JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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CALCULATIONS FOR

DESIGN

OF

FURUKAWA-BASHI

FOR

TOKUSHIMA-KEN

MADE BY JIUN MASUDA.

大正十五年
五月成

JIUN MASUDA
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SEIYU BLDG, TOKIO

MADE BY J. Masuda DATE 15-4-24 FILE NO.

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CALCULATIONS FOR

Final design of Furukawa-Bashi, Tokushima-Ken

This bridge shall be located across the Yoshinogawa on main highway between the City of Tokushima and the suburban town of Furukawa. The total length of the bridge 3528.381 shaku between center of End bearings of End truss spans or 17 spans of 203'-11½" and 16 spaces of 2'-6½" between bearings of trusses on piers. The entire bridge will be cambered at center 4.5 shaku to meet the grades of approaches at the both ends.

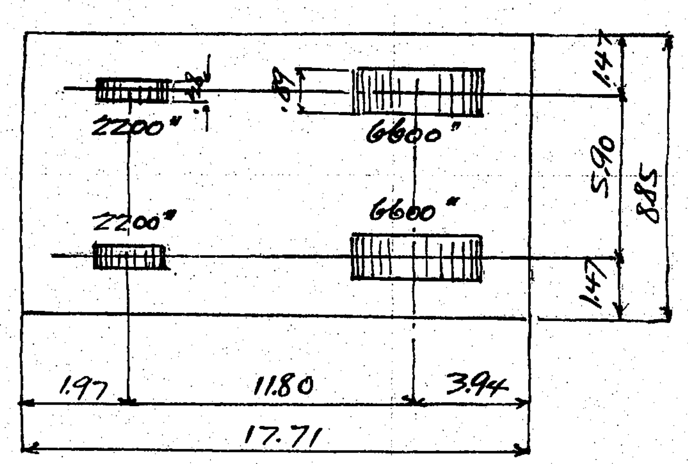
Clear roadway 20.0 shaku between curb lines, the roadway slab will be of reinforced concrete paved with 2" asphaltic concrete or Solidite.

Loadings on bridge.

Uniform live Load $q^{kg/m^2} = \frac{100.000}{170+l}$ where l = span length in meter

Under 30 meter in span use $q = 500$ kg/m2 or say 100.0 #/□'

8 ton motor truck loading (17600#).



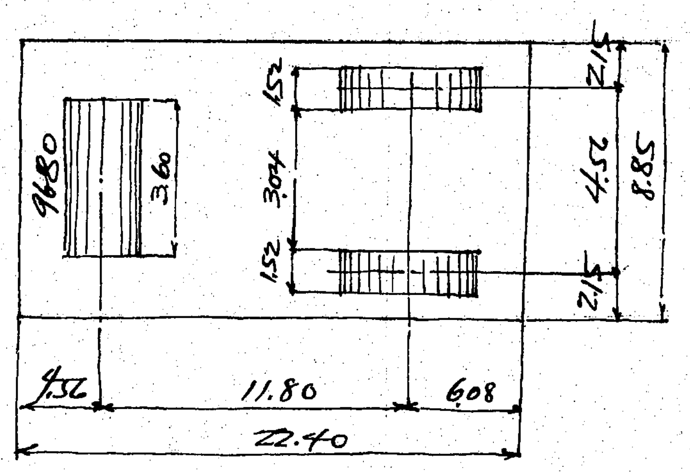
2 rows of motor traffic on roadway with impact; occupied space 8.85' each. Unoccupied space of motor trucks shall be filled in with above Uniform load.
One motor truck in each row on one span assumed.

One roadroller on span assumed. No impact.

Impact
For motor truck loading
Impact $= \frac{20}{60+l}$
where l = span length in meter
max impact limited to 30%

No impact for roadroller and Uniform load.

11 ton Roadroller (24200#).



Allowable working strength
Structural steel or reinforcing bars.

Tension		17000 #/□"
Extreme fibre stress		17000 "
Shear on web gross section		12800 "
Compression member gross section	21300 (1 − 0.0055 l/r) or not over	14000 #/□"
Compression flange of plate girder	17000 (1 − 0.012 l/b) or not over	15400 #/□"

where l = Unsupported length of flange in inches
b = width of flange in inches.

Shearing on shop driven rivets (machine driven)		12000 #/□"
" " field "	"	10000 #/□"
Extreme fibre stress of pin.		24000 #/□"

Final design of Furukawa-Bashi for Tokushima-Ken.

Bearing on shop rivets	24000 #/□"
Bearing on field rivets	20.000 #/□"
Bearing on pin	24000 #/□"

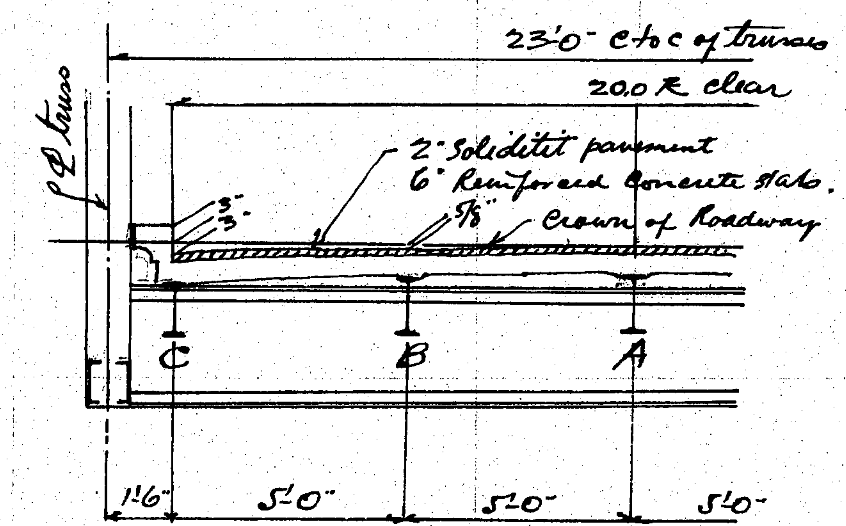
Expansion roller 610 d per lin. inch where d = diameter of roller in inches
Bearing on masonry 640 #/□"

Allowable stress in Concrete 1:2:4 mixture.

Compressive fibre stress	640 #/□"
Shear for plain Concrete	58 "
Punching shear	128 "
Bond stress of plain bar	85 "
Shear for reinf. concrete with web reinf.	128 "
Bond stress of deformed bar	130 #/□"

Considering wind and temperature stress in addition to dead, live and impact stresses the allowable working strength shall be increased 25% and proportioned the parts. In case of earthquake, the working strength shall be increased 80% and proportioned the parts. Acceleration of earthquake assumed 3000 mm/sec² or k = 0.3

Cross section of Bridge.



Span length of truss 203' 11½"
Head clearance above crown of roadway = 15.0
Centre to center of trusses 23'0"
The span divided into 10 panels of 20' 4 3/8" each

Roadway slab. spacing of stringers 5'0"
Dead Load.

Pavement 2" Soliditit	22
6" reinforced Concrete slab.	75
	97# per sq ft.

Dead Load moment $= \frac{1}{10} \cdot 97.0 \cdot 5.0^2 = 242$ '#
Dead Load shear $= \frac{1}{2} \cdot 97 \cdot 5.0 = 242$ #

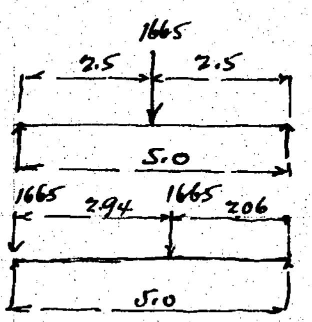
Live Load motor truck rear wheel concentration 6600#
Impact 30%. 1980
8580#

Front wheel conc $= \frac{1}{3} \cdot 8580 = 2860$#

Distribution of wheel concentration
Contact between wheel and pavement assumed 20 centimeter = 0.66
2 · pavement e.17 = 0.34
Longitudinal distribution a --- = 1.00
Transverse distribution b = 0.89 + 0.34 = 1.23

Effective width $E = \frac{2}{3}(l+b) + a = \frac{2}{3}(5.00 + 1.23) + 1.00 = 5.15'$
Load per ft strip = 8580 ÷ 5.15 = 1665#

moment due to single wheel load $\frac{1665}{2} \cdot 2.5 = 2080$



For continuity of beam 0.8 · 2080 = 1665 '#
Max End shear as simple beam
$1665 \cdot \frac{2.06}{5.0} = 687$
1665
2352#

Final design of Furukawa-Bashi for Tokushima-Ken.

Summary for moments and shears

	moments	shears
Dead Load	242	242
Live Load	1665	2352
	1907#	2594#

Effective depth of slab for steel stress of 17000#/□" and Concrete stress of 640#/□"

$d = \sqrt{\frac{1907}{102}} = 4.36''$

Use 6" slab and effective depth say 4.75"

Steel Area required $= \frac{1907 \cdot 12}{\frac{7}{8} \cdot 4.75 \cdot 17000} = 0.324$□" per ft. Use ½"ϕ bars 6" centers = 0.390□"

Shear $= \frac{2594}{\frac{7}{8} \cdot 4.75 \cdot 12} = 52$#/□"

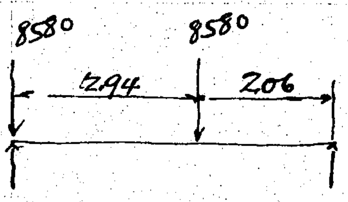
Bond stress $= \frac{2594}{\frac{7}{8} \cdot 4.75 \cdot 3.14} = 200$#/□" Lap ½"ϕ bars at center of bridge, the bond stress will be reduced to 100#/□" O.K.

Longitudinal stringer A. span length 20'-4¾" (20.4). Stringer spacing 5.0'

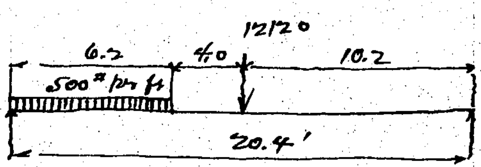
Dead Load of floor 97.0 · 5.0 = 485.0
beam assumed 60.0
545.0# per lin. ft.

Dead Load moment $= \frac{1}{8} \cdot 545 \cdot 20.4^2 = 28400$#
Dead Load shear $= \frac{1}{2} \cdot 545 \cdot 20.4 = 5560$#

Live Load motor truck loading Rear wheel Conc. 8580# with 30% impact
Front wheel Conc. 2860 " " "

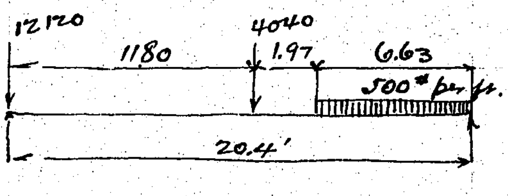


max load on stringer slab as simple beam
Rear wheel conc. $8580 \cdot \frac{2.06}{5.00} =$ 3540
8580
12120#



Uniform live load 100 · 5.0 = 500# per lin. ft. of span
Total unif. load 500 · 6.2 = 3100# $R = 3100 \cdot \frac{3.1}{20.4} = 470$#

moment due to motor trucks $\frac{12120}{2} \cdot 10.2 = 61900$#
" " " Unif. load 470 · 10.2 = 4810
66710#



Max End shear
shear due to motor trucks $4040 \cdot \frac{8.6}{20.4} =$ 1700
rear wheel conc. 12120
shear " " Unif. load $\frac{500 \cdot 6.63 \cdot 3.32}{20.4} =$ 540
14360#

Summary for moments and shears

	moment	shear
Dead Load	28400	5560
Live Load	66710	14360
	95110#	19920#

section modulus req'd $= \frac{95110 \cdot 12}{17000} = 67.0$

Use 18" · 6" @ 54.7# Sm = 88.4

Longitudinal stringers B. span length 20'-4¾"

Dead Load slab and pavement 97 · 5.0 = 485
beam assumed 45
530# per lin. ft. of span

Dead Load moment $= \frac{1}{8} \cdot 530 \cdot 20.4^2 = 27600$#
~~Live Load moment~~
Dead Load shear $= \frac{1}{2} \cdot 530 \cdot 20.4 = 5410$#

Live Load motor truck loading
Rear wheel 8580# with 30% impact
Front wheel 2680 " " "

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

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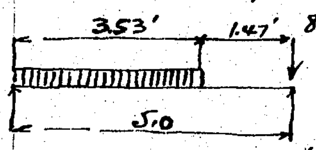
CALCULATIONS FOR

Final design of Furukawa-Bashi for Tokushima-Ken.



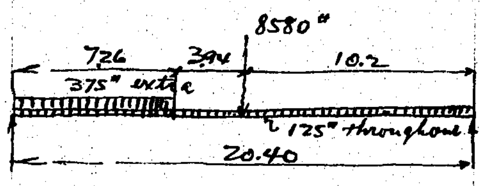
stringer B

wheel concentration 8580#

Full uniform load 100 · 5.0 = 500# per lin. ft.

Partial unif. load 100 · 3.53 · $\frac{1.77}{5.00}$ = 175# per ft on side of motor truck loading

At rear of motor truck the full unif. load shall be figured

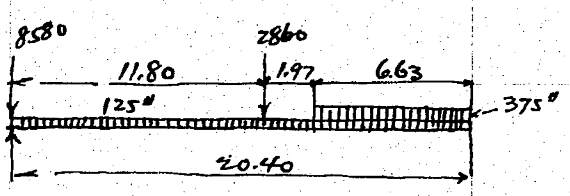


Moment due to motor truck $\frac{8580}{2}$ · 10.2 = 43800

Unif. load 125# A m = $\frac{1}{8}$ · 125 · 20.4² = 6500

Unif. load B 375# m = $\frac{375 \cdot 7.26 \cdot 3.63}{20.4}$ · 10.2 = 4940

55240#



Max End Shear

Rear wheel conc			8580
Front wheel conc	$\frac{2860 \cdot 8.6}{20.4}$	=	1200
Unif. load A	125 · $\frac{20.4}{2}$	=	1275
Unif. load B	$\frac{375 \cdot 6.63^2}{2 \cdot 20.4}$	=	395
			11450#

Summary for moments and shears.

	moments	shears
Dead Load	27600	5410
Live Load	55240	11450
	82840'#	16860#

section modulus req'd = $\frac{82840 \cdot 12}{17000}$ = 58.6

Use 15", 5½ I @ 42.9# Sm = 58.9 Ok

Longitudinal stringers C span length 20' 4¾"

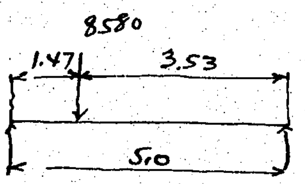
Dead Load

slab and pavement	97.0 · 2.5	=	243
Coping 0.5 · .75 = 0.375			
0.3 · .60 = 0.180			
0.42 · 0.4 = 0.168			
0.723 @ 150#		=	109
Handrails say			30
Beam assumed			35
			417# per lin ft of span

Dead Load moment = $\frac{1}{8}$ · 417 · 20.4² = 21600'#

Dead Load Shear = ½ · 417 · 20.4 = 4260#

Live Load motor truck loading Rear wheel 8580# with 30% impact

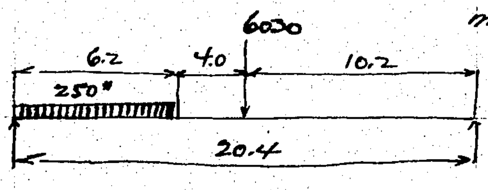


Load on stringer Rear 8580 · $\frac{3.53}{5.00}$ = 6030#

Front wheel 2010#

Uniform live load 100 · 2.5 = 250# per lin ft at front or rear of motor truck Tot. load = 250 · 6.2 = 1550#

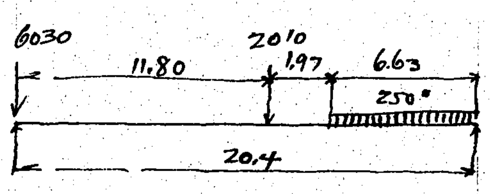
Reaction = 1550 · $\frac{3.1}{20.4}$ = 235#



Moment due to motor truck $\frac{6030}{2}$ · 10.2 = 30800

" " " Unif. load 235 · 10.2 = 2410

33210'#



Max End shear

Rear wheel			6030
Front wheel	2010 · $\frac{8.6}{20.4}$	=	846
Uniform load	250 · 6.63 · $\frac{3.32}{20.4}$	=	270
			7146#

Summary for moments and shears

	Moment	shear
Dead Load	21600	4260
Live Load	33210	7146
	54810'#	11406#

section modulus req'd = $\frac{54810 \cdot 12}{17000}$ = 38.7

Use 15", 33.9# I Sm = 41.7

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CALCULATIONS FOR

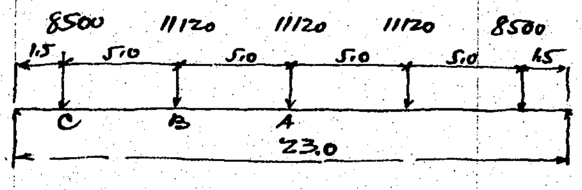
Final design of Furukawa-Bashi for Tokushima-ken.

Intermediate Floor Beam   span length 23'-0"   spacing 20'-4¾ (20.4).

Dead Load   Concentration at A   545 × 20.4 = 11.120#

at B   11.120#

at C   417 × 20.4 = 8500#



moment at A.

25180 × 11.50 = 290.000

11120 × 5.0 = 55600

8500 × 10.0 = 85000

149400

Reaction = 25180#

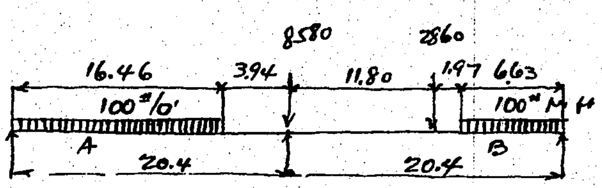
beam 120×11.5 = 1380

26560#

Floor Beam   m = ⅛ × 120 × 23.0² = 7940

157340#

Live Load

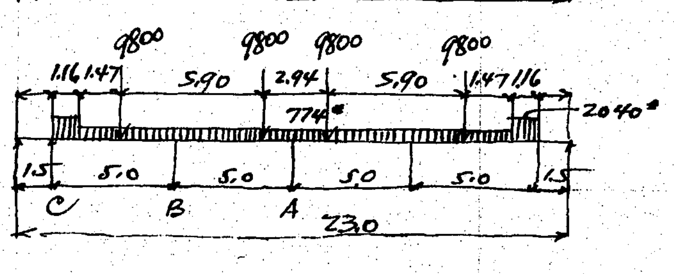


Uniform load at rear and front of motor truck

Unif. load A. $\frac{100 \times 16.46^2}{2 \times 20.4}$ = 666

Unif. load B. $\frac{100 \times 6.66^2}{2 \times 20.4}$ = 108

774#



For full unif. load   100 × 20.4 = 2040#

Reaction due to motor truck   2860 × $\frac{8.6}{20.4}$ = 1220

Rear wheel   8580

9800

Load on A.   Unif. load   774 × 5.0 = 3870

wheel conc. $\frac{2 \times 9800 \times 3.53}{5.0}$ = 13800

17670#

Reaction   8840
12090
7870
28800

Load on B.   Unif. load   774 × 2.5 = 1940

$\frac{774 \times 3.84 \times 3.08}{5.0}$ = 1830

$\frac{2040 \times 1.16^2}{2 \times 5.0}$ = 280

4050

Conc. $\frac{9800 \times 1.47}{5.0}$ = 2880

" $\frac{9800 \times 2.63}{5.0}$ = 5160

8040

12090#

moment at A

28800 × 11.5 = 331.200

12090 × 5.0 = 60500

7870 × 10.0 = 78700

− 139200

192.000#

Load on C   Unif. load $\frac{774 \times 3.84^2}{10}$ = 1140

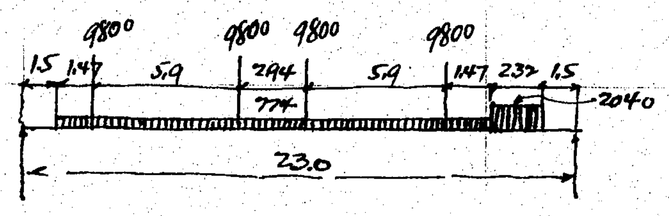
$\frac{2040 \times 1.16 \times 4.42}{5.0}$ = 2090

3230

Concentration $\frac{9800 \times 2.37}{5.0}$ = 4640

7870#

Max End shear.



Unif. load   774 × 17.68 × 12.66 ÷ 23 = 7500

"   "   2040 × 2.32 × 2.26 ÷ 23 = 470

7970#

motor trucks   9800 × $\frac{(20.03 + 14.13 + 11.19 + 5.29)}{23}$ = 21600

29570#

Summary for moments and shears

	moment	shear
Dead Load	157340	26560
Live Load	192000	29570
	349340#	56130#

Section of floor beam

web assumed 30 × 3/8 = 11.25○"   ⅛ web = 1.41○"

Back to back of flange Ls = 2'-6½"

Effective depth 2.54 − 0.15 = 2.39'

flange area required $\frac{349340}{17000 \times 2.39}$ − 1.41 = 7.18○" net

Use 2Ls 5 × 3½ × ½ = 8.00○" gross or 7.19○" net   OK

Final Design of Furukawa-Bashi for Tokushima-Ken.

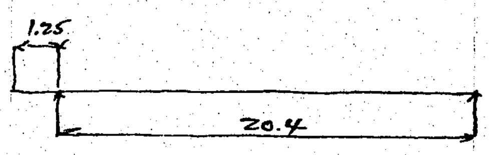
weight of intermediate floor beam

flanges	4Ls 5·3½·½	@ 13.6#	· 21'·9¾	=	1186.5
web.	1Pl 30·3/8	@ 38.25	· 21'·9¾"	=	832.7
End stiffs	4Ls 3½·3½·3/8	@ 8.5	· 3'·0½"	=	103.3
Int. stiffs	10Ls 4·3·5/16	@ 7.2	· 2'·5½	=	177.1
Int stiffs	8Ls 4·3·5/16	@ 7.2	· 2'·6½	=	146.3
fills.	4Pls. 3¾·½	@ 5.53	· 1'·11¼"	=	42.9
fills	10Pls. 3·½	@ 5.10	· 1'·11¼"	=	98.9
	10Ls 4·3·5/16	@ 7.2	· 1'·1¼"	=	79.2
	10Pls. 3·3/8	@ 3.83	· 0'·10¼"	=	32.6

2699.5
details say 80.5
2780.0#

$2780 \div 21.8 = 128$# per lin. ft.

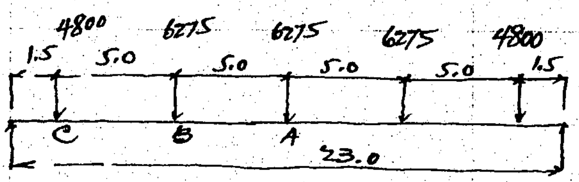
End Floor Beam
Dead Load



Floor load $97 \times 5 = 485$
stringer 60
545#

Concentration at A+B $545 \cdot \frac{21.65 \cdot 10.83}{20.4} = 6275$#

at C $417 \cdot \frac{21.65 \cdot 10.83}{20.4} = 4800$#



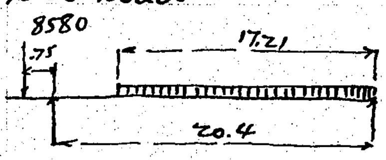
R = 4800
6275
3138
14123#

DL. moment $14213 \cdot 11.5 = 163500$
$6275 \cdot 5.0 = 31400$
$4800 \cdot 10.0 = 48000$
−79400
84100#

max End shear = 14123
Beam 1150
15363# call this 15500#

DL. Beam $\frac{1}{8} \cdot 100 \cdot 23^2 =$ 6600
90700#

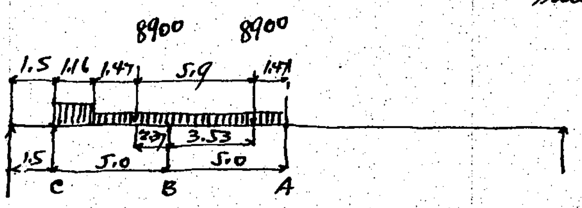
Live Load.



motor truck rear wheel 8580
Reaction $= 8580 \cdot \frac{21.15}{20.4} = 8900$#

Uniform load $17.21 \cdot 100 = 1721$# $R = \frac{1721 \cdot 8.60}{20.4} = 726$#

Full load. $21.65 \cdot \frac{10.83}{20.4} = 1255$#



A. motor truck $8900 \cdot \frac{3.53}{5.0} =$ 6280
6280
Unif. load 3630
16190#

B. motor truck $8900 - 6280 =$ 2620
" " $8900 \cdot \frac{2.63}{5.0} =$ 4680
Unif. load. 3630
Unif. load $529 \cdot \frac{0.58}{5.0} =$ 10930
60
10990#

1255
726
529# ½ A. 8095
B 10990
C 6504
25589#

C motor truck $8900 - 4680 =$ 4220
1815
$529 - 60 =$ 469
6504#

moment at A
$25589 \cdot 11.5$ 294000
$10990 \cdot 5.0 = 55000$
$6504 \cdot 10.0 = 65040$
120040
173960 call this 174000#

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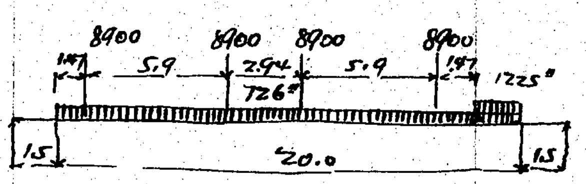
Final design of Furukawa-Bashi for Tokushima-Ken.



Max End shear

	$4 \cdot 8900 \cdot \frac{12.67}{23}$ =	19600
unif. load	$726 \cdot 17.68 \cdot \frac{12.67}{23}$ =	7040
" "	$1225 \cdot 2.32 \cdot \frac{2.66}{23}$ =	330
		26970 say 27000#

Summary for moments and shears

	moment	shear
Dead Load	90700	15500
Live Load	174000	27000
	264700'#	42500#

Section of End Floor Beam

web 30 · 3/8 = 11.25 1/8 web = 1.41

back to back of Ls 26 1/2" Effective d = 2.39'

flange stress 264700 ÷ 2.39 = 111000#

Section required 111000 ÷ 17000 = 6.53
 1.41
 5.12 □" net

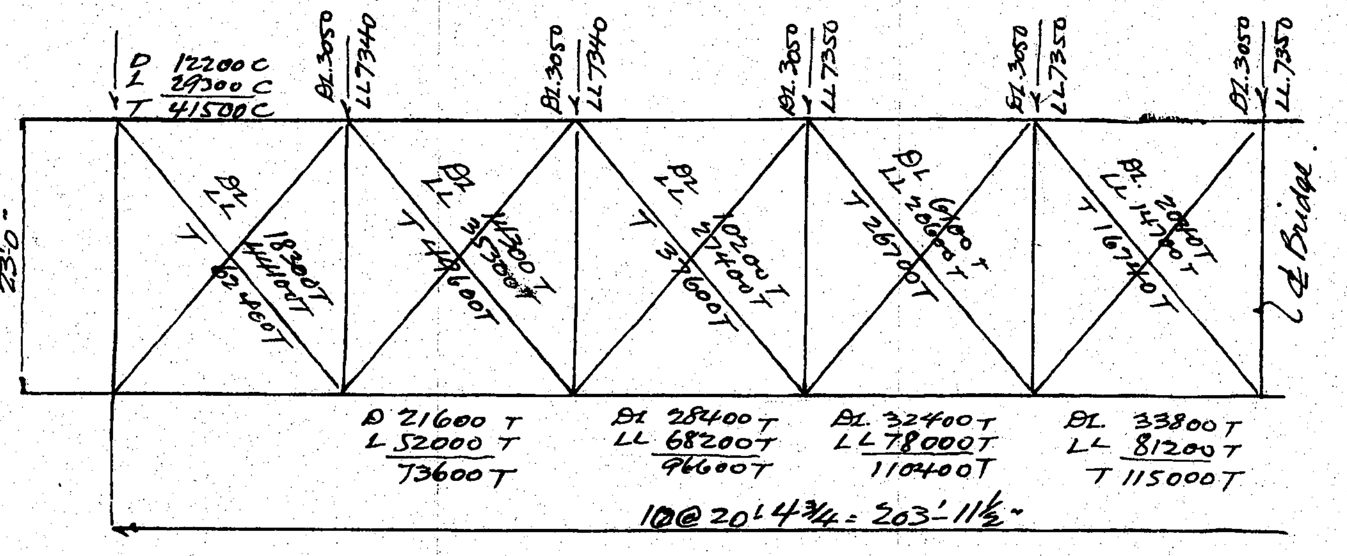
Use 2Ls 5·3 1/2·3/8 = 6.10 gr - 5.440 □" net.

weight of End Floor Beam

flange	4Ls	5·3 1/2·3/8	@ 10.4	· 21' 8 1/2	=	903.3
web	1Pl.	30·3/8	@ 38.25	· 21' 8"	=	828.9
End stiffs	4Ls	3 1/2·3 1/2·3/8	@ 8.5	· 2' 5 3/4	=	84.3
Int. stiffs	10Ls	4·3·5/16	@ 7.2	· 2' 5 3/4	=	178.6
" "	8Ls	4·3·5/16	@ 7.2	· 2' 6 1/2	=	146.3
fills	4Pls.	3 1/4·3/8	@ 4.14	· 1' 11 1/4	=	32.1
"	10Pls.	3·3/8	@ 3.83	· 1' 11 1/4	=	74.3
	5Ls	4·3·3/8	@ 8.5	· 1' 1 1/4	=	46.8
	5Pls.	3·3/8	@ 3.83	· 0' 10 1/4	=	16.3
						2310.9
		Rivet &c				70.1
						2381.0#

2381.0 ÷ 21.7 = 110# per lin ft.

Lower Lateral Bracing.



Static wind pressure 30#/□' of exposed surface

Assume 5×30 = 150# per lin ft. panel Conc = 150 · 20.4 = 3050#

moving wind load say 12 · 30 = 360# per lin. ft panel Conc 360 · 20.4 = 7340#

tan θ = 20.375 ÷ 23.0	=	0.885	W_1 tan θ	3050 · .885	=	2700#
sec θ = 30.75 ÷ 23.0	=	1.336	W_2 tan θ	7340 · .885	=	6500
W_1 = 3050			W_1 sec θ	3050 · 1.335	=	4070
W_2 = 7340			W_2 sec θ	7340 · 1.335	=	9800

Section of bottom laterals.

		SR
1st panel	62400# T	3.66 □"
2nd "	49600 T	2.92
3rd "	37600 T	2.21
4th "	26700 T	1.57
5th "	16740 T	0.98

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CALCULATIONS FOR

Final Design of Furukawa-Bashi for Tokushima-Ken

Bottom Lateral Bracings
Seismic stresses

Panel load	2 lines of handrails assumed	60	
	Stringers say	260	
	Lower Lateral Bracings	60	
	Floor beam	140	
	Trusses lower half	500	
			1020#
	Flooring & copings		2200
			3220#

Panel Conc = 3220 × 20.4 = 65600# Hor. Seismic force = 65600 × 0.3 = 19500#
working strength = 17000 × 1.8 = 30600#/□"
Shears and stresses.

	shear	stress	net section	Section	area	No. of Rivets
1	87800	117800#T	3.85	2Ls 5×3×3/8	5.72 □"	10
2	68200	91200	2.98	2Ls 5×3×3/8	5.72 "	8
3	48800	65200	2.13	2Ls 4×3×5/16	4.18 "	6
4	29200	39000	1.28	2Ls 4×3×5/16	4.18 "	6

Approximate weight of bottom lateral bracings.

4Ls 5×3×3/8 @ 9.80 × 28.25 = 1110
Center connection say 70
misc details say 120
1300 × 4 = 5200#
4Ls 4×3×5/16 @ 7.2 × 28.5 = 820
Center connection say 40
misc details say 100
960 × 6 = 5760
10960# per span

10960 ÷ 204 = 54# per lin ft of span.

Upper Lateral Bracings.
Static wind pressure 30#/□' panel load say 3050#
Diagonal stress. sec θ = 1.36 shear for end panel 3050 × 3.5 = 10700# S = 10700 × 1.36 = 14600#
Section required 14600 ÷ 17000 = 0.86 □"
Seismic stress :-
load on top chord upper half of truss = 500
top lateral & sways say 200
700# × 20.4 = 14300#
Horizontal seismic force = 14300 × 0.3 = 4300#
Shear = 4300 × 3.5 = 15000# S = 15000 × 1.36 = 20400# T
section required 20400 ÷ 30600 = 0.67 □" net.
Try 2Ls 6×4×3/8 ⅃L riveted back to back of Ls r = 1.50 r = 1.93
Unsupported length 31.2 × 12 = 374" l/r = 374 ÷ 1.93 = 194.0
" " 187 l/r = 187 ÷ 1.50 = 125.0
approximate weight of diagonals.

2Ls 6×4×3/8 @ 12.3 × 27'10 = 698.0
4Ls 6×4×3/8 @ 12.3 × 13'5 3/4 = 663.0
1 Pl. connection at center 50.0
details say 10.0
1421.0 say 1420#

8 panels @ 1420 = 11460# per span

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CALCULATIONS FOR

Final Design of Furukawa-Bashi for Tokushima-Ken

Sway bracings at U_2 U_3 U_4 and U_5

Top strut	4Ls	4.3.5/16	@	7.2	21'-0¼"	=	605.4
bott. strut	2Ls	6.4.3/8	@	12.3	21'-3¼"	=	523.2
diag.	4Ls	3.3.5/16	@	3.61	7'-0"	=	170.8
"	2.4Ls	3.3.5/16	@	3.61	7'-2"	=	349.9
misc details say							800.0
							2349.0 say 2350#

7@2350 = 16450#

Portal Bracings at U_1

Top	2Ls	4.3.5/16	@	7.2	20'-10"	=	300.0
Top	1L	4.3.5/16	@	7.2	21'-0"	=	151.0
bott.	4Ls	4.3.5/16	@	7.2	22'-0"	=	634.0
	1Pl.	15.5/16	@	15.94	20'-10"	=	332.0
	1Pl.	15.5/16	@	15.94	21'-9"	=	346.0
diag.	4Ls	4.3.5/16	@	7.2	4'-4¼"	=	125.0
"	2Pls.	15.5/16	@	15.94	4'-1¾"	=	132.0
	4Ls	4.3.5/16	@	7.2	4'-0"	=	115.0
	2Pls.	15.5/16	@	15.94	3'-9½"	=	121.0
	4Ls	4.3.5/16	@	7.2	3'-2½"	=	92.0
	2Pls.	15.5/16	@	15.94	3'-0"	=	95.0
	4Ls	4.3.5/16	@	7.2	3'-1"	=	89.0
	2Pls.	15.5/16	@	15.94	2'-10½"	=	92.0
misc details say							500.0
							3124.0#

2@3124 = 6248 say 6250#

Summary for top lateral Bracings.

diagonals.	11460
Sways	16450
Portal Bracings.	6250

34160# ÷ 204 = 167# per lin ft of span

Summary for stringers.

2 stringers	@ 15" [@	33.9	=	67.8
2 "	@ 15" I	42.9		85.8
1	@ 18 I	54.7		54.7
				208.3 call this 210# per lin ft.

Connections included into floor beams.

Intermediate Floor beam 2780 ÷ 20.4 = 137# per lin ft.

Design of 203'-11½" truss

2 lines of handrails	60
Stringers	210
Int. Floor Beam	137
Lower Laterals	54
upper Laterals	167
Truss assumed	1000
	1628#

Pavement	22" × 20 =	440
Slab	75 × 20 =	1500
copings	2 @ 120 =	240
misc conc. say		20
		2200
		3828 ÷ 2 = 1914# per lin ft.

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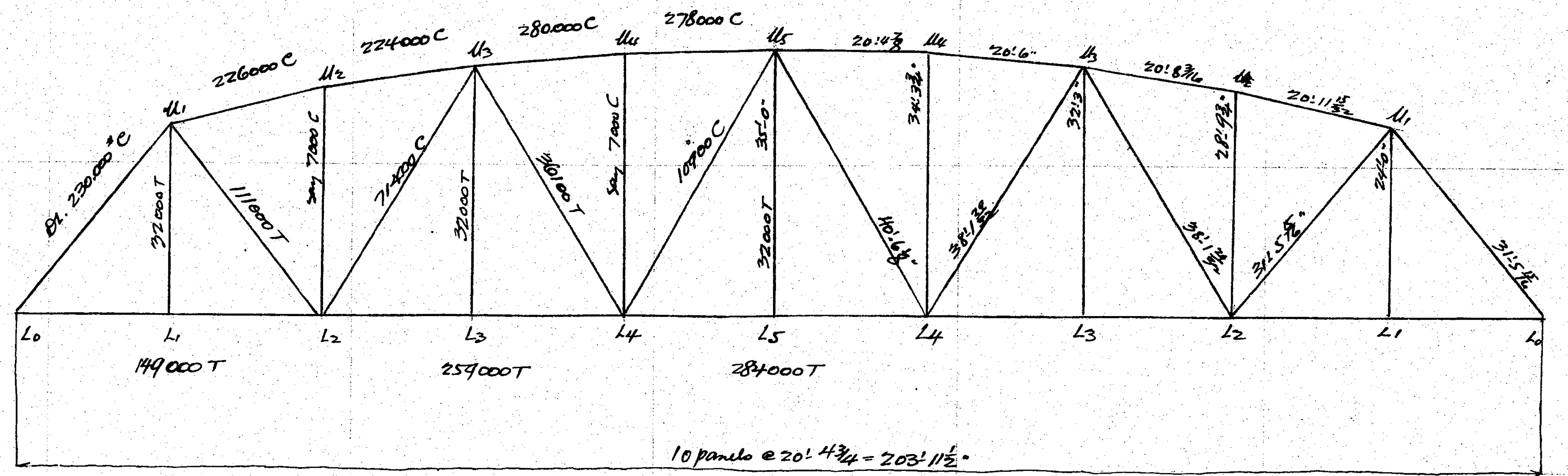
CALCULATIONS FOR

Final Design of Furukawa-Bashi for ~~Tokushima~~-Ken



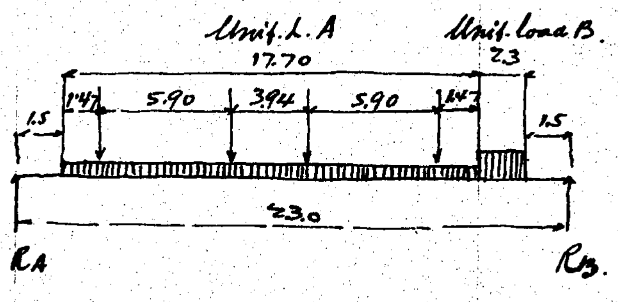
Points of intersection of both chords				Length of diagonal & Top chords			Lever Arm for Top chords				Arm for diagonals			
U_5	20.396 × $\frac{3.5}{.6875}$	=	1038.3	U_5-L_4	40'6 1/8	= 40.51	U_5-U_4	1017.9 × $\frac{34.31}{1018.5}$	=	34.30	U_5-L_4	1017.9 × $\frac{35.0}{40.51}$	=	879.4
U_4	20.396 × $\frac{34.31}{2.06}$	=	339.7	L_2-U_3-L_4	38'1 30/32	= 38.16	U_4-U_3	343.1 × $\frac{339.7}{341.4}$	=	34.05	U_3-L_4	339.7 × $\frac{32.25}{38.16}$	=	287.0
U_3	20.396 × $\frac{32.25}{3.44}$	=	191.2	L_0-U_1-L_2	31'5 15/16	= 31.49	U_3-U_2	28.81 × $\frac{170.8}{173.2}$	=	28.40	U_3-L_2	170.8 × $\frac{32.25}{38.16}$	=	144.4
U_2	20.396 × $\frac{28.81}{4.81}$	=	122.1	U_5-U_4	20'4 7/8		U_2-U_1	28.81 × $\frac{122.1}{125.4}$	=	28.10	U_1-L_2	122.1 × $\frac{24}{31.49}$	=	93.1
				U_4-U_3	20'6"									
				U_3-U_2	20'8 3/16									
				U_2-U_1	20'11 15/32"									

CALCULATIONS FOR

Final design of Furukawa-Bashi for Tokushima-Ken.

Panel concentration for dead load 1914 × 20.4 = 39000#
At top chord panel point
upper lateral system 167
upper half of trusses 500
667 ÷ 2 = 333.5# Conc. = 333.5 × 20.4 = 6800#
For lower panel point 39000 − 6800 = 32200#

Live load stresses
Uniform live load assumed 100#/□' Impact for motor truck loading assumed 25%.
In figuring the live load stresses in truss, the following are assumed
2 motor trucks side by side with occupied space of 2@8.85 = 17.7'
For 2.3' strip 100#/□' live load.



motor truck loading

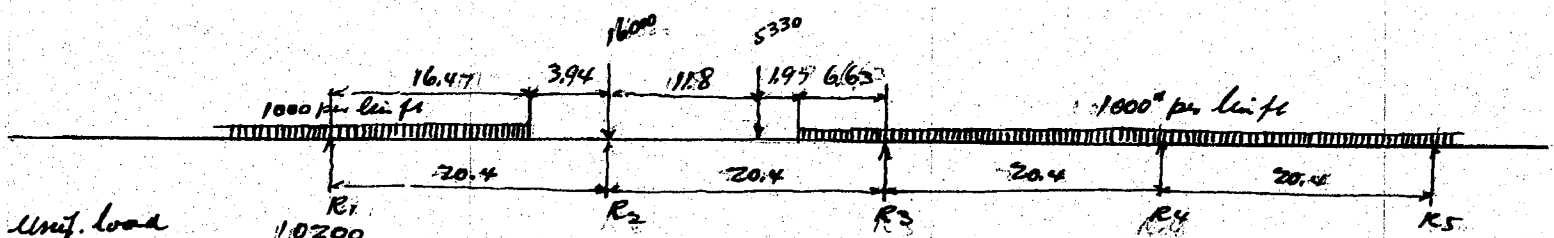
	Rear wheel	Front wheel
	6600	2200
25% impact	1650	550
	7250#	2750#

RA = 4 × 7250 × 12.65/23.0 = say 16000# for rear wheels
5330 " front wheels.

Uniform load A 1770 × 12.65/23.0 = 973#

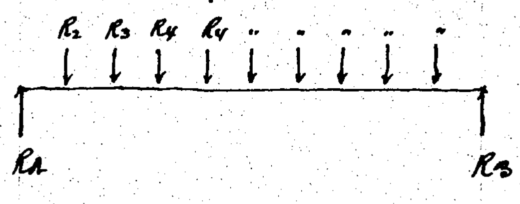
Uniform load B 230 × 2.65/23.0 = 27
1000# per lin ft.

neglect uniform load on side of motor truck and assume the 1000# uniform load as front and rear of motor trucks.
Panel concentration due to uniform and motor truck loadings.



	R1	R2	R3	R4	R5
Unif. load	10200				
Unif. load	9830	6640			
motor truck		16000			
motor truck		2240	3090		
Unif. load		1080	5550		
Unif. load			10200	20400	20400
	20030	25960	18840	20400	20400

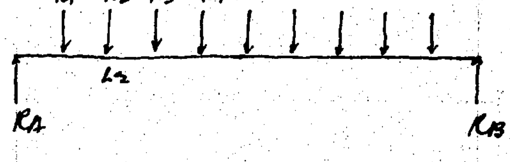
Lower chord L_0-L_2 and End post L_0-U_1
R_2 loading as L_1



RA
20400 × 2.8 = 57100
18840 × 0.8 = 15050
25960 × 0.9 = 23400
L_0-L_2 95550 × 20.4/24 = 81200#

L_0-U_1 95550 × 31.49/24 = 125500#

Top chord U_1-U_2-U_3.



Reaction
20400 × 2.1 = 42900
18840 × 0.7 = 13200
25960 × 0.8 = 20800
20030 × 0.9 = 18000
94900

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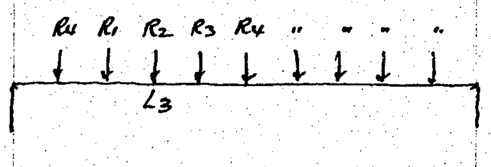
94900 · 2·20.4 = 3870000
20030 · 20.4 = 408.000
3462,000

U1-U2 3462.000 ÷ 28.10 = 123000 # C
U2-U3 3462.000 ÷ 28.40 = 122000 # C

Bottom chord L2-L3-L4

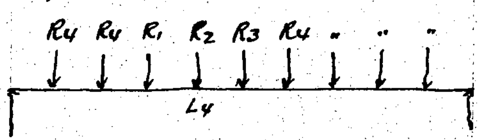


Reaction

R4	20400	· 2.40	=	49000
R3	18840	· 0.6	=	11300
R2	25960	· 0.7	=	18200
R1	20030	· 0.8	=	16000
				94500 #

moment at U3. 94500 · 3·20.4 = 5780.000
20030 · 20.4 = 408000
20400 · 2·20.4 = 833000
−1241.000
4539 000 ÷ 32.25 = 140600 # T

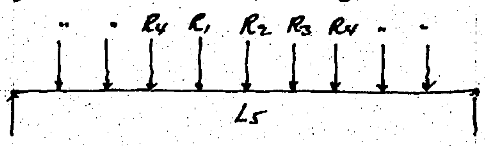
Top chord U3U4 - U4-U5 Reaction



R4	20400	· ~~4.1~~ 2.7	=	55100
R3	18840	· 0.5	=	9400
R2	25960	· 0.6	=	15600
R1	20030	· 0.7	=	14000
				94100 #

moment at L4
94100 · 4·20.4 = 7680.000
20030 · 20.4 = 408000
20400 · 5·20.4 = 2080000
2488.000
U3-U4 5192.000 ÷ 34.05 = 153000 # C
U4-U5 5192.000 ÷ 34.30 = 151500 # C

Bottom chord L4-L5

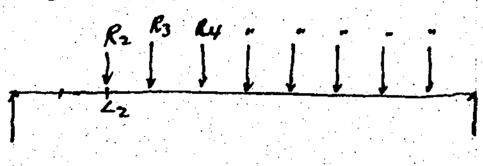


Reaction

R4	20400	· 3.0	=	61200
R3	18840	· 0.4	=	7550
R2	25960	· 0.5	=	13000
R1	20030	· 0.6	=	12000
				93750

moment at U5
93750 · 5·20.4 = 9550.000
20030 · 20.4 = 408000
20400 · 9·20.4 = 3745000
−4153.000
5,397,000 ÷ 35.0 = 154000 # T

Diagonals U1-L2
R2 at L2 Panel Concentration assumed for safe side

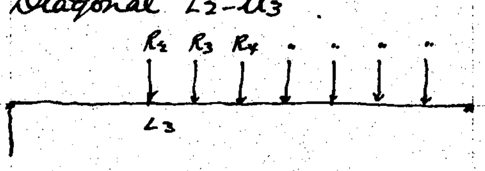


Reaction

R4	20400	· ~~3.0~~ 2.1	=	42900
R3	18840	· 0.7	=	13200
R2	25960	· 0.8	=	20800
				76900 #

stress = 76900 · $\frac{81.3}{93.1}$ = 67000 # T

Diagonal L2-U3



Reaction

R4	20400	· ~~2.4~~ 1.5	=	~~49000~~ 30600
R3	18840	· 0.6	=	11300
R2	25960	· 0.7	=	18200
				~~78500~~ # 60100 #

S = 60100 · $\frac{130.0}{144.4}$ = 54200 # C

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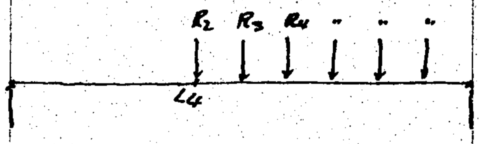
diagonal U3-L4

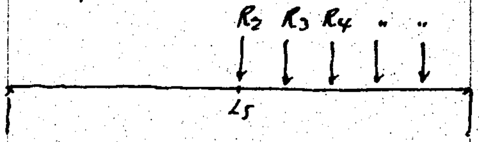


Reaction

R4	20400 · 1.0	=	20400
R3	18840 · 0.5	=	9420
R2	25960 · 0.6	=	15600
			45420#

Stress = 45420 · $\frac{258.1}{287.0}$ = 41000# T

diagonal L4-U5



Reaction

R4	20400 · 0.6	=	12200
R3	18840 · 0.4	=	7930
R2	25960 · 0.5	=	13000
			33130#

Stress = 33130 · $\frac{936.3}{879.4}$ = 35200# C

diagonal U5'-L4'
R2 at L4'

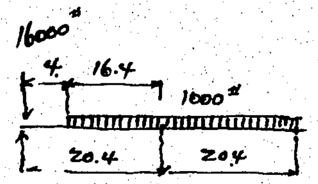
Reaction

R4	20400 · 0.3	=	6120
R3	18840 · 0.3	=	5650
R2	25960 · 0.4	=	10400
			22170#

Stress = 22170 · $\frac{936.3}{879.4}$ = 26600# T.

Live Load stress in Hangers U1-L1, U3-L3 U5-L5 29570# T see p5

Max Live Load reaction.
motor truck rear wheel at L0.



Reaction due to unif. load 200000 · $\frac{100}{204}$ = 98000#
motor truck rear wheel = 16000
114000#

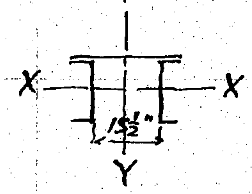
Dead Load on shoe. 3' 1914# · 103.25 = 198000#

max load on shoe

Dead load	198.000
Live Load	114000
	312000
weight of shoe say	3000
	315000 on bearing

Section of truss members.
Top chord section
U3-U4-U5

2[15" · 4" @41.94# = 24.66
1 Cov. 23 · ½ = 11.50
36.16□"



moment of inertia XX.

2[15"·4"	= 24.66	·	7.5	=	192.5	24.66 · 2.7²	=	180.0 + 754	= 934
1 Cov. Pl. 23·½	= 11.50	·	15.25	=	175.0	11.50 · 5.05²	=	293.0	293
	36.16		10.2		367.5				1227 in⁴

moment of inertia

Radius of gyration = $\sqrt{\frac{1227}{36.16}}$ = 5.82

moment of inertia YY

24.66 · 8.68²	= 1855	+ 29	=	1884
11.50				1081
				2965

radius of gyration = $\sqrt{\frac{2965}{36.16}}$ = 9.05

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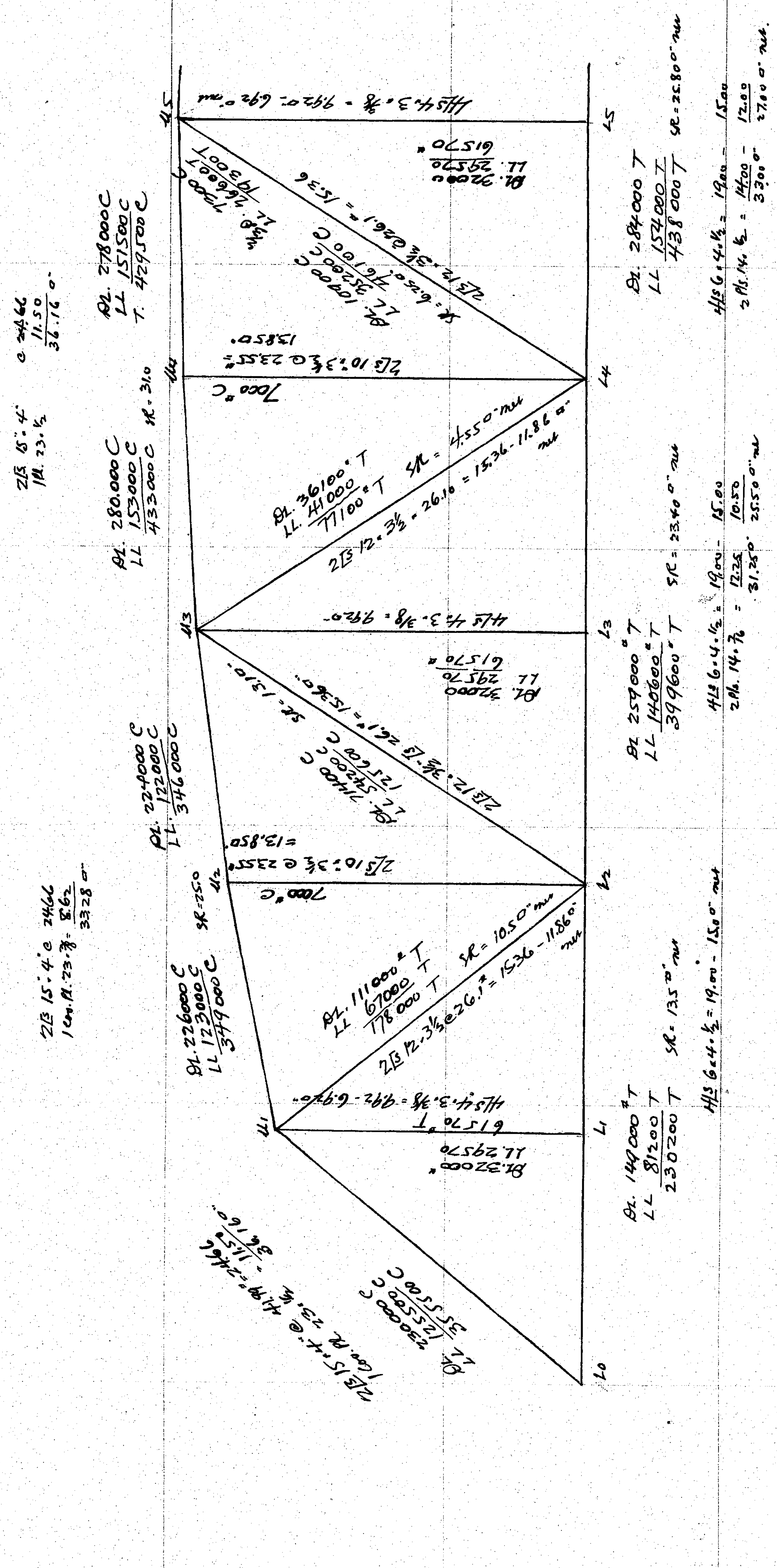
CALCULATIONS FOR

Final design of Furukawa-Bashi for Tokushimaken.



Stresses and sections of truss.

CALCULATIONS FOR

Preliminary design of Furukawa-Bashi for Tokushimaken

max stress 433000# C | Unsupported length 246" | allowable unit stress = 14000#/□"

section required 433000 ÷ 14000 = 31.0□" gross.

Top Chord U_1-U_2-U_3. max stress = 349.000# Unsupported length = 252"

2[15" × 4" @ 41.94# = 24.66 Unit allowable stress = 14000#/□"

1 Cov. Pl. 23" × 3/8" = 8.62 S.R. = 349.000 ÷ 14000 = 25.0□"

33.28□"

End post L_0-U_1 max direct stress 355500# C

Unsupported length = 378"

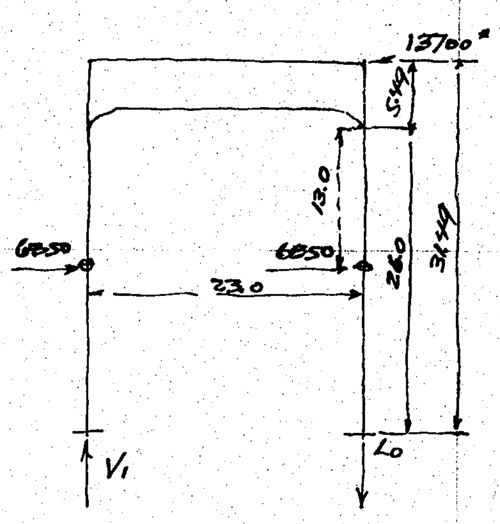
Allowable unit stress = 21300 (1 - 0.0055 l/r) = 13700#/□"

Assumed section
2[15" @ 41.94# = 24.66
1 Cov. 23" × 1/2 = 11.50
36.16□"

Section required for direct load
355500# C ÷ 13700 = 25.9□"

Radius of gyration = 5.82

For Bending due to wind load panel load 3050# × 4.5 = 13700#



Reaction V_1 = 13700 × $\frac{31.49}{23.0}$ = 18800
355500
374300#

Assume the point of Contra flexure = 13.0 above panel point L_0

moment = 6850 × 13.0 = 89000"#

moment of inertia YY axis 2965 (in)⁴

Bending stress = $\frac{89000 \times 12.5 \times 12}{2965}$ = 4150#/□"

Unit stress = 13700 × 1.25 = 17100#/□"
- 4150
12950#

Required section 374300 ÷ 12950 = 29.0□"

not increasing unit stress 13700 - 4150 = 9550#

Required section = 374300 ÷ 9550 = 39.2□"

For Emergencies' sake for such a long span some allowance is given for sections of compression members in top chords.

Bottom chord sections

	Stresses	S.R
L_0-L_2	230200# T	13.50□" net
L_2-L_4	399600 T	23.40□" net
L_4-L_6	438000 T	25.80□" net

L_0-L_2 4 ∟ 6 × 4 × 1/2 = 19.0 gr 15.0□" net.

L_2-L_4 4 ∟ 6 × 4 × 1/2 = 19.0 15.0
2 Pls. 14 × 7/16 = 6.12
6.12 10.5
31.24□" 25.5□" net

L_4-L_4' 4 ∟ 6 × 4 × 1/2 = 19.0 15.0
2 Pls. 14 × 1/2 = 14.0 12.0
33.0□" 27.0□" net

Diagonals. U_1-L_2 stress = 178000# T S.R. = 178000 ÷ 17000 = 10.50□" net
2[12" × 3½" @ 26.1# = 15.36□" - 3.5 = 11.86□" net.

Diagonals. L_2-U_3 stress = 125600# C
Try 2[12" × 3½" @ 26.1# = 15.36□" r = 4.55 Unsupported length = 457"
l/r = 100 allowable stress = 21300 (1 - 0.0055 × 100) = 9600#/□"
section required 125600 ÷ 9600 = 13.10□"

Final design of Furukawa-Bashi for Tokushima-Ken.

Diagonal U3-L4 stress = 77100# T SR = 4.55 □" net.
Use 2[s 12 × 3½ @ 26.1# = 15.36 - 3.5 = 11.86 □" net.

diagonal U5-L4 stress = 46100# C 46100
Reversal - 19300 T counting ½. 8650
54750#

Try 2[s 12 × 3½ @ 26.1# = 15.36 □" r = 4.55 Unsupported length = 486" l/r = 107.
allowable stress = 21300 (1 - 0.0055 × 107) = 8750 #/□"
section required = 54750 ÷ 8750 = 6.25 □"

Verticals U2-L2 and U4-L4
Use 2[s 10 × 3½ @ 23.55# = 13.85 □" r = 3.85
Unsupported length 387" l/r = 100.0 allowable stress = 9600 #/□"

Hangers U1-L1, U3-L3 and U5-L5 stress = 61570# T SR = 3.640"
4Ls 4 × 3 × 3/8 = 9.92 □" - 3.0 = 6.92 □" net.

Approximate weight of truss.

member	section				
L0-U1	36.16	@ 3.4	× 31.5	=	3870
U1-U2	33.28	"	× 21.00	=	2380
U2-U3	33.28	"	× 20.70	=	2340
U3-U4	36.16	"	× 20.50	=	2520
U4-U5	36.16	"	× 20.40	=	2500
L0-L2	19.00	"	× 40.80	=	2640
L2-L4	31.25	"	× 40.80	=	4340
L4-L5	33.00	"	× 20.40	=	2290
U1-L2	52.2#		× 31.50	=	1645
L2-U3	52.2#		× 38.20	=	1995
U3-L4	52.2#		× 38.20	=	1995
L4-U5	52.2#		× 40.50	=	2120
U1-L1	9.92 □"	@ 3.4	× 24.0	=	810
U3-L3	9.92	"	× 32.2	=	1080
U5-L5	½ - 9.92	"	× 35.0	=	590
U2-L2	47.1#		× 28.8	=	1355
U4-L4	47.1#		× 34.3	=	1615
					36085
	Details &c		35%		12600
					48685#

For one truss = 2 × 48685 = 97370 call this 97500#
For one span = 2 × 97500 = 195000#.
195000 ÷ 204 = 956# per lin ft of span

Approximate metal in one span

stringers	210 × 206.5	=	43400
Int. F.B.	9 @ 2780	=	25000
End Floor Beam	2 @ 2380	=	4760
Bottom laterals.			10960
Top lateral Bracings			34160
Trusses			195000
			313280#
Shoes assumed			8000
			321280# or 143.5 tons.

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CALCULATIONS FOR

Design of Furukawa-Bashi for Tokushima-Ken

Load on shoe. 312000#

Assume 6" ϕ pin $\frac{312.000}{6 \cdot 24000} = 2.17"$ or 1.09" per rib.

Used 1.5" ribs. Bending moment = 156.000 · 1.75 = 274500"#

6" pin good for 508900"# ok.

Load on Bearing 315000# Try 4" ϕ rollers. Unit stress = 610 · 4.0 = 2440# per lin inch

Length of rollers req'd = 315.000 ÷ 2440 = 129.0"

For 5 rollers $\frac{129}{5} = 25.8"$ Used 2'-2½"

Unit bearing on masonry $\frac{315000}{26 \cdot 38} =$ 320#/□" for roller shoes

$\frac{315000}{26 \cdot 29.75} =$ 408#/□" for fixed shoes.

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CALCULATIONS FOR

Design of Furukawa Bashi : List of materials

End Post $EP1^{R}_{L}$ 2 req'd each

2	Is	15"×4"	31'-4¼"	41.94#	1314.99	2629.98	
1	Pl	23"×½"	30'-5¾"	39.10	1191.74	1191.74	Cov. Pl
2	Ls	4½"×3½"×½"	2'-1¾"	15.30	82.83	165.66	
2	Pls	4¼"×½"	0'-7½"	7.23	4.52	9.04	Fillers
1	L	4×4"×⅜	1'-2½	9.80	11.86	11.86	
2	Pls	42¼"×½"	3'-10¼"	71.83	276.85	553.70	Gusset Pls
2	Pls	19½×⅜"	1'-11¾"	24.86	49.20	98.40	
2	Pls	15"×⅜"	1'-6¾"	19.13	29.88	59.78	
2	Ls	4½"×3½"×½"	2'-3⅝"	15.30	35.22	70.38	
2	Pls	4¼"×⅜"	1'-3"	5.42	6.78	13.50	Fillers
24	Bars	3"×⅜"	2'-6¾"	3.83	9.80	235.20	
1	Pl	19¼"×⅜"	2'-2"	24.55	53.27	53.27	
1	"	19½"×5/16"	3'-3"	20.72	67.34	67.34	
1	"	39"×5/16"	3'-3"	41.44	134.68	134.68	
1	"	14"×5/16"	3'-6"	14.88	52.08	52.08	
1	"	14"×5/16"	1'-8¼"	14.88	25.15	25.15	
1	"	15¼"×½"	1'-7¼	25.93	41.49	41.49	
1	"	3¾"×½"	1'-8"	6.38	10.65	10.65	
						5423.98 × 4 =	21696

Top Chord $U_1U_2{}^{R}_{L}$ 2 req'd each

2	Is	15"×4"	21'-2"	41.94#	887.87	1775.74	
1	Pl	23"×⅜"	21'-2⅛"	29.33	621.21	621.21	
2	Pls	22½"×½"	2'-0"	38.25	76.50	153.00	
16	Bars	3"×⅜"	2'-6¾"	3.83	9.81	156.96	
2	Pls	4¼"×⅜"	1'-8"	5.42	9.51	19.08	
2	"	4¼"×⅜"	0'-7½"	5.42	3.39	6.78	
2	Ls	4½"×3½×½"	2'-9"	15.30	42.08	84.16	
1	Pl	6½"×⅛"	1'-7½"	2.76	4.50	4.50	
2	Pls	29½"×½"	2'-11½"	50.14	148.56	297.12	
2	"	13¾×⅜"	1'-5⅜	17.53	25.42	50.84	
2	"	14½"×⅜"	1'-5"	18.49	26.26	52.52	
1	Pl	14½"×⅜"	3'-5⅝"	18.49	64.16	64.16	
1	"	19¼×⅜"	2'-2⅝"	24.55	54.50	54.50	
2	Pls	3¾"×½"	1'-7½"	6.38	10.40	20.80	
						3161.37 × 4 =	12645

Top Chord $U_2U_3{}^{R}_{L}$ 2 req'd each

2	Is	15"×4"	20'-9⅝"	41.94	870.67	1741.30	
1	Pl	23"×⅜"	20'-9"	29.33	608.60	608.60	
2	Pls	21"×½"	1'-10½"	35.70	67.12	134.20	
16	Bars	3"×⅜"	2'-6¾"	3.83	9.80	156.80	
2	Pls	32¼"×½"	3'-4¾"	54.83	186.42	372.80	
2	"	12¾"×⅜"	1'-4⅝"	16.26	22.60	45.20	
1	Pl	3⅝"×⅛"	1'-10¼"	1.54	2.85	2.90	
1	"	16½"×⅛"	1'-7¼"	7.01	11.22	11.20	
1	"	16½"×⅛"	1'-7¼"	7.01	11.22	11.20	
1	"	19¼"×½"	2'-8"	32.73	87.39	87.40	
1	"	14½"×⅜"	3'-8"	18.49	67.86	67.90	
2	Pls	3¾"×⅝"	2'-7½"	7.97	20.92	41.80	
						3281.30 × 4 =	13125

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CALCULATIONS FOR

List of materials for Furukawa-Bashi

			Top Chod $U_3U_4{}^R_L$ 2 req'd each					
2	[s	15"×4"	20'-6⅝"	41.94	862.71	1725.40		
1	Pl	23"×½"	20'-7"	39.10	804.68	804.70		
2	Pls	21"×½"	1'-10¼"	35.70	66.05	132.10		
16	Bars	3"×⅜"	2'-6¾"	3.83	9.80	156.80		
2	Pls	29¼"×½"	2'-10¼"	49.73	141.73	283.50		
2	"	14½"×⅜"	1'-3½"	18.49	23.85	47.70		
2	"	12¾"×⅜"	1'-4¼"	16.26	21.95	43.90		
1	Pl	19¼"×½"	2'-7½"	32.73	86.08	86.10		
1	"	14½"×⅜"	3'-7½"	18.49	67.12	67.10		
2	Pls	3¾"×⅝"	2'-6½"	7.97	20.24	40.48		
						3387.78 ×4 =	13551	
			Top Chord U_4U_5 2 req'd					
2	[s	15"×4"	20'-5¾"	41.94	858.93	1717.90		
1	Pl	23"×½"	20'-5⅝"	39.10	801.16	801.20		
2	Pls	21"×½"	1'-10½"	35.70	67.12	134.20		
16	Bars	3"×⅜"	2'-6¾"	3.83	9.80	156.80		
2	Pls	32"×½"	2'-10¾"	54.40	157.76	315.50		
2	"	12¾"×⅜"	1'-3½"	16.26	20.98	42.00		
1	Pl	19½"×½"	2'-8½"	33.15	89.83	89.80		13551
1	"	14½"×⅜"	3'-7"	18.49	66.19	66.20		6727
2	Pls	3¾"×⅝"	2'-6"	7.97	19.93	39.90		5620
						3363.50 ×2 =	6727	25898
			Top Chord U_4U_5A 2 req'd					
2	[s	15"×4"	20'-5¾"	41.94	858.93	1717.90		
1	Pl	23"×½"	20'-5⅝"	39.10	801.16	801.20		
2	Pls	21"×½"	1'-10½"	35.70	67.12	134.20		
16	Bars	3"×⅜"	2'-6¾"	3.83	9.80	156.80		
						2810.10 ×2 =	5620	
					Summary for Top Chord		70364# (73760)	
			Bottom Chord $L_0L_2{}^R_L$ 2 req'd each					
4	Ls	6"×4"×½"	36'-1⅜"	16.20	585.06	2340.20		
1	Pl	13"×⅜"	2'-3"	16.58	37.31	37.30		
6	Pls	9"×⅜"	1'-1"	11.48	12.40	74.70		
1	Pl	13"×⅜"	1'-6½"	16.58	25.53	25.50		
2	Pls	6"×4"×⅜"	2'-1"	12.30	25.58	51.20		
1	Pl	12"×⅜"	1'-11"	15.30	29.38	29.40		
2	Ls	6"×4"×⅜"	3'-1½"	12.30	38.50	76.90		
1	L	6"×4"×⅜"	2'-1"	12.30	25.58	25.60		
2	Pls	43½"×½"	4'-0"	73.95	295.80	591.60		
2	"	18½"×½"	2'-1¾"	31.45	67.62	135.20		
2	"	18½"×½"	2'-6¼"	31.45	79.25	158.50		
2	"	8⅜"×½"	1'-3¼"	14.24	18.08	36.20		
1	Pl	19"×⅜"	2'-2"	24.23	52.58	52.60		
1	Pl	19"×⅜"	3'-0"	24.23	72.69	72.70	Diaphragm	
1	L	4"×3"×⅜"	0'-10½"	8.50	7.48	7.50	Diaphragm	
4	Ls	4"×4"×⅜"	2'-6¼"	9.80	24.70	98.80		
1	Pl	12"×⅜"	2'-6¼"	15.30	38.56	38.60		

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CALCULATIONS FOR

List of Materials for Furukawa-Bashi

1	Washer	3"φ × 1/2"		5.10	1.52	1.50
1	"	3"φ × 3/8"		3.83	.90	.90
1	Ls	4 1/2 × 3 1/2 × 3/8"	1'-2 1/2"	11.30	13.67	13.70
1	Pl	17" × 3/8"	3'-1 1/2"	21.68	67.84	67.84
1	Pl	26 3/4" × 3/8"	3'-2"	34.11	108.13	108.13
						~~404.50~~ 404.50 × 4 = 1617.8
						1617.8

Bottom Chord $L_1L_3{}^R_L$ 2req'd each

4	Ls	6" × 4" × 1/2"	20'-4 5/8"	16.20	330.32	1321.30
2	Pls	14" × 7/16"	18'-6 1/4"	20.83	385.77	771.50
1	Pl	12" × 3/8"	1'-8 1/2"	15.30	26.16	26.20
3	Pls	9" × 3/8"	1'-0"	11.48	11.48	34.40
4	Ls	6 × 3 1/2" × 1/2"	3'-0 1/8"	15.30	46.05	184.20
4	Pls	5 1/4 × 7/16"	1'-6"	1.12	1.68	6.7
2	Pls	14" × 7/16"	3'-6 1/8"	20.83	73.18	146.20
1	Pl	12" × 3/8"	3'-0 1/8"	15.30	46.05	46.10
2	Ls	6" × 4" × 3/8"	2'-8 1/2"	12.30	33.33	66.70
2	Pls	36 1/2" × 3/8"	3'-8 1/2"	46.54	172.66	345.30
1	Pls	24" × 3/8"	2'-10"	30.60	86.60	86.60
						3035.20 × 4 = 12141

Bottom Chord $L_3L_4{}^R_L$ 2req'd each

4	Ls	6" × 4 × 1/2"	28'-4 5/8"	16.20	459.92	1839.70
2	Pls	14" × 1/2"	28'-4 5/8"	23.80	675.68	1351.40
1	Pl	12" × 3/8"	1'-7 1/2"	15.30	24.94	24.90
1	"	12" × 3/8"	1'-8 1/2"	15.30	26.16	26.10
4	Pls	9" × 3/8"	1'-0"	11.48	11.48	45.90
2	Ls	6" × 4" × 3/8"	2'-7 1/2"	12.30	32.35	64.60
2	"	6" × 4" × 3/8"	2'-8 1/2"	12.30	33.33	66.70
1	Pl	19 1/2" × 3/8"	3'-0"	24.86	74.58	74.60
1	Pl	19 1/2" × 3/8"	2'-2"	24.86	53.97	54.00
2	Pls	36 1/2 × 3/8"	3'-5 1/2"	46.54	161.03	322.10
4	Ls	6" × 3 1/2 × 1/2	3'-0 1/8"	15.30	46.05	184.20
2	Pls	14" × 1/2"	3'-11 1/8"	23.80	93.53	187.10
1	Pl	12 × 3/8"	3'-0 1/8"	15.30	46.05	46.10
1	"	22 × 3/8"	2'-8 1/2"	28.05	74.89	74.90
1	"	22 × 3/8"	2'-7 1/2"	28.05	73.77	~~73.80~~ 73.77
						4436.10 × 4 = 17744

Bottom Chord L_5L_5 2 req'd

4	Ls	6" × 4" × 1/2"	32'-9 3/8"	16.20	531.04	2124.20
2	Pls	14" × 1/2"	32'-9 3/8"	23.80	780.16	1560.30
1	Pl	12" × 3/8"	1'-7 1/2"	15.30	24.94	24.90
6	Pls	9" × 3/8"	1'-0"	11.48	11.48	68.90
2	Ls	6" × 4" × 3/8"	2'-7 1/2"	12.30	32.35	64.60
1	Pl	19 1/2" × 3/8"	3'-0"	24.86	74.58	74.60
1	"	19 1/2" × 3/8"	2'-2"	24.86	53.95	54.00
1	"	22" × 3/8"	2'-7 1/2"	28.05	73.77	73.77
						4045.30 × 2 = 8091

8091.

Summary for Bottom Chord : 54154#

54154

CALCULATIONS FOR

List of Materials for Furukawa-Bashi

			Diagonal	D1	4 req'd			
2	[s	12" x 3½"	29'-1½"	26.10	760.29	1520.60		
4	Pls	12" x 5/16"	1'-0¾"	12.22	12.95	51.80		
120	Bars	2¼" x 5/16"	1'-1½"	2.39	2.70	324.00		
						1896.40	x 4 =	7586#
			Diagonal	D2	4 req'd			
2	[s	12" x 3½"	35'-8½"	26.10	932.03	1864.10		
4	Pls	11½" x 5/16"	1'-0¾"	12.22	12.95	51.80		
146	Bars	2¼" x 5/16"	1'-1 7/16"	2.39	2.68	391.30		
						2307.20	x 4 =	9229#
			Diagonal	D3	4 req'd			
2	[s	12" x 3½"	35'-6"	26.10	926.55	1853.10		
4	Pls	11½" x 5/16"	1'-0¾"	12.22	12.95	51.80		
144	Bars	2¼" x 5/16"	1'-1 7/16"	2.39	2.68	385.90		
						2290.80	x 4 =	9163#
			Diagonal	D4	4 req'd			
2	[s	12" x 3½"	37'-11⅛"	26.10	990.50	1981.00		
4	Pls	11½" x 5/16"	1'-0¾"	12.22	12.95	51.80		
154	Bars	2¼" x 5/16"	1'-1 7/16"	2.39	2.68	412.70		
						2445.50	x 4 =	9782#
					Summary for Diagonals			35760#
			Ve-fical	V1E				
			Vertical	V1E	2 req'd each			
2	Ls	4" x 3" x 3/8"	21'-9"	8.50	184.88	369.80		
2	Ls	4" x 3" x 3/8"	22'-0"	8.50	187.00	364.00		
1	Pl	11½" x 3/8"	3'-8"	14.66	53.80	53.80		
1	"	11½" x 3/8"	1'-0¾"	14.66	15.54	15.50		
4	Ls	4" x 3" x 3/8"	0'-6⅜"	8.50	4.51	18.00		
24	Bars	2¼" x 3/8"	1'-2⅛"	2.87	3.39	81.40		
						902.50	x 4 =	3610#
			Vertical	V2	4 req'd			
2	[s	10" x 3½"	27'-3"	23.55	641.74	1283.50		
4	Ls	4" x 3" x 3/8"	0'-6⅜"	8.50	4.51	18.00		
4	"	3¾" x 3" x 3/8"	2'-4"	8.30	19.34	77.40		
1	Pl	12" x 3/8"	2'-10¾"	15.30	44.37	44.40		
4	Ls	3¾" x 3" x 3/8"	2'-8½"	8.30	22.49	90.00		
1	Pl	12" x 3/8"	2'-8½"	15.30	41.46	41.50		
4	Ls	3¾" x 3/8"	2'-6¾"	4.78	12.24	49.00		
1	Pl	12" x 3/8"	2'-6¾"	15.30	39.17	39.20		
4	Pls	11½" x 5/16"	1'-0¾"	12.22	12.95	51.80		
110	Bars	2¼" x 5/16"	1'-1 7/16"	2.39	2.68	294.80		
						1989.60	x 4 =	7958#
			Vertical	V3	4 req'd			
4	Ls	4" x 3" x 3/8"	32'-0"	8.50	272.00	1088.00		
4	"	4" x 3" x 3/8"	0'-6⅜"	8.50	4.51	18.00		
1	Pl	12½" x 3/8"	3'-8"	15.94	58.50	58.50		
1	"	12½" x 3/8"	3'-2½"	15.94	51.17	51.20		
1	"	12½" x 3/8"	2'-9¼"	15.94	44.15	44.20		
41	Bars	2¼" x 3/8"	1'-2⅛"	2.87	3.39	139.00		
						1398.90	x 4 =	5596#

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CALCULATIONS FOR

List of Materials for Furukaw-Bashi

			Vertical	V4	4 req'd		
2	Ls	10" × 3 1/2"	32'-9"	23.55	771.26	1542.50	
4	Pls	11 1/2" × 5/8"	1'-0 3/4"	12.22	12.95	51.80	
136	Bars	2 1/4" × 5/16"	1'-1 7/16"	2.39	2.68	364.50	
1	Pl	12" × 3/8"	2'-10 3/4"	15.30	44.37	44.40	
4	Ls	4" × 3" × 3/8"	0'-6 3/8"	8.50	4.51	18.00	
4	"	3 3/4" × 3" × 3/8"	2'-4"	8.30	19.34	77.40	
1	Pl	12" × 3/8"	2'-8 1/2"	15.30	41.46	41.50	
4	Ls	3 3/4" × 3" × 3/8"	2'-8 1/2"	8.30	22.49	90.00	
1	Pl	12" × 3/8"	2'-6 3/4"	15.30	39.17	39.20	
4	Ls	3 3/4" × 3" × 3/8"	2'-6 3/4"	8.48	21.71	86.80	
						3256.10 × 4 = 14024#	
			Vertical	V5	2 req'd		
4	Ls	4" × 3" × 3/8"	34'-9"	8.50	295.38	1181.50	
4	"	4" × 3" × 3/8"	0'-6 3/8"	8.50	4.51	18.00	
1	Pl	12 1/2" × 3/8"	4'-2 1/4"	15.94	66.79	66.80	
1	"	12 1/2" × 3/8"	3'-2 1/2"	15.94	51.17	51.20	
1	"	12 1/2" × 3/8"	2'-9 1/4"	15.94	44.15	44.20	
34	Bars	2 1/4" × 3/8"	1'-2 5/8"	2.89	2.82	115.30	
						1477.00 × 2 = ~~2454~~ 2954#	
				Summary for verticals			34142#
			Portal Bracing	PB1		2 req'd	
2	Ls	4" × 3" × 5/16"	20'-10"	7.20	149.98	300.0	
1	L	4" × 3" × 5/16"	21'-0"	7.20	151.00	151.0	
4	Ls	4" × 3" × 5/16"	22'-0"	7.20	158.40	633.6	
3	Pls	17 1/2" × 5/16"	1'-6 1/2"	18.59	28.63	85.9	
3	"	12" × 5/16"	1'-6 1/2"	12.75	19.64	58.9	
4	Pls	17" × 5/16"	1'-8 1/2"	18.06	30.88	123.5	
4	Pls	17" × 5/16"	1'-6 1/2"	18.06	27.81	111.2	
4	Ls	4" × 3" × 5/16"	4'-4 1/4"	7.20	31.32	125.3	PD1
2	Pls	15" × 5/16"	4'-1 3/4"	15.94	66.15	132.3	PD1
4	Ls	4" × 3" × 5/16"	4'-0"	7.20	28.80	115.2	PD2
2	Pls	15" × 5/16"	3'-9 1/2"	15.94	60.40	120.8	"
4	Ls	4" × 3" × 5/16"	3'-2 1/2"	7.20	23.11	92.4	PD3
2	Pls	15" × 5/16"	3'-0"	15.94	47.82	95.6	"
4	Ls	4" × 3" × 5/16"	3'-1"	7.20	22.18	88.7	PD4
2	Pls	15" × 5/16"	2'-10 1/2"	15.94	45.91	91.8	"
1	Pl	15" × 5/16"	20'-10"	15.94	332.03	332.0	
1	"	15" × 5/16"	21'-9"	15.94	346.70	346.7	
						3004.9 × 2 = 6010#	
			Top Laterals				
4	Ls	6" × 4" × 3/8"	27'-10"	12.30	342.31	1396.2	TL1 R(1) L(1)
2	Pls	21 3/4" × 3/8"	1'-11"	27.73	53.24	106.5	"
8	Ls	6" × 4" × 3/8"	13'-5 3/4"	12.30	165.80	1326.4	TL2 (4)
							2802.1
4	Ls	6" × 4" × 3/8"	27'-7 3/4"	12.30	340.10	1360.4	TL3 R L
2	Pls	21 3/4" × 3/8"	1'-11"	27.73	53.24	106.5	"
8	Ls	6" × 4" × 3/8"	13'-4 5/8"	12.30	164.70	1317.6	TL4
							2784.5

CALCULATIONS FOR

List of Materials for Furukawa-Bashi

4	Ls	6"×4"×3/8"	27' 6 3/8"	12.30	338.62	1354.5	TL5R
2	Pls	21 3/4"×3/8"	1' 11"	27.73	53.24	106.5	"
8	Ls	6"×4"×3/8"	13' 4"	12.30	163.96	1311.7	TL6
							2772.7#
4	Ls	6"×4"×3/8"	27' 5 3/8"	12.30	337.88	1351.5	TL7
2	Pls	21 3/4"×3/8"	1' 11"	27.73	53.24	106.5	"
8	Ls	6"×4"×3/8"	13' 3 9/16"	12.30	163.59	1308.7	TL8
							2766.7#
							11,127#

Bottom Lateral Bracing BL1R to BL7

4	Ls	5"×3"×3/8"	28' 1 1/2"	9.80	275.67	1102.7	BL1R
2	Pls	20"×3/8"	3' 0"	25.50	67.50	135.0	"
4	"	4 3/4"×5/16"	1' 1 1/2"	5.05	5.68	22.7	"
32	Bars	3"φ×5/16"		3.19	.80	25.6	"
2	Pls	8 1/4"×5/16"	1' 1 1/8"	8.77	9.56	19.2	"
4	Ls	3"×3"×5/16"	0' 10"	6.10	5.06	20.2	"
							1325.4
4	Ls	5"×3"×3/8"	13' 11 1/8"	9.80	136.51	546.0	BL2
4	Pls	4 3/4"×5/16"	1' 1 1/2"	5.05	5.68	22.7	"
16	Bars	3"φ×5/16"		3.19	.80	12.8	"
							581.5
4	Ls	5"×3"×3/8"	13' 5 3/8"	9.80	131.81	527.2	BL3
4	Pls	4 3/4"×5/16"	1' 1 1/2"	5.05	5.68	22.7	"
16	Bars	3"φ×5/16"		3.19	.80	12.8	"
							653.0
4	Ls	5"×3"×3/8"	28' 7 1/4"	9.80	280.28	1121.1	BL4R
2	Pls	17"×3/8"	2' 6 3/4"	21.68	55.50	111.0	"
4	"	4 3/4"×5/16"	0' 10 5/8"	5.05	4.60	18.4	"
32	Bars	3"φ×5/16"		3.19	.80	25.6	"
2	Pls	8 1/4"×5/16"	1' 1 1/8"	8.77	9.56	19.1	"
4	Ls	3"×3"×5/16"	0' 10"	3.61	2.65	10.6	"
							1305.8
8	Ls	5"×3"×3/8"	13' 11 1/8"	9.80	136.51	1092.1	BL5
8	Pls	4 3/4"×5/16"	0' 10 5/8"	5.05	4.60	36.8	"
32	Bars	3"φ×5/16"		3.19	.80	25.6	"
							1154.5
12	Ls	4"×3"×5/16"	27' 7 1/4"	7.2	205.92	2471.0	BL6R
6	Pls	15"×3/8"	2' 1 1/2"	19.13	40.75	244.5	"
12	"	3 3/4"×5/16"	0' 8 1/4"	3.98	2.75	33.0	"
96	Bars	3"φ×5/16"		3.19	.80	76.8	"
6	Pls	8 1/4"×5/16"	1' 1 5/8"	8.77	10.00	60.0	"
12	Ls	3"×3"×5/16"	0' 10"	6.10	5.06	60.7	
							2946.0
24	Ls	4"×3"×5/16"	13' 11 1/8"	7.20	100.30	2407.2	BL7
24	Pls	3 3/4"×5/16"	0' 8 1/4"	3.98	2.75	66.0	"
108	Bars	3"φ×5/16"		3.19	.80	86.4	"
							2560.0
							10436#

List of Materials for Furukawa-Bashi

Sway Bracing SB1.SB2.SB3 (2 req'd each) & SB4 (1 req'd)

4	Ls	4"×3"×5/16"	21'-0¼"	7.20	151.34	605.4	
2	"	6"×4"×3/8"	21'-3¼"	12.30	261.62	523.2	
4	"	4"×3½"×5/16"	1'-0"	7.70	30.80	30.8	
4	"	4"×3½"×5/16"	0'-9"	7.70	5.78	23.1	
4	"	4"×3½"×5/16"	2'-8½"	7.70	20.88	83.5	
4	"	3"×3"×5/16"	7'-0"	3.61	42.70	170.8	
8	"	3"×3"×5/16"	7'-2"	3.61	43.74	349.9	
2	Pls	15½"×5/16"	2'-2½"	16.47	36.40	72.8	
4	"	14"×5/16"	1'-10½"	14.88	27.97	111.9	
6	"	7"×5/16"	0'-9"	7.44	5.58	33.5	
2	"	17½"×5/16"	2'-8½"	18.95	50.38	100.8	
4	"	14½"×5/16"	1'-2½"	15.41	18.65	74.6	
5	"	4¾"×5/16"	0'-5½"	5.05	2.32	11.6	
						2191.9 × 7	= 15343

End Floor Beam FB1 2 req'd

4	Ls	5"×3½"×3/8"	21'-8½"	10.40	225.78	903.3	Flg-Ls
1	Pl	30"×3/8"	21'-8"	38.25	828.88	828.9	Web plates
4	Ls	3½"×3½"×3/8"	2'-5¾"	8.50	21.08	84.3	End stiff
10	"	4"×3"×5/16"	2'-5¾"	7.20	17.86	178.6	Stiff
8	"	4"×3"×5/16"	2'-6½"	7.20	18.29	146.3	"
4	Pls	3¾"×3/8"	1'-11¼"	4.14	8.03	32.1	Fillers
10	"	3"×3/8"	1'-11¼"	3.83	7.43	74.3	"
5	Ls	4"×3"×3/8"	1'-1¼"	8.50	9.35	46.8	
5	Pls	3"×3/8"	0'-10¼"	3.83	3.26	16.3	Fillers
						2310.9 × 2	= 4622

Intermediate Floor Beam FB2 (2 req'd) & FB3 (7 req'd)

4	Ls	5"×3½"×½"	21'-9¾"	13.60	296.62	1186.5	
1	Pl	30"×3/8"	21'-9¼"	38.25	832.70	832.7	
4	Ls	3½"×3½"×3/8"	3'-0½"	8.50	25.84	103.3	
10	"	4"×3"×5/16"	2'-5½"	7.20	17.71	177.1	
8	"	4"×3"×5/16"	2'-6½"	7.20	18.29	146.3	
4	Pls	3¾"×½"	1'-11¼"	5.53	10.72	42.9	
10	"	3"×½"	1'-11¼"	5.10	9.89	98.9	
10	Ls	4"×3"×5/16"	1'-1¼"	7.20	7.92	79.2	
10	Pls	3"×3/8"	0'-10¼"	3.83	3.26	32.6	
						2699.5 × 9	= 24296

Summary for Floor Beams 28917.#

Stringers $S1^{R}_{L}$ (10 req'd each) & S2 (20 req'd) & $S3^{R}_{L}$ (5 req'd each)

20	Is	15"×3".42	20'-1⅛"	33.90	681.13	13634.0	$S1^{R}_{L}$ Carnegie Sect
20	Is	15"×5½"	20'-1⅛"	42.90	862.70	17254.0	S2 "
10	Is	18"×6"	20'-1⅛"	54.70	1100.00	11000.0	$S3^{R}_{L}$ "
						42331.0	

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CALCULATIONS FOR

List of Materials for Furukawa-Bashi

Dimension	Valume	no	Total vol	Weight
	Shoe at movable end		2 req'd	
½" × 2'-2" × 1'-4¼" × 1½"	= 316.875			
− ½" × 3" × 5" × ½"	= 3.75	270.74 × 2	= 541.48 in³	(× .2835 = 153.51#) Part A
− π × 3"² × 1½"	= 42.39			
½" × 7" × 4½" × 1"	=	15.75 × 4	= 63.00"	
½" × 3" × 4½" × 1"	=	6.75 × 2	= 13.50"	
1'-3¾" × 4" × 1"	=	63.00 × 1	= 63.00"	
1'-3¾" × 1½" × 1"	= 23.63	39.76 × 2	= 79.52"	
10¾" × 1½" × 1"	= 16.13			
2'-5¾" × 2'-2" × 1½"	=	1160.25 × 1	= 1160.25"	
2" × ¼" × 2'-5¾"	=	14.88 × 1	= 14.88"	
			1935.63 in³ = 1.12 × 490	= 548.8
	Dust Guard	4 req'd		
6½" × ½" × 2'-2"	=	× 1	= 79.63	
¾" × ½" × 2'-2"	=	× 1	= 4.88	
3½" × ½" × 2'-2"	=	× 1	= 22.75	
½" × 5⅝" × 3½" × ½"	= 4.92	× 2	= 9.84	
			117.10 in³ × .2835	= 33.2#
	4 req'd		33.2# × 4 = 132.8#	
	Dust Guard Plate.		4 req'd	13.06#
2'-5½" × 5" × 5/16" @ 5.31#				
	4 req'd		13.06# × 4 = 52.24#	
	Bed	2 req'd		
2'-2" × 1½" × 3'-2"	= 148.2	× 1	= 148.2	
¼" × 2" × 2'-2"	= .13	× 1	= 13	
½(2½"+1½") × 2" × (3'-1"+2'-1")	= 248	240 × 1	= 240	
− ½(2½"+1½") × 2" × ½(2½"+1½")	= 8			
			1735 in³ × .2835	= 491.9
	2 req'd		491.9 × 2 = 984#	
	Roller-nest		2 req'd	
4"φ × 2'-4½" @ 42.73# = 101.27 × 5 = 506.35#				
3" × ½" × 1'-9" @ 5.1# = 8.93 × 2 = 17.86#				
			524.2 × 2 =	1048.4#
	Shoe at fixed end		2 req'd	
Part A (See "Shoe at movable end")				153.5#
5½" × 1½" × 2'-2"	= 214.5	× 1	= 214.5	
7" × 1" × 1'-3¾"	= 110.3	× 3	= 330.9	/
5½" × 1" × 8½"	= 46.8	× 2	= 275.2	
5½" × 1" × 1'-0½"	= 68.8	× 4	= 93.6	
2'-4¾" × 1½" × 2'-1"	= 1078.0	1070 × 1	= 1070.0	
− ½(2½"+1½") × 2 × ½(2½"+1½")	= 8			
			1984.2 × .2835	= 562.5#
				716.0#
	2 req'd		716.0 × 2 = 1432#	

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List of Materials for FuruKawa-Bashi

Bolts (tapped)

32 7/8"φ × 3/4" (Length underhead) @ 1.125# 36#

Pins 4 req'd

Pins 1'-11" long { 6"φ × 1'-8 1/2" @ 96.13# = 167.27
{ 4 1/2"φ × 3 1/2" @ 54.07 = 156.8

Two Nuts 2" @ 7.8 = 15.6

338.7#

4 req'd × 4 = 1354.8#

Anchor Bolts

24 1 1/4"φ @ 29.1# = 698.4#

Summary for the weight of Shoes and Pins

Shoes at movable end	1098.
Dust Guard	133
Dust Guard plates	52
Bed	984
Roller Nests	1048
Shoes at fixed end	1432
Bolts	36
Pins	1355
Anchor Bolts	698
	6836

Weight of Metal for one truss Span

Top Chords	70364
Bott Chords	54155
Diagonals	35760
Verticals	34142
Portal Bracing	6010
Top laterals	11127
Bott laterals	10436
Sway Bracings	15343
Floor Beams	28917
Stringers	42331
Shoes & pins	6836
Rivet heads	10102
	324523# or 144.876 tons

Number and weight of Rivet Heads.

Dia of riv	Head of Shop Riv	Head of Field Riv	Sum	Wt of 100 head	Total
3/4"	2×8280 = 16560	2×6566 = 13132	29692	14.25	3792#
7/8"	2×11089 = 22178	2×2218 = 4436	26614	21.25	6310#
Sum	38738	17568			10102# or 4.512 tons

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CALCULATIONS FOR

List of materials for Furukawa-Bashi for Tokushima-Ken.

Summary for structural steel 1 Truss span complete 144.876 tons
Rivet Heads. ~~shims~~ 4.512
149.388 tons per span

17 @ 149.388 tons = 2539.596 tons.

Concrete in slab and Copings.
Roadway slab. 0.5 · 20.00 = 10.000
filler 0.028
Coping 0.4 · 0.86 = 0.344
0.28 · (0.86+0.55) ½ = 0.196
0.5 · 0.49 = 0.245
0.785 · 2 = 1.570
11.598 sq ft

Total length of slab for one span 207.41 ft
Volume = 11.598 · 207.41 = 2405.54 立尺 or 11.136 立坪

17 spans @ 11.136 = 189.312 立坪

Reinforcing bars in slab. deformed bars. (see drawing # 8).
For Floor slab A. 2 @ 1.291 tons = 2.582
" Floor slab B 3 @ 1.252 tons = 3.756
6.338 tons per one span

17 spans @ 6.338 = 107.746 tons

Soliditit Pavement Pavement Area
Thickness 2" For one span 20. · 207.41/36 = 115.227 面坪

17 spans @ 115.227 = 1958.86 面坪

Finish of copings. 人造洗出仕上.
For one span 2 @ 2.84 · 207.41 = 1178.088 or 32.724 面坪
17 spans @ 32.724 = 556.3 面坪

Area of Forms for Concrete slab.
bottom + sides 22.51 · 207.41 = 4669
both ends. ·70 · 21.62 = 30
4699 or 130.53 面坪 for one span
17 spans @ 130.53 = 2219.0 面坪

Structural steel in expansion over pier 16 Required

4L	6"·3½"·5/16	· 19'-10 5/8	@ 9.8	=	195
1L	3½·3·5/16	· 19'-10 5/8	@ 6.6	=	131
2L	3·3·5/16	· 0'-10"	@ 6.1	=	11
1PL	10·3/8	· 19'-10 5/8"	@ 12.75	=	254
18 anchor bars	2½·5/16 @ 1'-6"		@ 2.66	=	72
	Rivet heads say				27
					690# per expansion.
					0.308 ton

16 @ 0.308 = 4.928 tons

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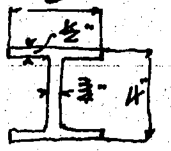
CALCULATIONS FOR

List of materials for Furukawa-Bashi for Tokushima Ken.

material for expansion joint over abutment A1
Lead Plate 12" • 1/8" • 22 ft about @ 7.35# = 162#
asphalt filling - .25 • .67 • 20 = 3.35 cubic ft

Drains. 170 required 46# per piece.

Handrails.
Cast iron posts 1398 Required



3" • 1/2" • 2" = 3.00
3" • 3/4" = 2.25
5.25 • 37.27 = 195.66 cubic inch
4.5 • 0.625 = 12.50
208.16

wt = 208.16 @ 0.26 = 54.12# per piece 1398 @ 54.12 = 75659.76#
or 33.777 tons.

Typical Panel 1360 panels required

1 -	2" gas pipe		5'-0 1/4"	@ 3.652	=	18
1	bar	2" • 5/8"	• 5'-5 1/4"	@ 4.25	=	23
8	bars	1 1/2" • 5/8	• 2'-3"	@ 3.19	=	58
4	"	1 1/2 • 5/8	• 0'-9 1/2	@ "	=	10
4		1 1/2 • 3/8	• 1'-8"	@ 1.91	=	13
1 L		2 1/4 • 2 1/4 • 5/16	• 5'-5 1/4	@ 4.28	=	23
						145#

1360 @ 145# = 197200#

Panel over pier 32 Required

1 -	2" gas pipe		2'-4 1/2"	@ 3.652	=	9
1 -	bar	2 • 5/8	• 2'-7"	@ 4.25	=	11
4 -	bars	1 1/2" • 5/8	• 2'-3"	@ 3.19	=	29
2 -	"	" • "	• 0'-9 1/2	@ "	=	5
2 -	"	1 1/2 • 3/8	• 1'-8"	@ 1.91	=	7
2	L3	4 • 3 • 3/8	• 0'-2"	@ 12.5	=	4
1	L	2 1/4 • 2 1/4	• 5/16 • 2'-7"	@ 4.28	=	11
						76#

32 @ 76 = 2432#

Panels at abutments 4 Required

1 -	2" gas pipe		• 0'-10"	@ 3.652	=	3
1 -	bar	2 • 5/8	• 1'-2 1/2	@ 4.25	=	5
1 -	"	1 1/2 • 5/8	• 2'-3"	@ 3.19	=	7
1 L		2 1/4 • 2 1/4 • 5/16	• 1'-2 1/2	@ 4.28	=	5
						20#

4 @ 20 = 80#

Anchor Plates and bolts.

1398 Pls	3 • 5/16	• 0'-7"	@ 3.19	=	2600
2796 bolts	1/2"	• 0'-6"	@ 0.46	=	1286
					3886#

For Typical Panels	197200
Panel over piers	2432
Panel over Abutments	80
Anchors.	3886
	203598# or 90.891 tons.
Handrail posts	33.777
	124.668 tons

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CALCULATIONS FOR

List of materials for Furukawa-Bashi for Tokushima-Ken.

End Pedestals. 4 Required

Volume of Concrete

2.8 · 2.8 · 4.0 = 31.40
1.6 · 2.7 · 0.05 · 2 = 0.43
4.0 · 2 · 2 = 16.00
8. · 0.25 · 4. · 1.55 = 1.24
49.07 ÷ 216 = 0.227 立坪   4 @ 0.227 = 0.908 立坪

Area of Forms for Concrete

2.8 · 4 · 4 = 44.8
2.0 · 4 · 4 = 32.0
0.25 · 1.8 · 16 = 7.2
84.0 ÷ 36 = 2.33 面坪   4 @ 2.33 = 9.32 面坪

Area for 人造仕上

0.1 · 11.6 = 1.16
0.05 · 16 = 0.80
1.96 ÷ 36 = 0.06 面坪
2.33
2.39 面坪   4 @ 2.39 = 9.56 面坪

Reinforcing bars

8 bars ½"ϕ · 5.0 @ .668 = 27
8 " " · 5.0 " = 27
2 " ⅜"ϕ · 10.5 @ .376 = 8
3 " " · 7.5 " = 7
69#   4 @ 69 = 276#

1 gas pipe 2½"ϕ · 11'-0" @ 5.793# = 64#   4 @ 64 = 256

Bronze fixture 254#   4 @ 254 = 1016#

袖柱. 8 Required.

Volume of Concrete

2.05 · 3.25 · 2.3 = ~~15.3~~ 15.3 立尺 or 0.071 立坪   8 @ 0.071 = 0.568 立坪

Forms for concrete

2.05 · 3.25 · 2 = 13.3
2.3 · 3.25 · 2 = 15.0
28.3 ÷ 36 = 0.79 面坪   8 @ ~~0.88~~ 79 = ~~7.04~~ 6.32 面坪

人造洗出面積

2.3 · 2.05 · = 4.7
.5 · 3.0 = − 1.5
+28.3
31.5 ÷ 36 = 0.88   8 @ .88 = 7.04 面坪

Reinforcing bars

3 - ⅜"ϕ · 8.0 @ .376 = 9
4 - ½"ϕ · 4.0 @ .668 = 10.7
19.7#   8 @ 19.7 = 157.6#

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CALCULATIONS FOR

List of materials for Furukawa-Bashi, for Tokushima-Ken.

Concrete Handrails   4 Required

Volume of Concrete

0.55 · 30 · 11.65 = 19.2
0.8 · 0.4 · .52 · 11 = -1.83
17.37 ÷ 216 = 0.08 立坪   4@0.08 = 0.32 立坪

Forms for Concrete ·

3.0 · 11.65 · 2 = 69.9
0.4 · 2.12 · 11.0 = 9.3
0.8 · .52 · 22.0 = -9.2
70.0 ÷ 36 = 1.95 面坪   4@1.95 = 7.80 面坪

人造石仕上面坪

30 · 11.65 · 2 = 69.9
0.52 · .8 · 22 = -9.2
0.6 · 11.65 = 7.0
2.8 · 0.4 · 11. = 12.3
80.0 ÷ 36 = 2.22 面坪   4@2.22 = 8.88 面坪

Reinforcing Bars

26 - 3/8 · 3'-6" @ .376 = 34
6 - 3/8 · 8'-6" " = 19
6 - 3/8 · 3'-10" " = 9
62# · 4 =   4@62# = 248#

Lamp Brackets on bridge   34 Required

1 Cast iron 247.5 @ 0.26 = 64
1 " " 4.0 @ 0.26 = 1
1 gas pipe 1¼"φ · 2'-2" @ 2.272 = 5
8 bolts ½"φ · 0'-2" 2
72# per piece

Bronze Name Plate 厚3/8 1.0' · 2.0' 3/4" 厚 with anchor bolts   66# about   二個
2 Required
設計者及監督人氏名標   do.   66#   二個

里程標及竣工月日標   0.5 · 1.0 · .75" about   25#   四個

Estimate of Cost Furukawa Bashi - Tokushimaken 1.

The total length of bridge between ℄ bearings on abutments = 3489.5
which will divide into the following spans.

1.	16 piers	17 spans @ 205 +
2	17 piers	18 spans @ 194 +
3.	18	19 spans @ 184
4	19	20 spans @ 174.5
5.	20	21 spans @ 166.0

From investigation, the cost of one pier = 35000 - 38000 yen
Assume the cost of pier 38000 yen & the price of steel 270 per ton

Approximate difference of the weight of steel in 19 spans @ 184' compared with 205 and 21 spans @ 166 compared with 205 are as follows

160# × 3490 = 558,000# 250 tons @ 270¥ = 67500 yen less
300# × 3490 = 1050000# 470 tons @ 270¥ = 127000 less

The cost of bridge for layout # 3. increase 2 @ 38000 = 76000
decrease 67500
8500 yen increase

The cost of bridge for layout #5 increase 4 @ 38000 = 152000
decrease 127000
25,000 yen increase

%
67500 ÷ 2 = 33750
127000 ÷ 4 = 31700

If the cost of one pier varies between 35000 - 38000 yen it seems to me No.1 layout will give minimum cost of structure. Let us estimate the superstructure of No.1 layout span length 201'-9" between ℄ of End bearings.
2-3" about between bearings on pier.
204'-0"

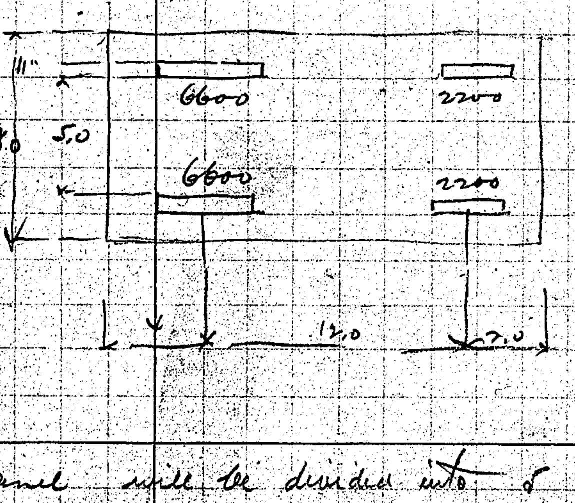
The span shall be divided into 9 panels @ 22'-5" = 201'-9"

See Sketch of general layout #1 for the form of truss.

Loading for bridge.

Uniform Load $q\ \#/\square' = \frac{20480}{170 + \frac{l}{328}}$ where l = span length in ft.

For 201.75' $q = 87.5\ \#/\square'$
For span under 30 meters 102.5 %
Motor truck loading.



Rear	13200	× 1.3 Impact coef = 1/3
Front	4400	× 1.3
	17600	
Impact	5860	
	$\frac{23460}{18 \times 8}$	= 163 #/□' average.

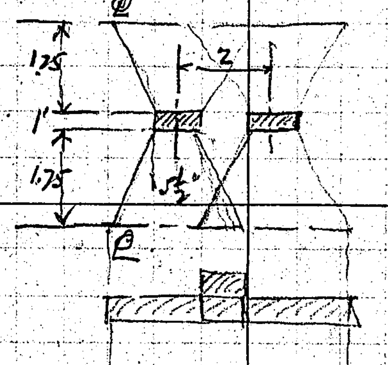
Floor slab.
One truss panel will be divided into 5 spacing of 4.5 centers.

Assumed pavement 3" wood blocks 15
cushion 7
7" concrete 88
110# per square ft

Estimate of cost Furukawa Bashi, Tokushimaken 2.

Dead Load moments $\pm \frac{1}{10} \times 110 \times 4.5^2 = 223^{1\#}$

Live Load moment — neglecting thickness of pavement wheel load will be distributed over



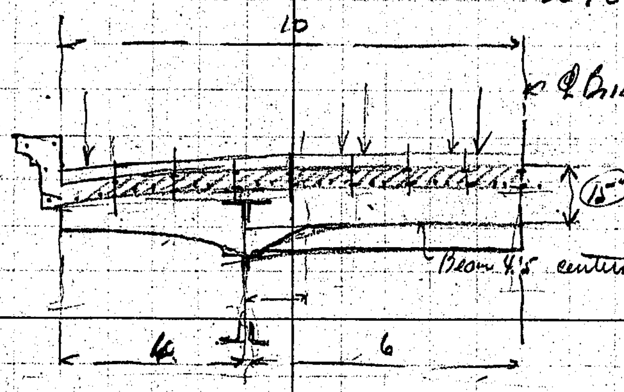
$3.5 \times 0.6 = 2.10$
0.9
3.0
wheel concentration 6600
1/3 impact 2200
8800#

Distribution $\frac{8800}{3} = 2940^{\#}$ per ft. $\frac{8800 \times 2}{5} = 3520$ 1760

Max moment 1760 1470 × 2.25 = 3520 − x 2/3 = 2340
220
2560

For 16000#/□" steel & 600#/□" unit stress
Effective Depth Required = $\sqrt{\frac{2560}{95}}$ = 5.2" make slab 6.25"
$\frac{2560 \times 12}{\frac{7}{8} \times 5.4 \times 16000}$ = 420/ft

End shear = Live Load 1760; DL 110 × 2.25 250; 2010# Unit shear = $\frac{2010}{12 \times 5.25 \times \frac{7}{8}}$ = 36.6#/□"



pavement 3" wood blocks 15
cushion 7
6¼" 78
100#

100 × 4.5 = 450
beam say 85
535#

9½
6
1'3½ weight say 120
Hand r 30
150

Dead Load moment over hanging Arm $535 \times \frac{4^2}{2}$ = 4280 1#
4.5 × 150 × 4.5 = 3040
7320

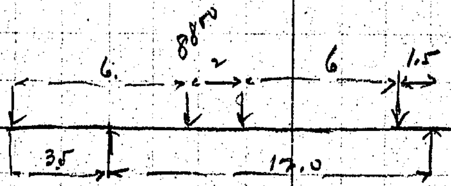
Moment center span $\frac{1}{8} \times 535 \times 12^2$ = 9630 1#
less 7320
2310 1# net positive

Live Load moment 8800 × 3.5 = 30800 1# cantilever
Center span
LL m. 8800 × 5 = 44000 1#

Summary of moments

	Over hanging Arm	Center Span
DL	7320	2310
	30800	44000
	38120 1#	46310 1#

End shear



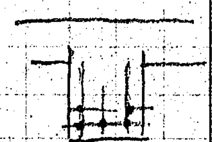
8800 × 1.5
7.5
9.5
LL shear 18.5 ÷ 12 = 13600#
DL shear 535 × 6 = 3200
16800#

width say 9" $\frac{16800}{9 \times 120}$ = 15.5"

Depth at stringer − 19"
15
4"

Reinforcement middle span $m = 46310$

steel Area $= \frac{46310 \times 12}{\frac{7}{8} \times 13 \times 16000} = 3.05''$ 5 - $\frac{7}{8}''$ ⌀ bars.



Reinforcement, cantilever Arm $m = 38120$ $D = 19''$

steel Area $= \frac{38120 \times 12}{\frac{7}{8} \times 17 \times 16000} = 1.92$ 3 - $\frac{7}{8}''$ ⌀ bars.

weight of slab & beam per panel.

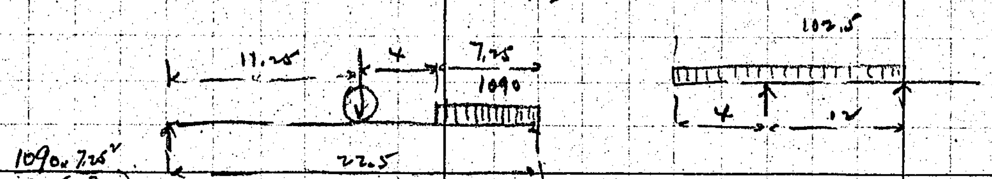
Concrete $20 \times 0.52 \times 22.5 = 234.0$
beam 4 @ $.75 \times .83 \times 21 = 52.2$ } 286.2 Average depth = 7.5''
curb $2 \times 22.5 = 45.0$
331.2

Average $\frac{331.2}{22.5} = 14.7$ @ 150 = 2210#
pavement $20 \times 22 = 440$.
Hand rail 60
2710# per lin ft.

Dead Load per stringer $2710 \div 2 = 1355$#
stringer say 65
1420 $m = \frac{1}{8} \times 1420 \times 22.5^2 = 90000$#

Live Load - $8800 \times \frac{34}{12} = 25000$#

$103.5 \times \frac{16}{12} \times \frac{16}{2} = 1090$#



$\frac{1090 \times 7.25^2}{22.5 \times 2} = 1275$

moment due to concentration $\frac{25000}{2} \times 11.25 = 140500$#
$1275 \times 11.25 = 14300$
LL. 154800
DL. 90000
Total 244800

$S_m = \frac{244,800 \times 12}{16000} = 183.0$

use 24'' - 90# 186.5

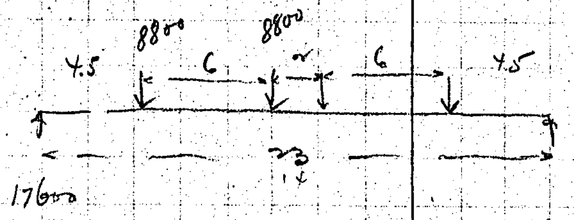
Floor Beam ℄ to ℄ of Trusses = 23.0'

Dead Load moment $2710 \div 20 = 136$#/0' $m = \frac{1}{8} \times 660 \times 23^2 = 43,600$#
$\frac{200 \div 2}{20}$ 10#/0'
146#/0'

Dead Load Uniform $146 \times 4.5 =$ say 660#
concentration at stringer connection - $1355 \times 9 = 12200$
stringer 1125
13325#

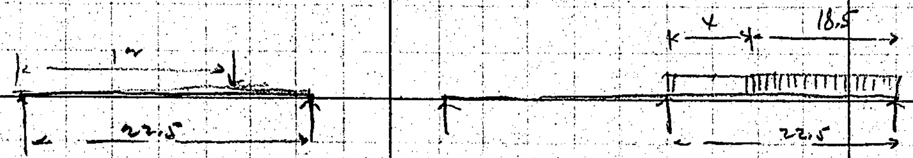
Moment due to concentration $13325 \times 5.5 = 73200$#
43600
Total Dead Load m - 116800#

Live Load Moment concentration. 8800#



moment = 17600 × 10.5 = 185000
8800 × 6 = − 52800
132200

Front wheel. 6600 × $\frac{10.5}{22.5}$ = 3100# moment = 3100 × 5.5 = 17000



Uniform load. — 102.5 × 10 = 1025 Reaction = 1025 × $\frac{18.5^2}{2 \times 22.5}$ = 7800#
moment = 7800 × 5.5 = 43,000

Summary of Live Load Moment

Front Wheel	17000	
Uniform load	43000	
Rear wheel	132200	
L.L. Total		192,200
D.L. Total		116800
		309,000#

Try 30 × 5/16 web = 9.360" ⅛ web = 1.17
Back to Back L's = 30½ Effective Depth = 2.54 − .15 = 2.39 Shear = 309000 ÷ 2.39 = 129000#
SR = 129000 ÷ 16000 = 8.00 − 1.17 = 6.830"
weight of one intermediate Floor Beam

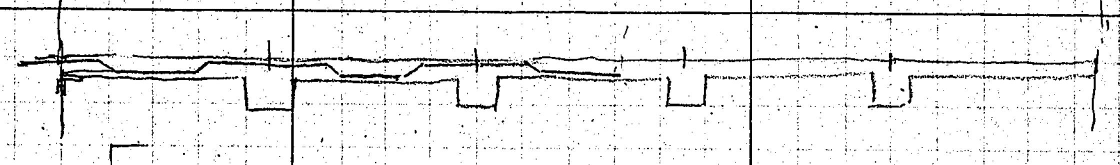
web	30 × 5/16	@ 31.88	× 22.0	=	700
Flanges	4L's 5 × 3½ × ½	@ 13.60	× 22.0	=	1196
Stiffeners	12L's 4 × 3 × 5/16	@ 7.2	× 2.5	=	216
Fills	3½ × ½	@ 5.95	× 1.8	=	43
End stiffs	4L's 3½ × 3½ × 3/8	@ 8.5	× 2.4	=	82
	Rivet heads & variation				120
					2357 call this 2400#

weight of End Floor Beam say 2100

331.2 ÷ 22.5 = 14.7 cubic ft

Concrete in one panel. 331.2 ÷ 216 = 1.53 cubic tsubo
For one span $\frac{1.53}{22.5}$ × 203.75 = 13.8 Cubic tsubo

Reinforcing bars.



72 −	½"	@ .67#	× 13.5 × 2	=	1300	
4 × 5	⅞	@ 2.04	× 22.0	=	900	− 700
4 × 2	⅞	@ 1.04	× 12.0	=	100	
Stirrups say					150	
10 −	½	@ .67	× 21.		141	− 100
					2591# ÷ 1.53	= 1900# per tsubo
					241	
					2350	

Total Reinf = $\frac{2350}{\cancel{2591}}$ × $\frac{203.75}{22.5}$ = 21200# 9.46 tons per span

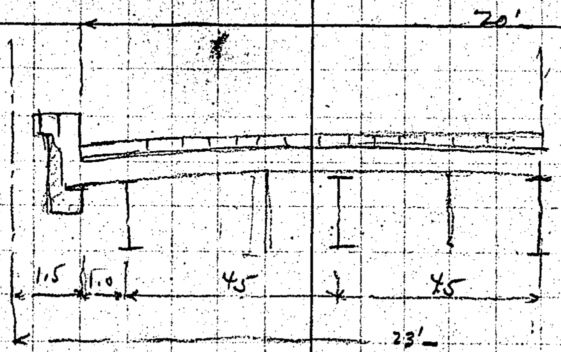
Estimate of Cost Furukawa Bashi - Tokushimaken 5

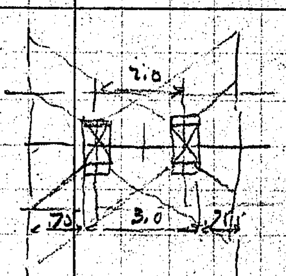
Structural Steel -

Stringers		200 · 203.75	=	40,800 #
F.B.	8 @	2400	=	19200
	2 @	2100		4200
				64200 # ÷ 203.75 = 315 # per ft.

315 × 203.75 = 64,000   28.6 tons

Try cross section of structure as shown below in sketch.





Distribution of load   .75 · 0.6 + 1.0 = 1.45

Assume 2.0 distribution -

$4.5 \div 6 = 2.7 + 1.0 = 3.7$

Concentrated wheel load with 1/3 impact   8800 $2 @ 8800 = \frac{17600}{3.7} = 4750$ #

LL. m = $\frac{4750}{2} \times .75 = 1780$ #

One concentration at center of span.   distribution $\frac{8800}{3} = 2940$ #

LL moment = 1470 × 2.25 = 3300 · 2/3 = 2200 #

DL moment = $\frac{1}{10} \times 110 \times 4.5^2$ = 220

2420

For 16000 #/□" steel + 600 #/□" concrete stress $\sqrt{\frac{2420}{95}} = 5.0$ make slab 6"

Reinforcing steel - $\frac{2420 \times 12}{\frac{7}{8} \times 5 \times 16000}$ = .42   use 1/2" □ bars 5" centers .49 □"

Stringers   span 22.5'

DL	3" wood block	15	
	1/2 cushion	7	
	6" concrete	75	
		97 #	call this 100 #

100 × 4.5 = 450
Beam say 65
515

m = $\frac{1}{8} \cdot 515 \cdot 22.5^2 = 32600$

Live Load   concentration   8800 — 8800
$8800 \times \frac{2.5}{4.5}$ 4900
13700 #

Moment = $\frac{13700}{2} \cdot 11.25 = 77,000$

Uniform load   102.5 × 4.5 = 460 $(460 \times \frac{7.25^2}{2 \times 22.5}) = 537$ #

moment = 537 × 11.25 = 6050
LL 77000
DL 32600
115650

Sm = $\frac{115650 \times 12}{16000} = 87.0$

Use 18" 55 # I   Sm = 88.4
say 60 #

5 @ 60 = 300 # per lin ft.

Intermediate Floor Beam

DL.   100 · 22.50 = 2250 #
100
2350

m = $\frac{1}{8} \cdot 2350 \cdot 23.0^2 = 155,000$ #

Front wheel. $3300 \cdot \frac{10.5}{22.5} = 1540$
8800
10340

Moment   20680 × 10.5 = 217000
10340 · 6   - 62000
155000

See pp. 4

Uniform load . . $\frac{102.5 \times 18.5^2}{2 \times 22.5} = 780^\#$

Uniform $m = \frac{1}{8} \times 780 \times 23.0^2 = 51500$
Concentration 155000
D.L. 155000
361500 $^{1\#}$

Try 30 × 5/16 web = 9.360" $\frac{1}{8}$ web = 1.17
B to B L's = 30½" Effective 2.54 − .15 = 2.39 $S = 361500 \div 2.39 = 151000^\#$
$SR = 151000 \div 16000 = \frac{9.42}{1.17} - 1.17 = 8.25$ net Use 2L's 6 × 4 × ½" = 9.50, 8.50 net

Weight of one intermediate Floor Beam

web	30 × 5/16	@ 31.88	× 22.0	=	700
Flanges	4L's 6 × 4 × ½	@ 16.2	× 22.0	=	1425
Stiffeners	8L's 4 × 3 × 5/16	@ 7.2	× 2.5	=	144
End Stiffs	4L's 3½ × 3½ × 3/8	@ 8.5	× 2.4	=	82
fillers	4 Pl's 3½ × ½	@ 5.95	× 1.8	=	43
Shelf L's	10 L's 3½ × 3½ × 5/16	@ 7.2	× 1.0	=	72
	Rivet heads & variation say				140
					2606 $^\#$

End Floor Beam say 2300 $^\#$

Summary of structural steel.
Intermediate F.B. 8 @ 2606 20848
2 @ 2300 = 4600
$25448 \div 203.75 = 126^\#$
Stringers say 5 @ 60 300
426 $^\#$
426 @ 203.75 = 86600 38.8 tons

Amount of Concrete in floor slab.
Slab 0.5 × 20 = 10.0
Curbs 2.4
12.4 × 203.75 = 2520 ÷ 216 = 11.7 cubic tsubo.

Reinforcing bars.
12/15 4 − ½" @ .67 × 23.0 = 49.33
36 − ½ @ .67 × 1.0 = 17.4
lap etc 5.3
72.03
72 × 203.75 = 14700 or 6.55 tons per span
1260 $^\#$ per cubic tsubo.

Effect on weight of truss due to extra weight of flooring.
14.7
12.4
2.3 @ 150 = say 350 $^\#$ per lin ft.

Moment at center of span $\frac{1}{8} \times 350 \times 203.75^2 = 1,780,000^{1\#}$

Depth of truss say 34' $\text{Stress} = \frac{1,780,000}{34} 1\!/2 = 52400^\#$ $SR = \frac{52400}{12000} = 4.37$

top & bottom 2 − 4.37 = 8.80 × 3.4 $^\#$ = 30 $^\#$ per lin ft.

30 $^\#$ × 203.75 = 61000 27.2 tons per truss.

Comparison of Cost between 2 systems of Flooring.

Case I
concrete in floor	13.8 tsubo @ 240 =	3310.00	
Reinforcing bars.	9.46 tons @ 200 =	1892.	17200
Structural steel in floor	28.6 tons @ 340 =	9720.	15839
" " " truss.	2.7 tons @ 340 =	917	1361.00
		15839.00	17 spans (1361.00) = 23,000 of less cost

Case II.
concrete in floor	11.7 tsubo @ 230 =	2690.00
Reinforcing bars	6.55 tons @ 200 =	1310.00
Structural steel in floor	38.8 tons @ 340 =	13200.00
		17200.00

For case II use reinforced concrete fascia girders & use 4 stringers instead of 5. Reduction of steel weight = 60 × 203.75 = 12,200# or 5.5 tons less.
5.5 tons @ 340 = 1870 per span less.

Using this design cost of cases I + II will be about same for both cases.

Steel in span 38.8
5.5
33.3 @ 17 = 567 tons

Preliminary Design of Furukawa Bashi Tokushimaken.

This bridge spans over Yoshinogawa on main highway between the city of Tokushima and Furukawa machi Tokushimaken. The total length of the crossing 3490 ft. 17 spans of 201'-9" and 2'-3" between ℄ bearings on pier will make proper layout for this crossing.

The structure will be of steel with 20' Roadway of concrete slab with 3" wood block pavement on ½" asphalt mortar cushion.

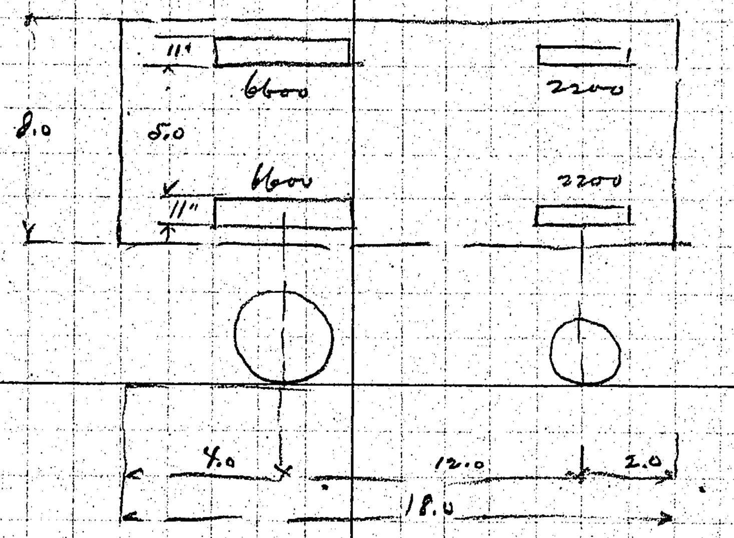
Loadings of bridge.

Uniform live load $q^{kg/m^2} = \frac{100.000}{170+l}$

under 30 meter in span $q = 500\ kg/m^2$

Equivalent load for 201'-9" $q = 87.5\ \#/□'$ under 100' span $q = 102.5\ \#/□'$

motor truck loading.



Impact ⅓

One motor truck on one span in the direction of bridge.

On sideway motor trucks assumed side by side with intervals of 8.0'.

Unoccupied space to be filled with uniform load.

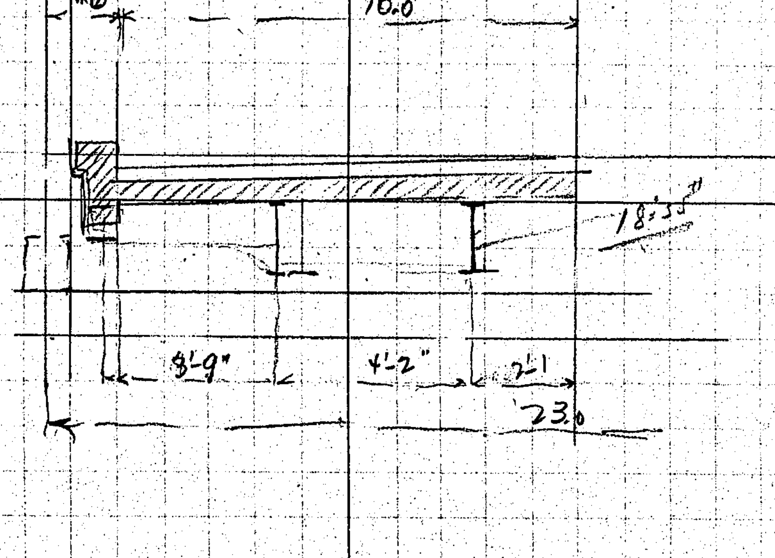
Assumed working strength.

Fibre stress in concrete	650 #/□"	Structural steel Tension	16000 #/□"
Shear reinforced	120 #/□"	Compression	16000 - 70 l/r
Shear plain	40 #/□"	Bending stress	16000
Bond stress	80 #/□"	Shear shop rivets	10,000
Tension in steel	16000 #/□"	field rivets	8,000
Shear in steel	10,000 #/□"	Bearing shop rivets	20,000
fs + fs = n = 15		field rivets	16,000
		bearing on pin	24,000
		Expansion roller	600 l
		On masonry	600 #/□"

For all other specification adopt AREA 1910 specification.

The span will be divided into 9 equal panels of 22'-5" making odd on account of curved chord truss. We made investigation between 9 panels + 11 panels and found the former economical compared with the latter.

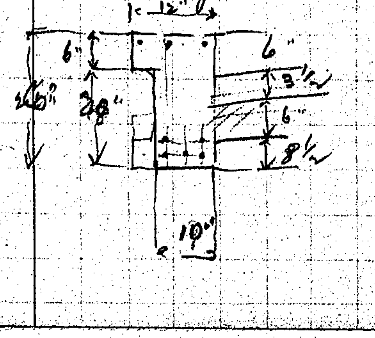
Cross section of structure assumed as shown on sketch.



Roadway slab.

pavement 3" wood block	15
½" Asphalt mortar	5
Concrete slab assumed	75
	95 #/□'

weight of curb



$0.5 \times 1.0 = 0.5$

$1.75 \times 1.5 = 1.125$

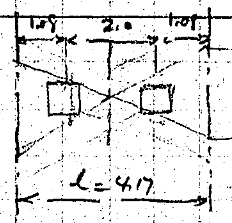
$1.625 @ 150 = 244^{\#}$

Including finish say $260^{\#}$

Weight of Handrail : 30# per lin ft. of structural steel or bar
Design to be properly made not to exceed this weight.

Design of Roadway Slab.
Spacing of stringer 4'-2". DL. m = $\frac{1}{10}$ × 95 × 4.17² = 165#
Live Load. Rear wheel concentration 6600
⅓ impact 2200
8800#

Distribution of wheel concentration.



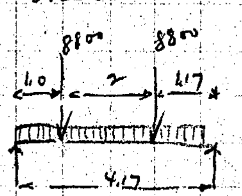
4.17 × 0.6 + 1.0 = 3.5
moment = 8800 × 1.08 = 9500 for 2 wheels.
4400 × 2.08 = 9150 for 1 wheel.
For one ft strip $\frac{9500}{3.5}$ = 2720'#
For continuity of slab. 2720 × 0.8 = 2180'#
DL. m 165
2345'#

Effective Depth required = $\sqrt{\frac{2345}{107}}$ = 4.7" for 650#/□" concrete stress + 16000#/□" steel stress

Use 6" slab with 5" effective Steel Area = $\frac{2345 \times 12}{\frac{7}{8} \times 5 \times 16000}$ = .400

Use ½"ϕ bars 5" centers as shown in sketch

End shear



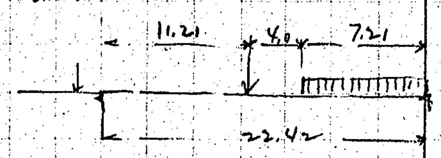
8800 × $\frac{4.34}{4.17}$ = 9150#
For one ft strip 9150 ÷ 3.5 = 2620#
DL. 95 × 2.08 = 200
2820

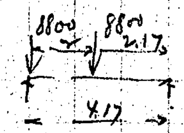
Unit Shear = $\frac{2820}{\frac{7}{8} \times 5 \times 12}$ = 537#/□" Use bent up bars to carry the excess of shear.

For 15" strip $\frac{2820 \times 15}{12}$ = 3520# $\frac{3520}{5}$ = 700#
perimeter for 3-½"ϕ bars = 4.7 700 ÷ 4.7 = 149#/□" O.K. for bond stress.

Steel Stringer span 22'-5" DL m = $\frac{1}{8}$ × 460 × 22.42² = 29,000'#
DL. 95 × 4.17 = 396#
Stringer say 64
460#

Live Load



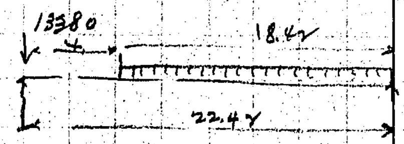


Uniform load 1025 × 4.21 = 432#
Reaction = $\frac{432 \times 7.21^2}{2 \times 22.42}$ = 501#
moment = 501 × 11.21 = 5520'# 5520
Rear wheel 8800 × $\frac{2.17}{4.17}$ = 4580
8800
13380#
moment = 13380 × 11.21 = 175,000
90520
Dead Load m 29000
109520

Sm Required = $\frac{109520 \times 12}{16000}$ = 82.2

Use 18" × 55# I Sm = 88.4

End shear

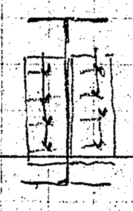


$\frac{4.3 \times 18.42^2}{2 \times 22.42}$ = 3280#
13380
17660#

no of Rivs. 7/16" bearing 7/8" Rivs

$\frac{17660}{4375}$ = 4.03

Use standard connection



4L 3½ × 3½ × 3/8 × @ 8.5 × 1.0 = 34
Rivs 6 3
37.

weight of 1 stringer

18" × 55# × 22.33 = 1230#
4Ls 3½ × 3½ × 3/8 @ 8.5 × 1.0 = 34
Rivs + variation say 30
1294 ÷ 22.42 = 57.5# per lin ft.

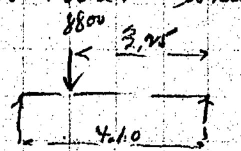
4 @ 57.5# = 230.0# per lin ft.

Fascia girder.
Assumed Dead Load.

Handrail	30
Beam say	260
95 × $\frac{3.75}{2}$ =	178
	468#

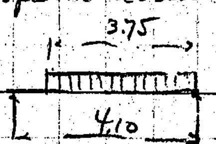
DLm = $\frac{1}{10}$ × 468 × 22.42² = 23,600#

Live Load. wheel load Reaction = 8800 × $\frac{3.75}{4.10}$ = 6970#



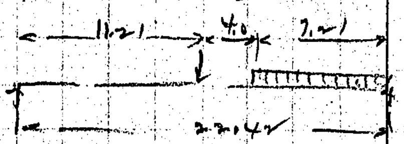
moment = 6970 × 11.42 = 39800
For continuity 39800 × 0.8 = 31700#

Uniform load



$\frac{102.5 \times 3.75^2}{2 \times 4.10}$ = 175#   Reaction $\frac{175 \times 7.21^2}{2 \times 22.42}$ = 203.0#

moment = 203 × 11.21 = 2270#
For continuity 2270 × 0.8 = 1810



Summary for moment

DL. m	23600
LL. wheel conc	31700
LL. uniform	1810
	57110#

for both neg. + pos. moment

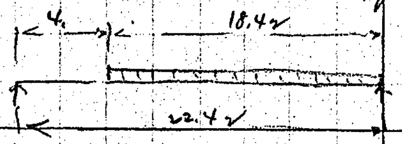
Effective Depth required = $\sqrt{\frac{57110 \times 12}{10 \times 107}}$ = $\sqrt{633}$ = 25.2" at support

Effective depth required = $\sqrt{\frac{57110}{107}}$ = 23.2" at center.

Steel Area required = $\frac{57110 \times 12}{\frac{7}{8} \times 23 \times 16000}$ = 2.12 □" Use 5-3/4"□ bars = 2.20"

Make depth of beam 26" throughout. Compressive rein for on compression side.

End shear. Uniform load $\frac{175 \times 18.42^2}{2 \times 22.42}$ = 1330



wheel conc. 6970
8300

Dead load 468 × 11.21 = 5250
13550#

Unit shear = $\frac{13550}{\frac{7}{8} \times 23 \times 10}$ = 67.5 #/□" use stirrups

Investigate moment +c for this beam and reduce ~~the~~ size.

Intermediate Floor Beam   span length 23.0'

Dead load

0.5 × 20 = 10.0
curb. 2-2.87 × .83 = 3.6
2-0.5 × .17 = .2
filler say = .2

14.0 cubic ft per lin. ft. @150 = 2100#
HR 60
stringers 230
2390

2390 ÷ 21.67 = say 110#/'

For Approximation take 110#/' uniform load throughout the span

Dead load   110 × 22.42 = 2470 $m = \frac{1}{8} \times 2585 \times 23^2 = 171,000$ '#
DL girder 115
2585

Live Load Uniform $\frac{102.5 \times 18.42^2 \times 10}{2 \times 22.42}$ = 777# $m = \frac{1}{8} \times 777 \times 23^2 = 51400$ '#

10.42   3300   17.0   8800

23.42

3300 × $\frac{10.42}{22.42}$ = 1530
8800
10330

4.5 6 2 6 4.5

23.

20660 × 10.5 = 217,000
10330 × 6 = 62,000
155,000 — 155,000

neglect uniform load on side

Summary   377,400 '#

Try 30 × 5/16 web   9.360"   ⅛ web = 1.170"
30½" back to back of Ls   Effective d = 25.4 - 1.5 = 23.9   S = $\frac{377,400}{23.9}$ = 158,000#
SR = 15800 ÷ 16000 = 9.87 - 1.17 = 8.70 □" net
Try 2Ls 5 × 3½ × 3/8 = 8.00 - 1.17   6.0
10½ × 3/8   3.93   3.18
9.18 $23 \times \sqrt{\frac{3.18}{9.11}} + 2 = 15.5$

weight of one intermediate Floor Beam

web	30 × 5/16	@ 31.88	× 22.0	= 700	
Flanges	4Ls 5 × 3½ × ½	@ 13.6	× 22.0	= 1196	2810#/23 = 122#/lin ft.
Cov. Pls.	2Pls. 10½ × 3/8	@ 13.39	× 15.0	= 402	
Stiffs	10Ls 4 × 3 × 5/16	@ 7.2	× 2.5	= 180	
End Stiffs	4Ls 3½ × 3½ × 3/8	@ ~~8.5~~	× 2.4	= 82	
fills	4Pls 3½ × ¼	@ 5.95	× 1.8	= 43	
Shelf Ls	12Ls 4 × 3 × 5/16	@ 7.2	× .83	= 72	
	Rivet heads + variations say			135	
				2810#	

End Floor Beam

Dead load say 110#/' $\frac{110 \times 23.5^2}{2 \times 22.42}$ = 1350# per lin ft

$m = \frac{1}{8} \times 1350 \times 23^2$ = 89300 '#   Add wt of FB

8800 3300
12   2   9.42
22.42

uniform load $\frac{102.5 \times 9.42^2}{2 \times 22.42}$ = 203#

$m = \frac{1}{8} \times 203 \times 23^2$ = 13,400

$m = \frac{1}{8} \times 100 \times 23^2$ = 6600

8800 × 23.42 = 206000
3300 × 11.42 = 37600

243600 ÷ 22.42 = 10850

10850
4.5 6 2 6 4.5

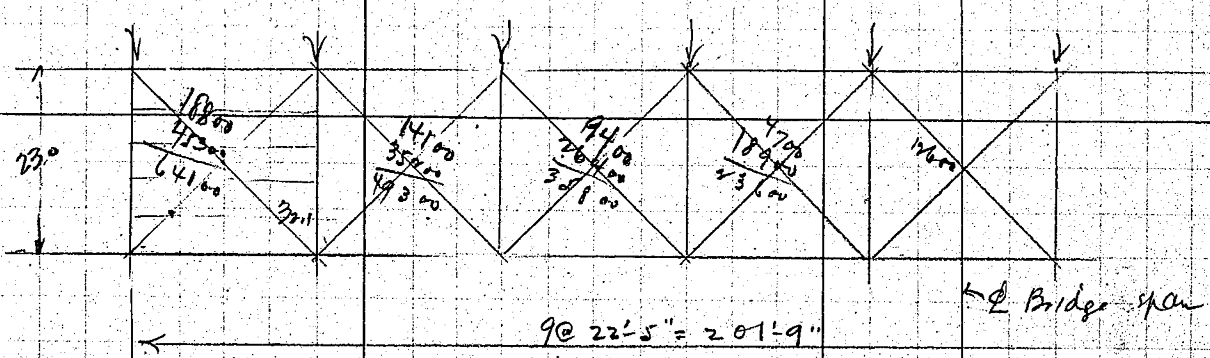
$m = 2 \times 10850 \times 10.5$ = 228000
10850 × 6 = 65000
163000
Total moment   265700 '#
6600
272300

Try 30 × 5/16" web = 9.360" 1/8 web = 1.170"
30½ backs to back of Ls d = 2.39 S = ~~260700~~ 272300 ÷ 2.39 = ~~111000~~ 114000#
SR = ~~111000~~ 114000 ÷ 16000 = 6.94 7.17 less 1.17 = 5.77
Use 2Ls 5 × 3½ × 7/16 = 7.06 or 6.19 net

weight of one 3nd Floor Beam

web	30 × 5/16	@ 31.88	× 22.0	=	700	2211/23 = 96#/lin ft
Flanges	4Ls 5×3½×7/16	@ 12.00	× 22.0	=	1055	
Stiffs	10Ls 4×3×5/16	@ 7.2	× 2.5	=	180	
3nd Stiffs	4Ls 3½×3½×3/8	@ 8.5	× 2.4	=	82	
fills	4 P/s 3½ × 7/16	@ 5.21	× 1.8	=	38	8 @ 2810 = 22500
shelfs	6Ls 4×3×5/16	@ 7.2	× 0.83	=	36	2 @ 2210 = 4420
		Rivet +c say			120	26920 ÷ 201.75 = 134
					2211#	

Bottom Laterals.



sec θ = 32.1/23.0 = 1.400
tan θ = 22.42/23 = 0.975
w₁ tan θ = 0.975 × 3360 = 3280
w₂ tan θ = 0.975 × 8100 = 7900
w₁ sec θ = 4700
w₂ sec θ = 11340

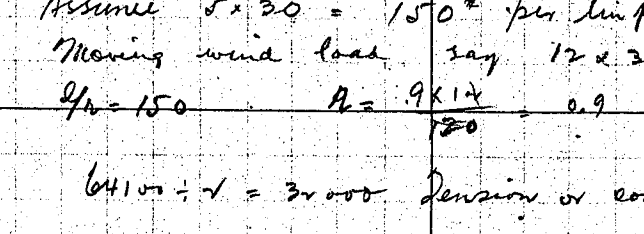
Static wind load 30#/□' of exposed surface
Assume 5 × 30 = 150# per lin ft panel conc = 150 × 22.42 = 3360#
Moving wind load say 12 × 30 = 360# panel conc 360 × 22.42 = say 8100#
l/r = 150 $A = \frac{9 \times 12}{120} = 0.9$

64100 ÷ 2 = 32000 Tension or comp. SR = 2.00" for Tension net
SR = 2.70"
Use 2Ls 3 × 2½ × 5/16 @ 5.6 × 30. = 336.
18 @ 336 = 6050#
Details say 1000
7050#
7050 ÷ 201.75 = 35# per lin ft.

Upper Lateral Bracing.

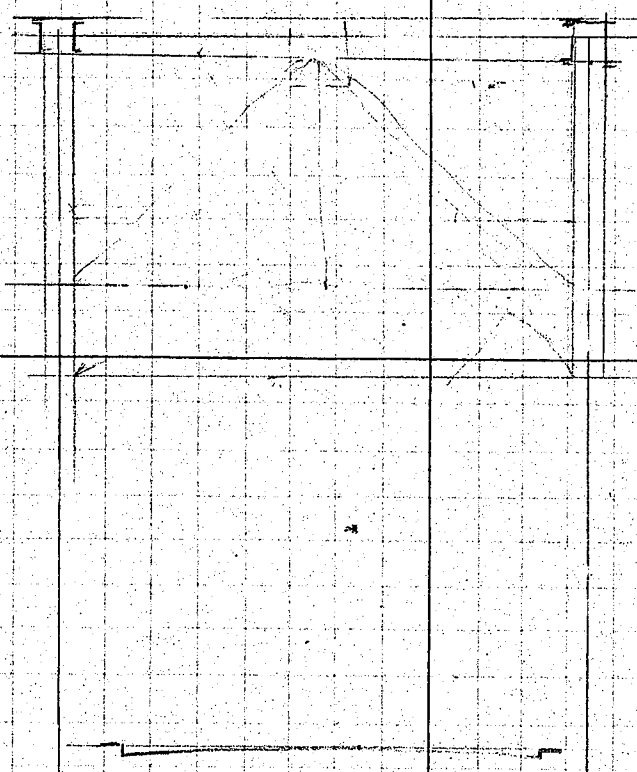
Use 8Ls 3 × 2½ × 5/16 @ 5.6 × 30 = 1350 4Ls 5×3½×5/16 @ 8.2 × 30 = 985.
connection pls say 50 Connection Pls. 25
6 Tie P/s @ 25 150
Single lacing 8# @ 50 400
1950
9 @ 1950 = 17550 ÷ 201.75 = 88#/lin ft.

End, Portal Bracing.



8Ls 4×3×5/16 @ 7.2 × 22.0 = 1272
12Ls 2½×2½×5/16 @ 4.5 × 4 = 216
Connection P/s. 7 @ 50 = 300
Lacing Bars. 400
Tie P/s. 450
2638
call this 2700#
2 @ 2700 = 5400#

Sway Bracing



4Ls 4x3x7/16 @ 7.2 × 21.0 = 605
2Ls do @ 7.2 × 21.0 = 300
4Ls do @ 7.2 × 14.0 = 403
2Ls 2½x2½x5/16 @ 5.10 × 10.0 = 100
6 connection + Tie Pls. @ 30 = 300
Lacing. 8@15 = 120
1828#

8
~~9~~@1828 = ~~16500~~ #
14600

Summary for bracings.
Diagonals 17550
Sway ~~14600~~
Portal 5400
~~39450~~ ÷ 201.75 = ~~196~~# per lin ft.
37550 186

Truss.
Panel concentration.
Lower panel load
2 lines of Handrails 2@30 = 60
Stringers 230
Intermediate Floor Beam 125
Lower Laterals 35
Trusses lower half. 425
875

Roadway pavement 22# x 20 = 440
concrete 75 x 20 = 1500
Fascia 2-1.9 @150 = 570
2510
3385

3385 ÷ 2 = 1693# per truss.
Panel concentration = 1693 x 22.42 = 38000#

Upper panel load.
upper laterals. 196.0
Trusses upper half 425.
621# ÷ 2 = 310
Panel conc. = 310 x 22.42 = say 7000#

Live Load.
Uniform live load. 87.5
For one truss 87.5 x 10 = 875# per ft
panel concentration 875 x 22.42 = 19600#

Motor Truck

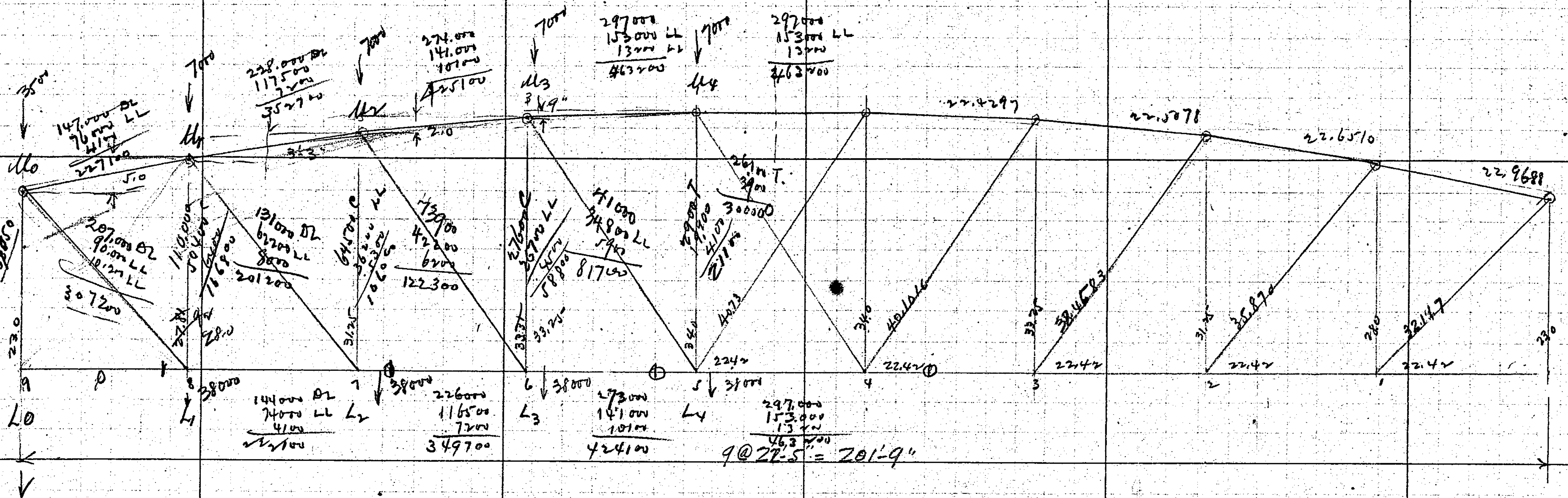
13200
4400
17600 x 1.3 = 22900 or 22900/18x8 = 159#

Distribution assumed 18x8

8x87.5 875
71.5 x 8 = 572# per ft.
Total load 572 x 18 = 10300#

Preliminary design of Furukawa Oashi, Tokushimaken

4.



28.0	×	22.4167 / 5	=	125.8	125.8	×	23 / 32.12	=	90.0	28.00	×	22.9688 / 22.4167	=	27.40
31.25	×	22.4167 / 3.25	=	216.0	216.0	×	28 / 35.87	=	168.5	31.25	×	22.6510 / 22.4167	=	31.00
33.25	×	22.4167 / 2	=	373.0	373.0	×	33.25 / 31.46	=	334.5	33.25	×	22.5078 / 22.4167	=	33.15
34.00	×	22.4167 / .75	=	1016.0	1016.0	×	34.0 / 40.10	=	864.	34.00	×	22.4297 / 22.4167	=	34.02

Reaction - DL. 38000 × 4 = 152000
LL. 19600 × 4 = 78400
DL. 7000 × 4 = 28000
- L.L. motor 5150
Shear ... 263550#
3500
267050

1	1/9	.111	× 19600 =	2180
2	3/9	.333		6520
3	6/9	.666		13040
4	10/9	1.110		21800
5	15/9	1.667		32600
6	21/9	2.333		45700
7	28/9	3.125		61200
8	36/9	4.000		78400

Top chord section. S

U_4-U_4	S = 463200#	$\frac{1}{12630}$ =	35.60"	36.60
U_3-U_4	do	"	35.6	36.60
U_2-U_3	425100	"	33.7	33.80
U_1-U_2	352900	"	27.1	28.00
U_0-U_1	227100	"	17.5	

$ps = 16000 - 70\,l/r = 12630$ $\frac{273}{3.67}$

$\frac{3370}{12630}$

U_3U_4	1 cov Pl. 20 × 1/2 2 [s 45#	36.48 0"
U_2-U_3	1 cov Pl. 20 × 1/2 2 [s 40#	33.52 0"
U_1-U_2	1 cov. Pl. 20 × ~~3/8~~ 1/2 7/16 2 [s 33#	29.80 ~~27.50~~
U_0-U_1	1 cov Pl. 20 × 3/8 2 [s 33#	27.20

Bottom chord.

463200	3K = 28.9 nw	2-40# -
424100	26.5 nw	
349700	21.8	
222100	13.9	

2-45 [s - 2 Pls. 12 × 3/8	36.48 9.00 35.48	21.23 7.5 28.73	4 on flange + 4 on web.
2 [s 40# 2 Pls. 12 × 3/8	23.50 9.00 32.50	19.00 7.5 26.52	15.76 7.5 23.26
2 [s 45#	~~23.5~~ 25	21.00	19.8 10.5 — 15.76 8.75 26.51
2 [s 33#	19.8 2.5	15.76	

U_0-L_1	307200 T		19.20" nw	2 [s 40# — 21.02	
U_1-L_2	201200		12.6	4 Ls 5 × 3 1/2 × 3/8 12.2 1 web 12 × 3/8 4.5 16.7	9.20 3.75 12.95
U_2-L_3	122300		7.6	4 Ls 5 × 3 1/2 × 3/8 — 12.2	9.2
U_3-L_4	81700		5.1	4 Ls 4 × 3 × 5/16 — 8.30	5.86
U_4-L_4	30000		1.9	4 Ls 4 × 3 × 5/16 — 8.26	5.86
U_1-L_1	166800	15.5	16000 - 70 l/r 5270 10730	$\frac{336}{4.41}$ 2 [s 12 × 30.#	17.64
U_2-L_2	106000	9.85	16000	$\frac{375}{3.61}$ 2 [s ~~10 × 25#~~	17
U_3-L_3				2 [s 10 × 20#	
U_4-L_4				2 [s 10 × 20#	

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

U0-L0- 2[33# + 1 cov pl. 20 × 3/8.

Approximate weight of truss.

member						
L0U0	27.30	× 3.40	93 ×	23.0	=	2140
U0-U1	27.30	"	93	23.0	=	2140
U1-U2	29.80	"	101.4	22.6	=	2300
U2-U3	33.52	"	114	22.5	=	2560
U3-U4	36.48	"	125	22.4	=	2800
U4	36.48	"	125	11.2	=	1400
L0-L1		66#		22.4	=	1480
L1-L2		66#		22.4	=	1480
L2-L3		90#		22.4	=	2020
L3-L4	34.52	× 3.40	111	22.4	=	2480
L4	35.48	"	121.	11.2	=	1350
U0-L1		80#		32.0	=	2560
U1-L2	16.7	× 3.4	57.0	35.0	=	2000
U2-L3	12.2	"	41.5	38.0	=	1580
U3-L4	8.36	"	28.5	40.0	=	1140
U4-	8.36	"	28.5	40.0	=	1140
U1 L1		60#		28.0	=	1680
U2-L2		50		31.0	=	1550
U3-L3		40		33.0	=	1320
U4-L4		40		34.0	=	1360

36,480 × 4 = 146,000
35% 51,000
197,000 ÷ 201.75 = 980#

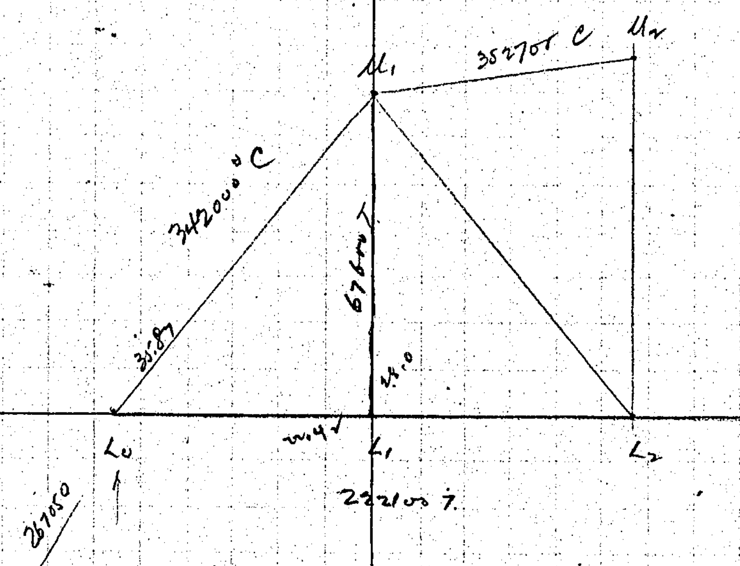
Approximate weight of steel in span

Handrails.	60.	
Stringers	230.	364
FB.	134	
upper lateral	186	
Lower Lateral	35	
Trusses.	980	
On piers.	50	
	1675	

1675 × 201.75 = 338,000 151 tons per truss

151 × 17 = 2567 tons.

15
13.2
1.8



U1-L1. 67600 att.
4L5 4×3×5/16 = 8.32 5.860" net

L0-U1 1 cov Pl. 20×1/2
2[5 15" 45° } 36.48

weight of truss.

L0-U1	36.48	× 3.40	× 35.0 =	4370
U1-U2				2300
U2-U3				2560
U3-U4				2800
U4				1400
L0-L1				1480
L1-L2				1480
L2-L3				2020
L3-L4				2480
L4				1350
U1-L2				2000
U2-L3				1580
U3-L4				1140
U4				1140
U1-L1	28.5#		× 28	800
U2-L2				1550
U3-L3				1320
U4-L4				1360

7730
1480
9210

33130 × 4 = 132000
35% 46500
179500 ÷ 201.75 = 890#

Upper Lateral - Diag 7 @ 1950 = 13650
Portal 2 @ 2700 = 5400
Sways 6 @ 1828 = 11000
30050 ÷ 201.75 = 149#

Approximate weight of Steel in span

Handrails	60
Stringers	230
F.B.	134
Upper laterals	149
Lower Laterals	35
Trusses	890
On piers	50

1548 × 201.75 = 313000 # or 140 tons.

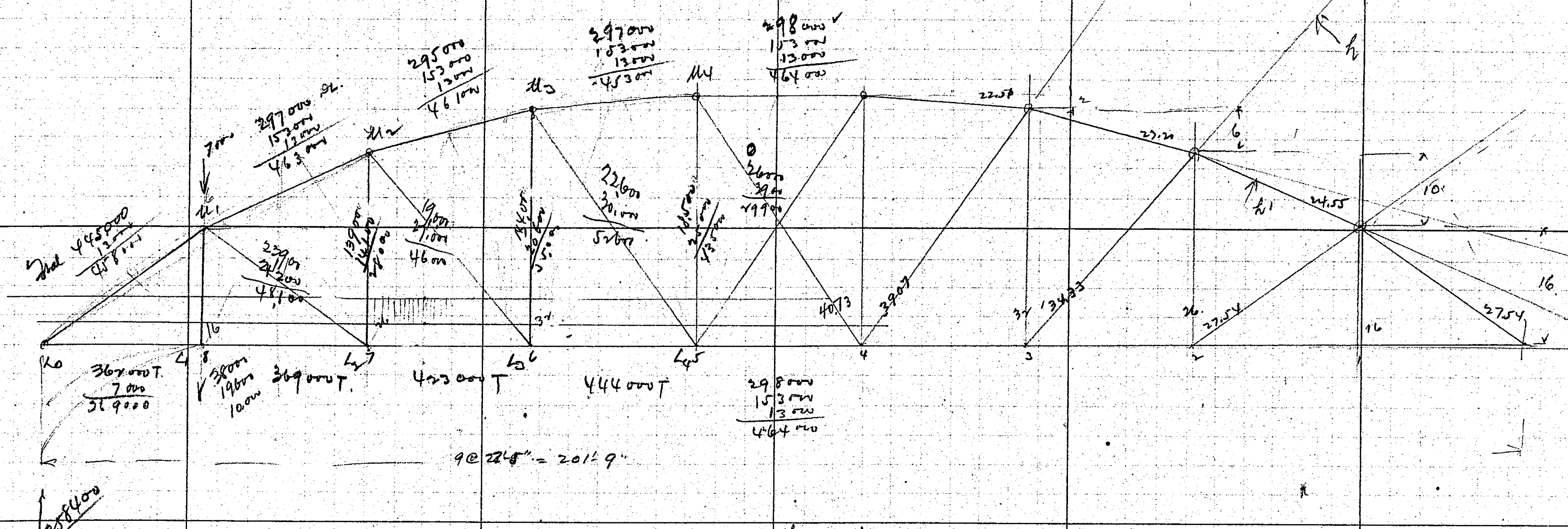
140 × 17 = 2380. (2567)

187 @ 260 = 50,000 yen

187 @ 350 = 65000 "

Preliminary Design of Furukawa Bashi, Tokushimaken 18.

Let us estimate the Bowstring type for this span and see how we can reduce the weight.



$26.0 \times \frac{22.42}{10} = 58.3$	$S = 13.46$	$58.3 \times \frac{16}{27.54} = 33.9$	$26.0 \times \frac{22.42}{24.55} = 23.8$
$34.0 \times \frac{22.43}{6} = 120.0$	$S = 52.74$	$120 \times \frac{21}{34.33} = 91.0$	$32.0 \times \frac{22.42}{23.00} = 30.8$
$34.0 \times \frac{22.42}{2} = 381.0$	$S = 291.3$	$381.0 \times \frac{32}{39.07} = 312.0$	$34.0 \times \frac{22.42}{22.51} = 33.9$

Reaction DL 38000 × 4 = 152,000
DL 7000 × 4 = 28,000
LL 19600 × 4 = 78400
258400

Preliminary Design of Furukawa Bashi, Tokushimaken

Top chord section.
16000 - 70 L/r
4070
11930
Sk. 38.4 □"

330 / 5.67 =

Try 1 cov Pl. 22 × 1/4, 2 [s 45# } 37.6 □" gross.

Bottom chord
465,200 Sk. 28.9 net
369,000 23.0 net

2 [s 45# - 26.48 21.20
2 Pls. 12 × 3/8 9.00 7.50
35.48 28.70

2 [s 45# 26.49 23.98 -

M1 L1. 4 Ls 4 × 3 × 5/16 = 8.32
12 × 5/16 = 3.75
12.07

weight of Truss.

L0-U1	37.6 × 3.40	128 × 27.5	=	3520
U1-U2	"	24.5	=	3130
U2-U3	"	23.2	=	2970
U3-U4	"	22.5	=	2880
U4	"	11.2	=	1430
L0-L2	90#	44.8	=	4040
L2-L3 –	121	56.0	=	6800
U1 L1	40	16.0	=	640
U2-L2	40	26.0	=	1040
U3-L3	40	32.0	=	1280
U4-L4	40	34.0	=	1360
U1-L2	8.32 @ 3.40 = 28.5	27.5	=	785
U2-L3		34.0	=	970
U3-L4		39.0	=	1110
U4--		40.0	=	1140

33090 × 4 = 132500
35%. 46500
179000 ÷ 201.75 = 890#

upper lateral Diag. 5 @ 1950 = 9750
2 @ 2700 = 5400
4 @ 1828 = 7300
22450 ÷ 201.75 = 112# per lin ft.

Approx weight of Steel in span

Handrails.	60
Stringers	220
F.B.	134
upper laterals.	112
Lower.	35
Trusses.	890
On line.	50
	1511

1511 × 201.75 = 304000 136 tons.

136 tons × 17 = 2312 tons.

2567
2312
255 @ 260 =

2380
2312
68 @ 260 = 18000

¥ 66,000

Estimate of Cost superstructure.

Floor slab. 0.5 × 20 = 10.0
Fascia girder 2 @ 1.9 = 3.8
13.8
.2
14.0 cubic ft per ft.

$\frac{14.0 \times 3470}{216}$ = 225 tsubo

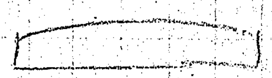
Total length.
201'-9" × 17 =
2'-3" × 17 = } call this - 3470 ft.

Reinforcing steel in floor slab.

$\frac{12}{15}$ 4 - ½"φ	@ .67	× 23.5 =	50.5	
20 - ½	@ .67	× 1.0 =	13.4	
14 - ¾"	@ 1.5	× 1.0 =	21.0	
2 - ⅜	@ 3.8	× 5.5 =	4.2	
			89.1	call this 90# per lin ft

$\frac{90 \times 3470}{2240}$ = 140 tons   140 ÷ 225 = .62 tons per cubic tsubo.

wood block pavement $\frac{20 \times 3470}{36}$ = 1930 tsubo in ft.

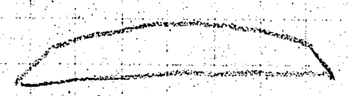


Estimate of cost. #1.

Structural steel. furnished	2567 tons	@ 340.00	=	872,500.00	
Deck: Reserved for variation of market price of steel		15.00		38500.00	
Deck: Concrete in slab	225 tsubo	@ 155.00		34900.00	115150
Deck: forms.	2500 @ 3	5.00		7500.00	
Deck: finish of curb	3500 ft	@ 1.50		5250.00	
Deck: wood block pavement with cushion -	1930	@ 35.00		67500.00	
				1,026,150.00	

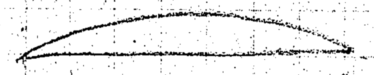
Estimate of cost #2

Structural steel	2380 tons	@ 340	=	810,000.00
	do.	15	=	35700.00
	Deck.			115150
				960850.00



Estimate of cost #3

	2312 tons	@ 340	=	785.000
	do	15		34700
	Deck			115150
				934850.00



102
90.000

1930
1600
700
+ 6.4

Load on shoe

Structural steel per span 338000

Roadway pavement $22^{\#} \times 20 = 440$

concrete slab. $75 \times 20 = 1500$

Fascia $2\text{-}1.9$ @ $150 = \underline{570}$

$2510 \times 2030 = \underline{510.000}$

	848.000
Live Load. $20 \times 70^{\#} \times \cdot 203 =$	384.000
Extra motor truck	20.000
	1,152,000 # on pier.

Load on shoe. $848000 \cdot 44 = 212000$

$10 \times 87.5 \times \frac{203}{2} = \underline{89000}$

$301000^{\#}$

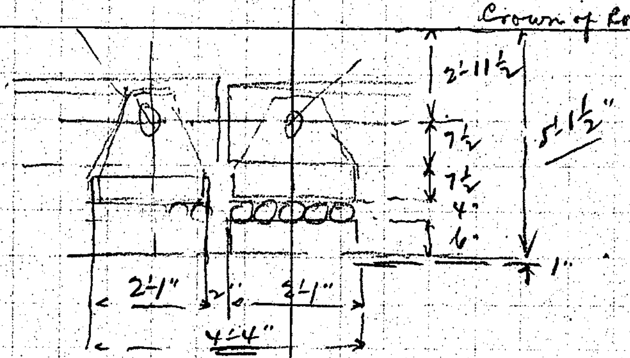
Bearing on masonry $25" \times 36 = 900 \square"$

Unit bearing pressure $= 301.000 \div 900 = 333\#/\square"$ O.K. on masonry

Length of roller $600d = 2400^{\#}$ per lin inch.

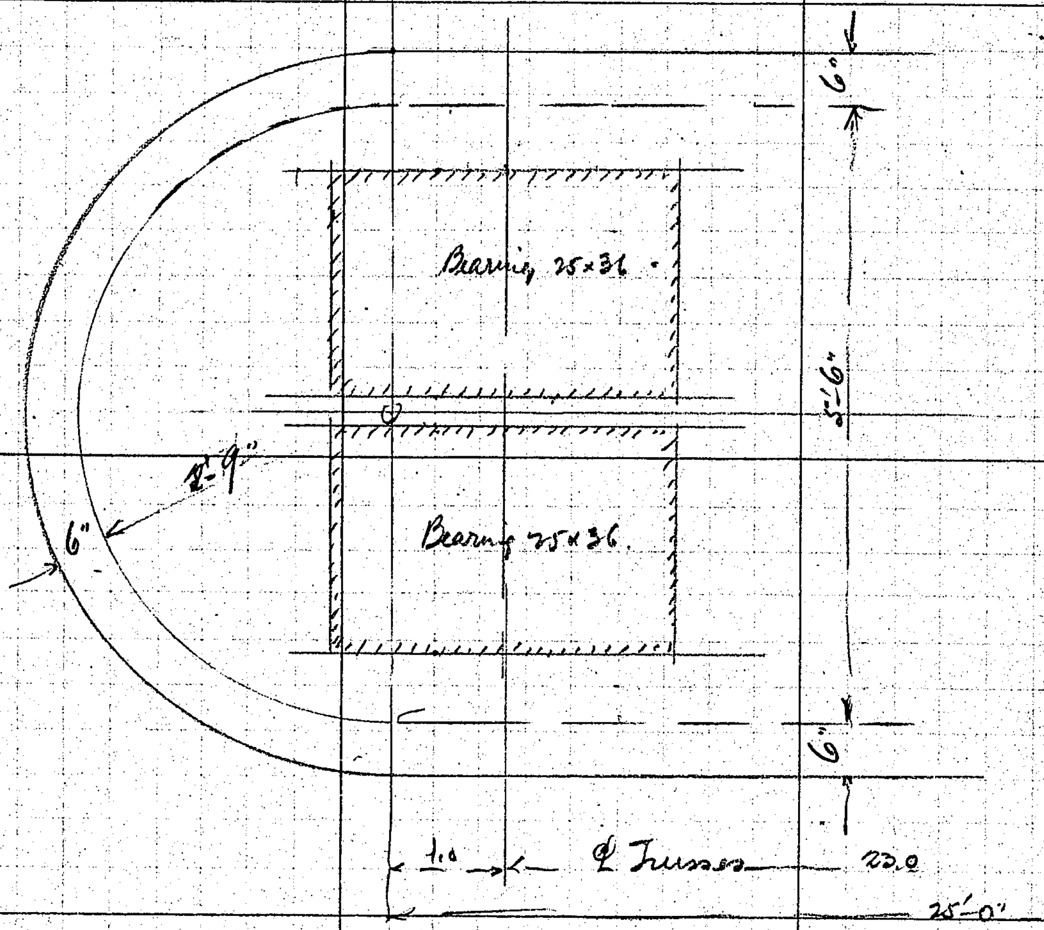
$301.000 \div 2400 = 126"$

Use 5 rollers @ $25" = 125"$ net



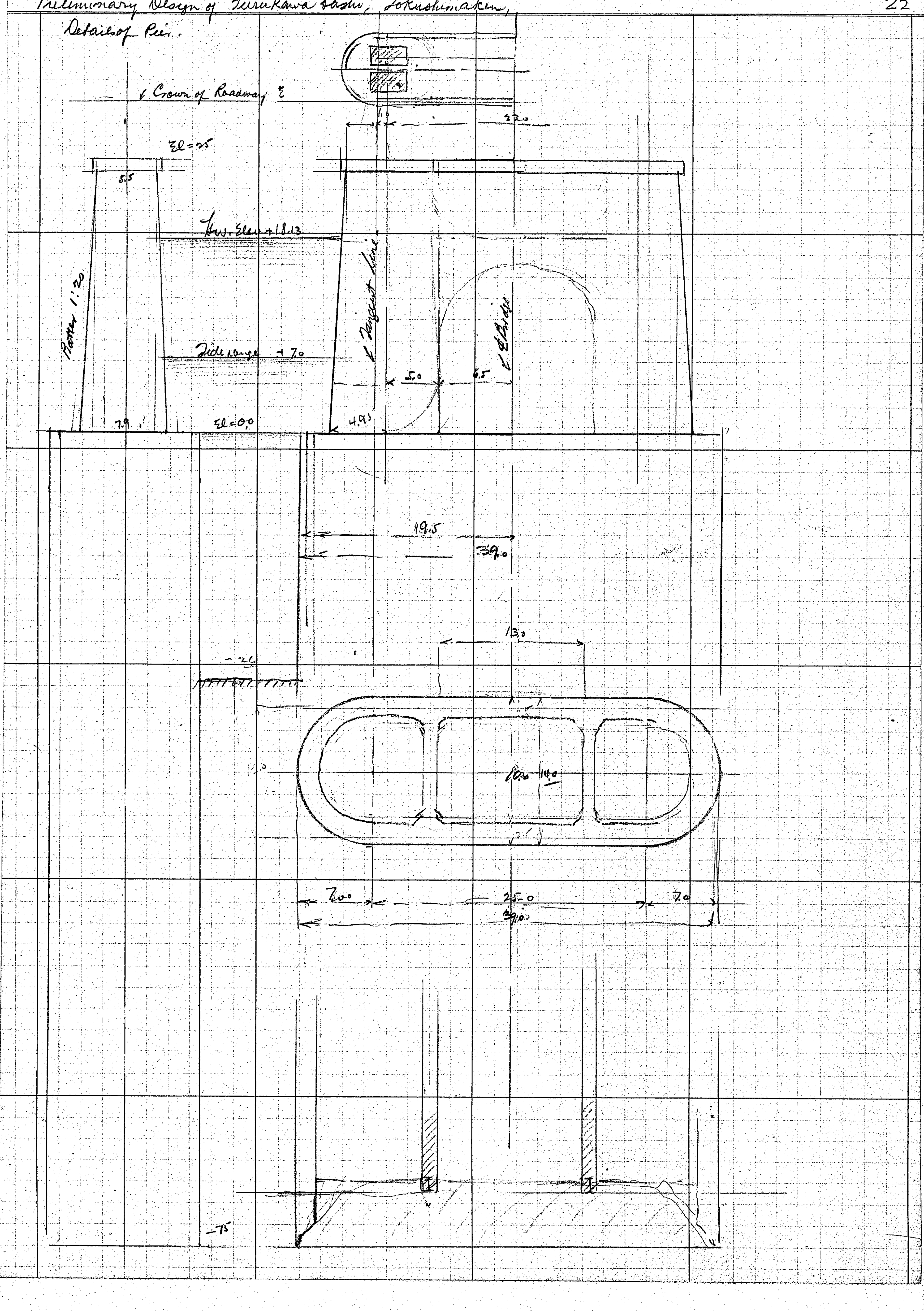
Average height of Roadway say +30.0

Top of pier say 25.00



Preliminary Design of Furukawa-bashi, Tokushimaken,

Details of Pier.



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Approximate Design of well.
cohesive strength of material to be excavated Assumed 40#
Frictional resistance assumed 400#/□' or $\frac{400}{40}$ = 10'
Assume weight of earth 100#
distributed friction say 40
140

Design well for earth pressure of 140# & trench width of 10'.

Pressure at 80' — 800#/□' × 1.40 = 1120#

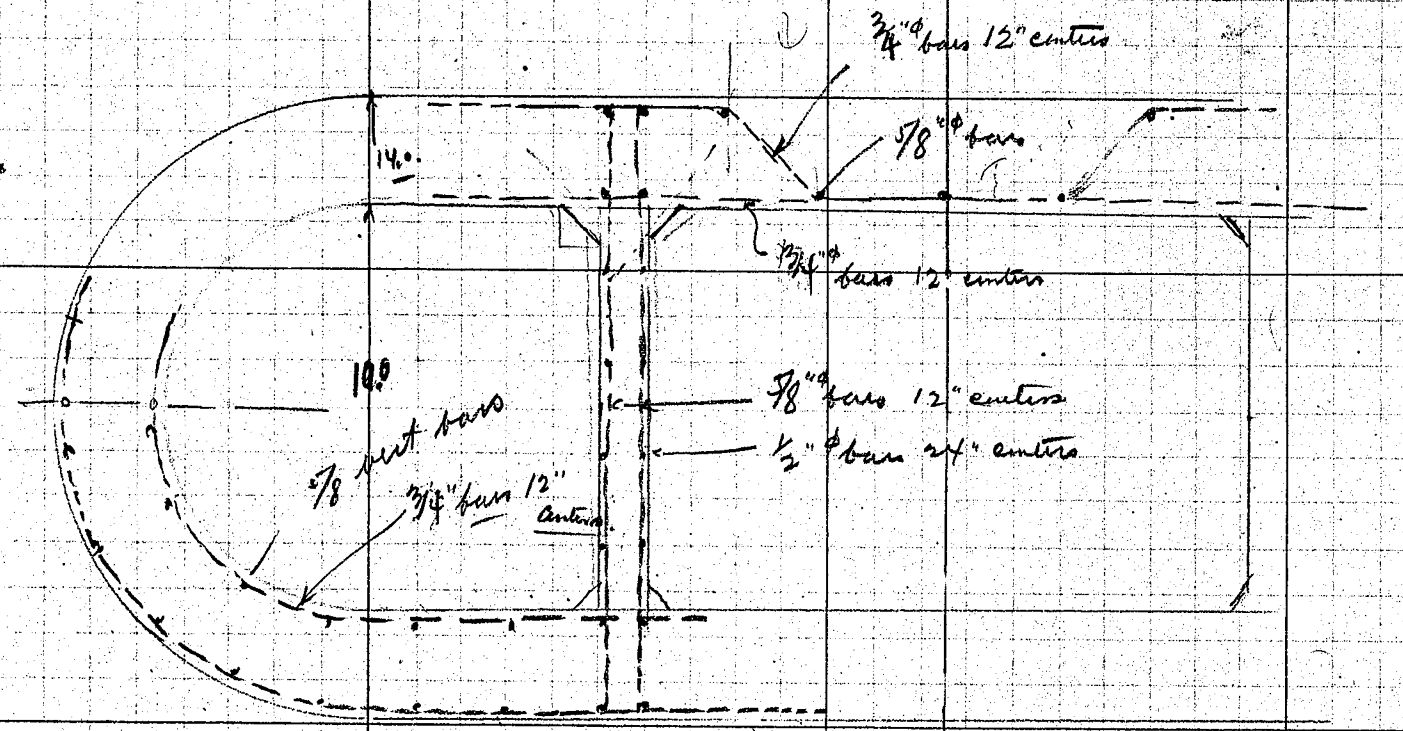
water pressure at 32 deep × 62.5 = 2000#.
Design wall for 2000# pressure.
middle section. 14.0 $m = \frac{1}{12} \times 2000 \times 14^2$ = 18300 1#
Depth required for fs=1600 fc=600 $\sqrt{\frac{18300}{95}}$ = 19.3"

make thickness 2'-6". Reinforcement = $\frac{18300 \times 12}{\frac{7}{8} \times 27 \times 16000}$ = .75 □ 1.0".

Use $\frac{3}{4}$"φ bars 6" centers.

Reinforcement at end of pier.
$m = \frac{1}{16} \times 2000 \times 11.5^2$ = 16500

Reinf. = $\frac{16500 \times 12}{\frac{7}{8} \times 27 \times 16000}$ = 0.525 $\frac{7}{8}$"φ bars 1'0" centers.



Concrete in one ft strip.

Ring 14 154
9 78
76 — 76
25 × 4 = 100
2-10×1. 20
4
200 ÷ 21.6 = 9.25 ÷ 12

Reinforcing bars.

4 —	3/4 @	1.50	× 22.0	=	130	
4 —	"	1.50	× 22.0	=	130	
4 —	"		× 16.0	=	96	
4 —	5/8	1.04	× 14.0	=	58	
70 —	5/8	1.04	× 1.0	=	73	
					491	per ft.
					Call this 500#	

69 @ 9.25 = 640 + 7 Reinf. 75 @ 500 = 37500 = 16.75 1. call 17 tons

Area at bottom = 25 × 14 = 350
14φ 154
504 × 6 / 6 × 36 = 14. tons.

Filling of concrete in base. up + down stream ends.

Area $= 10^{\phi}$ 78.5
2 - 5 × 10 100
178.5 ÷ 216 = .82 tsubo × 30 = 25.0 tsubo.

volume of shafts.
coping - 6.5^{ϕ} = 33.
6.5 × 25 = 163.
196 ÷ 216 = .90 tsubo.

Shaft 5.5^{ϕ} = 23.7
7.9^{ϕ} = 49.0
72.7 ÷ 2 = 36.3

5.5
7.9
13.4 ÷ 2 = 6.7 × 25 168.0
$204.3 \times \frac{25}{216}$ = 23.6 tsubo.

Total concrete in shaft 23.6
.9
24.5 tsubo.

concrete filling 1:3:6. 25.0 tsubo
Concrete in base. 64.0
14.0
78.0

44
50

Total. 127.5 tsubo.

Reinforcing Bars. 17 tons.

Structural Steel in cutting Edges

1 - 30 × 3/8 @ 38.25 38.25 145
2 - 36 × 3/8 @ 45.9 45.9
1 - 12 × 3/4 @ 30.6 30.6
Diaphragm +c 28.0
139.75 call this 140# per ft

15
3
18

circumference of pier. 94 × 140 = 13200# say 6 tons.

Excavation. $\frac{504}{216}$ = 2.34 tsubo per ft.

2.34 × 75 = 176 T.

Estimate of cost

concrete cement 1.1 @ 9.00 = 99.00	66	Concrete	127.5 @ 159.00	=	20300	
sand 0.5 = 8		Reinf.	17 tons @ 190.00	=	3230	
1.0 25.00		Curb shoe.	6 tons @ 300.00	=	1800	
Labor for mixing etc 27.00	126	excavating.	176 T @ 75.00	=	13200	+ 450
159.00					38530	

16 @ 38530 616.000 16 @ 50000 = 800.000

2 abutments 30.000 30.000
Misc. 50.000 50.000
696.000 call this 700.000 880.000
83.0
18 100.000
150.000
1950.000
175

weight of one cubic tsubo 216@150 = 32400 #

Total weight of pier. 127.5 × 32400 = 4,130,000 #

Superimposed load. = 1152,000

5,282,000 #

Area of bottom of base = 504

Friction. Circumference of base 94 × 400 × 75 = 2820,000 lbs

2,462,000 ÷ 504 = 4900 #/□' or 2.2 tons/□' ok

5,282000

1410,000

3872,000 ÷ 504 = 7700 3.43 tons

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40 - 150 60/2
60 - 90/2

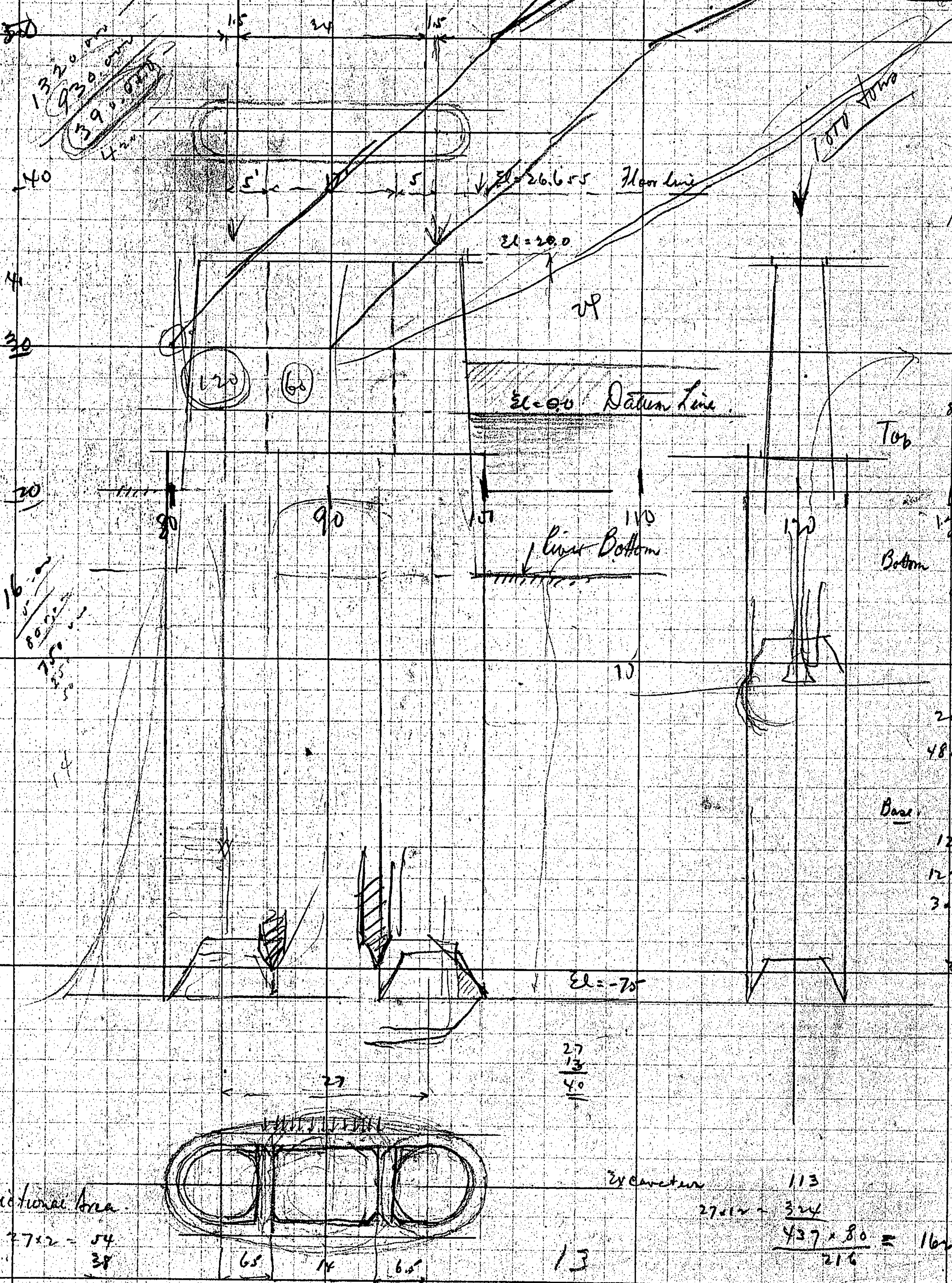
160

3.6 216

300 1600
1.00 20

204.5' span center to center of piers.

Dead Load 120 × 240 = 2880 #
metal 1700
4580
Live Load 2000
6580 # × 207 = 1360000 # on pier.
600 tons



volume of coping
27 × 7 = 189
38
227 × 1.0 = 227
227 ÷ 216 = 1.05 ㎥

Shaft
Top 6' φ = 28.
6 × 80 = 60
3 × 17 = 51
139
Bottom 9' φ = 64
9 × 10 = 90
6 × 17 = 102
256
395 ÷ 2 = say 200
200 × 24 = 4800 cu. ft.
4800 @ 216 = 22.2 ㎥

Base
12 φ = 113
12 × 13 = 156
3 × 14 = 42
311 × 70 = 21800
21800 ÷ 216 = 101.
10
111. ㎥
23.0
134 ㎥

steel say 15 tons
37.5

Frictional Area
27 × 2 = 54
38
92 × 55 = 5060
5060 × 400 = 2024,000

Excavation
113
27 × 12 = 324
437 × 80 = 162 ㎥
216

13

Estimate
134 ㎥ @ 150 = 20100
15 tons @ 200 = 3000.
cutting Edges 2000
162 ㎥ @ 100 = 16200
41300

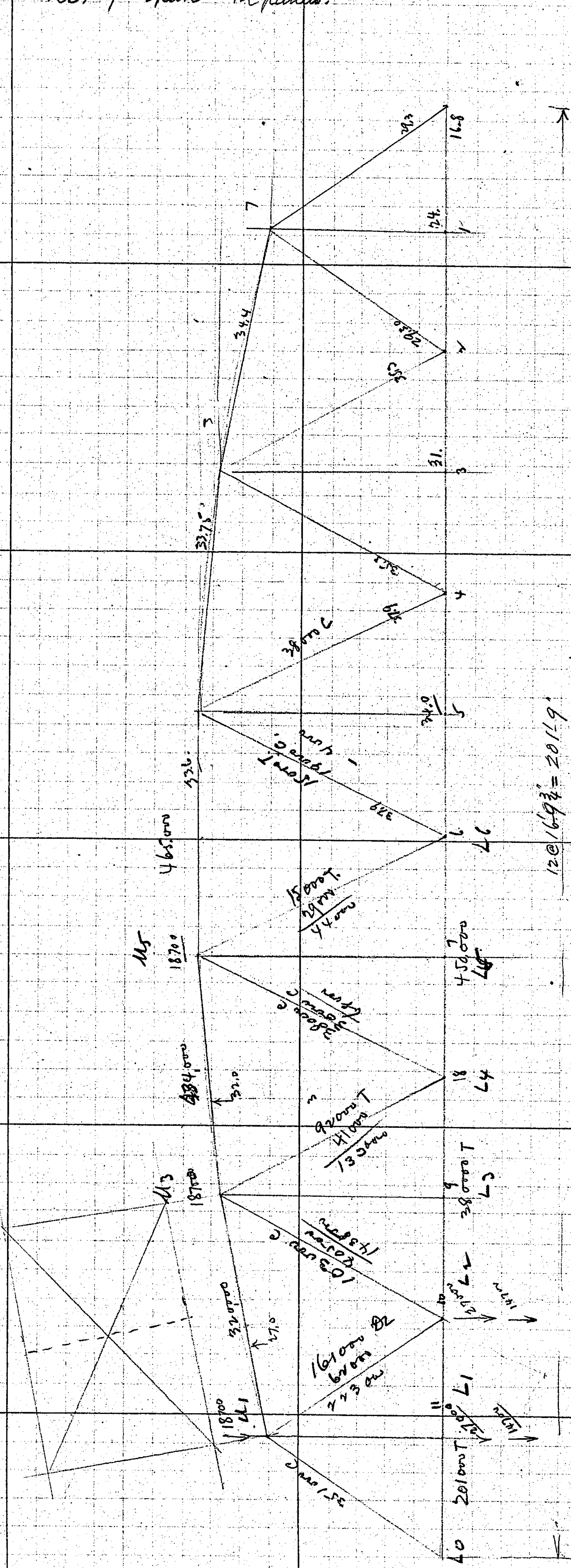
134 × 216.0 = 29,000 @ 150 = 4350,000 #
1360,000
5,710,000 = 13100 #/□' 5.85 tons/□'
437

5,710,000
2024,000
3,686,000 = 8440 #/□' or 3.76 tons/□'
437

143
139.00
1400

Preliminary Design of Furukawa Bashi, Tokushimaken.

201'-9" span 12 panels.



Floor Slabs same as other design.
6" thick.

Stringer span length 16.8'

DL 95 × 4.17 = 396
stringer say 44
440#

$m = \frac{1}{8} \times 440 \times 16.8^2 = 15,500$ '#

Live Load unif. 102.5 × 4.21 = 432#

8.4 — 4 — 4.4

16.8

Reaction $\frac{432 \times 4.42}{2 \times 16.8} = 250$

$m = 250 \times 8.4 = 2100$ '#

Rear wheel $8800 \times \frac{2.17}{4.17} = 4580$
8800
13380

moment $= \frac{13380}{2} \times 8.4 = 56,100$
2100
15,500
73,700 '#

$Sm = \frac{73,700 \times 12}{16000} = 55.2$

Use 15" × 42# I.

weight of ~~stringer~~ 42#
connection 3
45#

4 @ 45# = 180# per lin ft

Fascia girder.
Assumed dead load
Handrail 30
Beam say 260
$95 \times \frac{3.75}{2}$ 178
468#

$DL\ m = \frac{1}{10} \times 468 \times 16.8^2 = 13,200$ '#

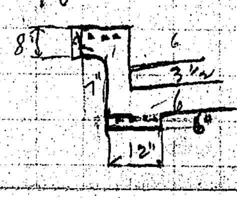
Live Load see pp 10
unif. load 175#
Reaction $= \frac{175 \times 4.4^2}{2 \times 16.8} = 101$#

$m = 101 \times 8.4 = 850$#

wheel load. $\frac{6970}{2} \times 8.4 = 29,300$
For continuity 29300 × 0.8 = 23400

Summary for moment
DL 13200
LL unif. 850
LL wheel 23400
37450 '#

Effective d $= \sqrt{\frac{37450}{107}} = \sqrt{350} = 18.7$"



make depth 21½

0.5 × .67 = .33
0.5 × 1.0 = .50
.58 .64 = .36
1.19 @ 150 = 180#

Steel Area reqd = $\frac{37450 \times 12}{\frac{7}{8} \times 19 \times 16000}$ = 1.69 use 4-3/4"φ bars 1.760" ok.

Intermediate Floor Beam

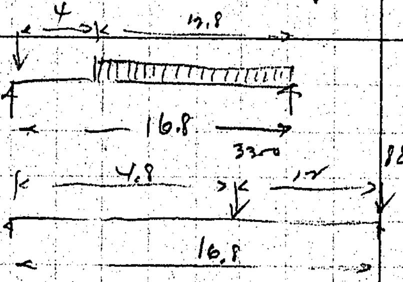
DL uniform say 110#/' throughout the span

110 × 16.8 = 1850 m = 1/8 × 1970 × 23² = 130,000 — 130,000

120

1970

Live Load - uniform load $\frac{102.5 \times 12.8^2}{2 \times 16.8}$ = 500 m = 1/8 × 500 × 23² = 33000



3300 × $\frac{4.8}{16.8}$ = 950

8800

9750#

19500

19500 × 10.5 = 205000

9750 × 6 = 58500

147500

310500#

Try 30 × 5/16 web = 9.360" 1/8 web = 1.17

30 3/4 back to back Ls d = 29 S = $\frac{310500}{2.39}$ = 130000 Sk = $\frac{8.12}{1.17}$ = 700 ur

Use 2Ls 5 × 3½ × ½ — 8.00 or 7.00 ur

weight of one intermediate Floor Beam

web 30 × 5/16 @ 31.88 × 22.0 = 700 2400 ÷ 23 = 105#

Flanges 4Ls 5 × 3½ × ½ @ 13.6 × 22.0 = 1196

Stiffs 10Ls 4 × 3 × 5/16 @ 7.2 × 2.5 = 180 2400 ÷ 16.8 = 143#

End Stiffs 4Ls 3½ × 3½ × 3/8 @ 8.5 × 2.4 = 82

fills 4 Pls 3½ × ½ @ 5.95 × 1.8 = 43

Shelfs 12Ls 4 × 3 × 5/16 @ 7.2 × .83 = 72

Rivit + @ 120

2393 2400#

For end use 4Ls 5 × 3½ × 3/8 @ 10.4 × 22 = 915 (1200, 285)

End Floor Beam say 2100 11 @ 2400 = 26400

2 @ 2100 = 4200

30600 ÷ 201.75 = 152#

Stringer 180

332

Lower Lateral Bracing

$\sqrt{23^2 + 16.8^2}$ = 28.5 wind load 150 × 200 = 30000

360 × 200 = 72000

102000 ÷ 2 = 51000

D = 51000 × $\frac{28.5}{23}$ = 63200#

Use 2Ls 3 × 2½ × 5/16 @ 5.6 × 27.0 = 300

24 @ 300 = 7200

Details 1500

8700 ÷ say 45#

Upper Lateral Bracing

length 20' l/r = 150 r = $\frac{240}{150}$ = 1.6

Use 8Ls 3 × 2½ × 5/16 @ 5.6 @ 40 = 1800 5 @ 2660 = 13300#

con pls 100

8 Tie Pls 25 200

single lacing 70 @ 8 560

2660# per panel

Upper Laterals.
Diag — 5@2660 = 13300
Portal 2@2700 = 5400
Sway 4@1900 = 7600
26300 ÷ 201.75 = 130# per lin ft.

Truss
Panel concentrations
Lower Panel
2 line of Handrails 2@30 = 60
Stringer 180
F.B. 143
Lower Laterals 45
Trusses Lower half 425
853# — 853
Roadway pavement 22×20 = 440
Concrete 75×20 = 1500
Fascia 200@2 = 400
2340
3193

Upper Panel Point.
Upper laterals 130
trusses 425
555 ÷ 2
555 × 16.8 × 2 = 18700 — 9350

Live load
Unif. LL. 87.5
For one truss 87.5 × 10 = 875
Panel conc = 875 × 16.8 = 14700#
Extra load of motor trucks over uniform load say 10,000#

3200 ÷ 2 = 1600# per truss.
Panelconc = 1600 × 16.8 = say 27000#

31.0 × 33.6/7 = 149.0 — 50.4 = 99
50
34.0 × 33.6/3 = 381.0 — 84.0 = 297

132.6 × 24/29.30 = 108.5
155 × 31/35.3 = 136
364 × 31/35.3 = 320
382 × 24/27.9 = 343.

Reaction 27000 × 5.5 = 148500
14700 × 5.5 = 80800
} 229300
18700 × 3 = (56100)
9350
285400

1	1	0.083	× 14700
2	3	0.25	
3	6	.50	
4	10	.833	= 12300
5	15	1.25	17000
6	21	1.75	25900
7	28	2.33	34600
8	36	3.00	44400
9	45	3.75	55500
10	55	4.58	67700
11	66	5.50	81500

Top chord section —
486000 40.500
434.000 37.8
320.000 27.8

16000 — 70 l/r
4500
11500

3.44 × 2 / 6.26

2Pls 16 × 3/8
20 × 7/16
2Ls 3½ × 3½ × 7/8
2Ls 5 × 3½ × 7/8
} 31.81

Add 2Pls 9 × 3/8 — 6.75 Use 16 × 7/16 pl.
3 3.81
38.56

40.56

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Preliminary Design of Furukawabashi, Tokushimaken

Bottom chord section

450,000	÷ 16000 =	28.0 □" req	2[45# [	12" × 3/8	35.48	28.73
380,000		23.7	2[40 [s	~~2-~~12 × ~~5/16~~ 3/8	32.52	26.52
201,000		12.6	2[33"	~~2[~~	19.8	15.76

Web

227000	14.0 □"	2Ls 5×3 1/2 × 3/8	12.2	9.20
133000	8.3	1 web 12 × ~~5/16~~ 1/2	~~6.05~~ 18.2	~~5.0~~ ~~14.6~~
44000	2.8	2Ls 5×3 1/2 × 3/8	12.2	9.2
		4×3 × 5/16	~~8.36~~	5.86
143500	16.0	2[12 × 30#	~~14.70~~ 17.40	
68000	7.5	2[10" × 20#	11.76	

37.9 × 12 / 44.3

All hangers 2Ls 4×3×5/16 8.36 5.86 net

weight of truss

L_0M_1	~~31.81~~ 38.56	@ 3.40	131	× 29.3	=	3840
M_1-M_3	31.81		108	34.4	=	3720
M_3-M_5	38.56		131	33.75	=	4400
M_5-	40.56		138	16.8	=	2320
L_0-L_2	19.8		67.2	33.6	=	2260
L_2-L_4	32.52		111	33.6	=	3720
L_4-L_6	35.48		121	33.6	=	4070
M_1-L_2	18.2	60#		29.3	=	1760
L_2-M_3	〃	60#		35.3	=	2120
M_3-L_4	~~14.7~~	40		35.3	=	1420
L_4-M_5		40		37.9	=	1520
M_5-L_6	~~12.2~~	40		37.9	=	1520
M_1L_1	8.36	@3.4 28.5		24	=	690
M_3L_3		〃		31	=	880
M_5-L_5		〃		34	=	970

10000

35210 × 4 = 141000
49300
190300 ÷ 201.75 = 945

Approx weight of steel in span

Handrail*	60
Stringers	180
Floor Beam	152
Upper lateral	120
Lower Lateral	45
Trusses	945
On piers	50
	1562

1562 × 201.75 = 315,000 141 tons.

141 × 17 = 2400 tons.

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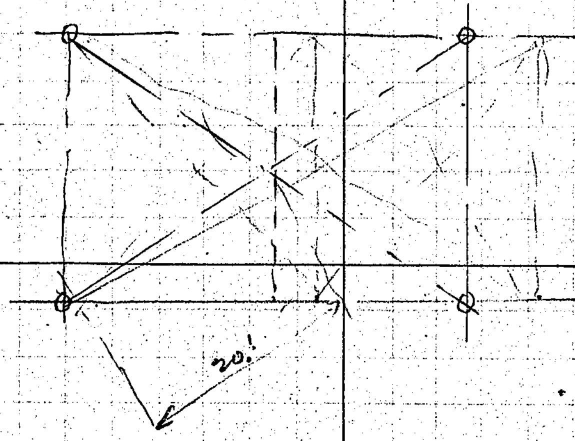
Revised design -
Upper lateral Bracing.



2Ls 3.3.5/16 @ 6.1 × 23.0 = 280
1 @ 8# 130
Tie Pls - 70
480# × 5

5 @ 480 = 2400 7 1200 per truss

4Ls 4.3.5/16 @ 7.2 × 31.0 = 900# × 6 5400
6600#
33# per ft truss.
66#

Top chord section -

$\frac{180 \times 12 \times 70}{7.0} = \frac{16000}{\frac{205}{12950}}$ 40.5 / 33.5 0" / 7.0 @ 3.4 = 23.8# per ft.

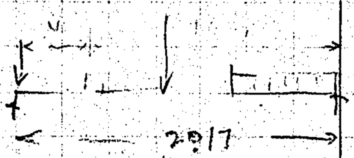
~~Truss unit 10 @ 20.175 = 201.75~~

Stringer 20.17'

Dead load - 95%

$DLm = \frac{1}{8} \times 460 \times 20.17^2 = 23,400$ 1#

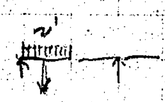
Live load



$\frac{10.08}{6.1'}$

$R = \frac{432 \times 6.08''}{2 \times 20.17} = 396$

$m = 396 \times 10.08 = 4,000$



$205'' \times \frac{1}{4} =$ say $50^{\#}$; $m = \frac{1}{8} \times 50 \times 20.17^2 = 2500$
4000

wheel. $\frac{8800}{2} \times 10.08$ 44500
74400

$Sm = \frac{74400 \times 12}{16000} = 55.7$

use 15" × 42# I

Concrete Fascia. 5710# × .81 = 46500 1#

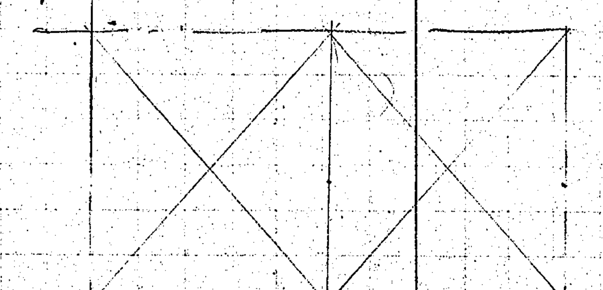
$d = \sqrt{\frac{46500}{107}} = 21''$ make d = 24" -

weight of stringer = 2 @ 57.5 = 115.
2 @ 44.5 = 89.
204# per lin ft.

Floor Beam say 145
349# per ft.

Lower Lateral Bracing. 40# per lin ft

Upper Lateral Bracing.



8Ls 3 × 2½ × 5/16 @ 5.16 × 30 = 1350
connection pls 50
6 Tie Pls - 150
Single lacing - 400
1950

8 @ 1950 = 15600#

Cross struts. 4 Ls 4×3×5/16 @ 7.4 × 21.5 = 620#
2 tie pls. 50
Lacing - 8@20 160
810

4 @ 810 = 3240#

Summary.
Diagonals. 15600
struts 3240
Sway Bracing 30@183 5500
Portal 5400
29740 ÷ 201.75 = 148# lin ft.

Truss
Panel concentration.
Lower Panel
2 lines of Handrails. 2@30 = 60
Stringers 204
FB. 135
Lower Laterals. 40
Trusses Lower half. 425
864
Roadway pavement 22×20 = 440
concrete 75×20 = 1500
Fascia say 500
2440
3304

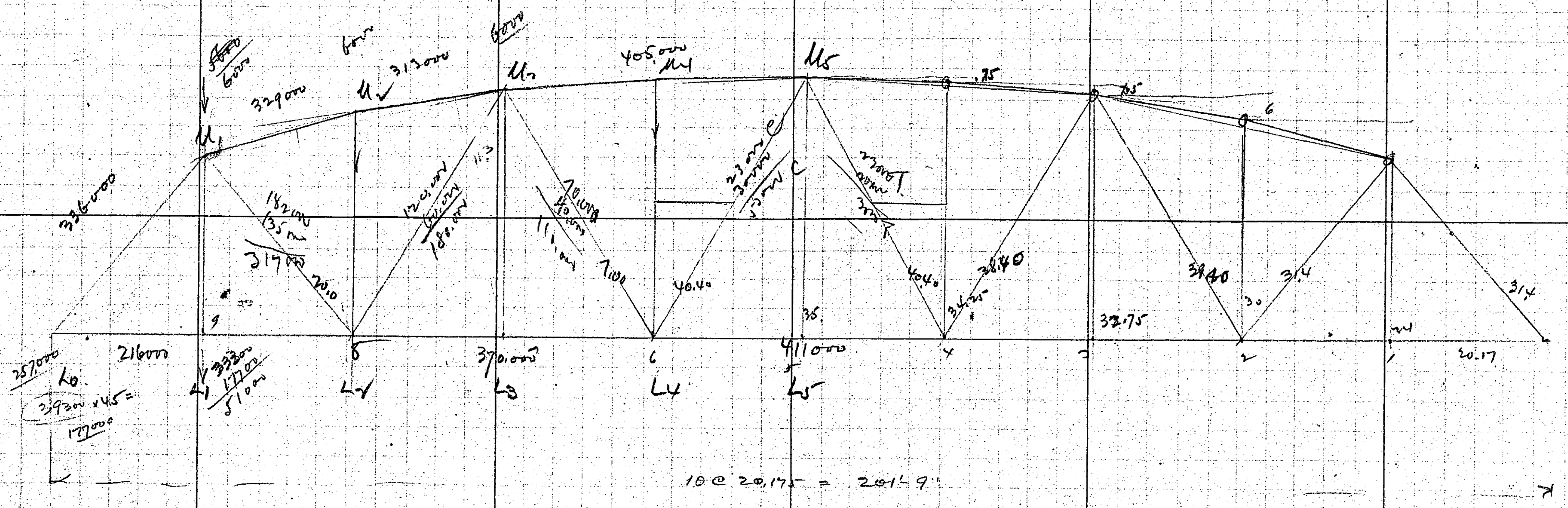
3304 ÷ 2 = 1650 per truss
Panel conc = 1650 × 20.17 = 33200#

Upper Panel
upper laterals. 148
Trusses - upper half. 425
553 ÷ 2 = 277
Panel conc. = 277 × 20.17 = 5600#

Live load.
Uniform live load. 87.5
For one truss 87.5 × 10 = 875#
Panel conc. = 875 × 20.17 = say 17700

For motor truck loading
use 10,000 per motor extra

Preliminary Design of Furukawa bashi, Tokushimaken. 32.



51000
6000
57000 × 4.5 = 257000

1	1			
2	3	200 ×		
3	6	1,600		
4	10	1,600	17700	2000
5	15	1,500	20600	3000
6	21	2,100	37000	4000
7	28	2,800	49500	6000
8	36	3,600 × 17700 =	63700	11350
9	45	4,500		
10				

$16000 - 70 \times \frac{19.2 \times 12}{290} = 10{,}000$. 1.80

11000

2 L 6 × 3½ × ½ — 18.00 —

2 L 5 × 3½ × 3/8 — 12.2 — 9.2 mm

Top chord. 16000 − 70 20.17×12 / 6.66

2400 / 13600 — 52.8

405000 — 30.8 / 15.0 — 32.94 / 29.06

329000

15 × 3/8 — 19 × 7/8

3 L 3 × 3 × 3/8 ; 4 × 3 × 3/8

2 web 15 × 3/8

1.8 × 3/8

2 L 3½ × 3½ × 3/8

2 L 5 × 3½ × 3/8

Bottom chord

411000	25.6	106.8	2 [15 × 33 × 2 Pl 15 × ½
370000	23.2	96.6	2 [15 × 33 · 2 Pl 15 × 3/8 —
216000	13.5	66.0	2 [15 × 33 — 19.8

15.96
10.
25.96

15.96
7.5
23.46

15.96

web Tension — 4 L 6 × 3½ × ½ — 1 web 12 × 3/8

18.00 / 6.00 / 24.00 — 14.00 / 5.00 / 19.00 — OK

5 × 3½ × 3/8 — 14.2 — 9.2

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Preliminary Design of Furukawabashi Totsuhimikes.

weight of Truss

L0U1	32.81 @ 3.40	=	115	×	31.4	=	3600
U1U2	29.06 @ 3.40		100	×	22.0	=	2200
U2-U3	29.51 @ 3.40		100	×	22.0	=	2200
U3-U4	32.81 do		115	×	21.0	=	2400
U4-U5	32.81		115	×	10.0	=	1150
L0-L2	66.0				41.	=	2700
L2-L4	96.6				41.	=	3960
L4-L5	106.8				20.5	=	2200
U1 L2	84				31.4	=	2570
L2-U3	62				38.40	=	2400
U3-L4	42				38.40	=	1600
L4-U5	28.5				40.40	=	1150
U1-L1	28.5				24	=	690
U2-L2	28.5				30	=	860
U3-L3	28.5				32.75	=	920
U4-L4	28.5				34.75	=	980
U5-L5 (1/2)	28.5				35/2	=	500
	20				20	=	400

31480 × 4 = 126000
35% 44000
170000 ÷ 201.75 = 850#

Approximate weight of Iron.

Handrails.	60
stringers	204
FB.	145
upper laterals	150
Lower laterals	40
Trusses.	850
On piers.	50
	1499

1499 × 201.75 = 302000   135 tons

17 @ 135 = 2300 tons.

Cost of Superstructure.

	2300 @ 15	34400
Structural Steel	2300 @ 340 =	780000
	Deck	115150
		929550
	call this	930000 ✓
		700000
		1630000

1630 / 7850 / 8150

Final design of Furukawabashi, Tokushimaken. 34.

Panel length. 20'- 2¼

10 2.25

200

1'- 10½

201'- 10½ = 201.875

17.

1413125

201875

3431875

36.667

3468.542

1005842

6937084

16937084

27748336

17342710

3468542

3488735851524

16 = 2'-3½

12

32

4-8

34'-8"

201'- 10½

2- 3½

204'- 2" center to center of piers.

204.1667

100.5842

4083334

4083334

16333336

10208335

2041667

205.3553585274

1.13

1.14

201'- 10½

1- 1¾

203. 0¼

203.0208

100.5842

4060416

4060416

16241664

10151040

2030208

204.2027870976

205.355

15

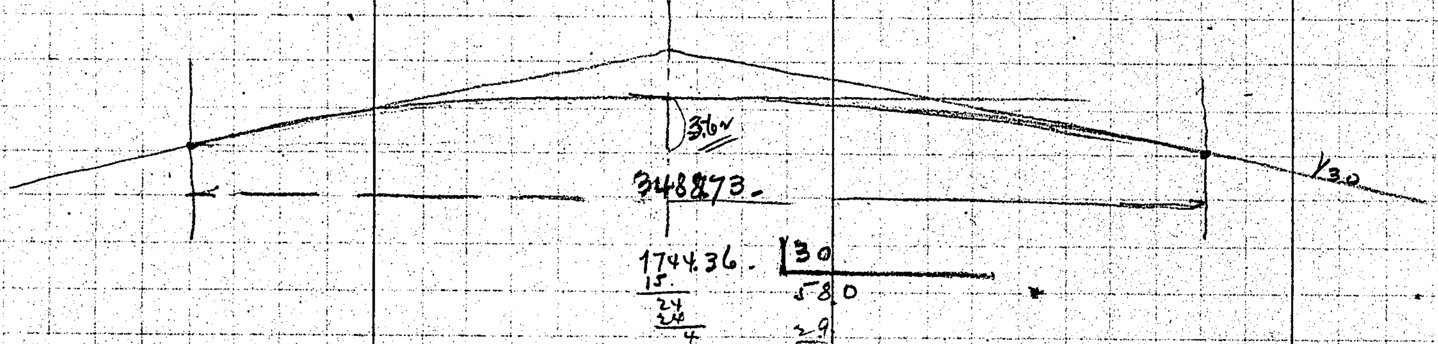
1026775

205355

3080325

40840.5

3488.730

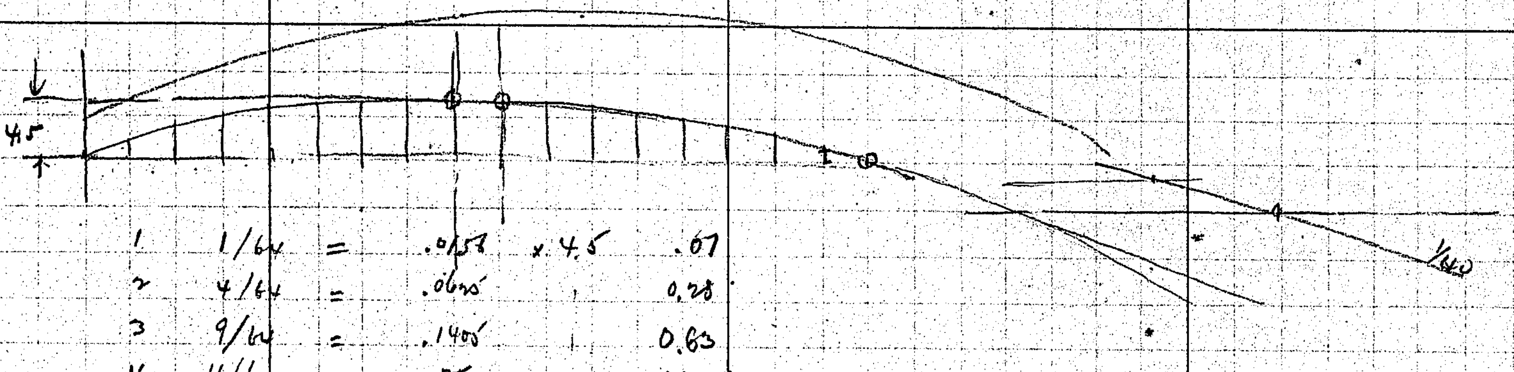


1744.36 | 30

15 | 58.0

24 | 29

4



1	1/64	=	.0156 × 4.5	.07
2	4/64	=	.0625	0.28
3	9/64	=	.1405	0.63
4	16/64	=	.2500	1.125
5	25	=	.39	1.755
6	36	=	.5625	2.53
7	49	=	.765	3.44
8	64	=	1.000	4.50

101 + 202 = 1/100

Final Design of Furukawa Oashi, Tokushimaken.

This bridge is locacated across the Yoshinogawa on main highway between the city of Tokushima and the suburban town of Furukawa. The total length of the bridge will be 3488.73 shaku between center lines of end bearings of end trusses or 17 truss spans of 201'-10½" and 16 spaces of 2'-3½" between bearings of trusses on piers. The entire bridge will be cambered at center about 4.5 shaku to meet the grade of approaches at both ends. After several preliminary designs and investigations we found the longer span than the above mentioned is more economical, however, the increased depth of truss does not suit the given profile and finally decided to adopt the 201'-10½" span layout after consultation with engineers and officials of Home Affairs' Department. The design of piers which will be sunk to the elevation of -75 (for all piers) was approved by the Engineers of the same office.

The structure will be ~~made~~ of steel with roadway of 20'. The Roadway will be paved with 3" woodblock with ½" asphaltic mortar cushion on reinforced concrete slab.

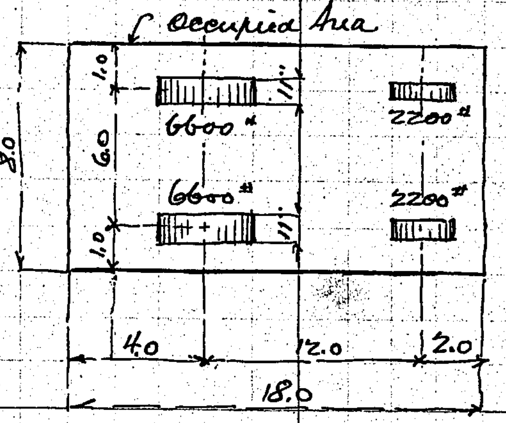
Loadings of bridge.

Uniform Live Load $q^{kg/m^2} = \dfrac{100.000}{170+l}$ where l = span length loaded in meter.

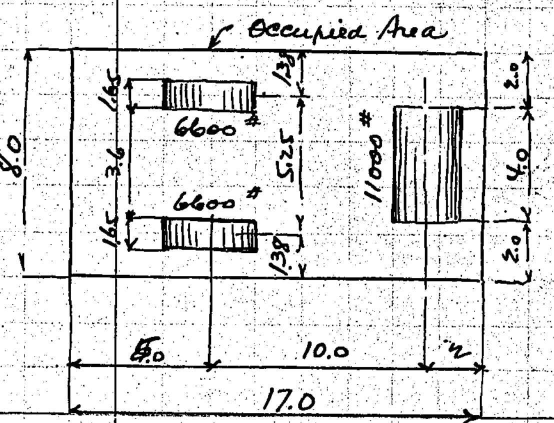
Under 30 meter in span use $q = 500\ kg/m^2$ or 102.5 #/□'

For 201'-10½" span $q = 87.5$ #/□'

Motor truck Loading



Road Roller



⅓ impact for motor truck loading; no impact for uniform Load and Road Roller Concentration. Assumed one motor truck on one span in the direction of bridge; two or more trucks on sideway with occupied space of 8.0' each truck; Unoccupied space to be filled with uniform load. Only one road roller on span assumed.

Assumed Working Strength

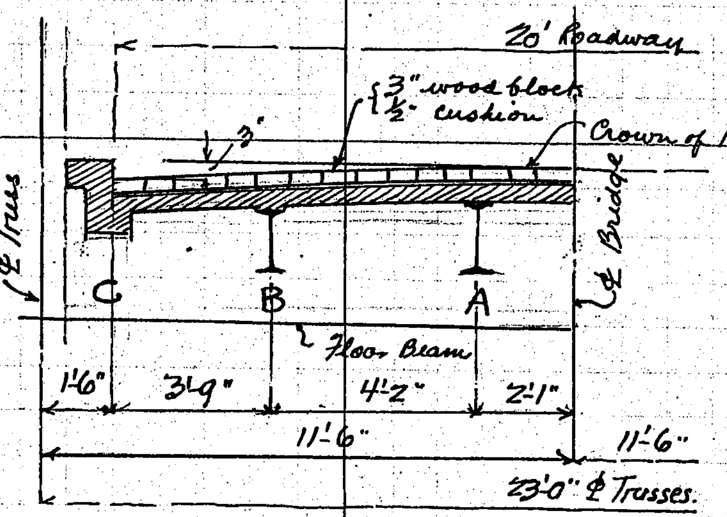
Concrete 1:2:4 mixture			Reinforcing Bar	
Compressive Fibre Stress	pos. moment	650 #/□"	Tension	16000 #/□"
" "	neg. moment	750 #/□"	Shear	10.000 #/□"
Shear plain concrete		40 #/□"		
" reinforced concrete		120 #/□"		
Punching shear		120 #/□"		
Bond Stress		80 #/□"		

Structural Steel

Tension		16000 #/□"
Compression		16000 - 70 l/r
Fibre stress		16000
Shear	shop rivet	10000
	field rivet	8000
Bearing	shop rivet	20.000
	field rivet	16.000
Bearing on pins		24.000
Expansion Roller		600d

Design of 201'-10½" Truss Span
10 panels @ 20'-2¼", center to center of trusses = 23'-0", Head clearance 15' above crown of roadway, Roadway 20'.

Cross Section of Structure.



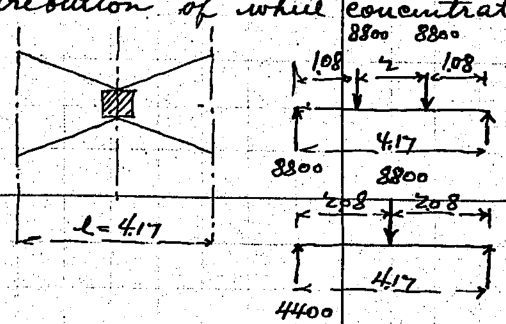
Roadway Slab.

Wood block pavement 3"	15
½" Asphaltic mortar	7
Concrete Slab 6"	75
	97#

Weight of Handrail 30# per lin ft made of structural steel or bar. design to be made properly not to exceed this assumed weight.

Roadway slab Span length 4'-2"
For continuity of slab use $\frac{1}{10}wl^2$ moment for both negative and positive moments.
Dead Load Moment = $\frac{1}{10} \times 97 \times 4.17^2$ = 169# per 1' strip
Live Load Rear wheel concentration 6600 Front wheel 2200
⅓ impact 2200 ⅓ impact 730
8800# 2930#

Distribution of wheel concentration = 4.17 × 0.6 + 1.0 = 3.5'



L.L.m = 8800 × 1.08 = 9500# for 2 wheels
L.L.m = 4400 × 2.08 = 9150# for 1 wheel

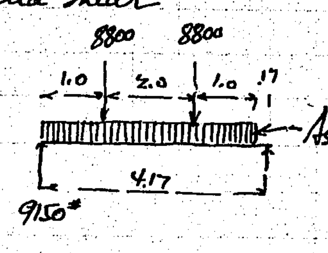
For one ft strip = $\frac{9500}{3.5}$ = 2720#

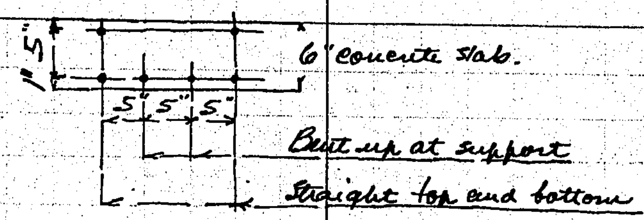
For continuity of slab = 2720 × 0.8 = say 2180#
Dead Load moment 169
Total moment 2349#

Effective depth required for
650#/□" concrete stress + 16000#/□" Steel Stress = $\sqrt{\frac{2349}{107}}$ = 4.7" Use 6" concrete slab with 5" effective depth.

Steel Area required = $\frac{2349 \times 12}{\frac{7}{8} \times 5 \times 16000}$ = .40 □" per ft Use ½"⌀ bars 5" centers = .470 □" as shown in Sketch

End Shear





8800 × $\frac{4.34}{4.17}$ = 9150#

For one ft strip say 9150 ÷ 3.5 = 2610
D.L. 97 × 2.08 say = 200
2810#

Unit Shear = $\frac{2810}{\frac{7}{8} \times 5 \times 12}$ = 53.5#/□"

Plain Concrete good for 40.

13.5#/□" to be carried by stirrups or bent up bars

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Steel Stringer A Span length 20'-2½"

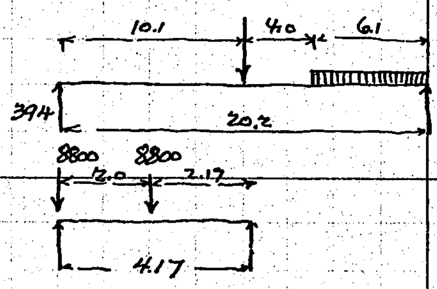
Dead Load slab and pavement $97 \times 4.17 = 405$ $DLm = \frac{1}{8} \times 465 \times 20.2^2 = 23700$ ′#

Stringer assumed 60

465

Live Load Uniform Load $102.5^{\#} \times 4.17 = 428^{\#}$

Reaction $= \frac{428 \times 6.1^2}{2 \times 20.2} = 394^{\#}$



Live Load Uniform Moment $= 394 \times 10.1 =$ 3980 ′#

Rear wheel $8800 \times \frac{2.17}{4.17} = 4580$

8800

13380

Live Load moment $= \frac{13380}{2} \times 10.10 =$ 67600 ′#

Total DL LL moment 95280

Section modulus required $= \frac{95280 \times 12}{16000} = 71.5$

Use 18" × 55# I Beam Sm = 88.4 Unit Stress = $16000 \times \frac{71.5}{88.4} = 13000$#/□"

Steel Stringer B Span length 20'-2½"

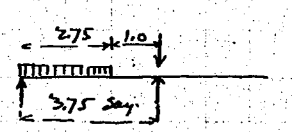
Dead Load Slab + Pavement $97 \times 4.17 = 405$ $DL.m = \frac{1}{8} \times 450 \times 20.2^2 = 23000$ ′#

Stringer assumed 45

450

Uniform Live Load same as for stringer A moment = 3980

Uniform Live Load on Side of motor trucks



Reaction $= \frac{102.5 \times 2.75^2}{2 \times 3.75} = 103$ moment $= \frac{1}{8} \times 103 \times 20.2^2 = 5250$

Wheel concentration moment $= \frac{8800}{2} \times 10.1 =$ 44440

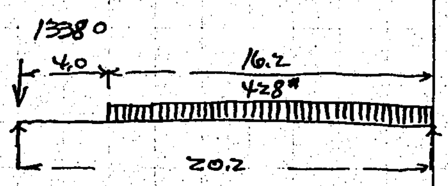
76670 ′#

Sm required $= \frac{76670 \times 12}{16000} = 57.5$

Use 15" × 42# I Beam Sm = 58.9 Unit Stress = $16000 \times \frac{57.5}{58.9} = 15600$#/□"

Max Shear at end.

Stringer A.

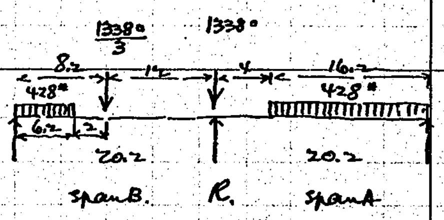


Unif. Load $\frac{428 \times 16.2^2}{2 \times 20.2} = 2780$

wheel Load 13380

16160 # for single shear

Reaction R = $\frac{428 \times 6.2^2}{2 \times 20.2}$ = say 410



$\frac{13380 \times 8.2}{3 \times 20.2} = 1810$

2220

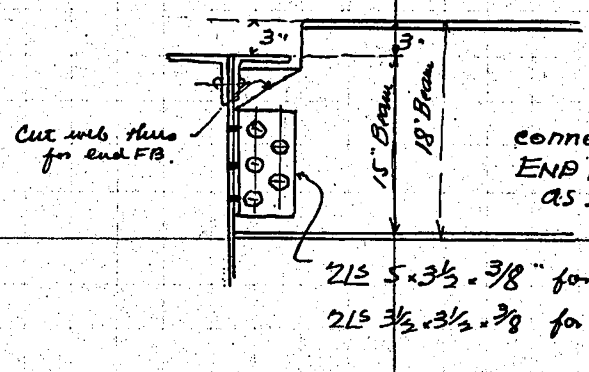
16160

18380 # for bearing

No of Rivet $= \frac{18380}{4375} = 4.2$ for bearing of 5/16" Plate

$\frac{16160}{8750} = 2$ Rivets

Bearing value of ½" web of 18" I.



connection to END FLOOR BEAM. as SHOWN

2Ls 5×3½×3/8" for end FB.

2Ls 3½×3½×3/8 for Int. FB.

weight of stringers

1I. 18" × 55#	× 20.1	=	111.0
4Ls 3½×3½×3/8 @ 8.5 ×	1.0	=	34
Rivet heads + variation			15
			1160 # × 2 = 2320
1I 15" × 42#	× 20.1	=	845
details + var.			50
			895 × 2 = 1790
			4110 #

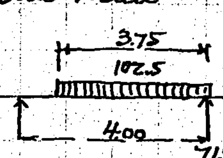
$4110^{\#} \div 20.20 = 204^{\#}$ per lin ft.

Fascia girder C Span length 20'-2½"
Reinforced Concrete
Assumed Dead Load

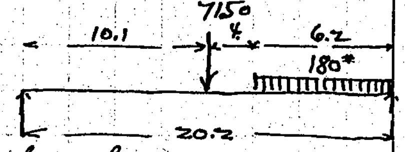
Handrail say	30
Floor say $97 \times \frac{3.75}{2}$ =	182
Beam assumed say	250
	462#

Dead Load Moment $= \frac{1}{10} \times 462 \times 20.2^2 =$ 18800'#

Live Load Uniform

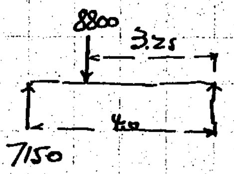


$\frac{102.5 \times 3.75^2}{2 \times 4.00} = 180$#



$\frac{180 \times 6.2^2}{2 \times 20.2} = 172$#   moment = 172×10.1 = 1740
For continuity of beam $1740 \times 0.8 = 1390$'#

Live Load motor Truck



Reaction = $8800 \times \frac{3.25}{4.00} = 7150$#   moment = $\frac{7150}{2} \times 10.1 = \frac{72300}{2}$'#
For continuity of beam $\frac{72300}{2} \times 0.8 = 28900$'#

Summary for moments

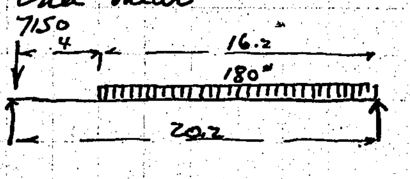
DL m	18800
LL Uniform	1390
LL wheel load	28900
	49090'#

Effective depth required at support $= \sqrt{\frac{49090}{133.5}} = 19.2"$ for Concrete stress 750#/□" + Steel stress of 16000#/□"

Effective depth required at center $= \sqrt{\frac{49090}{107}} = 21.4"$ for concrete 650#/□" + Steel 16000#/□"

make depth of beam 24" over all making effective depth 22".
Required Steel Area = $\frac{49090 \times 12}{\frac{7}{8} \times 22 \times 16000} = 1.910$□" Use 1-⅞"φ = 0.60
3-¾"φ = 1.32
1.92 □"

End shear

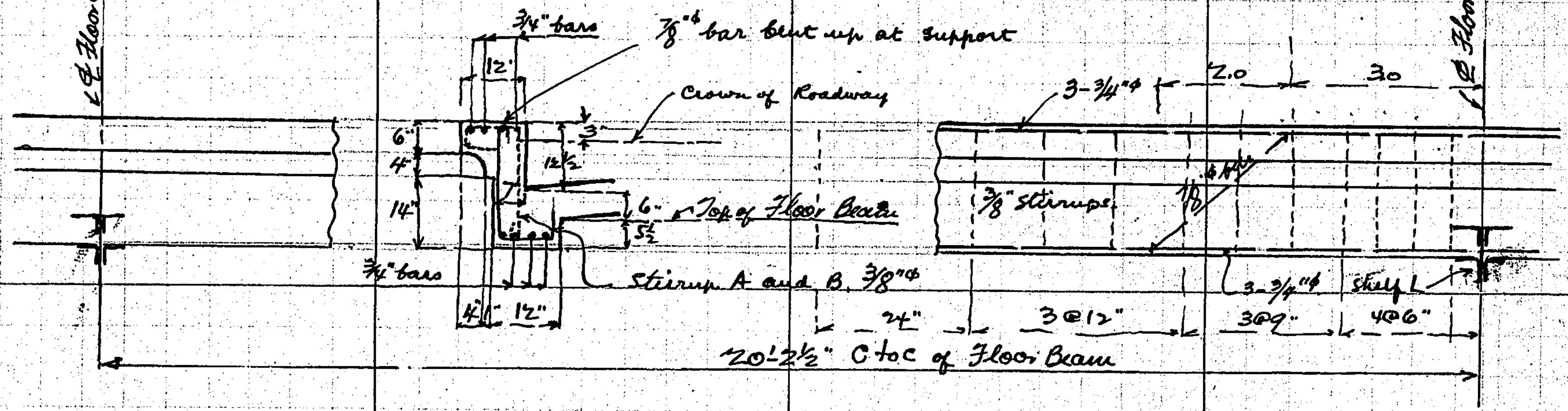


End shear uniform load = $\frac{180 \times 16.2^2}{2 \times 20.2} =$ 1170
wheel concentration 7150
Live Load shear 8320#
Dead Load shear say $462 \times \frac{5}{4} \times \frac{20.2}{2} =$ 5840
14160#

Unit shear approx $= \frac{14160}{\frac{7}{8} \times 22" \times 10" \text{att}} = 73.0$#/□"

Stirrup for 33#/□"

⅜" Stirrup good for $.22 \times 10{,}000 = 2200$#
Stirrup Spacing $= \frac{2200}{33 \times 10} = 6½"$ at end.



Sketch of Fascia Girder C
Scale ⅜" = 1'-0"

Intermediate Floor Beam   Span length 23.0'

Dead Load

weight of Fascia Girder C

Stem	.58 × 2.0	=	1.16	
Coping	.50 × .42	=	.21	
Filler	say		.07	
Bottom	.42 × .46	=	.19	
			1.63 call this 1.70 @ 150 = say	250#
			Handrail say	30
			Floor say 97 × 3.75/2 =	182

Concentration at C.   462# × 20.2 = 9350#

Concentration at B

Pavement and Slab   97 × 3.93 = 381

Steel Stringer   45

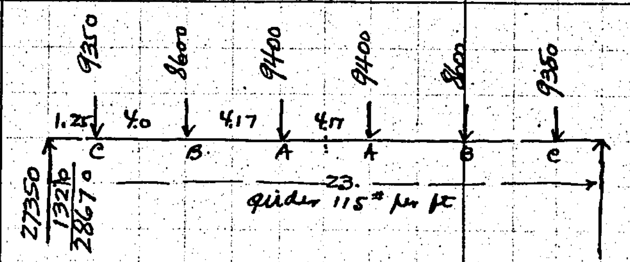
426 × 20.2 = 8600

Concentration at A

Pavement and Slab   97 × 4.17 = 405

Steel Stringer   60

465 × 20.2 = 9400



Dead Load moment at A

27350 × 9.42 = 258000

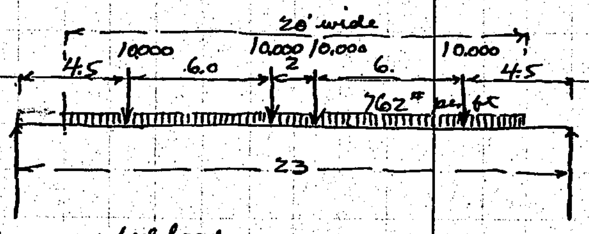
8600 × 4.17 = 35900

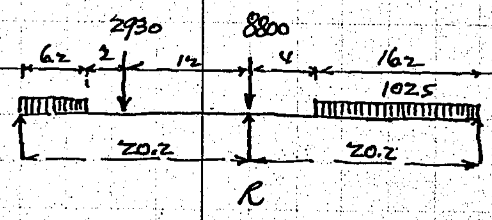
9350 × 8.17 = 76400

−112300

145700#

Live Load   2 motor trucks on roadway, Occupied space 16' wide uniform load on 2' wide on both sides of roadway neglected.





R due to uniform load

$\frac{102.5 \times 16.2^2}{2 \times 20.2} = 665.0$

$\frac{102.5 \times 6.2^2}{2 \times 20.2} = 97.0$

762.0# per ft

20.000 wheel load.

7620 uniform load.

Reaction R due to conc = $\frac{2930 \times 8.2}{20.2}$ = 1190#

8800

Total. 9990 say 10.000

Live Load moment

wheel. 20.000 × 10.5 = 210.000

10.000 × 6.0 = −60.000

150.000

Unif. 7620 × 11.5 = 87700

$7620 \times \frac{10^2}{2}$ = −38100

49600

Total Live Load Moment   199600#

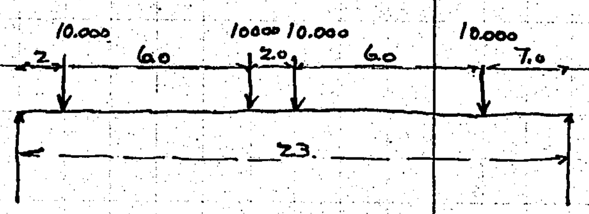
Dead Load Moment   145700

345300#

Dead Load girder assumed = $\frac{1}{8} \times 115 \times 23^2$ = 7600

352900

Max End Shear



max end shear motor trucks loading

Trucks   10.000 × 56/23 = 24400#

Unif.   7620

D.L. deck + girder   28670

60690# call this 61,000#

Intermediate Floor Beam

moment = 352900 "# End shear = 61000# web section reqd for shear = 6.10 □" gross.

At end 24.5" × 3/8" = 9.20□ At center 30 × 3/8 = 11.25 □ 1/8 web = 1.41 □"

30 1/2" b to b L's Effective depth = 2.54 − .24 = 2.30' Stress = 352900 ÷ 2.30 = 154,000#

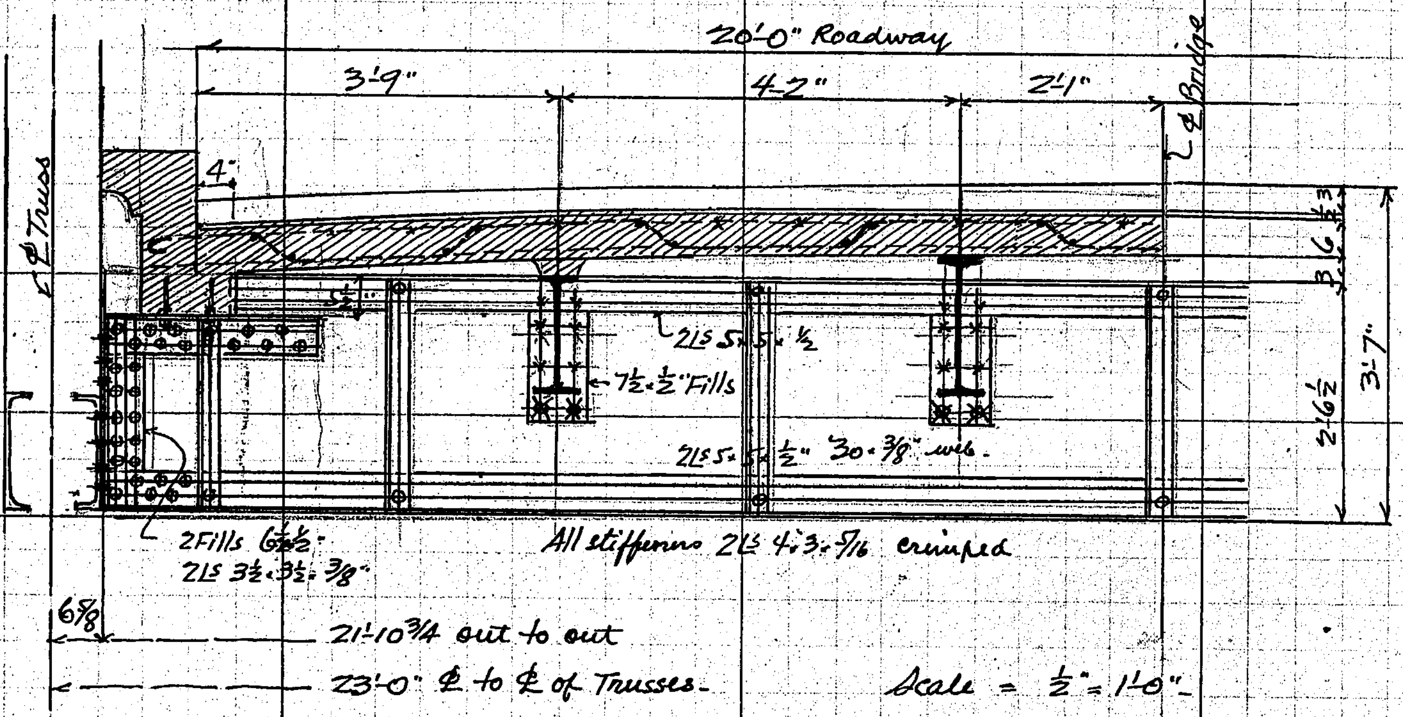
Section required = 154,000 ÷ 16000 = 9.6 − 1.41 = 8.19 □ net

Use 2L's 5 × 5 × 1/2 = 9.50 □ gross or 8.5 □ net

No of Rivet for end connection = 61000 ÷ 4810 = 12.7 single shear

61000 ÷ 4375 = 14.0 5/16" Pl. bearing

For Bearing 3/8" Plate shop 61000 ÷ 6562 = 9.3



Weight of one intermediate Floor Beam

web	30 × 3/8	@	38.25	×	21.89	=	835
Flanges	2L's 5 × 5 × 1/2"	@	16.2	×	21.89	=	709
"	2L's 5 × 5 × 1/2"	@	16.2	×	19.33	=	625
"	4L's 5 × 5 × 1/2"	@	16.2	×	2.00	=	130
Stiffeners	10L's 4 × 3 × 5/16	@	7.2	×	2.50	=	180
"	4L's 4 × 3 × 5/16	@	7.2	×	2.10	=	60
End Stiffs	4L's 3 1/2 × 3 1/2 × 3/8	@	8.5	×	2.00	=	68
Fills	4Pl's 6 1/2 × 1/2	@	11.05	×	1.27	=	56
Fills	8Pl's 7 1/2 × 1/2	@	12.75	×	1.12	=	114
	Rivet heads + variation say						123
							2900#

2900 ÷ 23 = 126.0# per ft of girder

2900 ÷ 20.2 = 143.5 per ft of truss.

End Floor Beam span length 23'-0"

Floor slab beyond end Floor Beam to be overhanged in the direction of bridge; Fascia girder C to be extended to the end of span.

Projection of slab say 1.10' Dead Load = 97# per sq ft

moment = $\frac{97 \times 1.1^2}{2}$ = say 60'#

Live Load concentration with impact = 8800# distribution of this load assumed on 2' × 2' or 2200# per sq ft

LL moment = $2200 \times \frac{1.1^2}{2}$ = 1330

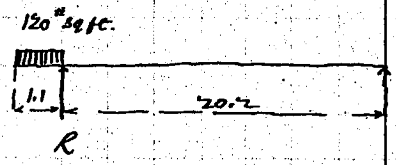
DL " 60

1390 '#

Steel Area required = $\frac{1390 \times 12}{\frac{7}{8} \times 5 \times 16000}$ = .24 □" per ft of strip Use 1/2" φ bars 10" ctrs = .24

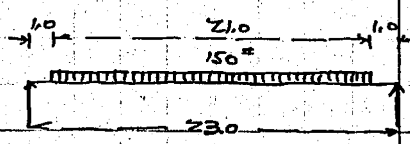
End Floor Beam

Dead Load Dead Load of overhang assumed 120# per sq. ft including filler and fascia girder
width assumed 21.0



Reaction $R = 120 \times 1.1 \times \frac{20.75}{20.2}$ = say 136 or call this 150# per Lin. ft.

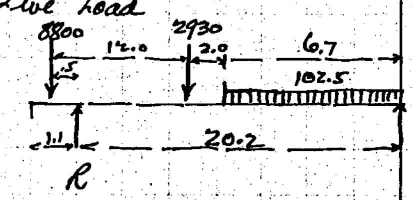
moment due to this loading $1575 \times 11.5 =$ 18100
$150 \times \frac{10.5^2}{2} =$ 8270
9830 1#



½ Dead Load moment of Intermediate FB p39 72850
82680 1#

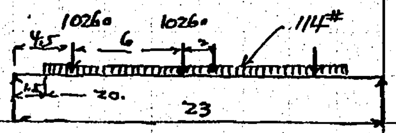
1575#
13675 ½ Interm. F.B.
15250

Live Load



Uniform load $R = \frac{102.5 \times 6.7^2}{2 \times 20.2} = 114$# per lin ft of girder

Wheel load $R = 8800 \times \frac{20.7}{20.2} =$ 9000
$2930 \times \frac{8.7}{20.2} =$ 1260
10260#



moment wheel load $20520 \times 10.5 =$ 215500
$10260 \times 6.0 =$ -61600
153900 1#

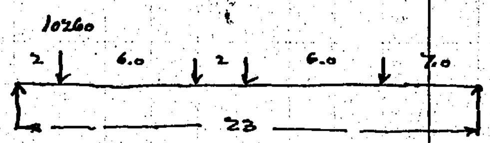
20520
1140

moment Unif. load $1140 \times 11.5 =$ 13100
$114 \times \frac{10^2}{2} =$ 5700
7400 1#

Summary of moments

Dead Load moment	82680
Live Load moment Conc	153900
" " " Unif.	7400
	243980
DL Floor Beam $= \frac{1}{8} \times 115 \times 23^2 =$	7600
	251580

Max End Shear



motor truck loading	$10260 \times \frac{56}{23} =$	25.000#
Uniform load		1140
DL Floor		15250
DL girder	$115 \times 11.5 =$	1320
		42710#

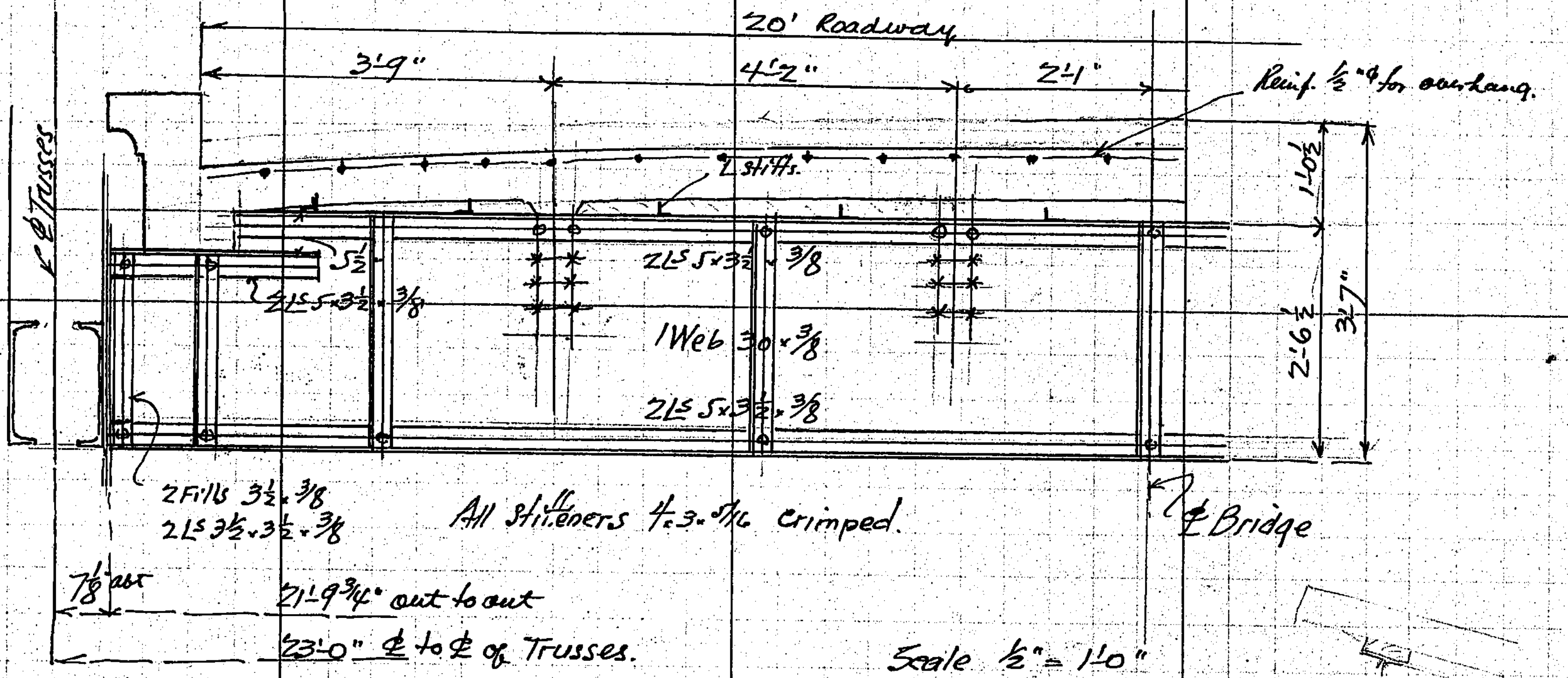
moment = 251580 1# Shear = 42710# web section required for shear = 4.27 □"
Try 30 × 3/8" web = 11.25 □" $\frac{1}{8}$ web = 1.410" 30½" b to b Ls Effective d = 2.54-.14 = 2.4
Stress = 251580 ÷ 2.40 = 10.5000 1# Section Required = 105000 ÷ 16000 = 6.56 □"
or 6.56 − 1.41 = 5.15 □" net Use 2Ls 5 × 3½ × 3/8 = 6.10 □" gr or 5.35 □" net

42710 ÷ 4810 = 9.0 single shear
42710 ÷ 4375 = 10.0 7/16 bearing field
42710 ÷ 6562 = 6.5 3/8 bearing shop

Rivet spacing at end $= \frac{6562 \times 21}{42710} = 3.24''$ Use rivet spacing of less than 3" at ends

For intermediate point figure direct load from motor truck concentration.

End Floor Beam



Weight of one End Floor Beam

web	30×3/8	@ 38.25#	× 21.80	=	835
Flanges	2Ls 5×3½×3/8	@ 10.4	× 21.80	=	453
"	2Ls 5×3½×3/8	@ 10.4	× 19.33	=	402
"	4Ls 5×3½×3/8	@ 10.4	× 20	=	83
Stiffeners	10Ls 4×3×5/16	@ 7.2	× 2.50	=	180
"	4Ls 4×3×5/16	@ 7.2	× 2.10	=	60
End Stiffs	4Ls 3½×3½×3/8	@ 8.5	× 2.00	=	68
Fills	4Pls 3½×3/8	@ 4.46	× 1.5	=	27
Stiffs	10Ls 2½×2½×¼	@ 4.1	× .75	=	31
	Rivet heads + variation say				111
					2250#

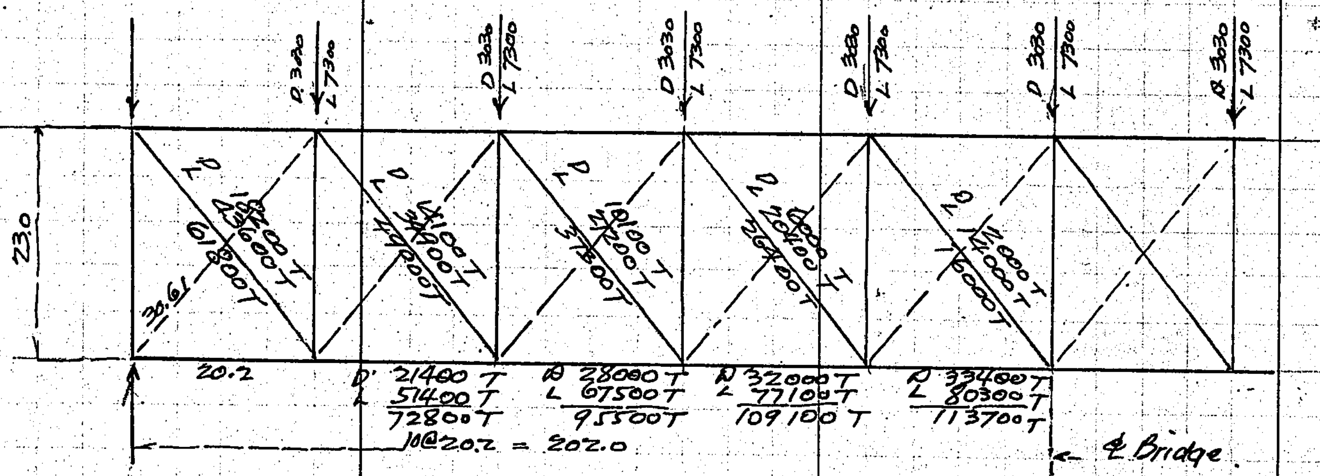
2250 ÷ 23.0 = 98# per lin ft girder
2250 ÷ 20.2 = 112 per lin ft truss

Bottom Laterals.

Static wind load 30#/□' of exposed surface.

Assume 5 × 30 = 150# per lin ft panel concentration = 150 × 20.2 = 3030#

Moving wind load say 12 × 30 = 360# per lin ft panel conc. = 360 × 20.2 = say 7300#



$\tan\theta = 20.2 \div 23.0 = .88$

$\sec\theta = 30.61 \div 23.0 = 1.33$

$W_1 = 3030^{\#}$

$W_2 = 7300^{\#}$

$W_1 \tan\theta = 3030 \times .88 = 2670$

$W_2 \tan\theta = 7300 \times .88 = 6425$

$W_1 \sec\theta = 3030 \times 1.33 = 4030$

$W_2 \sec\theta = 7300 \times 1.33 = 9700$

Sections of Bottom Laterals.

	Stress	3/4" Rivets 10.000#/□" = 4420#	Sq. net
1st Panel	61800 T	14	3.86 □"
2nd "	49000 T	11	3.06
3rd "	37300 T	8.5	2.33
4th "	26400 T	6.0	1.65
5th "	16000 T	3.6	1.00

1st Panel Stress = 61800 #T

Try 2Ls 4.3.5/16 = 4.18 □" or 3.64 □" net @ 16000 = 58200 # good for.

Unsupported length = 16.0 about r = 1.28

16000 − 70 l/r = 5500 #/□" l/r = 150

Stress to be carried by compression member = 5500 × 4.18 = 23000 # C 61800

38800

Use 10 − 3/4" Rivets for connection

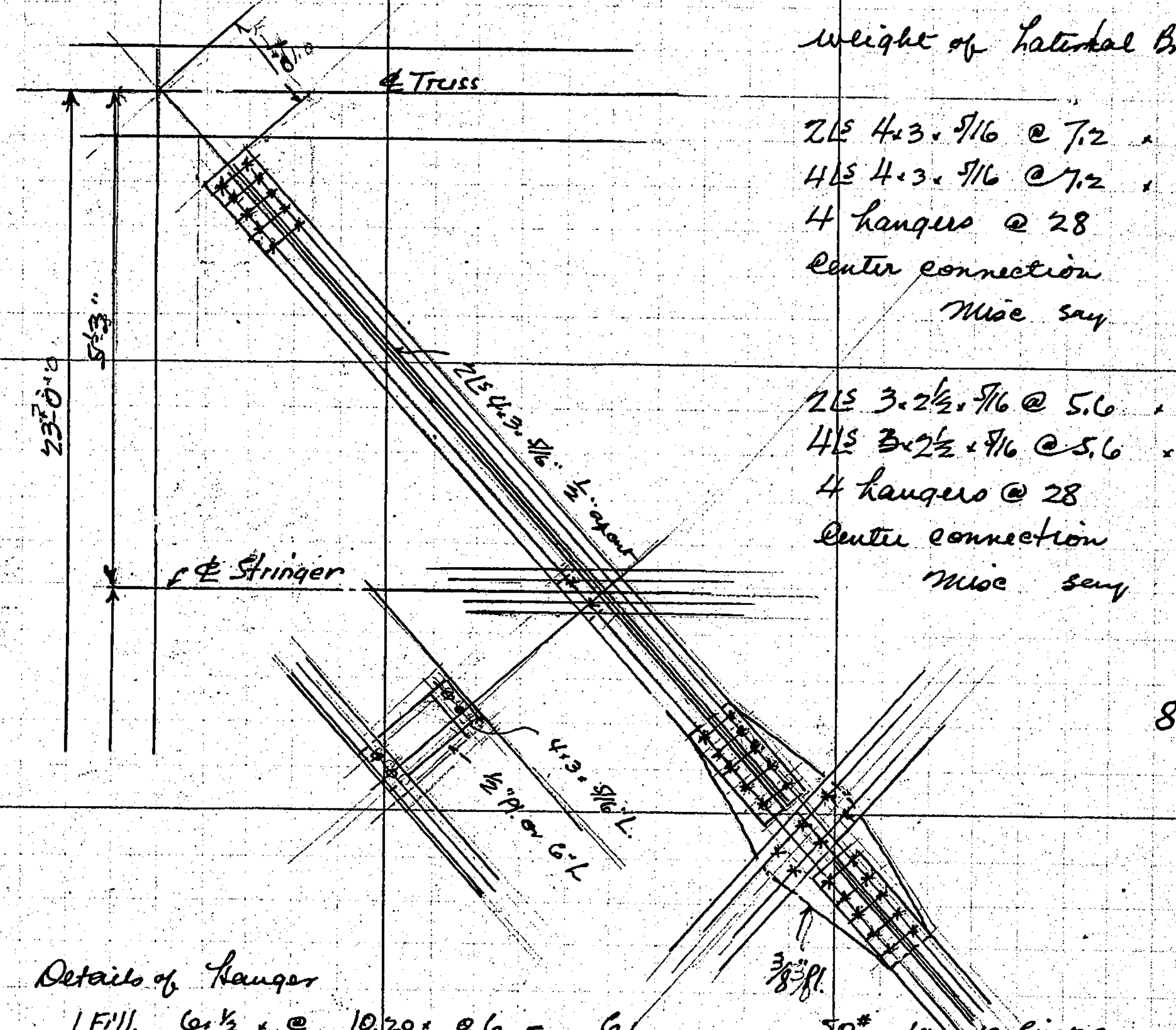
2nd Panel Use same section as for 1st panel

3rd Panel stress = 37300 # T

Try 2Ls 3.2½.5/16 = 3.24 □" or 2.70 □" net @ 16000 = 43200 # good for

Use 8 Rivets for connection.

4th and 5th panels same as for 3rd.



Weight of Lateral Bracing

2Ls 4.3.5/16 @ 7.2	×	28.6	=	411
4Ls 4.3.5/16 @ 7.2	×	14.0	=	402
4 hangers @ 28			=	112
Center connection				50
Misc say				10
				985 × 4 = 3940
2Ls 3.2½.5/16 @ 5.6	×	28.6	=	320
4Ls 3.2½.5/16 @ 5.6	×	14.0	=	313
4 hangers @ 28			=	112
Center connection				40
Misc say				10
				495 × 6 = 4770
				8710 #

8710 # ÷ 202.0 = 43 # per lin ft.

Details of Hanger

1 Fill 6 × ½ × @	10.20 × 0.6	=	6.1
1L 6.3½.3/8 @	11.7 × 1.25	=	15.0
1L 4.3.5/16 @	7.2 × 0.6	=	4.3
variation			2.6
			28.0 #

50 # for 10 Rivet conn

45 # for 8 Rivet conn

Weight of Sway Bracings

Top Strut	4Ls 4×3×5/16	@ 7.2#	× 21.4	=	615
Tie Pls.	2Pls. 15×5/16	@ 15.94	× 1.0	=	32
Lacing bars	19 @ 5#				95
Bottom Strut	2Ls 4×3×5/16	@ 7.2	× 24.0	=	345
Diagonals	1L 3×3×5/16	@ 6.1	× 54.0	=	330
6 conn Pls.	6 @ 28#				168
	Rivet heads + variations - say				55
					1640#

7@1640 = 11480#

Summary for top bracings

Diagonals	13440
Portals	5700
Sway bracing	11480
	30320# per truss

or say 150# per lin. ft of span.

Design of 201'-10½" truss

Panel concentrations

Lower Panel

2 Lines of Handrails assumed =	60
Stringers	204
Floor Beam (Intermediate)	143.5
Lower Laterals	43.
Trusses Lower Half assumed	430.
	880.5#

Roadway

Pavement	22# × 20	=	440	
Concrete slab	75 × 20	=	1500	
Fascia girder	2 @ 250#	=	500	2440.0

33205# ÷ 2 = 1660# per truss.

Lower Panel concentration = 1660 × 20.2 = 33530#

Upper Panel

Upper Laterals	150
Trusses upper half assumed	430

580 ÷ 2 = 290# per truss

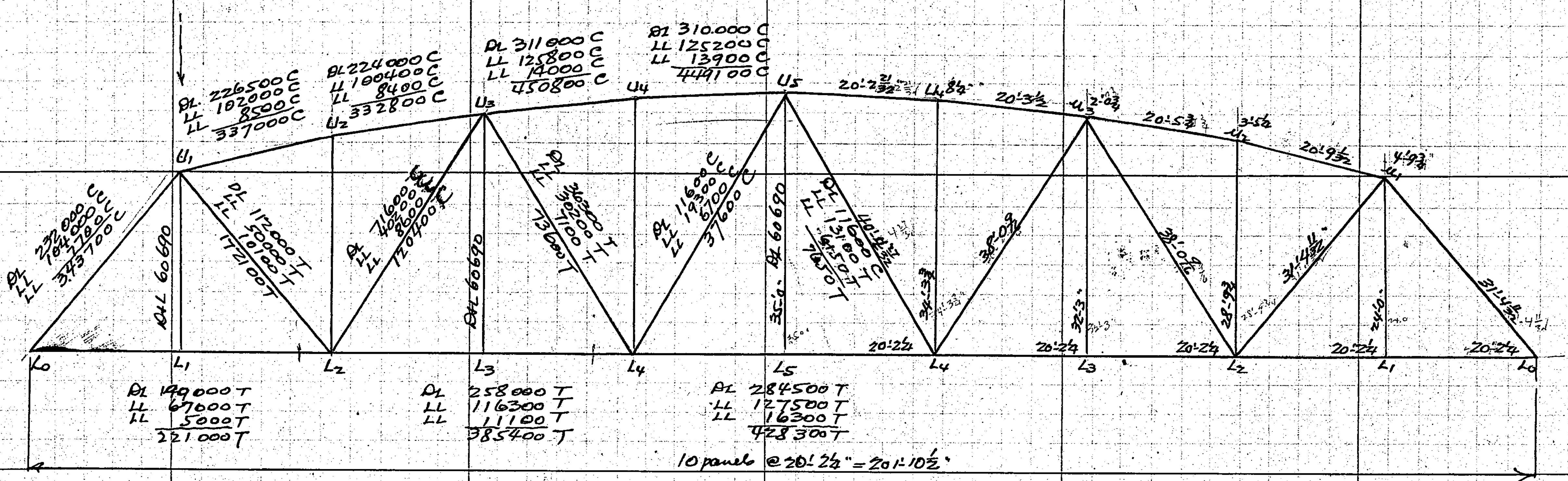
Upper Panel concentration = 290 × 20.2 = 5860#

Uniform Live Load on span

For full loading	87.5#	87.5 × 10 = 875# per truss
" 9 panels	91.0	91.0 × 10 = 910 " "
8 panels	93.5	93.5 × 10 = 935 " "
7 panels	96.0	96.0 × 10 = 960 " "
6 panels	99.0	99.0 × 10 = 990 " "
5 panels	102.5	102.5 × 10 = 1025 " "
4 panels	102.5	do
3 panels.	102.5	do

Motor trucks Loading. Use 1/3 impact wheel load distribution and assumed occupied area as shown on sketch on pp 35.

Stresses in truss.



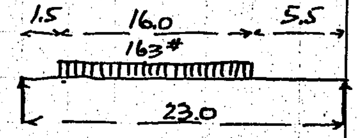
Points of Intersection of both chords			Lever Arm for top chords			Lever Arms for Diagonals		
At U_5	$20.1875 \times \frac{35}{.6875}$	= 1027.0	U_5-U_4	$\frac{1007.2 \times 35}{1008}$	= 34.2	U_5-L_4	$\frac{100.68 \times 35}{40.409}$	= 873.0
At U_4	$20.1875 \times \frac{34.31}{2.06}$	= 335.0	U_4-U_3	$\frac{335 \times 34.31}{336.8}$	= 34.1	L_4-U_3	$\frac{322.5 \times 33.5}{38.05}$	= 284.0
U_3	$20.1875 \times \frac{32.25}{3.44}$	= 189.0	U_3-U_2	$\frac{168.8 \times 32.25}{191.8}$	= 28.4	U_3-L_2	$\frac{168.8 \times 32.25}{38.05}$	= 143.0
U_2	$20.1875 \times \frac{28.81}{4.81}$	= 121.0	U_2-U_1	$\frac{121.0 \times 28.81}{124.4}$	= 28.05	L_2-U_1	$\frac{121.0 \times 24}{31.36}$	= 92.6

Upper Panel load 5860#
Lower Panel load 33530#

Live Load for top and bottom chords
Uniform load $87.5^{\#}/\text{'}$ or $87.5 \times 10 = 875^{\#}$ per truss.
Extra motor truck loading with impact of 1/3
Rear wheel 2 @ 6600 = 13200#
Front wheel 2 @ 2200 = 4400

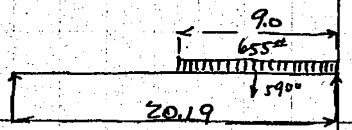
17600#
1/3 impact 5860
23460 Assuming this load distributed over 8×18
Average load = 163#/□'

Reaction on one truss



$\dfrac{163^{\#} \times 16 \times 13.5}{23} = 1530$
less 875
655# per ft extra load for 18' long.

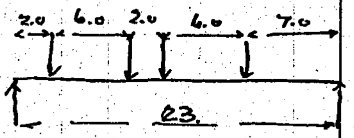
This extra load of 655 × 18 = 11800# placed on center of span
Panel Concentration as follows



$5900 \times \dfrac{4.5}{20.19} = 1320$

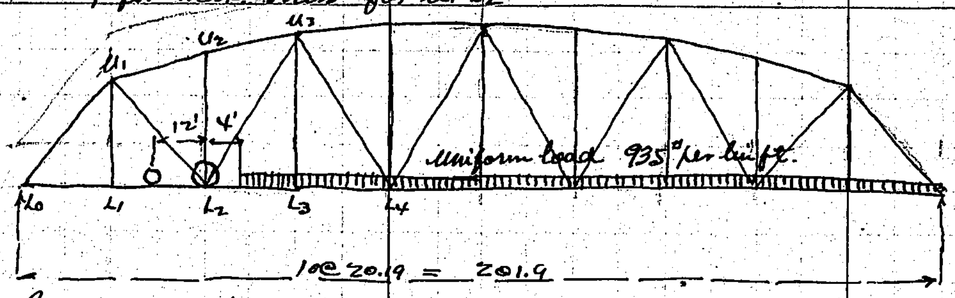
1320 4580#

For diagonals - motor truck loading
Rear wheel with impact 8800#
Front wheel 1/3 of rear wheel.



Rear wheel $8800 \times \dfrac{56}{23} = 13800^{\#}$
Front wheel $13800 \div 3 = 4600^{\#}$

Loading for max stress for $U_1 L_2$



Panel load at L_1 $4600 \times \dfrac{12}{20.19} = 2730^{\#}$

Panel load at L_2 4600 - 2730 = 1870
Rear wheel 13800
15670#

Max Live Load Reaction on shoes.
Uniform load 875 per lin ft Reaction = $\dfrac{875 \times 197.9^2}{2 \times 201.9} = 84500^{\#}$

Motor truck loading rear wheel with impact 13800
98300# per shoe.

Top chord sections.

U_3-U_4-U_5 max stress = 450800#C

2[15" × 40# = 23.52 Radius of gyration Hor. axis 5.76

1 cov Pl. 22" × 1/2" = 11.00 " " " vert axis 7.52

34.52 □"

Unsupported length of member = 243"

Allowable Unit Stress = 16000 − 70 × $\frac{243}{5.76}$ = 13050 #/□"

section required = 450800 ÷ 13050 = 34.60□" Assumed section Ok.

U_1-U_2-U_3 maxstress = 337000#C

2[15" × 33# = 19.80 Radius of gyration Hor Axis 5.84

1 cov Pl. 22 × 1/2 = 11.00 " " " Vert Axis 7.56

30.80 □"

Unsupported length = 249"

Unit stress = 16000 − 70 × $\frac{249}{5.84}$ = 13020#

Section required = 337000 ÷ 13020 = 25.85 □" Assumed section Ok.

End Post L_0-U_1 max Stress = 343700# C

2[15" × 40# = 23.52 r Hor Axis 5.76

1 cov Pl. 22 × 1/2 = 11.00 r vert Axis 7.52

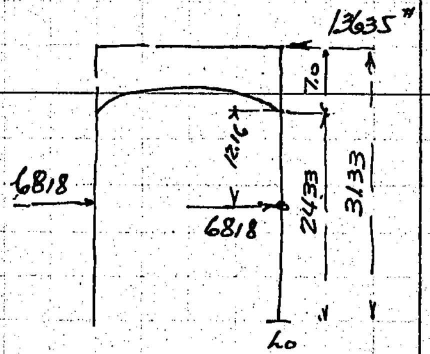
34.52 □"

Unsupported length of member = 376"

Unit Stress = 16000 − 70 $\frac{376}{5.76}$ = 11440 #/□

section required = 343700 ÷ 11440 = 30.0 □"

For Bending due to wind wind load 3030 × 4.5 = 13635#



Assume the point of contra flexure = 12.16' above L_0

Moment due to wind load = 6818 × 12.16 = 83.000 '#

Moment of Inertia parallel to bridge = 2011.9

Bending Stress = $\frac{83000 \times 12 \times 11}{2011.9}$ = 5450 #/□"

Reduce live Load stress 1/3 considering Wind-load Stress

DL 232000

LL 111700 × 2/3 74500

306500#

Unit Stress = 11440 × 1.3 = 14900 Section Required = $\frac{306500}{9450}$ = 32.40□"

less 5450

9450 Assumed Section Ok.

Bottom Chord Sections.

L_0-L_2	211000 T	÷ 16000	=	13.20□	net
L_2-L_4	385400 T	÷ 16000	=	24.10□	"
L_4-L_4	428300 T	÷ 16000	=	26.80□"	"

L_0-L_2 2[15" × 33# = 19.80 gross 4 holes in flanges = 4 @ .625 = 2.5

or 4.10 4 @ .40 = 1.6

15.70 □" net Reduction 4.1 □"

L_2-L_4 2[15" × 33# = 19.80 gross 15.70 net

2 Pls 12 × 7/8 = 10.50 8.75

30.30 24.45 □"

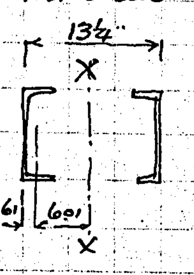
Bottom Chord L_4-L_4'

2[s 15" × 40# = 23.52   18.92 □" net   4 holes from flanges + 4 holes from webs.
2 Pls 12 × 7/16" = 10.50   8.75
34.02   27.67 □" net

Diagonals.

U_1-L_2 S = 172100#T ÷ 16000 = 10.8 □" net   Try 2[s 12" × 25# = 14.70 □"
* 4 holes from web 7/8" Rivets 4 @ 0.39 = 1.56 □"
4 " " flanges 3/4" Rivets 4 @ 0.437 = 1.75
Holes out either in flange or web   min net area = 12.95 □"

L_2-U_3 S = 120400#C
Try 2[s 10" × 20# = 11.76   r = 3.66,   Unsupported length $38 \times \frac{12}{2}$ = 228"
Unit Stress = $16000 - 70 \times \frac{228}{3.66}$ = 11640 #/□"   SR = 10.35 □"



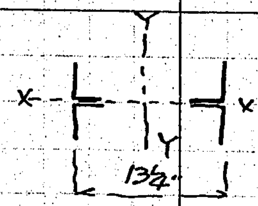
Radius of Gyration axis XX.
m. of Inertia $r = \sqrt{\frac{428.7}{11.76}} = 6.05$
$11.76 \times 6.01^2 + 5.7 = 428.7$
Unit stress = $16000 - 70 \times \frac{456}{6.05}$ = 10730 #/□"

Section required = 120400 ÷ 10730 = 11.20 □"   Assumed section OK.

U_3-L_4 73600#T
L_4-U_5 37600#C or 23060#T
} Use 2[s 10" × 20# = 11.76 □"

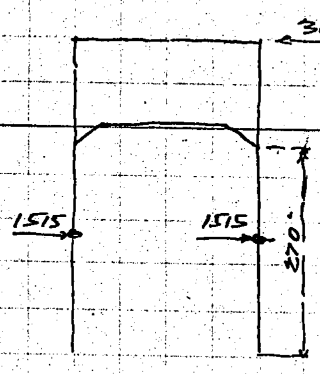
Hangers U_1-L_1 U_3-L_3 U_5-L_5 S = 60690#T   SR = 3.8 □" net
Use 4 ∟s 4 × 3 × 5/16 = 8.36
2.35
6.01 □" net
Deduction rivet holes 4 - 3/4" = 1.10
4 - 7/8" = 1.25
2.35
Unit stress = 60690 ÷ 6.01 = 10100 #/□"

Verticals U_2-L_2 U_4-L_4
Section 4 ∟s 4 × 3 × 5/16 = 8.36 □"



r_{YY} = 5.93 (inch)²   I = 294
r_{XX} = 1.90 "   I = 30.3

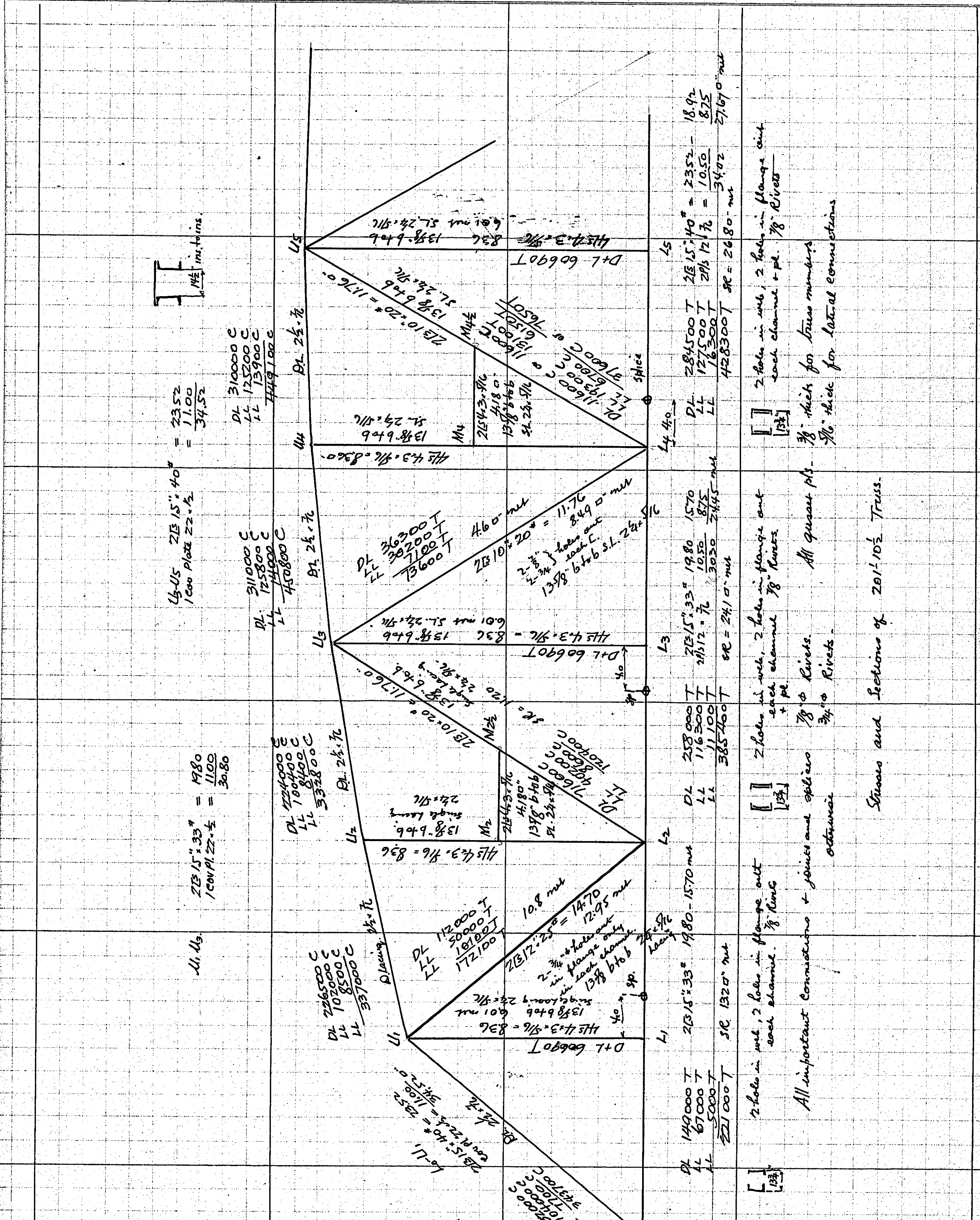
Unsupported length in elevation 17.5 × 12 = 210" $l/r = \frac{210}{1.90} = 111.$
Unsupported length in crosswise 22.5 × 12 = 270" $l/r = \frac{270}{5.93} = 45.5$



Moment due to wind load 1515 × 135" = 204500"#
Fibre Stress = $\frac{204500 \times 6.62}{294}$ = 4600 #/□"

Unit stress = 16000 - 70 × 111 = 8230 × 1.3 = 10700#
less 4600
6100#

Section good for 8.36 × 6100 = 51000#   ok.


All important connections + joints and splices 7/8" ⌀ Rivets.
otherwise 3/4" ⌀ Rivets.
All gusset pls. 3/8" thick for truss members
5/16" thick for lateral connections
Stresses and Sections of 201'-10½" Truss.

Approximate weight of Truss

member	wt main member	wt details
L_0-U_1	3520	500
U_1-U_2-U_3	4290	1516
U_3-U_4-U_5	4752	1883
L_0-L_1	1485	1120
L_1-L_3	2765	960
L_3-L_4	2875	1178
L_4-L_5	1410	710
U_1-L_2	1450	350
L_2-U_3	1400	440
U_3-L_4	1400	440
U_5-L_4	1500	500
U_1-L_1	625	140
U_2 L_2	820	200
U_3-L_3	910	185
U_4-L_4	990	220
U_5-L_5 (½)	505	100
M_2-$M_{2\frac{1}{2}}$	145	110
M_4-$M_{4\frac{1}{2}}$	145	110
	30987	10662

Main section 30987
Details 10662
41649 × 4 = 166600
Rivet heads + variation 5% 8330
174930 #

weight of trusses say 175000 #
175000 ÷ 201.87 = 860 # per ft of span.

wt of details 10662 × 4 = 42648
8330
50978
say 51000 #

51000 ÷ 124000 = 41%.

Load on End Pin.
metal in Bridge

stringers	10 @ 4410	=	41100
Floor Beams	9 @ 2900	=	26100
do	2 @ 2250	=	4500
Bottom Laterals			8710
Top Laterals etc			30320
Trusses			175000

285730
Handrail 2 lines @ 30 # = 60 # × 204 = 12240
297970
Roadway pavement + slab etc 3320.5 × 204.7 = 680000
977970
call this 978000 #

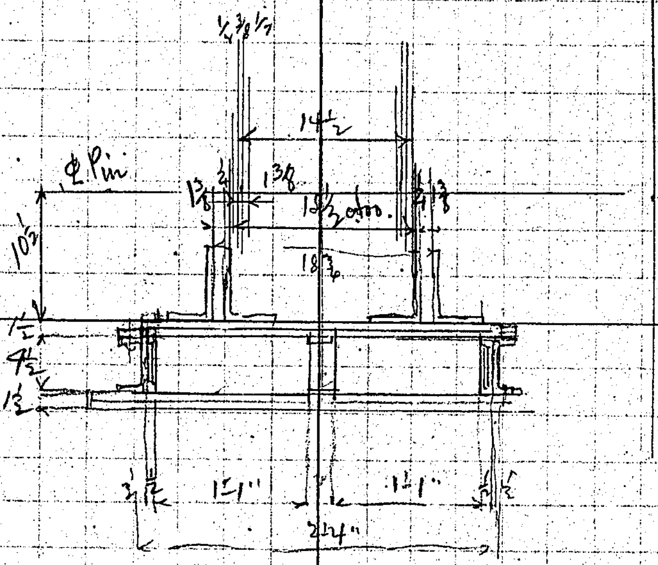
Load on pin Dead Load 978000 ÷ 4 = 244500
Live Load 98300
342800 #

Assume 6" pin Unit bearing 6 × 24000 = 144000 #
Thickness bearing pls = 342800 ÷ 144000 = 2.38"
or 1.19" per rib.
Use 2-½" Pl reinforcement to 3/8" gusset pl. thickness 1.375"
Bending moment = 171400 × 1.625 = 279000 "#
6" pin good for moment of 508900 "# ok

Load on Bearing
Load on pin 342800
wt of shoe say 2000
344800 #

Try 4½" Rollers Unit stress = 600 × 4.5 = 2700 # per lin. inch
344800 ÷ 2700 = 127"
Use 5 Rollers 25½" net

Final Design of Furukawa-bashi, Tokushimaken



Final Design of Furukawa-Bashi, Tokushimaken

This bridge is located across the Yoshinogawa on main highway between the city of Tokushima and the suburban town of Furukawa. The total length of the bridge will be 3528.381 shaku between center lines of end bearings of end trusses or 17 truss spans of 203'-11 1/2" and 16 spaces of 2'-6 1/2" between bearings of trusses on piers. The entire bridge will be cambered at center about 4.5 shaku to meet the grade of approaches at both ends. After several preliminary designs and estimates we found the longer span than the span mentioned above, is more economical, however, the increased depth of truss does not suit the given profile and finally decided to adopt the 203'-11 1/2" span layout after consultation with engineers and officials of Home Affairs' department. The design of piers which will be sunk to the elevation -75.0 was approved by the Engineers of the same office.

The bridge will be made of structural steel with roadway of 20 R. The roadway slabs will be of reinforced concrete slabs paved with 2" asphaltic concrete.

Loadings on bridge.

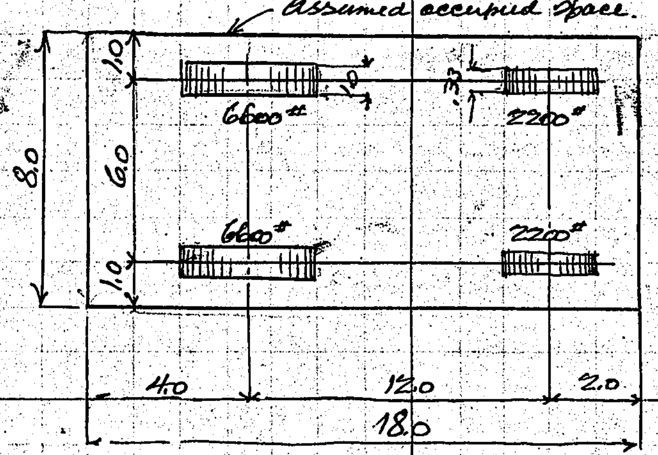
Uniform Live Load q kg/m² $= \dfrac{100.000}{170+l}$ where l = span length loaded in meter.

Under 20 meter in span use q = 500 kg/m² or 102.5 #/□'

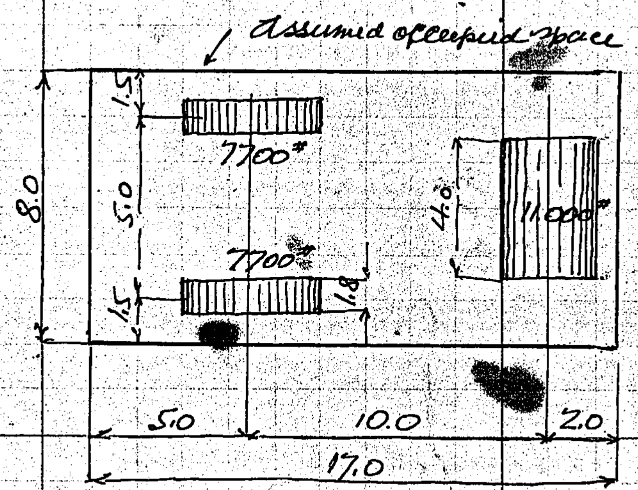
Motor trucks loading 8 ton motor trucks (8000 kg) approx. equivalent in lbs as shown on sketch

Road roller 12 ton road roller (12000 kg) approx equiv. in lbs as shown on sketch below.

Motor Truck Loading



Road roller



Impact formula :- $I = \dfrac{20}{60+l}$ where loaded length in meter

For 203'-11 1/2" span I = 16.4 %

Let us assume impact for truss span 25% and 1/3 impact for all others.

no impact for uniform load and Road roller concentration : Assume one motor truck on one span in the direction of bridge. two or more trucks side by side with occupied space of 8' each; Unoccupied space to be filled in with uniform load ; Only one road roller on span assumed.

Assumed Working Strength

Concrete 1:2:4 Mixture

Compressive Fibre stress positive moment	640 #/□"	
" " " negative moment	710 #/□"	
Shear without web reinforcement	57 #/□"	= 4 kg/cm²
" with web reinforcement	128 #/□"	9 kg/cm²
Punching shear	128 #/□"	
Bond stress	85 #/□"	6 kg/cm²

Structural Steel or Reinforcing Steel

Tension	18000 #/□"
~~Fibre stress~~ structural member	17000 (1 − 0.012 l/b)
Compression	21300 (1 − 0.0055 l/r) #/□"

where l = length of member in ft

r = least radius of gyration in inches

max stress = 14000 #/□"

Assumed working Strength (continued).

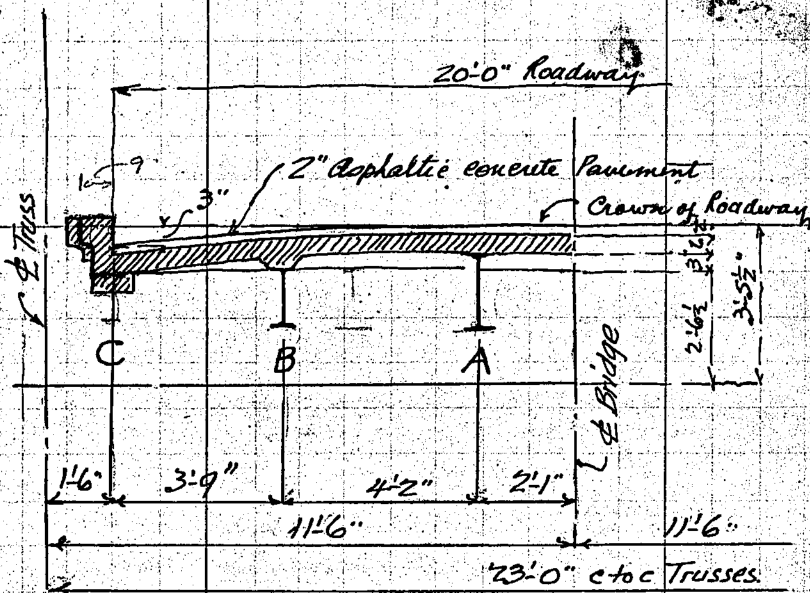
Shear	shop rivet	10.000 #/□"
	field rivet	8.000 "
Bearing	Shop rivet	20.000
	field rivet	16.000
Bearing on pins		24.000
Expansion roller		600d where d = diameter of roller in inches.

Design of 203' - 11½" truss Span

20' Roadway Head clearance above crown of roadway center to center of trusses = 23'-0"

The span will be divided into 10 panels of 20'-4¾" each.

Cross section of Structure



Roadway Slab:-

2" Asphaltic Concrete say	22.0
Concrete slab	75.0
	97.0

weight of Handrail 30# per lin ft structural steel shape or bar. Design to be made properly not to exceed this assumed weight.

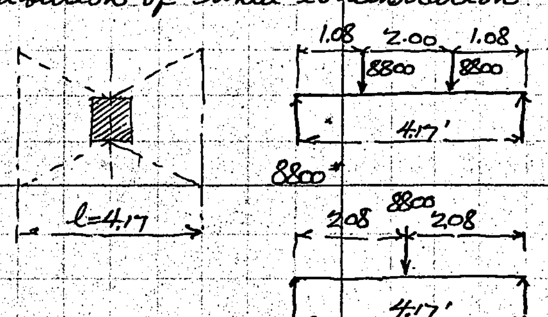
Roadway slab span length 4'-2"

For continuity of slab use $\frac{1}{10}wl^2$ moment for both positive and negative moments.

Dead Load moment $= \frac{1}{10} \cdot 97 \cdot 4.17^2 = 169$ '# per one ft strip

Live Load	Rear wheel concentration	6600	Front wheel	2200
	⅓ impact	2200	⅓ impact	730
		8800#		2930#

Distribution of wheel concentration $= 4.17 \times 0.6 + 1.0 = 3.5$ ft



Live Load position assumed as shown on sketch

Live Load Moment $= 8800 \cdot 1.08 = 9500$ '# for 2 wheels

" " " $= 4400 \cdot 2.08 = 9150$ " 1 wheel

For one ft strip $= \frac{9500}{3.5} = 2720$ '#

For continuity of slab $= 2720 \times 0.8 =$ say 2180#

Dead Load moment 169

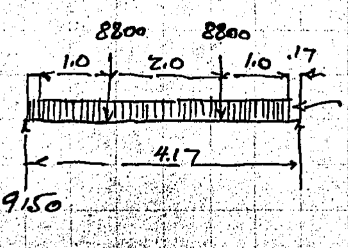
Total moment 2349 '#

Effective depth required for 640#/□" concrete stress and 17000#/□" steel stress.

$d = \sqrt{\frac{2349}{102}} = 4.8"$ Use 6" concrete slabs with 5" effective depth

Steel Area required $= \frac{2349 \times 12}{5 \times \frac{7}{8} \times 17000} = 0.379$ □" per ft strip Use ½"□ bars 6" centres $= 0.390$ □"

End shear



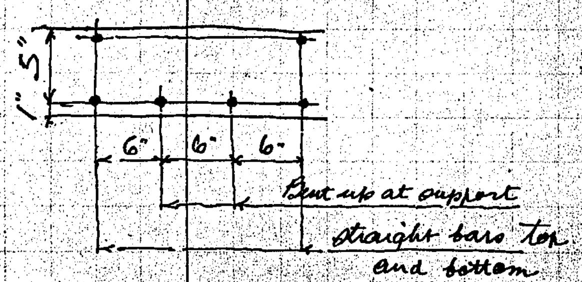
Reaction $= 8800 \cdot \frac{4.34}{4.17} = 9150$ #/c

For one ft strip $= 9150 \div 3.5 = 2610$

Assumed distribution DL say $97 \times 2.08 = 200$

2810#

Unit shear $= \frac{2810}{\frac{7}{8} \cdot 5 \cdot 12} = 53.5$ #/□" ok



増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Steel Stringer A span length $20'-4\frac{3}{4}''$

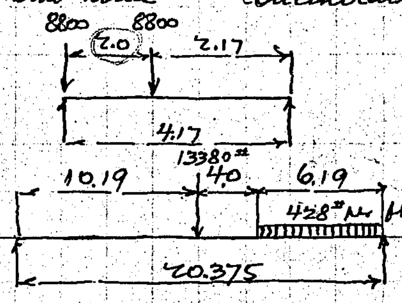
Dead Load Slab and pavement $97 \times 4.17 = 405$

Stringer assumed 60

$465^{\#}$

Dead Load $M = \frac{1}{8} \times 465 \times 20.375^2 = 24100^{1\#}$

Live Load Concentration rear wheel of motor trucks with impact $= 8800^{\#}$



Rear wheel $8800 \times \frac{2.17}{4.17} = 4580$

8800

$13380^{\#}$

Uniform Live Load $= 102.5 \times 4.17 = 428^{\#}$ per lin ft.

Live Load Moment motor trucks $13380 \times 10.19 = 68000$

" " " Uniform $\frac{428 \times 6.19^2}{2 \times 20.375} \times 10.19 = 4030$

Total $m = 72030^{1\#}$

Dead Load moment 24100

$96130^{1\#}$

Section modulus required $= \frac{96130 \times 12}{17000} = 68.0$ Use $18'' I \times 54.7^{\#}$ $S_m = 88.4$

Unit stress $= 17000 \times \frac{68}{88.4} = 13100^{\#}/\square''$ ok

Steel Stringer B span length $20'-4\frac{3}{4}''$

Dead Load Slab and pavement $97 \times 4.17 = 405$

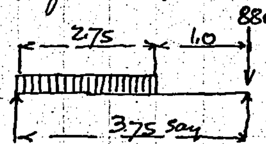
stringer assumed 45

$450^{\#}$

$DLm = \frac{1}{8} \times 450 \times 20.375^2 = 23300^{1\#}$

Uniform Live Load same as for stringer A. moment $= 4030$

Uniform Live Load on side of motor trucks loading



Reaction $= \frac{102.5 \times 2.75^2}{2 \times 3.75} = 103$ moment $= \frac{1}{8} \times 103 \times 20.375^2 = 5330$

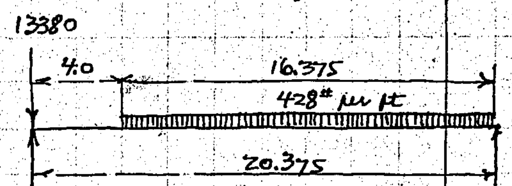
Rear wheel concentration moment $= \frac{8800}{2} \times 10.19 = 44800$

$77460^{1\#}$

Section modulus required $= \frac{77460 \times 12}{17000} = 54.6$

Use $15'' I \times 42.9^{\#}$ $S_m = 58.9$ Unit stress $= 17000 \times \frac{54.6}{58.9} = 15800^{\#}/\square''$

Max End shear for stringer A.

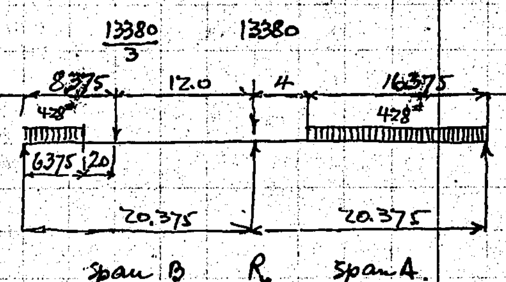


Uniform Load $\frac{428 \times 16.375^2}{2 \times 20.375} = 2820$

wheel concentration 13380

$16200^{\#}$ for single shear

Reaction $R = \frac{428 \times 6.375^2}{2 \times 20.375} =$ say 430



$\frac{13380 \times 8.375}{3 \times 20.375} = 1830$

2260

16200

For bearing $18460^{\#}$

weight of stringers.

1 I 18" × 54.7# × 20.3	=	111.0	
4 Ls 5 × 3½ × 3/8 @ 10.4 × 1.0	=	42	
Rivet heads + variation say		10	
		1162 × 2	= 2324
1 I 15" × 42.9 × 20.3	=	870	
4 Ls 5 × 3½ × 3/8 @ 10.4# × 1.0	=	42	
Rivet heads + variation say		10	
		922 × 2	= 1844
			4168# say 4170#

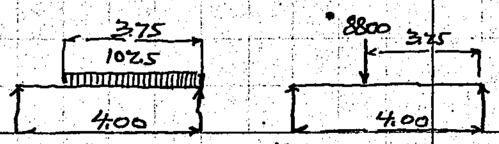
$4170^{\#} \div 20.375 = 205^{\#}$ per lin. ft of span

Fascia Girder C span length 20'-4 3/4"
Reinforced concrete

Assumed dead load Handrail say 30
Floor say $97 \times \frac{3.75}{2} = 182$
Beam assumed 250
462#

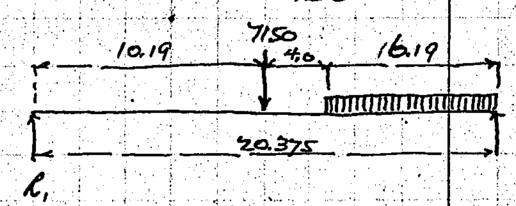
Dead Load Moment $= \frac{1}{10} \times 462 \times 20.375^2 = 19100$ 1#

Live Load



Uniform load $\frac{102.5 \times 3.75^2}{2 \times 4.00} = 180$#

motor trucks $8800 \times \frac{3.25}{4.00} = 7150$#



Reaction $R_1 = \frac{180 \times 6.19^2}{2 \times 20.375} = 169$#

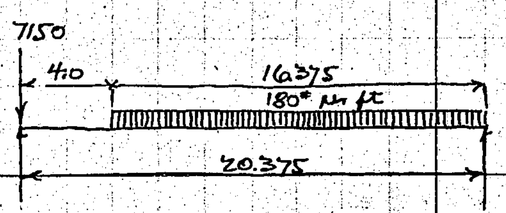
Live Load moment unif. $169 \times 10.19 = 1720$ 1#
For continuity of beam $1720 \times 0.8 = 1380$ 1#
Live Load moment conc. $\frac{7150}{2} \times 10.19 = 36400$
For continuity of beam $36400 \times 0.8 = 28200$
Dead Load moment 19100
Total Dead and Live Load Moment − 48680 1#

Effective Depth required at support $= \sqrt{\frac{48680}{129.5}} = 19.4"$ for $f_c = 750$#/□" $f_s = 17000$#/□"

Effective Depth required at center $= \sqrt{\frac{48680}{102}} = 21.8"$ for $f_c = 640$#/□" $f_s = 17000$#/□"

Make depth of beam 24" over all making effective depth 22".
Required steel area $= \frac{48680 \times 12}{\frac{7}{8} \times 22 \times 17000} = 1.78$ □" Use 4-3/4"□ bars = 1.76 □"

End shear



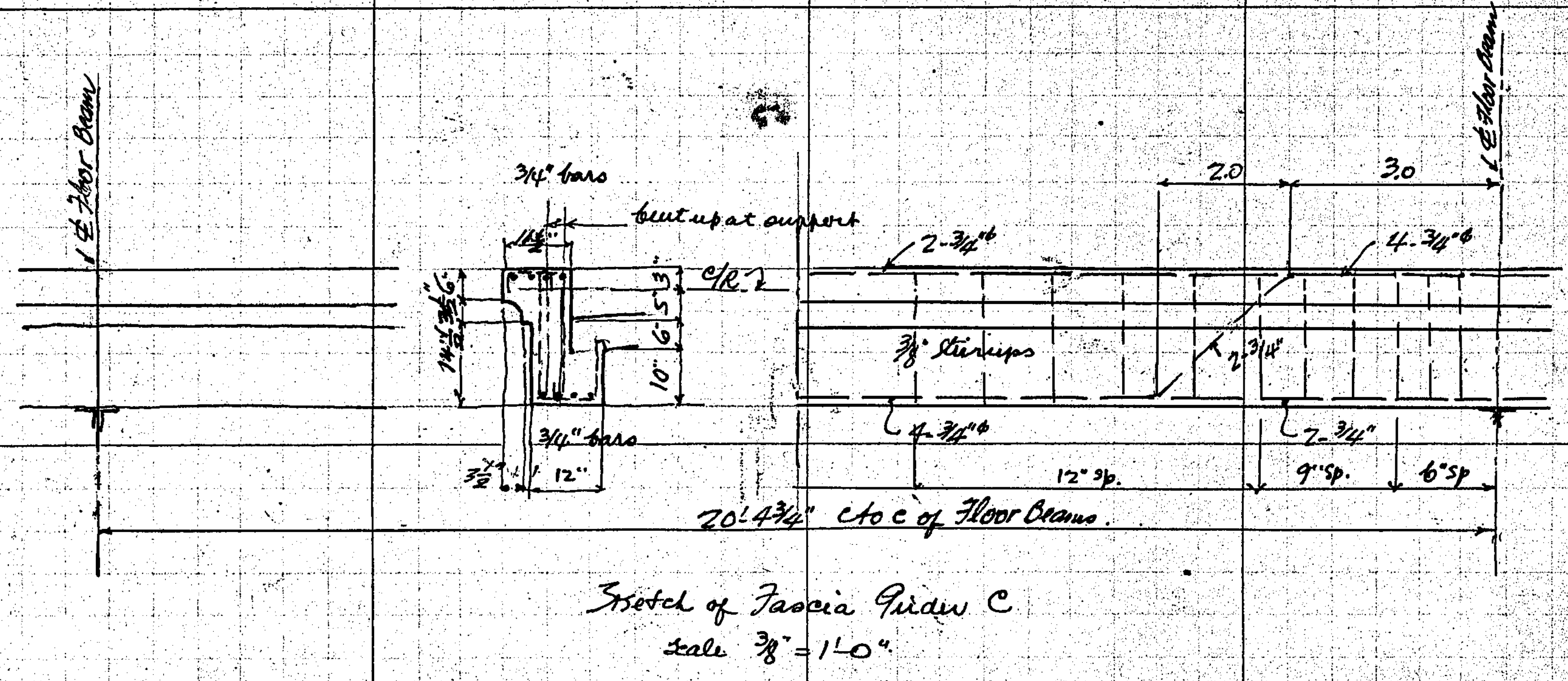
End shear unif. load $\frac{180 \times 16.375^2}{2 \times 20.375} = 1190$#
wheel concentration 7150
Live Load shear 8340
Dead Load shear $462 \times \frac{20.375}{2}$ 4700
13040#

Due to continuity of beam End shear say $13040 \times \frac{5}{4} = 16300$#

Approximate Unit shear $= \frac{16300}{\frac{7}{8} \times 22" \times 10" \text{att}} = 84.5$#/□"

Shear for Plain concrete 57.0
27.5#/□" for stirrups.

3/8" stirrups good for $.22 \times 10.000 = 2200$# stirrup spacing $= \frac{2200}{27.5 \times 10} = 8"$ at end



Sketch of Fascia Girder C
Scale 3/8" = 1'-0"

Intermediate Floor Beam span length 23.0' spacing 20'-4 3/4"

Dead load

weight of Fascia Girder C.

Stem	.58 × 2.0	=	1.16
Coping	.50 × .42	=	.21
Filler say		=	.07
Bottom	.83 × .42	=	.35
			1.79 @ 150 = say 270

Handrail say 30

Floor slab + Pavement $97 \times \frac{3.75}{2}$ = 182

482# × 20.375 = 9800#

Concentration at Stringer B

Pavement and slab 97 × 3.93 = 381

Steel stringer say 45

426 × 20.375 = 8700#

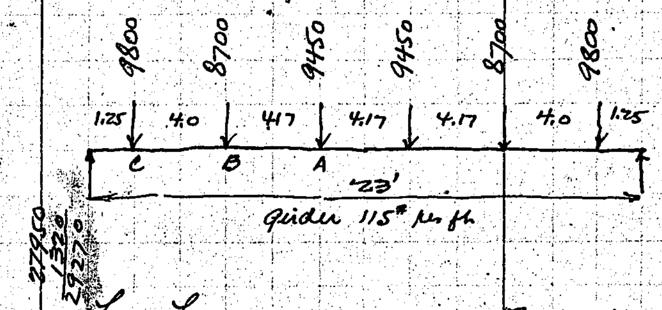
Concentration at stringer A

Pavement and slab 97 × 4.17 = 405

Steel stringer say 60

465 × 20.375 = 9450#

Dead Load moment



moment at A

27950 × 9.42 = 263500

8700 × 4.17 = 36300

9800 × 8.17 = 80000

= −116300

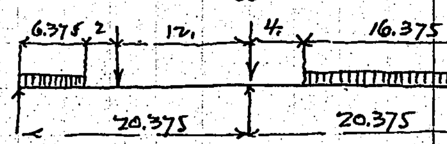
$\frac{1}{8} \times 115 \times 23^2$ = 147200#

7600

154800#

Live Load moment 2 motor trucks on roadway with occupied space of 16'; Uniform load 2' wide on both sides of motor trucks

2930 8800

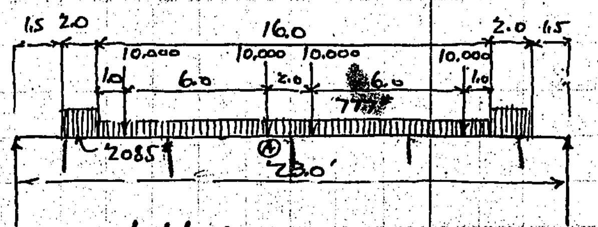


Reaction due to uniform load $\frac{102.5 \times 16.375^2}{2 \times 20.375}$ = 675

$\frac{102.5 \times 6.375^2}{2 \times 20.375}$ = 102

777# per lin ft

For full uniform load 102.5 × 20.375 = 2085# " "



Reaction due to wheel Conc. $2930 \times \frac{8.375}{20.375}$ = 1205

8800

10005#

call this 10.000#

20.000 wheel load

6200 unif.

4170

10370 total

Live Load Moment at (A)

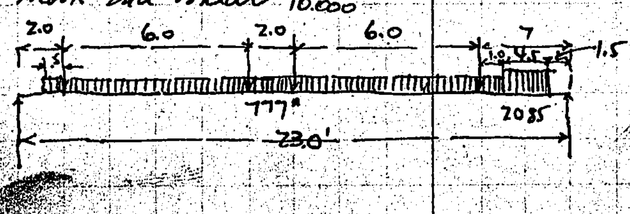
wheel load	20.000 × 10.5	=	210.000	
	10.000 × 6.0	=	−60.000	
				150.000
Unif. load	6200 × 11.5	=	71400	
	$777 \times \frac{8^2}{2}$	=	−24900	
				46500
Unif load	4170 × 11.5	=	48000	
	2085 × 9	=	−18750	
				29250

Live Load Moment 225750

Dead Load Moment 154800

Total 380550#

Max End shear



Unif. load $777 \times 15.5 \times \frac{13.75}{23.0}$ = 7200#

$2085 \times 4.5 \times \frac{3.75}{23.0}$ = 1530

Trucks. $10.000 \times \frac{56}{23}$ = 24400

33130#

Dead Load End shear 29270
Live Load End shear 33130
62400 #

Section for Intermediate Floor Beam
Moment = 380550 '# End shear = 62400 # web section required for shear = 6.24 □" gross.
web assumed 30 × 5/16 = 9.38 □" ⅛ web = 1.18 □" 30½" b to b of Ls
Effective depth = 2.54 - .24 = 2.30 Stress in flange = 380550 ÷ 2.30 = 166,000 #
section required = 166.000 ÷ 17.000 = 9.76 - 1.48 = 8.50 net
Use 2Ls 5 × 5 × ½" = 9.5 □" gross or 8.5 □" net

No of Rivet for End conn. 62400 ÷ 4810 = 13.0 Single Shear
62400 ÷ 4375 = 14.3 5/16" Pl. bearing. field

Allowable stress in compression Flange = $17.000\,(1 - 0.012\,\frac{50}{10\,5/16}) = 16000$ #
166000 ÷ 16000 = 10.40 - 1.48 = 8.92 □" ok.

weight of one intermediate Floor beam

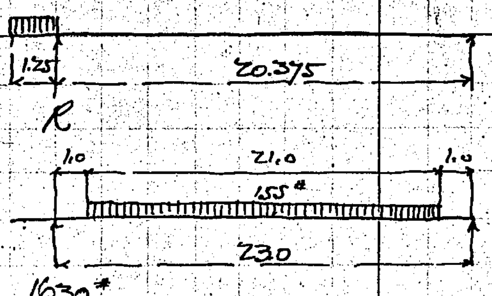
web.	30 × 5/16	@ 31.88	× 21.89	=	697
Flanges	4Ls 5 × 5 × ½"	@ 16.2	× 21.89	=	1415
Stiffeners	14Ls 4 × 3 × 5/16	@ 7.2	× 2.54	=	256
End Stiffs	4Ls 3½ × 3½ × ⅜	@ 8.5	× 2.46	=	84
Fills	4Pls 6½ × ½	@ 11.05	× 1.70	=	75
Fills	8Pls 7½ × ½	@ 12.75	× 1.12	=	114
	Rivet heads + variation				129
					2770 #

End Rivet Pitch
$\frac{5470 \times 24.75}{62400} = 2.17"$
$2770 \div \overset{21.89}{\cancel{23}} = 126.5$ # per ft
$2770 \div 20.375 = 136.0$ # per ft

End Floor Beam Span length 23'-0"
Floor slab beyond end floor beam will be over hanged without extension of stringers; Fascia Girders C also be extended to the end of span
Projection of slab 1'-3"
Dead Load = 97 # per sq ft moment = $97 \times \frac{1.25^2}{2} = 76$ '#
Live Load concentration with impact = 8800 # distribution of this load assumed 2' × 2' or
2200 # per sq ft Live Load moment = $2200 \times \frac{1.25^2}{2} = 1720$
Dead Load moment = 76
1796 '#

Steel area required = $\frac{1796 \times 12}{\frac{7}{8} \times 5 \times 17000} = .29$ □" ½"ϕ bars 8" centers = 0.29 □"

End Floor Beam
Dead Load of cantilever slab with pavement including fascia girders rc say 120 #/□

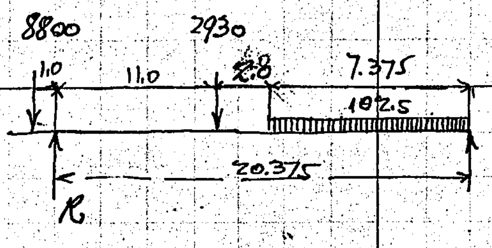


Reaction R = $120 \times 1.25 \times \frac{21.0}{20.375} = 155$ # per ft

Moment = 1630 × 11.5 = 18700
$155 \times \frac{10.5^2}{2}$ = - 8550
10150
½ Dead Load Moment intermediate F.B. 73600
83750 #

1630 #
14000 ½ F.B.
15630

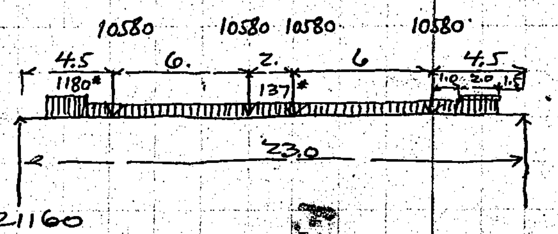
Live Load



Uniform load R = $\frac{102.5 \times 7.375^2}{2 \times 20.375} = 137$ #

wheel load R = $8800 \times \frac{21.375}{20.375} = 9230$
$2930 \times \frac{9.375}{20.375} = 1350$
10580

Full Unif. load R = $102.5 \times \frac{21.625^2}{20.375 \times 2} = 1180$ #



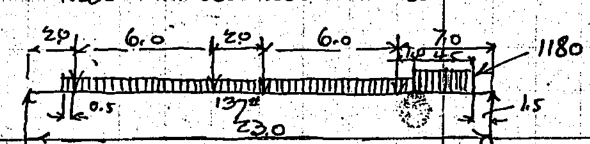
21160
1100
2360.

Moment due to wheel load	21160 × 10.5	= 222000	
	10580 × 6	= −63500	
			158500 1#
moment Unif. load	1100 × 11.5	= 12650	
	$137 \times \frac{8^2}{2}$	= −4380	
			8270
moment Unif. load	2360 × 11.5 =		
	2360 × 9 =		5900
	Total live load moment		172670 1#

Summary of moments

Dead Load moment	83750	
Live Load Moment	172670	
		256420
DL. Floor Beam	$= \frac{1}{8} \times 115 \times 23^2 =$	7600
		264020 1#

Max End Shear



Uniform load	$137 \times 15.5 \times \frac{13.75}{23.0}$	= 1280	
Unif. load	$1180 \times 4.5 \times \frac{3.75}{23.0}$	= 870	
motor trucks	$10580 \times \frac{56}{23}$	= 25800	
Live Load			27950 #
Dead Load Floor			15630
" Floor Beam	115 × 11.5 =		1320
			44900
		call this	45000 #

Section of End Floor Beam

Moment = 264000 1# Shear = 45000# Try 30 × 5/16" web = 9.38 □" 1/8 web = 1.18 □"

30½" b to b of L's Effective depth = 2.54 − .14 = 2.4'

stress = 264000 ÷ 2.4 = 110,000 # Section req'd = 110,000 ÷ 17000 = 6.47 − 1.18 = 5.29 □" net

Use 2L's 5 × 3½" × 3/8" = 6.10 □" gr. or 5.35 □" net

Rivet spacing at end = $\frac{5470 \times 26.5}{45000}$ = 3.22"

Weight of one end Floor Beam

web	30 × 5/16	@ 31.88	× 21.89	=	698
Flanges	4L's 5 × 3½ × 3/8	@ 10.40	× 21.89	=	910
Stiffeners	14L's 4 × 3 × 5/16	@ 7.2	× 2.54	=	256
End Stiffs	4L's 3½ × 3½ × 3/8	@ 8.5	× 2.46	=	84
Fills	4 Pl's 3½ × 3/8	@ 4.46	× 1.96	=	35
Fills	4 Pl's 7½ × 3/8	@ 9.56	× 1.30	=	50
cov Pl.	1 Pl. 10⅜ × 5/16	@ 11.16	× 21.89	=	254
	Rivet heads & variation say				120
					2407
			call this		2400#

2400 ÷ 21.89 = 110# per ft

Bottom Laterals

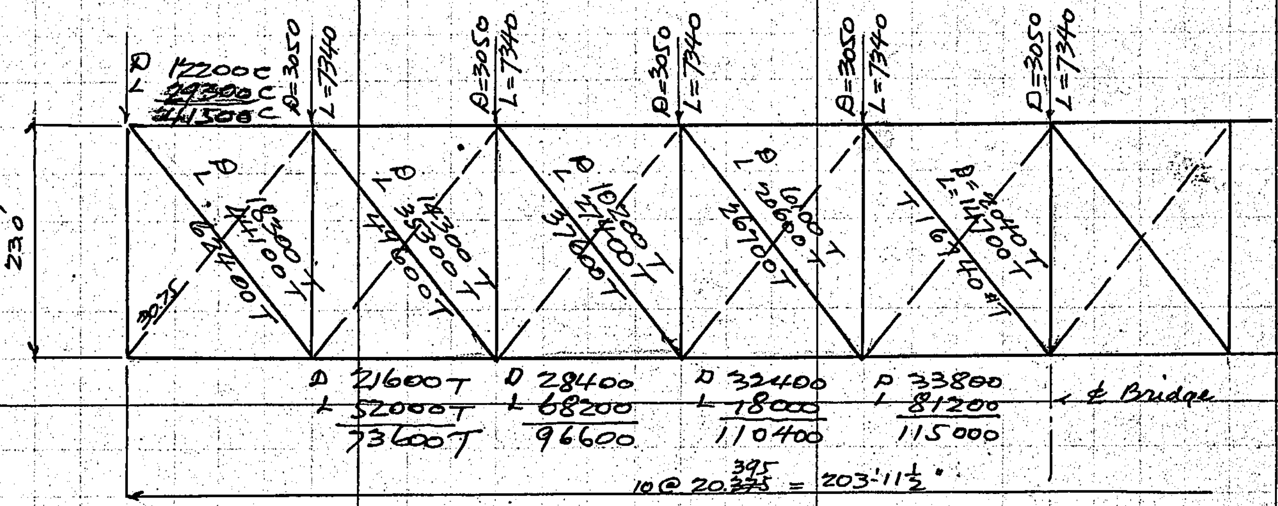
Static wind load 30#/□' of exposed surface

Assume 5 × 30 = 150# per lin. ft. panel concentration = 150 × 20.375 = 3050#

Moving wind load say 12 × 30 = 360# per lin ft. panel conc = 360 × 20.375 = 7340#

$\tan\theta = 20.375 \div 23.0 = .885$	$W_1 \tan\theta = 3050 \times .885 = 2700$
$\sec\theta = 30.75 \div 23.0 \quad 1.336$	$W_2 \tan\theta = 7340 \times .885 = 6500$
$W_1 = 3050$#	$W_1 \sec\theta = 3050 \times 1.335 = 4070$
$W_2 = 7340$#	$W_2 \sec\theta = 7340 \times 1.335 = 9800$

Final Design of Furukawa-bashi for Tokushimaken



Section of bottom laterals.

	stress	¾" Rivet = 4420#	S.R. 17000#/□"
1st panel	62400# T	14.1	3.66 □" net
2nd	49600	11.2	2.92
3rd	37600	8.5	2.21
4th	26700	6.0	1.57
5th	16740	3.8	.98

1st Panel Stress = 62400# T

Try 2Ls 4×3×5/16 = 4.18 □" or 3.64 □" net

Unsupported length 9.5' = 114" r = 1.28 l/r = 89

Unit stress = 21300 (1 − 0.0055 l/r) = 10850#/□" @ 4.18 = good for 45300#

Use 10 - ¾" rivets for connections.

2nd Panel Use same section as for 1st panel.

3rd Panel Stress = 37600#

Try 2Ls: 3×2½×5/16 = 3.24 □" gr or 2.70 □" net

Use 6 rivets for connection

4th and 5th panels same as for 3rd Panel.

Weight of Bottom Lateral Bracing

2Ls 4×3×5/16	@ 7.2 × 28.6	=	411	
4Ls 4×3×5/16	@ 7.2 × 14.0	=	402	
4 hangers @ 28.#		=	112	
Center connection			50	
misc say			10	
			985 × 4	= 3940
2Ls 3×2½×5/16	@ 5.6 × 28.6	=	320	
4Ls 3×2½×5/16	@ 5.6 × 14.0	=	313	
4 hangers @ 28		=	112	
Center connection			40	
misc say			10	
			795 × 6	= 4770
				8710#

call this 9000# per span

Details of Hanger

1 Fill 6×½	@ 10.20 × 0.6	=	6.1
1L 6×3½×3/8	@ 11.7 × 1.25	=	15.0
1L 4×3×5/16	@ 7.2 × 0.6	=	4.3
	variation		2.6
			28.0#

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Upper Lateral.

Static wind pressure 30#/□' assumed panel load say 3030#

For diagonal use 4Ls stiff section there is no need to figure stresses.

Unsupported length say 14.0 on sideway

Try 4Ls 3 × 2½ × 5/16 = 2 × 3.24□" r = 1.50 about l/r = 112. Stress = 8150 × 3.24 = 26600

26600

For 4Ls — 53200

No of rivet 26600 ÷ 4420 = 6. Use 6 rivets top and bottom

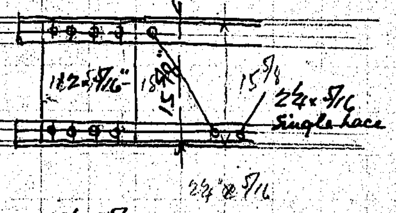
Weight of Diagonals

Diagonals	4Ls 3 × 2½ × 5/16 @ 5.6	× say 28.6	=	642
	8Ls do	say 14.0	=	628
	7 tie P/s @ 18		=	126
	Lacing bars 2¼ × 5/16 5# × 48		=	240
	misc detail say			44
				1680# per panel

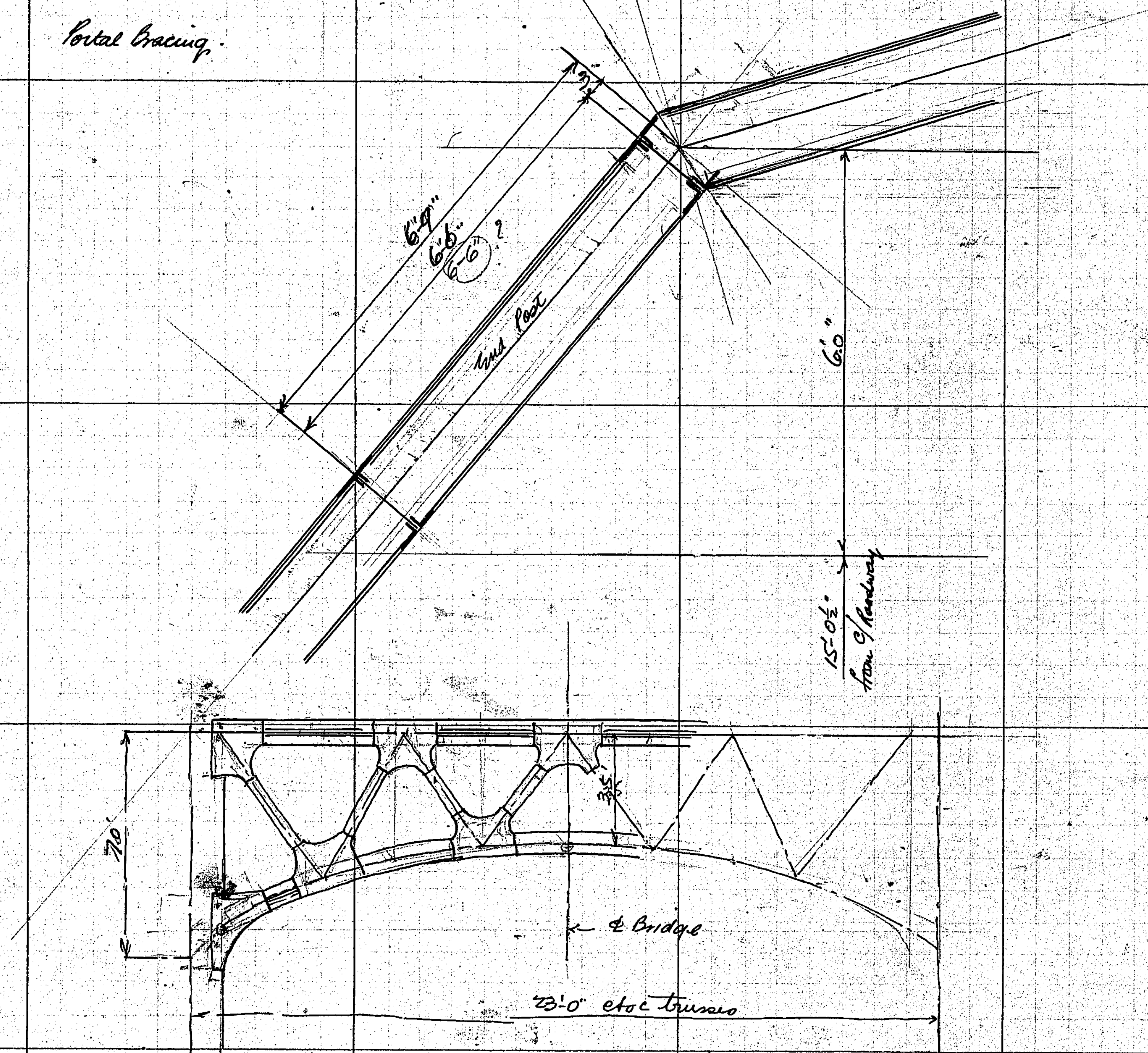
8 panels @ 1680 = 13440 # per span

2¼ × 5/16 @ 2.4 × 1.5 = 3.6#

3.6 ÷ .75 say 5#" per length



Portal Bracing.



Final Design of Furukawa Bashi, Tokushimaken

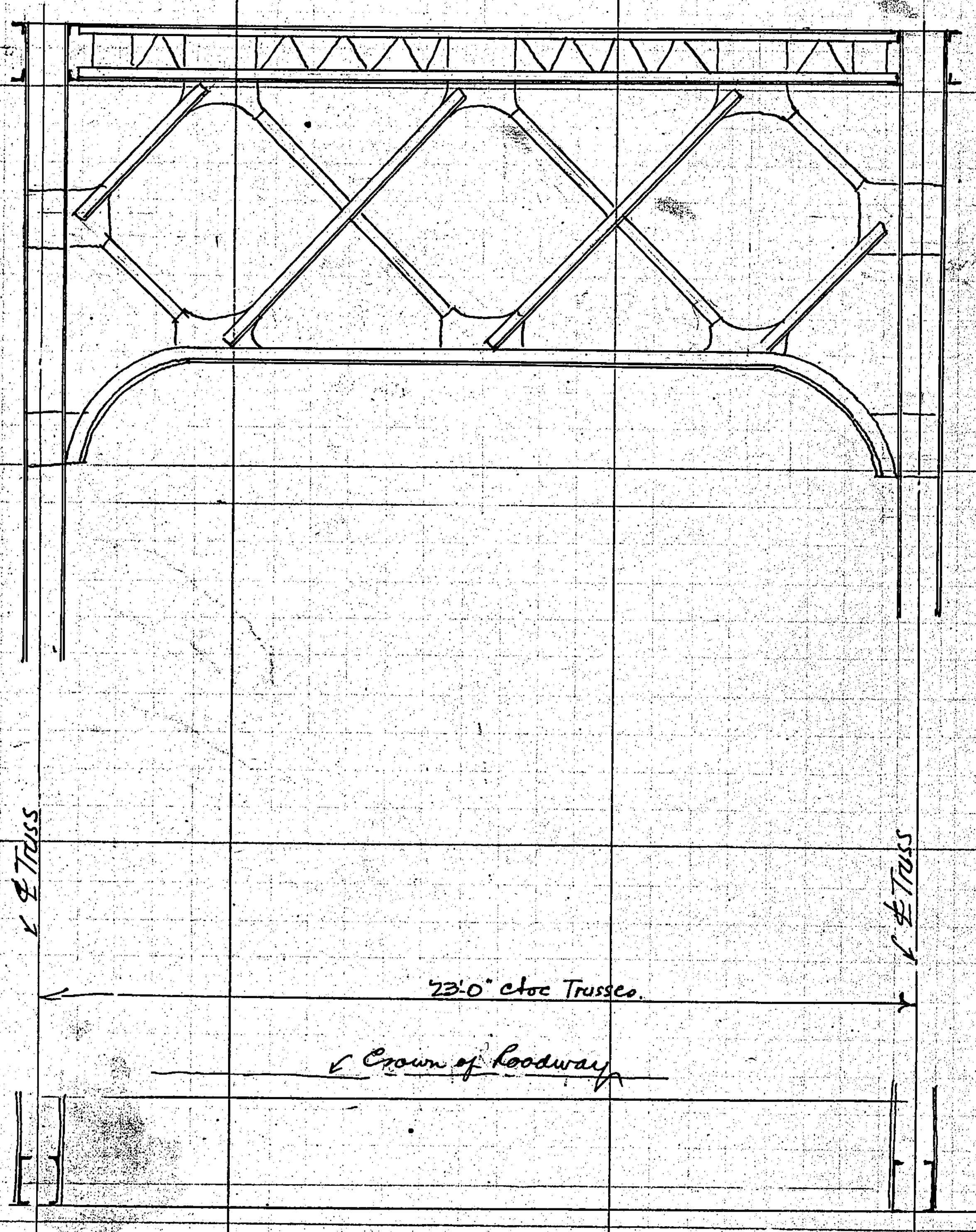
Portal Bracings.

weight of portal Bracing.

Top strut	4Ls	4 × 3 × 5/16	@ 7.2 ×	21.4	=	615
Bottom Strut	4Ls	4 × 3 × 5/16	@ 7.2 ×	21.8	=	628
web pls	1Pl.	15 × 5/16	@ 15.94 ×	21.4	=	341
stiffeners	3 Ls	2½ × 2½ × 5/16	@ 5.0 ×	1.25		19
web pl	1Pl	15 × 5/16	@ 15.94 ×	21.8	=	347
Stiffeners	4Ls	2½ × 2½ × 5/16	@ 5.0 ×	1.25	=	25
Connection Pls	7 @	27#			=	189
do ends	8 @	30			=	240
Diagonals.	1L	3 × 3 × 5/16 Lu	@ 6.1 ×	29.0'	=	177
		Rivet heads + variation say				119
						2700#

2 Portals @ 2700 = 5400# per truss

Sway Bracings -



Weight of sway bracing

Top strut	4Ls 4 × 3 × 5/16	@ 7.2# × 21.4	=	615
Tie Pls	2Pls 15 × 5/16	@ 15.94 × 1.0	=	32
Lacing bars	19' @ 5#			95
Bottom struts	2Ls 4 × 3 × 5/16	@ 7.2 × 24.0	=	345
Diagonals	1L 3 × 3 × 5/16	@ 6.1 × 54.0	=	330
Conn.	3 @ 60			180
"	5 @ 30			150
"	2 @ 40			80
		Rivet heads + variation		50
				1877 call this 1880#

7 @ 1880# = 13160#

Summary for top bracings

Diagonals	13440
Portals	5400
Sway bracings	13160

32000# per one span
or say 157# per lin ft of span

Design of 203'-11½" truss
Panel concentration -
Lower Panel

2 Lines of handrail assumed	= 60#	
Stringers	205#	
Floor Beam intermediate	136	
Lower Laterals	44	
Trusses Lower half assumed	430	
		875

Roadway

Pavement	22# × 20	=	440	
Concrete slab	75# × 20	=	1500	
Fascia girders	2 @ 270#	=	540	
Filler etc say			70	
				2550

3425# ÷ 2 = 1712.5# per truss

Lower Panel Concentration = 1712.5 × 20.395 = say 35000#

Upper Panel

Upper Laterals	157
Trusses upper half assumed	430

587 ÷ 2 = 293.5# per truss

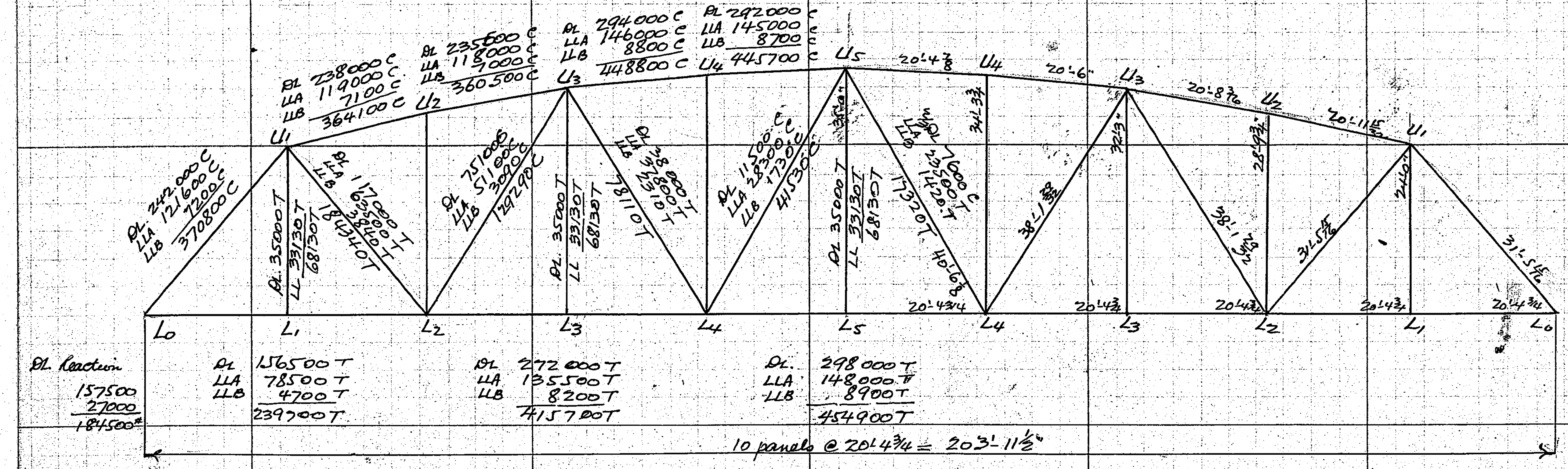
Upper Panel Concentration = 293.5 × 20.395 = say 6000#

Live Load
Uniform Live Load assumed 100#/□' combined with motor truck loading; Impact assumed 25% for motor truck

To figure the live load stresses in truss 2 systems of live load considered
(A) 2 motor trucks side by side with occupied width of 16' preceded and followed by uniform load of 100#/□'
(B) 4' strip of uniform load 100#/□'

Final Design of Furukawa-Bashi for Tokushimaken

Stresses in Truss.



Points of Intersection of both chords

$U_5 \quad 20.396 \times \frac{35}{.6875} = 1038.3$

$U_4 \quad 20.396 \cdot \frac{34.31}{2.06} = 339.7$

$U_3 \quad 20.396 \times \frac{32.25}{3.44} = 191.2$

$U_2 \quad 20.396 \cdot \frac{28.81}{4.81} = 122.1$

Length of Diagonals & Top chords

$U_5-L_4 \quad 40'-6\frac{7}{8} = 40.51$

$L_2-U_3-L_4 \quad 38'-1\frac{29}{32} = 38.16$

$L_0-U_1-L_2 \quad 31'-5\frac{15}{16} = 31.49$

$U_5-U_4 \quad 20'-4\frac{7}{8}$

$U_4-U_3 \quad 20'-6"$

$U_3-U_2 \quad 20'-8\frac{3}{16}"$

$U_2-U_1 \quad 20'-11\frac{15}{32}"$

Lever arm for top chords

$U_5-U_4 \quad 1017.9 \times \frac{34.31}{1018.5} = 34.30$

$U_4-U_3 \quad 34.31 \times \frac{339.7}{344.4} = 34.05$

$U_3-U_2 \quad 28.81 \cdot \frac{170.8}{173.2} = 28.40$

$U_2-U_1 \quad 28.81 \cdot \frac{122.1}{125.4} = 28.10$

Lower Panel concentration = 35000#

Upper Panel concentration = 6000#

Lever Arm for Diagonals.

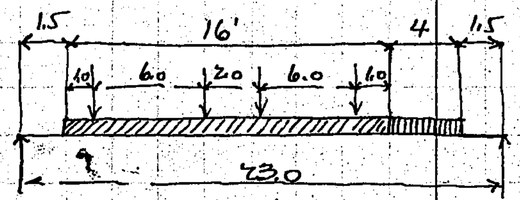
$U_5-L_4 \quad 1017.9 \times \frac{35.0}{40.51} = 879.4$

$U_3-L_4 \quad 339.7 \times \frac{32.25}{38.16} = 287.0$

$U_3-L_2 \quad 170.8 \times \frac{32.25}{38.16} = 144.4$

$U_1-L_2 \quad 122.1 \times \frac{24}{31.49} = 93.1$

Live Load on truss



Uniform load 100#/0'

Live Load A 1600 × $\frac{13.5}{23.0}$ = 940#

Live Load B 400 × $\frac{3.5}{23.0}$ = 60#

1000#

Motor trucks loading

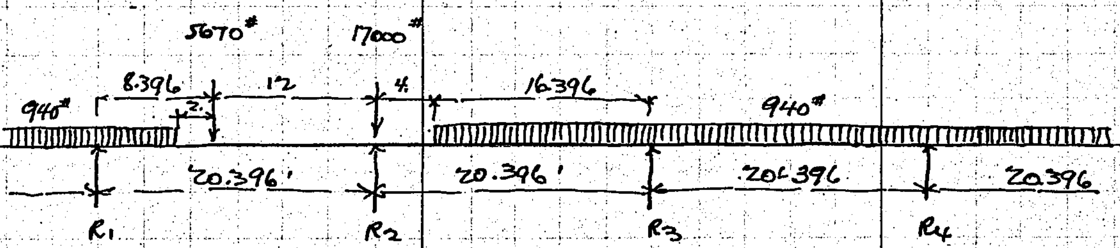
RA.

	Rear wheel	Front wheel
	6600	2200
25% impact	1650	550
	7250	2750

Reaction RA. 4 × 7250 × $\frac{13.5}{23.0}$ = 17000# for Rear 4 × 2750 × $\frac{13.5}{23}$ = 5670# for Front wheel

Live Load A

Panel concentration as shown in sketch below for chord members and webs.

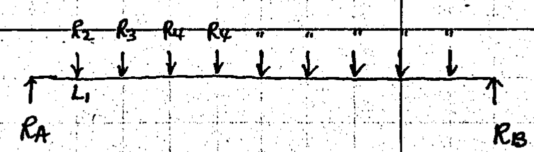


	R_1	R_2	R_3	R_4	
Unif. load	9600				} Omitted for diagonals
" "	5060	940			
Front wheel	3330	2340			
Rear wheel		17000			
Unif. load		6200	9200		
Unif. load			9600	9600	
Unif. load				9600	
	17990#	26480#	18800#	19200#	for chord members
	3330	25540#	18800	19200	for diagonal members

Live Load B Uniform load 60# per lin ft. Panel concentration = 60 × 20.396 = 1225#

Lower chord L_0-L_2 and End Post L_0-U_1

R_2 loading at L_1



RA :– R4 19200 × 2.8 = 53750

R3 18800 × 0.8 = 15050

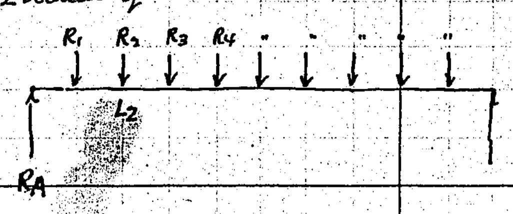
R2 26480 × 0.9 = 23800

L_0-L_2 92600 × $\frac{20.396}{24}$ = 78500#

L_0-U_1 – 92600 × $\frac{31.49}{24}$ = 121600#

Diagonals U_1-L_2

R_2 loading at L_2



R4 19200 × 2.1 = 40300

R3 18800 × 0.7 = 13150

R2 25540 × 0.8 = 20400

R1 3330 × 0.9 = 3000

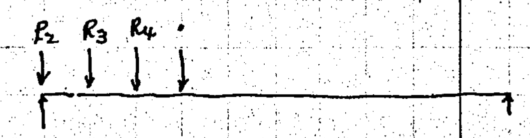
76850 × 81.3 = 6250.000

3330 × 101.7 = 338.000

5912.000

Stress = 5912000 ÷ 93.1 = 63500#

Max Live Load on shoe.
Live Load A.
R_2 loading at panel point Lo



R4 19200 × 3.6 = 69100
R3 18800 × 0.9 = 16900
R2 26480 × 1.0 = 26480
112480 #
Live Load B. 1225 × 4.5 5500
117980
Call this 118 000 #
D.L. 211500
load on shoe 329500 #
2500 #
For bearing 332000 #

Dead Load shear at Lo 184500 #
Panel load at Lo. say 27000 # approximate only
201500
weight of shoes say 2500
For bearing 214 000 #

Sections of truss members.
Top chord section
U3-U4-U5 max stress = 448800 # C
2[15" × 40 # = 23.40 Radius of gyration hor axis 5.76
1 Cov Pl 22" × ½" = 11.00 " " " vert axis 7.52
34.40 □"

Unsupported length of member = 246"
Allowable unit stress = 14000 #/□" section required = 448800 ÷ 14000 = 32.0 □" Ok

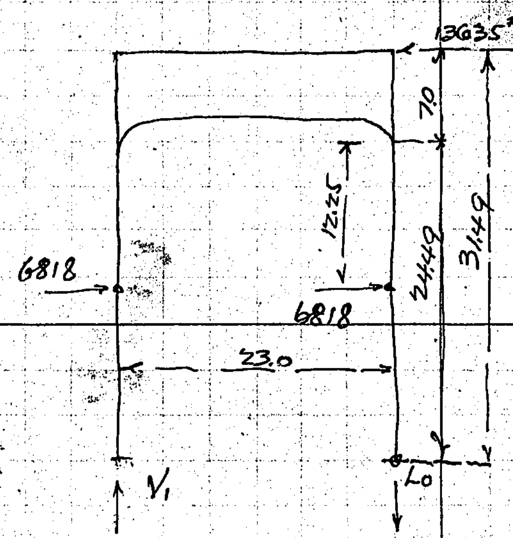
U1-U2-U3 max stress = 364100 # C
2[15" × 33.9 # = 19.80 Radius of gyration hor axis = 5.84
1 cov. pl. 22" × ½" = 11.00 " " " vert axis = 7.56
30.80 □"

Unsupported length = 252" Unit stress = 14000 #/□"
section required = 364100 ÷ 14000 = 26.0 □" Ok.

End Post Lo-U1 max stress = 370800 # C
2[15" × 40 # = 23.40 Radius of gyration hor. axis = 5.76
1 cov. pl 22" × ½" = 11.00 " " " vert axis = 7.52
34.40 □"

Unsupported length = 378"
allowable Unit stress = 21300 (1 − 0.0055 l/r) = 13650 #/□" l/r = 65.5
section required for direct stress = 370800 ÷ 13650 = 27.20 □"

For bending due to wind load. Panel load = 3030 × 4.5 = 13635 #



V1 = 13635 × 31.49/23.0 = 18600 #
370800
389400 #
Assume the point of contra-flexure = 12.25 above panel point Lo
moment = 6818 × 12.25 = 83500 '#
moment of inertia vertical axis = 2011.9 (inch)^4

Bending stress = (83500 × 12 × 11)/2011.9 = 5480 #/□"

Unit stress increased 25%
13650 × 1.25 = 17100 #
5480
11620 #

Section req'd = 389400 ÷ 11620 = 33.50 □" Ok

Bottom chord sections - Unit stress section req'd net

L_0-L_2	stress = 239700 T	17000#	14.10 □"
$L_2-L_3-L_4$	415700 T	"	24.50 □"
$L_4-L_5-L_4'$	454900 T		26.80 □"

$L_0-L_1-L_2$ 2[15" × 33.9# = 19.80 gross   4 holes in flanges 4@.625 = 2.5
4.10   in web 4@.40 = 1.6
15.70 net   4.1

$L_2-L_3-L_4$ 2[15" × 33.9# = 19.80 gross   15.70 net
2 Pls 12 × 7/16" = 10.50   8.75
30.30   24.45 □"

$L_4-L_5-L_4'$ 2[15" × 40# = 23.40   18.80
2 Pls 12 × 7/16 = 10.50   8.75
33.90 gross.   27.55 □" net

Diagonals.

U_1-L_2 Stress = 184340# T   SR = 184340 ÷ 17000 = 10.83 □" net
Try 2[12" × 25# = 14.70 gross − 3.31 = 11.39 □" net

4 - 7/8" rivets in webs 4@ 0.39 = 1.56
4 - 3/4" rivets in flanges 4@ 0.437 = 1.75
3.31

L_2-U_3 Stress = 129290# C
Try 2[12" × 25# = 14.70   r = 4.43   Unsupported length = 457"   l/r = 103.
allowable unit stress = 21300 (1 − 0.0055 × 103) = 9250#/□"
Section required = 129290 ÷ 9250 = 14.00 □" ok

U_3-L_4 Stress = 78110# T   SR = 4.60 □" net
Use 2[12 × 25# = 14.70 gr. − 3.31 = 11.39 □" net ok

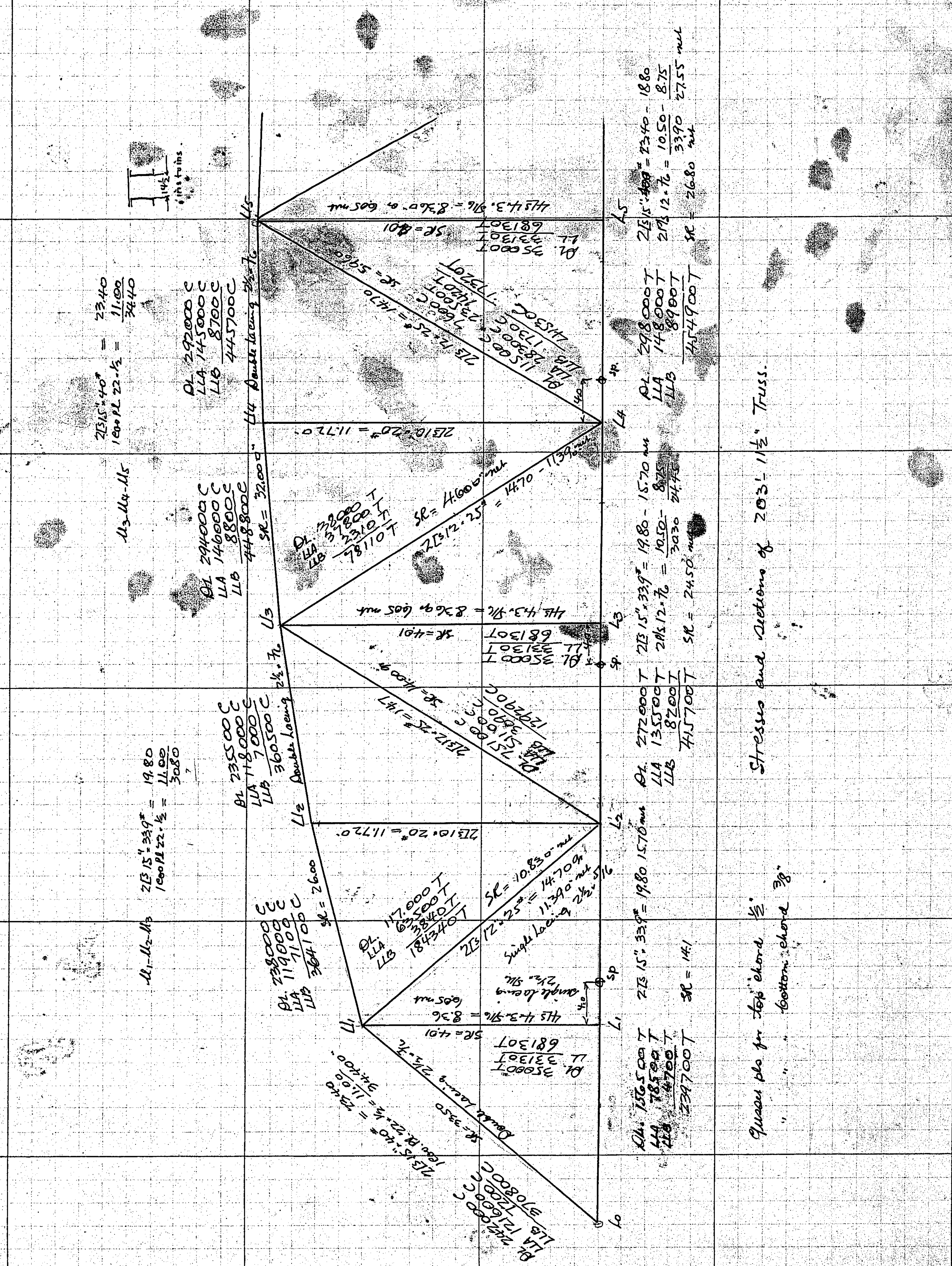
L_4-U_5 Stress = 41530 C
Reversal 8660
50190# C
Try 2[12" × 25# = 14.70   r = 4.43   unsupported length = 486"   l/r = 110.
Allowable Unit stress = 21300 (1 − 0.0055 × 110) = 8420#/□"
Section required = 50190 ÷ 8420 = 5.96 □" ok.

Verticals U_2-L_2 and U_4-L_4
Use 2[10 × 20# = 11.72 □"   r = 3.66   Unsupported length 387   l/r = 106.0 ok

Hangers U_1-L_1 U_3-L_3 U_5-L_5
Total stress = 68130# T   Section req'd 4.01 □" net
Use 4L 4 × 3 × 5/16" = 8.36 □" − 2.31 = 6.05 □" net

7/8" Rivet 4" leg   4 holes @ .312 = 1.25   Unit stress = $\frac{68130}{6.05}$ = 11250#/□" ok
3/4" Rivet   4 holes @ .265 = 1.06
2.31

Final Design of Turukawa bashi for Tokushimaken



Approximate weight of truss

member	section			Length		
L_0-U_1	34.40	@ 3.40	×	31.50	=	3680
U_1-U_2	30.80		×	21.00	=	2200
U_2-U_3	30.80		×	20.70	=	2170
U_3-U_4	34.40		×	20.50	=	2400
U_4-U_5	34.40		×	20.40	=	2390
L_0-L_2	19.80		×	40.80	=	2740
L_2-L_4	30.30		×	40.80	=	4200
L_4-L_5	33.90		×	20.40	=	2350
U_1-L_2		50#	×	31.50	=	1575
L_2-U_3		50#	×	38.20	=	1910
U_3-L_4		50#	×	38.20	=	1910
L_4-L_5		50#	×	40.50	=	2025
U_1-L_1	8.36	4 @ 3.40	×	24.0	=	680
U_3-L_3	8.36		×	32.2	=	910
U_5-L_5	8.36 ÷ 2		×	35.00	=	995
U_2-L_2		40#	×	28.8	=	1150
U_4-L_4		40#	×	34.3	=	1370
						34655#
		Details 38.5%				13345
						48000#

For one truss 96000#
For one span 192000# ÷ 203.96 = 941# per lin ft.

Approximate metal in one span

Stringers	205# × 206.5	=	42400
Inter. Floor Beam	9@2770	=	24900
End Floor Beam	2@2400	=	4800
Bottom Laterals			9000
Top Laterals + sway etc			32000
Trusses			192000

305100#
2 Lines of Handrails @30# = 60 × 206.5 = 12400
317500#
Roadway Pavement and Slabs 2550 × 206.5 = 527000
844500

Max load on pier Dead Load 844500 ÷ 4 = 211000#
Live Load 118000#
329000
weight of shoe 2500
331500#

Assume 6" pin Unit bearing 6 × 24000 = 144000
Thickness of bearing pls = 331500 ÷ 144000 = 2.31"
or 1.16" per rib.

Use ¾" Reinforcing Pls.

Bending Moment = 164500 × 1.5 = 247000"# 6" pin good for 508900"# OK

Load on Bearing - 331500# Try 4½" Rollers Unit stress = 600 × 4.5 = 2700# per inch
331500 ÷ 2700 = 123"
Use 5 rollers 24½" net

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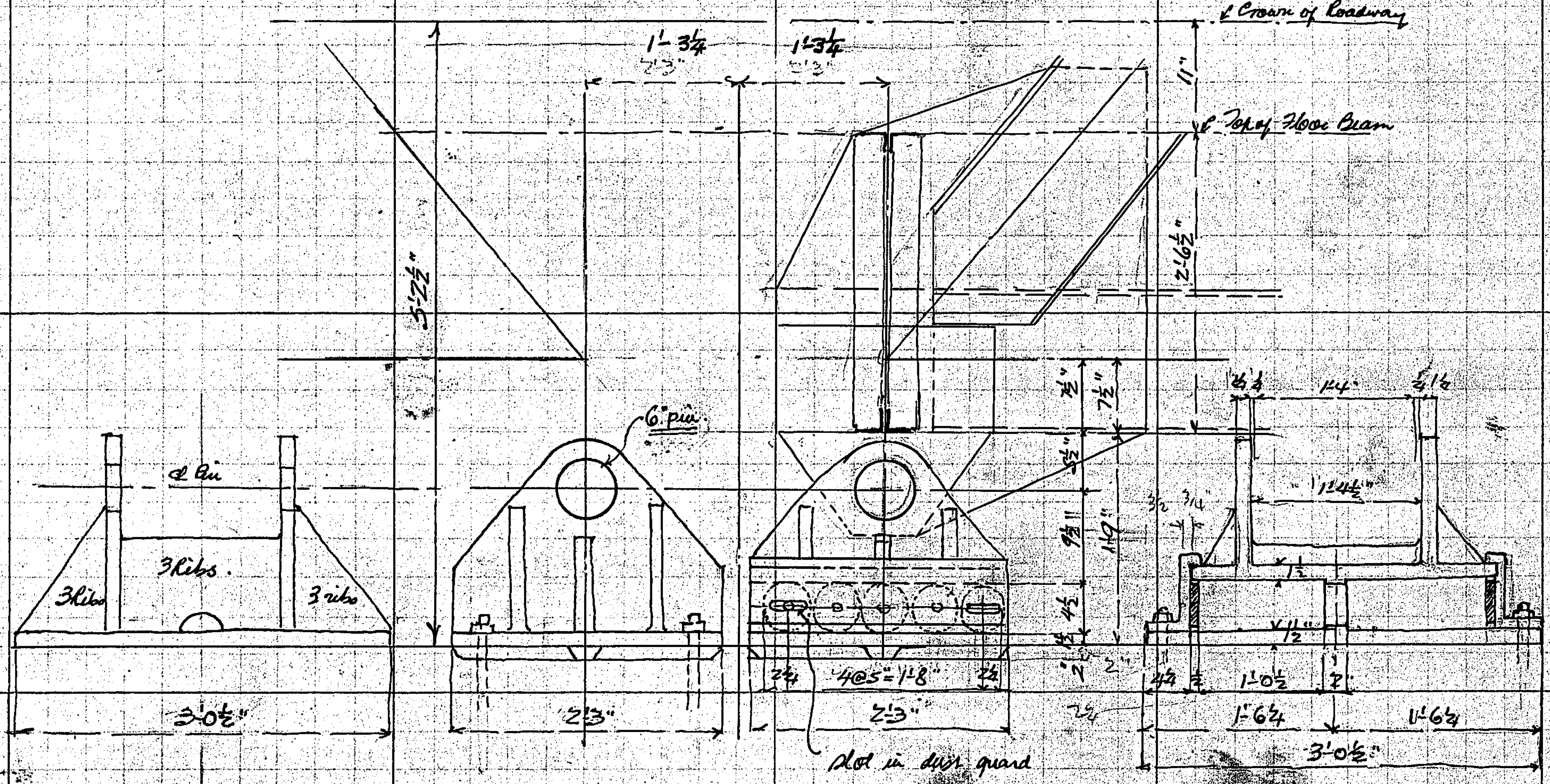
Details of shoe



Fixed Shoe

Expansion shoe

scale ¾" = 1'-0"

Approximate weight of shoe (Expansion Side).

Ribs -	2 -	1¼" × 17" × 12"	=	510 cubic inches	
"				100	
base -		36½" × 27 × 1½"	=	1480	1.89 cubic ft @ 490 = 925#
"		projection say	=	50	
"		28 × 27 × 1½	=	1130	
Rollers	5 @	4½ dia. @ 54" × 2'-3"			610#
spacers	2 @	4½ × ½ × 2'-1" @ 7.65			32
Guards	2 @	8.25° × 3.4 × 2.25 add details say			150
		Pins + other misc details.			250
					1967 call this 2000# per piece.

Approximate weight of Fixed Shoe.

main Ribs - 2 - 1¼ × 27 × 15" = ~~415~~ 1010

3 @ 675 = 2025

base 1480

projection. 50

4560 cubic inch or 2.64 cubic ft @ 490 = say 1300#

Pin and other misc details - 250

1550.

call this 1600#

Weight of shoes Expansion shoes 2 @ 2000 = 4000

fixed shoes 2 @ 1600 = 3200

7200#

7200 ÷ 204 = 35.5# per lin ft

Unit bearing on masonry $\frac{331500}{27 \times 36.5}$ = 335# per sq inch OK

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Approximate metal in one span

Stringers	42400	47000
Int. Floor Beam	24900	28900
End Floor Beam	4800	10550
Bottom Laterals	9000	30750
Top Laterals sway etc	32000	
Trusses	192000	
Shoes	7200	
	312300# call this 140 tons.	

17 spans @ 140 = 2380 tons.

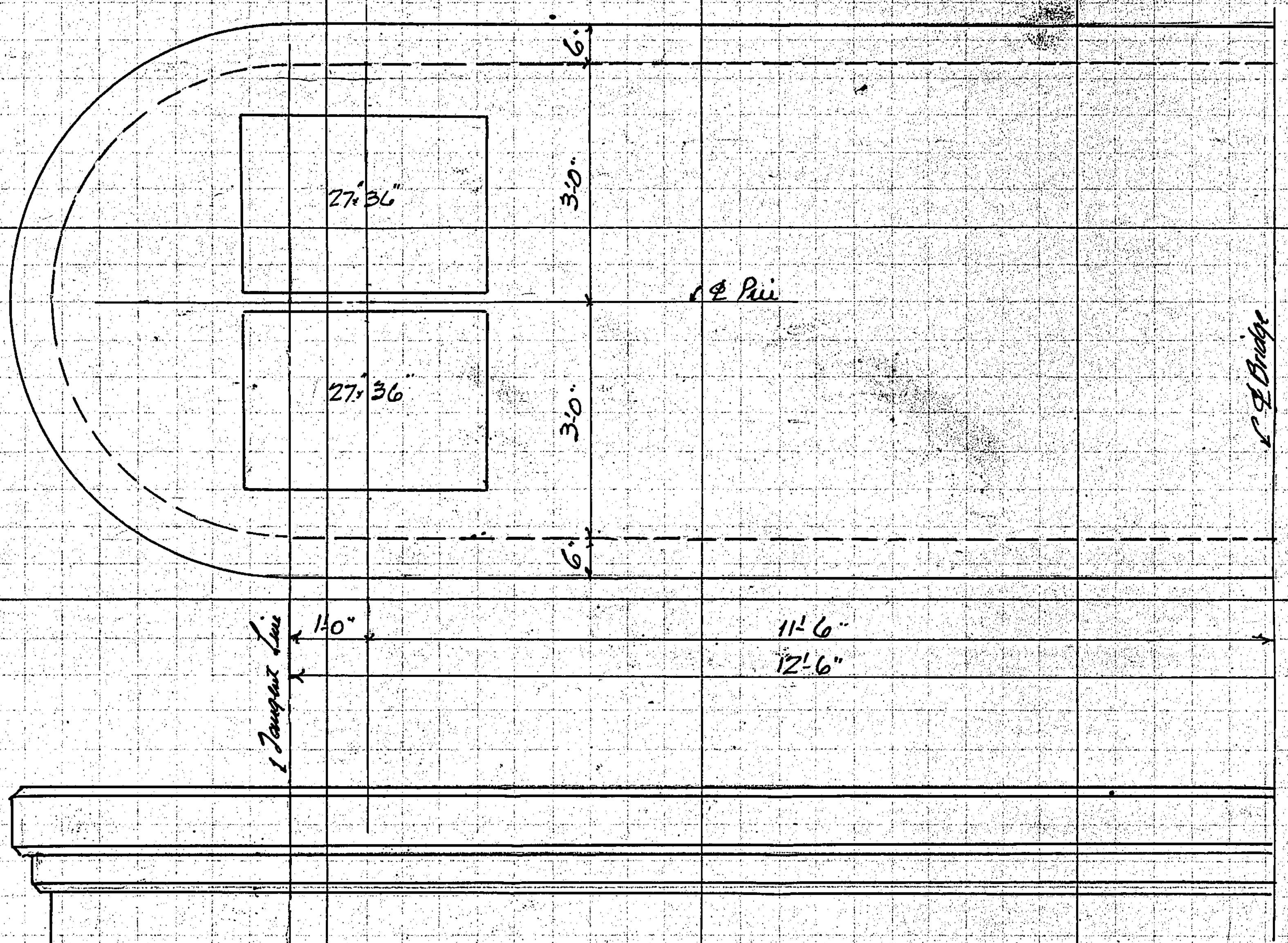
117200
188360
205560
105..
160..

metal - total 312300
2 lines of Handrails. 30@ = 60# × 206.5 = 12400
Roadway Pavement and slab 2550 × 206.5 = 527000
Total Dead Load on pier 851700#

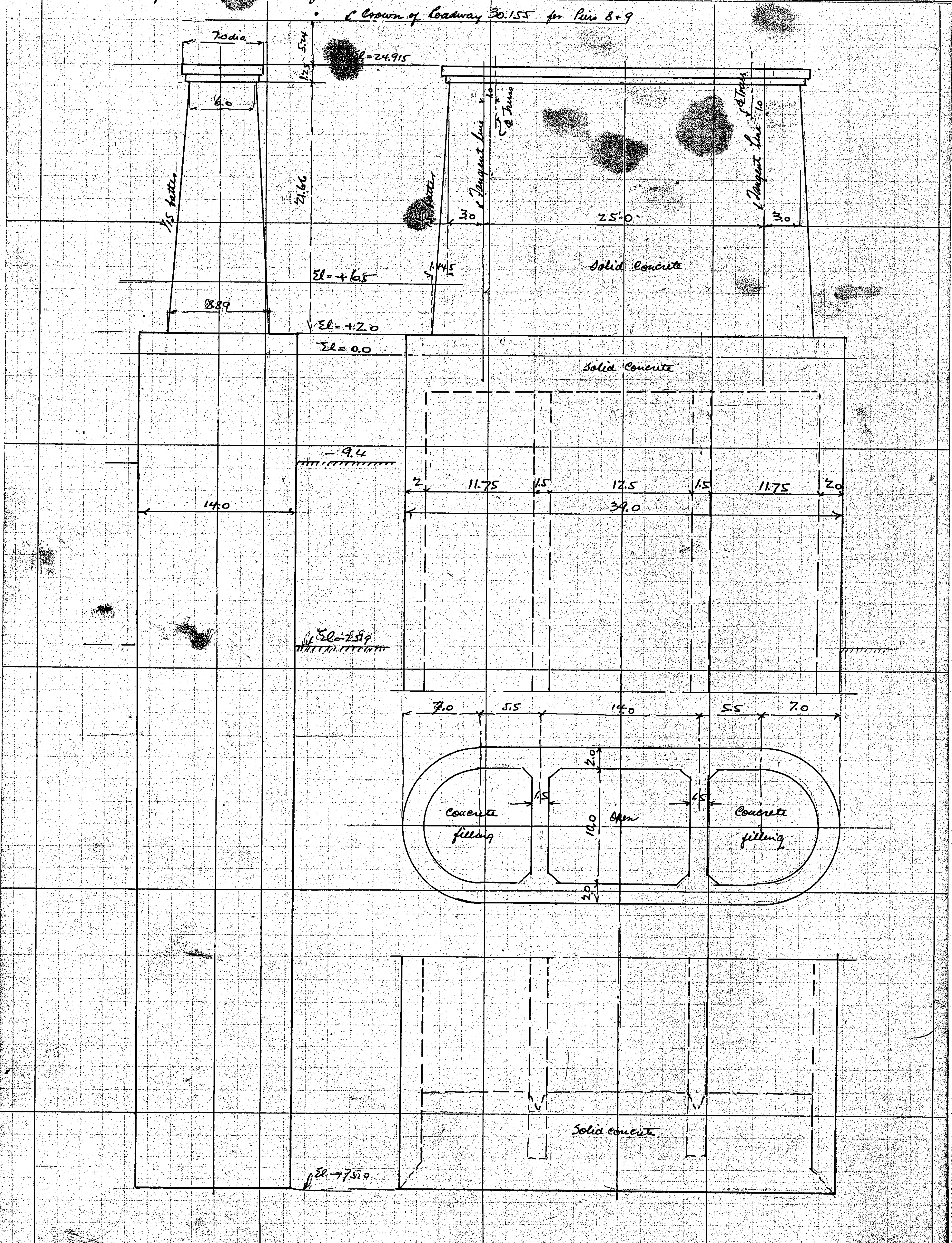
Live Load
Full load on 2 span
Center to center of span = 206.5
2@206.5 = 413' or 126 meter

$$\text{Uniform load} = \frac{100{,}000}{170+126} = 338\ \text{kg/m}^2 \text{ or say } 70\#/\square'$$

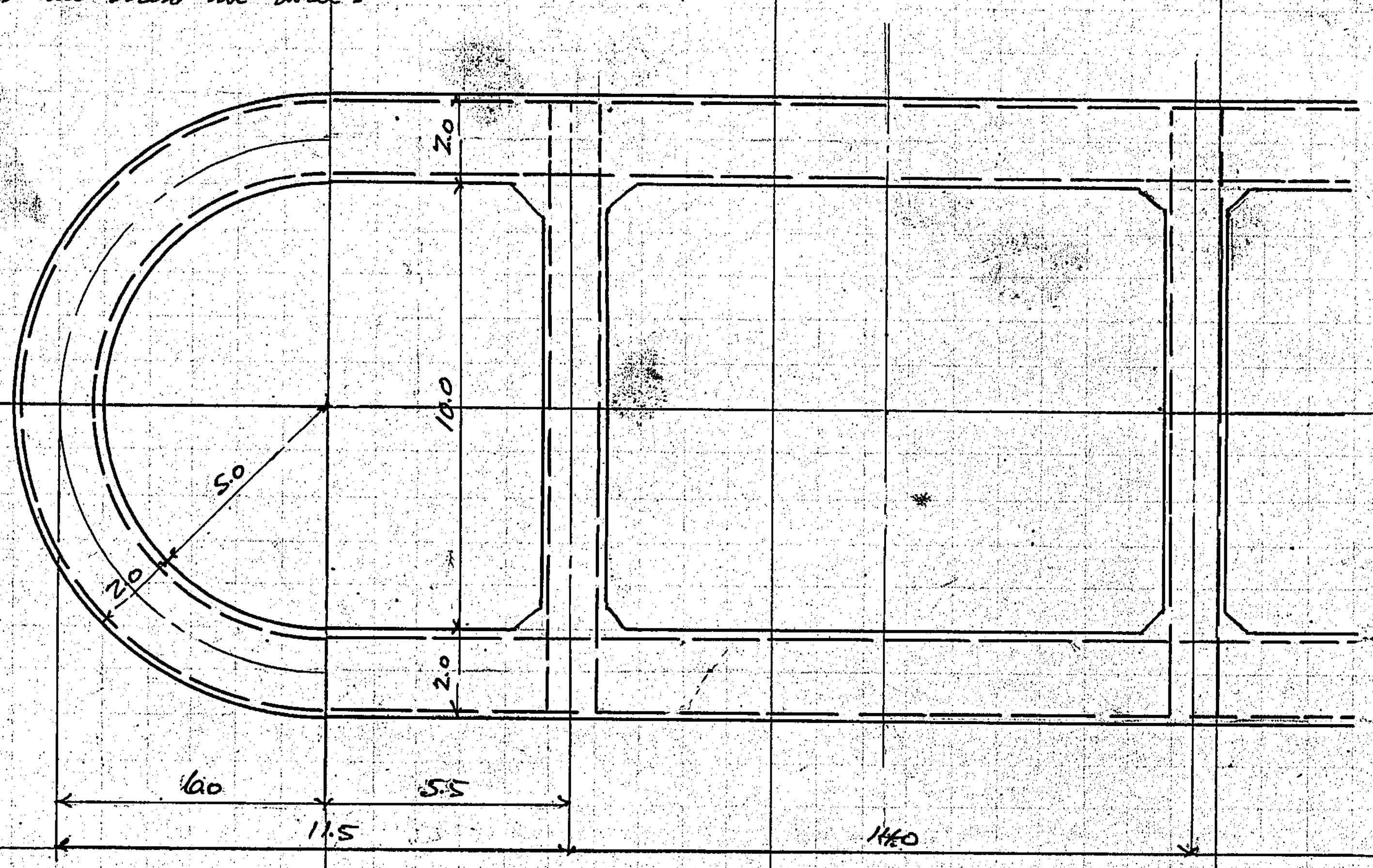
70# × 20 × 206.5 = 289000
Total load on pier 1140700#



Final Design of "Furukawa-bashi" for Tokushimaken



Approximate stress in well.



The well will be sunk approximate depth of 80' more or less and after depositing concrete of bottom of well, the up and down stream portion will be filled in with concrete and middle section will be remained unfilled

Approximate Earth pressure during sinking of well.

Temporary Earth pressure figured by the following formula

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

where L = Lateral unit pressure in lbs per sq ft at depth y.
V = vertical unit pressure in lbs per sq ft at depth y.
w = weight of Earth in lbs per cubic ft
ϕ = angle of repose of earth
μ = $\tan\phi$ coefficient of friction of earth on earth
b = the distance in ft that the earth breaks around the well.
ϕ' = angle of friction of earth on the surface of well.

Assume that $\phi = \phi' = 30°$ $\quad k = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1}{3}$ $\quad b = 10'$ $\quad y = 80'$

In case of $w = 100^{\#}$ per cubic ft $\quad L = 800^{\#}$ per sq ft

In case of sinking of well; the weight of well will be carried by friction and the load will be transferred to surrounding earth; Assuming $400^{\#}$ per square ft of friction on the surface of well and distributing area of 10', the uniform distribution of friction will be $40^{\#}$ per square ft; Let us assume the weight of earth $140^{\#}$ instead of $100^{\#}$

Then lateral pressure $= 800 \cdot 1.4 = 1120^{\#}$ per sq ft at the bottom of well.

Equivalent depth of water $= \frac{1120}{62.5} = 17.9'$

Special reinforcements should be used below this depth to resist water pressure when the inside of well is dry.

The stresses in the shell of well are hard to solve

Let us make the following assumption -

For end of pier - as ring of 12' in dia $= \frac{1}{16} wl^2$ approx

where w = unit weight of earth

and l = diameter of ring

load assumed uniformly distributed over diameter of ring.

Side of well between partition walls.

On account of continuity of beam, the moment assumed as $\frac{1}{12}wl^2$ for negative and positive

middle section $m = \frac{1}{12} \times 1120 \times 14^2 = 18300'^{\#}$

Depth required for 17000 #/□" steel stress and 640 #/□" concrete stress

$$d = \sqrt{\frac{18300}{102}} = 13.5''$$

Try 2' wall reinforcement $= \frac{18300 \times 12}{\frac{7}{8} \times 21'' \times 17000} = .703$ □" per ft strip.

Shear $= 1120 \times 7 = 7840^{\#}$ Unit shear $= \frac{7840}{\frac{7}{8} \times 21 \times 12} = 35.6$ #/□" ok without web reinforcement

End of well $m = \frac{1}{16} \times 1120 \times 12^2 = 10100'^{\#}$

Depth of concrete assumed 2'-0"

Steel area reqd $= \frac{10100 \times 12}{\frac{7}{8} \times 21 \times 17000} = 0.39$ □"

Use all vertical bars 5/8" dia round

Reinforcement in shaft

volume of coping 7' dia = 38.48

25 × 7 = 175.00

213.48 × 0.75 = 160.0

6.5' dia = 33.18

25 × 6.5 = 162.50

195.68 × 0.50 = 97.7

257.7 ÷ 216 = 11.9 立坪

volume of concrete in shaft 6' dia = 28.27

25 × 6 = 150.00

178.27

8.89' dia = 62.07

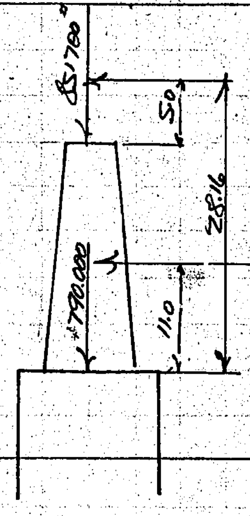
25 × 8.89 = 222.00

284.07

462.34 ÷ 2 = 231.17

vol = 231.17 × 21.66 ÷ 216 = 23.20

Total 24.39 立坪



weight = 24.39 × 216 × 150 = 790,000#

Dead load superstructure 851,700

Total 1641,700#

Earth quake acceleration assumed as 3000 mm/sec²

And seismic force assumed 30% of load or $k = 0.3$

Horizontal seismic force = 851700 × 0.3 = 255600#

790000 × 0.3 = 237,000#

moment at bottom of shaft 255600 × 28.16 = 7,200,000

237000 × 11.00 = 2,607,000

9807,000'#

Moment of Inertia at bottom of shaft

Middle section $= \frac{bd^3}{12} = \frac{25 \times 12 \times 8.89^3 \times 12^3}{12} = 30,300,000 \text{ (in)}^4$

Circular Ends $= 0.049 \times (8.89 \times 12)^4 = 6280,000$

$36580,000 \text{ (in)}^4$

Fibre stress due to seismic moment

$\frac{9807,000 \times 12 \times 53.25}{36580,000} = 171.0$ #/□" Tension or compression

Dried load $= \frac{1641.000}{28407 \times 144} = 40\ \#/\text{o}''$

Summary $171.0 - 40 = 131.0\ \#/\text{o}''$ Tension
$171.0 + 40 = 211.0$ " Compression.

Reinforcement in shaft due to seismic force ; tension in concrete neglected.
moment per ft strip say $340.000\ \#$
Dried load per ft " $565,000\ \#$ Eccentricity $= \frac{340.000}{56500} = 6.02'$

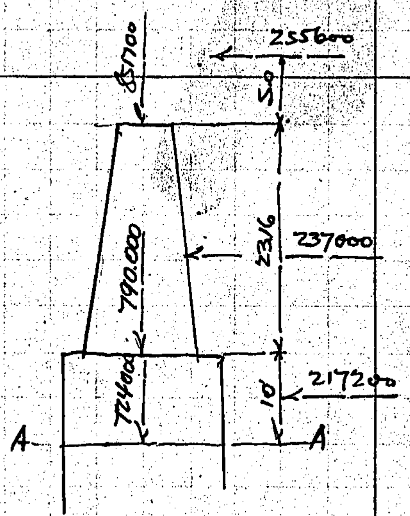
Assume $\frac{7}{8}''\phi$ bars 9" centers $= .8?$ % of reinforcement $= \frac{1.88}{12 \times 103.5} = 0.148$

Ratio eccentricity to depth $= 6.02 \div 8.89 = .677$ value of k/s (neutral axis) $= 0.20$

Stress in concrete $= \frac{340.000}{0.0677 \times 8.64^2} = 67300\ \#/\text{o}'$ or $467\ \#/\text{o}''$

Stress in steel $= 15 \times 467 \times (\frac{103.5}{21.3} - 1) = 27.000\ \#/\text{o}''$ OK.

Seismic moment at 10' below top of well
weight of well $10' = 724000\ \#$
seismic force $= 724000 \times 0.3 = 217200\ \#$



Moment at AA.
$255600 \times 38.16 = 9750.000$
$237000 \times 21.00 = 4980\ 000$
$217200 \times 5.00 = \underline{1.086.000}$
$15.816.000$

Total Load
superstructure 851700
Shaft 790000
well 10' 724000
2365700 #

moment of inertia of well $= \frac{25 \times 14^3}{12} = 5720^{(1)4} = 118.800.000$
$0.049 \times (14 \times 12)^4 = \underline{39.000.000}$
$157.800.000\ ''^4$

Fibre stress $= \frac{15816.000 \times 12 \times 84}{157800\ 000} = 101.0\ \#$ Tension or compression

Dried load $= \frac{2365700}{503.9 \times 144} = \underline{32.6}$ compression

Summary $101.0 - 32.6 = 68.4\ \#/\text{o}''$ tension
$101.0 + 32.6 = 133.6\ \#/\text{o}''$ compression

OK without reinforcement

Soil pressure at bottom of base
weight of well
shell 14.0 dia 153.9
25 × 14 $\underline{350.0}$
503.9

10.0 dia 78.53
25 × 10 $\underline{250.0}$

$\underline{328.53}$
$175.37 \times \frac{75}{216} = 60.9$ ミヤ

Partition wall $10 \times 1.5 = 15.0$
fillet $\underline{1.0}$
$2 @ 16.0 = 32.0 \times \frac{61}{216} = 9.04$ ミヤ

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Final Design of Furukawa-bashi for Tokushimaken 77

Bottom filling 9' 328.53 × $\frac{9}{216}$ = 13.7 yd³
Top filling 5' 328.53 × $\frac{5}{216}$ = 7.6 " "

Side filling between top and bottom layers
10'-dia 78.53
10 × 9.5 95.00
173.53
less filler say 1.00
172.53 × $\frac{61}{216}$ = 48.70 yd³

weight of pier
Shaft and coping 24.39 @ 32400 = 790.000#
well 60.9
partition 9.04
69.94 @ 32400 = 2265.000
Bottom filling 13.70
Top filling 7.60
21.30
Side filling 48.70
70.00 @ 30200 = 2114.000
5169.000
Superimposed Load Dead and Live 1140.000
6309.000#
Frictional Resistance assumed 2265000 × 2/3 1510.000
4799.000#

Friction circumference of well 94.0' per ft. for 70' 94×70 = 6580 □'
Friction per sq. ft = 1510.000 ÷ 6580 = 230#/□' OK
Bottom area of base 14.5 dia = 165
14.5 × 25 = 362
527 sq ft

Unit bearing = 4799000 ÷ 527 = 9100#/ft or 4.07 tons per sq. ft
Earth load 70' below 100 × 70 = 7000
2100#/□' or 0.94 tons " " extra
soil will be able to carry 0.94 tons extra load at this depth
or uniform load of 9100 ÷ 70 = 130# per cubic ft of soil.
Frictional Resistance will be added to weight of the soil.
Soil Pressure during Earthquake. Fixed point assumed as 10' below top of well.
Seismic moment k = 0.3 m = 15,816.000 '#
Frictional couple $\frac{1,510.000}{2 \times 2_2}$ × 14 = factor of safety 5290.000
10,526.000 '#

moment of inertia of bottom of base = 176.800.000 (inch)⁴
Fibre stress = $\frac{10.526.000 \times 12 \times 87}{176.800.000}$ = 62#/□" or 8920#/□' = 3.98 tons/□'

Dead Load of pier 5169.000
Superimposed Dead Load 851.700
6020.700
Less ½ friction 1,510.000 ÷ 2 = 755000
5,265700 ÷ 527 = 10.000#/□' or 4.46 tons/□'
8920 or 3.98
18920 8.44 tons/□'

Uniform load 18920 ÷ 70 = 270#/cubic ft
130
140# extra during earthquake
not counting factor of safety on frictional Resistance during earth quake
Seismic moment 15,816,000
Frictional couple 10580.000
5,236,000 '#

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Side of well between partition walls

On account of continuity of beam the moment assumed $\frac{1}{12} wl^2$ for negative and positive.

middle section - $m = \frac{1}{12} \times 1120 \times 14^2 = 18.300^{1\#}$

Depth required for 17000# Steel stress and 640#/□" concrete stress

$d = \sqrt{\frac{18300}{102}} = 13.5"$

Try 2' wall then reinforcement $= \frac{18300 \times 12}{\frac{7}{8} \cdot 21 \cdot 17000} = .703□"$ per ft strip

Use 3/4"φ bars $7\frac{1}{2}$" centres

shear $= 1120 \times 7 = 7840^{\#}$ Unit shear $= \frac{7840}{\frac{7}{8} \times 21 \times 12} = 356$ #/□" ok without reinf.

End of well:- $m = \frac{1}{16} \times 1120 \times 12^2 = 10100^{1\#}$

Depth of concrete assumed 2'-0"

steel area required $= \frac{10100 \times 12}{\frac{7}{8} \times 21 \times 17000} = .39□"$

Use $\frac{1}{2}$"φ bars 6" centres

All vertical bars 7/8" round.

Reinforcement in shaft

volume of concrete in coping

7' dia	=	38.48		
25 × 7	=	175.00		
		213.48	× .75 =	160.0
6.5' dia	=	33.18		
25 × 6.5	=	162.50		
		195.68	× .50 =	97.7

257.7 ÷ 216 = 1.19 立坪

volume of concrete in shaft

6' dia	=	28.27
25 × 6	=	150.00
		178.27
8.89' dia	=	62.07
25 × 8.89	=	222.00
		284.07

462.34 ÷ 2 = 231.17

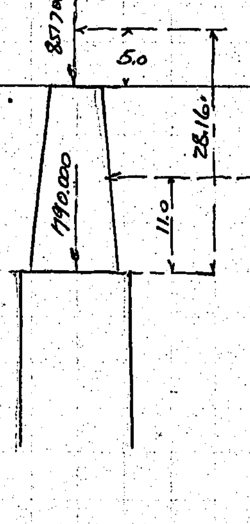
vol = 231.17 × $\frac{21.66}{216}$ = 23.20

Total 24.39 立坪

weight = 24.39 × 216 × 150# = 790.000#

Dead load superstructure = 851.700

Total 1641.700#



Seismic force acceleration of falling body $g = 9800^{mm}$ per sec.

Assumed acc. of earth quake $= 2000^{mm}$ per sec.

moment due to acc $= M = \frac{W}{g} a y$ where W = weight

g = acceleration of gravity

a = acc. of earthquake

y = lever arm

Horizontal force superstructure $- 851.700 \times \frac{2000}{9800} = 174000^{\#}$

shaft $790.000 \times \frac{2000}{9800} = 161500^{\#}$

moment at bottom of shaft

174.000 × 28.16 = 4900.000

161500 × 11.0 = 1776500

6676500 1#

moment of inertia at bottom of shaft

middle section $= \frac{bd^3}{12} = \frac{25 \times 12 \times 8.89^3 \times 12^3}{12} = 30.300.000^{(in)^4}$

Circular Ends $= 0.049 \times (8.89 \times 12)^4 = 6.280.000$

$36580.000^{(in)^4}$

Fibre stress due to moment $= \frac{6676500 \times 12 \times 5.325}{36.580.000} = 117.0\#/\square''$ Tension or Comp.

Direct load $= \frac{1641.700}{284.07 \times 144} = 40\#/\square''$

Summary $117.0 - 40.0 = 77.0\#/\square''$ Tension.

$117.0 + 40.0 = 157.0\#/\square''$ Compression.

Reinforce shaft neglecting tension in concrete

moment per ft strip say $230.000''\#$

Direct load say $\frac{1641700}{29 \times 8.89} = 6\ 56500\#$ Eccentricity $= \frac{230.000}{56500} = 4.07'$

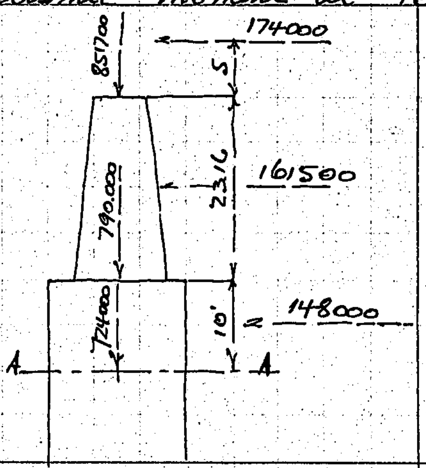
$1''\phi$ bars 9" centers assumed $= 1.050\square''$ % of reinforcement $= \frac{2.10}{12 \times 1035} = 0.17\%$

Ratio Eccentricity to depth $= 4.07 \div 8.89 = .47$ value of k (neutral axis) $= 0.354$

Stress in Concrete $= \frac{230.000}{0.0835 \times 8.64^2} = 36900\#/\square'$ or $257\#/\square''$

Stress in steel $= 15 \times 257\left(\frac{10.35}{3.87} - 1\right) = 6500\#/\square''$ O.K.

Seismic moment at 10' below top of well



weight of well 10' $= 724000\#$

seismic force $= 724000 \times \frac{2000}{9800} = 148000\#$

moment AA

174 000	$\times$	38.16	=	6.630.000
161 500	$\times$	21.00	=	3390.000
148.000	$\times$	5.00	=	740.000
				10760.000 $'\#$

Total load

superstructure	851700
shaft	790000
well 10'	724000
	2365700 #

moment of inertia of well $= \frac{25 \times 14^3}{12} = 5720\ ('')^4$ or $118.800.000\ ('')^4$

$0.049 \times (14 \times 12)^4 = \underline{39\ 000.000}$

$157800\ 000$

Fibre stress $= \frac{10760.000 \times 12 \times 84}{157800\ 000} = 68.75\#$ T or Comp.

Direct load $\frac{2365700}{503.9 \times 144} = 32.60\#$ comp.

Summary $68.75 - 32.60 = 36.15\#/\square''$ Tension

$68.75 + 32.60 = 101.35\#/\square''$ Comp.

No reinforcement required.

Soil pressure at bottom of base.

weight of well.

Shell- 14.0 dia 153.9

25 x 14 $\underline{350.0}$

503.9

10.0 dia 78.53

25 x 10 $\underline{250}$

328.53

$175.37 \times \frac{77}{216} = 62.5$ 立坪

Partition wall $10 \times 1.5 = 15.0$

filler $\underline{1.0}$

$2 @ 16.0 = 32.0 \times \frac{63}{216} = 9.33$ 立坪

bottom filling 9' $328.53 \times \frac{9}{216} = 13.7$ 立坪

Top filling 5 $328.53 \times 5 \div 216 = 7.6$ 立坪

Side filling between top and bottom layers.

10' dia 78.53
10 × 9.5 95.00
173.53
less say 1.00
172.53 · 63/216 = 50 cubic yd

Weight of Pier
shaft and coping 24.39 @ 32400 = 790.000 #
well 62.5
Partition 9.33
71.83 71.83 @ 32400 = 2325.000
Bottom filling 13.70
Top filling 7.60
21.30 @ 30200 = 642.000
Side filling 50.30 @ 30200 = 1520.000
5277000
superimposed load Dead + Live 1140000
6417.000 #
Frictional Resistance assumed 2325000 · 2/3 1.550.000
4867.000 #

Friction circumference of well 94.0' per ft For 70' 94 × 70 = 6580
Friction per sq ft = 1550.000 ÷ 6580 = 236# per sq ft OK

Bottom Area of base 14.5 dia = 165
14.5 × 25 = 362
527 sq ft
Unit Bearing = 4867.000 ÷ 527 = 9230 #/□' or 4.13 tons/□'
load at 70' below 100 × 70' = 7000
2230 #/□' = 1.00 ton/□'

Soil will be able to carry 1 ton extra load at this depth.
or uniform load of 9230 ÷ 70 = 132# per cubic ft of soil.

Soil Pressure during earthquake.
Seismic moment at 10' below top of base = 10,760.000 '#
Frictional couple $\frac{1550.000}{2 \times 2.0}$ · 14 = 5420.000
2 factor of safety 5340.000 '#
moment of inertia of bottom of base = 176800.000 (inch)^4
Bending stress = $\frac{5340000 \times 12 \times 87}{176800.000}$ = 31.6 #/□"
or 31.60 × 144 = 4550 #/□' or 2.03 tons/□'

Dead Load of Pier 5277.000
superimposed load dead load 851700
6128700
Less ½ friction 1550.000 ÷ 2 775.000
5353700 ÷ 527 = 10150 #/□' or 4.53 tons
4550 2.03
14700 #/□' 6.56 tons

Uniform load of 14700 ÷ 70 = 210 #/cubic ft
132
78 ÷ 132 = 59% increase during earth quake.

Not counting factor of safety on frictional resistance, the earthquake moment will be carried by friction and the bearing pressure
6128700 ÷ 527 = 11650 #/□' or 5.20 tons
11650 ÷ 70 = 166 # cubic ft
132
34 ÷ 132 = 25.8% increase. OK

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Fibrestress = $\frac{5236.000 \times 12 \times 87}{176800.000}$ = 30.9 #/□" or 4450 #/□' or 1.99 tons/□'

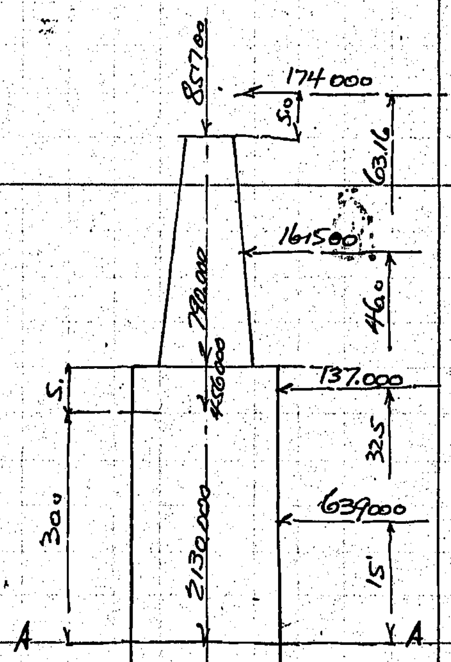
Dead Load of pier 5169.000
superimposed dead load 851.700

6020700

Less friction 1510000

4510700 ÷ 527 = 8550 #/□' or 3.82 tons/□'
4450 1.99
13000 #/□' 5.81 tons/□'

Design of Pier no. 14 in river channel.
Details of Pier shaft and reinforcement thereto same as for other piers.
Diameter or width of pier well assumed 17'0"
Seismic moment at 35' below top of well.



weight of well - 35'

5' layer at top 17' dia 227.0
17 × 25 = 425.0
652.0 × $\frac{5}{216}$ = 15.10 立坪

shell of well

13' dia = ~~132.7~~
13 × 25 = 325.0 652.0
~~457.7~~
~~194.3~~ × $\frac{30}{216}$ = 27.00 立坪

Partition wall 13 × 1.5 = 19.5
filler 1.0
2@ 20.5 = 41.0 × $\frac{30}{216}$ = 5.70 立坪

Side filling 13.0 dia = 132.7
13.0 × 9.5 = 123.5
256.2
less filler 1.0
255.2 × $\frac{30}{216}$ = 35.40 立坪

weight = well 27.00
Partition 5.70
32.70 × 32400 = 1,060.000
side filling 35.40 × 30200 = 1,070.000
2,130.000 #
Top filling 15.10 @ 30200 = 456.000 #

Seismic force k = 0.3
well 2,130.000 × 0.3 = 639.000 #
Top layer 456.000 × 0.3 = 137.000 #

moment at AA

174.000 × 63.16 = 11,000.000
161.500 × 46.00 = 7420.000
137.000 × 32.50 = 4450.000
639.000 × 15.00 = 959.000
23,829,000 #

Total Load superstructure 851700
shaft 790.000
well 456.000
well 2130.000
4,227,700 #

Moment of inertia of well (solid section assumed)

$\frac{25 \times 17^3}{12} = 10220^{(1)4}$ or $212,000,000^{(11)4}$

$0.049 \times (17 \times 12)^4 = \underline{84800.000}$

296,800.000

Fibre stress $= \frac{23829.000 \times 12 \times 102}{296800.000} = 97.2$ #/□" Tension or compression

Dead load $\frac{4227700}{652 \times 144} = 45.1$ # $97.2 - 45.1 = 52.10$ Tension

$97.2 + 45.1 = 142.30$ Comp.

O.K. without reinforcement

Soil Pressure at bottom of base.

weight of well

shell - 17' dia 227.0

$17 \times 25 = \underline{425.0}$

652.0

13' dia = 132.7

$13 \times 25 = \underline{325.0}$

$\underline{457.7}$

$1943 \times \frac{75}{216} = 67.50$ cu.yd.

Partition wall. $41.0 \times \frac{61}{216} = 11.56$ " "

Bottom filling 9' $\frac{457.7 - 41.0}{416.7} \times 9 \div 216 = 17.40$ " "

Top filling 5' $416.7 \times 5 \div 216 = 9.65$ " "

Side filling between top and bottom layers

$2552 \times \frac{61}{216} = 72.00$ " "

Weight of pier.

shaft and coping 790.000

well 67.50

partition $\underline{11.56}$

79.06 @ 32400 = 2.560.000

Bottom filling 17.40

Top filling 9.65

Side filling $\underline{72.00}$

99.05 @ 30200 = $\underline{2.990.000}$

6,340.000

Superimposed Dead + Live Loads $\underline{1140.000}$

7480.000 #

Frictional Resistance assumed $2,560,000 \times \frac{1}{2}$ $\underline{1.280.000}$

6200.000 #

Area of base assumed 17.5' dia = 240.5

$17.5 \times 25 = \underline{437.5}$

678.0

circumference of well - = 105.0 × 40 = 4200 □'

frictional Resistance per □' = 1280.000 ÷ 4200 = 305 #/□'

assume this 230 #/□' about × 4200 = 966.000 #

Revised load on bottom Total Dead Load 7480.000

frictional Resistance $\underline{966.000}$

6,514.000

Unit bearing = 6514.000 ÷ 678. = 9600 #/□' or 4.29 tons/□'

Soil Pressure during Earthquake.
Fixed Point assumed as 35' below top of well.
Seismic moment $k = 0.3$ $m = 23829.000^{\#}$
Frictional couple $\frac{966.000}{2 \times 2} \times 17 = 3870.000$
2 factor safety 19,959,000 $^{\#}$

Moment of inertia at bottom of well.
$\frac{25 \times 17.5^3}{12} = 11150^{(')4}$ or 231.500.000
$0.049 \cdot (17.5 \times 12)^4$ 95300.000
326,800,000

Fibre stress $= \frac{19.959.000 \times 12 \times 105}{326800.000} = 768^{\#/\square''}$ or $11050^{\#/\square'}$ or 4.93 tons

without of factor of safety or full frictional resistance assumed
Seismic moment 19,959.000
3870.000
16,089.000

Fibre stress $= \frac{16089.000 \times 12 \times 105}{326.800.000} = 62.0^{\#/\square''}$ $8920^{\#/\square'}$ or $3.98^{tons/\square'}$

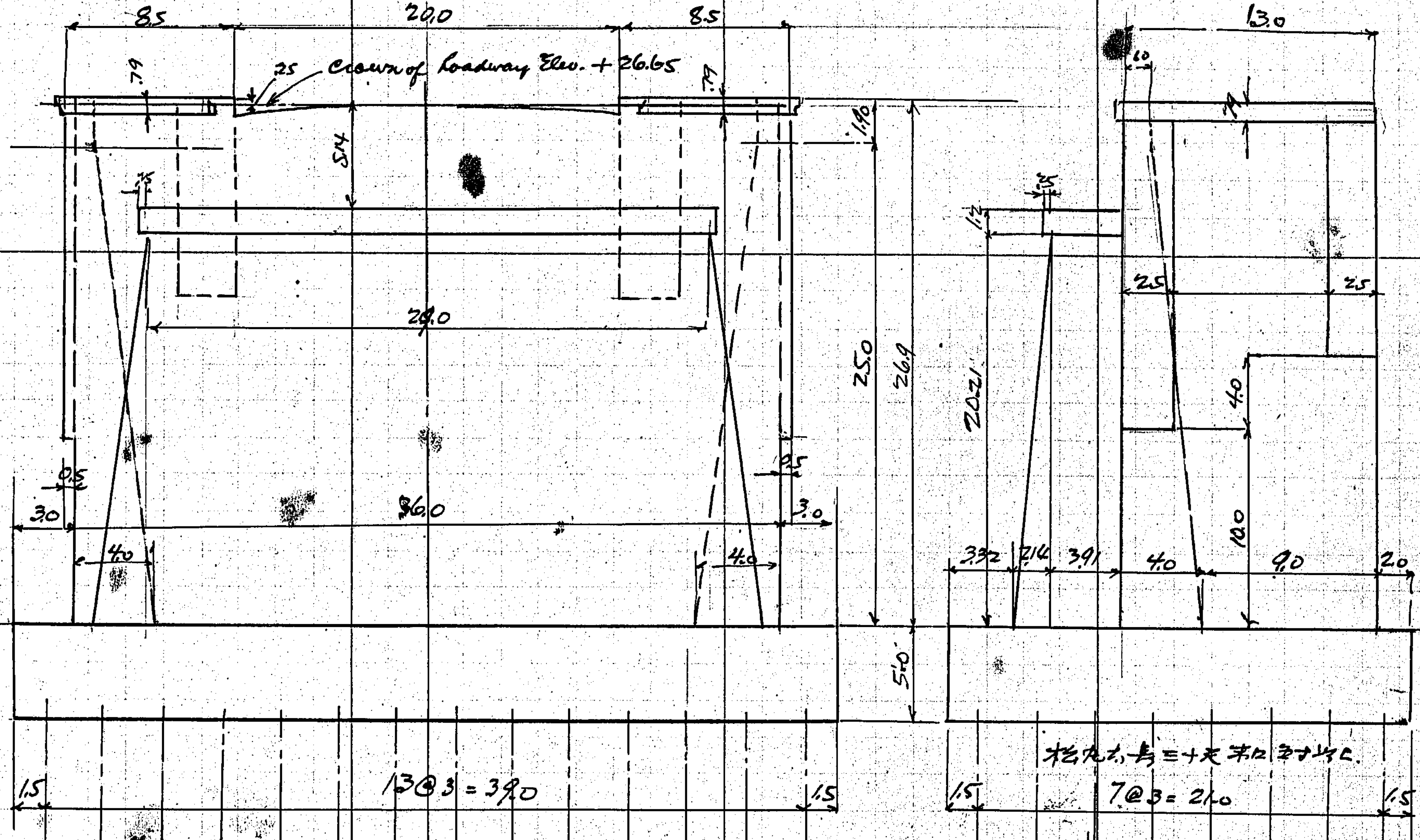
Total Dead Load of pier 7480.000
· superimposed load 851.700
$8,331,700^{\#} \div 6780 = 1245.0^{\#/\square'}$ or 5.55 tons.
Due to moment 3.98
9.53 tons

Details of Design.
Reinforcement in well.
Reinforcement in the shell of well between partition wall. same as for other piers.

Reinforcement in circular end
Soil pressure (horizontal). $1120^{\#}$ per sq. ft
$m = \frac{1}{10} \times 1120 \times 15^2 = 15750^{\#}$
Depth of concrete 2.0
Steel Area $= \frac{15750 \times 12}{\frac{7}{8} \times 21 \times 17000} = 0.605 \square''$ per ft

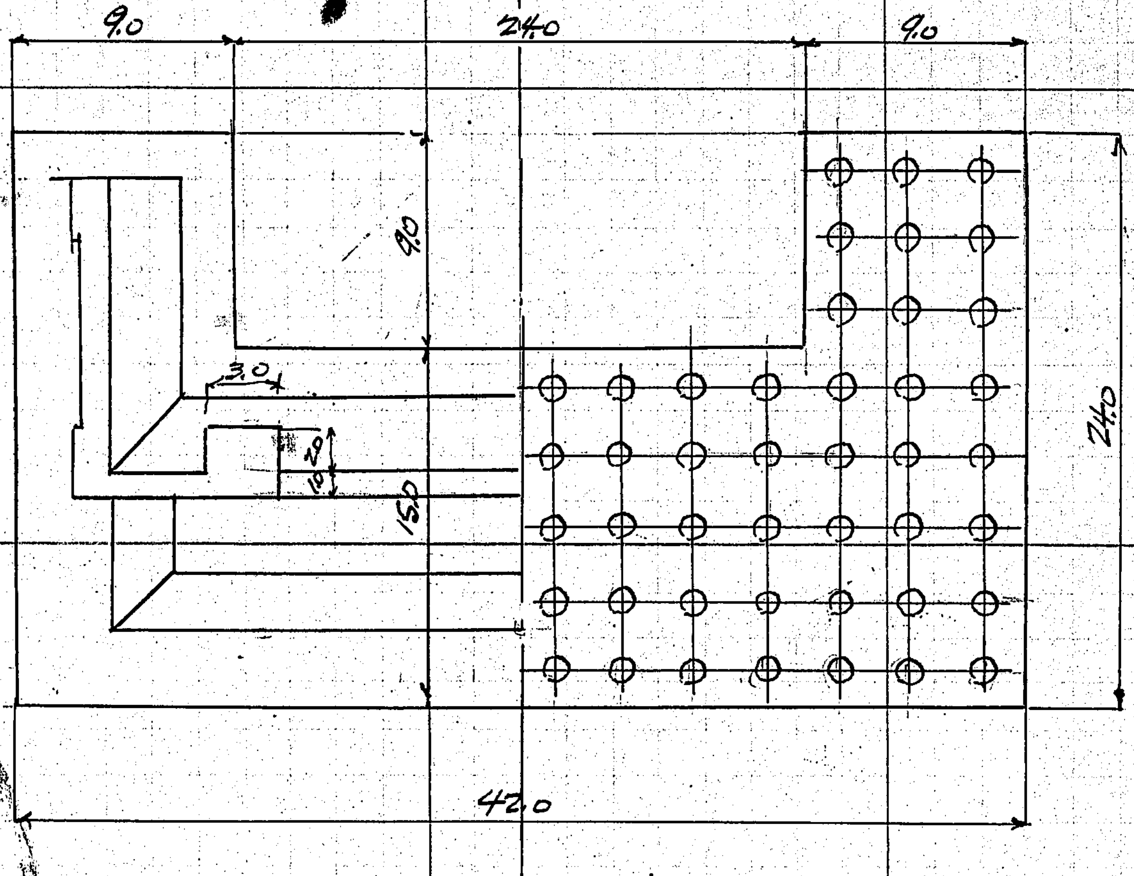
Reinforcement in the shaft same as for other piers.

Design of abutment



Front Elevation

Side View.



½ Plan View of abutment

½ plan of abutment base showing piling.

Sketch of abutment
scale ⅛" = 1'-0"

Volume of concrete in abutment and center of gravity for both axes.

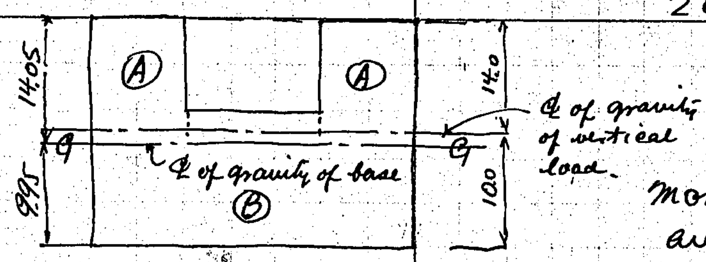
				cubic ft			
Body of abutment	coping	3.91 × 29.25 × 12.0	=	137.24	=	0.635 立坪	1
body of abut.	batter	2.02 × 20.21 × 31.7 × ½	=	647.06	=	2.996	2
" " "	rear	3.66 × 20.21 × 31.02	=	2294.51	=	10.623	3
Front wall to the top of base							
1' strip at front		1.0 × 26.90 × 36.0 − Roadway opening	=	958.40	=	4.437	4
batter		3.0 × 25.0 × ½ × 36.0	=	1350.00	=	6.250	5
coping at front				3.26	=	0.015	6
wings							
1' wall at top		2@ 1.0 × 1.9 × 12	=	45.60	=	0.211	7
1' wall to top of base		2@ 1.0 × 25.0 × 10.5	=	525.00	=	2.431	8
batter		2@ 3.0 × 25.0 × ½ × 10.0	=	750.00	=	3.472	9
Coping				5.80	=	0.027	10
Projection for posts		2@ 2.5 × 0.5 × (16.9 + 12.9)	=	74.50	=	0.345	11
Concrete under Pedestal		2@ (2.0 × 1.9 + 1.58 × 7.0 + 1.23) × 3 =		93.66	=	0.434	12
				6885.03 c		31.876 立坪	

Volume of Concrete in base.

Front	42 × 15 × 5	=	3150	
wings	2@ 9 × 9 × 5	=	810	
			3960	− 18.333 cubic tsubo

Center of gravity of area of base

		area		arm		
2@ 24 × 9	=	432	×	12.0	=	5184
24 × 15	=	360	×	7.5	=	2700
		792	×	9.95	=	7884



Moment of inertia of base area about axis GG

area						
A	432	× 2.05²	+	2@ 10368	=	22,551.48 (1)⁴
B	360	× 2.45²	+	6750	=	8910.9
						31462.38

Taking moment at heel and finding center of gravity of vertical load we found the center of gravity 10.0 from edge of toe.

Center of gravity horizontal axis − moments taken at bottom of base.

			volume		arm		
Front wall (body of abutment)	1	coping	0.635	×	25.81	=	16.38935 立坪
	2	body	2.996	×	11.74	=	35.17304
	3	body	10.623	×	15.11	=	160.57353
Front wall	4	wall	4.437	×	18.45	=	81.86265
	5	batter	6.250	×	13.33	=	83.31250
	6	coping	0.015	×			
wings.	7		0.211	×	30.95	=	6.53045
	8		2.084	×	17.50	=	36.47000
	8'		0.347	×	21.67	=	7.51949
	9		3.125	×	13.33	=	41.65625
	9		0.347	×	17.50	=	6.07250
	11		0.345	×	22.60	=	7.79700
	12		0.434	×	24.32	=	10.55488
Footing Equivalent			17.111	×	2.500	=	42.77750

summary 536.63

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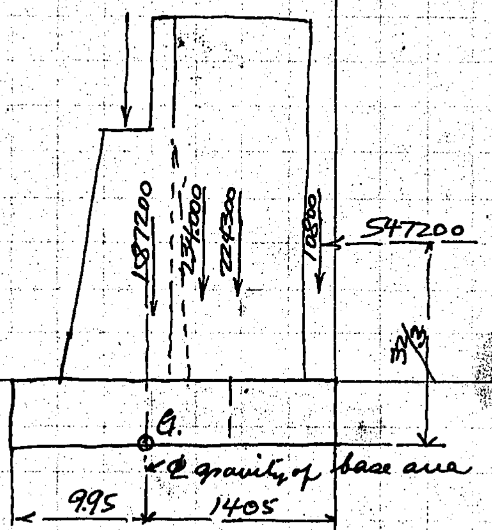
Total vol = 48.987 ㎥ Tot. moment 536.63 ㎥
center of gravity = $\frac{536.63}{48.987}$ = 10.95 ft above bottom of base.

Superimposed load including live load = 663.000#
Superimposed load without live load = 426.000#

weight of abutment = 1587.200#
weight of earth (rear of front wall) = 234.000#
" " " (side of wing wall) = 224300#
" " " (side of wing wall) = 10.800#

Horizontal earth pressure 32.0 high. Total pressure 1' strip = $\frac{100}{6} \cdot h^2$ = 17.100#
For 36.0 wide 17100 × 36 = 547.200#
moment = 547.200 × $\frac{32}{3}$ = 5.830.000#

Stability of abutment in normal stage with or without live load on span
without live load.
moment about ℄ of gravity of base area.



	load		arm		moment about G
Abutment	1587.200	×	0.05	=	79360
Earth fill	234.000	×	3.13	=	732.000
" "	224300	×	5.55	=	1246.000
" "	10800	×	13.05	=	141.000
Superimposed load	426.000	×	− 2.48	=	− 1057.000
	2482300		+ 0.46		1.141.360

with earth pressure
Summary of moments vert m. = +1.141.360
−5.830.000
−4.688.640#

Eccentricity −4688640 ÷ 2482300 = 1.88

Toe Pressure $\frac{2482300}{792}$ + $\frac{4688.640}{31462}$ × 9.95 = 4610 #/□' comp. 2.05 tons/□'

Pressure at Heel $\frac{2482300}{792}$ − $\frac{4688640}{31462}$ × 14.05 = 1040 #/□' comp. 0.465 ton/□'

With Live Load

	Load		Arm	moment about G
Abutment	1587.200	·	0.05	79360
Earth fill	234.000	·	3.13	732.000
" "	224.300	·	5.55	1246.000
" "	10.800	·	13.05	141.000
Superimposed load	663.000	·	− 2.48	− 1644.200
	2719300			554.160

Horizontal m. = − 5830000
Summary of moments − 5275.840#

Toe Pressure = $\frac{2719300}{792}$ + $\frac{5275.840}{31462}$ × 9.95 = 5110 #/□' or 2.28 tons/□'

Pressure at Heel = $\frac{2719300}{792}$ − $\frac{5275840}{31462}$ × 14.05 = 1080 #/□' 0.48 ton/□'

Stability of abutment during Earthquake.
Horizontal earthquake force assumed 30% of own weight or acc. of earthquake 3000 mm/sec.
k assumed 0.3. Hence $3/4\,k = 0.225$

Case A. Weight of abutment and earth filling with earthquake force of $3/4\,k$
Earth pressure with $3/4\,k$

Let us figure necessary data for earthquake force of $k = 0.3 \cdot 3/4$
$\tan\theta = 3/4 \times 0.3 = 0.225 \quad \theta = 12°\text{-}40'$
$\tan\beta = \cancel{21°\text{-}15'}\ 0.38806 \quad \beta = 21°\text{-}15'$

Earth Pressure during earthquake $= \frac{wh^2}{2} \cdot 0.640625$
$= \frac{100 \cdot 32^2 \cdot 0.641}{2}$ say $= 32.800$# per ft

$\cos\beta = 0.9320 \quad \sin\beta = 0.3624$ Horiz component $= 32800 \cdot .9320 = 30.600$# per ft
Vert. component $= 32800 \cdot .3624 = 11.900$# per ft

Summary for moments due to horizontal pressure.

Horizontal component	Horizontal Pressure at center section	24 × 30600	=	734400#
	" " wings	12 × 30600	=	367200#
Vert. Comp.	Vertical Pressure at center section	24 × 11900	=	285600#
	" " wings	12 × 11900	=	142800#
				428400

moment about ℄ gravity of base area

734400 × 10.67	=	7820.000#	
367200 × 10.67	=	3910.000	
			11,730000
285600 × 5.05	=	1.442.000	
142800 × 14.05	=	2005.000	
		less	3447,000
			8283,000#

For moment due to ~~dead~~ earthquake force of abutment including earth filling at rear.

	weight		Earthquake force	Arm	Moment about ℄ G of base area at bottom of base.
Abutment	1587.200 × 0.225	=	357.100	10.95	3910.000
Earth fill.	458300 × 0.225	=	125100	19.00	2380.000
"	10800 × 0.225	=	2430	18.50	45.000
~~D. weight from truss 426000 × 0.~~			484630		6,335,000#

Vertical moment due to dead load of abutment and earth filling

	load	arm	moment
Abutment	1587.200	0.05	79.360
Earth fill	234.000	3.13	732.000
" "	224300	5.55	1246.000
" "	10800	13.05	141.000
	2056300		2198360#

Summary of moments

vertical moment	− 2198360
Horizontal moment	+ 6335.000
Moment due to Earth pressure	+ 8283.000
	12419640#

Summary of vertical load

abutment + fill	2056300
vertical comp of earth p	428400
	2,484,700#

Toe Pressure $= \frac{2484700}{792} + \frac{12419600}{31462} \times 9.95 =$ 7060 #/□' comp. 3.16 tons/□'

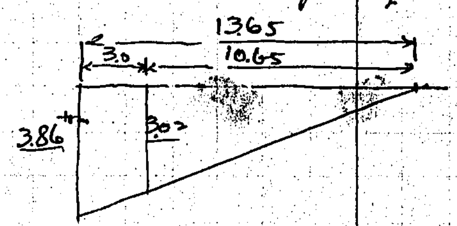
Pressure at Heel $= \frac{2484700}{792} - \frac{12419600}{31462} \times 14.05 =$ 2410 #/□' Ten. 1.08 tons/□'

Eccentricity = 12419600 ÷ 2484700 = 5.00' 9.55 − 5.00 = 4.55

Pressure area = 4.55 × 3 = 13.65' × width

Max intensity at toe pressure $= \frac{2484700 \times 2}{1365 \times 42} =$ 8650 #/□' or 3.86 tons/□'

Total load on piling neglecting bearing of soil.



$3.86 \times \frac{10.65}{13.65} = 3.02$ average pressure = 3.02 + 3.86 = 6.88 ÷ 2 = 3.44 tons/□'

Load on pile = 3.44 × 9 = 30.96 tons

Case B. Dead load superimposed load and body of abutment with k = 0.3 earthquake force.

Earthquake forces and moments (Hor).

	Load		Earthq. Force		moment (Hor)
superimposed load	426000 × 0.3	=	138600 × ~~24.50~~ 32.0	=	~~3,400,000~~ say 4,400,000
body of abutment	1587000 × 0.3	=	476200 × 10.95	=	5214,000
					9,614,000 #

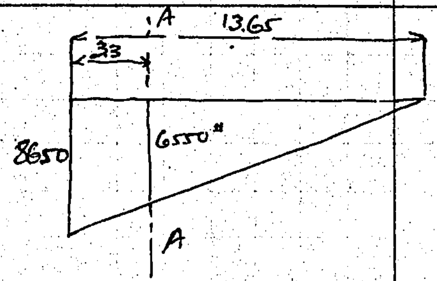
vertical moments

	Load	arm	moment
superimposed load	426000	× −2.48	1056,000
body of abutment	1587000	× 0.05 =	79,000
	2013000		977,000 # less 977,000
			8,637,000 #

Toe Pressure (Bar) $= \frac{2013000}{792} + \frac{8637000}{31462} \times 14.05 =$ ~~5950~~ 6400 #/□' comp ~~2.66~~ 2.86 tons/□'

Pressure at Heel $= \frac{2013000}{792} - \frac{8637000}{31462} \times 9.95 =$ 180 #/□' ~~Comp.~~ Ten ~~0.05~~ 0.08 tons/□'

Stress at toe of footing Projection 3.3 depth of beam = 5.0



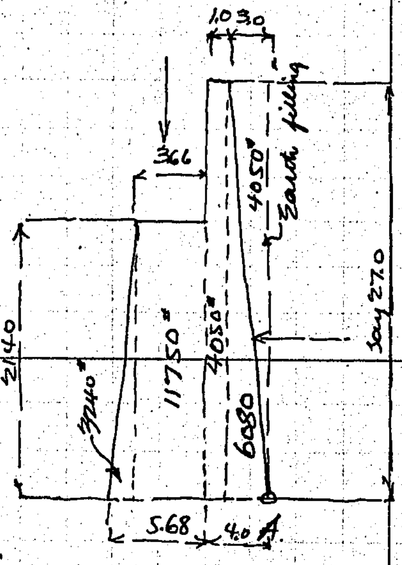
$8650 \times \frac{10.35}{13.65} = 6550$#

				arm		moment
moment at AA.	6550 × 3.3	=	21600	× 1.65	=	35600 #
	2100 × $\frac{3.3}{2}$	=	3460	× 2.2	=	7600
						43200
Less.	150 × 5 × 3.3 =		2480	× 1.65	=	4100
						39100 #

Fibre Stress $= \frac{6 \times 39100 \times 12}{12 \times 60^2} =$ 65 #/□" Tension or comp
O.K. during earthquake.

Reinforcement in shaft of abutment
Stress during earthquake
Case A. weights of abutment and horizontal pressure with k = 0.225

Reinforcement in shaft of abutment



Moment about A (vertical Axis)

Load	Arm	moment
4050	1.0	4050
6080	2.0	12160
4050	3.5	14175
11750	5.83	68502
3240	8.33	26989
29170#		125876#

Moment about A (Horizontal Axis) due to earthquake force

Load		Kam		Arm	moment
4050	×	0.225	×	18.0	16402
6080	×	0.225	×	9.0	12311
4050	×	0.225	×	13.5	12300
11750	×	0.225	×	10.7	28288
3240	×	0.225		7.13	5200
					74501#

Horizontal Earth Pressure due to Earthquake.

$= \frac{wH^2}{2} \times 0.641 = \frac{100}{2} \times 27^2 \times 0.641 = 23400\#$ per ft

Horizontal Component $= 23400 \times 0.932 = 21800\#$

vertical component neglected.

moment $= 21800 \times 9.0 = 196000\#$

Summary for moments

moment due to vertical force	125876
" " Horizontal force	74501
" " Earth Pressure	196000
	396377#

Resultant Arm $= 396377 \div 29170 = 13.6$ $\quad 13.6 - 4.84 = 8.76$

moment about neutral axis $= 29170 \times 8.76 = \underline{\underline{256,000}}\#$

Case B. superimposed dead load with earthquake force of $K = 0.3$
and body of abutment " " " " "
no earth pressure.

Superimposed Dead Load $426000 \div 29 = 14700\#$ per lin ft

moment about A (vertical axis).

Load		arm		moment
6080	×	2.0	=	12160
4050	×	3.5	=	14175
11750	×	5.83	=	68502
3240	×	8.33	=	26989
14700	×	5.50 about	=	80850
39820#				202,676#

Moment about A (Horizontal axis).

Load		K		Arm		moment
6080	×	0.3	×	9.0	=	16400
4050	×	0.3	×	13.5	=	16400
11750	×	0.3	×	10.7	=	37700
3240	×	0.3	×	7.13	=	6900
14700	×	0.3	×	27.00	=	119000
						196400#

Total moment $= 202.676 - 196400 =$ ~~399076~~# 6276.# about A.

Resultant arm $= 6276 \div 39820 = 0.16$

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Eccentricity $= 4.84 - 0.16 = 4.68$

moment $= 39820 \times 4.68 = 186500$ ¹#

Let us reinforce abutment for case A.

Total normal pressure $29170^{\#}$ Eccentricity $= 8.76'$

" moment $= 256.000$ ¹#

Depth of beam $11.6'' = 9.68$ Effective say $113''$

Assume $\frac{7}{8}''\phi$ bars 1'-0" centers $= 0.60$ % of reinforcement $= \frac{1.20}{12 \times 113} = .088\%$

Ratio eccentricity to depth $= 8.76 \div 9.68 = 0.93$

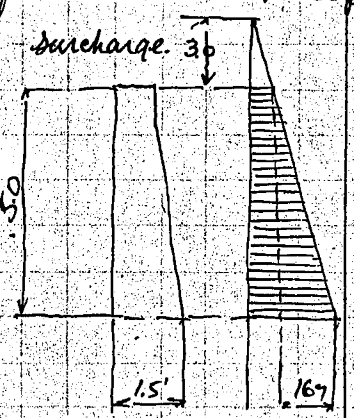
value of k (neutral axis) — say 0.1

Neglecting reinforcement, figure fibre stress

$$\text{Fibre stress} = \frac{29170}{144 \times 9.68} \pm \frac{6 \times 256.000 \times 12}{12 \times 113^2} = 141^{\#}\text{ comp or } 99^{\#}\text{ tension}$$

will be ok during earthquake, However use $\frac{7}{8}''\phi$ bars 1'-0 centers at bottom of shaft

Reinforcement in Parapet wall.



Earth Pressure $= \frac{100}{3} \times 3 = 100^{\#}$

" " $\frac{100}{3} \times 8 = 267$

$100 \times 5 = 500 \times 2.5 = 1250$ ¹#

$\frac{167}{2} \times 5 = 417 \times 1.67 = 695$

1945

call this 2000 ¹#

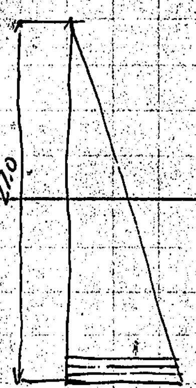
Depth of beam $= 18''$ effective say $16''$

Reinforcement reqd $= \frac{2000 \times 12}{\frac{7}{8} \times 16 \times 17000} = 0.10\square''$ per ft

use $\frac{1}{2}''\phi$ bars 12" centers $0.20\square''$ per ft

Reinforcement in wing wall.

~~Depth of fill~~ say 27.0



moment at bottom of wall.

moment $= \frac{100}{6} \times 27^2 \times \frac{27}{3} = 109.500$ ¹#

Depth $= 4.0$

Effective depth reqd for $f_c = 640$ + $f_s = 17000^{\#}/\square''$

$$d = \sqrt{\frac{109.500}{102}} = 33''$$

Steel Area $= \frac{109.500 \times 12}{\frac{7}{8} \times 45 \times 17000} = 1.970\square''$

use $\frac{7}{8}''$ 1" ϕ bars 4" centers ~~1.80~~ $\square''$ - $2.340\square''$

Moment at 17'-0" below top

moment $= \frac{100}{6} \times 17^2 \times \frac{17}{3} = 27400$ ¹#

Depth say 2.8' or 33" Effective d say 30"

Steel Area $= \frac{27400 \times 12}{\frac{7}{8} \times 30 \times 17000} = 0.74$ use 1" bar 12" centers $\square$ .78

Note: Wing wall buried into the ground and will carry earth pressure from both inside and outside, Hence in final details reinforcement in the wing reduced as shown on detail drawing. (Sheet no 7.)

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