JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

CALCULATIONS FOR

MADE BY ______ DATE ______ FILE NO ______
CHECKED BY ______ DATE ______ PAGE NO ______

昭和四年九月

岡山縣中川橋應力計算書

附材料計算書

JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

MADE BY J. Masuda DATE 4-7-22 FILE NO
CHECKED BY M Jinda DATE 4-9-17 PAGE NO 1

CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama-Ken

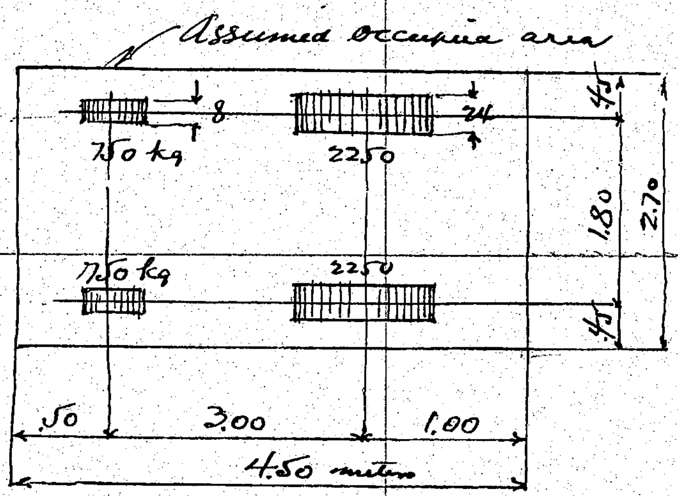
Total length of bridge 134.5 meters about between faces of parapet walls on both abutment. On right bank over stream 2 girder spans of 21.0 meters each out to out; the rest of bridge will be concrete span of 10.3 meter about. 9 spans of 3 span continuous girder.
Clear roadway 5.4 meters between curb lines, pavement 5 cm granolithic pavement on reinforced concrete slab.

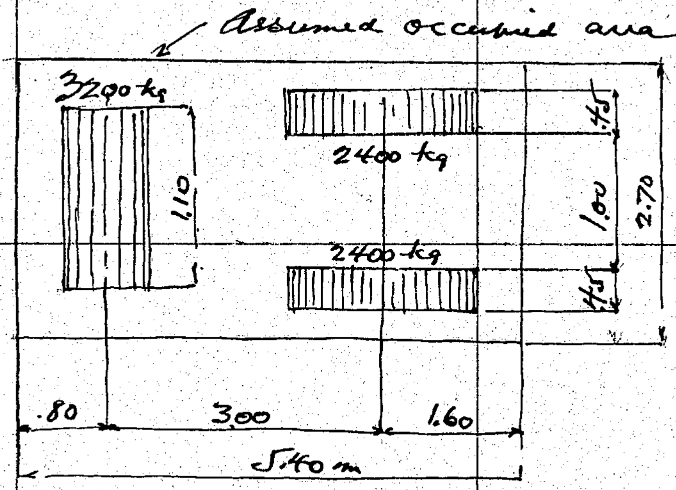
Assumed loadings

$$\text{Uniform load } w = \frac{100.000}{170+l} \leq 500 \text{ kg/m}^2$$

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

6 ton motor truck loading. 8 ton road roller.





2 rows of motor traffic on roadway with occupied width of 270 cm each; Unoccupied space around the motor truck shall be filled with uniform load specified above.
One road roller on one span.
Impact for motor truck loading

$$\text{Coef} = \frac{20}{60+l}$$ where l = loaded length in meter
max impact 30%

No impact for road roller and uniform load

Allowable Working Strength
Reinforcing Steel or structural Steel

Tension net	1200 kg/cm²
Extreme fibre stress net	1200
Shear of web gross section	900
Compression member	
$1500\,(1-0.0050\ l/r)$ not over	1000
where l = length of member in cm	
r = least radius of gyration	
Compression flange of girder	
$1200\,(1-0.012\ l/b)$ = not over	1100
where l = Unsupported length of flange in cm	
b = width of flange in cm	
Shear on shop driven rivets (machine driven)	850
" " field " and turned bolts (machine driven)	750
Shear on pin	900
Bearing on shop driven rivets (machine)	1700
" " field " "	1500
" " pin " "	1800

Rollers 45d kg where d = diameter of roller in cm

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MADE BY J. Masuda DATE 4-7-22 **FILE NO**

CHECKED BY M. Jinda DATE 4-9-17 **PAGE NO** 2

CALCULATIONS FOR

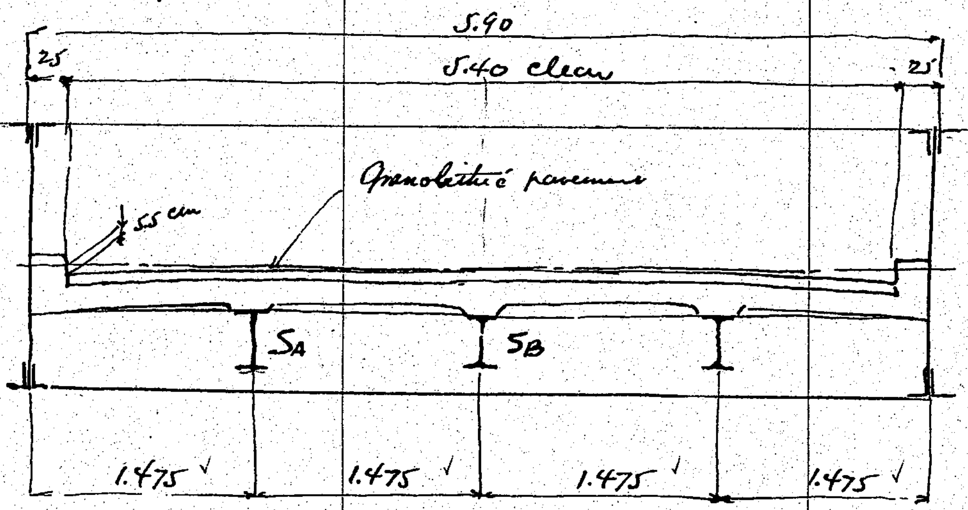
Design of Nakagawa-Bashi for Okayama-ken.

Allowable working strength
Concrete 1:2:4 mixture

Direct compression	35	kg/cm²
Fibre stress due to bending	45	"
Combined stress direct and bending	35	"
Punching shear of Concrete	9	"
Shear of plain Concrete	4	"
Bearing	45	"
Bond stress for plain bars	6	"
" " " deformed bars assumed	9	"

Considering wind or temperature stress in addition to dead live and impact stresses the allowable working strength shall be increased 25%. In case of Earthquake increase unit stress 80%
Seismic accelleration 1000 mm/sec² $k = 0.10$

Cross section of girder span.



Design of floor slab span length 1.475 meters
Dead Load

Pavement 5 cm Granolithic @ 22 kg	=	110
Concrete slab 13 cm @ 24 kg		312
Add for misc concrete		18
		440 kg/m²

Dead load moment $= \frac{1}{10} \times 440 \times 1.475^2 = 96.0$ kgm
shear $= \frac{1}{2} \times 440 \times 1.475 = 325.0$ kg.

Live load motor truck loading

Rear wheel	2250
impact 30%	675
	2925 kg

Front wheel $2925 \div 3 = 975$.

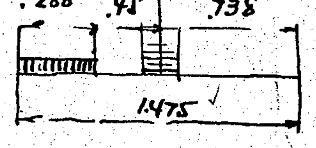
Distribution of wheel Concentration

Contact between wheel and pavement		20
distribution	2 × 5	10
Longitudinal distribution	a =	30 cm
Transverse "	b = 24 + 10 =	34 cm

Effective width $E = \frac{2}{3} l + a$ where l = span length in meter
$= 1.285$

Load per meter strip $2925 \div 1.285 = 2280$ kg.

Uniform live load 500 kg per sq. meter
Uniform load $\frac{500 \times .288^2}{2 \times 1.475} = 14.0$ kg.



Design of Nakagawa-Bashi for Okayama-ken

moment at center of span

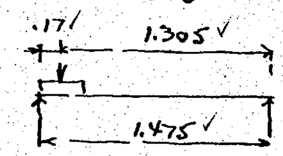
Due to unif. load $14.0 \times .738 = 10.3$

" " motor truck $\frac{2280}{2} \times .738 = 841.0$

851.3 kgm

For continuity of slab. $0.8 \times 851.3 = 680$ kgm

End shear



$2280 \times \frac{1.305}{1.475} = 2020$ kg.

Summary for moments and shears

	moment	shear
Dead Load	96	325
Live load	680	2020
	776 kgm	2345 kg.

Effective depth req'd for $f_s = 1200$ kg/cm² and $f_c = 45$ kg/cm²

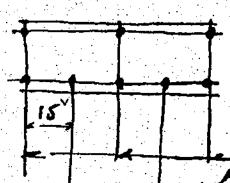
$R = \frac{M}{bd^2}$ $d = \sqrt{\frac{M}{bR}}$ $R = 7.18$ $d = \sqrt{\frac{776 \times 100}{100 \times 7.18}} = 10.4$ cm use 13 cm slab

insulation at bottom 2.6 cm meter

Steel area req'd $= \frac{776 \times 100}{7/8 \times 10.4 \times 1200} = 7.11$ cm² per meter strip.

13 mm bar spacing $\frac{1.33 \times 100}{7.11} = 18.7$ cm use 15 cm spacing

13 mm bars 30 cm spacing for top and bottom bottom bars 15 cm spacing every other bars to be bent up at support.



	dia	circumference		
S_1	13	4.08 ×	3.33 =	13.6
S_2	13	4.08	"	13.6
				27.2

without lap

Unit bond $= \frac{2345}{7/8 \times 10.4 \times 27.2} = 9.46$

Required extra reinforcements for bond stress.

Design of I beam Stringer SA. span length. 5.0 meters spacing 1.475 meters

Dead Load floor slab and pavement $440 \times 1.475 = 648$

beam assumed 50

698 kg.

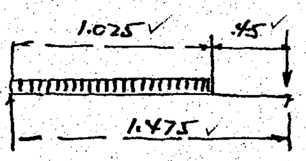
Dead Load moment $= 1/8 \times 698 \times 5.0^2 = 2180$ kgm

Dead Load shear $= 1/2 \times 698 \times 5.0 = 1750$ kg.

Live Load motor truck rear wheel with impact 2925 kg.

Uniform live load 500 kg per square meter

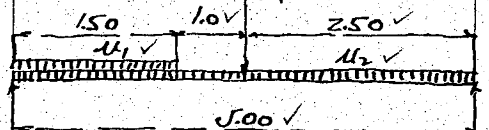
Load on stringer $\frac{500 \times 1.025 \times .512}{1.475} = 178$ u_2



full unif. load $500 \times 1.475 = 737$

559 u_1

u_1 Reaction $559 \times \frac{1.50 \times .75}{5.00} = 126.0$ kg.



Moment

Due to motor truck $\frac{2925}{2} \times 2.50 = 3660$

" " Unif. u_2 $1/8 \times 178 \times 5.0^2 = 557$

" " " u_1 $126 \times 2.50 = 315$

4532 kgm

End shear u_1 $559 \times \frac{4.0 \times 2.0}{5.00} = 895$

u_2 $178 \times \frac{5.0}{2} = 445$

1340

motor truck loading 2925

4265 kg.

Design of Nakagawa-Bashi for Okayama-Ken

Summary for moments and shears

	moment	shear
Dead Load	2180	1750
Live Load	4532	4265
	6712 kgm	6015 kg.

Section modulus req'd = $\frac{671200}{1100}$ = 610.0

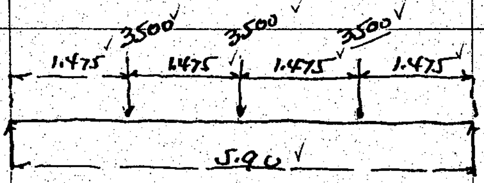
use 300 × 150 I @ 48.34 kg Sm = 633.2

Unit Stress = $\frac{671200}{633.2}$ = 1060 kg/cm²

Stringer SB. See page 7

Design of intermediate Floor Beam span length 5.90 m spacing 5.0 meters

Dead Load



Concentration on stringer 698 × 5.0 = 3500 kg.

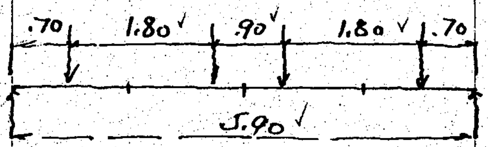
Dead Load beam assumed 140 kg.

moment 3500 × 1.5 × 2 × 1.475 = 15500
3500 × 1.475 = 5150
10350
1/8 · 140 · 5.90² = 609
10959 kgm

$\frac{140}{2}$ × 5.90 = 413 kg.

End shear 3500 × 1.5 = 5250
413
5663 kg.

Live Load motor truck loading rear wheel concentration with impact = 2925 kg
front " " " = 975 "



motor truck loading
Front wheel 975 × $\frac{2}{5}$ = 390
rear wheel 2925
3315

moment due to motor truck
2 × 3315 × 2.5 = 16600
3315 × 1.8 = 5970
10630 kgm

End shear
3315 × 2 = 6630
1180 × 1.5 = 1770
8400 kg

Unif load 500 kg/m² 500 × 1.475 = 737

load on floor beam 737 × $\frac{4.0^2}{2 \times 5}$ = 1180

moment 1180 × 1.5 × 2 × 1.475 = 5220
1180 × 1.475 = 1740
3480

Summary for moments and shears

	moment	shear
Dead Load	10959	5663
Live Load	10630	8400
	21589 kgm	14063 kg

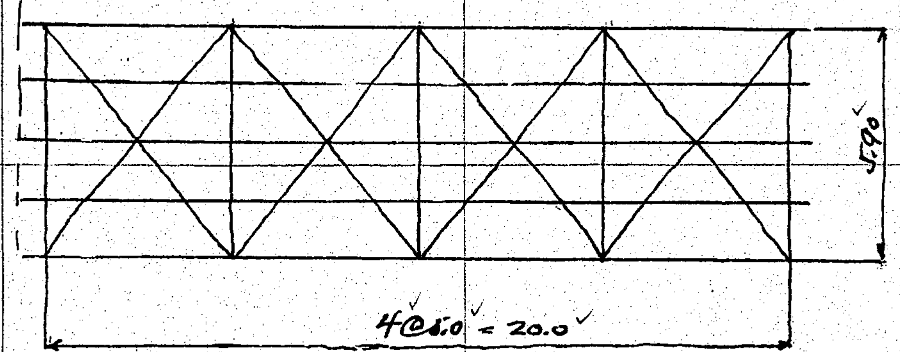
section modulus req'd = $\frac{2158900}{1100}$ = 1960

use 450 × 175 I @ 114.68 kg Sm = 2169

Unit stress = $\frac{2158900}{2169}$ = 995 kg/cm²

End floor beam use same as for intermediate floor beam

Lateral Bracing. Wind load 400 kg per lin. meter Concentration = 400 × 5 = 2000 kg.



Diagonal length 5.0² = 25.0
5.9² = 34.81
59.81 — 7.73

sec θ = $\frac{7.73}{5.90}$ = 1.31

End panel shear = 3000
stress = 3000 × 1.31 = 3930 kg.

use 3 - 19 mm rivets each connection.

use 1 L 75×75×9 = 12.69 cm² r = 1.44 — 1.49
r = 2.25

l/r = $\frac{387}{2.25}$ = 172

l/r = $\frac{194}{1.44}$ = 135 — 135.
144

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CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama-Ken.

Approximate weight of bottom lateral bracings.
for one panel

2L 75×75×9 @ 9.96 × 7.2 =	143.5	143.5
Center connection	10.0	
other misc details say	12.0	
	165.5 call this 166 kg.	165.5 166.

4 × 166 = 664 kg.

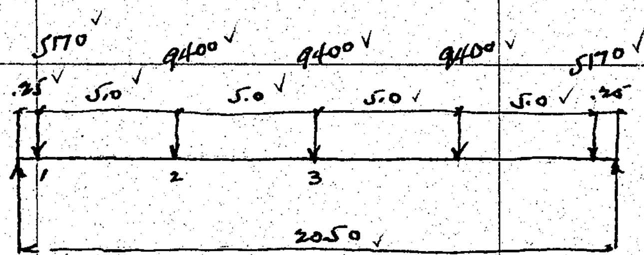
166 ÷ 5 = say 33 kg per lin. meter. 33

Design of main girder

Dead Load. Floor slab and pavement 440 × 5.4 = 2380
at curb. 2 × .25 × .33 @ 2400 = 396
2776

structural Steel 992 — 992

stringers 3 @ 52 =	156	3768 ÷ 2 = 1884 kg.
cross beam	153	call this 1880 kg per lin. meter.
Lateral Bracing	33 33	3768
Girders assumed	650	
	992 — 992	



Concentration = 1880 × 5 = 9400
at 1. 9400 ÷ 2 = 4700
1880 × .25 = 470
5170 kg

Reaction 9400 × 1.5 = 14100
5170
19270

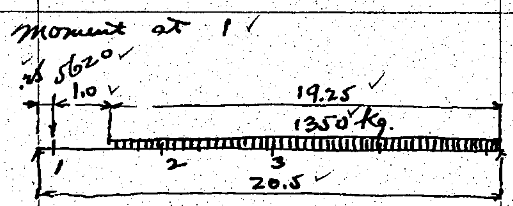
moment at 1 19270 × 0.25 = 4810 kgm

moment at 2 19270 × 5.25 = 101000
5170 × 5.00 = −25850
75150 kgm

moment at 3 19270 × 10.25 = 197500
9400 × 5.00 = −47000
5170 × 10.00 = −51700
98800 kgm

Live Load motor truck loading impact $\frac{20}{60+20.5}$ = 24.9 % rear wheel 2250
impact 24.9% 560
2810
front wheel 2810 ÷ 3 = 938

moment at 1

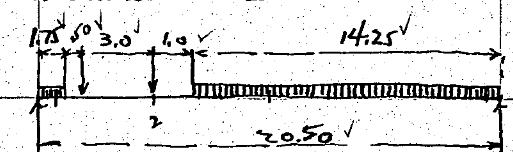


$1350 \times \frac{19.25^2}{2 \times 20.5} = 12200$ kg.

Uniform live load 500 kg per sq meter.
500 × 2.70 = 1350 kg per meter

moment at 1. motor truck $5620 \times \frac{20.25}{20.50} \times 0.25$ = 1390
Unif. 12200 × 0.25 = 3050
4440 kgm

moment at 2



motor truck reaction 5620 × 5.25 = 29500
1876 × 2.25 = 4220
33720 ÷ 20.50 = 1640.

moment at 2 1640 × 15.25 = 25000 kgm

1350 × 14.25 = 19200
1350 × 1.75 = 2360

Unif. load Reaction $2360 \times \frac{.87}{20.50}$ = 100
$19200 \times \frac{13.38}{20.50}$ = 12500
12600

moment 12600 × 15.25 = 192000
19200 × 8.12 = −156000
36000 kgm

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Design of Nakagawa Bashi for Okayama-Ken.

moment at 2 — motor truck 25000
Unif. 36000
61000 kgm

moment at center of span.

6.75 | .5 | 3.0 | 1.0 | 9.25
20.50

motor truck reaction $1878 \times \frac{7.25}{20.50} = 663$
2810
3473 kg.

moment = $3473 \times 10.25 = 35600$

$1350 \times 9.25 = 12500$
$1350 \times 6.75 = 9100$

Unif. load reaction $9100 \times \frac{3.38}{20.50} = 1500$
$12500 \times \frac{15.88}{20.50} = 9680$
11180

moment = $11180 \times 10.25 = 114700$
$12500 \times 5.62 = -70200$
44500 kgm

motor truck 35600
Unif. load 44500
80100 kgm

max End load. motor truck on ℄ bearing.
uniform load. $1350 \times \frac{19.50^2}{2 \times 20.50} = 12500$
motor truck 5620
18120 kg.

Summary for moments and shears

	moment 1	moment 2	moment cent.	shear
Dead Load	4810	75150	98800	19270
Live Load	4440	61000	80100	18120
	9250	136150	178900 kgm	37390 kg.

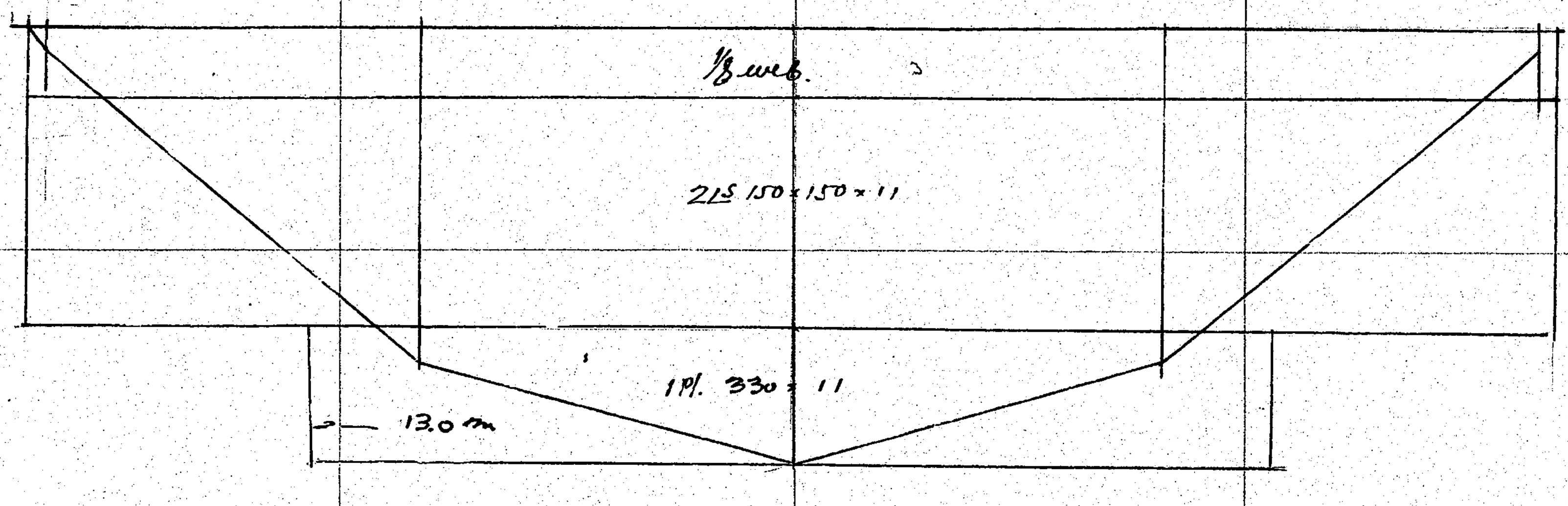
Section of main girder
web $1500 \times 9 = 135.0$ □cm   1/8 web = 16.90   back to back of Ls 1510 m/m

assumed section   2Ls $150 \times 150 \times 11$ = 63.58 × 4.07 = 258.50
1Pl. 330×11 = 36.30 × 0.55 = − 20.00
99.88   2.4   238.50

Effective depth = $1.510 - .048 = 1.462$
flange stress = $178900 \div 1.462 = 122100$ kg
section reqd = $122100 \div 1200 = 101.80$
− 16.90
84.90 □cm net

2Ls $150 \times 150 \times 11$ = $63.58 - 9.7 = 53.88$
1Pl 330×11 = $36.30 - 4.85 = 31.45$
85.33 □cm net

Approximate length of cover plate



Design of Nakagawa-Bashi for Okayama-ken

Approximate weight of main girder.

web	1 Pl.	1500 × 9 @	105.98	× 21.00	=	2220
flange	4 Ls	150×150×11 @	24.95	× 21.00	=	2100
	2 Pls.	330 × 11 @	28.50	× 13.00	=	740
Stiffs.	32 Ls	125×90×10 @	16.09	× 1.50	=	770
End stiffs.	4 Ls	150×150×11 @	24.95	× 1.50	=	150
" "	8 Ls	125×90×10 @	16.09	× 1.50	=	193
fills.	4 Pls.	180 × 11 @	15.60	× 1.20	=	75
"	5 Pls	90 × 11 @	7.77	× 1.20	=	47
						6295

Splices shoes & Rivet Heads &c say 750
7045 kg.

7045 ÷ 21 = 335 kg per lin. meter.

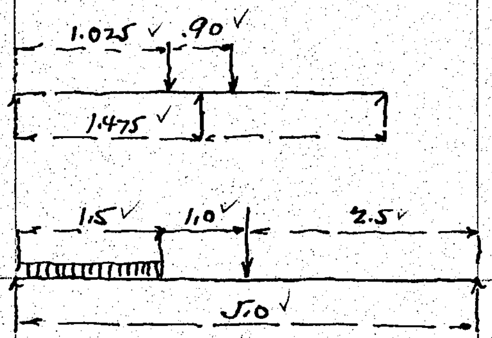
Approximate weight of one span

Stringers.	156 × 21.0	=	3280
Cross beams	5 × 760	=	3800
Lateral Bracings with connection say			600
main girders.	2 × 7045	=	14090
misc steel say			500
			22270 kg.

For 2 spans 2 @ 22270 = 44540 kg.

Design of stringer S.B. at center span length 5.0 m spacing 1.475 m
Dead Load floor slab and pavement 440 × 1.475 = 648
beam assumed 70
718 kg

Dead load moment = 1/8 × ~~648~~ 718 × 5.0² = 2240 kgm
Dead load shear = 1/2 × 718 × 5.0 = 1800 kg.
Live Load motor truck loading rear wheel with impact 2925 kg.
Uniform live load 500 kg per square meter



2925 × 1.025/1.475 = 2035
2035
4070 kg.

Unif. load 500 × 1.475 = 737 kg per lin. meter
Reaction = 737 × 1.5² / (2 × 5.0) = 166.0 kg.

Moment
due to motor truck 4070/2 × 2.5 = 5080
" " unif. load 166 × 2.5 = 415
5495

End shear Unif load 737 × 4² / (2 × 5.0) = 1180
motor truck 4070
5250 kg.

Summary for moments and shears

	moment	shear
Dead Load	2240	1800
Live Load	5495	5250
	7735	7050

Section modulus req'd = 773500 / 1100 = 702.0

12" × 6" @ 44.02# or 19.967 kg per ft.
Section modulus = 52.573 (in)³ 860 (cm)³

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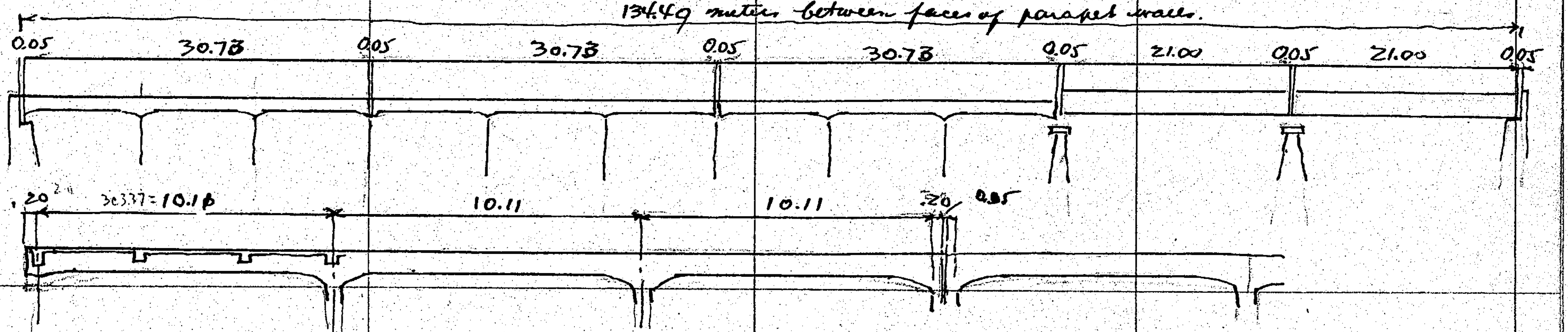
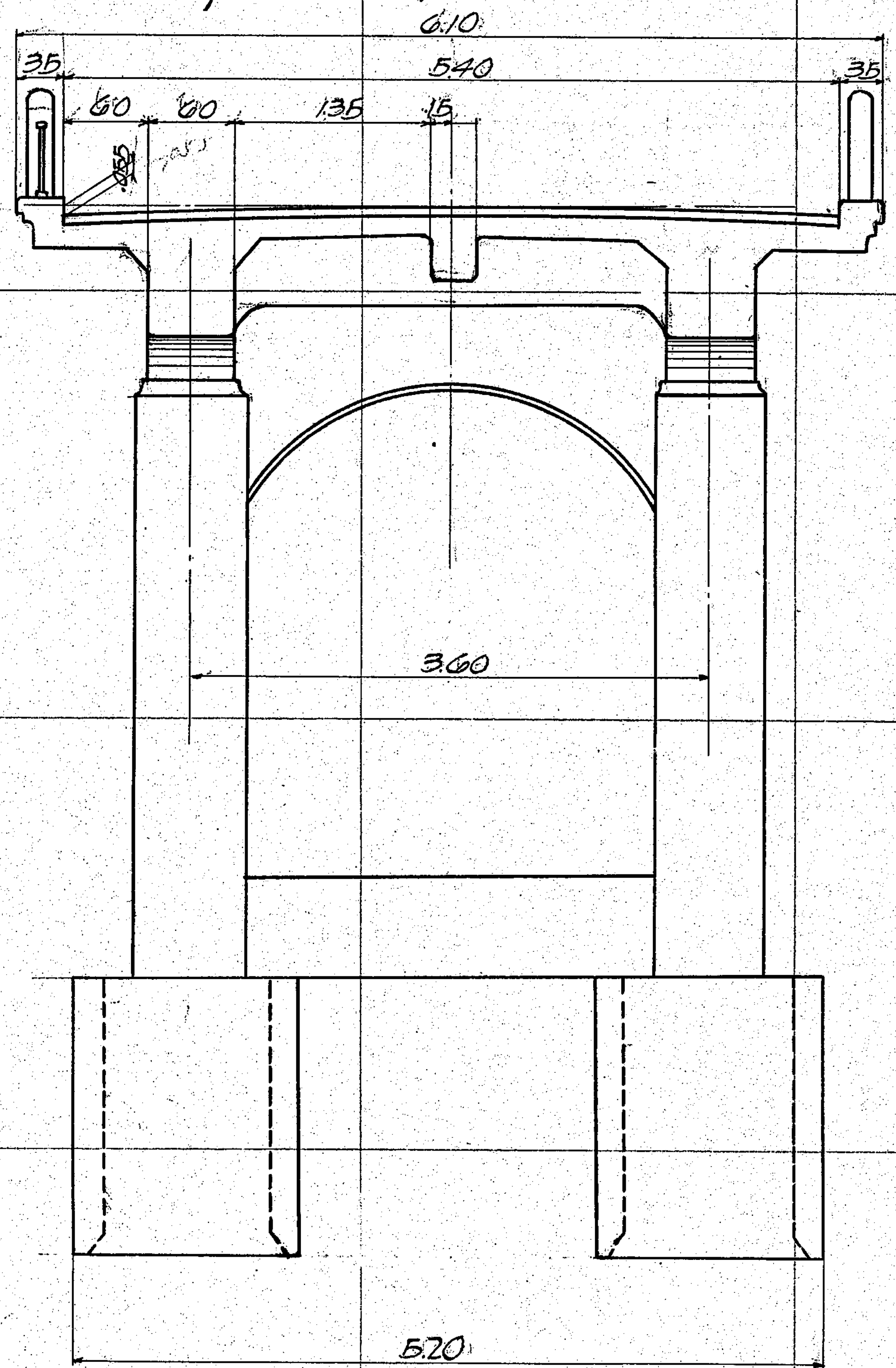
Design of Nakagawa-Bashi for Okayama-Ken.

Concrete girder spans. Clear roadway 5.4 meter
Span length of concrete span determined as shown below



Cross section of Concrete span



Design of floor slabs
between main girders
span length assumed 1.50 meters.

Dead Load
pavement 5cm granolithic @ 22 kg = 110
Concrete slab 13.0 cm @ 24 = 312
misc concrete 18
440

Dead Load moment = $\frac{1}{10} \times 440 \times 1.5^2$ = 99.0
" " Shear = $\frac{1}{2} \times 440 \times 1.5$ = 330.0

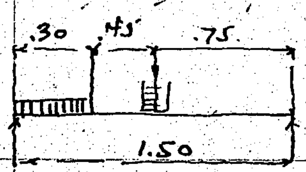
Live Load
motor truck loading
Rear wheel 2250
impact 30% 675
2925 kg
Front wheel 2925 ÷ 3 = 975 "

Distribution of wheel concentration;
contact on pavement 20
distribution 2 × 5 = 10
Longitudinal distribution a = 30 cm
Transv. " b = 24 + 10 = 34 cm

Effective width $E = \frac{2}{3} l + a$
where l = span length in meter
$E = \frac{2}{3} \times 1.5 + .30 = 1.30$

Load per meter strip 2925 ÷ 1.30 = 2250 kg

Uniform live load 500 kg per sq meter

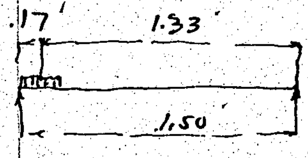


Uniform load reaction = $\frac{500 \times 0.3^2}{2 \times 1.50}$ = 15.0 kg.

moment at center of span
Due to unif. load 15.0 × .75 = 11.3
" " motor truck $\frac{2250}{2}$ × .75 = 845.0
856.3 kgm

For continuity of slab. 0.8 × 856.3 = 685.0 kgm

End shear



2250 × $\frac{1.33}{1.50}$ = 2000 kg.

Design of Nakagawa Bashi for Okayama Ken

Summary for moments and shears

	moment	shear
Dead Load	99	330
Live Load	685	2000
	784 kgm	2330 kg.

Effective depth req'd for $f_s = 1200$ $f_c = 45$ kg/cm²

$k = \frac{M}{bd^2}$ $d = \sqrt{\frac{M}{bk}}$ $k = 7.18$ $d = \sqrt{\frac{784 \times 100}{100 \times 7.18}} = 10.4$ cm

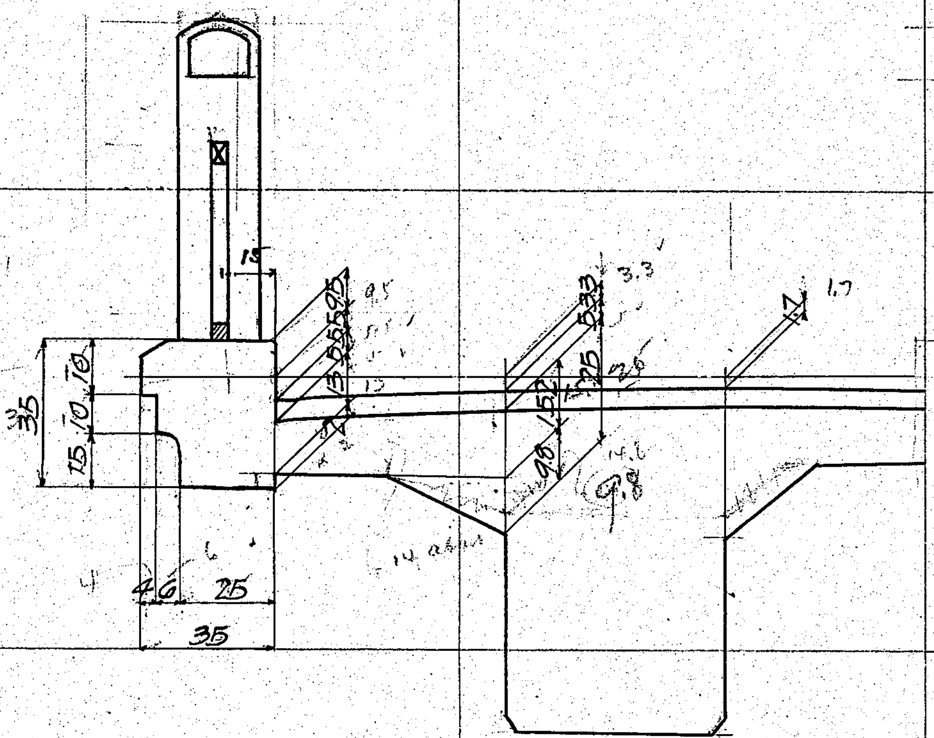
use slab 13.0 cm 2.5 cm insulation at bottom

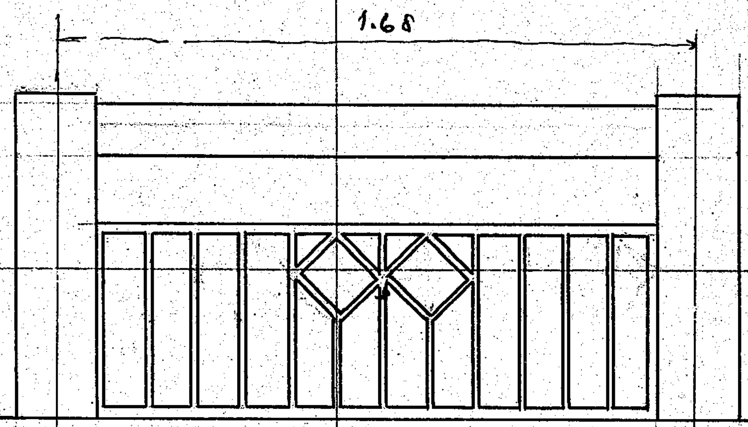
steel area req'd $= \frac{784 \times 100}{7/8 \times 10.5 \times 1200} = 7.11$ cm² per meter strip

13mm bar spacing $\frac{1.33 \times 100}{7.11} = 18.7$ cm use 15 cm spacing.

Slab reinf same as for slab over girder span see page 3.

Overhanging Slab beyond main girder.





weight of Handrail.
Cast iron grate

top & bottom 2 - 2.5 × 2.5 × 1.48 = 1850 cm³
13 - 2.5 × 1.5 × 0.45 = 2200
4050

4050 @ 0.00785 = 31.8 kg.
19. kg per lin. meter.

Top Concrete rail .15 × .15 @ 2400 = 54.0
Concrete post .20 × .20 × .80 @ 2400 = 77.0
77.0 ÷ 1.68 = 46.0

Summary =
grate 19
Top rail 54
post 46
119

Call this 120 kg per lin. meter

volume of coping.
.35 × .35 = 0.1225
less. - .190
0.1035 @ 2400 = 248

pavement and slab.
.14 × .60 = .084
$\frac{.148}{2}$ × .30 = .022
0.106 @ 2400 = 254

average 254 ÷ .60 = 423
110
533 kg/m²

533 · 0.60 = 320 kg per meter

moment at face of main girder

		arm	moment
Handrail	120	× .75	= 90
Coping.	248	× .76 abt	= 189
Slab + pavement	320	× .30 abt	= 96
	688	.545	375

moment at ℄ of main girder
688 × .845 = 582.0 kgm

Live Load moment motor truck loading.
Distribution of wheel concentration

Longitudinal a = 30 cm
Transverse b = 34 cm

Effective width assumed 2/3 × .60 × 2 + a = 1.10 meters.
Load per meter strip 2925 ÷ 1.10 = 2660 kg.

増田淳氏関係資料（独立行政法人 土木研究所蔵）

Design of Nakagawa-Bashi for Okayama-Ken

moment $2660 \cdot (.60 - \frac{.24}{2}) = 1278$

Summary for moments and shears

	moment	shear
Dead Load	375	688
Live Load	1278	2660
	1653 kgm	3348 kg.

depth of Concrete slab 25 cm
Effective depth 22.5 cm
steel area reqd $= \frac{165300}{7/8 \cdot 22.5 \cdot 1200} = 7.04$

13mm bars spacing $= \frac{1.33 \times 100}{7.04} = 18.9$ cm
use 15 cm spacing

Bond stress. 13mm bars cir. 4.08 $\cdot$ no 6.67 $= 27.20$

Bond stress $= \frac{3348}{7/8 \cdot 22.5 \cdot 27.20} = 6.28$

Stress in Concrete Tensile Steel % $\frac{1.33 \times 6.67}{100 \times 22.5} = .394$ %

value of $k = .289$ $kj = .260$

$f_c = \frac{2 M_c}{kj \cdot bd^2} = \frac{165300 \cdot 2}{.26 \cdot 100 \cdot 22.5^2} = 25.2$ kg/cm²

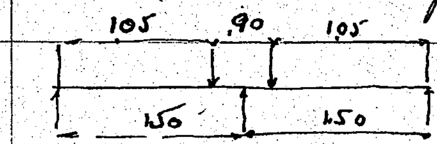
Stringer at center line of bridge span length 3.37 meters

Dead Load

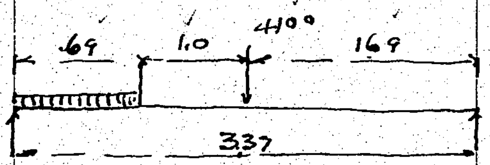
Floor slab and pavement $440 \cdot 1.5 = 660$
stem of beam assumed 216
876 kg per lin. meter

Dead Load moment $= \frac{1}{10} \cdot 876 \cdot 3.37^2 = 995.0$ kgm
Dead Load shear $= \frac{1}{2} \cdot 876 \cdot 3.37 = 1475.0$ kg.

Live Load motor truck rear wheel with impact 2925 kg.
Uniform load 500 kg per sq meter



motor truck loading
$2925 \cdot \frac{1.05}{1.50} = 2050$
2050
4100 kg.



Unif load $500 \cdot 1.5 = 750$ kg. per lin. meter
Reaction $\frac{750 \cdot .69^2}{2 \cdot 3.37} = 53.0$

moment
due to motor truck $2050 \cdot 1.69 = 3460$
" " Unif. load $53 \cdot 1.69 = \cancel{895}$ 90
~~4355~~ 3550 kgm

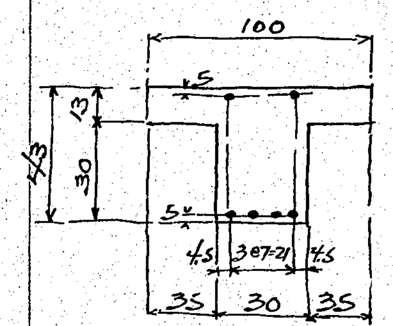
End shear say $\frac{750 \cdot 2.37^2}{2 \cdot 3.37} = 625$
~~4610~~ 4100
4725

Summary for moments and shears

	moment	shear
Dead load	995	1475
Live Load	3550	4725
	4545 kgm	6200 kg.

Design of Nakagawa-Bashi for Okayama-ken.

Assumed Cross-section of Stringer.



Effective depth $= 43 - 5 = 38$ cm

Approximate steel area required $= \frac{4545 \times 100}{1200 \times \frac{7}{8} \times 38} = 11.4$ cm²

Use 4-19 mmφ bars $= 11.4$ cm²

Steel ratio $p = \frac{11.4}{100 \times 38} = .003$, $t/d = 13/38 = .342$.

Neutral axis in the flange, design the beam as a double reinforced rectangular beam. From the prepared diagrams, we have for
$d'/d = 5/38 = .13$, $p' = \frac{1}{2}p = .0015$

$k = 0.252$, $j = .908$

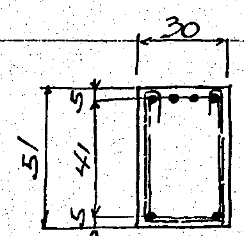
$f_s = \frac{M}{A_s j d} = \frac{4545 \times 100}{11.4 \times .908 \times 38} = 1157$ kg/cm² o.k.

$f_c = \frac{f_s k}{n(1-k)} = \frac{1157 \times .252}{15(1-.252)} = 26.0$ kg/cm² o.k.

Cross-section at supports

Moment $= -4545$ kgm , End shear $= 6200$ kg assumed.

Effective depth required $= \sqrt{\frac{4545 \times 100}{30 \times 7.18}} = 46$ cm



Use insulation at top 5 cm total depth of 51 cm.

Steel area required $= \frac{454500}{1200 \times \frac{7}{8} \times 46} = 9.4$ cm²

Use 4-19 mmφ bars $= 11.4$ cm²

Unit shear $= \frac{6200}{30 \times \frac{7}{8} \times 46} = 5.13$ kg/cm² Use stirrups.

9 mmφ U-stirrups. $A_s = 2 \times .636 = 1.27$ cm² $f_s = 1200$

Stirrup spacing at supports $= \frac{3}{2} \cdot \frac{A_s f_s j d}{V} = \frac{3}{2} \cdot \frac{1.27 \times 1200 \times \frac{7}{8} \times 46}{6200} =$ 15 cm c to c.

Bond stress.

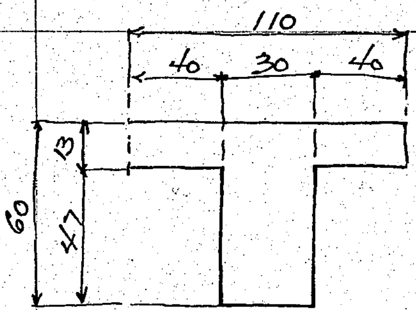
perimeter of bars req'd for plain bars. $= \frac{6200}{6.0 \times \frac{7}{8} \times 46} = 25.7$ cm

main reinforcements 4-19φ @ 5.97 = 23.9
extra bond bars. 2-13φ @ 4.08 = 8.16
(longitudinal bars of slab will be used.) 32.06 cm

Design of Cross Beams.

Intermediate Cross Beam. Span length 3.6 meters. Spacing 3.37 meters.

Assumed cross-section



Dead Load

Cross Beam.

floor and pavement	440 × 1.1 =	484
Stem of beam	.3 × .47 @ 2400 =	338
fillets etc say		18
		840 kg per lin meter.

Dead Load moment (beam) $= \frac{1}{10} \times 840 \times 3.6^2 =$ 1090 kgm

Dead Load shear (〃) $= \frac{1}{2} \times 840 \times 3.6 =$ 1510 kg

Stringer concentration on floor beam. 876 kg/lin m. of stringer. see page 10.

876 × (3.37 − .3) = 2690 kg
less floor 660 × .4 × 2 = −530
2160 kg.

moment due to stringer con. $= \frac{1}{4} \times 2160 \times 3.6 =$ 1945 kgm

for continuity of Beam moment $= 0.8 \times 1945 =$ 1555 kgm.

End shear due to stringer concentration 2160 ÷ 2 = ~~2160~~ 1080 kg

Design of Nakagawa-Bashi for Okayama-ken

Negative moment due to overhanging slab. Effective width of overhanging slab say 1.1 meters see page 9.

weight of Handrail, coping, slab + pavement = 688 kg per meter of main span.

for 1.1 meter width = 688 × 1.1 = 757 kg

Dead Load moment due to overhanging slab = 757 × .545 = −413 kgm Shear = $\frac{413}{36}$ = 11

Summary of Dead Load moments and end shears.

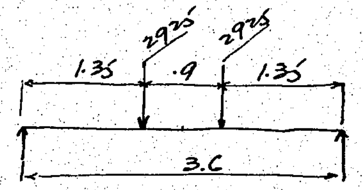
	moment at ℄	Shear	neg. moment at support assumed
D.L. due to wt. of cross beam	1090	1510	1090 × $\frac{10}{12}$ = −910
〃 〃 Stringer concentration	1555	1080	1555 × $\frac{10}{12}$ = −1295
〃 〃 overhanging slab.	−413	−11	−413
	2232 kgm	2579 kg	−2618 kgm

Live Load. Uniform live load 500 kg/m²

motor truck rear wheel concentration with impact 2925 kg

〃 〃 front wheel 〃 〃 〃 975 〃

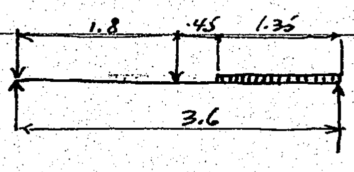
Rear wheel directly on floor beam assumed.



moment due to rear wheel = 2925 × 1.35 = 3,905 kgm

for continuity of crossbeam m = 3905 × .8 = 3,125 kgm

neg. moment at support say 3125 × $\frac{10}{12}$ = −2,600 kgm

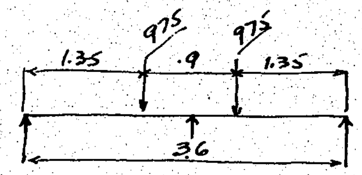


End shear 2925 × 1.8 ÷ 3.6 = 1463

2925

4388 kg

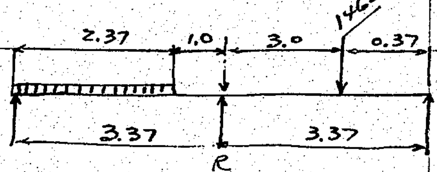
Front wheel. transverse distribution.



~~moment due to~~ Reaction on center stringer = $\frac{975 \times 1.35}{1.8}$ × 2 = 1462 kg

Uniform load full. 500 × 1.8 = 900 kg per lin meter of stringer.

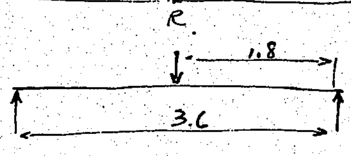
Concentration on center cross beam.



~~Rear~~ front wheel. = $\frac{1462 \times 0.37}{3.37}$ = 160

Unif. load = $\frac{900 \times 2.37^2}{2 \times 3.37}$ = 750

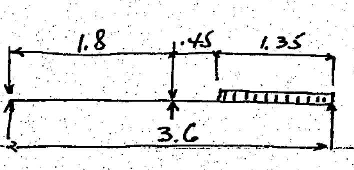
910 kg



moment on cross beam = $\frac{910}{2}$ × 1.8 = 820 kgm.

for continuity of beam m = 820 × .8 = 656 kgm

(End shear = 910 ÷ 2 = 455 kg)



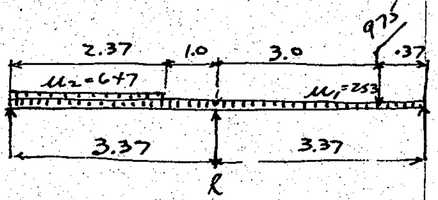
neg. moment at end say 656 × $\frac{10}{12}$ = −547 kgm

max end shear.

unif. load on side of truck = $\frac{1.35^2 \times 500}{2 \times 1.8}$ = ~~675~~ 253 kg/lin m. = u_1

〃 〃 on front + rear of truck = $\frac{3.6}{2}$ × 500 = 900 kg/lin m.

647 〃 = u_2



Stringer concentration on floor beam

front wheel = 975 × 0.37 ÷ 3.37 = 107

unif load u_1 = 253 × 3.37 = 852

〃 u_2 = $\frac{647 \times 2.37^2}{2 \times 3.37}$ = 540

1,499 kg

End shear = 1499 ÷ 2 = 750 kg

Summary of Live Load moments + shears.

	moment at center	end shear	neg. moment at support.
Live Load stresses due to rear wheel.	3125	4388	−2600
〃 〃 front wheel / unif load.	656	750	−547
	3781 kgm	5138 kg	−3147 kgm

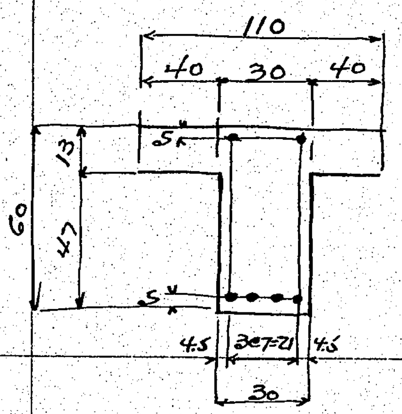
Summary for D.L. + L.L.

Dead Load	2232	2579	−2618
Live Load.	3781	5138	−3147
	6,013 kgm	7717 kg	−5765 kgm.

Design of Nakagawa-Bashi for Okayama-Ken.

Sections of Cross-Beam.

Section at Center of span. moment $= +6013$ kgm. Effective depth say $60-5=55$ cm.



Approx. steel area req'd $= \dfrac{6013 \times 100}{1200 \times \frac{7}{8} \times 55} = 10.4$ cm²

Use 4-19 mm φ bars $= 11.4$ cm²

Steel ratio $p = \dfrac{11.4}{110 \times 55} = .0019$ $p' = \frac{1}{2}p = .00095$

$t/d = 13/55 = .236$

Neutral axis in the flange, design the beam as a double reinforced rectangular beam. From the prepared diagrams, we have for

$d'/d = 5/55 = .091$

$k = .24$, $j = .92$

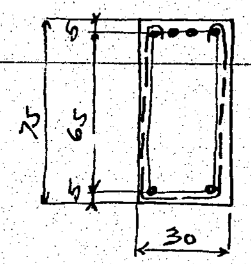
$f_s = \dfrac{6013 \times 100}{11.4 \times .92 \times 55} = 1{,}043$ kg/cm² ok.

$f_c = \dfrac{1043 \times .24}{15(1-.24)} = 22.$ kg/cm² ok.

Section at End support.

moment $= -5765$ kgm, Shear $= 7717$ kg

Effective depth req'd $= \sqrt{\dfrac{5765 \times 100}{30 \times 7.18}} = 51.7$ cm



Use 70 cm effective depth with 5 cm insulation at top or 75 cm in total.

Steel area req'd $= \dfrac{5765 \times 100}{1200 \times \frac{7}{8} \times 70} = 7.85$ cm²

Use 4-19 mm φ bars $= 11.4$ cm² ok.

Unit shear $= \dfrac{7717}{30 \times \frac{7}{8} \times 70} = 4.2$ kg/cm² use Stirrups.

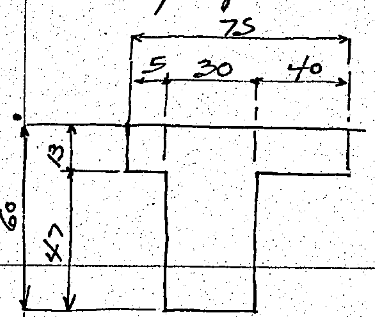
13 mm φ U-Stirrups $A_s = \cancel{2 \times 2.65 = 5.3}$ cm² $1.327 \times 2 = 2.65$ cm²

Stirrup spacing $= \dfrac{3}{2} \cdot \dfrac{\cancel{5.3}\,2.65 \times 1200 \times .875 \times 70}{7717} = 37.8$ cm c to c. at support.

Use 30 cm c to c about at support.

Unit bond $= \dfrac{7717}{4 \times 5.97 \times \frac{7}{8} \times 70} = 4.4$ kg/cm² ok.

Design of End Cross Beam Span length 3.6 meters, spacing 3.37 meters to int. cross beam.



Dead Load:-

Cross Beam

floor and pavement	$440 \times .75 =$	330
Stem of beam		338
fillets &c say		12
		680 kg per lin meter.

Moment due to wt. of beam $= \frac{1}{10} \times 680 \times 3.6^2 = \underline{881}$ kgm.

End Shear " " $= \frac{1}{2} \times 680 \times 3.6 = \underline{1225}$ kg

Stringer concentration on cross beam. 876 kg/lin m. of stringer see page 10.

$876 \times (3.37-.3) = 2690$ kg

less floor $660 \times .4 \times 2 = -530$

$2160 \div 2 = 1080$ kg

add overhang $660 \times .05$ say $= 33$

$1{,}113$ kg

Moment due to stringer concentration $= \frac{1}{4} \times 1113 \times 3.6 = 1000$

for continuity of Beam moment $= 1000 \times 0.8 = \underline{800}$ kgm

End shear $= 1113 \div 2 = \underline{557}$ kg

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CHECKED BY DATE PAGE NO 14

CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama-ken.

Negative moment due to overhanging slab.

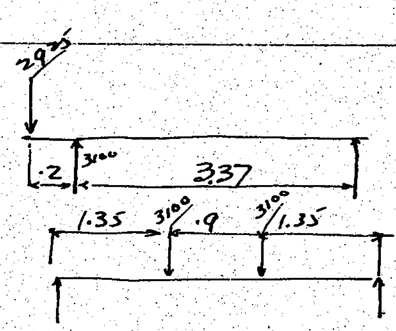
Effective width say $\frac{1}{2} \times 1.1 = 0.55$
0.20
0.75^m

weight of Handrail, coping, slab, and pavement = 688 kg/meter of main span.
for 0.75 meter width $688 \times 0.75 = 516$ kg
moment due to overhanging slab $= 516 \times .545 = -281$ kgm, Shear $= \frac{281}{3.6} = -8$ kg

Summary of Dead Load moments and shears.

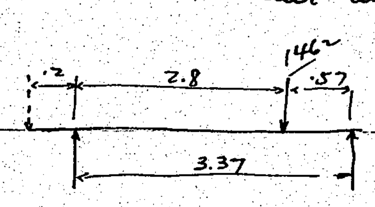
	moment at center of span	moment at support	End shear
Due to wt. of crossbeam	881	$881 \times \frac{10}{12} = -734$	1225
〃 Stringer concentration	800	$880 \times \frac{10}{12} = -667$	557
〃 overhanging slab.	-281	-281	-8
	+1400 kgm	-1682 kgm	1774 kg

Live Load. Uniform Live Load = 500 kg/m²
motor truck rear wheel concentration with imp. = 2925 kg
〃 〃 front 〃 〃 〃 〃 = 975 〃
moment and shears due to rear wheel of motor truck.
Load on cross beam $= 2925 \times \frac{3.57}{3.37} = 3100$ kg

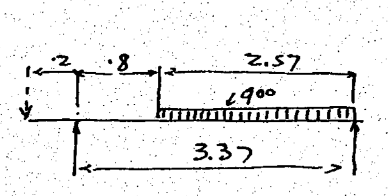


moment = $3100 \times 1.35 = 4185$
for continuity of beam moment $= 4185 \times .8 = 3350$ kgm

Moment and shears due to front wheel of motor truck. or unif. load.
front wheel concentration on stringer = 1462 kg see page 12.
Concentration on End cross beam $= \frac{1462 \times .57}{3.37} = 247$ kg

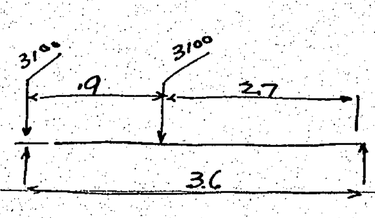


Uniform load 900 kg/lin meter.
Stringer concentration on cross beam $= \frac{900 \times 2.57^2}{2 \times 3.37} = 882$ kg this case governs.



moment $= \frac{882}{2} \times 1.8 = 794$
for continuity of beam moment $= 0.8 \times 794 = 635$ kgm.

End shears due to rear wheels of motor truck.
$3100 \times 2.7 \div 3.6 = 2330$
3100
5430 kg



End shear due to unif. load $882 \div 2 = 441$. kg

Summary of Live Load moment and end shear.

	moment at center of span	moment at support	End shear.
motor truck rear wheels.	+ 3350	$3350 \times \frac{10}{12} = -2790$	5430
unif. load	+ 635	635 × 〃 = -530	441
	+ 3985 kgm	-3320 kgm	5871 kg

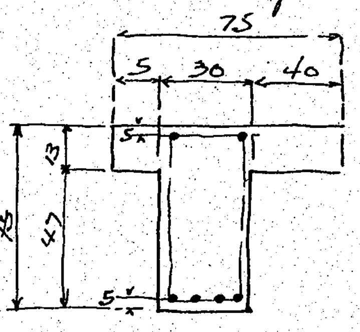
Summary for Dead + Live load moments + shears.

	moment at mid span	moment at support	End shear
Dead Load	+ 1400	- 1682	1774
Live Load.	+ 3985	- 3320	5871
	+ 5385 kgm	- 5002 kgm	7645 kg.

CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama-ken.

Cross-section of End Cross Beam.



Section at center of span moment $= +5385$ kgm effective depth 55 cm.

Steel area req'd $= \dfrac{5385\times100}{1200\times\frac{7}{8}\times55} = 9.33$ cm²

Use 4-19 mmφ bars $= 11.4$ cm²

Steel ratio $= \dfrac{11.4}{75\times55} = .00277$ $\quad p' = \frac{1}{2}p = .00139$

$t/d = 13/55 = .236 = \Delta$. $\quad d'/d = 5/55 = .091$, $\Delta^2 = .056$

Neutral axis in the web.

$$k = \frac{p + p'\left(\frac{d'}{d}\right) + \frac{\Delta^2}{2n}}{p + p' + \frac{\Delta}{n}} = \frac{.00277 + .00139\times.091 + .0019}{.00277 + .00139 + .0157} = 0.242$$

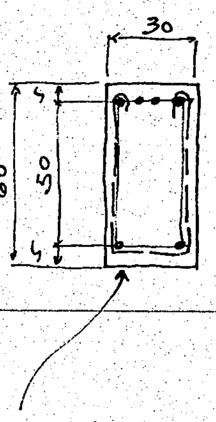
$$j = \frac{\Delta(2k-\Delta) - \frac{\Delta^2}{3}(3k-2\Delta) + 2p'n(k-\frac{d'}{d})(1-\frac{d'}{d})}{\Delta(2k-\Delta) + 2p'n(k-d'/d)}$$

$$= \frac{.236\times.248 - .0186\times.254 + .0417\times.151\times.909}{.236\times.248 + .0417\times.151} = \frac{.0595}{.0648} = 0.919$$

$$f_s = \frac{5385\times100}{11.4\times.919\times55} = 935 \text{ kg/cm}^2 \quad \text{ok}$$

$$f_c = \frac{935\times.242}{15(1-.242)} = 19.9 \quad \text{ok.}$$

Section at end.



Use fillet if possible

moment $= -5002$ kgm, end shear $= 7645$ kg

Effective depth req'd $= \sqrt{\dfrac{5002\times100}{30\times7.18}} = 48.2$ cm

Use 55 cm effective depth with 5 cm insulation at top.

Steel area req'd $= \dfrac{5002\times100}{1200\times\frac{7}{8}\times55} = 8.68$ cm²

$p = \dfrac{11.4}{30\times55} = .0069$ $\quad d'/d = 5/55 = .091$

Use 4-19 mmφ bars $= 11.4$ cm² ok

$k = .336$ $\quad j = .89$

~~Unit shear $= \dfrac{7645}{30\times.89\times55} = 5.2$ kg/cm²~~

Use 13 mmφ U-stirrups $A_s = 2.65$ cm²

Stirrup spacing $= \dfrac{3}{2}\times\dfrac{2.65\times1200\times.89\times55}{7645} = 30.5$ cm c to c at support.

Unit bond $= \dfrac{7645}{4\times5.97\times.89\times55} = 6.54$ kg/cm²

2 bond bars 13 mmφ be used.

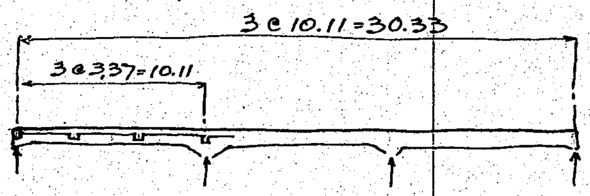
perimeter $= 2 @ 4.08 = 8.16$ — slal reinforcements will be avarable.
$4 @ 5.97 = 23.88$
32.04 cm²

bond stress $= \dfrac{7645}{32.04\times.89\times55} = 4.9$ kg/cm² ok

Design of Nakagawa-Bashi for Okayama-ken.

Design of Main Beam. Continuous beam 3 spans @ 10.11 = 30.33 m, one span = 3 panel @ 3.37 = 10.11

Dead load:-

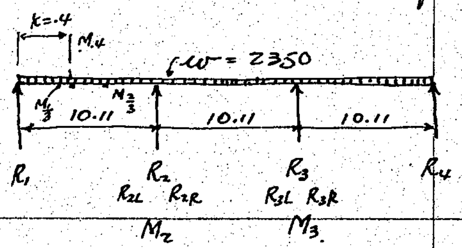


Floor beam concentration on main beam due to
Intermediate cross beam = 2590 kg
End cross beam = 1780 kg

Uniform load.
Handrail, coping, overhanging floor 688
floor 1.2 @ 440 = 528
fillet inside $\frac{.148}{2} \cdot .3$ @ 2400 = 54
1270 kg per lin meter of span

Stem of main beam assumed $25^{cm} \times 60^{cm}$ average.
.25 × .6 @ 2400 = 1080
Handrail floor etc 1270
2350 kg per lin meter of span.

Moments and shears for uniform load.



$M_2 = M_3 = -\frac{1}{10} wl^2 = -\frac{1}{10} \times 2350 \times 10.11^2 = -24050$ kgm

Shear $R_1 = R_4 = .40\,wl = .40 \times 2350 \times 10.11 = \pm 9510$ kg

" $R_{2L} = R_{3R} = .60\,wl = .60$ " " $= \mp 14270$ "

" $R_{2R} = R_{3L} = .50\,wl = .50$ " " $= \pm 11900$ "

reaction $R_2 = R_3 = 1.10\,wl = 1.10$ " " $= 26170$ "

$M.4 = 9510 \times 0.4 \times 10.11 - 2350 \times \frac{(10.11 \times .4)^2}{2} = +19200$ kgm

$M_{\frac{1}{3}} = 9510 \times \frac{10.11}{3} - 2350 \times \frac{(10.11 \div 3)^2}{2} = +18700$ "

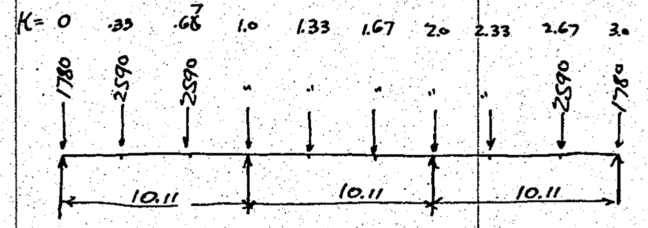
$M_{\frac{2}{3}} = 9510 \times \frac{10.11 \times 2}{3} - 2350 \times \frac{(10.11 \times \frac{2}{3})^2}{2} = +10700$ "

$M_{\frac{1}{2}} = 9510 \times \frac{10.11}{2} - 2350 \times \frac{(10.11 \times \frac{1}{2})^2}{2} = +18000$ "

$M_{1\frac{1}{3}} = 9510 \times 10.11 \times \frac{4}{3} - 2350 \times \frac{(10.11 \times \frac{4}{3})^2}{2}$
$+ 26170 \times \frac{10.11}{3} = +2800$ "

$M_{1\frac{1}{2}} = 9510 \times 10.11 \times \frac{3}{2} - 2350 \times \frac{(10.11 \times \frac{3}{2})^2}{2}$
$+ 26170 \times \frac{10.11}{2} = +6300$ "

Moments and shears due to Cross-beam concentrations.



By the prepared diagram.

K	M_2 unit load.	$10 M_2 / l$ (moment for 10 m span).
.33	−.780	
.67	−.990	
1.00	−.000	
1.33	−.790	
1.67	−.540	
2.00	−.000	
2.33	+.242	
2.67	+.200	
	−2.658 × 2590 = −6880	

$M_2 = M_3 = -6880 \times \frac{10.11}{10} = -6960$ kgm.

Design of Nakagawa Bashi for Okayama-ken.

Dead Load Reactions and Shears due to Cross Beam concentrations.

Reaction R1.

K.	R_1 unit load	load
.00	+ 1.00	× 1780 = 1780
.33	+ .59	
.67	+ .23	
1.00	.00	
1.33	− .08	
1.67	− .06	+.73 × 2590 = 1890
2.00	.00	
2.33	+ .025	
2.67	+ .025	
3.00	.00	× 1780 = 0

R_1 = +3670 kg

End shear less $\frac{1780}{2}$ = 890

+2780 kg

Reaction R2.

R_2 unit load	load
.000	× 1780 = 0
+ .505	
+ .890	
+ 1.000	
+ .770	
+ .360	+3.255 × 2590 = +8420
.000	
− .150	
− .120	
.000	× 1780 = 0

R_2 = +8420 kg

~~less~~

End Shears R2L

K.	R_{2L} unit load	load
.00	.000	× 1780 = 0
.33	− .410	
.67	− .770	
1.00	− 1.000	
1.33	− .078	−2.268 × 2590 = −5870
1.67	− .055	
2.00	.000	
2.33	+ .025	
2.67	+ .020	
3.00	.000	× 1780 = 0

−5870 kg

less 2590 ÷ 2 = +1300

Shear R_{2L} = −4570 kg

End Shears R2R.

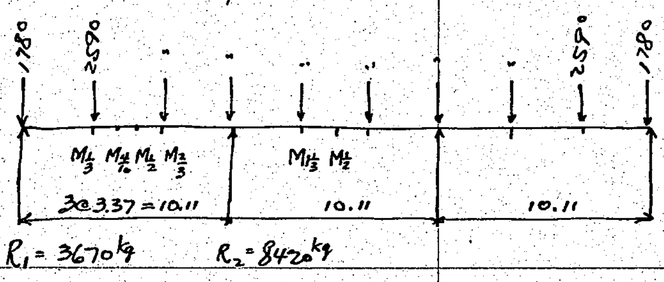
R_{2R} unit load	load
+ .000	× 1780 = 0
+ .098	
+ .122	
+ 1.000	
+ .691	+2.00 × 2590 = +5180 kg
+ .309	
.000	
− .122	
− .098	
.000	× 1780 = 0

+5180 kg

less 2590 ÷ 2 = −1300

Shear R_{2R} +3880 kg

Dead Load positive moments due to cross beam concentrations at various points.



$M_{\frac{1}{3}}$ = 3670 − 1780 = 1890 × 3.37 = +6,370 kgm

$M_{\frac{4}{10}}$ = 1890 × 4.04 = +7645
−2590 × .67 = −1735
+5,910 kgm

$M_{\frac{1}{2}}$ = 1890 × 5.05 = +9540
2590 × 1.68 = −4350
+5,190 kgm

$M_{\frac{2}{3}}$ = 1890 × 6.74 = +12740
2590 × 3.37 = −8730
+4,010 kgm

$M_{1\frac{1}{3}}$ = 1890 × 13.48 = +25500
8420 × 3.37 = +28380
2590 × 3.37 × 6 = −52400
+1,480 kgm.

$M_{1\frac{1}{2}}$ = 1890 × 15.17 = +28650
8420 × 5.06 = +42600
2590 × 3.37 × 8 = −69830
+1,420 kgm.

Summary of Dead Load Moments, Shears and Reactions.

	$M_{\frac{1}{3}}$	$M_{\frac{4}{10}}$	$M_{\frac{1}{2}}$	$M_{\frac{2}{3}}$	M_2	$M_{1\frac{1}{3}}$	$M_{1\frac{1}{2}}$	R_1	R_{1R}	R_2	R_{2L}	R_{2R}
Uniform load (direct)	+18700	+19200	+18,000	+10700	−24,050	+2800	+6300	9,510	9,510	26,170	14,270	11,900
Cross beam concentrations	+6370	+5910	+5910	+4010	−6960	+1480	+1420	3670	2780	8420	4570	3880
	+25,070	+25,110	+23,190	+14,710	−31,010	+4,280	+7,720	13,180	12,290	34,590	18,840	15,780

CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama-ken

Live Load :-

motor truck rear wheel concentration = 2250 kg

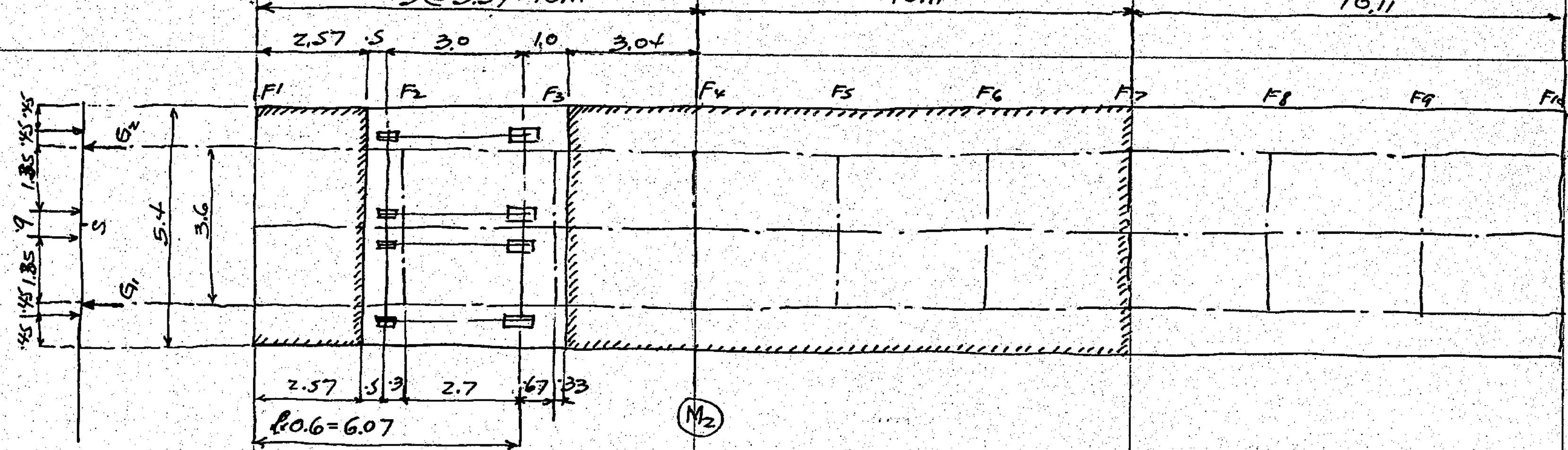
Impact $\frac{20}{60+l} = \frac{20}{60+10.11} = 28.5\%$ = 640

2890 kg

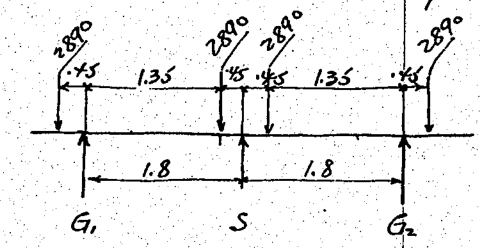
front wheel concentration with imp. = 2890 ÷ 3 = 960 kg say.

Uniform load on Roadway = $\frac{10,000}{170+10.11}$ = 555 use 500 kg/m²

Max. negative moment on pier M_2. Crysterion for max M_2, rear wheel at 0.6 l from left support. (By the preparea diagrams)



Rear wheel concentration on Stringer + main girder G1.

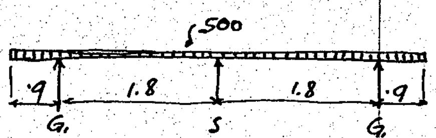


Extra load on main girder

$2890 \times \frac{.45}{1.8} = 720$ kg

G1	S	G2
2890	2170	720
720	2170	2890
720	−1440	720
4330 kg	2900 kg	4330 kg for rear wheel.
1445	970	1445 for front wheel.

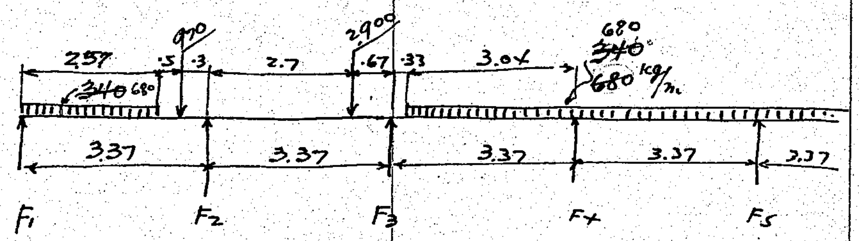
Unif. load concentration on Stringer + main girder G1.



$\frac{500 \times 2.7^2}{2 \times 1.8} = \frac{1010}{940}$

G1	S	G2
1010	340	
	340	1010
1010 kg	680	1010

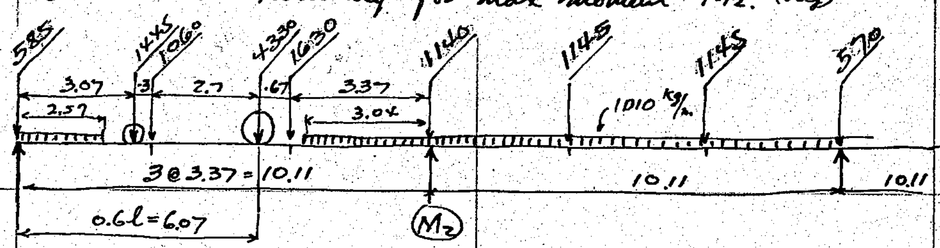
Wheel concentration on Cross beams F1, F2, F3, F4 F5



F1	F2	F3	F4	F5
1084	666	935	1135 / 1145	2290
86	884			
	575	2325		
1170	2125	3260	2280	2290
½ 585	1060	1630	1140	1145

Unif. load on main girder u = 1010 kg/m.

Combined Loading for max moment M_2. (neg)



Neg. moment M_2 due to concentration

K =	M_2 unit load.	load.	$10\,M_2/l$
0.304	−.735	1445	−1,061
0.33	−.780	1060	−828
0.60	−1.024	4330	−4435
0.67	−.990	1630	−1615
1.00	.000	1140	0
1.33	−.790	1145	−905
1.67	−.540	1145	−619
2.00	.000	570	0
			−9,463

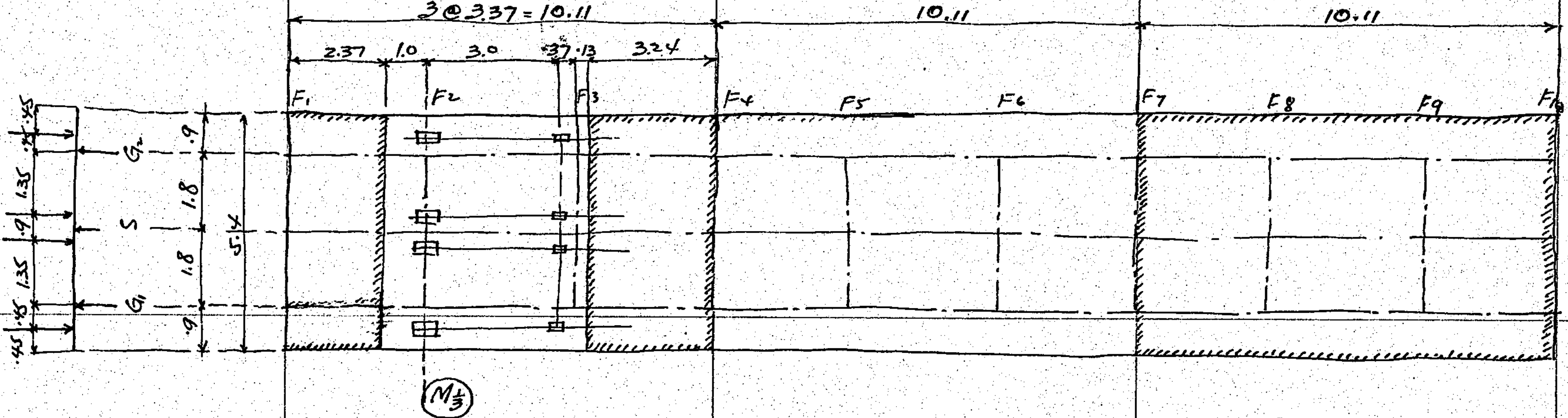
$M_2 = -16973 \times \frac{10.11}{10} = -17,100$ kgm.

M_2 due to unif. load. 1010 kg/m² directly on girder.

K.	M_2 unit	load.	
.00	.0	505	0
.1	−.265	1010	−268
.2	−.51	1050	−535
.7	−.95	505	−480
.8	−.77	1010	−778
.9	−.45	1010	−449
1.0	.0	1010	0
1.1 to 1.9	−4.95	1010	−5000
2.0	.0	505	0
			−7510
			−9463
			−16,973

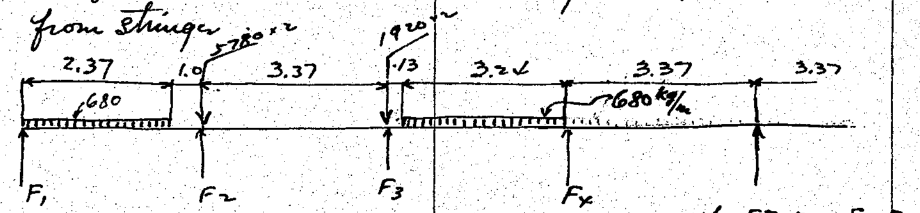
Design of Nakagawa-Bashi for Okayama-ken

Max. pos. moment in the 1st span
Pos. moment at 1/3 l from left support $M_{\frac{1}{3}}$



Rear wheel concentration on main girder
2@ 2890 = 5780 kg (direct on cross beam)
Front wheel do.
2@ 960 = 1,920 kg assumed direct on girder

Unif. load concentration on main girder
from stringer

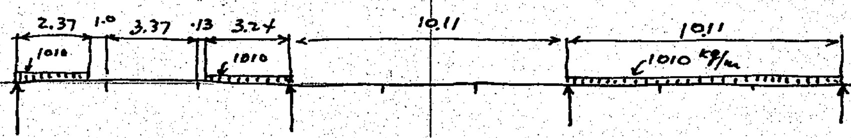


	F1	F2	F3	F4	for F7+F10	F8+F9
	1048	567	1058	1144	1145	2290
1/2 of above for one girder	524	284	529	~~572~~	573	1145
Wheels		5780	1920			
	524	6064	2449	572	573	1145

Pos.m. $M_{\frac{1}{3}}$ due to concentrations

K	$M_{\frac{1}{3}}$ unit	load	$\frac{10}{l} M_{\frac{1}{3}}$
.0	.00	524	0
.33	1.96	6,064	+11,880
.67	.78	2,449	+ 1,910
1.00	.00	572	0
2.00	.00	1,145	0
2.33	.07	2290	+ 160
2.67	.062	2290	+ 142
3.00	.00	1,145	0
			+14092

Unif load directly on girder.



moment $M_{\frac{1}{3}}$ due to unif load directly on girder.

k.			
.1	.58	1010	+ 586
.2	1.20	860	+ 1032
.7	.68	748	+ 508
.8	.41	1,020	+ 414
.9	.18	1,020	+ 182
2.1 to 2.9	.54	1020	+ 546
			+ 3268
			+ 14,092
		total.	+ 17360

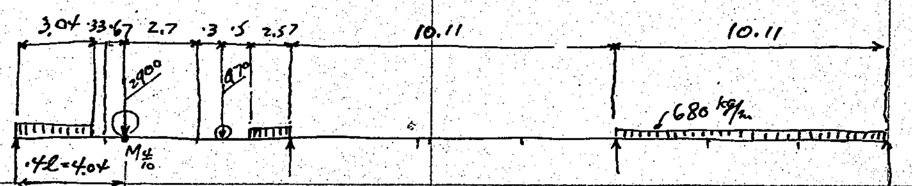
$1010 \times \frac{10.11}{10} = 1020$

Pos moment $M_{\frac{1}{3}} = 17360 \times \frac{10.11}{10} = +17,560$ kgm.

Live load positive max. moment at 0.4 l from left support. $+M_{\frac{4}{10}}$.
$+M_{\frac{4}{10}}$ due to stringer

K.		load.	$M_{\frac{4}{10}} \cdot \frac{10}{l}$
.33	1.68	1630	2736
.67	.93	1063	990
2.33	~~.07~~ .10	1145	115
2.67	.09	1145	103
			3944

Concentration on main girder due to stringer.

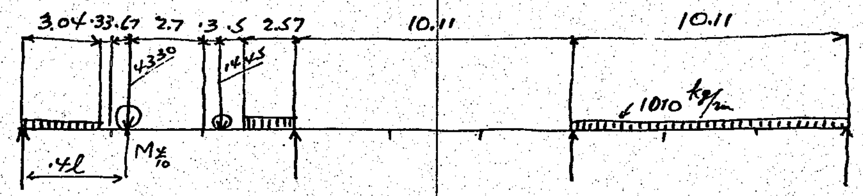


1135	935	666	1084	1145	2290	2290	1145
	2326	575					
		884	86				
1135	3260	2125	1170	1145	2290	2290	1145
$\frac{1}{2}$ ... 568	1630	1063	585	573	1145	1145	573

Design of Nakagawa-Bashi for Okayama-ken

$M_{\frac{4}{10}}$ due to loads on main girder directly.

K		loads.	
·1	·50	1020	510
·2	1.00	1020	1,020
·3	1.51	515	778
·4	2.04	4,330	8830
·7	·82	1445	1185
·8	·50	1,080	540
·9	·22	1020	224
2.1 to 2.9	·66	1020	674
			13761
			3944
			17705



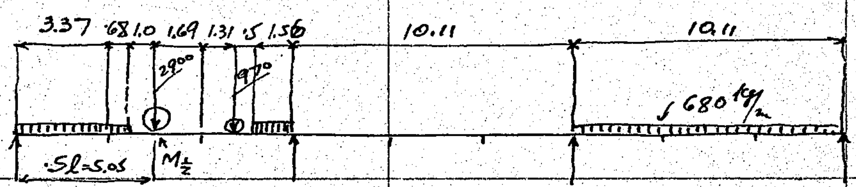
$1010 \times \frac{10.11}{10} = 1020$

$+M_{\frac{4}{10}} = 17705 \times \frac{10.11}{10} = +17900$ kgm.

Max pos. live load moment at center of left span $M_{\frac{l}{2}}$.

$+M_{\frac{l}{2}}$ due to load on stringer.

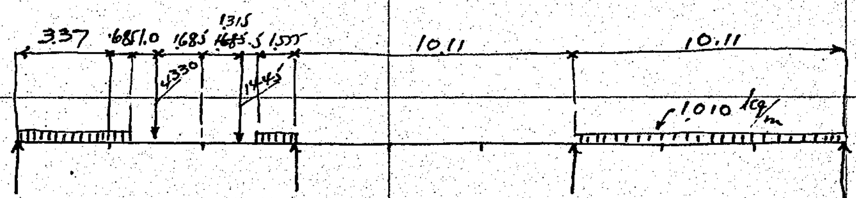
k.		load.	
·33	1.26	1,505	1,895
·67	1.15	1,168	1,343
2.33	·13	1,145	149
2.67	·10	1,145	115
			3,502



1145	1145	246	814	1145	2290	2290	1145
	415	47					
	1450	1450					
		593	377				
1145	3010	2336	1191	1145	2290	2290	1145
½ … 573	1,505	1168	596	573	1145	1145	573

$+M_{\frac{l}{2}}$ due to loads directly on main girder.

k.			
·1	·37	1,020	378
·2	·75	1,020	765
·3	1.13	1,020	1,153
·4	1.55	515	798
·5	2.00	4330	8660
·797	·62	1,445	896
·9	·27	1,060	286
2.1 to 2.9	·84	1,020	857
			13793
			3,502
			17295



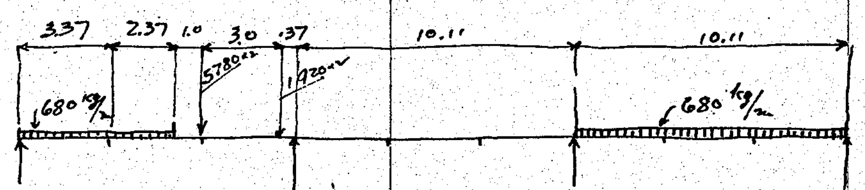
$1010 \times \frac{10.11}{10} = 1020$

$+M_{\frac{l}{2}} = +17295 \times \frac{10.11}{10} = +17,480$ kgm.

Max. pos. live load moment at $\frac{2}{3}l$ from left support, $+M_{\frac{l}{3}}$.

$+M_{\frac{l}{3}}$ due to panel concentration

k			
·33	·58	1,095	635
·67	1.62	6,273	10,170
2.33	·165	1,145	189
2.67	·13	1,145	149
			11,143



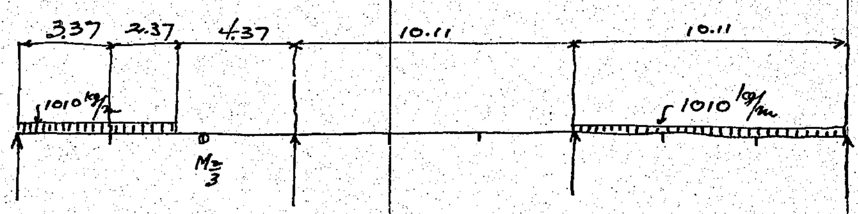
1145	1145			1145	2290	2290	1145
	1044	566					
1145	2189	566		1145	2290	2290	1145
½ … 573	1095	283		573	1145	1145	573
		210	1710				
		5780	~~1920~~				
573	1,095	~~6063~~ 6,273	~~1920~~ 1710	573	1145	1145	573

CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama ken.

$+M_{\frac{2}{3}}$ due to uniform load directly on girder.

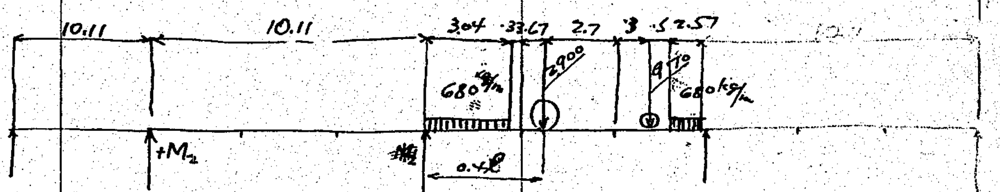
K.		load	
1 to 5	2.73	1020	2785
.6	.132	182	240
2.1 to 2.9	1.115	1020	1137
			4162
			11,143
			15305



$+M_{\frac{2}{3}} = +15305 \cdot \frac{10.11}{10} = \underline{15480}$ kgm.

Max. positive moment at 2nd support $+M_2$.
$+M_2$ due to load on stringer

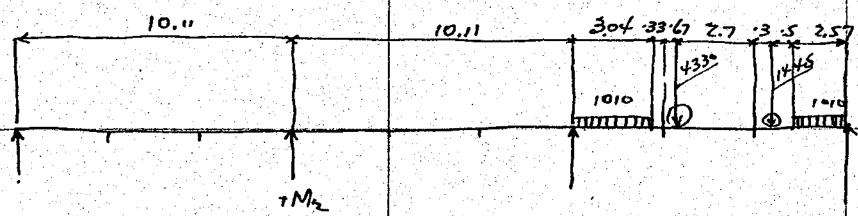
K.			
2.33	.245	1630	400
1.67	.205	1603	329
			+ 729



see P19 ($+M_{\frac{4}{10}}$) $\frac{1}{2}$ ---- 568 1630 1063 585

Pos. moment M_2 due to direct load on girder.

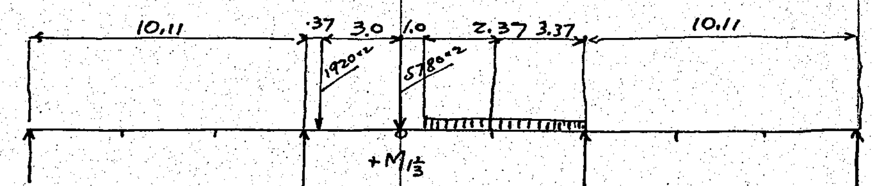
k			
2.1	.115	1020	117.3
2.2	.188	1020	192
2.3	.240	515	124
2.4	.255	4330	1104
2.7	.190	1445	275
2.8	.135	1080	146
2.9	.07	1020	71
			+ 2029
			729
			+ 2758



$+M_2 = +2758 \cdot \frac{10.11}{10} = \underline{+2790}$ kgm.

Max. pos. moment at $\frac{1}{3}l$ right side of 2nd support. $+M_{1\frac{1}{3}}$
$+M_{1\frac{1}{3}}$ due to loads on stringers + wheel load.

K.			
1.33	1.53	6273	9600
1.67	.48	1095	526
			10126

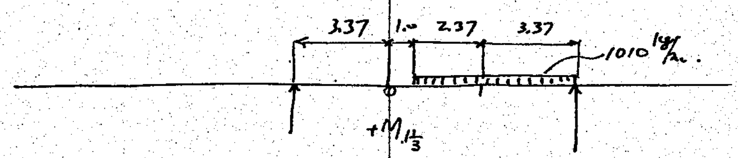


$\frac{1}{2}$ ---- 1710 6273 1095 573

See page 20 for $+M_{\frac{2}{3}}$

$+M_{1\frac{1}{3}}$ due to unif load directly on main girder.

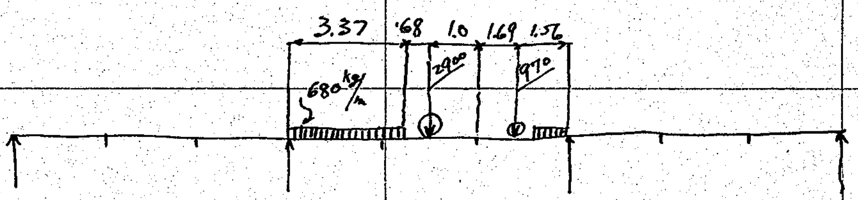
K			
1.4 (~~2.4~~)	1.255	182	229
1.5 to 1.9 (~~2.5 to 2.9~~)	2.36	1020	2408
			2637
			10126
			12763



$+M_{1\frac{1}{3}} = +12763 \cdot \frac{10.11}{10} = \underline{+12900}$ kgm.

Pos. max. moment at center of center span $+M_{1\frac{1}{2}}$.
$+M_{1\frac{1}{2}}$ due to panel concentration from stringer

K.			
1.33	.98	1505	1475
1.67	.98	1168	1145
			2620



$\frac{1}{2}$ ---- 573 1505 1168 596

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JIJI BLDG, TOKYO

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CHECKED BY DATE PAGE NO 22.

CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama-ken.

$+M_{1\frac{1}{2}}$ due to direct load on main beam.

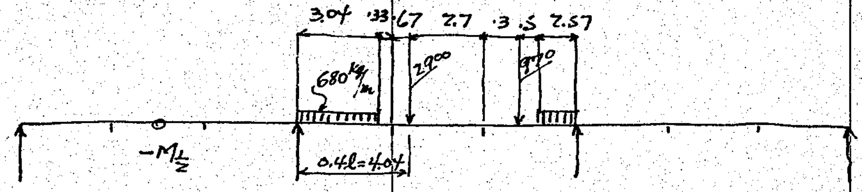
See for $+M_{\frac{1}{2}}$ 1st span. load in center span only.

k.			
1.1	.023	1020	235
1.2	.52	1020	530
1.3	.87	1020	888
1.4	1.28	515	659
1.5	1.75	4330	7580
1.797	.52	1445	752
1.9	.23	1060	244
			10,888
			2620
			13,508

$+M_{1\frac{1}{2}} = +13,508 \cdot \frac{10.11}{10} = +13660$ kgm.

Live Load negative moment at center of left span. $-M_{\frac{1}{2}}$.

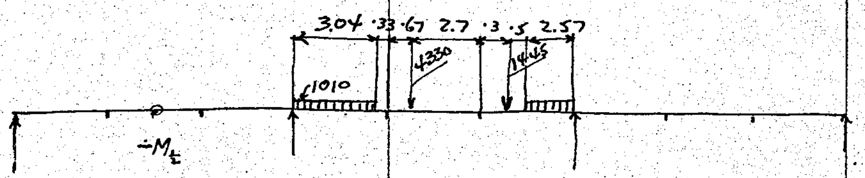
k.			
1.33	.39	1630	636
1.67	~~.36~~ ~~.27~~ .27	1063	287
			923



Live Load neg. moment due to direct load on girder.

$\frac{1}{2}$--- 568 1630 1063 585

k.			
1.1	.19	1020	194
1.2	.32	1020	326
1.3	.38	515	196
1.4	.40	4330	1,733
1.7	.25	1445	362
1.8	.16	1080	173
1.9	.08	1020	82
			3066
			923
			3989



$-M_{\frac{1}{2}} = -3989 \cdot \frac{10.11}{10} = -4030$ kgm.

Live Load negative moment at ⅔l from left support. $-M_{\frac{2}{3}}$

$-M_{\frac{2}{3}}$ due to panel concentration.

Same load for $-M_{\frac{1}{2}}$ as above.

k.			
1.33	.525	1630	856
1.67	.36	1063	383
			1,239

$-M_{\frac{2}{3}}$ due to direct load on girder.

do

k.			
1.1	.26	1020	265
1.2	.43	1020	438
1.3	.51	515	263
1.4	.535	4330	2317
1.7	.33	1445	477
1.8	.215	1080	232
1.9	.10	1020	102
			4094
			1,239
			5,333

$-M_{\frac{2}{3}} = -5333 \cdot \frac{10.11}{10} = -5390$ kgm.

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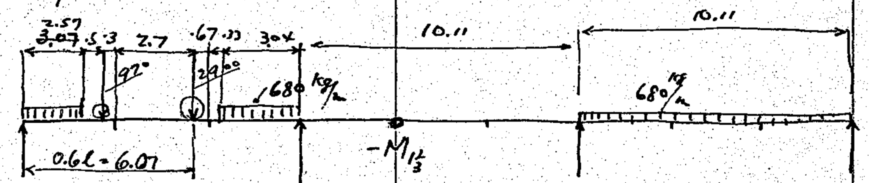
CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama-ken

Negative moment at 1/3 l right side of 2nd support in center span $-M_{1\frac{1}{3}}$.

$-M_{1\frac{1}{3}}$ due to panel load.



½ ... 585 1063 1630 568 573 1445 1445 573

K.			
.33	.46	1063	489
.67	.57	1630	929
2.33	.16	1145	183
2.67	.135	1145	155
			1756

$-M_{1\frac{1}{3}}$ due to direct load on girder.

For loading see $-M_{\frac{1}{2}}$ p.22.

K.			
.1	.15	1020	153
.2	.29	1080	~~296~~ 313
.3	.43	1445	621
.6	.60	4330	2600
.7	.57	515	294
.8	.45	1020	459
.9	.27	1020	276
2.1 to 2.9	1.10	1020	1122
			5838
			1756
			7594

$-M_{1\frac{1}{3}} = -7594 \times \frac{10.11}{10} = -7680$ kgm.

Negative moment at center of center span. $-M_{1\frac{1}{2}}$.

$-M_{1\frac{1}{2}}$ due to panel load.

Loading same as above.

K			
.33	.29	1063	308
.67	.37	1630	603
2.33	.37	1145	424
2.67	.29	1145	332
			1667

do.

$-M_{1\frac{1}{2}}$ due to load directly on girder.

K			
.1	.10	1020	102
.2	.19	1080	205
.3	.265	1445	383
.6	.382	4330	1655
.7	.375	515	193
.8	.29	1020	296
.9	.17	1020	173
2.1 to 2.9	2.48	1020	2530
			5537
			1667
			7204

$-M_{1\frac{1}{2}} = -7204 \times \frac{10.11}{10} = -7280$ kgm.

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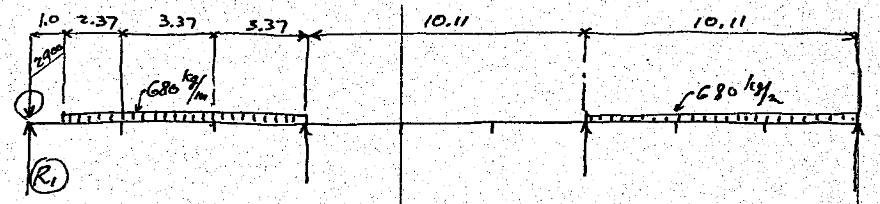
CALCULATIONS FOR

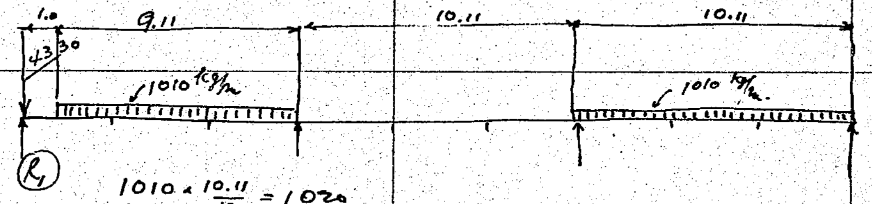
Design of Nakagawa-Bashi for Okayama-ken.

Live Load Shears and Reactions
End Reaction R_1 & Shear R_{1R}.
R_1 due to panel concentration

K.			
~~.00~~	1.00	1733	1733
~~.33~~	.59	1096	647
.67	.23	1145	263
2.33	.25	1145	286
2.67	.25	1145	286
			3215



566 1047
2900 1145 2290 1145 1145 2290 2290 1145
3466 2192 2290 1145 1145 2290 2290 1145
1/2 --- 1733 1096 1145 573 573 1145 1145 573



$1010 \times \frac{10.11}{10} = 1,020$

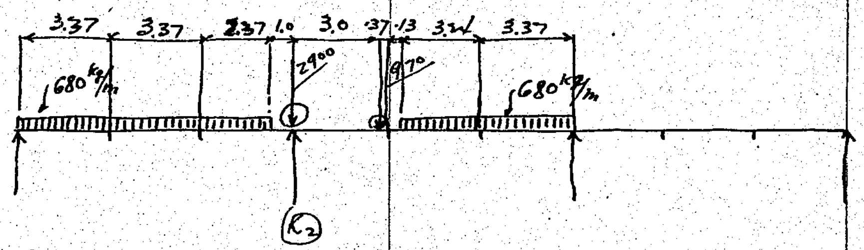
R_1 due to direct load on girder.

K.			
.00	1.00	4,330	4,330
.10	.874	535	468
.2to.9	2.966	1,020	3,025
2.1to2.9	.165	1,020	168
			7991 kg
			3215

R_1 = 11206 call this 11210 kg
less 1733÷2 = say 770
End shear R_{1R} = 10440 kg

Reaction at 2nd Support R_2.
R_2 due to panel concentrations.

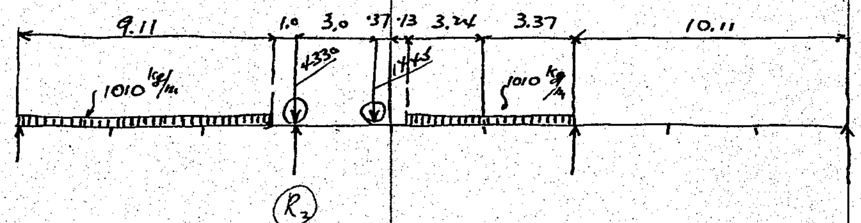
K.			
.33	.51	1145	584
.67	.89	1096	976
1.00	1.00	1787	1787
1.33	.77	960	739
1.67	.23	1145	263
			4349



1145 ~~573~~ 2290 1145 1047 ~~566~~ 1058 1145 1145 1144
2900
107 863
1145 2290 2192 3573 1921 2289 1145
1/2 --- 573 1145 1096 1787 960 1145 573

R_2 due to direct load on girder.

K.			
.1to.8	4.982	1,020	5080
.9	.997	510	509
1.0	1.000	4330	4330
1.297	.800	1445	1,155
1.4	.696	1050	731
1.5to1.9	1.635	1020	1668
			13473
			4349



R_2 = 17822 call this 17820 kg

Shear R_{2L}
R_{2L} due to panel concentrations.

Loading same as above for R_2.

K.			
.33	.412	1145	472
.67	.764	1096	837
1.00	1.000	1787	1787
1.33	.078	960	75
1.67	.054	1145	62
			3233

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R_{2L} due to direct load on main Beam.

Loading same as for R_2 page 24.

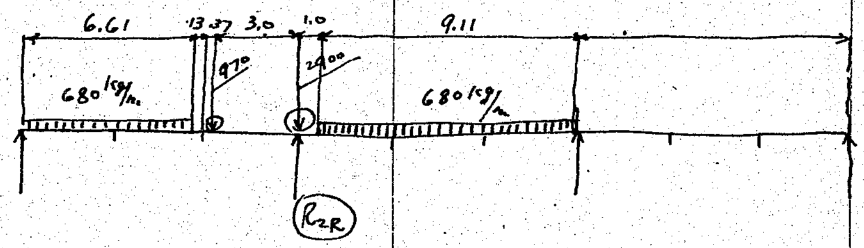
K.			
.1 to .8	4.114	1020	4,195
.9	.946	510	482
1.0	1.000	4330	4,330
1.297	.076	1,445	110
1.4	.080	1050	84
1.5 to 1.9	.235	1020	240.
			9441
			3233

R_{2L} = 12,674 call this 12,670 kg

Reaction R_{2R}.

R_{2R} due to panel concentrations.

K.	~~.098~~		
.33	.098	1145	112
.67	.122	960	117
1.00	1.000	1787	1787
1.33	.765	1096	838
1.67	.309	1145	354
			3208



$\frac{1}{2}$..573 1145 960 1787 1096 1145 573

R_{2R} due to direct load on main beam.

K			
0.1 to 0.5	1020	4.25	434
0.6	1050	.128	134
0.703	1,445	.120	173
1.00	4330	1.000	4330
1.1	510	.924	471
1.2 to 1.9	1,020	3.576	3648
			9190
			3208

R_{2R} = 12,398 call this 12,400 kg

Summary of Dead and Live Load moments. (in Kgm)

	$+M_{\frac{1}{3}}$	$+M_{\frac{4}{10}}$	$+M_{\frac{1}{2}}$	$-M_{\frac{1}{2}}$	$+M_{\frac{2}{3}}$	$-M_{\frac{2}{3}}$	$-M_2$	$+M_2$	$+M_{1\frac{1}{3}}$	$-M_{1\frac{1}{3}}$	$+M_{1\frac{1}{2}}$	$-M_{1\frac{1}{2}}$
Dead Load	+25,070	+25,110	+23,190	+23,190	+14,710	+14,710	-31,010	-31,010	+4280	+4280	+7720	+7,720
Live Load	+17,560	+17,900	+17,480	- 4030	+15,480	- 5390	-17,100	+ 2790	+12,900	-7680	+13,660	-7280
	+42,630	+43,010	+40,670	+19,160	+30,190	+9,320	+48,110	-28,220	+17,180	-3400	+21,380	+440

Summary of Dead and Live Load Shears and Reactions. (in kg.)

	R_1	R_{1R}	R_2	R_{2L}	R_{2R}
Dead Load	13,180	12,290	34,590	18,840	15,780
Live Load.	11,210	10,440	17,820	12,670	12,400
	24390	22,730	52,410	31,510	28,180

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CALCULATIONS FOR

Design of Nakagawa Bashi for Okayama-Ken.

Equivalent Uniform Loads:-

Dead Load:-

Uniform load directly on main beam = 2350 kg per lin meter.

Cross beam concentrations 2590 kg

Conversing to uniform load $\frac{2590}{337}$ = $\frac{770}{3120}$ kg per lin meter. $= w_d$.

$w_d l^2 = 3120 \cdot 10.11^2 = 318800$

Live Load:-

Pos. moment $+M_{\frac{1}{3}} = +17560$ kgm

for equivalent unif. load $M = .0942\,pl^2$

Equiv. unif. load $P_{+\frac{1}{3}} = \frac{17560}{.0942 \cdot 10.11^2} = 1825$ kg, $P_{+\frac{1}{3}} l^2 = 186500$

Pos. moment $+M_{\frac{4}{10}} = +17900$ kgm

for equivalent unif. load $M = .10\,pl^2$

Equiv. unif. load $P_{+\frac{4}{10}} = \frac{17900}{.10 \cdot 10.11^2} = \cancel{1830}$ 1752, $P_{+\frac{4}{10}} l^2 = 1\cancel{87}$ 79 000

Pos. moment $+M_{\frac{1}{2}} = +17480$ kgm.

for Equiv. unif. load, $M = .10\,pl^2$

Equiv. unif. load $P_{+\frac{1}{2}} = \frac{17480}{.10 \cdot 10.11^2} = 1712$, $P_{+\frac{1}{2}} l^2 = 175000$

Pos. moment $+M_{\frac{2}{3}} = +15480$ kgm.

for Equiv. unif. load $M = .0783\,pl^2$

Equiv. unif. load $P_{+\frac{2}{3}} = \frac{15480}{.0783 \cdot 10.11^2} = 1935$, $P_{+\frac{2}{3}} l^2 = 197800$

Pos. moment $+M_2 = +2790$ kgm.

for equiv. unif. load. $M = .01667\,pl^2$

Equiv. unif. load $P_{+1} = \frac{2790}{.01667 \cdot 10.11^2} = 1638$, $P_{+1} l^2 = 167500$

Pos. moment $+M_{1\frac{1}{3}} = +12900$ kgm

for unif load. $M = .0616\,pl^2$

Equiv. unif. load $P_{+1\frac{1}{3}} = \frac{12900}{.0616 \cdot 10.11^2} = 2050$, $P_{+1\frac{1}{3}} l^2 = 209200$

Pos. moment $+M_{1\frac{1}{2}} = +13660$ kgm.

for Equiv. unif. load, $M = .075\,pl^2$

Equiv. unif. load $P_{+1\frac{1}{2}} = \frac{13660}{.075 \cdot 10.11^2} = 1783$, $P_{+1\frac{1}{2}} l^2 = 182500$

Neg. moment $-M_{\frac{1}{2}} = -4030$ kgm.

for Equiv. unif. load, $M = .025\,pl^2$

Equiv. ~~&~~ unif. load, $P_{-\frac{1}{2}} = \frac{4030}{.025 \cdot 10.11^2} = \cancel{16}$ 1578, $P_{-\frac{1}{2}} l^2 = 161200$

Neg. moment $-M_{\frac{2}{3}} = -5390$ kgm.

for Equiv. unif. load, $M = .0333\,pl^2$

Equiv. unif. load $P_{-\frac{2}{3}} = \frac{5390}{.0333 \cdot 10.11^2} = 1583$, $P_{-\frac{2}{3}} l^2 = 161800$

Neg. moment $-M_2 = -17100$ kgm

for Equiv. unif. load, $M = .11667\,pl^2$

Equiv. unif. load $P_{-1} = \frac{17100}{.11667 \cdot 10.11^2} = 1433$, $P_{-1} l^2 = 146500$

Design of Nakagawa-Bashi for Okayama-ken.

Neg. moment $-M_{1\frac{1}{3}} = -7680$ kgm
for Equivalent unif. load, $M = .0500\,pl^2$
Equiv. unif. load $P_{1\frac{1}{3}} = \dfrac{7680}{.05 \times 10.11^2} = 1{,}502$, $P_{1\frac{1}{3}} l^2 = 153600$

Neg. moment $-M_{1\frac{1}{2}} = 7280$ kgm
for Equiv. unif. load, $M = .0500\,pl^2$
Equiv. unif. load $P_{1\frac{1}{2}} = \dfrac{7280}{.05 \times 10.11^2} = 1{,}425$, $P_{1\frac{1}{2}} l^2 = 145{,}600$

Moments at several points through main Beam calculated by Equivalent uniform loads.

		Dead Load moments			Live Load negative moments			Live Load Positive moments			
	x/l	Coef. of $Wd\,l^2$	By Equiv. load	By actual load	Coef. of Pl^2	By Equiv. load	By actual load	Coef. of	Pl^2	By Equiv. load	by Actual load
End span	.10	+ .035 $Wd\,l^2$	+ 11,170		.005 $P_{\frac{1}{3}}l^2$	− 810		.040	$P_{\frac{1}{3}}l^2$	+ 7,460	
	.20	+ .060	+ 19,130		.010 〃	− 1,610		.070	〃	+ 13,050	
	.30	+ .075	+ 23,900		.015 〃	− 2,420		.090	〃	+ 16,800	
	.33	+ .0778	+ 24,800	+ 25,070	.0167 〃	− 2,690		.0942	$P_{\frac{1}{3}}l^2$	+ 17,560	+ 17,560
	.40	+ .080	+ 25,500	+ 25,110	.020 〃	− 3,230		.100	$P_{\frac{4}{10}}l^2$	+ 17,900	+ 17,900
	.50	+ .075	+ 23,900	+ 23,190	.025 $P_{\frac{1}{2}}l^2$	− 4,030	−4,030	.100	$P_{\frac{1}{2}}l^2$	+ 17,480	+ 17,480
	.60	+ .060	+ 19,130		.030 $P_{\frac{2}{3}}l^2$	− 4,860		.090	$P_{\frac{2}{3}}l^2$	+ 17,800	
	.67	+ .0438	+ 13,970	+ 14,710	.0333 $P_{\frac{2}{3}}l^2$	− 5,390	− 5,390	.0783	$P_{\frac{2}{3}}l^2$	+ 15,480	+ 15,480
	.70	+ .035	+ 11,170		.035 〃	− 5,660		.070	〃	+ 13,850	
	.80	+ .000	+ 0		.04022 〃	− 6,510		.04022	〃	+ 7,950	
	.85	− .02125	− 6770		.04898 P_1l^2	− 7,180		.02773	P_1l^2	+ 4,640	
	.90	− .045	− 14,350		.06542 〃	− 9,580		.02042	〃	+ 3,420	
	.95	− .07125	− 22,720		.08831 〃	− 13,620		.01706	〃	+ 2,860	
	1.00	− .100	− 31,880	− 31,010	.11667 P_1l^2	− 17,100	− 17,100	.01667	P_1l^2	+ 2,790	+ 2,790
Center span	1.05	− .07625	− 24,300		.09033 〃	− 13,940		.01408	〃	+ 2,360	
	1.10	− .0550	− 17,540		.07014 〃	− 10,820		.01514	〃	+ 2,540	
	1.15	− .03625	− 11,560		.05678 〃	− 8,320		.02053	〃	+ 3,440	
	1.20	− .0200	− 6,380		.0500 $P_{1\frac{1}{3}}l^2$	− 7,680		.0300	$P_{1\frac{1}{3}}l^2$	+ 6,270	
	1.2764	− .0000	− 0		.0500 〃	− 7,680		.0500	〃	+ 10,460	
	1.300	+ .0050	+ 1,600		.0500 〃	− 7,680		.0550	〃	+ 11,510	
	1.33	+ .0118	+ 3,760	+ 4,280	.0500 $P_{1\frac{1}{3}}l^2$	− 7,680	− 7,680	.0616	$P_{1\frac{1}{3}}l^2$	+ 12,900	+ 12,900
	1.40	+ .0200	+ 6,380		.0500 〃	− 7,680		.0700	〃	+ 14,650	
	1.50	+ .0250	+ 7,970	+ 7,720	.0500 $P_{1\frac{1}{2}}l^2$	− 7,280	− 7,280	.0750	$P_{1\frac{1}{2}}l^2$	+ 13,660	+ 13,660

Summary of Dead and Live Load moments.

	x/l	Negative moment	Positive moment		x/l	Negative moment	Positive moment
End span	.10	+ 10,360	+ 18,630	Center span	1.05	− 38,240	− 21,940
	.20	+ 17,520	+ 32,180		1.10	− 28,360	− 15,000
	.30	+ 21,480	+ 40,700		1.15	− 19,880	− 8,120
	.33	+ 22,380	+ 42,630		1.20	− 14,060	− 110
	.40	+ 21,880	+ 43,010		1.2764	− 7,680	+ 10,460
	.50	+ 19,160	+ 40,670		1.30	− 6,080	+ 13,110
	.60	+ 14,270	+ 36,930		1.33	− 3,400	+ 17,180
	.67	+ 9,320	+ 30,190		1.40	− 1,300	+ 21,030
	.70	+ 5,510	+ 25,020		1.50	+ 440	+ 21,380
	.80	− 6,510	+ 7,950				
	.85	− 13,950	− 2,130				
	.90	− 23,930	− 10,930				
	.95	− 36,340	− 19,860				
	1.00	− 48,110	− 28,220				

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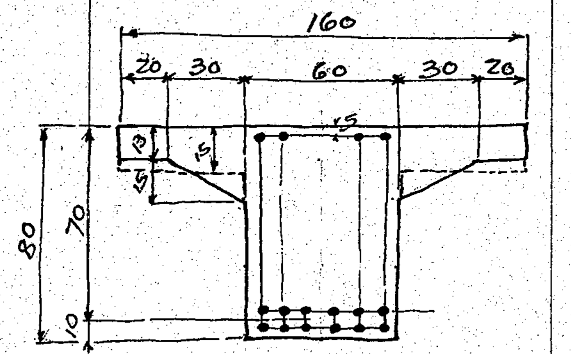
Design of Nakagawa-Bashi for Okayama-ken.

Sections of main girder.



Section at $\frac{1}{3}l$ from left support.

moment $+M_{\frac{l}{3}} = +42,630$ kgm

approximate steel area req'd $= \dfrac{42,630 \times 100}{1200 \times \frac{7}{8} \times 70} = 58.0\ \text{cm}^2$

use 12-25mmφ bars = 58.9 cm²

Steel ratio $p = \dfrac{58.9}{160 \times 70} = .00526$, $p' = \frac{1}{3}p = .00175$

t/d = say $15/70 = .215 = \Delta$, $d'/d = 5/70 = .0715$, $\Delta^2 = .0462$

$$k = \frac{p + p'\left(\frac{d'}{d}\right) + \frac{\Delta^2}{2n}}{p + p' + \frac{\Delta}{n}} = \frac{.00526 + \overset{.000125}{.00175 \times .0715} + \overset{.00154}{\frac{.0462}{30}}}{.00526 + .00175 + \overset{.01433}{\frac{.215}{15}}} = \frac{.006925}{.02134} = .3245$$

$$j = \frac{\Delta(2k-\Delta) - \frac{\Delta^2}{3}(3k - 2\Delta) + 2p'n\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)}{\Delta(2k-\Delta) + 2p'n\left(k - \frac{d'}{d}\right)}$$

[注意 分子第二項 $\frac{\Delta^2}{3}(2k-2\Delta)$ ト書イテ アルハ $\frac{\Delta^2}{3}(3k-2\Delta)$ ノ誤リ]

$$= \frac{\overset{.0933}{.215 \times .434} - \overset{.00837}{.0154 \times .5435} + \overset{.01233}{.0525 \times .253 \times .9285}}{.0933 + .01328} = .912$$

$$f_s = \frac{M}{A_s jd} = \frac{42630 \times 100}{58.9 \times .912 \times 70} = 1135\ \text{kg/cm}^2 \quad \text{ok}$$

$$f_c = \frac{f_s k}{n(1-k)} = \frac{1135 \times .3245}{15(1 - .3245)} = 36.4\ \text{kg/cm}^2 \quad \text{ok.}$$

Section at $4/10\,l$ from left support.

moment $= +43,010$ kgm.

Steel area req'd $= \dfrac{43010 \times 100}{1200 \times .912 \times 70} = 56.6\ \text{cm}^2$

use 12-25mmφ bars = 58.9 cm²

Section at $\frac{1}{2}l$ from left support

moment $= +40670$ kgm.

Steel area req'd $= \dfrac{40670 \times 100}{1200 \times .912 \times 70} = 53.1\ \text{cm}^2$

use 12-25mmφ bars = 58.9 cm²

Section at $\frac{2}{3}l$ from left support.

moment $= +30,190$ kgm

Steel area req'd $= \dfrac{30190 \times 100}{1200 \times .9 \times 70} = 40.0\ \text{cm}^2$

use 10-25mmφ bars = 49.1 cm²

Section at center of center span.

moment $= +21,380$ kgm.

Steel area req'd. $= \dfrac{21380 \times 100}{1200 \times .9 \times 70} = 28.3\ \text{cm}^2$

use 6-25mmφ bars = 29.5 cm²
or 8-22mmφ bars = 30.4 cm²

Section at $\frac{1}{3}l$ right side of 2nd support.

moment $= +17180$ kgm

Steel area req'd $= \dfrac{17180 \times 100}{1200 \times .9 \times 70} = 22.7\ \text{cm}^2$

use 6-25mmφ bars = 29.5 cm²
or ~~8~~ 6-22mmφ " = ~~30.4~~ 22.8 cm²

at 0.2l from left support 10-25φ = 49.1
" 0.3l " " 12-25φ = 58.9

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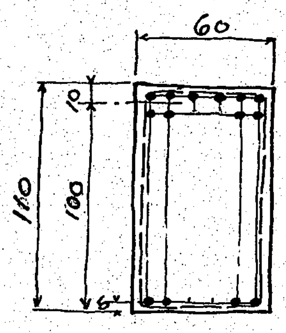
CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama-Ken

Section of Beam for negative moments.



Section at 2nd Support.

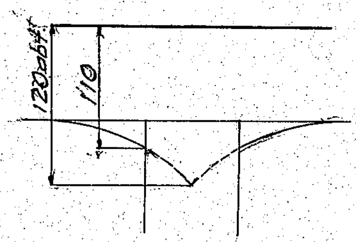
moment $= -M_2 = -48110$ kgm. End Shear $= R_{2L} = 31510$ kg, $R_{2R} = 28180$ kg

Effective depth req'd $= \sqrt{\dfrac{48110 \times 100}{60 \times 7.18}} = 105.7$ cm

Try 100 cm effective depth with 10 cm insulation at top.

Steel area req'd $= \dfrac{48110 \times 100}{1200 \times \frac{7}{8} \times 100} = 45.8$ cm²

Use 10 - 25 mmϕ bars = 49.1 cm² at ~~bottom~~ top
4 - 25ϕ " = 19.65 " ~~top~~ bottom



Steel ratio $p = \dfrac{49.1}{60 \times 100} = .0082$, $p' = \dfrac{19.65}{60 \times 100} = .00328$, $d'/d = 5/100 = .05$

From the prepared diagrams, $k = .355$, $j = .895$

$f_s = \dfrac{48110 \times 100}{49.1 \times .895 \times 100} = 1095$ kg/cm² ok

$f_c = \dfrac{1095 \times .355}{15(1-.355)} = 40.2$ kg/cm² ok

unit shear $= \dfrac{31510}{60 \times .895 \times 100} = 5.98$ kg/cm² use stirrups

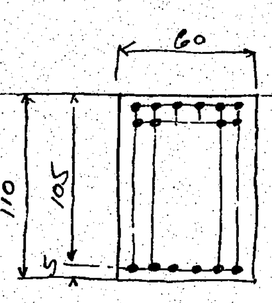
unit bond $= \dfrac{31510}{7.85 \times 10 \times .895 \times 100} = 4.49$ " ok.

16 mmϕ U-stirrups. $A_s = 2 \times 2.01 = 4.02$ cm²

Stirrup spacing at support $= \dfrac{3}{2} \cdot \dfrac{4.02 \times 1200 \times .895 \times 100}{31510} = 20.5$ cm left side of 2nd support

$20.5 \times \dfrac{31510}{28180} = 23.0$ cm right side " "

Section at left support.



End shear $R_{1R} = 22730$ kg

unit shear $= \dfrac{22730}{60 \times \frac{7}{8} \times 105} = 4.12$ kg/cm² use stirrups.

use 16 mmϕ U-stirrups. $A_s = 2 \times 2.01 = 4.02$ cm²

Stirrup spacing at support $= \dfrac{3}{2} \cdot \dfrac{4.02 \times 1200 \times .875 \times 105}{22730} = 29.2$ cm

unit bond $= \dfrac{22730}{6 \times 7.85 \times \frac{7}{8} \times 105} = 5.25$ kg/cm² ok.

Section at 0.9 l from left support

moment $= -23930$ kgm

Steel area req'd $= \dfrac{23930 \times 100}{1200 \times \frac{7}{8} \times 80} = 28.5$ cm²

use 6 - 25ϕ bars = 29.5 cm²

Section at 0.95 l from left support.

m = -36340 kgm. steel area = 38.5 cm²

use 10 - 25 mmϕ bars = 49.1 cm² 49.1

Section at 0.85 l from 2nd support in center span.

m = -38240 kgm Steel area req'd = 40.5 cm² 40.5

use 8 - 25 mmϕ bars = 39.3 cm²

Section at 0.1 l from 2nd support

m = -28360 kgm Steel area req'd = 33.8 cm² 33.8

use 8 - 25 mmϕ = 39.3 cm² 39.3

Section at 1.2 l from 2nd Support.

m = -14060 kgm, Steel area req'd = 19.2 cm² 19.2

use 4 - 25 mmϕ = 19.7 cm² 19.7

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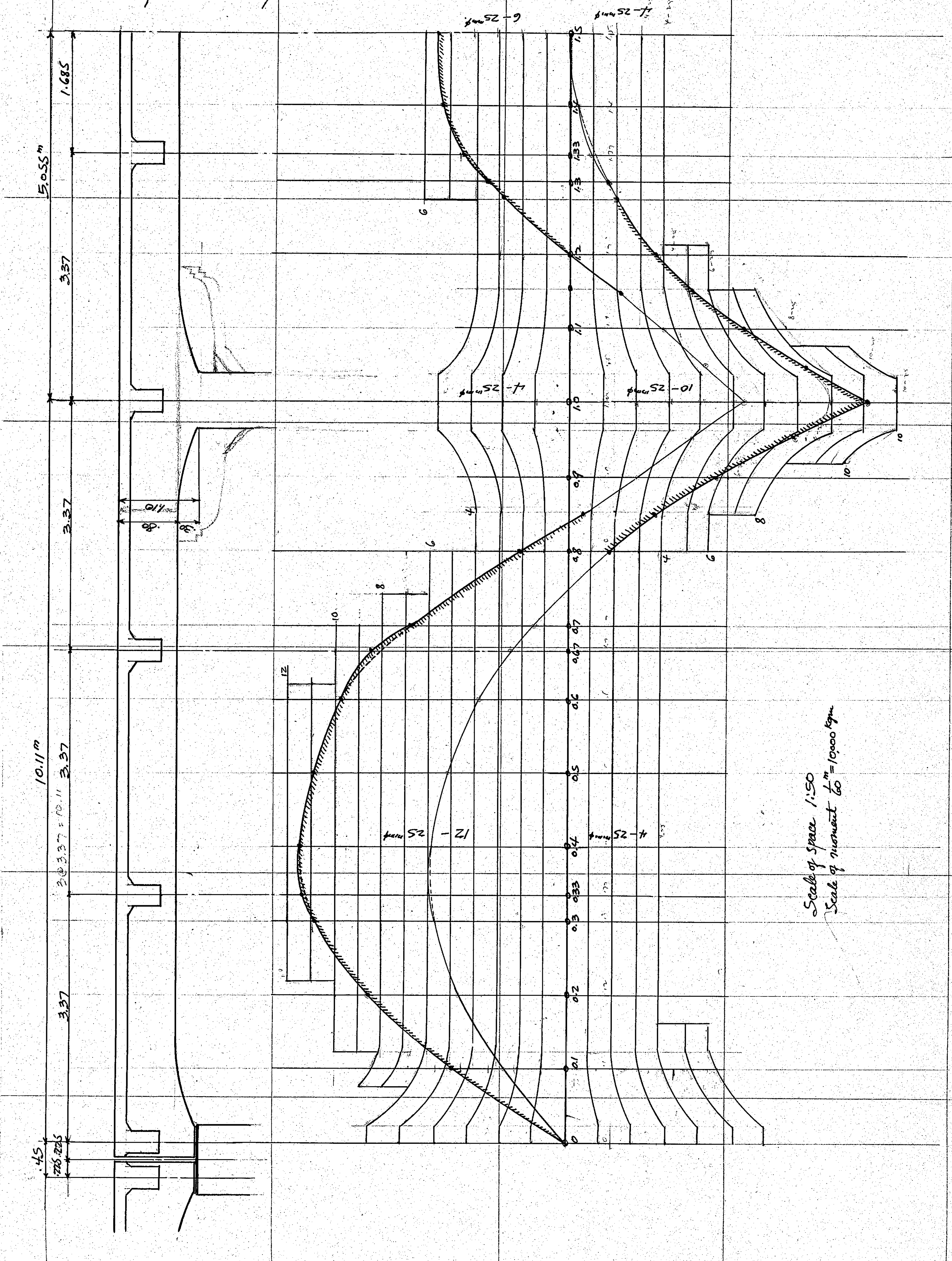
CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama-ken.

Bending moment diagram.



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MADE BY K. Inaba DATE 4-8-20 FILE NO ______

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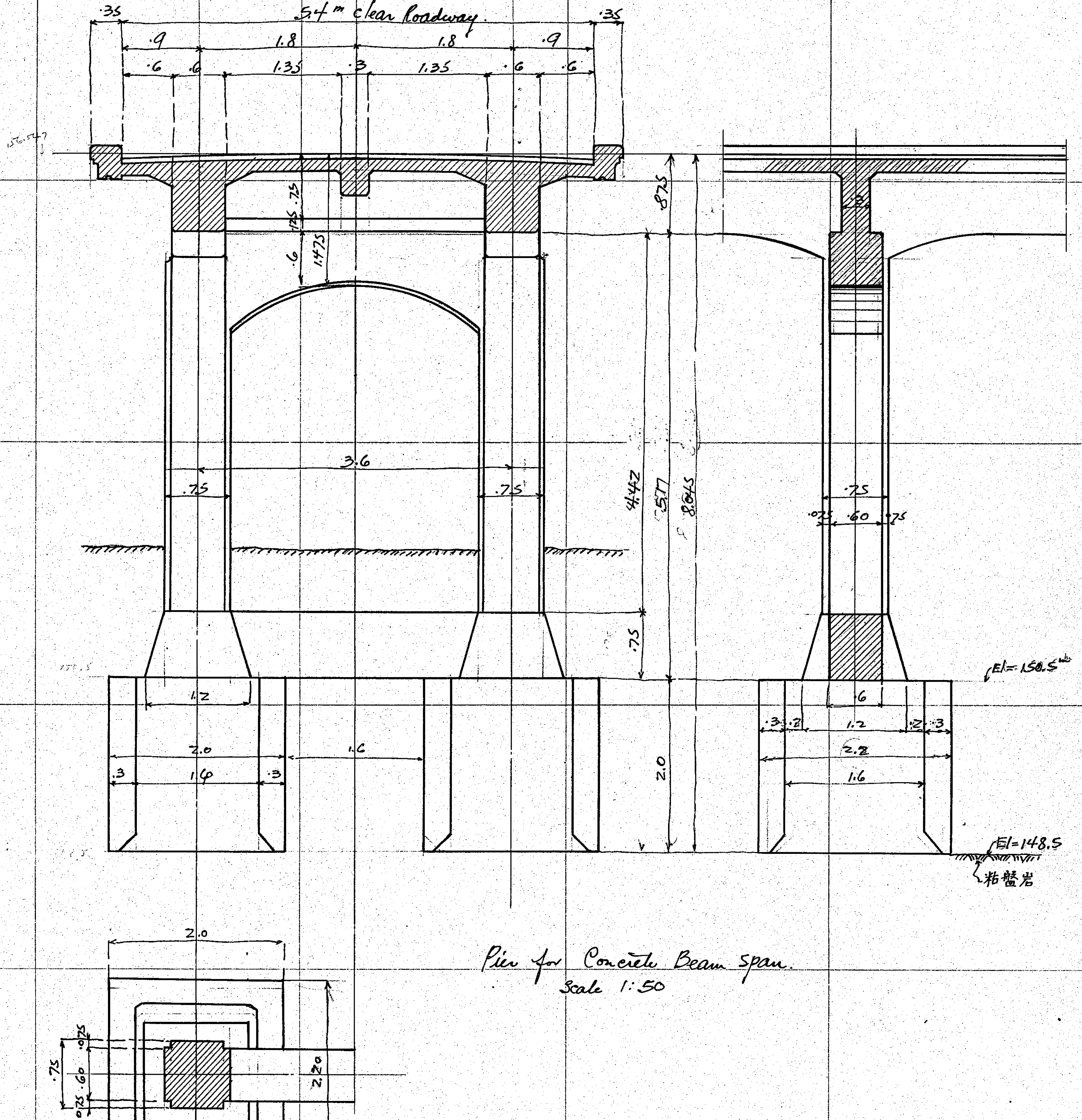
CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama-ken.

Design of Pier for Concrete Beam spans.

General details of pier is as shown on sketch below.



Pier for Concrete Beam span.
Scale 1:50

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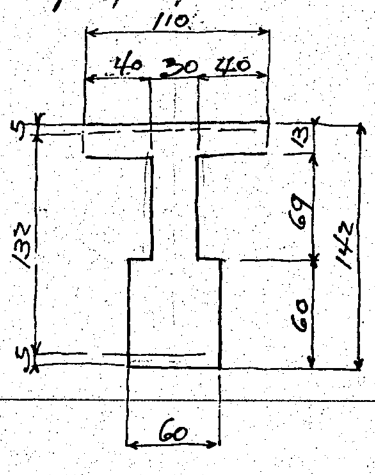
CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama-Ken.

Design of Top Beam. Span length = 3.6 meters.



Dead Load:-

Floor and pavement $440 \times 1.1 = 484$

Stem of Beam. $.3 \times .69 @ 2400 = 497$

" $.6 \times .85 @$ " $= 1223$

2204

Fillet and misc. details say. 16

2220 kg per lin meter.

D.L. moment due to wt. of Beam $= \frac{1}{10} \times 2220 \times 3.6^2 = 2880$ kgm.

D.L. Shear " " " $= \frac{1}{2} \times 2220 \times 3.6 = 4000$ kg

Stringer concentration on Beam. 2160 kg see page 11.

D.L. moment due to Stringer concentration $= \frac{1}{4} \cdot 2160 \times 3.6 = 1945$

For continuity of Beam moment $= .8 \times 1945 = 1555$ kgm.

End shear $= 2160 \div 2 = 1080$ kg

Summary of D.L. moments and end shears.

	pos. moment at center		neg. moment at end	End shear.
Weight of Beam.	2880	say $\times \frac{10}{12}$	2400	4000
Stringer concentration	1555	" "	1295	1080
	4435 kgm		3695 kgm	5080 kg

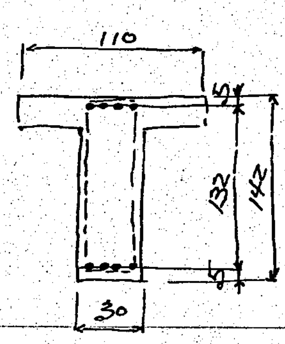
Effect of overhanging slab neglected.

Live Load — Live Load moments and shears same as for intermediate cross beam, page 12.

Summary of Dead and Live Load moments and End Shears.

	Pos. moment at center	Neg. moment at end	End shear.
Dead Load	4435	3695	5080
Live Load.	3781	3147	5138
	8216	6842	10218
In round numbers.	8220 kgm	6840 kgm	10220 kg

Assumed section at center of span.



moment $= + 8220$ kgm

Steel area req'd $= \frac{8220 \times 100}{1200 \times \frac{7}{8} \times 137} = 5.71$ cm²

use 4-16mmφ bars $= 8.04$ cm² on top and bottom.

Steel ratio $= \frac{5.71}{110 \times 137} = .00038 = p = p'$, $t/d = 13/137 = .095$

neutral axis in the web.

approx. $k = .15$, $j = .95$

$f_s = \frac{8220 \times 100}{8.04 \times .95 \times 137} = 785$ kg/cm² ok.

$f_c = \frac{785 \times .15}{15(1 - .15)} = 9.2$ kg/cm² ok.

End shear $= 10220$ kg Effective depth say 190 cm

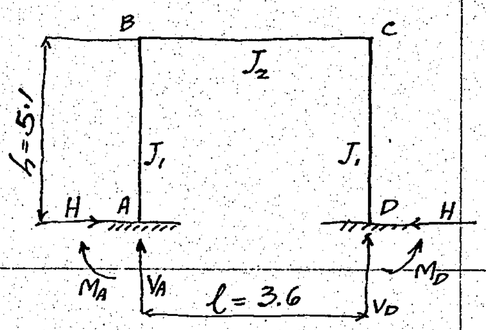
unit shear $= \frac{10220}{30 \times \frac{7}{8} \times 190} = 2.1$ kg/cm² ok.

unit bond $= \frac{10220}{5.03 \times 4 \times \frac{7}{8} \times 190} = 3.1$ " ok.

CALCULATIONS FOR

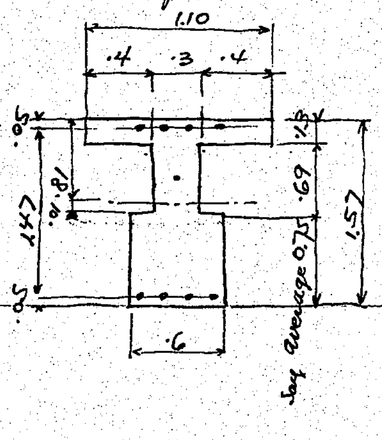
Design of Nakagawa-Bashi for Okayama-ken.

Design of Pier Frame.



Moment of inertias J.

Moment of inertia of Top Beam J_2



Center of gravity of the section.

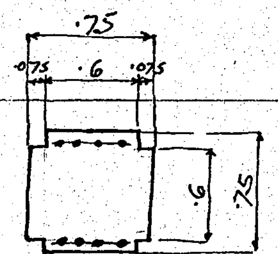
$1.1 \times .13 = .143 \times .065 = .0093$
$.3 \times .69 = .207 \times .475 = .0983$
$.6 \times .75 = .450 \times 1.195 = .5380$
$.800 \times .81 = .6456$

J_2.

$\frac{1.1 \times .13^3}{12} + .143 \times .745^2 = .0002 + .0793 = .0795$
$\frac{.3 \times .69^3}{12} + .207 \times .335^2 = .0081 + .0232 = .0313$
$\frac{.6 \times .75^3}{12} + .450 \times .385^2 = .0211 + .0667 = .0878$
Steel $.0008 \times 14 \times .76^2 = .0065 = .0065$
" $.0008 \times 14 \times .71^2 = .0056 = .0056$
$.2107\ m^4$

$h = 5.10 + .01 = 5.11$
Call this 5.1^m

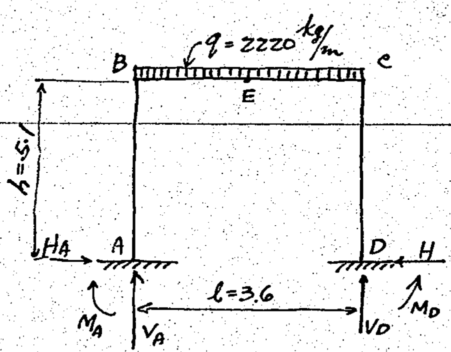
Moment of inertia of Columns J_1.



$\frac{.6 \times .75^3}{12} = .0211$
$\frac{.15 \times .6^3}{12} = .0027$
Steel say $.028 \times .325^2 \times 2 = .0059$
$.0297\ m^4$

$$k = \frac{J_2}{J_1} \cdot \frac{h}{l} = \frac{.2107}{.0297} \times \frac{5.1}{3.6} \doteqdot 10.0$$

Dead Load Stresses on Frame.



Moment due to Dead weight of Top beam. $q = 2220$ kg/lin m page 31.
Referring to Kleinlogel's Rahmenformeln. Page 88 – 95.

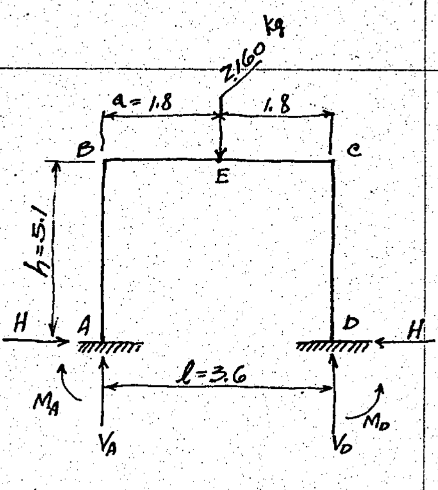
$V_A = V_D = \frac{ql}{2} = \frac{2220 \times 3.6}{2} = 4,000$ kg

$H_A = H_D = \frac{ql^2}{4h(k+2)} = \frac{2220 \times 3.6^2}{4 \times 5.1 \times 12} = 118$ kg

$M_B = M_C = -\frac{ql^2}{6(k+2)} = -\frac{2220 \times 3.6^2}{6 \times 12} = -400$ kgm

$M_A = M_D = +\frac{ql^2}{12(k+2)} = +200$ kgm.

$M_E = \frac{ql^2}{8} - \frac{ql^2}{6(k+2)} = \frac{2220 \times 3.6^2}{8} - 400 = +3200$ kgm.



Moment due to D.L. stringer concentration.

Stringer concentration $= 2160$ kg page 11.

$V_A = V_D = \frac{P}{2} = \frac{2160}{2} = 1,080$ kg $\quad \delta = \frac{a}{l} = \frac{1}{2}$

$H_A = H_D = \frac{3Pl}{8h(k+2)} = \frac{3 \times 2160 \times 3.6}{8 \times 5.1 \times 12} = 47.6$ call this 48 kg.

$M_A = +\frac{Pl}{8} \cdot \frac{5k - 1 + 2\delta(k+2)}{(k+2)(6k+1)} = \frac{2160 \times 3.6}{8} \times \frac{49 + 12}{12 \times 61} = +81.$ kgm $= M_D$.

$M_B = M_C = M_A - Hh = +81 - 48 \times 5.1 = -164$ kgm

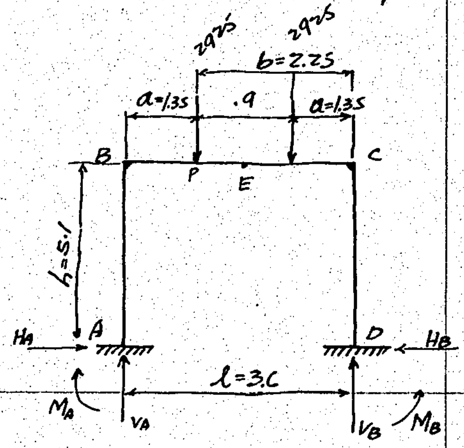
$M_E = M_A - Hh + V_A \cdot a = +81 - 48 \times 5.1 + 1080 \times 1.8 = +1,781$ kgm.

Summary of Dead Load Stresses.

	$M_A = M_D$	$M_B = M_C$	M_E	$H_A = H_D$	$V_A = V_D$
Due to wt. of Top beam	+200	−400	+3200	118	4000
" " Stringer concentration	81	−164	1781	48	1080
	+281 kgm	−564 kgm	+4981 kgm	166 kg	5080 kg

Design of Nakagawa-Bashi for Okayama-ken.

Live Load Stresses on frame:-



Moment due to Rear wheel of motor truck.

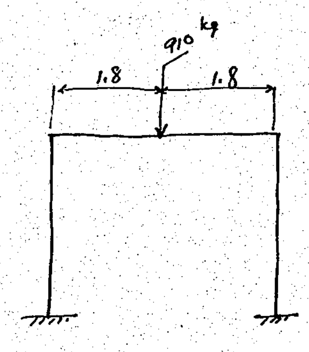
$V_A = P = 2925^{kg} = V_B$

$H_A = H_B = \dfrac{3Pab}{hl(k+2)} = \dfrac{3 \times 2925 \times 1.35 \times 2.25}{5.1 \times 3.6 \times 12} = 121^{kg}$

$M_A = M_D = +H\dfrac{h}{3} = 121 \times \dfrac{5.1}{3} = +206^{kgm}$

$M_B = M_C = -H \cdot \dfrac{2h}{3} = -412^{kgm}$

$M_E = Pa - M_B = 2925 \times 1.35 - 412 = +3535^{kgm} = M_P$



Moment due to Stringer concentration.

By propotion to D.L. ratio $\dfrac{910}{2160} = .421$

$V = 455^{kg}$

$H = 48 \times .421 = 20^{kg}$

$M_A = 81 \times " = +34^{kgm}$

$M_B = 164 \times " = -69 "$

$M_E = 1781 \times " = +750 "$

Summary of Live Load Stresses.

	$M_A=M_D$	$M_B=M_C$	M_E	$H_A=H_D$	$V_A=V_D$
Due to motor truck rear wheel	+206	-412	+3535	121	2925
" " front wheel and unif load.	+34	-69	+750	20	455
	+240 kgm	-481 kgm	+4285 kgm	141 kg	3380 kg

Summary for Dead and Live Load Stresses.

	$M_A=M_D$	$M_B=M_C$	M_E	$H_A=H_D$	$V_A=V_D$
Dead Load	+281	-564	+4981	166	5080
Live Load	+240	-481	+4285	141	3380
	+521 kgm	-1045 kgm	+9266 kgm	307 kg	8460 kg

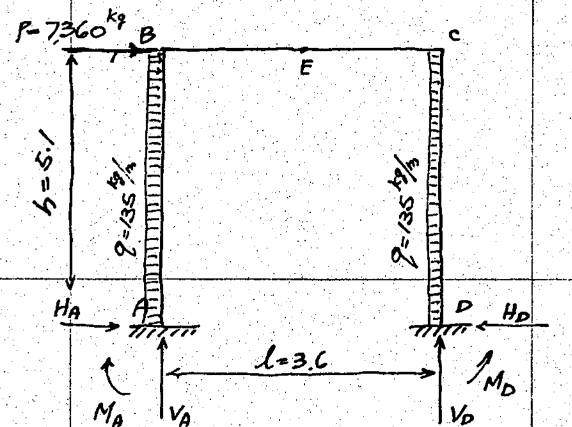
Max load on column $V_A = V_D$.

D.L. from main Beam (R_2)		34590 kg	page 25
" wt. of column	.75 × .75 × 5.1 @ 2400 =	6880	
		41470 kg	
Live Load from main girder		17820	page 25.
		59290 kg	

Seismic Stresses on Frame. k = 0.1 assumed.

Dead Load Floor 34590 on one column. 2 @ 34590 = 69180 kg on one pier

Seismic force = 34590 × .1 = 3459 kg 2 @ 3459 = 6920 " "



Top Beam say .6 × .85 × 3.6 @ 2400 = 4400 kg for one pier

Seismic force = 4400 × .1 = 440 kg

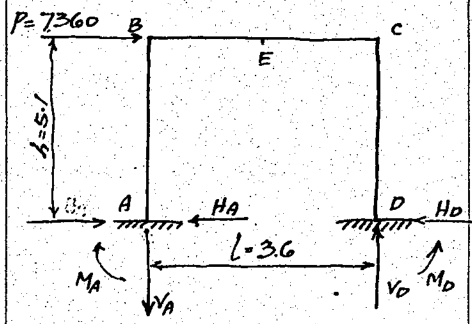
Column .75 × .75 @ 2400 = 1350 × 5.1 = 6880 kg

Seismic force 1350 × 0.1 = 135 kg/lin m for one col.

Summary of P. P = 6920 + 440 = 7360 kg

q q = 135 kg/m on both columns each

Design of Nakagawa-Bashi for Okayama ken.



Moment due to seismic force $P = 7360^{kg}$ on top of frame.

$$V_A = V_D = \mp \frac{3Phk}{l(6k+1)} = \mp \frac{3\times7360\times5.1\times10}{3.6\times61} = \mp 5130\ kg$$

$$H_D = H_A = \frac{P}{2} = \frac{7360}{2} = \pm 3680\ kg$$

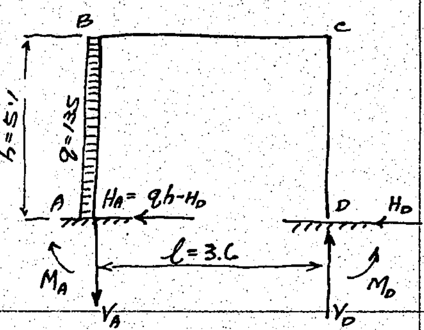
$$M_A = -\frac{Ph}{2}\cdot\frac{3k+1}{6k+1} = -\frac{7360\times5.1}{2}\times\frac{31}{61} = -9540\ kgm.$$

$$M_D = -M_A = +9540\ "$$

$$M_B = +\frac{Ph}{2}\cdot\frac{3k}{6k+1} = +\frac{7360\times5.1}{2}\times\frac{30}{61} = +9230\ "$$

$$M_C = -M_B = -9230\ "$$

$$M_E = 0.$$



Moment due to $q = 135^{kg/m}$ on AB. column only.

$$V_A = V_D = \mp \frac{qh^2k}{l(6k+1)} = \mp \frac{135\times5.1^2\times10}{3.6\times61} = \mp 160\ kg$$

$$H_A = -(qh - H_D) = -(135\times5.1-165) = -523^{kg}$$

$$H_D = +\frac{qh}{8}\cdot\frac{2k+3}{k+2} = \frac{135\times5.1}{8}\times\frac{23}{12} = +165^{kg}$$

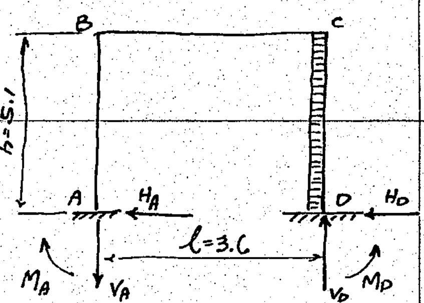
$$M_A = -\frac{qh^2}{24}\left(12 - \frac{5k+9}{k+2} - \frac{12k}{6k+1}\right) = -\frac{135\times5.1^2}{24}\left(12 - \frac{59}{12} - \frac{120}{61}\right) = -748\ kgm$$

$$\cancel{M_B = M_A - H_A h + \frac{qh^2}{2} = -748 + 165\times5.1 - \frac{135\times5.1^2}{2} = +165\ kgm}$$

$$M_D = +\frac{qh^2}{24}\left(+\frac{5k+9}{k+2} - \frac{12k}{6k+1}\right) = +\frac{135\times5.1^2}{24}\left(+\frac{59}{12} - \frac{120}{61}\right) = +432\ kgm.$$

$$M_C = M_D - H_D h = 432 - 165\times5.1 = -410\ kgm.$$

$$M_E = \frac{M_B + M_C}{2} = \frac{165 - 410}{2} = -123\ "$$



Moment due to $q = 135^{kg/m}$ on Column CD. only.

$V_A = V_D = \mp\ 160\ kg$
$H_A = -\ 165\ "$
$H_D = +\ 523\ "$
$M_A = -\ 432\ kgm$
$M_D = +\ 748\ "$
$M_B = +\ 410\ "$
$M_C = -\ 165\ "$
$M_E = +\ 123\ "$

Summary of Seismic Stresses on frame.

Due to.	M_A	M_B	M_C	M_D	M_E	H_A	H_D	V_A	V_D
Floor + Top beam.	-9540	+9230	-9230	+9540	0	-3680	+3680	-5130	+5130
Hor. load on col. AB	-748	+165	-410	+432	-123	-523	+165	-160	+160
" " " CD.	-432	+410	-165	+748	+123	-165	+523	-160	+160
	-10,720 kgm	+9805 kgm	-9805 kgm	+10720 kgm	~~-246~~ kgm 0	-4370 kg	+4370 kg	-5450 kg	+5450 kg

Load on column V_A + V_D.

D.L. from main beam (R_2) 34590
" wt. of column 6880
+ 41470 kg

$V_A = 41470 - 5450 = +36020\ kg$
$V_D = 41470 + 5450 = +46920$
$V_B = 34590 - 5450 = +$ ~~30050~~ 29140
$V_C = 34590 + 5450 = +40040$

Design of Nakagawa-Bashi for Okayama-ken.

Summary of Dead Load and Seismic Stresses.

	M_A	M_B	M_C	M_D	M_E	H_A	H_D	V_A	V_D
Dead Load	+281	−564	−564	+281	+4,981	+166	+166	+5,080	+5,080
Seismic	−10,720	+9,805	−9,805	+10,720	−246	−4,370	+4,370	−5,450	+5,450
Total in round nos.	−10,440 kgm	+9,240 "	−10,370 "	+11,000 "	+4,988 kgm	−4,205 kg	−4,535 "	−370 "	+10,530 kg

Sections at several points of Frame.

Top Beam :— Checking the section assumed on page 31.

At center of span moment = +9,266 kgm at normal state M_E P33
= +4,988 " during earthquake. " P35

$f_s = \dfrac{9266 \times 100}{8.04 \times .95 \times 137} = 886$ kg/cm² ok

$f_c = \dfrac{886 \times .15}{15(1-.15)} = 10.4$ " ok.

At End of span moment = −1,045 kgm at normal state $M_B = M_C$
= +9,240 " during earthquake M_B
or −10,370 " " " M_C

Shear = 8,460 kg at normal state.
" = 10,530 " during earthquake

$f_s = \dfrac{10370 \times 100}{8.04 \times \frac{7}{8} \times 190} = 775$ kg/cm² ok.

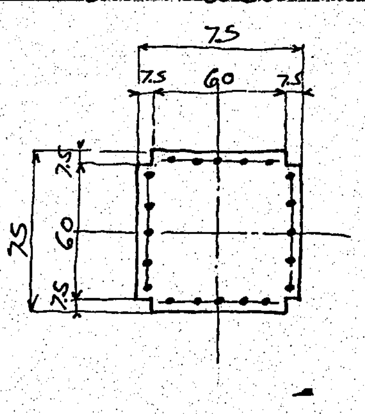
$f_c = \dfrac{775 \times .15}{15(1-.15)} = 9.1$ " ok.

Shear & bond ok.

Assumed Section top beam is ample.

Columns:— At Bottom of Column. A & D.

Assumed section



moment = +521 kgm at normal state $M_A = M_D$
= +11,000 " during earthquake M_D
= −10,440 " " " M_A

max. load = +59,290 kg at normal state $V_A = V_D$
= +46,920 during earthquake V_D
= +36,020 " V_A

Sectional area $75 \times 60 = 4500$
$15 \times 60 = \underline{900}$
5400 cm²

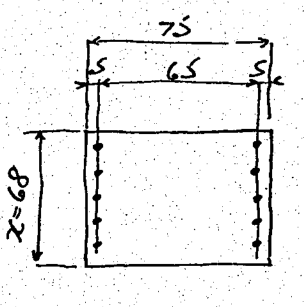
Equivalent rectangular section of equal moment of inertia.

moment of inertia $\dfrac{.60 \times .75^3}{12} = .0211$

$\dfrac{.15 \times .6^3}{12} = \underline{.0027}$

$I = .0238$ m⁴

Equivalent section



$I = \dfrac{x \times .75^3}{12} = .0238$ $\quad x = \dfrac{.0238 \times 12}{.75^3} = .675$ m

call this 68 cm

Steel, try 5 − 19φ bars = 14.18 cm² on both sides each. (revised to 4−22φ = 15.2 4-9-24.)

$p_o = 2p = \dfrac{14.18 \times 2}{68 \times 75} = .00556$ $\quad d'/h = 5/75 = .067$

Stresses at normal state $m = 521$ kgm $\quad V = 59290$ kg $= N$.

Eccentricity $e = \dfrac{521 \times 100}{59290} = 1$ cm

$e/h = 1/75 = .013$

From the prepared diagrams

$k = .98$

$f_c = \dfrac{Nk}{bh} = \dfrac{59290 \times .98}{68 \times 75} = 11.4$ kg/cm² ok.

Design of Nakagawa-Bashi for Okayama-ken

Stresses during Earthquake. at A.

moment = 10,440 kgm , vert. load at A N = 36,020 kg Shear at D = 4,535 (max) (Seismic)

Ecc. ε = 10 440 ÷ 36020 = .29 m or 29 cm

ε/h = 29/75 = .387

From the prepared diagrams.

k = .57 , L = .115

d'/d	k	L
.05	.585	.1185
.067	.58	
.10	.56	.1118

$f_c = \frac{M}{Lbh^2} = \frac{10440 \times 100}{.115 \times 68 \times 75^2}$ = 23.8 kg/cm² ok.

$f_s = nf_c(\frac{d}{kh}-1) = 15 \times 23.8(\frac{70}{.57 \times 75}-1)$ = 229 kg/cm² ok. (.64)

unit shear = $\frac{4535}{.68 \times \frac{7}{8} \times 70}$ = 1.09 kg/cm² ok.

Unit bond = $\frac{4535}{5.97 \times 5 \times \frac{7}{8} \times 70}$ = 2.5 " ok.

At D.

moment = 11,000 kgm , Vert. load N = 46920 kg , Shear = 4535

Ecc. $\varepsilon = \frac{11000 \times 100}{46920}$ = 23.5 cm ε/h = 23.5/75 = .314

	k	L
.05	.70	.118
.067	.69	.116
.10	.695	.113

k = .69 , L = .116

$f_c = \frac{11000 \times 100}{.116 \times 68 \times 75^2}$ = 24.8 kg/cm² ok

$f_s = 15 \times 24.8(\frac{70}{.69 \times 75}-1)$ = 131 kg/cm² ok.

At top B & C.

The section at top assumed same as for bottom.

moment at B. = 9,240 kgm. N = 29,140 kg during earthquake.

" " C = 10,370 " N = 40040 "

at B.

Ecc. $\varepsilon = \frac{9240 \times 100}{29140}$ = 31.7 cm ε/h = 31.7 ÷ 75 = .423

P_0 = .00556 , d'/d = .067

~~k~~ L = .115 , k = .535

	k	L
.05	.545	.118
.067	.53	.115
.1	.52	.1105

$f_c = \frac{9240 \times 100}{.115 \times 68 \times 75^2}$ = 21.0 kg/cm² ok.

$f_s = 21.0 \times 15(\frac{70}{.535 \times 75}-1)$ = 234 " ok

at C

Ecc. $\varepsilon = \frac{10370 \times 100}{40040}$ = 26 cm ε/h = 26/75 = .345

	k	L
.05	.65	.1185
.067	.64	.1165
.1	.62	.1125

k = .64 , L = .116

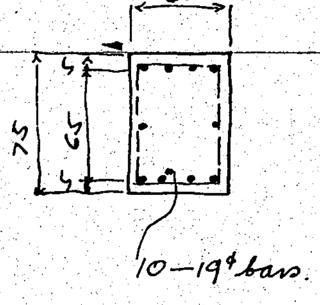
$f_c = \frac{10370 \times 100}{.116 \times 68 \times 75^2}$ = 23.4 kg/cm² ok

$f_s = 15 \times 23.4(\frac{70}{.64 \times 75}-1)$ = 162 " ok.

Bottom strut.

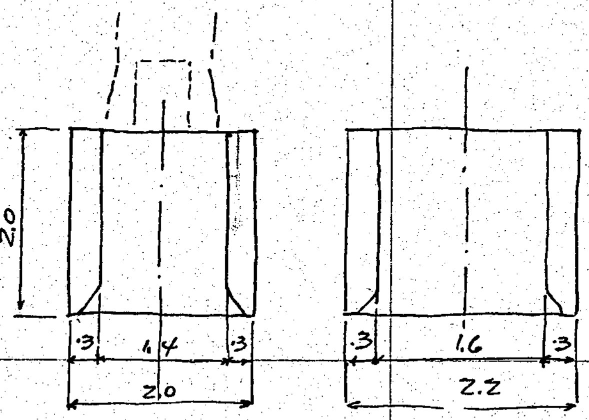
4,535 kg T or C.

use 60 × 75 cm strut with with 4 — 19 mm ϕ bars on top & bottom.



Design of Nakagawa-Bashi for Okayama Prefecture.

Design of Base for Fixed pier.



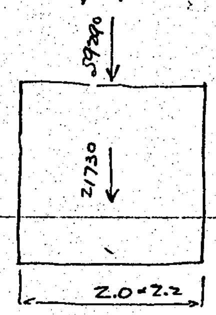
Section perpendicular to ℄ Bridge.

Section parallel to ℄ Bridge

Volume and weight of Base.

Strut.	.6 × .75 × 2.8 ÷ 2 =	.63	@2400 = 1,510 × 2.38 =		3,590
Shell of well.	2.0×2.2 = 4.40				
	1.4×1.6 = 2.24				
	2.16 × 2.0 =	4.32	@2400 = 10,370 × 1.0 =		10,370
Fill	1.4×1.6×2.0 =	4.48	@2200 = 9850 × 1.0 =		9,850
		9.43^{m^3}	21730^{kg} · 1.10^{m} =		23,810

Stability of Base. Case 1. Stability at normal state.



Superimposed Loads on Base D.L. = 41,470
L.L. = 17,820
59290 kg

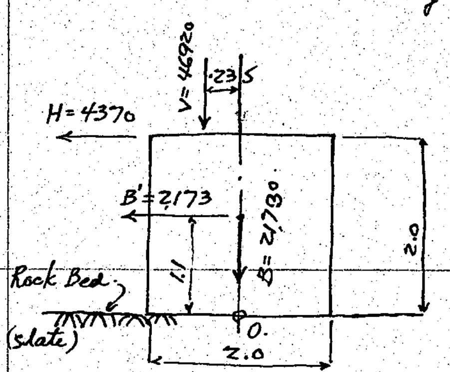
$m = 521^{kgm}$ neglected.
$H = 307^{kg}$ neglected.

Weight of Base 21,730
81,020 kg.

Bottom area of Base = 2.0×2.2 = 4.4^{0m}

Unit Bearing pressure on Bed. = 81,020 ÷ 4.4 = $18,420^{kg/m^2}$ (or $1.69^{ton/\square'}$) ok.

Case 2. Stability of Base during Earthquake. k = 0.1 assumed. (transverse direction).



moment from column = 11,000 kgm, vert. load on column = $46,920^{kg}$

Eccentricity Σ = 11000 ÷ 46920 = 0.235^{m} H = 4,370 kg

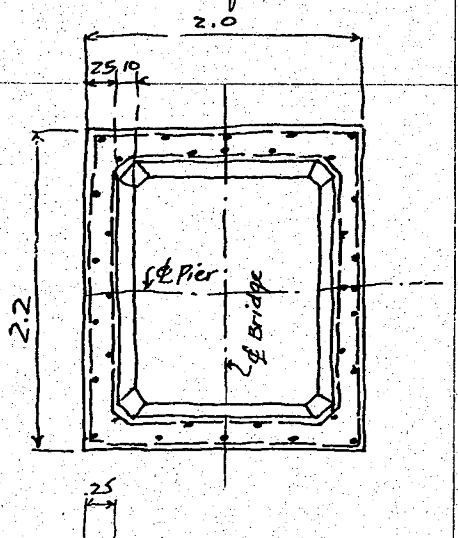
Taking moment about center of bed area O.

Loads	Hor forces	Vert. forces	Lev. arms	moments
V		46,920 ×	0.235 =	11,000
H	4370	×	2.0 =	8740
B		21730 ×	0 =	0
B'	2,173	×	1.1 =	2390
	6543^{kg}	68650^{kg} ÷	$.32^{m}$ =	22,130 kgm.

Resultant force within middle third.

max. toe pressure = $\frac{68650}{2.2 \times 2.0}\left(1 \pm \frac{6 \times .32}{2.0}\right)$ = $30,580^{kg/m^2}$ (or $2.8^{tons/\square'}$) ok.
or = 620 "

Section of well shell.



2.0×2.2 m well, thickness of Shell 25 cm at top and 35 cm at bottom.

Reinforcements.

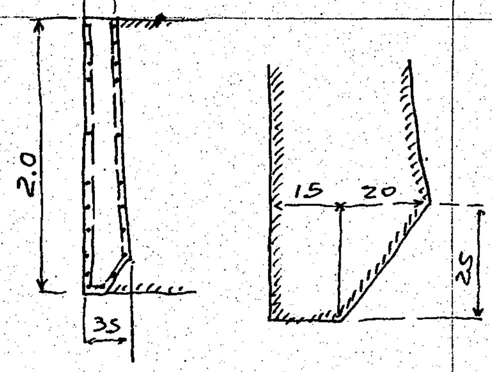
Horizontal bars 13 mm φ bars at 15 cm c/c at top and bottom
13 mm φ bars at 30 cm c/c for intermediate portion.

Vertical Bars. 13 mm φ bars at 50 cm c/c about on both sides.

No. Steel shoe for cutting edge.

Bond stress between shell and fill.

area of shell = 2.0 × 2.2 = 4.40
1.4 × 1.6 = 2.24
2.16^{0m}



upward pressure on shell at normal state = 2.16 × 18,420 = 39750^{kg}

bond area = 1.6 + 1.4 = 3.0 × 2 = 6.0 × 2.0 = 12.0^{0m}

unit bond = $\frac{39750}{12}$ = $3310^{kg/m^2}$ or ($680^{\#/\square'}$) ok.

Design of Nakagawa-Bashi for Okayama-ken.

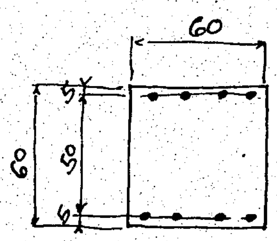
Design of Expansion Pier.

Details of Expansion pier same as for fixed pier except the top beam is separated from cross beam.

Assumed section of Top beam.

Center of span 60×60^{cm} effective depth 55^{cm} steel $4-22^{\phi} = 15.2^{cm^2}$

end of span 60×100 say " " 95 " " "



weight of top beam.

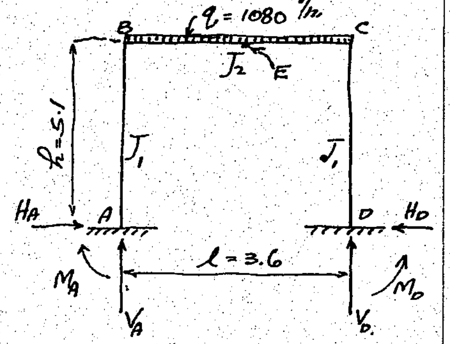
$.6 \times .75 @ 2400 = 1080$ kg/lin.m. average.

Moment of inertia of Top Beam average depth say 75 cm.

$J_2 = \frac{60 \times 75^3}{12} = 0.0211$

$.0015 \times .325 \times 2^2 = \underline{0.0003}$

$J_2 = 0.0214^{(m)^4}$



moment of inertia of column J_1 Section of col. same as for fixed pier page 32

$J_1 = 0.0297^{(m)^4}$

$$k = \frac{J_2}{J_1} \cdot \frac{h}{l} = \frac{.0214}{.0297} \cdot \frac{5.1}{3.6} = 1.02$$

Moment due to weight of Top Beam $q = 1080$ kg/lin.meter.

$V_A = V_D = \frac{ql}{2} = \frac{1080 \times 3.6}{2} = 1945$ kg.

$H_A = H_D = \frac{ql^2}{4h(k+2)} = \frac{1080 \times 3.6^2}{4 \times 5.1 \times 3.02} = 227$ kg

$M_B = M_C = -\frac{ql^2}{6(k+2)} = -\frac{1080 \times 3.6^2}{6 \times 3.02} = -773$ kgm

$M_A = M_D = +\frac{ql^2}{12(k+2)} = +\frac{1080 \times 3.6^2}{12 \times 3.02} = +387$ kgm.

$M_E = \frac{ql^2}{8} - \frac{ql^2}{6(k+2)} = \frac{1080 \times 3.6^2}{8} - \frac{1080 \times 3.6^2}{6 \times 3.02} = +978$ kgm

Dead Load on column due to main beam. $R_1 = \overset{13180}{\cancel{24390}} \times 2 = \overset{26360}{\cancel{48780}}$ kg on one column. P25

" wt. of column $.75 \times .75 @ 2400 = 1350 \times 5.1 = \underline{6880}$

33240 kg " " "

Live Load on column due to main beam $R_1 =$ say $2 \times 11210 = 22420$ kg " "

Summary of Dead Load and Live Load moments. &c.

	$M_A = M_D$	$M_B = M_C$	M_E	$H_A = H_D$	$V_A = V_D$
Dead Load.	+387 kgm	-773 kgm	+978 kgm	227 kg	1945 kg
Live Load.	—	—	—	—	
	+387	-773	+978	227	1945

max load on col.

		$V_A = V_D$		$V_B = V_C$
D.L.	Top beam	1945	28305	1945
"	main beam	26360		26360
~~col.~~ "	column	6880		
		35185 kg		28305
L.L.	main beam	22420		22420
		57605 kg.		50725 kg.

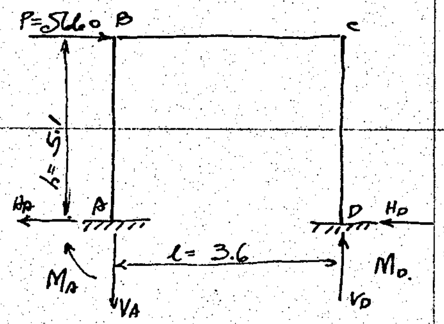
Design of Nakagawa-Bashi for Okayamaken

Seismic Stresses on Frame — Seismic coef. $K = 0.1$ assumed.

Dead load floor + Top beam $28305 \times 2 = 56610$ kg for one pier.
Seismic force $56610 \times .1 = 5660$ kg $= P$.

Column seismic force $= 1350 \times .1 = 135$ kg/m on each col. $= q$.

Moment due to seismic force $P = 5660$ kg on top.



$$V_A = V_D = \mp \frac{3Phk}{l(6k+1)} = \mp \frac{3 \times 5660 \times 5.1 \times 1.02}{3.6 \times 7.12} = \mp 3445 \text{ kg}$$

$$H_A = H_D = \mp \frac{P}{2} = \mp \frac{5660}{2} = \mp 2830 \text{ kg}$$

$$M_A = -\frac{Ph}{2} \cdot \frac{3k+1}{6k+1} = \frac{5660 \times 5.1}{2} \cdot \frac{4.06}{7.12} = -8230 \text{ kgm}$$

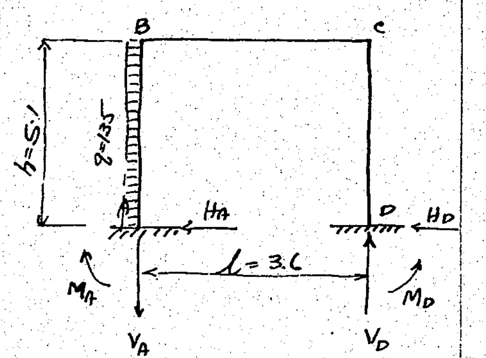
$$M_D = -M_A = +8230$$

$$M_B = +\frac{Ph}{2} \cdot \frac{3k}{6k+1} = \frac{5660 \times 5.1}{2} \cdot \frac{3.06}{7.12} = +6210$$

$$M_C = -M_B = -6210$$

$$M_E = 0$$

Moment due to Seismic force $q = 135$ kg/lin m. on column AB only



$$V_A = V_D = \mp \frac{qh^2k}{l(6k+1)} = \mp \frac{135 \times 5.1^2 \times 1.02}{3.6 \times 7.12} = \mp 140 \text{ kg}$$

$$H_D = +\frac{qh}{8} \cdot \frac{2k+3}{k+2} = +\frac{135 \times 5.1}{8} \cdot \frac{5.04}{3.02} = +144 \text{ kg}$$

$$H_A = -(qh - H_D) = -(135 \times 5.1 - 144) = -544 \text{ kg}$$

$$M_A = -\frac{qh^2}{24}\left(12 - \frac{5k+9}{k+2} - \frac{12k}{6k+1}\right) = -\frac{135 \times 5.1^2}{24}\left(12 - \frac{14.1}{3.02} - \frac{12.24}{7.12}\right) = -821 \text{ kgm}$$

5.61

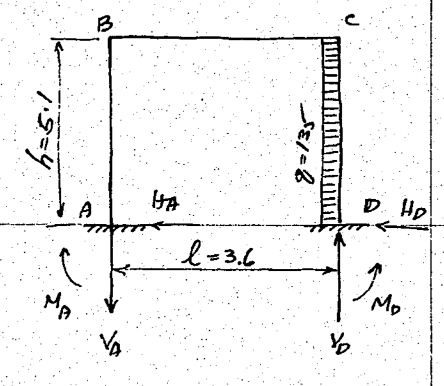
$$M_B = M_A - H_A h - \frac{qh^2}{2} = -821 + 544 \times 5.1 - \frac{135 \times 5.1^2}{2} = +197 \text{ kgm}$$

$$M_D = +\frac{qh^2}{24}\left(\frac{5k+9}{k+2} - \frac{12k}{6k+1}\right) = +\frac{135 \times 5.1^2}{24}\left(\frac{14.1}{3.02} - \frac{12.24}{7.12}\right) = +432 \text{ kgm}$$

$$M_C = M_D - H_D h = 432 - 144 \times 5.1 = -303$$

$$M_E = \frac{M_B + M_C}{2} = \frac{197 + (-303)}{2} = -53 \text{ kgm}$$

Moment due to q on column CD only.



$V_A = V_D = \mp 140$ kg
$H_A = -144$
$H_D = +544$
$M_A = -432$
$M_D = +821$
$M_B = +303$
$M_C = -197$
$M_E = +53$

Summary of D.L. and Seismic moment on frame.

	M_A	M_D	M_B	M_C	M_E	H_A	H_D	V_A	V_D
Seismic force Floor	−8230	+8230	+6210	−6210	0	−2830	+2830	−3445	+3445
〃 〃 P	−821	+432	+197	−303	−53	−544	+144	−140	+140
〃 〃 P	−432	+821	+303	−197	+53	−144	+544	−140	+140
Summary for Seismic stresses	−9483 kgm	+9483	+6710	−6710	0	−3518 kg	+3518	−3725	+3725
Dead load stresses.	+387	+387	−773	−773	+978	+227	+227	+35185	+35185
	9870	+9870	+5940	−7480	+980	−745	+3745	+31460 kg	+38910 kg
	−9100					3290			

$V_B + V_C$.

	V_B	V_C
$V_B + V_C$ for D.L.	= −3,445	+3,445
D.L. due to floor.	+28305	+28305
	+24,860 kg	+31,750 kg

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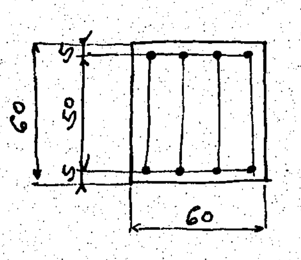
CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama-ken.

Sections at several points of Frame.

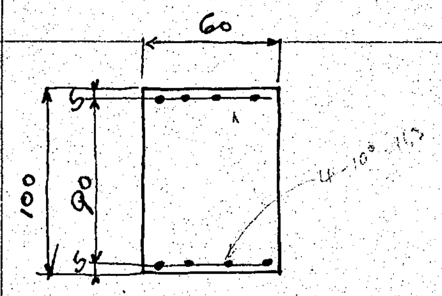
Top Beam :-



Section at Center of span moment = +980 kgm. at normal state.

Steel area req'd. = $\dfrac{980 \times 100}{1200 \times \frac{7}{8} \times 55} = 1.7^{cm^2}$

Section at End of span moment = + ~~9870~~ 5940 kgm during earthquake.
" = -7480 " "

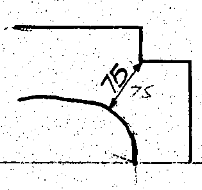


Shear seismic 3725
dead load 1945
5670 kg.

Steel area req'd = $\dfrac{7480 \times 100}{1200 \times 1.8 \times \frac{7}{8} \times 70} = 5.65^{cm^2}$

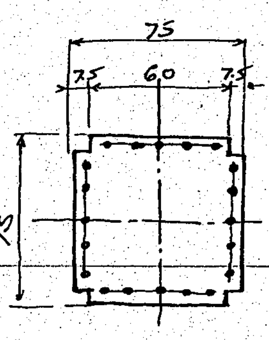
use 4-19φ bars = ~~15.2~~ 11.3 cm² on top and bottom each.

unit shear = $\dfrac{5670}{60 \times \frac{7}{8} \times 70} = 1.54^{kg/cm^2}$ ok



unit bond = $\dfrac{5670}{5.97 \times 4 \times \frac{7}{8} \times 70} = 3.90$ ok.

Columns:-
assumed section



At Bottom of column A + D.

			V.
moment =	+9870 kgm	during earthquake.	38910 kg
	-9100 "	"	31460 "

For equivalent section see page 36, Seismic. Shear 3745 kg

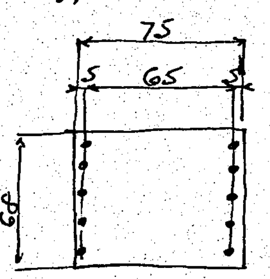
try 5-19mmφ bars = 14.18 cm² on both sides.

Ecc. $\varepsilon = \dfrac{9870 \times 100}{38910} = 25.4^{cm}$

$\varepsilon/h = 25.4/75 = .34$, $d'/h = 5/75 = .067$, $p = .00556$

$k = .64$, $L = .116$

Equivalent Section



$f_c = \dfrac{9870 \times 100}{.116 \times 68 \times 75^2} = 22.2^{kg/cm^2}$ ok.

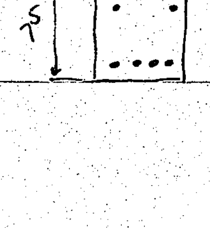
$f_s = 15 \times 22.2 \left(\dfrac{70}{.64 \times 75} - 1\right) = 156^{kg/cm^2}$ ok.

Shear and bond ok page 36.

At Top of column B + C.
Use same section as A + D.

Bottom strut

3745 kg T or C.
Use 60 × 75 cm strut with 4 - 19mmφ bars on top + bottom



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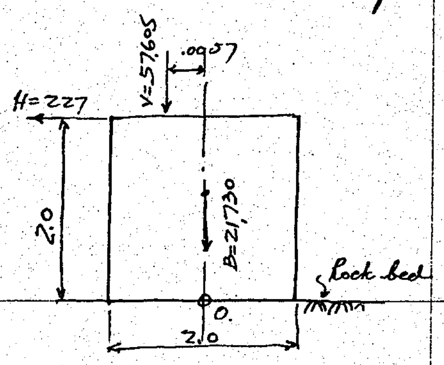
CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayamaken

Design of Base for Expansion Pier. Use same details as for Fixed Pier

Stability of Base.

Case 1. Stability at normal state.



Superimposed Loads on base D.L. = 35,185 kg
L.L. = 22,420 kg
57,605

Weight of base = 21,730
Total. 79,335 kg

Horizontal Load. 227 kg

moment from column = 387 kgm.

Eccentricity $\varepsilon = \frac{387}{57605} = 0.0057^m$

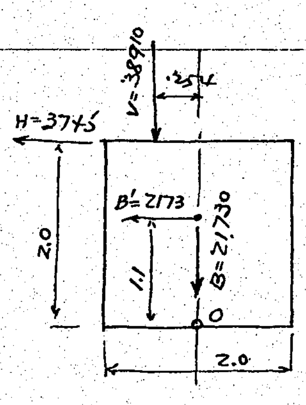
moment about point O. $227 \times 2.0 = 454$
387
841 kgm. $\varepsilon = \frac{841}{79335} = .0106$

max. bearing pressure $= \frac{79335}{2.2 \times 2.0}\left(1 \pm \frac{6 \times .0106}{2.0}\right) = 18600 \ kg/m^2$ or (1.70 tons/□') ok.

Case 2. Stability during Earthquake. $k = 0.1$. (transverse).

(D) moment from column $M_D = +9870$ kgm vert. load $V_D = 38,910$ kg $H_D = 3745$

Ecc. $\varepsilon = \frac{9870}{38910} = .254^m$



(A) moment from col. $M_A = -9,100$ kgm vert. load $V_A = 31,460$ kg H_A -3290

Ecc. $\varepsilon = \frac{9100}{31460} = .289^m$

Taking moment about O. for Base D.

Loads	Hor. forces	Vert. forces	Lev. arms	Moments
V.		38,910	× .254	= 9,870
H	3,745		× 2.0	= 7,490
B		21,730	× 0	= 0
B'	2173		× 1.1	= 2390
	5918	60,640 kg	× .326 m	= 19,750

Resultant force ~~outside~~ within of middle third.

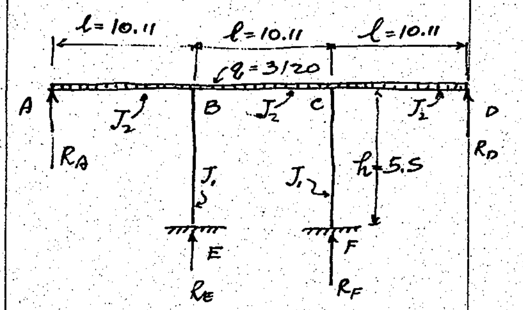
max. Bearing pressure $= \frac{60640}{2.2 \times 2.0}\left(1 \pm \frac{6 \times .326}{2.0}\right) = 27,300 \ kg/m^2$ or (2.5 tons/□') ok.

Use same details of well as for Fixed pier see page 37.

Design of Nakagawa-Bashi for Okayama-ken.

Approximate Column stresses for longitudinal frame of 3 spans. (both end supports free.)

Dead Load :-

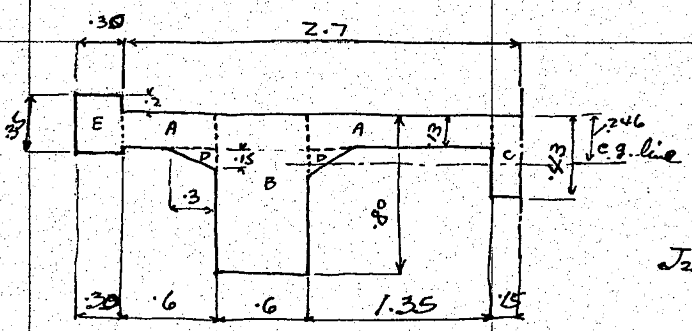


Cross Beam concentration = 2590 kg
Conversing to uniform load = 2590 ÷ 3.37 = 770 kg/m.
Unif. dead load = 2350
q = 3120 kg/lin m of span.

Moment of inertia J_2

Center of gravity.

A	1.95 · .13	=	.254 × .065	=	.0165
B	.6 × .8	=	.480 × .40	=	.1920
C	.15 × .43	=	.065 × .215	=	.0140
D	.3 × .15 ÷ 2 × 2	=	.045 × .18	=	.0081
E	.3 × .35	=	.105 × −.025	=	.0026
			.949 ㎡ × .246	=	.2332



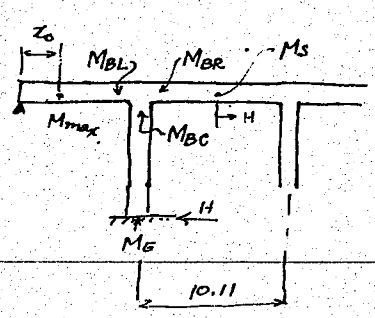
J_2.

A.	$1.95 \times .13^3 \div 12$ + $.254 \times .181^2$	=	.000357 + .00832 =	.008677
B.	$.6 \times .8^3 \div 12$ + $.48 \times .158^2$	=	.02560 + .0120 =	.037602
C	$.15 \times .43^3 \div 12$ + $.065 \times .031^2$	=	.000995 + .000625 =	.00162
D.	$.045 \times .066^2$	=	.000196 =	.000196
E	$.3 \times .35^3 \div 12$ + $.105 \times .271^2$	=	.00307 + .00772 =	.010790

J_2 = .05889 m⁴

J_1 See page 38. = 0.0297 m⁴

$$\frac{J_2 h}{J_1 l} = \frac{.05889}{.0297} \times \frac{5.5}{10.11} = 1.08 = k.$$



$$R_A = \frac{(3+4k)}{2(4+5k)} ql = \frac{7.32}{2 \times 9.4} \times 3120 \times 10.11 = 12280 \text{ kg}$$

$$H = -\frac{1}{4(4+5k)} ql^2 = -\frac{1}{4 \times 9.4} \times 3120 \times 10.11^2 = -8,490 \text{ kg}$$

$$M_{BR} = -\frac{(2+3k)}{6(4+5k)} ql^2 = -\frac{5.24}{6 \times 9.4} \times 3120 \times 10.11^2 = -29,600 \text{ kgm}$$

$$M_{BL} = -\frac{1+k}{2(4+5k)} ql^2 = -\frac{2.08}{2 \times 9.4} \times 3120 \times 10.11^2 = -35,250 \text{ kgm}$$

$$M_{BC} = \frac{1}{6(4+5k)} ql^2 = \frac{1}{6 \times 9.4} \times 3120 \times 10.11^2 = +5650 \text{ kgm}$$

$$M_E = M_F = -\frac{1}{12(4+5k)} ql^2 = -2825 \text{ kgm}$$

$$M_S = \frac{4+3k}{24(4+5k)} ql^2 = \frac{7.24}{24 \times 9.4} \times 3120 \times 10.11^2 = +10,200 \text{ kgm}$$

$$x_0 = \frac{3+4k}{2(4+5k)} \cancel{ql^2}\, l = \frac{7.32}{2 \times 9.4} \times \cancel{3120}\ 10.11 = \cancel{4.18}\text{ m } 3.94 \text{ m}$$

$$M_{max} = \frac{(3+2k)^2}{8(4+5k)^2} ql^2 = \frac{5.16^2}{8 \times 9.4^2} \times 3120 \times 10.11^2 = +12,000 \text{ kgm}$$

$$R_E = 1.5ql - R_A = 1.5 \times 3120 \times 10.11 - 12280 = 35,020 \text{ kg}$$

Live Load

Approx. equivalent unif. live load = 1433 kg/lin m. see page 26. (fa-M₂)

Stresses by proportion. 1433 ÷ 3120 = 0.46.

R_A	=	12280 × 0.46	=	5650
H	=	− 8490 × "	= −	3900
M_{BR}	=	− 29600 × "	= −	13620
M_{BL}	=	− 35250 × "	−	16220
M_{BC}	=	+ 5650 × "	+	2600
M_E	=	− 2825 × "	−	1300
M_S	=	+ 10200 × "	+	4690
M_{max}	=	+ 12000 × "	+	5520
R_E	=	35020 × "		16100

Design of Nakagawa-Bashi for Okayama ken.

Summary of Dead and Live Load moments + Reactions (approximate).

	M_{BR}	M_{BL}	M_{BC}	M_E	M_S	M_{max}	R_A	R_G	H
Dead Load	−29600	−35250	+5650	−2825	+10200	+12000	12280	35020	−8490
Live Load	−13620	−16220	+2600	−1300	+4690	+5520	5650	16100	−3900
	−43,220	−51,470	+8250	−4125	+14890	+17520	17930	51120	8800 / −12390

Approximate stresses on frame during earthquake $k = 0.1$ assumed

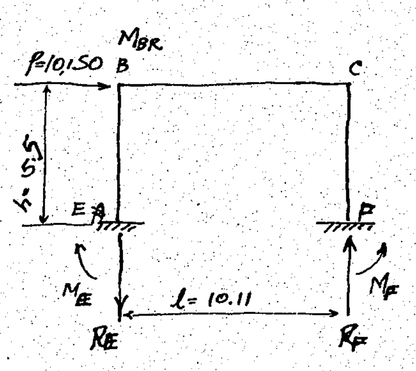
Dead load = 3120 kg per lin meter   P.42

Total Dead Load for 3 spans.

$= 3120 \times 10.11 \times 3 = 94600$ kg

$\frac{1}{2}$ wt of 2 cols. $= \underline{6900}$

$101,500$ kg



Seismic force $= 101,500 \times .1 = 10150$ kg on top $h = 5.975 - .475 = 5.5^m$ assumed

$K = \frac{J_2}{J_1} \cdot \frac{h}{l} = 1.08$ page 42

$R_E = \frac{3Phk}{l(6k+1)} = \frac{3 \times 10150 \times 5.5 \times 1.08}{10.11 \times 7.48} = 23900$ kg

$H = \frac{P}{2} = \frac{10150}{2} = 5080$ kg

$M_E = M_F = \mp \frac{Ph}{2} \cdot \frac{3k+1}{6k+1} = \mp \frac{10150 \times 5.5}{2} \times \frac{4.24}{7.48} = \mp 16,000$ kgm

$M_{BR} = \pm \frac{Ph}{2} \cdot \frac{3k}{6k+1} = \pm \frac{10150 \times 5.5}{2} \times \frac{3.24}{7.48} = \pm 12,100$ kgm.

Summary of Dead and Seismic moment + reactions.

	M_{BR}	M_{BC}	M_E	R_E	H
Dead Load	−29600	+5650	−2825	35020	−8490
Live Load	−12100	+12100	−16000	23900	−5080
					−690 --- column wt.
	−41,700 kgm	+17750"	−18825"	58920 kg	14260"

Check of main girder stress at second support. $M_{BR} = -51470$ kgm at normal state.

Reffer to page 29.

$f_s = \frac{51470 \times 100}{49.1 \times .895 \times 100} = 1170$ kg/cm² ok.

$f_c = \frac{1170 \times .355}{15(1-.355)} = 42.9$ kg/cm² ok.

The designed section of main girder is ample for all points.

Stresses of Column.

moment at top of column $= M_{BC} = +17750$ kgm during earthquake

" bottom " $= M_E = -18825$ " "

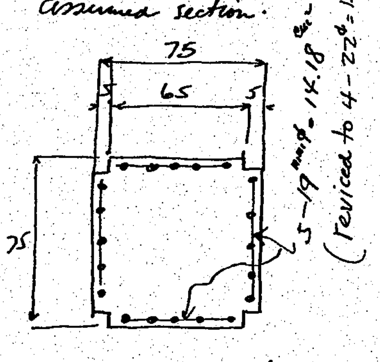
Column shall be designed at bottom and use same section throughout the length.

moment $= -18825$ kgm, $N = 58920$ kg, Shear $= 14260$ kg

Refer to page 40.

Eccentricity $\varepsilon = \frac{18825}{58920} = 0.32^m$ $\varepsilon/h = .32/.75 = .43$ $d'/h = .067$ $p_0 = .00556$

Assumed section.

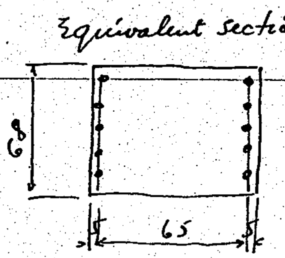


$k = .540$, $L = .116$

$f_c = \frac{18825 \times 100}{.116 \times 68 \times 75^2} = 42.5$ kg/cm² ok.

$f_s = 15 \times 42.5 \left(\frac{70}{.54 \times 75} - 1\right) = 465$ kg/cm²

Equivalent section



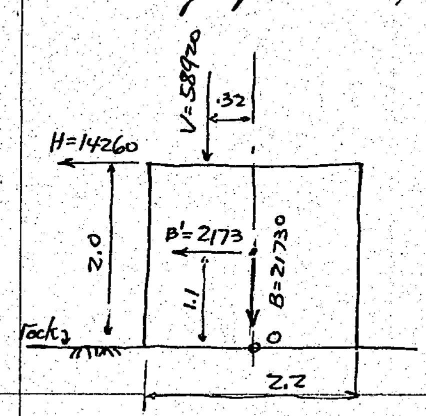
Unit Shear $= \frac{14260}{68 \times \frac{7}{8} \times 70} = 3.4$ kg/cm² ok.

Unit bond $= \frac{14260}{5.97 \times 5 \times \frac{7}{8} \times 70} = 7.8$ kg/cm² $< 6 \times 1.8$ ok.

assumed section ok.

Design of Nakagawa-Bashi for Okayama-Ken.

Stability of Fixed Pier Base during Longitudinal Earthquake. $k=0.1$



Moment from column $= 18825$ kgm, vert. load from col. $= 58920^{kg}$; $H = 14260$ kg

Ecc. $\varepsilon = \dfrac{18825}{58920} = 0.32^m$

Referring to page 37, take moment about 0.

Loads	Hor. forces	Vert. forces		Lev. arm		Moment
V		58,920	×	.32	=	18825
H	14,260		×	2.00	=	28,520
B		21,730	×	0	=	0
B'	2173		×	1.1	=	2390
	16433^{kg}	$80,650^{kg}$	×	0.617^m	=	49,735

Resultant force outside of middle third, neglecting tension at heel.

pressure area $= (1.1 - .617) \times 3 \times 2 = 2.898^{\square m}$

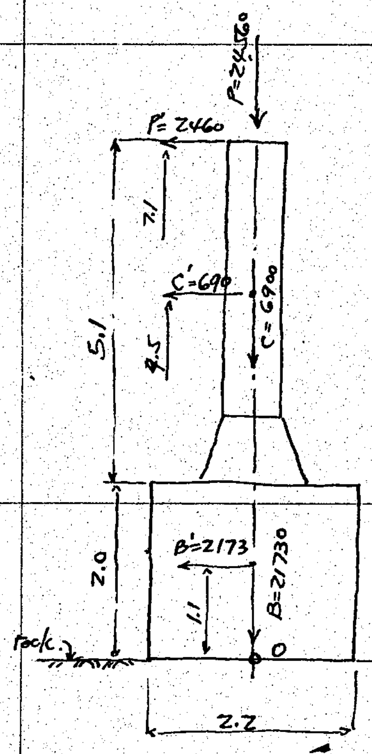
max. Bearing pressure $= \dfrac{80650 \times 2}{2.898} = 55600\ \text{kg/m}^2$ or $(5.08\ \text{tons}/\square')$ ok.

Stability of Expansion Pier during the Longitudinal Earthquake.

Dead Load on column from main girder $= 12280 \times 2 = 24,560^{kg} = P$.

wt. of column $= .75 \times .75 \times 5.1 = 2.87 @ 2400 = 6900$ " $= C$

wt. of Base, Same as for fixed pier. $= 21,730$ " $= B$.



Coeff. of friction of shoe for main beam assumed 0.10

Seismic force on top of col $= 24560 \times .1 = 2460$ kg $= P'$

Taking moment about 0

Loads	Hor. forces	Vert. forces		Lev. arm		Moments
P		24,560	×	0	=	0
P'	2,460		×	7.10	=	17,470
C		6,900	×	0	=	0
C'	690		×	4.50	=	3100
B		21,730	×	0	=	0
B'	2173		×	1.1	=	2390
	5,323	$53,190^{kg}$	×	$.432^m$	=	22,960

Resultant force outside of middle third, neglecting tension.

Pressure area $= (1.1 - .432) \times 3 \times 2.0 = 4.01^{\square m}$

max. Bearing pressure $= \dfrac{53190 \times 2}{4.01} = 26500\ \text{kg/m}^2$ (or $2.42\ \text{tons}/\square'$) ok.

Use the same base as for fixed pier on page 37.

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CALCULATIONS FOR

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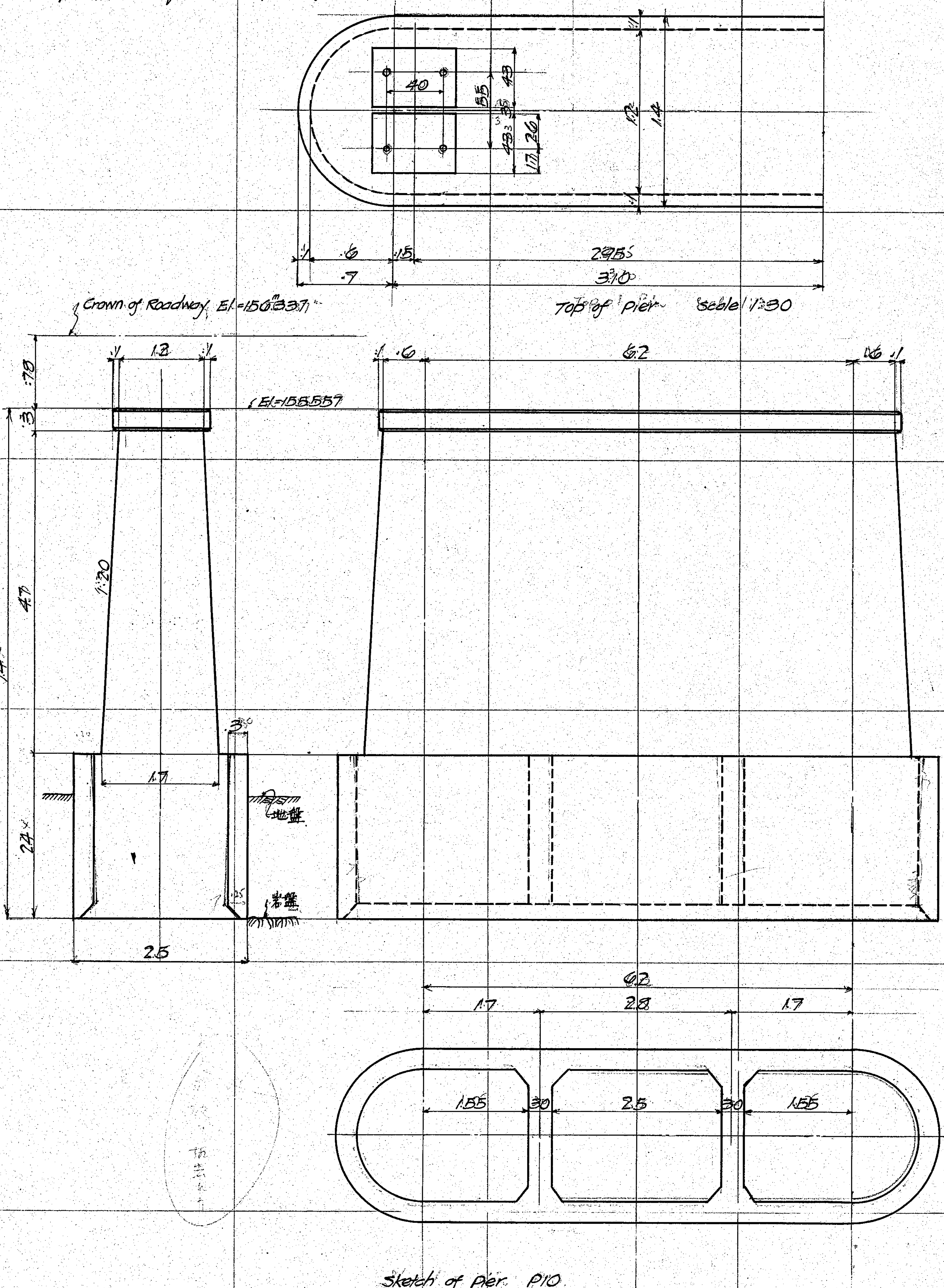
Design of Nakagawa-Bashi for Okayama Prefecture.
Design of Pier for Steel plate girder span. (P10)



Sketch of Pier P10
Scale 1:60

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CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama ken.

Volume, weight and center of gravity of pier.

Coping	0.3 × 1.4 × 6.2 =	2.61	@2400 =	6260 ×	4.85 =	30,350
"	0.3 × 1.4$^{\phi}$ =	0.46	@ " =	1,100 ×	4.85 =	5,330
Shaft	1.45 × 4.7 × 6.2 =	42.25	@ " =	101,440 ×	2.21 =	224,100
"	1.45$^{\phi}$ × 4.7 =	7.76	@ " =	18,600 ×	2.09 =	38,870
		52.98	S =	127,400 kg ×	2.35 m =	298,600

well Shell — average sectional area.

Side walls.	.3 × 6.2 × 2 =	3.72
Circular ends.	2.5$^{\phi}$ − 1.9$^{\phi}$ =	2.07
partition walls.	.3 × 1.9 × 2 =	1.14
fillets	.25 × .25 × 4 =	.25
		7.18 × 2.4 = 17.23 m³ @ 2400 = 41,350

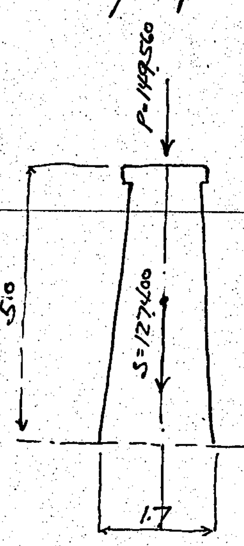
Fill. 2.5 × 6.2 = 15.50
2.5$^{\phi}$ = 4.91
20.41 × 2.4 = 49.00
17.23
31.77 m³ @ 2200 = 69,900

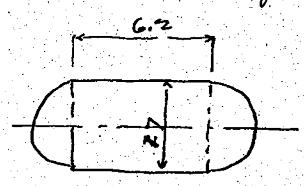
Total concrete for one pier = 102. cub.meter 111,250 kg call this 111,300 kg

Superimposed Loads on Pier

			For one girder	For 2 girders	
D.L.		2 @ 19270 =	38,540	77,080	see page 6.
L.L.	say	2 @ 18120 =	36,240	72,480	
			74,780	P = 149,560 kg	for one pier.

Design of Pier Shaft. moment of inertia of Bottom section

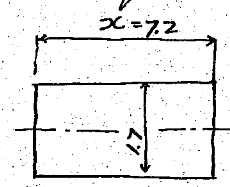




$\frac{6.2 \times 1.7^3}{12} = 2.535$

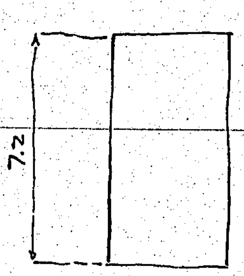
$0.0491 \times 1.7^4 = \underline{0.409}$
2.944 m⁴

Equivalent rectangular section of same moment of inertia



$\frac{x \times 1.7^3}{12} = 2.944$

$x = \frac{2.944 \times 12}{1.7^3} = 7.2$ m



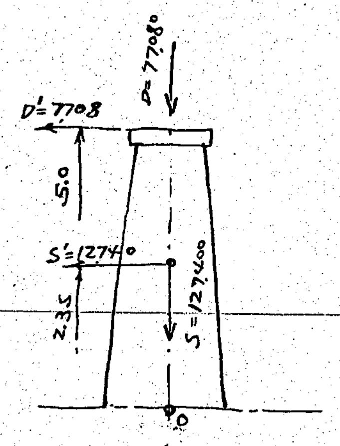
Case 1. Stability at normal state.

Superimposed D.L. and L.L. = 149,560
wt. of shaft = 127,400
276,960 kg

unit bearing on concrete = $\frac{276,960}{170 \times 720}$ = 2.3 kg/cm² o.k.

Case 2. Stability during earthquake. k = 0.1 assumed.

Taking moment about center O.



Loads	Hor. forces	Vert. forces		Lev. arm		Moments
D		77,080	×	0	=	0
D'	7,708		×	5.0	=	38,550
S		127,400	×	0	=	0
S'	12,740		×	2.35	=	29,950
	20,448 kg	204,480 kg	×	0.335	=	68,500 kgm

$e/h = \frac{.335}{1.7} = .197$, $d'/h = 5/170 = .03$,

Try reinforcements 19 mm$^{\phi}$ bars at 30 cm c/c = 9.45 × 7.2 = 68.0 cm² for one side

$p_0 = 2p = \frac{9.45 \times 2}{170 \times 100} = .00111$

k = .924 L = .093

CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama-ken.

$f_c = \frac{M}{Lbh^2} = \frac{68500 \times 100}{.093 \times 720 \times 170^2} = 3.5\ kg/cm^2$ ok.

$f_s = nf_c\left(\frac{d}{kh} - 1\right) = 15 \times 3.5\left(\frac{165}{.924 \times 170} - 1\right) = 3\ kg/cm^2$ ok

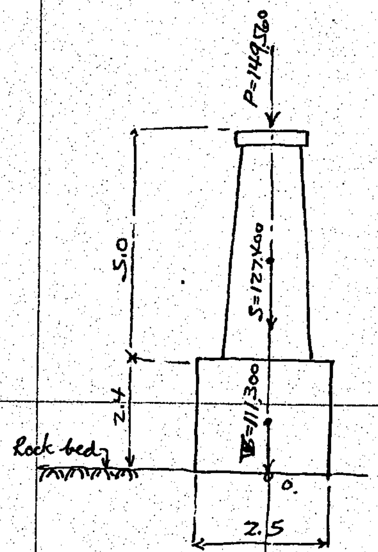
Steel required for moment only $= \frac{68500 \times 100}{1200 \times 1.8 \times \frac{7}{8} \times 165} = 22\ cm^2$

$\frac{22}{7.2} = 3.05\ cm^2$ per lin meter

use 19 m/m ϕ bars at 50 cm ctoc = 5.67 cm².

Stability of Pier as a whole.

Case 1. Stability ~~during~~ at normal state.

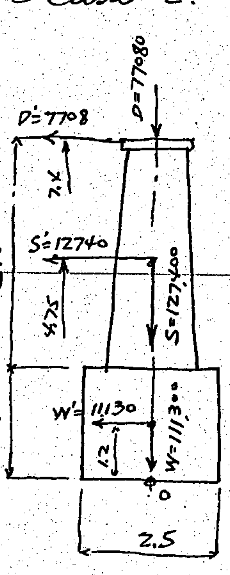


Superimposed Dead + Live Load = 149,560
weight of Shaft = 127,400
weight of well = 111,300
388,260 kg

bottom area = 2.5 × 6.2 = 15.50
2.5ϕ = 4.91
20.41 m²

Unit bearing pressure at bottom $= \frac{388260}{20.41} = 19000\ kg/m^2$ or (1.74 tons/□') ok

Case 2. Stability during Earthquake. k = 0.1 assumed.



moment about O.

Loads	Hor. forces	Vert. forces		Lev. arms		Moments
D		77,080	×	0	=	0
D'	7,708		×	7.4	=	57,000
S		127,400	×	0	=	0
S'	12,740		×	4.75	=	60,500
W		111,300	×	0	=	0
W'	11,130		×	1.2	=	13,400
	31,578	315,780 kg	×	.415 m	=	130,900 kgm.

Resultant force just upon the middle third point.

max. bearing pressure $= \frac{315780 \times 2}{20.41} = 30900\ kg/m^2$ or (2.83 tons/□') ok.

Design of well shell.

Earth pressure at bottom $= 1600 \times 2.4 \times \frac{1}{3} = 1,280\ kg/m^2$

Span length of sidewall = 2.8 m

moment on wall $= 1280 \times 2.8^2 \times \frac{1}{10} = 1,005$ kgm per horizontal meter strip.

Effective depth required $= \sqrt{\frac{1005 \times 100}{100 \times 7.18}} = 11.8$ cm

Use ~~280~~ 27 cm eff. depth with 3 cm insulation or 30 cm total depth.

Steel area required $= \frac{1005 \times 100}{1200 \times \frac{7}{8} \times 27} = 3.55$ cm²/meter strip at bottom

Use ϕ 13 m/m ϕ bars at 30 cm ctoc = 4.43

Pier between concrete beam and steel girder spans. P9.

Use simillar details as for Pier P10. as above figured.

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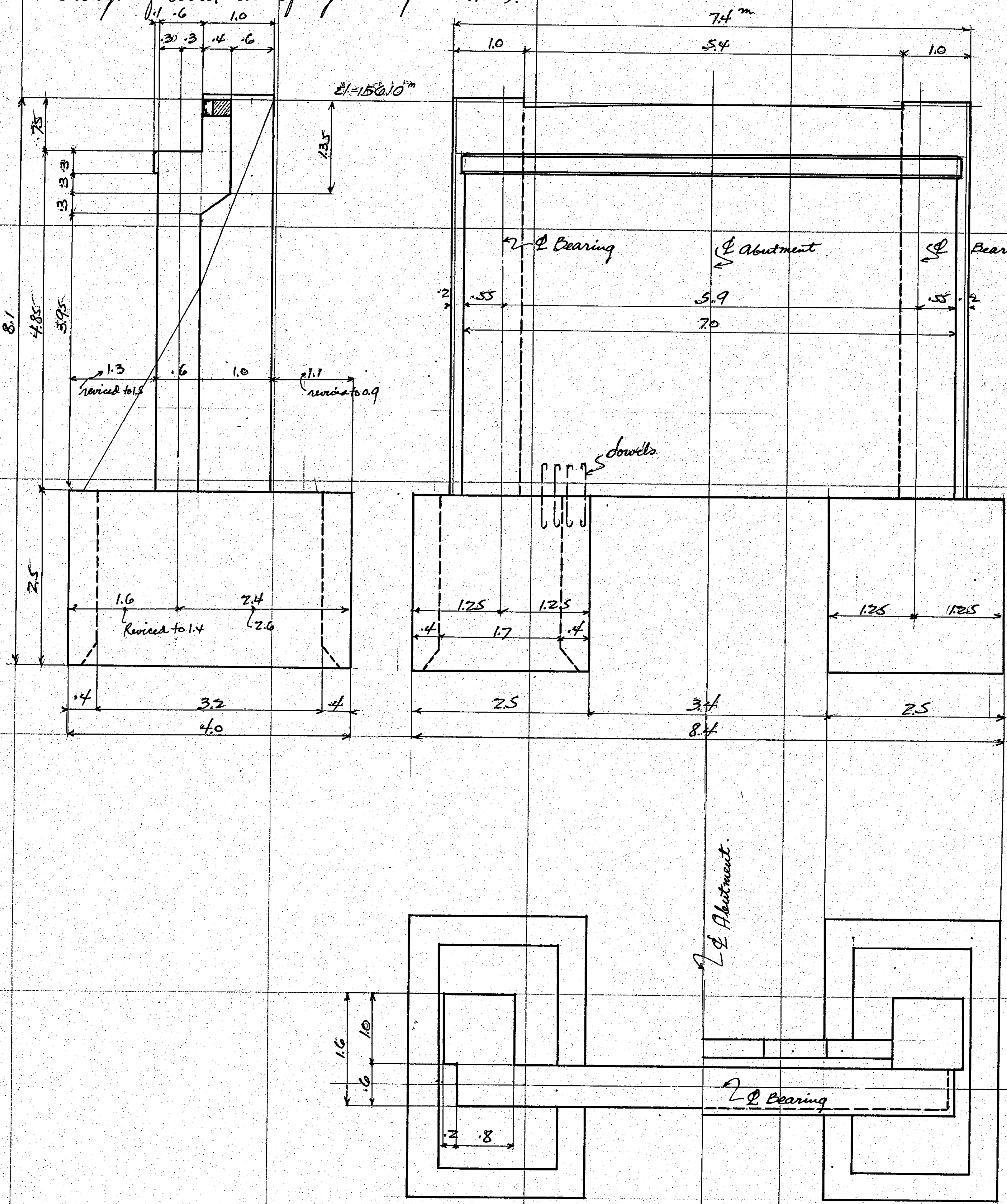
CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama ken.

Design of Abutment for Girder span (A2).

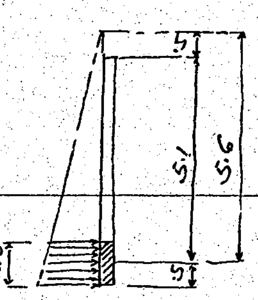


Sketch of Abutment A2.
Scale 1:60.

Design of Nakagawa-Bashi for Okayama-ken.

Parapet wall. Height 75cm max.
use 40cm wall, effective depth being 37cm
Reinforcements 13mmφ bars at 30cm c to c on both sides.

Curtain wall. Depth of Earth filling = 5.6m Surcharge for Live Load assumed 0.5m.
Span length of wall at bottom = 5.4m wall is fixed to the wells by proper dowels and the span length will be able to take its clear span.



Earth pressure on the lowest 1 meter strip of wall.
Depth of earth 5.6 − 0.5 + 0.5 = 5.6m.
Earth pressure $= \frac{1}{3} \times 1600 \times 5.6 = 2990$ kg/m² average.
moment on wall $= \frac{1}{10} \times 2990 \times 5.4^2 = 8{,}710$ kgm for one meter strip at bottom.
Effective depth required $= \sqrt{\frac{8710 \times 100}{100 \times 7.18}} = 35$cm.
use 60cm thickness, effective depth 55cm, insulation 5cm
Steel area required $= \frac{8710 \times 100}{1200 \times \frac{7}{8} \times 55} = 15.1$ cm²
use 22mmφ bars at 25cm c to c = 15.2 cm² on both sides at bottom.
Section at 2 meters above bottom. span length say 6.0m.
Earth pressure $= \frac{1}{3} \times 1600 \times 4.1 = 2190$ kg/m²
moment $= \frac{1}{10} \times 2190 \times 6.0^2 = 7{,}880$ kgm per meter strip.
Steel area req'd $= \frac{7880 \times 100}{1200 \times \frac{7}{8} \times 55} = 13.6$ cm²
use 22mmφ bars at 28cm c to c. use 30cm spacing

Stresses on Curtain wall during Earthquake. k = 0.1 assumed.
Earth pressure during Earthquake. $.37 \times 1600 \times 5.1 = 3020$ kg/m² at bottom 1m strip.
Seismic force due to wt. of wall $= .6 \times 2400 \times .1 = \underline{144}$
3,164
moment $= \frac{1}{10} \times 3164 \times 5.4^2 = 9220$ kgm per meter strip at bottom. ok.

Design of Shaft. Weight and center of gravity of Shaft.

				Hor. arm		vert. arm	
Parapet wall.	.4 × 1.5 × 5.4 = 3.24	@2400 =	7,780 ×	.8	6,220	4.85	37,700
Columns	1.0 × 1.0 × 5.7 × 2 = 11.40 ~~20.38~~	@ " =	27,350 ~~48,900~~ ×	1.1	30,100	2.85	78,000
Curtain wall.	.6 × 4.85 × 7.0 = 20.38	@ " =	48,900 ×	.3	14,680	2.43	118,800
Coping	.1 × .3 × 8.4 = .25	@ " =	600 ×	0	0	4.70	2,820
Light pedestals.	.8 × .8 × 1.5 × 2 = 1.92	@ " =	4,620 ×	1.1	5,080	6.45	29,780
	37.19m³	=	89,250	.63m	56,080	2.99m	267,100

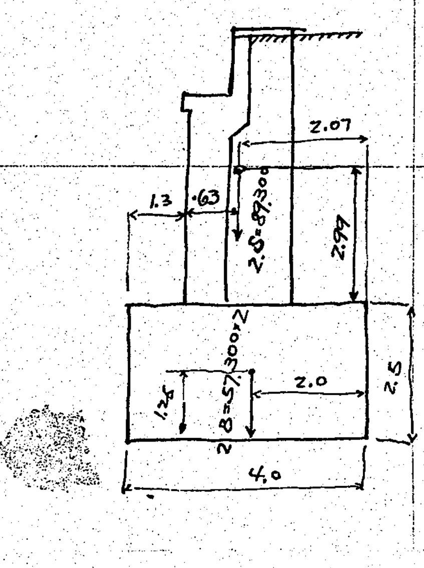
Call this 89,300 kg = S × 2

Weight of Base.

Shell. 2.5 × 4.0 = 10.00
1.7 × 3.2 = −5.44
4.56 × 2.5 = 11.40m³ @ 2400 = 27,350

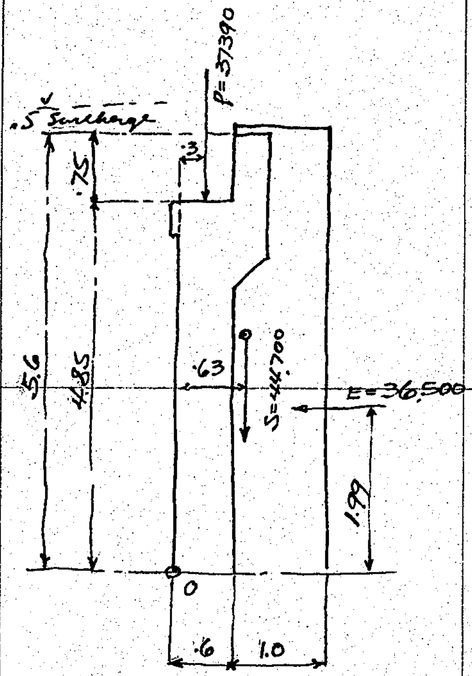
Fill 5.44 × 2.5 = 13.60 @ 2200 = 29,950
25.00m³ 57,300 kg = B/2

Total concrete = ~~62.19m³~~ 87.19m³



Design of Nakagawa-Bashi for Okayama-prefecture.

Case 1. Stability at normal state



Superimposed load on abutment. (load for one half of abutment).

D.L. 19270 = D
L.L. 18120
P = 37390 kg for one girder.

weight of Shaft S = ½ · 89300 = say 44,700 kg for ½ abutment.

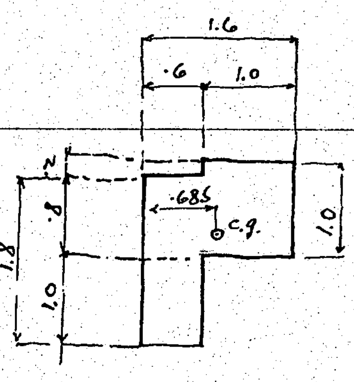
Earthpressure

⅓ · 1600 · .5 = 267
⅓ · 1600 · 6.1 = 3253
3520 ÷ 2 = 1,760 kg/m² average.

E = 1760 · 5.6 · 3.7 = 36,500 kg for ½ abutment.

Taking moment about toe 0.

Loads.	Hor. forces	Vert. forces		Lev. arm.		Moments.
P		37,390	×	.3	=	−11,220
S		44,700	×	.63	=	−28,150
E	36,500		×	1.99	=	72,600
	36,500 kg	82,090 kg	·	.405 m	=	33,230



Center of gravity of bottom area.

1.8 · 0.6 = 1.08 · .3 = 0.324
1.0 · 1.0 = 1.00 · 1.1 = 1.100
2.08 m² · .685 = 1.424

Eccentricity ε = .685 + .405 = 1.09 m, h = 160 cm, d' = 10, d = 150

ε/h = 109/160 = .682, d'/h = 10/160 = .063

moment at bottom of shaft = 82090 · 1.09 = 89500 kgm. for ½ of abutment.

Reinforcements: try 10 − 22 mm ϕ bars. on both sides = 38.01 cm²

$p_0 = 2p = \dfrac{38.01 \times 2}{160 \times 100}$ = .00475

k = .37, L = .107

$f_c = \dfrac{89500 \times 100}{.107 \times 100 \times 160^2}$ = 33.7 kg/cm² ok < 35.0

$f_s = 15 \times 33.7 \left(\dfrac{150}{.37 \times 160} - 1\right)$ = 776 kg/cm² ok.

Unit shear = $\dfrac{36500}{100 \times \frac{7}{8} \times 150}$ = 2.8, ok

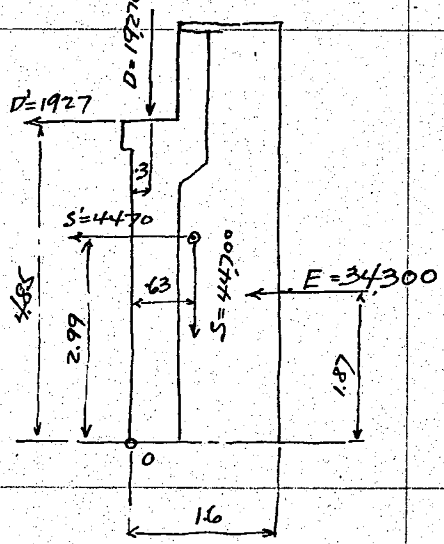
Unit bond = $\dfrac{36500}{6.91 \times 10 \times \frac{7}{8} \times 150}$ = 4.0, ok.

	k	L
.05	.4037, .140	.109
.063	.395	.107
.10	.3836, .1095	.100

Case 2. Stability during Earthquake.

Earthpressure during Earthquake = $\dfrac{1600 \times 5.6^2}{2} \cdot .370$ = ~~10930~~ 9280 kg per meter strip

E = 9280 · 3.7 = 34,300 kg



Moment about 0.

Loads	Hor. forces	Vert. forces		Lev. arm		Moments
D		19,270	×	−.3	=	−5,780
D'	1927		×	4.85	=	9,340
S		44,700	×	−.63	=	−28,200
S'	4470		×	2.99	=	13,370
E	34,300	~~34,300~~	×	1.87	=	64,180
	40,697	63,970	·	0.827 m	=	52,910

Ecc. ε = .827 + .405 = 1.232 m m = 63970 · 1.232 = 78800 kgm.

ε/h = 1.232/1.6 = .77 k = .350, L = .106

$f_c = \dfrac{78800 \times 100}{.106 \times 100 \times 160^2}$ = 29.0 kg/cm² ok

$f_s = 15 \times 29 \left(\dfrac{150}{.35 \times 160} - 1\right)$ = 730 kg/cm² ok

Unit shear = $\dfrac{40697}{100 \times \frac{7}{8} \times 150}$ = 3.1 kg/cm² ok

Unit bond = $\dfrac{40697}{6.91 \times 10 \times \frac{7}{8} \times 150}$ = 4.5, ok.

	k	L
.05	.345	.108
.063	.358	.106
.10	.330	.099

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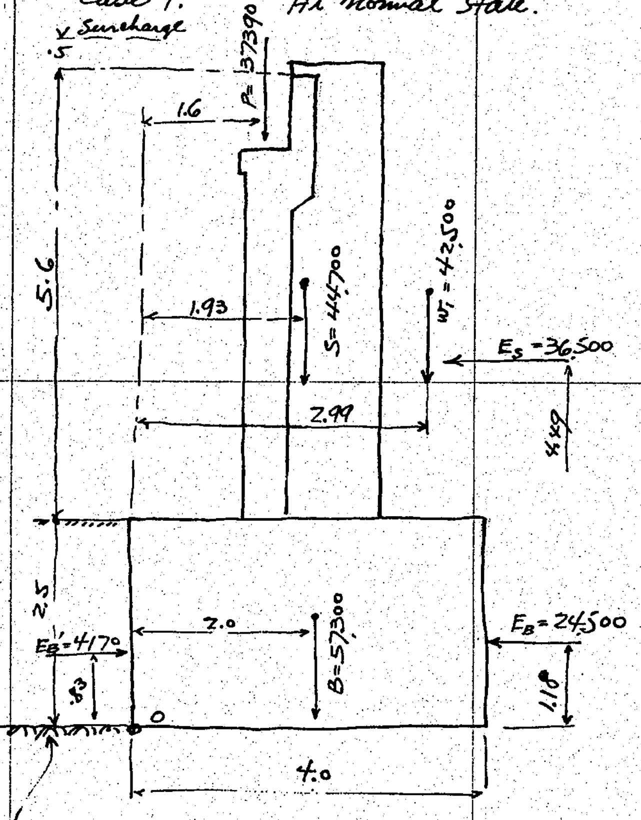
CALCULATIONS FOR

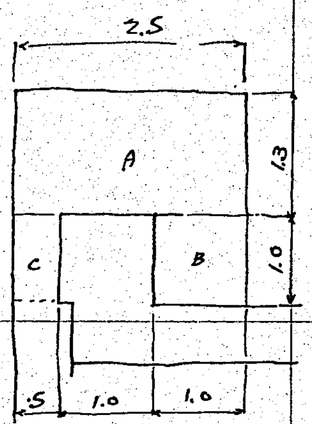
Design of Nakagawa-Bashi for Okayama-ken

Stability of Abutment. All figures below are for one-half of abutment.

Case 1. At normal state.



weight of earth on rear footing.



A. 2.5×1.3×5.6 @ 1600 = 29,100
B 1.0×1.0×5.6 @ " = 8960
C say .5×1.0×5.6 @ " = 4480
42,540 kg call this 42,500 kg

Earth pressure on base. rear side.
at top. $\frac{1}{3}\times 1600\times 6.1 = 3250$
" bottom $\frac{1}{3}\times 1600\times 8.6 = 4580$
$7,830 \div 2 = 3915$ kg/m² average.

$E_B = 3915\times 2.5\times 2.5 = 24,500$ kg

$E_B' = \frac{1}{3}\times 1600\times \frac{2.5^2}{2}\times 2.5 = 4170$ kg front side.

Taking moment about toe O.

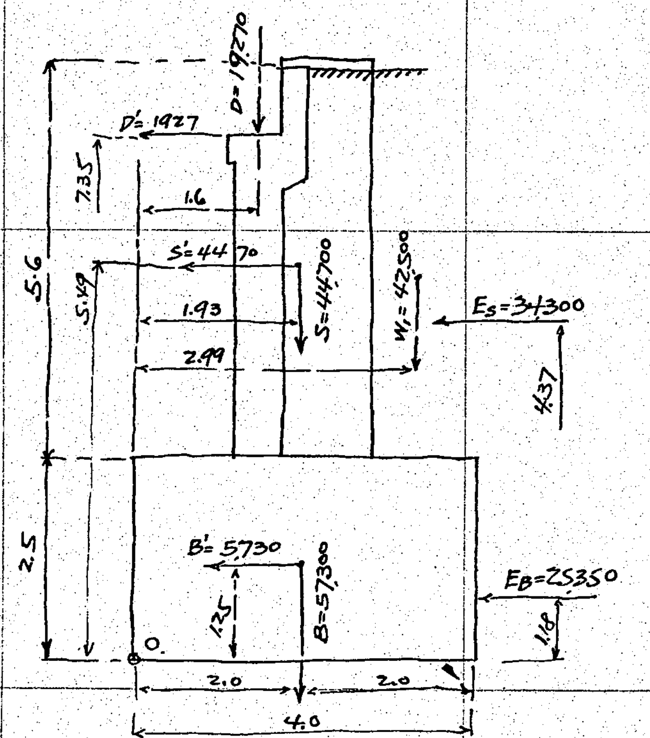
Loads	Hor. forces	Vert. forces	Lev. arm	moments
P		37,390	× -1.6	= -59800
S		44,700	× -1.93	= -86,200
B		57,300	× -2.00	= -114,600
W_1		42,500	× -2.99	= -127,200
E_s	36,500		× 4.49	= 164,000
E_B	24,500		× 1.18	= 28,900
E_B'	-4170		× .83	= -3,500
	56,830 kg	181,890 kg	· 1.09 m	= -198,400

Eccentricity ε = 2.0 − 1.09 = 0.91 m

Resultant force ~~within~~ outside of middle third. pressure area = 1.09×3×2.5 = 8.18 m²

max. bearing pressure = $\frac{181,890\times 2}{8.18}$ = 44,400 kg/m² or (4.06 tons/□') ok. on Rock bed.

Case 2. Stability during Earthquake. (Seismic forces forward)



Earth pressure on base during earthquake. $\frac{1600\times 8.1^2}{2}\times .37 = 19420$
9,280
10,140 kg/m strip.

$E_B = 10140\times 2.5 = 25350$ kg

Taking moment about toe O.

Loads	Hor. forces	Vert. forces	Lev. arm	Moments
D		19,270	× -1.6	= -30,800
D'	1,927		× 7.35	= 14,170
S		44,700	× -1.93	= -86,200
S'	4,470		× 5.49	= 24,550
B		57,300	× -2.00	= -114,550
B'	5,730		× 1.25	= 7,160
W_1		42,500	× -2.99	= -127,150
E_s	34,300		× 4.37	= 149,800
E_B.	25,350		× 1.18	= 29,900
	71,777	163,770 kg	· 0.813 m	= -133,120

Eccentricity ε = 2.0 − 0.813 = 1.187 m   pressure area = .813×3×2.5 = 6.1 m²

max. Bearing pressure at toe = $\frac{163,770\times 2}{6.1}$ = 53600 kg/m² or (4.9 tons/□') ok. on Rock bed.

Design of Nakagawa-Bashi for Okayama ken.

Case 3. Stability during Earthquake (Seismic forces backward).

Earth pressure during earthquake front side. $\frac{1}{2} \times 1600 \times 2.5^2 \times .370 = 1850$ kg/m strip.

$E_B = 1850 \times 2.5 = 4630$ kg.

Referring to the figure for case 2.

Loads	Hor. forces	Vert. forces		Lev. arm		Moments.
D		19,270	×	1.60	=	30,800
D'	1,927		×	7.35	=	14,170
S		44,700	×	1.93	=	86,200
S'	4,470		×	5.49	=	24,550
B		57,300	×	2.00	=	114,600
B'	5,730		×	1.25	=	7,160
W_1		42,500	×	2.99	=	127,150
E_B	4630		×	.83	=	3,840
	16,757	163770 kg	×	2.50 m	=	408,470

Eccentricity $\varepsilon = 2.50 - 2.0 = 0.50$ m

Resultant force within middle third.

max. bearing pressure $= \frac{163770}{2.5 \times 4.0}\left(1 \pm \frac{6 \times .5}{4.0}\right) = 28700$ kg/m² or (2.62 tons/□') ok.

Revise the position of Base, to shift 20 cm forward.

Approx. calculation for Case #1.

moment about o. $-198,400$

$124,590 \times .2 = -24,900$

a part of W_1. $0.2 \times 2.5 \times 5.6 = 2.8 \times 1600 \times 4.1 = +18,300$

~~191,800~~

$-205,000$

Vert forces. $181890 - 4480 = 177,410$ kg

$205,000 \div 177,410 = 1.155$ m

Eccentricity $\varepsilon = 2.0 - 1.155 = .845$ m   pressure area $= 1.155 \times 3 \times 2.5 = 8.66$ □m

max. bearing pressure $= \frac{177,410 \times 2}{8.66} = 41,000$ kg/m² (or 3.74 tons/□') ok.

Approx. calculation for case 2.

moment about O. $-133,120$ vert force 163770

$106,470 \times .2 = -21,290$ -4480

$+18,300$ 159,290

$-136,110 \div 159290 = .89$

Eccentricity $\varepsilon = 2.0 - .89 = 1.11$ m   pressure area $= .89 \times 3 \times 2.5 = 6.7$ □m

max bearing pressure $= \frac{159290 \times 2}{6.7} = 47,500$ kg/m² or (4.34 tons/□') ok.

Approx. calculation for case 3.

moment about O. 408470

$106470 \times .2$ 21,290

18,300

448,060   vert. force $= 159290$ kg

$448060 \div 159,290 = 2.82$ m

Eccentricity $\varepsilon = 2.82 - 2.0 = .82$ m   pressure area $= (1.18 \times 3) \times 2.5 = 8.85$ □m

max. Bearing pressure $= \frac{159290 \times 2}{8.85} = 36,000$ kg/m² or (3.3 tons/□') ok.

Design of Well.

Depth of Earthfill causing earthpressure on the well during execution assumed 4.0 m.

Earth pressure at bottom $= \frac{1}{3} \times 1600 \times 4 = 2130$ kg/m²   span length $= 3.6$ m

moment on longer sidewall $= \frac{1}{10} \times 2130 \times 3.6^2 = 2760$ kgm/meter strip

Effective depth req'd $= \sqrt{\frac{2760 \times 100}{100 \times 7.18}} = 19.6$ cm   use 37 cm eff. depth with 3 cm insulation.

Steel area req'd. $= \frac{2760 \times 100}{1200 \times \frac{7}{8} \times 37} = 7.1$ cm²

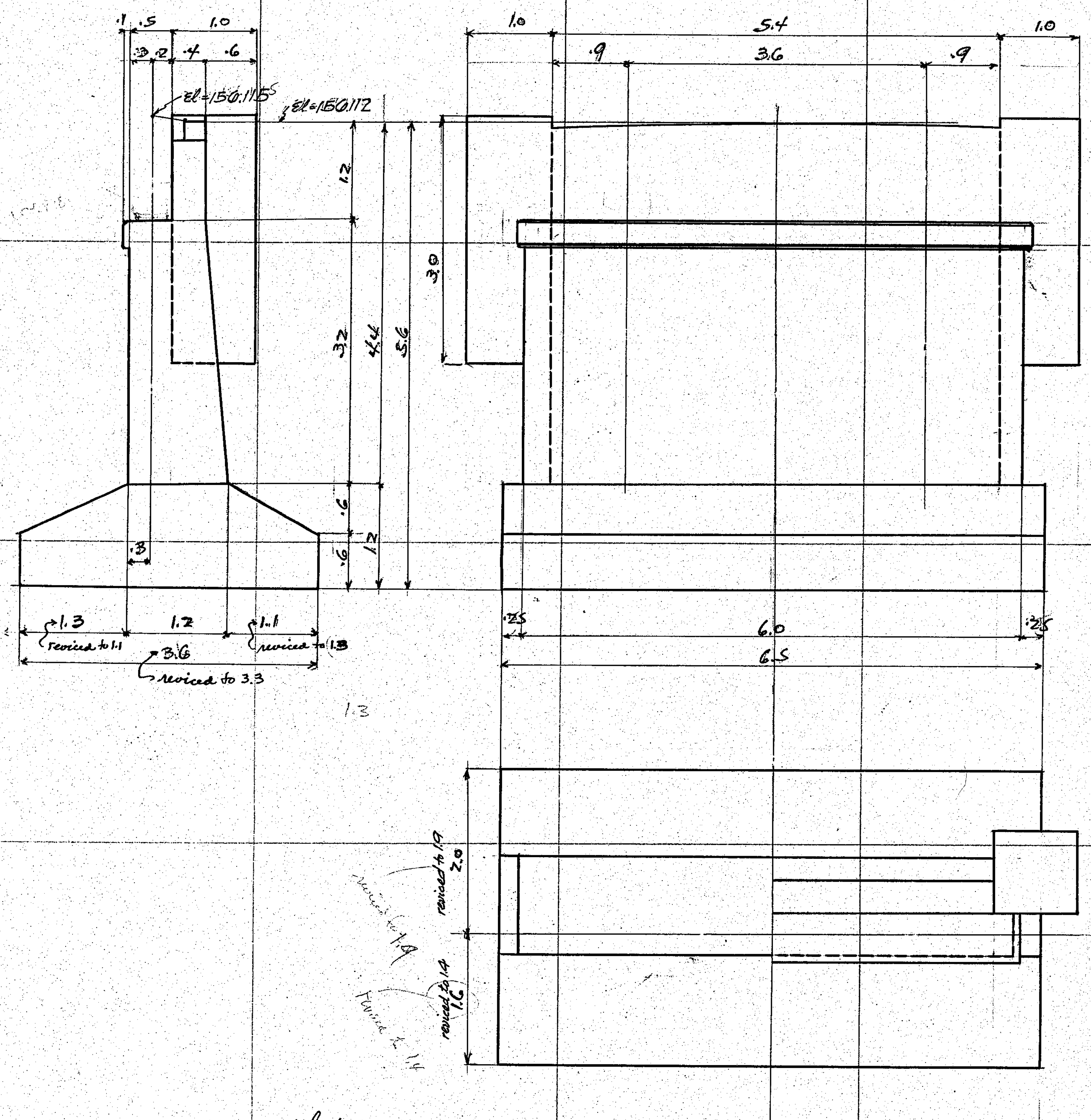
use 13 mm ϕ bars at 20 cm ctoc for lowest 1.0 m

" 13 mm " " 30 cm " for upper 1.5 m.

vertical bars 13 mm ϕ bars 50 cm ctoc about.

Design of Nakagawa-Bashi for Okayama ken.

Design of Abutment A1.



Sketch of Abutment A1. (for concrete span).
Scale 1:60.

Parapet wall.

Normal state. Height of wall 1.2 meters. with 0.5 m Surcharge for L.L.

Earth pressure. $= \frac{1}{3} \times 1600 \times \frac{1.7^2}{2} = 770$ kg per lin m.

moment $= 770 \times .615 = 474$ kgm " "

During Earthquake.

Earth pressure $= .370 \times \frac{1600 \times 1.7^2}{2} = 856.$ kg per lin m $\quad .4 \times 2400 \times .1 = 96$ kg/m

moment $= 856 \times .4 = 343$

" $96 \times \frac{1.5^2}{2} = 108$

451 kgm " "

Steel req'd $= \frac{47400}{1200 \times \frac{7}{8} \times 37} = 1.22$ cm² per meter strip. for normal state.

Use 13 mmφ bars at 30 cm c to c $= 4.42$ cm² " on both sides

Design of Nakagawa-Bashi for Okayama ken

Weight and Center of gravity
Shaft.

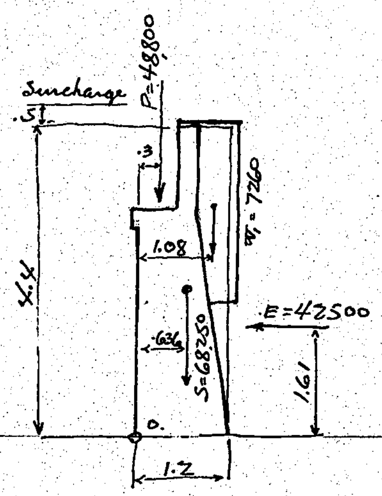
						hor. arm		vert. arm	
parapet wall	1.2×.4×5.4 =	2.59	@ 2400 =	6220	×	.70	4350	3.80	23650
columns.	1.0×1.0×3.0×2 =	6.00	@ 〃 =	14400	×	1.00	14400	3.00	43200
〃 less.	.5×.3×1.8×2×(−)	.54	@ 〃 =(−)	1300	×	.75	− 980	3.00	− 3900
Shaft.	1.05×3.2×6.0 =	20.18	@ 〃 =	48400	×	.53	25650	1.53	74000
Coping	.1×.3×7.2 =	.22	@ 〃 =	530	×	0	0	3.05	1620
		28.45 m³	=	68250 kg		.636 m	43420	2.03 m	138570

Base. Sectional area.

1.3×.9 =	1.17	× .722	.845	.467	.546
1.2×1.2 =	1.44	× 1.90	2.735	.600	.864
1.1×.9 =	.99	× 2.99	2.960	.467	.462
	3.60 m²	1.82	6.54	.52	1.872

Concrete 3.60×6.5 = 23.40 m³ @ 2400 = 56200 kg
28.45
51.85 m³ = Total concrete for abutment. A1.

Stability of Shaft.
Case 1. Stability at normal state.



Superimposed Loads on abutment. (for one abutment).
Dead Load. 2@13180 ≒ 26400 kg = D See page 25
LL. 2@11210 ≒ 22400
48800 = P.

weight of earth on the inclined back of wall
.3×1.2 = .36
$.3 \times \frac{3.2}{2}$ = .48
.84×5.4 = 4.54 @1600 = 7260 kg = W1

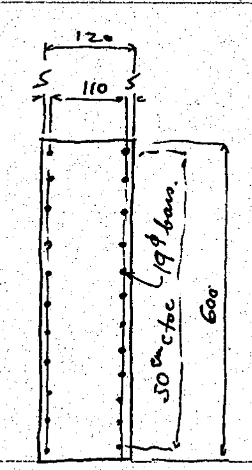
Earth pressure = $\frac{1}{3}$×1600×.5 = 267.
$\frac{1}{3}$×1600×4.9 = 2613
2880 ÷ 2 = 1440 kg/m² average.

1440×4.4×6.7 = 42500 kg = E

Taking moment about O.

Loads	Hor. forces.	Vert. forces.		Lev. arm		moment.
P		48800	×	.30	=	− 14650
S		68250	×	.636	=	− 43400
W1		7260	×	1.08	=	− 7840
E	42500		×	1.61	=	68410
	42500 k	124310 k	×	0.02 m	=	2520

Eccentricity ε = 0.6 + .02 = 0.62 m
moment = 124310 × .62 = 77100 kgm.



Reinforcements, Try 19mmφ bars at 50cm ctoc on both sides
Steel area = 2.835×2×6.0 = 34.0 cm²
Steel ratio $p_o = 2p = \frac{34 \times 2}{120 \times 600}$ = .00089

ε/h = 62/120 = .516, d/h = 5/120 = .042
k = .24, L = .062

$f_c = \frac{77100 \times 100}{.062 \times 600 \times 120^2}$ = 14.4 kg/cm² ok

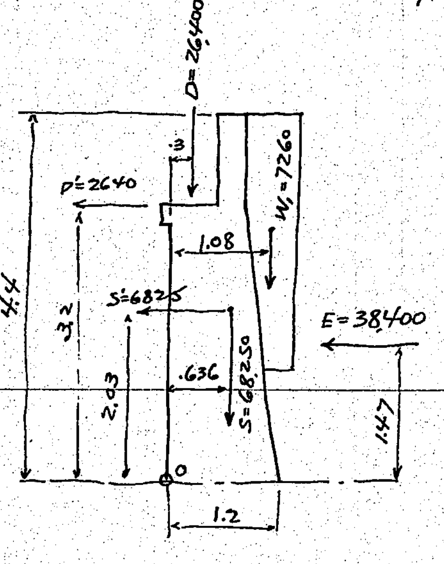
$f_s = 15 \times 14.4 \left(\frac{115}{.24 \times 120} - 1\right)$ = 648 〃 ok.

Unit shear = $\frac{42500}{600 \times \frac{7}{8} \times 115}$ = 0.7 〃 ok

unit bond = $\frac{42500}{597 \times 12 \times \frac{7}{8} \times 115}$ = 59 〃 ok.

Design of Nakagawa-Bashi for Okayamaken.

Case 2. Stability during Earthquake. (Seismic force forward).



Earth pressure during earthquake.

$\frac{1}{2} \cdot 1600 \cdot 4.4^2 \cdot .37 = 5730^{kg}$/lin m.

$E = 5730 \cdot 6.7 = 38,400$ kg.

Taking moment about O.

Load	Hor. forces	Vert. forces		Lev. arm		moment
D		26,400	×	-.3	=	- 7,920
D'	2640		×	3.2	=	8,450
S		68,250	×	-.636	=	-43,400
S'	6825		×	2.03	=	13,850
W1		7,260	×	-1.08	=	- 7,840
E	38400		×	1.47	=	56,400
	47865 kg	101,910 kg	×	0.192 m	=	19,540

(Seismic for backward)

absolute value = 81460

$\frac{81460}{101910} = .80^m$

$\varepsilon = .8 - .6 = .20^m$
H = 9465 kg
V = 101910 kg.

Eccentricity $\varepsilon = 0.6 + .192 = 0.792^m$

moment $= 101,910 \times .792 = 80,700$ kgm.

$\varepsilon/h = .792/1.2 = .66 \quad d'/h = .042, \quad p_3 = .00089$

$k = .195, \quad L = .057$

$f_c = \frac{80700 \times 100}{.057 \times 600 \times 120^2} = 16.4$ kg/cm² ok

$f_s = 15 \times 16.4 \left(\frac{115}{.195 \times 120} - 1\right) = 963$ " ok

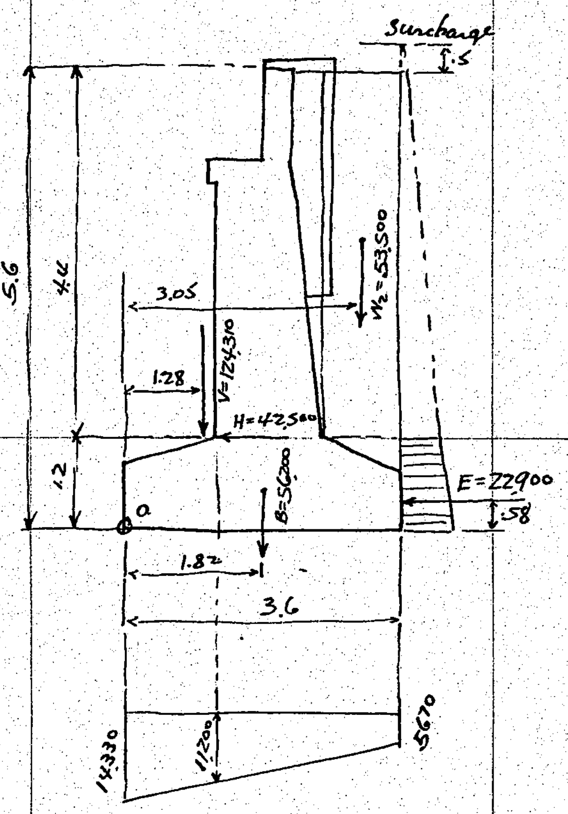
Unit shear $= \frac{47865}{600 \times \frac{7}{8} \times 115} = 0.8$ " ok

Unit bond $= \frac{47865}{5.97 \times 12 \times \frac{7}{8} \times 115} = 6.7$ " ok.

Assumed section is ample.

Stability of Abutment.

Case 1. Stability at normal state.



Weight of Earth on rear footing $1.1 \times 4.7 \times 6.5 @ 1600 = 53,500$ kg = W2

Earth pressure on base. average depth of fill = 5.0

Surcharge L.L. $= 0.5$

$5.5^m @ \frac{1600}{3} = \frac{8800}{3} = 2935$ kg/m²

$E = 2935 \times 1.2 \times 6.5 = 22,900$ kg

Taking moment about toe O.

Load	Hor. forces	Vert. forces		Lev. arm		Moments
V		124,310	×	-1.28	=	-159,200
H	42,500		×	1.20	=	51,000
W2		53,500	×	-3.05	=	-163,200
B		56,200	×	-1.82	=	-102,300
E	22900		×	.58	=	13,300
	65400 kg	234,010 kg	×	1.54 m	=	360,400

Eccentricity $\varepsilon = 1.80 - 1.54 = 0.26^m$

Resultant force within middle third.

max. toe pressure $= \frac{234010}{6.5 \cdot 3.6}\left(1 \pm \frac{6 \times .26}{3.6}\right) = 14,330$ kg/m² (or 1.31 tons/□') ok.
on gravel foundation.

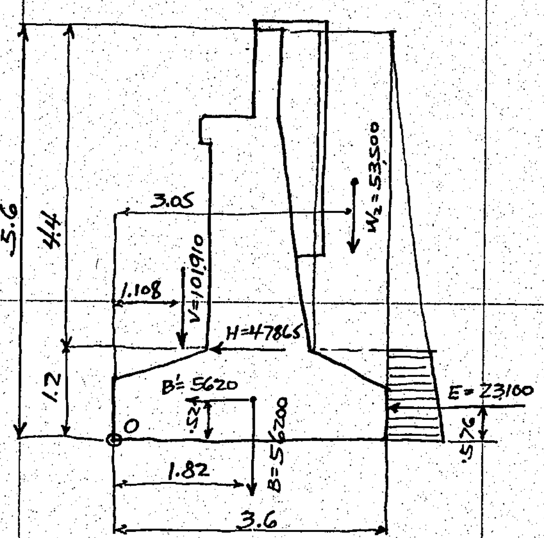
Design of Nakagawa-Bashi for Okayama-ken

Case 2. Stability of abutment during earthquake (seismic forces foward) k=0.1

Earth pressure average. 1600 × 5.0 × .37 = 2960 kg/m²

E = 2960 × 1.2 × 6.5 = 23,100 kg.

Taking moment about toe O.



Loads	Hor. forces	Vert. forces	Lev. arm	moments
V		101,910	× − 1.108 =	−113,000
H	47865		× 1.20 =	57,400
B		56,200	× − 1.82 =	−102,300
B'	5620		× .52 =	2,900
W_2		53,500	× − 3.05 =	−163,200
E	23100		× .576 =	13,300
	76,585	211,610 kg	× 1.44 m =	−304,900

Eccentricity ε = 1.80 − 1.44 = 0.36 m

Resultant force within middle third.

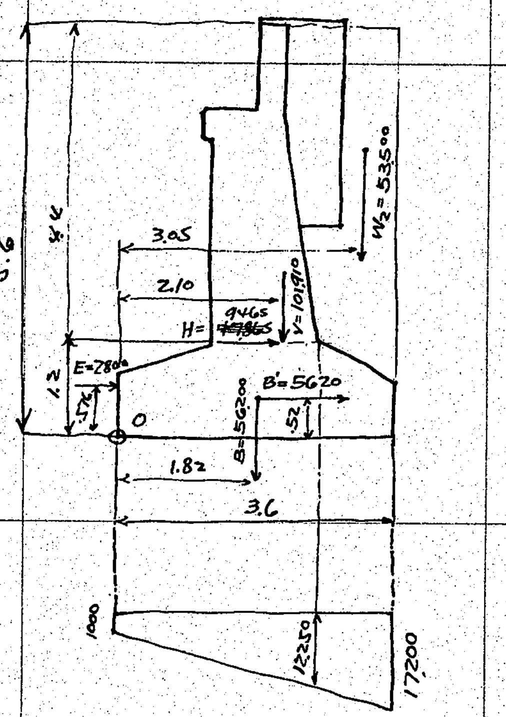
max. toe pressure = $\frac{211,610}{6.5 \times 3.6}\left(1 \pm \frac{6 \times .36}{3.6}\right)$ = 14480 kg/m² (= 1.32 tons/□') ok

= 3620 "

Case 3. Stability during earthquake (Seismic forces backward.)

Earth pressure $\frac{1600 \times 1.2^2}{2} \times .37 = 430 \times 6.5 = 2800$ kg

Taking moment about O.



Loads	Hor. forces	Vert. forces	Lev. arms	moments
V		101,910	× 2.10 =	−214,000
H	9465		× 1.2 =	−11,400
B		56200	× 1.82 =	−102,300
B'	5620		× .52 =	−2,900
W_2		53,500	× 3.05 =	−163,200
E	2800		× .576 =	−1,600
	17,885 kg	211,610	× 2.34 m =	495,400

Eccentricity ε = 2.34 − 1.8 = .54 m

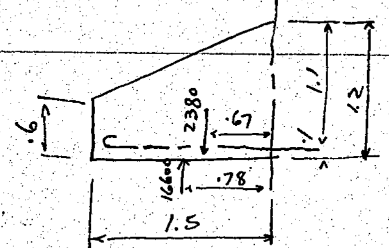
Resultant force within middle third.

max. bearing pressure at heel.

= $\frac{211,610}{6.5 \times 3.6}\left(1 \pm \frac{6 \times .54}{3.6}\right)$ = 17200 kg/m² (= 1.57 tons/□') ok.

= 1000 "

Design of Cantilever footing at toe.



Upward pressure 14330
11200
25530 ÷ 2 = 12,765 × 1.3 = 16,600 kg × 0.78 = 12,950

wt. of footing = 0.9 × 1.1/... × 2400 = −2,380 × 0.67 = −1600
14,220 kg   11,350 kgm

Effective depth required = $\sqrt{\frac{11350 \times 100}{100 \times 7.18}}$ = 40 cm

Use eff. depth of 110 cm with 10 cm insulation at bottom = total 120.

Steel area req'd. = $\frac{11350 \times 100}{1200 \times \frac{7}{8} \times 110}$ = 9.85 cm² per meter strip.

Use 22 mmφ bars at 30 cm ctoc = 12.67 cm² " ok

Unit shear = $\frac{14220}{100 \times \frac{7}{8} \times 110}$ = 1.48 kg/cm² ok

Unit bond = $\frac{14220}{6.91 \times 3.33 \times \frac{7}{8} \times 110}$ = 6.42 kg/cm²

Use 23 bars for total width of base.

= $\frac{23}{6.5}$ = 3.54/m

bond stress = 6.42 × $\frac{3.33}{3.54}$ = 6.0 kg/cm² ok.

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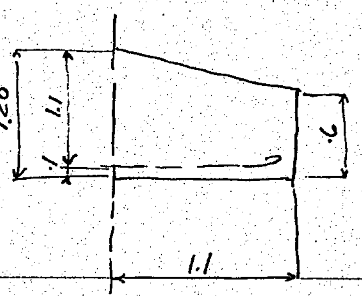
CALCULATIONS FOR

Design of Nakagawa-Bashi for Okayama-ken

Design of Cantilever footing at heel. (case 1)



		at face of wall		at end.
upward pressure		12,250		17,200
wt. of footing	1.2×2400 =	− 2,880		− 1,440
" " earth fill.	4.4×1600 =	− 7,040	5×1600 =	− 8,000
		2,330		7,760

average. 2330
7760
10,090 ÷ 2 = 5045 kg

moment = 5045 × ~~.884~~ .65 = ~~4460~~ 3280 kgm. during Earthquake.

case 2.

	at face of wall	at end.
upward pressure	6,940	3,620
wt. of footing	− 2,880	− 1,440
wt. of earth filling	− 7,040	− 8000
	− 2,980	− 5,820

Average. 2980
5820
8800 ÷ 2 = −4400 kg

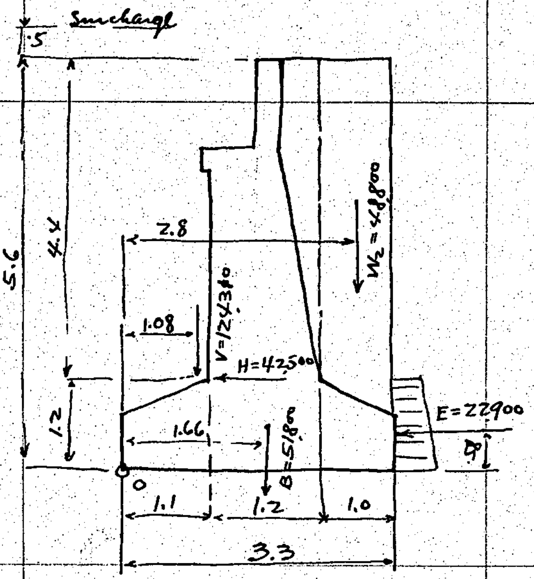
moment − 4400 × .61 = − 2,680 kgm.

Steel area req'd = $\frac{3280 \times 100}{1200 \times 1.8 \times \frac{7}{8} \times 110}$ = 1.58 cm² per lin meter

Use 19 mm φ bars at 60 cm c to c = 6.32 " "

unit bond = $\frac{4400}{6.91 \times 1.67 \times \frac{7}{8} \times 110}$ = 4.0 kg/cm² ok.

Revicing the width of base to 3.3 meters, the max bearing pressure at toe will be as follows.
At normal state
weight of earth on rear footing 1.0 × 4.7 × 6.5 × 1600 = 48800 kg
wt. of base.
1.1 × .9 = .99
1.2 × 1.2 = 1.44
1.0 × .9 = .90
3.33 × 6.5 × 2400 = 51800 kg



moment about O.

Loads	Hor. forces	Vert forces		Lev. arm		Moment
V		124,310	×	− 1.08	=	− 134,300
H	42,500		×	1.20	=	51,000
B		51,800	×	− 1.66	=	− 86,000
W₂		48,800	×	− 2.80	=	− 136,500
E	22,900		×	.58	=	13,300
	65,400 kg	224,910	÷	1.30 m	=	− 292,500

Eccentricity ε = 1.65 − 1.30 = 0.35 m
Resultant force within middle third
max. toe pressure = $\frac{224,910}{6.5 \times 3.3}\left(1 + \frac{6 \times .35}{3.3}\right)$ = 17200 kg/m² (or 1.57 tons/ft²) ok.

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MADE BY M. Jinda DATE 4-8-29 **FILE NO**

CHECKED BY M. Kojima DATE 4-8-30 **PAGE NO** 58

CALCULATIONS FOR

Material List of Nakagawa-Bashi for Okayama Prefecture

No	Description			Length	Unit Weight	Total Wts.
		Main Girder	$G1^R_L, G2^R_L, G3^R_L$		2-Required	
4	Cov. Pls	330	×11	2.705	28.496	308.3
2	〃	〃	〃	7.520	〃	428.6
6	Flg. Ls	150 × 150	×11	6.735	24.95	1,008.2
4	〃	〃 〃	〃	7.520	〃	750.5
2	〃	〃 〃	〃	8.235	〃	410.9
2	Web. Pls.	1,600	×9	6.735	113.04	1,522.6
1	Web Pl.	〃	〃	7.520	〃	850.1
4	Spl. Pls.	330	×11	1.230	28.496	140.2
8	Spl. Ls.	150 × 150	×15	1.230	33.55	330.1
4	Spl. Pls.	320	×11	.840	27.632	92.8
8	〃	230	×11	.920	19.861	146.2
12	Stiff. Ls	125 × 90	×10	1.588	16.09	306.6
20	〃	〃 〃	〃	1.610	〃	518.1
8	〃	〃 〃	〃	.730	〃	94.0
12	Fills.	90	×11	1.305	7.772	121.7
4	〃	180	×11	.590	15.543	36.7
5	〃	90	×11	.335	7.772	13.0
4	Stiff. Ls	100 × 90	×10	1.558	14.13	88.1
4	Fills.	90	×15	1.280	10.598	54.3
3	Ls	100 × 75	×10	.210	12.95	8.2
3	Fills.	60φ		10	@ 0.222	0.7
2	Ls	100 × 75	×10	1.085	12.95	28.1
9	〃	〃 〃	〃	1.230	〃	143.4
2	〃	〃 〃	〃	1.045	〃	27.1
2	〃	〃 〃	〃	.150	〃	3.9
2	〃	〃 〃	〃	1.075	〃	27.8
1	L.	〃 〃	〃	1.150	〃	14.9
1	〃	〃 〃	〃	.205	〃	2.7
2	Sole Pls.	410	×20	.500	64.370	64.4
2	Pls.	350	×9	.425	24.728	21.0
2	〃	〃	〃	.685	〃	33.9
1	Pl.	〃	〃	.675	〃	16.7
						7,613.8
						× 2
						15,227.6
		Floor Beams			1-Required	
5	I	450 × 175 @ 114.68 Kgs.		5.845		3,351.5
10	Ls	125 × 90.	×10	330	16.09	53.1
12	〃	〃 〃	〃	190	〃	36.7
						3,441.3
		Stringers	$S1, S2^R_L, S3$ & $S4^R_L$		1-Required	
8	I	300 × 150 @ 48.34 Kgs.		4.970		1,922.0
4	〃	305 × 152 @ 65.50 Kgs.		4.970		1,302.1
48	Ls	125 × 90	×10	210	16.09	162.2
						3,386.3

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MADE BY M. Jinda DATE 4-8-29 FILE NO
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CALCULATIONS FOR

Material List of Nakagawa-Bashi for Okayama-Prefecture

No.	Description			Length	Unit Weight	Total Weight
		Stringer Brackets		BR1 R/L	1-Req'd	
6	I	300 × 150 @48.34 Kgs		495		147.0
12	Ls	125 × 90	× 10	210	16.09	40.5
6	Pls.	160	× 9	500	11.304	33.9
						215.1
		Lateral Bracings			4-Required	
1	L	75 × 75	× 9	7,230	9.96	72.0
2	Ls	" "	"	3,560	"	70.9
4	"	150 × 90	× 9	190	16.32	12.4
1	Pl.	280	× 9	550	19.782	10.9
						166.2
						× 4
						664.8
		Bed Plate			4-Required	
1	Pl.	430	× 35	500	118.143	59.1
2	Ls	75 × 75	× 9	210	9.96	4.2
						63.3
						× 4
						253.2
		Anchor Bolt			8-Required	
1	Bolt	32φ		700	@ 5.203	5.2
1	Washer	150	× 9	150	10.598	1.6
						6.8
						× 8
						54.4
					Grand Total	23,242.7 Kgs.
		Expansion Joint EJ1			3-Required	
1	Bar	50	× 10	5,400	3.925	21.1
1	L	125 × 75	× 10	"	14.91	80.5
9	"	90 × 75	× 9	160	11.02	15.9
9	Bolts	19φ		200	@ 0.637	5.7
						123.2
						× 3
						369.6
		Expansion Joint EJ2			3-Required	
1	Checkered Pl.	165	× 9	5,400	13.133	70.9
1	L	65 × 65	× 8	"	7.66	41.3
2	Ls	90 × 75	× 9	180	11.02	4.0
7	"	" "	"	160	"	12.3
9	Bolts	19φ		200	@ 0.637	5.7
						134.2
						× 3
						402.6

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MADE BY M. Jinda DATE 4-8-29 FILE NO

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CALCULATIONS FOR

Material List of [illegible]akagawa-Bashi for Okayama Prefecture

No	Description			Length	Unit Weight	Total Weight
		Expansion	Joint EJ3		1-Required	
1	Checkered Pl.	190	×9	5400	15.123	81.6
1	L	65×65	×8	"	7.66	41.3
2	Ls	90×75	×9	180	11.02	4.0
7	"	"		160	"	12.3
9	Bolts	19φ		200	@0.637	5.7
						144.9
		Expansion	Joint EJ4		1-Required	
2	Bars	50	×10	6200	3.925	48.6
2	Ls	125×75	×10	"	14.91	184.8
4	Pls.	310	×9	455	21.902	39.8
4	"	160	×9	200	11.304	9.0
2	"	"	"	225	"	5.0
4	Ls	90×75	×9	455	11.02	20.0
6	"	"		160	"	10.5
10	Bars	70	×9	400	4.946	19.7
5	Bolts (on Abutment only)	19φ		200	@ 0.637	3.1
						340.5
		Expansion	Joint EJ4A		1-Required	
1	Bars	50	×10	5440	3.925	21.3
1	L	125×75	×10	5865	14.91	87.4
2	Pls.	170	×9	455	12.011	10.9
2	Pls	160	×9	200	11.304	4.5
1	Pl.	"		225	"	2.5
2	Ls	90×75	×9	455	11.02	10.0
3	"	"		160	"	5.2
5	Bars	70	×9	400	4.946	9.8
						151.6
		Expansion	Joint EJ5		2-Required	
1	Checkered Pl.	215	×9	6200	17.112	106.1
1	L	65×65	×8	"	7.66	47.5
2	Pls.	310	×9	455	21.902	19.9
2	"	160	×9	200	11.304	4.5
1	Pl.	"		225	"	2.5
2	Ls	90×75	×9	455	11.02	10.0
3	"	"		160	"	5.3
5	Bars	70	×9	400	4.946	9.9
						205.7
						×2
						411.4
					Grand Total	1,820.6

Total Summary of Weight

2 Girder span @ 23,242.7		=	46,485.4
Expansion Joint		=	1,820.6
Rivet heads	2.3 % abt.	=	1,100.0
			49,406.0 Kgs

or 49.4060 Kg. tons

増田淳氏関係資料（独立行政法人 土木研究所蔵）

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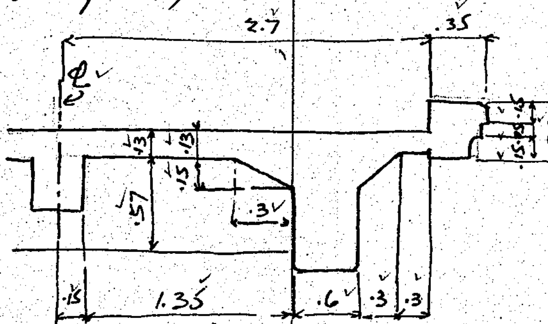
MADE BY K. Inaba DATE 4-9-30 FILE NO
CHECKED BY M.J DATE 4-10-2 PAGE NO 61

CALCULATIONS FOR

Materials of Nakagawa-Bashi for Okayama ken.

Materials for Floor. (Materials for 1 set of 3 continuous spans.)
Concrete 1:2:4 mixture.



Sectional area
Slab .13 × 5.4 = .702
coping .35 × .30 (average) × 2 = .210
main beam web. .6 × .744 × 2 = .893
" fillets .15 × .3 × 2 = .090
Stringer web .3 × .3 × 1 = .090
1.985 □m

Total length of floor = 30.63 m

1.985 × 30.63 = 60.80 m³

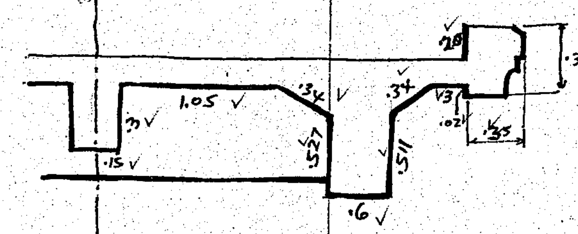
Cross beams. 8 @ .3 × .57 × 1.05 × 2 = 2.88
8 @ .3 × .27 × .3 = .19
8 @ .3 × .3 × .602 × 2 = .87
fillets for main beams—12 @ .6 × .3 × .98 = 2.12
fillets for stringer ends.—18 @ .3 × .08 × .25 = .11

cross beam over fixed piers not included here.

Total concrete for 1 set of 3 continuous spans = 66.97 cub. meters
" " for 3 sets " = 200.91 " "

Reinforcements Plain Bars. see drawing 9.948 kg tons × 3 = 29.8440 kg tons.

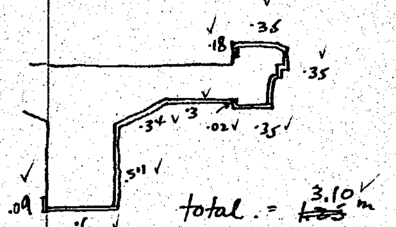
Forms.
1 Total width of form 10.08 m total length of floor = 30.63 m
Slab beam stringer + coping 10.08 × 30.63 = 309.08 m
Cross Beam 8 @ 1.05 × .57 × 4 = 19.16
" 8 @ .3 × .27 × 2 = 1.30
" 8 @ .3 × .602 × 2 = 5.78
main beam fillet 12 @ .3 × .98 × 2 = 7.05
Both ends area 2 × 1.985 □ = 3.97
less on piers 2 × .3 × 3.0 = (−) 1.80



total width 5.038 × 2 = 10.08 m

Forms for 1 — 3 continuous spans = 344.5 sq meter
for 3 sets of " " = 1033.5 " "

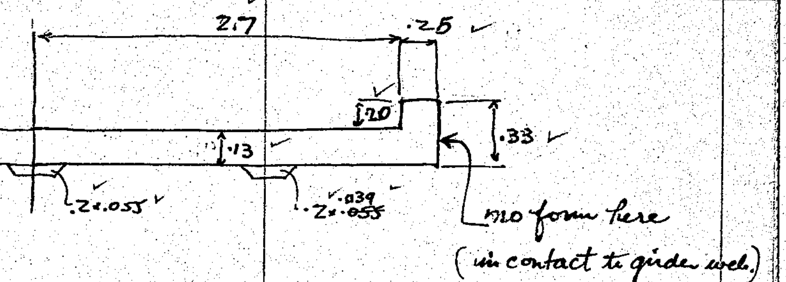
人造洗出仕上.



~~1.25~~ 3.10 × 2 × 30.63 = 190.00 m²
12 @ .3 × .98 × 2 = 7.05
less 12 @ .6 × .375 = (−) 2.70
1 set of 3 cont. spans = 194.35 □m
3 sets " " = 583.1 sq. meters.

total = ~~1.25~~ 3.10 m

Steel Girder spans.
Concrete 1:2:4 mixture.



Sectional area = 2.7 × .13 × 2 = .702
.25 × .33 × 2 = .165
.2 × .055 × 1 = .011
.2 × .039 × 2 = .016
.894 □m

Total length of floor = 2 @ 20.95 = 41.9 m
.894 × 41.9 = 37.45 m³
on cross beams. 10 @ .028 × .2 × 5.9 = .33
For 2-spans = 37.78 cub. meters

Forms. Total width 6.46 m
6.46 × 41.9 = 270.8
less stringer top. .15 × 4.9 × 3 × 2 = (−) 4.3
" floor beam .18 × 5.9 × 10 = (−) 10.6
255.6 m²

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MADE BY K. Inaba DATE 4-9-30 FILE NO
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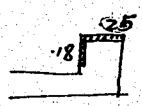
CALCULATIONS FOR

Materials of Nakagawa Bashi for Okayama Ken

Reinforcement for Steel girder span slab. Plain Bars.
2 spans @ 2.4452 = 4.8904 kg.tons.

人造洗出仕上.
0.43 × 2 × 41.9 = 36.0 m² for 2 spans.



Summary of materials for Floor.

	Concrete Beam spans.		Steel girder spans		Handrail (P63)		Total.	
Concrete 1:2:4 mix.	200.91	+	37.78	+	7.02	=	245.71	m³
Reinforcements, plain bars	29.844	+	4.8904	+	1.9607	=	36.6951	kg tons
Forms.	1,033.50	+	255.6	+	167.4	=	1,456.50	m²
人造仕上	583.10	+	36.0	+	172.0	=	791.1	m²

Pavement. Granolithic pavement 5 cm thick.
Total length 134.165 m face to face of parapet walls.
Less Exp. jts. .1×2 + .075 + .05×3 = − .425
" Const. jts .01 × 2 = − .020
133.72 m
Total area of pavement = 133.72 × 5.4 = 722.0 sq. meters.

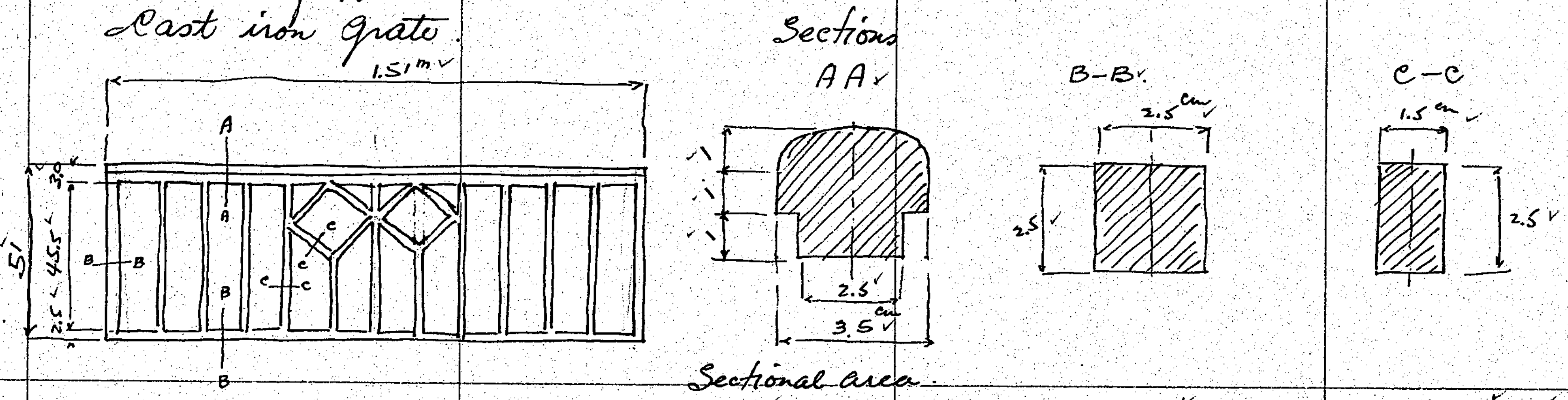
Cast iron drains
10 drains req'd. for steel girder spans
18 " " " concrete beam "
28 drains.
weight of cast iron 14 kg per drain specified.

Construction joints 2 joints for steel spans only. 3/8" Elastite filling.

Structural steel for Beam Shoes. (Concrete spans).
Shoes on ~~fixed~~ piers P3 ~~&~~ P6 4 @ .2547 = 1.0188 kg tons
" for end bearings 4 @ .1340 = .5360
1.5548 kg. tons.

Materials of Nakagawa-Bashi for Okayama Ken.

Materials of Handrail
Cast iron grate



Sectional area.

$3.5 \times 1.7 = 5.95$
$1.0 \times 2.5 = 2.50$
8.45 cm²

$2.5 \times 2.5 = 6.25$ cm²

$1.5 \times 2.5 = 3.75$ cm²

Top horizontal bars	$8.45 \times 151 =$	1275
Bott. 〃 〃	$6.25 \times 151 =$	944
End vertical bars	$6.25 \times 45.5 \times 2 =$	569
Int. 〃 〃	$3.75 \times 45.5 \times 9 =$	1537
〃 〃 〃	$3.75 \times 23 \times 2 =$	173
Diagonal 〃	$3.75 \times 17 \times 8 =$	510
		5008 cu.cm @ 0.0072 = 36.1 kg

54 grates required for one side @ 36.1 = 1949 kg or 1.949 kg tons × 2 = 3.90 kg tons. for both sides

Light pole. Bronze Lamp 8 req'd for the whole bridge (including pole on pedestals).
wt. of Bronze specified 9.0 kg per lamp.
Cast iron pole 8 required for the whole bridge
wt. of Cast iron specified 75.0 kg per pole.
gas pipe 1"φ × 1.5m − 1本

Concrete 1:2:4 mixture.
Concrete stanchions.
$57 \times 2 = 114$ poles req'd for both sides.
$.2 \times .2 \times .85 \times 114 = 3.88$ cub meters.
Toprails
$54 \times 2 = 108$ rails req'd for both sides
$.14 \times .14 \times 1.485 \times 108 = 3.14$

Total concrete for Handrail = 7.02 cub meters

Forms. Posts $114 @ .8 \times .85 = 77.5$
Toprails $108 @ .56 \times 1.485 = 89.9$
167.4 sq meters.

Reinforcements Plain Bars 1.9607 kg tons. see drawings.

人造洗ヒ仕上
Area of Forms − − − 167.4
plus top of posts $114 @ .04 =$ 4.6
∴ 172.0 sq. meters.

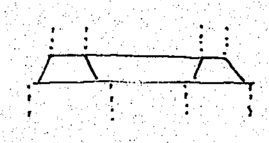
CALCULATIONS FOR

Materials for Nakagawa-Bashi for Okayama Ken.

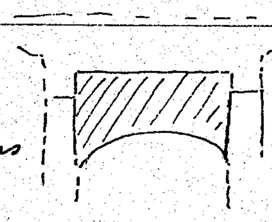
Materials of Piers for Concrete Beam Spans.
Concrete 1:2:4 mixture

Base 2.0m deep. 2.0 × 2.2 × 2.0 = 8.8 × 2 = 17.60 m^3 P1.2.3.4
" 1.7m " 2.0 × 2.2 × 1.7 = 7.48 × 2 = 14.96 m^3 P 5.6.7.8

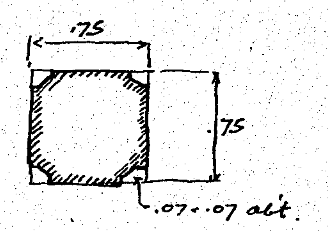
Bottom of columns. .975 × .975 × .75 = 0.71 × 2 = 1.42
Bottom strut. .6 × .75 × 2.63 = 1.18
Top beam. .6 × .9 × 2.85 = 1.54 } 2.60
4.14 m^3 for Exp. piers



wall above top beam .3 × .69 × 3.0 = .62
less stringer .3 × .3 × .3 = −.03
.59
4.73 m^3 for fixed piers



Column sectional area.
.75 × .75 = .562
chamffer .07 × .07 × 4 = −.020
.542 $\square^m$ × 2 = 1.084 $\square^m$ for 2 cols.



Summary of concrete for One pier.

Pier nos.	Column length	Section	Column	Top strut	wall	Bott. strut & block	Base	Total concrete for 1 pier	
P1	3.813	× 1.084 =	4.13 +	1.54 +	.59 +	2.60 +	14.96 =	23.82 m^3	average P1.2.3.4
P2	3.916	" " =	4.25 +	1.54 +	.59 +	2.60 +	14.96 =	23.94	
P3	3.974	" " =	4.31 +	1.54 +	0 +	2.60 +	14.96 =	23.41	23.95
P4	4.061	" " =	4.40 +	1.54 +	.59 +	2.60 +	14.96 =	24.09	
P5	4.102	" " =	4.45 +	1.54 +	.59 +	2.60 +	17.60 =	26.78	P5.6.7.8
P6	4.098	" " =	4.44 +	1.54 +	0 +	2.60 +	17.60 =	26.18	
P7	4.123	" " =	4.47 +	1.54 +	.59 +	2.60 +	17.60 =	26.80	26.79
P8	4.103	" " =	4.45 +	1.54 +	.59 +	2.60 +	17.60 =	26.78	

Forms.
Well
outside 8.4 × 2.0 = 16.8
inside 6.0 × 2.05 = 12.3
bottom 7.8 × .15 = 1.2
30.3 × 2 = 60.60 m^2 for Piers P5.6.7.8

outside 8.4 × 1.7 = 14.27
inside 6.0 × 1.75 = 10.50
bottom 7.8 × .15 = 1.2
25.97 × 2 = 51.94 m^2 for Piers P1.2.3.4

Bottom footings of column.
0.80 × (3.9 − .6) = 2.64
Bottom strut. 0.75 × 2 × 2.625 = 3.94
0.60 × 1.6 = .96
7.54 m^2 for all piers.

Top strut + wall. 0.9 × 2.85 × 2 = 5.13
bott. 0.6 × 3.2 = 1.92
top .3 × 3.0 = .90
wall .69 × 3.0 × 2 = 4.14
end .6 × .3 × 2 = .36
less .3 × .3 × 2 = (−) .18
12.27 m^2 for P1.2.4, 5.7.8

Top strut ~~+ wall~~. 0.9 × 2.85 × 2 = 5.13
bott. 0.6 × 3.2 = 1.92
top ~~0.6 × 3.0 = 1.80~~
end 0.6 × .3 × 2 = .36
7.41 m^2 for P3 and P6.

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MADE BY K. Inaba DATE 4-9-30 FILE NO

CHECKED BY M.K DATE 4-10-2 PAGE NO 65

CALCULATIONS FOR

Materials of Nakagawa-Bashi for Okayama ken

Summary of Forms for one pier

Pier nos.	Column length	width	.6×.9×2		Column		Topstrut + wall		Bottom footing + strut		wells.		Total.	average.
P1	3.813 ×	3.0 −	1.08	=	10.36	+	12.27	+	7.54	+	51.94	=	82.11 m^2	P1,2,4
P2	3.916	"	− "	=	10.67	+	12.27	+	7.54	+	51.94	=	82.42	82.40
P3	3.974	"	− "	=	10.84	+	7.41	+	7.54	+	51.94	=	77.73	
P4	4.061	"	− "	=	11.10	+	12.27	+	7.54	+	51.94	=	82.85	
P5	4.102	"	− "	=	11.23	+	12.27	+	7.54	+	60.60	=	91.64	P5,7,8
P6	4.098	"	− "	=	11.21	+	7.41	+	7.54	+	60.60	=	86.70	91.06
P7	4.123	"	− "	=	11.29	+	12.27	+	7.54	+	60.60	=	91.70	
P8	4.103	"	− "	=	11.23	+	12.27	+	7.54	+	60.60	=	91.64	

Reinforcements. Plain Bars see drawing (no 7)

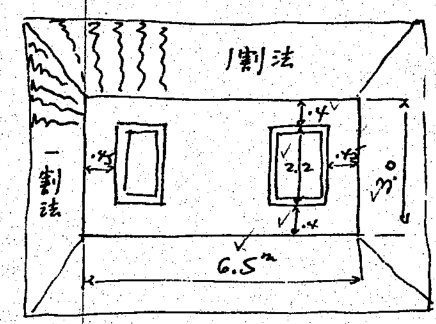
Pier nos.	P1, 2, +4	P3	P6	P5, 7, +8
Pier frame	1.2818	1.2144	1.2144	1.2818
2 − wells.	.5232	.5232	.5886	.5886
Total	1.8050 Kg tons	1.7376	1.8030	1.8704 Kg tons.

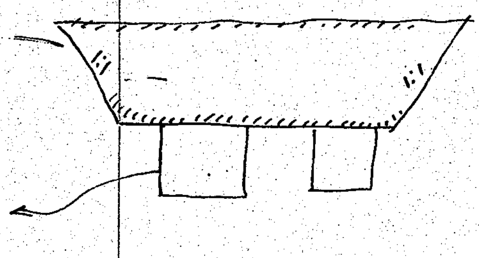
Excavation.

well sinking for P1,2,3,4 2.0×2.2×1.7×2 = 15.0 m^3 for one pier
" P5,6,7,8 2.0×2.2×2.0×2 = 17.6 m^3

Excavation above top of wells.

Pier nos.	mean depth		mean width		mean length		Excavation m^3	
P1	2.2	×	5.2	×	8.7	=	99.5	average
P2	2.0		5.0		8.5		85.0	74.4 m^3
P4	1.2		4.2		7.7		38.8	
P3	1.0		4.0		7.5		30.0	
P5	1.8		4.8		8.3		71.7	
P7	1.9		4.9		8.4		78.2	68.0
P8	1.5		4.5		8.0		54.0	
P6	2.0		5.0		8.5		85.0	





Excavation for sinking well.

2.0 × 2.2 × 1.7 × 2 = 14.96 m^3 for P1, 2, 4, + 3
2.0 × 2.2 × 2.0 × 2 = 17.60 for P5, 7, 8, + 6

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CALCULATIONS FOR

Materials for Nakagawa-Bashi, Okayama-ken.

Materials for Pier P10. (Plate girder span)
Concrete 1:2:4 mixture.

Shaft.	Coping	1.35 × .30 × 6.70 =	2.51	
	"	1.35ϕ × .30 =	.43	
				2.94 m^3
	Shaft.	1.438 × 4.757 × 6.20 =	42.40	
	"	1.438ϕ × 4.757 =	7.73	
				50.13
well. shell + fill.		2.5 × 2.4 × 6.20 =	37.20	
		2.5ϕ × 2.40 =	11.78	
				48.98
	Total concrete for Pier P10	=		102.05 cub. meters.

Reinforcements. Plain Bars. 2.1455 kg tons. see drawing.

Forms.	Coping	.375 × 6.20 × 2 =	4.65	
	"	1.35ϕ × .375 =	1.59	
				6.24 m^2
	Shaft.	4.757 × 6.20 × 2 =	59.00	
	"	1.438ϕ × 4.757 =	21.51	
				80.51
	Well outside	2.25 × 6.20 × 2 =	27.90	
	" "	2.5ϕ × 2.25 =	17.67	
	" inside	2.49 × 6.20 × 2 =	30.87	
	" "	1.9ϕ × 2.49 =	14.87	
	" less	0.7 × 2.0 × 4 =	(−) 5.60	
	" partition	1.5 × 1.95 × 4 =	11.70	
	" " bott.	0.46 × 1.9 × 2 =	1.75	
	" fillet	.28 × 1.95 × 8 =	4.37	
				103.53
	Total Form for Pier P10.	=		190.28 sq. meters

Structural steel for curb shoe = 0.4605 kg tons. see drawing.

Materials for Pier P9
Concrete. 1:2:4 mixture.

Shaft.	Coping	same as for Pier P10. =	2.94 m^3	
	Shaft.	1.425 × 4.488 × 6.20 =	39.64	
	"	1.425ϕ × 4.488 =	7.16	
				49.74
well.	Shell + fill	2.5 × 2.2 × 6.2 =	34.10	
	"	2.5ϕ × 2.2 =	10.80	
				44.90
	Total concrete for pier P9	=		94.64 cub. meters + .38 = 95.02 m^3

Reinforcements. Plain Bars 2.1395 kg tons. see drawing.

Structural steel for Curb shoe = 0.4605 kg tons. see drawing.

Concrete blocks under bearings
.65 × .70 × .417 × 2 = .38 m^3

Structural steel for Beam Shoes. Beam ノ部 = 計上ス.
~~0.1268 kg tons.~~

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(独立行政法人 土木研究所蔵)

Materials of Nakagawa-Bashi for Okayama ken.

Forms.

Coping	Same as for Pier P10	=	6.24 m²
Shaft.	4.488 × 6.20 × 2	=	55.31
〃	1.425φ × 4.488	=	20.09
			81.64
Blocks under bearing	.417 × 2.7 × 2	=	2.25
			83.89
Well outside	2.05 × 6.20 × 2	=	25.40
〃 〃	2.5φ × 2.05	=	16.10
〃 inside	2.29 × 6.20 × 2	=	28.40
〃 〃	1.9φ × 2.29	=	13.67
〃 less	0.7 × 1.8 × 4	= (−)	5.04
〃 partition	1.5 × 1.75 × 4	=	10.50
〃 bottom	0.46 × 1.9 × 2	=	1.75
〃 fillet	.28 × 1.75 × 8	=	3.92
			94.70
Total form for Pier P9		=	178.59 sq. meters

Materials of Abutment for plate girder span A2.

Concrete 1:2:4 mixture.

Shaft.			
Parapet wall.	.40 × 1.28 × 5.4	=	2.76
Columns	1.0 × 1.0 × 5.695 × 2	=	11.39
Curtain wall.	0.6 × 4.82 × 7.0	=	20.25
coping projection	.075 × .3 × ~~7.27~~ 8.35	=	.19
less notches.	.25 × .15 × .345 × 2	= (−)	.03
			34.56
2-Wells. shell + fill	2.5 × 2.5 × 4 × 2	=	50.00
			84.56 cub. meters

Granite 踏掛石

6 blocks @	.24 × .25 × .90	=	0.324 cub. meter

Forms. Shaft.

Parapet wall. front	.53 × 5.40	=	2.86
〃 rear	1.63 × 5.40	=	8.80
Coping	.375 × 8.50	=	3.19
Curtain wall front.	4.52 × 8.20	=	37.05
〃 〃 rear	3.92 × 5.40	=	21.18
Column outside + rear	2.0 × 5.695 × 2	=	22.79
〃 inside	.6 × 1.475 × 2	=	1.77
〃 〃	1.0 × 4.07 × 2	=	8.14
〃 〃	.095 × 1.0 × 2	=	.19
〃 front	1.0 × .875 × 2	=	1.75
〃 〃	.2 × 4.82 × 2	=	1.93
Curtain wall bottom	.6 × 3.4	=	2.04
			111.69
2-Wells			
outside	2.5 × 4.0 × 4	=	40.00
〃	2.5 × 2.5 × 4	=	25.00
inside	2.56 × 3.2 × 4	=	32.78
〃	2.56 × 1.7 × 4	=	17.41
bottom.	0.2 × 4.0 × 4	=	3.20
〃	0.2 × 2.1 × 4	=	1.68
			120.07
Total forms for abutment A2.		=	231.76 sq. meters

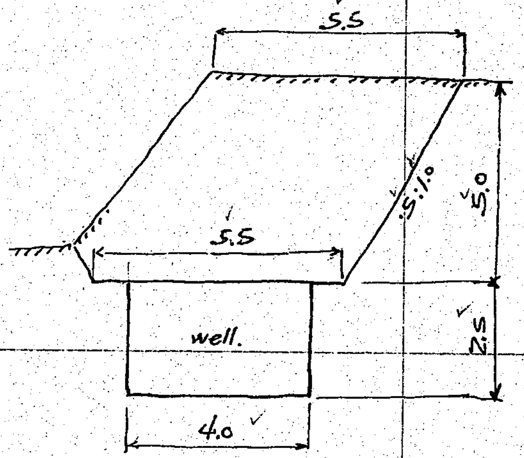
Materials of Nakagawa Bashi for Okayama ken.

Reinforcements for Abutment A2.
Plain Bars. 3.4744 kg. tons. See drawing.

人造洗出仕上

Column	front	.75 × .875 × 2	=	1.31
〃	outside	1.0 × 2.5 × 2	=	5.00
〃	front	0.2 × 2.1 × 2	=	.84
〃	rear + inside,	0.2 × 1.0 × 2 × 2	=	.80
〃	top.	2 × {(1.0 × 1.0) − (.8 × .85)}	=	.64
				8.59 sq. meters. for A2.

Excavation.



Excavation above top of well. (水上).
5.5 × 5.0 = 27.5 ㎡
27.5 × 13.4 = 368 cub. meters

Excavation for Sinking wells. (水中)
4.0 × 2.5 × 2.5 × 2 = 50 cub. meters

Materials for Abutment A1. for concrete beam span.
Concrete 1:2:4 mixture.

Parapet wall	.14 × .15 × 5.4	=		.11
〃	.4 × .946 × 5.4	=		2.04
Columns	1.0 × 1.0 × 3.2 × 2	=		6.40
〃 less	.489 × 1.91 × .3 × 2	=	(−)	.56
Shaft.	1.05 × 3.22 × 6.0	=		20.30
Coping	.3 × .075 × 7.15	=		.16
Base	2.25 × .6 × 6.5	=		8.78
〃	.6 × 3.3 × 6.5	=		12.87
				50.10 cub meters.

Granite 踏掛石
6 blocks @ .24 × .25 × .90 = 0.324 cub. meter.

Forms.	Parapet wall	front.	1.086 × 5.4	=	5.86
	〃	〃 rear	.14 × 5.4	=	.76
	〃	〃	.946 × 5.4	=	5.11
	Column	outside + rear	1.0 × 3.2 × 2 × 2	=	12.80
	〃	front	1.0 × 1.291 × 2	=	2.58
	〃	〃	.7 × 1.909 × 2	=	2.67
	〃	inside	.6 × 1.291 × 2	=	~~2.29~~ 1.55
	〃	〃	.511 × 1.909 × 2	=	1.95
	〃	bottom	.422 × 1.0 × 2	=	.844
	〃	〃	.7 × .578 × 2	=	.81
	〃	inside (curb)	.4 × .2 × 2	=	.16
	Shaft.	coping	.375 × 7.3	=	2.74
	〃	front	6.0 × 2.92	=	17.53
	〃	side	.5 × 2.92 × 2	=	2.92
	〃	〃	.739 × 1.309 × 2	=	1.94
	〃	rear	5.4 × 3.22	=	17.39
	〃	〃	.3 × 1.309 × 2	=	.79
	Base	all around.	.6 × 19.6	=	11.75
	〃	end	2.25 × .6 × 2	=	2.70
					92.85 sq. meters for A1

Materials of Nakagawa Bashi for Okayama ken.

Reinforcements for abutment A1.
Plain Bars. 1.2522 kg tons. see drawing

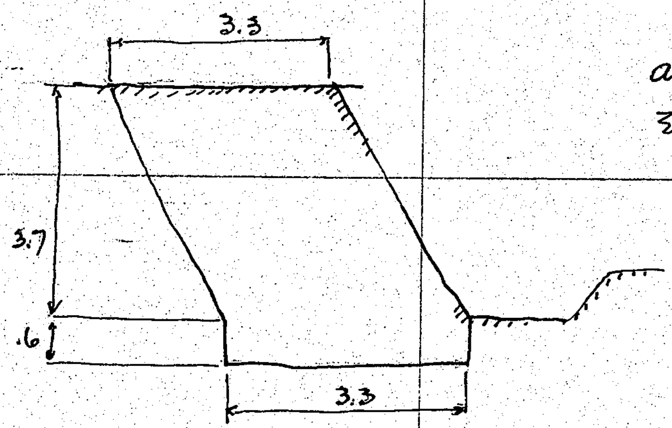
Structural steel for Beam shoes. Beam 1 当り = ?

人造洗出仕上

Column	front.	1.0 × 1.291 × 2	=	2.58
〃	outside	1.0 × 2.5 × 2	=	5.00
〃	front	0.7 × 1.71 × 2	=	2.39
〃	rear & inside	.2 × 1.0 × 2 × 2	=	.80
〃	top	{(1.0×1.0) − (.8×.85)} × 2	=	.64
parapet	front	1.086 × .6 × 2	=	1.30
				12.71 sq. meters

Excavation

average area = 3.3 × 3.7 = 12.20
3.3 × .6 = 1.98
14.18 m²



average length say 10m
Excavation 14.18 × 10 = 142 cub meters.

Materials for Light Pedestal. 親柱.
Concrete 1:2:4 mixture.

.8 × .85 × .85	=	.58
.8 × .55 × .40	=	.20
.45 × .7 × .06	=	.02
.3 × .3 × .25	=	.03
		.83 cub. meter.

人造洗出仕上

.85 × 3.3	=	2.81	4.27 area of forms.
.40 × 2.7	=	1.22	
.04 × 2.3	=	.09	
.25 × .3 × 2	=	.15	
.8 × .85	=	.68	
		4.95 sq. meters.	

Bronze name plate .20m × .50m —— 1枚. wt. of bronze 5.0 kg. specified.

Lamp. 1-set. same as for light pole in handrail
wt of bronze. 9.0 kg specified.
wt. of cast iron 75.0 kg 〃
~~gas pipe 1"φ~~

Gas pipe 〃 2"φ pipe × 1.6m long. 1本.

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CALCULATIONS FOR

昭和四年七月

岡山縣

中川橋予算設計書

$\frac{5.5}{270^2} = \frac{x}{180^2}$ $x = 24$.

729 324

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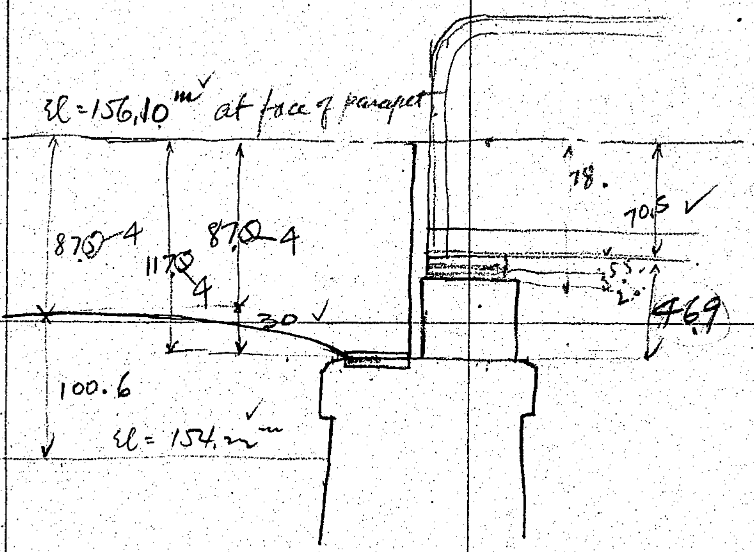
CALCULATIONS FOR

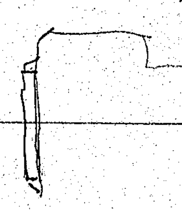
中川橋

Distance between faces of parapet wall = 134.19 m

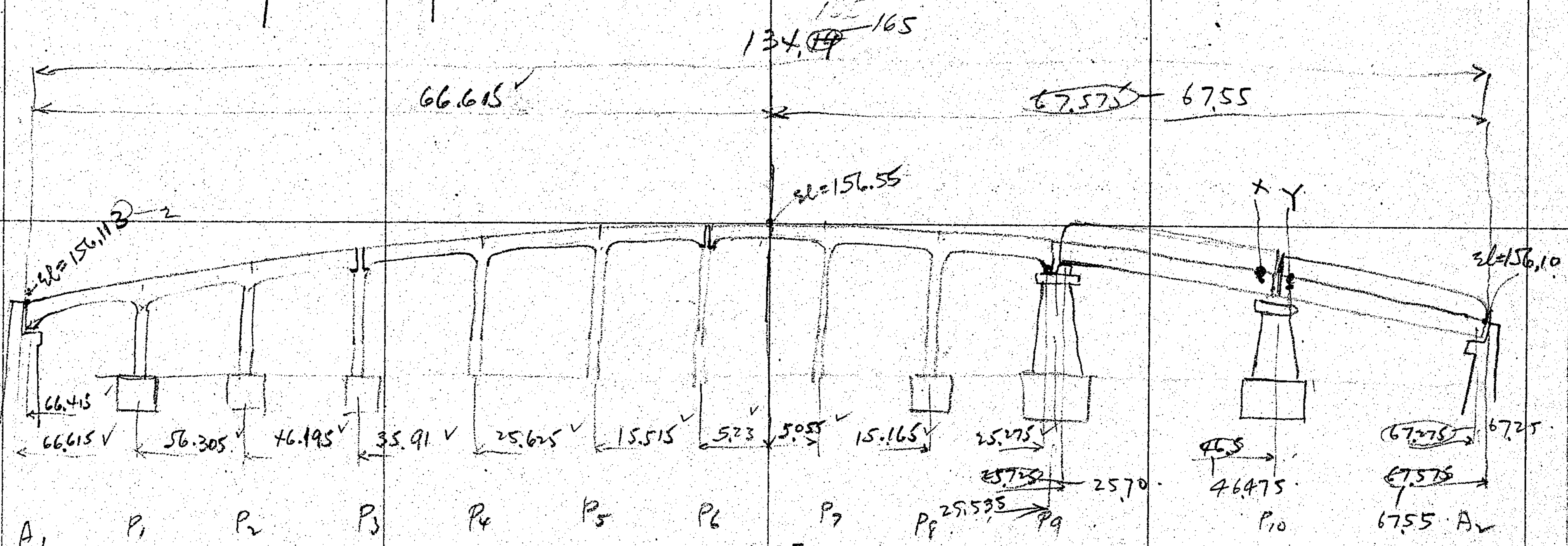
134.19 ÷ 2 = 67.095

156.104
.705
155.399
154.22
1.179





27.5 27.5



Parabola 1/150 right side f = Rise = ~~67.575~~ 67.55 ÷ 150 = 45 cm

$x = cl.$ $l = 2 \times$ ~~67.575~~ 67.55 = ~~135.15 m~~ 135.10

$y = 4fc(1-c) = 180\,c(1-c)$

	X.	$(1-c)$ = x/l	c	C	1-C	180 × C(1-C)	El. C/R
A1 parapet	66.615 ✓	0.4929	.5071	.0071 69	.9929 .9931	1.3 12	156.113 2
A1 bearing ℄	66.415 ✓	0.4914	.5086	.0086 84	.9914 .9916	1.5 ✓	156.115 ✓
P1	56.305 ✓	.4166	.5834	.0834 832	.9166 .9168	13.8 13.7	156.238 7
P2	46.195 ✓	.3418	.6582	.1582 1	.8418 .8419	24.0 ✓	156.340 ✓
P3	35.91 ✓	.2657	.7343	.2343 2	.7657 .7658	32.3 ✓	156.423 ✓
P4	25.625 ✓	.1896	.8104	.3104 3	.6896 .6897	38.5 ✓	156.485 ✓
P5	15.515 ✓	.1148	.8852	.3852 ✓	.6148 ✓	42.6 ✓	156.526 ✓
P6	5.23 ✓	.0387	.9613	.4613 ✓	.5387 ✓	44.7 ✓ vertex 45.0	156.547 ✓ 156.550 ✓
P7	5.055 ✓	.0374	.9626	.4626 ✓	.5374 ✓	44.7 ✓	156.547 ✓
P8	15.165 ✓	.1122	.8878	.3878 7	.6122 .6123	42.7 ✓	156.527 ✓
P9 Beam Bearing	25.275 ✓ 25.535	.1870	.8130	.3130 29	.6870 .6871	38.7 ✓	156.487 ✓
〃 Girder 〃	25.725 70	.1903	.8097	.3097 8	.6903 .6902	38.5 ✓	156.485 ✓
P10	46.50 475	.3441	.6559	.1559 60	.8441 .8440	23.7 ✓	156.337 ✓
A2 Bearing	67.275 25	.4978	.5022	.0022 ✓	.9978 ✓	0.4 ✓	156.104 ✓
A2 parapet	67.575 55	.5000	.5000	.0000 ✓	1.0000 ✓	✓	156.100 ✓
	46.2			.1580	0.8420	23.9	156.339 X
	46.75			.1540	0.8460	23.5	156.335 Y

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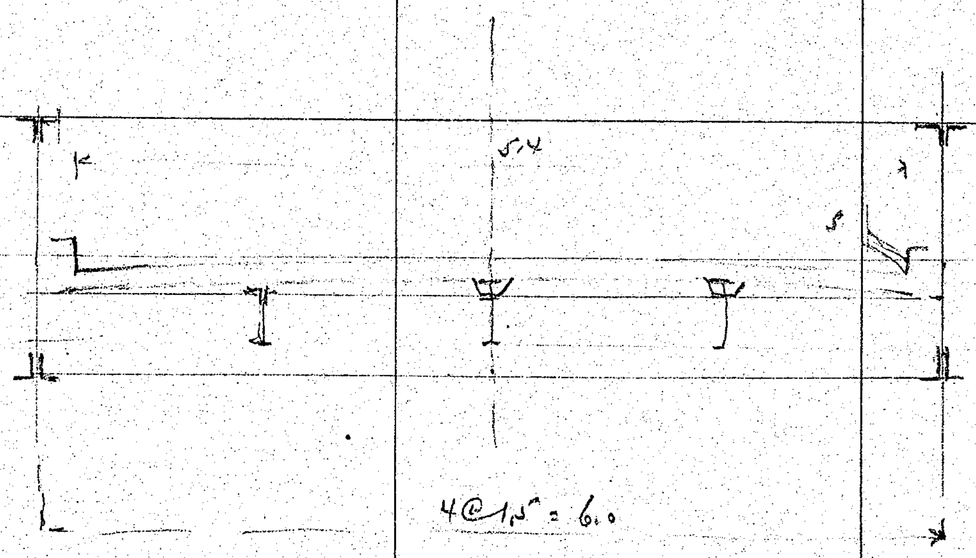
CALCULATIONS FOR

Preliminary Estimate of cost Nakagawa-Bashi.

Total span - 134.5 meters / width 5.4
Steel girder spans. 2 @ 21.0 meters.
Concrete span. 9 @ 10.3 meters

Design of steel span
cross section -

5.4
.6
6.0



13
5
450
1.475
1.500

Slab. 13 cm } @ 24. = 31.2
5 cm } @ 22 = 11.0
misc. 2.0
44.2 kg per sq meter

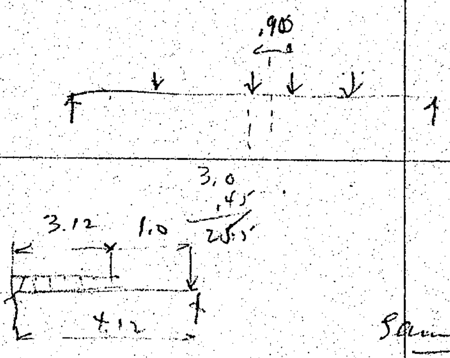
Panel length. 21.00 out to out
.40
20.60 ÷ 5 = 4.12

Stringers 3 @ 50 kg = 150 kg.
Floor beams span length 6.0 spacing 4.12
Dead load say 500 × 4.12 = 2060 kg per meter.

$m = 1/8 \times 2060 \times 6.0^2 = 9260$

motor truck loading. rear wheel. 2925
front wheel. 975 × $\frac{1.12}{4.12}$ = 265
2925
3190
2
6380 × 2.55 = 16300
3190 × 1.80 = 5750
10550
2660
13210
9260
22470

unif 1/8 × 590 × 6.0² =



Same as Tokiwa Bashi section modulus $\frac{22470}{11}$ = 2040
450 × 175 I @ 114.68 kg.
Sm = 2169

700 kg.

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CALCULATIONS FOR

Preliminary Estimate of Cost Nakagawa-Bashi Okayama-Ken

Main Girder.
Dead load.

500 · 3.0 = 1500.
misc. 200
1700 kg.

structural Steel say stringer 150
floor beam say 170
lateral say 75
Girder say 600
995 call this 1000 ÷ 2 = 500 for one girder.

1700
500
2200 kg.

moment = 1/8 × 2200 × 20.60² = 117000
9000
207000

Live Load. 3 × 500 = 1500

1/8 × 1700 × 20.60² =

web assumed 150 × 9 = 135 cm 1/8 web = 16.90

flange stress. 207000 / 1.47 = 141000 / 12 = 118.0
16.9
101.10 cm net

2 Ls 150 × 150 × 11 = 63.58 - 9.70 = 53.88
1 Pl 330 × 9 29.70 - 3.96 = 25.74
9 29.70 25.74
122.98 105.36

29.70
3.96
25.74

122.98 × 2 = 246.
135
381 @ .785 = 299. × 1.25 = 370. kg.

150
170
75
740
misc. 35
1170 × 21.0 = 25 tons = 50.0 tons
2 @ 25

50.0 @ 200 = 10,000
2270
12270

Floor. .18 @ 18.6 cubic @ 2.80
20 kg @ .130 2.60
forms 2.00
pavement. 1.50
8.90
1.1
10.00

54 × 42 =

10.0 @ 227. = 2270

Steel - 50 @ 200 = 10000
Deck. 2270
12270

Super structure. 12270

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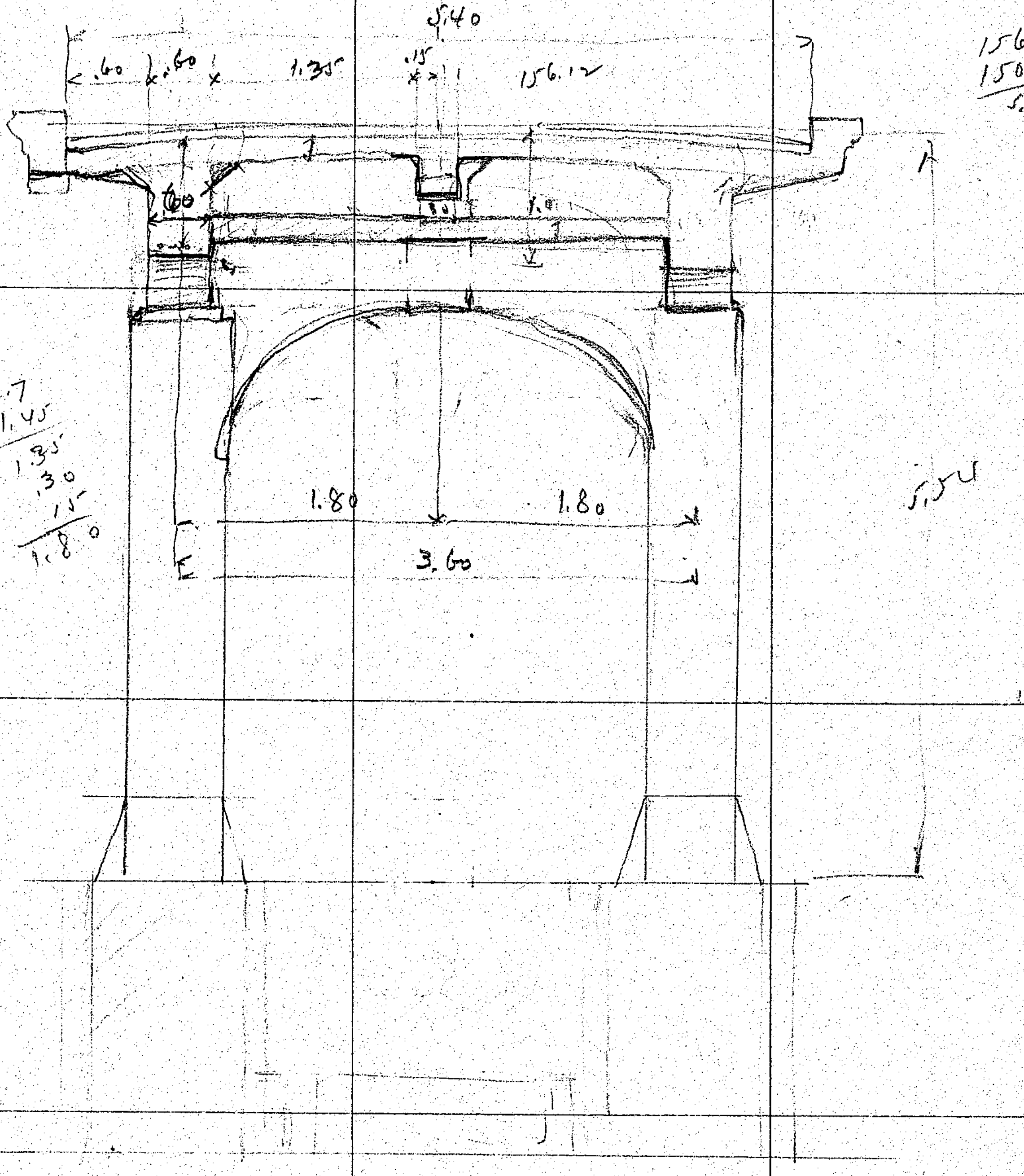
CALCULATIONS FOR

Preliminary Estimate of cost Nakagawa-Bashi for Okayama-ken

concrete 5 span.
span length — 10.3 meters.



156.12
150.58
5.54

2.7
1.45
1.85
.30
.15
1.80

5.40
.60
.60
3.00
.60
1.20
11.40

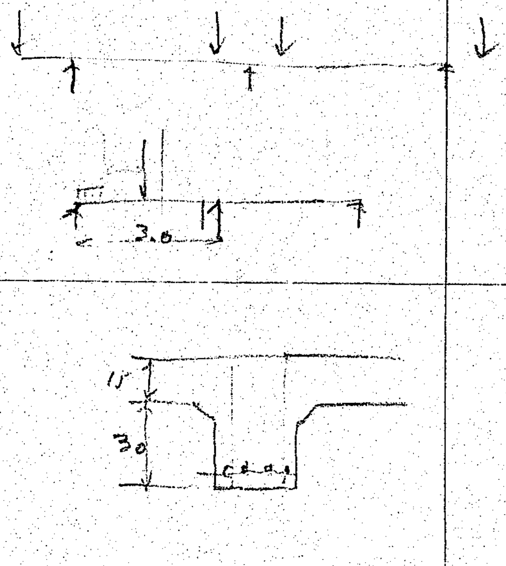
1.35
.30

concrete in floor slabs. 15 cm average assumed C. 24 = 360
5 cm pavement 110
470 kg.

span length of stringer — 3.0 meters

Dead load. 470 × 1.65 = 775
Stringer stem .30×.30 216
991

$m = \frac{1}{10} \times 991 \times 3.0^2 = 892.0$



Live load $2925 \times \frac{1.35}{1.80} = 2190$
2190
4380

$m = \frac{4380}{2} \times 1.5 = 3290.$

Unif. Bay 310
3600 × 0.8 = 2880
Dead load. 892
3772 kg.

steel reqd $= \frac{3772}{.40 \times 1200} = 7.86$
16 cm bars. 2.01   4 bars.

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CALCULATIONS FOR

Preliminary cost of Nakagawa Bashi for Okayama-Ken

cross beam. span length 3.60.
concentration -

991 × 3.0 = 2973   m = 2973 × 1.8 = 5350 × 0.8 = 4280
cross beam stem. .30 × .45 - e 2400 430   m = 1/10 × 430 × 3.6² = 558

4838
4369
9207

Live Load. 4380/2 × 1.8 = 3940.

Uniform load 1000 × 1/3 = 330   m = 1/10 × 330 × 3.6² = 429.0

3940
429.0
4369

Steel area = 9207 / (.55 × 1200) = 14.0 □cm

.45
.15
.60

19 mm bars. 6 × 2.83 = 17.0

Main girder
Dead Load.
slab. 470 × 2.7 = 1270
coping. 215
Handrail 150
stringer. 107
cross beam 165
main girder. .60 × .75 e=2400 = 1080
2987

m = 1/10 × 2989 × 10.3² = 31,600 kg.

Live Load.

5.15 4.15 1.0 3.0 2.15 10.3

2925
5850 ÷ 2 = 2925
1900 × 2.15/10.3 = 407
3332

270
3500
1350 × .5 × 5.15
4.0

Unif say

m = 3332 × 5.15 = 17150
8700

as simple beam — 25850 × .8 = 20600
31600 31600
57450 52200

Steel area = 52700 / (.80 × 1200) = 54.4

25 mm bars 4.90   54.4 / 4.90 = 11.1   12.0

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CALCULATIONS FOR Preliminary Estimate of Cost Nakagawa-Bashi Okayama-Ken

Concrete in one span.

slab	.15 × 5.4 =	.81
coping		.18
stringer		.09
cross beam		.135
main girder	.75 × .60 × 2	.900
		2.115 cubic meter say 2.2 cubic meter

Concrete	10.3 × 2.2 = 22.7	@ 18.61 =	422.—
Reinf. bars	22.7 × 165 = 3750	@ 130	488
forms	117.0 @	.270	316.—
finish	2.5 × 10.3 = 25.7 @	5.70 =	146.—
pavement 5.4 × 10.3 = 55.6		@ 1.50	83.5
Handrail	20.6 meters	20	416
			1841.00

2130

165

Cost of one bent.

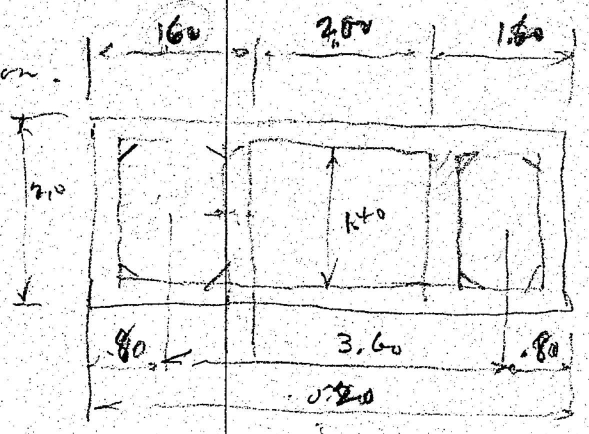
Dead load. $300 \times 10.3 \times \frac{5}{4} = 38600$ kg.

.75 × .75 @ 2400 = 1350 × 4.2 = 5700 / 44300

$\frac{44300}{.75 \times .75} = 7.7$

Base 5.4 × 2.0

Caisson.



concrete caisson 5.2 × 2 = 10.4
1.40 × 4 = 5.6
16.00 × .30 = 4.8
4.8 × 2 = 9.60 cubic m

inside filling 5.20 × 2 = 10.4
4.8
5.6

3.60

Concrete filling say 5.6 × 1.5 = 8.40 cubic meters.

volume of concrete in cols. 2 – .75 × .75 × 4.2 = 4.71 say including strut.

volume of concrete 5.00
5.60
9.60
20.20

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CALCULATIONS FOR

Preliminary Estimate Nakagawa Bashi Okayama ken

Estimate of one pier

	concrete	20-20 @	18.61 =	376
	steel -			170
	forms -			150
2.2 × 3.0 × 6.0	Excavation	40 metres @	1.20 =	48
	well			200
				944

~~1000~~ 1000

18
22
36
3.6
40

1000
1841
2841 × 9 = 25.600

concrete abut ments - 2000 2000 ✓
concrete piers - 20 2500 = 5000 ✓
3000 ✓
10,000

2 girder spans complete with deck 12270
concrete spans complete 25600
~~substructure~~ for girder + abut. piers 10000
47870

46500 予算額