JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

CALCULATION
FOR
DESIGN OF HAKUCHI-BASHI

MADE BY J. MASUDA

大正14-10-25

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

Nakuchi-Baski for Tokushimaken

After preliminary investigation of type of bridge we decided to adopt a suspension type with anchor arms on both sides and two more simple spans at one end of bridge. The center span is 459'-0" with 36 panels @12'-9" each; side spans are 102'-0", 8 panels @12'-9" each. simple truss spans are 102'-0" center to center of end bearings, 8 panels @12'-9"

The total length of steel structure is

main span	459'-0"	between towers
Side spans 2 @102'-0"	204'-0"	
Simple spans 2 @ 102'-0"	204'-0"	
spaces on pier 2 @ 2'-6"	5'-0"	
	872'-0"	between ℄ of End pins.

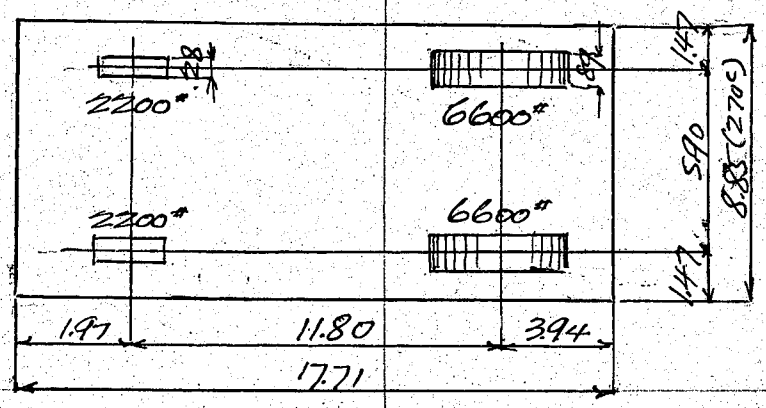
width of roadway 20.0 R Reinforced Concrete slab Soliditit Concrete pavement.

Assumed Loadings

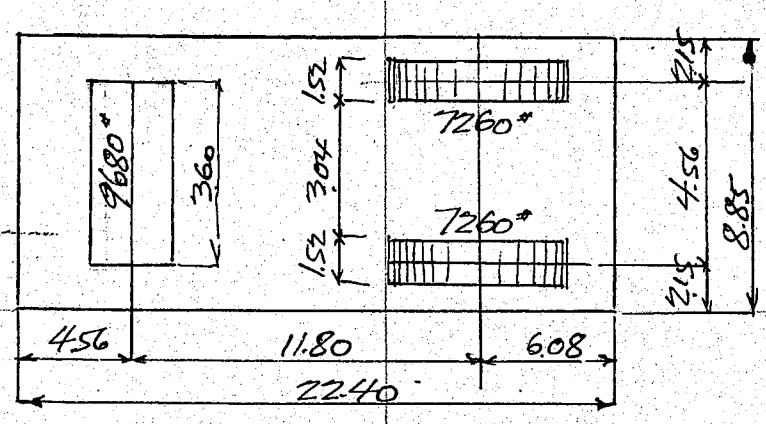
Uniform Load on roadway $w = \frac{100.000}{170+l} = 500\ ^{kg}/_{m^2}$

where w = Uniform load in kg per sq meter
l = span length in meter

8 ton motor trucks loading (17600#).



11 ton Roadroller (24200#).



2 rows of motor traffic on roadway with impact; occupied width 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 without impact

Impact

For motor truck Loading

$$\text{Impact} = \frac{20}{60+l}$$

where l = span length in meter
max. impact limited to 30%

No impact for roadroller and Uniform load.

Assumed weight of Materials

plain Concrete	140#	per cubic ft
Reinforced Concrete	150#	" " "
Structural Steel	490#	" " "
Cast iron	450#	" " "
Masonry	160#	" " "
Soliditit pavement	140#	" " "

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CALCULATIONS FOR

Sakuchi-Baski for Tokushimaken

Allowable working Strength
Structural steel or reinforcing bars

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

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

Shearing on shop driven rivets (machine driven)	13000 #/□"
" " field " " "	10.000 #/□"
Extreme fibre stress of pin	24.000 #/□"
Bearing on shop rivets	24.000 #/□"
" " field rivets	20.000 #/□"
Bearing on pin	24.000 #/□"

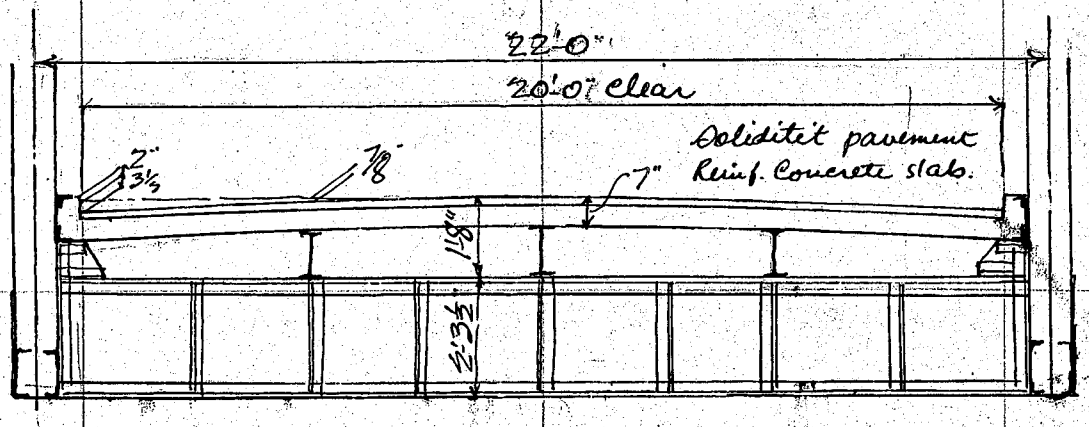
Expansion roller 610 d per lin inch where d = diameter of roller in inches
Bearing on masonry 640 #/□"

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 50% and proportioned the parts
Acceleration of Earthquake assumed 2000 mm/sec²

Design of 102' Approach span
Cross section of bridge assumed as follows.



Reinforced Concrete slab. stringer spacing 5'-0"
Dead Load

1/2" Solidited Pavement	17.5
5" Concrete slab.	68.5
	86" per sq ft.

Dead Load moment = 1/10 × 86 × 5² = 215 '#
Dead Load shear = 86 × 2.5 = 215 #
Live Load motor truck rear wheel concentration = 6600
Impact 30% 1980
8580 #

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CALCULATIONS FOR

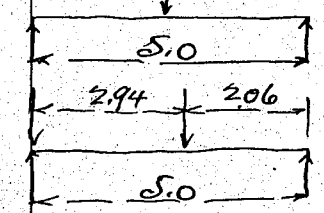
Hakuchi-Bashi for Tokushimaken

Distribution of wheel concentration

Contact between wheel and pavement = 0.66
Thickness of pavement 2@2" = 0.34
Longitudinal dist a = 1.00
Transv. dist. b = 89+34 = 1.23

Effective width $E = \frac{2}{3}(l+b)+a$
$= \frac{2}{3}(5+1.23)+1.00 = 5.15'$

Load for one ft strip = 8580 ÷ 5.15 = 1670 #



moment due to single wheel load = 8.35 × 2.5 = 2070 1#
For continuity of slab take m = 0.8 × 2070 = 1660 1#
max End shear as simple beam = 1670 × 2.06/5.00 = 690
1670
2360 #

Summary for moments and shears

	moment	shear
Dead Load	215	215
Live Load	1660	2360
	1875 1#	2575 #

Effective depth of slab for steel stress of 17000 #/□" and concrete stress of 640 #/□"
$d = \sqrt{\frac{1875}{102}} = 4.29"$
Use 3/4" insulation at bottom and 1/2" at top with soliditit pavement on top of slab.

Steel Area required = $\frac{1875 \times 12}{\frac{7}{8} \times 4.25 \times 17000}$ = 0.356 □" per ft strip
Use 1/2" d bars 6" centers = 0.390 □"

Unit shear at end = $\frac{2575}{\frac{7}{8} \times 4.25 \times 12}$ = 57.5 #/□"

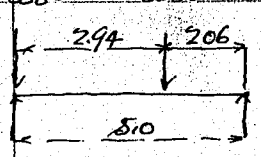
Bond stress = $\frac{2575}{\frac{7}{8} \times 4.25}$ = 690 # typ 4-1/2" d bars @ 1.57 × 130 # = 818 #
Add 2-1/2" d bars over center stringer for bond.

Design of Longitudinal Stringer Stringer spacing 5.0' span length = 12.75'

Dead Load
Slab and pavement . 86 × 5 = 430 #
stringer assumed 45
475 # per lin. ft

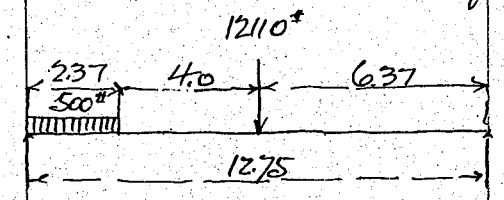
Dead Load moment = 1/8 × 475 × 12.75² = 9650 1#
Dead Load shear = 475 × 12.75 ÷ 2 = 3030 #

Live Load motor truck loading Rear wheel concentration with impact = 8580 #
Front " " " " = 2680 #



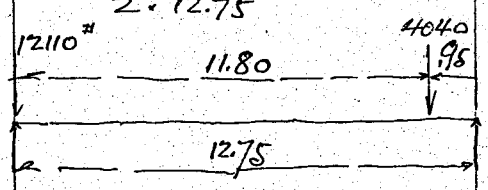
Rear wheel reaction = 8580 × 2.06/5.00 = 3530
8580
12110 #

Uniform load = 100 × 5 = 500 # per lin. ft.
moment due to wheel load = 12110/2 × 6.37 = 38500 1#
" " " Unif. load 110 × 6.37 = 700
Live Load m 39200 1#



Max shear 4040 × .95/12.75 = 300
12110
12410 #

$L = \frac{500 \times 2.37^2}{2 \times 12.75} = 110$ #



Summary for moments and shears

	moment	shear
Dead Load	9650	3030 #
Live Load	39200	12410
	48850 1#	15440

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CALCULATIONS FOR

Nakuchi-Bashi for Tokushimaken

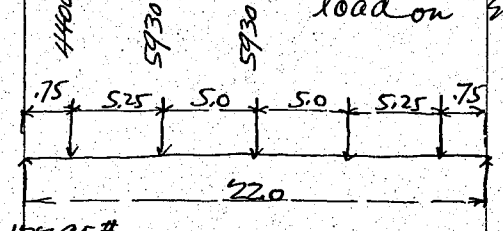
Section modulus required = $\frac{48850 \times 12}{17000}$ = 34.5 Use 12"×5" I = 31.99" Sm = 36.69

Stringer at curb

Dead Load Floor slab and pavement 86 × 2.5 = 215#
Curb and stringer say 70 " ⑧ add Handrail.
Handrail 60 "
345 # per lin ft.

Dead Load moment = ⅛ × 345 × 12.75² = 7000 ft#
Dead Load shear = 345 × 12.75 ÷ 2 = 2200#

Live Load motor truck loading 8580 × $\frac{4.0}{5.25}$ = 6530# for rear wheel
2180# for front wheel



Reaction = $\frac{250 \times 2.37^2}{2 \times 12.75}$ = 55#

Uniform load = 100 × 2.5 = 250# per lin ft
moment due to wheel load = $\frac{6530}{2}$ × 6.37 = 20800 ft#
" " " Unif. load 55 × 6.37 = 350
21150 ft#

End Shear 2180 × $\frac{9.5}{12.75}$ = 160
6530
6690#

Summary for moments and shears

	Moments	Shears
Dead Load	7000	2200
Live Load	21150	6690
	28150#	8890#

web assumed 12" × 5/16 = 3.75 ℄ web = 0.470" 12½" back to back of angles.
Effective depth = 1.04 − .14 = 0.90 Stress = 28150 ÷ 9 = 31300#
Section required = 31300 ÷ 17000 = 1.84" − .47 = 1.370" net required
Use 1L 3 × 3 × 5/16 = 1.78 gross or 1.510" net

Design of Intermediate Floor beam

Dead Load. load on intermediate panel point 86 × 5 = 430
Stringer 35
465# per ft.

load on End Stringer 86 × 2.5 = 215
curb say 70
Handrail say 60
345# per ft

12795#

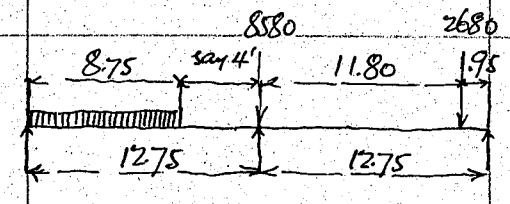
Concentrations
intermediate point 465 × 12.75 = 5930#
End Stringer 345 × 12.75 = 4400#

Dead Load moment
12795 × 11 = 141800
5930 × 5.0 = 29700
4400 × 10.25 = 45100
− 74800
67000 ft#

Dead Load girder assumed = ⅛ × 100 × 22² = 6050
73050 ft#

Dead Load shear from floor say 12800
Girder 1100
13900#

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



Reaction = 2680 × $\frac{0.95}{12.75}$ = 200
8580
8780#

Uniform load 100#/□'
Reaction = $\frac{100 \times 8.75^2}{2 \times 12.75}$ = 300# per lin. ft of span

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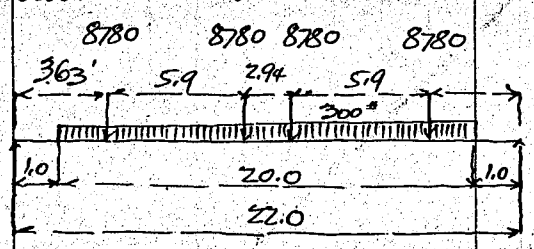
CALCULATIONS FOR

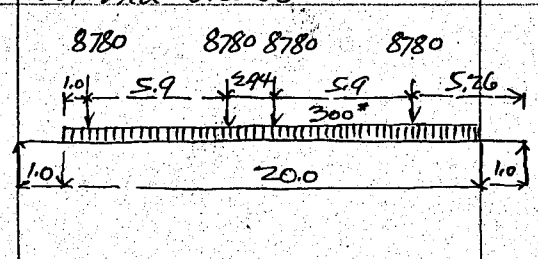
Hakuchi-Bashi for Tokushima-Ken

Intermediate Floor continued



R = 2@8780 = 17560
3000
20560

Max End shear



moment at center due to motor trucks loading

17560	× 9.53	=	167400	1#
8780	× 5.9	=	51800	
			115,600	1#
Uniform load 3000 × 11.0		=	33000	
$\frac{300 \times 10^2}{2}$		=	-15000	
			18.000	
			133,600	1#
Reaction 8780 × $\frac{50.52}{22.}$		=	20200	
Uniform load			3000	
			23200	#

Summary for moments and shears

	moment	shear
Dead Load	73050	13900
Live Load	133600	23200
	206650 1#	37100 #

Try 27" × 5/16 = 8.43 □" ⅛ web = 1.05 Depth of girder = 2'-3½" back to back of Ls
⅓ effective depth = 2.15' Stress = 206650 ÷ 2.15 = 96100 #
section required = 96100 ÷ 17000 = 5.65
1.05
4.60 □" net

2 Ls 5 × 3½ × ⅜ = 6.10 gross or 4.780" net

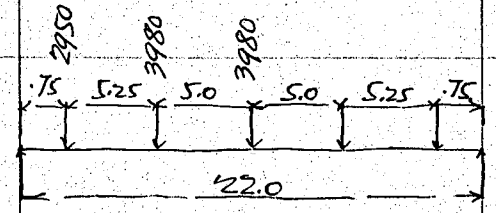
Approximate weight of Intermediate Floor Beam

web pl	1 Pl	27 × 5/16	@	28.69	×	20.75	= 595
flanges	4 Ls	5 × 3½ × ⅜	@	10.40	×	20.75	= 865
Interm. Stiffs	14 Ls	3 × 3 × 5/16	@	6.10	×	2.24	= 190
End Stiffs	4 Ls	4 × 3½ × ⅜	@	9.10	×	3.00	= 110
fills	4 Pls	3½ × ⅜	@	4.46	×	2.50	= 45
brackets say							70
Rivit heads and variation say							65
							1940 #

1940 # ÷ 20.75 = 93.5 # per ft
1940 # ÷ 12.75 = 152 # per ft of truss

End Floor Beam

Dead Load



Stringer concentration on Floor Beam

Intermediate 465 × $\frac{14.75^2}{2 \times 12.75}$ = 3980

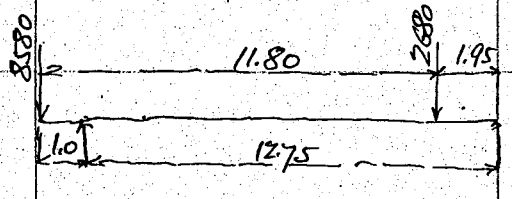
End 345 × $\frac{14.75^2}{2 \times 12.75}$ = 2950

Dead Load moment 8920 × 11 = 98120
3980 × 5.0 = 19900
2950 × 10.25 = 30200
-50100
48020 1#

8920
M due to concentration = 48020
Unif. = ⅛ × 100 × 22² = 6050
54070 1#

Shear 8920
1100
10020 #

Live Load



motor truck loading with impact Rear wheel 8580 #
Front wheel 2680 #

Reaction 8580 × $\frac{13.75}{12.75}$ = 9250
2680 × $\frac{1.95}{12.75}$ = 410
9660 #

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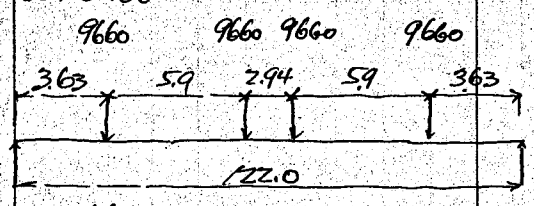
CALCULATIONS FOR

Hakuchi-Bashi for Tokushima Ken

Moment



No. Unif. load

2@9660 = 19320

End shear = 9660 × 50.52/22 = 22200

motor truck loading

19320 × 9.53 = 184000
9660 × 5.9 = 57000
127000 #

Summary for moments and shears

	moment	shear
Dead Load	54070	10020
Live Load	127000	22200
	181070 #	32220 #

Approximate weight of End Floor Beam

1 web pl.	1 Pl.	27 × 5/16 @ 28.69	×	20'9"	= 595
flanges	2 Ls	3½ × 3½ × 3/8 @ 8.5	×	20.15	= 352
"	2 Ls	3½ × 3½ × 3/8 @ 8.5	×	19.63	= 284
Stiffs	14 Ls	3 × 3 × 5/16 @ 6.1	×	2'-2¾	= 190
End conn	4 Ls	4 × 3½ × 3/8 @ 9.1	×	2'-0¾	= 75
fills	4 Pls	3½ × 3/8 @ 4.46	×	1'-6½	= 28
brackets	4 Ls	3 × 3 × 3/8 @ 7.2	×	0'-6½	= 15
"	4 Ls	3 × 3 × 3/8 @ 7.2	×	1'-1½	= 32
"	2 Pls	9 × 5/16 @ 9.56	×	1'-5"	= 27
at ends	4 Ls	3½ × 3½ × 3/8 @ 8.5	×	1'-5"	= 48
Rivet heads and variations say					60
					1706 #

1706 ÷ 21.0 = 81.0 # per lin ft.

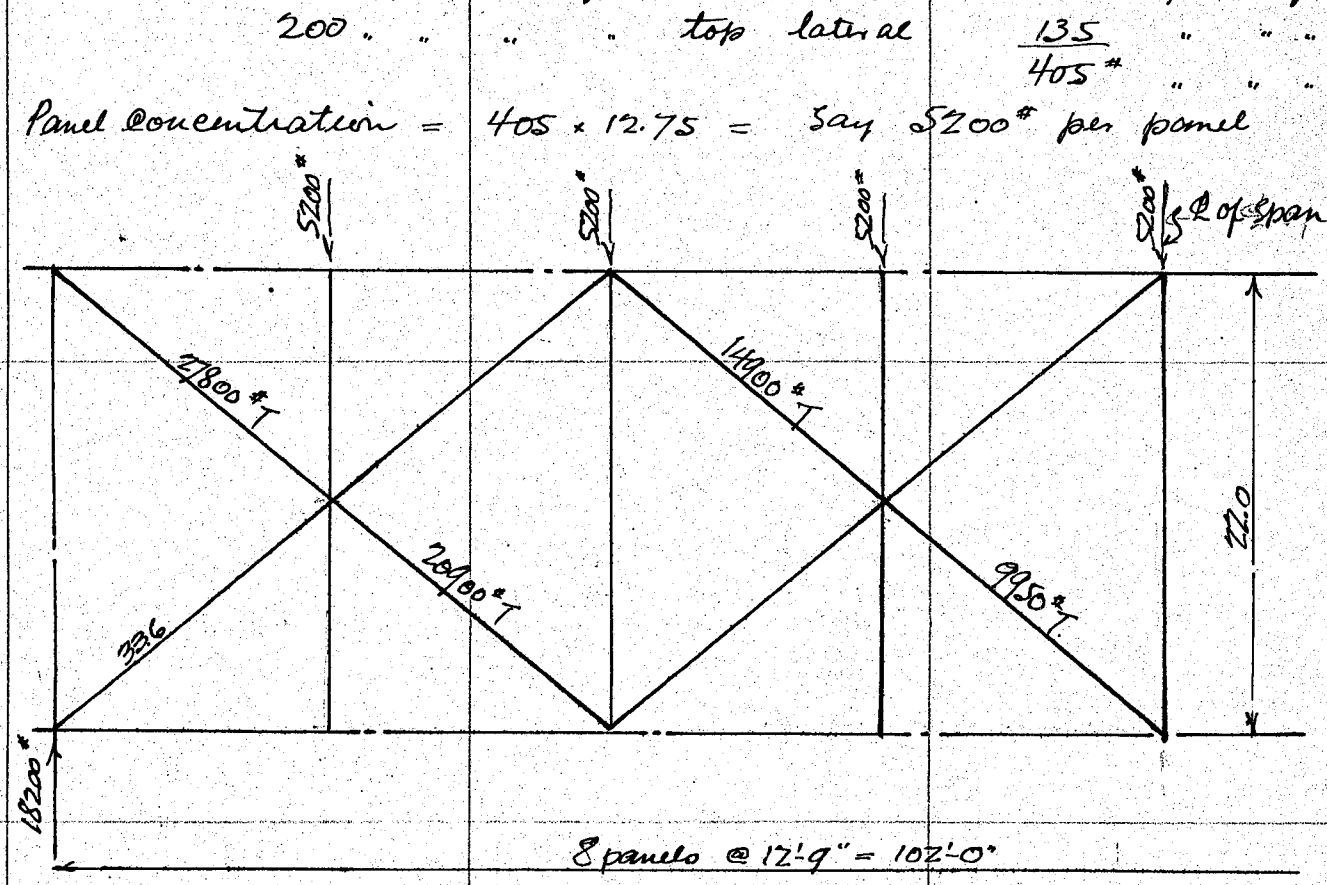
Dep 27" × 5/16 = 8.430" & web = 1.05"
Depth of girder = 2'-3½" backs to backs of Ls
Effective depth say 2.13'
stress = 181070 ÷ 2.13 = 85000 #
SR = 85000 ÷ 17000 = 5.00 - 1.05 = 3.95" net
2 Ls 3½ × 3½ × 3/8 = 4.96" gr 4.300" net

Lower Lateral Bracings.

wind pressure assumed 405# per lin ft

400 kg per meter for lower lateral = 270 # per lin ft
200 " " " " top lateral = 135 " " "
405 # " " " Combined

Panel concentration = 405 × 12.75 = say 5200 # per panel



sec θ = 336/22.0 = 1.53

Approximate Dead load 3400# per lin ft Assuming earthquake coefficient k = 0.2 force per ft = 3400 × 0.2 = 680# this will be less than wind load of 405 × 1.8 = 720# per lin ft. The section of lateral Bracings shall be determined by ordinary wind load instead of earthquake effect.

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Panel	Stress	Section rqd.	Section used	Area
I	27800#T	1.640" net	2ls 4x3x5/16	4.18
II	20900#T	1.23 "	"	"
III	14900#T	.88 "	"	"
IV	9950#T	.59 "	"	"

Stiffness of 2ls 4x3x5/16 riveted back to back of legs. 4" vertical,
Radius of gyration 1.17 Unsupported length = $\frac{33.6}{2} \times 12 = 202"$ $l/r = 173$
Unit stress = 21300 (1 − 0.0055 × 173) = 1070 #/□"
There is no stiffness as compression member, the stresses should be carried by tension only

Approximate weight of Lateral Bracings -
For two panels.

main section	8ls 4x3x5/16 @ 7.2# . 14'9"	=	850#
Connection pl	say		100
	variation & rivet heads say		10
			960#

4@960 = 3840#
Weight = 3840 ÷ 102 = 38# per lin ft of span

Approximate weight of metal in span

Stringers	3@ 35#	=	105#
Side stringers	2@ 30#	=	60
			165#
Intermediate floor beam			152#
Lower Latual Bracings.			38#
			355# per lin ft
Two trusses assumed			560
			915# per ft

Dead Load of Floor slab etc

Floor slab and pavement		= 86# . 20 =	1720#
2 curbs	2@75	=	150
2 handrail	2@60	=	120
			1990#
metal			915
			2905#

For one truss 2905 ÷ 2 = 1450# say.

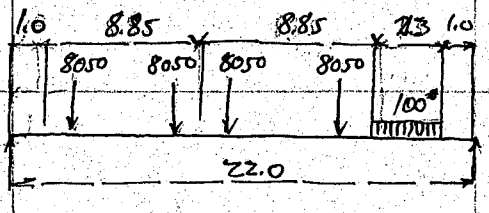
Live Load
Uniform Live Load on truss $\frac{100.000}{170 + \frac{102}{3.28}}$ = 496 kg/m² or say 100#/□'

Impact allowance = $\frac{20}{60 + \frac{102}{3.28}}$ = 22%.

Rear wheel of motor truck 6600#
Impact 22% 1450#
8050#

Front wheel conc = 8050 ÷ 3 = 2680#

Max Reaction on truss



Occupied space of two motor trucks side by side
2@8.85 = 17.7'
Reaction from motor truck loading (Rear wheel).
2@16100 . $\frac{12.15}{22}$ = 17800#
Front wheel reaction = $\frac{1}{3}$. 17800 = 5940#

Uniform load	2.3' wide	230# . $\frac{2.15}{22}$ =	say 23# per lin ft.	M_A
" "	17.7' wide	1770 . $\frac{12.15}{22}$ =	977# per lin ft.	M_B

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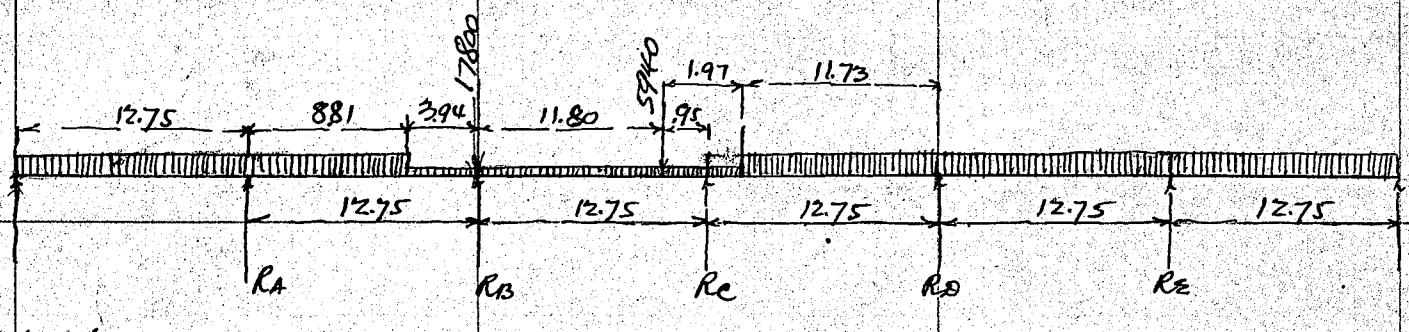
CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Live Load panel Concentration

Uniform load 23" × 12.75 = 294 say 290#
977 · 12.75 = 12460
12,750#

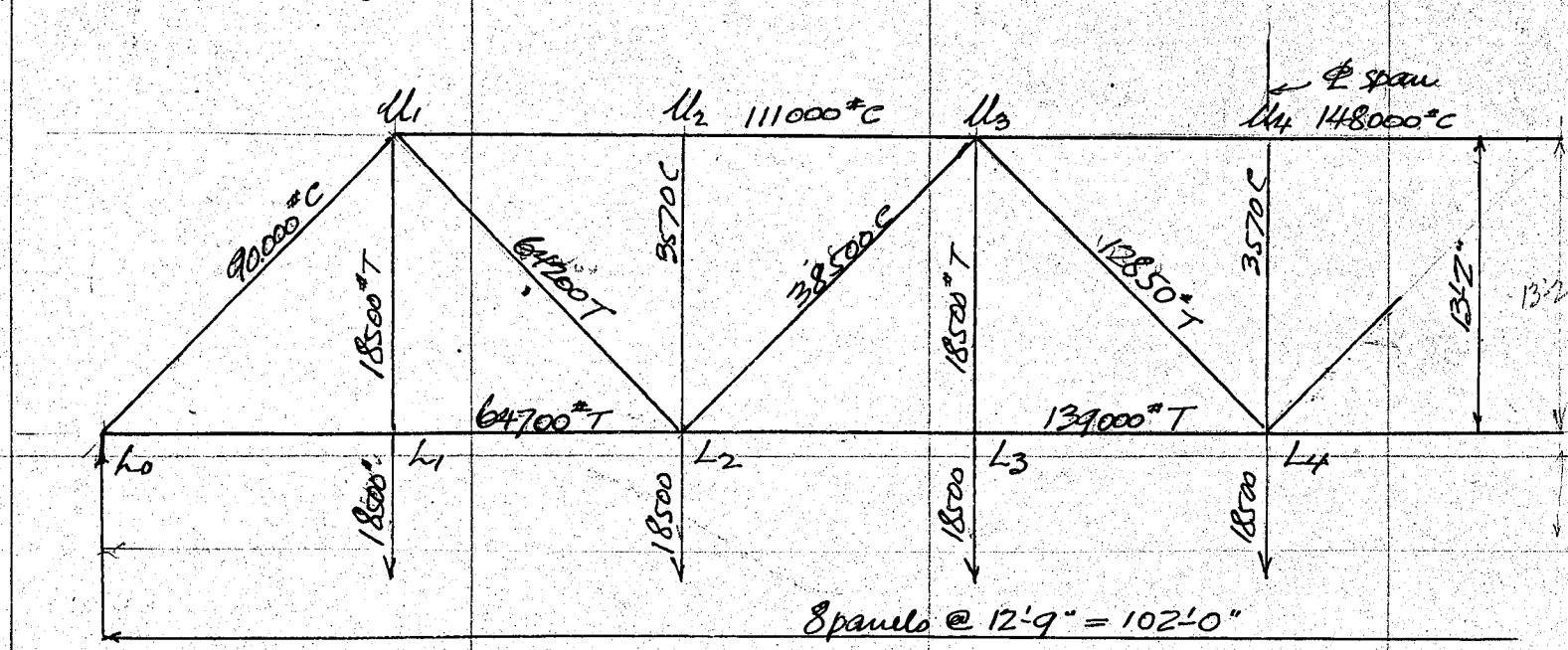
Panel Point reaction due to live load



Unif. L. LA	290	290	290	290	290
Unif. L. LB	6230				
LB	5640 ---	2970	5260 -----	6190	
LB				6230 ---	6230
LB					6230 --
Front wheel		440	5500		
Rear wheel	____	17800	____	____	____
	12160#	21500#	11050	12710	12750

For figuring chord stresses assume Uniform panel Concentration of 12750# throughout and add 21500-12750 = 8750# at panel point to be considered
For figuring web members assume same panel Concentration of 12750# throughout and add 18530# - 12750# = 5780# at panel point to be considered, where
18530 = 290 + 440 + 17800# (Unif. load LA and motor truck loading)
Dead Load panel Concentration = 1450 × 12.75 = 18500#
For dead Load upper panel Conc = 280 × 12.75 = 3570# for verticals only

Design of 102'-0" truss
Dead Load Stresses



Diagonal member = 18.30'

$\sec\theta = \frac{18.30}{13.17} = 1.39$

$\tan\theta = \frac{12.75}{13.17} = .968$

W tan θ = 18500 · .968 = 17900#
W sec θ = 18500 · 1.39 = 25700#

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Live Load stresses.

Stresses due to uniform concentration of 12750#

wtanθ = 12750 × .968 = 12300#

wsecθ = 12750 × 1.39 = 17700#

							Summary
Chord members							
L_0-L_1-L_2	12300	× 3.5	= 43000# T	+	7400	=	50400# T
U_1-U_2-U_3	12300	× 6.0	= 73800# C	+	12700	=	86500 C
L_2-L_3-L_4	12300	× 7.5	= 92200# T	+	15900	=	108100 T
U_3-U_4-U_3'	12300	× 8.0	= 98400# C	+	17000	=	115400 C
Web members							
L_0-U_1	17700	× 3.5	= 62000# C	+	7000	=	69000# C
U_1-L_2	17700	× 2 1/8	= 46500# T	+	6000	=	52500# T
L_2-U_3	17700	× 1 5/8	= 33200# C	+	5000	=	38200# C
U_3-L_4	17700	× 1 0/8	= 22100# T	+	4000	=	26100 T
L_4-U_3'	17700	× 6/8	= 13300 C	+	3000	=	16300 C

Chord stress due to 8750# concentration

L_0-L_1-L_2	.968 × 8750 ×	7/8	=	7400# T
U_1-U_2-U_3	do ×	12/8	=	12700# C
L_2-L_3-L_4	do ×	15/8	=	15900# T
U_3-U_4-U_3'	do ×	16/8	=	17000# C

Web members due to 5780#

L_0-U_1	1.39 × 5780 ×	7/8	=	7000# C
U_1-L_2	" " ×	6/8	=	6000# T
L_2-U_3	" " ×	5/8	=	5000# C
U_3-L_4	" " ×	4/8	=	4000# T
L_4-U_3'	" " ×	3/8	=	3000# C

Max Live Load reaction for Hanger = 23200# see pp 5

End Post Stress = 159000# C 2 L's 12"·3½" @ 26.1# = 15.35 □"

radius of gyration = 4.45 Unsupported length 220"

Unit stress = $21300 \left(1 - 0.0055 \times \frac{220}{4.45}\right)$ = 15600#/□" Use 14000#/□"

Section required = 159000 ÷ 14000 = 1.14 □" gross O.K.

Diagonal L_2-U_3 76700# C 4 L's 5"·3"·5/16 = 9.60 □"

r = 2.45 Unsupported length = 220" about

Unit stress = $21300 \times \left(1 - 0.0055 \frac{220}{2.45}\right)$ = 10700#/□"

Section required = 76700 ÷ 10700 = 7.15 □"

Approximate weight of truss

member	Section	Unit wt	length	weight.
End post	15.35 □"	3.4#	18.3 =	955
U_1-U_3	15.35		25.5 =	1330
U_3-U_4	19.34		12.75 =	835
L_0-L_2	15.35		25.5 =	1330
L_2-L_4	19.34		25.5 =	1670
U_1-L_2	9.60		18.3 =	600
L_2-U_3	9.60		18.3 =	600
U_3-L_4	8.36		18.3 =	520
Verticals 3.5 ×	8.36	3.4	13.2 =	1310
		For one truss	9150# × 2 =	18300#
		Details say 38%.		6950
				25250

252500 ÷ 102 = 248# per lin ft

wt of two trusses 25250 × 2 = 50500#

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CALCULATIONS FOR

Nakauchi-Itabashi for Johnan-shi.

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2[s 12"×3½"@26.1* = 15.35°

Dl. 111000*C
LL 86500 (86500)
197500* SR = 14.10

2[s 12"×3½"@32.88 = 19.34°

Dl. 148000C
LL 115400
263400 (113) SR = 18.8

2[s 12"×3½"×26.1* = 15.35°

Dl. 90.000*C
LL 69000
159,000 SR = 11.40°

183

Dl. 185500*T
LL 232200
417700

4Ls 43.5/16 = 8.36° - 2.20 = 6.16° net SR = 2.45° net

Dl. 69200*T
LL 53500
116700

4Ls 5×3.5/16 = 9.60° - 2.23 = 7.40° net SR = 6850° net

3570*C

4Ls 43.5/16 = 8.36

Dl. 338500*C
LL 428200
767700 SR = 7.15

4Ls 5×3.5/16 = 9.60°

Dl. 185500*T
LL 232200
417700*

4Ls 43.5/16 = 8.36 - 2.20 = 6.16° net SR = 2.45° net

Dl. 128500*T or LL
LL 36100*T or Dl
389000*

4Ls 4.3.5/16 = 8.36 - 2.22 = 6.160° net SR = 2.29

Dl. 163900C
LL 117000*C

4Ls 43.5/16 = 8.36

132"

L0 L1 L2 L3 L4

Dl. 64700*T
LL 50400
115100 SR = 6.75° net

2[s 12"×3½" @ 26.1 = 15.35
308
12.270" net
2 from web
2 from flange.

Dl. 139000*T
LL 108100
247100 SR = 14.5° net

2[s 12"×3½" @ 32.88 = 19.34
- 386
15148° net
2 from web
2 from flange

8 panels @ 12'-9" = 102'-0" c to c of End Bearings

Stresses and sections of 102'-0" truss.

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CALCULATIONS FOR

Nakuchi-Bashi for Tokushimaken

Approximate weight of metal in span

Stringers	165# × 104	=	17200#
Intern. F.B.	7 × 1940	=	13600
End Floor beam	2 @ 1710	=	3420
Lower Laterals		=	3840
Trusses		=	50500.
misc bracket &c	14 @ 150#	=	2100
			90660#
weight of shoes &c			3340#
			94000# or 42 tons. per span.

Load on pin.

Dead Load Floor slab &c 2905 × 104 = 302000#
weight of metal say 91000
393000# ÷ 4 = 98500#

Live Load -

max reaction. Uniform load and motor truck load combined see pp 8.

Unif. Load say 12750 × 3.5 = 44600
Conc. say 18530
63130#
Dead Load added 98500
161630#

Pin 4¼" dia. pin. Bearing on pin = $\frac{161630}{4.25 \times 1.125 \times 2}$ = 16900#/□".

Bending moment on pin. 80800 × 1.375 = 111000 in#
Unit fibre stress on pin = 18000#/□". ok.

Max Load on Bearing From above 161630
shoes say 870
162500# on masonry

Pin 3½" rollers 3.5 × 610 = 2140# per lin inch
162500 ÷ 2140 = 76" net or 5 rollers 15.2" net.

Size of base for fixed shoe 19¼" × 20" = 384□"
Unit bearing = 162500 ÷ 384 = 424#/□".

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CALCULATIONS FOR

Nakuchi-Bashi for Tokushima-Ken

Design of 459' suspension span with 102'-0" side spans.
Roadway 20'-0" clear same as for 102' simple span
Center span divided into 36 panels @ 12'-9" each = 459'-0"
Side span divided into 8 panels @ 12'-9" each = 102'-0"

Floor slab same as for simple span 5½" Reinforced concrete slab 1½" Soliditit Concrete Pavement For calculation see pp. 3

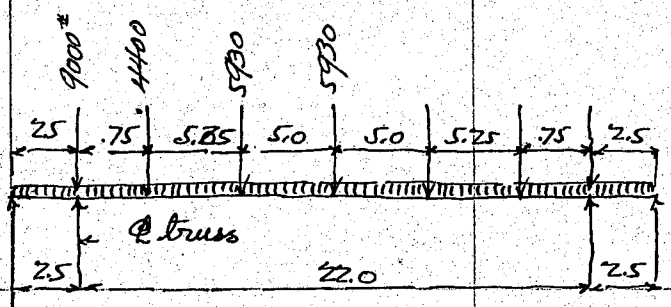
Stringers for side spans and Center span
Use 12"×5" I @ 31.88# for intermediate stringers see pp 3-4
1Pl 12"×5/16 2L's 3"×3"×5/16 for End stringers. see pp 4

Floor Beams for side spans
same as simple span see pp 5

End floor beam for side span same as for simple span see pp 5 to 6

Floor Beam for Center span with hanger connection.
Portion between main trusses = 22'-0", same as for ordinary intermediate floor beam and Overhanging portion 2'-6" each both ends.

Typical Intermediate floor Beam
Dead Load of flooring same as for simple beam see pp. 4



℄ of Suspender
floor beam = 100# per ft

Total load =

From floor	12795
truss	9000
Floor beam	1350
	23145

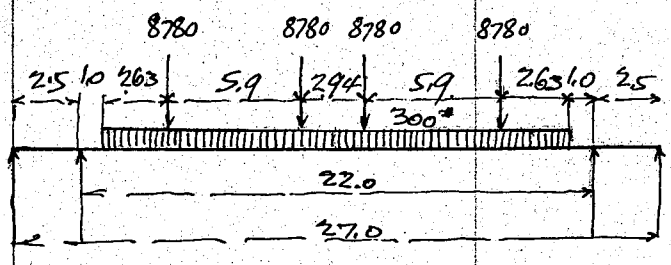
Moment at truss = 11570# × 2.5 = 29000'#

Dead Load from truss 670# per lin ft of truss
Panel Concentration = 670 × 12.75 = 8550#
D.L. Lower Lateral Bracing 32# per lin ft
Panel conc = 32 × 12.75 = 410#
Total panel conc = 8550 + 410 = 8960# say 9000#

Assumed the load carried equally by truss and cable
Assume reaction 11570# each
Moment at center

11570	× 13.5	=	156.000
11570	× 11.0	=	127300
			283300'#
5930	× 5.0	=	29600
4400	× 10.25	=	45100
9000	× 11.0	=	99000
1350	× 6.75	=	9100
			182800
			100500'#

Live Load refer to pp 5



Reaction 2@8780 = 17560
3000
20560#

10280# each
Moment at center

10280	× 13.5	=	138900
10280	× 11.0	=	113000
			251900
8780	× 1.47	=	12900
8780	× 7.37	=	64700
300	× 10²/₂		15000
			92600
			159300'#

Moment at ℄ truss = 10280 × 2.5 = 25700'#

Summary for moments

	at ℄ truss			
DL m	100500	29000	8780 × 1.47	= 12900
LL m	159300	25700	8780 × 7.37	= 64700
	259800'#	54700'#	300 × 10²/₂	= 15000

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushimaken

Intermediate floor beam continued
Typical floor beam depth 27½"
Dy 27" ÷ 5/16 = 8.43" say 8 web = 1.05 Depth of beam 2-3½" b to b of Ls
Effective depth 2.15' stress = 120900# section reqd = 120900 ÷ 17000 = 7.10
1.05
6.05" net

Use 2Ls 5×3½×7/16" = 7.06 0" gross or 6.300" net.

At ℄ of truss :- moment = 54700'#
Stress in bottom flange 54700 ÷ 2.15 = 25400# section reqd = 1.50"
For detail increase this 50% or say 2.25 0" net
Stress say 38000# no of rivets 8-3/4"ϕ

at Panel Point LC5 depth of beam 2-9"
web assumed = 32½ × 5/16 = 10.18 ⅛ web = 1.27 depth = 2-9" back to back of Ls
Effective depth = 2.61 stress = 259700 ÷ 2.61 = 99500# SR = 99500 ÷ 17000 = 5.85
1.27
4.58" net

Use 2Ls 5×3½×3/8" = 6.100" gross or 4.780" net.

Floor Beam at Panel Point LC4 depth of beam 3-2½"
web assumed = 38 × 5/16 = 11.900" ⅛ web = 1.49 depth = 3-2½ back to back of Ls
Effective depth = 3.07 stress = 259700 ÷ 3.07 = 84500# SR = 84500 ÷ 17000 = 4.99
1.49
3.50" net

Use 2Ls 5×3½×3/8 = 6.100" gr. or 4.780" net.

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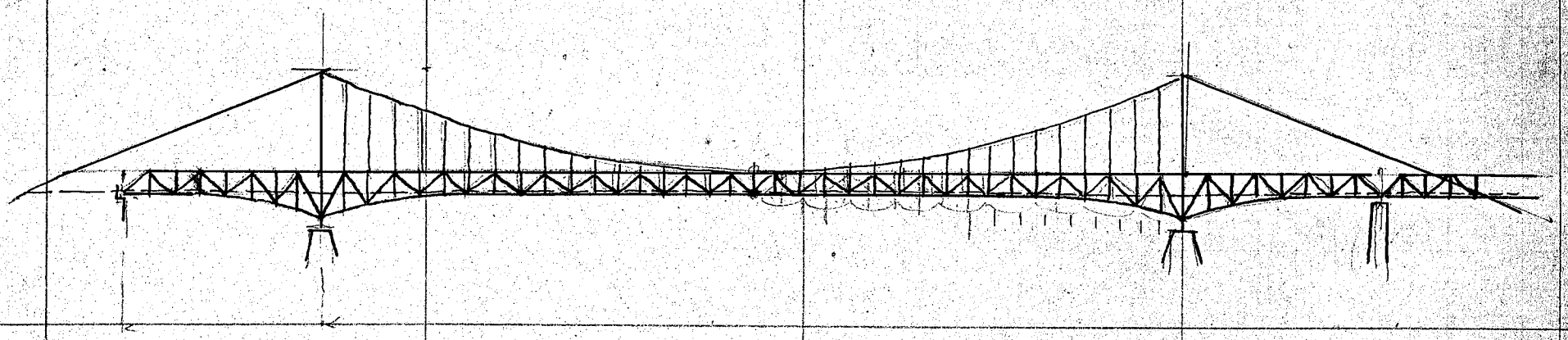
CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Design of suspension bridge -

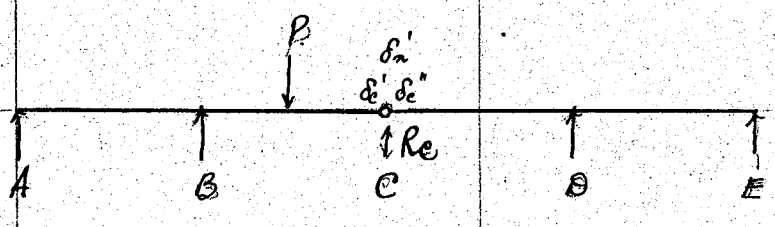
The general dimensions of the bridge are determined as shown in sketch below.

On account of erection purpose and stiffness of bridge we determined to use the stiffening truss cantilevered out from side span toward ℄ of span where the said truss will be hinged. Considering location of bridge the side span limited to 102'-0" and the anchorage beyond the end of the bridge, and therefore we adopted straight back stay instead of using suspender type of stay.



When the cable is removed the stiffening truss forms a beam continuous over 4 supports, but hinged at the center of the main span. As the truss is hinged at both ends of side spans also the moments at these points are zero. The moment of inertia of truss assumed constant throughout.

Reaction at Center Hinge.



δ_n' = Deflection at C of the structure AC, for a single load P placed at any point of AC.

δ_c' = Deflection at C of the structure AC, for unit load applied at C

δ_c'' = Deflection at C of the structure CE for unit load placed at C.

If the structures be connected at common point C will deflect a distance δ_n under the load P, which will be less than δ_n' by reason of the restraining action of the structure CE.

Let Rc be the reaction at C, assumed upward on AC.

Then we have, with respect to the structure AC $\quad \delta_n = \delta_n' - R_c \delta_c'$

and with respect to the structure CE $\quad \delta_n = R_c \delta_c''$

Eliminating δ_n we have

$$R_c = \frac{\delta_n'}{\delta_c' + \delta_c''}$$

If the two structures are alike and symmetrical about hinge C $\quad \delta_c' = \delta_c''$

Then

$$R_c = \frac{\delta_n'}{2\delta_c'}$$

But $\frac{\delta_n'}{\delta_c'}$ is the right reaction for the structure AC on rigid supports, since the reaction Rc is one half that of a structure on rigid supports.

The stresses in stiffening truss with center hinge.

Single load P in the main span, a distance kl from the left support, where $k < \frac{1}{2}$, the moments at supports are as follows.

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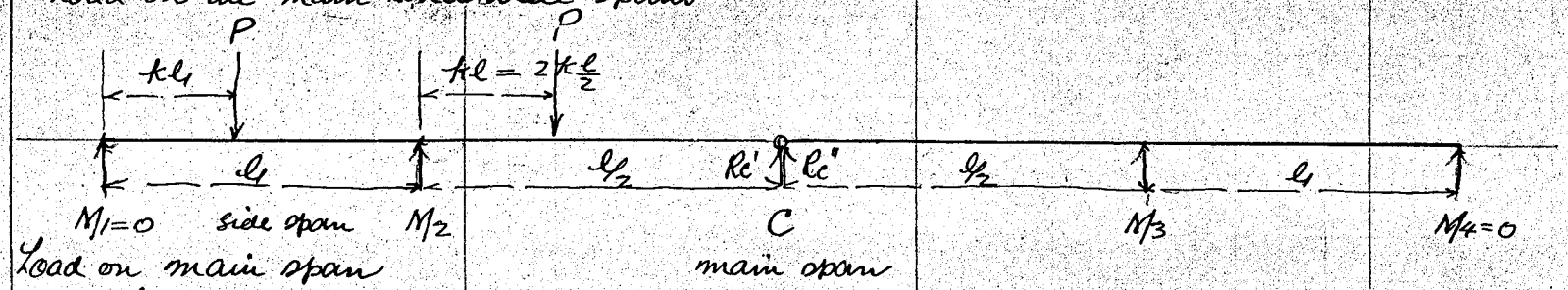
CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Load on the main and side spans



Load on main span
Center hinge as rigid support

$$M_1 l + 2M_2\left(l + \frac{l}{2}\right) + M_3' \frac{l}{2} = -P 2k \left(\frac{l}{2}\right)^2 (2 - 6k + 4k^2)$$

Placing $M_1 = M_3'$ equal to zero,

$$2M_2\left(l + \frac{l}{2}\right) = -2Pk\frac{l^2}{4}(2 - 6k + 4k^2)$$

$$M_2 = -\frac{Pk\frac{l^2}{2}(2 - 6k + 4k^2)}{2\left(l + \frac{l}{2}\right)}$$

$$Rc = -\frac{Pk\frac{l^2}{2}(2 - 6k + 4k^2)}{2\left(l + \frac{l}{2}\right)\cdot\frac{l}{2}} + \frac{Pkl}{\frac{l}{2}}$$

$$= -\frac{Pk\frac{l}{2}(2 - 6k + 4k^2)}{\left(l + \frac{l}{2}\right)} + 2Pk$$

$$= -\frac{\frac{Pkl}{2}(2 - 6k + 4k^2)}{\left(\frac{2l_1}{l} + 1\right)\frac{l}{2}} + 2Pk$$

For Elastic support

$$Rc' = -\frac{\frac{Pkl}{2}(2 - 6k + 4k^2)}{2\left(1 + \frac{2l_1}{l}\right)\frac{l}{2}} + Pk$$

Moment M_2 due to elastic support

$$M_2 = -\frac{\frac{Pkl}{2}(2 - 6k + 4k^2)\times\frac{l}{2}}{2\left(1 + \frac{2l_1}{l}\right)\frac{l}{2}} + Pk\cdot\frac{l}{2} - Pkl.$$

$$= -\frac{\frac{Pkl}{2}\left\{(2 - 6k + 4k^2) + 2\left(1 + \frac{2l_1}{l}\right)\right\}}{2\left(1 + \frac{2l_1}{l}\right)}$$

$$= -\frac{\frac{Pkl}{2}\left(4 - 6k + 4k^2 + \frac{4l_1}{l}\right)}{2\left(1 + \frac{2l_1}{l}\right)}$$

$$= -\frac{\frac{Pkl}{2}\left(2 - 3k + 2k^2 + \frac{2l_1}{l}\right)}{\left(1 + \frac{2l_1}{l}\right)}$$

For Elastic support

$$Rc'' = -\frac{\frac{Pkl}{2}(2 - 6k + 4k^2)}{2\left(1 + \frac{2l_1}{l}\right)\frac{l}{2}} - Pk$$

Moment M_3 due to Elastic support

$$M_3 = \frac{\frac{Pkl}{2}(2 - 6k + 4k^2)\frac{l}{2}}{2\left(1 + 2\frac{l_1}{l}\right)\frac{l}{2}} - Pk\frac{l}{2}$$

$$= \frac{\frac{Pkl}{2}\left\{(2 - 6k + 4k^2) - 2\left(1 + \frac{2l_1}{l}\right)\right\}}{2\left(1 + \frac{2l_1}{l}\right)}$$

$$= \frac{\frac{Pkl}{2}\left(-6k + 4k^2 - \frac{4l_1}{l}\right)}{2\left(1 + \frac{2l_1}{l}\right)}$$

$$= -\frac{\frac{Pkl}{2}\left(3k - 3k^2 + 2\frac{l_1}{l}\right)}{\left(1 + \frac{2l_1}{l}\right)}$$

Load on side span
Center hinge as rigid support

$$M_1 l + 2M_2\left(l + \frac{l}{2}\right) + M_3'\frac{l}{2} = -Pkl^2(1 - k^2). \quad \text{where } M_1 = M_3' = 0$$

$$M_2 = -\frac{Pkl^2(1 - k^2)}{2\left(l + \frac{l}{2}\right)}$$

$$Rc' = \frac{Pkl^2(1 - k^2)}{2\left(\frac{2l_1}{l} + 1\right)\frac{l}{2}\times\frac{l}{2}} = \frac{Pkl(1 - k^2)}{\left(1 + \frac{2l_1}{l}\right)\frac{l}{2}}$$

For Elastic support

$$Rc' = -\frac{\frac{Pkl}{2}(1 - k^2)}{\left(1 + \frac{2l_1}{l}\right)\frac{l}{2}}$$

$$M_2 = -\frac{\frac{Pkl}{2}(1 - k^2)}{\left(1 + \frac{2l_1}{l}\right)} \qquad M_3 = -M_2.$$

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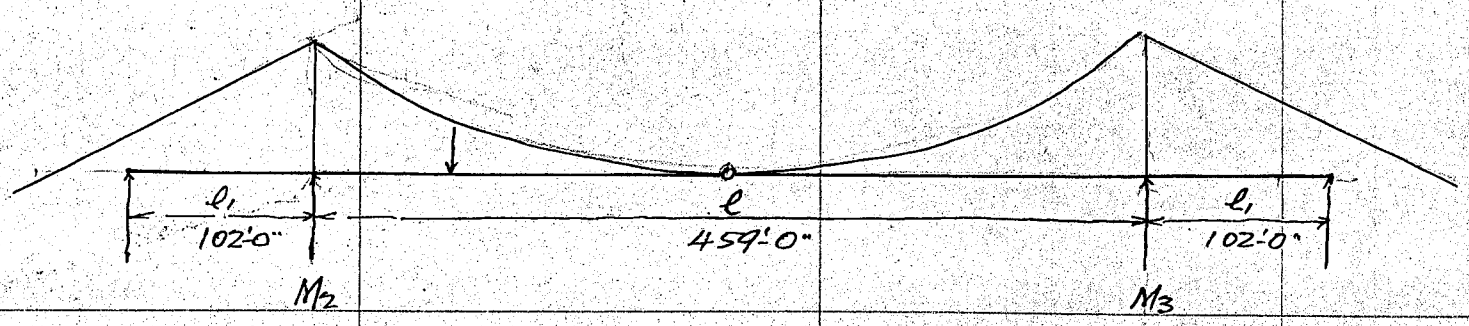
CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-ken

Stresses due to unit load on center span
Continuous over towers, hinge at centre.



$$M_2 = -\frac{\frac{Pkl}{2}\left[(2-3k+2k^2)+\frac{2l_1}{l}\right]}{\left(1+\frac{2l_1}{l}\right)}$$

$$M_3 = -\frac{\frac{Pkl}{2}\left[3k-2k^2+\frac{2l_1}{l}\right]}{\left(1+\frac{2l_1}{l}\right)}$$

$1+\frac{2l_1}{l} = 1+\frac{2\times102}{459} = 1.4444$ $\quad \frac{2l_1}{l} = 0.4444$ $\quad P$ = assumed = 1.0

Panel Pt.	k	k^2	$3k$	$2+2k^2-3k$	$2+2k^2-3k+\frac{2l_1}{l}$	$\frac{Pkl}{2}$	$\frac{Pkl}{2(1+\frac{2l_1}{l})}$	M_2
1	0.0278	0.0005	0.0833	1.9177	2.3621	6.3801	4.4170	−10.4334
2	0.0556	0.0031	0.1667	1.8395	2.7839	12.7602	8.8340	−20.1760
3	0.0833	0.0069	0.2500	1.7638	2.2082	19.1403	13.2510	−29.2609
4	0.1111	0.0123	0.3333	1.6913	2.1357	25.5204	17.668	−37.7335
5	0.1389	0.0193	0.4167	1.6219	2.0663	31.9005	22.085	−45.6342
6	0.1667	0.0278	0.5000	1.5556	2.0000	38.2806	26.502	−53.0040
7	0.1944	0.0378	0.5833	1.4923	1.9367	44.0067	30.919	−59.8808
8	0.2222	0.0494	0.6667	1.4321	1.8765	51.0408	35.336	−66.3080
9	0.2500	0.0625	0.7500	1.3750	1.8194	57.4709	39.753	−72.3266
10	0.2778	0.0772	0.8333	1.3211	1.7655	63.8010	44.170	−77.9821
11	0.3056	0.0934	0.9167	1.2701	1.7145	70.1811	48.587	−83.3024
12	0.3333	0.1111	1.0000	1.2222	1.6666	76.5612	53.004	−88.3365
13	0.3611	0.1304	1.0833	1.1775	1.6219	82.9413	57.421	−93.1311
14	0.3889	0.1512	1.1667	1.1357	1.5801	89.3214	61.838	−97.7102
15	0.4166	0.1736	1.2500	1.0972	1.5416	95.7015	66.255	−102.1387
16	0.4444	0.1975	1.3333	1.0617	1.5061	102.0816	70.672	−106.4391
17	0.4722	0.2230	1.4167	1.0293	1.4737	108.4617	75.089	−110.6587
18	0.5000	0.2500	1.5000	1.0000	1.4444	114.8418	79.506	−114.8385

	$3k-2k^2$	$3k-2k^2+\frac{2l_1}{l}$	$\frac{Pkl}{2(1+\frac{2l_1}{l})}$	M_3
1	0.0823	0.5267	4.417	−2.3264
2	0.1605	0.6049	8.834	−5.3437
3	0.2362	0.6806	13.251	−9.0186
4	0.3087	0.7531	17.668	−13.3058
5	0.3781	0.8225	22.085	−18.1649
6	0.4444	0.8888	26.502	−23.5550
7	0.5077	0.9521	30.919	−29.4380
8	0.5679	1.0123	35.336	−35.7706
9	0.6250	1.0694	39.753	−42.5119
10	0.6789	1.1233	44.170	−49.6162
11	0.7299	1.1743	48.587	−57.0557
12	0.7778	1.2222	53.004	−64.7815
13	0.8225	1.2669	57.421	−72.7467
14	0.8643	1.3087	61.838	−80.9274
15	0.9068	1.3512	66.255	−89.5238
16	0.9383	1.3827	70.675	−97.7182
17	0.9707	1.4151	75.089	−106.2584
18	1.0000	1.4444	79.506	−114.8385

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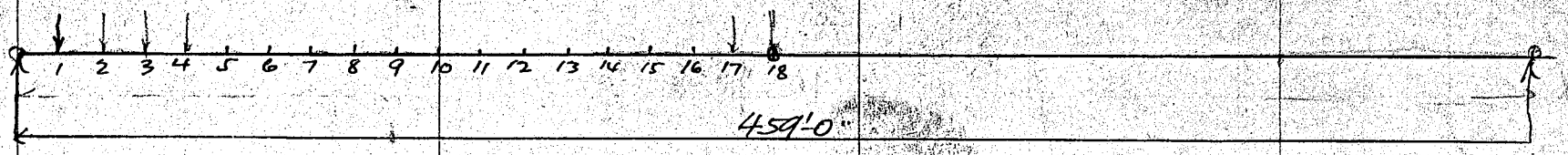
CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-ken

Center Span moments as simple beam



Load	M'1	M'2	M'3	M'4	M'5	M'6	M'7	M'8	M'9
1	0.3541	0.7083	1.0624	1.4166	1.7708	2.1249	2.4791	2.8332	3.1874
2	0.7083	1.4166	2.1250	2.8333	3.5416	4.2499	4.9582	5.6666	6.3749
3	1.0624	2.1250	3.1874	4.2499	5.3124	6.3749	7.4374	8.4998	9.5623
4	1.4166	2.8333	4.2499	5.6666	7.0832	8.4998	9.9165	11.3331	12.7498
5	1.7708	3.5416	5.3124	7.0832	8.8540	10.6248	12.3956	14.1664	15.9372
6	2.1249	4.2499	6.3749	8.4998	10.6248	12.7498	14.8747	16.9997	19.1246
7	2.4791	4.9582	7.4374	9.9165	12.3956	14.8747	17.3538	19.8330	22.3121
8	2.8332	5.6666	8.4998	11.3331	14.1664	16.9997	19.8330	22.6662	25.4995
9	3.1874	6.3749	9.5623	12.7498	15.9372	19.1446	22.3121	25.4995	28.6870
10	3.5416	7.0832	10.6248	14.1664	17.7080	21.2496	24.7912	28.3328	31.8744
11	3.8957	7.7915	11.6873	15.5830	19.4788	23.3746	27.2703	31.1661	35.0618
12	4.2499	8.4998	12.7498	16.9997	21.2496	25.4995	29.7494	33.9994	38.2493
13	4.6040	9.2082	13.8122	18.4163	23.0204	27.6245	32.2286	36.8326	41.4367
14	4.9582	9.9165	14.8747	19.8330	24.7912	29.7494	34.7077	39.6659	44.6242
15	5.3124	10.6248	15.9372	21.2496	26.5620	31.8744	37.1868	42.4992	47.8116
16	5.6665	11.3331	16.9997	22.6662	28.3328	33.9994	39.6659	45.3325	50.9990
17	6.0207	12.0414	18.0622	24.0829	30.1036	36.1243	42.1450	48.1658	54.1865
18	6.3748	12.7498	19.1246	25.4995	31.8744	38.2493	44.6242	50.9990	57.3739
19	6.7290	13.4581	20.1871	26.9162	33.6452	40.3742	47.1033	53.8323	60.5614
16	7.0832	14.1664	21.2496	28.3328	35.4160	42.4992	49.5824	56.6656	63.7488
15	7.4373	14.8747	22.3121	29.7494	37.1868	44.6242	52.0615	59.4989	66.9362
14	7.7915	15.5830	23.3746	31.1661	38.9576	46.7491	54.5406	62.3322	70.1237
13	8.1456	12.2914	24.4370	32.5827	40.7284	48.8741	57.0198	65.1654	73.3111
12	8.4998	16.9997	25.4995	33.9994	42.4992	50.9990	59.4989	67.9987	76.4786
11	8.8540	17.7080	26.5620	35.4160	44.2700	53.1240	61.9780	70.8320	79.6860
10	9.2081	18.4163	27.6245	36.8326	46.0408	55.2490	64.4571	73.6653	82.8734
9	9.5623	19.1246	28.6870	38.2493	47.8116	57.3739	66.9362	76.4986	86.0609
8	9.9164	19.8330	29.7494	39.6659	49.5824	59.4989	69.4154	79.3318	76.4983
7	10.2706	20.5413	30.8119	41.0826	51.3532	61.6238	71.8945	69.4151	66.9358
6	10.6248	21.2496	31.8744	42.4992	53.1240	63.7488	61.6236	59.4984	57.3732
5	10.9789	21.9579	32.9369	43.9158	54.8948	53.1238	51.3527	49.5817	47.8106
4	11.3331	22.6662	33.9994	45.3325	43.9156	42.4987	40.0818	39.6650	38.2481
3	11.6872	23.3746	35.0618	33.9991	32.9364	31.8737	30.8110	29.7482	28.6855
2	12.0414	24.0829	24.3743	22.6658	21.9572	21.2486	20.5401	19.8315	19.1230
1	12.3956	12.0412	11.6868	11.3324	10.9780	10.6236	10.2692	9.9148	9.5604

Left.

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Center span moments as simple beam

	M10'	M11	M12	M13	M14	M15	M16	M17	M18
1	3.5416	3.8957	4.2499	4.6040	4.9582	5.3124	5.6665	6.0207	6.3748
2	7.0832	7.7915	8.4998	9.2082	9.9165	10.6248	11.3331	12.0414	12.7498
3	10.6248	11.6873	12.7498	13.8122	14.8747	15.9372	16.9997	18.0622	19.1246
4	14.1664	15.5830	16.9997	18.4163	19.8330	21.2496	22.6662	24.0829	25.2995
5	17.7080	19.4788	21.2496	23.0204	24.7912	26.5620	28.3328	30.1036	31.8744
6	21.2496	23.3746	25.4995	27.6245	29.7494	31.8744	33.9994	32.1243	38.2493
7	24.7912	27.2703	29.7494	32.2286	34.7077	37.1868	39.6659	42.1450	44.6242
8	28.3328	31.1661	33.9994	36.8326	39.6659	42.4992	45.3325	48.1658	50.9990
9	31.8744	35.0618	38.2493	41.4367	44.6242	47.8116	50.9990	54.1865	57.3739
10	35.4160	38.9576	42.4992	46.0408	49.5824	53.1240	56.6656	60.2072	63.7488
11	38.9576	42.8534	46.7491	50.6449	54.5406	58.4364	62.3322	66.2279	70.1237
12	42.4992	46.7491	50.9990	55.2490	59.4989	63.7488	67.9987	72.2486	76.4986
13	46.0408	50.6449	55.2490	59.8530	64.4571	69.0612	73.6653	78.2694	82.8734
14	49.5824	54.5406	59.4989	64.4571	69.4154	74.3736	79.3318	84.2901	89.2483
15	53.1240	58.4364	63.7488	69.0612	74.3736	79.6860	84.9984	90.3108	95.6232
16	56.6651	62.3322	67.9987	73.6653	79.3318	84.9984	90.6650	96.3315	101.9981
17	60.2072	66.2279	72.2486	78.2694	84.2901	90.3108	96.3315	102.3522	108.3730
18	63.7488	70.1237	76.4986	82.8734	89.2483	95.6232	101.9981	108.3730	114.7500
17	67.2904	74.0194	80.7485	87.4775	94.2066	100.9356	107.6646	114.3937	108.3727
16	70.8320	77.9152	84.9984	92.0816	99.1648	106.2480	113.3312	107.6644	101.9976
15	74.3736	81.8110	89.2483	96.6857	104.1230	111.5604	106.2478	100.9351	95.6225
14	77.9152	85.7067	93.4982	101.2898	109.0813	104.1228	99.1643	94.2058	89.2474
13	81.4568	89.6025	97.7482	105.8938	101.2895	96.6852	92.0809	87.4766	82.8722
12	84.9984	93.4982	101.9981	97.7479	93.4978	89.2476	84.9974	80.7473	76.4971
11	88.5400	97.3940	93.4980	89.6020	85.4060	81.5100	77.9140	74.0180	70.1220
10	92.0816	88.5398	84.9979	81.4561	77.9142	74.3724	70.8306	67.2887	63.7469
9	87.8732	79.6855	76.4978	73.3102	70.1225	66.9348	63.7471	60.5594	57.3718
8	73.6648	70.8313	67.9978	65.1642	62.3307	59.4972	56.6637	53.8302	50.9966
7	64.4564	61.9770	59.4977	57.0183	54.5390	52.0596	49.5802	47.1009	44.6215
6	55.2480	53.1228	50.9976	48.8724	46.7472	44.6220	42.4968	40.3716	38.2464
5	46.0396	44.2686	42.4975	40.7265	38.9554	37.1844	35.4134	33.6423	31.8713
4	36.8312	35.4143	33.9974	32.5806	31.1637	29.7468	28.3299	26.9130	25.4962
3	27.6228	26.5601	25.4974	24.4346	23.3710	22.3092	21.2465	20.1838	19.1210
2	18.4144	17.7058	16.9973	16.2887	15.5802	14.8716	14.1630	13.4545	12.7459
1	9.2060	8.8516	8.4972	8.1428	7.7884	7.4340	7.0796	6.7252	6.3708

Horizontal Pull Center span (see pp. 262 Johnson's Framed structures Part II).

$$H = \frac{\frac{1}{6} P k l f \left[k(2k^2 - 4k + 3) + \frac{2l_1}{l}\right]}{\frac{f^2}{15}\left[3 + 10\frac{l_1}{l}\right] + \frac{I L_s}{l A}}$$ Equ (68).

L_s Equ (68) $= L_s$ eq (7) $+ 2l_1 \sec^3 \alpha_1$ for symmetrical backstays. ($2l_1$ ↙ C)

$$L_s \text{ eq (7)} = \int_0^L ds^3 = 2\int_0^{\frac{l}{2}} \left[1 + \frac{64 f^2 x^2}{l^4}\right]^{\frac{3}{2}} dx$$

$$= l\left\{\underbrace{\frac{1}{4}\left(\frac{5}{2} + \frac{16f^2}{l^2}\right)\left(1 + \frac{16f^2}{l^2}\right)^{\frac{1}{2}}}_{A} + \underbrace{\frac{3}{32}\frac{l}{f}\log_e\left[\frac{4f}{l} + \left(1 + \frac{16f^2}{l^2}\right)^{\frac{1}{2}}\right]}_{B}\right\}$$

where $f = 55.0'$ $l = 459'-0'$ $2l_1 =$ say $325'-0''$ $\sec\alpha_1 = 1.125$

$A = 0.7569$

$B = \underline{0.3620}$

$1.1189 \times 459. = 5135.75$

$C = 2l_1 \sec^3\alpha_1 = 1.4238 \times 325 = \underline{462.735}$

5598.485

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CALCULATIONS FOR

Nakuchi-Bashi for Tokushima-Ken

Moment of inertia of cable $0.0037 (ft)^4$
Area of cable $0.195 (ft)^2$
$\frac{I}{A} = 0.019$

$\frac{I}{Ae} Ls$ eq (68) $= \frac{5598.485 \times 0.019}{459} = 0.226$

$\frac{f^2}{15}\left(3 + 10\frac{e}{\ell}\right) = 1501.290$

$\frac{I}{Ae} Ls$ eq (68) $= \frac{.226}{1501.516}$

Summary for denominator call this 1501.5

	$k(3k^2-4(k+3^2))$	$+2\frac{e}{\ell}$		$\frac{4459 \times 155}{459 \times 6}$ k	Nominator	Denominator	H.
1	0.0803	+ 4444	= 0.5247	× 116.969	= 61.3736	÷ 1,501.5	= 0.0409
2	0.1546	"	= 0.5990	× 233.937	140.1283		0.0933
3	0.2227	"	= 0.6671	× 350.485	233.8085		0.1557
4	0.2857	"	= 0.7295	× 467.453	341.0070		0.2271
5	0.3422	"	= 0.7866	× 584.422	459.7063		0.3062
6	0.3936	"	= 0.8380	× 701.390	587.7648		0.3915
7	0.4393	"	= 0.8837	× 817.938	722.1181		0.4809
8	0.4801	"	= 0.9245	× 934.907	864.3215		0.5756
9	0.5156	"	= 0.9600	× 1051.875	1009.8000		0.6725
10	0.5462	"	= 0.9906	× 1168.844	1157.8568		0.7711
11	0.5718	"	= 1.0162	× 1285.812	1306.6422		0.8702
12	0.5925	"	= 1.0369	× 1402.360	1454.1071		0.9684
13	0.6016	"	= 1.0460	× 1519.328	1589.2171		1.0584
14	0.6205	"	= 1.0649	× 1636.297	1742.4927		1.1605
15	0.6278	"	= 1.0722	× 1752.845	1879.4004		1.2517
16	0.6309	"	= 1.0753	× 1869.813	2010.6099		1.3390
17	0.6300	"	= 1.0744	× 1986.782	2134.5986		1.4216
18	0.6250	"	= 1.0694	× 2103.750	2249.7503		1.5116

Summary of H 1 to 17 = 11.7846
2
23.5692
1.5116

Horizontal Pull for full load —— 25.0808.

Moments at towers due to load on side span

$$M_2 = -\frac{P_1 k_1 l_1 (1-k_1^2)\frac{e_1}{e}}{(1+2\frac{e_1}{e})}$$

	k_1	k_1^2	$1-k_1^2$	$\frac{k_1 l_1^2}{2e}(1+2\frac{e_1}{e})$	M_2
1	0.1250	0.0156	0.9844	× 0.9807	− 0.9654
2	0.2500	0.0625	0.9375	× 1.9615	− 1.8389
3	0.3750	0.1406	0.8594	× 2.9423	− 2.5286
4	0.5000	0.2500	0.7500	× 3.9231	− 2.9423
5	0.6250	0.3906	0.6094	× 4.9038	− 2.9884
6	0.7500	0.5625	0.4375	× 5.8846	− 2.5745
7	0.8750	0.7656	0.2344	× 6.8653	− 1.6092
8	1.0000	1.0000	0.0000	× 7.8461	− 0.0000

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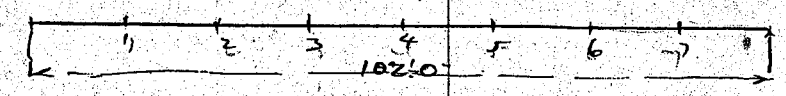
CALCULATIONS FOR

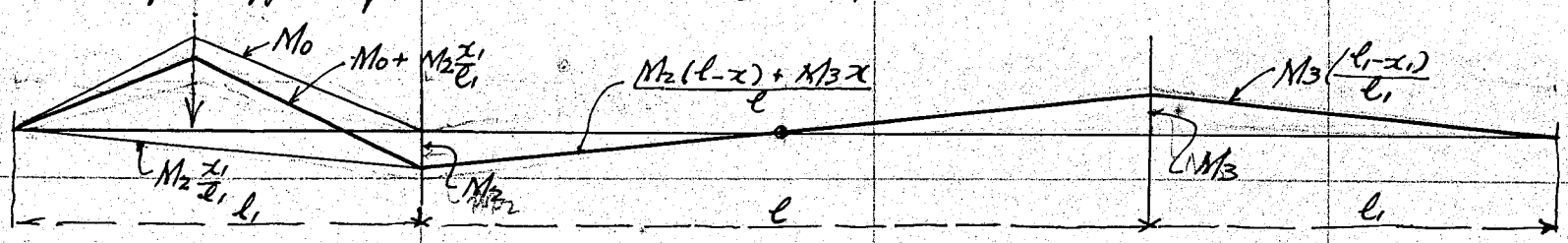
Nakuchi-Bashi for Tokushima-ken

Moments of side span as simple beam



Load at	M'_1	M'_2	M'_3	M'_4	M'_5	M'_6	M'_7
7	1.5938	3.1875	4.7812	6.375	7.9687	9.5625	11.1563
6	3.1875	6.3750	9.5625	12.7500	15.9375	19.1250	9.5625
5	4.7813	9.5625	14.3437	19.1250	23.9063	15.9375	7.9688
4	6.3750	12.7500	19.1250	25.5000	19.1250	12.7500	6.3750
3	7.9688	15.9375	23.9060	19.1250	14.3438	9.5625	4.7813
2	9.5625	19.1250	15.9375	12.7500	9.5625	6.3750	3.1875
1	11.1563	9.5625	7.9688	6.3750	4.7812	3.1875	1.5938
0							

Moments of stiffening truss Load on side span



Load at		1	2	3	4	5	6	7
1	Left	+11.0357	+9.3327	7.6528	6.0072	4.4077	2.8657	1.3930
2		+9.3212	+18.6653	15.3053	12.0144	8.8154	5.7314	2.7855
3		+7.6068	+15.2479	22.9578	18.0217	13.2231	8.5971	4.1780
4		+5.8923	+11.8305	17.8607	24.0289	17.6308	11.4627	5.5706
5		+4.1779	+8.4132	12.7633	17.2860	22.0385	14.3284	6.9631
6		+2.4635	+4.9958	7.6661	10.5433	13.6762	17.1941	8.3556
7		+0.7491	+1.5785	2.5688	3.8005	5.3539	7.3098	9.7482
8		−0.9654	−1.8389	−2.5286	−2.9423	−2.9884	−2.5745	−1.6092
9		−0.9118	−1.7367	−2.3881	−2.7788	−2.8224	2.4315	1.5198
2		−0.8581	−1.6345	−2.2476	−2.6154	−2.6564	2.2884	1.4304
3		−0.8045	−1.5324	−2.1072	−2.4519	−2.4903	2.1454	1.3410
4		−0.7509	−1.4303	−1.9667	−2.2885	−2.3243	2.0024	1.2516
5		−0.6972	−1.3281	−1.8262	−2.1250	−2.1583	1.8594	1.1622
6		−0.6436	−1.2259	−1.6857	−1.9615	−1.9923	1.7163	1.0728
7		−0.5900	−1.1238	−1.5452	−1.7981	−1.8262	1.5733	0.9834
8		−0.5363	−1.0216	−1.4048	−1.6346	−1.6602	1.4303	0.8940
9		−0.4827	−0.9194	−1.2642	−1.4711	−1.4942	1.2873	0.8046
10		−0.4291	−0.8173	−1.1238	−1.3077	−1.3282	1.1442	0.7152
11		−0.3754	−0.7151	−0.9833	−1.1442	−1.1621	1.0012	0.6258
12		−0.3218	−0.6130	−0.8429	−0.9808	−0.9961	0.8582	0.5364
13		−0.2681	−0.5108	−0.7024	−0.8173	−0.8301	0.7151	0.4470
14		−0.2145	−0.4086	−0.5619	−0.6538	−0.6641	0.5721	0.3576
15		−0.1609	−0.3065	−0.4214	−0.4904	−0.4981	0.4291	0.2682
16		−0.1073	−0.2043	−0.2809	−0.3269	−0.3320	0.2861	0.1788
17		−0.0536	−0.1021	−0.1405	−0.1635	−0.1660	0.1430	0.0894
18	Centre	−0.0000	−0.0000	0.00	0.0	0.0	0.0	0.0
8′	Right	+0.9654	+1.8389	2.5286	2.9423	2.9884	2.5745	1.6092
7′		+0.8447	1.6090	2.2125	2.5745	2.6149	2.2527	1.4081
6′		+0.7240	1.3792	1.8964	2.2067	2.2413	1.9309	1.2069
5′		+0.6034	1.1493	1.5804	1.8390	1.8678	1.6091	1.0098
4′		+0.4827	0.9195	1.2643	1.4711	1.4942	1.2873	0.8046
3′		+0.3620	0.6896	0.9482	1.1033	1.1207	0.9654	0.6035
2′		+0.2413	0.4597	0.6322	0.7356	0.7471	0.6436	0.4023
1′		+0.1207	0.2298	0.3160	0.3678	0.3736	0.3218	0.2012

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JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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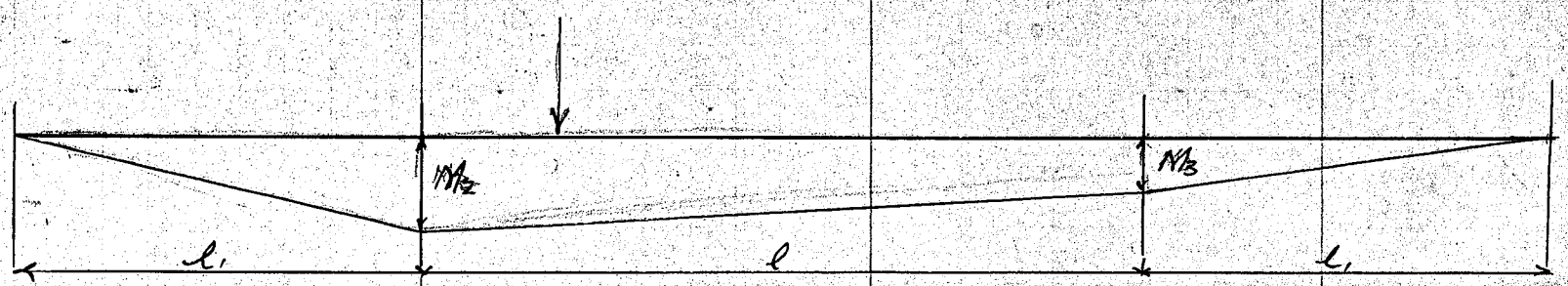
CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Moments in side spans Load on center span



Left span

Load at →	1	2	3	4	5	6	7	8	9
1	-1.3042	-2.5220	-3.6576	-4.7167	-5.7043	-6.6255	-7.4851	-8.2885	-9.0408
2	-2.6083	-5.0440	-7.2152	-9.4334	-11.4085	-13.2510	-14.9702	-16.5770	-18.0817
3	-3.9125	-7.5660	-10.9728	-14.1500	-17.1128	-19.8765	-22.4553	-24.8655	-27.1225
4	-5.2167	-10.0880	-14.6304	-18.8667	-22.8171	-26.5020	-29.9404	-33.1540	-36.1633
5	-6.5209	-12.6100	-18.2880	-23.5834	-28.5213	-33.1275	-37.4255	-41.4425	-45.2042
6	-7.8250	-15.1320	-21.9457	-28.3001	-34.2256	-39.7530	-44.9106	-49.7310	-54.2450
7	-9.1292	-17.6540	-25.6033	-33.0168	-39.9299	-46.3785	-52.3957	-58.0195	-63.2858
8	-10.4334	-20.1760	-29.2609	-37.7335	-45.6342	-53.0040	-59.8808	-66.3080	-72.3266

	10	11	12	13	14	15	16	17	18
1	-9.7478	-10.4128	-11.0421	-11.6414	-12.2138	-12.7673	-13.3049	-13.8324	-14.3548
2	-19.4955	-20.8256	-22.0842	-23.2828	-24.4276	-25.5346	-26.6098	-27.6648	-28.7096
3	-29.2433	-31.2384	-33.1263	-34.9242	-36.6414	-38.3019	-39.9147	-41.4972	-43.0644
4	-38.9910	-41.6512	-44.1684	-46.5656	-48.8552	-51.0692	-53.2196	-55.3296	-57.4192
5	-48.7388	-52.0640	-55.2105	-48.2070	-61.0690	-63.8365	-66.5245	-69.1620	-71.7742
6	-58.4866	-62.4768	-66.2526	-69.8484	-73.2528	-76.6038	-75.8294	-82.9944	-86.1288
7	-68.2343	-72.8896	-77.2947	-81.4908	-85.4966	-89.3711	-93.1343	-96.8268	-100.4836
8	-77.9821	-83.3024	-88.3365	-93.1311	-97.7102	-102.1387	-108.4391	-110.6587	-114.8385

Right span

	1	2	3	4	5	6	7	8	9
8'	-2.3264	-5.3437	-9.0186	-13.3058	-18.1649	-23.5550	-29.4380	-35.7706	-42.5119
7'	-2.0336	-4.6757	-7.8912	-11.6425	-15.8943	-20.6106	-25.7583	-31.2992	-37.1979
6'	-1.7448	-4.0078	-6.7639	-9.9793	-13.6237	-17.6662	-22.0785	-26.8279	-31.8839
5'	-1.4540	-3.3398	-5.6366	-8.3161	-11.3531	-14.7219	-18.3988	-22.3566	-26.5700
4'	-1.1632	-2.6719	-4.5093	-6.6529	-9.0824	-11.7775	-14.7190	-17.8853	-21.2560
3'	-0.8724	-2.0039	-3.3820	-4.9897	-6.8118	-8.8331	-11.0393	-13.4140	-15.9420
2'	-0.5816	-1.3359	-2.2546	-3.3264	-4.5412	-5.8887	-7.3595	-8.9426	-10.6280
1'	-0.2908	-0.6680	-1.1273	-1.6632	-2.2706	-2.9444	-3.6797	-4.4713	-5.3139

	10	11	12	13	14	15	16	17	18
8'	-49.6162	-57.0557	-64.5815	-72.7467	-80.9274	-89.5238	-97.7182	-106.2584	-114.8385
7'	-43.4141	-49.9237	-56.5088	-63.6534	-70.8113	-78.3335	-85.5036	-92.9761	-100.4836
6'	-37.2121	-42.7918	-48.4361	-54.5600	-60.6954	-67.1430	-73.2888	-79.6538	-86.1288
5'	-31.0101	-35.6598	-40.3635	-45.4667	-50.5795	-55.9525	-61.0740	-66.4115	-71.7740
4'	-24.8081	-28.5278	-32.2908	-36.3734	-40.4636	-44.7620	-48.8592	-53.1292	-57.4192
3'	-18.6061	-21.3959	-24.2181	-27.2800	-30.3477	-33.5715	-36.6444	-39.8469	-43.0644
2'	-12.4040	-14.2639	-16.1454	-18.1867	-20.2318	-22.3810	-24.4296	-26.5646	-28.7096
1'	-6.2020	-7.1319	-8.0727	-9.0933	-10.1159	-11.1905	-12.2148	-13.2823	-14.3548

増田淳氏関係資料
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JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY J. Masuda DATE 14-10-13 FILE NO
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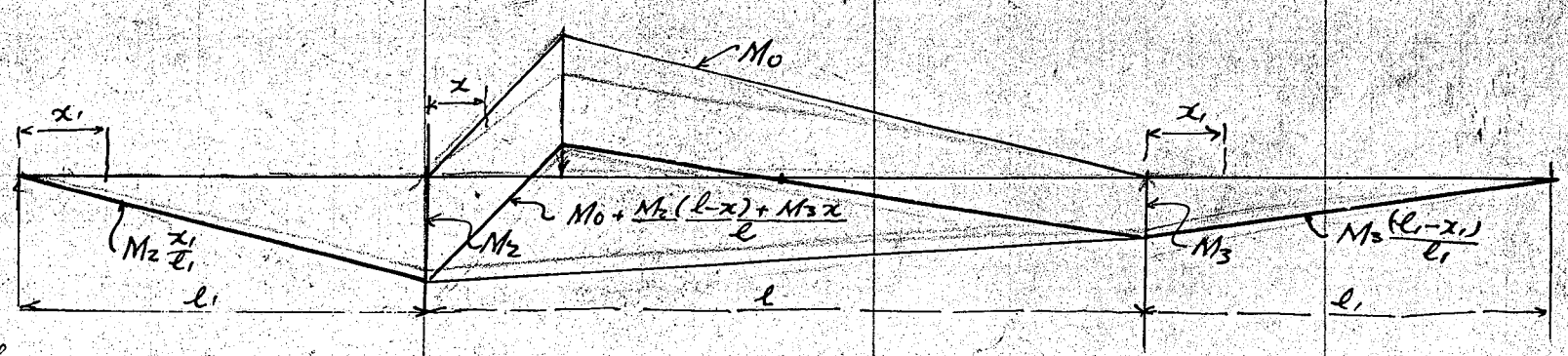
CALCULATIONS FOR

Kakuchi Bashi for Tokushima-Ken

Moments in Center span Load on Center span



Load at →	1	2	3	4	5	6	7	8	9
0	−10.4334	−20.1760	−29.2609	−37.7335	−45.6342	−53.0040	−59.8808	−66.3080	−72.3266
1	+ 2.1874	− 7.7228	−12.0018	−25.7225	−33.8932	− 41.5623	− 48.7661	−55.5450	− 61.9380
2	+ 2.0584	+ 4.8309	− 3.7620	−13.7106	− 22.1510	− 30.1194	− 37.6495	− 44.7801	− 51.5482
3	+ 1.9294	+ 4.4346	+ 7.4878	− +1.6988	− 10.4087	− 18.6762	− 26.5331	− 34.0151	−41.1565
4	+ 1.8005	4.1383	+ 6.9877	+10.0132	+ 1.3335	− 7.2332	− 16.4166	− 23.2500	− 30.7657
5	+ 1.6050	3.8419	6.1874	+ 9.5750	+ 13.0758	+ 4.2099	− 4.3001	− 12.4850	− 20.3751
6	+ 1.5426	3.5456	5.9872	8.8370	12.0680	+ 15.6530	+ 6.8165	− 1.7201	− 9.9842
7	+ 1.4135	3.2493	5.4870	8.0989	11.0602	14.3460	17.9330	+ 4.0449	+ 0.4066
8	+ 1.2746	2.9530	4.9868	7.3608	10.0052	13.0391	16.2995	19.8099	10.7972
9	+ 1.1556	2.6567	4.4867	6.6227	9.0447	11.7321	14.6660	17.8249	21.1880
10	+ 1.0268	2.3603	3.9861	5.8846	8.0369	10.4253	13.0325	15.8099	18.8287
11	+ 0.8978	2.0641	3.4862	5.1645	7.0292	9.1183	11.3990	13.8548	16.4694
12	+ 0.7687	1.7678	2.9860	4.4085	6.0214	7.8113	9.7656	11.8698	14.1102
13	+ 0.6398	1.4715	2.4858	3.6703	5.0137	6.5045	8.1312	9.8847	11.7509
14	+ 0.5106	1.1751	1.9857	2.9332	4.0059	5.1975	6.4986	7.8998	9.3917
15	+ 0.3879	0.8789	1.4855	2.1941	2.9979	3.8906	4.8651	5.9148	7.0329
16	+ 0.2529	0.5825	0.9853	1.4560	1.9904	2.5836	3.2316	3.9298	4.6732
17	+ 0.1239	0.2862	0.4851	0.7180	0.9826	1.2767	1.5982	1.9447	2.3140
18	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	−0.1292	−0.2969	−0.5010	− 0.7392	1.0092	− 1.3086	− 1.6354	− 1.9873	− 2.3618
16	−0.2585	−0.5937	−1.0020	− 1.4784	− 2.0183	− 2.6172	− 3.2709	− 3.9745	− 4.7236
15	− 0.3877	− 0.8906	− 1.5031	− 2.2176	− 3.0275	− 3.9258	− 4.9063	− 5.9618	− 7.0853
14	− 0.5170	− 0.1875	− 2.0041	− 2.9568	− 4.0366	− 5.2344	− 6.5418	− 7.9490	− 9.4471
13	− 0.6462	− 1.4844	− 2.5052	− 3.6961	− 5.0458	− 6.5431	− 8.1772	− 9.9363	− 11.8089
12	− 0.7755	− 1.7812	− 3.0062	− 4.4353	− 6.0550	− 7.8517	− 9.8127	− 11.9235	− 14.1707
11	− 0.9047	− 2.0781	− 3.5072	− 5.1745	− 7.0641	− 9.1603	− 11.4481	− 13.9108	− 16.5325
10	− 1.0340	− 2.3750	− 4.0083	− 5.9137	− 8.0733	− 10.4689	− 13.0836	− 15.8980	− 18.8942
9	− 1.1632	− 2.6718	− 4.5093	− 6.6529	− 9.0825	− 11.9775	− 14.7190	− 17.8853	− 21.2560
8	− 1.2924	− 2.9687	− 5.0103	− 7.3921	− 10.0916	− 13.0861	− 16.3545	− 19.8726	− 23.6178
7	− 1.4217	− 3.2656	− 5.5114	− 8.1313	− 11.1008	− 14.3947	− 17.9899	− 21.8598	− 25.9796
6	− 1.5509	− 3.5625	− 6.0124	− 8.8705	− 12.1099	− 15.7033	− 19.6254	− 23.8471	− 28.3414
5	− 1.6802	− 3.8593	− 6.5134	− 9.6098	− 13.1191	− 17.0120	− 21.2608	− 25.8343	− 30.7031
4	− 1.8094	− 4.1562	− 7.0145	− 10.3490	− 14.1283	− 18.3206	− 22.8962	− 27.8216	− 33.0649
3	− 1.9387	− 4.4531	− 7.5155	− 11.0882	− 15.1374	− 19.6292	− 24.5317	− 29.8088	− 35.4267
2	− 2.0679	− 4.7500	− 8.0165	− 11.8274	− 16.1466	− 20.9378	− 26.1671	− 31.7961	− 37.7885
1	− 2.1972	− 5.0468	− 8.5176	− 12.5666	− 17.1558	− 22.2464	− 27.8026	− 33.7834	− 40.1503
0	− 2.3264	− 5.3437	− 9.0186	− 13.3058	− 18.1649	− 23.5550	− 29.4380	− 35.7706	− 42.5119

増田淳氏関係資料
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JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY J. Masuda DATE 14-10-14 FILE NO ____
CHECKED BY ____ DATE ____ PAGE NO 23

CALCULATIONS FOR

"Kakuchi-Bashi" for Tokushima-Ken

Moments in Center span Load on center span

	10	11	12	13	14	15	16	17	18
0	-77.9821	-83.3024	-88.3365	-93.1311	-97.7102	-102.1387	-106.4391	-110.6587	-114.7500
1	-67.9882	-73.7217	-79.1850	-84.4222	-89.4556	-94.3914	-99.1172	-103.8105	-108.3733
2	-57.9919	-64.1384	-70.0306	-75.7100	-81.1975	-86.5664	-91.7916	-96.9589	-101.9981
3	-47.9505	-54.5551	-60.8762	-66.9974	-72.9396	-78.8784	-84.4657	-90.1074	-95.6232
4	-37.9991	-44.9718	-51.7219	-58.2858	-64.6816	-70.9904	-77.1402	-83.2560	-89.2483
5	-28.0029	-35.3884	-42.5674	-49.5736	-56.4237	-63.2024	-69.8144	-76.4044	-82.8734
6	-18.0065	-25.8051	-33.4131	-40.9857	-48.1658	-55.4143	-62.4888	-69.5530	-76.4986
7	-8.0101	-16.2219	-24.2587	-32.1493	-37.9078	-47.6263	-55.1632	-62.7195	-70.1237
8	+1.9862	-6.6385	-15.1042	-23.4371	-31.6499	-39.8383	-47.8374	-55.8498	-63.7488
9	+11.9825	+2.9448	-5.9498	-14.7250	-23.3919	-32.0502	-40.5118	-48.9985	-57.3739
10	+22.9789	12.5281	+3.2044	-6.0127	-15.1331	-24.2624	-33.1860	-42.1470	-50.9990
11	+17.2252	22.1114	12.3588	+2.6994	-7.1761	-16.4742	-25.8604	-35.2955	-44.6242
12	+16.4715	18.9447	21.5133	11.4115	+1.3819	-8.6862	-18.5348	-23.4439	-38.2493
13	+13.3179	15.7781	17.9177	20.1237	9.6399	-0.8982	11.2090	-21.5924	-31.8744
14	+10.9643	12.6114	14.3170	16.0894	17.8979	+6.8898	3.8834	-14.7410	-25.2995
15	+8.2105	9.4447	10.7264	12.0480	13.4057	+14.6778	+3.4424	-7.8895	-19.1246
16	+5.4569	6.2780	7.1308	8.0102	8.9137	+9.7159	+10.7680	-1.0560	-12.7498
17	+2.7033	3.1113	3.5352	3.7723	4.4217	+4.7539	+5.3436	+5.8135	-6.3749
18	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	-2.7565	-3.1698	-3.5990	-4.0414	-4.4959	-4.9735	-5.4288	-5.9032	-6.3749
16	-5.5129	-6.3395	-7.1980	-8.0830	-8.9919	-9.9471	-10.8576	-11.8064	-12.7498
15	-8.2694	-9.5092	-10.7969	-12.1244	-13.4879	-14.9206	-16.2864	-17.7097	-19.1246
14	-11.0258	-12.6790	-14.3959	-16.1659	-17.9838	-19.8942	-21.7152	-23.6128	-25.2995
13	-13.7823	-15.8488	-17.9949	-20.2074	-22.4798	-24.8677	-27.1440	-29.5161	-31.8744
12	-16.5388	-19.0185	-21.5939	-24.2489	-26.9758	-29.8412	-32.5728	-35.4193	-38.2493
11	-19.2952	-22.1883	-25.1929	-28.2904	-31.4717	-34.8148	-38.0016	-41.3225	-44.6242
10	-22.0517	-25.3580	-28.7918	-32.3318	-35.9677	-39.7883	-43.4304	-47.2258	-50.9990
9	-24.8081	-28.5278	-32.3907	-36.3733	-40.4636	-44.7619	-48.8592	-53.1290	-57.3739
8	-27.5646	-31.6976	-35.9898	-40.4148	-44.9596	-49.7354	-54.2880	-59.0322	-63.7488
7	-30.3211	-34.8673	-39.5888	-44.4563	-49.4556	-54.7089	-59.7168	-64.9354	-70.1237
6	-33.0775	-38.0371	-43.1878	-48.4978	-53.9515	-59.6825	-65.1456	-70.8386	-76.4986
5	-35.8340	-41.2068	-46.7867	-52.5392	-58.4475	-64.6560	-70.5744	-76.7419	-82.8734
4	-38.5904	-44.3766	-50.3857	-56.5807	-62.9434	-69.6296	-76.0032	-82.6451	-89.2483
3	-41.3469	-47.5464	-53.9847	-60.6222	-67.4394	-74.6031	-81.4320	-88.5483	-95.6232
2	-44.1034	-50.7161	-57.5837	-64.6637	-71.9354	-79.5766	-86.8608	-94.4515	-101.9981
1	-46.8598	-53.8859	-61.1827	-68.7052	-76.4313	-84.5502	-92.2896	-100.3547	-108.3733
0	-49.6162	-57.0557	-64.7815	-72.7466	-80.9274	-89.5238	-97.7182	-106.2584	-114.7500
									(114.8385)

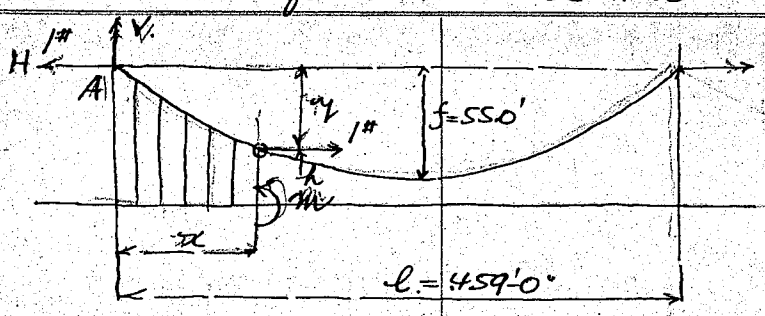
増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY J. Masuda DATE 14-10-14 FILE NO
CHECKED BY DATE PAGE NO 24

CALCULATIONS FOR

Nakuchi-Bashi for Tokushima-Ken



The value of m, the bending moment at any point in the stiffening truss for a 1-lb. load at A, acting horizontally is found by taking moments about the neutral axis of the truss

$$m = -(h+y) + h = -y.$$

which is equal to ordinate of parabolic cable or $m = -\frac{4fx}{l^2}(l-x) = -y$

Values of H×m

m →		5.5500	49.060	43.450	38.214	33.264	28.688	24.442	20.548	16.984
	H	0	1	2	3	4	5	6	7	8
0	0.0000	0	0	0	0	0	0	0	0	0
1	0.0409	2.2495	2.0070	1.7771	1.5630	1.3605	1.1733	0.9997	0.8404	0.6946
2	0.0933	5.1315	4.5773	4.0539	3.5654	3.1035	2.6766	2.2804	1.9171	1.5846
3	0.1557	8.5635	7.6386	6.7652	5.9499	5.1792	4.4667	3.8056	3.1993	2.6444
4	0.2271	12.4905	11.1415	9.8674	8.6784	7.5543	6.5150	5.5508	4.6665	3.8570
5	0.3062	16.2410	15.0221	13.3044	11.7011	10.1854	8.7843	7.4841	6.2918	5.2005
6	0.3915	21.5325	19.2070	17.0107	14.9608	13.0229	11.2314	9.5690	8.0445	6.6492
7	0.4809	26.4495	23.5930	20.8951	18.3771	15.9967	13.7961	11.7542	9.8815	8.1676
8	0.5756	31.6580	28.2389	25.0098	21.9960	19.1468	16.5128	14.0688	11.8274	9.7760
9	0.6725	36.9875	32.9928	29.2201	25.6989	22.3700	19.2927	16.4372	13.8185	11.4217
10	0.7711	42.4105	37.8302	33.5043	29.4668	25.6499	22.1213	18.8472	15.8446	13.0964
11	0.8702	47.8610	42.6920	37.8102	33.2538	28.9463	24.9643	21.2694	17.8809	14.7795
12	0.9684	53.2620	47.5097	42.0770	37.0064	32.2129	27.7815	23.6599	19.8987	16.4473
13	1.0584	58.2120	51.9251	45.9875	40.4457	35.2066	30.3634	25.8694	21.7480	17.9759
14	1.1605	63.8275	56.9341	50.4237	44.3473	38.6029	33.2924	28.3649	23.8460	19.7099
15	1.2517	68.8435	61.4084	54.3864	47.8325	41.6365	35.9088	30.5941	25.7199	21.2589
16	1.3390	73.6450	65.6913	58.1796	51.1685	44.5405	38.4432	32.7278	27.5138	22.7416
17	1.4216	78.1880	69.7437	61.7685	54.3250	47.2881	40.7829	34.7467	29.3110	24.1445
18	1.5116	83.1380	74.1591	65.6790	57.7642	50.2819	43.3648	36.9465	31.0604	25.6730

m →		13.750	10.868	8.116	4.246	2.796	8.316	1.540	0.672	0.176
	H	9	10	12	13	14	14	15	16	17
0	0	0	0	0	0	0	0	0	0	0
1	0.0409	0.5624	0.4445	0.2501	0.1737	0.1107	0.3041	0.0630	0.0275	0.0071
2	0.0933	1.2829	1.0140	0.5706	0.3962	0.2525	0.7759	0.1437	0.0627	0.0164
3	0.1557	2.1409	1.6921	0.9523	0.6611	0.4213	1.2948	0.2398	0.1046	0.0274
4	0.2271	3.1226	2.4681	1.3889	0.9643	0.6145	1.8886	0.3497	0.1526	0.0400
5	0.3062	4.2103	3.3278	1.8727	1.3001	0.8286	2.5464	0.4715	0.2058	0.0539
6	0.3915	5.3831	4.2548	2.3944	1.6623	1.0594	3.2557	0.6029	0.2631	0.0689
7	0.4809	6.6124	5.2264	2.9412	2.0419	1.3013	3.9992	0.7406	0.3232	0.0846
8	0.5756	7.9145	6.2556	3.5204	2.4440	1.5576	4.7867	0.8864	0.3868	0.1013
9	0.6725	9.2469	7.3087	4.1130	2.8554	1.8198	5.5925	1.0357	0.4519	0.1184
10	0.7711	10.6026	8.3803	4.7160	3.2740	2.0866	6.4125	1.1875	0.5182	0.1357
11	0.8702	11.9653	9.4573	5.3221	3.6949	2.3548	7.2366	1.3401	0.5848	0.1532
12	0.9684	13.3155	10.5246	5.9227	4.1118	2.6205	8.0532	1.4913	0.6508	0.1704
13	1.0584	14.5530	11.5027	6.4732	4.4940	2.8640	8.8017	1.6299	0.7112	0.1863
14	1.1605	15.9569	12.6123	7.0976	4.9275	3.1403	9.6507	1.7633	0.7799	0.2042
15	1.2517	17.2109	13.6035	7.6554	5.3147	3.3871	10.4091	1.9276	0.8411	0.2203
16	1.3390	18.4113	14.5523	8.1893	5.6854	3.6233	11.1351	2.0621	0.8998	0.2357
17	1.4216	19.5470	15.4499	8.6945	6.0361	3.8468	11.8220	2.1893	0.9553	0.2502
18	1.5116	20.7845	16.4281	9.2449	6.4183	4.0904	12.5705	2.3279	1.0158	0.2660

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushimaken

Moment in Centre span due to load on centre span
M-Hm.
M-Hm.

Load at	moment at 0	1	2	3	4	5	6	7	8
1	-8.1839	+4.1944	+3.8355	+3.4924	+3.1610	+2.7783	+2.5424	+2.2539	+1.9692
2	-15.0445	-3.1455	+8.8848	+7.9991	+7.2418	+6.5185	+5.8160	+5.1664	+4.5376
3	-20.6974	-4.3632	+3.0032	+13.4377	+12.1669	+10.6541	+9.7928	+8.6863	+7.6312
4	-25.2430	-14.5810	-3.8432	+6.9796	+17.8675	+16.0900	+14.3878	+12.7654	+11.2178
5	-28.7932	-18.8711	-8.8466	+1.2924	+11.5189	+21.8600	+19.5421	+17.3538	+15.2057
6	-31.4715	-22.3553	-13.1087	-3.7154	+5.7897	+15.4413	+25.2220	+22.3905	+19.6883
7	-33.4313	-25.1731	-16.7544	-8.1560	-0.4199	+9.4960	+18.5707	+27.8145	+24.4671
8	-34.6500	-27.3061	-19.7703	-12.0191	-4.1032	+4.0278	+12.3487	+15.8723	+29.5859
9	-35.3391	-28.9452	-22.3281	-15.4569	-7.9957	-1.0824	+6.4530	+14.2251	+22.2189
10	-36.5716	-30.1580	-24.4876	-18.4837	-12.3492	-5.8816	+0.8407	+7.8345	+15.0826
11	-35.4414	-31.0297	-26.3282	-21.3013	-16.0255	-10.4241	-4.5357	+1.6590	+8.1410
12	-35.0745	-31.6753	-27.9536	-23.3698	-19.5090	-14.7859	-9.7532	-4.3600	+1.3431
13	-34.9191	-32.4971	-29.7725	-26.5517	-23.0792	-19.2102	-15.0763	-10.4013	-5.4612
14	-33.8827	-32.5215	-30.7738	-28.5923	-26.0787	-23.1313	-19.8009	-16.0618	-11.9400
15	-33.2952	-32.9460	-32.1800	-31.0459	-29.3539	-27.2936	-24.8202	-21.9064	-18.5784
16	-32.7941	-33.4259	-33.6120	-33.2972	-32.5997	-31.4012	-29.7610	-27.6494	-25.0958
17	-32.4707	-34.0668	-35.1904	-35.7824	-35.9679	-35.6215	-34.8063	-33.5085	-31.7053
18	-31.6120	-34.2139	-36.3191	-37.8590	-38.9664	-39.5086	-39.5521	-39.0633	-38.0758
17	-28.0704	-30.6910	-32.6830	-34.2233	-35.3570	-35.9590	-36.0919	-35.7244	-34.8877
16	-24.0732	-26.5983	-28.6812	-30.2635	-31.4627	-32.1612	-32.4178	-32.2030	-31.5464
15	-20.6803	-23.1418	-25.1896	-26.7706	-27.9931	-28.7472	-29.0884	-28.9890	-28.4765
14	-17.1000	-19.4972	-21.5117	-23.0921	-24.3405	-25.1551	-25.5866	-25.6096	-25.2497
13	-14.5346	-16.3807	-18.6762	-20.1765	-21.3741	-22.1758	-22.6284	-22.7083	-22.4389
12	-11.5195	-13.6730	-15.5067	-16.9783	-18.1728	-19.0052	-19.5279	-19.6901	-19.5425
11	-9.1947	-11.1939	-12.9059	-14.2926	-15.4303	-16.2425	-16.7677	-16.9864	-16.9181
10	-7.2057	-9.0296	-10.5991	-11.8801	-12.9405	-13.7127	-14.2303	-14.4765	-14.4682
9	-5.5244	-8.1575	-8.5685	-9.7778	-10.6949	-11.4104	-11.9042	-12.1611	-12.1961
8	-4.1126	-5.5445	-6.7863	-7.8128	-8.6748	-9.3215	-9.7783	-10.0324	-10.0966
7	-2.9885	-4.2096	-5.2720	-6.1546	-6.8995	-7.4647	-7.8712	-8.1084	-8.1869
6	-2.0225	-3.6394	-3.9271	-4.6684	-5.2944	-5.7806	-6.1343	-6.3502	-6.4369
5	-1.3239	-2.1337	-2.8422	-3.4363	-3.9429	-4.3348	-4.6258	-4.8090	-4.8911
4	-0.8153	-1.4251	-1.9600	-2.4098	-2.7947	-3.0948	-3.3197	-3.4648	-3.5351
3	-0.4551	-0.8796	-1.2513	-1.9656	-1.8353	-2.0467	-2.2068	-2.3121	-2.3659
2	-0.2122	-0.6924	-0.6961	-0.8877	-1.0527	-1.1827	-1.2821	-1.3485	-1.3841
1	-0.0769	0.0	-0.2908	-0.3757	-0.4489	-0.5069	-0.5512	-0.5813	-0.5978
		+4.1944	+15.7235	+33.2012	+57.7458	+86.8660	+115.5162	+136.0217	+161.5884
	-688.8250	-604.5620	-558.5662	-510.8964	-475.0574	-446.6422	-422.1183	-398.5058	-368.2300

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Moments in center span due to load on center span

Load at	Moment at 9	10	11	12	13	14	15	16	17
1	+ 1.7180	+ 1.4713	+ 1.2319	+ 1.0188	+ 0.8135	+ 0.6213	+ 0.4509	+ 0.2804	+ 0.1310
2	+ 3.9396	+ 3.3743	+ 2.8400	+ 2.3384	+ 1.8677	+ 1.4276	+ 1.0226	+ 0.6452	+ 0.3024
3	+ 6.6276	+ 5.6782	+ 5.7810	+ 3.9385	+ 3.1469	+ 2.4070	+ 1.7253	+ 1.0899	+ 0.5125
4	+ 9.7453	+ 8.3527	+ 7.0351	+ 5.7974	+ 4.6346	+ 3.5477	+ 2.5438	+ 1.6086	+ 0.7570
5	+ 13.2556	+ 11.3647	+ 9.5156	+ 7.8941	+ 6.3138	+ 4.8345	+ 3.4694	+ 2.1962	+ 1.0365
6	+ 17.1152	+ 14.6801	+ 12.3740	+ 10.2057	+ 8.1668	+ 6.2569	+ 4.4935	+ 2.8467	+ 1.3456
7	+ 21.2784	+ 18.2589	+ 15.3982	+ 12.7068	+ 10.1731	+ 7.7999	+ 5.6057	+ 3.5548	+ 1.6828
8	+ 25.7394	+ 22.0655	+ 18.6415	+ 15.3902	+ 12.3287	+ 9.4574	+ 6.8012	+ 4.3166	+ 2.0460
9	+ 30.4349	+ 26.1374	+ 22.0619	+ 18.2232	+ 14.6063	+ 11.2115	+ 8.0680	+ 5.1251	+ 2.4324
10	+ 22.5857	+ 31.3592	+ 25.6377	+ 21.1875	+ 16.9919	+ 13.0409	+ 9.3980	+ 5.9751	+ 2.8390
11	+ 14.9101	+ 21.9854	+ 29.3480	+ 24.2668	+ 19.4730	+ 14.9442	+ 10.7848	+ 6.8128	+ 3.2645
12	+ 7.3657	+ 13.7290	+ 20.4120	+ 27.4360	+ 22.0295	+ 16.9375	+ 12.2177	+ 7.7816	+ 3.7256
13	- 0.1720	+ 5.4940	+ 11.5011	+ 17.8847	+ 24.6177	+ 18.9534	+ 13.6779	+ 8.7214	+ 3.9586
14	- 7.8350	- 2.5208	+ 2.4746	+ 8.4795	+ 14.5674	+ 21.0382	+ 15.1690	+ 9.6936	+ 4.6259
15	- 14.8393	- 10.6589	- 6.0651	- 1.0308	+ 4.4165	+ 10.2769	+ 16.6054	+ 10.5570	+ 4.9742
16	- 22.1005	- 18.6337	- 14.7253	- 10.3455	- 5.5236	- 0.2601	+ 5.5045	+ 11.6678	+ 5.5793
17	- 29.4515	- 26.6971	- 23.4735	- 19.7494	- 15.5563	- 10.8942	- 5.7002	- 0.1007	+ 6.0637
18	- 36.5894	- 34.5709	- 32.0537	- 29.0044	- 25.4561	- 21.2091	- 16.7967	- 11.7340	- 6.1089
17	- 33.5820	- 31.7559	- 29.5005	- 26.7248	- 23.4800	- 19.7660	- 15.5204	- 10.3511	- 5.6530
16	- 30.4479	- 28.8781	- 26.8665	- 24.3835	- 21.4586	- 18.0919	- 14.2243	- 9.9578	- 5.1931
15	- 27.5510	- 26.1848	- 24.4057	- 22.1858	- 19.5530	- 16.5071	- 12.9930	- 9.1060	- 4.7532
14	- 24.5067	- 23.3554	- 21.8210	- 19.8780	- 17.5523	- 14.8435	- 11.7246	- 8.2120	- 4.2917
13	- 21.8203	- 20.8291	- 19.4887	- 17.7757	- 15.7134	- 13.3019	- 10.4945	- 7.3718	- 3.8551
12	- 19.0752	↓ 18.2672	- 17.1397	- 15.6712	- 13.8831	- 11.7754	- 9.3056	- 6.5472	- 3.4286
11	- 16.5625	- 15.9007	- 14.9517	- 13.6964	- 12.1539	- 10.3242	- 8.1691	- 5.7547	- 3.0166
10	- 14.2055	- 13.6714	- 12.8827	- 11.8228	- 10.5083	- 8.9382	- 7.0819	- 4.9947	- 2.6208
9	- 12.0091	- 11.5855	- 10.9400	- 10.0577	- 8.9535	- 7.6273	- 6.0496	- 4.2717	- 2.2434
8	- 9.9708	- 9.6424	- 9.1241	- 8.4031	- 7.4923	- 6.3914	- 5.0754	- 3.5877	- 1.8860
7	- 8.1066	- 7.8572	- 7.4189	- 6.8715	- 6.1353	- 5.2405	- 4.1657	- 2.9477	- 1.5508
6	- 6.3944	- 6.2141	- 5.9046	- 5.4573	- 4.8808	- 4.1750	- 3.3229	- 2.3541	- 1.2397
5	- 4.8722	- 4.7455	- 4.5177	- 4.1823	- 3.7457	- 3.2060	- 2.5560	- 1.8125	- 0.9553
4	- 3.5303	- 3.4456	- 3.2859	- 3.0464	- 2.7318	- 2.3423	- 1.8679	- 1.3258	- 0.6992
3	- 2.3684	- 2.3162	- 2.2124	- 2.0539	- 1.8441	- 1.3828	- 1.2633	- 0.8974	- 0.4736
2	- 1.3889	- 1.3210	- 1.3022	- 1.2106	- 1.0882	- 0.9350	- 0.7469	- 0.5310	- 0.2805
1	- 0.6008	- 0.5895	- 0.5646	- 0.5254	- 0.4725	- 0.4063	- 0.3247	- 0.2310	- 0.1221
	+ 17.4.7143	+ 183.9507	+ 184.2586	+ 176.7676	+ 164.1474	+ 142.7549	+ 117.5368	+ 82.8728	+ 43.2770
	- 347.9803	- 319.6410	- 288.6450	- 254.0765	- 218.1828	- 177.6247	- 137.3817	- 92.5889	- 50.5890

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushimaken

Moments in Center span load on left side span

Load on	moment at tower	1	2	3	4	5	6	7	8
1	-0.9654	-0.9118	-0.8581	-0.8045	-0.7509	-0.6972	-0.6436	-0.5900	-0.5363
2	-1.8389	-1.7367	-1.6345	-1.5324	-1.4303	-1.3281	-1.2259	-1.1238	-1.0216
3	-2.5286	-2.3881	-2.2476	-2.1072	-1.9667	-1.8262	-1.6857	-1.5452	-1.4048
4	-2.9423	-2.7788	-2.6154	-2.4719	-2.2885	-2.1250	-1.9615	-1.7981	-1.6346
5	-2.9884	-2.8224	-2.6564	-2.4903	-2.3243	-2.1583	-1.9923	-1.8262	-1.6602
6	-2.5745	-2.4315	-2.2884	-2.1454	-2.0024	-1.8594	-1.7163	-1.5733	-1.4303
7	-1.6092	-1.5198	-1.4304	-1.3410	-1.2516	-1.1622	-1.0728	-0.9834	-0.8940
	-15.4473	-14.5891	13.7308	-12.8727	-12.0147	-11.1564	10.2981	-9.4400	-8.5818

Load on	9	10	11	12	13	14	15	16	17
1	-0.4827	-0.4291	-0.3754	-0.3218	-0.2681	-0.2145	-0.1609	-0.1073	-0.0536
2	-0.9194	-0.8173	-0.7151	-0.6130	-0.5108	-0.4086	-0.3065	-0.2043	-0.1022
3	-1.2642	-1.1238	-0.9833	-0.8429	-0.7024	-0.5619	-0.4214	-0.2809	-0.1405
4	-1.4711	-1.3077	-1.1442	-0.9808	-0.8173	-0.6538	-0.4904	-0.3269	-0.1635
5	-1.4942	-1.3282	-1.1621	-0.9961	-0.8301	-0.6641	-0.4981	-0.3320	-0.1660
6	-1.2873	-1.1442	-1.0012	-0.6582	-0.7151	-0.5721	-0.4291	-0.2861	-0.1430
7	-0.8046	-0.7152	-0.6258	-0.5364	-0.4470	-0.3570	-0.2682	-0.1788	-0.0894
	-7.7235	-6.8655	-6.0071	-5.1492	-4.2908	-3.4320	-2.5746	-1.7163	-0.8582

Moments in Center span due to load on right side span reverse to the above and should be positive on left half of center span

Moments in Left side span due to load on left span

Left span Load on	moment at 1	2	3	4	5	6	7	tower
1	11.0357	9.3212	7.6068	5.8923	4.1779	2.4635	0.7491	-0.9654
2	9.3327	18.6653	15.2479	11.8305	8.4132	4.9958	1.5785	-1.8389
3	7.6528	15.9575	23.9060	19.1250	14.3437	9.5625	2.7813	-2.5286
4	6.0072	12.0144	18.0217	24.0289	17.2860	10.5433	3.8005	-2.9423
5	4.4077	8.8154	13.2231	17.6308	22.0385	13.6762	5.3539	-2.9884
6	2.8657	5.7314	8.5971	11.4627	14.3284	17.1941	7.3098	-2.5745
7	1.3930	2.7855	4.1780	5.5706	6.9631	9.3556	9.7482	-1.6092
8	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.000	-0.0000
	+42.6948	+73.2707	+90.7806	+95.5408	+87.5508	+67.7910	31.3213	-15.4473

Moments in left side span due to load on right span

Right span Load on	moment at 1	2	3	4	5	6	7	tower
8	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
7	0.1207	0.2413	0.3620	0.4827	0.6034	0.7240	0.8447	0.9654
6	0.2298	0.4597	0.6896	0.9195	1.1493	1.3792	1.6090	1.8389
5	0.3160	0.6322	0.9482	1.2643	1.5804	1.8964	2.2125	2.5286
4	0.3678	0.7356	1.1033	1.4711	1.8390	2.2067	2.5745	2.9423
3	0.3736	0.7471	1.1207	1.4942	1.3678	2.2413	2.6149	2.9884
2	0.3218	0.6436	0.9654	1.2873	1.6091	1.9309	2.2527	2.5745
1	0.2011	0.4023	0.6035	0.8046	1.0058	1.2069	1.4081	1.6092
	+1.9308	+3.8618	+5.7927	+7.7237	+9.6548	+11.5894	+13.5794	+15.4473

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushimaken

Moments in Left side span due to load on center span
Full panel load max neg. moment at tower = −688.825 max panel moment m = 36.5716 due to load on panel point 10
moments at various points are

9	1	2	3	4	5	6	7	8
Full load	−84.104	−172.208	−258.312	−344.416	−430.520	−516.624	−602.728	−688.825
Load on 10	−4.5714	−9.1428	−13.7142	−18.2856	−22.8570	−27.4284	−31.9998	−36.5716

Slope of cable at panel points

$\tan\theta = \frac{4f}{l}(1-2n)$ where f = rise l = span length n = 1/ no of panels.
= 55'-0" = 459'-0" = 1/36

tower			tanθ		
1	1.		1.0000	× 0.4793 =	0.4793
1	1 − 0.05556	=	0.9444	"	0.4526
2	0.1111	=	0.8889	"	0.4260
3	0.1667	=	0.8333	"	0.3994
4	0.2222	=	0.7778	"	0.3728
5	0.2778	=	0.7222	"	0.3462
6	0.3333	=	0.6667	"	0.3195
7	0.3889	=	0.6111	"	0.2929
8	0.4444	=	0.5555	"	0.2663
9	0.5000	=	0.5000	"	0.2397
10	0.5555	=	0.4444	"	0.2130
11	0.6111	=	0.3889	"	0.1864
12	0.6667	=	0.3333	"	0.1596
13	0.7222	=	0.2778	"	0.1331
14	0.7778	=	0.2222	"	0.1065
15	0.8333	=	0.1667	"	0.0799
16	0.8889	=	0.1111	"	0.0533
17	0.9444	=	0.0556	"	0.0266
18	1.0000	=	0.0000	"	0.0000

ΔtanθH for center span

tanθ →	1	2	3	4	5	6	7	8	9
	.4526	.4240	.3994	.3728	.3462	.3195	.2929	.2663	.2397
1 ↓H	0.0409	0.0185	0.0174	0.0163	0.0152	0.0142	0.0131	0.0120	0.0109	0.0098
2	0.0933	0.0422	0.0397	0.0373	0.0348	0.0323	0.0298	0.0273	0.0249	0.0224
3	0.1557	0.0705	0.0663	0.0622	0.0580	0.0538	0.0497	0.0456	0.0415	0.0373
4	0.2271	0.1028	0.0968	0.0907	0.0846	0.0786	0.0725	0.0665	0.0604	0.0544
5	0.3062	0.1400	0.1305	0.1223	0.1138	0.1060	0.0978	0.0899	0.0816	0.0734
6	0.3915	0.1772	0.1667	0.1564	0.1460	0.1356	0.1252	0.1146	0.1043	0.0938
7	0.4809	0.2175	0.2048	0.1920	0.1794	0.1700	0.1537	0.1408	0.1281	0.1152
8	0.5756	0.2602	0.2450	0.2296	0.2145	0.1993	0.1840	0.1685	0.1532	0.1379
9	0.6725	0.3042	0.2862	0.2685	0.2510	0.2330	0.2150	0.1968	0.1792	0.1611
10	0.7711	0.3490	0.3290	0.3078	0.2875	0.2670	0.2464	0.2258	0.2052	0.1848
11	0.8702	0.3938	0.3708	0.3474	0.3244	0.3013	0.2780	0.2548	0.2318	0.2085
12	0.9684	0.4382	0.4125	0.3870	0.3610	0.3355	0.3095	0.2836	0.2579	0.2320
13	1.0584	0.4790	0.4510	0.4230	0.3945	0.3663	0.3380	0.3098	0.2820	0.2538
14	1.1605	0.5255	0.4945	0.4650	0.4330	0.4019	0.3720	0.3396	0.3090	0.2780
15	1.2517	0.5665	0.5330	0.4993	0.4666	0.4430	0.4000	0.3665	0.3335	0.3000
16	1.3390	0.6060	0.5705	0.5345	0.4990	0.4635	0.4274	0.3920	0.3567	0.3208
17	1.4216	0.6430	0.6055	0.5675	0.5300	0.4920	0.4540	0.4260	0.3786	0.3405
18	1.5116	0.6840	0.6440	0.6035	0.5630	0.5235	0.4826	0.4425	0.4025	0.3620
	25.0809	5.3341	5.0702	4.7068						
Σ tanθH		10.6682	10.0404	9.4136	9.33	8.66	8.00	7.33	6.66	6.000
		11.3527	10.67	10.680						
			10.6846	10.0771						

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushimaken

H tanθ continued

	tanθ →	.2130	.1824	.1596	.1331	.1065	.0799	.0533	.0266
	H	10	11	12	13	14	15	16	17
1	0.0409	0.0087	0.0076	0.0065	0.0054	0.0044	0.0033	0.0022	0.0011
2	0.0933	0.0199	0.0174	0.0149	0.0124	0.0099	0.0075	0.0050	0.0025
3	0.1557	0.0332	0.0290	0.0248	0.0207	0.0166	0.0124	0.0082	0.0041
4	0.2271	0.0484	0.0423	0.0362	0.0304	0.0242	0.0182	0.0120	0.0060
5	0.3062	0.0652	0.0571	0.0489	0.0407	0.0326	0.0245	0.0162	0.0081
6	0.3915	0.0834	0.0730	0.0625	0.0521	0.0417	0.0313	0.0208	0.0104
7	0.4809	0.1025	0.0896	0.0768	0.0640	0.0512	0.0384	0.0256	0.0128
8	0.5756	0.1225	0.1073	0.0919	0.0765	0.0613	0.0459	0.0306	0.0153
9	0.6725	0.1432	0.1254	0.1073	0.0895	0.0716	0.0537	0.0358	0.0179
10	0.7711	0.1642	0.1437	0.1231	0.1025	0.0822	0.0616	0.0410	0.0205
11	0.8702	0.1853	0.1622	0.1384	0.1158	0.0926	0.0696	0.0463	0.0232
12	0.9684	0.2062	0.1805	0.1546	0.1289	0.1032	0.0774	0.0516	0.0258
13	1.0584	0.2252	0.1973	0.1689	0.1409	0.1127	0.0845	0.0568	0.0284
14	1.1605	0.2470	0.2162	0.1852	0.1545	0.1236	0.0927	0.0618	0.0309
15	1.2517	0.2667	0.2332	0.1998	0.1665	0.1332	0.1000	0.0666	0.0333
16	1.3390	0.2850	0.2485	0.2151	0.1783	0.1426	0.1070	0.0714	0.0357
17	1.4216	0.3030	0.2650	0.2268	0.1891	0.1514	0.1135	0.0756	0.0378
18	1.5116	0.3220	0.2818	0.2412	0.2010	0.1610	0.1207	0.0804	0.0402
	Σ H tanθ	5.33	4.67	4.00	3.33	2.67	2.00	1.33	.67

Shear for center span

Load on left span

Load on	M_3	M_2	M_3-M_2	$\frac{M_3-M_2}{\ell}$
Left 1	-2.3264	-10.4934	+8.1070	+0.0177
2	-5.3437	-20.1760	+14.8323	+0.0323
3	-9.0186	-29.2609	+20.2423	+0.0441
4	-13.3058	-37.7335	+24.4277	+0.0532
5	-18.1649	-45.6342	+27.4693	+0.0598
6	-23.5550	-53.0040	+29.4490	+0.0642
7	-29.4380	-59.8808	+30.4428	+0.0659
8	-35.7706	-66.3080	+30.5374	+0.0666
9	-42.5119	-72.3266	+29.8147	+0.0650
10	-49.6162	-77.9821	+28.3659	+0.0618
11	-57.0557	-83.3024	+26.2467	+0.0572
12	-64.7815	-88.3365	+23.5550	+0.0513
13	-72.7467	-93.1311	+20.3844	+0.0444
14	-80.9274	-97.7102	+16.7828	+0.0366
15	-89.5238	-102.1387	+12.6149	+0.0275
16	-97.7182	-106.4391	+8.7209	+0.0190
17	-106.2584	-110.6587	+4.4003	+0.0096
18	-114.8385	-114.8385	0.0000	0.0000
17				-0.0096
16				-0.0190

Load on side span

Load on	M_2	M_3	M_3-M_2	$\frac{M_3-M_2}{\ell}$
1	-0.9654	+0.9654	+1.9208	+0.0042
2	-1.8389	+0.8389	+3.6778	+0.0080
3	-2.5286	+2.5286	+5.0572	+0.0110
4	-2.9423	+2.9423	+5.8846	+0.0128
5	-2.9884	+2.9884	+5.9768	+0.0130
6	-2.5745	+2.5745	+5.1490	+0.0112
7	-1.6092	+1.6092	+3.2184	+0.0070
8	0.0	0.0	0.0	0.0000
				+0.0672

Full load on Left side span + shear = 0.0672
Full load on right side span − shear = 0.0672
Full load on both side spans shear = 0.0000

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CALCULATIONS FOR

Naruchi-Ebashi for 20ft. Yoshima-Line

Shear in Center span Vo as simple beam

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	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1	0.0278																	
2	0.0556																	
3	0.0833																	
4	0.1111																	
5	0.1389																	
6	0.1667																	
7	0.1944																	
8	0.2222																	
9	0.2500																	
10	0.2778																	
11	0.3056																	
12	0.3333																	
13	0.3611																	
14	0.3889																	
15	0.4167																	
16	0.4444																	
17	0.4722																	
18	0.5000																	
17	0.5278																	-0.4722
16	0.5555																-0.4444	-0.4444
15	0.5833															-0.4167		-0.4167
14	0.6111														-0.3889			-0.3889
13	0.6389													-0.3611				-0.3611
12	0.6667												-0.3333					-0.3333
11	0.6944											-0.3056						-0.3056
10	0.7222										-0.2778							-0.2778
9	0.7500									-0.2500								-0.2500
8	0.7778								-0.2222									-0.2222
7	0.8056							-0.1944										-0.1944
6	0.8333						-0.1667											-0.1667
5	0.8611					-0.1389												-0.1389
4	0.8889				-0.1111													-0.1111
3	0.9167			-0.0833														-0.0833
2	0.9444		-0.0555															-0.0555
1	0.9722	-0.0278																-0.0278
0	1.0000																	-0.0000

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Shears for centre span $= V_0 + \frac{M_3 - M_2}{l} - H \tan\theta$

		1	2	3	4	5	6	7	8	9
1		-0.0084	-0.0073	-0.0062	-0.0051	-0.0041	-0.0020	-0.0019	-0.0008	+0.0005
2		-0.0189	-0.0164	-0.0140	-0.0115	-0.0090	-0.0065	-0.0040	-0.0016	+0.0009
3		-0.0313	-0.0271	-0.0230	-0.0188	-0.0146	-0.0105	-0.0064	-0.0023	+0.0019
4		-0.0449	-0.0389	-0.0328	-0.0267	-0.0207	-0.0146	-0.0086	-0.0025	+0.0035
5		-0.0609	-0.0514	-0.0432	-0.0347	-0.0269	-0.0187	-0.0106	-0.0025	+0.0057
6		-0.0747	-0.0642	-0.0539	-0.0435	-0.0331	-0.0227	-0.0121	-0.0018	+0.0087
7		-0.0890	-0.0763	-0.0635	-0.0509	-0.0390	-0.0252	-0.0123	+0.0004	+0.0133
8		-0.1046	-0.0894	-0.0740	-0.0589	-0.0437	-0.0284	-0.0129	+0.0024	+0.0177
9		-0.1192	-0.1012	-0.0835	-0.0660	-0.0480	-0.0300	-0.0118	0.0058	+0.0239
10	Left	-0.1330	-0.1130	-0.0918	-0.0715	-0.0510	-0.0304	-0.0093	0.0108	+0.0312
11		-0.1454	-0.1224	-0.0990	-0.0760	-0.0529	-0.0296	-0.0064	0.0166	+0.0399
12		-0.1562	-0.1305	-0.1050	-0.0790	-0.0535	-0.0275	-0.0016	0.0241	+0.0500
13		-0.1623	-0.1343	-0.1063	-0.0802	-0.0523	-0.0240	+0.0069	0.0347	+0.0629
14		-0.1732	-0.1389	-0.1127	-0.0807	-0.0496	-0.0193	0.0127	0.0433	+0.0743
15		-0.1773	-0.1438	-0.1101	-0.0794	-0.0440	-0.0108	0.0227	0.0557	0.0892
16		-0.1806	-0.1451	-0.1091	-0.0736	-0.0381	-0.0020	0.0334	0.0687	0.1046
17		-0.1823	-0.1428	-0.1048	-0.0678	-0.0293	+0.0087	0.0367	0.0841	0.1222
18		-0.1840	-0.1444	-0.1035	-0.0630	-0.0135	0.0174	0.0575	0.0975	0.1380
17		-0.1056	-0.0681	-0.0301	+0.0089	+0.0454	0.0834	0.1114	0.1588	0.1969
16		-0.0486	-0.0131	+0.0229	+0.0584	+0.0939	0.1300	0.1654	0.2007	0.2366
15		+0.0443	+0.0778	+0.1115	+0.1448	0.1678	0.2108	0.2443	0.2773	0.3108
14		+0.1222	0.1532	+0.1827	+0.2147	0.2458	0.2757	0.3081	0.3387	0.3697
13		+0.2043	0.2323	+0.2603	+0.2888	0.3170	0.3453	0.3735	0.4013	0.4295
12		+0.2798	0.3055	+0.3310	+0.3570	0.3825	0.4085	0.4344	0.4601	0.4860
11		+0.3578	0.3808	+0.4042	+0.4272	0.4503	0.4736	0.5258	0.5198	0.5431
10	Right	+0.4350	0.4550	+0.4762	+0.4965	0.5170	0.5376	0.5582	0.5788	0.5992
9		+0.5108	0.5288	+0.5465	+0.5640	0.5820	0.6000	0.6182	+0.6358	0.6539
8		+0.5842	0.5994	+0.6148	+0.6299	0.6451	0.6604	0.6759	+0.6912	-0.2935
7		+0.6570	0.6667	+0.6795	+0.6921	0.6965	+0.7178	+0.7307	-0.2566	-0.2437
6		+0.7203	0.7308	+0.7411	+0.7515	0.7619	+0.7723	-0.2171	-0.2068	-0.1963
5		+0.7809	0.7904	+0.7986	+0.8071	+0.8149	-0.1769	-0.1688	-0.1607	-0.1525
4		+0.8393	0.8453	+0.8514	+0.8575	-0.1365	-0.1304	-0.1244	-0.1183	-0.1123
3		+0.8903	0.8945	+0.8986	-0.0972	-0.0930	-0.0889	-0.0848	-0.0807	-0.0765
2		+0.9345	0.9370	-0.0606	-0.0581	-0.0556	-0.0531	-0.0506	-0.0482	-0.0457
1		+0.9714	-0.0275	-0.0264	-0.0253	-0.0243	-0.0232	-0.0221	-0.0210	-0.0197
		-2.2004	-1.7686	-1.3665	-.9848	-0.6213	-0.3022	-0.0979	-0.0115	
		+8.3321	+7.5975	+6.9193	+6.2984	+5.7201	+5.2154	+4.9258	+4.7066	+4.6141
			-0.0275	-0.0870	-.1806	-0.3094	-0.4725	-0.6678	-0.8923	-1.1402
			-1.7961	-1.4535	-1.1654	-0.9307	-0.7747	-0.7657	-0.8938	-1.1402

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CALCULATIONS FOR

Hatuchi-Bashi for Tokushimaken

Shears for Center span $V_0 + \frac{M_3 - M_2}{\ell} - H\tan\theta$

	10	11	12	13	14	15	16	17	18
1	+0.0014	+0.0025	0.0036	0.0047	0.0057	0.0068	0.0079	0.0090	0.0101
2	0.0033	0.0059	0.0084	0.0109	0.0134	0.0158	0.0183	0.0205	0.0233
3	0.0060	0.0102	0.0148	0.0185	0.0226	0.0268	0.0310	0.0351	0.0392
4	0.0095	0.0156	0.0217	0.0275	0.0337	0.0397	0.0454	0.0519	0.0579
5	0.0139	0.0220	0.0302	0.0384	0.0465	0.0546	0.0629	0.0710	0.0791
6	0.0191	0.0295	0.0400	0.0504	0.0608	0.0712	0.0818	0.0921	0.1025
7	0.0260	0.0389	0.0517	0.0645	0.0773	0.0901	0.1029	0.1157	0.1285
8	0.0331	0.0483	0.0637	0.0791	0.0943	0.1096	0.1250	0.1403	0.1556
9	0.0418	0.0596	0.0777	0.0955	0.1134	0.1313	0.1492	0.1679	0.1850
10	0.0518	0.0723	0.0929	0.1135	0.1338	0.1544	0.1750	0.1955	0.2160
11	0.0631	0.0840	0.1095	0.1327	0.1558	0.1788	0.2021	0.2252	0.2484
12	0.0740	0.1015	0.1274	0.1531	0.1788	0.2046	0.2304	0.2562	0.2820
13	0.0915	0.1194	0.1499	0.1758	0.2040	0.2322	0.2599	0.2883	0.3167
14	0.1053	0.1361	0.1671	0.1978	0.2287	0.2596	0.2905	0.3214	0.3523
15	0.1225	0.1560	0.1894	0.2227	0.2560	0.2892	0.3226	0.3559	0.3892
16	0.1404	0.1769	0.2117	0.2471	0.2828	0.3184	0.3540	0.3897	0.4254
17	0.1597	0.1977	0.2359	0.2736	0.3113	0.3492	0.3896	0.4249	0.4627
18	0.1780	0.2182	0.2588	0.2990	0.3390	0.3793	0.4196	0.4598	0.5000
17	0.2344	0.2724	0.3106	0.3483	0.3860	0.4239	0.4738	+0.4994	-0.4627
16	0.2724	0.3089	0.3437	0.3791	0.4148	0.4504	+0.4800	-0.4611	-0.4254
15	0.3441	0.3776	0.4110	0.4443	0.4776	+0.5108	-0.4554	-0.4225	-0.3892
14	0.4007	0.4315	0.4625	+0.4932	+0.5241	-0.4450	-0.4141	-0.3832	-0.3523
13	0.4581	0.4860	0.5144	+0.5424	-0.4294	-0.4012	-0.3735	-0.3451	-0.3167
12	0.5118	0.5375	+0.5634	-0.4109	-0.3852	-0.3594	-0.3334	-0.3078	-0.2820
11	0.5663	+0.5894	-0.3873	-0.3681	-0.3410	-0.3180	-0.2947	-0.2716	-0.2484
10	+0.6198	-0.3597	-0.3391	-0.3185	-0.2982	-0.2776	-0.2570	-0.2365	-0.2160
9	-0.3282	-0.3103	-0.2923	-0.2745	-0.2566	-0.2387	-0.2180	-0.2029	-0.1850
8	-0.2781	-0.2629	-0.2475	-0.2321	-0.2169	-0.2015	-0.1862	-0.1709	-0.1556
7	-0.2310	-0.2181	-0.2053	-0.1745	-0.1797	-0.1669	-0.1541	-0.1413	-0.1285
6	-0.1859	-0.1755	-0.1650	-0.1546	-0.1442	-0.1338	-0.1232	-0.1129	-0.1025
5	-0.1443	-0.1362	-0.1280	-0.1198	-0.1117	-0.1036	-0.0953	-0.0872	-0.0791
4	-0.1063	-0.1002	-0.0941	-0.0883	-0.0821	-0.0761	-0.0694	-0.0639	-0.0579
3	-0.0724	-0.0682	-0.0640	-0.0599	-0.0558	-0.0516	-0.0474	-0.0433	-0.0392
2	-0.0432	-0.0407	-0.0382	-0.0357	-0.0332	-0.0308	-0.0283	-0.0258	-0.0233
1	-0.0188	-0.0177	-0.0166	-0.0155	-0.0145	-0.0134	-0.0123	-0.0112	-0.0101
	+4.5480	+4.4979	+4.4600	+4.4351	+4.3609	+4.2967	+4.2219	+4.1149	+3.9739
	-1.4082	-1.6895	-1.9774	-2.2524	-2.6485	-2.8174	-3.0623	-3.2782	-3.4740

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Shear for side span
shear due to moments Shears as simple beam Vo.

Load at	M_2	$\frac{M_2}{\ell}$	1	2	3	4	5	6	7	8
8	0.0000	-0.0000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
7	1.6092	-0.0157	0.125	0.125	0.125	0.125	0.125	0.125	0.125	-0.875
6	2.5745	-0.0252	0.250	0.250	0.250	0.250	0.250	0.250	-0.750	-0.750
5	2.9884	-0.0293	0.375	0.375	0.375	0.375	0.375	-0.625	-0.625	-0.625
4	2.9423	-0.0288	0.500	0.500	0.500	0.500	-0.500	-0.500	-0.500	-0.500
3	2.5286	-0.0248	0.625	0.625	0.625	-0.375	-0.375	-0.375	-0.375	-0.375
2	1.8389	-0.0180	0.750	0.750	-0.250	-0.250	-0.250	-0.250	-0.250	-0.250
1	0.9654	-0.0093	0.875	-0.125	-0.125	-0.125	-0.125	-0.125	-0.125	-0.125
0	0.0000	-0.0000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Summary shear for side span load on side span $Vo + \frac{M_2}{\ell}$

	1	2	3	4	5	6	7	8
8	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
7	0.1093	0.1093	0.1093	0.1093	0.1093	0.1093	0.1093	-0.8907
6	0.2248	0.2248	0.2248	0.2248	0.2248	0.2248	-0.7752	-0.7752
5	0.3457	0.3457	0.3457	0.3457	0.3457	-0.6543	-0.6543	-0.6543
4	0.4712	0.4712	0.4712	0.4712	-0.5288	-0.5288	-0.5288	-0.5288
3	0.6002	0.6002	0.6002	-0.3998	-0.3998	-0.3998	-0.3998	-0.3998
2	0.7320	0.7320	-0.2680	-0.2680	-0.2680	-0.2680	-0.2680	-0.2680
1	0.8657	-0.1343	-0.1343	-0.1343	-0.1343	-0.1343	-0.1343	-0.1343
0								
	+3.3489	+2.4832	+1.7512	+1.1510	+0.6798	+0.3341	+0.1093	+0.0000
	-0.0000	-1.1343	-0.4023	-0.8021	-1.3309	-1.9852	-2.7604	-3.6511

For load on opposite span the shears will be reverse

Shear for side span due to load on center span
For full loads moment at tower = -688.825
Shear for side span -688.825 ÷ 102 = -6.7532
For single load the shears are

Load at	Left Half. Moment at Tower		Load at	Right Half. Moment at Tower	
1	-8.1839	÷102 = -0.0802	17	-28.0704	÷102 = -0.2752
2	-15.0445	= -0.1475	16	-24.0732	= -0.2360
3	-20.6974	= -0.2029	15	-20.6803	= -0.2027
4	-25.2430	= -0.2475	14	-17.1000	= -0.1676
5	-28.7932	= -0.2823	13	-14.5346	= -0.1425
6	-31.4715	= -0.3085	12	-11.5195	= -0.1129
7	-33.4313	= -0.3278	11	-9.1947	= -0.0901
8	-34.6500	= -0.3397	10	-7.2057	= -0.0706
9	-35.3391	= -0.3460	9	-5.5244	= -0.0542
10	-36.5716	= -0.3580	8	-4.1126	= -0.0403
11	-35.4414	= -0.3480	7	-2.9885	= -0.0293
12	-35.0745	= -0.3439	6	-2.0225	= -0.0188
13	-34.9191	= -0.3423	5	-1.3239	= -0.0130
14	-33.8827	= -0.3322	4	-0.8153	= -0.0080
15	-33.2952	= -0.3264	3	-0.4551	= -0.0045
16	-32.7941	= -0.3215	2	-0.2122	= -0.0021
17	-32.4707	= -0.3183	1	-0.0769	= -0.0008
18	-31.6120	= -0.3099	Summary for total load		= -6.7532

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Summary moments for stiffening truss

Moments for center span

	For Full Load		Max Panel Moment		Load on Left span	Load on Right span	Max panel moment	
	Neg. moment	Pos. moment	neg. moment	pos. m	neg. m	pos. m	Load on left span neg m	Load on right span pos. m
Tower	- 688.8250	0.0000	-36.5716	0.0000	-15.4473	+15.4473	- 2.98874	+ 2.98874
1	- 604.5620	4.1944	-34.2139	4.1944	-14.5891	+14.5891	- 2.8224	+ 2.8224
2	- 558.5662	15.7235	- 36.3191	8.8848	- 13.7308	↓	- 2.6564	↓
3	- 510.8964	33.2012	- 37.8590	13.4377	- 12.8727		- 2.4903	
4	- 475.0574	57.7458	- 38.9664	17.8675	- 12.0147		- 2.3243	
5	- 446.6422	86.8660	- 39.5086	21.8600	- 11.1564		- 2.1583	
6	- 422.1183	115.5162	- 39.5521	25.2220	- 10.2981		- 1.9923	
7	- 398.5058	136.0217	- 39.0633	27.8145	- 9.4400		- 1.8262	
8	- 368.2300	161.5884	- 38.0758	29.5859	- 8.5818		- 1.6602	
9	- 347.9803	174.7143	- 36.5894	30.4349	- 7.7235		- 1.4943	
10	- 319.6410	183.9507	- 34.5709	31.3592	- 6.8655		- 1.3282	
11	- 288.6450	184.2586	- 32.0537	29.3480	- 6.0071		- 1.1621	
12	- 254.0765	176.7676	- 29.0044	27.4360	- 5.1492		- 0.9961	
13	- 218.1828	164.1474	- 25.4561	24.6177	- 4.2908		- 0.8301	
14	- 177.6247	147.7549	- 21.2091	21.0382	- 3.4320		- 0.6641	
15	- 137.3817	117.5368	- 16.7967	16.6054	- 2.5746		- 0.4981	
16	- 92.5889	82.8728	- 11.7340	11.6678	- 1.7163		- 0.3320	
17	- 50.5890	43.2770	- 6.1089	6.0637	- 0.8582		- 0.1660	
18	- 0.0000	0.0000	0.0000	0.0000	0.0000		0.0000	

	Full Load on Center span	Max neg. Load on left and center spans	Max pos. Load on right and center spans
Tower	- 688.8250	- 704.2723	+ 15.4473
1	- 600.3676	619.1511	18.7147
2	- 542.8427	572.2970	29.4543
3	- 477.6952	523.7691	47.0839
4	- 417.3116	487.0721	69.7605
5	- 359.7762	457.7486	98.0224
6	- 306.6021	432.4164	125.8143
7	- 262.4841	407.9458	145.4617
8	- 206.6416	376.8118	170.1702
9	- 173.2660	355.7038	182.4378
10	- 135.6903	326.5065	190.8162
11	- 104.3864	294.6521	195.2657
12	- 77.3089	259.2257	181.9568
13	- 54.0354	222.4736	168.4582
14	- 34.8698	181.0567	146.1867
15	- 19.8449	139.9563	120.1114
16	- 9.8061	94.3052	84.5891
17	- 7.3120	51.4472	44.1352
18	0.0000	0.0000	0.0000

Full load on both side spans cancels pos. and neg. moments.

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CALCULATIONS FOR

Hakuchi - Bashi for Tokushima-Ken.

Moments for side span

	For Full Load on Side spans			Max Panel moment				Summary moments for full load -
	Load on left span	Load on right span	Both spans loaded	Load on left span	Load on right span	Load on centr span	max panel moment	
0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1	+ 42.6948	+ 1.9308	+ 44.6256	+11.0357	+ 0.3736	-84.104	- 4.5714	- 39.4784
2	+ 73.2707	+ 3.8618	+ 77.1325	+18.6653	+ 0.7471	-172.208	- 9.1428	- 95.0855
3	+ 90.7806	+ 5.7927	+ 96.5733	+23.9060	+ 1.1207	-258.312	- 13.7142	- 161.7397
4	+ 95.5408	+ 7.7237	+ 103.2645	+24.0289	+ 1.4942	-344.416	- 18.2856	- 241.1515
5	+ 87.5508	+ 9.6548	+ 97.2056	+22.0385	+ 1.8678	-430.520	- 22.8570	- 333.3144
6	+ 67.7910	+ 11.5854	+ 79.3764	+17.1941	+ 2.2413	-516.624	- 27.4284	- 437.2476
7	+ 33.3213	+ 13.5194	+ 46.8407	+ 9.7482	+ 2.6149	-602.728	- 31.9998	- 555.8873
8	- 15.4473	+ 15.4473	0.0000	- 2.9884	+ 2.9884	-688.825	- 36.5716	- 688.8250

15.4473 ÷ 102 = 0.1515

Shears for Center span

	max pos.	max neg.	Center span loaded Summary	max panel shear	Side span loaded +shear on left -shear on right loaded.	max + shear	max - shear	3 spans loaded same as center span load.
1	+8.3321	- 2.2004	+ 6.1317	+ 0.9714	± 0.0672	+ 8.4665	- 2.2672	+ 6.1317
2	7.5975	- 1.7961	+ 5.8014	+ 0.9370	"	+ 7.6647	- 1.8666	+ 5.8014
3	6.9193	- 1.4535	+ 5.4658	+ 0.8986	"	+ 6.9865	- 1.5270	+ 5.4658
4	6.2984	- 1.1654	+ 5.1330	+ 0.8575	"	+ 6.3656	- 1.2868	+ 5.1330
5	5.7201	- 0.9307	+ 4.7894	+ 0.8149	"	+ 5.7873	- 0.9979	+ 4.7894
6	5.2154	- 0.7747	+ 4.4407	+ 0.7723	"	+ 5.3826	- 0.9419	+ 4.4407
7	4.9258	- 0.7657	+ 4.1601	+ 0.7307	"	+ 4.9930	- 0.8329	+ 4.1601
8	4.7066	- 0.8938	+ 3.8128	+ 0.6912	"	+ 4.7738	- 0.9610	+ 3.8128
9	4.6141	- 1.1402	+ 3.4739	+ 0.6539	"	+ 4.6813	- 1.2074	+ 3.4739
10	4.5480	- 1.4082	+ 3.1398	+ 0.6198	"	+ 4.6152	- 1.4754	+ 3.1398
11	4.4979	- 1.6895	+ 2.8084	+ 0.5894	"	+ 4.5651	- 1.7567	+ 2.8084
12	4.4600	- 1.9774	+ 2.4826	+ 0.5634	"	+ 4.5272	- 2.0446	+ 2.4826
13	4.4351	- 2.2524	+ 2.1827	+ 0.5424	"	+ 4.5023	- 2.3196	+ 2.1827
14	4.3609	- 2.6485	+ 1.7124	+ 0.5241	"	+ 4.4276	- 2.7157	+ 1.7124
15	4.2967	- 2.8174	+ 1.4793	+ 0.5108	"	+ 4.3639	- 2.8846	+ 1.4793
16	4.2219	- 3.0623	+ 1.1596	+ 0.4800	"	+ 4.2891	- 3.1295	+ 1.1596
17	4.1149	- 3.2782	+ 0.8367	+ 0.4994	"	+ 4.1821	- 3.3454	+ 0.8367
18	3.9739	- 3.4740	+ 0.5000	+ 0.5000	"	+ 4.1411	- 3.5412	+ 0.5000

Shears for side span

Left Span loaded

	+ shear	- shear	Sum	Center Span loaded	Right side span loaded	To full load	Max neg.
1	3.3489	0.0000	+ 3.3489	- 6.7532	+.1515	- 3.2528	- 6.7532
2	2.4832	- 0.1343	+ 2.3489	- 6.7532	"	- 4.2528	- 6.8875
3	1.7512	- 0.4023	+ 1.3489	"	"	- 5.2528	- 7.1555
4	1.1510	- 0.8021	+ 0.3489	"	"	- 6.2528	- 7.5555
5	0.6798	- 1.3309	- 0.6511	"	"	- 7.2528	- 8.0841
6	0.3341	- 1.9850	- 1.6509	"	"	- 8.2528	- 8.7382
7	0.1093	- 2.7604	- 2.6511	"	"	- 9.2528	- 9.5172
8	0.0000	- 3.6511	- 3.6511	"	"	- 10.2528	- 10.4046

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Dead Load for suspension bridge

Approximate weight of metal

Stringers	165
Floor beam	190
Lateral Bracing	65
trusses	1340
misc.	50
	1810# per lin ft

weight of deck flooring
86 × 20 = 1720#
Curb 2@75 = 150
Handrails 2@60 = 120
1990#
1810
3800# ÷ 2 = 1900# per lin ft of truss

Dead Load Panel Concentration 1900 × 12.75 = 24200#

Live Load for suspension bridge -

Uniform load $w = \frac{100.000}{170+\ell} = \frac{100000}{170+\frac{459}{328}} = 323 kg/m^2 = 66\#/0'$

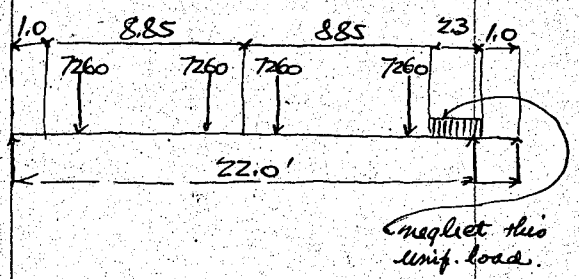
Impact allowance $= \frac{20}{60+\frac{459}{328}} = 10\%$.

Including both side spans use the same uniform load and impact throughout.

Motor truck loading

Rear wheel	6600	Front wheel
Impact 10%	660	1/3 × 7260 = 2420#
	7260#	

Max Reaction on trusses

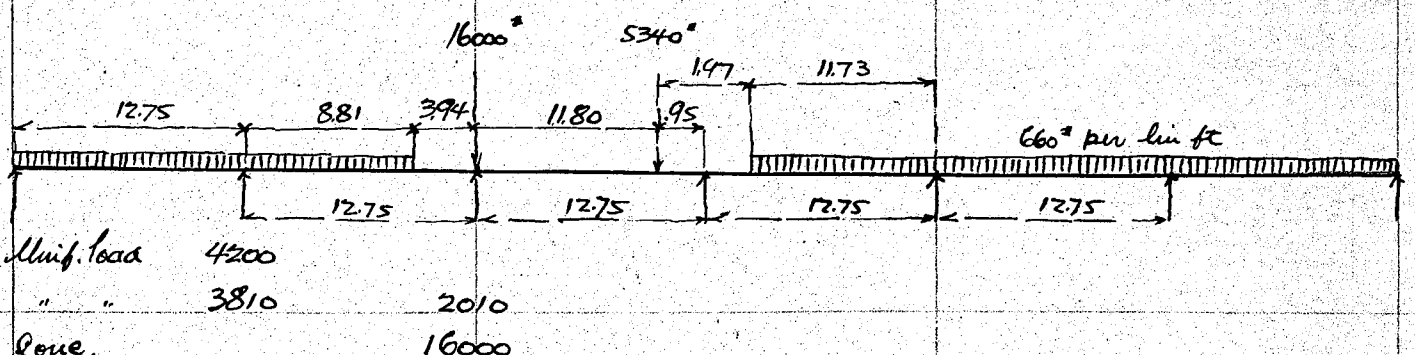


Occupied space of two motor trucks side by side
2@ 8.85 = 17.7'
Reaction from motor truck loading (Rear wheel).
2@14520 × 12.15/22 = say 16000#

Front wheel reaction = 1/3 × 16000 = 5340#

Concentration due to uniform load 66 × 10 = 660# per lin ft of truss
or 660 × 12.75 = say 8400#

Live Load Panel concentration on truss



Unif. load	4200				
" "	3810	2010			
Cone.		16000			
Cone.		400	4940		
Unif. load			3560	4190	4200
				4200	4200
	8010	18410	8500	8390	8400

Assume 8400# panel concentration throughout and add 10,000# single concentration at panel point considered for motor truck loading.

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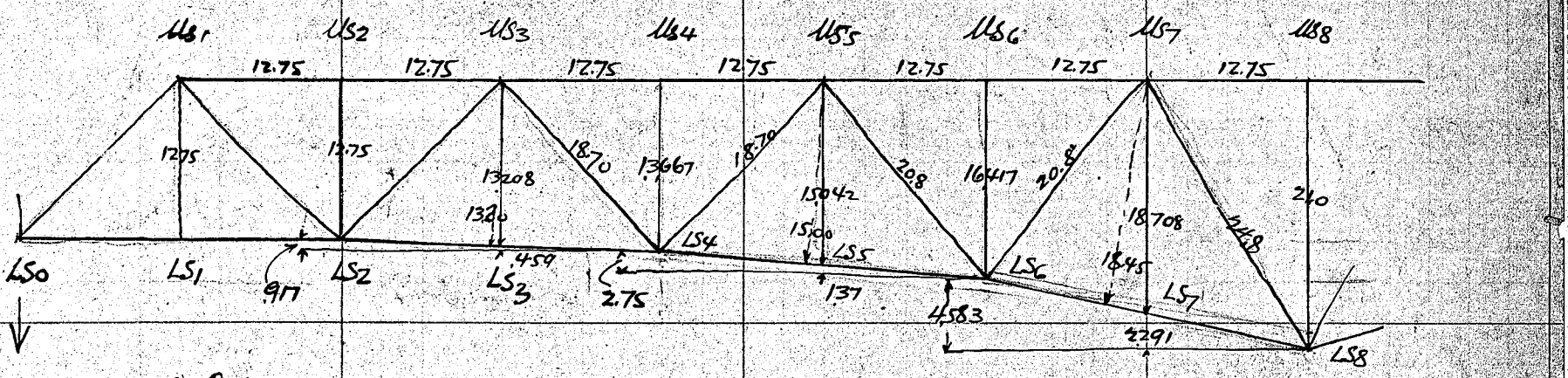
CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Dead and Live Load stresses of Side span



Sec θ = 1.41 Sec θ = 1.370 Sec θ = 1.265

Dead Load Chord Stresses

Member			Moment	Arm		Stress
Ls0-Ls2	- 39.4784	× 24200#	= - 955,000#	12.75	=	74900# C
Us1-Us3	- 95.0855		- 2300.000	12.75	=	180400 T
Ls2-Ls4	- 161.7397		- 3910.000	13.20	=	296000 C
Us3-Us5	- 241.1515		- 5840.000	13.67	=	427500 T
Ls4-Ls6	- 333.3144		- 8050.000	15.00	=	538000 C 538
Us5-Us7	- 437.2476		- 10590.000	16.42	=	643.000 T
Ls6-Ls8	- 555.8873		- 13460.000	18.45	=	730.000 C
Us7-Us8	- 688.825		- 16650.000	21.00	=	793.000 T

Live Load Chord Stresses

Due to Uniform Panel Concentration

Member				Arm			Summary Stresses
Ls0-Ls2	- 84.104	× 8400	= - 706.000	12.75	=	55300# C	59900# C
Us1-Us3	- 172.208	× 8400	= - 1449.000	12.75	=	113.500 T	120700 T
Ls2-Ls4	- 258.312		- 2170.000	13.20	=	164500 C	174900 C
Us3-Us5	- 344.416		- 2890.000	13.67	=	212.000 T	225400 T
Ls4-Ls6	- 430.520		- 3610.000	15.00	=	240500 C	255800 C
Us5-Us7	- 516.624		- 4330.000	16.42	=	264000 T	280700 T
Ls6-Ls8	- 602.728		- 5060.000	18.45	=	274000 C	291400 C
Us7-Us8	- 688.825		- 5775.000	21.00	=	275000 T	292400 T

Due to Extra motor truck loading

Member				Arm			
Ls0-Ls2	- 4.5714	× 10.000	= - 45700	12.75	=	3600# C +	
Us1-Us3	- 9.1428		- 91400	12.75	=	7200 T	
Ls2-Ls4	- 13.7142		- 137000	13.20	=	10400 C	
Us3-Us5	- 18.2856		- 182800	13.67	=	13400 T	
Ls4-Ls6	- 22.8570		- 228600	15.00	=	15300 C	
Us5-Us7	- 27.4284		- 274300	16.42	=	16700 T	
Ls6-Ls8	- 31.9998		- 320000	18.45	=	17400 C	
Us7-Us8	- 36.5716		- 365700	21.00	=	17400 T	

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MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Hakuchi-Bashi- for Tokushima-Ken

Dead and Live load stresses of side span

Dead load web stresses

member			shear				stress	
LS_0-US_1	-3.2528	× 24200 =	78600#	× 1.41		=	111000#	T
US_1-LS_2	-4.2528		102800	1.41		=	145000	C
LS_2-US_3	-5.2528		127000	1.41	× .965	=	173000	T
US_3-LS_4	-6.2528		151200	1.37	× .965	=	200.000	C
LS_4-US_5	-7.2528		175400	1.37	× .907	=	218.000	T
US_5-LS_6	-8.2528		199600	1.265	× .925	=	234.000	C
LS_6-US_7	-9.2528		223800	1.265	× .878	=	248.000	T
US_7-LS_8	-10.2528		248000	1.170	× .902	=	262.000	C

Live load web stresses

Due to uniform concentration.

member								Summary Live loads	
LS_0-US_1	-6.7532	× 8400 =	56800	× 1.41		=	80.000#	T	85150# T
US_1-LS_2	-6.8875		57800	× 1.41		=	81500	C	88600 C
LS_2-US_3	-7.1555		60000	× 1.41	× .965	=	81700	T	92100 T
US_3-LS_4	-7.5555		63500	× 1.37	× .965	=	84000	C	99400 C
LS_4-US_5	-8.0841		67800	× 1.37	× .907	=	84000	T	105100 T
US_5-LS_6	-8.7382		73400	× 1.265	× .925	=	86000	C	113500 C
LS_6-US_7	-9.5172		80000	× 1.265	× .878	=	88800	T	123500 T
US_7-LS_8	-10.4046		87500	× 1.17	× .902	=	92200	C	134500 C

Due to extra motor truck concentration.

member	left span	center span						
LS_0-US_1	0.0000	+.3650 =	-0.3650	× 10,000 =	3650	× 1.41	=	5150# T
US_1-LS_2	-0.1343	+.3650	-0.4993		5000	× 1.41	=	7100 C
LS_2-US_3	-0.4023	"	-0.7673		7670	× 1.41 × .965	=	10400 T.
US_3-LS_4	-0.8021	"	-1.1671		11670	× 1.37 × .965	=	15400 C
LS_4-US_5	-1.3309	"	-1.6959		17000	× 1.37 × .907	=	21100 T
US_5-LS_6	-1.9850	"	-2.3500		23500	× 1.265 × .925	=	27500 C
LS_6-US_7	-2.7604	"	-3.1254		31250	× 1.265 × .878	=	34700 T
US_7-LS_8	-3.6511	"	-4.0161		40160	× 1.17 × .902	=	42300 C

Max hanger stresses

D.L. say 23145# T
L.L. say 20560# T
43605# T

Dead Load stress of verticals.
say 4500# C

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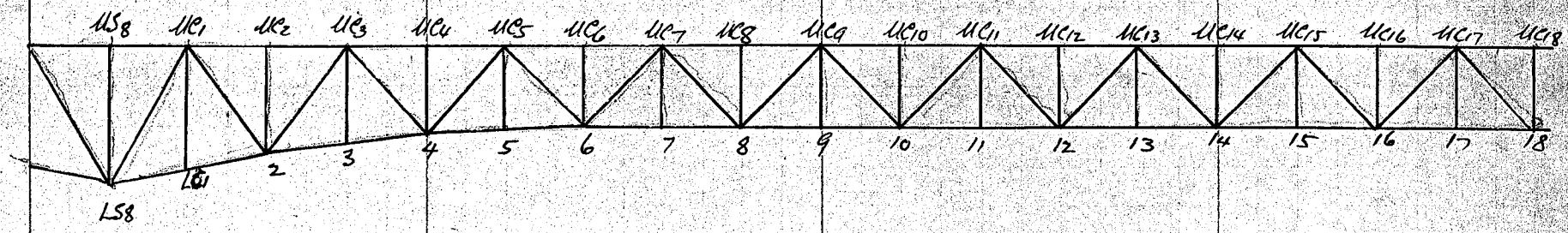
CALCULATIONS FOR

Hatsuchi-Bashi for Tokushima-Ken

Dead and Live Load Stresses of Centre span



	Member	D.L.	•24200#	Arm	D.L. Stress	-L.L.	8400#	
							Live Load Unif. concentration.	
0	Tower	-688.825	16650000 #	21.00	793000 # T	-7042723	-5910,000	+ 281000 # T
1	LS8-LS2	-600.3676	14520.000	18.45	787.000 C	-619.1511	-5200.000	- 282000 C
2	UC1-UC3	-542.8427	13120.000	16.42	800.000 T	-572.2970	-4800.000	- 292000 T
3	LS2-LS4	-477.6952	11560.000	15.00	771.000 C	-523.7691	-4400.000	- 293500 C
4	UC3-UC5	-417.3116	10100.000	13.67	740.000 T	-487.0721	-4090.000	300,000 T
5	LS4-LS6	-359.7762	8680.000	13.20	658.000 C	-457.7986	-3850.000	292.000 C
6	UC5-UC7	-306.6021	7420.000	12.75	582.000 T	-432.4164	-3630.000	284500 T
7	LS6-LS8	-262.4841	6340.000	"	497.000 C	-407.9458	-3430.000	269.000 C
8	UC7-UC9	-206.8416	5000.000	"	392.000 T	-376.8118	-3165.000	248.000 T
9	LS8-LS10	-173.2660	4190.000	"	328.000 C	-355.7038	-2985.000	234000 C
10	UC9-UC11	-135.6903	3280.000	"	257.000 T	-326.5065	-2740.000	215000 T
11	LS10-LS12	-104.3864	2525000	"	198.000 C	-294.6521	-2470.000	194000 C
12	UC11-UC13	-77.3089	1870000	"	146500 T	-259.2257	-2180.000	171000 T
13	LS12-LS14	-54.0354	1310.000	"	103000 C	-222.4736	-1870.000	146600 C
14	UC13-UC15	-34.8698	843000	"	66000 T	-181.0567	-1520.000	119000 T
15	LS14-LS16	-19.8449	480.000	"	37600 C	-139.9563	-1175.000	92.000 C
16	UC15-UC17	-9.8061	237.000	"	18600 T	-94.3052	-792.000	62.000 T
17	LS16-LS18	-7.3120	177.000	"	13900 C	-51.4472	-432.000	33800 C

	Live Load Unif. Concentration +LL.	8400#		Motor truck on centre span 10,000#		Motor truck on left side span 10.000#		ditto right side span	Motor truck on centre span 10.000#	
0	+ 15.4473	+ 129800		-365700 #	17400 # T	-154000 #	7300 # T	7300 # C	0000	
1	+ 18.7147	+ 157000		-342100	18550 C	-145900	7900 C	7900 T	+ 42000	
2	+ 29.4543	+ 247000		-363200	22100 T	-137300	8300 T	8300 C	+ 888.00	
3	+ 47.0839	+ 396000		-378500	25300 C	-128700	8600 C	8600 T	+ 1344.00	
4	+ 69.7605	+ 585000		-389700	28500 T	-120100	8800 T	8800 C	+ 178700	
5	+ 98.0224	+ 823000		-395100	29900 C	-111600	8400 C	8400 T	+ 218600	
6	+ 125.8143	+ 1055000	82700 C	-390600	31000 T	-103000	8100 T	8100 C	+ 252000	19800 C
7	+ 145.4617	+ 1221000	95700 T	-390600	30600 C	-94400	7400 C	7400 T	+ 278000	21800 T
8	+ 170.1702	+ 1430000	112000 C	-380700	29900 T	-85800	6700 T	6700 C	+ 295000	23200 C
9	+ 182.4378	+ 1530.000	120,000 T	-365900	28700 C	-77200	6100 C	6100 T	+ 304000	23800 T
10	+ 190.8162	+ 1600.000	125500 C	-345700	27100 T	-68600	5400 T	5400 C	+ 313000	24600 C
11	+ 195.2657	+ 1640.000	128500 T	-320500	25100 C	-60000	4700 C	4700 T	+ 293000	23000 T
12	+ 181.9568	+ 1530000	120000 C	-290000	22800 T	-51500	4000 T	4000 C	+ 274000	21500 C
13	+ 168.4582	+ 1414.000	111.000 T	-254600	20000 C	-43000	3400 C	3400 T	+ 246000	19300 T
14	+ 146.1867	+ 1230.000	96500 C	-212100	16600 T	-34000	2700 T	2700 C	+ 210000	16500 C
15	+ 120.1114	+ 1010.000	79200 T	-168000	13200 C	-25700	2000 C	2000 T	+ 166000	13000 T
16	+ 84.5891	+ 710.000	55700 C	-117300	9200 T	-17200	1300 T	1300 C	+ 117000	9200 C
17	+ 44.1352	+ 370.000	29000 # T	-61100	4800 C	-8600	700 C	700 T	+ 60600	4800 T

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CALCULATIONS FOR

Nakuchi-Bashi for Tokushima-Ken

Dead and Live Load Stresses of Center span

Summary for Live Load Stresses.

					Summary					Summary
0		281.000 T	17400	7300	305700 T					
1	LS_8-LC_2	282000 C	18550	7900	308450 C					
2	UC_1-UC_3	292000 T	22100	8300	322400 T					
3	LC_2-LC_4	293500 C	25300	8600	327400 C					
4	UC_3-UC_5	300000 T	28500	8800	337300 T					
5	LC_4-LC_6	292.000 C	29900	8400	330300 C					
6	UC_5-UC_7	284500 T	31000	8100	323600 T	82700 C	19800	8100	110600 C	
7	LC_6-LC_8	269000 C	30600	7400	307000 C	95700 T	21800	7400	124900 T	
8	UC_7-UC_9	248000 T	29900	6700	284600 T	112000 C	23200	6700	141900 C	
9	LC_8-LC_{10}	234000 C	28700	6100	268800 C	120000 T	23800	6100	149900 T	
10	UC_9-UC_{11}	215000 T	27100	5400	247500 T	125500 C	24600	5400	155500 C	
11	LC_{10}-LC_{12}	194000 C	25100	4700	223800 C	128500 T	23000	4700	156200 T	
12	UC_{11}-UC_{13}	171000 T	22800	4000	197800 T	120000 C	21500	4000	145500 C	
13	LC_{12}-LC_{14}	146600 C	20000	3400	170000 C	111000 T	19300	3400	133700 T	
14	UC_{13}-UC_{15}	119000 T	16600	2700	138300 T	96500 C	16500	2700	115700 C	
15	LC_{14}-LC_{16}	92000 C	13200	2000	107200 C	79200 T	13000	2000	94200 T	
16	UC_{15}-UC_{17}	62000 T	9200	1300	72500 T	55700 C	9200	1300	66200 C	
17	LC_{16}-LC_{18}	33800 C	4800	700	39300 C	29000 T	4800	700	34500 T	

Dead load web stresses of center span

	Number		24200#				
1	LC_8-UC_1	+ 6.1317	148000	× 1.17	× .902	=	156000# C
2	UC_1-LC_2	+ 5.8014	140200	× 1.265	× .878	=	154000 T
3	LC_2-UC_3	+ 5.4658	132000	× 1.265	× .925	=	154000 C
4	UC_3-LC_4	+ 5.1330	124000	× 1.37	× .907	=	154000 T
5	LC_4-UC_5	+ 4.7894	116000	× 1.37	× .965	=	153000 C
6	UC_5-LC_6	+ 4.4407	107300	× 1.41	× .965	=	146000 T
7	LC_6-UC_7	+ 4.1601	100500	× 1.41	× 1.000	=	142000 C
8	UC_7-LC_8	+ 3.8128	92200				130000 T
9	LC_8-UC_9	+ 3.4739	84000				118500 C
10	UC_9-LC_{10}	+ 3.1398	76000				107000 T
11	LC_{10}-UC_{11}	+ 2.8084	68000				96000 C
12	UC_{11}-LC_{12}	+ 2.4826	60000				84500 T
13	LC_{12}-UC_{13}	+ 2.1827	52800				74500 C
14	UC_{13}-LC_{14}	+ 1.7124	41500				58500 T
15	LC_{14}-UC_{15}	+ 1.4793	35800				50500 C
16	UC_{15}-LC_{16}	+ 1.1596	28000				39500 T
17	LC_{16}-UC_{17}	+ 0.8367	20200				28500 C
18	UC_{17}-LC_{18}	+ 0.5000	12100				17100 T

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Live load web stresses of Center span

Stresses due to positive shear

		8400#					10.000#			Summary	
1	+.84665	71200	× 1.17	× .902 =	75000 C	+0.9714	9714	× 1.17 × .902 =	10400 C	85400 C
2	+.76647	64400	× 1.265	.878 =	71500 T	0.9370	9370	× 1.265 .878 =	10400 T	81900 T
3	+.69865	58700	× 1.265	.925 =	68600 C	0.8986	8986	× 1.265 .925 =	10500	79100 C
4	+.63656	53400	× 1.37	.907 =	66200 T	0.8575	8575	× 1.37 .907 =	10700	76900 T
5	+.57873	48600	× 1.37	.965 =	64000 C	0.8149	8149	× 1.37 .965 =	10800	74800 C
6	+.53826	45200	× 1.41	.965 =	61500 T	0.7723	7723	× 1.41 .965 =	10500	72000 T
7	+.49930	42000	× 1.41	1.000 =	59200 C	0.7307	7307	× 1.41 1.000 =	10300	69500 C
8	+.47738	40100		=	56500 T	0.6912	6912	=	9800	66300 T
9	+.46813	39300		=	55400 C	0.6539	6539	=	9200	64600 C
10	+.46152	38800		=	54800 T	0.6198	6198	=	8700	63500 T
11	+.45651	38400		=	54200 C	0.5894	5894	=	8300	62500 C
12	+.45272	38000		=	53600 T	0.5634	5634	=	7900	61500 T
13	+.45023	37800		=	53200 C	0.5423	5423	=	7600	60800 C
14	+.44276	37200		=	52500 T	0.5241	5241	=	7400	59900 T
15	+.43639	36600		=	51500 C	0.5108	5108	=	7200	58700 C
16	+.42891	36000		=	50800 T	0.4800	4800	=	6800	57600 T
17	+.41821	35200		=	49600 C	0.4994	4994	=	7000 C	56600 C
18	+.41411	34800		=	49000 T	0.5000	5000	=	7000 T	56000 T

Stresses due to negative shear.

		8400#					10.000			Summary	
13	-.23196	19500	× 1.41	=	27500 T	-0.4109	4109	× 1.41	=	5800 T	33300 T
14	-.27157	22800	"	=	32200 C	-0.4294	4294	"	=	6000 C	38200 C
15	-.28846	24200	"	=	34200 T	-0.4450	4450	"	=	6300 T	40500 T
16	-.31295	26300	"	=	37000 C	-0.4554	4554	"	=	6400 C	43400 C
17	-.33454	28100	"	=	39600 T	-0.4611	4611	"	=	6500 T	46100 T
18	-.35412	29700	"	=	41900 C	-0.4627	4627	"	=	6500 C	48400 C

Section of truss members.

L

L_1L_2 and L_2L_2 233600# C

4Ls 6×4×½" = 19.0 qr. $r = 2.90$ $l = 216"$

Stress = $21300 (1 - 0.0055 l/r) = 12580$#"

SR = 18.60"

L_2L_3-L_3L_4 299400# C

				Moment of I.
4Ls 6×4×⅜	= 14.44	× 2.25² = 73.2	+ 27.0 × 2 =	127.2
2Pls 12¾"×⅜	= 9.36			130.0
	23.80			257.2

$r = \sqrt{\frac{257.2}{23.80}} = 3.28$

Unsupported length 225"

$l/r = 225/3.28 = 68.6$

$P = 21300 (1 - 0.0055 l/r) = 13250$#/□"

Section Required = 299400 ÷ 13250 = 22.6 □"

L_5L_5-L_5L_6. 347,500# C

4Ls 6×4×⅜	= 14.44	× 2.25² = 73.2	+ 54 =	172.2
2Pls 14×½	= 14.00			228.6
	28.44□"			400.8

$r = \sqrt{\frac{400.8}{28.44}} = 3.75$

Unsupported length 250"

$P = 21300 (1 - 0.0055 l/r) = 13500$#/□"

SR = 347500 ÷ 13500 = 2.58□"

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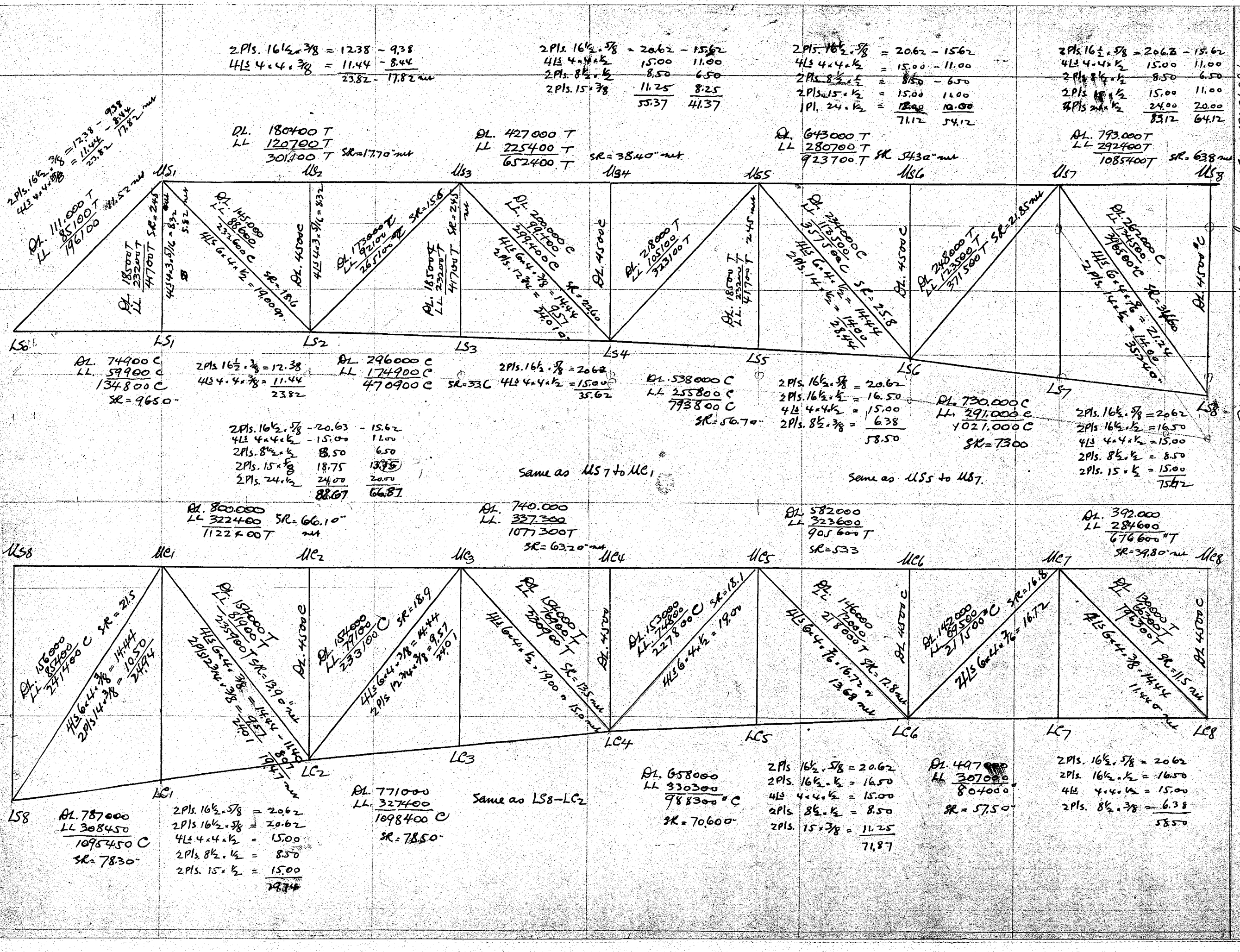

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横田淳民間系資料

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CALCULATIONS FOR

Katsuchi-Bashi for Yoshima-Ten

House and Section

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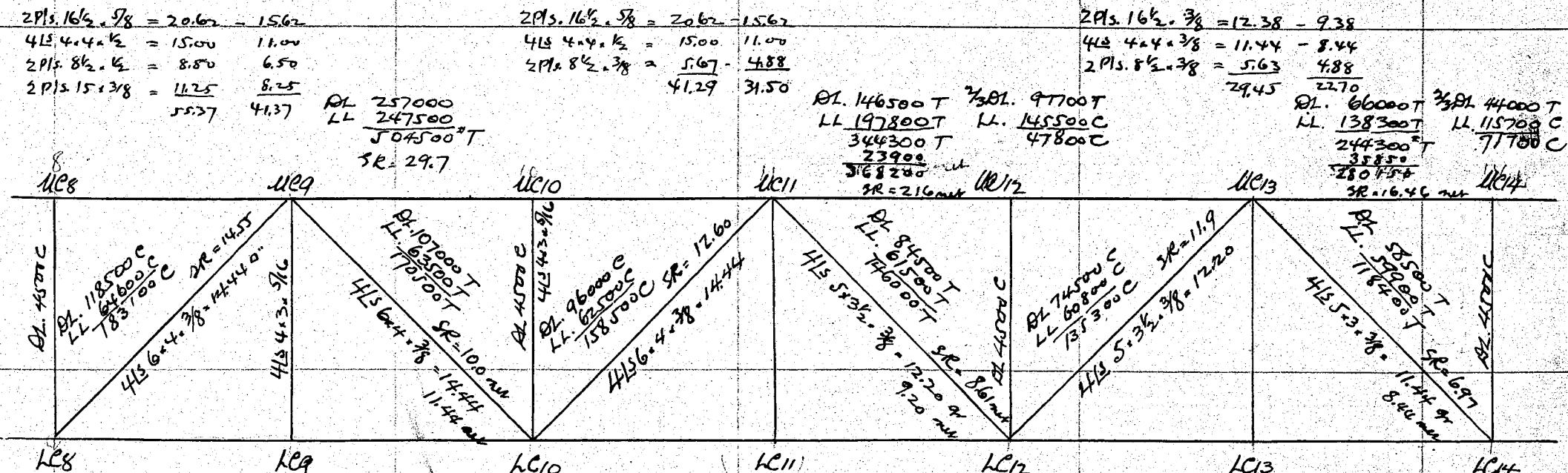

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CALCULATIONS FOR

Hatuchi-Baski for Iokushima-Ku

Ls-Ls8 Stress = 396500 # C
4Ls 6×4×9/16 = 21.24 × 2.52² + 77 = 212 r = √(498/38.24) = 3.6
2Pls. 14×5/8 = 17.00 286
38.24 498
Unsupported Length 24.8×12 = 298"
P = 21300 (1 − 0.0055 l/r) = 11600 #/□"
Section required 396500 ÷ 11600 = 34.10"

Try 4Ls 6×4×9/16 = 21.24 × 2.52² + 77 = 212 r = √(440/35.24) = 3.54
2Pls 14×½ = 14.00 228
35.24 440
Unsupported length 298"
P = 21300 (1 − 0.0055 l/r) = 11450 #/□"
Section required 396500 ÷ 11450 = 34.60" ok

LS8-UC1 Stress = 241400 # C
4Ls 6×4×3/8 = 14.44 × 2.25² + 54 = 127.0
2Pls 14×3/8 = 10.50 171.0
24.94 298
Unsupported Length 298" P = 21300 (1 − 0.0055 l/r) = 11200
Section required = 241400 ÷ 11200 = 21.50" r = √(298/24.94) = 3.46

LC2-UC3.1 Stress = 233100 # C
4Ls 6×4×3/8 = 14.44 r = 3.28 l = 250
2Pls 12¾×3/8 = 9.57
24.01 P = 21300 (1 − 0.0055 l/r) = 12360
Section = 233100 ÷ 12360 = 18.9 □"

LC4-UC5 Stress = 227800 # C
4Ls 6×4×½ = 19.00 r = 2.90 l = 225" P = 12280 #/□"
Section required = 227800 ÷ 12280 = 18.7 □"

LC6-UC7 211500 # C.
4Ls 6×4×7/16 = 13.68 net = 16.72 gr. P = 12,580 #/□"
Section required = 211500 ÷ 12.580 = 16.80 □"

LC8-UC9 183100 # C
4Ls 6×4×3/8 = 14.44 P = 12580 #/□" SR = 14.55

LC10-UC11 158500 # C SR = 12.6
4Ls 6×4×3/8 = 14.44

LC12-UC13 135300 # C r = 2.55 l = 216" P = 11400 #/□" SR = 11.9
4Ls 5×3½×3/8 = 12.20

LC14-UC15 109200 # C 6800 # T. 4Ls 5×3×3/8 - r = 2.62 l = 216 P = 11630 #/□"
3400
112600 # C SR = 9.670" 4Ls 5×3×3/8 = 11.44 □"

LC16-UC17 85100 C 27100 T
13550
98650 C SR = 8.47 4Ls 5×3×3/8 = 11.44

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CALCULATIONS FOR

Hatuchi-Bashi for Tokushima-Ken

Stress in cable

Horizontal thrust = 25.0808 × 24200# = 607.000# Dead Load
25.0808 × 8400 = 210500 Live Load unif.
1.5116 × 10000 = 15000 Live Load extra motor truck
832500#

tanθ at tower = 0.4793
secθ = 1.109 Stress in cable = 832500 · 1.109 = 925000#

Use 19 cables @ 1.59" dia circumference 5"
Required strength each cable 925000 ÷ 19 = 48700# or 21.75 tons direct stress.

Bending stress of each cable at Saddle on tower
Diameter of bend = 108" diameter of cable = 1.59" diameter of each wire = .106"
See Pocket book by Kent PP 1185
The bending stress = 25.000# about or 11.15 tons on sheaves.
Direct load 21.75
32.90 tons per cable

The Breaking strength of wire rope 88 tons. or factor of safety = 2.7 this will be all right for statical stress.

For dead load Stress only = 607000 · 1.109 = 673000# (673.000)
673000 ÷ 19 = 35500# or 18.65 tons

Stresses due to eretion.
weight of metal 1810 ÷ 2 = 905
Live Load during eretion say 45
950 · 12.75# = 12100#
Horizontal thrust = 25.0808 · 12100 = 307500#
Stress in cable 307500 · 1.109 ÷ 19 = 17900# or 8.• tons per cable.

Max load on tower
Horizontal stress = 832500#
load on tower = 832500 × .53 = 441.000#

Upward Pull at anchorage 441.000# each cables ÷ 30.000 = 14.7 ½ ft of concrete
Horizontal pull " " 832500# each cables ÷ 30.000 = 27.7 " "

For both sides of suspension cables 2 @ 14.7 = 29.4 ½ ft
2 @ 27.7 = 55.4 " "

Load from stiffening truss
See pp 35 Dt. 10.2528 × 24200 = 248000#
LL. 10.4046 × 8400 = 87300
335300#

Design shoes for 350.000# each
Try 6" pin. bearing for pin = 1.875" area = 11.250"
Unit bearing = 350.000 / 2 · 11.25 = 15550#/□" ok.

Bending Moment = 175 000 · 2" = 350.000"#
fibre stress = 350.000 · 3 / 63.6 = 16500#/□" ok.

Length of roller :- 4½" roller - 4.5 × 610 = 2740#
350.000 ÷ 2740 = 128" 6 Rollers 21.5" net
Bearing on masonry wt = 350.000 area = 33" × 37½ = 1240□"
Unit bearing = 350.000 ÷ 1240 = 282#/□" for Roller shoe
Bearing area of fixed shoe = 35" × 29.5 = 1030□"
Unit bearing = 350.000 ÷ 1030 = 340#/□" for fixed shoe.

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima Ken

Anchorage for side span of Stiffening truss. see page 35

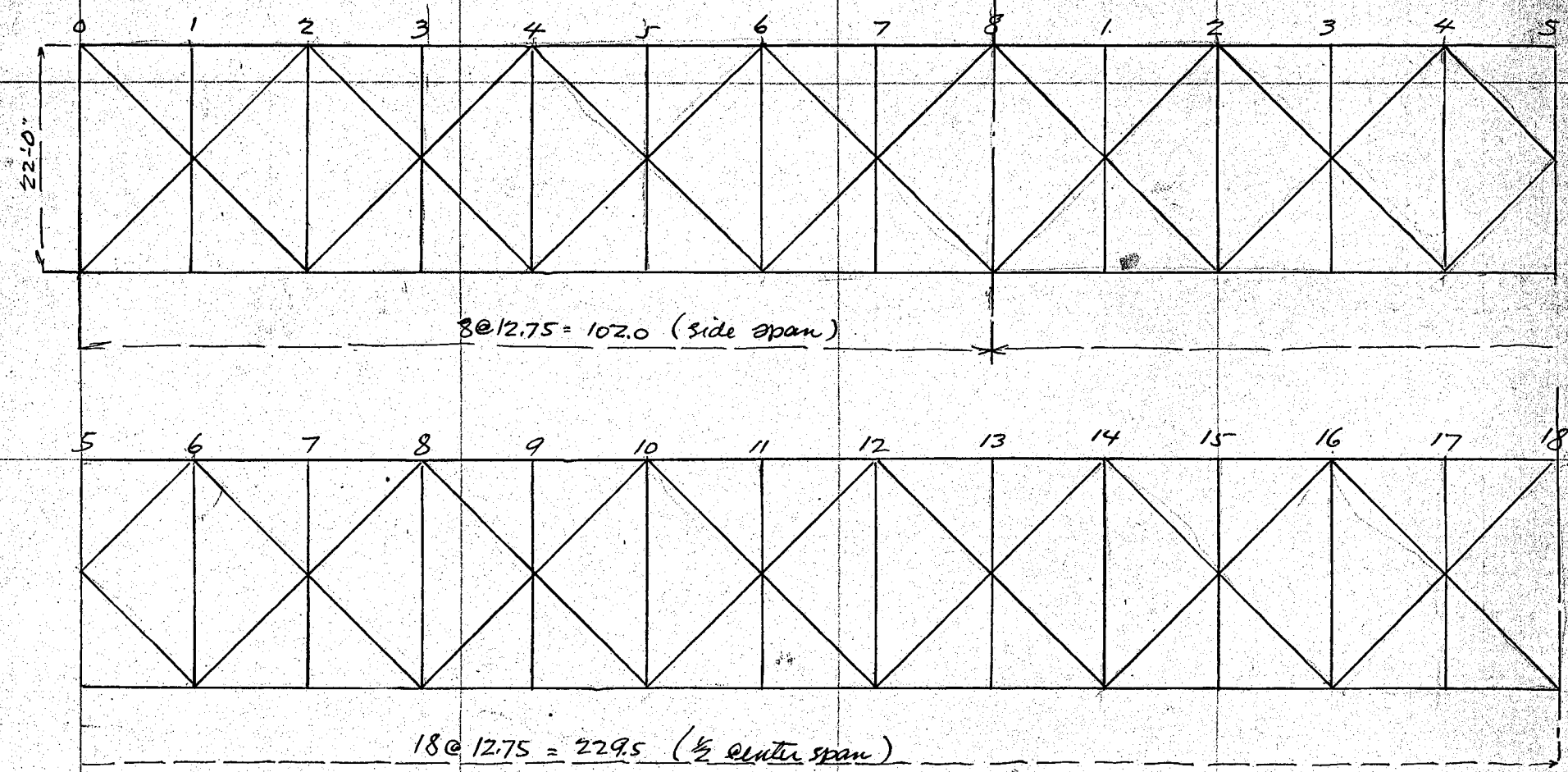
Dead Load 3.2528 × 24200 = 78600 − ½ · 24200 = 66500
Live Load 6.7532 × 8400 = 56800
123300# neg. reaction
Design for 130.000# Pull.

Use 4½" pin. thickness of plate 1½"
bearing area 4.5 × 1.5 = 6.75 Unit Bearing = $\frac{130.000}{2 @ 6.75}$ = 9650#/□" for anchorage.

Roller 4" 4 × 610 = 2440# 130.000 ÷ 2440 = 53.3" 5 @ 10.7" net.

Weight of anchorage required 260.000# ÷ 30.000 = 8.67 ≒ 7 ton of Concrete.

Lateral Bracing



Diagonal length = 16.82' Sec θ = $\frac{16.82}{11.}$ = 1.53.

Wind load assumed span length = 140 meter
additional wind load 15 × 9 = 135 kg per meter.
For loaded chord 400 For unloaded chord 265
135
535 = 800 kg per meter

Unloaded chord assumed as ½ of loaded chord, = 535# per lin ft.
panel concentration = 535 × 12.75 = 6820#
stress in diagonal = 6820 · 1.53 = 10450.0# multiplied by influence values.

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Stresses in Lower Lateral.

Shear due to unit load

Center span see pp 35 max shear + 8.4665 +C + Htan θ. =

$V_0 + \frac{M_3 - M_2}{\ell} =$ shear

Shear on side span

Due to full load on center span $\frac{M_2}{\ell_1}$ where $M_2 = -668.825$

$\Sigma H_m = 1379.444$

$2048.269 \div 10Z =$ say 20.0

Summary of shears

							net section		
1	20.0	+ 0.00	= 20.00	@ 10450# =	209.000	104500	6.15	2Ls 6×6×7/16	10.12 - 8.37 net
2	20.0	0.13	20.13		210.000	105000	6.18	"	
3	20.0	0.40	20.40		213.000	106500	6.28	"	
4	20.0	0.80	20.80		217.000	108500	6.39	"	
5	20.0	1.33	21.33		223.000	111500	6.55	2Ls 6×6×1/2	11.50 - 9.50 net
6	20.0	1.98	21.98		228.000	114000	6.71	"	
7	20.0	2.76	22.76		238 000	119000	7.00	"	
8 tower	20.0	3.65	3365		247.000	123500	7.26	"	
1	8.47	11.33	19.80		207.000	103500	6.10	2Ls 6×4×1/2	9.50 - 7.50 net
2	7.66	10.67	1833		192.000	96000	5.66	"	
3	6.97	10.00	16.97		177.000	88500	5.21	2Ls 6×4×7/16	8.36 - 6.61 net
4	637	9.33	15.70		164.000	82000	4.83	"	
5	5.79	867	14.46		151.000	75500	4.45	2Ls 6×4×3/8	7.22 - 5.72 net
6	5.29	8.00	1328		139.000	69500	4.10	"	
7	4.99	7.33	12.32		129.000	64500	3.80	2Ls 5×3½×3/8	6.10 - 4.60 net
8	4.77	6.66	11.43		119.000	59500	3.50	"	
9	4.68	6.00	10.68		111.000	55500	3.27	"	
10	4.62	5.33	9.85		104.000	50200	2.96	"	
11	4.57	4.67	9.29		96.000	48000	2.83	2Ls 5×3½×7/16	5.12 - 3.77 net
12	4.53	4.00	8.53		89.000	44500	2.62	"	
13	4.50	3.33	7.83		82.000	41000	2.42	"	
14	4.43	2.67	7.10		74.000	37000	2.18	"	
15	4.36	2.00	6.36		66.000	33000	1.95	"	
16	4.29	1.33	5.62		59.000	29500	1.74	"	
17	4.18	0.67	4.85		51.000	25500	1.50	"	
18	4.14	0.00	4.14		43.000	21500	1.27	"	

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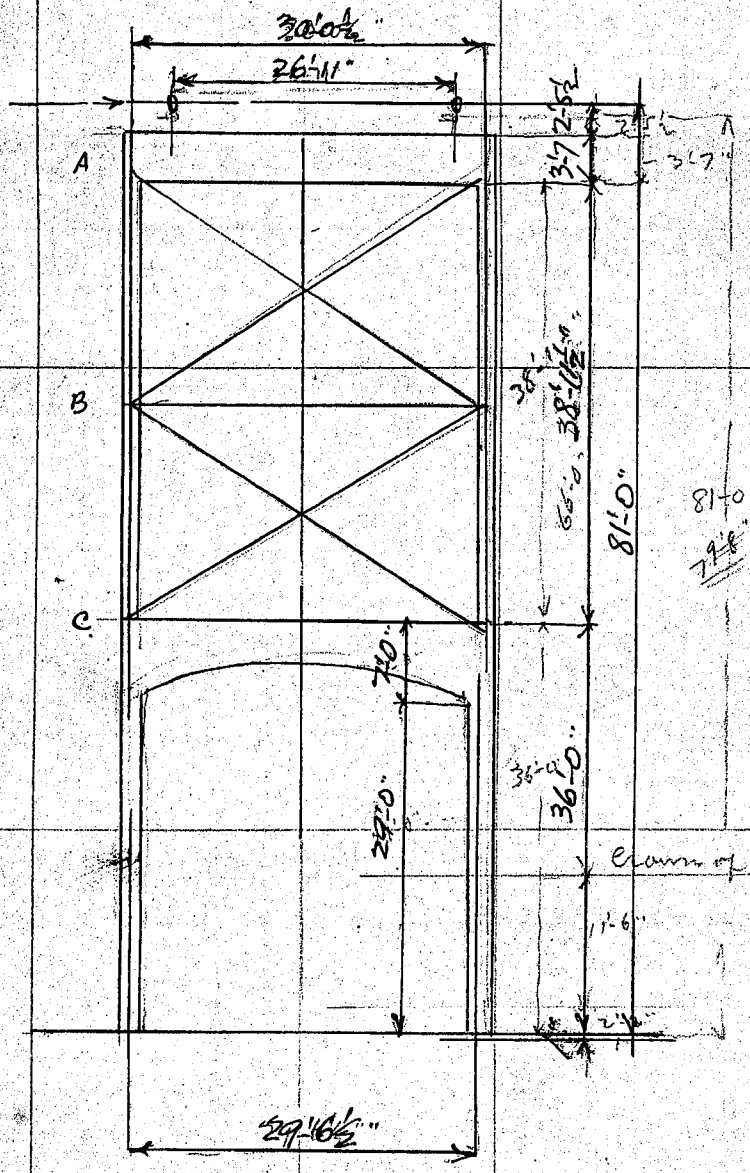
CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-ken

Design of tower



Load on tower 441,000# for Dead and Live Loads.
Transverse frame due to Earthquake
assumed $k = 0.20$
Horizontal earthquake force = 441000 × 0.2 = 88200#

Stress in diagonal $88200 \times \frac{36.9}{31.} = 105.000^{\#}$

$P = 21300\,(1 - 0.0055\,\ell/r) = 10200\ \#/\square"$
where $\ell = 18.45 \times 12 = 222"$ $r = 2.35$
for earthquake stress $10200 \times 1.8 = 18350\ \#/\square"$
Section required = 105000 ÷ 18350 = 5.73 □"
4Ls 5×5×3/8 = 14.44 □"
tie together two groups of 2Ls at intervals of 7' about.
During earthquake the cable is not stiff enough to carry the seismic force to top of tower and the stiffening truss will carry the greater part of the said force
To figure the strength of tower the seismic force reduced to ½ i.e. $k = 0.1$. and this assumption is big enough to my opinion.

Wind pressure assumed 400# per lin ft of height.

at A	400 × 10 =	4000#
" B	400 × 20 =	8000#
" C	400 × 46 =	18400#

Reaction from wind load.

	4000 × 20 =	80000
	8000 × 10 =	80000
at C.	18400 ×	160.000'# ÷ 30 = 5330.0#
	4000 × 76 =	304000
	8000 × 56 =	447500
m of I	18400 × 36 =	662000
30400.		1413500 ÷ 29.5 = 48000#

Column Section above C
441000# Ds L Loads.
32000 Dead Load of Column
473000#

Assumed section

4Ls 6.6.3/8	= 17.44	× 10.61^2 + 61 =	2026	
2Pls. 24×½	= 2400	=	1152	
	41.44		$3178^{(n)4}$	

$r = \sqrt{\frac{3178}{41.44}} = 8.75$

$P = 21300\,(1 - 0.0055\,\frac{240}{8.75}) = 15500^{\#}$ call this 14000#/□"

Required section 473.000 ÷ 14000 = 33.8 □" Assumed section Ok.

Column Section below point C.
Total wind load 30400# or 15200# per column
moment due to wind load = 15200 × 14.50 = 220.000'# direct load = 48000#
Assumed section.

		m of I		
8Ls 6×6×½	= 46.00	× 13.57^2 + 160 =	8630	$r = \sqrt{\frac{10880}{76.}} = 11.94$
2Pls 30×½	= 30.00		2250	
	76.00		$10880^{(n)4}$	$\ell = 29 \times 12 = 348"$

$P = 21300\,(1 - 0.0055\,\ell/r) = 17900\ \#/\square"$ say 14000#/□"

Fibre stress due to wind = $\frac{220.000 \times 12 \times 15.25}{10880} = 3680\ \#/\square"$

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Direct load 441.000
Column 56.000

497000
From wind 48000
545000 ÷ 76.0" = 7170"
From bending. 3680
10850 #/o" -

Stresses due to Earthquake.
Seismic force per Column say 44100 # Contra flexure Point taken at $\frac{29}{2} = 14.5$
Moment = 44100 · 14.5 = 640.000 1#
Moment = 11200 · 14.5 = 162.000
802000 1#
Seismic force of Col. 56000 × 0.2 = 11200 #

Fibre stress = $\frac{802000 \times 12 \times 15.25}{10880} = 13500$ #/o"

Direct Load from earthquake thrust 2 × 44100 × $\frac{81}{29.5}$ = ± 242000 #
Direct D+L load. 441.000
weight of Column 56000
739000 # ÷ 760 = 9720
13500
23220 #/o" ok

Unit stress during earthquake 14000 × 1.8 = 25600 #/o.

Stress in longitudinal Elevation - 2 Ribs 3'0" at top 7'0" at bottom at bottom of base.

Direct load - 441 000
wt of Column 56000
497000 ÷ 2 = 258500 #
Moment due to Earthquake. 441.00 × 81.0 = 3,570.000
wt of col. 56000 × 0.2 = 11200 · 40.0 = 448.000
4018000 1#

Reaction on one rib = $\frac{4018,000}{7}$ = 575000
Direct load 258500
833500 #

Sectional area = 76 ÷ 2 = 38
Unit stress = 833500 ÷ 38 = 21900 #/o"
Section is good for 38 · 25600 = 972,000 #

Section of Anchorage as tension
Tension. 575000
Comp. 258500
316500 # Tension.
Use 4 - 2½" ϕ rods = 4 × 4.90 = 19.60"
net dia = 2" . 4 @ 3.14 = 12.60" net.
Unit stress = 316500 ÷ 12.6 = 25000 #/o" ok

Bearing on base. max load say 972.000 ÷ 1590 = 610 #/o".
Base pl = 36", 3'-8" = 1590 o".
For base of 30" · 44 = 1320 972.000 ÷ 1320 = 735 #/o. ok. including earthquake stress.

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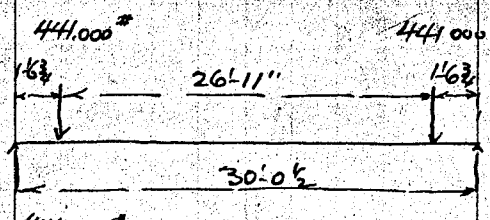
CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Saddle girder at top of tower



Dead Load assumed. 700# per ft

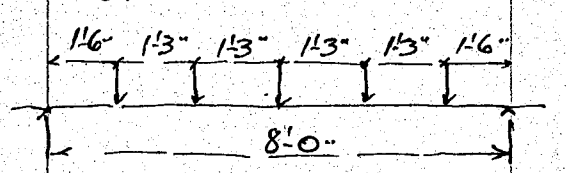
Moment = 441000 . 1.56 = 687000
Girder = 8 × 200 . 30.04² = 22500
709500 1#

Depth of girder = 3.55 Effective depth say 3.40'
Stress = 709500 ÷ 3.40 = 208000#
section reqd 208000 ÷ 17000 = 12.25
For one half . 6.120" net
Use 1L 6×6. 3/8 = 4.36 - .99 = 3.37
1Pl. 1 × 3/8 4.12 - 1.32 = 2.80
6.170" net.

Tension in suspender assumed 50.000#
net section required = 50.000 ÷ 17000 = 2.940" net
Use 2½" rod threaded net section at thread 2"ϕ = 3.10" o/c.

Tension in each cable 21.75 tons or 48800# call this 50000#
2. Anchorage rod for adjustment 25.000# each rod.
2" gross 1½" net net area reqd. 25.000 ÷ 17000 = 1.470"
1½"nd = 1.770" Unit stress = 25.000 ÷ 1.77 = 14100#/o" ok

Design of Anchorage Girder.



Effective depth = 1.77

Center to center of End bearings - 8'-0"
Weight of girder neglected.
Moment at center
= 125.000 × 4 = 500.000
50000 × 3.75 = 187.000
313.000 1#

Ends of girder anchored into concrete for continuity
Let us assume the moment = 313.000 . ⅔ = 208.000 1#
Section assumed 2Pl.24 × 3/8 = 18.0
2Ls 5×3½×3/8 = 6.10 5.35 0" net
208000 ÷ 1.77 = 117500#
section reqd = 117500 ÷ 17000 = 6.92
8 web = 2.25
4.67 0" net.

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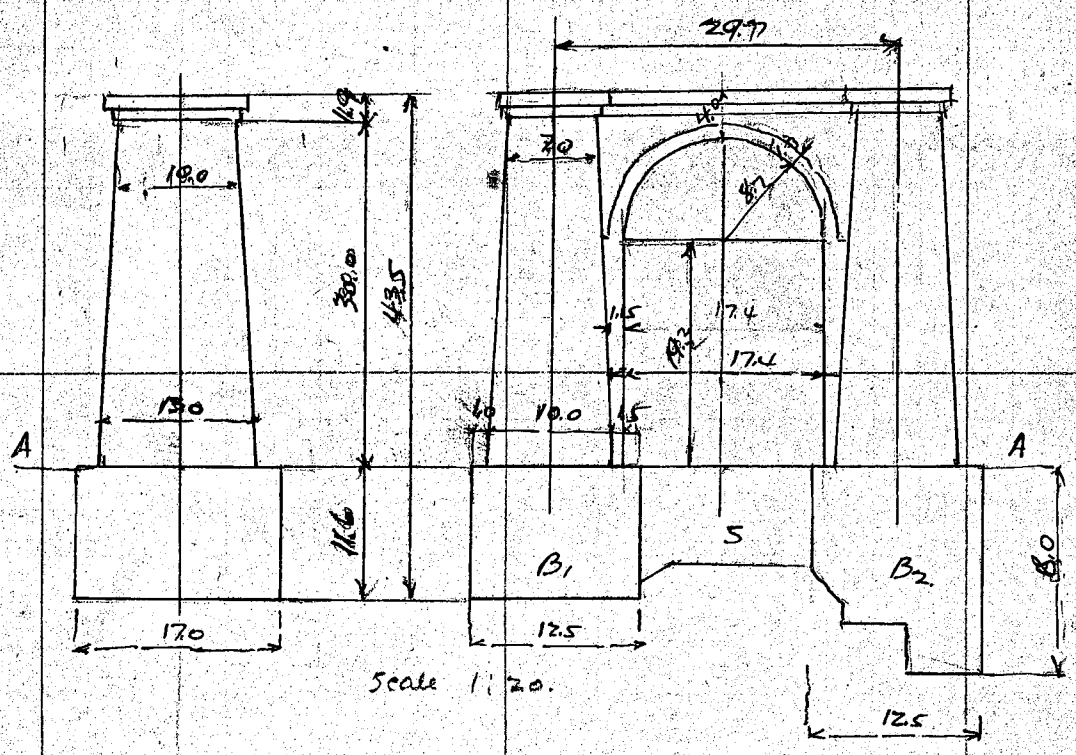
CALCULATIONS FOR

Takeuchi-Baski for Tokushima-Ken

Design of Pier P2-2.



Scale 1:120.

Volume of Concrete ½ Pier.

Coping. 8.5 × 11.5 = 97.8
5.5 × 10.6 = 58.3
156.1 × 1.2 = 187.0
11.0 7.7 × 10.7 = 82.5
4.7 × 12.5 = 51.7.
134.2 × .7 = 94.0
281.0

Shaft. 7 × 10 = 70 neglecting corners
10 × 13 = 130
200 ÷ 2 = 100
volume = 100 × 30 = 3000

fillet. 1.63 × 19.2 × 4.5 = 132.0
arch 12.02 × 10.8 = 130 285 206
- ins ¼ × 80 = 78.5 206
51 51.5 × 4 = 206.0
do:- 19 × 4.5 51.5 85.0
291.0

Summary of concrete above line AA. ½ pier.

coping	281	
shaft	3000	
Between shafts.	291	add 132.0.
	3572 ÷ 216 = 16.5 cubic tsubo.	

weight of shaft &c above line AA
16.5 @ 30200 = say 500.000#
concrete taken 140# per cubic ft.

Horizontal force due to Earthquake k = 0.2 500.000 × 0.2 = 100.000#

Concrete in base B1
17 × 12.5 × 11.6 = 2470
2470 ÷ 216 = 11.42 cubic tsubo
15.75

Concrete in base B2
12.5 × 18 = 225.0
4.0 × 5.3 = 21.2
1.5 × 2.8 = 4.2
254

Strut 7.0 × 10.0 × 16.7 = 11.70 cubic ft
or 5.41 cubic tsubo
60.17.
65.58 = Total concrete in Pier P2

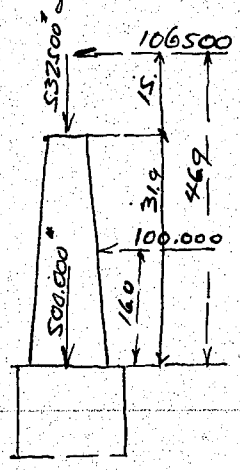
27.17
2@16.5 = 33.00
60.17 cubic tsubo

weight of Base B1 11.42 × 30200 = 345000#

weight of base B2 15.75 @ 30200 = 476000#

199.6 × 17.0 = 3400 ÷ 216 = 15.75 cubic tsubo

Superimposed load.
Dead Load say 3800# per lin ft. ½ span of both side and center span = 280.5'
Total Dead Load on pier 3800 × 280.5 = 1065000
For one truss say 532500#
Earthquake force with k=0.2 532500 × 0.2 = 106500#
Point of action application 15.0' about above top of pier.



℄ of gravity of weight of shaft assumed 16.0' above top of base. assumed.
moment at top of base.

532500	106500 × 46.9	=	5000.000
500000	100.000 × 16.0	=	1600.000
1032500			6600.000 ft#

Eccentricity = 6600.000 ÷ 1032500 = 6.4'
Unit pressure due to direct load = 1032500 / 10×13 = 7950 #/□' or 55 #/□".

Fibre stress = 6.600.000 × 6 / 10×13² = ± 23400 #/□" or 162 #/□"

Summary

	162 #/□" C	162 #/□" T
	-55	-55
	217 #/□" C	107 #/□" T

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

$t/h = 6.4/13 = .490$ $d/h = 0.016$ Steel Ratio assumed 0.0004
approximate value of neutral axis to edge $k = 0.22$ Coef = .0543 about

$$fc = \frac{6600.000}{0.0543 \times 10 \times 132} = 7200\ \#/\square' \text{ or } 500\ \#/\square''$$

$fs = 15 \times 500 (4.4 - 1) = 25400\ \#/\square''$ about

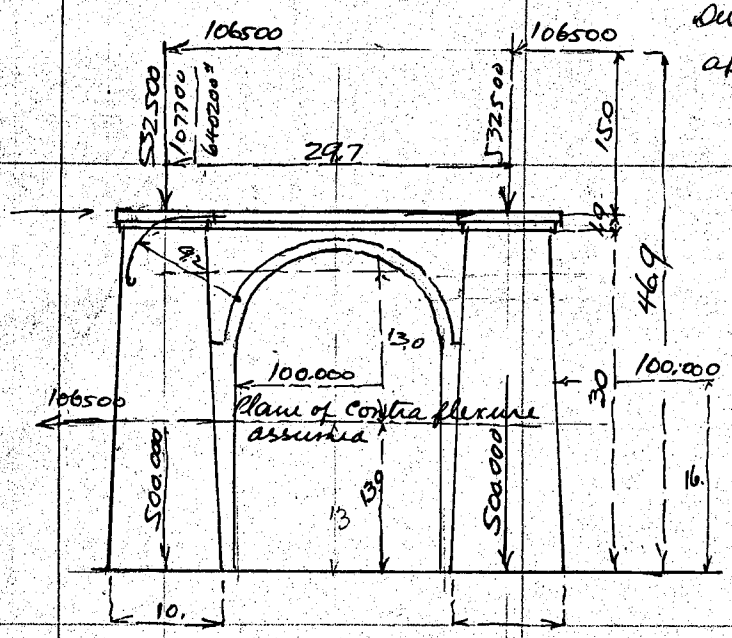
Steel Area $= 10 \times 13 \times 0.0004 = 7.85\ \square''$ or 9-3/4"ϕ = 3.96 each face of shaft.

Let us try to figure the steel area required for bending moment only neglecting direct load.

$$\text{Steel Area} = \frac{6600.000}{7/8 \times 13 \times 30600} = 19.0\ \square''$$

Use - 19 - 1"ϕ bars = 19.0 $\square''$ throughout the face.

Earthquake in transverse direction.



Superimposed load on pier. 532500
approximate reaction due to horizontal force

$$106500 \times 2 \times \frac{15}{29.7} = 107.700\ \#$$

532500	532500
+ 107700	− 107700
640200	424800 #
500000	500000
1140200	924800 #

Load at bottom due to horizontal forces.

$106500 \times 46.9 : 29.7$	=	168000
$100,000 \times \frac{16.0}{29.7}$	=	54000
		222000 #
superimposed load		532000
wt of pier.		500.000
		1,254,000 #

Point of Contraflexure assumed 13.0 above top of base.

Total horizontal force 106500
100.000
206500 #

Moment at top of base
206500 × 13 = Say 2,680.000 #
Moment at top. same as for Above.

Approximate steel area required at top of base (neglecting direct load) $= \frac{2,680.000}{7/8 \times 10 \times 30600} = 10.00\ \square''$ 10 bars 13-1" = 10.10 $\square''$

Approximate steel area required at top. $= \frac{2,680.000}{7/8 \times 9.2 \times 30600} = 10.85$ Use do.

Stability of Pier -

Moment at bottom of base B1.	106500	× 58.5	= 6230.000	
	100000	× 27.6	= 2760.000	
Base B1 11.42@30200 = 345,000 × 0.2	69000	× 5.8	= 400.000	
				9390,000 #
max load at base.	1254.000			
base B1	345.000	Σc =	9390.000 / 1,599.000 =	5.86
	1,599,000 #			

Fibre stress

$$\frac{1599000}{125 \times 17}\left(1 \pm \frac{5.86 \times 6}{17}\right) = 23800\ \#/\square' \text{ comp. } 16\ \#/\square'' \quad 13650\ \#/\square' \text{ ten. } 86\ \#/\square''$$

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CALCULATIONS FOR

Hatsuchi-Baski for Tokushima-ken

Stability of Pier
weight of base B_2.
assumed weight $12.5 \times \frac{17 \times 18}{216} = 17.7$ cubic tsubo wt $= 17.7 @ 30200 = 535000^{\#}$

For Earthquake $535000 \times 0.2 = 107.000^{\#}$ Horizontal force
Moment at bottom of Base B_2.

	$106500 \cdot$	64.9	$= 6.910.000$
	$100.000 \cdot$	34.0	$= 3400.000$
	$107.000 \cdot$	9.0	$= 960.000$
			$11,270.000^{\#}$

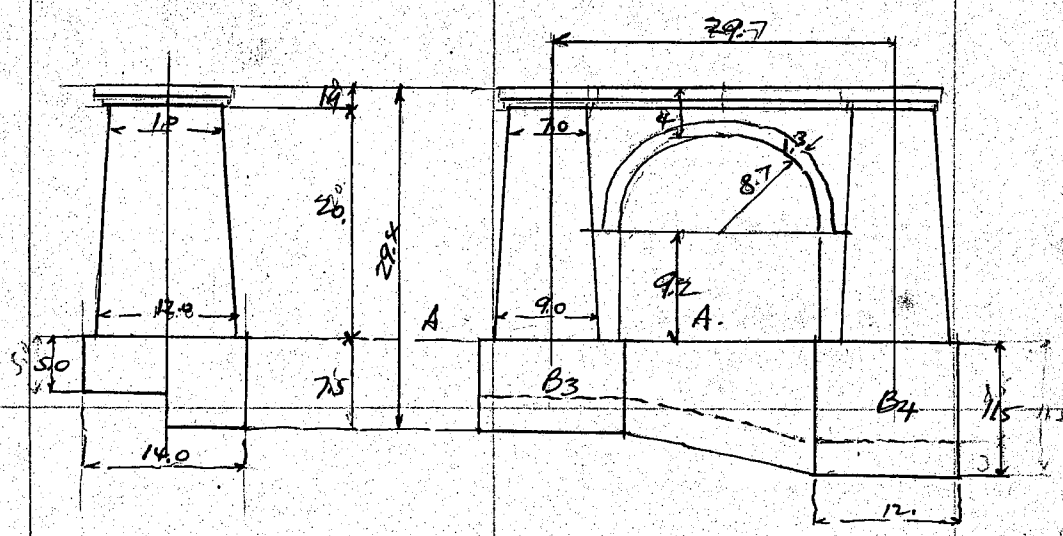
Max load on base. 1254.000
Base B_2 535.000
1789.000

$\text{Soc} = \frac{11.270.000}{1789000} = 6.31'$

Fibre stress $= \frac{1789000}{12.5 \times 17}\left(1 \pm \frac{6 \cdot 6.31}{17}\right) =$ $27100^{\#}/0'$ c $10200^{\#}/0'$ t
$188^{\#}/0''$ $71^{\#}/0''$

Use 5-1¼" anchorage at toe and heel on both ends of base

Design of Pier P_1



volume of Concrete ½ Pier.
Coping same as P_2. $= 281.0\ (ft)^3$

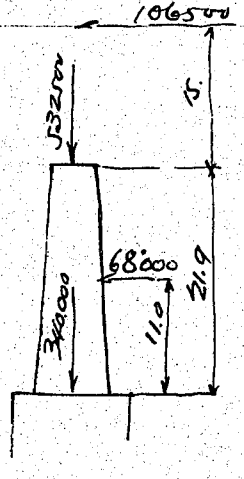
Shaft $7 \cdot 10 = 70.$
$12 \cdot 9 = 108$
$178 \div 2 = 89.0$
vol $= 89.0 \cdot 20 = 1780\ (ft)^3$
fillet. $1.88 \cdot 9.2 \cdot 4.5 = 79$
Arch 291
370

Summary of Concrete above AA.

Coping	=	281.0
Shaft	=	1780.0
Between Shafts	=	370.0
		$2431.0 \div 216 = 11.25$ cu ft

weight of pier above AA
$11.25 @ 30200 = 340.000^{\#}$

Superimposed load $532500^{\#}$



Point of application of this load assumed 15.0 above top of pier.
C of gravity of shaft assumed 11.0 above top of base.

$532500 \cdot 0.2 = 106500^{\#}$
$\frac{340000}{872.500} \times 0.2 = 68000^{\#}$

moment at top of base

	$106500 \cdot$	36.9	$= 3.930.000$
	$68000 \cdot$	11.0	$= 750.000$
			$4,680,000^{\#}$

Eccentricity $= \frac{4680.000}{872500} = 5.36$

Unit pressure due to direct load $= \frac{872500}{9 \cdot 12} = 8070^{\#}/0'$ or $56.0^{\#}/0''$

Fibre stress $= \frac{4680.000 \times 6}{9 \cdot 12^2} = 21600^{\#}/0'$ or $150^{\#}/0''$

Summary

150 C	150 T
56	56
$206^{\#}$ C	$94^{\#}/0''$ T

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Reinforcement in the shaft due to bending moment only

Steel Area = $\frac{4680.000}{\frac{7}{8} \times 8.75 \times 30600} = 19.9$

Use 13-7/8" bars = 7.80" Same as for Pz. considering direct load.

Earthquake in transverse direction.

Max load on top of base. 4680.000 ÷ 29.7 = 158000#
Superimposed load. 532500
per. 340000
1030500 #

Summary for Horizontal forces 106500
68000
174500 #

Point of Contra flexure 8' above top of base. Assumed.

Moment at bottom = 174500 × 8 = 1.400.000

Moment at top = 174500 × 8 = 1.400.000

Steel Area at top. $\frac{1.400.000}{\frac{7}{8} \times 9.2 \times 30600}$ = 5.68" Use 10-7/8"ϕ = 6.00"

Steel Area at bottom → $\frac{1.400.000}{\frac{7}{8} \times 8.75 \times 30600}$ = 5.97". Use 10-7/8"ϕ =

Stability of Pier.

assumed weight of base B4 12 × 14 × 11.5 ÷ 216 = 7.77 cubic tsubo
wt = 7.77 @ 30200 = Say 235000
Horizontal force k = 0.2 235000 × 0.2 = 47000 #

Moment at bottom of base B4

106500 × 48.4 = 5150.000
68000 × 22.5 = 1530.000
47000 × 5.75 = 270.000
6,950,000 in#

Max load at Base.

1030500
235000
1,265,500 #

Ecc = $\frac{6950.000}{1265.500}$ = 5.5'

Bearing Pressure = $\frac{1265.500}{12 \times 14}\left(1 \pm \frac{5.5 \times 6}{14}\right)$ = 25300 #/o' C or 10250 #/o' T
175 #/o" 71 #/o"

Assumed weight of base B3. 12 × 14 × 7.5 ÷ 216 = 5.84 坪 @ 30200 = 176000 #
20% = 35200 #

Moment at bottom of base B3

106500 × 44.4 = 4730.000
68000 × 18.5 = 1.260.000
35200 × 3.75 = 132.000
6,122,000 in#

max load at base

1030500
base 176000
1206500

Ecc = $\frac{6122.000}{1206500}$ = 5.07'

Bearing Pressure = $\frac{1206500}{12 \times 14}\left(1 \pm \frac{5.07 \times 6}{14}\right)$ = 22800 #/o' C 8420 #/o' T
158 #/o" 59 #/o"

Use 5-1¼"ϕ anchor bolts into rock.

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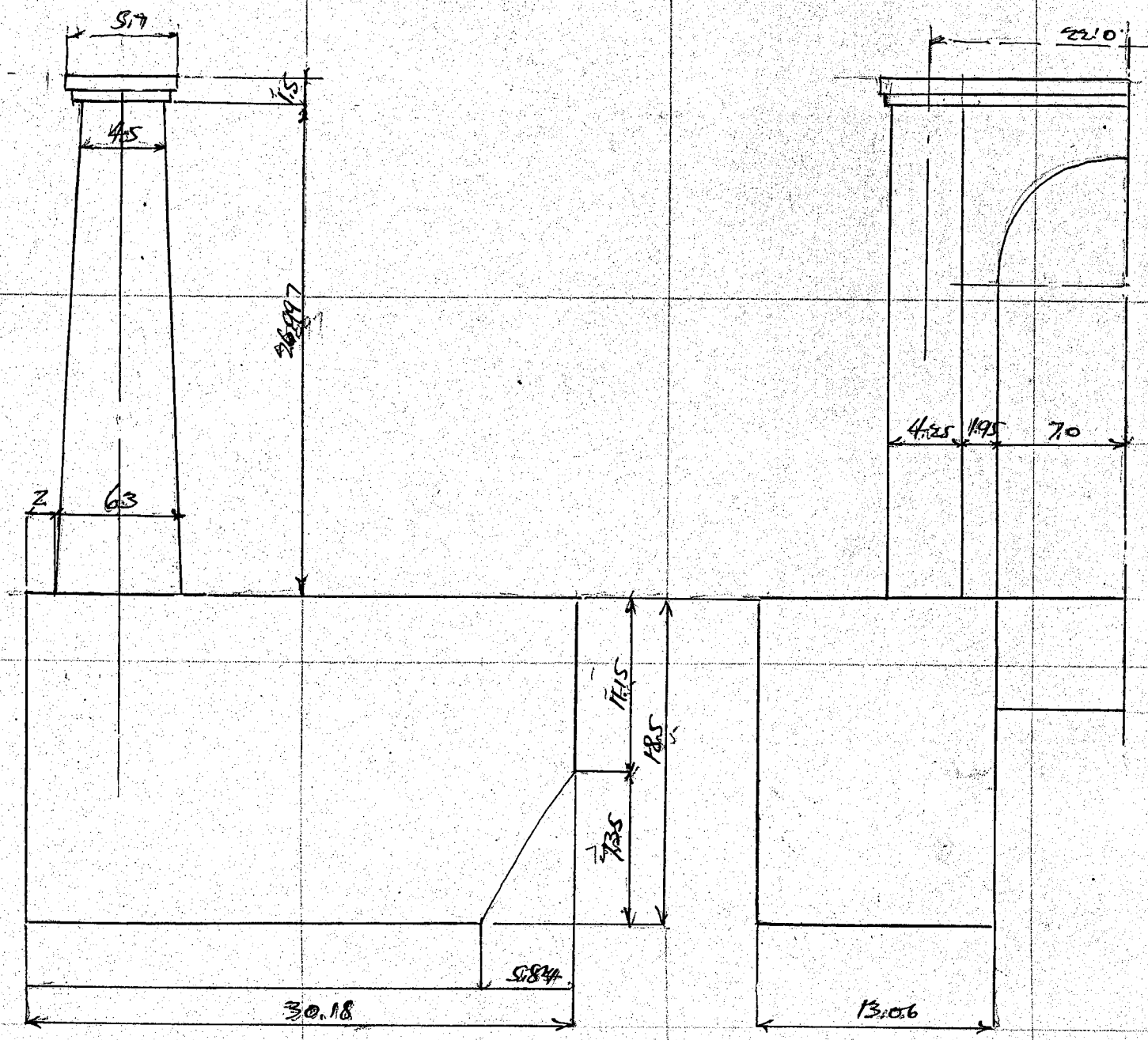
CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Design of Pier P4 carrying simple spans and Anchorage for cables.



Approximate Dead Load of pier.

Coping. 5.45 × 5.70 = 31.0
3.2 × 16.68/2 = 26.7
57.7 × 1.0 = 57.7
" 4.85 × 5.10 = 24.7
2.6 × 17.15/2 = 22.3
47.0 × 0.5 = 23.5
81.2 cubic R

shaft 4.25 × 4.50 = 19.15
4.25 × 6.30 = 26.80
45.95 ÷ 2 = 23.0 × 27 = 621.0 cubic R

web- 1.95 × 2.5 × 17.5 = 85.0
8.95 × 9.5 = 85.0
less 38.5
46.5 × 2.0 = 93.0
178.0

Base (1/2) 18.5 × 30.18 = 559.0
less. 5.84 × 7.35/2 = 21.5
537.5 × 13.06 = 7020. cubic R or 32.5 tsubo.
less 5 × 5 × 18.5 ÷ 216 = 2.14
30.36 tsubo. ÷ 2 = 60.72 ミヨヲ
weight of 1 base = 30.36 @ 30200 = 917.000 #
strut 35400
shafts + web. 123000
1075400 for 1/2 of pier.

Summary for concrete above base.- (1/2)
coping 81.2
shaft 621.0
web 178.0
880.2 cubic R
say 407 cubic tsubo

strut between bases.
6.0 × 6.0 × 7.0/216 = 1.17
407.
524
10.48 cubic tsubo.

weight of shaft and web (1/2) 4.07 × 30200 = 123,000 #
" " strut (1/2) 1.17 × 30200 = 35400 #

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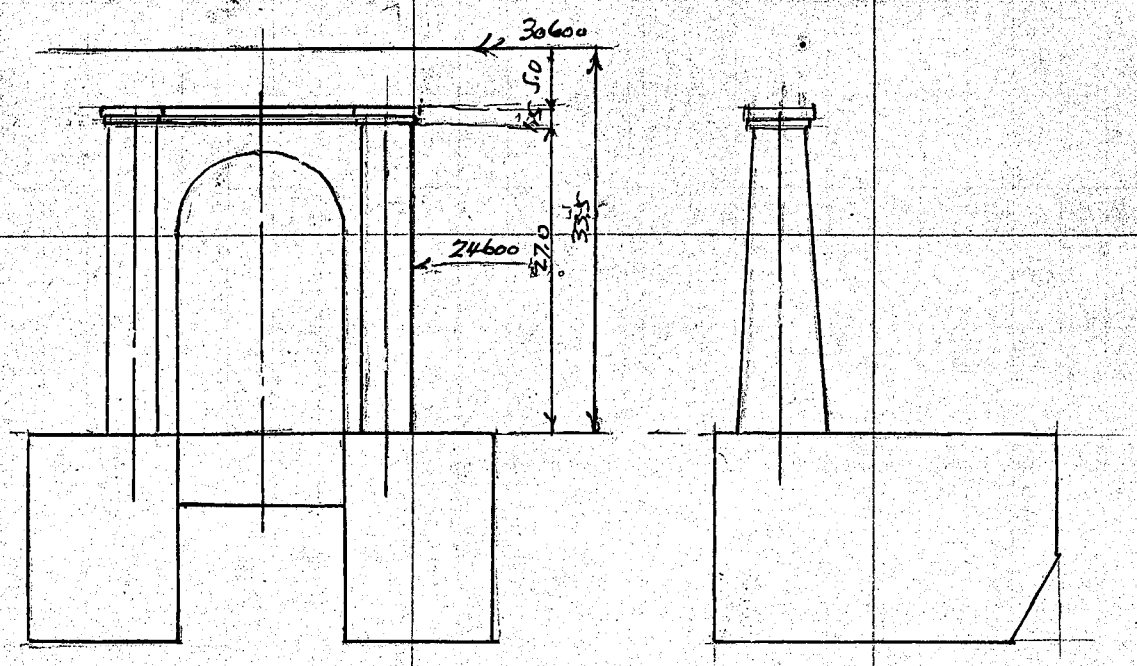
CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Assumed superimposed load.
For one truss 2905 ÷ 2 = say 1450#
Including shoe say Dload 1500# per lin. ft.
Load on pier = 1500 × 102 = 153,000# per truss on pier



Earthquake Coeff assumed 0.2. 153000 × 0.2 = 30600#
weight of shaft (½ pier) 123000 123000 × 0.2 = 24600#
Moment at top of base. 30600 × 33.5 = 1025000
24600 × 14.5 about = 357000
1,382,000 ft#

Direct load superimposed load 153000
wt of pier 123000
276000#

At longitudinal Elevation Ecc = 1382.000 / 276000 = 5.0' Area at bottom = 4.25 × 6.30 = 26.8 □'

Unit pressure $P = \frac{276000}{26.8}\left(1 \pm \frac{6 \times 5}{6.3}\right)$ = 9200 #/□' C or 38600 #/□' T
or 411 #/□" or 268 #/□"

Steel Area required for moment (direct load neglected) Unit Stress of steel = 17000 × 1.8

$$\text{Steel Area} = \frac{1382.000 \times 12}{\frac{7}{8} \times 72" \times 17000 \times 1.8} = 8.6\ \square"$$

Use 12-1"ϕ bars = @ .78 = 9.360".

Steel Area required at 17' above base.
Approximate weight of shaft say 50.000# 50000 × 0.2 = 10.000
moment 30600 × 16.5 = 505000
10000 × 5.5 = 55000
560.000 ft#

neglecting direct load Steel Area = $\frac{560.000 \times 12}{\frac{7}{8} \times 60" \times 30600} = 4.17\ \square"$

Use 6-1"ϕ bars @ .78 = 4.67 □"

Transverse view. moment at bottom From Rahmen formula the moment assumed as 2/3 and at top moment assumed as ½
moment at bottom 1382.000 × ⅔ = 921.000 ft#
" " top 1382.000 × ½ = 690.000 ft#

Steel Area at bottom $\frac{921.000 \times 12}{\frac{7}{8} \times 72 \times 30600}$ = 5.75 □" Use 8-1"ϕ bars @.78 = 6.23 □"

Steel Area at top $\frac{690.000}{\frac{7}{8} \times 7 \times 30600}$ = 3.68 □" Use 5-1"ϕ bars @.78 = 3.90 □"

For top strut use 3.680" throughout or 6-⅞"ϕ bars 3.60 □".

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CALCULATIONS FOR

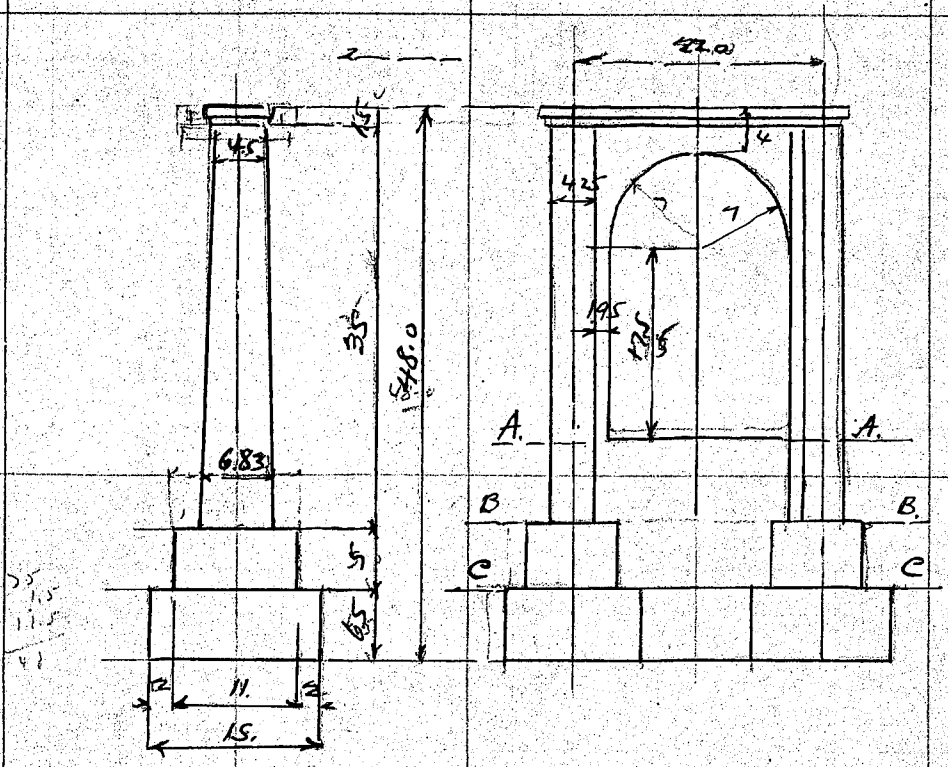
Hakuchi-Bashi for Tokushima-Ken

weight of pier - 1075400 × 2 = 2150800#
superimposed load. D.L only = 306000
2456800#

max upward pull at anchorage 2@441000 = 882,000#
max horizontal pull at " 2@832500 = 1,665,000#

Pier is heavy enough to resist the pull of the suspension cable.

Design of Pier P3



Approximate volume of Concrete in pier.

Coping 81.2 cubic ft.

Shaft 4.25 × 4.50 = 19.15
4.25 × 6.83 = 29.00
48.15 ÷ 2 = 24.07
vol = 24.07 × 35 = 843.0

web and arch above AA.
same as pier 4. = 178.0 cubic ft

web below line AA. = $\frac{17.9}{2}$ × 8 × 2.5 = 179.0
178
357 cubic.

Summary Shaft 843.0
Coping 81.2
web 357.0
1281.2 ÷ 216 = 5.95 say 6 cubic yards
877
1472
2942
1357
5067

volume of base. 5 × 11.0 × 8 = 440
6.5 × 15.0 × 12 = 1170
5.0 × 2.5 × 7.05 = 88
6 × 6.5 × 5.05 = 197
1895 ÷ 216 = 8.77 cubic yards.

1.5 × 15 × 12 = 270 ÷ 216 = 1.25 cubic yards.

Load at BB. ½ pier -
weight of pier 5.95 @ 30200 = 180.000#
superimposed Dead Load - 153.000
333.000#

seismic force.
0.2 = 36,000#
0.2 = 30600#

Moment at bottom line BB.
30600 × 41.5 = 1.270.000
36000 × 18.0 = 648.000
1,918.000 lbs

Counting upward reaction -
D.L. 66500# × 0.2 = 13300#
- moment say 13300 × 18 = 240000
1,678,000 lbs

Steel Area required = $\frac{1678.000 \times 12}{\frac{7}{8} \times 78'' \times 30600}$ = 8.15 □" use 12-1"φ bars = 9.360"

Detail reinforcement above line AA. Use same as for P4. on page 60.

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Total weight above line C-C.
528 cubic ÷ 216 = 2.44 ± ft wt 2,44 @ 30200 = 73700
From wt above BB 180.000
253700 — × 0.2 = 50720
Superimposed load 153000 × 0.2 = 30600
406700 #

Moment at line CC.
30600 × 46.5 = 1.422.000
50720 × 20.5 = 1040.000
2462.000 lft

Upward reaction 66500 pay 13300 . 23 = 306.000
2,156,000 lft

Depth assumed 10.75
Reinforcement required = $\frac{2156.000 \cdot 12}{\frac{7}{8} \cdot 10.75 \times 12 \times 30600}$ = 7.46 o" Use 10-1"ϕ bars = 7.80"

Stability of pier
weight of pier. 8.77 @ 30200 = 265000 × 0.2 = 53000
From above 180.000 × 0.2 = 36000
445000
Superimposed load. 153000 × 0.2 = 30600
598000 # − 66500 = 531500

moment about bottom of base.
30600 . 53.00 = 1.620.000
36000 . 29.50 = 1060.000
53000 . 5.00 = 265.000
2945000 lft
Less upward pull pay 13300 . 29.5 = − 393000
2552000 lft

Eccentricity = 2552000 ÷ 531500 = 4.8' area of base pay 12.15 = 180 o'

$P = \frac{531500}{180} \times \left(1 \pm \frac{6 \cdot 4.8}{15}\right) = +$ ~~291~~ 86100 #/o'c or 27100 #/o'
598 #/o" 188 #/o" tension.

Anchorage at bottom of base for moment only

Steel Area = $\frac{2.552.000}{\frac{7}{8} \times 15 \times 30600}$ = 6.33 o" Use 5-1¼"ϕ anchor bars = 6.100".

Anchorage Abutment A1

volume of Concrete 20.66 ± ft per cable.
wt = 2066 @ 30200 = 624000 #
upward pull of cable = 441.000 #
Horizontal " " " = 832.500 #
abutment built into rock 832500 − 624000 = 208500 # or 6.0 ± ft of rock required at front of abutment to resist the horizontal pull.

Anchorage Abutment A2 for stiffening truss. See pp. 46
upward pull of truss = 123300 # per truss.
2 × 123300 = 246600 #.
Concrete required 246600 ÷ 30200 = 8.15 ± ft

volume of Concrete in abutment A2 = 12.1 ± ft wt 12.1 × 30200 = 365000 # ok.

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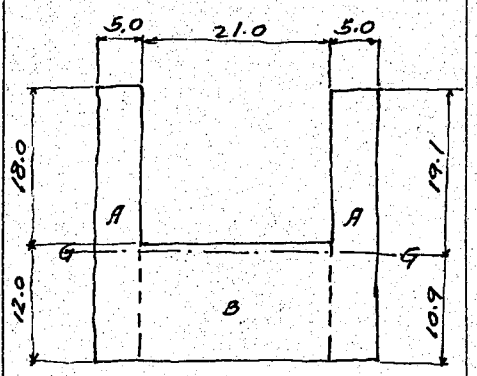
CALCULATIONS FOR

Design of Hakuchi Bashi. Tokushimaken.

Design of Abutment A3
Volume of Concrete in Abutment and base. See Drawing sheet #25

main body	A		3×24.88×26.0 =		1940
Body sloped	B		2.4×23.88×26×½ =		800
Base	C		4.0×12.0×31.0 =		1490
Base of wing	D	2@	6.0×2.0×5.0 =	2×60 =	120
Bottom of front wall	E		3.0×21.0×6.0×½ =		189
Front wall	F		30.×21.0×1.0 =		630
Corner post	P	2@	4.5×4.0×30.0 =	2×540 =	1080
Side wall	G	2@	1.5×8.5×24.5 =	2×311. =	622
Side wall buttress	H	4@	1.5×4.0×24.5 =	4×147. =	588
Pedestals	K	2@	3.0×3.0×7.0 =	2×63 =	126
Handrail posts	M	2@	1.2×1.2×3.0 =	2×4.5 =	9
Bottom horizontal struts	N	2@	1.5×3.0×22.0 =	2×99 =	198
Wall horizontal struts	Q	2@	1.5×2.0×22.0 =	2×66 =	132
Base of wing wall.	R	2@	5.0×3.0×18.0 =	2×270 =	540
					8464 ±RR or 39.2 ±17

Center of gravity of base area. G-G.



		Area		Arm (from Toe)	
A	2@	5.0×30 = 300	×	15.0 =	4500
B		12.0×21. = 252	×	6.0 =	1512
		552		10.9	6012

Moment of inertia of base area about axis G-G.

A	300 × 4.1² +	2@5×30³÷12 =	27,550
B	252 × 4.9² +	21.×12³÷12 =	9,110
			36,660 (H)4

Taking moment at Toe and finding center of gravity of vertical load, we find also the center of gravity of abutment in horizontal plane in likewise manner.

	Volume	Arm, horiz.		Arm, vertical	
A	1940	4.5	8730	16.5	32,000
B	800	2.2	1760	12.5	10,000
C	1490	6.0	8940	2.0	2,980
D	120	11.0	1320	7.0	840
E	189	8.0	1512	6.0	1,134
F	630	6.5	4095	19.0	11,970
P	1080	8.0	8640	19.0	20,500
G	622	{311 24.25	7500	19.0	6,600
		{311 14.25	4420		6,600
H	588	{294 29.25	8600	19.0	5,600
		{294 19.25	5830		
K	126	8.0	1010	37.0	4.600
M	9	{4.5 29.25	147	35.0	306
		{4.5 19.25	83		
N	198	{99 29.25	2900	8.5	1680
		{99 19.25	1930		
Q	132	{66 29.25	1910	26.0	3.440
		{66 19.25	1270		
R	540	21.0	11400	8.5	4.600
	8464		81997		118,450
			82000 say.		

$y = \frac{82.000}{8464} = 9.68.$

$y = \frac{118,450}{8464} = 14.0$

Stability of abutment

Case 1). Normal state without consideration of earth quake.

Superimposed load including live load	= 145,300# × 2	= 290,600#
Superimposed load without live load	= 94, 300# × 2	= 188,600#
Weight of abutment	= 150# × 8464	= 1,270,000#
Weight of rear fill 5.0×27.0×21×100 =		= 283,500#

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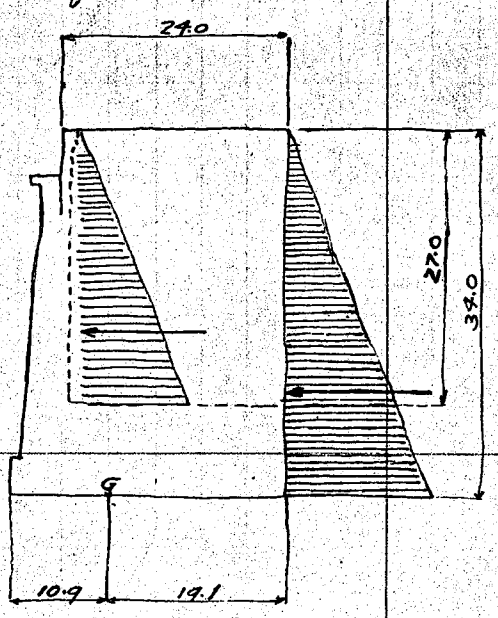
CALCULATIONS FOR

Design of Hakuchi Bashi, Tokushimaken.



Horizontal earth pressure

a) 27.0 deep. Total pressure per 1' strip $\frac{100}{6}h^2 = 12,200^{\#}$
Total pressure for 21' wide $= 256,200^{\#}$

b) 34.0 deep. Total pressure per 1' strip $\frac{100}{6}h^2 = 19,250^{\#}$
Total pressure for 2@4'-0. $154,000^{\#}$

Moment about ℄ of gravity of base area.

a). With live load

	Load		Arm		
Superimposed load	290,600	×	-6.05	=	-1,760,000
Abutment	1,270,000	×	-1.22	=	-1,548,000
Earth	283,500	×	-1.4	=	-358,700
Earth pressure b).	154,000	×	-11.3	=	-1,745,000
a).	256,200	×	-16.0	=	-4,100,000
					9,511,700

Sum of vertical force = 1844,100#

Eccentricity $= 9.511.700 \div 1,844,100 = 5.16$

Toe pressure $= \frac{1.884,100}{552} + \frac{9.511,700}{3660} = 6,170^{\#}/_{0'}$

Heel pressure $= \frac{1.844,100}{552} - \frac{9,511,700}{3660} = -1,620^{\#}/_{0'}$

Considering possitive pressure only, Toe pressure $= 6,890^{\#}/_{0'}$

b) Without live load

	Load		Arm		
Superimposed load	188,600	×	-6.05	=	-1,140,000
Abutment	1.270,000	×	-1.22	=	-1,548,000
Earth	283,500	×	-1.4	=	-358,700
Earth pressure b)	154,000	×	-11.3	=	-1,745,000
a)	256,200	×	-16.0	=	-4,100,000
					8,891,700

Sum of vertical forces = 1,742,100#

Eccentricity $= 8.891,700 \div 1,742,100 = 5.16$

Toe pressure $= \frac{1.742,100}{552} + \frac{8,891,700}{3,660} = 5800^{\#}/_{0'}$

Heel pressure $= \frac{1.742,100}{552} - \frac{8,891,700}{3.660} = 1,470^{\#}/_{0'}$

Considering possitive pressure only, Toe pressure $= 6,520^{\#}/_{0'}$

Case 2). During earth quake, weight of abutment with earth quake force $\frac{3}{4}K$, earth filling and Earth pressure with $\frac{3}{4}K$. are to be taken into consideration.

$\frac{3}{4}K = .75 \times .2 = .15$ $\beta = 14°38'$ $E = .416$

Earth pressure, rear of wing	$= \frac{1}{2} \times 100 \times 34^2 \times .416$	$= 24,000^{\#}$ per 1' strip
Total pressure 2@4.0		$= 192,000^{\#}$
Horizontal component	$= 192,000 \times \cos\beta$	$= 186,000^{\#}$
Vertical component	$= \quad " \quad \times \sin\beta$	$= 48,600^{\#}$
Earth pressure, rear of front wall	$= \frac{1}{2} \times 100 \times 27^2 \times .416$	$= 15,200^{\#}$
Horizontal component	$= 31,9,000 \times \cos\beta$	$= 309,000^{\#}$
Vertical component	$= 31,9000 \times \sin\beta$	$= 80,700^{\#}$
Total pressure 21.0		$= 319.000$

In above equation, $\sin\beta = .253$, $\cos\beta = .968$.

319,000

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CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Horizontal thrust of abutment 1.270.000 × 1.5 = 190.500 #

Moment about ℄ of gravity of base area

		Load		Arm		
Abutment	vert	1.270.000	×	−1.22	=	−1550.000
"	Hor.	190.500	×	−17.00	=	−3230.000
Earth fill		283.600	×	−1.4	=	−358 700
Earth pressure (a)	vert	48.600	×	+19.10	=	+928 000
	hor.	186.000	×	−11.30	=	−2102.000
Earth pressure (b)	vert	80.700	×	−3.15	=	−252.000
	hor	309.000	×	−16.00	=	−4945.000
						−12.437.700 ft#

Total vertical forces = 1682800 # $\quad Ecc = \frac{12437.700}{1682800} = 7.39'$

Toe Pressure $= \frac{1682800}{552} + \frac{12437700 \times 10.9}{3660} = +6740$ #/□'

Heel Pressure $= \frac{1682800}{552} - \frac{12437700 \cdot 19.1}{3660} = -3420$ #/□'

Neglecting tension max toe pressure = 10.310 #/□' or 4.60 tons/□'

Case 3 Superimposed Dead Load and body of abutment with earthquake force k=0.2

Abutment	1270.000
Superimposed load	188 600
	1458 600 #

Horizontal thrust

abutment	1270.000	× 0.2	=	254.000 #
superstructure	188 600	× 0.2	=	37.720 #

Moment about ℄ of gravity of base area

		Load		Arm		
Abutment	vert	1.270.000	×	−1.22	=	1.548.000
	hor.	254.000	×	−14.0	=	3.555.000
Superstructure	vert	188.000	×	−6.05	=	1.140.000
	hor	37.720	×	−34.00	=	1.280.000
						7.523.000 ft#

$Ecc = \frac{7523.000}{1458.600} = 5.16'$

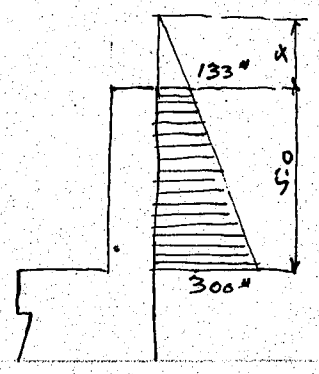
Toe Pressure $= \frac{1458600}{552} + \frac{7523.000 \times 10.9}{3660} = +4880$ #/□'

Heel " $= \frac{7523000 \times 19.1}{3660} = -1.275$ #/□'

Neglecting tension max toe Pressure = 5460 #/□ or 2.44 tons/□'

Details of Design.

Parapet wall. 1.0' thick Surcharge assumed 400 #/□'



		load	×	arm	=	
133 × 5	=	665	×	2.5	=	1660
167/2 × 5	=	418	×	1.67	=	700
		1083.				2360

Effective depth 10" Steel reqd $= \frac{2360 \times 12}{8 \times 10 \times 17000} = 0.19$ □"

Use ½"□ bars 12" centers 0.20 □" per ft.

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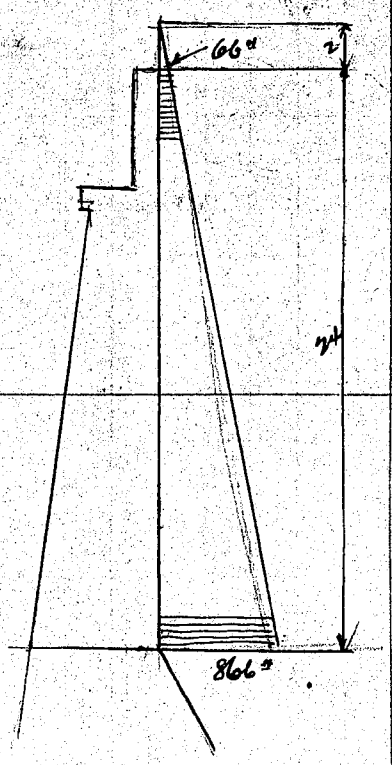
CALCULATIONS FOR

Hakuchi-Bashi for Tokushima-Ken

Section at 24' from crown of roadway



Surcharge assumed 2.0'
Horizontal moment

			moment
66 . 24	=	1580 . 12	= 19000
$\frac{800}{2}$. 24	=	9600 . 8	= 72000
		11180#	96000 ft

Depth 64"

Steel Area = $\frac{96000 \times 12}{\frac{7}{8} \times 64 \times 17000}$ = 1.21 □"

Use 7/8"ϕ bars 6" centers 1.21 □"

Section at 14' below top
moment

			moment
66 . 14	=	925 . 7.5	= 6930
$\frac{467}{2}$. 14	=	3270 . 467	= 15300
			22230 ft

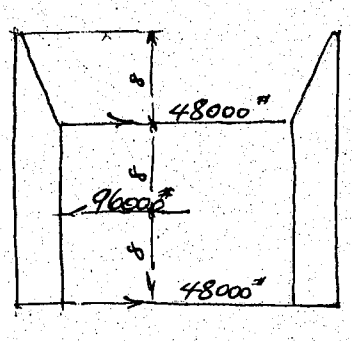
Steel Area = $\frac{22230 \times 12}{\frac{7}{8} \times 53" \times 17000}$ = .34

Use 3/4"ϕ bars 12" centers = 0.44 □"

Reinforcement in the wing wall -
Pressure at bottom 800#/0' Span length 10.0
moment at bottom = $\frac{1}{12} \times 800 \times 10^2$ = 6670 ft#
Thickness of wall. 1.5
Steel Area = $\frac{6670 \times 12}{\frac{7}{8} \times 16 \times 17000}$ = 0.336 □" ½"ϕ bars 6" centers 0.40 □"

at 14' above use ½"ϕ bars 12" centers -

Stress in Buttress wall.



Pressure 800# at bottom
Total Pressure = $\frac{800}{2} \times 24$ = 9600
for 10' strip 96000# assume unif. P = 800# per ft.
moment for buttress wall. = $\frac{1}{10} \times 8000 \times 16^2$ = 205000 ft#

Steel area = $\frac{205000 \times 12}{\frac{7}{8} \times 45" \times 17000}$ = 3.67 □" 6-7/8"ϕ bars = 5 × .6 = 3.0 □"

205000 ft# will be reduced by cantilever projection of 8'

Strut span length 24'0" section = 24" × 18"
Dead Load - 450#
fill on top say 800
1250#
moment = $\frac{1}{10} \times 1250 \times 24^2$ = 72.000 ft#

Steel Area = $\frac{72.000 \times 12}{\frac{7}{8} \times 22" \times 17000}$ = 2.624 use 5-7/8"ϕ = 3.0 on top and bottom

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CALCULATIONS FOR

Revised design of Yakuchi-Bashi Approach spans

To make main suspension span prominent compared to approach, the approach spans changed to 2-66'-3" girder spans instead of 2-102'-0" through truss spans shortening the total length of bridge.

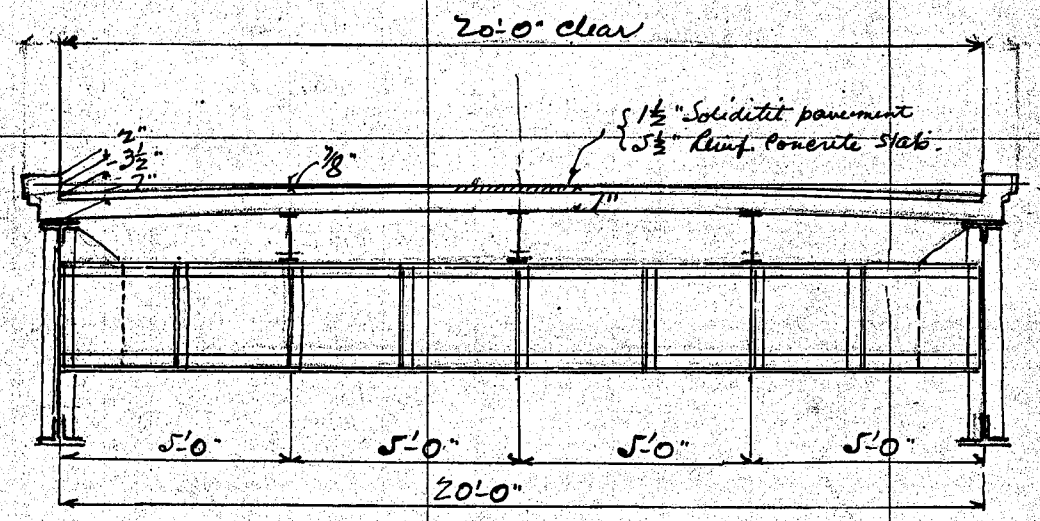
As final layout

Old pier no 3 Revised
Old pier no 2 for pier and anchorage abutment (abandoned)
New pier no 2 to be designed to carry girder spans.
Old abutment A3 (abandoned)
New abutment A3 for cable anchorages and girder abutment combined to be designed.

Assumed loadings shown on page no 1.

Allowable working strength same as shown on page no 2.

Design of 66'-3" approach span
Cross section of bridge assumed as follows.



Reinforced Concrete slab Stringer spacing 5'-0"

Dead Load

1½" Solidtit Pavement		17.5
5½" Concrete slab.		68.5
		860 per sq. ft

Dead Load moment = 1/10 · 86 · 5² = 215 1#
Dead Load shear = 86 · 2.5 = 215 #

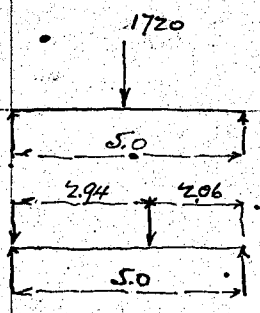
Live Load motor truck rear wheel concentration = 6600
Impact 30% 1980
8580 #

Distribution of wheel concentration

Contact between wheel and pavement = 0.66
Thickness of pavement 2 @ 1½" = 0.25
Longitudinal distribution a = 0.91'
Transv. dist. b = .89 + 0.25 = 1.14

Effective width E = ⅔ (l + b) + a
= ⅔ (5.0 + 1.14) + 0.91 = 5.01'

Load for one ft strip = 8580 ÷ 5.01 = 1720 #



Moment due to single wheel load = 860# · 2.5 = 2150 1#
For continuity of slabs m = 0.8 · 2150 = 1720 1#

Max End shear as simple beam = 1720 × 2.06/5.00 = 710
1720
2430 #

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CALCULATIONS FOR

Revised design of Gakuchi-Bashi Approach Spans.

Summary for moments and shears for slab-

	Moment	Shear
Dead Load	215	215
Live Load	1720	2430
	1935 #/ft	2645 #

Effective depth of slab for steel stress of 17000 #/□" and Concrete stress of 840 #/□"

$d = \sqrt{\frac{1935}{102}} = 4.35"$

Use ¾" insulation at bottom and ½" at top with Solidtit Pavement on top of slab. make total thickness of slab 5½" over all.

Steel area reqd $= \frac{1935 \cdot 12}{\frac{7}{8} \cdot 4.25 \cdot 17000} = 0.358$ □" per ft strip

Use ½"□ bars 6" centers = 0.390 □"

Unit shear at end $= \frac{2645\#}{\frac{7}{8} \cdot 4.25 \cdot 12} = 59$ #/□"

Bond stress $= \frac{2645}{\frac{7}{8} \cdot 4.25} = 710\#$

Use 2- ½"□ @ 1.57" · 130 = 408
2- ⅜" @ 1.18 · 130 = 306
714 #/□" ok for bond.

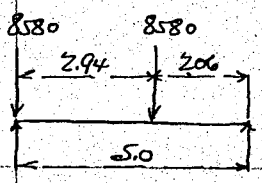
Design of Longitudinal Stringer stringer spacing 5.0' span length = 12.75'

Dead Load

Slab and pavement 86 · 5.0 = 430
Stringer assumed 45
475 # per lin ft

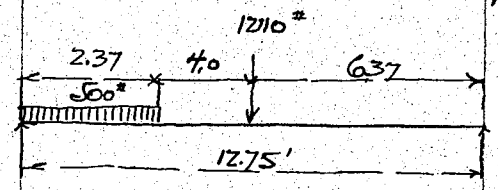
Dead Load moment $= \frac{1}{8} \cdot 475 \cdot 12.75^2 = 9650$ #'
Dead Load shear $= 475 \cdot 12.75 \div 2 = 3030\#$

Live Load motor truck loading Rear wheel concentration with impact = 8580 #
Front " " " " = 2680 #



Rear wheel reaction $= 8580 \cdot \frac{2.06}{5.00} = 3530$
8580
12110 #

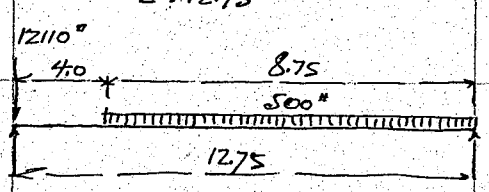
Uniform load = 100 · 5 = 500 # per lin ft.



Moment due to wheel load $= \frac{12110}{2} \cdot 6.37 = 38500$ #'
" " " Uniform load $110 \cdot 6.37 = 700$
39200 #'

$R = \frac{500 \cdot 2.37^2}{2 \cdot 12.75} = 110\#$

Max End shear
Uniform load $\frac{500 \cdot 8.75^2}{2 \cdot 12.75} = 1500\#$



Motor truck loading 12110
13610 #

Summary for moments and shears

	Moment	Shear
Dead Load	9650	3030
Live Load	39200	13610
	48850 #'	16640 #

Section modulus reqd $= \frac{48850 \times 12}{17000} = 34.5$ Use 12", 5"I · 31.99 □" Om = 36.6□"

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CALCULATIONS FOR

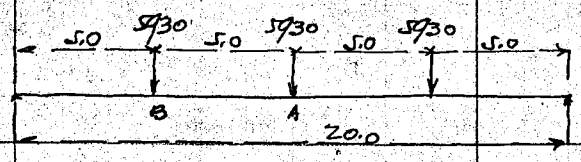
Revised Design of Hakuchi-Bashi Approach Spans

Intermediate Floor Beam span length 20.0' spacing 12'-9"

Dead Load

Slab and Pavement 86# · 5.0 = 430
stringer assumed 35
465# per lin ft

Concentration at stringer = 465 · 12.75 = 5930#



moment

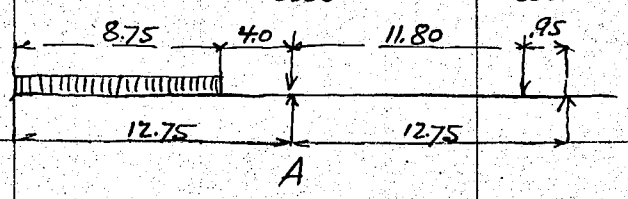
8895 · 10.0 = 88950
5930 · 5.0 = 29650
59300 1#

beam say $\frac{1}{8}$ · 85 · 20² = 4250
63550#

5930
2965
8895

Dead Load shear Reaction 8895
beam 85×10 = 850
9745#

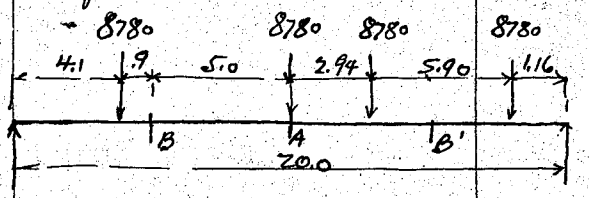
Live Load motor truck loading



Rear wheel conc. with impact = 8580#
Front " " " " = 2680

Reaction A.
wheel load 2680 · $\frac{.95}{12.75}$ = 200
8580
8780

Uniform load 100#/ft' Reaction = $\frac{100 \times 8.75^2}{2 \times 12.75}$ = 300# per lin ft.



Reaction 7200
6200
13400#

Concentration A 8780 · $\frac{2.06}{5.00}$ = 3620
8780
12400#

Concentration B' 8780 − 3620 = 5160
8780 · $\frac{1.16}{5.00}$ = 2040
7200#

Concentration B 8780 · $\frac{5.00}{5.00}$ = 7200#

Moment at A. 13400 · 10.0 = 134000
7200 · 5.0 = 36000
98000 1#

moment due to uniform load 300 · 5 = 1500# at stringer connection.
moment at Center 2250 · 10.0 = 22500
1500 · 5.0 = 7500
15000 1#

Live load moment motor truck loading 98000 1#
Uniform load 15000 1#
Total moment 113000 1#

max End shear 13400# for motor truck loading,
2250 " uniform load
Total. 15650#

Summary for moments and shears

	moment	Shear
Dead Load	63550	9745
Live Load	113000	15650
	176550 1#	25395#

web assumed 25" · 5/16 = 7.820" ½ web = .98 0" Effective depth = 1.98'
Stress = 176550 ÷ 1.98 = 89200# section req'd 89200 ÷ 17000 = 5.25 0"
½ web .98
4.27 0" net

Use 2 L 4.3 · 3/8 = 4.96 − 0.66 = 4.30 0" net.

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CALCULATIONS FOR

Revised Design of Hakuchi-Bashi Approach span

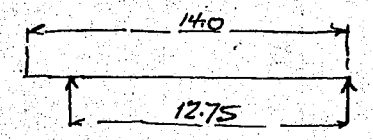
Approximate weight of Intermediate Floor Beam

2 flange Ls	4. 3 × 3/8	@	8.5#	× 19.00	=	323
"	4. 3 × 3/8	@	8.5	× 19' 11½"	=	339
web Pl	1 Pl 25" × 5/16	@	26.56	× 16' 6½	=	439
"	2 Pls 19¾ × 5/16	@	20.98	× 2' 9"	=	115
Stiff	14 Ls 3. 3. × 5/16	@	6.10	× 2' 1½	=	181
End Conn	2 Ls 5. 3½ × 3/8	@	10.4	× 2' 0¾	=	43
fills	2 Pls 5. × 3/8	@	6.38	× 1' 7¼	=	20
Splice	4 Pls 5½ × 5/16	@	5.84	× 1' 7¼"	=	37
						1497
			River heads and variations			53
						1550#

1550 ÷ 20 = 77.5# per lin ft of girder.

End Floor Beam span length 20.0

Dead load



Concentration at stringers = $\frac{465^{\#} \times 14.0^2}{2 \times 12.75}$ = 3570#

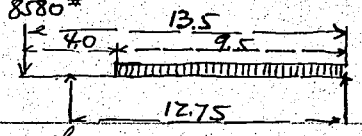
Dead Load moment

3570
1785
5355 × 10 = 53550
3570 × 5 = 17850
35700 1#

Dead Load girder say $\frac{1}{8} \cdot 85 \cdot 20^2$ = 4250
39950 1#

Dead Load shear = stringer concentration 5355
Floor beam 85 × 10 = 850
6205#

Live Load motor truck loading Rear wheel $8580 \times \frac{13.5}{12.75}$ = 9100#



Uniform load 100#/' stringer Conc = $\frac{5 \times 100 \times 9.5^2}{2 \times 12.75}$ = 1770#

Live Load motor truck loading moment = $98000 \times \frac{9100}{8780}$ = 101500 1#

" " Uniform load " = $15000 \times \frac{1770}{1500}$ = 17700
119200 1#

max End shear motor truck loading shear $13400 \times \frac{9100}{8780}$ = 13900

Uniform Live Load " 2655
16555#

Summary for moments and shears

	Moment	Shear
Dead Load	39950	6205
Live Load	119200	13900
	159150 1#	20105#

web assumed 25" × 5/16 = 7.82" ⅛ web = 0.98" Effective depth = 1.98'
stress = 159150 ÷ 1.98 = 80500# section rqd = 80500 ÷ 17000 = 4.73
0.98
3.75" net

2 Ls 4. 3. × 5/16 = 4.18 - 0.54 = 3.64" net
Use 2 Ls 4. 3. × 5/16 = 4.96 - 0.66 = 4.30" net

Approximate weight of End Floor Beam same as for intermediate F.B.

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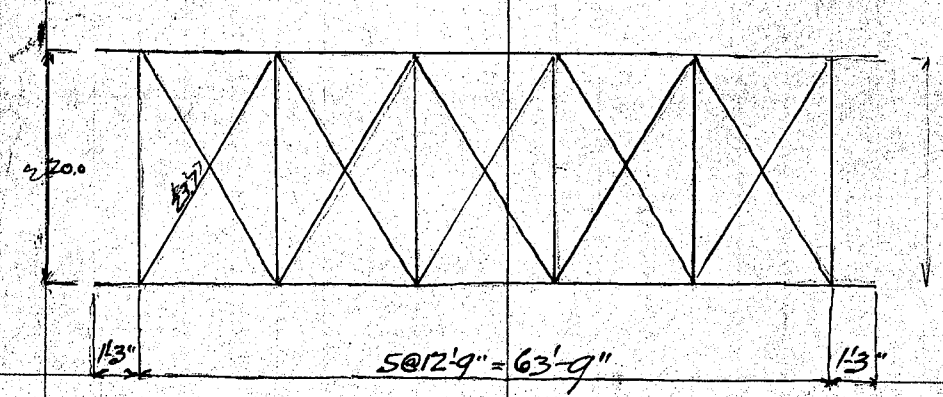
CALCULATIONS FOR

Revised Design of Hakuchi-Bashi Approach spans

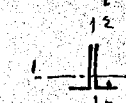
Lateral Bracing



Wind pressure assumed 30#/0'
Exposed area say 8.0' a 8.0 · 30 = 240#/
panel conc = 240 · 12.75 = 3060#
Reaction = 2 · 3060 = 6120#

$\sec\theta = \frac{23.70}{20.0} = 1.185$

Stress = 6120 · 1.185 = 7250#
Section req'd = 7250 ÷ 17000 = 0.427 □" net



r = 1.61

Use 2L 5 · 3 · 5/16 = 4.80 □" gross or 4.14 □" net
Unsupported length = 23.7 × 12 = 284"
l/r = 284 ÷ 1.61 = 177 in 1-1 axis.
Stiffeness in 2-2 axis = Unsupported length 142" r = 1.09 l/r = 142 ÷ 1.09 = 130.

Approximate weight of bracing
4L 5 · 3 · 5/16 @ 8.2# · 23.5 = 770
Plate Connection 50
Connection at panel 4@25 = 100
920#

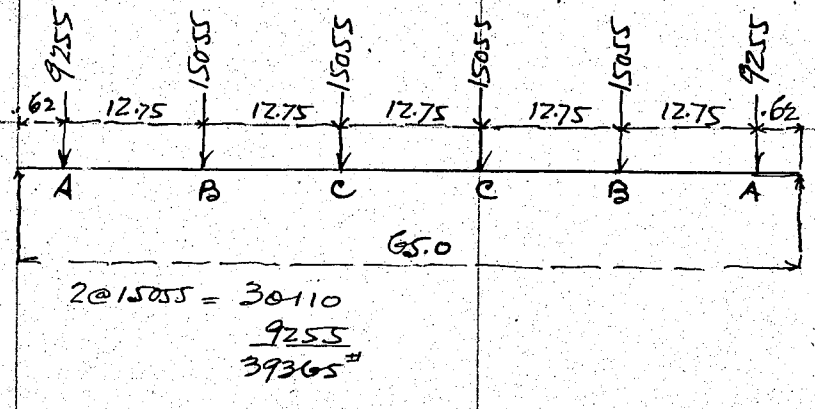
920# ÷ 12.75 = 72# per lin ft.

Design of Main Girder
Dead Load Floor slab 86# · 20 = 1720
Coping .75 @ 150# = 113#
Handrail say 60
173 × 2 = 346
2066# per lin ft of span

Metal in floor system and Floor Concentration
Intermediate panel point
Stringers 3@35# · 12.75 = 1340
Floor beam 1550
Lateral Bracing 920
3810
Floor slab &c 2066 · 12.75 = 26300
30110#
On one girder 15055#

End Panel Point
Stringers 3@35# · 7.62 = 800
Floor Beam 1550
Lateral Bracing 460
2810
Floor slab 2066 · 7.62 = 15700
18510
On one girder 9255#

Dead Load Moment due to panel Concentrations.



2@15055 = 30110
9255
39365#

Moment at A 39365 · 0.62 = 24400 ##

Moment at B 39365 · 13.37 = 525.000
9255 · 12.75 = 118.000
407000 ##

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CALCULATIONS FOR

Revised Design of Hakuchi-Bashi Approach Spans

Moment at C

39365 × 26.12 = 1,030.000
15055 × 12.75 = 192.000
9255 × 25.50 = 236.000

428.000
602.000 #

weight of main girder assumed 270# per lin ft

moment at A $\frac{270}{2}$ × 0.62 × 64.38 = 5400 #
" " B " × 13.37 × 51.63 = 93300
" C " × 26.12 × 38.88 = 137000
" Centre $\frac{1}{8}$ × 270 × 65.0² = 142500

Summary for dead load moments.

	A	B	C	Centre
Concentration	24400	407000	602000	602000
Own weight of girder	5400	93300	137000	142500
	29800	500300	739000	744500

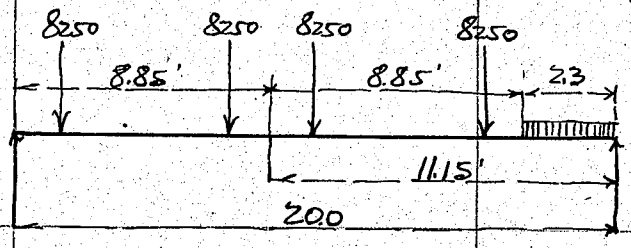
End shear Panel Concentration 39365
Girder 270.33 = 8900
48265 #

Live Load motor truck loading

Impact = $\frac{20}{60+19.8}$ = 25%

Rear wheel 6600
impact 25% 1650
8250 #

Front wheel $\frac{1}{3}$ · 8250 = 2750 #



motor truck reaction
4 × 8250 × $\frac{11.15}{20.0}$ = 18400 #

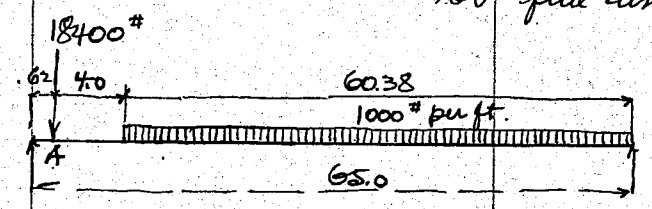
Front wheel 18400 · $\frac{1}{3}$ = 6140 #

Uniform live load say 100 #/0'
Reaction = $\frac{100 \times 2.3^2}{2 \times 20.}$ = 13 # per lin ft.

Neglect this uniform load on side of motor truck

For full uniform load 1000 # per lin ft on main girder.

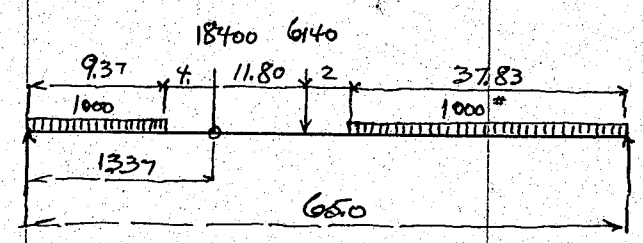
moment at A



Reaction Unif. load $\frac{1000 \times 60.38^2}{2 \times 65.0}$ = 28700 #
Motor truck 18400 · $\frac{64.38}{65.00}$ = 18200
46900

moment = 46900 × 0.62 = 29000 1#

moment at B.



Reactions
motor truck 18400 × $\frac{51.63}{65.0}$ = 14600
6140 · $\frac{39.83}{65.0}$ = 3760
18360 #
Unif. load 37830 × $\frac{18.92}{65.0}$ = 11000
9370 × $\frac{60.31}{65.0}$ = 8700
19700
Total 38060 #

moment motor truck 18360 · 13.37 = 245000
Unif. load 19700 · 13.37 = 263000
9370 · 6.69 = 62.500
200500
Total m 445500 1#

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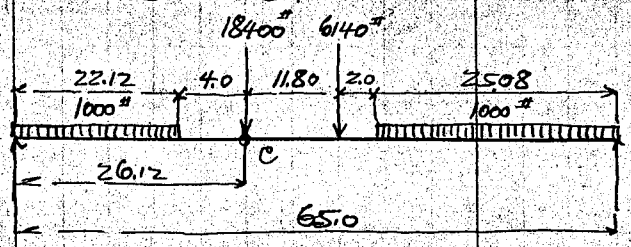
CALCULATIONS FOR

Revised Design of Hakuchi-Bashi Approach spans

Moment at C.



Reactions

motor truck	18400 ×	38.88/65.0	=	11000	
	6140 ×	27.08/65.0	=	2560	13560#
Unif. load	25080 ×	12.54/65.0	=	4840	
	22120 ×	53.94/65.0	=	18350	23190#

Moment				
	motor truck	13560 × 26.12	=	354000
	Unif. load	23190 × 26.12	=	605.000
		22120 × 15.09	=	333.000
				272.000
				626000 lb#

Summary for moments

	A	B	C	Centre
Dead Load	29800	500300	739000	744500
Live Load -	29000	445500	626000	say 626000
	58800	945800	1365000	1370500 lb#

End shear

Dead Load shear		48265
Live Load shear		46900
		95165 #

Web assumed 56" × 3/8" = 21.00" ⅛ web = 2.62 back to back of Ls = 4'8½"
Effective depth say 4.54' Stress = 1370500 ÷ 4.54 = 303000
Section reqd = 303000 ÷ 17000 = 17.80
2.62
15.18 0" net

Use 2 Ls 6×6×½	=	11.50 qn	9.50"
1 Pl. 13½ × ½	=	6.35	5.75
		17.85"	15.25" net

Approximate weight of main girder

web	1 Pl	56" × 3/8	@	71.4	× 66.25 =	4730
flange	4 Ls	6×6×½	@	19.6	× 66.25 =	5200
Cov. Pl	2 Pls.	13½ × ½	@	22.95	× 40.00 =	1840
Int Stiffs	28 Ls	5×3½×⅜	@	10.4	× 4.7 =	1370
End Stiffs	8 Ls	5×3½×½	@	13.6	× 4.7 =	512
fills -	8 Pls.	3½ × ½	@	6.0	× 3.7 =	177
"	8 Pls	3½ × ½	@	6.0	× 3.7 =	177
Splices and other misc details						2500
						16506#

call this 16500#

16500 ÷ 66.25 = 250# per lin ft.

Approximate weight of steel in one span

Stringers	105 × 66.25	= 6950
Floor Beams	6 × 1550	= 9300
Bracing	5 × 920	= 4600
Girders		= 33000
		53850 # or 24.0 tons per span

For 2 spans @ 24.0 = 48.0 tons. of structural steel.

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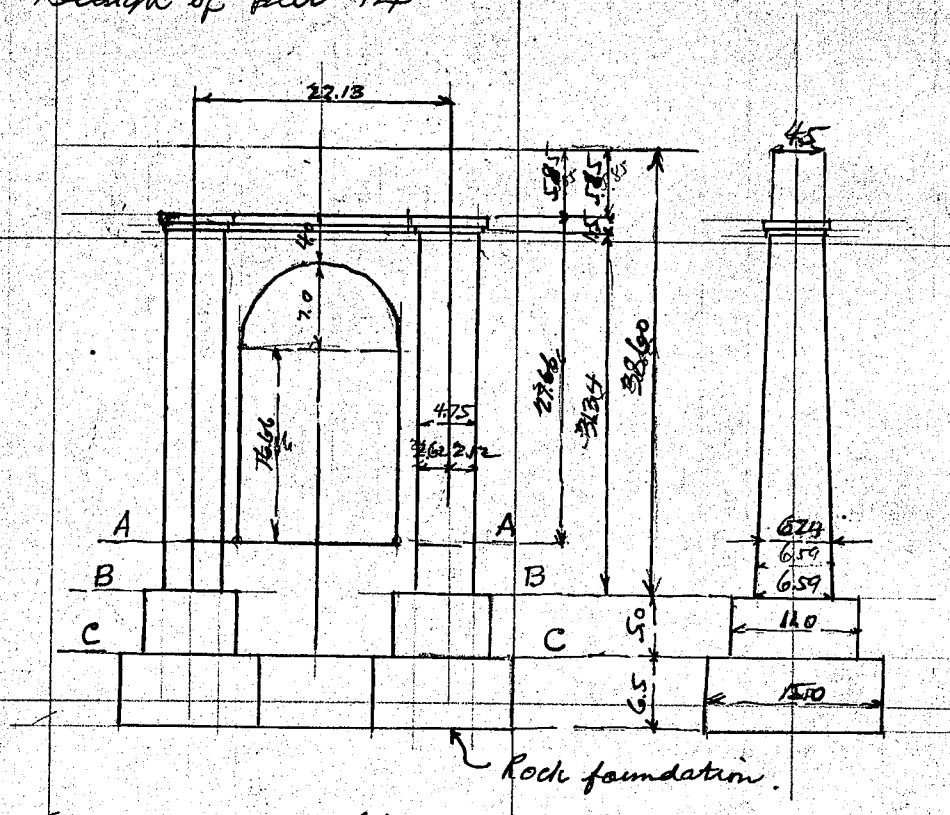
CALCULATIONS FOR

Revised Design of Hakuchi-Bashi Approach Spans

max load on bearing
max reaction say same as End shear 95165 # Design for 96000 #
Bearing area = 15" x 20 = 300 □" Unit bearing = 96000 ÷ 300 = 320 #/□" ok.

Design of pier P4



Approximate volume of Concrete in Pier.
Coping (½) 85.0 cubic R

Shaft 4.75 × 4.50 = 21.4
4.75 × 6.24 = 29.6
51.0 ÷ 2 = 25.50 av
volume = 25.50 × 26.16 = 667.0 cubic R
web and arch above line AA.
16.90 × 9.5 = 160.5
100.5
60. × 2.0 = 120.0
Arch 100.5
77.0
23.5 × 2.5 = 58.7
2.9 × 16.66 × 2.5 = 121.0
299.7
For ½ say 150 cubic R

Summary above line AA
Coping 85
Shaft 667.
web say 150.
902 ÷ 216 = 4.17 cubic tsubo wt = 902 × 140 # = 126000 #

volume of Concrete between AA and BB.
Shaft 6.24 × 4.75 = 29.60
6.59 × 4.75 = 31.25
60.85 ÷ 2 = 30.42 × 5.18 = 158.0
web ½ 8.45 × 5.18 × 2.5 = 109.5
267.5 ÷ 216 = 1.24 cubic tsubo
weight of Concrete 267.5 @ 140 = 37400 #

volume of Concrete between BB and CC (½)
8.5 × 11.0 × 5 = 467.5
5.18 × 6.56 × 2.5 = 85.0
552.5 ÷ 216 = 2.56 cubic tsubo
weight of Concrete 552.5 @140 = 77500 #

volume of Concrete in base. (½)
12.5 × 15.0 × 6.5 = 1220.0
6.0 × 6.5 × 4.56 = 178.0
1398.0 ÷ 216 = 6.47 cubic tsubo
weight = 1398 × 140 = 1960.00 #

Summary of Concrete in pier.
above line AA 4.17
Between AA and BB 1.24
" BB and CC 2.56
In base 6.47
14.44 × 2 = 28.88 cubic tsubo

Superimposed load on one shaft Dead Load say 49000 #
For two bearings 96000 #

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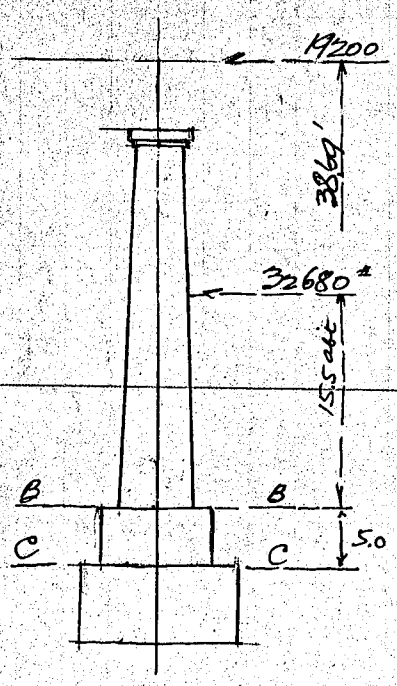
CALCULATIONS FOR

Revised Design of Hakuchi-Bashi Approach Spans

Reinforcement in the shaft (Longtudinal Direction)



Seismic Coff assumed 0.2

Superimposed load 96000 × 0.2 = 19200#

Shaft and Copings

above line AA 126000

AA and BB 37400

163400 × 0.2 = 32680#

Total load 96000 + 163400 = 259400#

Moment about line BB.

19200 × 38.69 = 743000 ft#

32680 × 15.5 abt = 507000

1350.000 ft#

At longitudinal Elevation

$Eoc = \frac{1350.000}{259400} = 5.2'$

Area at BB 4.75 × 6.59 = 31.3 □'

Unit Pressure $= \frac{259400}{31.3}\left(1 \pm \frac{6 \times 5.2}{6.59}\right) = 4760\ ^{\#}/\square'\ ^{C}$ or $3100\ ^{\#}/\square'\ T$

or $330\ ^{\#}/\square''$ or $215\ ^{\#}/\square''\ T$

Steel Area required for moment (direct load neglected) Unit stress of steel = 17000 × 1.8

Steel Area $= \frac{1350.000 \times 12}{\frac{7}{8} \cdot 75 \cdot 17000 \cdot 1.8} = 8.05\ \square''$

Use 12-1"□ bars @.78 = 9.36 □". Same as for Pier #3.

Reinforcement in 12' below top of coping approximate wt of shaft say 60.000# about

moment 19200 × 17.85 = 342000

12000 × 6.0 = 72000

414000 ft#

neglecting direct load $\frac{414000 \times 12}{\frac{7}{8} \cdot 60 \cdot 30600^{\#}} = 3.10\ \square''$ required

Transverse view From Rahmen formula the moment at bottom of shaft (say AA) assumed as 2/3 and moment at top assumed 1/2

moment about line BA. Load above line AA = 126000#

Seismic force = 126000 × 0.2 = 25200#

moment 19200 × 35.51 = 682.000

25200 × 15.0 abt = 378.000

1060.000 ft#

moment at AA 1060.000 × 2/3 = 707000 ft#

moment at top 1060.000 × 1/2 = 530.000 ft#

Reinforcement used at bottom 8-1"□ @.78 = 6.23 □"

" " " top 6-1"□ @.78 = 4.68 □"

Unit stress in steel bottom $= \frac{707000 \times 12}{\frac{7}{8} \cdot 72 \cdot 6.23} = 2060\ ^{\#}/\square''$

Unit stress in steel top $= \frac{530.000 \times 12}{\frac{7}{8} \cdot 7 \cdot 12 \cdot 4.68} = 1850\ ^{\#}/\square''$

Use 6-7/8"□ bars throughout the strut top and bottom area = 3.600".

Reinforcement in line CC.

weight of Concrete between lines BB and CC 77500.# Seismic f = 77500 × 0.2 = 15500#

moment at line CC

19200 × 43.69 = 838000

32680 × 20.50 = 670000

15500 × 2.5 = 39.000

1547.000 ft#

neglecting direct load and the reinforcements required for moment only are :—

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CALCULATIONS FOR

Revised Design of Hakuchi-Bashi Approach Spans

Depth of beam assumed 10.5' Stul = $\frac{1547.000}{\frac{7}{8} \cdot 10.5 \cdot 30600} = 5.5$ Use 10-1" @ .78 = 7.80"

Stability of pier.

Weight of Concrete in base 196,000 # Seismic force = 196000 · .02 = 39200 #

Moment at bottom of base

				load
19200	·	50.19	= 962000	96000 #
32680	·	27.00	= 882000	126000
15500	·	9.00	= 139,000	77500
39200	·	3.25	= 127,000	196000
			2,110.000 ft#	495500 #

e_{oc} = 2110.000 ÷ 495500 = 4.26'

Area of base = 12.5 · 15.0 = 187.50'

$P = \frac{495500}{187.5}\left(1 \pm \frac{6 \times 4.26}{15.}\right)$ = 1500 #/□' C or 1800 #/□' T
496 #/□" or 129 #/□"

Anchorage at bottom of base for moment only

Stul Area = $\frac{2110.000}{\frac{7}{8} \times 14.5 \times 30600} = 5.42$□" Use 5-1¼"ϕ anchor bars = 6.100□"
Anchored into rock.

Design of Abutment and Cable anchorage A3.

Cable Anchorage Proper including foundation at front of abutment

Concrete	1:3:6	64.33
Concrete	1:4:8	14.66
		78.99 ÷ 2 = 39.50 cubic tsubo.

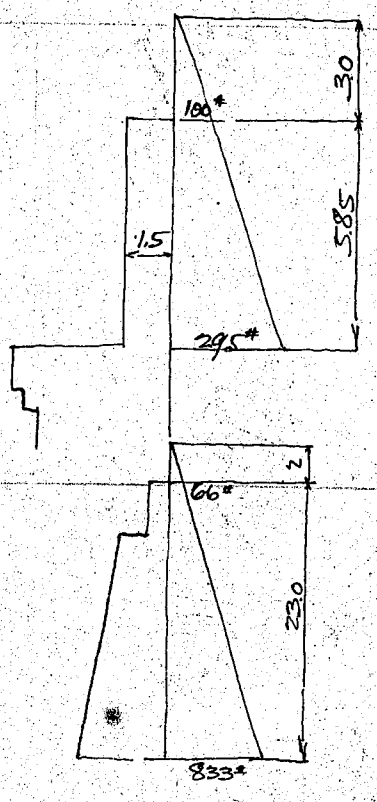
Approximate weight 39.50 · 3020 # = 1192.000 # per cable.
Upward pull of one cable = 441.000 #
Horizontal pull of " " = 832.500 #

Anchorage block is heavy enough to carry the horizontal pull of cable, Investigating rock formation, & the Concrete block at rear to be made smaller by judgement of Field Engineer to resist the pull of Cable.

Abutment proper is built on solid rock foundation and Combined with cable anchorage there is no need to test stability of the solid abutment

Let us design the reinforcement in parts of abutment

Parapet wall.



				moment
100 · 5.85	=	585 #	· 2.92	= 1710
195/2 · 585	=	570	· 1.95	= 1110
				2820 ft#

Effective depth 16" Stul Area = $\frac{2820 \times 12}{\frac{7}{8} \cdot 16 \cdot 17000} = 0.14$□"

Use ½"ϕ bars 12" centers 0.20□" per ft

				moment
66 · 23	=	1520	· 11.5	= 17500
767/2 · 23	=	8820	· 7.67	= 67500
		10340		85000 ft#

Depth 6.0'

Stul Area = $\frac{85000 \times 12}{\frac{7}{8} \cdot 60 \cdot 17000} = 1.14$

Use ⅞"ϕ bars 6" centers 1.21□" per ft

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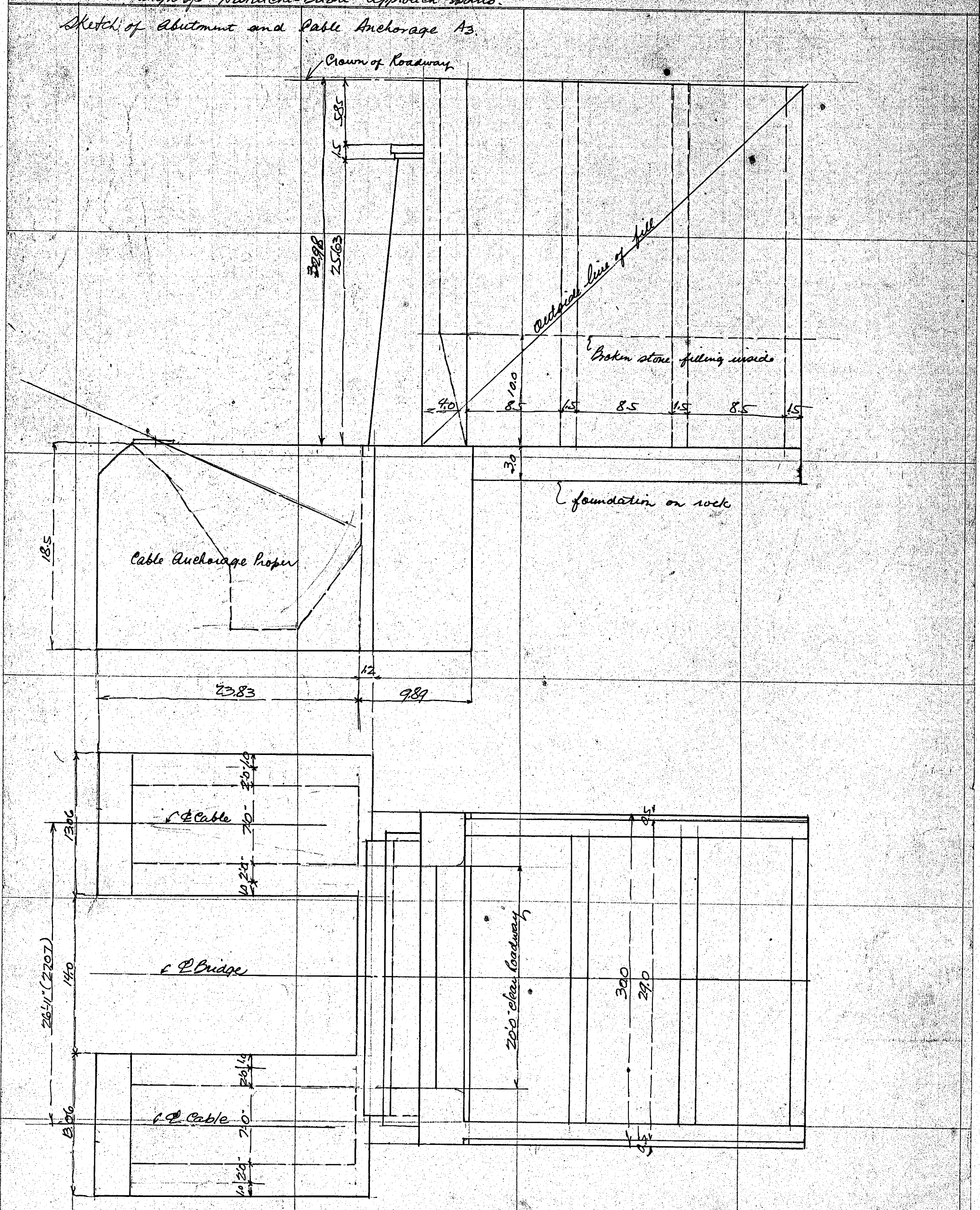
CALCULATIONS FOR

Revised Design of Takuchi-Bashi Approach Spans.

Sketch of Abutment and Cable Anchorage A3.



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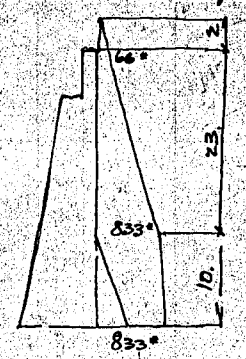
CALCULATIONS FOR

Revised Design of Hakuchi-Bashi Approach Spans.



Moment at 33' below top
10' at bottom assumed same horizontal pressure as for 23' except using broken stone filling.

			arm		moment
66 . 23	=	1520	. 21.5	=	32700
$\frac{76}{2}$. 23	=	8820	. 17.67	=	156000
833 . 10	=	8330	. 5.00	=	41700
		18670			230400 #'

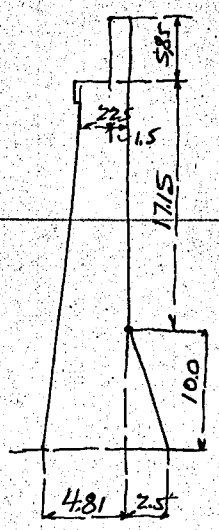
Depth 9.4' = 110"

$$\text{Steel area} = \frac{230400 \cdot 12}{\frac{7}{8} \cdot 110 \cdot 17000} = 1.69 \square''$$

Rear Reinforcement :- Use 7/8" bars 6" centers = 1.710" assuming the part of shear carried by horizontal beam.

Front Reinforcement in shaft of abutment

Superimposed Dead Load from girder span
2 @ 48800 = 97600 #
For one ft strip 97600 ÷ 25 = 3900 # per ft



Weight of shaft one ft strip

				arm		moment
parapet 5.85 . 1.5	=	8.8	@150# =	1320	. 30.23	= 40000
3.95 . 27.15	=	107.0	" =	15300	. 13.57	= 207000
$\frac{2.56}{2}$. 25.63	=	32.8	" =	4920	. 8.55	= 42000
2.5 . $\frac{10}{2}$	=	12.5	" =	1880	. 3.33	= 6200
				23420		295200

As seismic moment
295200 . 0.2 say 59000
Superimposed load 0.2 . 3900 . 3.30 = 25700
84700 #

Effective depth say 110"

$$\text{steel area} = \frac{84700 \cdot 12}{\frac{7}{8} \cdot 110 \cdot 17000 \cdot 1.8} = 0.345 \square''$$

Use 5/8" bars 1.0 spacing = 0.310" for bending only

$$\text{Fibre stress} = \frac{6 \cdot 84700 \cdot 12}{12 \cdot 110^2} = \pm 70.5 \#/\square''$$

$$\text{vertical pressure} = \frac{23420}{110 \times 12} = 18 \#/\square'' \qquad 70.5 - 18.0 = 52.5 \#/\square'' \text{ tension.}$$

Reinforcement in wing wall.
23.0 below top Pressure 833 # per sq ft span length 10.0'

$$m = \frac{1}{12} \cdot 833 \cdot 10^2 = 6940 \text{ #}'$$

Effective depth 16"

$$\text{Steel Area} = \frac{6940 \times 12}{\frac{7}{8} \cdot 16 \cdot 17000} = 0.35 \square''$$

1/2" bars 6" centers = 0.390"

18.0 below top Pressure = 20 . 33 = 660 #

$$m = \frac{1}{12} \cdot 660 \cdot 10^2 = 5500 \text{ #}'$$

$$\text{Steel Area} = \frac{5500 \times 12}{\frac{7}{8} \cdot 16 \cdot 17000} = .277 \quad \text{Use } \tfrac{1}{2}\text{" } \square \text{ bars } 8\text{" centers} = 0.290\text{"}$$

10.0 below top Pressure = 12 . 33.3 = 400 #/□'

$$m = \frac{1}{12} \cdot 400 \cdot 10^2 = 3330 \text{ #}'$$

$$\text{Steel Area} = \frac{3330 \times 12}{\frac{7}{8} \cdot 16 \cdot 17000} = 0.168 \quad \text{Use } \tfrac{1}{2}\text{" } \square \text{ bars } 12\text{" centers} = 0.190\text{"}$$

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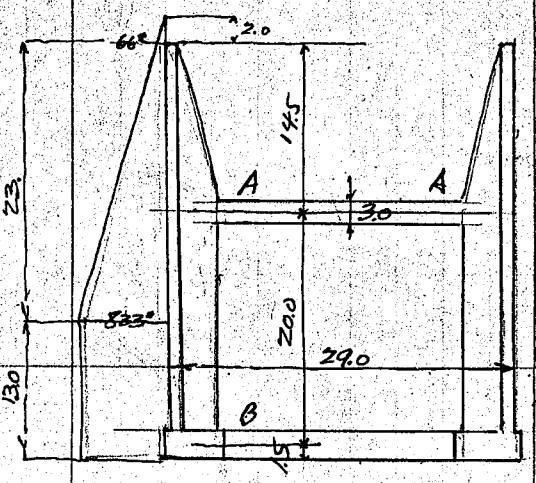
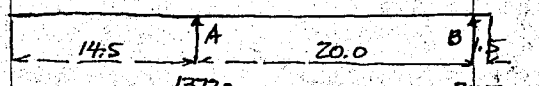
CALCULATIONS FOR

Revised Design of Hakuchi-Bashi Approach Spans

Transverse Frame





Reaction at A.

			arm		
66 . 23.0	=	1520	. 23.0	=	35000
$\frac{767}{2}$. 23.0	=	8820	. 19.17	=	169000
833 . 13.0	=	10830	. 6.5	=	70400
		21170#			274400

Reaction A = 274400 ÷ 20 = 13720# per ft.
Reaction B = 21170 − 13720 = 7450#

Point of max moment 7450 ÷ 833 = 8.93
1.50
7.43

Moment = 7450 . 7.43 = 55300
833 . $\frac{8.93^2}{2}$ = 33300
22000

For 10' panel 22000 . 10 = 220000#
Effective depth required = $\sqrt{\frac{220.000}{1.5 \times 102}}$ = 38"
Effective depth assumed = 51"
Steel Area = $\frac{220.000 \times 12}{\frac{7}{8} \times 51 \times 17000}$ = 3.47 □" 6-7/8"ϕ = 3.60"

Moment at A. center of strut 14.5' below top

66 . 14.5	=	956	× 7.25	=	6930
$\frac{483}{2}$. 14.5	=	3500	. 4.83	=	16900
		4456			23830#

Steel Area = $\frac{23830 \times 12}{\frac{7}{8} \cdot 51 \cdot 17000}$ = 3.76 □" use 6-7/8"ϕ = 3.60 □"

Transverse strut AA.
as beam weight of Earth 1300 . 1.5 = 2080
beam say 675
2755# per lin ft

Span length say 22.0 moment = $\frac{1}{12}$. 2755 . 22^2 = 111.000#
steel area required
for binding only = $\frac{111000 \times 12}{\frac{7}{8} \cdot 34" \cdot 17000}$ = 2.63 □" 5-7/8"ϕ = 3.00 □"

For direct tension. 137200 ÷ 17000 = 8.06 □" add 15-7/8"ϕ bars = 9.0 □"
for direct tension for strut.

max End shear 2755 . $\frac{22}{2}$ = 30300#
Unit shear = $\frac{30300}{18 \cdot 34 \cdot \frac{7}{8}}$ = 57#/□" for dead load shear only.

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CALCULATIONS FOR

List of Material for Hakuchi-Bashi

Descr.		No.	Section in ins.	Length ft.	Length ins.	Wt. in lbs One ft	Wt. in lbs. One piece	Tot. Wt. in lbs.	Remarks
			Upper Chord Ls0-Ls1				**4 Req'd.**		
Web	Pl	2	16½ × ⅜	17	4	21.04	364.692	729	
Flg	L	2	4 × 4 × ⅜	17	0	9.80	166.600	333	
Tie	Pl	3	23¼ × ⅜	1	9	29.65	51.888	156	
	"	1	"	3	4½	"	100.069	100	
	"	2	21 × ⅜	2	9½	26.78	74.762	150	
	"	2	39 × ⅝	4	1½	82.88	341.880	684	
	L	2	3 × 3 × 5/16	2	0	6.10	12.200	24	
	Fills	2	3 × ⅜	1	3½	3.83	4.947	10	
	Pl	1	12 × 5/16	3	6	12.75	44.625	45	
	"	2	4¼ × ½	2	2	7.23	15.665	31	
	L	2	4 × 4 × ⅜	16	6½	9.8	162.109	324	
Lac. Bars		20	2½ × ½	2	7½	4.25	11.156	223	
								2,809 × 4 = 11,236	
			Upper chord Us1-Us3 4 Req'd.						
Web	Pl	2	16½ × ⅜	25	9¾	21.04	543.095	1,086	
Flg	L	2	4 × 4 × ⅜	25	9¾	9.80	252.963	506	
"	"	2	"	25	4¼	"	248.471	497	
	Pl	2	23¼ × ⅜	2	0	29.65	59.300	119	
	"	2	21 × ⅜	1	11¼	26.78	51.886	104	
	"	1	23¼ × ⅜	2	6	29.65	74.125	74	
	"	1	23¾ × ⅜	2	5¾	30.29	75.095	75	
	Fill	2	8½ × ¼	0	11¾	7.225	7.075	14	
	"	4	3 × ¼	0	11¾	2.55	2.497	10	
	Pl.	2	15 × ½	2	2⅝	25.50	56.579	113	
	"	2	38 × ⅝	5	2	80.75	417.211	834	
	"	2	4½ × ½	1	11⅝	7.65	15.061	30	
	"	2	14⅝ × ⅝	1	10½	31.135	58.378	117	
Lac. Bars.		36	2½ × ½	2	7½	4.25	11.156	402	
								3981 × 4 = 15,924	
			Upper Chord Us3-Us5 4 Req'd.						
Web	Pl	2	16½ × 5/8	24	5	35.06	856.050	1,712	
Side	"	2	8½ × ½	25	6¼	14.45	368.776	738	
	"	2	15 × ⅜	26	7½	19.13	509.336	1,019	
Flg	L	4	4 × 4 × ½	25	6¼	12.80	326.666	1,307	
	Pl	2	23¾ × ⅜	2	0	30.29	60.580	121	
	"	2	21 × ⅜	1	11¾	26.78	53.003	106	
	"	1	23¾ × ⅜	2	4½	30.29	71.939	72	
	L	2	4 × 4 × ½	3	2½	12.80	42.112	84	
	Pl	2	4½ × ½	1	11½	7.65	14.947	30	
	Fill	2	13 × ⅛	1	3	5.53	6.913	14	
	"	2	16½ × 5/8	2	2½	35.06	77.423	155	
	Pl	2	15 × ½	2	2½	25.50	56.312	113	
	"	2	46 × 5/8	5	0	97.75	488.750	978	
	"	2	14½ × ½	1	10½	24.65	46.219	92	
Lac. Bars.		34	2½ × ½	2	8¼	4.25	11.422	388	
								6,929 × 4 = 27,716	

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Descr.	No.	Section in ins.	Length ft.	Length ins.	Wt. in lbs. One ft.	Wt. in lbs. one piece	Tot. Wt. in lbs.	Remarks.
Upper Chord Us5-Us7 4 Req'd.								
Web Pl	2	16½ × 5/8	23	3¾	35.06	817.336	1,635	
Side "	2	8½ × ½	25	6¼	14.45	368.776	738	
" "	2	15 × ½	25	6¼	25.50	650.780	1,302	
Flg L	4	4 × 4 × ½	25	6¼	12.80	326.666	1,307	
Cov Pl	1	24 × ½	28	1¼	40.80	1,146.480	1,146	
"	2	23¾ × 3/8	2	0	30.29	60.580	121	
"	2	21 × 3/8	1	11¾	26.78	53.002	106	
L	2	6 × 4 × ½	4	8½	16.2	76.302	153	
Pl.	1	24 × ½	2	5½	40.8	100.368	100	
"	2	4½ × ½	1	11½	7.65	15.141	30	
"	2	16½ × 5/8	2	2½	35.06	77.423	155	
"	2	15 × ½	2	2½	25.50	56.312	113	
"	2	48½ × 5/8	4	6	103.06	463.770	927	
"	2	14½ × 5/8	1	10½	30.81	57.769	116	
Lac. Bars	12	2½ × ½	2	8¼	4.25	11.422	137	
							8.086 × 4 = 32,344	
Upper Chord Us7-Us8 4 Req'd.								
Cov. Pl.	1	24 × ½	12	10¾	40.8	526.32	526	
" "	1	24 × ½	14	10⅝	"	607.512	608	
Web "	2	16½ × 5/8	11	9½	35.06	413.417	827	
Side "	2	8½ × ½	12	10¾	14.45	186.344	373	
" "	2	15 × ½	11	9⅜	25.50	300.423	601	
Flg Ls	4	4 × 4 × ½	12	10¾	12.80	165.066	660	
Pl.	1	23¾ × ¾	2	0	30.29	60.580	61	
"	1	21 × ⅜	1	11¾	26.78	53.003	53	
Ls	2	6 × 4 × ½	3	3	16.2	52.65	105	
Fill	1	24 × ½	3	6	40.8	142.800	143	
Pl.	1	24 × ½	8	1	"	329.664	330	
"	2	4½ × ½	2	0	7.65	15.300	31	
Fill	2	15¼ × ½	2	3	25.93	58.343	116	
Pl.	2	27 × 5/8	3	3	57.38	186.485	373	
Lac. Bars	6	2½ × ½	2	8¼	4.25	11.422	68	
							4875 × 4 = 19,500	
Upper Chord Us8-U61 4 Req'd.								
Web Pl.	2	16½ × 5/8	11	6⅞	35.06	405.746	812	
Flg Ls	4	4 × 4 × ½	12	8⅛	12.80	162.267	649	
Side Pl.	2	8½ × ½	12	8⅛	14.45	183.184	366	
"	2	15 × ½	11	5¾	25.50	292.720	585	
Cov. "	1	24 × ½	12	8⅛	40.8	517.344	517	
" "	1	24 × ½	9	2½	40.8	375.768	376	
"	1	21 × ⅜	1	11¾	26.78	53.003	53	
"	1	23¾ × ⅜	2	0	30.29	60.580	61	
Ls	2	6 × 4 × ½	3	2⅝	16.2	52.164	104	
Fill.	1	24 × ½	3	6	40.8	142.80	143	
"	2	13¼ × ⅛	1	3	5.636	7.045	14	
							3,680 × 4 = 14,720	

増田淳氏関係資料
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JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Desc.		No.	Section. in. ins.	Length ft.	Length ins	Wt. in lbs. One ft.	Wt. in lbs. One piece	Tot. Wt. in. lbs.	Remarks
			U$8-U$1 (Continued)						
	Pl	1	24 × 1/2	8	1	40.80	329.664	330	
Spl.	"	2	4 1/2 × 1/2	1	10	7.65	14.025	28	
Fill.		2	16 1/2 × 5/8	2	2 5/8	35.06	77.791	155	
Spl.	Pl.	2	15 × 1/2	2	2 3/4	25.50	56.845	114	
Guss	P/s	2	46 1/2 × 5/8	4	6	98.81	444.645	889	
Lat. Bars		6	2 1/2 × 1/2	2	8 1/4	4.25	11.422	68	
								1,584 × 4 = 6,336	
			Upper Chord U$1 - U$3 4 Reqd.						
	Fill.	2	13 1/4 × 1/8	1	3	5.636	7.045	14	
Web.	P/s	2	16 1/2 × 5/8	23	3 5/8	35.06	816.972	1633	
Flg	L$	4	4 × 4 × 1/2	25	6 1/4	12.80	326.66	1307	
Side	P/s	2	8 1/2 × 1/2	25	6 1/4	14.45	368.776	738	
"	"	2	15 × 5/8	25	6 1/4	31.88	813.578	1627	
Cov.	P/s	1	24 × 1/2	25	6 1/4	40.80	1041.216	1041	
"	"	1	24 × 1/2	22	0 7/8	40.80	899.660	900	
Tie	"	2	23 3/4 × 3/8	2	0	30.29	60.580	121	
"	"	2	21 × 3/8	1	11 3/4	26.78	53.003	106	
	L$	2	6 × 4 × 1/2	3	2 5/8	16.20	52.164	104	
Fill.		1	24 × 1/2	3	5 5/8	40.8	141.576	142	
Spl.	Pl.	1	24 × 1/2	8	1	40.8	329.664	330	
"	"	2	4 1/2 × 1/2	1	10	7.65	14.025	28	
Fill.		2	16 1/2 × 5/8	2	2 5/8	35.06	77.791	156	
Spl.	P/s	2	15 × 1/2	2	2 3/4	25.50	56.845	114	
Guss	"	2	43 1/2 × 5/8	4	9	92.44	439.09	878	
	"	2	14 1/2 × 5/8	1	10 1/2	30.81	57.769	116	
Lat. Bars		12	2 1/2 × 1/2	2	8 1/4	4.25	11.422	137	
								~~9,167 × 4 = 36,668~~	
								9492 × 4 = 37,968	
			Upper Chord U$3 - U$5 4 Reqd.						
Cov.	Pl	1	24 × 1/2	27	6 1/4	40.80	1122.816	1,123	
"	"	1	24 × 1/2	25	6 1/4	40.80	1041.216	1,041	
Flg	L$	4	4 × 4 × 1/2	25	6 1/4	12.80	326.666	1,307	
Web.	P/s	2	16 1/2 × 5/8	23	3 5/8	35.06	816.972	1.633	
Side	"	2	8 1/2 × 1/2	25	6 1/4	14.45	368.776	737	
"	"	2	15 × 1/2	25	6 1/4	25.50	650.780	1301	
Tie	"	2	23 3/4 × 3/8	2	0	30.29	60.580	121	
"	"	2	21 × 3/8	1	11 3/4	26.78	53.003	106	
	L$	2	6 × 4 × 1/2	4	10 3/4	16.20	79.380	159	
Spl.	Pl.	1	24 × 1/2	2	7 3/4	40.80	108.12	108	
"	"	2	4 1/2 × 1/2	2	1 7/8	7.65	16.336	33	
Fill.		2	16 1/2 × 5/8	2	2 5/8	35.06	77.791	155	
Spl.	P/s	2	15 × 1/2	2	2 5/8	25.50	56.579	113	
Guss	"	2	37 1/2 × 5/8	4	10	79.69	385.166	770	
	P/s	2	14 5/8 × 5/8	1	10 1/2	31.08	58.275	117	
Lat. Bars		12	2 1/2 × 1/2	2	8 1/4	4.25	11.422	137	
								8.961 × 4 = 35,844	

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CALCULATIONS FOR

Desc.		NO	Section in ins	Length ft.	Length ins	Wt. lbs. One ft.	Wt. in lbs. one piece	Tot. Wt. in lbs	Remarks.
			Upper Chord Uc5-Uc7 4 Req'd.						
Cov.	Pls	1	24 × 1/2	28	1/4	40.80	1146.480	1146	
Flg.	Ls	4	4 × 4 × 1/2	25	6 1/4	12.80	326.666	1307	
Web.	Pls	2	16 1/2 × 5/8	23	3 5/8	35.06	816.972	1634	
Side	"	2	8 1/2 × 1/2	25	6 1/4	14.45	368.776	738	
"	"	2	15 × 1/2	25	6 1/4	25.50	650.780	1301	
Tie	"	2	23 3/4 × 3/8	2	0	30.29	60.580	121	
"	"	2	21 × 3/8	1	11 3/4	26.78	53.003	106	
	Ls	2	4 × 4 × 1/2	3	4 5/8	12.80	43.392	86	
Fill		2	16 1/2 × 5/8	2	2 5/8	35.06	777.91	156	
Spl.	Pls	2	15 × 1/2	2	2 5/8	25.50	56.579	113	
"	"	2	4 1/2 × 1/2	2	1 5/8	7.65	16.336	33	
Guss	"	2	35 × 5/8	4	8 1/4	74.38	348.656	697	
	Pls	2	14 5/8 × 5/8	1	10 1/2	31.08	58.275	117	
Latt. Bars		12	2 1/2 × 1/2	2	8 1/4	4.25	11.422	137	
								7,693 × 4 = 30,772	
			Upper Chord Uc7-Uc9 4 Req'd.						
	Fill	2	8 1/2 × 1/8	1	1 1/4	3.613	3.974	8	
Web.	Pls	2	16 1/2 × 5/8	23	3 5/8	35.06	816.972	1633	
Flg	Ls	4	4 × 4 × 1/2	25	6 1/4	12.80	326.666	1307	
Side	Pls	2	8 1/2 × 1/2	25	6 1/4	14.45	368.776	738	
"	"	2	15 × 3/8	26	7 5/8	19.13	509.535	1020	
Tie	"	3	23 5/8 × 3/8	2	0	30.125	60.25	181	
"	"	2	21 × 3/8	1	1 5/8	26.78	30.460	61	
"	"	1	23 5/8 × 3/8	2	6	30.125	75.313	75	
Fills		2	17 × 5/8	2	2 1/2	36.13	79.786	159	
Guss	Pls	2	34 × 5/8	4	8 5/8	72.25	340.933	681	
	Pls	2	14 5/8 × 5/8	1	10 1/2	31.08	58.275	117	
Spl.	"	2	4 5/8 × 1/2	2	0	7.865	15.730	31	
Latt. Bars		16	2 1/2 × 1/2	2	8 1/4	4.25	11.422	183	
								6,194 × 4 = 24778	
			Upper Chord Uc9-Uc11 4 Req'd.						
	Fill	2	8 1/2 × 1/8	1	1 1/4	3.613	3.974	8	
Web	Pls	2	16 1/2 × 5/8	24	4 7/8	35.06	858.605	1717	
Flg.	Ls	4	4 × 4 × 1/2	25	6 1/4	12.80	326.666	1307	
Side	pls	2	8 1/2 × 3/8	25	6 1/4	10.84	276.627	553	
Tie	Pls	3	23 5/8 × 3/8	2	0	30.125	60.25	181	
"	"	2	21 × 3/8	1	11 5/8	26.78	51.118	106	
"	"	1	23 5/8 × 3/8	2	6	30.125	35.313	75	
Spl.	"	2	4 5/8 × 1/2	2	0	7.865	15.730	31	
Fills		2	13 1/4 × 1/8	1	3	5.636	7.046	14	
Spl.	Pls	2	15 × 5/8	2	2 1/2	31.88	70.423	141	
Guss	"	2	34 × 5/8	4	2 5/8	72.25	304.808	609	
	Pls	2	14 5/8 × 5/8	1	10 1/2	31.08	58.275	117	
Latt. Bars		36	2 1/2 × 1/2	2	8	4.25	11.333	408	
								~~5,485 × 4 = 21940~~	
								5,267 × 4 = 21,068	

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CALCULATIONS FOR

Desc.		No.	Section in ins	Length ft.	Length ins.	Wt. in lbs. One ft.	Wt. in lbs. One piece	Tot. Wt. in lbs	Remarks.
Upper Chord Uc11–Uc13 4 Req'd.									
Web	P/s	2	16½ × 5/8	25	6¼	35.06	894.759	1785	
Flg	Ls	4	4 × 4 × 3/8	25	6¼	9.8	250.104	1001	
Side	P/s	2	8½ × 3/8	26	7⅝	10.84	288.728	577	
Tie	"	3	23⅛ × 3/8	2	0	29.49	58.980	177	
"	"	2	21 × 3/8	1	11⅛	26.78	51.608	103	
Spl.	"	2	4⅜ × ½	2	0	7.44	14.88	30	
Tie	"	1	23⅛ × 3/8	2	6	2.949	7.373	74	
Spl.	"	2	15 × 3/8	2	2⅝	19.13	42.446	85	
Guss	"	2	34 × 5/8	4	2⅝	72.25	304.808	610	
	P/s	2	14⅝ × 5/8	1	10½	31.08	58.275	117	
Latt. Bars		36	2½ × ½	2	8	4.25	11.333	408	
								4,967 × 4 = 19,868	
Upper Chord Uc13–Uc15 4 Req'd.									
Web	P/s	2	16½ × 3/8	25	6¼	21.04	536.958	1074	
Flg	Ls	4	4 × 4 × 3/8	25	6¼	9.80	250.104	1000	
Tie	P/s	3	23⅛ × 3/8	2	0	29.49	58.980	177	
"	"	2	21 × 3/8	1	11⅛	26.78	51.607	103	
Fills		2	16½ × ¼	2	1¼	14.03	29.522	59	
"	"	2	15 × ¼	1	1¼	12.75	14.079	28	
"		4	8½ × 3/8	1	1¼	10.84	11.970	48	
Spl.	P/s	2	15 × 3/8	2	2¼	19.13	41.847	84	
Guss	"	2	34 × 3/8	3	11⅝	43.35	172.047	344	
	P/s	2	14⅝ × 5/8	1	10½	31.08	58.275	117	
Latt Bars		36	2½ × ½	2	8	4.25	11.333	408	
Tie	P/s	1	23⅛ × 3/8	2	6	29.49	73.725	74	
Spl.	"	2	4⅜ × ½	2	0	7.44	14.880	30	
								3,546 × 4 = 14,184	
Upper Chord Uc15–Uc17 4 Req'd.									
Web.	P/s	2	16½ × 3/8	26	6⅝	21.04	558.656	1117	
Flg.	Ls	4	4 × 4 × 3/8	26	6⅝	9.80	260.211	1041	
Tie	P/s	3	22⅝ × 3/8	2	0	28.845	57.69	173	
"	"	2	21 × 3/8	1	10⅝	26.78	50.491	101	
Fills		2	8½ × 3/8	1	0	10.84	10.840	22	
Spl.	P/s	2	15 × 3/8	1	0	19.13	19.130	38	
Guss	"	2	29 × 3/8	3	8⅝	36.98	137.521	275	
	P/s	2	14⅝ × 3/8	1	10½	18.645	34.96	70	
Lac. Bars		36	2½ × ½	2	8	4.25	11.333	408	
Tie	P/s	1	22⅝ × 3/8	2	6	28.845	57.69	58	
								3,303 × 4 = 13,212	

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CALCULATIONS FOR

Descr.		NO.	Section in ins.	Length ft.	Length ins.	Wt. in lbs One ft.	Wt. in lbs. One piece	Tot. Wt. in lbs.	Remarks.
Upper Chord Uc17-Uc18 4 Reqd.									
Web.	P/s	2	16½ × ⅜	11	3⅛	21.04	236.919	474	
Flg	Ls	4	4 × 4 × ⅜	11	3⅛	9.80	110.352	441	
Tie	P/s	2	22⅝ × ⅜	2	0	28.845	57.69	115	
"	"	1	21 × ⅜	1	10⅝	26.78	50.491	51	
	P/s	2	14⅝ × ⅜	1	10½	18.645	34.96	70	
Lac. Bars		18	2½ × ½	2	8	4.25	11.333	204	
								1,355 × 4 = 5,420	
Lower Chord Ls0-Ls2 4 Reqd.									
	Fil	4	4 × ⅜	1	11¼	5.1	9.884	40	
Web.	P/s	2	16½ × ⅜	26	2⅝	21.04	551.648	1103	
Flg	Ls	2	4 × 4 × ⅜	24	9⅜	9.80	242.857	486	
"	"	2	"	26	2⅝	9.80	256.946	514	
Tie	P/s	4	23¼ × ⅜	2	0	29.65	59.300	237	
"	"	1	23⅝ × ⅜	3	0	30.125	90.375	90	
Guss	"	1	38 × ⅜	5	5½	48.45	264.455	264	
Spl.	"	2	4½ × ½	2	0½	7.65	15.619	31	
Fill		2	8½ × ⅜	1	1¼	10.84	11.970	24	
"		2	15 × ⅜	1	1¼	19.13	21.123	42	
"		2	8½ × ½	1	1¾	14.45	16.557	33	
Spl.	P/s	2	15¼ × ½	2	3½	25.93	59.424	119	
Guss	"	2	37½ × ⅝	4	11	79.69	391.812	784	
Fills		2	11½ × ⅝	1	4½	24.44	33.605	67	
"		2	8½ × ⅜	0	11½	10.84	10.388	21	
Diaphragm	Ls	8	3 × 3 × ⅜	1	4½	7.20	9.900	79	
"	P/s	2	14 × ½	1	4½	23.80	32.725	65	
Lac. Bars		32	2½ × ½	2	7½	4.25	11.156	357	
	Pl	1	12½ × ⅜	2	10½	15.94	45.828	46	
	Ls	4	4 × 4 × ⅜	2	10½	9.80	28.175	113	
	"	2	"	0	11½	9.80	19.518	39	
	P/s	2	3" ϕ × ⅜				0.55	1	
	"	1	8¼ × ⅝	1	0	17.53	17.530	18	
	Ls	4	4 × 4 × ½	3	0	12.80	38.400	154	
	P/s	2	4 × ⅜	1	7½	5.10	8.289	33	
	"	2	8½ × ⅜	1	4½	10.84	14.905	30	
Guss	"	2	43½ × ⅝	4	0	92.44	369.760	740	
								5,530 × 4 = 22,120	

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CALCULATIONS FOR

Deser.		No.	Section. in ins.	Length ft	Length ins	Wt. in lbs One ft.	Wt. in lbs One piece	Tot. Wt. in lbs.	Remarks
			Lower Chord L52-L54 4 Req'd.						
Web	P/s	2	16½ × 5/8	24	3⅛	35.06	850.570	1701	
Flg	Ls	2	4×4× 1/2	25	6½	12.80	326.934	654	
"	"	2	"	25	6	12.80	326.400	653	
Tie	P/s	2	23¾ × ⅜	2	0	30.29	60.580	121	
"	"	1	21 × ⅜	1	11¾	26.78	53.003	106	
"	"	1	23¾ × ⅜	3	0	30.29	90.87	91	
Guss	"	1	39 × ⅜	5	6	49.73	273.515	274	
Spl.	"	2	5 × ½	2	1½	8.50	18.063	36	
Fills		2	8½ × ½	1	1¾	14.45	16.857	33	
"		2	15 × ½	1	1¾	25.50	29.218	58	
"		2	17¼ × 5/8	2	4	36.66	85.539	171	
Spl.	P/s	2	15¾ × ½	2	4	26.78	62.486	125	
Guss	"	2	46 × 5/8	5	1	97.75	496.893	994	
Fills		2	12 × 5/8	1	4½	25.50	35.063	70	
"		2	8½ × ½	1	0	14.45	14.450	29	
Diaph.	Ls	8	3×3× ⅜	1	4½	7.20	9.900	79	
"	P/s	2	14 × ½	1	4½	23.80	32.725	65	
Lac. Bars		32	2½ × ½	2	8¼	4.25	11.422	366	
								5,626 × 4 = 22,504	
			Lower Chord L54-L56 4 Req'd.						
Web	P/s	2	16½ × 5/8	23	3⅝	35.06	816.972	1634	
"	"	2	16½ × ½	25	8	28.05	719.951	1440	
Side	"	2	8½ × ⅜	25	8	10.84	310.785	622	
Flg	Ls	2	4×4× ½	25	8	12.80	328.534	657	
"	"	2	"	25	7¾	12.80	328.266	656	
Tie	P/s	2	24 × ½	2	0¾	40.80	84.150	168	
"	"	2	21 × ½	2	0¾	35.70	73.631	147	
"	"	1	25 × ⅜	3	0	31.88	95.640	96	
Spl.	"	2	5 × ½	2	2¼	8.50	18.594	37	
Guss	"	1	41 × ⅜	5	10	52.28	304.965	305	
Fill		2	12 × ½	1	3	20.40	25.50	51	
"		2	17 × ½	2	5	28.90	69.843	40	
"		2	15½ × ½	2	5	26.35	63.680	127	
Spl.	P/s	2	16 × ½	4	5	27.20	120.130	240	
Guss	"	2	51½ × 5/8	5	4	109.46	583.783	1167	
Lac Bars		32	3 × ½	2	9⅝	5.10	14.291	457	
Fills		2	15 × 5/8	1	2½	31.88	38.521	77	
Diaph.	Ls	8	3×3× ⅜	1	4½	7.20	9.900	79	
"	P/s	2	14 × ½	1	4½	23.80	32.725	65	
								[illegible]	
	Fill	2	8½ × ⅛	1	2	3.613	4.227	8	
	"	2	8½ × ⅛	1	2½	"	4.372	9	
	"	2	8½ × ⅛	2	2¼	"	7.912	16	
								8,098 × 4 = 32,392	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
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CALCULATIONS FOR

Descr.		No.	Section in ins	Length ft.	Length ins.	Wt in lbs one ft.	Wt. in lbs one piece	Tot Wt. in lbs.	Remarks
Lower Chord Ls6-Ls8 4 Req'd.									
Web.	Pls	2	16½ × 5/8	23	0	35.06	806.380	1613	
"	"	2	16½ × ½	26	0	28.05	729.300	1459	
Side	"	2	8½ × ½	25	0¼	14.45	361.557	723	
"	"	2	15 × ½	23	0	25.50	586.500	1173	
Flg	Ls	2	4 × 4 × ½	25	0¼	12.80	320266	641	
"	"	2	"	26	0	12.80	332.800	666	
Tie	Pls	2	24 × ½	2	0¾	40.80	84.150	168	
"	"	2	21 × ½	2	0¾	35.70	73.635	147	
"	"	1	23 × 3/8	3	0	29.33	87.990	88	
Spl.	"	2	5 × ½	1	11	8.50	16.290	32	
Guss	"	1	42 × 3/8	6	4	53.55	339.148	339	
	Pls	2	43½ × 5/8	4	3	92.44	392.87	786	
Fills		2	19 × ½	3	5½	32.30	111.703	223	
~~"~~		~~2~~	~~15 × 1/4~~	~~1~~	~~3~~	~~12.75~~	~~15.938~~	~~32~~	
Guss	Pls	5	67 × 5/8	5	10½	142.40	836.760	1673	
	Ls	4	6 × 4 × ½	2	5	16.20	39.151	156	
	Pls	1	11½ × ½	2	5	19.55	47.246	47	
	"	2	27 × 5/8	3	2½	57.38	184.092	368	
Lac Bars		32	3 × ½	2	9 5/8	5.10	14.291	467	
Fills		2	15 × 5/8	1	2½	31.88	38.521	77	
Diaph.	Ls	8	3 × 3 × 3/8	1	4½	7.20	9.900	79	
"	Pls	2	14 × ½	1	4½	23.80	32.725	65	
								10,990 × 4 = 43,960	
Lower Chord Ls8-L02 4 Req'd									
Web	Pls	2	16½ × 5/8	22	11	35.06	803.460	1607	
"	"	2	"	"	26	0¾	35.06	913.751	1837
Flg	Ls	2	4 × 4 × ½	25	11	12.80	331.734	663	
"	"	2	"	26	0½	12.80	333.333	667	
Side	Pls	2	8½ × ½	26	0½	14.45	376.303	753	
"	"	2	15 × ½	22	11	2550	584.460	1,169	
Tie	"	2	24 × ½	2	1	40.80	84.986	170	
"	"	2	21 × ½	2	1	35.70	74.374	149	
"	"	1	25 × 3/8	3	0	31.88	98.640	96	
Guss	"	1	42 × 3/8	5	10	53.55	312.373	312	
Spl.	"	2	5 × ½	2	1¾	8.50	18.239	36	
Fill.		2	17½ × 5/8	2	10¾	37.19	107.695	215	
"		2	16 × ½	2	10¾	27.20	78.766	157	
Guss	Pls	2	46 × 5/8	5	4½	97.75	525.406	1051	
Fills		2	15 × 5/8	1	3	31.88	39.850	80	
Lac Bars		32	3 × ½	2	7/8	5.10	13.228	423	
Diaph.	Ls	8	3 × 3 × 3/8	1	4½	7.20	9.900	79	
"	Pls	2	14 × ½	1	4½	23.80	32.725	65	
								9529 × 4 = 38,116	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
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Descr.		No.	Section in ins.	Length ft.	Length ins	Wt. in lbs One ft.	Wt. in lbs. One piece	Tot. Wt. in lbs.	Remarks
			Lower Chord Lc2 – Lc4 4 Req'd.						
	Fill	2	15 × 1/8	1	5	6.38	9.060	18	
Web	Pls	2	16½ × 5/8	22	9 5/8	35.06	799.442	1599	
"	"	2	"	25	8 1/4	35.06	900.604	1801	
Flg	Ls	2	4 × 4 × 1/2	25	8 1/4	12.80	328.800	658	
"	"	2	"	25	7 1/2	12.80	328.000	656	
Side	Pls	2	8½ × 1/2	25	8	14.45	370.884	742	
"	"	2	15 × 5/8	22	9 7/8	25.5	58.1400	1163	
Tie	"	2	24 × 1/2	2	0 3/4	40.80	84.150	168	
"	"	2	21 × 1/2	2	1	35.70	74.374	149	
"	"	1	25 1/4 × 3/8	3	0	32.20	96.600	97	
Spl.	"	2	5 × 1/2	2	3 3/4	8.50	19.656	39	
Guss	"	1	42 × 3/8	5	11½	53.55	531.778	532	
Fills		2	13 × 5/8	1	4½	27.63	37.991	76	
"		2	17 × 5/8	2	8	36.13	96.348	193	
Spl.	Pls	2	15½ × 1/2	2	10	26.35	74.657	149	
"	"	2	16 3/4 × 1/2	5	4	28.48	151.892	304	
Guss	"	2	42 × 5/8	5	4	89.25	475.997	952	
Lac. Bars		32	3 × 1/2	2	7 1/8	5.10	13.228	423	
Diaph.	Ls	8	3 × 3 × 3/8	1	4½	7.20	9.900	79	
"	Pls	2	14 × 1/2	1	4½	23.80	32.725	65	
								~~40,136 × 4 = 40,544~~	
								9.863 × 4 = 39.452	
			Lower Chord Lc4 – Lc6 4 Req'd.						
	Fill	2	8½ × 1/8	2	7 1/4	3.613	9.394	19	
Web	Pls	2	16½ × 5/8	22	8 7/8	35.06	797.250	1594	
"	"	2	16½ × 1/2	25	0 1/4	28.05	701.833	1404	
Side	"	2	8½ × 1/2	25	6½	14.45	369.078	738	
"	"	2	15 × 3/8	22	8 7/8	19.13	435.016	1870	
Flg	Ls	2	4 × 4 × 1/2	25	6½	12.80	326.934	655	
"	"	2	"	25	6	12.80	326.400	653	
Tie	Pls	2	24 × 1/2	2	0 3/4	40.80	84.150	168	
"	"	2	21 × 1/2	2	0 3/4	35.70	73.631	147	
"	"	1	25½ × 3/8	3	0	32.52	97.560	98	
Spl.	"	2	5 × 1/2	2	3 3/4	8.50	19.656	39	
Guss	"	1	31 3/4 × 3/8	5	8	40.49	229.445	229	
Fills		2	14 × 5/8	1	4½	29.75	40.906	82	
"		2	17 × 5/8	2	9 1/4	36.13	100.109	200	
"		2	15½ × 1/2	4	0 1/4	26.35	105.948	212	
Spl.	Pls	2	15 3/4 × 1/2	5	5 1/4	26.78	145.616	291	
Guss	"	2	37 × 5/8	5	4	78.63	419.357	839	
Lac Bars		32	2½ × 1/2	2	9 5/8	4.25	11.909	381	
Diaph.	Ls	8	3 × 3 × 3/8	1	4½	7.2	9.900	79	
"	Pls	2	14 × 1/2	1	4½	23.80	32.725	65	
								8,763 × 4 = 35,052	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CONSULTING ENGINEER
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CALCULATIONS FOR

Descr.		No.	Section in ins.	Length Ft.	Length Ins	Wt. in lbs. One ft.	Wt. in lbs One piece	Tot. Wt. in lbs	Remarks.
			Lower Chord LC6-LC8 4 Reqd						
	Fills	2	8½ × ⅛	1	4½	3.613	4.986	10	
Web.	Pls	2	16½ × ⅝	22	9¾	35.06	799.806	1600	
"	"	2	16½ × ½	25	6	28.05	715.275	1431	
Side	"	2	8½ × ⅜	25	6	10.84	276.420	553	
Flg	Ls	4	4 × 4 × ½	25	6	12.80	326.400	1306	
Tie	Pls	2	24 × ⅜	2	0⅝	30.60	62.794	126	
"	"	2	24 × ⅜	2	0⅝	26.78	54.955	110	
"	"	1	25½ × ⅜	3	0	32.52	97.560	98	
Spl.	"	2	5⅛ × ½	2	0	87.13	17.430	35	
Fills		2	11¼ × ⅝	1	4½	23.91	32.876	66	
"		2	16½ × ⅝	2	2½	35.06	77.423	155	
"		2	15 × ½	1	1¼	25.50	28.157	56	
Spl.	Pls	2	15 × ½	2	2½	25.50	56.312	113	
Guss	"	2	34 × ⅝	4	8⅝	72.25	340.933	682	
Lac. Bars		32	2½ × ½	2	9⅝	4.25	11.909	381	
Diaph	Ls	8	3 × 3 × ⅜	1	4½	7.2	9.900	79	
"	Pls	2	14 × ½	1	4½	23.80	32.725	65	
	Fill.	2	8½ × ⅛	1	1¼	3.613	3.974	8	
								1718	
								6,874 × 4 = 27,496	
			Lower chord Lc8-Lc10 4 Reqd.						
	Fill	2	15 × ⅛	2	1¼	6.38	13.398	27	
Web	Pls	2	16½ × ⅝	24	4¾	35.06	855.317	1711	
Flg	Ls	4	4 × 4 × ½	25	6	12.80	326.400	1306	
Side	Pls	2	8½ × ½	25	6	14.45	368.475	737	
Tie	"	2	23⅝ × ⅜	2	0	30.125	60.240	120	
"	"	2	21 × ⅜	1	11⅝	26.78	52.724	105	
"	"	1	28⅜ × ⅜	2	10⅝	36.18	104.393	104	
Spl.	"	2	4⅜ × ⅜	2	0	8.58	11.160	22	
Guss	"	1	34 × ⅜	4	4⅜	43.35	189.205	189	
Fills		2	11½ × ⅝	1	5	24.44	34.624	69	
"		2	8½ × ½	0	10¼	14.45	12.317	25	
Spl.	Pls	2	15 × ½	1	8½	25.50	43.562	87	
Guss	"	2	34 × ⅝	4	2⅝	72.25	304.808	610	
Lac Bars		34	2½ × ½	2	8	4.25	11.335	385	
Diaph.	Ls	8	3 × 3 × ⅜	1	4½	7.2	9.900	79	
"	Pls	2	14 × ½	1	4½	23.80	32.725	65	
								5641	
								5,641 × 4 = 22,564	

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

Descr.		No.	Section in ins	Length ft.	Length ins.	Wt. in lbs One ft.	Wt. in lbs One piece	Tot Wt. in lbs.	Remarks
			Lower Chord LC10 - LC12 4 Req'd.						
	Fill	4	15 x 1/8	2	1 1/4	6.38	13.398	54	
Web.	Pls	2	16 1/2 x 3/8	25	6	28.05	715.275	1431	
Flg	Ls	4	4 x 4 x 1/2	25	6	12.80	326.40	1306	
Tie	Pls	2	23 5/8 x 3/8	2	0	30.125	60.24	120	
"	"	2	21 x 3/8	1	11 5/8	26.78	52.724	105	
"	"	1	28 3/8 x 3/8	2	10 5/8	36.18	104.394	104	
Spl.	"	2	4 5/8 x 1/2	2	0	7.865	15.730	31	
Guss	"	1	34 x 3/8	4	4 3/8	43.35	189.205	189	
Fills		2	8 1/2 x 1/2	0	11 1/2	14.45	13.847	28	
"		2	11 1/2 x 3/4	1	5	29.33	41.649	83	
"		2	8 1/2 x 3/8	1	1 1/4	10.84	11.970	24	
"		2	15 x 3/8	1	1 1/4	19.13	21.123	42	
"		2	8 1/2 x 3/8	2	2 1/2	10.84	23.938	48	
Spl	Pls	2	15 x 1/2	2	2 1/2	25.50	56.312	113	
Guss	"	2	34 x 5/8	4	7	72.25	331.143	662	
Lac Bars		32	2 1/2 x 1/2	2	8	4.25	11.333	363	
Diaph	Ls	8	3 x 3 x 3/8	1	4 1/2	7.20	9.900	79	
"	Pls	2	14 x 1/2	1	4 1/2	23.80	32.725	65	
								4,846 x 4 = 19,384	
			Lower Chord LC12 - LC14 4 Req'd.						
Web	Pls	2	16 1/2 x 3/8	25	6	21.04	536.52	1073	
Flg	Ls	4	4 x 4 x 3/8	25	6	9.80	249.90	1000	
Tie	Pls	2	23 5/8 x 3/8	2	0	30.13	60.26	121	
"	"	2	21 x 3/8	1	11 1/8	26.78	51.608	103	
"	"	1	28 3/8 x 3/8	2	10	36.18	102.509	103	
Spl	"	2	4 5/8 x 3/8	2	0	5.90	11.800	24	
Guss	"	1	34 x 3/8	4	4 3/8	43.35	189.205	189	
Fills		2	8 1/2 x 3/8	0	11 1/2	10.84	10.388	21	
"		2	11 1/2 x 5/8	1	5	24.44	34.624	69	
"		2	16 1/2 x 1/4	2	0 1/4	14.03	28.352	57	
"		2	15 x 1/4	1	1 1/4	12.75	14.079	28	
"		4	8 1/2 x 3/8	1	1 1/4	10.84	11.970	48	
Spl.	Pls	2	15 x 3/8	2	0 1/2	19.13	39.058	78	
Guss	"	2	34 x 3/8	4	3 5/8	43.35	182.885	366	
Lac Bars		32	2 1/2 x 1/2	2	8	4.25	11.333	363	
Diaph	Ls	8	3 x 3 x 3/8	1	4 1/2	7.2	9.90	79	
"	Pls	2	14 x 1/2	1	4 1/2	23.80	32.725	65	
								3,787 x 4 = 15,148	

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

Descr.		No.	Section in ins.	Length ft.	Length ins.	Wt in lbs One ft.	Wt. in lbs One piece	Tot Wt. in lbs.	Remarks
Lower Chord Lc14-Lc16 4 Req'd.									
Web	Pls	2	16 × 3/8	25	6	21.04	536.52	1073	
Flg	Ls	4	4 × 4 × 3/8	25	6	9.80	249.90	1000	
Tie	Pls	2	22 5/8 × 3/8	2	0	28.85	57.70	116	
"	"	2	21 × 3/8	1	11 1/8	26.78	57.608	103	
"	"	1	28 3/8 × 3/8	2	10	36.18	102.509	103	
Spl.	"	2	4 3/8 × 3/8	2	0	5.58	11.16	22	
Guss	"	1	34 × 3/8	4	4 3/8	43.35	189.205	189	
Fills		2	8 1/2 × 3/8	0	11 1/2	10.84	10.388	21	
"		2	11 1/2 × 3/8	1	5	14.66	20.769	42	
"		2	8 1/2 × 3/8	2	2 1/2	10.84	23.938	48	
Spl.	Pls	2	15 × 3/8	2	2 1/2	19.13	42.245	85	
Guss	"	2	29 × 3/8	3	8 5/8	36.98	137.521	275	
Lac. Bars		36	2 1/2 × 1/2	2	8	4.25	11.333	408	
Diaph.	Ls	8	3 × 3 × 3/8	1	4 1/2	7.20	9.90	79	
"	Pls	2	14 × 1/2	1	4 1/2	23.80	32.725	65	
								3,629 × 4 = 14,516	
Lower Chord Lc16-Lc18 4 Req'd.									
Web	Pls	2	16 1/2 × 3/8	25	5 3/4	21.04	536.082	1072	
Flg	Ls	4	4 × 4 × 3/8	25	5 3/4	9.80	249.696	999	
Tie	Pls	2	22 5/8 × 3/8	2	0	28.85	57.700	115	
"	"	2	21 × 3/8	1	10 5/8	26.78	50.491	101	
"	"	1	28 3/8 × 3/8	2	10	36.18	102.509	103	
Guss	"	1	34 × 3/8	3	1 1/2	43.35	135.469	136	
Fills		2	11 1/2 × 3/8	1	5	14.66	20.769	42	
"		2	8 1/2 × 3/8	0	11 1/2	10.84	10.388	21	
Pin	Pls	2	8 1/2 × 3/8	1	7	10.84	17.163	34	
"	"	2	15 × 5/8	1	6 3/4	31.88	49.813	100	
Guss	"	2	29 × 3/8	2	10	36.98	104.775	210	
Diaph.	Ls	8	3 × 3 × 3/8	1	4 1/2	7.2	9.90	79	
"	Pls	2	14 × 1/2	1	4 1/2	23.80	32.725	65	
								3,077 × 4 = 12,308	
Pin for Center of Span.									
Pins		2	4 1/2" ϕ	1	8 1/8	54.07	90.675	181	
Nuts		4					4.6	18	
								199	

増田淳氏関係資料
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CALCULATIONS FOR

Descri	No.	Section in ins.	Length ft.	Length ins.	Wt. in lbs One ft.	Wt in lbs One piece	Tot. Wt in. lbs.	Remarks.
Verticals Us1-Ls1 4 Req'd.								
Ls	4	4 x 3 x 5/16	12	1 1/4	7.2	87.15	349	
Pls	1	9 x 5/16	1	7	9.86	15.136	15	
"	1	12 1/2 x 5/16	1	2 1/2	13.28	16.046	16	
Fills	1	8 3/8 x 5/8	1	3 1/2	17.795	22.986	23	
Lac. Bars	9	2 1/4 x 5/16	1	1 5/8	2.39	2.714	24	
							427 x 4 = 1,708 ✓	
Vertical Us2-Ls2 4 Req'd.								
Ls	4	4 x 3 x 5/16	14	1 1/2	7.2	101.70	407	
Pls	1	12 1/2 x 5/16	1	8 1/2	13.28	22.686	23	
"	1	9 x 5/16	1	7	9.56	15.136	15	
Fill	1	8 3/8 x 5/8	1	9	17.795	31.141	31	
Lat. Bars	11	2 1/4 x 5/16	1	1 5/8	2.39	2.714	30	
							506 x 4 = 2,024 ✓	
Vertical Us3-Ls3 4 Req'd.								
Ls	4	4 x 3 x 5/16	14	7	7.2	105.00	420	
Pls	1	12 1/2 x 5/16	2	5 1/2	13.28	32.646	33	
Ls	2	3 x 3 x 3/8	0	11 3/4	7.2	7.05	14	
Fills	2	2 3/4 x 5/16	0	6 1/2	2.92	1.582	3	
"	1	8 3/8 x 5/8	1	3 1/2	17.795	22.988	23	
Pl	1	9 x 5/16	1	7	9.56	15.136	15	
Lac. Bars	10	2 1/4 x 5/16	1	1 5/8	2.39	2.714	27	
							535 x 4 = 2,140 ✓	
Vertical Us4-Ls4 4 Req'd.								
Ls	4	4 x 3 x 5/16	15	1	7.2	108.60	434	
Fill	1	8 3/8 x 5/8	2	3	17.795	40.039	40	
Pl	1	12 1/2 x 5/16	1	8 1/2	13.28	22.686	23	
"	1	9 x 5/16	1	7	9.56	15.136	15	
Lac. Bars	11	2 1/4 x 5/16	1	1 5/8	2.39	2.714	30	
							542 x 4 = 2,168 ✓	
Vertical Us5-Ls5 4 Req'd.								
Ls	4	4 x 3 x 5/16	16	1 1/2	7.2	116.10	464	
Fill	1	8 3/8 x 5/8	1	3 1/2	17.795	22.986	23	
Pl	1	12 1/2 x 5/16	2	11 1/2	13.28	39.286	39	
"	1	9 x 5/16	1	7	9.56	15.136	15	
Lac Bars	9	2 1/4 x 5/16	1	1 5/8	2.39	2.714	24	
L	1	3 x 3 x 3/8	0	8 1/2	7.2	5.100	5	
							570 x 4 = 2,280 ✓	

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Descr.	No.	Section in ins	Length ft	Length ins	Wt. in lbs One ft.	Wt. in lbs One piece	Tot. Wt. in lbs.	Remarks
Vertical Us6 – Ls6						4 Req'd.		
Ls	4	4 × 3 × 5/16	17	10	7.2	128.40	514	
Fill	1	8 3/8 × 5/8	1	3 1/2	17.795	22.986	23	
Pl	1	12 1/2 × 5/16	1	8 1/2	13.28	22.686	23	
"	1	9 × 5/16	1	7	9.56	15.136	15	
Lac. Bars	11	2 1/4 × 5/16	1	1 5/8	2.39	2.714	30	
							605 × 4 = 2,420	✓
Vertical Us7 – Ls7						4 Req'd.		
Ls	4	4 × 3 × 5/16	19	9 1/2	7.2	142.50	570	
Fill	1	8 3/8 × 5/8	1	3 1/2	17.795	22.986	23	
Pl.	1	12 1/2 × 5/16	2	11 1/2	13.28	39.286	39	
"	1	9 × 5/16	1	7	9.56	15.136	15	
Lac. Bars	9	2 1/4 × 5/16	1	1 5/8	2.39	2.714	24	
							671 × 4 = 2,684	✓
Vertical Us8 – Ls8						4 Req'd.		
Ls	4	4 × 3 × 5/16	19	11 1/4	7.2	143.550	574	
Fill	1	8 3/8 × 5/8	1	3 1/2	17.795	22.986	23	
Pl	1	12 1/2 × 5/16	2	0	13.28	26.560	27	
Lac. Bars	11	2 1/4 × 5/16	1	1 5/8	2.39	2.714	30	
							654 × 4 = 2,616	✓
Vertical Uc1 – Lc1						4 Req'd.		
Ls	2	4 × 3 × 5/16	19	9	7.2	142.20	284	
"	2	"	19	8 1/2	7.2	141.90	284	
Fill.	1	8 3/8 × 5/8	1	3 1/2	17.795	22.986	23	
Pl.	1	12 1/2 × 5/16	2	11 1/2	13.28	39.286	39	
"	1	9 × 5/16	1	7	9.56	15.136	15	
Lac. Bars	6	2 1/4 × 5/16	1	1 5/8	2.39	2.714	16	
							661 × 4 = 2,644	✓
Vertical Uc2 – Lc2								
Ls	2	4 × 3 × 5/16	17	10 1/2	7.2	128.70	257	
	2	"	17	10	7.2	128.40	257	
Fill	1	8 3/8 × 5/8	1	3 1/2	17.795	22.986	23	
Pl.	1	12 1/2 × 5/16	1	8 1/2	13.28	22.686	23	
"	1	9 × 5/16	1	7	9.56	15.136	15	
Lac. Bars	8	2 1/4 × 5/16	1	1 5/8	2.39	2.714	22	
							597 × 4 = 2,388	✓
Vertical Uc3 – Lc3						4 Req'd.		
Ls	4	4 × 3 × 5/16	16	5 1/2	7.2	118.50	474	
Fill	1	8 3/8 × 5/8	1	3 1/2	17.795	22.986	23	
Pl.	1	12 1/2 × 5/16	2	11 1/2	13.28	39.286	39	
"	1	9 × 5/16	1	7	9.56	15.136	15	
Lac Bars	6	2 1/4 × 5/16	1	1 5/8	2.39	2.714	16	
							567 × 4 = 2,268	✓

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Desc.	No.	Section in ins.	Length ft.	Length ins.	Wt. in lbs. One ft.	Wt. in lbs. One piece	Tot. Wt. in lbs.	Remarks
Vertical Uc4-Lc4 4 Req'd.								
Ls	4	4 x 3 x 5/16	15	0 1/2	7.2	108.30	433	
Fill	1	8 3/8 x 5/8	2	3 3/4	17.795	41.151	41	
Pl.	1	12 1/2 x 5/16	1	8 1/2	13.28	22.686	23	
"	1	9 x 5/16	1	7	9.56	15.136	15	
Lac. Bars	8	2 1/4 x 5/16	1	1 5/8	2.39	2.714	22	
							534 x 4 = 2,136 ✓	
Vertical Uc5-Lc5 4 Req'd.								
Ls	4	4 x 3 x 5/16	14	3	7.2	102.60	410	
Fill	1	8 3/8 x 5/8	1	3 1/2	17.795	22.986	23	
Pl.	1	12 1/2 x 5/16	2	11 1/2	13.28	39.286	39	
"	1	9 x 5/16	1	7	9.56	15.136	15	
Lac. Bars	6	2 1/4 x 5/16	1	1 5/8	2.39	2.714	16	
							503 x 4 = 2,012 ✓	
Vertical Uc6-Lc6 4 Req'd.								
Ls	4	4 x 3 x 5/16	14	1 1/2	7.2	101.70	407	
Fill	1	8 3/8 x 5/8	2	0 3/4	17.795	36.702	37	
Pl.	1	12 1/2 x 5/16	1	8 1/2	13.28	22.686	23	
"	1	9 x 5/16	1	7	9.56	15.136	15	
Lac Bars	8	2 1/4 x 5/16	1	1 5/8	2.39	2.714	22	
							504 x 4 = 2,016 ✓	
Vertical Uc7-Lc7 4 Req'd.								
Ls	4	4 x 3 x 5/16	14	1	7.2	101.4	406	
Fill	1	8 3/8 x 5/8	1	3 1/2	17.795	22.986	23	
Pl.	1	12 1/2 x 5/16	1	9	13.28	23.240	23	
"	1	9 x 5/16	1	7	9.56	15.136	15	
Lac. Bars	8	2 1/4 x 5/16	1	1 5/8	2.39	2.714	22	
							489 x 4 = 1,956	
Verticals Uc8-Lc8, Uc10-Lc10, Uc12-Lc12, Uc14-Lc14 ✓ Uc16-Lc16, Uc18-Lc18, 24 Req'd.								
Ls	4	4 x 3 x 5/16	14	1	7.2	101.40	406	
Pl	1	12 1/2 x 5/16	1	9	13.28	23.24	23	
"	1	9 x 5/16	1	7	9.56	15.136	15	
Lac Bars	8	2 1/4 x 5/16	1	1 5/8	2.39	2.714	22	
							466 x 24 = 11,184 ✓	
1 Req'd.								
Fill	3	8 3/8 x 5/8	2	0 3/4	17.795	36.702	110	
"	3	8 3/8 x 3/8	2	0 3/4	10.68	22.027	66	
							176 ✓	

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Desc.	NO.	Section in ins.	Length ft.	Length ins.	Wt. in lbs One ft.	Wt. in lbs One piece	Tot. Wt. in. lbs.	Remarks
		Verticals Uc9-Lc9, Uc11-Lc11, Uc13-Lc13 Uc15-Lc15, Uc17-Lc17, 20 Req'd.						
Ls	4	4 x 3 x 5/16	14	1	7.2	101.40	406	
Pl.	1	12 1/2 x 5/16	2	6	13.28	33.20	33	
Ls	2	3 x 3 x 3/8	1	0 1/4	7.2	7.35	15	
Fills	2	3 x 5/16	0	6 1/2	3.19	1.728	3	
Pl.	1	9 x 5/16	1	7	9.56	15.136	15	
Lac. Bars	7	2 1/4 x 5/16	1	1 5/8	2.39	2.714	19	
							491 x 20 = 9.820 ✓	
						1 Req'd.		
Fills	3	8 3/8 x 5/8	1	3 1/2	17.795	22.986	69	
	2	8 3/8 x 3/8	1	3 1/2	10.68	13.795	28	
							97 ✓	

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CALCULATIONS FOR

Description	No	Section in ins	Length ft	Length ins	Wt in lbs one ft	Wt in lbs one piece	Total wt in lbs	Remarks
Diagonal Us4—Ls2 4 Req'd								
Ls	4	6×4×½	15	4½	16.20	249.075	996	
Pls	2	10½×⅜	1	3	15.94	19.925	40	
Fills	4	3¾×¼	1	3	3.188	3.985	16	
LaH Bars	30	2½×5/16	1	0	2.66	2.660	80	
							1132×4=4528	
Diagonal Ls2—Us3 4 Req'd								
Ls	4	6×4×½	15	4½	16.20	249.156	997	
Pls	2	10½×⅜	1	3	15.94	19.925	40	
Fills	4	3¾×¼	1	3	3.188	3.995	16	
LaH Bars	30	2½×5/16	1	0	2.66	2.66	80	
							~~1249×4=4996~~	
							1133×4=4532	
Diagonal Us3—Ls4 4 Req'd								
Ls	2	6×4×⅜	15	10½	12.30	195.263	391	
"	2	" " "	16	1½	12.30	198.338	397	
Cov Pls	2	12¾×⅜	16	4½	16.26	266.258	533	
Pls	2	11¾×⅜	1	5	14.98	18.725	37	
Fills	4	3¾×¼	1	3	3.188	3.985	16	
LaH Bars	32	2½×5/16	0	11	2.66	2.438	78	
							1452×4=5808	
Diagonal Ls4—Us5 4 Req'd								
Ls	2	6×4×⅜	15	9	12.30	193.725	387	
"	2	" " "	16	0	12.30	196.800	394	
Cov Pls	2	14×⅜	16	0	17.85	285.600	571	
Pls	2	11¾×⅜	1	9	14.98	26.215	52	
Fills	4	3¾×¼	1	9	3.188	5.579	22	
LaH Bars	33	2½×5/16	0	11	2.66	2.438	80	
							1506×4=6024	
Diagonal Us5—Ls6 4 Req'd								
Ls	4	6×4 3/8	17	9½	12.30	218.838	875	
Cov Pls	2	14×½	17	9½	23.80	423.440	847	
Pls	2	11½×⅜	2	3	14.66	32.985	66	
Fills	4	3¾×¼	2	3	3.188	7.173	29	
							1817×4=7268	

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Description	No	Section in ins	Length ft	Length ins	Wt in lbs one ft	Wt in lbs one piece	Total wt in lbs	Remarks
			Diagonal	Ls6—Us7	4 Req'd			
Ls	2	6×4 × 3/8	17	11½	12.30	220.887	442	
"	2	" " "	18	2½	12.30	223.962	448	
Cov Pls	2	14 × ½	18	2½	23.80	433.358	867	
Pl	2	11½ × 3/8	2	0	14.66	29.320	59	
Fills	4	3¾ × ¼	2	0	31.88	6.376	26	
LaH Bars	42	2½ × 5/16	0	10¾	2.66	2.383	100	
							1942 ×4 = 7.768 ✓	
		Diagonal	Us7—Ls8	4Req'd				
Ls	2	6×4 × 9/16	21	8½	18.10	392.920	786	
"	2	" " "	21	11½	18.10	397.445	795	
Cov Pls	2	14 × ½	21	11½	23.80	522.608	1045	
Pls	2	14 × ½	2	10½	23.80	68.425	137	
Fills	2	14 × 5/8	1	0	29.75	29.750	60	
Pls	2	11½ × 3/8	3	3	14.66	47.645	95	
Fills	4	3¾ × ¼	3	3	3.188	10.364	41	
"	2	12 × 5/8	1	7½	25.50	41.438	83	
Pls	2	14 × ½	3	1	23.80	73.383	147	
LaH Bars	45	2½ × 5/16	0	10¾	2.66	2.383	107	
							3296 ×4 = 13.184 ✓	
		Diagonal	Ls8—Uc1	4Req'd				
Ls	2	6×4 × 3/8	21	4½	12.30	262.913	526	
"	2	" " "	21	10½	12.30	269.063	538	
Cov Pls	2	14 × 3/8	21	10½	17.85	390.469	781	
Pls	2	11¾ × 3/8	2	3	14.98	33.705	67	
Fills	4	3¾ × ¼	2	3	3.188	7.173	29	
LaH Bars	47	2½ × 5/16	0	11	2.66	2.438	115	
							2056 × 4 = 8.224 ✓	
		Diagonal	Uc1—Lc2	4Req'd				
Ls	4	6×4 × 3/8	18	2½	12.30	223.962	896	
Cov Pls	2	12¾ × 3/8	18	2½	16.26	296.067	592	
Pls	2	11¾ × 3/8	2	0	14.98	29.960	60	
Fills	4	3¾ × ¼	2	0	3.188	6.376	26	
LaH Bars	39	2½ × 5/16	0	11	2.66	2.438	95	
							1669 × 4 = 6.676 ✓	

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Description	No	Section in ins	Length ft	Length ins	Wt in lbs one ft	Wt in lbs one piece	Total wt in lbs	Remarks
Diagonal Lc2—Uc3 4 Req'd								
Ls	4	6×4×3/8	17	9½	12.30	218.838	875	
Cor Pls	2	12¾×3/8	17	9½	16.26	289.293	579	
Pls	2	11¾×3/8	2	3	14.98	33.705	67	
Fills	4	3¾×¼	2	3	3.188	7.173	29	
LaH Bars	36	2½×5/16	0	11	2.66	2.438	89	
							1039×4=6556	
Diagonal Uc3—Lc4 4 Req'd								
Ls	4	6×4×½	16	0	16.20	259.200	1037	
Pls	2	12½×3/8	1	9	1594	27.895	56	
Fills	4	3¾×¼	1	9	3.188	5.579	22	
LaH Bars	31	2½×5/16	1	0	2.66	2.660	82	
							~~1318×4=5272~~ 1.197×4=4,788	
Diagonal Lc4—Uc5 4 Req'd								
Ls	4	6×4×½	15	10¾	16.2	257.580	1030	
Pls	2	12½×3/8	1	9	15.94	27.895	56	
Fills	4	3¾×¼	1	9	3.188	5579	22	
LaH Bars	30	2½×5/16	1	1⅝	2.66	3.020	91	
							~~1320×4=5280~~ 1,199×4=4,796	
Diagonal (Uc5—Lc6)—(Lc6—Uc7) 8 Req'd								
Ls	4	6×4×7/16	15	5½	14.30	221.078	884	
Pls	2	12½×3/8	1	6	15.94	23.910	48	
Fills	4	3¾×¼	1	6	3.188	4.782	19	
LaH Bars	30	2½×5/16	1	1⅝	2.66	3.020	91	
							~~1160×8=9280~~ 1,042×8=8,336	
Diagonal Uc7—Lc8 4 Req'd								
Ls	4	6×4×3/8	15	5½	12.30	190.158	761	
Pls	2	12½×3/8	1	6	15.94	23.910	48	
Fills	4	3¾×¼	1	6	3.188	4.782	19	
LaH Bars	30	2½×5/16	1	1⅝	2.66	3.020	91	
							~~1042×4=4168~~ 919×4=3,676	

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CALCULATIONS FOR

Description	No	Section in ins	Length ft	Length ins	Wt in lbs one ft	Wt in lbs one piece	Total wt in lbs	Remarks
		Diagonal (Lc8–Uc9)(Uc9–Lc10)(Lc10–Uc11)						12 Req'd
Ls	4	6×4 × 3/8	15	5½	12.30	190.130	761	
Pls	2	10½ × 3/8	1	3	15.97	19.925	40	
Fills	4	3¾ × 1/4	1	3	3.188	3.985	16	
Lat Bars	30	2½ × 5/16	0	11	2.66	2.438	78	
							890 × 12 = 10680 ✓	
		Diagonal (Uc11–Lc12)(Lc12–Uc13)						8 Req'd
Ls	4	5×3½×3/8	15	4½	10.40	159.900	640	
Pls	2	10½ × 3/8	1	3	15.94	19.925	40	
Fills	4	3¼ × 1/4	1	3	3.188	3.985	16	
Lat Bars	30	2½ × 5/16	1	1¼	2.66	2.937	88	
							784 × 8 = 6272 ✓	
		Diagonal (Uc13–Lc14) To (Lc16–Uc17)						16 Req'd
Ls	4	5×3×3/8	15	4½	9.80	150.675	603	
Pls	2	10½ × 3/8	1	3	15.94	19.925	40	
Fills	4	2¾ × 1/4	1	3	3.188	3.985	16	
Lat Bars	30	2½ × 5/16	1	1¼	2.66	2.937	88	
							747 × 16 = 11,952 ✓	
		Diagonal Uc17 — Lc18						4 Req'd
Ls	4	5×3×3/8	14	8	9.80	143.734	575	
Pls	2	10½ × 3/8	1	3	15.94	19.925	40	
Fills	4	2¾ × 1/4	1	3	3.188	3.985	16	
Lat Bars	30	2½ × 5/16	1	1¼	2.66	2.937	88	
							719 × 4 = 2876 ✓	
Ring Fills	200	3"φ × 5/16			0.625	125.000	125	2 Req'd for one member for Diagonal
							125	
		Stiffening truss					~~863.253#÷2240=385.381 Tons~~	
							854.907 ÷ 2240 = 381.655	

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CALCULATIONS FOR

Description	No	Section in ins	Length ft	Length ins	Wt in lbs one ft	Wt in lbs one piece	Total wt in lbs	Remarks
		Bott. Lateral Bracing Ls0—Ls2 2Req'd						
Ls	4	6×6×7/16	13	8	17.20	235.072	940	
"	4	" " "	14	5	"	247.972	992	
Guss Pl	1	41×3/8	3	5½	52.28	180.800	181	
							2.113×2=4.226	
		Bott Lateral Bracing Ls2—Ls4 2Req'd						
Ls	4	6×6×7/16	14	2	17.20	243.724	975	
"	4	" " "	14	8	"	252.324	1.009	
Guss Pl	1	41×3/8	3	5½	52.28	180.800	181	
							2.165×2=4.330	
		Bott Lateral Bracing Ls4—Ls6 2Req'd						
Ls	4	6×6×½	14	1	19.60	275.968	1.104	
"	4	" " "	14	10	"	290.668	1.163	
Guss Pls	1	43×3/8	3	8	54.85	201.045	201	
							2.468×2=4.936	
		Bott Lateral Bracing Ls6—Ls8 2Req'd						
Ls	4	6×6×½	14	1¼	19.60	276.360	1.105	
"	4	" " "	14	10¼	"	296.060	1.184	
Guss Pl	1	47×3/8	4	0	59.93	239.720	240	
							2.509×2=5.018	
		Bott Lateral Bracing Ls8—Lc2 2Req'd						
Fills	8	4¾×½	1	8	8.08	13.467	108	
Ls	4	6×4×½	13	11¾	16.20	226.463	906	
"	4	" " "	14	7¼	16.20	236.588	946	
Guss Pl	1	48×3/8	4	3	61.20	260.100	260	
Ring Fills	28	3"ϕ×½			1.0	1.000	28	
							2248×2=4496	
		Bott Lateral Bracing Lc2—Lc4 2Req'd						
Ls	4	6×4×7/16	14	0½	14.30	200.772	803	
"	4	" " "	14	5½	"	206.778	827	
Fills	8	4¾×½	1	8	8.08	13.467	108	
Guss Pl	1	46×3/8	4	0	58.65	234.600	235	
Ring Fills	28	3"ϕ×½			1.0	1.000	28	
							2.001×2=4.002	

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Description	No	Section in	Length ins	Length ft.	Length ins	Wt in lbs one ft	Wt in lbs each piece	Total wt in lbs	Remarks
Bott. Lateral Bracing Lc4–Lc6 2Req'd									
	Ls	4	6×4×⅜	13	11½	12.30	177.708	687	
	"	4	" " "	14	7	(1 "	179.334	717	
Guss	Pl	1	44×⅜	3	10½	56.10	217.388	217	
Fills		8	4¾×½	1	8	8.08	13.467	107	
Ring	Fills	28	3"Φ×½			1.00	1.000	28	
								1.756×2=3.512	
Bott Lateral Bracing Lc6—Lc8 2Req'd									
	Ls	4	5×3½×⅜	13	11	10.40	144.734	579	
	"	4	" " "	14	6½	10.40	151.234	605	
Guss	Pl	1	38×⅜	3	2½	48.45	155.442	155	
Fills		8	4¾×½	1	0½	8.08	8.417	67	
Ring	Fills	28	3"Φ×½			1.00	1.000	28	
								1434×2=2868 ✓	
Bott Lateral Bracing Lc8—Lc10 2Req'd									
	Ls	2	5×3½×⅜	14	3½	10.40	148.616	297	
	"	4	" " "	14	1	"	146.432	586	
	"	2	" " "	14	6	"	150.800	302	
Guss	Pl	1	32×⅜	2	9	40.80	112.200	112	
Fills		8	5×½	1	1	8.50	9.208	74	
Ring	Fills	28	3"Φ×½			1.00	1.000	28	
								1399×2=2.798 ✓	
Bott Lateral Bracing (Lc10–Lc12) to (Lc14–Lc16) 6Req'd									
	Ls	4	5×3½×⅜	14	6	8.70	126.150	505	
	"	4	" " "	14	1	8.70	122.525	490	
Guss	Pl	1	32×⅜	2	9	40.80	112.200	112	
Fills		8	5×½	1	1	8.50	9.208	74	
Ring	Fills	28	3"Φ×½			1.00	1.000	28	
								1209×6=7254 ✓	
Bott Lateral Bracing Lc16—Lc18 2Req'd									
	Ls	2	5×3½×⅜	14	6	8.70	126.150	252	
	"	2	" " "	14	1	8.70	122.525	245	
	"	2	" " "	13	9½	8.70	119.988	240	
	"	2	" " "	13	4½	8.70	116.363	233	
Guss	Pl	1	32×⅜	2	9	40.80	112.200	112	
Fills		8	5×½	1	1	8.50	9.208	74	
Ring	Fills	28	3"Φ×½			1.00	1.000	28	
								1184×2=2368 ✓	

Tons

Bottom lateral Bracings ~~41.942 #12240=18.724~~
45,808 ÷ 224 00 = 200.45.00

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Desc.	No.	Section in ins.	Length ft.	Length ins.	Wt in lbs. One ft.	Wt. in lbs. One piece	Tot. Wt. in lbs.	Remarks
Floor Beam FBSOL 2 Req'd.								
Web Pl	2	19½ × 5/16	3	0½	20.72	63.030	126	
Web Pl.	1	27 × 5/16	17	6¾	28.69	502.534	503	
Flg. Ls	2	5 × 3½ × 3/8	20	2½	8.5	171.768	344	
" "	2	"	19	7¼	"	166.634	333	
Stiff. "	14	3 × 3 × 5/16	2	2¾	6.10	13.598	190	
Conn. "	4	4 × 3½ × 3/8	1	10¹⁵/₁₆	9.10	15.632	63	
Fills	4	3½ × 3/8	1	4½	4.46	5.945	24	
Ls	4	5 × 3½ × 3/8	1	11	8.5	16.273	65	
"	4	3 × 3 × 3/8	0	6½	7.20	2.900	16	
"	4	"	1	1½	7.20	8.100	32	
Pls	4	5½ × 5/16	1	8¾	5.84	7.858	39	
							1,735 × 2 = 3470	
Floor Beam FBS1L 2 Req'd.								
Web Pl.	1	27 × 5/16	20	10½	28.69	598.904	599	
Flg Ls	2	5 × 3½ × 3/8	20	3½	10.40	211.033	422	
" "	2	"	20	2	"	209.734	419	
Stiff. "	14	3 × 3 × 5/16	2	2¾	6.10	13.598	190	
Conn. "	4	4 × 3½ × 3/8	1	3	9.10	11.375	46	
" "	4	"	1	7⅜	"	14.693	59	
Fills	4	3½ × 3/8	1	4½	4.46	6.133	25	
Ls	4	3 × 3 × 3/8	0	6½	7.2	3.900	16	
"	4	"	0	10½	"	6.300	25	
Pls	2	9 × 5/16	1	2	9.56	11.154	22	
"	2	12½ × 5/16	3	11¾	13.28	52.844	106	
Ls	4	3 × 3 × 3/8	1	0	7.2	7.2	29	
Fills	4	2¾ × 5/16	0	6½	2.92	1.582	6	
							1,964 × 2 = 3,928 ✓	
Floor Beam FBS2L 2 Req'd.								
Web Pl	1	27 × 5/16	20	9¼	28.69	595.914	596	
Flg. Ls	2	5 × 3½ × 3/8	20	2½	10.40	209.950	420	
" "	2	" " "	20	0	"	208.000	416	
Stiff. Ls	14	3 × 3 × 5/16	2	2¾	6.10	13.598	190	
Conn. Ls	4	4 × 3½ × 3/8	1	3⅜	9.10	11.470	46	
" "	4	" " "	1	7⅜	"	14.693	59	
Fills.	4	3½ × 3/8	1	4½	4.46	6.133	25	
Ls	4	3 × 3 × 3/8	0	6½	7.20	3.900	16	
"	4	" " "	0	10½	"	6.300	25	
Pls.	2	9 × 5/16	1	2	9.56	11.154	22	
"	2	12½ × 5/16	3	11¾	13.28	52.844	106	
Ls.	4	3 × 3 × 3/8	1	0	7.2	7.2	29	
Fills.	4	2¾ × 5/16	0	6½	2.92	1.582	6	
							1956 × 2 = 3912 ✓	

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Desc.		No.	Section in ins.	Length ft.	Length ins.	Wt. in lbs. One ft.	Wt. in lbs. One piece	Tot. Wt. in lbs.	Remarks
			Floor Beam FB33				2 Req'd.		
Web	Pls.	1	32½ × 5/16"	20	10½	34.53	720.814	721	
Flg.	Ls.	2	5 × 3½ × 3/8	20	3½	10.40	211.034	422.	
"	"	2	" " "	20	1/4	"	209.084	418.	
Stiff	Ls	14	3 × 3 × 5/16	2	8¼	6.10	16.394	230	
Con.n.	Ls	4	4 × 3½ × 3/8	1	3	9.1	11.375	46.	
"	"	4	" " "	2	0⅝	"	18.674	75	
Fills.		4	3½ × 3/8	1	9¾	4.46	8.084	32	
	Ls	4	3 × 3 × 3/8	0	6½	7.2	3.900	16	
	"	4	" " "	0	10½	"	6.300	25	
	Pls.	2	9 × 5/16	1	2	9.66	11.154	22	
	"	2	12½ × 5/16	4	3	13.28	56.440	113	
	Ls	4	3 × 3 × 3/8	1	0	7.2	7.200	29.	
Fills.		4	2¾ × 5/16	0	6½	2.92	1.582	6	
								2.155 × 2 = 4.310	
			Floor Beam FB34 2 Req'd.						
Web	Pl	1	38 × 5/16	20	9¼	40.38	838.725	839	
Flg.	Ls	2	5 × 3½ × 3/8	20	2¼	10.40	209.950	419	
"	"	2	" " "	19	9	"	205.400	411	
Stiff.	Ls	14	3 × 3 × 5/16	3	1¾	6.10	19.189	269	
Con.n	Ls	4	4 × 3½ × 3/8	1	3½	9.10	11.755	47	
"	"	4	" " "	2	6⅜	"	22.845	91	
Fills.		4	3½ × 3/8	2	3¼	4.46	10.128	40	
	Ls	4	3 × 3 × 3/8	0	6½	7.20	3.900	16	
	"	4	" " "	0	10½	"	6.300	25	
	Pls.	2	9 × 5/16	1	2	9.56	11.154	22	
	"	2	12½ × 5/16	4	9½	13.28	63.634	127	
	Ls	4	3 × 3 × 3/8	1	0	7.2	7.200	29	
Fills.		4	2¾ × 5/16	0	6½	2.92	1.582	6	
								2.341 × 2 = 4.682	
			Floor Beam FB35 2 Req'd						
Web	Pl.	1	27 × 5/16	20	10½	28.69	598.904	599	
Flg.	Ls	2	5 × 3½ × 3/8	20	3½	10.40	211.033	422	
"	"	2	" " "	20	10¼	"	217.100	434	
Stiff.	Ls	14	3 × 3 × 5/16	2	3¾	6.10	14.106	198	
Con.n	Ls	4	4 × 3½ × 3/8	3	0¼	9.10	27.489	110	
Fills.		4	3½ × 3/8	2	7	4.46	11.522	46	
	Ls	4	3 × 3 × 3/8	0	6½	7.2	3.900	16	
	"	4	3 × 3 × 3/8	0	10½	"	6.300	25	
	Pls.	2	9 × 5/16	1	2	9.56	11.154	22	
	"	2	12½ × 5/16	6	1½	13.28	81.340	163	
	Ls	4	3 × 3 × 3/8	1	0	7.2	7.200	29	
Fills.		4	2¾ × 5/16	0	6½	2.92	1.582	6	
								2.070 × 2 = 4.140	

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Desc.	No.	Section in ins.	Length ft.	Length ins.	Wt. in lbs. One ft.	Wt. in lbs. One piece	Tot. Wt. in lbs.	Remarks
		Floor Beam FB56 2 Req'd.						
Web Pl.	1	27 × 5/16	20	10½	28.69	598.904	599	
Flg. L	2	5 × 3½ × 3/8	20	3½	10.40	284.080	568	
" "	2	" " "	20	10¼	"	291.959	584	
Stiff. L	14	3 × 3 × 5/16	2	2¾	6.10	13598	190	
Con. n. L	4	4 × 3½ × 3/8	3	0¼	9.10	27.489	110	
Fills.	4	3½ × 3/8	2	7	4.46	11.521	46	
L	4	3 × 3 × 3/8	0	6½	7.20	3.900	16	
"	4	" " "	0	10½	"	6.300	25	
Pls.	2	9 × 5/16	1	2	9.56	11.154	22	
"	2	12½ × 5/16	6	9½	13.28	90.194	180	
L	4	3 × 3 × 3/8	1	0	7.2	7.200	29	
Fills.	4	2¾ × 5/16	0	6½	2.92	15.820	6	
L	4	5 × 3½ × 3/8	2	0¼	10.40	21.016	84	Strut.
"	4	" " "	1	4	"	13.866	56	"
"	4	3½ × 3½ × 3/8	2	2¾	8.5	18.948	76	"
"	4	4 × 3½ × 3/8	1	3¼	9.1	11.564	46	"
Pls.	2	22¼ × 5/16	2	2⅜	23.65	51.980	104	"
Fills.	4	3½ × 3/8	1	8⅜	4.46	7.666	31	"
"	4	" "	1	7⅜	"	7.294	29	"
							2.801	× 2 = 5.602
		Floor Beam FB57 2 Req'd.						
Web Pl.	1	27 × 5/16	20	10½	28.69	598.904	599	
Flg. L	2	5 × 3½ × 3/8	20	3½	10.40	211.034	422	
" "	2	" " "	20	10¼	"	216,884	584	
Stiff. L	14	3 × 3 × 5/16	2	2¾	6.10	13.598	190	
Con.n L	4	4 × 3½ × 3/8	3	0¼	9.10	27.489	110	
Fills. /	4	3½ × 3/8	2	7	4.46	11.521	46	
L	4	3 × 3 × 3/8	0	6½	7.20	3.900	16	
"	4	" " "	0	10½	"	6.300	25	
Pls.	2	9 × 5/16	1	2	9.56	11.253	23	
"	2	12½ × 5/16	4	11	13.28	65.294	131	
"	2	" "	1	6	"	19.920	40	
L.	4	3 × 3 × 3/8	1	0	6.1	6.100	24	
Fills.	4	2¾ × 5/16	0	6½	2.92	1.582	6	
L	8	4 × 3 × 5/16	5	10½	7.2	42.300	338	Laterals.
Con.n. L	4	5 × 3½ × 3/8	1	10½	10.4	19.500	78	
" "	4	" " "	1	0⅛	"	10.508	42	
Guss. Pls.	2	13 × 5/16	1	0¼	13.81	14.097	28	
Con.n L	2	5 × 3½ × 3/8	2	7⅛	10.4	27.144	54	
Guss. Pl	1	13 × 5/16	2	7⅛	13.81	36.395	36	
Latt. Bars	10	2 × 5/16	1	1⅛	2.13	2.418	24	
							2.816	× 2 = 5.632

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Desc.	No.	Section in ins.	Length in	Length ins	Wt. in lbs. One ft.	Wt. in lbs. One piece	Tot. Wt. in lbs.	Remarks
		Floor Beam FB38 2 Req'd.						
Web Pl.	1	27 × 5/16	20	10½	28.69	598.903	599	
Flg. L	2	5×3½×7/16	20	3½	10.4	211.033	422	
" "	2	" " "	20	10¼	"	216.883	434	
Stiff. L	14	3×3×5/16	2	2¾	6.1	13.598	190	
Con.n. L	4	4×3½×7/16	3	0¼	9.1	27.489	110	
Fills.	4	3½ × 7/16	2	7	4.46	11.521	46	
L	4	3×3×7/16	0	6½	7.20	3900	16	
"	4	" " "	0	10½	"	6.300	25	
Pls.	2	9 × 5/16	1	2	9.56	11.154	22	
"	2	12½ × 5/16	3	10	13.28	50.906	102	Vertical.
"	2	13½ × 5/16	1	10½	14.34	26.888	54	
"	2	12½ × 5/16	2	0	13.28	26.560	53	
L	8	4×3×5/16	7	10½	7.2	56.700	454	Laterals.
"	4	3½×3½×7/16	0	9½	8.5	6.729	27	
"	2	" " "	2	0	"	17.000	34	
Pl.	1	13 × 5/16	2	0	13.81	27.620	28	
Latt. Bars.	8	2¼ × 5/16	1	1⅝	2.39	2.758	22	Vertical.
Ring. Fills.	12	3"φ × 5/16				0.500	6	
							2644 × 2 = 5288.	
		Floor Beam FBC1 2 Req'd.						
Web Pl.	1	27 × 5/16	18	4	28.69	525982	526	
" "	2	36 × 5/16	4	0¾	38.25	155390	311	
Flg. L	2	5×3½×7/16	20	3½	12.00	243.504	487	
" "	2	" " "	30	1½	"	361.500	723	
Stiff. L	14	3×3×5/16	2	2¾	6.1	13.598	190	
Con.n. L	8	4×3½×7/16	3	0¼	9.1	27.489	220	both. sides.
Fills.	8	3½ × 7/16	2	6¾	4.46	11.428	91	
L	4	3×3×7/16	0	6½	7.20	3.900	16	
"	4	5×5×7/16	1	0	12.30	12.300	49	
Pls.	2	9 × 5/16	3	7½	9.56	34.655	69	
Spl. Pls.	4	6 × 7/16	1	8	7.65	12.750	51	
L	4	3×3×7/16	1	0¾	7.2	7.650	31	
"	4	5×3½×7/16	1	1¾	10.4	11.916	48	
"	4	" " "	1	6¾	"	16.250	65	
"	4	3×3×5/16	4	8¾	6.1	28.848	150	
Pls.	2	9 × 5/16	1	0½	9.56	9.958	20	Vertical.
Spl. Pls.	4	6 × 7/16	1	6	7.65	11.475	46	" "
Fills.	4	6 × 7/16	0	6	"	3.825	15	"
L	4	3½×3½×7/16	1	0	7.2	7.200	29	"
"	4	5×3½×7/16	1	0	10.4	10.400	42	"
Fills.	4	3½ × 7/16	0	6	4.46	2.230	9	"
Pls.	2	12½ × 5/16	1	6½	13.28	20.473	41	"
L	8	4×3×5/16	6	11	8.5	50.292	402	
"	4	4×3×5/16	6	4	7.2	45.600	182	Laterals.
"	4	" " "	5	10½	"	42.300	169	"
"	2	5×3½×7/16	2	7⅝	10.4	27.408	55	"
Pl.	1	13 × 5/16	2	7⅝	13.81	36.395	36	"
Latt. Bars.	16	24 × 5/16	1	1⅝	2.39	2.714	43	Verticals.
							4.116 × 2 = 8.232.	

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Desc.		NO.	Section in ins.		Length ft.	Length ins	Wt. in lbs one ft.	Wt. in lbs one Piece	Tot. Wt. in lbs	Remarks
			Floor Beam FBC2					2 Req'd.		
Web	Pl.	1	27 ×	5/16	18	4	28.69	525.982	526	
"	"	2	36 ×	5/16	4	0¼	38.25	155390	311	
Flg.	L	2	5 × 3½ ×	7/16	20	3½	12.00	243.504	487	
"	"	2	" "	"	30	½	"	361.500	723	
Stiff.	L	14	3 × 3 ×	5/16	2	2¼	6.1	13.598	190	
Conn	L	8	4 × 3½ ×	5/16	3	0¼	9.1	27.489	220	both sides
Fills.		8	3½ ×	5/16	2	6¼	4.46	11.428	91	
	L	4	3 × 3 ×	5/16	0	6½	7.20	3900	16	
	"	4	5 × 5 ×	5/16	1	0	12.30	12.300	49	
	Pls.	2	9 ×	5/16	3	½	9.56	34.655	69	
Spl.	Pl.	4	6 ×	5/16	1	8	7.65	12.750	51	
	L	4	3 × 3 ×	5/16	1	0¼	7.2	7.650	31	
	"	4	5 × 3½ ×	5/16	1	1¼	10.4	11.916	48	
	"	4	" " "	"	1	6¼	"	16.250	65	
	Pl.	2	9 ×	5/16	1	0½	9.56	9.958	20	Vertical
Spl.	Pls.	4	6 ×	5/16	1	6	7.65	11.475	46	"
	L	4	3 × 3 ×	5/16	4	8¼	6.1	28.848	115	"
	"	8	4 × 3 ×	5/16	3	8½	8.5	31.255	250	"
	Pl.	2	12½ ×	5/16	2	10½	13.28	38.180	76	"
	L	4	3½ × 3½ ×	5/16	1	0	7.2	7.200	29	"
	Pl	4	6 ×	5/16	1	3	7.65	9.562	38	"
	L	4	5 × 3½ ×	5/16	1	0	10.4	10.400	42	"
Fills.		4	3½ ×	5/16	0	6	4.46	2.230	9	"
	L	4	4 × 3½ ×	5/16	2	0¼	9.1	18.389	74	Strut.
	"	4	" " "	"	1	4	"	12.133	49	"
	"	4	3½ × 3½ ×	5/16	2	2⅛	7.2	15.825	63	"
	"	4	4 × 3½ ×	5/16	1	3¼	9.1	11.564	46	"
	Pl	2	22¼ ×	5/16	2	2⅛	23.65	51.980	104	"
Fills.		4	3½ ×	5/16	1	8⅛	4.46	7.666	31	"
"		4	"	5/16	1	7⅛	"	7.294	29	"
Latt. Bars		6	24 ×	5/16	1	1⅛	2.39	2.714	16	"
									3.914 × 2 = 7.828	
			Floor Beam FBC3					2 Req'd.		
Web	Pl.	1	27 ×	5/16	18	4	28.69	525.974	526	
"	"	2	36 ×	5/16	4	0¼	38.25	155.391	311	
Flg.	L	2	5 × 3½ ×	7/16	20	3½	12.00	243.504	487	
"	"	2	" "	"	30	½	"	361.500	723	
Stiff.	L	14	3 × 3 ×	5/16	2	2¼	6.1	13.598	190	
Con.n	L	8	4 × 3½ ×	5/16	3	0¼	9.1	27.489	200	
Fills.		8	3½ ×	5/16	2	6¼	4.46	11.429	91	
	L	4	3 × 3 ×	5/16	0	6½	7.2	3.900	16	
	"	4	5 × 5 ×	5/16	1	0	12.3	12.300	49	
Pls.		2	9 ×	5/16	3	7½	9.56	34.656	69	
Spl.	Pls.	4	6 ×	5/16	1	8	7.65	12.750	51	
	L	4	3 × 3 ×	5/16	4	8¼	6.1	28.848	115	
	"	4	3 × 3 ×	5/16	1	0¼	7.2	7.650	31	
	"	4	5 × 3½ ×	5/16	1	1¼	10.4	11.916	48	
	"	4	" "	"	1	6¼	"	16.250	65	

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CALCULATIONS FOR

Desc.	No.	Section in ins.	Length ft.	Length ins	Wt. in lbs one ft.	Wt. in lbs one piece	Tot. Wt. in lbs.	Remarks.
Pls.	2	9 × 5/16	1	0½	9.56	9.959	20	
Spl. Pls.	4	6 × 3/8	1	6	7.65	11.475	46	
Ls	8	4 × 3 × 3/8	2	3⅝	8.5	19.568	157	
"	4	3½ × 3½ × 5/16	1	0	7.2	7.200	29	
"	4	5 × 3½ × 5/16	1	0	10.4	10.400	42	
Pls.	4	6 × 3/8	1	3	7.65	9.563	38	
"	2	12½ × 5/16	1	6	13.28	19.920	40	
Fills.	4	3½ × 5/16	0	6	4.46	2.230	9	
Latt. Bars.	6	2¼ × 5/16	1	1⅝	2.39	2.714	16	
						3369	× 2 = 6,738	

Floor Beam FBC4 2 Req'd.

Web	Pl.	1	38 × 5/16	20	9½	40.38	839.569	840	
Flg.	Ls	2	5 × 3½ × 7/16	20	2¼	12.00	242.256	485	
"	"	2	" " "	19	10	"	237.996	476	
Stiff.	Ls	14	3 × 3 × 5/16	3	1¾	6.1	19.189	269	
Con.n	Ls	8	4 × 3½ × 3/8	2	6⅜	9.1	22.845	183	
"	"	8	" " "	1	3½	"	11.755	94	
Fillş.		8	3½ × 3/8	2	3¾	4.46	10.314	83	
	Ls	4	3 × 3 × 3/8	0	6½	7.20	3.900	16	
	"	4	5 × 5 × 3/8	1	0	12.30	12.300	49	
	Pls.	2	9 × 5/16	3	7½	9.56	34.656	69	
	Ls	4	3 × 3 × 5/16	4	8¾	6.1	28.848	115	strut.
	"	4	3 × 3 × 3/8	1	0¾	7.2	7.650	31	
	"	4	5 × 3½ × 3/8	1	1¾	10.4	11.916	48	
	"	4	" " "	1	6¾	"	16.250	65	
Flg.	Ls	4	" " "	3	10½	"	40.300	161	
Web	Pls.	2	21⅜ × 5/16	3	2	22.71	71.916	144	
	Pls.	2	9 × 5/16	1	0½	9.56	9.959	20	Vertical.
Spl.	Pls.	4	6 × 3/8	1	8	7.65	12.750	51	"
"		2	12½ × 5/16	3	2	13.28	42.054	84	
	Ls	4	5 × 3½ × 3/8	1	0	10.4	10.400	42	
Fillş.		4	3½ × 3/8	0	6	4.46	2.230	9	
Tie	Pls.	2	10¼ × 5/16	3	8	17.43	63.911	128	
Latt.	Bars.	6	2¼ × 5/16	1	1⅝	2.39	2.714	16	
							3.415	× 2 = 6,830	

増田淳氏関係資料
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CALCULATIONS FOR

Desc.		No.	Section in ins.	Length ft.	Length ins.	Wt. in lbs. one ft.	Wt. in lbs one piece	Tot. Wt. in lbs.	Remarks.
			Floor Beam FBC5 2 Req'd.						
Web	Pl.	1	32½ × 5/16	20	10½	3453	720814	721	
Flg.	L	2	5 × 3½ × 7/16	20	3½	12.00	243.504	487	
"	"	2	" " "	19	10	"	237.996	476	
Stiff.	L	14	3 × 3 × 5/16	2	8¼	6.1	16.394	230	
Conn.	L	8	4 × 3½ × 3/8	2	0⅝	9.1	18.674	149	
"	"	8	" " "	1	3	"	11.375	91	
Fills.		8	3½ × 3/8	1	9¼	4.46	8.084	65	
	L	4	3 × 3 × 3/8	0	6½	7.20	3.900	16	
	"	4	5 × 5 × 3/8	1	0	12.30	12.300	49	
	Pls.	2	9 × 5/16	3	7½	9.56	34.981	70	
	L	4	3 × 3 × 5/16	4	8¼	6.1	28.848	115	
	"	4	3 × 3 × 3/8	1	0¾	7.2	7.650	31	
	"	4	5 × 3½ × 3/8	1	1¾	10.4	11.916	48	
	"	4	" " "	1	6¾	"	16.250	65	
Flg.	L	4	" " "	3	6¼	"	36.616	146	
Web	Pls.	2	22 × 5/16	2	9	2338	64.295	129	
	Pls	2	9 × 5/16	1	0½	9.56	9.959	20	Vertical.
Spl.	Pls.	4	6 × 3/8	1	7	7.65	12.117	48	"
	Pls.	2	12½ × 5/16	2	4½	13.28	31.540	63	"
	L	4	5 × 3½ × 3/8	1	0	10.4	10.400	42	"
Fills.		4	3½ × 3/8	0	6	446	2.230	9	"
Tie	Pls.	2	10¼ × ½	3	8	10.89	39.930	80	
Latt.	Bars	6	24 × 5/16	1	1⅝	2.39	2.714	16	
								3,166 × 2 = 6,332	
			Floor Beam FBC6-7-8-9-10-11-12-13-14-15-16-17 24 Req'd.						
Stiff.	L	14	3 × 3 × 5/16	2	2¾	6.1	13.598	190	
Conn.	L	8	4 × 3½ × 3/8	1	7⅛	9.1	14.504	116	
"	"	8	" " "	1	3	"	11.470	92	
Fills.	"	8	3½ × 3/8	1	4¼	4.46	6.225	50	
	L	4	3 × 3 × 3/8	0	6½	7.20	3.900	16	
	"	4	5 × 5 × 3/8	1	0	12.30	12.300	49	
	Pls.	2	9 × 5/16	3	7½	9.56	34.655	69	
	L	4	3 × 3 × 5/16	4	8¼	6.1	28.848	115	
	"	4	3 × 3 × 3/8	1	0¾	7.2	7.650	31	
	"	4	5 × 3½ × 3/8	1	1¾	10.4	11.916	48	
	"	4	" " "	1	6¾	"	16.250	65	
Spl.	Pls.	4	6 × 3/8	1	7½	7.65	12.431	50	
	Pls.	2	9 × 5/16	1	0½	9.56	9.959	20	
	"	2	12½ × 5/16	2	3	13.28	29.880	60	
	L	4	5 × 3½ × 3/8	1	0	1040	10.400	42	
Fills.		4	3½ × 3/8	0	6	446	2.230	9	
Latt Bars.		6	24 × 5/16	1	1⅝	2.39	2.714	16	
								1038 × 24 = 24912	

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CALCULATIONS FOR

Desc.	NO.	Section in ins.	Length ft.	Length ins.	Wt. in lbs one ft.	Wt. in lbs one piece	Tot. Wt. in lbs	Remarks.
Floor Beam FBC6 2 Req'd.								
Web Pl.	1	27 × 5/16	20	9¼	28.69	595.914	596	
Flg. ∠s	2	5×3½×7/16	20	2¼	12.0	242.256	485	
" "	2	" " "	19	9	"	237.000	474	
" "	4	" " "	2	11⅞	"	31.091	124	
Web Pls.	2	2½ × 5/16	2	3½	22.84	52.342	105	
Tie Pls.	2	10¼ × ½	3	9	17.43	65.362	131	
							~~1,787 × 2 = 3,574~~	
							1,915 × 2 = 3,830	
Floor Beam FBC7 2 Req'd.								
Web Pl.	1	27 × 5/16	20	10½	28.69	598.904	599	
Flg. ∠s	2	5×3½×7/16	20	3½	12.0	243.504	487	
" "	2	" " "	19	11½	"	359.52	719	
" "	4	" " "	3	0⅝	"	31.526	126	
Web Pls.	2	22 × 5/16	2	3½	23.38	53.580	107	
Tie Pls.	2	10¼ × ½	3	6½	17.43	61.732	123	
							2,161 × 2 = 4,322	
Floor Beam FBC8 2 Req'd.								
Web Pl.	1	27 × 5/16	20	9¼	28.69	595.914	596	
Flg. ∠s	2	5×3½×7/16	20	2¼	12.0	242.280	485	
" "	2	" " "	19	10	"	237.96	476	
" "	4	" " "	3	0⅝	"	31.526	126	
Web Pls.	2	2½ × 5/16	2	3½	22.84	52.342	105	
Tie Pls.	2	10¼ × ½	3	8	17.43	63.911	128	
							1,976 × 2 = 3,832	
Floor Beam FBC9-11 4 Req'd.								
Web Pl.	1	27 × 5/16	20	10½	28.69	598.904	599	
Flg. ∠s	2	5×3½×7/16	20	3½	12.0	243.504	487	
" "	2	" " "	20	0½	"	240.48	481	
" "	4	" " "	3	1⅛	"	32.176	129	
Web Pls.	2	22 × 5/16	2	3½	23.38	53.580	107	
Tie Pls.	2	10¼ × ½	3	5½	17.43	60.278	121	
							~~1,795 × 4 = 7,180~~	
							1,924 × 4 = 7,696	
Floor Beam FBC10-12 4 Req'd								
Web Pls	1	27 × 5/16	20	9¼	28.69	595.914	596	
Flg. ∠s	2	5×3½×7/16	20	2¼	12.0	242.280	485	
" "	2	" " "	19	10	"	237.960	476	
" "	4	" " "	3	0⅞	"	31.958	128	
Web Pls.	2	2½ × 5/16	2	3½	22.84	52.342	105	
Tie Pls.	2	10¼ × ½	3	7	17.43	62.569	125	
							1,915 × 4 = 7,660	

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CALCULATIONS FOR

Desc.	No.	Section in ins.	Length Ft.	Length ins	Wt. in lbs one ft.	Wt. in lbs one piece	Tot. Wt. in lbs.	Remarks
					Floor Beam FBC13	2 Req'd.		
Web Pl.	1	27 × 5/16	20	10½	28.69	598.904	599	
Flg. Ls	2	5×3½×7/16	20	3½	12.0	243.504	487	
" "	2	" " "	20	0¾	"	240.720	881	
" "	4	" " "	3	1½	"	32.500	130	
Web Pls.	2	22 × 5/16	2	3½	23.38	53.580	107	
Tie Pls.	2	10¼ × ½	3	5¼	17.43	59.916	120	
								~~1,795 × 2 = 3,590~~
								2,324 × 2 = 4,648
					Floor Beam FBC14	2 Req'd.		
Web Pl.	1	27 × 5/16	20	9¾	28.69	597.111	597	
Flg. Ls	2	5×3½×7/16	20	2¾	12.0	242.760	486	
" "	2	" " "	20	0	"	240.000	480	
" "	4	" " "	3	11	"	40,734	163	
Web Pls.	2	21½ × 5/16	2	3½	22.84	52.342	105	
Tie Pls.	2	10¼ × ½	3	6	17.43	61.005	122	
								~~1,824 × 2 = 3,648~~
								1,953 × 2 = 3,900
					Floor Beam FBC15-17	4 Req'd.		
Web Pls.	1	27 × 5/16	20	10½	28.69	598.904	599	
Flg. Ls	2	5×3½×7/16	20	3½	12.00	243.504	487	
" "	2	" " "	20	1¼	"	241.200	482	
" "	4	" " "	3	1¼	"	32.284	129	
Web Pls.	2	22 × 5/16	2	3½	23.38	53.580	107	
Tie Pls.	2	10¼ × ½	3	4¼	17.43	59.189	118	
								~~1,795 × 4 = 7,172~~
								1,922 × 4 = 7,688
					Floor Beam FBC16	2 Req'd.		
Web Pl.	1	27 × 5/16	20	9¾	28.69	597.111	597	
Flg. Ls	2	5×3½×7/16	20	2¾	12.0	242.760	486	
" "	2	" " "	20	0½	"	240.487	481	
" "	4	" " "	3	1¼	"	40.950	164	
Web Pls.	2	21½ × 5/16	2	3½	22.84	52.342	105	
Tie Pls.	2	10¼ × ½	3	5½	17.43	60.278	121	
								1,954 × 2 = 3,908

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CALCULATIONS FOR

Desc.		No.	Section in ins.	Length ft.	Length ins.	Wt. in lbs. one ft.	Wt. in lbs. one piece	Tot. Wt. in lbs.	Remarks.
			Floor Beam FBC18 2 Req'd.						
Web	Pl.	1	27 × 5/16	20	10¾	28.69	599.501	600	
Flg.	L	2	5×3½×⅜	20	2¾	10.4	210.384	421	
"	"	2	" " "	20	0	"	208.000	416	
Stiff.	L	14	3×3×5/16	2	2¾	6.1	13.598	190	
	L	4	3×3×⅜	0	6½	7.2	3.900	16	
	"	4	5×5×⅜	1	0	12.3	12.300	49	
	Plt.	2	9 × 5/16	3	7½	9.56	34.655	69	
	L	4	3×3×5/16	4	8¾	6.1	28.848	115	Strut.
	"	4	3×3×⅜	1	0¾	7.2	7.650	31	
	"	4	5×3½×⅜	1	1¾	10.4	11.916	48	
	"	4	" " "	1	6¾	"	16.250	65	
Conn	L	8	4×3½×⅜	1	7⅜	9.1	14.504	116	
"	"	8	" " "	1	3	9.1	11.375	91	
Flg.	L	4	5×3½×⅜	2	1⅜	10.4	30.658	123	
Web	Plt.	2	22 × 5/16	2	3½	23.38	53.580	107	
	Plt.	2	9 × 5/16	1	0½	9.56	9.560	20	Vertical.
Spl.	Plt.	4	6 × ⅜	1	7½	7.65	12.431	50	"
	Plt.	2	12½ × 5/16	2	3	13.28	29.880	60	"
	L	4	5 × 3½ × ⅜	1	0	10.4	10.400	42	"
Fills.		4	3½ × ⅜	0	6	4.46	2.230	9	"
Tie	Plt.	2	10¼ × ½	3	6	17.43	38.115	76	"
Latt.	Bars	6	2¼ × 5/16	1	1⅝	2.39	2.714	16	
								2730×2=5460	
Tie	Plt.	2	18 × ⅜	3	2	22.95	72.676	145	} At center of Span
"	"	1	30 × ⅜	3	2	38.25	121.126	121	
								266	1 Req'd.
								5.460+266=5.726 ✓	

Floor Beams ~~149.552#÷2240#=66.764 TONS~~
155,046÷2,240=69.217

増田淳氏関係資料
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CALCULATIONS FOR

Desc.	No.	Section in ins.	Length. ft.	Length. ins.	Wt. in lbs. one ft.	Wt. in lbs one piece	Tot. Wt. in lbs.	Remarks.
		Side strut at LS1. 2 Req'd.						
LS1. L's	4	3 × 3 × 5/16	7	9	6.1	47.275	189	
Pl's.	2	15½ × 5/16	1	4½	16.47	22.646	45	
L's	4	4 × 3 × 3/8	1	3½	8.5	10.975	44	
"	4	" " "	0	6	"	4.250	17	
Pl's.	2	6 × 5/16	1	2½	6.38	7.709	15	
Ring Fills.	8	2½"φ × 5/16				0.62	5	
							315	× 2 = 630 ✓
		Side strut at LS2 2 Req'd.						
L's	4	3 × 3 × 5/16	7	8½	6.1	47.021	188	
Pl's.	2	15½ × 5/16	1	3⅜	16.47	21.103	42	
L's	4	4 × 3 × 3/8	1	3½	8.5	10.979	44	
"	4	" " "	0	6	"	4.250	17	
Pl's.	2	6 × 5/16	1	2½	6.38	7.709	15	
Ring. Fills.	8	2½"φ × 5/16				0.620	5	
							311	× 2 = 622 ✓
		Side strut at LS3 2 Req'd.						
L's	4	3 × 3 × 5/16	8	2½	6.1	50.071	200	
Pl's.	2	15½ × 5/16	1	4⅛	16.47	22.132	44	
L's	4	4 × 3 × 3/8	1	3½	8.5	10.979	44	
"	4	" " "	0	6	"	4.250	17	
Pl's.	2	6 × 5/16	1	2½	6.38	7.709	15	
Ring Fills.	8	2½"φ × 5/16				0.620	6	
							325	× 2 = 650
		Side strut at LS4 2 Req'd.						
L's	4	3 × 3 × 5/16	8	7	6.1	52.358	209	
Pl's.	2	14 × 5/16	1	3½	14.88	19.220	38	
L's	4	4 × 3 × 3/8	1	3½	8.5	10.979	44	
"	4	" " "	0	6	"	4.250	17	
Pl's	2	6 × 5/16	1	2½	6.83	7.709	15	
Ring. Fills.	8	2½"φ × 5/16				0.62	5	
							328	× 2 = 656 ✓
		Side strut at LS5. LS6 & LS7 6 Req'd.						
L's	4	3 × 3 × 5/16	7	9⅜	6.1	47.466	190	
"	4	4 × 3 × 3/8	1	3½	8.5	10.979	44	
Pl's.	2	15 × 5/16	1	3½	15.94	20.590	41	
"	2	6 × 5/16	1	2½	6.83	7.709	15	
L's	4	4 × 3 × 3/8	0	6	8.5	4.250	17	
Ring. Fills	8	2½"φ × 5/16				0.620	5	
							312	× 6 = 1,872 ✓
						Side Struts	4,430 ÷ 2240 = 1.978	

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CALCULATIONS FOR

Desc.	No.	Section in ins.	Length Ft.	Length ins.	Wt. in lbs one ft.	Wt. in lbs one piece	Tot. Wt. in lbs	Remark.
Bottom Strut S1						2 Req'd		
Flg.	Ls	4.	4 × 3 × 5/16	20	0¼	7.12	144.150	577
Conn.	Ls	4	4 × 3½ × 3/8	1	4	9.1	12.133	24
Tie	Pls.	2	16½ × 5/16	2	0½	17.53	35.791	72
"	"	1	16½ × 5/16	2	0	"	35.060	35
Latt. Bars		24	2½ × 5/16	1	10½	2.66	4988	120
								828 × 2 = 1,656 ✓
Bottom Strut S2						2 Req'd.		
Flg.	Ls	4	4 × 3 × 5/16	19	9	7.12	142.200	569
Conn	Ls	4	4 × 3½ × 3/8	1	4½	9.1	12.513	50
Tie	Pls.	2	16½ × 5/16	1	10¼	17.53	32.504	65
Latt.	Bars.	28	2½ × 5/16	1	10½	2.66	4.988	140
								824 × 2 = 1,648 ✓
Bottom Strut S3						2 Req'd.		
Flg.	Ls	4	4 × 3 × 5/16	19	11¼	7.12	143.550	574
Conn.	Ls	4	5 × 3½ × 3/8	1	5	10.4	14.734	59
Tie	Pls.	2	16½ × 5/16	1	7	17.53	27.755	56
"	"	1	"	"		"	35.060	35
Guss.	Pls.	2	24 × 5/16	2	3	25.50	57.375	115
Latt.	Bars	20	2½ × 5/16	1	10½	2.66	4.988	100
								939 × 2 = 1,878 ✓
Bottom Strut S5						2 Req'd.		
Flg.	Ls	4	4 × 3 × 5/16	19	11¼	7.12	143.550	574
Conn	Ls	4	4 × 3½ × 3/8	1	4½	9.1	12.513	50
Tie	Pls.	2	16½ × 5/16	1	7⅞	17.53	27.940	56
"	"	1	"	"		"	35.060	35
Guss.	Pls.	2	24 × 5/16	2	3	25.50	57.375	115
Latt.	Bars.	20	2½ × 5/16	1	10½	2.66	4.988	100
								930 × 2 = 1,860 ✓
Bottom Strut S6						2 Req'd.		
Flg.	Ls	4	4 × 3 × 5/16	19	8¾	7.12	142.050	568
Conn.	Ls	4	4 × 3½ × 3/8	1	5	9.1	10.200	41
Tie	Pls.	2	16½ × 5/16	1	10	17.53	32.138	64
Latt.	Bars.	28	2½ × 5/16	1	10½	2.66	4.988	140
								813 × 2 = 1,626 ✓

増田淳氏関係資料
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CALCULATIONS FOR

Desc.	NO	Section in ins.	Length Ft.	Length ins.	Wt. in lbs. one ft.	Wt. in lbs one piece	Total Wt. in lbs.	Remarks
		Bottom strut 37				2 Req'd.		
Flg. Ls	4	4 × 3 × 5/16	19	11¼	7.2	143.550	574	
Con.n Ls	4	4 × 3½ × ⅜	1	4½	9.1	12.513	50	
Tie Pls	2	16½ × 5/16	1	10½	17.53	32.869	66	
" "	1	"	2	0	"	35.060	35	
Latt Bars	24	2½ × 5/16	1	10½	2.66	4.988	120	
							845 × 2 = 1,690 ✓	
		Bottom strut 34				2 Req'd.		
Flg. Ls	4	4 × 3 × 5/16	19	11¼	7.2	143.850	575	
Con'n Ls	4	4 × 3½ × ⅜	1	3	9.1	11.375	46	
Tie Pls	2	16½ × 5/16	1	6½	17.53	27.026	54	
" "	2	19 × 5/16	2	3½	20.19	26.269	53	
Latt. Bars	24	2½ × 5/16	1	10½	2.66	3.113	75	
							803 × 2 = 1,606 ✓	
						Bottom Struts 16,394# ÷ 2240 = 7.319		
		Stringer 4 Req'd.						
Flg. L	1	3 × 3 × 5/16	3	4	6.1	20.318	20	
" "	1	" " "	2	6	"	18,000	18	
Web Pl.	1	12 × 5/16	3	4	12.75	42.500	43	
Flg. L	1	3 × 3 × 5/16	8	6	6.1	51.850	62	
" "	1	" " "	9	3½	"	66.900	67	
Web Pl.	1	12 × 5/16	9	3½	12.75	118.469	118	
							318 × 4 = 1272	
		Stringer 100 Req'd.						
Flg. Ls	2	3 × 3 × 5/16	12	8¾	6.1	71.653	155	
Web Pl.	1	12 × 5/16	12	8¾	12.75	162.297	162	
							317 × 100 = 31,700	
		Stringer 1 Req'd.						
I Section	6	12 × 5	13	7¾	31.99	436.865	2621	
"	150	" "	12	8¾	"	407.207	61.081	
Con'n Pls	306	9 × ⅝	1	0	9.56	9.560	2.925	
							66627 × 1 = 66.627	
						Stringers ~~102.451 ÷ 2240 = 45.737~~ TONS		
						99,599# ÷ 2,240 = 44.464		

増田淳氏関係資料
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CALCULATIONS FOR

Descr.	No.	Section in ins.	Length ft.	Length ins.	Wt in lbs One ft.	Wt. in lbs. One piece.	Tot. wt. in lbs.	Remarks.
		Towers 4 Req'd.						
Sole Pl	4	30 × 3/4	3	8	76.5	280.755	1,123	
"	2	30 × 3/8	9	6	38.25	363.375	726	
Ls	2	6 × 6 × 1/2	9	6	19.6	186.200	372	
"	2	"	8	0	"	156.800	314	
"	6	"	2	4 1/2	"	46.648	280	
"	16	6 × 4 × 1/2	0	8	16.2	10.854	174	
"	8	5 × 4 × 1/2	2	5	14.5	35.090	281	
"	8	4 × 4 × 1/2	2	5	12.8	30.976	248	
"	4	"	2	5 1/4	"	31.232	125	
P/s	4	15 × 1/2	2	6	25.50	63.750	255	
Fills	4	11 1/2 × 1/2	1	5 3/4	19.55	28.934	116	
"	4	12 1/4 × 1/2	2	6	20.83	52.075	208	
"	2	17 3/4 × 1/2	1	10 1/2	30.18	56.738	113	
"	2	8 1/2 × 1/2	1	11 3/4	14.45	28.611	57	
L	8	6 × 6 × 1/2	18	8 1/8	19.6	366.128	2929	
P/	2	30 × 1/2	18	8 1/8	51.00	952.680	1905	
L	2	6 × 6 × 1/2	22	9 1/8	19.6	446.096	892	
"	2	"	20	8 5/8	"	406.112	812	
Pl.	2	30 × 1/2	22	9 1/8	51.00	1160.760	2322	
L	2	6 × 6 × 1/2	10	0	19.6	196.000	392	
"	2	"	19	5	"	380.632	761	
"	2	"	9	0	"	176.400	353	
Pl.	2	36 × 3/8	7	2	45.90	329.103	658	
L	2	6 × 6 × 1/2	5	6	19.6	107.800	216	
"	4	6 × 6 × 3/8	19	0 1/8	14.90	283.249	1133	
P/s	2	24 × 1/2	19	0 1/8	40.80	775.608	1551	
L	4	6 × 6 × 3/8	14	5 1/8	14.90	215.007	860	
Pl.	2	24 × 1/2	14	5 1/8	40.80	588.744	1177	
Fill	8	5 1/4 × 1/8	1	6	2.23	3.345	27	
"	4	18 1/4 × 1/2	2	6	21.03	77.575	310	
Pl.	4	29 × 1/2	2	6	49.30	123.250	493	
Fill	2	12 1/4 × 1/2	2	6	20.83	52.075	104	
Pl.	2	22 1/2 × 1/2	2	6	38.25	95.625	191	
"	2	24 × 1/2	2	6	40.80	102.000	204	
"	2	22 1/2 × 1/2	2	6	38.25	95.250	191	
Fill	2	12 1/4 × 3/8	2	6	15.62	39.050	78	
L	4	6 × 2 1/2 @ 12.#04	5	2	12.04	62.247	249	
"	4	"	4	11	"	59.237	237	
"	2	"	7	8	"	92.347	185	
"	4	"	4	11	"	59.237	237	
"	8	"	4	7	"	55.143	441	
"	2	"	7	2	"	86.327	173	
"	8	"	4	4	"	52.133	417	
"	2	"	6	9	"	81.270	163	
"	6	"	4	2	"	50.207	301	
"	2	"	6	4	"	76.213	152	
"	1	"	3	11	"	47.197	47	
"	2	"	5	11	"	71.277	143	
"	2	"	3	10	"	46.113	92	
"	2	"	3	8	"	44.187	88	
"	2	"	5	6	"	66.220	132	

増田淳氏関係資料
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Descr.	No.	Section in ins.	Length ft.	Length ins.	Wt in lbs One ft.	Wt in lbs. One piece	Tot. Wt. in lbs.	Remarks.
L	1	6×2½@12.04#	4	6	12.04	54.180	54	
,	4	5×2½@10.98#	3	8	10.98	40.297	161	
,	4	,	3	6	,	38.430	154	
,	2	,	5	2	,	56.767	114	
,	4	,	3	5	,	37.552	150	
,	8	,	3	3	,	35.685	286	
,	2	,	4	9	,	53.155	106	
,	6	,	3	0	,	32.940	132	
,	2	,	4	5	,	48.532	97	
,	6	,	2	10	,	31.073	186	
,	2	,	4	1	,	44.798	90	
,	8	,	2	7½	,	28.877	231	
,	2	,	3	9	,	41.175	82	
,	4	,	2	6	,	27.450	110	
,	4	,	2	5	,	26.572	106	
,	2	,	3	5	,	37.552	75	
,	4	,	2	4	,	25.583	102	
,	2	,	3	2	,	34.807	70	
,	4	,	2	3½	,	25.144	101	
,	1	,	3	1	,	33.818	34	
Pl	24	10×3/8	1	0	12.75	12.750	306	
,	4	12 3/8×½	2	0	21.04	42.080	168	
,	4	9×3/8	2	11½	11.48	33.981	136	
L	1	3×3×3/8	5	1	7.2	36.576	37	
,	1	"	5	0	,	36.000	36	
Pl.	1	28×3/8	5	1	35.70	181.356	181	
L	1	3×3×3/8	4	9	7.2	34.200	34	
Pl.	1	36×3/8	4	9	45.90	218.025	218	
,	4	6×½	1	9	10.20	17.850	71	
,	4	"	1	9	,	17.850	71	
,	1	37½×½	4	4	63.75	276.038	276	
,	1	23½×½	2	7	39.95	103.071	103	
L	4	6×6×½	2	7½	19.6	51.548	206	
,	2	,	1	10½	,	36.848	74	
Fill	2	6×½	0	1 3/4	10.2	9.996	20	
,	2	12½×3/8	2	7	15.94	41.125	82	
L	4	6×6×½	2	7	19.6	50.568	202	
Fill	2	12½×½	2	2	21.25	46.113	92	
L	2	6×6×½	1	11½	19.6	38.416	77	
,	2	3×3×3/8	1	0 3/4	7.2	7.632	15	
,	2	,	1	2	,	25.366	51	
Pl.	1	12 3/4×½	1	6	21.68	35.520	36	
,	1	7½×½	1	3	12.75	15.938	16	

29.957×4=119.828

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CALCULATIONS FOR

Description	No	Section in ins	Length Ft	Length ins	Wt in lbs one Ft	Wt in lbs one piece	Total Wt in lbs	Remarks
		Portal Bracing (middle) 2 Req'd						
Flg Le	4	6 × 6 × 3/8	17	0	14.90	402.300	1609	Top
" "	4	" " "	28	6	14.90	424.650	1699	Bottom
Pls	4	18½ × 3/8	5	3	17.21	90.353	361	Top
"	4	" "	7	3	17.21	124.773	499	"
"	4	" "	5	11	"	101.826	407	Bottom
"	4	15 × 3/8	7	6	19.13	143.475	574	"
Conn Le	4	6 × 6 × 3/8	7	1½	14.90	106.163	425	
Fills	4	5¾ × 3/8	6	1	7.33	44.591	178	
Le	8	3 × 3 × 5/16	6	9	7.20	48.600	389	
"	8	" " "	6	8	"	48.000	384	
"	4	" " "	6	8	"	45.000	180	
"	4	" " "	5	10	"	42.000	168	
"	4	" " "	5	6	"	39.600	158	
"	4	" " "	5	2	"	37.200	149	
"	8	" " "	4	11	"	35.400	283	
"	8	" " "	4	10	"	34.800	278	
"	4	" " "	4	7	"	33.000	132	
"	4	" " "	4	5	"	31.800	127	
"	4	" " "	4	3	"	30.600	122	
"	4	" " "	4	1½	"	29.700	119	
"	4	" " "	4	0	"	28.800	115	
"	4	" " "	3	11½	"	28.500	114	
"	8	" " "	3	11½	"	28.500	228	
Fills	4	3 × 3/8	4	5	3.83	16.916	68	
"	8	6 × 3/8	5	9	7.65	43.988	352	
"	40	3 × 3/8	0	7¾	3.83	24.754	989	Top
"	40	" "	0	5½	"	1.755	70	Bottom
"	4	6¼ × 3/8	2	9	7.97	21.918	88	
"	8	6¼ × 3/8	3	11	"	31.216	250	
"	4	" "	3	0	"	23.910	96	
"	2	" "	1	10	"	14.614	29	
Tie Pls	2	31 × ½	6	2	52.70	325.154	650	Bottom
" "	1	" "	6	0	52.70	316.200	316	
Pls	2	50½ × 3/8	5	0	64.44	322.200	644	Diaph Ⓐ
"	1	42½ × 3/8	4	10	54.19	261.738	262	" Ⓑ

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CALCULATIONS FOR

Description		No	Section in ins	Length ft	Length ins	Wt in lbs one ft	Wt in lbs one piece	Total wt in lbs	Remarks
Tie	Pls	2	25½ × ½	5	10	43.35	252.731	505	Top
"	"	3	17½ × ⅜	5	10	22.31	130.067	390	"
	∠s	6	3×3×⅜	3	9	7.20	27.000	162	"
LaH	Bars	4	4.4×⅜	5	6	9.80	53.900	~~220~~ 216	
"	"	8	" "	5	5	9.80	53.116	425	
								~~14.208 × 2 = 28.416~~	
								14.204 × 2 = 28,408	
	Pl	4	12×⅜	4	2	Bracing 15.3	2 Req'd 62.424	(Bottom) 250	
	Pl	4	12×⅜	3	10	15.3	58.599	234	
	∠s	4	5×5×⅜	14	3	12.30	175.275	701	
	"	4	" " "	14	7	12.30	179.375	718	
	"	8	" " "	14	11	12.30	183.475	1468	
	"	4	5×3×⅜	2	3	9.80	22.050	88	Diaph
	Pl	1	27×⅜	4	0	34.43	137.720	138	Diaph
	∠s	4	6×6×⅜	1	7½	14.90	24.213	97	
	"	8	" " "	2	1½	14.90	31.663	253	
	Pls	4	25½ × ⅜	2	2	32.52	70.461	282	
Guss	Pls	2	39 × ⅜	4	1½	49.73	205.136	410	
								4,639 × 2 = 9,278	
	Pl	4	12×⅜	2	10	Bracing 15.3	2 Req'd 43.249	(Top) 173	
	"	4	12 "	2	3	"	34.435	138	
	∠s	8	5×5×⅜	14	2¾	12.30	175.019	1400	
	"	4	" " "	14	8¾	12.30	181.169	725	
	"	4	" " "	14	4¾	12.30	177.068	708	
Guss	Pls	2	28½ × ⅜	4	3	49.09	208.692	817	Diaph
	∠s	4	5×3×⅜	2	3	9.80	22.050	88	Diaph
	Pl	1	27 × ⅜	2	6	34.43	86.075	86	
Guss	Pls	4	47 × ⅜	4	5	59.93	264.695	1069	
"	∠s	4	6×6×⅜	3	10	14.90	57.116	228	
"	Pls	4	27½ × ⅜	2	7½	35.07	92.059	368	
	∠s	4	6×6×⅜	1	8½	14.90	25.454	103	
	"	4	" " "	2	7½	14.90	39.113	156	
								6,059 × 2 = 12118	
						Strut	2 Req'd		
	∠s	4	6×4×⅜	32	1	12.30	394.625	1579	
	Pls	2	24 × ⅜	3	0	30.60	91.800	184	
Fills		4	3¾ × ⅛	2	0	1.594	3.188	13	
	Pls	1	27 × ⅜	3	0	34.43	103.290	103	
Fills		2	3¾ × ⅛	2	3	1.504	3.587	7	
LaH	Bars	20	3 × ½	3	9	5.10	19.125	383	
								2,269 × 2 = 4538	

増田淳氏関係資料
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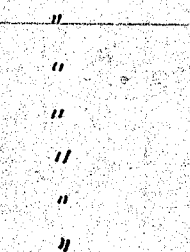
CALCULATIONS FOR

Description		No	Section in ins	Length ft	Length ins	Wt in lbs one Ft	Wt in lbs one piece	Total wt in lbs	Remarks
			Top girder				2 Req'd		
Flg	∠s	4	6×6×⅜	32	1	14.90	478.041	1912	
	∠s	8	6×6×½	3	5¾	19.60	68.192	546	
	Pls	4	42×½	5	6	71.40	392.700	1571	
Fills		4	30¼×⅜	4	9	38.57	183.208	733	
	∠s	4	6×4×½	2	11⅞	16.20	48.432	194	
Spl	Pls	4	12½×⅜	1	9	15.94	27.895	112	
	Pls	4	12½×⅜	10	0½	15.94	160.065	640	
Spl	Pls	4	11¼×⅜	2	0	14.34	28.680	115	
"	"	4	11¼×⅜	1	9	14.34	25.095	100	
	Pls	4	11¼×⅜	10	0½	14.34	143.998	576	
	"	3	18×⅜	2	7½	22.95	60.244	181	
	"	2	23¾×½	3	6½	40.88	143.014	286	Diaph
	∠s	8	6×6×½	3	6½	19.60	69.417	555	"
	"	4	" " "	1	10½	19.60	36.750	147	"
	"	4	6×4×½	1	10½	16.20	30.375	122	"
Fills		8	6×½	0	11¾	10.20	9.988	80	"
	Pls	2	23¾×¾	3	0¼	60.61	183.091	366	"
	∠s	8	6×6×½	3	0¼	19.60	58.604	469	"
	"	4	6×4×½	1	10½	16.20	30.375	122	"
Fills		4	6×½	0	11¾	10.20	9.988	40	"
	Pls	2	23¾×½	3	0¼	40.38	121.980	244	"
	∠s	8	6×6×½	3	0¼	19.60	58.604	469	"
	"	4	6×4×½	1	10½	16.20	30.375	122	"
Fills		4	6×½	0	11¾	10.20	9.988	40	"
	Pls	4	23¾×½	3	1¼	40.38	125.348	501	"
	∠s	8	6×4×½	3	0¼	16.20	48.937	391	"
	"	4	6×4×½	1	10½	16.20	30.375	122	"
Fills		4	6×½	0	11¾	10.20	9.988	40	"
	Pls	2	24½×½	3	1¼	41.65	129.290	260	
Tie	Pls	2	32×⅜	3	1	40.80	125.799	252	
Latl	Bars	24	4×½	4	4½	6.80	29.750	714	
Fills		10	6×⅜	0	6	7.65	3.825	38	
"		10	5¼×⅜	0	6	6.69	3.345	34	
"		10	3×⅜	0	5¼	3.83	1.675	17	
	∠s	20	3×3×⅜	3	5¾	7.20	25.050	501	
	"	32	" " "	3	5¾	7.20	25.050	802	
Fills		32	3×⅜	0	5¼	3.83	1.676	54	
"		32	3×⅜	0	6	3.83	1.915	61	
Bolts		28	⅞" ϕ	0	2¾		1.050	29	
"		24	"	0	3¾		1.050	25	
"		24	"	0	3		1.090	26	



13609 × 2 = 27218

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

Description	No	Section in ins	Length ft	Length ins	Wt in lbs one ft	Wt in lbs one piece	Total wt in lbs	Remarks
		Anchorage for Tower						
Anch. bolts	32	2½"ø	10	0	with one head & nut	179.900	5751	
Washers	32	5 × ½	0	5	8.150	3.542	113	
Pls	8	21 × ⅜	4	2	26.78	111.584	893	
E	8	6, 9½@12.04	9	0	12.04	108.360	867	
Ls	8	3 × 3 × ⅜	9	6	7.20	68.400	547	
Washer	4	3"ø × ⅜				0.600	2	
Pls	8	12 × ⅜	4	0	15.30	61.200	490	
Ls	8	3 × 3 × ⅜	8	9	7.20	63.000	504	
Washer	4	3"ø × ⅜				0.600	2	
							9.175 × 1 = 9,175	
					Towers		~~199,564 ÷ 2240 = 89.097~~ Tons	
							210,563 ÷ 2240 = 94.001	

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

Description	No	Section in ins	Length ft	Length ins	Wt in lbs one ft	Wt in lbs one piece	Total wt in lbs	Remarks
			Suspender					
Rods	72	2½"φ	4	8	16.69	77.887	5608	RD 1
"	4	"	28	9⅛	"	481.054	1924	RD 2
"	4	"	25	8⅛	"	429.594	1718	"
"	4	"	21	4⅜	"	356.575	1426	"
"	4	"	17	3⅜	"	288.425	1154	"
"	4	"	13	6⅞	"	226.532	906	"
"	4	"	10	2½	"	170.377	682	"
"	4	"	26	8	"	445.067	1780	RD 3
"	4	"	24	8	"	411.687	1647	"
"	4	"	23	8	"	394.997	1580	"
"	4	"	22	8	"	378.307	1513	"
"	4	"	21	8	"	361.617	1446	"
"	4	"	20	8	"	344.927	1380	"
"	4	"	27	2¼	"	453.759	1815	RD 4
"	4	"	23	4⅜	"	398.955	1560	"
"	4	"	20	1¾	"	336.233	1345	"
"	4	"	17	2⅛	"	286.686	1147	"
"	4	"	14	6¾	"	243.048	972	"
"	4	"	12	4	"	205.843	823	"
"	4	"	10	5⅛	"	174.028	696	"
"	4	"	8	10¼	"	147.777	591	"
"	4	"	7	8⅛	"	128.131	513	"
"	4	"	6	9⅝	"	113.527	454	"
"	4	"	6	3¼	"	104.660	419	"
"	4	"	6	1	"	101.530	406	"
Turnbuckles	72		1	1½		25.000	1800	
Sleeve nuts	24		0	10		19.000	456	
							35,761	

Suspenders 35.761 ℔ ÷ 2240 = 15.965 Tons

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CALCULATIONS FOR

Description	No	Section in	Section ins	Length Ft	Length ins	Wt in lbs one ft	Wt in lbs one piece	Total wt in lbs	Remarks
Anchorage at abutment A 2 Req'd									
Pls	4	9	x 1/2	2	3	15.30	34.425	138	
"	2	9	x 1/2	2	0	15.30	30.600	61	
"	8	5 1/2	x 3/8	0	9	7.01	5.258	42	
Ls	4	4x4	x 3/8	11	6	9.80	112.700	450	
"	2	3x3	x 3/8	6	0	7.20	43.200	86	
Tie Pls	2	11 1/2	x 3/8	1	11 1/4	14.66	28.404	57	
Fills	8	4	x 5/16	0	11 1/2	4.25	4.078	33	
Lat Bars	2	3	x 1/2	3	4	5.10	17.000	34	
Washers	4	3"ϕ	x 1/2				0.900	4	
Web Pls	2	24	x 5/8	6	0	30.60	183.600	367	
Flg Ls	4	4x4	x 3/8	6	0	9.80	58.800	235	
Pl	1	19	x 5/8	1	11 1/4	24.23	46.946	47	Diaph
Ls	4	4x4	x 3/8	1	4	9.80	13.066	52	"
"	4	3x3	x 3/8	1	6 3/4	7.20	11.250	45	"
"	8	4x4	x 3/8	0	5 1/2	9.80	4.491	36	
Pls	2	9	x 3/8	1	7 1/4	11.48	18.416	37	
Bolts	32	3/4"ϕ		0	2	-	0.630	20	
Isections	4	12	x 5	6	0	31.99	191.940	768	
								2,512 x 2 = 5,024	
Anchorage at pier P3 2 Req'd									
Pls	4	9	x 1/2	2	3	15.30	34.425	138	
"	2	"	"	2	0	15.30	30.600	61	
Spl Pls	2	9	x 3/8	1	8	11.48	19.134	38	
Ls	32	3x3	x 3/8	1	6	7.20	10.800	346	
ISection	2	8	x 5	3	6	28.02	63.070	126	
C "	2	12	x 3/16	3	0	26.10	78.300	157	
Ls	4	4x4	x 5/8	0	8 1/2	9.80	6.941	28	Diaph
Pls	1	8 1/2	x 3/8	1	7	10.84	17.163	17	"
Pls	2	9	x 3/8	1	8	11.48	19.134	38	
Ls	4	4x4	x 3/8	4	0	9.80	39.200	157	
Spl Pls	4	4	x 1/2	2	11	6.80	19.834	74	
Tie Pl	1	10	x 3/8	1	11 1/4	12.75	24.703	25	
Fills	4	4	x 5/16	0	10	4.25	3.542	14	
Ls	4	4x4	x 3/8	16	0	9.80	156.800	627	
Tie Pls	16	14 1/2	x 3/8	1	11 1/4	18.49	35.824	573	
Fills	64	4	x 5/16	1	2 1/2	4.25	5.135	329	
Ls	4	4x4	x 3/8	18	0	9.80	176.400	706	
								3459 x 2 = 6.918	

Anchorage For Abutments 11.942 #1 ÷ 2240 = 5.331 Tons

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CALCULATIONS FOR

Description		No	Section in ins	Length Ft	Length ins	Wt in lbs one ft	Wt in lbs one piece	Total wt in lbs	Remarks
Cable Anchorage Girder							**8 Req'd**		
Web	Pls	2	24 × 3/8	11	0	30.60	336.600	673	
Flg	Le	4	5×3½×3/8	11	0	10.40	114.400	458	
Bear	Pls	2	12 × 1/2	1	9	20.40	35.700	71	
StH	Le	22	3×3×3/8	1	11¾	7.20	14.250	314	
Fills		4	6½ × 3/8	1	2¼	8.29	9.844	39	
"		6	11 × 3/8	1	2¼	14.03	16.661	100	
Diaph.	Pls	2	10½ × 3/8	1	11¾	15.94	31.548	63	
"	Le	4	3×3×3/8	1	0	7.20	7.200	29	
								1,747 × 8 = 13,976 ✓	
Girder							**8 Req'd**		
Web	Pls	2	24 × 3/8	11	0	30.60	336.600	673	
Flg	Le	4	5×3½×3/8	11	0	10.40	114.400	458	
Bear	Pls	2	12 × 1/2	1	9	20.40	35.700	71	
StH	Le	24	3×3×3/8	1	11¾	7.20	14.250	342	
Fills		4	6½ × 3/8	1	2¼	8.29	9.844	39	
"		8	11 × 3/8	1	2¼	14.03	16.661	133	
Diaph	Pls	2	12½ × 3/8	1	11¾	15.94	31.548	63	
"	Le	4	3×3×3/8	1	0	7.20	7.200	29	
								1,808 × 8 = 14,464 ✓	
Girder							**4 Req'd**		
Web	Pls	2	24 × 3/8	11	0	30.60	336.600	673	
Flg	Le	4	5×3½×3/8	11	0	10.40	114.400	458	
Bear	Pls	2	12 × 1/2	1	9	20.40	35.700	71	
StH	Le	26	3×3×3/8	1	11¾	7.20	14.250	371	
Fills		4	6½ × 3/8	1	2¼	8.29	9.844	39	
"		10	11 × 3/8	1	2¼	14.03	16.661	167	
Diaph	Pls	2	12½ × 3/8	1	11¾	15.94	31.548	63	
"	Le	4	3×3×3/8	1	0	7.20	7.200	29	
								1,871 × 4 = 7,484 ✓	

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CALCULATIONS FOR

Description	No	Section in ins	Length ft	Length ins	Wt in lbs one ft	Wt in lbs one piece	Total wt in lbs	Remarks
			Girder			8 Req'd		
Web Pls	2	18½ × ⅜	6	2	23.59	145.472	291	
" "	2	19½ × ⅜	6	2	24.86	153.304	307	
Flg Ls	2	3 × 3 × ⅜	11	8½	7.20	84.300	169	
" "	2	" " "	13	4	7.20	96.000	192	
Ls	2	" " "	0	7½	7.20	4.500	9	
"	2	" " "	0	8½	7.20	5.100	10	
Spl Pls	2	11½ × ⅜	0	11½	14.66	14.049	28	
Diaph Pls	8	11½ × ⅜	1	10½	14.66	27.488	220	1'-0"
Bear Pls	8	6 × ⅜	0	9	7.65	5.738	46	
Anch Bolts	16	3¼" ϕ	1	0		1.870	30	
							1,302 × 8 = 10,416 ✓	
						1 Req'd.		
Rods	152.	2" ϕ	7	0	10.681	24.767	1 1.365	
Nuts	608					3.400	2067	
							13,432.	13,432. ✓
						10 Req'd		
Rods	2.	2" ϕ	2	7½	10.681	28.035	56	
Rod with heads	2.	"	2	0	[illegible]	24.700	49	
Turnbuckles	2.		1	0		14.000	28	
Nut	4					3400	14	
							147 × 10 = 1,470 ✓	

Anchorage For Cable 61.242 #. ÷ 2240 = 27.340 Tons

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CALCULATIONS FOR

Cast steel shoes Fig. 2 Req'd.

Cast steel.

1 Shoe	368#
1 Bed Frame	292
	660

660 × 2 = 1320#

Structual steel.

5 - Rollers	4"φ × 0'-11½	@ 41.0#	=	205#
2 - Bars	2½ × ½ × 1'-9	@ 7.45	=	15
2 - "	5 × ⅜ × 1'-0¾	@ 6.8	=	14
8 - Tapped Bolts	½"φ × 0'-1¼	@ 0.15	=	1.2
4 - " "	¾"φ × 0'-1⅞	@ 0.55	=	2.2
6 - " "	¾"φ × 0'-1⅞	@ 0.55	=	3.3
				241#

241 × 2 = 482#

Total Wt. of shoes A1

Cast steel	1.320#
Struc. steel	482
	1.802#

Cast steel shoes F2. 2 Req'd

Cast steel.

1 Shoe	328#
1 Bed Frame	260
	588

588 × 2 = 1.176#

Struc. steel

Rollers, Bars and Bolts 241# 241 × 2 = 482#

Total Wt. of shoes F2

Cast steel	1.176#
Struc. steel	482
	1,658#

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CALCULATIONS FOR

Desc	NO.	Section in ins.	Length ft.	Length ins.	Wt. in lbs one ft.	Wt. in lbs one piece	Tot. Wt. in lbs.	Remarks.
Shoes B1 2 Req'd.								
Structural Steel.								
Pls	6	24½ × 5/8	2	11	52.06	151.843	911	
Le	8	3½×3½×½	1	2½	11.1	13.412	107	
"	8	" " "	0	1½	"	10.637	85	
Pls	3	11½ × ½	1	2⅜	19.55	23419	70	Diaph.
Le	4	6×4×½	2	11	16.2	47.251	189	"
Fills	8	3½ × ½	0	9	5.95	4.463	36	
"	4	6 × ½	0	7½	10.20	6.375	26	
Le	6	6×4×½	0	7½	16.2	10.125	61	
Pls	2	5½ × ½	2	5	9.35	22.596	45	
"	2	3 × ½	2	4	5.1	11.900	24	
Rollers. &	6	4½"Φ	2	2	54.07	117.153	703	
Tapped Bolts	16	½ Φ	0	1⅜		0.15	2	
" "	12	⅞ Φ	0	2⅛		0.75	9	
							2268	× 2 = 4.536#
Cast steel.								
2 – Cast steel guards		@ 745#	= 1149#					
1 – Shoe		@ 445	= 445					
1 – Cast stee Base Pl.		@ 590	= 590					
							1.184	1.184 × 2 = 2.368#
Total Wt. of Shoe B1								
Structural steel			4.536					
Cast steel.			2.368					
			6.904#					
Shoes B2 2 Req'd.								
Structural steel.								
Pls	6	30½ × ⅝	2	11	64.81	189.031	1.134	
Stiff. Le	24	3½×3½× ½	1	5½	11.1	16.187	389	
" "	8	" " "	0	1½	"	10.637	85	
Le	6	6×3½ × ½	0	7	15.3	8.924	54	Diaph.
Pls	3	14½ × ½	1	5½	24.65	35.947	109	"
Fills.	4	12 × ½	1	1½	20.40	39.949	160	
"	8	3½ × ½	0	6	5.95	2.975	24	
Le	4	6×4× ½	2	11	16.20	47.251	189	
							2.144	× 2 = 4.288#
Cast steel								
1 cast steel Base Pl. @ 498# = 498								498 × 2 = 996#
Total Wt. of Shoe B2								
Structural steel			4288					
Cast. steel.			996					
			5.284#					

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CALCULATIONS FOR

Pins and Nuts For A1 and A2.

4- Pins 4½"φ x 1'-11⅞" @ 107.015 = 428
8- Nuts @ 4.600 = 37
465#

Pins and Nuts For B1 and B2

4- Pins 6"φ x 1'-11⅞ @ 189.261 = 757
8- Nuts @ 7.800 = 62
819#

Anchor Bolts for Shoes B1-B2

32- Bolts ¼"φ x 2'-0" @ 23.7 = 758#
32- Hex. Nuts. @ 0.865 = 28
32- Washers 3"φ x ¼" @ 0.500 = 16
802#

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CALCULATIONS FOR

Saddle on tower 4 Req'd.

Cast steel.

1- Cast steel Cap @ 157#	=	157#
1- Cast steel Pedestal @ 1.440	=	1.440
1- " " Base Pl. @ 560	=	560
1- " " guard @ 216	=	216
		2.373

2.373 × 4 = 9.492#

Structural steel.

7- Rollers 4"ϕ × 2'-7½ @ 108	=	756#
2- Bars 2½ × ½ × 2'-4 @ 9.9	=	20
12- Bolts ¾"ϕ × 0'-2⅞ @ 0.735	=	9
14- Tapped Bolts ¾"ϕ × 0'-1⅞ @ 0.58	=	8
		802

802 × 4 = 3.208#

Total Wt. of suddles on tower.

Cast steel.	9.492
Struc. steel.	3.208
	12.700#

Suspender Block (Top) 72 Req'd

Cast steel. 72 @ 114# = 8.200#

Structural steel.

288- Bolts. ¾"ϕ × 0'-2½" @ 0.69	=	199#
288- " ¾"ϕ × 0'-4" @ 0.88	=	255
		453#

Total Wt. of Suspender Blocks.

Cast steel.	8.200#
Struc. steel.	453
	8.653#

Suspender Block (bottom) 72 Req'd.

Cast steel. 72 @ 94# = 6780#

Pins for Suspender

structural steel.	144- Pins	2¼"ϕ × 0'-5¼"	@ 5.9#	= 850#
	288- Nuts.		@ 1.1	= 320
				1.170#

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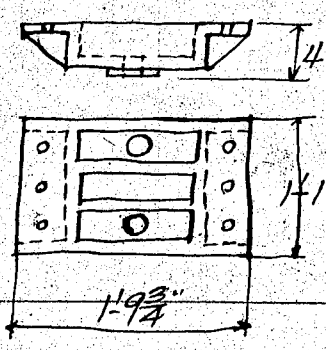
CALCULATIONS FOR

Cast steel Block for cable anchorage.



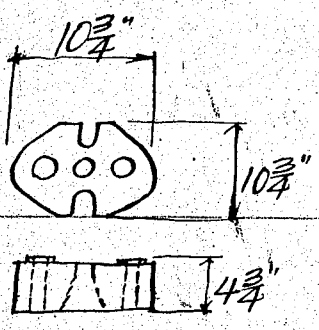
Cast steel block 13"×4"×1'-9¾" 76 Req'd.

76 @ 177# = 13450#

Total Wt.
13450#
315
13765#

Structural steel.

465 Bolts ¾"φ × 0'-2½ @ 0.69 = 315#

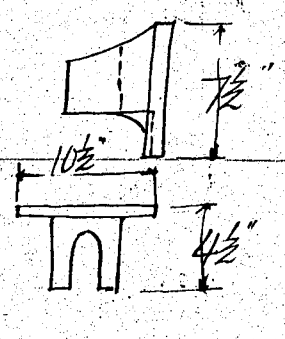


Cast steel Block 10¾" × 4¾" × 0'-10¾" 76 Req'd.

76 @ 117# = 8992#

Total wt.
8992#

Cast steel Block 7½ × 4½ × 0'-10½ 20 Req'd.

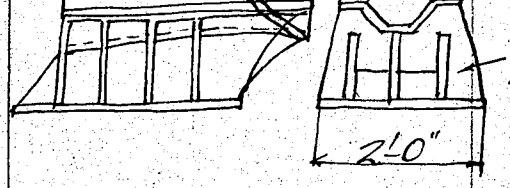


Cast steel.
20 @ 33.4# = 668#

Struc. steel.
120 Bolts ¾"φ × 0'-2½" @ 0.69# = 82#

Total wt
668
82
750

Cable saddle at anchorage. 4 Req'd.



Cast steel Cap.
4 @ 99# = 396#

Cast steel Pedestal.
4 @ 725# = 2900#

total Wt. of Cast steel 396
2900
3296#

Total Wt. of Cable saddle
Cast steel 3296#
Struc. steel. 334
3630#

Structural steel.

48 Bolts ¾"φ × 0'-2¾" @ 0.72 = 36#
24 anchor Bolts 1"φ × 1'-6" @ 1.24 = 298
334#

Shoes-Casting ETC 74.174 ÷ 2240 = 33.13

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CALCULATIONS FOR

Summary for Suspension Bridge

Stiffening trusses		854, 907 #	381. 655 TONS
Bottom Laterals		45, 808	20. 450
Floor Beams		155. 046	69. 217
Side struts		4, 430	1. 978
Bottom struts		16, 394	7. 319
Stringer		99. 599	44. 464
Towers		210, 563	94. 001
Suspenders		35, 761	15. 965
Anchorage for truss		11, 942	5. 331
" " cable		61. 242	27. 340
Shoes, castings ETC.		74, 174	33. 113
		1, 569, 866	700. 833
Rivet heads	7/8"φ		23. 716 TONS
" "	3/4"φ		9. 581
			33. 297
Cable 38 - 902'-0" × 4. 605'# @ 157. 841#			70. 464 TONS
	Total sum.		804. 594 TONS

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CALCULATIONS FOR

Description	No	Section in ins	Length ft	Length ins	Wt in lbs one ft	Wt in lbs one piece	Total wt in lbs	Remarks
		Materials for Simple Span						
		2 Spans Required						
		Upper Chord Lo—U1. 4 Reqd						
Channels	2	12 x 3½	19	6	26.10	508.950	1018	
Tie Pls	3	21 x ⅜	1	9	26.78	46.865	141	
Pl	1	12 x 5/16	2	9	12.75	35.063	35	
Ls	2	3 x 3 x 5/16	1	5	6.18	18.662	17	
Bent Pl	1	21 x ⅜	4	3	26.78	113.815	114	
"	2	3½ x ½	2	1	5.95	12.396	25	
SPl Pls	2	13 x ⅜	1	5	16.58	23.489	47	
Guss Pls	2	31 x ⅜	3	8½	39.53	146.590	293	
Latt Bars	24	2½ x ½	2	4	4.25	9.917	238	
							1.928	
								4@1928=7.712
		Upper chord U1—U3 4 Req'd						
Channels	2	12 x 3½	25	8¾	26.10	671.532	1343	
Tie Pls	2	21 x ⅜	1	9	26.78	46.865	94	
" "	1	21 x ⅜	2	0	26.78	53.560	54	
" "	2	15 x ⅜	1	9	19.13	33.478	67	
" "	1	21 x ⅜	3	6	26.78	93.730	94	
SPl Pls	2	3½ x ⅝	3	0	7.44	22.320	45	
Fills	2	10 x ⅛	0	10½	4.25	3.719	7	
SPl Pls	2	10 x ⅜	1	9½	12.75	22.717	45	
Guss Pls	2	28 x ⅜	3	4	35.70	118.999	238	
Pls	2	13½ x ⅜	1	2½	17.21	20.795	42	
Latt Bars	40	2½ x ½	2	4	4.25	9.917	397	
							2426	
								4@2426=9.704
		Upper chord U3—U3 2 Req'd						
Channels	2	12 x 3½	25	6¼	32.88	839.123	1678	
Tie Pl	1	21 x ⅜	2	0	26.78	53.560	54	
" "	2	15 x ⅜	1	9	19.13	33.478	67	
Pls	2	13½ x ⅜	1	2½	17.21	20.795	42	
Latt Bars	40	2½ x ½	2	4	4.25	9.917	397	
Tie Pl	2	21 x ⅜	1	9	26.78	46.865	94	
							2332	
								2@2332=4664

増田淳氏関係資料
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CALCULATIONS FOR

Description	No	Section in ins	Length ft	Length ins	Wt in lbs one ft	Wt in lbs one piece	Total wt in lbs	Remarks
		Lower chord Lo—L2 4 Req'd					47.019	
Channel	2	12 × 3½	24	9½	26.10	647.019	1.294	1.294
Ls	4	4 × 3 × ⅜	3	1	8.50	26.208	105	
Pl	1	12 × ⅜	3	1	15.30	47.174	47	
Ls	2	3 × 3 × 5/16	0	11¼	6.10	5.719	11	
Fills	2	2½ × ⅜	0	2½	3.19	0.664	1	
Pls	2	16 × ⅜	3	0¾	20.40	62.475	125	
Fills	2	12 × ⅜	2	1½	15.30	32.513	65	
Guss Pls	2	36¾ × ⅝	3	2	46.86	148.392	297	
Pls	2	13½ × ⅜	2	0½	17.21	35.138	70	
Fills	2	10 × ⅜	0	10½	4.25	3.719	7	
SPl Pls	2	10 × ⅜	1	9½	12.75	22.844	46	
Guss Pls	2	31½ × ⅜	3	8½	40.17	148.962	298	
Pl	1	12½ × 5/16	1	5½	13.28	19.389	19	
"	1	12½ × 5/16	1	4	"	17.662	18	
"	1	" × "	3	2½	"	42.629	43	
Guss Pls	1	24 × 5/16	3	2½	25.50	81.855	82	
Latt Bars	88	2¼ × 5/16	1	1⅛	2.39	2.614	230	
Pl	1	16 × 5/16	1	10	17.00	31.110	31	
Pl	1	12 × 5/16	1	5	12.75	18.063	18	
Fills	2	8¼ × ⅜	1	0	10.52	10.520	21	
Pl	1	22 × 5/16	3	6½	23.38	82.765	83	
							2.911	4@2.911 = 11.644
		Lower chord L2—L4 4 Req'd						
Channel	2	12 × 3½	25	6	32.88	838.440	1677	
Pls	2	13½ × ⅜	2	0½	17.21	35.138	70	
"	1	16 × 5/16	1	10	17.00	31.110	31	
Latt Bars	92	2¼ × 5/16	1	1⅛	2.39	2.614	241	
Pl	1	12½ × 5/16	1	4	13.28	17.662	18	
							2.037	4@2,037 = 8.148
		2 Req'd						
Guss Pls	2	27½ × ⅜	3	6½	35.07	124.207	248	At Center of span
" "	1	24 × 5/16	3	2½	~~13.28~~ 25.5	81.855	82	
Pl	1	12½ × 5/16	3	2½	13.28	42.629	43	
							373	2@373 = 746

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CALCULATIONS FOR

Description	No	Section in ins	Length ft	Length ins	Wt in lbs one Ft	Wt in lbs one piece	Total wt in lbs	Remarks
			Vertical U1—L1			4 Req'd		
Ls	4	4×3×5/16	11	5¾	7.20	82.650	331	
Fill	1	8¼×3/8	2	3½	10.52	24.109	24	
Pl	1	12½×5/16	3	0	13.28	39.840	40	
"	1	9×5/16	1	7	9.56	15.136	15	
"	1	11½×5/16	1	0½	12.22	12.730	13	
Latt Bars	9	2¼×5/16	1	1⅞	2.39	2.714	24	
							447	4@447=1788
		Vertical (U2—L2)(U4—L4) 6 Req'd						
				11¾		93.449	374	
Ls	4	4×3×5/16	12	~~6¼~~	7.20	~~90.144~~	~~361~~	
Pl	1	8¼×3/8	2	3½	10.52	24.109	24	
"	1	12½×5/16	3	0	13.28	39.840	40	
"	1	9×5/16	1	7	9.56	15.136	15	
"	1	11½×5/16	1	0½	12.22	12.730	13	
Latt Bars	12	2¼×5/16	1	1⅞	2.39	2.714	33	
							~~486~~ 499	6@~~486~~ 499 = ~~2.916~~ 2.994
		Vertical U6—L3 4 Req'd						
				11¾		93.449	374	
Ls	4	4×3×5/16	12	~~6¼~~	7.20	~~90.144~~	~~361~~	
Pl	1	8½×3/8	2	3½	10.52	24.109	24	
"	1	12½×5/16	3	0	13.28	39.840	40	
"	1	9×5/16	1	7	9.56	15.136	15	
"	1	12½×5/16	1	9	13.28	23.240	23	
Latt Bars	11	2¼×5/16	1	1⅞	2.39	2.714	30	
							~~475~~ 488	4@~~475~~ 488 = ~~1.900~~ 1.952
		Lower Lateral L0—L2 2 Req'd						
Ls	4	4×3×5/16	14	5½	7.20	104.100	416	
"	4	" " "	14	9	7.20	106.200	425	
Guss Pl	1	29×3/8	2	8½	36.98	100.153	100	
							941	2@941 = 1882
		Lower Lateral L2—L4 2 Req'd						
Ls	8	4×3×5/16	14	9	7.20	106.200	850	
Guss Pl	1	29×3/8	2	8½	36.98	100.153	100	
							950	2@950 = 1900

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CALCULATIONS FOR

Description	No	Section in ins	Length ft	Length ins	Wt in lbs one ft	Wt in lbs one piece	Total wt in lbs	Remarks
		List of materials for Simple span 102'—0" 2 Spans Req'd						
		End Floor beam 2 Req'd						
Web Plate	1	27 × 5/16	20	9	28.69	595.518	595	
Flg Ls	2	3½ × 3½ × 3/8	20	1⅛	8.50	171.36	343	
" "	2	" " "	19	7⅛	10.40	204.208	408	
Stiff Ls	14	3 × 3 × 5/16	2	2¾	6.10	13.598	190	
Conn Ls	4	4 × 3½ × 3/8	2	0¼	9.10	18.769	75	
Fills	4	3½ × 5/8	1	6½	4.46	6.876	28	
Ls	4	3 × 3 × 3/8	0	6½	7.20	3.846	15	
"	4	" " "	1	1½	7.20	8.100	32	
Pl	2	9 × 5/16	1	5	9.56	13.544	27	
Ls	4	3½ × 3½ × 3/8	1	5	8.50	12.070	48	
							~~1772~~ 1761	~~2@1772 = 3544~~ 2@1761 = 3522
		Intermediate Floor beam FB1 & FB2. 7 Req'd						
Web Plate	1	27 × 5/16	20	9⅜	28.69	595.980	596	
Flg Ls	2	5 × 3½ × 3/8	20	2¾	10.40	204.315	409	
" "	2	" " "	20	9⅜	10.40	216.342	433	
Stiff Ls	14	3 × 3 × 5/16	2	2¾	6.10	13.598	190	
Conn Ls	4	4 × 3½ × 3/8	3	0¼	9.10	27.489	110	
Fills	4	3½ × 5/8	2	6	4.46	11.150	45	
Ls	4	3 × 3 × 3/8	0	6½	7.20	3.900	16	
"	4	" " "	0	10½	7.20	6.300	25	
Pls	2	9 × 5/16	1	1¾	9.56	10.954	22	
Ls	4	3 × 3 × 5/16	7	9	6.10	42.275	189	Side struts
"	4	4 × 3 × 3/8	1	3¼	8.50	10.802	43	
Pls	2	15 × 5/16	1	4½	15.94	21.918	44	
"	2	6 × 5/16	1	1½	6.38	7.178	14	
Ls	4	3 × 3 × 3/8	0	6	7.20	3.600	14	
Ring Fills	8	3" ø × 5/16				0.625	5	
							2155	
								7@2155 = 15085
		Stringer						
I	6	12 × 5/16	13	7⅛	31.99	436.865	2621	
"	18	" "	12	8¾	31.99	407.207	7330	
Conn Pls	42	9 × 5/16	1	0	9.56	9.560	402	
							10353	
								10353

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CALCULATIONS FOR

Description	No	Section in ins	Length ft	Length ins	Wt in lbs one ft	Wt in lbs one piece	Total wt in lbs	Remarks
			Stringer			4 Req'd		
Flg L	1	3×3×5/16	8	8	6.10	52.829	53	
" "	1	" " "	9	5	6.10	57.462	57	
Web Pl	1	12 × 5/16	9	5	12.75	120.063	120	
Flg L	1	3×3×5/8	4	4	7.20	31.200	31	
" "	1	" " "	3	6	7.20	25.200	25	
Web Pl	1	12 × 5/16	4	4	12.75	55.250	55	
Fill	1	3 × 3/8	0	6¼	3.83	1.995	2	
							~~354~~ 343	A @ ~~354~~ 343 = ~~1,416~~ 1,372
			Stringer			12 Req'd		
Flg Ls	2	3×3×5/16	12	8¾	6.1	77.647	155	
Web Pl	1	12 × 5/16	12	8¾	12.75	162.297	162	
							317	12 @ 317 = 3,804

SUMMARY FOR SIMPLE SPAN (Include page 57)

		TONS
TRUSSES	49,352#	22.032
LOWER LATERALS	3,782	1.688
FLOOR BEAMS	18,607	8.307
STRINGERS	15,529	6.933
SHOES, PINS & ANCH. BOLTS	3,176	1.418
	90,446	40.378
RIVET HEADS 3/4" dia	1,667# (3,734#)	1.667

40.378
1.667
FOR ONE SPAN ··· 42.045

FOR TWO SPANS ··· 42.045 × 2 = 84.090

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CALCULATIONS FOR

Description	No	Section in ins	Wt in lbs Ft	Wt in lbs ins	Wt in lbs one Ft	Wt in lbs one piece	Total wt in lbs	Remarks
			Shoe C1	2 Req'd	For	simple Span		
Pls	2	4 1/4 × 3/8	1	10	5.42	9.919	20	
Pls	6	16 1/4 × 3/8	1	0	20.72	20.720	124	
Stiff Ls	8	3 × 3 × 3/8	0	8 1/4	7.20	4.950	40	
Pl	1	8 1/4 × 1/2	0	8 1/2	14.03	9.937	10	4 1/4 + 4 1/4 + 8
Ls	4	4 × 3 1/2 × 1/2	1	0	11.90	11.900	48	
Pls	2	4 × 3/8	1	7	5.10	8.075	16	
Bars	2	2 1/2 × 1/2	1	6 1/2	4.25	6.552	13	
Rollers	5	3 1/2" φ	1	5	32.71	46.448	232	
Ls	2	4 × 3 × 3/8	1	10	8.50	15.583	31	
Cast steel bed pl.	1	22 × 1 1/4	25 1/2		0.283 #/in²	161.693	162	
Tapped bolts	26	5/16" φ	0	1 1/4		0.120	3	
"	10	3/4" φ	0	1 9/16		0.500	5	
							704	2 @ 704 = 1,408
			Shoe C2	2 Req'd				
Pls	6	20 3/4 × 3/8	1	10	26.46	30.176	181	
Stiff Ls	16	3 × 3 × 3/8	0	8 1/4	7.20	4.950	79	
" "	8	" " "	1	1	7.20	7.800	62	
Fills	2	7 1/2 × 1/2	1	1 1/2	12.75	14.344	29	
"	2	2 3/4 × 1/2	1	9	4.68	8.190	16	
"	4	" "	0	3	4.68	1.170	5	
Ls	4	6 × 4 × 1/2	1	10	16.20	29.699	119	
Base Pl	1	19 1/4 × 1	1	10	65.45	119.989	120	
							611	2 @ 611 = 1222
			Pin & Nut					
Pins	4	4 1/4" φ	1	6 1/8	48.23	72.827	291	
Nuts	8					4.600	37	
							328	1 @ 328 = 328
			Anchor Bolt					
Anchor bolts	16	1" φ	1	6		12.400	199	
Hex nuts	16					0.800	13	
Washers	16					0.350	6	
							218	1 @ 218 = 218

18. 90,446# ÷ 2240 = 40.378 TONS ~~× 2 = 80.776 TONS~~

~~85,162# ÷ 2240 = 38.019 TONS × 2 = 76.038 TONS~~

Rivet Head ~~3/4" φ~~ ~~1.667 × 2 = 3.334 TONS~~ For 2 spans

~~80.776 TONS~~
~~3.334~~
~~Tot. wt. 84.110~~

~~76.038 TONS~~
~~3.334~~
~~Total Wt = 79.372~~

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CALCULATIONS FOR

Lists of Materials for Hakuchi-Bashi.

Super structure for Suspension span

Expansion Joint EX1 1 Required

1 checkered Pl	63⅜" . ⅜"	. 21'-8"	@ 171.172	=	171
14 Anchored Plo	3. 5/16	. 1'-2"	@ 3.722	=	52
2 flange Ls	3.3. 5/16	. 20'-9⅛"	@ 126.638	=	253
1 web Pl	10. 5/16	. 20'-0"	@ 212.600	=	213
2 Plo	4. 3/8	. 1'-3"	@ 6375	=	13
1 fill	3. 1"	. 0'-5"	@ 425	=	4
1 Pl	9. 3/8	. 20'-0"	@ 2296	=	230
1 L	3.3. 5/16	. 20'-0"	@ 122.0	=	122
1 bar	1". 3/8"	. 20'-0"	@ 25.6	=	26
					1084#

Expansion Joint EX2 1 Required

1 checkered Pl	11⅜ . ⅜"	. 21'-8"	@ 152.518	=	153
14 Anchored Plo	3. 5/16	. 1'-2"	@ 3.722	=	52
2 flange Ls	3.3. 5/16	. 20'-9⅛	@ 126.638	=	253
1 web pl	10. 5/16	. 20'-0"	@ 212.600	=	213
1 flange L	3.3. 5/16	. 20'-0"	@ 122.0	=	122
1 " "	3.3. 5/16	. 20'-9⅜	@ 126.75	=	127
1 web pl	10. 5/16	. 20'-0"	@ 212.600	=	213
2 Plo	4. 3/8	. 1'-3"	@ 6375	=	13
1 fill	3. 1	. 0'-5"	@ 425	=	4
1 Pl	7. 3/8	. 20'-0"	@ 178.60	=	179
1 bar	1. 3/8	. 20'-0"	@ 25.60	=	26
					1355#

Summary EX1 1084#
EX2 1355
2439# or 1.089 tons.

Drains (Cast iron) 52 Required 52 @ 87# = 4524# or 2.019 tons

Handrails

Handrail Panel A 2 Required

1-2"φ gas pipe	. 2'-3½"	@ 8.217	=	8
2-1¼"φ "	. 2'-3½"	@ 5.117	=	10
3-bars 2½ . ¾	. 2'-2½"	@ 14.089	=	42
			60 . 2 =	120#

Handrail Panel B 2 Required

1-2"φ gas pipe	. 2'-0"	@ 7.304	=	7
2-1¼"φ "	. 2'-0"	@ 4.544	=	9
3-bars 2½ . ¾	. 2'-2½	@ 14.089	=	42
			58 . 2 =	116#

Handrail Panel D 204 Required

1-2"φ gas pipe	. 6'-4"	@ 27.971	=	23
2-1¼"φ " "	. 6'-4"	@ 14.291	=	29
12 bars 2½ . ¾"	. 2'-2½	@ 14.089	=	169
			221 . 204 =	45.084#

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CALCULATIONS FOR

REVISED

DATE_____

DATE_____

DATE_____

DATE_____

Panel E. 4 Req'd.

1 - 2"φ gas pipe × 5'-4" @ 19.326 = 19
2 - 1¼"φ " " × 5'-4" @ 12.023 = 24
11 - Bars 2½ × ¾ × 2'-2½ @ 14.089 = 155
198 × 4 = 792#

Bolts 208 - ½"φ × 0'-3" @ .3 = 62
" 472 - ⅝"φ × 0'-2 @ .4 = 190
252#

Handrail Post P1. 14 Req'd.

86#/piece 86@14 = 1204#

Post P2 104 Req'd.

69#/piece 69@104 = 7176#

Past P3 104 Req'd.

100#/piece 100@104 = 10400#

Near End Post 2 Req'd.

120#/piece 120@2 = 240#
19020# or 8.491 tons

Total weight of Panels and Bolts.

Panel A = 120#
" B = 116
" D = 45.084
" E = 792
Bolts = 252
46364#

or 20.700 tons.

Total weight of handrails.

Panels = 20.700 tons.
Posts = 8.491
29.191 tons.

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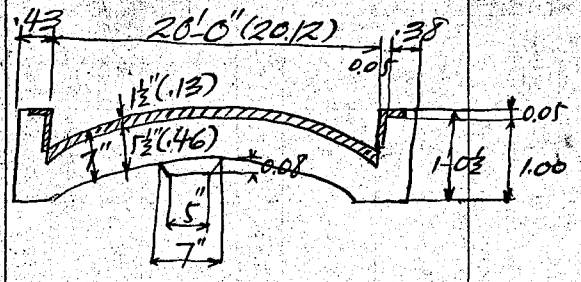
CALCULATIONS FOR

Materials for super structure

Concrete in slab and Curb.



sectional area of slab.

$(20.12 \times 0.46) + 2 \times (1.0 \times .38) = 9.26 + 0.76 = 10.02$ 平尺

$\frac{.42+.59}{2} \times 0.08 = \frac{0.04}{10.06}$ 平尺

1:2:4 concrete in slab A. length 5'-0"(5.13R) 10 Required

$1.006 \times 5.13 \times$ = 5.1608 平尺 or 2.39 立坪

10@2.39 = 23.90 立坪

1:2:4 conc. in slab B. length 25'-6"(25.649R) 2 Req'd.

10.06×25.649 = 258.03 平尺 or 1.19 立坪

2@1.19 = 2.38 立坪

1:2:4 conc. in slab C. length 51'-11"(52.22R) 2 Req'd.

10.06×52.22 = 525.33 or 2.43 立坪

2@2.43 = 4.86 立坪

Total Volume of 1:2:4 conc. for slab.

slab A	10 @ 2.39	= 23.90
" B	2 @ 1.19	= 2.38
" C	2 @ 2.43	= 4.86
		31.14 立坪

Pavement area over slab.

10- slab A $20.12 \times 5.13 = 1032.16$ 平尺 or 28.671 立坪

10@28.671 = 286.71 立坪

2- " B $20.12 \times 25.649 = 516.06$ 平尺 or 14.335 立坪

2@14.335 = 28.67 立坪

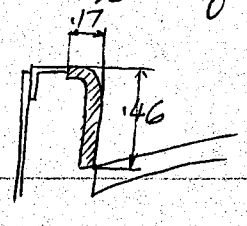
2- " C $20.12 \times 52.22 = 1050.666$ 平尺 or 29.185 立坪

2@29.185 = 58.37 立坪

Total area of pavement over slab

slab A	10 @ 28.671	= 286.71
" B	2 @ 14.335	= 28.67
" C	2 @ 29.185	= 158.37
Over Abutment		= 1.23 面坪
		375.28

Finish of curb. (人造花崗岩仕上げ)



.46
.17
.63

10-slab A. $0.63 \times 5.13 \times 2 = 64.638$ 平尺 or 1.800 立坪

10 @ 1.80 = 18.00 立坪

2- " B $0.63 \times 25.649 \times 2 = 32.318$ or 0.898 "

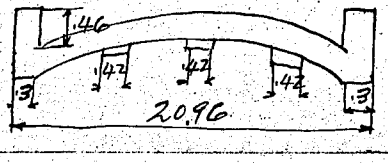
2@.0898 = 1.80 "

2- " C $0.63 \times 52.220 \times 2 = 65.797$ or 1.827

2@1.827 = 3.65

total 23.45 立坪

Forms.



(20.96+.92) - 1.86 = 20.02R say 20.0R

20. × 670.00 = 13400 平尺 or 3.72 立坪

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CALCULATIONS FOR

Materials for superstructure.

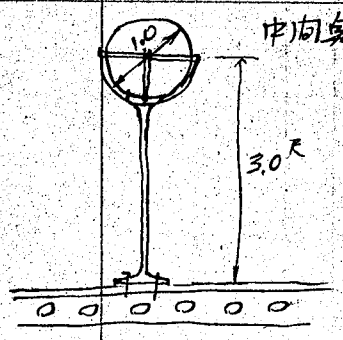
Reinforcement in slab (See sheet no. 4)

slab A	10 @ 2930	= 29.300 #	or 13.080	tons
" B	2 @ 1.447	= 2.894	or 1.292	
" C	2 @ 2.988	= 5.976	or 2.668	
		total	17.040	tons

Summary of Materials for superstructure.

Expansion joints (Structural steel)	= 1.089	tons
Drains (cast iron)	= 2.019	tons
Handrails {Panels (steel)	= 20.700	"
Posts (cast iron)	= 8.491	" = 29.191 tons
Reinforcement	= 17.040	tons

Concrete (1:2:4) in slab and copings.	31.14 立坪
Pavement area in slab	375.28 方坪
Finished of curb (人造 洗出仕上げ)	23.45 方坪
Forms	372.0 方坪



中間瓦燈 10 個

Upper chord.

Length of gas pipes.

2" φ

Panel A	2 @ 2'3"	= 4'-6"	
" B	2 @ 2'-1"	= 4'-2"	
" C	4 @ 2'-0"	= 8'-0"	
" D	268 @ 6'-3½"	= 1686'-2"	
" E	4 @ 5'-3½"	= 21'-2"	
End panel	2 @ 1'-10"	= 3'-8"	
" "	2 @ 1'-8"	= 3'-4"	
		1731'-0"	

1¼" φ 1731'-0" × 2 = 3462'-0"

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CALCULATIONS FOR

Materials of super structure for girder span.

Concrete (1:2:4)

in slab. (5½" thickness .46) length 20.12+.1 = 20.22 R

Sectional area = .46 × 20.22 = 9.30 平方尺

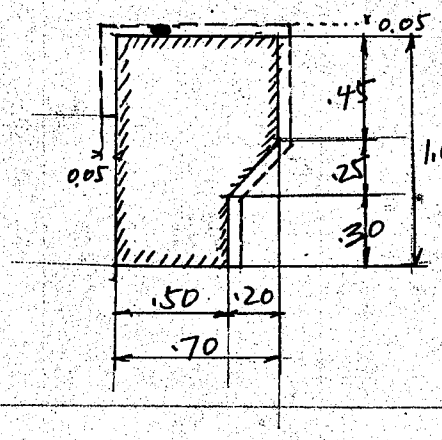
Volume. = 9.30 × 66.64 = 619.75 2@619.75 = 1239.50 立方尺

filling conc. on girder flanges. = 2.21 2@2.21 = 4.42

1243.92 立方尺

or 5.751 立方米.

in Copings.



Sectional area of copings.

.45 × .7 = 0.315

$\frac{.5+.7}{2}$ × .25 = 0.150

.3 × .5 = 0.150

0.615 平方尺

2@0.615 = 1.23 平方尺

Volume = 1.23 × (66.64+.3) = 82.34

1.23 × (66.64+.12) = 82.11

164.45 立方尺

or 0.761 立方米

total Volume of conc (1:2:4) =

in slabs. = 5.751

in copings. = 0.768

6.512 立方米

Reinforcement in slab (D.) see sheet no. 21.

2@ 2.027 = 4.054 tons

Pavement area of Roadway.

width of Roadway. = 20.12

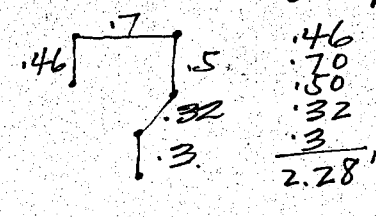
length = 66.64.

area = 20.12 × 66.64 = 1340.8 平方尺

or 37.24 立方米

2@37.24 = 74.48 立方米

Finished area of copings.



.46
.70
.50
.32
.3
2.28 R

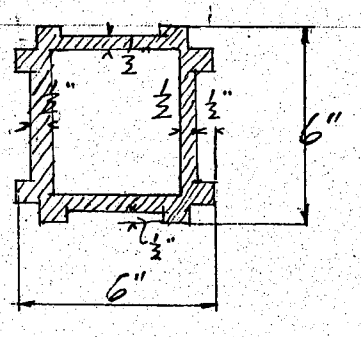
2 × 2.28 × (66.64 + 0.3) = 305.25

2 × 2.28 × (66.64 + 0.12) = 304.43

609.68 平方尺 or 16.94 立方米

Intermediate lamp post. (Cast iron). 2 Required.

(approximate weight.)



height = 3'-8"

Mean Sectional area. = 5 × 5 = 25

4 × 4 = 16

11 □"

Volume. = 11.0 × 44.0 = 484.0 □"

Base " = 11.0 × 10.0 × ⅓ = 82.5

566.5 □"

weight = 566.5 × 0.26 = 142.0

top Casting 30.0

172.0 # for castings.

gas pipe 1-2½"φ @ 5.79 × 4.5 = 26.0

" " 1-2"φ @ 3.65 × 4.75 = 17.0

215.0 # for total wt of one post.

2@215.0 = 430.0 #

{ 4-½"φ × 0'-2½" Bolt.
1.0 dia globe.

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CALCULATIONS FOR

List of materials for Girder span (superstructure).

Handrails.

Handrail Posts P1. 26 Required. (cast iron)

95# / Piece 26@95 = 2470#

Handrail Posts P2 2 Required (cast iron)

120# / Piece 2@120 = 240#

Total weight of Posts.

P1 = 2470
P2 = 240
2710#
or 1.210 tons.

Handrail Panels.

Panel A. 2 Required.

1-2"φ gaspipe @ 3.652 × 2'-6 = 9.
2-1¼"φ " " @ 2.272 × 2'-6 = 11.
4-Bars 2½ × ¾ @ 6.38 × 2'-2½ = 56
76 × 2 = 152#

Panel B. 2 Req'd.

1-2"φ gaspipe @ 3.652 × 2'-7" = 9.
2-1¼"φ " @ 2.272 × 2'-7" = 12.
4-Bars. 2½ × ¾ @ 6.38 × 2'-2½ = 56
77 × 2 = 154#

Panel C. 2 Required

1-2"φ gas pipe @ 3.652 × 6'-2¾" = 23
2-1¼"φ " " @ 2.272 × 6'-2¾" = 28
12-Bars 2½ × ¾ @ 6.38 × 2'-2½ = 169
220 × 2 = 440#

Panel D. 22 Required

1-2"φ gas pipe @ 3.652 × 6'-4" = 23
2-1¼"φ " " @ 2.272 × 6'-4 = 29
12-Bars 2½ × ¾ @ 6.38 × 2'-2½ = 169
221 × 22 = 4862#

Panel E 2 Req'd.

1-2"φ gas pipe @ 3.652 × 1'-10¾ = 7
2-1¼"φ " " @ 2.272 × 1'-10¾ = 9
3-Bars 2½ × ¾ @ 6.38 × 2'-2½ = 42
58 × 2 = 116#
5724#
or 2.555 tons.

Total weight of handrails.

Posts. = 1.210.
Panels = 2.555
3.765 tons

31.8# Per ft of bridge.

Forms for slab. 83.0 平方呎

Drains (cast iron.) 12 Required.

87# / Piece 12@87 = 1044# or 0.466 ton.

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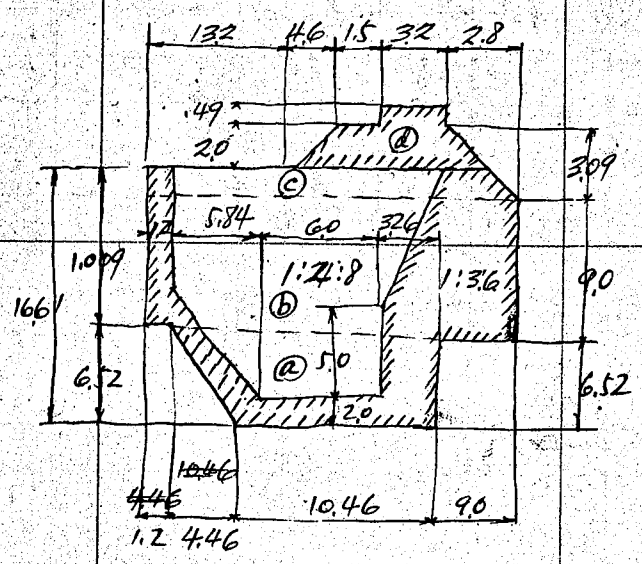
CALCULATIONS FOR

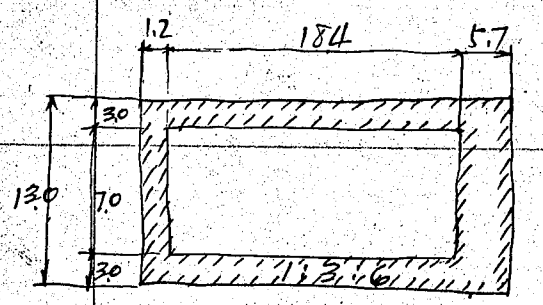
Materials for Substructures.

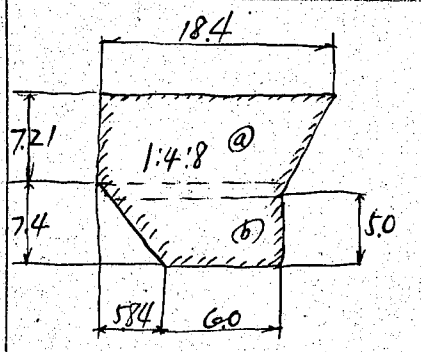
Revised 14-12-25

Abutment A1 2 Req'd.

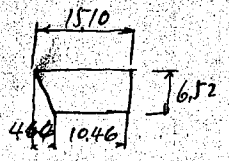
Concrete (1:3:6)





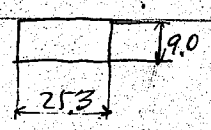


ⓐ area.



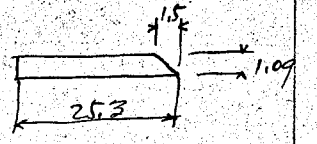
$\frac{10.46 + 15.10}{2} \times 6.52 = 83.33$ 平尺

ⓑ area



$25.3 \times 9.0 = 228.00$ 平尺

ⓒ area

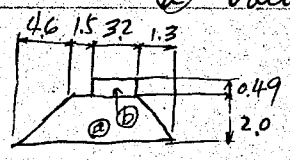


$25.3 \times 1.09 - \frac{1.5 \times 1.09}{2} = 26.80$ 平尺

338.13 平尺

Volume $338.13 \times 13.0 = 4395.69$ 平尺

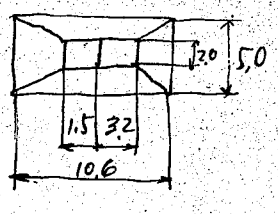
ⓓ Volume.



ⓐ $\frac{(4.7 \times 2.0) + (10.6 \times 5.0)}{2} \times 2 = 62.4$ 立尺

ⓑ $3.2 \times 2.0 \times 0.49 = 3.14$

65.54 立尺



total Volume of Abutment A1.

4395.69
65.54
4461.23 立尺

ⓐ area $\frac{18.4 + 13.24}{2} \times 7.21 = 114.6$ 平尺

ⓑ " $\frac{6.0 + 11.84}{2} \times 7.4 = 66$

182.7 平尺

Volume $182.7 \times 7.0 = 1278.9$ 立尺

total Volume of Conc.(1:3:6) $= 4461.23 - 1225 = 3182.33$ 立尺 or 147.4 立坪

$2 @ 14.74 = 29.48$ 立坪

total Volume of Conc.(1:4:8) $= 12.78$ 立尺 or 5.92 立坪

$2 @ 5.92$ 立坪 $= 11.84$ 立坪

Summary of materials for abutment A1.

Concrete (1:3:6)			29.48 立坪
" (1:4:8)			11.84 "
Forms.		2@28	=56.00 立坪
Excavation for earth		2@20	=40.00 立坪
" for Rock		2@5.5	=11.00 "
Reinforcement.			=0.602 Ton.

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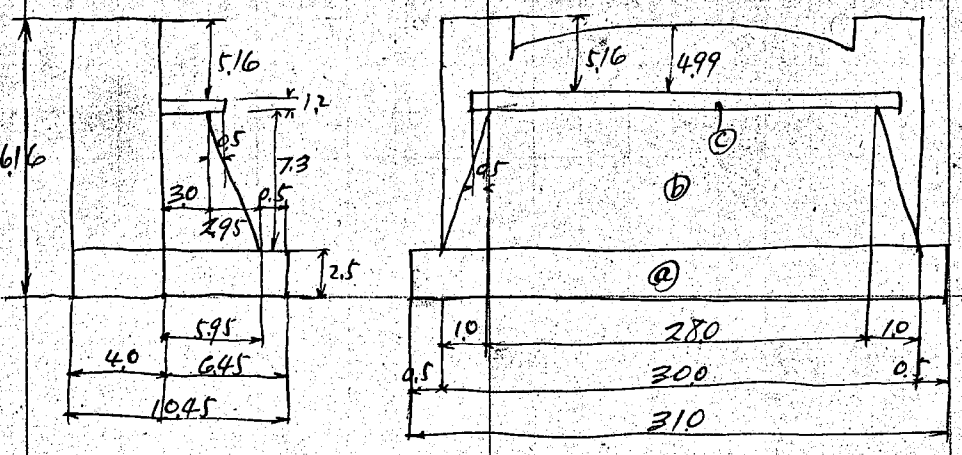
CALCULATIONS FOR

Materials for substructures.

Abutment A2 1 Req'd.

Concrete (1:3:6)



Volume of abutment:

Ⓐ $6.45 \times 31.0 \times 2.5 = 500.0$ 立米

Ⓑ $V = \frac{h}{3}(B + b + \sqrt{B.b})$

where $h = 7.3$

B = top area $= 3 \times 28 = 84$

b = bott. " $= 5.95 \times 30 = 178$

and $\sqrt{B.b} = \sqrt{84 \times 178} = 122.5$

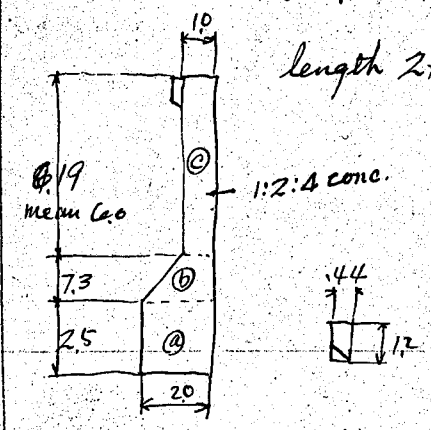
$\therefore$ Volume $= \frac{7.3}{3} \times (84 + 178 + 122.5) = 937$ 立米

Ⓒ $1.2 \times 3.5 \times 29.0 = 124$ 立米

total Volume of Abutment

Part Ⓐ 500 立米
" Ⓑ 937 "
" Ⓒ 124 "
1561 立米

Volume of parapet wall:



length 21.0

Ⓐ area $2 \times 2.5 = 5.0$ 平米

Ⓑ " $\frac{1+2}{2} \times 7.3 = 11.0$

Ⓒ " $1. \times 6 = 6.0$

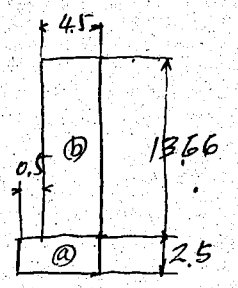
22.0 平米

Volume $22 \times 21 = 462$ 立米

$1.2 \times .44 \times 26.0 = 14$

476 立米

Volume of wing walls:



Volume Ⓐ $2.5 \times 5.0 \times 4.0 \times 2 = 100$ 立米

" Ⓑ $4.5 \times 4 \times 13.66 \times 2 = 492$

592 立米

total Volume of Conc (1:3:6) for abutment A2.

abutment	1561 立米		
Parapet wall	496		
Wing wall	592		
	2629 立米	or	12.17 立坪

Summary of materials for abutment A2

Reinforcement for abutment A2 (see sheet no. 22)	0.132 ton
Forms. " " "	40.00 立坪
Excavation " " " for earth	7.00 立坪
" " " " Rock	5.00 "
Concrete (1:3:6)	12.17 "
" (1:2:4)	0.65 "

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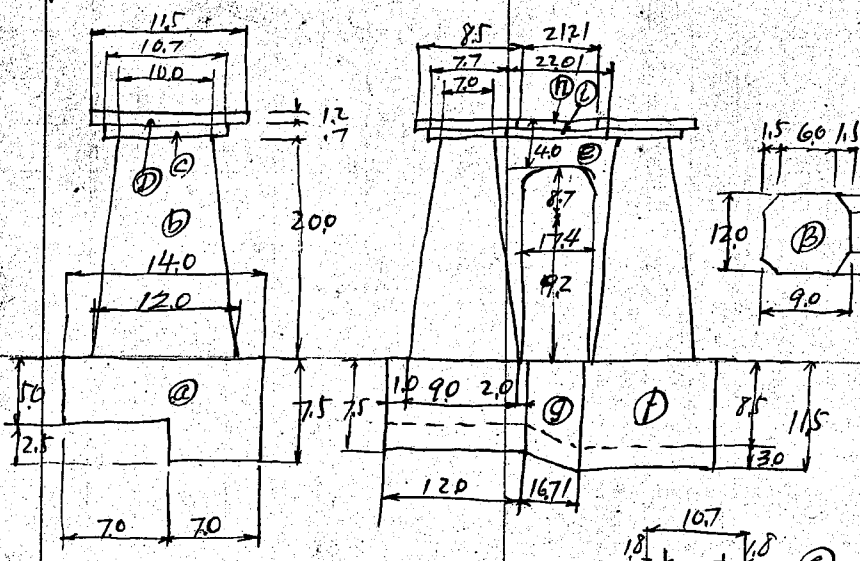
CALCULATIONS FOR

Materials for substructures.

Pier P1



Volume ⓐ $(14.0 \times 12.0 \times 5.0) + (7.0 \times 12.0 \times 2.5) = 1050$ 立米

〃 ⓑ $V = \frac{h}{3}(B + b + \sqrt{Bb})$

Where $h = 20.0$

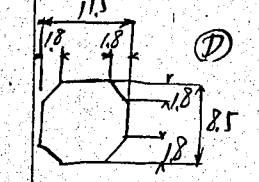
$B = (9.0 \times 12.0) - (1.5 \times 1.5 \times 2) = 103.5$

ⓑ $= (7.0 \times 10.0) - (1.5 \times 1.5 \times 2) = 66.5$

$\therefore$ Volume $= \frac{20}{3} \times (103.5 + 66.5 + \sqrt{103.5 \times 66.5})$

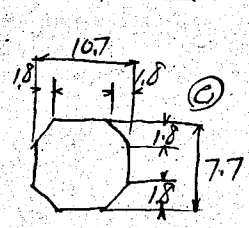
$= \frac{1}{3} \times 20 \times 251.6 = 1678$ 立米

$2 @ 1668 = 3356$ 立米



Volume ⓓ $\{(11.5 \times 8.5) - (1.8 \times 1.8 \times 2)\} 1.2 = 109.5$ 立米

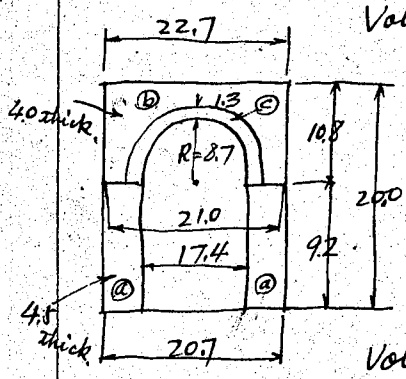
$2 @ 109.5 = 219$ 立米



Volume ⓒ $\{(10.7 \times 7.7) - (1.8 \times 1.8 \times 2)\} \times 0.7 = 53.0$ 立米

$2 @ 53.0 = 106$ 立米

ⓔ



Volume ⓔ

Volume ⓐ $\left(\frac{21.0 + 20.7}{2} \times 9.2 - 17.4 \times 9.2\right) \times 4.5 = 144$

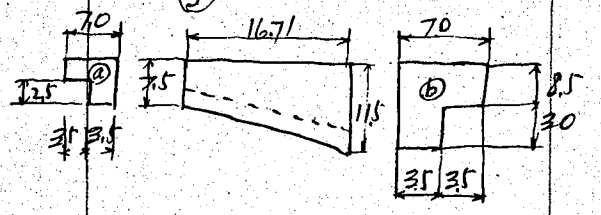
〃 ⓑ $\left(\frac{21.0 + 22.7}{2} \times 10.8 - \frac{\pi \times 20^2}{4} \times \frac{1}{2}\right) \times 4.0 = 316$

〃 ⓒ $\left(\frac{\pi \times 20^2}{4} - \frac{\pi \times 17.4^2}{4}\right) \times 4.5 \times \frac{1}{2} = 171$

631 立米

Volume ⓕ $12 \times 14 \times 8.5 + 12 \times 7 \times 3 = 1677$ 立米

ⓖ



Volume ⓖ

area ⓐ $7.0 \times 7.5 - 2.5 \times 3.5 = 43.75$ 立米

〃 ⓑ $7.0 \times 11.5 - 3 \times 3.5 = 70.0$ 〃

Volume $= \frac{43.5 + 70}{2} \times 16.71 = 950$ 立米

Volume ⓗ $21.21 \times 1.2 \times 5.5 = 140$ 立米

〃 ⓘ $22.01 \times 0.7 \times 4.7 = 72$ 〃

Summary of Materials for Pier P1

Concrete (1:3:6)	37.98	立坪
Reinforcement. (see sheet no. 23)	349	Tons
Forms.	86.00	石坪
Excavation Rock	8.00	立坪

Total Volume of conc. (1:3:6) for pier P1

ⓐ	1050	立米
ⓑ	3386	
ⓒ	106	
ⓓ	219	
ⓔ	631	
ⓕ	1680	
ⓗ	960	
ⓘ	72	
	8204	立米 or 37.98 立坪

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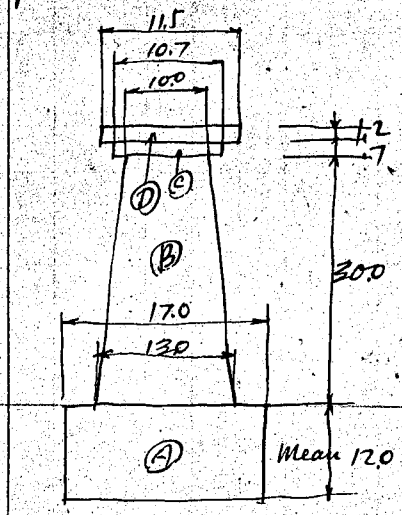
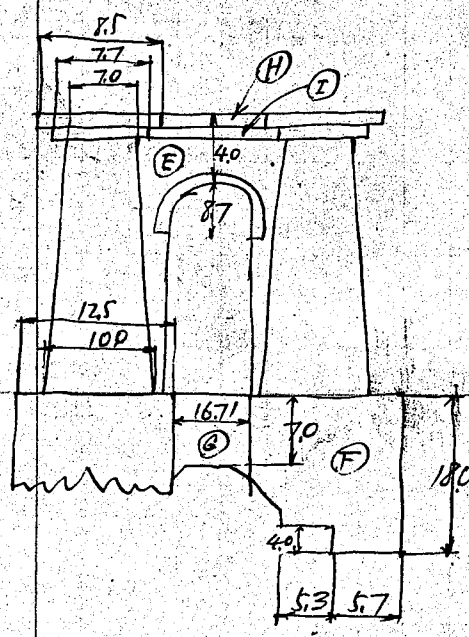
CALCULATIONS FOR

Materials for substructures.

Pier P2



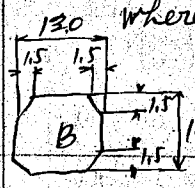


Concrete (1:3:6)

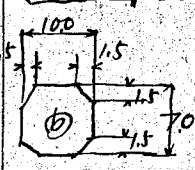
Ⓐ $17.0 \times 12.5 \times 12.0 = 2550$ 立方尺

Ⓑ Volume $= \frac{1}{3}h(B+b+\sqrt{B.b})$

where $h = 30.0$ 尺



$B = (13.0 \times 10.0) - (1.5 \times 1.5 \times 2) = 125.5$ 平尺



$b = (10.0 \times 7.0) - (1.5 \times 1.5 \times 2) = 65.5$ "

Volume $= \frac{30}{3} \times (125.5 + 65.5 + \sqrt{125.5 \times 65.5})$

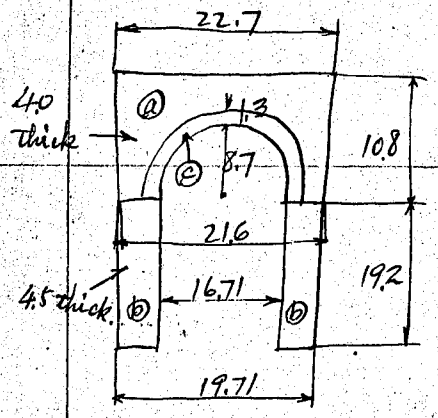
$= 10 \times (125.5 + 65.5 + 90.5) = 2815$ 立方尺

$2.815 \times 2 = 5630$ 立方尺

Ⓒ same as for pier P1 Ⓒ 106 立方尺

Ⓓ " " " " " Ⓓ 219 "

Volume of Ⓔ

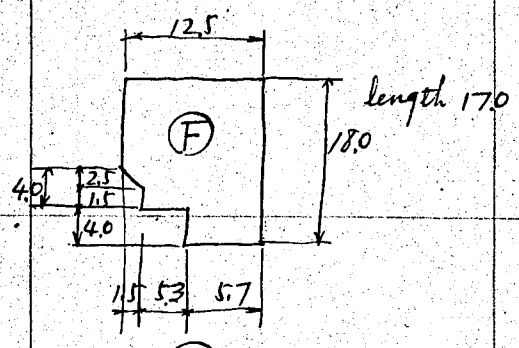


Ⓐ same as for pier P1 Ⓑ $= 316$ 立方尺

Ⓑ $\left(\frac{21.6+19.71}{2} \times 19.2 - 16.71 \times 19.2\right) \times 4.5$ $= 359$ "

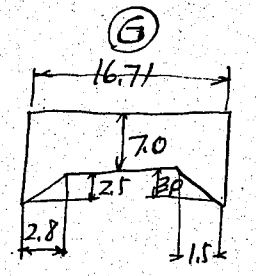
Ⓒ same as for pier P1 Ⓒ $= 171$ "

total Volume of Ⓔ 846 立方尺



area of Ⓕ $12.5 \times 18.0 - \left(\frac{1.5+4.0}{2} \times 1.5 + 4.0 \times 6.8\right) = 195.55$ 平尺

Volume of Ⓕ 195.55×17.0 $= 3326$ 立方尺



Volume of Ⓖ $\left(16.71 \times 7.0 + \frac{3 \times 1.5}{2} + \frac{2.5 \times 2.8}{2}\right) \times 10.0 = 1228$ 立方尺

" " Ⓗ same as for pier P1 Ⓗ $= 140$ "

" " Ⓘ " " " " " Ⓘ $= 72$ "

total Volume of conc. (1:3:6) for pier P2

Ⓐ	2550 立方尺
Ⓑ	5630
Ⓒ	106
Ⓓ	219
Ⓔ Ⓕ	846
Ⓖ	3326
Ⓗ	1228
Ⓘ	140
	72
	14077 立方尺
or	65.17 立坪
	65.17

Summary of Materials for Pier P2

Concrete (1:3:6)	65.17	立坪
Reinforcement (see sheet No. 23)	591.	tons.
Forms. ~~Rock~~	133.00	方坪
Excavation Rock.	10.00	立坪

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CALCULATIONS FOR

Pier P3.

Concrete (1:2:4)

in Copings.

5.95 × 5.70 × 1.0 × 2	=	68.0
~~5.10~~ × 5.35 × .5 × 2	=	27.4
32 × 16.18 × 1.0	=	51.7
26 × 16.78 × 0.5	=	21.8
1678		1689 立米 or 0.783 立坪

in Web.

16.9 × 9.5 × 2.0	=	321.0
100.5 × 2.0	= −	201.0
		120.0
105 × 2.5	=	252.5
100.5 76.97 × 2.5	= −	193.0
		69.0 — 59.
16.66 × 1.45 × 2.5 × 2	=	121.0
16.90 × 8.0 × 2.5	=	338.0
5.0 × 13.13 × 2.5	=	164.2
~~812.2~~ 802.2		立方尺 or 3.760 立坪

Shafts.

$\frac{4.5 + 6.78}{2} \times 34.16 \times 4.75 \times 2$ = 1832.0 立方尺 or 8.500 立坪.

in Base. 5.0 × 11.0 × 8.5 × 2 = 935.0 立方尺 or 4.375 立坪

in truss bearings. 3.0 × 2.5 × 0.86 × 2 = 12.9 立方尺 or 0.060 立坪

77.428 立坪

Concrete (1:3:6)

in Base.

6.5 × 15.0 × 12.5	=	1220.0
6.5 × 6.0 × 9.13	=	356.0
7.5 × 6.5 × 15.0	=	730.3
8.5 × 6.0 × 15.0	=	765.0
		3071.3 立方尺 or 14.219 立坪

Reinforcement. (see sheet no. 24) 11.998 # or 5.401 tons.

Anchored Bars for Pier P3. 20 - 1¼"φ × 10.0 @ 41.72 = 834.40 # or 0.372 ton.

Forms. 91.00 面坪

Excavation. (Rock) 6.00 立坪.

Summary of Materials for pier P3.

Concrete	(1:2:4)	17.428 立坪.
"	(1:3:6)	14.219 立坪.
Reinforcement.		5.401 tons.
Anchored Bars		0.372 "
Forms		91.00 面坪
Excavation.		6.00 立坪

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CALCULATIONS FOR

Pier P4.

Concrete (1:2:4)

in Copings. Same as for P3. = 168.90 主尺 or 0.783 主坪

in Web.

16.9 × 9.5 × 2.0 = 321.0
100.5 × 2.0 = 201.0
100.5 — 108 × 2.5 = ~~260.0~~ 120.0
76.97 × 2.5 = 193.0

~~69.0~~ 59.0

16.66 × 1.45 × 2.5 × 2 = 121.0
5.18 × 16.9 × 2.5 = 219.0
5.0 × 13.13 × 2.5 = 164.2
~~6933~~ 683 主尺 or 3210 主坪

Shaft Piers.

$\frac{4.5 + 6.59}{2}$ × 31.34 × 4.75 × 2 = 1649.0 or 7.630 主坪

in Base. Same as for Pier P3. = 935.0 or 4.325 主坪
15.948 主坪

Concrete (1:3:6)

in Base.

6.5 × 15.0 × 12.5 × 2 = 2440.0
6.5 × 6.0 × 9.13 = 356.0
2796.0 or 12.944 主坪

Reinforcement (see sheet no. 24) 11.132 # or 4.970 tons.

Anchored Bars for Pier P4 20-1¼"φ × 10.0 @ 41.72 = 834.40 # or 0.372 ton

Forms. 86.0 面坪

Excavation. (Rock.) 4.0 主坪

Summary of Materials for Pier P4.

Concrete (1:2:4)	15.948	主坪
" (1:3:6)	12.944	主坪
Reinforcement.	4.970	tons.
Anchored bars.	0.372	ton.
Forms.	86.00	百坪
Excavation.	4.00	主坪

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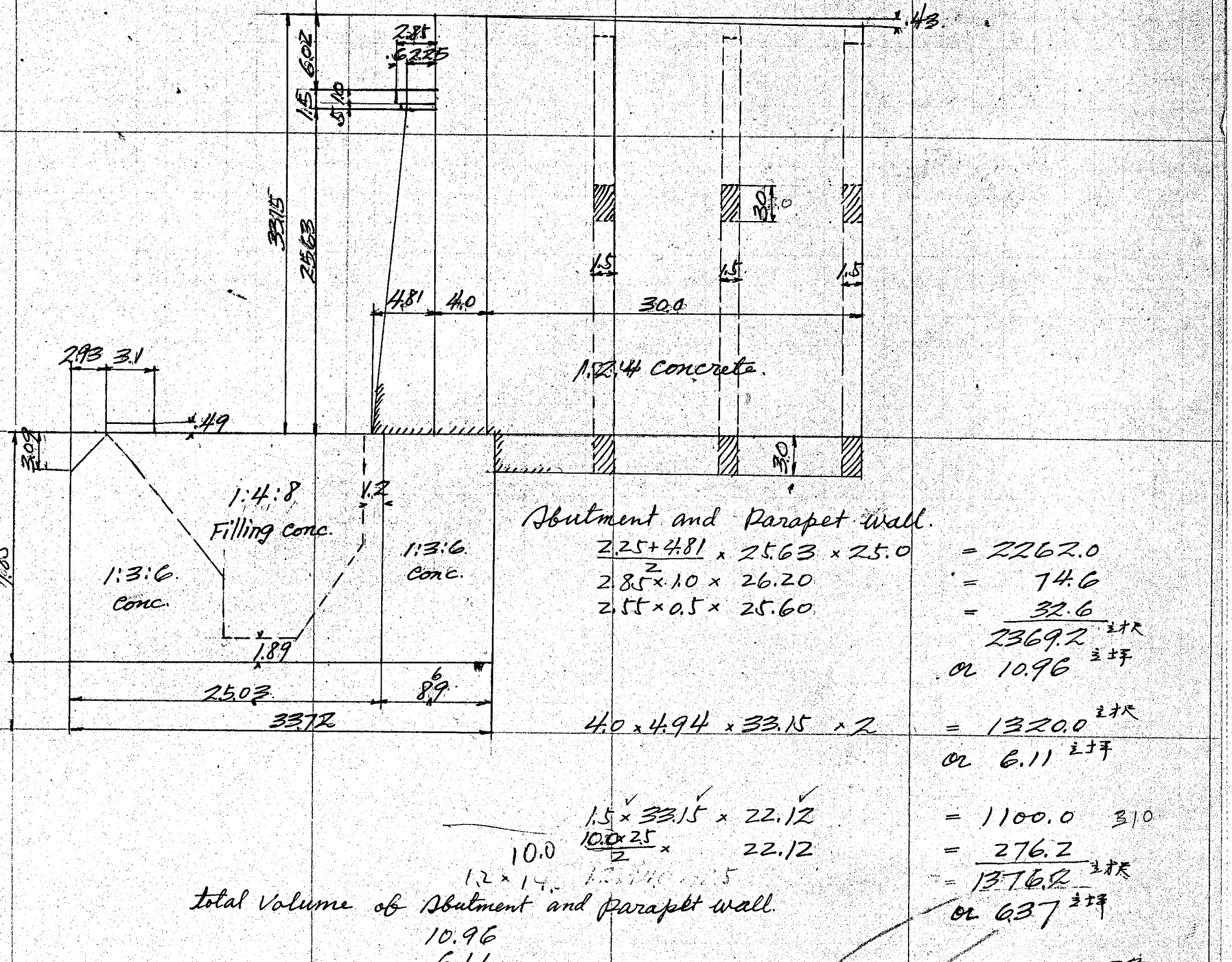
CALCULATIONS FOR

Abutment A3.
in Concrete (1:2:4) for abulment.



Abutment and Parapet wall.

$\frac{2.25+4.81}{2} \times 25.63 \times 25.0 = 2262.0$
$2.85 \times 1.0 \times 26.20 = 74.6$
$2.55 \times 0.5 \times 25.60 = 32.6$
2369.2 立方尺
or 10.96 立坪

$4.0 \times 4.94 \times 33.15 \times 2 = 1320.0$ 立方尺
or 6.11 立坪

$1.5 \times 33.15 \times 22.12 = 1100.0$ B10
$\frac{10.0 \times 2.5}{2} \times 22.12 = 276.2$
1376.2 立方尺
or 637 立坪

1376.2
310
1687.2
or 7.81

total Volume of Abutment and parapet wall.

10.96
6.11
6.37 — 781
23.44 立坪.
2406

Wing wall and base.

$5.5 \times 3.0 \times 29.5 \times 2 = 975.0$ 立方尺 or 4.51 立坪 for base

$\frac{32.72+33.15}{2} \times 30.0 \times 1.5 \times 2 = 2964.0$ 立方尺 or 13.72 立坪 for wing wall

$\frac{12.0 \times 3.0}{2} \times 1.5 = 27.0$
$3.0 \times 20.0 \times 1.5 = 90.0$
$\frac{1.5 \times 1.5}{2} \times 1.5 \times 2 = 3.5$
102.5 6@102.5 = 723.0 立方尺 or 3.35 立坪

$1.5 \times 3.0 \times 20.0 \times 3 = 270.0$
$1.5 \times 3.0 \times 19.0 \times 3 = 256.5$
526.5 立方尺 or 2.44 立坪.

total Volume of wing wall and base.

24.02 立坪

total Volume of 1:2:4 conc.

47.46 立坪 — 4890

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CALCULATIONS FOR

List of Materials for Hakuchi-Bashi

Abutment A3.

1:3:6 Concrete in cable anchorage

2.0 × 3.1 × 0.49	3.04	
18.5 × 25.03 × 13.0	= 6019.72	
3.09/2 × 2.93 × 13.0	= − 58.85	
	5963.91 × 2 =	11927.82
1.2 × 14.0 × 18.5	=	311.00
18.5 × 8.69 × 30.0	=	4822.95
	17061.77 立方尺 or	78.99 立方米
Less 1:4:8 Concrete filling		14.66 "
		64.33 "

1:4:8 Concrete filling 2 @ 7.33 = 14.66 立方米

Summary of Concrete in Abutment A3

Concrete (1:2:4)	= 47.46 立方米	
(1:3:6)	= 64.33 "	
(1:4:8)	= 14.66 "	

Reinforcing bars (see sheet no 25).

in abutment	8.855 tons
in cable Anchorage	0.704 "
	9.559 "

Forms in abutment = 25.80
in cable anchorage = 7.10
329.0 平方米

Excavation 68.0 立方米
Earth excavation assumed as 13.6 立方米
Rock Excavation " " 54.4 "
68.0 "

Lamp Post. (Details on sheet no 21).

4 Required

Concrete (1:2:4)

3.0 × 3.0 × 4.0	= 36
2.5 × 2.5 × 1.0	= 6
1.5 × 1.5 × 4.4	= 10
1.2 × 1.2 × 1.0	= 2
	54 立方尺 or 0.25 立方米

Reinforcing bars 108.3# per post or 0.048 tons

Gas pipe 2" dia 13.0 long 1' milk white globe-finish (人造石上) 3.5 平方米 forms 2.5 平方米

Handrails on abutment A3

4 - 2"ϕ gas pipes	× 4'-7"	@ 16.75#	= 67#	3.652# per ft
8 - 1¼"ϕ " "	× 4'-7"	@ 10.22	= 82	2.272# " "
88 - Bars 2½ × ¾	× 2'-2½"	@ 14.089	= 1240	
12 collars		@ 0.743	= 9	
24 "		@ 0.343	= 8	
8 - 2"ϕ gas pipes	× 4'-4½"	@ 15.98	= 128	3.652# per ft
16 - 1¼"ϕ " "	× 4'-4½"	@ 9.94	= 158	2.272# " "
6 - posts P1		@ 86.00#	= 516	
24 - bolts ⅝"ϕ	× 0'-6"	@ 0.75#	= 18	
			2226#	or 0.994 tons

Concrete Handrail post 6 Required Size 1.2 × 1.0 × 3.4
see drawing # 21.

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CALCULATIONS FOR

Descr.	No.	Section in ins	Length ft.	Length ins.	Wt in lbs One ft.	Wt in lbs One piece	Tot wt in lbs	Remarks
GIRDER SPAN 2. REQ'D.								
Main. girder 2 Req'd.								
Flg	Ls	8.	6×.6×1/2	2.0	2½	19.6	396.08	3169
"	"	4.	6×.6×1/2	25.	1.0	"	506.33	2025
Web	Pls	2.	56×3/8	20.	2½	71.4	1442.87	2886
"	"	1.	56×3/8	25	1.0	"	1844.50	1844
Stiff.	Ls	1.6	5×3½×½	4.	7½	13.6	62.90	1006
"	"	20	5×3½×3/8.	4	8½	10.4	48.97	979
	Fills	16	3½×½	3.	8¼	5.95	21.94	351
Cov.	Pls	4.	13½×½	7.	2	22.95	164.48	658
"	"	2.	13½×½	2.5	1.0	"	592.87	1186
	"	4.	13½×½	2.	1.0	"	65.02	260
	Ls	8.	6×.6×½	2.	1.0	19.6	55.53	444
	Pls	8.	9¾×½	2.	6.	16.58	41.45	332
	"	4.	12×3/8	2.	0½	15.3	31.69	127
	Ls	6.	4×3×3/8	1.	0.	8.5	8.50	51
	"	4.	4×3×3/8.	0.	9.	"	6.06	24
	Pls.	2	6×½	1.	0½	10.2	10.63	21
	"	4.	15×¾	1.	8.	38.3	63.83	255
								15,618×2=31,236
Floor Beam 6. Req'd.								
Flg	Ls	2.	4×3×3/8.	19.	0	8.5	161.50	323
"	"	2.	4×3×3/8	19.	11½	"	169.66	339
Web	Pls.	1.	25×5/16	16.	6½	26.56	439.35	439
	Ls	14.	3×3×5/16	2.	1½	6.1	12.96	181
	"	2.	5×3½×3/8	2.	0¾	10.4	21.45	43
	Fills	2.	5×3/8	1.	7¼	6.38	10.21	20
	Pls	2.	19¾×5/16	2.	9	20.98	57.70	115
	Pls	4	5½×5/16	1.	7¼	5.84	9.37	37
								1,497×6=8,982
Stringer								
I		6.	12×5@31.99#	13.	11¾	31.99	447.19	2,683
"		9.	12×5@31.99	12.	8½	"	406.54	3,659
Pls		24	8½×3/8	1.	0	10.84	10.84	260
								6,602×1=6,602

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CALCULATIONS FOR

Descr.	No.	Section in ins.	Length ft	Length ins.	Wt in lbs One ft	Wt in lbs One piece	Tot wt in lbs	Remarks
Lateral bracing								
Ls	10.	5. × 3. × 5/16.	21.	10½	8.2	179.38	1794	
"	10.	5. × 3. × 5/16.	10	8½	"	87.82	878	
"	10.	5. × 3. × 5/16	10	3½	"	84.38	844	
P/s	5.	19¼ × 5/16	2.	1½	20.45	43.46	217	
"	4.	17½ × 5/16	1	5½	18.59	27.11	108	
"	8.	17½ × 5/16	2	2¼	"	40.67	325	
							4.166 × 1 = 4.166	
Anchor bolts								
	16.	1" dia.	3.	6.	2.67	9.35	150	
P/s	16.	6. × 3/8	0	6.	7.65	3.83	61	
							211 × 1 = 211	

51,197 × 2 = 102,394#
102,394 ÷ 2240 = 45.712 Tons

Rivets

shop rivet	2,210	7/8" dia	grip abt. 1 3/8"	1,613
field "	830	" "	" " 1½	639
shop "	1,230	¾" "	" " 1	566
field "	590	" "	" " 1	271
				3,089

3,089 × 2 = 6,178#
6,178 ÷ 2,240 = 2.758 Tons

Total weight 2 Girder spans

Tons
45.712
2.758
48.470

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Final Design of Katsuura and Omatsugawa Bridges, Tokushimaken

The proposed new bridges span over Katsuuragawa and Omatsugawa between Nakata and Nishionga on main highway from Komatsushima to the city of Tokushima. The former crosses the river square, the latter skew as shown on general plan. These two bridges designed already by Bridge Engineer mr. T. Hanabusa and their details made. But after estimating cost it is found the cost does not come under the first estimation of cost, and we are obliged to change the design to reduce the total cost. We are proposing concrete girder spans of 60 and 40 which are most economic for these crossings.

The total length of Katsuuragawa Bridge

4 - 60 R	girder spans	= 240
6 - 40 R	" "	= 240
		480 R between ℄ Bearings on abutments

The total length of Omatsugawa Bridge (Skew).

5 - 54 R	girder spans	= 270
6 - 40 R	girder spans	= 240
		510 R

The spans will be all deck girders concrete or encased in concrete with reinforced concrete slab. The roadway 24 R wide between curb lines; handrail are made of concrete; Pavement will be the creosoted wood block with ½" asphaltic cement cushion.

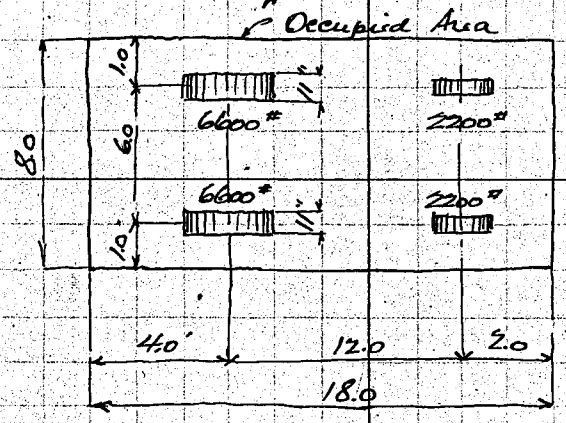
Assumed Loadings.

Uniform Live Load $q\ ^{kg/m^2} = \frac{100.000}{170+l}$ where l = the span length of bridge in meters

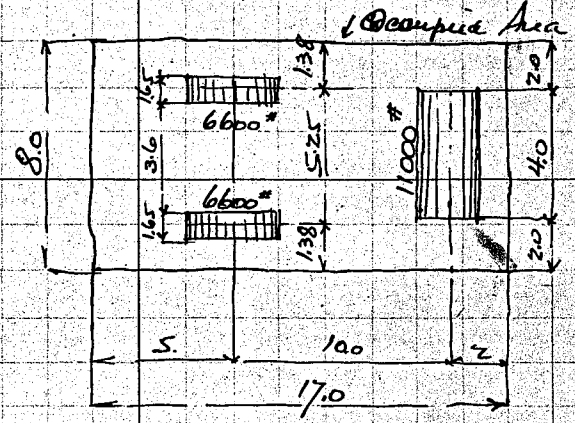
Max U. L. L. for spans under 30 meters = 102.5 #/0'

Load Roller and Motor Truck Loadings.

motor truck loading



Road Roller



⅓ impact assumed for motor trucks loading; no impact for uniform live load and road roller concentration: One motor trucks on one span in the direction of bridge, On sideway more than one trucks is assumed with occupied space of 8'. All unoccupied space to be filled with uniform loading. Only one road roller on span assumed.

Assumed Working Strengths.

Concrete 1:2:4 and reinforcing bars

Compressive Fibre Stress	pos. moment	650 #/0"
" " "	neg. moment	750 #/0"
Shear plain concrete		40
" reinforced Concrete		120
Punching shear		120
Bond stress		80
Tension in bar		16000 #/0"
Shear in bar		10000 #/0"

Structural Steel

Tension		16000 #/0"
Compression		16000 − 70 $\frac{l}{r}$
Fibre Stress		16000
Shear	shop rivet	10.000
	field rivet	8.000
Bearing	shop rivet	20.000
	field rivets	16.000
Bearing on pin		24.000
Expansion Roller		600 d

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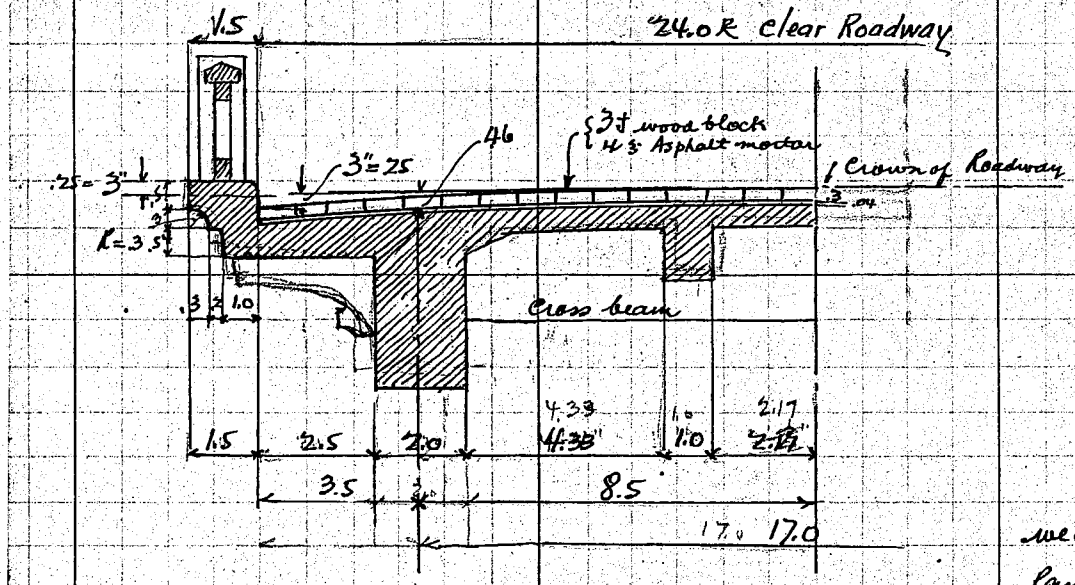
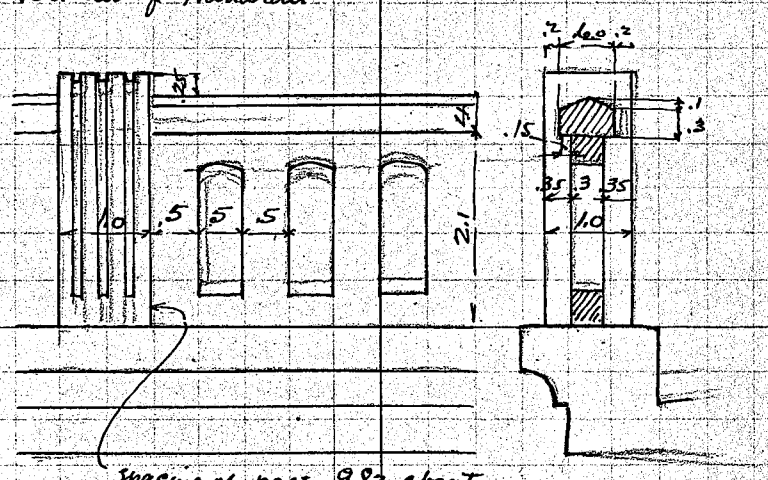
Design of 60' span for Katsuura gawa Bridge.

Cross section of bridge.



Details of Handrail



Scale = 1/20

weight of Coping

.5 × 1.5	= .75	× .75	= .56		
.3 × 1.3	= .39	× .65	= .254		
.54 × 1.0	= .54	× .5	= .127		
	1.68	@150#	= 252#	1.084	

Call this 260# per lin ft.

C of gravity = 1.084 / 1.68 = say .65 from curb line.

weight of Handrail

Panel Post	1.0 × 1.0 × 2.75	= 2.75	
Top rail	.6 × .37 × 8.83	= 1.96	
Ext. post	.6 × .6 × 2.1	= .76	
panel	.3 × 2.1 × 7.83	= 4.95	
		10.42	
Less panel cutting say		1.50	
		8.92	

weight = 8.92 × 150 = 134#

or 134 ÷ 9.83 = 136# per lin ft

assuming variation of details say 145# per lin ft.

Overhanging Arm of Floor Slab.

Moment at face of main girder

Dead Load 3" wood block = .18

½" cushion = .7

.25#

Concrete Slab. assumed 125#

150# × 2.5 = 375

	wt		arm		moment
Handrail	145	×	3.25	=	470
Coping	260	×	3.15	=	820
Roadway	375	×	1.25	=	470
	780				

Arm

1760 ÷ 780 = 2.26'

Dead Load M = 1760'# per ft.

Live Load

Motor truck loading Rear wheel with impact 8800#

Distribution of load on face of main beam = .6 × 2.0 + 1.0 = 2.2

Live Load Moment. 8800 × 2.0 = 17600'#

17600 ÷ 2.2 = 8000'# per ft strip.

1760

Total D and L.L. moment 9760'#

Depth of slab at face of main Girder. $d = \sqrt{\frac{9760}{107.5}} = \sqrt{91.0} = 9.5"$ for $f_s = 16000$#" $f_c = 650$#"

make depth 11 (12)" Effective depth say 10 (12)" Steel = 9760 × 12 / ⅞ × 10 × 16000 = .76 (.91) sq in per ft

Use ½"ϕ bars 3" (3½") centers = 0.78" (.66")

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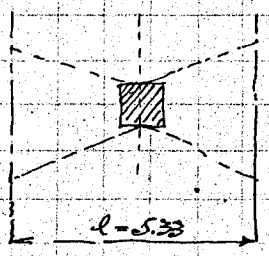
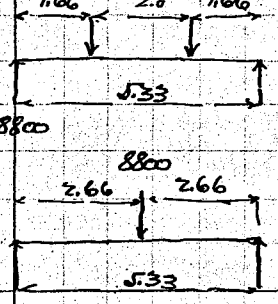
Dead Load moment at ℄ of Beam = 780 × (2.26 + 1.0) = 2620 ″ per one ft of strip

Roadway slab at center of bridge.
Span length 5.33′ clear span 4.33′
Dead Load- pavement 25″ moment = 1/10 × 113″ × 5.33² = 320 ″
slab assumed 88
113″

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

Distribution of wheel concentration
Single load at center of span 5.33 × 0.6 + 1.0 = 3.2 + 1.0 = 4.2





4400

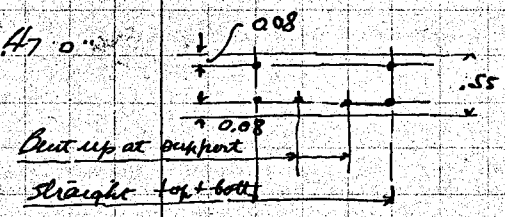
LL moment 8800 × 1.66 = 14600″ for 2 wheels
do 4400 × 2.66 = 11700 for 1 wheel

For one ft strip 14600 ÷ 4.2 = 3480″
For continuity of slab 3480 × 0.8 = 2780″
Dead Load moment 320
3100″

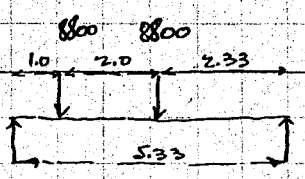
Design slab for pos and neg moment of 3100″

Effective depth required for 650 ″/□″ concrete stress and 16000 ″/□″ steel stress
= √(3100/107) = 5.4″ Use 6½″ slab effective d = 5½″

Steel Area = 3100 × 12 / (⅞ × 5.5 × 16000) = .483 Use ½″ ϕ bars 5″ centers = .47 □″
or ½″ ϕ bars 4″ centers



End Shear



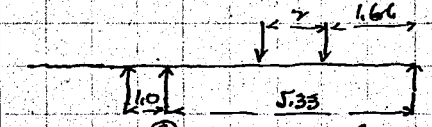
Shear = 8800 × 6.66/5.33 = 11000 ″

For one ft strip = 11000/4.2 = 2620
Dead Load shear 113 × 2.66 = 300
2920

Unit shear = 2920 / (⅞ × 5½ × 12) = 50.5 ″/□″

Plain concrete good for 40.0
10.5 ″/□″ to be taken care of by stirrup or bent up bars.

Roadway slab next to main beam
Max positive moment free ends assumed.
span length say 5.33 Dead load moment = 1/8 × 113 × 5.33² = 402
Dead load moment overhanging arm = 780 × (2.66 + 0.5) = −2460
say +400
−2060″



Max positive live Load moment 2 wheels as shown M = 8800 × 1.66 = 14600″
For one ft strip 14600 ÷ 4.2 = 3480
−2060
Combined moment = +1420″
Dead Load Max negative moment ~~2480~~ ″ say −2460 at support

Live Load moment
Distribution of wheel load at @ 3.5 × 0.6 + 1.0 = 3.1
moment = 8800 × 2.5 = 22000″ 22000 ÷ 3.1 = 7100″ per 1′ strip
Dead load moment 2460
9560″

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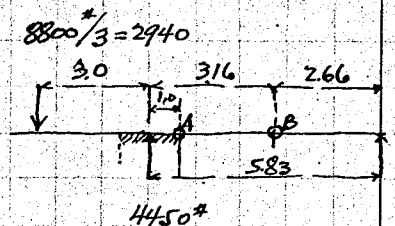


Distribution of wheel load assumed 3.0

8800 ÷ 3.0 = 2940 #

Reaction = 2940 × 8.83/5.83 = 4450 #

Moment at A = 2940 × 4.0 = 11780

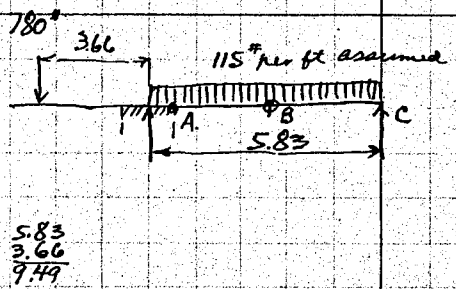
less 4450 × 1.0 = -4450

7330 1# negative

Moment at B = 2940 × 6.16 = 18100

less 4450 × 3.16 = -14100

4000 1# negative.



5.83
3.66
9.49

Dead Load Reaction 780 × 9.49/5.83 = 1270 #

115 × 5.83/2 = 335

1605 #

Moment at A 780 × 4.66 = 3640

115 × 0.5 = 57

3697 1#

Less 1605 × 1.0 = 1605

2092 1# neg

Moment at B 780 × 6.16 = 4800

115 × 3.16²/2 = 575

5375

Less 1605 × 3.16 = 5070

305 1# neg.

Summary of moments

	at A.	at B	
Live Load neg.	7330	4000	Theoretically these stresses are not correct however design slab for these moment as safe side.
Dead Load neg.	2092	305	
	-9422 1#	-4305 1#	

Positive moment at B = +1420 See page 3

negative moment at support C say same as center panel m = 3100 1#

Depth of slab required at A $\sqrt{\frac{9422}{107.5}}$ = 8.9" use 12" slab reinf. same as for overhang.

B $\sqrt{\frac{4305}{107.5}}$ = 6.3" use slab 6½" steel on compressive side to reduce unit stress of concrete

Steel required at B (center of slab). $\frac{4305 \times 12}{7/8 \times 5.5 \times 16000}$ = .67 on top. ½"ϕ bars 3.4" spacing

$\frac{1420 \times 12}{7/8 \times 5.5 \times 16000}$ = .22 on bottom ½"ϕ bars 11.0' spacing.

Concrete Stringer. span length 14.5

Dead load Reaction at C 780 × 3.66/5.83 = -490

115 × 5.83/2 = +335

-155

113 × 5.33/2 = +301

+246 #

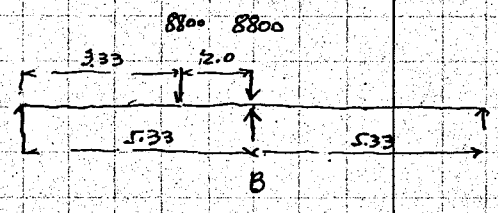
Dead Load Beam assumed 187

433 # per lin ft.

moment = 1/10 × 433 × 14.5² = 9100 1# for both pos and neg.

Live Load

motor trucks with ⅓ impact Rear wheel 8800# Front wheel 2930#



Reaction at B = 8800 × 3.33/5.33 = 5500

8800

14300 #

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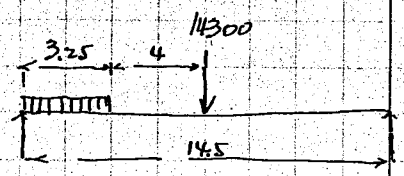
Uniform live load 107.5# × 5.33 = say 550#



Reaction due to uniform load = $\frac{550 \times 3.25^2}{2 \times 14.5}$ = 200#

Reaction due to conc. = 7150#

moment = 7150 × 7.25 = 51900

200 × 7.25 = 1450

53350 1#

For continuity of beam 53350 × 0.8 = 42700 1#

Dead Load moment 9100

51800 1#

Depth rqd for req moment = $\frac{\sqrt{51800}}{107.5}$ = 22"

make depth 2.05' or 1.5 under slab.

Steel = $\frac{51800 \times 12}{\frac{7}{8} \cdot 22 \cdot 16000}$ = 2.02"

Use 4 - $\frac{7}{8}$"ϕ bars = 2.40" at bottom

2 - 3/4" bars at top.

End shear Dead Load = 433 × $\frac{14.5}{2}$ = 3150#

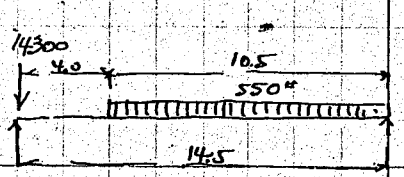
Reaction = $\frac{550 \times 10.5^2}{2 \times 14.5}$ = 2090#

14300

16390

3150

19540#



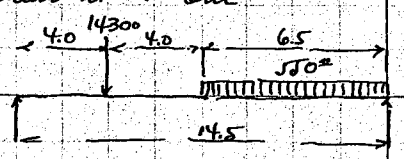
Unit shear = $\frac{19540}{\frac{7}{8} \times 22 \times 12}$ = 84.5#/o.

Plain concrete good for - 40.

44.5# for stirrups.

3/8" Stirrup spacing - = $\frac{2200}{12 \times 44.5}$ = 4.12"

Shear at 4' out



Reaction = $\frac{550 \times 6.5^2}{2 \times 14.5}$ = 800

14300 × $\frac{10.5}{14.5}$ = 10350

11150

Dead Load 3150 - 1730 = 1420

12570

Unit shear = $\frac{12570}{\frac{7}{8} \times 22 \times 12}$ = 54

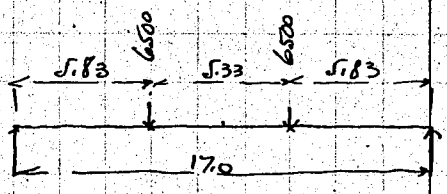
less 40

14#/o.

Stirrup spacing = $\frac{2200}{12 \times 14}$ = 13".

Intermediate Floor Beam span length 17.0 spacing 14.5'

Dead Load



Concentration say 450 × 14.5 = 6500#

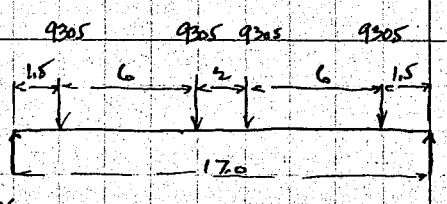
moment = 6500 × 5.83 × 0.8 = 30300 1#

Dead Load beam assumed 2.5 × 1.25 × 150 = 468# per ft

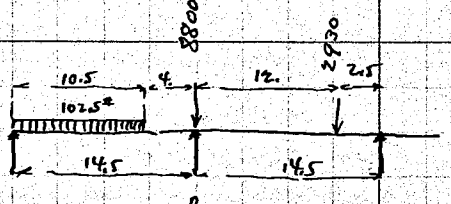
moment = $\frac{1}{10}$ × 468 × 17² = 13500 1#

Total Dead Load Moment = 30300 + 13500 = 43800 1#

Live Load



18610



R = $\frac{2930 \times 2.5}{14.5}$ = 505

8800

9305#

R = $\frac{102.5 \times 10.5^2}{2 \times 14.5}$ = 390# per lin ft

moment = 18610 × 7.5 = 139600

9305 × 6.0 = - 55908

83700 × 0.8 = 67000 1#

$\frac{1}{10}$ × 390 × 17² 11300

78300 1#

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Intermediate Floor Beam

Dead Load Moment = 43800
Live Load Moment = 78300
122,100 #

depth at support = $\sqrt{\frac{122,100}{107.5 \times 1.25}}$ = ~~33.6~~ 30.2"

Make depth 25' under slab 1.25' wide. Effective d say 33" ✓

Steel = $\frac{122.100 \times 12}{\frac{7}{8} \times 33 \times 16000}$ = 3.16 use 6-$\frac{7}{8}$"ϕ = 3.60"

End Shear 18610

$390 \times \frac{17}{2}$ = 3300

21910 #

Dead Load 6500

" " $468 \times \frac{17}{2}$ = 4000

32410 #

Unit shear = $\frac{32410}{\frac{7}{8} \times (15) \times 33}$ = 75 # 103 / 40 / 63

$\frac{40}{35}$ #

$\frac{3}{8}$"ϕ stirrups s/p = $\frac{2200}{15 \times 35}$ = 4.7" at end.

End Floor Beam span length 17.0

Dead Load Concentration $450 \times 8.25 = 3720$ #

Moment = $3720 \times 5.83 \times 0.8 = 17350$

Dead Load beam assumed $2.5 \times 150 = 375$

Moment = $\frac{1}{10} \times 375 \times 17^2 = 10800$ #

Total Dead Load Moment = 17350 + 10800 = 28150 #

Live Load wheel concentration only moment 67000

95150 #

depth required = $\sqrt{\frac{95150}{107.5}} = 29.8$" make beam 2.5' under slab 1.0' wide.

Steel Area = $\frac{95150 \times 12}{\frac{7}{8} \times 33 \times 16000}$ = 2.47 □" use 4-$\frac{7}{8}$"ϕ = 2.40"

Use same stirrups as for intermediate Floor Beams.

Main Girder span length 58.0

Concrete in flooring ½ width of Roadway

Concrete in coping		1.68 □
Roadway slab	12 × .55 =	6.60
Fillet	10 × .45 =	.45
Beam above fillet	2.0 × .45 =	.90
Stringer	1.0 × 1.5 =	1.50
		11.13
Floor Beam	$2.5 \times 1.25 \times \frac{7.5}{14.5}$ =	1.62
		12.75 Call this 13.0 cubic ft. per ft

weight of Concrete	13 @150 =	1950 #
Roadway Pavement &c	12 × 25 =	306
		2256 #
weight of Handrail		145
		2395 #
Dead Load Beam assumed below fillet	3.2 = 6@150 =	900
		3295 # per ft

Dead Load Bending moment = $\frac{1}{8} \times 3295 \times 58^2$ = 1,390,000 #

Live Load.

Motor trucks loading 3 trucks side by side. 3 wheels on one girder.

Rear wheel	8800 × 3 =	26400 #
Front wheel	2930 × 3 =	8790 #
Uniform load	102.5 × 12 =	1230 # per lin. ft.

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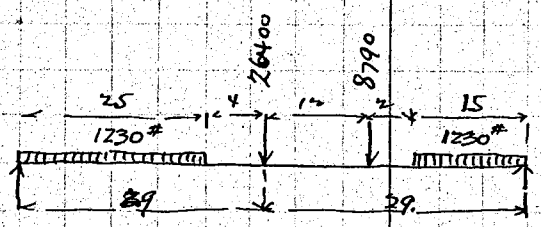
Live Load Moment



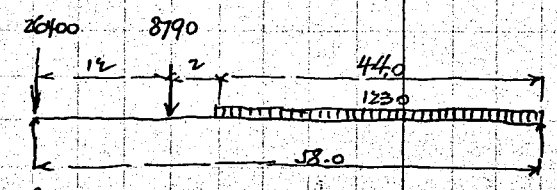
R_1 26400 × 29/58 = 13200
8790 × 17/58 = 2580
15780

R_1 1230 × 15 = 18450 · 7.5/58 = 2380

R_1 1230 × 25 = 30750 · 45.5/58 = 21100
42260 #

Bending Moment at center 42260 · 29 = 1225000
Less 3075 × 16.5 = 507000
718,000 1#

End shear



R_1 = 53860

R_1 1230 × 44 = 54.000 · 22/58 = 20500
8790 × 46/58 = 6960
26400
53860

Dead load shear 3295 × 58/2 = 95500
149360 #

Max shear in concrete = 149360 / (7/8 · 45" · 24) = 158 #/□"

Excess of shearing to be taken care of reinf or structural Steel.

Total Bending moment
Dead Load 1390.000
Live Load 718.000
2108.000 1#

Assumed depth of beam is too shallow. Let us make depth 0.5 deeper.
Extra load 150# m = 1/8 × 150 × 58² = 63000

Dead Load moment 1453.000
Live Load moment 718.000
2171,000 1#

depth of girder 48½" using 48 × ⅜" = 18.0□"
Effective depth = 4.04 − 0.19 = 3.85 Stress = 2.171.000 ÷ 3.85 = 564.000 #
Section required for tension = 564.000 ÷ 16000 = 35.20"
Try the following section.

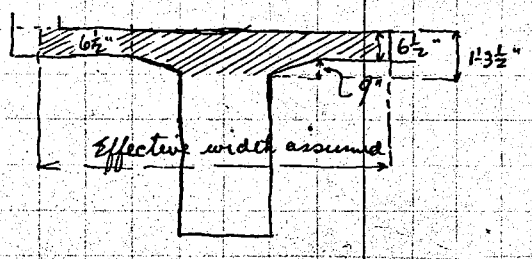
2Ls 6×6×⅝	=	14.22 gross	11.72 □" net
1 cov Pl. 18×½	=	9.00	8.00
do	=	9.00	8.00
do	=	9.00	8.00
		41.22 □"	35.72 □"

For top flange use 2Ls 6×6×⅝"

Length of cov pl.
58 × 0.473 + 2 = 29.5'
58 × 0.661 + 2 = 40.4
58 × 0.820 + 2 = 49.5

Resisting moment 2Ls 6×6×⅝ 11.72 □" × 16000 × 3.85 = 722000 1#
18×½ Pl. 8.00 × 16000 × 3.85 = 493.000 1#

Resisting moment of reinforced concrete slab = 2171.000
− 722.000
1,449,000 1#



Average depth of flange.

6½"	× 3.5	=	22.8
11"	× 2.0	=	22.0
15½	× 2.0	=	31.0
	7.5		75.8 ÷ 7.5 = say 10"

Assume flange depth 10"

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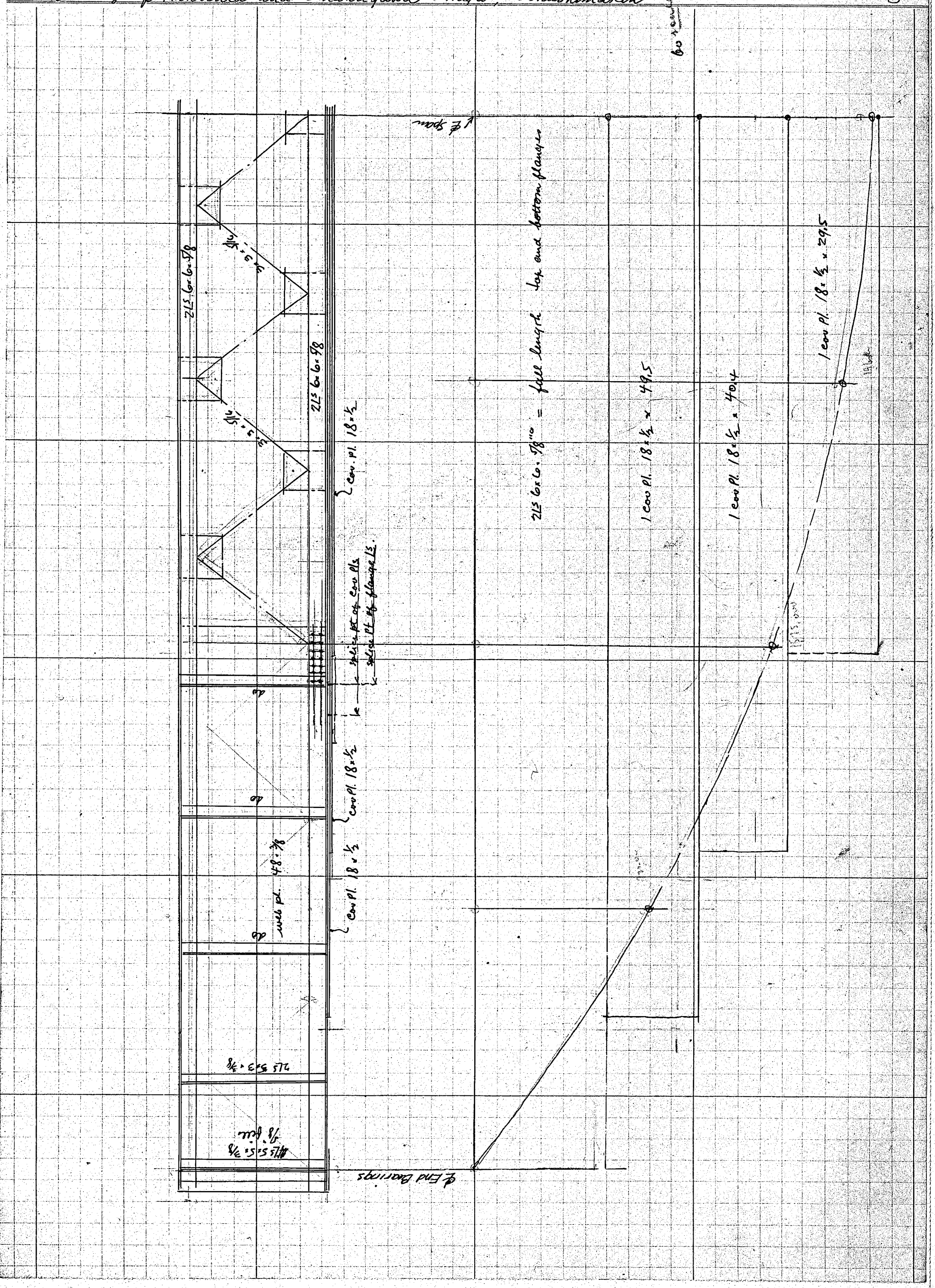

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Depth of beam 51" to top of concrete

$t/d = 10/51 = .196$

Values of $\frac{M}{bd^2} = \frac{1449.000 \times 12}{75 \times 51^2 \times 12} = 74.2$

Stress in concrete 575 #/□" ok from curve.

End shear say 150.000 $sR =$ 150" Use 48 × 3/8 = 18.00"

Rivet spacing at end

7/8" Rivet on 3/8" pl. 6562 $sp = \frac{6562 \times 42}{150.000} = 185"$ Use 2" spacing at end

Concrete will take care of 40# shear.

Approximate weight of girder

Flange Ls	4 Ls	6×6×5/8	@ 24.2 #	×	60.0	=	5800
cov Pl.	1 Pl.	18×1/2	@ 30.60	×	49.5	=	1515
"	1 Pl.	18×1/2	@ 30.60	×	40.4	=	1240
"	1 Pl	18×1/2	@ 30.60	×	29.5	=	905
web Pls.	2 webs	48×3/8	@ 61.20	×	16.0	=	1960
End stiffs -	8 Ls	5×5×3/8	@ 12.30		4.0	=	394
fills	4 Pls	10×5/8	@ 21.25	×	3.0	=	255
stiffs	16 Ls	5×3×3/8	@ 9.8	×	4.0	=	628
connection Pls	11 Pls.		@ 40#			=	440
web.	24 Ls	3×3×5/16	@ 6.1.	×	3.5	=	512
		Splices + variation say					500
							14149 # 6.3 tons

call this 6.5 tons per girder

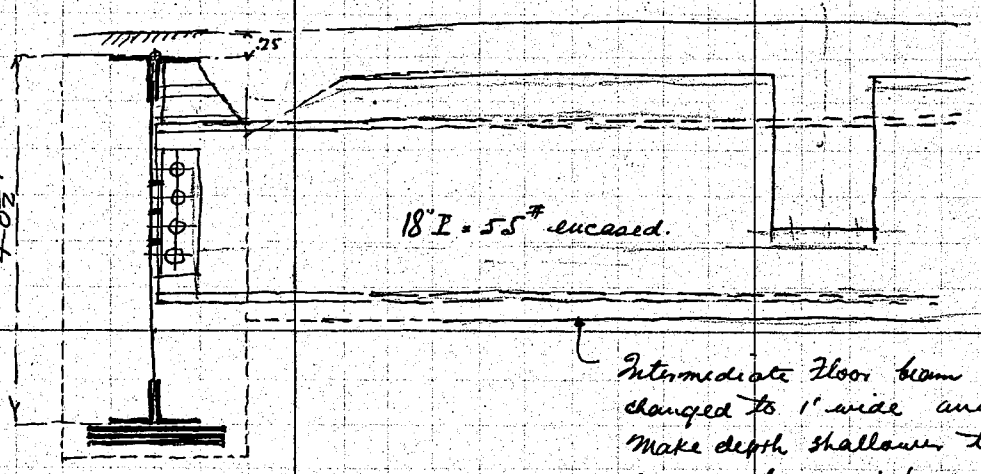
or 13 tons per span

Cross Struts.

Cross Struts to be used at intervals of floor beams and encased into concrete

max. bending moment of Intermediate Floor Beam = 122.100 #

Section modulus required = 92.0



Intermediate Floor beam changed to 1' wide and make depth shallower to cover I beam into concrete

All forms to be carried by this I beam

weight of cross strut and beam

1 I 18" × 55#	× 17.0	=	935 #
4 Ls 3½×3½×3/8	@ 8.5# × 12.5	=	42
Rivet heads			8
			985 call this 1000 # each

5 @ 1000 = 5000 #

Misc details - 200

5200 # 2.3 tons per span

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40' Concrete girder span

Details of Floor Slab same as for 60' span

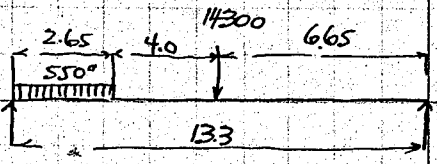
Concrete Stringer Span length 13.3

Dead Load 433# per lin ft see p4. moment = $\frac{1}{10} \times 433 \times 13.3^2 = 7670$ 1# both neg and pos.

Live Load Concentration 14300 # p4.

Uniform Live load 107.5×5.33 = say 550#

Reaction due to uniform load = $\frac{550 \times 2.65^2}{2 \times 13.3} = 145$#



Reaction due to Concentration = 7150#

Moment 7150 × 6.65 = 47500

145 × 6.65 = 965

48465

For continuity of beam 48465 × 0.8 = 38800 1#

Dead Load moment 7670

46470 1#

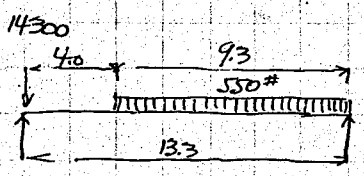
Depth required for neg. moment = $\sqrt{\frac{46470}{107.5}}$ = 20.8"

Make depth 1.75 or 1.2 under slab Steel Area = $\frac{46470 \times 12}{\frac{7}{8} \times \frac{20}{19} \times 16000} = 2.10$"

use 4-$\frac{7}{8}$"ϕ bars = 2.40"

2-$\frac{3}{4}$"ϕ bars at top

End shear



Dead Load = $433 \times \frac{13.3}{2}$ = 2900#

Uniform load $\frac{550 \times 9.3^2}{2 \times 13.3}$ = 1790

14300

16090

2900

18990

Unit shear = $\frac{18990}{\frac{7}{8} \times 19 \times 12}$ = 95# 10.

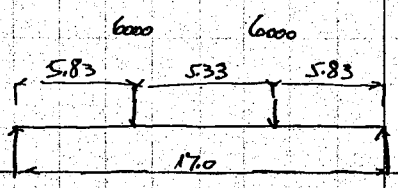
40

55# by stirrups.

$\frac{3}{8}$" Stirrups spacing = $\frac{2200}{12 \times 55}$ = 33"

Intermediate Floor Beam span 17.0 spacing 13.3'

Dead Load



Concentration say 450 × 13.3 = 6000#

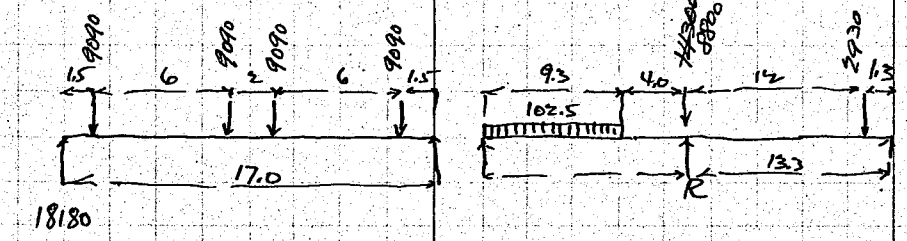
Moment = 6000 × 5.83 × 0.8 = 28000 1#

Dead Load beam assumed 2.5 × 1.85 × 150 = 468# per ft

Moment = $\frac{1}{10} \times 468 \times 17^2$ = 13500 1#

Total Dead Load Moment = 28000 + 13500 = 41500

Live Load



R = $2930 \times \frac{1.3}{13.3}$ = 290

8800

9090#

R = $\frac{102.5 \times 9.3^2}{2 \times 13.3}$ = 330# per ft

Moment 18180 × 7.5 = 136300

9090 × 6.0 = 54600

81700 × 0.8 = 65400

Unif. load $\frac{1}{10} \times 330 \times 17^2$ = 9500

74900 1#

Summary of Moment

Dead Load Moment	41500
Live Load Moment	74900
	116400 1#

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Depth at support $\sqrt{\frac{1164000}{107.5 \times 12.5}} = \sqrt{865} = 29.4"$

Make beam 1.25 × 2.25 under slab. Steel Area $= \frac{1164400 \times 12}{\frac{7}{8} \times 30 \times 16000} = 3.320"$

Use $6 - \frac{7}{8}"\phi = 3.60"$

Stirrups for shear Use same stirrups as shown on pp 6.

End Floor Beam Span length 17.0

Dead Load Concentration $= 450 \times \left(\frac{13.3}{2} + 0.5\right) = 3220^{\#}$ (7.15)

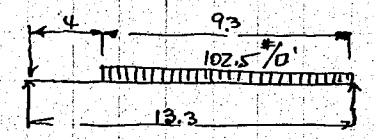
Moment $= 3220 \times 5.83 \times 0.8 = 15040^{\text{lft}}$

Dead Load beam assumed $2.25 \times 150 = 338$

Moment $= \frac{1}{10} \times 338 \times 17^2 = 9750$

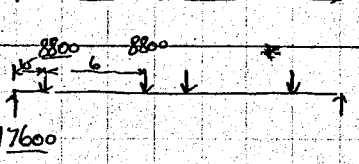
Total Dead Load moment $15040 + 9750 = 24790^{\#}$

Live Load



Uniform load $= \frac{102.5 \times 9.3^2}{2 \times 13.3} = 330^{\#}$

Moment $= \frac{1}{10} \times 330 \times 17^2 = 9500$



Concentration $17600 \times 7.5 = 132,000$

$8800 \times 6.0 = 52800$

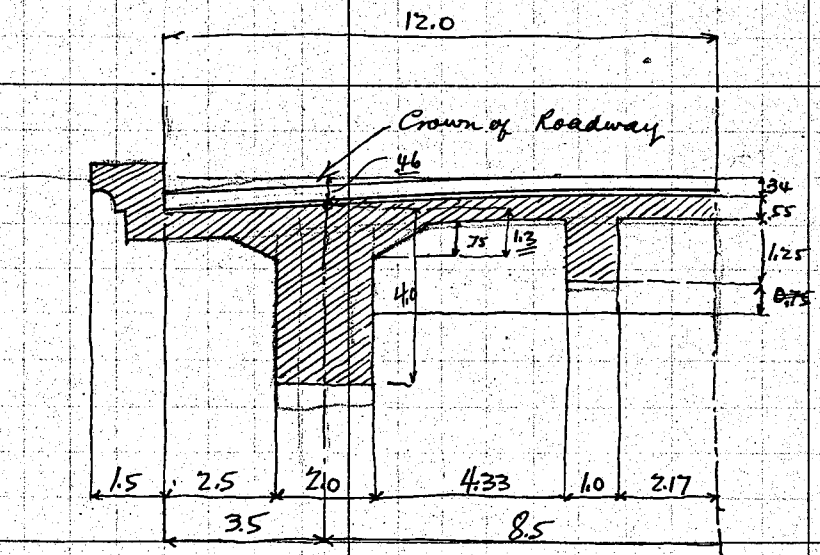
79200

$88700^{\#}$

Make beam = 1.0 × 2.25 under slab.

Steel Area $= \frac{88700 \times 12}{\frac{7}{8} \times 30 \times 16000} = 2.54"$ Use $4 - \frac{7}{8}"\phi$ bars $= 2.40"$

Main Girder.



Concrete in Floor. ½ Roadway		
Concrete in coping		1.68
Roadway slab	12 × .55 =	6.60
Fillet	.75 × 1.0	.75
Beam for fillet	2. × .75	1.50
Stringer		1.25
Floor Beam	$2.25 \times 1.25 \times \frac{7.5}{13.3}$ =	1.59
Beam assumed	3 × 2 =	6.00
		19.37
weight of concrete	19.37 @ 150 =	2900
Pavement ↔	12 × 25 =	300
Handrail		145
		$3345^{\#}$

Dead Load Moment 3 span continuous beams

End span	positive moment	$= 0.080 \times 3345 \times 40^2 =$	428000
2nd support	negative moment	$= 0.100 \times 3345 \times 40^2 =$	535000
Int. span	positive moment	$= 0.025 \times 3345 \times 40^2 =$	134.000

Live Load

Motor trucks loading, 3 trucks side by side. 3 wheels on one girder

Rear wheel	3@8800 =	26400#
Front wheel	3@2930 =	8790

Uniform load $102.5 \times 12 = 1230^{\#}$ per lin ft.

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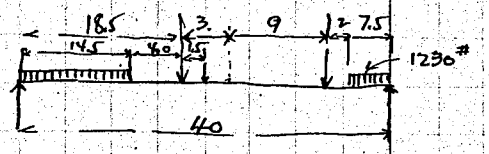
By Bending moment as simple span



Absolute max of loading

Center of gravity of loading $8790 \times 12 = 105500$

26400×0

35190

$105.500 \div 35190 = 3.0'$

$$\text{Moment} = 35190 \times \frac{18.5'}{40} = 300.000\ \#$$

$$\text{Reaction} = \frac{1230 \times 7.5^{2'}}{2 \times 40} = 865$$

Moment $15465 \times 18.5 = 286000$

less. $1230 \times 14.5 \times 11.5 = 205500$

$80500\ \#$

300.000

Total moment - 380.500

$$1230 \times \frac{14.5 \times 3275}{40} = 14600$$

15465

Equivalent uniform load for simple span $\frac{8}{8} wl^2 = m$ $w = \frac{8^2 m}{l^2}$

$$w = \frac{380500 \times 8}{40^2} = 1900\ \# \text{ per lin ft of span}$$

Let us take this uniform load for continuous beam.

Live load stress

End span	Positive moment	$0.100 \times 1900 \times 40^2 =$	$304.000\ \#$
2nd support	Negative moment	$0.117 \times 1900 \times 40^2 =$	$356000\ \#$
Int. span	Positive moment	$0.075 \times 1900 \times 40^2 =$	$267.000\ \#$

Summary of moments

	End span Pos. m.	2nd support neg. m	Center span Pos. m
Dead Load	428000	535000	134000
Live Load	304000	356000	267000
Total	732000	891000	401000

Depth of Beam at center assumed $4.0 = 48"$ Depth to ℄ steel say 43"

1% steel $jd =$ say .91

$k/d = 9/43 = .21$ about Steel Area $= \frac{732000 \times 12}{.91 \times 43 \times 16000} = 14.1$ a"

use $18 - 1"\phi$ bars $= 14.10"$

Approximate concrete stress in flange. Average depth of 9". $k/d = .21$

Effective width say 7.5

$$R = \frac{M}{bd^2} = \frac{732000}{7.5 \times 43^2} = 52.9$$ concrete stress = 430#/□" ok.

Percentage of steel $= \frac{14.1}{7.5 \times 12 \times 43} = .36\ \%$

Effective width for 650# stress $R = 89.5$ $b = \frac{732000}{89.5 \times 43^2} = 4.42'$ ok

Effective depth at support $d = \sqrt{\frac{891000}{107.5 \times 2}} = 64.5"$ at center line of col.

Steel Area $= \frac{891000 \times 12}{\frac{7}{8} \times 64.5 \times 16000} = 11.9$ use $15 - 1"\phi$ bars

Steel Area required for center span $= \frac{401000 \times 12}{.91 \times 43 \times 16000} = 7.7$ o" use $10 - 1"\phi$ bars $= 7.85$ o"

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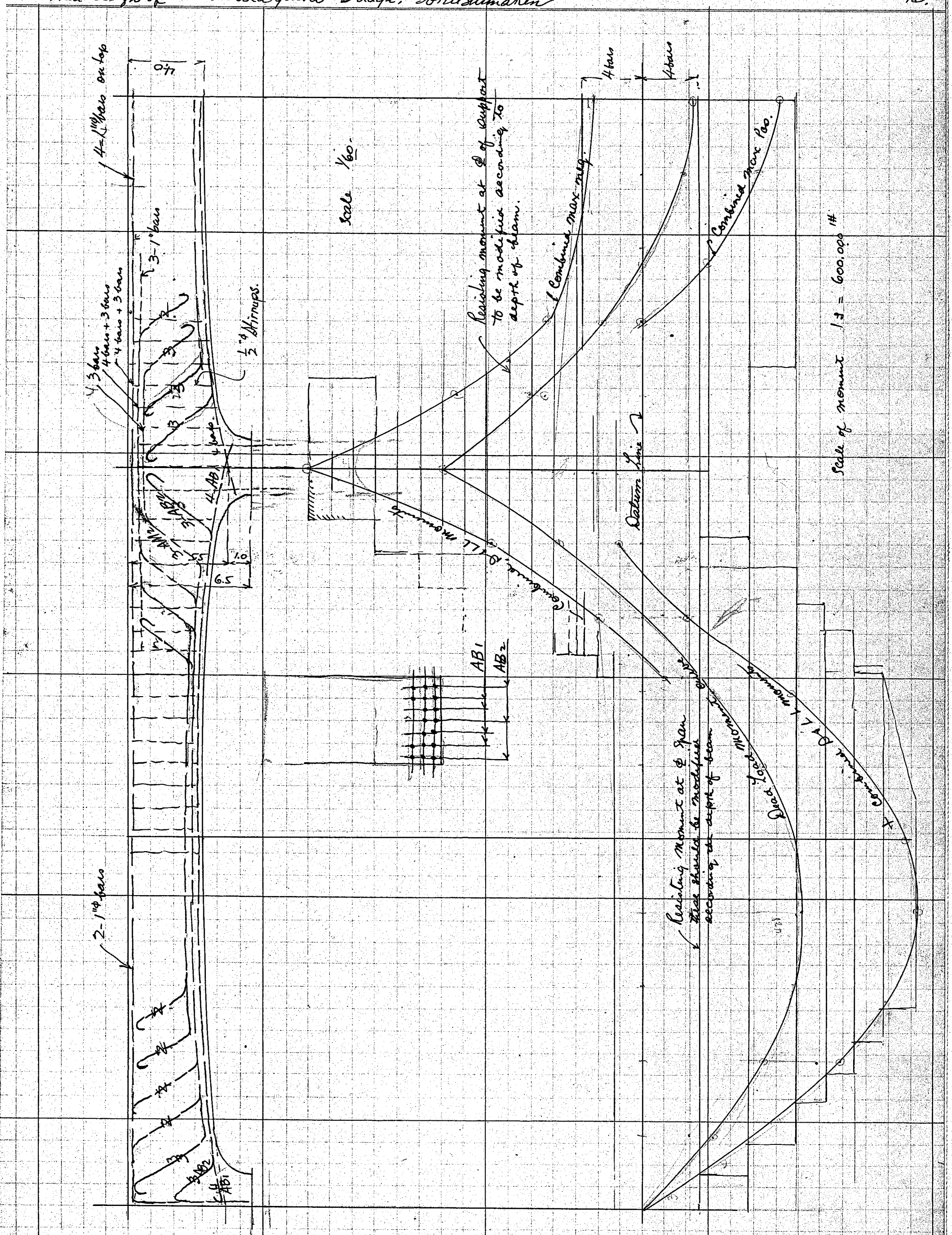

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End shear

Dead Load	1st span	End support	0.4 × 3345 × 40	=	53500 #
		near 2nd support	0.6 × 3345 × 40	=	80400
	Center span	near 2nd support	0.5 × 3345 × 40	=	67.000

max Reaction at 2nd support = 147400 #

Live Load	1st span	End support	0.45 × 1900 × 40	=	34200 #
			0.616 × 1900 × 40	=	46800
			0.583 × 1900 × 40	=	44200

max Reaction at 2nd support = 91000 #

Summary of End shear

	1st Span 1st Support	1st Span 2nd support	2nd Span 2nd support	Reaction 2nd Support
Dead Load	53500	80400	67000	147400
Live Load	34200	46800	44200	91000
Total	87700	127200	111200	238.400

Max End shear = 127200 # Unit shear = $\frac{127200}{7 \times 65 \times 24}$ = 93 #/□"

excess of shear over 40# to be carried by bent up bars or stirrups.

Design of Substructure.

60' span ½ Roadway

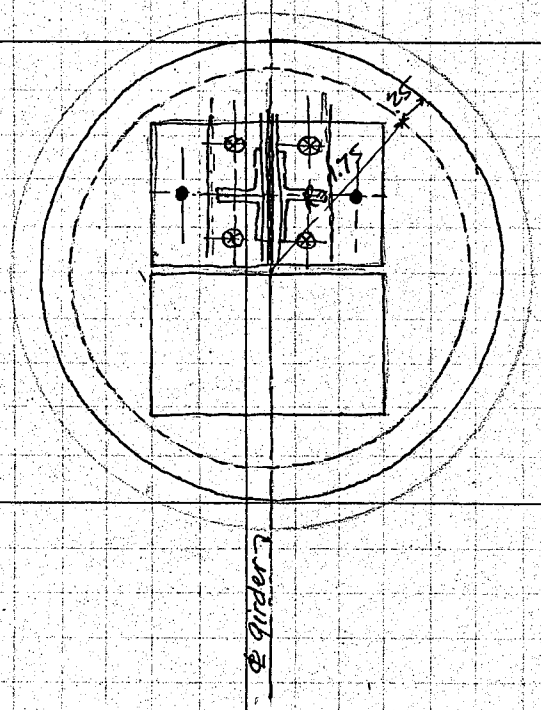
Dead Load	3295	see P6.	Dead load on pier =	3445 × 60 =	207.000 #
	150	" P7	Live Load on	1900 × 60 =	114.000
	3445 #		Total		321.000 #

For end span 160500 #

40' span ½ Roadway Total load = 238.400 for intermediate support
175400 " end support

Pier for 40' and 60' spans For 60' span 160500
" 40' span 87700
248200 #

Typical Design of Pier for 60' spans.



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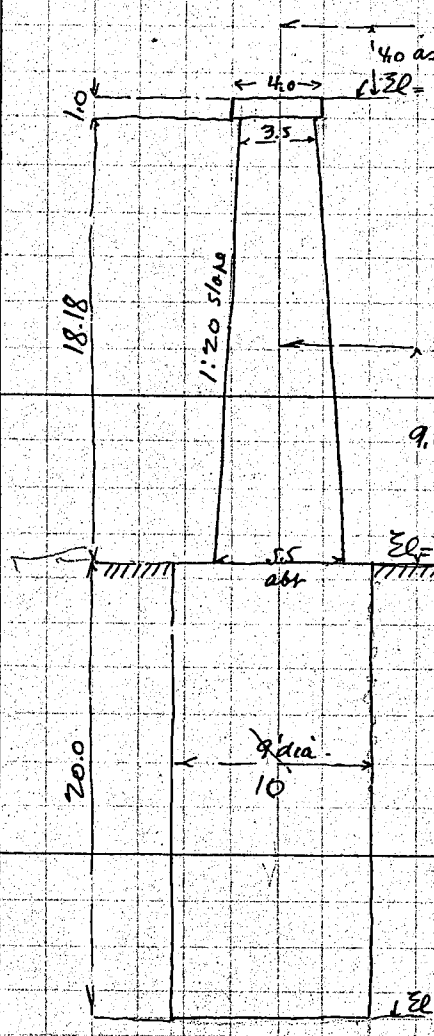
volume of shaft



4' dia 12.5 coping 12.5 cubic ft

shaft 3.5'φ 9.6

5.5 φ 23.7

33.3 ÷ 2 = 16.65

volume = 16.65 × 18.18 = 302.0

Total 314.5 @ 150 = 47000#

Base 9'φ = 63.6

volume = 63.6 × 20 = 1272 cft wt = 191000

238.000#

(Revised)

substructure 238.000

superstructure 321.000

Total load 559.000#

Friction of well 200 × 28.2 × 20 = 113.000

446.000 net

Unit bearing Pressure = $\frac{446.000}{63.6}$ = 7000# or 3.13

Try 10' Base

Base 10' 78.5 × 20 = 157.0 @ 150 = 236000

shaft 47000

substructure 283000

superstructure 321000

604000#

Friction of well 200 × 31.4 × 20 = less 126000

478000#

Unit bearing Pressure = $\frac{478000}{78.5}$ = 6100 #/□' = 2.72 tons/□' ok

Seismic stability Acceleration of falling body g = 32.2' per sec²

Assume acc. of earth quake 1250 mm/sec² = 4.1' per sec²

Moment due to acc = $M = \frac{W}{g} ay$. where W = weight

g = acceleration of gravity

a = " " Earthquake

y = lever arm

Deck $\frac{321000}{32.2} \times 4.1 \times 23.18$ = 950.000

Shaft $\frac{47000}{32.2} \times 4.1 \times 9.0$ = 54.000

1.004.000

Moment of Inertia of shaft at base $I = 0.049 \times 66^4$ = 932.000 (inch)⁴

Fibre stress = $\frac{1004.000 \times 12 \times 33}{932.000}$ = 427 #/□" comp or tension.

Direct load 604000# area of shaft = 3421.

unit stress = $\frac{604000}{3421}$ = 176 #/□"

Summary

Comp.	Tension
427	427
10.8	10.8
176	176
603 #/□"	254 #/□"
535	319

Let us figure the stresses in case of dead load only

Deck $\frac{207.000}{32.2} \times 4.1 \times 23.18$ = 611.000.

$\frac{47000}{32.2} \times 4.1 \times 9.0$ = 54.000

665.000#

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Fibre stress = $\frac{665000 \times 12 \times 33}{932000} = 283$ #/o.

Direct stress = 207.000 + 47000 = 254000

Unit stress = $\frac{254000}{3421} = 74$ #/o.

Combined Direct and Fibre stress

	Comp.	Tension
	283	283
	74	− 74
	357 #	209 #

Reinforcement in tension side required. For simplicity assume the round sheaft of a square beam of 40" × 62"

Reinforcing bars 10 − 3/4"ϕ bars = 4.40" Percentage of steel = $\frac{4.4}{40 \times 62} = 0.0018$ Tension side

0.0036 for both side

Value of eccentricity 665.000 ÷ 254000 = 2.62' or 31.4"

e/h = 31.4 : 62 = 0.505 value of k = .413 R = .103

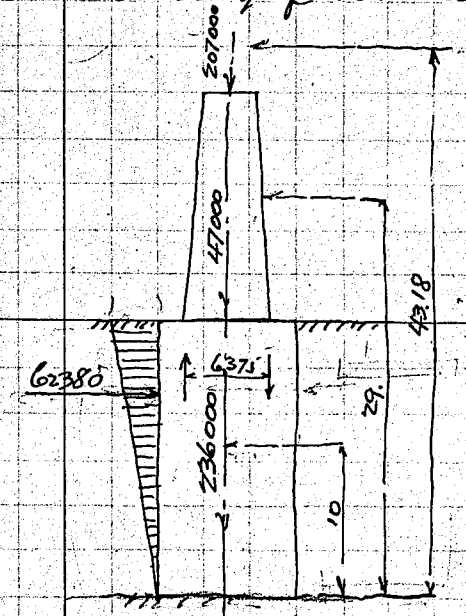
Concrete stress = $\frac{665000 \times 12}{.103 \times 40 \times 62^2} = \frac{7980}{505}$ #/o.

Steel stress = $15 \times 505 \left(\frac{59}{62 \times .413} - 1.\right) = 9900$ #/o. ok

Use 8 − 3/4"ϕ bars in quadrant which be ok

or total in shaft 32 − 3/4"ϕ bars

Stability of Pier due to Seismic motion approximate only



Direct load

207000
47000
236000
Total 490.000 #

$\frac{207000}{32.2} \times 4.1 = 26400 \times 43.18 = 1.140.000$

$\frac{47000}{32.2} \times 4.1 = 5980 \times 29.0 = 173600$

$\frac{236000}{32.2} \times 4.1 = \frac{30000}{62380\#} \times 10.0 = \frac{300.000}{1,613,600}$

less 62380 × 13.3 = 815.000

798600 1#

Frictional Resistance in circumference of well = 31.4 × 300 × 20 = 189000

300 #/o' friction assumed

Resisting Moment = $\frac{189000}{2} \times 6.375$ = say 600.000 1#

Assuming 200 #/o' friction − RM = 400.000 1#

Resultant moment = 798600 − 400.000 = 398600 1#

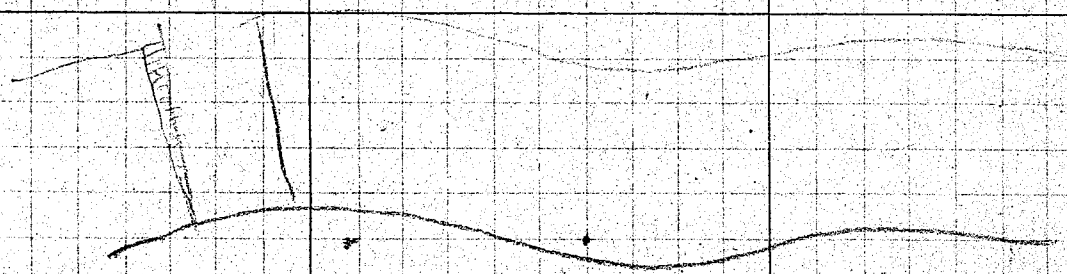
Unit bearing pressure due to bending = $\frac{398600 \times 5}{490} = 4070$ #/o' or 1.82 tons

Direct Bearing 490.000 ÷ 78.5 = 6250 #/o' or 2.79

4.61 tons ok

Pier for 40' span max load 238400 #

else same base as for 60' span making the depth of well 15' instead of 20'



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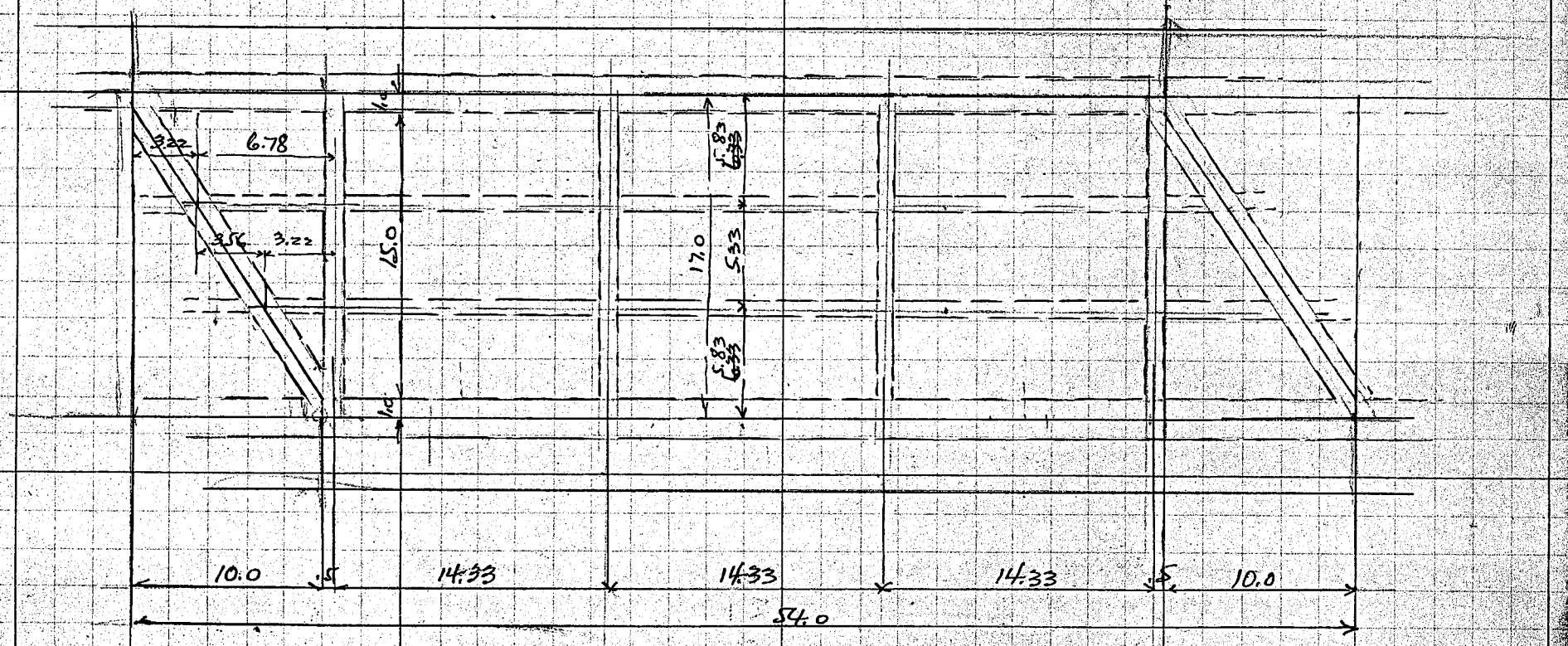
Final Design of Omatsugawa Bridge Tokushimaken

As stated on pp 1. this crossing will be on skew of 1.7 to 1.0. The total length of bridge 5-54R steel girder span encased with concrete and 6-40R continuous concrete girder span. The general cross section of bridge is same as for Katsuura gawa Bridge.

54R Span



Cross Section of Structure
Same as shown on pp 2.

Floor slab same as for Katsuuragawa Bridge.

Concrete Stringer Span length 14.33
Use same reinf. as for 60' girder span which is shown on pp 4 and 5

Intermediate Floor Beam
For figure see pp 5
By 1/8 moment instead of 1/10 m

Dead Load moment 65.00 × 5.83 = 37900
1/8 × 468 × 17² = 16900
54800

Live Load moment 83700
1/8 × 390 × 17² = 14100
97800
152600

section modulus = 152600 × 12 / 16000 = 114.0

Use 20" × 65# I Sm = 117.0

Use same I beam for end floor beam.

Skew Beam at end made out of concrete use 4-7/8" bars.

Main Girder Span length say 53.0
Dead Load moment = 1/8 × 3295 × 53² = 1,160,000 #

Live Load 3 motor trucks side by side
Uniform load at front and rear of motor trucks

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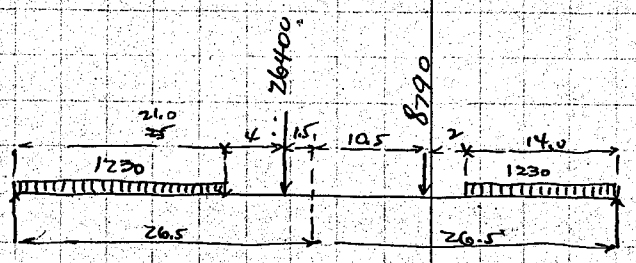

$35190 \times \frac{25^2}{53} = 415000$ #

$1230 \times 14 = 17200 \times \frac{7}{53} = 2270$

$1230 \times 21 = 25800 \times \frac{42.5}{53} = 20700$

22970

Moment = $22970 \times 25 = 574000$

less $25800 \times 14.5 = 374000$

200.000

415000

Total 615000 #

Dead load moment 1160.000

Extra Pl. $8 \times 150 \times 53^2 =$ 53.000

1213.000

Live load moment 615000

1828.000 #

Depth of girder 48½" web = $48 \times \frac{3}{8} = 18.0$ ⅛web = 2.25

Effective Depth = 3.85 Stress = $1828000 \div 3.85 = 474.000$ #

Section required = $\frac{474.000}{16000} = 29.6$ □" − ⅛ web = 2.25 = 27.35 □" net

Try the following section

2 L's 6 × 6 × ⅝	=	14.22	Gross	11.720 □" net
1 Cov Pl 18 × ½	=	9.00		8.00
do	=	9.00		8.00
		32.22		27.72 □"

For top flange use 2 L's 6 × 6 × ⅝"

Length of Cov Pl $53.0 \times .536 + 2 = 30.4'$

$53.0 \times .758 + 2 = 42.2'$

Resisting moment of Concrete Slab. Ok.

see pp 7.

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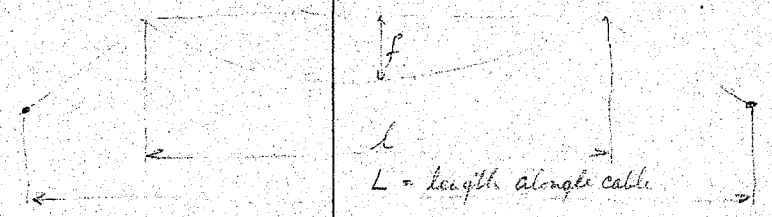
CALCULATIONS FOR

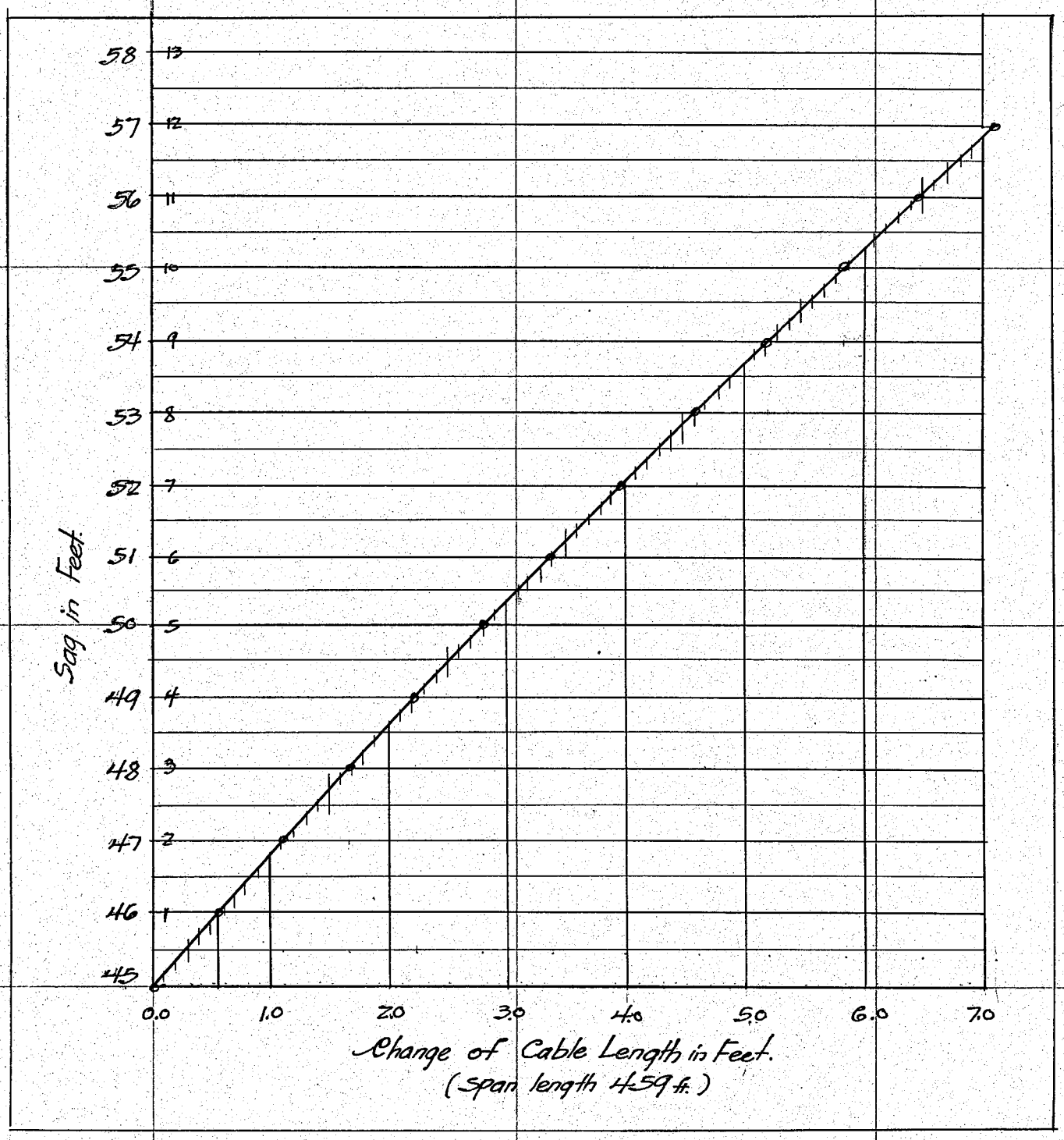
Miyoshi Bashi for Tokushima ken.

Relation Between Cable Length and its Sag.

$$L = l\left(1 + \frac{8}{3}n^2\right)$$

where L = Total length of the cable in feet.
l = Span length in ft. =(459')
f = Sag of the cable in ft. =(55')
n = sag ratio $\frac{f}{l}$.





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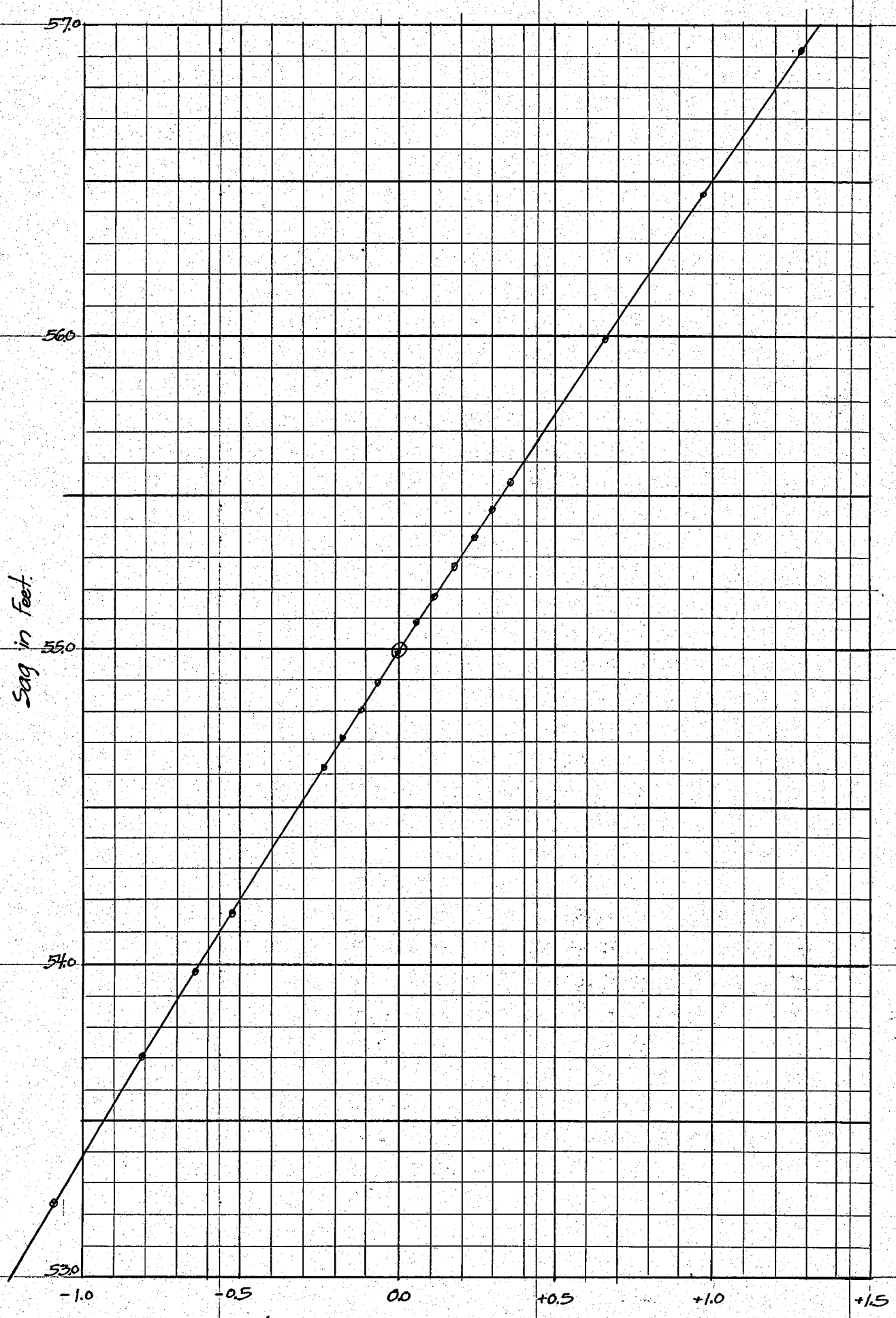
CALCULATIONS FOR

Miyoshi-Bashi for Tokushima Ken
Relation between Cable Length and its Sag.



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CALCULATIONS FOR

Miyoshi Bashi for Tokushima Prefecture

Change of Sag due to horizontal displacement of tower rollers (change of span length).

$L = l(1+\frac{8}{3}n^2) = l(1+\frac{8}{3}\frac{f^2}{l^2})$ L constant, f + l variables.

Let f' be the sag for the changed span length l'

$$l(1+\frac{8}{3}\frac{f^2}{l^2}) = l'(1+\frac{8}{3}\frac{f'^2}{l'^2})$$

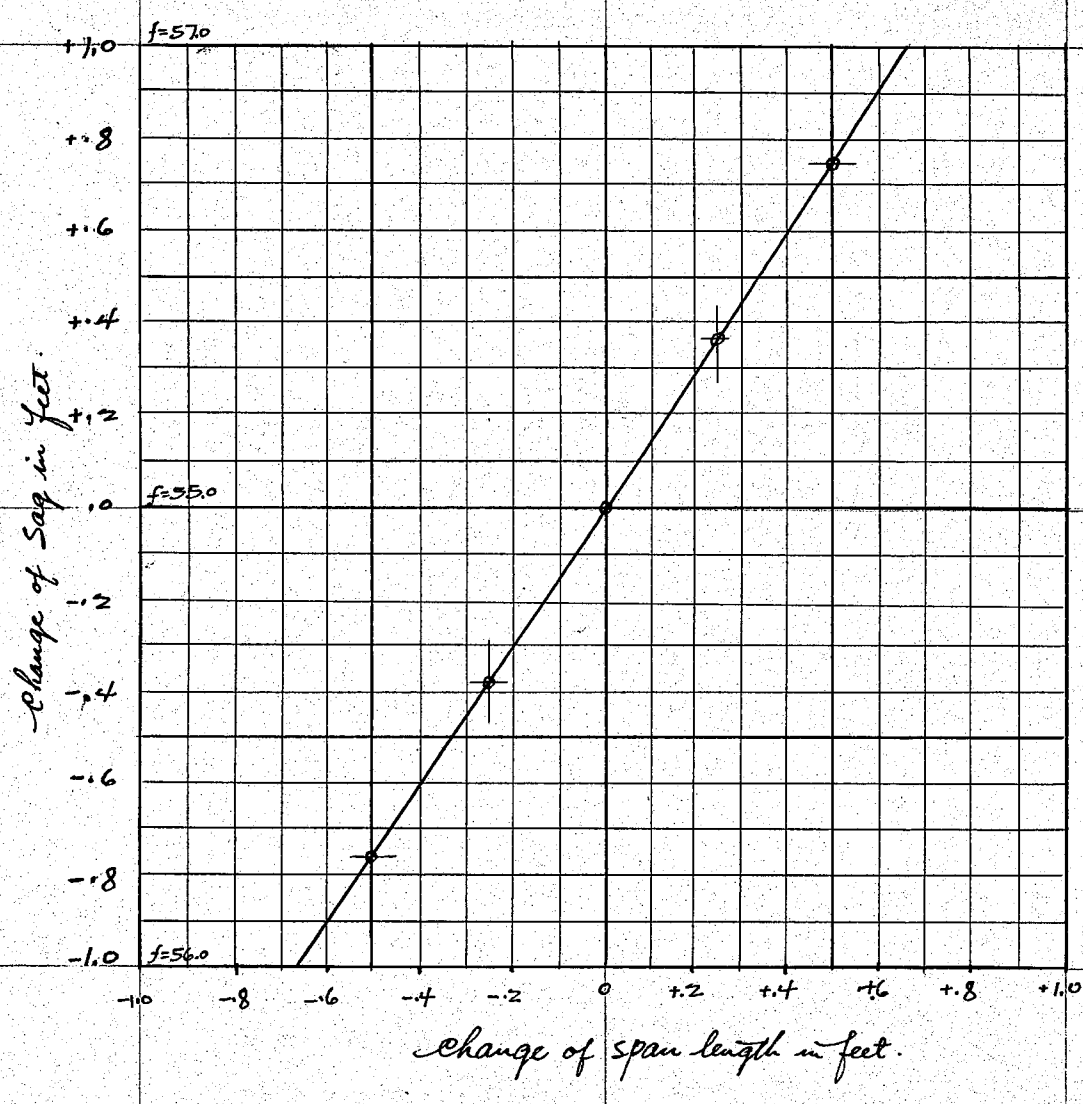
$$l+\frac{8}{3}\frac{f^2}{l} = l'+\frac{8}{3}\frac{f'^2}{l'}$$

$$l-l'+\frac{8}{3}\frac{f^2}{l} = \frac{8}{3}\frac{f'^2}{l'}$$

$$f'^2 = (l-l'+\frac{8f^2}{3l})\frac{3}{8}l'$$

$$= \frac{3l'}{8}(l-l')+\frac{f^2l'}{l}$$

l	l'	$l-l'$	$\frac{3}{8}l'$	$\frac{3l'}{8}(l-l')$	f^2	$\frac{l'}{l}$	$f^2\frac{l'}{l}$	f'^2	f'	$f'-f$
459.0	458.5	0.50	171.9375	85.9698	3025.0	0.9989	3021.6726	3107.6423	55.746	0.746
459.0	458.75	0.25	172.0313	.43.0078	3025.0	0.9995	3023.4875	3066.4953	55.376	0.376
459.0	459.00	0.00	172.1250	0.0000	3025.0	1.0000	3025.0000	3025.0000	55.000	0.000
459.0	459.25	−0.25	172.2188	−43.0547	3025.0	1.0005	3026.5126	2983.4578	54.621	−0.379
459.0	459.50	−0.50	172.3125	−86.1563	3025.0	1.0011	3028.3275	2942.1712	54.242	−0.758



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CALCULATIONS FOR

Deflection of Cable due to the change of its length. ケーブルの伸び.

Johnson Bryan Turneaure, Modern Framed Structures, page 193 Eqn. 210

$L = l\left(1 + \frac{8}{3}n^2\right)$ where L = Total length of the cable
l = Span length in ft. = 459'
f = Sag in ft. = 55.0'
n = ratio $\frac{f}{l}$

For $l = 1$

$L_{unit} = 1 + \frac{8}{3}n^2$

n	n^2	$1+\frac{8}{3}n^2$	$l(1+\frac{8}{3}n^2)$
53.244	0.1160	0.013456	1.03588	475.469 ✓	
53.703	0.1170	0.013689	1.03650	475.754 ✓	
53.9784	0.1176	—		475.929 ✓	
54.162	0.1180	0.013924	1.03713	476.043 ✓	
54.621	0.1190	0.01416100	1.03776271	476.3331 ✓ ✓	54.8 ✓
54.713	0.1192	0.01420864	1.03788981	476.3914 ✓ ✓	
805	0.1194	0.01425636 ✓	1.03801170 ✓	476.4498 ✓ ✓	0.8
896	0.1196	0.01430416	1.03814474	5083 ✓ ✓	
54.988	0.1198	0.01435204	1.03827210	5669 ✓ ✓	55.0
55.080	0.1200	0.01440000	1.03840000	6256 ✓ ✓	
.172	0.1202	0.01444804	1.03852810	6844 ✓ ✓	
.264	0.1204	0.01449616	1.03865644	7433 ✓ ✓	
.355	0.1206	0.01454436	1.03878504	8023 ✓ ✓	
.447	0.1208	0.01459264	1.03891370	8614 ✓ ✓	
.539	0.1210	0.01464100	1.03904270	476.9206 ✓ ✓	55.54
55.998	0.122	0.014884		477.2180 ✓✓	
56.457	0.123	0.015129	1.04034	477.530 ✓	
56.916	0.124	0.015376	1.04100	477.849 ✓	

$n' = \frac{55+f'}{459} = n + g'$

$n' = \frac{f'}{459} = \frac{2}{10000}$

$\frac{f}{l} = .1190$

$f' = \frac{1cm}{100cm} = .1$

$\frac{55+2}{100cm}$ 100 101

$\frac{1}{1600}$ .1198 .1200

$\frac{55}{l}$ $\frac{55}{459}$ $\frac{f'}{459} = \frac{.092}{10000}$

$\frac{.046}{10000}$ $x = \frac{459 \times .1}{10cm}$

$\frac{1cm}{900}$ $\frac{x}{10000} \times 459 = .0918$

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CALCULATIONS FOR

Sag f	span l	$\frac{f}{l}=n$	n^2	$1+\frac{8}{3}n^2$	$l(1+\frac{8}{3}n^2)$		
45.00 ft	÷ 459	0.0980	.009600	1.02560	470.750 ft	✓	✓
46.		0.1002	.010040	1.02677	471.287	✓	✓
47		0.1024	.010486	1.02796	471.834	✓	✓
48		0.1046	.010941	1.02918	472.394	✓	✓
49		0.1068	.011406	1.03042	472.963	✓	✓
50		0.1089	.011859	1.03162	473.514	✓	✓
51		0.1111	.012343	1.03291	474.106	✓	✓
52		0.1133	.012837	1.03423	474.712	✓	✓
53		0.1155	.013340	1.03557	475.327	✓	✓
54		0.1176	.013830	1.03688	475.928	✓	✓
55		0.1198	.014352	1.03827	476.566	✓	✓
56		0.1220	.014884	1.03968	477.218	✓	✓
57		0.1242	.015426	1.04114	477.833	✓	✓

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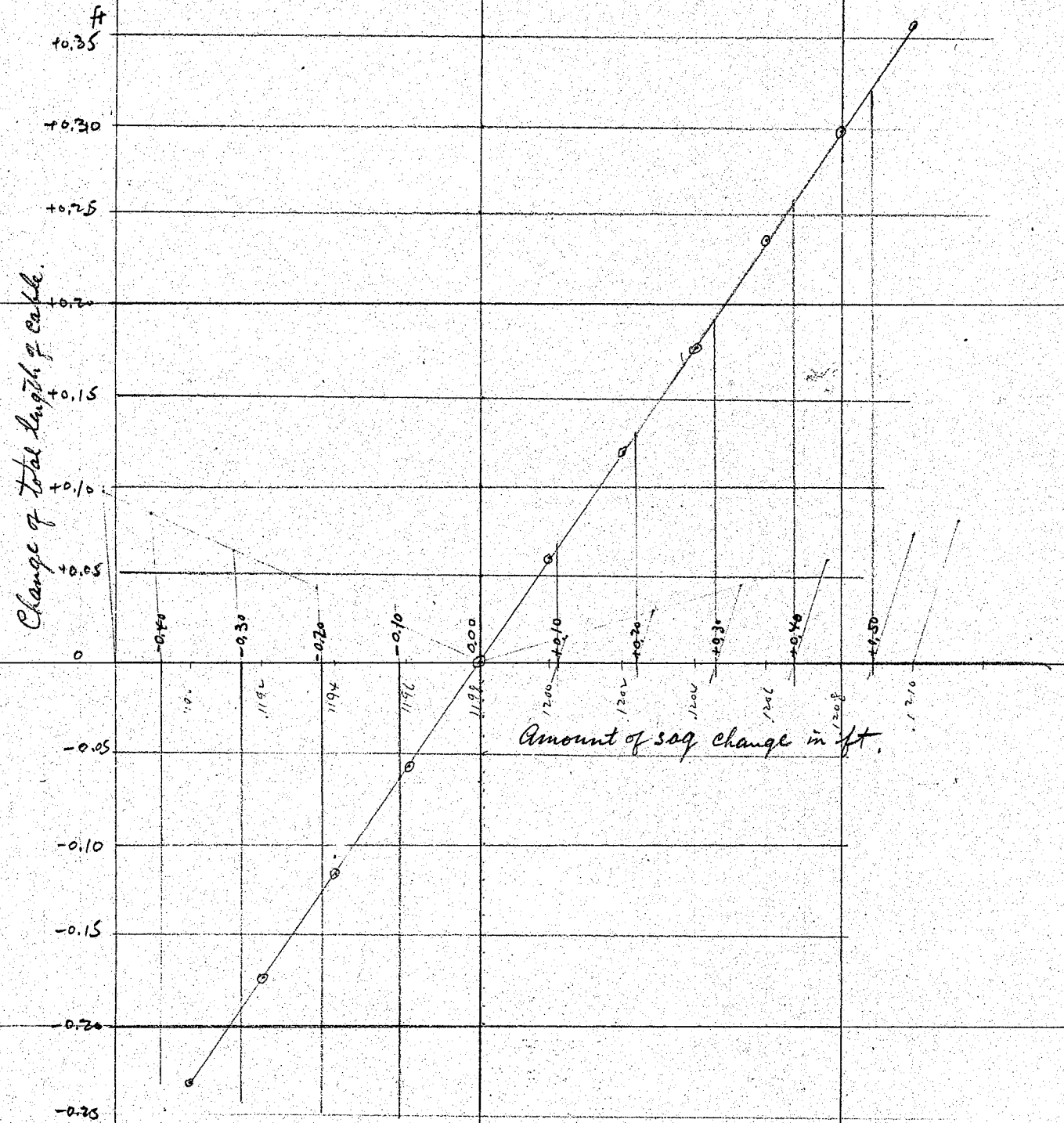
CALCULATIONS FOR

三径間



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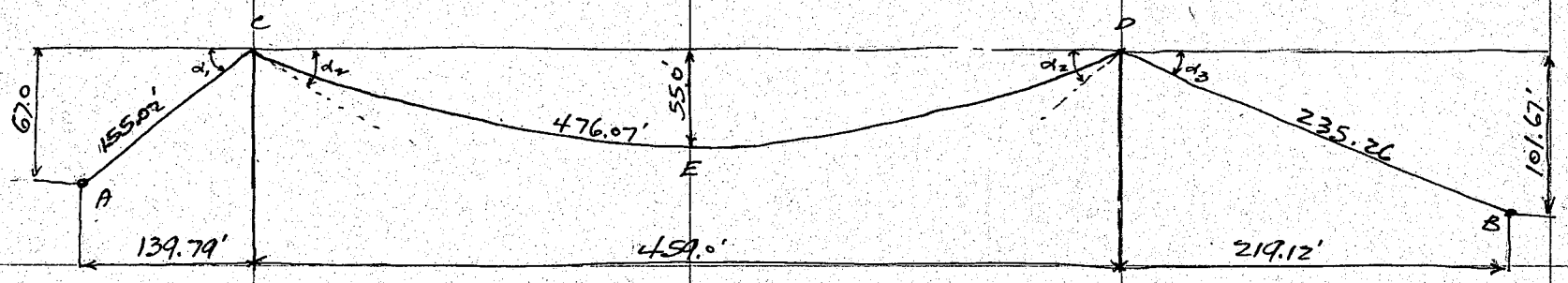
CALCULATIONS FOR

Miyoshi-Bashi for Tokushima-ken
Modulus of elasticity of the cable.



Dead load Horizontal thrust on the cable = 607,000 # (Calculation page 45.) for each cable.

Slope $\alpha_1 = \text{Sec}^{-1} \frac{155.02}{139.79} = {}^{\text{Sec}^{-1}}_{1}1.109 = 25°-40'$

Slope $\alpha_2 = \text{Sec}^{-1}\ \text{tan}^{-1} \frac{55.2}{229.5} = {}^{\tan^{-1}}_{1}.479 = 25°-40'$ $\quad \text{Sec}\,\alpha_2 = 1.109.$

$\therefore \quad \alpha_3 = \text{Sec}^{-1} \frac{235.26}{219.12} = \text{Sec}^{-1}\ 1.073 = 21°-20'$

Dead load stress on cable.

				Average	length	
A–C.	607,000 × 1.109	=	673,000 #	673000	× 155.02'	= 104,300,000
C–E at C	do	=	673,000			
C–E at E		=	607,000	640,000	× 476.07'	= 304,500,000
E–D at D	607,000 × 1.109	=	673,000			
D–B	607,000 × 1.073	=	652,000	652,000	× 235.26'/866.35'	= 153,400,000
						562,200,000

Average stress on cable = $\frac{562,200,000}{866.35} = 649,500$ #

for one cable = 649,500 ÷ 19 = 34,200 #

Horizontal thrust due to wt. of cable. 4.5 # per lin ft.

$H = \frac{4.5 \times 459^2}{8 \times 55} = 2160$ #

Average throughout the cable = $2160 \times \frac{649,500}{607,000} = 2,300$ # nearly.

Net average D.L. stress = 34200 − 2300 = 31,900 #/cable.

Net area of one cable = $\frac{4.5}{34} = 1.324$ □"

Unit stress = $\frac{31900}{1.324} = 24,100$ #/□" tension (one strand).

Modulus of Elasticity of the cable assumed 6,800,000 #/□" ~ 7,500,000 #/□"

Elongation of the cable = $\frac{\text{stress}}{E} \times 866.35 = \frac{24100}{E} \times 866.35$

Elongation for E = 6,800,000 $\quad \Delta l = \frac{24100}{6,800,000} \times 866.35 = 3.07$ ft.

" " E = 7,500,000 $\quad \Delta l = \frac{24100}{7,500,000} \times 866.35 = 2.78$ ft.

Sag change due to elongation

Δl = 3.07 ∴ Increase of sag = 5.1 ft.
Δl = 2.78 ∴ " = 4.6 ft.

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