昭和二年一月成

京都府淀大橋及御幸橋

設計計算書及

材料調書

JIUN MASUDA
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SEIYU BLDG, TOKIO

MADE BY J. Masuda DATE 15-12-11 FILE NO.
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CALCULATIONS FOR

Design of Goko-Bashi for Kioto-Prefecture.

There are two crossings for this bridge one over Shinujigawa and ~~the other~~ Kizugawa. The length of former 266 meter with

2 End spans @ 17.363 (~~55.775'~~) 18.125 = 35.488
8 Center spans @ 29.0 (95.146') = 232.0
267.488 meters between end bearings.

The length of latter crossing is 353 meter with

2 End spans @ 17.363 (~~55.775~~) = 34.726
11 Center span @ 29.0 (95.146) = 319.0
353.726 meters between end bearings

Embankment between two crossings 56.5 meters between end bearings of two crossings.

Then the total length of the bridge

Shinujigawa Crossings 267.488
Embankment (56.5 about) out
Kizugawa Crossing 353.726
671.214 meter between end bearings of girders.
2040.0' about

The both end spans of Kizugawa crossing will have 22.536' overhanging arm, and one of end span of Shinujigawa crossing same as above, and the other end span will have the same overhanging span from next span 55.775' - 22.536 = 23.239' suspended span. There are two kinds of center span; one for continuous throughout between piers 95.146' and the other 22.536' overhanging arm from both ends and 50.072' suspended span at center.

Total width of bridge 11 meter between inside lines of hand rails

The pavement on roadway will be soliditit pavement 7.3 meters at center and 1.85 meter each on both sides will be asphalt block pavement 1½" thick with ½" mortar cushion underneath of asphalt block.

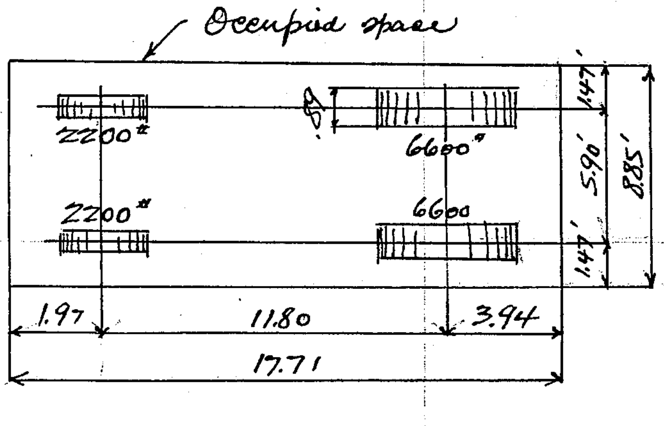
The handrail will be of ornamental casting with concrete post and rails.

Assumed Loadings

Uniform load on Roadway $w = \frac{100.000}{170 + l} = 500\ kg/m^2$ or say $100\#/\square'$

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

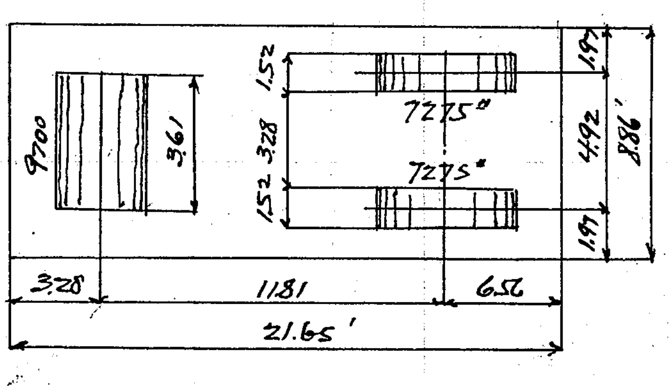
Motor truck Loading 8 ton motor truck (17600#).



4 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 uniform load.

11 ton Roadroller (24200#).



One roadroller on one span without impact

Impact for motor truck loading

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

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

no impact for roadroller and uniform load.

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CALCULATIONS FOR

Design of Gokō-Bashi for Kioto-Prefecture.

Allowable working strength
structural steel or reinforcing bars

Tension	1200. kg/cm^2 or	17000 #/□"
Extreme fibre stress	1200 "	17000 "
Shear of web gross section	900 "	12800 "

Compression member

$1500(1-0.0055\ l/r)$ or not over $1.000\ kg/cm^2$

where l = length of member in cm
r = least radius of gyration in cm^2

Equivalent formula in inch-lbs

$21300(1-0.0055\ l/r)$ or not over 14000 #/□".

Compression flange of plate girder

$1200(1-0.012\ l/b) \leq 1.100\ kg/cm^2$

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

Equivalent formula in inch lbs.

$17000(1-0.012\ l/b) \leq 15400$ #/□".

Shearing on shop driven rivets (machine driven).	12000 #/□"
Shear on field driven rivets and turned bolts (machine driven)	10.000 "
Extreme fibre stress of pin.	24.000 "
Bearing on shop rivet	24.000 "
Bearing on ~~shop~~ field rivets and turned bolts	20.000 "
Bearing on pin.	24.000

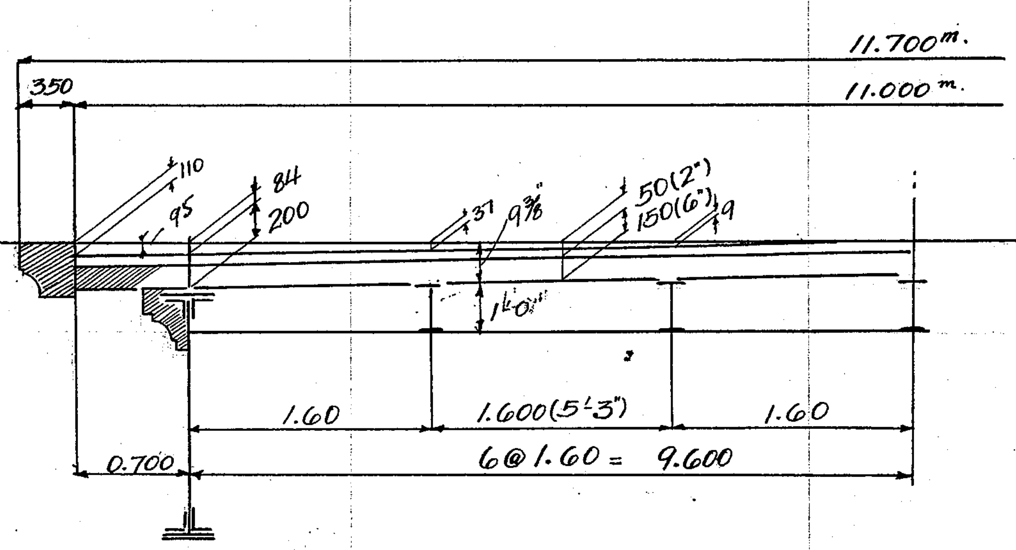
Expansion roller $45d\ kg/cm^2$ where d = diameter of roller in centimeter
In inch-lbs. $610d$ # per lin. inch where d = dia. of roller in inches.
Bearing on masonry 1:2:4 Concrete $45\ kg/cm^2$ = 640 #/□"

Strength of Concrete 1:2:4 mixture.

Compressive fibre stress	45 kg/cm^2	640 #/□"
Shear for plain Concrete	4 "	58
Punching shear	9 "	128
Bond stress of plain bar	6 "	85
Bond stress of deformed bar	9 "	128
Shear for reinforced Concrete.	9 "	128

Considering wind and temperature stresses in addition to dead live and impact stresses, ~~the~~ the allowable working strength shall be increased 25% and proportioned the parts. In considering earthquake, the working strength shall be increased 80% and proportioned the parts.
Assumed acceleration of Earthquake = 2500 mm/sec^2

Cross section of bridge as shown on sketch below.



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CALCULATIONS FOR

Design of Gokō-Bashi for Kioto Prefecture

Design of floor slab. span length 5'3" center to center of stringers

Dead Load

2" Solidetit or Asphaltic Concrete 25#

6" Reinforced Concrete slab. 75#

100 #/□'

Dead Load moment = $\frac{1}{10} \times 100 \times 5.25^2$ = 276 '#

Dead Load shear = $\frac{1}{2} \times 100 \times 5.25$ = 263 #

Live Load

motor truck rear wheel 6600 Front wheel $\frac{1}{3} \times 8580$ = 2860 #

Impact 30% 1980 #

8580

Distribution of wheel Concentration

Contact space between wheel and pavement assumed 20 cm = 0.66'

$2 \times .17$ = 0.34

Longitudinal distribution 1.00' = a

Transverse distribution b = 0.89 + 0.34 = 1.23'

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

$= \frac{2}{3}(5.25 + 1.23) + 1.00 = 5.32'$

Load per ft strip = 8580 ÷ 5.32 = 1615 #

2.63 1615 5.25

moment due to wheel Concentration = 807×2.63 = 2120 '#

For Continuity of slab. m = 0.8×2120 = 1700 '#

1615 1615 2.94 2.31 5.25

Max End shear $1615 \times \frac{2.31}{5.25}$ = 710

1615

2325 # per ft strip

Summary for moments and shears

Dead Load	276	263
Live Load	1700	2325
	1976 '#	2588 #

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

$d = \sqrt{\frac{1976}{102}} = 4.4"$

Use 6" slab with insulation 1¼" at bottom

Effective depth = 4.75"

Steel area required = $\frac{1976 \times 12}{\frac{7}{8} \times 4.75 \times 17000}$ = 0.336 □" per ft ½"φ bars 6" center = 0.393 □"

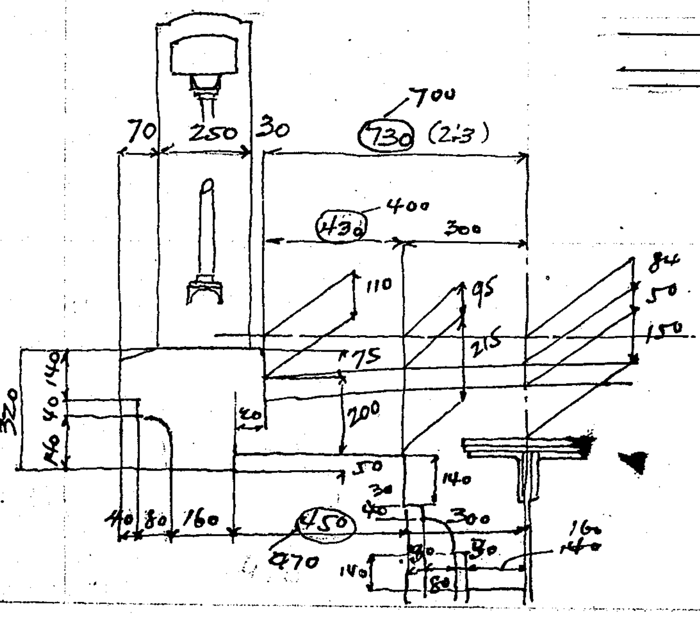
Unit shearing stress = $\frac{2588}{\frac{7}{8} \times 4.75 \times 12}$ = 52 #/□" ok

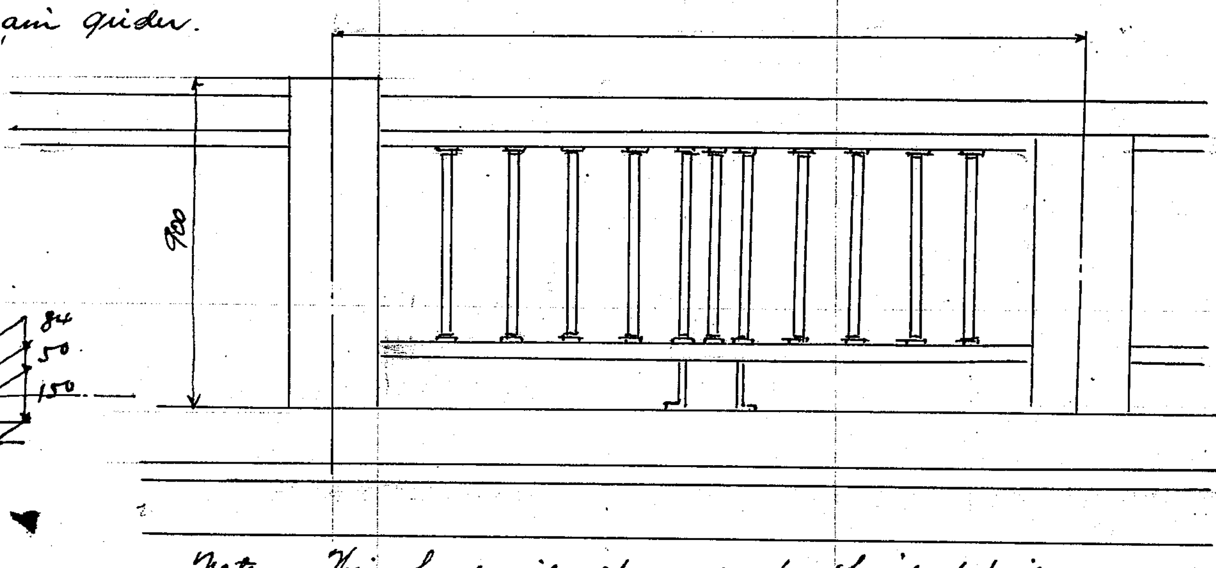
Bond stress = $\frac{2588}{\frac{7}{8} \times 4.75 \times 3.14}$ = 194 #/□" for 2-½"φ bars for 1' strip.

Use extra reinforcing bars 2-3/8"φ circumference = 2.35" on I beam stringers.

Unit bond stress = $\frac{2588}{\frac{7}{8} \times 4.75 \times 5.49}$ = 113 #/□" ok.

Overhanging slab beyond main girder.





Note. This hand rail changed for final detail see drawing

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CALCULATIONS FOR

Design of Goko-Bashi for Kioto Prefecture.

Approximate weights of Handrail.

Panel length - 11.89' ½ panel = 5.95'

Post.	.8 × .8 × 2.95 @ 150#	= 302#		302.0
Intermediate post	.55 × .33 = .182			
and top rail.	.25 × .12 = .030			
	0.212 × 11.09	=	2.350	
	.55 × .80 × 2.35	=	1.03	
			3.38 @ 150 =	507.0#

Structural steel in Handrail.

4 [3" 1½"	@ 5.27 × 5.25	=	111#
22 - 1¼"φ pipe	@ 2.99 × 2.70	=	178
44 Collars	@ 1.00	=	44
2 cast iron chairs.	@ 15	=	30
			363.0
			1172.0#

1172 ÷ 11.89' = 99# call this 100# per lin ft of span.

Approximate of Coping.

1.15 × .46	=	.53
.13 × 1.02	=	.13
.46 × .52	=	.24
.40 × .07	=	.03
		.93 call this 1.00 @ 150 = 150# per lin ft.

slab and pavement .69 @ 150 = 103# per square ft.

Concrete under slab at side of main girder.

.46 × .98	=	.45
.10 × .88	=	.09
.26 × .66	=	.17
.46 × .52	=	.24
		.95 call this 1.00 @ 150 = 150# per lin. ft.

Dead Load Moment at face of Concrete facing of main girder.

Depth of slab. 165 = 6½"

Dead Load moment	Handrail -	100 × 1.92 =	192
	Coping	150 × 1.92 say =	288
	slab + pavement	103 × 1.4 × .70 =	101
		395#	581 1#

Live Load   Rear wheel motor truck with impact 8580#

This load assumed distributed over 2' × 2' or 8580 ÷ 4 = 2140# per sq ft.

moment about face of Concrete facing $2140 \times \frac{1.4^2}{2} = 2100$ 1#

Shear = 2140 × 1.4 = 3000#

Summary for moments and shears

	Moment	Shear
Dead Load	581	395
Live Load	2100	3000
	2681 1#	3395 1#

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

$d = \sqrt{\frac{2681}{102}} = 5.1"$

Use slab 6½" with insulation 1¼" at bottom

Effective depth 5.25"

Steel Area reqd $= \frac{2681 \times 12}{7/8 \times 5.25 \times 17000} = .410$ □" Use main reinf. ½"φ - 6" centres = .39
, with ⅜" - 6" " .22
.61 □"

Unit shear $= \frac{3395}{7/8 \times 12 \times 5.25} = 61.5$ #/□" say ok

Bond stress $= \frac{3395}{7/8 \times 5.25 \times 5.49} = 135$ #/□" say ok

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CALCULATIONS FOR

Design of Goko-Bashi for Kioto-Prefecture.

Design of Longitudinal Stringer span length 11.89' spacing 5'-3"
Dead Load

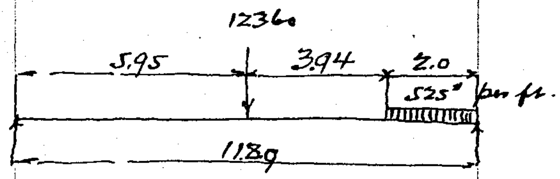
2" pavement 25
6" Reinforced Concrete Slab. 75
100#/□' × 5.25 = 525#

Stringer assumed 35
560# per lin. ft.

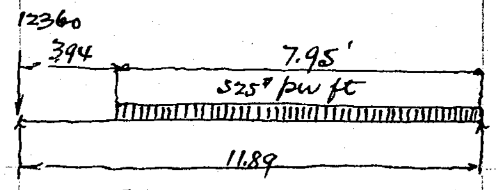
Dead Load moment = 1/8 · 560 · 11.89² = 9900'#
Dead Load shear = 1/2 · 560 · 11.89 = 3320#

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

max load on stringer 8580 · 2.31/5.25 = 3780
8580
12360#



Uniform live load = 100 × 5.25 = 525# per lin. ft.
Total unif. load 2 × 525 = 1050#
Reaction = 1050 · 1/11.89 = 88#



moment due to motor truck loading 6180 · 5.95 = 36800
" " " Uniform load 88 · 5.95 = 520
37320'#

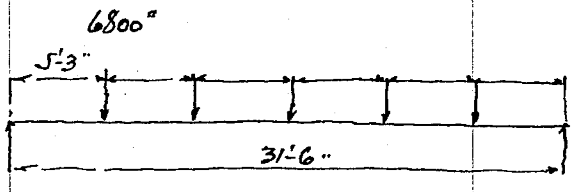
max End shear
Uniform load 525 × (7.95 × 3.97)/11.89 = 1400
12360
13760#

Summary for moments and shears

	moment	shear
Dead Load	9900	3320
Live Load	37320	13760
	47220'#	17080#

Try 12" × 5" @ 31.99# I S. modulus = 36.686
Unit stress = (47220 × 12)/36.686 = 15450#/□"

Intermediate floor Beam span length 31'-6" spacing 11.89'
Dead Load flooring including stringer 560# × 11.89 = 6650
allowing filler on stringer say 150
6800#



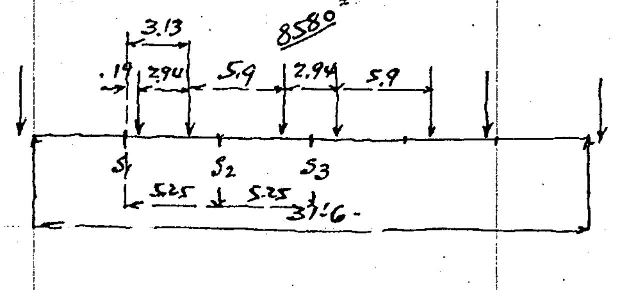
Dead Load moment 17000 × 15.75 = 268000
3 × 6800 × 5.25 = 107000
161000'#

Beam assumed 130# 1/8 · 130 · 31.5² = 16200
177200'#

R = 6800 × 2.5 = 17000#

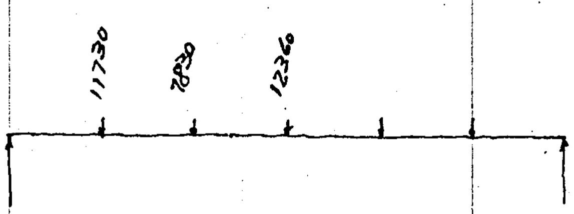
End shear From Conc = 17000
Beam 130 × 31.5/2 = 2050
19050#

Live Load. motor truck on Roadway impact say 30%.
Rear wheel Concentration 8580#
Front wheel concentration 2860#
On S_3. 8580 · 3.78/5.25 = 6180 × 2 = 12360#



On S_2 8580 − 6180 = 2400
8580 · 3.13/5.25 = 5120
8580 · .19/5.25 = 310
7830#

On S_1 8580 − 5120 = 3460
8580 − 310 = 8270
11730#



R = 11730 + 7830 + 6180 = 25740

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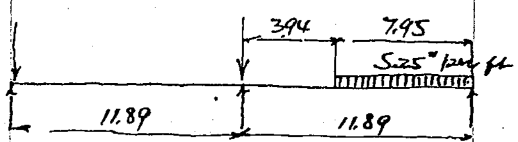
Design of Gokō-Bashi for Kioto-Prefecture.

Live Load Uniform live load 100#/□' 5.25 × 100 = 525# per ft.

Reaction = 525 × $\frac{7.95 \times 3.97}{11.89}$ = 1400#.



Reaction = 1400 × 2.5 = 3500

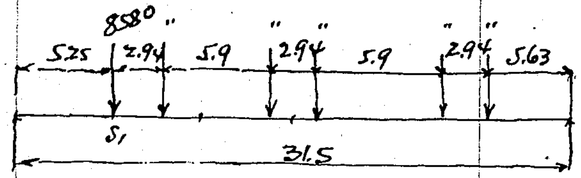
moment due to motor truck
25740 × 15.75 = 405000
7830 × ½ 5.25 = 41100
11730 × 2 × 5.25 = 123000

- 164100
240900#

Moment due to unif. load. 3500 × 15.75 = 55100
3 × 1400 × 5.25 = − 22000

33100
274000#

Max End shear

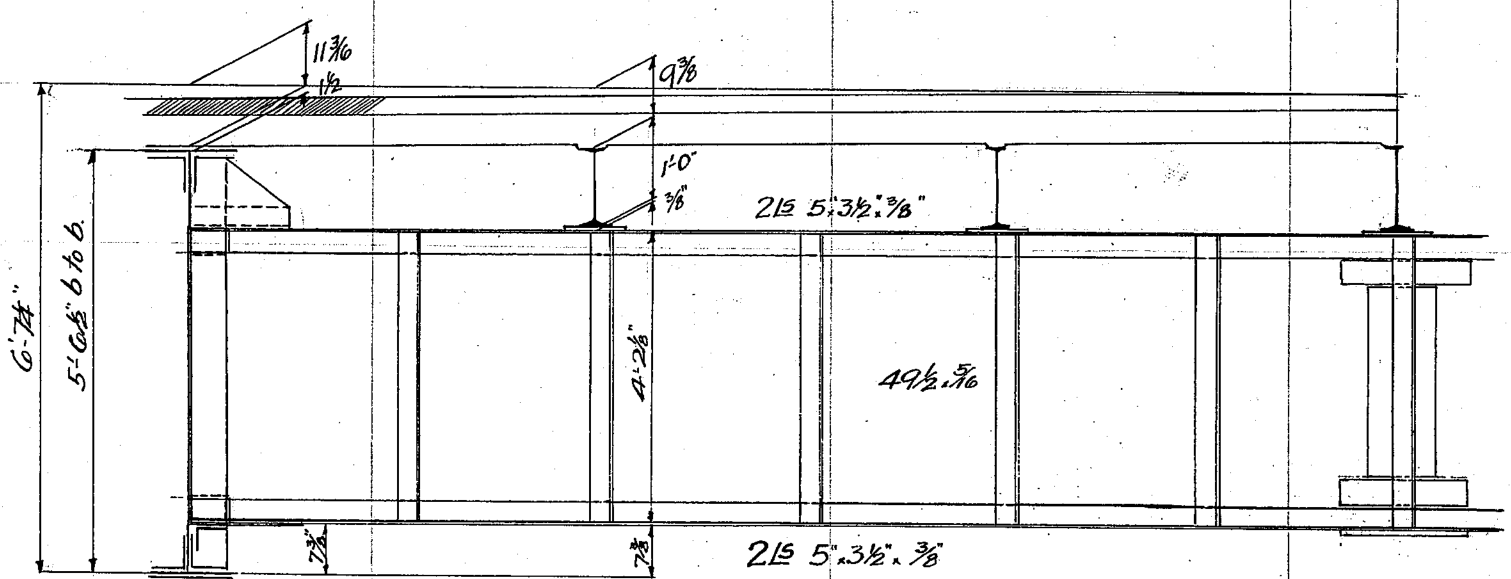


3 × 8580 × $\frac{15.94}{31.5}$ = 13050
2
26100#

Uniform load. 3500
29600#

Summary for moments and shear

	moment	shear
Dead Load	177200	19050
Live Load	274000	29600
	451200#	48650#



Depth of girder 4'-2⅞" web 49½" . 5/16" = 15.45 □" ⅛ web = 1.93 □"
Effective depth 4.17 − .14 = 4.03 flange stress = 451200 ÷ 4.03 = 112000#
Section req'd = 112000 ÷ 17000 = 6.59
− 1.93
4.66 □" net.

Try 2Ls 5 × 3½ . 3/8 = 6.10 □" gross. or − 1.33 = 4.77 □" net deducting of − 7/8" 3/4"φ holes.

Unit shear gross section = $\frac{48650}{15.45}$ = 3150 #/□" ok

moment of inertia. (gross section).
1 web pl. 49½ . 5/16 = 15.45   3160
4Ls 5 × 3½ . 3/8 = 12.20 × 24.14² + 12.8 = 7103
10263 (in)⁴

Unit flange stress = $\frac{451200 \times 12 \times 25}{10263}$ = 13200 #/□".

allowable unit stress of Comp. flange = 17000 (1 − 0.012 × $\frac{63}{10.3}$) = 15750 #/□".
Max = 15400 #/□" for max.

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CALCULATIONS FOR

Design of Gokō-Bashi for Kioto Prefecture.

Approximate weight of stringer.

For one panel	5 Is 12" × 5" @ 31.99#	× 11.85'	=	1895
web splice	10 Pls. 9" × 3/8 @ 11.48	× 0'-8"	=	77
Bed plate	5 Pls. 7 × 3/8 @ 8.93	× 0'-10½"	=	39
	Rivet heads and variation - 2%.			40
				2051#

3/4" Rivets

2051 ÷ 11.89 = 173# per lin. ft of span

Approximate weight of Intermediate floor beam

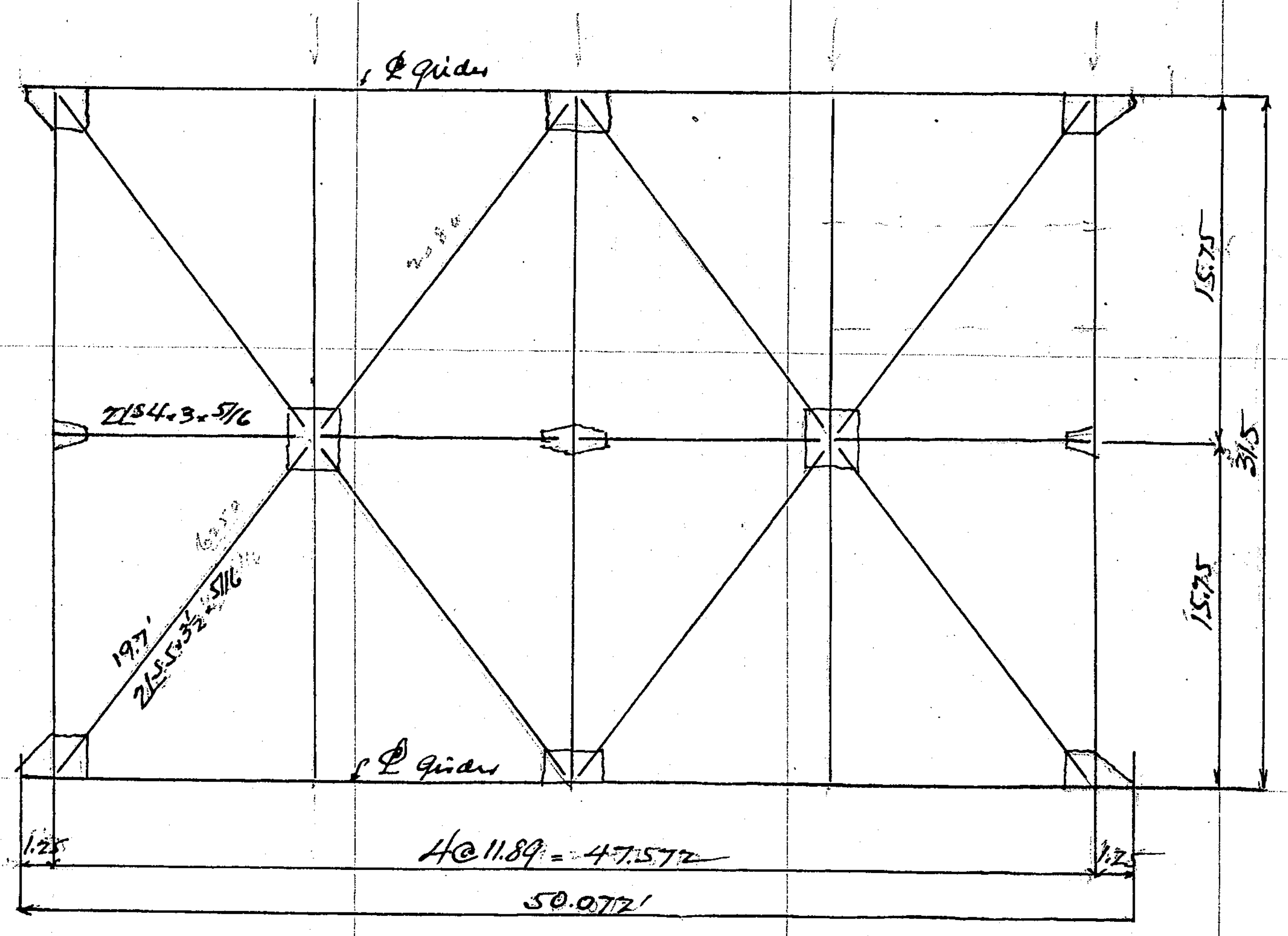
web	1 Pl.	49½ × 5/16	@ 52.6#	× 31.5	=	1660
flange	4 Ls	5 × 3½ × 3/8	@ 10.4#	× 31.5	=	1310
stiffs (interm.)	22 Ls	3½ × 3½ × 5/16	@ 7.2	× 4.15	=	657
End stiffs	2 Ls	5 × 3½ × 3/8	@ 10.4	× 4.15	=	86
filler	2 Pls.	5 × 3/8	@ 6.38	× 3.57	=	46
center splice flange	4 Pls.	6 × 3/8	@ 7.65	× 2.5	=	76
" " web	2 Pls.	10½ × 3/8	@ 13.39	× 2.57	=	69
Rivet heads and variation say 3½ %					=	137
						4142#

3/4" Rivets.

4142 ÷ 31.5 = 131.5# per lin. ft. of girder.
or 4142 ÷ 11.89 = 348# per lin ft of span

Section for End floor Beam assumed same as for intermediate floor beam

Bottom Lateral Bracings.



Wind pressure for loaded and unloaded portion assumed carried down to bottom panel point and then transferred to support.

For loaded chord	268
For unloaded chord	132
	400# per lin. ft.

panel concentration = 400 × 11.89 = 4750#

Reaction say 4750
2375
7125#

$\sec\theta = \frac{19.7}{15.75} = 1.25$

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CALCULATIONS FOR

Design of Gokō-Bashi for Kioto-Prefecture

Stress in end panel	7125 × 1.25 =	8900#
Stress in 2nd panel	2375 × 1.25 =	2970#

For Earthquake of 2500 mm/sec²
Approximate weight of floor.

coping	2 @ 150	300
concrete facing	2 @ 150	300
overhanging slab and pavement	2 - 103 × 1.40 =	290
floor slab + pavement	100 × 33.5 =	3350
misc concrete		60
Handrails	2 @ 100	200
		4500# per lin. ft of span
Structural Steel in span assumed		1181
		5681#

stringers	173
Beam	348
lateral	60
girders say	600
	1181

For lateral force 5681 × .25 = 1420# per lin ft.
panel concentration = 1420 × 11.89 = 16900#
Reaction = 16900 × 1.5 = 25400#
stress in end panel = 25400 × 1.25 = 31800#
Stress in second panel. as moving load.
Reaction 16900 × 3/4 = 12700#
stress = 12700 × 1.25 = 15900#
Unit stress to tension 17000 × ~~1.8~~ 1.6 = ~~30600~~ 27200 #/□"
section required = 31800 ÷ ~~30600~~ 27200 = ~~1.04~~ 1.17 □" net
= 15900 ÷ ~~30600~~ 27200 = ~~0.52~~ 0.58 □" net.

Unsupported length 19.7 × 12 = 237"
radius of gyration required for tension member = 237 ÷ 200 = 1.19
Try 2Ls 5 × 3½ × 5/16 = 5.12 □" or 4.04
r = 1.33 and 1.61
no of rivet required 31800 ÷ ~~7950~~ 7075 = ~~4.0~~ 4.5 ¾" Rivet = 4420 × ~~1.8~~ 1.6 = ~~7950~~ 7075#
Use 6 Rivets for connection
Use longitudinal strut to brace the bottom flange of floor beam
2Ls 4 × 3 × 5/16 Riveted back to back of Ls
Unsupported length = 11.89 × 12 = 143" radius of gyration = 1.16
l/r = 123.
For gusset pls 5/16"
Approximate weight of bottom lateral bracing.

16 Ls 5 × 3½ × 5/16	@ 8.7	× 18.0	=	2510
8 Ls 4 × 3 × 5/16	@ 7.2	× 11.0	=	720
Connection pls. for girder 3/8" thick	6 @ 58#		=	348
" " for floor beam 5/16	2 @ 90		=	180
do 5/16	3 @ 25# aver.		=	75
Rivet heads and variation - 3% about				117
				3950#

3950 ÷ 47.57 = 83# per lin. ft. of span

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Design of Gokō-Bashi for Kioto Prefecture.

Design of suspended main girder span span length 50.072' center to center of end bearings.

Dead Load.

Uniform load on main girder

Handrail	100
Coping	150
Concrete facing.	150
overhanging slab + pave	145
slab + pavement 100 × 3.62 =	362
	907# per lin ft. for one girder

Concentration from floor beam (floor slab and pavement).

100 × 5.25 = 525
misc say = 10
535# per lin ft.

Floor beam reaction = 535 × 2.5 = 1340.0#
For one panel = 1340 × 11.89 = 15900.# Intermediate panel.
For end panel = 1340 × ~~13.76~~ 7.82 = ~~18400~~ 10500

~~Structural Steel~~ in span

Stringers	173 × 11.89 =	2050
Intermediate floor beam	=	4142
Lateral Bracing	83 × 11.89 =	986

For intermediate panel point 7178# call this 7200#÷2 = 3600#

stringers	$173 \times (\frac{11.89}{2} + 1.87) =$	1350
floor beam say		4140
Lateral Bracing -		1493

For end panel point. — 5983# call this 6000#÷2 = 3000#

structural steel in main girder assumed 275# per lin ft of one girder.

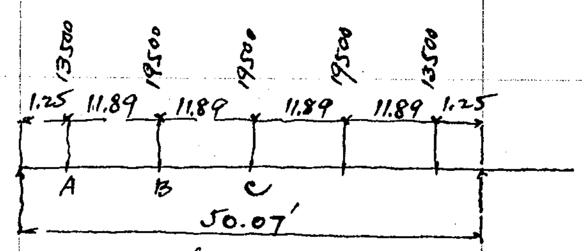
Summary for panel Concentration -

	Intermediate Panel Point	End panel Point
flooring	15900	10500
structural steel	3600	3000
	19500#	13500#

For Uniform load flooring &c 907
girder. 275
1182# per lin. ft of span.

Out to out of girder 51.312'

Dead Load moment.



Unif. load - 1182#
42750

moment at A. 42750 × 1.25 = 53500 1#

moment at B. 42750 × 13.14 = 563000
13500 × 11.89 = − 161000
402000 1#

moment at C 42750 × 25.03 = 1070.000
19500 × 11.89 = − 232.000
13500 × 2 × 11.89 = − 322.000
516000 1#

For Uniform load at C $\frac{1}{8} \cdot 1182 \times 50.07^2 =$ 370.000 1#

at B $\frac{1}{2} \times 1182 \times 13.14 \times 36.93 =$ 287.000

at A $\frac{1}{2} \times 1182 \times 1.25 \times 48.82 =$ 37.000

End shear
Unif. 1182 × 25.03 = 29600
Conc. 42750
72350#

Summary for dead Load moment

	A	B	C
Concentration	53500	402000	516000
Uniform load.	37000	287.000	370.000
	90500 1#	689.000 1#	886.000 1#

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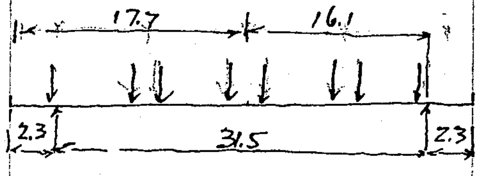
Design of Gokō-Bashi for Kioto-Prefecture

Live Load motor truck loading impact $= \frac{20}{60+\frac{50}{3.28}} = 26.6\%$.



Rear wheel Concentration 6600 #
Impact 26.6 % 1760
8360 #

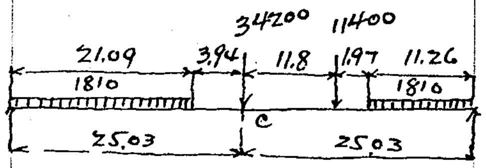
Front wheel Concentration = 8360 ÷ 3 = 2790 #

Rear wheels 2@ 8360 = 16720 #
For 4 motor trucks $4 \cdot 16720 \cdot \frac{16.1}{31.5} = 34200$ #

For Front wheels 34200 ÷ 3 = 11400 #

Uniform live load 100 #/□' × 33.8' = 3380 #
Reaction = 3380 · 16.9 ÷ 31.5 = 1810 # per lin. ft of span.

The above live load assumed on main girder without panel point.
max bending moment at center of span C.



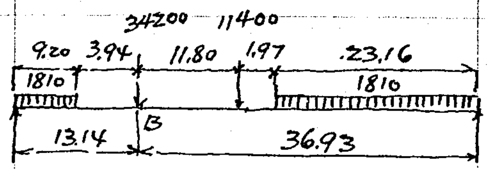
Uniform load - 1810 · 21.09 = 38200 #
1810 · 11.26 = 20400 #

Reaction 20400 · 5.63 ÷ 50.06 = 2300
38200 · 39.52 ÷ 50.06 = 30200
32500 #

moment due to motor truck. 17100 #
$11400 \cdot \frac{13.23}{50.06} =$ 3020
20120 · 25.03 = 503 000

moment due to uniform load
32500 · 25.03 = 813000
38200 · 14.53 = - 555000
258 000
761 000 '#

Bending moment at B.

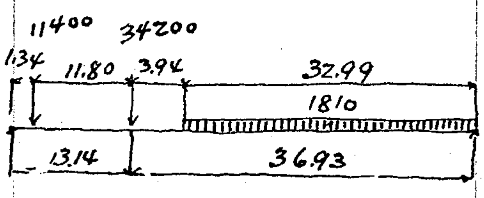


Uniform load 1810 · 9.20 = 16650
1810 · 23.16 = 41900

Reaction 16650 · 45.47 ÷ 50.07 = 15150
41900 · 11.58 ÷ 50.07 = 9700
24850 #

motor truck loading
34200 · 36.93 ÷ 50.07 = 25200
11400 · 25.13 ÷ 50.07 = 5720
30920 #

moment due to motor truck 30920 · 13.14 = 406000
moment due to unif. load 24850 · 13.14 = 326 000
16650 · 8.54 = - 142 000
184 000
590 000 '#



Uniform load 1810 · 32.99 = 59750
Reaction 59750 · 16.5 ÷ 50.07 = 19700 #

motor truck loading
34200 · 36.93 ÷ 50.07 = 25200
11400 · 48.73 ÷ 50.07 = 11100
36300

moment due to motor truck 36300 · 13.14 = 477000
11400 · 11.80 = 134500
342500 '#

Uniform live load 19700 · 13.14 = 259 000
601500 '#

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CALCULATIONS FOR

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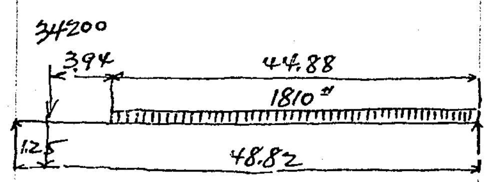
Moment at A.

Uniform load 1810 × 44.88 = 81300#
Reaction 81300 × 22.44 ÷ 50.07 = 36400#



motor truck R 34200 × 48.82 ÷ 50.07 = 33400

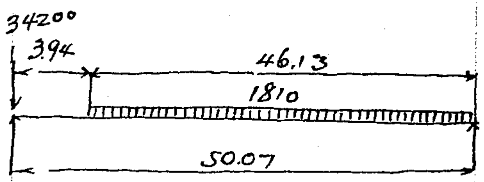
moment due to motor truck 33400 × 1.25 = 41800
Moment due to unif. load 36400 × 1.25 = 45500
87300#

Summary for Bending moment

	A	B	C
Dead Load	90500	689000	886000
Live Load	87300	601500	761000
	177800	1290500	1647000

Max End shear due to live load.

Uniform load 1810 × 46.13 = 83500
Reaction 83500 × 23.06 ÷ 50.07 = 38500



motor truck loading 34200
Live Load shear 72700#
Dead Load shear p9 72350
145050#

Section of main girder.
web assumed 66 × 3/8 = 24.75 □" 1/8 web = 3.09 □" 6 to b of Ls = 5'-6½"
flange section assumed

2Ls 6×6×½	=	11.50	× 1.68" =	19.30	- 9.50
1Pl 14×½	=	7.00	× .25" =	-1.75	- 6.00
		18.50 □"		17.55	15.50 □" net

Center of gravity = 17.55 ÷ 18.5 = .95"
2@ .95" = 1.9" or .16'

Effective depth = 5.54 - .16 = 5.38
flange stress = 1647000 ÷ 5.38 = 306.000#
SR = 306.000 ÷ 17000 = 18.00
3.09
14.91 □" net

Moment diagram and sketch of main girder as shown on p 12.
approximate weight of main girder.

web pl	1Pl.	66 × 3/8	@	84.2	×	50 av.	=	4210
flange Ls	4Ls	6×6×½	@	19.6	×	50 av.	=	3920
Cov. pl.	2Pls.	14×½	@	23.8	×	31.	=	1476
floor beam conn.	10Ls	5×3½×3/8	@	10.4	×	5.5	=	572
fillers	4Pls.	7½×½	@	12.75	×	4.5	=	229
Stiffs	16Ls	5×3½×3/8	@	10.4	×	5.5	=	915
center splice.	4Pls.	6×½	@	10.20	×	3.0 abt	=	123
" "	2Pls.	13½×½	@	22.95	×	3.5	=	161
angle splice	8Ls	6×6×½	@	19.6	×	3.0	=	471
at bearing.	8Ls	6×4×½	@	16.2	×	2.5	=	324
filler	4Pls.	36×½	@	61.2	×	4.5	=	1100
Bearing	4Ls	6×6×½	@	19.6	×	1.5	=	118
"	2Pls.	14×3/4	@	35.7	×	1.5	=	107
misc details say								300

Rivet heads + variation say 490
14516#

14516 ÷ 50. = 290# per lin ft of girder.
For two girders 580# per lin ft.

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CALCULATIONS FOR

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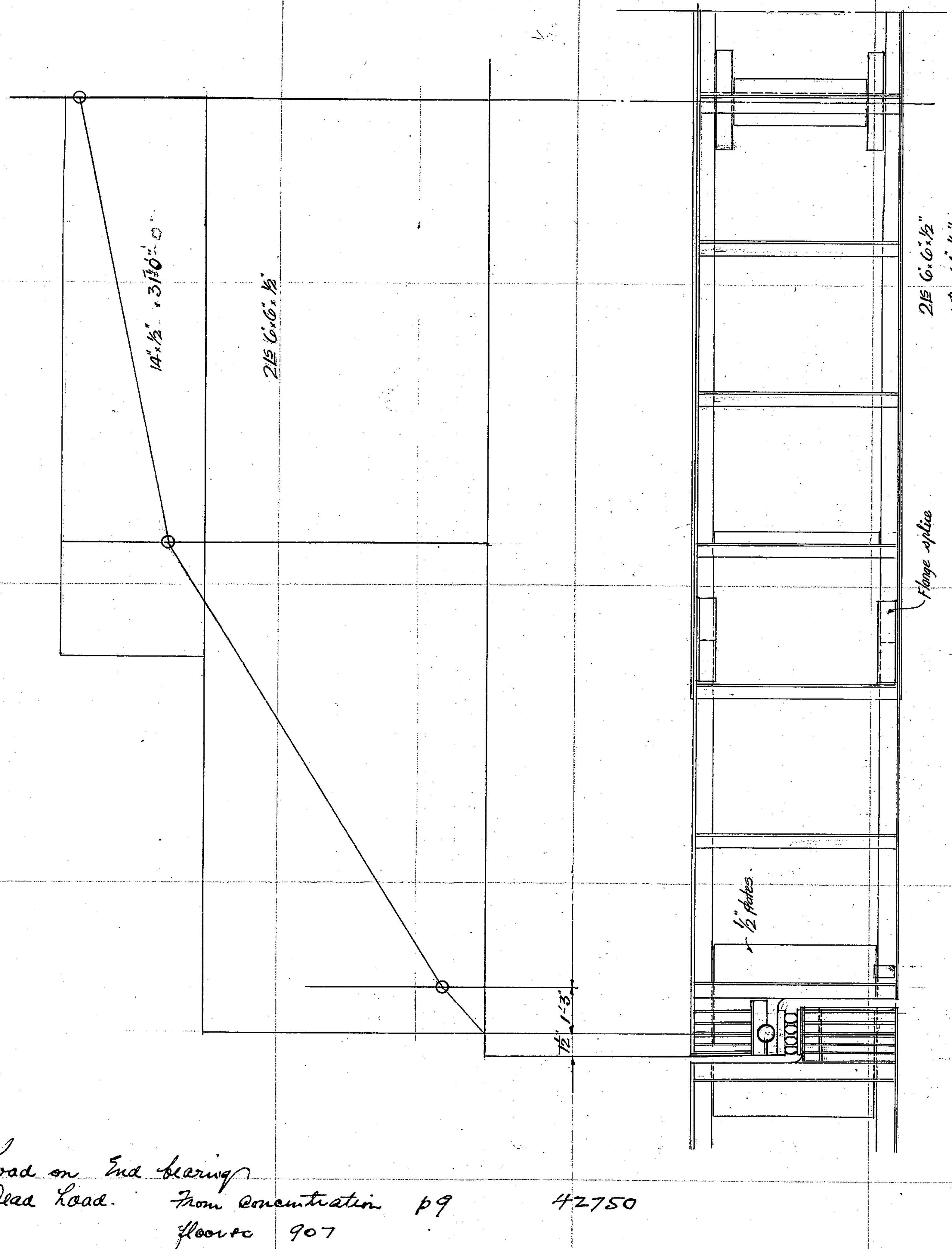


Load on End bearing

Dead Load. From concentration p9 42750
floor etc 907
girder 290
1197 × 25 = 30000

72750#
Live Load p11 72700#
Total load 145450 " call this 146.000# per girder.

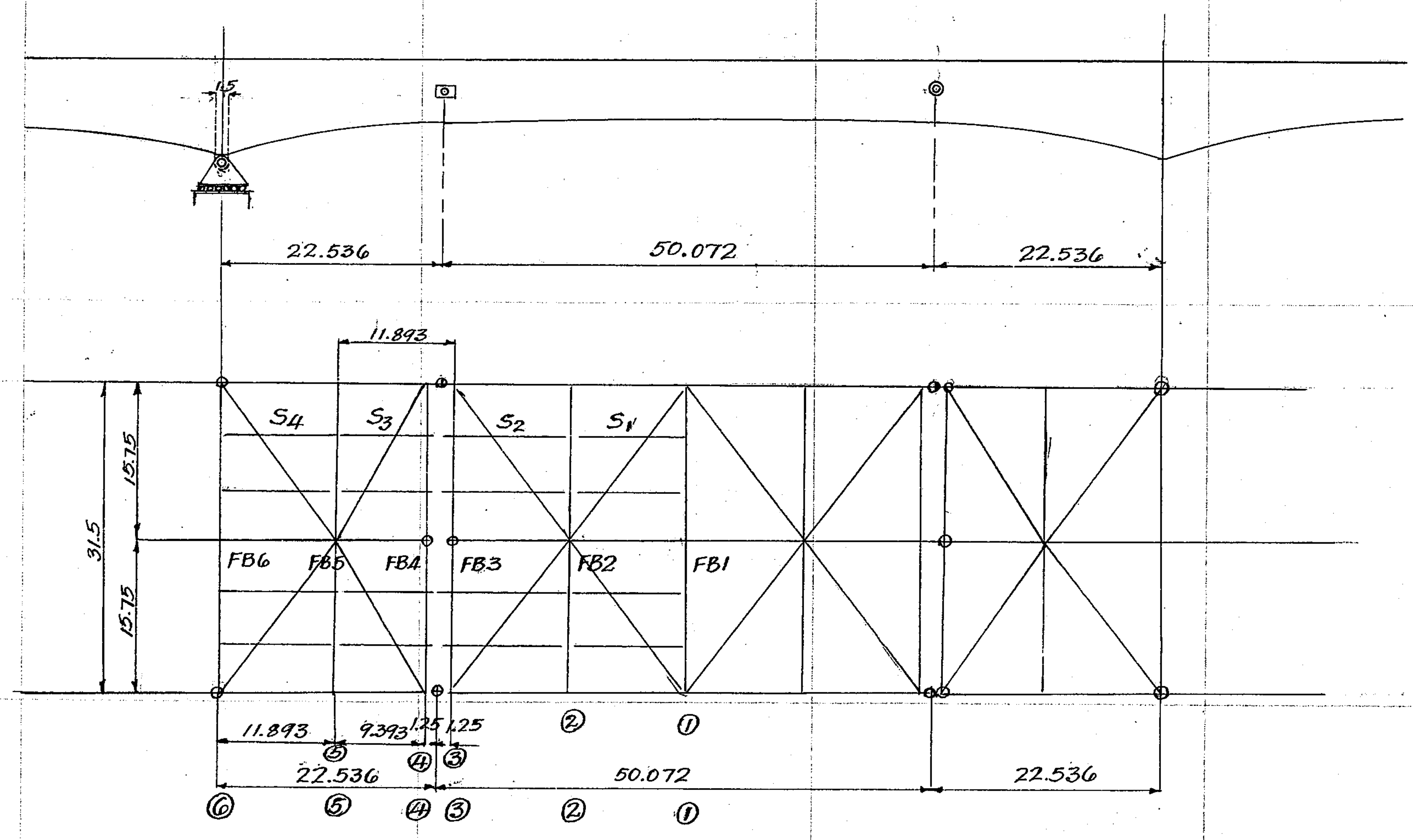
For roller bearing. use 6" Roller 6 × 610 = 3660# per lin. inch

Length of roller = $\frac{146.000}{3660}$ = 40" net

Using 3 rollers 40 ÷ 3 = 13½" net each roller

For 6½" roller 6.5 × 610 = 3970# 146.000 ÷ 3970 = 36.8"
For 3 rollers 36.8 ÷ 3 = 12¼" net.

Design of Goko-Bashi for Kioto Prefecture
Design of overhanging arm of main girder



Design of floor slab same as for suspended span see p3.
Design of stringers S3 and S4 same as for suspended span see p.5
Use 1 I 12"×5" @ 31.99#

Floor Beams FB 4-5-6 use same section as for standard intermediate floor beam
if required, use end bracket to brace compression flange of main girder.
depth of floor beam 4'-2 1/16" b to b Ls
1 web. 49 1/2" × 5/16"
flange 2 Ls 5×3 1/2 × 3/8"

Lateral Bracing.
Lateral bracing for Earthquake of 2500 mm/sec²
approximate weight of floor see p 8 4500# per lin ft
1600
6100

Structural steel in span
stringers 173
Floor beam 348
Lateral Bracing 83
Girders assumed 1000
1604 call this 1600

For lateral force = 6100 × .25 = 1525# per lin ft.
panel Concentration = 1525 × 11.89 = 18150#

length of lateral for panels 4 + 5 $\sqrt{9.39^2 + 15.75^2} = 18.35'$

$\sec\theta = \frac{18.35}{15.75} = 1.165$

For standard panel $\sec\theta = \frac{19.7}{15.75} = 1.25$

Lateral force assumed as shown on sketch in next page.

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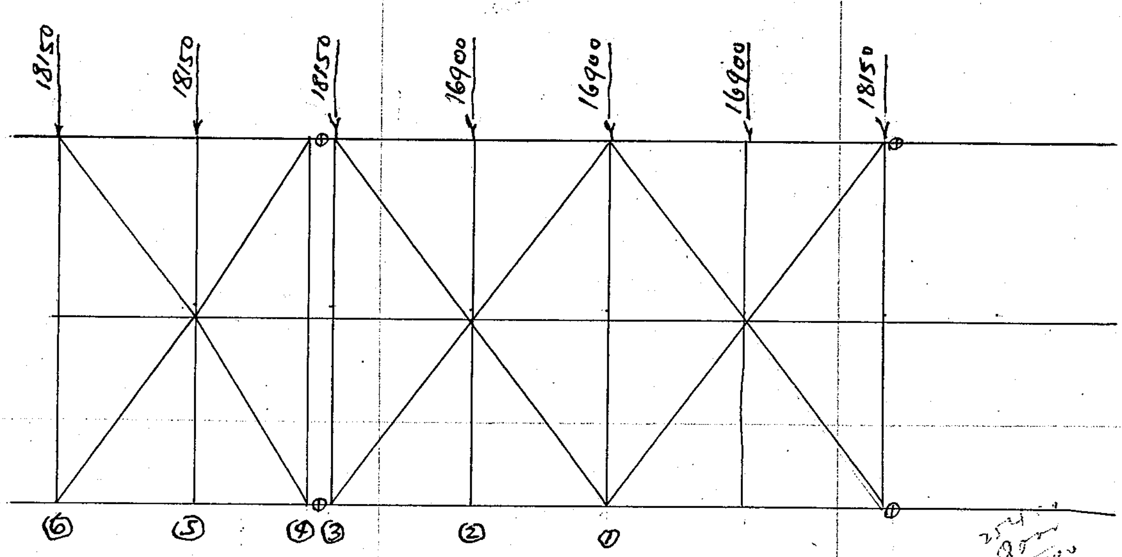
CALCULATIONS FOR

Design of Gokō-Bashi for Kioto-Prefecture.



Shear in panel 4+5 16900 × 1.5 = 25400 Stress in diagonals.
18150
43550 × 1.165 = 50700#
18150
Shear in panel 5+6 61700# × 1.25 = 77200#

				No of Rivet 3/4"φ	Use
Section required for 4-5	50700 ÷ ~~30600~~ 27200	=	~~1.65~~ 1.87 □" net	~~6.4~~ 7.2	8
" " " 5-6	77200 ÷ ~~30600~~ 27200	=	~~2.52~~ 2.84 □" net	~~9.7~~ 11.0	~~10~~ 12

Try 2Ls 5 × 3½ × 5/16 = 5.12 □" gross or 4.04 □" net.
radius of gyration = 1.33 and 1.61 riveted back to back of Ls
l/r = 18.35 × 12 ÷ 1.33 = 165
l/r = 19.7 × 12 ÷ 1.33 = 178
Use diagonal for tension stress only.

For longitudinal strut use 2Ls 4 × 3 × 5/16
Approximate weight of bottom lateral bracing assumed 83# per lin ft of span same as for suspended span shown on p 8

Design of main girder (Cantilever Portion).

Dead Load floor concentrated load panel point 4 1340 × 5.32 = 7120# on one girder
(see page 9.) 5 1340 × 10.65 = 14280 "
6 1340 × 11.89 = 15900 "

Structural steel concentrated load

Panel Point 4. stringers 173 × 5.32 = 920
Floor beam say 4140
Lateral Bracing 83 × 4.70 = 390
5450 ÷ 2 = 2725# on one girder

Panel Point 5 stringers 173 × 10.65 = 1840
Floor beam say 4500
Lateral Bracing 83 × 10.65 = 885
7225 ÷ 2 = say 3610# on one girder

Panel Point 6 stringers 173 × 11.89 = 2050
Floor beam say 4500
Lateral Bracing 83 × 11.89 = 986
7536 ÷ 2 = say 3770# on one girder

Uniform floor load on girder 907# per lin. ft of span
Dead Load of girder assumed 500
1407#

For panel point 4 1407 × 5.32 = 7480# add 900 = 8380#
5 1407 × 10.65 = 15000#
6 1407 × 11.89 = 16750#
} for panel load.

CALCULATIONS FOR

Design of Yoko-Bashi for Kioto-Prefecture

Summary for Panel load

	Panel Point 4	Panel Point 5	Panel Point 6
From floor load	7120	14280	15900
" structural steel.	2725	3610	3770
" Unif. floor load at girder	8380	15000	16750
	18225	32890	36420

Dead load reaction from suspended span = 72750# p12

Dead Load moment at 4 72750 · 1.25 = 91000 '#

Dead Load shear 72750 #

Dead Load moment at 5 18225 · 9.39 = 171.000

72750 · 10.65 = 775.000

90975 946.000 '#

Dead load shear 90.975 # call this 92 000 #

Dead Load moment at 6 18225 · 21.28 = 388 000

32890 · 11.89 = 391 000

72750 · 22.54 = 1640 000

123865 2419,000 '#

Dead Load shear 123865 # call this 125.000 #

Live Load impact $\frac{20}{60 + \frac{725}{32.8}} = .244$

motor truck loading rear wheel 6600

impact 24.4%. 1610

8210 # for axle load 16420 #

max reaction on girder = 4@16420 · $\frac{16.1}{31.5}$ = 33600 #

For front wheels. 11200 #

Uniform live load 100 #/□' max reaction = 1810 # per lin ft on girder.

Live Load reaction from suspended span referred to p.11

Uniform live load reaction = 1810 · $\frac{46.13 \cdot 23.06}{50.07}$ = 38500

Rear wheel concentration 33600

Total reaction 72100 #

Live Load moment at 4 72100 · 1.25 = 90100

" " " at 5 72100 · 10.65 = 768 000

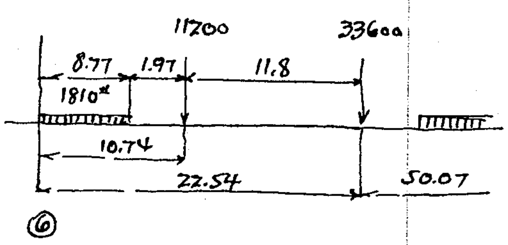
Live Load moment at 6. Uniform load 1810 · 8.77 = 15900 #

moment 72100 · 22.54 = 1.620.000

11200 · 10.74 = 120.000

15900 · 4.39 = 70.000

99200 1810.000 '#

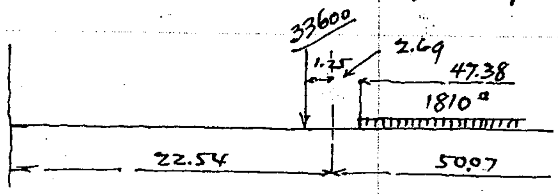


max live load shear at panel point 4.

Uniform load 1810 · $\frac{47.38 \times 23.69}{50.07}$ = 40500 #

motor truck loading 33600

74100 #



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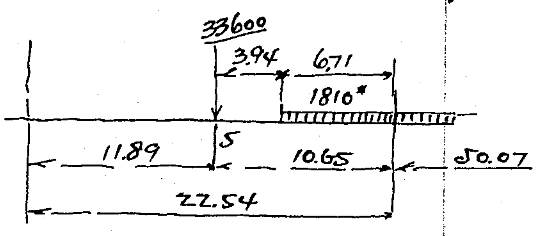
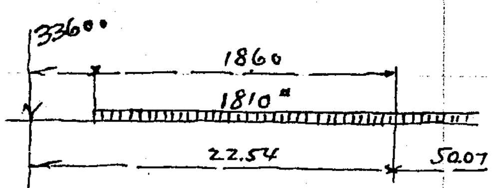
Max live load shear for Panel Point 5



Uniform load 1810 × 25.03 = 45200
1810 × 6.71 = 12150
57350
motor truck loading 33600
90950#

Max live load shear for Panel Point 6.

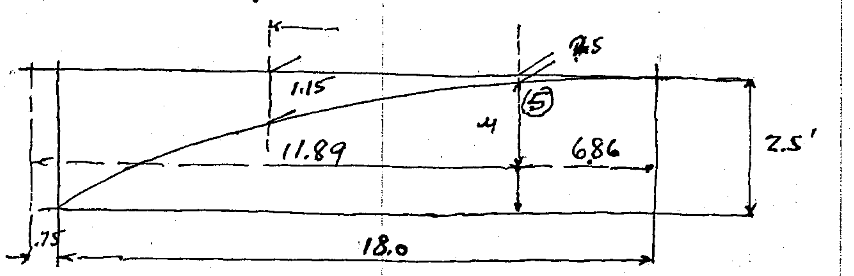


Uniform load 1810 × 25.03 = 45200
1810 × 18.60 = 33600
78800
motor truck loading 33600
112400#

Summary for moments and shears.

	M. 6	M5	M4	shear 6	shear 5	shear 4
Dead load	2419.000	946000	91000	125000	92000	72750
Live load	1810.000	768000	90100	112400	90950	74100
	4229.000#	1714000#	181100#	237400#	182950#	146850#

Shape of bottom flange.



Radius $r = \frac{4 \times 2.5^2 + 36^2}{8 \times 2.5} = 66.05'$

$y = 2.5 - 66.05 + \sqrt{66.05^2 - 6.86^2} =$
65.80
.45

$y = 66.05 + \sqrt{66.05^2 - 12.81^2}$
64.90
1.15

Depth of girder at 5 5'-6½" + 3" = 5'-9½" b to b of Ls

Depth of girder at mid point of 5 & 6 5'-6½" + 1'-2" = 6'-8½" b to b of Ls.

Depth of girder at panel point 6 8'-0½" b to b of Ls

Section of girder at panel point 6. moment = 4229.000 ᶦ#
Depth of girder 8'-0½ web = 96 × ½ = 48.0 □" ⅛ web = 6.0 □"
assumed flange section

2Ls 6 × 6 × 5/8	= 14.22 □" × 1.73 =	24.6	11.72	13.88
1 Pl. 14 × 5/8	= 8.75 } 17.5 × .62	10.9	7.50	6.
1 Pl. 14 × 5/8	= 8.75 }		7.50	6.
	31.72 □"	13.7	26.72 □" net	25.88

13.7 ÷ 31.72 = .43 2 @ .43 = .86" = say .10'

Effective depth 8.04 - .10 = 7.94'
flange stress = 4229000 ÷ 7.94' = 533.000
Sk = 533.000 ÷ 17000 = 31.40
6.00
25.40 □" net. for tension flange.

Unit Compression Stress = 17000 (1 - 0.012 l/b) = 14900 #/□"
l say 145" b = 14"

Gross flange area 31.72 533.000 ÷ 37.72 = 14150 #/□" Ok
⅛ web 6.00
37.72

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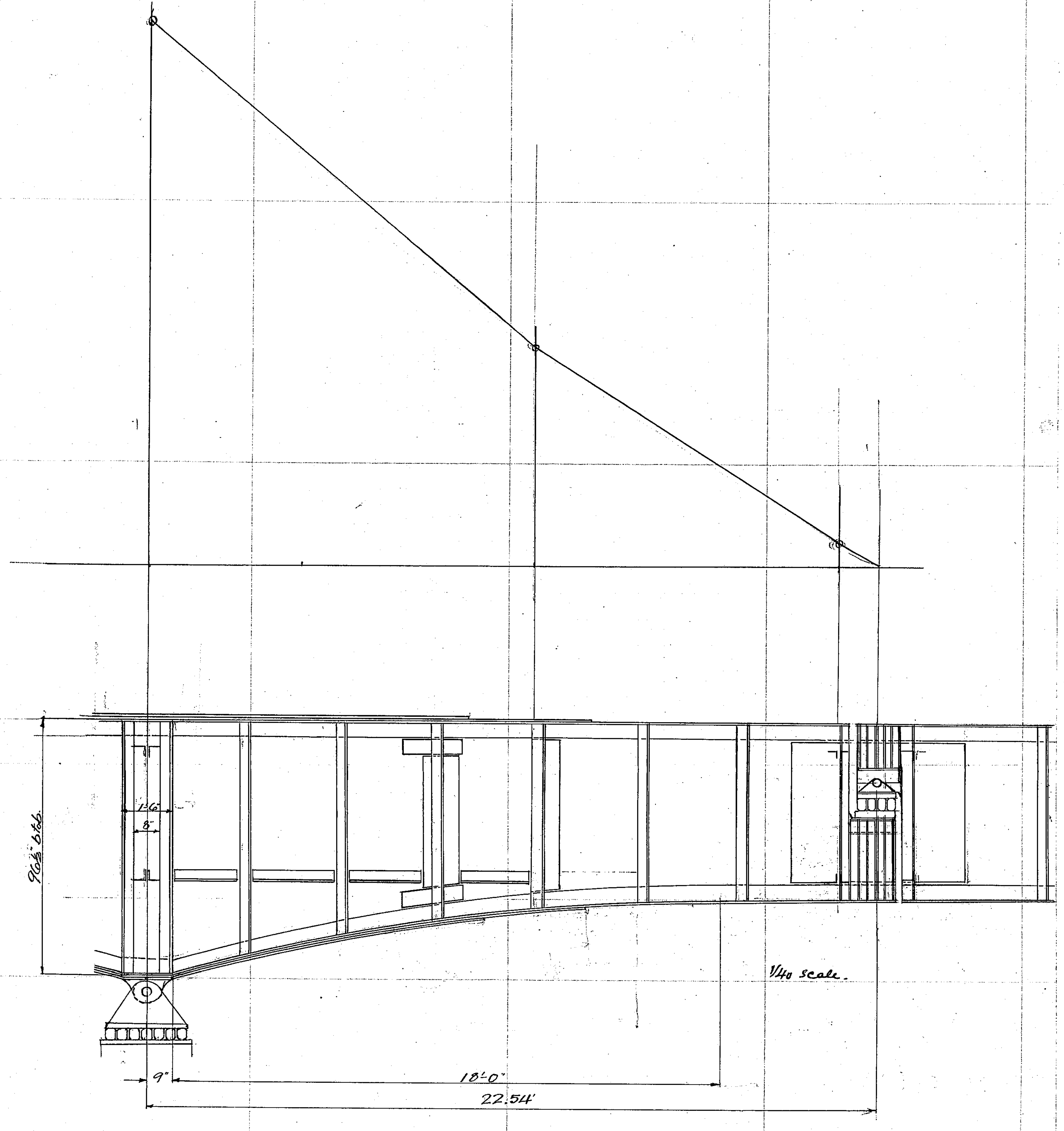
Sketch of main girder (Cantilever Portion).



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Design of Yokō-Bashi for Kioto-Prefecture.

Section of main girder at mid point of 6+5 m = 2972.000#
depth of girder 6'-8½ web = 80" × ½ = 40 □" ⅛ web = 5.0 □"
Section assumed

2Ls 6.6.⅝ = 14.22	× 1.73 =	24.6	11.72	13.88
1Pl. 14 × ⅝ = 8.75	× .62 =	−5.4	7.50	
22.97		19.2	19.22	

19.2 ÷ 22.97 = .83   2 @ .83 = 1.66" or .14'
Effective depth 6.71 − .14 = 6.57'   flange stress = 2972.000 ÷ 6.57 = 452000#
Section required = 452000 ÷ 17.000 = 26.60
5.00
21.60 □" net reqd.

Try section same as panel 6. Use 2Ls 6.6.⅝ = 14.22 − 11.72
2 Pls. 14 × ⅝ = 17.50   15.00
31.72   26.72 □" net.
Effective depth. 6.71 − .10 (say) = 6.61
flange stress = 2972.000 ÷ 6.61 = 450.000
Section reqd = 450.000 ÷ 17000 = 26.45
5.00
21.450 □" net.

Section of girder at panel point 5. m = 1714000#
Depth of girder 5'-9½ b to b of Ls web = 69 × ½ = 34.5 ⅛ web = 4.31 □"
Effective depth 5.79 − .14 = 5.65 flange stress = 1714.000 ÷ 5.65 = 304.000#
Section required = 304.000 ÷ 17000 = 17.86
4.31
13.550 □" net

Use 2Ls 6.6.~~⅝~~ ½ = ~~14.22~~ 11.50 − ~~11.72~~ 9.50
1Pl 14 × ⅝ = 8.75 − 7.50
~~22.97~~ 20.25   ~~19.22~~ 17.00 □" net   21.89

Approximate weight of main girder (Cantilever portion only)

web.	1Pl.	96 × ½	@ 163.2 ×	9.00 =	1470
"		75 × ½	@ 127.5 ×	13.54 =	1730
flanges.	4Ls	6.6.⅝	@ 24.2 ×	9.00 =	870
"	4Ls	6.6 × ½	@ 19.6 ×	13.54 =	1060
Cover pls	2Pls.	14 × ⅝	@ 29.75 ×	13.50 =	803
"	2Pls.	14 × ⅝	@ 29.75 ×	8.00 =	476
End stiffs	4Ls	5 × 4 × ½	@ 14.5 ×	8.00 =	464
fills	2Pls.	9½ × ⅝	@ 19.66 ×	8.00 =	314
Int. stiffs	2Ls	5 × 3½ × ⅜	@ 10.4 ×	7.2 =	150
& fills	2Ls	5 × 3½ × ⅜	@ 10.4 ×	6.6 =	137
	2Ls	5 × 3½ × ⅜	@ 10.4 ×	6.2 =	129
	2Ls	5 × 4 × ⅜	@ 11.0 ×	5.8 =	128
	2 fills	8½ × ½	@ 14.45 ×	4.8 =	139
	2Ls	5 × 3½ × ⅜	@ 10.4 ×	5.6 =	117
	2Ls	5 × 3½ × ⅜	@ 10.4 ×	5.5 =	115
	2Ls	5 × 4 × ⅜	@ 11.0 ×	5.5 =	121
	2 fills	36 × ½	@ 54.4 ×	4.5 =	490
shelf.	2Ls	6.6 × ½	@ 19.6 ×	1.5 =	59
stiffs	8Ls	5 × 4 × ½	@ 14.5 ×	2.5 =	290
web splice	4Pls	6 × ⅝	@ 12.75 ×	3.5 =	178
"	2Pls.	12½ × ⅝	@ 26.56 ×	4.2 =	223
	misc details say				300
	Rivet heads say				300
					9163#

9163 ÷ 22.54 = 407# per lin ft   call this 410# per ft.

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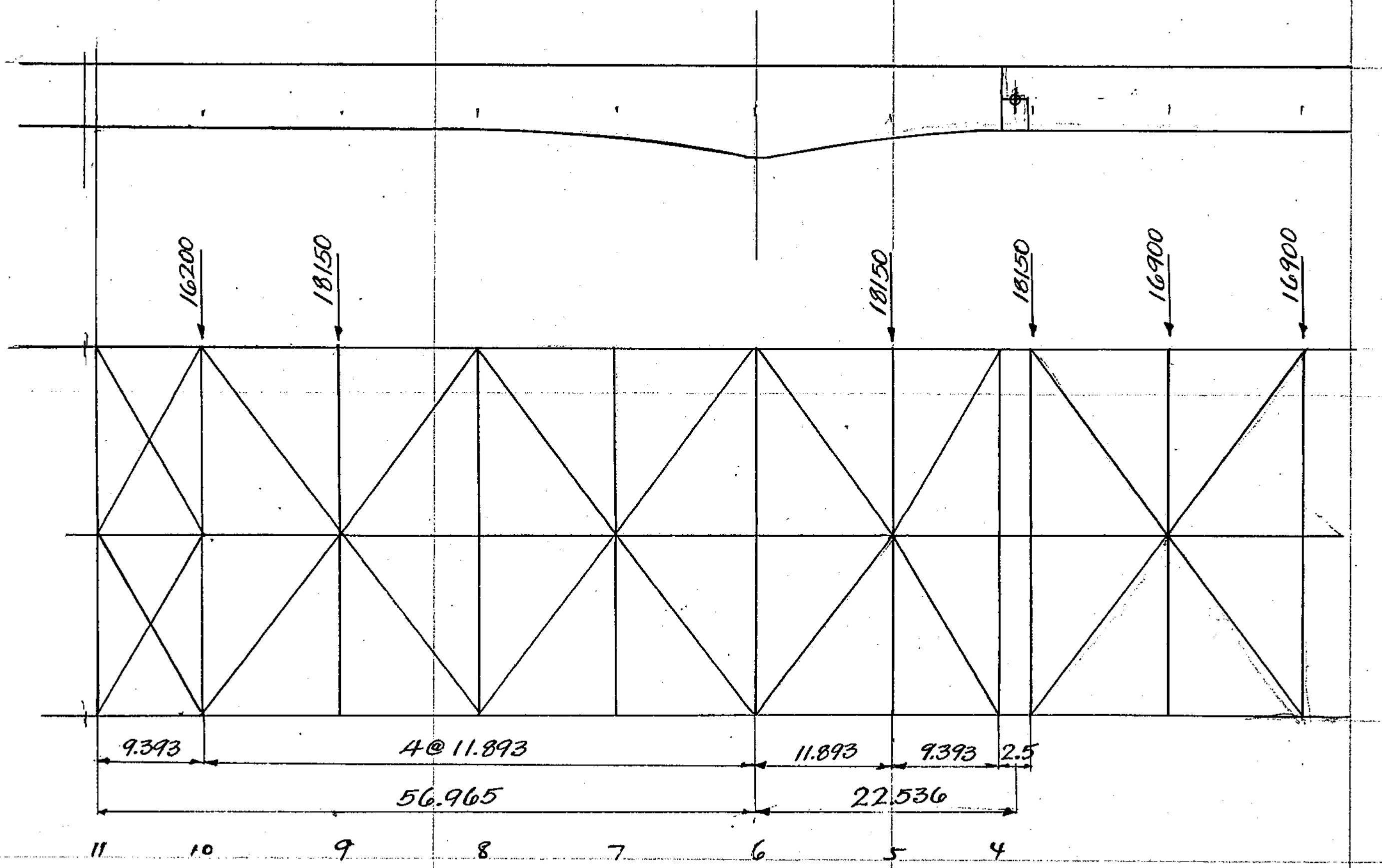

Design of floor slab, stringers and floor beams same as for suspended span

Lateral Bracing

Lateral force due to Earthquake of 2500 mm/sec²

Dead Load per lin. ft = 6100 Lateral force = 6100 · 0.25 = 1525# per lin ft.

6-7-8-9	panel Concentration	=	1525	×	11.89	=	18150#
10-	"	" =	1525	×	10.65	=	16200#
11		=	1525	×	6.95	=	10600#

negative reaction due to cantilever arm

Lateral force 16900 × 1.5 = 25400
18150
43550

moment at 6 43550 · 22.536 = 980.000
18150 · 11.893 = 216.000
1196.000 #

Reaction at 11 = 1196.000 ÷ 56.96 = 21000#

					stress				3/4 Rivet	No.
Shear in panel	11-10	36300	× 1.165	=	42300	÷ 27200	=	1.56 net	6.0	
	10-9	-21000	× 1.25	=	26200	÷ 27200	=	0.96	3.6	6
	9-8	-21000	× 1.25	=	26200	÷ 27200	=	0.96	3.6	6
	8-7	-41000	× 1.25	=	51200	÷ 27200	=	1.88 net	7.2	8
	7-6	-55500	× 1.25	=	69300	÷ 27200	=	2.55	9.8	10

Use 2∟s 5·3½·5/16 = 5.120□" gross or 4.04□" net for diagonal

strut at center line of bridge 2∟s 4·3·5/16

For end panel use double bracing of 2∟s 5·3½·5/16 with 6 rivet connection each

approximate weight of bottom lateral Bracing assumed 83# per lin. ft.

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CALCULATIONS FOR

Design of Gokō-Bashi for Kioto Prefecture

Dead Load stresses of main girder

panel concentration - panel point 7-8-9 36420#

Panel Point 11. Panel point 10

floor load concentration 1340 × 5.70 = 7650 1340 × 10.65 = 14300

stringers 173 × 5.70 = 987 173 × 10.65 = 1840

floor beam say 4140 floor beam 4140

Lateral Bracing 83 × 5.70 = 475 (say) 83 × 10.65 = 885

5602 ÷ 2 = 2800 6865 ÷ 2 = 3430 (say)

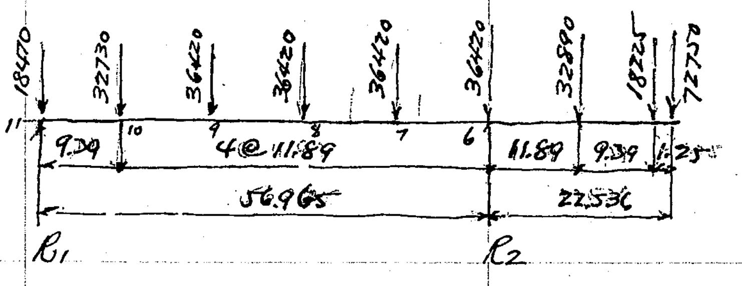
Unif. load floor 907 1407 × 10.65 = 15000

Dead Load girder 500 32730#

1407 × 5.7 = 8020

18470#

Dead Load moment



Dead Load moment at 6

18225 × 21.28 = 388 000

32890 × 11.89 = 391 000

72750 × 2254 = 1640 000

123865 2419 000 1#

R_1 = 2419 000 ÷ 56.965 = -42400#

R_1 +72900

30500#

R_1 36420 × $\frac{6 \times 11.89}{56.97}$ = 45500

32730 × $\frac{4 \times 11.89}{56.97}$ = 27400

72900#

moment at 10 30500 × 9.39 = 286000 1#

moment at 9 30500 × 21.28 = 650.000

- 32730 × 11.89 = -389.000

261000

moment at 8 30500 × 33.17 = 1010.000

- 32730 × 2 × 11.89 = - 778.000

- 36420 × 11.89 = - 434.000

- 202000

moment at 7 30500 × 45.06 = 1375000

- 32730 × 3 × 11.89 = -1170000

- 36420 × 3 × 11.89 = -1300.000

- 1095000

moment at 6. - 2419 000

Load at R_1 30500

18470

48970# or say 49000#

Load at R_2 from cantilever 123865

" simple span 105500

2419000 ÷ 56965 = 42400

271765

call this 280.000#

Live Load Coefficient for impact assumed same as shown on page 15 24.4%

motor truck rear wheel 8210# with impact

Front wheel ⅓ of 8210#

Uniform live load - 100#/□' For load on one girder see page 15

Max live load moment at 6 1.810.000 1# negative Reaction R_1 = 1.810 000 ÷ 56.96 = 31800#

negative moment at 10 31800 × 9.39 = 299000 1#

9 × 21.28 = 677000 "

8 × 33.17 = 1055000 "

7 × 45.06 = 1430 000 "

6 × 56.96 = 1810.000 "

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Design of Gokō-Bashi for Kioto Prefecture.

Positive live load moment of anchor span

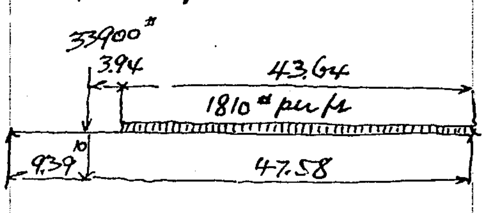
motor truck loading span length 56.965' impact $= \dfrac{20}{60 + \frac{56.97}{3.28}} = .259$

Rear wheel concentration 6600
Impact 25.9% 1710
8310# 2@ 8310 = 16620#

max reaction on girder see p10 $4 \times 16620 \times \frac{16.1}{31.5} = 33900$#
Front wheel 11300#

Uniform live load 100#/□' Reaction on girder = 1810# per lin. ft of girder.

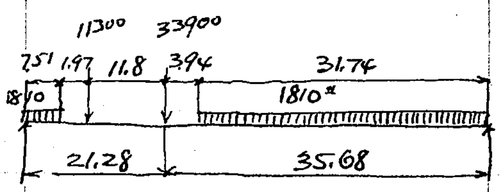
At panel point 10



Uniform load 1810 × 43.64 = 79000
Reaction = 79000 × 21.82 ÷ 56.97 = 30200# 30200
motor truck loading $R = 33900 \times \frac{47.58}{56.97}$ = 28300
58500

Moment = 58500 × 9.39 = 548000'#

At panel point 9



Uniform load - 1810 × 7.51 = 13550
1810 × 31.74 = 57400
70950#

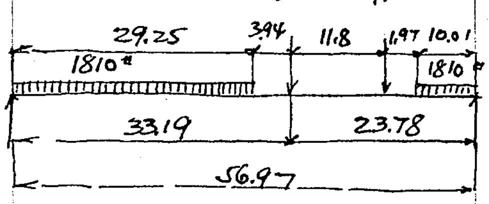
Reaction 13550 × 53.22 ÷ 56.97 = 12650
57400 × 15.87 ÷ 56.97 = 16000
28650

motor truck rear 33900 × 35.68 ÷ 56.97 = 21300
front 11300 × 47.48 ÷ 56.97 = 9430
30730

Moment due to motor truck 30730 × 21.28 = 655000
11300 × 11.80 = − 134000
521000

Uniform load 28650 × 21.28 = 610.000
12650 × 17.53 = − 222.000
388000
909000'#

At panel Point 8



Uniform load. 1810 × 10.01 = 18100
1810 × 29.25 = 53000

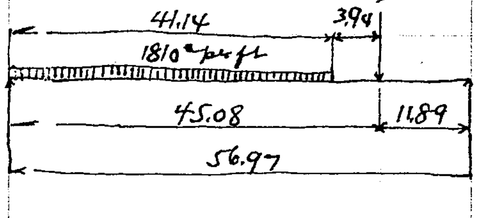
Reaction 18100 × 51.97 ÷ 56.97 = 16500
53000 × 14.62 ÷ 56.97 = 13600
71100 30100#

motor truck 33900 × 33.19 ÷ 56.97 = 19700
11300 × 44.99 ÷ 56.97 = 8900
28600#

Moment due to motor truck 28600 × 23.78 = 680000
11300 × 11.8 = − 134000
546000

Uniform load 30100 × 23.78 = 716000
18100 × 18.78 = − 340000
376000
922000'#

At panel Point 7



Unif. load 1810 × 41.14 = 74500
Unif. R = 74500 × 20.57 ÷ 56.97 = 26800
truck 33900 × 45.08 ÷ 56.97 = 26800
53600#

moment = 53600 × 11.89 = 670000'#

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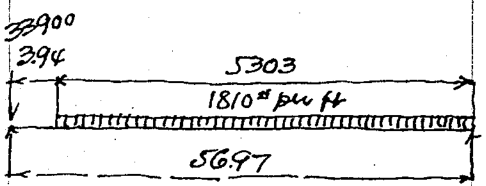
max End shear at 11



Uniform load $1810 \times \frac{53.03^2}{2 \times 56.97} =$ = 44600
motor truck 33900
78500#

max shear at 9
Uniform load 16000
motor truck 30730
46730
− 11300
35430

max shear at 10
Unif. load 30200
motor truck 28300
58500#

max shear at 8
Uniform load 13600
motor truck 28600
11300
17300
30900#

max shear at 7
Uniform load 26800
motor truck 26800
53600#

at 6 − 78500

Dead Load shear see p20.

	11-10	10-9	9-8	8-7	7-6
Shear as simple beam	+ 72900	40170	3750	− 32670	− 69090
Due to cantilever	− 42400	− 42400	− 42400	− 42400	− 42400
	+ 30500	− 2230	− 38750	− 75070	− 111490

Live Load shear

	11-10	10-9	9-8	8-7	7-6
As simple beam	78500	58500	35430	− 53600	− 78500
Due to cantilever	− 31800	− 31800	− 31800	− 31800	− 31800
max	+ 78500	+ 58500	− 31800	− 85400	− 110300

Summary for shears

	11-10	10-9	9-8	8-7	7-6
Dead Load	+ 30500	− 2230	− 38750	− 75070	− 111490
Live Load	+ 78500	+ 58500	− 31800	− 85400	− 110300
	+ 109000	+ 56270	− 70550	− 160470	− 221790

Summary for moments

	10	9	8	7	6
Dead Load	+ 286000	+ 261000	− 202000	− 1095000	− 2419000
LL neg	− 299000	− 677000	− 1055000	− 1430000	− 1810000
	− 13000	− 416000	− 1257000	− 2525000	− 4229000
Dead Load	+ 286000	+ 261000	− 202000	− 1095000	− 2419000
LL pos.	+ 548000	+ 909000	+ 922000	+ 670000	0
	+ 834000	+ 1170000	+ 720000	− 425000	− 2419000

Section of girder at 6 moment = − 4229000 1#
Depth of girder 8'-0½" b + ab of 13 web 96 × ½ = 48.0 1/8 web = 6.0
flange section 2∠s 6 × 6 × 5/8 = 14.22 − 11.72
1 Pl. 14 × 5/8 = 8.75 7.50
1 Pl. 14 × 5/8 = 8.75 7.50
31.72 □" 26.72 □" net

Effective depth = 7.94' flange stress = 4229000 ÷ 7.94 = 533000#
section reqd = 533000 ÷ 17000 = 31.40
6.00
25.40 □" net.

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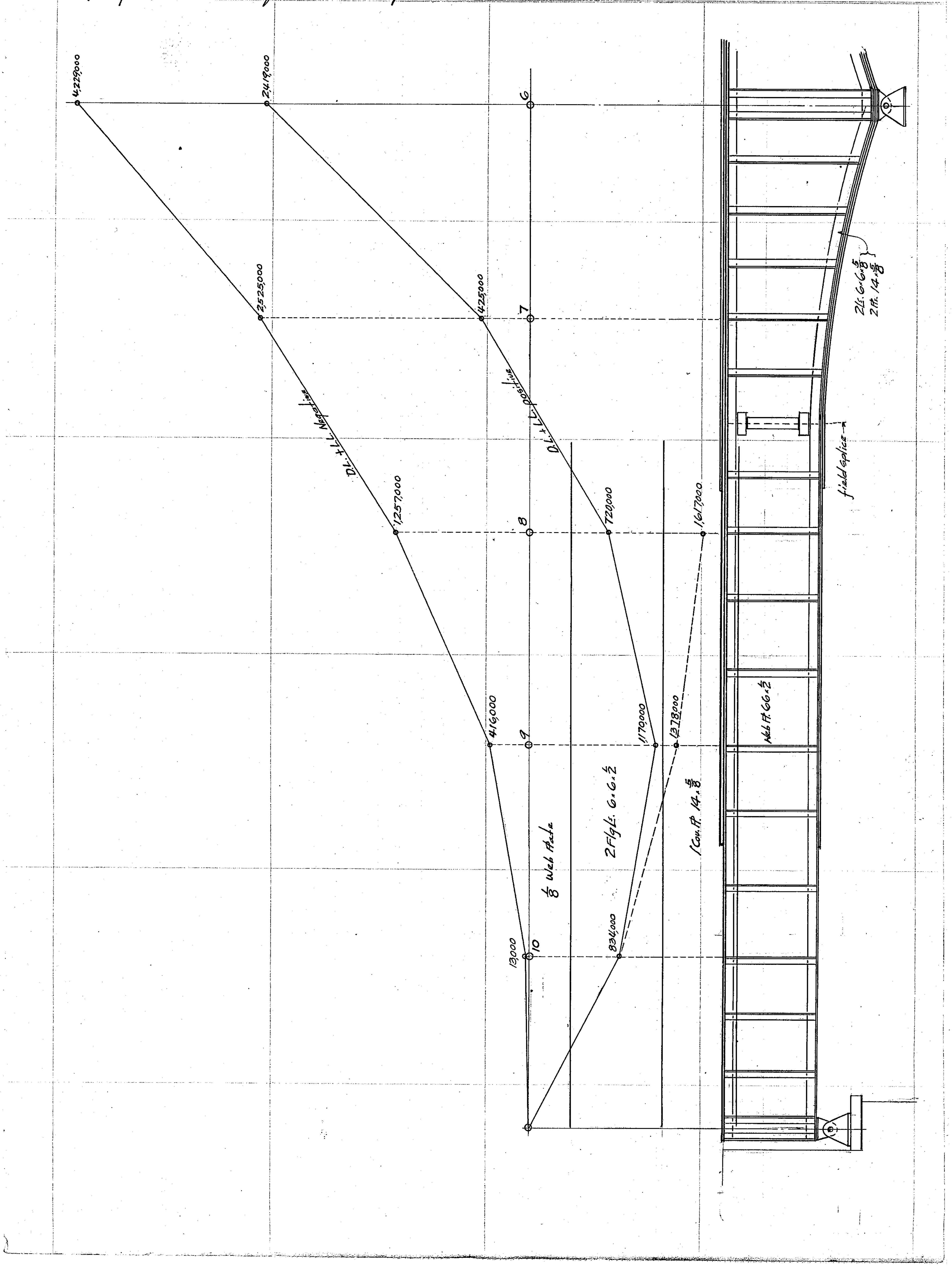

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CALCULATIONS FOR

Design of Goko-Bashi for Kioto-Prefecture.

Unit Compressive stress = 17000 (1 - 0.012 l/b) = 14900 #/□"
Gross area of flange = 31.72
1/8 web - 6.00
37.72
Unit stress = 533000 ÷ 37.72 = 14150 #/□"

mid point of 6 to 7 moment - 4229 000
- 2525 000
- 6754 000 ÷ 2 = 3377 000 "#

flange section same as above.
Depth of girder 6'-8½" b to b of Ls web = 80 × ½ = 40.0 □" 1/8 web = 5.0 □"
Effective depth 6.71 - .10 say = 6.61' flange stress = 3377000 ÷ 6.61 = 511 000 #
Section req'd = 511 000 ÷ 17000 = 30.00
5.00
25.00 □" net

Unit Compressive stress = 14900 #/□"
Gross area of flange = 31.72
1/8 web - 5.00
36.72 □"
Unit Stress = 511,000 ÷ 36.72 = 13900 #/□"

Section at 7 m = 2.525.000 "#
Depth of girder 5'-9½" b to b of Ls web = 69 × ½ = 34.5 □" 1/8 web = 4.31 □"
flange assumed same as above
Effective depth 5.79 - .10 = 5.69 flange stress = 2525.000 ÷ 5.69 = 443.000 #
section req'd = 443.000 ÷ 17000 = 26.00
4.31
21.69 □" net

Section at splice 18' from panel point 6. m = 1900.000 "# about
Depth of girder 5.54' b to b of Ls web 66 × ½ = 33.0 □" 1/8 web = 4.13 □"
Effective depth say 5.54 - .14 = 5.40 flange stress = 1900.000 ÷ 5.40 = 352 000 #
section req'd = 352.000 ÷ 17000 = 20.70
4.13
16.57 □" net

Use 2 Ls 6.6.½ = 11.50 - 9.5
1 Pl. 14 × 5/8 = 8.75 - 7.5
20.25 17.0 □" net

Section at panel point 8 m = 1257.000
720,000 ÷ 2 = 360.000
1617.000 "#

web assumed 66 × ½ = 33.0 1/8 web = 4.13 □"
Effective depth 5.40 flange stress = 1617.000 ÷ 5.40 = 300.000 #
section required 300.000 ÷ 17000 = 17.65
4.13
13.52 □ net

Use 2 Ls 6.6.½ = 11.50 - 9.50
1 Pl. 14 × 5/8 = 8.75 7.50
20.25 17.00 □" net

Section at 9 m = 1170.000
416000 ÷ 2 = 208.000
1378.000 "#

Effective depth = 5.4 flange stress = 1378000 ÷ 5.4 = 255000 #
Section required = 255000 ÷ 17000 = 15.00
4.13
10.87 □" net

Design of Goko-Bashi for Kioto-Prefecture.

assumed section	2 Ls 6.6.½	11.50	9.50
	1 Pl 14.½	7.00	6.00
		18.50	14.50 □" net.

web.	4.13 × 17000 × 5.40 =	380.000	
Ls	9.50	871.000	
covpl.	6.00	550.000	
		1801.000 '"	

Approximate weight of girder (anchor span).

web	1 Pl.	96 × ½	@ 163.2	× 17.5	=	2860
	1 Pl.	66 . ½	@ 112.2	× 40.5	=	4550
flange.	4 Ls	6.6. ⅝	@ 24.2	× 17.5	=	1690
	4 Ls	6.6. ½	@ 19.6	. 40.5	=	3180
Cov.	2 Pls.	14 . ⅝	@ 29.75	. 17.5	=	1040
	2 Pls	14 . ⅝	@ 29.75	. 24.0	=	1430
	2 Pls.	14 . ⅝	@ 29.75	× 19.5	=	1160
stiff at bearing ½	4 Ls	5 × 4 × ½	@ 14.5	. 8.0	=	464
½	2 Pls.	9¼ . ⅞	@ 19.46	. 8.0	=	314
Int. stiffs	6 Ls	5 × 3½ . ⅜	@ 10.40	. 7.4	=	461
	6 Ls	do		. 5.7	=	356
	12 Ls	do		. 5.5	=	686
Stiff at panel pt.	8 Ls	5 × 4 . ⅜	@ 11.0	. 5.5	=	485
	4 fills	8½ . ½	@ 14.45	. 4.5	=	260
	4 fills	4 . ½	@ 6.80	. 4.5	=	123
End stiffs	8 Ls	5 × 4 . ½	@ 14.5	. 5.5	=	637
fills	2 fills	18 × ½	@ 30.8	. 4.5	=	270
splice	8 Pls.	6 × ½	@ 10.20	. 3.0	=	245
flange splice						500
splice	4 Pls	12½ . ½	@ 21.25	. 3.5	=	297
misc details say						1255
Rivet heads &c						500
						22763 "

call this 23000 "

22763 ÷ 56.97 = 399" per lin ft.

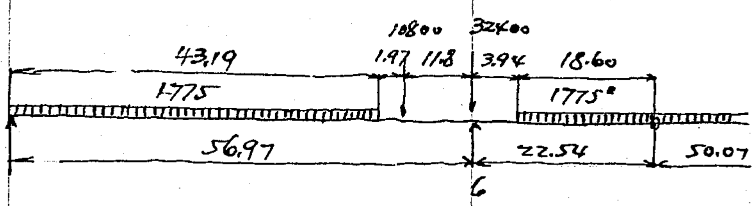
Load on shoe at panel point 11

Dead Load	49000	p20
Live load.	78500	
	127500	
Shoe say	2500	
	130000 "	

Load on shoe at panel point 6 Dead Load say 280.000 " p20

Live Load on shoe loading length

50.07
22.54
56.97
129.58



$$\text{impact} = \frac{20}{60 + \frac{129.58}{328}} = \text{say } 20.0\,\%$$

motor truck loading rear wheel 6600
impact 20% 1320
7920 × 2 = 15840

Reaction on girder = 4 @ 15840 × $\frac{16.1}{31.5}$ = 32400 "
Front wheel 10800 "

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CALCULATIONS FOR

Design of Goko-Bashi for Kioto-Prefecture

Uniform live load $\frac{100.000}{170 + \frac{129.58}{3.28}}$ = say 475 kg/m² 97 %

load on girder by proportion = $1810 \times \frac{97}{100} = 1775$#

Uniform load 1775 × 18.60 = 33000
1775 × 25.03 = 44400
77400#

moment at 6 44400 × 22.54 = 1.000.000
33000 × 13.24 = 437.000
1437.000 ÷ 56.96 = 25200#

From cantilever arm 77400
" Cantilever action 25200
102600#

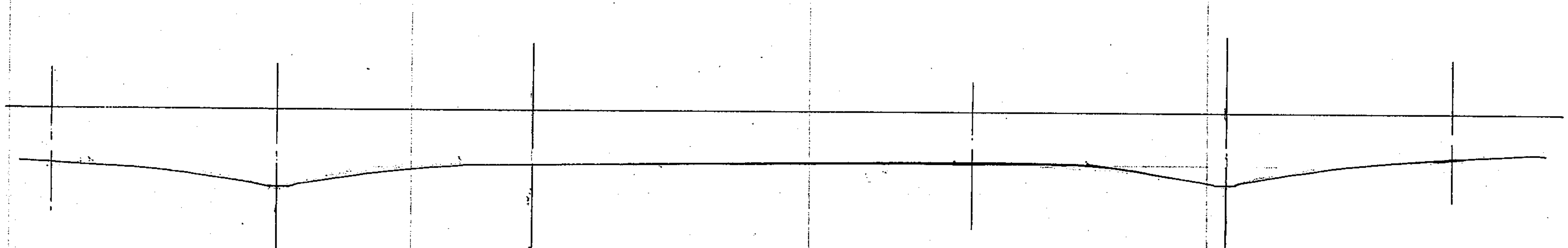
From anchor span Unif. load 1775 × 43.19 = 76500

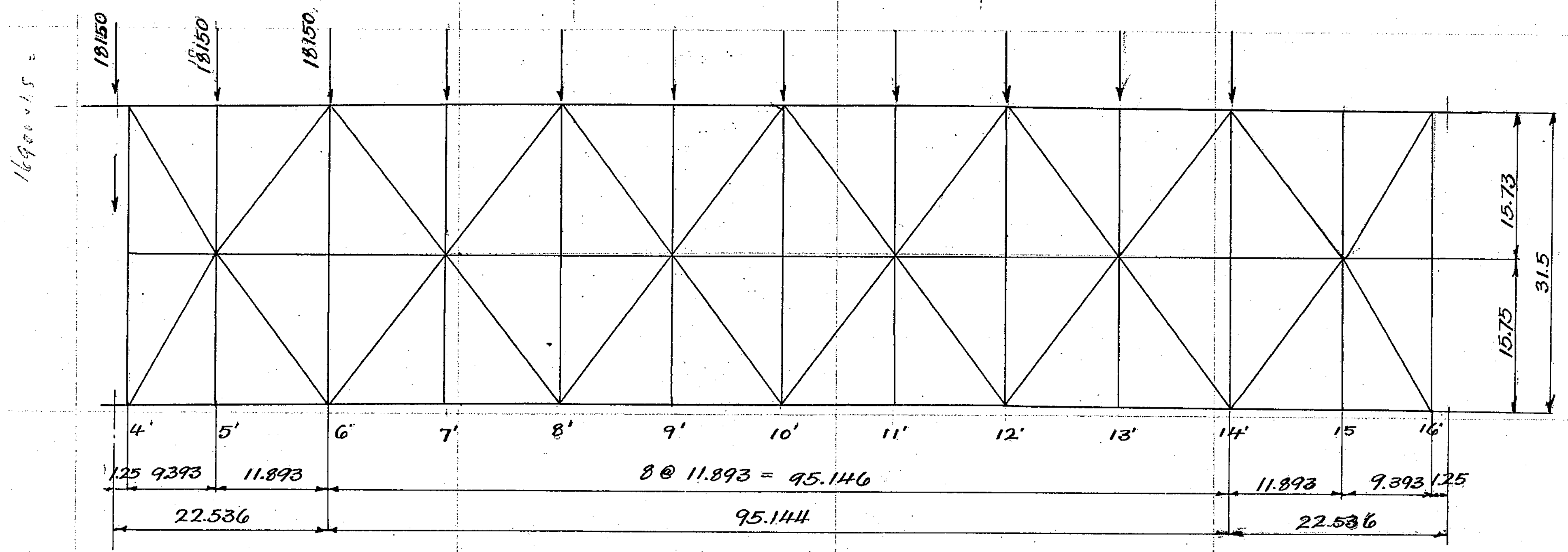
Reaction 76500 × 21.6 ÷ 56.97 = 29000
motor truck 10800 × 45.17 ÷ 56.97 = 8580
" " 32400
69980
From cantilever 102600
172580# call this 175000#

Dead Load 280.000
Live Load 175.000
455000
Add for shoe 10000
Design shoe for load of 465000#

Design of Goko-Bashi for Kioto-Prefecture.

Design of main girder for centre span span length = 95.144 (28.0 meters).





Design of floor slab, stringers and floor beams same as for suspended span.

Lateral Bracing

Lateral force due to Earthquake of 2500 mm/sec².

Dead load per lin. ft assumed 6100# Lateral force = 6100 × 0.25 = 1525# per lin. ft.

panel concentration = 1525 × 11.89 = 18150#

negative reaction due to cantilever effect

Lateral force from suspended span 16900 × 1.5 = 25400

18150

43550#

moment at 6' 43550 × 22.536 = 980.000

18150 × 11.893 = 216.000

1.196.000 '#

Reaction at 14' = 1196.000 ÷ 95.14 = 12560#

Shear for centre span as moving load.

	6'-7'	7'-8'	8'-9'	9'-10'
shear	63500	47700	34100	22700
shear from cantilever	12560	12560	12560	12560
	76060	60260	46660	35260#
Diagonal stress × 1.25	95000#	75000	58200	44000#
section reqd. unit s. 27200	3.50"	2.76	2.14	1.620" net
3/4" Rivet. 7075#	12.8	10.6	8.2	6.35
	12	10.	8	6
Use 3/4 Rivet	14	12	10	8

Some stress will be carried by compression member.

section of diagonal. 2LS 5 × 3½ × 5/16 = 5.12 - 4.040" net

Strut at centre line of bridge 2LS 4 × 3 × 5/16"

Approximate weight of bottom lateral bracing assumed 83# per lin ft.

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CALCULATIONS FOR

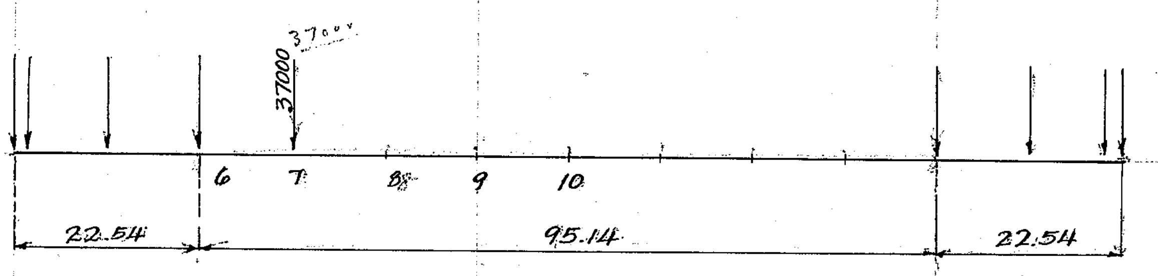
Design of Gokō-Bashi for Kioto-Prefecture.
Design of main girder.
Dead Load stresses of main girder.
panel Concentration.

floor slab + pavement Concentrated Load 1340 × 11.89 = 15900# on one girder
structural steel concentrated load
stringers 173 × 11.89 = 2050
Floor beam say 4050
Lateral Bracings 83 × 11.89 = 986
7536 ÷ 2 = say 3770# on one girder.

Uniform floor load 907# per lin. ft. of girder
Assumed weight. 553
1460# × 11.89 = 17350#
from floor 15900
" steel 3770
Total panel Concentration ———— 37020 call this 37000#

Dead Load moment.

Reaction = 37000 × 3.5 = 129700#



moment as simple span
moment at 7' 129700 × 11.89 = 1,540,000'#

moment at 8' 129700 × 2 × 11.89 = 3,080,000
37000 × 11.89 = − 440,000
2640,000'#

moment at 9' 129700 × 3 × 11.89 = 4630,000
37000 × 3 × 11.89 = −1320,000
3310,000'#

moment at 10' 129700 × 4 × 11.89 = 6,160,000
37000 × 6 × 11.89 = −2640,000
3520,000'#

Dead load Cantilever moment at support = 2419,000'# See page. 20
Summary moments at panel points

	7'	8'	9'	10'
As simple beam	+ 1540,000	+ 2640,000	+ 3310,000	+ 3520,000
Cantilever moment	− 2419,000	− 2419,000	− 2419,000	− 2419,000
	− 879,000	+ 221,000	+ 891,000	+ 1101,000'#

Shear at 6' 37000 × 3.5 = 129700#
7' 37000 × 2.5 = 92500
8' 37000 × 1.5 = 55500
9' 37000 × 0.5 = 18500

Live Load.
Live load negative moment
span length loaded. 50.07 impact assumed 24.4%.
22.54 motor truck rear wheel conc = 8210
72.61 front wheel " = 1/3 × 8210
Uniform live load 100#/□' load on one girder see page 15

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CALCULATIONS FOR

Design of Goko-Bashi for Kioto-Prefecture.

Max live load moment at 6' 1.810.000'# see page 15

negative reaction = 1.810.000 ÷ 95.14 = 19040#

negative moment at 13'	19040 × 11.89	=	− 226000
12'	" × 2 × 11.89	=	− 453000
11'	" × 3 × 11.89	=	− 680.000
10'	" × 4 × 11.89	=	− 905.000
9'	" × 5 × 11.89	=	− 1131.000
8'	" × 6 × 11.89	=	− 1360.000
7'	" × 7 × 11.89	=	− 1585.000
6'	" × 8 × 11.89	=	− 1810.000'#

Positive live load moment of center span

span length 95.14   impact $= \dfrac{20}{60 + \frac{95.14}{3.28}} = 22.5\%$.

motor truck loading   Rear wheel concentration   6600

impact 22.5%   1485

8085# × 2 = 16170#

max reaction on girder see p10 $4 \times 16170 \times \dfrac{16.1}{31.5} = 33000$#

Front wheel concentration   1/3 × 33000 = 11000#

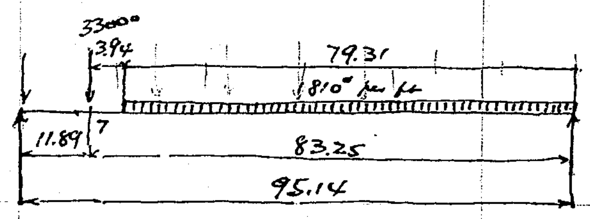
Uniform live load. $\dfrac{100.000}{170+29} = 502$ kg/m²

Use 100#/□'   Reaction on girder = 1810# per lin ft.

Bending moment at 7'

Uniform load − 1810 × 79.31 = 143,500#

Reaction − 143500 × 39.65 ÷ 95.14 = 59700#



motor truck loading   R = 33000 × $\dfrac{83.25}{95.14}$ = 28900#

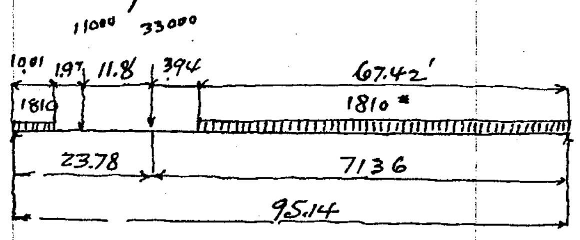
Front wheel of motor truck neglected

moment due to motor trucks   28900 × 11.89 = 344.000

" " Unif. load −   59700 × 11.89 = 710.000

1,054,000'#

Bending moment at 8'



Uniform load   1810 × 10.01 = 18100

1810 × 67.42 = 122000

140100

Reaction   18100 × 90.14 ÷ 95.14 = 17110

122000 × 33.71 ÷ 95.14 = 43100

60210#

motor truck loading   Rear wheel − 33000 × 71.36 ÷ 95.14 = 24800

Front wheel − 11000 × 83.16 ÷ 95.14 = 9600

34400#

moment due to motor truck   34400 × 23.78 = 818.000

11000 × 11.80 = 130.000

688,000'#

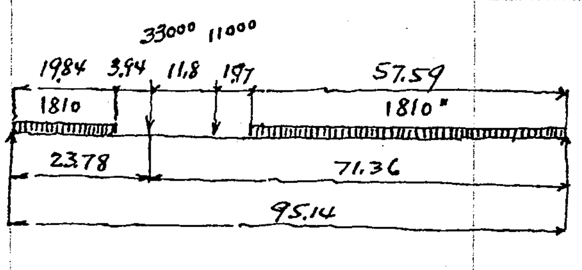
moment due to Unif. load −   60210 × 23.78 = 1430.000

18100 × 18.78 = 340.000

1.090.000

1.778,000'#

Case II.



Uniform load   1810 × 19.84 = 35900

1810 × 57.59 = 104300

140200

Reaction   35900 × 85.22 ÷ 95.14 = 32200

104300 × 28.79 ÷ 95.14 = 31600

63800#

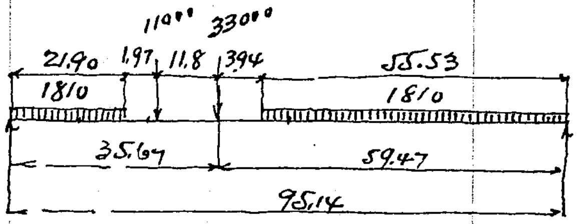
Design of Gokō-Bashi for Kioto-Prefecture.

motor truck loading rear wheel 33000 · 71.36 ÷ 95.14 = 24750
Front wheel 11000 · 59.56 ÷ 95.14 = 6900
31650 #

moment due to motor truck 31650 · 23.78 = 752.000

moment due to unif. load 63800 · 23.78 = 1520.000
35900 · 13.86 = −497.000
1023.000
1775.000 "#

Bending moment at 9' Uniform load 1810 · 21.90 = 39600
1810 · 55.53 = 100500
140100



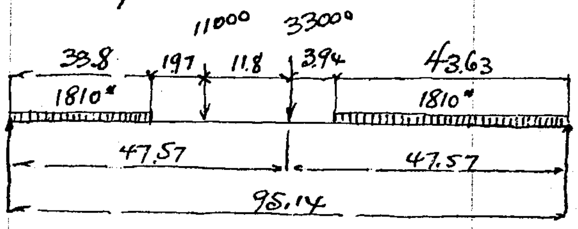
Reaction = 39600 · 84.19 · 95.14 = 35000
100500 · 27.72 · 95.14 = 29300
64300 #

motor truck 33000 · 59.47 ÷ 95.14 = 20600
11000 · 71.27 ÷ 95.14 = 8250
28850 #

Bending moment due to motor truck 28850 · 35.67 = 1030.000
11000 · 11.80 = 130.000
900.000 "#

Moment due to uniform load 64300 · 35.67 = 2300.000
396.. 25000 · 24.72 = −865.000 (880)
1435000 (1320000)
2335.000 "# (2220000)

Bending moment at 10' Uniform load 1810 · 33.8 = 611000
1810 · 43.63 = 790000
1401000 #



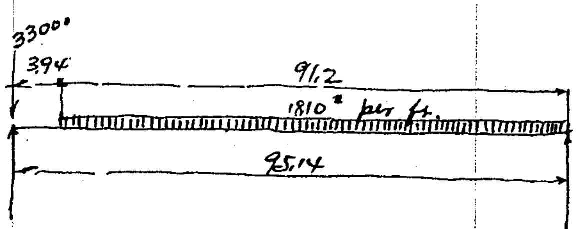
Reaction 611.000 · 78.24 ÷ 95.14 = 50250
790.000 · 21.82 ÷ 95.14 = 18100
68350 #

Reaction due to motor trucks Front wheel 11000 · 59.37 ÷ 95.14 = 6870
Rear wheel. 33000 ÷ 2 = 16500
23370 #

moment due to motor truck 23370 · 47.57 = 1110.000
11000 · 11.80 = −130.000
980.000 "#

Moment due to Unif. load. 68350 · 47.57 = 3250.000
61100 · 30.67 = −1875.000
1375000
2355000 "#.

Max End shear at 6' Reaction. Unif. load 1810 · 91.2 · $\frac{45.6}{95.14}$ = 79200

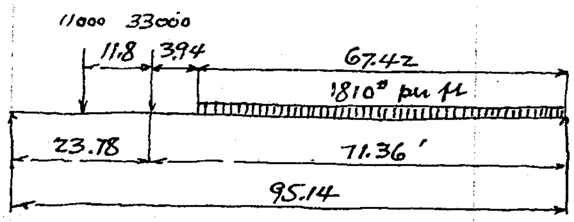


Motor truck rear wheel 33000
112200 #

Max shear at 7' loading same as in case of finding Bm at 7'. see p. 29
Reaction due to unif. load 59700
" " " m. truck 28900
88600 #

Max shear at 8'
case I. front wheel at front.

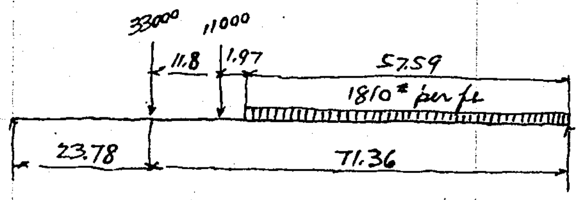
Design of Gokō-Bashi for Kioto Prefecture



Uniform load 1810 × 67.42 = 122000
Reaction = 122000 × 33.71 ÷ 95.14 = 43100#
motor trucks
Total Reaction 34400
− 11000

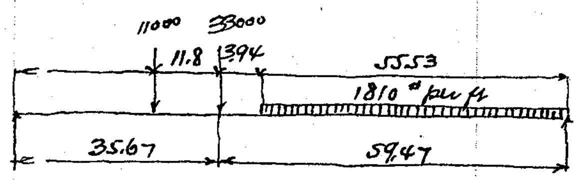
23400
66500#

Case II Front wheel at rear



Uniform load 1810 × 57.59 = 104300#
Reaction = 104300 × 28.79 ÷ 95.14 = 31600
Reaction due to motor truck 31650
63250#

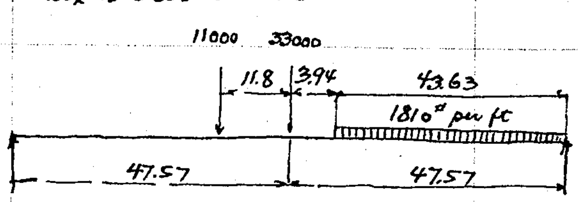
Max shear at 9'



Uniform load 1810 × 55.53 = 100500
Reaction = 100500 × 27.72 ÷ 95.14 = 29300#
motor trucks
Total Reaction = 28850
− 11000

17850
47150#

Max shear at 10'



Uniform load = 1810 × 43.63 = 79000
Reaction 79000 × 21.82 ÷ 95.14 = 18100#
motor trucks
Reaction 23370
− 11000

12370
30470#

Summary for Bending moment

	6'	7'	8'	9'	10'
Dead Load.	−2419.000	−879000	+221.000	+891000	+1101000
LL neg.	−1810.000	−1810000	−1810.000	−1810000	−1810.000
	−4229000	−2689000	−1589000	−919000	−709000

Dead Load	−2419000	−879000	+221000	+891000	+1101.000
LL pos.	0	+1054000	+1778000	+2220000	+2355000
	−2419000	+175000	+1999000	3111000	+3456.000

Design girder at 10' 3456.000
709000 ÷ 2 = 354500
3810500 1#

at 9' 3111000
919000 ÷ 2 = 459500
3670500 1#

at 8' 1999000
1589000 ÷ 2 = 794500
2793500 1#

at 7' 2689000
175000 ÷ 2 = 87500
− 2776500 1#

at 6' − 4229000

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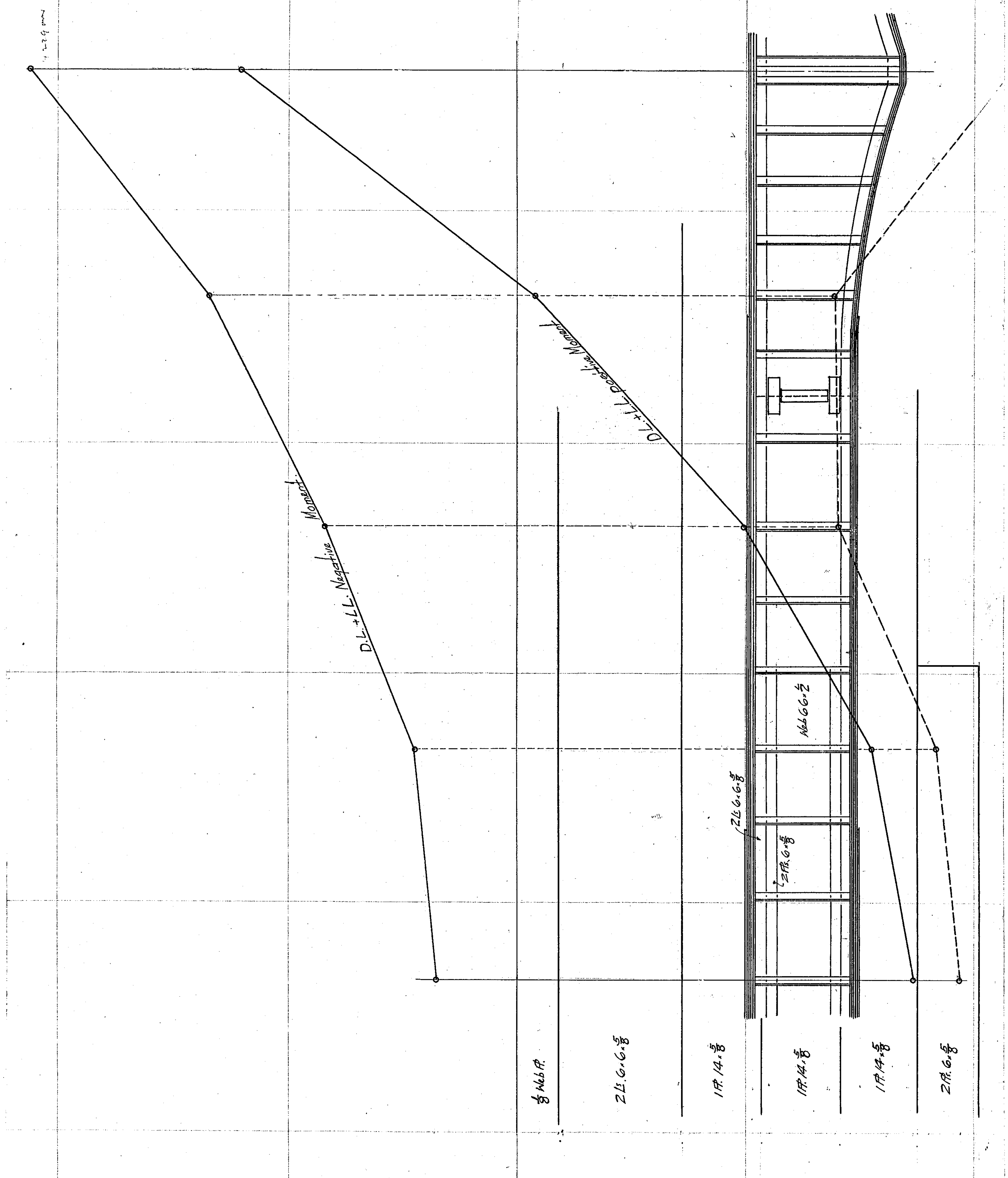
CALCULATIONS FOR

Design of Gokō-Bashi for Kioto-Prefecture.



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 Gokō-Bashi for Kioto-Prefecture

Section of girder at center m = 3810500"# pos.
Depth of girder 5'-6½" b to b of Ls web 66 × ½ = 33.00□" ⅛ web = 4.13□"
Assumed flange section

2Ls 6×6×5/8	= 14.22	× 1.73" = 24.60		11.72 net
3Pls 14×5/8	= 26.25	× .94" = −24.60		22.50
2Pls 6×5/8	= 7.50	× 7.50 = 56.20		6.25
	47.97	56.20		40.47□" net

56.20 ÷ 47.97 = 1.17" 2.34" = 0.19'
Effective depth = 5.54 − 0.19 = 5.35' flange stress = 3810500 ÷ 5.35 = 711000#
section required = 711000 ÷ 17000 = 41.90
4.13
37.77□"

moment carried by web. 4.13 × 17000 × 5.35 = 376000
11.72 = 1070.000
7.50 = 680.000
7.50 = 680.000
7.50 = 680.000
3.12 = 294.000
3.12 3294.000

Compressive stress
flange area 47.97 allowable unit stress = 14900#/□"
⅛ web 4.13 unit stress = 711.000 / 52.10 = 13650#/□"
52.10□"

Section of main girder at support m = 4229000"# neg.
Depth of girder 8'-0½ b to b of Ls ~~web assumed~~ 96 × ½ = 48.0□" ⅛ web = 6.0□"
flange section 2Ls 6×6×5/8 = 14.22 11.72
1Pl 14×5/8 = 8.75 7.50
1Pl 14×5/8 = 8.75 7.50
31.72 26.72□" net
Effective depth 7.94' flange stress = 4229.000 ÷ 7.94 = 533000#
Section reqd = 533.000 ÷ 17000 = 31.40
−6.00
25.40□" net.

Section at mid point of 6'-7' m = 4229000
2776500
7005500 ÷ 2 = 3502700"#
Effective depth = 6.61' flange stress = 3502700 ÷ 6.61 = 529.000
section reqd = 529000 ÷ 17000 = 31.10
−5.00
26.10□" net

Use same section as for support 6'

Section at panel point 7' m = 2776500"# neg.
section assumed same as above. Depth of girder 5'-9½" b to b of Ls
web 69 × ½ = 34.5□" ⅛ web = 4.31□"
Effective depth = 5.69' flange stress = 2776500 ÷ 5.69 = 488000#
section required = 488000 ÷ 17000 = 28.70
4.31
24.39□" net.

Section at panel point 8' m = 2793500"#
Section assumed same as above Depth of girder 5'-6½ b to b of Ls
web 66 × ½ = 33.0□" ⅛ web = 4.13□"
Effective depth = 5.44 flange stress = 2793500 ÷ 5.44 = 514.000#
section reqd = 514000 ÷ 17000 = 30.20
4.13
26.07□" net.

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CALCULATIONS FOR

Design of Gokō-Bashi for Kioto-Prefecture

Approximate weight of girder (Center span).

web.	1 Pl.	96. ½	@ 163.2	× 17.5	=	2860
	1 Pl.	66. ½	@ 112.2	× 30.0	=	3365
flange	4 Ls	6×6. 5/8	@ 24.2	× 48.0	=	4650
cov. pl.	4 Pls.	14 × 5/8	@ 29.75	× 48.0	=	5720
"	2 Pls.	14 × 5/8	@ 29.75	× 18.0	=	1070
stiff at pier. ½	4 Ls	5×4. ½	@ 14.5	× 8.0	=	464
½	2 fills	9¼ × 5/8	@ 19.66	× 8.0	=	314
Int. stiffs.	6 Ls	5×3½. 3/8	@ 10.4	× 7.4	=	461
	6 Ls	do		× 5.7	=	356
	8 Ls	do		× 5.5	=	457
Stiffs at panel Pt.	7 Ls	5×4 × 3/8	@ 11.0	× 5.5	=	423
	3.5 Pl.	8½ × 5/8	@ 18.06	× 4.5	=	285
web splice	6 Pls.	6½ × 5/8	@ 13.81	× 3.5	=	290
	3 Pls.	12½ × 5/8	@ 26.56	× 3.5	=	280
flange splice				say		1500
		misc details say				500
		Rivet heads &c	3½ %.			805
						23700 #

For one girder 2 @ 23700 = 47400 #

47400 ÷ 95.0 = say 500 # per lin ft.

Summary for shear

Live Load shears	6'-7'	7'-8'	8'-9'	9'-10'
As simple beam	112200	88600	67500	47150
Cantilever effect	19040	19040	19040	19040
	131240	107640	86540	66190
Dead Load shears	129700	92500	55500	18500
Total shear	260940 #	200140 #	142040 #	84690 #

Load on shoe.

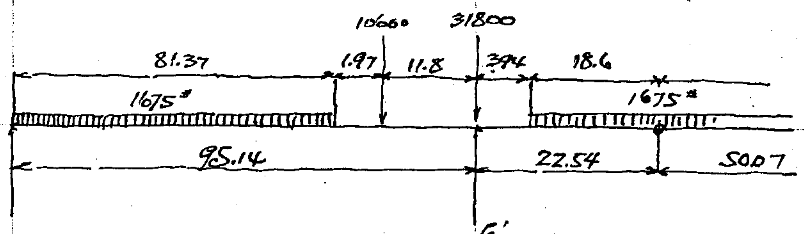
Dead Load shear from center span 37000 × 3.5 = 129700

at 6' p20 36420

from Cantilever beam. p20 123865

289985 call this 290.000 #

Live Load on shoe



loaded length 50.07
Cantilever arm 22.54
Simple span 95.14
167.75

$$\text{impact} = \frac{20}{60 + \frac{167.75}{3.28}} = 18\%.$$

motor truck rear wheel 6600

Impact 18% 1190

7790 # × 2 = 15580

$4 @ 15580 \times \frac{16.1}{31.5} = 31800$ on girder

31800 ÷ 3 = 10600 # for front wheel.

$$\text{Uniform live load} \quad \frac{100.000}{170 + \frac{167.75}{3.28}} = 452 \ \text{kg/m}^2 \quad 92.5\# \text{ per sq. ft.}$$

$$\text{load on girder by propotion} = 1810 \times \frac{92.5}{100} = 1675\# \text{ per lin ft of girder.}$$

Uniform load

1675 × 18.6 = 31200

1675 × 25.03 = 42000

73200 #

moment at 6'

42000 × 22.54 = 945.000

31200 × 13.24 = 413.000

1358,000 ÷ 95.14 = 14260 #

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CALCULATIONS FOR

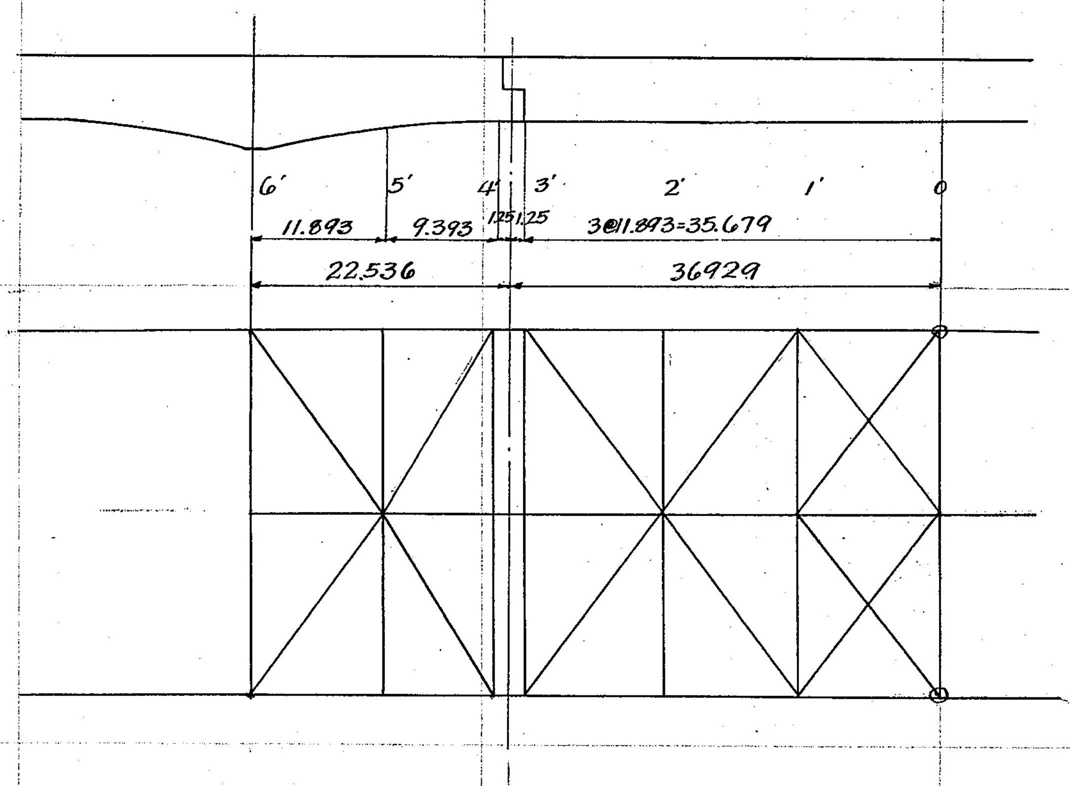
Design of Goko-Bashi for Kioto-Prefecture

From cantilever arm 73200
from cantilever effect 14260
87460 #

From center span
Uniform load 1675 × 81.37 = 136000
Reaction at 6' 136000 × 40.69 ÷ 95.14 = 58200
motor truck Front wheel 10600 × 83.44 ÷ 95.14 = 9300
rear wheel 31800
99300
From cantilever 87460
Total load 186760 #
call this 187000 #

Dead Load on pier. 290.000
Live load. 187.000
477000
Add for shoe 10000
Design shoe for. 487000 # load.

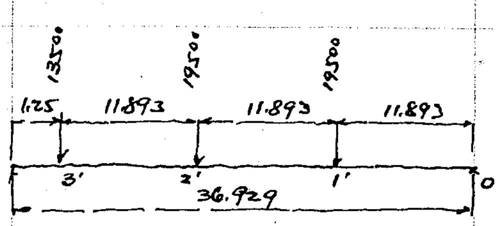
Design of suspended span at end. span length 36.929'



Design of floor slab., stringers + floor beams same as for 50.07' suspended span.
Lateral Bracing similar and as shown on sketch above.

Design of main girder.
Dead load. panel Concentration at 3' 13500 # p9
at 2' + 1' 19500 #
at 0 $19500 \times \frac{695}{1189} = 11400$ # assumed.

Dead load moment



Reaction 19500 × 3 × 11.893 ÷ 36.929 = 18850
13500 × 3 × 11.893 ÷ 36.929 = 13050
31900 #

moment at 3' 31900 × 1.25 = 39900 '#
moment at 2' 31900 × 13.14 = 420000
13500 × 11.89 = -160600
259400 '#

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CALCULATIONS FOR

Design of Goko-Bashi for Kioto Prefecture.

Reaction at 0 2@19500 = 39000
13500
52500 - 31900 = 20600#

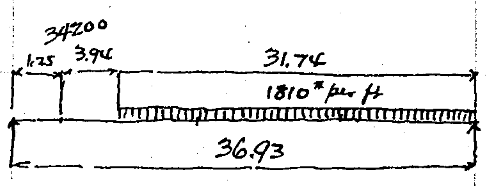
Note. Bending moment due to load as uniform see p37

D.L. moment at 1' = 20600 · 11.89 = 245000'#

Live Load p10. motor truck loading rear wheel = 34200# on one girder
front wheel = 11400#

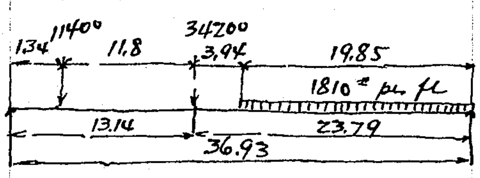
Uniform live load 100#/□' 1810# per ft on girder.

max Bending moment at 3'



Uniform load - 1810 · 31.74 = 57400
Reaction = 57400 · 15.87 ÷ 36.93 = 24700#
motor truck R = 34200 · 35.68 ÷ 36.93 = 33100
moment unif. load · 24700 · 1.25 = 30900
motor truck = 33100 · 1.25 = 41400
72300'#

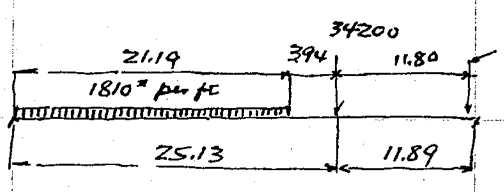
Bending moment at 2'



Uniform load 1810 × 19.85 = 35900#
Reaction = 35900 · 9.92 ÷ 36.93 = 9650#
motor truck loading
34200 · 23.79 ÷ 36.93 = 22000
11400 · 35.68 ÷ 36.93 = 11000
33000#

Moment due to uniform load 9650 · 13.14 = 127000
" " " motor trucks 33000 · 13.14 = 434000
11400 · 11.80 = -134500
299500
426500'#

Bending moment at 1'



unif. load = 1810 · 21.19 = 38300#
Reaction = 38300 · 10.60 ÷ 36.93 = 11000#
motor truck loading 34200 · 25.13 ÷ 36.93 = 23250#

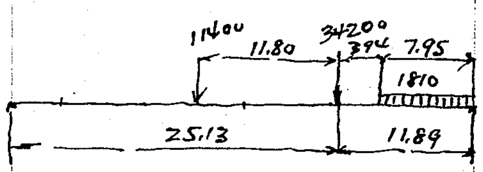
Moment due to uniform load 11000 · 11.89 = 131000
" " " motor trucks 23250 · 11.89 = 276500
407500'#

max shear at support

Uniform load - 24700
motor truck 33100
57800#

Shear at 3'-2' Uniform load - 9650
motor truck 33000
- 11400
21600
31250#

Shear at 2'-1' Uniform load 1810 · 7.95 · $\frac{3.98}{36.93}$ = 1550



motor truck 34200 · 11.89 ÷ 36.93 = 11020
11400 · 23.69 ÷ 36.93 = 7310
18330
- 11400
6930
1550
8480#

Shear at 1'-0 Uniform load 11000
motor truck 23250
34250#

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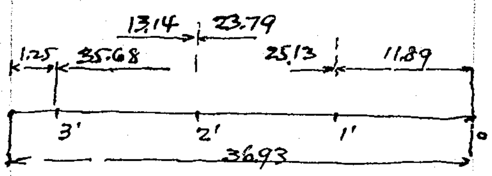
CALCULATIONS FOR

Design of Goko-Bashi for Kioto-Prefecture.

Dead load moment due to load as uniform



Uniform load assumed 1182# per lin ft.

at 3' $\frac{1}{2}$ × 1182 × 1.25 × 35.68 = 26400'#

2' " × 13.14 × 23.79 = 185000

1' " × 25.13 × 11.89 = 177000

Shear max at end $\frac{1}{2}$ × 1182 × 36.93 = 21800

Shear at 3' $1182 \times \frac{35.68^2}{2 \times 36.93}$ = 20400

Shear at 2' $1182 \times \frac{23.79^2}{2 \times 36.93}$ = 9150

Shear at 1' $1182 \times \frac{25.13^2}{2 \times 36.93}$ = 10100

Summary for Dead load moments and shears.

	-3'	2'	1'	End-3'	3'-2'	2'-1'	1'-0
due to conc.	39900	259400	245000	31900	18400	1100	20600
due to unif.	26400	185000	177000	21800	20400	10100	21800
	66300	444400	422000'#	53700#	38800	11200	42400#

Summary for live + Dead load moment and shear

	3'	2'	1'
Dead load moment	66300	444400	422000
live load moment	72300	426500	407500
	138600	870900	829500

	End-3'	3'-2'	2'-1'	1'-0
Dead Load shear	53700	38800	11200	42400
live load shear	57800	31250	8480	34250
	111500	70050	19680	76650

Section of main girder M = 870900'#

web assumed 66.3/8 = 24.75 □" 1/8 web = 3.09 □" Depth 5'-6½" b to b of L^s.

flange angle assumed 2L^s 6.6.½. Effective depth = 5.54 - .28 = 5.26'

flange stress = 870900 ÷ 5.26 = 166000#

section req'd = 166000 ÷ 17000 = 9.76

3.09

6.67 □" net.

Try 2L^s 6.6.½ = 11.50 - 10.50 □" net.

allowable comp. stress = $17000 \left(1 - 0.012 \frac{144}{12.37}\right)$ = 14650#/□"

Gross section in flange - 11.50

1/8 web 3.09

14.59 □"

Unit stress = 166000 ÷ 14.59 = 11400#/□" O.K.

Load on shoe over abutment

Dead load. 42400

Concentration - 11400

53800#

Max live load

unif. load. $1810 \times \frac{33^2}{2 \times 36.93}$ = 26600

motor truck 34200

60800

114600

weight of shoe assumed 5000

119600 call this 120.000#

Design of shoe same as for anchor span at abutment

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CALCULATIONS FOR

Design of Goko-Bashi for Kioto-Prefecture.

Approximate weight of main girder.

web.	1 Pl.	66 × 3/8	@	84.2	× 37.0 av	=	3040
flange.	4 Ls	6 · 6 · 1/2	@	19.6	× 37.0 av	=	2900
floor beam conn.	8 Ls	5 × 4 × 3/8	@	11.0	× 5.5	=	363
	3 fill	8 1/2 × 1/2	@	14.45	× 4.5	=	293
	12 Ls	5 × 3 1/2 × 3/8	@	10.4	× 5.5	=	685
	3 fill	4 × 1/2	@	6.80	× 4.5	=	92
Bearing	8 Ls	5 × 4 × 1/2	@	14.5	× 5.5	=	638
	2 fill	18 × 1/2	@	30.80	× 4.5	=	277
	8 Ls	5 × 4 × 1/2	@	14.5	× 2.75	=	320
	2 fills	36 × 1/2	@	61.2	× 4.5	=	550
Splice web	4 Pls.	6 × 1/2	@	10.2	× 3.0	=	123
"	2 Pls.	13 1/2 × 1/2	@	22.95	× 3.5	=	160
Flange. say							150
	misc detail						500
	Rivet heads etc say						450
							10541

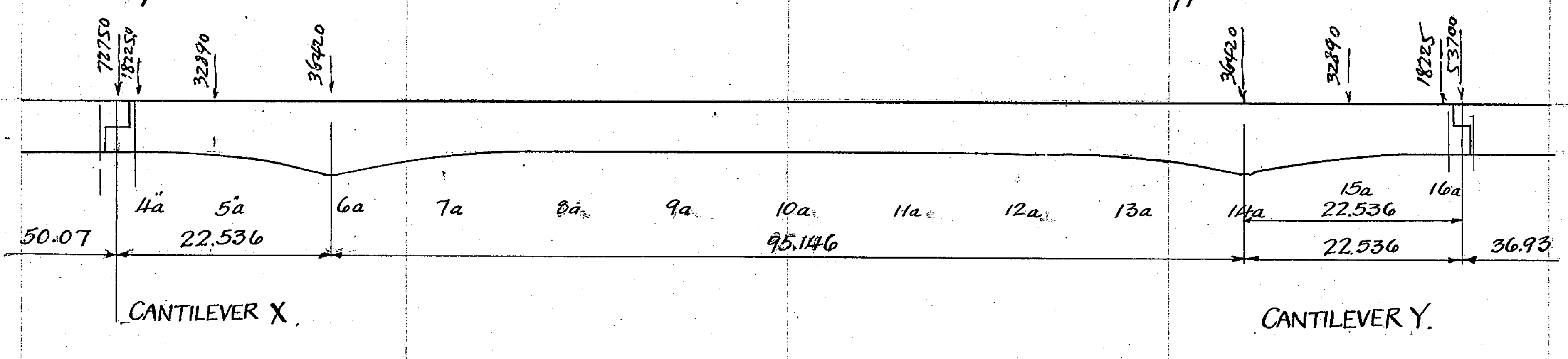
10541 ÷ 36.9 = 286# or say 290# per lin ft of girder.

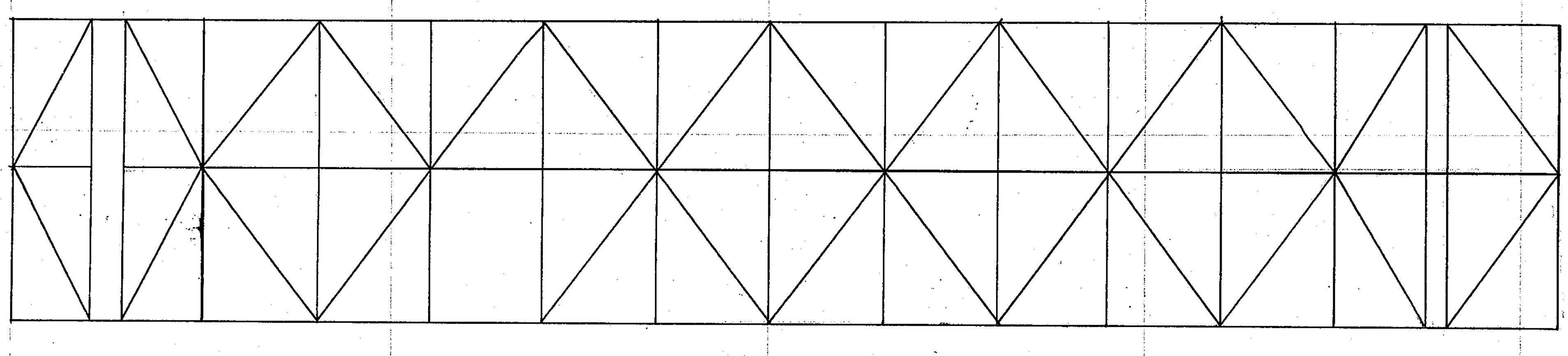
Load at cantilever Bearing

Dead Load	53700
Live load	57800
	111500#

Design bearing same as for End bearing of 50' suspended span

Design of main span with overhanging arm suspended spans different.





Overhanging arm X as figured on p 13-18

Dead Load moment at 6a

18225	× 21.28	=	388 000	
32890	× 11.89	=	391 000	
72750	× 22.54	=	1640.000	
123865#			2419.000'#	÷ 95.14 = 25400#

Dead Load moment at 14a

18225	× 21.28	=	388 000	
32890	× 11.89	=	391 000	
53700	× 22.54	=	1210.000	
104815#			1989000	÷ 95.14 = 20900#

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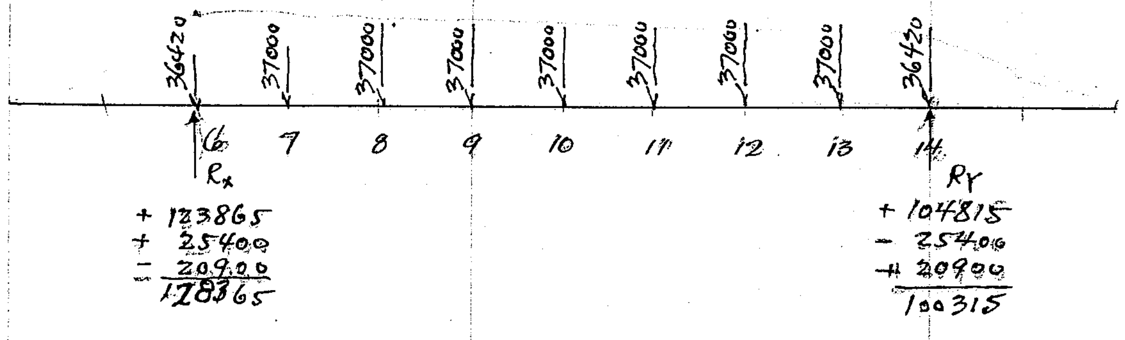
CALCULATIONS FOR

Design of Toko-Bashi for Kioto-Prefecture.



Cantilever moments.

X.	2419.000
Y	1989.000
	430.000

increment = 430.000 ÷ 8 = 53800#

	6a	7	8	9	10	11	12	13	14a
moment in cantilever	−2419.000	−2365000	−2311800	−2258000	−2204200	−2150400	−2096600	−2042800	−1989000
as simple beam	0	+1540000	+2640000	+3310000	+3520000	+3310000	+2640000	+1540000	0
	−2419000	−825,000	+328,200	+1,052,000	+1315800	+1169600	+543400	−502800	−1989000

Shear as	6–7	7–8	8–9	9–10	10–11	11–12	12–13	13–14
simple span	129700	92500	55500	18500	−18500	−55500	−92500	−129700
as cantilever effect	4500	4500	4500	4500	4500	4500	4500	4500
	134200	97000	60000	23000	−14000	−51000	−88000	−125200

max Dead load at Rx. 128365 / 36420 / 129700 / 294485#

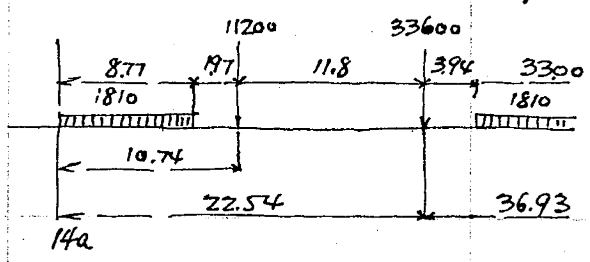
max dead load at Ry 100315 / 36420 / 129700 / 266435#

Max live load negative moment due to cantilever X see p 29

at	
14a	0
13	−226000
12	−453000
11	−680.000
10	−905000
9	−1131000
8	−1360.000
7	−1585.000
6a	−1810.000.

impact assumed 24.4 %.
motor truck rear wheel conc. 8210
Front wheel = 1/3 × 8210 =
Uniform load 100#/0'

Max live load negative moment due to cantilever Y.
load and impact assumption same as above.



Uniform live load on suspended span
$= 1810 \times \frac{33.0^2}{2 \times 36.93} = 26700$

motor truck loading 33600
60300#

moment at 14a			
60300 × 22.54	=	1360.000
11200 × 10.74	=	120.000
15900 · 4.39	=	70.000
		1,550.000#

Reaction at 6a 1,550.000 ÷ 95.14 = 16300#

at			
14a	16300 × 11.89 × 8	=	−1,550.000
13	16300 × 11.89 × 7	=	−1358.000
12	× 6	=	−1163.000
11	× 5	=	−970.000
10	× 4	=	−775.000
9	× 3	=	−582.000
8	× 2	=	−388.000
7	× 1	=	−194.000
6a	×		0

Design of Gokō-Bashi for Kioto-Prefecture.

Summary for negative live load moments

6a	7	8	9	10	11	12	13	14a
-1810.000	-1585000	-1360.000	-1131.000	-905000	-680.000	-453000	-226.000	0
0	-194000	-388000	-582.000	-775000	-970.000	-1163000	-1358000	-1550.000
-1.810.000	-1779000	-1748000	-1713000	-1680000	-1650000	-1616000	-1584000	-1550.000

Summary for positive live load moments

6a	7	8	9	10	11	12	13	14a
	1054000	1778000	2220.000	2355000	2220.000	1778000	1054000	0

Summary for moments

	6a	7	8	9	10	11	12	13	14a
Dead load	-2419000	-825000	+328200	+1052.000	+1315800	+1169600	+543400	-502800	-1989000
LL neg.	-1.810.000	-1779000	-1748000	-1713.000	-1680000	-1650000	-1616000	-1584000	-1550.000
	-4229000	-2604000	~~-2076200~~ 1419800	-661.000	-364200	-480400	-1072600	-2086800	-3539000

	6a	7	8	9	10	11	12	13	14a
Dead Load	-2419000	-825000	+328200	+1052000	+1315800	+1169600	+543400	-502800	-1989000
LL. pos.	0	+1054000	+1778000	+2220.000	+2355000	+2220.000	+1778000	+1054000	0
	-2419000	+229000	+2106200	+3272000	3670800	+3389600	+2321400	+551200	-1989000

Design girder for | | For sym Cantilever arm (p31).

at 6a 4229000 # neg. —— 4229000

at 7a 2604000
229000÷2 = 114500
2718500 neg. —— 2776500

at 8a 2106200
1419800÷2 = 709900
2816100 pos. —— 2793500

at 9a 3272000
661000÷2 = 330500
3602500 pos. —— 3610500

at 10a 3670800
364200÷2 = 182100
3852900 pos. —— 3810500

at 11a 3389600
480400÷2 = 240200
3629800 pos. —— 3610500

at 12a 2321400
1072600÷2 = 536300
2857700 pos. —— 2793500

at 13a 2086800
551200÷2 = 275600
2362400 neg. —— 2776500

at 14a ~~1989000~~
3539000 —— 4229000

For symmetry design cantilever arm Y same as for cantilever arm X.

Design of Gokō-Bashi for Kioto-Prefecture.

Section of girder at center m = 3852900# req. refer to p33.
Depth of girder 5'-6½" b to b of Ls web 66 × ½ = 33.0 □" ⅛ web = 4.13 □"
Assumed flange section

2Ls 6.6. 5/8	= 14.22	1.73"	= 24.60		- 11.72
3Pls. 14 × 5/8	= 26.25	.94"	- 24.60		22.50
2Pls. 6. 5/8	= 7.50	7.50	56.20		6.25
	47.97 □"		56.20		40.47 □" net

5620 ÷ 47.97 = 1.17" 2.34" = 0.19'
Effective depth = 5.54 - 0.19 = 5.35' flange stress = 3852900 ÷ 5.35 = 720.000#
Section required = 720.000 ÷ 17000 = 42.40
4.13
38.27 □" net

Section of main girder at support m = 4229000#
Section same as for standard span see p33.

Section at 7a - 6a m = 4229000
2718.500
6947500 ÷ 2 = 3,473,700#
which is less than moment in standard span use same section as for standard span.

Section at 12a - 11a M = 2857700# pos. instead of 2793500
assumed section - 2Ls 6 × 6 × 5/8 = 14.22 - 11.72
2Pls. 14 × 5/8 17.50 15.00
31.72 26.72 □" net.
Depth of girder 5'-6½" b to b of Ls web 66 × ½ = 33.0 □" ⅛ web = 4.13 □"
Effective depth = 5.44 flange stress = 2857700 ÷ 5.44 = 524000#
section required = 524000 ÷ 17000 = 30.80
4.13
26.67 □" net

Summary for shears
Live Load max shear same as standard span see p34

Live Load shears	6a-7a	7-8	8-9	9-10
as simple beam	112200	88600	66500	47150
Cantilever Effect.	19040	19040	19040	19040
	131240	107640	85540	66190.
Dead Load shears	134200	97000	60000	23000
	265440#	204640	145540	89190#

Load on shoe -
Dead load shear from center span 37000 × 35 = 129700
at 6a 36420
from cantilever see p39 128365
294485 call this 295000#
Live load max see p34-35 187000
482000
Add for shoe 10000
Design shoes for. 492000#

Roller 5½" dia. 5.5 × 610 = 3360# per lin. inch 492000 ÷ 3360 = 146.4"
For 7 rollers 146.4 ÷ 7 = 21.0" net each

Pin assumed 5" bearing per inch = 5 × 24000 = 120,000#
492000 ÷ 120.000 = 4.1" for direct bearing top and bottom
To prevent up and down motion or to anchor the superstructure to pier use eye holes to top and bottom castings of shoe

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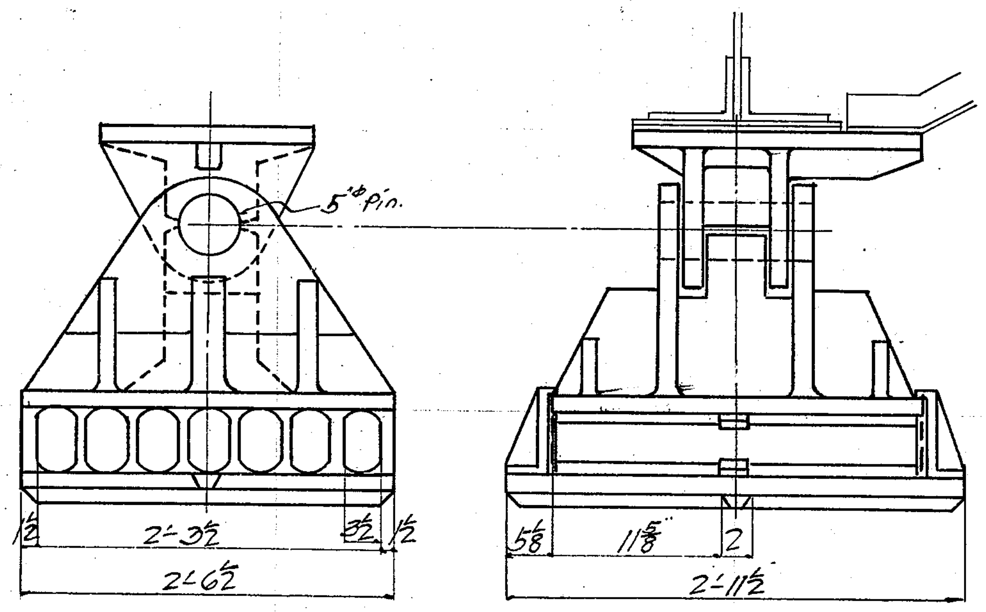
CALCULATIONS FOR

Design of Goko-Bashi for Kioto-Prefecture.



Approximate bearing area = 30.5 × 34" = 1040 □"
unit bearing 492000 ÷ 1040 = 473 #/□"
For approximate weight of casting 5000 # for roller shoes } assumed.
3500 # for fixed shoe

Approximate weight of structural steel in super structure.

50' suspended span

stringers	173 × 51.5	=	8900
floor beam	5@4140	=	21700
laterals	83 × 51.5	=	4280
girders	2@14500	=	29000
			63880 ÷ 2240 = 28.5 tons

37' suspended span

stringers	173 × 38.65	=	6700
floor beam	4@4140	=	16560
laterals say		=	3500
girder	2@10600	=	21200
			47960 ÷ 2240 = 21.4 tons

Anchor span

stringers	173 @ 82.5	=	14300
floor beam	8@ 4140	=	33100
laterals & bracket	say	=	7300
girder cantilever portion	2@9200	=	18400
girder anchor arm	2@23000	=	46000
			119100 ÷ 2240 = 53.2 tons

Center span

stringers	173 × 139	=	24100
floor beam	13@4140	=	53800
laterals	say		12000
girder cantilever portion	4@9200	=	36800
girder center span	2@47400	=	94800
			221500 ÷ 2240 = 99.0 tons.

Shoes

22 roller shoes	@5000	=	110.000
20 fixed shoes	@3500	=	70.000
8 shoes	@1500	=	12.000
			192.000 ÷ 2240 = 85.5 tons

Summary of steel in bridge structure

10 suspended span 50'	@ 28.5	=	285.0
1 - 37' suspended span			21.4
3 - anchor spans	@ 53.2	=	159.6
9 - center spans	@ 99.0	=	891.0
Shoes. say			85.5
			1442.5 tons

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CALCULATIONS FOR

Design of Toko-Bashi for Kioto-Prefecture.

Design of piers. No 10 to No 21 inclusive

Superimposed dead load p34 2 @ 290.000 = 580.000 #

Live load see p 34 for loading

Uniform load 92.5 × 36' = 3330# per ft

Unif. load 3330 × 18.6 = 61800
3330 × 25.03 = 83300
145100 #

Moment at 6 83300 × 22.54 = 1.875.000
61800 × 13.24 = 819.000
2.694.000 ÷ 95.14 = 28300
145100
From cantilever arm 173400 #

From center span

motor truck loading rear wheel 4 @ 15580 = 62300
Front wheel 62300 ÷ 3 = 20770

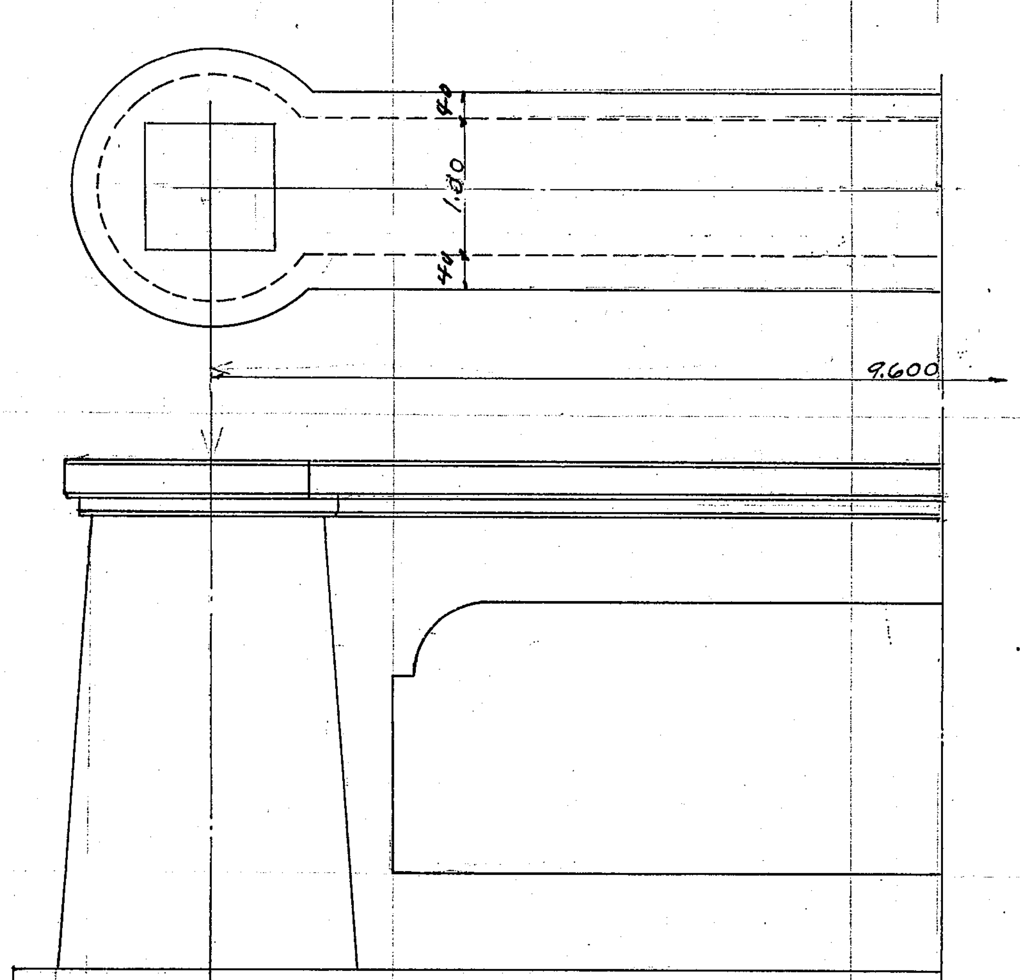
Reaction due to uniform load 3330 × $\frac{81.37^2}{2 \times 95.14}$ = 115.800

Reaction due to motor truck front wheel 20770 × $\frac{83.44}{95.14}$ = 18 200
rear wheel 62300
196 300
From cantilever arm 173 400
369700 call this 380.000 #

Total Dead load 580.000
Live Load 380.000
960.000 # on one pier.

max load on bearing. p35 487.000 #
p41 492.000 #

Size of shoe 34" × 34" about.



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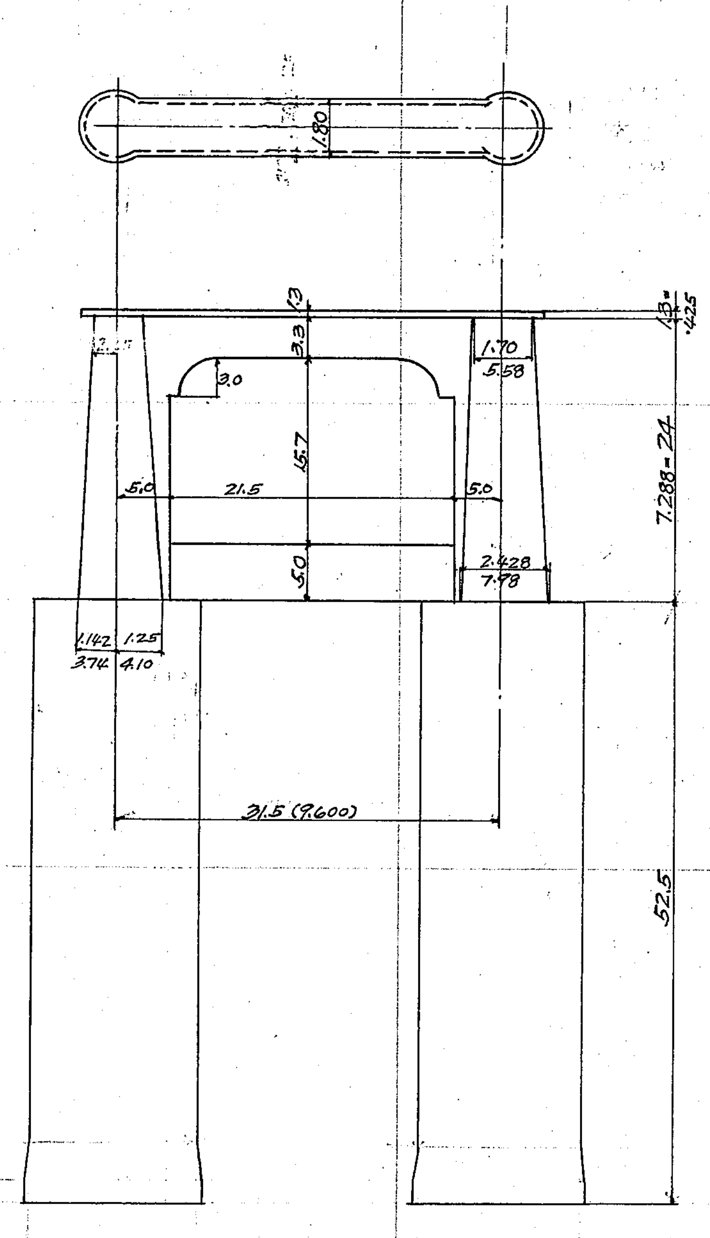
CALCULATIONS FOR

Design of Gokō-Bashi for Kioto-Prefecture.



Coping 6.88ϕ 37.2 × 0.8 = 29.80
6.13ϕ 29.5 × 0.5 = 14.75
44.55

4.3 × 0.8 = 3.44
3.5 × 0.5 = 1.75
5.19 × 26.0 = 135.0
2 @ 44.55 = 89.1
224.10 cubic ft

volume of shaft
top area = 5.58ϕ = 24.45
7.98ϕ = 50.00
74.45 ÷ 2 = 37.23 ◻'
volume = 37.23 × 24.0 = 893
2 @ 893 = 1786.0 cubic ft.

volume of web:-
15.7 × 21.5 = 338.
less 17.1
320.9 × 2' = 641.8 cubic ft.

between shaft.
area 25 × 24 = 600
less 320.9
279.1 × 3' = 890.0
1531.8 cubic ft.

Coping. 224.10
2 shafts 1786.00
web. 1531.80
3541.90 @ 150# = 531.000#

well. shell. 13.45'ϕ 142.0
10.17 81.1
60.9 × 52.5 = 3200 @ 150# = 480.000
top filling 81.1 × 10 = 81.1
bottom filling 81.1 × 10 = 81.1
162.2 @ 140# = 227.000

Int. filling 81.1 × 32.5 = 2640 @ 100 = 264.000
971.000 × 2 = 1942.000#

Bearing Pressure at bottom of base.
Superimposed Dead load 580.000
Live Load 380.000
960.000
shaft. 531.000
well. 1942.000
3433.000#

Surface friction of well assumed 250#/◻' circumference of well 2 @ 42.2 = 84.4'
84.4 × 250 = 21100# per lin ft.
Total friction = 21100 × 47.5 = 1.000.000
Load at bottom of base 3433.000 −
1000.000
2433.000#

Bottom area of well 14'ϕ 154 sq ft 2 @ 154 = 308.
Unit pressure = 2433.000 ÷ 308 = 7900#/◻' or 3.53 tons/◻'

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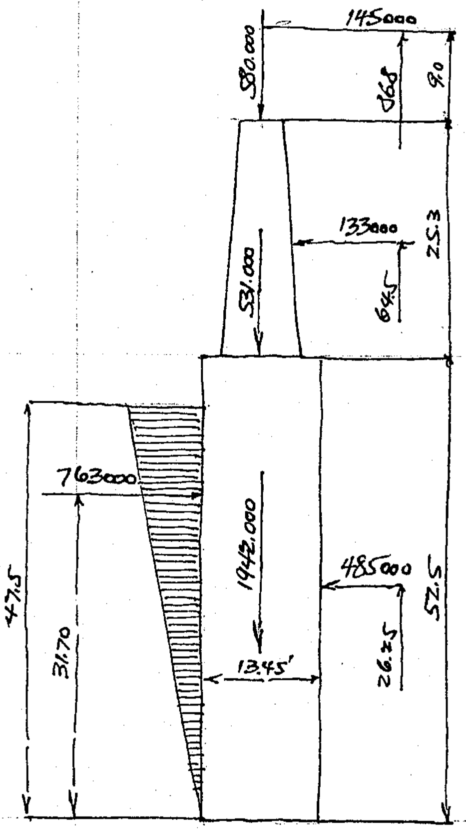
CALCULATIONS FOR

Design of Goko-Bashi for Kioto-Prefecture

Bearing Pressure at bottom of base during Earthquake.
Seismic force = 2500 mm/sec²



			Hor. Force	Arm		moment
Superimposed D.L.	580.000	× .25	= 145.000	× 86.8	=	12.600.000
weight of shaft	531.000	× .25	= 133.000	× 64.5	=	8570.000
weight of well	1942.000	× .25	= 485.000	× 26.25	=	12750.000
	3053.000		763.000			33920000
			Less 763.000 × 31.70		=	24200000
						9.720,000 ′#

Arm for frictional couple = 13.45 - .64 = 8.6′
Total friction = 1,000.000
moment = 500.000 × 8.6 = -4300.000

5.420,000 ′#

Moment of inertia of bottom area = 0.049 × 14⁴ = 1885 (1)⁴
2 @ 1885 = 3770

Fibre stress = $\frac{5420.000 \times 7}{3770}$ = 10100 #/□′ or 4.51 tons/□′

Direct soil pressure load 3,053.000
less friction 1,000.000
2053.000 #

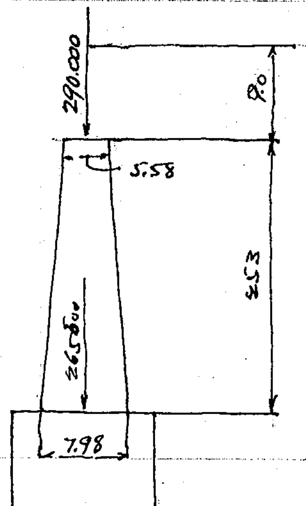
Unit pressure = 2053000 ÷ 308 = 6670 #/□′ or 2.92 tons/□′

summary for bearing pressure.

Direct load	2.92	2.92
Due to bending	4.51	- 4.51
	7.43 tons/□′	- 1.59 tons/□′

Reinforcement in shaft due to Earthquake.

Superimposed dead load on one shaft = 580.000 ÷ 2 = 290.000 #
weight of shaft. 531.000 ÷ 2 = 265500 #



Superimposed D.L.	290.000	× .25	= 72500	× 34.3	= 2485.000
weight of shaft	265.500	× .25	= 66400	× 12.0 abt	= 797.000
	555.500				3.282,000 ′#

Moment of inertia of bottom section of shaft
Concrete = 0.049 × 7.98⁴ = 415.00 × 0.049 = 203.0
Steel = 0.098 × 15 × $\frac{7.48^2}{8}$ = 10.3
213.3

fibre stress = $\frac{3282.000 \times 3.99}{213.3}$ = 61400 #/□′ 426 #/□″

Equivalent concrete area of section.

32-3/4"φ bars = 14.1 □″
0.098 □′

Concrete = 7.98φ = 50.0 × 144 = 7200
14.1 × 15 = 212
7412

Unit direct stress = 555500 ÷ 7412 = 75 #/□″

Summary of stress

Due to bending	426	426
" " direct load	75	75
	501 #/□″ comp.	- 351 #/□″ tension.

Reinforce shaft neglecting tension in concrete of shaft.

Note:- Diameter of shaft revised see next page.

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CALCULATIONS FOR

Design of Gokō-Bashi for Kioto-Prefecture.

Revise shaft.

Top area 5.58 ϕ = 24.45

8.58 ϕ = 57.81

82.26 ÷ 2 = 41.13 × 24.0 = 987 @150 = 148 000

2@ 987. = 1974

coping 224.10

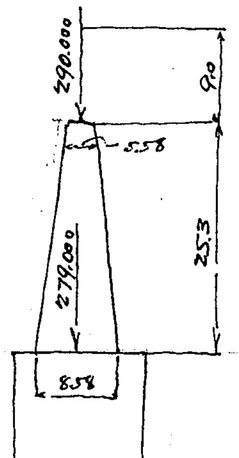
2 shafts 1974.00

web. 1531.80

3729.90 @150 = 558 000

For one shaft 558 000 ÷ 2 = 279.000 #

Reinforcement in shaft due to Earthquake.



				H.F.	Arm		Moment
Superimposed D.L.	290.000	× .25	=	72500	× 34.3	=	2485.000
Weight of shaft	279.000	× .25	=	69800	× 12.0 abt	=	837.000
	569.000						3322000 ″#

Moment of inertia of bottom section of shaft

Concrete 0.049 × 8.58⁴ = 266.0

steel say $15 \times 0.098 \times \frac{8.08^2}{8}$ abt = 12.0

278.0

Fibre stress = $\frac{3322000 \times 4.29}{278.}$ = 51200 #/□′ or 356 #/□″

32. 3/4″ϕ = 14.1□″ or 0.098□′

Equivalent concrete area of section

Concrete 8.58′ ϕ = 57.81 × 144 = 8320

Steel 14.1 × 15 212

8532 □″

Unit direct stress = 569.000 ÷ 8532 = 67 #/□″

Summary of stress

Due to bending	356	−356
Due to direct load	67	67
	423 #/□″ C	−289 #/□″ T.

Reinforce shaft neglecting tension in concrete.

Approximate stress in concrete and steel in shaft.

Round section assumed as square section of 7.0′ × 7.7′

Reinforcing bars assumed 13- 3/4″ϕ bars @.44 = 5.72 on one face

On both sides 2@5.72 = 11.44 □″

Cross sectional area of Concrete 7.0 × 7.7 × 144 = 7750

~~Steel~~ % 11.44 : 7750 = .148 %.

Eccentricity = 3322 000 ÷ 569.000 = 5.85

Ecc to depth = 5.85 ÷ 7.7 = 0.76 k = 0.24 C = 0.073

Fibre stress $f_c = \frac{3322.000 \times}{0.073 \times 7.0 \times 7.7^2}$ = 110000 #/□′ or 765 #/□″

$f_s = 15 \times 765 \left(1 - \frac{7.5}{7.7} \times \frac{1}{.24}\right)$ = 35000 #/□″

Try double rows of reinforcement 26- 3/4″ϕ bars. 22.88 □″ on both faces.

steel % = .296 %.

Eccentricity 5.85 E/h = 0.76 k = 0.31 C = 0.097

Fibre stress $f_c = \frac{3322.000}{0.097 \times 7. \times 7.7^2}$ = 82500 #/□′ or 573 #/□″

$f_s = 15 \times 573 \left(1 - \frac{7.5}{7.7} \times \frac{1}{.31}\right)$ = 18400 #/□″

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CALCULATIONS FOR

Design of Goko-Bashi for Kioto Prefecture.

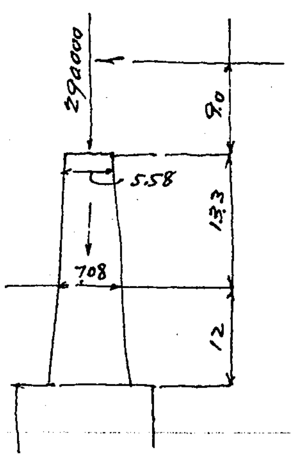
Try Reinforcing bars 20- 3/4"Ø bars. @ .44 = 8.80
on both faces 17.60 □"
steel % 17.60 ÷ 7750 = 0.227%

Ecc/h = 0.76 k = 0.29 C = 0.088

fc = 3322000 / (0.088 × 7 × 7.7²) = 90800 #/□' or 630 #/□" ok

fs = 15 × 630 (1 - 7.5/7.7 × 1/.29) = 22200 #/□" ok

Reinforcement in the shaft 12' above bottom of shaft.



				AF		arm		
Superimposed Dead load	290.000	× .25	=	72500	×	22.3	=	1.615.000
weight of shaft assumed	140.000	× .25	=	35000	×	say 6.0	=	210.000
	430.000							1.825.000

moment of inertia of section
Concrete 0.049 × 7.08⁴ = 123.5
Steel say 7.5
131.0

fibre stress = 1825.000 × 3.54 / 131.0 = 49300 #/□' or 343 #/□"

Area of section 7.08 39.36 × 144 = 5670
Steel Equiv. concrete area say 212
5882 □"

Unit direct stress = 430.000 ÷ 5882 = 73 #/□"

Due to bending stress 343 -343
" " direct stress 73 +73
416 #/□" C -270 #/□" T

Approximate stress in Concrete and steel.
Round section assumed as square beam of 5.75' × 6.5'
Reinforcing bars assumed 13 3/4"Ø @ .44 = 5.72 2 @ 5.72 = 11.44 □" on both faces
area of beam 5.75 × 6.5 × 144 = 5375
steel %. 11.44 ÷ 5375 = .213 %

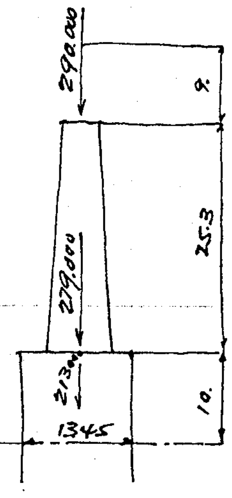
Eccentricity 1825.000 ÷ 430.000 = 4.25 Ecc/h = 4.25/6.5 = .653 k = .31
C = 0.097

Fibre stress = 1825000 / (0.097 × 5.75 × 6.5²) = 77400 #/□' or 538 #/□"

fs = 15 × 538 (1 - 6.0/6.5 × 1/.31) = 16000 #/□"

Reinforcing bars can be reduced somewhat in this section.

Reinforcement in the well 10' below top of well. dia = 13.45



weight of well. 13.45' dia area = 142.0 □' × 10 = 1420
weight = 1420 × 150 = say 213.000 #

						arm		moment
Superimposed load	290.000	× 0.25	=	72500	×	44.3	=	3220.000
shaft.	279.000	× 0.25	=	69800	×	22.0	=	1540.000
well.	213.000	× 0.25	=	53200	×	5.0	=	266.000
	782.000 #							5.026.000

moment of inertia of Concrete section.
As ring. 13.45' dia 0.049 × (13.45⁴ - 10.17⁴) = 1080

fibre stress = 5026.000 × 6.72 / 1080 = 31300 #/□' or 217 #/□"

Design of Gokō-Bashi for Kioto Prefecture.

Concrete area in ring - $60.9 \cdot 144 = 8770\square''$
Unit direct stress = $782.000 \div 8770 = 88^{\#}/\square''$

Due to bending stress	217	-217
" " direct stress	88	88
	$305^{\#}/\square''$ C	$129^{\#}/\square''$ T.

Neglecting tension in concrete
Approximate stress in well.
Eccentricity $5026000 \div 782.000 = 6.43'$

Radius of ring - 13.45 / 10.17 thickness of ring = 1.64' = 19.7"

$23.62 \div 2 = 11.81' \div 2 = 5.90'$ $\quad Ecc/r = \frac{6.43}{5.90} = 1.09$

Reinforcing bars. 96 - 3/4"φ @ .44 = 42.3 □" % 423 ÷ 8770 = .477 %.

Refer to Principles of reinforced concrete construction, Turneaure and Maurer P. 408.

$m = 3.90$

Stress in concrete $fc = 3.90 \cdot 88 = 343^{\#}/\square''$

Stress in steel $fs = 343 \cdot 24$ about $= 8250^{\#}/\square''$

$~~f =~~ fc + (fc + fs/n)t/4r$

fibre stress in concrete $= 343 + \frac{(343 + 550)\,19.7}{4 \cdot 5.90 \cdot 12} = 405^{\#}/\square''$

steel stress $= 405 \cdot \cancel{825}\ 24 = 9730^{\#}/\square''$ about.

Moment at bottom of well. 5.420.000 '#

Assumed section and reinforcing bars are ample for stresses.
Reinforcing bars will be reduced somewhat ~~to~~ ~~increase~~ increasing unit stress of the same.

Reinforcement in the well shell during sinking due to earth pressure.
Side pressure on well
Temporary Earth pressure during sinking well shall be figured by the following formula see p 120 "Side pressure for temporary trench work" Ketchum's Walls, Bins, and Elevators.

$$L = \frac{wb}{2\mu}\left(1 - e^{-\frac{2k\mu y}{b}}\right) \qquad V = \frac{wb}{2k\mu}\left(1 - e^{-\frac{2k\mu y}{b}}\right)$$

where L = Lateral unit pressure in lbs per sq ft at depth y
V = Vertical unit pressure in lbs per sq ft at depth y
w = weight of Earth in lbs per cubic ft.
ϕ = Angle of repose of Earth
μ = $\tan\phi$ Coefficient of friction of Earth on Earth
b = the distance in ft that the Earth breaks around the well.
ϕ' = Angle of friction of earth on the surface of well.

Assume $\phi = \phi' = 30°$ $\quad k = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1}{3}$ $\quad b = 15$ $\quad w = 100^{\#}$ per cubic ft.

From curve prepared by the same author.

For	60'	$1100^{\#}/\square'$
	50'	960 "
	40	880 "
	30'	740 "
	20'	580 "
	10'	340 "

Let us the following assumption of stress in the ring.
Side pressure from one side only
moment will be $= \frac{1}{6} wl^2$
where w = unit side pressure
l = diameter of ring.

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CALCULATIONS FOR

Design of Gokō-Bashi for Kioto-Prefecture.

at 60' of Earth $m = \frac{1}{6} \cdot 1100 \cdot 11.81^2 = 9600^{\#}$

Thickness of wall $1.64 \times 12 = 19.7''$ depth to reinf. say $18'' + 3''$

steel area $= \frac{9600 \times 12}{\frac{7}{8} \cdot \frac{18}{21} \cdot 17000} = 0.\overset{37}{43}\ \square''$ per ft.

at 50' of Earth

$m = \frac{1}{6} \cdot 960 \cdot 11.81^2 = 9200^{\#}$

at 40' $m = \cdot 880$ " $= 8450^{\#}$

at 30' $m = \cdot 740$ " $= 7100''$

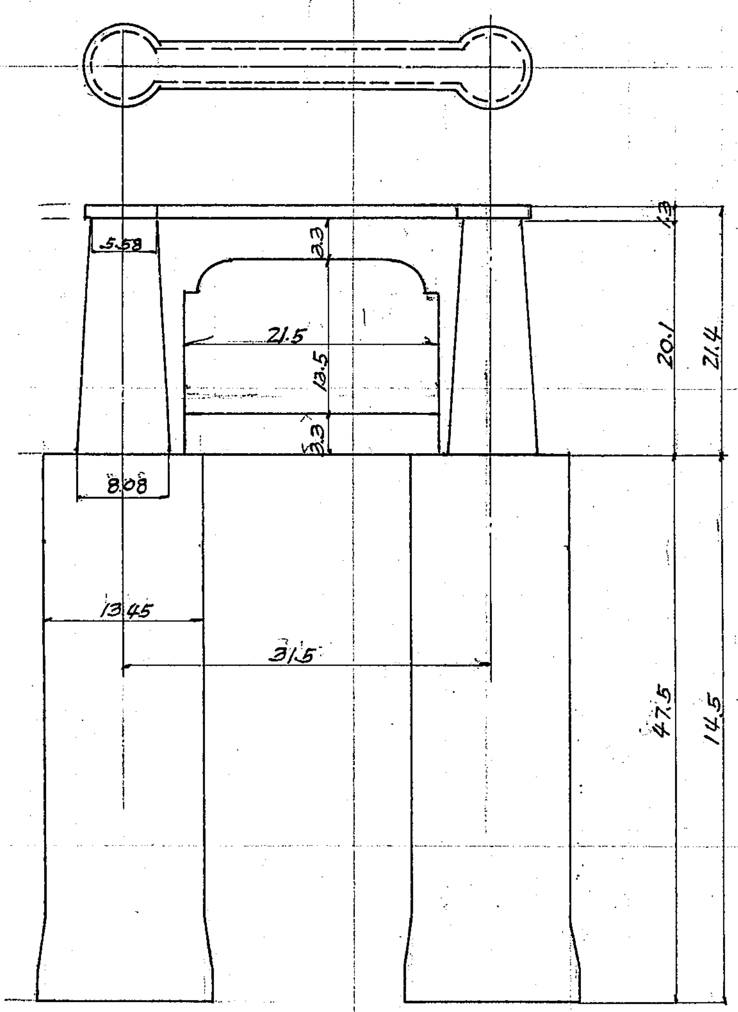
at 20' $m = \cdot 580$ " $= 5560''$

at 10' $m = \cdot 340$ " $= 3260''$

at 50' steel area $= \frac{9200 \times 12}{\frac{7}{8} \cdot 18 \cdot 17000} = 0.412\ \square''$ per ft strip

at 40	steel area =	= 0.380 □"
" 30	" =	= 0.320 □"
" 20	" =	= 0.250 □"
" 10	" =	= 0.146 □"

Design of Piers No 1 to 9 inclusive.



coping same as for Pier No 10. = 224.10 cu ft.

volume of shaft

top area $= 5.58'^{\phi} = 24.45$

$8.08'^{\phi} = \underline{51.27}$

$75.72 \div 2 = 37.86$

vol $= 37.86 \times 20.1 = 760$

$2 @ 760 = 1520$ cubic ft.

volume of web.

$13.5 \times 21.5 = 290.0$

Less $\underline{17.1}$

$272.9 \times 2' = 545.8$ cu ft.

between shaft. moulding

area $25 \times 20.1 = 502.5$

$- \underline{272.9}$

$229.6 \times 3' = \underline{689.0}$

1234.8 cubic ft

coping . 224.10

shafts 1520.00

web. $\underline{1234.80}$

$2978.90\ @ 150^{\#} = 446\,000^{\#}$

well shell. 13.45^{ϕ} 142.0

10.17^{ϕ} $\underline{81.1}$

$60.9 \times 47.5 = 2900\ @ 150^{\#} = 435.000$

Top and bottom filling 10' each $162.2\ @ 140^{\#}$ 227.000

Intermediate filling $81.1 \times 27.5 = 223.0\ @ 100$ $\underline{223.000}$

$885.000 \times 2 = 1,770.000^{\#}$

Bearing Pressure at bottom of base

superimposed Dead Load 580.000

Live Load $\underline{380.000}$

960.000

shaft 446.000

well. $\underline{1770.000}$

$3176.000^{\#}$

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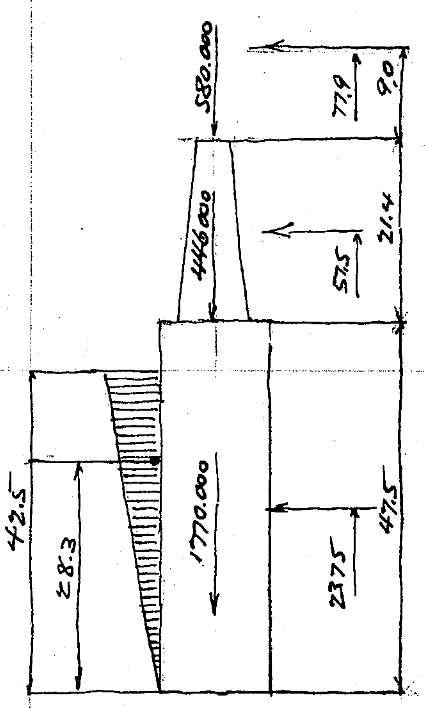
CALCULATIONS FOR

Design of Goko-Bashi for Kioto-Prefecture.

Surface friction of well assumed 250 #/□' Circumference of well 2@42.2 = 84.4'
844 @250# = 21100# per lin ft of depth
In ground. assumed 42.5' 21100 × 42.5 = 887.000#
Load at bottom of base. 3176.000
- 887.000
2,289.000 #

Base area 2@154 = 308 □'
Unit pressure = 2289.000 ÷ 308 = 7430 #/□' or 332 tons/□'

Bearing Pressure at bottom of base during Earthquake.
Seismic force = 2500 mm/sec²



				Hor. F.		Arm		moment
superimposed DL.	580.000	× 0.25	=	145000	×	77.9	=	11.300.000
weight of shaft	446.000	× 0.25	=	111000	×	57.5	=	6390.000
weight of well.	1770.000	× 0.25	=	442000	×	23.75	=	10.500.000
	2796.000			698.000#				28,190,000
		Less		698.000	×	28.3	=	19.750.000
								8,440,000 '#

Arm for frictional Couple = 13.45 × .64 = 8.6
Total Friction say 887000 443500 × 8.6 = 3820.000
4,620,000 '#

Moment of inertia of bottom area 2 @ 1885 = 3770 (')⁴

Fibre Stress = $\frac{4620.000 \times 7}{3770}$ = 8570 #/□' or 3.82 tons/□'

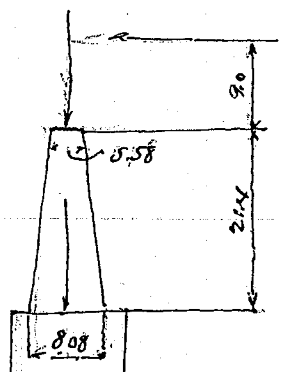
Direct load 2796.000
less friction 887.000
1909.000

Unit pressure = 1909000 ÷ 308 = 6210 #/□' or 2.77 tons/□'

Summary for bearing pressure

Due to Direct load	2.77	2.77
" " Bending	3.82	-3.82
	6.59 tons/□'	1.05 tons/□'

Reinforcement in shaft due to Earthquake.



superimposed Dead load on one shaft 580.000 ÷ 2 = 290.000#
weight of shaft 446000 ÷ 2 = 223.000#

Superimposed DL.	290.000	× 0.25	=	72500	×	30.4	=	2200.000
weight of shaft	223.000	× 0.25	=	55800	×	10 say	=	558.000
	513.000							2758.000 '#

Moment of inertia of bottom section of shaft.
Concrete 0.049 × 8.08⁴ = 210
Reinforcement say 10.
220.

32 - 3/4"φ bars - 14.10"
0.098 □'

Fibre stress = $\frac{2758000 \times 4.04}{220.}$ = 50600 #/□' or 352 #/□"

Equivalent Concrete area of section
Concrete 8.08' dia. 51.27 × 144 = 7400
Steel 14.1 × 15 = 212
7612

Unit direct stress 513.000 ÷ 7612 = 67. #/□"

Due to bending	352	352
" " direct load	67	67
	419 #/□" C	285 #/□" T

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CALCULATIONS FOR

Design of Goko-Bashi for Kioto-Prefecture.

Approximate stress in concrete and steel neglecting tension in concrete.
Round section assumed as square beam of 6.5' × 7.2'
Reinforcing bars 13-3/4"ϕ @ .44 = 5.72 2 @ 5.72 = 11.44 □" assumed

Concrete area say 6.5 × 7.2 × 144 = 6750 □" 11.44 ÷ 6750 = 0.17 %.

$\Sigma 0$ = 2758.000 ÷ 513000 = .538 ε/h = 5.35 ÷ 7.2 = 0.747 k = .26 C = 0.079

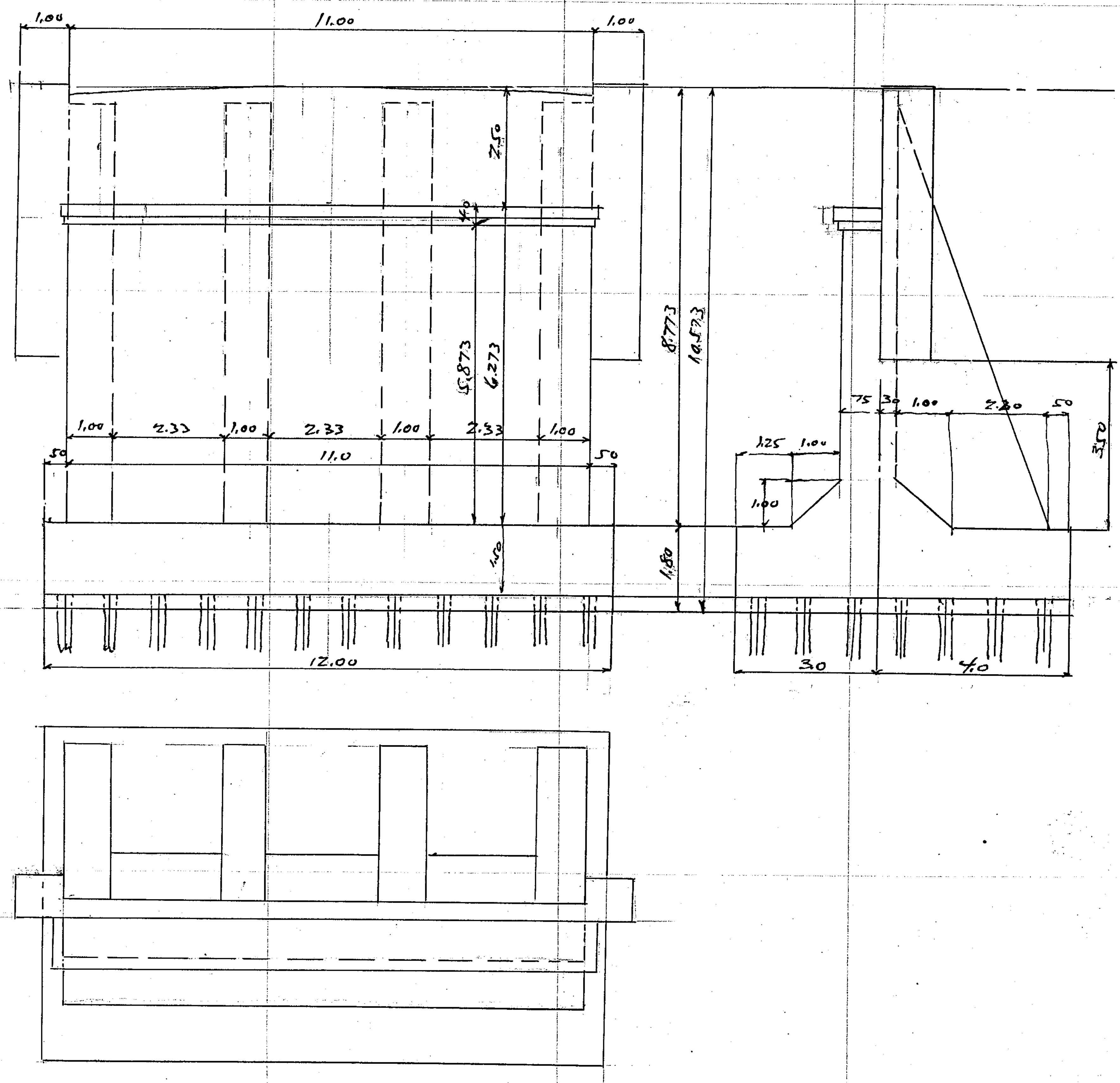
Fibre stress = $\frac{2758.000}{0.079 \times 6.5 \times 7.2^2}$ = 103600 #/□' or 720 #/□"

Steel stress = $15 \times 720 \left(1 - \frac{7.0}{7.2} \times \frac{1}{.26}\right)$ = 29600 #/□"

Reinforcement in the well. Use similar reinforcement as shown for Piers 10-21 see page 47 and 48.

Reinforcement in the well due to earth pressure during sinking same as for shown on page 48 and 49. according to depth of earth.

Design of abutment



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CALCULATIONS FOR

Design of Yoko-Bashi for Kioto Prefecture

Concrete in abutment and weight

Base	23' · 39.4' · 5.9 =	5340 cubic ft.	@150# =	800.000#	1.
Coping say	1.3 · 3.12 · 37.4 =	151		22600	2
Shaft under coping	3.44 · 19.25 · 36 =	2380		357.000	3
Filler at front	3.28 · 3.28/2 · 36 =	193		29.000	4
" " rear	do · 23.=	124		18600	5
Parapet wall	9.50 · 1.00 · 36 =	342		51300	6
Under post	2-3.28 · 32.8 × 17.2 =	370		55500	7
Buttress	4-3.28 · 10.5 · 27.8/2 =	1920		288000	8
Pedestal lamp post		10820	say	30.000#	9

Moment about Heel.

	weight	arm		arm	moment about base.
1	800.000#	11.50'	= 9200.000	2.95	= 2360.000
2	22600	14.60	330.000	25.80	583.000
3	357.000	14.35	5120.000	15.50	5530.000
4	29.000	16.65	483.000	7.00	203.000
5	18600	11.00	204.000	7.00	130.000
6	51300	12.60	646.000	30.00	1540.000
7	55500	11.50	638.000	26.00	1440.000
8	288.000	8.62	2480.000	15.20	4375.000
9	30.000	11.50	345.000	40.00	1200.000
	1652000	11.70'	19446.000	10.50	17361.000
		3.57 m		3.20 m	

Earth fill at Rear	12.10 · 28.80 · 36 =	12550		
	Less concrete	2044		
		10506 cubic ft @100 =	1.050.600#	
Rear Outsides.	12.10 · 28.80 · 3.3 =	1150 @100 =	115.000#	
Earth fill at front	13.0 say · 7.4 · 36.0 =	3460		
	less concrete	193		
		3267 cubic ft @100 =	326.700	
Front outsides	13.0 · 10.8 · 3.3. =	464 " " @100 =	46400	

Moment about Heel

	arm		
1050.600	· 6.00	=	6300.000
115.000	· 6.00	=	690.000
326.000	· 19.30	=	6300.000
46400	· 17.55	=	639.000
1538700			13.929.000'#

Superimposed Load

				moment about Heel.
Dead Load	103.000#	· 14.30	=	1,470.000
Live Load	157.000	· 14.30	=	2240.000
	260.000			3710.000

Summary

Concrete in abutment	1652.000	· 11.70	=	19.446.000
Earth fill	1538.700			13 929.000
Superimposed loads	260.000			3 710.000
	3440.700	10.9		37,085,000
		3.32 metres		

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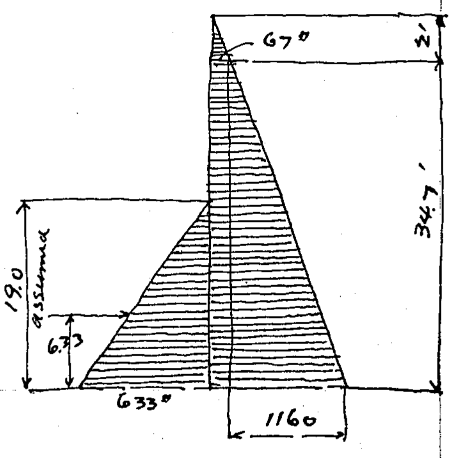
CALCULATIONS FOR

Design of Goko-Bashi for Kioto-Prefecture.

Earth Pressure — Horizontal Pressure assumed 1/3



Pressure from rear

67 × 34.7	=	2320	× 17.35	=	40.200
1160 × 34.7/2	=	20100	× 11.60	=	233.000
		22420			273.200

Pressure from front

633 × 19/2 = 6000 × 6.33 = 38.000 "

For 39.4' wide — Total Hor. moment

273.200 × 39.4' = 10.750.000
38.000 × 39.4 = 1.500.000
9250.000 '#

Summary for moment

Moment due to vertical load 37.085.000
" " " Hor. Pressure 9250.000
46335.000

Resultant arm = 46335.000 ÷ 3440700 = 13.45' or 4.03 meter.
call this ~~13.5~~

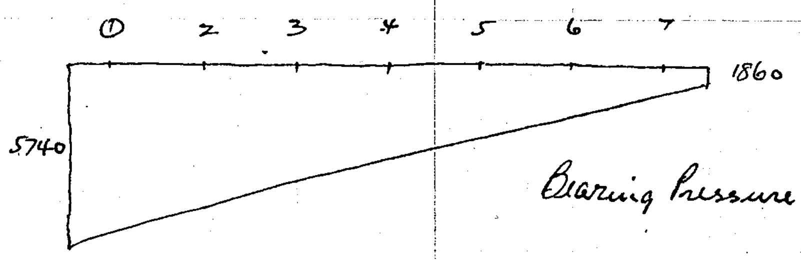
Base. 23.0' ~~no~~ eccentricity 13.45 − 11.50 = 1.95

Bearing Pressure = $\frac{3440.700}{23 \times 39.4}\left(1 \pm \frac{6 \times 1.95}{23}\right)$ = 5740 #/□' at toe
1860 #/□' at Heel
1.51
.49

without earth pressure

Resultant arm = 37.085.000 ÷ 3440700 = 10.8' or 3.29 m
Eccentricity = 11.5 − 10.8 = 0.7'

Bearing Pressure = $\frac{3440.700}{23 \times 39.4}\left(1 \pm \frac{6 \times 0.7}{23}\right)$ = 4500 #/□' or 3100 #/□' at toe



5740 − 1860 = 3880 "

Bearing Pressure

					#/□'	For 1 meter □' #	tons
7	278	+ 1860	=	2138		23000	10.3 tons
6	830	"	=	2690		29000	13.0 "
5	1390	"	=	3250		35000	15.6 "
4	1950	"	=	3810		41000	18.3 "
3	2500	"	=	4360		47000	21.0 "
2	3060	"	=	4920		53000	23.6 "
1	3610	"	=	5470		59000	26.3 "

Driving pile per square meter max bearing pressure per pile 26.3 tons.

Max toe pressure 5740 # or 2.56 tons per square ft. Soil good enough for this load however use pilings to protect scoring.

Stability during Earthquake k = 2500 mm/sec² No Earth filling
Horizontal moment body of abutment 17.361.000 × 0.25 = 4.350.000
Superimposed Dead Load 103000
Hor. Force 103000 × 0.25 = 25800 #
m = 25.800 × 33.5 abt = 864.000
5214.000 '#

Moment due to load about Heel.

Abutment 1.652.000 19.446.000
Superimposed DL. 103.000 × 14.3 = 1.475.000
1755.000
20.921.000
26135.000 '#

Resultant arm = 26135.000 ÷ 1755000 = 14.9'

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Design of Yoko-Bashi for Kioto-Prefecture.

Ecc = 14.9 - 11.5 = 3.4'

max toe pressure = $\frac{1755.000}{23 \times 39.4}$ $\times$ (1 ± $\frac{3.4 \times 6}{23}$) = 3650 #/□' at toe or 223 #/□' at Heel.

1940

Horizontal force toward rear 20.921.000
-5214.000

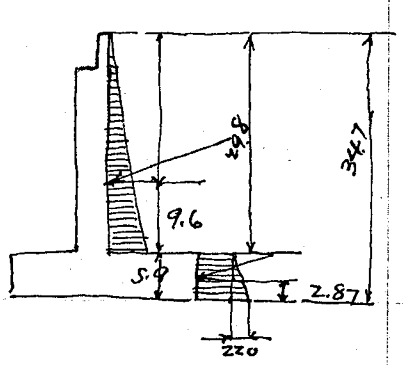
Resultant arm 15707.000 ÷ 1755.000 = 8.95'

Ecc = 11.5 - 8.95 = 2.55'

max toe pressure = $\frac{1755.000}{23 \times 39.4}$ $\times$ (1 ± $\frac{2.55 \times 6}{23}$) = 650 #/□' toe 3230 #/□' Heel.

Earth pressure from rear

h = 0.25 × 3/4 = 0.1875
angle θ = 18°0'
Coefficient for Earth pressure 0.462 × $\frac{wh^2}{2}$
sin θ = 0.309
cos θ = 0.951



46.2# × 29.8 = 1380
46.2 × 34.7 = 1600
220#

Earth pressure $\frac{1380}{2}$ × 29.8 = 20600

1380 × 5.9 =	8150	×	2.95 =	24000
$\frac{220}{2}$ × 5.9 =	650	×	1.96 =	1275
	8800		2.87'	25275

Hor. 20600 × say 39.4 = 812.000 × 0.951 = 772.000 #
Vert. 812.000 × 0.309 = 251.000 #

Hor 8800 × 39.4 = 347.000 × 0.951 = 330.000 #
Vert. 347.000 × 0.309 = 107.000 #

Vertical moment about Heel

Body of abutment	1.652.000	19.446.000
Earth	1538.000	13 929.000
Dead Load on	103.000	1.470.000
	3.293.000	34.845.000
Component	251.000 × 12.12 =	3.040.000
	107.000	000
	3,651.000#	37,885.000'#

Horizontal moment about bottom of base

abutment m 17.361.000 × 0.1875 = 3.260.000
Dead Load on 103000 × 0.1875 = 19,300
19300 × 35.0 att = 675.000

3935.000

Hor. Component 772.000 × 15.5 = 11,96.0000
330.000 × 2.87 = 946000

Hor. m 16841000 #
~~Hor.~~ vertical m 37885000
54,726.000 '#

Resultant arm 54726.000 ÷ 3651.000 = 15.00'

Eccentricity 15.00 - 11.50 = 3.50'

Soil Pressure = $\frac{3651.000}{23 \times 39.4}$ (1 ± $\frac{3.50 \times 6}{23}$) = 7700 #/□' at toe 3.44 tons/□'
355 #/□' Heel

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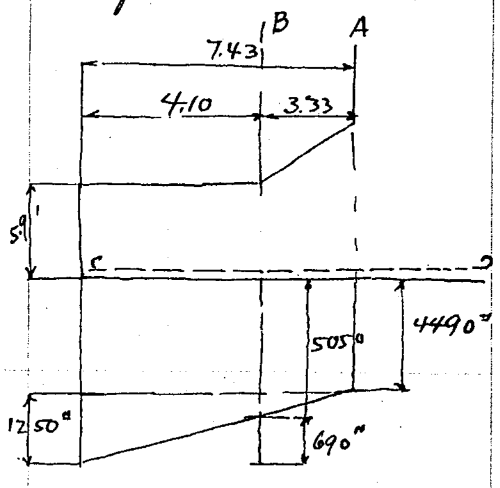
CALCULATIONS FOR

Design of Goko-Bashi for Kioto-Prefecture.

Details of design.
Reinforcement in toe.



Own weight of beam $5.9 \times 150 = 885^{\#}$ per lin ft. assumed
$4490 - 885^{\#} =$ say $3600^{\#}$

Section at A.
moment $3600 \times \frac{7.43^2}{2} = 99200$
$\frac{1250}{2} \cdot 7.43 = 4650$
$4650 \times 4.95 = \underline{23000}$
$122200^{'\#}$

Depth say 103.5"
fibre stress $= \frac{122200 \times 12 \times 6}{12 \times 103.5^2} = 68.5^{\#}/\square''$

Steel $= \frac{122200 \times 12}{\frac{7}{8} \times 103.5 \times 17000} = 0.950\square''$ per ft.
7/8"ϕ – 7" spacing $= 1.030\square''$ per ft.

Section B. upward pressure $5050 - 885 = 4165^{\#}$ uniform pressure.
moment $4165 \times \frac{4.10^2}{2} = 35.000$
$690 \times \frac{4.10}{2} = 1410$
$1410 \times 2.73 = \underline{3850}^{'\#}$
$38850^{'\#}$

Depth assumed 63.5" fibre stress $= \frac{38850 \times 12 \times 6}{12 \times 63.5^2} = 57.8^{\#}/\square''$

Steel area $= \frac{38850 \times 12}{\frac{7}{8} \cdot 63.5 \times 17000} = 0.493\square''$ per ft.
7/8"ϕ bars 12" centers – $0.60\square''$

Reinforcement in rear span length 10.95'
Depth of fill – 28.8
Surcharge 2.0
30.8' @ 100 = 3080
beam say 885
$3965^{\#}$ per lin. ft.

$m = \frac{1}{12} \times 3965 \times 10.95^2 = 39600$
steel area $= \frac{39600 \times 12}{\frac{7}{8} \cdot 63.5 \times 17000} = 0.504\square''$ per ft
also 7/8"ϕ bars 12" centers about.

Reinforcement in Front wall. Earth pressure
at 25.5 × 33# = ~~850~~ 850# per sq ft.
20.0 " " = 660
15.0 " " = 500
10.0 " " = 330

Depth say 3.5' or 42" Effective depth say 39".

at 25.5 ~~steel area~~ m $= \frac{1}{12} \times 850 \times 10.95^2 = 8500^{'\#}$ steel $= \frac{8500 \times 12}{\frac{7}{8} \cdot 39 \cdot 17000} = 0.176\square''$
20.0 660 " = 6600 0.137
15.0 500 " = 5000 0.103
10.0 300 " = 3000 0.070
1/2"ϕ bars 12" centers $= 0.192\square''$ per ft.

Reinforcement in Parapet wall.
Surcharge assumed 3.0' Earth pressure $= (8+3) \times 33 = 363^{\#}$ per sq ft.
at bottom moment $= \frac{1}{2} \times 363 \times 10.95^2 = 3630^{'\#}$
Effective depth say 10"

Design of Yoko-Bashi for Kioto Prefecture

Steel area = $\frac{3630 \times 12}{7/8 \times 10'' \times 17000}$ = 0.293 □" per ft.

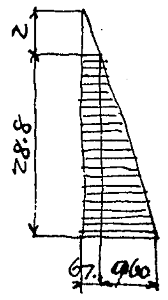
at 5' from top. Surcharge 3.0' Earth pressure = 264#/□'

moment = $\frac{1}{12} \times 264 \times 10.95^2$ = 2640''#

Steel area = $\frac{2640 \times 12}{7/8 \times 10 \times 17000}$ = 0.214 □" per ft.

The bottom portion of parapet wall will be carried by 2 ways (vertical and Hor. reinforcement). Use ½"φ bars 12" centers about

Reinforcement in Buttress wall in rear.
Depth of fill 28.8 surcharge 2.0'



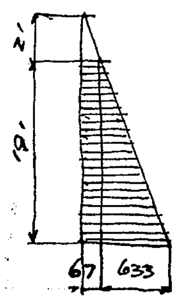
67 × 28.8 = 1930 moment 1930 . 14.4 = 27800
$\frac{960 \times 28.8}{2}$ = 13800 13800 . 9.6 = 132500
160.300''#

For 10.95' wide m = 160.300 × 10.95 = 1.750.000''#
Depth assumed 13.1' arm say 12.5' about

Steel area reqd = $\frac{1750.000}{7/8 \times 12.5 \times 17000}$ = 9.40 □"

Use 16-7/8"φ bars = 9.60 □"

at 19'

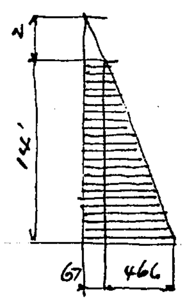


Pressure = 67 × 19 = 1275 moment 1275 . 9.5 = 12.100
$\frac{633}{2} \times 19$ = 6000 6000 . 6.33 = 38.000
50100

For one buttress 50100 × 10.95 = 550.000''#
arm say 9.2'
~~steel area~~ = $\frac{550.000}{7/8 \times 9.2 \times 17000}$ = 4.02 □"

Use 8 - 7/8"φ = 4.8 □"

at 14'



Pressure 67 × 14 = 937 937 . 7.0 = 6550
$\frac{466}{2} \times 14$ = 3260 3260 . 4.67 = 15200
21750''#

For one buttress = 21750 × 10.95 = 238,000''#

Steel area = $\frac{238,000}{7/8 \times 7.2 \times 17000}$ = 2.22 □"

4 - 7/8"φ bars = 2.40 □"

Reinforcement in Outside buttress wall.
width ~~to~~ carry earth pressure = ~~8.8'~~ 7.15 ~~8.8~~ 7.15 : 10.95 = 65%.

at 28.8'	steel area reqd =	9.4	× 65%	=	6.10 □"
at 19.0'	" " "	4.02	"	=	2.62 □"
at 14.0'	" " "	222	.	=	1.45 □"

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MADE BY I. Matano DATE 2-1-14 FILE NO.
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CALCULATIONS FOR

List of Materials for Goko- & Yodo-o-Hashi for Kioto-Prefecture.

MAIN GIRDER G1 R/L — 2 Req'd for Yodo, 4 Req'd for Goko

1	Web Pl.	66" × 1/2"	18'-0 1/2"	@ 112.2	2,024	Main Section
2	Ls	6" × 6" × 1/2"	40'-4 3/32"	@ 19.6	1,581	Main Section
2	"	"	38'-6 9/16"	"	1,511	Main Section
1	Web Pl.	66" × 1/2"	22'-1 1/32"	@ 112.2	2,478	Main Section
2	Cov. Pls.	14" × 5/8"	24'-4 3/8"	@ 29.75	1,450	Main Section
4	Ls	5" × 3 1/2" × 3/8"	5'-6"	@ 10.4	229	Stiffener
2	"	5" × 5" × 3/8"	2'-2 3/8"	@ 12.3	54	"
14	"	5" × 3 1/2" × 3/8"	5'-5 1/2"	@ 10.4	795	"
6	"	5" × 4" × 3/8"	5'-5 1/2"	@ 11.0	360	"
2	Pls.	19" × 1/2"	5'-11 5/8"	32.3	386	
14	Fills	3 1/2" × 1/2"	4'-6 1/4"	5.95	377	
3	"	4" × 1/2"	4'-6 1/4"	6.8	92	
3	"	8 3/4" × 1/2"	4'-6 1/4"	14.88	202	
1	L	6" × 4" × 3/8"	1'-6 1/4"	12.3	19	
1	Fill.	6" × 3/8"	0'-11 3/4"	7.65	7	
1	L	6" × 4" × 3/8"	1'-1 1/2"	12.3	14	
1	Fill.	12" × 1/2"	1'-7"	20.4	32	
2	Ls	6" × 6" × 1/2"	2'-3 1/4"	19.6	89	
1	Washer	3 φ × 1/2"		1.0	1	
2	Ls	6" × 6" × 3/8"	1'-2"	12.3	29	
1	L	"	1'-1 1/2"	"	14	
1	"	"	1'-3"	"	15	
1	Pls	20" × 5/16"	2'-8"	21.25	57	Lateral Plates
1	"	19" × 5/16"	2'-3 1/4"	20.19	41	Lateral Plates
1	"	19 × 5/16"	2'-3 1/4"	20.19	46	Lateral Plates
900	Shop Rivets	7/8 φ	grip abt. 1 3/8"	.73	657	
25	Field Rivets	"	1 1/2"	.77	17	
95	" "	3/4 φ	3/4"	.43	41	
4	Pls	6" × 1/2	2'-0 1/2"	10.2	83	
2	"	12 1/2" × 1/2"	3'-5 3/4"	21.25	222	
					12,923	× 2 = 25,846 for Yodo
					"	× 4 = 51,692 " Goko

SPLICE SP1 — 2 Req'd. for Yodo, 4 " " Goko

2	Ls	6" × 6" × 5/8"	3'-5 1/2"	24.2	167
2	"	"	3'-6"	"	169
4	Fills	5 1/4" × 1/8	1'-8 3/4"	2.231	15
4	"		1'-9"		16
2	Pls	6" × 5/8"	2'-0 1/2"	12.75	52
2	"	6 1/2" × 5/8"	2'-1"	13.81	58
2	"	12 1/2" × 1/2"	3'-6"	21.25	149
210	Field Rivets	7/8 φ	grip abt. 2"	.88	185
					811 × 2 = 1,622 For Yodo
					" × 4 = 3244 " Goko

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MADE BY J. Matano DATE 2-1-15 FILE NO. 3

CHECKED BY M. Kojima DATE " PAGE NO M 2

CALCULATIONS FOR

List of Materials for Goko- & Yodo-O-Hashi for Kioto-Prefecture.

MAIN GIRDER $G2^R_L$ $G2A^R_L$ — 2 Req'd for Yodo, 4 " " Goko

2	Ls	6"×6"×5/8"	26'-6 17/32"	@ 24.18	1,284	Main Section
2	"	"	27'-2"	"	1,314	
1	Web.Pl.	97"×1/2"	26'-6 17/32"	@ 164.9	4,378	 -650
1	Cov.Pl.	14"×5/8"	27'-4 5/8"	@ 28.75	787	
1	"	"	30'-7 5/8"	"	881	
1	"	"	28'-5"	"	817	
1	"	"	31'-8	"	910	
2	Ls	6"×6"×1/2"	12'-8 15/32"	@ 19.56	497	
2	"	"	14'-7"	"	570	
1	Web Pl.	75"×1/2"	14'-3 23/32"	@ 127.5	1,825	 -228
2	Ls	5×3½×3/8"	5'-6¼"	@ 10.37	115	
2	"	"	6'-2½"	"	129	
4	"	"	6'-9"	"	280	
4	"	"	7'-5"	"	308	
2	"	"	6'-1½"	"	127	
2	"	"	5'-6¼"	"	115	
2	"	"	5'-5½"	"	113	
4	"	5"×4"×3/8"	5'-10"	11.0	257	
2	"	"	5'-5½"	"	120	
4	"	5"×5"×3/8"	7'-11¼"	12.27	390	
2	"	5"×4"×3/8"	7'-11¼"	11.0	175	
4	"	3½"×3½"×3/8"	2'-7½"	8.5	89	
2	"	"	1'-11¾"	"	34	
1	L	"	2'-4½"	"	20	
1	"	"	2'-1"	"	18	
2	Fills	3½"×5/8"	4'-7¼"	7.44	69	
1	Fill	4"×5/8"	4'-11"	8.5	42	
1	"	8¾"×5/8"	4'-11"	18.59	91	
2	Fills	3½"×5/8"	5'-3½"	7.44	79	
4	"	"	5'-10"	"	174	
4	"	"	6'-6"	"	193	
2	"	19½"×5/8"	7'-0½"	41.44	584	
1	Fill	4"×1/2"	4'-11"	6.8	33	
1	"	8¾"×1/2"	4'-11"	14.88	73	
2	Fills	3½"×1/2"	4'-7¾"	5.95	55	
2	"	"	4'-6½"	"	54	
2	Ls	6"×6"×5/8"	3'-2"	24.18	153	Splice
2	"	"	3'-3	"	157	
4	Fills	5¼"×1/8"	1'-7"	2.231	14	
4	"	"	1'-7½"	"	15	
2	Pls	6"×5/8"	2'-1"	12.75	53	
2	"	6½"×5/8"	2'-2"	13.81	60	
2	"	13"×5/8"	4'-3"	27.63	235	
2	Fills	3½"×5/8"	5'-2½"	7.44	77	
1	L	6"×4"×3/8"	1'-0"	12.27	12	
2	Ls	6"×6"×1/2"	1'-7"	19.56	62	
1	Pl.	22½"×3/8"	2'-9"	28.69	79	Bracket Pl. 
2	Ls	4"×3½"×3/8"	1'-6"	9.08	27	
1	Pl.	34½"×7/16"	5'-0¾"	36.66	186	Lateral Pls.
1	"	21¼"×3/8"	2'-2"	27.095	59	
2	Ls	6"×4"×3/8"	2'-11¼"	12.27	72	
1	Pl.	11¾"×3/8"	2'-9¾"	14.98	42	

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CALCULATIONS FOR

List of Materials for Goko- & Yodo-o-Hashi for Kioto-Prefecture.

1250	Shop Rivets	3/8 φ	grip ab't.	2"	@ .88	1100
120	Field "	"	" "	1½"	@ .77	92
8	Shop "	3/4 φ	" "	1"	@ .46	4
70	Field "	"	" "	3/4"	@ .43	30

19,529
− 878
18,651 × 2 = 37,302 for Yodo
" × 4 = 74,604 " Goko

FOR G2 R/L — 2 Req'd. for Yodo, 2 " " Goko

2	Pls	49" × ½"	4'-6¼"	@ 83.3	753
8	Ls	5" × 3½" × 3/8"	2'-2½"	@ 10.4	184
2	Fills	19" × ½"	1'-9"	@ 32.3	113
45	Shop Rivets	3/8 φ grip	3¼"	@ 1.11	50

− 160

1100
− 160
940 × 2 = 1,880 for Yodo
" × 2 = 1,880 " Goko

FOR G2A R/L — 0 Req'd for Yodo, 2 " " Goko

2	Pls.	49" × ½"	4'-6¼"	@ 83.3	753
8	Ls	5" × 3½" × 3/8"	2'-9¼"	@ 10.4	231
2	Fills	19" × ½"	2'-3¾"	@ 32.3	149
55	Shop Rivets	3/8 φ grip.	3¼	@ 1.11	61

− 160

1194
− 160
1034 × 0 = 0 for Yodo
" × 2 = 2068 " Goko

MAIN GIRDER G3 R/L — 8 Req'd for Yodo, 12 " " Goko

4	Ls	6" × 6" × ½"	11'-1 27/32"	@ 19.6	874	Main section
2	"	"	9'-6 23/32"	"	375	Main section
4	"	"	29'-1 1/16"	"	2,285	Main section
2	Web Pls.	66" × 3/8"	25'-8¾"	84.2	4,333	Main section
2	Ls	6" × 6" × ½"	9'-8 3/32"	19.6	381	Main section
2	Cov. Pls.	14" × ½"	32'-4 3/8"	23.8	1,543	Main section
8	Ls	6" × 6" × ½"	2'-7½"	19.6	412	
10	"	5" × 4" × 3/8"	5'-5½"	11.0	600	
16	"	5" × 3½" × 3/8"	5'-5½"	10.4	908	
16	"	"	1'-7 3/8"	"	269	
4	"	6" × 6" × 3/4"	1'-7"	28.7	182	
4	Pls.	49" × ½"	4'-6¼"	83.3	1,506	
16	Fills	3½" × ½"	4'-6¼"	5.95	430	
2	"	4" × ½"	4'-6¼"	6.8	61	
2	"	8¾" × ½"	4'-6¼"	14.88	135	
2	Pls.	6" × ½"	2'-3⅜"	10.2	46	splice
2	"	8" × ½"	2'-3⅜"	13.6	61	splice
2	"	15⅛" × ½"	3'-4⅜"	25.7	172	splice
2	Ls	6" × 4" × 3/8"	1'-0"	12.3	25	
1	"	"	1'-3"	"	15	
1	"	"	1'-6"	"	18	

−160

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MADE BY I. Matano DATE 2-1-15 FILE NO
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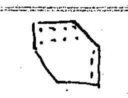
CALCULATIONS FOR

List of Materials for Goko- & Yodo-O-Hashi for Kioto-Prefecture

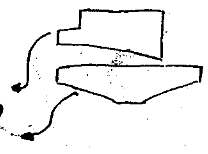
2	Pls.	21" × 3/8"	2'-0 1/2"	@	26.78	109	Bracket Pls
4	Ls	4" × 3 1/2" × 3/8"	1'-6"	@	9.1	55	
2	Pls.	18" × 3/8"	1'-8 1/4"	@	22.95	77	Lateral Pls.
1	"	17" × 5/16"	2'-3 1/2"	@	18.06	41	
1395	Shop Rivets	7/8"φ	grip abt. 2"	@	.88	1225	
75	Field "	"	" 2"	@	.88	66	
10	Shop "	3/4"φ	" 1"	@	.46	5	
95	Field "	"	" 3/4"	@	.43	41	
						16,253	
						- 160	



16,093 × 8 = 128,744 for Yodo
" × 12 = 193,116 " Goko

MAIN GIRDER $G4^R_L$ $G4A^R_L$ — 16 Req'd for Yodo; 20 " " Goko

2	Ls	6" × 6" × 1/2"	12'-10 19/32"	@	19.6	505	Main Section
2	"	"	14'-7"		"	572	
2	"	6" × 6" × 5/8"	26'-6 17/32"	@	24.2	1285	
2	"	"	27'-2"		"	1,315	
1	Web Pl.	75" × 1/2"	14'-3 23/32"		127.5	1,825	
1	"	97" × 1/2"	26'-6 17/32"		164.9	4,378	
1	Cov. Pl.	14" × 5/8"	30'-7 5/8"		29.75	911	
1	"	"	22'-7 7/8"		"	674	
1	"	"	31'-8"		"	942	
1	"	"	23'-8 1/2"		"	705	
2	Ls	5" × 3 1/2" × 3/8"	5'-6 1/4"		10.4	115	
2	"	"	6'-2 1/2"		"	129	
4	"	"	6'-9"		"	281	
4	"	"	7'-5"		"	309	
2	"	"	6'-1 1/2"		"	127	
2	"	"	5'-6 3/4"		"	116	
2	"	"	5'-5 1/2"		"	114	
4	"	5" × 4" × 3/8"	5'-10"		11.0	257	
2	"	"	5'-5 1/2"		"	120	
4	"	5" × 5" × 3/8"	7'-11 1/4"		12.3	391	
2	"	5" × 4" × 3/8"	7'-11 1/4"		11.0	175	
4	"	3 1/2" × 3 1/2" × 3/8"	2'-7 1/2"		8.5	89	
2	"	"	1'-11 3/4"		"	34	
1	"	"	2'-4 1/2"		"	20	
1	"	"	2'-1"		"	18	
2	Fills	3 1/2" × 5/8"	4'-7 1/4"		7.44	69	
1	"	4" × 5/8"	4'-11"		8.5	42	
1	"	8 3/4" × 5/8"	4'-11"		18.59	91	
2	"	3 1/2" × 5/8"	5'-3 1/2"		7.44	79	
4	"	"	5'-10"		"	174	
4	"	"	6'-6"		"	193	
2	"	19 1/2" × 5/8"	7'-0 1/2"		41.44	584	
1	"	4" × 1/2"	4'-11"		6.8	33	
1	"	8 3/4" × 1/2"	4'-11"		14.88	73	
2	"	3 1/2" × 1/2"	4'-7 3/4"		5.95	55	
2	"	"	4'-6 1/2"		"	54	
2	Ls	6" × 6" × 5/8"	3'-2"		24.2	153	Splice
2	"	"	3'-3"		"	157	
4	Fills	5 1/4" × 1/8"	1'-7"		2.231	14	



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- 650

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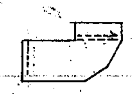
CALCULATIONS FOR

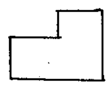
List of Materials for Goko- & Yodo-O-Hashi for Koto-Prefecture

2	Ls	6"×4"×3/8"	2'-1¼"	@ 12.27	72	
1	Pl.	11¾ × 3/8"	2-9¾	14.98	42	
4	Fills	5¼ × 1/8	1-7½	2.231	15	
2	Pls	6 × 5/8	2-1	12.75	53	
2	"	6½ × 5/8	2-2	13.81	60	
2	"	13 × 5/8	4-3	27.63	235	
2	Fills	3½ × 5/8	5-2½	7.44	77	
1	L	6×4×3/8	1-0	12.3	12	
2	Ls	6×6×½	1-7	19.6	62	
1	Pl	22½ × 3/8	2-9	28.69	79	Bracket Pl 
2	Ls	4×3½×3/8	1-6	9.1	27	
1	Pl.	21¼ × 3/8	2-2	27.095	59	Lateral Pl.
1	"	34½ × 5/16	5-3¾	36.66	195	
1250	shop Rivets	7/8ϕ Grip ab't.	2	0.88	1,100	
120	Field "	" "	1½	0.77	92	
8	shop "	¾ϕ "	1	0.46	4	
70	Field "	" "	¾	0.43	30	

19,398
\- 878
18,520 × 16 = 296,320 For YODO
" × 20 = 370,400 " GOKO

For G4 R/L

8 Rig'd For YODO
10 " " GOKO

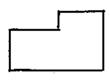


2	Pls	49 × ½	4-6¼	83.3	753	-160
8	Ls	5×3½×3/8	2-9¼	10.4	231	
2	Fills	19 × ½	2-3¾	32.3	149	
55	shop Rivets	7/8ϕ Grip 3¼		1.11	61	

1194
\- 160
1034 × 8 = 8272 For YODO
" × 10 = 10340 " GOKO

For G4A R/L

8 Rig'd For YODO
10 " " GOKO



2	Pls	49 × ½	4-6¼	83.3	753	-160
8	Ls	5×3½×3/8	2-2½	10.4	184	
2	Fills	19 × ½	1-9	32.3	113	
45	shop Rivets	7/8ϕ Grip	3¼	1.11	50	

1100
\- 160
940 × 8 = 7520 For YODO
" × 10 = 9400 " GOKO

Splice SP2

16 Rig'd For YODO
20 " " GOKO

2	Ls	6×6×5/8	3-5½	24.2	167
2	"	"	3-6	"	169
2	Pls	6 × 5/8	2-0½	12.75	52
2	"	6½ × 5/8	2-1	13.81	58
2	"	12½ × ½	3-6	21.25	149
1	Fill	14 × 5/8	4-8½	29.75	140
1	"	"	4-9	"	141
280	Field Rivets 7/8ϕ Grip ab't.		2¼	0.92	258

1,134 × 16 = 18,144 For YODO
" × 20 = 22,680 " GOKO

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SEIYU BLDG, TOKIO

MADE BY I. Matano DATE 2-1-15 FILE NO

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CALCULATIONS FOR

List of Materials for Goko- & Yodo-o-Hashi for Kioto-Prefecture.

Main Girder G5^R_L & G5A^R_L

16 Rig'd For YODO
20 〃 〃 GOKO

4	Ls	6"×6"×5/8"	29'-11 3/8" @	24.2	2,899
4	Pls.	6"×5/8"	13-2 7/8	12.75	673
1	Web pl.	66"×1/2	29-11 3/8	112.2	3360
2	Cov. Pls.	14×5/8	29-11 3/8	29.75	1,782
2	〃	〃	25-1	〃	1,492
2	〃	〃	29-8 1/8	〃	1,766
10	Ls	5×3 1/2×3/8	5-5 1/4	10.4	566
4	〃	5×4×3/8	5-5 1/4	11.	239
10	Fills.	3 1/2×5/8	4-6 1/4	7.44	336
1	Fill	4×5/8	4-6 1/4	8.5	38
1	〃	8 3/4×5/8	4-6 1/4	18.59	84
4	Pls	6×5/8	2-0 7/8	12.75	106
1	L	6×4×3/8	1-8	12.3	21
1	〃	〃	1-7 1/2	〃	20
1	Pl	21"×5/16	2-10 3/4	22.31	65 Lateral Pl.
1	Fill	4×5/8	3-5 3/4	8.5	30
1	〃	8 3/4×5/8	3-5 3/4	18.59	65
675	Shop Rivets	7/8"ϕ Grip ab't.	2 1/4	0.92	621
51	Field 〃	〃 〃	2 3/4	1.01	52
20	Field 〃	3/4"ϕ 〃	3/4	0.43	9

14,224 × 16 = 227,584 For YODO
〃 × 20 = 284,480 For GOKO

Splice Sp3

8 Rig'd For YODO
10 〃 〃 GOKO

4	Ls	6"×6"×5/8"	3-10 5/8	24.2	376
4	Pls	6×5/8	5-11 1/2	12.75	303
2	Ls	5×4×3/8	5-4	11.	117
1	Fill	8 3/4×5/8	4-5 3/4	18.59	83
2	Pls	14 3/4×5/8	1-11 1/2	31.345	123
4	〃	9×5/8	2-2 1/4	19.13	167
1	Fill	4×5/8	4-5 3/4	8.5	38
2	Cov. Pls	14×5/8	14-2 1/2	29.75	845
2	〃	〃	9-11 1/2	〃	593
2	〃	〃	4-11	〃	293
1	L	6×4×3/8	1-1	12.3	13
1	〃	〃	1-4	〃	16
1	Pl	17×5/16	2-3 1/2	18.06	41 Lateral Pl.
385	Field Rivets	7/8ϕ Grip ab't.	3	1.07	412
15	〃 〃	3/4ϕ	3/4	0.43	6

3,426 × 8 = 27,408 For YODO
〃 × 10 = 34,260 For GOKO

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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CALCULATIONS FOR

List of Materials for Goko- & Yodo-o-Hashi for Kioto-Prefecture

Main Girder G6 R/L — 2 Riq'd For YODO; 0 For GOKO

1	Web pl.	66" × 3/8"	16'-7"	@ 84.2	1,396	
2	Ls	6" × 6" × 1/2"	38'-7 1/2"	@ 19.6	1,514	Main section
2	"	"	35-2 3/4	"	1,381	Main section
1	Web Pl.	66 × 3/8"	21'-9 3/32	84.2	1,832	Main section -80
6	Ls	5 × 4 × 3/8	5-5 1/2	11.	360	
12	"	5 × 3 1/2 × 3/8	5-5 1/2	10.4	681	
4	"	"	5-6	"	229	
2	"	5 × 5 × 3/8	2-2 7/8	12.3	54	
12	Fills	3 1/2 × 1/2	4-6 1/4	5.95	323	
2	"	4 × 1/2	4-6 1/4	6.8	61	
2	"	8 3/4 × 1/2	4-6 1/4	14.88	135	
2	"	19 × 1/2	5-11 7/8	32.3	386	
1	L	6 × 4 × 3/8	1-0	12.3	12	
2	Pls	49 × 1/2	4-6 1/4	83.3	753	-160
8	Ls	5 × 3 1/2 × 3/8	1-7 3/8	10.4	134	
2	"	6 × 6 × 3/4	1-7	28.7	91	
1	"	6 × 4 × 3/8	1-1 7/8	12.3	14	
1	"	"	1-2	"	14	
1	"	"	1-1 1/2	"	14	
1	"	"	1-6 1/4	"	19	
1	Fill	6 × 3/8	0-11 3/4	7.65	7	
1	"	12 × 3/8	1-7	15.3	24	
2	Ls	6 × 6 × 1/2	2-3 1/4	19.6	89	
1	Washer	3" φ × 3/8		1	1	
1	Pl.	21 × 3/8	2-0 1/2	26.78	55	Bracket pl.
2	Ls	4 × 3 1/2 × 3/8	1-6	9.1	27	
1	Pl	18 × 3/8	1-8 1/4	22.95	39	Lateral pl.
1	"	17 × 5/16	2-3 1/4	18.06	41	Lateral pl.
1	"	20 × 5/16	2-8	21.25	57	Lateral pl.
700	Shop Rivets	7/8 φ Grip abt.	1 3/4	0.84	588	
75	Field "	" "	1 3/4	"	63	
6	Shop Rivets	3/4 φ "	1	0.46	3	
70	Field "	" "	3/4	0.43	30	
4	Pls	6" × 1/2"	1'-6 1/2"	10.2	[illegible]	[illegible]
2	"	12 1/2" × 1/2"	3'-5 3/4"	21.25	148	[illegible]

~~10,646 × 2 = 21,292 for Yodo~~

10,638 × 2 = 21,276 for Yodo.

Floor Beams FB15, FB1 & FB5 — 11 Riq'd For YODO; 14 " " GOKO

4	Ls	5 × 3 1/2 × 3/8	31-4	10.4	1,303
2	Web Pls	50 × 5/16	15-8	53.1	1,664
4	Ls	5 × 4 × 3/8	4-1 3/4	11.	182
4	Fills	5 × 3/8	3-7 1/4	6.38	92
20	Ls	4 × 3 × 5/16	4-2 1/2	7.2	606
2	"	"	4-1 3/4	"	597
2	Pls	12 × 3/8	2-6 7/8	15.3	79
4	"	6 1/4 × 3/8	1-6	7.97	48
5	"	10 1/2 × 3/8	1-3 1/2	13.39	86
460	Shop Rivets	3/4" φ Grip abt.	1 1/2	0.54	248

4,905 × 11 = 53,955 For YODO

" × 14 = 68,670 For GOKO

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

CALCULATIONS FOR

List of Materials for Goko- & Yodo-o-Hashi for Kioto-Prefecture.

Floor Beams FB2. FB2A. FB3. FB4. FB6. FB9. FB10. FB11. FB12. FB8. FB7A & FB14
Y = 64. G = 86 Req'd

2	Ls	5 x 3½ x 3/8	31'-4"	@ 10.4	652
2	"	"	30-6	"	634
2	Web pls	50 x 5/16	15-8	53.1	1,664
2	Ls	5 x 4 x 3/8	4-1¾	11	91
2	Fills	5 x 3/8	3-7¼	6.38	46
20	Ls	4 x 3 x 5/16	4-2½	7.2	606
2	"	"	4-1¾	"	60
2	Pls	12 x 3/8	2-6⅞	15.3	79
4	"	6¼ x 3/8	1-6	7.97	48
5	"	10½ x 3/8	1-3½	13.39	86
2	Pls	6 x 5/16	0-10	6.38	11
2	Ls	5 x 3½ x 3/8	0-10	10.4	17
455	Shop Rivets	¾"φ Grip abt	1½	0.54	246

4,240 x 64 = 271,360 For YODO
" x 86 = 364,640 For GOKO

FLOOR BEAM FB7 Y = 5 Req'd For YODO
G = 6 " For GOKO

2	Ls	5 x 3½ x 3/8	31'-4"	10.4	652
2	"	"	30-6	"	634
2	Web pls	50 x 5/16	15-8	53.1	1,664
2	Ls	5 x 4 x 3/8	4-1¾	11	91
2	Fills	5 x 3/8	3-7¼	6.3	46
20	Ls	4 x 3 x 5/16	4-2½	7.2	606
2	"	"	4-1¾	"	60
2	Pls	12 x 3/8	2-6⅞	15.3	79
4	"	6¼ x 3/8	1-6	7.97	48
5	"	9¼ x 3/8	1-3½	11.79	76
2	Pls	6 x 5/16	0-10	6.38	11
2	Ls	5 x 3½ x 3/8	0-10	10.4	17
455	Shop Rivets	¾"φ Grip abt.	1½	0.54	246

4,230 x 5 = 21,150 For YODO
" x 6 = 25,380 For GOKO

List of Materials for Goko- & Yodo-O-Hachi for Kioto-Prefecture

FLOOR BEAM FB13
4 Req'd for Yodo,
5 " " GOKO

2	Ls	5" × 3½" × ⅜"	31'-2½"	@	10.4	649
2	"	"	30'-4½"	@	"	632
2	Web.pls.	50" × 5/16"	15'-7¼"	@	53.1	1,657
2	Ls	5" × 4" × ⅜"	4'-1¼"		11.0	91
2	Fills	5" × ⅜"	3'-7¼"		6.38	46
20	Ls	4" × 3" × 5/16"	4'-2½"		7.2	606
2	"	"	4'-1¼"		"	60
2	pls	12" × ⅜"	2'-6⅜"		15.3	79
4	"	6¼" × ⅜"	1'-6"		7.97	48
5	"	10½" × ⅜"	1'-3½"		13.39	86
2	"	6" × 5/16"	0'-9¾"		6.38	10
2	Ls	5" × 3½" × ⅜"	0'-9¾"		10.4	17
455	Shop Rivets	¾" grip ab't.	1½"		.54	246

4,227 × 4 = 16,908 ✓ for Yodo
" × 5 = 21,135 ✓ " Goko

BRACKET BR1
18 Req'd. for Yodo
24 " " Goko

2	Ls	5" × 3½" × ⅜"	2'-10½"	@	10.4	60
2	"	5" × 4" × ⅜"	3'-0⅛"	@	11.0	66
2	Fills	4¾" × ⅜"	2'-8¾"		6.06	33
2	Ls	5" × 3½" × ⅜"	3'-11½"		10.4	82
1	pl.	34½" × 5/16"	3'-2¼"		36.66	117
35	Shop Rivets	¾" grip ab't.	1¼"		.49	17
4	Field "	" "	1⅝"		.55	2

377 × 18 = 6,786 ✓ for Yodo
" × 24 = 9,048 ✓ " Goko

STRINGER S1
5 Req'd. for Yodo
10 " " Goko

1	I	12" × 5" @ 31.99#	10'-4½"			332
12	Field Rivets	¾" grip ab't	1"	@	.46	6

338 × 5 = 1,690 ✓ for Yodo
" × 10 = 3,380 " Goko

STRINGER S2
270 Req'd for Yodo
360 " " Goko

1	I	12" × 5" @ 31.99	11'-10¼			379
16	Field Rivets	¾" grip ab't.	1"	@	.46	7

386 × 270 = 104,220 ✓ for Yodo
" × 360 = 138,960 ✓ " Goko

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CALCULATIONS FOR

List of Materials for Goko- & Yodo-o-Hashi for Kioto-Prefecture.

STRINGERS S3 & S4
25 Req'd for Yodo
30 " " Goko

1	I	12" × 5" @ 31.99#		9'-8 5/8"	@ ---	311
1	"	"		13'-9 3/4"		442
24	Field Rivets	3/4"φ	grip abt.	1"	@ .46	11
						764

764 × 25 = 19,100 ✓ for Yodo
" × 30 = 22,920 ✓ " Goko

STRINGERS S5 & S6
20 Req'd. for Yodo
30 " " Goko

1	I	12" × 5" @ 31.99#		13'-9 3/4"		442
1	"	"		9'-10 5/8"		316
24	Field Rivets	3/4"φ	grip abt.	1"	@ .46	11
						769

769 × 20 = 15,380 ✓ for Yodo
" × 30 = 23,070 ✓ " Goko

STRINGER S7
5 Req'd for Yodo
0 " " Goko

1	I	12" × 5" @ 31.99#		12'-10 1/2"		412
12	Field Rivets	3/4"φ	grip abt.	1"	@ .46	6
						418

418 × 5 = 2,090 ✓ for Yodo
" × 0

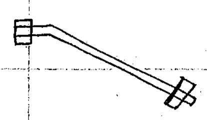
CONNECTION PLATE CP1
640 Req'd for Yodo
850 " " Goko

1	Pl.	9 1/2" × 5/16"	1'-1"	@ 10.09	11

11 × 640 = 7,040 ✓ for Yodo
" × 850 = 9,350 ✓ " Goko

EXPANSION JOINT EJ1
5 Req'd for Yodo
6 " " Goko

1	L	3" × 3" × 5/16"		18'-9 1/2"	@ 6.1	115
1	"	"		17'-6 1/2"	"	107
1	"	"		17'-6 1/4"	"	107
1	"	"		18'-9 1/4"	"	115
2	Pls	9 1/2" × 5/16"		18'-1 3/4"	10.09	366
1	"	8 3/4" × 3/8"		18'-11 1/2"	11.16	212
1	"	"		17'-3 1/2"	"	193
2	"	3" × 3/8"		18'-1 1/2"	3.83	139
14	Bolts	5/8"φ		1'-3"	1.52	21
365	Shop Rivets	3/4"φ	grip abt.	3/4"	.43	157
22	Field "	"	"	7/8"	.44	10
						1,542



1,542 × 5 = 7,710 ✓ for Yodo
" × 6 = 9,252 ✓ " Goko

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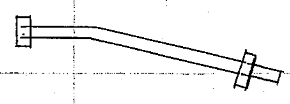
CALCULATIONS FOR

List of Materials for Goko- & Yodo-O-Hashi for Kioto-Prefecture.

Expansion Joint EJ2

5 Rigid For YODO
6 " For GOKO

1	L	3×3×5/16	18-9½	@ 6.1	115
1	"	"	17-6½	"	107
1	"	"	17-6¼	"	107
1	"	"	18-9¼	"	115
2	Pls	10×5/16	18-1¾	10.63	386
2	checkered Pls	10½×3/8	18-2½	14.26	519
14	Bolts	5/8"φ	1-3	1.52	21
265	Shop Rivets	3/4" Grip abt.	3/4	0.43	114
20	Field "	" "	7/8	0.44	9
					1,493



1,493 × 5 = 7,465 For YODO
× 6 = 8,958 For GOKO

Lateral Bracings For YODO

4	Ls	5×3½×5/16	16-1½	8.7	561	LB1
4	"	"	7-9	"	270	LB2
4	"	"	7-4	"	255	"
2	Pls	19½×5/16	2-3½	20.72	95	LB1 Gusset Pl.
112	Ls	5×3½×5/16	17-9¼	8.7	17,316	LB3 R/L
36	"	"	17-9¼	"	5566	LB4 R/L
2	Ls	"	17-6¾	"	306	LB5 R/L
2	"	"	17-9¼	"	309	"
32	"	"	17-9¼	"	4,947	LB5A R/L
34	"	"	17-6¾	"	5,195	LB6 R/L
34	"	"	17-9¼	"	5,257	"
36	"	"	16-1½	"	5,050	LB7 R/L
20	"	4×3×5/16	8-5¼	7.2	1,221	LB8
128	"	"	10-11¼	"	10,118	LB9
4	"	5×3½×5/16	17-9¼	8.7	618	LB10 R/L
2	"	"	8-3⅜	"	144	LB11 R/L
2	"	"	8-8⅜	"	151	"
2	"	"	8-5⅞	"	147	LB12 R/L
2	"	"	8-3⅜	"	144	"
2	Pls	20×5/16	2-1	21.25	89	LB10 R/L Gusset Pls
1	"	18¼×5/16	3-6	19.40	68	P1
1	"	26×5/16	3-6	27.63	97	P2
10	"	26×5/16	3-1¾	"	87	P3
26	"	15×5/16	2-2	15.94	898	P4
1	"	28½×5/16	3-9½	30.28	115	P5
9	"	29½×5/16	4-1½	31.35	1,164	P6
18	"	15×5/16	1-6¼	15.94	436	P7
8	"	31½×5/16	4-1½	33.47	1,005	P8
8	"	26½×5/16	3-5½	28.16	779	P9
1	"	26×5/16	3-1¾	27.63	87	P10
1		18¼×5/16	3-1¾	19.40	61	P11
2000	Shop Rivets	3/4"φ Grip abt.	5/8	0.41	820	
4450	" "	" "	5/8	0.41	1,825	
					65,201	

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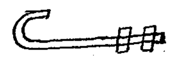
CALCULATIONS FOR

List of Materials for Goko- & Yodo-o-Hashi for Kioto-Prefecture.

LATERAL BRACING GOKO

8	∟s	5"×3½"×5/16"	16'-1½"	@ 8.7	1,122	LB1
4	pls	19½"×5/16"	2'-3½"	@ 20.72	1901	" gusset pls.
8	∟s	5"×3½"×5/16"	7'-9"	@ 8.7	539	LB2
8	∟s	"	7'-4"	"	510	"
152	"	"	17'-9¼"	"	23,500	LB3 R/L
48	"	"	17'-9¼"	"	7,421	LB4 R/L
4	"	"	17'-6¾"	"	611	LB5 R/L
4	"	"	17'-9¼"	"	618	"
40	"	"	17'-9¼"	"	6,184	LB5A R/L
44	"	"	17'-6¾"	"	6,723	LB6 R/L
44	"	"	17'-9¼"	"	6,803	"
48	"	"	16'-1½"	"	6,734	LB7 R/L
28	"	4"×3"×5/16"	8'-5¾"	7.2	1,709	LB8
168	"	"	10'-11¾"	"	13,280	LB9
2	pls.	18¼"×5/16"	3'-6"	19.4	136	P1
2	"	26"×5/16"	3'-6"	27.63	193	P2
14	"	26"×"	3'-1¾"	"	1,217	P3
35	"	15"×"	2'-2"	15.94	1,209	P4
2	"	28½"×"	3'-9½"	30.28	230	P5
12	"	29½"×"	4'-1½"	31.35	1,552	P6
24	"	15"×"	1'-6¼"	15.94	582	P7
10	"	31½"×"	4'-1½"	33.47	1,381	P8
10	"	26½"×"	3'-5½"	28.16	974	P9
2590	shop Rivets	¾"φ	grip ab't. 5/8"	.41	1,062	
5850	Field "	"	" 5/8"	.41	2,399	
					86,879	

HOOK BOLTS for Yodo

1104	Bolts	5/8"φ	0'-10"	@ 1.05	~~1163~~ 1,159	

HOOK BOLTS for Goko

1460	Bolts	5/8"φ	0'-10"	@ 1.05	~~1537~~ 1,533	"

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CALCULATIONS FOR

List of Materials for Goko- & Yodo-O-Hashi for Kioto-Prefecture

SHOES, PINS, ROLLERS AND BOLTS for Yodo

4	Cast steel shoes			@ 507.#	2.028	FH1
10	〃			180.	1.800	HG1
10	〃 Bed Pls			@ 73.0	730	RB1
40	Rollers	6"φ	1'0"	@ 71.4	2,856	RN1
40	Teeth Pls	2" × ½"	0'7½"	@ 3.4	86	〃
40	Side Pls	2" × 5/16"	1'2"	2.13	100	〃
80	Tapped bolts	¾"φ	0'2⅛"	Per ft. wt 1.5	22	〃
80	〃 〃	〃	0'1⅝"	〃 1.5	17	〃
20	Cast steel dust guards			@ 27.0	540	DG1
8	Cast steel shoe			170.0	1,360	HG2
18	〃 〃 〃			268.0	4.824	SH1
10	〃 〃 〃			807.0	8.070	RS1
10	〃 〃 〃 Bed Pls			498.0	4,980	BP1
70	Rollers	5½"φ	2'0"	131.0	9,170	RN2
40	Side Pls.	2" × ½"	2'2"	3.4	295	〃
160	Tapped Bolts	¾"φ	0'1 13/16"	Per ft. wt. 1.5	38	〃
120	〃 〃	〃	0'1 13/16"	〃 1.5	29	〃
20	Cast steel dust guards			@ 90.0	1,800	DG2
8	Cast steel shoe			1280.0	10,240	FS2
20	Pls	6½" × 5/16"	2'1"	6.91	287	DG3
10	Cast steel shear blooks			124.0	1,240	SB1$^{R}_{L}$
8	〃 〃 〃 〃			129.0	1,032	SB3$^{R}_{L}$
10	〃 〃 〃 〃			58.0	580	SB2
8	〃 〃 〃 〃			62.0	496	SB4
22	Pins	4½"φ	0'9¼"	Per ft. wt. 54.07	916	PN1
44	Nuts for	〃 pin		@ 4.6	202	〃
18	Pins	5"φ	0'11½"	66.76	1,154	PN2
36	Nuts for	5"φ pin		6.2	223	〃
72	1⅝"φ Anchor bolts			74.4	5,357	AB1
72	Hex nuts			@ 2.0	144	〃
72	Washers	3½"φ × ⅜"		32.71	73	〃
16	1¼"φ Anchor bolts			43.9	702	AB2
16	Hex nuts			0.7	11	〃
16	Washers	3"φ × ⅜"		24.03	12	〃
144	Turned bolts	1¼"φ	0'5"	3.47	500	for SH1
80	Tapped bolts	½"φ	0'1½"	0.13	10	〃 DG1
140	〃 〃	½"φ	0'1¼"	0.12	17	〃 DG3 〃
40	Bolts	⅞"φ	0'3½"	1.18	47	〃 BP1
216	〃	¾"φ	0'3"	0.75	162	〃 SB1$^{R}_{L}$ & SB3$^{R}_{L}$
					62150	

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MADE BY I. Motano DATE 2-1-16 FILE NO

CHECKED BY M. Kojima DATE 2-1-18 PAGE NO M 14

CALCULATIONS FOR

List of Materials for Goko- & Yodo-o-Hashi for Kioto-Prefecture

SHOES, PINS, ROLLERS AND BOLTS for GOKO

4	Cast steel shoes			@ 507.	2,028	FH1
12	" " "			@ 180.	2,160	HG1
12	" " " Bed Pls.			73.	876	RB1
48	Rollers	6"φ	1'-0"	71.4	3,427	RN1
48	Teeth Pls	2" × 1/2"	0'-7 1/2"	3.4	103	"
48	Side Pls	2" × 5/16"	1'-2"	2.13	120	"
96	Tapped bolts	3/4"φ	0'-2 1/8"	1.5	26	"
96	" "	"	0'-1 5/8"	1.5	23	"
24	Cast steel dust guards			24.0	576	DG1
12	Cast steel shoes			170.0	2,040	HG2
24	" " "			268.0	6,432	SH1
14	" " "			807.0	11,298	RS1
14	" " " Bed Pls.			498.0	6,972	BP1
98	Rollers	5 1/2"φ	2'-0"	131.0	12,838	RN2
56	Side Pls	2" × 1/2"	2'-2"	3.4	413	"
224	Tapped bolts	3/4"φ	0'-1 13/16"	Per ft. wt. 1.5	51	"
168	" "	"	0'-1 13/16"	1.5	38	"
28	Cast steel dust guards			@ 90.0	2,520	DG2
10	" " shoe			1280.0	12,800	FS2
28	Pls.	6 1/2" × 5/16"	2'-1"	6.91	402	DG3
12	Cast steel shoe block			124.0	1,488	SB1 R/L
12	" " " "			129.0	1,548	SB3 R/L
12	" " " "			58.0	696	SB2
12	" " " "			62.0	744	SB4
28	Pins	4 1/2"φ	0'-9 1/4"	54.07	1,166	PN1
56	Nuts for	4 1/2"φ		4.6	258	"
24	Pins	5"φ	0'-11 1/2"	66.76	1,538	PN2
48	Nuts for	4 1/2"φ pins		6.2	298	"
96	1 5/8"φ Anchor Bolts			74.4	7,142	AB1
96	Hex Nuts			2.0	192	"
96	Washers	3 1/2"φ × 3/8"		32.71	94	"
16	1 1/4"φ Anchor Bolts			43.92	703	AB2
16	Hex Nuts			0.7	11	"
16	Washers	3"φ × 3/8"		24.03	12	"
196	Tunned bolts	1 1/4"φ	0'-5"	3.47	680	for SH1
96	Tapped bolts	1/2"φ	0'-1 1/2"	.13	12	" DG1
196	" "	1/2"φ	0'-1 1/2"	.12	24	" DG3
56	Bolts	7/8"φ	0'-3 1/2"	1.18	66	" BP1
288	"	3/4"φ	0'-3"	.75	216	" SB1 R/L & SB3 R/L
					82,031	

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CALCULATIONS FOR

List of Materials for Goko- & Yodo-o-Hashi for Kioto-Prefecture.

Summary of Weight

		Yodo	Goko
Main girder	G1R	25,846#	51,692#
" "	Splice SP1	1,622	3,244
" "	G2R - G2AR	37,302	74,604
	G2R	1,880	1,880
	G2AR	0	2,068
" "	G3R	128,744	193,116
" "	G4R - G4AR	296,320	370,400
	G4R	8,272	10,340
	G4AR	7,520	9,400
	splice SP2	18,144	22,680
" "	G5R - G5AR	227,584	284,480
	Splice SP3	27,408	34,260
" "	G6R	21,276	0
		801,918	1,058,164
Floor Beams	FB1-5	53,955	68,670
" "	FB2-2A-3-4-6-9-10-11-12-8-7A	271,360	364,640
" "	FB7	21,150	25,380
" "	FB13	16,908	21,135
Bracket	BR1	6,786	9,048
		370,159	488,873
Stringer	S1	1,690	3,380
"	S2	104,220	138,960
"	S3-4	19,100	22,920
"	S5-6	15,380	23,070
"	S7	2,090	0
Connection Pls	CP1	7,040	9,350
		149,520	197,680
Exp. Joint	EJ1	7,710	9,252
" "	EJ2	7,465	8,958
		15,175	18,210
Laterals		65,201	86,879
		65,201	86,879
Hook Bolts		1,159	1,533
		1,159	1,533
Shoes, Pins, Rollers, Anchor bolts, Shear block etc.		62,150	82,031
		62,150	82,031
Total Weight =		1,465,282#	1,933,370#
		= 654.144 Tons	= 863.112 Tons

Grand Total Weight for Yodo & Goko = 1,517.256 Tons

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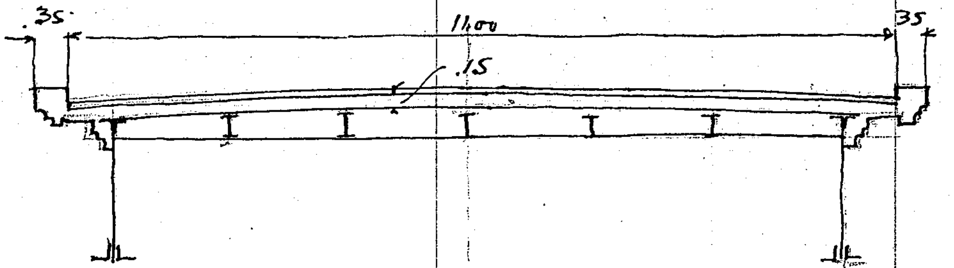
CALCULATIONS FOR

List of materials for Goko and Yodo-o-hashi for Kioto-Prefecture

Volume of Concrete in floor slab.

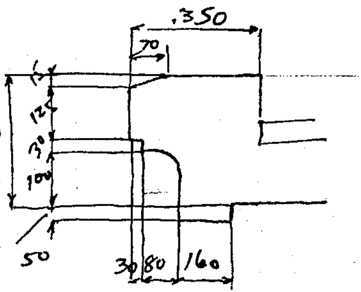


Sectional area of floor slab.

between main girder	9.6 × .15	=	1.440
Outside of main girder	1.40 × .175	=	0.245
coping	2 @ 0.0938	=	0.188
moulding at girder	2 @ .103	=	0.206
filler on top of girder + stringers say		=	0.024
			2.103 ☐m

call this 2.10 ☐m

finish of coping not counted.



area of coping. 350 × 270 = 94500
− 1500
+ 800
.093800

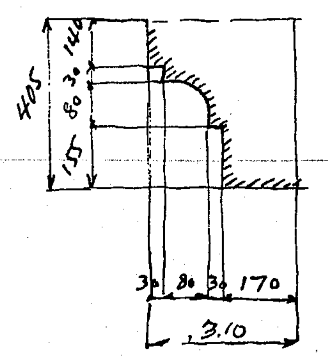
2 @ .0938 = 0.1876

Total length of slab.

For Yodo-o-hashi.
268.193 − .500 for expansion = 267.7 m
assumed.

For Goko-bashi.
354.436 − .600 = 353.8 m

area of moulding



405 × 310 = 125500
− 22530
.102970

call this .103

filler on girder 2 @ .0075 = 0.015
stringers 1 @ 55
stringers 30
0.0235

Volume of Concrete

Yodo	2.10 × 267.7	=	562.17 cubic m
			call this 562.0
Goko.	2.10 × 353.8	=	742.98
			call this 743.0

Area of forms for coping and floor slab.

Total length 11.70
1.80
13.50

At coping
.32
.405
.05
.125

Less stringer 5 × .125 = −.625
girder 2 @ .175 = −.350
12.525 ☐m

2 @ .900 = 1.80

Yodo. 12.525 × 268.2 (say) = 3360.0
Goko. 12.525 × 354.4 (say) = 4440.0

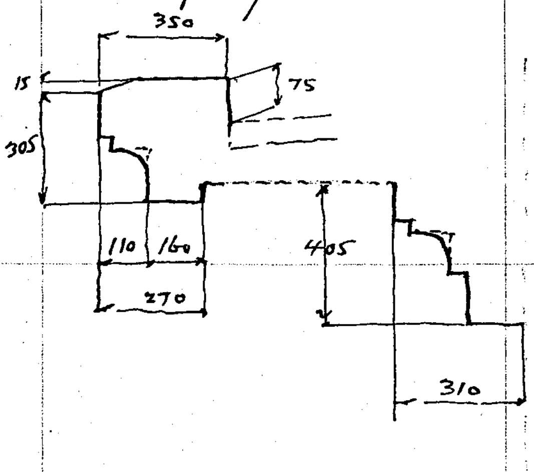
Area of pavement

	淀大橋		御幸橋
Total length between parapet walls.	268.198		354.436
Expansion Joint 5 @ 337 =	1.685	6 @ 337	2.022
	266.503		352.414

Asphaltic Concrete pavement — Granolithic pavement

For Yodō	7.0 × 266.503	= 1865.521	4.00 × 266.503	= 1066.012
For Gokō	7.0 × 352.414	= 2466.898	4.00 × 352.414	= 1409.656

Area of finish



75
350
305
270
405
310

2 @ 1.715 = 3.430 m

length of finish.
Yodo − 268.193
5 @ 63 = .315
267.878

Goko-bashi 354.436
6 @ 63 .378
354.058

For Yodo-o-hashi 3.43 × 267.878 = 918.82 sq meters (882.154)

For Goko-bashi. 3.43 × 354.058 = 1214.42 sq meters (1141.89)

Reinforcement in slab.

For Yodo-ohashi 51.007 tons For Goko-bashi. 67.411 tons see sheet no 4

List of materials for Yoko + Yodo-ohashi for Kioto-Prefecture

volume of Concrete in piers nos 1 to 9 (淀大橋)

Coping 2.10^{ϕ} = 3.46 2 @ 3.46 = 6.92
1.40 × 7.70 abt 10.80
17.72 × .26 = 4.60 cubic meters
1.90^{ϕ} = 2.84 2 @ 2.84 = 5.68
1.20 × 7.9 about 9.50
15.18 × .14 = 2.12
6.72 cubic meters.

Shafts. top area - 1.7 dia = 2.270

	Dz.	Bottom area		Top area			Av.		Hz		vol.	for 2 shafts
Pier no. 1-9	2.41	4.562	+	2.27	=	6.832	3.416	×	5.683	=	19.42	38.84 cubic m
2-8	2.429	4.635	+	2.27	=	6.905	3.452	×	5.832	=	20.12	40.24
3-7	2.442	4.680	+	2.27	=	6.950	3.475	×	5.938	=	20.62	41.24
4-6	2.450	4.714	+	2.27	=	6.984	3.492	×	6.001	=	20.95	41.90
5	2.453	4.714	+	2.27	=	6.984	3.492	×	6.023	=	21.05	42.10

web panel

			Hz		Area				Area net		width		
Pier no 1-9	6.70	×	3.683	=	24.70	−	.93	=	23.77	×	.60	=	14.25 cubic m
2-8	6.70	×	3.832	=	25.69	−	.93	=	24.76	×	.60	=	14.85
3-7	6.70	×	3.938	=	26.39	−	.93	=	25.46	×	.60	=	15.27
4-6	6.70	×	4.001	=	26.80	−	.93	=	25.87	×	.60	=	15.52
5	6.70	×	4.023	=	26.90	−	.93	=	25.97	×	.60	=	15.57

web moulding

			Hz										
Pier no. 1-9	7.74	×	5.683	=	44.00	−	23.77	=	20.23	×	1.000	=	20.23 cm
2-8	7.74	×	5.832	=	45.20	−	24.76	=	20.44	×	"	=	20.44
3-7	7.73	×	5.938	=	45.90	−	25.46	=	20.44	×	"	=	20.44
4-6	7.72	×	6.001	=	46.32	−	25.87	=	20.45	×	"	=	20.45
5	7.72	×	6.023	=	46.50	−	25.97	=	20.53	×	"	=	20.53

Summary for shaft coping + web.

Pier no.	1-9	2-8	3-7	4-6	5
coping	6.72	6.72	6.72	6.72	6.72
shafts	38.84	40.24	41.24	41.90	42.10
web panel	14.25	14.85	15.27	15.52	15.57
web moulding	20.23	20.44	20.44	20.45	20.53
	80.04	82.25	83.67	84.59	84.92
	237.14	237.14	237.14	237.14	237.14
	317.18	319.39	320.81	321.73	322.06

Concrete in well pier nos 1-9 inclusive
Concrete in shell at 1.5 meter at bottom.
outside dia 4.25 - 14.186
Inside dia 3.10 7.548
7.638 sq m

7.638 × 1.5 = 11.46 cubic meters

Less concrete outside $\frac{.075 \times 8.74}{2} \times 14.2$ = 0.46 cubic meter

Less Concrete at curb shoe volume of cone

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CALCULATIONS FOR

List of materials for Gokō and Yodo-o-hashi for Kioto-Prefecture.

2/18

Outside area (cone) 4.25 · 14.186 × 0.063 = 0.90
Cone. 14.186 (bottom)
Inside dia 3.226 8.160 (top)
$\sqrt{14.186 \times 8.16}$ 10.73
33.076 × .493/3 = 5.44
6.34 cubic meter
volume of inside dia - ~~8.16~~ 7.548 × .556 = - ~~4.20~~ ~~1.79~~ 2.14 cubic meter
0.46
~~2.25~~ 2.60 " "

net volume = 11.46 - ~~2.25~~ 2.60 = ~~9.21~~ 8.86 cubic meter.

Top portion of well shell. outside dia 4.10 · 13.200
Inside dia 3.10 · 7.548
5.652 × 12.0 = 67.8 cubic meter
8.86
76.66 " " for one well.

Concrete in bottom filling
under curb shoe 6.34
Inside dia 3.10 - 7.548 × ~~2.44~~ 2.444 = 18.40
24.74 cubic meters.

Concrete in top filling
dia 3.10 7.548 × 1.000 = 7.548
" 3.50 9.621 × 1.000 = 9.621
17.167 say 17.17 cubic meter.

Intermediate sand fill 7.548 × 9.5 = 71.7 cubic meters

Concrete block. outside 13.20
Inside 9.62
3.58 × 1.00 = 3.58 cubic meters.

Summary for well

	For one well.	For one pier		
shell.	76.66	153.32	cubic meters	153.32
bottom filling	24.74	49.48	"	49.48
top filling	17.17	34.34	"	34.34
intermediate sand filling	71.70	143.40	"	237.14
Concrete block	3.58	7.16	"	

Area of forms.
Coping - circ. 6.60 - 1.4 = 5.40 × 2 = 10.80 + 16.2 = 27.00
5.97 - 1.2 = 4.77 × 2 = 9.54 + 16.3 = 25.84

27.00 × .26 = 7.02
25.84 × .14 = 3.62
10.64
Bottom area 17.72 - 12.44 = 5.28 = 5.28
15.92 call this 16.00 sq meters.

Shaft. pier no 3 for average
top circumference 1.70 dia 5.34
bottom " 2.442 " 7.67
13.01 ÷ 2 = 6.50
less 1.00
5.50 × 5.938 = 32.7 sq meters

12 @ 32.70 = 65.4 square meters.

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CALCULATIONS FOR

List of materials for Goko and Yodo-o-hashi for Kioto Prefecture. 3/19

Area of web. 45.90 × 2 = 91.80
for moulding. 2.938 × 2 = 5.876
2@.25 = .500
Curve = 3.150
under. = 4.200
13.726 × .40 = 5.50
under web. 1.00 × say 5.70 = 5.70
103.00 sq meters.

coping 16.00
shafts 65.40
web. 103.00
184.40 sq meters
582.00
766.40

Area of forms for well.
Outside dia 4.10 12.88
Inside .. 3.10 9.74
22.62 × 12.874 = 291.0 sq meters
2@ 291.0 = 582.0 .. ..

Excavation bottom area - 14.19 × 14.5 = 205.8 cubic meters.
say 206.0
2@ 206 = ~~416~~ 412 cubic meters

Reinforcing bars in well + shafts

Pier no	1-9	2-8	3-7	4-6	5	
	~~9.090~~ 8.501	~~9.104~~ 8.515	~~9.116~~ 8.525	~~9.121~~ 8.531	~~9.123~~ 8.534	tons

Curb shoe 2 curb shoes = ~~4.077~~ ~~3.828~~ 4.096 tons.

List of materials for Pier no 10-21 inclusive 橋脚

Volume of Concrete
Coping. 6.72 cubic meters
shaft. top area 1.7 dia = 2.270

	dia Dz	area			aver. area	H/2	For one	For 2 shafts
Pier no 10-21	2.548	5.100 + 2.27	=	7.37	3.69 × 6.782	=	25.00	50.00
11-20	2.567	5.18 + "	=	7.45	3.72 × 6.936	=	25.80	51.60
12-19	2.582	5.23 + "	=	7.50	3.75 × 7.058	=	26.48	52.96
13-18	2.593	5.27 + "	=	7.54	3.77 × 7.144	=	26.94	53.88
14-17	2.599	5.31 + "	=	7.58	3.79 × 7.196	=	27.28	54.56
15-16	2.602	5.31 + "	=	7.58	3.79 × 7.213	=	27.30	54.60

Web panel.

Pier no. 10-21	6.700 × 3.882	=	26.00 - .93	=	25.07 × .60	=	15.05	
11-20	6.70 × 4.036	=	27.05 "	=	26.12 × .60	=	15.67	
12-19	6.70 × 4.158	=	27.85 "	=	26.92 × .60	=	16.15	
13-18	6.70 × 4.244	=	28.42 "	=	27.49 × .60	=	16.50	
14-17	6.70 × 4.291	=	28.73 "	=	27.80 × .60	=	16.68	
15-16	6.70 × 4.313	=	28.90 "	=	27.97 × .60	=	16.78	

web moulding

Pier no. 10-21	7.68 × 6.782	=	52.05	-	25.07	=	26.98 × 1.00	= 26.98
11-20	7.67 × 6.936	=	53.20	-	26.12	=	27.08	27.08
12-19	7.66 × 7.058	=	54.00	-	26.92	=	27.~~13~~ 08	27.~~13~~ 08
13-18	7.65 × 7.144	=	54.65	-	27.49	=	27.16	27.16
14-17	7.65 × 7.196	=	55.00	-	27.80	=	27.20	27.20
15-16	7.65 × 7.213	=	55.20	-	27.97	=	27.23	27.23

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CALCULATIONS FOR

List of materials for Goko- + Yodo-o-hashi for Kito-Prefecture

Summary of Concrete in coping shafts + web.

Piers	10-21	11-20	12-19	13-18	14-17	15-16
Coping	6.72	6.72	6.72	6.72	6.72	6.72
shafts	50.00	51.60	52.96	53.88	54.56	54.60
web panel	15.05	15.67	16.15	16.50	16.68	16.78
web moulding	26.98	27.08	~~27.13~~ 08	27.16	27.20	27.23
	98.75	101.07	102.96	104.26	105.16	105.33
	254.14	254.14	254.14	254.14	254.14	254.14
	352.89	355.21	357.05	358.40	359.30	359.47

Concrete in well shell.

Concrete in 1.5 m at bottom 8.86
" " top 5.652 × 13.5 = 76.30
85.16 cubic meter.

Bottom filling - 24.74 cubic meters
Top filling - 17.17 " "
Sand filling - 7.548 × 11.0 = 83.0 cubic meters
Concrete block. 3.58 cubic meters

Summary for well

	For one well	For two well.	
shell	85.16	170.32	170.32
bottom filling	24.74	49.48	49.48
top filling	17.17	34.34	34.34
intermediate filling	83.00	166.00	~~254.14~~
Concrete block	3.58	7.16	

Area of forms. coping 16.00 sq meters.
shaft top. 1.70 dia 5.34
bottom 2.593 dia 8.14
13.48 ÷ 2 = 6.74
1.00
5.74 × 7.144 = 41.0 2 @ 41.0 = 82.0 sq meters

web. 54.65 × 2 = 109.3
under moulding = 5.74
bottom of well = 1.00 × 5.7 = 5.70
110.74

Coping. 16.00
shafts 82.00
web. 110.70
208.70 sq. meters
650
858.70 ~~call this~~ 860.

Forms for well. 22.62 × 14.374 = 325.0 sq m
For 2 wells @ 325 = 650 sq meters

Excavation. 14.19 × 16.00 = 227.0 2 @ 227 = 454.0 cubic meters.

Reinforcing

Pier	10-21	11-20	12-19	13-18	14-17	15-16
	~~9.876~~ 75	9.871 ~~9.892~~	9.903 ~~9.907~~	9.911 ~~9.912~~	9.916	9.917 ~~9.918~~ tons.
	9.875	9.871	9.903	9.911		7

Weight of curb shoes. 2 curb shoes = ~~4.077~~ ~~3.828~~ 4.096 tons of structural

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Materials of Abutment for Yodo-ohashi and Goko-bashi

Volume of Concrete

Base		12 × 7 × 1.8	= 151.2 cub.m
Front wall	Under pedestal	11 × 1.05 × 6.29	= 72.65
	Front footing	½ × 1.0 × 1.0 × 11	= 5.5
	Back footing	½ × 1.0 × 1.0 × 7	= 3.50
	Parapet wall	2.24 × .3 × 11	= 7.36
	Side column	2 × 1.0 × 1.0 × 5.0	= 10.0
Butress walls		4 × ½ × 3.2 × 8.4	= 53.76
			303.97 cub.m or 51.0 立坪

4@ 303.97 = 1215.88 cub.m 204.0 立坪

Area of Concrete forms

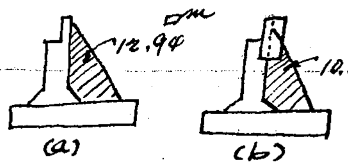
Base side
2 × 12 × 1.8 = 43.2
2 × 7 × 1.8 = 25.2

Front face
Parapet wall 2.24 × 11. = 24.64
Wall 5.29 × 11. = 58.19
Front footing 1.0 × 1.41 × 11 = 15.51
Side column 2 × 5 × 1.0 = 10.00
= 108.34

Back face
Buttress walls 4 × 1.0 × 9 = 36.
Front wall
8.773
−1.0
−0.26
7.513
1.41
8.923 × 7 = 62.46

Side column 2 × 1.0 × 5 = 10.0
= 72.46

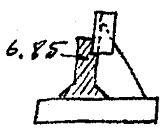
Side face
Buttress walls (a) ½ × 3.2 × 8.4 = 13.44
− ½ × 1.0 × 1.0 = .5 □m
6@ 12.94 = 77.66



(b)
12.94
− .70 × 4 = 2.8
2@ 10.14 = 20.28

Side Column
2 × 1.0 × 5.0 = 10
2(½ × .7 × 2) = 1.4
= 11.40

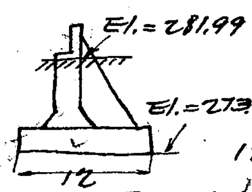
Front wall
1.05 × 6.29 = 6.6
2 × ½ × 1.1. = 1.0
−.30 × 2.5 = −.75
2@ 6.85 = 13.70



372.24 □m
4@ 372.24 = 1488.96 □m or 546.7 面坪

Volume of excavation

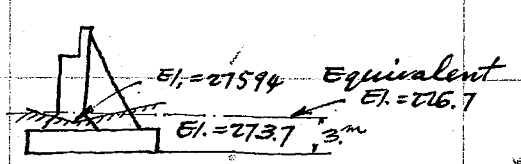
淀大橋右岸



281.99
273.7
8.29

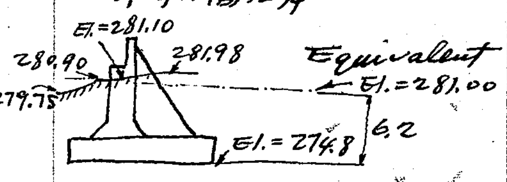
12 × 7 × 8.29 = 700 cub.m or 116 立坪

淀大橋左岸



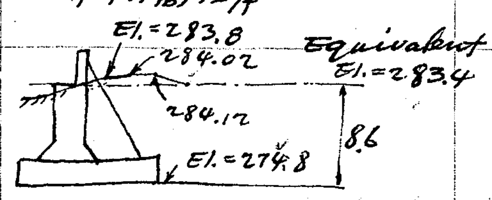
12 × 7 × 3 = 252 cub.m or 42 立坪

御幸橋右岸



12 × 7 × 6.2 = 520.8 cub.m or 87 立坪

御幸橋左岸



12 × 7 × 8.6 = 722.4 cub.m or 120 立坪

2195.2 365 立坪

~~割栗石~~
~~12 × 7 × .3 = 25.2 cub.m or 4.2 立坪~~
~~4@ 25.2 = 101. cub.m or 16.8 立坪~~

跳込石
.30 × .15 × 11 = .5 cub.m
4@ .5 = 2. cub.m or .33 立坪

Finished Area

Front face under pedestal
Front 1. × 11. = 11.0
Side 2 × 1. × 1.05 = 2.2
Top 2 × .9 × 1.25 = 2.25
parapet 2 × .7 × 2.24 = 3.14
Side col. 2 × 1. × 2.5 = 5.0
2 × 1. × 3.2 = 6.4
29.89 □m

372.24

4@ 29.89 = 120.0 □m − 36 面坪

Reinforcement

12,555 # per one abutment = 5.605 Tons
4@ 5.605 Tons = 22.42 Tons

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

MADE BY Y. Hirata DATE 2-1-18 FILE NO
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CALCULATIONS FOR

List of materials for Goko + Yodo-o-hashi for Kioto-Prefecture. 22

Panel Casting A1

3 × 856 × 15 × 25 = 963,000 cub. mirimeter
8 × 141 × 19 × 25 = 535,800
8 × 433 × 19 × 25 = 1,645,400
4 × 70 × 15 × 25 = 105,000
3 × 135 × 15 × 25 = 151,875
2 × 100 × 19 × 25 = 95,000
3,496,075 = 213.5 cub. inch @ 0.26# 55.5#
2.5 assumed Rivets wt.
58.0

Panel Casting X1

759 767 × 19 × 25 = 36,525 ~~360,525~~
2 × 15 × 25 × 70 = 52,500
1 × 15 × 25 × 135 = 50,625
~~463,650~~
463,650

3,496,075
−) 463,650
3,032,425 = 185 cub.in. @ 0.26# 48.0#
3,032,425
2.0
50.0

Panel Casting X2

2 × 357 × 15 × 25 = 267,750
4 × 589 × 19 × 25 = 1,119,100
2 × 70 × 15 × 25 = 52,500
135 × 15 × 25 = 50,625
2 × 100 × 19 × 25 = 95,000
281 × 15 × 25 = 105,375
1,690,350 = 103 @ 0.26 27.0
3.0
30.0

Panel Casting X3 246,000

2 × 268 × 15 × 25 = 246,000
3 × 589 × 19 × 25 = 839,325
135 × 15 × 25 = 50,625
281 × 15 × 25 = 105,375
1,241,325 = 76.1 @ 0.26 19.7
76.1
(1.3)
21.0

246,000
839,325
50,625
105,375
1,241,325

Post P1 & P2

75 × 20 × 856
$-(74^2 \times \frac{\pi}{4} + 48^2 \times \frac{\pi}{4}) \times 20$ = 1,162,000 (438)
2 × 15 × 100 × 856 = 2,568,000
$2 \times 20 \times 6.2(1660 + 105 \times \frac{\pi}{2})$ = 438,000 438,000
4,168,000 = 254.3 @ 0.26 66.1
416,8,000 254.3
2 - Bolts $\frac{1}{2}$" × 200 (8") @ 58.00 1.2
1 Pl. 3" × 5/16" × 195 (7 3/4) @ 3.19 2.1
69.4 69.4
69.4 69.4

Pipe $2\frac{1}{2}$" $1\frac{1}{2}$"

H_1 & H_6	1,800 = 5'-11"	@ 5.79	34.3#	@ 2.72	16.1#
H_2	1,400 = 4'-7 6/8"	〃	26.6	〃	12.5
H_3	2,160 = 7'-1 6/8"	〃	41.0	〃	19.3
H_4	1,778 = 5'-10"	〃	33.8	〃	15.9
H_5	1,786 = 5'-10 3/8"	〃	34.0	〃	15.9

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

MADE BY Y. Hirata DATE 2-1-18 FILE NO.

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CALCULATIONS FOR

15000
35
625000
1250

23

Lamp post LP.

$100 \times 15 \times \cancel{325}$ 350	=	525,000 ~~487,500~~	a		
$180 \times 15 \times 340$	=	918,000	b		
$215 \times 15 \times 180$	=	580,500	c		
$150 \times 270 \times 15 \times \frac{1}{2}$	=	303,750	d		
$250^2 \times 20$	=	1250,000 1250,000	e		
$215^2 \times 20$	=	924,500 924,500	f		
$660 \times 15 \times 910$			g		
$-[(74^2+48^2)\times\frac{\pi}{4}\times 15\times 2]$	=	8,825,800	h		
$4 \times 5 \times 20 \times 741$	=	296,400	i		
$(140^2\times\frac{\pi}{4} - 100^2\times\frac{\pi}{4}) \times 60$	=	45,250	j		
$(120^2\times\frac{\pi}{4} - 80^2\times\frac{\pi}{4}) \times 80$	=	50,250	k		
$60 \times (70^2\times\frac{\pi}{4} - 50^2\times\frac{\pi}{4})$	=	1,130	l		
$2 \times 20 (104^2\times\frac{\pi}{4} - 74^2\times\frac{\pi}{4})$	=	168,000 168	m		
$2 \times 170 \times 50 \times 15$	=	255,000	n		
$50^2 \times \frac{\pi}{4} \times 70$	=	137,200	o		
$20^2 \times \frac{\pi}{4} \times 400$	=	125,700	p		
		14,406,480 14406,480	= ~~880~~ 880	@ 0.26	~~229.0#~~ 229.0#
1 pipe $3\frac{1}{2}$" × 800 (2-8")				@ 9.11	24.3
1 " $2\frac{1}{2}$ × 900 (3-0)				@ 5.79	17.4
1 " $1\frac{1}{2}$ × 350 (1-2)		1440648		@ 2.72	31.8
4 Bolts $\frac{5}{8}$" × 50 (2")				@ 40.00	1.6
					304.1 304.1

15000
17

304.1#
304.1#

Cast Iron Drain

$12 \times 230 \times 224$	=	618,240			
$240 \times 12 \times 42 \times 2$	=	241,920			
$330 \times 12 \times 150$	=	594,000			
$12 \times 224 \times 320$	=	860,160			
$12 \times 30 \times 80 \times 16$	=	460,800			
$12 \times 30 \times 236 \times 3$	=	254,880			
		3030,000	= 185	@ 0.26	48.0#

Pedestal.

$0.94^2 \times 1.10 - 4 \times 0.06 \times (0.37 + 0.46) \times \frac{1}{2} = 0.87236$ cub. meter

$0.69^2 \times 1.42 - (2 \times 0.12^2 \times 1.42 + 0.03 \times 0.29 \times 1.42) + 0.1 \times 0.15 \times 0.03 \times 8 = 0.626412$

$0.4^2 \times 0.08 = 0.0128$

$0.53^2 \times 0.10 = 0.02809$

$1.0^2 \times 0.225 - 0.4^2 \times 0.225 = 0.18900$

$0.4^2 \times 0.225$

1.728662
1.728662

~~1.721~~ cub. meter
1.721

$0.08 \times 0.4 \times 4 = 0.128$ □ meter
$0.1 \times 0.53 \times 4 = 0.212$
$0.53 \times 0.065 \times 2 = 0.0689$
$0.4 \times 0.065 \times 2 = 0.052$
$0.76 \times 1.42 \times 4 = 4.3168$
$0.08 \times 0.53 \times 4 = 0.1696$
$1.04 \times 1.1 \times 4 = 4.576$
$0.125 \times 0.82 \times 4 = 0.41$
$0.225 \times 4 + 0.06 \times 4 = 1.14$
11.0733

11.07 □ meter

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

MADE BY Y. Hirata DATE 2-1-18 FILE NO.

CHECKED BY DATE PAGE NO. M24

CALCULATIONS FOR

Hand Rail

Yodo-Ōhashi						Gokō-Bashi					
270	Post	P_1	@	69.4#	18,738	358	Post	P_1	@	69.4	24,845
10	〃	P_2	〃	〃	694	12	〃	P_2	〃	〃	833
544	Panel Casting	A_1	〃	58.0	31,552	720	Panel Casting	A_1	〃	58.0	41,760
46	〃 〃	X_1	〃	50.0	2,300	60	〃 〃	X_1	〃	50.0	3,000
2	〃 〃	X_2	〃	30.0	60	4	〃 〃	X_2	〃	30.0	120
2	〃 〃	X_3	〃	21.0	42	0					
274	Pipe 2½"φ	H_1 & H_6	〃	34.3	9,398	362	Pipe 2½"φ	H_1 & H_6	@	34.3	12,417
2	〃	H_2	〃	26.6	53	4	〃	H_2	〃	26.6	106
2	〃	H_3	〃	41.0	82	0					
8	〃	H_4	〃	33.8	270	12		H_4	〃	33.8	406
10	〃	H_5	〃	34.0	340	14		H_5	〃	34.0	476
274	〃	H_1	〃	16.1	4,411	362		H_1	〃	16.1	5,828
2	〃	H_2	〃	12.5	25	4		H_2	〃	12.5	50
2	〃	H_3	〃	19.3	39	0					
8	〃	H_4	〃	15.9	127	12		H_4	〃	15.9	191
10	〃	H_5	〃	15.9	159	14		H_5	〃	15.9	223
					68,290# or 30.5 T.						90,255# or 40.3 T.

Lamp Post

18 @ 304.1 5,474# — 24 @ 304.1 7,298#

Cast Iron Drain

76 @ 48.0 3648# — 100 @ 48.0 4,800#

Pedestal

4 @ 1.721 cub. meter 6.884 c.m. — 4 @ 1.721 6.884 c.m.

4 @ 11.07 $\square^m$ 44.28 $\square^m$. — 4 @ 11.07 $\square^m$ 44.28 $\square^m$.

増田淳氏関係資料（独立行政法人 土木研究所蔵）

Bearing Power of soil of Present Bridge —

Present bridge consists of 10 – 85' span of Howe Truss design made of ~~wood~~ timber. The width of roadway — 18.0

Let us estimate weight of bridge approximate only.

Floor Planking and Joists assumed .35 × 18 = 6.30 cubic ft @ 40# = 380# per lin ft

Truss (one truss only) Top chord 2@ .8 × .8 = 1.28
bottom chord 2@ .8 × .8 = 1.28
diagonals say 2.00
For two trusses — 2@ 4.56 cubic ft @ 40# = 365#

Lateral Bracing — say 65#

810# per lin ft of span

Total load on pier = 810 × 85 = 69000# on one pier.

Live Load on bridge neglected in figuring soil pressure of present pier.

Weight of Present Pier. (approximate only)

Shaft 6 × 25 × 15.2 = 2280
less say 280
2000 @ 137# = 274000#

weight of wells 9' dia 2@ 63 × 21 @ 150 = 396000

670000#

Weight of superstructure 69000

Total load 739000#

Unit Bearing Pressure = $\frac{739000}{2@63.4}$ = 5820# or 2.6 tons per sq ft.
not counting frictional surface resistance of wells.

Surface friction of well assumed 200# per sq ft

9' dia 2@ 28 × 200 × 21 = 236000#

Counting surface friction of well Resulting load on bottom of base 739000
236000
503000#

Unit Bearing Pressure = $\frac{503000}{2@63.4}$ = say 4000# or 1.8 tons per sq ft of base.

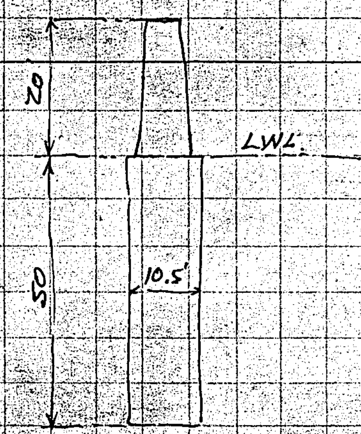
From findings of borings these piers rest on the clay soil or sand soil over the clay and its safe bearing power will be safe enough to assume 2.5 tons per sq ft at the depth of 20' below river bed.

京阪電氣鉄道橋梁 Single Track Electric Railway Bridge.

Span length 124'0" 127'0" center to center of piers.

Dead load (superstructure) 100 tons about including decks

Live Load 35 ton car with impact of 25% = say 43.7 tons assumed on one pier



weight of shaft say 6 × 20 × 20 = 2400
less cut say 400
2000 @ 140# = 480.000#

wells 2 @ 86.6 × 50 = 8660 @ 150 = 1300.000#

1780.000#

Superimposed load 143.7 × 2240 = say 320.000

2,100,000#

Unit Bearing Pressure = $\frac{2100.000}{2@86.6}$ = 12100# or 5.40 tons per sq ft
not counting surface friction.

Counting frictional resistance of wells @ 200#/□'

2100.000
Less say 660.000
1,440,000# ÷ 2@86.6 = 8300#/□'
or 3.7 tons □'

These piers may be rest on gravel and ~~the~~ safe bearing pressure can be assumed 4. to 5. tons per square ft at 50' below water level.

Preliminary design and Estimate of 淀大橋 for Kioto Prefecture.

From the findings of borings there will be 3 kinds of pier ~~due to~~ according to the depth of base

1st Stop the base at Elev. 50.0 which is 28' below present river bed and penetrate clay soil 10' to 15' and rest on sandy clay.

2nd Stop the base at Elev. 35.0 about and rest on fine gravel soil.

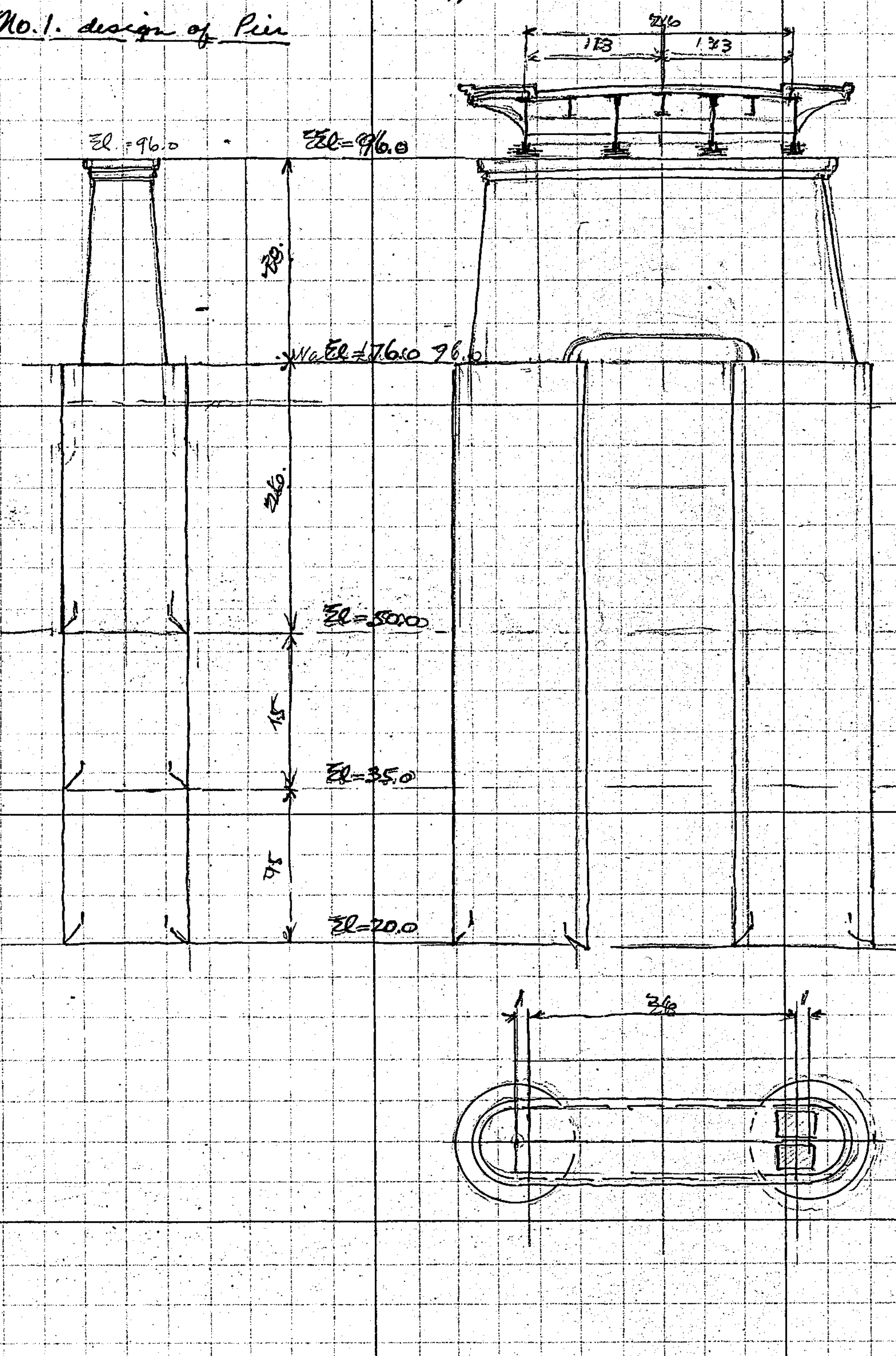
3rd Stop the base at Elev. 20.0 about and rest on solid rock.

Low water level about 76.0 and the piers 3rd is 56' below low water level.

Let us estimate of cost of one pier approximately to find out the economic span length of the bridge.

Assumed data. 24' Roadway 2-6' sidewalks on both sides.

No. 1. design of Pier



1st Pier at Elev. 50.0

Approximate volume of concrete in shaft of pier =

top area 6×28 = 168
6'dia. 28
196

Bottom area 8×28 = 224
8'dia. 50
274
470

Average 235 □'

volume of concrete 235×20 = 4700 cubic ft

neglecting projection of coping and cut at top of base. 21.8 立坪

weight = 4700×140 = say 660.000#

volume of well.

volume of shell 1.25 thick.

outside dia 12 — 113.0

Inside dia 9.5 70.9

42.1

volume of concrete — 42.1×26 = 1095 cubic ft

~~wells~~ or 5.06 立坪

or 2@5.06 = 10.12 立坪

Filling 26×70.9 = 1840 cu.ft or 8.52 立坪

2@8.52 = 17.04

Total volume of concrete in wells = 27.16 立坪

weight = 27.16×216×140 = 820,000#

Excavation 2@113.0×22 = say 5000

or 23.0 立坪

weight of curb shoe say approximately — 50×37 = 1850#

adding misc. details say 2000#

Reinforcing bars in shaft etc say ~~5000#~~ 7000

Bearing area = 2@113 = 226 □'

Total load pier only

shaft = 660.000

wells 820.000

1480.000#

Approximate Estimate.

shaft	1:3:6	concrete	21.8 立坪	@	146.50	=	3190.00
wells	1:3:6	concrete	17.04 "	@	128.00	=	2180.00
do	1:2:4	concrete	10.12 "	@	168.50	=	1705.00
Excavation			23.0 "	@	60.00	=	1380.00
Steel in shoes	2@2000 =		4000#	@	15	=	600.00
Reinf. bars			7000#	@	10	=	700.00
							9755.00

for Pier at Elev. 50.0

Unit Bearing = $\frac{1480.000}{226}$ = 6530#/□'

292 tons/□'

No 2 design of Pier. well stopped at Elev. 35.0
construction of Pier above Elev. 50.0 same as for No 1 Pier.
volume of shell of well:-
Outside dia = 113.0
Inside dia = 70.9

	2@	42.1 × 15	=	1265	or	5.85 立坪	
Inside filling	2@	70.9 × 15	=	2120	or	9.85	"
Excavation -	2@	113.0 × 15	=	3390	or	15.70	"

Total weight of concrete 3390 × 140 = 475000#
From above 1480000
Unit Bearing, 1955000 ÷ 226 = 8640#/□' or 3.86 tons/□'

Estimate of cost

well	1:2:4 concrete	5.85 坪	@ 168.50	=	985.00
	1:3:6 concrete	9.85	@ 128.00	=	1260.00
Excavation		15.70	@ 150.00	=	2360.00
Reinforcing bars say					500.00
					5105.00
Pier no 1.					9755.00
					14960.00
10% Risks					1496.00
					16456.00 call this 16500.00 円

No 3 design of Pier well stopped at Elev. 20.0
Bearing on soil. load above 1955000
load 475000
2430000 ÷ 226 = 10750#/□' or 4.8 tons.

Estimate of cost

well	1:2:4 concrete	5.85 立坪	@ 168.50	=	985.00
	1:3:6 "	9.85	" 128.00	=	1260.00
Excavation		15.70	" 300.00	=	4720.00
Reinforcing bars					500.00
					9465.00
Pier no 2					16456.00
					25921.00 - call this 26000.00
~~10% risks~~					~~2592.00~~
					~~28513.00~~ ~~call this 28500.00~~ 円

Economic span length for Type No 1. Pier.
Total length of bridge end to End = 870' about
This length will be divided into the following spans

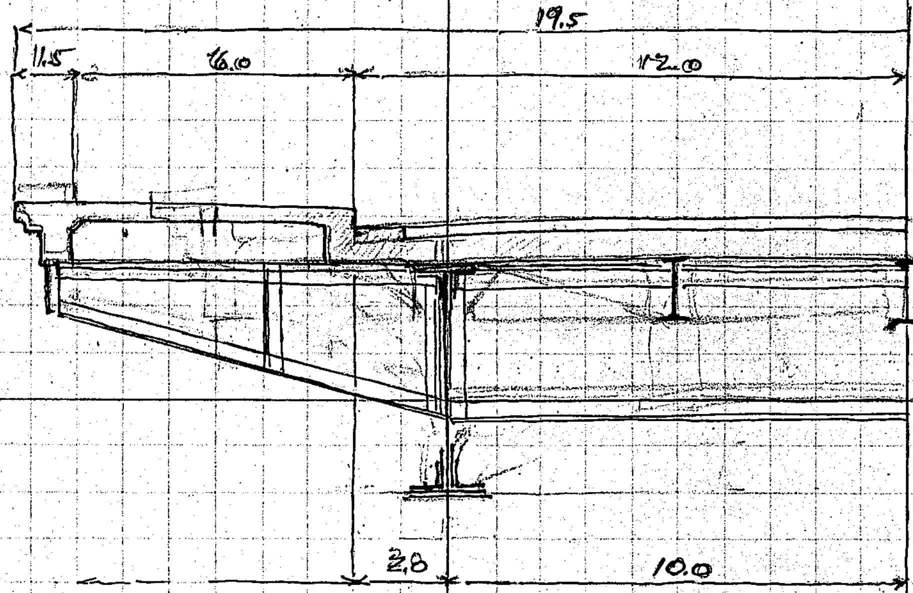
15 spans @ 58.0	9 spans @ 96.7
14 spans @ 62.2	8 spans @ 109.0
13 spans @ 67.0	7 spans @ 124.0
12 spans @ 72.5	6 spans @ 145.0
11 spans @ 79.2	5 spans @ 174.0
10 spans @ 87.0	4 spans @ 218.0

Let us estimate the superstructure of all steel and concrete slab with Deck Type.
Loading on the bridge. 8 ton motor trucks with 1/3 impact

Concentrated load:
- Rear wheel concentration 6600#
- Front wheel concentration 2200#
- 12 ton Road Roller without impact
- Rear wheel concentration 7700#
- Front wheel concentration 11000#

Uniform load 100# per square ft.

Cross section of structure (Deck girder span).



Floor slab span length 5.0'

assumed pavement - 3" asphaltic concrete

3" asphalt @ $\frac{130}{12}$ = 33#

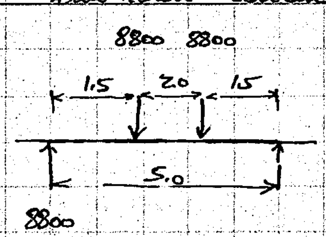
7" concrete slab - 88#

121#

Dead Load moment = $\frac{1}{10} \times 121 \times 5^2 = 302$#

Dead Load shear = 121 × 2.5 = 300#

Live Load concentration 6600 + 2200 impact = 8800#



moment as simple beam = 8800 × 1.5 = 13200 1#

Distribution of concentrated load - 0.6 × 5.0 + 1.0 = 4.0 assumed

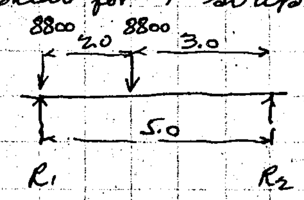
moment per ft strip = $\frac{13200}{4}$ = 3300 1#

For continuity of slab 0.8 × 3300 = 2640 1#

Dead Load moment say 300 1#

Total moment 2940

Shear for 1' strip



R_1 = 8800 × $\frac{3}{5}$ = 5280

8800

Live Load shear 14080 ÷ 4 = 3520#

Dead Load shear 300

Total shear 3820#

Effective depth of slab for 600#/□" concrete stress and 16000#/□" steel stress.

$d = \sqrt{\frac{2940}{95}} = 5.6"$

Covering for protection 1.4

7.00" make slab 7.0" over all with effective depth of 6".

Steel required = $\frac{2940 \times 12}{\frac{7}{8} \times 6 \times 16000}$ = .42 □" per ft $\frac{1}{2}$"φ bars $5\frac{1}{2}$" centers = .43

Unit shear = $\frac{3820}{\frac{7}{8} \times 6 \times 12}$ = 60.7#/□" Use bent up bars for shear at ends.

Bond stress = $\frac{3820}{\frac{7}{8} \times 6}$ = 726# Bond Area required = $\frac{726}{85}$ = 8.55 □"

$\frac{1}{2}$"φ bars = 1.57 8.55 ÷ 1.57 = 5.5 reqd

Sidewalk slab Use $3\frac{1}{4}$" with $\frac{3}{4}$" wearing course on top.

Longitudinal Stringer

Use 12" I beam 31.5# section modulus 36.0 For span length ~~15.0~~ 12.5? about or less

" 15" I " 42.0# " " 58.9 For span length 15.0 about

" 18" I " 55.0# " " 88.4

Let us figure the limiting span length for these I beams approximately.

Dead Load Floor 121 × 5 = 605

" " beam say 35

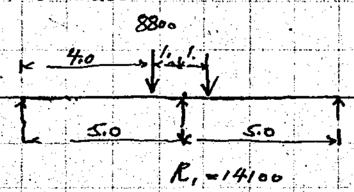
640#

Live Load

$R_1 = 8800 \times \frac{4}{5}$ = 7050

7050

14100



Try span length = 15.0 m = $\frac{1}{8} \times 640 \times 15^2$ = 18000 1#

Live load ~~52800~~ ~~26400~~

Total m ~~44400~~ 70800

section modulus = $\frac{\cancel{44400}\ 70800 \times 12}{16000}$ = ~~33.4~~ 53.0

Apply this load at center of span

moment = $\frac{14100}{2}$ × 7.5 = ~~26400~~ 52800

Try span length = 13.0 DL m = $\frac{1}{8} \times 640 \times 13^2$ = 13500

Live Load m (absolute) 18800 × $\frac{5.25^2}{13}$ = 39900

53400

see next page

Sm = $\frac{53400 \times 12}{16000}$ = 40.0

15" I beam span assumed 17.5'.

Dead Load Floor 121 × 5 = 605 Live Load Rear conc. 14100# × 0

Beam 45 Front " 4700 × 6 = 28200

650' 18800 28200

17.5 8.75 .75 8.0

m = 1/8 · 650 × 17.5² = 24800# Center of gravity = 1.5 from rear conc.

Live Load moment 68800 Bending moment = 18800 × 8.0²/17.5 = 68800

93600

Sm = 93600 × 12 / 16000 = ~~630~~ 70.2

Try 17.0 span D.L. m = 1/8 · 650 · 17.0² = 23500 Live Load m = 18800 × 7.75²/17.0 = 66300#

Live Load m 66300

89800 Sm = 89800 × 12 / 16000 = 67.3

Try 16.0 span D.L m = 1/8 · 650 · 16.0² = 20800

Live Load m = 18800 × 7.25²/16. 61800 Sm = 82600 × 12 / 16000 = 62.0

Total m 82600

Try 15.0 span D.L m = 1/8 · 650 · 15.0² = 18300

Live Load Moment = 18800 × 6.75²/15.0 = ~~48900~~ 57000 Sm = 75300 × 12 / 16000 = 56.5

75300

18" I beam span say 18.5'

Dead Load Floor 121 × 5 = 605 Live Load moment = 18800 · 8.5²/18.5 = 73400

beam say 60 D.L m 28400

665 101800

m = 1/8 · 665 × 18.5² = 28400# Sm = 101800 × 12 / 16000 = 76.2

Try span length 19.5

Dead Load m = 1/8 · 665 × 19.5² = 31600#

Live Load m = 18800 × 9²/19.5 = 78000 Sm = 109600 × 12 / 16000 = 82.0

109600

Try span length 20.0

Dead Load m = 1/8 · 665 × 20² = 33200

Live Load m = 18800 · 9.25²/20 = 80500 Sm = 113700 × 12 / 16000 = 85.3

113700

From the above figures, figure floor beams for 12.5' span 15.0 span and 20.0 span

Span length of floor beam assumed 20.0' and over-hang of cantilever bracket as shown on sketch on pp.4.

Cantilever bracket

				Arm		m about ℄ main girder
Handrail say 150# per lin ft.			150	8.75	=	1310
Fascia girder say 2.0 @ 150 = 300# per lin ft			300	8.75	=	2620
Sidewalk slab say .33 × 6 . 150 = 300#			300	5.00	=	1500
curb and gutter beyond main girder	1.25 × .75 = .94 @ 150	=	141	2.38	=	336
Gutter	.8 × 1.00 = .80 @ 150	=	120	1.50	=	180
Floor slab	.58 × 1.00 = .58 @ 150	=	87	.50	=	44
Pavement			33	.50	=	16
			1131#			6006
Cantilever bracket say			30#	3.50	=	105
			1161#			6111#

					Total m
Cantilever bracket Dead Load moment	12.5 span @ 6111	= 76500#	37500#	15000#	129000#
	15.0 span @ 6111	= 91800#	45000	15000#	151800
	20.0 span @ 6111	= 122200#	60000	15000	197200

Live Load moment 100 · 6 = 600 × arm 5.0 = 3000

concentration say 10,000 × 1.5 = 15000 per bracket assumed.

増田淳氏関係資料（独立行政法人 土木研究所蔵）

Depth of bracket at connection 3.0'

			section reqd	section used	
12.5 spacing	129000 '#	44500 #	3.750 □"	2Ls 3½ . 3½ . 5/16	= 4.18
15.0 spacing	157800	52300	4.360 □"	2Ls 3½ . 3½ . 3/8	= 4.98
20.0 spacing	197200	68000	5.670 □"	2Ls 4 . 4 . 3/8	= 5.72

Use 5/16" web plate throughout

weight of Cantilever bracket

12.5 spacing

4Ls 3½ . 3½ . 5/16 @ 7.2 . 8.5 = 245
1 web 24" . 5/16 @ 25.50 . 8.5 = 217
Details say 160
622#

15.0 spacing

4Ls 3½ . 3½ . 3/8 @ 8.5 . 8.5 = 289
web 217
details 160
666#

20.0 spacing

4Ls 4 . 4 . 3/8 @ 9.8 . 8.5 = 334
web 217
details 160
711#

Cross Beam between Main Girders. Span 20.0'

Approximate section of Cross Beam

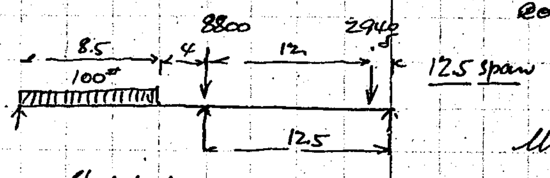
Dead Load Floor 121#
Beam say 10
131#

moment lin ft = ⅛ . 131 . 20² = 6550 #

			Resulting moment
For 12.5 spacing	@ 6550 =	82000 '# less D.L.m. Cant. 76500 =	5500 '#
15.0 spacing	" =	98200 '# 91800 =	6400
20.0 spacing	" =	131000 '# 122200 =	8800

Dead Load Cross beam = ⅛ . 100 . 20² = 5000 '#

Live Load moment 12.5 span



Sketch for 12.5 span

concentration = 2940 . .5/12.5 = 120
8800
8920#

Uniform load = 850 . 4.25/12.5 = 289# per lin ft.

15.0 span

concentration = 2940 . 3/15 = 590
8800
9390

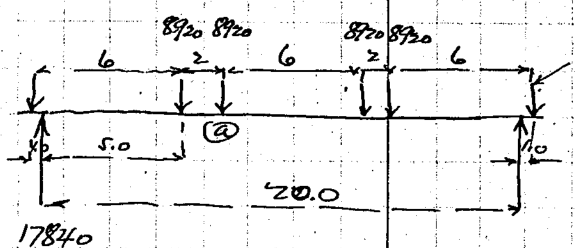
Uniform load = 1100 . 5.5/15 = 404# per lin ft

20.0 span

concentration = 2940 . 8/20 = 1180
8800 . 20/20 = 8800
9980

Uniform load = 1600 . 8/20 = 640
600 . 3/20 = 90
730# per lin ft.

Live Load moment 12.5' span



Moment at (a) 17840 . 7 =
− 8920 . 2 =
17840 . 6 = 107,000 '#
Unif. ⅛ . 289 . 20² = 14500
121,500 '# = 121500 '#

15.0 span moment at (a) 18780 . 6 = 112500
Unif. ⅛ . 404 . 20² = 20200
132700 '#

20.0 span moment at (a) 19960 . 6 = 120.000
Unif. ⅛ . 730 . 20² = 36500
156500 '#

Summary of moments

	12.5 spacing	15.0 Spacing	20.0 Spacing
Resulting D.L.m	5500	6400	8800
cross beam	5000	5000	5000
Live Load m	121500	132700	156500
	132000	144100	170300 '#

Section of Cross Beam

Depth of beam 3.0

	moment	stress	section	section used
12.5'	132000 #	45400	2.84 net	2 L's 3½ × 3½ × 5/16 = 4.18 gross
15.0	144100	49600	3.10 "	2 L's 3½ × 3½ × 3/8 = 4.98 "
20.0	170300	58600	3.66 "	2 L's 4 × 4 × 3/8" = 5.72 "

weight of Cross Beam

12.5 4 L's 3½ × 3½ × 5/16 @ 7.2 × 20.0 = 576
1 web 36 × 5/16 @ 38.25 × 20.0 = 765
details say 300
1641#

15.0 4 L's 3½ × 3½ × 3/8 @ 8.5 × 20.0 = 680
web 765
details 300
1745

20.0 4 L's 4 × 4 × 3/8 @ 9.8 × 20.0 = 783
web 765
details 300
1848

Summary for cross beam and cantilever brackets

2 @ 622 = 1244
1641
2885#

2 @ 666 = 1332
1745
3077#

2 @ 711 = 1422
1848
3270#

Summary for Floor system

12.5 3 @ 35# = 105#
2885 ÷ 12.5 = 231
336#

15.0 3 @ 45 = 135
3077 ÷ 15 = 205
340#

20.0 3 @ 60 = 180
3270 ÷ 20 = 163.5
343.5#

with End Floor Beams

5 @ 12.5 = 62.5
3 @ 35# = 105
6 × 2885 / 62.5 = 277
382

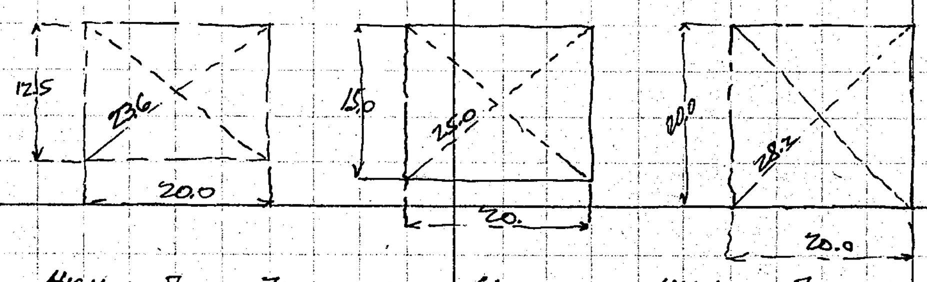
4 @ 15.0 = 60.0
3 @ 45 = 135
5 × 3077 / 60 = 257
392#

3 @ 20.0 = 60.0
3 @ 60 = 180
3270 × 4 / 60 = 218
398#

Metal in Floor system say 400# per lin ft of span.

Lateral system at bottom plane of Cross Beam.

12.5 spacing



4 L's 4 × 3 × 5/16 @ 7.2 × 23. = 660
Details say 5 @ 25 125
785
785 ÷ 12.5 = 63.# per ft

4 L's 4 × 3 × 5/16 @ 7.2 × 24. = 690
details 125
815
815 ÷ 15.0 = 55# per ft

4 L's 4 × 3 × 5/16 @ 7.2 × 27.5 = 790#
details 125
915
915 ÷ 20 = 46# per ft

Design of main Girder.

Dead Load Floor.

pavement	33 × 11.0 =	363
Concrete slab	88 × 11.0 =	968
Gutter		120
Curb	say	140
Sidewalk		300
Fascia girder		300
Hand rail		150
		2341# per lin ft.

Metal in bridge –

Floor system	400		
Lateral system say	50		
Main girder say	800		
	1250# ÷ 2 = 625# per lin. ft		
weight of Flooring complete	2340		
	2965# per lin. ft		

Dead Load moments: –

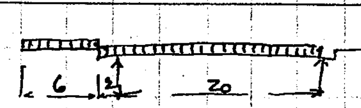
$m = \frac{1}{8} \times 2965 \times 60.2^2 = \underline{890.000}^{1\#}$?

$m = \frac{1}{8} \times 2965 \times 70.5^2 = \underline{1220.000}^{1\#}$?

$m = \frac{1}{8} \times 2965 \times 85.0^2 = \underline{1770.000}^{1\#}$?

Uniform Live Load

$2800 \times \frac{14}{20} = 1960$# per lin ft



Extra concentration at center of span say 25.000# assumed.

60.2 span	U. moment	$= \frac{1}{8} \times 1960 \times 60.2^2$	= say	890.000
	conc.	12500×30.1	=	376.000
				1,266,000 1#
70.5 span	U. moment	$= \frac{1}{8} \times 1960 \times 70.5^2$	=	1220.000
	conc.	12500×35.2	=	440.000
				1660,000 1#
85.0 span	U moment	$= \frac{1}{8} \times 1960 \times 85.0^2$	=	1770.000
	conc.	12500×42.5	=	530.000
				2300,000 1#

Summary for moment

	60.2' span	70.5' span	85.0 span
DLm	890.000	1220.000	1770.000
LLm	1266.000	1660.000	2300.000
	2156.000 1#	2880,000 1#	4070.000 1#

Summary for shear

	60.2' span	70.5' span	85.0 span
DL shear	59000	69000	83500
LL shear say	59000	69000	83500
LL shear say	25000	25000	25000
	143000	163000	192000#
section req'd	14.3 □"	16.3 □"	19.2 □"
	$60 \times \frac{3}{8} = 22.5$	$66 \times \frac{7}{16} = \cancel{25.5}$ 28.0	$72 \times \frac{1}{2} = 36.0$ □"

Main girder section: – for 60.2' span 62.2 out to out

~~Stress = 143000~~ Moment = 2156000 Stress = 440000# 2h = 27.4 – 2.8 = 24.60"

Use	2 Ls 6 × 6 × 3/4	=	16.88 □"	13.88 □" net	
1 Pl.	14 × 1/2	=	7.00	6.00	$60.2 \times \sqrt{\frac{10.5}{24.38}}^{.655} + 2 = 42.0'$
1 Pl.	14 × 3/8	=	5.25	4.50	$60.2 \times \sqrt{\frac{4.5}{24.38}}^{.43} + 2 = 28.0'$
			29.13 □"	24.38	

weight of main girder

1 web pl	60 × 3/8	@ 76.5	×	62.2	=	4750
4 Ls 6 × 6 × 3/4		@ 28.7	×	62.2	=	7150
2 Pls 14 × 1/2		@ 23.8	×	42.0	=	2000
2 Pls 14 × 3/8		@ 17.85	×	28.0	=	1000
						14900
		Details say 25%				3700
						18600#

315# per lin ft

2 @ 18600 = 39200# 630# per lin ft.

Main girder section for 70.5' span 72.5 out to out.

moment 2880,000 Depth = 5.54 S = 533000 SR = 3330 − 3.5 = 29.8 □" net

Try 2Ls 8×8×3/4 = 22.88 gr. 19.88 net

2Pls 18×5/8 = 11.25 10.00 45.0

34.13 □" 29.88 □" net

weight of main girder

1 web pl. 66×7/16 @ 98.2 × 72.5 = 7140

4Ls 8×8×3/4 @ 38.9 × 72.5 = 11280

2Pls 18×5/8 @ 38.25 × 45.0 = 3440

21860

22% 4800

26660 ÷ 72.5 = 368.0# per ft

2@26660 = 53320 ÷ 72.5 = 736# " "

Main girder section for 85.0' span 87.0 out to out.

moment 4070.000 depth 6.04 Effective d say 5.5 S = 740.000# SR = 46.3 − 4.5 = 41.8 □" net

Try 2Ls 8×8×3/4 = 22.88 gr 19.88

1Pl. 18×5/8 = 11.25 10.00 44'

1Pl. 18×3/4 = 13.50 11.50 63'

47.63 41.38

weight of main girder

1 web pl 72×1/2 @ 122.4 × 87.0 = 10650

4Ls 8×8×3/4 @ 38.9 × 87.0 = 13550

2Pls 18×3/4 @ 45.9 × 63.0 = 5780

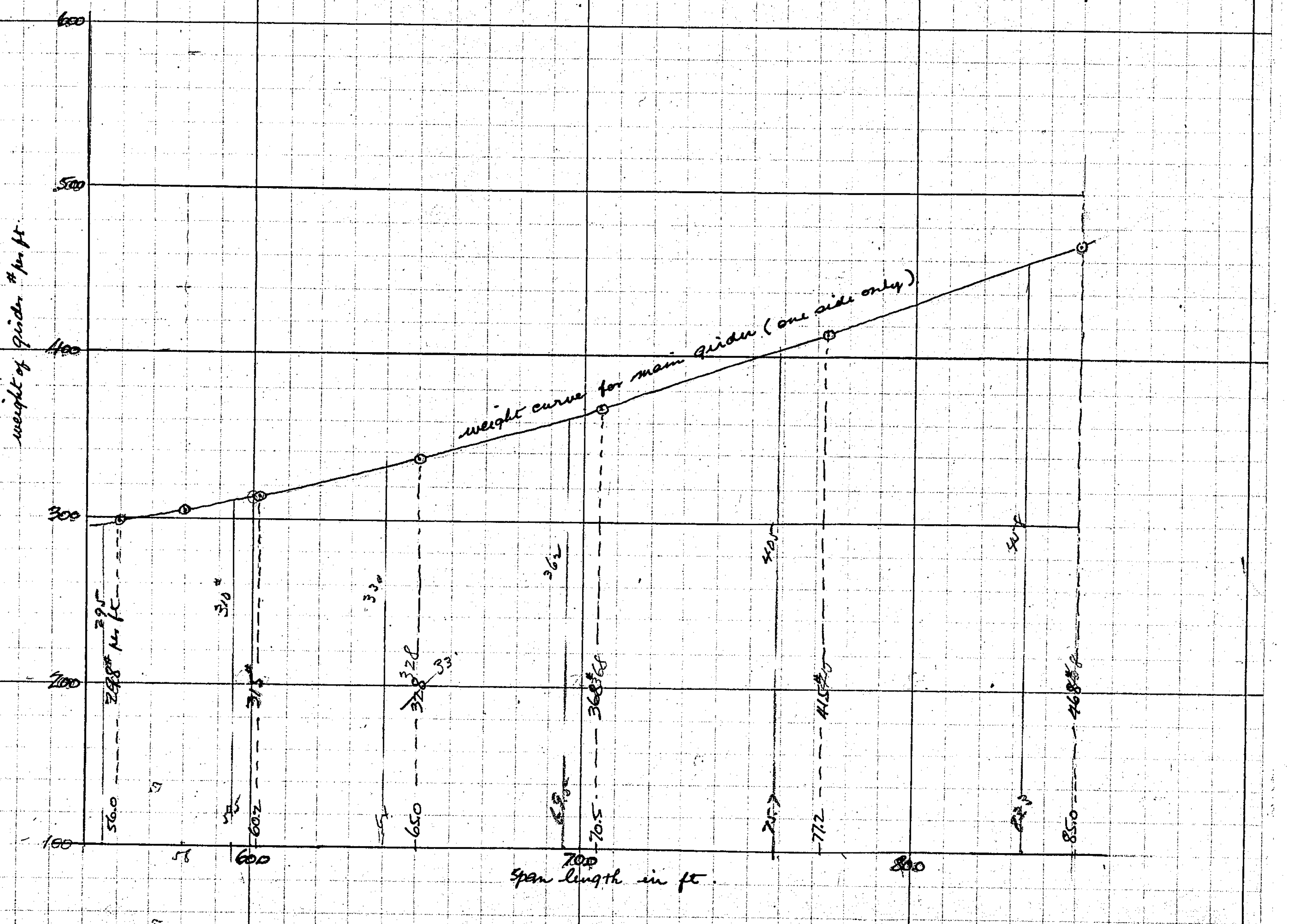
2Pls 18×5/8 @ 38.25 × 44.0 = 3360

33340 ÷ 87

Details 22% 7330

40670 ÷ 87 = 468# per lin ft

2@40670 = 81240 ÷ 87 = 936# per lin ft



Preliminary design and Estimate of 淀大橋 for Kioto Prefecture

metal in bridge –

	15@ ~~58~~		14@ ~~62.2~~		13@ ~~67.0~~		12@ ~~72.5~~		11@ ~~79.2~~		10@ ~~87.0~~	
Main girder	596#	590	630	620	656	660	736	724	830	810	936	916
Laterals	70		63	63	60	62	54	54	50	50	46	46
Floor system	390		382	382	400	400	392	392	400	400	~~400~~ 390	390
	~~1056~~	1050	1075	1065	~~1116~~	1120	1182	1170	~~1280~~	1260	~~1372~~	1352
For 875'	412. tons	391.	420 tons	397	436 tons	417	463 tons	436.	500 tons	470	538 tons	504
with 5% var.	433 tons	410	441 tons	417	458 tons	438	486 tons	458	525 tons	494	565 tons	530 –
280.00 per ton	121000.00	115000	123500.00	117000	128200.00	123000	136100.00	129,000	146000.00	139000	158500.00	149.000

Design of pier for 87' span

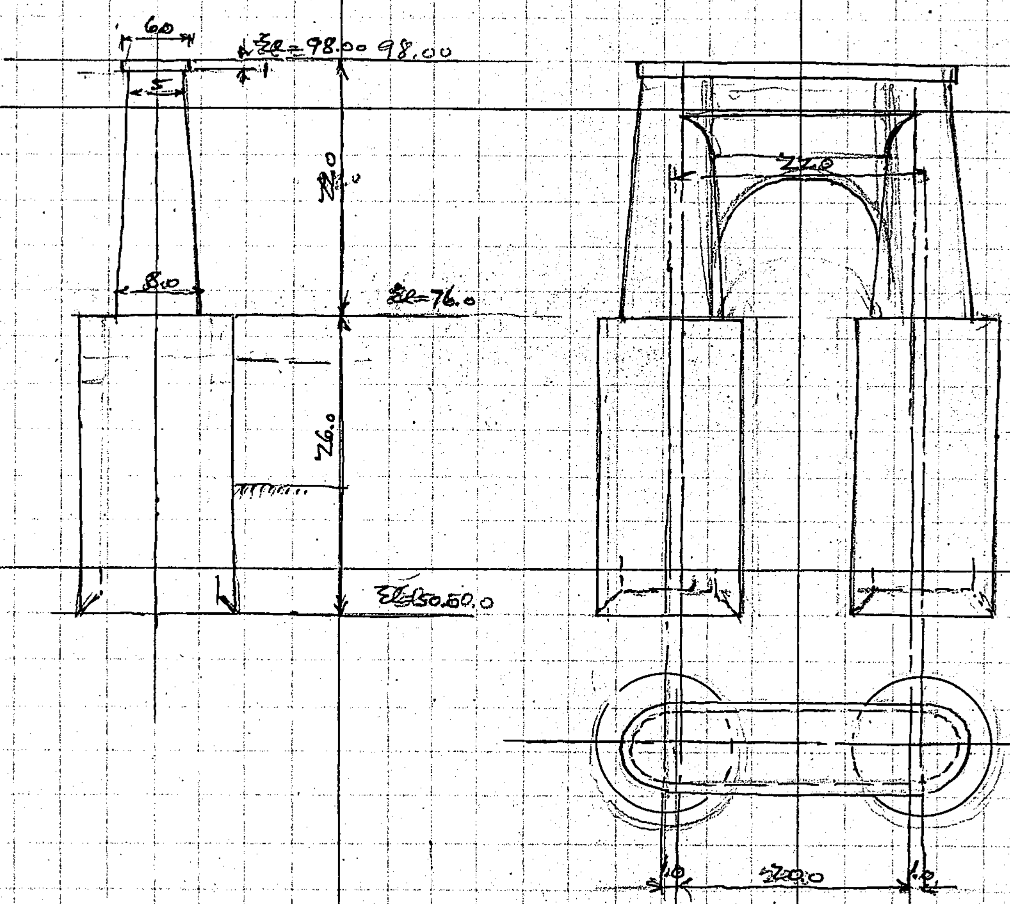
Solid shaft and 2 wells. as shown on pp 2.

superimposed load 4 × 192000 = 768000 call this 800.000#

weight of pier pp 2 1480.000

2280.000# = 9270# 415 tons/□'

make shoes 13.5' dia at bottom 122.7 × 2 = 245.4



coping

6 × 22 = 132

6' dia 28

160.0 cubic shaken

shaft

top 5 × 22 = 110

5' dia 19.6

129.6 – .60

~~bottom~~

8 × 22 = 176

8' dia 50

226.0

355.6

average – 177.8

volume = 177.8 × 21 = 3730 cu ft 17.30 cu ton

17.90

wells.

13 dia 132.7

10.5 dia 86.6

46.1

volume of shell 2 – 46.1 × 26 = 2400 11.10 cu yd

filling 2 – 86.6 × 26 = 4500 20.90

6900

Excavation 2 – 132.7 × 22 = 5840 – 27.00 cu yd

Approximate Estimate

shaft + coping	136	17.9 cu yd	@ 146.50	=	2620
well – shell	1:2:4	11.1	@ 168.50	=	1870
filling	1:3:6	20.9	@ 128.00	=	2680
Excavation		27.0	@ ~~60.00~~ 97.00	=	~~1620~~ 2620
curb shoes		2. tons	@ 320.00	=	640
Reinf. bars		2 tons	@ 230.00	=	460
					19890.00
			call this		16.000.00

add ~~1000 for misc~~ 1000

~~17000.00~~

12000.00

Total weight of pier –

shaft + coping 3860

wells 6900

10760 @ 140 = 1,510.000

superimposed load 800.000

2,310.000#

bottom area 13.5 dia – 2 @ 143 = 286

Unit bearing $\frac{2310.000}{286}$ = 8070 #/□'

3.6 tons/□'

not counting friction.

Friction 200 × 40.8 × 10 = 820.00

2 @ 82000 = 164000#

2310.000
164.000
2,146,000#

Unit bearing = $\frac{2146.000}{286}$ = 7500 #/□'

3.35 tons

Preliminary Design and Estimate of 琵大橋 for Kioto Prefecture

Concrete in Deck.

Fascia girder	2.0	
sidewalk slab	1.98	
curb	.94	
gutter	.80	
Floor slab	.58 × 11 =	6.38
	12.10 × 2 =	24.20 □'

volume of concrete in Floor + sidewalks

24.20 × ~~875~~ 835 = ~~21200~~ 20200 cubic ft

~~98.0~~ 93.5 立坪

Reinforcing bars ~~980~~ 935 × 1600 = ~~157000~~ 150,000 #

~~70 tons~~ 67 tons.

Pavement 875 × 22 / 36 = ~~535~~ 510 坪

Finish of sidewalks ~~146~~ 139 × 2.5 = ~~365~~ 347 坪

Handrail ~~875~~ 835 × 2 = ~~1750~~ 1670 call this ~~1800~~ 1700 lin ft.

Estimate of Cost of Deck

concrete in Floor	~~98.0~~ 93.5 立坪	@ 245.00	=	24000.00	23000
Reinforcing bars	~~70.0~~ 67.0 tons	@ 230.00	=	16100.00	15400
Pavement in Roadway = ?	~~535~~ 510 坪	@ 25.00	=	~~28600.00~~ 13.4	12750
Finish of sidewalks + coping	~~365~~ 347 坪	@ ~~4.00~~ 5.00	=	1460.00	1740
Handrail	~~1800~~ 1700 ft	@ 8.50	=	15300.00	14400
		misc.		5200.00	misc. 5000
				~~85460.00~~	
				75460.00	72290
					call this 73000.00

Estimate of Cost

	15 @ ~~58~~ 55.5	14 @ ~~62.2~~ 59.5	13 @ ~~67.0~~ 64.0	12 @ ~~72.5~~ 69.5	11 @ ~~79.2~~ 75.7	10 @ ~~87.0~~ 83.3
Deck	~~85460~~ .00	~~85460~~	~~85460~~	~~85460~~	~~85460~~	~~85460~~ .00
Steel	121000.00	123500	128200	136100	146000	158500.00
substructure	~~140000~~ .00 168000	~~130000~~ 156000	~~120000~~ 144000	~~110000~~ 132000	~~100000~~ 120000	~~90000~~ .00 108
abutments	18000	18000	18000	18000	18000	18000.00
Total cost	364460	356960	361660	349560	349460	351960.00
misc Exp.	14000	14000	14000	14000	14000	14000.00
	378460	370960	365660	363560	363460	365960.00
	~~14000~~	~~13000~~	~~12000~~	~~11000~~	~~10000~~	18000
	~~392460~~	~~383960~~	~~377660~~	~~374560~~	~~373460~~	~~374960~~
	384460	373960	367660	364560	363460	364960

22

16000

Revised Estimate of Cost

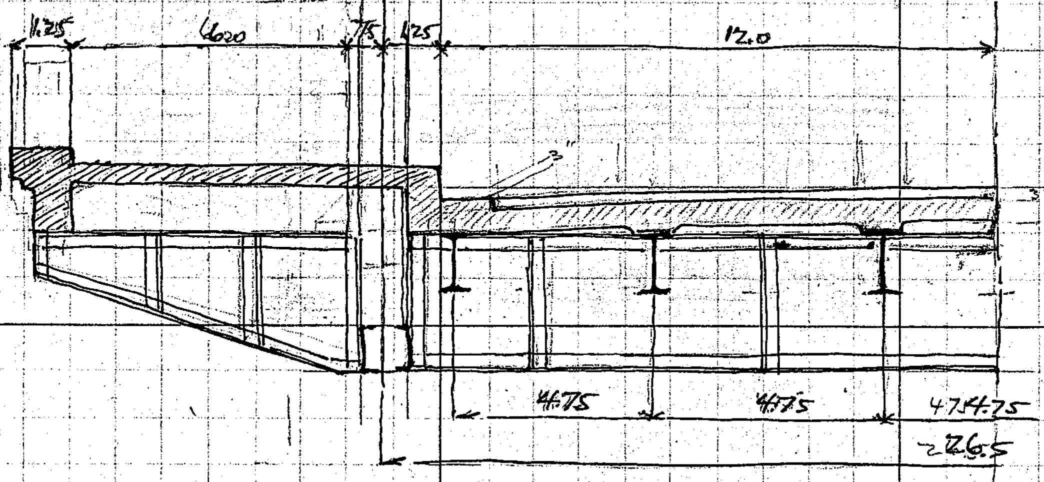
	15 @ 55.5	14 @ 59.5	13 @ 64.0	12 @ 69.5	11 @ 75.7	10 @ 83.3
Deck	73.000	73.000	73000	73000	73000	73000
Steel	115.000	117.000	123000	129000	139000	149.000
Substructure	154.000	143.000	132000	121000	110000	99000
abutment	18000	18000	18000	18000	18000	18000
Misc Exp.	14000	14000	14000	14000	14000	14000
	374000	365000	360000	355000	354000	353000

485000

838000

485000

Design No 2. Through Truss bridge for Pier Elev. 35.0
Cross section of structure.



Size of stringer same as for deck span

For this cross section try	9 @ 96.7	span length	94.5	5 @ 18.9
	8 @ 109	" "	109.0	6 @ 17.8
	7 @ 124	" "	122.0	6 @ 20.3
	6 @ 145	" "	143.0	7 @ 20.4

Stringer use 6 @ 55# 18" Is = 360# per lin ft of span

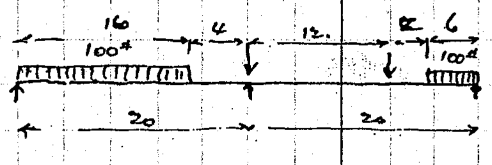
Cross Beam and Cantilever Brackets.

Cantilever Bracket		Arm		moment about ℄ Truss.
Handrail say	150# ×	7.38	=	1110
Fascia girder	300# ×	7.38	=	2220
Sidewalk slab .33 × 8 @ 150 = say	400# ×	2.75	=	1100
				4430
	cantilever bracket say			100
				4530 '#

For 20.0 panel dead Load moment = 4530 × 20 = 90600 '#
Live Load moment 100 × 6 = 600 600 × 3.75 = 2250
For 20.0 panel 2250 × 20 = 45000
Dead Load moment 90600
Total moment 135600 '#
use 2 Ls 3½ × 3½ × 5/16 flange Ls and 5/16" web plate
weight of Cantilever bracket
4 Ls 3½ × 3½ × 5/16 @ 7.2 × 7.2 = 208.
web 5/16 × 24 @ 25.50 × 7.0 = 179
details say 150
537# 2 @ 537 = 1074#

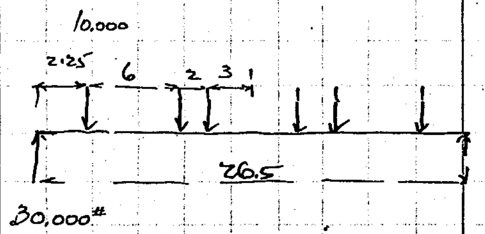
Cross Beam between main trusses.
~~Span~~ 26.5 spacing 20.0 about
Dead Load assumed 131# per sq ft Moment per ft = 1/8 × 131 × 26.5² = 11500 '#
For 20 11500 × 20 = 230000 '#
Less Cantilever m 90600
139400 '#

Live Load moment



Uniform load 1600 × 8/20 = 640
600 × 3/20 = 90
730#
Concentration 2940 × 8/20 = 1180
8800
9980# say 10.000#

Cross Beam continued



Live Load Moment	30.000 × 10.25	=	307500 #
	10,000 × 10	=	-100.000
			207500 #
Unif. live Load	1/8 × 730 × 265² =		41100
			248600 #
Dead Load moment say			139400
			388000 #

Design beam for 400.000 # moment

Depth of beam 36½" Stress = 138000# SR = 8.62 − 1.4 = 7.22 □"

Use 2L3 6×4×7/16 = 8.36 or 7.49 net

weight of cross beam

4L3 6×4×7/16	@ 14.3 × 25.5	=	1460 #	Cross Beam	2935
1 web 36×5/16	@ 38.25 × 25.5	=	975	brackets	1074
	Details say		500		4009 call this 4000 #
			2935 #		

Summary for Cross beam and cantilever

	94.5 span	107.0 span	122.0 span	143.0 span
Cross beam + Cant.	254 #	261	230	224
Stringers	360	360	360	360
	614	621	590	584 # per ft of span

Lateral System say 60 # per lin ft throughout

Main truss Dead Load on truss assumed 2400 #
Live load 2600
5000 # per lin ft

Approximate section at Center of span			Depth	Stress	SR gross.	weight per ft
94.5 span	1/8 × 5000 × 94.5² =	558.0000 #	13.5	413.000	~~34.4~~ □" 30.7	~~443~~ # 375
107.0 span	1/8 × 5000 × 107² =	7160.000	14.5	495000	~~41.2~~ 36.7	~~495~~ # 450
122.0 span	1/8 × 5000 × 122² =	9300.000	15.5	600 000	~~50.0~~ 44.5	~~600~~ # 540
143.0 span	1/8 × 5000 × 143² =	12,800.000	17.0	753000	~~63.0~~ 56.0	~~755~~ # 680

122.0' span approximate weight

top chord	50	44.5
bottom chord	50	44.5
web	~~35~~	30.0
details	~~40~~	40.0
	~~175~~ 159.0	@ 3.4 = say ~~600~~ 540 #

	94.5 span	107.0 span	122.0 span	143.0 span
Floor system	614	621	590	584
Lateral	60	60	60	60
Trusses	~~826~~ 750	~~990~~ 900	~~1200~~ 1080	~~1510~~ 1360
	1424 #	1581	1.730	2004
	~~1500~~	~~1671~~	~~1850~~	~~2154~~
875'	556 ~~585~~ tons	614 ~~652~~ tons	675 ~~721~~ tons	780 ~~840~~ tons
~~340~~ per ton	~~199000~~	~~222000~~	~~245000~~	~~286000~~
	186000	209000	230000	265000

Let us try trusses 143.0 span, 172' span and 215' span with top lateral bracings.
Dead + Live Loads on trusses assumed 5000 # per lin ft as above.

		moment	depth	stresses	section	weight of truss
143.0 span	1/8 × 5000 × 143.0² =	12.800.000 #	25.0	511000	37.90"	460 # per ft
172.0 span	1/8 × 5000 × 172.0² =	18.500.000	~~28.0~~ 27.5	670.000	49.6	603
215.0 span	1/8 × 5000 × 215.0² =	28900.000	~~37.0~~ 35.0	~~903.000~~ 825000	~~67.0~~ 61.2	~~845~~ # 745

Lateral Bracing assumed 180# per lin ft of truss

	143' span	172' span	215.0 span
Floor system	584	580	575
Lower Lateral	60	60	60
Upper Lateral	180	180	180
trusses	920	1200	1490
shoes say	60	60	60
	1804	2080	2365# per ft
875'	705 tons	811 tons	923 tons
¥340.00 per ton	240,000.00	276000.00	314000.00

Design of Pier bottom of pier El = 35.0 about
Center to center of trusses = 26.5 tangent = 28.5

coping 6 × 28.5 = 171.0
6' dia 28
199 .92 立坪 92 立坪

shaft 5 × 28.5 = 142.5
5' dia 19.6
162.1

8 × 28.5 = 228.0
8' dia 50
278.0

Average 440.1 ÷ 2 = 220

volume of shaft 220 × 21 = 4620 20.9 立坪
volume of well 2 - 46.1 × 41 = 3780 17.5
filling 2 - 86.6 × 41 = 7100 32.8
50.3 立坪
Excavation 2 - 132.7 × 37 = 9820 45.5 立坪

Approximate Cost of one pier -

shaft and coping	136	21.8 立坪	@ 146.50	=	3200
shell of well		17.5 "	@ 168.50	=	2930
filling of well		32.8 "	@ 128.00	=	4200
Excavation		45.5 "	@ 140.00	=	6360
curb shoes					640
Reinf bars					460
					17790
		add for misc exp.			2000
					19790 call this 20.000.00

Total weight of pier - 15700 @ 140 = 2200.000#
superimposed load say 10.000 × 100 1000.000
3200.000#

Unit bearing = $\frac{3200.000}{286}$ = 11200# or 5.0 tons per sq ft
not counting friction

Frictional resistance assumed = 200 × 40.8 × 25 = 204000
2 @ 204000 = 408000

Load at base 3200.000
408.000
2792 000

Unit bearing = $\frac{2792.000}{286}$ = 9730#/□' or 4.3 tons ok

Design of Pier at Elev. 20.0 about dia 12.0

shaft same as above – 21.8 立坪

Well 12' dia	113.0	volume of Concrete shell	42.1 × 56 × 2 = 4720	21.8 立坪
Inside dia 9.5	~~70.9~~	filling	70.9 × 56 × 2 = 7930	36.7 "
	42.1	Excavation –	113 × 52 × 2 = 11750	54.3 "

Estimate of Cost

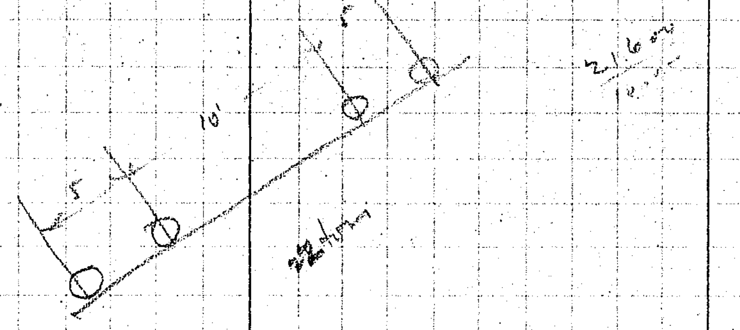
shaft and coping	21.8 立坪	@	146.50	=	3200
Shell of well	21.8	@	168.50	=	3670
filling " "	36.7	@	128.00	=	4700
Excavation	54.3	@	250.00	=	13600
Curb shoes					640
Reinf bars					460
					26270
	Add for misc exp.				2000
					28270 call this 29000.00

Estimate of cost

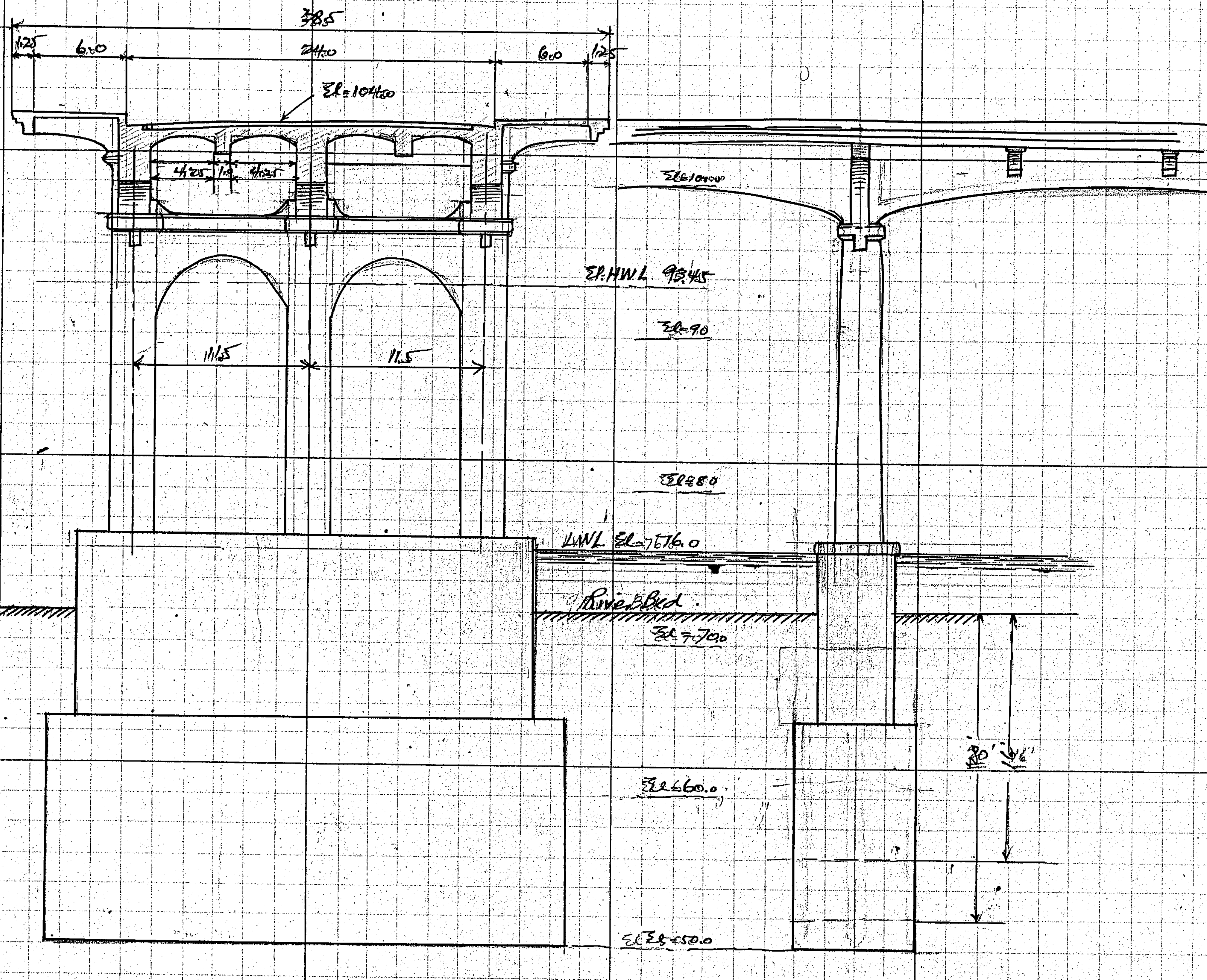
	9 @ 96.7	8 @ 109.0	7 @ 124.0	6 @ 145.0
Deck say	~~87000~~ 73000	~~87000~~ 73000	~~87000~~ 73000	~~87000~~ 73000
steel	186000	209000	230000	265000
Piers	160000	140000	120000	100000
abutments	20000	20000	20000	20000
misc	14000	14000	14000	14000
	453000	456000	457000	472000
misc exp.	~~14000~~	~~14000~~	~~14000~~	~~14000~~
	~~467000~~	~~470000~~	~~471000~~ ×	~~486000~~

Estimate of Cost

	6 @ 145.0	5 @ 174	4 @ 218
Deck say	~~87000~~ 73000	~~87000~~ 73000	~~87000~~ 73000
steel	240000	276000	314000
Piers	145000	116000	87000
abutments	20000	20000	20000
misc	14000	14000	14000
	492000	499000	508000.00
	~~14000~~	~~14000~~	~~14000~~.00
	~~506000~~	~~513000~~ ×	~~522000~~



Reinforced concrete design -
max span of reinforced concrete beam will be 40±. Let us estimate the bridge for 21- 41.5' span with 7 sets of 3 continuous girder span
Cross section of structure as shown in sketch below:-



Floor slab clear span 4.25
Dead Load pavement .25 asphalt 33#
Concrete slab 7" assumed 88
121# per sq. ft

slab say 7" same as for steel girder span see page 4

Concrete stringer span length say 10.0
Rear wheel motor trucks with impact 8800#
moment as simple span = 4400 × 5 = 22000 '#
as continuous beam 0.8 × 22000 = 17600 '#

Dead Load 121 × 5.25 = 635 #
beam say 150
785#

$m = \frac{1}{10} \cdot 785 \cdot 10^2 = 7850$
L L M 17600
25450 '#

Depth required $= \sqrt{\frac{25450 \times 12}{12 \times 95}} = 16.4"$ make depth of beam 19" over all

Cross beam and cantilever bracket make 24" deep. 12" wide

Sidewalk slab 4" thick with 3/4" wearing course.
Fascia girder 1.5 0' in cross section ok.

増田淳氏関係資料（独立行政法人 土木研究所蔵）

Dead Load Cantilever moment at ℄ girder

			arm say		moment
Handrail say	150# per ft		× 7.0	=	1050
Fascia girder 1.5@150 =	225#		× 7.0	=	1575
Sidewalk slab	300		× 3.5	=	1050
Cantilever bracket say	120		× 3.0	=	360
	795				4035'# per lin ft.

Load on outside girder

Extra load on outside girder $\frac{4035}{11.5} = 350$#

Cantilever side say 800#

Between girder	9.5 × .58 × 150 / 2 (pavement 160)	415	Pos. Dead Load moment = $\frac{1}{10} \cdot 3400 \cdot 40.5^2$ = 558000
Concrete stringer		75	neg. = $\frac{1}{8} \cdot 3400 \cdot 40.5^2$ = 700.000
Cross beam say		110	
Filler		75	
main beam and curb + gutter	9.0 × 150 =	1350	
		~~3175~~# 3335	call this 3400 #

Load on center girder

Slab and pavement	121 × 11.5 = say	1400	Pos Dead Load moment = $\frac{1}{10} \cdot 3120 \cdot 40.5^2$ = 512000
Stringer		150	neg. = $\frac{1}{8} \cdot 3120 \cdot 40.5^2$ = 640000
Cross beam say		220	
filler say		150	
main beam	8@150	1200	
		3120	

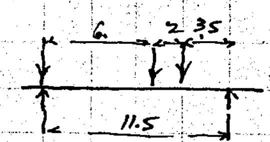
Live load moment on outside girder

600 × 3.5 = 2100

Extra load on main girder $\frac{2100}{11.5}$ = say 180

load 600

Concentrated load

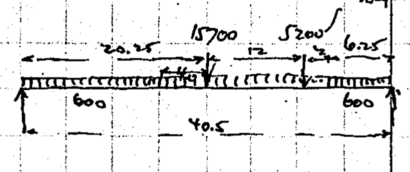


8800 × $\frac{20.5}{11.5}$ = 15700# Rear wheel

5200 Front wheel

Uniform load say 600# per ft

Live Load moment



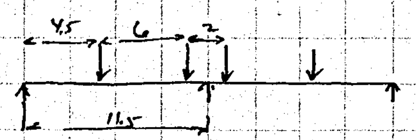
concentrated load $\frac{15700}{2} \times 20.25 \times 0.8$ = 126,000

$\frac{1}{10} \cdot 600 \cdot 40.5^2$ = 226000

600
180
1380

+ 352000'# neg m say 440.000'#

Live Load moment on center girder



8800 × $\frac{30}{11.5}$ = 23000# Rear wheel

7700 Front wheel

neglect front wheel and add rear wheel conc. on ℄ span combined with unif. live load.

Conc. 11500 × 20.25 × 0.8 = 186000

Unif. $\frac{1}{10}$ × 1150 × 40.5^2 = 189 000

+ 375000'# − 470,000'#

Summary of moments

	~~Outside~~ Center girder		Outside girder		
	+ moment	− moment	+ moment	− moment	
LL ~~DL~~ m	~~352000~~ 375000	~~440.0~~ 470.000	352000	440.000	live load
DL m	512000	640.000	558000	700.000	Dead load
	887000	1110000	910000	1140000	

Design beam for 850000'# pos moment and 1000.000'# neg moment

Make depth 4.5' from crown of roadway.

Concrete in Floor Slab etc
Fascia girder 1.75 | concrete $\frac{4680}{216}$ = .217 tsubo per ft
Sidewalk slab 2.00
curb .25 | Concrete CB. $\frac{3}{216}$ = .014
Outside girder 9.00 | .231 tsubo per ft.
Inside girder 4.00 | Reinforcing bars .231 @ 1800 = 420# per lin ft.
Floor slab 5.40
Stringer 1.00 | Total Concrete in superstructure = 875 × .231 = 202 tsubo
23.40 × 2 = 46.80 sq ft | " reinf. in " = 875 × $\frac{420}{2240}$ = 164 tons
cross beam say 3.0 sq ft for full ditch

Approximate Area of Forms
bottom ~~40~~.0
Fascia 2.0
beam 8.0
" 3.5
" 2.0
35.5 call this 1.2 tsubo per ft. | 5.0 面坪 per 1. tsubo of concrete

Design of Pier
Superimposed load per girder Dead Load say 3000
Live Load " 2000
5000#

$\frac{5000}{2240}$ × 40.5 = say 93 tons per girder
or 3 × 93 = say 280 tons per pier
column assumed 3.3 × 9 or 9 × 144 = 1300 ☐" 160%☐" ok
Concrete in columns and cross beam
columns 3 @ 3.3 × 22 = 594
bent 5 × 3 × 17 = 255
849 @ 150 = 127,500# 3.93 坪
Reinforcing bars - 34 - 1"☐ @ 2.67# × 26 = 2360#
misc bars 400
3 @ 2760 = 8300
cross beam say 700
9000# = 4. tons

concrete in Shaft
26 × 5 = 130
5' dia say = 20
150 × 12 = 1800 @ 150 = 270000#
Concrete = 8.33 cubic tsubo
Concrete Base - 8 × 34 × 12 av = ~~2860~~ 3270 @ 150 = ~~429~~ 490.000# Area = ~~46~~ 272
concrete = ~~13.25~~ 15.10 tsubo

Total load on base
Superimposed load 15000 × 415 = 622000
column & cross beams 127500
Shaft 270.000
base ~~429~~ 490.000
~~1148~~ 1509 500 ÷ ~~46~~ 272 = 5550 #/☐' = 2.48 tons

Estimate of Cost of one pier
Base 15.10 立坪 @ 146.50 = 2210.00
Shaft 8.33 " @ 146.50 = 1220.00
columns 3.93 " @ ~~188.5~~ = 740.00
Reinf. bars 4 tons @ 230.00 = 920.00
Excavation 30.0 立坪 @ 100.00 3000
8090.00 call this ~~8500~~ 8000.00

Estimate of cost :—

concrete in slab + beams	202 tsubo	@	260.00	=	52500.00
Reinf. bars	164 tons	@	230.00	=	37700.00
Pavement in Roadway	535 坪	@	25.00	=	13400.00 ~~28600.00~~
Finish of sidewalks &c	365 坪	@	4.00	=	1460.00
Handrail	1800 ft	@	8.50	=	15300.00
			misc		15700
					135560.00

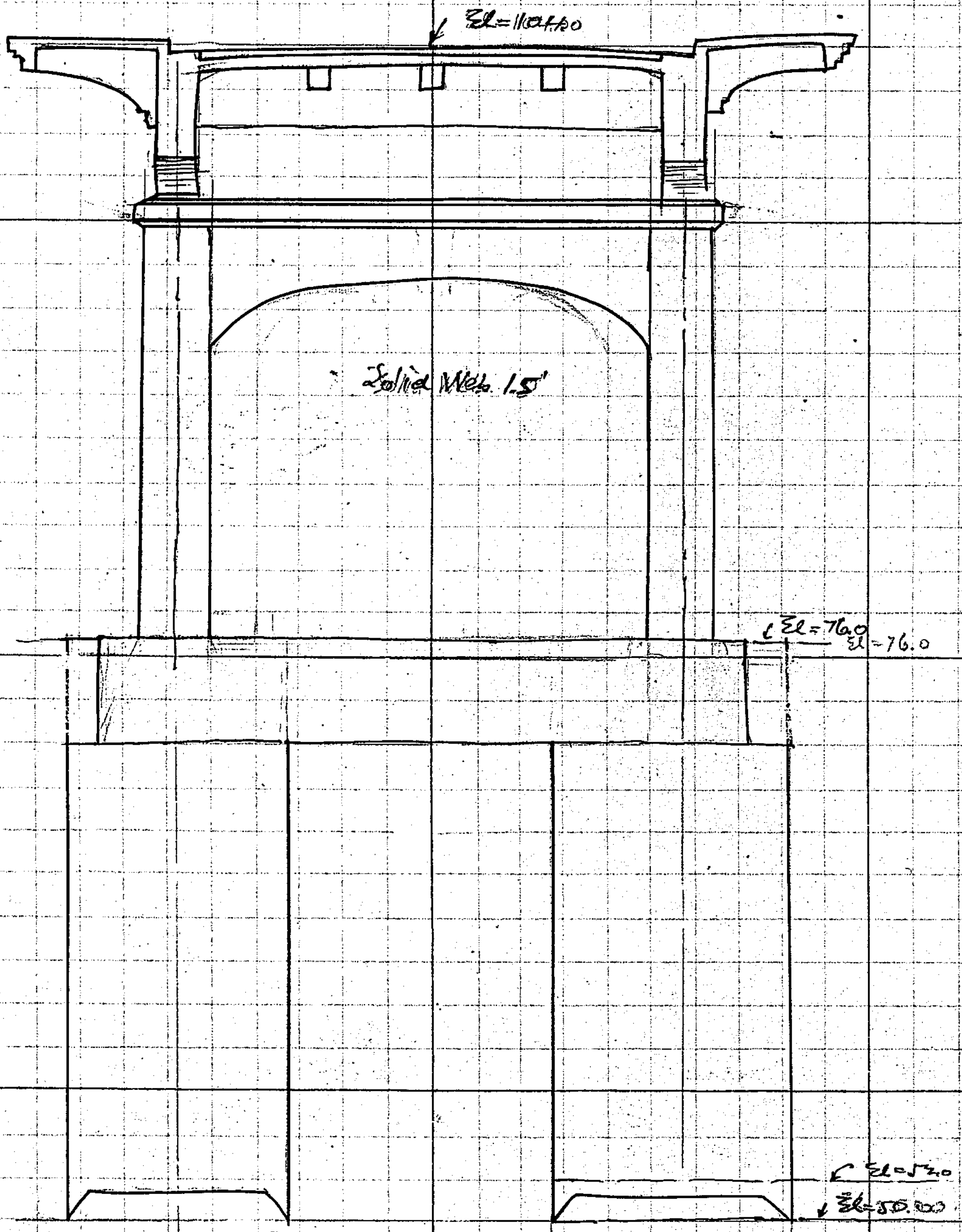
20 piers @ 8000.00 ~~8500.00~~ 160000.00

2 abutment 18000.00

misc Expense 14000.00

327560.00

Design of Concrete girder



concrete in Decks

Fascia girder	1.75
sidewalk slab	2.00
curb + gutter	1.00
slab	7.10
Stringer	1.50
main beam	8.50
cross beam say	4.50

26.35 × 2 = 52.70

weight 52.70 @ 150 = 7900 #

Handrails 2 @ 150 300

Roadway Pavement 33 × 10.5 = 350

8550 # per ft

Live load full 24 × 150 = 3600

12 × 100 = 1200

4800

13350 #

load on pier 13350 × 41.5 = 553.000 #

concrete in Deck $\frac{52.70}{216}$ = .244 tsubo per ft

.244 × 875 = 214 坪

Reinf. bars 172 tons

Substructure :—

well = 11' dia 95.0

filling 9' dia = 63.6

$\frac{31.4 \times 20}{216}$ = 2.9 cubic tsubo

filling $\frac{63.6 \times 20}{216}$ = 5.9 cubic tsubo

excavation $\frac{95.0 \times 24}{216}$ = 10.5

2 @ 2.9 = 5.8 立坪

2 @ 5.9 = 11.8 立坪 } 17.6 立坪

Excavation 2 @ 10.5 = 21.0 立坪

shaft 8.0 立坪 @ 32400 # = 260000

well 17.6 立坪 @ 30000 = 528000

788000

superimposed load 553000

1341,000

Unit bearing $\frac{1341000}{2 @ 95}$ = 7050 #/□' or 315 tons

Counting friction 200 × 34.5 × 12' = 83000

1341,000

1258,000 unit bearing = 6600 #/□' or 295 tons ok

Estimate of cost of one pier.

Shaft concrete	7.9 立坪	@ 150.00	=	1200.00
Shaft forms				450.00
well shell	5.8 立坪	@ 150.00	=	870.00
" forms				400.00
filling	11.8 立坪	@ 120.00		1420.00
Excavation	21.0 立坪	@ 75.00		1575.00
Reinforcing bars + shoes				1300.00
				7215.00

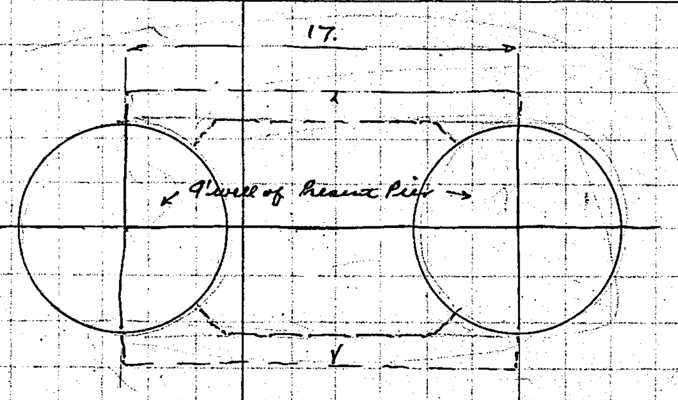
Estimate of cost of bridge – Concrete spans 41.5' Pier Type B.

Concrete in floor	214 tsubo	@ 250.00	=	51300.00	
Reinforcing bars	172 tons	@ 230.00	=	39600.00	
Pavement in roadway	535 坪	@ 25.00	=	~~28600.00~~ 13400	
Finish of sidewalks	365 坪	@ 4.00	=	1460.00	
Handrail	1800 lin ft	@ 8.50	=	15300.00	
			misc	5200	1~~3~~2 6260.00
	20 piers @ 7215				144300.00
	2 abutments				18000.00
	misc Expense				14000.00
					30~~2~~ 560.00

Remodeling of present substructure for 87' deck girder spans.
Cross section of superstructure as shown on pp 4.
Bearing on base. see pp. 1.

weight of Pier	670.000
From deck 192000 × 4 =	768.000
	1438.000

$\frac{1438.000}{2 @ 63.4} = 11400\#/0'$ or 5.06 tons/0'



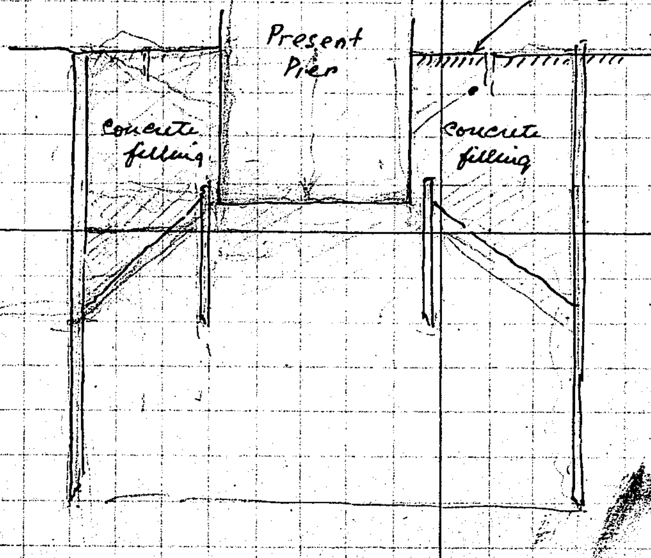
Remodeling of 4 Shore Piers.
Filling of space between wells.

volume of concrete 12 × 17 = 204 − 64 = 140
volume say $140 \times \frac{18}{216}$ = 11.7 tsubo
Excavation say 15 tsubo

Estimate of cost

concrete	11.7 立坪	@ 130	=	1520.00
Excavation	15.0 立坪	@ 100.00	=	1500
Sheet piling	18 坪	@ 20.00	=	360
				3380 call this 3500.00

Remodeling of 5 river Piers.



$\cancel{22}^{15} \times \cancel{38}^{32} \times \frac{10}{216} = \cancel{38}^{22}$ 坪 ~~say less~~

Concrete sheet piling 60 坪

Estimate of cost	~~38~~ 22 @ 120 = ~~4560.00~~		2640.00
Excavation	~~38~~ 22 @ 40 = 1520.00		880.00
Sheet piling	60 坪 @ 65.00 = 3900.00		3900.00
			7420.00

Preliminary Estimate of 淀大橋 for Kioto Prefecture

Estimate of cost

4 piers	@ 3500^{00}	=	14000^{00}
5 piers	@ 7420^{00}	=	37000^{00}
9 piers casing	400	=	3600^{00}
Remodelling 2 abutments	1000	=	2000^{00}
Temparary Bridge	$2.5 \times 146 = 365$ @165	=	60000^{00}
			116600^{00}
Deck			85460^{00}
Steel			158500^{00}
misc Expense			14000^{00}
			374560^{00}

Preliminary Estimate of 郷車村 for Kioto Prefecture

General layout of this Bridge from right bank 4 @ 60-0 = 240
21 @ 43-0 = 903
1143 ft about

Total length say 1145 ft

General cross section same as shown pp. 16 24' roadway, 2-6 sidewalks

Reinforced concrete spans.

Concrete in Deck	.244 坪 per ft × 1145 =	279.0 坪	@	250.00	=	69700.00
Reinforcing bars		224 tons	@	230.00	=	31500.00
Pavement in Roadway		668 坪	@	25.00	=	16700.00
Finish of sidewalks + curb + gutter		475 坪	@	4.00	=	1900.00
Sidewalk Handrails		2350 lin ft	@	8.50	=	20000.00
		misc say				5000.00
						164800.00

Design of pier for 60' span

Base $10 \times 36 \times \frac{20}{216}$ = 33.3 坪

shaft average 5.5 dia. 23.7

5.5 × 24.0 = 132.0

$155.7 \times \frac{17}{216}$ = 12.25 tsubo

Reinforcement say 3 tons

Estimate of cost

Base	33.3	@	150.00	=	5000.00
shaft	12.25	@	200.00	=	2450.00
Reinf.	3 tons	@	230.00	=	690.00
Excavation	40 坪	@	120.00	=	4800.00
					12940.00

call this 13000.00

Design of pier for 43' span

Base $8 \times 34 \times \frac{12}{216}$ = 12.6 坪

$5 \times 5 \times \frac{30}{216}$ = 3.5 坪

16.1 坪

Bent say 4.00 tsubo

Reinforcing bars 3.5 tons

Estimate of Cost

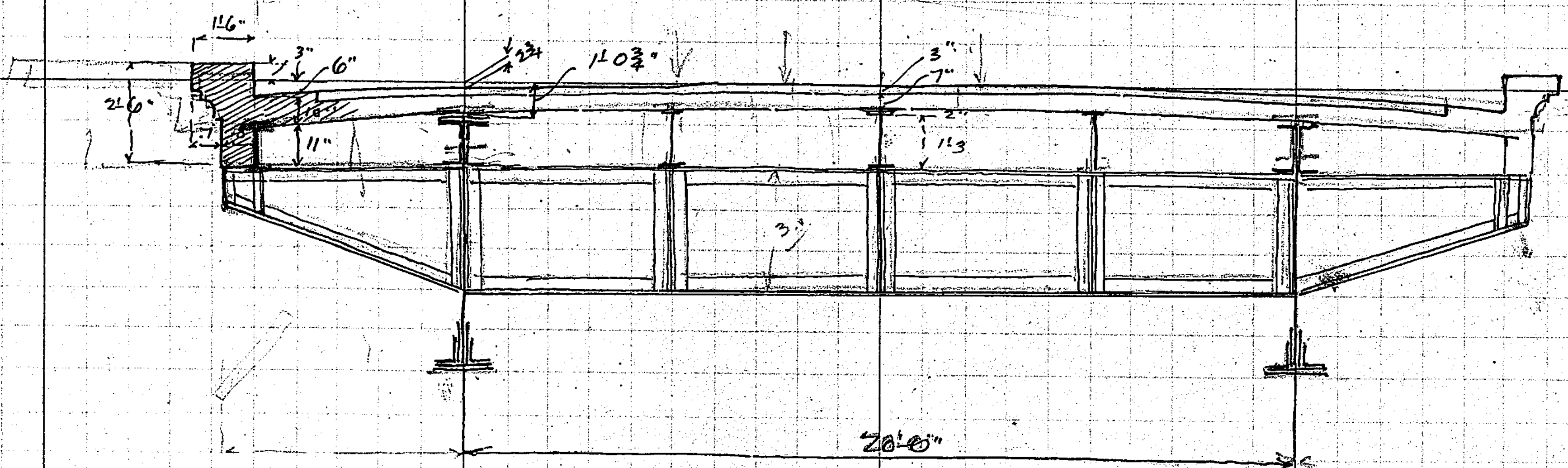
Base	16.1 tsubo	@	150.00	=	2420.00
Bent	4.0 "	@	220.00	=	880.00
Reinf.	3.5 tons	@	230.00	=	805.00
Excavation	20.0	@	100.00	=	2000.00
					6105.00

call this 6100.00

Estimate of cost of bridge -

super structure concrete portion -		164,800.00
Extra steel in 60' span		10,000.00
4 piers	@ 13000	52000.00
3 piers	@ 7500	22500.00
17 piers	@ 6100	103500.00
2 abutments	@ 9000	18000.00
misc.		14000.00
		424800.00

Consulting with Resident Engineer of 淀川改修事務所, Face to face of both abutments is determined as 828.0 ft counting bearing on abutment, let us assume 10 spans @ 83.3 or 82'-9½" or 81'-0" between end bearings. This layout approved by the Engineer Mr. Sakamoto and we will make estimate for this span length and roadway 5 ton clear without sidewalk.
panel length assumed 5@16.2' stringer spacing etc as shown on sketch below.



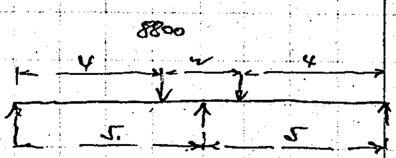
Roadway slab. see pp 4 7" slab
½" bars 5½" centers

Longitudinal stringers. Span length 16.2 spacing of stringer = 5.0'
Dead load moment

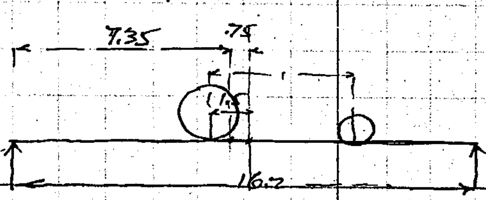
3" asphalt pavement	33#	
7" concrete slab	88	
	121 × 5 =	605
Dead load beam assumed		45
		650#

$m = \frac{1}{8} \cdot 650 \cdot 16.2^2 = 21300^{\#}$

Live Load motor trucks with impact Rear wheel = 8800#
Front wheel = ⅓ of rear



$R_1 = 8800 \cdot \frac{4}{5} = 7050$
7050
Rear wheel ---- 14100# center of gravity = 1.5
Front wheel 4700
18800



		with impact	without impact
Bending moment =	$18800 \times \frac{7.35^2}{16.2}$ =	62700	47100
Dead Load moment		21300	21300
		84000#	68400

$Sm = \frac{84000 \times 12}{16000} = 63.0$

$Sm = \frac{68400 \times 12}{16000^{\#}}$ 51.2

Single concentration at center with impact = 14100 $m = 7050 \cdot 8.1 =$ 57100
DL 21300
78400

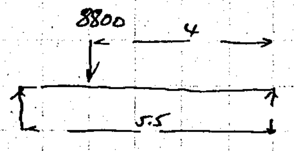
$Sm = \frac{78400 \times 12}{16000} = 58.7$ 15" × 42# I $Sm = 58.9$ ok

Fascia girder (Reinforced concrete). —
Dead Load 2.5 × .8 = 2.00
.7 × .7 = 0.50
2.5 @ 150# = 375#
Handrail 150
525# per lin. ft.

Dead Load Floor 2.5 × 121 = say 355
880#

$m = \frac{1}{10} \times 880 \times 16.2^2 = 23100^{\#}$

Live Load.



$R = 8800 \times \frac{4}{5.5} = 6400^{\#}$

$m = \frac{6400}{2} \times 8.1 = 26000 \times 0.8 = 21000$

D.L.m 23100 23000

49100 44000

Steel Area $= \frac{44000 \times 12}{\frac{7}{8} \times 27 \times 16000} = 1.40''$

use 3 - 3/4" ▫ bars = 1.32 ▫" top and bottom

Approximate Cantilever moment

Dead load 880# × 5.5 = 4840 1# × 16.2 = 78400 1#

Live Load say 10.000 × 5.5 = 55000

133400 1#

Depth say 3.0 Stress = 44200# 44200 ÷ 16000 = 2.76 ▫" net

use 2L 3½ × 3½ × 3/8" 4.96 ▫" gr 4.21 ▫" net. ok

Cross beam between main girders.

span length 20.0 spacing 16.2'

Dead load say 130.0" × 16.2 = 2100# $m = \frac{1}{8} \times 2100 \times 20^2 = 105000$

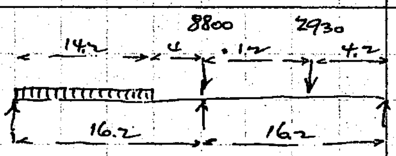
less cantilever moment = 78400

26600 1#

Live Load. $R = 2930 \times \frac{4.2}{16.2} = 760^{\#}$

8800

9560 call this 10.000#

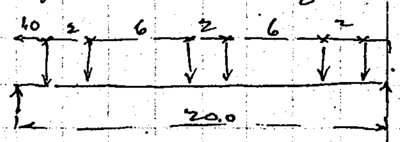


Uniform load $= \frac{14.2^2 \times 100}{16.2 \times 2} = 620^{\#}$

Dead load girder say 100

720#

Unif. L.L. m = $\frac{1}{8} \times 720 \times 20^2 = 36000^{1\#}$



30.000

moment = 30.000 × 9 = 270.000 1#

10.000 × 14 = 140.000

130.000

U.LL m 36.000

D.L. 26600

192600

web 36" × 5/16 = 11.25 ▫" 1/8 web = 1.41 ▫" D = 2.85 Stress = 67500 SR = 4.22 - 1.41 = 2.81 ▫"

use 2LS 3½ × 3½ × 3/8 = 4.96 ▫" gr 4.21 ▫" net ok

main section - 4LS 3½ × 3½ × 3/8 @8.5 = 34.0

36 × 5/16 - 38.25

72.25

25% - 16.00

88.25 call this 90# per lin. ft

wt of 1 floor beam = 90 × 32 = 2900#

use same section for End floor beam 6@ 2900 = 17400# per span

Bottom bracing 60# per lin ft

Design of main girder -

Dead load Floor 121 × 15 = 1820 $m = \frac{1}{8} \times 2345 \times 81.0^2 = 1930.000^{1\#}$

Fascia girder 375

Handrail - 150

2345# per ft

Live Load $2500 \times \frac{12.5}{20} = 1560^{\#}$ per lin ft $m = \frac{1}{8} \times 1560 \times 81.0^2 = 1.280.000^{1\#}$

25.000# concentration at Center $m = \frac{25000}{2} \times 40.5 = 506.000$

3.716.000

Design girder for 3.800.000 1# moment

web = 72 × 1/2" = 360" ⅛ web = 4.50"
m = 3,800,000 1# Depth = 6'-0½" b to b. d = say ~~5~~.5 = S = 690.000 SK = 43.0 − 4.5 = 38.50"
Compression flange.
4Ls 6×6×5/8" @ ~~5.75~~ 7.11 = ~~23.00~~ 28.44
1Pl. 14"×5/8 = 8.75 52'
~~1Pl. 14×5/8~~ = ~~8.75~~
1Pl. 14×1/2 = 7.00 38'
52.94
8.75
44.19 □"

Tension flange.
2Ls 6×6×5/8 @ 7.11 = 14.22 11.72 net
2Pls. *12×5/8 = 15.00 12.50
1Pl. 14×5/8 = 8.75 7.50 52'
1Pl. 14×5/8 = ~~8.75~~ 7.00 ~~7.50~~ 6.00 38'
44.97 ~~37.72~~ □" net
46.72 39.22

weight of one girder − 82'-9" out to out
1 web. 72"×1/2 @ 12.24 × 82.75 = 10030
flanges 4Ls 6×6×5/8 @ 24.2 × 82.75 = ~~4~~100
2Ls 6×6×5/8 @ 24.2 × 82.75 = 4050
2Pls. 12×5/8 @ 25.5 × 82.75 = 4220
2Pls 14×5/8 @ 29.75 × 52 = 3100
2Pls. 14×5/8 @ 29.75 × 38 = 2260
32760#
Details say 22% − 7200
39960 call this 40000#
483# per lin ft
966# per lin ft

Structural steel in bridge
Flooring 3@50" = 150#
Floor beams − 210#
Bottom bracings 60
main girder 966
1386# × 830/2240 = 513 tons
5% 27
540 tons

Design of Pier same as for PP 10.

Paving − 30 × 833/36 = 695 坪

Concrete in deck. .58 × 30 = 17.5
coping − 4.5 (5.0)
22.0 ÷ 216 = .1050 × 833 = 88 坪
Reinforcing bars. 88 × 1650 ÷ 2240 = 64 tons

Finish of coping − 7 × 833/36 = 162 坪

Handrail − say 1750 lin ft

Revised design of 淀大橋 for Kioto Prefecture —

Estimate of Cost

Deck

Concrete in Floor	88 立坪	@ 245^{00}	=	21600^{00}
Reinforcing bars	64 tons	@ 230^{00}	=	14700^{00}
Asphalt Pavement	695 坪	@ 25^{00}	=	17400^{00}
Finish of Coping	160 坪	@ 5^{00}	=	800^{00}
Handrail	1750 尺	@ 8^{50}	=	14900
misc. say				69400
		misc.		4000
				73400^{00}

Estimate of Cost

	Deck	73400^{00}
	Steel 540@280 =	151000^{00}
Substructure	9@11000 =	99000^{00}
Abutment		18000^{00}
misc. Expense		14000^{00}
		355400

355400 --- call this 355000^{00}

Total length of bridge between face to face of abutments 1146.7 ft or between bearings say 1150 ft

17 spans @ 67.5 = 67'-1 5/16" — 66'-9" between end bearings
16 spans @ 71.8 = 71'-4 19/32 — 70'-0" between end bearings
15 spans @ 76.5 = 76'-0 3/32 — 75'-6" " " "

Cross section of roadway same as shown on pp. 23.

Let us estimate 70'-0" span

Floor system complete say 3@55 = 165# per lin ft
Floor Beam say — 210
375# per lin ft

Bottom bracing say 70# per lin ft

Design of main girder —

D.L. on one girder say 2400# per lin ft $m = \frac{1}{8} \cdot 2400 \cdot 70^2 = 1470000$ '#

Live Load Uniform load — 1560 $m = \frac{1}{8} \cdot 1560 \cdot 70^2 = 955000$

Conc. 25000# at center $m = \frac{25000}{2} \cdot 35.0 = 437000$

2862000 '#

Design girder for moment of 2900000 '#

web assumed 5.5' high 66" × 7/16 = 28.9 □" 1/8 web = 3.61 □"

Depth = 5.54 − .34 = 5.2 S = 557000# SR = 34.8 − 3.61 = 31.20 □" net required.

main gross section — 40.5

weight of girder

web —	28.9
flanges	48.0
details	15.1
	92.0 □" · 3.4# = 314#

For 2 girders say 630# per ft

Total steel in structure

Floor system complete	375#
Bottom bracings	70
main girders —	650
shoes + misc —	50
	1145# per lin ft

Total weights of one span = $1145 \times \frac{71.5}{2240}$ = 36.6 tons or

For 16 spans @ 36.6 = 585 tons or call this 600 tons —

Estimate :—

Concrete in structure $88 \cdot \frac{1150}{833}$ = 122 立坪
Reinforcing bars — 90 tons

Pavement $4.5^k \cdot \frac{1150}{6}$ = 870 平坪
finish of coping + gutter 230 平坪
Handrails — 2400 lin ft

Estimate of cost in deck

concrete in Floor	122 立坪	@	245.00	=	29900.00
Reinf. bars	90 tons	@	230.00	=	20700.00
Asphalt pavement	870 坪	@	25.00	=	21800.00
Finish of Coping + gutter	230 坪	@	5.00	=	11500.00
Handrail —	2400 lin ft	@	8.50	=	20400.00
					104300.00

Preliminary Estimate of Cost 郷峯村 for Kioto Prefecture

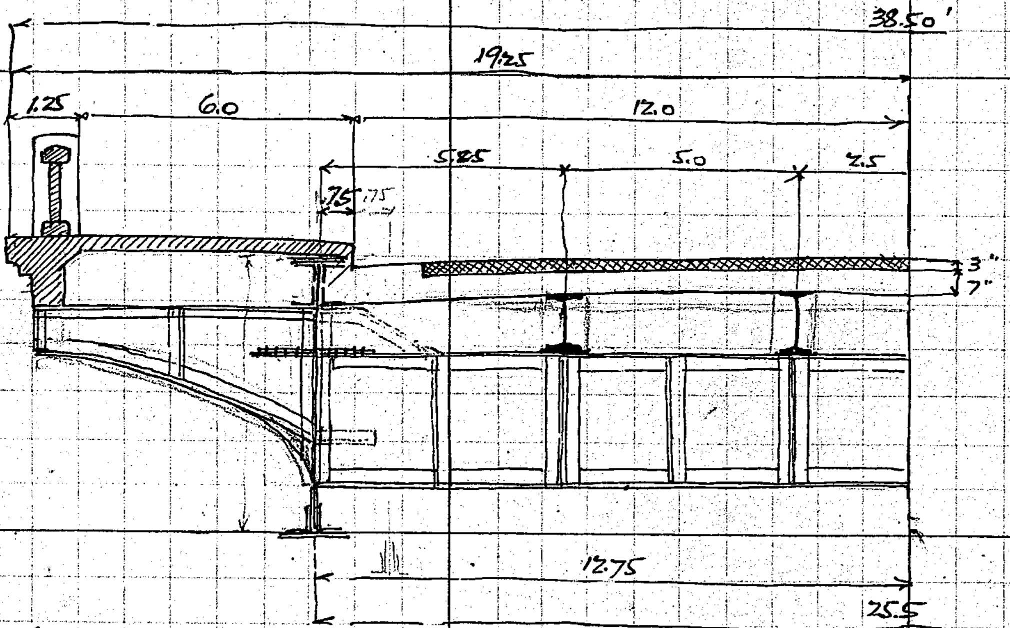
Estimate of Cost	Deck	104,300.00			
Steel*	600 @ 280 =	168,000.00			
Piers	15 @ 11000 =	165,000.00			
abutments		20,000.00			
misc Expense say		20,000.00			
		477,300.00	475,000.00	960#	495.00
		355,400	360,000.00	695	517.00
		832,700			

Total length of bridge is determined to be 828.ft between faces of abutments and center to center of end bearings 833.ft making 10 spans, each span length will be 83.3ft or 82'-9½" span length between ℄ of end bearings assumed 81'-0" panel length = 5 @ 16.2 = 81'-0"

Consulting with Mr. Ikeda, chief Engineer of Civil Engineering office, Home Affairs' Department and Mr. Miura, Bridge Engineer of the same office 24' Roadway and 2-6' sidewalks were fixed and we will make design on this basis.

Loading 国道 loading as fixed by 内務省 bridge specification.

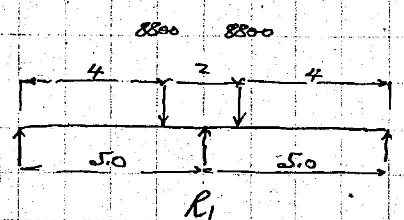
Cross section of bridge as shown in sketch below:—



Slab thickness 7"
Reinforcing bars ½"⌀ - 5.5" centers
Stringer span length 16.2 spacing - 5.0'
Dead load -
3" Asphalt pavement 33
7" Concrete slab 88
121# · 5 = 605
beam assumed 50
655#

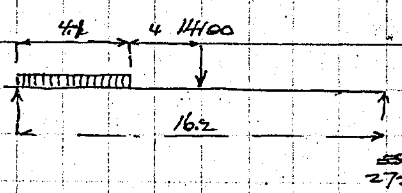
$m = \frac{1}{8} \cdot 655 \cdot 16.2^2 = 21500^{\#}$

Live load motor trucks with impact Rear wheel = 8800# Front wheel ⅓ of rear wheel = 2930#



$R_1 = 8800 \cdot \frac{4}{5} = 7050$
7050
Rear wheel 14100
Front wheel 4700
Total 18800#

Uniform load $\frac{100 \cdot 4.2^2}{2 \cdot 16.2} = 54.5 \quad .5 = 27.2.0$



moment = 27.2 · 8.1 = 2200#
$\frac{14100}{2} \cdot 8.1 = 57100$
59300
Dead load m 21500
80800

96.5
5
6.85
108.35

6'-6½
2½
¾
6'-9¾"

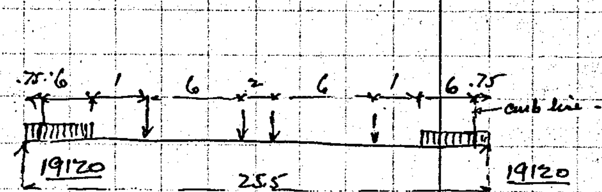
6'-9¾"
6.7895
.063
6.8525

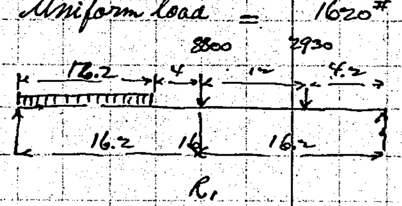
section modulus = $\frac{80800 \cdot 12}{16000} = 60.5$

Use 15" · 45# I Sm = 60.8 for inside stringers
15" · 42# I Sm = 58.9 for outside stringers

Approximate weight of Floor Beam

Uniform load = 1620#





$R_1 = 8800 \cdot \frac{1}{1} = 8800$
$2930 \cdot \frac{4.2}{16.2} = 760$
9560#

Uniform load $\frac{12.2^2 \cdot 100}{2 \cdot 16.2} = 460^{\#}$ per ft

1620# · 6.75 = 10900
460 · 8 = 3680

moment due to concentration = 19120 · 12.75 = 244000#
9560 · 8 = -76500"
167500#

86'-6
1-9
84'-9

Moment U.L. 10900 · 12.75 = 139.000
less 10900 · 9.37 = 102.000
37.000

Moment U.L. 3680 · 12.75 = 47.000
$\frac{460 \cdot 8^2}{2}$ = 14700
32.300
236.800#

Dead Load cantilever moment

Handrail –	150	moment = 10100 × 5.75 = 58000#
Fascia girder	325#	Dead load beam say 5000
3.33 @ 150	150#	63000#
	625# per ft × 16.2 = 10100# conc.	

Dead Load moment of Floor beam Uniformly distributed load say 130# per sq ft × 16.2 = 2100#

100

2200

$m = \frac{1}{8} \times 2200 \times 25.5^2 = 179000$

less 63000

Resulting Dead Load m 116,000#

Live Load m 236,800

352800#

Depth = 36 × 5/16 = 11.25 ⅛ web = 1.41 D = 2.89 S = 122.000 SR = 7.62 − 1.41 = 6.21 □" net

Use 2∟s 5 × 3½ × 7/16 = 7.06 gr or 6.19 □" net

weight of cross beam

Web	1 Pl. 36 × 5/16	@ 38.25	× 25.5	=	975
flanges	4∟s 5 × 3½ × 7/16	@ 12.0	× 25.5	=	1225
Int stiffs	10∟s 3½ × 3½ × 5/16	@ 7.2	× 3.0	=	216
stiffs	8∟s 3½ × 3½ × 5/16	@ 7.2	× 3.0	=	173
stiffs	4∟s 3½ × 3½ × 3/8	@ 8.5	× 3.0	=	102
Fills	8 Pls 7 × 7/16	@ 10.41	× 2.4	=	200
Fills	4 Pls 3½ × 7/16	@ 5.20	× 2.4	=	50
					2941
		5%			149
					3090# per piece

Cantilever Bracket

web	1 Pl. 24 × 5/16	@ 25.50	× 6.0	=	153
flanges	4∟s 3½ × 3½ × 3/8	@ 8.5	× 6.0	=	204
stiffs	4∟s 3 × 3 × 5/16	@ 6.1	× av. 2.0	=	49
		mis details say			70
					476 call this 500#

Total for Floor ~~beam~~ system complete

Floor beam 3090

2 cantilevers @ 500 = 1000

4090 ÷ 16.2 = 252# per lin ft

2 stringers @ 48 96

2 " @ 50 100

448

call this 450# per lin ft

4090 × 5 = 20450

8275 × 196 = 16200

36650# call this ~~36500~~ 40500#

4090

40740

~~450# per ft~~ 500# per lin ft

Lower Lateral Bracings –

4∟s 4 × 3 × 5/16	@ 7.2	× 30	=	865#
strut 2∟s 3 × 3 × 5/16	@ 6.1	× 16.2	=	198#
1 Pl. 6 × 5/16	@ 6.38	× 16.2	=	100
connections –	4 @ 30			120
	1 @ 50			50
	2 @ 15			30

1263# × 5 = 6320# 78# per lin ft

Design of 鴨大橋 for Kioto Prefecture 31.

Main girder

Dead Load 2341# per lin ft see pp 7.

metal - Floor system - 500

lower Laterals 78

Main girder 1000 } 1578

~~3919~~ call this 4000#

Floor - 2341
metal - 790
3131#

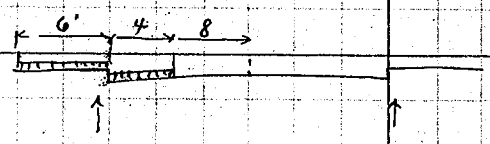
$m = \frac{1}{8} \times \cancel{4000 \times 81.0^2} = \cancel{3280.000}$ 1#

$m = \frac{1}{8} \times 3131 \times 81.0^2 = 2570.000$ 1#

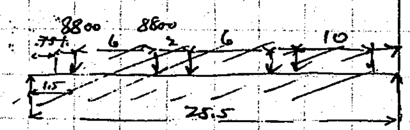
Live Load -

Uniform load = 1000# per lin ft

$m = \frac{1}{8} \times \cancel{1000}\ 600 \times 81.0^2 = \cancel{820.000}\ 493.000$ 1#

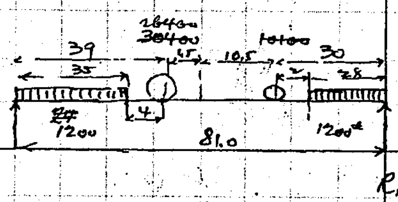


Live Load Concentration -



$8800 \times \frac{\cancel{18}}{\cancel{25.5}} \times 3 = \cancel{30400}\ 26400$# Rear wheel -
~~10100~~ 8800 Front wheel -
~~40500~~ 35200

moment due to concentration $= \cancel{40500}\ 35200 \times \frac{39^2}{81} = \cancel{760.000}\ 660.000$



R_1 $\frac{1200 \times 35^2}{81 \times 2} = 9080$
$\frac{1200 \times 28 \times 67}{81} = 27700$
36780#

moment = 36780 × 40.5 = 1490.000
1200 × 28 × 26.5 = − 890.000
600,000

Summary of L.L.m Unif. 493.000
Conc. 660.000
Unif. 600.000
1,753.000
Dead Load 2570.000
4,323,000#

web assumed 72 × ½ = 36.0□" ⅛ web = 4.50□" 76 × ½ = 38.0 ⅛ web = 4.75

assumed flange section = 4Ls 6×6 × ¾ @ 8.44 = 33.76 d = 6.1 707.000 44.2
1 cov. Pl. 14 × ⅝ = 8.75 50' 4.75
1 cov. Pl. 14 × ⅝ = 8.75 36' 39.45
51.26 □" gross

bottom flange. 2Ls 6×6 × ¾ @ 8.44□" = 16.88 net
2 Pls 12 × ~~¾~~ ⅝ @ 18.00 = 15.00
1 cov. Pl 14 × ⅝ @ 8.75 = 7.50 50'
1 cov. Pl. 14 × ⅝ @ 8.75 = 7.50 36'
52.38 gr. 43.88

weight of one girder -

web	1 Pl. 72 × ½"	@	122.4	×	82.7	=	10050	
Flanges.	6Ls 6×6 × ¾	@	28.7	×	82.7	=	14250	
2 Pls.	2 Pls 12 × ¾	@	30.60	×	82.7	=	5070	34485
2 cov.	2 Pls. 14 × ⅝	@	29.75	×	50.0	=	2975	32700
2 cov.	2 Pls 14 × ⅝	@	29.75	×	36.0	=	2140	1785
Stiffs	30 Ls 5 × 3½ × ½	@	13.60	×	5.0	=	2040	
End Stiffs -	8 Ls 5 × 3½ × ⅝	@	16.80	×	6.0	=	805	
fills -	4 Pls 18 × ¾	@	45.90	×	5.5	=	1010	
web splice -							1200	
Flange splice							2000	
misc. details -							1500	
							42840#	

call this 43000
Shoes say 3000
46000#

For 2 girders = 90.000#

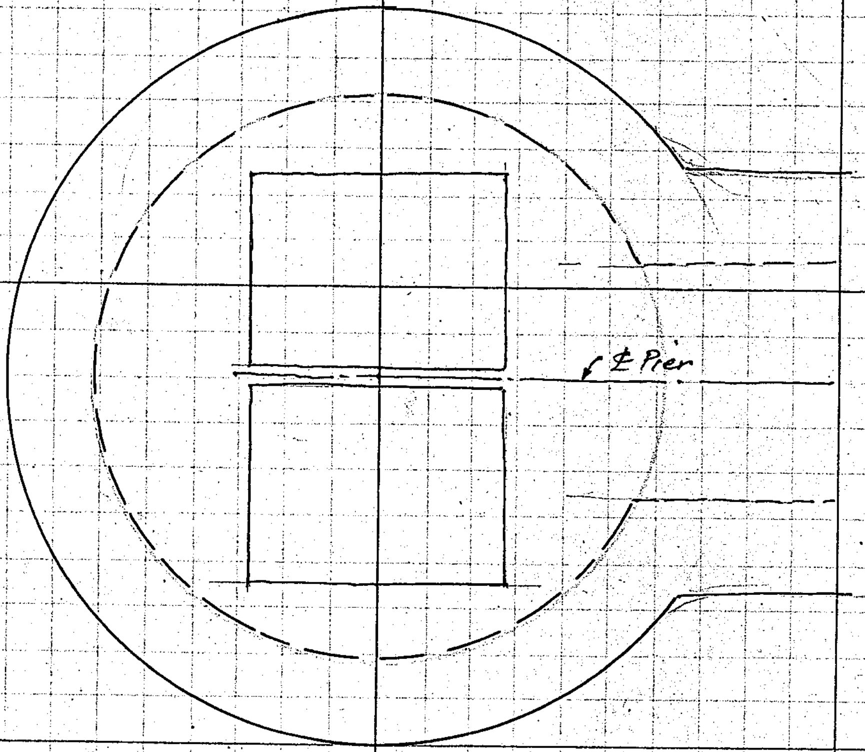
560# per lin ft

Summary for steel - Floor beams & stringers 40500#
Lateral Bracing - 6300
Main girders & shoes 90.000
136800# or 61 tons

For 10 spans @ 61.0 = 610 tons - @ 280 = 171,000
73.000

Design of pier
Superimposed load -
Dead load 2 @ ~~4000~~ 3200 × 81 - 518.000 ~~647.000~~
Live load 3600 × 81 = say 300.000
818.000#

bearing area = 220.000 / 400# = 550　21" × 27" = 550



coping 6.5 dia × ~~25.5~~
2 - 6.5 dia　33.2 × 2 = 66.4
3.5 × 20 = 70.
136.4
say 140

shaft
top 5' dia - 19.6
bottom 7.5 dia - 44.2
63.8 ÷ 2 = 31.9

height = 25'　vol = 31.9 × 25 = 800 / 216 × 2 = 1600

web. 25.5
6.25
19.25 × 2 × 10 = 385
2125

call this 10.0 立方

wt = 324.000#

well 24'
13' dia - 132.7
10.5 dia 86.6
46.1

volume of shell - 2 - 46.1 × 24 = 2220
top + bottom filling 2 - 86.6 × 8 = 1390
middle filling - 2 - 86.6 × 16 = 2770
Total - 6380 @ 150 = 960.000#

Total weight shaft 324.000
well 960.000
1284.000
Superimposed load - 818.000
2,102,000

bottom area 13.5 dia - 2 @ 143 = 286.0
unit bearing = 2102.000 / 286 = 7350　3.28 tons

Let us try 12.5 dia well -
12.5' dia = 122.7
10 = 78.5
44.20

volume of shell - 2 × 44.20 × 24 = 2120　10.0 立方
top + bottom filling 2 × 78.5 × 8 = 1260　5.8 立方
2 × 78.5 × 16 = 2520　11.7 立方
5900　27.5 立方

wt 5900 @ 140 = 825.000
superimposed load = 818.000
shaft 324.000
1,967.000

13' dia　132.7 × 2 = 265.4　unit bearing = 7420
3.3 tons

Counting friction of 12' at bottom @ 200# per sq. ft
200 × 41 × 12 = say 100.000
weight of concrete shell - 310.000 -
Counting 1/2 as friction - 150.000 ÷ 265.4 ÷ 2240 = 0.25 tons
~~1967.000~~

Resulting bearing = 3.05 tons

増田淳氏関係資料（独立行政法人 土木研究所蔵）

Estimate of pier –

1:2:4 Concrete	25.8	@	150.00 =	3870
1:3:6 Concrete	11.7	@	120.00 =	1410
Forms for shaft	44 7/4	@	15.00 =	660
Forms for shell of well –	94 7/4	@	10.00 =	940
Excavation	27.5	@	100.00 =	2750
curb shoe				500
Reinf bars				650
				10780.00 call this 11000.00

Estimate of Cost

Deck see pp. 11	73000
Steel – 610 tons @ 280	171000
Piers 9 @ 11000	99000
abutments say 2 @ 10,000.00	20000
misc Expense –	10000
	373000.00

After several preliminary designs and layouts, the span length of bridge is determined by the officials concerned in this case (京都府土木課長近新三郎氏, 淀川改修事務所長内務技師坂本助太郎氏, 内務省大阪土木出張所長岡崎芳樹氏, 内務省土木局第一技術課長池田圓男氏 仝内務技師三浦七郎氏) and the following is the layout

Location of bridge - The present wooden bridge is old enough to be replaced by the new bridge on account of heavier traffic and piers not strong enough to carry new loading. We decided to build new bridge 6 Ken from edge of the present bridge upstream so as not to interfere with the present bridge during construction, and also on account of flood situation old and new piers to be in line of stream, that is, to keep the same span length. Assumed flood elevation = 96.5 and the top of bank will be 5.0 ft higher, that is, El = 101.5 and the low point of steel not lower than this elevation. Several investigations of preliminary estimates we found the girder span is most economical in this layout, we adopted the following span lengths for this bridge site.

8 spans	87.10	=	696.8	center to center of piers.
2 spans	69.0	=	138.0	
			834.8	between faces of parapet walls End abutments

The end spans shortened by river channel which will be narrowed widening embankment at this bridge site and down.

Depth of pier - By the opinion of Mr Sakamoto of Yodogawa improvement office max depth of scouring is 24 below low water level which is El = 74.0 and we decided to make bottom of well Elev = 50.0 which will be OK for this stream.

Width of Roadway - After investigation Naimusho officials directed us to make Roadway 24.0 ft with 2 sidewalks of 6.0 ft at both ends.

Bearing of Soil. assumed 3.5 tons per sq ft which will be safe from the figure of the present bridge -

Present bridge 85 ft span wooden Howe truss span with wooden floor planking and joists

Floor Planking and Joists assumed .35 × 18 = 6.3 cubic ft @ 40# = 380# per lin ft

Truss (one truss) Top chord 2@ .8 × .8 = 1.28
bottom chord 1.28
diagonals say 2.00

For two trusses 2@ 4.56 cubic ft @ 40# = 365
Lateral bracings 65
810# per ft of span

Total Dead load on pier 810 × 85 = 69000#
Live Load say 18 × 50 × 85 = 76000
145000#

Weight of Present Pier.
Shaft 6 × 25 × 15.2 = 2280
less say 280
2000 @ 140 = 280000
Wells 9' dia 2@ 63 × 21 @ 140 = 370000
650000
Total load. 795000#

Unit bearing pressure = $\frac{795000}{2@634}$ = 6290 #/□' or 2.8 tons per sq ft.

These Piers stand on clay soil. From the findings of borings soil at Elev = 50.0 is sand and gravel soil and its bearing power will be 50% more than the clay soil in which the present piers stands. Hence let us assume safe bearing power of 2.8 × 1.5 = 4.2 tons per square ft

京阪電氣鐵道橋 Double tracks Electric Ry bridge.
Span length 124'-0" 127'-0" center to center of piers.
Dead Load of superstructure 150 tons assumed including deck
Live load 35 tons car with impact of 25% = 43.7 tons assumed on one track or For two tracks say 87.4 tons per pier.
Total Superimposed load = 237.4 tons.

weight of shaft say 480.000 #
wells 2 @ 86.6 · 30 = 5200 @ 140 # = 728.000

1208.000 # or 538 tons

Superimposed load - 237.4

775.4 tons

Unit bearing pressure = $\frac{775.4}{2 @ 86.6}$ = 4.52 tons per square ft

Let us assume the bearing power of 3.5 tons per square ft for designing substructure.

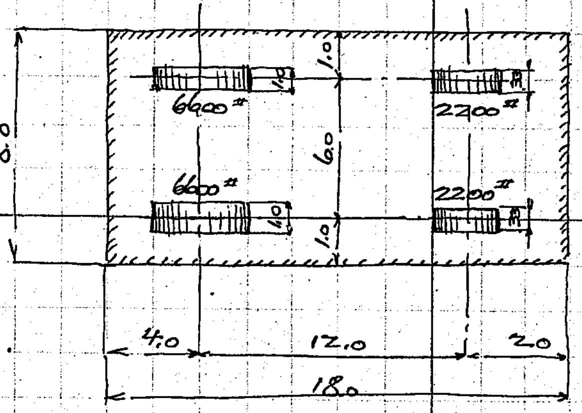
Assumed Loadings

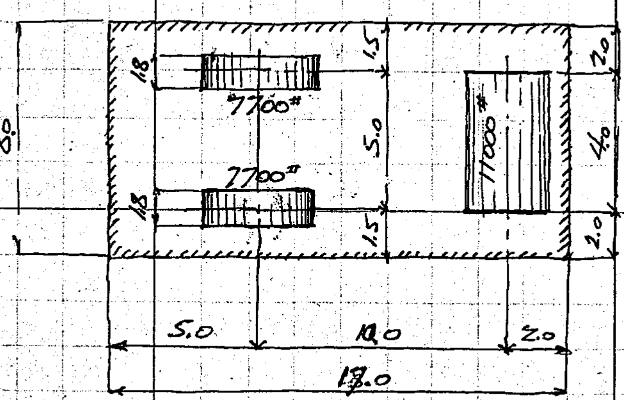
Uniform load $q^{kg/m^2} = \frac{100.000}{170 + l}$ where l = span length in meter

under 30 meter in span $q = 500\ {}^{kg}/{}_{m^2}$ = 102.5 #/□'

motor trucks loading - 8000 kg Road Roller concentration 12000 kg.

Equivalent in ft and pounds Equivalent in ft and pounds





1/3 impact assumed for motor trucks when running; No impact for uniform live load and roadroller concentration. Two lines of motor trucks loading for roadway; when running only one motor trucks on one span with impact; Without impact continuous line of motor-trucks assumed on span; Only one roadroller assumed on one span without impact, Unoccupied space of motor trucks or road roller to be filled by Specified Uniform load

Assumed weight of materials -

Creosoted wood block	56 #	per	cubic	ft
mortar	120	"	"	"
Asphalt	130	"	"	"
Plain Concrete	140	"	"	"
Reinforced Concrete	150	"	"	"
Structural Steel	490	"	"	"

Allowable stresses

Tension of structural steel or reinforcing bars	16000 #/□"
Extreme fibre stress in rolled shapes net section	16000
Tension in flange of girder net section	16000
Shearing on plate girder web gross section	10.000
Shearing on shop driven rivets	10.000
Shearing on field driven rivets and turned bolts	8000
Extreme fibre stress of pins	24000
Bearing on field rivets and turned bolts or Pins	16.000
Bearing on Shop driven rivets	20.000
Expansion roller per lin inch. d = diameter of roller	600 d
Bearing on concrete	500 #/□"

Compression on plate girder flange $16000 (1 - 0.012\ l/b)$

where l = unsupported length of flange

b = width of flange

Compression member gross section $16000 (1 - 0.0055\ l/r)$

where l = unsupported length of member

r = radius of gyration

with max. of 14000#/□" and l/r limited to 100 for main member and 150 for secondary members.

Stresses in 1:2:4 concrete

Direct compression		500#/□"
Compression due to bending	positive moment	640
	negative moment	710
Combined stresses of direct compression and bending		500
Punching shear		128
Shearing stress without web reinforcement		57
" " with " "		128
Bond stress of reinforcing bars		85

Miscellaneous data

Ratio between moduli of elasticities of steel and concrete 15

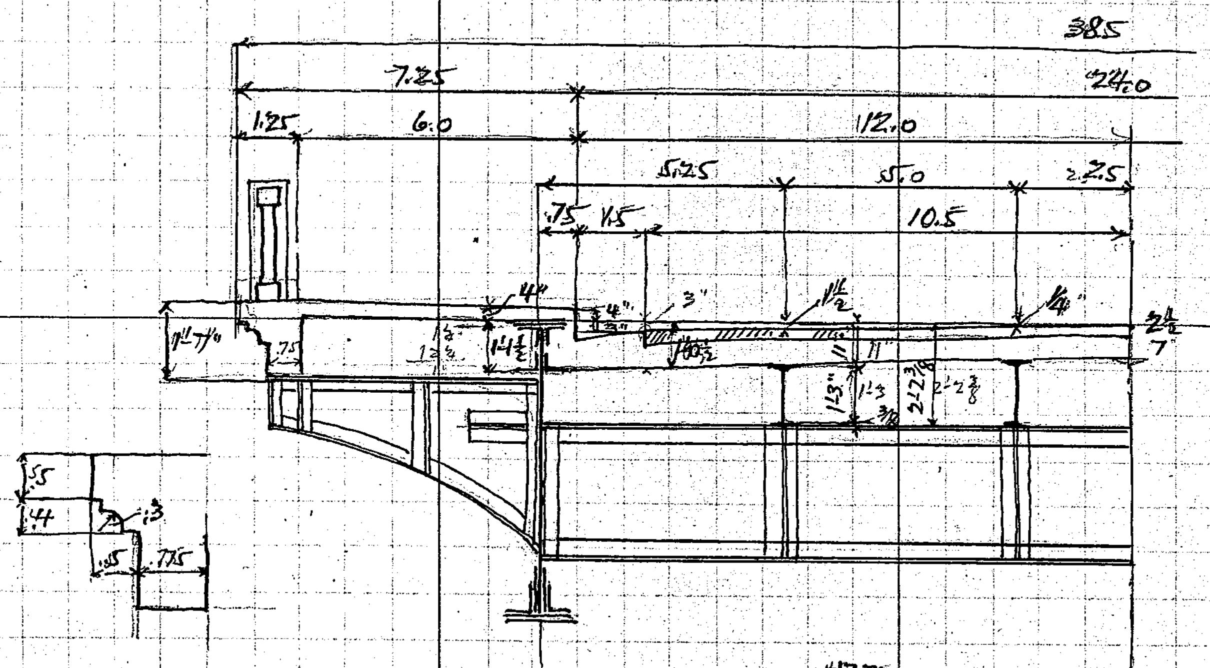
Expansion Coefficient of Concrete = 0.0000055 per 1° of Fahrenheit

" " " steel = 0.0000060 " " "

Temperature change of concrete = ±40° where standard temperature

" " " steel = ±60° assumed as 62°F

Cross section of structure



Reinforced Concrete Roadway slab.

span length 5.0 center to center of stringers

Dead Load

$moment = \frac{1}{10} \times 115 \times 5^2 = 288^{\#'}$

Asphaltic Concrete pavement 2½" thick 27#

Concrete slab assumed 7" 88

115# per square ft.

Live Load

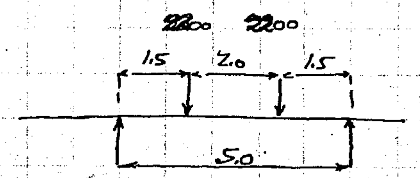
Rear wheel of motor truck	6600		Front wheel of motor truck	2200
⅓ impact	2200		⅓ impact	730
	8800# per conc.			2930#

Distribution of wheel concentration

$= 0.6 \times 5. + 1.0 = 4.0$ in direction of bridge.

For one ft strip of slab 2200# concentration is assumed.

Live Load moment

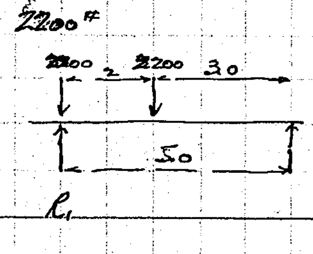


moment = 2200 × 1.5 = 3300#

For continuity of slabs m = 3300 × 0.8 = 2640#

Dead Load moment 288

Total moment = 2928#



Reaction R_1 2200

2200 × $\frac{3}{5}$ 1320

3520#

Dead Load shear say 290

3810#

Effective depth of slab for 640#/□" concrete stress and 16000#/□" steel stress

$d = \sqrt{\frac{2928}{105}} = 5.28"$

Insulation 1.22

6.50" make slab 7" over all with effective depth of 6"

Steel Required $= \frac{2928 \times 12}{\frac{7}{8} \cdot 6 \cdot 16000} = 0.42$ □" per 1' strip

Use ½"φ bars 5½" centers = .43 □"

Unit shear $= \frac{3810}{\frac{7}{8} \cdot 6 \cdot 12} = 60.7$ #/□" Use bent-up bars for shear at supports

Bond stress $= \frac{3810}{\frac{7}{8} \cdot 6} = 726$# Bond area required $= \frac{726}{85} = 8.55$

½"φ bars perimeter = 1.57 8.55 ÷ 1.57 = 5.5 bars req'd per 1' strip.

Sidewalk slab Assume 3¼" slab with ¾" wearing course on top

Dead Load 4" slab = 50# per sq. ft. $m = \frac{1}{10} \times 152.5 \times 6.0^2 = 550$#

Live Load 102.5

152.5

Effective depth $= \sqrt{\frac{550}{105}} = 2.3"$

.95

3.25" o.k.

Steel required $= \frac{550 \times 12}{\frac{7}{8} \cdot 2.3 \cdot 16000} = .205$ Use ⅜"φ bars 6" centers = .22

Every other bars to be bent up at support to take care of negative moment

Roadway stringer (S_1) near center of bridge.

Span length of bridge.

Center to center of piers = 87.1 = 86'- 7⅛"

For expansion space 1⅛

86'- 6"

Bearing. 1'- 6"

85'- 0" center to center of bearings.

Span will be divided into 5 spaces of 16.2 = 81'- 0"

86'- 6"

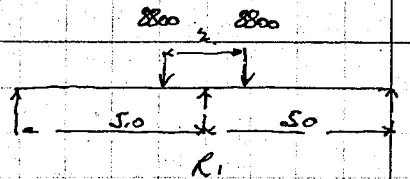
5'- 6" ÷ 2 = 2'- 9" over hang over pier.

Dead Load Floor 115 × 5 = 575 moment $= \frac{1}{8} \cdot 625 \times 16.2^2 = 20800$#

beam .50

625

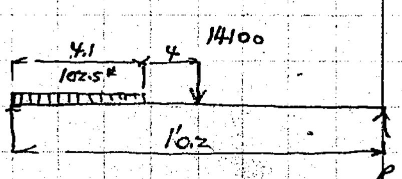
Live Load motor trucks with impact Rear wheel = 8800# Front wheel = 2930#



$R_1 = 8800 \times \frac{4}{5} = 7050$ R Unif. load $= \frac{102.5 \times 4.1^2}{2 \times 16.2} = 53.2$

7050 53.2 × 5 = 266.0

14100#



Moment Conc. 7050 × 8.1 = 57100

Unif. 266 × 8.1 = 2200

59300

Dead Load moment 20500

79800

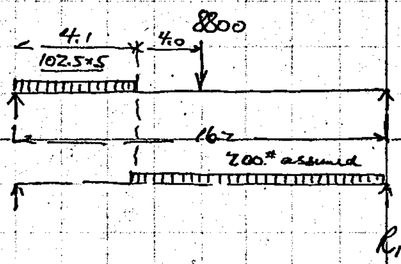
Section modulus required $= \frac{79800 \times 12}{16000} = 59.9$

Use 15" I beam × 45# per ft $S_m = 60.8$ for stringer S_1.

Steel Roadway Stringer S_2 near curb line.

Dead Load on stringer assumed 656# $m = \frac{1}{8} \cdot 656 \cdot 16.2^2 = 21500$ 1#

wheel reaction = 8800#



Unif load U_1; R_1 = 266#

" " U_2 $R_1 = \frac{200 \times 12.1 \times 10.15}{16.2} = 1510$#

moment due to unif loads 266 × 8.1 = 2200

1510 × 8.1 = 12200

14400

Less $\frac{200 \times 8.1^2}{2}$ 6550

7850 1#

Moment due to concentration 4400 × 8.1 = 35600

~~section~~ 43450

Dead Load moment 21500

64950

section modulus required $= \frac{64950 \times 12}{16000} = 48.7$

Use 15" × 42# I $S_m = 58.9$ for side stringer S_2.

Fascia Girder span length 16.2

weight of fascia girder

1.58 × .75 = 1.19

.50 × .75 = .38

1.57 @ 150# = 236# per ft

Handrail assumed 150

Dead Load slab 50 × $\frac{5.25}{2}$ = 132

518 call this 520#

moment $= \frac{1}{10} \cdot 520 \cdot 16.2^2 = 13600$ 1#

Live load on sidewalk 102.5 × $\frac{5.25}{2}$ = 270# per ft

moment $= \frac{1}{10} \cdot 270 \cdot 16.2^2 = 7100$

20700 1#

Depth of beam required for negative moment 710#/□" + 16000#/□" stresses of concrete + steel

$d = \sqrt{\frac{20700 \times 12}{9 \times 122}} = 15.1$" make total depth of beam 18½"

Effective assumed 16".

Steel Area $= \frac{20700 \times 12}{\frac{7}{8} \times 16 \times 16000} = 1.11$ □" or 3 - ¾"□ bars @ .44 = 1.32 □"

Cantilever Bracket

Dead Load moment about ℄ main girder

520# × 5.75 = 3020 1# For 16.2' strip · 3020 = 49000 1#

Dead load cantilever assumed 540# × 2.2 assumed = 1200

50200 1#

Live Load moment 270 × 5.75 = 1550 1# 16.2 × 1550 = 25100

75300 1#

Flange $\frac{75300}{2.0}$ = 37600# Section Required = 2.35 □" net for tension plate

Use 2 L 3½ × 3½ × 3/8" = 4.96 □" 1 web 5/16" thick.

Cross Beam

Dead Load uniformly distributed load say 125# per sq ft 125 × 16.2 = 2030# per ft

Dead ~~Load~~ Floor Beam say 120

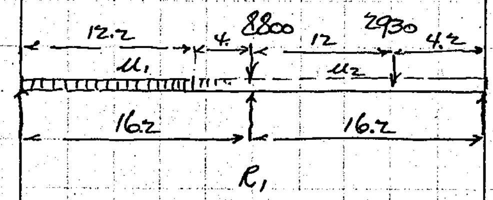
2150# per ft

$m = \frac{1}{8} \cdot 2150 \cdot 25.5^2 = 175.000$ 1#

Less 50200 · 0.8 = 40.000

135.000 1#

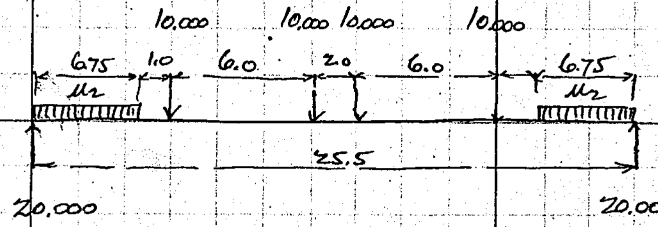
Live Load



Motor truck loading = $2930 \times \frac{4.2}{16.2}$ = 760
8800
9560 call this 10.000#

Uniform load u_1 $\frac{12.2^2 \times 102.5}{2 \times 16.2}$ = 470# per lin. ft.

Uniform load u_2 $\frac{102.5 \times 20.2^2}{2 \times 16.2}$ = 1290# per ft.



1290 × 6.75 = 8700#

moment due to concentration
20.000 × 11.75 = 235.000 1#
less 10.000 × 6 = 60.000
175.000 1#

moment unif. load u_2
8700 × 12.75 =
8700 × $\frac{9.37}{3.38}$ =
29400 1#

moment unif. load u_1 $\frac{1}{8} \times 470 \times 25.5^2$ = 38200
242600 1#

Dead Load Moment 135000
377600

Depth assumed = 36½" web = 36 × 5/16 = 11.25 □" ⅛ web = 1.41 d = 2.89
Stress = 130700# SR = 8.15 □" − 1.41 = 6.74 □" net
Use 2Ls 6 × 3½ × 7/16 = 7.94 gr. or 7.07 □" net

weight of cross beam

web	1 Pl.	36 × 5/16	@	38.25	× 25.50	=	975
flanges	4 Ls	6 × 3½ × 7/16	@	13.5	× 25.50	=	1375
Int. stiffs	18 Ls	3½ × 3½ × 5/16	@	7.2	× 3.0	=	390
End stiffs	4 Ls	3½ × 3½ × 3/8	@	8.5	× 3.0	=	102
Fills	4 Pls	3½ × 7/16	@	5.20	× 2.4	=	50
web splice	2 Pls	12 × 5/16	@	12.75	× 2.4	=	61
misc. sp. say							40
	variation + Rivet heads						150
							3143 call this 3150#

weight of Cantilever Bracket

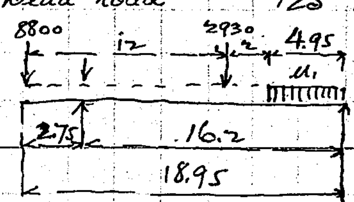
web							
flanges	1 Pl.	24 × 5/16	@	25.5	× 6.0	=	153
stiffs	4 Ls	3½ × 3½ × 3/8	@	8.5	× 6.0	=	204
details say							150
							507 call this 510#

Floor Beam complete cross beam 1 3150
Cantilever 2@510 = 1020
4170# per piece.

End Floor Beam

Br. Use same section as for Int. cantilever bracket
Cross beam span length 16.2 overhang 2'-9"

Dead Load 125#/□' × $\frac{18.95^2}{2 \times 16.2}$ = 1390#
Floor Beam say 110
1500

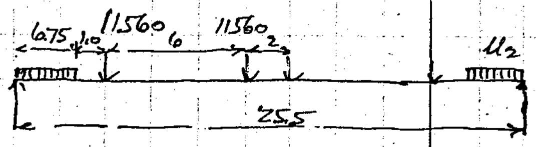


$m = \frac{1}{8} \times 1500 \times 25.5^2$ = 122000 1#
less dead load cantilever moment say 30.000
92.000 1#

Live Load 8800 × $\frac{18.95}{16.2}$ = 10300
2930 × $\frac{6.95}{16.2}$ = 1260
11560#

Unif. load u_1 $\frac{102.5 \times 4.95^2}{2 \times 16.2}$ = 77.6 call this 80#
" " u_2 $102.5 \times 12 \times \frac{12.95}{16.2}$ = 980#

Live Load moment



23120
Unif. 6600

moment conc. 23120 × 11.75 = 271000
less 11560 × 6 = 69300
201700

moment Unif. load U2
16600 × 3.38 = 22300

moment Unif. load U1 $\frac{1}{8}$ × 80 × 25.5² = 6500
230500

Dead Load moment 92000
322500

web assumed 36 × 5/16 = 11.25 $\frac{1}{8}$ web = 1.41 d = 2.89 Stress = 112000 SR = 7.00 − 1.41 = 5.59□"

Use 2∠s 6 × 3½ × 3/8 = 6.84 □" 609 □" net

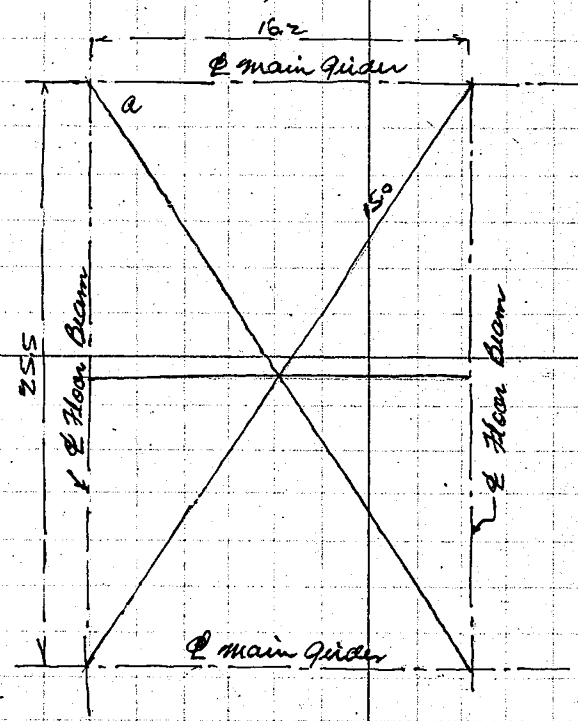
weight of End cross beam

web	1 Pl. 36 × 5/16	@ 38.25	× 25.50	=	975
Flanges	4∠s 6 × 3½ × 3/8	@ 11.70	× 25.50	=	1195
Int. Stiffs	18∠s 3½ × 3½ × 5/16	@ 7.2	× 3.0	=	390
End Stiffs	4∠s 3½ × 3½ × 3/8	@ 8.5	× 3.0	=	102
Fills	4 Pls 3½ × 3/8	@ 4.46	× 2.4	=	43
Misc details					250
					2955 #

weight of End Floor Beam
1 cross beam 2955
2 cant @ 510 = 1020
3975 call this 4000 #

Floor system in one span
2 stringers @ 50 = 100
2 stringers @ 47 = 94
194 # × 86.75 = 15950
Int. Floor Beam 4 @ 4170 16680
End Floor Beam 2 @ 4000 8000
40630 # per span

Lower Lateral Bracing



l/r assumed as 150
r reqd = $\frac{15 \times 12}{150}$ = 1.2

Use 2∠s 4 × 3 × 5/16 in contact back to back

4∠s 4 × 3 × 5/16	@ 7.2	× 29.0	=	836
2∠s 3 × 3 × 5/16	@ 6.1	× 16.2	=	196
1 Pl. 6 × 5/16	@ 6.38	× 16.2	=	100
conns @	4 @ 50			200
center conn				50
details say				30
				1412

call this 5 @ 1400 = 7000 #

Main girder span
span length 85'-0"

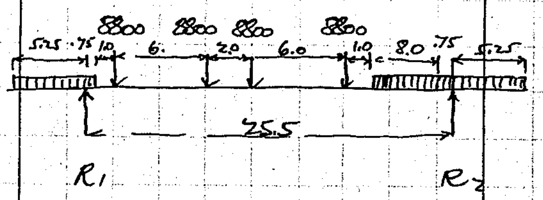
Dead Load	Pavement	½ Roadway	27 × 11 =	297
	concrete slab	"	88 × 11 =	968
	gutter		.75 × 1.5 @ 150 =	169
	curb		.75 × 1.1 @ 150 =	124
	slab		.33 × 6 @ 150 =	300
	Fascia girder		1.57 @ 150 =	236
	Handrail say			150
				2243

Including filler say 2260 #
½ Floor system steel 275
Pl. girder assumed 525
3060 # per lin ft

$m = \frac{1}{8} \cdot 3060 \cdot 85^2 = 2{,}765{,}000$ '#

增田淳氏関係資料
（独立行政法人 土木研究所蔵）

Live Load



Reaction on main girder full uniform load assumed as shown in sketch and motor concentration as shown

Reaction due to Concentration $R_1 = 8800 \times \frac{67}{25.5} = 23100$ Rear wheel

7700 Front wheel

30800#

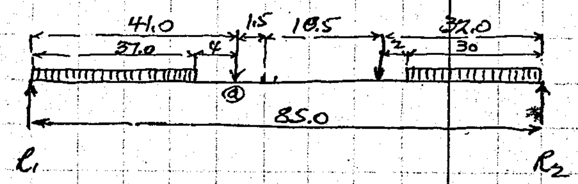
Uniform load $R_1 = \frac{102.5 \times 8 \times 4.75}{25.5} = 133$#

Uniform load on sidewalk $102.5 \times 6 = 615$

748 call this 750# per ft

Uniform load at rear or front of motor truck 16' wide

Reaction $R_1 = 102.5 \times 16 \times \frac{16.75}{25.5} = 1080$# per lin ft



37000

Reaction due to unif. load

32400

$1080 \times 30 \times \frac{15}{85} = 5700$

40.000

$1080 \times 37 \times \frac{66.5}{85} = 31300$

37.000#

moment unif. load at (a) $37.000 \times 41. = 1,518.000$

less $40.000 \times 22.5 = 900.000$

618 000

Moment motor truck loading $30800 \times \frac{41^2}{85} =$ 609000

Uniform load $\frac{1}{8} \times 750 \times 85^2 =$ 678000

Total Live Load moment 1905000

Dead Load moment 2765000

4,670,000 #

web assumed $78 \times \frac{1}{2}'' = 39\square''$ $\frac{1}{8}$ web $= 4.87$ d = 6.1' about Stress = 765000# SR = 47.8 − 4.87 = 42.93

Bottom flange	2Ls 6×6×3/4	@ 8.44	=	16.88	gr	13.88	
	2Pls 12×3/4		=	18.00		15.00	
	1Pl 14×5/8		=	8.75		7.50	52
	1Pl 14×5/8		=	8.75		7.50	37'
				52.38□"		43.88 □" net	

Top flange	4Ls 6×6×3/4	@ 8.44	=	33.76	
	1Pl 14×5/8			8.75	52'
	1Pl 14×5/8			8.75	37'
				51.26□" gross.	

weight of one girder 85'-0" span

web	1Pl 78"×1/2"	@	132.6	×	86.5	=	11480
Flanges	6Ls 6×6×3/4	@	28.7	×	86.5	=	14900
2Pls	2Pls 12×3/4	@	30.60	×	86.5	=	5300
2Pls	2Pls 14×5/8	@	29.75	×	52.0	=	3100
2 cov.	2Pls 14×5/8	@	29.75	×	37.0	=	2200
Stiffs	30Ls 5×3½×3/8	@	10.4	×	5.5	=	1720
Stiffs	8Ls 5×3½×5/8	@	16.8	×	6.5	=	873
30 fills	30Pls 3½×3/4	@	8.93	×	4.5	=	1210
fills	4Pls 18×3/4	@	45.9	×	4.5	=	827
web splices	3@ 320						960
flange splice	4@ 700						2800
Rivet heads + misc. details							2000
							47370#

47370# or $\frac{47370}{86.5} = 547$# per ft

wt of 2 girders	2@ 47370 =	94740 □"
Floor system		40630
Lateral system		7000
Shoes.		4500
		146870# call this 147000# 65.6 tons.

main girder.

1 - overhang = 2'-9" same as for 85'-0" span

4 @ 16'-2 = 64'-9 5/8

67'-6 5/8

1'-5 3/8

69'-0" out to out

1/8

69'-1/8 — 69.4973 call this 69.5 ft

span length of girder = 67.55'

Details of Floor slab, Steel stringer and Floor beam, Lower laterals same as for 85'-0" span

Steel stringer = 194 × 69.0 = 13400 Lower Lateral Bracing 4 @ 1400 = 5600

Intermediate FB 3 @ 4170 12510

End Floor Beam say 2 @ 4000 = 8000

33910

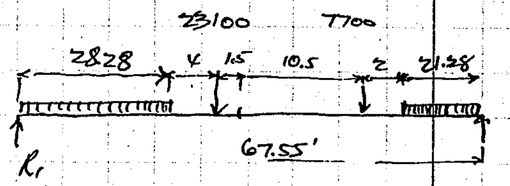
5600

39510 #

Dead Load say 3060# see pp40 $m = \frac{1}{8} \cdot 3060 \times 67.55^2 = 1,750,000$ '#

Live Load moment

Live Load same as for 85'-0" span see pp44



Reaction Unif. load - $1080 \times 21.28 = 23000 \times \frac{10.64}{67.55} = 3620$ #

$1080 \times 28.28 = 30600 \times \frac{53.41}{67.55} = 24200$

27820 #

moment unif. load - 27820 × 32.28 = 898000

Less 30600 × 18.28 = 559000

339000 '#

moment motor truck $30800 \times \frac{32.28^2}{67.55} = 475000$

moment unif. load $\frac{1}{8} \times 750 \times 67.55^2 = 427000$

Total live load moment 1,241,000

Dead load m 1,750,000

Total moment 2,991,000 '#

Try 78 × 1/2" = 39.0" 1/8 web = 4.87 d = 6.1' stress = 490,000 SR = 30.6 - 4.87 = 25.73 □"

Use for bottom flange 2 Ls 6 × 6 × 3/4 = 16.88 □" 13.88 □" net

1 Pl 14 × 1/2 = 7.00 6.00 47'

1 Pl. 14 × 1/2 = 7.00 6.00 35'

30.88 □" 25.88 □" net

Use for top flange 2 Ls 6 × 6 × 3/4 16.88 13.88

2 Ls 6 × 6 × 3/4 16.88

33.76 □"

weight of one girder 67.55' span

web	1 Pl	78 × 1/2"	@	132.6	×	69.0	=	9150
flanges	6 Ls	6 × 6 × 3/4	@	28.7	×	69.0	=	11900
Cov.	1 Pl	14 × 1/2	@	23.8	×	47.0	=	1120
Cov	1 Pl	14 × 1/2	@	23.8	×	35.0	=	832
Stiffs	22 Ls	5 × 3 1/2 × 1/2	@	13.6	@	5.5	=	1645
"	8 Ls	5 × 3 1/2 × 5/8	@	16.8	@	6.5	=	873
fills	4 Pls	18 × 3/4	@	45.9	×	5.5	=	1010
web splices		2 @ 350						700
flange splice		4 @ 500						2000
Rivet heads & misc details								1500
								30730

30730 or $\frac{30730}{69} = 445$ #

weight of two girders 2@ 30730 = 61460#

Floor system 33910

Lateral system 5600

Shoes 4000

104970 call this 105000# 46.8 tons

Summary Structural steel 8 - 85' spans @ 65.6 = 525.0

2 - 67.55' spans @ 46.8 = 93.6

618.6 tons call this 625 tons for Entire bridge -

Design of piers -

Superimposed Load DL say $2 \times 3060 \times \frac{86.5}{2} =$ 264000

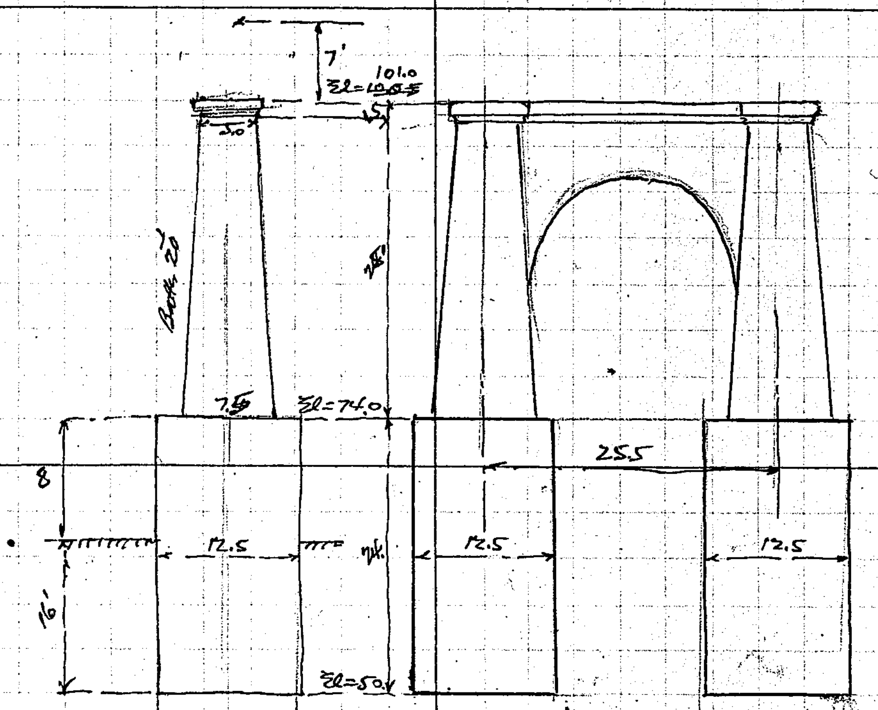
Live load $102.5 \times 36 \times \frac{86.5}{2} =$ 160000

424000

For one shoe 212000# Bearing Area = 18×30 = 540

Unit bearing = $\frac{212000}{540}$ = 393#/□" [illegible]

Superimposed load on pier 848.000#



Coping 2 - 6.25' dia 30.7 × 2 = 61.4

32.5 × 20 = 65.0

126.4 × 1.5 = 190.0

shaft

top area 5' dia 19.6

bottom area 7.5 dia 44.2

63.8 ÷ 2 = 31.9

height = 25' vol = 31.9 × 25 = 800 × 2 = 1600.0

web say 19.25 × 2 × 10 385.0

2175.'

call this 10.0 #7 wt = 324000#

well 24' high

Try 12.5 dia well

12.5' dia = 122.7

10. dia = 78.5

44.2

volume of shell 2-44.2 × 24 = 2120 10.0 #7

top and bottom filling 2-78.5 × 8 = 1260 5.8 #7

middle filling 2-78.5 × 16 = 2520 11.7 #7

5900 27.5 #7

weight of well 5900 × 140 = 825000

shaft 324000

1149000

Superimposed load DL + LL. 848000

1997000

Bottom Area 13' dia = 132.7 × 2 = 265.4 Unit bearing = 7520 #/□' or 3.36 tons/□'

Effect of Earthquake - a = 2000 mm/sec² $\frac{2000 \text{ mm/sec}}{[illegible]}$ = 6.1 ft/sec² g = 32.2

moment due to acceleration $M = \frac{W}{g} a y$

Dead Load superstructure for one shaft = 264000 horizontal force = $264000 \times \frac{6.1}{32.2}$ (.1895) = 50000#

weight of shaft + 1/2 web 162000 " " = 162000 × .1895 = 30700

Total load 426000

Moment at top of well = 50.000 × 33.5 = 1,675,000

say 30700 × 12.5 = 384.000

2,059,000 '#

Moment of Inertia of bottom section - $I = 0.049 \times 90^4 = 3220.000 \text{ (inch)}^4$

Fibre stress = $\frac{2059000 \times 12.45}{74220000}$ = 345 #/□"

Dead load = $\frac{426000}{144 \times 44.2}$ = 67 #/□"   Summary for stress 345 + 67 = 512 #/□"
345 − 67 = 278 #/□"

Use 32 - 1"□ bars @ 2.67 × 30.0 = 2560
32 - 1" bars @ 2.67 × 15.0 = 1280
3840 including hooking call this 2 tons per shaft or 4 tons per pier.

Moment at 8' below top of well

moment superimposed load - 50000 × 41.5 = 2080000
shaft 30700 × 20.5 = 630000
well $\frac{550000}{2 \times 2}$ × .1895 = 26000 × 4.0 = 104000
2814000 '#

moment of Inertia = 0.049 × 12.5^4 = 1200

Fibre stress = $\frac{2814.000 \times 6.25}{1200}$ = 14600   6.5 tons per sq ft

moment of Inertia = 0.049 × 13^4 = 1400
Fibre stress = $\frac{2814000 \times 6.5}{1400}$ = 13050   5.82 tons □'

Counting friction of well below 8' line 12.5' dia 39.2 × 200 × 16 = 125500 #
Resisting moment = 125500 × $\frac{12.5}{2}$ × .675 = 530000 '#
Resulting moment = 2814.000
530.000
2284.000

Fibre stress = $\frac{2284000 \times 6.5}{1400}$ = 10600 or 4.73
3.36
8.09 tons □'

weight of well below 8' line 550000 ÷ 2 = 275000
Resisting moment = 275000 × $\frac{12.5}{2}$ × (.675) (.6375) = 1160000
2284000
1124000 ← !!!

Fibre stress = $\frac{1124000 \times 6.5}{1400}$ = 5220 or 2.33 tons
3.36
5.69 tons   1.03 tons.

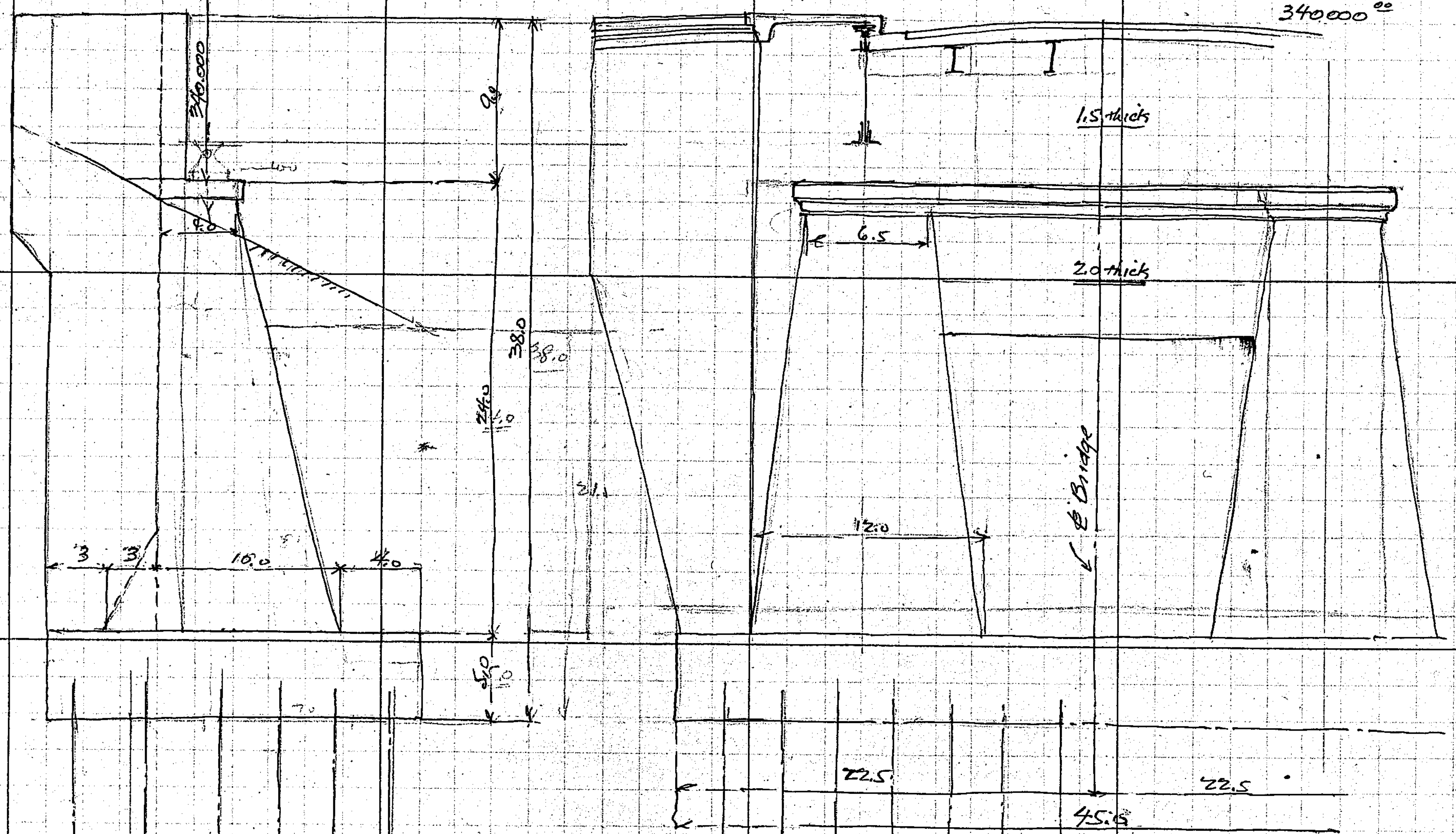
Weight of curb shoe -
24 × 3/8 @ 30.60 #
6 × 6 × 3/4 @ 28.70
59.30 × 40 = 2380 call this 1 ton 2 @ 1.0 = 2 tons

Area of forms in shaft
5.0 dia 15.7
7.5 dia 23.5
39.2 ÷ 2 = 18.6
Area = 18.6 × 26.5 = 493 - 2 @ 493 = 986
20 × 20 = 400
misc say 100
1486
41.3 坪

Area of forms in well -
outside 12.5 dia - 39.3
Inside 10.0 dia 31.4
2 - 70.7 × 24 = 3400 □' or 95.0 坪

Design of abutment

Superimposed load DL. $6120 \times \frac{69}{2} = 212000$
Live Load $102.5 \times 36 \times \frac{69}{2} = 128000$
340000 ##



Approximate Estimate of Concrete in abutment.

½	Front wall	1.5 × 19 × 9 =	256
	wing wall	3.0 × 33 × 10 =	1000
	web	2.0 × 9 × 10 =	180
	Shaft	6.5 × 4 = 26.0	
		12 × 10 = 120.0	
		146.0 ÷ 2 = 73 × 24 =	1750
	Base	20 × 5 × 22.5 =	2250
			5436 ÷ 216 = 25.1 坪 × 2 = 50.2 坪

approximate area of Forms = 150 面坪

Timber Piling - 84 本 18 = 1510 lin ft
Excavation - 20 × 45 × 20 = 1800 8.3 坪
Filling say 15.0 坪

Approximate Reinf. bars. ~~17.5~~ 10.0 tons.

Estimate of materials

Concrete in Floor slabs. forms. 7.K × 140 = 980 面坪

Concrete slabs in Roadway	.58 × 10.5	=	6.10
gutters	.75 × 1.5	=	1.13
curbs	.75 × 1.1	=	0.83
Sidewalks	.33 × 6.0	=	2.00
Fascia girder			1.57
			11.63 call this 12.0

12.0 × 2 = 24.0 cubic ft

volume of Concrete = $24 \times \frac{835.0}{216}$ = 93.0 坪

Design of 淀大橋 for Kioto Prefecture

Reinforcement in Floor slabs 1600# @ $\frac{93.0}{2240}$ = say 67.0 tons.

Length of Handrails. say 1750 lin ft.
curb angle – 835 × 2 = 1670 lin ft.
Expansion joint – 10
do. in slab 20
Electric wiring &c –

Pavement = 3.5 × 140 = 490 面坪

concrete 1:2:4

Cement 11 @ 8.50 =	93.5	93.5
Sand	7.5	5.0
gravel	25.0	25.0
mixing	20.0	20.0
	145.0	143.5

Estimate of cost in superstructure.

Struct. Steel in superstructure	625 tons	@ 280.00	=	175000.00	
Concrete in Flooring 1:2:4	93 立坪	@ 145.00	=	13500.00	49720.00
Forms	980 面坪	@ 5.00	=	4900.00	
Reinforcing bars	67.0 tons	@ 230.00	=	15400.00	
Pavement in Roadway 三寸五分厚	490.0 面坪	@ 25.00	=	12250.00	
Finish of sidewalks & coping say	350.0 面坪	@ 5.00	=	1750.00	
Expansion Joint & other miscellaneous				1000.00	
curb angles	1670.0	@ .55	=	920.00	
				224720	

Estimate of cost of one pier.

Concrete in shaft and coping 1:2:4	10.0 立坪	@ 145.00	=	1450.00
do Forms	~~18.6~~ 41. 面坪	@ 10.00	=	410.00
concrete in shell of well 1:2:4	10.0 立坪	@ 145.00	=	1450.00
Forms	95.0 面坪	@ 6.00	=	570.00
concrete filling 1:2:4 acqueo	5.8 面坪	@ 155.00	=	900.00
Concrete filling 1:3:6	11.7 立坪	@ 119.5	=	1400.00
Excavation say	27.5 立坪	@ 120.00	=	3300.00
Curb shoe	2.0 tons	@ 300.00	=	600.00
Reinf. bars	4.0 tons	@ 230.00	=	920.00
				11000.00

7620 × $\frac{20.5}{24}$ =

Estimate of Cost of one abutment

Concrete 1:2:4	50.0 立坪	@ 145.00	=	7250.00	120.00 6000
Forms	150.0 面坪	@ 15.00	=	2250.00	2250
Reinf. bars	10.0 tons	@ 230.00	=	2300.00	2300
Timber Piling 18' long	84.0 本	@ 20.00	=	1680.00	1680
Excavation Earth	8.0 立坪	@ 5.00	=	40.00	40
Re Filling & filling Earth	15.0 立坪	@ 7.00	=	95.00	95
misc facing stone & other finishing				500.00	500
				14115.00	12815

Handrail 1750 @ 6.00 = 10500.00
Electric lighting &c – 5000.00

Summary of Estimate of Cost 淀大橋.

structural steel in superstructure	175000.00
Concrete slab & Pavement	49720.00
Piers 9 @ 11000.00	99000.00
abutment 2 @ 14115.00	28225.00
Handrail	10500.00
Electric lighting &c	5000.00
	367445.00
	13555
	380000
	485000
	865000

面坪 6.145 = 835

455.00 立坪

総坪 1160 @ 480.00

385000
460000
385000

The total length of the bridge = 1151.2 ft face to face of end abutments
Span length fixed after several preliminary estimates = 16 spans @ 71.85 = 1149.6 ft c to c of piers
center to center of piers in ft = 70' 7 1/16

9 17/32

19/32

71' - 4 3/16 — space between ends of girder 1/16" — bearing of plate — 1' 6"

1' 6 1/2

69' - 9 1/2"

4 panels @ 16' - 2 3/8 — 64' - 9 1/2

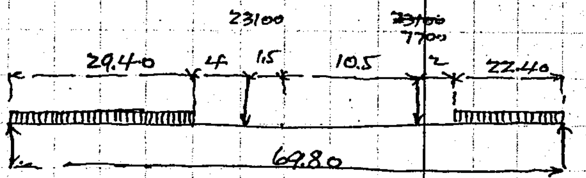
5' - 0" overhang = 2' - 6" at ends

Cross-section of structure same as for Yodo ohashi - see pp 36
span length of main girder 69.80
Details of floor slabs, steel stringer, floor beam & Lower Laterals same as for Yodo ohashi
out to out say 71.35

steel stringer	194 × 71.35	=	13850
Intermediate Floor Beam	3 @ 4170	=	12510
End Floor Beam say	2 @ 4000	=	8000
Lateral bracing	4 @ 1400	=	5600
			39960 #

Dead Load say 3060 # see pp 40 $m = \frac{1}{8} \times 3060 \times 69.80^2 = 1865,000$ 1#

Live Load moment
Live Load same as for 85'0" span see pp 44



Reaction unif load $1080 \times 22.40 = 24200 \times \frac{11.20}{69.80} = 3890$

$1080 \times 29.40 = 31800 \times \frac{55.6}{69.80} = 25100$

28990 #

moment unif. load $28990 \times 33.40 = 970\,000$
less $31800 \times 18.7 = 595\,000$

375000

moment motor truck $30800 \times \frac{33.40^2}{69.80} =$ 492000

moment unif. load $\frac{1}{8} \times 750 \times 69.8^2 =$ 457000

Total LLm 1324000
DLm 1865000
Total moment 3189000 1#

Try web 72" × 1/2" = 36.0" 1/8 web = 4.50" Effective depth say 5.6 S = 570.000
SR = 35.60 − 4.5 = 31.10 □" net

For bottom flange				
2 Ls 6 × 6 × 3/4	16.88	13.88		
1 Pl. 14 × 5/8	8.75	7.50	55'	
1 Pl. 14 × 1/2	7.00	6.00	42'	
1 Pl. 14 × 3/8	5.25	4.50	28'	
	37.88 □"	31.88 □" net		

For top flange 2 Ls 6 × 6 × 3/4 = 16.88
2 Ls 6 × 6 × 3/4 = 16.88
33.76 □"

weight of one girder 69.8'

web	1 Pl. 72" × 1/2	@ 122.4	× 71.3	=	8700
flanges	6 Ls 6 × 6 × 3/4	@ 28.7	× 71.3	=	12300
cov	14 × 5/8	@ 23.8	× 55.0	=	1310
cov	14 × 1/2	@ 23.8	× 42.0	=	1000
cov	14 × 1/2	@ 23.2	× 28.0	=	665
Stiffs	22 Ls 5 × 3 1/2 × 1/2	@ 13.6	× 5.0	=	1495
"	8 Ls 5 × 3 1/2 × 5/8	@ 16.8	× 6.0	=	805
fills	4 Pls 18 × 3/4	@ 45.9	× 5.0	=	920
web splices		2 @ 350			700
flange splices		4 @ 600			2400
Rivet heads & details					2000
					32295

32295 or $\frac{32295}{71.3} = 452$ # per ft

weight of 2 girders 2@ 32300 = 64600
Floor system + Laterals - 40000
Shoes - 4000
108600 # or 48.5 tons

16 spans @ 48.5 tons = 776 tons

Design of Piers -
Assume height of shaft same as for 淀大橋
Depth of well - 25' - 21' - 17'

Superimposed load on pier = say $848000 \times \frac{71.3}{86.5} = 700.000$ #

shaft 10.0 立坪 wt = 324000 #

				area
Try 12.0 dia well	12.0 dia	113.0		37.7
	9.5 dia	70.9		29.9
		42.2		67.6 ÷ 36 = 1.88

25'	volume of shell	2 - 42.2 × 25 =	2110	9.8	forms 94.0 面坪
	top + bott filling	2 - 70.9 × 8 =	1140	5.3	
	middle filling	2 - 70.9 × 17 =	2420	11.2	
			5670	26.3 立坪	
21	volume of shell	2 - 42.2 × 21 =	1770	8.2	forms 79.0 面坪
	top + bott filling	2 - 70.9 × 8 =	1140	5.3	
	middle filling	2 - 70.9 × 13 =	1850	8.6	
			4760	22.1 立坪	
17	vol of shell	2 - 42.2 × 17 =	1430	6.7	forms 64.0 面坪
	top + bott filling	2 - 70.9 × 8 =	1140	5.3	
	middle filling	2 - 70.9 × 9 =	1280	5.9	
			3850	17.9 立坪	

25' well

weight of shaft 324000
" " well 795000
1,119,000
Superimposed load 700.000
1.819,000

Bottom area say 12.5 dia 2 × 122.7 = 245.4

1819.000 ÷ 245.4 = 7420 #/□' or 3.32 tons/□'

Estimate of cost of one pier - 25' well -

A	Concrete in shaft and coping	1:2:4	10.0 立坪 @ 145.00 =	1450.00	
	Forms		41.0 面坪 @ 10.00 =	410.00	
	Curb shoes			600.00	
	Reinf. bars			900.00	
					3360
	Concrete in shell of well	1:2:4	9.8 立坪 @ 145.00 =		1420
	Concrete Forms		94.0 面坪 @ 6.00 =		565
	Concrete filling	1:2:4	5.3 立坪 @ 155.00 =		820
	Concrete filling	1:3:6	11.2 立坪 @ 119.50 =		1340
	Excavation -		26.3 立坪 @ 120.00 =		3160
					10665.00

Estimate of Cost of one pier 21' well

Estimate A same as above		3360.00
Concrete in shell of well	8.2 立坪 @ 145.00 =	1190
Forms	79.0 面坪 @ 6.00 =	475
Concrete filling	5.3 立坪 @ 155.00 =	820
Concrete filling	8.6 立坪 @ 119.50 =	1030
Excavation	22.1 " @ 120.00 =	2650
		9525.00

Estimate of Cost of one pier 17 well -

Estimate A same as preceding -		3360
Concrete in shell	67 立坪 @ 145.00	970
Forms	640 @ 6.00	385
Concrete filling	53 @ 155.00	820
Concrete filling	5.9 @ 119.50	705
Excavation	17.9 @ 100.00	1790
		8030.00

Design of abutment same as for 三条大橋 cost of one abutment 14115.00 see pp 46

Structural steel	776 @ 280.00	=	217500
Cost of Deck :-			
Concrete in flooring	128.0 立坪 @ 145.00	=	18550
Forms	1350.0 面坪 @ 5.00	=	6750
Reinf. bars	92.5 tons @ 230.00	=	21300
Pavement in roadway	675 面坪 @ 25.00	=	16900
Finish of sidewalks + coping	480 " @ 5.00	=	2400
Expansion joints + other misc.		=	1300
curb angles -	2300 lin ft @ .55	=	1270
			68470.00

Handrail say 2400 lin ft @ 6.00 = 14400.00

Electric wiring + lighting &c 6000.00

Summary of Estimate of Cost

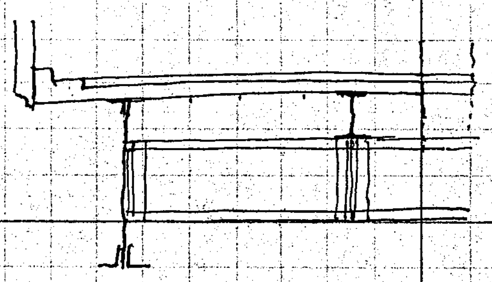
Structural steel		217500.00	
Concrete slabs + pavement		68470.00	
4 Piers @	10665.00	42600.00	
5 Piers @	9525.00	47600.00	
6 Piers @	8030.00	48180.00	367500.00
2 abutments @	14115	28230.00	473000.00
Handrails		14400.00	840500.00
Electric lighting &c		6000.00	30000.00
		472980.00	870500.00
		19020	
		492000	

Estimate of cost of 淀大橋. 18' wide.

span length same as old.
Roadway 18'-0"

$D.5 \times 20 = \frac{10.0}{216} = 0.045 \times 835 = 38$ 立坪 Reinf. bar 27.0 tons.
call this 40 立坪

Stringer -



2@ 45# =	90#	$\frac{1}{8} \times 2000 \times 85^2 = 1820000$ "#	
cross beam =	80	d = 5.5 S = 343.000 SR =	21.50
bracing -	50	$66 \times \frac{7}{16} = 28.8$ $\frac{1}{8}$ web =	36.0
	220#		179.0"
Main girder -	600#	2L 6×6×½ = 11.50	9.5
Shoes + C	50	1Pl. 14×⅜ = 5.25	4.5
	870	14×⅜ = 5.25	4.5
		21.50	18.5

21.50
28.80
71.80
details 16.20
88.00 @ 3.4 = 300#

Structural steel = 870 × 835 = 726000 say 330 tons.

Estimate of cost

Structural Steel	330 tons @ 280	=	92300.00
Floor concrete	40 立坪 @ 200	=	8000
Reinf.	27 tons @ 200	=	5400
Pavement	435 坪 @ 20	=	8700
Handrail -	1700 lin ft @ 4.00	=	6800.00
Remodeling of abutments			1000.00
Remodeling of Piers -	9 @ 200.00	=	1800.00
Remodeling of Piers -	4 @ 4000.00	=	16000.00
			140000.00
misc expense say			10.000.00
			150.000.00

面坪當, 150.000 ÷ 435 = 344.00
140.000 ÷ 435 = 320.00
435
× 300
130500.00