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CALCULATIONS FOR

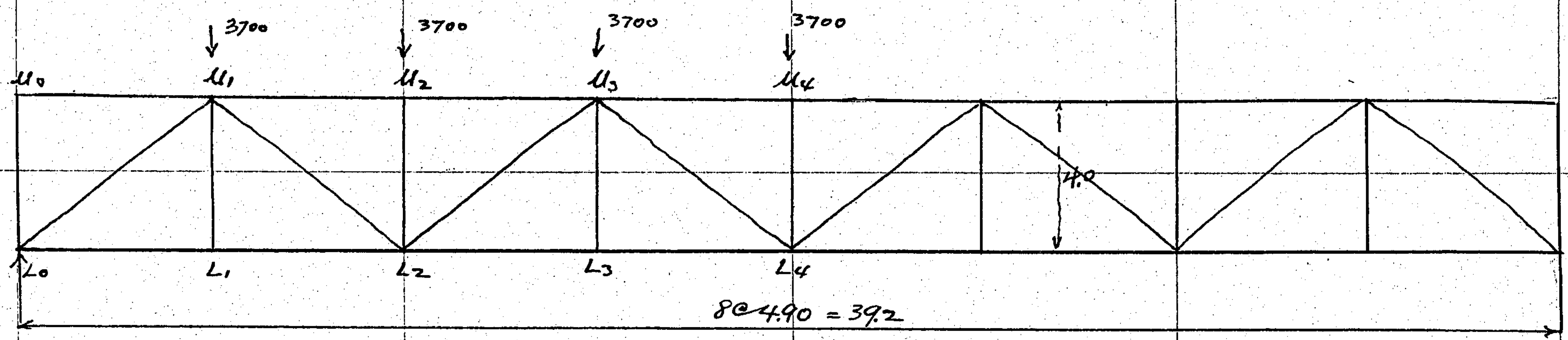
Design of Jinzu - Bashi

Design of truss — span length 39.2 meters — 8@4.90
Height of truss 4.0 meters

Dead Load — Deck construction complete — 500

structural steel
stringers $600 \div 49$ = 123
Floor Beam $420 \div 49$ = 86
Bracing complete ~~160~~ 160
trusses assumed ~~640~~ 640
1009
$1509 \div 2 = 755$ kg per lin. meter.

Panel Concentration $755 \times 4.90 = 3700$ kg.




Diagonal length
$4.0^2 = 16.0$
$4.9^2 = \underline{24.01}$
$40.01 - \underline{6.33}$.

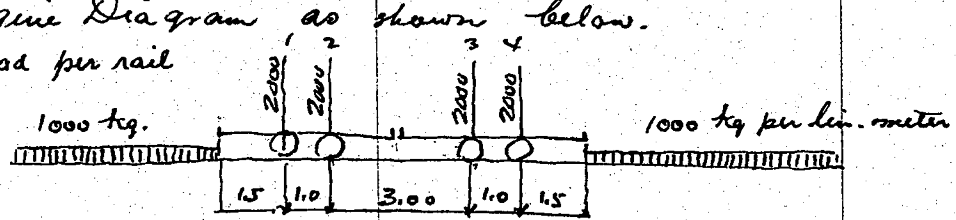
$\sec\theta = \frac{6.33}{4.00} = 1.583$
$\tan\theta = \frac{4.90}{4.00} = 1.227$

Chord stresses

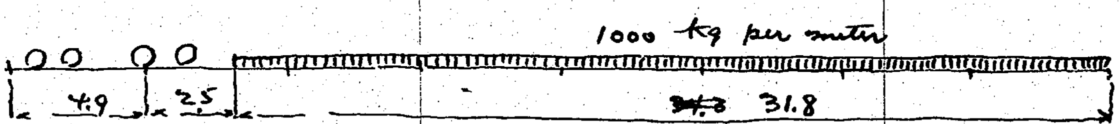
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| L0-L2 | 3.50 | × 3700 | × 1.227 | = | 15900 | kg | T |
| U1-U3 | 6.00 | | | = | 27200 | " | C |
| L2-L4 | 7.50 | | | = | 34000 | " | T |
| U3-U4 | 8.00 | | | = | 36400 | " | C |
| web member | | | | | | | |
| L0-U1 | 3.50 | × 3700 | × 1.583 | = | 20500 | " | C |
| U1-L2 | 2.50 | | | = | 14700 | " | T |
| L2-U3 | 1.50 | | | = | 8800 | " | C |
| U3-L4 | 0.50 | | | = | 2940 | " | T |

Live Load Stresses.
Chord stresses.

Engine Diagram as shown below.
Load per rail




moment at U1
wheel no 3 at L1




Reaction = $8000 \times \frac{35.8}{39.2} = 7320$
12900
$31800 \times \frac{31.8}{39.2 \times 2} = \cancel{13740}$
~~21060~~ ÷
20220

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CALCULATIONS FOR

Design of Jinzu-Bashi

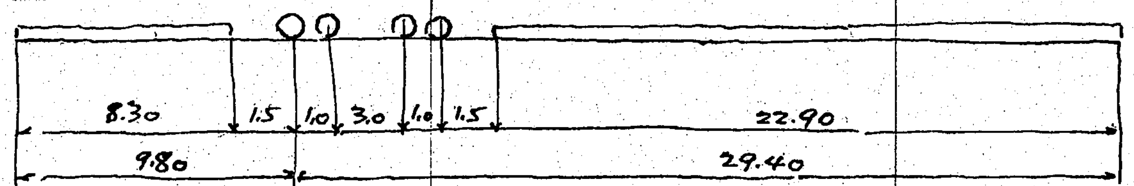
moment at $U_1$ 20220 ~~21060~~ × 4.90 = 99200 ~~103.000~~
4000 × 3.50 = −14.000
85200 ~~89.000~~ kgm

stress in $L_0$-$L_2$ 85200 ~~89000~~ ÷ 4.0 = 21300 ~~22250~~ kg.

try the following method of calculation considering traction engine as uniform load of 1000 kg per linear meter. Panel Concentration 4.900 kg.

Stress in $L_0$-$L_2$ 3.500 × 4900 × 1.227 = 21200 kg.

Stress in $U_1$-$U_3$.



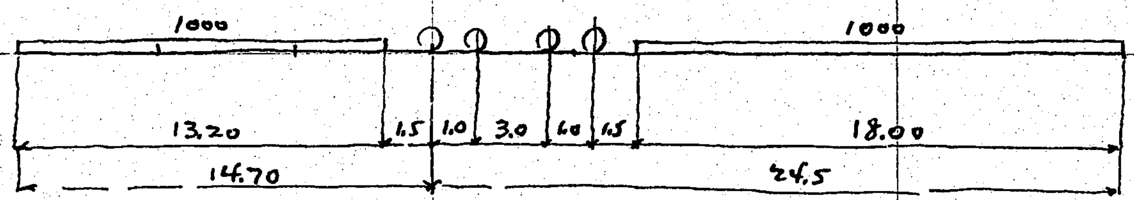

Reaction 8000 × 26.90/39.2 = 5480
22900 × 11.45/39.20 = 6680
8300 × 34.05/39.20 = 7200
19360

moment 19360 × 9.80 = 190.000
8300 × 5.65 = −47.000
143 000

stress $U_1$-$U_3$ 143.000 ÷ 4 = 35800 kg.

as uniform load
stress in $U_1$-$U_3$ 6.00 × 4900 × 1.227 = 36000 kg.

Stress in $L_2$-$L_4$




Reaction 8000 × 22.0/39.20 = 4490
18000 × 9.0/39.20 = 4130
13200 × 32.60/39.20 = 10950
19570

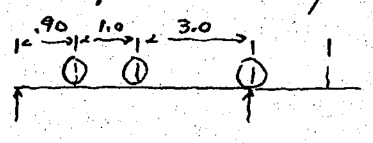
moment 19570 × 14.70 = 288.000
13200 × 8.10 = 106800
181200

stress $L_2$-$L_4$ 181200 ÷ 4 = 45400

As uniform load 7.50 × 4900 × 1.227 = 45100 kg.

Stress in $U_3$-$U_4$ 8.00 × 4900 × 1.227 = 48000 kg.

Diagonal Stress $L_0$-$U_1$
Loading same as for max moment at $U_1$



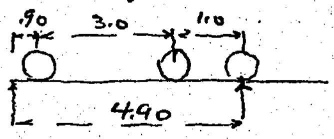

2000 × 2.80/4.90 = 1145
2000
3145 × 34.80 = 108,000
2000 × 33.30 = 66600
174600 ÷ 39.20 = 4460

31800 × 31.80/(2 × 39.20) = 12900 ~~13740~~
16360 ~~18200~~ kg.

stress = 16360 ~~18200~~ × 1.583 = 25850 ~~28800~~ kg.

as uniform load 3.5 × 4900 × 1.583 = 27200 "

Try the following loading




2000 × 4.80/4.90 = 1956
2000
3956 × 34.30/39.20 = 3460

32800 × 32.8/(2 × 39.20) = 13750
16210

Design of Jinzu- Bashi

Use panel concentrated load from uniform load which will give max stress for diagonal stresses

| | | | | | impact | stress |
|---|---|---|---|---|---|---|
| $U_1-L_2$ | 21/8 | × 4900 × 1.583 | = | 20400 kg. | 50.2% | 10200 kg |
| $L_2-U_3$ | 15/8 | " " | = | 14600 " | 52.4 | 7650 " |
| $U_3-L_4$ | 10/8 | " " | = | 9700 " | 54.7 | 5300 " |
| $L_4-U_3'$ | 6/8 | " " | = | 5800 " | 57.3 | 3330 " |
| $U_3'-L_2'$ | 3/8 | " " | = | 2900 " | 60.0 | 1740 " |

Impact for full load = 46.4%

For stress diagram see page 9.

Section of truss members.

Top chord. 4Ls 150×90×9 = 83.38 × $5.46^2$ + 1872 = 4362
1Pl. 300×10 = 30.00
113.38

$r = \sqrt{\frac{4362}{113.38}} = 6.20$

Length of member 490

allowable unit stress = $1200 - 5 \times \frac{490}{6.2} = 805$ kg/cm²

Length of member $\frac{4.90}{2} = 2.45$ m using strut at center of panel from stringer in deck

$l/r = \frac{245}{6.2} = 39.6$ allowable unit stress 1000 kg/cm²

Top chord $U_1-U_3$ 79600 ÷ 805 = 99.0 □cm reqd

Top chord $U_3-U_4$ 106700 ÷ 1000 = 106.7 □cm reqd

Diagonal $L_0-U_1$ 4Ls 150×90×9 = 83.38

A, B, B, 190, A

AA axis 1.0 cm apart $r = 7.22$ $l/r = \frac{633}{7.22} = 88.$

BB axis 83.38 × $7.52^2$ + 516 = 5216

$r = \sqrt{\frac{5216}{83.38}} = 7.90$

Allowable unit stress = $1200 - 5 \times \frac{633}{7.22} = 760$ kg/cm²

Section reqd 59900 ÷ 760 = 79.0 □cm reqd

Diagonal $L_2-U_3$ Section same as for $L_0-U_1$

Section reqd 31050 ÷ 760 = 40.9 □cm

Diagonal $U_1-L_2$ 45300 ÷ 1200 = 37.8 □cm net
4Ls 90×90×10 = 68.0 − 20.0 = 48.0 net
Rivet holes 25 mm.

Diagonal $U_3-L_4$ 17940 T 7070 C
Design stress 3535 3535
21475 T 10605 C

try 2Ls 130×130×9 $r = 5.05$ $l = 633$ cm

allowable stress = $\frac{21.000.000}{3} \times \left(\frac{r}{l}\right)^2 = 4445$ kg/cm² SR 10605 ÷ 4445 = 23.9 gross

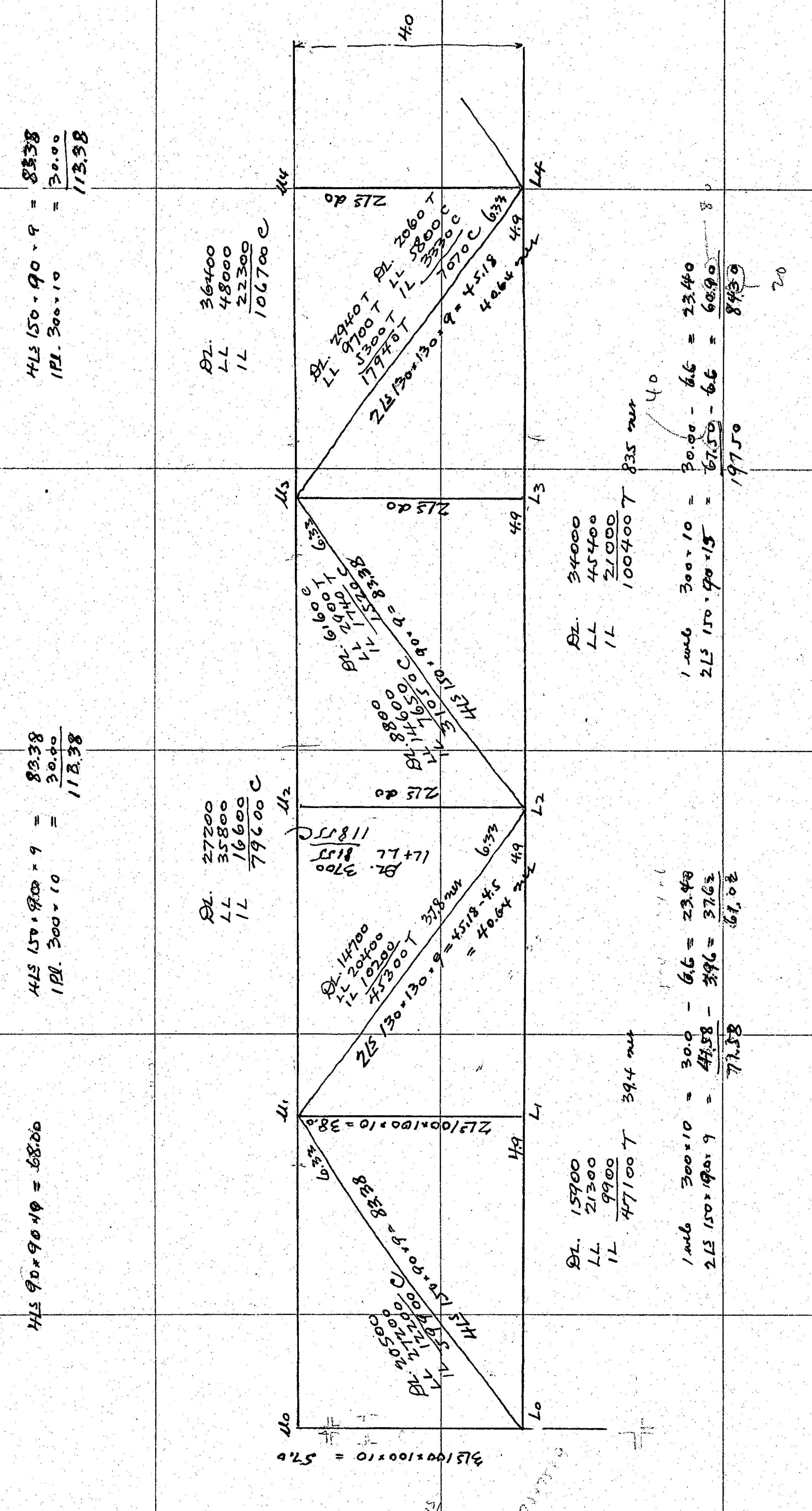
For tension SR 21475 ÷ 1200 = 17.9 net

2Ls 130×130×9 = 45.18 − 4.5 = 40.64 net

CALCULATIONS FOR

Design of Jinzu-Bashi

stresses and section of truss



CALCULATIONS FOR

Design of Jinzu Bashi.

Diagonal $U_1-L_2$ 45300 T 37.8 net

2 L$^s$ 130 × 130 × 9 = 45.18 − 4.5 = 40.64 net 22$^{mm}$ Rivet for connection only

Bottom chord $L_0-L_2$ 47100 T SR = 39.4

| | | | | | |
|---|---|---|---|---|---|
| 1 Pl | 300 × 10 = | 30.00 | − 6.60 | = 23.40 | 19$^{mm}$ Rivet |
| 2 L$^s$ | 150 × 90 × 9 = | 41.58 | − 3.96 | = 37.62 | |
| | | 71.58 | | 61.02 | |

Bottom chord $L_2-L_4$ 100400 T SR = 83.50

| | | | | |
|---|---|---|---|---|
| 1 Pl | 300 × 10 = | 30.00 | − 6.60 | = 23.40 |
| 2 L$^s$ | 150 × 90 × 15 = | 67.50 | − 6.60 | = 60.90 |
| | | 97.50 | | 84.30 |

Verticals Dead Load concentration 3700
Live Load say with impact 8155
11855 kg. C.

2 L$^s$ 100 × 100 × 10 = 38.0 r = 3.83 l = 3.45 m
allowable unit stress $1200 - 5 \times \frac{345}{3.83} = 750$

Section required 11855 ÷ 750 = 15.8 gross.

All hangers use same section as for verticals.

approximate weight of truss.

| | | | | | |
|---|---|---|---|---|---|
| $U_0-U_1$ | 68.0 | @ .785 | × 4.90 | = | 262 |
| $U_1-U_4$ | 113.38 | " | × 3.490 | = | 1310 |
| $L_0-L_2$ | 71.58 | " | × 2.490 | = | 550 |
| $L_2-L_4$ | 97.50 | " | × 2.490 | = | 750 |
| $L_0-U_1$ $U_3-L_2$ | 83.38 | " | × 2.633 | = | 825 |
| $U_1-L_2-U_3-L_4$ | 45.18 | " | × 2.633 | = | 450 |
| verticals | 38.00 | " | × 35.40 | = | 417 |
| End vert. | 57.00 | " | × 4.0 | = | 179 |
| | | | | | 4743 × 2 = 9486 |

Details assumed 30% 2840
12326

12326 ÷ 39.2 = 314

For two trusses 24650 kgr. 630 kg per lin. meter

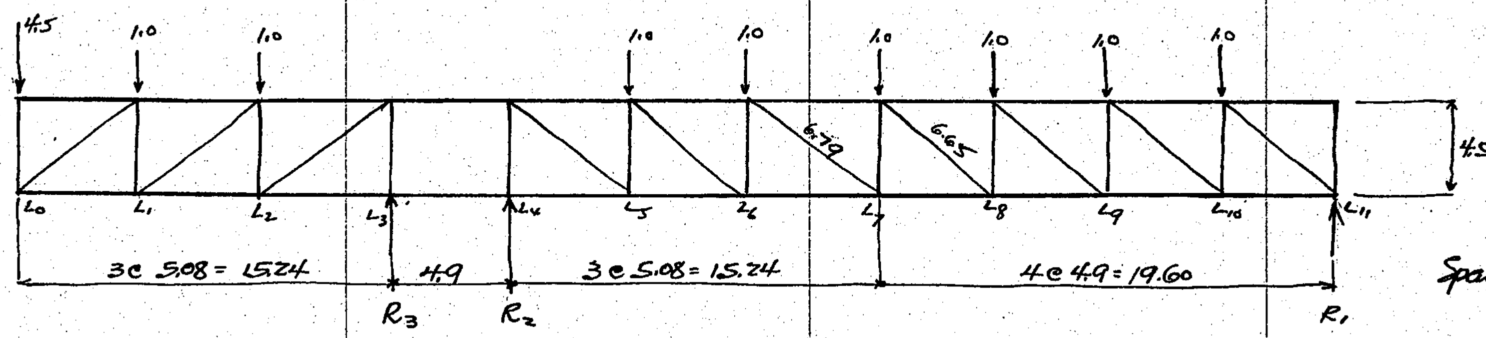
Load on shoe. DL. 755 Design shoes for 45.000 kg.
LL. 1000
IL 464
$2219 \times \frac{39.2}{2} = \cancel{87000}$ 43500 kg.

Structural steel in 39.2 meter span

| | | | |
|---|---|---|---|
| Stringers | 600 × 8 | = | 4800 |
| Floor Beam | 420 × 8 | = | 3360 |
| End F.B. | 600 - | | 600 |
| Sway & laterals | 150 × 39.2 | = | 5870 |
| Trusses | | | 24650 |
| shoe & misc. | | | 2500 |
| | | | 41780 call this 42.0 tons |

Design of Juizū Bashi

Design of Lower Lateral Bracings for Anchor span with a cantilever arm.




Span length developed.

$4.5^2 = 20.25$
$4.9^2 = 24.00$
$44.25$
$\sqrt{44.25} = 6.65$, $\frac{6.65}{4.5} = 1.478$

$4.5^2 = 20.25$
$5.08^2 = 25.80$
$46.05$
$\sqrt{46.05} = 6.79$, $\frac{6.79}{4.5} = 1.51$

Reaction due to single unit load at any panel point.

$b = 15.24^m$, $a = 34.84^m$, $b/a = 15.24/34.84 = .437$

Unit load in anchor span.

$R_1 = 1 - k$, $R_2 = k$, $R_3 = 0$.

Unit load in cantilever arm.

$R_1 = -\frac{kb}{a} = -.437k$

$R_2 = +\frac{kb}{a} = .437k$

$R_3 = 1.000$

| | Panel Point | $k$ | $1-k$ | $.437k$ | $R_1$ | $R_2$ | $R_3$ |
|---|---|---|---|---|---|---|---|
| Cantilever arm | $L_0$ | 1.000 | | .437 | − .437 | + .437 | + 1.000 |
| | $L_1$ | .667 | | .292 | − .292 | + .292 | + 1.000 |
| | $L_2$ | .333 | | .146 | − .146 | + .146 | + 1.000 |
| | $L_3$ | .000 | | .000 | .000 | .000 | + 1.000 |
| Anchor span | $L_4$ | 1.000 | .000 | | .000 | + 1.000 | 0 |
| | $L_5$ | .854 | .146 | | + .146 | + .854 | 0 |
| | $L_6$ | .708 | .292 | | + .292 | + .708 | 0 |
| | $L_7$ | .563 | .437 | | + .437 | + .563 | 0 |
| | $L_8$ | .422 | .578 | | + .578 | + .422 | 0 |
| | $L_9$ | .281 | .719 | | + .719 | + .281 | 0 |
| | $L_{10}$ | .141 | .859 | | + .859 | + .141 | 0 |
| | $L_{11}$ | .000 | 1.000 | | + 1.000 | + .000 | 0 |

Reactions for full unit loads.

| Panel Points | Loads | $R_1$ | $R_2$ | $R_3$ |
|---|---|---|---|---|
| $L_0$ | 4.50 | − 1.965 | + 1.965 | + 4.500 |
| $L_1$ | 1.0 | − .292 | + .292 | + 1.000 |
| $L_2$ | 1.0 | − .146 | + .146 | + 1.000 |
| $L_3$ | | | | |
| $L_4$ | | | | |
| $L_5$ | 1.0 | + .146 | + .854 | 0 |
| $L_6$ | 1.0 | + .292 | + .708 | 0 |
| $L_7$ | 1.0 | + .437 | + .563 | 0 |
| $L_8$ | 1.0 | + .578 | + .422 | 0 |
| $L_9$ | 1.0 | + .719 | + .281 | 0 |
| $L_{10}$ | 1.0 | + .859 | + .141 | 0 |
| $L_{11}$ | | | | |
| | | + .628 | + 5.372 | + 6.500 |

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CALCULATIONS FOR

Design of Jinzu Bashi.

Diagonal stresses. Panel load 4.90 × 300 = 1470 kg

| Panel | shear | stress unity | Panel load | stress | S.R. net | 19φ rivet no. | use | | |
|---|---|---|---|---|---|---|---|---|---|
| 0-1 | 4.500 × 1.510 | 6.79 | 1,470 kg | 9,980 kg | 8.31 cm² | 4.7 | 6 | 1L | 125×90×9 = 18.54 − 1.98 = 16.56 cm² net |
| 1-2 | 5.500 " | 8.30 | " | 12,200 | 10.15 | 5.7 | 6 | 1L | do |
| 2-3 | 6.500 " | 9.81 | " | 14,420 | 12.01 | 6.8 | 8 | 1L | do |
| 4-5 | 5.372 × 1.510 | 8.11 | " | 11,920 | 9.94 | 5.6 | 6 | 1L | do |
| 5-6 | 4.372 " | 6.60 | " | 9,700 | 8.08 | 4.6 | 6 | 1L | do |
| 6-7 | 3.372 " | 5.09 | " | 7,480 | 6.23 | 3.5 | 4 | 1L | do |
| 7-8 | 2.372 × 1.478 | 3.51 | " | 5,160 | 4.30 | 2.4 | 3 | 1L | {125×75×9 -- continuous member; 75×75×9 --- broken " |
| 8-9 | 1.372 " | 2.03 | " | 2980 | 2.48 | 1.4 | 3 | do | |
| 9-10 | 0.372 " | 0.55 | " | 810 | .67 | 0.4 | 3 | do | |
| 10-11 | −0.628 " | −0.93 | " | −1,370 | 1.14 | 0.6 | 3 | do | |

Design of Top Lateral Bracings for Anchor span with a cantilever arm.

Reactions same as for main truss, see page 11. Panel load 4.9 × 600 = 2940 kg

For full unit loads $R_1 = +0.640$

$R_2 = +5.360$

$R_3 = +6.500$

Diagonal stresses.

| Panel | shear | stress unity | Panel load | stress (Tension only) | S.R. net | 19φ rivet no | stress (Tension + Compression) | 19φ rivet no | use | |
|---|---|---|---|---|---|---|---|---|---|---|
| 0-1 | 4.500 × 1.478 | 6.65 | 2940 kg | 19,550 kg | 16.30 cm² | 9.2 | 9,780 kg | 4.6 | 6 | 2L 75×75×9 |
| 1-2 | 5.500 " | 8.13 | " | 23,900 | 19.91 | 11.3 | 11,950 | 5.7 | 6 | do |
| 2-3 | 6.500 " | 9.61 | " | 28,250 | 23.55 | 13.3 | 14,125 | 6.7 | 8 | do |
| 4-5 | 5.360 " | 7.92 | " | 23,300 | 19.41 | 11.0 | 11,650 | 5.5 | 6 | do |
| 5-6 | 4.360 " | 6.45 | " | 18,960 | 15.80 | 8.9 | 9,480 | 4.5 | 6 | do |
| 6-7 | 3.360 " | 4.97 | " | 14,610 | 12.18 | 6.9 | 7,300 | 3.5 | 4 | do |
| 7-8 | 2.360 " | 3.49 | " | 10,260 | 8.55 | 4.8 | 5,130 | 2.4 | 4 | do |
| 8-9 | 1.360 " | 2.01 | " | 5,910 | 4.93 | 2.8 | 2,960 | 1.4 | 3 | 1L 75×75×9 |
| 9-10 | 0.360 " | 0.53 | " | 1,560 | 1.30 | 0.7 | 780 | | 3 | do |
| 10-11 | −0.640 " | −0.95 | " | −2,790 | 2.33 | 1.3 | | | 3 | do |

member 2-3, make diagonal member to be able to take care of compression and tension.

Stress for one member = 28250 ÷ 2 = 14,125 kg C or T.

2L 75×75×9 = 25.38 cm² gr − 7.92 = 17.46 cm² net

least radius of gyration = 2.25 cm unsupported length 1.65 × 1.478 = 2.44 m $r/l = 2.25/244 = 1/108$

allowable unit compression = $\frac{21000000}{3} \times \left(\frac{1}{108}\right)^2 = 600$ kg/cm²

S.R. for compression member = $\frac{14125}{600}$ = 23.54 cm² gr ok

S.R. " tension " = $\frac{14125}{1200}$ = 11.78 cm² net ok

Approximate weight of Bottom Lateral Bracings.

| | | | | | |
|---|---|---|---|---|---|
| 14L | 125×90×9 | @ | 14.60 × 6.7 | = | 1,370 |
| 4L | 125×75×9 | @ | 13.50 × 6.5 | = | 351 |
| 4L | 75×75×9 | @ | 9.96 × 6.5 | = | 259 |
| Center conn. | 11 | @ | 20 | = | 220 |
| End " | 22 | @ | 20 | = | 440 |
| | | | | | 2640 kg |

2640 ÷ 53.9 = 48.6 kg/lin m.

Approximate weight of Top Lateral Bracings.

| | | | | | |
|---|---|---|---|---|---|
| 32L | 75×75×9 | @ 9.96 × | 6.5 | = | 2072 |
| 6L | " | @ " × | 6.5 | = | 388 |
| Center conn. | 11 | @ 25 | | = | 275 |
| End " | 22 | @ 25 | | = | 550 |
| | | | | | 3285 kg |

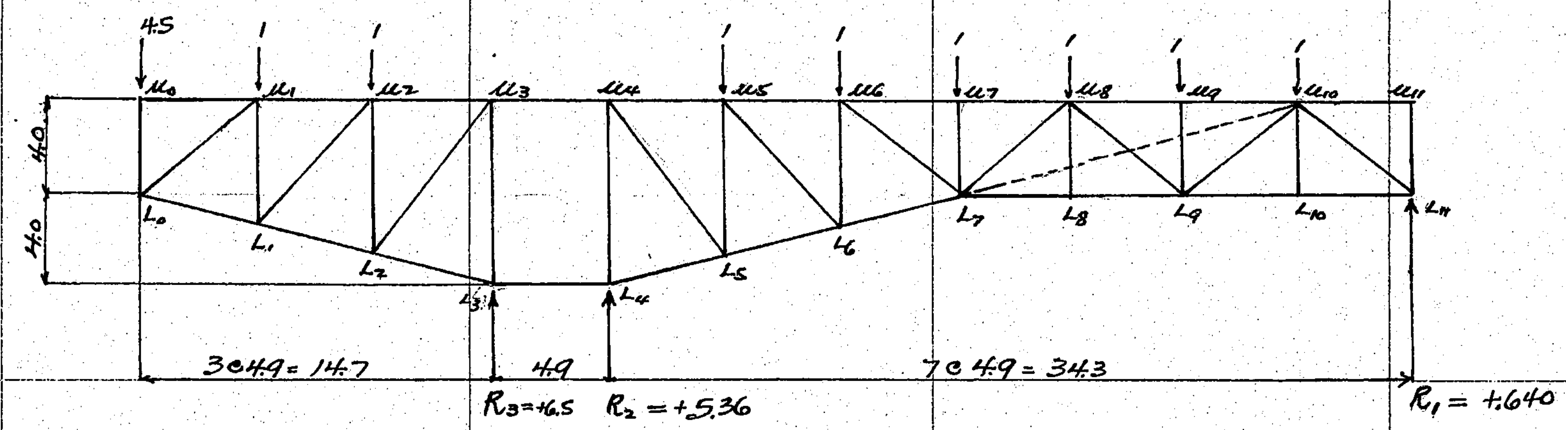
3285 ÷ 53.9 = 61 kg/lin m.

Total weight of Lateral bracings and sway bracings for Anchor span with a ~~cho~~ cantilever arm.

| | | |
|---|---|---|
| Top Lateral bracings | | 3285 |
| Bottom " " | | 2640 |
| Sway Bracings | 12 @ 400 = | 4800 |
| | | 10,725 kg ÷ 53.9 = 199 kg/lin meter. |

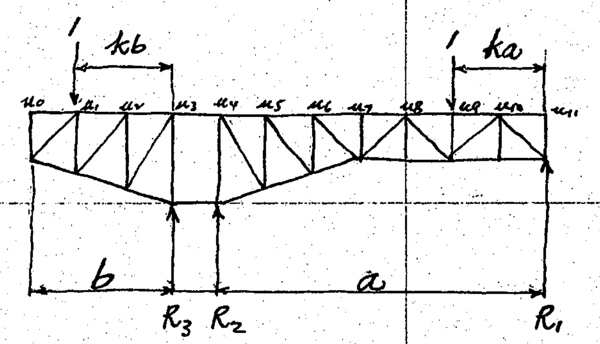
Design of Jinzū Bashi.

Stresses of Main truss for Anchor span with a cantilever arm.




This truss will be designed as a partially continuous truss owing to the lack of diagonal members in the panel over the pier.

Rections due to single unit load at any panel point.




Unit load in the anchor span

$R_1 = +P(1-k) = 1-k$

$R_2 = +Pk = k$

$R_3 = 0$

Unit load on the cantilever arm

$R_1 = -P\frac{kb}{a} = -1\ \frac{k \cdot 3}{7} = -.429\,k$

$R_2 = \cdot P\frac{kb}{a} = .429\,k$

$R_3 = P = 1.000$

for k=1. (end of arm)

$R_1 = -.429$

$R_2 = .429$

$R_3 = 1.000$

| | Panel Point | k | 1-k | .429 k | $R_1$ | $R_2$ | $R_3$ |
|---|---|---|---|---|---|---|---|
| Cantilever arm | $u_0$ | 1.000 | . | .429 | − .429 | + .429 | + 1.000 |
| | $u_1$ | .667 | | .286 | − .286 | + .286 | + 1.000 |
| | $u_2$ | .333 | | .143 | − .143 | + .143 | + 1.000 |
| | $u_3$ | .000 | | .000 | − .000 | + .000 | + 1.000 |
| Anchor span | $u_4$ | 1.000 | .000 | | .000 | + 1.000 | 0 |
| | $u_5$ | .857 | .143 | | + .143 | + .857 | 0 |
| | $u_6$ | .714 | .286 | | + .286 | + .714 | 0 |
| | $u_7$ | .571 | .429 | | + .429 | + .571 | 0 |
| | $u_8$ | .429 | .571 | | + .571 | + .429 | 0 |
| | $u_9$ | .286 | .714 | | + .714 | + .286 | 0 |
| | $u_{10}$ | .143 | .857 | | + .857 | + .143 | 0 |
| | $u_{11}$ | .000 | 1.000 | | + 1.000 | + .000 | 0 |

For full unit loads.

| Panel point | Load | $R_1$ | $R_2$ | $R_3$ |
|---|---|---|---|---|
| $u_0$ | 4.5 | − 1.931 | + 1.931 | + 4.500 |
| $u_1$ | 1.0 | − .286 | + .286 | + 1.000 |
| $u_2$ | 1.0 | − .143 | + .143 | + 1.000 |
| $u_3$ | | | | |
| $u_4$ | | | | |
| $u_5$ | 1.0 | + .143 | + .857 | 0 |
| $u_6$ | 1.0 | + .286 | + .714 | 0 |
| $u_7$ | 1.0 | + .429 | + .571 | 0 |
| $u_8$ | 1.0 | + .571 | + .429 | 0 |
| $u_9$ | 1.0 | + .714 | + .286 | 0 |
| $u_{10}$ | 1.0 | + .857 | + .143 | 0 |
| $u_{11}$ | | | | |
| | | + .640 | + 5.360 | + 6.500 |

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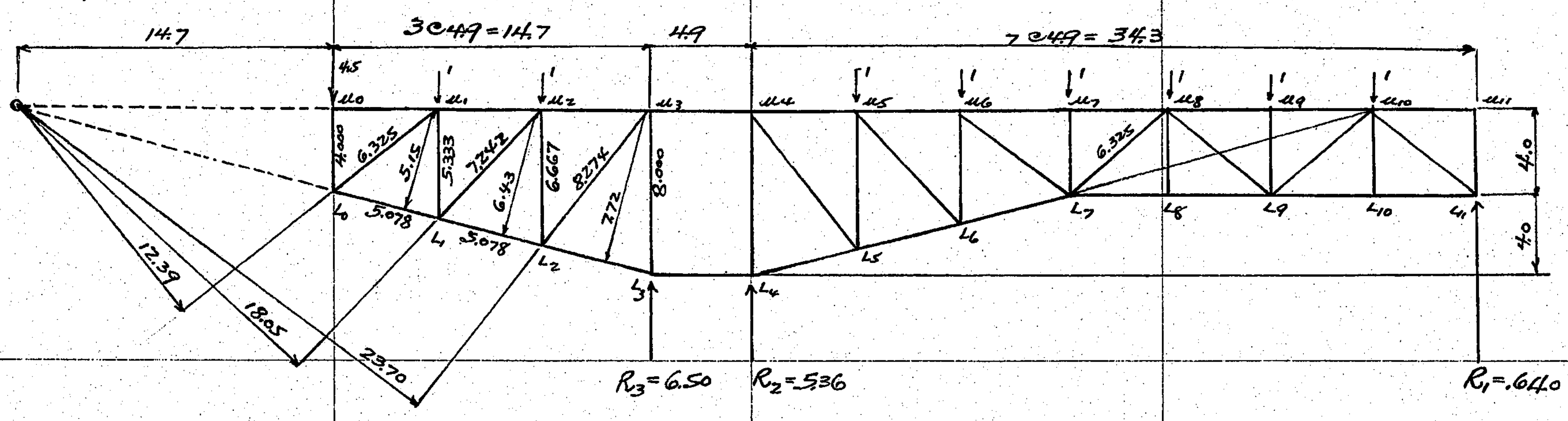
CALCULATIONS FOR

Design of Jinzu - Bashi

Length of members and lever arms.




Stresses of each members due to full unit loads.

Cantelever arm

| | | | | | |
|---|---|---|---|---|---|
| $U_1-U_2$ | moment | | 4.50 · 4.90 | = | 22.05 |
| | Stress | = | 22.05 ÷ 5.333 | = | 4.135 T |
| $L_0-L_1$ | " | = | 22.05 ÷ 5.15 | = | 4.282 C |
| $U_2-U_3$ | moment | | 4.50 · 9.80 = 44.10 | | |
| | | | 1.00 · 4.90 = 4.90 | | |
| | | | 49.00 | | |
| | Stress | = | 49.00 ÷ 6.667 | = | 7.35 T |
| $L_1-L_2$ | " | = | " ÷ 6.43 | = | 7.62 C |
| $L_2-L_3$ | moment | | 16.5 · 4.90 = 80.80 | | |
| | stress | = | 80.80 ÷ 7.72 | = | 10.46 C |
| $U_0-L_0$ | stress | | | = | 4.50 C |
| $L_0-U_1$ | moment | = | 4.5 · 14.7 = 66.12 | | |
| | stress | = | 66.12 ÷ 12.39 | = | 5.34 T |
| $U_1-L_1$ | moment | = | 66.12 | | |
| | | | 1.0 · 19.6 = 19.60 | | |
| | | | 85.72 | | |
| | stress | = | 85.72 ÷ 19.60 | = | 4.38 C |
| $L_1-U_2$ | stress | = | 85.72 ÷ 18.05 | = | 4.75 T |
| $U_2-L_2$ | moment | | 85.72 + 24.50 = 110.22 | | |
| | stress | = | 110.22 ÷ 24.50 | = | 4.50 C |
| $L_2-U_3$ | stress | = | 110.22 ÷ 23.70 | = | 4.79 T |
| $U_3-L_3$ | " | = | 110.22 ÷ 29.4 | = | 3.75 |
| | | | load on $U_3$ | = | 1.00 |
| | | | | | 4.75 C |

Anchor span.

| | | | | | |
|---|---|---|---|---|---|
| $U_{11}-L_{11}$ | stress | | | = | 0.50 C |
| $L_9-L_{11}$ | moment | = | 0.64 · 4.9 = 3.14 | | |
| | stress | = | 3.14 ÷ 4.0 | = | 0.79 T |
| $U_8-U_{10}$ | moment | = | .64 · 9.8 = 6.27 | | |
| | | | − 4.90 | | |
| | | | 1.37 | | |
| | stress | = | 1.37 ÷ 4.0 | = | 0.34 C |

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CALCULATIONS FOR

Design of Jinzu-Bashi

| | | | |
|---|---|---|---|
| $L_7-L_9$. | moment | 0.64 × 14.70 = | 9.41 |
| | | 4.90 × 3 = − | 14.70 |
| | | | − 5.29 ÷ 4.0 = 1.32 C |
| $U_6-U_8$ | moment | 0.64 × 19.60 = | 12.54 |
| | | 4.90 × 6 = − | 29.40 |
| | | | − 16.86 ÷ 4.0 = 4.22 T |
| $U_{10}-L_{11}$ | Shear | − 0.64 | 1.581 × .64 = 1.01 C |
| $L_9-U_{10}$ | 〃 | + 0.36 | 〃 × .36 = .57 C |
| $U_8-L_9$ | | + 1.36 | 〃 × 1.36 = 2.15 T |
| $L_7-U_8$ | | + 2.36 | 〃 × 2.36 = 3.73 C |
| $U_{10}-L_{10}$ | | | 0 |
| $U_8-L_8$ | | | 0 |
| $U_9-L_9$ | | | 1.00 C |
| $U_7-L_7$ | | | 1.00 C |
| $U_5-U_6$ | moment | 0.64 × 24.5 = | 15.79 |
| | | 1.0 × 4.9 × 10 = − | 49.00 |
| | | | − 33.21 ÷ 5.333 = 6.23 T |
| $L_6-L_7$ | | | − 33.21 ÷ 5.15 = 6.45 C |
| $U_4-U_5$ | moment | 0.64 × 29.4 = | 18.82 |
| | | 1.0 × 4.9 × 15 = | − 73.50 |
| | | | − 54.68 ÷ 6.667 = 8.20 T |
| $L_5-L_6$ | | | − 54.68 ÷ 6.43 = 8.51 C |
| $L_4-L_5$ | moment | 0.64 × 34.3 = | 21.95 |
| | | 1.0 × 4.9 × 21 = | − 102.90 |
| | | | − 80.95 ÷ 7.72 = 10.49 C |
| $U_3-U_4$ | | | − 80.95 ÷ 8.00 = 10.12 T |
| $L_3-L_4$ | | | 〃 〃 = 10.12 C |
| $U_6-L_7$ | moment. | 0.64 × 4.90 = | + 3.14 |
| | | 1.0 × 4.9 × 6 = | + 29.40 |
| | | | + 32.54 ÷ 12.39 = 2.63 T |
| $U_6-L_6$ | moment | 0.64 × 4.9 = | + 3.14 |
| | | 1.0 × 4.9 × 10 = | + 49.00 |
| | | | + 52.14 ÷ 19.60 = 2.66 C |
| $U_5-L_6$ | | | 〃 ÷ 18.05 = 2.89 T |
| $U_5-L_5$ | moment | 0.64 × 4.9 = | + 3.14 |
| | | 1.0 × 4.9 × 15 = | + 73.50 |
| | | | + 76.64 ÷ 24.50 = 3.13 C |
| $U_4-L_5$ | | | 〃 ÷ 23.70 = 3.24 T |
| $U_4-L_4$ | moment | 0.64 × 4.9 = | + 3.14 |
| | | 1.0 × 4.9 × 15 = | + 73.50 |
| | | | + 76.64 ÷ 34.3 = 2.24 |
| | | Direct load on $U_4$ = | 1.00 |
| | | | 3.24 C |
| Load on tower bent. | | on $L_3$ 6.50 + 1.0 = | 7.50 C |
| | | on $L_2$ 5.36 + 1.0 = | 6.36 C |
| | | on $L_{11}$ 0.64 + 0.5 = | 1.14 C |

Coef. = $\frac{6.325}{4.0}$ = 1.581

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 Jinzū-Bashi

Dead Load stresses.

Dead Load assumed as follows:

| | | |
|---|---|---|
| Deck construction complete | | 500 |
| structural steel | | |
| stringers $600 \div 4.9 =$ | 123 | |
| floor-beam $420 \div 4.9 =$ | 86 | |
| bracing complete | 190 | |
| trusses assumed | 760 | |
| | | 1,159 |

$1659 \div 2 = 830$ kg per lin meter.

Panel concentration $= 830 \times 4.90 = 4,070$

| members | Load | Stress due to unity | | Dead Load stress | |
|---|---|---|---|---|---|
| $U_0 - U_1$ | 4070 kg. | .00 | | 0 kg | |
| $U_1 - U_2$ | " | 4.14 | T | 16850 | T |
| $U_2 - U_3$ | " | 7.35 | T | 29900 | T |
| $U_3 - U_4$ | " | 10.12 | T | 41,150 | T |
| $U_4 - U_5$ | " | 8.20 | T | 33350 | T |
| $U_5 - U_6$ | " | 6.23 | T | 25350 | T |
| $U_6 - U_8$ | " | 4.22 | T | 17180 | T |
| $U_8 - U_{10}$ | " | 0.34 | C | 1,380 | C |
| $U_{10} - U_{11}$ | " | .00 | | 0 | |
| $L_0 - L_1$ | " | 4.28 | C | 17420 | C |
| $L_1 - L_2$ | " | 7.62 | C | 31,000 | C |
| $L_2 - L_3$ | " | 10.46 | C | 42600 | C |
| $L_3 - L_4$ | " | 10.12 | C | 41,150 | C |
| $L_4 - L_5$ | " | 10.49 | C | 42700 | C |
| $L_5 - L_6$ | " | 8.51 | C | 34600 | C |
| $L_6 - L_7$ | " | 6.45 | C | 26250 | C |
| $L_7 - L_9$ | " | 1.32 | C | 5350 | C |
| $L_9 - L_{11}$ | " | 0.79 | T | 3220 | T |
| Load on tower bent at | | | | | |
| on $L_3$ | 4070 | 7.50 | C | 30500 | C |
| $L_4$ | " | 6.36 | C | 25900 | C |
| $L_{11}$ | " | 1.14 | C | 4600 | C |

| members | Load | Stress due to unity | | Dead Load stress | |
|---|---|---|---|---|---|
| $U_0 - L_0$ | 4070 | 4.50 | C | 18,310 kg | C |
| $U_1 - L_1$ | " | 4.38 | C | 17820 | C |
| $U_2 - L_2$ | " | 4.50 | C | 18310 | C |
| $U_3 - L_3$ | " | 4.75 | C | 19320 | C |
| $U_4 - L_4$ | " | 3.24 | C | 13190 | C |
| $U_5 - L_5$ | " | 3.13 | C | 12750 | C |
| $U_6 - L_6$ | " | 2.66 | C | 10820 | C |
| $U_7 - L_7$ | " | 1.00 | C | 4070 | C |
| $U_8 - L_8$ | " | 0.00 | | 0 | |
| $U_9 - L_9$ | " | 1.00 | C | 4070 | C |
| $U_{10} - L_{10}$ | " | 0.00 | | 0 | |
| $U_{11} - L_{11}$ | " | 0.50 | C | 2,040 | C |
| $L_0 - U_1$ | " | 5.34 | T | 21,720 | T |
| $L_1 - U_2$ | " | 4.75 | T | 19320 | T |
| $L_2 - U_3$ | " | 4.79 | T | 19500 | T |
| $U_4 - L_5$ | " | 3.24 | T | 13190 | T |
| $U_5 - L_6$ | " | 2.89 | T | 11,760 | T |
| $U_6 - L_7$ | " | 2.63 | T | 10700 | T |
| $L_7 - U_8$ | " | 3.73 | C | 15,170 | C |
| $U_8 - L_9$ | " | 2.15 | T | 8750 | T |
| $L_9 - U_{10}$ | " | 0.57 | C | 2320 | C |
| $U_{10} - L_{11}$ | " | 1.01 | C | 4110 | C |

Live Load stresses

Live Load assumed as follows.

Engine Load 8000 kg length of engine = 4.0 m

Uniform load = $8000 \div 4.0$ = 2000 kg per lin meter

for one truss = 1,000 " "

Truck Load 4600 kg length of truck = 2.3 m

Uniform load = $4600 \div 2.3$ = 2000 kg per lin meter

for one truss = 1,000 " "

Use a uniform live load of 1000 kg per lin meter through out.

Live Load panel concentration = 4.9 @ 1000 = 4,900 kg

Impact Stress

$$I = L\left(\frac{60}{90+l}\right)$$

where $I$ = Impact stress of a member considered.

$L$ = Live Load stress of a member in consideration.

$l$ = Length of load in meters to produce a max. stress in the member in consideration.

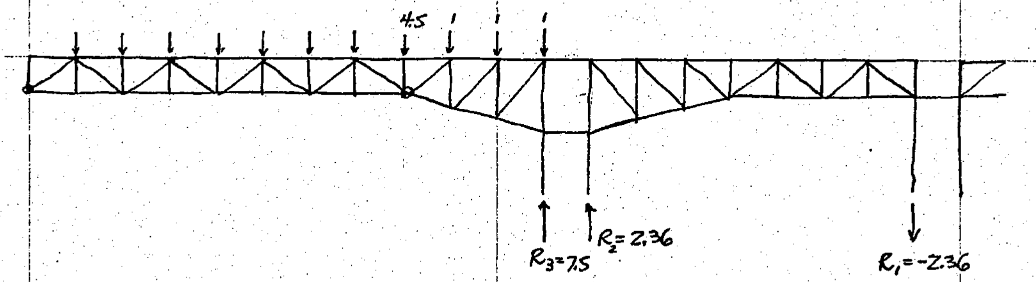
Design of Jinzu - Bashi

Impact coefficients for several load lengths

$$I = L\left(\frac{60}{90+l}\right) = L.C.$$

| Load Length | | Coefficient C | Load Lengths | | Coefficient C |
|---|---|---|---|---|---|
| 1 Panel or | 4.90 m | .632 | 11 panels or | 53.9 m | .417 |
| 2 " | 9.80 | .601 | 12 " | 58.8 | .404 |
| 3 " | 14.70 | .573 | 13 " | 63.7 | .391 |
| 4 " | 19.60 | .547 | 14 " | 68.6 | .378 |
| 5 " | 24.50 | .524 | 15 " | 73.5 | .367 |
| 6 " | 29.40 | .503 | 16 " | 78.4 | .357 |
| 7 " | 34.30 | .483 | 17 " | 83.3 | .347 |
| 8 " | 39.20 | .465 | 18 " | 88.2 | .337 |
| 9 " | 44.10 | .448 | 19 " | 93.1 | .328 |
| 10 " | 49.00 | .432 | 20 " | 98.0 | .319 |

Live Load stresses for members in the cantilever arm.




Stress due to full unit loads over suspended span and cantilever arm same as for Dead Load stress calculated above.

Impact. loaded length = 11 Panels. Impact = 0.417 L

For Reaction see on page

Chord stresses in anchor span. Live load on suspended span and cantilever arm.

Loaded length $11 \times 4.9 = 53.9$ m
impact 0.417 L

| Member | | | | Stress |
|---|---|---|---|---|
| $L_9 - L_{11}$ | m | $= -2.36 \times 4.9 = -11.57$ | $\div 4.0$ | = 2.89 C |
| $U_8 - U_{10}$ | m | $= -2.36 \times 4.9 \times 2 = -23.14$ | $\div 4.0$ | = 5.78 T |
| $L_7 - L_9$ | m | $= -2.36 \times 4.9 \times 3 = -34.70$ | $\div 4.0$ | = 8.68 C |
| $U_6 - U_8$ | m | $= -2.36 \times 4.9 \times 4 = -46.25$ | $\div 4.0$ | = 11.57 T |
| $L_6 - L_7$ | m | $= 11.57 \times 5 = -57.80$ | $\div 5.15$ | = 11.24 C |
| $U_5 - U_6$ | | = " | $\div 5.333$ | = 10.85 T |
| $L_5 - L_6$ | m | $= 11.57 \times 6 = -69.40$ | $\div 6.43$ | = 10.80 C |
| $U_4 - U_5$ | | " | $\div 6.667$ | = 10.42 T |
| $L_4 - L_5$ | m | $= 11.57 \times 7 = -80.90$ | $\div 7.72$ | = 10.48 C |
| $U_3 - U_4$ | | " | $\div 8.00$ | = 10.11 T |
| $L_3 - L_4$ | | " | " | = 10.11 C |

Live load on anchor span only.

Reaction $R_1 = 3.00$

Loaded length $7 \times 4.9 = 34.3$ m
impact = 0.483 L.

Stress

$L_9 - L_{11}$ m = $3.0 \times 4.9 = 14.70 \div 4.0$ = 3.68 T

$U_8 - U_{10}$ m = $3.0 \times 4.9 \times 2 = 29.4$
$1.0 \times 4.9 = -4.9$
$24.5 \div 4.0$ = 6.12 C

$L_7 - L_9$ m = $3.0 \times 4.9 \times 3 = 44.10$
$1.0 \times 4.9 \times 3 = -14.70$
$29.4 \div 4.0$ = 7.35 T

$U_6 - U_8$ m = $3.0 \times 4.9 \times 4 = 58.80$
$1.0 \times 4.9 \times 6 = -29.40$
$29.4 \div 4.0$ = 7.35 C

Design of Jinzū - Bashi.

$L_6 - L_7$  m = 3.0 × 4.9 × 5 = 73.50
1.0 × 4.9 × 10 = − 49.00
24.50 ÷ 5.15 = 4.76 T
$M_5 - M_6$  " ÷ 5.33 = 4.60 C

$L_5 - L_6$  m = 3.0 × 4.90 × 6 = 88.20
1.0 × 4.9 × 15 = − 73.50
14.70 ÷ 6.43 = 2.29 T
$M_4 - M_5$  " ÷ 6.667 = 2.21 C

$L_4 - L_5$  m = 3.0 × 4.9 × 7 = 102.90
1.0 × 4.9 × 21 = − 102.90
0.00  0.00
$M_3 - M_4$  0.00

Web stresses in anchor span. due to positive max. shear. Load on panel pts in anchor span to the left of section.

| | | | Stress | | | impact |
|---|---|---|---|---|---|---|
| $M_{10} - L_{11}$ | Load on $M_5$ to $M_{10}$ | $R_1$ = 3.00 | 3.00 × 1.581 = | 4.74 C | 6 panels | .503 L |
| $L_9 - M_{10}$ | " $M_5 - M_9$ | $R_1$ = 2.143 | 2.143 × 1.581 = | 3.39 T | 5 " | .524 L |
| $M_8 - L_9$ | " $M_5 - M_8$ | $R_1$ = 1.429 | 1.429 × 1.581 = | 2.26 C | 4 " | .547 L |
| $L_7 - M_8$ | " $M_5 - M_7$ | $R_1$ = .858 | .858 × 1.584 = | 1.36 T | 3 " | .573 L |
| $M_{11} - L_{11}$, $M_9 - L_9$, & $M_7 - L_7$ | | | = | 1.00 C | 1 " | .632 L |
| $M_{10} - L_{10}$, & $M_8 - L_8$ | | | = | 0.00 | | |
| $M_6 - L_6$ | Load on $M_5$. | $R_1$ = .143 | .143 × $\frac{1}{4}$ = | 0.04 C | 1 " | .632 L |
| $M_5 - L_6$ | " | " | .143 × $\frac{4.9}{18.05}$ = | 0.04 T | 1 " | .632 L |
| $M_6 - L_7$ | Load on $M_5$ & $M_6$ | $R_1$ = .429 | .429 × $\frac{4.9}{12.39}$ = | 0.17 T | 2 " | .601 L |

due to negative shear. Load on suspended span & cantilever arm and from the section to right hand support

| | | Shear | Stress | | | impact |
|---|---|---|---|---|---|---|
| $M_{10} - L_{11}$ | $R_1$ = − 2.36 | 2.36 | 2.36 × 1.581 = | 3.73 T | 11 Panels | .417 L |
| $L_9 - M_{10}$ | $R_1$ = − 2.36 + .857 = 1.503 | 2.503 | 2.503 × 1.581 = | 3.96 C | 12 " | .404 L |
| $M_8 - L_9$ | $R_1$ = − .789 | 2.789 | 2.789 × 1.581 = | 4.41 T | 13 " | .391 L |
| $L_7 - M_8$ | $R_1$ = − .218 | 3.218 | 3.218 × 1.581 = | 5.08 C | 14 " | .378 L |
| $M_{11} - L_{11}$, $M_9 - L_9$, & $M_7 - L_7$ | | | = | 1.00 C | 1 " | .632 L |
| $M_{10} - L_{10}$ & $M_8 - L_8$ | | | = | 0.00 | | |

$M_6 - L_7$  $R_1$ = + .211  m  .211 × 4.9 = 1.03
1.00 × 4.9 × 6 = 29.40
30.43 ÷ 12.39 = 2.46 T  15 "  .367 L

$M_6 - L_6$  $R_1$ = + .497  m  .497 × 4.9 = 2.44
1.00 × 4.9 × 10 = 49.00
51.44 ÷ 19.6 = 2.63 C  16 "  .357 L
$M_5 - L_6$  " ÷ 18.05 = 2.85 T  "  "
$M_4 - L_5$  Same as for Dead Load. 3.24 T  19 "  .328 L
$M_5 - L_5$  " " 3.13 C  "  "

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CALCULATIONS FOR

Design of Jinzu - Bashi

Summary of Live Load stresses

| Members | Load | Stress due to unity | Live Load stress | Impact stress | Stress due to unity | Live Load stress | Impact stress |
|---|---|---|---|---|---|---|---|
| $M_0 - L_1$ | 4,900 kg | .00 | 0 kg | 0 | — | — | — |
| $M_1 - M_2$ | 〃 | 4.14 T | 20,450 T | 8,520 T | — | — | — |
| $M_2 - M_3$ | 〃 | 7.35 T | 36,000 T | 15,010 T | — | — | — |
| $M_3 - M_4$ | 〃 | 10.12 T | 49,600 T | 20,680 T | — | — | — |
| $M_4 - M_5$ | 〃 | 10.42 T | 51,000 T | 21,280 T | 2.21 C | 10,820 C | 5,220 C |
| $M_5 - M_6$ | 〃 | 10.85 T | 53,180 T | 22,170 T | 4.60 C | 22,540 C | 10,880 C |
| $M_6 - M_8$ | 〃 | 11.57 T | 56,700 T | 23,650 T | 7.35 C | 36,000 C | 17,400 C |
| $M_8 - M_{10}$ | 〃 | 5.78 T | 28,350 T | 11,820 T | 6.12 C | 30,000 C | 14,500 C |
| $M_{10} - M_{11}$ | 〃 | .00 | | | .00 | 0 | |
| $L_0 - L_1$ | 4,900 | 4.28 C | 20,980 C | 8,750 C | — | — | — |
| $L_1 - L_2$ | 〃 | 7.62 C | 37,350 C | 15,580 C | — | — | — |
| $L_2 - L_3$ | 〃 | 10.46 C | 51,250 C | 21,380 C | — | — | — |
| $L_3 - L_4$ | 〃 | 10.12 C | 49,600 C | 20,680 C | — | — | — |
| $L_4 - L_5$ | 〃 | 10.48 C | 51,400 C | 21,450 C | .00 | 0 | |
| $L_5 - L_6$ | 〃 | 10.80 C | 52,900 C | 22,050 C | 2.29 T | 11,220 T | 5,420 T |
| $L_6 - L_7$ | 〃 | 11.24 C | 55,100 C | 22,970 C | 4.76 T | 23,350 T | 11,280 T |
| $L_7 - L_9$ | 〃 | 8.68 C | 42,550 C | 17,750 C | 7.35 T | 36,000 T | 17,400 T |
| $L_9 - L_{11}$ | 〃 | 2.89 C | 14,170 C | 5,900 C | 3.68 T | 18,050 T | 8,720 T |
| $M_0 - L_0$ | 4,900 | 4.50 C | 22,050 C | 9,200 C | — | — | — |
| $M_1 - L_1$ | 〃 | 4.38 C | 21,450 C | 8,950 C | — | — | — |
| $M_2 - L_2$ | 〃 | 4.50 C | 22,050 C | 9,200 C | — | — | — |
| $M_3 - L_3$ | 〃 | 4.75 C | 23,300 C | 9,710 C | — | — | — |
| $M_4 - L_4$ | 〃 | 3.24 C | 15,880 C | 6,620 C | — | — | — |
| $M_5 - L_5$ | 〃 | 3.13 C | 15,350 C | 5,030 C | .00 | 0 | 0 |
| $M_6 - L_6$ | 〃 | 2.63 C | 12,900 C | 4,600 C | .04 C | 200 C | 130 C |
| $M_7 - L_7$ | 〃 | 1.00 C | 4,900 C | 3,100 C | 1.00 C | 4,900 C | 3100 C |
| $M_8 - L_8$ | 〃 | .00 | 0 | | .00 | 0 | 0 |
| $M_9 - L_9$ | 〃 | 1.00 C | 4,900 C | 3,100 C | 1.00 C | 4,900 C | 3,100 C |
| $M_{10} - L_{10}$ | 〃 | .00 | 0 | | .00 | 0 | 0 |
| $M_{11} - L_{11}$ | 〃 | 1.00 C | 4,900 C | 3,100 C | 1.00 C | 4,900 C | 3,100 C |
| $L_0 - M_1$ | 4,900 | 5.34 T | 26,150 T | 10,900 T | — | — | — |
| $L_1 - M_2$ | 〃 | 4.75 T | 23,300 T | 9,710 T | — | — | — |
| $L_2 - M_3$ | 〃 | 4.79 T | 23,480 T | 9,790 T | — | — | — |
| $M_4 - L_5$ | 〃 | 3.24 T | 15,880 T | 5,210 T | .00 | 0 | |
| $M_5 - L_6$ | 〃 | 2.85 T | 13,970 T | 4,980 T | .04 T | 200 T | 130 T |
| $M_6 - L_7$ | 〃 | 2.46 T | 12,060 T | 4,430 T | .17 T | 830 T | 500 T |
| $L_7 - M_8$ | 〃 | 5.08 C | 24,900 C | 9,410 C | 1.36 T | 6,660 T | 3,820 T |
| $M_8 - L_9$ | 〃 | 4.41 T | 21,600 T | 8,450 T | 2.26 C | 11,070 C | 6,050 C |
| $L_9 - M_{10}$ | 〃 | 3.96 C | 19,410 C | 7,840 C | 3.39 T | 16,620 T | 8,710 T |
| $M_{10} - L_{11}$ | 〃 | 3.73 T | 18,280 T | 7,620 T | 4.74 C | 23,250 C | 11,700 C |
| Max. Load on Tower Bent. | | | | | | | |
| on $L_3$ | 4,900 | 7.50 C | 36,750 C | 15,340 C | | | |
| 〃 $L_4$ | 〃 | 6.36 C | 31,200 C | 10,230 C | | | |
| 〃 $L_{11}$ | 〃 | 4.00 C | 19,600 C | 9,460 C | | | |
| 〃 $L_{11}$ | 〃 | 2.36 T | 11,560 T | 4,820 T | | | |

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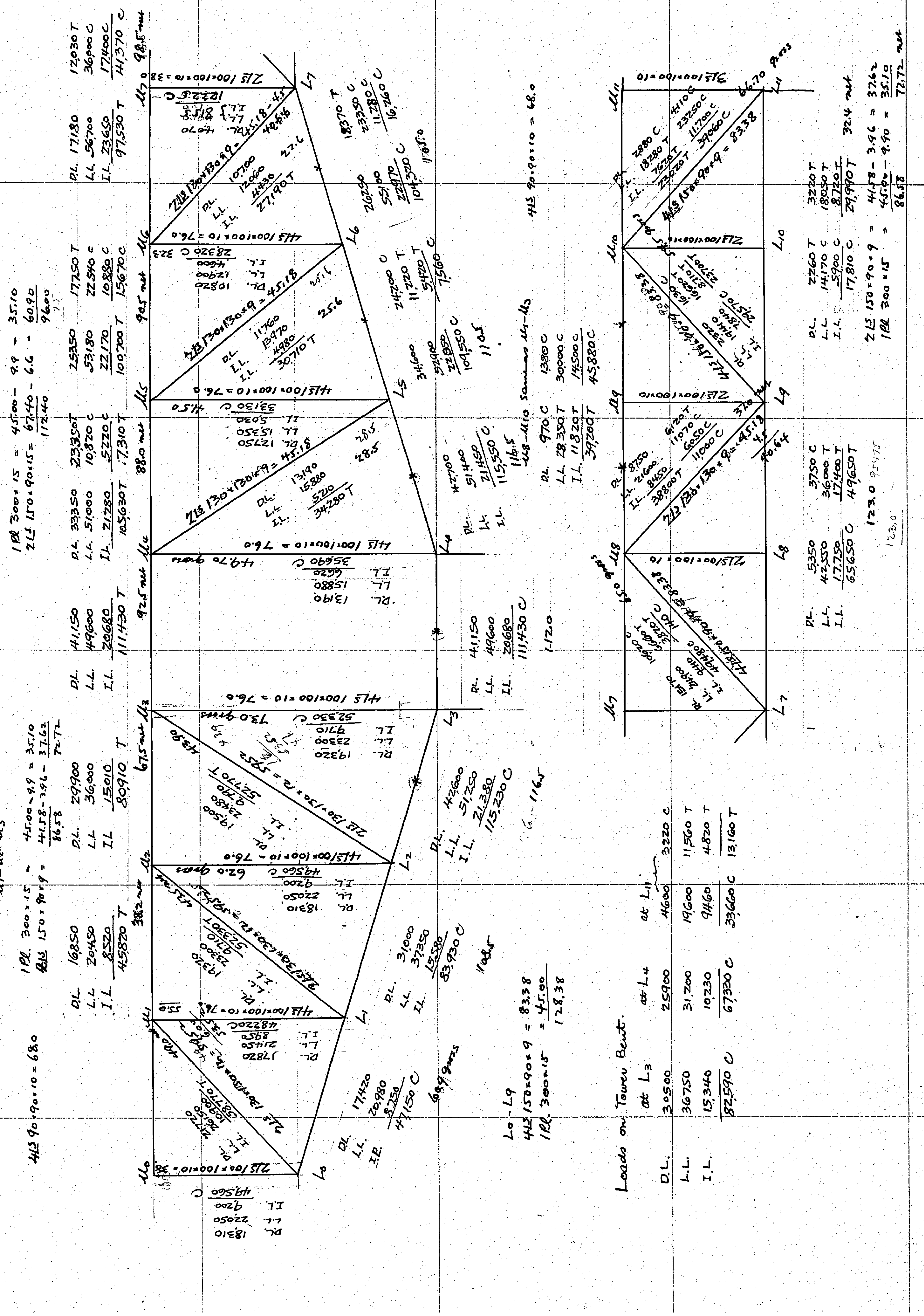
CALCULATIONS FOR

Design of Jinzu - Bashi

Stresses and section of truss members.



増田淳氏関係資料
(独立行政法人 土木研究所蔵)

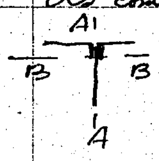
Design of Jinzu- Bashi.

Sections of member
Top chord member

| | | | | | |
|---|---|---|---|---|---|
| 1 web | 300 × 15 | = 45.00 | - 9.9 | = 35.10 | |
| 2 ∠s | 150 × 90 × 9 | = 41.58 | - 3.96 | = 37.62 | section A |
| | | 86.58 | | 72.72 net | |

| | | | | | |
|---|---|---|---|---|---|
| 1 web | 300 × 15 | = 45.00 | - 9.9 | = 35.10 | |
| 2 ∠s | 150 × 90 × 15 | = 67.40 | - 6.6 | = 60.90 | Section B. |
| | | 112.40 | | 96.00 net | |

As compression member — center of gravity — moment of inertia AA.



| | | | | | | |
|---|---|---|---|---|---|---|
| 2 ∠s 150 × 90 × 15 | = 67.40 × 2.2 | = 148.20 | 67.40 | × $5.33^2$ + 408 | = 2240 | $r = \sqrt{\frac{8470}{112.40}} = 9.11$ |
| 1 Pl. 300 × 15 | = 45.00 × 15.5 | 697.00 | 45.00 | × $7.97^2$ + 3370 | = 6230 | |
| | 112.40 7.53 | 845.20 | 112.40 | | 8470 | |

For BB Axis. 67.40 × $5.93^2$ + 3020 = 5396 $r = \sqrt{\frac{5396}{112.40}} = 6.92$
45.00
112.40

allowable unit stress as compression member
For 4.9 meter $1200 - 5 \times \frac{490}{6.92} = 846$ kg/cm²

2.45 meter $l/r = 35.4$ 1000 kg/cm²

| | stress | | | |
|---|---|---|---|---|
| $U_0$-$U_1$ | S | | SR | 4 ∠s 90 × 90 × 10 |
| $U_1$-$U_2$ | 45822 T | ÷ 1200 = | 38.2 | Section A |
| $U_2$-$U_3$ | 80910 T | | 67.5 | " A |
| $U_3$-$U_4$ | 111430 T | | 92.0 | " B |
| $U_4$-$U_5$ | 105630 T | | 88.0 | " B |
| $U_5$-$U_6$ | 100700 T | | ~~90.5~~ | |
| | 7835 C | | | |
| | 108535 | | 90.5 | " B |
| $U_6$-$U_7$-$U_8$ | 97530 T | | | |
| | 20685 C | | | |
| | 118215 | | 98.5 | " B |
| $U_8$-$U_9$-$U_{10}$ | 45880 C | | | |
| | 19600 T | | | |
| | 65480 C | | | " A |
| $U_{10}$-$U_{11}$ | | | | 4 ∠s 90 × 90 × 10 |

Bottom chord sections — Moment of Inertia

4 ∠s 150 × 90 × 9 = 83.38 × $5.71^2$ + 1872 = 4592 $r = \sqrt{\frac{4592}{128.38}} = 5.98$
1 Pl 300 × 15 = 45.00
128.38

Unsupported length 4.90 $1200 - 5 \times \frac{490}{5.98} = 790$ kg/cm²
" " 2.45 $1200 - 5 \times \frac{245}{5.98} = 995$ kg/cm²

Unsupported length 5.08 $1200 - 5 \times \frac{5.08}{5.98} = 775$ kg/cm²
" " 2.54 $1200 - 5 \times \frac{254}{5.98} = 988$ kg/cm²

| | | Unsupported length | Unit stress | Section Req'd |
|---|---|---|---|---|
| $L_0$-$L_1$ | 47150 C | 5.08 meters | 775 kg/cm² | 60.9 |
| $L_1$-$L_2$ | 83930 C | 5.08 " | 775 | 108.5 |
| $L_2$-$L_3$ | 115230 C | 2.54 " | 988 | 116.5 |
| $L_3$-$L_4$ | 111430 C | 2.45 " | 995 | 112.0 |
| $L_4$-$L_5$ | 115550 C | 2.54 " | 988 | 116.5 |
| $L_5$-$L_6$ | 109550 C | 2.54 " | 988 | ~~105.0~~ 110.5 |
| $L_6$-$L_7$ | 104320 C | 2.54 " | 988 | 105.0 |

use
4 ∠s 150 × 90 × 9 = 83.38
1 Pl. 300 × 15 = 45.00
128.38

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CALCULATIONS FOR

Design of Jinzu-Bashi

$L_7$-$L_8$-$L_9$ 65650
24825 length unit stress SR
90475 C 4.90 790 114.5

$L_9$-$L_{10}$-$L_{11}$ 17810 C
½C 8905
25715 4.90 790 32.6

as tension member 29990 T
8905
38895 T ÷ 1200 = 32.4 cm² net

Try { 2L̲s 150×90×9 = 41.58 − 9.96 = 37.62
1 Pl. 300×15 = 45.0 − 9.90 = 35.10 }
86.58 72.72 net

Diagonals tension member SR

| | | | | | | |
|---|---|---|---|---|---|---|
| $L_0$-$U_1$ | 58770 T | ÷ 1200 = 49.0 net | 2L̲s 130×130×12 = 59.52 | − 6.0 | = 53.52 | net |
| $L_1$-$U_2$ | 52330 | = 43.5 | " | − 12.0 | = 47.52 | " |
| $L_2$-$U_3$ | 52770 | = 43.9 | " | − 12.0 | = 47.52 | " |
| $U_4$-$L_5$ | 34280 | = 28.5 | 2L̲s 130×130×9 = 45.18 | − 4.5 | = 40.64 | " |
| $U_5$-$L_6$ | 30710 | = 25.6 | " | | " | " |
| $U_6$-$L_7$ | 27190 | = 22.6 | " | | " | " |

$L_7$-$U_8$ 49480 C try 4L̲s 150×90×9 = 83.38 r = 7.22 $l/r = \frac{633}{7.22} = 88$

Unit stress = 1200 − 5 × $\frac{633}{7.22}$ = 760

49480 ÷ 760 = 65.0 □cm gross

$U_8$-$L_9$ 38800 T assumed section 2L̲s 130×130×9 = 45.18 − 4.5 = 40.64 net.
5500 ½C
44300 T ÷ 1200 = 37.0 □cm net

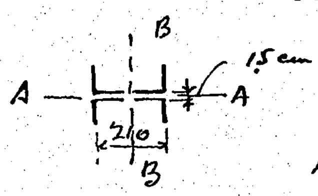
compression Try 2L̲s 130×130×9 = 45.18 r = 5.05 l = 633 cm
allowable unit stress = 444.5 kg/cm²

stress 11000
5500
16500 ÷ 444.5 = 37.2 □cm gross.

$L_9$-$U_{10}$ 4L̲s 150×90×9 = 83.38 Unit stress 760
29570 C
11850 ½T
41420 ÷ 760 = 54.50 □cm gross required

$U_{10}$-$L_{11}$ 4L̲s 150×90×9 = 83.38
39060 C
11510 ½T
50570 ÷ 760 = 66.70 □cm gross required

vertical members 4L̲s 100×100×10 = 76.0 □cm
r AA axis 15 mm apart 4.68
r BB Axis



4L̲s 100×100×10 = 76.0 × 7.69² + 698 = 5200 $r = \sqrt{\frac{5200}{76.0}} = 8.28$

5.33 $l/r = \frac{828}{\cancel{5}} = \frac{533}{8.28} = 64.5$ 1200 − 5 × 64.5 = 878

$l/r = \frac{266}{4.68} = 56.8$ 1200 − 5 × 56.8 = 916

6.66 $l/r = \frac{666}{8.28} = 80.5$ 1200 − 5 × 80.5 = 800

$l/r = \frac{333}{4.68} = 71.1$ 1200 − 5 × 71.1 = 844

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CALCULATIONS FOR

Design of Jinzu Bashi

8.00 $\quad \frac{l}{r} = \frac{800}{8.28} = 96.5 \quad 1200 - 5 \times 96.5 = 718$

4.00 $\quad \frac{l}{r} = \frac{400}{4.68} = 85.5 \quad 1200 - 5 \times 85.5 = 772$

| Section required | stress | Unit stress | section reqd | section used |
|---|---|---|---|---|
| $U_1$-$L_1$ | 48220 c | 878 | 55.0 | 4L 100×100×10 = 76.0 |
| $U_2$-$L_2$ | 49560 c | 800 | 62.0 | " |
| $U_3$-$L_3$ | 52330 c | 718 | 73.0 | " |
| $U_4$-$L_4$ | 35690 c | 718 | 49.7 | " |
| $U_5$-$L_5$ | 33130 c | 800 | 41.5 | " |
| $U_6$-$L_6$ | 28320 c | 878 | 32.3 | " |

$U_{11}$-$L_{11}$ 3L 100×100×10 = 57.0

All other verticals 2L 100×100×10 = 38.

Approximate weight of truss.

| | | | | | | |
|---|---|---|---|---|---|---|
| $U_0$-$U_1$, $U_{10}$-$U_{11}$ | 68.0 | @ .785 | × | 2 × | 4.9 | = 524 |
| Top chord | 86.58 | " | × | 4 × | 4.9 | = 1330 |
| Top chord | 112.40 | " | × | 5 × | 4.9 | = 2160 |
| Bottom chord | 128.38 | | | 6 × | 5.08 | = 3070 |
| " " | 128.38 | | | 3 × | 4.9 | = 1480 |
| " " | 86.58 | | | 2 × | 4.9 | = 665 |
| Diagonals | 59.52 | | | 1 × | 6.33 | = 296 |
| " | 59.52 | | | × | 7.25 | = 338 |
| " | 59.52 | | | × | 8.27 | = 386 |
| " | 45.18 | | | × | 8.27 | = 293 |
| " | 45.18 | | | × | 7.25 | = 257 |
| " | 45.18 | | | 2 × | 6.33 | = 450 |
| " | 83.38 | | | 3 × | 6.33 | = 1240 |
| verticals | 38.00 | | | 5 × | 4.0 | = 596 |
| " | 57.00 | | | × | 4.0 | = 179 |
| | 76.00 | | | 2 × | 5.33 | = 635 |
| | 76.00 | | | 2 × | 6.66 | = 795 |
| | 76.00 | | | 2 × | 8.00 | = 955 |

15649 × 2 = 31298

Details 30%. 9400

40698

40698 ÷ 53.9 = 755 kg per lin. meter

Assumed weight of truss span complete

| | | |
|---|---|---|
| stringers | 600 ÷ 4.9 | = 123 |
| floor beam | 420 ÷ 4.9 | = 86 |
| bracings complete say | | 190 |
| trusses assumed | | 755 |
| | | 1154 × 53.9 = 62300 |
| misc say | | 2700 |
| | | 65000 |

Design of Jinzu Bashi

Design of Tower Bent.

Wind Load.

Exposed area of superstructure

| | | | | |
|---|---|---|---|---|
| Handrail | | = 0.20 | × 8.80 = | 1.76 |
| Floor | | = 0.20 | × 8.30 = | 1.66 |
| Chords. | 2 @ .30 | = 0.60 | × 6.00 = | 3.60 |
| web | | = 0.35 | × 6.00 = | 2.10 |
| misc. say | | = 0.05 | × = | 0.30 |
| | | 1.45 | 6.50ᵐ | 9.42 |
| 50% | | .73 | | |
| | | 2.18 | | |

wind load = 250 × 2.18 = 545 kg per lin m.

When moving load is taken in consideration.

| | | | |
|---|---|---|---|
| Exposed area of structure | 2.18 | × 6.50 = | 14.17 |
| " " " live load | 1.00 | × 1.13 = | 1.13 |
| | 3.18 | 4.8ᵐ | 15.30 |

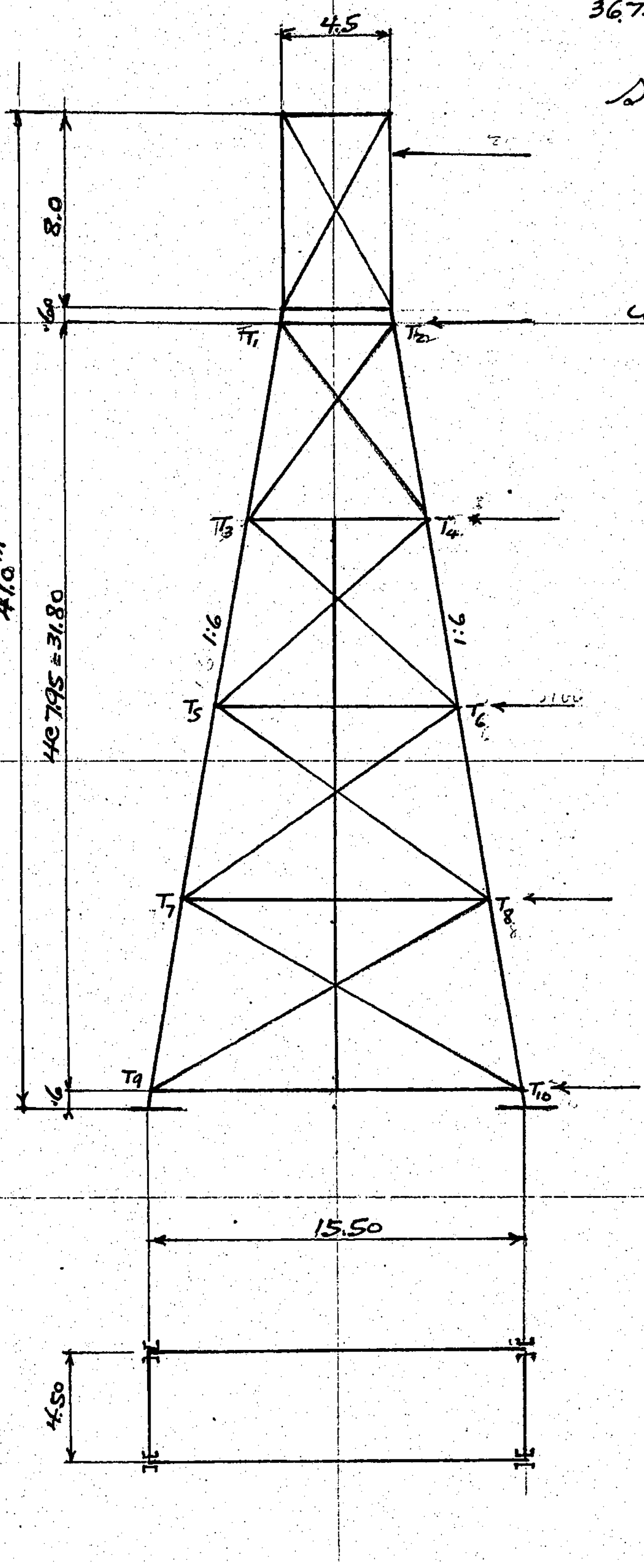
wind load = 150 × 3.18 = 475 kg per lin m.

Use a wind load of 545 kg per lin m, whose point of application is 6.5ᵐ above top of tower.

for 7.5 panels. 7.5 × 4.9 = 36.75 m

36.75 × 545 = 20,000 kg. on one truss. in case of no live load.

36.75 × 475 = 17,500 kg " " " " full load.




Superimposed loads on pier.

Dead Load 30500 ÷ 2 = 15,300 kg on one column

Live load with impact 52090 ÷ 2 = 26,000 " " " "

41,300 " " " "

weight of pier assumed 33,000 kg

33000 ÷ 33.0 = 1000 kg per lin m

for one panel 7.95 × 1000 = 7950 kg

for one column 7950 ÷ 4 ≒ 2000 kg.

Horizontal wind load on bent.

exposed area of bent.

| | | |
|---|---|---|
| Column | 0.50 × 8.0 = | 4.0 |
| Strut | 0.26 × 5.0 = | 1.3 |
| diagonals | 0.13 × 14.0 = | 1.8 |
| vert. tie | 0.15 × 8.0 = | 1.2 |
| | | 8.30 |
| 50% | | 4.20 |
| | | 12.50 |

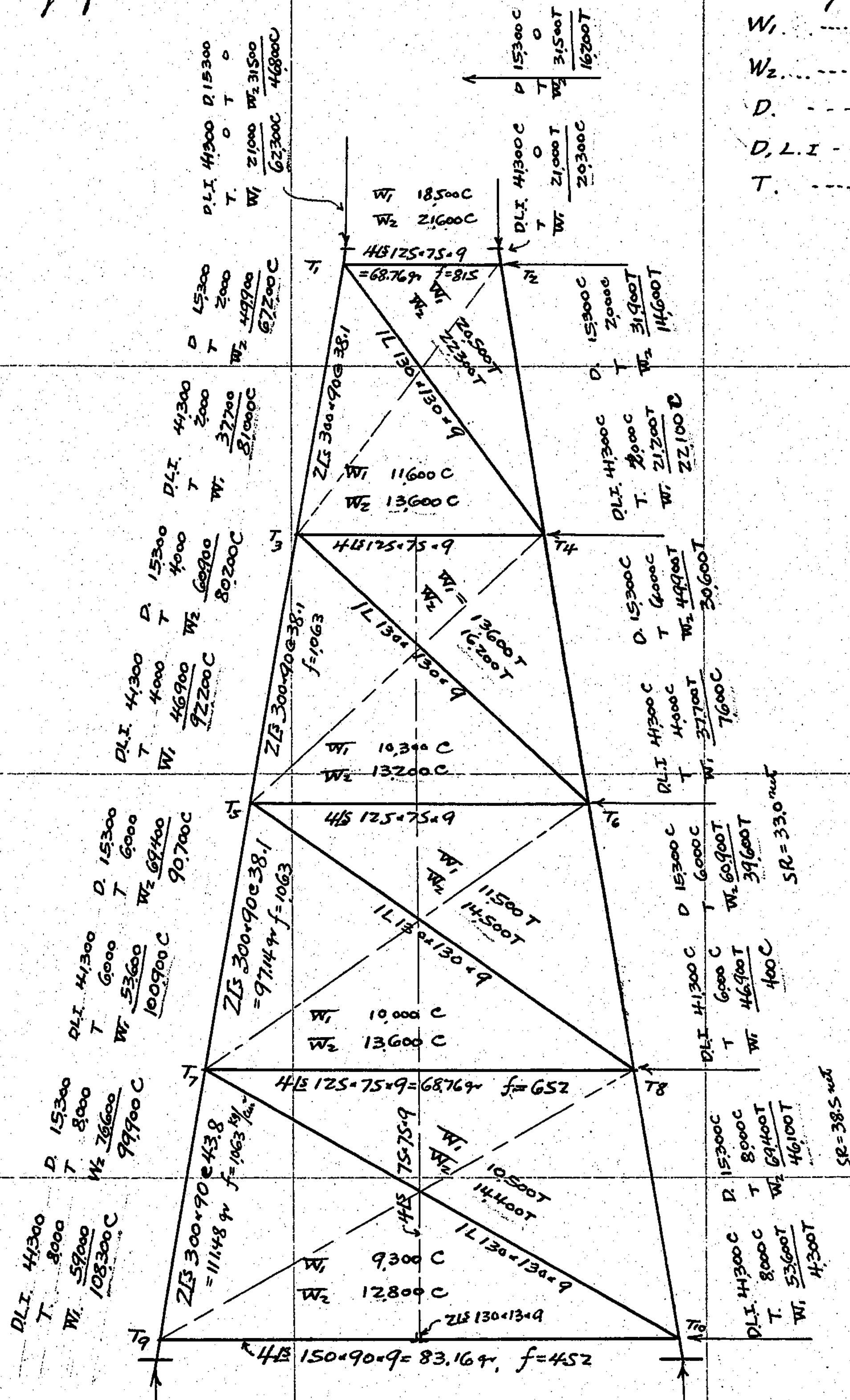
12.50 @ 250 = 3100 kg one truss. no live load

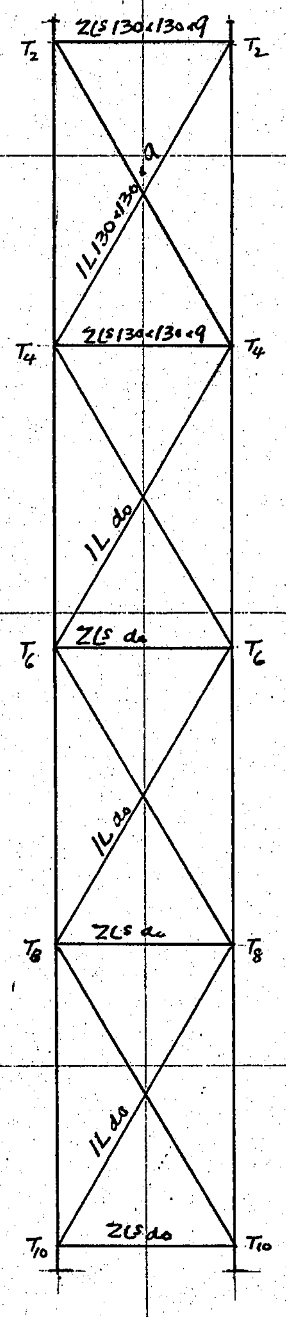
12.50 @ 150 = 1900 " full load.

Design of Jinzu Bashi

Summary of Stresses in Tower Bent. Wind stress by graphical method.

$W_1$ ...... wind load 150 kg/m² on structure + moving load.
$W_2$ ...... " " 250 " on structure only.
D. ---- Dead Load on column
D.L.I --- D.L. + L.L. + impact load.
T. ---- weight of tower bent.




| | | | |
|---|---|---|---|
| D.L.I. 41300 | D=15300 C | D.L.I. 41300 C | D. 15300 C |
| T 8,000 | T 8,000 C | T 8,000 C | T 8,000 C |
| $W_1$ 58,000 | $W_2$ 75400 C | $W_1$ 58,000 T | $W_2$ 75400 T |
| 107300 C | 98700 C | 8700 T | 52,100 T |

~~bearing area reqd.~~ = 3,070 cm² on concrete.

anchor bolts.
43.5 cm² net
(4 - 45φ bolts / or 6 - 38φ ") — 75 cm² [illegible]

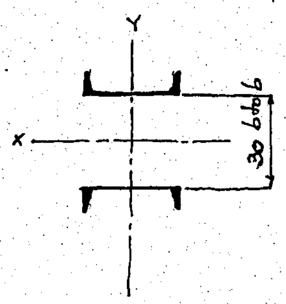
Approximate Length of members. (neutral axis).

| members | Length | members | Length | members | Length |
|---|---|---|---|---|---|
| Columns | 8.06 m | $T_1 - T_4$ | 9.98 | $T_2 - T_2$ | 4.90 |
| $T_1 - T_2$ | 4.70 | $T_3 - T_6$ | 11.77 | $T_4 - T_4$ | |
| $T_3 - T_4$ | 7.35 | $T_5 - T_8$ | 13.84 | — — | |
| $T_5 - T_6$ | 10.00 | $T_7 - T_{10}$ | 16.07 | $T_{10} - T_{10}$ | |
| $T_7 - T_8$ | 12.65 | | | $T_2 - T_4$ | 9.34 |
| $T_9 - T_{10}$ | 15.30 | | | $T_4 - T_6$ | |
| | | | | — — | |
| | | | | $T_8 - T_{10}$ | |

Design of Jinzu Bashi.

Sections of Tower members.

Column $T_7 - T_9$




max. stress 108,300 C (D.L. + L.L. + I.L. + W.L.)

2[s 300×90 @ 43.8 kg = 111.48 cm² gr. − 26.40 = 85.08 cm² net.

radius of gyration Y−Y = 11.52 cm (least).

unsupported length 8.06 meters $l/r$ = 806/11.52 = 70.0

allowable unit compression = 1200 − 5×70 = 850

850 × 1.25 = 1,063 kg/cm²

Required gross sectional area = $\frac{108300}{1063}$ = 102.0 cm² ok.

Column $T_5 - T_7$

max. stress 100,900 C (D.L. + L.L. + I.L. + W.L.)

2[s 300×90 @ 38.10 kg = 97.14 cm² gr. − 22.43 = 74.71 cm² net.

radius of gyration Y−Y = 11.51 cm (least)

unsupported length = 8.06 meters $l/r$ = 806/11.51 = 70.0

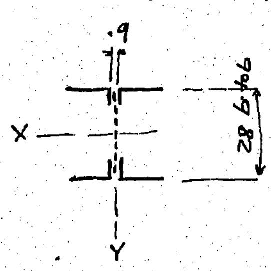
allowable unit compression = (1200 − 5×70) × 1.25 = 1063 kg/cm²

Required gross sectional area = $\frac{100900}{1063}$ = 95.00 cm² ok

Columns $T_3 - T_5$ + $T_1 - T_3$

Use same section as for $T_5 - T_7$.

Struts. $T_1 - T_2$




Stress = 21,600 kg C unsupported length l = 4.70 meters.

4Ls 125×75×9 = 68.76 cm² gr

least radius of gyration $r_y$ = 6.1 cm $l/r$ = 470/6.1 = 77.0

allowable unit stress f = 1200 − 5×77 = 815 kg/cm²

Required gross sectional area = $\frac{21600}{815}$ = 26.5 cm² ok.

Struts $T_7 - T_8$

Stress = 13600 kg C Unsupported length l = $\frac{12.65}{2}$ meters = 6.33

4Ls 125×75×9 = 68.76 cm² gr.

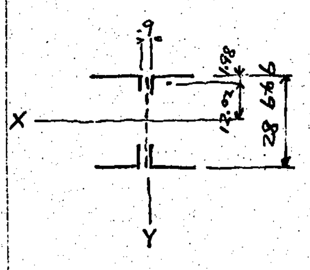
least radius of gyration $r_y$ = 6.1 cm $l/r$ = ~~1265~~ 6.33/6.1 = 103.7 $l/r_x = \frac{1265}{12.46}$ = 101.5

allowable unit stress = f = $\frac{21,000,000}{3} \times \left(\frac{1}{103.7}\right)^2$ = 652 kg/cm²

Required gross sectional area = $\frac{13600}{652}$ = 24.65 cm² ok.

Struts $T_3 - T_4$ + $T_5 - T_6$ use same section as for $T_7 - T_8$.

Struts $T_9 - T_{10}$ Stress = 12800 kg C unsupported length 15.3 meters horizontal, 7.65 " vertical




4Ls 150×90×9 = 83.16 cm² gr

$r_y$ = 7.2 cm $l_v/r_y$ = 765/7.2 = 106.4

$I_x$ 129 × 4 = 516

83.16 × 12.02² = 12020

12536 cm⁴

$r_x = \sqrt{\frac{12536}{83.16}}$ = 12.28 cm $l_x/r_x$ = 1530/12.28 = 124.5

allowable unit compression f = $\frac{21,000,000}{3}\left(\frac{1}{124.5}\right)^2$ = 452 kg/cm²

Required sectional area = $\frac{12800}{452}$ = 28.3 cm² gr ok

Design of Jinzu Bashi.

Diagonal member $T_1-T_4$ stress 22,300 kg T. S.R. = 18.60 cm² net.
1L 130×130×9 = 22.59 − 2.25 = 20.34 cm² net 8 − 22ᶲ rivets for connection.

all other diagonal members use same section as for $T_1-T_4$.

Struts $T_2-T_2$, $T_4-T_4$, ---- $T_{10}-T_{10}$
unsupported length 4.90 − 0.30 = 4.60 meters
least radius of gyration required = 460 ÷ 120 = 3.84 cm.

2Ls 130×130×9 = 45.18 cm²
least radius of gyration 3.96 cm ok.

Approximate weight of Tower Bent.

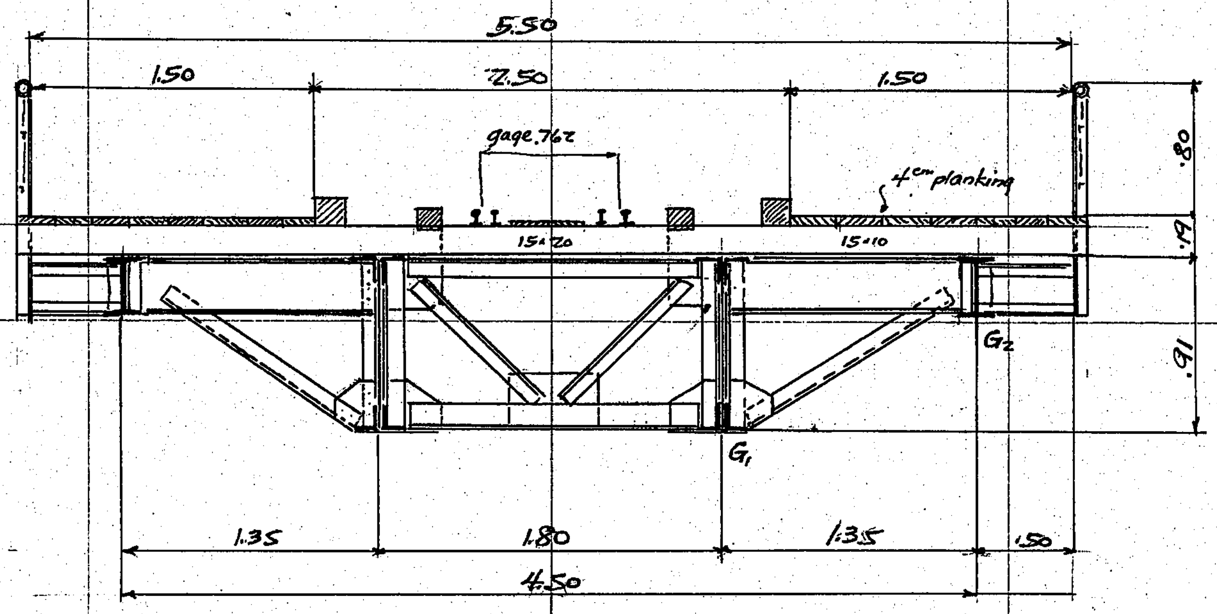
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Column $T_7-T_9$ | 2Ls 300×90 @ 43.80 | × 8.66 | = | 758 | | | |
| | Details | 60% say | = | 455 | | | |
| | | | | | 1213 × 4 | = | 4852 |
| Columns $T_1-T_7$ | 2Ls 300×90 @ 38.10 | × 24.80 | = | 1,890 | | | |
| | Details | say 60% | = | 1,135 | | | |
| | | | | | 3025 × 4 | = | 12,100 |
| Struts | 4Ls 125×75×9 @ 13.5 | × 34.70 | = | 1,873 | | | |
| | 4Ls 150×90×9 @ 16.3 | × 15.30 | = | 997 | | | |
| | Details | say 20% | = | 574 | | | |
| Struts | | | | | 3444 × 2 | = | 6888 |
| | 2Ls 130×130×9 @ 17.7 | × 4.90 | = | 174 | | | |
| | Details | say 10% | = | 17 | | | |
| | | | | | 191 × 10 | = | 1910 |
| Diagonals | 1L 130×130×9 @ 17.7 | × 51.66 | = | 914 | | | |
| | Details | say 10% | = | 91 | | | |
| | | | | | 1005 × 4 | = | 4020 |
| Diagonals | 1L 130×130×9 @ 17.7 | × 9.34 | = | 165 | | | |
| | Details | say 10% | = | 17 | | | |
| | | | | 2 | 182 × 16 | = | 2,910 |
| Vertical tie | 4Ls 75×75×9 @ 9.96 | × 23.85 | = | 950 | | | |
| | Details | say 25% | = | 248 | | | |
| | | | | | 1198 × 2 | = | 2,396 |
| Center strut | 2Ls 130×130×9 @ 17.7 | × 4.9 | = | 174 | | | |
| | Details | say 10% | = | 17 | | | |
| | | | | | 191 × 1 | = | 191 |
| Shoes + anchor bolts say | 4 @ 180 | | | | | = | 720 |
| | | | | | | | 35,987 kg |

Call this 36.00 kg tons.

Design of Jinzu Bashi.

Design of Approach Girder spans.

Cross section of Bridge assumed as shown on sketch below.




scale 1:40.

Deck construction same as for truss span. see on page 1, 2, + 3.

Dead Load due to deck and footwalk.

| | | on girder $G_1$ | | on Girder $G_2$ |
|---|---|---|---|---|
| Track | | 100 | | — |
| foot walk | $100 \times \frac{.3}{1.35}$ | 22 | $100 \times \frac{1.05}{1.35} =$ | 78 |
| handrail | | −20 | $50 \times \frac{1.9}{1.35} =$ | 70 |
| | | 102 kg/m | | 145 kg/m |

center concentration on $G_1$.
$145 \times 6.0 = 870$ kg

Live Load on foot walk.
$600 \times \frac{.25}{1.35} = 110$ kg/m $\quad 600 \times \frac{1.1}{1.35} = 490$ kg/m. $\quad 490 \times 6.0 = 2940$ kg.

Design of Fascia girder $G_2$.

Dead Load

| | |
|---|---|
| Footwalk and handrail | 145 |
| beam assumed | 50 |
| | 195 kg per lin meter. |

Dead Load moment $= 1/8 \times 195 \times 6.0^2 = 878$ kgm.
Dead Load shear $= 1/2 \times 195 \times 6.0 = 585$ kg.

Live Load

Live Load moment $= 1/8 \times 490 \times 6.0^2 = 2202$ kgm
Live Load shear $= 1/2 \times 490 \times 6.0 = 1470$ kg

Summary of moments and shears.

| | moment | shear |
|---|---|---|
| Dead Load | 878 | 585 |
| Live Load | 2202 | 1470 |
| | 3080 kgm | 2055 kg |

Try 1 I $300 \times 150$ @ 48.3 kg $\quad S_m = 633.2$ cm³

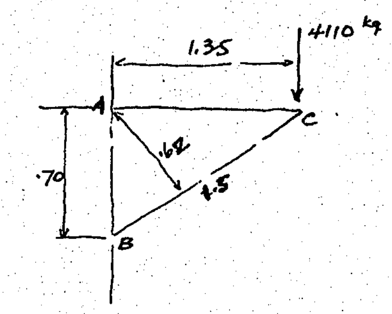
Fibre stress $= \frac{3080 \times 100}{633.2} = 487$ kg/cm²

Allowable unit compression on flange
$= 1100 - 15\frac{l}{b} = 1100 - 15\frac{600}{15} = 500$ kg/cm² ok.

Shear on web $= \frac{2055}{30 \times .8} = 86$ kg/cm² ok.

Design of Jinzu Bashi.

Design of Cantilever bracket to support $G_2$.




D.L. of bracket assumed 60 kg/lin m.
moment = ½ × 60 × 1.35² = 55 kgm.
Shear = 60 × 1.35 = 80 kg.

Dead Load + Live Load due to foot walk.
moment = 4110 × 1.35 = 5545 kgm
Shear = 4110 kg

Summary for moments and shears.

| | moment | shear |
|---|---|---|
| bracket | 55 | 80 |
| foot walk | 5545 | 4110 |
| | 5600 kgm | 4190 kg. |

Stress AC $\frac{5600}{.70}$ = 8,000 kg T
" BC $\frac{5600}{.62}$ = 9,050 kg C

B.C. 1L 90×90×10 = 17.00 cm² gr.
$l/r = \frac{150}{1.71} = 88$ f = 1200 − 5×88 = 760
gross area reqd = $\frac{9050}{760}$ = 11.9 gr ok.

AC use 1[ 300×90 @ 38.1 kg. = 48.57 − 8 = 40.57 net.

Design of Main girder G1.

Dead Load.

| | |
|---|---|
| Foot walk + truck | 102 |
| Lateral bracing say | 20 |
| main girder assumed. | 200 |
| | 322 |

call this 320 kg per lin m.

moment = ⅛ × 320 × 12.0² = 5750 kgm
Shear = ½ × 320 × 12.0 = 1,920 kg

| | |
|---|---|
| floor concentration at center 585×2 = | 1170 |
| bracket | 80 |
| Sway + strut. say | 130 |
| | 1380 kg |

moment = ¼ × 1380 × 12.0 = 4140 kgm
Shear = ½ × 1380 = 690 kg.

Summary of Dead Load moments + shears.

| | moment | shear. |
|---|---|---|
| uniform load | 5750 | 1920 |
| Concentration | 4140 | 690 |
| | 9890 kgm | 2610 kg. |

Live Load assumed 1000 kg per lin m. 1000
impact = $\frac{60}{90+12}$ = .588 588
1588
uniform live load from foot walk = 110
1688 call this 1690 kg per lin m

moment = ⅛ × 1690 × 12.0² = 30400 kgm.
Shear = ½ × 1690 × 12.0 = 10,140 kg

live load concentration from cantilever bracket at center 2 @ 1470 = 2940 kg
moment = ¼ × 2940 × 12.0 = 8820 kgm
Shear = ½ × 2940 = 1,470 kg

Summary for Live Load moments + shears.

| | moments | Shears. |
|---|---|---|
| uniform load | 30,400 | 10,140 |
| concentration | 8,820 | 1,470 |
| | 39,220 kgm | 11,610 kg. |

Design of Jiuzu Bashi.

Summary for Dead Load and Live Load moments and shears.

| | moment | shear |
|---|---|---|
| Dead Load | 9,890 | 2610 |
| Live Load | 39220 | 11,610 |
| | 49,110 kgm | 14,220 kg |

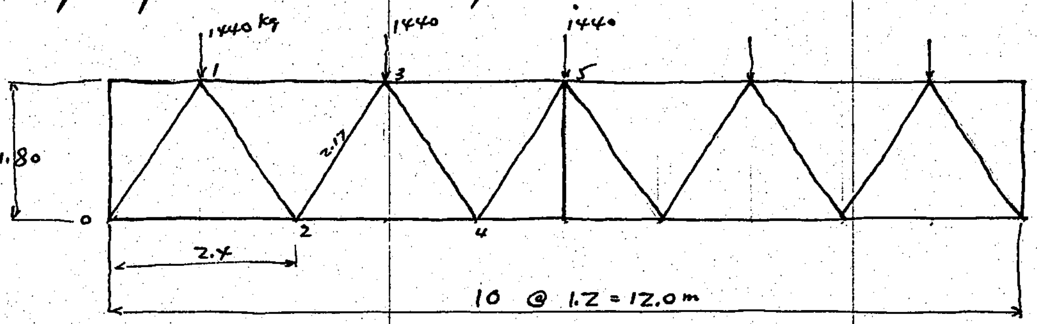
Try web plate 900 × 9  back to back of angles 91.0 cm  1/8 web area = 10.13 cm²
Effective depth = 91.0 − 2×4.07 = 82.86 cm
flange stress = $\frac{49110}{.8286}$ = 59250 kg.

bottom flange area required = 59250 ÷ 1200 = 49.38
$\frac{1}{8}$ web area − 10.13
39.25 cm² net

use 2L 150×150×11 = 63.56 − 11.0 = 52.56 cm² net.
allowable unit compression on top flange = 1100 − 15 × $\frac{240}{30.9}$ = 983 kg/cm² c.

Top flange area required = 59250 ÷ 983 = 60.30
1/8 web area = − 10.13
50.17 cm² gross. < 63.56 ok

Design of Lateral bracings.




wind load assumed 600 kg/lin m.
panel load = 2.4 × 600 = 1440 kg.

Reaction 1440 × 2.5 = 3600 kg
Diagonal length = $\sqrt{1.8^2+1.2^2}$ = 2.17
Coefficient = $\frac{2.17}{1.8}$ = 1.20

max. stress = 3600 × 1.2 = 4320 kg c

1L 90×90×10 = 17.00 gr − 2.5 = 14.50 net.
allowable stress c = $\frac{21000000}{3}\left(\frac{1}{117}\right)^2$ = 510 kg/cm² c

gross area req'd. = $\frac{4320}{510}$ = 8.5 cm² gr  ok.

Approximate weight of structural steel in one span.
Fascia girder. 12.3 @ 50 = 615 kg.

Cantilever bracket 1[

| | | |
|---|---|---|
| 1[ 300×90 @ 38.1 × 1.35 | = | 51. |
| 1L 90×90×10 @ 13.3 × 1.30 | = | 17 |
| details say | = | 32 |
| | | 100 kg |

Sway bracing

| | | |
|---|---|---|
| 4Ls 90×90×10 @ 13.3 × 1.80 | = | 96 |
| 2Ls 75×75×9 × 9.96 × .90 | = | 18 |
| details say | = | 36 |
| | | 150 kg. |

Lateral bracings.

| | | |
|---|---|---|
| 10 Ls 90×90×10 @ 13.3 × 1.80 | = | 239 |
| 10 connections @ 25 | = | 250 |
| | | 489 |
| call this | | 490 kg. |

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CALCULATIONS FOR

Design of Juizu Bashi.

Approximate weight of main girders.

| | | | | | | |
|---|---|---|---|---|---|---|
| Flange | 4Ls | 150×150×11 | @ 24.9 | × 12.30 | = | 1225 |
| web pl. | 1Pl. | 900×9 | @ 63.59 | × 12.30 | = | 783 |
| End stiffs. | 8Ls | 130×130×12 | @ 23.40 | × .89 | = | 167 |
| fills | 4Pls | 260×11 | @ 22.45 | × .60 | = | 54 |
| int. stiffs. | 20Ls | 125×75×9 | @ 13.5 | × .91 | = | 246 |
| center splice | | | say | | = | 140 |
| Rivets | | | say 7% | | = | 188 |
| | | | | | | 2798 |

call this 2800 kg

Summary for structural steel in one span.

| | | | | |
|---|---|---|---|---|
| Frascia girders | 2 @ | 615 | = | 1,230 |
| cantilever brackets | 6 @ | 100 | = | 600 |
| sway bracings | 3 @ | 150 | = | 450 |
| lateral bracings | 1 @ | 490 | = | 490 |
| main girders | 2 @ | 2800 | = | 5600 |
| Shoes and anchor bolts etc say | 4 @ | 140 | = | 560 |
| misc details | | | | 70 |
| | | | | 9,000 kg |

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

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CALCULATIONS FOR

昭和六年二月

三井鑛山會社

神岡軌道橋梁設計々算書

及材料調書

深谷橋梁。船渡谷橋梁及

七百米突附近陸橋

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MADE BY J. Masuda DATE 6-1-31 FILE NO
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CALCULATIONS FOR

Electric Rwy Bridges for Kamioka-Electric Ry Co.

Fukaya - Crossing 30.0 meter clear span and 4m clear beam span

Funawatashi-Dani Bridge 1-30.0 meter clear span on abutments

700 meter Bridge 1-40.0 meter clear span on abutments.

For span length of bridge - 6 panels @ 5.14 = 30.84 meters between end bearings for 30.0 meter clear span and 8 panels @ 5.10 meters = 40.80 meters between end bearings for 40.0 meter clear span. beam span will be framed into truss span

Loading Coupled 8 ton electric traction engines followed by freight cars of 2 tons per lin. meter; track gage being 0.763 meter wide.

Impact allowance $I = L\left(\frac{60}{90+l}\right)$

where $L$ = Live load stress of member

$l$ = loaded length in meter

Wind load On loaded deck 600 kg per lin. meter

Unloaded deck 300 kg " " "

Dead load track 200 kg per lin. meter.

Temperature change 80°F

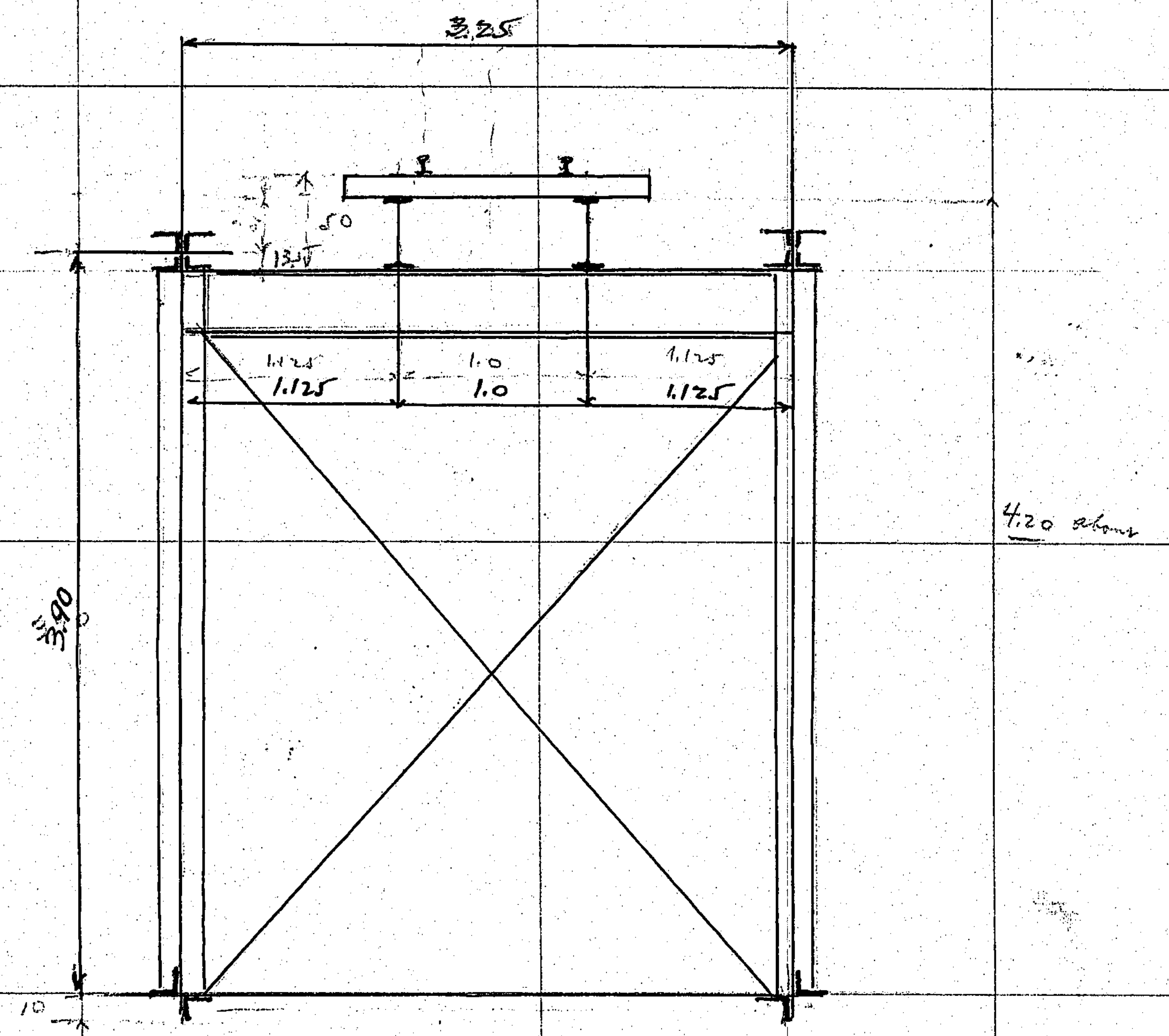
Allowable working strength of material and specification for detail design - Imperial Railway Beaurea's Standard excepting data mentioned below.

Compression members $l/r \leq 120$

For wind stress $l/r \leq 150$

Tension member of truss $l/r \leq 300$

Cross section of bridge assumed as shown below in sketch

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CALCULATIONS FOR

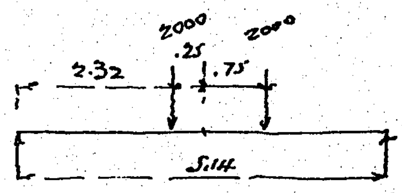
Bridges for Kamioka Electric Ry Co.

Span length 6 @ 5.14 = 30.84 meter between c to c of End bearings.

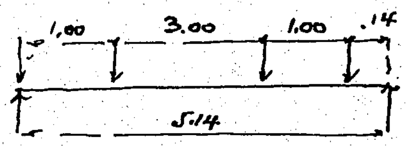
Stringer span length 5.14 meters
Dead Load for one stringer Deck construction 100
stringer assumed 65
165 kg per lin. meter

moment = $\frac{1}{8} \cdot 165 \cdot 5.14^2$ = 548 kgm
shear = $\frac{1}{2} \cdot 165 \cdot 5.14$ = 425 kg

Live Load traction Engines per rail concentration 2000 kg.




Absolute moment = $4000 \cdot \frac{2.32^2}{5.14}$ = 4190
impact $\frac{60}{90+5.14}$ = 63.1% 2645
6835 kgm



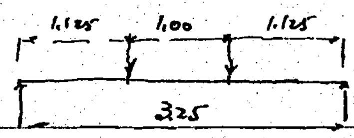

End shear $2000 \cdot \frac{10.56}{5.14}$ = 4110
impact 63.1% 2600
6710 kg.

Summary for moments and shear

| | moment | shear |
|---|---|---|
| Dead Load | 548 | 425 |
| Live Load | 6835 | 6710 |
| | 7383 kgm | 7135 kg |

Try I 350 · 150 @ 58.5 kg per meter $S_m$ = 870.6
Unit stress = $\frac{738300}{870.6}$ = 847.5 kg/cm²
allowable unit stress = $1150 - 15 \cdot \frac{2597}{15}$ = 899 kg/cm²

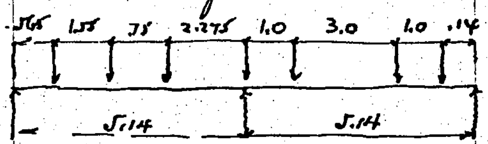
Cross Beam span length 3.25 meters spacing 5.14 meters
Dead Load Concentration at stringer connection 165 · 5.14 = 850




moment = 850 · 1.125 = 955
beam $\frac{1}{8} \cdot 80 \cdot 3.25^2$ = 105
1060 kgm

End shear 850
$\frac{80}{2} \cdot 3.25$ 130
980 kg.

Live Load Traction Engine.
Load on floor beam $1150 \cdot \frac{5.545}{5.14}$ = 1240
$2000 \cdot \frac{10.56}{5.14}$ = 4110
5350




impact $\frac{60}{90+10.28}$ = 59.7% 3190
8540 kg

Live Load moment 8540 · 1.125 = 9600 kgm
Live Load max End shear 8540 kg.

Summary for moment and shear

| | moment | shear |
|---|---|---|
| Dead Load | 1060 | 980 |
| Live Load | 9600 | 8540 |
| | 10660 kgm | 9520 kg |

Try 14 · 6" @ 20.870 kg per ft. $S_m$ = 1030 cm³ (62.946 inch³)
350 · 150
fibre stress = $\frac{1066000}{1030}$ = 1034 kg/cm²
allowable unit stress = $1150 - 15 \cdot \frac{112.5}{15}$ = 1037 kg/cm² OK

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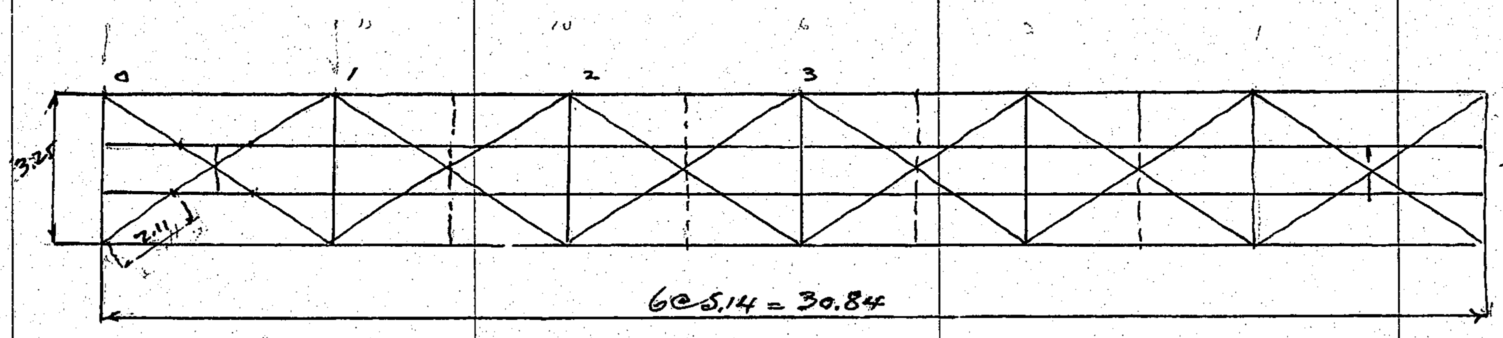
CALCULATIONS FOR

Bridge for Kamioka - Electric Co.

Top Lateral Bracing




Diagonal length

$3.25^2 = 10.56$
$5.14^2 = 26.42$
$6.08 \quad 36.98$

$\frac{6.08}{3.25} = 1.875$

$\frac{5.14}{3.25} = 1.580$

Wind load 600 kg per lin. meter Panel Concentration = 600 × 5.14 = 3080 kg

Diagonal Stress

| Panel | | Shear | | stress | one member | net section | 19φ Rivet | Unit stress | Sr. cm² | Section used |
|---|---|---|---|---|---|---|---|---|---|---|
| 0-1 | 3080 × 15/6 = | 7700 | · 1.875 = | 14450 | 7225 | 6.01 cm² | 3.4 | 460 | 15.75 | 1L 90×90×10 = 17.00 cm² |
| 1-2 | " 10/6 = | 5140 | = | 9650 | 4825 | 4.02 " | 2.3 | 460 | 10.50 | 1L 90×90×10 = 17.00 |
| 2-3 | " 6/6 = | 3080 | | 5780 | 2890 | 2.40 " | 1.4 | 326 | 8.88 | 1L 75×75×9 = 12.69 |

Chord stress

| panel | | |
|---|---|---|
| 0-1 | 3080 × 2.5 × 1.580 | = 12200 |
| 1-2 | × 4.0 | = 19500 |
| 2-3 | × 4.5 | = 21900 |

1L 75×75×9 = 12.69 − 1.98 = 10.71 net $r = 1.44$ $\frac{l}{r} = \frac{211}{1.44} = 146.5$

unit stress $\frac{21.000.000}{3} \times \left(\frac{r}{l}\right)^2 = 326.$

1L 90×90×10 = 17.00 − 2.20 = 14.80 net $r = 1.71$ $\frac{l}{r} = \frac{211}{1.71} = 123.5$

unit stress $\frac{21.000.000}{3} \times \left(\frac{r}{l}\right)^2 = 460.0$

Bottom Lateral Bracing 300 kg per lin. meter
Considering the diagonals ~~work~~ act for tension stress only
Diagonal

| Panel | stress | net section | 19φ Rivet | Section used |
|---|---|---|---|---|
| 0-1 | 7225 T | 6.01 cm² | 3.4 | 1L 75×75×9 = 12.69 |
| 1-2 | 4825 T | 4.02 | 2.3 | " |
| 2-3 | 2890 T | 2.40 | 1.4 | " |

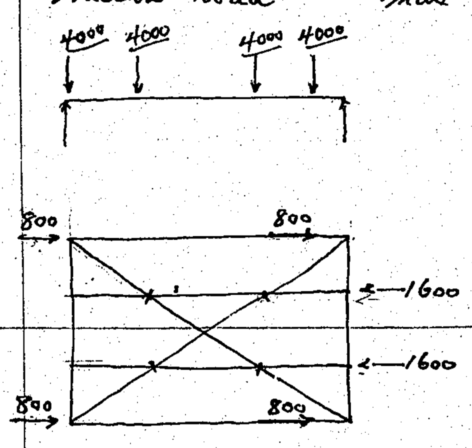
1L 75×75×9 = 12.69 − 1.98 = 10.71 $r = 1.44$, $r = 2.25$

$\frac{l}{r} = \frac{304}{1.44} = 211$

$\frac{l}{r} = \frac{608}{2.25} = 270$

Traction Load max load on one panel 16.000 kg for 2 Rails

Traction load = 16.000 × 0.2 = 3200 kg.




stress in diagonal = $800 \times \frac{6.08}{5.14} = 950$ kg.

Riveting diagonals and stringers at intersection points the traction load will be carried to chord member through diagonal

max stress 7225
950
no of rivets — 8175 ÷ 2126 = 3.8

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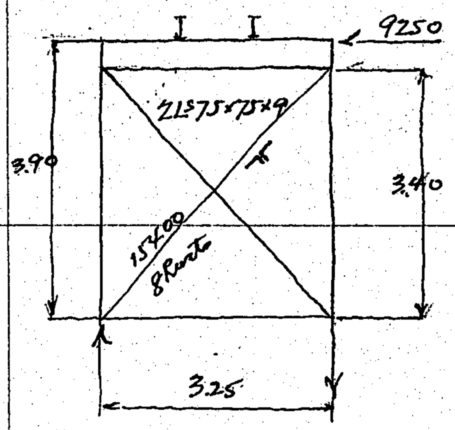
CALCULATIONS FOR

Bridges for Kamioka - Electric Co.

Strut at bottom lateral Bracing at panel points
2L 75×75×9 = 25.38 cm² r = 2.25 l/r = 325/2.25 = 144.5

Sway Bracing at End panel point.




Diagonal length 3.4² = 11.58
3.25² = 10.56
22.14 – 4.70

wind load 600 × 30.84/2 = 9250 kg.

Reaction 9250 × 3.9/3.25 = 11100 kg

Stress in diagonal = 11100 × 4.70/3.40 = 15400 kg. ÷ 1200 = 12.80 net
no of rivet 15400 ÷ 2126 = 7.25 use 8 Rivets for connection.
use 2L 75×75×9 = 25.38 – 3.96 = 21.42 cm² net

Approximate weight of top lateral Bracings

| | | | | | |
|---|---|---|---|---|---|
| 8L 90×90×10 | @ 13.30 | × | 5.70 | = | 606 |
| 4L 75×75×9 | @ 9.96 | × | 5.70 | = | 227 |
| Connections | 14@20 | | | | 280 |
| Splice at intersections | 6@10 | | | | 60 |
| Cross strut and connection | 6@30 | | | | 180 |
| | | | | | 1353 |
| Add for misc | | | | | 47 |
| | | | | | 1400 kg. |

Approximate weight of bottom lateral bracings

| | | | | | | |
|---|---|---|---|---|---|---|
| diagonal | 12L 75×75×9 | @ 9.96 | × | 5.70 | = | 680 |
| strut | 14L 75×75×9 | @ 9.96 | × | 2.95 | = | 411 |
| | Connections | 14@20 | | | = | 280 |
| | Splice at intersections | 6@10 | | | = | 60 |
| | misc say | | | | | 29 |
| | | | | | | 1460 kg |

Approximate weight of Sway Bracing.

End Sway

| | | | | | | |
|---|---|---|---|---|---|---|
| diagonal | 4L 75×75×9 | @ 9.96 | × | 4.30 | = | 172 |
| | Connections | 4 @ 15 | | | = | 60 |
| | misc say | | | | | 8 |
| | | | | | | 240 |
| Center Sway | 2L 75×75×9 | @ 9.96 | × | 4.30 | = | 86 |
| | Connections | 4@15 | | | | 60 |
| | misc say | | | | | 4 |
| | | | | | | 150 |

Total for Sway 2@240 = 480
150
630 kg.

Approximate weight of stringer 2@65 × 31.6 = 4100 kg

Floor Beam 14"×6" @ 68.5 kg × 3.25 = 222
Connection angles etc 13
235 × 7 = 1645 kg.

CALCULATIONS FOR

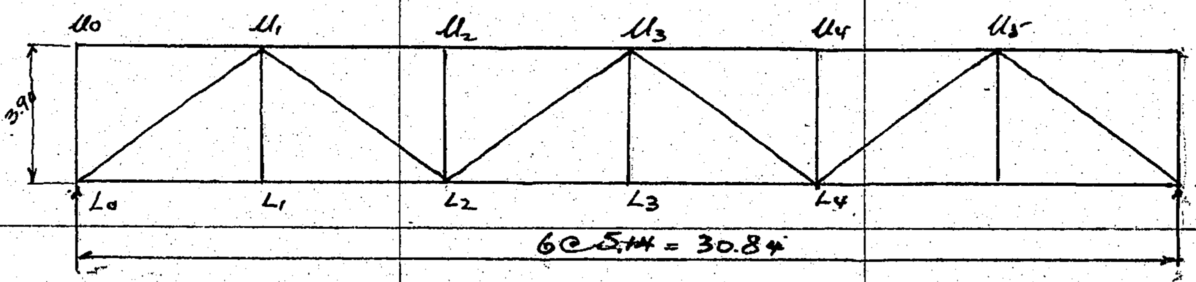
Bridge for Kamioka - Electric Co.

Design of truss  span length $6 @ 5.14 = 30.84$ meters
Height of truss 3.90 meters

Dead Load

| | | |
|---|---|---|
| structural Steel | stringers | 130 |
| | Floor Beam $235 \div 5.14 =$ | 46 |
| | Top Lateral | 46 |
| | Bottom Lateral | 48 |
| | Sway Bracing | 20 |
| | trusses assumed | 610 |
| | | 900 |
| | track construction | 200 |
| | | $1100 \div 2 =$ 550 kg per lin. meter |

Panel Concentration $550 \times 5.14 = 2820$ kg.




Diagonal length
$3.90^2 = 15.21$
$5.14^2 = 26.42$
$41.63 - 6.42$

$\sec\theta = \frac{6.42}{3.90} = 1.645$

$\tan\theta = \frac{5.14}{3.90} = 1.320$

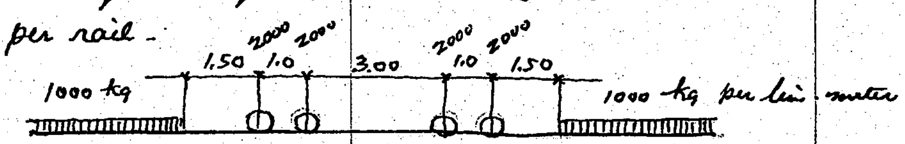
Chord stresses

| | | | | | |
|---|---|---|---|---|---|
| $L_0-L_2$ | 2.50 | × 2820 | × 1.320 | = 9310 | kg T |
| $U_1-U_3$ | 4.00 | | | = 14900 | " C |
| $L_2-L_3$ | 4.50 | | | = 16750 | " T |

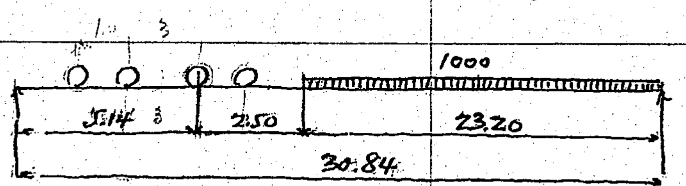
Diagonal Stresses

| | | | | | |
|---|---|---|---|---|---|
| $L_0-U_1$ | 2.50 | × 2820 | × 1.645 | = 11600 | " C |
| $U_1-L_2$ | 1.50 | | | = 6950 | " T |
| $L_2-U_3$ | .50 | | | = 2320 | " C |

Live Load
Chord stresses
traction engine diagram as shown on sketch
Load per rail.




moment at $U_1$, wheel load no 3 at $L_1$




Reaction $8000 \times \frac{27.20}{30.84} = 7050$

$23200 \times \frac{23.20}{30.84 \times 2} = 8720$

15770

moment at $U_1$  $15770 \times 5.14 = 81000$
$4000 \times 3.50 = -14000$
67000

Stress in $L_0-L_2$  $\frac{67000}{3.90} = 17200$ T

For simplicity's sake  Consider the traction engine as uniformly distributed load of 1000 kg per linear meter  panel Concentration = 5140 kg.

| | | | | | Impact |
|---|---|---|---|---|---|
| stress in $L_0-L_2$ | = 2.50 | × 5140 | × 1.320 | = 17000 kg T | 8450 |
| $U_1-U_3$ | 4.00 | | | = 27200 " C | 13500 |
| $L_2-L_3$ | 4.50 | | | = 30500 " T | 15150 |

Impact $= \frac{60}{90 + 30.84} = 49.7\%$

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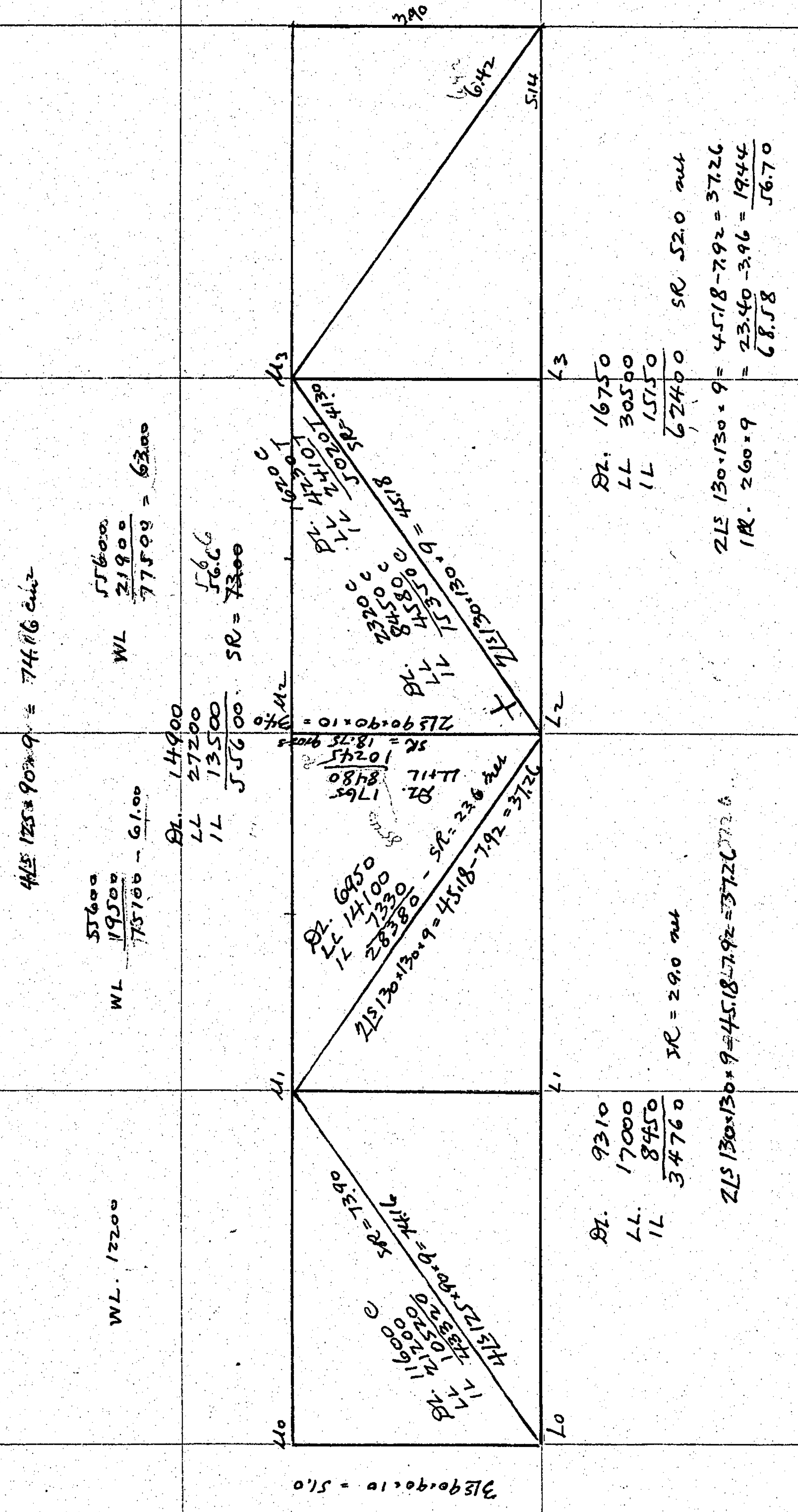
CALCULATIONS FOR

Bridges for Kamioka - Electric Co

Diagonal Stresses Panel Concentration 5140 kg

| | | | | | | | impact | |
|---|---|---|---|---|---|---|---|---|
| $L_0$-$U_1$ | 15/6 | × 5140 | × 1.645 | = | 21200 | C | 49.7 % | 10520 |
| $U_1$-$L_2$ | 10/6 | | | = | 14100 | T | 52.0 | 7330 |
| $L_2$-$U_3$ | 6/6 | | | = | 8450 | C | 54.2 | 4580 |
| $U_3$-$L_4$ | 3/6 | | | = | 4230 | T | 57.0 | 2410 |
| $L_4$-$U_5$ | 1/6 | | | = | 1410 | C | 59.7 | 840 |



増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Bridges for Kamioka Electric Ry Co.

Section of truss members.

Top chord 4Ls 130×130×9 = 90.36  r = 5.60  Unit stress = 1200 − 5 × $\frac{514}{5.60}$ = 741

section required = 55600 ÷ 741 = 75.0 gross. ( See Revised Section ).

Bottom chord

$L_0$-$L_2$  34740 ÷ 1200 = 29.0 cm² net  2Ls 130×130×9 = 45.18 − 7.92 = 37.26 cm² net

$L_2$-$L_3$  62400 ÷ 1200 = 52.0 cm² net  2Ls 130×130×9 = 45.18 − 7.92 = 37.26

1 Pl. 260×9 = 23.40 − 3.96 = 19.44

68.58  56.70

Top chord 4Ls 125×90×9 = 74.16  r = 5.88  Unit stress = 1200 − 5 × $\frac{514}{5.88}$ = 763.0 kg/cm²

Section req'd = 55600 ÷ ~~763~~ 982 = ~~73.00~~ 56.6 gross.  982×1.25 = 1230  1200 − 5 × $\frac{257}{5.88}$ = 982

77500 ÷ 1230 = 63.00

75100 ÷ 1230 = 61.00

Diagonal $L_0$-$U_1$  4Ls 125×90×9 = 74.16  r = 5.88  Unit stress $\frac{21.000.000}{3}$ × $\left(\frac{5.88}{642}\right)^2$ = 586.0 kg/cm²

Section req'd = 43320 ÷ 586 = 73.90 cm²

Diagonal $L_2$-$U_3$  2Ls 130×130×9 = 45.18  r = 5.05 least  $\frac{21.000.000}{3}$ × $\left(\frac{5.05}{642}\right)^2$ = 433.

stress 15350

5020 ÷ 2 = 2510

17860 ÷ 433 = 41.30

Diagonal $U_1$-$L_2$  2Ls 130×130×9 = 45.18 − 7.92 = 37.26

Section req = 28380 ÷ 1200 = 23.60 net

Verticals Dead Load panel load from floor beam 980

truss $\frac{610}{4}$ × 5.14 = 785

1765

Live Load Floor Beam Reaction − 8540

10305 ÷ 547 = 18.90 gross

2Ls 90×90×10 = 34.00  r = 3.43  $\frac{21.000.000}{3}$ × $\left(\frac{3.43}{390}\right)^2$ = 547. kg/cm²

Approximate weight of truss

| | | | | | | |
|---|---|---|---|---|---|---|
| Top chord | 74.16 | × .785 | × 3 | × 5.14 | = | 900 |
| Bottom chord | 45.18 | | × 2 | × 5.14 | = | 364 |
| " | 68.58 | | × | × 5.14 | = | 278 |
| Diagonals | 74.16 | | × | 6.42 | = | 374 |
| " | 45.18 | | × 2 | × 6.42 | = | 455 |
| Verticals | 34.0 | | × 4 | × 3.90 | = | 417 |

2788 × 2 = 5576

Details say 1/3  1860

7436 × 2 = 14872 kg
say 15000

7436 ÷ 30.84 = 240.0 kg per lin. meter.

For 2 trusses say 480.0

Load on shoe  Dead Load 550
Live Load 1000
Impact 497
2047 × $\frac{30.84}{2}$ = 31.500 kg.

Structural Steel in one span ( 30.84 meters ).

| | |
|---|---|
| stringers | 4100 |
| Floor beams | 1645 |
| Top Lateral | 1400 |
| Bottom Lateral | 1460 |
| Sway | 630 |
| trusses say | 15000 |
| Shoes etc | 1000 |
| | 25235 |

call this 25.5 tons

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CALCULATIONS FOR

Bridges for Kamioka Electric Rwy Co

Span Length 8 @ 5.10 = 40.80 meters between c to c of End Bearings

Stringer span length 5.10 meters

| | | | | |
|---|---|---|---|---|
| Dead Load | one stringer | Deck Construction | 100 | |
| | | beam assumed | 65 | |
| | | | 165 | kg per lin. meter |

moment = $\frac{1}{8} \times 165 \times 5.10^2$ = 537. kgm

Shear = $\frac{1}{2} \times 165 \times 5.10$ = 421. kg.

For stringer use 350 × 150 @ 58.5 kg I beam

For Cross beam use 14" × 6" @ 20.87 kg per ft I beam

Load on truss from floor beam

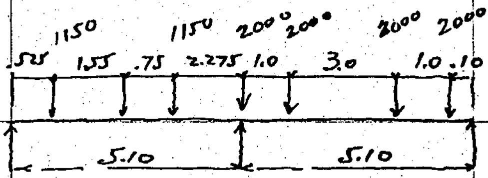
Dead Load 165 × 5.10 = 842

beam $\frac{80}{2} \times 3.25$ = 130

972 kg

Dead Load 972
Live load 8465
9437

Live Load




Load on floor beam

1150 × $\frac{5.425}{5.10}$ = 1225

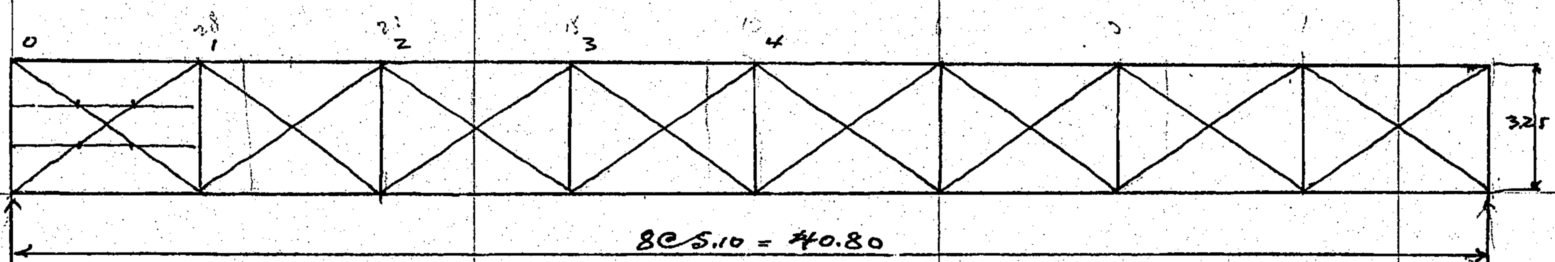
2000 × $\frac{10.40}{5.10}$ = 4080

5305

impact $\frac{60}{90 + 10.2}$ = 597 3160

8465

Top Lateral Bracings




Diagonal length

$3.25^2 = 10.56$
$5.10^2 = 26.00$
6.05 36.56

$\frac{6.05}{3.25} = 1.860$

$\frac{5.10}{3.25} = 1.570$

wind load 600 kg per lin. meter Panel Concentration 600 × 5.10 = 3060 kg.

Diagonal Stresses

| Panel | | shear | | stress | one member | net section | 19φ Rivet | Unit stress | | Section used |
|---|---|---|---|---|---|---|---|---|---|---|
| 0-1 | 3060 × $\frac{28}{8}$ = | 10800 | × 1.86 = | 20100 | 10050 | 8.38 | 4.7 | 572 | 17.60 | 1L 125 × 90 × 9 = 18.54 |
| 1-2 | $\frac{21}{8}$ = | 8090 | | 15050 | 7525 | 6.26 | 3.5 | 460 | 16.35 | 1L 90 × 90 × 10 = 17.0 |
| 2-3 | $\frac{15}{8}$ | 5770 | | 10750 | 5375 | 4.48 | 2.5 | 460 | 11.70 | 1L 90 × 90 × 10 = 17.0 |
| 3-4 | $\frac{10}{8}$ | 3850 | | 7160 | 3580 | 2.98 | 1.7 | 326 | 10.98 | 1L 75 × 75 × 9 = 12.69 |

Chord Stresses

| Panel | | | | |
|---|---|---|---|---|
| 0-1 | 3060 × 3.5 × 1.57 | = | 16800 | |
| 1-2 | 6.0 | = | 28800 | |
| 2-3 | 7.5 | = | 36000 | |
| 3-4 | 8.0 | = | 38400 | |

1L 125 × 90 × 9 = 18.54 r = 1.90

$\frac{21000.000}{3} \times \left(\frac{1.90}{2.10}\right)^2 = 572$

Bottom Lateral Bracings 300 kg per lin. meter

Diagonals for tension stress only

Diagonal

| Panel | stress | Section | 19φ Rivet | Section used |
|---|---|---|---|---|
| 0-1 | 10050 T | 8.38 | 4.7 | 1L 90 × 90 × 10 = 17.00 − 2.2 = 14.80 net |
| 1-2 | 7525 | 6.26 | 3.5 | 1L 75 × 75 × 9 = 12.69 − 1.98 = 10.71 net |
| 2-3 | 5375 | 4.48 | 2.5 | " |
| 3-4 | 3580 | 2.98 | 1.7 | " |

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CALCULATIONS FOR

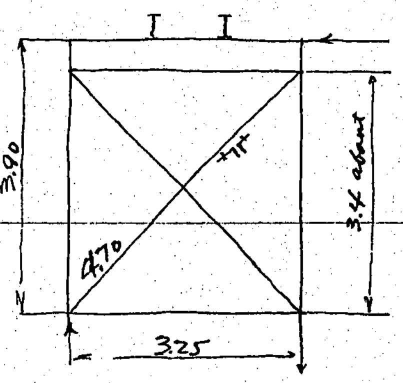
Bridges for Kamioka Electric Ry Co

Bottom Lateral Strut at Panel Points

2Ls 75×75×9 = 25.38 cm² r = 2.25 l/r = 325/2.25 = 144.5

Sway Bracing at End Panel Point

wind load 600 × 40.8/2 = 12250 kg




Reaction 12250 × 3.90/3.25 = 14700 stress = 14700 × 4.70/3.40 = 20300

Section required 20300 ÷ 1200 = 16.90 net

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

no of Rivet 19φ - 10

Approximate weight of top lateral bracings

| | | | | |
|---|---|---|---|---|
| 4Ls 125×90×9 | @ 14.60 | × 5.70 | = | 334 |
| 8Ls 90×90×10 | @ 13.30 | × 5.70 | = | 606 |
| 4Ls 75×75×9 | @ 9.96 | × 5.70 | = | 227 |
| connections | 18 @ 20 | | = | 360 |
| splice at intersections | 8 @ 10 | | = | 80 |
| Cross struts and connections | 6 @ 30 | | = | 180 |
| misc say | | | | 53 |
| | | | | 1840 kg |

Approximate weight of bottom lateral bracings

| | | | | | |
|---|---|---|---|---|---|
| diagonal | 4Ls 90×90×10 | @ 13.30 | × 5.70 | = | 303 |
| | 12 Ls 75×75×9 | @ 9.96 | × 5.70 | = | 680 |
| strut | 18 Ls 75×75×9 | @ 9.96 | × 2.95 | = | 530 |
| | connections | 18 @ 20 | | | 360 |
| | splice at intersections | 8 @ 10 | | | 80 |
| | misc say | | | | 47 |
| | | | | | 2000 kg. |

Approximate weight of Sway Bracings

Same as for 30.0 meter span

| | | | | |
|---|---|---|---|---|
| End sways | 2 @ | 240 | = | 480 |
| Inter sways | 2 @ | 150 | = | 300 |
| | | | | 780 kg |

Approximate weight of stringers 2 @ 65 × 40.8 = 5300 kg

Floor Beams 9 @ 235 = 2120 kg.

Design of Truss span length 8@5.10 = 40.80 meters
truss height 3.90 meters

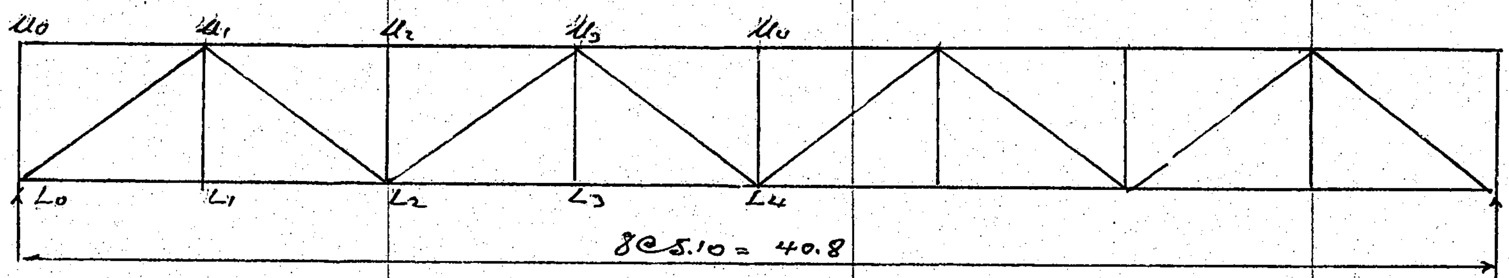
Dead Load

| | |
|---|---|
| structural steel stringers | 130 |
| Floor Beam | 46 |
| Top Lateral | 46 |
| Bottom Lateral | 49 |
| Sway Bracings | 20 |
| trusses assumed | 680 |
| | 971 |
| track Construction | 200 |
| | 1171 ÷ 2 = 585 kg per lin. meter |

Panel Concentration 585 × 5.10 = 2980 kg.

CALCULATIONS FOR

Bridge for Kamioka Electric Ry Co




Diagonal length

$3.90^2$ = 15.21 Secθ = $\frac{6.40}{3.90}$ = 1.645

$5.10^2$ = 26.00

41.21 − 6.40 tanθ = $\frac{5.10}{3.90}$ = 1.310

| Chord stresses | | | | Diagonal members | | | |
|---|---|---|---|---|---|---|---|
| L0-L2 | 3.50 × 2980 × 1.310 = | 13700 | T | L0-U1 | 3.50 × 2980 × 1.645 = | 17200 | C |
| U1-U3 | 6.00 | 23400 | C | U1-L2 | 2.50 | 12300 | T |
| L2-L4 | 7.50 | 29300 | T | L2-U3 | 1.50 | 7350 | C |
| U3-U4 | 8.00 | 31200 | C | U3-L4 | .50 | 2450 | T |

Live Load traction engine 1000 kg per lin meter throughout

Panel Concentration 5100 kg.

Chord Stresses

impact $\frac{60}{90+40.8}$ 46%

| | | | |
|---|---|---|---|
| L0-L2 | 3.50 × 5100 × 1.310 | = 23400 T | 10800 T |
| U1-U3 | 6.00 | = 40000 C | 18400 C |
| L2-L4 | 7.50 | = 50000 T | 23000 T |
| U3-U4 | 8.00 | = 53500 C | 24600 C |

Diagonal Stresses

| | | | | impact |
|---|---|---|---|---|
| L0-U1 | 28/8 × 5100 × 1.645 | = 29400 | 46.% | 13500 |
| U1-L2 | 21/8 | = 22000 | 47.75 | 10500 |
| L2-U3 | 15/8 | = 15700 | 49.80 | 7820 |
| U3-L4 | 10/8 | = 10500 | 52.00 | 5460 |
| L4-U5 | 6/8 | = 6300 | 54.30 | 3420 |

Load on shoe

Dead Load 585.0
live load 1000.0
impact 460.0

2045 × $\frac{40.8}{2}$ = 41800 kg.

Load on verticals

Dead Load from floor beam 972
truss $\frac{680}{4}$ × 5.10 = 868

1840

Live load floor beam reaction with impact 8465

10305 kg

Section of truss members.

Top chord

4 ∟S 150 × 90 × 9 = 83.38 × $5.56^2$ + 18.72 = 4450

1 Pl 300 × 12 = 36.00

119.38

310

$r = \sqrt{\frac{4450}{119.38}}$ = 6.10

unsupported length $\frac{5.10}{2}$ = 2.55 1200 − 5 × $\frac{2.55}{6.10}$ = 991 kg/cm²

for wind load 991 × 1.25 = 1240 kg/cm²

max stress 147700 ÷ 1240 = 119.0 cm²

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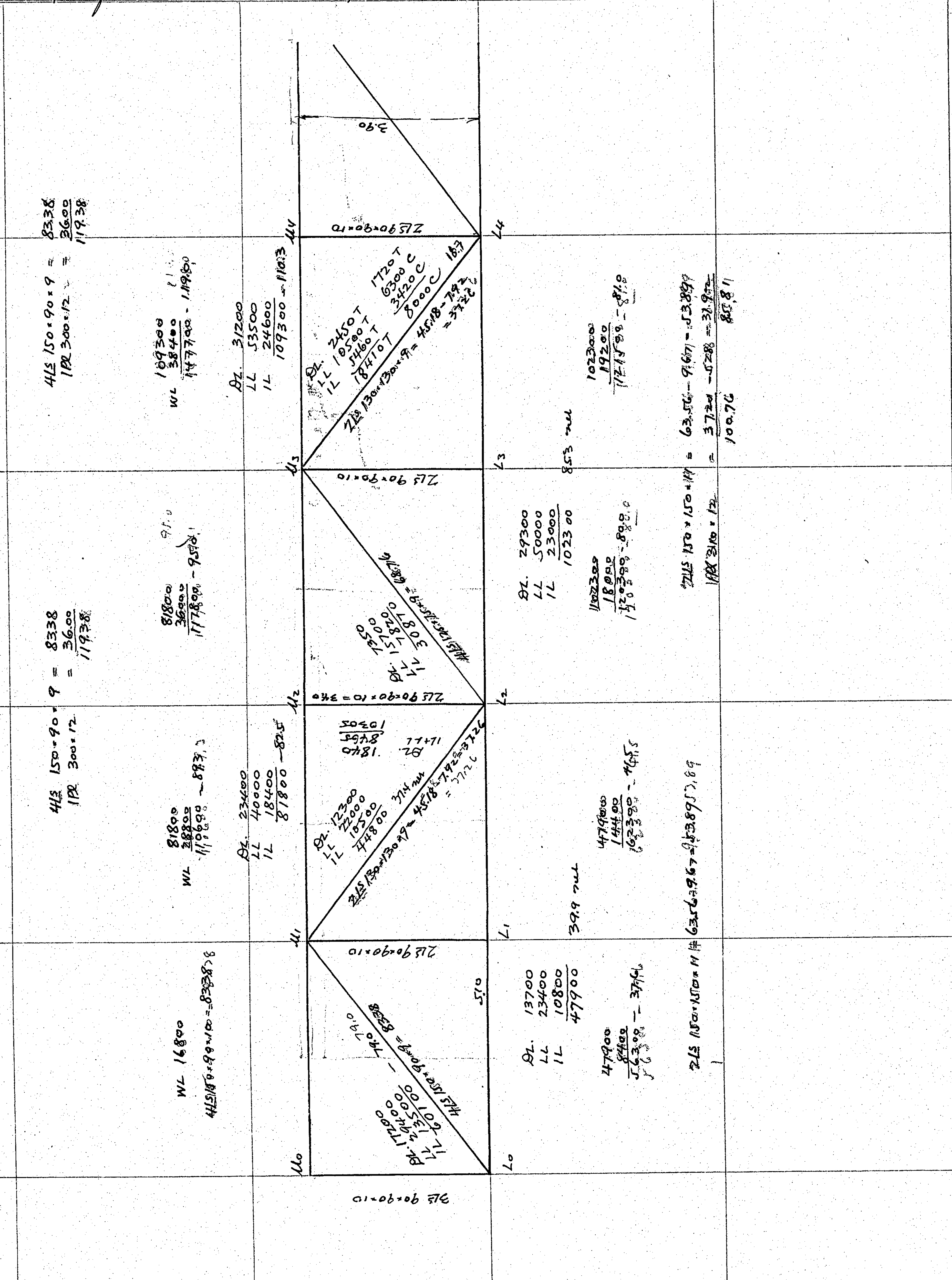
CHECKED BY ______ DATE ______ **PAGE NO** *11*

CALCULATIONS FOR

*Bridges for Kamioka-Electric Ry Co*



CALCULATIONS FOR

Bridges for Kamioka Electric Ry Co

$U_2-U_3$ 117800 ÷ 1240 = 95.0 cm² including wind stress
$U_1-U_2$ 110600 ÷ 1240 = 89.3 "

$U_1-U_2-U_3$ 81800 ÷ 991 = 82.5 cm² without wind stress.

Bottom chord $L_0-L_1-L_2$ 47900 ÷ 1200 = 39.9 net 2L 150×150×11 = 63.56 - 9.67 = 53.89
$L_0-L_1$ 56300 ÷ 1500 = 37.6 net
$L_1-L_2$ 62300 ÷ 1500 = 41.5 net

$L_2-L_3-L_4$ 102300 ÷ 1200 = 85.3 net 2L 150×150×11 = 63.56 - 9.67 = 53.89
$L_2-L_3$ 120300 ÷ 1500 = 80.0 net 1PL 310×12 = 37.20 - 5.28 = 31.92
$L_3-L_4$ 121500 ÷ 1500 = 81.0 net 85.81

End Post
4L 150×90×9 = 83.38 r = 7.31 $1200 - 5 \times \frac{641}{7.31} = 762$ kg/cm²
SR = 60100 ÷ 762 = 79.0 cm²

$U_1-L_2$ 44800 ÷ 1200 = 37.4 cm² net
2L 130×130×9 = 45.18 - 7.92 = 37.26 cm² net

$L_2-U_3$
4L 125×75×9 = 68.76 r = 6.20 $\frac{21000.000}{3} \times \left(\frac{6.20}{641}\right)^2 = 655$ kg/cm²
SR = 30870 ÷ 655 = 47.0

$U_3-L_4$ 18410
4000
22410 ÷ 1200 = 18.7 net ┼ 2L 130×130×9 = 45.18 - 7.92 = 37.26 net

Verticals 10305 ÷ 547 = 23.8 gross ┼ 2L 90×90×10 = 34.0

Approximate weight of truss

| | | | | | |
|---|---|---|---|---|---|
| Top chord | 119.38 | × .785 | × 3 × 5.10 | = | 1430 |
| " | 83.38 | × | 5.10 | = | 334 |
| Bottom chord | 63.56 | × | 2 × 5.10 | = | 510 |
| | 100.76 | × | 2 × 5.10 | = | 805 |
| Diagonals | 83.38 | × | 6.41 | = | 420 |
| " | 45.18 | | 2 × 6.41 | = | 455 |
| " | 68.76 | | × 6.41 | = | 346 |
| Verticals | 34.0 | | 5 × 3.90 | = | 520 |

4820 × 2 = 9640
Details 1/3 3210
12850

12850 ÷ 40.8 = 315 kg per lin. meter
For 2 trusses 25700 630 " " " "

Structural Steel in 40.0 meter span

| | |
|---|---|
| Stringers | 5300 |
| Floor Beams | 2120 |
| Top Laterals | 1840 |
| Bottom Laterals | 2000 |
| Sways | 780 |
| Trusses | 25700 |
| Shoes &c | 1500 |
| | 39240 kg. call this 39.5 tons |

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Bridges for Kamioka Electric Co.

I beam span over Highway
span length 5.06 meter about use 5.14 meter span

Stresses same as for ordinary stringer for 30.0 meter truss span.
One end will be fixed over cross beam and other end slide on abutment
else. 350 × 150 I @ 58.5 kg

Approximate weight 130 × 5.5 = 715
misc 35
750 kg.

Summary steel

| | | | |
|---|---|---|---|
| Fukaya Bridge | 30.0 meter span | 25.5 | |
| | 4.0 " " | .75 | |
| | | | 26.25 |
| Funawatashi tani Bridge | 30.0 meter span | | 25.50 |
| 700 meter Crossing | 40.0 meter span | | 39.50 |
| | | | 91.25 tons |

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CALCULATIONS FOR

Bridges for Kamiaka Electric Ry Co.

weight of 30.0 meter truss span

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Top chord | | | | | | | | | |
| $U_0-U_1$ | 8Ls | 125×90×9 | @ | 14.60 | × | 4.740 | = | 553.0 | |
| Gusset | 2Pls. | 370×10 | @ | 29.05 | × | .625 | = | 36.4 | |
| tie Pls. | 8Pls. | 190×10 | @ | 14.92 | × | .26 | = | 31.1 | |
| $U_1-U_2$ | 8Ls | 125×90×9 | @ | 14.60 | × | 6.540 | = | 765.0 | |
| gusset | 2Pls. | 625×10 | @ | 49.06 | × | 1.34 | = | 131.0 | |
| " | 2Pls. | 400×10 | @ | 31.40 | × | .625 | = | 39.3 | |
| tie Pls. | 10Pls. | 190×10 | @ | 14.92 | × | .26 | = | 38.8 | |
| $U_2-U_3-U_2$ | 4Ls | 125×90×9 | @ | 14.60 | × | 8.48 | = | 495.0 | |
| gusset | 1Pl | 625×10 | @ | 49.06 | × | 1.16 | = | 57.0 | |
| tie Pls. | 8Pls. | 190×10 | @ | 14.92 | × | .26 | = | 31.0 | |
| Splice at $U_1$ | 2Pls. | 260×9 | @ | 18.37 | × | .40 | = | 14.7 | |
| | 4Pls. | 110×9 | @ | 7.77 | × | .40 | = | 12.4 | |
| | 2Pls. | 260×10 | @ | 20.41 | × | .40 | = | 16.3 | |
| | 4Pls. | 230×9 | @ | 16.25 | × | .40 | = | 26.0 | |
| | 4fill | 80×9 | @ | 5.65 | × | .40 | = | 9.1 | |
| Splice at $U_2$ | 4Pls. | 260×9 | @ | 18.37 | × | .60 | = | 43.0 | |
| | 2Pls. | 260×10 | @ | 20.41 | × | .60 | = | 24.5 | |
| | 4Pls. | 230×9 | @ | 16.25 | × | .38 | = | 24.7 | |
| | 4fill | 80×9 | @ | 5.65 | × | .38 | = | 8.6 | |
| Bracket for FB | 7Ls | 90×75×9 | @ | 11.00 | × | .25 | = | 19.3 | |
| | 7fill | 75×9 | @ | 5.30 | × | .08 | = | 3.0 | |
| | 7Pls. | 250×9 | @ | 17.66 | × | .30 | = | 37.0 | |
| | 14Ls | 75×75×9 | @ | 9.96 | × | .18 | = | 25.1 | |
| | | | | | | | | | 2442.3 ✓ |
| Bottom chord | | | | | | | | | |
| $L_0-L_2$ | 4Ls | 130×130×9 | @ | 17.70 | × | 9.12 | = | 646.0 | |
| Gusset | 2Pls. | 700×10 | @ | 54.95 | × | .96 | = | 105.0 | |
| tie Pls. | 10Pls. | 190×10 | @ | 14.92 | × | .26 | = | 38.8 | |
| " | 2Pls. | 260×10 | @ | 20.41 | × | .46 | = | 18.8 | |
| Stiffener | 4Ls | 125×90×9 | @ | 14.60 | × | .60 | = | 35.0 | |
| | 4Ls | 125×90×9 | @ | 14.60 | × | .70 | = | 41.0 | |
| | 2Ls | 90×90×10 | @ | 13.30 | × | .70 | = | 18.6 | |
| fill | 4Pls. | 190×15 | @ | 22.37 | × | .50 | = | 44.7 | |
| | 4Ls | 150×150×15 | @ | 33.60 | × | .40 | = | 53.7 | |
| Sole Pl | 2Pls. | 350×30 | @ | 82.43 | × | .40 | = | 66.0 | |
| $L_2-L_3-L_2$ | 2Ls | 130×130×9 | @ | 17.70 | × | 12.20 | = | 431.0 | |
| " | 1Pl | 260×10 | @ | 20.41 | × | 9.12 | = | 186.0 | |
| Splice | 2Pls. | 260×10 | @ | 20.41 | × | 0.72 | = | 29.4 | |
| | 4Ls | 130×130×9 | @ | 17.70 | × | .72 | = | 51.0 | |
| | 4Pls. | 120×12 | @ | 11.30 | × | .72 | = | 32.5 | |
| | | | | | | | | | 1797.5 ✓ |
| Diagonals | | | | | | | | | |
| $L_0-U_1$ | 8Ls | 125×90×9 | @ | 14.60 | × | 5.80 | = | 678.0 | |
| tie Pls. | 10Pls. | 190×10 | @ | 14.92 | × | .26 | = | 38.8 | |
| lug | 16Ls | 90×90×10 | @ | 13.30 | × | .18 | = | 38.3 | |
| $U_1-L_2$ & $L_2-U_3$ | 8Ls | 130×130×9 | @ | 14.60 | × | 5.90 | = | 690.0 | |
| tie Pls. | 16Pls. | 190×10 | @ | 14.92 | × | .26 | = | 62.0 | |
| | | | | | | | | | 1507.1 ✓ |
| Verticals | | | | | | | | | |
| $L_0-U_0$ | 6Ls | 90×90×10 | @ | 13.30 | × | 3.90 | = | 311.0 | |
| tie Pls. | 6Pls. | 190×10 | @ | 14.92 | × | .19 | = | 17.0 | |
| " | 8Pls. | 190×9 | @ | 13.42 | × | .19 | = | 20.4 | |
| Gusset | 4Pls. | 300×9 | @ | 21.20 | × | .45 | = | 38.2 | |
| Conn Ls | 4Ls | 75×75×9 | @ | 9.96 | × | .30 | = | 12.0 | |

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CALCULATIONS FOR

Bridges for Kamioka- Electric Ry Co.

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Vertical Continued | | | | | | | | |
| U1-L1, U2-L2 - U3-L3 | 10Ls | 90×90×10 | @ | 13.30 | × | 3.64 | = | 485.0 |
| tie Pls. | 10Pls. | 190 × 10 | @ | 14.92 | × | .19 | = | 28.4 |
| " | 5Pls. | 190 × 9 | @ | 13.42 | × | .19 | = | 12.7 |
| Gusset | 10Pls. | 300 × 9 | @ | 21.20 | × | .45 | = | 95.5 |
| | | | | | | | | 1020.2 ✓ |
| | | Total weight for one truss 6767.1 ✓ | | | | | | |
| | | for two trusses 13534.2 kg. ✓ | | | | | | |
| Top Lateral Bracings | | | | | | | | |
| Diagonal | 4Ls | 90×90×10 | @ | 13.30 | × | 5.60 | = | 298.0 |
| | 8Ls | 90×90×10 | @ | " | × | 2.73 | = | 290.0 |
| | 2Ls | 75×75×9 | @ | 9.96 | × | 5.60 | = | 111.5 |
| | 4Ls | 75×75×9 | @ | 9.96 | × | 2.73 | = | 109.5 |
| Gusset center | 4Pls. | 180 × 9 | @ | 12.72 | × | .62 | = | 31.6 |
| " " | 2Pls. | 180 × 9 | @ | 12.72 | × | .50 | = | 12.7 |
| Gusset side | 4Pls | 300 × 9 | @ | 21.20 | × | .55 | = | 46.6 |
| | 4Pls. | 300 × 9 | @ | 21.20 | × | 1.00 | = | 84.8 |
| | 6Pls. | 300 × 9 | @ | 21.20 | × | .90 | = | 114.5 |
| | | | | | | | | 1099.2 ✓ |
| Bottom Lateral Bracings | | | | | | | | |
| Diagonals | 6Ls | 75×75×9 | @ | 9.96 | × | 5.60 | = | 334.0 |
| | 12Ls | 75×75×9 | @ | 9.96 | × | 2.73 | = | 326.0 |
| Gusset (center) | 6Pls | 180 × 9 | @ | 12.72 | × | .50 | = | 38.1 |
| (side) | 4Pls | 300 × 9 | @ | 21.20 | × | .45 | = | 38.2 |
| | 4Pls. | 300 × 9 | @ | 21.20 | × | .90 | = | 76.5 |
| | 6Pls. | 250 × 9 | @ | 17.66 | × | .90 | = | 95.5 |
| Struts (bottom) | 14Ls | 75×75×9 | @ | 9.96 | × | 3.24 | = | 453.0 |
| | | | | | | | | 1361.3 ✓ |
| Sway Frames | | | | | | | | |
| Diagonals | 10Ls | 75×75×9 | @ | 9.96 | × | 4.40 | = | 438.0 |
| fills | 3Pls. | 150 × 9 | @ | 10.60 | × | .15 | = | 4.8 |
| | | | | | | | | 442.8 ✓ |
| Struts on Top chord | | | | | | | | |
| strut | 8Ls | 75×75×9 | @ | 9.96 | × | 1.12 | = | 89.2 |
| Conn Ls | 16Ls | 75×75×9 | @ | 9.96 | × | .24 | = | 38.2 |
| " Pls. | 16Pls | 200 × 9 | @ | 14.13 | × | .24 | = | 54.2 |
| fills | 8Pls. | 75 × 9 | @ | 5.30 | × | .09 | = | 3.8 |
| | | | | | | | | 185.4 ✓ |
| Cross Beams | | | | | | | | |
| | 7Is | 14"×6" | @ | 68.50 | × | 3.23 | = | 1549.0 |
| Conn Ls | 14Ls | 90×90×10 | @ | 13.30 | × | .30 | = | 55.9 |
| | | | | | | | | 1604.9 ✓ |
| Stringers | | | | | | | | |
| intermediate | 8Is | 350 × 150 | @ | 58.50 | × | 5.14 | = | 2405.0 |
| ends | 4Is | 350 × 150 | @ | " | × | 5.40 | = | 1264.0 |
| Conn Pls. seat | 10Pls. | 150 × 9 | @ | 10.60 | × | .30 | = | 31.8 |
| " " web | 20Pls. | 300 × 9 | @ | 21.20 | × | .25 | = | 106.0 |
| struts | 6Ls | 75×75×9 | @ | 9.96 | × | .84 | = | 50.2 |
| Gussets | 12Pls. | 130 × 9 | @ | 9.19 | × | .30 | = | 33.1 |
| | | | | | | | | 3890.1 ✓ |

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CALCULATIONS FOR

Bridges for Kamioka Electric Ry Co

Summary of Structural Steel in 30.0 meter truss span

| | | | |
|---|---|---|---|
| main trusses | 2 @ 6767.1 | | = 13534.2 |
| Top Lateral Bracings | | | 1099.2 |
| Bottom " " | | | 1361.3 |
| Sway Frames | | | 442.8 |
| Top Strut | | | 185.4 |
| Cross Beams | | | 1604.9 |
| Stringers with Strut | | | 3890.1 |
| Shoes & anchor bolts etc | | say | 1200.0 |
| Rivet head & misc. | | say | 1680.0 |
| | | | 24997.9 kg |
| | | Call this | 25.00 tons |

Beam Span on Highway

| | | | | | | |
|---|---|---|---|---|---|---|
| main beam | 2 I3 | 350 × 150 | @ 58.50 | × 5.40 | = | 630.0 |
| connection Pls. | 2 Pls. | 150 × 9 | @ 10.60 | × .30 | = | 6.4 |
| " | 4 Pls. | 300 × 9 | @ 21.20 | × .25 | = | 21.2 |
| struts | 4 L3 | 75 × 75 × 9 | @ 9.96 | × .84 | = | 33.5 |
| Gusset | 4 Pls. | 130 × 9 | @ 9.19 | × .30 | = | 11.0 |
| lattice | 8 Pls. | 60 × 9 | @ 4.24 | × .45 | = | 15.3 |
| Sole Pls | 2 Pls | 200 × 19 | @ 29.83 | × .25 | = | 14.9 |
| bed Pls. | 2 Pls | 250 × 25 | @ 49.06 | × .25 | = | 24.5 |
| Anchor bolts | | | | | | 9.6 |
| | | Rivet & misc say | | | | 33.6 |
| | | | | | | 800.0 kg. |

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Bridges for Kamioka Electric Ry Co.

Structural Steel in 40.0 meter truss span

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Top Chord | | | | | | | |
| U0-U1 | 8Ls | 150 × 90 × 9 | @ | 16.30 | × 4.70 | = | 613.0 |
| gusset | 2Pls | 370 × 12 | @ | 29.05 | × .67 | = | 38.9 |
| tie Pls. | 8Pls. | 190 × 12 | @ | 14.92 | × .30 | = | 35.8 |
| U1-U3 | 8Ls | 150 × 90 × 9 | @ | 16.30 | × 10.00 | = | 1304.0 |
| web | 2Pls | 300 × 12 | @ | 28.26 | × 8.77 | = | 495.6 |
| gusset | 2Pls | 670 × 12 | @ | 63.11 | × 1.50 | = | 189.4 |
| U3-U4-U3 | 4Ls | 150 × 90 × 9 | @ | 16.30 | × 11.56 | = | 754.0 |
| web | 1Pl | 300 × 12 | @ | 28.26 | × 8.92 | = | 252.0 |
| gusset | 2Pls | 670 × 12 | @ | 63.11 | × 1.32 | = | 166.6 |
| splice cover | 2Pls. | 310 × 9 | @ | 21.90 | × .50 | = | 21.9 |
| side | 4Pls. | 270 × 9 | @ | 19.08 | × .36 | = | 27.5 |
| | 4fill | 120 × 9 | @ | 8.48 | × .36 | = | 12.2 |
| side | 4Pls. | 270 × 9 | @ | 19.08 | × .60 | = | 45.8 |
| | 4fill | 120 × 9 | @ | 8.48 | × .60 | = | 20.4 |
| cover U3 | 4Pls. | 310 × 9 | @ | 21.90 | × .40 | = | 35.0 |
| | 8Ls | 150 × 90 × 9 | @ | 16.30 | × .86 | = | 112.0 |
| Side Pl. | 4Pls | 120 × 9 | @ | 8.48 | × .50 | = | 17.0 |
| | 4Pls. | 270 × 9 | @ | 19.08 | × .60 | = | 45.8 |
| | 4fill | 120 × 9 | @ | 8.48 | × .60 | = | 20.4 |
| Bracket for FB | 9Ls | 90 × 75 × 9 | @ | 11.00 | × .292 | = | 28.9 |
| | 9fill | 75 × 9 | @ | 5.30 | × .12 | = | 6.7 |
| | 9Pls. | 290 × 9 | @ | 20.49 | × .30 | = | 55.3 |
| | 18Ls | 75 × 75 × 9 | @ | 9.96 | × .18 | = | 32.3 |
| | | | | | | | 4330.5 |
| Bottom Chord | | | | | | | |
| L0-L2 | 4Ls | 150 × 150 × 11 | @ | 24.90 | × 9.25 | = | 921.0 |
| gusset | 2Pls | 700 × 12 | @ | 65.94 | × .96 | = | 126.5 |
| tie | 14Pls. | 190 × 12 | @ | 17.90 | × .31 | = | 77.7 |
| stiffener | 4Ls | 125 × 90 × 9 | @ | 14.60 | × .60 | = | 35.0 |
| | 4Ls | 125 × 90 × 9 | @ | 14.60 | × .70 | = | 40.9 |
| | 2Ls | 90 × 90 × 10 | @ | 13.30 | × .70 | = | 18.6 |
| fill | 4Pls. | 190 × 15 | @ | 22.37 | × .55 | = | 49.2 |
| | 4Ls | 150 × 150 × 15 | @ | 33.60 | × .45 | = | 60.4 |
| Sole Pl. | 2Pls. | 350 × 30 | @ | 82.42 | × .45 | = | 74.2 |
| L2-L3 | 4Ls | 150 × 150 × 11 | @ | 24.90 | × 6.64 | = | 662.0 |
| web | 2Pls. | 310 × 12 | @ | 29.20 | × 5.80 | = | 338.5 |
| gusset | 2Pls. | 700 × 12 | @ | 65.94 | × 1.34 | = | 176.6 |
| L3-L4-L3 | 2Ls | 150 × 150 × 11 | @ | 24.90 | × 8.60 | = | 428.0 |
| web | 1Pl | 310 × 12 | @ | 29.20 | × 6.32 | = | 184.5 |
| gusset | 1Pl | 600 × 12 | @ | 56.52 | × 1.26 | = | 71.2 |
| splice (L2) | 4Ls | 150 × 150 × 11 | @ | 24.90 | × .96 | = | 95.6 |
| fill | 2Pls. | 310 × 12 | @ | 29.20 | × .48 | = | 28.0 |
| web splice | 4Pls. | 135 × 15 | @ | 15.90 | × .96 | = | 61.0 |
| splice (L3) | 4Ls | 150 × 150 × 11 | @ | 24.90 | × 1.45 | = | 144.5 |
| | | | | | | | 3593.4 |

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CALCULATIONS FOR

Bridges for Kamioka Electric Ry. Co.

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Diagonal members. | | | | | | | |
| $L_0-M_1$. | 8Ls | 150 × 90 × 9 | @ | 16.30 | × 5.700 = | 743.5 | |
| tie plates | 10Pls | 190 × 12 | @ | 17.90 | × .300 = | 53.7 | |
| lug Ls | 16Ls | 90 × 90 × 10 | @ | 13.30 | × .220 = | 46.8 | |
| $M_1-L_2$ + $M_3-L_4$ | 8Ls | 130 × 130 × 9 | @ | 17.70 | × 5.700 = | 807.0 | |
| tie plates | 8Pls | 190 × 12 | @ | 17.90 | × .300 = | 42.9 | |
| $L_2-M_3$ | 8Ls | 125 × 75 × 9 | @ | 13.50 | × 5.700 = | 615.5 | |
| tie plates | 8Pls | 190 × 12 | @ | 17.90 | × .300 = | 42.9 | |
| ~~$M_3-L_4$~~ | ~~8Ls~~ | ~~130 × 130 × 9~~ | ~~@~~ | ~~17.70~~ | ~~× 5.700 =~~ | ~~807.0~~ | |
| ~~tie plates~~ | ~~8Pls~~ | ~~190 × 12~~ | ~~@~~ | ~~17.90~~ | ~~× .300 =~~ | ~~42.9~~ | 2325.? |
| | | | | | | | 3202.2 kg |
| Vertical members. | | | | | | | |
| $L_0-M_0$ | 6Ls | 90 × 90 × 10 | @ | 13.30 | × 3.820 = | 305.0 | |
| tie pls. | 6Pls | 190 × 12 | @ | 17.90 | × .190 = | 20.4 | |
| " | 8Pls | 190 × 9 | @ | 13.42 | × .190 = | 20.4 | |
| gusset pls. | 4Pls | 300 × 9 | @ | 21.20 | × .450 = | 38.2 | |
| Conn Ls | 4Ls | 75 × 75 × 9 | @ | 9.96 | × .300 = | 12.0 | |
| $M_1-L_1$ + $M_3-L_3$ | 8Ls | 90 × 90 × 10 | @ | 13.30 | × 3.680 = | 391.5 | |
| tie pls | 4Pls | 190 × 9 | @ | 13.42 | × .190 = | 10.2 | |
| " | 8Pls | 190 × 12 | @ | 17.90 | × .190 = | 27.2 | |
| $M_2-L_2$ + $M_4-L_4$ | 6Ls | 90 × 90 × 10 | @ | 13.30 | × 3.540 = | 282.5 | |
| tie pls | 3Pls | 190 × 9 | @ | 13.42 | × .190 = | 7.7 | |
| " | 6Pls | 190 × 12 | @ | 17.90 | × .190 = | 20.4 | |
| bracket | 6Ls | 90 × 90 × 10 | @ | 13.30 | × .480 = | 38.3 | |
| " | 3Pls | 350 × 12 | @ | 32.97 | × .480 = | 47.5 | |
| gusset pls | 7Pls | 300 × 9 | @ | 21.20 | × .450 = | 66.8 | |
| | | | | | | | 1288.1 kg. |
| | | Summary for main truss | | | | = | 12414 kg (11564) |
| Top lateral bracings. | | | | | | | |
| Diagonals | 2Ls | 125 × 90 × 9 | @ | 14.60 | × 5.550 = | 162.1 | |
| " | 4Ls | 125 × 90 × 9 | @ | 14.60 | × 2.700 = | 157.6 | |
| " | 4Ls | 90 × 90 × 10 | @ | 13.30 | × 5.550 = | 295.0 | |
| " | 8Ls | 90 × 90 × 10 | @ | 13.30 | × 2.700 = | 287.2 | |
| " | 2Ls | 75 × 75 × 9 | @ | 9.96 | × 5.550 = | 110.5 | |
| " | 4Ls | 75 × 75 × 9 | @ | 9.96 | × 2.700 = | 107.5 | |
| gusset plates (center) | 2Pls | 230 × 9 | @ | 16.25 | × .800 = | 26.0 | |
| " " | 4Pls | 180 × 9 | @ | 12.72 | × .620 = | 31.5 | |
| " " | 2Pls | 180 × 9 | @ | 12.72 | × .500 = | 12.7 | |
| " (sides) | 4Pls | 320 × 9 | @ | 22.61 | × .550 = | 49.8 | |
| " " | 4Pls | 360 × 9 | @ | 25.43 | × 1.100 = | 111.9 | |
| " " | 4Pls | 300 × 9 | @ | 21.20 | × 1.000 = | 84.8 | |
| " " | 6Pls | 270 × 9 | @ | 19.08 | × 1.000 = | 114.5 | |
| | | | | | | | 1551.1 kg |
| Bottom lateral bracings. | | | | | | | |
| Diagonals | 2Ls | 90 × 90 × 10 | @ | 13.30 | × 5.550 = | 147.5 | |
| " | 4Ls | 90 × 90 × 10 | @ | 13.30 | × 2.700 = | 143.6 | |
| " | 6Ls | 75 × 75 × 9 | @ | 9.96 | × 5.550 = | 331.5 | |
| " | 12Ls | 75 × 75 × 9 | @ | 9.96 | × 2.700 = | 323.0 | |
| gusset plates (center) | 2Pls | 230 × 9 | @ | 16.25 | × .800 = | 26.0 | |
| " " | 2Pls | 210 × 9 | @ | 14.84 | × .700 = | 20.8 | |
| " " | 4Pls | 180 × 9 | @ | 12.72 | × .620 = | 31.6 | |
| " (side) | 4Pls | 360 × 9 | @ | 25.43 | × .550 = | 56.0 | |
| " " | 4Pls | 360 × 9 | @ | 25.43 | × 1.100 = | 111.9 | |
| " " | 4Pls | 300 × 9 | @ | 21.20 | × 1.000 = | 84.8 | |
| " " | 6Pls | 250 × 9 | @ | 17.66 | × 1.000 = | 106.0 | |
| Struts (bottom) | 18Ls | 75 × 75 × 9 | @ | 9.96 | × 3.240 = | 581.0 | 1963.7 kg. |

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CALCULATIONS FOR

Bridges for Kamioka Electric Ry Co.

| | | | | | | |
|---|---|---|---|---|---|---|
| Sway Bracings | | | | | | |
| Diagonals | 12Ls 75×75×9 | @ 9.96 | × 4.40 | = 526.0 | | |
| | 4 fill 150×9 | @ 10.60 | × .15 | = 6.4 | | |
| | | | | | 532.4 | |
| Struts on Top Chord | | | | | | |
| | 12Ls 75×75×9 | @ 9.96 | × 1.12 | = 134.0 | | |
| Connection | 24Ls 75×75×9 | @ 9.96 | × .28 | = 66.9 | | |
| " | 24Pls 200×9 | @ 14.13 | × .28 | = 95.0 | | |
| fill | 12Pls 75×9 | @ 5.30 | × .12 | = 7.6 | | |
| | | | | | 303.5 | |
| Cross Beams | | | | | | |
| | 9Is 14"×6" | @ 68.50 | × 3.23 | = 1990.0 | | |
| Connection | 18Ls 90×90×10 | @ 13.30 | × .30 | = 71.8 | | |
| | | | | | 2061.8 | |
| Stringers | | | | | | |
| Intermediate | 12Is 350×150 | @ 58.50 | × 5.10 | = 3582.0 | | |
| End | 4Is 350×150 | @ 58.50 | × 5.35 | = 1251.0 | | |
| Connection | 14Pls 150×9 | @ 10.60 | × .30 | = 44.5 | | |
| splice | 24Pls 300×9 | @ 21.20 | × .25 | = 127.2 | | |
| strut | 8Ls 75×75×9 | @ 9.96 | × .84 | = 67.0 | | |
| Gussets | 16Pls 130×9 | @ 9.19 | × .30 | = 44.1 | | |
| | | | | | 5115.8 | |

Summary of structural steel in 40.0 meter truss span

| | | | |
|---|---|---|---|
| main trusses | 2 @ 12.414 (11564) | = 24828 | 23128 |
| Top Lateral Bracings | | 1551 | |
| Bottom Lateral Bracings | | 1964 | |
| Sway Frames | | 532 | |
| Top Strut | | 304 | |
| Cross Beams | | 2062 | |
| Stringers with struts | | 5116 | |
| shoes + anchor bolts | | 1200 | |
| Rivets and miscellaneous details say 7½% | | 2713 | |
| | | 40.270 | 38.570 |

Call this 40.30 tons. (38.60)

| | | | |
|---|---|---|---|
| 深谷橋梁 | 1 @ 25.0 tons + 1 @ 0.80 | = 25.80 | |
| 船渡谷橋梁 | 1 span | 25.00 | |
| 七百米突附近陸橋 | 1 span | 40.30 | 38.60 |
| | | 91.10 tons | 89.40 |

Design of Jinzu Bashi.

Design of Tower Bent.

wind Load

Exposed area of super structure.

| | | | |
|---|---|---|---|
| Track | = 0.20 | × 8.30 = | 1.66 |
| Chords 2@.30 | = 0.60 | × 6.00 = | 3.60 |
| web | = 0.35 | × 6.00 = | 2.10 |
| misc say | = 0.05 | × 6.00 = | 0.30 |
| | 1.20 | 6.4 m | 7.66 |
| add 100 % | 1.20 | | |
| | 2.40 m² | | |

wind load = 250 × 2.40 = 600 kg per lin meter. ------ $W_2$

when the moving load is taken into consideration.

Exposed area of structure 2.40 × 6.4 = 15.32

" " live load 1.00 × 9.14 = 9.14

3.40 m² 7.2 m 24.46

wind load = 150 × 3.40 = 510 kg per lin meter. ----- $W_1$

Superimposed Loads on pier.

Dead Load 30.500 ÷ 2 = 15,300 kg on one column.

Live load with impact $\frac{52090}{2}$ = 26000 " " "

41,300 " " "

weight of tower bent assumed 46.500 kg

46500 ÷ 33.0 = 1410 kg per lin meter of tower

for one panel = 1410 × 7.95 = 11200 kg

for one column = 11200 ÷ 4 = 2800 kg

Horizontal wind load on bent.

| | | |
|---|---|---|
| Column | 0.40 × 8.0 = | 3.2 |
| Strut | 0.26 × 5.0 = | 1.3 |
| diagonals | 0.13 × 14.0 = | 1.8 |
| vertical tie | 0.15 × 8.0 = | 1.2 |
| | | 7.5 |
| add 100 % | | 7.5 |
| | | 15.0 m² |

15.0 @ 250 = 3750 kg one truss no live load

15.0 @ 150 = 2250 " " full live load.

max. wind load on tower. (one truss of tower)

$W_2$ 7.5 × 600 × 4.9 = 22000 kg. (See page 17. --- $R_3$ = 7.50)

vert. load on column = 22000 × $\frac{7.0}{4.5}$ = 34,200 kg T or C

$W_1$ 7.5 × 510 × 4.9 = 18700 kg

vert. load on column = 18700 × $\frac{7.8}{4.5}$ = 32400 kg T or C

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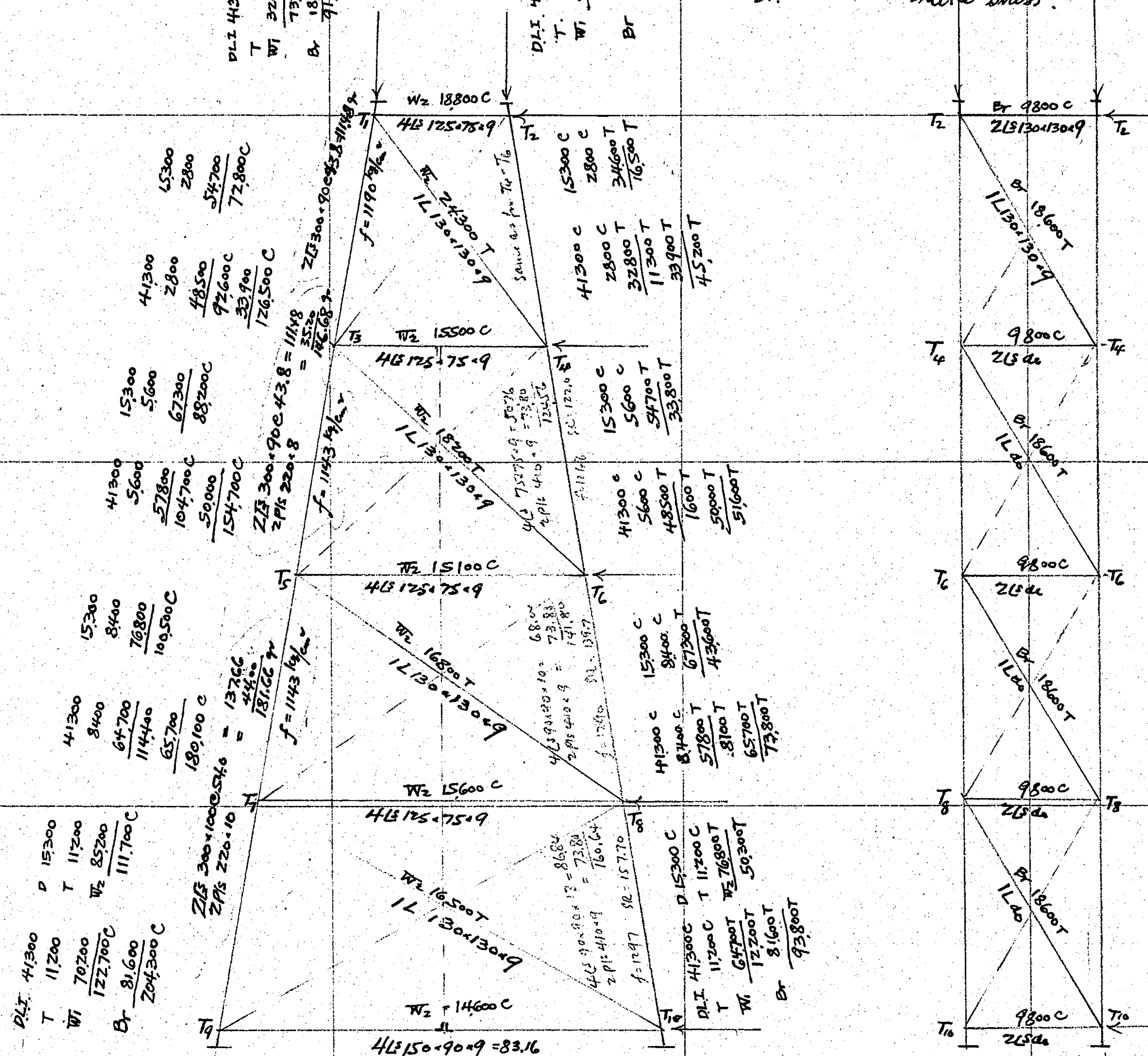
CALCULATIONS FOR

Design of Juizu Bashi

Summary of stresses in Tower Bent. Wind stresses by Graphical method.

W1 ------ wind load 150 kg/m² on structure + moving load.
W2 ------ " " 250 " on structure only.
D ------ Dead Load on column
D.L.I ------ D.L. + L.L. + impact load.
T. ------ weight of Tower bent.
Br. ------ Brake stress.




max. bearing pressure 202000 kg
bearing area on concrete 4120 cm²

max uplift 99500 kg
anchor bolt 58.2 cm² net

25A

Design of Jinzu Bashi.

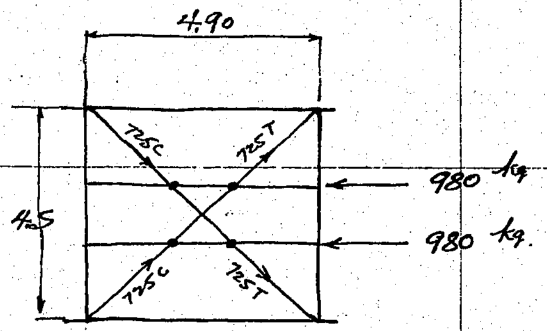
Stresses due to brake force.

Brake force assumed 20% of Live Load specified.

$2{,}000 \times .20 = 400$ kg per lin meter of bridge.

$400 \times 4.90 = 1960$ kg per panel.

Stress on Top Lateral bracing.




Stress in diagonal members $1960 \times 1.478 = 2900$ kg (coef.)

Stress for one diagonal $2900 \div 4 = 725$ kg T or C.

max. wind stress in diagonal = 14125 page 12.

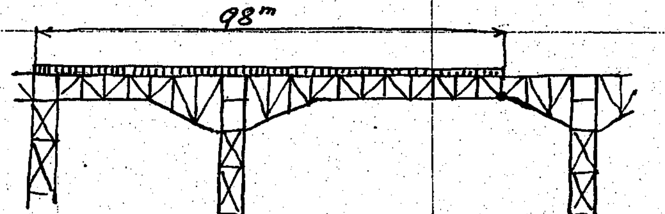
brake stress " " = $\frac{725}{14900}$ kg T or C

Section required for compression member

$= \frac{14900}{600} = 24.82$ cm² gr $< 2\angle 75 \times 75 \times 9 = 25.38$ cm² gr ok.

Rivet no. reqd. $= \frac{14900}{2126} = 7.0 < 8-19\phi$ rivets ok

Brake stress in Tower Bent.



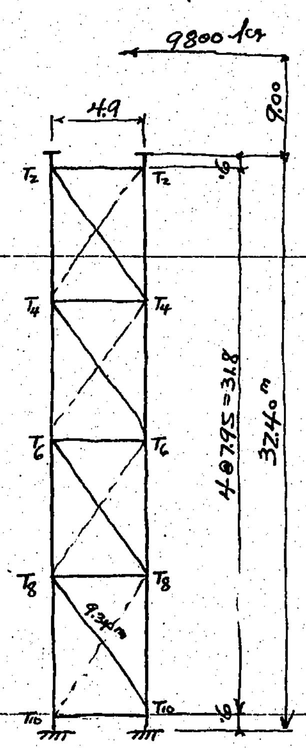

Brake force = 400 kg per lin meter of bridge.

max. brake load on pier

$98^m @ 400 = 39{,}200$ kg on 2 piers.

for one side of one ~~truss~~ tower $= \frac{39200}{4} = 9800$ kg.

Brake force assumed to be applicated on rail top.




Stresses in columns.

| Columns. | brake force | arm | moment | arm | | Stress |
|---|---|---|---|---|---|---|
| T2-T4 | 9800 | × 16.95 | = 166,000 | ÷ 4.90 | = | 33900 kg C or T |
| T4-T6 | " | × 24.95 | = 245,000 | ÷ " | = | 50,000 " |
| T6-T8 | " | × 32.85 | = 322,000 | ÷ " | = | 65,700 " |
| T8-T10 | " | × 40.80 | = 400,000 | ÷ " | = | 81,600 " |
| on shoe. | " | × 41.40 | = 405,000 | ÷ " | = | 82,600 " |

| Diagonals | Shear | Coeff. | | Stress |
|---|---|---|---|---|
| T2-T4 | 9800 kg | × 1.90 | = | 18600 kg T |
| T4-T6 | " | " | = | 18,600 " |
| T6-T8 | " | " | = | 18,600 " |
| T8-T10 | " | " | = | 18600 " |

S.R. = 12.4 cm² net
1L 130×130×9
= 22.59 − 2.25 = 20.34 net
ok

Struts T2-T2, T4-T4, T6-T6, T8-T8, T10-T10 ------ = 9800 kg C

$9800 \times \frac{9.0}{4.9} = 18000$ kg T or C on top.

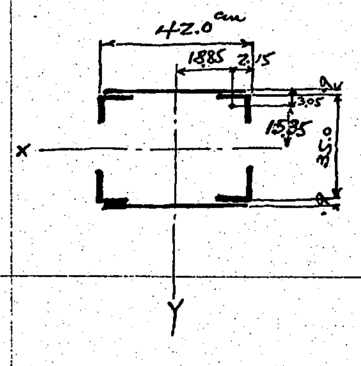
26

Design of Juizu Bashi

Sections of column.

$T_1 - T_3$ max. stress = 126,500 kg C (D.L.+L.L.+I.L.+W.L.)

$T_3 - T_5$ " " = 154,700 " "




2 Pls 410 × 9 = 73.80

4 Ls 75 × 75 × 9 = 50.76

124.56 cm²

moment of inertia about X-X axis.

73.80 × 17.95² = 23,800

64 × 4 + 50.76 × 15.35² = 12,210

36,010 cm⁴

$r_x = \sqrt{\frac{36010}{124.56}} = 17.0$ cm

moment of inertia about Y-Y axis.

.9 × 41³ ÷ 12 = 5,170

64 × 4 + 50.76 × 18.85² = 18,300

23,470 cm⁴

$r_y = \sqrt{\frac{23470}{124.56}} = 13.73$ cm (~~18.85~~)

unsupported length = 806 cm

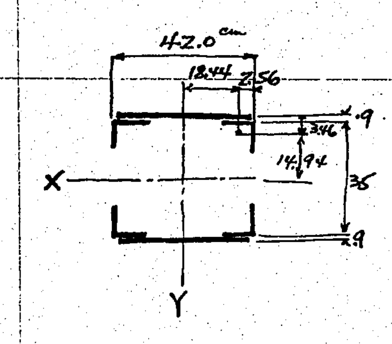
$l/r$ = 806 ÷ 13.73 = 58.7

allowable unit compression

= 1200 − 5 × 58.5 = 907 × 1.40 = 1270 kg/cm²

Safe load on column = 124.56 × 1270 = 158,200 kg C ok.

$T_5 - T_7$ max. stress = 180,100 kg C (D.L.+L.L.+I.L.+W.L.)




2 Pls 410 × 9 = 73.80

4 Ls 90 × 90 × 10 = 68.00

141.80 cm²

moment of inertia about

X-X axis. 73.80 × 17.95² = 23,800

125 × 4 + 68.0 × 14.94² = 15,670

39,470 cm⁴

$r_x = \sqrt{\frac{39470}{141.80}} = 16.7$ cm

Y-Y axis. .9 × 41³ ÷ 12 = 5170

125 × 4 + 68.0 × 18.44² = 23600

28,770 cm⁴

$l/r$ = 806 ÷ 14.27 = 56.5

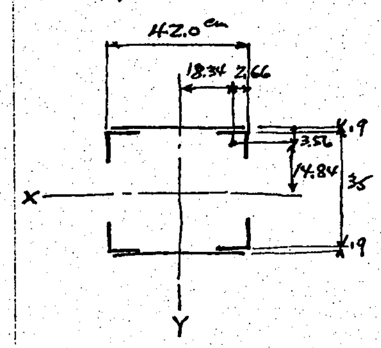
$r_y = \sqrt{\frac{28770}{141.80}} = 14.27$ cm

allowable unit compression

= 1200 − 5 × 56.5 = 917 × 1.4 = 1285 kg/cm²

Safe load on column = 141.80 × 1285 = 182,300 kg C. ok

$T_7 - T_{10}$ max. stress = 204,300 kg C (D.L.+L.L.+I.L.+W.L.)




X-X axis 73.8 × 17.95² = 23800

2 Pls. 410 × 9 = 73.80

158 × 4 + 86.84 × 14.84² = 19,550

4 Ls 90 × 90 × 13 = 86.84

43,350 cm⁴

160.64 cm²

$r_x = \sqrt{\frac{43350}{160.64}} = 16.43$ cm

y-y axis .9 × 41³ ÷ 12 = 5170

158 × 4 + 86.84 × 18.34² = 29820

34990 cm⁴

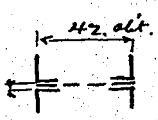
$l/r$ = 806 ÷ 14.76 = 54.6

$r_y = \sqrt{\frac{34990}{160.64}} = 14.76$ cm

$f$ = 1200 − 5 × 54.6 = 927 × 1.4 = 1297 kg/cm²

Safe load on column = 160.64 × 1297 = 208,200 kg C ok.

Strut $T_1 - T_2$ max. stress = 18800 kg C (W.L.)

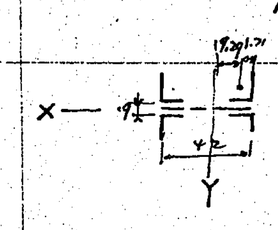



4 Ls 125 × 75 × 9 = 68.76 cm² $l/r$ = 470/6.1 = 77.0

$f$ = 1200 − 5 × 77.0 = 815 kg/cm² C.

safe load = 68.76 × 815 = 56,000 kg C ok.

$T_7 - T_8$ max. stress = 15600 kg C (W.L.)




4 Ls 125 × 75 × 9 = 68.76 cm² $r_x$ = 6.1 $l_x/r_x$ = 633/6.1 = 103.7

74 × 4 + 68.76 × 19.29² = 25860

$r_y = \sqrt{\frac{25860}{68.76}} = 19.40$ $l_y/r_y = \frac{12.65}{19.4} = 65.2$

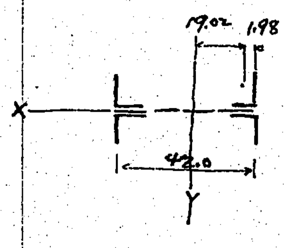
$f = \frac{21000000}{3}\left(\frac{1}{103.7}\right)^2 = 652$ kg/cm²

Safe load = 68.76 × 652 = 44800 kg C ok.

Use same section for struts $T_3 - T_4$ & $T_5 - T_6$.

Design of Juzu Bashi.

Struts $T_9-T_{10}$ max. stress = 14600 kg c. (wind load).




4Ls 150×90×9 = 83.16 cm² gr

$r_x = 7.2$ cm $\quad l_x/r_x = 765/7.2 = 106.4$

$129 \times 4 + 83.16 \times 19.02^2 = 30570$

$r_y = \sqrt{\dfrac{30570}{83.16}} = 19.18 \quad l_y/r_y = 1530/19.18 = 79.8$

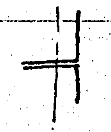
$f = \dfrac{21{,}000{,}000}{3}\left(\dfrac{1}{106.4}\right)^2 = 619$ kg/cm² C.

Safe load = 83.16 × 619 = 51,500 kg c. ok.

Diagonal members.
$T_1 - T_4$

max. stress = 24300 kg T. (wind load) Sr. = 20.24 cm² net.
1L 130×130×9 = 22.59 − 2.25 = 20.34 cm² net ok.
For all other diagonals use same section as for $T_1-T_4$.

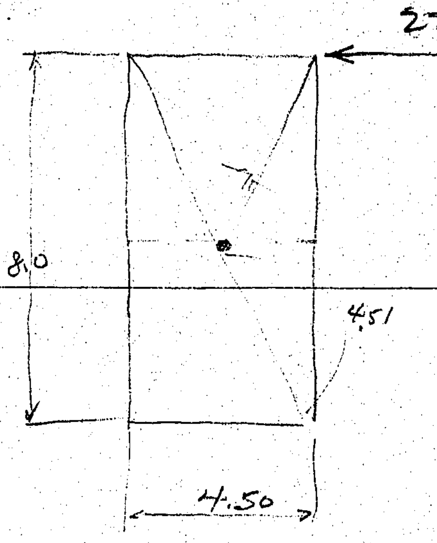
Struts $T_2-T_2$, $T_4-T_4$, $T_6-T_6$, $T_8-T_8$ + $T_{10}-T_{10}$ Stress = 9800 kg C (brake)
Unsupported length 4.90 − .30 = 4.60 m
2Ls 130×130×9 = 45.18 cm² $\quad r = 3.96 \quad l/r = 460/3.96 = 116$ ok



$f = \dfrac{21000000}{3}\left(\dfrac{1}{116}\right)^2 = 520$ kg c.

Safe load = 45.18 × 520 = 23,500 kg c ok.

Sway Bracing at panel point $M_3 - L_3$. $7.50 \times 2940 = 22000$




Diagonal Length $= \sqrt{8.0^2+4.5^2} = \sqrt{64.0+20.25} = \sqrt{84.25} = 9.2^m$

coeff. $= \frac{9.2}{4.5} = 2.1$

Diagonal stress $= 22000 \times 2.1 = 46300$ kg.

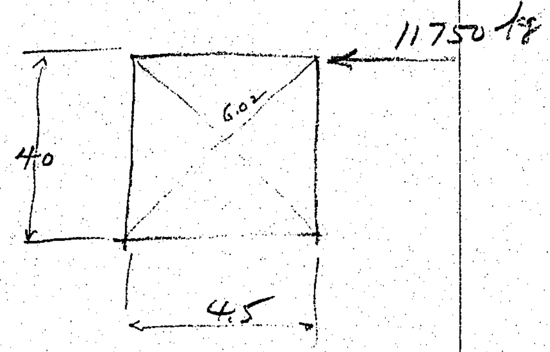
net area req'd $= \frac{46300}{1200} = 38.5$ cm² net

$2\angle s$ $150 \times 100 \times 9 = 43.4 - 4.5 = 38.9$ net ok

rivet for conn 16 − 22φ.

Sway bracing at end of simple span $M_0 L_0$

$2940 \times 4 = 11750$




Diagonal length $= \sqrt{4.5^2+4.0^2} = \sqrt{36.25} = 6.02$

coef. $= \frac{6.02}{4.5} = 1.34$

Diagonal stress $= 11750 \times 1.34 = 15750$ kg.

net area $= \frac{15750}{1200} = 13.15$ net

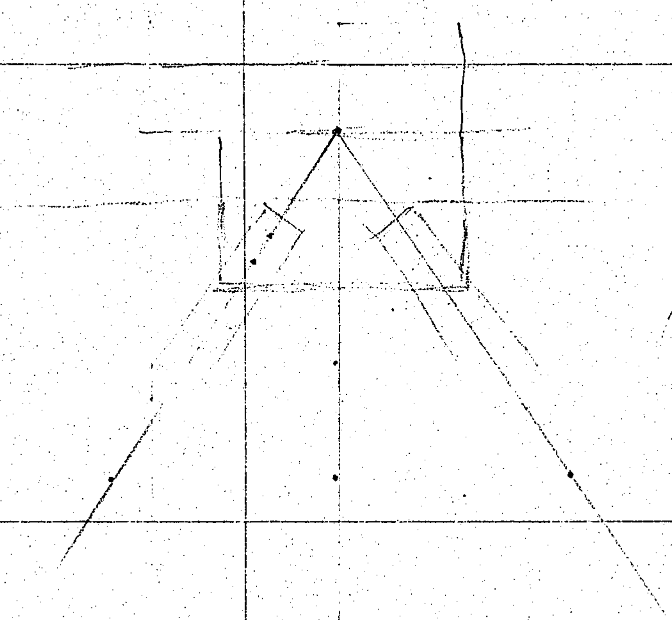
$1\angle$ $125 \times 75 \times 9 = 17.19 - 2.25 = 14.94$ net.

Rivet for conn 6 − 22φ.

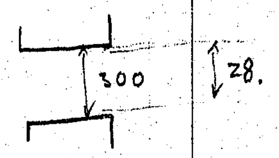
Approximate weight of tower bent

| | | | | |
|---|---|---|---|---|
| columns. | 2[ 250×90 @ 34.6 × 33.5 = | 2320 | | |
| | Details 60% (gusset #) | 1390 | | |
| | | | 3710 × 4 | 14,830 kg |
| Struts | 4Ls 125×75×9 @ 13.5 × 69.2 = | 3740 | | |
| | 4Ls 130×90×9 @ 16.3 × 30.8 = | 2010 | | |
| | Details 15% | 860 | | |
| | | | | 6610 |
| | 2Ls 130×130×9 @ 17.7 × 45 × 10 = | | 1600 | 1600 |
| Diagonals | 1L 130×130×9 @ 17.7 × 205 = | | | 3630 |
| | 1L " @ 17.7 × 152 = | | | 2690 |
| Vert. strut | 4Ls 75×75×9 @ 9.96 × 24.0 × 2 = | | 1910 | |
| | Details | | 190 | |
| | | | 2100 | 2100 |
| center str. | 2Ls 130×130×9 @ 17.7 × 4.5 | | | 160 |
| | | | | 31640 |
| | | | misc | 2,360 |
| | | | | 34,000 |

1030 kg/lin m. of height.



50×60

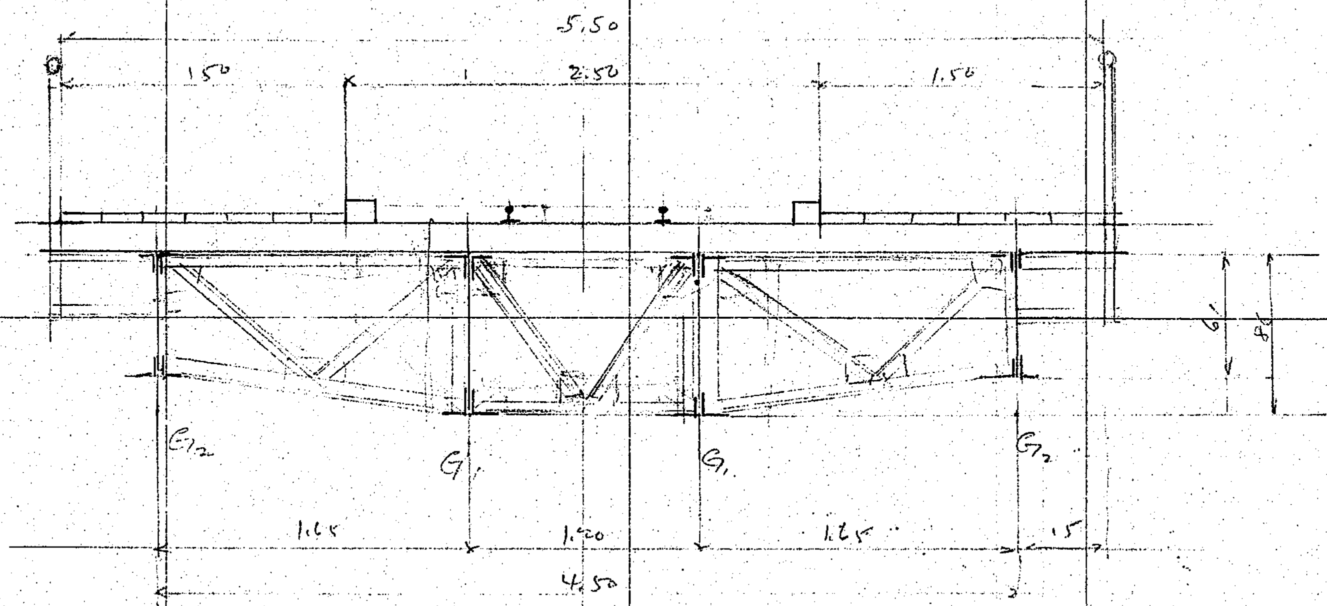



21

Design of Jiuzu Bashi.

Design of Approach Girder spans.

Cross section of Bridge assumed as shown on sketch below.




Deck construction same as for truss span. See on page 1. 2. + 3

Dead Load due to deck.

| | | Girder G1 | | Girder G2 |
|---|---|---|---|---|
| Track | | 100. | | |
| foot walk | | 30 | | 70 |
| handrail | $50 \cdot \frac{.5}{1.65} =$ | − 15 | $50+15 =$ | 65 |
| | | 115 kg/lin m | | 140 kg/lin m. |
| Live Load on foot walk | | 90 " | | 510 " |

Design of main girder G1.

Dead Load

| | | |
|---|---|---|
| Deck construction | | 115 |
| Lateral bracings say | $40 \div 2 =$ | 20 |
| Sway " say. | | 5 |
| main girder assumed. | | 140 |
| | | 280 kg/lin meter. |

Dead Load moment $= \frac{1}{8} \cdot 280 \cdot 12.0^2 =$ 5040 kgm

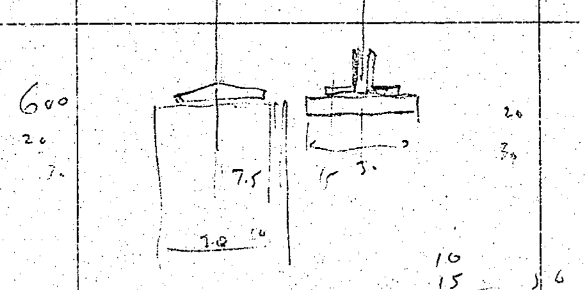
Dead Load shear $= \frac{1}{2} \cdot 280 \cdot 12.0 =$ 1680 kg.

Live Load

Uniform live load on foot walk = 90 kg/lin m on girder G1.

Moment $= 1/8 \cdot 90 \cdot 12.0^2 =$ 1,620 kgm

Shear $= 1/2 \cdot 90 \cdot 12.0 =$ 540 kg



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CALCULATIONS FOR

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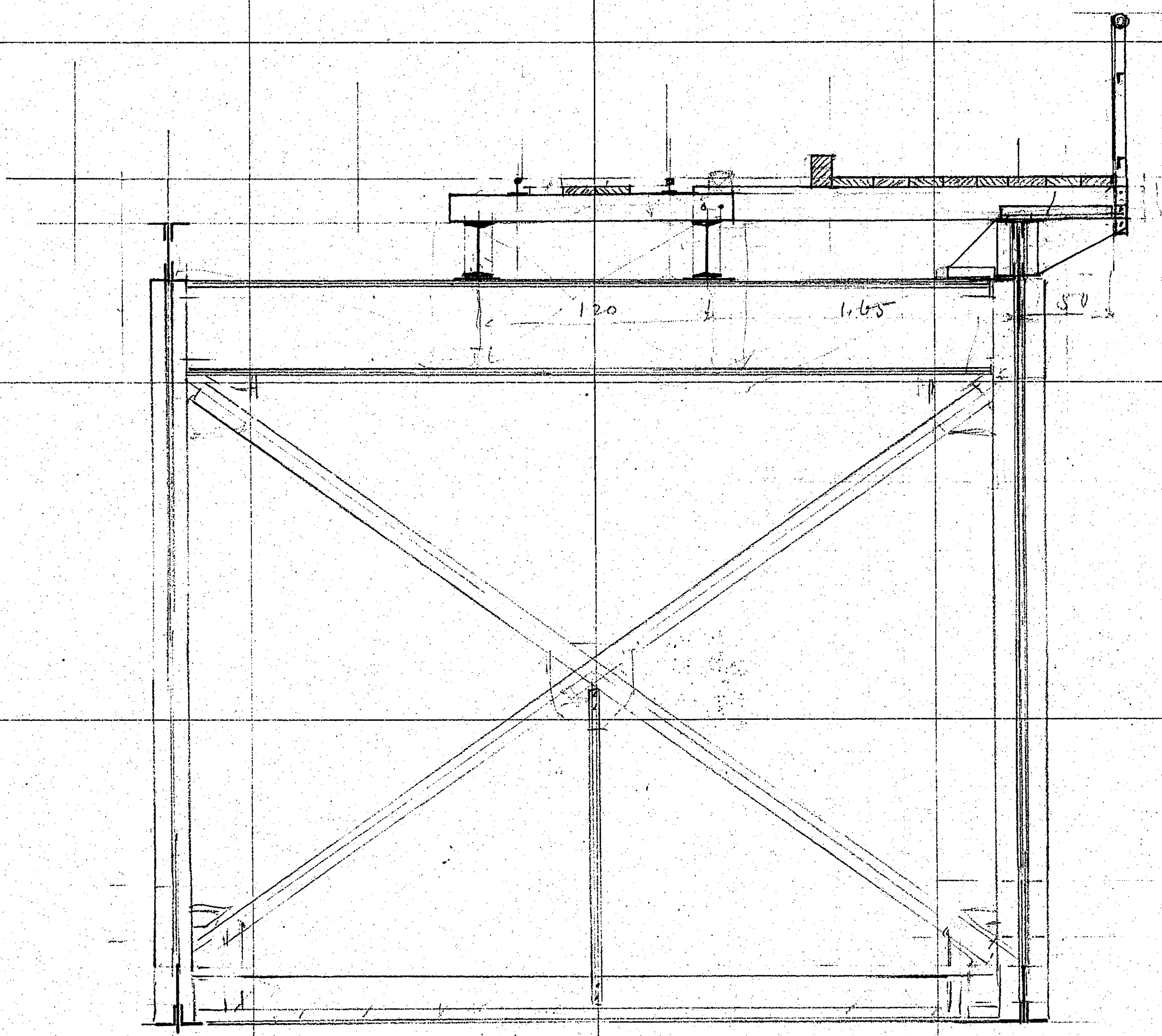
42
3
126
154
280
153
50
483

$\frac{2.0}{2.2}$ $\frac{3.28}{3.0}$




4
2.25
1.60
1.65

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CALCULATIONS FOR

Estimate
truss span 8 @ 4.9 m = 39.2 m

Dead Load. structural steel = 1,200 per meter.
track const. = 200
sidewalks. = 150
1550 ÷ 2 = 775

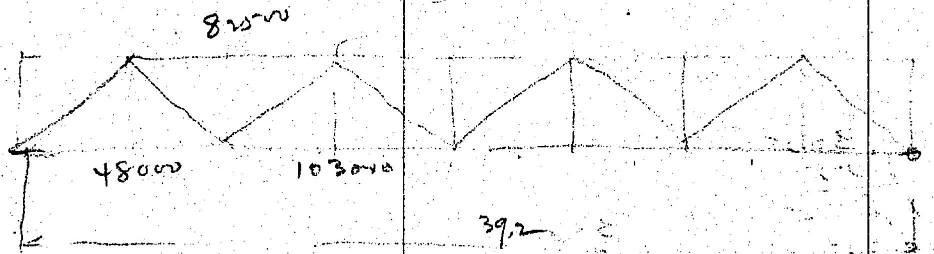
Live Load. 2000
impact $\frac{60}{90+39.2}$ = 930
2930 — say 1500

$DL.\ m = \frac{1}{8} \times 775 \times 39.2^2 = 149,000$

stress $\frac{149,000}{4.00} = 37,300$
72500
109,800

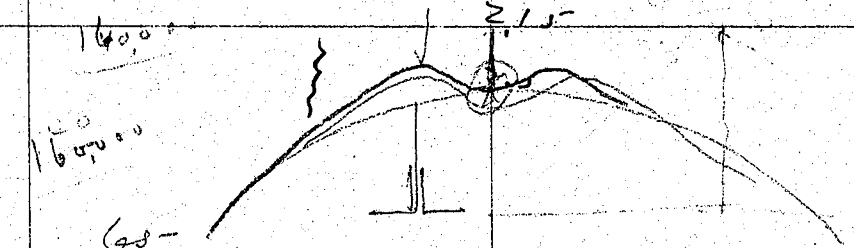
109800

live load.




16000
9150 net

103,000 ÷ 1200 = 860 net




| | | | | |
|---|---|---|---|---|
| 130×130×9 = | 45.18 | — 7.93 | = | 38.25 |
| 300×10.9 = | 27.00 | — 5.96 | | 21.00 |
| | 72.18 | | | 59.25 |
| | | | | 21 |
| | | | | 80 |

1 Pl. 300×9

65
75
140

6
134

2.16

$\frac{66600 \times 6}{15 \times 15^2}$ =

2.15
1.10

118.5

4.90
2.45

48 1.75

.65
.75
1.40

16
25
41 — 6.4 m

6.4

2.3
1.15

22.9
45.18

22
9
19.8

45
5.96
39.04

45.18
7.93
38.25

27.00
5.96
21.04

215

130×130×12 =

cantilever portion D.L. $775 \times \frac{39.2}{2} = 15200$

$DL.\ m = 15200 \times 3 \times 4.9 = 224,000$
$\frac{1}{2} \times 775 \times 14.7^2 = 84,000$
308,000

Live Load — $308000 \times \frac{1500}{775} = 596,000$
904,000
8.0

= 113,000

$\frac{600}{8.2 \times 3.5^2} = 42$

56 kg

6 m

$f = \frac{My}{I}$

$\frac{6m}{bd^2}$

$d^2 = \frac{6 \times 2850}{1.00 \times 56}$ 24.5 / 2.5

303

39.2
9.8
29.4

221.9
49

918
1.5
8.3

415
15

83
.49
13.2

15

39.20
6.60
32.60

18
6.5
24.5

29.40
2.50
26.90

39.2
4.15
34.05

29.40
6.5
22.9

49
9.7

4.9
9.5

Top + bottom

approximate weight of Top lateral.

2L$^s$ 125×75×9 @13.5 @ × 6.65 = 179
2L$^s$ 75×75×9 @ 9.96 × 6.65 = 133

4@ 179 = 715
4@ 133 = 531
8@ 25 = 200
16 @ 20 = 320
1766 ÷ 39.2

45 kg

4.9 4.5 203 2× 44.5 66.5

203

84
.6
50.4

1100 − 15 245 4.90 15.0
245
855

4.5

163

6 483 65.5 8

65.5
48.3
1

2.0

.325
1.55
1.875
.75
2.625

.325
1.875
2.625
4.825

150

1400
145
959

15 1.65
175

Bull. 1 @ 65 × 25 + 6.5

65 + 19.5 + 10

.775

1.50
.775
2.275
.75
1.55
4.575

168

75
19.5
94.5

4900
4575
.325

600
4.9 420

623.

Top 150

1.15
25
96.5

Top bottom sway
45
45
53 — 60
143
150
call 155

2L$^s$ 75×75×9 @ 9.96 × 5.2 = 104
center
corners @ 15 — 20
60
bottom strut. 2L$^s$ 75×75×9 @ 9.96 × 4.5 = ~~90~~
1 Pl. 200×9 @ 14.13 × 4.5 64
misc
248
12
65
260 ÷ 4.9 = 53

Sway

450

300

2 Pls. 450 × 9 = 81.0
4 ∟s 90×90×10 = 68.0
149.0 × 0.785 × 31 = 3,630

Detail 60%

2160
5790

17/4 6.8 strut (4.1) 130×130×9 3.96 - 529

$\frac{4.70}{120}$ =

$\frac{7.35}{120}$ = 6.12 125 × 75 × 9

25

68.76 × 10.79 (1.71, 125) + 294.8 = 8300

$r = \sqrt{\frac{8300}{68.76}} = 12.1$

17.19 × 4 = 68.76

28
14

68.76 × 12.29 (14.0, 1.71) + 294.8 = 10580

$r = \sqrt{\frac{10580}{68.76}} = 15.4$

73.7 × 4 = 29.48

10380 / 3

7.12 ÷ 1.20 = 5.94 (2.97) 4∟ 125 × 75 × 9
9.80 8.16 (4.08)
12.20 10.20 5.10
14.70 (12.20) 6.10 4∟ 125 × 75 × 9 6.10

7.5 meters vertical strut
4.0 ÷ 1.20 = 3.34

$\frac{8.5}{2.00}$ = 4.20

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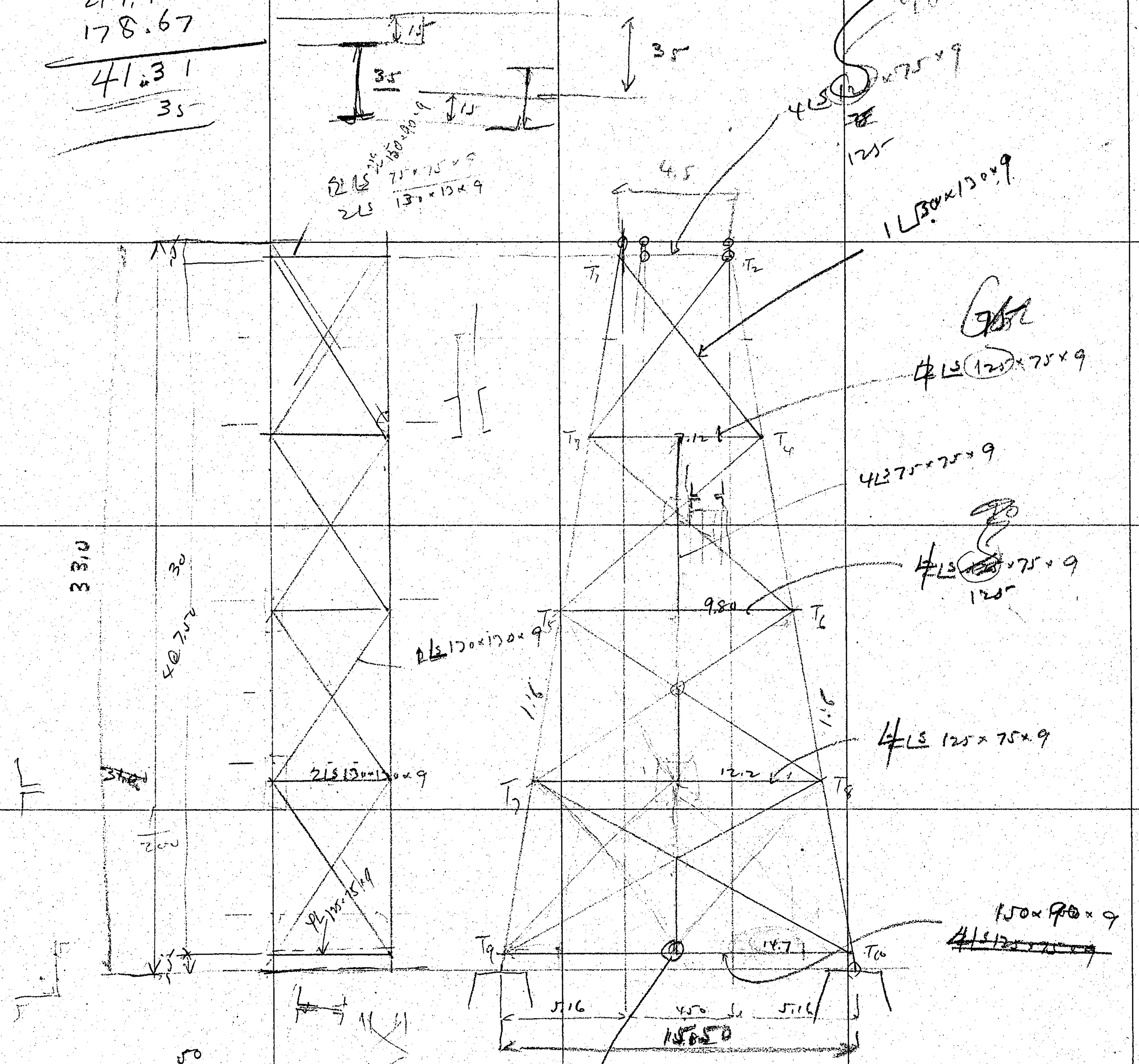
CALCULATIONS. FOR 4.90 4.1 / 120

219.98
178.67
41.31
35




50
30.5
28
14
9

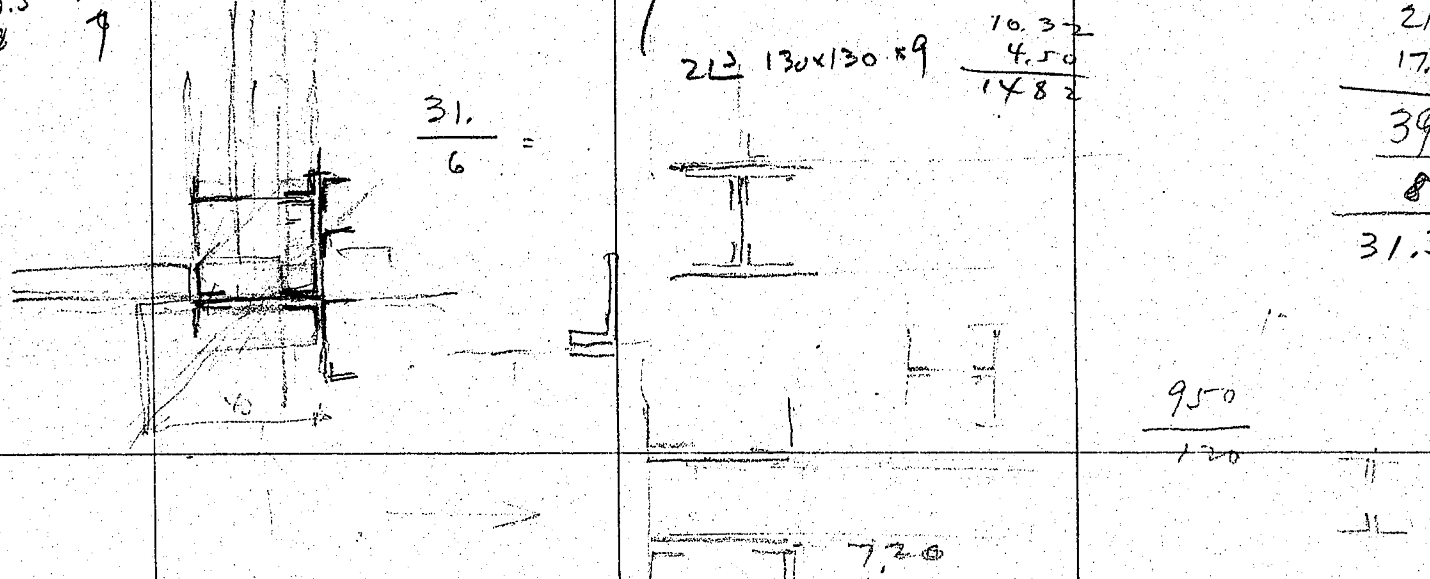
2LS 130×130×9

10.32
4.50
14.82

21.58
17.870
39.88
8.50
31.38

31. / 6 =

length.



950 / 120

7.20
12
14.4
12
2.64

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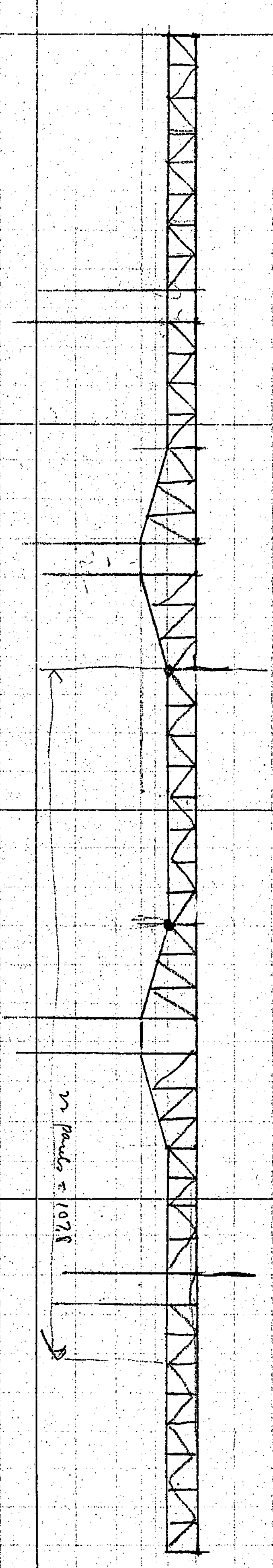
CALCULATIONS FOR

*Stresses of each member due to full unit load.*



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CALCULATIONS FOR

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Torsion member

1 web 300 × 10 = 30. − 6.6 = 23.40
4Ls 150 × 90 × 9 41.58 − 3.96 37.62
71.58 61.02

10070
7835
108535

300 × 15 30. − 6.6 = 23.40
4Ls 150 × 100 × 9 =

300 × 15 = 45.0 − 9.9 = 35.10
37.62
72.72

35.10
60.90
96.00

300 × 12 = 36.0 − 7.9 = 28.10
37.62
65.72

300 × 13 = ~~39~~.0 − 8.58 = 30.

41137.0

97.580
20685
118215

150 × 90 × 15

30 × 15

2Ls 150 × 90 × 15 = 67.40 × 2.2 = 148.20 @ 5.33² + 408 = 2240
1PL 300 × 15 = 45.00 × 15.15 = 697.00 @ 7.97² + 3370 = 6230
11240 7.53 845.20 6470

$r = \sqrt{\frac{6470}{11240}} = \cancel{575}$ 7.58

70.50

7.53
2.2
5.33

15.15
7.53
7.97

2860
3370
6230

2700

1850

204.4
2

900

45880
19600
65480

2Ls 150 × 90 × 15 = 67.40 × 5.93² + 3026
45.00 2370
11240 5396

$\sqrt{\frac{5396}{11240}} = 6.92$

41370
20685

5.18
75
5.93

2.37

30.26

1200 − 354 = 846

$\frac{400}{6.92}$ $\frac{245}{6.92}$

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CALCULATIONS FOR

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Compression –

150 × 90 × 9 = 83.38 × 5.71² + 1872 = 4592
300 × 15 = 45.00
128.38

$\sqrt{\frac{4592}{128.38}}$ = 5.98

5.46
25
5.71

length of m 5 × 245 / 5.98 =

9.40
5.27
7.25

1200
205
995

1200
410
790

5 × 490 / 5.98

64.0 24.0 88
44.30 24.00 68.30
28.40 24.00 52.40

8.0² 4.9²
6.66² 4.90²
5.33² 4.90²

4.9² = 24.00
4 1.33 1.77
25.77 – 5.075

1200 212 988
48 200 × 878 / 5.98 5500
49 560 800 62.00

1200 420 770
52330 × 718 / 5.98 730
35690 718 49.7
33130 800 41.5
28320 878 32.3

65.650
29.825
95.475

29.76
59.52
16
47.52

8.105

85

29.990
9

17.810
8905
26.715

8.100 5.33

38
19
57

4.00
2.66
6.66

approximate weight 128.38
117.40
242.78 × c.785 × 11 × 4.9 =
117.00
.76
435 8383 × 1.4
7600

1200
322
878

281 1200
284
916

30%

3129.8
940.0
4069.8
539

4.9
4.49
539

435 @ .785 = 342.
30% 100
442

1200
428
772

10.5
2.81
7.69

4500
698

1200
356
844

174.5
6980

1200
482
718

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MADE BY I.M. DATE 11-25 FILE NO

CHECKED BY DATE PAGE NO

CALCULATIONS FOR

Jingū Bashi

For Rivets.

1 Ton 当り Shop rivets 98 本
Field rivets 81 本

for Simple span
Rivet with shank 1.6 Ton
総重量 4.3 %

Rivet heads 13,336 T × 0.065 = 870 kgs
総重量 2.35 %

646
478
160
1484 - 1500

20

86

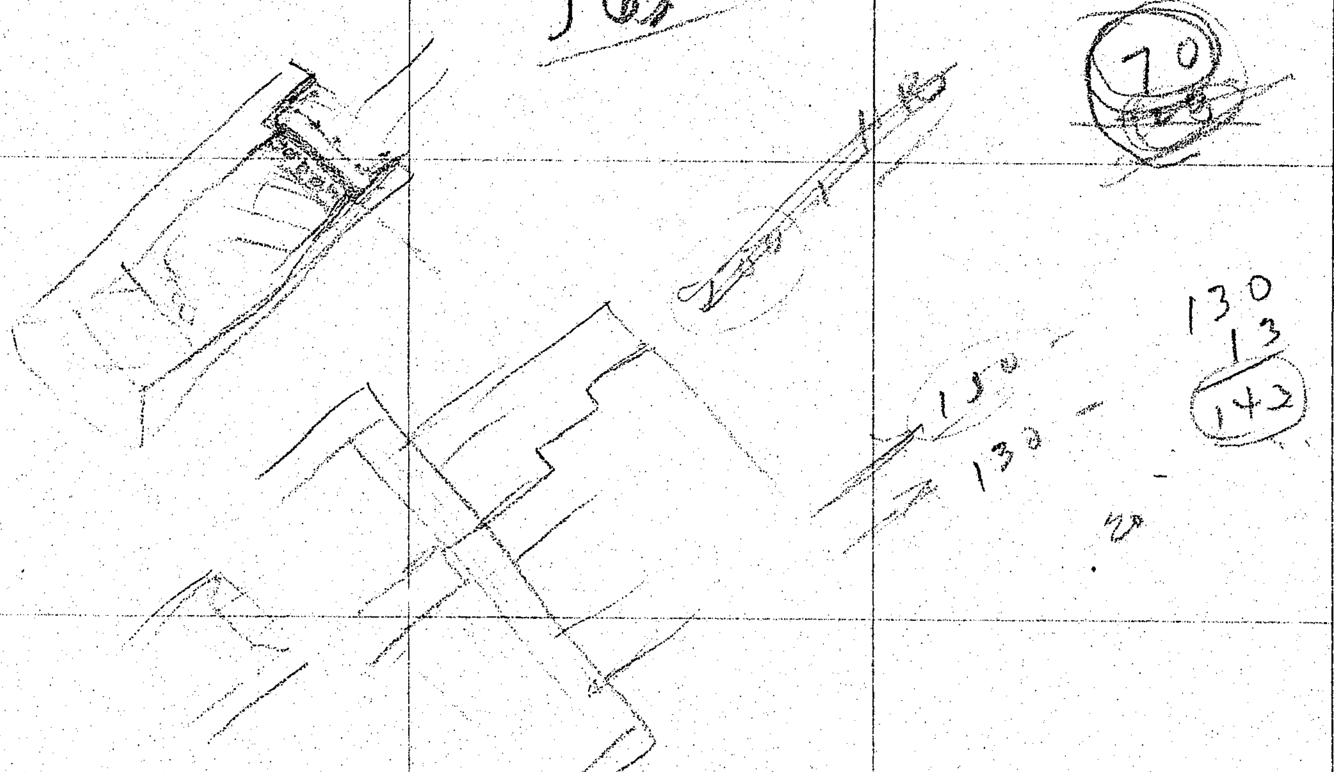
456 - 20 - 436
35
37 -
493 tons - 470 - tons
70
563

70

40

130 - 130
13
130 - 143

20



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CALCULATIONS FOR

1.

| | | | | | |
|---|---|---|---|---|---|
| Top chords Uao-UA1 | 8 Ls | 90×90×10 | @ 13.3 × | 4.700 | = 500 |
| " Guss. Pl. | 2 Pl | 650×10 | @ 51.03 × | .700 | 71 |
| for UA0 only | 2 Ls | 150×90×9 | 16.3 × | .300 | 10 |
| tie Pls | 6 Pls | 170×10 | 13.35 × | 290 | 23 |
| " for handrail Conn. | 4 " | 300×10 | 23.55 × | 290 | 27 — 5 |
| for UA0 only | 1 Pl | 300×10 | 23.55 × | 200 | 5 |
| splice | 8 Pls | 75×9 | 5.30 × | 460 | 20 |
| " | 2 " | 150×9 | 10.60 × | 460 | 10 |
| " | 2 " | 300×9 | 21.20 × | 460 | 20 |
| " UA1-UA1' | 4 Ls | 150×90×9 | 16.3 × | 30.400 | 1,982 |
| web Pl. | 2 Pls | 300×10 | 23.55 × | 8.500 | 401 |
| " | 1 " | " | " × | 8.600 | 203 |
| Gusset Pls | 4 " | 700×10 | 54.95 × | 1.350 | 297 |
| " | 3 Ls | 100×100×10 | 14.9 × | 500 | 22 |
| " | 3 Pls | 500×10 | 39.25 × | 400 | 47 |
| Splice side | 4 Pls | 270×9 | 19.08 × | 460 | 35 |
| " | 4 fills | 115×9 | 8.13 × | 460 | 15 |
| side | 4 Pls | 270×9 | 19.08 × | 900 | 69 |
| | 4 fills | 115×9 | 8.13 × | 900 | 29 |
| Cover. | 4 Pls | 310×9 | 21.90 × | 650 | 57 |
| Top lateral Pls (1) | 2 Pls | 450×9 | 31.79 × | 1.200 | 76 |
| (2), (4) | 3 Pls | 600×9 | 42.39 × | 850 | 108 |
| (3) | 2 " | 380×9 | 26.85 × | 850 | 46 |
| Brackets | 18 Ls | 90×90×9 — 10 | 13.3 × | 292 | 70 |
| " " | 18 " | 75×75×9 | 9.96 × | 240 | 43 |
| " " | 9 Pls | 300×9 | 21.20 × | 300 | 57 |
| " " | 9 Fills | 115×9 | 8.13 × | 190 | 14 |
| Top lateral Pl. (0) | 2 Pls | 350×9 | 24.73 × | 700 | 35 |
| | | | | | 4290 — 4,292 |

| | | | | | |
|---|---|---|---|---|---|
| Bottom chords | 4 Ls | 150×90×9 | 16.3 | 9.250 | 604 |
| | 2 Ls | 150×90×15 | 26.5 | 21.300 | 1,130 |
| " web Pl. | 2 Pls | 300×10 | 23.55 × | 8.100 | 382 |
| " " | 2 Pls | " | " × | 8.200 | 386 |
| " Gusset Pls | 2 " | 550×10 | 43.18 × | 1.200 | 104 |
| " | 2 " | " | " × | 1.600 | 138 |
| " | 1 Pl | 350×10 | 27.48 × | 1.650 | 45 |
| splice Pl. | 20 Pls | 210×9 | 14.84 × | 660 | 196 |
| " | 4 Pls | 75×12 | 7.07 × | 750 | 21 |
| | 2 " | 310×9 | 21.90 × | 750 | 33 |
| | 4 " | 75×15 | 8.83 × | 1.00 | 35 |
| | 2 " | 310×19 | 46.24 × | 1.40 | 130 |
| Reinforced Pls at (0) | 4 Pls | 210×9 | 14.84 × | .500 | 30 |
| L " | 2 Ls | 100×100×10 | 14.9 × | 600 | 18 |
| Bottom lateral Pls | 2 Pls | 550×9 | 38.86 × | 650 | 51 |
| | 2 " | " | " × | 800 | 62 |
| | 5 " | 500×9 | 35.33 × | 800 | 141 |
| Sole Pls | 2 " | 310×22 | 53.54 × | 500 | 54 |
| fill for splice | 4 Fills | 75×6 | 3.53 × | 380 | 5 |
| | | | | | 3,565 |

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| | | | | | |
|---|---|---|---|---|---|
| Virticals (end) | 6 Ls | 100×100×10 | @ 14.9 | × 3.410 | = 350 |
| 〃 | 14 〃 | 〃 | @ 〃 | × 3.810 | 795 |
| Tie Pls | 27 Pls | 170×10 | 13.35 | × .210 | 76 |
| Diagonals | 8 Ls | 150×90×9 | 16.3 | × 5.700 | 744 763 |
| 〃 | 8 〃 | 〃 | 〃 | × 5.850 | 753 |
| 〃 | 4 〃 | 130×130×9 | 17.7 | × 5.700 | 404 |
| 〃 | 4 〃 | 〃 | 〃 | × 5.850 | 414 |
| Tie Pls | 20 Pls | 170×10 | 13.35 | × 270 | 72 |
| 〃 | 20 〃 | 〃 | 〃 | × 220 | 59 |
| lag Ls | 16 Ls | 90×90×9 10 | 13.3 | × 350 | 75 |
| 〃 | 8 〃 | 130×130×9 | 17.7 | × 500 | 71 |
| | | | | | 3813 3,823 |

32
37
22
2602
1221
3823

11,668
2
23,336

11,680
2
23,360

| | | | |
|---|---|---|---|
| ~~Bracket for handrail.~~ | ~~1 L~~ | ~~90×90×9~~ | ~~× 280~~ |
| | ~~1 〃~~ | ~~75×75×9~~ | ~~× 460~~ |
| | ~~1 〃~~ | ~~〃~~ | ~~× 560~~ |
| | ~~1 Pl.~~ | ~~280×9~~ | ~~× 560~~ |
| | | | ~~× 25~~ |

以上 ~~one truss~~

TOP LATERAL BRACINGS.

| | | | | | |
|---|---|---|---|---|---|
| Toplateral | 4 Ls | 125×75×9 | 13.5 | × 6.000 | = 324 |
| | 8 〃 | 〃 | 〃 | × 2.950 | 319 |
| | 4 〃 | 75×75×9 | 9.96 | × 6.100 | 243 |
| | 8 〃 | 〃 | 〃 | × 3.000 | 239 |
| lag Ls | 4 〃 | 75×75×9 | 〃 | × .250 | 10 |
| | 4 〃 | 380×9 | 26.85 | × 800 | 86 |
| | 4 〃 | 300×9 | 21.20 | × 600 | 51 |
| | | | | | 1,272 |

BOTTOM LATERAL BRACINGS.

| | | | | |
|---|---|---|---|---|
| 8 Ls | 125×75×9 | 13.5 | × 6.100 | = 659 |
| 16 〃 | 75×75×9 | 9.96 | × 3.000 | 478 |
| 2 Pls | 300×9 | 21.20 | × 700 | 30 |
| 6 〃 | 300×9 | 〃 | × 500 | 64 |
| | | | | 1,231 |

STRINGERS & STRUTS

| | | | | | |
|---|---|---|---|---|---|
| | 16 Is | 350×150@58.5kg | | × 4.900 | 4584 |
| Splice | 28 Pls | 170×9 | 12.01 | × 280 | 94 |
| Strut | 8 Ls | 75×75×9 | 9.96 | × 1.040 | 80 |
| | 16 Pls | 150×9 | 10.60 | × 300 | 50 |
| 〃 | 4 Ls | 75×75×9 | 9.96 | × 1.500 | 60 |
| | 8 Pls | 25×9 | 1.77 | × .280 | 4 |
| Conn. Ls | 16 Ls | 75×75×9 | 9.96 | × 280 | 45 |
| | | | | | 4,917 |

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CALCULATIONS FOR

3

バット 7.5% 最後に加ヘ

FLOOR BEAMS.

| | | | | | | |
|---|---|---|---|---|---|---|
| | 9 Is | 450×175 @ 91.7 kgs. | | × 4,470 | = | 3,690 |
| Conn. | 18 Ls | 100×100×10 | @ 14.9 | × 350 | | 94 |
| Stringer bed Pl. | 18 Pls | 175×9 | 12.36 | × 230 | | 51 |
| | | | | | | 3,835 |

SWAY BRACINGS

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| SB1 | | 7 Ls | 75×75×9 | @ 9.96 | × 5,200 | = 363 | |
| | | 14 〃 | 〃 | 〃 | × 2,600 | 363 | |
| | Struts | 7 〃 | 150×90×9 | 16.3 | × 4,180 | 477 | |
| | 〃 | 7 〃 | 90×90×10 | 13.3 | × 4,180 | 389 | SB1 = 1978/7 = 283 |
| | splice | 7 Pls | 400×9 | 28.26 | × 500 | 99 | |
| | Conn. | 14 〃 | 270×9 | 19.08 | × 460 | 123 | |
| | | 28 Ls | 75×75×9 | 9.96 | × 230 | 64 | |
| | 〃 | 14 Pls | 270×9 | 19.08 | × 330 | 88 | |
| | Washers | 35 Pls | 70×9 | 4.95 | × 70 | 12 | |
| SB2 | | 2 Ls | 125×75×9 | 13.5 | × 5,200 | 140 | |
| | | 4 〃 | 〃 | 〃 | × 2,500 | 135 | |
| | splice | 2 Pls | 450×9 | 31.79 | × 550 | 35 | |
| | Conn. | 2 〃 | 350×9 | 24.73 | × 500 | 25 | SB2 = 622/2 = 311 |
| | | 8 Ls | 75×75×9 | 9.96 | × 250 | 20 | |
| | | 2 Pl | 350×9 | 24.73 | × 400 | 20 | |
| | Strut | 2 Ls | 150×90×9 | 16.3 | × 4,180 | 136 | |
| | 〃 | 2 〃 | 90×90×10 | 13.3 | × 4,180 | 111 | |
| | | | | | | 2,600 | |

37,191 37,215

Say 37.00

Rivet heads, shoes & miscellaneous 5.

42.00

.475
.400

1660
3,2227
1.77

.030 × 2.23 = .067

1213 19 × 49,580 = 10500/2 = 5.25

= 10,500

37,215

30.

113,980. — 1.77 — S .0155 19φ
1.45 — F .0127

47φ

36,784 — .468 — S —.0127 19φ
.396 — F .0108

20

208

CALCULATIONS FOR

×

Jingu Bashi

Tower span

~~ANCHOR SPAN.~~ & CANTILEVER ARM.

| | | | | | |
|---|---|---|---|---|---|
| Top chord flg. Ls | 2 Ls | 90×90×10 | @ ✓ 13.34 × 4.700 | ~~105~~ — 125. | |
| | 2 " | " | @ ✓ " × 5.700 | ~~266~~ — 152 | |
| | 2 " | 150×90×9 | @ ✓ 16.32 × 9.500 | 310 | ✓ |
| | 2 " | 150×90×15 | @ ✓ 26.49 × 26.000 | 1,377 | ✓ |
| | 2 " | 150×90×9 | @ ✓ 16.32 × 8.750 | 286 | ✓ |
| | 2 " | 90×90×10 | @ ✓ 13.34 × 4.250 | ~~114~~ — 113 | |
| | 2 " | " | ✓ " × 540 | 14 | ✓ |
| Tie Pl. | 1 Pl. | 290×15 | ✓ 34.15 × 900 | 31 | ✓ |
| web Pl. | 1 " | 300×15 | ✓ 35.33 × 39.250 | 1,387 | ✓ |
| Guss. Pls | 1 " | 700×15 | ✓ 82.43 × .700 | 58 | ✓ |
| " | 1 " | " | ✓ " × 11.700 | 964 | ✓ |
| at U11 | 2 Ls | 150×90×9 | ✓ 16.32 × 300 | 10 | ✓ |
| " | 1 Pl. | 310×10 | ✓ 24.34 × 200 | 5 | ✓ |
| Splice Pls | 29 Pls | 210×9 | ✓ 14.84 × 700 | 301 | ✓ |
| " | 12 Pls | 80×16 | ✓ 10.05 × 700 | 84 | ✓ |
| " | 6 " | 310×16 | ✓ 38.94 × 1.000 | 234 | ✓ |
| at U0 | 1 L | 150×90×9 | ✓ 16.32 × 1.350 | 22 | ✓ |
| Bracket 12 1/2 | 24 Ls | 90×90×10 | ✓ 13.34 × 292 | 93 | ✓ |
| " | 24 " | 75×75×9 | ✓ 9.96 × 240 | 57 | ✓ |
| " | 12 Pls | 300×9 | ✓ 21.20 × 300 | 76 | ✓ |
| " | 12 " | 115×9 | ✓ 8.13 × 190 | 19 | ✓ |
| top lateral Pls | 12 " | 350×9 | ✓ 24.73 × 700 | 208 | ✓ |
| " conn Ls | 17 Ls | ~~150×90×9~~ 90×90×10 | ✓ 13.34 × 300 | 68 | ✓ |
| Bottom chord. | 2 Ls | 150×90×9 | (11) × 9.400 | 307 | ✓ |
| | 4 " | 150×90×9 | 16.32 (11) × 50.400 | 3,290 | ✓ |
| web Pls | 1 Pl. | 300×15 | ✓ 35.33 × 44.300 | 1,565 | ✓ |
| Guss. Pls | 1 " | 600×15 | ✓ 70.65 × 2.800 | 198 | ✓ |
| " | 1 " | 800×15 | ✓ 94.20 × 1.900 | ~~180~~ — 179 | |
| " | 1 " | 900×15 | ✓ 105.98 × 5.600 | 593 | ✓ |
| | 1 " | 800×15 | ✓ 94.20 × 1.750 | 165 | ✓ |
| Splice | 24 Pls | 280×9 | ✓ 19.78 × 580. | 275 | ✓ |
| " | 12 " | 280×14 | ✓ 30.77 × 900 | 332 | ✓ |
| " | 12 " | 320×16 | ✓ 40.19 × 700 | 338 | ✓ |
| " fill | 32 Pls | 120×15 | ✓ 14.13 × 680 | 307 | ✓ |
| for L1. L3 | 4 Fills | 180×15 | ✓ 21.20 × 1.000 | 85 | ✓ |
| " | 4 Pls | 330×9 | ✓ 23.32 × 800 | 75 | ✓ |
| " Stiff. | 16 Ls | 125×90×9 | ✓ 14.6 × 310 | 72 | ✓ |
| Bott. lateral Pls | 1 Pls | 550×9 | ✓ 38.86 11.000 | 427 | ✓ |
| | | | | 14,036 ✓ | |
| | | | | 14,202 | |

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CALCULATIONS FOR

5

| | | | | | | |
|---|---|---|---|---|---|---|
| Virticals | 14 Ls | 100×100×10 | @14.9 | × 3,850 | 803 | ✓ |
| | 1 [ | 250×90 @ 34.6 | | × 23.000 | 796 | ✓ |
| | 1 " | 300×90 @ 38.1 | | × 15.600 | 594 | ✓ |
| | 2 Ls | 100×100×10 | 14.91 | × 23.000 | 686 | ✓ |
| | 2 " | " | " | × 16.100 | 480 | ✓ |
| | 50 Fills | 150×15 | 17.66 ✓ | × 220 | 194 | ✓ |
| Diagonals | 12 Ls | 150×90×9 | 16.32 ✓ | × 5,550 | 1,087 | ✓ |
| | 4 " | 130×130×9 | 17.73 ✓ | × 5,600 | 397 | ✓ |
| | 2 " | 130×130×9 | " ✓ | × 7,550 | 267 | ✓ |
| | 2 " | " | " ✓ | × 6,520 | 231 | ✓ |
| | 2 " | 130×130×12 | 23.36 ✓ | × 5,600 | 26~~2~~ | |
| | 2 " | " | " ✓ | × 6500 | 304 | ✓ |
| | 2 " | " | " ✓ | × 7600 | 355 | ✓ |
| lag Ls | 8 Ls | 75×75×9 | 9.96 ✓ | × 250 | 20 | ✓ |
| " | 28 " | 100×100×10 | 14.91 ✓ | × 300 | 125 | ✓ |
| Filler | 56 Pls | 150×15 | 17.66 ✓ | × 220 | 218 | ✓ |
| Horizontal struts M4-3 | 2 Ls | 100×100×10 | 14.91 ✓ | × 4,600 | 137 | ✓ |
| " " | 2 Pls | 400×15 | 47.10 ✓ | × 450 | 42 | ✓ |
| " " | 5 Fills | 150×15 | 17.66 ✓ | × 220 | 19 | ✓ |
| | | | | | 7,017 | ✓ 以上 one truss |

$$\frac{\cancel{21,053} \times 2}{\cancel{42,106}} \qquad \frac{21,219 \times 2}{42,438}$$

TOP LATERAL BRACINGS.

| | | | | | | |
|---|---|---|---|---|---|---|
| | 19 Ls | 75×75×9 | 9.96 ✓ | × 6.300 | 1,192 | ✓ |
| | 22 " | " | " ✓ | × 3.100 | 679 | ✓ |
| | 3 Pls | 300×9 | 21.20 ✓ | × 500 | 32 | ✓ |
| | 8 " | 400×9 | 28.26 ✓ | × 700 | 158 | ✓ |
| | 12 Fills | 75×9 | 5.30 ✓ | × 200 | 13 | ✓ |
| | 32 " | 150×9 | 10.60 ✓ | × 300 | 102 | ✓ |
| | | | | | 2,176 | ✓ |

BOTTOM LATERAL BRACINGS.

| | | | | | | |
|---|---|---|---|---|---|---|
| | 4 Ls | 75×75×9 | 9.96 ✓ | × 6,100 | 243 | ✓ |
| | 8 " | " | " ✓ | × 3.000 | 239 | ✓ |
| | 4 Pls | 300×9 | 21.20 ✓ | × 600 | 51 | ✓ |
| | 6 Ls | 125×75×9 | 13.5 ✓ | × 6,200 | 502 | ✓ |
| | 12 " | " | " ✓ | × 3,050 | 494 | ✓ |
| | 36 " | 75×75×9 | 9.96 ✓ | × 250 | 90 | ✓ |
| | 4 Pls | 600×9 | 42.39 ✓ | × 600 | 102 | ✓ |
| | 2 " | 300×9 | 21.20 ✓ | × 700 | 30 | ✓ |
| Struts | 4 Ls | 125×90×9 | 14.6 ✓ | × 4,200 | 245 | ✓ |
| " Conn. | 8 Pls | 280×9 | 19.78 ✓ | × 350 | 55 | ✓ |
| " " | 16 Ls | 90×90×10 | 13.3 ✓ | × 280 | 60 | ✓ |
| Strut at L4-3 | 3 " | 150×90×9 | 16.3 ✓ | × 4,200 | 205 | ✓ |
| | 3 " | 90×90×10 | 13.3 ✓ | × 4,200 | 168 | ✓ |
| Lacing Ls | 16 " | 75×75×9 | 9.96 ✓ | × 750 | 120 | ✓ |
| | 14 Pls | 230×9 | 16.25 ✓ | × 350 | 80 | ✓ |
| | | | | | 2,684 | ✓ |

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CALCULATIONS FOR

6

STRINGERS

| | | | | | | |
|---|---|---|---|---|---|---|
| | 22 Is | 350×150 @ 58.5 | | × 4,900 | 6300 | ✓ |
| Splice | 44 P/s | 170 × 9 | 12.01 ✓ | × 360 | 190 | ✓ |
| Strut | 22 P/s | 150 × 9 | 10.60 ✓ | × 300 | 70 | ✓ |
| 〃 | 11 Ls | 75×75×9 | 9.96 ✓ | × 1,040 | 114 | ✓ |
| 〃 | 4 Ls | 75×75×9 | 〃 ✓ | × 1,550 | 62 | ✓ |
| 〃 | 8 P/s | 280 × 9 | 19.78 ✓ | × 280 | 44 | ✓ |
| 〃 | 16 Ls | 90×90×10 | 13.3 ✓ | × 280 | 60 | ✓ |
| | | | | | 6,840 | ✓ |

FLOOR BEAMS

| | | | | | | |
|---|---|---|---|---|---|---|
| | 12 Is | 450×175 @ 91.7 | | × 4,470 | 4910 | ✓ |
| Conn. | 10 Ls | 100×100×10 | 14.9 ✓ | × 350 | 52 | ✓ |
| Stringer bed pl. | 24 P/s | 175 × 9 | 12.36 ✓ | × 230 | 68 | ✓ |
| | | | | | 5,030 | ✓ |

SWAY BRACINGS

| | | | | | | |
|---|---|---|---|---|---|---|
| 5 × SB1 See page no. 3. | | 283 × 5 = | | | 1,415 | |
| 1 × SB2 〃 〃 | | 311 × 1 = | | | 311 | |
| SB3 & SB4 | 1 L | 75×75×9 | 9.96 ✓ | × 54,000 | 538 | ✓ |
| 〃 (二組合ブロック) | 2 Ls | 125×75×9 | 13.5 ✓ | × 4,300 | 116 | ✓ |
| Conn. Ls | 16 〃 | 75×75×9 | 9.96 ✓ | × 200 | 319 — 32 | |
| Pl | 1 Pl | 250 × 9 | 17.66 ✓ | × 8,400 | 148 | ✓ |
| 〃 | 4 P/s | 350 × 9 | 24.73 ✓ | × 450 | 45 | ✓ |
| Strut | 4 Ls | 150×90×9 | 16.3 ✓ | × 4,200 | 274 | ✓ |
| 〃 | 4 〃 | 90×90×10 | 13.3 ✓ | × 4,200 | 224 | ✓ |
| SB5. (二組合) | 2 Ls | 150×100×9 | 17.0 ✓ | 32,000 | 1086 | ✓ |
| | 2 〃 | 125×75×9 | 13.5 | × 4,100 | 107 | 111 |
| Conn Ls | 8 〃 | 75×75×9 | 9.96 ✓ | × 500 | 40 | ✓ |
| 〃 Pl | 4 P/s | 200 × 9 | 14.13 ✓ | × 250 | 14 | ✓ |
| 〃 〃 | 2 P/s | 400 × 9 | 28.26 ✓ | × 500 | 28 | ✓ |
| 〃 〃 | 8 P/s | 550 × 9 | 38.858 ✓ | × 700 | 218 | ✓ |
| Struts | 2 Ls | 150×90×9 | 16.3 ✓ | × 4,200 | 137 | ✓ |
| 〃 | 2 〃 | 90×90×10 | 13.3 ✓ | × 4,200 | 112 | ✓ |
| Washers | 30 P/s | 70 × 9 | 4.946 ✓ | × 70 | 10 | ✓ |
| | | | | | 5,142 | ✓ |
| | | | | | 4,859. | |

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CALCULATIONS FOR

Floor system at ~~U12~~ U12-U11 7

~~TOP CHORD ON TOWER~~ ( )

| | | | | | | |
|---|---|---|---|---|---|---|
| | 4 Ls | 90×90×10 | @ 13.3 | × 4,400 | 234 | ✓ |
| Tie Pl. Handrail conn | 2 Pls | 300 × 10 | @ 23.55 | × 290 | 14 | ✓ |
| Tie Pl | 1 " | 170 × 10 | 13.35 | × 290 | 4 | ✓ |
| end Tie Pl. | 2 " | 290 × 10 | 22.77 | × 500 | 23 | ✓ |
| lateral Pls | 2 | 400 × 9 | 28.26 | × 550 | 31 | |
| | | | | | ~~275~~ | ✓ |
| | | | | | 306 × 2 = ~~550~~ 612 | |
| Lateral bracings | 1 L | 75×75×9 | 9.96 | × 5,600 | 56 | ✓ |
| " | 2 Ls | " | " | × 2,700 | 54 | ✓ |
| " | 1 Pl | 300×9 | 21.20 | × 500 | 10 | ✓ |
| Stringers | 2 I | 350 × 15 @ 58.5 | | × 4,750 | 556 | ✓ |
| brackets | 4 Pls | 300 × 9 | 21.20 | × 4,300 | 365 | ✓ |
| " a | 8 Ls | 90×90×10 | 13.3 | × 360 | 39 | ✓ |
| " b | 8 Ls | 150×90×9 | 16.3 | × 200 | ~~261~~ 26 | |
| " c | 8 Ls | 125×75×9 | 13.5 | × 250 | 27 | ✓ |
| strut conn. | 2 Pls | 150 × 9 | 10.6 | × 300 | 6 | ✓ |
| " | 1 L | 75×75×9 | 9.96 | × 1,090 | 10 | ✓ |
| Struts | 8 Ls | 100×75×~~9~~ 10 | 12.95 | × 1,500 | 155 | ✓ |
| | 4 " | 100 " | " | × 1,040 | 54 | ✓ |
| Tie | 2 Pls | 300 × 9 | 21.20 | × 1,800 | 76 | ✓ |
| Conn. | 24 Ls | 90×90×10 | 13.3 | × 280 | 89 | ✓ |
| | | | | | 1,758 — 1,523 | |

550
1,758
2,308
2

66,162 ✓

~~66,286~~
Rivet head ~~5.0~~
~~71,286~~

66,348 ✓
4.97
0.7

72.

66,348
5.1

44
4.97

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CALCULATIONS FOR

Material list for Jinzu bashi 1

Simple truss Top chord UA0-UA1

Top chord UA1-UA3 4 Req'd.

| | | | | | | |
|---|---|---|---|---|---|---|
| 4 Ls | 150 × 90 × 9 | × | 8.305 | 16.32 | 542.15 | main |
| 1 Pl. | 300 × 10 | × | 8.305 | 23.55 | 195.58 | |
| 1 " | 720 × 10 | × | 1.425 | 56.52 | 80.54 | gusset Pl |
| 1 " | 145 × 9 | × | 450 | 10.244 | 4.61 | splice. |
| 1 " | 310 × 9 | × | 450 | 21.902 | 9.86 | |
| 4 Pls | 75 × 9 | × | 450 | 5.299 | 9.54 | |
| 2 Pls | 280 × 9 | × | 440 | 19.782 | 17.41 | " |
| 2 Fills | 120 × 9 | × | 440 | 8.478 | 7.46 | |
| 2 Pls | 280 × 9 | × | 750 | 19.782 | 29.67 | " |
| 2 Fills | 120 × 9 | × | 750 | 8.478 | 12.72 | |
| 2 Pls | 310 × 9 | × | 750 | 21.902 | 32.85 | |
| 1 Pl. | 425 × 9 | × | 1.215 | 30.026 | 36.48 | Top lateral Pls |
| 1 " | 590 × 9 | × | 870 | 41.684 | 36.27 | |
| 6 Ls | 90 × 90 × 10 | × | 292 | 13.34 | 23.37 | stiffeners |
| 2 Fills | 120 × 9 | × | 190 | 8.478 | 3.22 | |
| 2 " | 90 × 9 | × | 120 | 6.359 | 1.53 | |
| | | | | | 1,043.26 × 4 = 4,173.04 | |

Top chord UA3-UA3' 2 Req'd.

| | | | | | | |
|---|---|---|---|---|---|---|
| 4 Ls | 150 × 90 × 9 | × | 11.430 | 16.32 | 740.15 | |
| 1 Pl. | 300 × 10 | × | 8.670 | 23.55 | 204.18 | |
| 2 Pls | 720 × 10 | × | 1.380 | 56.52 | 156.00 | Gusset Pls |
| 4 Pls | 280 × 9 | × | 440 | 19.782 | 34.82 | splice |
| 4 " | 120 × 9 | × | 440 | 8.478 | 14.92 | |
| 3 " | 550 × 9 | × | 850 | 38.858 | 99.09 | lateral Pls |
| 9 | 90 × 90 × 10 | × | 292 | 13.34 | 35.06 | stiff. |
| 3 Fills | 120 × 9 | × | 190 | 8.478 | 4.83 | |
| 3 " | 90 × 9 | × | 120 | 6.359 | 2.29 | |
| | | | | | 1,297.34 × 2 = 2,594.68 | |

6,767.72

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CALCULATIONS FOR

Jingū Bashi 2.

Bottom chord LA0-LA2 4 Req'd.

| | | | | | |
|---|---|---|---|---|---|
| 1 Pl. | 300 × 10 | × 8,105 | 23.55 | 190.87 | |
| 2 Ls | 150 × 90 × 9 | × 5,240 | 16.32 | 171.03 | |
| 1 Pl. | 615 × 10 | × 1,130 | 48.278 | 54.55 | gusset Pl. |
| 2 Fills | 210 × 9 | × 520 | 14.837 | 15.43 | |
| 2 Pls | 210 × 9 | × 655 | 〃 | 19.44 | splice |
| 2 〃 | 〃 | × 655 | 〃 | 〃 | |
| 2 〃 | 75 × 12 | × 755 | 7.065 | 10.67 | |
| 2 Fills | 75 × 6 | × 525 | 3.533 | 3.67 | splice. |
| 1 Pl. | 310 × 9 | × 755 | 21.902 | 16.54 | |
| 1 Pl. | 555 × 12 | × 700 | 52.281 | 36.60 | } lateral Pls |
| 1 〃 | 555 × 9 | × 810 | 39.211 | 31.76 | |
| | | | | 570.00 × 4 = 2,280.0 | |

Bottom chord LA2-LA3 4 Req'd.

| | | | | | |
|---|---|---|---|---|---|
| 1 Pl. | 300 × 10 | × 5,275 | 23.55 | 124.23 | |
| 2 Ls | 150 × 90 × 15 | × 6,860 | 26.49 | 363.44 | |
| 1 Pl. | 520 × 10 | × 1,580 | 40.82 | 64.50 | gusset Pl. |
| 2 Pls | 210 × 9 | × 655 | 14.837 | 19.44 | - splice. |
| 1 Pl. | 70 × 9 | × 1,170 | 4.946 | 5.79 | |
| 2 Pls | 210 × 9 | × 875 | 14.837 | 25.96 | |
| 2 Ls | 150 × 90 × 15 | × 1,095 | 26.49 | 58.01 | } splice. |
| 1 Pl. | 310 × 9 | × 1,095 | 21.902 | 23.98 | |
| 2 Pls | 505 × 9 | × 750 | 35.678 | 53.52 | lateral Pl. |
| | | | | 738.87 × 4 = 2955.48 | |

Bottom chord LA3-LA3' 2 Req'd.

| | | | | |
|---|---|---|---|---|
| 2 Pls | 300 × 10 | × 2,950 | 23.55 | 138.95 |
| 2 Ls | 150 × 90 × 15 | × 7,510 | 26.49 | 397.88 |
| 1 Pl. | 430 × 10 | × 1,600 | 33.755 | 54.01 |
| 4 Pls | 210 × 9 | × 655 | 14.837 | 38.87 |
| 1 Pl. | 505 × 9 | × 750 | 35.678 | 26.76 |
| | | | | 656.47 × 2 = 1,312.94 |

654842

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Jinzu bashi 3

Diagonals 4 Reqd.

| | | | | | |
|---|---|---|---|---|---|
| 4 Ls | 150 × 90 × 9 | × 5,665 | 16.32 | 369.81 | |
| 4 " | " | × 5,740 | " | 374.71 | |
| 10 Tie pls | 150 × 10 | × 190 | 11.775 | 22.37 | |
| 2 Ls | 130 × 130 × 9 | × 5,725 | 17.73 | 203.01 | |
| 2 Ls | " | × 5,805 | " | 205.85 | |
| 10 Tie pls | 150 × 10 | × 270 | 11.775 | 31.79 | |
| 8 Ls | 90 × 90 × 10 | × 305 | 13.34 | 32.55 | |
| 4 " | 150 × 150 × 11 | × 505 | 24.95 | 50.40 | |
| | | | | 1290.49 × 4 | = 5161.96 |

Verticals 2 Reqd.

| | | | | | |
|---|---|---|---|---|---|
| 4 Ls | 100 × 100 × 10 | × 3,745 | 14.91 | 223.35 | |
| 4 Ls | " | × 3,800 | " | 226.63 | |
| 3 Ls | " | × 3,745 | " | 167.51 | |
| 3 Ls | " | × 3,795 | " | 169.75 | |
| 3 Ls | 150 × 150 × 11 | × 450 | 24.95 | 33.68 | |
| 3 Pls | 395 × 10 | × 450 | 31.008 | 41.86 | |
| 21 Tie Pls | 150 × 10 | × 210 | 11.775 | 51.93 | |
| | | | | 914.71 × 2 | = 1,829.42 |

Verticals $L_{A0}$-$U_{A1}$ 4 Reqd.

| | | | | | |
|---|---|---|---|---|---|
| 3 Ls | 100 × 100 × 10 | × 3,900 | 14.91 | 174.45 | |
| 1 L | | 1,080 | " | 16.10 | |
| 1 Fill. | 210 × 10 | × 470 | 16.485 | 7.75 | |
| 2 " | 170 × 9 | × 210 | 12.011 | 5.04 | |
| 3 Tie pls | 150 × 10 | × 210 | 11.775 | 7.42 | |
| | | | | 210.76 × 4 | = 843.04 |

2,672.46

21,150.56

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CALCULATIONS FOR

Jinzū bashi 4

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Floor system at UA0-U11 | | | UA0' - U11 | 2 | Req'd | | |
| 4 | Ls | 90 × 90 × 10 | 4,310 | 13.34 | 230.0 | | |
| 3 | Tie Pls | 150 × 10 | 300 | 11.775 | | 10.6 | |
| 2 | Pls | 300 × 10 | 495 | 23.550 | | 23.4 | |
| 2 | Pls | 435 × 9 | 475 | 30.733 | | 29.2 | |
| | | | | | 230.0 + | 63.2 = | 293.2 × 2 = 586.4 |
| | | | STRUT ST1 | 2 | Req'd. | | |
| 2 | Ls | 100 × 75 × 10 | 1,000 | 12.95 | | 25.9 | |
| 1 | Pl. | 150 × 9 | 300 | 10.598 | | 3.2 | |
| 2 | Pls | 215 × 9 | 300 | 15.190 | | 9.1 | |
| 4 | Ls | 90 × 90 × 10 | 270 | 13.34 | | 14.4 | |
| | | | | | | 52.6 × 2 | = 105.2 |
| | | | STRUT ST2 | | 4 Req'd | | |
| 2 | Ls | 100 × 75 × 10 | 1,440 | 12.95 | | 37.3 | |
| 1 | Pl | 150 × 9 | 300 | 10.598 | | 3.2 | |
| 1 | Pl. | 215 × 9 | 300 | 15.190 | | 4.9 | |
| 1 | 〃 | 〃 | 290 | 〃 | | 4.7 | |
| 2 | Ls | 90 × 90 × 10 | 270 | 13.34 | | 7.2 | |
| 2 | Ls | 〃 | 290 | 〃 | | 7.7 | |
| | | | | | | 65.0 × 4 | = 260.0 |
| | | | LATERAL BRACING | | 1 Req'd. | | |
| 1 | L | 75 × 75 × 9 | 5,560 | 9.96 | 55.4 | | |
| 2 | Ls | 〃 | 2,725 | 〃 | 54.3 | | |
| 1 | Pl. | 270 × 9 | 550 | 19.076 | | 10.5 | |
| 4 | Pls | 75 × 9 | 320 | 5.299 | | 6.9 | |
| | | | | | 109.7 + | 17.4 | = 127.1 |

362
13
492

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~~U8~~

Top chords

| U10 – U11 | U8 – U10 | U3 – U8 | U1 – U3 |
|---|---|---|---|
| 4L 90×90×10 | 1Pl. 300×15<br>2L 150×90×9 | 1Pl. 300×15<br>2L 150×90×15 | 1Pl. 300×15 ×<br>2L 150×90×9 |

diagonals

| L11 – U10 | U10 – L9 | L9 – U8 | U8 – L7 | L7 – U6 | 6 – 5 | L5 U4 | U3 – L2 | U2 – L1 | U1 – L0 |
|---|---|---|---|---|---|---|---|---|---|
| 4L 150×90×9 | 4L 150×90×9 | 2L 130×130×9 | 4L 150×90×9 | 2L 130×130×9 | 〃 | 〃 | 2L 130×130×12 | 〃 | 〃 |

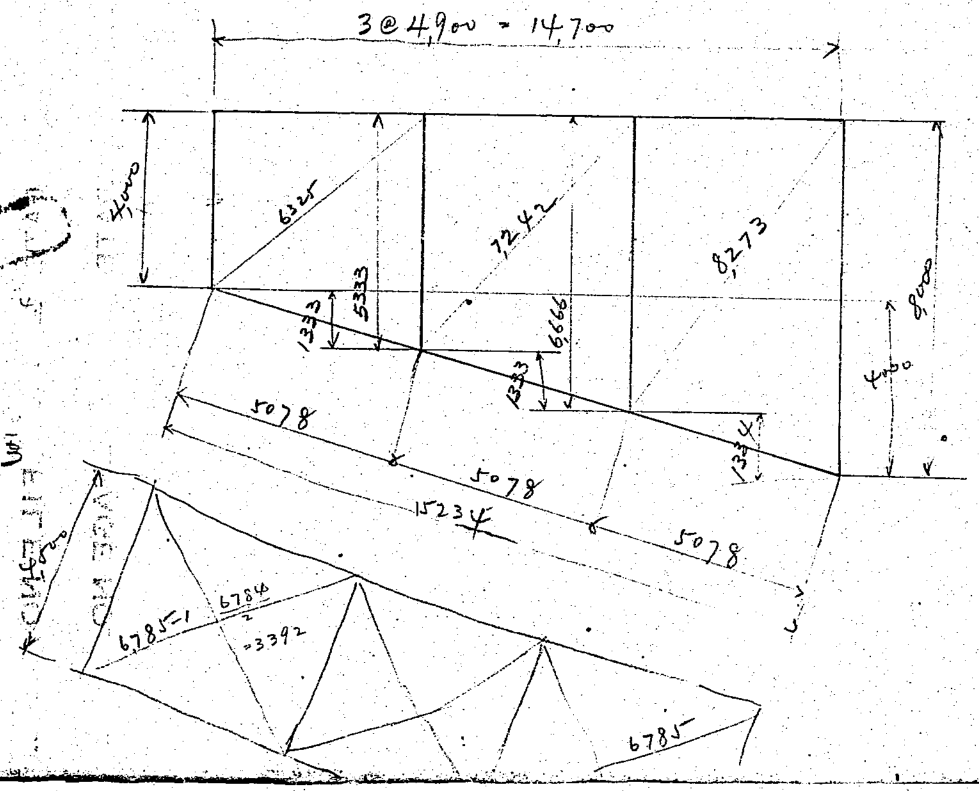
hangers

| U6 – L6 | U5 – L5 | L4 – U4 | L3 – U3 |
|---|---|---|---|
| 2L 100×100×10<br>1C 250×90 @ 34.6 | 〃 | 2L 100×100×10<br>1C 300×90 @ 38.1 kg | 〃 |

STRUT 100×100×10 (2L)

Bottom

| L0 – L9 | L0 – L9 |
|---|---|
| 4L 150×90×9<br>1Pl. 300×15 | 4L 150×90×9<br>1Pl. 300×15 |




4900 24010000
5333 28440889
52450889 7,242

4900 24010000
6666 44435556
68445556 8273

14700 216090000
4000 16000000
232090000

15234.535

5078 = 25786889
4500 20250000
46036889

6785

24010000
1776889
25786889
5078

24010000
1779556
25789556

25789556
2025
46039556

**JIUN MASUDA**
CONSULTING ENGINEER
JIJI BLDG, TOKYO

CALCULATIONS FOR

MADE BY K. I. DATE 6-2-3 FILE NO
CHECKED BY DATE PAGE NO M1

Bridges for Kamioka Electric Ry. Co.

Weight of 30 meter simple truss span.

| | | | | | |
|---|---|---|---|---|---|
| Top chord M0-M1 | 8Ls | 125×90×9 | @ 14.60 | × 4.740 | = 553.0 |
| gusset pls. | 2Pls. | 370×10 | @ 29.05 | × .625 | = 36.4 |
| tie pls. | 8Pls. | 190×10 | @ 14.92 | × .260 | = 31.1 |
| M1-M2 | 8Ls | 125×90×9 | @ 14.60 | × 6.540 | = 765.0 |
| gusset pls. | 2Pls. | 625×10 | @ 49.06 | × 1.340 | = 131.0 |
| " | 2Pls. | 400×10 | @ 31.40 | × .625 | = 39.3 |
| tie pls. | 10Pls | 190×10 | @ 14.92 | × .260 | = 38.8 |
| M2-M3-M2 | 4Ls | 125×90×9 | @ 14.60 | × 8.480 | = 495.0 |
| ~~gusset pls~~ | 1Pl. | 625×10 | @ 49.06 | × 1.160 | = 57.0 |
| tie pls. | 8pls. | 190×10 | @ 14.92 | × .260 | = 31.0 |
| splice, near M1 | 2Pls. | 260×9 | @ 18.37 | × .400 | = 14.7 |
| | 4 | 110×9 | @ 7.77 | × .400 | = 12.4 |
| " web | 2Pls. | 260×10 | @ 20.41 | × .400 | = 16.3 |
| " | 4Pls | 230×9 | @ 16.25 | × .400 | = 26.0 |
| " fills | 4 fills | 80×9 | @ 5.65 | × .400 | = 9.1 |
| splices near M2 | 4Pls. | 260×9 | @ 18.37 | × .600 | = 43.0 |
| web | 2Pls | 260×10 | @ 20.41 | × .600 | = 24.5 |
| spl. pl. | 4Pls | 230×9 | @ 16.25 | × .380 | = 24.7 |
| | 4 fills | 80×9 | @ 5.65 | × .380 | = 8.6 |
| Brackets for F.B | 7Ls | 90×75×9 | @ 11.00 | × .250 | = 19.3 |
| " | 7 fills | 75×9 | @ 5.30 | × .080 | = 29.7 — 3.00 |
| " | 7Pls. | 250×9 | @ 17.66 | × .300 | = 37.0 |
| " | 14Ls | 75×75×9 | @ 9.96 | × .180 | = 25.1 |
| | | | | | 2469.0 |
| | | | | | 2442.3 |
| Bottom chord L0-L2 | 4Ls | 130×130×9 | @ 17.70 | × 9.120 | = 646.0 |
| gusset pls. | 2Pls. | 700×10 | @ 54.95 | × .960 | = 105.0 |
| tie pls. | 10pls | 190×10 | @ 14.92 | × .260 | = 38.8 |
| " | 2Pls | 260×10 | @ 20.41 | × .460 | = 18.8 |
| stiffeners | 4Ls | 125×90×9 | @ 14.60 | × .600 | = 35.0 |
| | 4Ls | 125×90×9 | @ 14.60 | × .700 | = 41.0 |
| " | 2Ls | 90×90×10 | @ 13.30 | × .700 | = 18.6 |
| fills | 4pls. | 190×15 | @ 22.37 | × .500 | = 44.7 |
| " | 4Ls | 150×150×15 | @ 33.60 | × .400 | = 53.7 |
| Sole pls. | 2pls. | 350×30 | @ 82.43 | × .400 | = 66.0 |
| L2-L3-L2 | 2Ls | 130×130×9 | @ 17.70 | × 12.200 | = 431.0 |
| | 1Pl. | 260×10 | @ 20.41 | × 9.120 | = 186.0 |
| splice | 2Pls. | 260×10 | @ 20.41 | × 0.720 | = 29.4 |
| " | 4Ls | 130×130×9 | @ 17.70 | × .720 | = 51.0 |
| " | 4Pls | 120×12 | @ 11.30 | × .720 | = 32.5 |
| | | | | | 1797.5 |
| Diagonals L0-M1 | 8Ls | 125×90×9 | @ 14.60 | × 5.800 | = 678.0 |
| tie plates | 10pls. | 190×10 | @ 14.92 | × .260 | = 38.8 |
| lug Ls | 16Ls | 90×90×10 | @ 13.30 | × .180 | = 38.3 |
| M1-L2 & L2-M3 | 8Ls | 130×130×9 | @ 14.60 | × 5.900 | = 690.0 |
| tie plate | 16pls | 190×10 | @ 14.92 | × .260 | = 62.0 |
| | | | | | 1507.1 |
| Verticals L0-M0 | 6Ls | 90×90×10 | @ 13.30 | × 3.900 | 311.0 |
| tie plates | 6Pls | 190×10 | @ 14.92 | × .190 | 17.0 |
| | 8 | 190×9 | @ 13.42 | × .190 | 20.4 |
| gusset pls. | 4pls. | 300×9 | @ 21.20 | × .450 | 38.2 |
| conn. Ls | 4Ls | 75×75×9 | @ 9.96 | × .300 | 12.0 |
| M1-L1, M2-L2 & M3-L3 | 10Ls | 90×90×10 | @ 13.30 | × 3.640 | 485.0 |
| tie plates | 10Pls | 190×10 | @ 14.92 | × .190 | 28.4 |
| | 10 | 190×9 | @ 13.42 | × .190 | 12.7 |
| gusset pls. | 10pls | 300×9 | @ 21.20 | × .450 | 95.5 |
| | | | | | 709.2 |

29.7
3
26.7

60
123
64
17
246.0
26.7
247.3

2×1.32
3

45
73
52
12
1797.5

2442.3
1797.5
1507.1
709.2
6456.1
2
12912.2
800
13716.6

13
6
78

**JIUN MASUDA**
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JIJI BLDG, TOKYO

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CALCULATIONS FOR

Bridge for Kamioka Electric Ry. Co.
Summary for main truss
for 2 trusses

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Top lateral bracings.** | | | | | | | |
| Diagonals | 4 Ls | 90 × 90 × 10 | @ | 13.30 | × 5.60 | = | 298.0 |
| 〃 | 8 Ls | 90 × 90 × 10 | @ | 〃 | × 2.73 | = | 290.0 |
| 〃 | 2 Ls | 75 × 75 × 9 | @ | 9.96 | × 5.60 | = | 111.5 |
| 〃 | 4 Ls | 75 × 75 × 9 | @ | 〃 | × 2.73 | = | 109.5 |
| Gusset plates (center) | 4 Pls | 180 × 9 | @ | 12.72 | × .62 | = | 31.6 |
| 〃 〃 | 2 Pls | 180 × 9 | @ | 12.72 | × .50 | = | 12.7 |
| 〃 (Side) | 4 Pls | 300 × 9 | @ | 21.20 | × .55 | = | 46.6 |
| 〃 〃 | 4 Pls | 300 × 9 | @ | 21.20 | × 1.00 | = | 84.8 |
| 〃 〃 | 6 Pls | 300 × 9 | @ | 21.20 | × .90 | = | 114.5 |
| | | | | | | | 1099.2 |
| **Bottom lateral bracings.** | | | | | | | |
| Diagonals | 6 Ls | 75 × 75 × 9 | @ | 9.96 | × 5.60 | = | 334.0 |
| 〃 | 12 Ls | 75 × 75 × 9 | @ | 〃 | × 2.73 | = | 326.0 |
| Gusset plates (center) | 6 Pls | 180 × 9 | @ | 12.72 | × .50 | = | 38.1 |
| 〃 (Side) | 4 Pls | 300 × 9 | @ | 21.20 | × .45 | = | 38.2 |
| 〃 〃 | 4 Pls | 300 × 9 | @ | 21.20 | × .90 | = | 76.5 |
| 〃 〃 | 6 Pls | 250 × 9 | @ | 17.66 | × .90 | = | 95.5 |
| Struts (bottom) | 14 Ls | 75 × 75 × 9 | @ | 9.96 | × 3.24 | = | 453.0 |
| | | | | | | | 1361.3 |
| **Sway frames.** | | | | | | | |
| Diagonals | ~~12~~ 10 Ls | 75 × 75 × 9 | @ | 9.96 | × 4.40 | = | 438.0 |
| | 3 fills | 150 × 9 | @ | 10.60 | × .15 | = | 4.8 |
| | | | | | | | 442.8 |
| Struts on top chord | 8 Ls | 75 × 75 × 9 | @ | 9.96 | × 1.12 | = | 89.2 |
| Conn. Ls | 16 Ls | 75 × 75 × 9 | @ | 〃 | × .24 | = | 38.2 |
| 〃 Pls | 16 Pls | 200 × 9 | @ | 14.13 | × .24 | = | 54.2 |
| fills | 8 Pls | 75 × 9 | @ | 5.30 | × .09 | = | 3.8 |
| | | | | | | | 185.4 |
| **Cross beams.** | 7 Is | 14" × 6" | @ | 68.50 | × 3.23 | = | 1549.0 |
| Conn. Ls | 14 Ls | 90 × 90 × 10 | @ | 13.30 | × .30 | = | 55.9 |
| | | | | | | | 1604.9 |
| **Stringers** (intermediate) | 8 Is | 350 × 150 | @ | 58.50 | × 5.14 | = | 2405.0 |
| 〃 (end) | 4 Is | 350 × 150 | @ | 58.50 | × 5.40 | = | 1264.0 |
| Conn. Pls | 10 Pls | 150 × 9 | @ | 10.60 | × .30 | = | 31.8 |
| 〃 〃 | 20 Pls | 300 × 9 | @ | 21.20 | × .25 | = | 106.0 |
| Struts | 6 Ls | 75 × 75 × 9 | @ | 9.960 | × .84 | = | 50.2 |
| Gusset plates | 12 Pls | 130 × 9 | @ | 9.19 | × .30 | = | 33.1 |
| | | | | | | | 3890.1 |

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CALCULATIONS FOR

Bridges for Kamioka Electric Ry. Co.

Summary of structural steel in 30 meter truss span.

| | | | |
|---|---|---|---|
| Main truss | 2 trusses @ | = | 12912.2 |
| Top lateral bracings | | = | 1099.2 |
| Bottom lateral bracings | | = | 1361.3 |
| Sway frames | | = | 442.8 |
| Top struts | | = | 185.4 |
| Cross beams | | = | 1604.9 |
| Stringers with struts | | = | 3890.8 |
| [illegible] bolts | | | 1200.0 |
| Rivets & misc | | | 1700.0 |
| | | | 24395.9 |
| | | | Call this 24.40 |

Structural steel for 5.06 meter I beam span.

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | 5.40 | | 630.0 |
| Main beams | 2 Is | 350 × 150 | @ | 58.50 | × | 5.14 | = | ~~601.8~~ |
| Conn. pls. | 2 pls. | 150 × 9 | @ | 10.60 | × | .30 | = | 6.4 |
| " | ~~4~~ pls. | 300 × 9 | @ | 21.20 | × | .25 | = | 21.2 |
| ~~Struts~~ | 4 ~~8~~ Ls | 75 × 75 × 9 | @ | 9.96 | × | .84 | = | 50.2 |
| ~~Gusset pls~~ | 4 ~~8~~ pls | 130 × 9 | @ | 9.19 | × | .30 | = | 16.5 |
| Lattice | 8 ~~16~~ pls. | 60 × 9 | @ | 4.24 | × | .45 | = | 22.9 |
| Sole pl. | 2 pls. | 200 × 19 | @ | 29.83 | × | .250 | = | 14.9 |
| bed pl | 2 pls. | 250 × 25 | @ | 49.06 | × | .250 | = | 24.5 |
| Anchor bolts | 4 bolts | 25 ϕ 50 cm long | @ | 2.40 | | | | 9.6 |
| | | Rivets heads | | | | | | ~~768.0~~ |

.800

24.5
.8
25.3
24.5
40.3
90.1

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CALCULATIONS FOR

MADE BY k. I. DATE 6-2-3 FILE NO

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Bridges for Kamioka Electric Ry Co.

Structural steel for 40 meter truss span.

| | | | | | | |
|---|---|---|---|---|---|---|
| Top chord. | | | | | | |
| $M_0 - M_1$ | 8Ls | 150 × 90 × 9 | @ 16.30 | × 4.700 | = | 613.0 |
| gusset pls. | 2Pls. | 370 × 12 | @ 29.05 | × .670 | = | 38.9 |
| tie plates. | 8Pls | 190 × 12 | @ 14.92 | × .300 | = | 35.8 |
| $M_1 - M_3$ | 8Ls | 150 × 90 × 9 | @ 16.30 | × 10.000 | = | 1304.0 |
| web pls | 2Pls. | 300 × 12 | @ 28.26 | × 8.770 | = | 495.6 |
| gusset pls | 2Pls | 670 × 12 | @ 63.11 | × 1.500 | = | 189.4 |
| $M_3 - M_4 - M_3$ | 4Ls | 150 × 90 × 9 | @ 16.30 | × 11.560 | = | 754.0 |
| web pl. | 1Pl | 300 × 12 | @ 28.26 | × 8.920 | = | 252.0 |
| gusset pls | 2Pls | 670 × 12 | @ 63.11 | × 1.320 | = | 166.6 |
| splice cov pl. ($M_1$) | 2Pls. | 310 × 9 | @ 21.90 | × .500 | = | 21.9 |
| 〃 side pls. | 4Pls | 270 × 9 | @ 19.08 | × .360 | = | 27.5 |
| 〃 | 4 fills | 120 × 9 | @ 8.48 | × .360 | = | 12.2 |
| 〃 side pls | 4Pls | 270 × 9 | @ 19.08 | × .600 | = | 45.8 |
| 〃 | 4 fills | 120 × 9 | @ 8.48 | × .600 | = | 20.4 |
| 〃 cov pl. ($M_3$) | 4Pls. | 310 × 9 | @ 21.90 | × .400 | = | 35.0 |
| 〃 | 8Ls | 150 × 90 × 9 | @ 16.30 | × .860 | = | 112.0 |
| 〃 side pls | 4Pls | 120 × 9 | @ 8.48 | × .500 | = | 17.0 |
| 〃 〃 | 4Pls | 270 × 9 | @ 19.08 | × .600 | = | 45.8 |
| 〃 | 4 fills | 120 × 9 | @ 8.48 | × .600 | = | 20.4 |
| Brackets for F.B. | 9Ls | 90 × 75 × 9 | @ 11.00 | × .292 | = | 28.9 |
| 〃 | 9 fills | 75 × 9 | @ 5.30 | × .120 | = | 6.7 |
| 〃 | 9Pls | 290 × 9 | @ 20.49 | × .300 | = | 55.3 |
| 〃 | 18Ls | 75 × 75 × 9 | @ 9.96 | × .180 | = | 32.3 |
| | | | | | | 4330.5 kgs. ✓ |
| Bottom chord | | | | | | |
| $L_0 - L_2$ | 4Ls | 150 × 150 × 11 | @ 24.90 | × 9.250 | = | 921.0 |
| ~~gusset pls.~~ | 2Pls | 700 × 12 | @ 65.94 | × .960 | = | 126.5 |
| tie plates | 14Pls | 190 × 12 | @ 17.90 | × .3100 | = | 77.7 |
| stiffeners | 4Ls | 125 × 90 × 9 | @ 14.60 | × .600 | = | 35.0 |
| 〃 | 4Ls | 125 × 90 × 9 | @ 14.60 | × .700 | = | 40.9 |
| 〃 | 2Ls | 90 × 90 × 10 | @ 13.30 | × .700 | = | 18.6 |
| fills | 4Pls | 190 × 15 | @ 22.37 | × .550 | = | 49.2 |
| | 4Ls | 150 × 150 × 15 | @ 33.60 | × .450 | = | 60.4 |
| sole pl. | 2Pls | 350 × 30 | @ 82.42 | × .450 | = | 74.2 |
| $L_2 - L_3$ | 4Ls | 150 × 150 × 11 | @ 24.90 | × 6.640 | = | 662.0 |
| web | 2Pls | 310 × 12 | @ 29.20 | × 5.800 | = | 338.5 |
| gusset pl | 2Pls | 700 × 12 | @ 65.94 | × 1.340 | = | 176.6 |
| $L_3 - L_4 - L_3$ | 2Ls | 150 × 150 × 11 | @ 24.90 | × 8.600 | = | 428.0 |
| web | 1Pl. | 310 × 12 | @ 29.20 | × 6.320 | = | 184.5 |
| ~~gusset pl.~~ | 1Pl | 600 × 12 | @ 56.52 | × 1.260 | = | 71.2 |
| splice ($L_2$) | 4Ls | 150 × 150 × 11 | @ 24.90 | × .960 | = | 95.6 |
| fills | 2Pls | 310 × 12 | @ 29.20 | × .480 | = | 28.0 |
| web spl. | 4Pls. | 135 × 15 | @ 15.90 | × .960 | = | 61.0 |
| splice ($L_3$) | 4Ls | 150 × 150 × 11 | @ 24.90 | × 1.450 | = | 144.5 |
| | | | | | | 3593.4 kgs ✓ |

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

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MADE BY K.I. DATE 6-2-3 FILE NO
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CALCULATIONS FOR

Bridges for Kamioka Electric Ry Co

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Diagonal Lo–M1 | 8Ls | 150 × 90 × 9 | @ | 16.30 × | 5.700 = | 743.5 | |
| tie plates | 10pls. | 190 × 12 | @ | 17.90 × | .300 = | 53.7 | |
| lug Ls | 16Ls | 90 × 90 × 10 | @ | 13.30 × | .220 = | 46.8 | |
| M1–L2 + M5–L4 | 8Ls | 130 × 130 × 9 | @ | 17.70 × | 5.700 = | 807.0 | |
| tie pls | 8pls | 190 × 12 | @ | 17.90 × | .300 = | 42.9 | |
| L2–M3 | 8Ls | 125 × 75 × 9 | @ | 13.50 × | 5.700 = | 615.5 | |
| tie pls | 8pls | 190 × 12 | @ | 17.90 × | .300 = | 42.9 | |
| M3–L4 | 8Ls | 130 × 130 × 9 | @ | 17.70 × | 5.700 = | ~~806~~ 807.0 | |
| tie pls. | 8pls | 190 × 12 | @ | 17.90 × | .300 = | 42.9 | |
| | | | | | | ~~4201.5~~ | |
| | | | | | | 3,202.2 | 2352.2 |
| Verticals Lo–Mo | 6Ls | 90 × 90 × 10 | @ | 13.30 × | 3.820 = | 305.0 | |
| tie pls | 6pls | 190 × 12 | @ | 17.90 × | .190 = | 20.4 | |
| 〃 | 8pls | 190 × 9 | @ | 13.42 × | .190 = | 20.4 | |
| gusset pls | 4pls | 300 × 9 | @ | 21.20 × | .450 = | 38.2 | |
| conn Ls | 4Ls | 75 × 75 × 9 | @ | 9.96 × | .300 = | 12.0 | |
| M1–L1 + M5–L5 | 8Ls | 90 × 90 × 10 | @ | 13.30 × | 3.680 = | 391.5 | |
| tie pls | 4 ~~8~~pls | 190 × 9 | @ | 13.42 × | .190 = | ~~20.4~~ | 10.2 |
| 〃 | 8 ~~4~~pls | 190 × 12 | @ | 17.90 × | .190 = | ~~13.6~~ | 27.2 |
| M2–L2 + M4–L4 | 6Ls | 90 × 90 × 10 | @ | 13.30 × | 3.540 = | 282.5 | |
| tie pls. | 3 ~~8~~pls | 190 × 9 | @ | 13.42 × | .190 = | ~~15.3~~ | 7.7 |
| | 6 ~~3~~pls. | 190 × 12 | @ | 17.90 × | .190 = | ~~10.2~~ | 20.4 |
| bracket | 6Ls | 90 × 90 × 10 | @ | 13.30 × | .480 = | 38.3 | |
| 〃 pls. | 3pls. | 350 × 12 | @ | 32.97 × | .480 = | 47.5 | |
| Gusset pls (bott. strut) | 7pls | 300 × 9 | @ | 21.20 × | .450 = | 66.8 | |
| | | | | | | 1,288.1 | |

Summary for main truss = ~~13408.~~ kgs 12414 kg
for 2 trusses = ~~26816.~~ kg tons 24.828

Top lateral bracings

| | | | | | | |
|---|---|---|---|---|---|---|
| Diagonals | 2Ls | 125 × 90 × 9 | @ | 14.60 × | 5.550 = | 162.1 |
| 〃 | 4Ls | 125 × 90 × 9 | @ | 14.60 × | 2.700 = | 157.6 |
| 〃 | 4Ls | 90 × 90 × 10 | @ | 13.30 × | 5.550 = | 295.0 |
| 〃 | 8Ls | 90 × 90 × 10 | @ | 13.30 × | 2.700 = | 287.2 |
| 〃 | 2Ls | 75 × 75 × 9 | @ | 9.96 × | 5.550 = | 110.5 |
| 〃 | 4Ls | 75 × 75 × 9 | @ | 9.96 × | 2.700 = | 107.5 |
| Gusset pls (center) | 2pls | 230 × 9 | @ | 16.25 × | .800 = | 26.0 |
| 〃 〃 | 4pls | 180 × 9 | @ | 12.72 × | .620 = | 31.5 |
| | 2pls | 180 × 9 | @ | 12.72 × | .500 = | 12.7 |
| 〃 (side) | 4pls | 320 × 9 | @ | 22.61 × | .550 = | 49.8 |
| 〃 〃 | 4pls | 360 × 9 | @ | 25.43 × | 1.100 = | 111.9 |
| 〃 〃 | 4pls | 300 × 9 | @ | 21.20 × | 1.000 = | 84.8 |
| 〃 〃 | 6pls | 270 × 9 | @ | 19.08 × | 1.000 = | 114.5 ✓ |
| | | | | | | 1,551.1 |

Bottom lateral bracing

| | | | | | | |
|---|---|---|---|---|---|---|
| Diagonals | 2Ls | 90 × 90 × 10 | @ | 13.30 × | 5.550 = | 147.5 |
| 〃 | 4Ls | 90 × 90 × 10 | @ | 13.30 × | 2.700 = | 143.6 |
| 〃 | 6Ls | 75 × 75 × 9 | @ | 9.96 × | 5.550 = | 331.5 |
| 〃 | 12Ls | 75 × 75 × 9 | @ | 9.96 × | 2.700 = | 323.0 |
| gusset pls (center) | 2pls | 230 × 9 | @ | 16.25 × | .800 = | 26.0 |
| 〃 〃 | 2pls | 210 × 9 | @ | 14.84 × | .700 = | 20.8 |
| 〃 〃 | 4pls | 180 × 9 | @ | 12.72 × | .620 = | 31.6 |
| 〃 (side) | 4pls | 360 × 9 | @ | 25.43 × | .550 = | 56.0 |
| 〃 〃 | 4pls | 360 × 9 | @ | 25.43 × | 1.100 = | 111.9 |
| 〃 〃 | 4pls | 300 × 9 | @ | 21.20 × | 1.000 = | 84.8 |
| 〃 | 6pls | 250 × 9 | @ | 17.66 × | 1.000 = | 106.0 |
| Struts (bottom) | 18Ls | 75 × 75 × 9 | @ | 9.96 × | 3.240 = | 581.0 ✓ |
| | | | | | | 1,963.7 |

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CALCULATIONS FOR

Bridges for Kamioka Electric Ry. Co

Sway bracings

| | | | | | | |
|---|---|---|---|---|---|---|
| Diagonals | 12 Ls | 75 × 75 × 9 | @ | 9.96 | × 4.40 = | 526.0 |
| | 4 fills | 150 × 9 | @ | 10.60 | × .150 = | 6.4 |
| | | | | | | 532.4 |
| ~~Struts~~ on top chord | 12 Ls | 75 × 75 × 9 | @ | 9.96 | × 1.120 = | 134.0 |
| conn Ls | 24 Ls | 75 × 75 × 9 | @ | 9.96 | × .280 = | 66.9 |
| " pls | 24 pls | 200 × 9 | @ | 14.13 | × .280 = | 95.0 |
| fills | 12 pls | 75 × 9 | @ | 5.30 | × .120 = | 7.6 |
| | | | | | | 303.5 |
| Cross Beams | 9 Is | 14" × 6" | @ | 68.50 | × 3.230 = | 1,990.0 |
| Conn Ls | 18 Ls | 90 × 90 × 10 | @ | 13.30 | × .300 = | 71.8 |
| | | | | | | 2,061.8 |
| Stringers (Intermediate) | 12 Is | 350 × 150 | @ | 58.50 | × 5.100 = | 3582.0 |
| " (end) | 4 Is | 350 × 150 | @ | 58.50 | × 5.350 = | 1,251.0 |
| Conn pls. | 14 pls | 150 × 9 | @ | 10.60 | × .300 = | 44.5 |
| " web. | 24 pls | 300 × 9 | @ | 21.20 | × .250 = | 127.2 |
| ~~struts~~ | 8 Ls | 75 × 75 × 9 | @ | 9.96 | × .840 = | 67.0 |
| gusset pls | 16 pls | 130 × 9 | @ | 9.19 | × .300 = | 44.1 |
| | | | | | | 5,115.8 |

Summary of structural steel in 40 meter truss span.

| | | | |
|---|---|---|---|
| Main trusses | 2 trusses @ 12,414 | = | 24,828 |
| Top lateral bracings | | = | 1,551 |
| Bottom lateral bracings | | = | 1,964 |
| Sway frames | | = | 532 |
| Top struts | | = | 304 |
| Cross beams | | = | 2062 |
| Stringers with struts | | = | 5116 |
| Shoes + anchor bolts | say | = | 1,200 |
| Rivets and miscellaneous. | say 7½ % | = | 2713 |
| | | | 40,270 kg |
| | | | 40.30 tons |

37557
2713

| | | | | |
|---|---|---|---|---|
| 鋼桁材 — | 1@25.0 | 1@80 | = | 25.8 |
| 既設桁 — | | | | 25.0 |
| 七百米突陸桁 — | | | | 40.3 |
| | | | | 91.1 tons |

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JIJI BLDG, TOKYO

MADE BY K. Iwaka DATE 5-12-12 FILE NO
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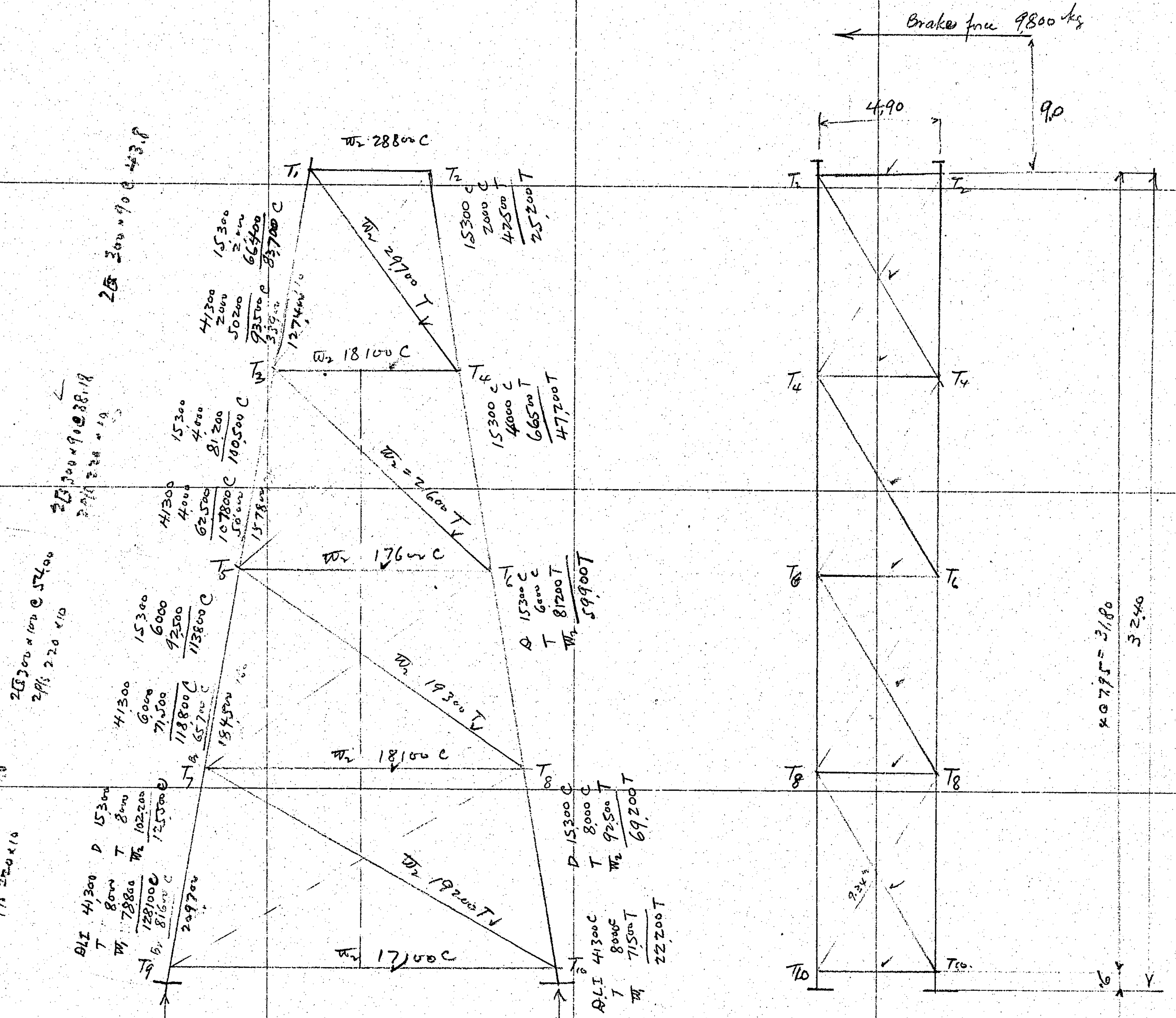
CALCULATIONS FOR

不用

Jinzu Bashi

Tower Bent.

Wind pressure ニ対シ Exposed area ヲ製図面積ノ二倍ニトル時、其ノ程ナ 1.5倍ニ対スル Stress ノ $\frac{2.0}{1.5} = \frac{4}{3}$ 倍ニトルベシ.




| | | | |
|---|---|---|---|
| DLI 41300 C | D 15300 | DLI 41300 C | D 15300 C |
| T 8000 C | T 8000 | T 8000 C | T 8000 C |
| W1 77300 C | W2 100500 | W1 77300 T | W2 100500 T |
| 126600 C | 123800 C | 28000 T | 77200 T |

bearing area
3,620 cm² on concrete

Anchor bolts 51.5 cm² net.

2400
9.34
4.9

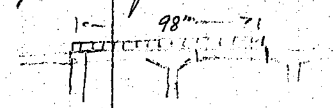
不可

Jinzu Bashi

Brake force — Live load 2000 kg per lin meter or 1000 kg/lin m for one truss

Brake force 1000 × 0,20 = 200 kg/lin m.

Loaded length for max. thrust on tower bent 98 meters




Thrust = 200 × 98 = 19,600 kg

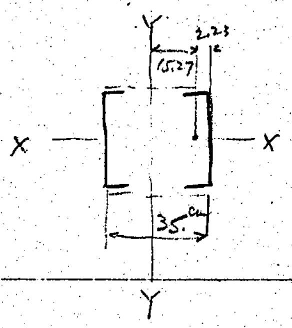
for one truss of one tower = 19600 ÷ 2 = 9,800 kg

Stresses on Tower bent due to brake force.

| Columns. | brake force | arm | moment | arm | Stress |
|---|---|---|---|---|---|
| $T_2 - T_4$ | 9800 kg × | 16.95 = | 166000 kgm ÷ | 4.9 = | 33,900 kg C or T |
| $T_4 - T_6$ | 〃 | × 24.95 = | 245,000 ÷ | 〃 = | 50,000 〃 |
| $T_6 - T_8$ | 〃 | × 32.85 = | 322,000 ÷ | 〃 = | 65,700 〃 |
| $T_8 - T_{10}$ | 〃 | × 40.80 = | 400,000 ÷ | 〃 = | 81,600 〃 |
| $T_{10}$ - 2.T. | 〃 | × 41.4 = | 405,000 ÷ | 〃 = | 82,600 〃 |

| Diagonals | Shear | Coefficient | Stress | S.R. | |
|---|---|---|---|---|---|
| $T_2 - T_4$ ✓ | 9800 kg | 1.90 | 18,600 kg T | 12.4 cm² net | 1L 130×130×9 = 22.59 − 2.25 = 20.34 net ok |
| $T_4 - T_6$ ✓ | 〃 | 〃 | 18,600 〃 | 〃 | 〃 |
| $T_6 - T_8$ ✓ | 〃 | 〃 | 18,600 〃 | 〃 | 〃 |
| $T_8 - T_{10}$ ✓ | 〃 | 〃 | 18,600 〃 | 〃 | 〃 |
| $T_2 - T_2$ ✓ $T_4 - T_4$, $T_6 - T_6$, $T_8 - T_8$, $T_{10} - T_{10}$ — | | | 9,800 kg C | | |

Sections for Tower Columns.




(A) 2[ 300 × 90 @ 38.10 kg = 97.14 cm² − 26.4 = 85.08 cm² net

radius of gyration X−X $r_x$ = 11.51 cm

〃 〃 〃 Y-Y

moment of inertia Y-Y. 2 × 325 + 97.14 × 15.27² = 23300 cm⁴

$r_y = \sqrt{\frac{23300}{97.14}} = 15.16$ cm

$l/r_x = \frac{8.06}{11.51} = 70.0$

allowable unit compression = 1200 − 5 × 70 = 850 kg/cm² { 850 × 1.25 = 1063 kg/cm² C; 850 × 1.40 = 1190 〃 }

(B) 2[ 300 × 90 @ 43.8 kg = 111.48 cm² gr

$l/r_x = \frac{806}{11.52} = 70.0$

allowable unit comp. same as for (A) — 850 × 1.25 = 1063 kg/cm²; 850 × 1.40 = 1190 〃

(C) 2[ 300 × 100 @ 54.0 kg = 137.66 cm² gr 〃 〃

(A) + 2Pls (D) 2[ 300 × 90 @ 38.18 = 97.14
2Pls 220 × 10 = 44.00
141.14 cm² gr.

(B) + 2Pls (E) 2[ 300 × 90 @ 43.8 = 111.48
2Pls 220 × 10 = 44.00
155.48 cm² gr

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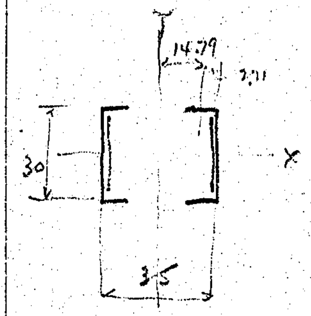
Ⓒ+ Ⓕ

2[ 300×100 @54.0 = 137.66
2 p/s 220×10 = 44.00
181.66 cm² gr.




X-X. moment of I. 2×9166 = 18,332
$\frac{1\times22^3}{12}\times2$ = 1776
20,108
$r_x = \sqrt{\frac{20108}{181.66}} = 10.53$ cm

Y-Y 2×574 + 137.66×14.79² = 31250
44.0×16² = 11250
42500
$r_y = \sqrt{\frac{42500}{181.66}} = 15.3$ cm

$l/r_x = \frac{80.6}{10.53} = 76.4$

allowable unit compr. = 1200 − 76.4×5 = 818    818×1.25 = 1023 kg/cm²
818×1.40 = 1145

allowable load on above sections.

| | | D.L.I + W | D.L.I.+W+Br. | |
|---|---|---|---|---|
| Section | Ⓐ | 103,400 kg C | 115,700 kg C | |
| | Ⓑ | 118,500 | 132,600 | 2[ 300×90 @43.8 |
| | Ⓒ | 146,400 | 163,700 | |
| | Ⓓ | 150,000 | 168,000 | |
| | Ⓔ | 162,000 | 182,000 | 2[ 300×90 @43.8; 2 p/s 220×10 9 |
| | Ⓕ | 186,000 | 208,200 | 2[ 300×100 @54.0; 2 p/s 220×10 9 |

D.L.I.+W.

| Columns | Section | cm² | | | | | |
|---|---|---|---|---|---|---|---|
| T1−T3 | Ⓐ | 97.14 | @ .785 | ×8.66 | ×4 | = | 2640 |
| T3−T5 | B | 111.48 | | 8.06 | ×4 | = | 2820 |
| T5−T7 | B | 111.48 | | 8.06 | ×4 | = | 2820 |
| T7−T9 | Ⓒ | 137.66 | | 8.66 | ×4 | = | 3740 |

計 12020 kg
一部分 1600

1,420
Details 20% 284
1,704
call this 1.70 kg tons
3.5 基 @ 1.70 = 5.95 kg tons. 増

D.L. I. +W+Br.

| Columns | Section | | | | | | |
|---|---|---|---|---|---|---|---|
| T1−T3 | Ⓑ | 111.48 | @ .785 | × 8.66 | × 4 | = | 3120 |
| T3−T5 | D | 141.14 | | 8.06 | × 4 | = | 3575 |
| T5−T7 | F | 181.66 | | 8.06 | × 4 | = | 4600 |
| T7−T9 | Ⓕ | 181.66 | | 8.66 | × 4 | = | 4935 |

計 16230
一部分 1600

5,630
Details 20% 1126
6,756
call this 6.80 kg tons for one bent
3.5 @ 6.80 = 23.80 kg tons. 増

風荷重増加ノタメ鉄材増加総量 5.95 kg tons. 増 見込ミ
風荷重増加及制動荷重ノタメ鉄材増加 23.80 〃 増 〃
制動荷重ノタメノ増加 17.85 〃 増 〃

Sidewalk ヲ除去シタルタメ truss span ニテ 6.00 kg tons 減 見込ミ
〃 〃 girder 4 span ニテ 8.00 〃 〃 〃
14.00 〃 減 〃

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MADE BY K. Inaba DATE 5-12-12 FILE NO

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CALCULATIONS FOR

不用.

Sway Bracings, at panel Points 4 and 6. 3 required.

Design of Jinzu Bashi.

Design of Tower Bent.

Wind load.

Exposed area of super structure.

| | | | | |
|---|---|---|---|---|
| Track | | = 0.30 | × 8.4 | = 2.52 |
| Chords | 2@·32 | = 0.64 | × 6.0 | = 3.84 |
| web with gusset pls. | | = 0.45 | × 6.0 | = 2.70 |
| misc say | | = 0.06 | × 6.0 | = 0.36 |
| | | 1.45 | 6.50 m | 9.42 |
| add. 100 % | | 1.45 | | |
| | | 2.90 m² | | |

Wind load = 250 × 2.90 = 725 Kg per lin meter.

When moving load is taken into consideration

| | | |
|---|---|---|
| Exposed area of structure | 2.90 × 6.5 = | [illegible] |
| " " " live load | 1.00 × 1.13 = | [illegible] |

Design of Jinzu Bashi

Approximate weight of High Tower Bent

| | | | | |
|---|---|---|---|---|
| Column | 2[s 300×90@43.80 kg | ×16.72 | = 1465 | |
| " | 2Pls 220×8 @ 13.82 | × 8.36 | = 231 | |
| " | 2[s 300×100@ 54.00 | ×16.72 | = 1805 | |
| " | 2Pls 220×10 @ 17.27 | × 16.72 | = 577 | |
| | | | 4078 ×4 = | 16312 |
| Struts | 4Ls 125×75×9@13.5 | × 34.70 | = 1873 | |
| | 4Ls 150×90×9@16.3 | × 15.30 | = 997 | |
| | | | 2870 ×2 = | 5740 |
| Struts | 2Ls 130×130×9@17.7 | × 4.90 | = 174 ×10 = | 1740 |
| Diagonals. | 1L 130×130×9@17.7 | × 51.66 | = 914 × 4 = | 3656 |
| Diagonals | 1L 130×130×9@17.7 | × 9.34 | = 165 × 16 = | 2640 |
| Vertical tie | 4Ls 75×75×9 @ 9.96 | × 23.85 | = 950 × 2 = | 1900 |
| Center strut (bottom) | 2Ls 130×130×9@17.7 | × 4.90 | = 174 × 1 = | 174 |
| Shoes and anchor bolts say | 4@ | | 530 = | 2120 |
| | | | | 34282 kg |

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CALCULATIONS FOR

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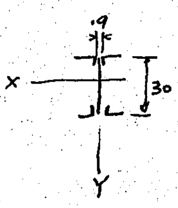
Design of Jinzu Bashi.

Struts $T_7$-$T_8$




max. stress 15600 c (wind) unsupported length $l = \frac{12.65}{2} = 6.33$ m

HS 125×75×9 = 68.76 cm² qr.

least radius of gyration $r_y = 6.1$ cm $\quad l/r = 633/6.1 = 103.7 \quad l/r_x = \frac{1265}{13.35} = 95$

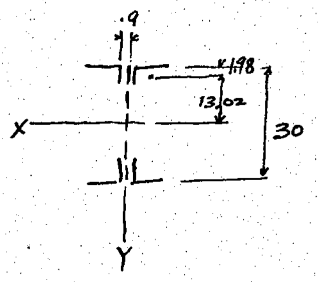
allowable unit compression $f = \frac{21000000}{3} \times \left(\frac{1}{103.7}\right)^2 = 652$ kg/cm²

Required gross sectional area $= \frac{15600}{652} = 23.95$ cm² qr

Safe load = 68.76 × 652 = 44800 kg c. ok.

Struts $T_3$-$T_4$, & $T_5$-$T_6$ use same section as for $T_7$-$T_8$.

Strut $T_9$-$T_{10}$




stress 14600 kg c. (wind) unsupported length 15.3 m horizontal
7.65 vertical.

HS 150×90×9 = 83.16 cm² qr.

$r_y = 7.2$ cm $\quad l_0/r_y = 765/7.2 = 106.4$

$I_x$ 129×4 = 516

83.16×13.02² = 14084
14600

$r_x = \sqrt{\frac{14600}{83.16}} = 13.26 \quad l_0/r_x = 1530/13.26 = 115.3$

Allowable unit compression $f = \frac{21000000}{3}\left(\frac{1}{115.3}\right)^2 = 526$ kg/cm² c.

Safe load = 83.16 × 526 = 43700 kg c. ok.

Diagonal members.

$T_1$-$T_4$ max. stress 24300 kg T wind. S.R. = 20.24 cm² net.

1L 130×130×9 = 22.59 − 2.25 = 20.34 cm² net. ok

For all other diagonal members, use same section as above.

Struts $T_2$-$T_2$, $T_4$-$T_4$, - - - - $T_{10}$-$T_{10}$.. Stress = 9800 kg c (Brake) for all members.

Unsupported length $l = 4.90 - .30 = 4.60$ meters.

least radius of gyration required = 460 ÷ 120 = 3.84 cm

2LS 130×130×9 = 45.18 cm² qr

least radius of gyr. = 3.96 cm $\quad l/r = 460/3.96 = 116$.

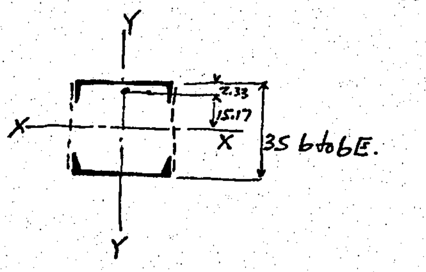
Allowable unit compression $= \frac{21000000}{3}\left(\frac{1}{116}\right)^2 = 520$ kg/cm²

Safe load = 45.18 × 520 = 23500 kg c. ok.

Design of Jiuzu Bashi.

Sections of Column.

$T_1 - T_3$. max. stress 126,500 c (D.L.+L.L.+I.L.+W.L.)




ZE 300×90@43.8 = 111.48 $cm^2$   radius of gyration Y-Y axis 11.52 cm

moment of inertia X-X axis

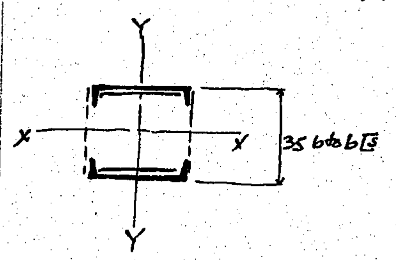
$2 \times 373 + 111.48 \times 15.17^2 = 26350$

radius of gyration X-X axis $\sqrt{\frac{26350}{111.48}} = 15.38$ cm

allowable unit compression $= 1200 - 5 \times \frac{806}{11.52} = 850 \times 1.40 = 1190$ kg/cm²

allowable load $= 1190 \times 111.48 = 132,500$ kg c. ok.

$T_3 - T_5$ max. stress = 154,700 kg c (D.L.+L.L.+I.L.+W.L.)




ZE 300×90@43.8 = 111.48
2Pls 220×8 = 35.20
146.68 cm²

moment of inertia about X-X axis

ZE $2 \times 373 + 111.48 \times 15.17^2 = 26,350$
2Pls. $35.2 \times 16.1^2 = 9120$
35,470 cm⁴   $r_x = \sqrt{\frac{35470}{146.68}} = 15.57$ cm

moment of inertia about Y-Y axis

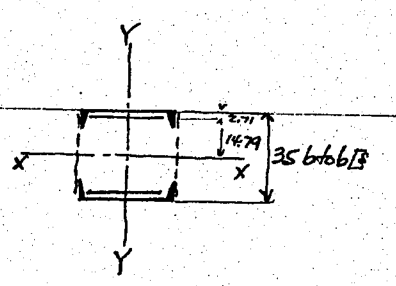
ZE $2 \times 7403 = 14,806$
2Pls $2 \times \frac{22^3 \times 0.8}{12} = 1418$
16224 cm⁴   $r_y = \sqrt{\frac{16224}{146.68}} = 10.52$ cm

allowable unit compression $= 1200 - 5 \times \frac{806}{10.52} = 817 \times 1.40 = 1143$ kg/cm²

allowable load on column $= 1143 \times 146.68 = 167,600$ kg c ok.

$T_7 - T_9$ max. stress 204,300 kg c. (D.L.+L.L.+I.L.+W.L.)

ZE 300×100@54.0 = 137.66
2Pls 220×10 = 44.00
181.66 cm²




moment of inertia about X-X axis.

ZE $2 \times 574 + 137.66 \times 14.79^2 = 31,250$
2Pls $44.00 \times 15.80^2 = 10980$
42,230 cm⁴   $r_x = \sqrt{\frac{42230}{181.66}} = 15.25$ cm

moment of inertia about Y-Y axis

ZE $2 \times 9166 = 18332$
2Pls $\frac{1 \times 22^3}{12} \times 2 = 1,773$
20,105 cm⁴   $r_y = \sqrt{\frac{20105}{181.66}} = 10.53$ cm

allowable unit compression $= 1200 - 5 \times \frac{806}{10.53} = 817 \times 1.40 = 1143$ kg/cm

allowable load on column $= 1143 \times 181.66 = 208,000$ kg c ok.

max. tension on column = 93800 kg

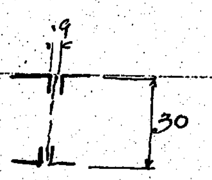
net section req'd. $= \frac{93800}{1200 \times 1.4} = 55.9$ cm² net.

$T_5 - T_7$ max. stress 180100 kg c

use same section as for column $T_7 - T_9$

Struts.

$T_1 - T_2$ max. stress 18800 kg c unsupported length 4.70 m




HE 125×75×9 = 68.76 cm² gr.

least radius of gyration $r_y = 6.1$ cm   $l/r = 470/6.1 = 77.0$

allowable unit compression $f = 1200 - 5 \times 77 = 815$ kg/cm²

Safe load $= 68.76 \times 815 = 56000$ kg c ok.

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電話丸ノ内七七七番

main sections.

Column 4 @ 2723 = 10,892
shoes 4 @ 478 = 1,912

longit. br. 2 @ 2199 = 4,398 –

transv. br. 2 @ 5269 = 10,538

bott. str. at d. 2 @ 163 = 326

28.066

6.216

484

6.70 × 3.50 = 23.4

2.8

20.6

1600

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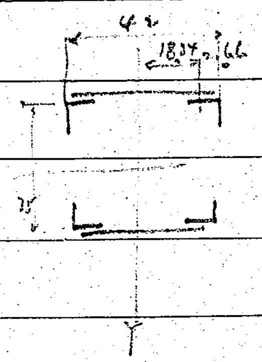


2 Pls 410×9 = 73.80 × $\frac{9\times41^3}{12}$ = 5180

4 Ls 90×90×13 = 86.84 × 18.34² + 158×4 = 29830

160.64 35010

$r_y = \sqrt{\frac{35010}{160.64}} = 14.77$

$l/r = 806/14.77 = 54.6$

$f = 1200 - 5\times54.6 = 927\times1.4 = 1297$

$SR = \frac{204300}{1297} = 157.70$

2 Pls 410×9 = 73.80 $\frac{9\times41^3}{12}$ = 5170

4 Ls 75×75×9 = 50.76 × 18.85² + 54×4 = 18250

124.56 23420

$\sqrt{\frac{23420}{124.56}} = 13.71$

$f = 1200 - 58.8\times5 = 906\times1.4 = 1268$ $l/r = \frac{806}{13.71} = 58.8$

SR = 122.0

29510 / 141.8

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138.72 × 24.78 = 3405.0
171.42 × 8.66 = 1485.0
4926.0 × .1785 = 886 × 4 = 15450

111.48 × 8.66 = 964.
146.68 × 8.06 = 1182.
181.66 × 16.72 = 3035
5181 × .1785 = 4067 × 4 = 16.260
.810

126.36 × 8.50 1050.
126.36 × 8.06 1018
143.60 × 8.06 1157
162.44 × 9.66 1568
4793 × .1785 = 3.760 ×
2723
1038
100
1138 × 4 = 4.550 × 3.0 = 13.65
1.85
15.50

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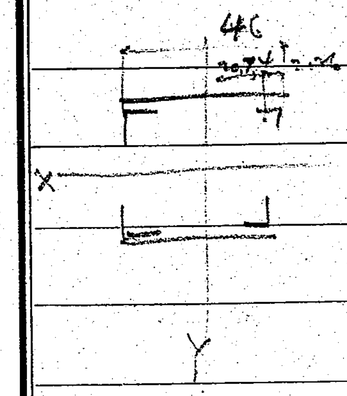
131,76

6840



2 Pls 450×9 = 81,00 $\frac{.9 \times 45^3}{12}$ = 6840

4 L 75×75×12 = 66,24 × 20,74² + 4×83. 28830

147,24

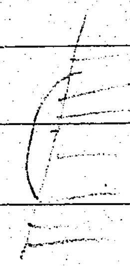
35670

$\sqrt{\frac{35670}{147,24}}$ = 15,58

$\frac{l}{r}$ = 806/15,58 = 51,7

f = 1200 − 5×51,7 = 941 ×1,4 = 1317

SR = $\frac{204300}{1317}$ = 155,0



2 [ 380×100 @ 54,5 = 138,7

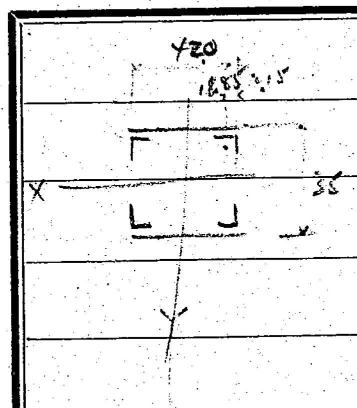
2 [ 380×100 @ 67,5 = 171,4

16,5
15,7

55,5

164
155,0

14,0
15,75





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2Pls $420 \times 9 = 75.6 \times 18.85^2 + 54 \times 4 = \cancel{5?0?0}$ 18270

4L $75 \times 75 \times 9 = 50.76$　$\frac{9 \times 42^3}{12} =$　5170

126.36　23440

$r_y = \sqrt{\frac{\cancel{23440}}{126.36}} = \cancel{13.??}$ 13.62 cm

$l/r = 806/13.62 = \cancel{?} 59.2$

$f = 1200 - 5 \times \frac{\cancel{?}}{59.2} = \cancel{9??}$ 904 $\times 1.4 = 1267$

$440 \times 9 = 79.2$

2Pls $270 \times 9 = 48.6$

126.36

174.96

75.6

66.24

141.84

[illegible]

[illegible]

JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

CALCULATIONS FOR

MADE BY_____ DATE_____ FILE NO_____
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昭和四年四月

越中鐵道株式會社

鋼鈑桁橋應力計算書並材料表

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MADE BY K. Inaba DATE 6-4-7 FILE NO.

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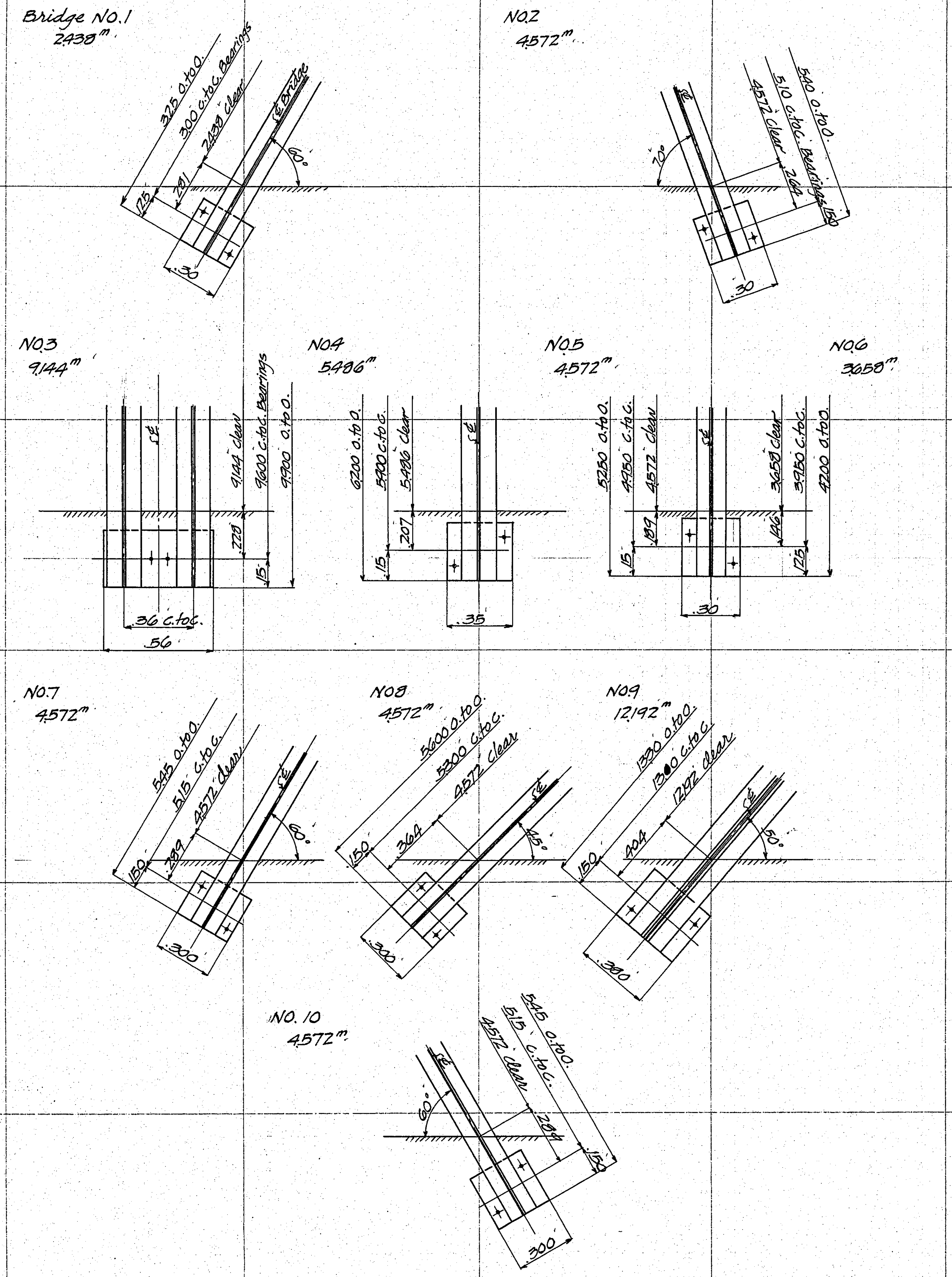
CALCULATIONS FOR

Bridges for Etchu Tetsudo

Effective Span length of Bridges assumed as follows




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

Bridges for Etchu Tetsudo.

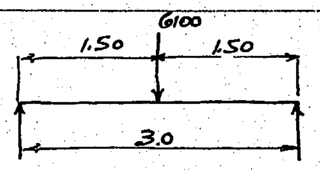
Design of Bridge no. 1. Clear span length = 2.438 m, Effective span length = 3.00 meters. 60° Right skew.

Dead Load assumed

Track construction complete 90 ÷ 2 = 45 #/ft = 72 kg/lin m.
Beam with details assumed 83
155 " for one beam.

Dead Load moment = 1/8 × 155 × 3.0² = 175 kgm.
Dead Load shear = 1/2 × 155 × 3.0 = 233 kg.

Live Load — Wheel concentration = 13440 # = 6100 kg. wheel base = 10' = 3.05 meters.




Live Load moment = 3050 × 1.50 = 4575 kgm.
50% impact = 2288 "
Live Load shear = 6100 kg
50% Impact = 3050 "

Summary of moments + shears.

| | moments | end shear. |
|---|---|---|
| Dead Load | 175 | 233 |
| Live Load | 4575 | 6100 |
| Impact | 2288 | 3050 |
| | 7038 kgm. | 9383 kg. |

Allowable unit fibre stress 15000 #/□" = 1055 kg/cm²

Section modulus required = $\frac{7038 \times 100}{1055}$ = 667 cm³

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

fibre stress = $\frac{7038 \times 100}{870.6}$ = 809 kg/cm² = 11,500 #/□"

Unit shear = $\frac{9383}{35.0 \times .9}$ = 298 " = 4240 "

Design of Bridge No. 2. Clear span length = 4.572 m, Effective span length = 5.100 meters 70° left skew.

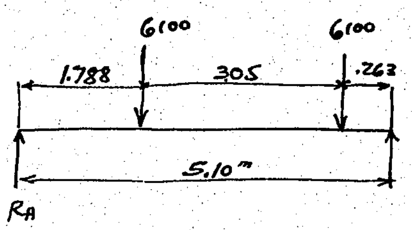
Dead Load assumed

Track construction complete 72.
Beam with details say 148
220 kg per lin meter.

Dead Load moment = 1/8 × 220 × 5.1² = 715 kgm.
Dead Load shear = 1/2 × 220 × 5.1 = 511 kg.

Live Load.

Reaction $R_A$
6100 × .263 = 1604
6100 × 3.313 = 20206
21810 ÷ 5.10 = 4280 kg




Live Load moment = 4280 × 1.788 = 7660 kgm
50% impact = 3830 "
Shear. 6100 × $\frac{2.05}{5.10}$ = 2450
6100
8550 kg.
50% impact 4275 "

Summary for moments and shears.

| | moments | shears. |
|---|---|---|
| Dead Load | 715 | 511 |
| Live Load | 7660 | 8550 |
| Impact " | 3830 | 4275 |
| | 12205 kgm. | 13336 kg. |

Section modulus required = $\frac{12205 \times 100}{1055}$ = 1158 cm³

Use 350 × 150 @ 87.2 kg Sm = 1283 cm³

fibre stress = $\frac{12205 \times 100}{1283}$ = 952 kg/cm² or (13520 #/□")

Unit shear = $\frac{13336}{35.0 \times 1.2}$ = 318 " or (4515 ")

Bridges for Etchu Tetsudo.

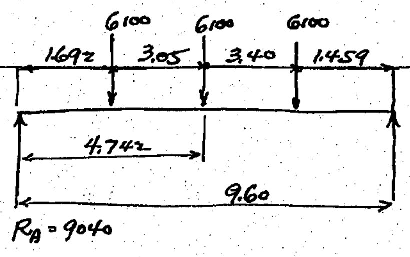
Design of Bridge No 3. Clear span length = 9.144 m Effective span length = 9.600 meters. Square bridge.

Dead Load assumed

Track construction complete 72.
Beams with details say 388
460 kg per lin meter.

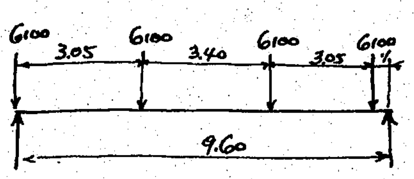
Dead Load moment = 1/8 × 460 × 9.60² = 5,300 kgm.
Dead Load shear = 1/2 × 460 × 9.60 = 2210 kg

Live Load.




Reaction RA
6100 × 1.459 = 8900
6100 × 4.859 = 29620
6100 × 7.909 = 48240
86,760 ÷ 9.60 = 9040 kg.

moment at P.
9040 × 4.742 = 42,870
6100 × 305 = −18610
24,260 kgm.
50% impact 12,130 "




End shear.
6100 × .10 = 610
6100 × 3.15 = 19230
6100 × 6.55 = 39550
59390 ÷ 9.6 = 6190
6100
12290 kg.

50% impact 6145 "

Summary for moments and shear.

| | moments | End shear. |
|---|---|---|
| Dead Load | 5300 | 2210 |
| Live Load | 24260 | 12290 |
| Impact | 12130 | 6145 |
| | 41690 kgm | 20645 kg. |

Section modulus req'd = 41690×100 / 1055 = 3950 cm³

use 2-Is 24"×7½" @ 45.328 kg/ft Sm = 7250 cm³

fibre stress = 41690×100 / 7250 = 575 kg/cm² = 8180 #/□"

unit shear = 20645 / 34 60×1.5 = 230 " 3270 "

Design of Bridge No 4. Clear span 5.486 m, Effective span 5.900 m Square bridge.

Dead Load assumed

Track construction complete 72
Beam with details say 148
220 kg per lin meter.

Dead Load moment = 1/8 × 220 × 5.9² = 958 kgm
Dead Load shear = 1/2 × 220 × 5.9 = 650 kg.

Live Load.

RA
6100 × .67 = 4090
6100 × 3.72 = 22700
26790 ÷ 5.9 = 4540 kg.


moment 4540 × 2.19 = 9,940 kgm.
Impact 4,970 "

Shear 6100 × 2.85/5.9 = 2950
6100
9050 kg imp. 4525 kg.

Summary of moments + shears

| | moments | shears - |
|---|---|---|
| Dead Load | 958 | 650 |
| Live Load | 9940 | 9050 |
| Impact. | 4970 | 4525 |
| | 15,868 kgm | 14225 kg. |

Sm. req'd. = 15868×100 / 1055 = 1510 cm³

use 1 I 450×175 @ 91.7 kg Sm = 1743

fibre stress = 15868×100 / 1743 = 910 kg/cm² = (12950 #/□")

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CALCULATIONS FOR

*Bridges for Etchu Tetsudo.*

Design of Bridge no. 5. Clear span length 4.572 m Effective span length = 4.95 meter, Square.

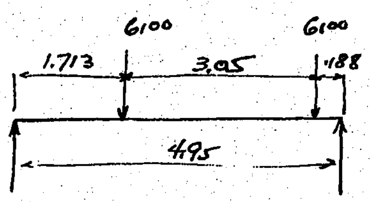
Dead Load.

Track construction 72
Beam with details 123
195 kg per lin m.

Dead load moment $= 1/8 \times 195 \times 4.95^2 = 598$ kgm.
Dead load shear $= 1/2 \times 195 \times 4.95 = 482$ kg.

Live load.

Reaction $R_B$ $6100 \times .188 \div 4.95 = 230$
$6100 \times 3.238 \div 4.95 = 3990$
$4220$ kg.




Live load moment $= 4220 \times 1.713 = 7240$ kgm
50% impact 3620 "
Shear. $6100 \times 1.90 \div 4.95 = 2340$
6100
8440 kg
50% imp. 4220 "

Summary for moments and shears.

| | moments | shears |
|---|---|---|
| Dead load | 598 | 482 |
| Live load | 7240 | 8440 |
| Impact. | 3620 | 4220 |
| | 11458 kgm. | 13142 kg. |

Sm. reqd. $= \frac{11458 \times 100}{1055} = 1086$ cm³

Use I, 350×150 @ 87.2 kg  Sm = 1283

fibre stress $= \frac{1145800}{1283} = 894$ kg/cm² or 12700 #/□"

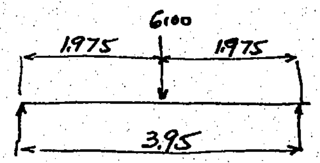
Design of Bridge no. 6. Clear span = 3.658 m, Effective depth = 3.950 m, square bridge.

Dead load assumed.

Track construction complete 72
Beam with details say 113
185 kg per lin m.

Dead load moment $= 1/8 \times 185 \times 3.95^2 = 361$ kgm.
Dead load shear $= 1/2 \times 185 \times 3.95 = 365$ kg.

Live load




Live load moment $6100 \times 3.95 \div 4 = 6020$ kgm.
50% imp $= 3010$ "
Live load shear $6100 \times \frac{.9}{3.95} = 1390$
6100
7490 kg.
50% imp. 3745 "

Summary for moments & shears

| | moments | shears. |
|---|---|---|
| Dead load. | 361 | 365 |
| Live load | 6020 | 7490 |
| Impact load | 3010 | 3745 |
| | 9391 kgm | 11600 kg. |

Sm. reqd. $= \frac{9391 \times 100}{1055} = 890$ cm³.

Use I 14"×6" @ 20.87 kg/ft.  Sm = 1030 cm³.

fibre stress $= \frac{939100}{1030} = 912$ kg/cm² or (12970 #/□")

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CALCULATIONS FOR

Bridges for Etchu Tetsudo.

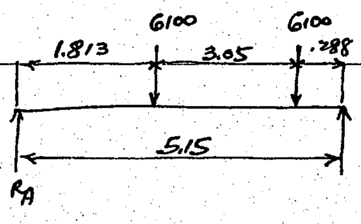
Design of Bridge no 7. Clear span length = 4.572 m, Effective span length = 5.15 m, Skew 60 right.

Dead Load.

Track construction complete 72
Beam with details &c 123
195 kg per lin m.

Dead Load moment = 1/8 × 195 × 5.15² = 647 kgm.
Dead Load shear = 1/2 × 195 × 5.15 = 502 kg.

Live Load




Reaction $R_A$ 6100 × .288 = 1757
6100 × 3.338 = 20350
22107 ÷ 5.15 = 4290 kg

Live Load moment = 4290 × 1.813 = 7780 kgm.

Shear. 6100 × 2.1 ÷ 5.15 = 2485
6100 × 6100
8585 kg.

Summary for moments & shears.

| | moments | shears |
|---|---|---|
| Dead Load | 647 | 502 |
| Live Load | 7780 | 8585 |
| Impact. | 3890 | 4293 |
| | 12317 kgm | 13380 kg |

Section modulus required = $\frac{12317 \times 100}{1055}$ = 1167.0 cm³

Use 1 I 350×150 @ 87.2 kg   Sm = 1283 cm³.

fibre stress = $\frac{1231700}{1283}$ = 960 kg/cm² or 13,640 #/□"

unit shear = $\frac{13380}{35 \times 1.2}$ = 319 kg/cm² or 4,530 #/□"

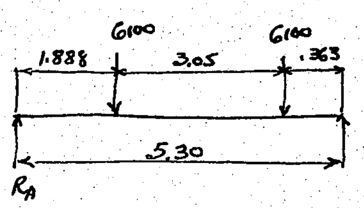
Design of Bridge no 8. Clear span length 4.572 m. Effective span length = 5.30 m, Skew 45° right.

Dead Load assumed.

Track construction complete 72
Beam with details say 123
195 kg per lin m.

Dead Load moment = 1/8 × 195 × 5.3² = 685 kgm.
Dead Load shear = 1/2 × 195 × 5.3 = 517 kg.

Live Load




Live Load reaction $R_A$ 6100 × .363 = 2215
6100 × 3.413 = 20815
23030 ÷ 5.3 = 4350 kg.

Live Load moment = 4350 × 1.888 = 8220 kgm.
Live Load shear = 6100 × $\frac{2.25}{5.3}$ = 2590
6100
8690 kg.

Summary for moments & shears.

| | moments | shears. |
|---|---|---|
| Dead Load | 685 | 517 |
| Live Load | 8220 | 8690 |
| Impact 50% | 4110 | 4345 |
| | 13015 kgm | 13552 kg. |

Sm. required = $\frac{13015 \times 100}{1055}$ = 1235 cm³

Use 1 I. 450×175 @ 91.7   Sm = 1743.

fibre stress = $\frac{1301500}{1743}$ = 747 kg/cm² or (10620 #/□")

Design of Bridge no 10. Clear span length = 4.572 m, Effective span = 5.15 m, Skew 60° Left.

Use same section as for Bridge no 7.

Bridges for Etchu Tetsudo.

Design of Bridge no. 9. Clear span length = 12.192 m; Effective span length = 13.0 m; Skew 50° right.

Dead load assumed

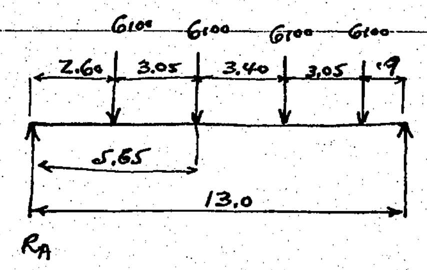
Track construction complete 72.
girders, bracings etc say 328
400 kg per lin meter.

Dead load moments = 1/8 × 400 × 13.0² = 8450 kgm.
Dead load shear = 1/2 × 400 × 13.0 = 2600 kg.

Live load

Reaction $R_A$ 6100 × (.9 + 3.95 + 7.35 + 10.4 [22.6]) / 13.0 = 10600 kg

moment = 10600 × 5.65 = 59900 kgm.
6100 × 3.05 = −18600
41300 kgm.




50% impact 20650

Shear. 6100 × (13.0 + 9.95 + 6.55 + 3.5 + .1 [33.1]) / 13.0 = 15530 kg

50% impact 7765 kg.

Summary for moments and shears.

| | moments | shears |
|---|---|---|
| Dead load | 8450 | 2600 |
| Live load | 41300 | 15530 |
| Impact | 20650 | 7765 |
| | 70400 kgm | 25895 kg |

web assumed 1100 × 9. 1/8 web area = 99/8 = 12.38 cm²
Effective depth say 111.0 − .4 = 109.6 cm
flange stress = 70400 / 1.096 = 64200 kg C or T

use 2 cov pl 230×9 = 41.40 − 7.92 = 33.48
2 ∠ 100×75×10 = 33.00 − 4.40 = 28.60
1/8 web area = 12.38 = 12.38
86.78 cm² gr 74.46 cm² net.

Unit stresses on top flange = 64200 / 86.78 = 741.0 kg/cm² ~ (1052.0 #/□")

do on bottom flange = 64200 / 74.46 = 862.0 " ~ (12250 ")

Unit shear on web = 25895 / 99 = 262 kg/cm² ~ (3710 #/□")

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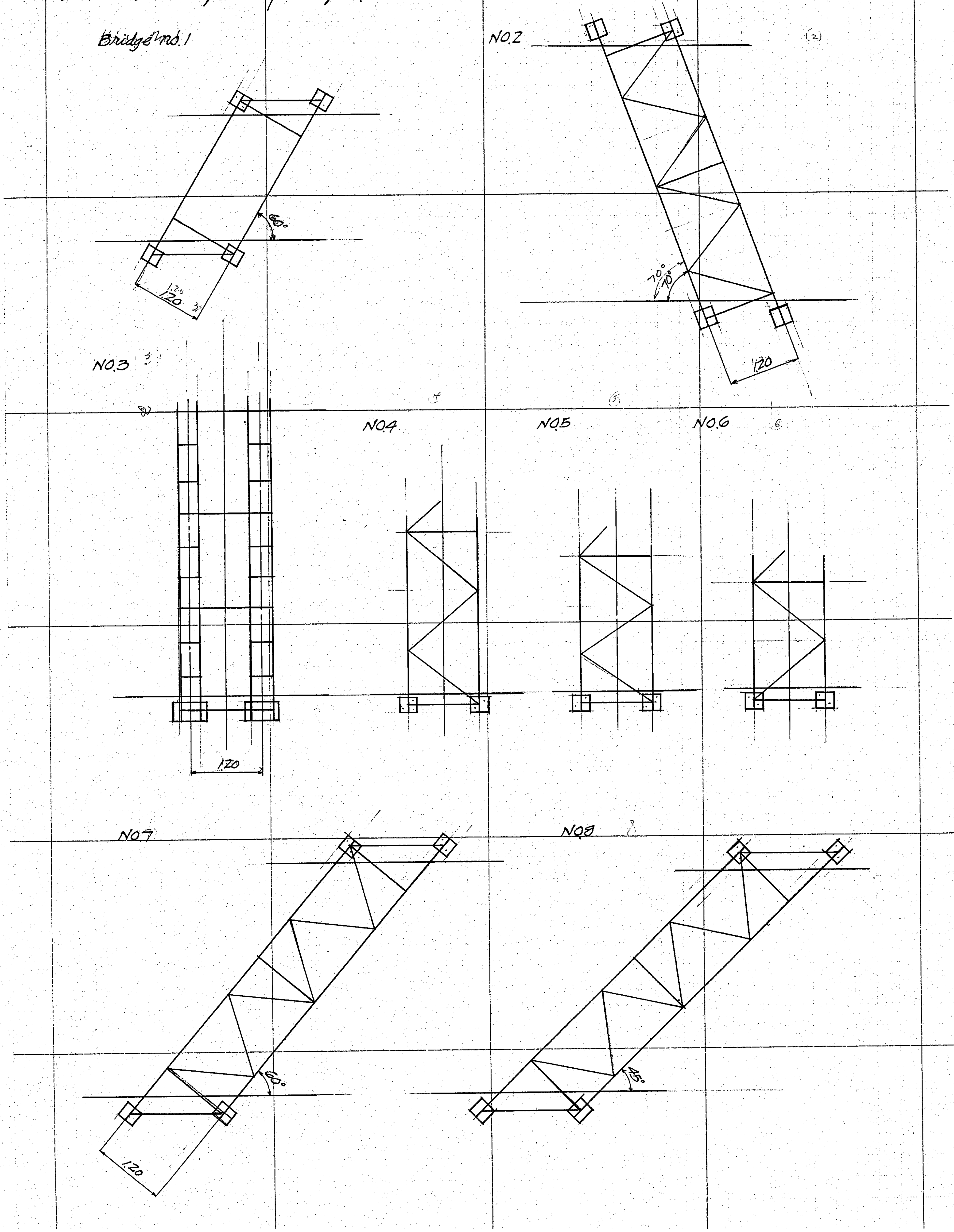
CALCULATIONS FOR

Bridges for Etchu Tetsudo.
Skeleton Diagrams of Bridges.

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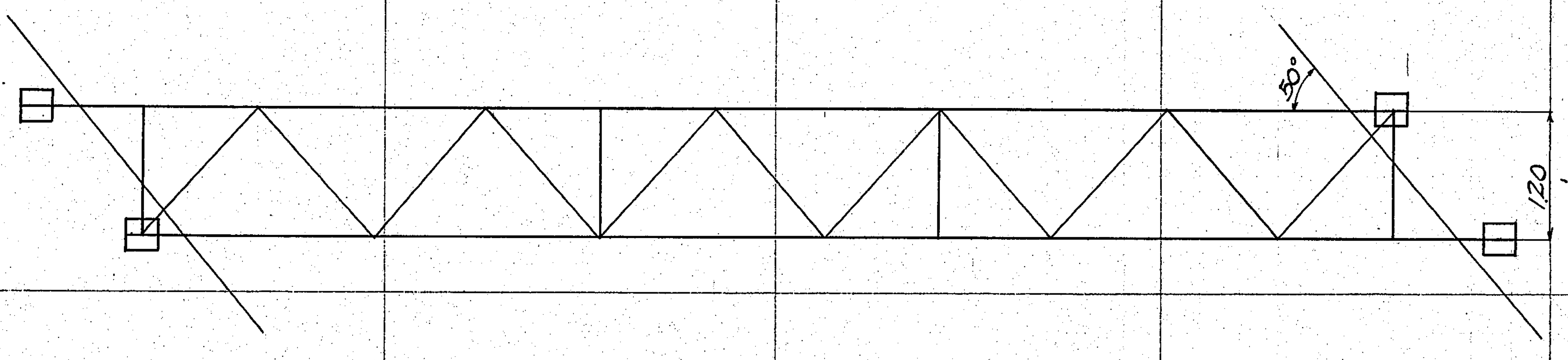
CALCULATIONS FOR

Bridges for Etchu Tetsudo

NO.9

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CALCULATIONS FOR

Bridges for Etchu Tetsudo.

Estimate of Structural steel.

Bridge no. 1.

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| main Beams. | 2 I's | 350 × 150 | @ 58.5 | × 3.25 | = | 380. | |
| Struts | 2 ['s | 230 × 80 | @ 24.8 | × 1.20 | = | 60. | |
| ties | 2 L's | 75×75×9 | @ 9.96 | × 0.90 | = | 18 | |
| Shoes anchor bolts, stiffeners, gusset pls &c say | | | | | = | 122 | |
| | | | | | | | 580 kg |

Bridge no. 2.

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| main Beams. | 2 I's | 350 × 150 | @ 87.2 | × 5.40 | = | 942. | |
| Struts | 3 ['s | 250 × 90 | @ 34.6 | × 1.20 | = | 125 | |
| bracings | 6 L's | 75×75×9 | @ 9.96 | × 1.20 | = | 72 | |
| miscellaneous details say. | | | | | | 201 | |
| | | | | | | | 1340 kg. |

Bridge no 3.

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| main Beams. | 4 I's | 24" × 7½" | @ 148.6 | × 9.900 | = | 5880 | |
| ties, rail bases, Shoes, anchor bolts &c say | | | | | = | 2220 | |
| | | | | | | | 8,100 kg |

Bridge no. 4.

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| main Beams | 2 I's | 450×175 | @ 91.7 | × 6.200 | = | 1136 | |
| Struts | 3 ['s | 300×90 | @ 38.1 | × 1.200 | = | 137 | |
| bracings | 6 L's | 75×75×9 | @ 9.96 | × 1.300 | = | 78 | |
| miscellaneous details say. | | | | | = | 209 | |
| | | | | | | | 1,560 kg |

Bridge no. 5.

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| main Beams | 2 I's | 350×150 | @ 87.2 | × 5.250 | = | 919 | |
| Struts | 3 ['s | 250 × 90 | @ 34.6 | × 1.200 | = | 125 | |
| bracings | 6 L's | 75×75×9 | @ 9.96 | × 1.200 | = | 72 | |
| miscellaneous details say | | | | | | 194 | |
| | | | | | | | 1,310 kg |

Bridge no. 6

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| main Beams. | 2 I's | 14" × 6" | @ 67.5 | × 4.200 | = | 567 | |
| Struts | 3 ['s | 230×80 | @ 24.8 | × 1.200 | = | 89. | |
| bracings | 4 L's | 75×75×9 | @ 9.96 | × 1.300 | = | 52 | |
| miscellaneous details say | | | | | = | 142 | |
| | | | | | | | 850 kg |

Bridge no. 7.

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| main Beams. | 2 I's | 350×150 | @ 87.2 | × 5.450 | = | 950 | |
| Struts | 3 ['s | 250×9 | @ 34.6 | × 1.200 | = | 125 | |
| bracings | 8 L's | 75×75×9 | @ 9.96 | × 1.200 | = | 96 | |
| miscellaneous details say. | | | | | | 229 | |
| | | | | | | | 1,400 kg |

Bridge no. 8.

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| main Beams. | 2 I's | 450 × 175 | @ 91.7 | × 5.600 | = | 1026 | |
| Struts | 3 ['s | 300 × 9 | @ 38.1 | × 1.200 | = | 137 | |
| bracings | 8 L's | 75×75×9 | @ 9.96 | × 1.200 | = | 96 | |
| misc. details say. | | | | | = | 231 | |
| | | | | | | | 1,490 kg. |

Bridge no 10.

Same as for no 7. = 1,400 kg

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CALCULATIONS FOR

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Bridges for Etchu Tetsudo.

Bridge no. 9.

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Flanges. Ls | 8Ls | 100×75×10 | @ | 13.00× | 13.300 | = | 1,384 |
| web pls. | 2Pls | 1100×9 | @ | 77.72× | 13.300 | = | 2070 |
| Cov. pls. top. | 2Pls | 230×9 | @ | 16.25× | 13.300 | = | 432 |
| 〃 〃 | 2Pls | 230×9 | @ | 16.25× | 7.000 | = | 228 |
| 〃 bottom | 2Pls | 230×9 | @ | 16.25× | 9.500 | = | 309 |
| 〃 〃 | 2Pls | 230×9 | @ | 16.25× | 7.000 | = | 228 |
| 〃 splice | 4Pls | 230×9 | @ | 16.25× | 1.500 | = | 98 |
| Spl. Ls | 8Ls | 100×75×10 | @ | 13.00× | .900 | = | 94 |
| Spl. Pls. | 8Pls | 250×9 | @ | 17.66× | .600 | = | 85 |
| 〃 | 4Pls | 300×9 | @ | 21.20× | .450 | = | 38 |
| Stiffeners end. | 16Ls | 100×75×10 | @ | 13.00× | 1.090 | = | 227 |
| fills | 8 fills | 150×10× | @ | 11.78× | 0.950 | = | 89 |
| stiffs. int. | 32 Ls | 75×75×9 | @ | 9.96× | 1.110 | = | 354 |
| 〃 〃 | 8 Ls | 75×75×9 | @ | 9.96× | 1.090 | = | 87 |
| fills 〃 | 4 fills | 75×10 | @ | 5.89× | .950 | = | 22 |
| Cross frames | 4 sets. | | @ | 50.0 | | = | 200 |
| laterals | 11 Ls | 75×75×9 | @ | 9.96× | 1.42 | = | 156 |
| lateral pls. | 12 Pls | | @ | 11.0 | | = | 132 |
| Shoes, beapls, anchor bolts to say | | | | | | = | 480 |
| Rivet heads. say | | | | | | = | 217 |
| | | | | | | | 6,930 kg |

Summary for Structural steel.

| Bridge no | Clear span length. | Skew angles. | Structural steel. kg |
|---|---|---|---|
| 1 | 2.438 m | 60° Right | 580 |
| 2 | 4.572 | 70° Left. | 1,340 |
| 3 | 9.144 | square | 8100 |
| 4 | 5.486 | 〃 | 1,560 |
| 5 | 4.572 | 〃 | 1,310 |
| 6 | 3.658 | 〃 | 2@850 = 1,700 |
| 7 | 4.572 | 60° Right | 1,400 |
| 8 | 4.572 | 45° 〃 | 1,490 |
| 9 | 12.192 | 50° 〃 | 6930 |
| 10 | 4.572 | 60° Left. | 1,400 |
| | | | 25,810 kg. |

Bridges for Etchū Tetsudo.

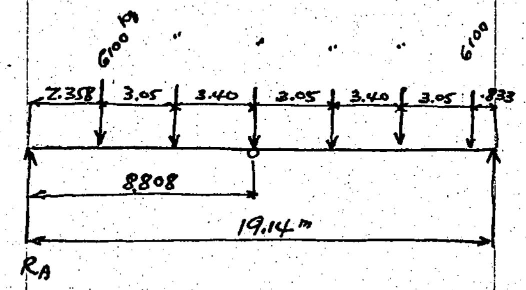
Design of 18.288 meter Girder span. Effective span length = 19.14 meters.

Dead load assumed.

| | |
|---|---|
| Track construction | 72 |
| main girder assumed | 320 |
| lateral bracing &c say | 28 |
| | 420 kg per lin m of one girder. |

Dead load moment = $\frac{1}{8} \times 420 \times 19.14^2 = 19210$ kgm
Dead load shear = $\frac{1}{2} \times 420 \times 19.14 = 4020$ kg

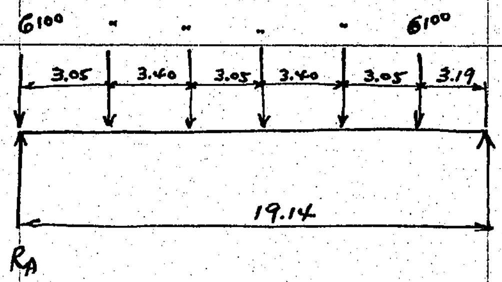
Live load.




Reaction $R_A$ = 6100 × .833
3.883
7.283
10.333
13.733
16.783

6100 × 52.848 ÷ 19.14 = 16,850 kg

moment. 16850 × 8.808 = 148300
6100 × 3.400 = − 20750
6100 × 6.450 = − 39300
88,250 kgm



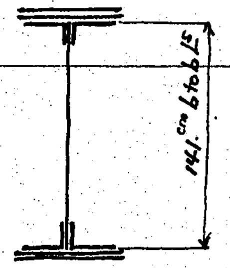

End shear.
Reaction $R_A$ 6100 × 3.19
6.24
9.64
12.69
16.09
19.14

6100 × 66.99 ÷ 19.14 = 21,360 kg

Summary for moments and shears.

| | moment | End shear |
|---|---|---|
| Dead load | 19,210 | 4,020 |
| Live load | 88,250 | 21,360 |
| Impact · 50% | 44175 | 10680 |
| | 151,585 kgm | 36,060 kg. |

Web assumed 1400 mm × 9, web area = 136.0 cm², 1/8 web area = 15.75 cm²
Effective depth say 141 − .72×2 = 139.56 cm




flange stress = $\frac{151585}{1.3956}$ = 108,700 kg T or C

flange area required = $\frac{108.700}{1055}$ = 103.0 cm².

use

| | | | |
|---|---|---|---|
| 2 L's 150 × 100 × 12 | = 57.12 − 12.0 = 45.12 | | |
| 2 cov. pls. 320 × 9 | = 57.60 − 9.0 = 48.60 | | |
| 1/8 web area | | 15.75 | 15.75 |
| | | 130.47 cm² gr | 109.47 cm² net. |

Unit stress on bottom flange = $\frac{108.700}{109.47}$ = 993 kg/cm² or (14120 #/□")

" " top flange = $\frac{108.700}{130.47}$ = 806 " or (11,470 ")

unit shear on web = $\frac{36060}{136}$ = 258 " or (3660 ")

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CALCULATIONS FOR

MADE BY K. Inaba DATE 6-4-10 FILE NO

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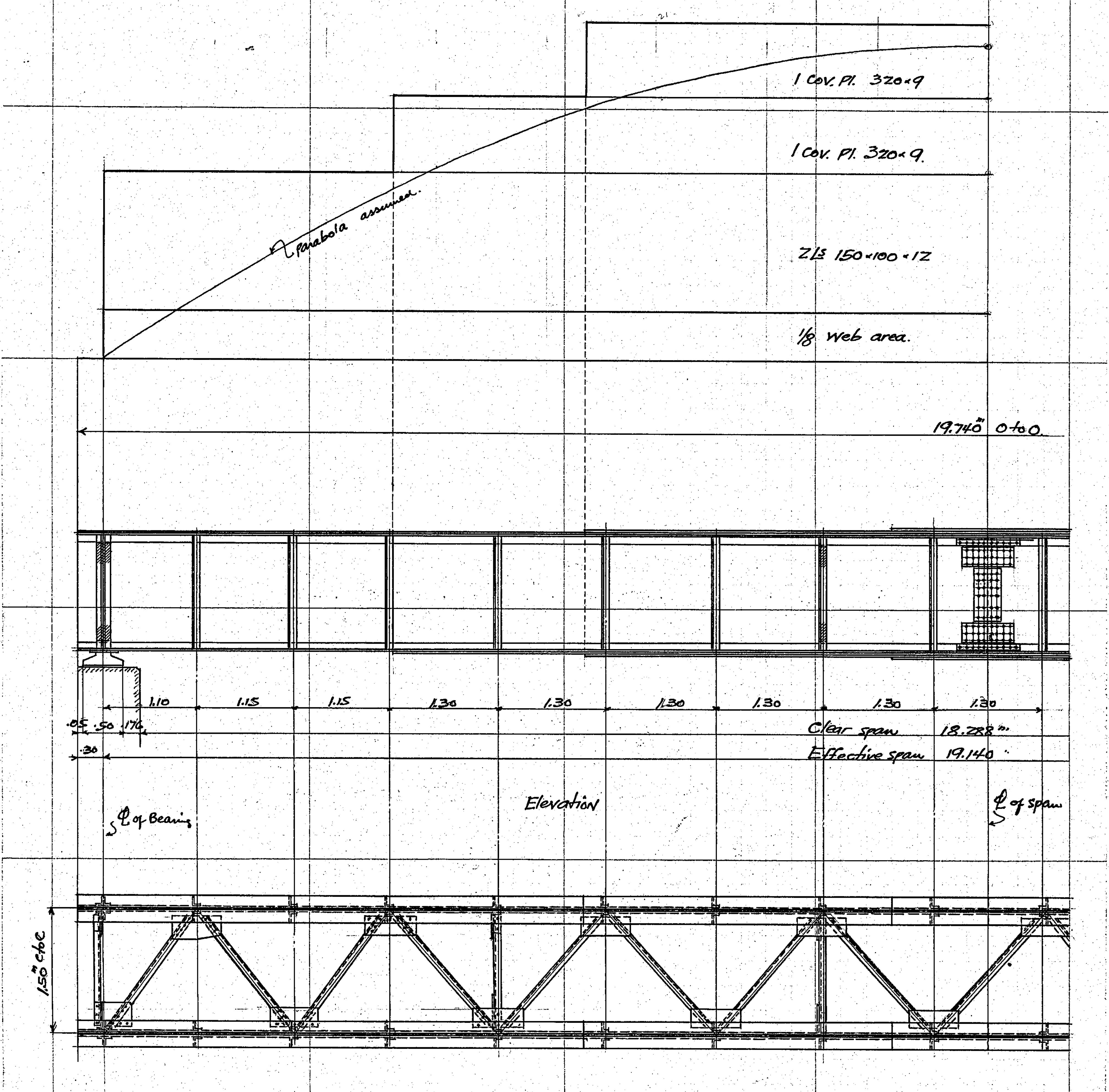
Bridges for Etchu Tetsudo.

General Dimensions and Details of main girders.




PLAN.

Scale 1:50.

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

Bridges for Etchu Tetsudo.

Estimate of structural steel in one span.

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Flange | 8Ls | 150 × 100 × 12 | @ 22.40 kg | × 19.740 m | = | 3,535 kg | |
| web. | 2Pls | 1400 × 9 | @ 98.91 | × 19.740 | = | 3,900 | |
| Cov. pl. | 2Pls | 320 × 9 | @ 22.61 | × 19.740 | = | 893 | |
| " | 2Pls. | 320 × 9 | @ 22.61 | × 14.200 | = | 642 | |
| " | 4Pls | 320 × 9 | @ 22.61 | × 9.600 | = | 869 | |
| " splice | 4Pls | 320 × 9 | @ 22.61 | × 2.300 | = | 208 | |
| Splice | 8Ls | 150 × 100 × 12 | @ 22.40 | × 0.800 | = | 143 | |
| " | 8Pls. | 250 × 9 | @ 17.66 | × 0.640 | = | 91 | |
| " | 4Pls | 320 × 9 | @ 22.61 | × 0.690 | = | 62 | |
| Stiffeners (end) | 16Ls | 125 × 90 × 10 | @ 16.10 | × 1.386 | = | 357 | |
| fills | 8 fills | 180 × 12 | @ 16.96 | × 1.200 | = | 163 | |
| Stiffeners (int.) | 56Ls | 125 × 75 × 9 | @ 13.50 | × 1.410 | = | 1066 | |
| " | 8Ls | 125 × 75 × 9 | @ 13.50 | × 1.386 | = | 150 | |
| fills | 8 fills | 75 × 12 | @ 7.07 | × 1.200 | = | 68 | |
| bed pls | 2Pls | 350 × 25 | @ 68.69 | × 0.400 | = | 55 | |
| " | 2Pls | 350 × 25 | @ 68.69 | × 0.330 | = | 45 | |
| | | | | | | | 12,247 kg |
| Lateral bracings | 4Ls | 100 × 100 × 10 | @ 14.90 | × 1.700 | = | 101 | |
| " | 6Ls | 90 × 90 × 10 | @ 13.30 | × 1.800 | = | 144 | |
| " | 7Ls | 75 × 75 × 9 | @ 9.96 | × 1.800 | = | 125 | |
| " Plates. | 2Pls | 300 × 9 | @ 21.20 | × 0.400 | = | 17 | |
| " " | 2Pls | 300 × 9. | @ 21.20 | × 0.630 | = | 27 | |
| " " | 14Pls | 235 × 9 | @ 16.60 | × 0.610 | = | 142 | |
| Cross frames. | 12Ls | 90 × 90 × 10 | @ 13.30 | × 1.200 | = | 192 | |
| " " | 12Ls | 90 × 90 × 10 | @ 13.30 | × 1.700 | = | 271 | |
| " " | 24Pls | 300 × 9 | @ 21.20 | × 0.320 | = | 163 | |
| " " | 6 Pls | 75 × 9 | @ 5.30 | × 0.180 | = | 6 | |
| " " | 6 fills | 75□ × 9 | @ .35 | | = | 2 | |
| | | | | | | | 1,190 kg. |
| Rivet heads | say 4 % | | | | = | | 537 kg |
| Shoes, anchor bolts &c say | | | | | = | | 556 kg |
| | | | | | | | 14,530 kg |

for 21 spans @ 14.530 = 305.130 kg tons

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Bridges for Etchu Tetsudo.

Design of 24.775 m Girder span. Effective span length = 25.700 meter. (26.5526 m c to c of piers)

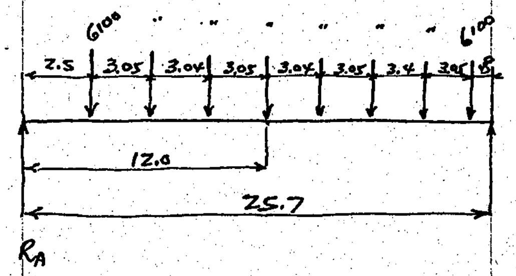
Dead load assumed

| | |
|---|---|
| Track construction complete | 82 |
| main girder assumed | 500 |
| lateral bracings etc say | 48 |
| | 630 kg per lin meter for one girder. |

Dead load moment = 1/8 × 630 × 25.7² = 52000 kgm

Dead load shear = 1/2 × 630 × 25.7 = 8,100 kg

Live load.



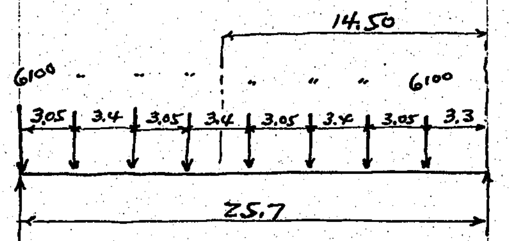

Reaction $R_A$ 6100 × .80
3.85
7.25
10.30
13.70
16.75
20.15
23.20

6100 × 96.00 ÷ 25.70 = 22,780 kg

Live load moment.

22780 × 12.0 = 273000
6100 × 19.0 = − 116,000
157000 kgm.

50% impact. 78,500 "




Live load end shear.

$6100 \times 8 \times \frac{14.50}{25.70} =$ 27,500 kg

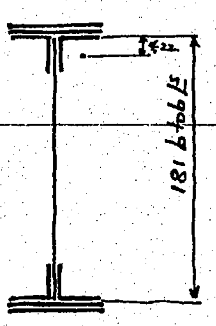
50% impact 13,750 kg.

Summary for moments and shears.

| | moments | End shears |
|---|---|---|
| Dead load | 52000 | 8,100 |
| Live load | 157,000 | 27,500 |
| Impact 50%. | 78,500 | 13,750 |
| | 287,500 kgm. | 49,350 kg. |

Web assumed 1800 × 9. web area = 180 × .9 = 162.0 cm², 1/8 web area = 20.25 cm²

back to back of L's 181 cm. Effective depth say 181.0 − 2 × 1.66 = 177.68 cm.




flange stress = $\frac{287500}{1.7768}$ = 161,800 kg C or T.

Flange area required = $\frac{161800}{1055}$ = 153.5 cm²

Use.

| | | | |
|---|---|---|---|
| 2L's 150 × 150 × 15 = | 85.48 | − 15.00 = | 70.48 |
| 2 cov. pls. 320 × 12 = | 76.80 | − 12.00 = | 64.80 |
| 1/8 web area = | 20.25 | | 20.25 |
| | 182.53 cm² gr. | | 155.53 cm² net |

85.48 × 4.22 = − 361.0
76.80 × 1.20 = + 92.0
162.28 × 1.66 cm = − 269.0

Unit stress on bottom flange = $\frac{161800}{155.53}$ = 1040 kg/cm² or (14,770 #/□")

" " " top flange = $\frac{161800}{182.53}$ = 886 " or (12,600 ")

Unit shear on web = $\frac{49350}{162}$ = 305 kg/cm² or (4330 #/□")

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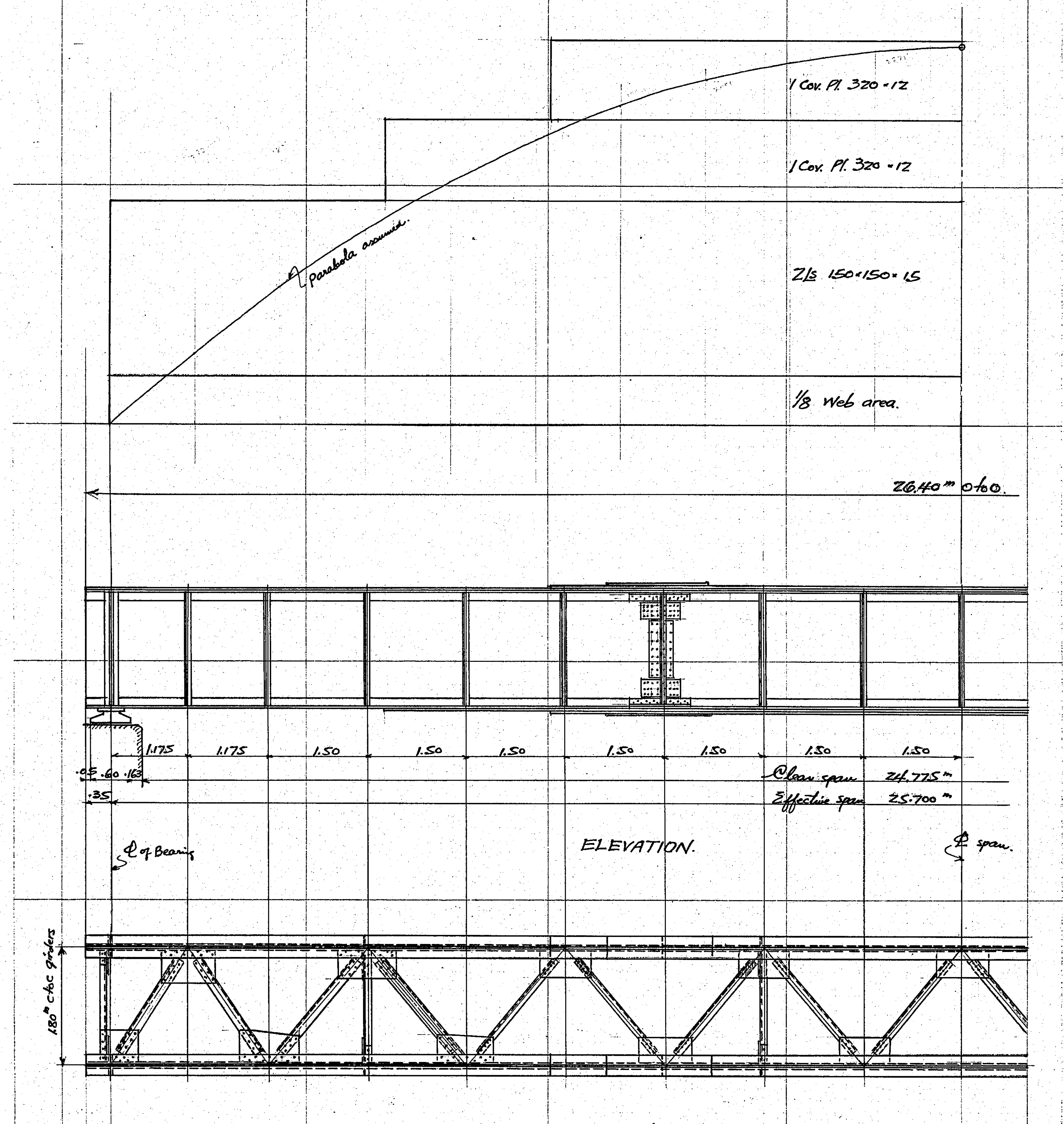
CALCULATIONS FOR

Bridges for Etchu Tetsudo.
General Dimensions and Details of main girders.




PLAN.

Scale 1:60.

3

Bridges for Etchu Tetsudo.

Wind Bracings.

Wind Load assumed as follows.

Exposed area of Bridge girder 1.85
rail + sleeper .25
2.10 m

Wind load. 2.10 @ 300 = 630 kg per lin meter of Bridge.

or. 2.10 @ 200 = 420
wind load on cars. 600
1020 kg " " " (moving load)

| Panel | Shear. | Coeff. | Stress in diagonal | 22φ rivet req'd. | use | |
|---|---|---|---|---|---|---|
| 1 st | 13,100 kg × | 1.195 = | 15650 kg T or C | 5.5 | 6 | } 1L 150×100×12 = 28.56-3.0 = 25.56 net |
| 2 nd | 11,940 | " | 14260 | 5.0 | 6 | |
| 3rd | 10,820 | 1.300 | 14060 | 4.9 | 5 | } 1L 150×100×9 = 21.69-2.25 = 19.44 net |
| 4th | 9480 | " | 12320 | 4.3 | 5 | |
| 5th | 8210 | " | 10670 | 3.7 | 4 | } 1L 100×100×10 = 19.00 - 2.50 = 16.50 net |
| 6th | | | | | 4 | |
| 7th | | | | | 4 | |
| 8th | | | | | 4 | |
| 9th | | | | | 4 | |

for 1st + 2nd panels unsupported length $l$ = 180 cm
1L 150×100×12 = 28.56 - 3.0 = 25.56 cm² net. least radius of gyration $r$ = 2.14 cm $l/r = \frac{180}{2.14} = 84.1$

allowable unit comp. $f = 1200 - 5 \times 84.1 = 780$ kg/cm² gross area req'd. = $\frac{15650}{780}$ = 20.20 cm² gr.

net area req'd = $\frac{15650}{1055}$ = 14.83 cm² net

for 3rd + 4th panels $l$ = ~~180~~ 200 cm $r_{min}$ = 2.04
1L 150×100×9 = 21.69 - 2.25 = 19.44 cm² net $l/r = \frac{200}{2.04} = 98$

$f = 1200 - 5 \times 98 = 710$ kg/cm² C gross area req'd. = $\frac{14060}{710}$ = 19.81 cm² gr.

net area req'd. = $\frac{14060}{1055}$ = 13.35 cm² net.

for 5th to 9th panels + inclusive. $l$ = 200 cm
1L 100×100×10 = 19.00 - 2.50 = 16.50 cm² net. $r_{min}$ = 1.92 cm. $l/r$ = 200/1.92 = 104.

$f = \frac{21000,000}{3} \times \left(\frac{1}{104}\right)^2 = 647$ kg/cm² C gross area req'd. = $\frac{10670}{647}$ = 16.49 cm² gr.

net area req'd. = $\frac{10670}{1055}$ = 10.12 cm² net.

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CALCULATIONS FOR

Bridges for Etchu Tetsudo.

Estimate of Structural Steel in one span.

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Flange | 8 Ls | 150 × 150 × 15 | @ | 33.60 | × | 24.60 m | = | 6,610 kg |
| web. | 2 Pls. | 1800 × 9 | @ | 127.17 | × | 24.60 | = | 6,250 |
| Cov. pls. | 2 Pls | 320 × 12 | @ | 30.14 | × | 24.60 | = | 1,483 |
| 〃 | 2 Pls | 320 × 12 | @ | 30.14 | × | 17.50 | = | 1,055 |
| 〃 | 4 Pls | 320 × 12 | @ | 30.14 | × | 12.50 | = | 1,507 |
| 〃 splice | 8 Pls | 320 × 12 | @ | 30.14 | × | 2.40 | = | 578 |
| Splice | 16 Ls | 150 × 150 × 15 | @ | 33.60 | × | 0.94 | = | 506 |
| 〃 | 16 Pls | 280 × 9 × | @ | 19.78 | × | 0.63 | = | 199 |
| 〃 | 8 Pls | 320 × 9 | @ | 22.61 | × | 0.93 | = | 168 |
| Stiffeners (end) | 16 Ls | 125 × 90 × 13 | @ | 20.60 | × | 1.78 | = | 586 |
| fills | 8 Pls | 180 × 15 | @ | 21.20 | × | 1.50 | = | 255 |
| Stiffeners (int) | 52 Ls | 125 × 75 × 9 | @ | 13.50 | × | 1.81 | = | 1,270 |
| 〃 〃 | 16 Ls | 125 × 75 × 9 | @ | 13.50 | × | 1.78 | = | 384 |
| fills | 8 Pls | 75 × 15 | @ | 8.83 | × | 1.50 | = | 106 |
| bed pls | 2 Pls | 360 × 25 | @ | 70.65 | × | 0.45 | = | 64 |
| 〃 | 2 Pls | 360 × 25 | @ | 70.65 | × | 0.38 | = | 54 |
| | | | | | | | | 21,075 |
| Lateral Bracing | 4 Ls | 150 × 100 × 12 | @ | 22.40 | × | 1.70 | = | 152 |
| 〃 〃 | 4 Ls | 150 × 100 × 9 | @ | 17.00 | × | 1.80 | = | 123 |
| 〃 〃 | 10 Ls | 100 × 100 × 10 | @ | 14.90 | × | 1.85 | = | 276 |
| 〃 plates | 2 Pls | 480 × 9 | @ | 33.91 | × | 0.50 | = | 34 |
| 〃 〃 | 4 Pls | 480 × 9 | @ | 33.91 | × | 0.80 | = | 109 |
| 〃 〃 | 4 Pls | 480 × 9 | @ | 33.91 | × | 0.80 | = | 109 |
| 〃 〃 | 9 Pls | 340 × 9 | @ | 24.02 | × | 0.72 | = | 156 |
| fills (Ls) | 4 Pls | 100 × 15 | @ | 11.78 | × | 0.50 | = | 24 |
| Cross frames. | 4 Ls | 150 × 100 × 9 | @ | 17.00 | × | 1.75 | = | 119 |
| 〃 | 8 Ls | 90 × 90 × 10 | @ | 13.30 | × | 1.75 | = | 186 |
| 〃 | 12 Ls | 90 × 90 × 10 | @ | 13.30 | × | 1.50 | = | 239 |
| 〃 | 8 Pls | 340 × 9 | @ | 24.02 | × | 0.40 | = | 77 |
| 〃 | 16 Pls | 320 × 9 | @ | 22.61 | × | 0.32 | = | 116 |
| 〃 | 6 Pls | 90 × 9 | @ | 6.36 | × | 0.18 | = | 7 |
| 〃 | 6 〃 | 75φ × 9 | @ | .35 | | | = | 2 |
| | | | | | | | | 1729 |
| Rivet heads. say | | 4% | | | | | = | 826 |
| Shoes, anchor bolts etc say | | | | | | | = | 770 |
| | | | | | | | | 24,400 kg |

or 24.40 kg tons for one span.