JIUN MASUDA
CONSULTIEG ENGINEER
SEIYU BLDG, TOKIO

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CALCULATIONS FOR

昭和三年六月

岡山縣芦田橋予算設計書

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost : Ashida-Bashi. Okayama-Ken

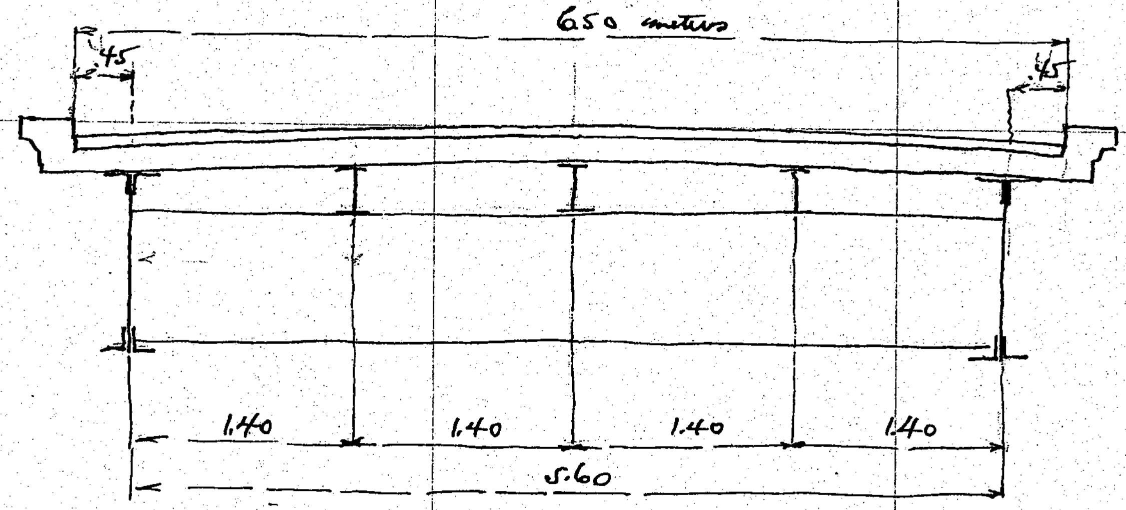
Present bridge structure : wooden Bowstring type width of Roadway 2.0 B.
Center to Center of spans = 15.0 B. 14 spans 90 R × 14 = 1260 R or 382 meters

Panel length 3.3 meters 8 @ 3.3 = 26.4 meters c to c of piers
Suspended span 4 @ 3.3 = 13.20
.75
13.95 meters
Cantilever arm 2 @ 3.3 = 6.60
−.37
6.23 meters

Span length next to abutment 6.23 + 13.95 = 20.18
Total length of bridge = 13 @ 26.4 = 343.2
2 @ 20.18 = 40.36
383.56 meters.

Clear width of Roadway 6.5 meters.

Cross section assumed



Floor slab. span length 1.40 meters
Dead Load. 5c Asphalt block pavement @ 21 kg = 105
1.5c mortar cushion @ 22 = 33
15.0c Concrete slab @ 24 = 360
misc 12
510 kg per sq meters

Dead Load moment = $\frac{1}{10} \times 510 \times 1.40^2 = 100.$ kgm

Live Load motor truck loading
Rear wheel 3000
impact 30% 900
3900

distribution Longl distribution a- Contact betw. wheel + pavement 20
distribution 2@6.5 13
a = 33

Effective width $E = \frac{2}{3} \times 1.40 + .33 = 1.26$

Load per meter strip 3900 ÷ 1.26 = 3100 kg.
moment = $\frac{3100}{2} \times 0.7$ = 1085
To continuity 1085 × 0.8 = 868
Dl. 100
968

Effective depth = $\sqrt{\frac{968 \times 100}{100 \times 7.18}}$ = 11.6 cm use 15 cm slab.

Steel area = $\frac{968 \times 100}{7/8 \times 12.5 \times 1200}$ = .735

try ½" φ 13 mm $\frac{133 \times 100}{.735}$ = 18.1 use 15 cm spacing.

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost: Ashi-da-Bashi Okayama-ken.

Approximate weight of Reinforcing Bars in slab.

Transverse bars 13mm @1.04 kg × 6.67 = 7.0 kg.

Longl bars. " " × 3 = 3.1

~~Misc~~. bars and lap, &c 100% = 10.0

" 20.1 kg.

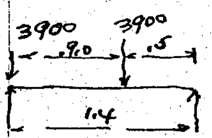
Stringer. span length 3.3 meters.

Dead Load. 510 × 1.4 = 714

~~stringer~~ assumed 56

770    $m = \frac{1}{8} \times 770 \times 3.3^2 = 1050.0$

Live Load



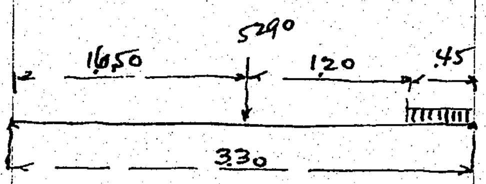
$3900 \times \frac{.5}{1.4} = 1390$

3900

5290 kg.

Uniform load = 500 × 1.4 = 700 kg.

$700 \times \frac{.45 \times .225}{3.30} = 21.5$



moment $\frac{5290}{2} \times 1.650 = 4360$

21.5 × 1.650 = 35

Live load moment 4395

Dead Load 1050

5445 kgm

section modulus required $= \frac{5445 \times 100}{1100} = 495.0$

use 300 × 150 @48.34 kg = Sm = 633.2

with details say 50 kg per lin. meters.

Cross beam Span length 5.6 meters

Dead Load.

510

stringer 35

misc say 5

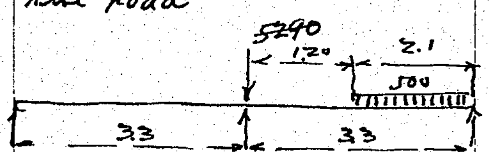
550 × 3.3 = 1820 kg per lin. meters

Floor beam assumed 150

1970 Call this 2000 kg.

Dead load moment $= \frac{1}{8} \times 2000 \times 5.6^2 = 7850$

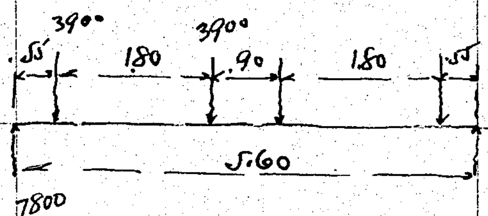
Live Load



Rear wheel ~~5290~~ 3900

Front wheel ~~5290~~ 3900 ÷ 3 = ~~1760~~ 1300

$\frac{500 \times 2.1^2}{2 \times 3.3} = 334$ kg.



moment = 7800 × ~~2.35~~ = 18300 − 3900 × 1.80 = 11280 (7020)

$= \frac{1}{8} \times 334 \times 5.60^2 =$ 1310   1310

Live Load moment ~~19610~~   12590

Dead Load moment 7850   7850

20440

~~web assumed~~ 750 × 8 = 60.0 cm   1/8 web = 7.5 cm   Back to back of ∠s 76.0 cm

assume 2∠s 90 × 75 × 9 = ~~22.04~~ 28.08 □ cm   Effective depth 72.0 cm

flange stress $= 20440 \times \frac{100}{72} = 28400$ kg.   28400 ÷ 1200 = ~~28.08~~ 23.60 net

7.50 ℓ

~~16.10~~ □ net

~~20.58~~

2∠s 90 × 75 × 9 = ~~22.04~~ 28.08

3.96

~~18.08~~ 24.12 □ net.

allowable stress in top flange $= 1200\left(1 - 0.012 \times \frac{1.40}{.188}\right) = 1090$

S Required for comp. = ~~28400 ÷ 1090 = 26.00~~

28.08

~~22.04~~

7.50

~~29.54~~

35.58

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi, Okayama-Ken

Approximate weight of intermediate floor beam span length 5.6 meters

	1 web. 750 × 8	@ 47.0	× 5.6	=	263
	4 Ls 90 × 75 × 9	@ 11.02	× 5.6	=	248
	6 Ls 75 × 75 × 9	@ 9.98	× .75	=	45
	2 Ls 125 × 90 × 10	@ 16.09	× .75	=	24
	2 fills 90 × 9	@ 6.36	× .160	=	8
Stringer Seat	3 pls. 220 × 9	@ 15.50	× 30	=	14
	Rivet heads and variations	5%			30
					632 kg.

632 ÷ 5.6 = 113 kg per lin meters

632 ÷ 3.3 = 191 " " " " of girder.

Lateral Bracing

~~2 Ls 125 × 75 × 10 @ 14.91 kg × 6.5 = 194~~
~~2~~
~~388 per panel~~

~~Try 2 Ls 125 × 75 × 10 @ 14.91 kg × 4.3 = 128~~
~~128~~
~~Center Connection. 10~~
~~266 per panel~~

~~266 ÷ 3.3 = 80 kg per lin meter.~~

Try 2 Ls 125 × 90 × 10 = 41.0 × 2.22² + 276.2 = 478.2 $r = \sqrt{\frac{478.2}{41}} = 3.42$ YY axis.

Unsupported length 430 cm $l/r = \frac{430}{3.42} = 126.$

weight 2 Ls 125 × 90 × 10 @ 16.09 kg × 4.3 = 138
138
Center connection. 10
286 per panel

286 ÷ 3.3 = 87 kg per lin meter.
Call this 90 " " "

Main girder
Dead Load.

Flooring	510 × 6.50	=	3320
Coping.	2 @ 200	=	400
Handrails	2 @ 75	=	150
			3870 kg per meter.

structural Steel.

stringers	3 @ 50 =	150
floor beam		191
lateral Bracings		90
Girders	2 @ 450	900
		1331

1330
5200 ÷ 2 = 2600 kg per lin. meter

Suspended span span length 13.95 meters.

m = 1/8 × 2600 × 13.95² = ~~4530~~ 63200 kgm

Live Load 500 kg/m² × $\frac{6.5}{2}$ = 1625 kg per meter.

m = 1/8 × 1625 × 13.95² = 39600 kgm

Concentration assumed. 3000 kg at center.
m = 1500 × $\frac{13.95}{2}$ = 10460

Summary

DL. m	63200
LL m	39600
"	10460
	113260 kgm

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CHECKED BY ______ DATE ______ **PAGE NO** 4

CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi, Okayama-Ken.

~~web assumed~~ 1300 × 9 = 117.0 □ cm 1/8 web = 14.6 □ cm Back to back of Ls 131.0 cm

Effective depth say 123 cm Flange stress = 113260 ÷ 123 = 92000 kg. Σ = 92000 ÷ 1200 = 76.7

14.6

62.1 □ cm net

2Ls 150 × 150 × 11 = 63.58 − 11.00 = 52.58

1Pl. 350 × ~~10~~ 9 = 31.50 − 4.50 = 26.00

95.08

78.58 □ cm net

2Ls 150 × 150 × 15 = 85.50 − ~~18.00~~ 7.5 = ~~70.50~~ 78.0 □ cm net

Approximate weight of suspended girder.

1 web. 117.0

flanges 2 @ 85.50 = 171.0

288.0 @ .785 = 226.0

Details say. 25% 56.5

282.5 call this 290 kg per lin meter.

Summary for structural steel in suspended span.

stringers	3 @ 50 × 150 × 13.95	=	2100
floor beams	5 @ 632	=	3160
lateral bracings	90 × 13.95	=	1250
main girder	580 × 13.95	=	8100
misc details say			1000
			15,610 kg.

main girder Cantilever arm Length of arm 6.23

Dead Load. 2600 kg per lin meter

m = ½ × 2600 × 6.23² = 50500

Concentration 2600 × 13.95/2 = 18100

m = 18100 × 6.23 = 113 000

163,500 kg meter.

Dead Load m = 163.500

Live Load m = 120900

284,400 kgm

Live Load 1625 kg per lin. meter

m = ½ × 1625 × 6.23² = 31600

Concentration 1625 × 13.95/2 = 11320

assumed 3000

14320

m = 14320 × 6.23 = 89300

120900 kg meter

~~web assumed~~ 1800 × 13 = ~~117.0~~ 234.0 1/8 web = ~~14.6~~ 29.3 □ cm Back to back of Ls 181. cm

Effective depth say ~~183~~ 173 cm Flange stress = ~~2844~~00 ÷ 173 = 165.000 ÷ 1200 = 137.50

29.3 / 29.3

~~135.0~~ / □ cm net 108.2 net

2Ls 150 × 150 × 18 = 85.50 − 15.00 = 70.50

2Pls. 350 × 10 = 75.00 − 10.00 = 65.00

160.50 135.50

Approximate weight of cantilever arm

1 web. 234.0

flanges 2 @ 160.5 = 321.0

555. @ .785 = 435

Details say 25% 109

544 kg per lin. meters.

Main girder between Piers. span length 26.4 meters

Dead Load. 2600 kg per lin. meters

m = ⅛ × 2600 × 26.4² = 226.000

Less cantilever moment = 163.500

6

82,500 kgm

Preliminary Design and Estimate of Cost, Ashida-Bashi Okayama-Ken

Live Load 1625 kg per lin. meter.

m = 1/8 · 1625 · 26.4² = 141600

concentration say 3000 kg at center of span

m = 1500 · 13.2 = 19800

161400 -

Dead Load moment 82500

243900 kg m

Live Load negative moment - 120900

D.L. m + 82500

- 38400

243900

282300 for design moment

~~web assumed~~ 130 × 13 = 169.0 1/8 web. = 21.1 □ cm Back to back of L's 131.0 cm.

Effective depth 125 cm about flange stress = 282300 ÷ 125 = 226000

Section req'd = 226000 ÷ 1200 = 188.0

21.1

166.9 □ cm net.

Try 2L's 150 × 150 × 15 = 85.50 - 15.00 = 70.5

2Pl's 350 × 15 = 105.00 - 15.00 = 90.

1Pl. 350 × 10 = 35.00 - 5.00 = 30.0

190.5 □ cm net

Try web. 140 × 13 = 182.0 1/8 web = 22.7 cm² Back to back of L's 141.0 cm

Effective depth say 135 flange stress = 282300 ÷ 135 = 210000

SR = 210000 ÷ 1200 = 175.0

22.70

152.30 cm² net.

Try 2L's 150 × 150 × 15 = 85.50 - 15.00 = 70.50

2Pl's 350 × 15 = 105.00 - 15.00 = 90.00

190.50 160.50

Approximate weight of main girder

1 web. 182

flange 2 @ 190.5 = 381

563 @ .785 = 442.

25%. details 111

553 kg per lin. meter.

Approximate weight of Structural Steel in Center span

stringer 150 × 38.86 = 5830

floor beam 13 × 632 = 8200

Lateral Bracing 90 × 38.86 = 3500

main girder Cant. 1088 × 12.46 = 13500

" " Center 1106 × 26.4 = 29200

misc. 2000

62,230 kg.

Summary for Structural Steel in bridge.

Suspended span 8 @ 15.61 = 124.88

Anchor span 7 @ 62.23 = 435.61

560.49 tons

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CHECKED BY ______ DATE ______ **PAGE NO** 6

CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi Okayama-Ken

Load on pier.

Dead Load	2・2600 ・ 26.4	=	137,500
Live Load	2・1625 ・ 〃	=	85,800
Extra load say			10.000
			233,300 kg.

For one shoe 233300 ÷ 2 = 116650 kg

Bearing area = 116650 ÷ 45 = 2590 51^2

Try 60・60 = 3600 116650 ÷ 3600 = 32.4 kg/cm²

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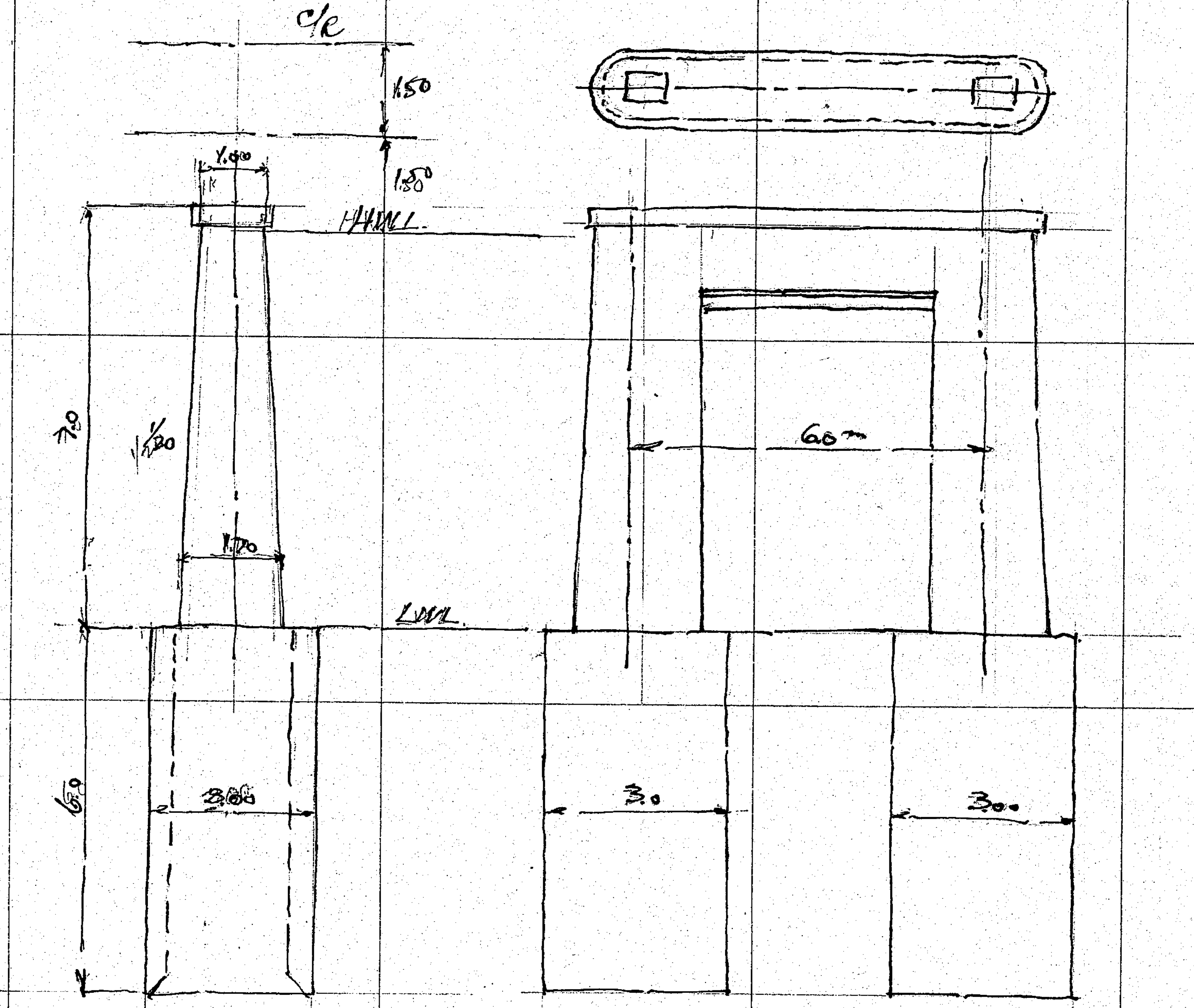
CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi, Okayama-ken.



Approximate volume of Concrete

Top area 1^{ϕ} .78
2.00
2.78

Bottom area 1.7^{ϕ} 2.26
2.0 × 1.7 = 3.40
5.66

2.78
5.66
8.44 ÷ 2 = 4.22

vol = 4.22 × 7.0 = 29.50
1. × 1.5 × 4.0 = 6.00
web. .75 × 5.5 × 4.0 16.50
52.00 cubic meters

Concrete in well.
3 × 3 = 9.00
2.2 × 2.2 = 4.84
4.16 sq m × 6.00 = 25.00
Top and bottom filling 1 m each 2.0 × 4.84 = 9.70 } 34.70 2 @ 34.70 = 69.40 cm

Intermediate filling 4.0 × 4.84 = 19.40 2 @ 19.40 = 38.80 cm

Forms. (2 × 8 + 2 × 2) × 7 = 140.
24 × 6 × 2 = 290
430 sq meters

Reinforcing Bars 4.0 tons.
Curb shoe. 1.20 tons.

Excavation for well. 9 × 6 = 54 2 @ 54 = 108 cubic meters

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi Okayama-ken.

Total Concrete shaft 52.00
well. 69.40
38.00
159.40 @ 2200 kg = 351 000 kg.
Superimposed load. 233000
584 000 kg.
Base area = 9×2 = 18.0 584000 ÷ 18 = 325 tons / per sq meter.
302 tons/□'

Approximate Cost of one pier.

Concrete	1:2:4	121.40	@ 19.00	=	2346
"	1:4:8	38.80	@ 13.00	=	504
Reinforcing steel		4.0 tons	@ 130	=	520
Cut shoes		1.2 "	@ 250	=	300
Form		430 sq m	@ 2.00	=	860
Excavation.		108 c m	@ 5.00	=	540
					5034.00

Cost of one abutment 4500.00

Approximate Estimate of Cost of Deck Construction 382 meters Roadway 6.5 meters

Concrete in slab.	~~382~~ 460 cubic meters	@ 19.00	=	~~7450~~ 8750
Reinforcing Bars in slab.	50 tons	@ 160.00	=	8000
Forms	2900 sq meters	@ 2.20	=	6380
pavement	2480 " "	@ 3.00	=	7450
Handrails	58.0 tons	@ 300.00	=	17400
				46680

46680 ÷ 2480 = 18.80 2500
wiring &c
49180
1300
50480

Summary for Estimate of Cost
Roadway 6.5 meters span 13 @ 26.4 m = 343.2
2 @ 20.18 = 40.36
383.56 meters.

structural steel	560.49	@ 230.00	=	129 000
Deck complete	HR + Pavement			~~49180~~ 50480
Piers	14 piers	@ 5000	=	70000
abutment	2	@ 4500	=	9000
				~~257180~~ 258480

Preliminary Design and Estimate of Cost Ashida-Bashi, Okayama-Ken

The present bridge Consists of 14- 90ft spans. Roadway 2. ken

Let us estimate the cost of Construction for the following span arrangement

10 spans 90ft on present piers and abutment

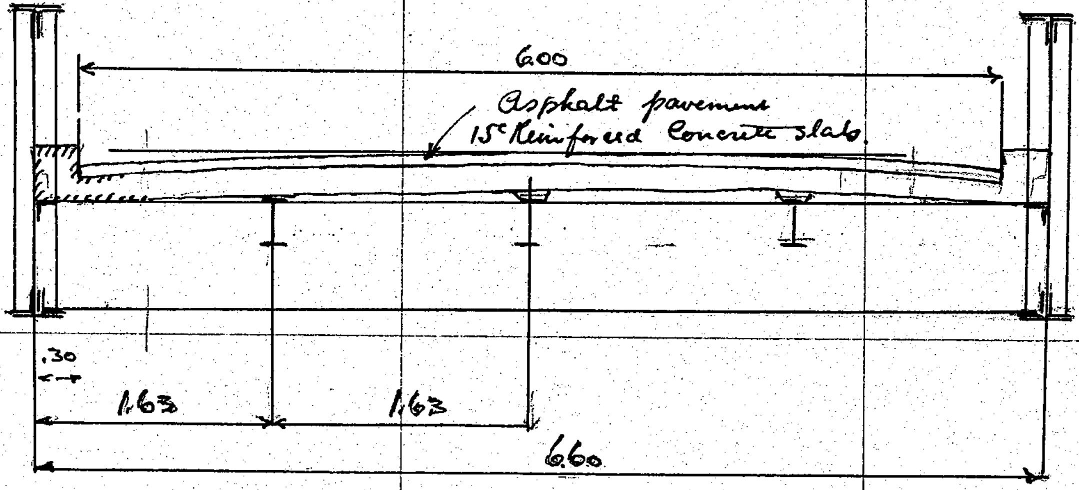
Piers and abutment shall be remodeled for wider roadway and scouring of river bed.

2 spans @ 135 ft truss span

2 Piers to be abandoned + One new pier shall be built.

1 span 90 ft Between pier and abutment both shall be remodeled same way as for first 10 spans

Roadway 6.0 meters.



Span length of slab 1.63 m = 5.35' Slab. 15 cm.

Span length of girder. 89'6" out to out center to center of bearings 88'0"
27.3 m 26.8 meters

7 Panels @ 3.83 meters = 26.8 m

Stringer span length 3.83

Dead Load 510 × 1.63 = 830
stringer 50
880

$m = \frac{1}{8} \times 880 \times 383^2 = 1610$

Live Load motor truck rear wheel with impact - 3900 kg.

1.02 .61 .29 1.34
1.63 1.63

Reaction on stringers $3900 \times \frac{2.36}{1.63} = 5650$

$3900 \times \frac{2.36}{1.63} = 5650.$

5650
1.91 1.20 .71
3.83

Uniform live load 500 × 1.63 = 815 kg per lin. meter

$815 \times .71 \times \frac{.35}{383} = 53.0$

moment $\frac{5650}{2} \times 1.91 = 5400$
$53. \times 1.91 = 103$

5503

Dead Load moment 1610

7113 kgm

Section modulus req'd $= \frac{711300}{1100} = 645.0$

Try I 300 × 150 @ 48.34 kg Sm = 633.2

Cross Beam span length 6.6 meters

Dead Load 510
stringer say 35
misc. say 5
550 × 3.83 = 2110 kg per meter

Floor Beam assumed 150
2260

DL. moment $= \frac{1}{8} \times 2260 \times 6.6^2 = 12300$ kgm

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CHECKED BY ____ DATE ____ **PAGE NO** *10*

CALCULATIONS FOR

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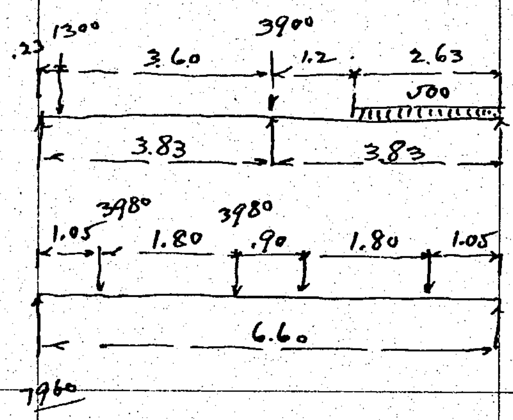
Live Load.



$1300 \times \frac{.23}{3.83} = 78$

3900

3978 say 3980 kg.

Unif. load $\frac{500 \times 2.63^2}{2 \times 3.83} = 452$ kg per meter

moment due to motor truck

$7960 \times 2.85 = 22700$

$3980 \times 1.80 = 7160$

15540

Unif. $\frac{1}{8} \cdot 452 \cdot 6.6^2 = 2460$

18000

Dead Load moment 12300

30300 kgm

web assumed $750 \times 8 = 60.0$ cm 1/8 web = 7.50 cm Back to back of Ls say 76 cm

Effective depth say 72.0 flange stress $= \frac{3030000}{72} = 42100$ $SR = \frac{42100}{1200} = 35.0$

7.5

27.5 net

try 2Ls $100 \times 75 \times 10 = 33.00 -$

4.4

28.60 net.

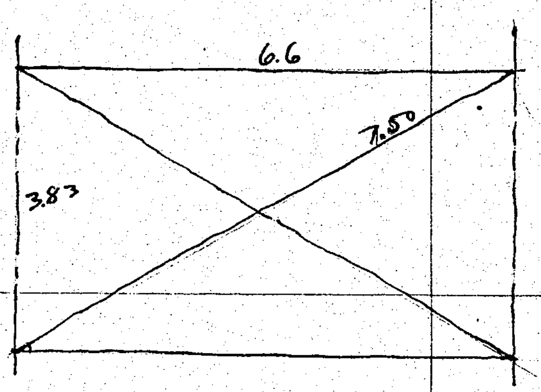
Approximate weight of intermediate floor beam span length 6.6.

	1 web.	750×8	@ 47.0	× 6.6	= 310
	4Ls	$100 \times 75 \times 10$	@ 12.95	× 6.6	= 342
	6Ls	$75 \times 75 \times 9$	@ 9.98	× .75	= 45
	2Ls	$125 \times 90 \times 10$	@ 16.09	× .75	= 24
	2 Pls.	90×9	@ 6.36	× .60	= 8
Stringer seat	3 Pls.	220×9	@ 15.50	× .30	= 14
	Rivet heads + variations		5%		37
					780 kg.

$780 \div 6.6 = 118$ kg.

$780 \div 3.83 = 204$ " of girder

Lateral Bracings.



2Ls $125 \times 90 \times 10$ @ 16.09 kg × 7.5 = 242

Center connection 10

252 per panel.

$252 \div 3.83 = 66.$ kg.

call this 70 kg per lin. meter of span

Main Girder

Dead Load

Flooring	510×6.0	=	3060
Coping	2 @ 200	=	400
H.R.	2 @ 75	=	150
			3610 kg per meter.

Structural Steel

Stringers	3 @ 50	=	150
Floor beam			204
Lateral Bracings			70
Girder say	2 @ 500	=	1000
			1424

3610

1424

$5034 \div 2 = 2517$ kg per lin. meter

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CHECKED BY ______ DATE ______ PAGE NO 11

CALCULATIONS FOR

Preliminary Design and Estimate of Cost. Ashida Bashi Okayama-Ken.

Dead Load moment = 1/8 · 2517 × 26.8² = 226,000

Live Load Uniform load 500 × 3.0 = 1500 kg per meter.

m = 1/8 · 1500 · 26.8² = 134,000

Concentration assumed 3000 kg. at center of span

m = 1/2 · 3000 · 13.4 = 20100

154.100

Dead Load moment = 226.000

Live Load Moment = 154.100

380.100 kgm

web assumed 1900 × 13 = 247.0 1/8 web = 30.9 cm Effective depth say 190 cm

flange stress = 380.100 00 ÷ 190 = 20.000

section required = 200.000 ÷ 1200 = 167.0

30.9

136.1 cm

Try 2L^S 150 · 150 × 15 = 85.50 - 15.00 = 70.50

2Pl. 350 · 15 = 105.00 - 15.00 = 90.00

190.50 160.50

Approximate weight of main girder.

1 web 247.0

flanges 2@ 190.50 = 381.0

628.0 @ .785 = 493.

Details say 25% - 123

616 kg per lin ~~ft~~ meter.

Approximate weight of structural steel in one span.

stringer 150 · 27.3 = 4100

floor beams 8 @ 780 = 6240

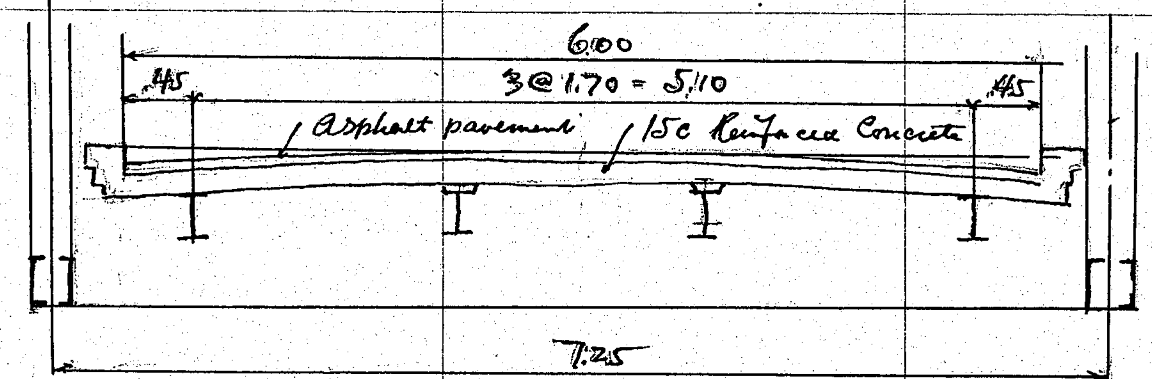
lateral 70 × 26.8 = 1880

main girder 1232 · 27.3 = 33600

45820 kg.

11 spans @ 45.8 = 503.8 tons.

Truss Span 135 ft = 134' 132' c to c of bearings say 40. meter



Floor slabs. 15 cm reinforced Concrete.

stringer span length 4.0 meters

Dead Load 510 × 1.70 = 867

stringer assumed 70

937 kg m = 1/8 · 937 · 4.0² = 1870

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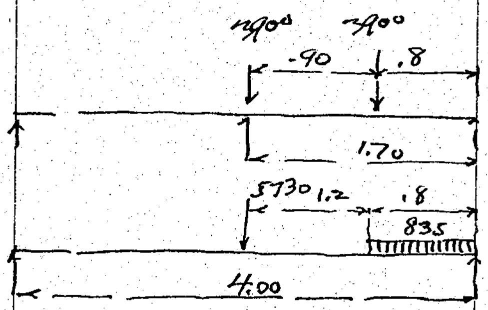
MADE BY J. Masuda DATE 3-6-30 **FILE NO** ______

CHECKED BY ______ DATE ______ **PAGE NO** 12

CALCULATIONS FOR

Preliminary Design and Estimate of Cost, Ashida - Bashi, Okayama - Ken.

Live Load motor truck loading rear wheel with impact = 3900 kg.



$3900 \times \frac{.8}{1.7} = 1830$

3900

Reaction on ~~stringer~~ = 5730 kg.

Uniform load $500 \times 1.70 = 835$ kg per lin. meter

Reaction $= \frac{835 \times .8^2}{2 \times 4.00} = 66.7$

moment $\frac{5730}{2} \times 2 = 5730$

$66.7 \times 2 = 133$

L.L. moment 5863

DL. moment 1870

7733

Zm. req'd $= \frac{773300}{1100} = 703.0$

try $350 \times 150 \times 9$ @ 58.54 kg.

~~2~~ @ 60 = ~~240~~ 120 kg per lin. meter

2 @ 50 = 100

220 " " " "

Cross beam span length = 7.25 meters.

Dead Load 510

stringer 35

misc. 5

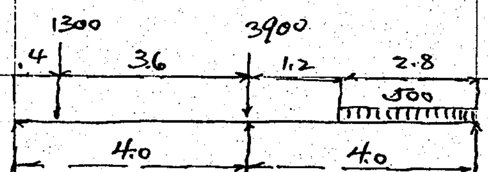
$550 \times 4.00 = 2200$ kg.

Floor Beam say 150

2350 kg per lin. meter.

DL. $m = \frac{1}{8} \times 2350 \times 7.25^2 = 15450$ kgm

Live Load

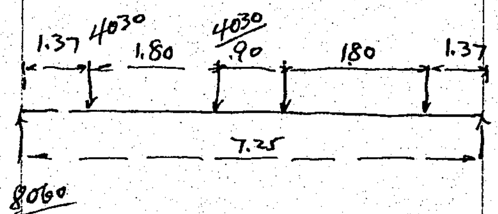


motor truck loading $1300 \times \frac{4}{4} = 130$

3900

4030 kg.

$500 \times \frac{2.8^2}{2 \times 4.0} = 490$



moment $8060 \times 3.17 = 25600$

$4030 \times 1.80 = 7250$

18350

Unif. $\frac{1}{8} \times 490 \times 7.25^2 = 3220$

21570

Dead Load moment 15450

37020 kgm

web assumed $750 \times 8 = 60.0$ cm² $\frac{1}{8}$ web $= 7.5$ □cm Back to back of L^s 76.0

Effective depth 72. cm flange stress $= \frac{3702000}{72} = 51500$ $3R = 51500 \div 1200 = 4300$

7.50

35.50 □cm net

$41.0 - 4.4 = 36.60$

try 2L^s ~~100~~ 125 × 90 × 10 = ~~36.0~~ − ~~4.4~~ = ~~31.6~~ □cm net

~~clear deep web~~ ~~770 × 8~~ about to use above flange L^s

approximate weight of floor beam 6.90 m out to out

1 web	770 × 8	@	~~48.4~~ 47.0	× 6.90	=	324
4 L^s	125 × 90 × 10	@	16.09	× 6.90	=	444
6 L^s	75 × 75 × 9	@	9.98	× .75	=	45
8 L^s	125 × 75 × 10	@	14.91	× .75	=	89
8 pls.	75 × 10	@	5.90	× .60	=	28
4 L^s	125 × 75 × 10	@	14.91	× .75	=	45
4 pls.	75 × 10	@	5.90	× .60	=	14
	variation 5%					49
						1038

say 1040.

$1040 \div 6.90 = 151.$ kg per meter

$1040 \div 4.0$ 260 kg per meter of span

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Bottom Lateral Bracing 90 kg per lin meter.

main truss

Dead Load Flooring 510 × 6.0 = 3060
Coping 2 @ 200 = 400
HR say 2 @ 75 150
3610 kg per meter
structural Steel 1950
stringer 200 5560 ÷ 2 = 2780 kg per meter.
floor beam 260
lateral 90
trusses. 2 @ 700 = 1400
1950

moment = 1/8 × 2780 × 40² = 556000 kgm

Live Load moment = 1/8 × 1500 × 40² = 300.000
Conc. say 5000 2500 × 20 = 50.000
906000 kgm

Depth of truss = 4.0 Chord stress = 906000/4.0 = 227000

A Required = 227 000 ÷ 1000 = 227.0 □cm

Approximate weight of truss 227 × 2.7 × .785 = 480
38% – 182
662 say 660 kg per meter

Summary for structural Steel in One span Out to out 40.6
stringers 220 × 40.6 = 8930
floor beam 11 @ 1040 = 11440
lateral Bracing 90 × 40 = 3600
trusses 1320 × 40 = 52800
shoes say 3000
79.740 tons call this 80 tons.

2 spans @ 80. = 160 tons.

Total steel in bridge
11 girder spans @ 45.8 = 503.8 @ 230 = 116000
2 truss spans @ 80 = 160.0 @ 260 = 41600
663.8 tons. 157600.00
34820
192420

Approximate Cost of Deck Construction.
Total length = 382 meters width of Roadway 6.0 meters.
Concrete in slab. 430 cubic meters @ 19.00 = 8170
Reinforcing bars. 46 tons @ 160.00 = 7350
Forms. 2860 sq meters @ 2.20 = 6300
Pavement 2300 sq meters @ 3.00 = 6900
Handrails. 160 meters 12 tons @ 300.00 = 3600
32320
wiring &c 2500.
34820

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CALCULATIONS FOR

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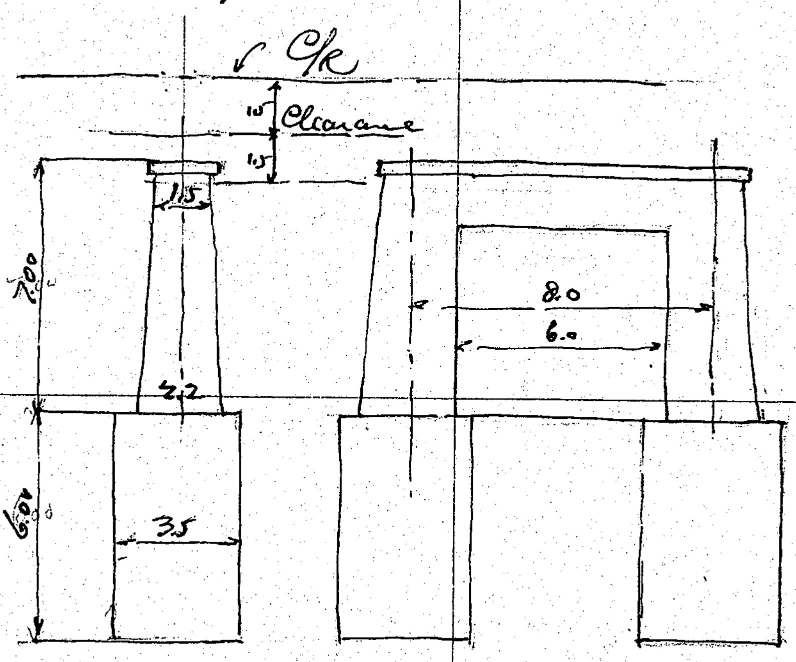
New Pier for truss span



Approximate volume of Concrete

Shaft

Top	1.77	Bottom	3.8
	3.00		4.4
	4.77		8.2
			4.77
			12.97 ÷ 2 = 6.5

volume = 6.5 × 7.0 = 45.5 cubic meters
2.25 × 6 = 13.5
.75 × 5.5 × 6.0 = 24.8
83.8

Concrete in well.
3.5 × 3.5 = 12.30
2.7 × 2.7 = 7.30
5.00

Shell. 5.00 × 6 = 30.0 2 @ 30 = 60.0 cubic meters
filling 7.30 × 2 = 14.6 2 @ 14.6 = 29.2
89.2

Intermediate filling 7.30 × 4 = 29.2 2 @ 29.2 = 58.4

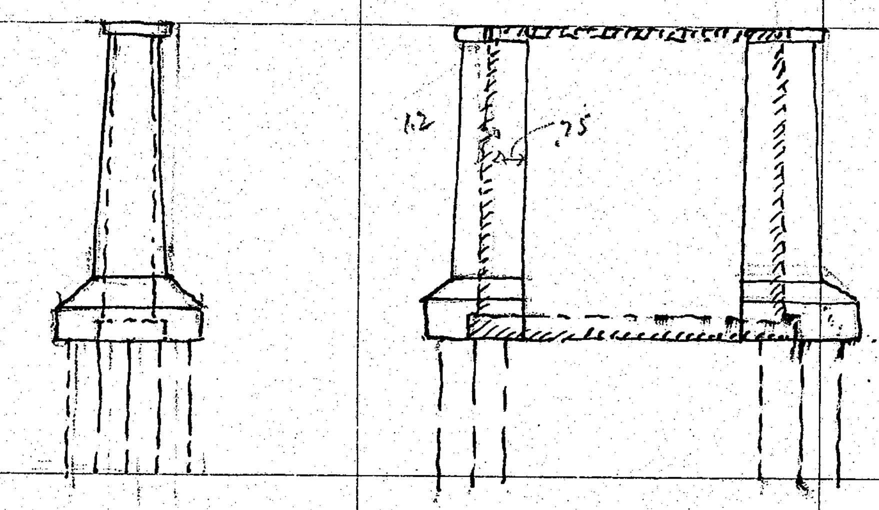
Forms say 500
Reinforcing bars say 6.0 tons curb shoe say 1.5 tons

Excavation 12.30 × 6 × 2 = 148.0 cubic meters.

Approximate Estimate

Concrete	1:2:4	173.0 ~~89.2~~	@ 19.00	= 3280 ~~1700~~
	1:4:8	58.4	@ 13.00	= 760
Steel		6.0 tons	@ ~~130~~.00	= 780
curb shoes		1.5 tons	@ 250	= 375
Form		500 sq meters	@ 3.00	= 1500
Excavation		148.	@ 5.00	= 740
				7435 say 7500.00

Remodeling of Present Piers for new truss spans 2 Piers Pier nos 10 and 13



Volume of new Concrete
approximate only

2 - 2.5 × 2. × 6.5 = 65.0
2 - 4.0 × 2.75 × 1.5 = 33.0
98.0 call this 100.0

no of piling 30 - 15' 40

Reinforcing bars. 2.5 tons

Forms. 140 sq meters

Excavation - —

Removing old pier. 2 @ 10 = 20.0 ~~20.0~~ cubic meters

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CALCULATIONS FOR

Preliminary Design and Estimate of cost Ashida-Bashi, Okayama-Ken

Estimate of Cost. Piers 10 + 13.
For one pier.

Concrete	100	@	19.00	=	1900
Reinf.	2.5 tons	@	130	=	325
Forms.	140	@	5.00	=	700
Excavation.					100
piling	30 -	@	15.00	=	450
misc say					200
					3725.00
Sheet piling.	30 meter. ~~4~~		120 sq m @10.00		1200
	~~2@ 3725~~		~~= 7450.00~~		4925 say 5000

2@5000 = 10.000.00

Remodeling of girder piers.

Concrete.	16	@	19.00	=	300
Reinforcing bars					100
forms					200
Excavation.					100
Sheet piling.	22 × 4 = 88	@	10	=	880
					1580.00
Removing.	16	@	—	—	100
					Call 1700.00

9@ 1700 = 15300.00

2@ 1000 = ~~3400~~.00 2000

Summary.

Structural steel +		~~192420~~	157600
Deck.			34820
Pier.			7500
2 piers	@ 5000	=	10000
9 piers.	@ 1700	=	15300
2 abutments			2000
			227220.00

All girder spans.

Structural Steel - 14 @ 45.8 = 640 tons @ 230 = 148000.00

Deck Construction - no HR.			31220.00
2 new piers.	@	6000	12000
11 old piers	@	1700	18700
2 abutments	@	1000	2000
			211920.00

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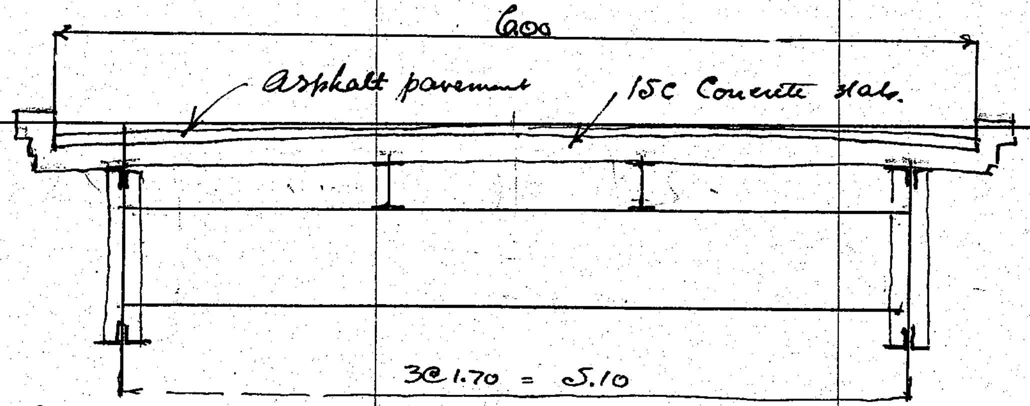
15 Spans gerber girder. Layout same as for p1 to p8

span length 13 @ 26.4 = 343.2
2 @ 20.18 = 40.36
383.56 meters

Clear Roadway 6.0 meters

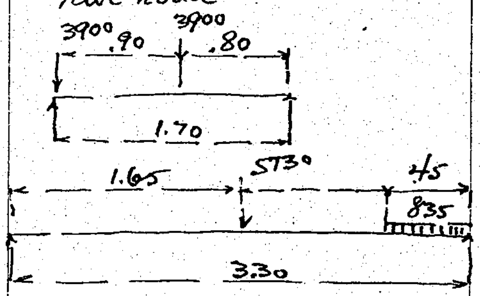
Cross section assumed



Reinforced Concrete Slab. 15.0 cm
stringer span length 3.30 meters

Dead Load 510 · 1.70 = 867
stringer 50
917 kg $m = \frac{1}{8} \cdot 917 \cdot 3.3^2 = 1250$

Live Load $3900 \cdot \frac{.8}{1.7} =$ 1830
3900
5730 kg



Uniform load 500 · 1.70 = 835 kg per lin. meter.

$835 \cdot \frac{.45^2}{2 \cdot 3.3} =$ 25.6

Moment $\frac{5730}{2} \cdot 1.65 =$ 4730
$25.6 \cdot 1.65 =$ ~~420~~ 42
5150

4772
~~5150~~
1250
~~6400~~
6022

Section modulus req'd $= \frac{\cancel{515000}\ \cancel{640000}\ 60220}{1100} = \cancel{582.0}\ 548.0$ 4772

Try I 300 · 150 @ 48.34 kg Zm = 633.2

Cross beam referred to p3.
weight say 5.1 × 120 kg = say 610 kg.
610 ÷ 3.3 =

Lateral Bracing say 80 kg per lin. meter.

Main girder

Dead Load

Flooring	510 · 6.0	=	3060
Coping	2 @ 200	=	400
Handrails	2 @ 75	=	150
			3610

structural steel

stringers	2 @ 50	=	100
Floor beam			185
Lateral Bracings			80
Girders	2 @ 500		1000
			1365

4975 ÷ 2 = say 2500 kg

Suspended span span length 13.95 meters
$m = \frac{1}{8} \cdot 2500 \cdot 13.95^2 =$ 60700 kgm

Live Load 500 · 3.00 = 1500 kg per meter
$m = \frac{1}{8} \cdot 1500 \cdot 13.95^2 =$ 36400 kgm

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi Okayama-Ken

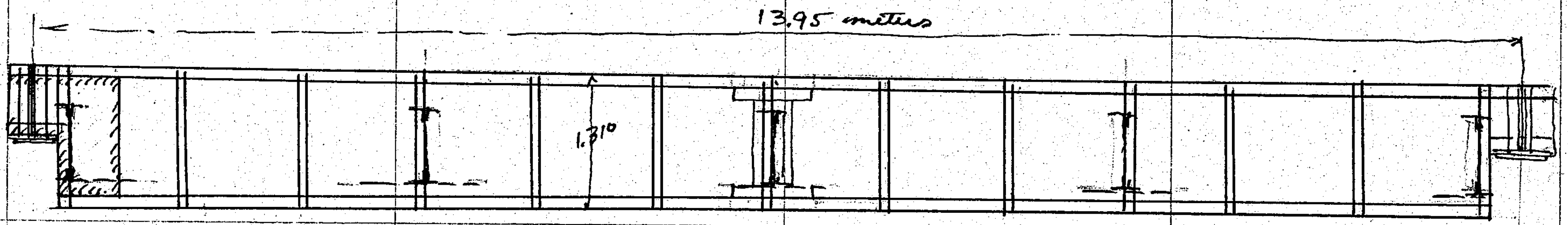
Concentration assumed 4250 kg. at ends

$m = 2125 \cdot \frac{13.95}{2} = 14800$
$\underline{36400}$

LL m 51200
DL m $\underline{60700}$
111900

~~web assumed~~ 1300 · 9 = 117.0 □cm 1/8 web = 14.6 □cm Back to back of Ls = 131.0 cm
Effective depth say 123 cm flange stress = 111900 ÷ 1.23 = 90300
section required = 90300 ÷ 1200 = 75.2
$\underline{14.6}$
60.6 □cm net

2Ls 150 · 100 · 15 = 70.50 − 7.50 = 63.0 cm net



Approximate weight of main girder. (Suspended span).

web.	130 · 9	@	91.7	×	14.70	= 1350
	4Ls 150×100×15	@	27.67	·	13.95	= 1550
	26Ls 125·90·10	@	16.09	·	1.30	= 543
	4Pls 90·15	@	10.60	·	1.10	= 47
	8Ls 150·150·15	@	33.55	·	0.60	= 161
	4Pls 1,000·15	@	116.0	·	1.30	= 603
	4Ls 150·100·15	@	27.67	·	.75	= 83
web splice	2Pls 350·9	@	24.70	·	.80	= 40
" "	4Pls 150·15	@	17.65	·	1.20	= 85
flange splice	2Pls 310·9	@	21.80	·	1.20	= 52
	4Ls 150·100·9	@	17.02	·	1.20	= 82
Lateral Conn.	8Ls 90·90·10	@	13.34	·	.40	= 43
	5Pls 350·9	@	24.70	·	.60	= 74
	Rivet heads say 3.5%.					$\underline{165}$
	misc					4878 kg.
						$\underline{100}$
						4978

4978 ÷ 13.95 = 366 kg per meter.

Summary for structural steel in suspended span

stringers	2@50 = 100 · 13.95	=	1395
floor beams	5@610 =		3050
Lateral bracings	80 · 13.95 =	=	1117
main Girders	2@4978 =		$\underline{9956}$
			15518 kg ÷ 13.95 = 1110

Main girder Cantilever arm Length of arm 6.23
Dead Load 2500 kg per lin. meter.
Concentration at hinge ~~2500~~
3610
$\underline{1110}$
4720 ÷ 2 = 2360 kg $\times \frac{13.95}{2} = 16450$

Dead Load moment $m = \frac{1}{2} \cdot 2500 \cdot 6.23^2 = 48500$
$m = 16450 \cdot 6.23 = \underline{102500}$
151,000 kgm

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CALCULATIONS FOR

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Live Load Concentration at hinge uniform load 1500 × 13.95/2 = 10500
motor truck extra. say 4250
14750

moment 14750 × 6.23 = 92000
½ × 1500 × 6.23² = 29200
121200. ~~1212~~

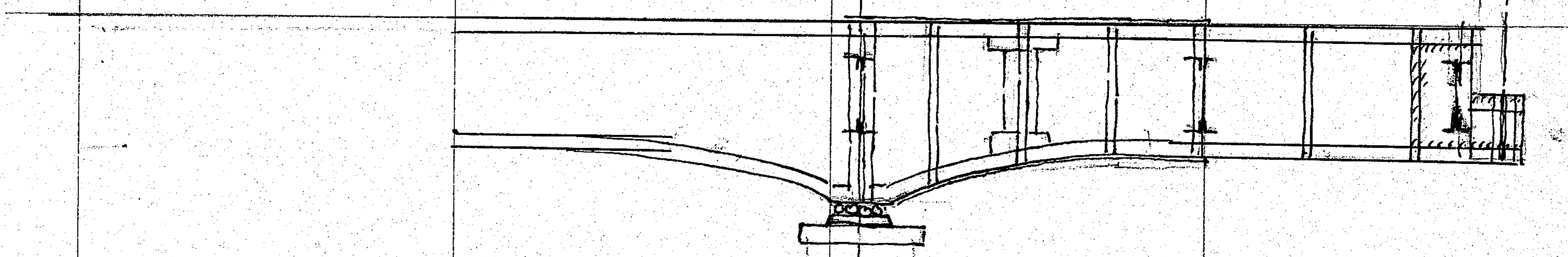
Dead Load moment 151000
272200

~~web assumed~~ 1800 × 9 = 162.0 ⅛ web = 20.3 □cm Back to back of Ls 181. cm
Effective depth 176.cm flange stress = 272200 ÷ 1.76. = 155000
section required = 155000 ÷ 1200 = 129.00
20.30
108.70 □cm net

2Ls 150 × 150 × 15 = 85.50 - 15.00 = 70.5
1Pl. 350 × 15 = 52.50 - 7.50 = 45.0
138.00 115.50 □cm net.

Approximate weight of Cantilever

6.23 meters.



web.	1Pl.	180 × 9	@ 127.0	× 6.60	=	840
flange	4Ls	150 × 150 × 15	@ 33.55	× 6.23	=	835
	2Pls.	350 × 15	@ 41.2	× 3.3	=	272
Stiffs	16Ls	125 × 90 × 10	@ 16.09	× 1.50	=	338
fillers	2Pls.	90 × 15	@ 10.60	× 1.10	=	23
End stiffs	2Ls	~~125 × 125 × 15~~ 130 × 130 × 12	@ 23.36	× 1.80	=	84
½	1 fill.	260 × 15	@ 30.60	× 1.50	=	46
End details	4Ls	130 × 130 × 12	@ 23.36	× .60	=	56
	2Ls	150 × 150 × 15	@ 33.55	× .60	=	40
fills	2Pls.	1,000 × 15	@ 116.0	× 1.20	=	278
flange splice	2Ls	150 × 150 × 15	@ 33.55	× 1.50	=	100
web splice	4Pls.	150 × 15	@ 17.65	× 1.20	=	85
	2Pls.	350 × 9	@ 24.70	× .70	=	35
Lateral Conn	5½Ls	90 × 90 × 10	@ 13.34	× .40	=	27
	2.5 Pls.	350 × 9	@ 24.70	× .60	=	37
Sole plate	1Pl.	350 × 19	@ 52.00	× .40	=	20
Base Pl.	½ Pl.	450 × 19	@ 67.10	× .45	=	15
Rivet heads &c						111
						3241 kg. say 3300

589

3241 ÷ 6.23 = 520 kg per lin meter.

Summary for structural steel in Cantilever arm

stringers	100 × 623	=	623
Floor beam	2 @ 610	=	1220
laterals	80 × 623	=	500
main girder.	say 2 @ 3300	=	6600
			8943 kg.

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi Okayama-Ken

main girder Anchor span between piers = 26.4 m

Dead Load say 2600 kg per lin meters

$m = \frac{1}{8} \cdot 2600 \cdot 26.4^2 =$ 226.000

Less Cantilever moment 151.000

75.000

Live Load 1500 kg per lin. meter

$m = \frac{1}{8} \cdot 1500 \cdot 26.4^2 =$ 131000

Conc $\frac{4250}{2} \cdot 13.2 =$ 28000

159000

DL m 75000

234.000

Live Load negative moment − 121200

\+ 75000

− 46200

Design moment − 46200 ÷ 2 = 23100

234000

257100 kgm

web assumed 1300 · 9 = 117.0 □cm 1/8 web = 14.6 □cm

Effective depth say 127. cm Flange Stress = 257100 ÷ 127 = 202000

Section reqd = 202000 ÷ 1200 = 168.0

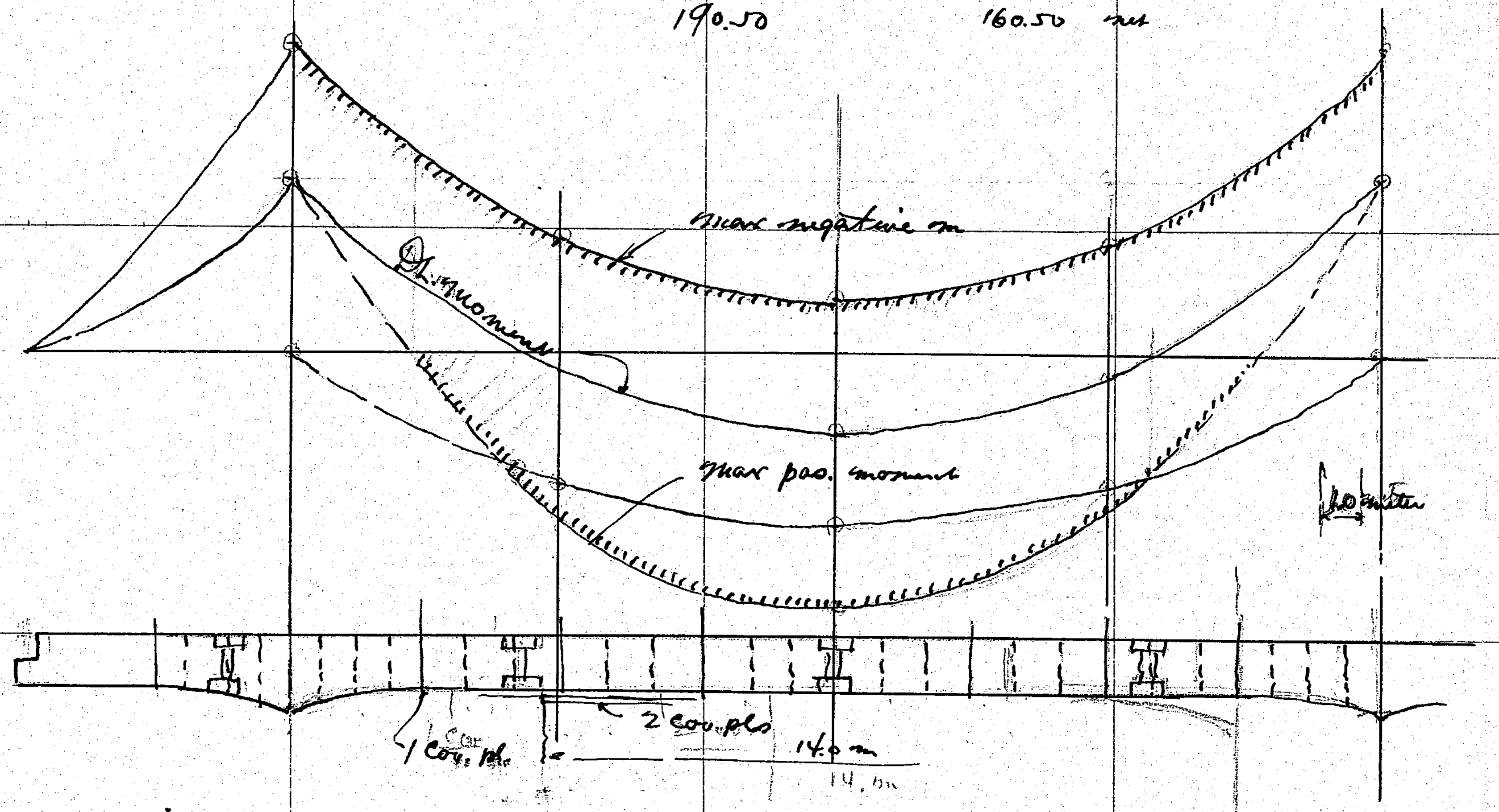
14.6

153.4 □cm net

Try 2 Ls 150 · 150 · 15 = 85.50 − 15.00 = 70.50

2 Pls. 350 · 15 = 105.00 − 15.00 = 90.00

190.50 160.50 net



Approximate weight of anchor span.

web. say	1 Pl. 130 · 9	@ 91.7	· 15.0	=	1380
	2 Pls. 180 · 9	@ 127.0	· 5.7	=	1450
flanges.	4 Ls 150 · 150 · 15	@ 33.55	· 26.4	=	3540
	2 Pls 350 · 15	@ 41.2	· 26.4	=	2180
	2 Pls. 350 · 15	@ 41.2	· 14.0	=	1150
	50 Ls 125 × 90 × 10	@ 16.09	· 1.4	=	1130
fills.	14 Pls. 90 · 15	@ 10.60	· 1.20	=	178
	2 Ls 130 · 130 · 12	@ 23.36	· 1.80	=	84
	1 Pl. 260 · 15	@ 30.60	· 1.50	=	46
					11138

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi, Okayama-Ken

webs flange splices 3 @ 300 = 1000
Lateral connection 16 Ls 90×90×10 @ 13.34 × .40 = 850
8 Pls. 350×9 @ 24.70 × .60 = 119.0
Sole plate 15
Shoes say 1000 11138
2219
465
13822
Call this 14000 kg

14000 ÷ 26.4 = 530 kg per lin. meter.

Approximate weight of Structural Steel in Anchor span

Stringer 100 × 26.4 = 2640
Floor beam 9 @ 610 = 5490
Lateral Bracings 80 × 26.4 = 2110
Girders with details 2 @ 14000 = 28000
38240
Cantilever arms 2 @ 8943 = ~~16686~~ 17886
~~54926~~ kg.
56126

Grand summary for structural steel in bridge

Suspended span 8 @ 15.10 = 120.8 120.8
Anchor span with Cant. arms 7 @ ~~55.0~~ 56.1 = ~~385.0~~ 392.7
~~505.8~~ 523.5
For variation say ~~3% say~~ ~~14.2~~
~~520.0~~ tons. 525. tons.

Design of pier. same as for 6.5 m Roadway. see page 7
Cost of one pier 5000.00

Cost of one abutment 4500.00

Estimate of Cost of Deck Construction.

Concrete in slab. 435 cubic meters @ 19.00 = 8250
Reinforcing Bars. 46. tons @ 160.00 = 7360
Forms 3060 sq meter @ 2.00 = 6120
Pavement 2300 sq meters @ 3.00 = 6900
Handrails. 58 tons @ 300.00 = 17400
wiring &c 2500
48530

Summary for Estimate of Cost
structural steel 525.00 tons @ 230.00 = 121.000
Deck Construction complete 48530
Piers 14 @ 5000 70000
Abutments 2 @ 4500 9000
248530.00
212
36000

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi Okayama-ken

17 spans Gerber träger
Anchor span 7 @ 3.33 = 23.31 meters
Suspended span 3@3.33 = 9.990
.37
10.36 meters

Cantilever arm 2@3.33 = 6.66
.37
6.29 - meters

10.36
6.29
16.65 meters

Total length of bridge 15 @ 23.31 = 349.65
2@ 16.65 = 33.30
382.95

Cross section of bridge same as p16.
Clear roadway 6.0 m.

Reinforced Concrete slab 15 cm
Stringers II Is. 300 × 150 @ 48.34 kg say 2@50 = 100 kg/m

Cross beam 610 kg per piece ÷ 3.33 = 183 kg/m

Lateral Bracing say 80 kg per lin. meter

Main girder
Dead Load Flooring 510 × 6.0 = 3060
Coping 2@200 = 400
HR 2@75 = 150
3610

Structural steel Stringer 100
Floor beam 183
Lateral Bracings 80
Girders assumed 800
1163
4773 ÷ 2 = say 2400 kg/m

Suspended span span length 10.36 m
m = 1/8 × 2400 × 10.36² = 32200 kgm

Live Load 500 × 3.0 = 1500 kg per lin. meter
m = 1/8 × 1500 × 10.36² = 20100
Concentration assumed 4250 kg
m = 4250/2 × 5.18 = 11000
31100

DL. m 32200
63300 kgm

Web assumed 1200 × 9 = 108.0 1/8 web = 13.5 □cm back to back of Ls 121
Effective depth assumed 113 cm flange stress = 63300 ÷ 1.13 = 56000 kg
section required = 56000 ÷ 1200 = 46.60
13.50
33.10 □cm net

2Ls 150 × 90 × 9 @ 16.32 kg = 41.58 - 6.75 = 34.83 □cm

Approximate weight of main girder (Suspended span)

web.	1Pl.	120 × 9	@	84.7	×	10.7	=	905
flange.	4Ls	150 × 90 × 9	@	16.32	×	10.36	=	675
stiffs	20Ls	125 × 90 × 10	@	16.09	×	1.20	=	386
	4Pls.	90 × 9	@	6.35	×	1.03	=	26
End fills	4Pls.	1000 × 9	@	70.50	×	1.03	=	290
	4Ls	150 × 90 × 9	@	16.32	×	.60	=	39
	8Ls	130 × 130 × 9	@	17.73	×	.60	=	85 - 2406

Preliminary Design and Estimate of Cost Ashida-Bashi, Okayama-Ken

web splice	2Pls.	350 × 9	@ 24.70	× .73	=	36.
	4Pls.	150 × 9	@ 10.40	× 1.00	=	42
flange splice	2Pls.	310 × 9	@ 21.80	× 1.20	=	52
	4Ls	130×130×9	@ 17.73	× 1.20	=	85
Lateral Connection	6Ls	90×90×10	@ 13.34	× .40	=	32
	4Pls.	350 × 9	@ 24.70	× .80	=	89
Base plates.	2Pls.	350 × 19	@ 52.0	× .40	=	42
						116
						3000 kg.

3000 ÷ 10.36 = 290 kg per meter.

Summary for structural steel in suspended span

stringers	100 × 10.36	=	1036
floor beams	4 @ 610	=	2440
Lateral bracings	80 × 10.36	=	840
Main girders	2@3000	=	6000
			10316 kg ÷ 10.36 = say 1000 kg.

main girder Cantilever arm Length of arm 6.29

Dead Load floor 3610
steel 1000
4610 ÷ 2 = 2305 kg per lin meter

Reaction = 2305 × $\frac{10.36}{2}$ = say 12000

Dead Load moment $m = \frac{1}{2} \times 2305 \times 6.29^2$ = 45600
$m = 12000 \times 6.29$ = 75500
121100 kgm

Live Load Concentration at hinge unif. load $1500 \times \frac{10.36}{2}$ = 7760
motor truck loading extra 4250
12010

moment = 12010 × 6.29 = 75500
" = $\frac{1}{2} \times 1500 \times 6.29^2$ = 29700
105200
Dead Load moment 121100
226300 kgm

web assumed 1800 × 9 = 162.0 1/8 web 20.3 cm² back to back of Ls 181. cm
Effective depth 176. cm flange ~~stress~~ = 226300 ÷ 1.76 = 129000
Section required = 129000 ÷ 1200 = 107.50
20.3
87.2 cm net

2Ls 150×150×11 = 63.58 − 11.00 = 52.58
1Pl 350 × 12 = 42.00 − 6.00 = 36.00
88.58 cm

Approximate weight of Cantilever

web.	1Pl.	180 × 9	@ 127.0	× 6.60	=	840
flange	4Ls	150×150×11	@ 24.95	× 6.29	=	627
	2Pls.	350 × 12	@ 32.9	× 3.3	=	217
fills	2Pls.	90 × 11	@ 7.75	× 1.10	=	17
	14Ls	125×90×10	@ 16.09	× 1.50	=	338
End Stiff	2Ls	130×130×12	@ 23.36	× 1.80	=	84
½	1 fill	260 × 11	@ 22.50	× 1.50	=	34
End details	4Ls	130×130×12	@ 23.36	× .60	=	56
	2Ls	150×150×11	@ 24.95	× .60	=	30
fills	2Pls.	1000 × 14	@ ~~70.00~~ 86.2	× 1.03	=	~~143~~ 177
misc details						589
	Rivet heads &c	say				111
						3120 kg.

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi Okayama-Ken.

Summary of structural steel in Cantilever arm

Stringers	100	× 629	=	629
floor beam	2 @ 610		=	1220
laterals	80 @ 629		=	502
main girders	2 @ 3120		=	6240
				8591 kg.

Main Girder Anchor span between piers = 23.31 meters

Dead Load say 2500 kg per lin. meter

m = 1/8 × 2500 × 23.31² = 170.000

Less cantilever moment − 121.100

48.900 kgm

Live load 1500 kg per lin. meter

m = 1/8 × 1500 × 23.31² = 102.000

Conc. 4250/2 × 11.65 = 24.800

126800

175.700 kgm

Live load negative moment 105200

48900

56300 ÷ 2 = 28150

175700

Design moment at center = 203850

web assumed 1200 × 9. = 108.0 1/8 web = 13.50 cm back to back of Ls = 121.

Effective depth say 113 flange stress = 203850 ÷ 1.13 = 180.000 kg.

Section required = 180.000 ÷ 1200 = 150.0

13.5

136.5

Try 2Ls 150 × 150 × 11. = 63.58 − 11.00 = 52.58

2Pls 350 × 13 = 91.00 − 13.00 = 78.00

130,58

Approximate weight of Anchor span

web	1 Pl.	120 × 9	@	84.7	× 11.90	=	1010
	2 Pls.	180 × 9	@	127.0	× 5.70	=	1450
flanges.	4 Ls	150 × 150 × 11	@	24.95	× 23.31	=	2320
	2 Pls.	350 × 13	@	35.70	× 23.31	=	1665
	2 Pls.	350 × 13	@	35.70	× 12.00	=	857
	44 Ls	125 × 90 × 10	@	16.09	× 1.30	=	920
	12 Pls.	90 × 11	@	7.75	× 1.00	=	93
	2 Ls	130 × 130 × 12	@	23.36	× 1.80	=	84
	1 Pl.	260 × 11	@	22.50	× 1.50	=	34
	web and flange splices			3 @ 300	say	=	900
	Lateral 14 Ls	90 × 90 × 10	@	13.34	× 1.40	=	75
	7 Pls.	350 × 9	@	24.70	× .80	=	138
	Sole plates say						30
	Shoes. say						800
	Rivet heads etc						310
							10686. say 10700 kg.

10700 ÷ 2331 = 46.0 kg per lin. meter

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Preliminary Design and Estimate of Cost Ashida Bashi Okayama-Ken

Approximate weight of structural Steel in anchor span

stringers	100 · 23.31	=	2330
floor beam	8 @ 610	=	4880
Lateral bracing	80 · 23.31	=	1860
Girders	2 @ 10700	=	21400
			30470
Cantilever arms	2 @ 8590	=	17180
			47650 kg.

Grand summary of structural steel in Bridge.

9 suspended spans	@ 10.30	=	92.7
8 anchor spans	@ 47.65	=	381.2
			473.9 tons.

Pier. say ~~5000~~ ⁰⁰ 4000⁰⁰
abutment . 4500 ⁰⁰

Summary for Estimate of Cost

Structural Steel	474. tons @ 230	=	109000
Deck construction complete	p20		48530
Piers	16 @ ~~5000~~	=	64000
abutments	2 @ 4500	=	9000
			236530

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CALCULATIONS FOR

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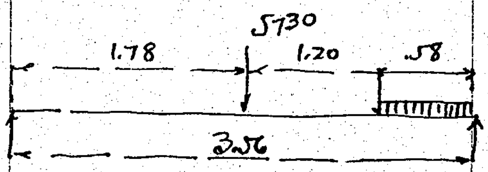
Simple girder span design
$382.0 \div 21.0 = 18.2$
Center to center of bearings - $18.2 - 0.4 = 17.8$ 5 @ 3.56 meters = 17.8 m

Cross section of bridge same as p16.
Reinforced Concrete slab. 15.0 cm
stringer span length 3.56
Dead Load $510 \times 1.70 = 867$
stringer 50
917 $m = 1/8 \times 917 \times 3.56^2 = 1450$

Live Load motor truck stringer reaction = 5730
Uniform load $500 \times 1.7 = 835$ kg per lin. meter



$R = \frac{835 \times .58^2}{2 \times 3.56} = 39.4$

$m = \frac{5730}{2} \times 1.78 = 5100$
$39.4 \times 1.78 = 70$
5170
1450
6620 kg m

section modulus reqd = $\frac{662000}{1100} = 602$

Use I 300 × 150 @ 48.34 kg Sm = 633.2

Cross beam weight say 610 kg per piece. $610 \div 3.56 = 172$ kg/m

Lateral Bracing 80 kg per lin. meter.

Main girder span length 17.8
Dead Load Flooring $510 \times 6.0 = 3060$
Capings 2 @ 200 = 400
Handrails 2 @ 75 = 150
3610

Structural steel
stringer 2 @ 50 = 100
floor beam 172
Lateral Bracings 80
Girder span 2 @ 500 = 1000
1352
$4962 \div 2 = 2481$
say 2500 kg/m

Dead Load moment $= 1/8 \times 2500 \times 17.8^2 = 99000$ kgm

Live Load $500 \times 3.0 = 1500$ kg per meter
Concentration assumed 4250.
$m = 1/8 \times 1500 \times 17.8^2 = 59400$
$\frac{4250}{2} \times \frac{17.8}{2} = 18900$
78300
177300 kg m

~~web assumed~~ $1300 \times 9 = 117.0$ □ cm 1/8 web = 14.6 cm Back to back of ⌊s 131.0
Effective depth say 125 cm flange stress $= 177300 \div 1.25 = 142000$ kg.
section reqd $= 142000 \div 1200 = 118.20$
14.6
103.6 □ cm

2⌊s 150 × 150 × 11 = 63.58 − 11.00 = 52.58
2 Pls. 310 × 11 = 68.20 − 11.00 = 57.20
131.78 109.78 net

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi Okayama-Ken

Alternate section

2Ls 150 × 150 × 15 @	85.50	− 15.00	= 70.50	
1Pl. 350 × ~~10~~ 12	= ~~35.00~~ 42.00	− ~~5.00~~ 6.00	= ~~30.00~~ 36.00	11. metres.
	~~120.50~~ 127.50		~~100.50~~ 106.50	

Approximate weight of girder.

web.	1Pl.	1300 × 9	@	91.8	× 18.2	=	1670
	4Ls	150 × 150 × 15	@	33.55	× 18.2	=	2440
	2Pls.	350 × 12	@	33.00	× 11.0	=	726
End stiffs	8Ls	~~130 × 130 × 12~~ 125 × 90 × 13	@	~~23.36~~ 20.61	× 1.30	=	214
Fill	8Pls.	90 × 15	@	10.48	× 1.00	=	84
int. Stiffs	28Ls	125 × 90 × 10	@	16.09	× 1.30	=	586
	8Pls.	90 × 15	@	10.48	× 1.00	=	84
Splice	8Pls.	200 × 15	@	23.60	× 1.00	=	189
	4Pls.	300 × 9	@	21.20	× .60	=	51
	8Ls	130 × 130 × 15	@	28.84	× 1.30	=	300
Sole plates	2Pls.	400 × 30	@	94.0	× .50	=	94

Variation − 5% say 312

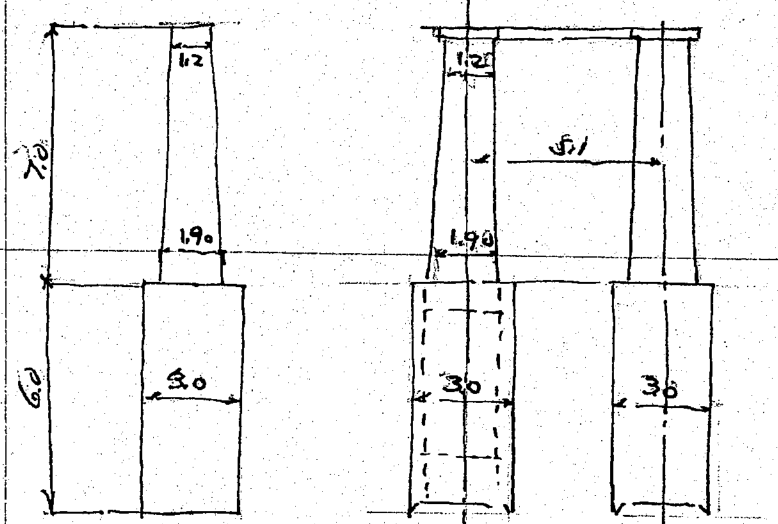
6750 kg.

Approximate weight of Steel in one span

stringers	100 × 18.2	=	1820
floor beam	6 @ 610	=	3660
Lateral	80 × 18.2	=	1460
girders.	2 @ 6750	=	13500
misc say			~~1500~~
			20940
			−500
			20440 kg.

21 spans @ 20.44 = 430 tons.

Pier



Approximate Concrete in pier.

shaft Top. 1.2ϕ 1.13
bott. 1.9ϕ 2.83
3.96 ÷ 2 = 1.98 × 7 = 13.9 × 2 = 27.8

web. .75 × 3.6 × 7 = 18.9

46.7

well 3ϕ = 7.07
2.4ϕ = 4.52
2.55 × 6 × 2 = 30.60

Top and bottom filling
2.5 × 4.52 × 2 = 22.60

53.20

Sand filling 3.5 × 4.52 × 2 = 31.7 cubic metres.

Excavation say 7.07 × 6 × 2 = 85.0 cubic metres

Forms. 2 × 4.9 × 7.0 = 68.0
3.60 × 7 × 2 = 50.0
17.0 × 6 × 2 = 204.0
322.0 sq metres.

Curb shoe 0.6 tons
Reinforcing bars 2.0 tons.

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi, Okayama-Ken.

Approximate Cost of one pier.

Concrete	1:2:4	100 cubic meters	@ 19.00	=	1900	
Sand filling		32 "	@ 2.00	=	64	
Reinforcing steel		2 tons	@ 130.00	=	260	
Curb shoe		0.6 ton	@ 230.00	=	138	
Form		322	@ 2.00	=	644	
Excavation		85	@ 5.00	=	425	
					3431	call this 3500.00

~~Estimate of Cost~~

~~Structural steel~~	430 @ 230.00	=	99000.00	
Deck complete	p20		48530.00	
piers	20 @ 3500.00		70000.00	
abutments	2 @ 4500.00		9000	
			226530.00	

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi Okayama-Ken.

Reinforced Concrete Spans. 382 meter

3 Continuous girder – 3 @ 12.5 = 37.5 meters End to End 38

$\times$ 8 ... 8

300.0 meters ... 304.0 meters

Between 382 – 304 = 78. 2 @ 39.0 out to out = 78 meters

382. "

Center to Center of End bearings say 38.0 meters

Design of truss span See page 11.

Cross section of bridge same as on page 11.

Roadway 6.0 meters ℄ to ℄ of trusses 7.25 meters

10 panels @ 3.80 = 38.0 meters

Stringers 2 @ 60 kg = 120

2 @ 50 " = 100

220 kg per lin. meter

Cross beam say 1040 ÷ 3.80 = 274 " " " "

Lateral Bracings 90 " " " "

trusses assumed 2 @ 700 = 1400

1984

flooring complete 3610

5594 ÷ 2 = 2800 kg per lin. meter

Moment at Center = 1/8 × 2800 × 38.0² = 505,000 kgm

Live Load Moment " " = 1/8 × 1500 × 38.0² = 270.000

Concentration 4250/2 × 19.0 = 40400

815400 kgm

Depth of truss 4.0 chord stress = 815400/4 = 204000 kg.

section reqd = 204000 ÷ 1000 = 204.0

Approximate weight of truss

204.0 × 2.7 × 0.785 = 432

38% 164

596 say 600 kg per meter

Summary for structural steel

stringers	220 × 39.0	=	8600	
floor beam	11 @ 1040	=	11440	
lateral bracing	90 × 38	=	3420	
trusses	1200 × 38.0	=	45600	
shoes say			2500	
			70960	say 71 tons

2 spans @ 71.0 = 142 tons

Deck construction – 48530 / (382 × 6) = 21.20 ¥20 per sq meter.

6 × 78 × 21.20 = say 10.000 ¥

Piers 7500 ¥ see page 14

Side pier say 5000

Total Cost

structural steel	142 @ 260	=	36700
Deck construction			10.000
piers			7500
"			10.000
			64200 ¥

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CALCULATIONS FOR

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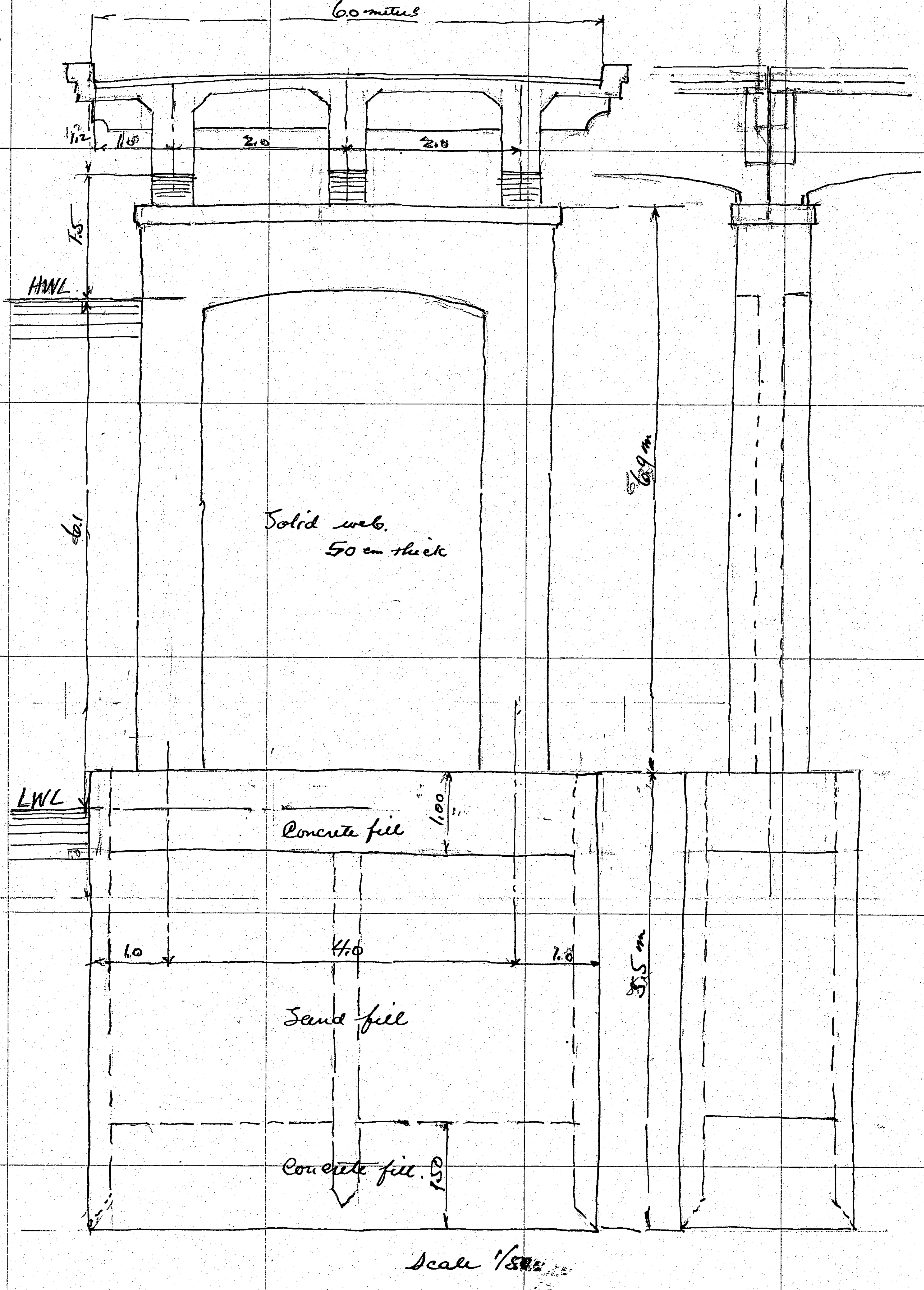
Reinforced Concrete Span
3 Continuous spans @ 12.5 = 37.5 meters



CALCULATIONS FOR

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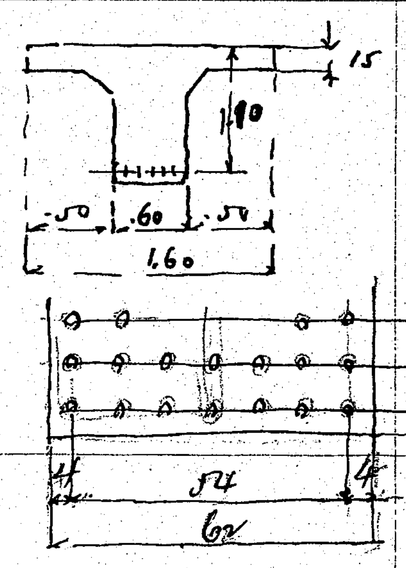
Concrete slab. 15.cm with pavement 510 kg per sq meter
Brackets .30 × .40 × 5.0 = 0.6 cubic meter
weight 0.6 × 2400 = 1440 kg per lin meter 3.13 m spacing.
1440 ÷ 3.13 = 465 kg per lin meter.
465 ÷ 6.0 = 77.5 say 80 kg.
slab and cross beam 510 + 80 = 590 kg per sq meter

Amount of Concrete .15 × .60 = .90
Coping 2 @ .12 = .24
Cross beam 0.60 ÷ 3.13 = .19
1.33 cubic meter per meter of span.

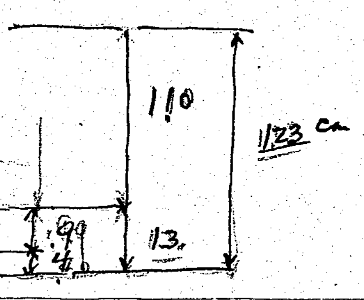
Dead Load on Outside girder.
Slab and cross beam 590 × 2.0 = 1180
Coping. 200
Handrail 100
1480
weight of beam .60 × 1.1 = .66
Fillet. .9
.75 × 2400 = 1800
3280 kg per lin meter.
Dead load moment = $\frac{1}{10}$ × 3280 × 12.5^2 = 51100 kgm

Live Load Uniform load 1000 kg per lin. meter.
Concentration assumed 5000
Live Load moment = $\frac{1}{10}$ × 1000 × 12.5^2 = 15600
.8 × 2500 × 6.25 = 12500
28100
D.L.m 51100
79200 kgm.



Steel area = $\frac{7920000}{110 \times 1200 \times 7/8}$ = 68.5 cm^2

Steel bars 22 cm ϕ. 68.5 ÷ 3.8 = 18



Stress in Concrete t = 15 d = 110
t/d = $\frac{15}{110}$ = .136

Reinf. %. $\frac{68.5}{110 \times 160}$ = .39 %

value of k = .34 j = .94

Stress in Concrete f_c = $\frac{2 \times 7920000 \times .34 \times 110}{[(2 \times 0.34 \times 110 - 15) \times 160 \times 15 + (.34 \times 110 - 15)^2\, 62]\, .94 \times 110}$ = 32.7 kg/cm²

volume of Concrete in beam. 1.08 × .62 = .67
Filler .09
3 @ .76 = 2.28
add .62
2.90 cubic meters.
Slab etc 1.33
4.23 cubic meters
4.23 × 170 kg/m = 720 kg per lin meter of span

Forms. 16. square meters per lin meter of span

CALCULATIONS FOR

Preliminary design and Estimate of Cost Ashida-Bashi, Okayama-Ken.

Bent ~~9 × 9~~ 2 - 1.0 × 1.0 = 2.00
.50 × 3.0 = 1.50
3.50 × 6.9 = 24.20
add - 1.80
26.00 cubic meters

Reinforcing steel. 26.0 × 200 = ~~5~~.200 tons.
forms. 12 × 6.9 = 83 sq meter.

Concrete	26.00 cubic meters	÷ 12.5	= 2.08	} per lin. meter
Reinf. bars	5.20 tons	"	= 410 kg.	
Forms	83 square meters	"	= 6.65 sq meter	

Well. 2^{ϕ} = 2^{ϕ} = 3.14
1.4^{ϕ} = 2 × 4 = 8.00
Outside area 11.14 square meter
Inside area 1.4^{ϕ} = 1.54 1.54
1.4 × 3.7 = 5.18 5.18 11.14
~~1.022~~ 6.72 6.72
4.42

Shell 4.42 × 5.5 = 24.30
Top and bottom filling 6.72 × 2.5 = 16.8 cubic meters
Intermediate filling 6.72 × 3.0 = 20.2 "

Reinforcing steel. say 24.3 × 80 = 1.95 tons.

Forms. 14 × 5.5 = 77.0
15.2 × 5.5 = 83.5
160.5

Curb shoe say 1.00 tons
Excavation say 11.14 × 5.5 = 61.0 cubic meters

Estimate of cost of Deck Construction (superstructure).

Concrete	4.23	@ 19.00	=	80.5
Reinf. bars	.72	@ 160.00	=	115.0
Forms.	16	@ 3.50	=	56.0
pavement	6.00	@ 3.00	=	18.0
Handrail	.15	@ 300.00	=	45.0
~~misc.~~				314.5 yen per lin meter

314.5 × 304. = 95600
misc say 2500
98100.00 yen

Substructure

Bent.	Concrete	26.00	@	19.00	=	494.
	Reinf. bars.	say 4.20	@	160.00	=	672
	Forms.	83.0	@	2.50	=	208
well.	Concrete	41.1	@	19.00	=	780
filling	sand	20.2	@	2.00	=	40
	Reinf.	1.95		160.00	=	312
	forms.	160.	@	1.50	=	240
	Curb shoe	1.00	@	220	=	220
	Excavation	61.0	@	5.00	=	305
						3271 yen

22 @ 3270 = 71940
Abutments say 2 @ 4500 = 9000
80940

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi Okayama-Ken.

Concrete span	Superstructure	98100 .00	
	Substructure	80940	
		179040	— 325.00
Steel spans complete		64200	
		243240 .00	

Summary

①	15 spans 26.4 (86.5') meter span 21' (6.5 m). Roadway — (Gerber type)	258450.00
②	Remodeling present piers and abutment. 6.0m Roadway 12 - 90' girderspans 2 - pony trusses @ 40.0 meters (132') (half through)	211920.00
③	15 spans 26.4 meter span 6.0 m Roadway — (Gerber type)	248530.00
④	17 spans 23.31 (76.0) meter 6.0 m Roadway — (Gerber type)	230500.00
⑤	21. spans @ 18.2 (60') single span 6.m Roadway	226500.00
⑥	Concrete spans 24 spans 12.5 m (41') 6m Roadway 2 - 38 meter poney truss spans	243240.00

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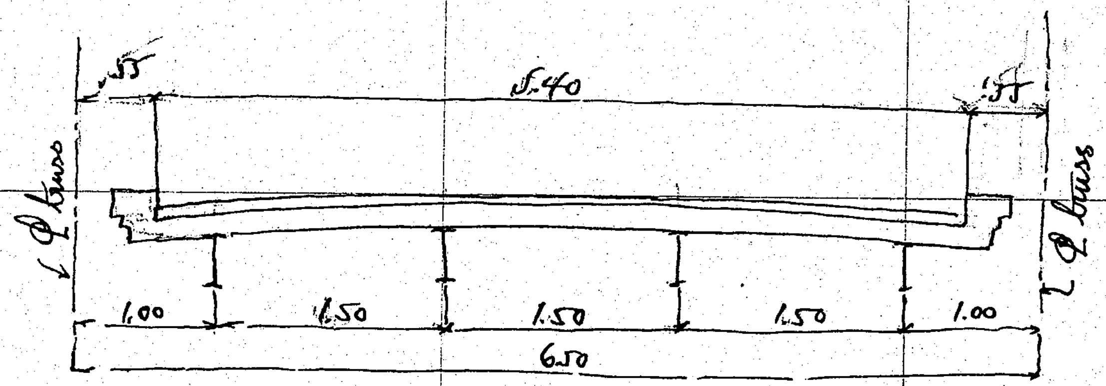
CHECKED BY ____ DATE ____ **PAGE NO** 33

CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi, Okayama-Ken

Span length 5- 37.5 meter truss spans.
18- 17.7 meter girder spans. } Total length 395. meters
Roadway 5.4 meters clear.

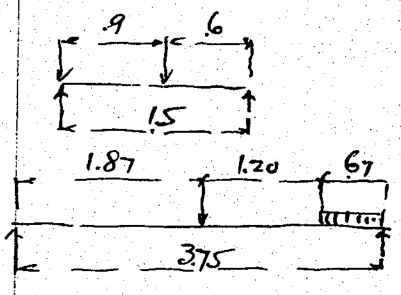
Design of truss span 10 @ 3.75 = 375



Stringer. 3.75 meters Dead Load 510 × 1.5 = 765
stringer 50
815

DL m = 1/8 × 815 × 3.75² = 1430 kgm

Live Load motor truck loading. 3900 kg.



3900 × .6/1.5 = 1560
3900
5460 kg.

Uniform live load 500 × 1.5 = 750 kg per lin. meter
Reaction = 750 × .67²/(2 × 3.75) = 45

moment 2730 × 1.87 = 5110
45 × 1.87 = 84
5194

5194
1430
6624

Sm = 6624 / 514400/1100 = 602.0
use 300 × 150 × 8 @ 48.34 Sm = 633.2

Design of truss.
Dead Load Floor 510 × 5.40 = 2760
Coping 2 @ 200 = 400
HR 2 @ 75 = 150
3310

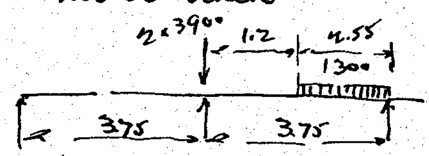
Structural steel stringer 200
floor beam 265
lateral 55
trusses say 1200
1720

3310
1720
5030 ÷ 2 = 2515 kg per lin. meter.

Dead Load moment at center DLm = 1/8 × 2515 × 37.5² = 442000 kgm

Live Load uniform load. 482 kg/m² 482 × 2.70 = 1300 kg.
1300 × 3.75 = 4870

motor truck



Unif. 1300 × 2.55² / (2 × 3.75) = 1130
motor truck 7800
8930
4870
4060 call this 4000 kg. extra concentration.

Live Load moment 1/8 × 1300 × 37.5² = 228500
2000 × 37.5/2 = 37500
266,000
DL. 442,000
708000

Height at center 4.0 stress = 708000/4 = 177,500 kg
SR = ~~708~~ 1775 cm

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida - Bashi. Okayama - ken.

From our standard design
weight of truss complete 59.3 tons
For 2 truss spans @ 59.3 = 118.6 tons or say 120 tons

Design of simple girder span 17.7 meter 5 panels @ 3.45 m = 17.25
1 bearing. .40
17.65 + .05 = 17.70 meters

Cross section similar to truss span.
Dead Load Deck 3310
Steel stringer 2@50 = 100 1225
floor beam 175 4535 ÷ 2 = 2270 kg.
lateral bracing. 50
Girder span say 2@450 = 900
1225

DL moment = 1/8 × 2270 × 17.25² = 84500 kgm

Live Load 500 × 270 = 1350 kg per lin. meter.
Extra Concentration say 4000 kg. at center.
m = 1/8 × 1350 × 17.25² = 50200
4000/2 × 17.25/2 = 17250
67450
84500
151950 kgm

web assumed 1200 × 9 = 108. 1/8 web = 13.5 Back to back 121.
Effective d = 117 flange stress = 152000 ÷ 1.17 = 130.000 kg
SR = 130.000 ÷ 1200 = 108.0 cm net = 13.5 = 94.5
Use 2L 150 × 150 × 11 = 63.58 - 11.0 = 52.58
2Pls. 310 - 10 9 = ~~58.20~~ 55.80 - ~~11.20~~ 9.0 = ~~57.20~~ 46.80
131.78 ~~109.78~~
129 38 99.38

Referring P26 6750/18.2 × 17.7 = 6550 kg.
say 6400 kg. 6400 ÷ 17.7 = 362

approximate weight of steel in one span.
Stringer 2 @ 50 = 100
floor beam 175
lateral bracing. 50
girders 2 @ 362 = 724
1049 say 1050 kg.
1050 × 17.7 = 18.6 tons.

18 Spans @ 18.6 = 334 tons @ ~~230~~ 220 = 77000 .00
2 Spans @ 60. = 120 @ ~~260~~ 255 = 31200
454 108,200 .00

Estimate of Cost of Deck Construction referred to p20

Concrete in slab.	435 cubic meters	@ 19.00	= 8250	
Reinforcing bars.	43 tons.	@ 155.00	= 6660.	
forms.	2800 sq meter	@ 2.00 / 5.50	= 5600	
Pavement	2140 sq meters	@ 3.60	= 11800	Asphalt block
Handrails.	59 tons	@ 300.00	= 17700	
wiring & misc.			2500	
			52510 .00	

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Ashida-Bashi Okayama-Ken.

Estimate of Cost substructure

For girder spans.	16 @ 3500	=	56000
2 abutment piers	2 @ 4500	=	9000
1 pier for truss spans.			8000
2 piers. truss and girder spans	2@ 6500	=	13000
			86000

Summary for Estimate of Cost

steel complete	girders	334 tons (325 @)	@ 230	=	77000	73500
	trusses	120 tons (110)	@ 260	=	31200	28600
Deck					52510	102100
Piers and abutments					86000	~~80000~~
					246710	8000
						232[illegible]

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CALCULATIONS FOR

昭和三年六月

岡山縣周匝橋予算設計書

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CALCULATIONS FOR

Sesai Bashi Okayama-Ken

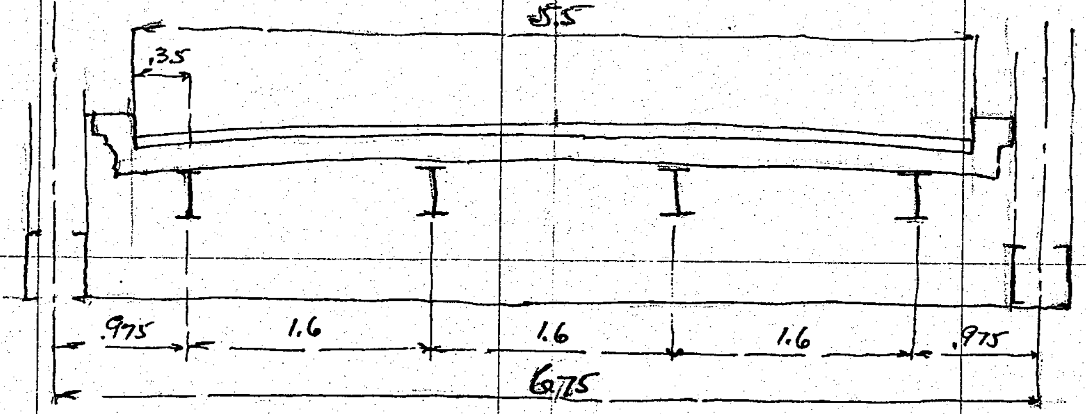
7 spans @ 36.0 = 252.0
6 spaces @ .6 = 3.6
255.6 meters.
.6
256.2 meters End to End of parapet walls

Roadway 5.5 meters clear.

Roadway slab.

15 centimeter slab.	360
5c pavement	105
1.5 pavement cushion.	33
misc. say	12
	510 kg per sq meter

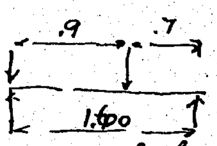
Stringer Cross section as shown below
Span length of stringer 4.5 meters



Load on stringer $\cancel{5}10 \times 1.6 = 816$
stringer say 50
550 kg. 866

$m = \frac{1}{8} \times \cancel{550}\,866 \times 4.5^2 = \cancel{1392}\ 2200$

Live load motor truck loading. rear wheel 2925 kg with 30% impact



$2925 \times \frac{.7}{1.6} = 1280$
2925
4205

L.L. moment $= \frac{4205}{2} \times 2.25 = 4730$
D.L. moment 2200
6930

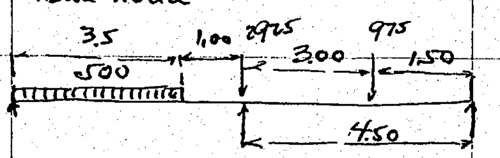
Sm req'd $= \sqrt{\frac{6930 \times 100}{1100}} = 633.$

Use 300 · 150 I @ 48.34 kg Sm = 633.2

Floor beam span length 6.75 spacing 4.5 meters
Dead Load $550 \times \cancel{4}.5 = 2480$ kg.
weight of girder say 120
2600 kg.

D.L. moment $= \frac{1}{8} \times 2600 \times 6.75^2 = 14800$

Live Load



Rear wheel 2925 ~~front wheel~~ 975
$975 \times \frac{1.50}{4.50} = 325$
2925
3250 kg.

Uniform load $500 \cdot .35 \times \frac{1.75}{4.50} = 680$ kg per lin ft.

Approximate moment
motor truck
$2 \times 3250 \times 2.92 = 19000$
$3250 \times 1.80 = 5850$
13150

3.37
.45
2.92

Unif. load say $\frac{1}{8} \times 680 \times 6.75^2 = 3870$
L.L. m. 17020
D.L. m. 14800
31820 kgm

web assumed $670 \times 8 = 536$ $\frac{1}{8}$ web $= 6.70$ cm²
flange stress $= 31820 \times 100 \div 63.6 = 50{,}000$ kg SR $= 50.000 \div 1200 = 41.7$
6.7
35.0 net

try 2L 125 × 90 × 10 = 40.50 gross − 4.4 = 36.10 □ cm net

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Susai-Bashi, Okayama-Ken.

Approximate weight of Intermediate Floor Beam

1 web	53.6		
flanges 2 @ 40.5 =	81.0		
	134.6 @ .785	= 106.	
Details say 25%.		27	
		133 × 6.75 =	900 kg per piece.

Lateral Bracing. 55 kg per lin meter of span.

Design of truss.

Dead Load.	Floor	510 × 5.5 =	2800	
	Coping	2 @ 250 =	500	
	Handrails.	2 @ 100 =	200	
			3500	3500
Structural Steel.	stringers	4 @ 50 = 200		
	floor beam	200		
	lateral system	55		
	trusses	1000		
		1455		1455
				4955 ÷ 2 = say 2500 per truss.

Dead Load moment at center of span = 1/8 × 2500 × 36.0² = 404,000 kg-m.

Live Load 500 × 5.5/2 = 1375 kg per m.

Live load moment = 1/8 × 1375 × 36.0² = 222.000 kg-m.
Extra conc. 2000 at center 1000 × 18 = 18 000
644,000 kg-m.

Depth of truss 4.35 assumed 644000/435 = 148.000 SR = 148.0 cm

Approximate section at center .148 × 2.7 = 400 @ .785 = 314 kg
Details say 38%. 119
433 kg per lin meter.

Summary of structural steel in one span

stringers	4 @ 50 = 200 × 36.6	=	7320
floor beam	9 @ 900	=	8100
Lateral Bracings	55 × 36.0	=	1980
trusses	866 × 36.0	=	31200
shoe &c	say		3000
			51.600 tons

For 7 spans @ 51.6 = 361.2 tons

Load on shoe DL. 1/2 × 2500 × 36.6 = 45700
LL. 1/2 × 1375 × 36.6 = 252.00
70900
Extra load say 2100
73000 kg.

Base. 50. × 75.0 = 3750
73000 ÷ 3750 = 19.5 kg/cm²

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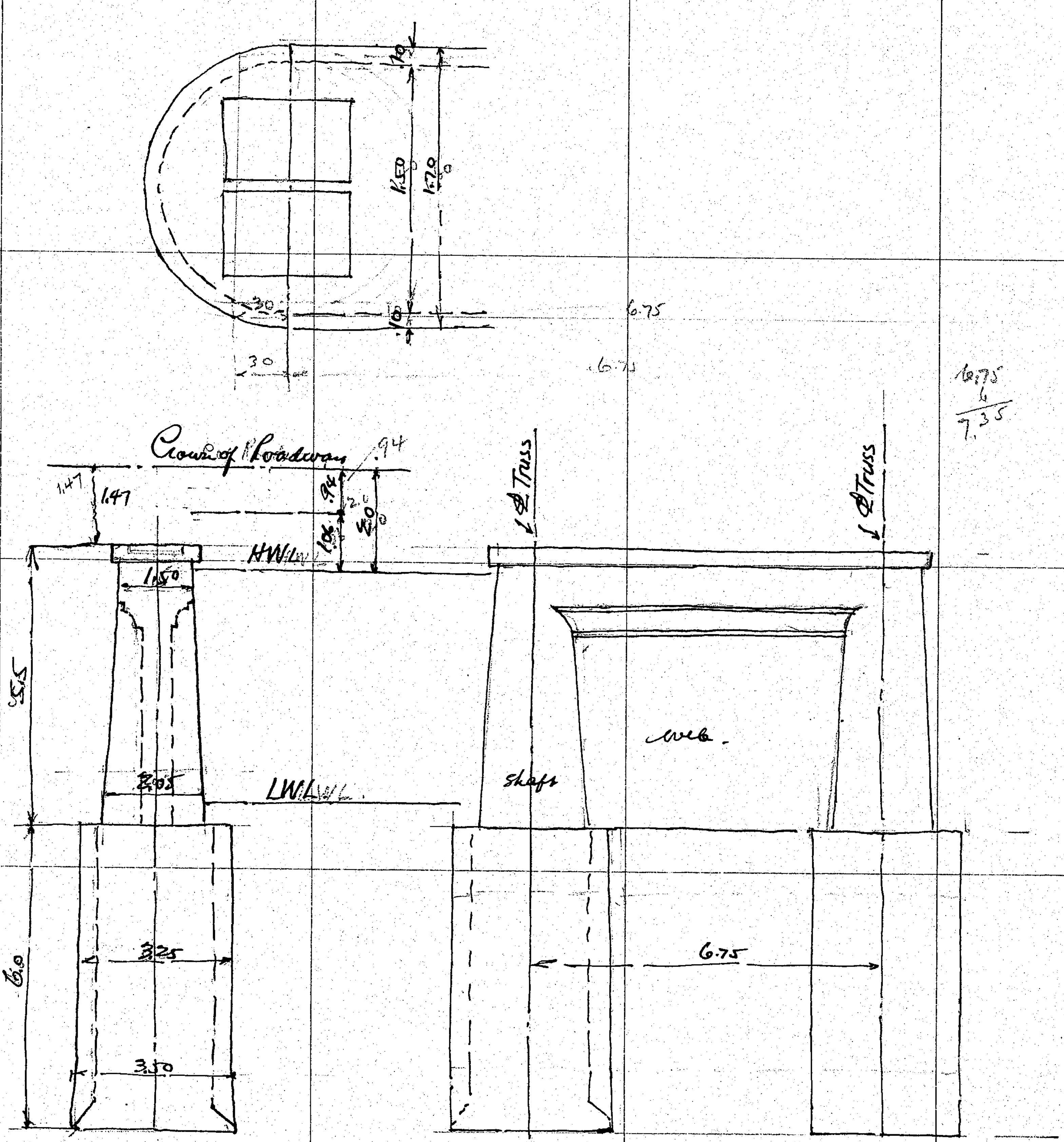
CALCULATIONS FOR

Preliminary Design and Estimate of Cost, Susai-Bashi, Okayama-Ken.



Approximate vol of Concrete

Top. area	1.5^{ϕ}	@	1.76	
Bott area	2.05		3.30	
			5.06 ÷ 2 = 2.53 × 2 × 5.5 =	27.90
web. Top.		1.5 × 1.5 =	2.25	
		4.0 × .75 =	3.00	
			5.25 × 5 =	26.25
volume of shaft.				54.15 cubic meters

well	3.0^{ϕ} =	7.07				
	2.4^{ϕ} =	4.52				
		2.55 × 6.0 =	~~33.30~~	~~2 @ 33.30~~ =	~~66.60~~	cubic meters
Inside filling at bottom		4.52 × 1.5 =	6.78	2 @ 6.78 =	13.56	" "
Inside filling at top		4.52 × 1.0 =	4.52	2 @ ~~4.52~~ =	9.04	" "
Intermediate filling		4.52 × 3.5 =	15.80	2 @ 15.80 =	31.60	" "

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost, Susai-Bashi, Okayama-Ken

Excavation of well Say volume of well
2 – 7.07 × 6.0 = 85.0 cubic meters

Reinforcing Steel in Shaft well } 2 tons Curb shoe .75 tons.

Summary for materials

Concrete in shaft	1:2:4	54.15 cubic m	@	19.00	=	1030
Concrete in well shell	"	66.60	@	"	=	1268
well filling	"	22.60	@	"	=	430
" "	1:4:8	31.60	@	13.00	=	410
Reinf.		2.00 tons	@	130.00	=	260
curb shoe.		.75 tons	@	250	=	188
Forms.	350 sq meters @ 2.00					700
Excavations	6.0 meter deep.					600
						4886.00

Total Concrete = 174.95 cubic meters @ 2200 = 385,000 kg
Super imposed load. 4 × 73 000 = 292000
677,000 kg.

Base area = 2 @ 7.07 = 14.14 677. ÷ 14.14 = 47.8 tons / m²
or 4.4 tons / ft²

Base area = 3.5$^\phi$ 2 @ 9.6 = 19.2
Unit bearing = 677.000 ÷ 19.2 = 35.30 tons / m²
3.28 tons / ft²

make well 3.5$^\phi$ at bottom 3.25$^\phi$ at top.
Revised volume of Concrete.
well out 3.25$^\phi$ – 8.30
in 2.66 5.56
2.74 × 6. = 16.5 2 @ 16.5 = 33.0

Inside filling at bottom	5.56	× 1.5	=	8.35	× 2	=	16.70	cubic m
at top		× 1.0	=	5.56	× 2	=	11.12	" "
Intermediate filling		× 3.5	=	19.5	× 2	=	39.00	" "

Shaft 54.15
well shell 33.00
filling 27.82
" 39.00
153.97 @ 2200 = 338.000 kg
Super imposed load 4 × 73000 = 292.000
630.000 kg.

Base area = 19.2 Unit bearing = 630.000 ÷ 19.2 = 32.8 tons / m²
3.05 tons / ft²

Estimate of Cost of one pier.

Concrete in shaft	54.15	@	19.00	=	1030
Concrete in well.	60.82	@	"	=	1160
" " "	39.00	@	13.00	=	~~455~~ 506
Reinforcing steel	2. tons	@	130.00	=	260
Curb shoe	.75 tons	@	250.00	=	188
Form say	350 sq m	@	2.00	=	700
Excavation + well sinking	2-6.00 meter				600
					43~~93~~ 4443

Call this 4500 yen

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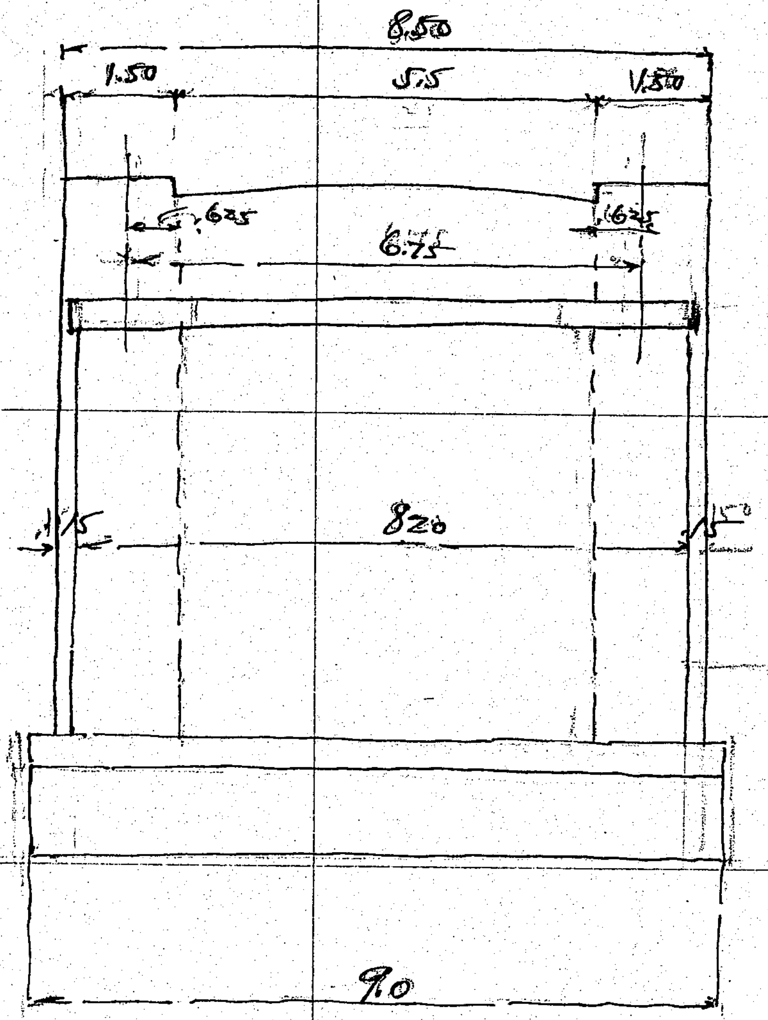
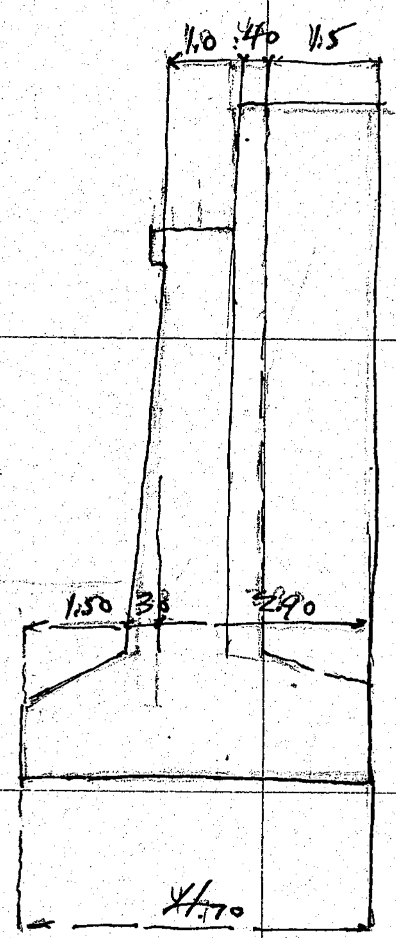
CALCULATIONS FOR

Preliminary Design and Estimate of cost Susai-Bashi, Okayama-Ken.





Approximate Volume of Concrete.

Base.	9.0 × 4.7 × 1.5	=	63.50 cubic meters
Shaft.	1.50 × 5.0 × 8.50	=	64.00 cubic meters
Parapet wall.	.40 × 1.5 × 8.50	=	5.10
Wing walls.	2 - 1.50 × 7.0	=	21.00
			153.60

Forms. 2 - 8.50 × 9.0 = 153.0
Sides 27 × 4 = 108
261 sq meters.

Reinforcing Bars. 2. tons.

Approximate Cost.

Concrete.	153.60	@ 19.00	=	2920
Forms.	261.0 sq m	@ 3.50	=	912
Reinforcing bars	2 tons	@ 130	=	260
Excavation &c.				400
				4492

Call this 4500 yen.

Concrete in slab. 7 @ 36.6 meters = 256.0 meters. total length.
.150 × 5.5 = .825
2@.100 .200
1.025 Call this 1.10 × 256.0 = 281.6 cubic meters

Reinforcing bars. 20 kg. × 256 × 5.5 = 28.2 tons.

Area of form 256 × 7.0 about = 1800 sq meters.

Area of pavement 256 × 5.5 = 1410 sq meters.

Handrails. 512 lin. meters @ 75 kg = 38.5 tons

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Susai-Bashi for Okayama-Ken

Approximate Estimate of Deck Construction -

Concrete in slab.	281.6 cm	@	19.00	=	5350.00
Reinforcing bars	28.2 tons	@	160.00	=	4500.00
Forms	1800.0 sqm	@	2.20	=	3960.00
pavement	1410.0 sqm	@	3.00	=	4220.00
Handrails	38.5 tons	@	300.00	=	11550.00
					29580.00
Pedestal + wiring + lighting apparatus					1420
					31000.00

Summary for Estimate of Cost.

7 - 36.0 meter span 5.5 meter Roadway
Total length between parapet walls. = 256.0 meters

Structural Steel -	361.2	@	260.00	=	94.000.00
Deck Complete with Handrails + Pavement					31.000.00
Piers	6 @ 4500.00			=	27.000.00
Abutments	2 @ 4500				9.000.00
					161,000.00

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CALCULATIONS FOR

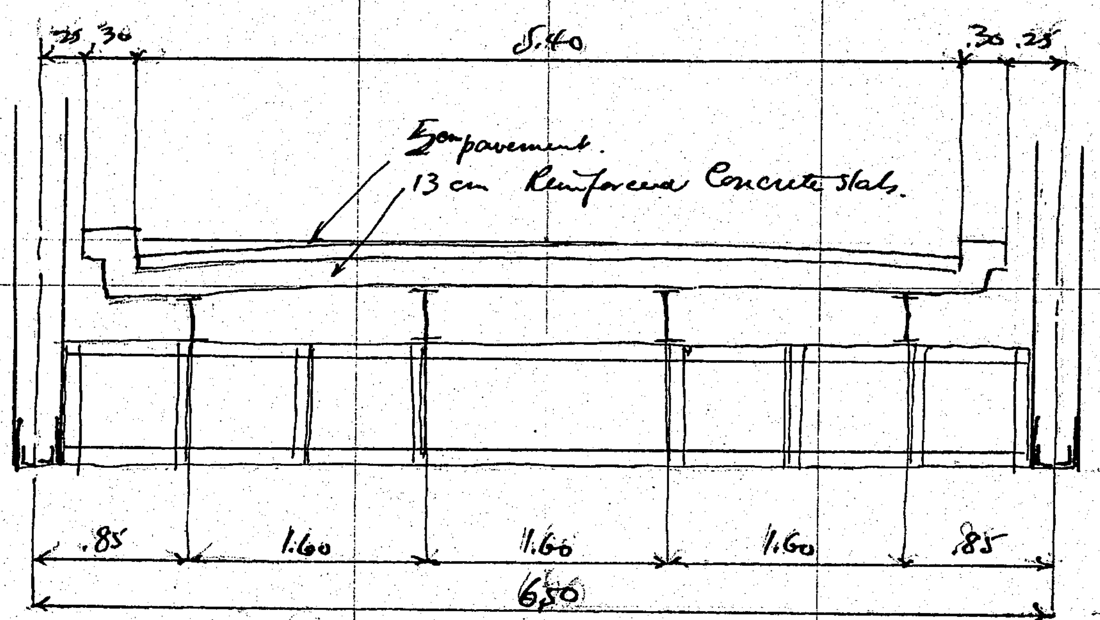
Susai-Bashi Okayama-ken.

7 spans @ 38.0 = 266.0
6 spaces @ .56 = 3.36
269.36
.50 + 2 @ .06 .62
269.98 out to out or between parapet walls.

Between banks. 273.70 meters
Between waterline 268.86 meters

Design of truss 8 @ 4.75 meters = 38.0 meters
Roadway 5.4 meters clear pavement ~~Granolithic~~ pavement. 5 cm

Cross section of bridge



Floor Slab.

Dead Load pavement 5 cm Soliditit @ 22 = 110
Concrete slab. 13 cm @ 24 312
Misc filler 28
450 kg

For design of slab. see page 1-2 Tokiwa-bashi.

Stringers span length 4.75 meters see Tokiwa-Bashi.
Use 300 - 150 I @ 48.34 kg Sm = 633.2

Cross beam span length 6.75 same as final design of Ashida-bashi.
weight of intermediate floor beam say 925 kg.
925 ÷ 6.43 = 144 kg per lin meter
925 ÷ 4.75 = 195 " " " " of span

Lower Lateral Bracings see Ashida-Bashi page 8
Approximate weight of bottom lateral bracings

4 Ls 125 × 75 × 10 @ 14.91 kg × 7.5 = 448
4 ~~6~~ Ls 75 × 75 × 9 @ 9.96 × 7.5 = ~~448~~ 299
747
~~896~~

Center connection say 5 @ 25 125

2 × ~~1021~~ 872 = ~~2042~~ 1744 kg

~~2042~~ 1744 ÷ ~~4.75~~ 38 = ~~37.~~ 46 kg

panel load ~~46~~ × 4.75 = 218 kg

4.75^2 = 22.55
6.75^2 = 45.50
68.05
8.25

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CALCULATIONS FOR

Susai Bashi for Okayama-ken

Design of truss   8@ 4.75 = 38.0 meters

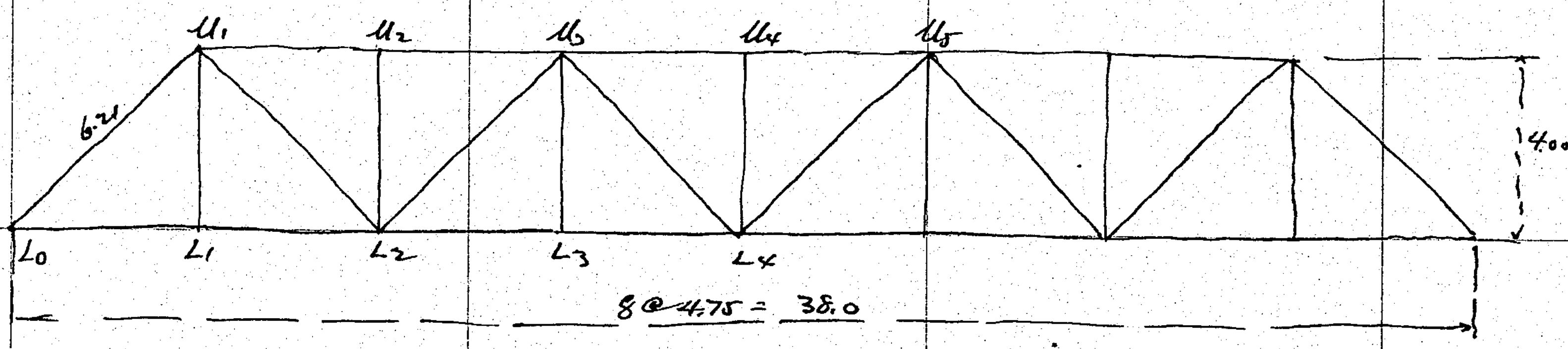
Dead Load.

450 × 5.4	=	2440	
Coping 2@ 220	=	440	
Handrails 2@ 60	=	120	
		3000	— — 3000

structural Steel   stringers 4@50 = 200
floor beam 925 ÷ 4.75 = 195
lateral bracing. 46
trusses 2@ 450 900

1341   1340
4340 ÷ 2 = 2170 kg.

Panel Concentration   2170 × 4.75 = 10300 kg.



$\sec\theta = \frac{6.21}{4.00} = 1.55$ W $\sec\theta$ = 10300 × 1.55 = 16000

$\tan\theta = \frac{4.75}{4.00} = 1.19$ W $\tan\theta$ = 10300 × 1.19 = 12300

Chord stresses

L_0-L_2	16000 / 12300	× 3.5	=	43000
U_1-U_3		× 6.0	=	73700
L_2-L_4		× 7.5	=	92000
U_3-U_4		× 8.0	=	98200

Diagonals

L_0-U_1	16000	× 3.5	=	56000
U_1-L_2		× 2.5	=	40000
L_2-U_3		× 1.5	=	24000
U_3-L_4		× .5	=	8000

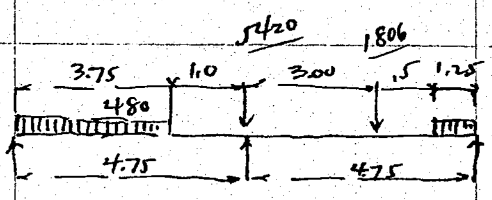
Live Load on truss

Uniform live load $w = \frac{100.000}{170+38} = 480$ kg/m²

Live load on truss   motor truck loading.   rear wheel   2250
Impact $\frac{20}{60+38}$ = 20.4 %   460
2710 kg. × 2 = 5420 kg

Front wheel — 2710 ÷ 3 = 903 kg × 2 = 1806 kg.

full uniform load 480 × 5.4 = 2600 kg. per lin meter



$\frac{480 \times 3.75^2}{2 \times 4.75}$ = 710

$\frac{480 \times 1.25^2}{2 \times 4.75}$ = 80

790 × 2.70 = 2140
5420

1806 × $\frac{1.75}{4.75}$ = 665
8225
6150
2075 kg. say 2100 kg.

Unif. load full   480 × 4.75 × 2.7 = 6150

6150 × $\sec\theta$ = 7300
" × $\tan\theta$ =

Chord stresses

L_0-L_2	25800	L_0-U_1	9530 / 6150×1.55 / ~~7300~~ /8	× 28	=	~~25600~~	33400
U_1-U_3	44200	U_1-L_2		21	=	~~19200~~	25000
L_2-L_4	55000	L_2-U_3		15	=	~~13700~~	18000
U_3-U_8	58800	U_3-L_4		10	=	~~7300~~	12000
				6	=	~~5500~~	7200

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Susai-Bashi Okayama-Ken

Extra Concentration 2100 kg.

Chord stress

L0-L2	2100 × 7/8	× 1.19	× 1.0	=	2180
U1-U3	6/8		2.0	=	3650
L2-L4	5/8		3.0	=	4700
U3-U4	4/8		4.0	=	5000

Diagonal

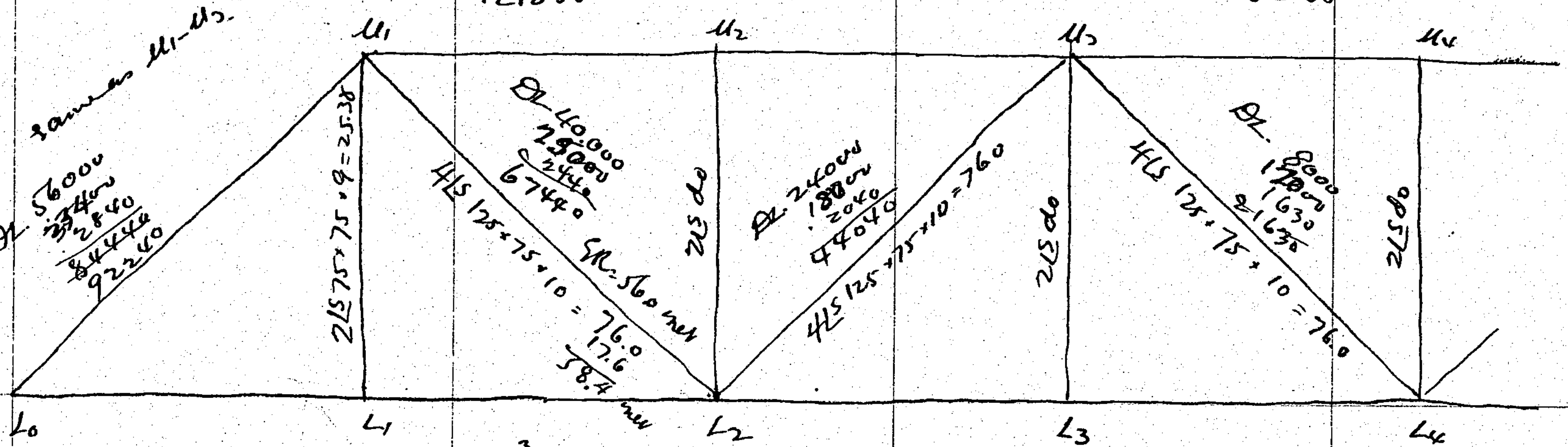
L0-U1	2100 × 7/8 × 1.55	=	2840
U1-L2	× 6/8	=	2440
L2-U3	× 5/8	=	2040
U3-L4	4/8	=	1630
L4-U4'	3/8	=	1220.

DL. 73700, 44200, 3650, 121550

1 Pl. ~~400~~ 455 × 9 = ~~36.0~~ 41.0
2 Ls 150×90×9 = 41.58
2 Pls. 250 × 10 = 50.00
132.58

DL. 98200, 58800, 5000, 162000

1 Pl. 400 × ~~10~~ 9 = ~~40.00~~ ~~36.00~~ 41
2 Ls 150×90×12 = 54.72
2 Pls. 250 × 10 = 50.00
2 Pls. 100 × 12 = 24.00
~~164.72~~ 169.72



DL. 43000, 25800, 2180, 70980

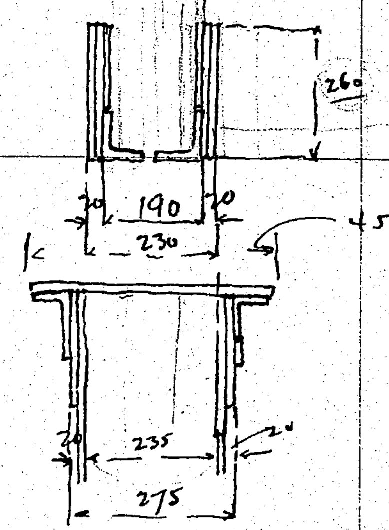
2 Pls. 250 × 10 = ~~45.0~~ 50.0 − ~~11.9~~ 13.2 = ~~33.1~~ 36.8
2 Ls 90×90×10 = 34.0 − 8.8 = 25.2
84. 62.0

SR = 59.0 net

DL. 92000, 55000, 4700, 151700

2 Pls. 250 × 10 = 50. − 11.9 ~~26.8~~ 38.1
2 Pls. 160 × 10 = 32 − ~~8.8~~ 4.4 27.6
166.0 129.0 (84, 84, ~~62.0~~, 63.3)

SR = 126.0 net



275
180
455

540
60
6,00
455
6,455
6,500 ℄ of trusses

Approximate weight of truss

L0-U1	132.58	@ .785 × 6.21	=	645
U1-U3	132.58	× 9.50		990
U3-U4	169.72	× 4.75		632
L0-L2	84.00	× 9.50		625
L2-L4	166.00	× 9.50		1240
diag 3 -	76.	× 6.21	=	1110
vert. 3.5	25.38	× 4.00	=	278
				5520 × 2 = 11040

Detail 30%. 3300
14340

weight of one span

stringer	200 × 38.6	=	7720
floor beams	925 × 9	=	8330
lateral bracing			1744
trusses	2 @ 14340	=	28680
shoes say			1500
			47974 call this 48.0 tons

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Susai bashi for Okayama-ken.

structural Steel - 7 × 48. = 336 tons

Deck Construction.

Concrete. .13 × 5.4 = .70
coping .20
.90 × 270 = ~~297~~ 243 cubic meters

Pavement	5.4 × 270 = 1460 sq meters	Concrete	11 @ 6.00	66.0
Reinf.	1460 × 20 = 29.2 tons	Sand.		7
forms.	5.4 / 1.2 / 6.6 × 270 = 1780 sq meters.	gravel.		20
Handrails	100 × 270 = 27. tons	mixing.		9.3
				17
				110

Say 18.00 per cubic meter

Estimate of Cost

	Concrete	243	@	18.00	= 4370
	Reinf. bars	29.2	@	155	= 4520
Soliditit	pavement	1460	@	2.70	= 3940
	forms.	1780	@	1.80	= 3200
	Handrail	27. tons	@	280.00	= 7600
	lighting				1500
					25130.00

19.00
76.00
26.00

Grand summary of Cost.

structural Steel.	336	@ 260.00	= 87500.00
Deck. Complete			25.100.00
Piers 6 @ 4500			27000.00
abutments 2 @ 4500			9000
			148600

Call this. 150.000.00

昭和三年八月

岡山縣高梁川架橋

常盤橋予算設計書

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Tokiwa-Bashi, Okayama-Ken

General layout

From left bank. 5 spans @ 21.27 meters c to c of piers . = 106.35 meters

Truss spans — Cantilever type

anchor span	2 @ 38.0	=	76.0
Center span	1 @		57.0
			133.0 meters
3 fixed spans @ 38.0		=	114.0
5 spaces @ .560		=	2.8
			249.8 meters

On right bank 5 spans @ 21.27 meters c to c of piers = 106.35 meters

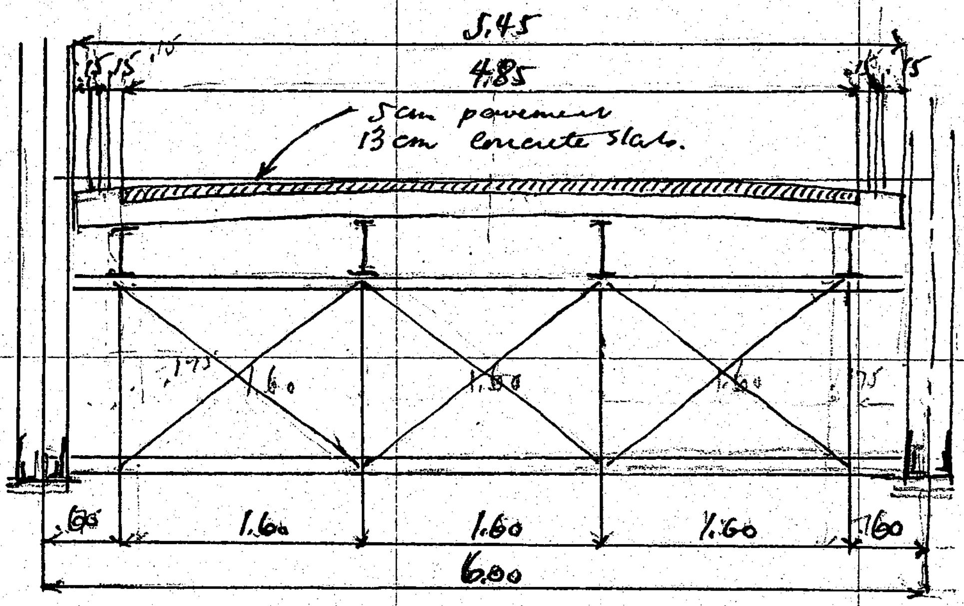
Summary

Girder spans	106.35
Truss spans	249.80
Girder spans	106.35
	462.5 meters between parapet walls of abutment

Design of 38.0 meter span 8 panels @ 4.75 = 38.0

Roadway 4.85 meters = 16.0 ft clear.

Cross section of bridge.



Design of slab. span length 1.60

Dead Load

Pavement 5 cm Soliditit or granolithic @ 22		=	110
Concrete slab 13 cm @ 24		=	312
misc fillet &c			20
			442 kg per m²
		call this	450

Dead Load moment $= \frac{1}{10} \times 450 \times 1.6^2 = 115$ kgm

Dead Load shear $= \frac{1}{2} \times 450 \times 1.6 = 360$ kg

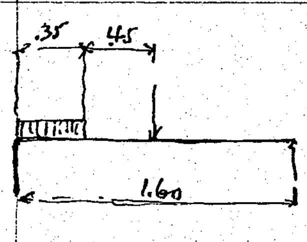
Live Load motor truck loading. Rear wheel with imp. = 2995

Front wheel - - = 975

Distribution Contact between wheel + pavement 20

distribution 2 @ 5.0 10

30



a --

Transverse distribution b 24 + 10 = 34

Effective width $E = \frac{2}{3}l + a = 1.37$ meter

Load per meter strip $= 2995 \div 1.37 = 2150$ kg

moment $= \frac{2150}{2} \times .8 = 860$

for continuity $860 \times 0.8 = 687$

uniform load $500 \times \frac{.35^2}{2 \times 1.6} = 19.2$ m $= 19.2 \times 0.8 = 15.$

687
.15
702 kgm

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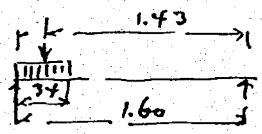
CALCULATIONS FOR

Preliminary Design and Estimate of Cost Tokiwa-Bashi Okayama-Ken.

Summary for moment.
Dead Load 115
Live Load 702
817

Summary for shear
Dead Load 360
Live Load 1920
~~1956~~
2280

End shear



$\frac{2150}{\cancel{2975}} \times \frac{1.43}{1.60} = 1920$

Effective depth $d = \sqrt{\frac{817 \times 100}{100 \times 7.18}} = 10.7$ cm
2.5
13.2

use 13cm slab insulation $2\frac{1}{2}$ cm depth 10.5

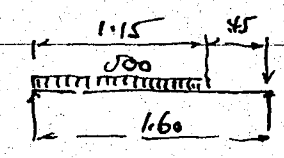
Steel $= \frac{81700}{7/8 \times 10.5 \times 1200} = \cancel{6.8}\ 7.4$ 13mm bars spacing $= \frac{1.33 \times 100}{7.40} = 18$ cm
use 15 cm spacing.

Bond stress 13mm bars 30 cm spacing top and bottom
" every other bars bent up and lapped.

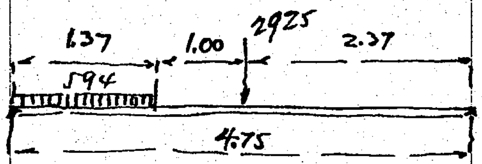
dia	Circumference			for meter strip
13	4.08	×	3.33 =	13.6
"	"	×	6.67 =	27.2
				40.8

Unit bond $= \frac{2280}{7/8 \times 10.5 \times 40.8} = 6.06$ kg/cm²

Stringer. span length 4.75 meter
Dead Load slab and pavement $450 \times 1.60 = 72.0$
stringer assumed 50
770 kg per lin. meter.
Dead Load moment $= 1/8 \times 770 \times 4.75^2 = 2170$ kgm
Live Load motor truck rear wheel with impact $= 2925$ kg
uniform load $\frac{500 \times 1.15^2}{2 \times 1.60} = 206$ kg per lin meter.



Full uniform load $500 \times 1.60 = 800$
$-\ 206$
594 kg per lin meter front or rear of truck



motor truck $\frac{2925}{2} \times 2.37 = 3470$ kgm.

Unif. load. $\frac{594 \times 1.37^2}{2 \times 4.75} \times 2.37 = 278$

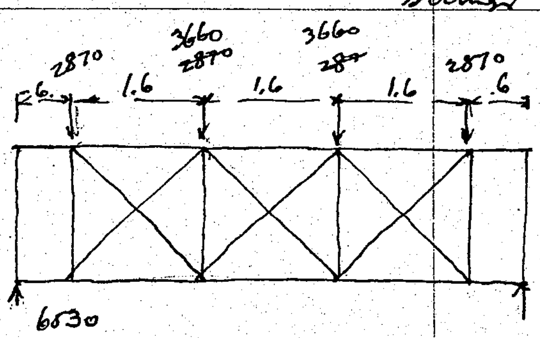
Unif. load - $1/8 \times 206 \times 4.75^2 = 581$
Live load moment 4329
2170
6499 kg meter

Section modulus reqd $= \frac{6499 \times 100}{1100} = 590.0$

use 300×150 I @ 48.34 kg Sm = 633.2

Cross beam span length ~~4.75~~ 6.00 meters. spacing 4.75 meters.
Dead Load $770 \times 4.75 = 3660$ kg

End panel. $450 \times 1.1 =$ say 500
stringer 50
$550 \times 4.75 = 2610$ kg.
Handrail - 250
2870



3660
2870
6530

moment $= 6530 \times 2.2 = 14400$
less $2870 \times 1.6 = 4600$
9800 kgm.

own weight 150 kg per meter assumed.
$150 \times 1.60 = 240$ kg.

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Tokiwa-Bashi Okayama-Ken.

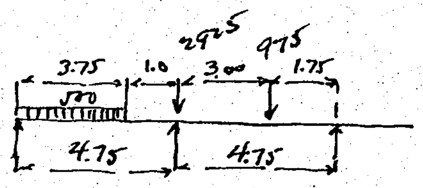
moment own weight

moment = 2 × 240 × 2.20 = 1060
240 × 1.60 = 380
680

9800
680
10480

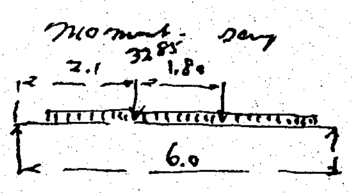
Live Load. motor truck loading

975 × $\frac{1.75}{4.75}$ = 360
2925
3285 kg.



Unif. load 500 × $\frac{3.75^2}{2 \times 4.75}$ = 740 kg. per lin. meter.

moment - say

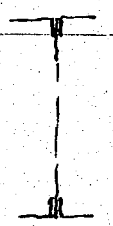


m = 3285 × 2.1 = 6900
Unif. say 1/8 × 740 × 6.0² = 3330
LL 10230
DL 10480
Summary 20710 kg. meters

Depth assumed 1.2 m chord stress = $\frac{20710}{1.2}$ = 17300 kg.

SR = 17.30 cm.

Try. 2Ls 100 × 75 × 10 @ 33.00 cm.



Diagonals shear - DL. 6830
480
7310 7310

LL unif. 740 × 2.42 = 1790
motor truck 2 × 3285 × $\frac{4.08}{6.00}$ = 4450
6240
13550 say 14000.

stress in diagonal 14000 × 1.41 say = 19800 kg. SR = $\frac{19.80}{2}$ = 9.90 cm

use 2Ls 75×75×9 = 12.69 no of Rivet

19800 ÷ 2126 = 9.3 5 Rivets for one angle. field.

Approximate weight

Top and bottom chord 4Ls 100 × 75 × 10 @ 12.95 × 6.0 = 310
verticals. 8Ls 75 × 75 × 9 @ 9.96 × 1.3 = 104
6Ls " @ " × 1.9 = 114
Connection plate - 4 Pls @ 15 kg = 60
2 Pls. 800 × 8 @ 50.2 × 1.2 = 121
misc. 30
739 - 740 kg

740 ÷ 6.0 = 123 kg per lin meter of cross beam

740 ÷ 4.75 = 156 kg per lin meter of span.

stringers. 4 @ 50.0 = 200. kg per lin meter of span.

Lower Lateral Bracings
Wind load 400
200
600 × 4.75 = 2850

diagonal length. 4.75² = 22.55
6.0² = 36.00
58.55 - 7.65²

sec θ = $\frac{7.65}{6.00}$ = 1.275 tan θ = $\frac{4.75}{6.00}$ = .79

Panel							Rivet	
0-1	$\frac{2850}{8}$ × 28	=	10000 × 1.275	=	12750		6.0	1L 125 × 75 × 10 = ~~14.91~~ 19.00
1-2	× 21	=	7500 "	=	9550		4.5	"
2-3	× 15	=	5350 "	=	6820		3.2	1L 75 × 75 × 9 = 12.69
3-4	× 10	=	3560 "	=	4540		2.1	"

Preliminary Design and Estimate of Cost Tokiwa-Bashi Okayama-Ken

Approximate weight of bottom lateral bracings.

4Ls	125×75×10	@ 14.91	× 7.3	=	435
4Ls	75×75×9	@ 9.96	× 7.3	=	290
					725
Center connections		4@30			120
					845×2 = 1690 kg.

1690 ÷ 38.0 = 44.5 kg say 45 kg
Panel load 45 × 4.75 = 214 kg

Design of truss span length 38 8@4.75

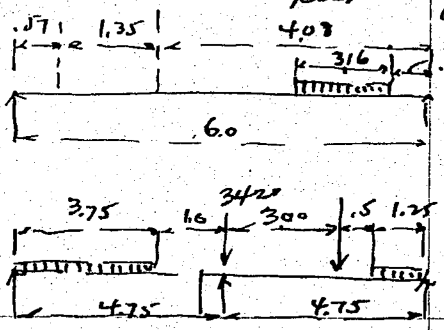
Dead Load. Floor and pavement 450 × 5.45 = 2450
Handrails 2 @ 70 = 120
2570

Structural steel. stringers 200
Floor beam 156
lateral Bracings 45
trusses 2@450 = 900
1301 — 1300
3870 ÷ 2 = 1935 kg per lin meter.

Live Load. $w = \frac{100.000}{170+380} = 480$ kg

motor truck loading. rear wheel ~~2925~~ kg 2250

Impact $\frac{20}{60+38}$ = 20.4% 260
2510 kg



$5020 \times \frac{4.08}{6.00} = 3420$ kg.
Front wheel 1140 kg

Unif. on side of motor truck $\frac{480 \times 3.16^2}{2 \times 6.00} = 400$ kg.

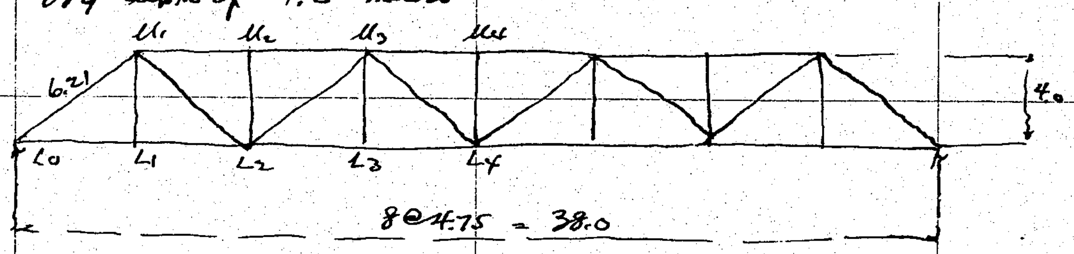
$480 \times \frac{3.75^2}{2 \times 4.75} = 710$

$480 \times \frac{1.25^2}{2 \times 4.75} = 80$
790

Full unif load. $480 \times \frac{4.85}{2} = 1160$
$1160 \times 4.75 = 5500$

400 × 4.75 = 1900
3420
$1140 \times \frac{1.75}{4.75} = 420$
6530
For motor truck 5500
1030 extra loading.

Try depth of 4.0 meters



$4^2 = 16.0$
$4.75^2 = 22.55$
38.55 = 6.21

$\sec\theta = \frac{6.21}{4.00} = 1.55$
$\tan\theta = \frac{4.75}{4.00} = 1.19$

19350 × 4.75 = 9200 kg.
9200 × secθ = 14250
9200 × tanθ = 10900

Dead Load stresses

					Diagonal				
L0-L2	10900	× 3.5	=	38100	L0-U1	14250 ×	3.5	=	50.000
U1-U3	"	× 6.0	=	65200	U1-L2		2.5	=	35600
L2-L4	"	× 7.5	=	81500	L2-U3		1.5	=	21400
U3-U4	"	× 8.0	=	87000	U3-L4		0.5	=	7100

Live Load stresses

Chord.

L0-L2	5500 × 1.19	× 3.5	=	23000	L0-U1	$\frac{5500}{8}$ × 1.55 ×	28	=	29800
U1-U3		× 6.0	=	39300	U1-L2		21	=	22400
L2-L4		× 7.5	=	49000	L2-U3		15	=	16000
U3-U4		× 8.0	=	52500	U3-L4		10	=	10700
					L4-L4'		6	=	6400

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Tokiwa-Bashi Okayama-Ken.

Concentration 1000 kg.

Chord stress

Lo-L2	1000 · 7/8 · 1.19 · 1.0	= 1040		Lo-U1	1000 · 7/8 · 1.55	= 1360
U1-U3	6/8 · 2.0	= 1780		U1-L2	6/8	= 1160
L2-L4	5/8 · 3.0	= 2220		L2-U3	5/8	= 970
U3-U4	4/8 · 4.0	= 2380		U3-L4	4/8	= 780
				L4-L4'	3/8	= 580

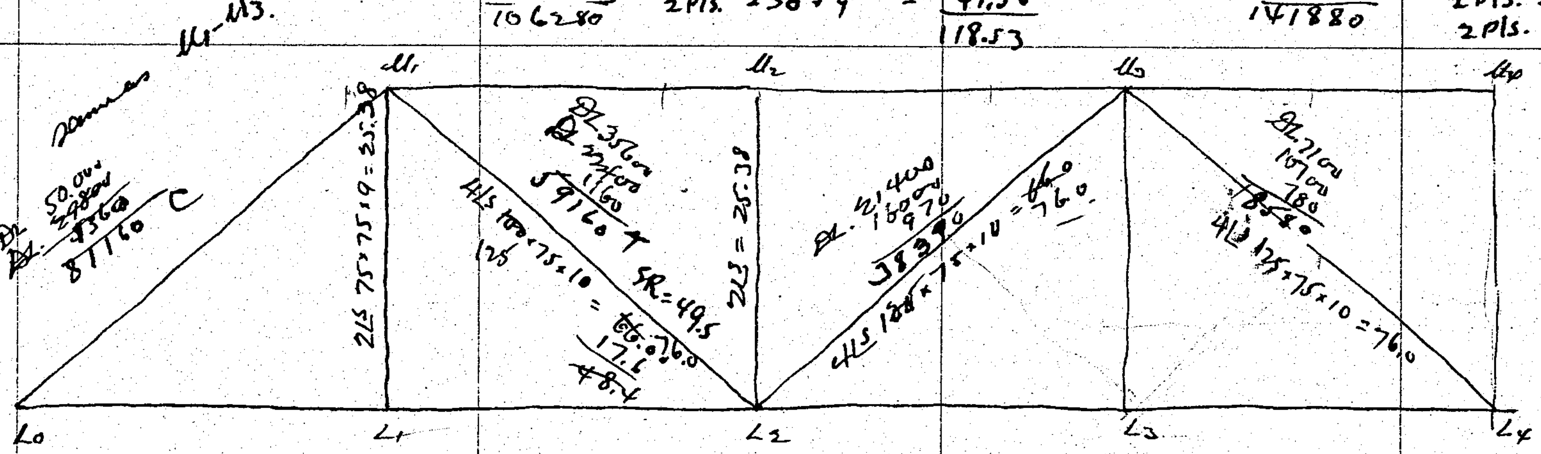
Diagonals

DL. 65300
LL 39300
1780
106280

1 Cov Pl. 390 × 9 = 35.05
2 Ls 150 × 90 × 9 = 41.58
2 Pls. 230 × 9 = 41.50
118.53

DL. 87000
52500
2380
141880

1 cov. Pl. 390 × 9 = 35.05
2 Ls 150 × 90 × 12 = 54.72
2 Pls. 230 × 9 = 41.50
2 Pls. 80 × 9 = 14.40
146.67



DL. 38100
LL 23000
1040
62140 T
SR = 52.0 net

2 Pls 230 × 9 = 41.50 − 11.8 = 32.7
2 Ls 90 × 90 × 10 = 34.0 − 8.8 = 25.2
75.50 57.9

DL 81500
49000
2220
132720
SR = 111.0 net

add 75.5 − 57.9
2 Pls. 140 × 10 = 28.0 − 4.4 = 23.6
2 Pls. 200 × 10 = 40.0 − 8.8 = 31.2
143.5 112.7

$$P = 1500\left(1 - 0.0005 \times \frac{621}{4.72}\right).$$

approximate weight of truss

Lo-U1	118.53	× .785	× 6.21	=	575
U1-U3	118.53	×	× 9.50	=	880
U3-U4	146.67	×	× 4.75	=	547
Lo-L2	75.50	×	× 9.50	=	565
L2-L4	143.50	×	× 9.50	=	1070
U1-L2 ... 3-	76.00	×	× 6.21	=	1110
vert. 3.5	25.38	×	× 4.00	=	278
					5025 × 2 = 10050

Details 30% 3000
13050

approx weight

stringer	200 × 38.6	=	7720
floor beams.	9 × 740	=	6660
lateral Bracing			1690
trusses	2 @ 13050	=	26100
shoes say			1500
			43670

say 45.0 tons

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Tokiwa-Bashi Okayama-Ken

Anchor and Cantilever Arm
Cantilever arm 2 @ 4.75 = 9.5 meters.

End reaction from ~~cantilever or~~ suspended span
Dead Load. 1935 × $\frac{38.6}{2}$ = 37400 kg.

Live Load say. 1160 × $\frac{38.6}{2}$ = 22400
59800 call this

Dead Load moment 37400 × 9.5 = 355.000 kgm.
$\frac{9200}{2}$ × 9.5 = 43700
9200 × 4.75 = 43700
87400 — 87400
442,400 kg m.

Live Load moment 22400 × 9.5 = 213000
1160 × $\frac{9.5^2}{2}$ = 52200
265,200
442.400
D + LL. 707.600 kg m

Depth = 5.0 stress = 707.600 ÷ 5.0 = 141,500
This stress same as max chord stress ~~in~~ center of simple span.

Top chord section. 14667 @ .785 = 114.
Bottom chord say 114
vertical. 25.38 @ .785 × $\frac{4.0}{4.75}$ = 17
Diagonals 760 @ .785 × $\frac{6.21}{4.75}$ = 78
323 kg
30%. 97
420 kg.

anchor span and cantilever arm 47.5
weight of truss 420 × 47.5 = 20.000
2 @ 20.000 = 40.000

Approximate weight of Anchor span and Cantilever arm
stringer 200 × 47.5 = 9500
floorbeams 11 × 740 = 8140
lateral bracings say 2200
trusses 2 @ 20.000 = 40.000
shoes + misc say 3000
62840 kg.

Summary for truss span

4 @ 45.0 tons = 1800
2 @ 63.0 = 126.0
306.0 tons.

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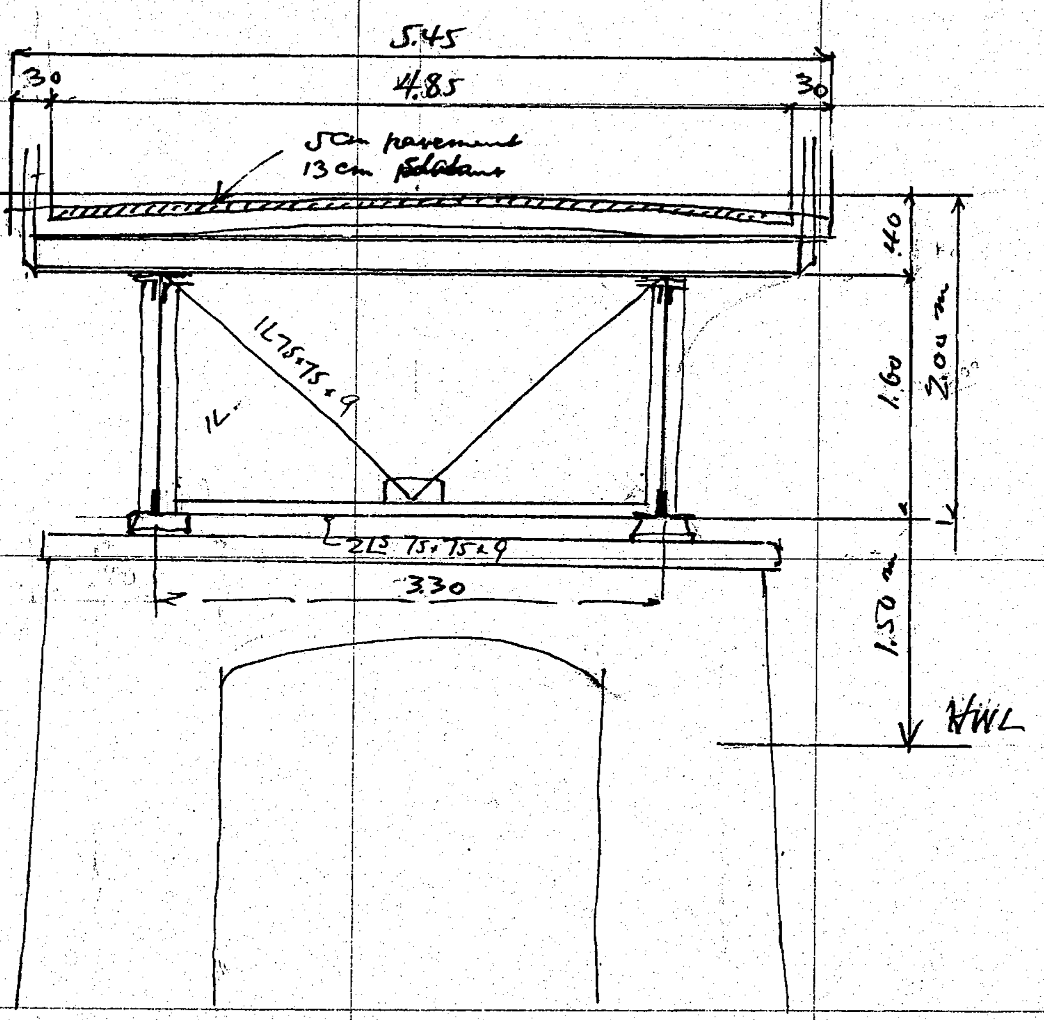
CALCULATIONS FOR

Preliminary Design and Estimate of Cost Tokiwa-Bashi, Okayama-Ken.

Deck truss span 21.25 out to out
.40
20.85 c to c of end bearings.

14 panels @ 1.49 meter Roadway 4.85 meters. pavement 2" Concrete.

Cross section of bridge



Dead Load
uniform load slab and pavement.
450 kg per sq. meter.

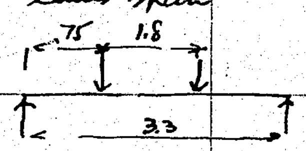
Cross beam spacing. 1.49 meters
Cantilever
Dead Load 450 × 1.49 = 670
Cantilever. 30
700 kg.

m = 700 × .925²/2 = 300
extra moment say 100
400 kgm.

Center span 1/8 × 700 × 3.3² = 950
less. 400
550 kgm

Live Load
Cantilever arm 2925 × .675 = 1975
DL. 400
2375

Center span



2925 × .75 = 2200
DLm 550
2750

Section mod. 27500/1100 = 250.

Use 230×100 @ 30.68 kg Sm = 291.0

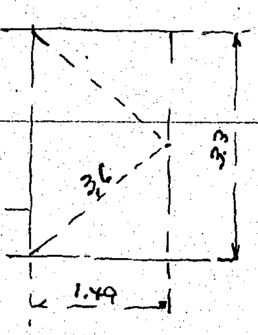
weight = 30.68 × 5.15 = 158/1.49 = 106 kg.

Transverse Framing.
2L 75×75×9 @ 9.96 × 2.1 = 42
2L " " × 3.3 = 66
misc. 5 @ 8 40
148

148/1.49 = 100 kg per lin meter.

Lateral Bracing. 1

3.3² = 10.9
1.49² = 2.22
13.12 3.6



2Ls 90×90×10 @ 13.34 × 3.6 = 96.0
3 @ 15 = 45.
141

141 ÷ 1.49 = 95. kg. including gusset plate

Cross beam 106
Transv. framing. 100
Lateral with conn 95
301 call this 300 kg per lin meter of span

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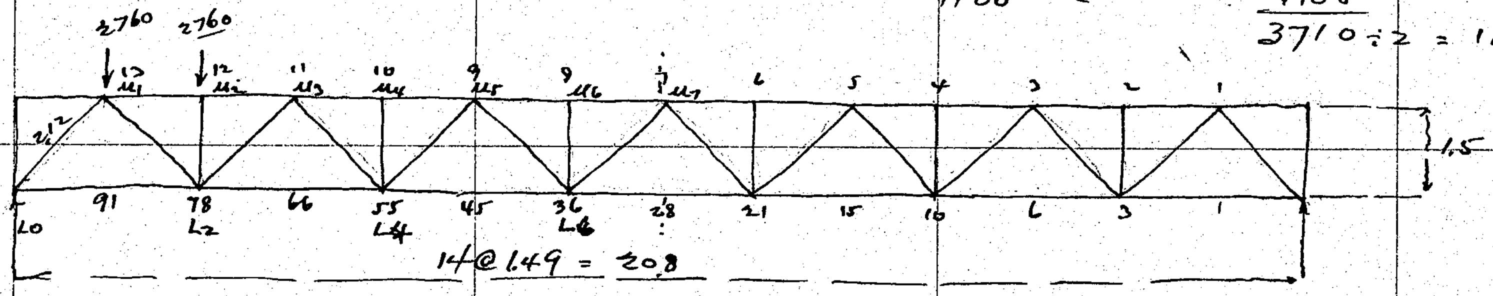
CALCULATIONS FOR

Preliminary Design and Estimate of Cost Tokiwa-Bashi Okayama-ken

Dead Load

Floor and pavement	450 × 5.45 =	2460	
Handrail		150	
			2610
Structural Steel stringers & lateral	300		
trusses 2@400 =	800		
	1100 —		1100
			3710 ÷ 2 = 1855 kg.



$\sec\theta = \frac{2.12}{1.5} = 1.41$

$\tan\theta = .995$

Panel load 1855 × 1.49 = 2760 R = 2760 × 6.5 = 18000

Chord stresses				Diagonal			
L0-L2	2760 × .995	× 6.5	= 17800	L0-U1	2760 × 1.41	× 6.5	= 25400
U1-U3		× 12.0	= 33000	U1-L2		× 5.5	= 21400
L2-L4		× 16.5	= 45200	L2-U3		× 4.5	= 17500
U3-U5		× 20.0	= 55000	U3-L4		× 3.5	= 13600
L4-L6		× 22.5	= 61600	L4-U5		× 2.5	= 9700
U5-U7		× 24.0	= 66000	U5-L6		× 1.5	= 5800
L6-L6'		× 24.5	= 67200	L6-U7		× 0.5	= 1950

Live load stress. say 500 × $\frac{5}{2}$ = 1250 including motor truck say 1500
Panel Conc. 1500 × 1.49 = say $\frac{2230}{2760}$ =

Chord stress		Diagonal			
L0-L2	= 14400	L0-U1	$\frac{2230}{14}$ × 1.41 ×	91	= 20400
U1-U3	= 26700	U1-L2		78	= 17500
L2-L4	= 36600	L2-U3		66	= 14800
U3-U5	= 44500	U3-L4		55	= 12350
L4-L6	= 49800	L4-U5		45	= 10100
U5-U7	= 53300	U5-L6		36	= 8100
L6-L6'	= 54300	L6-U7		28	= 6300

2L 150×150×11 = 63.58

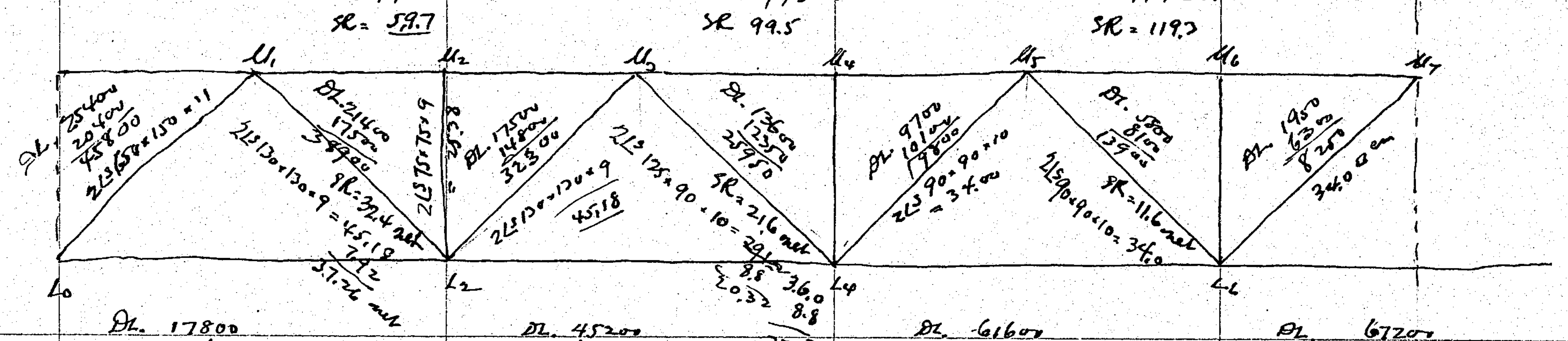
2L do — 63.58
1Pl. 350 × 11 = 38.50
102.08

2L do — 63.58
1Pl. 350 × 16 = 56.00
119.58

DL 33000
26700
59700
SR = 59.7

DL. 55000
44500
99500
SR 99.5

DL. 66000
53300
119300
SR = 119.3



DL. 17800
14400
32200
SR = 26.8 net

2L 150×150×11 = 63.58 −
9.70
53.88

DL. 45200
36600
81800
SR = 68.0 net

2L do — 63.58 — 53.88
1Pl. 350×9 — 31.50 — 27.54
81.42

DL. 61600
49800
111400
SR = 93.0 net

2L do 63.58 — 57.88
2Pls. 63.00 — 55.08
126.58 108.96

DL. 67200
54300
121500
SR 101.0 net
same as L4-L6

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CALCULATIONS FOR

Preliminary Design and Estimate of cost Tokiwa-Bashi Okayama-Ken.

Approximate weight of truss.

U0-U3		63.58	@ .785	× 3 × ~~4.75~~ 1.49	=	220
U3-U5		102.08	@	× 2 × ~~4.75~~ 1.49	=	304
U5-U7		119.58	@	× 2 × ~~4.75~~ 1.49	=	357
L0-L2		63.58	@	× 2 ~~9.5~~ 1.49	=	147
L2-L4		81.42	@	× ~~9.5~~	=	191
L4-L7		126.58	@	× 3 × ~~4.75~~ 1.49	=	565
L0-U1		63.58	@	× 2.12	=	105
U1-L2	2-	45.18	@	× 2.12	=	151
U3-L4	1-	~~29.12~~ 36.00	@	× 2.12	=	60
L4-U5	3-	34.00	@	× 2.12	=	170
vert.	4-	25.38	@	× 1.50	=	120

2390 × 2 = 4780

Details say 25%. 1200

5980 - call this 6000 kg.

Summary for structural steel in one span out to out = 21.25

stringers lateral &c	300 × 21.25	= 6360
trusses	2 @ 6.000	= 12000
shoes &c say		1000
		19.360 call this 19.5 tons.

10 Spans @ 19.5 = 195 tons.

Summary

Through trusses		3060 tons
deck spans	10 @ 19.5 =	195
		5010 tons

Alternate design of girder span.

Floor and bracings same as for deck truss.

Dead Load 1855
Live Load 1500
3355

$m = \frac{1}{8} \times 3355 \times 20.8^2 = 181{,}000$

Depth say 1500 × 9 = 135 1/8 web = 16.90 cm back to back 160.

Effective depth 160 - 10 = 150 flange = 181.000 ÷ 1.50 = 121.000

Section reqd 121.000 ÷ 1200 = 100.0
16.9
83.1 □ cm.

2Ls 150 × 150 × 11 = 63.58 - 9.70 = 53.88
1Pl. 350 × 10 = 35.00 - 4.4 = 30.60
84.48 □ cm.

Length of Cov. plate $\sqrt{\frac{30.6}{84.5}} + 0.5 = 12.5 + .5 = 13.0$ meters.

Approximate weight of main girder

	1 web.	1500 × 9	@	106.0	× 21.25	=	2250
	4Ls	150 × 150 × 11	@	24.95	× 21.25	=	2125
	2Pls.	350 × 10	@	27.40	× 13.00	=	712
Stiffs	32Ls	125 × 90 × 10	@	16.09	× 1.50	=	770
fills.	4Pls.	180 × 11	@	15.60	× 1.20	=	75
splices	2 @ 350					=	700
Sole plate							32
shoes	&c.						150
							6814
	Rivet heads &c	3½%.					240
							6754 kg. say 6.8 tons

(2250 + 2125 + 712 = 5087)

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Tokiwa Bashi Okayama-ken.

Approximate weight of Deck Girder span 21.25 out to out

Cross beam and bracings	300 × 21.25	=	6360
Girders.	2@ 6800	=	13600
			19960 say 20.0 tons.

Estimate of Steel.

306.0	@ 260.00	=	79500.00	250.00 =	76500
195	@ 220.00	=	430.00	220.00 =	43000
			122500		119500

Estimate of Cost of Deck Construction.

Concrete. 13.5 × 5.45 × 462.5 = 340 cubic meters

Reinforcing Bars. 45 tons.

Pavement = 4.85 × 462.5 = 2240 sq. meters

Forms say 2500 sq meters.

Handrail. toprail. 2" pipe 3.2754 5.6 kg per lin meter

2 L's 40×40×5 @ 2.94 = 5.88

$\frac{1.2}{1.5}$ kg × 8.00

Post. T 75, 15, 99 11.2, 13.5

24.7 × $\frac{1.2}{1.5}$ @ .785 15.50

34.48 call 40 kg. per lin meter

40 × 462 = 18.5 tons × 2 = 37.0 tons.

Cement	11.0 @ 550 =	60.50
Sand.		7.00
Gravel.		25.00
		92.50
		15.10 × 1.5 = 16.60

Estimate

Concrete	340	@	16.60	=	5650
bars	45 tons	@	155.00	=	7000
Pavement	2240	@	1.80	=	4030
Forms	2500	@	1.80	=	4500
Handrail.	37 tons	@	280.00	=	10400
					31580 call 31500.00

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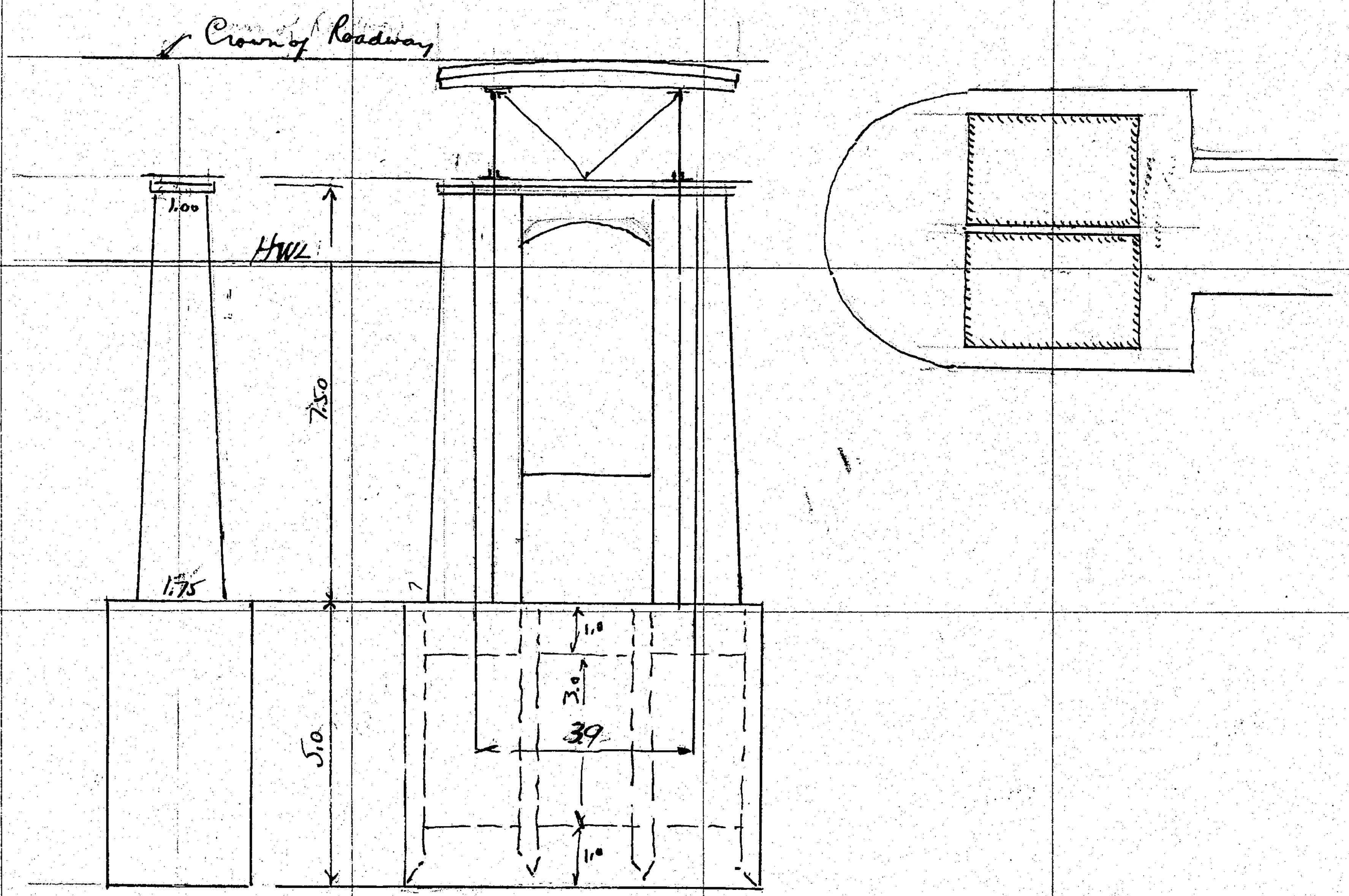
CALCULATIONS FOR

Preliminary Design ~~and~~ Estimate of Cost Tokiwa-Bashi Okayama-Ken
Piers for girder span



Approximate volume of concrete

shaft 1m .78 1.75 2.40
2-1.0×.8 = 1.60 2-1.75×.8 = 2.80
1.58 5.20 3.80
2.38 2.38 1.58
7.58 2.38 ÷ 2 = 3.79 × 7.5 = 28.4

web. 2.3 × .6 × 3.5 = 4.8
33.2 cubic meters.

Well. 2.5φ 4.90 1.9φ 2.83
(4.3)×2.5 9.90 (4.3)×1.9 = 8.16
3.9 14.80 outside area. 3.9 10.99
10.99.
say 3.80
Partition 3×1.9×2 = 1.14
4.94 × 5.0 = 24.7 cubic meters.

Top and bottom filling 11.00
1.14
9.86 × 2.0 = 19.7

Sand filling. 9.86 × 3.0 = 29.6

Own weight of pier. shaft. 33.2 @ 2200 kg = 73000
well shell 24.7 @ 2200 = 54400
filling 19.7 @ 2200 = 43400
sand fill 29.6 @ 1700 = 50200
221,000

Superimposed dead Load. 3710 kg × 21.25 = 79000 132.000
Live Load. 2500 × 21.25 = 53000 353.000 kg.
132000

Unit bearing 353.000 ÷ 14.80 = 23.8 tons / sq meter
2.2 tons / sq ft

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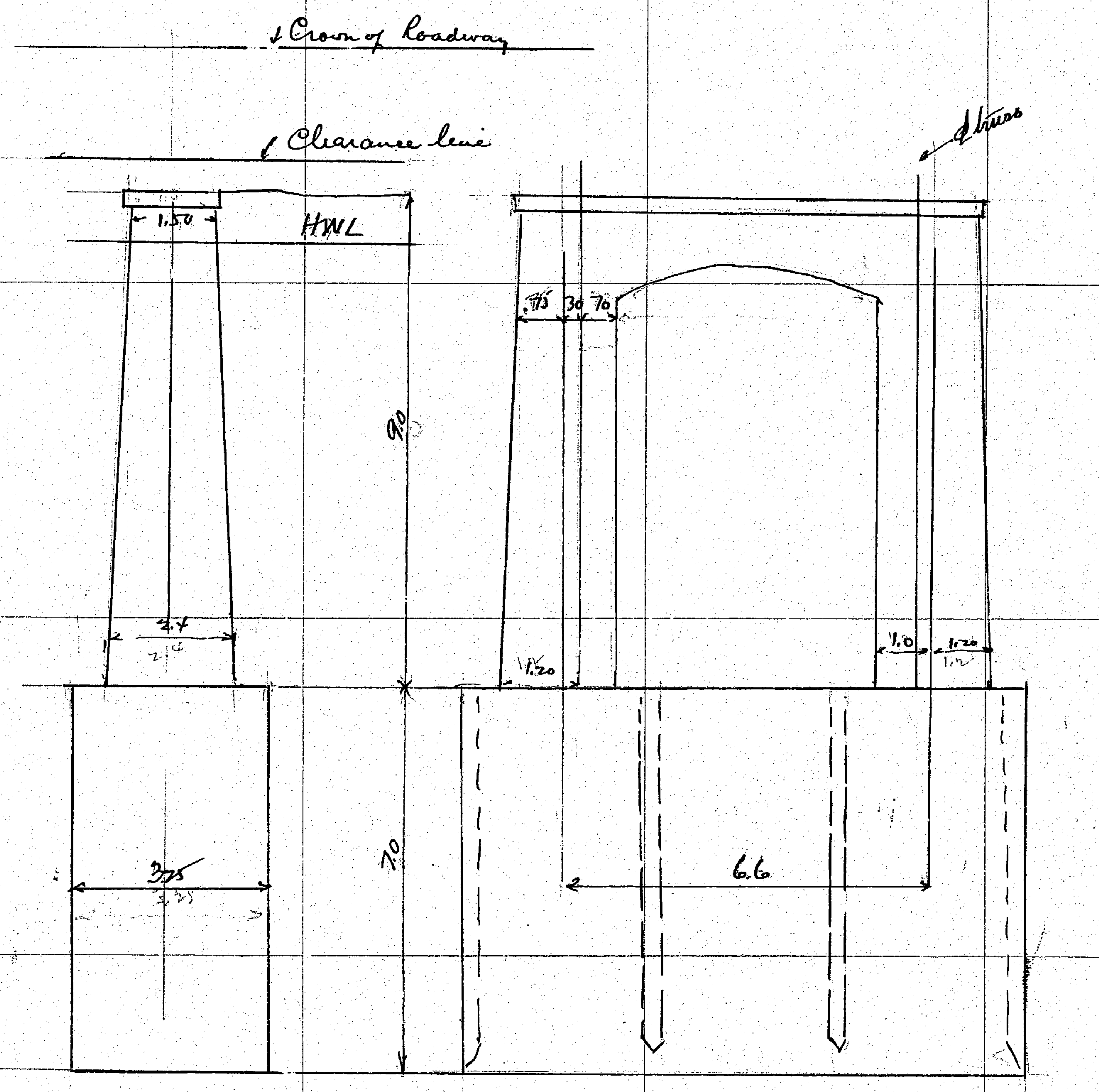
CALCULATIONS FOR

Preliminary Design and Estimate of cost Tokiwa-Bashi, Okayama-Ken

Approximate cost.

Concrete	1.2-4	77.6	@	16.60	=	1290
Sand fill		29.6	@	1.20	=	~~355~~ 35
Reinf. bars.		2.5 tons	@	130	=	325
Curb shoe		.8 tons	@	250.00	=	200
Forms.		250 sqm	@	1.50	=	376
Excavation.		74.0	@	3.50	=	260
						2486 - 2500.00
		misc expense say				200
						2700.00

Piers for truss span.



Approximate volume of concrete.

shaft 1.5^{ϕ} 1.76
$2.0 \times 1.5 =$ 3.00
4.76

Bottom 2.4^{ϕ} 4.53
$2 \times 2.4 =$ 4.80
9.33
4.76
$14.09 \div 2 = 7.05 \times 9.0 =$ 63.5
31.0
94.5 cubic meters

web. $1.5 \times 4.6 \times 1.5 =$ 10.35
$.6 \times 4.6 \times 7.5$ 20.70
31.05

well. outside area $3.25^{\phi} =$ 8.29
$3.25 \times 6.6 =$ 21.40
29.69
23.01
6.68
$2 - .30 \times 2.65 =$ 1.60
$8.28 \times 7.0 = 580$ cubic meters

inside area $2.65^{\phi} =$ 5.51
$2.65 \times 6.6 =$ 17.50
23.01

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost, Tokiwa-Bashi Okayama-Ken.

Top and bottom filling: 23.01 − 1.60 = 21.41 × 3.0 = 64.3 cubic meters

Sand filling: 21.41 × 4.0 = 86.0 cubic meters

Concrete	
shaft	94.5
well	58.0
filling	64.3
	216.8 cubic m

Own weight of piers: 216.8 @ 2200 = 477 000
Sand: 86.0 @ 1700 = 146 000
623000 kg

Superimposed Dead Load: 3870 × 38.0 = 147 000
Live Load: 2500 × 38.0 = 95000
242000
865,000 kg

Unit bearing: 865 ÷ 29.64 = 29.2 tons / sq m
2.7 tons / □'

Approximate Cost

Concrete	216.8 cubic meters	@	16.60	=	3600
Sand filling	86.0 " "	@	$1^{\underline{20}}$	=	103
Reinforcing bars	3.5 tons	@	$130^{\underline{00}}$	=	455
Curb shoe	1.6	@	250	=	250
forms	380	@	$1^{\underline{50}}$	=	570
Excavation	210	@	$4^{\underline{50}}$	=	945
					5923

$6000^{\underline{00}}$

Approximate Cost.

Girder piers	8 @ $2700^{\underline{00}}$	=	$21600^{\underline{00}}$
abutments pairs	2 @ 3500	=	7000
truss piers	5 @ 6000	=	30 000
" "	2 @ 7500	=	15 000
			$73600^{\underline{00}}$

Total Cost of bridge Layout No 1.

structural steel	119500
Deck Construction	31500
Substructure	73600
	$224600^{\underline{00}}$
	177 000
	47 600

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost - Tokiwa-Bashi Okayama-Ken

Layout no 2. Over stream 3 spans.
Anchor spans 2 @ 38.0 = 76.0
Center span 57.0
133.0
2 spaces @ .56 1.12
134.12

Out to out 462.5 meters
134.12
328.38 meters
15 spans @ 21.8 meters = 328.38

5 spans on left bank.
10 spans on right bank.

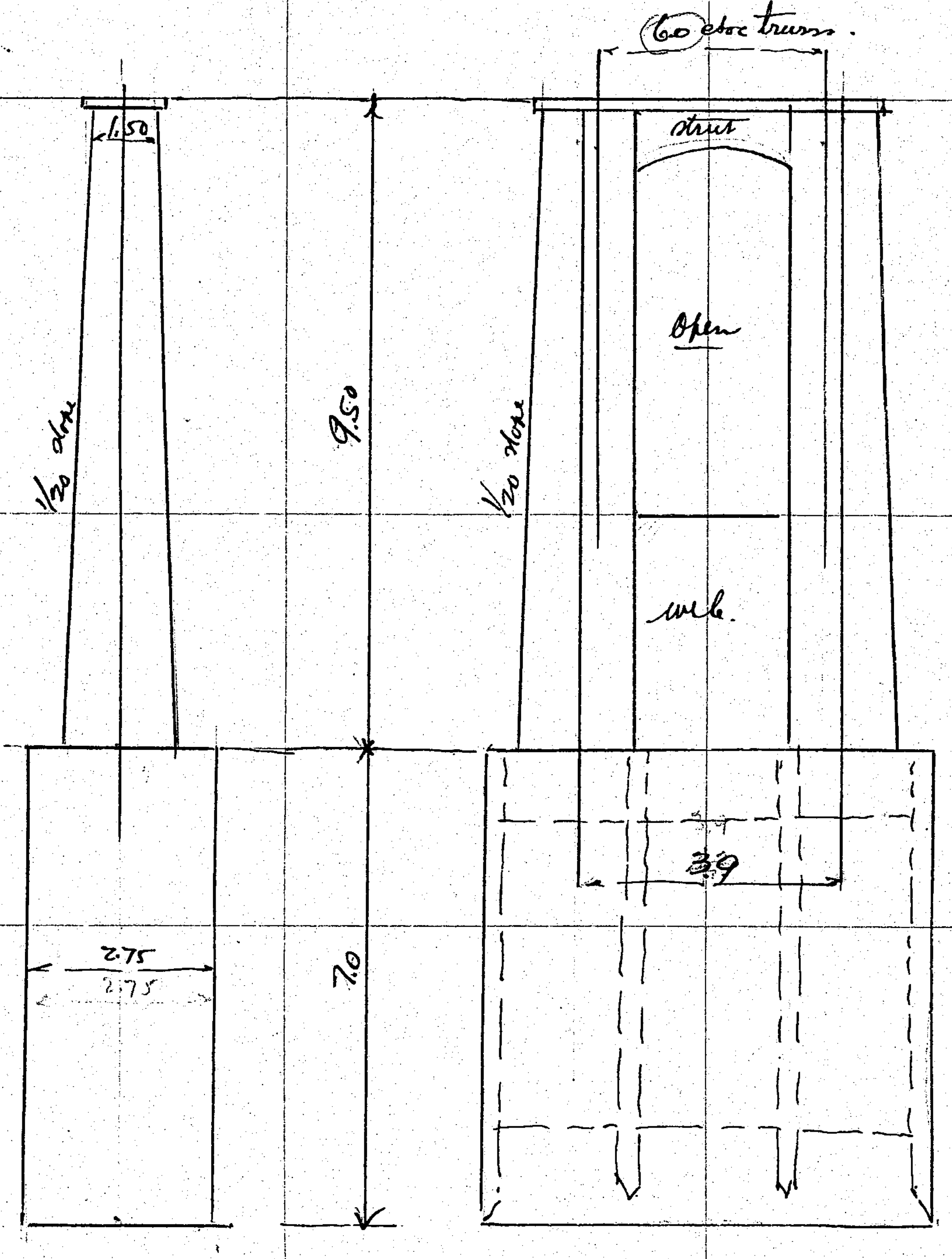
Truss spans.
Anchor spans 2 @ 63.0 tons = 123.00
Suspended span 45.00
168 tons.

Girder span say 20 tons.
15 spans @ 20.00 300
468 tons.

Estimate of Cost 168 @ 260.00 = 43600.00
300 @ 220.00 = 66000
109600.00
Deck Construction 31500.00

Substructure 7.0 meter well for girder span.
for sketch see page 11

Shaft.



4.8

3.3
6
39

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CALCULATIONS FOR

Preliminary Design and Estimate of Cost Tokiwa-Bashi Okayama-Ken.

Approximate volume of Concrete

shafts 1.0 m — .78 1.95 ϕ — 2.98
2 - 1.0 × 0.8 = 1.60 ~~2.98~~ 2 - 1.95 × .8 = 3.12
2.38 6.10
2.38
8.48 ÷ 2 = 4.24 × 9.5 = 40.3 cubic meters

web. 0.6 × 2.3 × 4.5 = 6.2
46.5 "

well. 2.75 ϕ — 5.94 2.15 3.63
3.9 × 2.75 = 10.70 3.9 × 2.15 = 8.40
16.64 outside 12.03 inside
12.03
4.61
Partition .3 × 2 × 2.15 = 1.30
5.91 × 7.0 = 41.5 cubic meters

Top and bottom filling 12.03
1.30
13.33 × 3. = 40.0 " "

Sand filling 13.33 × 4 = 53.3 " "

Concrete
shafts 46.5
well 41.5
filling 40.0
128.0

Own weight 128.00 @ 2200 = 282.000
53.3 @ 1700 = 90500
372.500
Superimposed load 132.000
504.500 t

Unit bearing 504.5 ÷ 16.64 = 30.3 tons / m²
2.80 tons / ft²

Approximate Cost of one pier.

Concrete	128.0 cubic meters	@ 16.60	=	2120
Sand fill	53.3	@ 1.20	=	
Reinforcing bars	3.00 tons	@ 130.—	=	64
				390.
Curb shoe	1.0 ton	@ 250	=	250
forms.				450
Excavation.	117. cubic meters	@ 4.00		470
				3744 call this 4000 t

Summary for substructure

2 abutments	@ 3500	=	7000
8 piers	@ 2700	=	21600
5 piers	@ 4000	=	20000
2 piers	@ 6000	=	12000
2 piers	@ 7500	=	15000
			75600 t

54300
6
60.0

15000

Total Cost

Structural Steel	109600
Deck Construction	31500
Substructure	75600
	216700

255 × 2.67 = 680 坪 216700 ÷ 680 = 320

216700
177000
40300

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