JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

MADE BY ________ DATE ________ FILE NO ________
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CALCULATIONS FOR

岡山縣霞橋設計々算書
及材料調書

大正十五年八月

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

MADE BY J. Masuda DATE 15-6-16 FILE NO
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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

This bridge is to be built on main highway between Kurashiki and Tamashima Okayama-Ken and to cross Takahashi-gawa at 50 ken down stream about of the present wooden highway bridge. For final layout we adopted the following beginning from left bank,

11 spans	60'-0" out to out	=	660'-0"
7 spans	177'-3" out to out	=	1240'-9"
2 spans	60'-0" " " "	=	120'-0"
Space at abutments	2@2"	=	4"
Space on girder piers	11@2"	=	1'-10"
2@1½" spaces		=	3"
			2023'-2"

Note:- The truss spans shown above will be out to out without expansion joints; 3" floor expansion shall be provided on truss piers. The span length between center to center of end pins = 175'-0"

The bridge shall be cambered toward centre of stream and the crown shall be located at ℄ of centre span of trusses.

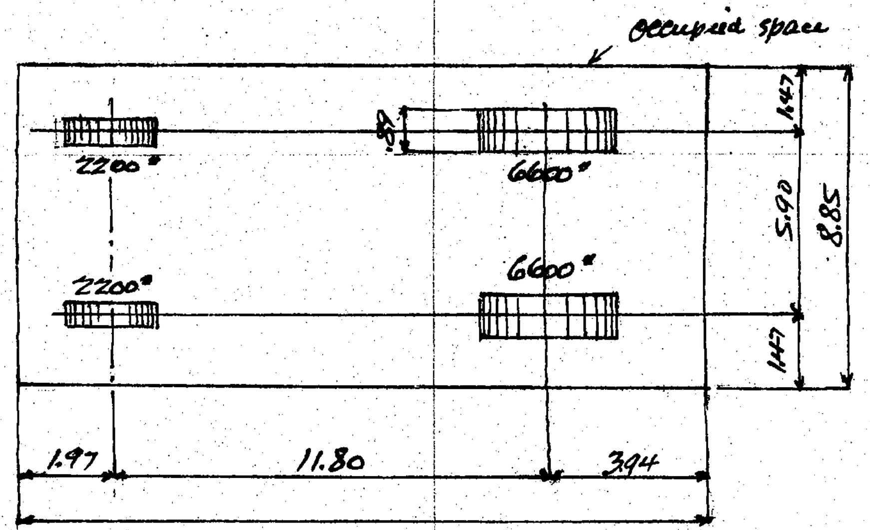
The width of roadway 21.0 ft between curb lines; Pavement shall be 2" Soliditit or asphaltic concrete on reinforced concrete slab. The handrail will be of structural steel of proper ornamental design.

Assumed Loadings

Uniform load on roadway $w = \frac{100000}{170+l} = 500^{kg/m^2}$ max say 100 #/□'

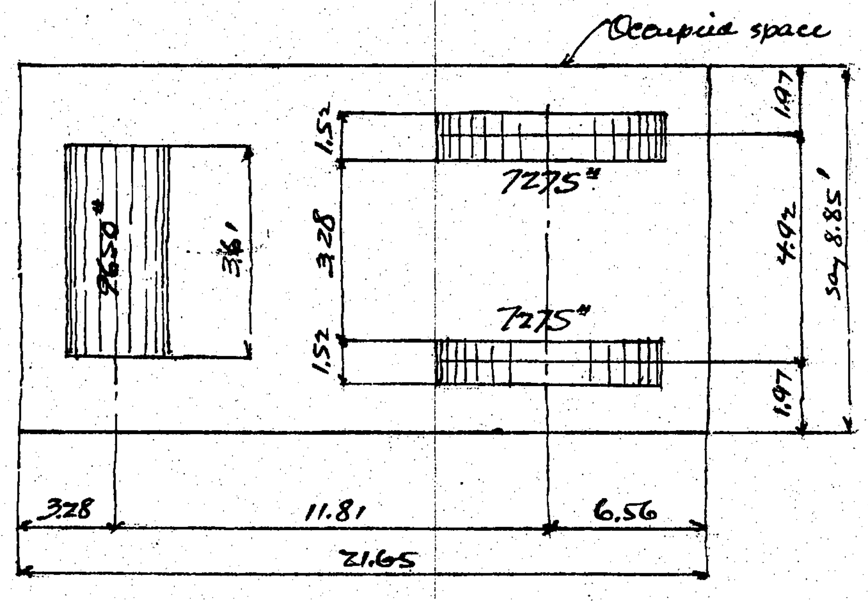
where w = Uniform load in kg per sq. meter
l = span length in meter.

8 ton motor truck loading (17600#).



2 rows of motor traffic on roadway with occupied width of 8.85' each
Impact into Consideration
Unoccupied space of motor truck to be filled with the uniform load.

11 ton Road roller (24200#).



One road roller on one span. no impact

Impact for motor truck loading

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

where l = span length in meter
max impact 30%
No impact for uniform load and road roller.

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Assumed weight of materials.

Plain concrete	140 # per cubic ft
Reinforced Concrete	150 " " "
Structural steel	490 " " "
Cast iron	450 " " "
masonry	160 " " "
Soledetit or asphalt pavement	140 " " "

Allowable working strength

Structural steel or Reinforcing bars.

Tension	17000
Extreme fibre stress	17000
Shear on web gross section	12800

Compression member 21300 (1- 0.0055 l/r) or not over 14000 #/□" gross area

Compression flange of plate girder

$$17000\ (1-.0012\ l/b) \leqq 15400\ \text{\#/□"}$$

where l = unsupported length of flange in inches

b = width of flange in inches.

Shearing on shop rivets (machine driven)	12000 #/□"
" " field " " "	10.000
Extreme fibre stress of pin	24.000
Bearing on shop rivets	24.000
" " field "	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 #/□"

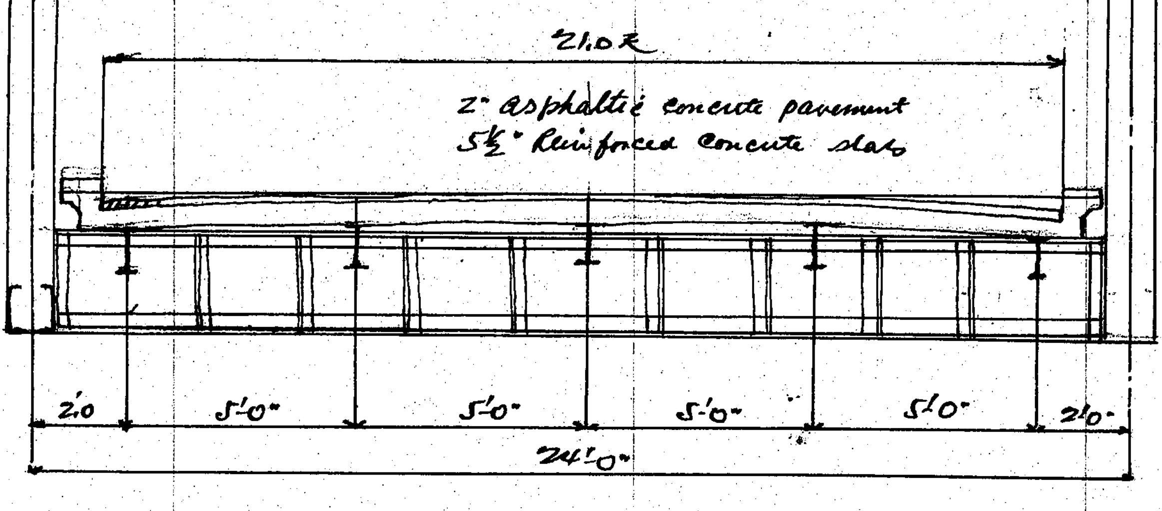
Concrete 1:2:4 mixture

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

Considering wind and temperature stresses in addition to dead, live and impact stresses, the allowable working strength shall be increased 25% and proportioned the parts. In figuring earthquake, the working strength shall be increased 80% and proportioned the parts.

Accelleration of Earthquake assumed 1000 mm/sec² or k = 0.1

Cross section of Truss span assumed as shown below in sketch.



CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Reinforced Concrete slab. span length of slab. = 5'-0" center to center of stringers

Dead Load

2" asphaltic Concrete pavement 24#
5½" slab. 69
93# per square ft.

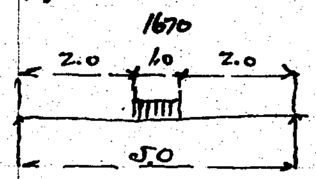
Dead Load moment = $\frac{1}{10} \cdot 93 \cdot 5^2$ = 232'#
Dead Load shear = $93 \cdot 2.5$ = 232#

Live Load motor truck rear wheel 6600
Impact 30% 1980
8580# Front wheel $\frac{1}{3} \cdot 8580 = 2860$#

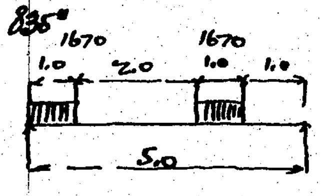
Distribution of wheel Concentration

Contact space between wheel and pavement assumed 20cm = 0.66
2 • .17 pavement = 0.34
long'l ~~Transverse~~ distribution a = 1.00'
Transverse distribution b = 0.89 + 0.34 = 1.23'

Effective width $E = \frac{2}{3}(l+b) + a$ where l = span length
$= \frac{2}{3}(5.0 + 1.23) + 1.0 = 5.15'$



Load per ft strip 8580 ÷ 5.15 = 1670# assume transverse distribution 1.0'
moment due to single wheel load 835 • 2.5 = 2090
Less moment 835 • .25 = - 209
1881'#



For continuity of slab. take moment as 0.8 • 1881 = 1505
max End shear as simple beam
$1670 \cdot \frac{6.0}{5.0} = 2000$#

Summary for moments and shears

	moment	shear
Dead Load	232	232
Live Load	1881	2000
	2113'#	2232#

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

Effective depth reqd = $\sqrt{\frac{2113}{102}} = 4.55$"
insulation at bottom say .95" make slab 5½" over all.

Steel Area required $\frac{2113 \cdot 12}{\frac{7}{8} \cdot 4.55 \cdot 17000} = .375$ Use ½"⌀ bars 6" centers = 0.39 □"

Shear = $\frac{2232}{\frac{7}{8} \cdot 4.55 \cdot 12}$ = 47#/□" ok

Bond stress $u = \frac{2232}{\frac{7}{8} \cdot 4.55} = 560$# per ft strip 2-½"⌀ @ 1.57 • 130# = 409#
with lapping 409
818#

Bent up bars will be lapped over center stringer.

For end stringer End shear = $1670 \cdot \frac{4.5}{5.0}$ = 1410
Dead Load shear 232
1642#

Bond stress $u = \frac{1642}{\frac{7}{8} \cdot 4.55} = 412$# 2-½"⌀ bars per ft strip ok for bond.

Design of Longitudinal Stringers

Stringer SC stringer spacing 5'-0" span length = 17.5'

Dead Load

slab and pavement 93 • 5 = 465
stringer assumed 45
510# per lin. ft.

Dead Load moment = $\frac{1}{8} \cdot 510 \cdot 17.5^2$ = 19550'#
" " shear = $\frac{1}{2} \cdot 510 \cdot 17.5$ = 4470#

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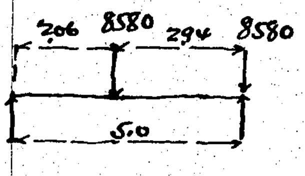
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

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

Load on stringer Rear wheel Reaction

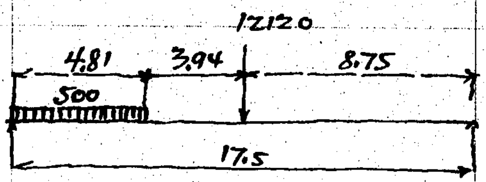


$8580 \cdot \frac{2.06}{5.00} = 3540$
8580
12120#

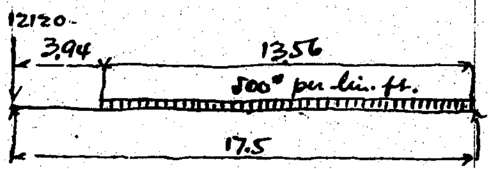
Front wheel Reaction 4040#

Uniform live load $5 \times 100 = 500$# per lin. ft.
$500 \cdot 4.81 = 2405$#

Live Load Moment



moment due to truck $\frac{12120}{2} \cdot 8.75 = 53000$
Uniform load $2405 \cdot \frac{2.41}{17.5} \cdot 8.75 = 2900$
55900'#



Max End Shear
Uniform live load $500 \cdot 13.56 = 6770$
Reaction $= 6770 \cdot \frac{6.78}{17.5} = 2620$
motor truck 12120
14740#

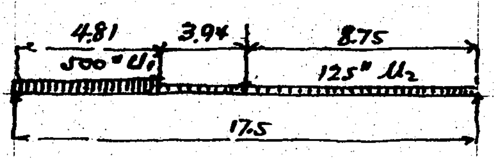
Summary for moments and shears

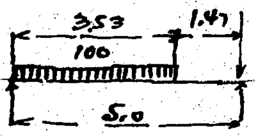
	moments	shears
Dead Load	19550	4470
Live Load	55900	14740
	75450'#	19210#

section modulus required
$\frac{75450 \times 12}{17000} = 53.2$
Use 12" × 6" @ 44.02# Sm = 52.57
Unit stress $= \frac{75450 \times 12}{52.57} = 17200$ #/□"
say Ok.

Stringer SB.
Dead Load assume same as Sc neglecting cantilever action at SA.
Live Load motor truck loading 8580#

Uniform load for 5.0' wide 500# per ft
Unif. load on side of motor truck
$R = 353 \cdot \frac{1.77}{5.0} = 125$#





$500 \cdot 4.81 = 2405$#
$125 \cdot 12.69 = 1585$#

Reaction $2405 \cdot \frac{2.41}{17.5} = 331$#
" $1585 \cdot \frac{11.15}{17.5} = 1010$
1341#

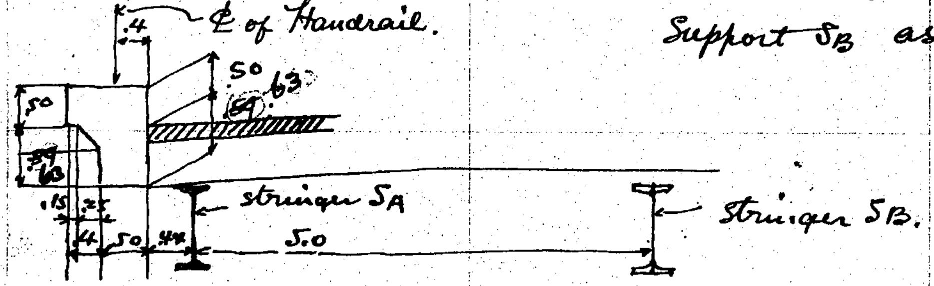
moment $= 1341 \cdot 8.75 = 11720$
$\frac{125 \cdot 8.75^2}{2} = 4800$
6920'#
motor truck $\frac{8580}{2} \cdot 8.75 = 37500$
44420'#
Dead Load moment 19550
63970'#

section modulus required $= \frac{63970 \cdot 12}{17000} = 45.1$ Use 12" × 6" @ 44.02# Sm = 5257

Unit stress $= \frac{63970 \cdot 12}{5257} = 14600$ #/□"

Design of End Stringer SA.
Dead Load

Reaction on stringer SA
Support SB assumed as simply supported.



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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken.

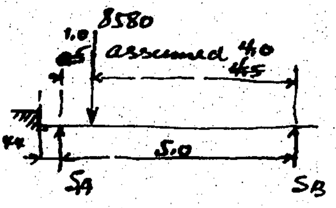
			weight		Arm		moment
pavement and slab.	93# × 5.44	=	505	×	2.72	=	1375
coping	.5 × 1.13 × 150#	=	85	×	5.69	=	484
"	0.03 × 150#	=	5	×	6.02	=	30
"	0.20 × 150	=	30	×	6.14	=	184
Handrail say			60	×	5.84	=	350
			685#				2423

Reaction on SA 2423 ÷ 5.0 = 485#
" " SB +485 - 685 = -200#

Dead Load on SA floor 485
Stringer 45
530# per lin ft.

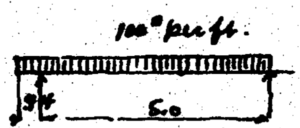
Dead Load moment = $\frac{1}{8} \times 530 \times 17.5^2$ = 20300'#
Dead Load shear = $\frac{1}{2} \times 530 \times 17.5$ = 4640#

Live Load motor truck loading Rear wheel Concentration = 8580# with impact.

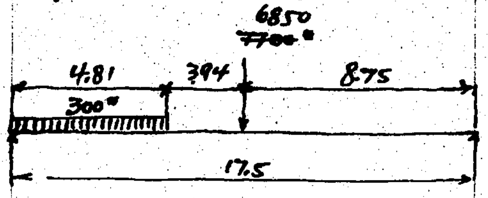


Reaction on SA = $8580 \times \frac{4.5}{5.0}$ = say 7700#

In case of 1.5' from curb line
Reaction on SA = $8580 \times \frac{4.0}{5.0}$ = 6850#



Uniform live load 100 × 5.44 = 544
Reaction on SA = $544 \times \frac{2.72}{5.0}$ = say 300# per lin ft of stringer
300 × 4.81 = 1443#
Reaction = $1443 \times \frac{2.41}{17.5}$ = 199#



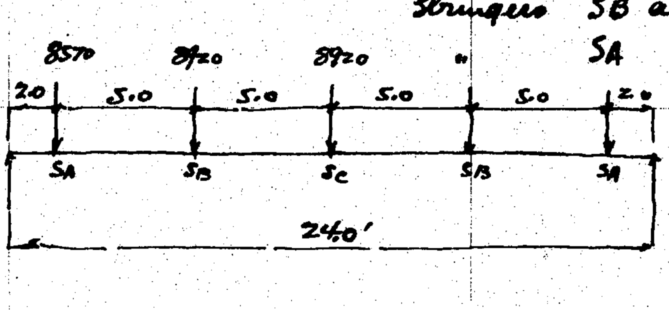
moment due to unif. load 199 × 8.75 = 1740'#
" " " truck $\frac{6850}{2} \times 8.75$ = 30000
31740'#
Dead Load moment = 20300
52040'#

Section modulus required = $\frac{52040 \times 12}{17000}$ = 366 Use 12" × 5" I @ 31.99# Sm = 36.69 Ok.

Approximate weight of stringers
3 @ 12" × 6" @ 44.02# I = 132#
2Is 12 × 5" @ 31.99# = 64
196#
Details + variation 5% = 10
206# per lin ft of truss span.

Design of floor beams.
Intermediate Floor Beam
Dead Load stringer SC 510# per lin ft.
stringer SB 510# " " " neglecting cantilever effect
Stringer SA 685 - 93 × 2.5 = 453#
Stringer say 35
488# call this 490# per lin ft.

Concentration on floor Beam
stringers SB and SC 510 × 17.5 = 8920#
SA 490 × 17.5 = 8570#



Reaction = 8920 × 1.5 = 13380
8570
21850#

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CALCULATIONS FOR

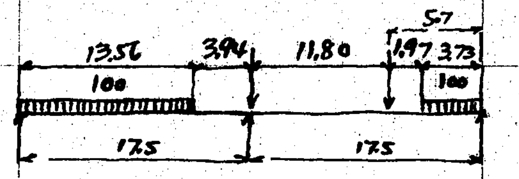
Design of Kasumi-Bashi for Okayama-Ken.

Dead Load moment
21850 · 12 262200
8920 · 5 = 44600
8570 · 10 = 85700
-130300
131900 ″#

End shear
Conc = 21850
Beam 1560
23410 #

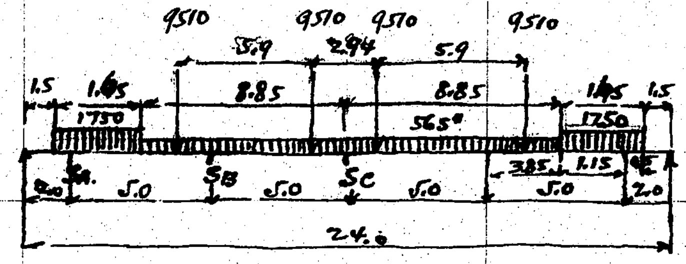
Dead Load of girder assumed 130 # per ft $m = \frac{1}{8} \cdot 130 \cdot 24^2$ = 9360
141260 ″#

Live Load motor truck loading. Rear wheel = 8580 #
Front wheel = 2860 #



Reaction on floor beam due to uniform load.
1356 # · 6.78 ÷ 17.50 = 525 #
373 # · 1.87 ÷ 17.50 = 40
565 # for rear and front of motor truck.

For sides of motor truck 1750 # per ft of floor beam.
Reaction on floor beam due to motor truck loading. 2860 · $\frac{5.7}{17.5}$ = 930 #
8580
9510 #



Reaction on stringer due to motor truck
stringer Sc 9510 · $\frac{3.53}{5.00}$ = 6720
6720
13440 #

stringer SB 9510 - 6720 = 2790
9510 · $\frac{2.63}{5.0}$ = 5000
7790 #

Stringer SA 9510 - 5000 = 4510 #

Uniform live live load
565 · 5.0 = 2820 #
565 · 3.85 = 2180 #
1750 · 1.15 = 2010 #
1750 · 0.5 = 875 #

Reaction on stringer due to uniform live load.
stringer Sc 1410 · 2 = 2820 #
Stringer SB 2180 · $\frac{3.08}{5.00}$ = 1340
2010 · $\frac{.58}{5.00}$ = 234
1410
2984 #

Stringer SA. 2180 - 1340 = 840
2010 - 234 = 1776
875
3491 #

moment due to motor truck loading
19020 · 12 = 228500
7790 · 5 = 38950
4510 · 10 = 45100
-84050
144450 ″#

moment due to Uniform load.
7885 · 12 94620
2984 · 5 = 14920
3491 · 10 = 34910
49830
44790
189240 ″#

Max Live Load End shear, motor trucks on one side of roadway.

			wt		arm		moment
motor truck	4 · 9510	=	38040	·	13.65	=	518000
Unif. load	565 · 24 ~~.85~~	=	10000	·	13.65	=	136500
〃 〃	1750 · 2.8	=	6780	·	3.15	=	18200

672700 ÷ 24 = 28.000 #

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken.

Intermediate Floor Beam continued.

Summary for moments and shears.

	moment	shear
Dead Load	141260	23410
Live Load	189240	28000
	330500"#	51410#

Depth of girder = 27½ × b to b of Ls
web assumed 27 × 5/16 = 8.440□" ⅛ web = 1.050□"
Effective depth 2.23'
flange stress = 330500 ÷ 2.23 = 148000#
ΣR = 148000 ÷ 17000 = 8.72 − 1.05 = 7.67□" net

Try 2Ls 5 × 3 × 3/8 = 5.72 − 4.40
1Pl 10½ × 3/8 = 3.94 − 3.28
9.66□" 7.68□" net ok

Length of cover plate $24 \times \sqrt{\frac{3.28}{7.68}} + 2 = 17.7'$

Weight of Floor Beam

Flange	4Ls	5 × 3 × 3/8	@ 9.8	× 23.0	= 900
web	1Pl	27 × 5/16	@ 28.69	× 23.0	= 660
cov. pls.	2Pls.	10½ × 3/8	@ 13.39	× 17.7	= 473
End stiffs	4Ls	3½ × 3½ × 3/8	@ 8.5	× 2.23	= 76
fillers	4Pls.	6½ × 3/8	@ 8.29	× 1.79	= 59
Stiff at stringer	10Ls	3½ × 3½ × 3/8	@ 8.5	× 2.23	= 190
filler	10Pls.	3½ × 3/8	@ 4.46	× 1.79	= 80
stiffs	8Ls	3 × 3 × 5/16	@ 6.10	× 2.30	= 112
		Rivet heads + variation 3½%			90
					2640 ÷ 23 = 115# per lin. ft.

End Floor Beam

Dead Load stringer S_C 510# per ft × $\frac{18.6^2}{2 \times 17.5}$ = 5040#
" S_B " " " do = 5040#
" S_A 490 " " do = 4840#

Reaction 5040
2520
4840
12400#

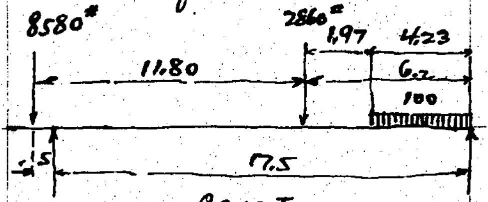
Dead Load moment at center 12400 × 12 148800
5040 × 5 = 25200
4840 × 10 = 48400
− 73600
75200"#

Dead Load Beam 100# per ft m = ⅛ × 100 × 24² = 7200
82400"#

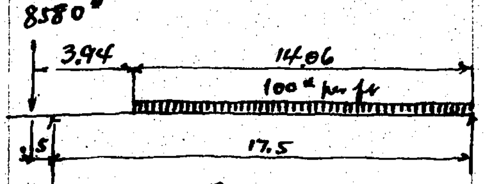
Max Dead Load shear = 12400 + 1200 = 13600#

Live Load

Load on floor beam



Case I



Case II

motor truck rear wheel with impact 8580#
front " " " 2860#

R = $8580 \times \frac{18.0}{17.5}$ = 8820
$2860 \times \frac{6.2}{17.5}$ = 1010
9830#

Uniform load $\overset{4.23}{\cancel{6.2}} \times 100$ = $\overset{423}{\cancel{620}}$#
Reaction = $\overset{423}{\cancel{620}} \times \frac{2.11}{17.5}$ = 51# per ft.

Case II
Uniform load 1406#
Reaction = $1406 \times \frac{7.03}{17.5}$ = 565#

Motor truck loading = $8580 \times \frac{18.0}{17.5}$ = 8820#

For full unif. load 100 × 18.6 = 1860
Reaction = $1860 \times \frac{9.30}{17.5}$ = 990# per ft of floor beam

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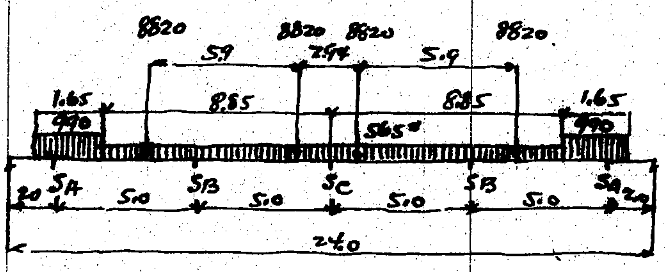
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

End Floor Beam



Uniform live load

565 · 5.0 = 2820
565 · 3.85 = 2180
990 · 1.15 = 1005
990 · 0.5 = 495

Reaction on stringer due to motor truck

Stringer SC 8820 · $\frac{3.53}{5.00}$ = 6220
6220
12440"

Stringer SB 8820 - 6220 = 2600
8820 · $\frac{2.63}{5.00}$ = 4650
7250"

Stringer SA 8820 - 4650 = 4170"

Reaction on stringer due to uniform load

Stringer SC 1410 · 2 = 2820"

Stringer SB 2180 · $\frac{308}{5.00}$ = 1340
1005 · $\frac{.58}{5.00}$ = 117
1410
2867"

Stringer SA 2180 - 1340 = 840
1005 - 117 = 888
495
2223"

Moment due to motor truck loading

17640 · 12 = 211680
7250 · 5 = 36250
4170 · 10 = 41700
77950
133730 '#

Moment due to uniform load

6500 · 12 = 78000
2867 · 5 = 14335
2223 · 10 = 22230
36565
41435
175165 '#

Max live load End shear motor truck on one side of roadway

Motor truck 4 · 8820 = 35280 · 13.65 = 481000
Unif. load 565 · 2 · 8.85 = 10.000 · 13.65 = 136500
" " 990 · 3.3 = 3270 · 3.15 = 10300
627800 ÷ 24 = 26200"

Summary for moments and shears

	Moment	Shear
Dead Load	82400	13600
Live Load	175165	26200
	257565 '#	39800"

Depth of girder = 27½" b to b of L^s
~~web assumed~~ 27 · 5/16 = 8.44 □" ⅛ web = 1.05 □"
Effective depth 2.23
flange stress = 257565 ÷ 2.23 = 115500"
SR = 115500 ÷ 17000 = 6.80 - 1.05 = 5.750" net

Try 2L^s 4 · 3 · 3/8" = 4.96 - 3.64
1 Pl. 8½ · 3/8 = 3.19 - 2.53
8.15 □" 6.17 □"

Approximate weight of End floor beam.

1 web. 27 · 5/16 @ 28.69 · 22' 10½ = 656
4L^s 4 · 3 · 3/8 @ 8.5 · 22' 10½ = 779
2 Pl/s 8½ · 3/8 @ 10.84 · 18.0 = 390
misc details and rivet heads say 400
2225"

2225 ÷ 22.87 = 97" per lin ft of floor beam

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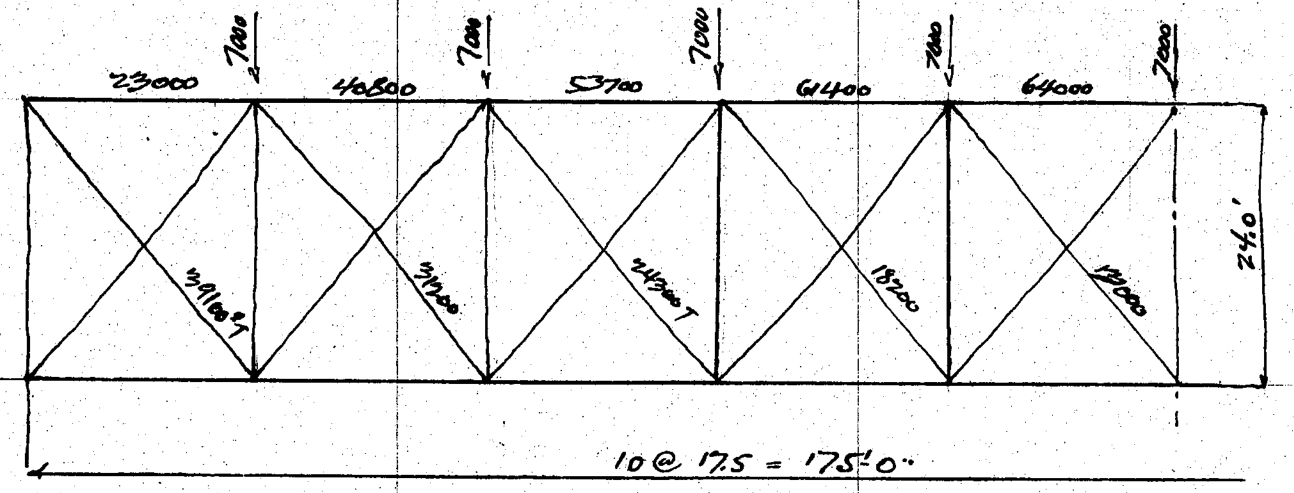
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

wind pressure assumed 400# per lin. ft Panel Concentration = 400 × 17.5 = 7000#



Sec θ = $\frac{29.70}{24.0}$ = 1.24

Panel	Shear		Stress	SR for Tension	Section	Area	Rivet 3/4
I	31500	× 1.24	= 39100# T	2.30 ☐"	2Ls 5×3×5/16	4.80	10
II	25200		31200	1.83	2Ls 5×3×5/16	4.80	8
III	19600		24300	1.43	2Ls 4×3×5/16	4.18	6
IV	14700		18200	1.07	2Ls 4×3×5/16	4.18	6
V	10500		13000	0.76	2Ls 4×3×5/16	4.18	6

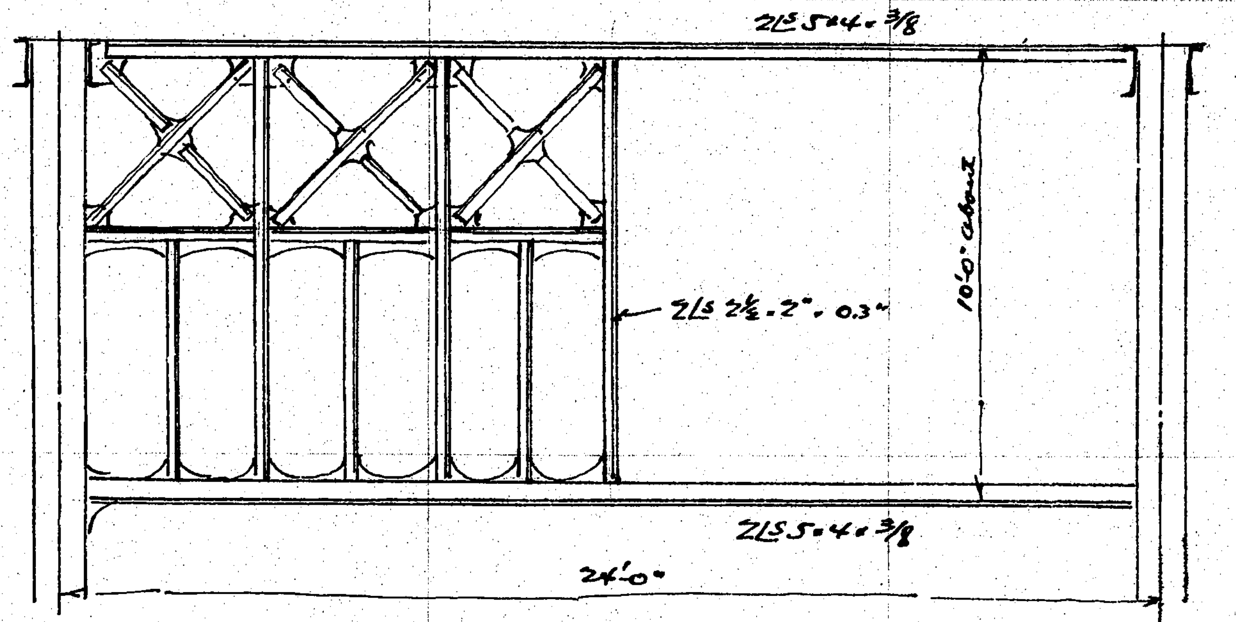
approximate weight of lower laterals

2Ls 5×3×5/16"	@ 8.2#	×	28.0	=	460
4Ls 5×3×5/16"	@ 8.2#	×	13.7	=	450
Center connection &c					75
					985#
2Ls 4×3×5/16	@ 7.2	×	28.0	=	403
4Ls 4×3×5/16	@ 7.2	×	13.7	=	394
Center Connection say					55
					852

Summary for Lower Lateral Bracing

4@ 985	=	3940
6@ 852	=	5110
Rivet heads &c		50
		9100# per truss.

Sway Bracings 4 Required for one span.



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CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Top and bottom struts	4Ls 5·4·3/8 @	11.0 · 22.4 = 986	
verticals and diagonals	2Ls 2½·2·0.3" @	4.28 · 130.0 = 1120	
Connection plates and details say		350	
		2456#	call this 2500# per sway
Top diagonal Bracings	2Ls 5·3·5/16	say 1000# per panel including	detail.
Transverse strut	4Ls 4·3·5/16 laced	760# per piece.	

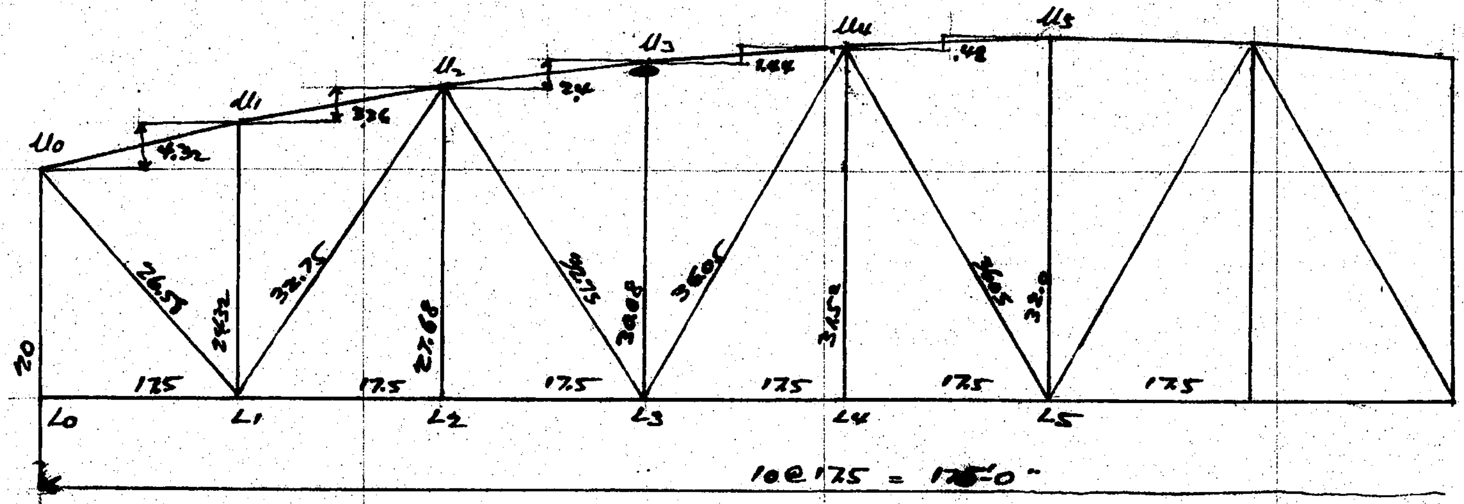
Portal Bracing

Top and bottom struts	4Ls 5×4·3/8 @	11.0 · 23.0 =	1010
diagonals	2Ls 3·3·5/16 @	6.1 · 24.0 =	300
verticals &c	2Ls 3·3·5/16 @	6.1 · 24.0 =	300
	Connection and misc details say		450
			2060# per piece.

Summary for weight of top lateral bracings

Diagonal Bracings	10 @ 1000 =	10.000
Transverse struts	5 @ 760 =	3800
Sway Bracings	4 @ 2500 =	10.000
Portal Bracings	2 @ 2060 =	4120
		27920# ÷ 175 = 159.5# per lin. ft. of truss

Design of truss
Span length 175'-0" c to c of End pins. truss spacing 24'-0" 10 panels @ 17.5 = 175'-0"



Dimensions of truss members given above are nominal. To give the camber to truss top chord shall be lengthened ¼" at end panel and 3/8" at center panel and other members proportioned to this added lengths. truss camber assumed 2¾" at center of span during erection.

Assumed dead Load

Floor slab. and pavement	93 · 21	=	1953
Copings	2@ 120	=	240
Handrails	2@ 60	=	120
			2313# per lin. ft.

Dead Load metal

Stringers	206#
Intermediate FB.	151
Lower Laterals.	52
upper Laterals	160.
trusses assumed.	920
	1489 ÷ 2 = 745# per lin ft.

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CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Dead Load metal for one truss 745#
" " floor 2313 ÷ 2 = 1157
1902# per lin ft.

Panel concentration = 1902 · 17.5 = 33300# for both top and bottom

For top chord panel point load
upper panel 160 ÷ 2 = 80
truss upper half. 230
310# · 17.5 = say 5400#

Dead Load stresses
Reaction = 33300 · 4.5 = 149850
Chord stresses
U_0-U_1 149850 · 17.5 ÷ 23.60 = 111000# C
U_1-U_2 ÷ 23.88 = 109800# C

L_1-L_2-L_3 149850 · 2 · 17.5 = 5245,000
33300 · 17.5 = 583,000
4662,000 ÷ 27.68 = 168,500# T

U_2-U_3-U_4
149850 · 3 · 17.5 = 7867,000
33300 · 3 · 17.5 = -1750,000
6117,000 ÷ 29.55 = 207,000# C U_2-U_3
÷ 29.80 = 205500 C U_3-U_4

L_3-L_4-L_5
149850 · 4 · 17.5 = 10490,000
33300 · 6 · 17.5 = 3500,000
6990000 ÷ 31.52 = 222,000# T

U_4-U_5
149850 · 5 · 17.5 = 13,112,000
33300 · 10 · 17.5 = 5827,000
7,285,000 ÷ 31.99 = 228,000# C

Web stresses.
U_0-L_1. 149850 · 81.1 ÷ 74.25 = 163500# T

L_1-U_2 149850 · 109.1 = 16,36.0000
33300 · 126.6 = 4215000
12145000 ÷ 107.1 = 113,300# C

U_2-L_3.
149850 · 166.83 = 25000,000
33300 · 386.16 = 12860,000
12140000 ÷ 186.8 = 65100# T

L_3-U_4
149850 · 313.05 = 46,850,000
33300 · 1044.15 = 34,800,000
12050000 ÷ 318.7 = 37800# C

U_4-L_5
149850 · 1079.2 = 161,600,000
33300 · 4473.8 = 149,000,000
12600000 ÷ 1020 = 12300# T

End post. DL. shear 149850
5400 Top chord panel concentration.
155250# C

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken.

Live Load on truss

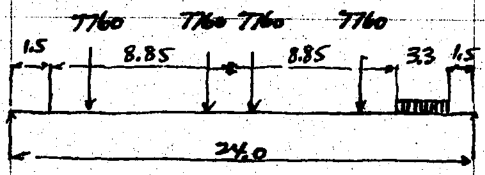
Uniform Load $w = \frac{100.000}{170+533} = 429^{kg}/m^2 = 88^{\#}/\square'$

Coefficient of impact $= \frac{20}{60+533} =$ say 17.6%.

Motor truck rear wheel concentration 6600#
17.6% impact 1160
7760#

Front wheel Conc = $\frac{1}{3} \cdot 7760 = 2590^{\#}$

Reaction on truss.



Motor truck loading

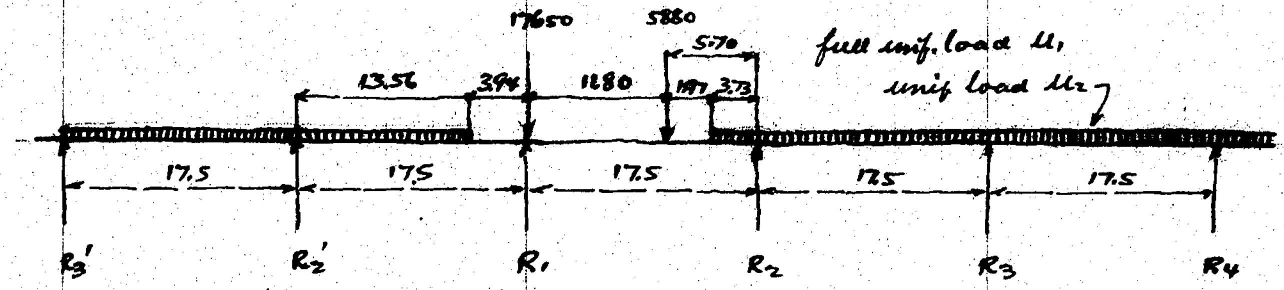
Rear wheel $.4 @ 7760 \cdot \frac{13.65}{24.} = 17650^{\#}$

Front wheel $\frac{1}{3} \cdot 17650 = 5883^{\#}$ call this 5880#

Uniform Load at front and rear of motor truck) 2@8.85 = 17.70' wide
17.70 · 88 = 1560#
Reaction on truss = $1560 \cdot \frac{13.65}{24.0} =$ 885# per lin ft of truss

Uniform Live Load on side of motor truck 3.3' wide
3.3 · 88 = 290#
Reaction on truss = $290 \cdot \frac{3.15}{24.0} =$ 38# per lin. ft. of truss.

u_1 38 · 17.5 = 665#
u_2 885 · 17.5 = 15500#
" 885 · 13.56 = 12000#
" 885 · 3.73 = 3300#



for chord members

R_3': 665 / 15500 / 16165

R_2': 665 / 7750 / 7350 / 15765

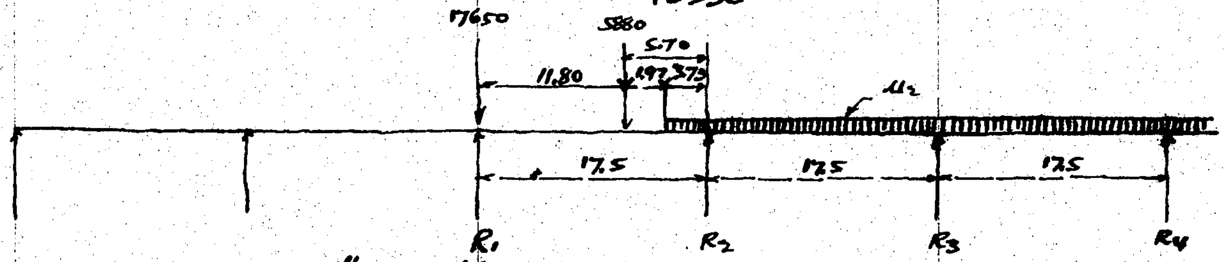
R_1: 665 / 4650 / truck → 17650 / truck → 1915 / 350 / 25230

R_2: 665 / 3965 / 2950 / 7750 / 15330

R_3: 665 / 15500 / 16165#

R_4: 665 / 15500 / 16165#

7350 - - - - - 4650
1915 - - - - - 3965
350 - - - - - 2950



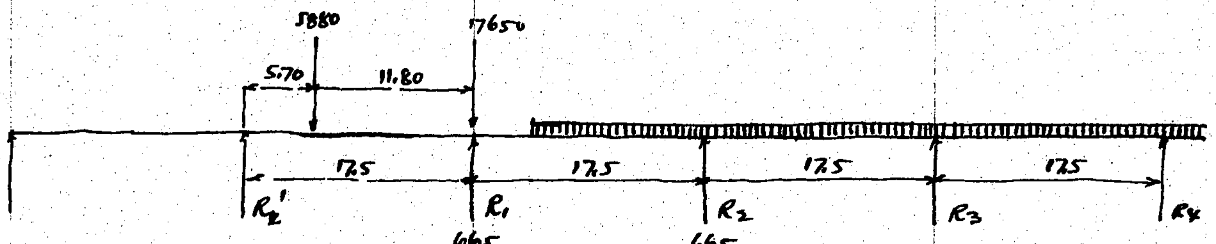
for web members

R_1: u_1 665 / T 17650 / T. 1915 / u_2 350 / 20580

R_2: 15330

R_3: 161650

R_4: 16165



for web. member.

R_2': 3965 - - - - - 1915

R_1: 665 / 1915 / 17650 / 4650 / 24880

R_2: 665 / 7350 / 7750 / 15765

R_3: 16165

R_4: 16165

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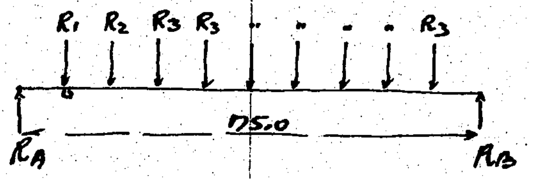
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Live Load Stresses
Chord Stresses.

U_0-U_1
U_1-U_2

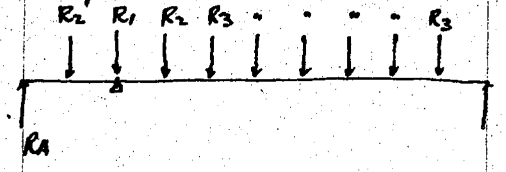


R_A.
25230 · 157.5 = 3980.000
15330 · 140.0 = 2145.000
16165 · 490.0 = 7920.000
14045.000

R_A = 14045.000 ÷ 175 = 80200#

U_0-U_1 80200 · 17.5 ÷ 23.60 = 59600# C
U_1-U_2 ÷ 23.88 = 58900# C

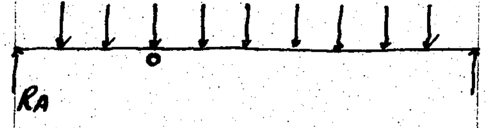
L_1-L_2-L_3



R_A
15765 · 157.5 = 2480.000
25230 · 140.0 = 3530.000
15330 · 122.5 = 1875.000
16165 · 367.5 = 5930.000
13,815.000 ÷ 175 = 79000#

79000 · 2 · 17.5 = 2765.000
15765 · 17.5 = 276.000
2489.000 ÷ 27.68 = 90.000# T

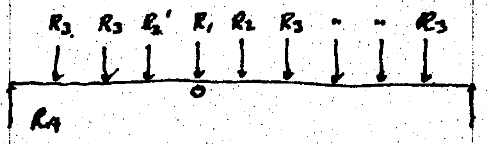
U_2-U_3
U_3-U_4



R_A
16165 · 157.5 = 2.545.000
15765 · 140.0 = 2208.000
25230 · 122.5 = 3090.000
15330 · 105.0 = 1610.000
16165 · 262.5 = 4240.000
13,693.000 ÷ 175 = 78200#

78200 · 3 · 17.5 = 4,110.000
15765 · 17.5 = 276.000
16165 · 35.0 = 565.000
841.000
3,269.000 ÷ 29.55 = 110800# C U_2-U_3
÷ 29.80 = 109900# C U_3-U_4

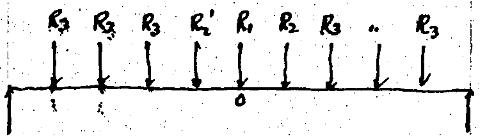
L_3-L_4-L_5



R_A
16165 · 472.5 = 7.640.000
15765 · 122.5 = 1.930.000
25230 · 105.0 = 2.650.000
15330 · 87.5 = 1341.000
13,561.000 ÷ 175 = 77400#

77400 · 4 · 17.5 = 5,420.000
15765 · 17.5 = 276.000
16165 · 5 · 17.5 = 1415.000
1691.000
3,729.000 ÷ 31.52 = 118400# T

U_4-U_5



R_A
15765 · 105.0 = 1655.000
25230 · 87.5 = 2210.000
15330 · 70.0 = 1072.000
4937.000 ÷ 175 = 28200
3@16165 = 48500
76700

76700 · 5 · 17.5 = 6700.000
15765 · 17.5 = 276.000
16165 · 9 · 17.5 = 2540.000
2,816.000
3,884.000 ÷ 31.99 = 121500# C

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

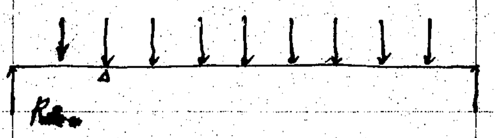
Design of Kasumi-Bashi for Okayama-Ken

Live Load web stresses.

U_0-L_1 RA = 25230 · 157.5 = 3.980.000
15765 · 140.0 = 2205.000
16165 · 490.0 = 7920.000
14105.000 ÷ 175 = 80600#

80600 · 81.1 ÷ 74.25 = 88100# T

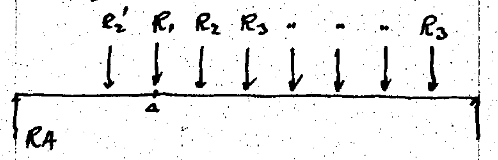
L_1-U_2



RA 3965 · 157.5 = 625.000
24880 · 140.0 = 3480.000
15765 · 122.5 = 1930.000
16165 · 367.5 = 5930.000
11965.000 ÷ 175 = 68200#

68200 · 109.1 = 7.450.000
3965 · 126.6 = 501.000
6949.000 ÷ 107.1 = 64700#

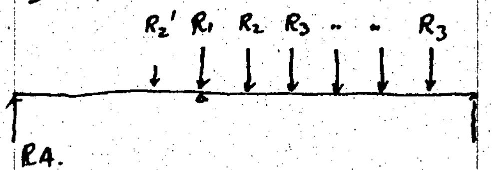
U_2-L_3



RA 3965 · 140.0 = 555000
24880 · 122.5 = 3050000
15765 · 105.0 = 1655000
16165 · 262.5 = 4240.000
9500.000 ÷ 175 = 54250#

54250 · 166.83 = 9.050.000
3965 · 201.83 = −797500
8252500 ÷ 186.8 = 44200#

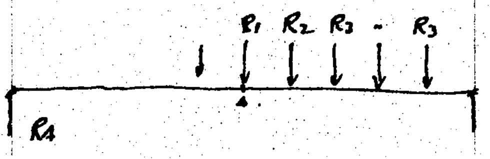
L_3-U_4



RA 3965 · 122.5 = 485.000
24880 · 105.0 = 2612.000
15765 · 87.5 = 1380.000
16165 · 175.0 = 2830.000
7307.000 ÷ 175 = 41750

41750 · 313.05 = 13,080,000
3965 · 365.55 = 1447.000
11,633,000 ÷ 318.7 = 36500#

U_4-L_5



RA 3965 · 105.0 = 416.000
24880 · 87.5 = 2172.000
15765 · 70.0 = 1102.000
16165 · 105.0 = 1696.000
5386.000 ÷ 175 = 30700#

30700 · 1079.2 = 33200.000
3965 · 1149.2 = 4550.000
28,650,000 ÷ 1020 = 28100#

Stress in End Post 80600# C

L_5-U_4' RA 3965 · 87.5 = 347.000
24880 · 70.0 = 1740.000
15765 · 52.5 = 827.000
16165 · 52.5 = 848.000
3762000 ÷ 175 = 21400#

21400 · 1254.2 = 26840,000
3965 · 1166.7 = 4625.000
22215.000 ÷ 1020 = 21800# C

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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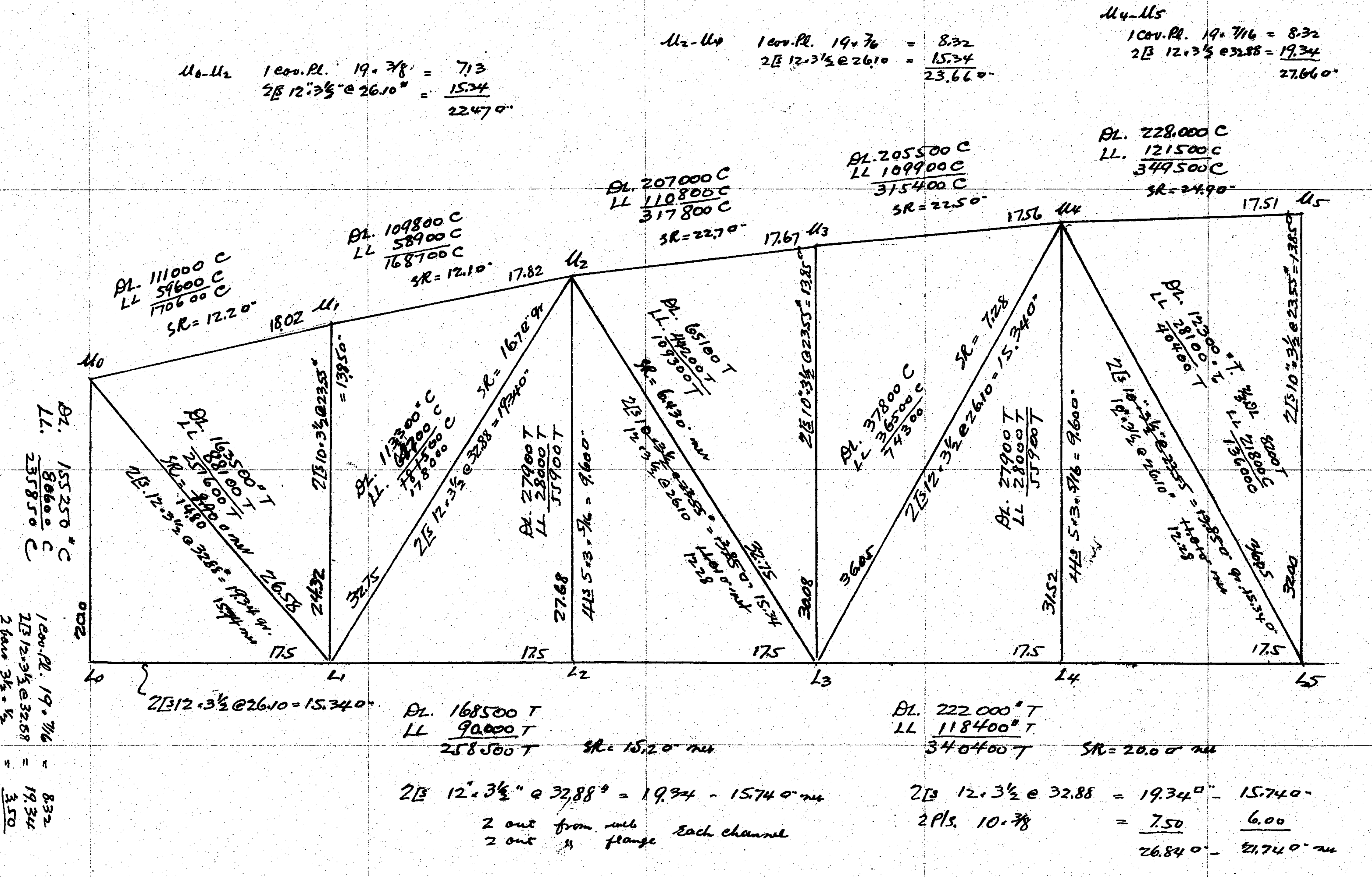
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken.

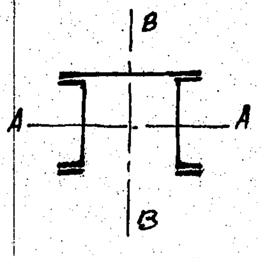
Stresses and sections of truss.



Design of Kasumi Bashi for Okayama-Ken.

Section of End Post U_0-L_0.

Assumed Section



Moment of inertia about AA Axis

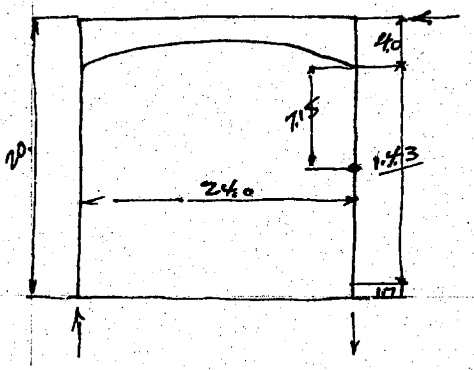
1 Cov. Pl. 19"·7/16"	= 8.32	· 12.22	= 101.7	8.32 · 5.27²	= 231
2[s 12"·3½" @ 32.88#	= 19.34	· 6.0	= 116.0	19.34 · 0.95² + 380	= 397
2 Pls 3½"·½	= 3.50	· -0.25	= -.8	3.50 · 6.70² +	= 157
	31.16	6.95	216.9		785

Eccentricity = 6.95 - 6.0 = 0.95" $r = \sqrt{\frac{785}{31.16}} = 5.02$

Moment of inertia about BB axis

1 Cov. Pl. 19·7/16	= 8.32			250.0
2[s 12" @ 32.88	= 19.34	· 6.8²	+ 18	= 912.0
2 Pls 3½"·½"	= 3.50	· 7.68²	+ 35	= 210.0
	31.16			1372.0

$r = \sqrt{\frac{1372}{31.16}} = 6.63$



Wind load $\frac{200}{2}$ · 175 = 17500
Extra load due to wind = 17500 · $\frac{20}{24}$ = 14600
Direct dead + live load. 235850
250450#

Bending moment due to wind load
Point of contraflexure assumed 11.15' from top or
moment = 8750 · 7.15 = 62500'#
· 750000"#

Fibre stress due to wind load = $\frac{750.000 \cdot 9.5}{1372}$ = 5200#/□"

Bending stress due to Eccentricity of Connection = 250450 · 0.95 = 238000"#

Fibre stress due to Eccentricity = $\frac{238000 \cdot 5.48}{785}$ = 1660#/□"

Unit stress due to direct stress = 250450 ÷ 31.16 = 8050#/□"
Due to wind load 5200
Due to eccentricity 1660
14910#/□".

Diagonal L_1-U_2 178000# C
2[s 12·3½ @ 32.88# = 19.34□" r = 4.44 Unsupported length = 32.75 · 12 = 393"
p = 21300 (1 - 0.0055 l/r) = 10660#/□" l/r = 393 ÷ 4.44 = 88.5
section required 178000 ÷ 10660 = 16.7□" gross.
assumed section ok.

Diagonal L_3-U_4 74300
Try 2[s 12·3½ @ 26.10# = 15.34□" r = 4.55 unsupported length = 432"
l/r = 432 ÷ 4.55 = 94.8
p = 21300 (1 - 0.0055 l/r) = 10200#/□"
Section required 74300 ÷ 10200 = 7.28□" gross.
assumed section ok

Diagonal U_4-L_5
40400 T Use 2[s 12·3½ @ 26.10# = 15.34□"
13600 ÷ 2 = 6800 l/r = 94.8
47200#

Verticals U_5-L_5 Unsupported length 32 · 12 = 384"
Use 2[s 10·3½ @ 23.55# = 13.84□" Single lacing both faces.

Hangers U_2-L_2.
Dead Load panel load 33300 - 5400 = 27900# see p11.
Live Load 28000
55900

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken.

Hangers U_2-L_2 4Ls 5×3×5/16 = 9.60 ○" gross or 7.420" net.

Unit stress = 55900 ÷ 7.42 = 7540 #/○"

Approximate weight of truss

		Section			
Top chord	U_0-U_1	22.47 ○" @ 3.4 ×	18.02	=	1379.
"	U_1-U_2	22.47	17.82	=	1364
"	U_2-U_3	23.66	17.67	=	1420
"	U_3-U_4	23.66	17.56	=	1415
"	U_4-U_5	27.66	17.51	=	1650
Bottom chord.	L_0-L_1	15.34	17.50	=	915
"	L_1-L_2-L_3	19.34	35.00	=	2310
"	L_3-L_4-L_5	26.84	35.00	=	3200
End post	U_0-L_0	31.16	20.00	=	2120
verticals	U_1-L_1	13.85	24.32	=	1140
"	U_3-L_3	13.85	30.00	=	1410
"	½ U_5-L_5	13.85	32.00	=	1505 ÷ 2
Hangers	U_2-L_2	9.6	27.68	=	905
"	U_4-L_4	9.6	31.52	=	1030
Diagonals	U_0-L_1	19.34	26.58	=	1750
"	L_1-U_2	19.34	32.75	=	2160
"	U_2-L_3	15.34	32.75	=	1705
"	L_3-U_4	15.34	36.05	=	1880
"	U_4-L_5	15.34	36.05	=	1880

31138 #　　31138
752
30386 #

For one truss 30386 × 2 = 60772
Details say 37% 22500
83272 #

83272 ÷ 175 = 477 # per lin ft.

Approximate metal in one span

Stringers	206 × 175	=	36000
Intermediate F.B.	9 @ 2640 #	=	23760
End Floor Beam	2 @ 2225 #	=	4450
Lower Lateral Bracing		=	9100
Upper Lateral Bracing.		=	27920
trusses		=	83270
		=	83270

267770 #
weight of shoes assumed 6230
274000 # = 122 tons

Load on pin.

weight of metal 267770
flooring. 2313 × 177.5 411000
678770 ÷ 4 = ~~144,690~~ # 169600 #

Live Load reaction. (see p12).

motor truck Rear wheel on ℄ bearing. (case 3).

Panel load Unif. say 400
Rear wheel 17650
Unif. load 4650
22700 #

panel concentration say 16165 #
16165 × 4.5 = 72800
22700
95500 #

DL. Reaction 169600
LL " 95500
265100 # Design pin for 270.000 # stress.

Design of Kasumi-Bashi for Okayama-Ken

Assume diameter of pin $5\frac{1}{4}$" Unit bearing = $\frac{135.000}{1.62 \times 5.25}$ = 15900 #/□ ok.

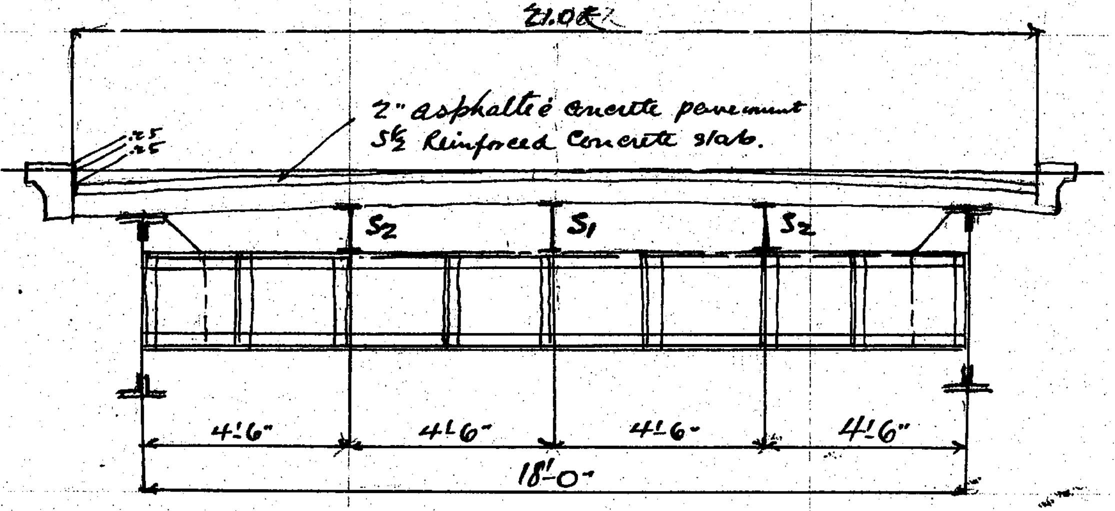
Bending moment of pin = 135000 × 1.87" = 252500 "#
fibre stress = $\frac{252500 \times 2.62"}{0.0049 \times 5.25^4}$ = 17700 #/□ ok

Shoes Load on shoe say 270.000#
diameter of roller 4" spacing $4\frac{1}{4}$" c to c of rollers
610 × 4 = 2440# per lin. inch. 270.000 ÷ 2440 = 110.8"
use 5 rollers 22" each 110.0 inches.

Size of shoe for Expansion Ends 24" × 33" = 792□"
Unit bearing 270.000 ÷ 792 = 341 #/□ ok

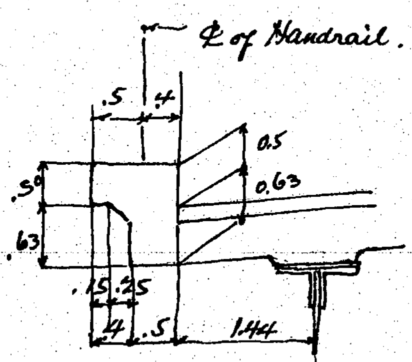
For fixed shoes. 24" × $23\frac{1}{2}$ = 564□
Unit bearing 270.000 ÷ 564 = 480 #/□ ok.

Design of Plate girder span 60'-0" out to out.



Design of Floor slab. span length 4'-6" between main girders
1'-6" over hang beyond main girder.

Overhanging portion of slab.



moment about ℄ main girder

slab and pavement	93# × 1.44 =	134	× 0.72	=	96.5
coping	0.5 × 1.13 @ 150 =	85	× 1.69	=	144.0
"	0.03 @ 150 =	5	× 2.02	=	10.2
"	0.20 @ 150 =	30	× 2.14	=	64.2
Handrail		60	× 1.84	=	110.4
		314#	1.36		1425.3

motor truck loading rear wheel 8580#
Front wheel. 2860#

Live Load distribution of rear wheel concentration assumed 2.0' wide
8580 ÷ 2 = 4290# per ft.

wheel concentration from curb line ~~assumed~~ 1.0'
Moment about ℄ of ~~girder~~ = 4290 × 0.44 = 1930
D.L.m = 425
2355 '#

Effective depth required for fs = 17000 #/□ and fc = 640 #/□
Effective depth = $\sqrt{\frac{2355}{102}}$ = 4.8" make slab depth 6" at ℄ of girder

Reinf. bars = $\frac{2355 \times 12}{7/8 \times 4.8 \times 17000}$ = 0.394□" 2-$\frac{1}{2}$"φ bars = 0.390□" per ft. strip

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken.

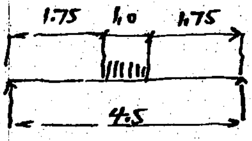
Floor slab between main girder Span length 4'-6" center to center of stringers

Dead Load 2" Asphaltic concrete 24#
5½" slab. <u>69#</u>
93# per square ft of Roadway

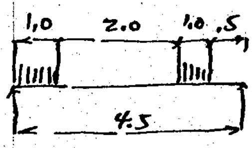
Dead Load moment $= \frac{1}{10} \cdot 93 \cdot 4.5^2 = 188$ '#
Dead Load shear $= 93 \cdot \frac{4.5}{2} = 209$ #

Live Load motor truck rear wheel 6600 Front wheel $\frac{1}{3} \cdot 8580 = 2860$ #
Impact 30% <u>1980</u>
8580

Transverse distribution b = 1.23
Longitudinal distribution a = 1.00



Effective width $\Sigma = \frac{2}{3}(l+b)+a$ where l = span length
$\Sigma = \frac{2}{3}(4.5+1.23)+1.00 = 4.82'$
Load per ft strip $8580 \div 4.82 = 1780$ #
Arbitrary transverse distribution assumed 1.0



moment due to Single wheel load $890 \cdot 2.25 = 2000$
less moment $890 \cdot 2.5 = \underline{222}$
1878 '#

For continuity of slab. take moment $0.8 \cdot 1878 = 1500$ '#
max End shear $= 1780 \cdot \frac{1.0}{4.5} = 396$
$1780 \cdot \frac{4.0}{4.5} = \underline{1580}$
1976 #

Summary for moments and shears

	moment	shear
Dead Load	188	209
Live Load	<u>1878</u>	<u>1976</u>
	2066 '#	2185 #

Effective depth of slab req'd $= \sqrt{\frac{2066}{102}} = 4.5"$

make slab 5½" with 1" insulation.

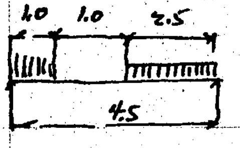
Steel Area $= \frac{2066 \cdot 12}{\frac{7}{8} \cdot 4.5 \cdot 17000} = .37 \square"$ Use ½" bars 6" ctrs
$= 0.39 \square"$ per ft.

Shear $= \frac{2185}{\frac{7}{8} \cdot 4.5 \cdot 12} = 46.2$ #/□"

Bond stress $= \frac{2185}{\frac{7}{8} \cdot 4.5} = 555$ # per ft strip 2- ½"φ @ 1.57 · 130# = 409
with lapping <u>409</u>
818 # ok.

Bent up bars shall be lapped over center stringer

Shear over stringer S2



Uniform load 250# Shear $= 250 \cdot \frac{1.25}{2.25} = 139$ #
motor truck loading $1780 \cdot \frac{4.0}{4.5} = \underline{1580}$
1719 #
Dead Load shear <u>209</u>
1928 #

Bond stress $u = \frac{1928}{\frac{7}{8} \cdot 4.5} = 488$ # per ft 2- ½"φ @ 1.57 · 130 = 409 #
add extra reinforcement for bond over stringer.

Design of Longitudinal stringer

Stringer S1 stringer spacing 4'-6" Span length 11.5'
Slab and pavement $93 \cdot 4.5 = 420$
stringer say <u>35</u>
455 #

Dead Load moment $= \frac{1}{8} \cdot 455 \cdot 11.5^2 = 7520$ '#
Dead Load Shear $= \frac{1}{2} \cdot 455 \cdot 11.5 = 2620$ #

Live Load motor truck loading Rear wheel conc. with impact = 8580 #
Front " " " " = 2860 #

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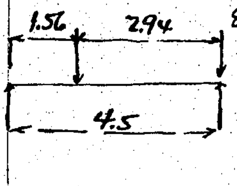
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama Ken.

Load on stringer



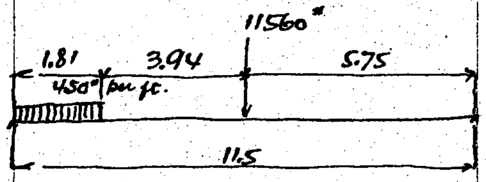
Rear wheel Reaction $8580 \cdot \frac{1.56}{4.5} = 2980$

$\underline{8580}$

11560 #

Uniform live load $100 \cdot 4.5 = 450$ # per ft of stringer

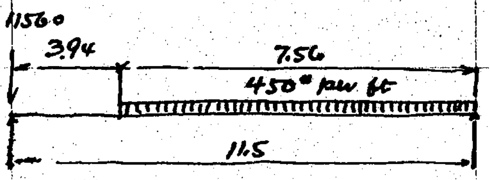
$450 \cdot 1.81 = 815$ # $R = 815 \cdot \frac{.91}{11.5} = 64.5$ #



moment due to truck $\frac{11560}{2} \cdot 5.75 = 34400$

" " · Unif. load $64.5 \cdot 5.75 = \underline{3710}$

38110 #



Max End shear

Uniform load $450 \cdot 7.56 = 3400$ #

$R = 3400 \cdot \frac{3.78}{11.5} = 1120$

motor truck loading $= \underline{11560}$

12680 #

Summary for moments and shears

	moment	shear
Dead Load	7520	2620
Live load	38110	12680
	45630 #	15300 #

Section modulus required

$\frac{45630 \cdot 12}{17000} = 32.3$

Use 12" · 5" @ 31.99 #   Sm = 36.69

Stringer S_2 Use 12" · 5" @ 31.99 #   Same as for S_1.

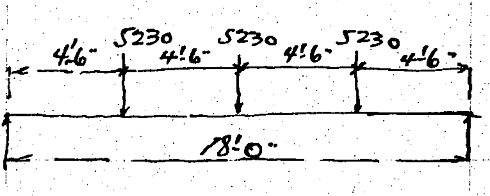
Design of Floor Beam

Intermediate floor beam   span length 18'-0"   spacing 11'-6"

Dead Load   stringer 455 # per lin ft.

panel Concentration $455 \cdot 11.5 = 5230$ #

Reaction $5230 \cdot 1.5 = 7845$



moment $= 7845 \cdot 9.0 = 70500$

$\overset{23}{5\cancel{32}0} \cdot 4.5 = -\underline{23\overset{5}{\cancel{9}}00}$

$\overset{70}{4\cancel{66}00}$ #

Dl. Floor Beam $\frac{1}{8} \cdot 100 \cdot 18^2 = \underline{4050}$

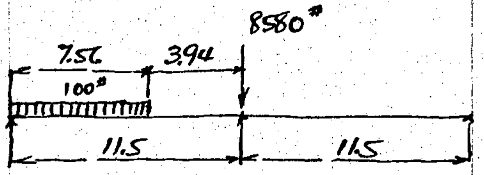
51050 #
~~42550~~

Dl. shear $7845 + 900 = 8745$ #

Live load on floor beam

motor truck rear wheel - 8580

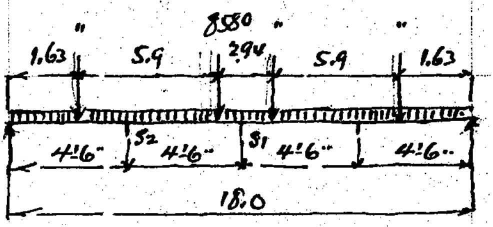
$756\text{\#} \cdot \frac{3.78}{11.50} = 248$ # per ft.



Load on stringer S_1 $8580 \cdot \frac{3.03}{4.50} = 5780$

$\underline{5780}$

11560 #



Load on stringer S_2 $8580 - 5780 = 2800$

$8580 \cdot \frac{1.63}{4.5} = \underline{\overset{3120}{\cancel{1350}}}$

5920
~~4150~~ #

Reaction = ~~9930~~ # 11680 #

Uniform load on stringer $248 \cdot 4.5 = 1115.0$

Reaction $= 1115 \cdot 1.5 = 1673$ #

moment due to motor truck $= \overset{11680}{\cancel{9930}} \cdot 9.0 = ~~89400~~$ 105000

$\overset{5920}{\cancel{4150}} \cdot 4.5 = ~~18700~~$ $\underline{26600}$

78300
~~70700~~

moment due to unif. load $= 1673 \cdot 9.0 = 15100$

$1115 \cdot 4.5 = \underline{5020}$

$\underline{10080}$

~~80780~~ #
88380

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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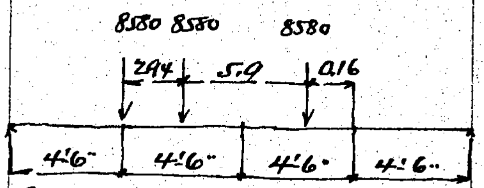
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Intermediate Floor Beam

Max End shear



Loading assumed as shown on diagram

$8580 \cdot \frac{2.872}{18.0} =$ 13700
Unif load say 1673
15373" call this 16000"

Summary for moments and shears

	moment	shear
Dead Load	51050	8745
Live Load	88380	16000
	139430"	24745"

Try 24 · 5/16 = 7.50□" 1/8 web = 0.94
Effective depth = 2.04 − 0.11 = 1.93
flange stress = 139430 ÷ 1.93 = 72200"
SR = 72200 ÷ 17000 = 4.24 − 0.94 = 3.30□" net

Use 2Ls 3.3. 3/8 = 4.22□" gross or 3.56□" net

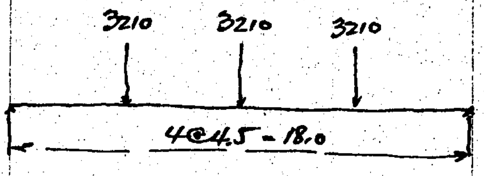
Approximate weight of Intermediate Floor Beam.

web	1Pl.	24 · 5/16	@	25.50	·	18.0	=	459
flanges	4Ls	3. 3. 3/8	@	7.2	·	18.0	=	520
End conn	4Ls	3. 3. 3/8	@	7.2	·	~~1.81~~ 2.00	=	58
fillers	4Pls.	3. 3/8	@	3.83	·	~~1.17~~ 1.34	=	21
Stiffs	6Ls	3. 2½. 5/16	@	5.6	·	2.00	=	67
		Rivet heads and variation say						40

1165" ÷ 18 = 65" per ft

End Floor Beam

Dead Load



Load on floor beam at stringer Connection.

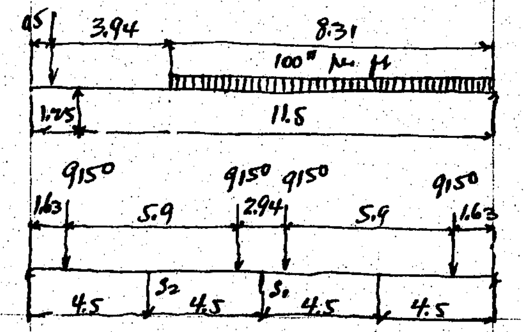
$455 \cdot \frac{12.75^2}{2 \cdot 11.5} =$ 3210"

Reaction = 3210 · 1.5 = 4815"
moment at center 4815 · 9.0 = 43400
3210 · 4.5 = 14450
28950'"

Dead Load beam $\frac{1}{8} \cdot 80 \cdot 18^2 =$ 3240
32190'"

Live Load motor truck rear wheel 8580"

motor truck loading $8580 \cdot \frac{12.25}{11.50} =$ 9150"



Unif. load $831 \cdot \frac{4.15}{11.50} =$ 300"
Concentration on stringer 300 · 4.5 = 1350"

Load on stringer S1 $9150 \cdot \frac{3.03}{4.50} =$ 6150
6150
12300"

Load on stringer S2 9150 − 6150 = 3000
$9150 \cdot \frac{1.63}{4.5} =$ 3320
6320"

Reaction = 12470"

Moment due to motor truck = 12470 · 9.0 = 112000
6320 · 4.5 = − 28450
83550'"

Moment due to unif. load = 1350 · 9 = 12130
95680'"

Dead Load moment = 32190
127870'"

Try 24 · 5/16 = 7.50□" 1/8 web = 0.94□" Effective depth = 1.93
flange stress = 127870 ÷ 1.93 = 66200" SR = 66200 ÷ 17000 = 3.90 − .94 = 2.96□"
Use 2Ls 3.3. 5/16 = 3.56□" gross or 3.02□" net

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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CALCULATIONS FOR

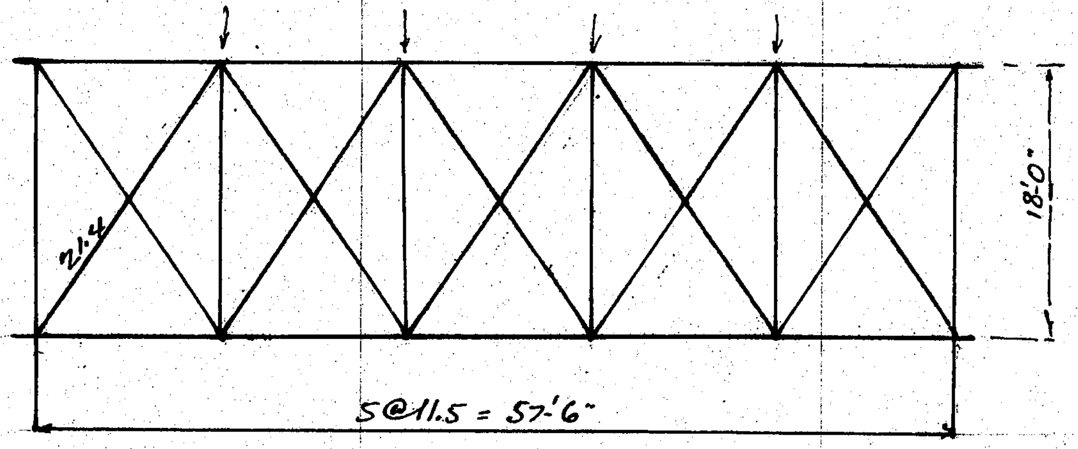
Design of Kasumi-Bashi for Tokushima-Ken

Approximate weight of End floor beam

web	1Pl. 24 × 5/16	@	25.50	× 18.0	=	459
flanges	4Ls 3×3×5/16	@	6.1	× 18.0	=	440
End conn	4Ls 3×3×3/8	@	7.2	× 2.0	=	58
fillers	4Pls. 3×5/16	@	3.19	× 1.5	=	19
Stiffeners	6Ls 3×2½×5/16	@	5.6	× 2.0	=	67
	Rivet heads + variation					40

1083# ÷ 18 = 60# per lin. ft.

Lateral Bracing.
wind pressure assumed 30#/□'. Exposed area say 8' or 240# per lin. ft.



Panel Concentration = 240 × 11.5 = 2760# sec θ = $\frac{21.4}{18.0}$ = 1.19
Reaction at end = 2760 × 2 = 4520#
Stress in End panel 4520 × 1.19 = 5370# Use 6 Rivets for connection.
Approximate weight of lower lateral Bracings.

2Ls 4×3×5/16 0.3" @ 684 × 20.0 = 274#
× 19.0 = 260
Center connection say 40
misc connection say 50
624# × 5 = 3120#

3120 ÷ 60 = 52# per lin ft of girder.

Design of main girder.

Dead Load
Floor slab and pavement 93 × 21 = 1953
Copings 2@ 120 = 240
Handrails 2@ 60 = 120
2313# per lin. ft

Dead Load metal
Stringers 3@ 34# = 102 @ 11.5 = say 1170
Intermediate floor beam 1165
Lateral Bracing 624
2959 say 2960#
Floor 2313 × 11.5 = 26600
29560

Panel Concentration. For one girder 29560 ÷ 2 = 14780#

Dead Load metal End panel.
Stringers 3@ 34# = 102 @ ~~12.75~~ 7.0 = 714#
End floor beam 1083
Lateral Bracing 312
2109 say 2110
Floor 2313 × 7.0 = 16200
18310

Panel Concentration For one girder 18310 ÷ 2 = 9155#

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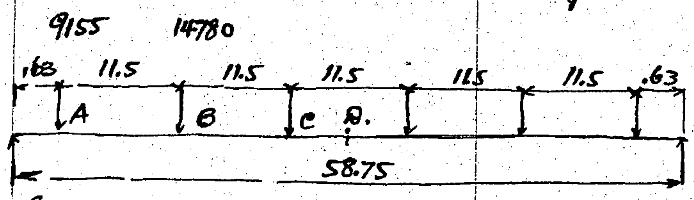
CALCULATIONS FOR

Design of Kasumi-Bashi for Tokushima-Ken

Dead Load moment due to panel concentration.



Reaction = 38715#
Girder 280 • 30 = 8400
47115#

moment at A = 38715 • 0.63 = 24400'#

Moment at B. 38715 × 12.13 = 470,000
9155 • 11.5 = 105.200
364800'#

Moment at C 38715 • 23.63 = 915000
91.55 • 23.0 = 210500
14780 • 11.5 = 170000
380500
534500'#

Dead Load of main girder assumed 280# per lin ft.

moment at A. $\frac{1}{2}$ • 280 • 0.63 • 58.12 = 5130'#

moment at B $\frac{1}{2}$ • 280 • 12.13 • 46.62 = 79300'#

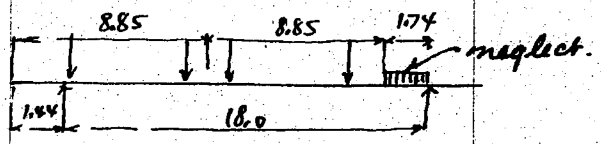
moment at C $\frac{1}{2}$ • 280 • 23.63 • 35.12 = 116300'#

moment at center $\frac{1}{8}$ • 280 • 58.75^2 = 121000'#

Summary for dead load moments

	A	B	C	center
Due to Concentration	24400	364800	534500	534500
Due to wt of girder	5130	79300	116300	121000
	29530'#	444100'#	650800'#	655500'#

Live Load motor truck loading Impact = $\frac{20}{60+l}$ = $\frac{20}{77.9}$ = 25.7%



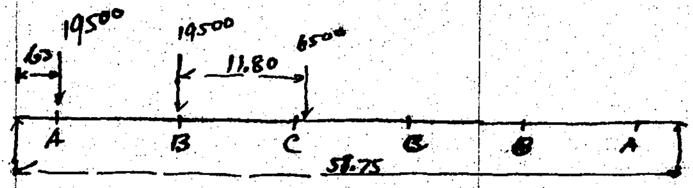
Rear wheel 6600
Impact 25.7% 1695
8295#

Front wheel $\frac{1}{3}$ • 8295 = 2760

Uniform load 174# • $\frac{.87}{18.0}$ = 8.4# neglect this uniform live load on side of motor truck.

Front wheel. $\frac{1}{3}$ • 19500 = 6500
Rear wheel 4 • 8295 • $\frac{10.59}{18.0}$ = 19500

Moment due to motor truck loading.



Moment at A.
R = 19500 • $\frac{58.11}{58.75}$ = 19300

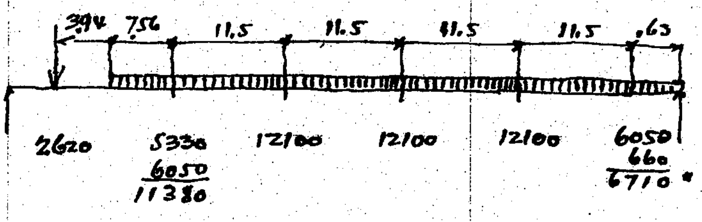
m = 19300 • 0.63 = 12170'#

moment at B R 19500 • 46.62 = 908000
6500 • 34.82 = 226500
1134500 ÷ 5875 = 19300
m = 19300 • 12.13 = 234000'#

moment at C R 19500 • 35.12 = 685000
6500 • 23.32 = 151500
836500 ÷ 5875 = 14250
m = 14250 • 23.63 = 337.000'#

Moment due to uniform load
Uniform load at the front and rear of motor truck 19.44' wide
1944# • $\frac{9.72}{18}$ = 1050# per lin ft.

Moment at A



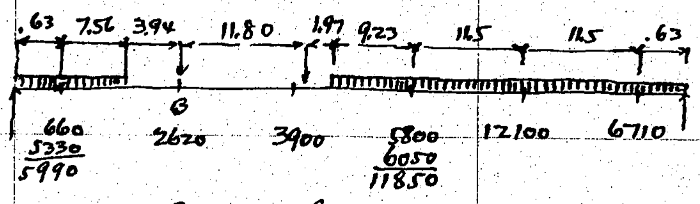
7.56 • $\frac{3.78}{11.5}$ = 2.49 @ 1050 = 2620
7950 - 2620 = 5330
0.63 • 1050 = 660#
11.5 @ 1050 = 12100

Design of Kasumi-Bashi for Okayama-Ken.

moment at A.

R 2620 · 58.12 = 152,500
11380 · 46.62 = 530.000
12100 · 70.86 = 857.000
6710 · 0.63 = 4200
1543700 ÷ 58.75 = 26300

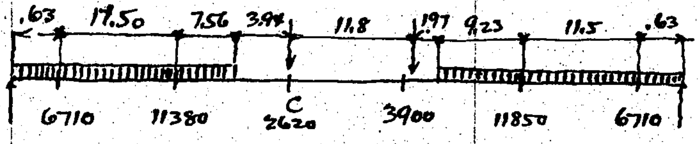
moment = 26300 · 0.63 = 16550 #



9.23 · 1050 = 9700 9700 · $\frac{4.62}{11.50}$ = 3900
9700 - 3900 = 5800 #

moment at B. R 5990 · 58.12 = 348.000
2620 · 46.62 = 122.000
3900 · 35.12 = 137.000
11850 · 23.62 = 280.000
12100 · 12.12 = 147.000
6710 · 0.63 = 4200
1038200 ÷ 58.75 = 17650 #

moment B 17650 · 12.12 = 214.000
5990 · 11.50 = 68900
145,100 #



moment at C R 11380 · 46.62 = 530.000
2620 · 35.12 = 92.000
3900 · 23.62 = 92100
11850 · 12.12 = 143800
757900 ÷ 58.75 = 12900
.6710
19610 #

moment C 19610 · 23.62 = 464000
11380 · 11.5 = 130800
6710 · 23.0 = 154400
285200
178800 #

Summary for live load

	A	B	C
Concentrated motor truck	12170	234000	337000
Uniform load	16550	145100	178800
	28720	379100	575800

Summary for dead and live load moments

	A	B	C	center
Dead Load	29530	444100	650800	655500
Live Load	28720	379100	515800	515800
	58250	823200	1166600	1171300 #

End shear

Dead Load shear	47115
Live Load motor truck	19300
" " Unif. load	26300
	92715 #

Section of main girder moment = 1171300 # shear = 92715 #

CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

web assumed 48"· 3/8" = 18.0 □" 1/8 web = 2.250□" depth = 4'-0½" b to b to L^s
Effective depth = 3.88' flange stress = 1171300 ÷ 3.88 = 302.000#
Section required = 302.000 ÷ 17000 = 17.80
− 2.25
15.55□" net

Use 2L^s 6·6·½ = 11.50□" gr. 9.50□"
1 Pl. 14·½ = 7.00 6.00
18.50 gr 15.50□" net.

Approximate weight of main girder

web.	1 Pl. 48"· 3/8"	@ 61.20	· 60.0	=	3570
flanges.	4L^s 6·6·½"	@ 19.60	· 60.0	=	4710
Cov. pls.	2Pl^s 14·½	@ 23.80	· 38.0	=	1810
End stiffs	8L^s 5·3½·½	@ 13.60	· 3.96	=	432
fills	8Pl^s 3½·½	@ 5.95	· 3.00	=	143
Int. stiffs	28L^s 5·3½·3/8	@ 10.40	· 4.04	=	1178
fillers.	4Pl^s 3½·½	@ 5.95	· 3.00	=	72
web splice	12Pl^s 9·½	@ 15.3	· 3.5	=	642
"	4Pl^s 12½·3/8	@ 15.94	· 1.5	=	95
"	2Pl^s 12½·½	@ 21.25	· 1.5	=	64
flange splice	4L^s 6·6·3/8	@ 14.9	· 4.0	=	238
Sole pls	2Pl^s 15"·1"	@ 51.0	· 1.75	=	178
	Rivet heads + variation say				660

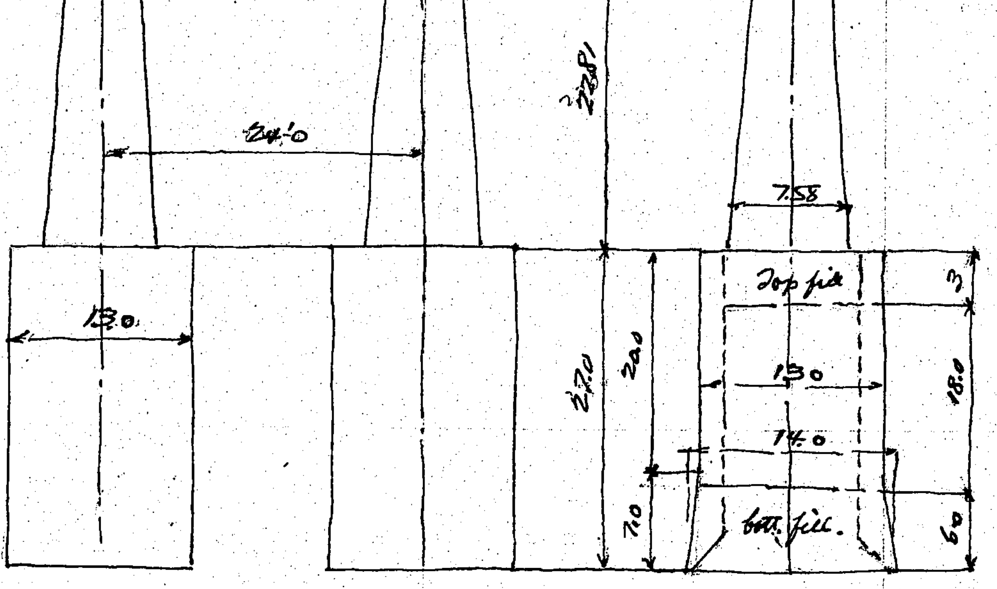
13792# ÷ 60 = 230# per lin. ft.

Approximate weight of metal in one span

Stringers	102# · 60.0	=	6120
Floor Beam	4 @ 1165	=	4660
" "	2 @ 1083	=	2166
Lateral Bracings			3120
Girders	2 @ 13800	=	27600
misc steel say			350
			44016 or 19.65 tons

For max bearing assume load 93.000#
Bearing plate 15×21 = 315□" Unit bearing = 93000 ÷ 315 = 295#/□" OK.

Design of Piers for truss spans. (TP 2-3-4-5-6-7)



Approximate Concrete in pier.
Coping 2-7.25^φ 2 @ 41.3 = 82.6
16.75 · 3.5 = 58.6
141.2 × 1.5 = 212.

Shaft 5.75^φ = 26.0
7.58^φ = 45.1
71.1 ÷ 2^c = 35.55

35.55 × 22.81 = 810
web = 2 · 17.33 · 22.81 = 790.
shaft + web.
coping 212
shaft 2 @ 810 = 1620
web 790
2622 cubic ft

or 12.15 cubic tsubo
wt = 12.15 @ 30200 = 367.000#

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

well shell 13.0 dia 132.7
10.0 dia -78.5
54.2 • 20 = 1084
$14'^{0}-10'^{0}$ = 75.4
$13^{\phi}-10^{\phi}$ = 54.2
129.6 ÷ 2 = 64.8 • 7 = 453

upper portion 1084
lower portion 453
1537

2 @ 1537 = 3074 cubic ft
or 14.2 cubic tsubo.

Bottom filling 6.0 • 78.5 = 471
Top filling 3.0 • 78.5 = 235.5
706.5 • 2 = 1413
6.55 cubic tsubo

Intermediate filling 18 • 78.5 = 1412 @ 2 = 2824 cubic ft or 13.10 cubic tsubo.

Summary of Concrete in well
shell 1:2:4 14.20
Top and bottom fillings 1:2:4 6.55
Intermediate filling lean concrete 13.10
33.85 cubic tsubo

wt = 3385 @ 30200 = 1.020.000#

Total dead Load of pier shafts 362.000
wells 1020.000
1382.000#

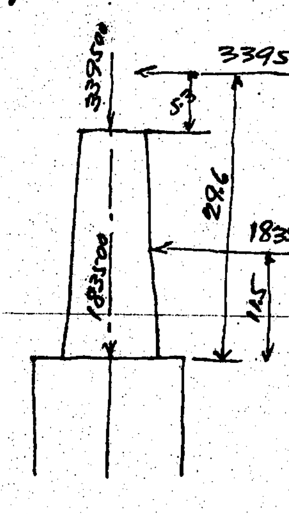
Superimposed load say page. 17. 679.000#

Live Load. Uniform live load for 2-177.5' spans $w = \frac{100.000}{170+105} = 364$ kg/m² or say 75#/□'

Total live load on pier 75 • 21 • 177.5 = 280.000#
Superimposed dead load 679.000
959.000#
weight of pier 1382.000
Total load on bottom 2,341,000#
Circumferencial friction assumed 27 • 200 • 40 = 216.000
2,125,000# net.

Bottom Area of well 2-14' dia = 2 • 153.9 = 307.8 □'
Unit bearing pressure = 2125.000 ÷ 307.8 = 6900#/□' or 3.08 tons/□'

Reinforcement in shaft due to Earthquake.



Assumed accelleration of Earthquake = 1000 mm/sec²
Hor. Force Dead Load on pier 339500 • 0.1 = 33950
shaft 183500 • 0.1 = 18350
52300

Moment about bottom of shaft
= 33950 • 29.6 = 1005000
18350 • 11.5 = 211000
1216.000'#

Moment of inertia of section
Concrete $0.049 \cdot 7.58^4$ = 162.0
Steel $15 \cdot 0.049 \cdot \frac{7^2}{8}$ = 4.5
166.5 (ft)⁴

16-3/4"ϕ bars assumed
Steel Area = 7.04 □" Steel % = 0.1 Fibre stress = $\frac{1216.000 \cdot 3.79}{166.5}$ = ± 27700#/□' or 192#/□"

Equivalent concrete area concrete 7.58^{ϕ} = 45.12 • 144 = 6500
steel 15 • 7 = 105
6605 □"

Direct pressure = 523000 ÷ 6605 = 79.2 #/□"

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Summary for fibre stress.

192 T	192 C
79.2 C	79.2 C
112.8 #/□" T	271.2 C

Approximate stress in shaft neglecting tension of concrete
Square beam of 4.5' × 6.8' assumed
Steel area 6-3/4" ⌀ .44 = 2.64□" Section of beam = 4.5 × 6.8 × 144 = 4410□"

Steel % = $\frac{2.64}{4410}$ = 0.06 % Eccentricity = $\frac{1216.000}{523.000}$ = 2.32 $e/h = \frac{2.32}{6.8}$ = 0.342

k = 0.53 Coefficient = 0.089

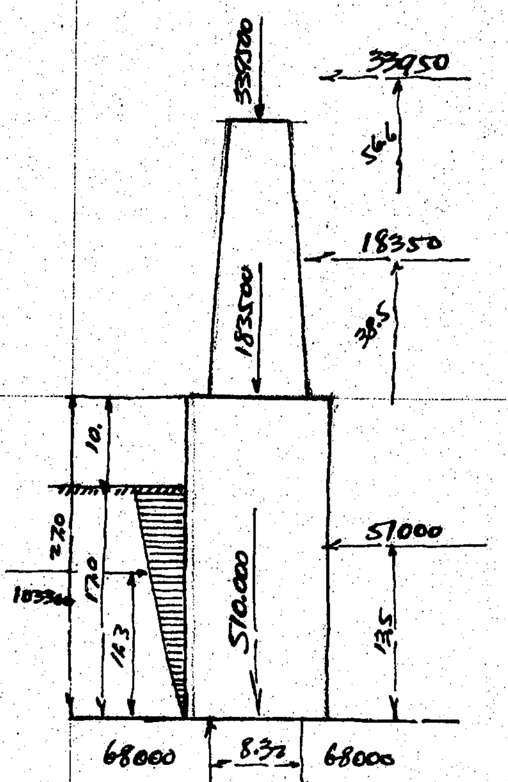
stress in concrete = $\frac{1216.000}{0.089 \times 4.5 \times 6.8^2}$ = 65600 #/□' or 455 #/□"

$f_s = 15 \times 455 \left(\frac{6.8}{0.53 \times 6.8} - 1 \right) = 6080$ #/□" -

From Journal of the institute of Japanese Architects 大正十一年六月号: Reinforced Concrete Round Column subjected to bending and direct load by messrs Naito and Kawai.
Steel % 0.1 Concrete stress = 5.0 × $\frac{523000}{6605}$ = 396 #/□"

neutral axis 0.5 Steel stress = $15 \times 396 \times \left(\frac{7.33}{7.58 \times 0.5} - 1 \right)$ = 5550 #/□"

Bearing Pressure at bottom of base during Earthquake



Firm ground assumed 10' below of river bed
Side resistance of firm ground and frictional resistance along circumference of well considered below line of firm ground

Moment about bottom of base.

33950 ×	56.6	=	1.920.000	
18350 ×	38.5	=	707.000	
51000 ×	13.5	=	690.000	
103300#				3,317,000
103300 ×	11.3	=	1.170.000	
68000 ×	8.32	=	565.000	
				1735.000
				1,582,000 #

Friction of well surface = 40.8 × 200 = 8000# per ft
Total friction = 17 × 8000 = 136000#

Direct load	superimposed dead load	339500	
	shaft	183500	
	well	510000	
			1033000
	Less friction		136000
			897000# net

Bottom area of well 14' = 153.9□'
Unit pressure = $\frac{897000}{153.9}$ = 5830 #/□' 2.60 tons/□'

Moment of inertia of bottom area of base = 0.049×14^4 = 1885 ft⁴

Design of Kasumi-Bashi for Okayama-Ken.

Fibre stress due to moment = $\frac{1582.000 \cdot 7}{1885}$ = 5870 #/□' 2.62 tons/□'

Summary for soil pressure

due to bending moment	2.62 C	2.62 T
" " direct load	2.60 C	2.60 C
	5.22 $^{tons}/□'$ C	0.02 $^{ton}/□'$ T

Design of well

Earth pressure during well sinking assumed

at 20' $p = \frac{1}{3} \cdot 100 \cdot 20 = 667$ #/□'

at 27' $p = \frac{1}{3} \cdot 100 \cdot 27 = 900$ "

Thickness of well at 20' = 1.5'

Diameter of well along neutral axis = 11.5' $m = \frac{1}{6} \cdot 667 \cdot 11.5^2 = 5520$ '#

Thickness of well at 27' = 2.0

Diameter of well along neutral axis = 12.0 $m = \frac{1}{6} \cdot 900 \cdot 12^2 = 8100$ '#

Reinforcement at bottom = $\frac{8100 \cdot 12}{\frac{7}{8} \cdot 22 \cdot 17000}$ = 0.297 □" per ft.

Reinforcement at 20' = $\frac{5520 \cdot 12}{\frac{7}{8} \cdot 16 \cdot 17000}$ = 0.278 □" per ft.

vertical bars - 3/4"ø throughout.

Design of pier for between truss and girder spans (TP 1 and 8).

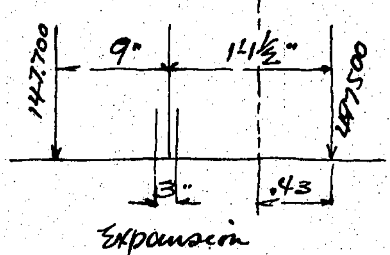
Live load for truss and girder spans

Total length = 177.5 + 60 = 237.5 ft or 72.5 meter

$w = \frac{100.000}{170 + 72.5}$ = 412 kg/m² or say 85 #/□'

Truss span Dead Load 339500
Live load $85 \cdot 21 \cdot \frac{177.5}{2}$ = 158000
497500

Girder span Dead Load
Floor and metal. 2 @ 47100 = 94200
Live load 85 · 21 · 30 = 53500
147700



Arm for Resultant
moment 147700 · 1.87 = 276.000 '#

Arm = 276.000 ÷ 645200 = 0.43 call this 5"

Total load 147700
497500
645200 #

For dead load only. 339500
94200
433700

m = 94200 · 1.87 = 176,000 '#

Arm = 176.000 ÷ 433.700 = 0.407'

Locate ℄ of pier 5" from ℄ bearing of truss span.

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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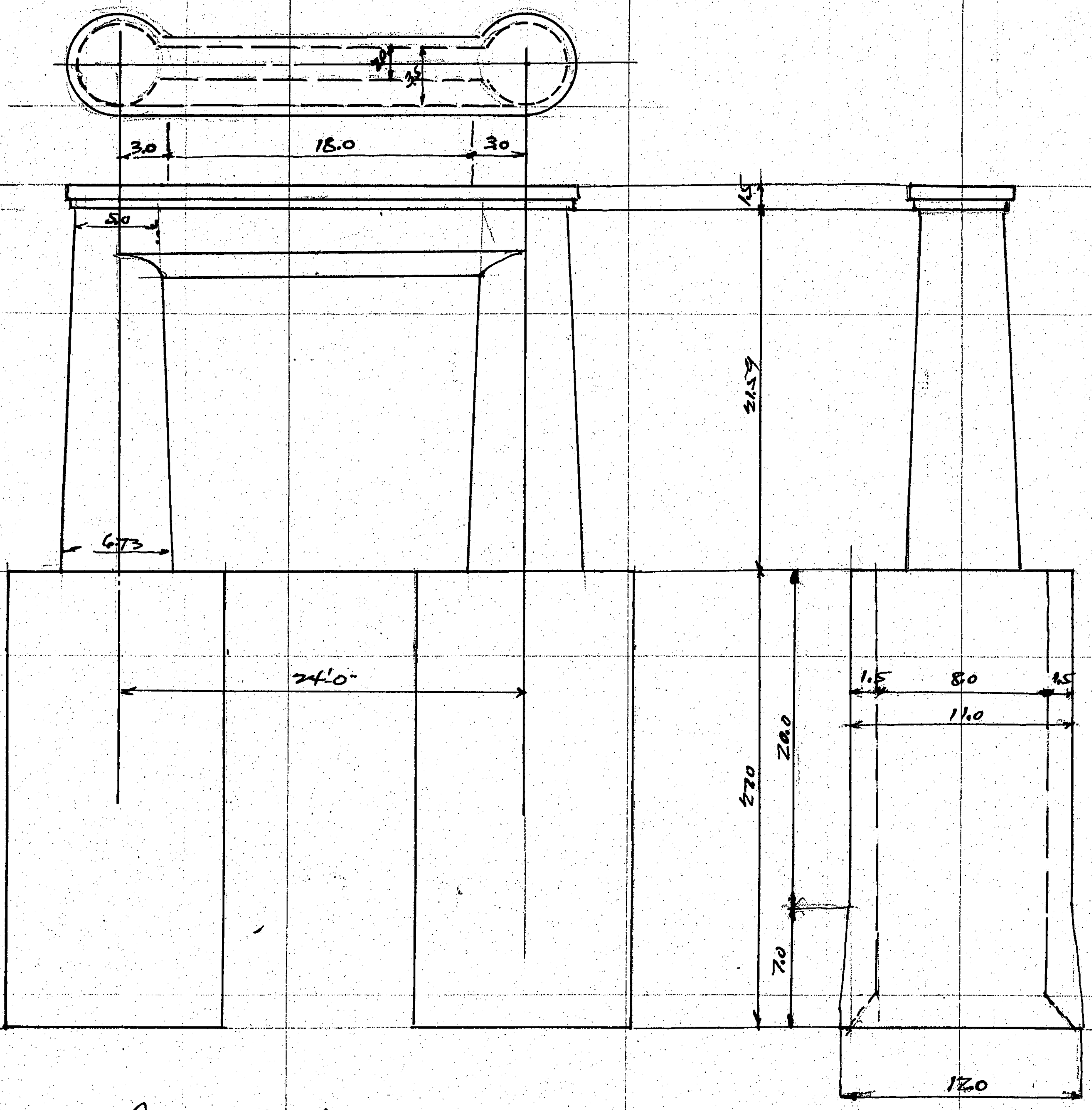
Design of Kasumi-Bashi for Okayama-Ken



Approximate Concrete in pier

Coping 2- 6.5' dia = 66.4
18.0 × 5 = 90.0
166.4 × 1.5 = 250

Shaft 5' dia = 19.6
6.73 dia = 35.6
55.2 ÷ 2 = 27.6 @ 21.59 = 595

Web. 18.13 × 2 × 21.59 = 593
18.0 × 3.25 × 1.5 = 88
681

Well. shell 11' dia 95.0
8' dia. 50.3
34.7 × 20 = 694
342
1036

2@1036 = 2072 cubic shaku
or 9.57 " tsubo

12' dia 113.1
8' dia. 50.3
62.8
34.7
97.5 ÷ 2 = 48.75 @ 7 = 342

Coping - 250
Shaft. 2@595 = 1190
Web. 681
2121
9.8 cubic tsubo
wt 9.8 × 30200 = 296000#

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Design of Kasumi-Bashi for Okayama-Ken.

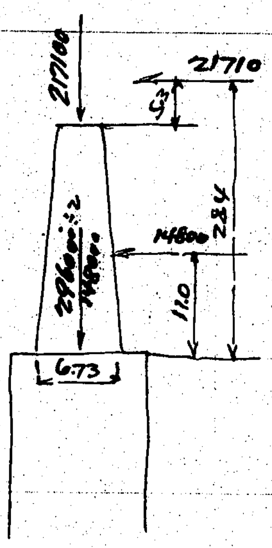
Top and bottom filling 503 · 9 = 4527 @ 2 = 905 cubic ft or 4.19 cubic tsubo.

Intermediate filling 503 · 18 = 905 · 2 = 1810 " or 8.38 " "

Summary of Concrete in wells.
shell 9.57
filling 4.19
filling 8.38
22.14 @ 30200 = 670.000
weight of shaft 296.000
966000
Superimposed dead and live loads. 645200
1611200
Circumferencial friction assumed 3.45 · 200 · 27 − 188000
1423200# net

Bottom area 12' dia 2@ 113.1 = 226.2□'
Unit bearing = 1423200 ÷ 226.2 = 6290#/□' or 2.81 tons/□'

Reinforcement in shaft due to Earthquake.



16-3/4"□ bars = 7.04□"
or. 0.049 □'

Assumed accelleration of Earthquake = 1000 mm/sec²
Horizontal force
Superimposed Dead Load 217100 · 0.1 = 21710#
weight of shaft. 148000 · 0.1 = 14800#
365100

moment about bottom of shaft
21710 · 28.4 = 617000
14800 · 11.0 = 163.000
780,000'#

moment of Inertia of section
Concrete 0.049 · 6.73^4 = 100.8
Steel 15 · 0.049 · $\frac{6.73^2}{8}$ = 3.6
104.4

Fibre stress = $\frac{780.000 \cdot 3.37}{104.4}$ = 25300#/□' or 176#/□"

Equivalent Concrete area of section
Concrete 6.73 dia 35.57 · 144 = 5110
Steel 15. 7. = 105
5215 □"

Direct Pressure 365100 ÷ 5215 = 70#/□"

Summary for stress (in fibre).

Due to bending stress	176 C	176 T
" " Direct stress	70 C	70 C
	246 C #/□"	106 T #/□"

This value approximately same as shown on page 27 for intermediate truss piers.
16-3/4"□ reinforcing bars are ample for this section of shaft.

Bearing Pressure at bottom of base during Earthquake.
Firm ground assumed 10.0' below river bed ; side resistance of firm ground and circumferencial friction of well considered below this line.
Friction of well surface = 34.5 · 200 · 17 = 117000#
Arm for frictional couple = 11.0 · .64 = 7.03' about

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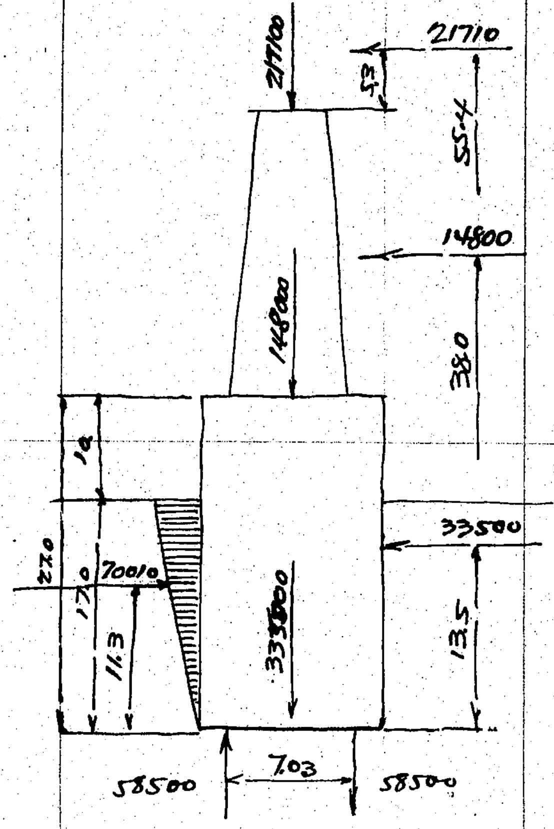
Design of Kasumi-Bashi for Okayama-Ken.



moment about bottom of base

21710	·	55.4	=	1205.000	
14800	·	38.0	=	563.000	
33500	·	13.5	=	452.000	
70010					2,220.000
70010	·	11.30	=	791.000	
58500	·	7.03	=	411.000	
					1,202.000
					1,018.000 $^{\#}$

Direct load

Superimposed Dead load	217100
shaft	148000
wells.	335000
	700100
Less friction	117000
	583100 #

Bottom area · 12' dia = 113.1 o'

unit bearing = 583100 ÷ 113.1 = 5150 #/o' or 2.30 tons/o'

Moment of inertia of bottom area = $0.049 \cdot 12^4 = 1015\ (ft)^4$

Fibre stress due to bending = $\frac{1018.000 \cdot 6}{1015}$ = 6010 #/o' or 2.68 tons/o'

Summary for soil pressure (Extreme fibre).

Bending	2.68 C	2.68 T
Direct load	2.30 C	2.30 C
	4.98 tons/o'	.38 tons/o'

Shell:-

Earth pressure during well sinking assumed

at 20' = 667 # per sq ft

at 27 = 900 " "

At 20' thickness 1.5' $m = \frac{1}{6} \cdot 667 \cdot 9.5^2$ = 3760 ′#

at 27' thickness 2.0 $m = \frac{1}{6} \cdot 900 \cdot 10.0^2$ = 5620 ′#

Reinforcement required at bottom = $\frac{5620 \cdot 12}{7/8 \cdot 22 \cdot 17000}$ = 0.20 □" per ft.

Reinforcement required at 20' = $\frac{3760 \cdot 12}{7/8 \cdot 16 \cdot 17000}$ = 0.19 □" per ft.

Design of Piers for Girder spans (GP 6-7-8-9-10).

Load on pier Dead Load 4 · 47100 = 188400 #

Live Load for 2-60' spans $w = \frac{100.000}{170 + 36.6}$ = 485 kg/m² or say 100 #/o'

Live Load 21 · 100 · 60.17 = 126000 #

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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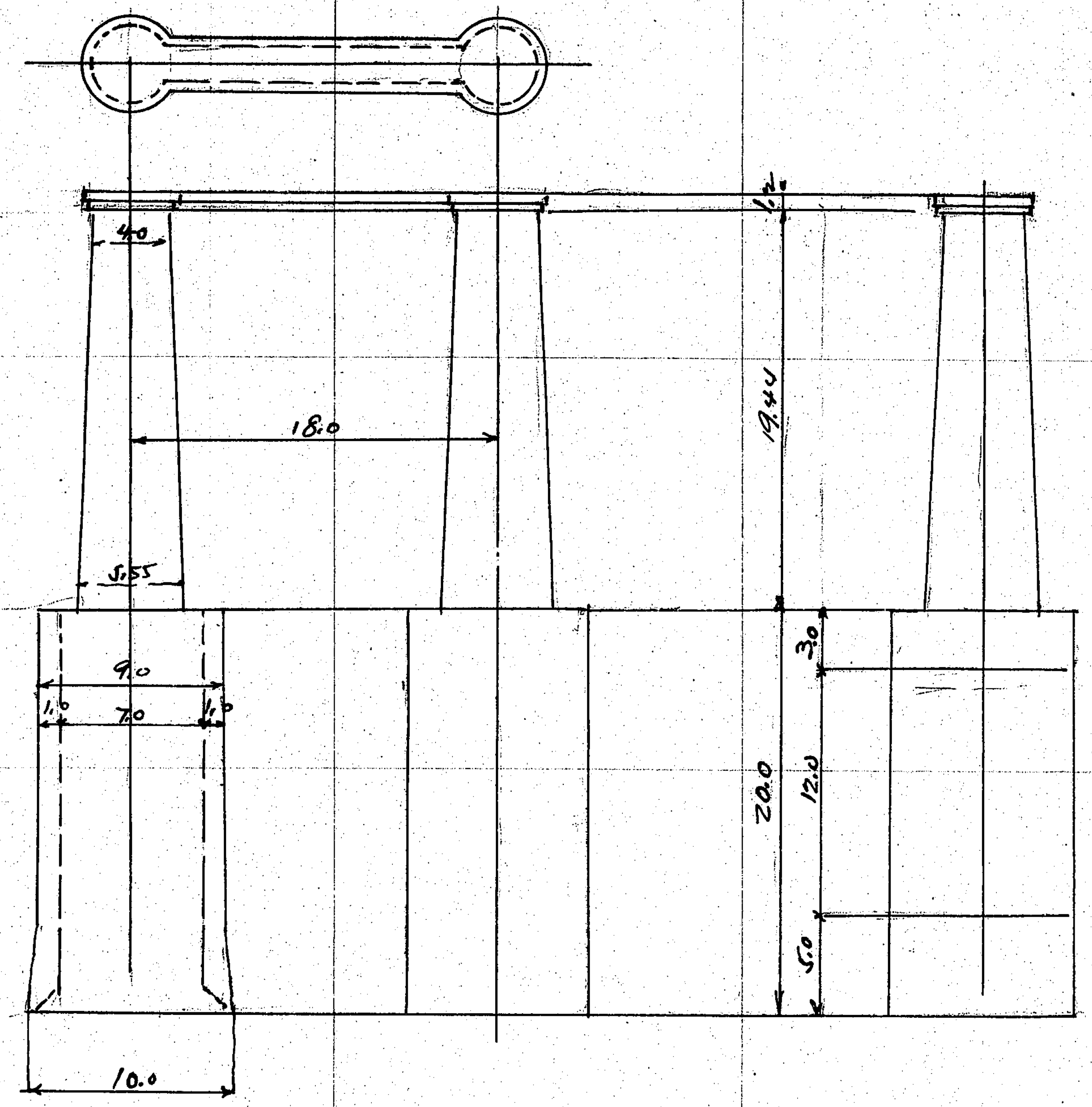
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken.



Approximate Concrete in pier

coping 2- 5' dia 39.2
13 • 2.5 32.5
71.7 • 1.2 = 86.0

Shaft 4' dia 12.6
5.55 dia 24.2
36.8 ÷ 2 = 18.4 vol = 18.4 • 2 • 19.44 = 715 wt = 5.5 @ 30200 = 166000#

web = 1.5 • 13.23 • 19.44 = 386

well. shell. 9.' dia 63.6
7' dia 38.5
25.1 • 15 = 377
163
540

10.0 dia 78.5
7.0 dia 38.5
40.0
25.1
65.1 ÷ 2 = 32.55 @ 5 = 163.

2 @ 540 = 1080 or 5.0 Cubic tsubo.

Summary

coping	86.0
Shaft	715.0
web	386.0
	1187.0

or 5.5 Cubic tsubo

Top and bottom filling 38.5 • 8 • 2 = 615 - 2.85 立坪

Intermediate filling 38.5 • 12 • 2 = 924 4.28

shell. 5.00

12.13 " "

Weight of well 12.13 @ 30200 = 367000

" " shaft 16$\overset{6}{3}$000

5$\overset{3}{3}$0000#

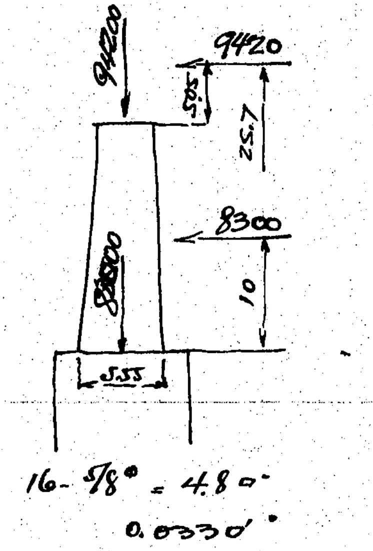
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

weight of pier 5$\overset{3}{3}$0,000
Superimposed load DL. 188400
LL. 126000
314400
84$\overset{7}{4}$400#
Less friction 9.0' 28.3 • 200 • 20 - 118.000
72$\overset{9}{8}$400#

Bearing area at bottom 2 - 10' dia = 2 • 78.5 = 157.0
Unit bearing pressure = 72$\overset{9}{6}$400 ÷ 157.0 = 4620 #/□' or 20.6 tons/□'

Reinforcement in shaft due to Earthquake



Assumed accelleration of Earthquake = 1000 mm/sec²
Horizontal force
superimposed dead load 94200 • 0.1 = 9420#
weight of shaft 83000 • 0.1 = 8300#
177200

moment about bottom of shaft
9420 • 25.7 = 242000
8300 • 10.0 = 83000
325.000'#

moment of inertia of bottom section
Concrete 0.049 • 5.55^4 = 46.5
Steel 15 • 0.033 • $\frac{5.05^2}{8}$ = 1.6
48.1

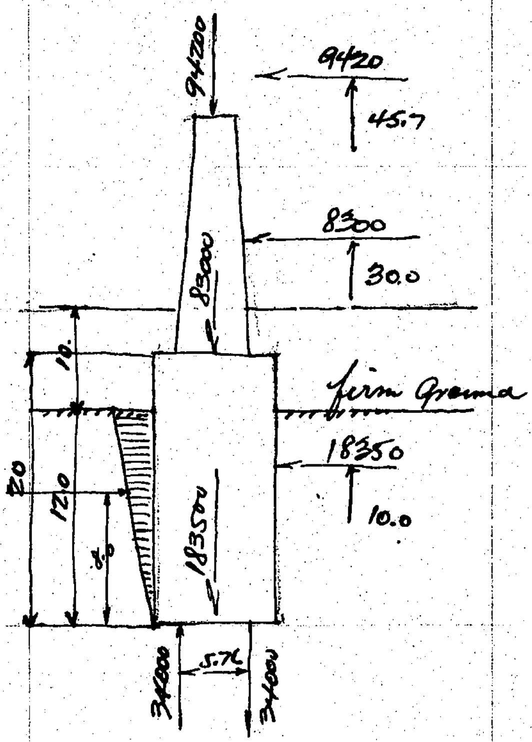
Fibre stress = $\frac{325.000 \cdot 2.78}{48.1}$ = 18800 #/□' or 130.0 #/□"

Equivalent Concrete area of section Concrete 5.55° 24.19 • 144 = 3480
steel 15 • 4.8 = 72
3552 □"

Unit direct stress = 177200 ÷ 3552 = 50 #/□"
Summary of fibre stress Bending 130 C 130 T
Direct load 50 C 50 C
180 C 80 #/□" T

Ok without reinforcement

Bearing Pressure at bottom of base during Earthquake.



firm ground assumed 12' above bottom of base, 10' loose ground on top assumed.
Friction of well surface 9'° = 28.3 • 200 • 12. = 68000#
Arm for frictional couple = 9 • .64 = 5.76'

moment about bottom of base
9420.0 • 45.7 = 430.000
8300.0 • 30.0 = 249.000
18350 • 10.0 = 183 500
36070
862500'#
36070 • 8.0 = 288.000
34000 • 5.76 = 196.000
-484.000
378500'#

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken.

Direct Load on bottom of base

Superimposed load	94200	
shaft	83000	
well	183500	
		360700#
Less friction		68000
		292700#

Bottom area 10.$^{\phi}$ = 78.5 Unit bearing = 292700 ÷ 78.5 = 3730 #/□' or 1.66 tons/□'

moment of inertia of bottom area = 0.049 · 10^4 = 490

fibre stress due to bending = $\frac{378500 \cdot 5}{490}$ = 3860 #/□' or 1.72 tons/□'

Summary for fibre stress

Due to bending	1.72 C	1.72 T
" " direct load	1.66 C	1.66 C
	3.38 tons/□'	.06 tons/□'

Design of shell.

Earth pressure during well sinking assumed.

at 15'+2' P = $\frac{1}{3}$ · 100 · 17 = say 570 #/□'

at bottom P = $\frac{1}{3}$ · 100 · 22 = " 750 #/□'

At 15' thickness 1.0' moment = $\frac{1}{16}$ · 570 · 8^2 = 2280'#

At bottom thickness 1.5' moment = $\frac{1}{16}$ · 750 · 8.5^2 = 3380'#

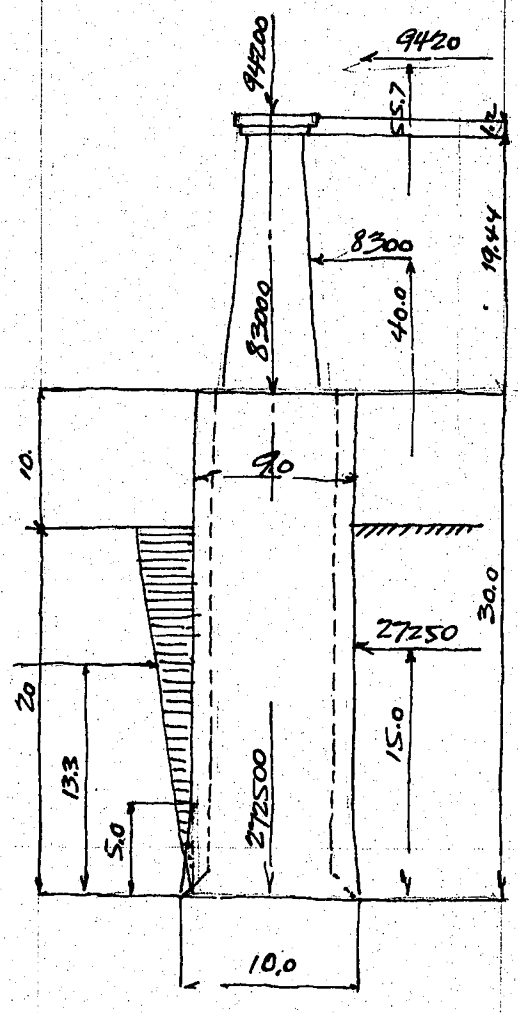
Reinforcement required at bottom = $\frac{3380 \cdot 12}{\frac{7}{8} \cdot 16 \cdot 17000}$ = 0.017 □" per ft

Reinforcement required at 15' = $\frac{2280 \cdot 12}{\frac{7}{8} \cdot 10 \cdot 17000}$ = 0.185 □" " "

Design of Piers for girder spans (GP 1 to 5).

make well 16' deep details same as for GP6 to 10 unless otherwise noted.

Design of Pier for Girder span (GP11)



Details of shaft same as for GP6 see page 32

well.

shell	9' dia	63.6	
	7' dia	38.5	
		25.1 · 25 =	627
		32.55 · 5 =	163
			790

2 @ 790 = 1580 or 7.33 cubic tsubo

Top and bottom filling	38.5 · 8 · 2 =	615	2.85 tsubo
Intermediate filling	38.5 · 22 · 2 =	1695	7.85 "
shell			7.33 "
			18.03 "

weight of well complete 18.03 @ 30200 =	545000	
" " shaft	166000	
		711000
Superimposed load		314400
		1025400
Less friction assumed 9'$^{\phi}$ 28.3 · 200 · 30 =		170000
		855400#

Bearing area of bottom 2-10$^{\phi}$ = 2 · 78.5 = 157.0

Unit Bearing 855400 ÷ 157 = 5450 #/□' or 2.43 tons/□'

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Bearing Pressure at bottom of base during Earthquake

Consider side pressure and frictional resistance along face of well; firm ground assumed 10.0' below top of well.

Friction of well surface assumed 9'° = 28.3 · 200 · 20 = 113000 #

Arm for frictional Couple = 9 · .64 = 5.76'

moment about bottom of base.

94200	· 55.7	=	524.000	
83000	· 40.0	=	332.000	
272500	· 15.0	=	408.000	
449700				1264.000
449700	· 13.3	=	597.000	
56500	· 5.76	=	325.000	
				922000
				342.000 1#

Direct load

Superimposed dead load	94200	
Shaft	83000	
well	272500	
		449700
Less friction say		113000
		336700 #

Bottom area 10.'ϕ = 78.5 □' Unit bearing 336700 ÷ 78.5 = 4300 #/□' or 1.92 tons/□'

Fibre stress due to bending = $\frac{342.000 \cdot 5}{490}$ = 3490 #/□' or 1.56 tons/□'

Summary for fibre stress

Bending	1.56 C	1.56 T
Direct load	1.92 C	1.92 C
	3.48 tons/□'	.36 tons/□' C

Design of shell.

Earth pressure during well sinking assumed

at 25' $P = \frac{1}{3} \cdot 100 \cdot 25$ = 830 #/□'

at 30' $P = \frac{1}{3} \cdot 100 \cdot 30$ = 1000 "

moment at 25' thickness 1.0 $m = \frac{1}{16} \cdot 830 \cdot 8^2$ = 3320 1#/□'

moment at 30' thickness 1.5' $m = \frac{1}{16} \cdot 1000 \cdot 8.5^2$ = 4510 1#/□'

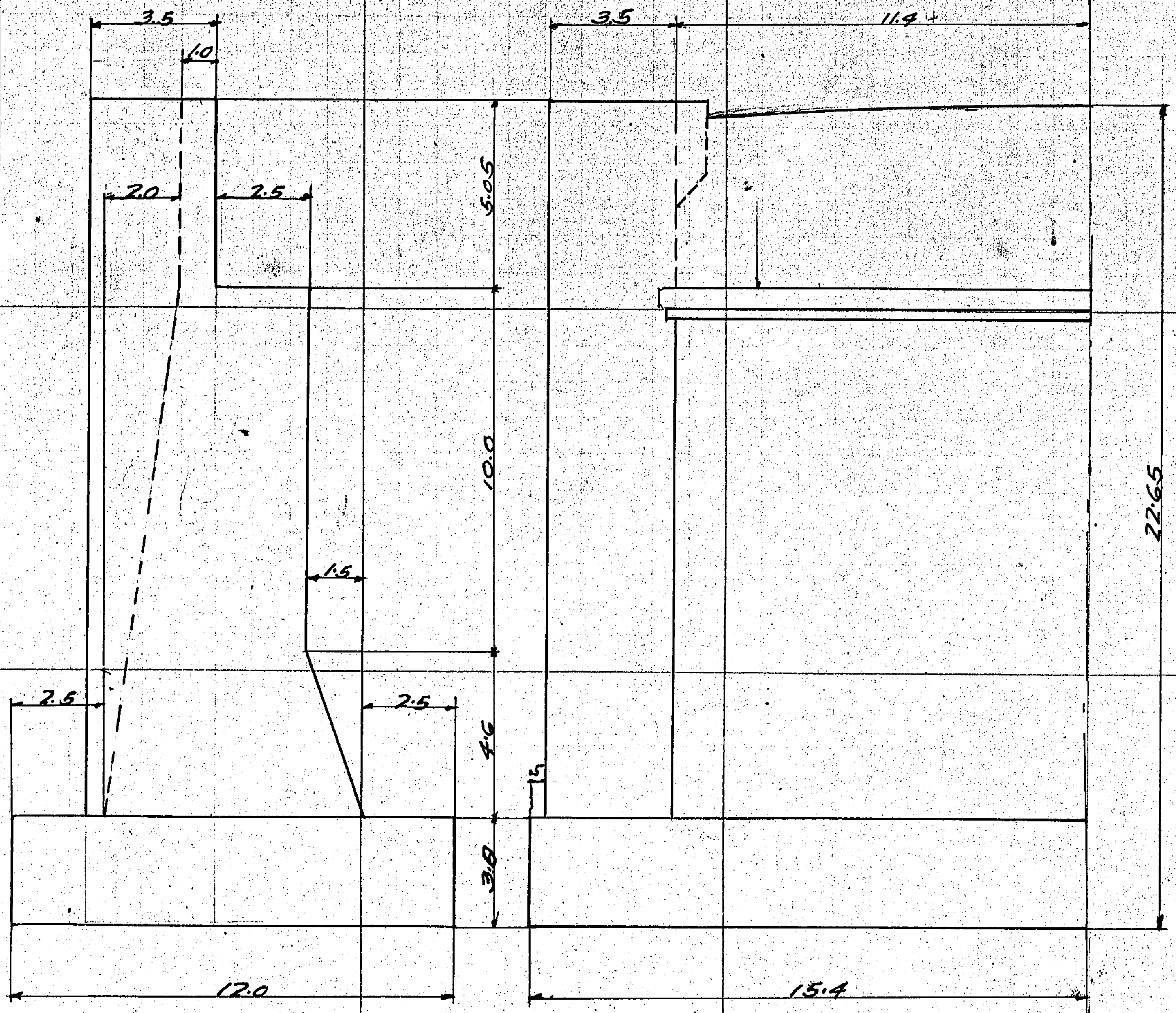
Reinforcement required at bottom = $\frac{4510 \cdot 12}{\frac{7}{8} \cdot 16 \cdot 17000}$ = 0.23 □" per ft.

Reinforcement required at 25' = $\frac{3320 \cdot 12}{\frac{7}{8} \cdot 10 \cdot 17000}$ = 0.27 □" per ft

CALCULATIONS FOR

Design of Kasumibashi, Okayama Ken.

Abutment A1, 22'.65 height



Load from superstructure

Dead Load. metal = 39,800# × ½ = 19,900#
" " floor = 2,150 × 30 = 64,500#
84,400#
Live Load 2,100 × 30 = 63,000#
147,400#

Weight and Volume of abutment Vol. cuft. wt
Base 3.0 × 12.0 × 24.8 = 893 × 150 = 134,000#
2@ 3.0 × 3.0 × 6.0 = 108 × do = 16,200
Wall 1.0 × 19.65 × 22.8 = 448 × do = 67,200
½ × 2.0 × 14.6 × 22.8 = 333 × do = 50,000
Main 2.5 × 14.6 × 22.8 = 832 × do = 124,800
½ × 1.5 × 4.6 × 22.8 = 78.6 × do = 11,800
Side 2@ 3.5 × 3.5 × 19.65 = 483. × do = 72,500
3175.6 476,500#
~ 147 ##

Weight of earth fill.

Front. Assume 5'.0 of earth from top of footing of abutment.
2.5 × 5 × 24.8 = 310 × 100 = 31,000
½ × 1.5 × 5 × 24.8 = 93 × do = 9,300
403 40,300

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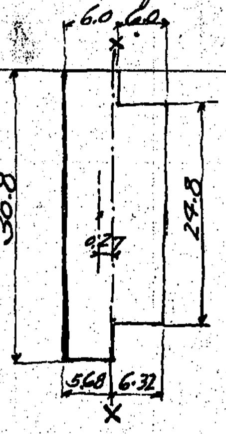
CALCULATIONS FOR

Design of Kasumi-bashi, Okayama Ken.

Rear.

		vol.			wt.
4.5 × 5.0 × 22.8	=	513.	× 100	=	51,300
½ × 2.0 × 14.6 × 22.8	=	333.	× do	=	33,300
2.5 × 14.65 × 22.8	=	835.	× do	=	83,500
2@ 4.0 × 2.0 × 79.65	=	314	× do	=	31,400
		1995			199,500#

Moment of Inertia of base area.



Moment of area abt Toe line

Area					
2@ 3.0×6.0	= 36.0	×	9	=	324
12.0×24.8	= 297.6	×	6	=	1785.6
	333.6		6.32		2109.6

$$I_{x-x} = \frac{24.8\times12^3}{12} + 297.6\times.32^2 + \frac{2\times3\times6^3}{12} + 36\times(3.0-.32)^2 = 3966.^{(f)^4}$$

Center of gravity of abutment.

	Weight		Horiz. arm from toe		
Base	134,000	×	6.0	=	804,000
	16,200	×	9.0	=	145,800
Wall	67,200	×	7.0	=	470,400
	50,000	×	8.5	=	425,000
main	124,800	×	5.25	=	656,000
	11,800	×	3.25	=	38,400
Side	72,500	×	8.25	=	598,000
	476,500		6.59		3137,600

6.59
6.32
.27 from ℄ of gravity of base area.

			Vert. arm from bottom.		
Base	134,000		1.5	=	201,000
	16,200	×	1.5	=	24,300
Wall	67,200	×	12.825	=	862,000
	50,000	×	7.9	=	395,000
main	124,800	×	10.3	=	1,285,000
	11,800	×	4.53	=	47,800
Side	72,500	×	12.825	=	930,000
	476,500		7.86		3745,100

distance of center from bottom.

Center of gravity of earth fill.

Front	wt.		Horiz. arm from toe		
	31,000	×	1.25	=	38,750
	9,300	×	3.0	=	27,900
	40,300		1.63		65,650

6.32
1.63
4.69 from ℄ of gravity of base area.

wt.		Vert. arm from bottom		
31,000	×	4.7	=	147,000
9,300	×	6.3	=	58,600
40,300		5.1		205,600

center of distance from base

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CALCULATIONS FOR

Design of Kasumi bashi, Okayama Ken

Rear. wt.		Horiz. arm from toe		
51,300	×	9.75	= 497,000	
33,300	×	8.8	= 293,000	
83,500	×	10.75	= 1,897,000	
31,400	×	11.0	= 345,000	10.18 6.32
199,500		10.18	2,032,000	3.86 from ℄ of gravity of base area.

wt	×	Vert. arm from bottom		
51,300	×	20.15	= 1,028,000	
33,300	×	12.8	= 426,000	
183,500	×	10.3	= 860,000	
31,400	×	10.3	= 323,500	
199.500		13.2	2637,500	Distance of center from bottom.

Earth pressure from rear fill.

Ordinary case

$P_0 = \frac{1}{3} \times \frac{wH^2}{2} = \frac{1}{3} \times \frac{1}{2} \times 100 \times 22.65 = 8550^{\#}$ per 1' strip.

Total pressure $8550 \times 29.8 = 255,000^{\#}$

In case of earth quake with seismic coefficient $\frac{3}{4}K$ or .0.75

$P = E \times \frac{wH^2}{2} = .388 \times 255,000 \times 3 = 297,000^{\#}$

$\beta = 7°30'$ $\sin 7°30' = .1305$ $\therefore P_{vert} = 38,800^{\#}$

$\cos 7°30' = .9914$ $\therefore P_{horiz.} = 294,500^{\#}$

Hight of point of action $= \frac{1}{3} \times 22.65 = 7.'55$ from bottom of abutment.

Stability of abutment.

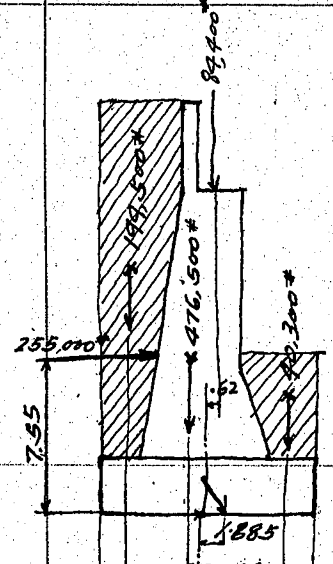
Case I. No load from superstructure, weight of earth fill and earth pressure from rear side acting.

Negative moment

$255,000^{\#} \times 7.55 = 1,926,000$

$40,300^{\#} \times 4.69 = 181,000$

$-2,107,000$



Positive moment

$476,500^{\#} \times .27 = 128,600$

$199,500^{\#} \times 3.86 = 771,000$

$+899,600$

$-2,107,000$

Resultant moment. $-1,207,400$

Sum of weight $716,300$

Acting point of resultant $-1,207,400 \div 716,300 = -1.'685$

Toe pressure $= \frac{716,300}{333.6} + \frac{1,207,400}{3966} = 4075^{\#}/□'$

Heel pressure $= \frac{716,300}{333.6} - \frac{1,207,400}{3966} = 420^{\#}/□'$

Case II Dead Load from superstructure, weight of earth fill and earth pressure from rear side acting.

Negative moment

$84,400 \times 0.62 = 52,300$

$1,207,400$

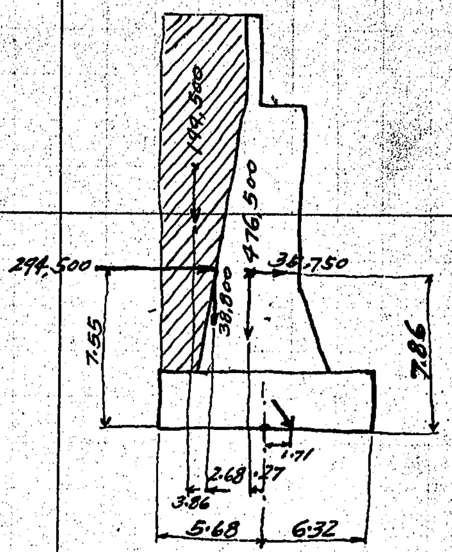
$1,259,700$

Sum of wt. $84,400 + 716,300 = 800,700$

Toe pressure $= 2,400 + 2005 = 4,405^{\#}/□'$

Heel pressure $= 2,400 - 1800 = 600^{\#}/□'$

Case III No load from superstructure, weight of abutment and earth quake thrust with seismic coefficient $\frac{3}{4}$K or 0.75, weight of rear fill and earth pressure from rear with seismic coefficient $\frac{3}{4}$K or 0.75.



Negative moment

294,500 × 7.55 = 1,926,000
35,750 × 7.81 = 281,000
-2,207,000

Positive moment

199,500 × 3.86 = 771,000
38,800 × 2.68 = 104,000
476,500 × 0.27 = 128,600
+1,003,600
-2207,000
-1,203,400'#

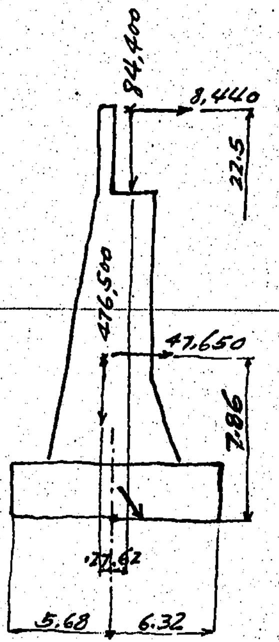
Sum of verticals = 714,800#

Point of action of resultant 1,220,400 ÷ 714,800 = 1.71

Toe pressure $= \frac{714,800}{333.6} + \frac{1,203,400}{3966} \times 6.32 = 4060$ #/□'

Heel pressure $= \frac{714,800}{333.6} - \frac{1,203,400}{3966} \times 5.68 = 1417$ #/□'

Case IV Dead load from superstructure with seismic coefficient 0.1 and weight of abutment with seismic coefficient 0.1.



Negative moment

8,440 × 22.5 = 190,000
47,650 × 7.86 = 374,000
84,400 × 0.62 = 52,300

Positive moment

476,500 × 0.27 = 128,600

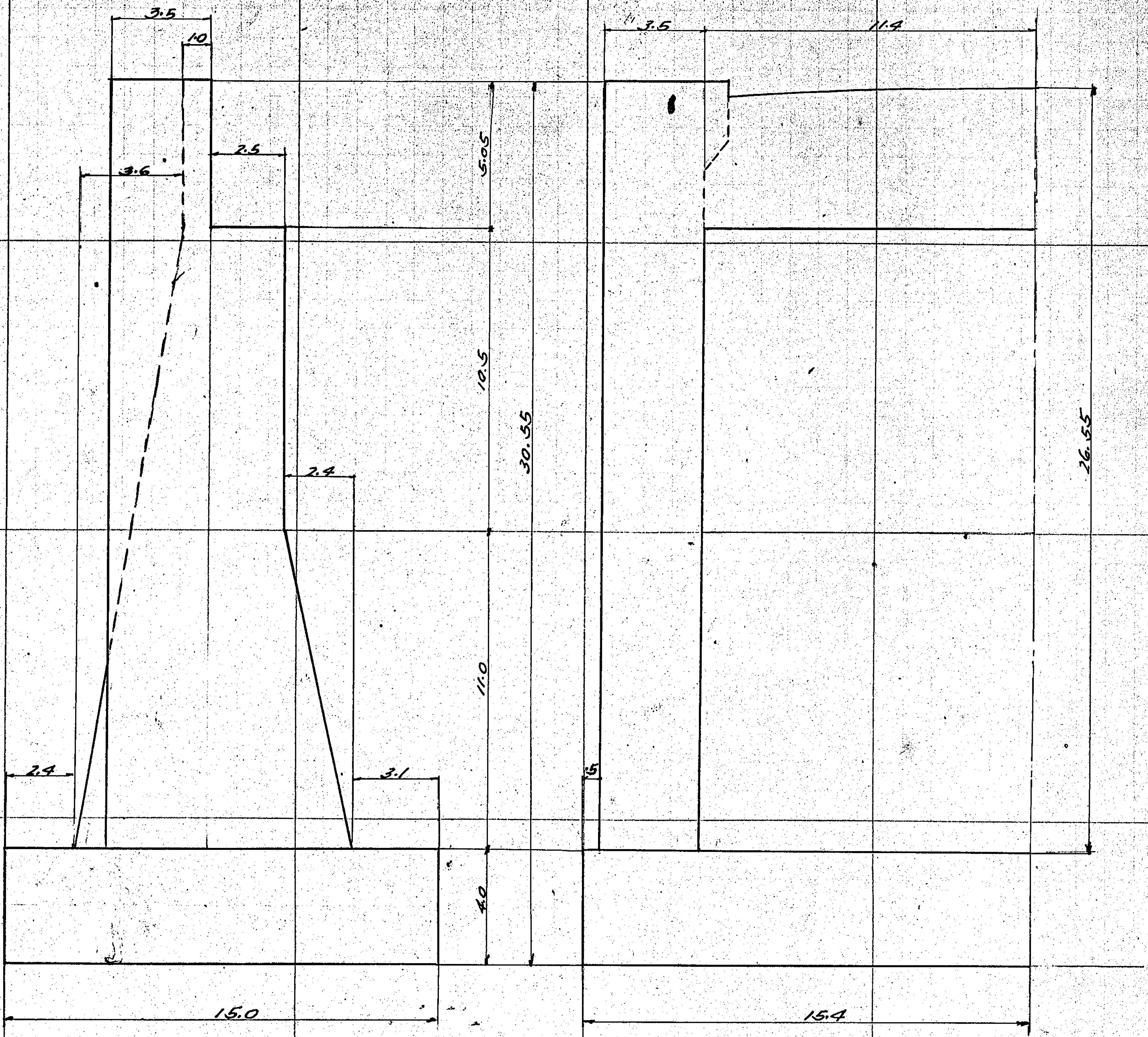
Resultant moment = 488,300

Sum of verticals = 560,900

Point of action of resultant 488,300 ÷ 560,900 = 0.871

Need no more study.

Design of Kasumibashi, Okayama Ken.
Abutment A2, 30.55 heigh



Weight and Volume of abutment,

				Vol.				wt.
Base		4.0 × 15.0 × 24.8	=	1,368	×	150	=	205,200
	2@	3.0 × 7.5 × 4.0	=	180	×	do	=	27,000
Wall		1.0 × 26.55 × 22.8	=	605	×	do	=	90,750
	½ ×	3.6 × 21.5 × 22.8	=	882	×	do	=	1,323,000
Main		2.5 × 21.5 × 22.8	=	1,225	×	do	=	1,837,500
	½ ×	2.4 × 11.0 × 22.8	=	302	×	do	=	45,300
Side	2@	3.5 × 3.5 × 26.55	=	651	×	do	=	97,650
				5213				781,950

or 2415立坪

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CALCULATIONS FOR

Design of Kasumi-boshi, Okayama Ken.

Weight of earth fill

Front. assume 11'0" of earth from top of footing of abutment.

3.1 × 11.0 × 24.8 = 846. × 100 = 84,600#

½ × 2.4 × 11.0 × 24.8 = 317. × 100 = 31,700

116,300#

Rear.

6.0 × 505 × 22.8 = 691. × 100 = 69,400

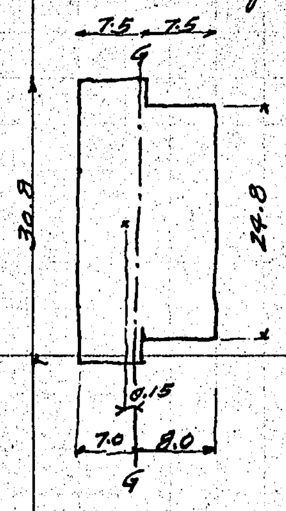
½ × 3.6 × 21.5 × 22.8 = 886. × do = 88,600

2.4 × 21.5 × 22.8 = 1177. × do = 117,700

2@ 3.5 × 4.0 × 26.55 = 743. × do = 74,300

349,700#

Moment of inertia of base area.



Moment of area abt. toe line

	area		
15.0 × 24.8 =	317.	× 7.5 =	2378.0
2@ 3.0 × 7.5 =	45.	× 11.25 =	517.5
	362	8.0	2895.5

$$I_{G\text{-}G} = \overset{6975}{\frac{24.8\times15^3}{12}} + \overset{317}{317\times1.0^2} + 2\times\overset{\sim11}{\frac{3\times7.5^3}{12}} + \overset{340}{45.0\times2.75^2} = 7843^{(s)4}$$

Center of gravity of abutment.

	Weight		Horiz. arm from toe.		
Base	205,200	×	7.5	=	1,54,0000
	27,000	×	11.25	=	303,700
Wall	90,750	×	8.5	=	771,000
	132,300	×	10.2	=	1,349,000
Main	183,750	×	6.75	=	1,240,000
	45,300	×	4.7	=	213,000
Side	97,650	×	9.75	=	952,000
	781,950		8.15		6,368,700

8.15
8.0
0.15 from ℄ of gravity from base area.

			Vert. arm from bottom.		
Base	205,200	×	2.0	=	410,400
	27,000	×	2.0	=	54,000
Wall	90,750	×	17.275	=	1,567,000
	132,300	×	11.2	=	1,482,000
Main	183,750	×	14.75	=	2,710,000
	45,300	×	7.7	=	348,800
Side	97,650	×	17.275	=	1,687,000
	781,950		10.56		8,259,200

Distance of center from abutment.

Center of gravity of earth fill.

Front	84,600	×	1.55	=	131,100
	31,700	×	3.9	=	123,700
	116,300		2.19		254,800

8.98
2.19
5.86 from ℄ of gravity from base area.

84,600	×	9.5	=	804,000
31,700	×	11.4	=	361,500
116,300		10.02		1,165,500

Center of distance from base.

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CALCULATIONS FOR

Design of Kasumi-bashi. Okayama Ken.

Rear. 69,100 × 12.0 = 829,000
88,600 × 11.4 = 1,010,000
117,700 × 13.8 = 1,625,000
74,300 × 13.25 = 985,000
349,700 12.72 4,449,000

12.72
8.00
4.72 from ℄ of gravity of base area.

69,100 × 28.05 = 1,940,000
88,600 × 18.4 = 1,630,000
117,700 × 17.28 = 2,033,000
74,300 × 17.28 = 1,284,000
349,700 19.69 6,887,000

centr of distance from base

Earth pressure from rear fill.

Ordinary case

$P_0 = \frac{1}{3} \times \frac{wH^2}{2} = \frac{1}{2} \times \frac{1}{3} \times 100 \times 30.55^2 = 15,570^{\#}$ per 1' strip

Total pressure $15,570^{\#} \times 29.8 = 464,000^{\#}$

In case of earth quake with seismic coefficient $\frac{3}{4}K$ or 0.075

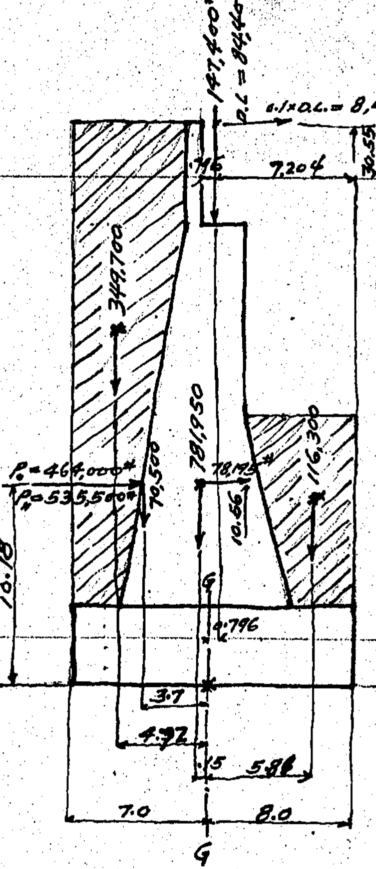
$P = E \times \frac{wH^2}{2} = .388 \times 464,000 \times 3 = 540,000^{\#}$

$P_{vert} = 540,000 \times .1305 = 70,500^{\#}$ Dist. of act. pt. = 11.7 from toe. $\frac{11.7}{8.0}$ 3.7

$P_{horiz.} = 540,000 \times .9914 = 535,500^{\#}$ Height of action $= \frac{H}{3} = 10.18$ from bottom

Stability of abutment.

Case I Load from superstructure, earth pressure from rear fill and weight of abutment itself with rear and front fill acting.



Moment abt ℄ of gravity of base area.

Positive Moment

464,000 × 10.18 = 4,725,000
147,400 × .796 = 117,300
116,300 × 5.81 = 676,000
+ 5,518,300

Negative moment

349,700 × 4.72 = 1,650,000
781,950 × 0.15 = 117,100
- 1,767,100

Resultant moment = + 3,751,200
Sum of verticals. = 1,395,350
Point of action = 3,751,200 ÷ 1395,350 = 2.69

Toe pressure $= \frac{1,395,350}{362}$ 3860 $+ \frac{3,751,200}{7843}$ 3826 $\times 8 = 7686^{\#}/□'$

Heel pressure $= \frac{1,395,350}{362} - \frac{3,751,200}{7843}$ 3347 $\times 7 = 513^{\#}/□'$

Case II Dead Load from superstructure and body of abutment with seismic coefficient K=0.1 and earth pressure at rear with seismic coefficient $\frac{3}{4}K = 0.075$

Positive moment

535,500 × 10.18 = 5,455,000
78,195 × 10.56 = 826,000
8,440 × 30.55 = 257,800
84,400 × .796 = 67,200
+ 6,606,000

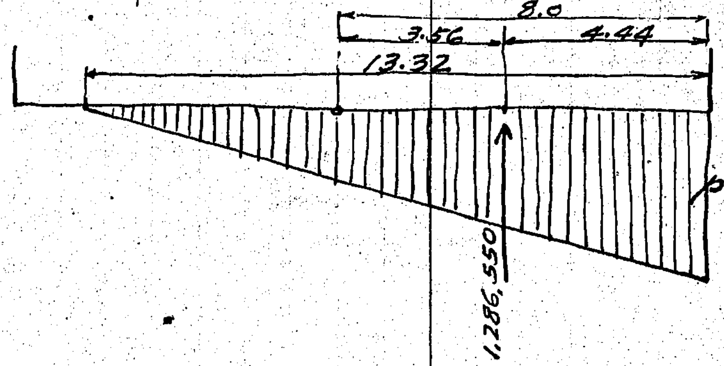
Negative moment

349,700 × 4.72 = 1,650,000
70,500 × 3.7 = 261,000
781,950 × .15 = 117,100
- 2,028,100
+ 6,606,000

Resultant moment + 4,577,900
Sum of verticals 1,286,550

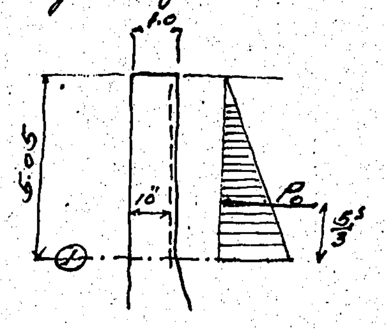
$$\eta = \frac{4,577,900}{1,286,550} = 3.56$$

$$\frac{\begin{array}{r}8.0\\3.56\end{array}}{4.44} \times 3 = 13.32$$



$$p = \frac{2 \times 1,286,550}{13.32 \times 24.8} = 7,790 \ {}^{\#}/_{\square'}$$

Design of detail



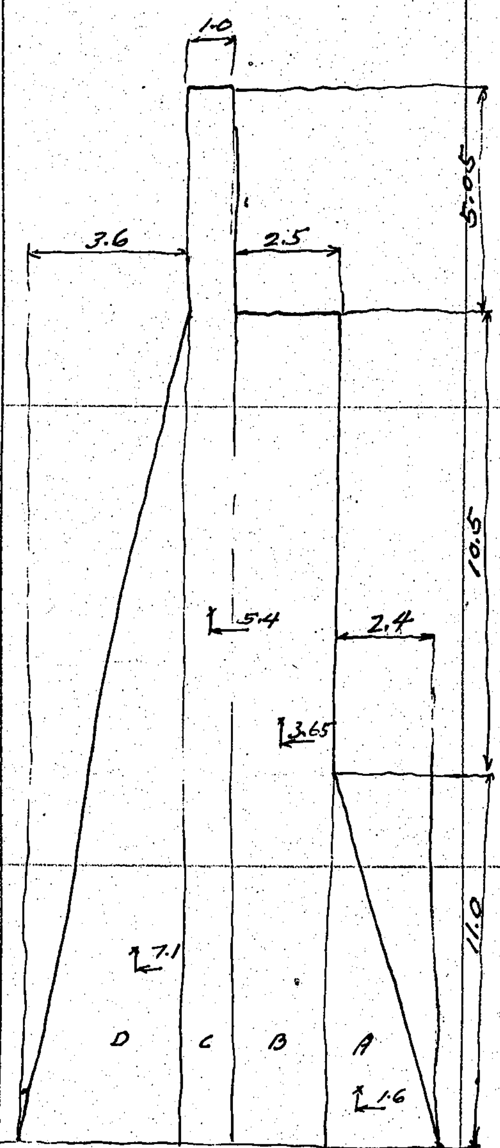
$$P_o = \frac{1}{3} \times 100 \times \frac{5.05^2}{2} = 417.0^{\#} \text{ for } 1' \text{ vert. strips.}$$

Bending moment at section ①

$$M = 417 \times \frac{5}{3} = 695'^{\#}$$

$$A_s = \frac{695 \times 12}{\frac{7}{8} \times 10 \times 17,000} = .0561$$

Use 1-½"φ with 1.5' pitch $A_s = .196 \times \frac{1}{1.5} = .13 \square''$
or 2.0' " $A_s =$ " $\frac{1}{2} = .098 \square''$



Weight of 1' strip of body of abutment above footing.

A	½ × 2.4 × 11.0	= 13.2	× 150	= 1,980	
B	2.5 × 21.5	= 53.75	× do	= 8,063	
C	1 × 26.55	= 26.55	× do	= 3,982	
D	½ × 3.6 × 21.5	= 38.7	× do	= 4805	
				18,840#	

Center of gravity

A	1,980	× 1.6	= 3,170
B	8,063	× 3.65	= 29,420
C	3,982	× 5.4	= 21,520
D	4,805	× 7.1	= 34,100
		4.61	88,210

Superimposed Load

$$\frac{147,400}{22.8} = 6,470^{\#}$$

Earth pressure at rear.

$$P_o = \frac{1}{3} \times \frac{100 \times 26.55^2}{2} = 17,400^{\#} \text{ acting at}$$

$$\frac{26.55}{3} = 8.85' \text{ high above this section.}$$

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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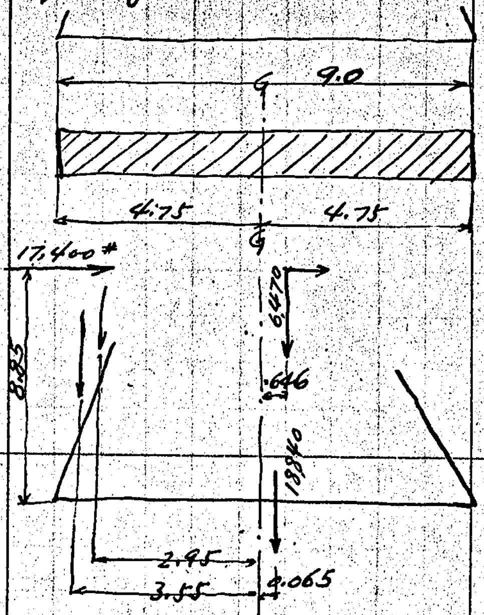
CALCULATIONS FOR

Design of Kasumi' bashi' Okayama Ken



Moment of inertia of the section.

$I = \frac{1.\times 9.5^3}{12} = 71.5^{(5)4}$

Rear earth fill

$5.0 \times 3.6 = 18.0 \quad \times 100 = 1800\#$

$\frac{1}{2} \times 3.6 \times 21.5 = 38.7 \quad \times 100 = \underline{3870\#}$

$5670\#$

moment abt ℄ of gravity of base area.

Body $18,840 \times +0.065 = 1,224$
Superimp. Ld. $6,470 \times +0.646 = 4,180$ } $+159,206\#$
P_o $17,400 \times +8.85 = 153,800$
Rear fill $1,800 \times -2.95 = -5,310$ } $-19,060'\#$
do $3,870 \times -3.55 = \underline{-13,750}$
Resultant moment $139,140$

Sum of verticals $30,980\#$

$p = \frac{139.140}{30,980} = 4.\overset{5}{3}$

Toe pressure $\frac{130,980}{9.5} + \frac{139,140}{71.5} \times 4.75 = 12,510\#/\square'$
or $87\#/\square''$

Heel pressure $= \frac{30,980}{9.5} - \frac{139,140}{71.5} \times 4.75 = -5990\#/\square'$
or $-41.6\#/\square''$

Reinforcement at heel

$A_s = \frac{159,200 \times 12}{\frac{7}{8} \times 111 \times 17000} = 1.15\square''$

use $2-\frac{7}{8}''\phi$. $A_s = 2 \times 0.6 = 1.2\square''$

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CALCULATIONS FOR

岡山縣霞橋材料調書

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

List of materials for Kasumi-Bashi, Okayama-Ken

End Post L_0-U_0 4-Required

Cover Plate	1 Pl.	19 × 7/16	×	21'-3	@	28.26	=	600
	2 [s.	12 × 3½	×	22'-1½	@	32.88	=	1450
Gusset Plate	2 Pls.	35½ × 7/16	×	3'-5	@	45.27	=	310
diaphragm	2 Ls.	4 × 4 × 5/16	×	0'-10⅜	@	8.20	=	14
"	4 Ls.	3 × 3 × 5/16	×	1'-0½	@	6.10	=	25
"	1 Pl.	10½ × 5/16	×	1'-0½	@	11.16	=	12
"	2 Wash.	3" × 5/16			@	0.62	=	1
"	4 Ls.	4 × 3 × 5/16	×	1'-4	@	7.2	=	38
"	1 Pl.	11 × 5/16	×	1'-4	@	11.69	=	16
Tie Plate	2 Pls.	17½ × 3/8	×	1'-7	@	22.31	=	70
	4 Ls.	3½ × 3½ × 3/8	×	2'-5½	@	8.5	=	84
	2 Ls.	3 × 3 × 3/8	×	2'-5½	@	7.2	=	35
	4 Ls.	"	×	2'-5½	@	7.2	=	71
	1 Pl.	9 × 3/8	×	2'-5½	@	11.48	=	28
	2 Pls.	11¾ × 3/4	×	3'-4	@	29.96	=	200
Gusset Plate	2 Pls.	29¼ × 3/8	×	3'-0	@	37.30	=	224
Lacing bars	18 bars.	2¼ × 3/8	×	2'-1¼	@	2.87	=	108
	1 L.	3 × 3 × 5/16	×	0'-10¾	@	6.1	=	6
	2 Pls.	3½ × ½	×	22'-1½	@	5.95	=	263
								3555 × 4 = 14,220#

Top chord. U_0-U_1 4-Required

Cov. Pl.	1 Pl.	19 × 3/8	×	18'-5¾	@	24.23	=	450
	2 [s.	12 × 3½	×	17'-7¾	@	26.10	=	921
	2 Ls.	3½ × 3½ × 3/8	×	1'-9¼	@	8.50	=	30
	1 Pl.	15 × 5/16	×	1'-11	@	15.94	=	31
	2 Pls.	17½ × 3/8	×	1'-7	@	22.31	=	70
	1 Pl.	16 × 3/8	×	1'-11½	@	20.40	=	40
	2 Ls.	3½ × 3 × 3/8	×	2'-2	@	7.9	=	34
	1 Pl.	13½ × 5/16	×	2'-9¼	@	14.34	=	40
	1 Pl.	14 × 5/16	×	1'-11½	@	14.88	=	29
	1 Pl.	3½ × ½	×	1'-11	@	5.95	=	11
	2 Pls.	10¼ × 3/8	×	0'-11	@	13.07	=	24
	2 Pls.	24¼ × 7/16	×	2'-2	@	36.07	=	157
	16 Lac. bars.	2¼ × 3/8	×	2'-1¼	@	2.87	=	101
								1938 × 4 = 7752#

Top chord U_1-U_2 4-Required

Cov. Pl.	1 Pl.	19 × 3/8	×	17'-10⅝	@	24.23	=	433
	2 [s.	12 × 3½	×	17'-10⅝	@	26.10	=	933
	1 Pl.	17½ × 3/8	×	1'-7	@	22.31	=	35
	1 Pl.	16 × 7/16	×	2'-2½	@	23.80	=	53
	2 Ls.	3½ × 3 × 3/8	×	2'-2½	@	7.9	=	35
	1 Pl.	13½ × 5/16	×	2'-8	@	14.34	=	38
	1 filla	2¾ × 1/16	×	1'-4⅛	@	0.584	=	1
	1 "	13⅝ × 1/16	×	1'-4	@	2.90	=	4
	2 "	3½ × ½	×	1'-11	@	5.95	=	23
	1 Pl.	19 × 3/8	×	1'-8½	@	24.23	=	41
	2 Pls.	10½ × ½	×	1'-1⅞	@	17.85	=	43
	2 Pls.	34 × 7/16	×	3'-3½	@	139.0	=	334
	16 bars.	2¼ × 3/8	×	2'-1¼	@	2.87	=	96
								2,069 × 4 = 8,276#

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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CALCULATIONS FOR

List of materials for Kasumi-Bashi. Okayama-Ken

Top chord. U2-U3 4-Required
Cov. Pl.

1 Pl.	19 × 7/16	×	17'-8½	@	28.26	-	500
2 [.	12 × 3½	×	17'-8½	@	26.10	-	925
2 Pl.	17½ × 3/8	×	1'-7	@	22.31	-	70
1 Pl.	16 × 7/16	×	2'-4½	@	23.80	-	57
2 L.	3½ × 3 × 3/8	×	2'-4½	@	7.9	-	38
1 Pl.	13½ × 5/16	×	2'-9½	@	14.34	-	40
1 Pl.	14 × 5/16	×	1'-11	@	14.88	-	29
1 Pl.	3½ × ½	×	1'-11	@	5.95	=	11
2 Pls.	10¼ × 3/8	×	0'-10¾	@	13.07	=	23
2 Pls.	24 × 7/16	×	2'-2	@	35.70	-	155
16 bars.	2¼ × 3/8	×	2'-1¼	@	2.87	-	97

1945 × 4 = 7.780#

Top chord U3-U4 4-Required
Cov. Pl.

1 Pl.	19 × 7/16	×	17'-7½	@	28.26	=	500
2 [s.	12 × 3½	×	17'-7½	@	26.10	-	920
2 Pls.	17½ × 3/8	×	1'-7	@	22.31	-	70
1 Pl.	16 × 7/16	×	2'-5	@	23.80	-	58
2 Ls.	3½ × 3 × 3/8	×	2'-7½	@	7.9	-	42
1 Pl.	13½ × 5/16	×	2'-10	@	14.34	-	41
2 Pl.	3½ × 5/8	×	3'-2½	@	7.44	-	48
2 Pls.	10¼ × 3/8	×	1'-4¼	@	13.07	=	35
2 fill.	7¾ × 1/8	×	0'-10	@	3.294	-	5
2 Pls.	28½ × 7/16	×	2'-10	@	42.40	-	204
16 bars.	2¼ × 3/8	×	2'-1¼	@	2.87	-	97
2 "	"	×	1'-10¼	@	2.87	-	11

2031 × 4 = 8.124#

Top chord U4-U5 2-Required
Cov. Pl.

2 Pls.	19 × 7/16	×	17'-6¾	@	28.26	993
4 [.	12 × 3½	×	17'-6¾	@	32.88	2309
4 Pls.	17½ × 3/8	×	1'-7	@	22.31	141
1 Pl.	16 × 7/16	×	2'-4½	@	23.80	57
2 L.	3½ × 3 × 3/8	×	2'-4½	@	7.9	38
1 Pl.	13½ × 5/16	×	2'-9½	@	14.34	40
1 Pl.	14 × 5/16	×	3'-2	@	14.88	47
1 Pl.	3½ × 5/8	×	3'-2	@	7.44	24
2 Pl.	10⅛ × 3/8	×	0'-10¾	@	12.91	23
2 "	24 × 7/16	×	2'-2	@	35.9	155
32 bars	2¼ × 3/8	×	2'-1¼	@	2.87	193

4020 × 2 = 8.040#

Bottom chord. L0-L1 4-Required.

2 [.	12 × 3½	×	15'-11¾	@	26.1	834
1 Pl.	25½ × 5/16	×	2'-3½	@	27.09	62
2 Pl.	10¾ × 5/16	×	1'-0½	@	11.42	24
4 Pl.	10 × 5/16	×	0'-10¾	@	9.6	38

958 × 4 = 3.832#

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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List of materials for Kasumi-Bashi.

Bottom chord. L_1-L_3 4-Required

2 [s.	12 × 3½ ×	31′0	@	32.88	2,039
2 Ps.	34½ × 7/16 ×	3′7½	@	51.32	373
8 Ls.	3½ × 3½ × 5/16 ×	0′8½	@	7.2	41
2 Ps.	8½ × 5/16 ×	0′9½	@	9.03	14
1 Ps.	27½ × 5/16 ×	24′0½	@	29.22	84
3 Ps.	10¾ × 5/16 ×	1′0½	@	11.42	36
10 Ps.	10 × 5/16 ×	0′10¾	@	10.63	96
1 Ps.	19 × 3/8 ×	2′1¼	@	24.23	51
1 Ps.	19 × 3/8 ×	2′8¾	@	〃	66
1 Ps.	25½ × 5/16 ×	2′6	@	27.09	68
2 Ps.	12 × 9/16 ×	2′6½	@	22.95	117
2 Ps.	10½ × 3/8 ×	2′4	@	13,39	62

3047 × 4 = 12,188#

Bottom chord. L_3-L_4 4-Required

2 [s.	12 × 3½ ×	25′6	@	32,88	1,677
2 Ps.	10 × 3/8 ×	21′5¾	@	12,75	548
2 Ps.	31½ × 3/8 ×	2′10	@	40.17	228
8 Ls.	3½ × 3½ × 5/16 ×	0′8½	@	7.2	41
2 Ps.	8½ × 5/16 ×	0′8¾	@	9.03	13
1 Ps.	23½ × 5/16 ×	2′4¼	@	24.97	59
3 Ps.	10¾ × 5/16 ×	1′0½	@	11.42	36
6 Ps.	10 × 5/16 ×	0′10¾	@	10.63	58
1 〃	19 × 3/8 ×	2′8¾	@	24.23	66
1 〃	19 × 3/8 ×	2′1¼	@	〃	51
1 〃	23½ × 5/16 ×	2′4½	@	24.97	59
2 〃	12 × 9/16 ×	2′6½	@	22.95	117
2 〃	10½ × 3/8 ×	2′4	@	13,39	62
2 〃	10 × 3/8 ×	1′3½	@	12.75	33

3048 × 4 = 11,792#

Bottom chord L_4-L_4 2-Required.

2 [s.	12 × 3½ ×	29′0½	@	32.88	1910
2 Ps.	10 × 3/8 ×	29′0½	@	12,75	741
2 Ps.	10¾ × 5/16 ×	1′0½	@	11.42	24
8 Ps.	10 × 5/16 ×	0′10¾	@	10.63	77
2 Ps.	29½ × 3/8 ×	2′10	@	37.62	213
4 Ps.	3½ × 3½ × 5/16 ×	0′8½	@	7.2	20
1 Ps.	8½ × 5/16 ×	0′8¾	@	9.03	7
1 Ps.	23½ × 5/16 ×	2′4¼	@	24.97	59

3051 × 2 = 6,102#

Diagonal U_0L_1 4-Required

2 [s.	12 × 3½ ×	24′7¾	@	32.88	1620
4 Ps.	10 × 5/16 ×	0′11½	@	10,63	41
112 bars.	2 × 5/16 ×	0′10¾	@	2.13	213

1,874 × 4 = 7,496#

Diagonal L_1-U_2 4-Required

2 [s.	12 × 3½ ×	30′5	@	32.88	2 000
4 Ps.	10 × 5/16 ×	0′11½	@	10.63	41
152 bars.	2 × 5/16 ×	0′10¾	@	2,13	290

2331 × 4 = 9,324#

Diagonal U_2-L_3 4-Required

2 [s.	12 × 3½ ×	30′8	@	26.10	1601
4 Ps.	10 × 5/16 ×	0′11½	@	10.63	41
158 bars.	2 × 5/16 ×	0′10¾	@	2.13	302

1944 × 4 = 7,776#

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List of materials for Kasumi Bashi.

Diagonal L3-U4 4-Required						
	2 [.	12 × 3½	× 33'-11¾	@ 26.1	1774	
	4 Pl.	10 × 5/16	× 0'-11½	@ 10.63	41	
	176 bars.	2 × 5/16	× 0'-10¾	@ 2.13	336	
					2151 × 4 =	8,604#
Diagonal U4-U5 4-Required						
	2 [.	12 × 3½	× 34'-0¼	@ 26.1	1776	
	4 Pl.	10 × 5/16	× 0'-11½	@ 10.6	41	
	178 bars.	2 × 5/16	× 0'-10¾	@ 2.13	340	
					2157 × 4 =	8,628#
Vertical U1-L1 4-Required						
	2 [.	10 × 3½	× 23'-0¼	@ 23.55	1081	
	4 Pls.	10 × 5/16	× 0'-11½	@ 10.63	41	
	4 L.	3½ × 3 × 5/16	× 1'-9	@ 6.6	46	
	4 L.	〃	× 1'-0½	@ 〃	27	
	1 Pl.	10 × 5/16	× 1'-9	@ 10.63	19	
	1 Pl.	〃	× 1'-0½	@ 〃	11	
	110 bars.	2 × 5/16	× 0'-10¾	@ 2,13	211	
					1436 × 4 =	5,744#
Vertical U2-L2 4-Required						
	4 L.	5 × 3 × 5/16	× 26'-4⅜	@ 8.2	865	
	1 Pl.	10 × 5/16	× 0'-10	@ 10.63	9	
	1 Pl.	14¼ × 5/16	× 1'-8¼	@ 15,15	26	
	1 Pl.	14½ × 5/16	× 1'-9	@ 15,41	27	
	1 Pl.	10 × 5/16	× 0'-11½	@ 10.63	10	
	35 bars.	2 × 5/16	× 0'-11⅜	@ 2.13	71	
					1008 × 4 =	4,032#
Vertical U3-L3. - 4-Required						
	2 [.	10 × 3½	× 28'-9¼	@ 23.55#	1350	
	4 Pl.	10 × 5/16	× 0'-11½	@ 10.63	41	
	1 Pl.	10 × 5/16	× 1'-0½	@ 10.63	11	
	1 Pl.	〃	× 1'-9	@ 〃	19	
	4 L.	3½ × 3 × 5/16	× 1'-9	@ 6.6	46	
	4 L.	〃	× 1'-0½	@ 〃	27	
	144 bars.	2 × 5/16	× 0'-10¾	@ 2,13	276	
					1770 × 4 -	7,080#
Vertical U4-L4 4-Required						
	4 L.	5 × 3 × 5/16	× 30'-2½	@ 8.2	986	
	1 Pl.	10 × 5/16	× 0'-10	@ 10.63	9	
	1 Pl.	15¾ × 5/16	× 1'-8¼	@ 16.74	28	
	1 Pl.	14½ × 5/16	× 1'-9	@ 15.41	27	
	1 Pl.	10 × 5/16	× 0'-11½	@ 10.63	10	
	41 bars.	2 × 5/16	× 0'-11⅜	@ 2.13	83	
					1143 × 4 -	4,572#
Vertical U5-L5 2-Required						
	2 [.	10 × 3½	× 30'-8¼	@ 23.55	1446	
	4 Pl.	10 × 5/16	× 0'-11½	@ 10.63	41	
	4 L.	3½ × 3 × 5/16	× 1'-9	@ 6.6	46	
	4 L.	〃	× 1'-0½	@ 〃	27	
	1 Pl.	10 × 5/16	× 1'-0½	@ 10.63	11	
	1 Pl.	〃	× 1'-9	@ 〃	19	
	156 bars	2 × 5/16	× 0'-10¾	@ 2.13	30	
					1620 × 2 -	3,240#

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CALCULATIONS FOR

List of material for Kasumi-Bashi:

Portal Bracing 2-Req'd.

2 Ls	5 × 4 × 3/8	×	22'-7"	@	11.0	=	497
2 Ls	5 × 4 × 3/8	×	25-3	@	"	=	556
4 Ls	3 × 3 × 5/16	×	2-11½	@	6.1	=	72
4 Ls	3 × 3 × 5/16	×	1-5¾	@	"	=	36
4 Fills	3 × 3/8	×	0'-9"	@	3.83	=	11
4 Ls	3 × 2½ × 5/16	×	2-8	@	5.6	=	60
4 Ls	"	×	2-5	@	"	=	54
4 Ls	"	×	2-4¼	@	"	=	53
4 Ls	"	×	2-4	@	"	=	52
4 Ls	"	×	2-4	@	"	=	52
4 Ls	"	×	2-1	@	"	=	46
4 Ls	"	×	1-11	@	"	=	43
4 Ls	"	×	1-10½	@	"	=	42
4 Ls	"	×	1-9¾	@	"	=	40
2 Ls	"	×	1-9½	@	"	=	20
2 Pls	10¼ × 5/16	×	1-1¼	@	10.89	=	24
4 Pls	8½ × 5/16	×	0-11½	@	9.03	=	35
10 Pls	9 × 5/16	×	0-11½	@	9.56	=	92
1 Pl.	5½ × 5/16	×	0-8	@	5.84	=	4
2 Pls	34 × 5/16	×	2-11½	@	36.13	=	214
1 Pl.	9 × 5/16	×	1-3	@	9.56	=	12

2,015 × 2 = 4,030#

Sway Bracing SB1 2 Req'd.

2 Ls	5 × 4 × 3/8	×	22'-3½"	@	11.0	491
2 Ls	"	×	23'-0½	@	"	507
2 "	2½ × 2 × 0.3	×	23'-0"	@	4.28	197
2 "	"	×	4-0	@	"	34
10 "	"	×	4-3¾	@	"	184
10 "	"	×	2-10¼	@	"	122
22 "	"	×	3-8	@	"	345
2 "	3½ × 3 × 3/8	×	0-10⅝	@	7.9	14
2 "	"	×	0-10½	@	"	14
10 "	2½ × 2 × 0.3	×	1'-0"	@	4.28	43
2 Pls	10¾ × 5/16	×	1-1"	@	11.42	25
5 "	8 × 5/16	×	1-2¾	@	8.50	53
5 "	7 × 5/16	×	1-1½	@	7.44	42
11 "	6¾ × 5/16	×	1-11¼	@	7.17	153
11 "	5 × 5/16	×	0-8	@	5.31	40
6 Washers	3"φ × 5/16	×	—			4
22 "	2½"φ × 5/16	×	—			9

2,277 × 2 = 4,554#

Sway Bracing SB2 2-Req'd.

2 Ls	5 × 4 × 3/8	×	22'-3½"	@	11.0	491
2 "	"	×	23-0½	@	"	507
2 "	2½ × 2 × 0.3	×	23-0	@	4.28	197
2 "	"	×	4-5⅜	@	"	38
10 "	"	×	4-10⅝	@	"	209
10 "	"	×	3-7	@	"	153
22 "	"	×	5-7¾	@		532
4 "	3½ × 3 × 3/8	×	0'-10¾	@	7.9	28

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CALCULATIONS FOR

List of materials for Kasumi-Bashi.

	10 Ls 2 1/2 × 2 × 0.3 ×	1'-0"	@	4.28	43	
	2 Pls 11 7/8 × 5/16 ×	1'-1"	@	12.61	27	
	5 " 8 × 5/16 ×	1-1	@	8.50	46	
	5 " 7 1/4 × 5/16 ×	1-0	@	7.7	39	
	11 " 6 3/4 × 5/16 ×	1-11 1/4	@	7.17	153	
	11 " 5 × 5/16 ×	0-8	@	5.31	40	
	6 Washers 3"φ × 5/16 ×	—			4	
	22 " 2 1/2"φ × 5/16 ×	—			9	
					2,516 × 2 = 5,032#	
Top Lateral Strut ST1 5-Req'd.						
	4 Ls 4 × 3 × 5/16 ×	22-3 5/8	@	7.2	642	
	2 Pls 11 1/2 × 5/16 ×	1-0 1/2	@	12.22	25	
	1 " 10 × 5/16 ×	0-11 1/2	@	10.63	10	
	28 Lac. Bars 2 × 5/16 ×	1-0 7/8	@	2.13	64	
					741 × 5 = 3705	
Top Lateral Bracing 2-Req'd.						
	2 Ls 5 × 3 × 0.3 ×	27-4 3/4	@	7.85	430	TB1R
	1 Pl. 18 × 5/16 ×	1-9 3/4	@	19.13	35	
	2 Ls 3 × 3 × 5/16 ×	13'-5 1/4	@	6.1	164	TB2
	2 " " ×	13'-3 1/4	@	"	162	TB3
	2 " 5 × 3 × 0.3 ×	27-5 1/2	@	7.85	431	TB4R
	4 " 3 × 3 × 5/16 ×	13-4 3/8	@	6.1	326	TB5
	1 Pl. 18 × 5/16 ×	1-9 3/4	@	19.13	35	
	2 Ls 5 × 3 × 0.3 ×	27-4 3/8	@	7.85	429	TB6R
	1 Pl. 18 × 5/16 ×	1-9 3/4	@	19.13	35	
	4 Ls 3 × 3 × 5/16 ×	13-3 3/4	@	6.1	325	TB7
	2 " 5 × 3 × 0.3 ×	27-3 5/8	@	7.85	428	TB8R
	1 Pl. 18 × 5/16 ×	1-9 3/4	@	19.13	35	
	4 Ls 3 × 3 × 5/16 ×	13-3 1/2	@	6.1	324	TB9
	2 " 5 × 3 × 0.3 ×	27-3 1/4	@	7.85	428	TB10R
	1 Pl. 18 × 5/16 ×	1-9 3/4	@	19.13	35	
	4 Ls 3 × 3 × 5/16 ×	13-3 1/4	@	6.1	324	TB11
					3,946 × 2 = 7,892#	
Floor Beam FB1 2 Req'd.						
Web Pl.	1 Pl 27 × 5/16 ×	22'-10 1/2"	@	28.69	656	
~~Flag~~ Flg. Ls	4 Ls 4 × 3 × 3/8 ×	22-10 1/2	@	8.5	778	
	4 " 3 × 3 × 3/8 ×	2-2 3/4	@	7.2	64	
	4 Fills 3 × 3/8 ×	1-9 1/4	@	3.83	27	
	16 Ls 3 × 3 × 5/16 ×	2-2 3/4	@	6.1	218	
	5 " 4 × 3 × 3/8 ×	0-6"	@	8.5	22	
	2 Pl. 7 1/2 × 3/8 ×	0-10 1/4	@	9.56	16	
	1 " 7 1/2 × 3/8 ×	0-9	@	"	7	
Cov. Pl.	2 Pls 8 1/2 × 3/8 ×	17-3 3/4	@	10.84	376	
					2,164 × 2 = 4,328	
Floor Beam FB2 & FB3 9-Required.						
Web Pl.	1 Pl. 27 × 5/16 ×	22'-11 1/2	@	28.69	659	
Flg. Ls.	4 Ls. 5 × 3 × 3/8 ×	22'-11 1/2	@	9.8	900	
Cov. Pls.	2 Pls. 10 1/2 × 3/8 ×	17'-3 3/4	@	13.39	464	
	4 Ls. 3 1/2 × 3 1/2 × 3/8 ×	2'-2 3/4	@	8.5	76	
	4 fill. 3 1/2 × 3/8 ×	1'-9 1/4	@	4.46	32	
	16 Ls. 3 × 3 × 5/16 ×	2'-2 3/4	@	6.1	218	
	10 Ls. 4 × 3 × 3/8 ×	0'-6	@	8.5	43	
	4 Pls. 7 1/2 × 3/8 ×	0'-10 1/4	@	9.56	32	
	2 Pls. " ×	0'-9	@	"	14	
					2438 × 9 = 21,942#	

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CALCULATIONS FOR

List of materials for Kasumi-Bashi.

Stringers 1-Required

20 I.	12×5	×	17'5	@	31.99	11146
20 I.	12×6	×	17'4½	@	44.2	15340
10 I.	〃	×	〃	@	〃	7670
80 L.	6×3½×3/8	×	0'7½	@	11.7	585
120 L.	〃	×	0'7¾	@	〃	905
						35646×1 = 35,646#

Bracing Hangers. H1 10-Required

1 L.	6×3½×3/8	×	0'7	@	11.7	7
1 L.	2½×2×0.3	×	1'0½	@	4.28	4
1 Pl.	6½×5/16	×	0'7¾	@	6.91	5
						16×10 = 160#

Bottom lateral Bracing 2-Required

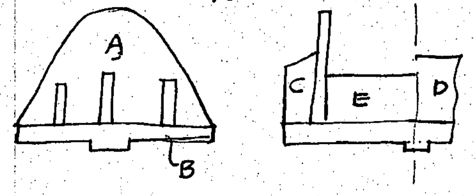
LB.1.3.5	10 L.	4×3×0.3	×	27'6	@	6.84	1881
LB 2.4.6	20 L.	〃	×	13'4¾	@	〃	1834
	1 Pl.	23¾×5/16	×	2'6	@	25.24	63
	1 Pl.	20¾×5/16	×	2'1¾	@	22.05	47
	3 Pl.	17¾×5/16	×	1'9¾	@	18.86	102
							3927×2 = 7,854#

Pin and Roller 1-Reqd.

4	Pins	5¼"φ	×	1'8½	@	73.6	496
8	nuts				@	6.2	50
10	Roller	4"φ	×	2'0	@	42.73	855
20	Bolts	¾"φ	×	0'1 13/16	@	0.25	5
4	Pls.	3×½	×	1'8	@	5.10	34
							1440×1 = 1,440#

Cast Steel Shoe & Bed Plate

RS1



$A = \frac{2\times2}{2}\times0.135\times2 = 0.5$

$B = 2.13\times2\times0.125 = 0.53$

$C = (0.375\times0.3\times0.08 + 0.21\times0.3\times0.08\times\frac{1}{2})\times6 = 0.072$

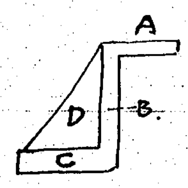
$D = 0.583\times0.08\times1.1\times2 = 0.103$

$E = 0.375\times0.08\times1.1 = 0.033$

1.238 ft³ @ 490 — 588×2 = 1,176#

720#
457
5

CG1



$A = 0.05\times0.1\times2 = 0.01$

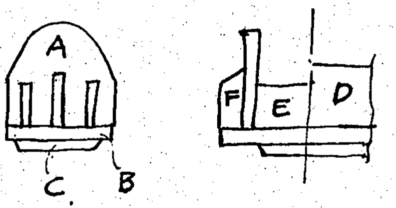
$B = 0.417\times0.05\times2 = 0.042$

$C = 0.312\times0.05\times2 = 0.031$

$D = 0.47\times0.26\times0.5\times2\times\frac{1}{2} = 0.006$

0.089 ft³ @ 490 — 41×4 = 164#

BP1 — $2\times2.75\times0.125 = 0.688$ ft³ @ 490 — 337×2 = 674#



$A = (0.458\times2\times0.135 + 2\times1.75\times0.135\times\frac{1}{2})\times2 = 0.69$

$B = 2\times1.96\times0.125 = 0.49$

$C = 1.64\times0.167\times0.135\times2 = 0.074$

$D = 1.04\times1.1\times0.083\times2 = 0.19$

$E = 0.83\times1.1\times0.083 = 0.076$

$F = (0.458\times0.292\times0.08\times + 0.42\times0.292\times0.08\times\frac{1}{2})\times6 = 0.094$

1.614 ft³ @ 490 — 784×2 = 1,568#

Anchor Bolts

16	Bolts	1¼"φ	×	2'0	@	8.172	133#
16	nuts				@	0.9	14
							147#

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CALCULATIONS FOR

List of materials for Kasumi-Bashi:

Summary of weight for one span (Truss span)

Trusses	154,602#	69.018 Tons.
Portals, Sways & struts	17,321	7.733
Top Bracings	7,892	3.523
Floor Beams	26,270	11.728
Stringers	35,646	15.913
Bottom Bracings	8,014	3.578
Pins, Rollers, Shoe & Anchs.	5,169	2.308
	254,914	113.801

Summary of weight, Rivets head for one span

Shop Rivets	3/4"φ	5780#	2.580 Tons.
Field "	3/4"φ	2,667#	1.191
Shop "	5/8"φ	72	0.032
Field "	5/8	123	0.055
		8,642	3.858

Grand summary for one span

254,914#	113.801 Tons.
8,642	3.858
263,542	117.659

Total grand summary for 7-Spans

117.659 Tons × 7 = 823.613 Tons.

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（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

List of materials for Kasumi-Bashi.

Main girders $G1_L$ & $G1A^R_L$ 4- Required

flange	4 Ls.	6 × 6 × ½	× 30'0	@ 19.60	= 2352
Web. Plate	1 Pl.	48 × ⅜	× 30'0	@ 61.2	= 1836
Stiffners	2 Ls.	5 3½ × ½	× 3'11½	@ 13.6	= 108
"	16 Ls.	5 × 3 × ⅜	× 3'11½	@ 9.8	= 621
Splice Pl.	2 Ls.	6 × 6 × ⅝	× 3'0	@ 24.2	= 145
"	1 Pl.	14 × ⅝	× 3'0	@ 29.75	= 89
"	1 Pl.	12 × ⅜	× 1'11½	@ 15.3	= 30
"	2 Pls.	6 × ½	× 3'0	@ 10.2	= 61
Cover Pl.	2 Pls.	14 × ½	× 19'0	@ 23.8	= 904
	5 fills	3½ × ½	× 3'0¼	@ 5.95	= 90
Connection	5 Ls.	4 × 3 × 5/16	× 0'7½	@ 7.2	= 23
Sole Pl.	2 Pls.	15 × ¾	× 1'9	@ 38.3	= 134
	1 Pl.	3 × ¼	× 1'3	@ 2.55	= 3

6396 × 4 = 25,584#

Main Girder 25,584 ÷ 2240 = 11.421 Tons.

Stringers S1 & S2. 1 Required

6 Is.	12 × 5	× 12'8¾	@ 31.99	= 2443
9 Is.	"	× 11'5½	@ "	= 3,299
24 Pls.	8½ × 5/16	× 0'11½	@ 9.03	= 185

5927 × 1 = 5,927#

Stringer 5,927 ÷ 2240 = 2.646 Tons.

Floor Beams FB1 2- Required

flange	2 Ls.	3 × 3 × 5/16	× 17'11	@ 6.1	= 219
"	2 Ls.	3 × 3 × 5/16	× 16'11¾	@ 6.1	= 207
Web Plate	1 Pl.	24 × 5/16	× 17'11	@ 25.5	= 457
Stiffner	2 Ls.	5 × 3½ × ⅜	× 1'11 2/8	@ 10.4	= 41
"	14 Ls.	3 × 2½ × 5/16	× 1'11⅜	@ 5.6	= 156
	2 fills	4¾ × 5/16	× 1'6¼	@ 5.05	= 15
	3 Pls.	8½ × ⅜	× 1'0⅜	@ 10.84	= 33

1128 × 2 = 2,256#

Floor Beam FB2 4- Required

flange	2 Ls.	3 × 3 × ⅜	× 17'11	@ 7.2	= 258
"	2 Ls.	3 × 3 × ⅜	× 16'11¾	@ 7.2	= 245
Web.	1 Pl.	24 × 5/16	× 17'11	@ 25.5	= 457
Stiff.	2 Ls.	5 × 3½ × ⅜	× 1'11¾	@ 10.4	= 41
"	14 Ls.	3 × 2½ × 5/16	× 1'11¾	@ 5.6	= 155
	2 fill.	4¾ × ⅜	× 1'6¼	@ 6.06	= 18
	3 Pls.	8½ × ⅜	× 1'0⅜	@ 10.84	= 33

1207 × 4 = 4828#

Floor Beams 7084 ÷ 2240 = 3.163 Tons.

Bottom lateral Bracing 5-Required

2 Ls.	4 × 3 × 5/16	× 19'10⅝	@ 7.2	= 286
2 Ls.	"	× 9'5⅝	@ "	= 136
2 Ls.	"	× 9'6⅞	@ "	= 138
1 Pl.	"	× 2'1⅛	@ 6.38	= 13

573 × 5 = 2,865#

Lateral gusset Plates GP1 & GP2 1-Required

4 Pls.	13¼ × 5/16	× 1'3	@ 14.08	= 70
8 Pls.	"	× 1'8½	@ "	= 193

263 × 1 = 263#

Bracing 3,128 ÷ 2240 = 1,396 Tons.

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CALCULATIONS FOR

List of materials for Kasumi-Bashi.

Cast steel Bed Plate 1-Required
2 cast steel Plate @ 121 242
2 " " " @ 117 234
476×1= 476#

Anchor Bolts 1-Required
12 Bolts 1¼"φ × 2'-0 @ 4.172 100
12 nuts @ 0.9 11
111×1= 111#

Bed Pts. & Anch. 587÷2240= 0.262 Tons.

Summary of Weight for one span (Girder span)

Main girders	25,024#	11.421 Tons
Stringers	5,927	2.646
Floor Beams	7,084	3.163
Bott. Bracings	3,128	1.396
Bed Pts. & Anchor Bolts	587	0.262
	42,310	18.888

Summary of Rivet head Weight for one span

Shop Rivets	7/8"φ	634#	0.283 Tons
Field "	7/8"φ	350	0.156
Shop "	3/4"φ	293	0.131
Field "	3/4"φ	113	0.050
		1,390	0.620

Grand Summary for one span

42,310#	18.888 Tons.
1,390	0.620
43,700	19.508

Edge steel for Expansion joint ES1 22-Required
10 Bolts 3/4"φ × 0'-2½ @ 0.69 7
1 L. 3×3×5/16 × 21'-0 @ 6.1 128
1 fill. 3×½ × 0'-5¼ @ 5.1 2
2 fills 3×1 × 1'-2½ @ 10.2 25
162×22= 3.564#
3.564÷2240= 1.591 Tons.

Total grand Summary for 13-Spans

19.508 × 13 = 253.604 Tons.
1.591
255.195

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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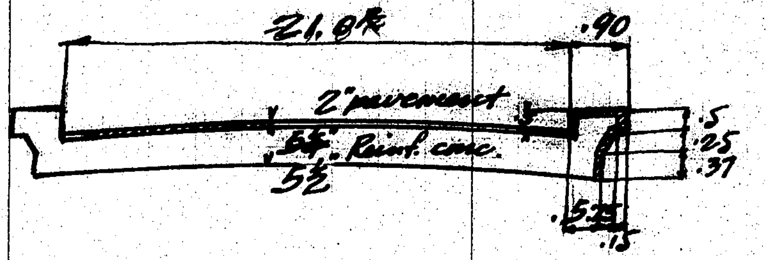
CALCULATIONS FOR

List of materials for Kasumi-Bashi

Materials for super structure

concrete (1:2:4) for slab and copings.



Sectional area 5½" slab and copings.

Slab 21.0×.46 = 9.66 立方尺

copings $.5\times.9+\frac{.5+.75}{2}\times.25+.5\times.37\times2$ = 1.86 "

11.52 "

表面仕上 −0.11 "

11.41 "

Concrete volume

Slab A 11.41×53.81 = 613.0 立方尺 slab and copings

$\frac{1}{3}\times20.0\times.23\times3.5$ = 5.4 " filling on floor beams

618.4 立方尺 or 2.86 立坪

14 @ 2.86 = 40.0 立坪

Slab B 11.41×70.41 = 803.4 立方尺

$\frac{1}{3}\times20.0\times.23\times4$ = 6.1 "

809.5 " or 3.75 立坪

7 @ 3.75 = 26.25 立坪

Slab C 11.41×60.35 = 688.6 立方尺 or 3.19 立坪

13 @ 3.19 = 41.47 立坪

107.72 立坪

Area of pavement (2" thickness)

Slab A	21.0×53.81 = 1130.0 平方尺	or 31.40 面坪	14@31.40	= 439.6 面坪
Slab B	21.0×70.41 = 1478.6 "	or 41.07 "	7@41.07	= 287.5 "
Slab C	21.0×60.35 = 1267.3	or 35.21 "	13@35.21	= 457.8 "
				1184.9 "

Expansion jt. (+1"×13 − 8"×7)×21尺 = −3.5"×21尺 = −73.5 平方尺 or −2.0 "

1182.9 面坪

Area of copings finish

Slab A	2.6×2×53.81 = 279.8 平方尺	or 7.8 面坪	14@ 7.8	= 109.2 面坪
Slab B	2.6×2×70.41 = 366.0 "	or 10.2 "	7@ 10.2	= 71.4 "
Slab C	2.6×2×60.35 = 314.0 "	or 8.7 "	13@ 8.7	= 113.1 "
				293.67 面坪

Reinforcement in slab (See sheet No. 2)

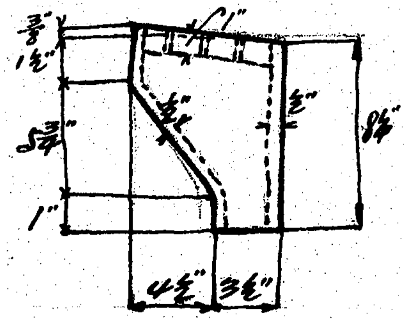
Slab A 14 @ 1.674 = 23.44 tons

Slab B 7 @ 2.140 = 14.98 "

Slab C 13 @ 1.966 = 25.56 "

63.98 tons

Drains 136 Req'd



$\{(1\frac{3}{8}''+7\frac{1}{4}+1)+8\frac{1}{4}''\}\times8\frac{1}{2}\times\frac{1}{2}''$ = 74.0 cub inch back and front

$\{\frac{\frac{3}{8}''\times7''}{2}+1\frac{1}{2}\times7''+\frac{7''+2\frac{1}{2}''}{2}\times5\frac{3}{4}''+1''\times2\frac{1}{2}\}\times2\times\frac{1}{2}''$

= (1.31+10.5+27.3+2.5)×2×.5 = 41.6 cub inch side

(1"×½"×3½×8)+(1×½"×7×1) = 17.5 " cover

133.1 cub inch ×0.26 = 35.#

136 @ 35. = 4760# or 2.12 tons.

CALCULATIONS FOR

List of materials for Kasumi-Bashi

Expansion Joint for truss spans 7 Req'd.

1-Pl. 9" × $\frac{3}{8}$" @ 11.48 × 20'-11" = 240#
1-L. 3 × 3 × $\frac{5}{16}$ @ 6.1 × 20'-11" = 128#
1-L 5 × 3 $\frac{5}{16}$ @ 8.2 × 20'-11" = 172"
2-Ls 2½ × 2½ × $\frac{5}{16}$ @ 5.0 × 0'-9" = 8"
22-Pls. 2½ × $\frac{5}{16}$ @ 2.66 × 1'-2" = 68"
616# or 0.275^{ton}
7 @ 0.275 = 1.93^{tons}

Expansion Joint for plate girder spans 13 Req'd

Handrails

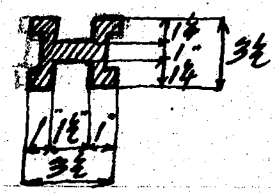
panel A 24 Req'd.
1-1½"φ gas pipe @ 2.717 #/ft × 5-6" = 15#
2-1"φ " " @ 1.678 × 5-6" = 18"
9 Bars 1½ × ¾ @ 3.83 × 2'-6" = 86" 119# 24 @ 119# = 2856.#

panel B 288 Req'd
1-1½"φ Gas pipe @ 2.717 × 4'-10½" = 13#
2-1"φ " " @ 1.678 × 4-10½ = 16#
9 Bars 1½ × ¾ @ 3.83 × 2-6" = 86# 115# 288 @ 115" = 33120.#

panel C 490 Req'd.
1-1½"φ Gas-pipe @ 2.717 × 4'-9⅞" = 13#
2-1" " " @ 1.678 × 4'-9⅞ = 16#
9 Bars 1½ × ¾ @ 3.83 × 2-6" = 86# 115# 490 @ 115" = 56350.#

panel D 14 Req'd.
1-1½" gas-pipe @ 2.717 × 5-5⅜ = 15#
1-1" " " @ 1.678 × 5-5⅜ = 18"
9 Bars 1½ × ¾ @ 3.83 × 2-6" = 86" 119# 14 @ 119" = 1666#
93,992.#
or 41.97^{tons}

Handrail posts 845 Req'd. (Cast iron)



approximate Volume
1½ × 1 × 33.0" = $49.5^{cub.in.}$
1" × 3½ × 72.0" = 252.0 "
8½ × 4½ × ¾ = 27.0 "
328.5" × $0.26^{\#/cub.in}$ = 85#
845 @ 85# = 71825.0#
or 32.06^{tons}

Anchor bolts and anchor plates
1690 Anch. bolts ½"φ × 0'-6" @ .4# = 676#
845 " plates 2½" × $\frac{5}{16}$ × 0'-8" @ 2.66 = 1502#
2178# or 0.97^{ton}

Lamp posts and its structural bracket

Lamp posts 11 Req'd
24" × ½" × 39" = $468.^{cub.in.}$
8" × 8" × 1 = 64. "
Foot casting 7" × 13⅜ = 94. "
7" × ¾ × 6 = 32. "
658 × 0.26# = 171#. 11 @ 171 = 1881# or 0.83^{ton}

1- 2½"φ gas pipe @ 5.79 × 3'-2" = 18#
1- 2"φ " " @ 3.65 × 3'-10" = 14"
1- 1"φ " " @ 1.678 × 1'-7" = 3"
35# 11 @ 35# = 385# 0.16^{ton}

CALCULATIONS FOR

List of materials for Kasumi Bridge.

Structural ~~Bracket~~

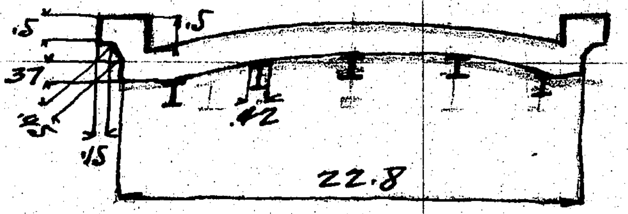
2-Ls	3 x 3 x 5/16 x 1'-9"	6.1 #/ft.	21#
2-Ls	3 x 3 x 5/16 x 2'-2¼"	"	27
2-Ls	3 x 3 x 5/16 x 2'-4½"	"	29
2-Ls	2½ x 2½ x 5/16 x 0'-7"	5.0 "	6
1-Webpl.	20½ x 3/8 x 2'-2"	26.14"	57
2-Fills.	3 x 5/16 x 1-2½"	3.19"	8
1- "	7 x ½ x 1-3½"	11.90"	15
1- "	6 x ¼ x 0'-7"	5.10"	3.
2- Bolts	½"φ x 0'-3"	.18 #/one	0.36
4-Anch.Bolts	½"φ x 0'-6"	.35"	1.4

168#   11@168# = 1848# or 0.825 ton

Summary of wt. lamppost and struc'l. bracket 374#   11@374 = 4114# or 1.84 tons

Forms

for truss spans



Copings
width (.5+.5+.15+.45+.37)x2 = 3.94
length 177'-0" = 178.00尺
area 3.94 x 178.0 = 701.0 平方尺
Slab
width 22.8-(5x.42) = 20.7尺
length 178.0尺
Area 20.7 x 178.0 = 3685.0 平方尺

4386.0" or 122.0 面坪

for girder spans

Copings
width 3.94尺
length 60'-0" = 60.35尺
area 3.94 x 60.35 = 239.0 平方尺

7@ 122 = 854.0 面坪

Slab
width 22.8-(1.0+1.0+.42x3) = ~~18.82~~ 19.54
length 60.35尺
Area 60.35 x 19.54 = 1179.0 平方尺

1418.0" or 39.39 面坪

13@39.39 = 512.1 面坪

1366.1 面坪

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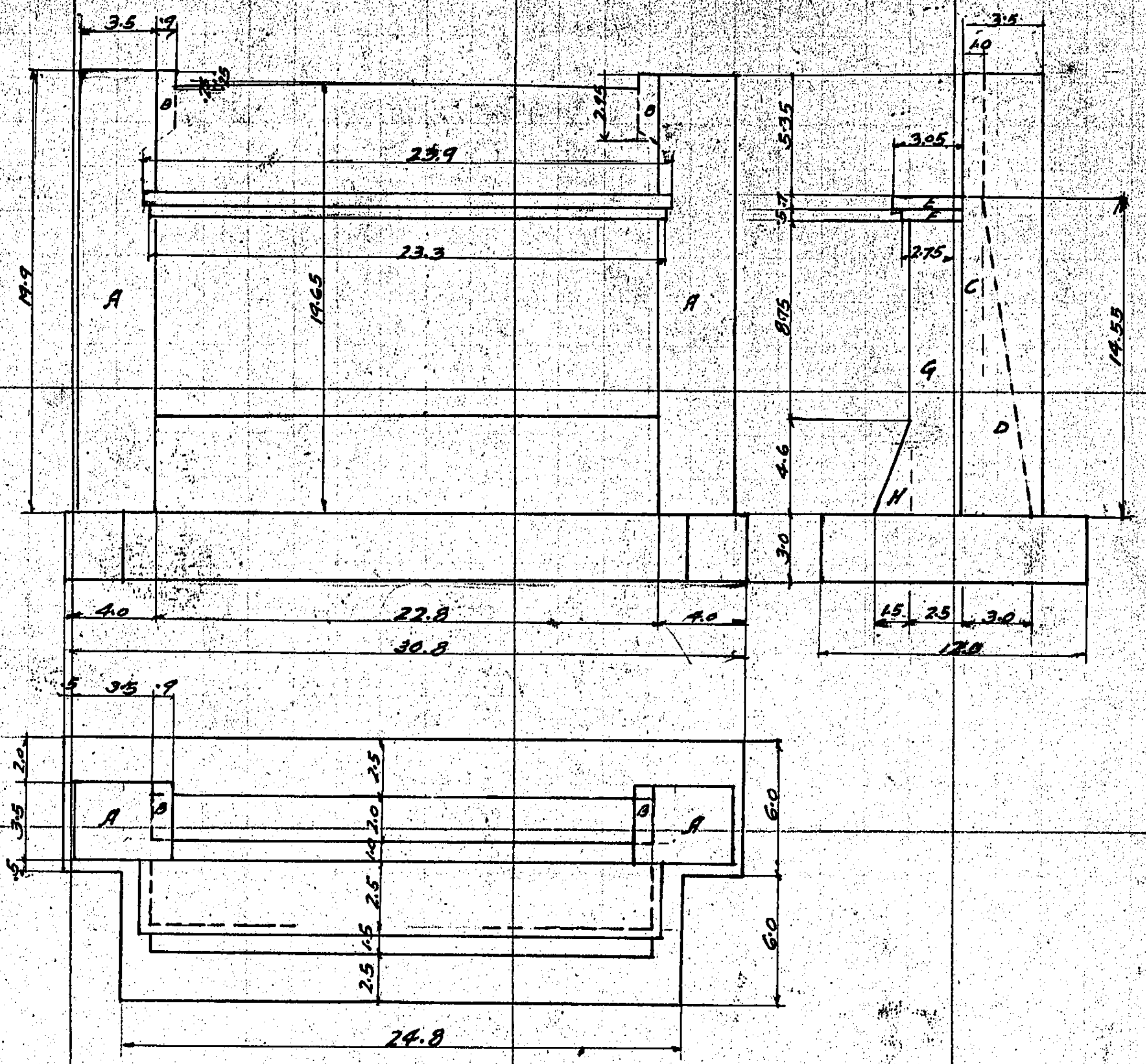
CALCULATIONS FOR

Materials of Substructure for Kasumi-Bashi, Okayama Ken

ABUTMENT A1 WITHOUT WING WALL AND HAND RAIL AND ORNAMENTS. 1-Req'd



Concrete 1:2:4

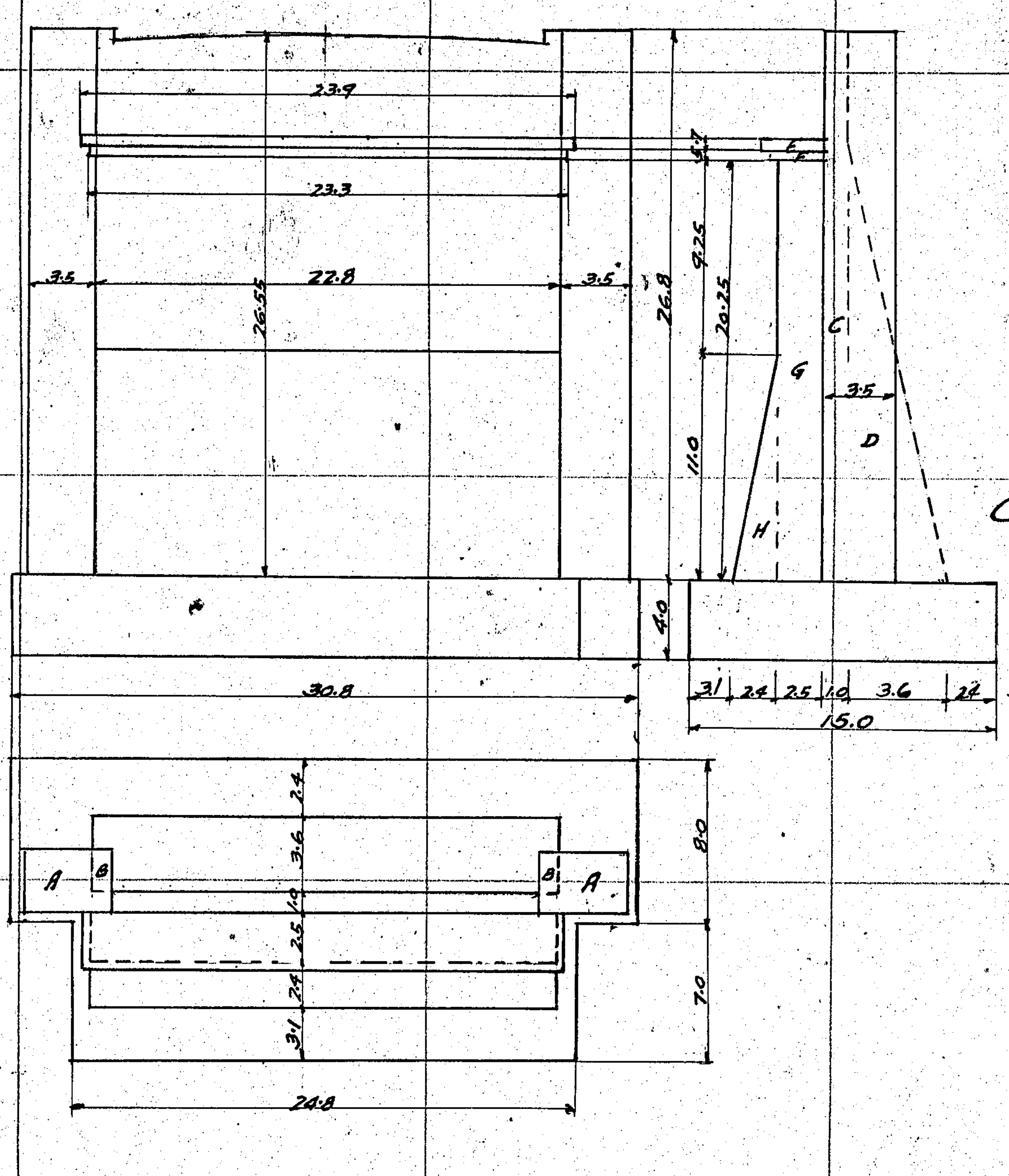
2-A 2×3.5² ×19.9 = 488.0
2-B 2×.9×2.5×2.95 = 13.3
C 1.0×19.6×22.8 = 446.5
D ½×14.55×2.0×22.8 = 330.5
E 3.05×.7×23.9 = 51.0
F 2.75×.5×23.3 = 32.0
G 2.5×13.35×22.8 = 761.0
H ½×1.5×4.6×22.8 = 78.6
2200.9 立方尺
or 10.2 立坪

Concrete 1:3:6

12.0×3.0×24.8 = 893.0
2× 6×3.0×3.0 = 108.0
1001.0 立方尺
or 4.63 立坪

Total 3201.9 立方尺 or 14.83 立坪

ABUTMENT A2 WITHOUT WING WALL & HAND RAIL AND ORNAMENTS. 1-Req'd.



Concrete 1:2:4

2-A 2×3.5² ×26.8 = 657.0
2 B 2×.9×2.5×2.95 = 13.3
C 1.0×26.5×22.8 = 604.0
D ½×3.6×21.45×22.8 = 880.0
E 3.05×.7×23.9 = 51.0
F 2.75×.5×23.3 = 32.0
G 2.5×20.25×22.8 = 1153.0
H ½×2.4×11.0×22.8 = 301.0
3619.3 立方尺
or 16.75 立坪

Concrete 1:3:6

15.×4.0×24.8 = 1488.0
2×4.0×8.0×3.0 = 192.0
1680.0 立方尺
or 7.78 立坪

Total 5299.3 立方尺 or 24.52 立坪

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CALCULATIONS FOR

Materials of Substructure for Kasumi-Bashi, Okayama-Ken

WING OF ABUTMENT. 2@2-Req'd Length of wing wall 9.96+1.4 = 11.36

Coping 11.36 × .5 × 1.4 = 7.95
Panel 11.36 × 1.0 × 8.5 = 96.56
11.36 × 2.5 × 1.0 = 28.40
132.91 × 2 = 265.82 立方尺 or 1.23 立坪 Concrete 1:2:4
× 2
2.46 立坪

PEDESTAL & HANDRAIL 2@2 - Req'd Length of handrail = 9.96

Pedestal 3.0 × 3.0 × 3.5 = 35.1
2.0 × 2.0 × 5.8 = 23.2
2@ Handrail posts 1.0 × 1.0 × 3.3 × 2 = 6.6
Top & Bott. rail 9.96 × .7 × .5 × 2 = 7.0
Panel 9.96 × .4 × 2.0 = 7.8
79.7 立方尺 × 2 = 159.4 立方尺 or 0.74 立坪 Concrete 1:2:4
× 2
1.48 立坪

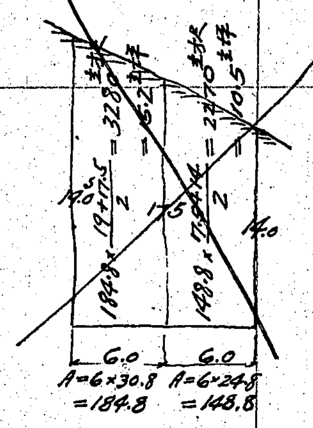
Area of form

Body of abutment A1 48.80
2@ Wing wall 2@ 7.76 = 15.52
2@ Pedestal & Railing 2@ 6.28 = 12.56
76.88 面坪

Body of abutment A2 65.20
2@ Wing wall 2@ 7.76 = 15.52
2@ Pedestal & Railing 2@ 6.28 = 12,56
93.28 面坪

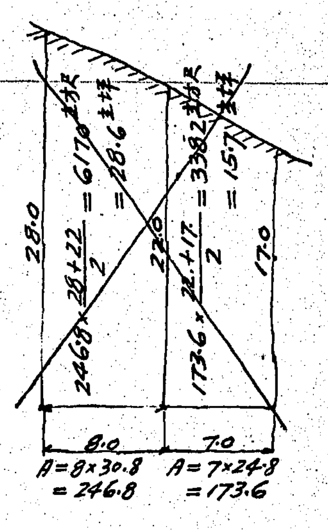
Excavation

Abutment A1



15.2
10.5
25.7 立坪

Abutment A2



28.6
15.7
44.3 立坪

Summery of Material for A1 & A2 each.

Abutment A1:—

Concrete 1:3:6 4.63 立坪
Concrete 1:2:4 Body 10.20
Wingwall 1.48
Pedestal &c. .74
} 12.42 立坪
12.42

~~Excavation~~ ~~25.7~~ 立坪

Form Body 48.80
Wingwall 15.52
Pedestal &c 12.56
} 76.88 面坪

Reinforcement Body 1.046
Other parts .142
} 1.188 tons as shown in the drawing sheet #

花崗石 12.6才
排水誘導用パイプ 2½" gas pipe 2本
モルタル仕上 111.44 平方尺 or 3.10 面坪
人造石仕上 374.42 平方尺 or 10.40 面坪

Materials of Substructure for Kasumi-Bashi, Okayama-Ken

Abutment A2 :–

Concrete 1:3:6			7.78 立坪	
Concrete 1:2:4	Body	16.75	18.97 立坪	
	Wingwall	1.48		
	Pedestal &c	.74		
~~Excavation~~			~~44.3~~ 立坪	
Form	Body	65.20	93.28 面坪	
	Wingwall	15.52		
	Pedestal &c	12.56		
Reinforcement	Body	1.583	1.741 tons	as shown in the drawing sheet #
	Other parts	.142		
Pile 生松丸太	末口六寸 長三間		44本	
花崗石		12.6才		
電纜誘導用パイプ 2吋 gas pipe		2本		
モルタル仕上		111.5 平方尺	or 3.1 面坪	
人造石仕上		374.5 〃	or 10.4 面坪	

CALCULATIONS FOR

Materials of Substructure for Kasumi-Bashi, Okayama-Ken

Materials for Piers
for GP1.
Concrete (1:2:4)

for shafts and coping 20.4 × 0.65 = 133
15.9 × 0.50 = 80
21.30 or 0.009 立坪 2@0.009 = 0.018 立坪

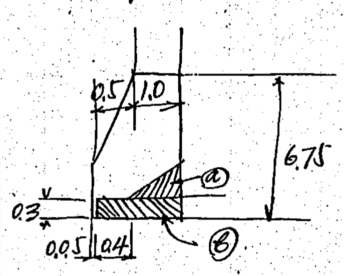
for cylinder. $V = \frac{h}{3} \times (B + b + \sqrt{B.b})$ where h = height of cylinder = 18.5
B = base area = 26.76
b = top area = 12.57
and $\sqrt{B.b} = \sqrt{26.76 \times 12.57} = 18.36$

$V = \frac{18.5}{3} \times (26.76 + 12.57 + 18.36)$ = 6.17 × 57.72 = 356.00 or 1.65 立坪
2@1.65 = 3.30 立坪

for Web
130 × 0.65. = 8.45
13.6 × 0.5 = 6.80
0.35 × 2.5 × 1.0 × 3 × 2 = 1.76
(coping)
$\frac{14.11 * 12.63}{2} \times 18.5 \times 1.5$ = 370.00
387.01 or 1.79 立坪

Summary of Volume for shaft and web.
coping 0.018
cylinder 3.30
web 1.79
5.108 立坪

for well (1:2:4) shell.



ⓐ $\frac{38.48 + 65.03}{2} \times 1.05 - 38.48 \times 1.05$ = 54.4 − 40.4 = 14.0
ⓑ 78.5 − 38.5 × 0.3 = 12.0
38.48 × 6.75 = 260.0

dia. 9.0 area 63.60
dia. 10.0 78.50

$\frac{78.5 + 63.60}{2} \times 6.75$ = 71.0 × 6.75 = 479.00
~~63.60 − 38.48 × 9.0 = 22.60~~

479.00
−14.00
−260.00
205.00 or 0.95 立坪

or 1.05
0.95
2.00 立坪 2@2.00 = 4.00 立坪

Summary of Volume in well.
(1:2:4) shell 4.00
(") filling 2.74
(1:4:8) " 3.22

" (1:2:4) filling 38.48 × 3 = 115.5
38.48 × 4 = 154.0
12.0
14.0
295.5 or 1.37 立坪 2@1.37 = 2.740 立坪

" (1:4:8) filling 38.48 × 9.0 = 346.00 or 1.61 立坪.
2@1.61 = 3.22 立坪

curb shoes e31
2Ls 5" × 3" × 0.3" @ 7.85 × 15'-7½" = 246#
2Pls. 12" × ¼" @ 10.20 × 15'-7½" = 319
2Pls. 4 × ¼ @ 3.40 × 0'-5¾" = 4
2Ls 8" × 3 × 0.3 @ 7.85 × 1'-4" = 22
32 bolts ½"φ × 1'-2 @ 0.91 = 29
620# or 0.28 ton

Forms (approx.)
for shafts. 17.21 × 20.0 × 2 = 684.4
" Web 13.50 × 20.0 × 2 = 540.0
1224.4 or 34.0 方坪

for Well 28.27 × 15.0 × 2 = 850.0 outside
22.00 × 16.4 × 2 = 723.0 inside
1573.0 or 44.0

Sum. of forms
shaft. Web 34.0 方坪
Well 44.0 "
78.0

CALCULATIONS FOR

Materials of Substructure for Kasumi-Bashi, Okayama Ken.

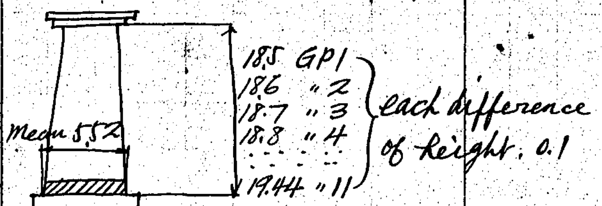
summary of Materials for Pier GP1 to GP5
(except concrete in shaft and web)

concrete (1:2:4) in shell of Wells 4.00 立坪
" (") in filling " " 2.74 "
" (1:4:8) " " " " 3.22 "
Curb shoes 2@0.28 = 0.56 ton
Reinforcement in shaft and web 0.695 "
in Wells 1.010
1.705 tons
Forms 78.00 面坪

concrete in Wells of Pier GP6 to GP10.
Conc. (1:2:4) for shell.
area 63.62-38.48 = 25.14
25.140 × 13.0 = 327 or 1.53 立坪
0.95
2.48 立坪
2@2.48 = 4.96 立坪

Conc. (1:2:4) 2@1.37 = 2.74 立坪
Con. (1:4:8) 38.48 × 13.0 = 500.0 or 2.32 立坪
2@2.32 = 4.64 立坪

Reinforcement 2@0.631 = 1.262 tons
Form 28.27 × 19.0 = 537.0 outside
22.00 × 20.4 = 450.0 inside
987.0 or 28.0 面坪
2@28.0 = 56.0 面坪



difference of Volume = 0.03 立坪

Conc. (1:2:4) in shaft and web for GP1 to GP11

GP1.	5.11 立坪
" 2.	5.14 "
" 3	5.17 "
" 4	5.20 "
" 5	5.23 "
" 6	5.26 "
" 7.	5.29 "
" 8	5.32 "
" 9	5.35 "
" 10.	5.38
" 11.	5.41 "

Summary of materials for pier GP6 to GP10
except conc, in shafts and web.

Conc. (1:2:4) for shell of Well 4.96 立坪
" (") " filling " " 2.74 "
" (1:4:8) " " " " 4.64 "
Curb shoes 2@0.28 = 0.56 ton
Reinforcement in shaft and web 0.695
" in Well 1.262
1.957 tons
Forms in shaft and web 34.0
in Wells 56.0
90.0 面坪

materials for Pier GP11

Conc. (1:2:4) in shell of wells
63.62-38.48 × 22.5 = 566.00 or 2.62 立坪
0.95
3.57 立坪
2@3.57 = 7.14 立坪

conc. (1:2:4) for filling in wells.
38.48 × 3.0 = 115.00 top
38.48 × 6.0 = 230.00 bottom
12.00
14.00
372.00 or 1.74 立坪
2@1.74 = 3.48 立坪

Conc. (1:4:8) for filling in wells.
38.48 × 20.5 = 772.00 or 3.57 立坪
2@3.57 = 7.14 立坪

Reinforcement in wells 2@1.033 = 2.066 tons.

CALCULATIONS FOR

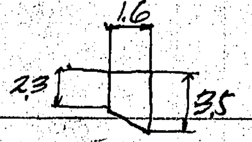
Materials of Substructure for Kasumi-Bashi, Okayama-Ken

Materials for pier TP1 and TP8
concrete (1:2:4) for shafts and web.
for shafts.
copings. 31.17 × 0.7 = 29.00
" 24.63 × .5 = 12.70
41.70 or 0.19 立坪
2@ 0.19 = 0.38 立坪

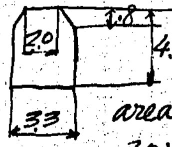
for cylinders. height = 21.59 R
B = base area = 35.57 平尺
b = top area = 19.64 "
and $\sqrt{B.b}=\sqrt{35.57\times19.64}=26.46$

Volume $V=\frac{21.59}{3}\times(35.57+19.64+26.46)$
$=7.2\times81.67=588.00$ 立尺 or 2.72 立坪
2@ 2.72 = 5.44 立坪

for web.
Coping 4.8 × 17.84 = 85.80
4.1 × 18.54 = 76.00
.65 × 3.0 × 1.0 × 3 = 5.85
167.65 立尺 or 0.78 立坪

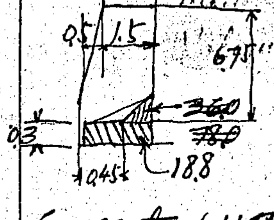


mean length 21.0 R
$\frac{2.3+3.5}{2}\times1.6\times21.0=93.00$
12.7 × 17.4 = 221.00
2.0 × 13.7 × 18.0 = 497.00
811.00 立尺 or 3.76 立坪



area
$\frac{2.0+3.3}{2}\times0.8=2.2$
3.2 × 3.3 = 10.5
12.7 平尺

Concrete (1:2:4) for wells.
for shell of wells
dia 12.0 area 113.10
dia 11.0 area 95.03
5.03 × 6.75 = 34.0
340.0
36.0
376.0



$\frac{113.1+95.0}{2}\times6.75=702.00$
376.00
326.0 or 1.51 立坪
98.0 − 50.3 × 20.0 = 894.0 or 4.14 "
4.14
1.51
5.65 立坪 2@ 5.65 = 11.30 立坪

Concrete (1:2:4) for wells filling.
95.0 − 50.3 = 44.70
44.7 × (3.0 + 6.0) = 402.5
18.8
78.0
499.3 or 2.31 2@ 2.31 = 4.62 立坪

for wells filling.
50.27 × 18.0 = 905.00 立尺 or 4.19 立坪
2@ 4.19 = 8.38 立坪

Curb shoes CS2
2-P/s 12" × ¼ @ 10.20 × 18'-9" = 383 #
2-Ls 5" × 3" × 0.3" @ 7.85 × 18'-9" = 295
2-P/s 4" × ¼" @ 3.40 × 0'-5¾" = 4
2-Ls 5" × 3" × 0.3" @ 7.85 × 1'-4" = 22
32-Bolts ½"φ × 1'-2" @ 0.91 = 29
733 #
Rivets head variation 2% 7
740 # or 0.33 ton
2@ 0.33 = 0.66 tons.

Forms.
28.27 × 28.5 = 807.00 平尺
22.00 × 29.9 = 660.00
1467.00 平尺 or 40.80 平坪
2@ 40.8 = 81.6 平坪

summary of materials for pier GP11.
(except conc in shafts and web)

Conc. (1:2:4) in wells 7.14 立坪
Conc. (1:3:6) in wells 3.48 "
in wells 7.14 "
curb shoes 2@ 0.28 = 0.56 tons
Reinforcement in shaft and web 0.695
in wells 2.066
2.761 tons
Forms. in shafts and web 34.0
in wells 81.6
115.6 平坪

———— o ————

summary of volume of conc. (1:2:4)
for shafts and web
copings 0.38 立坪
cylinders 5.44 "
web 3.76 "
9.58 立坪

Summary of volume of conc. for wells.
1:2:4 11.30 立坪
1:2:4 4.62 "
1:4:8 8.38 "

Reinforcement (see sheet no 9.)
in shafts. 2@ 600 = 1200 #
in web 814
2014 #
or 0.900 ton
in wells 2@ 2086 = 4172 #
or 1.86 tons.

Curb shoes.
2@ 0.33 = 0.66 tons

CALCULATIONS FOR

Materials of Substructure for Kasumi-Bashi, Okayama-Ken

Forms.

for shafts and Web. (approx.)

shafts 21.0 × 23.0 = 483.0 平方尺 or 13.5 平坪
2@13.5 = 27.0 平坪

Web 18.0 × 23.0 = 414.0 平方尺 or 11.5 平坪
2@11.5 = 23.0 平坪

Wells 34.5 × 26.0 = 900.0 outside
" 25.1 × 27.4 = 690.0 inside
1590.0 平方尺 or 44.0 平坪
2@44.0 = 88.0 平坪

Materials for Pier TP2 to TP7.

Concrete (1:2:4) for shafts and Web.

copings.
39.03 × 0.7 × 2 = 54.80 立方尺
31.67 × 0.5 × 2 = 31.40
17.09 × 0.7 × 3.3 = 39.50
17.69 × 0.5 × 2.7 = 24.00
.65 × 1.0 × 3.0 × 6 = 11.70
161.40 立方尺 0.75 立坪

Cylinders. height = 22.81 尺
B = base area = 45.13 平方尺
b = top area = 25.97
and $\sqrt{B.b} = \sqrt{45.13 \times 25.97} = 34.28$

Volume $V = \frac{22.81}{3} \times (45.13 + 25.97 + 34.28) = 800.00$ 立方尺 or 3.70 立坪
2@3.70 = 7.40 立坪

Web.

18.81 × 2.0 × 17.3 = 652.00
▨ 221.00
873.00 立方尺 or 4.04 立坪

Difference of Volume of conc. pier TP4 to TP7.

deff. of height TP4&5 to TP3&6 22.81
22.61
0.20 尺

deff. of ht. TP3&6 to TP2&7 22.61
22.20
0.41 尺

Mean ~~bottom~~ area 44.88 平方尺

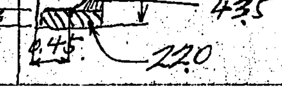
Volume 44.88 × 0.20 × 2 = 18.0 立方尺
17.3 × 0.2 × 2.0 = 6.0
24.0 立方尺 or 0.11 立坪 for TP3&6.

Volume 44.88 × 0.41 × 2 = 37.0
17.3 × 0.41 × 2.0 = 14.0
51.0 立方尺 or 0.24 for TP2&7.

Concrete (1:2:4) for Wells filling.

78.5 × (6.0+3.0) = 707.0
22.0
43.5
772.5 or 3.60 立坪
2@3.60 = 7.20 立坪

Concrete (1:2:4) for Wells shell.



dia 14.0 area 153.9 = $\frac{153.9 + 132.7}{2} \times 6.75 = 143.3 \times 6.75 = 970.0$
dia 13.0 area 132.7 = 785.0 × 6.75
−530.0
−43.5
396.50 or 1.97 立坪

132.7 − 78.5 × 20.0 = 1084.00 or 5.03
1.97
7.00 立坪

2@7.0 = 14.0 立坪

summary of Forms.

for shafts Web 27.0
23.0
50.0 平坪

for Wells 88.0 平坪

---

summary of materials for Pier TP1 and TP8.

Conc. (1:2:4)
for shafts and web 9.58 立坪
for Wells 11.30 "
20.89 立坪

Conc. (1:2:4)
for wells 4.62 立坪

Conc. (1:4:8)
for wells 8.38 立坪

Reinforcement
for shafts and Web. 0.900 ton
for Wells 1.860
2.760 tons

Curb shoes.
2@0.33 = 0.66 tons

Forms.
for shafts and Web 50.0 平坪
for wells 88.0
138.0 平坪

---

summary of conc. for shafts and Web.

copings. 0.75 立坪
cylinders. 7.40
Web. 4.04
for TP4&5 .... 12.19 立坪

for TP3&6 .... 12.19
0.11
12.08 立坪

for TP2&7 12.08
0.24
11.84 立坪

Materials of Substructure for Kasumi-Bashi. Okayama-Ken.

Conc. (1:4:8) for Wells
7.854 × 18.0 = 1414.00 立尺 or 6.54 立坪
2@6.54 = 13.08 立坪

Reinforcement. (see sheet no. 9)
in shafts 674# × 2 = 1,348#
in Web. 842
2190# or 0.98 ton
in Wells. 2@1.19 = 2.38 tons.

Curb shoes. C53.
4 Pls. 12" × ¼" @ 10.20 × 10'-11¼" = 446.#
4 Ls. 5" × 3" × 0.3" @ 7.85 × 10'-11¼" = 344
4 Pls. 4" × ¼" @ 3.40 × 0'-5¾" = 47
4 Ls. 5" × 3" × 0.3" @ 7.85 × 1'-4" = 44
32-Bolts ¾"φ × 1'-2" @ 0.91 = 29
870# or 0.39 ton
2@0.39 = 0.78 ton

Forms.
for shafts and web 23.75 × 22.81 = 542.00 平尺
17.30 × 24.31 = 420.00
962.00 × 2 = 1,924.00 平尺
or 53.4 平坪
for Wells. 40.84 × 26.0 = 1060.0 outside.
31.42 × 27.4 = 860.0 inside.
1920.0 平尺 or 53.0 平坪
2@53.0 = 106.0 平坪.

Summary of Materials for pier TP2-TP7

Concrete (1:2:4) shafts and web. 12.19 立坪
Wells 14.00
26.19 立坪 ... for TP4 & TP5

shafts and web 12.08
wells 15.50
27.58 立坪 for TP3 & TP6.

concrete (1:2:4) for Wells. 7.20 立坪
" (1:4:8) for Wells 13.08 "

shafts and web 11.84
Wells. 15.50
27.34 立坪 for TP2 & TP7.

Reinforcement for shafts and web. 0.98 ton
for Wells 2.38
3.36 tons.
Curb shoes C53. 2@0.39 = 0.78 ton.
Forms. for shafts and web. 53.4 平坪
for Wells 106.0
159.4. ≒ 159.0 平坪

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY J. Masuda DATE 14-9-10 FILE NO
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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

1\. This bridge is to be erected on main highway between Kurashiki and Tamashima Okayama-Ken and to cross Takahashi-gawa at 50 ken about down-stream of present wooden highway bridge. For final layout we adopted the following beginning from left bank;

11 spans 60'-0" out to out	=	660'-0"
7 spans 177'-3" out to out	=	1240'-9" ⓒ
2 spans 60'-0" " "	=	120'-0"
space at abutments 2@2"	=	4"
space on girder piers 11@2"	=	1'-10"
2@1½" space	=	3"
		2023'-2"

note:- The truss spans shown above will be out to out without expansion joint; 3" floor expansion shall be provided on truss piers. The span length between center to center of end pins = 175'-0"

The bridge shall be cambered toward center of stream and the crown should be located at ℄ of center span of trusses.

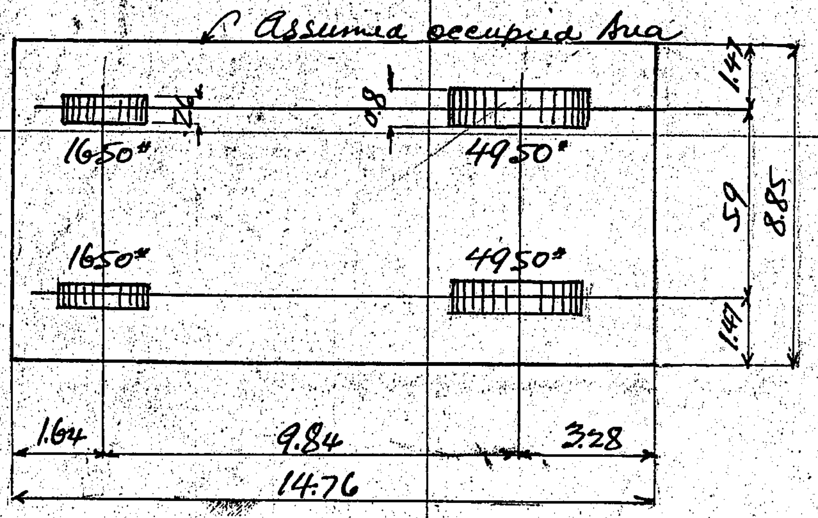
The width of roadway 21.0 ft between curb lines; Pavement shall be 2" Soliditit on reinforced concrete slab. The handrail will be made of structural steel of proper ornamental design.

Assumed Loadings

Uniform load on roadway $w = \frac{100.000}{170+l} \leqq 500\,{}^{kg}/{}_{m^2}$ say 100 #/□'

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

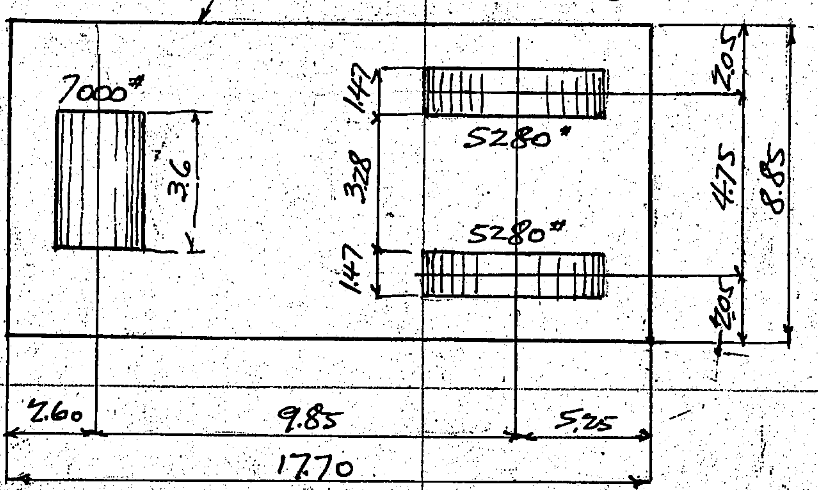
Motor trucks loading (6 tons).



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 trucks in each row on one span

One road roller on bridge considered without impact

8 ton Road roller



Impact
For motor trucks loading $\frac{20}{60+l}$

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

No impact for roadroller and Uniform load.

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Assumed weight of materials.

Plain concrete	140# per cubic ft
Reinforced concrete	150# " "
Structural steel	490# " "
Cast iron	450# " "
masonry	160# " "
Soliditit pavement	140# " "

Allowable Working Strength

Structural steel or Reinforcing Bars

Tension	17000 #/□"
Extreme fibre stress	17000 "
Shear on web gross section	12800 "

Compression member 21300 (1 − 0.0055 l/r) or not over 14000# gross area.

Compression flange of plate girder

$$17000\,(1-0.012\ l/b) \leqq 15400\ \#/\square$$

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

Shearing on shop driven rivets (machine driven)	12000 #/□
" " 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#/□.

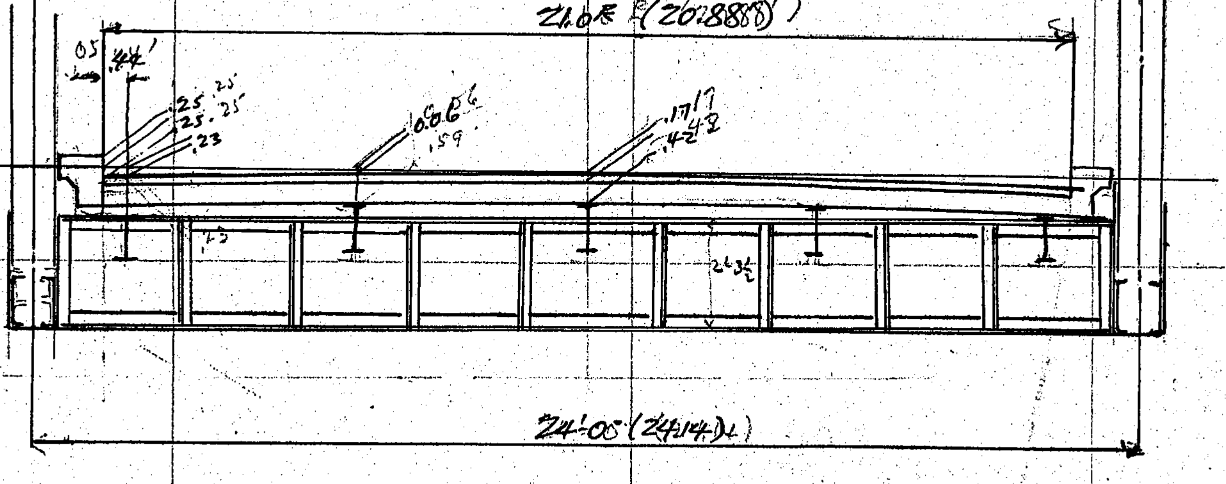
Concrete 1:2:4 mixture

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 stresses in addition to dead, Live and impact stresses, the allowable working strength shall be increased 25% and proportioned the parts. In figuring earthquake, the working strength shall be increased 80% and proportioned the parts.

Acceleration of Earthquake assumed 1000 mm/sec².

Cross section of Truss Span.



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

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Reinforced Concrete slab. Span length of slab = 5'-0" centre to centre of stringers

Dead Load

2" Soliditit Pavement 24#
5" Concrete slab 62
86# per sq ft.

Dead Load moment = $\frac{1}{10} \times 86 \cdot 5^2 =$ 215 1#
Dead Load shear = $86 \cdot 2.5$ = 215#

Live Load motor truck rear wheel concentration 4950#
Impact 30% 1485
6435#

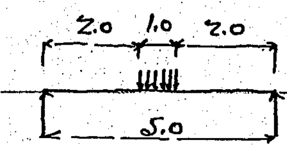
Front wheel conc. with impact = $\frac{1}{3}$ · rear = 2145#

Distribution of wheel concentration

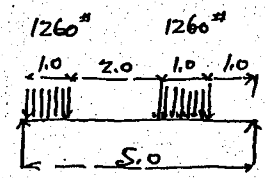
Contact between wheel and pavement assumed 20 centimeter = 0.66
2 × thickness of pavement .17 = 0.34
Longitudinal distribution 1.00 = a

Transverse distribution 0.80 + 0.34 = 1.14 = b

Effective width $E = \frac{2}{3}(l+b) + a$ where l = span length in ft
$= \frac{2}{3}(5.00 + 1.14) + 1.00 = 5.1$



Load for one ft strip = 6435 ÷ 5.1 = 1260# assumed
Moment due to single wheel load = 630 × 2.5 = 1575 1#
Less moment due to uniform 630 · 0.25 = - 157
1418 1#



For continuity of slab take moment as 0.8 · 1418 = 1135 1#
max End shear as simple beam
$1260 \cdot \frac{6.0}{5.0} = 1510$#

Summary for moments and shears

	moment	shear
Dead Load	215 1#	215#
Live Load	1135	1510
	1350	1725

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

$d = \sqrt{\frac{1350}{102}} = 3.64"$ Use 5" slab with effective depth of 3.75" with insulation of 1.25".

Steel area req'd = $\frac{1350 \cdot 12}{\frac{7}{8} \cdot 3.75 \cdot 17000}$ = 0.296□" per ft Use ½" dia bars 6" centers = 0.390□

max End shear = $\frac{1725}{\frac{7}{8} \cdot 3.75 \cdot 12}$ = 44#/□ ok

Bond stress $u = \frac{1725}{\frac{7}{8} \cdot 3.75}$ = 526# per ft Try 2-½"φ @ 1.57 · 130# = 409
2-⅜ @ 1.18 × 130# = 306
715#

This max shear at stringer on ℄ bridge only add 2-⅜"φ bars 3.0 long for bond stress.

At other supports max Live Load shear = $1260 \cdot \frac{4.5}{5.0}$ = 1135
D.L. shear 215
1350#

Bond stress $u = \frac{1350}{\frac{7}{8} \cdot 3.75}$ = 411# per ft strip Use no extra reinforcement

Design of Longitudinal Stringers.

Stringer Sc Stringer spacing 5'-0" Span length = 17.5'

Dead Load

Slab and Pavement 86 × 5 = 430
Stringer assumed 45
475# per lin. ft

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayamaken

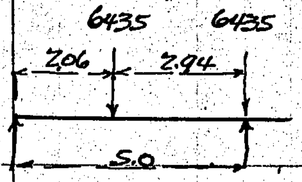
Stringer Sc

Dead Load Moment = $\frac{1}{8} \times 475 \times 17.5^2 = 18180'^{\#}$ Shear = $475 \times \frac{17.5}{2} = 41.60^{\#}$

Live Load motor trucks loading Rear wheel conc with impact = 6435

Front wheel conc " " = 2145

Load on stringer Rear wheel Reaction



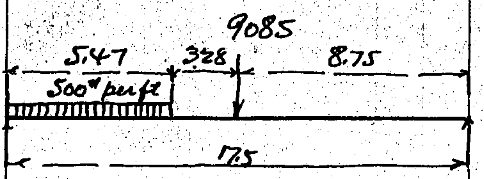
$6435 \times \frac{2.06}{5.0} = 2650$

6435

$9085^{\#}$

Front wheel Reaction ------- $3030^{\#}$

Live Load Moment

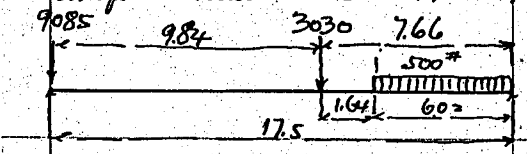


moment due to concentration $\frac{9085}{2} \times 8.75 = 39700$

Uniform load $2730 \times \frac{2.73}{17.5} \times 8.75 = 3720$

43420

Uniform load $500 \times 5.47 = 2730^{\#}$



max End shear

Uniform load $500 \times 6.02 =$ say $3000^{\#}$

Reaction = $3000 \times \frac{3.01}{17.5} = 515^{\#}$

Rear wheel 9085

Front wheel $3030 \times \frac{7.66}{17.50} = 1325$

10925

call this $11000^{\#}$

Summary for moments and shears

	moment	shear
Dead Load	18180	4160
Live load	43420	11000
	$61600'^{\#}$	$15160^{\#}$

Section modulus required

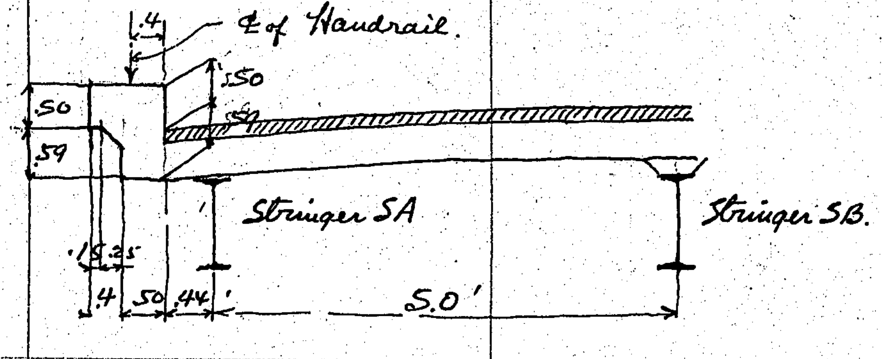
$\frac{61600 \times 12}{17000} = 43.5$

Use 12"×6" I beam $44.02^{\#}$ Sm = 52.57

Unit stress = $\frac{61600 \times 12}{5257} = 14100^{\#}/\square''$

Design of End stringer SA

Dead Load



Reaction on stringer SA

Support SB assumed as simply supported

	weight		arm		
$86^{\#} \times 5.44 =$	468	×	2.72	=	1275
$.5 \times 1.09 \times 150^{\#} =$	82	×	5.69	=	466
$0.03 \times 150^{\#} =$	5	×	6.02	=	30
$0.20 \times 150 =$	30	×	6.14	=	184
Handrail say	60	×	5.84	=	350
	645				2305

Reaction on SB = $645 - 461 = 184^{\#}$

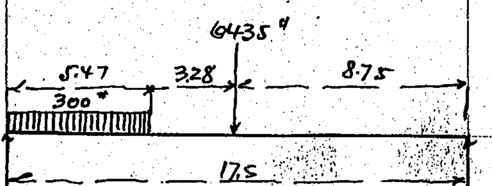
Reaction = $2305 \div 5.0 = 461^{\#}$

stringer assumed 35

$496^{\#}$

Dead Load Moment = $\frac{1}{8} \times 496 \times 17.5^2 = 19000'^{\#}$ shear = $496 \times \frac{17.5}{2} = 4350^{\#}$

Live Load



Uniform live Load assumed $300^{\#}$ per lin ft.

Rear wheel of motor truck on ℄ web of stringer assumed

moment due to conc. $\frac{6435}{2} \times 8.75 = 28200$

Uniform load $256 \times 8.75 = 2240$

Unif. L = $300 \times 5.47 = 1640^{\#}$

Reaction = $1640 \times \frac{2.73}{17.5} = 256$

Live Load m $30440'^{\#}$

Dead Load m 19000

$49440'^{\#}$

section modulus req'd = $\frac{49440 \times 12}{17000} = 34.9$

Use 12"×5" I beam $31.99^{\#}$ Sm = 36.69

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Stringer SB.
Dead Load assumed same as SC neglecting cantilever action at SA
Dead Load Moment 18180 1#
Live Load uniform assumed 500# per lin ft m = 3720
conc. 6435# at center m = 28200

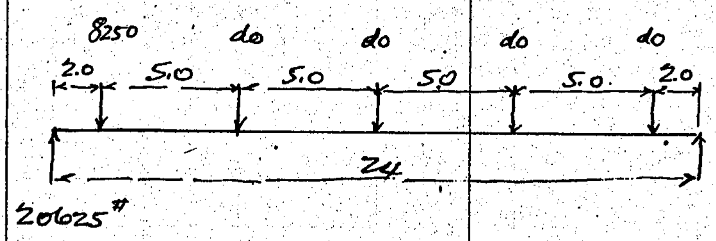
Total Live Load m 31920
Dead Load m 18180
50100

Section modulus reqd $\frac{50100 \times 12}{17000} = 35.4$

Use 12" × 5" I beam 31.99# Sm = 36.69

Design of Floor Beam
Intermediate floor beam
Dead Load of Floor - see pp4 Load beyond SB. 2 @ 645 = 1290#
Load between SB-SB. 86 × 10 = 860
Including Handrail - 2150#

Distribution of load assumed
On stringer SC 86 × 5 = 430# Stringer assumed 40#
" " SB 430# "
" " SA 645 − ½ × 430 = 430# "

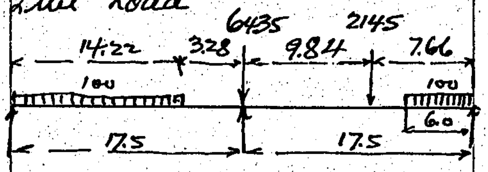


Concentration = 470# × 17.5 = 8250#

Dead Load Moment
20625 × 12 = 248000
8250 × 15 = −123500
124500 1#

Dead Load of girder assumed 120# m = $\frac{1}{8} \times 120 \times 24^2$ = 9930
134430 1#

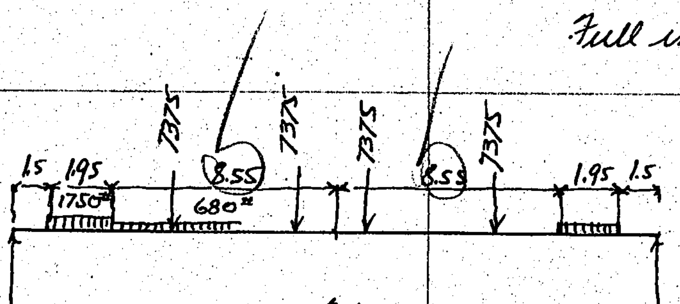
Live Load



Reaction = $2145 \times \frac{7.66}{17.5}$ = 940
6435
7375#

Uniform load per unit length
Reaction $600 \times \frac{3.0}{17.5}$ = 103#
$1422 \times \frac{7.11}{17.5}$ = 578
681 say 680# per lin ft

Full uniform load 1750# " " "



Reaction 2 @ 7375 = 14750#
680 × 8.55 = 5800
1750 × 1.95 = 3400

moment at ℄ of span
concentration 14750 × 12 = 177000
14750 × 4.27 = 63000
114000 1#

Uniform Load 5800 × 12 = 69600
5800 × 4.27 = 24800
45200 1#

Uniform Load 3400 × 12 = 40900
3400 × 9.53 = 32400
8500

Total Live Load Moment = 114000 + 45200 + 8500 = 167700#
" Dead Load moment = 134430
Total moment 302130 1#

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Maximum End shear
Uniform load on one side
motor truck wheel load $14750 \times 2 \times \frac{13.95}{24.0} = 17110^{\#}$
$11600 \times 13.95 \div 24 = 6750$
$6800 \times 3.45 \div 24 =$ say $\underline{1000}$
$24850^{\#}$
Dead Load Floor $20625^{\#}$ } 22065
Dead Load Floor Beam say $\underline{1440}$
46915
call this $47000^{\#}$

Main section of Floor Beam moment = say $302000'^{\#}$ shear $= 47000^{\#}$
Depth of beam $27\frac{1}{2}$" b to b of Ls.
web = 27" × 5/16 = 8.44 □" 1/8 web = 1.05 Effective depth say 2.21' stress = $137000^{\#}$
Section required = 137000 ÷ 17000 = 8.03 □" 8.03 − 1.05 = 6.98
Less 1/8 web = $\underline{1.05}$
6.98 □" net

Use 2Ls 5 × 3 × 3/8 = 5.72 □" 4.22 □" net
1Pl. $10\frac{1}{2}$ × 3/8 = 3.94 □" $\underline{3.18}$ Length $= 24 \times \sqrt{\frac{3.18}{7.40}} + 2 = 17.7'$
7.40 □"

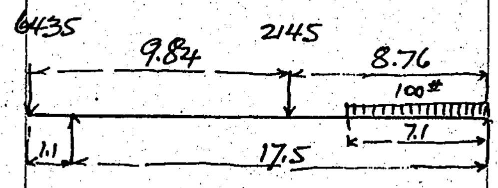
Weight of Floor Beam

Flange	4Ls	5 × 3 × 3/8	@	9.8#	×	23.0	=	900
web.	1Pl	27" × 5/16	@	28.69#	×	23.0	=	660
cov Pls	2Pls	$10\frac{1}{2}$ × 3/8	@	13.39	×	17.7	=	473
End Stiffs	4Ls	$3\frac{1}{2}$ × $3\frac{1}{2}$ × 3/8	@	8.5	×	2.23	=	76
fillers	4Pls	$6\frac{1}{2}$ × 3/8	@	8.29	×	1.79	=	59
Stiffs at Stringer	10Ls	$3\frac{1}{2}$ × $3\frac{1}{2}$ × 3/8	@	8.5	×	2.23	=	190
filler	10Pls	$3\frac{1}{2}$ × 3/8	@	4.46	×	1.79	=	80
stiffs	8Ls	3 × 3 × 5/16	@	6.10	×	2.30	=	112
				Rivet heads & variation $3\frac{1}{2}$%				90

$2640^{\#} \div 23 = 115^{\#}$ per lin ft

End Floor Beam
Dead Load concentration at stringer say $430 \times \underline{18.60} \times \frac{9.30}{17.5} = 4250^{\#}$
End reaction = 4250 × 2.5 = say $10600^{\#}$
Dead Load moment at center ! = $10600 \times 12 = 127200$
$4250 \times 15 = -\underline{63800}$
$63400'^{\#}$
Dead Load girder assumed $100^{\#}$ per ft $m = \frac{1}{8} \times 100 \times 24^2 =$ $\underline{7200}$
$70600'^{\#}$

Live Load



Reaction $2145 \times \frac{8.76}{17.5} = 1072$
$6435 \times \frac{18.6}{17.5} = \underline{6820}$
$7892^{\#} \times 2 = 15784^{\#}$
Uniform Load $710 \times \frac{355}{17.5} = 144^{\#} \times 8.55 = 1230^{\#}$
" $1860 \times \frac{9.3}{17.5} = 990^{\#} \times 1.95 = 1930^{\#}$

Moment at ℄ of span
motor trucks wheel $15784 \times (12 - 4.27) = 122000$
Uniform Load $1230 \times (12 - 4.27) = 9500$
" " $1930 \times (12 - 9.53) = \underline{4800}$
Live Load moment $136300'^{\#}$
Dead Load moment $\underline{70600}$
$206900'^{\#}$

CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Max End shear
Motor trucks 15784 × 2 × 13.95 ÷ 24 = 18350
Uniform Load 1230 × 2 × 13.95 ÷ 24 = 1430
" " 1930 × 2 × 3.45 ÷ 24 = 550
20330#

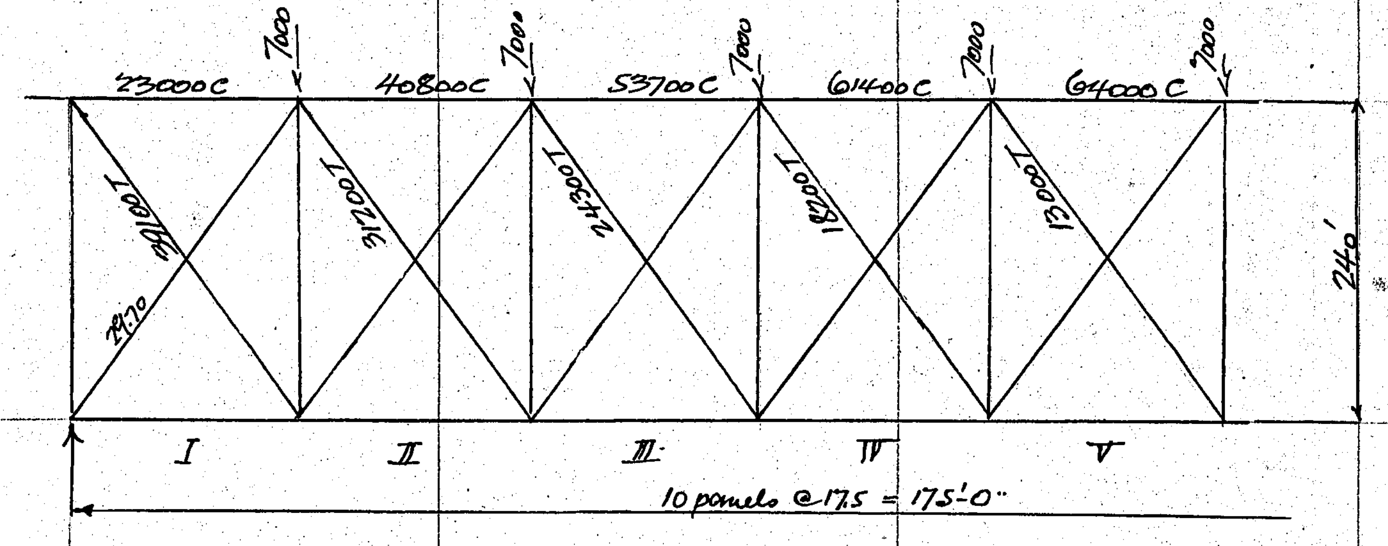
Dead Load shear Floor 10600
" " Floor Beam say 1200
11800
32130# call this 33000#

Main section moment say 207000'# Shear = 33000#
Depth of beam 27½" back to back of Ls web 27" × 5/16 = 8.44□" ⅛ web = 1.05□"
Effective Depth say 2.15' Stress = 96200# Section reqd = 96200 ÷ 17000 = 5.66□"
⅛ web − 1.05
4.61□" net

Use 2Ls 5 × 3 × 3/8 5.72□" gr or 4.97□" net

Weight of End Floor Beam
Flange 4Ls 5 × 3 × 3/8 @ 9.8 × 23.0 = 900
web 1Pl 27 × 5/16 @ 28.69 × 23.0 = 660
End stiffs 4Ls 3½ × 3½ × 3/8 @ 8.5 × 2.23 = 76
End filler 4Pls. 3½ × 3/8 @ 4.46 × 1.79 = 32
Stiff at stringer 5Ls 3½ × 3½ × 3/8 @ 8.5 × 2.23 = 95
filler 5Pls 3½ × 3/8 @ 4.46 × 1.79 = 40
Stiffs 8Ls 3 × 3 × 5/16 @ 6.10 × 2.30 = 112
Rivet heads + variation 3½% 67
1982# ÷ 23 = 86# per lin ft.

Lower Lateral Bracings.
wind pressure assumed 400# per lin. ft Panel conc. = 400 × 17.5 = 7000#



Section required for End panel. 39100# ÷ 17000 = 2.30□" net

Panel	Stress	SR for Tension	Section	area	No of Rivets
I	39100T	2.30□"	2Ls 5 × 3 × 5/16	4.80	10
II	31200T	1.83	2Ls 5 × 3 × 5/16	4.80	8
III	24300T	1.43	2Ls 4 × 3 × 5/16	4.18	6
IV	18200T	1.07	do	4.18	6
V	13000T	0.76	do	4.18	6

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Approximate weights of Lower Laterals.

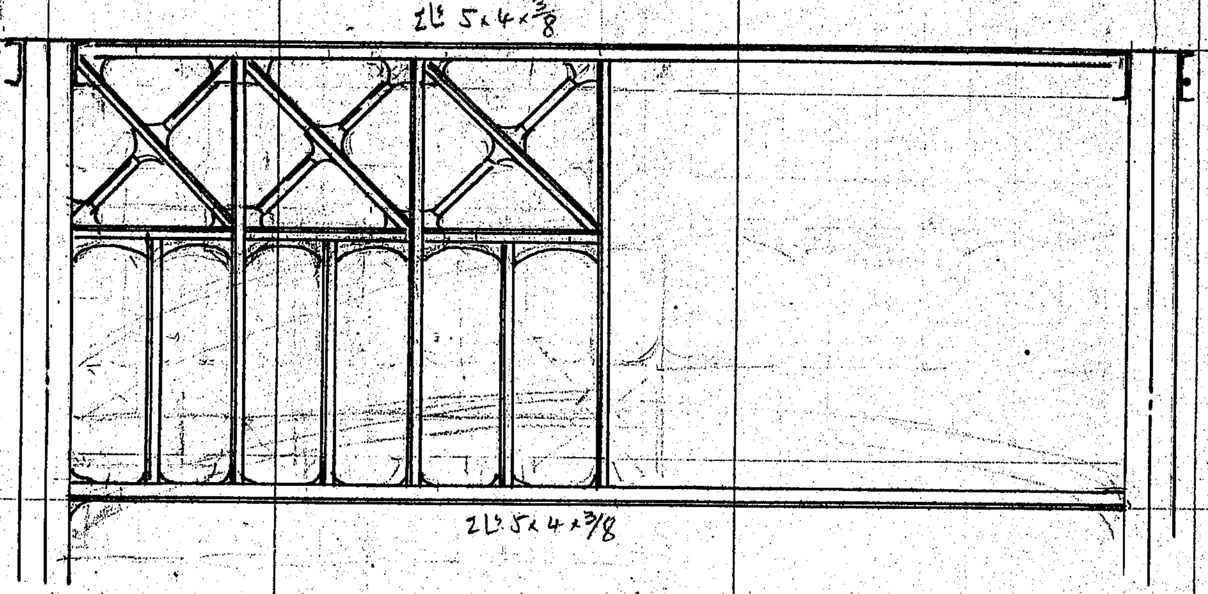
2Ls 5×3×5/16 @ 8.2 × 28.0 = 460#
4Ls 5×3×5/16 @ 8.2 × 13.7 = 450
misc enter connection say 75
985#

2Ls 4×3×5/16 @ 7.2 × 28.0 = 403#
4Ls 4×3×5/16 @ 7.2 13.7 - 394
misc enter connection say 55
852

Summary for Lower Lateral Bracing

4 @ 985 = 3940
6 @ 852 = 5110
misc. rivet headssc 40
9100# per truss

Sway Bracings 4 Required



Top and bottom struts 4Ls 5×4×3/8 @ 11.0 × 22.4 = 986#
verticals + diagonals 2Ls 2½×2"×0.3" @ 4.28 × 130.0 = 1120
connection pls + details say 350
2456 call this 2500# per sway

Diagonal Bracings 2Ls 5×3×5/16 call this 1000# per panel including detail.
Transverse strut 4Ls 4×3×5/16 including detail say 760# per piece.

Portal Bracing
Top and Bottom Struts 4Ls 5×4×3/8 @ 11.0 × 23.0 = 1010
diagonals 2Ls 3×3×5/16 @ 6.1 × 24.0 = 300
verticals 2Ls 3×3×5/16 @ 6.1 × 24.0 = 300
connection and ~~detail~~ say 450
2060# per piece.

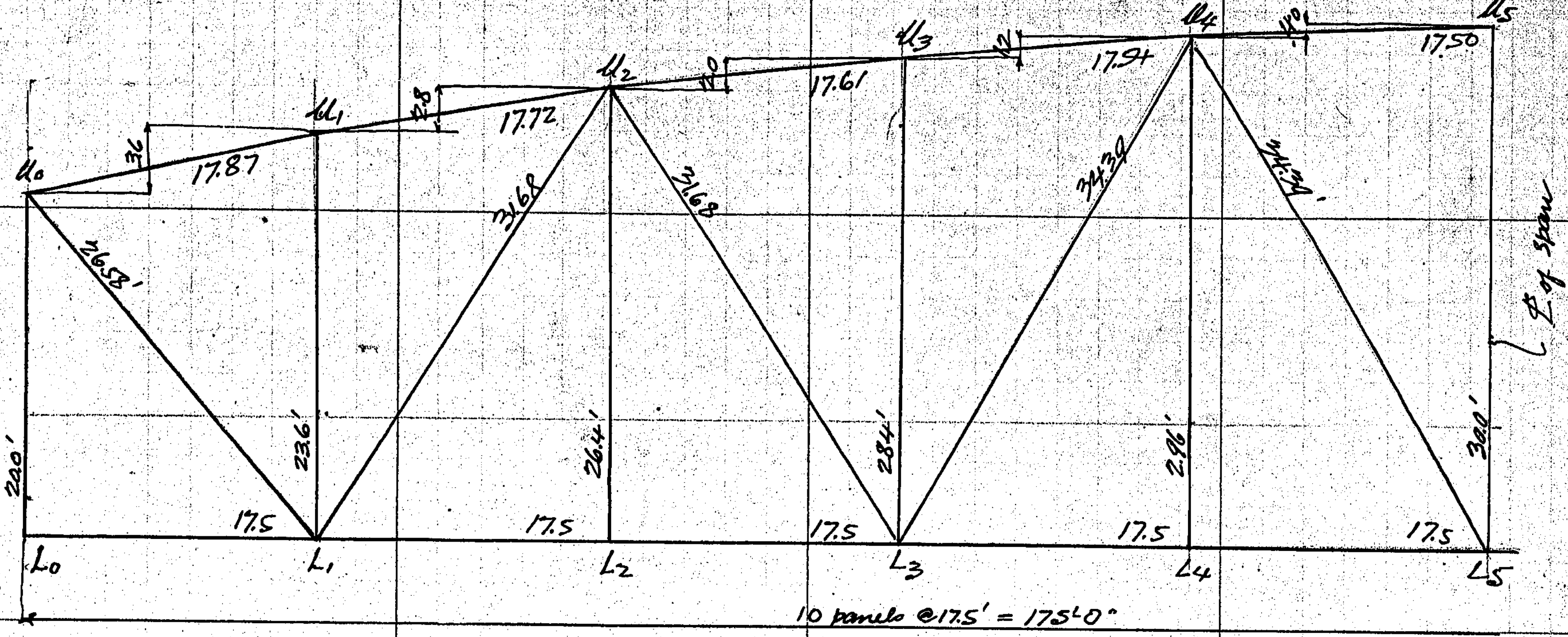
Summary for Top Lateral Bracings
Diagonal Bracings. 10 @ 1.000 = 10.000
Transverse strut 5 @ 760 = 3800
Sway Bracings 4 @ 2500 = 10.000
Portal Bracings - 2 @ 2060 = 4120
27920# ÷ 175 = 159.5# per lin. ft.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Design of Kasumi-Bashi for Okayama-Ken

Design of Truss span

175'-0" c to c of End Bearings truss centers 24'-0"; 10 Panels @ 17.5' each



Dimensions of truss members given above are nominal length of members; To give the camber to truss during erection top chord is lengthened 1/4" at end panel and 3/8" at center of span and all other members are proportioned to this added lengths. In nominal condition the camber of truss will be 2 3/4" about at center line of span.

Assumed Dead Load

Floor Slab pavement and Handrails 5 @ 430 = 2150# per lin. ft of truss

Dead Load metal

Stringers	187#
Intermediate Floor Beam	151
Lower Laterals	52
Upper Laterals	160
trusses assumed	920

1470# ÷ 2 = 735# per lin. ft

1/2 Dead Load of floor 1075

1810# per lin ft.

Panel concentration = 1810 × 17.5 = say 31700# for top and bottom panel points

For top chord panel point for one truss

Upper Lateral 160 ÷ 2 = 80

truss upper half 230

310# × 17.5 = say 5400#

Live Load on truss

Uniform Load $w = \frac{100.000}{170 + 533} = 429$ kg/m² $= 88\#/\square'$

Coefficient of Impact $= \frac{20}{60 + 533} =$ say 17.6%

motor truck Rear wheel concentration = 4950#

Impact 17.6% 870

5820# with impact

Front wheel conc $\frac{1}{3} \times 5820 =$ 1610# " "

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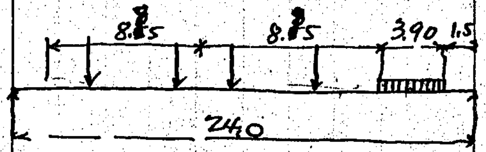
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Reaction on truss



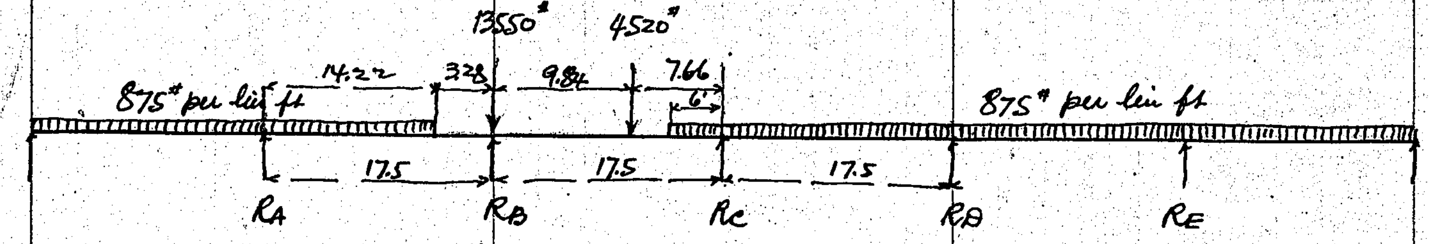
motor truck loading

Rear wheel 4@5820 · $\frac{13.95}{24}$ = 13550#

Front wheel ⅓ · 13550 = 4520#

Uniform Load at front and rear of motor truck 2@8.55 = 17.10' wide
88 × 17.10 = 1505# Reaction 1505 · $\frac{13.95}{24}$ = 875# per lin ft of truss
Uniform Load at side of motor truck 3.90' wide
88 × 3.90 = 344# Reaction 344 × $\frac{3.45}{24}$ = say 50# " " " " "

Panel Concentrations due to uniform and motor truck loads



1675#	7420 7650 875 15945	13550 1975 900 875 17300 5080 22380	2545 4350 7650 875 15420#	15300 875 16175	15300 875 16175	— Uniform load 50# per ft

motor truck · Front wheel = 4520 × $\frac{7.66}{17.5}$ = 1975 RB
2545 RC

Uniform Load 6 × 875 = 5250# 5250 × $\frac{3}{17.5}$ = 900# RB
5250 − 900 = 4350# RC

Uniform Load 14.22 × 875 = say 12500# 1250 × $\frac{7.11}{17.5}$ = 5080# RB
12500 5080 7420 RA

Uniform Load 17.5 × 875 = 15300# ½ = 7650#
Uniform Load 17.5 × 50 = 875#

For conveniencis sake live load panel concentration take 16500# throughout the panel points and add 6000# wheel concentration to uniform load
For web member ~~mem~~ use 16500# concentration -

Dead Load Stresses Reaction = 31700 × 4.5 = 142650#
chord stresses

$U_0 - U_1$ 142650 · 17.5 ÷ 23.1 = 108000# C
$U_1 - U_2$ " · ÷ 23.3 = 107000# C

$L_1 - L_2 - L_3$ 142650 · 2 · 17.5 = 4.990.000
31700 · 17.5 = 555000
4435000 ÷ 26.4 = 167700# T

$U_2 - U_3$ and $U_3 - U_4$ 142650 · 3 · 17.5 = 7.490.000
31700 · 3 · 17.5 = 1.663.000
5827000 ÷ 28.2 = 206500# C
÷ 28.3 = 206000# C

$L_3 - L_4 - L_5$ 142650 · 4 · 17.5 = 9.980.000
31700 · 6 · 17.5 = 3330.000
6650.000 ÷ 29.6 = 224000# T

$U_4 - U_5$ 142650 · 5 · 17.5 = 12.480.000
31700 · 10 · 17.5 = 5550.000
6930.000 ÷ 30.0 = 231000# C

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Dead Load web stresses

U_0-L_1 142650 · (97.3 ÷ 96.5) = 144.000 #

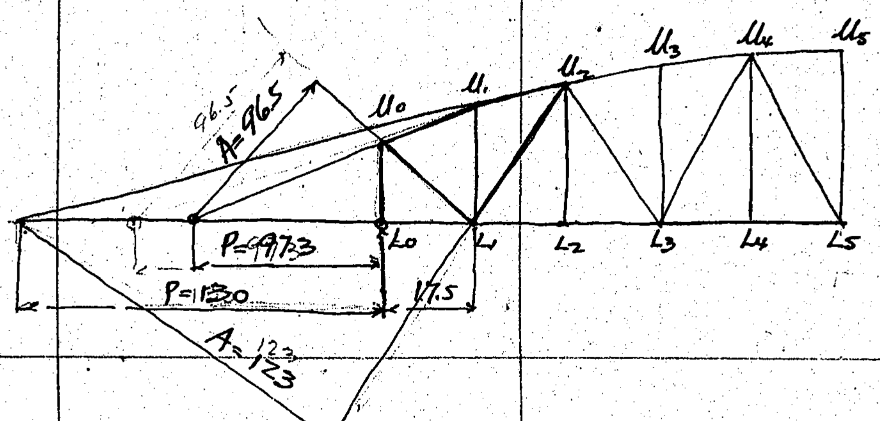
L_1-U_2 142650 · 130 = 18.550.000
31700 · 147.5 = − 4675.000
13875.000 ÷ 1230 = 112.800 #

U_2-L_3 142650 · 196 = 27.950.000
31700 · ~~44~~45 = 14100.000
13.850.000 ÷ 207. = 67.000 #

L_3-U_4 142650 · 362. = 51.600.000
31700 · 1191. = −37800.000
13800.000 ÷ 344.8 = 40.000 #

U_4-L_5 142650 · 1237.5 = 176.200.000
31700 5125 = 162500.000
13.700.000 ÷ 1144 = 12.000 #

End Post D.L. shear = 142650
½ panel conc. top chord 2700
145350 #



member	dist P.	arm A.
U_0-L_1	97.3′	96.5′
L_1-U_2	130.0	123.0
U_2-L_3	196.0	207.0
L_3-U_4	362.0	344.8
U_4-L_5	1237.5	1144.0

Live Load chord stresses

Reaction = 16500 × 4.5 = 74250 #

U_0-U_1 74250 · 17.5 ÷ 23.1 = 56300 #
U_1-U_2 " · ÷ 23.3 = 55900 #

L_1-L_2-L_3 74250 × 2 × 17.5 = 2.600.000
16500 × 17.5 = 289.000
2311000 ÷ 26.4 = 87500 #

U_2-U_3 and U_3-U_4 74250 · 3. × 17.5 = 3890.000
16500 · 3. 17.5 = 865.000
3025000 ÷ 28.2 = 107200 #
÷ 28.3 = 107000 #

L_3-L_4-L_5 74250 · 4 · 17.5 = 5195000
16500 · 6 · 17.5 = 1732.000
3463.000 ÷ 29.6 = 117.0.00 #

U_4-U_5 74250 · 5 × 17.5 = 6500.000
16500 · 10 × 17.5 = 2890.000
3610000 ÷ 30.0 = 120200 #

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Live Load chord stresses

6000# concentration at panel point

				Add		Summary
U_0-U_1	5400 × 17.5 ÷ 23.1	=	4090	56300	=	60390
U_1-U_2	5400 × 17.5 ÷ 23.3	=	4050	55900		59950
L_1-L_2-L_3	4800 · 35 ÷ 26.4	=	6360	87500		93860
U_2-U_3	4200 · 52.5 ÷ 28.2	=	7820	107200		115020
U_3-U_4	4200 · 52.5 ÷ 28.3	=	7800	107000		114800
L_3-L_4-L_5	3600 · 70. ÷ 29.6	=	8500	117000		125500
U_4-U_5-	3000 · 87.5 ÷ 30.0	=	8750	120200		128950

Live Load web stresses

				Reaction			
U_0-L_1	16500 ·	4.5	=	74300	74300 × 97.3 ÷ 96.5	=	75000# T
L_1-U_2	·	3.6	=	59500	59500 · 130 ÷ 123	=	63000 C
U_2-L_3	·	2.8	=	46200	46200 · 196 ÷ 207	=	43700 T
L_3-U_4	·	2.1	=	34700	34700 · 362 ÷ 344.8	=	36400 C
U_4-L_5	·	1.5	=	24800	24800 · 1237.5 ÷ 1144	=	26800 T
do	·	1.0	=	16500	16500 · do	=	17300 C

End post. stress = 74300#

Section of End Post U_0-L_0.

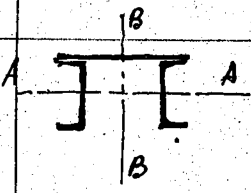
Total stress = 219650#

section assumed

1 cov Pl 19 × 3/8 = 7.13

2[s 12 × 3½ × 26.10# = 15.34

22.47□"



Moment of Inertia AA Axis

7.13 × 12.19 = 87.0

15.34 × 6.0 = 92.0

7.95" 179.0

7.13 × 4.24² = 127.8

15.34 × 1.95² + 316 = 374.4

502.2

Ecc = 1.95"

Radius of gyration = $\sqrt{\frac{502.2}{22.47}}$ = 4.72 (inch)⁴

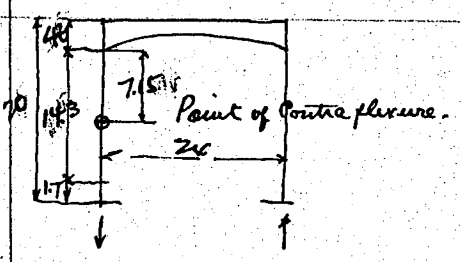
Moment of Inertia BB Axis.

cov Pl. 7.13 = 214

[s 15.34 × 6.86² + 15.2 = 737

22.47 951

$r = \sqrt{\frac{951}{22.47}} = 6.5$



wind load $\frac{200}{2}$ × 175 = 17500

Extra load due to wind = 17500 × $\frac{20}{24}$ = 14600

direct load - D + L 219650

234250#

Bending moment due to wind contra flexure point assumed at 11.25' from top = 8750 · 7.15 = 62500 '#

or 750.000 "#

Fibre stress = $\frac{750.000\text{"\#} \times 9.5}{951}$ = 7500 #/□"

Unit stress = 14000 × 1.25 = 17500 #/□"

7500

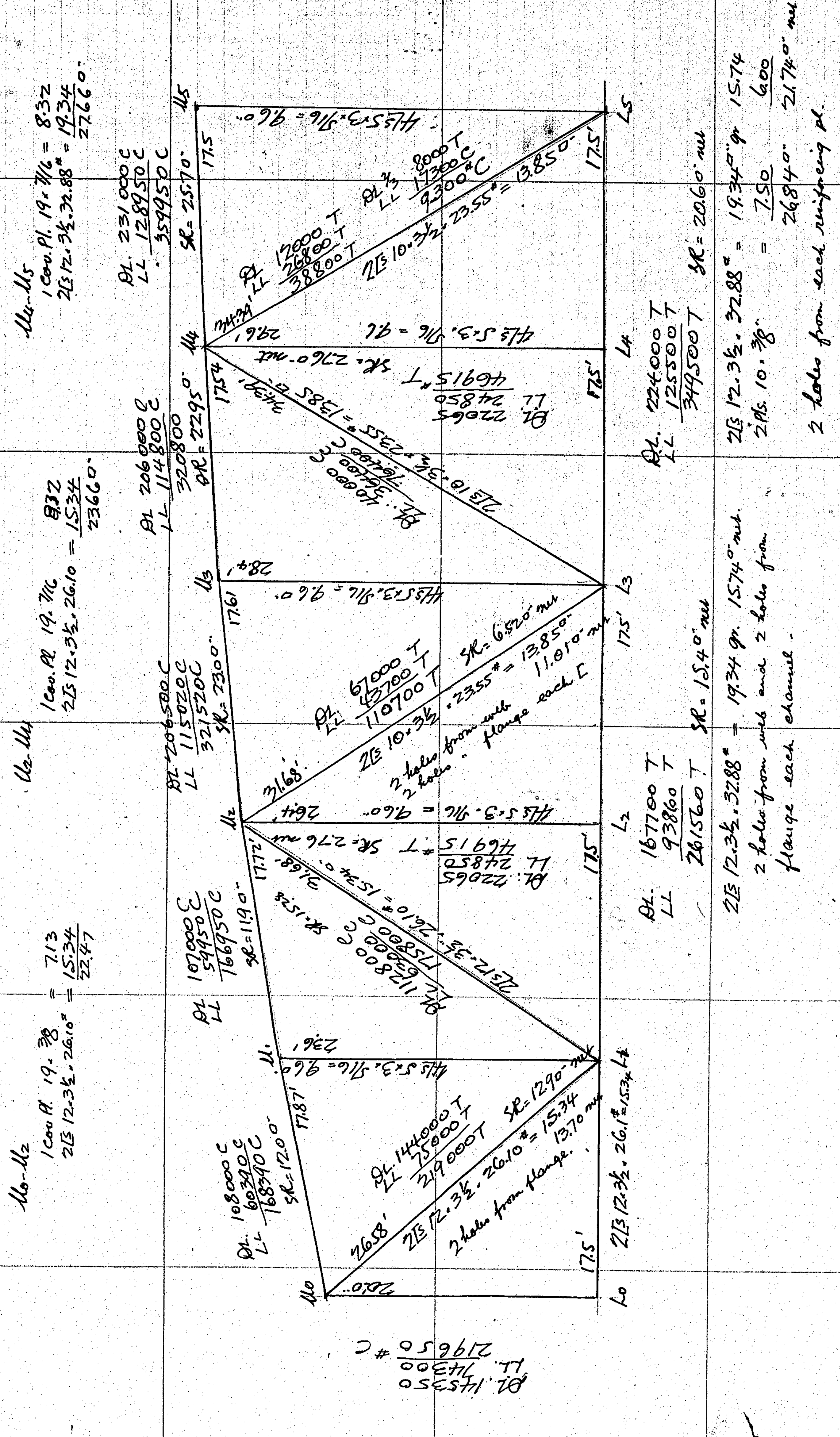
10000 #/□" for direct stress.

Unit stress due to direct stress = 219650 ÷ 22.47 = 9770 #/□" Ok.

Neglecting live Load stress this stress 145350 ÷ 22.47 = 6460 #/□"

Design of Kasumi-Bashi for Okayama-Ken

stresses and sections of truss



增田淳氏関係資料
（独立行政法人 土木研究所蔵）

Design of Kasumi-Bashi for Okayama-Ken

End Post U_0-L_0.

Unit stress = $21300 \times (1 - 0.0055\, l/r)$ = where $l = 20 \times 12 = 240"$ $r = 4.72$
= 15900#/□" Use 14000#/□"

Moment due to Eccentricity of connection = 219650 × 1.95" = 428000"#

Fibre stress = $\frac{428000 \times 4.43}{502.2}$ = 3780#

Direct stress 219650 ÷ 22.47 = 9770
13550#/□"

Diagonal L_1-U_2. 175800# C

$r = 4.55$ $l = 31.68 \times 12 = 380$ Unit stress = $21300 \times (1 - 0.0055\, l/r)$.
$l/r = 83.5$ = 11500#/□"

section required = 175800 ÷ 11500 = 15.28□"
Use 2[12 × 3½ @ 26.10# = 15.34□"

Diagonal L_3-U_4 76400# C

$r = 3.85$ $l = 34.39 \times 12 = 413"$ Unit stress = $21300 \times (1 - 0.0055\, l/r)$
$l/r = 107.5$ = 8750#/□"

section required = 76400 ÷ 8750 = 8.70□"
Use 2[10 × 3½ @ 23.55# = 13.85□"

Diagonal U_4-L_5 Use 2[10 × 3½ @ 23.55# = 13.85□"
For connection 38800# T
½ × 9300. = 4650
43450# T Proportion the section for this stress
SR = 43450 ÷ 17000 = 2.55□" net

Approximate weight of truss.

Top chord	U_0-U_1	22.47□"	@ 34#	× 17.87	= 1370#
"	U_1-U_2	do		× 17.72	= 1360
"	U_2-U_3	23.66	@ 34#	× 17.61	= 1420
"	U_3-U_4	do		× 17.54	= 1410
"	U_4-U_5	27.66	@ 34#	× 17.50	= 1650
Bott chord	L_0-L_1	52.2#		× 17.50	= 910
	L_1-L_2-L_3	66.0		× 35.0	= 2310
	L_3-L_4-L_5	26.84□"	@ 34#	× 35.0	= 3190
End Post	U_0-L_0	22.47□"	@ 34#	× 20.0	= 1530
vertical	U_1-L_1	9.60□"	@ 3.4	× 23.6	= 770
Hanger	U_2-L_2	"	"	× 26.4	= 860
vertical	U_3-L_3	"	"	× 28.4	= 930
Hanger	U_4-L_4	"	"	× 29.6	= 970
Vertical	½ U_5-L_5	"	"	× 15.0	= 490
diagonals	U_0-L_1	52.2#	"	× 26.58	= 1330
	L_1-U_2	"	"	× 31.68	= 1650
	U_2-L_3	47.1#	"	× 31.68	= 1450
	L_3-U_4	47.1	"	× 34.4	= 1620
	U_4-L_5	47.1	"	× 34.4	= 1620
					26840#

For one truss 2 × 26840 = 53680#
Details say 37% - 19900
73580#

73580 ÷ 175 = 420# per lin ft of truss.

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Approximate weight of metal in one span

Stringers	187
Floor Beam complete	159
Lower Laterals	52
Upper Laterals	160
trusses	840

1398# × 175.0 = 244500#
weight of shoes assumed 6500
2510.00# 112 tons per span

Load on pin

weight of metal 244 500
" " flooring 2150 × 177.5 = 382.000
626 500 ÷ 4 = 156500
Live Load reaction 16500 × 4.5 = 74250
230750#

Design Pin for 240.000# stress.

Thickness of bearing plates = 1.42" assume $4\frac{1}{2}$" ϕ pin bearing area = 4.5 × 1.42" = 6.4"
Unit bearing = 240.000 ÷ (6.4 × 2) = 18800#/□" Bending moment = 120.000 × 1.67 = 200.000"#
$4\frac{1}{2}$" ϕ pin good for moment of 201300"# with unit stress of 22500#/□"

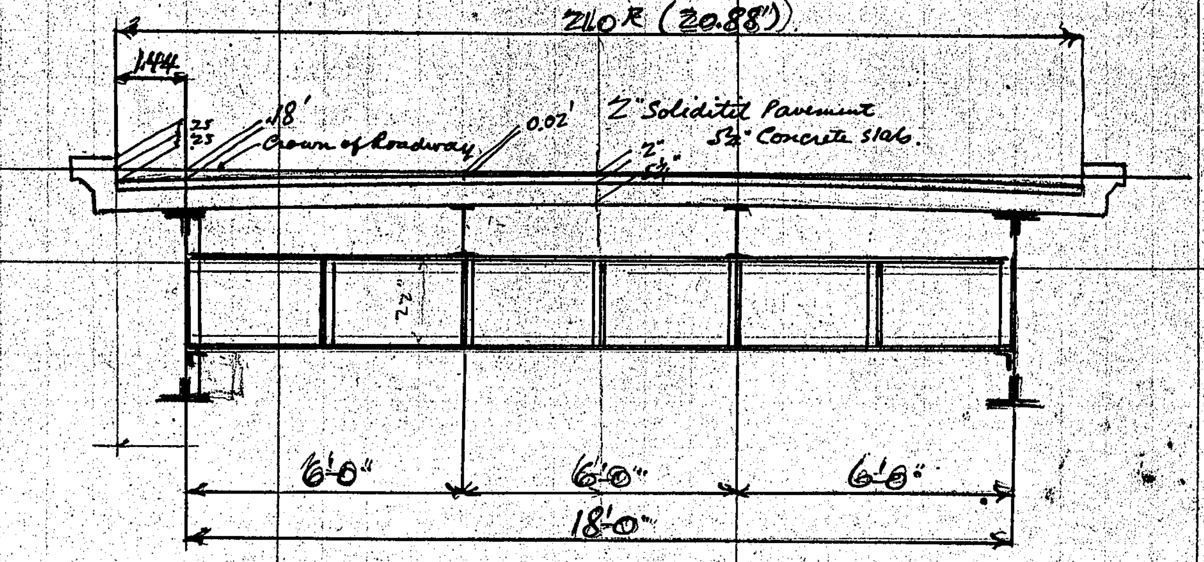
Shoes - design shoes for 240.000# load.
diameter of roller = 4" spacing $4\frac{1}{4}$" c to c of rollers.
610 × 4 = 2440# per lin inch. 240.000 ÷ 2440 = 98.5"
Use 5-4" @ 20".
2"
22" + 11" = 33" make bearing plate 24" × 33 = 792□"
Unit bearing = 240.000 ÷ 792 = 303#/□" on concrete

Bearing Area for fixed shoe same as for roller shoe.

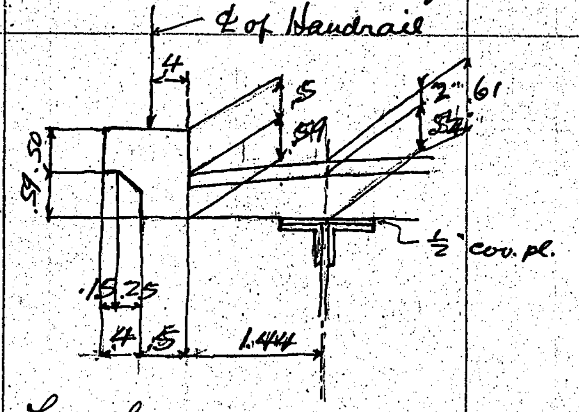
Design of Kasumi-Bashi for Okayama-Ken

Design of plate girder span 60'-0" out to out.

Cross Section of girder span



Design of Floor slab.
Overhanging Portion of slab.



moment about ℄ of girder.

Slab and Pavement	90# × 1.44 =	130#	× .72 =	93.5
coping	.5 × 1.09 @150 =	82	× 1.69 =	139.0
"	0.03 @150# =	5	× 2.02 =	10.1
"	0.20 @ 150 =	30	× 2.14 =	64.2
Handrail	say	60	× 1.84 =	110.4
		307#		417.2'#

Live Load 100#/□' moment = 150 × .72 = 108'#

Assume wheel concentration 0.5' out of ℄ girder distribution of load assumed 2.0 wide or 6435 ÷ 2 = 3217# moment = 3217 × 0.5 = 1608'#

Dead Load moment 417
2025'#

Effective depth required $= \sqrt{\frac{2025}{102}} = 4.45"$

Use reinforcement $\frac{2025 \times 12}{7/8 \times 4" \times 17000} = .409$ 2-½"□ bars 0.39□" per ft strip.

This will be all right counting flange Ls and cover plate of girder.

Intermediate Floor Slab. span length 6'-0"

Dead Load

2" Soliditit Pavement	24#
5½" Concrete slab.	66
	90# per sq ft

Dead Load moment = 1/10 × 90 × 6² = 324'#
Dead Load shear = 90 × 3 = 270#

Live Load motor truck rear wheel concentration 4950#
Impact 30% 1485
6435#
Front wheel concentration 2145#

Distribution of wheel concentration
Contact between wheel and pavement assumed 20cm = 0.66
2 × thickness of pavement = 0.34
Longitudinal distribution a = 1.00

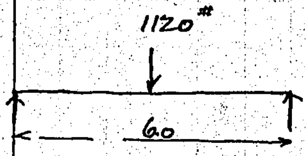
Design of Kasumi-Bashi for Okayama-Ken

Transverse distribution $b = 0.80 + .34 = 1.14$

Effective width $E = \frac{2}{3}(l + b) + a$

$= \frac{2}{3}(6 + 1.14) + 1.00 = 5.75$

Load per ft strip $= 6435 \div 5.75 = 1120^{\#}$

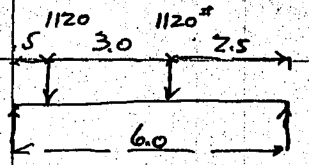


Moment due to single concentration at center $= 560 \times 3 = 1680'^{\#}$

For continuity of slab take moment as $0.8 \times 1680 = 1345^{\#}$

max End shear as simple beam

$1120 \times 8.0 \div 6.0 = 1494^{\#}$



Summary for moments and shears

	moment	shear
Dead Load	324	270
Live Load	1345	1494
	1669'#	1764#

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

$d = \sqrt{\frac{1669}{102}} = 4.04''$ Use slab 5¼" thick

Effective depth say 4.0"

Steel Area required $= \frac{1669 \times 12}{\frac{7}{8} \times 4 \times 17000} = .336$□" Use ½"⌀ bars 6" centers 0.39□" per ft

Max End shear $= \frac{1764}{\frac{7}{8} \times 4 \times 12} = 42$#/□"

Bond stress $u = \frac{1764}{\frac{7}{8} \times 4} = 505^{\#}$ try

2-½"⌀ @ $1.57 \times 130^{\#} = 409$

2-⅜"⌀ @ $1.18 \times 130 = 306$

715

Extra Reinforcement = ⅜" · 3'-0" long. 6" centers

This max shear for stringers only; at main girder no extra reinforcement required.

Design of Longitudinal Stringers spacing = 6'-0" span length 14'-4½" Overhang at both ends = 1'-3"

Dead Load floor $90^{\#} \times 6 = 540$

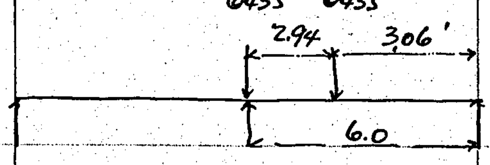
stringer assumed 35

$575^{\#}$

D.L. moment $= \frac{1}{8} \times 575 \times 14.37^2 = 14800'^{\#}$

D.L. shear $= 575 \times \frac{14.37}{2} = 4130^{\#}$

Live Load.



Load on stringer $6435 \times \frac{3.06}{6.00} = 3280$

6435

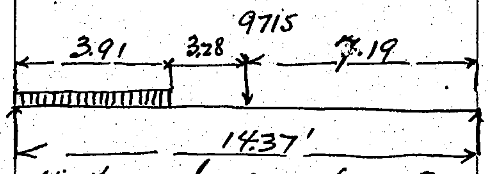
$9715^{\#}$

Rear wheel at center

moment $= \frac{9715}{2} \times 7.19 = 34950$

Uniform Load $318 \times 7.19 = 2290$

$37240'^{\#}$

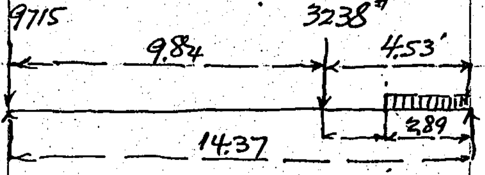


Uniform load $= 600 \times 3.91 = 2345^{\#}$

Reaction $= 2345 \times \frac{1.95}{14.37} = 318^{\#}$

Max End shear

Rear wheel = 9715 Front wheel $= \frac{1}{3} \times 9715 = 3238^{\#}$



Front wheel $3238 \times \frac{4.53}{14.37} = 1020^{\#}$

Rear wheel 9715

$10735^{\#}$

Uniform Load $1730 \times \frac{1.45}{14.37} = 175$

$10910^{\#}$

Uniform Load $600 \times 2.89 = 1730^{\#}$

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Summary for moments and shears

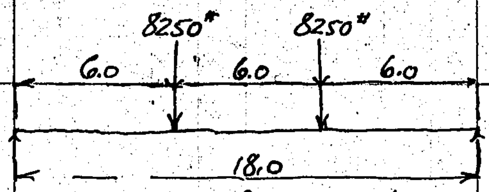
	Moments	Shear
Dead Load	14800	4130
Live Load	37240	10910
	52040'#	15040#

Section modulus reqd $= \frac{52040 \times 12}{17000} = 36.7$

Use 1I 12"×5" @ 31.99# SM = 36.64

Intermediate Floor Beam span length 18'-0" spacing = 14'-4½"

Dead Load



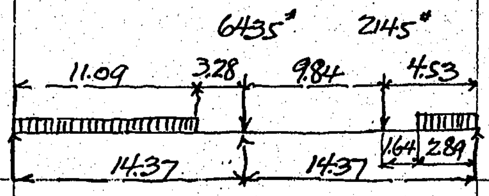
Concentration at stringer connection = 575 × 14.37 = 8250#
moment 8250 × 6.0 = 49500'#
Floor Beam $\frac{1}{8} \times 100 \times 18^2$ = 4050
53550'#

Dead Load shear = 9150#

8250
900
9150#

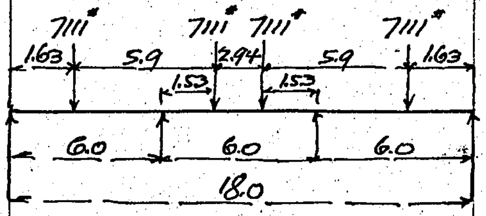
D.L. Floor beam assumed 100# per lin ft

Live Load



motor truck load Front wheel $2145\# \times \frac{4.53}{14.37}$ = 676
Rear wheel 6435
7111#

Uniform Load $600 \times 2.89 = 1730 @ \frac{1.45}{14.37} = 175$
$600 \times 11.09 = 6660 @ \frac{5.54}{14.37} = 2570$
2745#



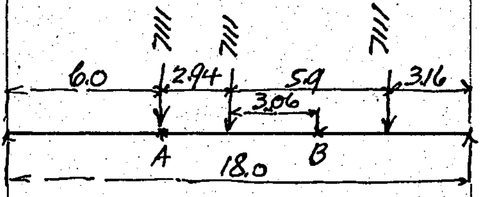
Reaction

motor truck load $7111 \times \frac{1.63}{6.0} = 1930$
7111
9041

Uniform Load 2745
11786#

Live Load Moment = 11786 × 6 = say 70700'#

Live Load shear motor truck loading wheel load at stringer connection



$7111 \times \frac{3.06}{6.0} = 3630$ at A
7111 − 3630 = 3481 at B
$7111 \times \frac{3.16}{6} = 3740$ at B

Concentration at A = 7111 + 3630 = 10741
" " B = 3481 + 3740 = 7221

Max End shear

$10741 \times \frac{2}{3} = 7160$
$7221 \times \frac{1}{3} = 2400$
9560#

From uniform Load 2745
12305# call this 12300#

Summary for moments and shears

	Moments	Shears
Dead Load	53550	9150
Live Load	70700	12300
	124250'#	21450#

Try 22 × 5/16 web $\frac{1}{8}$ web = 0.86□"
Depth of girder 22½" back to back of Ls
Effective depth 1.87 − .11 = 1.76'
Stress = 124250 ÷ 1.76 = 70600
Section reqd = 70600 ÷ 17000 = 4.15
− 0.86
3.29□" net

Use 2Ls 3 × 3 × 3/8" = 4.22□" gr or 3.56□" net

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CALCULATIONS FOR

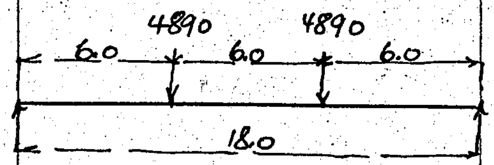
Design of Kasumi-Bashi for Okayama-Ken

Approximate weight of Intermediate Floor Beam

web	1 Pl	22 · 5/16	@ 23.38	· 18.0	=	420
flanges	4 Ls	3.3.3/8	@ 7.2	· 18.0	=	520
End Connection	4 Ls	3.3.3/8	@ 7.2	· 1.81	=	52
fillers	4 Pls	3.3/8	@ 3.83	· 1.17	=	18
Stiffs	4 Ls	3.2½.5/16	@ 5.6	· 1.86	=	42
Rivet heads + variation 3½%						37
						1089# ÷ 18 = 60# per lin ft

Assume weight of one floor beam 1100#

End Floor Beam
Dead Load



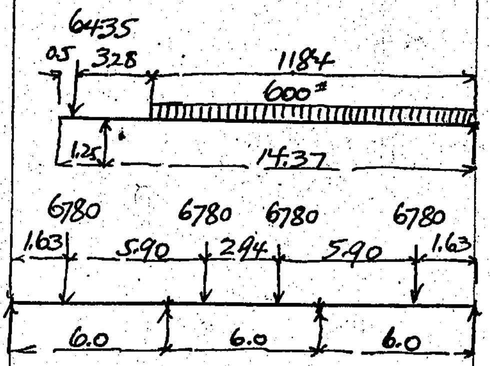
4890
720
5610# DL. Floor beam assumed 80# per ft

Concentration at stringer Conn $= 575 \cdot \frac{15.62^2}{14.37 \cdot 2} = 4890$#

Moment $4890 \cdot 6.0 = 29400$
Floor beam $\frac{1}{8} \cdot 80 \cdot 18^2 = 3240$
32640 '#

Dead Load shear = 5610#

Live Load



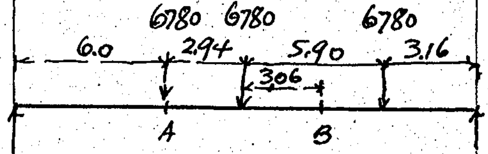
Motor truck load $6435 \times \frac{15.12}{14.37} = 6780$#

Uniform Load $600 \times 11.84 = 7100$# @ $\frac{5.92}{14.37} = 2930$#

Concentration $6780 \times \frac{1.63}{6.0} = 1840$#
6780
8620
Uniform load 2930
11550#

Moment = 11550 · 6 = 69300 '#

Live Load Shear



Motor truck loading
wheel load at stringer Connection.

$6780 \times \frac{3.06}{6.00} = 3460$ A
$6780 - 3460 = 3320$ B
$6780 \times \frac{3.16}{6.0} = 3570$ B
Concentration at A = 6780 + 3460 = 10240
" " B 3320 + 3570 = 6890

Max End shear

$10240 \cdot \frac{2}{3} = 6830$
$6890 \cdot \frac{1}{3} = 2300$
wheel load 9130#
Uniform load 2930
12060#

Summary for moments and shears

	Moment	Shear
Dead Load	32640	5610
Live Load	69300	12060
	101940 '#	17670#

Try 22" × 5/16 = 6.870 □" ⅛ web = 0.86 □" depth of girder 22½" back to back of Ls
Effective depth 1.76 Stress = 101940 ÷ 1.76 = 57800# SR = 57800 ÷ 17000 = 3.40
− .86
2.540 □" net

Use 2Ls 3.3.5/16 = 3.560" gr or 3.010" net

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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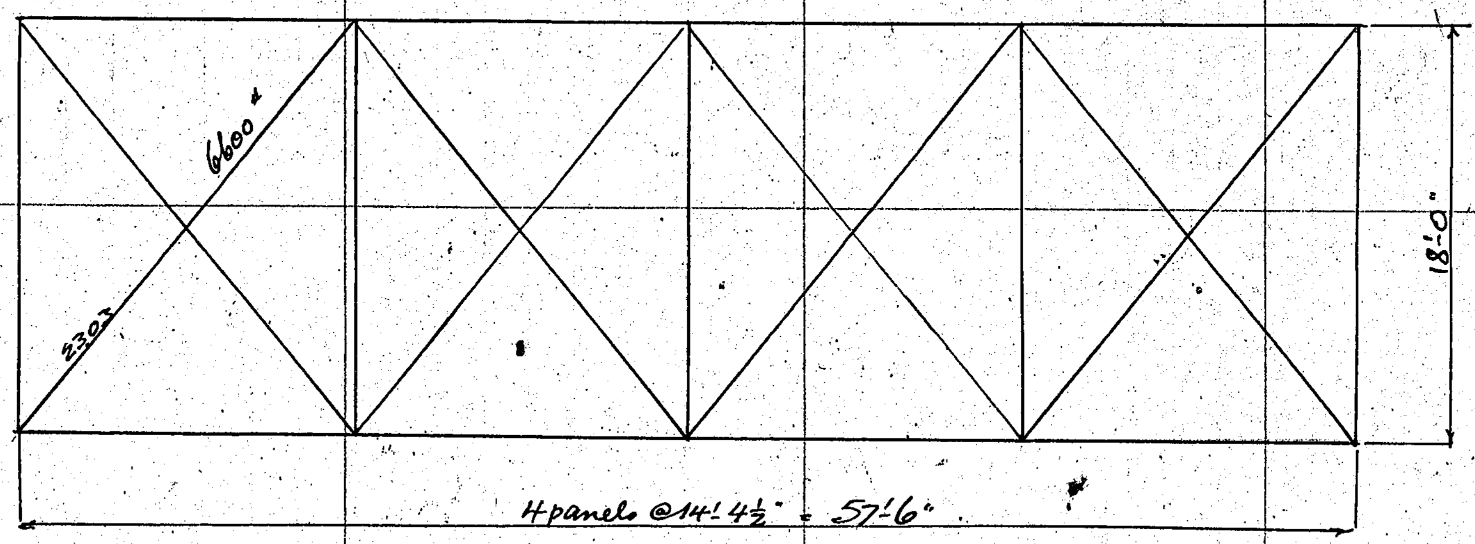
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Approximate weight of End Floor Beam
End Connected to stiffener of girder.

web.	1Pl. 22" × 5/16	@ 23.38#	× 18.0	= 420	
flanges	4Ls 3×3×5/16	@ 6.1	× 18.0	= 440	
End Connection	4Ls 3×3×3/8	@ 7.2	× 1.81	= 52	← Omitted
Stiffeners	10Ls 3×2½×5/16	@ 5.6	× 1.86	= 104	
		Rivet heads say		37	
				1001# ÷ 18 =	55.5# per ft.

Design of main girder. Lateral Bracings
Wind pressure assumed 30#/□' Exposed Area say 8' 240# per lin ft.



$\sec\theta = \frac{23.03}{18.00} = 1.28$ panel concentration = 240 × 14.37 = 3440#
Reaction = 3440 × 1.5 = 5160# stress in End Panel = 6600# Use 4 Rivets for conn.
Use 2Ls 4"×3"×3/8" @ 6.84 × 22.0 = 300
do. 300
Center connection 25
Girder connection say 80
705 × 4 = 2820#
2820 ÷ 60 = 47# per lin ft. of span.

Design of Main Girder
Dead Load. Floor Outside of main girder see pp 16 2@307 = 614
90# × 18 = 1620
2234#

Dead Load metal.
Stringers 2@35 = 70# × 14.37 = 1050# For intermediate panel 2234 × 14.37 = 32100#
Floor Beam. 1100 " End panel point 2234 × 8.44 = 18800#
Lateral Bracings 705
For intermediate panel pt. 2855 ÷ 2 = 1427#

Stringers 70# × 8.44 = 590
End Floor Beam 1000
Lateral Bracing say 350
For End panel 1940 ÷ 2 = 970#

Summary for Panel concentration from Floor system for one girder

	Intermediate panel	End panel
Floor's	16050	9400
metal.	1427	970
	17477#	10370#

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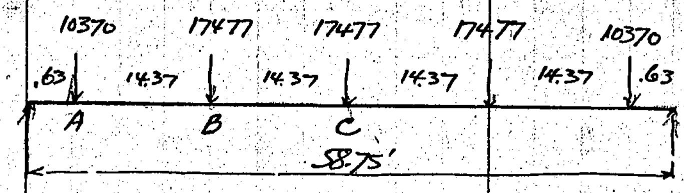
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Dead Load Moment



Dead Load reaction = 36586#
D.L. girder 235×30 = 7050
43636
Dead Load of
main girder assumed 235# per ft.

Moment at C 36586 · 29.37 = 1074500
17477 × 14.37 = 251100
10370 · 28.74 = 298100
- 549200
525300'#

Moment at B 36586 · 15.0 = 548.800
10370 · 14.37 = - 149000
399800'#

Moment at A 36586 · 0.63 = · 23000'#

Moment at C = $\frac{1}{8}$ · 235 · 58.75² = 101.400'#

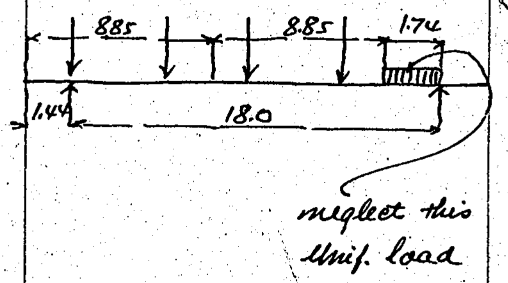
Moment at B = $\frac{235}{2}$ · 15.0 · 43.75 = 77200'#

Moment at A = $\frac{235}{2}$ · 0.63 · 58.12 = 4300'#

Summary for Dead Load moment

	C	B	A
Concentration	525300	399800	23000
Uniform	101400	77200	4300
	626700'#	477000'#	27300'#

Live Load
Motor trucks loading, Impact = $\frac{20}{60+l} = \frac{20}{77.9}$ = 25.7%



9.85
1.74
10.59

Rear wheels 2@4950 = 9900
Impact 25.7% = 2545
12445#

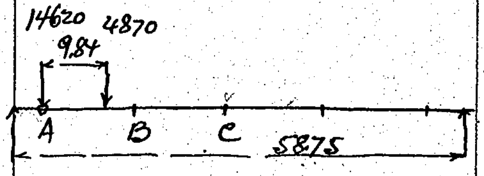
Front wheels 4130#

Rear wheel reaction 2@12445 · $\frac{10.59}{18.0}$ = 14620#

Front wheel reaction = ⅓ · 14620 = 4870#

Uniform load for front and rear of motor trucks 19.44' wide
Uniform load 100#/□' Reaction on girder = 1944 · $\frac{9.72}{18}$ = 1050# per lin ft.

Moment due to motor trucks loading

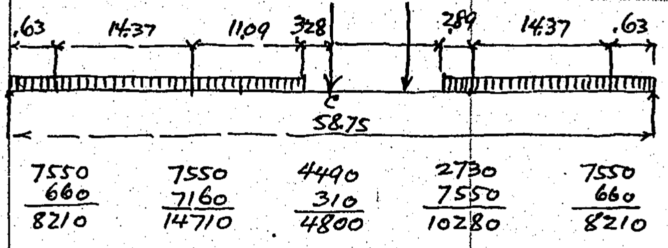


Moment at C
14620 × 29.37 = 429000
4870 × 19.53 = 95100
524100 ÷ 58.75 = 8930#
m = 8930 × 29.37 = 262000'#

Moment at B.
14620 · 43.74 = 649000
4870 · 33.90 = 165000
814000 ÷ 58.75 = 13860#
m = 13860 · 15.0 = 207900'#

Moment at A
14620 · 58.11 = 850000
4870 · 48.27 = 235000
1085000 ÷ 58.75 = 18470
m = 18470 × 0.63 = 11600'#

Moment due to uniform load.
1050# per lin ft of girder



Moment at C
10280 × 15.0 = 154200
4800 × 29.37 = 141000
14710 × 43.74 = 644000
939200 ÷ 58.75 = 15950
8210
24160#

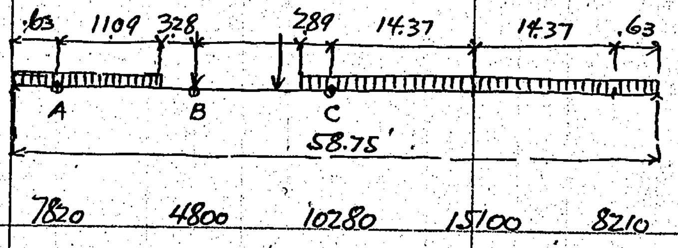
Design of Kasumi-Bashi for Okayama-Ken

Moment at C

24160 × 29.37 = 710000
1050 × 26.09 × 16.32 = − 447000
263000#

Moment at B

1050# per ft



Reaction

8210	×	0.63	=	5200
15100	×	15.00	=	226200
10280	×	29.37	=	302000
4800	×	43.74	=	210000
7820	×	58.12	=	454500

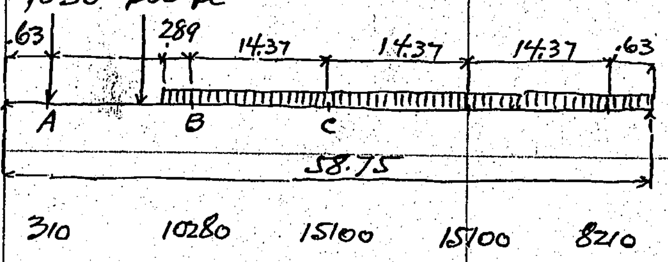
1197900 ÷ 58.75 = 20400#

Moment

20400 × 15.0 = 306000
1050 × 11.72 × 9.12 = − 112400
193600 1#

Moment at A

1050# per ft



Reaction

8210	×	0.63	=	5200
15100	×	15.00	=	226200
15100	×	29.37	=	443000
10280	×	43.74	=	450 000
310	×	58.12	=	18 000

1,142,400 ÷ 58.75 = 19430#

Moment

19430 × 0.63 = 12250#

Summary for Live Load

	C	B	A
motor truck loading	262000	207900	11600
Uniform Load	263000	193600	12250
	525000 1#	401500 1#	23850 1#

Summary for Dead and Live Loads

	C	B	A
Dead Load	626700	477000	27300
Live Load	525000	401500	23850
	1151700 1#	878500 1#	51150 1#

End Shears

Dead Load shear	43600	
Live Load shear	18470	motor truck
" " "	19430	Uniform load
	81500#	

Section of main girder moment = 1151700 1# shear = 81500#
~~web assumed~~ 48 × 3/8 = 18.0 □" ⅛ web = 2.25 □" depth = 4'-0½" back to back of Ls
Effective depth = 4.04 − .16 = 3.88 Stress = 1151700 ÷ 3.88 = 297000#
Section required = 297000 ÷ 17000 = 17.45
− 2.25
15.20 □" net

Use	2 Ls 6 × 6 × ½	11.50 □" gr	9.62 □" net
	1 Pl 13 × ½	6.50	5.56
		18.00	15.18 net

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CONSULTING ENGINEER
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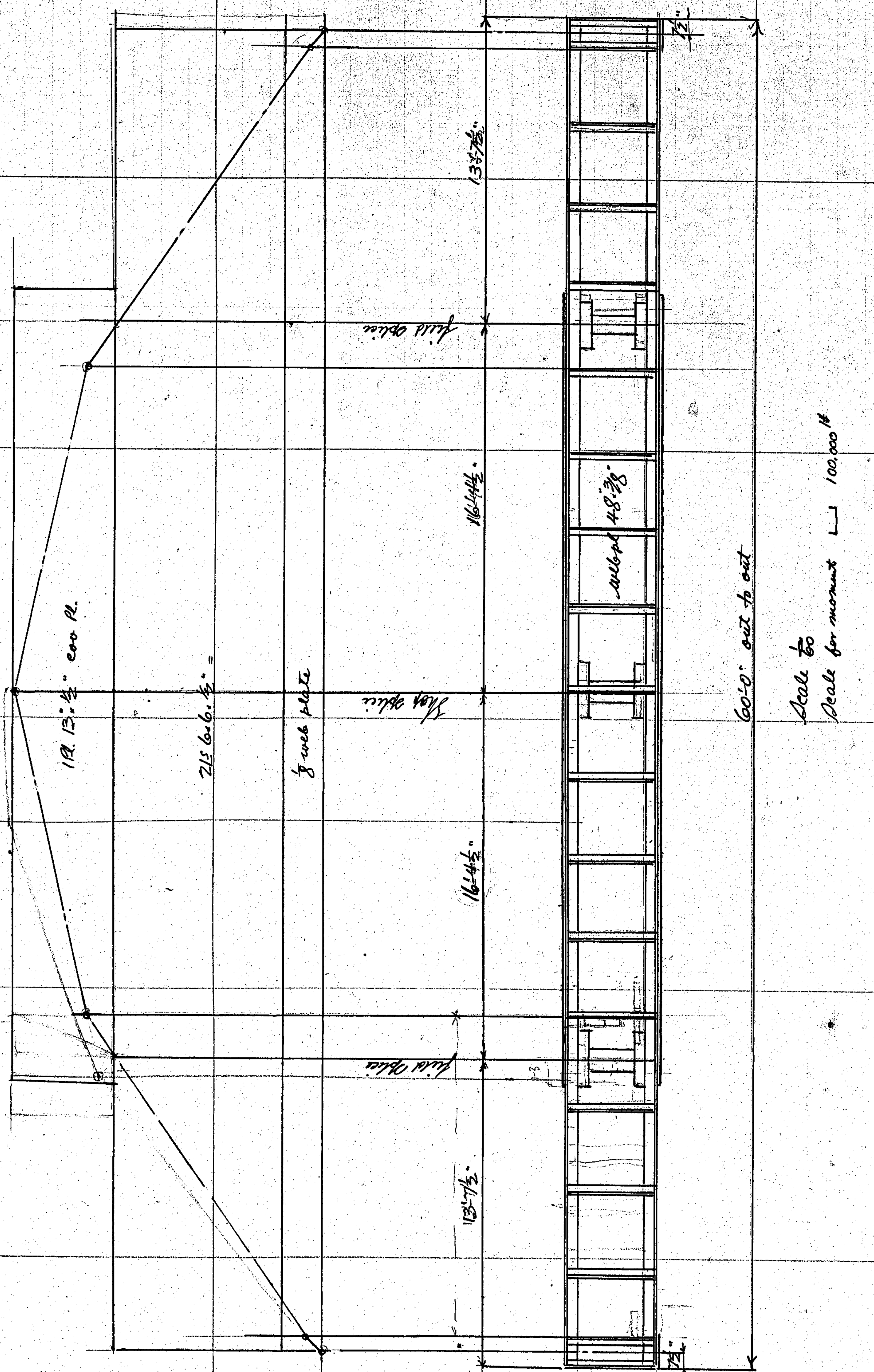
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken



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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Approximate weight of main girder

web	1 Pl	48"×3/8	@	61.20	×	60.0	=	3570
flange	4 Ls	6×6×1/2	@	19.60	×	60.0	=	4710
cooples	2 P/s	13×1/2	@	22.10	×	36.0	=	1590
End Stiffs	8 Ls	5×3½×1/2	@	13.60	×	3.96	=	432
fills	8 P/s	3½×1/2	@	5.95	×	3.00	=	143
Int. Stiffs	24 Ls	5×3½×5/16	@	8.7	×	4.04	=	845
Stiffs Floor Beam	6 Ls	5×3½×3/8	@	10.4	×	3.96	=	247
filler	4 P/s.	3½×1/2	@	5.95	×	3.00	=	72
web splice	12 P/s.	9×1/2	@	15.3	×	3.5	=	642
" "	4 P/s.	12½×3/8	@	15.94	×	1.5	=	95
" "	2 P/s	12½×1/2	@	21.25	×	1.5	=	64
flange splice	4 Ls	6×6×3/8	@	14.9	×	4.0	=	238
sole pls	2 P/s	15×1"	@	51.0	×	1.75	=	178
								12826
		5%						640
								13466 ÷ 60 = 225# per ft.

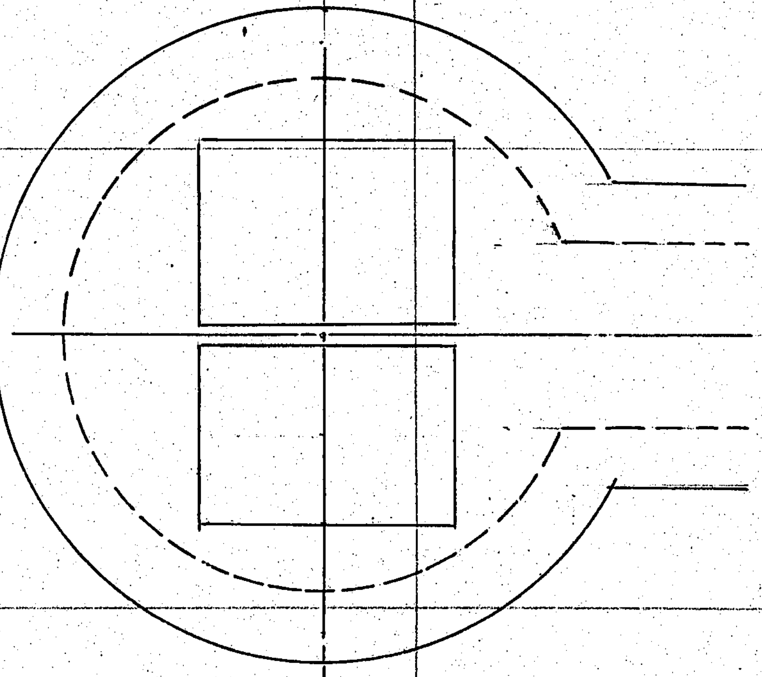
Approximate weight of metal in one span

Stringers	70#×60 =	4200
Floor Beam	3@1100 = 3300	
	2@1000 = 2000	
		5300
Lateral Bracings		2820
~~Girders~~	2@13500 =	27000
Girders misc. steel say		480
		39800# or 17.75 tons per span

For max bearing assume Load - 90.000 for dead and Live Loads

Bearing plate 15"×21" = 315□" Unit bearing = 90.000 ÷ 315□" = 286#/□" Ok

Design of piers for truss spans (TP2-3-4-5-6-7)



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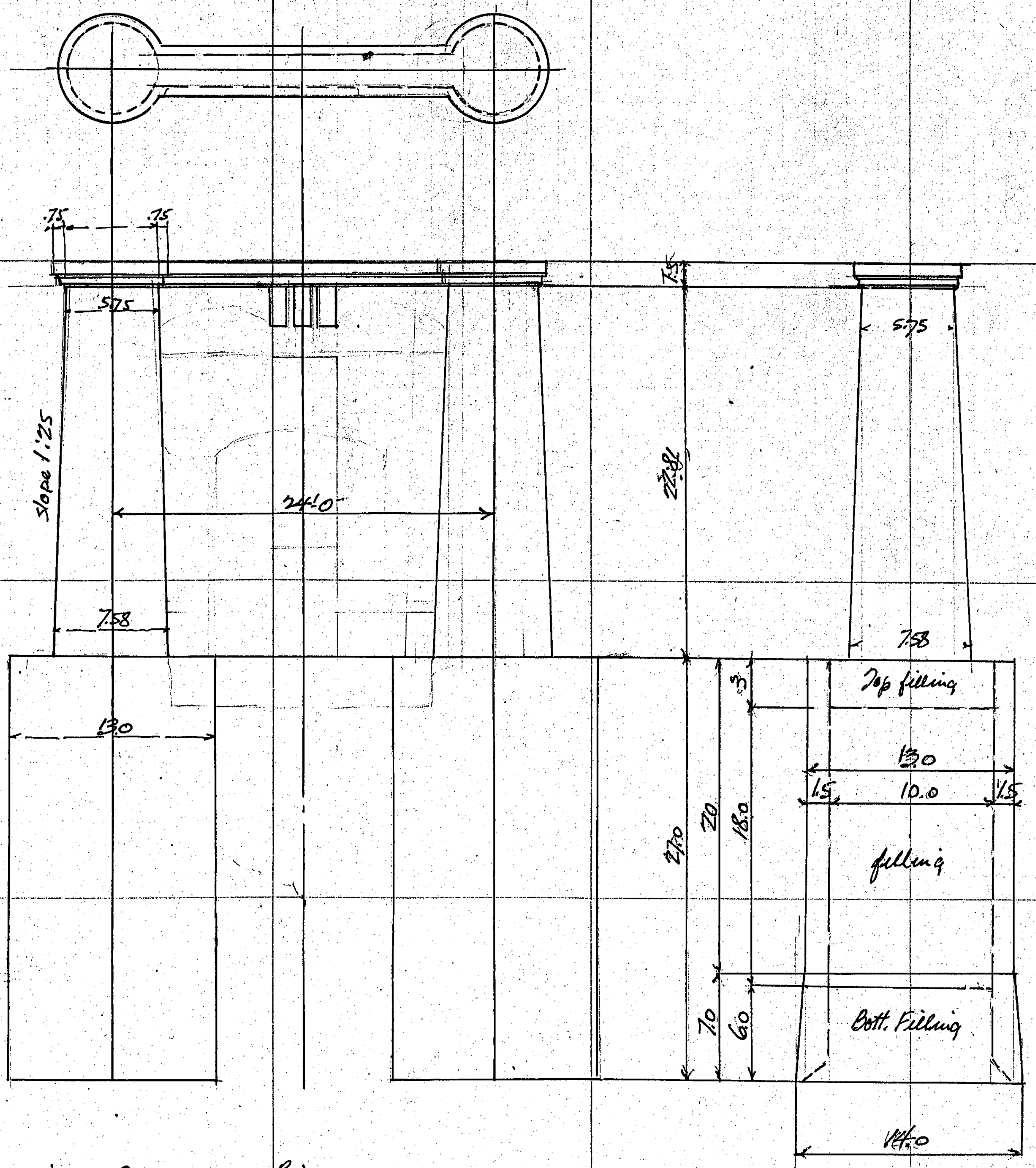
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken



Approximate Concrete in Pier.

coping 7.25ϕ - 41.3
16.75 × 3.5 = <u>58.6</u>
99.9 × 1.5 = 150

shaft 5.75ϕ = 26.0
7.58ϕ = <u>45.1</u>
71.1 ÷ 2 = 35.55 × 22.81 = 810

web 2 × 17.33 × 22.81 = 790

Well :-

shell - 13.0 dia 132.7
10.0 dia <u>78.5</u>
54.2 × 20 = 1084

14'dia - 10'dia = 75.4
13'dia - 10'dia = <u>54.2</u>
129.6 ÷ 2 = 64.8 × 7 = 453

shaft and web

coping - 150
shaft 2 @ 810 = 1620
web - <u>790</u>
2560 cubic ft
or 11.85 cubic tsubo

wt = 11.85 @ 30200 = 358000#

upper portion 1084
Lower portion <u>453</u>
1537

2 @ 1537 = 3074 cubic ft
or 14.2 cubic tsubo

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Bottom filling 6.0 × 78.5 = 471
Top filling 3.0 × 78.5 = 235.5
706.5 × 2 = 1413 cubic ft
6.55 cubic tsubo

Intermediate filling 18 × 78.5 = 1412 @ 2 = 2824 cubic ft
a 1310 cubic tsubo

Summary of Concrete in well.
shell - 1:2:4 14.20
Top and bottom filling 1:2:4 6.55
Intermediate filling lean concrete 13.10
33.85 cubic tsubo

weight = 33.85 × 30200 = 1,020.000 #

Total dead Load of pier shafts 358 000
wells 1020.000
1,378,000 #

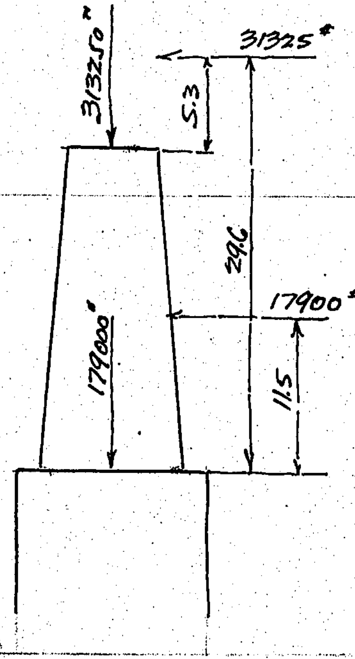
Superimposed Load on pier
weight of metal 244500 #
flooring 2150 # × 177.5 = 385.000
Dead Load 626.500 #

Live Load
Uniform Live Load for 2 - 177.5 spans $w = \frac{100.000}{170 + 105} = 364$ kg/□ m
or say 75 #/□'

Total Live Load = 75 × 21. × 177.5 = 280.000 #
" Dead Load - 626.500
906.500 #
weight of pier 1378 000
2284500 #
Circumferencial friction 27 × 200 × 40 = 216 000
2068 500 # net

Bottom Area of well 2 - 14' dia = 2 × 153.9 = 307.8
Unit Bearing Pressure = 2068500 ÷ 307.8 = 6730 #/□'
or 3.0 tons/□'

Reinforcement in shaft due to Earthquake.



Assumed accellalation of earthquake = 1000 $\mathrm{mm/sec^2}$
Horizontal force
superimposed Dead Load 313250 × 0.1 = 31325
shaft 179000 × 0.1 = 17900
492250

Moment about bottom of shaft
= 31325 × 29.6 = 926 000
17900 × 11.5 = 206000
1,132,000 ′#

Moment of inertia of section
Concrete $0.049 \times 7.58^4 = 162.0\ (\mathrm{ft})^4$
Steel $15 \times \frac{0.049 \times 7^2}{8} = 4.5$
166.5

Fibre stress = $\frac{1.132.000 \times 379}{166.5} = \pm 25700$ #/□' or 178 #/□"

16 - 3/4" ϕ bars assumed =
Steel Area = 7.04 □"
steel % = 0.1

Equivalent Concrete area
concrete = 7.58 dia = 45.12 × 144 = 6500 □"
steel 15 × 7 = 105
6605 □"

Direct Pressure = 492250 ÷ 6605 = 82 # per sq inch.

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

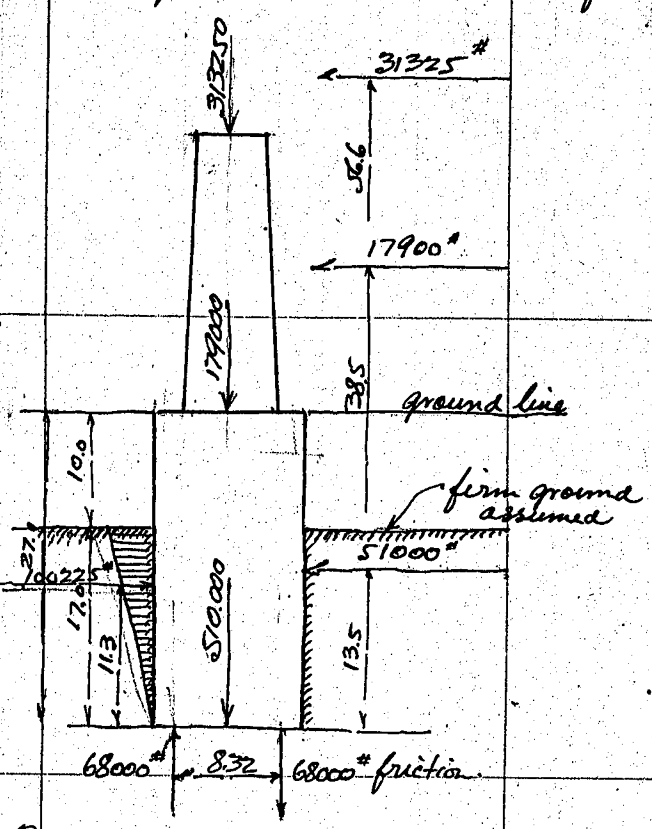
Summary for fibre stress

178 T	178 C
82 C	82 C
96#/□T	260#/□"

Approximate stress in shaft neglecting tension of Concrete
From Journal of The Institute of Japanese Architects 大正十一年六月号 "Reinforced Concrete Round Column subjected to Bending and Direct Load by Messrs Naito and Kawai.
steel % = 0.1 Concrete stress = $5.0 \times \frac{492250}{6605} = 410$#/□" about

neutral axis = 0.5 Steel Stress = $15 \times 410 \times \left(\frac{7.33}{7.58 \times 0.5} - 1\right) = 5700$#/□"

Bearing Pressure at Bottom of Base during Earthquake



Moment taken at bottom of base.
Firm ground assumed at 10' below of river bed
Side resistance of firm ground and frictional resistance along circumference of well considered below line of firm ground.
moment about bottom of base :-

31325 × 56.6 = 1.775.000
17900 × 38.5 = 690.000
51000 × 13.5 = 690.000
100225# 3.155.000 1#

100225 × 11.3 = 1.130.000
68000 × 8.32 = 565.000
-1695.000
1,460,000 1#

Friction of well surface = 40.8 × 200#/□" say 8000# pr ft
Total friction = 17 × 8000 = 136000#

Direct Load superimposed Load 313250
shaft 179000
well 510000
1002250#
Less friction 136000
866250# net

Bottom area 14' dia = 153.9 □'

Unit Pressure = $\frac{866250}{153.9}$ = 5640#/□' or 2.52 tons/□'

Moment of inertia of bottom area of base. = 0.049 × 14⁴ = 1885

Fibre stress due to moment = $\frac{1460.000 \times 7}{1885}$ = 5420#/□' or 2.42 tons/□'

Summary for soil pressure (Extreme Fibre).

Bending moment	2.42 C	2.42 T
Direct load	2.52 C	2.52 C
	4.94 tons/□' C	0.10 tons/□' C

Design of well

Earth pressure during well sinking
at 20' P = $\frac{1}{3} \times 100 \times 20$ = 667# per ft assumed
at 27' P = $\frac{1}{3} \times 100 \times 27$ = 900# per ft. "

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken

Depth at 20' thickness of well assumed 1.5'
Diameter of circle along neutral axis. = 11.5' m = $\frac{1}{16} \cdot 667 \cdot 11.5^2$ = 5520 '#
Depth at 27' thickness of well assumed 2.0
Diameter of circle on neutral axis = 11.0 m = $\frac{1}{16} \cdot 900 \cdot 11^2$ = 6800 '#

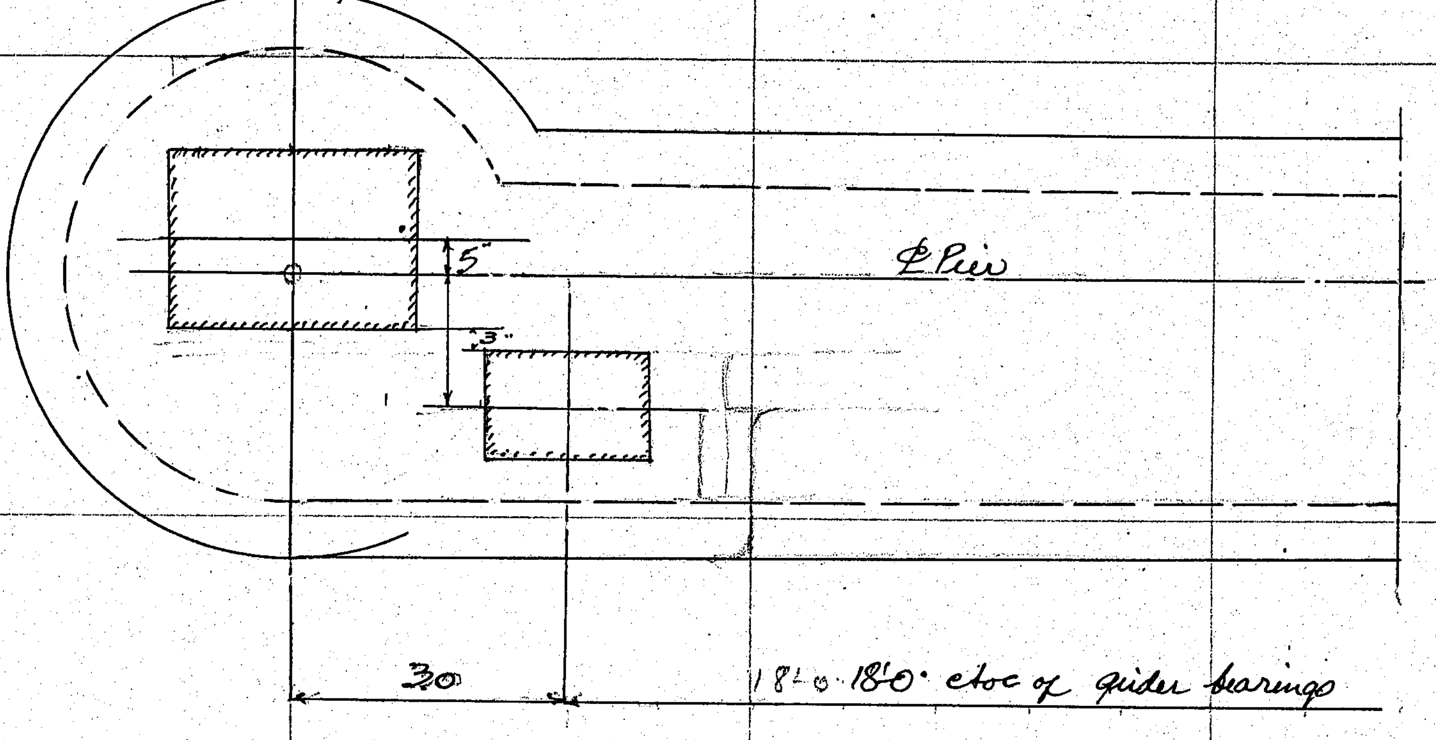
Reinforcement required at bottom = $\frac{6800 \cdot 12}{\frac{7}{8} \cdot 22'' \cdot 17000}$ = .25 □" per ft

Reinforcement required at 20' = $\frac{5520 \cdot 12}{\frac{7}{8} \cdot 16 \cdot 17000}$ = .278 □" per ft.

Use ½"ϕ bars 12" centers inside and outside = (0.40) 22

vertical Bars - (3/4)

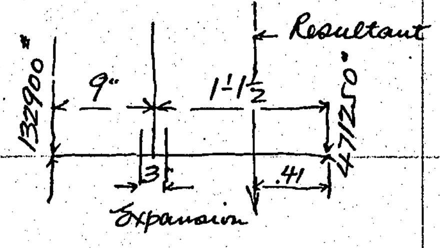
Design of Pier for between girder and truss spans (TP1 and 8)



Live Load for truss and girder spans
Total length = 177.5 + 60 = 237.5' or 72.5 meter
w = $\frac{100.000}{170+72.5}$ = 412 kg/m² or say 85#/□'

Truss span Floor and metal 313250
Live Load $85 \times 21 \times \frac{177.5}{2}$ = 158000
471250#

Girder span metal 39800 ÷ 2 = 14900
Floor 2150 × 30 = 64500
Dead Load 79400#
Live Load 85 × 21 × 30 = 53500
132900
604150# Total Load.



Arm for resultant := 132900 × 1.87 = 248 000 '#
arm = 248000 ÷ 604150 = 0.41' call this 5"

For dead Load only 313250
79400 × 1.87 = 148500
392650
Arm = 148500 ÷ 392650 = .38' call this 5"

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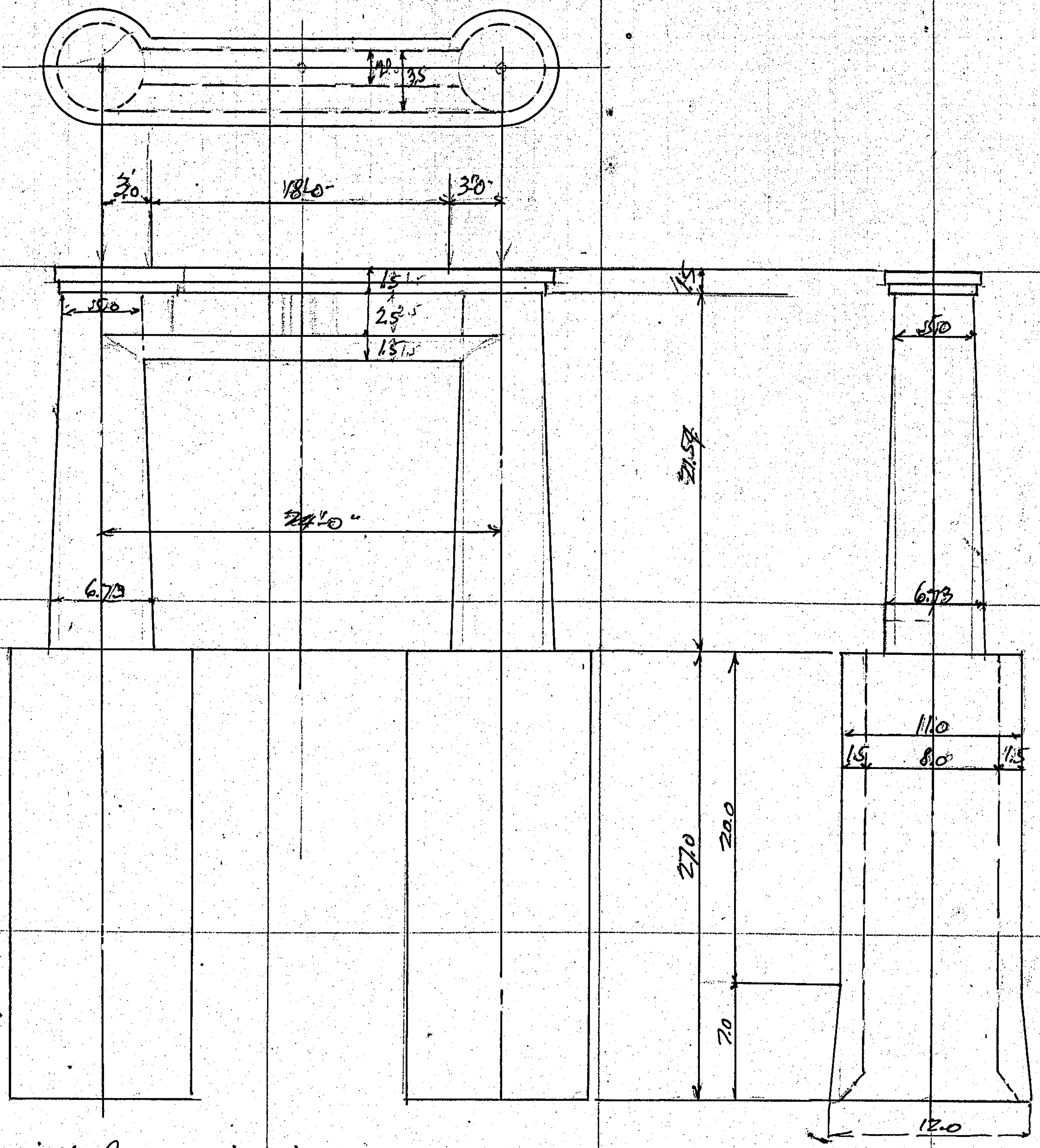
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken



Approximate Concrete in pier

Coping 6.5' dia = 33.2
18.0 × 5 = 90.0
123.2 × 1.5 = 185.

shaft 5' dia = 19.6
6.73 dia = 35.6
55.2 ÷ 2 = 27.6 @ 21.59 = 595.

web 18.13 × 2 × 21.59 = 593
18.0 × 3.25 × 1.5 = 88
681

Summary
Coping 185
Shaft 2@595 = 1190
web 681
2056 cubic shaku
or 9.5 cubic tsubo
wt = 9.5 @ 30200 = 287000 #

Well shell 11' dia 95.0
8' dia 50.3
34.7 × 20 = 694
342
1036

12' dia 113.1
8' dia 50.3
62.8
34.7
97.5 ÷ 2 = 48.75 @ 7 = 342.

2 × 1036 = 2072 cubic shaku
or 9.57 cubic tsubo

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken.

Top and bottom filling $503 \times 9 = 4527$ @ 2 = 905 ft^3 or 4.19 立坪

Intermediate filling $503 \times 18 = 905$ @ 2 = 1810 or 8.38 立坪

Summary of Concrete in wells

shell	9.57
filling	4.19
"	8.38
	22.14 @ 30200 = 670.000#

weight of shaft 287.000

957000#

superimposed Load dead and Live Loads pp 28 604150

1561150#

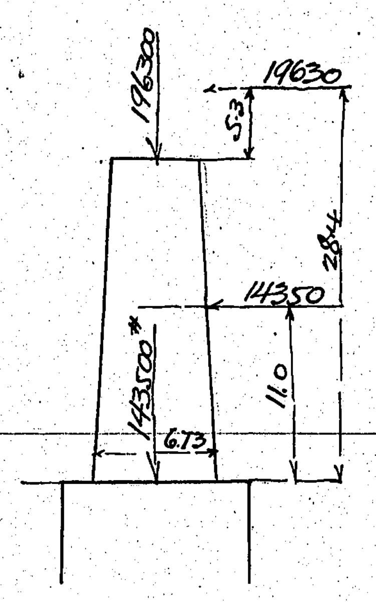
Circumferential friction $34.5 \times 200^{\#/\square'} \times 27$ − 188000

1373150# net

Bottom Area 12' dia 2 @ 113.1 = 226.2 □'

Unit Bearing = 1373150 ÷ 226.2 = 6060 #/□' or 2.71 tons/□'

Reinforcement in shaft due to Earthquake



16-3/4"φ bars = 7.04 □"
or 0.049 □'

Assumed acceleration of Earthquake = $1000\ ^{mm}/_{sec^2}$

Horizontal force

superimposed dead Load $196300 \times 0.1 = 19630$#

weight of shaft $143500 \times 0.1 = 14350$#

339800

Moment about bottom of shaft.

$19630 \times 28.4 = 558\,000$

$14350 \times 11.0 = 158\,000$

716000'#

Moment of Inertia of section

Concrete $0.049 \times 6.73^4 = 100.8$

Steel $15 \times 0.049 \times \frac{6.23^2}{8} = 3.6$

104.4

Fibre stress = $\frac{716.000 \times 3.37}{104.4} = 23150$ #/□' or 161 #/□"

Equivalent concrete area of section

Concrete 6.73 dia $35.57 \times 144 = 5110$

steel $15 \times 7 = 105$

5215 □"

Direct Pressure 339800 ÷ 5215 = 65 #/□"

Summary of stress (in fibre).

Bending stress	161 C	161 T
Direct stress	65 C	65 C
	226 #/□" C	96 #/□" T

This value approximately same as shown on pp 27 for intermediate truss piers
16-3/4"φ reinforcement is ample for this section of shaft.

Bearing Pressure at Bottom of Base during Earthquake

Moment taken at bottom of base; Firm ground assumed at 10' below river bed; side resistance of firm ground and circumferential friction of well considered below this line.

Friction of well surface = $34.5 \times 200 \times 17' = 117000$#

Arm for friction couple = $11. \times .64 = 7.03'$

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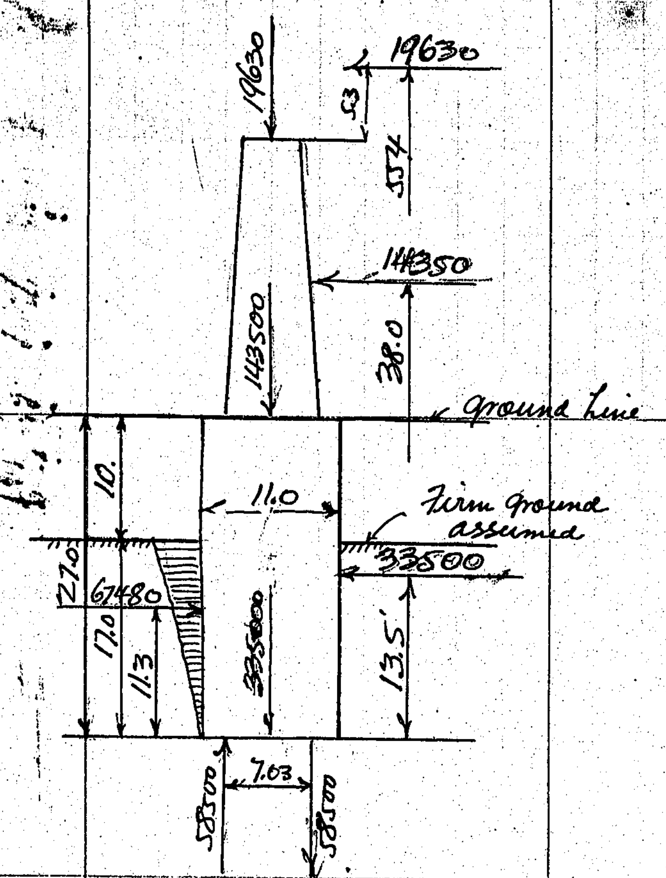
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken



Moment about bottom of Base

19630 × 55.4 = 1.090.000
14350 × 38.0 = 545.000
33500 × 13.5 = 452.000
67480# 2087000

67480 × 11.30 = 764000
58500 × 7.03 = 411000
−1175000
912000 1#

Direct Load
Superimposed Load 196300
Shaft 143500
Well 335000
674800
Less friction −117000
557800#

Bottom area 12' dia = 113.1 □'

Unit Bearing = 557800 ÷ 113.1 = 4930#/□' or 2.2 tons/□'

Moment of inertia of bottom area = $0.049 \times 12^4 = 1015 \text{ (ft)}^4$

Fibre stress due to bending = $\frac{912000 \times 6}{1015}$ = 5380#/□' or 2.4 tons/□'

Summary for soil pressure (Extreme fibre)

Bending	2.40 C	2.40 T
Direct	2.20 C	2.20 C
	4.60 tons/□' C	0.20 tons/□' T.

Shell :-
Earth pressure during well sinking assumed
at 20' = 667# per sq ft
at 27' = 900# " " "

At 20' thickness 1.5' $m = \frac{1}{16} \times 667 \times 9.5^2 = 3760$ 1#

At 27' thickness 2.0 $m = \frac{1}{16} \times 900 \times 10.0^2 = 5620$ 1#

Reinforcement required at bottom $= \frac{5620 \times 12}{\frac{7}{8} \times 22 \times 17000} = 0.20$□" per ft.

Reinforcement required at 20' $= \frac{3760 \times 12}{\frac{7}{8} \times 16 \times 17000} = 0.19$□" per ft.

Design of Piers for Girder Spans. (GP6-7-8-9-10)

Load on pier. Dead Load metal 39800
" " Floor 2150 × 60.17 = 129500
169300#

Live Load for 2-60' spans $w = \frac{100.000}{170 + 36.6} = 485$ kg/m² or say 100#/□'

Live Load 21. × 100 × 60.17 = 126000#

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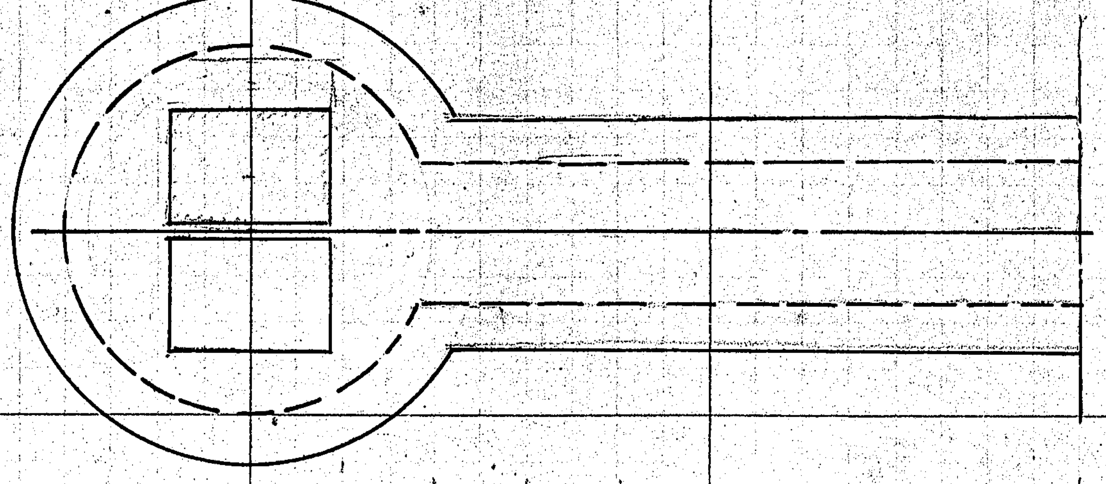
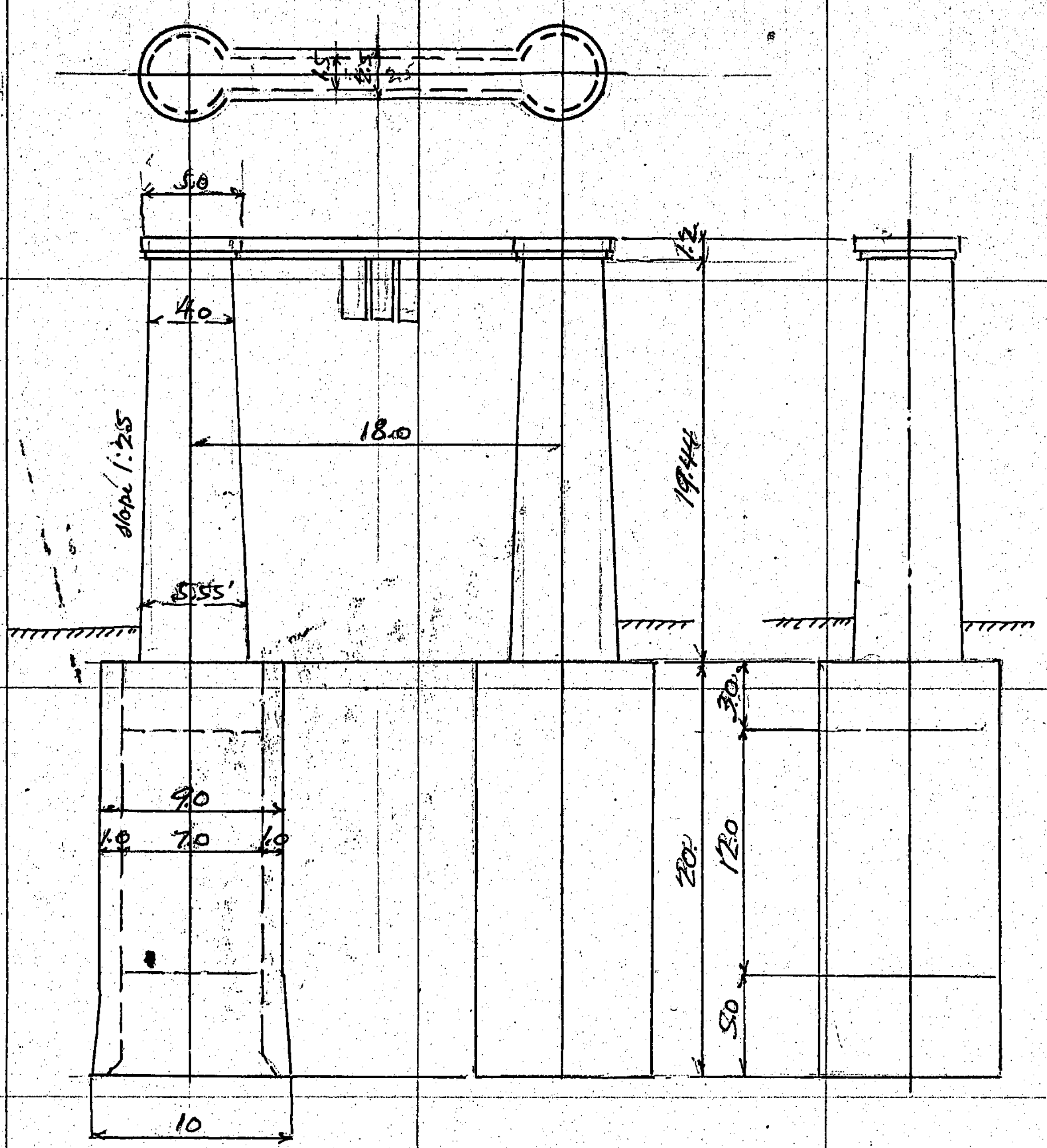
CALCULATIONS FOR

Design of Kasumi-Bashi for Okayama-Ken





Approximate Concrete in pier
coping
5' dia 19.6
13 × 2.5 32.5
52.1 × 1.2 = 63.0
shaft.
~~4~~'dia 19.6 12.6
5.55 dia 24.2
36.8 ÷ 2 = 18.4
vol = 18.4 × 2 × 19.44 = 715

web. = 1.5 × 13.23 × 19.44 = 386.

Summary
coping 63
shaft 715
web 386
1164 ÷ 216 = 5.4 立坪
weight = 5.4 @ 30200 = 163000 #

Well :-
shell
9' dia 63.6
7' dia 38.5
25.1 × 15 = 377
10'φ 78.5
7'φ 38.5
40.0
25.1
65.1 ÷ 2 = 32.55 @ 5 = 163
540
540 × 2 = 1080 or 5.00 立坪

Top and bottom filling 38.5 × 8 × 2 = 615 2.85 立坪
Intermediate filling 38.5 × 12 × 2 = 924 4.28 " "
shell - 5.00 " "
12.13 " "

weight of well. 12.13 × 30200 = 366000 #
" " shaft 163.000
529.000 #

Superimposed load DL 169300
LL 126000
295300
824300 #

Less friction 9'φ 283 × 200 × 20 - 118000
706300 #

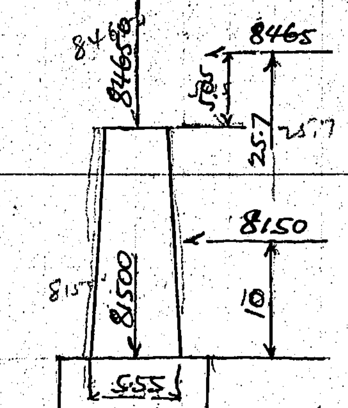
増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Design of Kasumi-Bashi for Okayama-Ken.

Bearing area at bottom 2-10' dia = 2 × 78.5 = 157.0

Unit Bearing Pressure = 706300 ÷ 157. = 4500 #/□' or 20.1 tons/□'

Reinforcement in shaft due to Earthquake.



16-5/8"ø = 4.80□"
0.033 □'

Assumed acceleration of Earthquake = 1000 mm/sec²
Horizontal force
superimposed Dead Load 84650 × 0.1 = 8465
weight of shaft 81500 × 0.1 = 8150
166150

moment about bottom of shaft
8465 × 25.7 = 217.000
8150 × 10.0 = 81500
298.500 1#

Moment of Inertia of section
Concrete 0.049 × 5.55⁴ = 46.5
Steel 15 × 0.033 × $\frac{5.05^2}{8}$ = 1.6
48.1

Fibre stress = $\frac{298.500 \times 2.78}{48.1}$ = 17300 #/□' or 120 #/□"

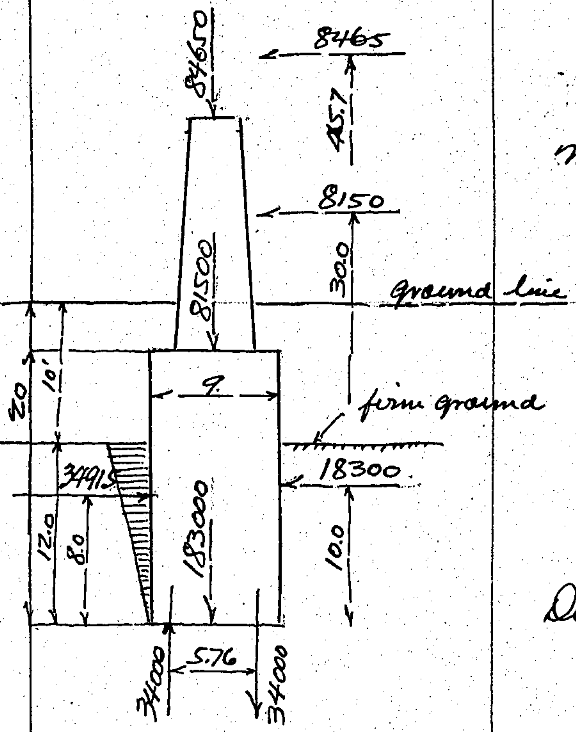
Equivalent Concrete area of section Concrete 5.55ø 24.19 × 144 = 3480
steel .15 × 4.8 = 72
3552 □"

Unit direct load = 166150 ÷ 3552 = 47 #/□"

Summary of Fibre Stress Bending 120 C 120 T
Direct 47 C 47 C
167 #/□" C 73 #/□" T

All right without reinforcement, however use reinf. as shown to insure elasticity of concrete in shaft.

Bearing Pressure at bottom of base during earthquake
Moment taken at bottom of base; firm ground assumed 12' above bottom of base
10' loose ground on top



Friction of well surface 9'ø = 28.3 × 200 × 12 = 68000 #
Arm for friction couple = 9 × .64 = 5.76'
moment about bottom of base
8465 × 45.7 = 387.000
8150 × 30.0 = 244500
18300 × 10.0 = 183000
34915
814500 1#

34915 × 8.0 = 279000
34000 × 5.76 = 196000
− 475000
339500 1# net

Direct Load
superimposed load 84650
shaft 81500
well 183000
349150 #
Less friction 68000
281150 #

Bottom area 10'ø = 78.5 □' Unit bearing = 281150 ÷ 78.5 = 3580 #/□'
or 1.60 tons/□'.

Design of Kasumi-Bashi for Okayama-Ken

Moment of inertia of bottom area = 0.049 × 10^4 = 490

fibre stress due to bending = $\frac{339500}{490}$ × 5 = 3460 #/□' or 1.54 tons/□'

Summary for fibre stress

Bending	1.54 C	1.54 T
Direct	1.60	1.60 C
	3.14 tons/□' C	0.06 ton/□' C.

Design of shell

Earth pressure during well sinking assumed

at 15' + 2' P = $\frac{1}{3}$ × 100 × 17 = say 570 #/□'

at bottom P = $\frac{1}{3}$ × 100 × 22 = " 750 #/□'

At 15' thickness 1.0' moment = $\frac{1}{16}$ × 570 × 8^2 = 2280 '#

At bottom thickness 1.5' moment = $\frac{1}{16}$ × 750 × 8.5^2 = 3380 '#

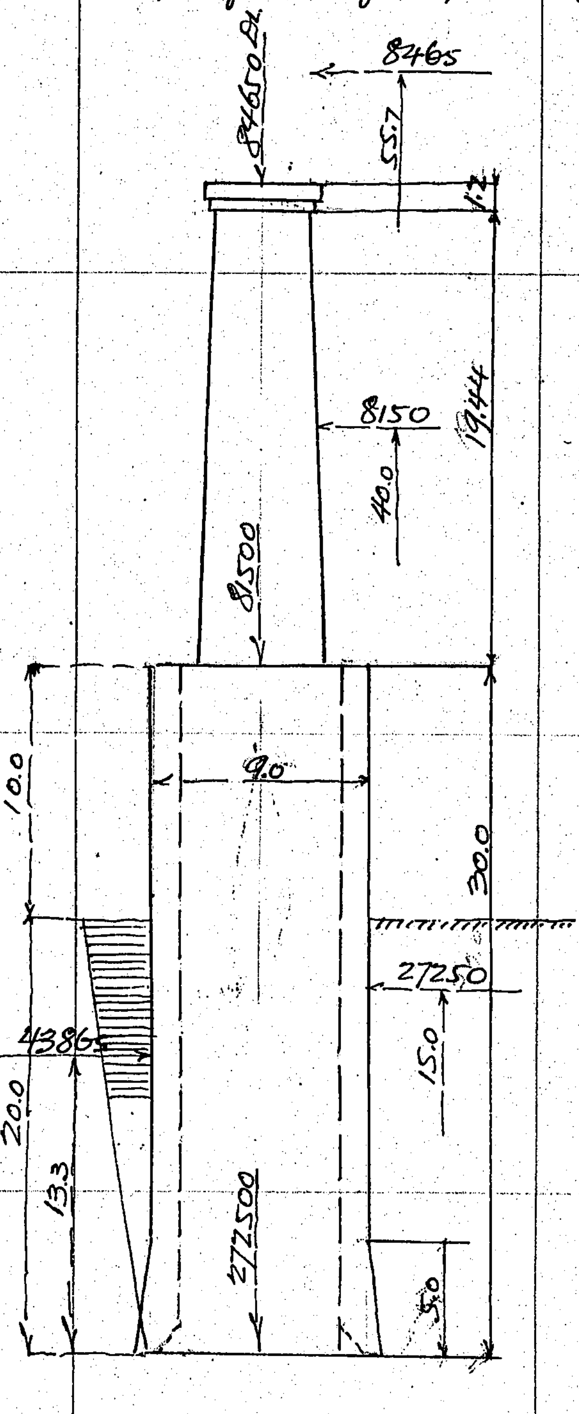
Reinforcement required at bottom = $\frac{3380 \times 12}{\frac{7}{8} \times 16 \times 17000}$ = 0.170" per ft

Reinforcement " at 15' = $\frac{2280 \times 12}{\frac{7}{8} \times 10 \times 17000}$ = 0.185" per ft.

Design of Pier for Girder spans (GP1 to 5).

make well 16' deep Unless otherwise details same as for GP6 to 10.

Design of Pier for Girder spans (GP11)



Details of shaft same as for GP6 see pp 32.

well

shell 9' dia 63.6

7' dia 38.5

25.1 × 25 = 627.

32.55 × 5 = 163

790

2 × 790 = 1580 or 733 cubic tsubo.

Top and bottom filling	38.5 × 8 × 2 =	615	2.85	立坪
Intermediate filling	38.5 × 22 × 2 =	1695	7.85	" "
shell			7.33	
			18.03	" "

weight of well 18.03 @ 30200 = 545 000

" " shaft 163 000

529 000

Superimposed Load 295300

824300

Less friction 9'φ 28.3 × 200 #/□' × 30 = 170 000

654300 #

Bearing area of bottom 2-10' dia 2 × 78.5 = 157.0 □'

Unit bearing = 654300 ÷ 157. = 4170 #/□' 1.86 tons/□'

Bearing Pressure at bottom of base during Earthquake moment taken at bottom of base considering side pressure and frictional resistance along face of well; firm ground assumed at 10' below top of well.

Friction of well surface 9'φ = 28.3 × 200 × 20 = 113000 #

Arm for friction couple = 9 × .64 = 5.76'

Design of Kasumi-Bashi for Okayama-Ken

Moment about bottom of base

8465	×	55.7	=	470.000	
8150	×	40.0	=	326.000	
27250	×	15.0	=	408.000	
43865					1204000
43865	×	13.3	=	583.000	
56500	×	5.76	=	325.000	
					− 908000
					296000 '#

Direct Load

superimposed load	84650	
Shaft	81500	
well	272500	
		438650
Less friction		113000
		325650 #

Bottom area 10'⌀ = 78.5 □' Unit bearing 325650 ÷ 78.5 = 4150 #/□' or 1.85 tons/□'

Fibre stress due to bending $\frac{296000 \times 5}{490}$ = 3020 #/□' or 1.35 tons/□'

Summary for fibre stress	Bending	1.35 C	1.35 T
	Direct load	1.85 C	1.85 C
		3.20 tons/□' C	0.50 tons/□' C.

Design of shell

Earth pressure during well sinking assumed

at 25' $P = \frac{1}{3} \times 100 \times 25 = 830$ #/□'

at 30' $P = \frac{1}{3} \times 100 \times 30 = 1000$ #/□'

Moment at 25' thickness 1.0 moment $= \frac{1}{16} \times 830 \times 8^2 = 3320$ '#

at 30 " 1.5' " $= \frac{1}{16} \times 100 \times 8.5^2 = 4510$ '#

Reinforcement required at bottom $= \frac{4510 \times 12}{\frac{7}{8} \times 16 \times 17000} = 0.230$ □"

" " " 25' $= \frac{3320 \times 12}{\frac{7}{8} \times 10 \times 17000} = 0.270$ □"

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY J.M. DATE 14.10.6 FILE NO
CHECKED BY DATE 14.10.12 PAGE NO 45

CALCULATIONS FOR

FOR TRUSS SPAN

No.	Descr.	Section in ins.	Length ft.	Length ins.	Wt. one ft. in lbs.	Wt. one piece in lbs.	Tot. wt. in lbs.	Remarks
		End Post L0U0 4. Req'd.						
1.	Cov. Pl.	1.9 × 3/8	21	2⅝	24.23	514.2	514	
2	[s	12. × 3½	22	1⅛	26.1	576.5	1,153	
2	Pls	35 × 7/16	3	3	52.06	169.2	338	
2	Ls	4. × 4. × 5/16	0.	10⅜	8.2	7.1	14	
4.	"	3 × 3. × 5/16	1.	0¼	6.1	6.1	24	
1.	Pl	10½ × 5/16	1.	0¾	11.16	11.2	11	
2.	Washers	3"φ × 5/16				0.62	1	
4.	Ls	4. × 3 × 5/16	1.	4.	7.2	9.6	38	
1.	Pl.	11. × 5/16	1.	4.	11.69	15.5	16	
2.	"	17½ × 3/8	1.	7.	22.31	35.2	70	
4.	Ls	3½ × 3½ × 3/8	2.	5½	8.50	20.9	84	
2	"	3. × 3. × 3/8	2.	5½	7.2	17.7	35	
4.	"	3. × 3 × 3/8	2.	5½	"	17.7	71	
1	Pl.	9½ × 3/8	2.	5½	11.48	28.2	28	
2.	"	11¾ × ¾	3	4.	29.96	99.8	200	
2	"	28¾ × 3/8	3.	4.	36.66	122.1	244	
18	Lac. Bars.	2¼ × 3/8	2.	1⅛	2.87	6.0	108	
1.	L.	3. × 3. × 5/16	0.	10¾	6.1	5.5	6	
							2,955 × 4 = 11,820	
		Top Chord U0U1 4. Req'd.						
1.	Cov. Pl	1.9 × 3/8	18	4 5¾	24.23	444.1	~~444~~	460.
2	[s	12 × 3½	17	6½ 7¾	26.1 ✓	457.8	~~916~~	921
2	Ls	3½ × 3½ × 3/8	1.	8 9¾	8.5	14.2	~~28~~	30.
1.	Pl.	15 × 5/16	1.	10 11	15.94	29.2	~~29~~	31.
2	"	17½ × 3/8	1.	7 ✓	22.31	35.2	70	
1.	"	16 × 3/8	1.	11½ ✓	20.40	40.0	40	
2	Ls	3½ × 3 × 3/8	2	2 ✓	7.9	17.1	34	
1.	Pl.	13½ × 5/16	2.	9¼ ✓	14.34	39.7	40	
1.	"	14 × 5/16	1.	11½ ✓	14.88	29.2	29	
1.	"	3½ × ½	1.	1.1 ✓	5.95	11.4	11	
2.	"	10¼ 10⅛ × 3/8	0.	10¾ 11	12.91 13.07	11.6	~~23~~	24.
2	"	24¼ 24½ × 3/8 7/16	2	1¾ 2	31.24 36.97	67.2	~~134~~	157
16	Lac. Bars.	2¼ × 3/8	2	1⅛ ✓ 1¼	2.87	6.0	~~96~~	101.
							1,894 × 4 = 7,576	
		Top Chord U1U2 4. Req'd.						
1.	Cov. Pl.	19 × 3/8	17	9¼ 10⅝	24.23	430.6	~~431~~	433
2	[s	12 × 3½	17	9¼	26.10	463.8	~~928~~	933.
2 1	Pl	17½ × 3/8	1.	7 ✓	22.31	35.2	~~70~~	35.
1.	"	16 × 7/16	2	2½ ✓	23.80	52.6	53	
2	Ls	3½ × 3 × 3/8.	2	2½ ✓	7.9	17.5	35	
1.	Pl	13½ × 5/16	2	8 ✓	14.34	38.3	38	
1	Fill	2¾ × 1/16	1	4⅛ ✓	0.584	0.8	1	
1.	"	13⅞ × 1/16	1.	4. ✓	2.90	3.9	4	
2	Pl.	3½ × ½	~~2~~	0 ✓ 1 11	5.95	11.9	~~24~~	23.
1,	Pl.	19 × 3/8	1	8½	24.23			41.

No.	Descr.	Section in ins.	Length ft.	Length ins.	Wt. one ft. in lbs.	Wt. one piece in lbs.	Tot. wt. in lbs	Remarks
2	P/s	10 1/2 ~~10 1/4~~ × 1/2	1.	~~1 7/8~~ 2	~~17.43~~ 17.85	20.2	~~40~~	43
2	"	31 1/2 × 3/8	3	5 1/2	~~40.17~~ 50.58	139.0	~~278~~	334
16	Lac. Bars.	2 1/4 × 3/8	2	1 7/8	2.87 ✓	6.0	96	96
							1,998 × 4 = 7,992	

34 2/16 1/4 14 3 1/2

Top Chord U2 U3 4 Req'd.

No.	Descr.	Section in ins.	Length ft.	Length ins.	Wt. one ft. in lbs.	Wt. one piece in lbs.	Tot. wt. in lbs	Remarks
1.	Cov. Pl	19 × 7/16	17	~~7 7/8~~ 8 1/4	28.26	499.1	499	500
2	∟s	12 × 3 1/2	17	7 7/8	26.1 ✓	460.9	922	925
2	Pl	17 1/2 × 3/8	1.	7. ✓	22.31	35.2	70	
1.	"	16 × 7/16	2	4 1/2 ✓	23.80	56.6	57	
2.	∟s	3 1/2 × 3 × 3/8	2	4 1/2 ✓	7.9	18.8	38	
1.	Pl.	13 1/2 × 5/16	2	9 1/2 ✓	14.34	40.0	40	
1.	"	14 × 5/16	1.	1.1 ✓	14.88	28.6	29	
1.	"	3 1/2 × 1/2	1.	1.1 ✓	5.95	11.4	11	
2	"	10 1/4 ~~10 1/8~~ × 3/8	0.	10 3/4 ✓	12.91 13.07	11.6	~~23~~	23
2	"	24 × 3/8	2	2 ✓	~~30.60~~ ✓ 35.70	66.4	~~133~~	155
16	Lac. Bars.	2 1/4 × 3/8	2	1 7/8 ✓	2.87	6.0	~~96~~	97
							1,918 × 4 = 7,672	

7/16 1/4 14

Top Chord U3 U4 4. Req'd.

No.	Descr.	Section in ins.	Length ft.	Length ins.	Wt. one ft. in lbs.	Wt. one piece in lbs.	Tot. wt. in lbs	Remarks
1.	Cov. Pl.	19. × 7/16	17	~~7 1/8~~ 7 1/2	28.26	496.8	~~497~~	500
2.	∟s.	12 × 3 1/2	17	7 1/8	26.1 ✓	458.8	~~918~~	920
2	P/s.	17 1/2 × 3/8	1.	7 ✓	22.31	35.2	70	
1.	"	16 × 7/16	2	5 ✓	23.80	57.6	58	
2.	∟s	3 1/2 × 3 × 3/8.	2.	7 1/2 ✓	7.9	20.8	42	
1.	Pl.	13 1/2 × 5/16	2.	1.0 ✓	14.34	40.6	41	
2	"	3 1/2 × 5/8	3.	2 1/2	7.44	23.9	48	
2	"	10 1/4 ~~10 1/8~~ × 3/8	1.	4 1/4	~~12.91~~ 13.07	17.4	~~35~~	35
2	Fill	7 3/4 × 1/8	0.	~~1.0~~ 10	3.294	2.7	5	
2	P/s	28 1/2 ~~28 1/4~~ × 3/8	2	~~9 3/4~~ 10	36.02 42.04	101.2	~~202~~	204
16	Lac. Bars	2 1/4 × 3/8	2	1 1/8 ✓	2.87	6.0	~~96~~	97
2	"	"	1	10 1/2	2.87			
							2012 × 4 = 8,048	

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11.

Top Chord U4 U5 2. Req'd.

No.	Descr.	Section in ins.	Length ft.	Length ins.	Wt. one ft. in lbs.	Wt. one piece in lbs.	Tot. wt. in lbs	Remarks
2	Cov. P/s	19 × 7/16	17	~~6 5/8~~ 6 3/4	28.26	496.0	~~992~~	993
4.	∟s.	12 × 3 1/2	17	6 5/8	32.88 ✓	577.0	2,308	2,309
4.	P/s	17 1/2 × 3/8	1	7. ✓	22.31	35.2	141	
1.	"	16 × 7/16	2	4 1/2 ✓	23.80	56.6	57	
2	∟s	3 1/2 × 3 × 3/8	2	4 1/2 ✓	7.9	18.8	38	
1.	Pl	13 1/2 × 5/16	2	9 1/2 ✓	14.34	40.0	40	
1.	"	14 ~~13 1/2~~ × 5/16	3.	2 ✓	14.88	45.5	~~46~~	47
1.	"	3 1/2 × 5/8	3.	2 ✓	7.44	23.6	24	
2	"	10 1/8 × 3/8	0.	10 3/4 ✓	12.91	11.6	23	
2	"	24 × 3/8	2	2 ✓	~~30.60~~ 35.7	66.4	~~133~~	155
32	Lac. Bars	2 1/4 × 3/8	2.	1 1/8 ✓	2.87	6.0	~~192~~	193
							3994 × 2 = 7,988	

7/16 1/4 14

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY______ DATE______ **FILE NO**______
CHECKED BY______ DATE______ **PAGE NO** 47

CALCULATIONS FOR

No.	Descr.	Section in ins	Length		Wt. one ft. in lbs.	Wt. one piece in lbs.	Tot wt. in lbs.	Remarks
			ft.	ins.				
		Bottom Chord L0L1 4. Req'd.						
2.	[s	12 × 3½	15	11¾ ✓	26.1	417.1	834	
1.	Pl	25½ × 5/16	2	3½ ✓	27.09	62.0	62	
2	″	10¾ × 5/16	1.	0½ ✓	11.42	11.9	24	
4	″	1.0 × 5/16	0	10¾ ✓	10.63	9.6	38	
							958 × 4 = 3,832	
		Bottom Chord L1L3 4. Req'd.				215.4		
2.	[s	36 12 × 3½	31	0 ½ ✓	32.88 ✓	1019.3	2,039	
2	P/s	~~35½~~ × 7/16	4.	1	~~37.72~~ 52.8 53.15	215.4	431	~~443~~
8.	Ls	3½ × 3½ × 5/16	0	8½ ✓	7.2	5.1	41	
2	P/s	8½ × 5/16	0.	9½ ✓	9.03	7.10	14.	14.
1.	″	27½ × 5/16	2.	10½ ✓	29.22	84.2	84	
3	″	10¾ × 5/16	1.	0½ ✓	11.42	11.9	36	
10	″	1.0 × 5/16	0	10¾ ✓	10.63	9.6	96	
1.	″	1.9 × 3/8	2	1¼ ✓	24.23	50.9	51	51
1.	″	1.9 × 3/8	2	8¾ ✓	″	66.1	66.	
1.	″	25½ × 5/16	2	6. ✓	27.09	67.7	68	
2.	″	12 × 9/16	2	6½ ✓	22.95	58.3	117	
2	″	10½ 3/8	2	4 ✓	13.39	31.2	62	
							3,105 × 4 = 12,420. 3,105	12,420
		Bottom Chord L3L4 4. Req'd.						
2	[s	12 × 3½	25	6 ✓	32.88	838.4	1,677	
2.	P/s	3½ 1.0 × 3/8	21	5¾ ✓	12.75	273.9	548	
2	″	3½ ~~31¾~~ × 3/8	~~3~~	~~0½~~ 1½	~~40.49~~ 41.44	223.1	~~446~~	257.
8	Ls	3½ × 3½ × 5/16	0.	8½ ✓	7.2	5.1	41	
2	Pl	8½ × 5/16	0.	8¾	9.03	6.6 6.6	13	13.
1.	″	23½ × 5/16	2.	4¼ ✓	24.97	58.7	59	
3.	″	10¾ × 5/16	1.	0½ ✓	11.42	11.9	36	
6.	″	1.0 × 5/16	0.	10¾ ✓	10.63	9.6	58	
1.	″	19 × 3/8	2.	8¾ ✓	24.23	66.1	66	
1.	″	1.9 × 3/8	2.	1¼ ✓	″	50.9	51	
1	″	23½ × 5/16	2	4½ ✓	24.97	59.4	59	
2	″	12 × 9/16	2	6½ ✓	22.95	58.3	117	
2	″	10½ × 3/8	2	4 ✓	13.39	31.2	62	
2	″	10 × 3/8	1	3½ ✓	12.75	16.4	33	
							3,266 × 4 = 13,064 3266	13,064
		Bottom Chord L4 L4 2. Req'd.						
2.	[s	12 × 3½	29	0½ ✓	32.88 ✓	954.8	1910	
2.	P/s	1.0 × 3/8	29	0.½ ✓	12.75	370.3	741	
2	P/s	10¾ × 5/16	1.	0½ ✓	11.42	11.9	24	
8.	″	1.0 × 5/16	0.	10¾ ✓	10.63	96	77	
2	″	29½ × 3/8	2.	1.0 ✓	37.62	106.5	213	
4.	Ls	3½ × 3½ × 5/16	0.	8½ ✓	7.2	5.1	20	
1.	Pl.	8½ × 5/16	0.	8¾ ✓	9.03	6.6	7	

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

CALCULATIONS FOR

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No.	Descr.	Section in ins.	Length ft.	Length ins.	Wt one ft. in lbs.	Wt. onepiece in lbs.	Tot. Wt. in lbs.	Remarks
1.	Pl.	23½ × 5/16	2	4¼ ✓	24.97	58.7	59	
							3,051 × 2 = 6,102	
		Diagonal U_0L_1 4. Req'd.						
2	[s	12 × 3½	24	~~3½~~ 7¾	~~26.1~~ 32.88	634.0	~~1,268~~	1,620
4	P/s	10 × 5/16	0.	11½ ✓	10.63	10.2	41	
112	Lac. Bars.	2 × 5/16	0.	10¾ ✓	2.13	1.9	213	
							1,522 × 4 = 6,088	
		Diagonal L_1U_2 4 Req'd.						
2.	[s	12 × 3½	~~29~~	~~0½~~ 30'5	~~26.1~~ 32.88	757.9	~~4516~~	2,000
4.	P/s	10 × 5/16	0.	11½	10.63	10.2	41	
14.6 152	Lac. Bars	2 × 5/16	0.	10¾	2.13	1.91	277	290
							1,834 × 4 = 7,336	
		12 Diagonal U_2L_3 4. Req'd.						
2.	[s	10 × 3½	29	2 30'8	~~23.55~~ 26.1	687.0	~~1,374~~	1,604
4.	P/s	10 × 5/16	0	11½ ✓	10.63	10.2	41	
15.0 158	Lac. Bars	2 × 5/16	0.	10¾	2.13	1.9	~~285~~	302
							1,700 × 4 = 6,800	
		12 Diagonal L_3U_4 4. Req'd.		33'11¾				
2.	[s	10 × 3½	31	1.1	~~23.55~~ 26.1	751.7	~~1,503~~	1774
4.	P/s	10 × 5/16	0.	11½ ✓	10.63	10.2	41	
16.8 176	Lac. Bars	2 × 5/16	0	10¾	2.13	1.9	~~319~~	336
							1,863 × 4 = 7,452	
		12 Diagonal U_4L_5 4. Req'd.		34'0¼				
2	[s	10 × 3½	~~31~~	~~11¾~~	~~23.55~~ 26.1	753.1	1506	1776
4	P/s	10 × 5/16	0.	11½ ✓	10.63	10.2	41	
168 178	Lac. Bars	2 × 5/16	0.	10¾ ✓	2.13	1.9	~~319~~	340
							1,866 × 4 = 7,464	
		Vertical U_1L_1 4. Req'd.						
4.	Ls.	5 × 3 × 5/16.	22	2	8.2	181.8	727	
1.	Pl	10 × 5/16	0.	10.	10.63	8.8	9	
1.	"	10 × 5/16	0.	11½	"	10.2	10	
33	Lac. Bars	2 × 5/16	0	11⅜	2.13	2.0	66	
							817 × 4 = 3,248	

No.	Descr.	Section in ins	Length ft.	Length ins.	Wt. one ft. in lbs.	Wt. one piece in lbs	Tot. Wt. in lbs.	Remarks
		Vertical	U2 L2		4 Req'd.			
			26∠2 2/8					
4.	Ls.	5 × 3 × 5/16	24	11½	8.2	204.7	~~819~~	861.
1.	Pl.	10 × 5/16	0.	10.	10.63	8.8	9	
1.	"	14¼ × 5/16	1.	8¼	15.15	25.6	26	
1.	"	14½ × 5/16	1.	9.	15.41	27.0	27	
1.	"	10 × 5/16	0.	11½	10.63	10.2	10	
33 → 35	Lac. Bars.	2 × 5/16	0.	11 3/8	2.13	2.0	~~64~~	71
							955 × 4 = 3,820	
		Vertical	U3 L3		4 Req'd.			
~~4.~~	~~Ls.~~	~~5 × 3 × 5/16~~	~~26~~	~~11½~~	~~8.2~~	~~221.1~~	~~884~~	
~~1.~~	~~Pl.~~	~~10 × 5/16~~	~~0.~~	~~10~~	~~10.63~~	~~8.8~~	~~9~~	
~~1.~~	~~"~~	~~10 × 5/16~~	~~0.~~	~~11½~~	~~"~~	~~10.2~~	~~10~~	
~~41~~	~~Lac. Bars.~~	~~2 × 5/16~~	~~0.~~	~~11 3/8~~	~~2.13~~	~~2.0~~	~~82~~	
							985 × 4 = 3,940	
		Vertical	U4 L4		4 Req'd.			
			30∠1					
4.	Ls.	5 × 3 × 5/16	28	2	8.2	231.0	~~924~~	1001
1.	Pl.	10 × 5/16	0.	10 ✓	10.63	8.8	9	
1.	"	15¾ × 5/16	1.	8¼ ✓	16.74	28.3	28	
1.	"	14½ × 5/16	1.	9. ✓	15.41	27.0	27	
1.	"	10 × 5/16	0.	11½ ✓	10.63	10.2	10	
38 → 41	Lac. Bars	2 × 5/16	0.	11 3/8	2.13	2.0	~~76~~	83
							1,074 × 4 = 4,296	
		Vertical	U5 L5		2 Req'd.			
~~4.~~	~~Ls.~~	~~5 × 3 × 5/16~~	~~28~~	~~6¾~~	~~8.2~~	~~234.2~~	~~937~~	
~~1.~~	~~Pl.~~	~~10 × 5/16~~	~~0.~~	~~10.~~	~~10.63~~	~~8.8~~	~~9~~	
~~1.~~	~~"~~	~~10 × 5/16~~	~~0.~~	~~11½~~	~~"~~	~~10.2~~	~~10~~	
~~43~~	~~Lac. Bars~~	~~2 × 5/16~~	~~0.~~	~~11 3/8~~	~~2.13~~	~~2.0~~	~~86~~	
							1,042 × 2 = 2,084	
						TRUSSES -- ~~139,108~~ 139,042 # ÷ 2240 =	~~62.103~~ 62.072 Tons	
		Portal Bracing			2 Req'd.			
2.	Ls.	5 × 4 × 3/8.	22	7	11.0	248.4	497	
2.	"	5 × 4 × 3/8	25	3.	"	277.8	556	
4.	Ls.	3 × 3 × 5/16	2	11½	6.1	18.1	72	
4.	"	3 × 3 × 5/16	1.	5¾	"	9.0	36	
4.	Fills	3 × 3/8	0.	9.	3.83	2.9	11	
4.	Ls.	3 × 2½ × 5/16	2	8.	5.6	15.0	60	
4.	"	3 × 2½ × 5/16	2	5.	"	13.6	54	
4.	"	3 × 2½ × 5/16	2.	4¼	"	13.2	53	
4.	"	3 × 2½ × 5/16	2	4.	"	13.0	52	
4.	"	3 × 2½ × 5/16	2	4.	"	13.0	52	
4.	"	3 × 2½ × 5/16	2	1.	"	11.6	46	

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______ DATE______ FILE NO______

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CALCULATIONS FOR

No.	Descr.	Section in ins	Length ft.	Length ins.	Wt. one ft. in lbs.	Wt. one piece in lbs	Tot. Wt. in lbs.	Remarks
4.	Ls.	3 × 2½ × 5/16	1.	11.	5.6	10.8	43	
4	〃	3. × 2½ × 5/16	1.	10½	〃	10.5	42	
4.	〃	3 × 2½ × 5/16	1.	9¾	〃	10.1	40	
2	〃	3. × 2½ × 5/16	1.	9½	〃	10.0	20	
2.	P/s	10¼ × 5/16	1.	1¼	10.89	12.0	24	
4.	〃	8½ × 5/16	0	11½	9.03	8.7	35	
10	〃	9. × 5/16	0	11½	9.56	9.2	92	
1.	〃	5½ × 5/16	0.	8.	5.84	3.9	4	
2	〃	34 × 5/16	2	11½	36.13	106.9	214	
1.	〃	9. × 5/16	1.	3	9.56	12.0	12	
							2,015 × 2 = 4,030	
		Sway Bracing SB 1 2. Req'd.						
2.	Ls.	5. × 4 × 3/8.	22	3⅝	11.0	245.3	491	
2.	〃	5. × 4 × 3/8.	23	0½	〃	253.4	507	
2.	Ls	2½ × 2 × 0.3	23	0	4.28	98.4	197	
2	〃	2½ × 2 × 0.3	4.	0.	〃	17.1	34	
10.	〃	2½ × 2 × 0.3	4.	3¾	〃	18.4	184	
10	〃	2½ × 2 × 0.3	2	10¼	〃	12.2	122	
22	〃	2½ × 2 × 0.3	3	8.	〃	15.7	345	
2.	〃	3½ × 3 × 3/8	0	10⅝	7.9	7.0	14	
2	〃	3½ × 3 × 3/8	0.	10½	〃	7.0	14	
10	〃	2½ × 2 × 0.3	1.	0.	4.28	4.3	43	
2	P/s	10¾ × 5/16	1.	1.	11.42	12.3	25	
5.	〃	8. × 5/16	1.	2¾	8.50	10.5	53	
5	〃	7 × 5/16	1	1½	7.44	8.4	42	
11	〃	6¾ × 5/16	1.	11¼	7.17	13.9	153	
11	〃	5. × 5/16	0.	8.	5.31	3.6	40	
6.	Washers.	3"φ × 5/16.				0.62	4	
22	〃	2½"φ × 5/16				0.43	9	
							2,277 × 2 = 4,554	
		Sway Bracing SB2 2. Req'd.						
2	Ls	5 × 4 × 3/8	22	3⅝	11.0	245.3	491	
2	〃	5 × 4 × 3/8	23	0½	〃	253.4	507	
2	Ls.	2½ × 2 × 0.3	23	0	4.28	98.4	197	
2	〃	2½ × 2 × 0.3	4	5⅜	〃	19.0	38	
10	〃	2½ × 2 × 0.3	4	10⅝	〃	20.9	209	
10	〃	2½ × 2 × 0.3	3.	7.	〃	15.3	153	
22	〃	2½ × 2 × 0.3	5.	7¾	〃	24.2	532	
4	〃	3½ × 3 × 3/8	0.	10¼	7.9	6.7	28	
10	〃	2½ × 2 × 0.3	1.	0.	4.28	4.3	43	
2	P/s	11⅞ × 5/16	1.	1.	12.61	13.6	27	
5.	〃	8. × 5/16	1.	1.	8.50	9.2	46	
5.	〃	7¼ × 5/16	1.	0.	7.70	7.7	39	
11	〃	6¾ × 5/16	1.	11¼	7.17	13.9	153	
11	〃	5. × 5/16	0.	8.	5.31	3.6	40	
6.	Washers	3"φ × 5/16				0.62	4	
22	〃	2½"φ × 5/16				0.43	9	
							2,516 × 2 = 5,032	

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

MADE BY ______ DATE ______ **FILE NO** ______

CHECKED BY ______ DATE ______ **PAGE NO** 51

CALCULATIONS FOR

No.	Descr.	Section in ins.	Length ft.	Length ins.	Wt. one ft. in lbs.	Wt. onepiece in lbs	Tot. Wt. in lbs.	Remarks
Top Lateral strut ST1 5. Reqd.								
4.	∠s	4×3×5/16	22	3¾	7.2	160.6	642	
2.	Pls	11½×5/16	1.	0½	12.22	12.7	25	
1.	〃	10×5/16	0.	11½	10.63	10.2	10	
28	Lac. Bars	2×5/16	1.	0⅞	2.13	2.3	64	
							741	× 5 = 3,705
					PORTAL, SWAY & STRUT …… 17,321# ÷ 2,240 = 7.733 TONS			
Top Lateral Bracing 2. Reqd.								
2.	∠s	5×3×0.3	27	3¾	7.85	214.4	429	TB1R
1.	Pl	18×5/16	1.	9¾	19.13	34.6	35	
2	∠s	3×3×5/16	13	2¾	6.1	80.7	161	TB2
2	〃	3×3×5/16	13	4¾	〃	81.7	163	TB3
2	〃	5×3×0.3	27	4¾	7.85	215.1	430	TB4R
4	〃	3×3×5/16	13	4	6.1	81.3	325	TB5
1	Pl	18×5/16	1.	9¾	19.13	34.6	35	
2	∠s	5×3×0.3	27	4	7.85	214.5	429	TB6R
1.	Pl.	18×5/16	1.	9¾	19.13	34.6	35	
4	∠s	3×3×5/16	13	3¾	6.1	81.2	325	TB7
2.	〃	5×3×0.3	27	3½	7.85	214.2	428	TB8R
1	Pl.	18×5/16	1	9¾	19.13	34.6	35	
4	∠s	3×3×5/16	13	3½	6.1	81.1	324	TB9
2	〃	5×3×0.3	27	3¼	7.85	214.1	428	TB10R
1	Pl.	18×5/16	1	9¾	19.13	34.6	35	
4	∠s	3×3×5/16	13	3¼	6.1	80.9	324	TB11
							3,941	× 2 = 7,882
						TOP BRACING 7,882 ÷ 2,240 = 3.519 TONS		
Floor Beam FB1 2. Reqd.								
1.	Web. Pl.	27.×5/16.	22.	10½	28.69	656.4	656	
4.	∠s.	5.×3.×3/8	22	10½	9.8	224.2	897	
4.	∠s.	3×3×3/8	2.	2¾	7.2	16.1	64	
4.	Fills	3.×3/8.	1.	9¼	3.83	6.8	27	
16.	∠s.	3×3×5/16.	2.	2¾	6.1	13.6	218	
5.	〃	4.×3.×3/8.	0.	6.	8.5	4.3	22	
2.	Pl.	7½×3/8	0.	10¼	9.56	8.1	16	
1.	〃	7½×3/8	0.	9.	〃	7.2	7	
							1,907	× 2 = 3,814
Floor Beam FB2 & FB3 9. Reqd.								
1.	Web. Pl.	27.×5/16	22	11½	28.69	658.7	659	
4.	∠s.	5.×3×3/8.	22	11½	9.8	225.0	900	
2.	Cov. Pls.	10½×3/8	17	3¾	13.39	231.8	464	
4	∠s.	3½×3½×3/8	2.	2¾	8.5	19.0	76	
4.	Fills	3½×3/8	1.	9¼	4.46	7.9	32	
16.	∠s.	3×3.×5/16	2.	2¾	6.1	13.6	218	
10	〃	4×3×3/8.	0.	6	8.5	4.3	43	

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

No.	Descr.	Section in ins.	Length ft.	Length ins.	Wt. one ft. in lbs.	Wt. one piece in lbs	Tot Wt. in lbs.	Remarks
4.	Pls.	7½ × 3/8.	0.	10¼.	9.56	8.1	32	
2.	"	7½ × 3/8	0.	9.	"	7.2	14	
							2,438 × 9 = 21,942	
						F1. BM.	25,756 ÷ 2,240 = 11.498 TONS	
		Stringers		1. Reqd.				
20	Is	12 × 5	17.	5.	31.99	557.3	11,146	S1
20.	"	12 × 5	17.	4¼.	"	555.0	11,100	S2
10.	"	12 × 5.	17.	4¼	"	555.0	5,550	S3
200	Ls	6 × 3½ × 3/8	0.	6½	11.7	16.3	1,260	
							29,056 × 1 = 29,056	
						STRINGER	29,056 ÷ 2,240 = 12.971 TONS	
		Bracings Hanger H.1		10. Reqd.				
1.	Ls.	6. × 3½ × 3/8.	0.	7	11.7	6.8	7	
1.	"	2½ × 2 × 0.3	1.	0¾	4.28	4.5	5	
1.	Pl.	6½ × 5/16	0.	7¾	6.91	4.5	5	
							17 × 10 = 170	
		Bottom Lateral Bracings				2. Reqd.		
10.	Ls.	4 × 3 × 0.3	27	6.	6.84	188.1	1,881	LB1.3.5.
20	"	4 × 3 × 0.3	13	4¾	"	91.7	1,834	LB2.4.6.
1.	Pl.	23¾ × 5/16	2.	6.	25.24	63.1	63	
1.	"	20¾ × 5/16	2.	1¾	22.05	47.4	47	
3.	"	17¾ × 5/16	1.	9¾	18.86	34.1	102	
							3,927 × 2 = 7,854	
						BOTT. BRACING	8,024 ÷ 2240 = 3.582 TONS	
		Pin & Roller			1. Reqd.			
4.	Pins	4½"φ	1.	7¾	54.07	89.2	357	PN1
8.	Nuts					4.6	37	
10.	Rollers	4"φ	1.	10.	42.73	78.2	782	RN1
20	Tapped Bolts	¾"φ	0.	1 13/16		0.25	5	
4.	side Pls.	3. × ½	1.	8.	5.10	8.5	34	
							1,215 × 1 = 1,215	
		Cast Steel Shoe & Bed Plate				1 Reqd.		
2.	Cast Steel					537.0	1,074	RS1
4.	" "					6.0	24	CG1
2.	" "					317.0	634	BP1
2.	" "					775.0	1550	FS1
							3,282 × 1 = 3,282	
		Anchor Bolts			1 Reqd.			
16.	Anch. Bolts	1¼"φ	2.	0.	4.172	8.3	133	

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CALCULATIONS FOR

No	Descr.	Section in ins.	Length ft.	Length ins.	Wt. one ft in lbs.	Wt one piece in lbs.	Tot. wt. in lbs.	Remarks
16	nuts					0.9	14	

147 x 1= 147

PIN, ROLLER, SHOE & ANCH BOLTS 4,644# ÷ 2,240 = 2.073 TONS

Tot. wt. 231,725#

231,725# ÷ 2,240 = 103.448 TONS.

SUMMARY OF WEIGHT FOR ONE SPAN (TRUSS SPAN)

		TONS.
TRUSSES	139,042#	62.072
PORTALS, SWAYS & STRUTS	17,321	7.733
TOP BRACINGS	7,882	3.519
FLOOR BEAMS	25,756	11.498
STRINGERS	29,056	12.971
BOTT. BRACINGS	8,024	3.582
PINS, ROLLERS, SHOES & ANCH. BOLTS.	4,644	2.073
	231,725	103.448

SUMMARY OF WEIGHT, RIVET HEAD FOR ONE SPAN

			TONS
SHOP RIVET HEADS	3/4" dia.	5,500#	2.455
FIELD " "	"	2,537	1.133
SHOP " "	5/8" dia	69	0.031
FIELD " "	"	117	0.052
		8,223	3.671

GRAND SUMMARY FOR ONE SPAN

	TONS
231,725#	103.448
8,223	3.671
239,948	107.119

TOTAL GRAND SUMMARY FOR 7 SPANS

107.119 TONS x 7 = 749.833 TONS

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY J. M. DATE 14.10.6 FILE NO
CHECKED BY DATE 14.10.13 PAGE NO 54

CALCULATIONS FOR

FOR GIRDER SPAN

NO.	Descr.	Section in ins.	Length ft.	Length ins.	Wt. one ft. in lbs.	Wt. onepiece in lbs.	Tot. Wt. in lbs.	Remarks
		Main Girders G.1$^{R}_{L}$ & A$^{R}_{L}$. 4. Req'd.						
4.	Ls.	6×6×½	13	10½	19.6	272.0	1,088	
1.	Web. Pl.	48 × ⅜	13.	10½	61.20	849.5	850	
2	Ls	5.×3½×½	3	11½	13.60	53.9	108	
2	"	5.×3½×⅜	3	11½	10.40	41.2	82	
6.	"	5×3½×5/16	3	11½	8.7	34.5	207	
3.	Fills	3½×½	3.	0¼	5.95	18.0	54	
1.	L.	4×3×0.3	0.	7½	6.84	4.3	4	
4.	"	6×6×½	2.	10	19.60	55.5	222	
4.	Pls	6.×½	2	10	10.20	28.9	116	
2.	"	12.×⅜	2.	0.	15.30	30.6	61	
1.	"	15.×¾	1.	9.	38.30	67.0	67	
							2,859	×4 = 11,436
		Main Girders. G2$^{R}_{L}$ 2 Req'd.						
4.	Ls	6×6×½	32	3.	19.60	632.1	2,528	
1.	Pl.	48.×⅜.	32	3.	61.20	1,973.7	1,974	
6.	Ls	5.×3½×⅜.	3.	11½	10.4	41.2	247	
12	"	5×3½×5/16	3	11½	8.7	34.5	414	
3.	Fill.	3½×½	3.	0¼	5.95	18.0	54	
2.	Cov. Pls	13.×½	35	1.	22.10	775.3	1,551	
6.	Ls.	4×3.×0.3	0.	7½	6.84	4.3	26	
							6,794	×2 = 13,588
					MAIN GIRDER		25,024 ÷ 2240 =	11.171 TONS
		Stringer 1. Req'd.						
4.	Is	12×5.	15.	7⅜	31.99	499.4	1,998	S1
4	"	12×5.	14	4	"	458.4	1,834	S2
12	Pls	8½×5/16	0.	11½	9.03	8.7	104	
							3,936	×1 = 3,936
					STRINGER		3936 ÷ 2240 =	1.757 TONS
		Floor Beam FB1 2 Req'd.						
1.	Web. Pl.	22×5/16	17.	11.	23.38	419.0	419	
2.	Ls.	3×3×5/16	17.	11	6.1	109.3	219	
2.	"	3×3×5/16.	16	11¾	"	103.6	207	
2	Ls.	5.×3½×⅜	1.	9⅞	10.4	18.9	38	
2	Fills	4¾×5/16	1.	4¼	5.05	6.8	14	
14	Ls.	3×2½×5/16	1	9⅞	5.6	10.2	143	
2	Pls	8½×⅜	1.	0½	10.84	11.3	23	
							1,063	×2 = 2,126

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY______ DATE______ **FILE NO**______

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CALCULATIONS FOR

No.	Descr.	Section in ins.	Length ft.	Length ins.	Wt. one ft. in lbs.	Wt one piece in lbs.	Tot. Wt. in lbs	Remarks
		Floor Beam FB.2 3. Req'd.						
1.	Web Pl.	22 × 5/16	17.	11.	23.38	419.0	419	
2.	∠s	3 × 3 × 3/8	17.	11.	7.2	129.0	258	
2	"	3 × 3 × 3/8	16.	11¾	7.2	122.3	245	
2.	∠s	5 × 3½ × 3/8	1.	9¾	10.4	18.8	38	
2.	Fills	4¾ × 3/8	1.	4¼	6.06	8.2	16	
14.	∠s.	3 × 2½ × 5/16	1.	9¾	5.6	10.1	141	
2.	Pls.	8½ × 3/8	1.	0½	10.84	11.3	23	
							1,140	× 3 = 3,420
						Fl. Bm.	5,546 ÷ 2240 =	2.476 TONS
		Bottom Lateral Bracing 4. Req'd.						
2.	∠s.	4 × 3 × 0.3	21.	5½	6.84	146.8	294	BC1
1.	Pl.	15 × 5/16	1.	5.	15.94	22.6	23	
4.	∠s.	4 × 3 × 0.3	10	4¾	6.84	71.1	284	BC2
							601	× 4 = 2,404
		Lateral Gusset Plate 1 Req'd.						
4.	Pls.	11¾ × 5/16	1.	2.	12.48	14.6	58	GP1
6.	"	11¾ × 5/16	1.	9¼	"	22.1	133	GP2
							191	× 1 = 191
						BRACING	2,595 ÷ 2,240 =	1.158 TONS
		Cast Steel Bed Plates 1 Req'd.						
2.	Cast steel					121.0	242	BP1
2.	" "					117.0	234	BP2
							476	× 1 = 476
		Anchor Bolts 1 Req'd.						
12.	Anch. Bolts	1¼"φ	2.	0.	4.172	8.3	100	
12.	Nut.					0.9	11	
							111	× 1 = 111
						BED PL. & ANCH.	587 ÷ 2,240 =	0.262 TONS

Total wt. 37,688
37,688 ÷ 2,240 = 16.825 TONS

SUMMARY OF WEIGHT FOR ONE SPAN (GIRDER SPAN)

MAIN GIRDERS	25,024 #	11.171 TONS
STRINGERS	3,936	1.757
FLOOR BEAMS	5,546	2.476
BOTT. BRACINGS	2,595	1.159
BED PL. & ANCH BOLTS.	587	0.262
	37,688	16.825

SUMMARY OF RIVET HEAD WEIGHT FOR ONE SPAN

2,840	Shop Rivs	7/8"φ	604 #	0.270 TONS
1,560	Field "	7/8"φ	332	0.148
1,960	Shop Rivs	3/4"φ	279	0.125
760	Field Rivs	3/4"φ	108	0.048
			1,323	0.591

GRAND SUMMARY FOR ONE SPAN

37,688 #	16.825 TONS
1,323	0.591
39,011	17.416

Edge Steel for Expansion Joint ES1 22 Req'd. (13 SPANS)

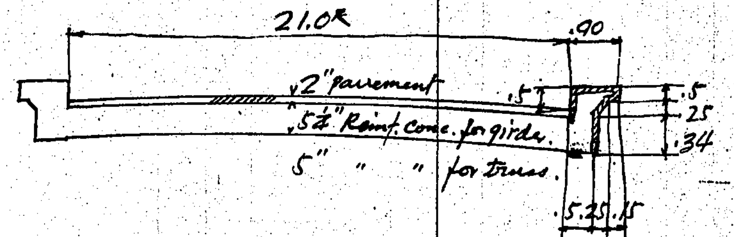
8	Bolts	3/4"φ ×	0'-2½"		0.69	6
1	L	3 × 3 × 5/16 ×	21'-0"	6.1	128.1	128
2	P/s	3 × ½ ×	1'-2½"	5.1	6.2	12

146 × 22 = 3,212
3,212 ÷ 2240 = 1.434 TONS

TOTAL GRAND SUMMARY FOR 13 SPANS

17.416 TONS × 13 = 226.408 TONS
1.434
227.842

Materials for super structure.
Concrete (1:2:4) for slab and copings.



Sectional area 5¼" slab and copings.
21.0 × .44 = $9.25^{\text{平尺}}$ for slab.
$.5 \times .9 + \frac{.5+.75}{2} \times .25 + .5 \times .34 \times 2$ = 1.56 " for copings.
$10.81^{\text{平尺}}$
mortal finish
10.81 − 0.11 = $\underline{10.7}^{\text{平尺}}$

Volume of conc. for slab C 13 Required.
10.70 × 60.35 = $645.0^{\text{立尺}}$ or $2.99^{\text{立坪}}$
13@2.99 = $\underline{38.8}^{\text{立坪}}$

Sectional area for 5" slab and copings.
21.0 × .42 = $8.83^{\text{平尺}}$ for slab.
1.56 − 0.11 = 1.45 for copings.
$10.28^{\text{平尺}}$

Volume of conc. for slab A. 14 Req'd.
10.28 × 54.81 = $563.0^{\text{立尺}}$ for slab and copings.
$\frac{1}{3} \times 20.0 \times .23 \times 3.5$ = 5.4 " for filling on floor beams.
$568.4^{\text{立尺}}$ or $2.63^{\text{立坪}}$

14@2.63 = $\underline{36.8}^{\text{立坪}}$

Volume of conc. for slab B 7 Req'd.
10.28 × 70.41 = $723.0^{\text{立尺}}$ for slab and copings.
$\frac{1}{3} \times 20.0 \times .23 \times 4$ = 6.1 " for filling on floor beams.
$729.1^{\text{立尺}}$ or $3.37^{\text{立坪}}$

7@3.37 = $\underline{23.59}^{\text{立坪}}$

Summary of conc. (1:2:4) in slab.
slab A. 14@2.63 = $36.80^{\text{立坪}}$
" B. 7@3.37 = 23.59 "
" C. 13@2.99 = 38.80 "
$\underline{\underline{99.19}}^{\text{立坪}}$

Area of pavement. (2" thickness.)
~~for slab A. 20.0 × 54.81 − .67 = 1082.8 平尺 or 30.08 平坪 14@30.08 = 421.1 平坪~~
~~for slab B. 20.0 × 70.41 − .67 = 1394.8 " or 38.70 " 7@38.70 = 270.9 "~~
~~for slab. C. 20.0 × 60.35 + .16 = 1210.2 " or 33.60 " 13@33.60 = 436.8 "~~
1187.20 − 4.3 = $1182.9^{\text{平坪}}$

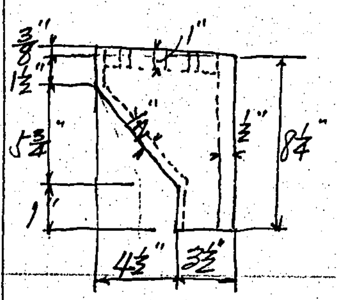
~~Summary of pavement area for slabs.~~
~~slab A 14@30.08 = 421.1 平坪~~
~~" B 7@38.70 = 270.9 "~~
~~" C 13@33.60 = 436.8 "~~
~~1128.8 平坪~~

Area of copings finish.
for slab A. 2.6 × 2 × 54.81 = $284.0^{\text{平尺}}$ or $7.9^{\text{平坪}}$ 14@7.9 = $110.6^{\text{平坪}}$
for slab B. 2.6 × 2 × 70.41 = 366.0 " or 10.2 " 7@10.2 = 71.4 "
for slab. C. 2.6 × 2 × 60.35 = 314.0 " or 8.7 " 13@8.7 = 113.1 "

Reinforcement in slab. (see sheet no. 2)
for slab A. 14@1.685 = 23.59 tons.
for slab B. 7@2.140 = 14.98 "
for slab C. 13@1.795 = $\underline{23.335}$ "

Summary of coping finish area.
slab A. 14@7.9 = $110.6^{\text{平坪}}$
" B. 7@10.2 = 71.4 "
" C. 13@8.7 = 113.1 "
$\underline{\underline{295.1}}^{\text{平坪}}$

Drains. 136 Required.



$\{(1\frac{7}{8}" + 7\frac{1}{4} + 1) + 8\frac{1}{4}"\} \times 8\frac{1}{2} \times \frac{1}{2}"$ = 74.0 cub. inches. for back & front.
$\left\{\frac{\frac{3}{8}" \times 7"}{2} + 1\frac{1}{2}" \times 7" + \frac{7" + 2\frac{1}{2}"}{2} \times 5\frac{3}{4}" + 1" \times 2\frac{1}{2}"\right\} \times 2 \times \frac{1}{2}"$
= (1.31 + 10.5 + 27.3 + 2.5) × 2 × 0.5 = 41.6 cub. ins. for side.
$(1" \times \frac{1}{2}" \times 3\frac{1}{2} \times 8) + (1 \times \frac{1}{2}" \times 7 \times 1)$ = 17.5 cub. ins. for cover.
Total Volume = 133.1 cub. ins.
wt. 133.1 × 0.26 = 35.0 lbs. per one piece.

136@35.0 = 4760# or 2.12 tons.

Summary of Reinforcements.
slab A. 23.59 tons
" B. 14.98 "
" C. $\underline{23.335}$ "
$\underline{\underline{61.905}}$ tons

Summary of Cast iron Drains.
136@35.0 = 4760#
or 2.12 tons.

Expansion Joint for truss spans. 7 Req'd.

1-Pl. 9" ⅜" @ 11.48 × 20'-11" = 240#
1-L. 3"×3×$\frac{7}{16}$ @ 6.10 × 20'-11 = 128#
1-L 5"×3×$\frac{7}{16}$ @ 8.20 × 20'-11. = 172
2-Ls 2½×2½×$\frac{7}{16}$ @ 5.00 × 0'-9 = 8
22-Pls. 2½ × $\frac{7}{16}$ @ 2.66 × 1'-2 = 68
616# or 0.275 ton.

7 @ 0.275 = 1.93 tons

Expansion Joint for plate girder spans. 13 4 T/T

Handrails.

Panel A. 38 Req'd.
1-1½"φ gas pipe @ 2.717 #/ft. × 0'-7 = 2#
2-1" " " @ 1.678 × 0'-7" = 2
4#

38 @ 4 = 152#

panel B 312 Req'd.

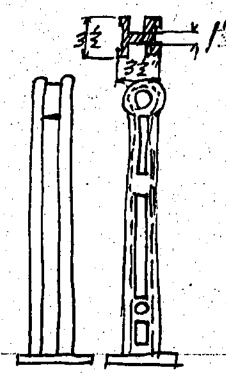
1-1½" gas pipe @ 2.717 × 4'-10½ = 13.#
2-1" " " @ 1.678 × 4'-10½ = 16.
9-Bars 1½" × ¾ @ 3.83 × 2'-6" = 86
115#

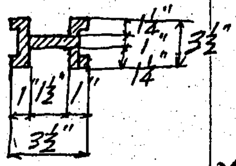
312 @ 115 = 35,880#

Panel C. 504 Req'd.
1-1½" gas pipe @ 2.717 × 4'-9⅞ = 13
2-1" " " @ 1.678 × 4'-9⅞ = 16
9-Bars 1½" × ¾" @ 3.83 × 2'-6" = 86
115#

504 @ 115 = 57,960#

Handrail posts. 843 Required (cast iron)





approximate Volume.
1½ × 1" × 33.0" = 49.5 cub. ins.
1"×3½ × 72.0" = 252.0 "
8½ × 4½ × ¾" = 27.0 "
328.5 cub. ins.

Weight 329.5 × 0.26 = 85.0#

843 @ 85.0 = 71,655.0# or 31.99 tons

Anchor bolts and anchor plates.

1686 Anch. bolts ½"φ × 0'-6" = 776#
843 Anch. plates 2½" × $\frac{5}{16}$ × 0'-8" = 1121
1897# or 0.85 ton.

Intermediate lamp posts. 11 Req'd.
Mean ½" thickness. 24" × ½" × 39" = 468.0 cub. ins
8" × 8" × 1" = 64.0 "
532.0 cub. ins.

wt. 532 × 0.26 = 138# 11 @ 138 = 1518# or 0.68 ton. for cast iron

52 Anch. bolts ½"φ × 0'-6" = 24#
13 Anch. plates 8" × 8 × $\frac{5}{16}$" = 74
98#

summary of structural steel
Expansion Joint for truss.

7 @ 0.275 = 1.93 tons

Expansion Joint for girder spans.
13 4 T/T

total wt of handrail panels.
Panel A 42 @ 4 = 152#
Panel B 312 @ 115 = 35,880
Panel C 504 @ 115 = 57,960
93,992#
or 41.97 tons

total wt of handrail posts.
843 @ 85.0 = 71,655#
or 31.99 ton.

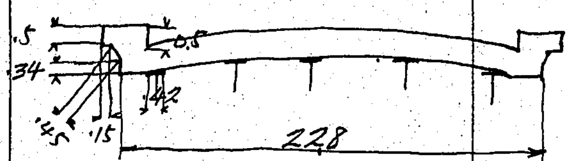
wt of Anchor bolts and plates.
1897# or 0.85 ton.

2½" gas pipe @ 5.79# × 3'-2" = 18#
2" " " @ 3.65 × 3'-10" = 14
32#

Total length of gas pipes of hand rail.
for 1½" pipes.
Panel A 38 @ 0'-7" = 22'-2"
Panel B 312 @ 4'-10½ = 1521'-0"
Panel C. 504 @ 4'-9⅞" = 2430'-9"
3973'-11"

for 1" gas pipes 2@3973'-11" =

Forms.
for truss spans.



copings.
width (.5+.5+.15+.45+.34)×2 = 3.88尺 ~~13@0.86~~=
length. 175'-0" = 176.00尺
area 3.88 × 176.0 = 680.0平方尺
slab.
width 22.8-(5×.42) = 20.7尺
length 176.0尺
area. 20.7×176.0 = 3640平方尺
total area. 3640+680 = 4320.0平方尺 or 120.0方坪
7@120.0 = 840.0方坪

for girder spans.
Copings. 680.0平方尺
slab. width 22.0-(1.0+1.0+.42+.42) = 19.16尺
length 60'-0" = 60.35尺
area. = 60.35×19.16 = 1155.0平方尺
total area 1155.0+680.0 = 1835.0平方尺 or 51.00方坪
13@51.0 = 663.0方坪

summary of weight for handrails.

handrail panels	41.97 tons
handrail posts	31.99 "
anch. bolts and plates	0.85 "
	74.81 tons.

summary of wt. intermediate lamp posts.
13本分.

Castings.	0.8 ton or 1794#
anch. bolts and plates	98
gas pipes	32
	1924#
	or 0.86 ton.

Summary of Materials for surper-structure.

Concrete (1:2:4) for slabs and copings. 99.19立坪
area of pavement for slabs 1182.90方坪
Finish area of copings. 295.10 "
Weight of reinforcements 61.905 tons.
Cast iron drains. 136 @ 35.0 = 4760# or 2.12 tons.
Structural steel expansion joints
for truss spans. 7@0.275 = 1.93 tons.
Expansion joints for girder spans 13 個所
Weight of handrails Panels. 41.97 tons. ...... ☆ total length of gas pipes
Cast iron handrail posts 843 @ 85.0 = 71,655.0# or 31.99 tons.
Anchor bolts and Anch. plates 0.85 ton
Intermediate lamp posts 13 Required casting 138#/one piece
Bolts and plates 98
236#
Forms 1503.0方坪

for 1½" 3973'-11"
for 1" 7947'-10"

Summary of area of forms
for truss spans 840
for girder spans 663
1503.0方坪

岡山縣 霞橋

豫備設計書

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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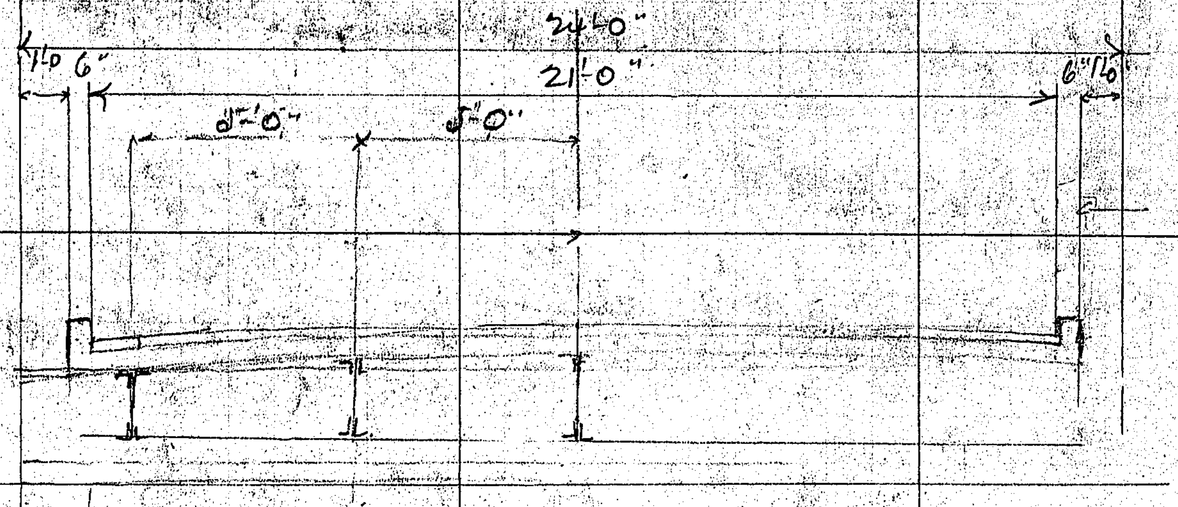
CALCULATIONS FOR

Estimate of cost for Kasumi bashi for Okayama ken

21' wide Roadway 15½E Loading
span length = 176'-0" clear. 12 panels @ 14.68'
or 10 panels @ 17.6'

Cross section of floor



Floor slab.
2" pavement 25#
5½" slab 69
94# call this 95#

Bending moment = $\frac{1}{10} \cdot 95 \cdot 5^2 = 238'^{\#}$

Live Load
Distribution of motor truck concentration $0.67 \cdot (5+1.0) + 1.33 = 5.33$
wheel conc with 30% impact = 6435 $\frac{6435}{5.0} = 1286^{\#}$ per ft
moment = $\frac{1286 \cdot 2.5}{2} = 3220'^{\#}$
For continuity $0.8 \cdot \frac{3220}{2} = \frac{2500'^{\#}}{2} = 1250'^{\#}$
238
1488

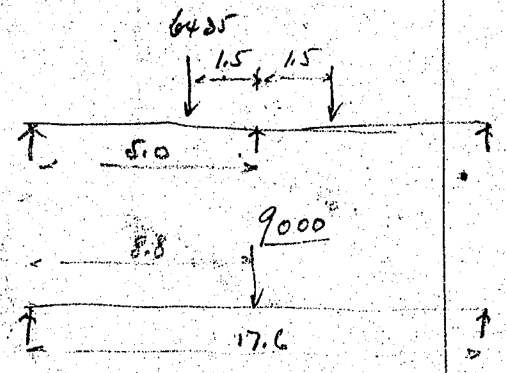
Effective depth = $\sqrt{\frac{1488}{103}} = \sqrt{14.5} = 3.8$
make slab 5".

Dead Load slab + pavement and copings.
21 × 87.5 = 1840
coping say 300
2140#
Handrail 2@80 160
2300 ÷ 2 = 1150# per ft. per truss

2154
1300
3454
1727

Stringer span length . 17.6' spacing 5.0'
Dead Load 90 × 5 = 450
D.L. stringer 45
495#
$m = \frac{1}{8} \cdot 495 \cdot 17.6^2 = 19200'^{\#}$

Live Load moment



$6435 \times \frac{3.5 \times 2}{5.0} = 9000^{\#}$

LLm 4500 × 8.8 = 39,600
DLm 19200
58800 call this 60.000

Sm reqd = $\frac{60.000 \times 12}{17.000} = 42.3$

use 12" × 6" = 44.0#

3@ say 46 = 132#
2@ 35 = 70
202# per lin ft

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CALCULATIONS FOR

Estimate of Cost for Kasumi Bashi for Okayama Ken

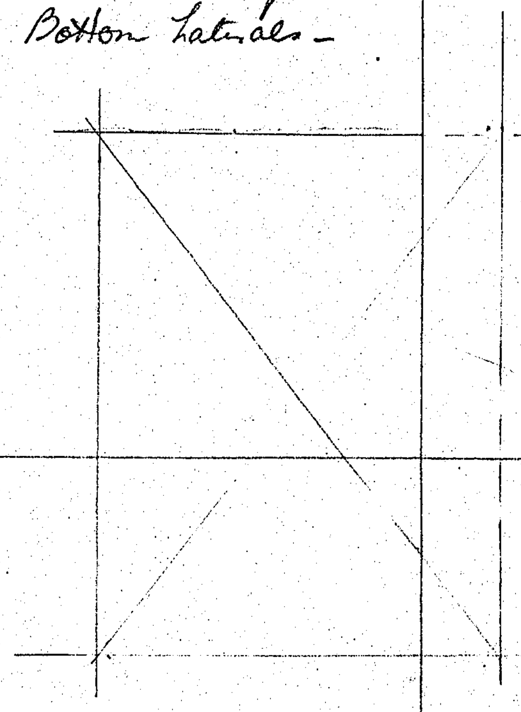
Floor Beam say 4Ls 5 × 3 × 3/8 @ 9.8 × 23.0 = 900#
1 Pl. 30 × 5/16 @ 31.88 × 23.0 = 732
1632# × 1.25 = 2040# per piece

9 @ 2040 = 18360
2 @ 1950 = 3900
22260

out to out of span 178.5' 22260 ÷ 178.5 = 125# per lin ft of span

Lateral Bracing

Bottom Laterals – 2Ls 5 × 3½ × 5/16 @ 8.7 × 27.0 = 470#
470
center connection – 50
990#



2Ls 4 × 3 × 5/16 @ 7.2 × 27.0 = 390#
390
50
830#

4 @ 1000 = 4000
6 @ 830 = 5000
9000 ÷ 178.5 = say 50# per lin ft

Top Lateral Bracing –
4Ls 4 × 3 × 5/16 @ 7.2 × 27.0 = 780
270
1050 × 2 = 2100#

10 @ 2100 = 21000
Sway. 9 @ 1200 = 10800
Portal Bracing. 2 @ 3000 = 6000
37800 ÷ 178.5 = 210# per lin ft

Making top diagonals, tension member
2Ls 5 × 3½ × 5/16 @ 8.7 × 27.0 = 470
470
center connection – 50
990# say 1000#

Sway Bracing – 4Ls 4 × 3 × 5/16 @ 7.2 × 23.0 = 660#
230
890
Sway lattice say 610
1500#

Diagonals – 10 @ 1000 = 10,000
Sway 9 @ 1500 = 13500
Portal 2 @ 3000 = 6000
29500 ÷ 178.5 = 165# per lin ft
call this 170# per lin ft

Load on truss –

stringer	202
Floor Beam	125
Laterals Bott	50
" top	170
trusses assumed	800
	1347

Dead Load Deck – 2300#
metal say 1400
3700
1850# per truss.

Dead Load moment
$\frac{1}{8} \times 1850 \times 176^2 = 7,250,000$

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CALCULATIONS FOR

Estimate of Cost for Kasumi Bashi for Okayama Ken

Live Load on truss -

Uniform load - say 7 × 1050 × 176² = 4,07.0000

conc. 6000/2 × 176/2 = 264 000

summary for moment = DL. 7250.000
LL 4070.000
LL 264.000
11,584,000 1#

stress in chord Depth of truss at center = 32.
stress = 1,1584.000 = 362.000#
sq = 362000 ÷ 14000 = 26.0□".

weight of truss = 26.0 × 3.0 × 3.4 = 266.0#
details say 37% 100.
366#

weight of steel -

stringer	202
Floor Beams	125
Bott Laterals	50
Top Laterals	170
trusses	732
	1279 call this 1280

Total weight = 1280 × 178.5 = 210,500# or 94 tons
or say 98 tons per truss including shoes.

5 trusses @ 98 tons = 490 tons.

885

Design of Pier

Load on pier say Dead Load Deck - 2300
Metal. 1300
3600#

DLoad 3600 × 178.5 = 642,000#
Live Load 2100 × 178.5 = 375,000
1017000 ÷ 4 = ~~509000~~ ~~254500~~ # per shoe.

Bearing area = 204500 / 450 = 455.

21" × 24"

4" Roller 204500 ÷ 2400 = 85" = 5 @ 4"

Center to center of shoe say 2'0"

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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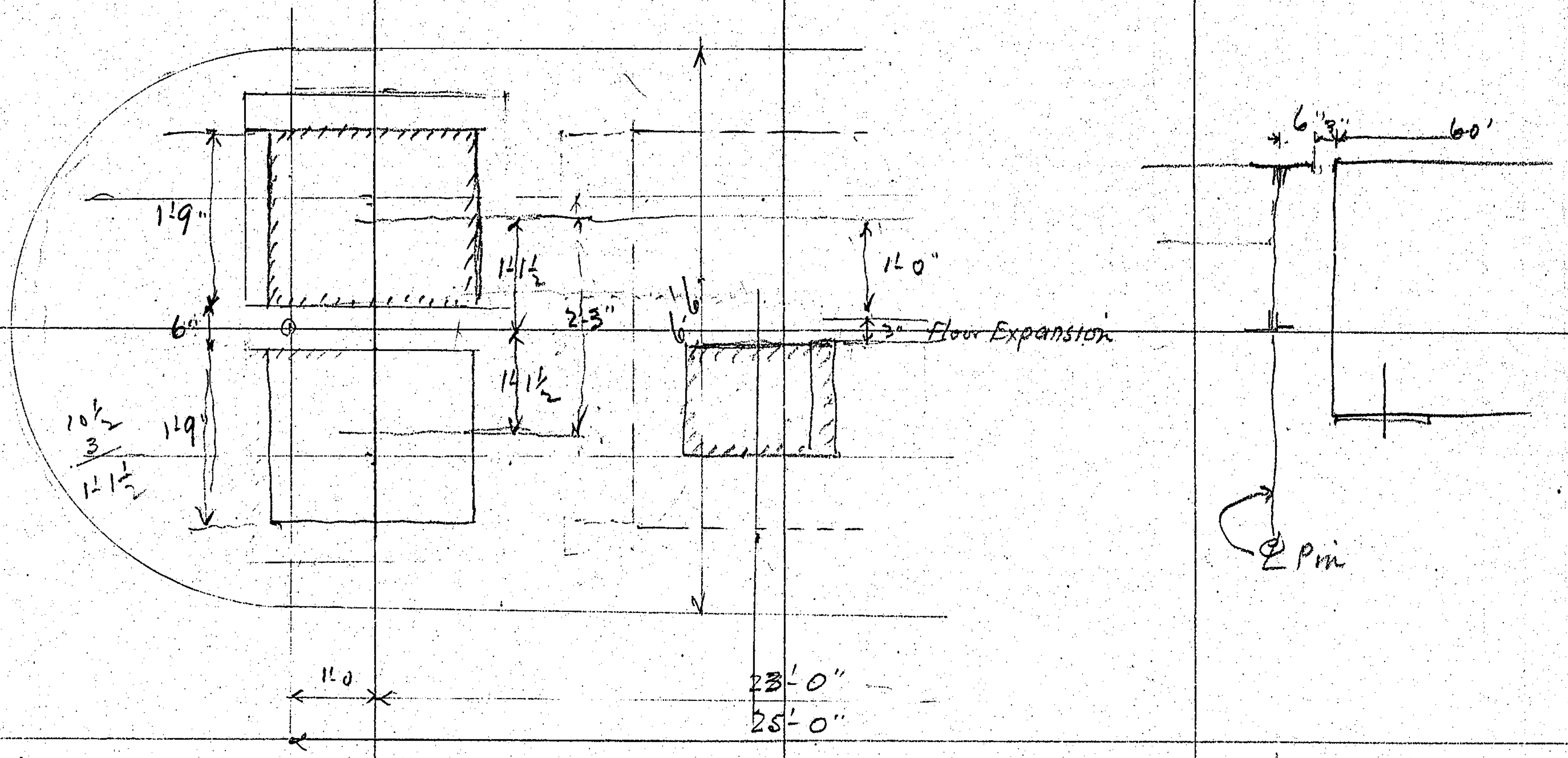
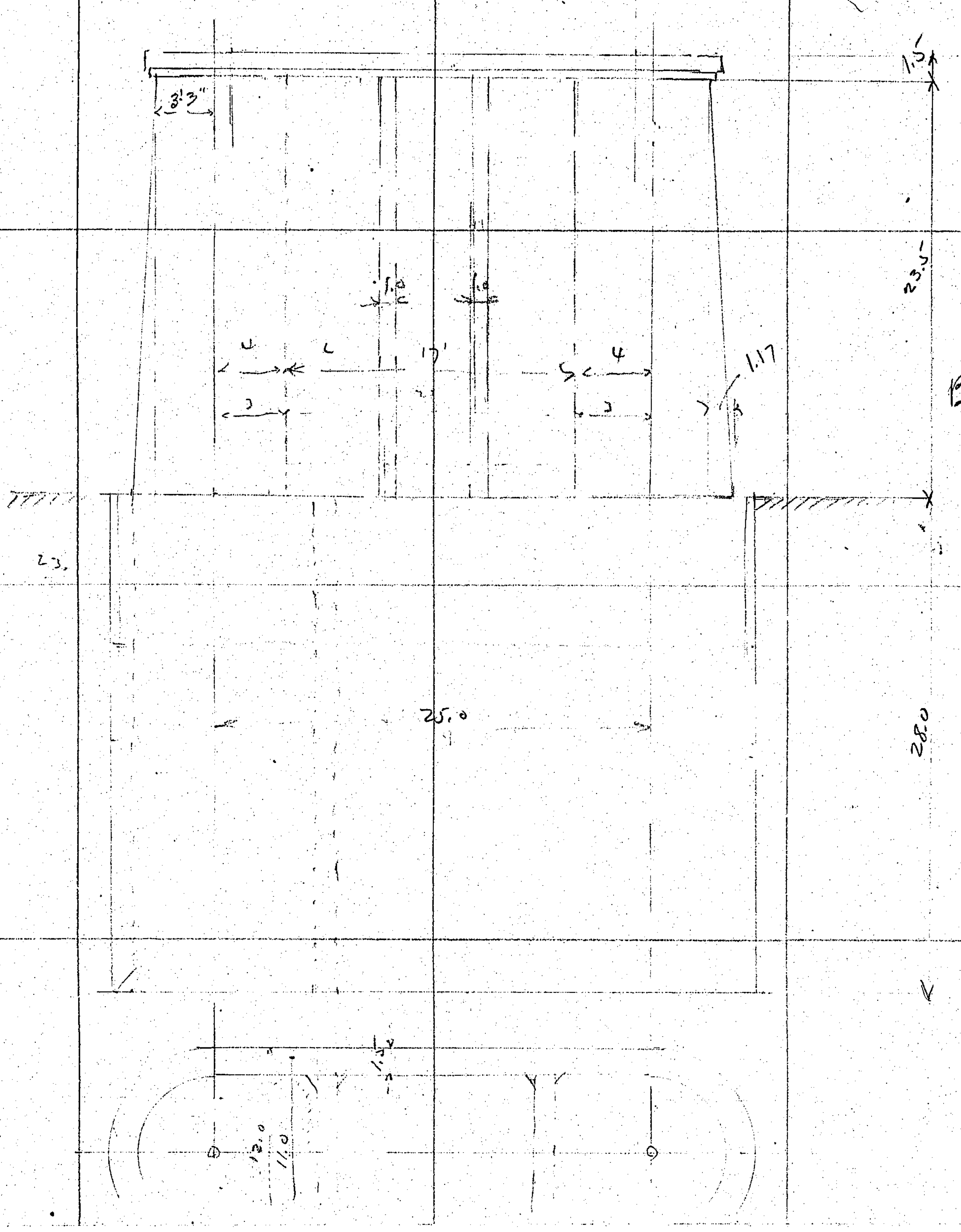
CALCULATIONS FOR

Estimate of Cost for Kasumi bashi for Okayama ken





6.5
2.35
8.85
27.5
12.00

57.3 @ 30.000.. = 1,720,000#
Super imposed load 1017,000
2,737,000

Bott Area 413.0 sq ft
$\frac{3737,000}{413}$ = 6620#/□'
or 2.96 tons/□'

2 3

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Estimate of Cost for Kasumi-Bashi for Okayama Ken

Estimate of pier

coping - 7.5° = 44.2
7.5 × 25 = 187.5
231.7 × 1.5 = 348. cubic ft

shaft 6.5° = 33.2
8.85° = 61.5
94.7 ÷ 2 = 47.3 av.
10 × 7.67 = 76.7
15 × 2 = 30.0

6.5
8.85
15.35
7.67

348.
154.0 × 23.5 = 3620.
3968 ÷ 216 = 18.4 坪

well :-
12° = 113.1
12 × 25 = 300.0
413.1 °'
9'° = 63.6
9 × 25 = 225.0
288.6

413.1
288.6.
124.5 °'
22.0 partition wall.
146.5

shell $\frac{146.5 \times 28.0}{216}$ = 19.0 坪

Excavation $\frac{413.1 \times 28}{216}$ = 53.5 坪

Inside filling 6' at bottom 1:2:4
$\frac{266.6 \times 6}{216}$ = 7.40 坪

Top filling 3' 3.70 坪.

Intermediate filling 28 - 9 = 19'

63.6
9 × 4 36.
99.6. × 19 ÷ 216 = 8.76 立坪

1:2:4 concrete Reinf.
coping + shaft 18.4 @ .2 3.6 tons
well 19.0 3.6
Top filling 3.7 7.2
41.1 立坪
1:2:4 concrete in water - 7.4 坪
Intermediate filling 8.76.

41.1
7.4
8.8
57.3

Estimate of cost

1:2:4 concrete 41.1 @ 126.00 = 5170
1:2:4 concrete 7.4 @ 130.00 = 960
1:4:8 concrete filling 8.76 @ 80.00 = 700
Reinf 5.0 @ 200 = 1000.00
7830.00
form say 800
curb shoe say ~~200~~
excavation 53.5 @ 50.00 2700
11680 call this ~~12000.00~~ 10600 price -
10680

cost of concrete
Cement 11 @ ~~6.50~~ 6.5 = 71.50
sand. 7.50
gravel - 20.00
labor 27.00
126.00

For side span say - 9000.00

Estimate of cost 4 @ ~~12000~~ 10600 = ~~48000~~ 42400
2 @ ~~9000~~ 8000 = 18000 16000
66000 58400

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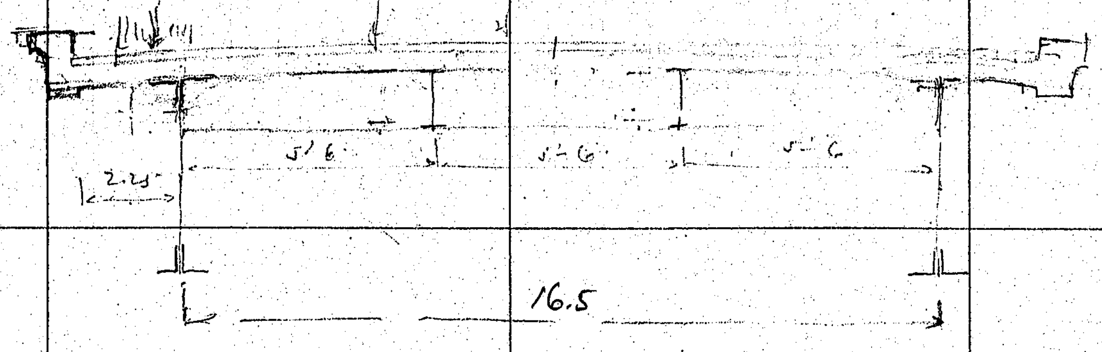
CALCULATIONS FOR

Estimate of Cost for Kasumi Bashi for Okayamaken

Estimate of Cost
60' girder span 21' wide - Slab

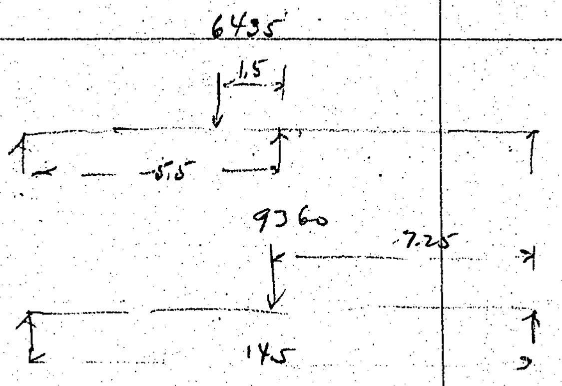


21'-0
15
6'-0

Floor slab. (5¼") 2" pavement 90# × 5.5 = 495
stringer 35
530

moment = 1/8 × 530 × 14.5² = 13900 1#

Live Load.



6435 × 4.0/5.5 = 4680
4680
9360#

LL moment = 4680 × 7.25 = 340.00
139 00
479.00

Sm = 47900 × 12 / 17000 = 33.8

Use 12" × 5" × 31.99# I Sm = 36.686

Floor beam web 24" × 5/16 25.5
4Ls 4 × 3 × 5/16 28.8
54.3 × 1.25 = say 70# per ft
weight of one piece = 70 × 16.5 = 1150
5 @ 1150 = 5750 95# per lin ft

Stringer say 2 @ 35# = 70
Floor beam 95
Lateral bracing 50
Girders assumed 400
615

weight of floor 90 × 21 = 1890
Coping 350
H Rail 160
2400
3015 ÷ 2 = 1510# per girder

Main girder
D.Load 1510
L.L 1050
moment = 1/8 × 2560 × 58² = 1075000
Conc at Center 3000 × 58/2 = 87000
1162000 1#

web assumed = 48 × 3/8 = 18.0 S web = 2250"
Effective depth 4.04
.10
3.94
295000
17350 net
2.25
15.10

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CALCULATIONS FOR

Estimate of Cost for Kasumi Bashi for Okayama Ken

Top 2Ls 6×6×½ = 11.50 q. 9.50 net
1Pl. 14×½ 7.00 6.00
15.50

15.10
9.5
5.6

Length of Cover pl. $\overset{65}{\cancel{58}}\times\sqrt{\frac{6.0}{15.50}}+2 = 38'$

main section 4Ls 6×6×½ @ 19.6 × 60.0 = 4710
1Pl 48×3/8 @ 61.2 × 60.0 = 3670
2Pls. 14×½ @ 23.8 × 38.0 = 1810

10190#
3810
14000#

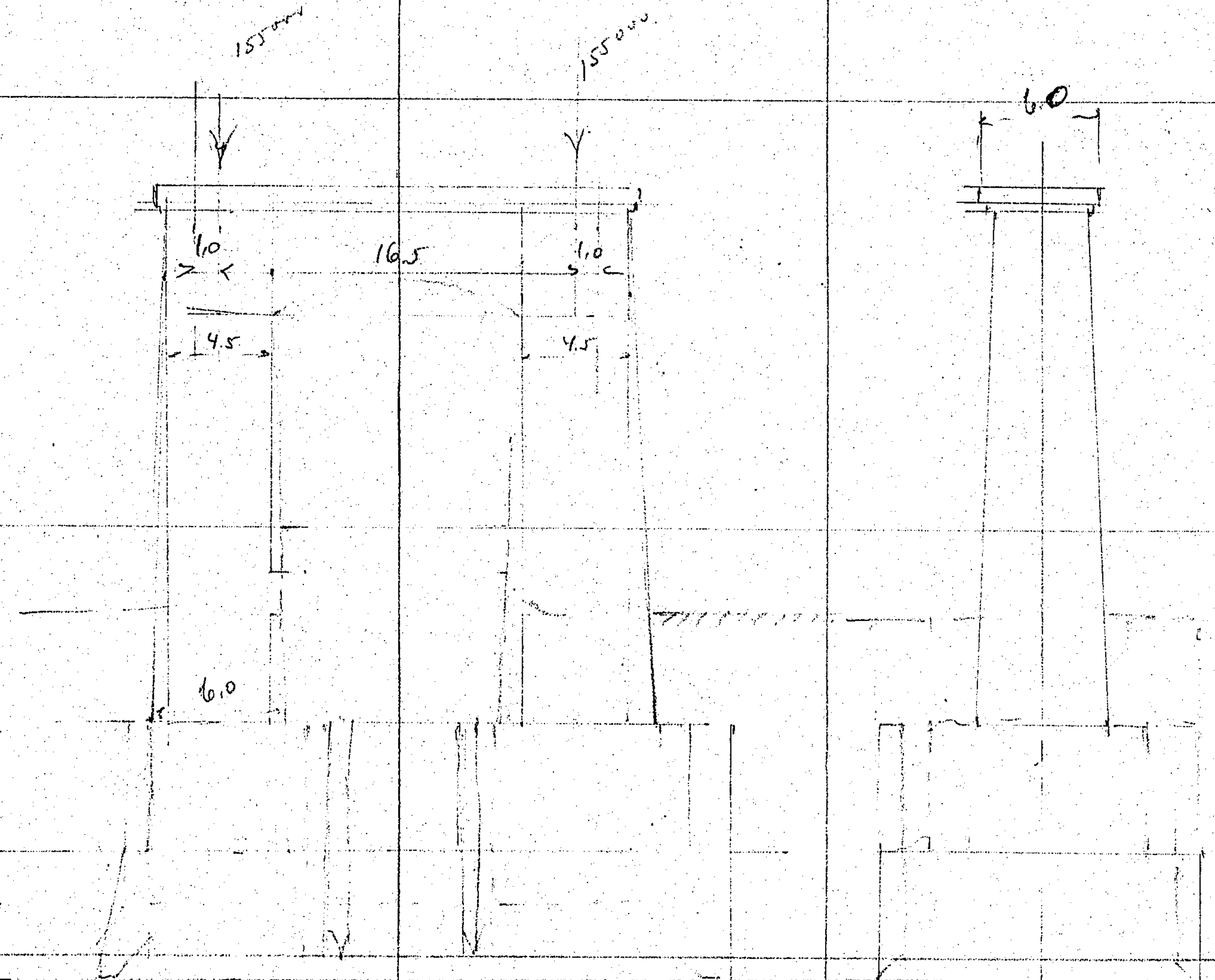
14000 ÷ 60 = 235#

Stringers 2@ 35 = 70
Floor beam 95
Lateral Bracing. 50
Girders 470
685# × 60 = 41100 or 18.35 tons

Load on pier.

DL. metal - 685
deck 2400
3085
LL. 2100
5185# × 30 = 155,000# ÷ 2 = 77500# on shoe.

Load on pier = 310.000 #



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CALCULATIONS FOR

Estimate of cost for Kasumi Bashi for Okayama Ken

Concrete in pier

Coping. 6' dia. 28.3
16 × 6 = 96.0
124.3 × 1.5 = 187.

Shaft 4.5 dia 15.9
28.3
44.2 / 2 = 22.1 × 23.5 = 520 / 216 = 2.4 立坪

Strut 12 × 4 × 3 / 216 = .66

Concrete in base. 15 × 12 × 31.5 / 216 = 26.2 立坪 Exc = 15 × 31.5 × 17 / 216 = 37.1

Area = 15 × 31.5 = 472.

Total concrete	coping	.87
	shaft	4.80
	strut	.67
	base	26.20
		32.54 @ 30,000 = 1,085,000#
Super imposed load		310,000
		1,395,000

Max bearing = 1395000 ÷ 472 = 2950# 1.3 tons/□'

Estimate of cost

concrete	32.54	@	126.	= 4100.00
Reinforcing bars	3	@	200	= 600
Excavation	37.1	@	35.00	= 1300
form say				500
				6500.00

30
24
72

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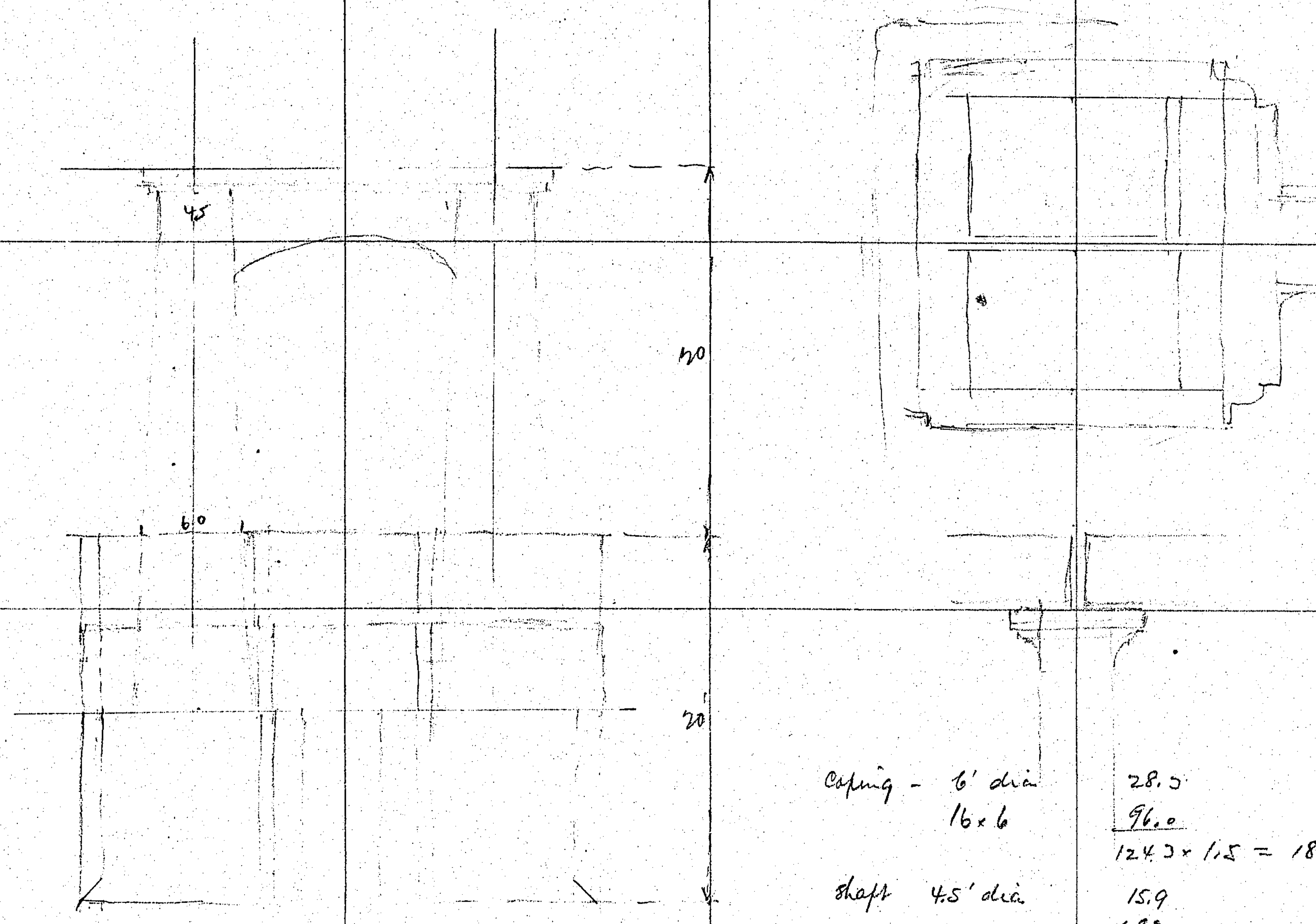
CALCULATIONS FOR

Estimate of Cost for Kasumi bashi for Okayamaken

Revised pier for girder span



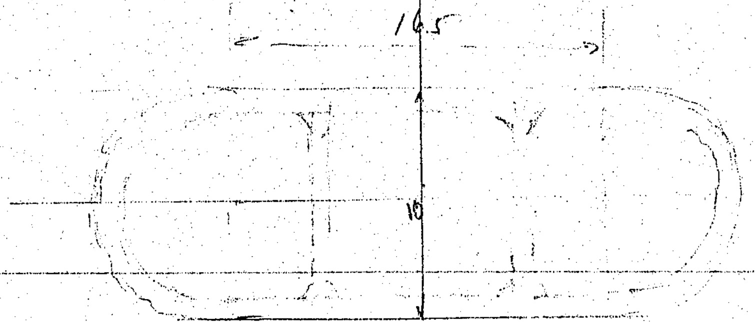
coping - 6' dia 28.5
16 × 6 96.0
124.5 × 1.5 = 187

shaft 4.5' dia 15.9
28.3
44.2 ÷ 2 = 22.1

$\frac{2 \times 22.1 \times 18.5}{216}$ = 3.80

Topstrut say .67
coping .87
5.34 立坪

Reinforcing bar 1.5 tons



Shell of well -
10' dia 78.5
16.5 × 10 = 165.0
243.5 outside

7 dia = 38.5
16.5 × 7 = 115.0
153.5

243.5
153.5
90.0

volume = 90 × 20 = 1800
3.0 × 7. × 20 420
4. × 20 80
$\frac{2300}{216}$ = 10.7 立坪

Reinf. 2.7 tons

Top and bottom filling
9 × $\frac{153.5}{216}$ = 6.40 立坪

Inside sand filling
38.5
7.0 × 7. 49.0
87.5 × 11. = 4.45 立坪

Excavation
$\frac{243.5 \times 25}{216}$ = 28.0 立坪.

concrete 5.34
10.70
6.40
4.45
26.89 @ 30.000 = 790.000 #

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CALCULATIONS FOR

Estimate of cost for Kasumi-Bashi for Okayamaken

Total wt pier 790.000
super imposed ld 310.000
1,100,000 ÷ 243.0 = 4500 2.0 tons/□

Estimate of cost

shaft	5.34	@	126^{00}	=	674
Reinf	1.5	@	150	=	225
form					150
					1049

well

shell -	10.7	@	126^{00}	-	1350
concrete	6.40	@	126^{00}	-	810
filling	4.45	@	10^{00}	=	44
Reinf -	2.7	@	150^{00}	=	405
Exc -	28.0	@	40^{00}	=	1120
form say					200
					3929
					1049
					4978 5000^{00}

18 span - on left bank 17 piers. @5000 = 85000
2 span " Right bank 1 piers. 5000
20 spans. 90,000^{00}

60' girder span 20×18.5 = 370 tons @210 = 77600
167,600
5 truss spans - 490 tons @250 122500
66000
356100^{00}
Deck say 77000
abutment etc 20000
453100^{00}

Assume cost of one pier - 4500^{00} per piece -
Economic span length of girder
15 @ 73.3' -
14 @ 78.5

73.3 clear 72.0 moment = $\frac{1}{8} \times 2700 \times 72^2 =$ 1,750,000
$3000 \times \frac{72}{2}$ 108 000
1858 000

1 web 60 × 3/8 = 22.5
$\frac{1}{8}$ web = 2.80

stress = 377 000
SR = 22.2
2.8
19.4

2LS 6.6.3/4 16.44 13.44
1 cov Pl. 14×1/2 = 7.00 6.00
23.44 19.44
22.00
23.44
69.38 @ 3.4 = 236$^{\#}$ × 1.30 306

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CALCULATIONS FOR

Estimate of cost for Kasumibashi for Okayama Ken

weight of girder span 733

Stringer 2@35 = 70
FB 5 95
Lateral Bracing - 50
Girders - 612
827# × 73.3 = 60500 27.0 tons per span

15 spans @ 27.0 = 405 tons

Girder span 14@ 78.5 77.0 clear

moment = $\frac{1}{8} \cdot 2700 \cdot 77^2$ = 2000.000
3000 × $\frac{77}{2}$ = 115.000
2115.000

1 web 60 · 3/8 = 22.5
1/8 web = 2.80

stress = 432000
SR = 25.4
2.8
22.6

2Ls 6. 6. 3/4 16.44 13.44 net
1 cov. pl. 13. 1/2 = 6.50 5.50
1 cov. pl. 13 × 3/8 = 4.87 4.12
27.81 23.06
27.81
22.50
78.12 @ 3.4 = 266 Including details say 336#
236
30

Stringer 70
FB 95
Lateral 50
girders - 672
887 × 78.5 = 69500# 31.0 tons - per span.

14@31 = 434 tons 42

	60.	73.3	78.5
girderspan	18@18.5 = 335@210 = 70400	405@210 = 85000	434@210 = 91,000
	17@4500 76500	13@4500 = 58500	12.@4500 54000
	146900	144500	145.000

5 —
4 —
— .75

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CALCULATIONS FOR

Estimate of cost for Kasumi bashi for Okayama Ken

Truss spans -
6 @ 149 = 894'
DL m = 1/8 × 1850 × 147² = 5000.000
LL m = 1/8 × 1000 × 147² = 2840.000
3000 × 147/2 = = 220.000
8,060,000

Depth of truss 27' Stress = 300.000 SR = 21.4 × 3. × 3.4 = 219.
37%. 81
300#

weight of steel

Stringer	202
FB.	125
Bott Lateral	50
Top lateral	150
trusses	600
	1127

Total weight = 1127 × 149 = 168000 - 75 tons 4 4/2 2
each truss 77 tons.
6 trusses @ 77 - 462 tons.

5 @ 178.5 6 @ 149 -

Steel piers 490 tons @ 250 = 122500 122500 462 @ 250 = 115,200 115,200
66000 46.600 77000 54600
188500 = 169100 192200 169800

Estimate of cost of Deck.
concrete in Floor slab - Total length 342.5
46 × 21 = 965 4
2. 338.5 say 2030'
11.65 × 2030/216 = 110, 立坪.
Reinforcing bars. 80 tons -

Pavement = 338.5 × 3.5 = 1180 平坪

cost

Floor slab.	110	@	126.00	=	13900
Reinf. -	80	@	180.00	=	14400
Pavement	1180	@	14.00	=	16500
HR.	4060	@	7.00	=	28420
forms					5000.
					78220.00

See Revised figures. pp18

Layout -

5 spans	@	178.5	=	892.0
2	@	60	=	120
17 spans	@	60	=	1020
				2032' -

2030
1012
1018

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MADE BY Jm DATE 14-8-17 FILE NO
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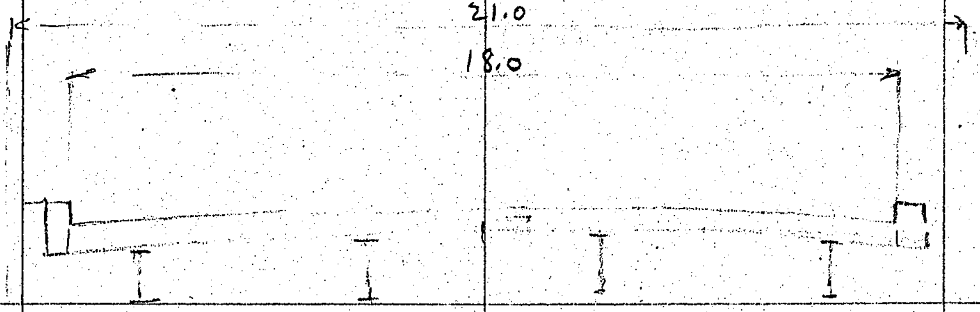
CALCULATIONS FOR

Estimate of cost for Kasumi-Bashi for Okayama Ken

Steel - 5 spans 178.5 490 tons @ 250 = 1225.00 ⁰⁰
piers 4 piers 4 @ 10600 = 42400
— 2 piers 2 @ 8000 = 16000
Deck span 19-60'. 352 tons @ 210 = 74000
piers - 17 @ 4500 = 76500
abutment 2 @ 5000 = 10000
Deck say 78200
misc - 5000
424600 ⁰⁰

See page no 18 for Revised figures -

18' wide Roadway -
truss span 178.5 span



slab 5" + 2" pavement
90 × 18 = 1620
coping - 150
H.R. = 2080 160
1930#
1144
3074 ÷ 2 = 1540# per ft per truss

stringers 4 @ 46 = 184
Floor Beam 110#
Lateral 45
Top Laterals - 155
truss assumed 650
1144#

D.L. 1540
L.L. 900
2440

moment = ⅛ × 2440 × 176² = 9.440.000
L.L. Conc. 264,000
9,704,000'#

Depth of truss = 32 S = 304000 SR = 21.7.
wt = 21.7 × 3 × 3.4 = 221
37% = 82
303#

metal in span = 184
110
45
155
606
1100# × 178.5 = 196.000 87.5 call this 90 tons

5 spans @ 90 = 450 tons.

Piers

coping - 6.5 ⌀
6.5 × 23.0 = 33.2
150.0
183.2 × 1.5 = 275/216 = 1.27 cubic 坪

shaft 5.5 ⌀ 23.7
7.85 48.4
13.35 72.1 ÷ 2 = 36.0
6.7
8 × 6.7 = 53.5
2 × 15 = 30.0
119.5 × 23.5/216 = 13.00
14.27 立坪

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MADE BY JM DATE 14-8-17 FILE NO

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CALCULATIONS FOR

Estimate of Cost for Kasumibashi for Okayamaken

well dia 11.5

11.5^{ϕ} = 103.8 8.5^{ϕ} = 56.7
11.5 × 23 = 264.0 8.5 × 23 = 196.0
367.8 252.7
252.7 − 29.5
115.1 223.2
85 × 3.0 = 25.5
4.0
144.6 × $\frac{235}{216}$ = 15.75 cubic tsubo

Top and bottom filling $\frac{223.2 \times 9}{216}$ = 9.3 cubic tsubo

Intermediate filling 56.7
8.5 × 4 34.0
90.7 × 19 ÷ 216 = 8.0 cubic tsubo

Summary of concrete
Con coping + shaft 14.27
well 15.75
Top + bottom filling 9.30
39.32

Excavation = 367.8 × $\frac{28}{216}$ = 48.0 cubic tsubo
Reinforcing bars = ~~6.0~~ say 5.0 tons

Estimate of cost
concrete 1:2:4 − 39.3 @ 126.00 = 4950
1:4:8 8.0 @ 5.00 = 40
Reinf. − 5 @ 180 = 900
5890
forms 500
curb shoe say 100
excavation say 48 @ 50 = 2480
8970.00 call this 9000.00

End piers say 7000.00 a piece.

Plate girder spans 19-60' spans
center to center of main girders 15.0' Revise
stringers say 2 @ 35 = 70 70 Floor. 90 × 18 = 1620
Floor beam 90 90 300
Lateral Bracing say 40 40 HR. 160
Girders assumed 400 430 2060
600# 630# per lin ft 600
2660 1330 per girder.

Main girder DL. 1330
LL. 900
2230
Moment = $\frac{1}{8}$ × 2230 × 58^2 = 938,000
conc. 3000 × $\frac{58}{2}$ = 87,000
1,025,000

~~web assumed~~ = 48 × 3/8 = 18.0
$\frac{1}{8}$ web = 2.25 stress = 264000 Sk = 15.5
2.25
13.25

Try − 2Ls 6 × 6 × $\frac{1}{2}$ = 11.50 9.50
1Pl 12½ × 3/8 = 4.70 4.00
15.20
15.20
18.00
48.40 × 3.4 = 165. × 1.30 = 215#
630 × 60 = 36700 17 tons
19 @ 16.5 = 313 tons

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CALCULATIONS FOR

Estimate of cost for Kasumi bashi for Okayamaken

Piers assumed 4000.00 per piece averaged

Estimate of cost of Deck

Floor slabs -	97. @ 126.00	=	12200	12200
Reinf	70 @ 180	=	12600	12600
Pavement	1015 @ 14.00	=	14200 13.00	13200
HR			28420	22300 Pavement =
forms			4000	4000
				5000 - misc.
			71420.00	69300

46 . 18 = 8.3
2
10.3 × $\frac{2030}{216}$ = 97. slabs
70 tons.
3 × $\frac{2030}{6}$ = 1015 t.

Estimate of cost 18' wide -

steel	5 @ 178.5	450 @ 250	=	112500
piers	4 @ 9000		=	36000
"	2 @ 7000			14000
Girder span	19-60	313 @ 210	=	65700
	17 @ 4000			68000
Deck		abutment		71400 9000
misc				5000
				372600.00
				9
				381600

820

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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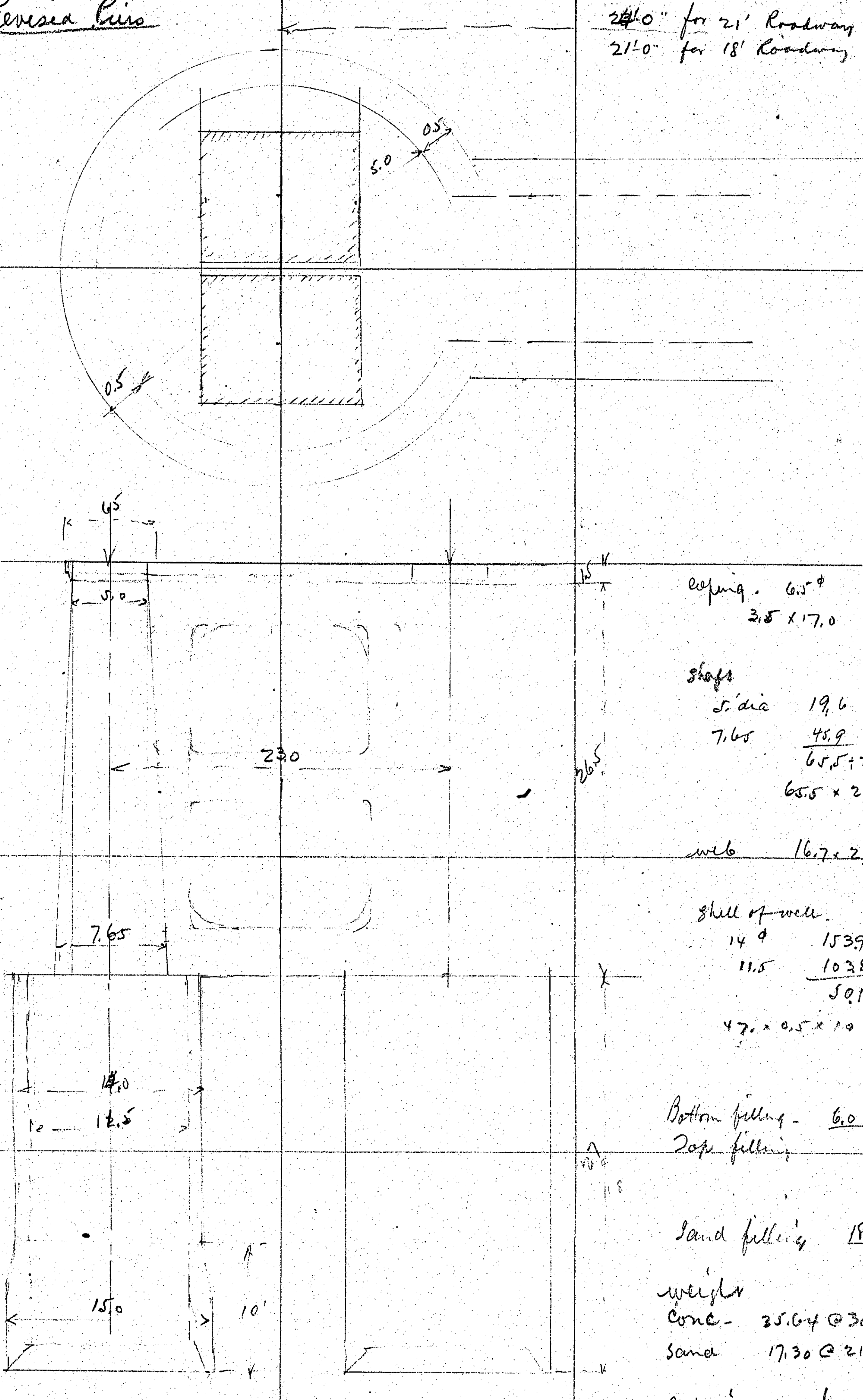
CALCULATIONS FOR

Estimate of Cost for Kasumi bashi for Okayamaken

Revised Piers

5.00
7.65
12.65
6.32

23.0
6.32
16.68



coping. 6.5 ϕ 33.2
3.5 × 17.0 59.5
92.7 × 1.5 = 140.0

shafts
5' dia 19.6
7.65 45.9
65.5 ÷ 2 = 32.8 ×
65.5 × 26.5 = 1735.0
1875.0 8.73
web 16.7 × 2 × 26.5 / 216 = 4.1
12.8

Shell of well.
14 ϕ 1539
11.5 1038
501 × 26 = 1300
47. × 0.5 × 10 = 235
1535 ÷ 216 =
2 @ 7.1 = 14.2

Bottom filling - 6.0 × 103.8 / 216 = 2.88
Top filling 1.44
4.32
2 @ 4.32 = 8.64

Sand filling 18 × 103.8 / 216 = 8.65 cubic -
2 @ 8.65 = 17.30

weight
Conc. - 35.64 @ 30,000 = 1070.000
Sand 17.30 @ 21600 = 374.000
1444 000
Super imposed load 1017 000
2461 000

Bearing Pressure = 2461000 / 353 = 7000 3.12 tons/□'
2-15ϕ

Excavation = 353 × 27 / 216 = say 45 ㎥

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CALCULATIONS FOR

Estimate of cost for Kasumibashi for Okayamaken

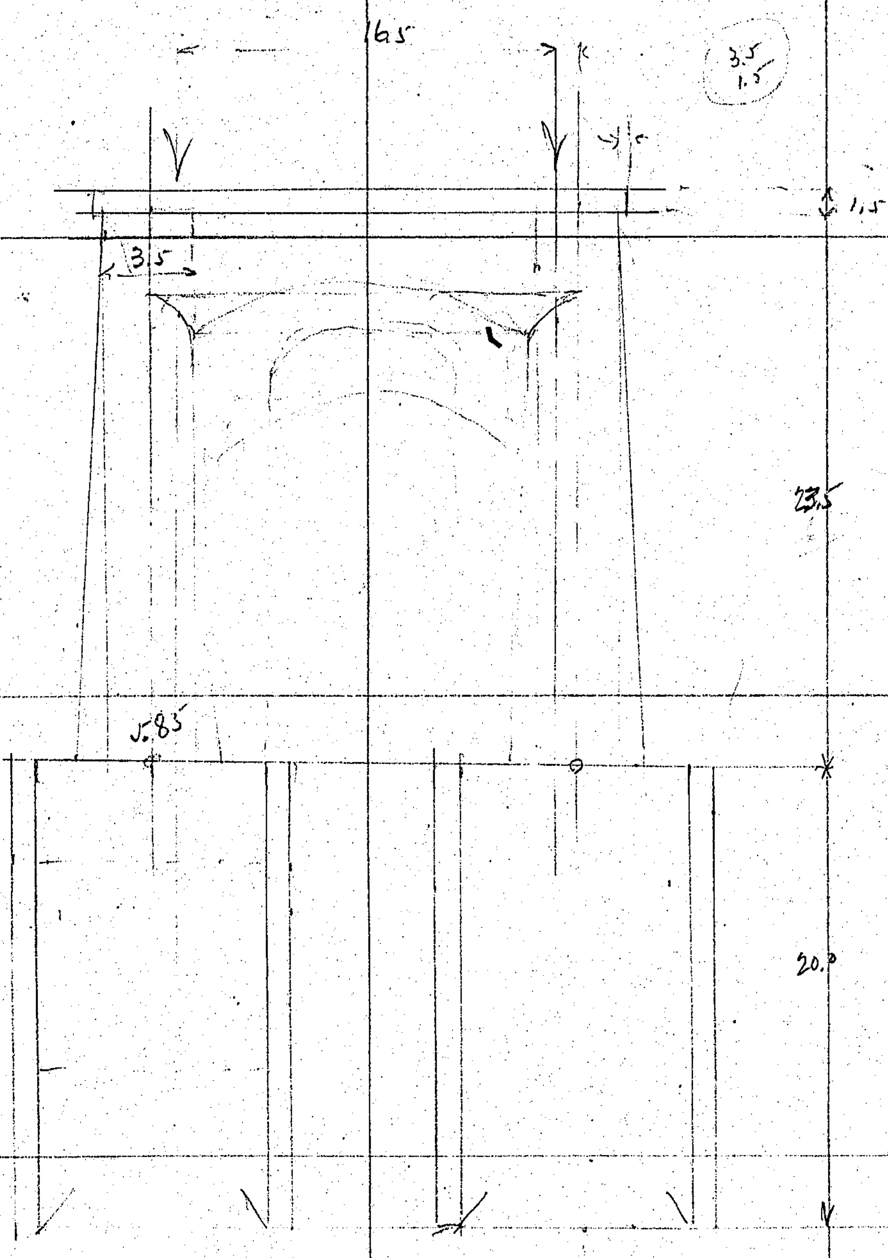
Estimate of cost

Concrete	35,64	@	126^{00}	=	4500^{00}
Sand fill	17.3	@	5^{00}	= say	90^{00}
Reinf -		5@	180	=	900^{00}
form say					400
excavation			45 @ 50	=	2250
					8140^{00}

4 @ 8140 = 3260̶00
2@ 7000 = 14000
46600

6 @ 8140 = 49000
14000
63000

Piers for girder spans.



Coping. 5'4 19.6
18.5 × 5.0 = 92.5
112.1 × 1.5 = 168.5

Shaft 3.5 : 9.6
5.85 26.8
36.4 × $\frac{23.5}{216}$ = 3.96
Coping .80

web. 1.5 × 13.5 × $\frac{30}{216}$ abt = say 3.00
7.76 cubic tsubo

well 12' dia 113.0
10' dia 78.5
34.5

2@ $\frac{34.5 \times 20}{216}$ = 6.39 tsubo

filling 2@ $\frac{78.5 \times 9}{216}$ = 6.55 tsubo

middle filling -

2@ 78.5 × $\frac{11}{216}$ = 8.00 tsubo

Excavation 12'
2@ $\frac{113 \times 23}{216}$ = 24.1 tsubo
call this 25 tsubo

Concrete
7.76
6.39
6.55
20.70 @ 30,000 = 620.000
Sand - 8.00 @ 21000 170.000
790.000
super imposed load 310.000
1,100,000

Unit bearing = $\frac{1.100,000}{226}$ = 4880
2,18 tons/□'

Estimate of cost

Concrete •	20.70	@	126^{00}	=	2610
Reinf -					400^{00}
Excavation -	25	@	35^{00}	=	880^{00}
form					250
					4140^{00}

call this 4000^{00} average.

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CALCULATIONS FOR

Estimate of Cost for Kasumibashi for Okayamaken

Estimate of Deck

Floor slab -	110 @ 126^{00}	=	13900^{00}
Reinf.	80 @ 180^{00}	=	14400
Pavement	1180 @ 13^{00}	=	15350
HR	4060 @ 5^{50}		22300
forms			5000
misc.			5000
			76050^{00}

12,000

16910

Final Estimate - 21' Roadway trusses + Girder Design -

5 @ 178.5	490 tons @ 250^{00}	=	122500
Piers -			46600
Deck 19 - 60'	352 tons @ 210	=	74000
piers -	17 @ 4000		68000
abutments	2 @ 5000		10000
Deck -			76000
			397100^{00} ÷ 1180 = 336^{00}/坪

142 000

178.5
5
892.5

19
60
1140

10 x 190
665 x 5.3 tons/坪

1.15
1.50

126
384

Estimate 18' Roadway truss + Girder design -

5 @ 178.5	450 tons @ 250^{00}	=	112500
piers -			44000
Deck - 19 - 60'	313 @ 210	=	65700
piers	17 @ 3600		61100
abutments	2 @ 4500		9000
Deck -			69300
			361600^{00} ÷ 1015 = 356^{00}/坪

16910
15650
1260

Girder spans. 21' Roadway -
approach girder spans same as for other layout
For river spans 10 @ 89.25' girders
Refer to pp 6.

stringers	70	70	Floor - HR etc 2400
Floor beam	95	95	
Lateral	50	50	
girders assumed	750	820	
	965	1035	

89.25
1.75
88.0

19600
48
250 @ 210

DL. 2400 + 965 = 3365 ÷ 2 = say 1680
LL 1050

Moment = ⅛ × 2730 × 88² = 2,640,000
cone 3000 × $\frac{88}{2}$ = 132,000
2,772,000^{00}

web assumed 72" × 7/16 = 31.5 ⅛ web = 3.94
Depth say 5.9 stress = 470.000^{00} SR = 27.60"
3.9
23.70" -

2Ls 6 × 6 × ¾ = 16.88 13.88
2 cov. Pls. 14 × ½ = 14.00 12.00
30.88 25.88
30.88
31.50
93.26 @ 3.4 = 316# × 1.30 = 410# 2 @ 410 = 820

weight = 1035 × 89.25 = 92400 41.2 tons

10 spans @ 41.2 = 412 tons

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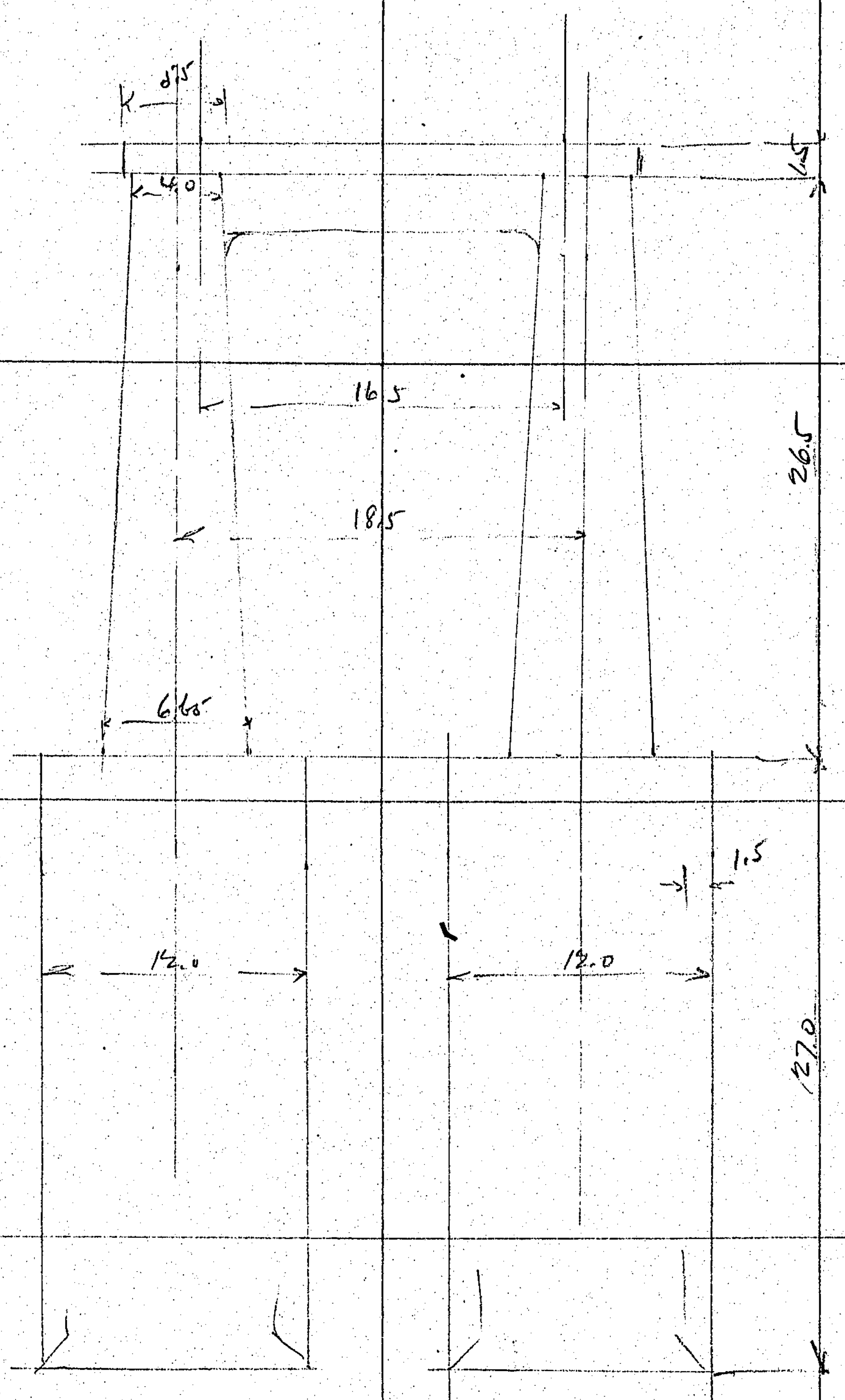
CALCULATIONS FOR

Estimate of Cost for Kasumi bashi for Okayama-Ken

Piers for Girder span for River channel-



Coping
dia 5.5 23.7
5.5 × 18.5 = 102.0
125.7 × $\frac{1.5}{216}$ = .87

Shaft 4' dia 12.6
6.65 34.7
47.3 ÷ 2 = 23.65
47.3 × $\frac{26.5}{216}$ = 5.8

.87
5.80
2.90
9.57

web. 1.5 × 13.5 × $\frac{26.5}{216}$ = 2.5
strut 2. × 3 × $\frac{14.5}{216}$ = .4
2.9 立坪

well -
12' dia - 113.0
9' 63.6
49.4

2 × 49.4 × $\frac{27}{216}$ = 12.3

filling 2 × $\frac{63.6 \times 9}{216}$ = 5.3 立坪

Sand fill 10.6

Excavation. 113.0 × $\frac{30}{216}$ = 16
2
32. tsubo

Concrete shaft 9.57
12.30
5.30
27.17 cubic tsubo

Conc. 27.17 @ 30.000 = 815.000
Sand 10.6 @ 21600 = 230.000
1,045.000
Superimposed load say 470.000
1,515 000

Unit bearing = $\frac{1515\,000}{226}$ = 6700
3.0 tons/□'
o.k.

Estimate of Cost of one pier
Concrete 27.17 @ 126.00 = 3420
Reinf. 600
Excavation 32 @ 40 = 1,280
form 300
5,600.00

10 @ 89.3
piers 11 @ 5600 = 61,600.00
~~steel~~ 412 tons @ 210 = 86500
158,100.00

12 @ 74.4
13 piers @ 5600 = 73,000
340 tons @ 210 = 71500
144500

12 spans 74.4' 850 × 74.4 = 63200 28.2 tons × 12 = 340 tons

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CALCULATIONS FOR

Estimate of Cost for Kasumibashi for Okayamaken

Final Estimate	Girder spans on River Channel - (21' Roadway)		
River span	12 spans @ 74.4 $^{28.2}$ =	340 @ 210^{00} =	71500
	13 piers @ 5600	=	73000
	19 - 60'	352 @ 210 =	74000
Approach	piers 17 @ 4000		68000
	abutments 2 @ 5000		10000
	Deck		76000
			372500^{00}
		(18' Roadway)	
River spans	12 spans @ 74.4 @ $^{26.0}$	312 @ 210 =	65500
	13 piers @ 5200	=	67600
	19 - 60	313 @ 210 =	65700
Approach	piers - 17 @ 3700	=	61100
	abutments 2 @ 4500	=	9000
	Deck say		69300
			338200^{00}