JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

CALCULATIONS FOR

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白鬚橋予算設計書

大正十五年六月

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CALCULATIONS FOR

Preliminary Estimate of Shirahige-Bashi Tokyo Prefecture

From sketches made for layout of the new Bridge we finally determined to adopt 2 side span of 147'-6" each with cantilever projection of 24'-6" and 196'-0" tied arch for center span. Between end bearings

2 @	147'-6"	=	295'-0
2 @	24'-6"	=	49'-0
1 @	196'-0"	=	196'-0
			540'0"

Making panel length 12'3" each

12.25 × 16	=	196'-0"
12.25 × 4	=	49'-0"
		245'-0"
2 @ 147'-0"	=	294-0
		539'-0

10 @ 12'-3" = 122.5
2 @ 12'-3" 24.5
147.0

19.2
5.4
197-4

16
4
54
64 5.4

Making panel length

12'-4" × 16	=	197'-4"
2 - 12'-4" × 2		49'-4"
		246'-8"
2 @ 148'-0"	=	296'-0
		542'-8"

24'-8
49'-4"

12'-4"
12
144
4
148

Use this span length for Preliminary Estimate

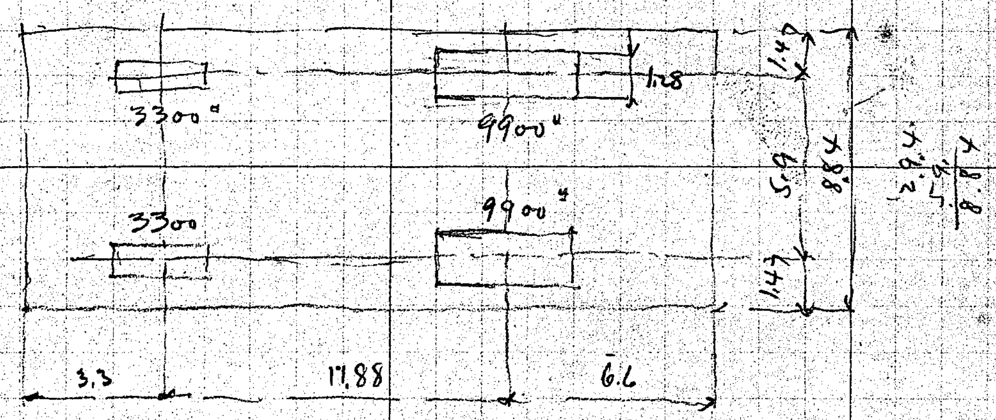
542.67
3.16
545.83

Assumed Loadings.

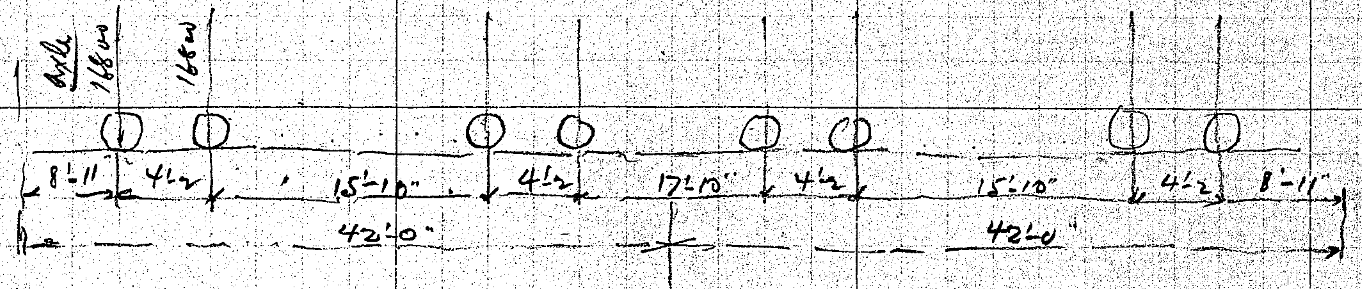
Uniform load 100# per lin ft on sidewalk $\frac{100.000}{170+l}$
120 " " " on roadway $\frac{120.000}{170+l}$

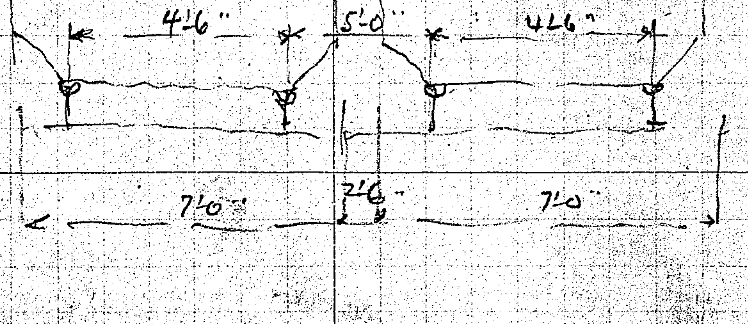
where l = span length in meter.

12 ton motor truck loading.



2 - 30 ton Bogie Car 4'-6" gauge.





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

No impact for uniform live load + road roller concentration.

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CALCULATIONS FOR

Preliminary design of Shirahige Bashi for Tokyo Prefecture

Let us Estimate structure for.

I. double track Electric Ry at center of bridge

16.5 @ 3.28 =	54.0	9 ken
2 side walks @ 12 =	24.0	4.0
	78.0	13 ken

II. no Electric Ry Roadway 6. ken

2 sidewalks. C 2 ken. 4.1 ken

10.0 ken

Design of Floor slabs.

Dead Load Soliditit pavement 2 3/8 28#

Concrete slab say 82

110# per sq. ft.

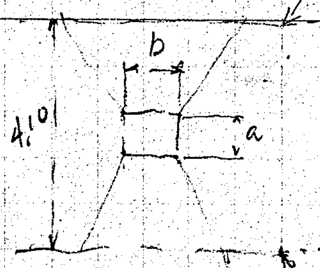
span length 4.10 $m = \frac{1}{10} \cdot 110 \cdot 4.10^2 = 185'^{\#}$

Live Load distribution motor truck wheel conc. 9900#

$a = .66 + 2 \cdot .2 =$ say 1.00

$b = 1.25 + 2 \times .2 = 1.65$

℄ Joist



Effective width $= \frac{2}{3}(4.1 + 1.65) + 1.00 = 4.84$

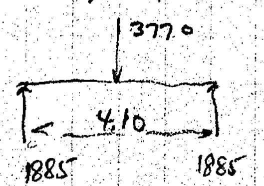
wheel conc.	9900
Impact 30%	3300
	13200#

$13200 \div 4.84 = 2730^{\#}$ per ft strip

Between rails. 4'-6". take effective width 3.5'

$13200 \div 3.5 = 3770^{\#}$ per ft strip.

Bending moment



$\frac{3770}{2} \cdot 2.05' = 3860'^{\#}$ for continuity of slab $3860 \times 0.8 = 3080'^{\#}$

DLm $= 185$

3265

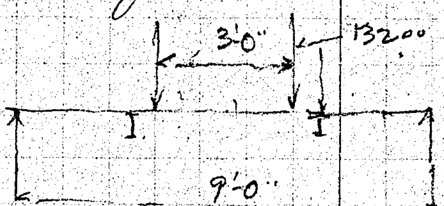
Effective depth reqd $= \sqrt{\frac{3265}{102}} = 5.65''$

1.10

$6.75''$

Steel area reqd $\frac{3265 \times 12}{\frac{7}{8} \cdot 5.65 \times 17000} = 0.465$ use ½" ϕ bars 4" centers.

Transverse Joist. span length 9'0"



Live load moment $13200 \times 3 = 39600'^{\#}$

Dead load.

pavement 28

slab. 6¾" @125. 84.

112# or say 115#/'

$115 \times 4.1 = 471.$

stringers 35

$506^{\#}$ $m = \frac{1}{8} \cdot 506 \cdot 9^2 = 5130'^{\#}$

39600

$44730'^{\#}$

$Sm = \frac{44730 \times 12}{\frac{?}{?} 17000} = 31.6$

use 12" · 31.9# I Sm = 36.6

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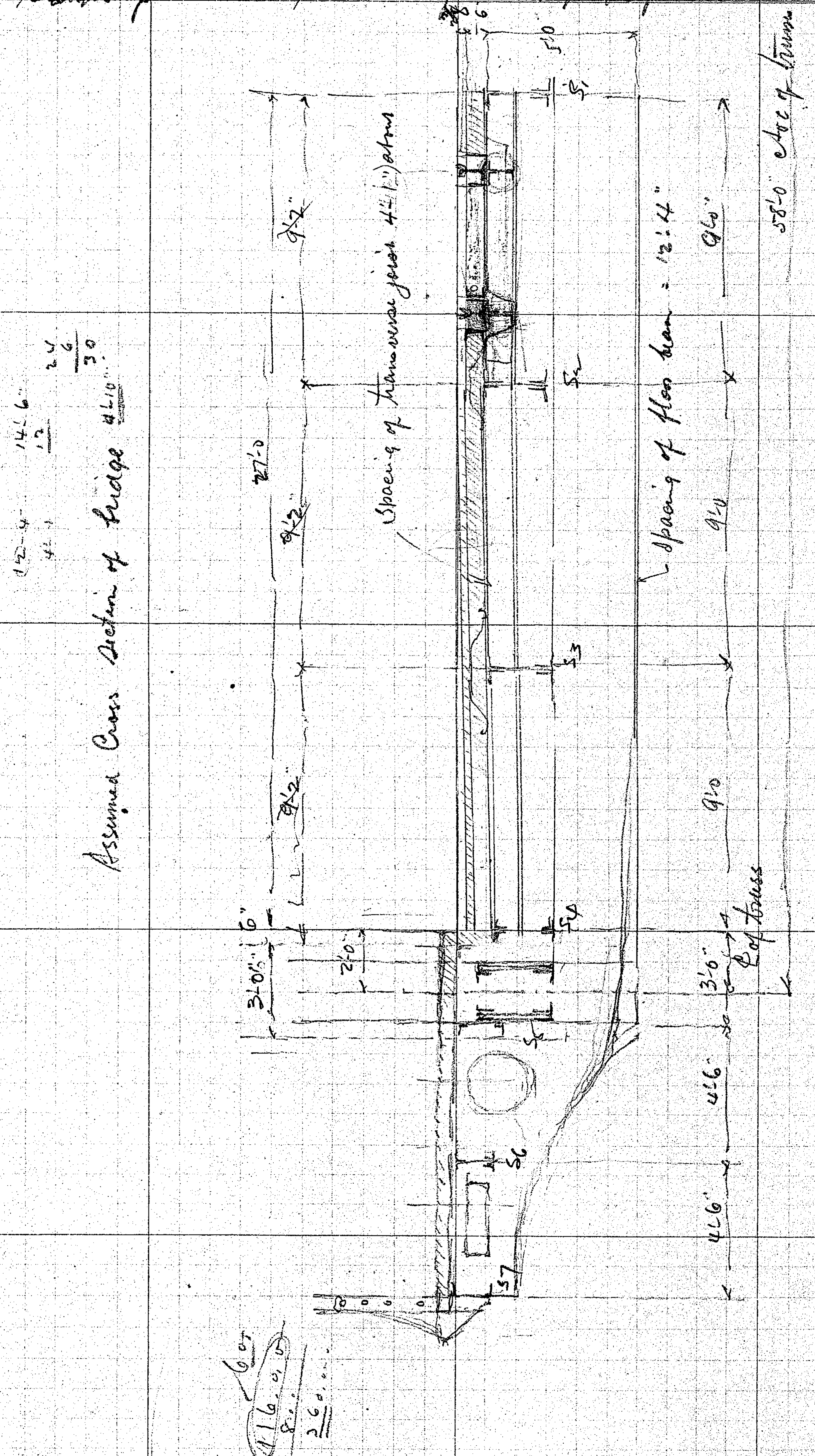
CALCULATIONS FOR

Preliminary Design of Shirahige-Bashi for Tokyo Prefecture



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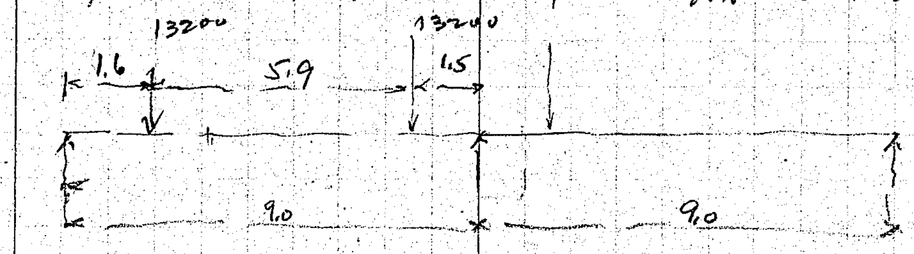
CALCULATIONS FOR

Preliminary Design of Shirahige Bashi for Tokyo Prefecture.

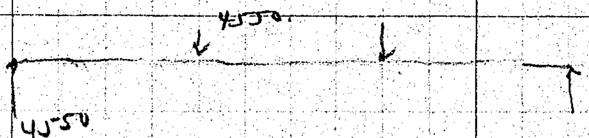
Longitudinal stringer   Span length 12'-4".



5.9
1.5
7.4
9.0
1.6
7.5
9.1

Dead Load Lone.   506# × 9.0 = 4550#

moment = 4550 × 4.11 = 18700'#



Live Load   single concentration of motor truck

13200 × 9.1/9.0 = 13350
13350
26700#

LL m = 26700/2 × 12.33/2 = 82300'#
DL m say 20000
102300

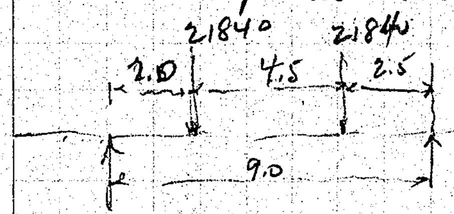
Depth of beam say 24½"   20.4 / .14 / 1.90 say 1.85   24 × 3/8 = 9.0   ⅛ web = 1.13

flange stress = 102300 ÷ 1.85 = 55300#

section required 55300 ÷ 17000 = 3.25
1.13
2.120" net   use 2 L 5 × 3½ × 3½ × 3/8 = 4.96"
1.50
3.46 net

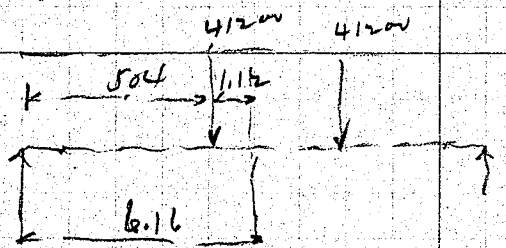
Electric Ry Car Loading.

16800
5040 30%.
21840# × 8.5/9.0 = 20600#
20600
41200#



2.0
6.5
8.5

4.5/4 = 1.12



12.33
6.16

LL moment = 82400 × 5.04² / 12.33 = ~~170000/2~~ = 85000
20000
105000

DL m say 20000
190000

flange stress = 190.000 ÷ ~~1.85~~ 2.10 = 90500 ~~102000~~

sR = ~~183000~~ 90500 ÷ 17000 = ~~6.06~~ 5.32
⅛ web = 1.13   1.27
4.93" net   4.05

.27" × 3/8 = 10.1   3½

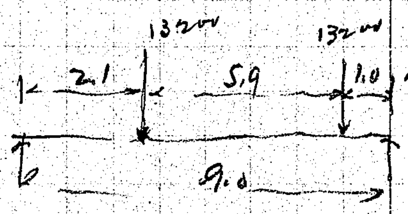
2 L 6 × 3½ × 3/8 = ~~6.10~~ 4.96
~~1.50~~ 1.50
~~4.60~~ 3.46
OK

Longitudinal stringer S2.

L.L moment   21840 × 9.5/9.0 = 23100#

2.5
7.0
9.5

absolute moment = 46200 × 5.04² / 12.33 = 95000



motor truck loading.

6.9

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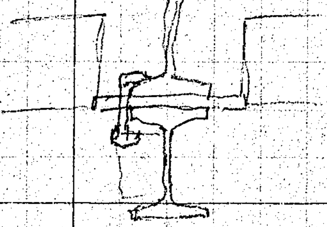
Sidewalk - Slab.

mortar 3/4" - 7.5
4 3/4" slab. 60.0
say 70.0 #/□'
Live Load say 100
170# · 4.5 = 765#
stringer say 35
800 $m = \frac{1}{8} \cdot 800 \cdot 12.33^2 = 15200'^{\#}$



$Sm = \frac{15200 \times 12}{17000} = 10.8$

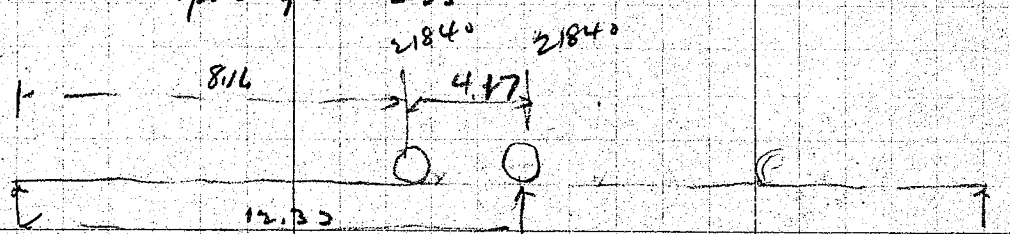
use - I 10" · 5" @ 29.99# - Sm 29.14

Approximate weight of transverse Joists
32# · 54 = 1730
Details say 90
1820# × 2 = 3640# per panel.
3640 ÷ 12.33 = 295# per lin ft.
Longitudinal stringer under rail - 8" · 4" I @ 18.01
1 Pl. 9 · 3/8 - 11.48
29.49
4 @ 32# = 128# per lin ft.

S1. 4 L. 3½ · 3½ · 3/8 @ 8.5 = 34.0
1 Pl. 24 · 3/8. 30.6
64.6 call this
12.0
76.6 say 77#

Approximate weight. 7 @ 77 = 540
4 @ 32 = 128
Sidewalk 2 @ 35 = 70
2 @ 25 = 50
fascia. 2 @ 50 = 100
888# — 888
Joist - 295#
1183# per lin ft of span.

Floor Beam spacing - 12.33



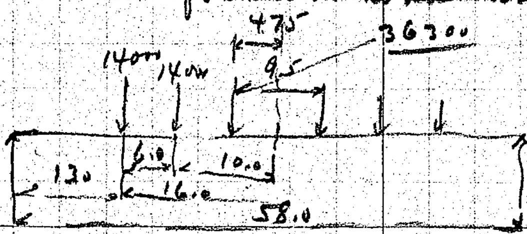
16800
5040
21840

12.33
4.17
8.16

21840
14500
36340

Motor truck conc. say ~~13200#~~ 14000 ~~8000~~
uniform live load assumed. 10 × 120 = ~~1200#~~ per lin ft.

5.
4.5



28000
36300
64300 - moment at center -
64300 × 29.0 = 1,865,000
36300 × 4.75 = 172,500 536,500
28000 × 13.0 = 364,000 1,328,500
536,500 332,000

5 29.0
16
13

Uniform load say $\frac{1}{8} \times 800 \times 58^2 = 7,665,500$ '#

337.
36000

Preliminary Design of Shirahige-Bashi for Tokyo Prefecture

64300
28000
92300

92300 × 29.0 = 2670000
36300 × 4.75 = 172500
28000 × 13.0 = 364000
28000 × 9.8/22.0 = 615000
1151500

2670000
1151500
1518500
840000
1602000

unif. 1/8 × 2000 × 58² =

12.33
6.6
5.73

Dead Load - 115# × 12.33 = 1420
570
1990 - call this 2000

170
400
570

D.L. m = 1/8 × 2000 × 58² = 840.000
negative moment say 140.000
700.000 #
L.L. moment. 1600.000
2300.000 #

Section at center of bridge -
60 × 7/16 = 26.30 □" 1/8 web - 3.29
Effective depth say 5.0.
flange stress = 2300.000 / 5.0 = 460.000
3R = 460.000 ÷ 17000 = 27.0
3.3
23.7 □" net.

2 L 6 × 6 × 3/4 - 16.88 - 3.00 = 13.88
2 Pls. 12½ × ½ = 12.50 10.50
29.38 24.38
29.38
26.30
85.06 @ 3.4 = 290#
Details say - 45% =

2 L 6 × 6 × 5/8 - 14.22 - 11.72
2 Pls. 12½ × 5/8 - 15.60 13.10
29.82 24.82
60 × 3/8 - 29.82
22.50
82.14 @ 3.4 = 280
Details say 40% = 120
400# per lin ft

weight of one floor beam = 400 × 58 = 23,200
150 × 20 = 3000
26,200# ÷ 12.33 = 2128 per lin ft
Stringers - 1200
3320# per lin ft.

810 tons

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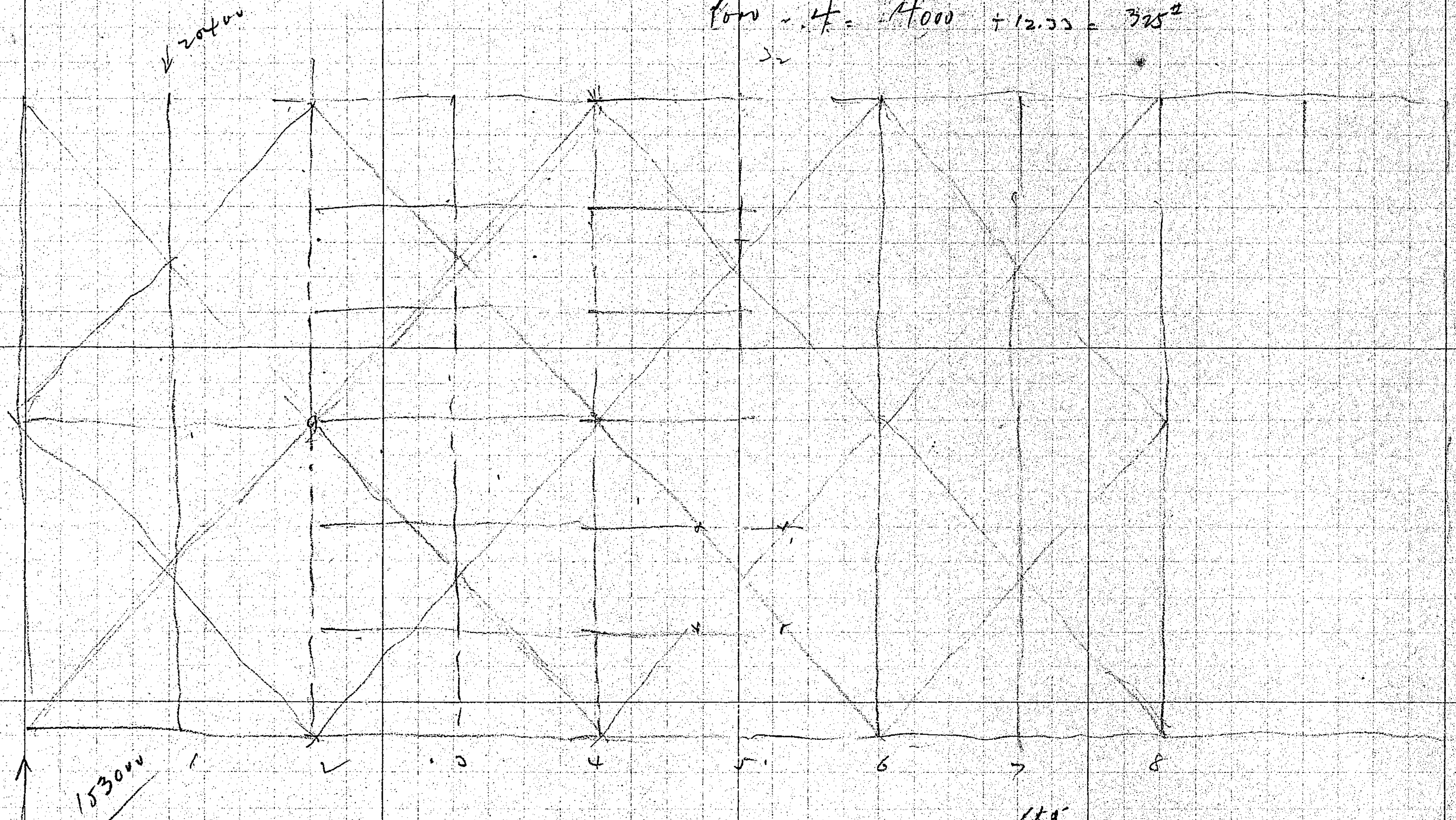
Lateral Bracings -

2Ls 6.6.½ @ 20° = 40 × 18 = 700
Details = 300
1000 × .4 = 4000 ÷ 12.33 = 325#
32



1700 × 12 = 20400.

180
20
20
160 × 40 = 6400
2000
8400 × 2 = 1700#

153000 × 1.4 = 214000 / 30600 = 7.00
17000 × 1.8 = 30600
8.72

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Preliminary Design of Shirahige-Bashi for Tokyo Prefecture-

Top lateral Bracings
Diagonals 325#
strut 5000 ÷ 12.33 405
730# per lin ft.

approximate wt of tied arch span length - 197.33'

approximate DL:

Flooring + pavement =	115# × 54 =	6200
Sidewalk slab	72# × 24 =	1720
curb		200
Handrails say 2@75		150
watermain + other misc. say		500
		8770 call this 8800

metal in floor. 3320
Lower laterals 325
upper laterals. 730
4375
13175# per lin ft.

Live Load 54 × 120 = 6500
24 × 100 = 2400
8900#

LL. 8900
misc. say 2000
24075
12000 per truss -

moment at center = 1/8 × 12,000 × 197.33² = 58,300,000 ft#
1500 40,000,000

Depth = 30 stress in the 58,300,000 ÷ 30 = 1,950,000 #
1,950,000 ÷ 17000 = 115.0 □ net
20
135.0 gross @ 3.4 = 460# per lin ft.
460 × 5.2 = 2400# per ft. 4800# for two trusses
4375
shoes say 225
9400# per lin ft.

9400 × 197.33 = 1860,000 # or 860 tons.

1650 × 800/300 = 1100 tons

Side spans. span length 148'0" 24'8" overhang.

Dead Load reaction from tied arch span 13175
4800
17975 say 18000#

18000 × 197.33/2 = 1,775,000# ÷ 2 = 887,500# per truss.

Live Load - 8900 × 197.33/2 = 880,000# ÷ 2 = 440,000# per truss.

Dead Load on side span 18000
upper laterals. - 730
17270 # per lin ft. 8635# per truss.

D.L. max negative moment.

R1 R2

tied arch 887,500 × 24.67 = 21,850,000
8635 × 24.67²/2 = 2620,000
262
24470,000 ft#

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Preliminary Design of Shirahige-Bashi for Tokyo Prefecture.

Live Load negative moment.

From arch 440.000 × 24.67 = 10.900.000 [#]
4450 × $\frac{24.67^2}{2}$ = 1.350.000

LL m 12,250.000
DL m 24.470.000
36,720.000

Depth 25.0 stress = 36.720.000 ÷ 25.0 = 1470.000 [#]
section required 1,470.000 ÷ 17000 = 86.50"
20% 17.3
103.8 □" @ 3.4 = 354 [#]

354 [#] × 3.2 = 1130
37% 420
1550 [#] per lin ft. of truss.

Positive moment between piers.
8635 [#] for truss m = $\frac{1}{8}$ · 8635 · 148² = 23.700.000 [#]
neg. m 2.620.000 = − 2.620.000
21,080,000 [#]

Live Load moment. $\frac{1}{8}$ · 4450 · 148² = + 12.200.000
33.280,000

Live Load max neg. 12.250.000

Depth of truss say 14.0 stress = 33.280.000 ÷ 14.0 = 2380,000 [#]
section required = 2380.000 ÷ 17000 = 140.0 □"
20% 28
168,0 □" @ 3.4 = 570 [#]

570 [#]
570 [#]
400
1540
38% 585
2125 [#] per lin ft. 2125 [#] × 2 = 4250 [#]
say 4000 [#]

Dead load metal truss 4000
3645
shoes say 255
7900 [#] per ft. × 147 = 116000

3100
3645
255
7000 × 24.67 = 17300
133.300 [#] or 593 tons

2 @ 593 tons = 1186 tons
1 @ 860
2046 tons call this 2050 tons

max load on pier.

147.
24.67
171.67

DL. 1,775,000 × $\frac{171.67}{147.0}$ = 2,070,000 [#]
LL. 880.000 · $\frac{171.67}{147}$ = 1,030.000
3,100,000 [#]

8635
4450
13085 × $\frac{171.67^2}{2 \cdot 147}$ = 1,310.000
4,410,000 [#] on pier.

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CALCULATIONS FOR

Preliminary Design of Shirahige-Bashi for Tokyo Prefecture

Max load on abutment.

negative reaction. 24,470,000 ÷ 147 = −167,000# } for one truss.

8635 × 147/2 = 635,000

4450 × 147/2 = 327,000

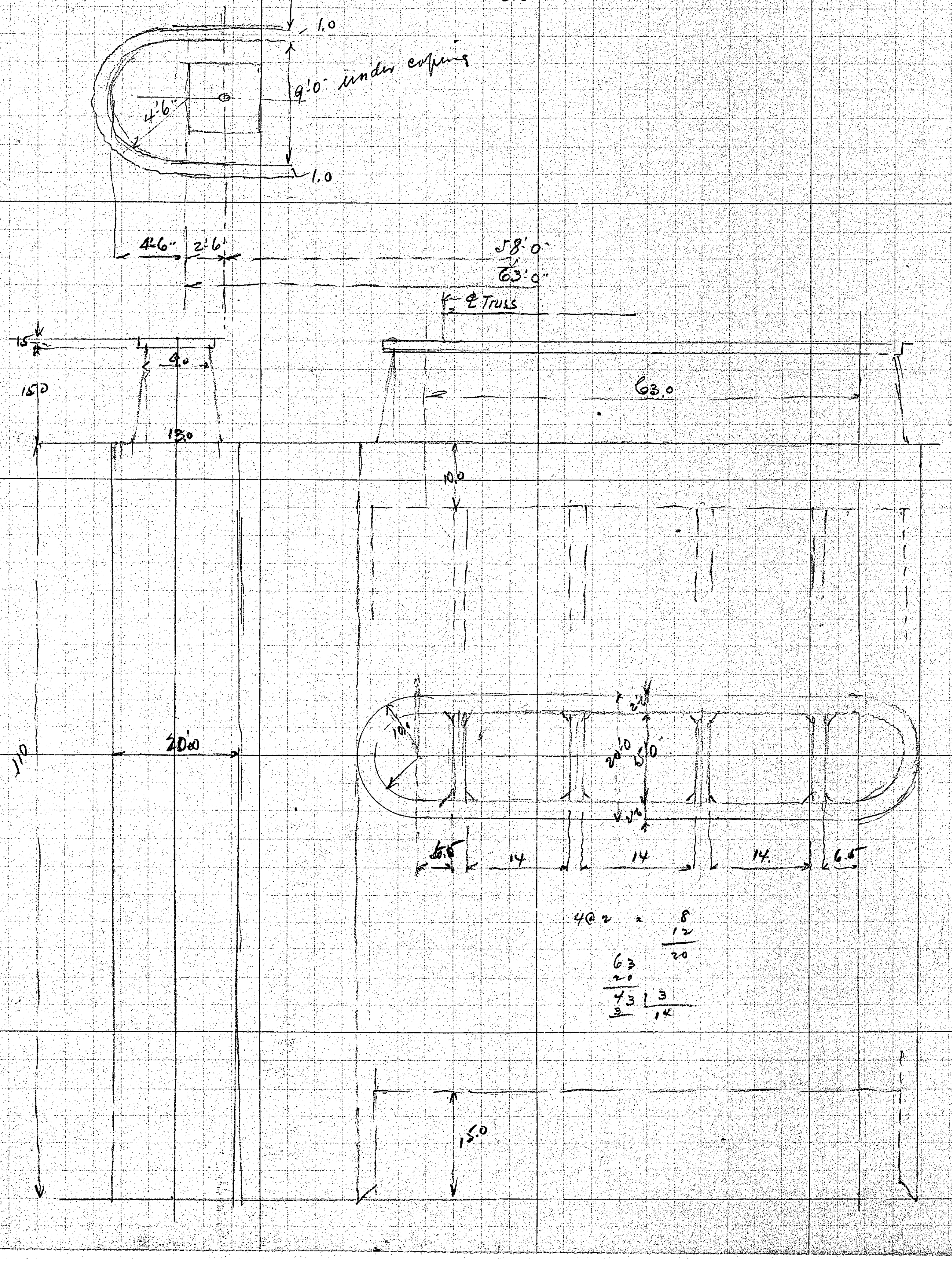
962,000
167,000
795,000# × 2 = 1,590,000

710 ton

Design of pier.

Bearing 4,410,000# on pier 2,205,000# on bearing.

2,205,000 / 600 = 3680□" 60" · 61.5" = 3690□"



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CALCULATIONS FOR

Preliminary Design of Shirahige-Bashi - for Tokyo Prefecture

Concrete in pier.

shaft and copings. coping - 11.0ϕ - 95
63 . 11 = 693.0
788 × 1.5 = 1180.

shaft top. 9.0ϕ = 63
63 . 9 = 567.
630 - 630
13.ϕ 132
13 × 63 818
950 - 950
1580 ÷ 2 = 790 × 15.0 = 11860
1180
13040 ÷ 216 = 60.4 立方

Top and bottom of well.

20.0ϕ 314
63 . 20 = 1260
1574 × 25 = 39400 cubic ft 182.2 立方

Intermediate well section -
20.0 - 314
15.ϕ - 176
138
63 . 5 = 315
Partition filler. 16 × 2 × 4 = 128
8
589 × 85 = 50000 - 232. 立方

110
25
85

1:2:4 Concrete 1180
11860
39400
50000
92440 ÷ 216 = 428 立方

wt = 428 × 32400 = 13,850,000 #

Inside filling.
30 15.14 = 630 × 85 = 53500 cubic ft 248 立方
wt = 264 × 21600 = 5,700,000 #

Inside filling
15ϕ 177
15 × 13 = 195
372 × 85 = 31,600 cubic ft 146 立方
wt = 146 × 30600 = 4,480,000 #

Total vol and weight

	vol	wt
coping + shaft	60.4	1960000
top + bottom of well	182.2	5890.000
Intermediate well	232.0	7500.000
Inside filling. sand	264	5700.000
" " concrete	146	4480.000
		25530,000
Superimposed load		4410.000
		29,940,000

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CALCULATIONS FOR

Preliminary Design of Shirahige-Bashi for Tokyo Prefecture

bottom area 20.5' 330
20.5 × 63 1290
1620 □' 29.940.000 ÷ 1620 = 18500 #/□' 8.25 tons/□'

Circumferencial friction.
circumference 20'φ 63
2 @ 63 = 126
189 × 200 = 37800# per lin ft of depth
37800 × 60 = 2,270.000#

29.940.000
2270.000
27.670.000 #

Unit bearing on soil = 27.670.000 / 1620 = 17000 #/□' or 7.61 tons/□'

Excavation. 1620 × say 70 = 113400 cubic ft 525 立坪 per pier in average.

volume of Concrete	428 立坪	@ 140	-	60,000
concrete filling	146 " "	@ 75	-	10950
Sand filling	264 " "	@ 35	-	9250
Reinforcing bars.	140 tons	@ 160	-	22400
Curb shoes.	50 tons	@ 250	-	12500
Excavation	525 立坪	@ 50	-	26200
				141300
	Incidentals	say		8700
				150000 yen

116 tons

3.87 cu

Abutment say 2 @ 75000 = 150.000.00

Substructure piers 2 @ 150.000 = 300.000.00
abutments 2 @ 75000 = 150.000
450.000.00

Estimate of Deck.
Concrete in slab. 54 × 0.5 / 216 = 1.25 立坪
4.K × 4/6 = 1.26
1.150 × 540 = 810 立坪 @ 140 = 11350 9.10

Reinforcing bars.	810 @ 1500 / 2240	=	542 tons.	@ 160	8680
forms.	13 Ken × 540/6		1170 □坪	5.00	5850
pavement on roadway	9 × 90 =	=	810 "	20.00	16200
pavement on sidewalks.	4 × 90 =		360 □坪	5.00	1800
Handrails.			1080 lin ft	@ 10.00	10800
					54680.00
			call this	~~5460~~	5468
					60148
			call this		60.000.00

Summary of cost
Steel 2050 tons @ ~~315~~ 290 = 595,000.00
Deck construction 60.000
Substructure 450.000
1105000.00
5% general expense = 55000
1160.000.00

1290 / 58

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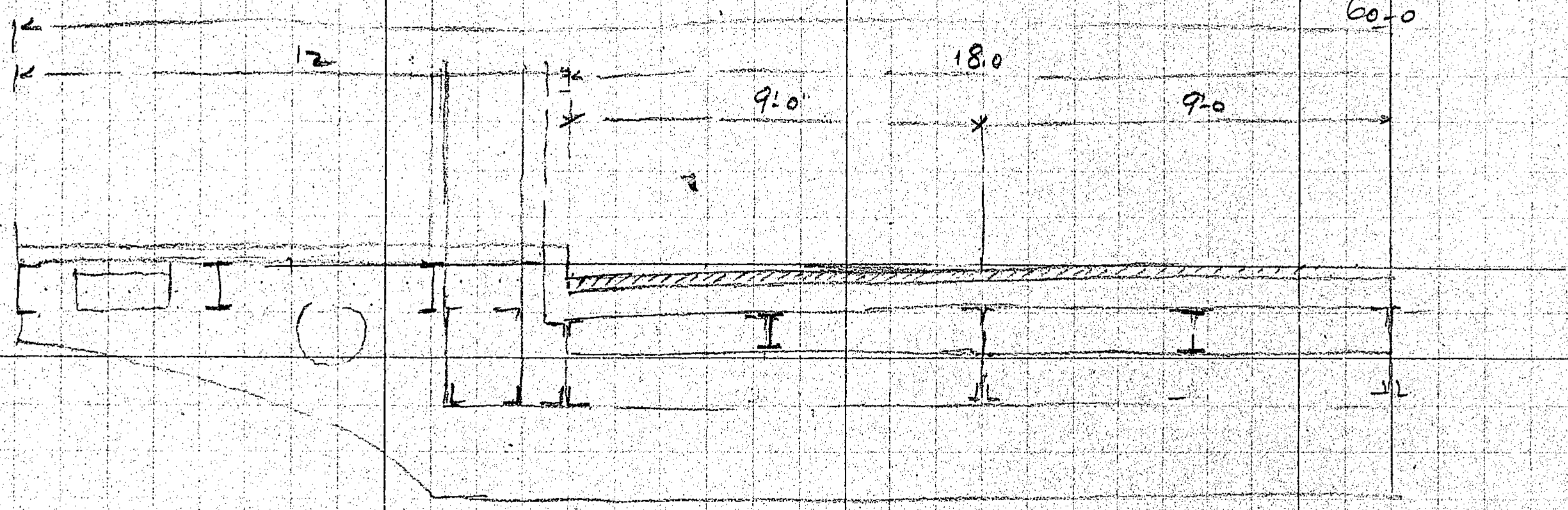
CALCULATIONS FOR

Preliminary Design of Shirahige-Bashi for Tokyo Prefecture

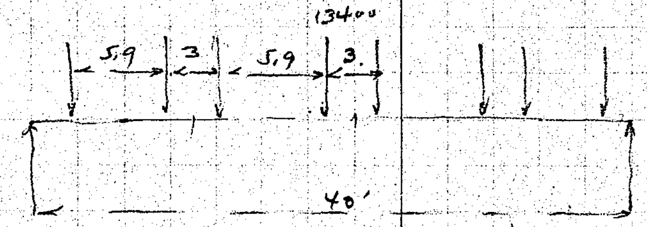


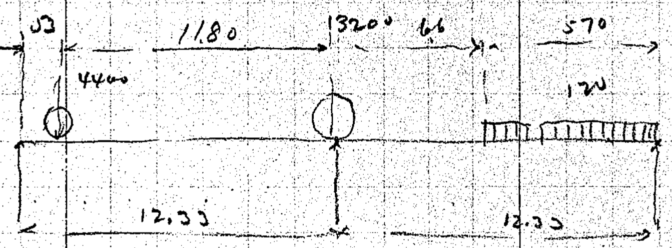
Approximate weight of of stringers and Joists.

	5 @ 77	=	385#
Sidewalk.	2 @ 35	=	70
	2 @ 25	=	50
fascia	2 @ 50	=	100
			605

Joist say 295 × 2/3 = 605, 197, 802# say 800# per lin ft.

Floor beam spacing 12.33





12.33
11.80
.53

12.33
6.6
5.70

13400 × 4 = 53600#

9900 × 1/3 = 3300, 13200

4400 × .53/12.33 = 190, 13200, say 13400 #

120 × 5.70²/(2 × 12.33) = 158 say 160#

5.9, 3, 8.9

moment at center 53600 × 20/8.9 11.1 = 595000 ##

m = 1/8 × 160 × 40² = 32000 ##

16000, 72

627000 ##

Dead Load moment.

Dead Load. 11.5# × 12.33 = 1420

7.3 say 400

1820#

D.L. m = 1/8 × 1820 × 40² = 364000 #

negative moment say 140,000

224,000

LL m 627,000

851,000

Section at center of bridge.

48 × 3/8" = 18.0 1/8 web = 2.25 □"

Effective depth say 3.9 stress = 851.000 ÷ 3.9 = 218.000 #

SR = 218.000 ÷ 17000 = 12.10" req

2 Ls 6 × 6 × 1/2 = 11.50 9.50

11.50

18.00

46.00 @ 3.4 = 140# per ft

Details. 25 40

180# per lin ft

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

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CALCULATIONS FOR

Preliminary Design of Shirahige-Bashi for Tokyo Prefecture.

weight of one floor beam 180 × 40 = 7200
150 × 20 = 3000
10200 ÷ 12.33 = 825# per lin ft.
stringers 800
1625# per lin ft.

1625 × 540 / 2240 = 390 tons

Lower Lateral Bracing - 200# per lin ft.
Upper lateral Bracing - 1730 × 2/3 = 490# per lin ft.

Approximate weight of tied arch.
Dead Load - Flooring & Pavement 115 × 36 = 4150
Sidewalk slabs. 72 × 24 = 1720
curb. 200
Handrails. 2 @ 75 150
water main and other misc. 500
6720#

metal in floor. 1625
Lower laterals 200
upper laterals 490
2315 2315
Trusses assumed 3000
12035

Live Load 36 × 120 = 4320
24 × 100 = 2400
6720 6720
18755# per lin ft.

For one truss 9380# per lin ft.

Approximate Wt = 4800 × 9380/12000 = 3750# per lin ft.
2315
shoes. 200
6265# per lin ft.

Approximate wt of one tied arch complete
6265 × 197.3/2240 = 553 tons

Side Span. 14000 × 9380/12000 = 3120#
Floor 1625
Lower laterals - 200
shoes say 200
5145 × 147/2240 = 338 tons.

2 @ 338 = 676 tons
1 @ 553
1229 call this 1230 tons.

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CALCULATIONS FOR

Preliminary Design of Shirahige-Bashi for Tokyo Prefecture.

Substructure -
approximate vol of concrete tangent length
shaft and coping. 11.0° = 95
45.1 11 = 495
590 × 1.5 = 885.

shaft. 9.0° - 63
45×9 = 405
468

13° - 132
13.45 585
717
1185 ÷ 2 = 592 × 15 = 8870
9755 ÷ 216 = 45.2 立ヅボ

Top and bottom of well.
20.0 314
20×45 900
1214 × 25 = 30400 cubic ft 140.5 立ヅボ

Intermediate well section.
20° - 314
15'° - 176
138
45×5 = 225
wall. 15×2×3 = 90
filler. 6
459 × 85 = 39000 180.0 立ヅボ

Inside filling.
Sand 26×15×85 = 33100 1530 立ヅボ

Inside filling concrete 146.0 立ヅボ

Total vol of concrete coping + shaft 45.2
well filling 140.5
well 180.0
365.7

Excavation - 20.5° = 330
20.5×45 = 920
1250 × 70/216 = 405 立ヅボ

Estimate.

Concrete	365.7	@ 140.00	=	51100
Concrete filling	146	@ 75.00	=	10950
Sand filling	153	@ 35.00	=	5350
Reinforcing bars	100 tons	@ 160.00	=	16000
Curb shoes	40 ton	@ 250.00	=	10000
excavation	405 立ヅボ	@ 50	=	20250
				113650

Incidentals say 6350
120000

2 @ 120.000 = 240.000
abutment say. 75000 × $\frac{10}{13}$ = 58,000 116.000

2 @ 58000 = 116.000 356,000.00

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CALCULATIONS FOR

Preliminary Estimate of Shirahige-Bashi - for Tokyo Prefecture -

Estimate of Deck

Concrete in slabs. $\frac{36 \times 0.5}{216} = .0835$

sidewalks. .026

	0.110 × 540 =	60 cu yd	@ 140	= 8400
Reinforcing bars. 60 @ 150 lb / 2240 =		40 tons.	@ 160	= 6400
forms. $10 \times \frac{540}{6}$ =		900 sq yd	@ 5.00	= 4500
pavement on Roadway	6 × 90 =	540 " "	@ 20.00	= 10800
" " sidewalk	4 × 90 =	360 " "	@ 5.00	= 1800
Handrails.		1080 lin ft	@ 10.00	10800
				42700.00
				4270
				46970

47000 yen

Summary of Cost

steel	1230 tons.	290 =	357,000.00
Deck -			47,000.00
substructure.			356,000.00
			760,000
General Expense 5%.			38,000
			798,000.00

call this 800000.00

$\frac{800,000}{1,160,000} = .69\%$

$\frac{1160,000}{} \cdot \frac{10}{13} =$

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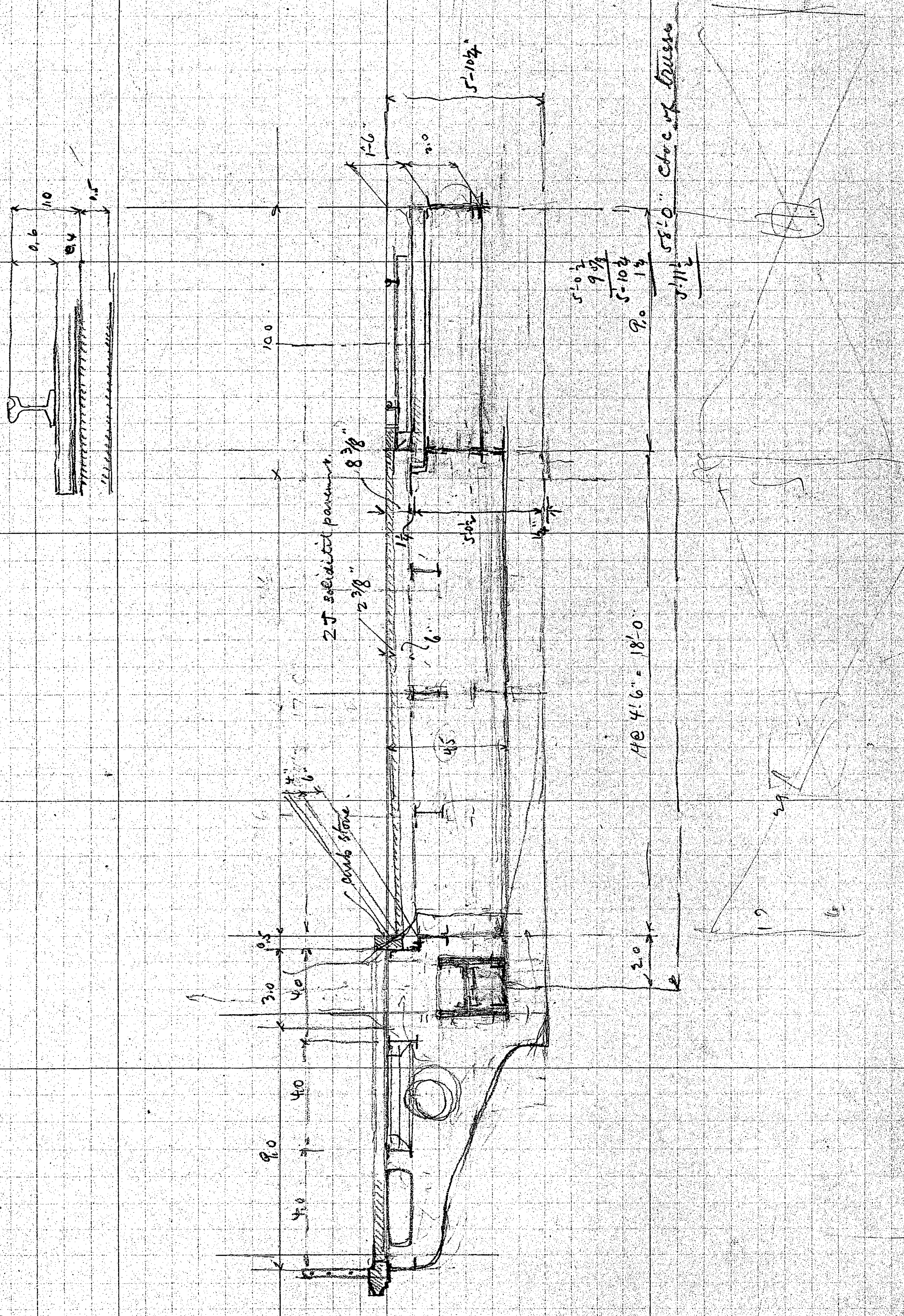
CALCULATIONS FOR

Revised Preliminary Design of Shirahige-Bashi for Tokyo Prefecture



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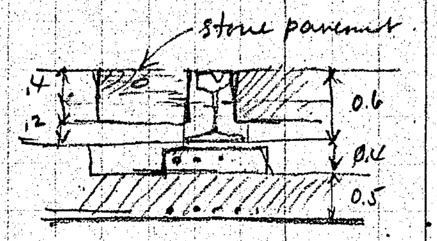
CALCULATIONS FOR

Revised Preliminary Design of Shirahige-Bashi for Tokyo Prefecture.

Slab under tracks.



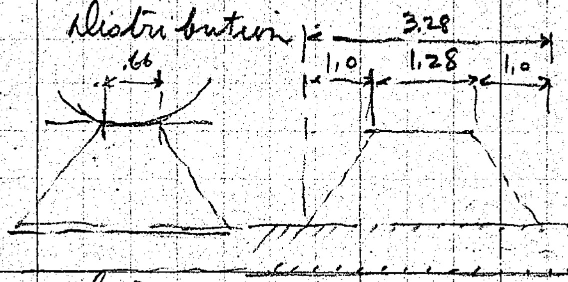
.4 stone pavement .4 × 160 = 64.#
.6 lean concrete 90
.5 Reinforced concrete slab. 75
229# call this 230# per ft.

span length for stringer = 14'-6"
dividing in 3 panels. 4'-10" each for floor slab.

DL. moment = 1/10 × 230 × 4.83² = 536'#
DL. shear = 230 × 4.83/2 = 556#

Motor truck loading rear wheel 9900
Impact 30% 2970
12870#

Distribution



transverse distribution = 3.28 = b
Longitudinal distribution = 2.66' = a

2/3 (l + a) + b = 2/3 (4.83 + 2.66) + 3.28 = 8.28

4.83
2.66
7.49

Longitudinal Transverse

distribution assumed 5.0'
12870 ÷ 5.0 = 2580# per ft.

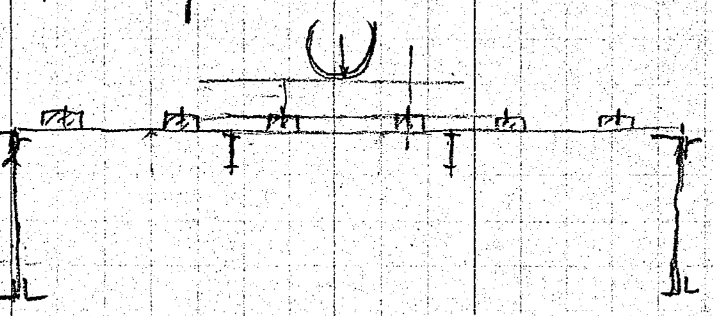
moment = 2580/2 × 4.83/2 = 3110 × 0.8 = 2480'#
Dead load moment 536
3016

Effective depth = √(3016/102) = 5.5" make depth 6.5"

30 ton Electric Ry car wheel load 8400#
Impact 30% 2520
10920#

Spacing of tie assumed 2.42'

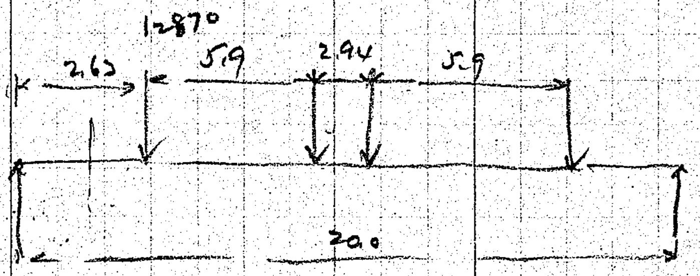
2 @ 10920 = 21840#
21840 ÷ 7 = 3120# per lin ft.



Depth of slab same as for motor truck loading.

40,000

stringer span length 20.0
DL. = 230 × 4.83 = 1110# moment = 1/8 × 1110 × 20² = 55,500
motor truck loading. 12870



2 × 12870 × 8.53 = 219,500
less 12870 × 5.9 = 76,000
143,500
DL. 55,500
199,000

1.47
5.90
7.37
2.63

5.9
2.63
8.53

Sm = 199,000 × 12 / 17000 = 140.5

use 20" × 7½ - 88.96#

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CALCULATIONS FOR

Revised Preliminary design of Shirahige-Bashi for Tokyo Prefecture.

span assumed - 18.0 DL. m = ⅛ · 1110 · 18² = 45,000 1#

motor truck loading.
2 × 12870 × 7.53 = 194,000
12870 × 5.9 = 76,000
LL 118,000
DL 45,000
163,000

$Sm = \frac{163000 \times 12}{17000} = 115.0$

20" @ 65.4# I. Sm = 116.9

Longitudinal stringer span length 14'-6"
load on stringer = 111.0
stringer say 9.0
1200# per lin ft.

End reaction = 1200 · 8 = 9600#

From highway side.
Dead load. 2⅜" pavement = 30#
slab 75
105# per sq ft.

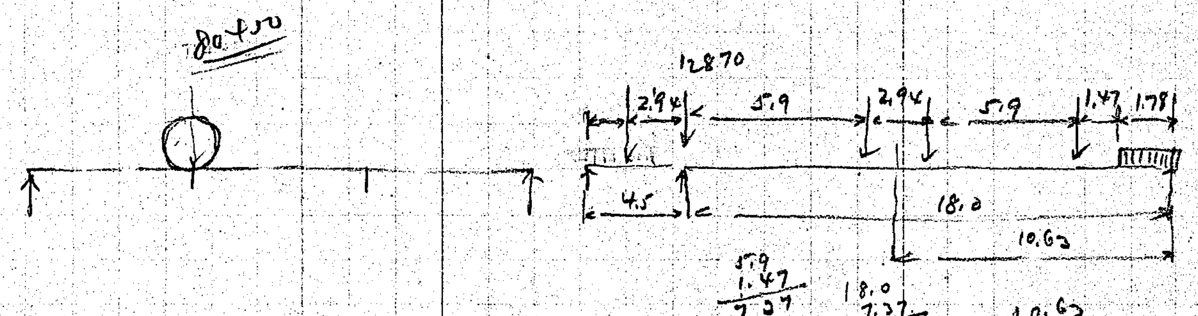
8⅜ 1.0 6½

105 × 4.5/2 = 236# per lin ft.
extra concrete 62
298 say 300# per lin ft.
Floor beam say 120
420# per lin ft.

18½
10

Moment = ⅛ · 420 · 14.5² = 11,050
conc. 9600 · 4.83 = 46400
57450 1#

Live load.



12870 · 4 · 10.63/18.0 = 30400#
12870 · 1.56/4.50 = 4450
34850

5.9 + 1.47 = 7.37
18.0 − 7.37 = 10.63
10.63 − 8.85 = 1.78
4.5 − 2.94 = 1.56

moment = 34850 · 4.83 = 168,000 1#
Dead load = 57,450
225450 1#

20
2.64
14
24

web assumed 30 × ⅜ = 11.25 □" ⅛ web = 11.25/8 = 1.41
flange stress = 225450 ÷ 23.5 = 96,000 #
sq. : 96000 ÷ 17000 = 5.65
1.41
4.24

2 L 5 × 3½ × ⅜ = 6.10 − 5.35 □" net
6.10
14.25
approximate weight. 23.45

23.45 @ 3.4 = 79.60
126.00
95560 per ft. · 14.5 = 7450# per girder.

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CALCULATIONS FOR

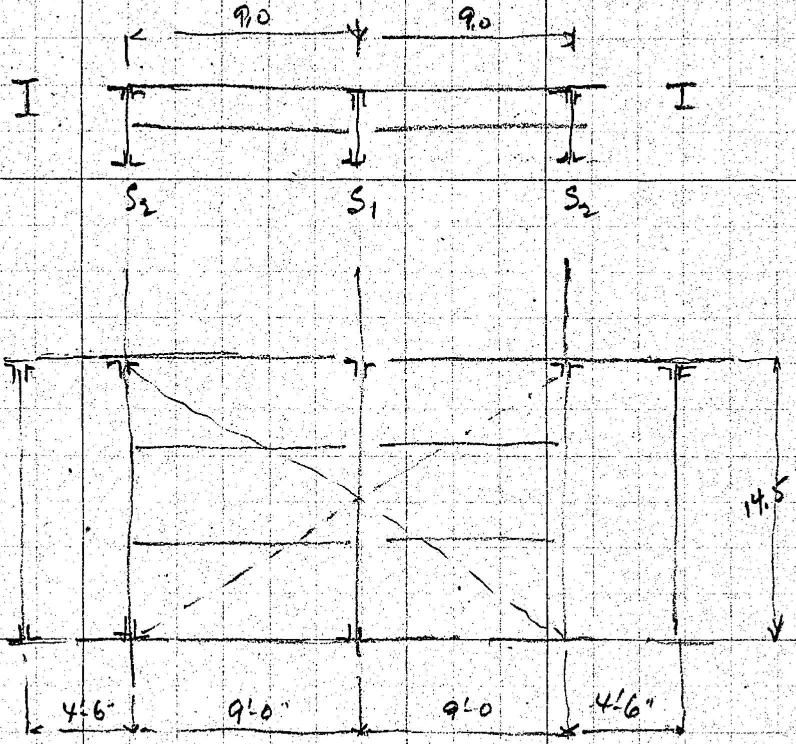
Revised Estimate of Shirahige-Bashi-for Tokyo Prefecture-

approximate weight.
Joist 2-70# = 140# × 18 = 2520#
Stringer 2@ 1450 2900
5420 ÷ 14.5 = 374 per lin ft.

Bay 3 girder system.
Transverse Joist - span length 9.0
load - 230 × 4.83 = 111.0
stringer say 40
1150# m = 1/8 × 1150 × 9² = 11,680
motor truck loading - 12870 × 3 = 38,600
50280

Sm = 50280 × 12 / 17000 = 35.5

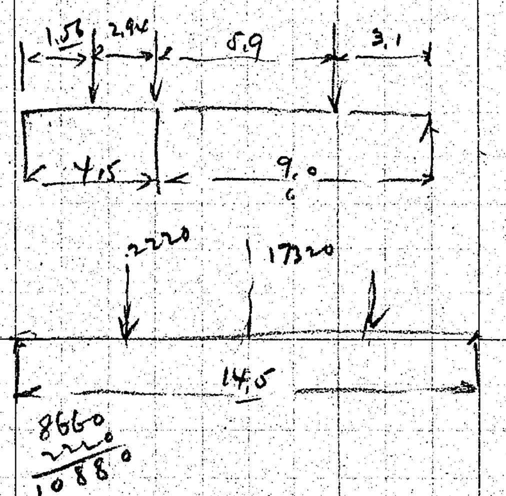
Use 12 × 31.99 I Sm = 36.6.



Stringer S2.
Dead load 230 × 4.83 = 1110
transverse Joist say 35
1145# per lin ft.
Conc = 1145 × 4.5 = 5150
From highway 420# including wt of girder

DL moment 1/8 × 420 × 14.5² = 11,050
Conc 5150 × 4.83 = 24900
35,950 (17)

Live load



12870 × 3.1/9.0 = 4440# ÷ 2 = 2220 at.

12870 × 1.56/4.5 = 4450
12870
17320 } at center.
8660

moment at center = 10880 × 7.25 = 78,800
2220 × 4.83 = 10700
68,100
DL m 35,950
104,050

8660
2220
10880

24 × 3/8 web = 9.00 1/8 web = 1.12
Effective depth - say 2.04
.16
1.88

flange stress = 104,050 ÷ 1.88 = 55,600 SR = 55600 ÷ 17000 = 3.28
use 2Ls (5) × 3½ × 3/8 = ~~6.10~~ 4.96 - 5.35 ~~4.96~~
3½ ~~6.10~~ 4.96 1.9
9.00
~~21.20~~

19.0
21.20 @ 3.4 = ~~72~~ 65
details 22% 15
80 × 14.5 = ~~1270~~ 1160#

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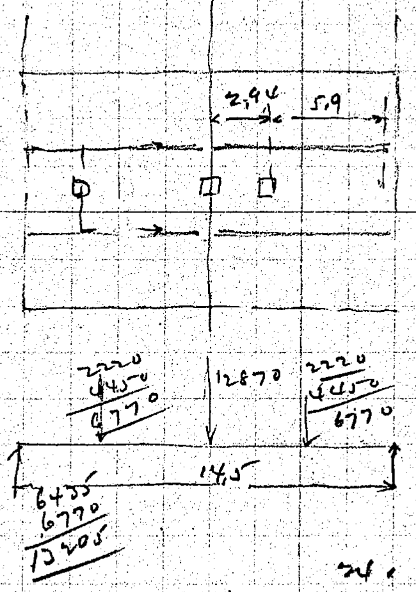
CALCULATIONS FOR

Revised Estimate of Shirahige bashi for Tokyo Prefecture

Stringer S_2 at ℄ Bridge -

Dead Load Concentration $24515 \cdot 4.83 = 59800$
DL $\frac{1}{8} \cdot 100 \cdot 14.5^2 = 2630$
62430 ‴

moment due to motor truck



$12870 \cdot \frac{3.1}{9.0} = 4440$

$12870 \cdot \frac{6.22}{9.0} = 8900$

moment $= 13205 \cdot 7.25 = 95700$
$6770 \cdot 4.83 = -32700$
$+63000$
DL moment 62430
125430

$24 \cdot 3/8 = 9.0$ $\frac{1}{8}$ web $= 1.12$

flange stress $= 125430 \div 1.88 = 66700 \div 17000 = 3.92$
1.12
2.80

use $2L\,3\frac{1}{2} \cdot 3\frac{1}{2} \cdot 3/8 = 4.96 - 4.21$
$.66$
3.55

wt of entire stringer 1160 ‴

weight of stringers and Joists:

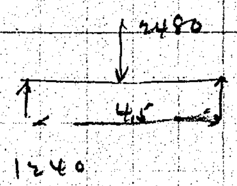
$2C35 \cdot 18 = 1260$
Floor beams $3@1160 = 3480$
$4740 \div 14.5 = 326$ ‴ per lin ft.

Highway flooring

2" solidite $= 30$ ‴
6" Concrete Slab $= 75$
105 ‴ per sq ft.

span length 4.5 DL m $= \frac{1}{10} \cdot 105 \cdot 4.5^2 = 213$ ‴

Live Load - motor truck loading - 12870 ‴



Distribution of load -

Longitudinal distribution $k = \frac{4}{1.06}$

Transverse distribution $= 1.28 + .4 = 1.68 = k$

$2/3\,(4.5 + 1.68) + 1.06 = 5.18$

$12870 \div 5.18 = 2480$

moment $= 1240 \cdot 2.25 = 2790$ ‴ $\cdot 0.8 = 2230$
DL m $= 213$
2443

$d = \sqrt{\frac{2443}{102}} = 4.9$ use 6" slab

Reinforcing bars $= \frac{2443 \cdot 12}{\frac{7}{8} \cdot 5 \cdot 17000} = .394$ use ½ bars 6" centers -

use - extra reinforcement for bond.

増田淳氏関係資料（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

Revised Estimate of Shirahige-Bashi for Tokyo prefecture.

Highway stringer.

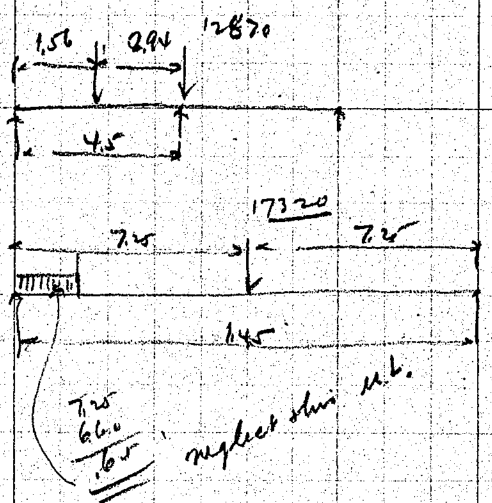
Dead load

slab + pavement $105 \times 4.5 = 473^{\#}$
stringer say $45^{\#}$
$518^{\#}$

$m = \frac{1}{8} \cdot 518 \times 14.5^2 = 13,600^{'\#}$

Live Load

$12870 \cdot \frac{1.56}{4.5} = 4450$
12870
17320



$m = \frac{17320}{2} \cdot 7.25 = 62,800$
13600
$76,400$

$Sm = \frac{76400 \cdot 12}{17000} = 54.0$

15" I 42.9# —

Sidewalk slab.

pavement + slab. $4\frac{3}{4}" = 60^{\#}$
live load 100
$160^{\#}$ per sq ft.

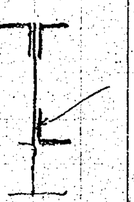
$m = \frac{1}{10} \cdot 160 \cdot 4^2 = 256^{'\#}$

stringer $160 \cdot 4 = 640^{\#}$
$\frac{1}{2}$ water main &c. say 250
so $890^{\#}$ $m = \frac{1}{8} \cdot 890 \cdot 14.5^2 = 23,400^{'\#}$

$Sm = \frac{23400 \cdot 12}{17000} = 16.5$

Use $12" \cdot 5" @ 31.99^{\#}$ $Sm = 36.6$

Fascia stringer.



$18 \times 5/16 = 5.625$
$3 L 3\frac{1}{2} \cdot 3\frac{1}{2} \cdot 3/8 = 7.441$
$13.065 \times 3.4 = 44.4$
$\frac{44.8}{7.4}$ Details say 16% $7.$
$51.5^{\#}$ per lin ft.

Stringer at curb

$12 \cdot 5/16$ 12.75
$2 L^s\ 3\frac{1}{2} \cdot 3\frac{1}{2} \cdot 3/8\ @ 8.5 \cdot 17.0$
29.75
Details say 5.25
$35.0^{\#}$ per lin ft.

For sidewalk

$2 @ 35 = 70$
fascia stringer 51.5
curb stringer 35.0
$156.5^{\#}$
misc bracing &c 33.5
$190.0^{\#}$ per lin. ft. $2 \cdot 190 = 380^{\#}$ per lin ft.

増田淳氏関係資料
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CALCULATIONS FOR

Revised Estimate of Shirahige-Bashi for Tokyo Prefecture.

Highway stringers

8 @ 45# = 360# per lin ft.

Stringers under tracks. 326

Sidewalk stringers &c. 380

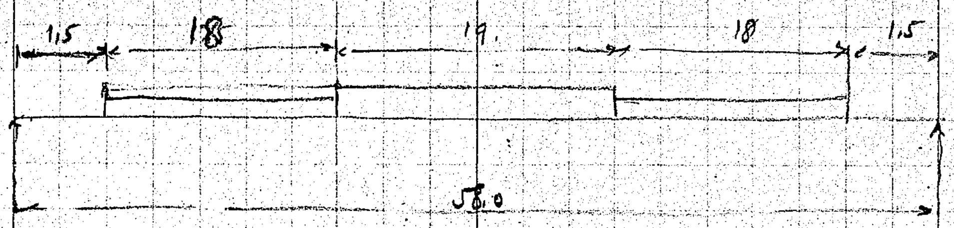
1066# per lin ft.

Design of floor beam.

Dead Load.

Uniform load under tracks.
Floor and paving 230
Stringers &c. say 20
250

Uniform load for highway
Floor + pavement 105
Stringers &c. say 20
125#



Dead load of floor beam assumed 400# per ft

19
36
55

Car tracks $\cancel{280}$ 51 × 19 = 4750 × 14.5 = 69,000#

Highway 115 × 18 = 2070 × 14.5 = 30,000#

R = 34500 - 3450
30000 3260
64500

moment at center = 64500 × 29 = 1,870,000

less m 34500 × 4.75 = 164,000

30000 × 18.50 = 555,000

719,000 — 719,000

1,151,000#

9.5
4.75

9.5
9
18.5

DL floor beam $\frac{1}{8}$ 400 × 58² = 168,000

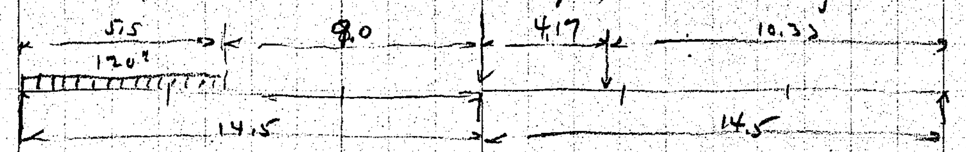
1319,000

DL negative moment say - 64000

1255,000#

cantilever moment.
80# per sq ft
80 × 14.5 = 1160#
11,600 × $\frac{10.5^2}{2}$ = 64,000

Live load. Electric Ry car loading.



axle. 16800
Impact 30%. 5040
21840 ÷ 2 = 10920# per wheel.

14.5
4.17
10.33

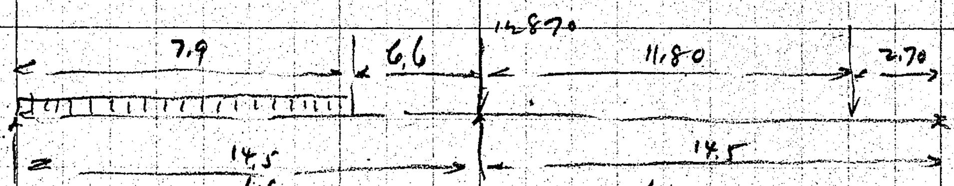
10920 × $\frac{10.33}{14.5}$ = 7800

18720# per rail

Uniform load. 120 × 5.5 = 660#

660 × $\frac{2.75}{14.5}$ = 125# per lin ft.

motor truck loading. with impact 12870#



12870 × $\frac{2.70}{14.5}$ = 2400
12870
15270

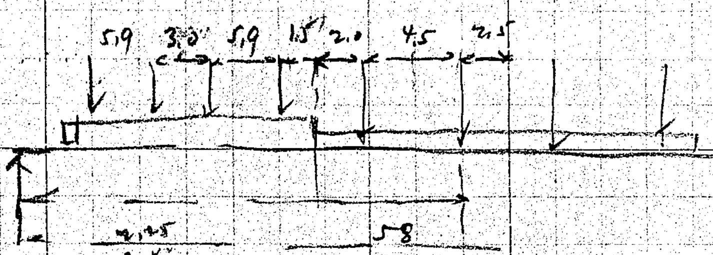
14.5
6.6
7.9

14.5
11.80
2.70

Unif load. 120 × 7.9 = 950

950 × $\frac{3.95}{14.5}$ = 258# per lin ft.

LL moment.



Conc. 2 @ 18720 = 37440
4 @ 15270 = 30540
67980

moment. = 67980 × 26.5 = 1,800,000

18720 × 4.5 = 84,200

30540 × 15.4 = 470,000
554200

554200
1,245,800#

2.25
2.0
4

29
2.5
26.5

5.9
2
8.9
6.5
15.4

増田淳氏関係資料
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CALCULATIONS FOR

Revised Estimate of Shirahige-Bashi for Tokyo Prefecture.

LL m. due to uniform loading.
at front of car 125 × 18 = 2250
at " " motor truck 258 × 18 = 4650
R = 1125
4650
5775 #

moment at center 5775 × 29 = 167500
1125 × 9 = 10100
4650 × 18 = 83600
93700 — 93700
53800
Due to motor trucks 1245800
1299600
Dead Load moment 1255000
2554600 #

web assumed 60 · 7/16 = 26.2 1/8 web = 3.270"
Effective depth 5.04 = 4.94 flange stress = 2554600 ÷ 4.94 = 517000

section reqd = 517000 ÷ 17000 = 30.40"
3.27
27.13

use 2Ls 6×6×3/4 = 16.88 − 3.00 = 13.88
2 Pls 12½ × 5/8 = 15.62 − 2.50 = 13.12
32.50 27.00" net.

approximate weight of one intermediate floor beam complete. — #

32.55
32.55
26.2
91.30 @ 3.4 = 310"
40% 124
434 × 58 = 25100

Cantilever bracket 2 @ 3000 = 6000
31100
misc details say 1900
33000 #

181
58
3
79

33000 ÷ 80 = 413" per ft of girder
33000 ÷ 14.5 = 2280." per lin ft of span
stringer say 1066
3346" per lin ft of span.

weight of floor system = 3346 × 555 = 186000 ÷ 2240 = 830 tons.

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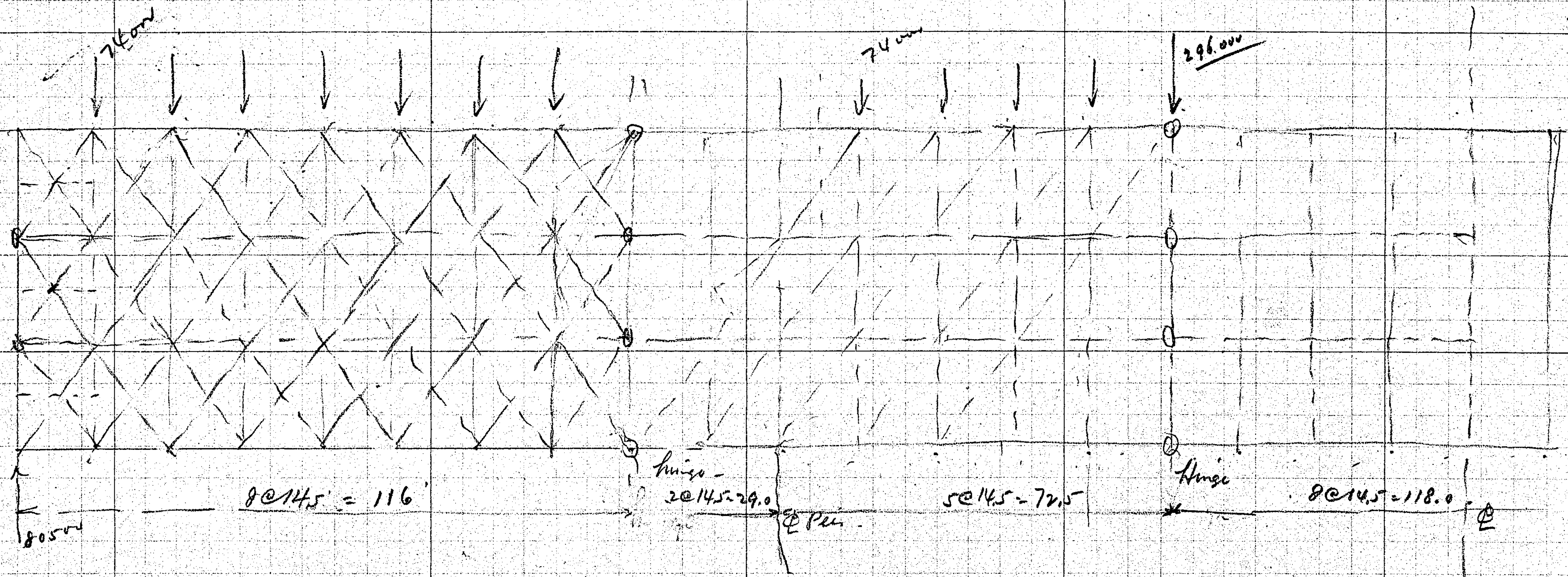
CALCULATIONS FOR

Revised Estimate of Shirahige-Bashi for Tokyo Prefecture.



approximate DL. 19 @ 250 = 4750
18 @ 115 = 2070
Floor system structural Steel. 3345
Bottom lateral. 300
Sidewalk & HR. 1500
water main &c. 250
trusses assumed. 4500

16915 × 14.5 = 245.000

245000 × 0.3 = 73600 call this 74000

R = 74000 × 3.5 = 259000 ÷ 3 86000

$14.5^2 = 210$
$18^2 = 324$
$\sqrt{534} = 23.1$ sec θ = $\frac{23.1}{18}$ = 1.286
17000 × 1.8 = 30600

Stress = 86000 × 1.286 = 111000 ÷ 30600 = 3.620 □″ net.
use 2 Ls 5 × 4 × 3/8 = 6.46 r = 1.59

11.55 × 12 = 139″ l/r = $\frac{139}{1.59}$ = 87.2

chord stress = 16915 × 0.3 = 5080#
m = $\frac{1}{8}$ × 5080 × 116^2 = 8,550,000#
stress in chord = 8,550.000 ÷ 58 = 147,000#
147,000 ÷ 25,200 = 5.82
14000 × 1.8 = 25200

74000 × 4 = 296.000
4 @ 74000 = 296.000
592000 ÷ 3 = 197000
stress = 197000 × 1.286 = 254,000#
SR = 254.000 ÷ 30600 = 8.30 □″ net
2Ls 6.6 × 7/16 = 10.16
1.75
8.41 □″ net

Revised Estimate of Shirahige-Bashi for Tokyo Prefecture

approximate weight of lateral bracing
side span.

2L 5×4×3/8 @ 11.0 × 22 = 485
6 @ 485 = 2910
strut - 2 - 20 × 14.5 = 580
misc connect 8 @ 100 = 800
3 @ 100 = 300
4590

4590 × 8 = 36800 #
2 @ 36800 = 73600

center span say - 45,000
over pier -
2L 6×6×7/16 @ 17.2 × 22 = 755 × 6 = 4530
580
800
300
6210

7 @ 6210 = 43500
misc bracing, say 15000
58500 call them 60,000

78
2
76

2 @ 36800 = 73600
center 45000
on pier - 2 @ 60000 120000
238600 ÷ 555 = 430 # per lin ft.

238.600 ÷ 2240 = 106 tons

Top lateral Bracing.
diagonal say 70,000
Sways 7 @ 12500 = 87500
portals 2 @ 16000 = 32000
189,500 = 84.5 tons.

Top lateral 757 # per lin ft.

12,000
3500
10,000
25,500
89,500
105,000
350 # lin ft.

Design of truss.

approximate Dead Load of floor.
19 × 250 = 4750
18 × 115 = 2070
Side walks & Handrail = 1700
Floor system = 3345
~~Main~~ laterals = 430
water main etc say = 300
truss assumed = 3000
15595 # ÷ 2 = 7800 # per lin ft.

Live Load
54 × 120 = 6500
24 × 100 = 2400
8900 ÷ 2 = 4450 # per lin ft.

Extra motor truck loading - assumed 20,000 # at center.

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CALCULATIONS FOR

Revised Estimate of Shirahige-Bashi for Tokyo Prefecture

Span length 116'-0" — truss height 14.0'

DL. moment = 1/8 × 7800 × 116² = 13,100,000 ÷ 14 = 936,000#

10,000,000

LL. 1/8 × 4400 × 116² = 7,480,000

20,000/2 × 58 = 580,000

8,060,000 ÷ 14 = 576,000

1,512,000#

SR = 1,512,000 ÷ 14000 = 108 □"

108 @ 3.4 = 370#

370#

web + post 500

1240#

37% = 460

1660#

1660 × 2 = 3320# × 116 = 385,000 ÷ 2240 = 172 tons.

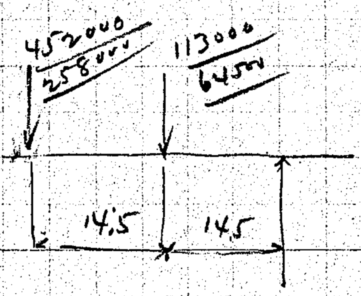
2 spans @ 172 = 344 tons.

Cantilever Tied arch.

Center span 265'-0" Cantilever portion 29'-0" on both sides -

Dead Load - cantilever.

panel concentration. 7800 × 14.5 = 113,000#



113000 × 4 = 452000

live load concentration - 4400 × 14.5 = 64500#

64500 × 4 = 258000

DL. moment 452,000 × 29.0 = 13,100,000

113,000 × 14.5 = 1636,000

14,736,000 ÷ 26 = 566,000#

LL moment. 252000 × 29.0 = 7,300,000

64500 × 14.5 = 935,000

8,235,000 ÷ 26 = 316,000

882,000#T.

882,000 ÷ 14000 = 63 □"

63 @ 3.4 = 214#

214#

web. say 500

928

37% details = 340

1268# × 29.0 = 36700

For 2 trusses 2 @ 36700 = 73400# ÷ 2240 = 33.0 tons

For both arms 2 @ 33 = 66 tons -

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CALCULATIONS FOR

Revised Estimate of Shirahige-Bashi for Tokyo Prefecture.

Center span

D.L. 7800
upper L. $\frac{750}{2}$ 375
8175#

D.L. m = ⅛ × 8175 × 265² = 71,800,000
less Cant. moment = 14,736,000
57,064,000 ÷ 43 = 1,330,000#

L.L. ⅛ × 4400 × 265² = 39,000,000
motor truck 10,000 × 132.5 1,325,000
40,325,000 ÷ 43 = 937,000
2,267,000

section for top chord 2,267,000 ÷ 14,000 = 161.5
161.5
middle chord + webs. here 242.0 1.5
565.0
Details 40% 226.
791 @ 3.4 = 2700#

For both trusses 2 @ 2700 = 5400#
wt of center span 5400 × 265 = 1,430,000# or 640 tons.
110

Summary for trusses

2 - Side spans. 344 tons.
2 - Cant portion. 66 "
Center span 640.
1050

Floor system complete. 830
Lower laterals. 106
top laterals 85
Shoes & castings - say - 35
2106 tons.

244
161
80

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CALCULATIONS FOR

昭和二年九月成

東京府

白鬚橋設計々算書

及材料調書

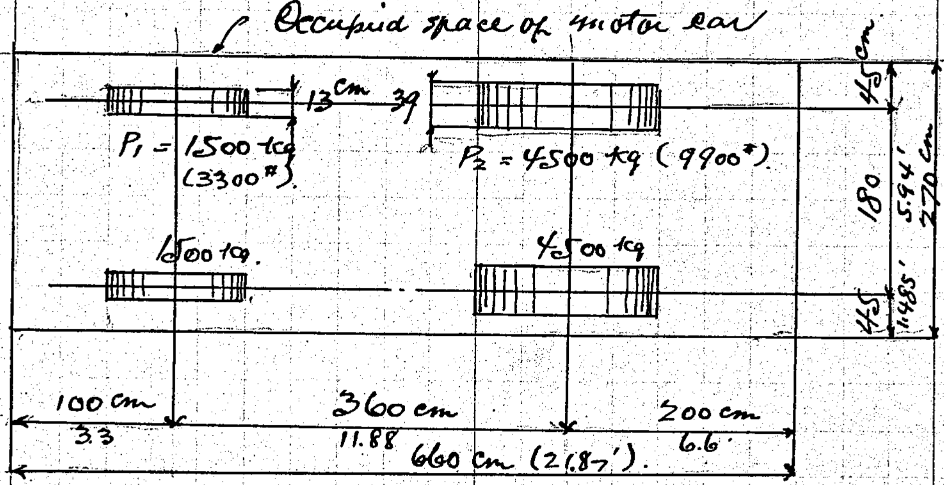
Design of Shirahige-Bashi for Tokyo-Prefecture

Assumed loading.

Uniform live load for carriage way $w = \frac{120000}{170+l} = 600\ kg/m^2$
" " " for sidewalk $w = \frac{100.000}{170+l} = 500$ "

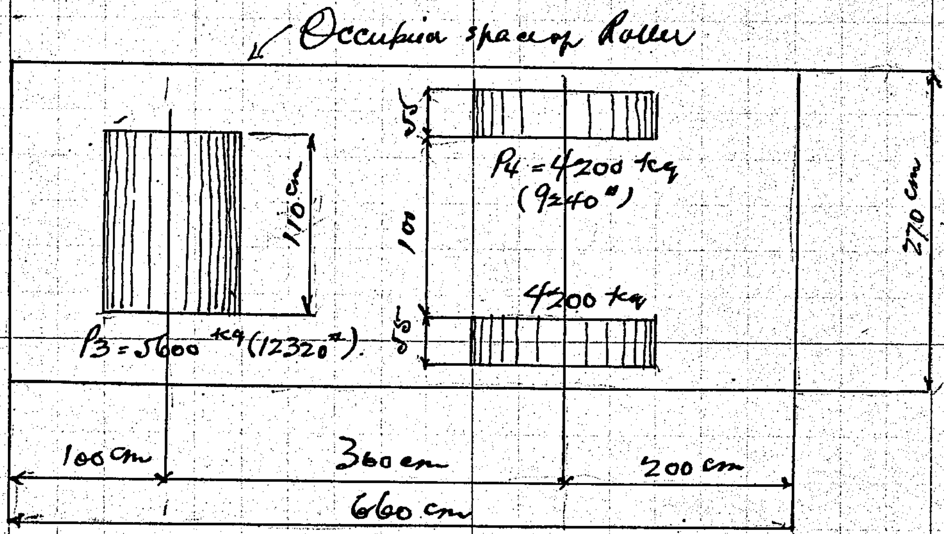
where l = span length in meter.

Motor car loading (12.0 tons).



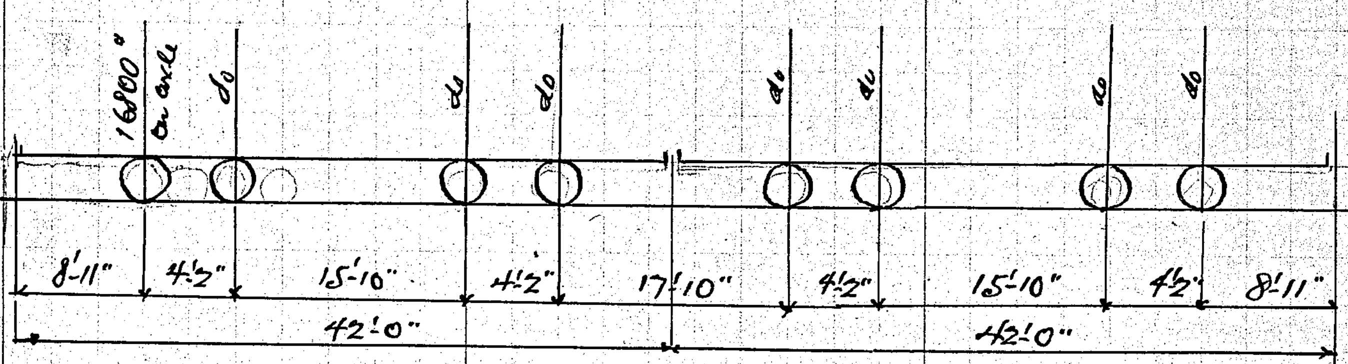
4 rows of motor traffic for roadway with occupied space of 8.91' each. Combined with electric car loading, two rows of motor traffic assumed.
Unoccupied space of motor trucks shall be filled with the specified uniform load.
Only one motor truck for each traffic line on the bridge assumed.

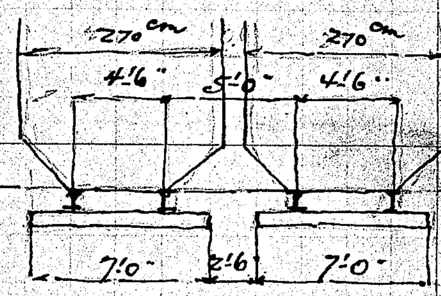
Road Roller (14.0 tons).



Only one road roller on the bridge assumed

Electric Car loading (2-30 tons Bogie Cars) 4'6" gage.





Impact   Impact coef $= \frac{20}{60+l}$ where l = loaded length in meter

The above impact for motor car and electric car loadings max impact 30%.

No impact for uniform live load and road roller concentration.

Assumed Weight of Materials.

Creosoted wooden block pavement	1000 kg/m³	or 62	# cubic ft
mortar	1700 "	106	"
Reinforced Concrete	2400 "	150	"
Plain Concrete	2200 "	140	"

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CALCULATIONS FOR

Design of Shirahige-Bashi for Tokyo Prefecture

Structural steel	7850 kg/m³	490 #	per cubic ft
Granite masonry	2600 "	160	"
Cast iron	7250 "	450	"
wrought iron	7800 "	487	"
earth	1600 "	100	"
Sand (Compact)	1700 "	106	"

Allowable Working strength of Materials.

Structural steel or Reinforcing Bars.

Tension	1.200 kg/cm²	17000 #	per sq inch
Extreme fibre stress (net).	1.200 "	17000	"
Shearing of web (Gross section).	900 "	12800	"

Compression members

$$1500\,(1-0.0055\,l/r) \equiv 1.000\ \text{kg/cm}^2$$

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

Equivalent formula in inch-lbs

$$21300\,(1-0.0055\,l/r) \equiv 14000\ \#/\square''\ \text{gross area.}$$

Compression flange of plate girder

$$1200\,(1-0.012\,\tfrac{l}{b}) \equiv 1.000\ \text{kg/cm}^2$$

where l = unsupported length of flange in cm
b = width of flange in cm

Equivalent formula for inch-lbs unit

$$17000\,(1-0.012\,\tfrac{l}{b}) \equiv 15400\ \#/\square''\ \text{Gross area}$$

Shearing on shop driven rivets (machine driven)	12.000 #/□"
Shearing on field driven rivets and turned bolts	10.000 "
Extreme fibre of pin	24.000 "
Bearing on shop rivets	24.000 "
Bearing on field rivets and turned bolts	20.000 "

Expansion roller 45 d kg/cm where d = diameter of roller in cm
or inch-lbs units 610 d lbs per lin. inch where d = diameter of roller in inches
Bearing on 1:2:4 Concrete 45 kg/cm² or 640 lbs/inch²

l/r for compression member not over 120 for truss members and 140 for wind bracings
l/r for riveted tension members not over 200.

1:2:4 Concrete

	kg/cm²	#/□"
Direct Compression for Col.	35	500
Compressive fibre stress	45	640
Combined stress for column	35	500
Combined stress for arch	45	640
Punching shear	9	128
Shear for plain Concrete	4	58
Bond stress for plain bar	6	85
" " deformed bar	9	128
Shear for Reinf. Concrete with web reinf.	9	128

Considering wind traction and temperature stresses in addition to dead, live and impact stresses, the allowable working stress shall be increased 25% and proportioned the parts of members. With earthquake effect, the working stress shall be increased 80% and proportioned the parts.

Ratio between moduli of elasticity of steel and Concrete = 15
Expansion Coefficient of Concrete 0.000 0011 per 1°C
" " Steel 0.000 0012 "
Accellaration of earthquake 3300 mm/sec²

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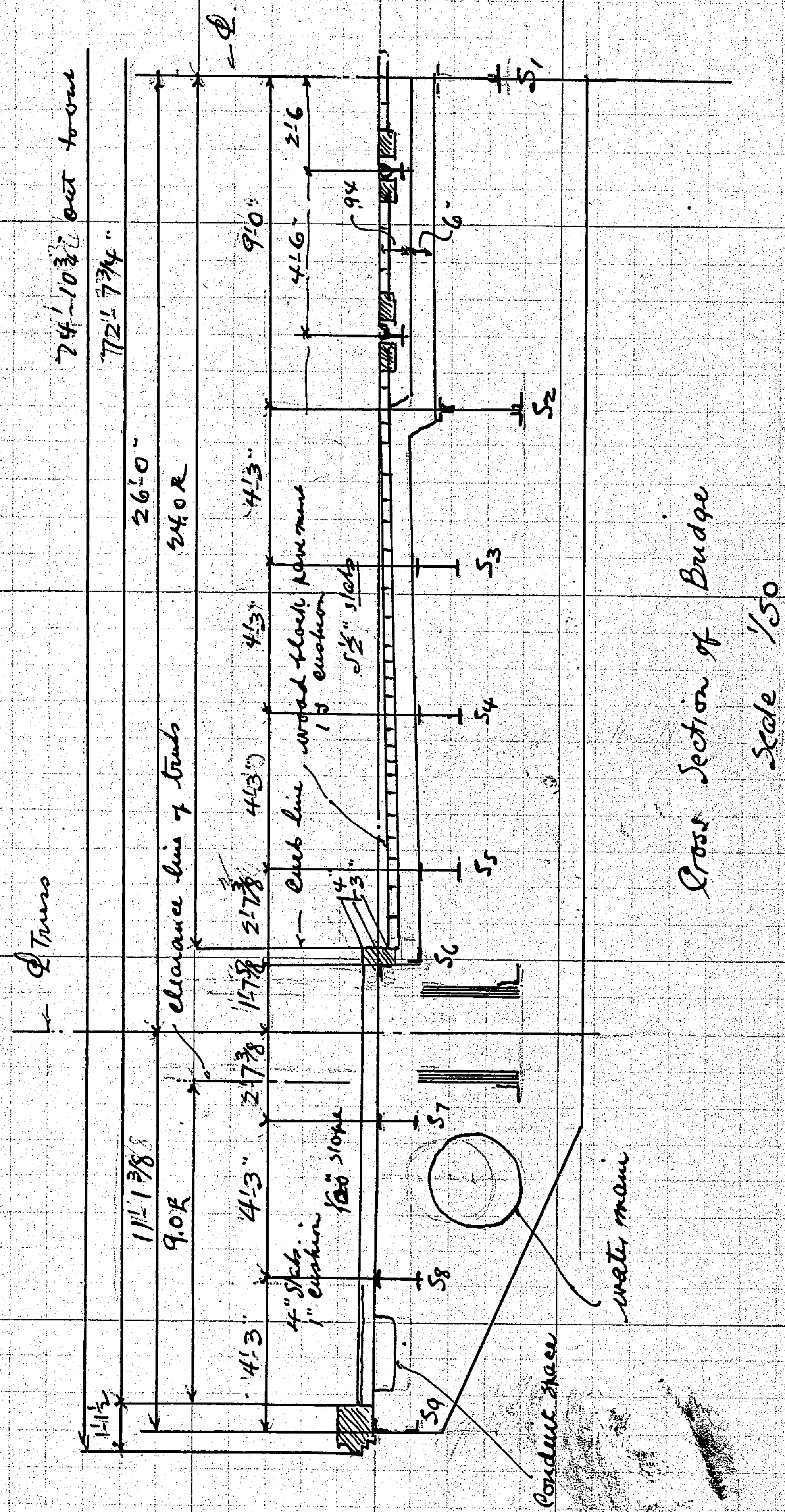
CALCULATIONS FOR

Design of Shirahige-Bashi for Tokyo Prefecture

Assumed cross section of bridge.



Design of Shin-hige-Bashi

Slab under carriage way   span length 4'3"

Dead Load

Wood block pavement   0.25 @ 56#   14.0

1¼" mortar cushion 106#   11.0

Slab assumed 5½" @ 150#   69.0

94.0

For fillet &c call this 95.0# per square ft.

Dead Load moment = $\frac{1}{10} \cdot 95 \cdot 4.25^2$ = 172.$^{1\#}$

Dead Load shear = $\frac{1}{2} \cdot 95 \cdot 4.25$ = 202#

Live Load. motor truck loading   Rear wheel Concentration 9900#.

Distribution of wheel Concentration

Longitudinal distribution   a = 0.66' + 2d' = 1.16' assumed

Transverse distribution   ~~b = t+2d = 1.29 + 0.5 = 1.79' assumed~~

Effective width E = ⅔ (l + b) + a

= ⅔ (4.25 + 1.79) + 1.16 = 5.19'

Load per ft strip = 9900 ÷ 5.19 = say 1900

30% impact = 570

2470$^{1\#}$

moment due to wheel Concentration = $2470 \cdot \frac{1}{4} \cdot 4.25 - 2.470 \cdot \frac{1}{2} \cdot \frac{1.79}{\cancel{2}\,4}$ = 1520$^{1\#}$

For Continuity of slab   1520 · 0.8 = 1216$^{1\#}$ per ft strip.

Max end shear = $2470 \cdot \frac{4.25 - .90}{4.25}$ = 1960#

Summary for moments and shears

	moment	shear
Dead Load	172	202
Live Load	1216	1960
	1388$^{1\#}$	2162#

Effective depth of slab for steel 17000#/□", Concrete 640#/□" with n = 15

$d = \sqrt{\frac{M}{bK}} = \sqrt{\frac{1388}{102}}$ = 3.7"

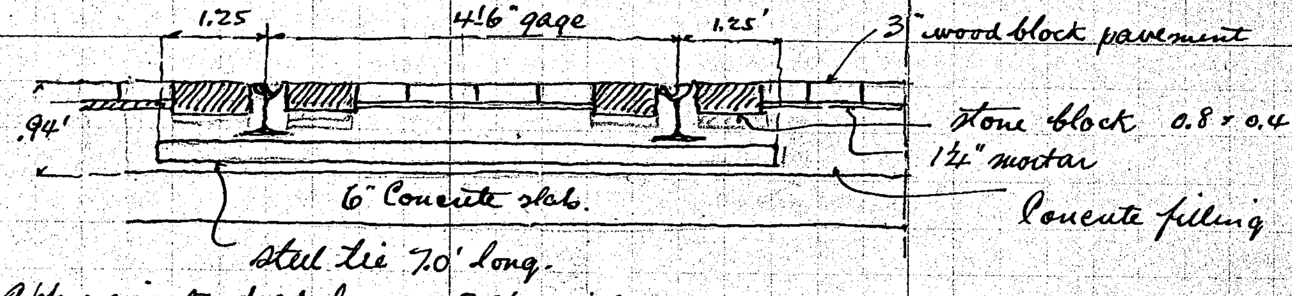
make slab 5½" over all with 1" insulation at bottom of slab.

Required steel area = $\frac{1388 \cdot 12}{7/8 \cdot 4.5 \cdot 17000}$ = 0.25□" per ft   Use ½" bars 4" centers = 0.585□"

Unit shear = $\frac{2162}{7/8 \cdot 4.5 \cdot 12}$ = 46#/□"   Bond Stress = $\frac{2162}{7/8 \cdot 4.5 \cdot 4.71}$ = 11.6#/□" < 128#/□" OK

Slab under electric Railway Tracks.   span length 4'10" in longitudinal direction.

Dead Load



Approximate dead load 3.5' wide.

weight of stone block	2 - 0.8 × 0.4 @ 160#	=	102
Rail with accessories say	110 ÷ 3	=	37
wood-block pavement say	0.25 @ 56# · 1.6'	=	23
1¼" mortar Cushion	11# @ 3.5	=	39
Concrete filling including tie	1.75 ~~cubic~~ sq ft @ 150#	=	262

463# ÷ 3.5 = 133# per sq ft.

Reinforced Concrete slab 6" thick -   75

208# per sq ft.

~~Call this 210# per sq ft.~~

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CALCULATIONS FOR

Design of Shirahige-Bashi

Dead Load Outside of track (tie).
3" wood block pavement = 14.0
1¼" mortar cushion = 11.0
Concrete filling .59 @ 140# = 83.0
108#
Reinforced Concrete slab 6" 75
183# per sq ft.

Average dead weight for 9.0' wide.
Concrete slab over side longitudinal girder assumed inside of 9'0"
for 1.0' wide. 3" pavement 14
1¼" cushion 11.0
.59 @ 150 = 89.0
114.0#

Dead Load over 7' tie 208 × 7.0 = 1456
Outside of tie 183 × 2.0 = 366
Extra weight beyond 9' .63 × .83 @ 150 = 52
1874 ÷ 9.0 = 208# per sq ft.

For Calculation of slab use dead load of 208#/□'.

~~Dead Load moment = 1/10 × 208 × 4.83² = 485'#~~

Dead Load shear = ½ × 208 × 4.83 = 503#

Live Load. Electric Car loading axle load = 16800#

Distribution of Concentration.
Transverse distribution 7.0 + 2 @ .30 = 7.6'
Longitudinal distribution 3.3 + 2 @ .30 = 3.9
area of distribution = 7.6 × 3.9 = 29.64

Load per sq ft = 16800 ÷ 29.64 = 570#
Impact 30%. 170
740# per sq ft.

Live Load moment = 1/10 × 740 × 4.83² = 1730'#
" " shear = ½ × 740 × 4.83 = 1790#

Summary for moment and shear

	moment	shear
Dead Load	485	503
Live Load	1730	1790
	2215	2293

Effective depth reqd = $\sqrt{\frac{2215}{102}}$ = 4.7"
make slab 6" thick with 1" insulation
Steel area = $\frac{2215 \times 12}{7/8 \times 5 \times 17000}$ = 0.360□" per ft
Use ½"⌀ bars 4" spacing 0.585□"

Unit shear = $\frac{2293}{7/8 \times 5" \times 12}$ = 44#/□" Unit Bond Stress = $\frac{2296}{7/8 \times 5 \times 4.71}$ = 111#/□" < 128#/□" OK.

Sidewalk slab span length = 4'3" c to c of stringers.

Dead Load.
1" wearing course mortar asphalt @ 131# = 11#
slab assumed 0.33 @ 150 = 50
61# call this 60#/□'.

Dead Load moment = 1/10 × 60 × 4.25² = 108'#
Dead Load shear = ½ × 60 × 4.25 = 128#

Live Load 100#/□'
Live Load moment = 1/10 × 100 × 4.25² = 180'#
Live Load shear = ½ × 100 × 4.25 = 218#

Design of Shirahige-Bashi.

Summary for moment and shear

	moment	shear
Dead Load	108	128
Live Load	180	218
	288 1#	346#

Effective depth of slab = $\sqrt{\frac{288}{102}}$ = 1.7"
make slab 4" thick with 1" insulation at bottom

Steel area = $\frac{288 \cdot 12}{7/8 \cdot 3" \cdot 17000}$ = 0.058 □" per ft

Use ½" ϕ bars 6" centers = 0.392 □"

Transverse Joist under Electric Ry Track span length 9'-0" spacing 4'-10"

Dead Load

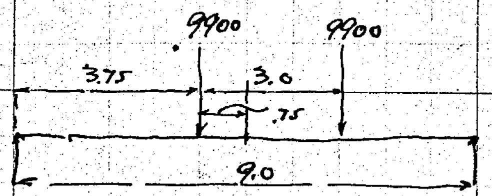
Slab + track &c 208 · 4.83 = 1010
stringer assumed 35
1045# per lin ft of span

Dead Load moment = ⅛ · 1045 · 9.0² = 10600 1#

Dead Load shear = ½ · 1045 · 9.0 = 4700 #

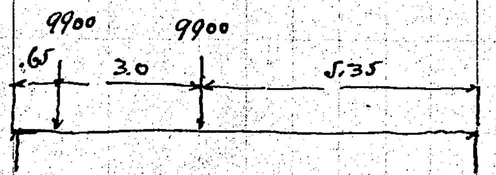
Live Load. motor truck loading Rear wheel Concentration = 9900#

moment



$2 \cdot 9900 \cdot \frac{3.75^2}{9}$ = 30938
30% impact 10210
41148 1#

End shear 9900 · (5.35 + 8.35) ÷ 9.0 = 15070
30% impact = 4530
19600 #



Summary for moments and shears

	moment	shear
Dead Load	10600	4700
Live load	41150	19600
	51750 1#	24300 #

Try. 12" I @ 31.99 Section modulus = 36.69

Unit fibre stress = $\frac{51750 \cdot 12}{36.69}$ = 16950 #/□"

Stringer S_1 under Electric Railway Tracks span length 14'-6" c to c of floor beams spacing 9'-0"

Dead Load

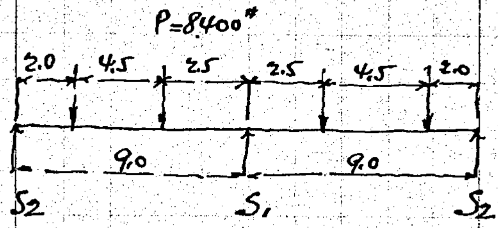
Floor slab. 208 · 9.0 = 1870
Transverse Joists 44
weight of stringer 76
1990# per lin. ft.

Joist 2 - 35# · 9'0" = 630#
630 ÷ 14.5 = 44# per ft.

Dead Load moment = ⅛ · 1990 · 14.5² = 52300 1#

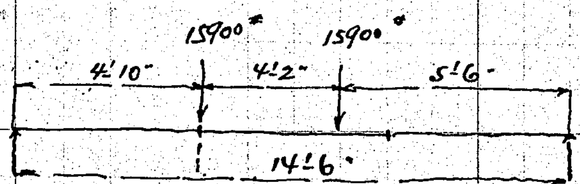
Dead Load shear = ½ · 1990 · 14.5 = 14500 #

Live Load Electric Railway Car loading axle load = 16800#



Reaction on S_1 $2P \cdot \frac{8.5}{9.0}$ = 1.89 · P = 1.89 · 8400 = 15900#

Reaction on S_2 $P \cdot \frac{9.5}{9.0}$ = 1.055 P = 1.055 · 8400 = 8860#



Moment = $15900 \cdot \frac{9.67 + 5.5}{14.5} \cdot 4.83$ = 80500
30% impact 24200
104700 1#

End shear = $15900 \cdot (\frac{10.33}{14.5} + 1)$ = 27200
30% impact 8200
35400 #

Summary for moments and shears

	moment	shear
Dead Load	52300	14500
Live Load	104700	35400
	157000 1#	49900 #

web assumed 24" · 5/16" = 7.50 □"
⅛ web = 0.94 □" 24½" b to b of L's
Effective depth = 22.58"

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Design of Shirahige-Bashi.

flange area required $= \dfrac{157000 \times 12}{27.58 \times 17000} = 4.92 \square''$

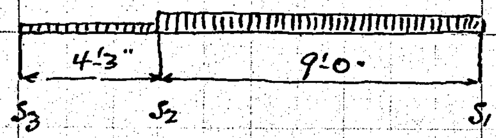
1/8 web $\quad .94$

$3.98 \square''$ net.

2 L^s 4 × 3½ × 3/8 = 5.34 □″ gross or 4.680 □″ net.
2 - ¾″ φ Rivets reduced.

Stringer S_2 at Edge of Electric Railway Tracks. span length = 14′-6″

Dead Load

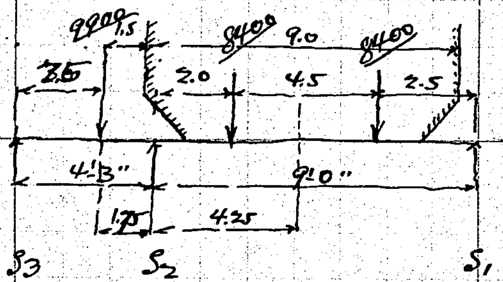


Slab under track	208 × 4.5	=	935
Highway slab	95 × 2.12	=	202
Transverse Joist			22
Assumed weight of stringer			90
			1249 call this 1250#

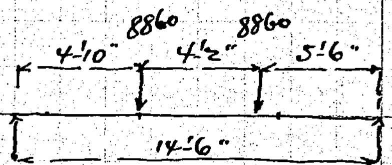
Dead Load moment $= \frac{1}{8} \times 1250 \times 14.5^2 = 32800$ ′#

Dead Load shear $= \frac{1}{2} \times 1250 \times 14.5 = 9050$ #

Live Load



Electric Car Loading.



Reaction on S_2. 8860#

moment $= 8860 \times \dfrac{9.67 + 5.5}{14.5} \times 4.83 = 44800$

30% impact $\quad 13500$

58300 ′#

End shear $= 8860 \times \left(\dfrac{10.33}{14.5} + 1\right) = 15200$

30% impact $= 4600$

19800 #

Motor Car loading rear wheel 9900#

Reaction on S_2 $\quad 9900 \times \dfrac{2.5}{4.25} = 5830$ #

Moment at center of span $= \dfrac{5830}{2} \times 7.25 = 21800$ ′#

30% impact $= 6550$

28350 ′#

Uniform live load at rear of motor truck.

Load $= 0.65 \times 120 \times \dfrac{4.25}{2} = 166$ # Reaction $= 166 \times 0.325 \div 14.5 = 37$ #

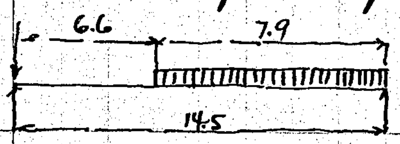
moment $= 3.7 \times 7.25 = 27.0$ ′#

End shear. motor truck loading. rear wheel $\quad 5830$

30% impact $\quad 1750$

7580 #

End shear for uniform load at rear of motor truck



Unif load $= 120 \times \dfrac{4.25}{2} = 255$ # per lin ft.

Reaction $= 255 \times 7.9 \times 3.95 \div 14.5 = 550$ #

Summary for live load moments and shears.

	Moment	Shear
Electric Car Loading	58300	19800
motor truck loading	28350	7580
Uniform load	27	550
	86677 ′#	27930 #

say 86700 ′#

Summary for Dead & Live load moments and shears

	Moment	Shear
Dead Load	32800	9050
Live Load	86700	27930
	119500 ′#	36980 #

web assumed 30½ × 5/16 = 9.52

1/8 web = 1.19 □″ $\quad$ 31″ b to b of L^s

Effective depth 29.08″

flange area $= \dfrac{119500 \times 12}{29.08 \times 17000} = 2.90 \square''$

1.19

$1.71 \square''$ net.

Use 2 L^s 4 × 3½ × 3/8 = 5.34 □″ gross. or 4.680 □″ net

same as for stringer S_1

Design of Shirahige-Bashi.

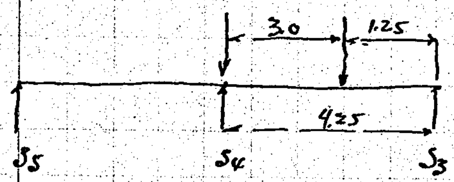
Highway stringers S3-S4-S5 span length 14'-6" spacing 4'-3"

Dead Load

Slab and pavement 95# × 4.25 = 404
stringer assumed 50
454# per lin. ft.

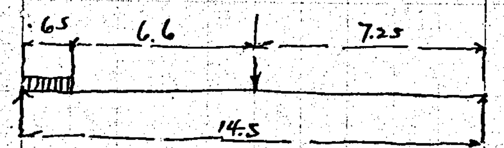
Dead Load moment = $\frac{1}{8} \cdot 454 \times 14.5^2$ = 11900 '#
Dead Load shear = $\frac{1}{2} \cdot 454 \times 14.5$ = 3300 #

Live Load motor car loading Rear wheel 9900
impact 30% 2970
12870 #



Load on S4 12870 × $\frac{1.25}{4.25}$ = 3790
12870
16660 #

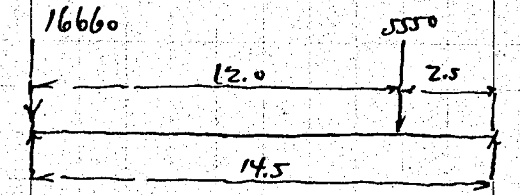
Front wheel 16660 ÷ 3 = 5550 #



Uniform load 120# × 4.25 = 510# per lin ft.
Reaction = 510 × $\frac{.325}{14.5}$ = 11.5 #

motor truck loading m = $\frac{16660}{2}$ × 7.25 = 60500
Uniform live load m = 11.5 × 7.25 = say 80
60580 '#

End shear



Front wheel 5550 × 2.5 ÷ 14.5 = 960
Rear wheel 16660
17620 '#

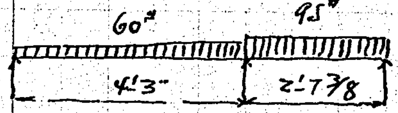
Summary for moments and shears Try 14" × 6" I @ 46.01# section modulus = 62.946

	moment	shear
Dead Load	11900	3300
Live Load	60580	17620
	72480 '#	20920 #

Unit stress = $\frac{72480 \times 12}{62.946}$ = 13860 #/□" ok

Stringer S6 at curb line span length = 14'-6" spacing 4'-3" and 2'-7 7/8"

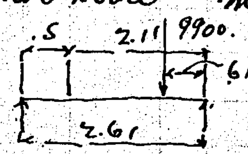
Dead Load



Highway slab r.c 95 × $\frac{2.61}{2}$ = $\frac{248}{2}$ = 124
Sidewalk slab 60 × $\frac{4.25}{2}$ = 128
Extra weight St. 0.5 @ 160 = 45
Gutter 35
80
Weight of stringer assumed 35
367# per lin. ft.

Dead load moment = $\frac{1}{8} \times 367 \times 14.5^2$ = 9650
Dead load shear = $\frac{1}{2} \times 367 \times 14.5$ = 2660

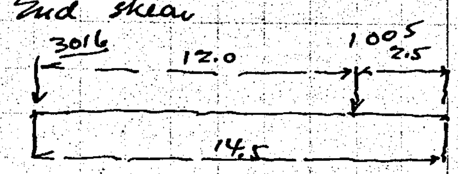
Live load motor truck loading on Highway.



Rear wheel 9900 × $\frac{.61}{2.61}$ = 2320
30% impact = 696
3016 #

m = $\frac{1}{4}$ × 3016 × 14.5 = 10940 '# Unif. load at rear neglected

End shear



1005 × 2.5 ÷ 14.5 = 172.
3016
3188 #

Uniform load on side walk 100 #/□'

Load on stringer = 100 × $\frac{4.25}{2}$ = 212.5# call this 210#

CALCULATIONS FOR

Design of Shiahige - Bashi.

moment = $\frac{1}{8} \cdot 210 \cdot 14.5^2 = 5530'^{\#}$ web assumed $12 \times 5/16 = 3.750''$

shear = $\frac{1}{2} \cdot 210 \cdot 14.5 = 1500^{\#}$ $\frac{1}{8}$ web = .470" Effective depth = 10.72"

Summary for moment and shear

Req flange section $= \frac{26120 \times 12}{10.72 \times 17000} = 1.72$
.47
1.260" net

	moment	shear
Dead Load	9650	2660
Live Load motor truck	10940	3188
" " Unif.	5530	1500
	26120'#	7348#

Use 1L 3 × 3 × 3/8 = 2.11 − .30 = 1.810" net.

Sidewalk stringer S7 and S8 span length 14'-6" spacing 4'-3"

Dead Load

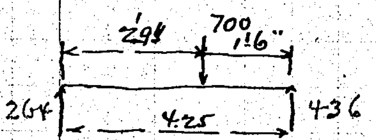
Slab + Pavement 60 × 4.25 = 255
Stringer assumed 45
300# per lin. ft.

Live Load 100 × 4.25 = 425# per lin ft.

Dead Load moment = $\frac{1}{8} \cdot 300 \cdot 14.5^2 = 7900'^{\#}$ Live Load moment = $\frac{1}{8} \times 425 \times 14.5^2 = 11200'^{\#}$

Dead Load shear = $\frac{1}{2} \cdot 300 \cdot 14.5 = 2200^{\#}$ Live Load shear = $\frac{1}{2} \times 425 \times 14.5 = 3080^{\#}$

water main assumed 700# per lin. ft.



moment $= 700 \times \frac{2.75}{4.25} \times \frac{14.5^2}{8} = 12.000'^{\#}$

shear $= 700 \times \frac{2.75}{4.25} \times \frac{14.5}{2} = 3300^{\#}$

Summary for moment and shear Try 12" × 5" @ 31.99# I. Sm. = 36.686

Unit stress $= \frac{31100 \times 12}{36.686} = 10200^{\#}/\square''$ ok

	moment	shear
Dead Load	7900	2200
Live Load	11200	3080
water main	12000	3300
	31100'#	8580#

Fascia Stringer S9 span length 14'-6"

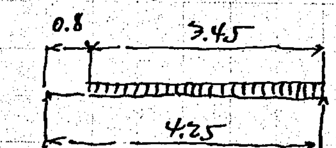
Dead Load

slab	60 × 4.25 ÷ 2	= 127.5
Coping stone	1.6 × 0.8 @ 160	= 204.8
Handrail assumed		100.0
stringer assumed		80.0
		512.3# call this 550# per lin ft.

Dead Load moment = $\frac{1}{8} \cdot 550 \cdot 14.5^2 = 14.500'^{\#}$

Dead Load shear = $\frac{1}{2} \cdot 550 \cdot 14.5 = 4.000^{\#}$

Live Load 100#/□' Load on stringer $= \frac{100 \times 3.45^2}{2 \times 4.25} = 140^{\#}$ per lin. ft.



moment = $\frac{1}{8} \cdot 140 \cdot 14.5^2 = 3680'^{\#}$

shear = $\frac{1}{2} \cdot 140 \cdot 14.5 = 1020^{\#}$

Summary for moments and shears web assumed 12" × 3/8 at center = 4.50"

$\frac{1}{8}$ web = 0.560" depth say 10.67"

	Moment	shear
Dead Load	14500	4.000
Live Load	3680	1020
	18180'#	5020#

flange stress $= \frac{18180 \times 12}{10.67} = 20400^{\#}$

SK = 20400 ÷ 17000 = 1.20□"
0.56
0.640" net.

Use for top flange 1L 6 × 4 × 3/8
" bottom flange 1L 3 × 3 × 3/8 = 2.11 − 0.3 = 1.81□" net.

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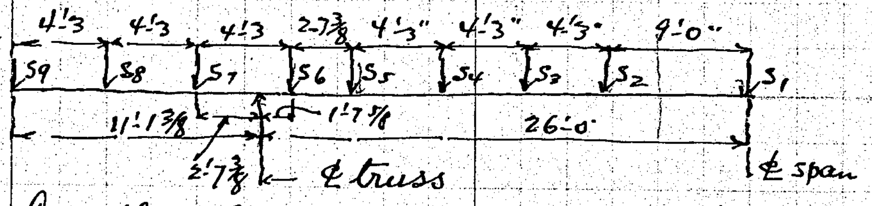
CALCULATIONS FOR

Design of Shirahige - Bashi.

Intermediate Floor Beam and Cantilever Bracket
span length 52'-0" c to c of trusses Cantilever arm = 11'-1 3/8 spacing 14'-6"
Dead Load.

From the preceding calculation the concentration at stringer connections

S_9	=	8000	the load including weight of stringers
S_8	=	8200	
S_7	=	10700	
S_6	=	5300	
S_5	=	5400	
S_4	=	6600	
S_3	=	6600	
S_2	=	18100	
$\frac{1}{2} S_1$	=	14500	
		83400	



Cantilever Bracket.

moment at S8 8000 · 4.25 = 34000'# neg.

at S7 8000 · 8.5 = 68000
8200 · 4.25 = 34800
102800'#

at center line of truss
8000 · 11.11 = 89000
8200 · 6.86 = 56300
10700 · 2.61 = 27900
26900# 173200'#

Floor Beam between trusses.

at S6 56500 · 1.64 = 92500'#
Cantilever m − 173200
− 80700'#

at S5 56500 · 4.25 = 240.000
5300 · 2.61 = − 13800
226200
Less Cantilever m − 173200
+ 53000'#

at S4 56500 · 8.5 = 480.000
5400 × 4.25 = 23.000
5300 · 6.86 = 36400
− 59400
420600
Less Cant. m = 173200
247400

at S3 56500 · 4.25 · 3 = 720.000
6600 · 4.25 = 28.000
5400 · 8.5 = 45900
5300 · 10.86 = 57500
− 131400
588600
Less Cant. m − 173200
415.400'#

at S2 56500 · 4.25 · 4 = 960.000
6600 · 12.75 = 84200
5400 · 12.75 = 69000
5300 · 15.36 = 81500
− 234700
725.300
Cantilever m − 173200
552100'#

at S1 56500 · 2600 = 1495.000
18100 · 9.00 = 163000
6600 · 30.75 = 203000
5400 · 21.75 = 117500
5300 · 24.35 = 129000
− 612500
882500
Cant. m − 173200
709300'#

End shear for cantilever 26900#
" " " floor beam 56500#
83400# panel Conc. for truss

Dead Load weight of Cantilever assumed 200# per lin ft.

moment S8 $200 \cdot \frac{4.25^2}{2}$ = 1800# shear = 200 · 11.11 = 2200#

S7 $200 \cdot \frac{8.5^2}{2}$ = 7200#

℄ truss $200 \cdot \frac{11.11^2}{2}$ = 12400'#

Dead Load own weight of Floor Beam 320# per lin ft.

S6 $\frac{320}{2} \cdot 50.36 \cdot 1.64$ = 13200'# − 12400 = + 800'#

S5 ½ · 320 · 47.75 · 4.25 = 32500 − 12400 = + 20100'#

S4 ½ · 320 · 43.5 · 8.5 = 59100 − 12400 = 46700

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CALCULATIONS FOR

Design of Shirahige-Bashi.

S_3 $\frac{1}{2} \cdot 320 \cdot 39.25 \cdot 12.75 = 80100^{'\#} - 12400 = 67700^{'\#}$

S_2 $\frac{1}{2} \cdot 320 \cdot 35.00 \cdot 17.00 = 95500 - 12400 = 83100^{'\#}$

S_1 $\frac{1}{8} \cdot 320 \cdot 52^2 = 108000 - 12400 = 95600^{'\#}$

End shear $= \frac{1}{2} \cdot 320 \cdot 52.0 = 8300^{\#}$

Summary for Dead Load ~~Moments and shear~~

	S_8	S_7	℄ Truss	S_6	S_5	S_4	S_3	S_2	S_1
Conc.	−34000	−102800	−173200	−80700	+53000	+247400	+415400	+552100	+709300
own wt	−1800	−7200	−12400	+800	+20100	+46700	+67700	+83100	+95600
	−35800	−110,000	−185,600	−79900	+73100	+294100	+483100	+635200	804900

End shear for Cantilever $26900 + 2200 = 29100^{\#}$
" " for Floor Beam $56500 + 8300 = 64800^{\#}$

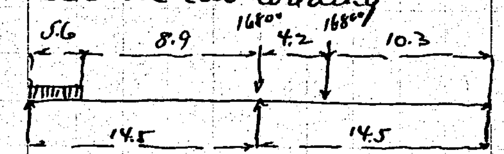
Live Load Uniform live load $= 100 \cdot 14.5 = 1450^{\#}$ per lin. ft

moment at S_8 $\frac{1450}{2} \cdot 4.5^2 = 14700^{'\#}$ shear $= \frac{1450}{2} \cdot 11.11 = 8050^{\#}$

at S_7 $\frac{1450}{2} \cdot 8.5^2 = 52300^{'\#}$

at ℄ $\frac{1450}{2} \cdot 11.11^2 = 89500^{'\#}$

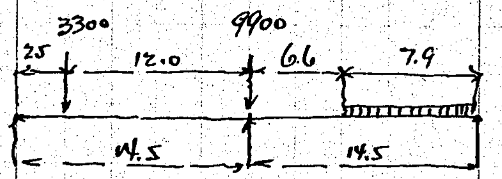
Electric car loading



Load on floor beam $16800 \cdot \frac{10.3}{14.5} = 11950$
16800
$28750^{\#}$
30% impact say 8650
$37400^{\#}$

Uniform load at front or rear $5.6 \cdot 120 \cdot \frac{2.8}{14.5} = 130^{\#}$ per lin ft.

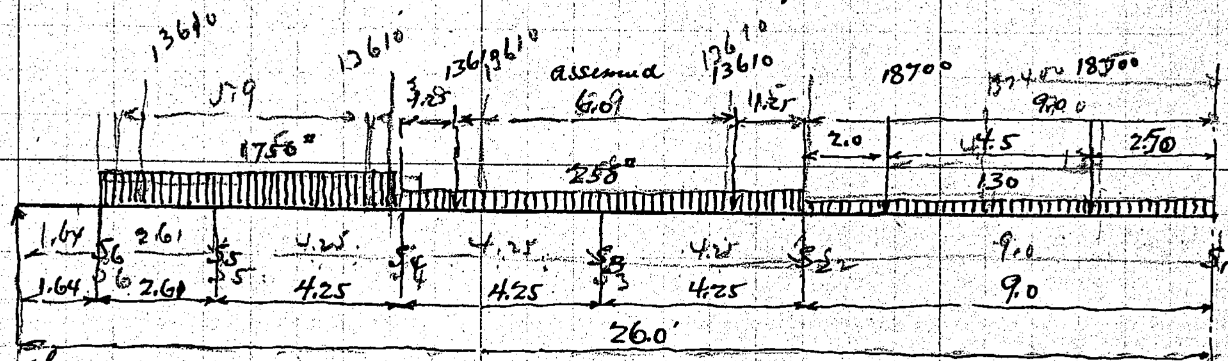
motor truck loading with uniform live load



Rear wheel with impact $= 12870^{\#}$
Front wheel " " $= 4290^{\#}$

Load on floor beam $= 4290 \cdot 2.5 \div 14.5 = 740$
12870
$13610^{\#}$ per wheel.

Unif. load $= 120 \cdot 7.9 \cdot \frac{3.95}{14.5} = 258^{\#}$ per lin ft.



Loading assumed as shown on sketch above.

Reaction on stringer

S_1. $37400 \cdot 4.25 \div 9.0 = 17650$
$130 \cdot 4.5 = 585$
$18235^{\#}$

S_2 $37400 - 17650 = 19750$
585
$13610 \cdot \frac{3.0}{4.25} = 9600$
$258 \cdot 2.12 = 545$
$30480^{\#}$

S_3 $(13610 - 9600) \cdot 2 = 8020$
$258 \cdot 4.25 = 1100$
9120

S_5 - $1750 \cdot 3.43 = 6000^{\#}$

S_4 9600
258×2.12 545
$1750 \cdot 2.12 = 3700$
$13845^{\#}$

S_6 - $1750 \cdot 1.31 = 2290^{\#}$

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CALCULATIONS FOR

Design of Shirahige-Bashi.

Reaction S1 18235
S2 30480
S3 9120
S4 13845
S5 6000
S6 2290
79970# say 80.000#

moment at S6 80.000 × 1.64 = 131.000#

S5 80.000 × 4.25 = 340.000
2290 × 2.61 = -6.000
334.000#

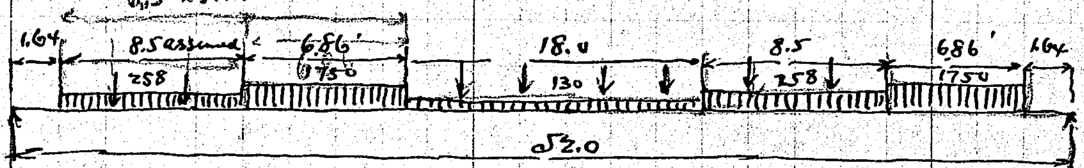
S4 80.000 × 8.5 = 680.000
6.000 × 4.25 = 25500
2290 × 6.86 = 15700
41200
638800#

S3 80.000 × 4.25 × 3 = 1,020.000
13845 × 4.25 = 58900
6000 × 8.50 = 51000
2290 × 10.86 = 24900
-134800
885200#

S2 80.000 × 4.25 × 4 1,360.000
9120 × 4.25 = 38700
13845 × 8.50 = 117700
6000 × 12.75 = 76500
2290 × 15.36 = 35200
266100
1,093900

S1 80000 × 26.0 2080.000
30.480 × 9.0 = 274000
9120 × 13.25 = 120800
13845 × 17.50 = 242000
6000 × 21.75 = 130500
2290 × 24.35 = 55700
793000
1287.000#

Max Live Load Shear.



Loading assumed as shown on diagram.

Uniform load
258 × 8.5 = 2200
130 × 18 = 2340
1750 × 6.86' = 12000

Electric car loading
37400 × 2 = 74800

motor truck loading
13610 × 2 = 27220#

Unif. load 12000 × 43.50 ÷ 52.0 = 10.000
2200 × 58.86 ÷ 52.0 = 2480
1170
13650

Motor Car 27220 × ~~54.31~~ 58.86 ÷ 52.0 = ~~28500~~ 30600
Electric Car loading 37400
~~79370~~
81650#

Max Reaction on Hanger.
Extra reaction due to load on sidewalk beyond ℄ truss Cant. m 89500 ÷ 52 = 1720#
81650
83370
Call this 84000#

Summary for moment

	S8	S7	℄ Truss	S6	S5	S4	S3	S2	S1
Dead Load	-35800	-110.000	-185600	-79900	+73100	+294100	483100	635200	804900
Live Load	-14700	-52300	-89500	131000	334000	638800	885200	1093900	1287.000
	-50500	-162300	-275100	+51100	+407100	+932900	1368300	1729.100	2091.900#

Max End shear.

Cantilever Bracket
Dead Load 29100
Live Load 8050
37150#

Floor Beam
64800
81650
146450#

max load on panel point
Dead Load 64800
Live Load 84000
148800#

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

Design of Shirahige-Bashi.

Section of Floor Beam

Try web 60 × 7/16 = 26.25 1/8 web = 3.28□" Effective depth say 59.46"

Flange area = $\frac{2091900 \times 12}{59.46 \times 17000}$ = 24.80

3.28

21.52 □" net

Use 2∠s 6×6×1/2 = 11.50 − 2.0 = 9.50

1 Pl. 13½ × ½ = 6.75 − 1.0 = 5.75

1 Pl 13½ × ½ = 6.75 − 1.0 = 5.75

25.00 21.00 □" net

Revised moment at center.

80.000 × 26.0 ~~156~~ = 2080.000

37400 × 4.25 = 159.000 804500

1170 × 4.50 = 5300 1275500 "# Live Load

2200 × 13.25 = 29200 804900 Dead Load

27220 × 13.25 = 360.000 2080400 "# Total

12000 × 20.93 = 251000

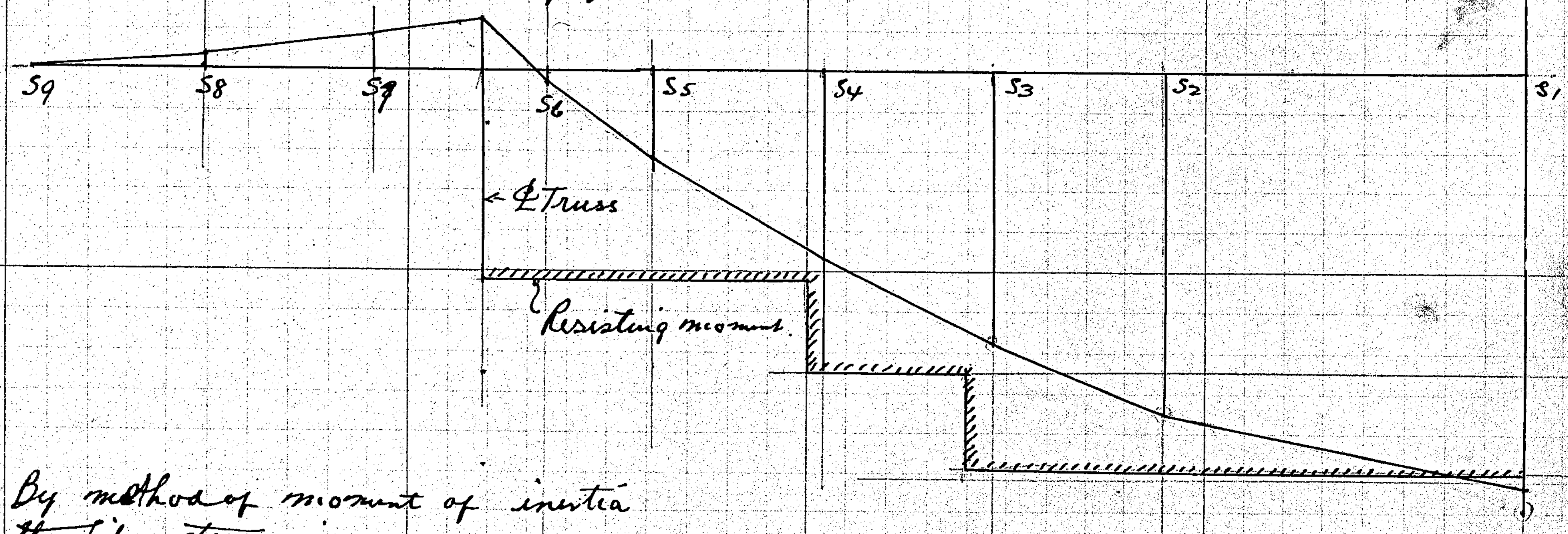
804500

Flange area $\frac{2080400 \times 12}{59.5 \times 17000}$ = 24.65

3.28

21.37 □" net.

Moment diagram and Section of floor beam



By method of moment of inertia
the fibre stress is

mo of I. 1 web. 60 × 7/16 = 7875

4∠s 6×6×½ = 23.00 × 28.57² + 80 = 17500

4 Pls. 13½ × ½ = 27.00 × 30.75² = 25600

50975 (")⁴

fibre stress = $\frac{2080400 \times 12 \times 30.25}{50975}$ = 15350 #/□"

For net section approx fibre stress = ~~15350 × 25/21~~ =

Cantilever Bracket use 2∠s 3½ × 3½ × 3/8 for flange

stress in tension plate 275100 ÷ 5.58 = 49300 #

8K = 49300 ÷ 17000 = 2.90□" net

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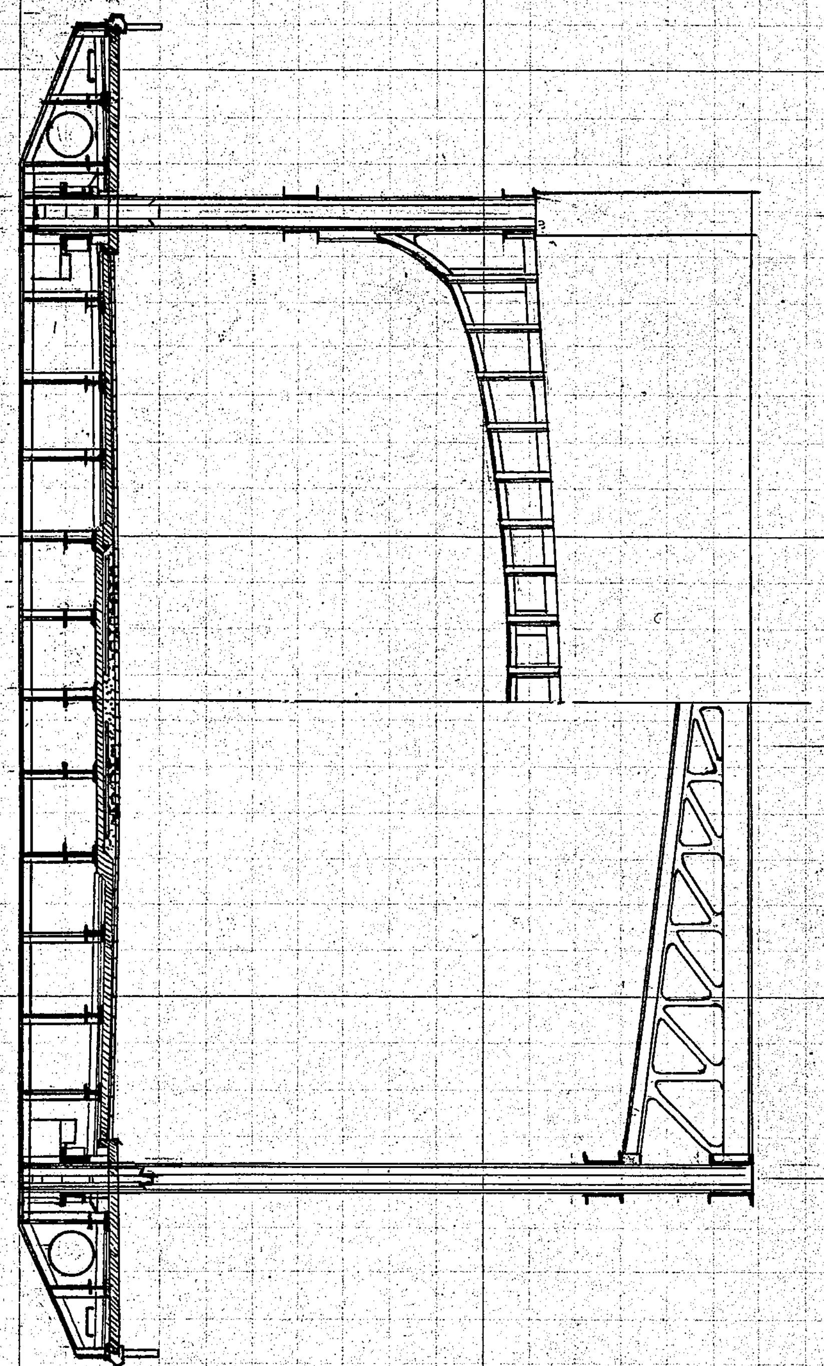
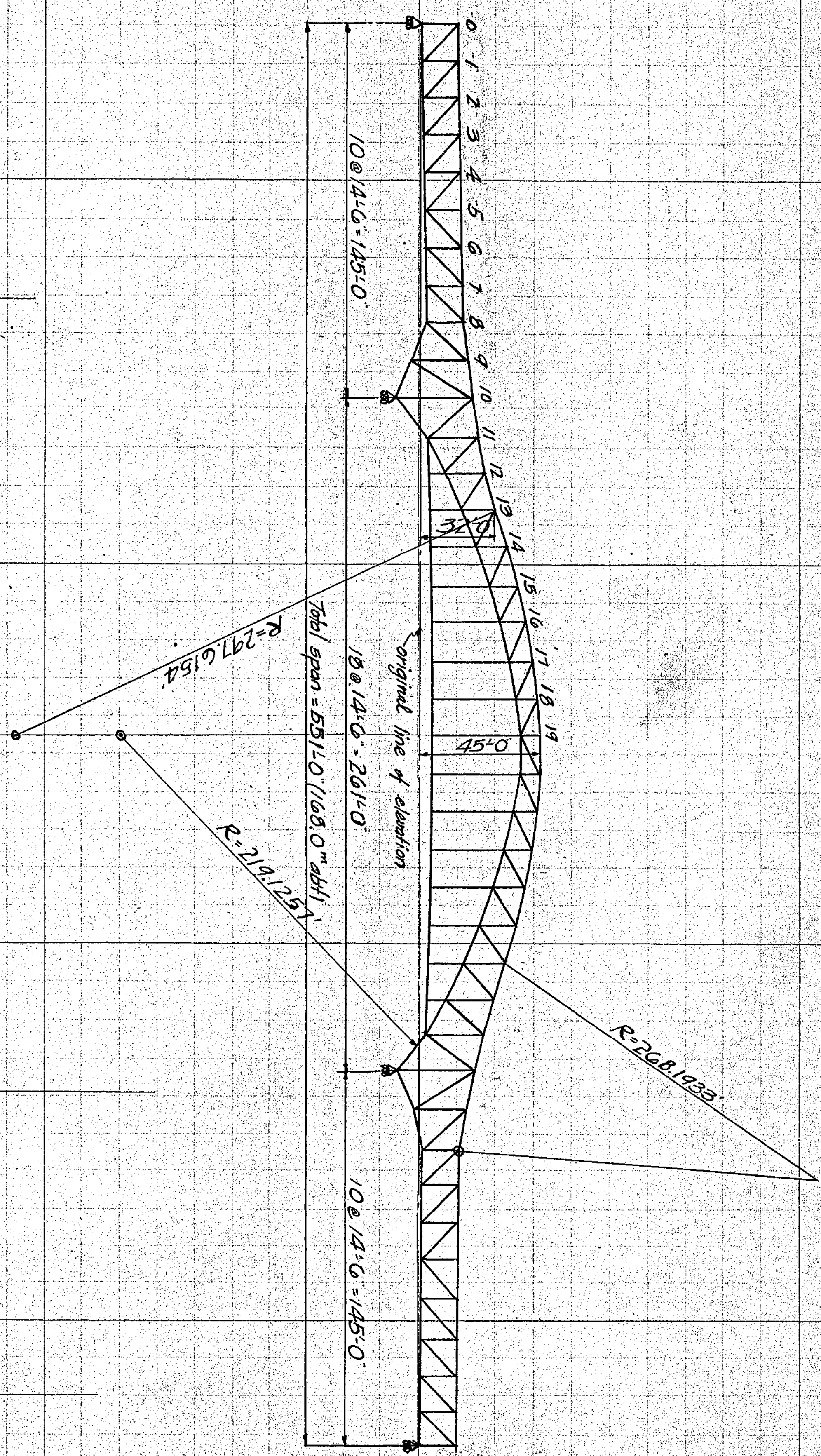
CALCULATIONS FOR





増田淳氏関係資料
(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTING ENGINEER
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CALCULATIONS FOR

Design of Shirahige-Bashi

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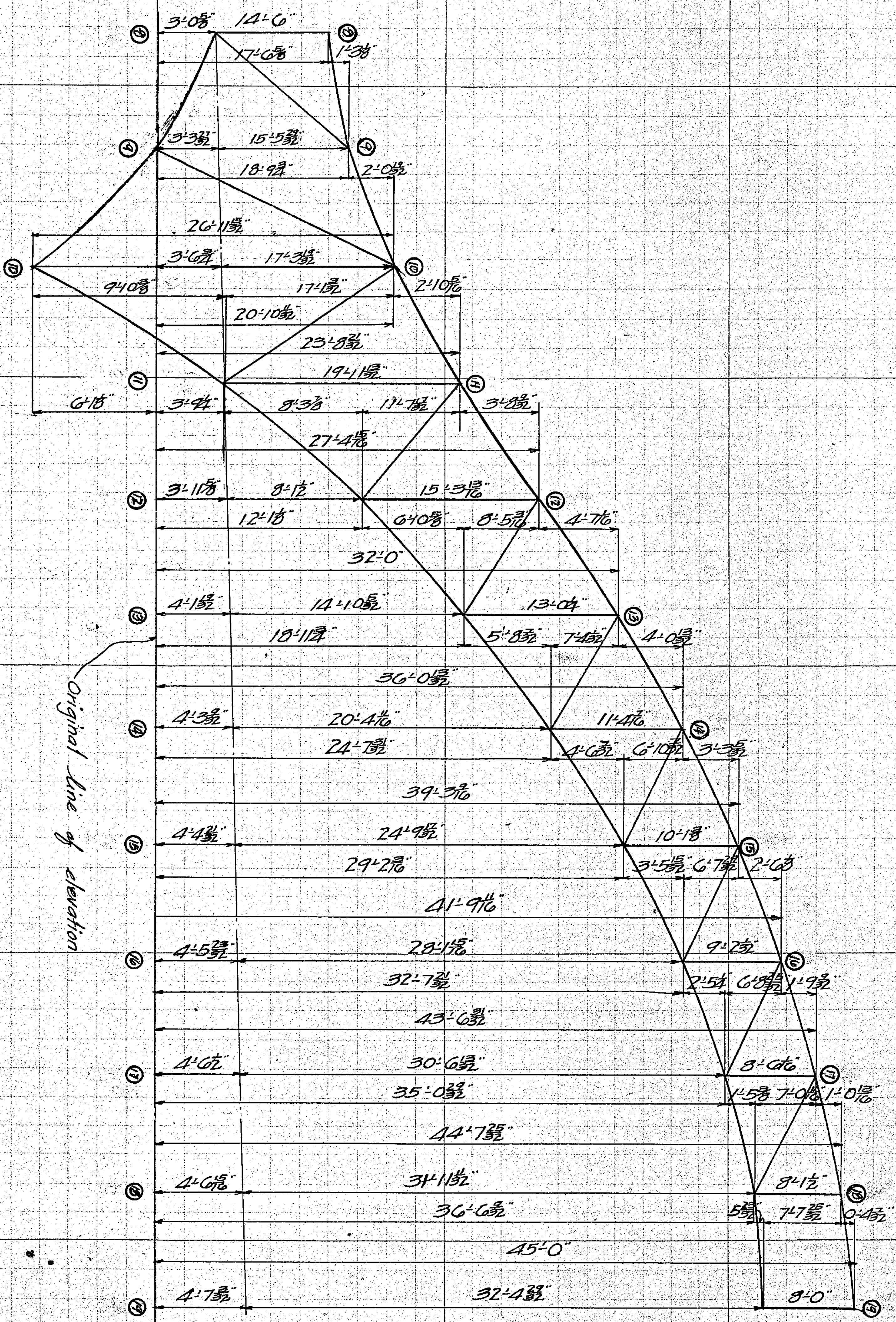

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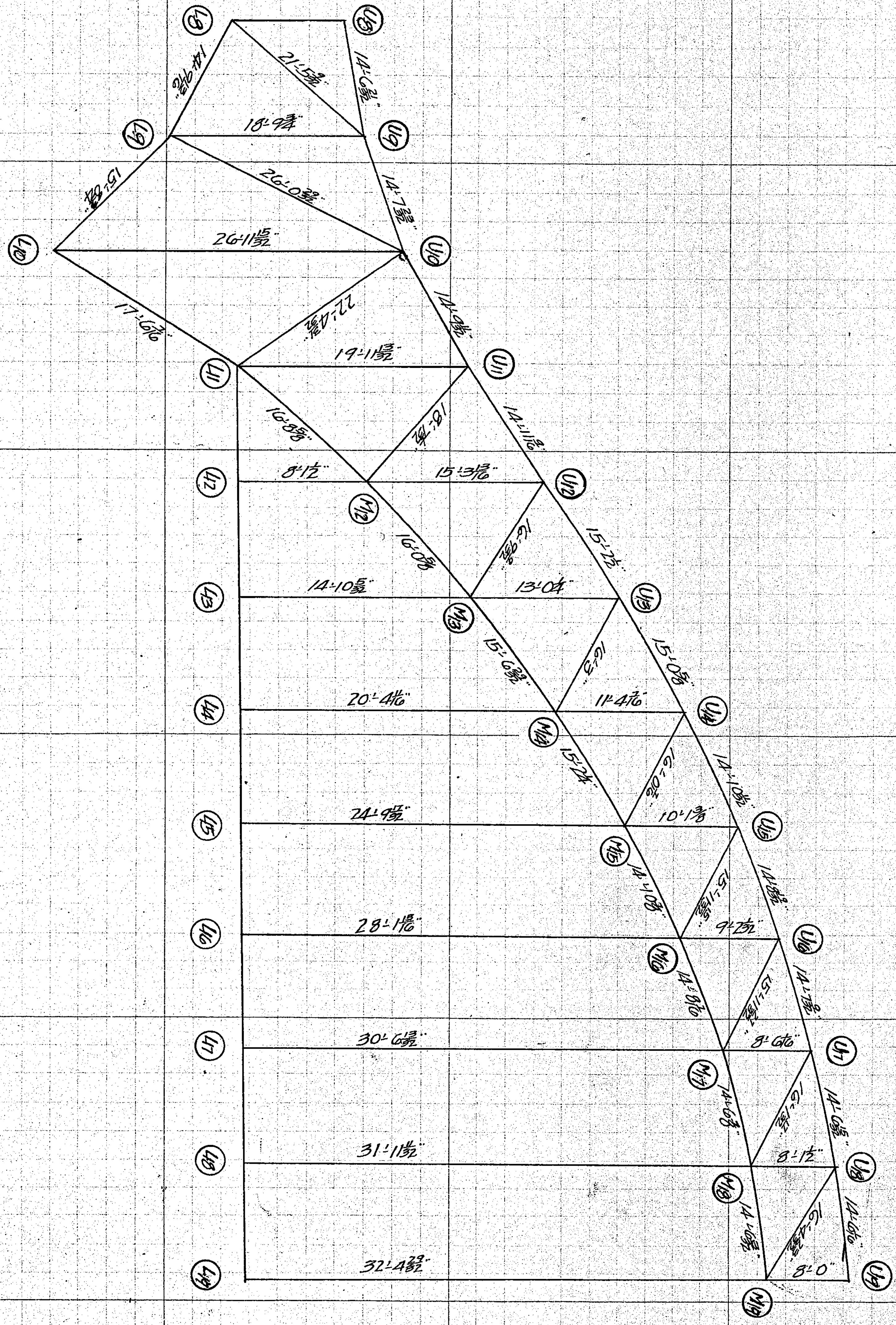
CALCULATIONS FOR

Design of Shirahige-Bashi



増田淳氏関係資料
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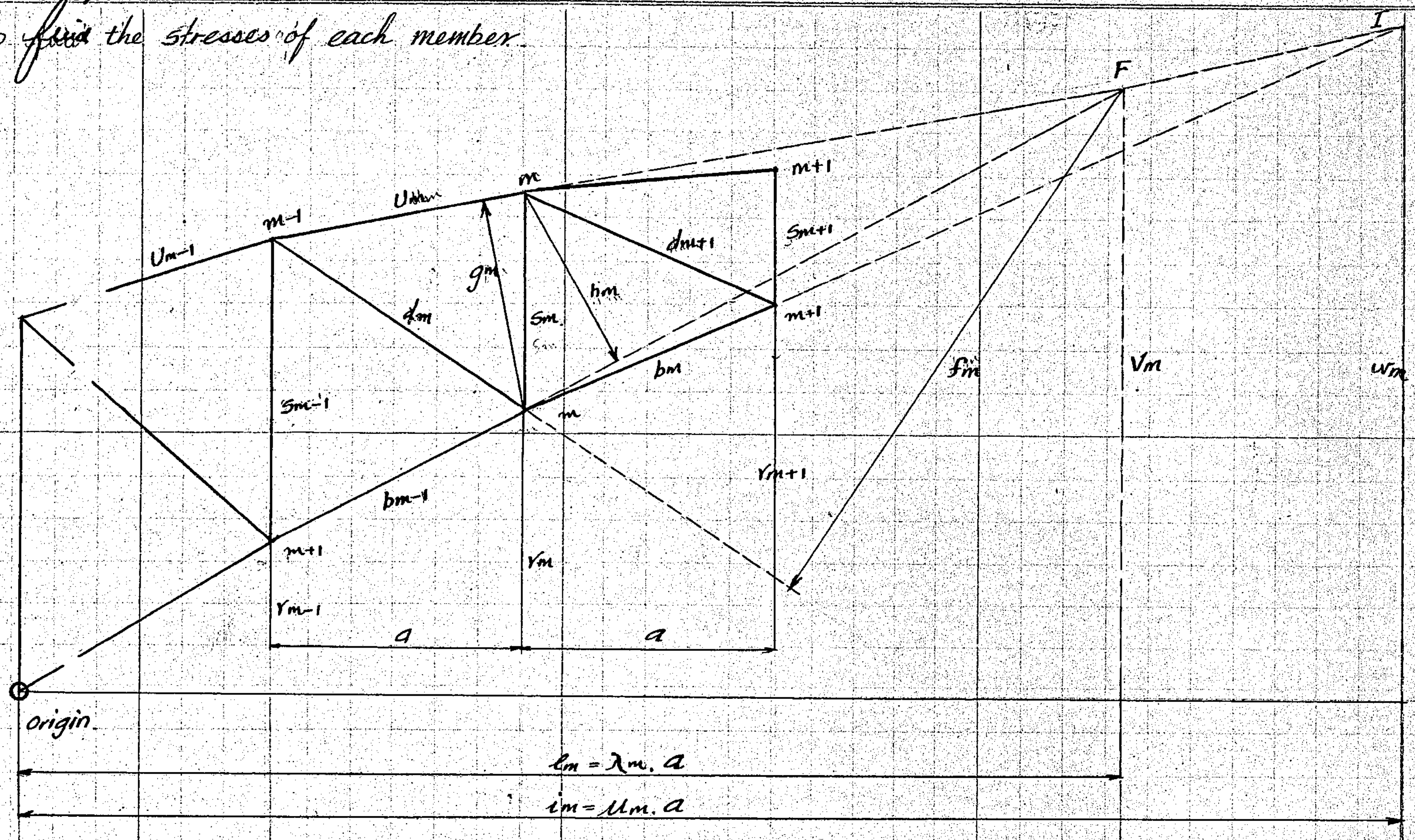
Design of Shirahige-Bashi

Date, needed to find the stresses of each member.



Notations

s_m = length of verticals.
d_m = length of diagonals.
u_m = length of upper chords.
b_m = length of lower chords.
r_m = length of hangers.
a = length of panel.
g_m = perpendicular distance from lower panel point m to upper chord u_m.
h_m = perpendicular distance from upper panel point m to lower chord b_m.
V_m = Vertical distance from intersection pt. (F) of chord u_m and chord b_{m-1}.
f_m = perpendicular distance from (F) to diagonals d_m.
w_m = Vertical distance from intersection pt (I) of chord u_m and b_m to left support.
e_m = horizontal distance from intersection pt (F) to left support $= \lambda_m . a$.
i_m = horizontal distance from intersection pt (I) to left support $= \mu_m . a$.

then in general.

$$g_m = \frac{a}{u_m} s_m$$

$$h_m = \frac{a}{b_m} s_m$$

$$V_m = \frac{s_{m-1}}{s_{m-1} - s_m} (r_m - r_{m-1}) + r_{m-1}$$

$$f_m = \frac{e_{m-1}}{s_{m-1} - s_m} \frac{a}{d_m} s_m.$$

$$e_m = \frac{s_{m-1}}{s_{m-1} - s_m} a + (m-1) a = \lambda_m . a$$

$$i_m = \frac{s_m}{(r_{m+1} - r_m) - (y_m - y_{m-1})} a + m a = \mu_m . a$$

where $y_m = r_m + s_m$.

$$w_m = \frac{s_m}{(r_{m+1} - r_m) - (y_m - y_{m-1})} (r_{m+1} - r_m) + r_m$$

Find $g_m = \frac{a}{u_m} s_m$ $a = 14'6"$ $\log a = 1.16137$.

m	u_m	s_m	$\log s_m$	$\log u_m$	$\log g_m$	g_m	g_m
8	$14'1\frac{1}{4}"$	$14'6"$	1.16137	1.17428	1.14846	$14'0\frac{29}{32}"$	14.08
9	$14'7\frac{23}{32}"$	$18'9\frac{3}{4}"$	1.27445	1.16564	1.27018	$18'7\frac{17}{32}"$	18.63
11	$14'9\frac{11}{32}"$	$19'11\frac{13}{32}"$	1.29995	1.16963	1.29169	$19'6\frac{29}{32}"$	1.958
12	$14'11\frac{9}{16}"$	$15'3\frac{13}{16}"$	1.18519	1.17503	1.17153	$14'10\frac{1}{8}"$	14.84
13	$15'2\frac{1}{2}"$	$13'0\frac{1}{4}"$	1.11464	1.18208	1.09393	$12'4\frac{31}{32}"$	12.41
14	$15'0\frac{5}{8}"$	$11'4\frac{7}{16}"$	1.05575	1.17760	1.03952	$10'11\frac{7}{16}"$	10.95
15	$14'10\frac{11}{32}"$	$10'1\frac{3}{8}"$	1.00495	1.17208	0.99424	$9'10\frac{13}{32}"$	9.88
16	$14'8\frac{19}{32}"$	$9'2\frac{1}{32}"$	0.96233	1.16779	0.95591	$9'0\frac{7}{16}"$	9.04
17	$14'7\frac{9}{32}"$	$8'6\frac{7}{16}"$	0.92968	1.16455	0.92650	$8'5\frac{5}{16}"$	8.44
18	$14'6\frac{15}{32}"$	$8'1\frac{1}{2}"$	0.90982	1.16254	0.90865	$8'1\frac{1}{4}"$	8.10
19	$14'6\frac{1}{16}"$	$8'0"$	0.90309	1.16152	0.90294	$7'11\frac{31}{32}"$	7.99

CALCULATIONS FOR

Design of Shirahige-Bashi

Find $h_m = \frac{a}{b_m} S_m$ $\log a = 1.16137$

m	log S_m	b_m	log b_m	log h_m	h_m	h_m
9	1.27445	14'-9 13/16"	1.17078	1.26504	18'-4 29/32"	18.41
10	1.43065	15'-8 3/4"	1.19671	1.39531	24'-10 3/16"	24.85
10	1.43065	17'-6 7/16"	1.24394	1.34808	22'-3 15/16"	22.33
11	1.29995	16'-8 5/8"	1.22320	1.18812	15'-5 1/16"	15.42
12	1.18519	16'-0 5/8"	1.20553	1.14013	13'-10 1/32"	13.84
13	1.11464	15'-6 29/32"	1.19244	1.08357	12'-1 15/32"	12.12
14	1.05575	15'-2 1/4"	1.18149	1.03563	10'-10 1/4"	10.85
15	1.00495	14'-10 7/8"	1.17339	0.99295	9'-10 1/16"	9.84
16	0.96233	14'-8 7/16"	1.16741	0.95629	9'-0 1/2"	9.04
17	0.92968	14'-6 7/8"	1.16355	0.92750	8'-5 9/16"	8.46
18	0.90982	14'-6 3/32"	1.16160	0.90959	8'-1 7/16"	8.12

Find $f_m = \frac{S_{m-1}}{S_{m-1}-S_m} \frac{a}{d_m} S_m$ $\log a = 1.16137$

注意　−mノ場合ハ先ヅ equation ニ +m ヲ挿入シテ出シタルモノヲ−符号ニ置キ換ヘベシ

m	S_{m-1}	S_m	$S_{m-1}-S_m$	log S_{m-1}	log S_m	log $S_{m-1}-S_m$	d_m	log d_m	log f_m	f_m
8	18'-9 3/4"	14'-6"	4'-3 3/4"	1.27445	1.16137	0.63473	21'-5 9/32"	1.33123	1.63123	42.78
9	26'-11 15/32"	18'-9 3/4"	8'-1 23/32"	1.43065	1.27445	0.91080	26'-0 23/32"	1.41615	1.53952	34.64
11	26'-11 15/32"	19'-11 13/32"	7'-0 1/16"	1.43065	1.29995	0.84542	22'-4 31/32"	1.35051	1.69604	49.66
12	19'-11 13/32"	15'-3 13/16"	4'-7 9/32"	1.29995	1.18519	0.66584	18'-7 1/32"	1.26918	1.71149	51.46
13	15'-3 13/16"	13'-0 1/4"	2'-3 9/16"	1.18519	1.11464	0.36114	16'-9 9/32"	1.22462	1.87544	75.07
14	13'-0 1/4"	11'-4 7/16"	1'-7 13/16"	1.11464	1.05575	0.21776	16'-3"	1.21085	1.90315	80.01
15	11'-4 7/16"	10'-1 3/8"	1'-3 1/16"	1.05575	1.00495	0.09872	16'-0 7/16"	1.20511	1.91824	82.84
16	10'-1 3/8"	9'-2 1/32"	0'-11 13/32"	1.00495	0.96233	$\bar{1}$.97558	15'-11 15/32"	1.20292	1.95009	89.14
17	9'-2 1/32"	8'-6 1/16"	0'-7 31/32"	0.96233	0.92968	$\bar{1}$.82221	15'-11 27/32"	1.20377	2.02740	106.52
18	8'-6 1/16"	8'-1 1/2"	0'-4 9/16"	0.92968	0.90982	$\bar{1}$.58002	16'-1 17/32"	1.20757	2.21328	163.41
19	8'-1 1/2"	8'-0"	0'-1 1/2"	0.90982	0.90309	$\bar{1}$.09691	16'-4 23/32"	1.21466	2.66271	459.95

Find $e_m = \frac{S_{m-1}}{S_{m-1}-S_m} a + (m-1)a$ $e_m' = \frac{S_{m-1}}{S_{m-1}-S_m} \cdot a$

m	log S_{m-1}	log $S_{m-1}-S_m$	log $\frac{S_{m-1}}{S_{m-1}-S_m} a$	e_m'	$(m-1)a$	e_m
8	1.27445	0.63473	1.80109	-63.26	-14.5	-77.76
9	1.43065	0.91080	1.68122	-48.00	0	-48.00
11	1.43065	0.84542	1.74660	55.80	0	55.80
12	1.29995	0.66584	1.79548	62.44	14.5	76.94
13	1.18519	0.36114	1.98542	96.70	29.0	125.70
14	1.11464	0.21776	2.05825	114.36	43.5	157.86
15	1.05575	0.09872	2.11840	131.34	58.0	189.34
16	1.00495	$\bar{1}$.97558	2.19074	155.15	72.5	227.65
17	0.96233	$\bar{1}$.82221	2.30149	200.22	87.0	287.22
18	0.92968	$\bar{1}$.58002	2.51103	324.37	101.5	425.87
19	0.90982	$\bar{1}$.09691	2.97428	942.50	116.0	1058.50

Find $V_m = \frac{S_{m-1}}{S_{m-1}-S_m}(Y_m - Y_{m-1}) + Y_m$ $V_m' = \frac{S_{m-1}}{S_{m-1}-S_m}(Y_m - Y_{m-1})$.

m	Y_m	Y_{m-1}	Y_m-Y_{m-1}	log Y_m-Y_{m-1}	log S_{m-1}	log $S_{m-1}-S_m$	log V_m'	V_m'	Y_m	V_m	V_m
8	3'-0 5/8"	0	3'-0 5/8"	0.48460	1.27445	0.63473	1.12432	13'-3 25/32"	3'-0 5/8"	16'-4 13/32"	16.37
9	0	-6'-1 1/8"	6'-1 1/8"	0.78488	1.43065	0.91080	1.30473	20'-2 1/16"	0	20'-2 1/16"	20.17
11	3'-9 1/4"	-6'-1 1/8"	9'-10 3/8"	0.99408	1.43065	0.84542	1.57931	37'-11 1/2"	3'-9 1/4"	41'-8 3/4"	41.73
12	12'-1 1/8"	3'-9 1/4"	8'-3 7/8"	0.92028	1.29995	0.66584	1.55439	35'-10 3/32"	12'-1 1/8"	47'-11 7/32"	47.93
13	18'-11 1/4"	12'-1 1/8"	6'-10 1/8"	0.89793	1.18519	0.36114	1.66298	46'-0 3/32"	18'-11 1/4"	65'-0 1/32"	65.00
14	24'-7 31/32"	18'-11 1/4"	5'-8 23/32"	0.75472	1.11464	0.21776	1.65160	44'-1 0"	24'-7 31/32"	69'-5 31/32"	69.50
15	29'-2 3/16"	24'-7 31/32"	4'-6 7/32"	0.65497	1.05575	0.09872	1.61200	40'-11 1/8"	29'-2 3/16"	70'-1 5/16"	70.11
16	32'-7 21/32"	29'-2 3/16"	3'-5 15/32"	0.53854	1.00495	$\bar{1}$.97558	1.56791	36'-11 1/16"	32'-7 21/32"	69'-7 11/32"	69.61
17	35'-0 29/32"	32'-7 21/32"	2'-5 1/4"	0.38694	0.96233	$\bar{1}$.82221	1.52706	33'-7 1/8"	35'-0 29/32"	68'-8 25/32"	68.73
18	36'-6 9/32"	35'-0 29/32"	1'-5 3/8"	0.16074	0.92968	$\bar{1}$.58002	1.51039	32'-4 21/32"	36'-6 9/32"	68'-10 15/16"	68.91
19	37'-0"	36'-6 9/32"	0'-5 23/32"	$\bar{1}$.67812	0.90982	$\bar{1}$.09691	1.49103	30'-11 23/32"	37'-0"	67'-11 21/32"	67.98

Design of Shirahige-Bashi

Find $i_m = \left(\dfrac{S_m}{(r_{m+1}-r_m)-(y_m-y_{m-1})}\right)a + ma$ $\qquad i_m' = \dfrac{S_m}{(r_{m+1}-r_m)-(y_m-y_{m-1})}\,a$

m	$(r_{m+1})-r_m)-(y_m-y_{m-1})$			$\log(r_{m+1}-r_m)-(y_m-y_{m-1})$	$\log S_m$	$\log i_m'$	i_m'	ma	i_m
9	3'-0 5/8"	1'-3 1/8"	1'-9 1/2"	0.25326	1.27445	2.18256	-152.25	-14.5	-166.75
11	8'-3 7/8"	2'-10 5/16"	5'-5 9/16"	0.73747	1.29995	1.72385	52.95	14.5	67.45
12	6'-10 5/8"	3'-8 9/32"	3'-2 11/32"	0.50451	1.18519	1.84205	69.51	29.0	98.51
13	5'-8 7/32"	4'-7 1/16"	1'-1 5/32"	0.03995	1.11464	2.23606	172.21	43.5	225.71
14	4'-6 7/32"	4'-0 13/32"	0'-5 13/16"	$\bar{1}.68518$	1.05575	2.53194	340.36	58.0	398.36
15	3'-5 15/32"	3'-3 5/32"	0'-2 5/16"	$\bar{1}.28490$	1.00495	2.88142	761.06	72.5	833.56
16	2'-5 1/4"	2'-6 1/8"	-0'-0 7/8"	$-\bar{2}.86283$	0.96233	-3.26087	-1823.40	87.0	-1736.40
17	1'-5 3/8"	1'-9 9/32"	-0'-3 29/32"	$-\bar{1}.51258$	0.92968	-2.57847	-378.96	101.5	-277.46
18	0'-5 23/32"	1'-0 13/16"	-0'-7 3/32"	$-\bar{1}.77169$	0.90982	-2.29950	-199.30	116.0	-83.30

Find $w_m = \dfrac{S_m}{(r_{m+1}-r_m)-(y_m-y_{m-1})}(r_{m+1}-r_m)+r_m$ $\qquad w_m' = \dfrac{S_m}{(r_{m+1}-r_m)-(y_m-y_{m-1})}(r_{m+1}-r_m)$

m	$\log S_m$	$\log(r_{m+1}-r_m)-(y_m-y_{m-1})$	$\log(r_{m+1}-r_m)$	$\log w_m'$	w_m'	r_m	w_m
8	1.27445	0.25326	0.48460	1.50579	-32.05	0	-32.05
11	1.29995	0.73747	0.92028	1.48276	30.39	3.77	34.16
12	1.18519	0.50451	0.83793	1.51861	33.02	12.09	45.11
13	1.11464	0.03995	0.75472	1.82941	67.52	18.98	86.50
14	1.05575	$\bar{1}.68518$	0.65497	2.02554	106.06	24.66	130.72
15	1.00495	$\bar{1}.28490$	0.53854	2.25859	181.38	29.18	210.56
16	0.96233	$-\bar{2}.86283$	0.38694	-2.48644	-306.51	32.64	-273.87
17	0.92968	$-\bar{1}.51258$	0.16074	-1.57784	-37.83	35.08	- 2.75
18	0.90982	$-\bar{1}.77169$	$\bar{1}.67812$	-0.81625	- 6.55	36.52	29.97

TABULATE THE TERMS U_m. b_m. d_m. S_m. r_m. g_m. h_m. f_m. e_m V_m. i_m and w_m.

m	U_m	b_m	d_m	S_m	r_m	g_m	h_m	f_m	e_m	V_m	i_m	w_m
8	14.94	—	21.44	14.50	3.05	14.08	—	42.78	-77.76	16.37	—	—
9	14.64	14.82	26.08	18.81	0	18.63	18.63	18.41	3	20.17	166.75	-32.05
10	—	15.73	—	26.96	-6.09	—	24.85	—	—	—	—	—
10	—	17.54	—	26.96	-6.09	—	22.33	—	—	—	—	—
11	14.78	16.72	22.41	19.95	3.77	19.58	15.42	49.66	55.80	41.73	67.45	34.16
12	14.96	16.05	18.59	15.32	12.09	14.84	13.84	51.46	76.94	47.93	98.51	45.11
13	15.21	15.58	16.77	13.02	18.98	12.41	12.12	75.07	125.70	65.00	225.71	86.50
14	15.05	15.19	16.25	11.37	24.66	10.95	10.85	80.01	157.86	69.50	398.36	130.72
15	14.86	14.91	16.04	10.11	29.18	9.88	9.84	82.84	189.34	70.11	833.56	210.56
16	14.72	14.70	15.96	9.17	32.64	9.04	9.04	89.14	227.65	69.61	-1736.40	-273.87
17	14.61	14.51	15.99	8.51	35.08	8.44	8.46	106.52	287.22	68.73	-277.46	-2.75
18	14.54	14.51	16.13	8.13	36.52	8.10	8.12	163.41	425.87	68.91	-83.30	29.97
19	14.51	—	16.39	8.00	37.00	7.99	—	459.95	1058.50	67.98	—	—

Formula used to find the stresses of tied arch.

$$X_a = \frac{\Sigma S_o S_a \delta + \Sigma \varepsilon t_o S_a l}{\Sigma S_a^2 \delta}$$

$$X_a = \frac{\Sigma S_o S_a \delta}{\Sigma S_a^2 \delta}$$ neglecting the temperature effect.

where X_a = redundancy.
S_o = stresses when redundancy removed.
S_a = stresses due to $X_a = -1$
$\delta = \frac{l}{AE}$
l = length of each member
E = modulus of elasticity of each member.
A = Sectional areas of member.

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CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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CHECKED BY ______ DATE ______ PAGE NO 20

CALCULATIONS FOR

Design of Shirahige-Bashi

Find Fo for upper chords

load on \ member	-2	-1	1	2	3	4	5	6	7	8	9
8'	0	0	0.082	0.217	0.390	0.561	0.815	1.069	1.336	1.591	1.815
9'	0	0	0.041	0.108	0.195	0.281	0.408	0.535	0.668	0.796	0.907
11'	0	0	-0.041	-0.108	-0.195	-0.281	-0.408	-0.535	-0.668	-0.796	-0.907
12'	0	0	-0.082	-0.217	-0.390	-0.561	-0.815	-1.069	-1.336	-1.591	-1.815
13'	0	0	-0.124	-0.326	-0.584	-0.842	-1.223	-1.604	-2.004	-2.387	-2.722
14'	0	0	-0.165	-0.434	-0.779	-1.122	-1.631	-2.139	-2.672	-3.182	-3.629
15'	0	0	-0.206	-0.543	-0.974	-1.403	-2.039	-2.673	-3.341	-3.978	-4.537
16'	0	0	0.247	-0.652	-1.169	-1.683	-2.446	-3.208	-4.009	-4.774	-5.444
17'	0	0	-0.288	-0.760	-1.364	-1.964	-2.854	-3.743	-4.677	-5.569	-6.351
18'	0	0	-0.330	-0.868	-1.558	-2.245	-3.262	-4.277	-5.345	-6.365	-7.258
19'	0	0	-0.371	-0.977	-1.753	-2.525	-3.669	-4.812	-6.013	-7.160	-8.166
18	0	0	-0.411	-1.075	-1.948	-2.806	-4.077	-5.347	-6.681	-7.956	-7.258
17	0	0	-0.452	-1.195	-2.143	-3.086	-4.485	-5.881	-7.349	-6.962	-6.351
16	0	0	-0.493	-1.303	-2.338	-3.367	-4.892	-6.416	-6.300	-5.967	-5.444
15	0	0	-0.535	-1.412	-2.532	-3.647	-5.300	-5.347	-5.250	-4.973	-4.537
14	0	0	-0.576	-1.520	-2.727	-3.928	-4.240	-4.277	-4.200	-3.978	-3.629
13	0	0	-0.617	-1.628	-2.922	-2.946	-3.180	-3.208	-3.150	-2.984	-2.722
12	0	0	-0.659	-1.737	-1.950	-1.964	-2.120	-2.139	-2.100	-1.989	-1.815
11	0	0	-0.700	-0.868	-0.975	-0.982	-1.060	-1.069	-1.050	-0.995	-0.907
9	0	0	-0.700	0.868	0.975	0.982	1.060	1.069	1.050	0.995	0.907
8	0	0.778	1.400	1.736	1.950	1.964	2.120	2.139	2.100	1.989	1.815

Fo for upper chords

35P

member \ load on.	-2	-1	1	2	3	4	5	6	7	8	9
8	0	0	0	0	0	0	0	0	0	0	0
9	0	0	0	0	0	0	0	0	0	0	0
11	0.082	0.041	-0.041	-0.082	-0.124	-0.165	-0.206	-0.247	-0.288	-0.330	-0.371
12	0.217	0.108	-0.108	-0.217	-0.326	-0.437	-0.543	-0.652	-0.760	-0.868	-0.977
13	0.390	0.195	-0.195	-0.390	-0.584	-0.779	-0.974	-1.169	-1.364	-1.558	-1.753
14	0.561	0.281	-0.281	-0.561	-0.842	-1.122	-1.403	-1.683	-1.964	-2.245	-2.525
15	0.815	0.408	-0.408	-0.815	-1.223	-1.631	-2.039	-2.446	-2.854	-3.262	-3.669
16	1.069	0.535	-0.535	-1.069	-1.604	-2.139	-2.673	-3.208	-3.743	-4.277	-4.812
17	1.336	0.668	-0.668	-1.336	-2.004	-2.672	-3.341	-4.009	-4.677	-5.345	-6.013
18	1.591	0.796	-0.796	-1.591	-2.387	-3.182	-3.978	-4.774	-5.569	-6.365	-7.160
19	1.815	0.907	-0.907	-1.815	-2.722	-3.629	-4.539	-5.444	-6.351	-7.258	-8.166
19'	1.815	0.907	-0.907	-1.815	-2.722	-3.629	-4.537	-5.444	-6.351	-7.258	-8.166
18'	1.989	0.995	-0.995	-1.989	-2.984	-3.978	-4.973	-5.967	-6.962	-7.956	-7.160
17'	2.100	1.050	-1.050	-2.100	-3.150	-4.200	-5.250	-6.300	-7.349	-6.681	-6.013
16'	2.139	1.069	-1.069	-2.139	-3.208	-4.277	-5.347	-6.416	-5.881	-5.347	-4.812
15'	2.120	1.060	-1.060	-2.120	-3.180	-4.240	-5.300	-4.892	-4.485	-4.077	-3.669
14'	1.964	0.982	-0.982	-1.964	-2.946	-3.928	-3.647	-3.367	-3.086	-2.806	-2.525
13'	1.950	0.975	-0.975	-1.950	-2.922	-2.727	-2.532	-2.338	-2.143	-1.948	-1.753
12'	1.736	0.868	-0.868	-1.736	-1.628	-1.520	-1.412	-1.303	-1.195	-1.075	-0.977
11'	1.400	0.700	-0.700	-0.659	-0.617	-0.576	-0.535	-0.493	-0.452	-0.411	-0.371

CALCULATIONS FOR

Design of Shirahige-Bashi

9'	0.778	0	0	0	0	0	0	0	0	0	0
8'	0.	0	0	0	0	0	0	0	0	0.	0

Find Io for lower chords

load on \ member	9	10	10	11	12	13	14	15	16	17	18
8'	0	0	0	-0.105	-0.233	-0.399	-0.600	-0.819	-1.069	-1.333	-1.588
9'	0	0	0	-0.052	-0.116	-0.200	-0.300	-0.409	-0.535	-0.667	-0.794
11'	0	0	0	0.052	0.116	0.200	0.300	0.409	0.535	0.667	0.794
12'	0	0	0	0.105	0.233	0.399	0.600	0.819	1.069	1.333	1.588
13'	0	0	0	0.157	0.349	0.599	0.900	1.228	1.604	2.000	2.382
14'	0	0	0	0.209	0.466	0.788	1.200	1.637	2.139	2.667	3.176
15'	0	0	0	0.261	0.582	0.998	1.500	2.046	2.674	3.333	3.970
16'	0	0	0	0.314	0.698	1.197	1.800	2.456	3.208	4.000	4.764
17'	0	0	0	0.366	0.815	1.397	2.100	2.865	3.743	4.667	5.558
18'	0	0	0	0.418	0.931	1.596	2.400	3.274	4.278	5.333	6.352
19'	0	0	0	0.470	1.048	1.796	2.700	3.683	4.812	6.000	7.145
18	0	0	0	0.523	1.164	1.995	3000	4.093	5.347	6.666	7.940
17	0	0	0	0.575	1.280	2.195	3.300	4.502	5.882	7.333	6.948
16	0	0	0	0.627	1.397	2.394	3.600	4.911	6.417	6.286	5.955
15	0	0	0	0.679	1.513	2.594	3.900	5.320	5.348	5.238	4.963
14	0	0	0	0.732	1.630	2.792	4.200	4.256	4.278	4.190	3.970
13	0	0	0	0.783	1.746	2.993	3.150	3.192	3.209	3.143	2.978
12	0	0	0	0.836	1.862	1.995	2.100	2.128	2.139	2.095	1.985
11	0	0	0	0.888	0.931	0.998	1.050	1.064	1.070	1.048	0.993
9	0	0.584	0.650	-0.888	-0.931	-0.998	-1.050	-1.064	-1.070	-1.048	-0.993
8	-0.778	-1.167	-1.300	-1.776	-1.862	-1.995	-2.100	-2.128	-2.139	-2.095	-1.985

member \ load on	8	9	11	12	13	14	15	16	17	18	19
9'	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0
10	0	0.	0	0	0	0	0	0	0	0	0
11	-0.105	-0.052	0.052	0.105	0.157	0.209	0.261	0.314	0.366	0.418	0.470
12	-0.233	-0.116	0.116	0.233	0.349	0.466	0.582	0.698	0.815	0.931	1.048
13	-0.399	-0.200	0.200	0.399	0.599	0.798	0.998	1.197	1.397	1.596	1.796
14	-0.600	-0.300	0.300	0.600	0.900	1.200	1.500	1.800	2.100	2.400	2.700
15	-0.819	-0.409	0.409	0.819	1.228	1.637	2.046	2.456	2.865	3.274	3.683
16	-1.069	-0.535	0.535	1.069	1.604	2.139	2.674	3.208	3.743	4.278	4.812
17	-1.333	-0.667	0.667	1.333	2.000	2.667	3.333	4.000	4.667	5.333	6.000
18	-1.588	-0.794	0.794	1.588	2.382	3.176	3.970	4.764	5.588	6.352	7.145
18'	-1.985	-0.993	0.933	0.985	2.978	3.970	4.963	5.955	6.948	7.940	7.145
17'	-2.095	-1.048	1.048	2.095	3.143	4.190	5.238	6.286	7.333	6.666	6.000
16'	-2.139	-1.070	1.070	2.139	3.209	4.278	5.348	6.417	5.882	5.347	4.812
15'	-2.128	-1.064	1.064	2.128	3.192	4.256	5.320	4.911	4.502	4.093	3.683
14'	-2.100	-1.050	1.050	2.100	3.150	4.200	3.900	3.600	3.300	3.000	2.700
13'	-1.995	-0.998	0.998	1.995	2.993	2.792	2.594	2.394	2.195	1.995	1.796
12'	-1.862	-0.931	0.931	1.862	1.746	1.630	1.513	1.397	1.280	1.164	1.048
11'	-1.776	-0.888	0.888	0.836	0.783	0.732	0.679	0.627	0.575	0.523	0.470
10'	-1.300	-0.650	0	0	0	0	0	0	0	0	0
10'	-1.167	-0.584	0	0	0	0	0	0	0	0	0
9'	-0.788	0	0	0	0	0	0	0	0	0	0

member \ load on	8	9	11	12	13	14	15	16	17	18	19
9	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0
11	-0.105	-0.052	0.052	0.105	0.157	0.209	0.261	0.314	0.366	0.418	0.470
12	-0.233	-0.116	0.116	0.233	0.349	0.466	0.582	0.698	0.815	0.931	1.048
13	-0.399	-0.200	0.200	0.399	0.599	0.798	0.998	1.197	1.397	1.596	1.796

CALCULATIONS FOR

Design of Shirahige - Bashi

member \ load on	8	9	11	12	13	14	15	16	17	18	19
14	-0.600	-0.300	0.300	0.600	0.900	1.200	1.500	1.800	2.100	2.400	2.700
15	-0.819	-0.409	0.409	0.819	1.228	1.637	2.046	2.456	2.865	3.274	3.683
16	-1.069	-0.535	0.535	1.069	1.604	2.139	2.674	3.208	3.743	4.278	4.812
17	-1.333	-0.667	0.667	1.333	2.000	2.667	3.333	4.000	4.667	5.333	6.000
18	-1.588	-0.794	0.794	1.588	2.382	3.176	3.970	4.764	5.588	6.352	7.145
18'	-1.985	-0.993	0.993	1.985	2.978	3.970	4.963	5.955	6.948	7.940	7.145
17'	-2.095	-1.048	1.048	2.095	3.143	4.190	5.238	6.286	7.333	6.666	6.000
16'	-2.139	-1.070	1.070	2.139	3.209	4.278	5.348	6.417	5.882	5.347	4.812
15'	-2.128	-1.064	1.064	2.128	3.192	4.256	5.320	4.911	4.502	4.093	3.683
14'	-2.100	-1.050	1.050	2.100	3.150	4.200	3.900	3.600	3.300	3.000	2.700
13'	-1.995	-0.998	0.998	1.995	2.993	2.792	2.594	2.394	2.195	1.995	1.796
12'	-1.862	-0.931	0.931	1.862	1.746	1.630	1.513	1.397	1.280	1.164	1.048
11'	-1.776	-0.888	0.888	0.836	0.783	0.732	0.679	0.627	0.575	0.523	0.470
10'	-1.300	-0.650	0	0	0	0	0	0	0	0	0
10'	-1.167	-0.584	0	0	0	0	0	0	0	0	0
9'	-0.788	0	0	0	0	0	0	0	0	0	0

Find So for diagonals

Load on \ member	8	9	11	12	13	14	15	16	17	18	19
8'	0	0	-0.125	-0.166	-0.186	-0.219	-0.254	-0.284	-0.300	-0.290	-0.256
9'	0	0	-0.062	-0.083	-0.093	-0.110	-0.127	-0.142	-0.150	-0.145	-0.128
11'	0	0	0.062	0.083	0.093	0.110	0.127	0.142	0.150	0.145	0.128
12'	0	0	0.125	0.166	0.186	0.219	0.254	0.284	0.300	0.290	0.256
13'	0	0	0.187	0.249	0.279	0.329	0.381	0.426	0.449	0.434	0.384
14'	0	0	0.250	0.332	0.372	0.438	0.508	0.568	0.599	0.579	0.512
15'	0	0	0.312	0.415	0.465	0.548	0.635	0.710	0.744	0.724	0.640
16'	0	0	0.374	0.498	0.558	0.658	0.762	0.851	0.899	0.869	0.767
17'	0	0	0.437	0.581	0.651	0.767	0.889	0.993	1.049	1.014	0.895
18'	0	0	0.499	0.664	0.744	0.877	1.016	1.135	1.198	1.158	1.023
19'	0	0	0.562	0.748	0.837	0.986	1.143	1.277	1.348	1.303	1.151
18	0	0	0.624	0.831	0.930	1.096	1.270	1.419	1.498	1.448	-0.770
17	0	0	0.687	0.914	1.023	1.206	1.400	1.561	1.648	-0.392	-0.673
16	0	0	0.749	0.997	1.116	1.315	1.524	1.703	-0.082	-0.336	-0.576
15	0	0	0.811	1.080	1.209	1.425	1.651	0.104	-0.074	-0.280	-0.480
14	0	0	0.874	1.163	1.302	1.535	0.193	0.083	-0.039	-0.224	-0.384
13	0	0	0.936	1.246	1.395	0.216	0.145	0.062	-0.044	-0.158	-0.288
12	0	0	0.999	1.329	0.200	0.144	0.097	0.042	-0.029	-0.112	-0.192
11	0	0	1.061	0.199	0.100	0.072	0.048	0.021	-0.015	-0.056	-0.096
9	0	0.967	-0.231	-0.199	-0.100	-0.072	-0.048	-0.021	0.015	0.056	0.096
8	1.139	0.548	-0.462	-0.398	-0.200	-0.144	-0.097	-0.042	0.029	0.112	0.192

Load on \ member	8	9	11	12	13	14	15	16	17	18	19
8	0	0	0	0	0	0	0	0	0	0	0
9	0	0	0	0	0	0	0	0	0	0	0
11'	-0.125	-0.062	0.062	0.125	0.187	0.250	0.312	0.374	0.437	0.499	0.562
12	-0.166	-0.083	0.083	0.166	0.249	0.332	0.415	0.498	0.581	0.664	0.748
13	-0.186	-0.093	0.093	0.186	0.279	0.372	0.465	0.558	0.651	0.744	0.837
14	-0.219	-0.110	0.110	0.219	0.329	0.438	0.548	0.658	0.767	0.877	0.986
15	-0.254	-0.127	0.127	0.254	0.381	0.508	0.635	0.762	0.889	1.016	1.143
16	-0.284	-0.142	0.142	0.284	0.426	0.568	0.710	0.851	0.993	1.135	1.277
17	-0.300	-0.150	0.150	0.300	0.449	0.599	0.744	0.899	1.049	1.198	1.348
18	-0.290	-0.145	0.145	0.290	0.434	0.579	0.724	0.869	1.014	1.158	1.303
19	-0.256	-0.128	0.128	0.256	0.384	0.512	0.640	0.767	0.895	1.023	1.151
19'	0.192	0.096	-0.096	-0.192	-0.288	-0.384	-0.480	-0.576	-0.673	-0.770	1.151

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

MADE BY ______ DATE ______ FILE NO. ______

CHECKED BY ______ DATE ______ PAGE NO. 23

CALCULATIONS FOR

Design of Shirahige-Bashi

10

member \ load on	8	9	11	12	13	14	15	16	17	18	19
18'	0.122	0.056	-0.056	-0.112	-0.158	-0.224	-0.280	-0.336	-0.392	1.448	1.303
17'	0.029	0.015	-0.015	-0.029	-0.044	-0.059	-0.074	-0.087	1.648	1.498	1.348
16'	-0.042	-0.021	0.021	0.042	0.062	0.083	0.104	1.703	1.561	1.419	1.277
15'	-0.097	-0.048	0.048	0.097	0.145	0.193	1.651	1.524	1.400	1.270	1.143
14'	-0.144	-0.072	0.072	0.144	0.216	1.535	1.425	1.315	1.206	1.096	0.986
13'	-0.200	-0.100	0.100	0.200	1.395	1.302	1.209	1.116	1.023	0.930	0.837
12'	-0.398	-0.199	0.199	1.329	1.246	1.163	1.080	0.997	0.914	0.831	0.748
11'	-0.462	-0.231	1.061	0.990	0.936	0.874	0.811	0.749	0.687	0.624	0.562
9'	0.548	0.967	0	0	0	0	0	0	0	0	0
	1.139	0	0	0	0	0	0	0	0	0	0

Find Lo for Verticals.

load on \ member	8	9	11	12	13	14	15	16	17	18	
8'	0	0	0.142	0.157	0.138	0.130	0.120	0.106	0.081	0.046	0
9'	0	0	0.071	0.079	0.069	0.065	0.060	0.053	0.041	0.023	
11'	0	0	-0.071	-0.079	-0.069	-0.065	-0.060	-0.053	-0.041	-0.023	
12'	0	0	-0.142	-0.157	-0.138	-0.130	-0.120	-0.106	-0.081	-0.046	
13'	0	0	-0.212	-0.236	-0.207	-0.195	-0.180	-0.159	-0.122	-0.070	
14'	0	0	-0.283	-0.315	-0.275	-0.260	-0.240	-0.212	-0.163	-0.093	
15'	0	0	-0.354	-0.394	-0.344	-0.325	-0.300	-0.265	-0.204	-0.116	
16'	0	0	-0.425	-0.472	-0.413	-0.390	-0.360	-0.318	-0.244	-0.139	
17'	0	0	-0.495	-0.551	-0.482	-0.455	-0.420	-0.371	-0.285	-0.162	
18'	0	0	-0.566	-0.630	-0.551	-0.520	-0.480	-0.424	-0.326	-0.186	
19'	0	0	-0.637	-0.708	-0.620	-0.585	-0.540	-0.477	-0.366	-0.209	
18	0	0	-0.708	-0.787	-0.689	-0.650	-0.600	-0.530	-0.470	0.768	
17	0	0	-0.778	-0.866	-0.757	-0.715	-0.660	-0.582	0.552	0.672	
16	0	0	-0.849	-0.944	-0.826	-0.780	-0.721	0.365	0.474	0.576	
15	0	0	-0.920	-1.023	-0.895	-0.846	0.219	0.304	0.395	0.480	
14	0	0	-0.991	-1.102	-0.962	0.089	0.175	0.243	0.316	0.384	
13	0	0	-1.061	-1.181	-0.033	0.067	0.131	0.182	0.237	0.288	
12	0	0	-1.132	-0.260	-0.022	0.045	0.088	0.122	0.158	0.192	
11	0	0	-0.203	-0.130	-0.011	0.022	0.044	0.061	0.079	0.096	
9	0	0	0.203	0.130	0.011	-0.022	-0.044	-0.061	-0.079	-0.096	
8	0	-0.904	0.406	0.260	0.022	-0.044	-0.088	-0.122	-0.158	-0.192	

member \ load on	8'	9'	11	12	13	14	15	16	17	18	19
8	0	0	0	0	0	0	0	0	0	0	0
9	0	0	0	0	0	0	0	0	0	0	0
11	0.142	0.071	-0.071	-0.142	-0.212	-0.283	-0.354	-0.425	-0.495	-0.566	-0.637
12	0.157	0.079	-0.079	-0.157	-0.236	-0.315	-0.394	-0.472	-0.551	-0.630	-0.708
13	0.138	0.069	-0.069	-0.138	-0.207	-0.275	-0.344	-0.413	-0.482	-0.551	-0.620
14	0.130	0.065	-0.065	-0.130	-0.195	-0.260	-0.325	-0.390	-0.455	-0.520	-0.585
15	0.120	0.060	-0.060	-0.120	-0.180	-0.240	-0.300	-0.360	-0.420	-0.480	-0.540
16	0.106	0.053	-0.053	-0.106	-0.159	-0.212	-0.265	-0.318	-0.371	-0.424	-0.477
17	0.081	0.041	-0.041	-0.081	-0.122	-0.163	-0.204	-0.244	-0.285	-0.326	-0.366
18	0.046	0.023	-0.023	-0.046	-0.070	-0.093	-0.116	-0.139	-0.162	-0.186	-0.209
18'	-0.192	-0.096	0.096	0.192	-0.288	-0.384	-0.480	-0.576	0.672	-0.768	-0.209
17'	-0.158	-0.079	0.079	0.158	0.237	0.316	0.395	0.474	0.552	-0.407	-0.366
16'	-0.122	-0.061	0.061	0.122	0.182	0.243	0.304	0.365	-0.582	-0.530	-0.477
15'	-0.088	-0.044	0.044	0.088	0.131	0.175	0.219	-0.721	-0.660	-0.600	-0.540
14'	-0.044	-0.022	0.022	0.044	0.067	0.089	-0.846	-0.780	-0.715	-0.650	-0.585
13'	0.022	0.011	-0.011	-0.022	-0.033	-0.964	-0.895	-0.826	-0.757	-0.689	-0.620
12'	0.260	0.130	-0.130	-0.260	-1.181	-1.102	-1.023	-0.944	-0.866	-0.787	-0.708
11'	0.406	0.203	-0.203	-1.132	-1.061	-0.991	-0.920	-0.849	-0.778	-0.708	-0.637
9'	-0.904	0	0	0	0	0	0	0	0	0	0
8'	0	0	0	0	0	0	0	0	0	0	0

Design of Shiraluge-Bashi //

Find to due to $X_a = -1$.

for upper chords $\gamma_1 = 3.77$ | for lower chords

m	γ_m	$\gamma_m - \gamma_1$	g_m	$S_a = \frac{\gamma_m - \gamma_1}{g_m}$	m	$y_m - \gamma_1$	h_m	$S_a = \frac{y_m - \gamma_1}{h_m}$
12	12.09	8.32	14.84	0.561	11	19.95	1542	-1.295
13	18.98	15.21	12.41	1.225	12	23.64	1384	-1.707
14	24.66	20.89	10.95	1.907	13	28.23	12.12	-2.329
15	29.18	25.41	9.88	2.573	14	32.26	10.85	-2.974
16	32.64	28.87	9.04	3.193	15	35.52	9.84	-3.610
17	35.08	31.31	8.44	3.708	16	38.04	9.04	-4.206
18	36.52	32.75	8.10	4.043	17	39.82	8.46	-4.707
19	37.00	33.23	7.99	4.158	18	40.88	8.12	-5.035

for Diagonals | for Verticals U

m	V_m	$V_m - \gamma_1$	f_m	$S_a = \frac{V_m - \gamma_1}{f_m}$	m	w_m	$w_m - \gamma_1$	$i_m - m_a$	$S_a = \frac{w_m - \gamma_1}{i_m - m_a}$
12	47.93	44.16	51.46	-0.858	11	34.16	30.39	52.95	0.574
13	65.00	61.23	75.07	-0.816	12	45.11	41.34	69.51	0.595
14	69.50	65.73	80.01	-0.822	13	86.50	82.73	182.21	0.454
15	70.11	66.34	82.84	-0.800	14	130.72	126.95	340.36	0.373
16	69.61	65.84	89.14	-0.750	15	210.56	206.79	761.06	0.272
17	68.73	64.96	106.52	-0.610	16	-273.87	-277.64	-1823.40	0.152
18	68.91	65.14	163.41	-0.398	17	-2.75	-6.52	-378.96	0.017
19	67.98	64.21	459.95	-0.140	18	29.97	26.20	-199.30	-0.131

Find H-Surface $X_a = \frac{\Sigma S_o S_a S}{\Sigma S_a^2 S}$

For upper chords.

Mark	Member	$S_{o.1}$	S_a	l	$S_{o.1} S_a l$	$S_a^2 l$	A	$S_{a.1} S_a \frac{l}{A}$	$S_a^2 \frac{l}{A}$
A	11	-0.041	0	14.78	0.000	0.000	63.58°"	0.000	0.000
B	12	-0.108	0.561	14.96	-0.907	4.708	"	-0.014	0.074
C	13	-0.195	1.225	15.21	-3.633	22.826	"	-0.057	0.359
D	14	-0.281	1.907	15.05	-8.065	54.737	"	-0.127	0.861
E	15	-0.408	2.573	14.86	-15.600	98.378	92.39	-0.169	1.065
F	16	-0.535	3.193	14.72	-25.146	150.070	118.38	-0.212	1.267
G	17	-0.668	3.708	1.461	-36.188	200.888	"	-0.306	1.697
H	18	-0.796	4.043	14.54	-46.793	237.671	"	-0.395	2.008
K	19	-0.907	4.158	14.51	-54.722	250.878	"	-0.462	2.118
L	19'	-0.907	4.158	14.51	-54.722	250.878	"	-0.462	2.118
M	18'	-0.995	4.043	14.54	-58.491	237.671	"	-0.494	2.008
N	17'	-1.050	3.708	14.61	-56.883	200.888	"	-0.481	1.697
O	16'	-1.069	3.193	14.72	-50.244	150.070	"	-0.424	1.267
P	15'	-1.060	2.573	14.86	-40.529	98.378	92.39	-0.439	1.065
Q	14'	-0.982	1.907	15.05	-28.184	54.737	63.58	-0.443	0.861
R	13'	-0.975	1.225	15.21	-18.161	22.826	"	-0.286	0.359
S	12'	-0.868	0.561	14.96	-7.285	4.708	"	-0.115	0.074
T	11'	-0.700	0	14.78	0.000	0.000	"	0.000	0.000

$\Sigma S_a^2 \frac{l}{A} = 18.898$

Find lower chords

Mark	Member	$S_{o.1}$	S_a	l	$S_{o.1} S_a l$	$S_a^2 l$	A	$S_{a.1} S_a \frac{l}{A}$	$S_a^2 \frac{l}{A}$
A	10	0.062	0	22.41	0	0	80.84	0.000	0.000
B	12	0.083	-0.858	18.59	-1.323	13.685	27.95	-0.047	0.490
C	13	0.093	-0.816	16.77	-1.273	11.166	27.95	-0.046	0.400
D	14	0.110	-0.822	16.25	-1.469	10.980	27.95	-0.053	0.393
E	15	0.127	-0.800	16.04	-1.630	10.266	27.95	-0.058	0.367
F	16	0.142	-0.750	15.96	-1.700	8.978	27.95	-0.061	0.321
G	17	0.150	-0.610	15.99	-1.463	5.950	36.00	-0.041	0.165
H	18	0.145	-0.398	16.13	-0.931	2.555	37.76	0.025	0.068
K	18	0.128	-0.140	16.39	-0.294	0.321	37.76	0.008	0.009
L	18'	-0.096	-0.140	16.39	0.220	0.321	37.76	0.006	0.009
M	17'	-0.056	-0.398	16.13	0.360	2.555	37.76	0.010	0.068

CALCULATIONS FOR

Design of Shirahige-Bashi 40p. 12

17'	-0.015	-0.610	15.99	0.146	5.950		0.004	0.165
16'	0.021	-0.750	15.96	-0.251	8.978	27.95	-0.009	0.321
15'	0.048	-0.800	16.04	-0.615	10.266	27.95	-0.022	0.367
14'	0.072	-0.822	16.25	-0.962	10.980	27.95	-0.034	0.393
13'	0.100	-0.816	16.77	-1.368	11.166	27.95	-0.049	0.400
12'	0.199	-0.858	18.59	-3.174	13.685	27.95	-0.114	0.490
11'	1.061	0	22.41	0	0	80.84	0.000	0.000

$\Sigma S_a^2 \frac{l}{A} = 4.426$

For Verticals

	$S_{0.1}$	S_a	l	$S_{a1} S_a l$	$S_a^2 l$	A	$S_{0.1} S_a \frac{l}{A}$	$S_a^2 \frac{l}{A}$
10	0	0	29.96	0	0	140.93	0.000	0.000
11	-0.071	0.574	19.95	-0.813	6.573	30.88	-0.026	0.213
12	-0.079	0.595	15.32	-0.720	5.424	26.88	-0.027	0.202
13	-0.069	0.454	13.02	-0.408	2.684	26.88	-0.015	0.100
14	-0.065	0.373	11.37	-0.276	1.582	26.88	-0.010	0.059
15	-0.060	0.272	10.11	-0.165	0.748	26.88	-0.006	0.028
16	-0.053	0.152	9.17	-0.074	0.212	26.88	-0.003	0.008
17	-0.041	0.017	8.51	-0.006	0.002	26.88	0.000	0.000
18	-0.023	-0.131	8.13	0.024	0.140	26.88	0.001	0.005
18'	0.096	-0.131	8.13	-0.102	0.140	26.88	-0.004	0.005
17'	0.079	0.017	8.51	0.114	0.002	26.88	0.004	0.000
16'	0.061	0.152	9.17	0.085	0.212	26.88	0.003	0.008
15'	0.044	0.272	10.11	0.121	0.748	26.88	0.005	0.028
14'	0.022	0.373	11.37	0.093	1.582	26.88	0.003	0.059
13'	-0.011	0.454	13.02	-0.065	2.684	26.88	-0.002	0.100
12'	-0.130	0.595	15.32	-1.185	5.424	26.88	-0.044	0.202
11'	-0.203	0.574	19.95	-2.324	6.573	30.88	-0.075	0.213
10'	—	0	26.96	0	0	140.93	0.000	0.000

$\Sigma S_a^2 \frac{l}{A} = 1.230$

$S_{0.1} S_a \frac{l}{A}$	$S_{0.2} S_a \frac{l}{A}$	$S_{0.3} S_a \frac{l}{A}$	$S_{0.4} S_a \frac{l}{A}$	$S_{0.5} S_a \frac{l}{A}$	$S_{a6} S_a \frac{l}{A}$	$S_{a7} S_a \frac{l}{A}$	$S_{a8} S_a \frac{l}{A}$	$S_{a9} S_a \frac{l}{A}$
A	A	A	A	A	A	A	A	A
B	B	B	B	B	B	B	B	B
C	C	C	C	C	C	C	C	C
D	D	D	D	D	D	D	D	D
E	E	E	E	E	E	E	8 E	9 E
F	F	F	F	F	F	7 F	F	F
G	G	G	4 G	G	6 G	G	G	G
H	2 H	3 H	H	5 H	H	H	H	H
K	K	K	K	K	K	K	K	K
L	L	L	L	L	L	L	L	K
M	M	M	M	M	M	M	H	H
N	N	N	N	N	N	G	G	G
O	O	O	O	O	F	F	F	F
P	P	P	P	E	E	E	10 E	9 E
Q	Q	Q	D	D	D	11 D	D	D
R	R	C	C	13 C	12 C	C	C	C
S	B	15 B	14 B	B	B	B	B	B
T	16 A	A	A	A	A	A	A	A

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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CHECKED BY ______ DATE ______ PAGE NO 26

CALCULATIONS FOR

Design of Shirahige-Bashi 12

$\Sigma S_0 S_a \frac{l}{A}$

Mark	Upper chords	Lower chords	Diagonals	Verticals	Summary	Summary	Remarks
A	0.000	0.000	0.000	0.000	0.000	0.000	A
B	-0.014	-0.006	-0.047	-0.026	-0.093	-0.093	A-B
C	-0.057	-0.019	-0.046	-0.027	-0.149	-0.242	C
D	-0.127	-0.049	-0.053	-0.015	-0.244	-0.486	D
E	-0.169	-0.105	-0.058	-0.010	-0.342	-0.828	E
F	-0.212	-0.192	-0.061	-0.006	-0.471	-1.299	F
G	-0.306	-0.345	-0.041	-0.003	-0.659	-1.958	G
H	-0.395	-0.566	-0.025	0.000	-0.986	-2.944	H
K	-0.462	-0.933	-0.008	0.001	-1.402	-4.346	K
L	-0.462	-1.167	0.006	-0.004	-1.627	-5.973	L
M	-0.494	-0.888	0.010	0.004	-1.368	-7.341	M
N	-0.481	-0.689	0.004	0.003	-1.163	-8.504	N
O	-0.424	-0.500	-0.009	0.005	-0.928	-9.432	O
P	-0.439	-0.366	-0.022	0.003	-0.824	-10.256	P
Q	-0.443	-0.244	-0.034	-0.002	-0.723	-10.979	Q
R	-0.286	-0.156	-0.049	-0.044	-0.535	-11.514	R
S	-0.115	-0.106	-0.114	-0.075	-0.410	-11.924	S
T	0.000	0.000	0.000	0.000	0.000	-11.924	T

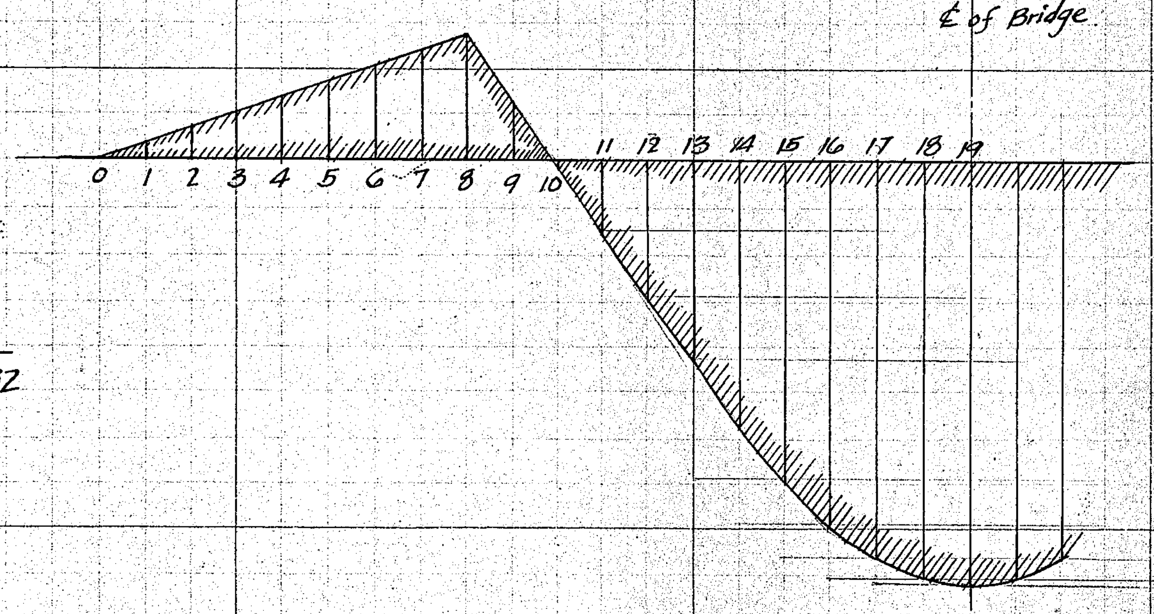
$\Sigma S_a^2 \frac{l}{A}$ for tie = 232÷144.22=1.608

$\Sigma S_a^2 \frac{l}{A}$ = 18.898+32.692+4.426+1.230+1.608 = 58.854

To Find H Surfaces

Load on			Total	÷58.854 Final H.	Primal H.
11	-1×11.924=-11.924		-11.924	-0.203	-0.209
12	-2×11.514=-23.028	-16×0.093=-1.488	-24.516	-0.418	-0.429
13	-3×10.979=-32.637	-15×0.242=-3.630	-36.267	-0.617	-0.641
14	-4×10.256=-41.024	-14×0.486=-6.804	-47.828	-0.814	-0.838
15	-5×9.432=-47.160	-13×0.828=-10.764	-57.924	-0.986	-1.018
16	-6×8.504=-51.024	-12×1.299=-15.588	-66.612	-1.133	-1.174
17	-7×7.341=-51.387	-11×1.958=-21.538	-72.925	-1.240	-1.292
18	-8×5.973=-47.784	-10×2.944=-29.440	-77.224	-1.314	-1.363
19	-9×4.346=-39.114	-9×4.346=-39.114	-78.224	-1.330	-1.380
				Σ=-14.780	Σ=-15.308

Load on		
10	0.000	0.000
9	0.203	0.209
8	0.406	0.418
7	0.355	0.366
6	0.305	0.314
5	0.254	0.261
4	0.203	0.209
3	0.152	0.157
2	0.102	0.105
1	0.051	0.052
0	0.000	0.000
	Σ=4.062	Σ=4.182



増田淳氏関係資料
(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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CHECKED BY ______ DATE ______ PAGE NO. 27

CALCULATIONS FOR

Design of Shirahige - Bashi 13

Find y_a X_a

For Upper chords

Member	y_a	panel H	11	12	13	14	15	16	17	18	19
			0.209	0.429	0.641	0.838	1.018	1.174	1.292	1.363	1.380
11	0.000		0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
12	0.561		0.117	0.241	0.369	0.470	0.571	0.659	0.725	0.766	0.774
13	1.225		0.256	0.528	0.785	1.027	1.247	1.438	1.583	1.672	1.692
14	1.907		0.399	0.818	1.222	1.598	1.942	2.238	2.463	2.600	2.630
15	2.573		0.538	1.103	1.648	2.155	2.620	3.020	3.324	3.510	3.550
16	3.193		0.667	1.379	2.045	2.675	3.250	3.747	4.124	4.355	4.405
17	3.708		0.775	1.592	2.375	3.108	3.777	4.354	4.790	5.060	5.117
18	4.043		0.845	1.733	2.592	3.387	4.116	4.746	5.220	5.511	5.580
19	4.158		0.869	1.783	2.664	3.484	4.234	4.882	5.370	5.667	5.738

For Lower chords

Member	y_a	panel H_1	1 11	2 12	3 13	4 14	5 15	16	7 17	8 18	9 19
			0.209	0.429	0.641	0.838	1.018	1.174	1.292	1.363	1.380
11	-1.295		-0.271	-0.556	-0.830	-1.085	-1.318	-1.520	-1.672	-1.765	-1.777
12	-1.707		-0.351	-0.732	-1.093	-1.430	-1.737	-2.004	-2.204	-2.326	-2.355
13	-2.329		-0.487	-1.000	-1.493	-1.952	-2.372	-2.735	-3.008	-3.176	-3.215
14	-2.974		-0.622	-1.276	-1.906	-2.492	-3.028	-3.490	-3.840	-4.055	-4.104
15	-3.610		-0.754	-1.549	-2.314	-3.025	-3.676	-4.236	-4.663	-4.920	-4.980
16	-4.206		-0.879	1.805	-2.696	-3.525	-4.284	-4.938	-5.432	-5.733	-5.804
17	-4.707		-0.984	-2.018	-3.017	-3.943	-4.792	-5.528	-6.080	-6.416	-6.496
18	-5.035		-1.052	-2.160	3.226	-4.220	-5.130	-5.910	-6.500	-6.863	-6.948

For Diagonals

Member	y_a	panel.	11	12	13	14	15	16	17	18	19
			0.209	0.429	0.641	0.838	1.018	1.174	1.292	1.363	1.380
11	0.000		0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
12	-0.858		-0.179	-0.368	-0.550	-0.719	-0.874	-1.008	-1.108	-1.170	-1.184
13	-0.816		-0.171	-0.350	-0.523	-0.684	-0.831	-0.958	-1.053	-1.113	-1.127
14	-0.822		-0.172	-0.353	-0.527	-0.689	-0.837	-0.966	-1.062	-1.121	1.131
15	-0.800		-0.167	-0.343	-0.513	-0.677	-0.815	-0.940	1.033	-1.092	-1.104
16	-0.750		-0.157	-0.322	-0.481	-0.629	-0.764	-0.881	-0.969	-1.023	-1.035
17	-0.610		-0.128	-0.262	-0.391	-0.511	-0.622	-0.717	-0.788	-0.832	-0.842
18	-0.398		-0.083	-0.171	-0.255	-0.334	-0.405	-0.468	-0.514	0.543	-0.549
19	-0.140		-0.029	-0.060	-0.090	-0.117	-0.143	-0.164	-0.181	0.191	0.193

For Verticals

Member	y_a	panel	11	12	13	14	15	16	17	18	19
			0.209	0.429	0.641	0.838	1.018	1.174	1.292	1.363	1.380
11	0.514		0.107	0.221	0.329	0.431	0.523	0.604	0.664	0.701	0.709
12	0.595		0.124	0.255	0.382	0.499	0.606	0.699	0.769	0.812	0.821
13	0.454		0.095	0.195	0.291	0.381	0.462	0.534	0.587	0.619	0.627
14	0.373		0.078	0.160	0.239	0.313	0.380	0.438	0.482	0.509	0.515
15	0.272		0.057	0.117	0.174	0.228	0.277	0.320	0.352	0.371	0.375
16	0.152		0.032	0.065	0.097	0.127	0.155	0.179	0.197	0.207	0.210
17	0.017		0.004	0.007	0.011	0.014	0.017	0.020	0.022	0.023	0.023
18	-0.131		-0.027	-0.056	-0.084	-0.110	-0.133	-0.154	-0.169	-0.179	-0.181

Influence surface for upper chords

member \ load on	8	9	11	12	13	14	15	16	17	18	19
8	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
11	0.082	0.041	-0.041	-0.082	-0.124	-0.165	-0.206	-0.247	-0.288	-0.330	-0.371
12	-0.018	-0.009	0.009	0.024	0.043	0.033	0.028	0.007	-0.035	-0.102	-0.203
13	-0.122	-0.061	0.061	0.138	0.201	0.248	0.273	0.269	0.219	0.144	-0.061
14	-0.236	-0.118	0.118	0.257	0.380	0.476	0.539	0.555	0.499	0.355	0.105
15	-0.260	-0.130	0.130	0.288	0.425	0.524	0.581	0.574	0.470	0.248	-0.119

CALCULATIONS FOR

Design of Shiraluge - Bashi　　　　19

16	-0.264	0.132	0.132	0.310	0.441	0.536	0.577	0.539	0.381	0.078	0.407
17	-0.214	-0.107	0.107	0.256	0.371	0.436	0.436	0.345	0.113	-0.285	-0.896
18	-0.098	-0.049	0.049	0.142	0.205	0.205	0.138	-0.028	-0.349	-0.854	-1.580
19	0.076	0.038	-0.038	-0.032	-0.058	-0.145	-0.303	-0.562	-0.981	-1.591	-2.428
19'	0.076	0.038	-0.038	-0.032	-0.058	-0.145	-0.303	-0.562	-0.981	-1.591	-2.428
18'	0.300	0.150	-0.150	-0.256	-0.392	-0.591	-0.857	-1.221	-1.742	-2.445	-1.580
17'	0.550	0.275	-0.275	-0.508	-0.775	-1.092	-1.473	-1.946	-2.559	-1.621	-0.896
16'	0.804	0.402	-0.402	-0.760	-1.163	-1.602	-2.097	-2.669	-1.757	-0.992	-0.407
15'	1.044	0.522	-0.522	-1.017	-1.532	-2.085	-2.680	-1.872	-1.161	-0.567	-0.119
14'	1.166	0.583	-0.583	-1.146	-1.724	-2.330	-1.705	-1.129	-0.623	-0.206	0.105
13'	1.438	0.719	-0.719	-1.422	-2.137	-1.700	-1.285	-0.900	0.560	-0.276	-0.061
12'	1.502	0.751	-0.751	-1.495	-1.259	-1.050	-0.841	-0.644	-0.478	-0.309	-0.203
11'	1.400	0.700	-0.700	-0.659	-0.617	-0.576	-0.535	-0.493	0.452	-0.411	0.371
9'	0.778	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8'	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Influence Surface for Lower chords

member \ load on	8	9	11	12	13	14	15	16	17	18	19
9	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
11	0.438	0.219	-0.219	-0.451	-0.673	-0.876	-1.057	-1.206	-1.306	-1.347	-1.307
12	0.482	0.241	-0.241	-0.499	-0.744	-0.964	-1.155	-1.306	-1.389	-1.395	-1.307
13	0.574	0.287	-0.287	-0.601	-0.894	-1.154	-1.374	-1.538	-1.611	-1.580	-1.419
14	0.644	0.322	-0.322	-0.672	-1.006	-1.292	-1.528	-1.690	-1.730	-1.655	-1.404
15	0.690	0.345	-0.345	-0.730	-1.086	-1.388	-1.630	-1.780	-1.798	-1.646	-1.297
16	0.688	0.344	-0.344	-0.736	-1.092	-1.386	-1.610	-1.730	-1.685	-1.492	-0.992
17	0.634	0.317	-0.317	-0.685	-1.017	-1.276	-1.459	-1.528	-1.413	-1.083	-0.496
18	0.516	0.258	-0.258	-0.572	-0.844	-1.044	-1.160	-1.146	-0.912	0.511	0.197
18'	0.118	0.059	-0.059	-0.175	-0.248	-0.250	-0.167	0.045	0.448	1.077	0.197
17'	-0.128	-0.064	0.064	0.077	0.126	0.247	0.446	0.758	1.253	0.250	-0.496
16'	-0.382	-0.191	0.191	0.334	0.513	0.753	1.064	1.479	0.450	-0.386	-0.992
15'	-0.620	-0.310	0.310	0.519	0.878	1.231	1.644	0.675	-0.101	-0.827	-1.297
14'	-0.856	-0.428	0.428	0.824	1.244	1.702	0.872	0.110	-0.541	-1.055	-1.404
13'	-1.022	-0.511	0.511	0.995	1.500	0.840	0.222	-0.341	-0.813	-1.181	-1.419
12'	-1.148	-0.574	0.574	1.130	0.653	0.200	-0.224	-0.607	-0.924	-1.162	-1.307
11'	-1.234	-0.617	0.617	0.280	-0.047	-0.353	-0.639	-0.893	-1.097	-1.242	-1.307
10'	-1.300	-0.650	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
10'	-1.167	-0.584	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9'	-0.788	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Influence Surface for Diagonals

member \ load on	8	9	11	12	13	14	15	16	17	18	19
8	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
11	-0.125	-0.062	0.062	0.125	0.187	0.250	0.312	0.374	0.437	0.499	0.562
12	0.192	0.096	-0.096	-0.202	-0.301	0.387	0.459	-0.510	-0.527	-0.506	-0.436
13	0.156	0.078	-0.078	-0.164	-0.244	-0.312	-0.366	-0.400	-0.402	-0.369	-0.290
14	0.124	0.062	-0.062	-0.134	-0.198	-0.251	-0.289	-0.308	-0.295	-0.244	-0.148
15	0.080	0.040	-0.040	-0.089	-0.132	-0.163	-0.180	-0.178	-0.144	-0.076	0.039
16	0.030	0.015	-0.015	-0.038	-0.055	-0.057	0.054	-0.030	0.024	0.112	0.242
17	-0.044	-0.022	0.022	0.038	0.058	0.088	0.122	0.182	0.261	0.366	0.506
18	-0.124	-0.062	0.062	0.119	0.179	0.245	0.319	0.401	0.500	0.615	0.754
19	-0.198	-0.099	0.099	0.196	0.294	0.395	0.497	0.603	0.714	0.832	0.958
19'	0.250	0.125	-0.125	-0.252	-0.378	-0.501	-0.623	-0.740	-0.854	-0.961	0.958
18'	0.278	0.139	-0.139	-0.283	-0.413	-0.558	-0.685	-0.804	-0.906	0.905	0.754
17'	0.286	0.143	-0.143	-0.291	-0.435	-0.570	-0.696	0.799	0.860	0.666	0.506

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

MADE BY ______ DATE ______ FILE NO ______

CHECKED BY ______ DATE ______ PAGE NO 29

CALCULATIONS FOR

Design of Shin-hige - Bashi

15

16'	0.252	0.126	-0.126	-0.280	-0.419	-0.546	-0.660	0.822	0.592	0.396	0.242
15'	0.238	0.119	-0.119	-0.246	-0.368	-0.478	0.836	0.584	0.367	0.178	0.039
14'	0.200	0.100	-0.100	-0.209	-0.311	-0.846	0.588	0.349	0.144	-0.025	-0.148
13'	0.142	0.071	-0.071	-0.150	0.872	0.618	0.378	0.158	-0.030	-0.183	-0.290
12'	-0.040	-0.020	0.020	+0.961	0.696	0.444	0.206	-0.011	-0.194	-0.339	-0.436
11'	-0.462	-0.231	1.061	0.990	0.936	0.874	0.811	0.749	0.687	0.624	0.562
9'	0.548	0.967	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8'	1.139	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Influence surfaces for Verticals

member \ load on	8	9	11	12	13	14	15	16	17	18	19
8	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
11	-0.072	-0.036	0.036	0.079	0.117	0.148	0.178	0.179	0.169	0.135	0.072
12	-0.090	-0.045	0.045	0.098	0.146	0.184	0.212	0.227	0.218	0.192	0.113
13	-0.056	-0.026	0.026	0.057	0.084	0.106	0.118	0.121	0.105	0.068	0.007
14	-0.026	-0.013	0.013	0.030	0.044	0.053	0.055	0.048	0.027	-0.011	-0.070
15	0.006	0.003	-0.003	-0.003	-0.006	-0.012	-0.023	-0.040	0.068	-0.109	-0.165
16	0.042	0.021	-0.021	-0.041	-0.062	-0.085	-0.110	-0.139	-0.174	-0.217	-0.267
17	0.074	0.037	-0.037	-0.054	-0.111	-0.149	-0.187	-0.224	-0.263	-0.303	-0.343
18	0.100	0.050	-0.050	-0.102	-0.154	-0.203	-0.250	-0.293	-0.331	-0.365	-0.390
18'	-0.138	-0.069	0.069	0.136	0.204	0.274	0.347	0.422	0.503	0.589	-0.390
17'	-0.166	-0.083	0.083	0.165	0.248	0.330	0.412	0.494	0.574	-0.384	-0.343
16'	-0.186	-0.093	0.093	0.187	0.279	0.370	0.459	0.544	-0.385	-0.323	-0.267
15'	-0.202	-0.101	0.101	0.205	0.305	0.403	0.496	-0.401	0.308	-0.229	-0.165
14'	-0.200	-0.100	0.100	0.204	0.306	0.402	0.466	0.324	0.233	-0.141	-0.070
13'	-0.168	-0.084	0.084	0.173	0.258	-0.583	-0.433	-0.328	-0.170	-0.070	0.007
12'	0.012	0.006	-0.006	-0.005	-0.793	-0.603	-0.417	-0.245	-0.097	0.025	0.113
11'	0.192	0.096	-0.096	-0.911	-0.732	-0.560	-0.397	-0.245	-0.114	-0.007	0.072
9'	-0.904	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
8'	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Influence surfaces for upper chords

load on \ Member	8	9	11	12	13	14	15	16	17	18	19
0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	0	0	0.010	-0.002	-0.015	-0.030	-0.033	-0.033	-0.027	-0.012	0.010
2	0	0	0.021	-0.005	-0.031	-0.059	-0.065	-0.066	-0.054	-0.025	0.019
3	0	0	0.031	-0.007	-0.046	-0.089	-0.098	-0.099	-0.080	-0.037	0.029
4	0	0	0.041	-0.009	-0.061	-0.118	-0.130	-0.132	-0.107	-0.049	0.038
5	0	0	0.051	-0.011	-0.076	-0.148	-0.163	-0.165	-0.134	-0.061	0.048
6	0	0	0.062	-0.014	-0.092	-0.177	-0.195	-0.198	-0.161	-0.074	0.057
7	0	0	0.072	-0.016	-0.107	-0.207	-0.228	-0.231	-0.187	-0.086	0.067
8	0	0	0.082	-0.018	-0.122	-0.236	-0.260	-0.264	-0.214	-0.098	0.076
9	0	0	0.041	-0.009	-0.061	-0.118	-0.130	-0.132	-0.107	0.049	0.038
10	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
11	0	0	-0.041	0.009	0.061	0.118	0.130	0.132	0.107	0.049	-0.038
12	0	0	-0.082	0.024	0.138	0.257	0.288	0.310	0.256	0.142	-0.032
13	0	0	-0.124	0.043	0.201	0.380	0.425	0.441	0.371	0.205	-0.058
14	0	0	-0.165	0.033	0.248	0.476	0.524	0.536	0.436	0.205	-0.145
15	0	0	-0.206	0.028	0.273	0.539	0.581	0.577	0.436	0.138	-0.303
16	0	0	-0.247	0.007	0.269	0.555	0.574	0.539	0.345	-0.028	-0.562
17	0	0	-0.288	-0.035	0.219	0.499	0.470	0.381	0.113	-0.349	-0.981
18	0	0	-0.330	-0.102	0.144	0.355	0.248	0.078	-0.285	-0.854	-1.591
19	0	0	-0.371	-0.203	-0.061	0.105	-0.119	-0.407	-0.896	-1.580	-2.428
18'	0	0	-0.411	-0.309	-0.276	-0.206	-0.567	-0.992	-1.621	-2.445	-1.591
17'	0	0	-0.452	-0.470	-0.560	-0.623	-1.161	-1.757	-2.559	-1.742	-0.981
16'	0	0	-0.493	-0.644	0.900	-1.129	-1.872	-2.669	-1.946	-1.221	-0.562

CALCULATIONS FOR

Design of Shirahige - Bashi

16

15'	0	0	-0.535	-0.841	-1.285	-0.705	-2.680	-2.097	-1.473	-0.857	-0.303
14'	0	0	-0.576	-1.050	-1.700	-2.330	-2.085	-1.602	-1.092	-0.591	-0.145
13'	0	0	-0.617	-1.259	-2.137	-1.724	-1.532	-1.163	-0.775	-0.392	-0.058
12'	0	0	-0.659	-1.495	-1.422	-1.146	-1.017	-0.760	-0.508	-0.256	-0.032
11'	0	0	-0.700	-0.751	-0.719	-0.583	-0.522	-0.402	-0.275	-0.150	-0.038
10'	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9'	0	0.000	0.700	0.751	0.719	0.583	0.522	0.402	0.275	0.150	0.038
8'	0	0.778	1.400	1.502	1.438	1.166	1.044	0.804	0.550	0.300	0.076
7'	0	0.681	1.225	1.314	1.258	1.020	0.914	0.704	0.481	0.263	0.067
6'	0	0.584	1.050	1.127	1.079	0.875	0.783	0.603	0.413	0.225	0.051
5'	0	0.486	0.875	0.939	0.899	0.729	0.653	0.503	0.344	0.188	0.048
4'	0	0.389	0.700	0.751	0.719	0.583	0.522	0.402	0.275	0.150	0.038
3'	0	0.292	0.525	0.563	0.539	0.437	0.392	0.302	0.206	0.113	0.029
2'	0	0.195	0.350	0.376	0.360	0.292	0.261	0.201	0.138	0.075	0.019
1'	0	0.097	0.175	0.188	0.180	0.146	0.131	0.101	0.069	0.038	0.010
0'	0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
+A'	0.000	0.000	0.000	0.144	1.553	3.284	3.240	3.696	2.064	0.739	0.000
-A'	0.000	0.000	6.296	7.159	9.060	9.446	11.555	11.849	11.430	10.465	9.848
+A	0.000	3.502	7.411	7.655	8.744	9.115	8.462	7.718	4.815	2.241	0.764
-A	0.000	0.000 -3.502	6.296	7.260	9.671	10.628	12.857	13.169	12.501	10.956	9.848

Influence surface for Lower Chords

load on. \ member	-9 1	-0 10	0 10	1 11	2 12	3 13	4 14	5 15	6 16	7 17	8 18
0'	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	0	0	0	0.055	0.060	0.072	0.081	0.086	0.086	0.079	0.065
2	0	0	0	0.110	0.121	0.144	0.161	0.173	0.172	0.159	0.129
3	0	0	0	0.164	0.181	0.215	0.242	0.259	0.258	0.238	0.194
4	0	0	0	0.219	0.241	0.287	0.322	0.345	0.344	0.317	0.258
5	0	0	0	0.274	0.301	0.359	0.403	0.431	0.430	0.396	0.323
6	0	0	0	0.329	0.362	0.431	0.483	0.518	0.516	0.476	0.387
7	0	0	0	0.383	0.422	0.502	0.564	0.604	0.602	0.555	0.452
8	0	0	0	0.4~~38~~32	0.482	0.574	0.644	0.690	0.688	0.634	0.516
9	0	0	0	0.219	0.241	0.287	0.322	0.345	0.344	0.317	0.258
10	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
11	0	0	0	-0.219	-0.241	-0.287	-0.322	-0.345	-0.344	-0.317	-0.258
12	0	0	0	-0.451	-0.499	-0.601	-0.672	-0.730	-0.736	-0.685	-0.572
13	0	0	0	-0.673	-0.744	-0.894	-1.006	-1.086	-1.092	-1.017	-0.844
14	0	0	0	-0.876	-0.964	-1.154	-1.292	-1.388	-1.386	-1.276	-1.044
15	0	0	0	-1.057	-1.155	-1.374	-1.528	-1.630	-1.610	-1.459	-1.160
16	0	0	0	-1.206	-1.306	-1.538	-1.690	-1.780	-1.730	-1.528	-1.146
17	0	0	0	-1.306	-1.389	-1.611	-1.730	-1.798	-1.685	-1.413	-0.912
18	0	0	0	-1.347	-1.395	-1.580	-1.655	-1.646	-1.492	-1.083	-0.511
19	0	0	0	-1.307	-1.307	-1.419	-1.404	-1.297	-0.992	-0.496	0.197
18'	0	0	0	-1.242	-1.162	-1.181	-1.055	-0.827	-0.386	+0.250	1.077
17'	0	0	0	-1.097	-0.924	-0.813	-0.540	-0.101	0.450	1.253	0.448
16'	0	0	0	-0.893	-0.607	-0.341	0.110	0.675	1.479	0.758	0.045
15'	0	0	0	-0.639	-0.224	0.222	0.872	1.644	1.064	0.446	-0.167
14'	0	0	0	-0.353	0.200	0.840	1.702	1.231	0.753	0.247	-0.250
13'	0	0	0	-0.047	0.653	1.500	1.244	0.878	0.513	0.126	-0.248
12'	0	0	0	0.280	1.130	0.995	0.824	0.579	0.334	0.077	-0.175
11'	0	0	0	0.617	0.574	0.511	0.428	0.310	0.191	0.064	-0.059
10'	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9'	0	-0.584	-0.650	-0.617	-0.574	-0.511	-0.428	-0.310	-0.191	-0.064	0.059
8'	-0.788	-1.167	-1.300	-1.234	-1.148	-1.022	-0.856	-0.620	-0.382	-0.128	0.118
7'	-0.690	1.022	1.138	-1.080	-1.005	-0.894	-0.749	-0.543	-0.334	-0.112	0.103

CALCULATIONS FOR

Design of Shia-lige-Bashi (7

6'	-0.591	0.876	-0.975	-0.926	-0.861	-0.766	-0.642	-0.465	-0.287	-0.096	0.089
5'	-0.493	0.730	-0.813	-0.771	-0.718	-0.638	-0.535	-0.388	-0.239	-0.080	0.074
4'	-0.394	0.584	-0.650	-0.617	-0.574	-0.515	-0.428	-0.310	-0.191	-0.064	0.059
3'	-0.296	0.438	-0.488	-0.463	-0.431	-0.383	-0.321	-0.233	-0.143	-0.048	0.044
2'	-0.197	0.292	-0.325	-0.309	-0.287	-0.255	-0.214	-0.155	-0.096	-0.032	0.030
1'	-0.099	0.146	-0.163	-0.154	-0.144	-0.128	-0.107	-0.078	-0.048	-0.016	0.015
0'	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
+A'	0.000	0.000	0.000	0.897	2.557	3.618	5.180	5.317	4.784	3.221	1.767
-A'	0.000	0.000	0.000	12.713	11.917	12.793	12.894	12.628	11.453	9.274	7.346
+A	0.000	0.000	0.000	3.088	4.968	6.489	8.402	8.768	8.224	6.392	4.940
-A	3.548	5.839	6.502	18.884	17.659	17.905	17.174	15.730	13.364	9.914	7.346

Influence surface for Diagonals

member / load on.	8	9	11	12	13	14	15	16	17	18	19
0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	0	0	-0.015	0.024	0.020	0.016	0.010	0.004	-0.006	-0.016	-0.025
2	0	0	-0.031	0.048	0.029	0.031	0.020	0.007	-0.011	-0.031	-0.050
3	0	0	-0.047	0.072	0.059	0.047	0.030	0.011	-0.017	-0.047	-0.074
4	0	0	-0.063	0.096	0.078	0.062	0.040	0.015	-0.022	-0.062	-0.099
5	0	0	-0.078	0.120	0.098	0.078	0.050	0.019	-0.027	-0.078	-0.124
6	0	0	-0.094	0.144	0.114	0.093	0.060	0.022	-0.033	-0.093	-0.149
7	0	0	-0.109	0.168	0.133	0.108	0.070	0.026	-0.037	-0.108	-0.173
8	0	0	-0.125	0.192	0.156	0.124	0.080	0.030	-0.044	-0.124	-0.198
9	0	0	-0.062	0.096	0.078	0.062	0.040	0.015	-0.022	-0.062	-0.099
10	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
11	0	0	0.062	-0.096	-0.078	-0.062	-0.040	-0.015	0.022	0.062	0.099
12	0	0	0.125	-0.202	-0.164	-0.134	-0.089	-0.038	0.038	0.119	0.196
13	0	0	0.187	-0.301	-0.244	-0.198	-0.132	-0.055	0.058	0.179	0.294
14	0	0	0.250	-0.387	-0.312	-0.251	-0.163	-0.057	0.088	0.245	0.395
15	0	0	0.312	-0.459	-0.366	-0.289	-0.180	-0.054	0.122	0.319	0.497
16	0	0	0.374	-0.510	-0400	-0308	-0.178	-0.030	0.182	0.401	0.603
17	0	0	0.437	-0.527	-0.402	-0.295	-0.144	+0.024	0.261	0.500	0.714
18	0	0	0.499	-0.506	-0.369	-0.244	-0.076	0.112	0.366	0.615	0.832
19	0	0	0.562	-0.436	-0.290	-0.148	+0.039	0.242	0.506	0.754	0.958
18'	0	0	0.624	-0.339	-0.183	-0.025	0.178	0.396	0.666	0.905	-0.961
17'	0	0	0.687	-0.194	-0.030	0.144	0.367	0.592	0.860	-0.906	-0.854
16'	0	0	0.749	-0.011	0.158	0.349	0.584	0.822	-0.799	-0.804	-0.740
15'	0	0	0.811	0.206	0.378	0.588	0.836	-0.660	-0.696	-0.685	-0.623
14'	0	0	0.874	0.444	0.618	0.846	-0.478	-0.546	-0.570	-0.558	-0.501
13'	0	0	0.936	0.696	0.872	-0.311	-0.368	-0.419	-0.435	-0.413	-0.378
12'	0	0	0.990	0.961	-0.150	-0.209	-0.246	-0.280	-0.291	-0.283	-0.252
11'	0	0	1.061	0.020	-0.071	-0.100	-0.119	-0.126	-0.143	-0.139	-0.125
10'	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9'	0.000	0.967	-0.231	-0.020	0.071	0.100	0.119	0.126	0.143	0.139	0.125
8'	1.139	0.548	-0.462	-0.040	0.142	0.200	0.238	0.252	0.286	0.278	0.250
7'	0.997	0.480	-0.404	-0.035	0.124	0.175	0.208	0.220	0.250	0.243	0.219
6'	0.854	0.411	-0.347	-0.030	0.106	0.150	0.178	0.189	0.214	0.208	0.187
5'	0.712	0.343	-0.289	-0.025	0.089	0.125	0.149	0.158	0.179	0.174	0.156
4'	0.570	0.274	-0.231	-0.020	0.071	0.100	0.119	0.126	0.143	0.139	0.125
3'	0.427	0.206	-0.173	-0.015	0.053	0.075	0.089	0.094	0.107	0.104	0.094
2	0.285	0.137	-0.116	-0.010	0.035	0.050	0.059	0.063	0.072	0.070	0.062
1'	0.142	0.069	-0.058	-0.005	0.018	0.025	0.030	0.031	0.036	0.035	0.031
0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

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CALCULATIONS FOR

Design of Shirahige-Bashi

+A'	0.000	0.000	9.540	2.327	2.026	1.927	2.004	2.188	3.169	4.099	3.588
-A'	0.000	0.000	0.000	3.968	3.059	2.574	2.213	2.280	2.934	3.788	4.434
+A	5.126	3.435	9.540	3.267	3.500	3.548	3.593	3.596	4.599	5.489	5.837
-A	0.000	0.000	2.935	4.168	3.059	2.574	2.213	2.280	3.153	4.409	5.425

Influence surface for Verticals

load on \ member	8	9	10	11	12	13	14	15	16	17	18
0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	0	0	-0.139	-0.009	-0.011	-0.007	-0.003	0.001	0.005	0.009	0.013
2	0	0	-0.278	-0.018	-0.023	-0.014	-0.007	0.002	0.011	0.019	0.025
3	0	0	-0.416	-0.027	-0.033	-0.021	-0.010	0.002	0.016	0.028	0.034
4	0	0	-0.555	-0.036	-0.045	-0.028	-0.013	0.003	0.021	0.037	0.050
5	0	0	-0.693	-0.045	-0.056	-0.035	-0.016	0.004	0.026	0.046	0.063
6	0	0	-0.832	-0.054	-0.068	-0.042	-0.020	0.005	0.032	0.056	0.075
7	0	0	-0.970	-0.063	-0.079	-0.049	-0.023	0.005	0.037	0.065	0.088
8	0	0	-1.111	-0.072	-0.090	-0.056	-0.026	0.006	0.042	0.074	0.100
9	0	0	-1.056	-0.036	-0.045	-0.026	-0.013	0.003	0.021	0.037	0.050
10	0.000	0.000	-1.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
11	0	0	-0.944	0.036	0.045	0.026	0.013	-0.003	-0.021	-0.037	-0.050
12	0	0	-0.889	0.079	0.098	0.057	0.030	-0.003	-0.041	-0.054	-0.102
13	0	0	-0.833	0.117	0.146	0.084	0.044	-0.006	-0.062	-0.111	-0.154
14	0	0	-0.778	0.148	0.184	0.106	0.053	-0.012	-0.085	-0.149	-0.203
15	0	0	-0.722	0.178	0.212	0.118	0.055	-0.023	-0.110	-0.187	-0.250
16	0	0	-0.667	0.179	0.227	0.121	0.048	-0.040	-0.139	-0.224	-0.293
17	0	0	-0.611	0.169	0.218	0.105	0.027	-0.068	-0.174	-0.263	-0.331
18	0	0	-0.556	0.135	0.192	0.068	-0.011	-0.109	-0.217	-0.303	-0.365
19	0	0	-0.500	0.072	0.113	0.000	-0.070	-0.165	-0.267	-0.343	-0.390
18'	0	0	-0.444	-0.007	0.025	0.070	-0.141	-0.229	-0.323	-0.384	0.589
17'	0	0	-0.389	-0.114	-0.097	-0.170	-0.233	-0.308	-0.385	-0.574	0.503
16'	0	0	-0.333	-0.245	-0.245	-0.328	-0.324	-0.401	0.544	0.494	0.422
15'	0	0	-0.278	-0.397	-0.417	-0.433	-0.466	0.496	0.459	0.412	0.347
14'	0	0	-0.222	-0.560	-0.603	-0.583	0.402	0.403	0.370	0.330	0.274
13'	0	0	-0.167	-0.732	-0.793	-0.258	0.306	0.305	0.279	0.248	0.204
12'	0	0	-0.111	-0.911	-0.005	0.173	0.204	0.205	0.187	0.165	0.136
11'	0	0	-0.056	-0.096	-0.006	0.084	0.100	0.101	0.093	0.083	0.069
10'	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
9'	0	0	0.056	0.096	0.006	-0.084	-0.100	-0.101	-0.093	-0.083	-0.069
8'	0	-0.904	0.111	0.192	0.012	-0.168	-0.200	-0.202	-0.186	-0.166	-0.138
7'	0	-0.791	0.097	0.168	0.011	-0.147	-0.175	-0.177	-0.163	-0.145	-0.121
6'	0	-0.678	0.083	0.144	0.009	-0.126	-0.150	-0.152	-0.140	-0.125	-0.104
5'	0	-0.565	0.069	0.120	0.008	-0.105	-0.125	-0.126	-0.116	-0.104	-0.086
4'	0	-0.452	0.055	0.096	0.006	-0.084	-0.100	-0.101	-0.093	-0.083	-0.069
3'	0	-0.339	0.042	0.072	0.005	-0.063	-0.075	-0.076	-0.070	-0.062	-0.052
2'	0	-0.226	0.028	0.048	0.003	-0.042	-0.050	-0.051	-0.047	-0.042	-0.035
1'	0	-0.113	0.014	0.024	0.002	-0.021	-0.025	-0.025	-0.023	-0.021	-0.017
0'	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
+A'	0.000	0.000	0.000	1.113	1.460	1.200	1.282	1.510	1.932	2.306	2.544
-A'	0.000	0.000	7.556	3.062	2.166	1.584	1.225	1.367	1.824	2.055	2.138
+A	0.000	0.000	0.555	2.073	1.522	1.200	1.282	1.541	2.163	2.677	3.042
-A	0.000	4.068	12.550	3.422	2.616	2.702	2.356	2.378	2.755	2.886	2.829

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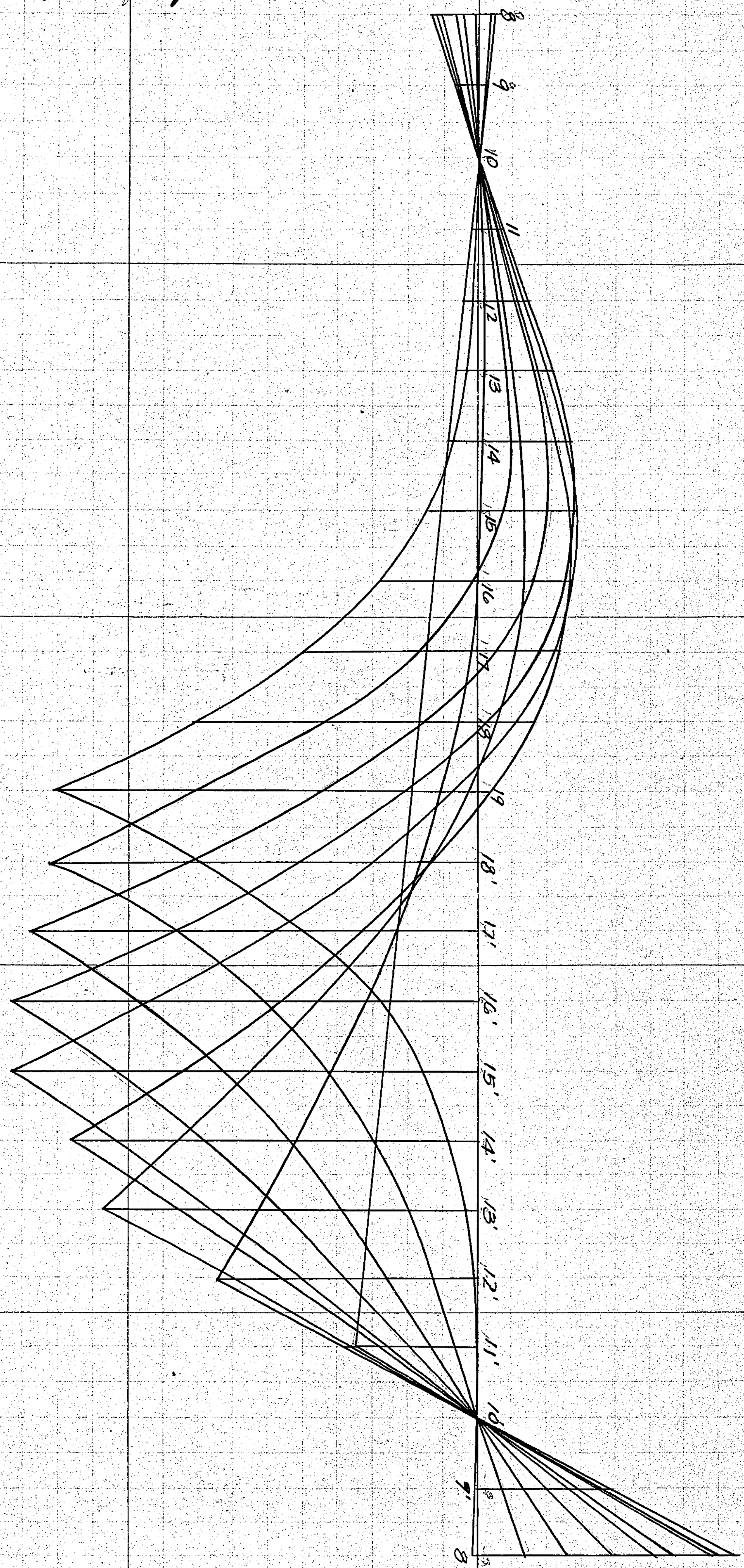
CALCULATIONS FOR

Design of Shirahige Bashi

Influence Surface for upper chords



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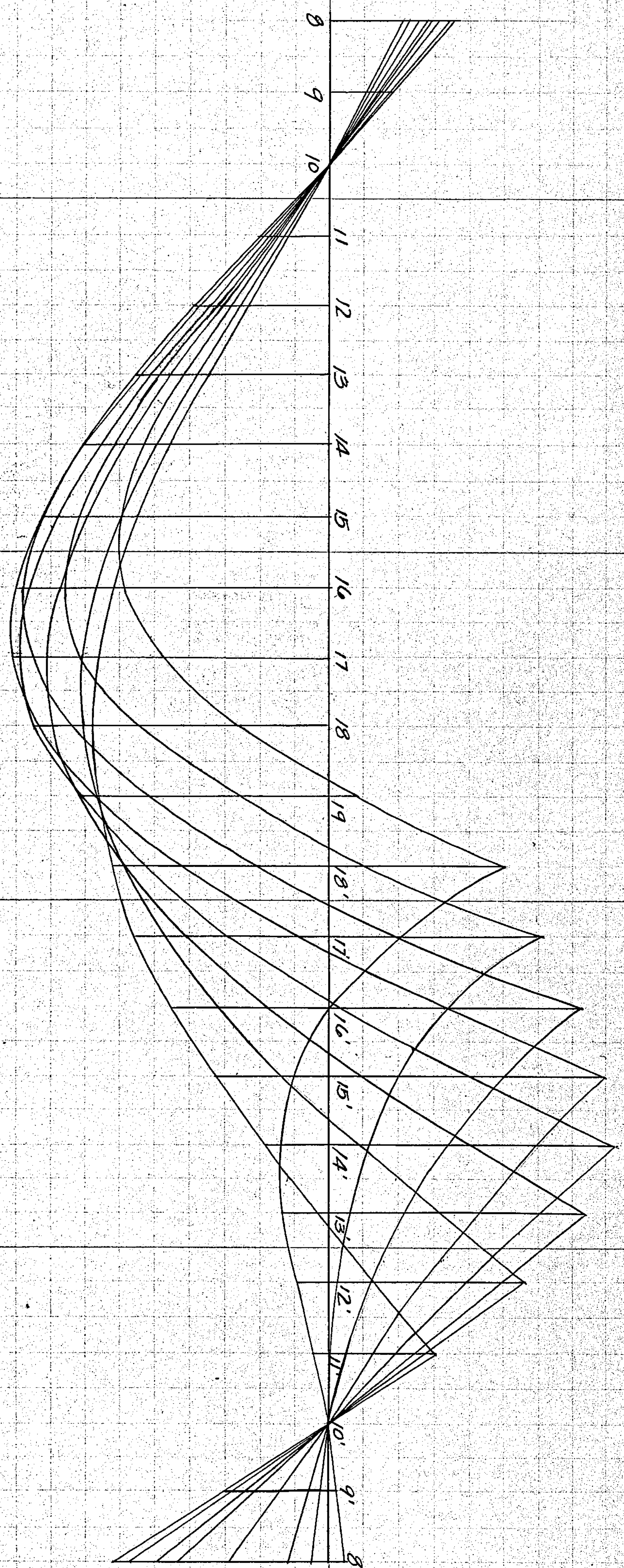
CALCULATIONS FOR

Design of Shirahige-Bashi

Influence Surface for Lower chords



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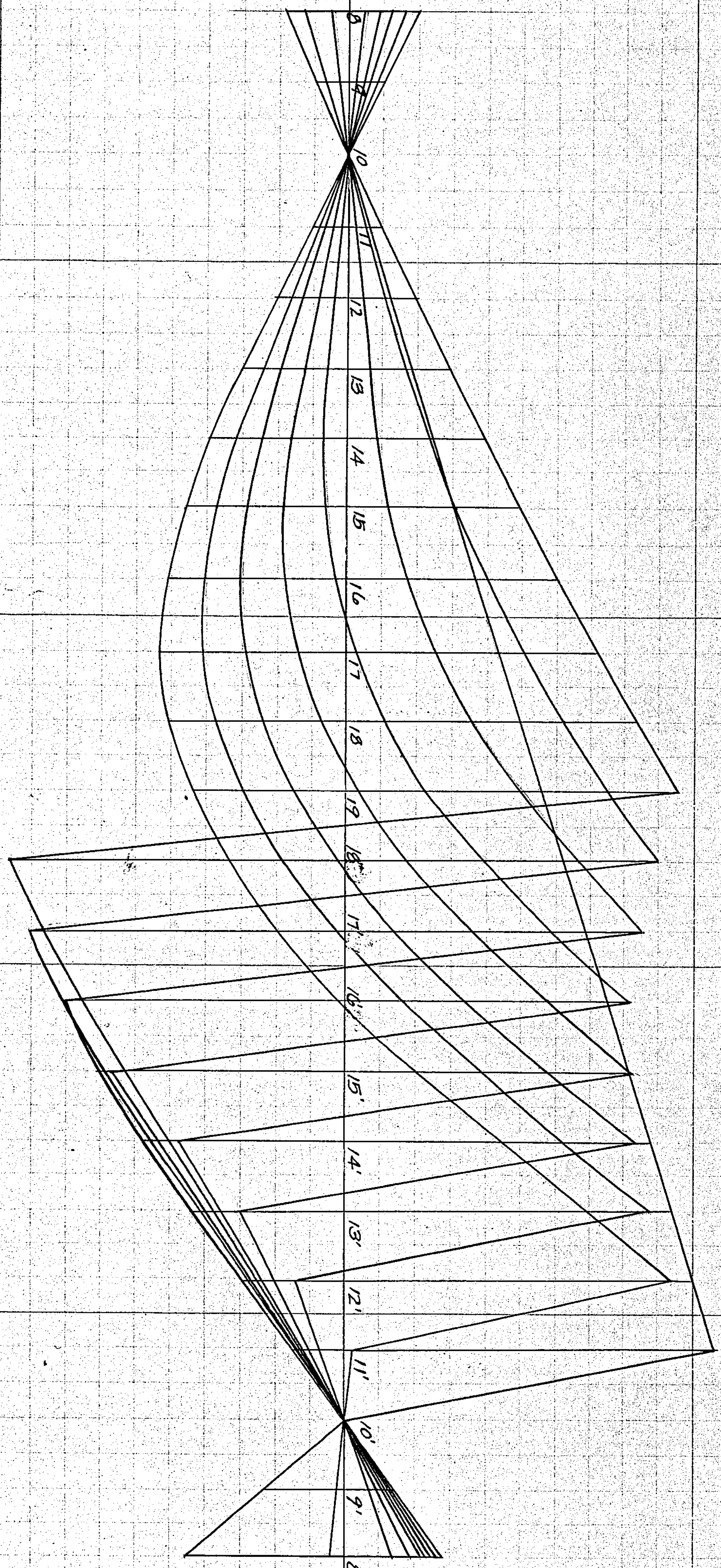
CALCULATIONS FOR

Design of Shirahige-Bashi

Influence Surface for Diagonals



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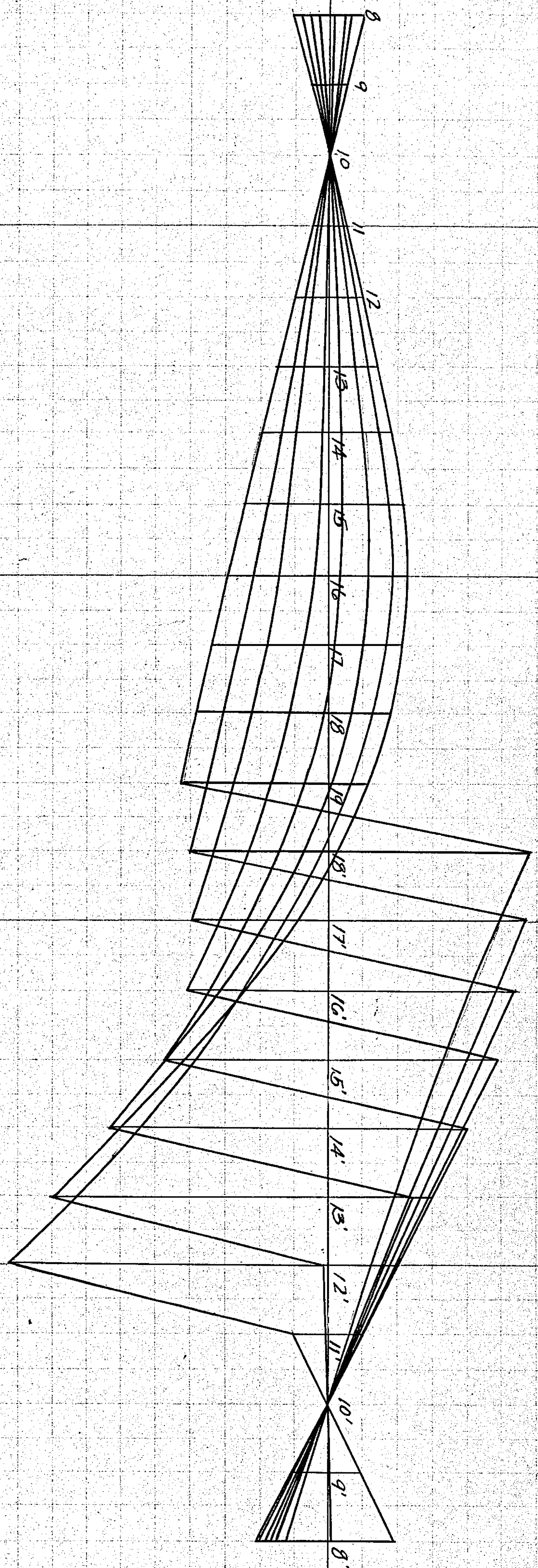
CALCULATIONS FOR

Design of Shirahige-Bashi

Influence Surface for Verticals



CALCULATIONS FOR

Design of Shirahige-Bashi. (1)

Panel Dead Load on truss
Weight of Floor &c
Slab and track for Electric Ry 9' @ 208# = 1874
Highway slab and pavement 95" × 13.86 = 1315
Filler 0.6×0.66 @150 = 60
1375
Gutter 1.0×.81 @ 150 122
Concrete at curb line 0.5×1.46 @ 150 = 110
Extra weight for granite 4×.8 @10 = 3
113
Sidewalk slab. 60"×11.96 = 718
Filler 0.6×.21 @ 150 18
Under coping stone 50"×0.5 = 25
761
Coping stone 0.5×1.12 = 0.56
0.54×0.5 = 0.27
0.83 @ 160 = 133
Handrail 85
Water main Contents 300
Steel 100 } 500
protection 100
Conduit pipes for Electric wirings 100
600
Panel Concentration -- 5063# × 14.5 = 73500#

Dead Load metal for Side span
Total weight of steel 1.205.000#
Rivets. 94.000#
3% allowance 40.000
1.339.000 ÷ 232 = 5770# per lin ft of span

Panel Concentration for one truss = $\frac{5770}{2}$ × 14.5 = 41.800#
weight of floor 73500
114300# call this 115000#

Dead Load metal for center span
Total weight of steel 2.600.000 Wt of Cantilever portion assumed same as
Rivets - 200.000 side span 5770×58 = 336.000#
5% allowance 140.000
2940.000#
Less Cantilever 336.000
2604.000 ÷ 261. 2660 = 9960# per lin. ft.
For one truss 9960 ÷ 2 = 4980# per lin ft.

Panel concentration of metal 4980 × 14.5 = 72500
Wt of floor 73500
146000 call this 150.000#.

Live Load
Panel load on side span
Uniform live load on Highway = $\frac{120.000}{170+\frac{116}{43}}$ = $\frac{120.000}{170+35}$ = 585 kg/m² or 117#/□'
" " " " Sidewalk = $\frac{100.000}{170+35}$ = 488 kg/m² = 98#/□'
Electric car loading 1500# per lin ft of span continuous loading on span assumed for 9.0' wide.

Design of Shira-hige-Bashi. 2

Electric car loading		1500 · 14.5	=	21.750
Uniform load on highway		117 · 15 · 14.5	=	25450
" " " sidewalk		98 · 9 · 14.5	=	12790
panel concentration.				59990# or say 60.000#

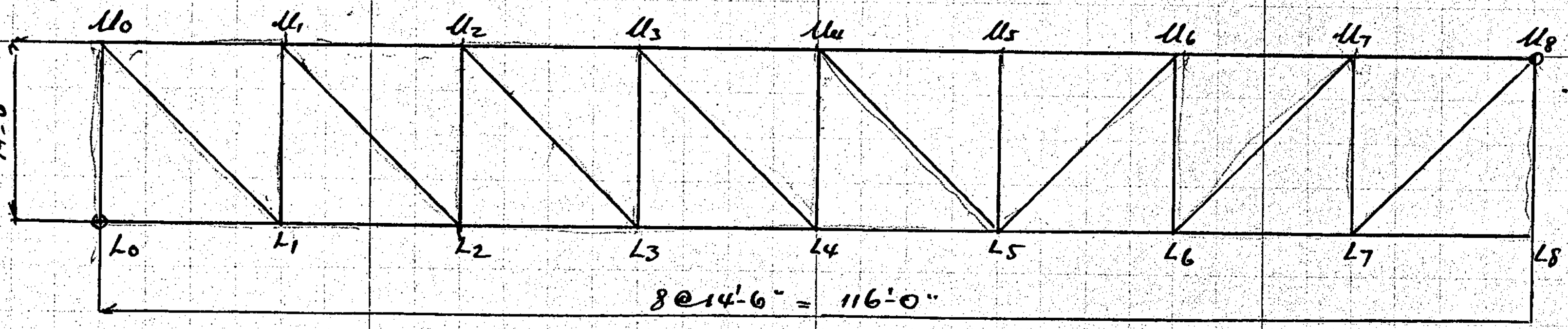
Panel load on centre span between piers

Uniform load on Highway $w = \frac{120.000}{170 + \frac{261}{33}} = \frac{120.000}{170+79} = 482\ ^{kg}/_{m^2} = 96\ ^{\#}/_{\square'}$

Uniform load on sidewalk $w = \frac{100.000}{170+79} = 400\ ^{kg}/_{m^2} = 80\ ^{\#}/_{\square'}$

Electric car loading		1500 × 14.5	=	21750
Uniform load on Highway		96 × 15 × 14.5	=	20880
" " " Sidewalk		80 × 9 × 14.5	=	10440
panel concentration				53070# or say 53000#

Design of side span   span length 8@14.5 = 116'-0"   Height = 14'6"



Dead Load panel concentration = 115000#
Live " " " = 60.000#

Stresses of upper chord

member	Coef. for influence surface	DL stress (115.000#)	L.L. stress (60.000#)	Total Stress	Section Required
				613.000 C	
U0-U1	-3.500	403.000# C	210.000 C	1050.000 C	43.8 □"
U1-U2	-6.000	690.000 C	360.000 C	1312.000 C	75.0 "
U2-U3	-7.500	862.000 C	450.000 C	1400.000 C	93.8 "
U3-U4	-8.000	920.000 C	480.000 C	1312.000 C	100.0 "
U4-U5	-7.500	862.000 C	450.000 C	1312.000 C	93.8 "
U5-U6	-7.500	862.000 C	450.000 C	1050.000 C	93.8 "
U6-U7	-6.000	690.000 C	360.000 C		75.0 "
U7-U8	-3.500	403.000 C	210.000 C	613.000 C	43.8 "

Stresses of Lower chord

member	Inf. surface	DL stress	LL stress	Total stress	section
L0-L1	+0.000	0.000	0	0	
L1-L2	+3.500	403.000 T	210.000 T	613.000	36.0 □" net
L2-L3	+6.000	690.000 T	360.000 T	1050.000	61.7 "
L3-L4	+7.500	862.000 T	450.000 T	1312.000	77.2 "
L4-L5	+8.000	920.000 T	480.000 T	1400.000	82.3 "
L5-L6	+6.000	690.000 T	360.000 T	1050.000	61.7 "
L6-L7	+3.500	403.000 T	210.000 T	613.000	36.0 "
L7-L8	+0.000	0.00	0	0	
L-					

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CALCULATIONS FOR

Design of Shira Hige-Bashi. 3

Stresses of Diagonals.

member	+	−	Total	DL stress	LL stress	Total stress		Section Reqd
U_0-L_1	4.948	0	4.948	570.000 T	296900 T	866900 T		51.00 □" net
U_1-L_2	3.712	0.177	3.535	406.000 T	222700 T	628700 T		37.00 "
U_2-L_3	2.652	0.531	2.121	244.000 T	159100 T	403100 T		23.70 "
U_3-L_4	1.768	1.061	0.707	81500 T	106100 T (−63700)	187600 T	17800 C	11.20 "
U_4-L_5	1.061	1.768	−0.707	81500 C	−106100 C (63700)	187600 C	17800 T	13.40 gross
U_6-L_5	2.652	0.531	2.121	244.000 T	159100 T	403100 T		23.70 □" net
U_7-L_6	3.712	0.177	3.535	406.000 T	222700 T	628700 T		37.00 "
U_8-L_7	4.948	0	4.948	570.000 T	296900 T	866900 T		51.00 "

Stresses of Verticals.

member	+	−	Total	DL stress	LL stress	Total stress	Section Reqd
U_0-L_0	0	3.500	−3.500	391.000 C	210.000 C	601.000 C	43.0 □" gr
U_1-L_1	0.125	2.625	−2.500	288.000 C	157500 C	445500 C	31.8 "
U_2-L_2	0.375	1.875	−1.500	172500 C	112500 C	285000 C	21.6 "
U_3-L_3	0.750	1.250	−0.500	57500 C	75000 C	132500 C	9.4 "
U_4-L_4	1.250	0.750	+0.500	57500 T	75000 T	132500 T	7.8 □" net
U_5-L_5	0	0	0	0	0	0	0
U_6-L_6	0.375	1.875	−1.500	172500 C	112500 C	285000 C	21.6 □" gross
U_7-L_7	0.125	2.625	−2.500	288000 C	157500 C	445500 C	31.8 "
U_8-L_8	0	3.500	−3.500	391.000 C	210000 C	601000 C	430 "

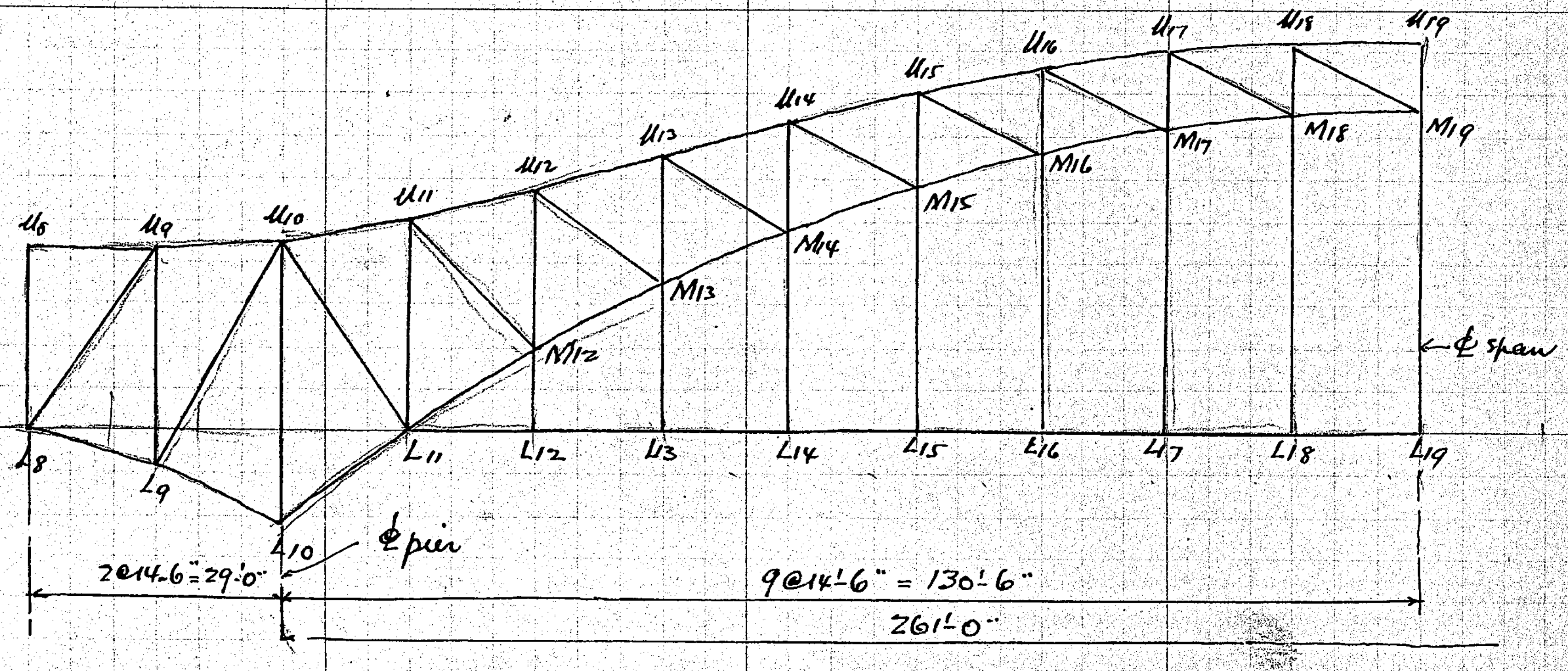
Load on pin on abutment

Dead Load	115.000 × 4	=	460.000
LL	60.000 × 4	=	240.000
			700.000
DL Live Extra loads beyond L_0 say			20.000
			720.000 #

Load on shoe

Dead Load	460.000	
Extra Dead Load beyond L_0	10.000	
weight of shoe	10.000	
		480.000
Live Load say		250.000
		730.000 #

Stresses and section of Center span with Cantilever arms



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Design of Shira Hige - Bashi. 4

Upper chord stresses and sections

DL. D2 LL

Member	+A	-A	Total	+A'	-A'	Total	DL.	DL.	DL. Total	L.L.	Total Stress	Design stress	SR.
	Total span			main span only			150.000	35000	Total	LL			
U8-U9	0.00	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
U9-U10	3.502	0.00	3.502	0.000	0.000	0.000	+525300	-122000	+403300	185600	588900	588900 T	34.6 nes
U10-U11	7.411	6.296	1.115	0.000	6.296	-6.296	+167300	-259000	-91700	392800	301100		
										(-333700)	-425400	575900 C	41.2 a"
U11-U12	7.655	7.250	0.405	0.144	7.159	-7.015	+60700	-259000	-198300	405700	207400		
										(-384300)	-582600	686300 C	49.0 a"
U12-U13	8.744	9.671	-0.927	1.553	9.060	-7.507	-139000	-230.000	369000	463400	94400		
										(-512600)	-881600	928800 C	66.4"
U13-U14	9.115	10.628	-1.513	3.284	9.446	-6.162	-227000	-163.000	390.000	483100	93100		
										(-563300)	-953300	999800 C	71.5
U14-U15	8.462	12.857	-4.395	3.240	11.555	-8.315	-659300	-136500	-795800	-681400	-1477200	1477200 C	105.5
U15-U16	7.718	13.169	-5.451	3.696	11.849	-8.153	-917700	-94500	-~~1012200~~ 912200	-698000	-~~1710200~~ 1610200	~~1710200~~ 1610200 C	115.7 ~~122.0~~
U16-U17	4.815	12.501	-7.686	2.064	11.430	-9.366	-1152900	-58800	-1211700	-662600	-1874300	1874300 C	134.0
U17-U18	2.241	10.956	-8.715	0.739	10.465	-9.726	-1307300	-35400	-1342700	-580700	-1923400	1923400 C	137.5
U18-U19	0.764	9.848	-9.084	0.000	9.848	-9.848	-1362600	-26800	-1389400	-521900	-1911400	1911400 C	136.7

Lower chord & Middle chord stresses and sections.

member	+A	-A	Total	+A'	-A'	Total	DL. 150000	DL. 35000	DL. Total	L.L.	Total stress	section
L8-L9	0.000	3.548	-3.548	0.000	0.000	0.000	-532200	+124000	-408200	-188000	-596200	42.6 a"
L9-L10	0.000	5.839	-5.839	0.000	0.000	0.000	-875800	+204000	-671800	-309500	-981300	70.1
L10-L11	0.000	6.502	-6.502	0.000	0.000	0.000	-975300	+228000	-747300	-344600	-1091900	78.0
L11-M12	3.088	18.884	-15.796	0.897	12.713	-11.816	-2369400	+139000	-2230400	-1000900	-3231300	231.0
M12-M13	4.968	17.659	-12.691	2.557	11.917	-9.360	-1903600	+116500	-1787100	-935900	-2723000	195.0
M13-M14	6.489	17.905	-11.416	3.618	12.793	-9.175	-1712400	+78500	-1633900	-949000	-2582900	184.0
M14-M15	8.402	17.174	-8.772	5.180	12.894	-7.714	-1315800	+37000	-1278800	-910200	-2189000	156.0
M15-M16	8.768	15.730	-6.962	5.317	12.628	-7.311	-1044300	-12200	-1056500	-833700	-1890200	135.0
M16-M17	8.224	13.364	-5.140	4.784	11.453	-6.669	-771.000	-53500	-824500	-708300	-1532800	109.0
M17-M18	6.392	9.914	-3.522	3.221	9.294	-6.053	-528300	-88500	-616800	-525400	-1142200	81.6
M18-M19	4.940	7.346	-2.406	1.767	7.346	-5.579	-360900	-111000	-471900	-389300	-861200	61.5

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CALCULATIONS FOR

Design of Shira Luge-Bashi:

Diagonal stresses and sections.

member	+A (Total span)	−A (Total span)	Total (Total span)	+A′ (main span only)	−A′ (main span only)	Total (main span only)	D.L. 150,000	D.L. 35,000	Total D.L. load	L.L.	Total stress	Design stress	section
L8-U9	5.125	0.000	5.125	0.000	0.000	0.000	768800	−180400	588400	260800	849200	849200T	50.0 □
L9-U10	3.435	0.000	3.435	0.000	0.000	0.000	515300	−120200	395100	182100	577200	577200T	33.9 ″
U10-L11	9.540	2.935	6.605	9.540	0.000	9.540	990800	102700	1093600	505600	1599100	1599100T	94.1 ″
U11-M12	3.267	4.168	−0.901	2.327	3.968	−1.641	−135200	−25900	−161100	−220900 +173200	−382.000 12100	388100C	27.7 □
U12-M13	3.500	3.059	0.441	2.026	3.059	−1.033	66200	−51600	14600	185500 −162100	200100 −147.500	273900T	16.1 □
U13-M14	3.548	2.574	0.974	1.927	2.574	−0.647	146100	−56700	89400	188.000 −136400	277400 −47000	300900T	17.7 ″
U14-M15	3.593	2.213	1.380	2.004	2.213	−0.209	207000	−55600	151400	190400	341800	341800T	20.1 ″
U15-M16	3.596	2.280	1.316	2.188	2.280	−0.092	197400	−49300	148100	190600	338700	338700T	20.0 ″
U16-M17	4.599	3.153	1.446	3.169	2.934	0.235	216900	−42400	174700	243700	418400	418400T	24.6 ″
U17-M18	5.489	4.409	1.080	4.099	3.788	0.311	162000	−26900	135100	290900 −233700	426000 −98600	475300	28.0 ″
U18-M19	5.837	5.425	0.412	3.588	4.434	−0.846	61800	−44.000	17800	309400 −287.500	327200 −269700	462100 404600	27.2 □

Stresses and Required Sections of Verticals.

member	+A	−A	Total	+A′	−A′	Total	D.L. 150,000	D.L. 35,000	Total D.L.	L.L.	Total stress	Design stress	Section
U8-L8	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	− 0.000	0	
U9-L9	0.000	4.068	−4.068	0.000	0.000	0.000	−610200	142400	−467800	−215600	−683400	683400C	51.7 □
U10-L10	0.555	12.550	−11.995	0.000	7.556	−7.556	−1799300	155400	−1643900	−665200	−2309100	2309100C	165.0
U11-L11	2.073	3.422	−1.349	1.113	3.062	−1.949	−202400	−21.000	−181400	−181400	−362800	362800C	25.9
U12-M12	1.522	2.616	−1.094	1.460	2.166	−0.706	−164100	13600	−150500	−138600	−289100	289100C	20.7
U13-M13	1.200	2.702	−1.502	1.200	1.584	−0.384	−225300	39100	−186200	−143200	−329400	329400C	23.5
U14-M14	1.282	2.356	−1.074	1.282	1.225	+0.057	−161100	39600	−121500	−124900	−246400	246400C	17.6
U15-M15	1.541	2.378	−0.837	1.510	1.367	+0.143	−125600	34300	−91300	−126000	−217300	217300C	15.5
U16-M16	2.163	2.755	−0.592	1.932	1.824	0.108	−88800	24500	−64300	−146000 114600	−210300 50300	235600C	16.7
U17-M17	2.677	2.886	−0.209	2.306	2.055	0.251	−31400	16100	−15300	−153000 141900	−168300 126600	231600C	16.6
U18-M18	3.042	2.829	0.213	2.544	2.138	0.406	32000	6800	25200	−149900 161200	−124700 186400	248800T	14.7 □

Design of Shirahige-Bashi. 6

Ties

Member	+A	-A	Total	+A'	-A'	Total	DL 150.000	DL 35000	Total DL	LL	Total Stress	SR
	Total span											
L11-L11	15.308	4.142	11.126	15.308	0.000	15.308	1668.900	146400	1815.300	811.300	2626.600	154.2□" net

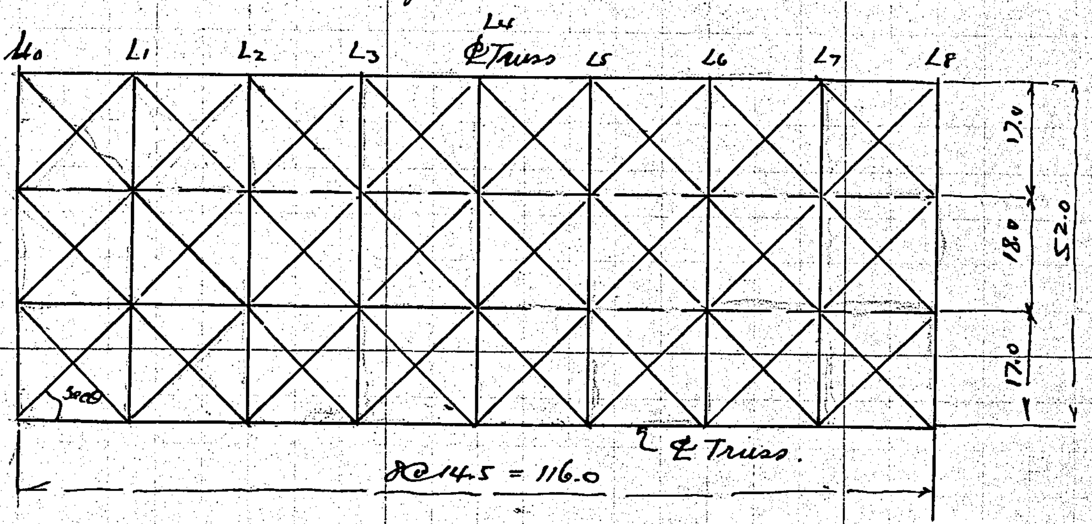
Stress of Hanger and Required Section

Dead Load assumed 150.000
Live Load 84.000
234000# ÷ 17000 = 13.75□" net

Load on Pin. Dead Load between piers 150.000 × 9.5 = 1425.000
" " side span 115.000 × 5.5 = 632.000
2057.000#
Live Load say 50.000 × 15.0 = 750.000
2807.000

Load on one shoe
Dead + Live Load 2807.000
Dead Load shoe say 13.000
2820.000# assumed.

Bottom Lateral Bracings for side span



Truss panel load due to Earthquake arbitrary assumed 80.000# for side span
Coefficient for Earthquake 1/3 unit stress 80% extra over normal allowable unit stress.

Panel load as normal stress = $\frac{80.000}{3 \times 1.8}$ = 14.800#

Reaction = 2 × 14.800 × 3.5 = 103.600#

Secθ = 1.32 bracing assumed as tension member neglecting compression

				SR	No of Rivet 7/8
Stress in diagonal	(L0-L1)	$\frac{103600}{3} \times 1.32$	= 45570#	2.68□" net	8
Stress in "	(L1-L2)	$\frac{74000}{3} \times 1.32$	= 32600#	1.92	6
	(L2-L3)	$\frac{44400}{3} \times 1.32$	= 19500#	1.15	6
	(L3-L4)	$\frac{14800}{3} \times 1.32$	= 6500#	0.38	6

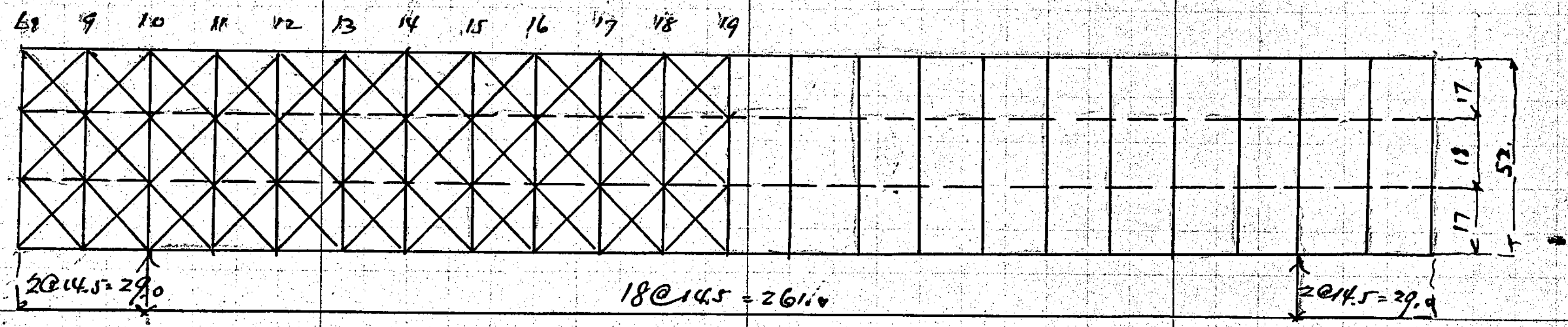
For L0-L1 2∠s 5×3½×3/8 riveted back to back × 6.10 − 0.54 = 5.56□" net
use same section for other panels.

CALCULATIONS FOR

7

Design of Shira Hige-Bashi.

Bottom Lateral Bracings for center span including cantilever arms.

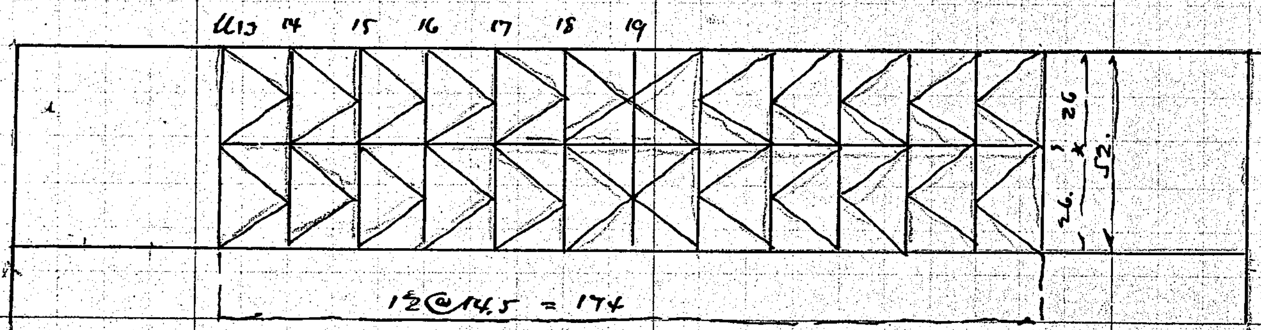


Panel dead load due to Earthquake assumed 100.000#
Horizontal panel load assumed $\frac{100.000}{3 \times 1.8}$ = 18,500# as normal stress
for two trusses 2@18.5 00 = 37000#

Panel concentration at L8 Reaction from side span at L8 = 103600
37000
140600

	Shear		No of member		sec θ	Diagonal stress	Section Req'd		
L8-L9	140600	÷	3	×	1.325	61600 T	3.63□" net	2L$\underline{s}$ 5×3½×3/8	= 6.10
L9-L10	177600	÷	6	×	1.28	37800 T or C	2.70 gr	4L$\underline{s}$ 4×3½×3/8	= 10.68
L10-L11	314000	÷	6	×	1.18	61800 T or C	4.42 gr	4L$\underline{s}$ 4×3½×3/8	= 10.68
L11-L12	277500	÷	3	×	1.32	122100 T	7.16 net	2L$\underline{s}$ 6×6×3/8	= 8.72 − 1.5 = 7.22
L12-L13	240500		"		"	105800 T	6.20 "	"	"
L13-L14	203500		"		"	89500 T	5.26 "	"	"
L14-L15	166500		"		"	73300 T	4.30 "	"	"
L15-L16	129500		"		"	57000 T	3.35 "	2L$\underline{s}$ 5×5×3/8	= 7.22
L16-L17	92500		"		"	40800 T	2.40 "	2L$\underline{s}$ 5×5×3/8	= 7.22
L17-L18	55500		"		"	24400 T	1.43 "	2L$\underline{s}$ 5×3½×3/8	= 6.10
L18-L19	18500		"		"	8150 T	0.48 "	"	= 6.10

Top Lateral Bracing



Wind panel load assumed 3100# for both trusses 2×3100 = 6200#
End reaction = 5.5 × 6200 = 34100#
Stress in diagonal = 34100 × 1.5 = 51150#
in one diagonal = 51150 ÷ 4 = 12800# C or T
Length of diagonal 19'4" section assumed 4L$\underline{s}$ 4×3×3/8 = 9.92□" r = 1.92
$l/r = \frac{19.33 \times 12}{1.92} = 121$ allowable stress = 21300 (1 − 0.0055 × 121) = 7140#/□"

For longitudinal strut use 4L$\underline{s}$ 4×3×3/8 = 9.92

For sway bracing. top 4L$\underline{s}$ 5×3½×3/8 = 12.20□"
bottom 2L$\underline{s}$ 6×6×3/8 = 8.72□" } braced with angle diagonals of 2L$\underline{s}$ 3½×3½×3/8
For End portal 4L$\underline{s}$ 6×6×3/8 = 17.44□" flange
and 1 web of 36×7/16 at ℄ of bridge

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CALCULATIONS FOR

8

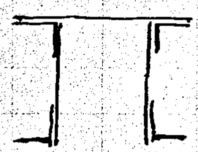
Design of Shirahige - Bashi.

Section of truss.
Side span

Upper chord — U_0-U_1: S = 613.000 C, SR = 43.80° — U_1-U_2: S = 1050.000#, SR = 75.0°

Section U_0-U_1 + U_1-U_2

1 cov. Pl. 33 × 5/8	=	~~33.76~~ 20.63
4 Ls 6.6. 1/2	=	23.00
2 webs 27 × 5/8	=	33.76
		77.39

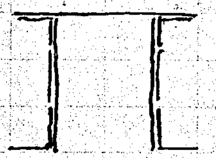


U_2-U_3. S = 1312000# SR = 93.80°

2 webs 27 × 5/8	=	33.76
4 Ls 6.6 × 5/8	=	28.44
1 Cov Pl 33 × 5/8	=	20.63
2 Pls. 15 × 3/8	=	11.25
		94.08 °"

U_3-U_4 S = 1400.000# SR = 100.0°"

2 webs 27 × 5/8	=	33.76
4 Ls 6.6. 5/8	=	28.44
1 Cov. Pl 33 × 5/8	=	20.63
2 Pls. 15 × 5/8	=	18.75
		101.58 °"



U_4-U_5-U_6. S = 1312000# SR = 93.8
Use section same as for U_2-U_3

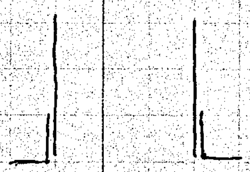
U_6-U_7 S = 1050.000# SR = 75.0°" — same as U_1-U_2

U_7-U_8 S = 643.000# SR = 43.80° — same as U_0-U_1

Bottom chord

L_0-L_1 S = 0 — L_1-L_2 S = 613.000# SR = 36.0°" net

2 webs 25 × 5/8	=	31.26 - 5.00	=	26.26
2 Ls 6.6. 1/2	=	11.50 - 2.00	=	9.50
		42.76		35.76 °" net



L_6-L_7-L_8 same as for L_0-L_1-L_2

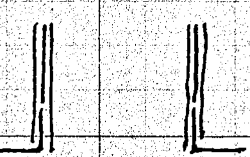
L_2-L_3 S = 1050.000# SR = 61.7°" net

2 webs 25 × 5/8	=	31.26 - 5.0	=	26.26
2 Ls 6.6. 3/4	=	16.88 - 3.0	=	13.88
2 Pls. 19 × 3/4	=	28.50 - 4.5	=	24.00
		76.64		64.14 °" net

L_5-L_6 same as L_2-L_3

L_3-L_4 S = 1312.000# SR = 77.2°" net
L_4-L_5 S = 1400.000# SR = 82.3°" net

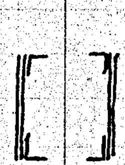
2 webs 25 × 5/8	=	31.26 - 5.0	=	26.26
2 Ls 6.6. 3/4	=	16.88 - 3.0	=	13.88
2 Pls. 19 × 3/4	=	28.50 - 4.5	=	24.00
2 Pls 24 1/4 × 1/2	=	24.25 - 4.00	=	20.25
		100.89 °"		84.39 °" net



Diagonals.

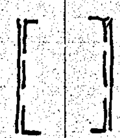
U_0-L_1 + L_7-U_8 S = 866900# T SR = 51.0°" net.

4 Pls. 18 × 5/8	=	45.00 - 10.0	=	35.0
4 Ls 6.6. 1/2	=	19.00 - ~~2.0~~	=	~~18.0~~ 17
		64.00 °"		~~53.0~~ 52.0 °" net



U_1-L_2 + L_6-U_7 S = 628700# T SR = 37.0 °" net

2 Pls. 18 × 5/8	=	22.50 - 5.0	=	17.50
4 Ls 6.4. 1/2	=	19.00 - 2.0	=	17.00
2 Pls. 6 × 1/2	=	6.00 - 2.0	=	4.00
				38.50 °" net



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CALCULATIONS FOR

Design of Shinohige-Bashi

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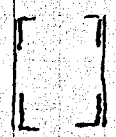
Diagonals

U2-L3 + L5-U6 S = 403100 # T SR = 23.7 □" net



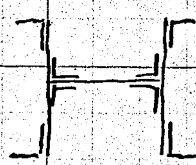
4Ls 6×4×3/8 = 14.44 - 3.00 = 11.44
2Pls. 18×1/2 = 18.00 - 4.00 = 14.00
32.44 □" 25.44 □" net

U3-L4 S = 187600 # T SR = 11.20 □" net
4Ls ~~6~~ 4×3/8 = ~~14~~ 11.44 - 3.00 = 8.44
2Pls. 18×3/8 = 13.50 - 3.00 = 10.50
24.94 ~~2~~ 18.94

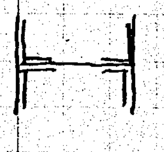
U4-L5 S = 187600 # C SR = 13.40 □" gross
4Ls 4×4×3/8 = 11.44
2Pls. 18×3/8 = 13.50
24.94 □"

Verticals

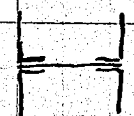
End Post U0-L0 S = 601000 # C SR = 43.0
2Pls. 24×5/8 = 30.00
4Ls 4×4×1/2 = 15.00
4Ls 4×4×1/2 = 15.00
1 web 17½×7/16 = 7.66
67.66 □"



U1-L1 S = 445500 C SR = 31.80 □" U7-L7 same as U1-L1
2Pls. 15×5/8 = 18.75
4Ls 6×4×1/2 = 19.00
1 web 16½×7/16 = 7.22
44.97



U2-L2 + U6-L6 S = 285000 # C SR = 21.6 □"
4Ls 6×4×1/2 = 19.00
1 web 18×7/16 = 7.88
26.88 □"



U3-L3 S = 132500 # C SR = 10.0
4Ls 6×4×3/8 = 11.44
1 web 18×7/16 = 7.88
22.32 □"

U4-L4 S = 132500 # T SR 7.80 □" net
4Ls 6×4×3/8 = 11.44 - 3.0 = 8.44
1 web 18×7/16 = 7.88 - .62 = 7.26
22.32 15.70 □" net.

U5-L5 S = 0.0 Use same section as above

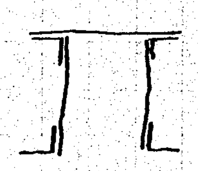
Center Span
upper chord
U8-U9 S = 0.
U9-U10 S = 588900 # T SR 34.60 □" net
U10-U11 S = 575900 C SR = 41.2 □" gross
U11-U12 S = 686300 C SR = 49.0
2 webs 27×5/8 = 33.76
4Ls 6×6×1/2 = 23.00
1 cov Pl. 33×1/2 = 16.50
73.26 □"

Design of Shirahige-Bashi

(10)

Upper Chord.

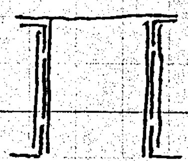
U12-U13 S = 928800 C SR = 66.4 □"
4Ls 6·6·½ = 23.00
2 webs 27·⅝ = 33.76
1 Cov. Pl 33·⅝ = 20.63
77.39 □"



U13-U14 S = 999800# C SR = 71.5 □"
4Ls 6·6·½ = 23.00
2 webs. 27·⅝ = 33.76
1 Cov. Pl. 33·¾ = 24.75
81.51 □"

U14-U15 S = 1477200# C SR = 105.5
4Ls 6·6·⅝ = 28.44
2 webs 27·⅝ = 33.76
1 Cov Pl. 33·¾ = 24.75
2 Side Pls. 15·⅝ = 18.75
105.70

U15-U16. S = 1610200# C SR = 115.2
Same as above 105.70
add 2 Pls. 26·⅜ 19.50
125.20 □"

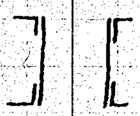


U16-U17 S = 1874300# C SR = 134.0
U17-U18 S = 1923400# C SR = 137.5
U18-U19 S = 1911400# C SR = 136.7

Same as U14-U15 105.70
add 2 Pls 26·⅝ = 32.50
138.20 □" gross.

Lower chord and middle chord.

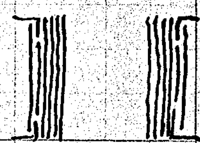
L8-L9 S = 596200# C SR = 42.60"
2 Pls. 25·⅝ = 31.26
4Ls 6·6·½ = 23.00
54.26 □"



L9-L10 S = 981300# C SR = 70.10 □"
2 Pls. 25·⅝ = 31.26
4Ls 6·6·⅝ = 28.44
2 Pls. 13·⅝ = 16.25
75.95 □"

L10-L11 S = 1091900# C SR = 78.0
4Ls 6·6·⅝ = 28.44
2 webs 27·¾ = 40.50
2 Pls. 15·⅝ = 18.75
87.69 □"

L11-M12 S = 3231300# C SR = 231.00
4Ls 6·6·¾ = 33.76
8 Pls. 27·¾ = 162.00
2 Pls 15·¾ = 22.50
2 Pls. 26·½ = 26.00
244.26 □"



M12-M13 S = 2723000# SR = 195.0 □"
4Ls 6·6·¾ = 33.76
8 Pls. 27·¾ = 162.00
195.76 □"

M13-M14 S = 2582900# C SR = 184.0
Same as M12-M13 195.76

M14-M15 S = 2189000# C SR = 156.0 □"
4Ls 6·6·¾ = 33.76
6 Pls. 27·¾ = 121.50
155.26

M15-M16 S = 1890200# C SR = 135.0
4Ls 6·6·¾ = 33.76
4 Pls. 27·¾ = 81.00
2 Pls. 15·¾ = 22.50
137.26 □"

M16-M17 S = 1532800# C SR = 109.0
4Ls 6·6·¾ = 33.76
4 Pls. 27·¾ = 81.00
114.76

M17-M18 S = ~~861200~~ 1142200# C SR = ~~61.5~~ 81.6
4Ls 6·6·¾ = 33.76
2 Pls. 27·¾ = 40.50
2 Pls. 15·¾ = 22.50
96.76 □"

M18-M19 S = 861200# C SR = 61.5
4Ls 6·6·¾ = 33.76
2 Pls. 27·¾ = 40.50
74.26 □"

Design of Shirahige-Bashi.

Diagonals.

M9L8 S = 849200# T SR = 50.0 □" net
4 Pls. 18 × 5/8 = 45.00 − 10.0 = 35.0
4 Ls 6 × 4 × 1/2 = 19.00 − 2.0 = 17.0
64.00 □" 52.0 □" net

L9-M10 S = 577200# T SR = 33.90" net
4 Ls 4 × 4 × 1/2 = 15.00 − 4.0 = 11.00
2 Pls. 24 × 5/8 = 30.00 − 5.0 = 25.00
45.00 36.00 □" net

M10-L11 S = 1599100# T SR = 94.100" net
4 Ls 6 × 4 × 5/8 = 23.44 − 5.00 = 18.44
6 Pls. 24 × 5/8 = 90.00 − 15. = 75.00
2 Pls. 12 × 5/8 = 15.00 − 5.0 = 10.00
103.44 □" net

M11-M12 S = 388100# C SR = 27.7 □"
4 Ls 6 × 4 × 3/8 = 14.44
2 Pls. 18 × 3/8 = 13.50
27.94

M12-M13 S = 273850# T SR = 16.1 □" net
4 Ls 6 × 4 × 3/8 = 14.44 − 3.00 = 11.44
2 Pls. 18 × 3/8 = 13.50 − 3.00 = 10.50
27.94 21.94 □" net

M13-M14 S = 300900# T SR = 17.7 □" net Same as M12-M13

M14-M15 S = 341800# T SR = 20.1 □" net Same as M12-M13

M15-M16 S = 338700# T SR = 20.0 net "

M16-M17 S = 418400# T SR = 24.6 □" net
4 Ls 6 × 4 × 1/2 = 19.00 − 4.00 = 15.00
2 Pls. 17 × 1/2 = 17.00 − 4.00 = 13.00
36.00 28.00 net

M17-M18 S = 475300# T SR = 28.00
4 Ls 6 × 4 × 1/2 = 19.00 − 4.00 = 15.00
2 Pls. 15 × 5/8 = 18.75 − 3.75 = 15.00
37.75 30.00 □" net

M18-M19 S = 462100# T SR = 27.20 net Same as M17-M18

Verticals

M8-L8 S = 601000# C SR = 43.0
4 Ls 6 × 4 × 1/2 = 19.00
1 Pl. 14½ × 7/16 = 6.34
2 Pls. 12½ × 1/2 = 12.50
2 Pls. 15 × 1/2 = 15.00
2 Pls. 15 × 5/8 = 18.75
71.59 □"

M9-L9 S = 683400# C SR = 51.7 □"
4 Ls 6 × 4 × 1/2 = 19.00
4 Pls. 15 × 5/8 = 37.50
1 web 15¼ × 7/16 = 6.68
63.18 □"

M10-L10 S = DL 1643900# C SR = 165.0
LL 665200
2309100# C
6 Pls. 27 × 5/8 = 101.25
4 Ls 6 × 6 × 5/8 = 28.44
2 Pls. 15 × 5/8 = 18.75
4 Ls 6 × 4 × 1/2 = 19.00
1 Pl. 15½ × 7/16 = 6.38
174.22 □"

M11-L11 S = 362800 SR = 25.9 □"
4 Ls 6 × 6 × 1/2 = 23.00
1 Pl. 18 × 7/16 = 7.88
30.88 □"

M12-M12 S = 289100# C SR = 20.7 □"
4 Ls 6 × 4 × 1/2 = 19.00
1 web 18 × 7/16 = 7.88
26.88 □"

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MADE BY J.M. DATE 2-9-19 FILE NO.

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CALCULATIONS FOR

Design of Shirahige-Bashi

Verticals

U13-M13	S = 329400# C	SR = 23.5 □"	same as U12-M12
U14-M14	S = 246400# C	SR = 17.6 □"	same as U12-M12
U15-M15	S = 217300# C	SR = 15.5 □"	same as U12-M12
U16-M16	S = 235500# C	SR = 16.7 □"	same as U12-M12
U17-M17	S = 231600# C	SR = 16.1 □"	Same as U12-M12
U18-M18	S = 248750# C	SR = ~~14.2~~ 17.7 □"	Same as U12-M12
U19-M19	S = 0.0		Same as U12-M12

Hangers S = 234000# T SR = 13.750 □" net

4 L^s 6·4·½ = 19.00 - 4.0 = 15.00

1 Pl. 18·7/16 = 7.88 - .88 = 7.00

26.88 □" 22.00 □"

Tie

L11-L11 S = 2626600# T SR = 184.20 □" net

8 Pls. 26·3/4 = 156.00

2 Pls. 20·3/4 = 30.00

2 L^s 6·6·3/4 = 16.88

202.88 - 45.0 = 157.88 □" net 6 rows of Rivet holes out

Pin for side span Load = 720.000#

On abutment L0. 6" pin assumed

Unit bearing = $\frac{720.000}{2.75 \times 2 \times 6}$ = 21900 #/□"

Bending moment = 360.000 · 1.25 = 450.000"# $fs = \frac{450.000 \times 3}{0.049 \times 6^4}$ = 21200 #/□" ok

Pin at L8 7½" pin assumed

Unit bearing = $\frac{720.000}{2.375 \times 2 \times 7.5}$ = 20300 #/□"

Bending moment = 360.000 · 2.5 say = 900.000"# $fs = \frac{900.000 \times 3.75}{0.049 \times 7.5^4}$ = 21700 #/□" ok

Shoe at abutment load 730.000#

4⅝" Roller 610 · 4.62 = 2820# 8 Rollers

Length of roller = $\frac{730.000}{2820 \times 8}$ = 32.3" net each roller

Bearing on Concrete = $\frac{730.000}{45 \times 44}$ = 370 #/□" ok

Pin at L10 on pier Load 2807.000# 10" pin assumed

Unit bearing = $\frac{2807.000}{11.75 \times 10}$ = 23900 #/□"

There is bearing on top and bottom and no moment on pin.

Shoe on pier Load 2820.000#

9½" Roller 9½ · 610 = 5800# per lin inch 9 Rollers

Length of Roller = $\frac{2820.000}{5800 \times 9}$ = 54" net each Roller.

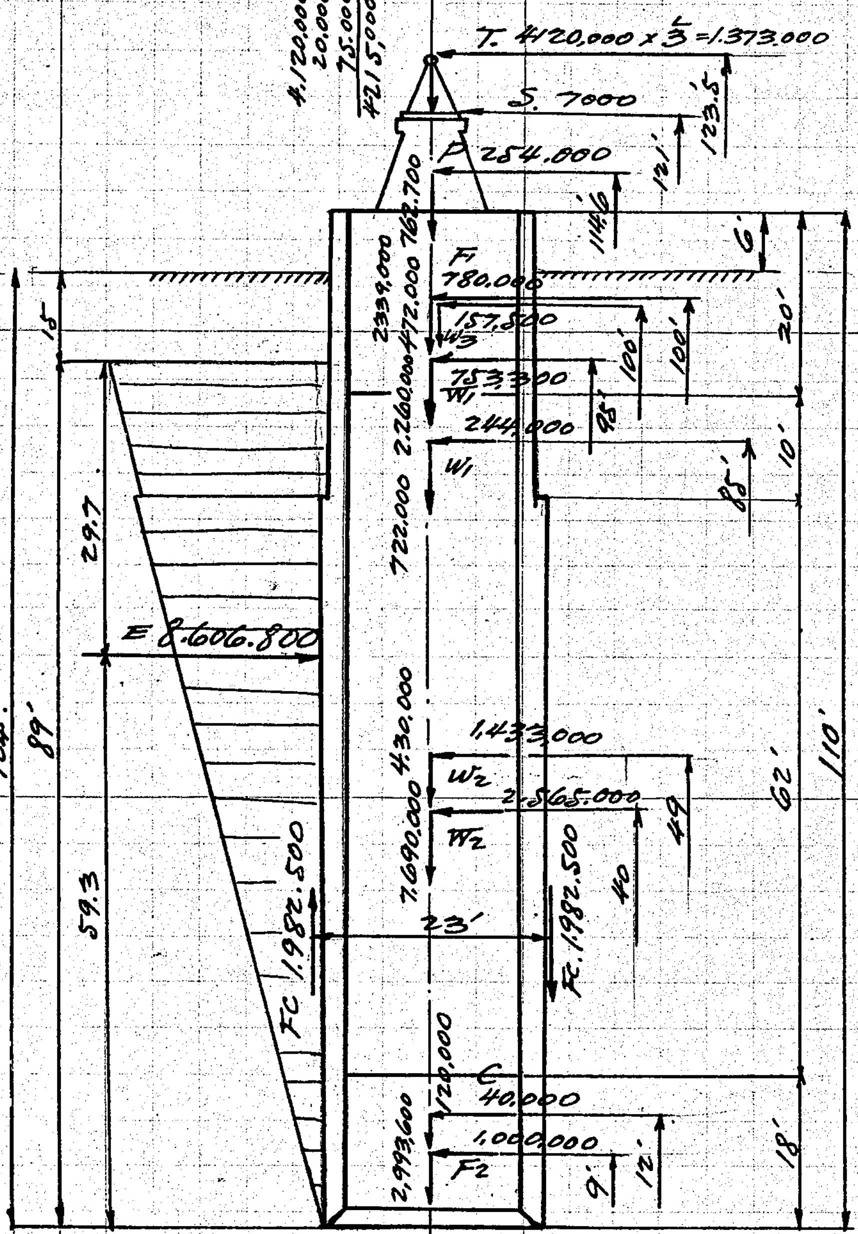
Bearing on Concrete = $\frac{2820.000}{68.5 \times 68.5}$ = 600 #/□"

make base 5'-8½ · 5'-8½"

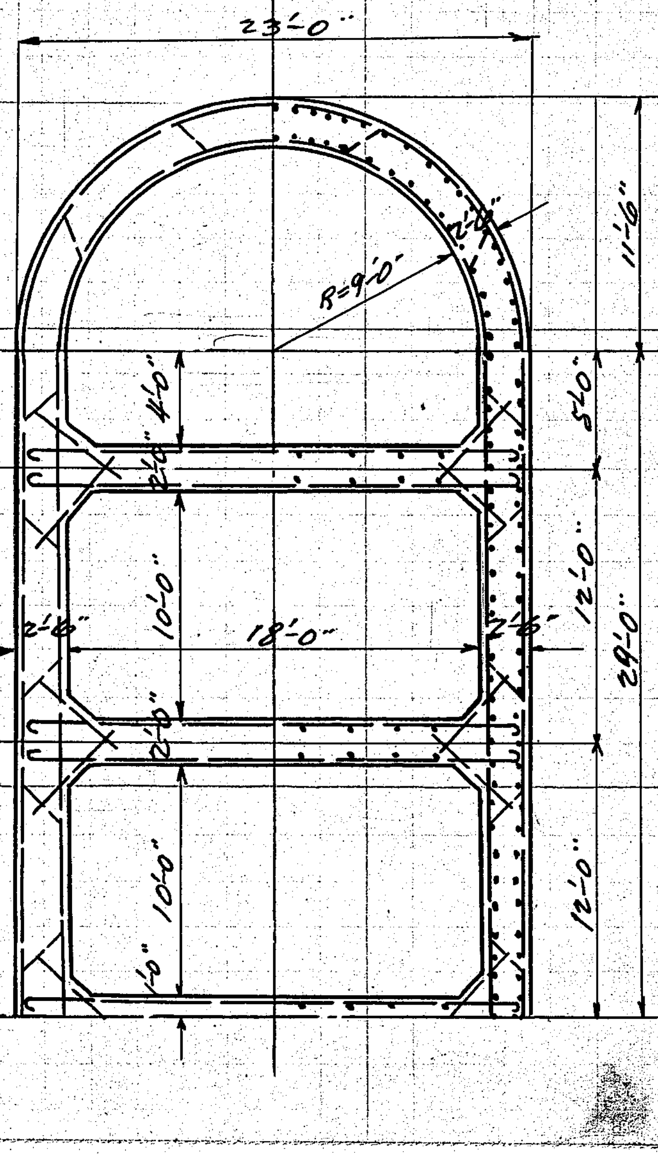
Shirahige Bashi for Tokyofu.

Design of well for Pier

Seismic stability of Pier. Acceleration of earthquake = $\frac{10000}{3}$ mm/sec²



Earthquake moment = 8.606.800 × 59.3
= 510,000,000.'#



Loads in case of earthquake.

Loads on shoes = 2060,000 × 2 = 4,120,000# arm 138' (T)

Extra load due to seismic moment = 175,000

Weight of shoes 10000 × 2 = 20.000 121' (S)

Shaft.

Coping 8×15×63 = 756 ¢ @ 150 = 113,500#

Shaft 9×8.5×64 = 4895 @ 150 = 732,500

Less holes 2 @ 4.4×6×10.5 = -555 @ 150 = − 83,300

762,700# 114.6 (P)

Weight of well, shell.

upper 30' shell

sectional area

sides 58×2×2' = 232.0□'

Ends 20×3.142×2 = 125.7

Walls 18×5×1.5 = 135.0

fillets 1×1×10 = 10.0

502.7□'

502.7×30 = 15,081 ¢ @ 150 = 2,260,000# 95' (W_1)

Lower 80' shell

sectional area

Sides 58×2×2.5 = 290.0□'

Ends 20.5×3.142×2.5 = 161.2

Partition wall 18×5×2.0 = 180.0

fillets 1×1×10 = 10.0

641.2□'

641.2×80 = 51,296 ¢ @ 150 = 7,690,000# 40' (W_2)

Shoe 24' @ 5000# = 120,000# 12' (C)

Concrete fill

Top fill

Interm partitions 10.5×18×2 = 378.0□'

End " 4.25×18×2 = 153.0

" " $(\frac{18}{2})^2$×3.142 = 254.7

fillets, less 1×1×6 = −6.0

779.7□'

779.7×20 = 15594 @ 150 = 2,339,000# 100' (F_1)

Bottom fill

Int. partitions 10×18×4 = 720.0□'

End " 4×18×2 = 144.0

" " $(\frac{18}{2})^2$×3.142 = 254.0

fillets less 1×1×10 = −10.0

1,108.7□'

1108.7 × 18 = 19957 @ 150 = 2,993,600# 9' (F_2)

Water fill

1153.7×10 @ 62.5 = 722000# 85' (W_1)

1108.7×62 @ 62.5 = 4,300,000 49' (W_2)

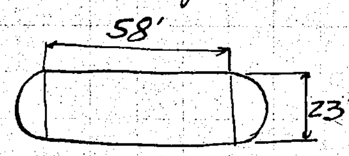
Top 7560 × @ 62.5 472,000 100 (W_3)

Shirahige Bashi for Tokyofu.

Surface friction of well assumed at 250#/□'
Circumference of well = 58×2 + 22.5×3.142 = 186.7'
Total friction = 186.7 × 86 × 250 = 3,965,000#
Frictional couple = $\frac{3,965,000}{2}$ × 22.5 = 44,600,000'# (FC)
Bearing pressure at Bottom of well during earthquake.

				Seismic forces	arm	Earthquake mom.
T.	Superimposed load (dead)	= 4,120,000	× 1/3 =	1,373,000	× 123.5'	= 169,500,000'#
S.	Weight of Shoes	20,000	× 〃 =	7,000	× 121.0	= 850,000
P.	〃 〃 Shaft	762,700	× 〃 =	254,000	× 114.6	= 29,150,000
W_1	〃 〃 well, upper 30' Shell	2,260,000	× 〃 =	753,300	× 95	= 71,560,000
W_2	〃 〃 〃 lower 80' 〃	7,690,000	× 〃 =	2,565,000	× 40	= 102,600,000
C	〃 〃 Curb shoes	120,000	× 〃 =	40,000	× 12	= 480,000
F_1	〃 〃 Top concrete fill	2,339,000	× 〃 =	780,000	× 100	= 78,000,000
F_2	〃 〃 Bottom 〃 〃	2,993,600	× 〃 =	1,000,000	× 9	= 9,000,000
w_1	〃 〃 Water fill	722,000	× 〃 =	244,000	× 85	= 20,460,000
w_2	〃 〃 〃 〃	4,300,000	× 〃 =	1,433,000	× 49	= 70,250,000
w_3	〃 〃 〃 〃	472,000	× 〃 =	157,500	× 100	= 15,750,000
	ΣV =	25,799,300#		ΣH = 8,606,800#		Σm = 567,600,000
	Less friction	= −3,965,000		Less earth press. moment		= −510,000,000
		21,834,300		〃 frictional couple		= −44,600,000
	Extra load on shoe	75,000				+13,000,000'#
		21,909,300#				

Moment of inertia of bottom area.



$\frac{58\times 23^3}{12} = 58,000$

$0.049\times 23^3 = 13,700$

$72,500'^4$

Bottom area of well.
$23\times 58 + 11.5^2\times 3.142 = 1,749$□'

Max. Bearing pressure at Toe.
$= \pm \frac{13,000,000\times 11.5}{72,500} + \frac{21,909,300}{1,749}$
$= \pm 2,060 + 12,540$ #/□' $= \pm 0.92 + 5.60$ tons/□'
$= 6.52$ tons/□' Compression.
or = 4.68 〃 〃

Stability of pier at normal state for full load.

Superimposed live load = 796,000 × 2 = 1,592,000#
From the previous calcurations, max. load on bottom of well
= 21,909,300 + 1,592,000 = 23,501,300#

Bearing pressure on bottom earth
$= \frac{23,501,300}{1,749} = 13,440$ #/□' = 6.0 tons/□'

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CALCULATIONS FOR

Shirahige Bashi for Tokyofu. (3)

Pressure on well

Case 1. Pressure on well after competion.

Rankine's formula

unit pressure $p = wh \frac{1-\sin\phi}{1+\sin\phi}$

in which $w = 120^{\#}/c.f.$, $\phi = 30°$

$p = \frac{1}{3} \times 120 \times h = 40h.$

Case 2. Temporary pressure on well during execution.

Ketchum's walls, bins and grain elevators Side pressure for temporary trench work Page 120 & 121

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

where L = Lateral unit pressure in lbs per sq.ft. at depth y

V = Vertical unit pressure in lbs per sq.ft. at depth y

w = weight of earth in lbs per cub. ft.

ϕ = angle of repose of earth.

$\mu = \tan\phi$ coeff. of friction of earth on earth.

b = the distance in ft. that the earth breaks arround the well.

ϕ' = angle of friction of earth on the surface of the well.

assumed that $\phi = \phi' = 30°$

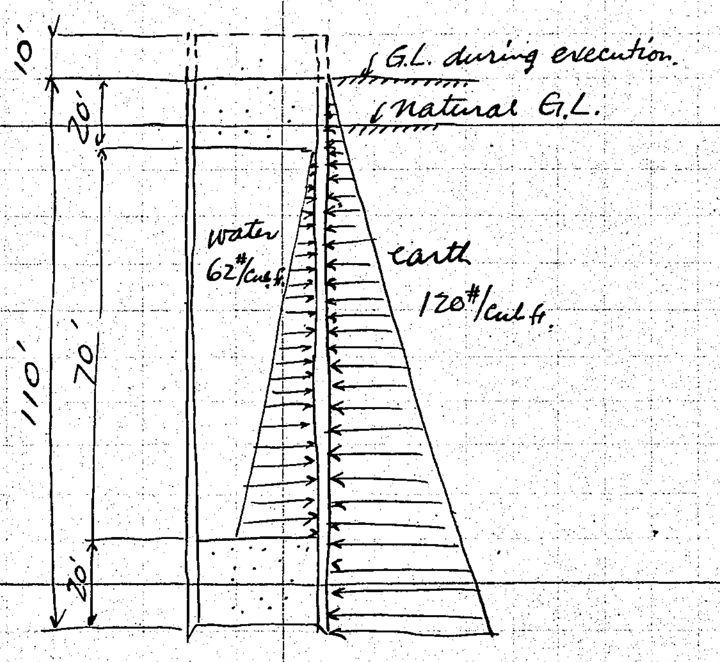
$K = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1}{3}$

$b = 10'$, $w = 120^{\#}/cub.ft.$

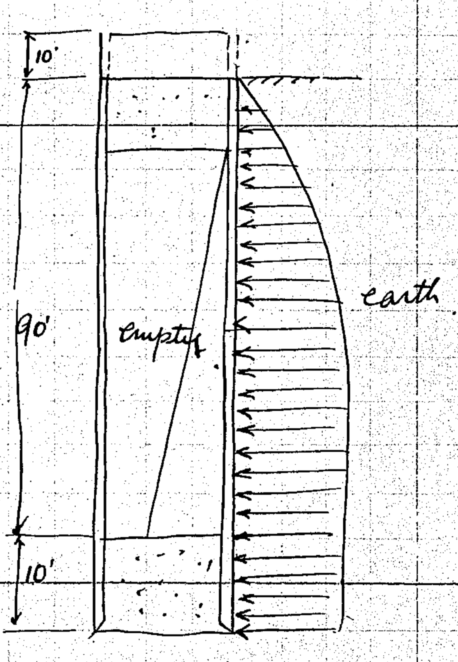
depth of earth	earth pressure	depth of water	water pres.	resulting pres.
110'	4,400#/□'	80'	5,000	-600#/□'
100	4,000	70'	4375	-375
80	3,200	50	3,125	+ 75
60	2,400	30	1,875	+525
40	1,600	10	625	+975
20	800			+800
10	400			+400

depth of earth	temporary pressure	
110'	+ 870 × 1.2 = + 1,050#/□'	
100	+ 850 × 1.2 = + 1,020	
80	+ 800 × 1.2 = + 960	1050
60	+ 780 × 1.2 = + 936	
40	+ 690 × 1.2 = + 828	
20	+ 480 × 1.2 = + 576	
10	+ 290 × 1.2 = + 348	800

Note + sign shows external pressure.
— " " internal pressure.



Case 1.



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(独立行政法人 土木研究所蔵)

Shirahige Bashi for Tokyofu.

Design of well continued.

Reinforcements in the shell. (40~110' deep)

Thickness of shell 2.5' = 30"

Side pressure = 1,050 #/□'

Moment on circular end

$= \frac{1}{16} \times 1050 \times 20.5^2 = 27{,}600$ '#

Moment on side wall

$= \frac{1}{12} \times 1050 \times 12.0^2 = 12{,}600$ '#

Reinforcements required.

for circular end $\frac{27600 \times 12}{17000 \times \frac{7}{8} \times 28} = 0.795$ □"/ft strip.

for side wall $\frac{12600 \times 12}{17000 \times \frac{7}{8} \times 28} = 0.364$ □"/ft strip.

at 0 ~ 40 ft deep.

Thickness of well = 2.0 = 24"

side pressure = 800 #/□'

Moment on circular end

$= \frac{1}{16} \times 800 \times 20.5^2 = 21{,}000$ '#

moment on side wall

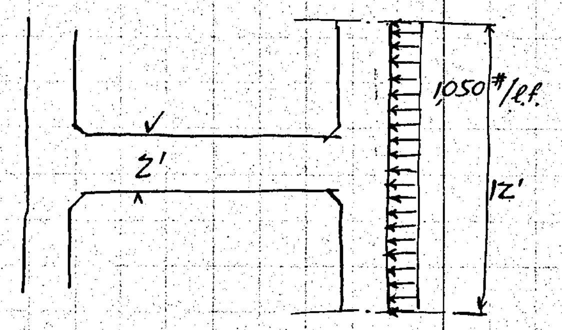
$= \frac{1}{12} \times 800 \times 12.0^2 = 9{,}600$ '#

Reinforcement required

for circular end $= \frac{21{,}000 \times 12}{17000 \times \frac{7}{8} \times 22} = 0.768$ □"/ft strip

for side wall $= \frac{9600 \times 12}{17000 \times \frac{7}{8} \times 22} = 0.352$ □"/ft strip

partition wall.



Load on partition wall = 1,050 × 12 = 12,600 #/ft strip

$l/d = 16/2 = 8$

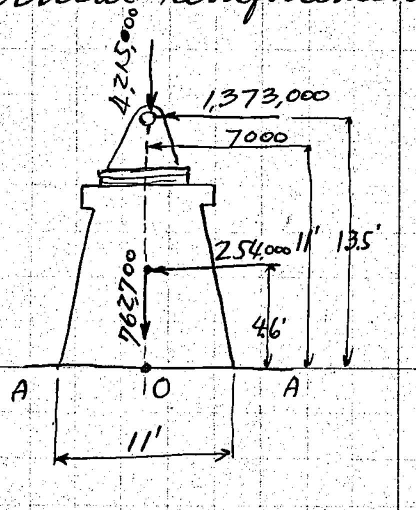
design as a short column.

unit ~~column~~ compression on concrete

$= \frac{12600}{2 \times 144} = 44$ #/□" OK.

Use some reinforcements on both sides.

Vertical Reinforcements in the shaft.



At Bottom of shaft. Length of shaft resisting to moment and vertical load assumed 23' on both ends = 2×23 = 46'

Moment about O.

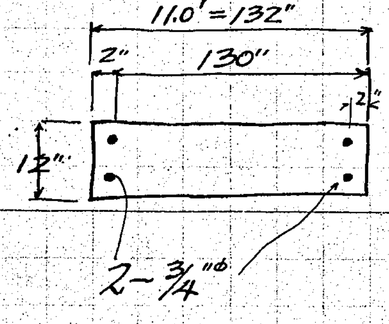
due to Hor. forces.

1,373,000	×	13.5	=	18,550,000 '#
7,000	×	11.0	=	77,000
254,000	×	4.6	=	1,168,000
ΣH = 1,634,000 #			Σm_h =	19,795,000 '# ÷ 46 = 431,000 '# per ft strip

due to vert. load. $m_v = 0$.

4,215,000 #
762,700

ΣV = 4,977,700 # ÷ 46 = 108,000 # per ft strip.



Eccentricity $\varepsilon = \frac{431{,}000}{108{,}000} = 3.99'$

$\varepsilon/h = 3.99/11 = 0.363$

$p_0 = 2p = \frac{.4418 \times 4}{12 \times 132} = 0.00112$

$d'/h = 2/132 = 0.015$

$f_c = M \div C_2 b h^2, \quad f_s = n f_c \left(\frac{d}{Kh} - 1\right)$

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CALCULATIONS FOR

Shirahige Bashi for Tokyofu.

By the prepared diagrams.

$K' = 0.500, \quad K'' = 0.465$

$K = K' - 5\left(\frac{d'}{h}\right)(K' - K'') = .500 - 5 \times .015 \times .035 = 0.497$

$A_2 = 0.0368, \quad B_2 = 0.0833$

$C_2 = 100 p_b A_2 + B_2 = 100 \times 0.00112 \times 0.0368 + 0.0833 = 0.0874$

$f_c = \frac{M}{C_2 b h^2} = \frac{431000 \times 12}{.0.0874 \times 12 \times 130^2} = 283$ #/□" OK.

$f_s = 15 \times 283 \left(\frac{130}{.497 \times 132} - 1\right) = 4,170$ #/□" OK.

Vertical Reinforcements at the Top of well.

Effective width of well resisting to moment + vertical forces, assumed to be 60'.

Moment on well = 19,795,000 '# ÷ 60 = 330,000 '# per ft strip of well.

vertical load " = 4,977,700 # ÷ 60 = 83,000 # " " on both sides.

or = 41,500 # " " on one side

resisting moment arm say 20.5'

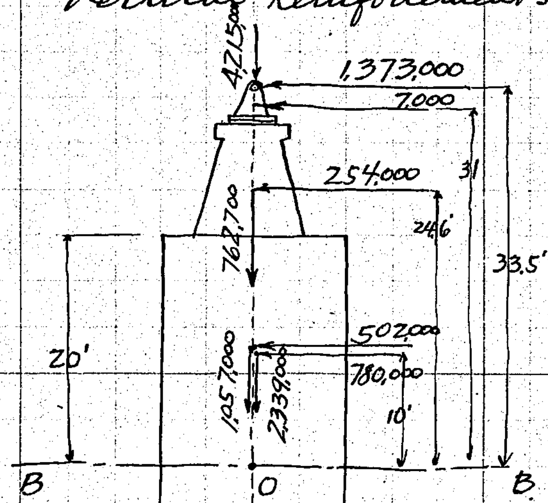
Total compression per ft strip of side wall.

$= \pm \frac{330,000}{20.5} + 41,500 = \pm 16,100 + 41,500 = 57,600 \div 2^{\square} = 28,800$ #/□" = 200 #/□" Comp.

or = 25,400 ÷ 2 = 12,700 □" = 88 #/□" Comp.

No vertical reinforcement required theoritically.

Vertical Reinforcements at 20' below top of well.



Taking moment about ~~section~~ O

Moment from shaft	19,795,000 + 1,634,000 × 20 =	52,475,000 '#
moment due to well	502,000 × 10 =	5,020,000
"	780,000 × 10 =	7,800,000
		65,295,000 '#

Summary of vertical load

vertical load from shaft	=	4,977,700 #
do well	=	1,057,000
do "	=	2,339,000
		8,373,700 #

Effective width of well resisting to moment + vert. load assumed at 70'

moment per foot strip of well

= 65,295,000 ÷ 70 = 933,000 '#

vert. load per ft strip

= 8,373,700 ÷ 70 = 119,700 # or 59,850 # on one side

Resisting moment arm say 21'

Total stress per ft strip of well.

$= \pm \frac{933,000}{21} + 59,850 = 104,300$ # compression

or = 15,400 # "

unit compression $= \frac{104,300}{2 \times 144} = 353$ #/□" OK

No vertical Reinforcements required theoritically.

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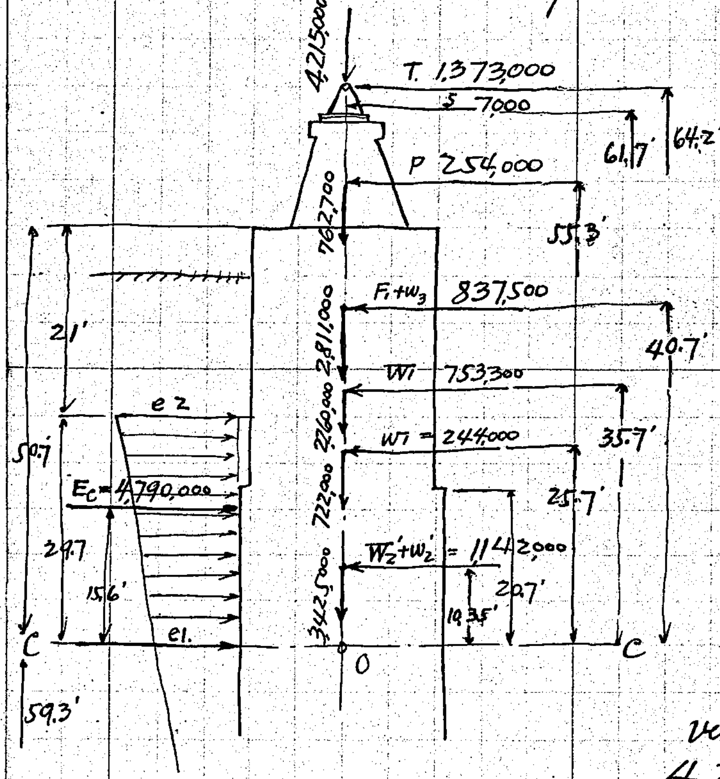
CALCULATIONS FOR

Shirahige Bashi for Tokyofu. 6

Vertical reinforcements at section 50.70' below top of well. i.e. point of application of resultant earth pressure.



$W_2' = 7690,000 \times \frac{20.7}{80} = 1990,000$

$W_2' = 4,300,000 \times \frac{20.7}{62} = \underline{1,435,000}$

$3,425,000 \div 3 = 1,142,000^{\#}$

$e_2 = \frac{8606,800 \times 2}{89} = 193,500^{\#}$

$e_1 = \frac{193500 \times 59.3}{89} = 129,000^{\#}$

$E_c = \frac{193500 + 129000}{2} \times 29.7 = 4,790,000$

moment $= 4,790,000 \times 15.6 = 74,700,000'^{\#}$

Total friction above section c-c

$187' \times 250^{\#} \times 29.7' = 1,390,000^{\#}$

frictional couple

$= \frac{1,390,000}{2} \times 23 = 16,000,000'^{\#}$

Line of action of E_c

$x = \frac{29.7\,(129,000 + 387,000)}{3\,(193,500 + 129,000)} = 15.6'$

Take moments about O.

	vertical forces		Seismic forces		Lever arm	moment about O.
	4,215,000 #	T	1,373,000	×	64.2	88,200,000
		S	7,000	×	61.7	432,000
	762,700	P	254,000	×	55.3	14,040,000
	2,812,700	F_1+W_3	837,500	×	40.7	34,080,000
	2,260,000	W_1	753,300	×	35.7	26,900,000
	722,000	W_1	244,000	×	25.7	6,270,000
	3,425,000	$W_2'+W_2'$	1,142,000	×	10.35	11,820,000
	14,197,400					181,742,000 '#
Less friction	−1,390,000		Less earth pressure m.			−74,700,000
			" frictional couple			−16,000,000
ΣV =	12,807,400 #				Σm =	91,042,000 '#

moment per ft strip of well

$= 91,042,000 \div 70 = 1,300,000'^{\#}$

Vertical load per ft strip of well

$12,807,400 \div 70 \div 2 = 91,500^{\#}/ft$ on one side

Total stress per ft strip of side wall. arm say 21.

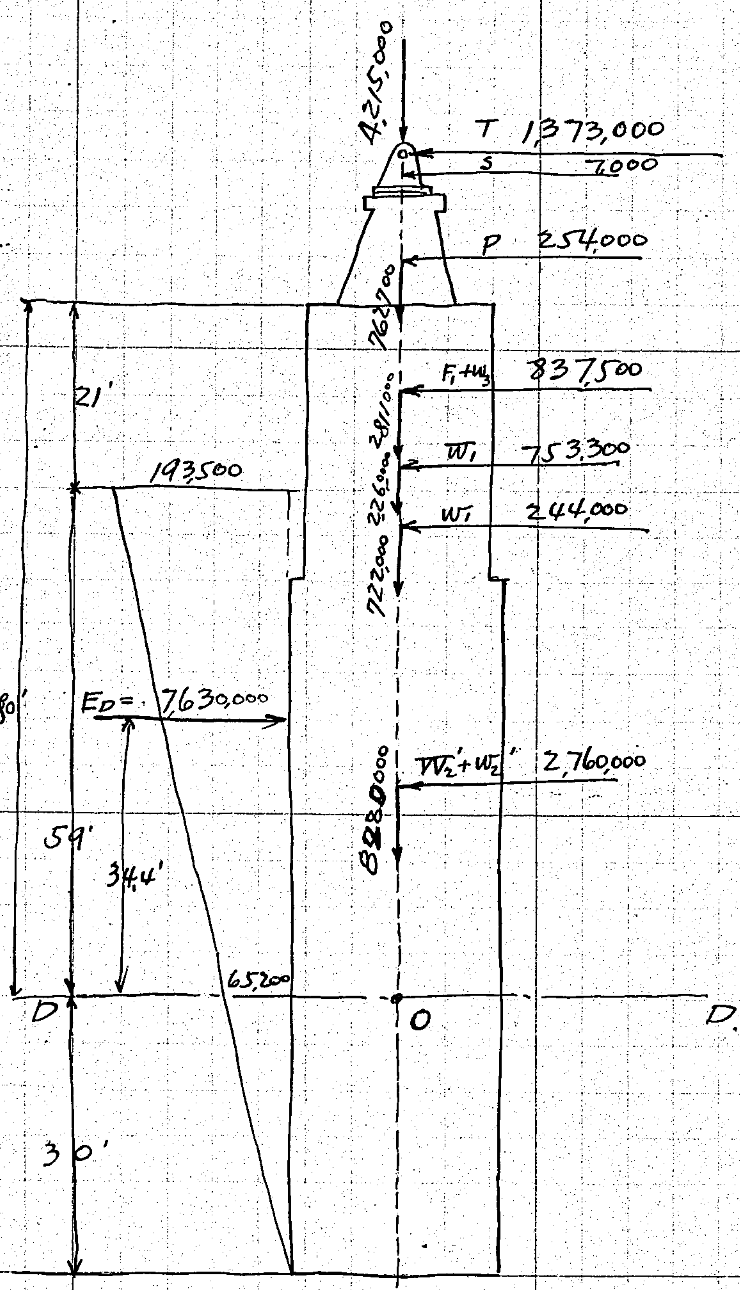
$= 91,500 \pm \frac{1300,000}{21} = 153,400^{\#}/ft = 76,700^{\#}/\square' = 533^{\#}/\square''$ ok

$= 29,600$ "

No vertical reinforcement required theoritically.

Shirahige Bashi for Tokyofu. 7.

Vertical reinforcement in the well.
Section at 80' below top of well.



$W_2' = 7{,}690{,}000 \times \frac{50}{80} = 4{,}810{,}000^{\#}$
$W_2' = 4{,}300{,}000 \times \frac{50}{62} = \underline{3{,}470{,}000}$
$8{,}280{,}000^{\#} \div 3 = 2{,}760{,}000$

$e_1 = \frac{193500 \times 30}{89} = 65{,}200^{\#}$

$E_D = \frac{193{,}500 + 65{,}200}{2} \times 59 = 7{,}630{,}000^{\#}$

Line of action of ED
$x = \frac{59}{3} \frac{(65200 + 387000)}{(65200 + 193500)} = 34.4$

Total friction above D-D
$187 \times 250 \times 59 = 2{,}755{,}000^{\#}$

Frictional couple
$\frac{2{,}755{,}000 \times 23}{2} = 31{,}700{,}000'^{\#}$

Resisting moment due to earth pressure
$= 7630{,}000 \times 34.4 = 254{,}800{,}000'^{\#}$

Taking moment about O.

Vert forces	Seismic forces		Lever arm		moment about O.
4,215,000	T 1,373,000	×	93.5	=	128,400,000'#
	S 7,000	×	91.0	=	637,000
762,700	P 254,000	×	84.6	=	21,500,000
281,1000	F1+W3 837,500	×	70.0	=	58,600,000
2,260,000	W1 753,300	×	65.0	=	49,000,000
722,000	W1 244,000	×	55.0	=	13,420,000
8,280,000	W2+W2 2,760,000	×	25.0	=	69,000,000

ΣV = 19,050,700
less friction 2,755,000
16,295,700

Σm = 340,557,000'#
Less earth pressure m − 254,800,000
" frictional couple − 31,700,000
54,057,000

Moment per ft strip of well
$= 54{,}057{,}000 \div 70 = 772{,}000'^{\#}$
Vertical load per ft strip of well
$= 16{,}295{,}700 \div 70 = 232900 \div 2 = 116{,}450^{\#}$ on one side.

Total stress per ft strip of side wall arm = 21'
$= 116{,}450 \pm \frac{772{,}000}{21} = 153{,}250^{\#} \div 288 = 532^{\#}/\square''$ O.K.
or = 79,650
No vert reinforcement required theoritically.

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CALCULATIONS FOR

Shirahige Bashi for Tokyofu

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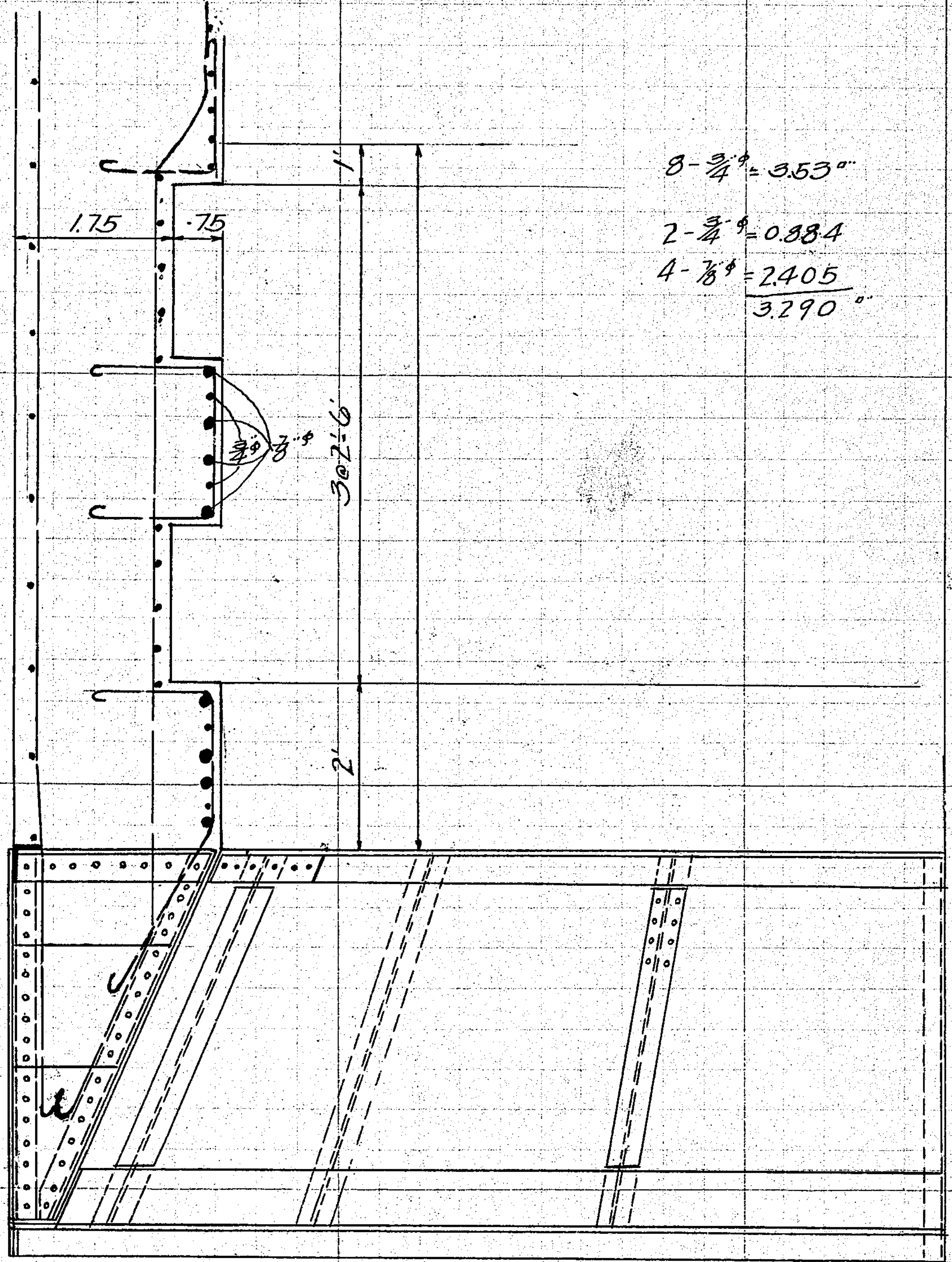


8-3/4"φ = 3.53"

2-3/4"φ = 0.884
4-7/8"φ = 2.405
3.290"

Outer shell

20 Pls	60" x 1/4" x 9'-6"	9.690#
20 Pls	3" x 3" x 1/4" x 9'-6"	932#
20 spl. Pls	6 1/2" x 1/4" x 4'-6"	496
20 " "	6" x 1/4" x 2'-6"	256
20 " "	3" x 1/4" x 2'-6"	128
		11.502#

Inner shell

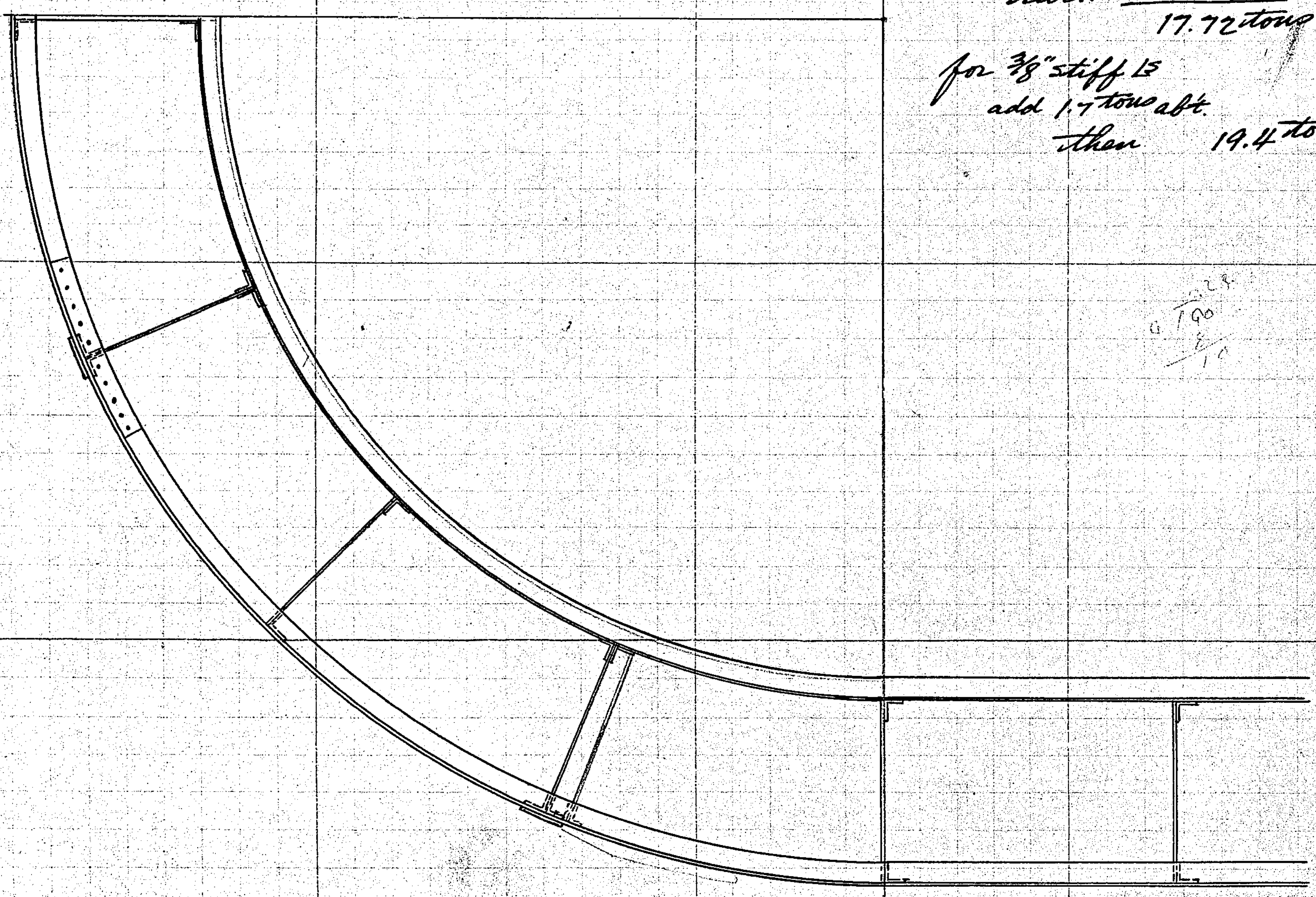
20 Pls	60" x 1/4" x 9'-6"	9.690
20 bent Pls	3" x 3" x 1/4" x 7'-3"	710
20 Ls	6" x 6" x 3/8" x 9'-6"	2.830
20 Ls	6" x 6" x 1/2" x 9'-6"	3,725
20 spl. Pls	6 1/2" x 1/4" x 3'-9"	412
20 " Ls	3" x 3" x 1/4" x 2'-6"	245
20 " "	6" x 6" x 3/8" x 2'-6"	745
20 " "	6" x 6" x 1/2" x 3'-0"	1,176
		19.533

Diaphram

60 Ls	3" x 3" x 1/4" x 2'-6"	735
60 Ls	3" x 3" x 1/4" x 5'-0"	1470
60 Ls	3" x 3" x 1/4" x 4'-6"	1323
40 web Pls	12" x 3/8" x 2'-6"	1,530
40 " "	15" x 3/8" x 2'-0"	1,830
20 fills.	3" x 1/4" x 2'-0"	102
20 "	3" x 1/4" x 1'-6"	77
		6.767

Total 37.802#
or 16.88 tons
5% rivet 0.84
17.72 tons

for 3/8" stiff Ls
add 1.7 tons abt.
then 19.4 tons



Shirahige Bashi for Tokyofu.

Bouyancy of water on well during execution

Weight of well shell, upper 30'	2,260,000#
" " lower 80'	7,690,000
Curb Shoes	40,000
Bottom fill say 12'	2,000,000
Concrete blocks at top 10' high	378,000
	12,368,000# = 5,520 tons

Bouyancy of water on empty well

206,200 c.f. @ 72# say = 14,846,000 = 6,625 tons

resultant bouyant effect = 1,105 tons

Sectional area of well
upper 30' — 1,657 □'
lower 80' — 1,749 □'
volume of well
= 1657×40 = 66,280 c.f.
1749×80 = 139,920
206,200 c.f.

Friction of earth on well.

upper 24'	185×24×250 =	1,110,000
lower 80'	189×80×250 =	3,780,000
		4,890,000# = 2185 tons

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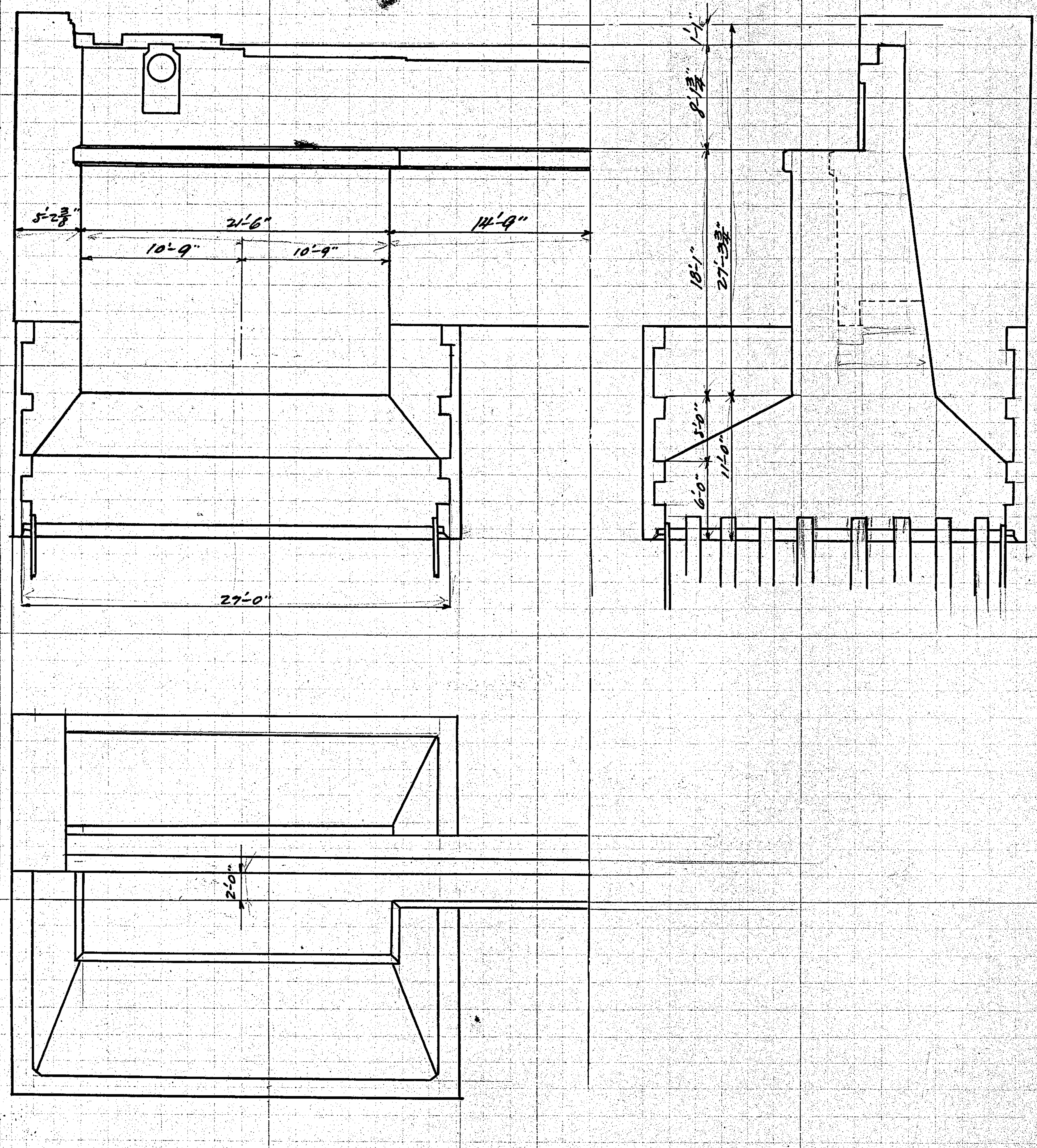

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CALCULATIONS FOR

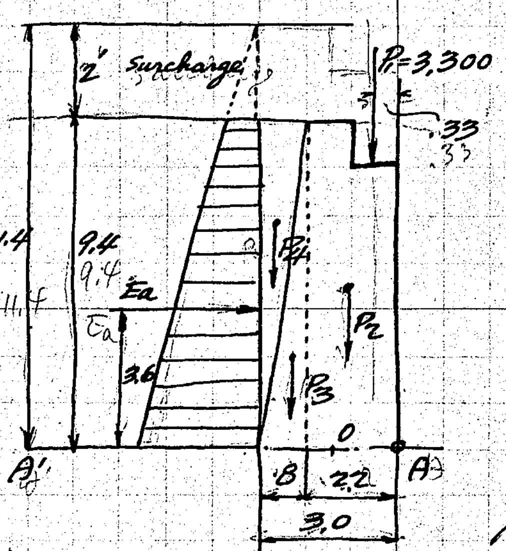
Shirahige Bashi for Tokyofu

Design of abutment

Prapet Wall Case 1.

Stability at normal case

Live load



Stringer reaction on abutment D.L. 300#/lin. ft. of parapet wall
L.L. 3000
P_1 = 3300#

$P_2 = 2.2 \times 9.4 \times 150 = 3,100$

$P_3 = \dfrac{0.8 \times 9.4}{2} \times 150 = 560$

$P_4 = \dfrac{0.8 \times 9.4}{2} \times 100 = 380$

$Ea = \dfrac{67+380}{2} \times 9.4 = 2,100$

Moment about A

	Forces		lever arm		moment
P_1	3,300	×	0.33	=	1,090'#
P_2	3,100	×	1.10	=	3,410
P_3	560	×	2.47	=	1,380
P_4	380	×	2.73	=	1,040
Ea	2,100	×	3.60	=	−7,560
	ΣV = 7,340#				− 640

} 5830'# ÷ 4040 = 1.44'

Point of appication of Earth pressure

$p_1 = \dfrac{wh}{3} = \dfrac{100 \times 2}{3} = 67$ #/□'

$p_2 = " = \dfrac{100 \times 11.4}{3} = 380"$

$X = \dfrac{9.4(67+760)}{3(380+67)} = 5.8$

$X_1 = 9.4 - 5.8 = 3.6$

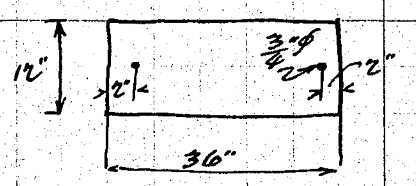
$Ea = \dfrac{(67+380)}{2} \times 9.4 = 2100$#

Moment about N.A. of Section AA'

$= -640 - 7,340 \times 1.5 = -11650$'#

Eccentricity $\varepsilon = \dfrac{11650}{7340} = 1.59' = 19.1"$

$p = \dfrac{.8836}{36 \times 12} = 0.002 \qquad \varepsilon/h = \dfrac{19.1}{36} = 0.53$



$d'/h = \dfrac{2}{36} = 0.056$

$f_c = \dfrac{M}{c_2 b h^2} \cdot \quad f_s = n f_c \left(\dfrac{d}{kh} - 1\right)$

By the prepared diagram

$k' = 0.36 \quad k'' = 0.3 \quad k = k' - 5\left(\dfrac{d'}{h}\right)(k' - k'')$

$= 0.36 - 5 \times 0.056 \times 0.06 = 0.343 \quad j = 1 - \dfrac{k}{3} = .886$

$A_2 = 0.0875 \quad B_2 = 0.066$

$C_2 = 100 p A_2 + B_2 = 100 \times 0.002 \times 0.0875 + 0.066 = .084$

$f_c = \dfrac{11650 \times 12}{0.084 \times 12 \times 36^2} = 107$ #/□" Ok

$f_s = 15 \times 107 \left(\dfrac{34}{0.343 \times 36} - 1\right) = 2,810$ #/□" Ok

$v = \dfrac{2100}{12 \times .886 \times 34} = 5.8$ #/□" ok

$u = \dfrac{2100}{2.36 \times .886 \times 34} = 29.5$ #/□" ok

Shirahige Bashi for Tokyofu

Case 2 Stability during earthquake of which acceleration $= \frac{10,000}{3}$ mm/sec²

Moment about O

	Forces	K	Earthquake force	arm		moment
P_2'	3100 ×	$\frac{1}{3}$ =	1,030 ×	4.7	=	4,840
P_3'	560 ×	" =	190 ×	3.1	=	590
P_4'	380 ×	" =	130 ×	6.3	=	820
Ea'	3840 ×	1 =	3,840 ×	3.5	=	12,400
			$\Sigma H = 4,890^{\#}$			

P_1	300 ×	1.17	=	350
P_2	3,100 ×	0.4	=	1,240
P_3	560 ×	−2.46	=	−1,380
P_4	380 ×	−2.74	=	−1,040
	$\Sigma V = 4340^{\#}$			$17,820'^{\#}$

Eccentricity $\varepsilon = \frac{17,820}{4,340} = 4.1'$

$\varepsilon/h = \frac{4.1}{3.0} = 1.37 \qquad \frac{d'}{h} = \frac{2}{36} = 0.056$

$p = 0.002$

$e'a_1 = 0.562 \times 100 \times 2 = 113^{\#/□'}$

$e'a_2 = 0.562 \times 100 \times 11.4 = 641^{\#/□'}$

$X = \frac{9.4(113+1282)}{3(113+641)} = 5.9'$

$X_1 = 9.4 - 5.9 = 3.5$

$Ea' = \frac{113+641}{2} \times 9.4 = 3,540^{\#}$

By the Prepared diagram on Page 399 Hool's Conc. eng. Handbook

$K = 0.20 \qquad L = 0.073$

$f_c = \frac{17820 \times 12}{0.073 \times 12 \times 36^2} = 188^{\#/□''}$

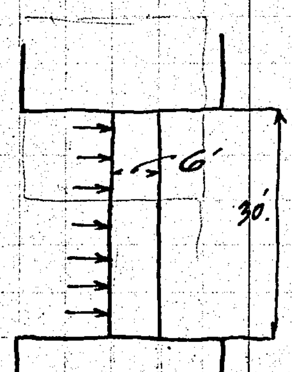
$f_s = n f_c\left(\frac{d}{kh} - 1\right) = 15 \times 188\left(\frac{34}{0.2 \times 36} - 1\right) = 10,500^{\#/□''}$

Use 1-5/8"φ = 0.306

$f_s = 10500 \times \frac{.442}{.306} = 15,200^{\#/□''}$ approx.

$v = \frac{4,890}{12 \times .967 \times 24} = 17.6^{\#/□''}$ Ok

$u = \frac{4,890}{2.36 \times .967 \times 24} = 89.4^{\#/□''}$ Ok



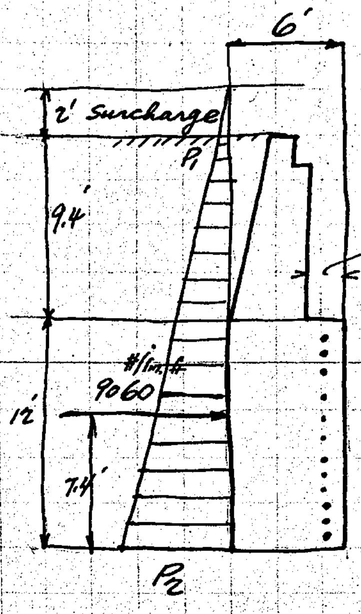
Hor. strut (Curtain wall) 6' Thick assumed

Case 1 Stability at normal state

$P_1 = \frac{100 \times 2}{3} = 67^{\#/□'}$

$P_2 = \frac{100 \times 23.4}{3} = 780^{\#/□'}$

$E = \frac{67+780}{2} \times 21.4 = 9,060^{\#/\text{lin.ft. of strut}}$



Span 30' say

moment $= \frac{9060 \times 30^2}{12} = 679,000'^{\#}$

steel req'd $= \frac{679,000 \times 12}{17000 \times \frac{7}{8} \times 70} = 7.3^{□''}$

Use 15 - 7/8"φ = 9.02□" (See next page)

End reaction $= 9060 \times 15 = 135,900^{\#}$

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CALCULATIONS FOR

Shirahige Bashi for Tokyofu

Case 2 During Earthquake

Earth pressure $E_B = 0.562 \times \frac{100 \times 23.4^2}{2} = 15,400^{\#}$/L of beam

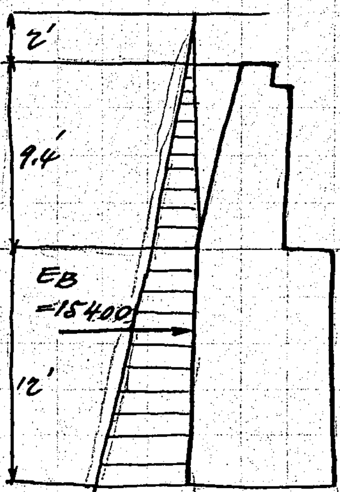
Hor. force from parapet wall except earth pressure

$P = 4340 \times \frac{1}{3} = 1450^{\#}$/L of beam

" " due to dead wt. beam of its own

$P_B = 6 \times 12 \times 150 \times \frac{1}{3} = 3.600^{\#}$/L of beam

Total $= 20,450^{\#}$/lin L span of beam



$M = \frac{20450 \times 30^2}{12} = 1,535,000'^{\#}$

Steel req'd $= \frac{1,535,000 \times 12}{30600 \times \frac{7}{8} \times 70} = 8.35^{\square''}$

Use $15 - \frac{7}{8}''\phi = 9.02^{\square''}$

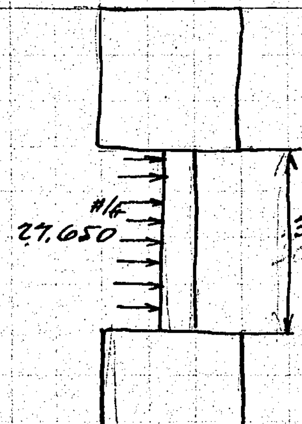
$p = \frac{9.02}{12 \times 6 \times 144} = 0.00087$ $k = 0.153$ $j = .949$

$f_s = \frac{1,535,000 \times 12}{9.02 \times .949 \times 70} = 30,600^{\#/\square''}$ OK

$f_c = \frac{2 f_s p}{k} = \frac{2 \times 30.600 \times 0.00087}{0.153} = 348^{\#/\square''}$ OK

$v = \frac{30700}{144 \times .949 \times 70} = 32.1^{\#/\square''}$ (shear $V = \frac{20450 \times 30}{2} = 307,000^{\#}$) OK

$u = \frac{30700}{15 \times 2.75 \times .949 \times 70} = 112^{\#/\square''}$ OK



Earthquake forces backward with no earth pressure

Loads on Beam

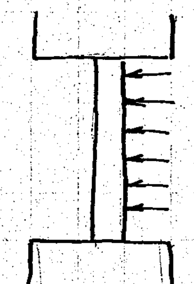
Forces form parapet wall $= 1.450$

Dead wt. of its own $= 3600$

$5,050^{\#}$/lin L of span of beam

Moment $= \frac{5050 \times 30^2}{12} = 379,000'^{\#}$

Steel req'd $= \frac{379000 \times 12}{30600 \times 0.95 \times 70} = 2.235^{\square''}$



Use $8 - \frac{5}{8}''\phi = 2.45^{\square''}$

End reaction $= 5050 \times 15 = 75,750^{\#}$

Point of application of resultant forces

Taking Moments about bottom of beam

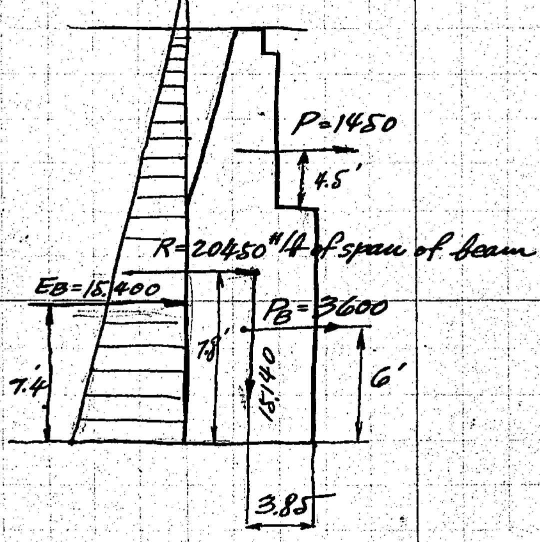
P	1450	×	16.5	= 23,900 '#
P_B	3,600	×	6.0	= 21,600
E_B	15,400	×	7.4	= 114,000
	20,450			159,500 '# ÷ 20,450 = 7.8 (ordinate)

Hor. reaction on shaft $= 20,450 \times 15 = 307,000^{\#}$

$4340 \times 3.94 = 17,100$

$10,800 \times 3.00 = 32,400$

$49,500 \div 14800 = 3.85'$ (abscissa)



Vert. reaction on shaft $= 15,140 \times 15 = 227,100^{\#}$

増田淳氏関係資料
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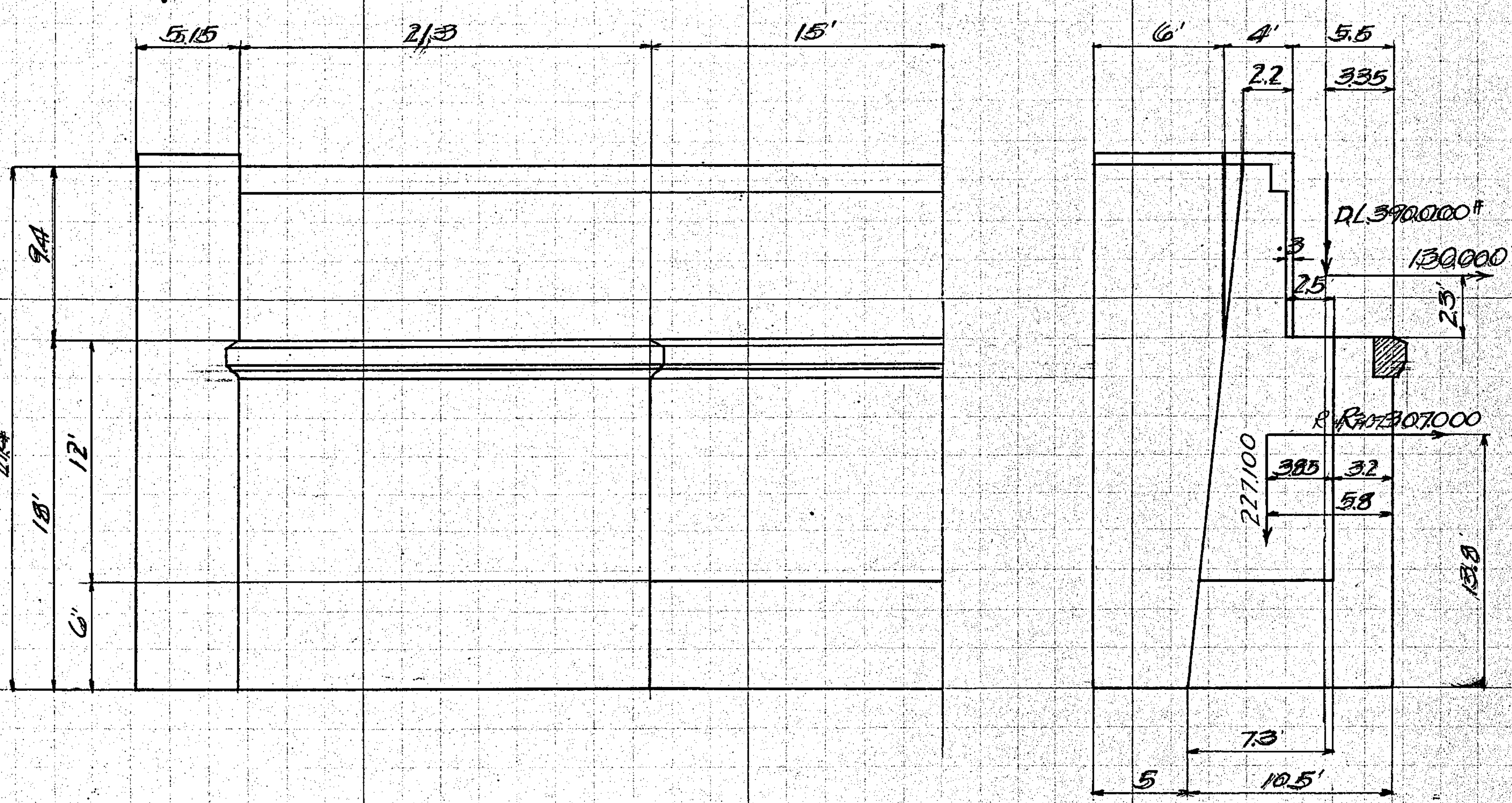
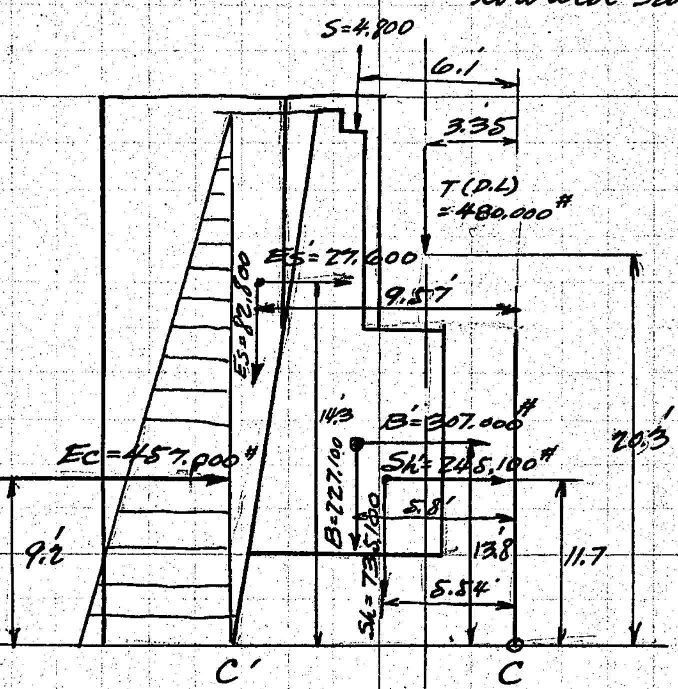
CALCULATIONS FOR

Shirahige Bashi for Tokyofu



Shaft. During earthquake

Case A Seismic forces due to Back earth fill and weight of abutment structure acting at their center of gravity toward river side.



		vert. force	Hor. force
parapet wall on shaft	4340 × 21.5 =	93,300# × 1/3 =	31.100
shaft	9.5 × 21.5 × 18 × 150 =	551.000 × 〃 =	183.700
Column	4 × 4 × 27.5 × 150 =	66.000 × 〃 =	22.000
Wing wall	6 × 27.5 × 1. × 150 =	24.800 × 〃 =	8,300

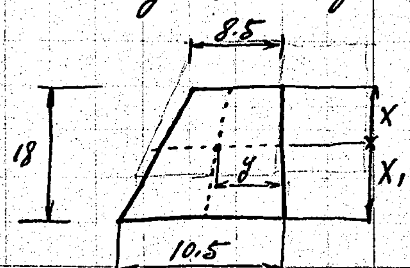
C.G. of shaft, Parapet wall on shaft. Col. and wing

Moment of vert. forces about C

Parapet wall on shaft	93,300 × 7.15 =	666,500'#
Shaft	551.000 × 4.73 =	2,608,000
Column	66.000 × 7.40 =	488.000
Wing wall	24.800 × 12.50 =	310,000
	ΣV = 735,100#	4.072,500'#

∴ arm = 4,072,500 ÷ 735,100# = 5.54'

C.g. of shaft



$X = \frac{18(8.5+21)}{3(8.5+10.5)}$

$= 9.32$

$X_1 = 18 - 9.32$

$= 8.68 \doteqdot 8.7$

$y = (8.5 + \frac{10.5-8.5}{18} \times 8.7) \div 2$

$= 4.73'$

Moment of Hor. forces about C

Parapet wall on shaft	31,100 × 22.5 =	700,000'#
Shaft	183.700 × 8.7 =	1.598,000
Column	22,000 × 13.75 =	114,000
Wing wall	8,300 × 13.75 =	114.000
	245,100	2,715,000'#

∴ arm = 2,715,000 ÷ 245,100 = 11.7'

Earth behind the shaft

$Es = \frac{2.8 \times 27.5 \times 21.5}{2} \times 100 = 82,800\#$ $Es' = 27,600\#$

Earth pressure on shaft during earthquake

$Ec = 0.562 \times \frac{21.5 \times 27.5^2}{2} \times 100 = 457,000\#$

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CALCULATIONS FOR

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Shirahige Bashi for Tokyofu

Seismic stability at bottom of shaft

Moment about C due to hor. forces.

	Seismic forces		arm		moments
Es'	24.600	×	9.57	=	264,000
Ec	457.000	×	9.2	=	4.205,000
Sh'	248.100	×	11.7	=	2,870,000
B	307.000	×	13.8	=	4.237.000
	ΣH = 1.036.700				+11.576.000'#

Moment about C due to Vert. forces

	Vert. forces		arm		moments
S	4.800	×	6.1	=	29.000
T	480.000	×	3.35	=	1.607.000
Es	82.800	×	9.57	=	792.000
Sh	735,100	×	5.54	=	4.071.000
B	227.100	×	5.8	=	1.316.000
	ΣV = 1.529.800				Σm = −7,815.000'#

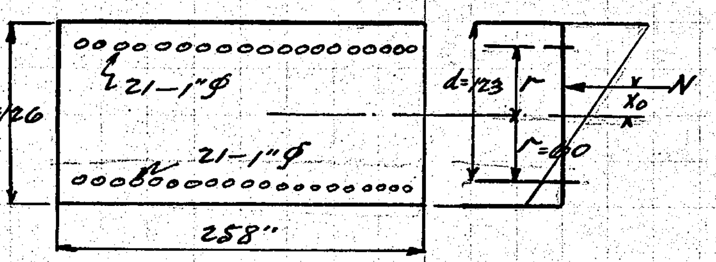
Total moment = +3.761.000'#

Moment about N.A. of Section CC'

$+3.761.000 + 1,529.800 \times \frac{10.5}{2} = +11.791.000'\#$

$\text{Eccentricity} = \frac{11.791.000}{1.529.800} = 7.71' = 92.5''$

$p_0 = 0.00102 \quad \frac{x_0}{t} = \frac{92.5}{126} = 0.734 \quad \frac{r^2}{t^2} = \frac{60^2}{126^2} = 0.227$



To find Neutral axis

$k^3 - 3\left(\frac{1}{2} - \frac{x_0}{t}\right)k^2 + 6np_0 k\frac{x_0}{t} = 3np_0\left(\frac{x_0}{t} + 2\frac{r^2}{t^2}\right)$

$k^3 + .702k^2 + 0.0515k - 0.03925 = 0$

By trial $k = 0.215 \quad j = 1 - \frac{k}{3} = 0.928$

$42 - 1''\phi = 33^{\square''}$

$p_0 = \frac{33}{258 \times 126} = 0.00102$

$L = \frac{np_0 r^2}{kt^2} + \frac{k}{12}(3-2k)$

$= \frac{15 \times 0.00102 \times 60^2}{0.215 \times 126^2} + \frac{.215}{12}(3 - 2\times .215)$

$= 0.0622$

$f_c = \frac{M}{Lbt^2} = \frac{11.791.000 \times 12}{0.0622 \times 258 \times 126^2} = 555\,\#/\square'' \text{ Ok}$

$f_s = nf_c\left(\frac{d}{kt} - 1\right) = 15 \times 555\left(\frac{123}{.215 \times 126} - 1\right)$

$= 29.400\,\#/\square'' \text{ Ok}$

$\text{Unit shear} = \frac{1.036.700}{258 \times \cancel{0.8} \times .928 \times 123} = 35.2\,\#/\square'' \text{ Ok}$

$\text{Unit bond} = \frac{1.036.700}{3.142 \times 21 \times .928 \times 123} = 138\,\#/\square'' \text{ Ok}$

(allowable bond $127 \times 1.8 = 228\,\#/\square''$)

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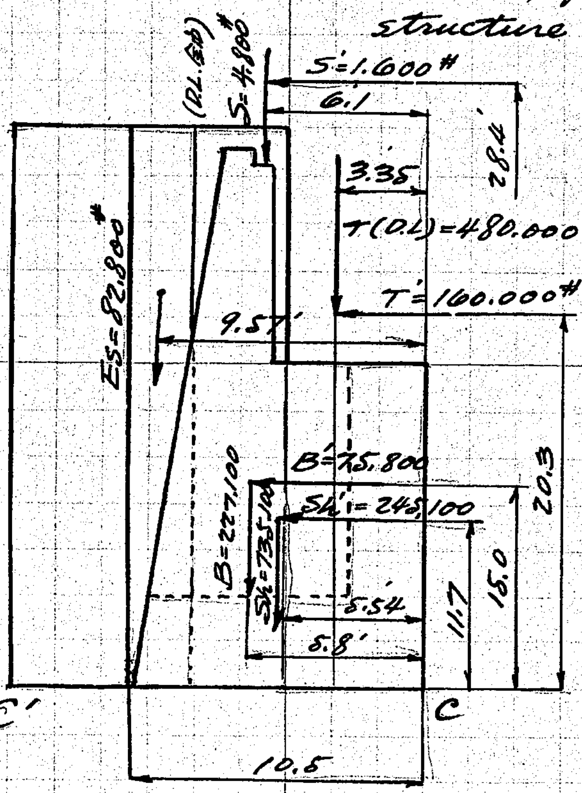
Shirahige Bashi for Tokyofu

During earthquake

Case B Seismic forces due to abutment structure and super structure acting at their center of gravity backward



Taking moment about C of Hor. forces

Seismic forces		arm		moments
S	1.600 ×	28.4	=	45.000
T'	160.000 ×	20.3	=	3.245.000
B'	75,800 ×	15.0	=	1.137.000
Sh'	245.100 ×	11.7	=	2.868.000
ΣH = 482.500				+ 7.295.000

note! + sign shows counter clockwise direction

Moment of Vert. forces about C

	Vert. forces	arm		moments
Es	82.800 ×	9.57	=	792.000
S	4.800 ×	6.1	=	29.000
T	480.000 ×	3.35	=	1.608.000
B	227.100 ×	5.8	=	1.317.000
Sh	735.100 ×	5.54	=	4.072.000
ΣV = 1.529.800				+ 7.818.000

Summary of moment about C
= 15.113.000

Point of application of B'

$$= \frac{1450 \times 22.5 + 3600 \times 12}{8050}$$

= 15'

Moment about N.A. of section CC'

$$M = 15.113.000 - 1.529.800 \times \frac{10.5}{2} = 7.083.000'^{\#}$$

$$\text{Eccentricity} = \frac{6.646.000}{1.439.800} = 55.4''$$

Since the resultant moment at bottom of shaft is by far the smaller for this case than for case B, so the section designed for the previous case is quite safe for this case.

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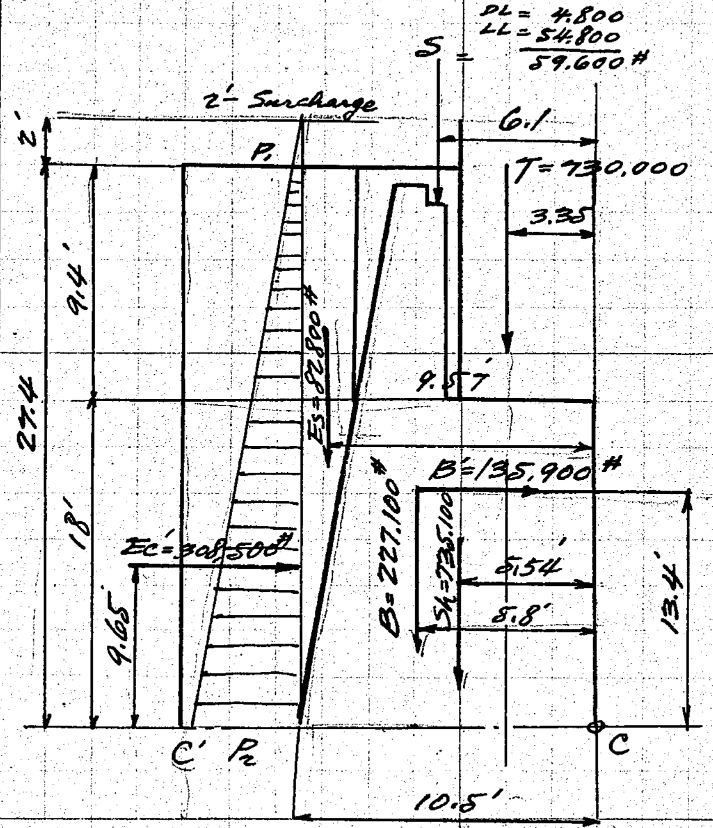
Shirahige Bashi for Tokyofu

Stability of abutment at bottom of shaft

For the case of max. Live Load



Stringer reaction on parapet wall due to live load

Electric car	$16.800 \times 1 \times 1.3$	$= 21.850^{\#}$
Motor truck	$9900 \times 2 \times 1.3$	$= 25.750$
Unif. load	$120 \times 6.1 \times (1.5+2.2)$	$= 2.720$
" "	$100 \times 12 \times (1.5+2.2)$	$= 4.440$
		$54.760^{\#}$

Call this $54.800^{\#}$

do. dead load $4.800^{\#}$

$59.600^{\#}$

Earth pressure on shaft only Ec'

$$P_1 = \frac{100 \times 2}{3} = 67 \ \#/\square'$$

$$P_2 = \frac{100 \times 29.4}{3} = 980 \ \#/\square'$$

$$\frac{67+980}{2} \times 27.4 = 14.340 \ \#/1' \text{ of width of shaft}$$

$$Ec' = 14.340 \times 21.5 = 308.800 \ \#$$

Taking moments about C

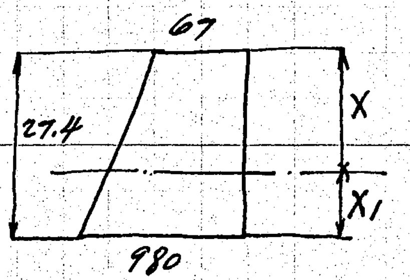
Moments due to Hor. forces

	forces	arm	moments
B'	135.900	× 13.4'	= 1.821.000
Ec'	308.500	× 9.65	= 2.985.000
	ΣH = 444.400		+ 4.806.000

Point of application of EC'

$$X = \frac{27.4(67+1960)}{3(67+980)} = 17.75'$$

$$X_1 = 27.4 - 17.75 = 9.65'$$



Moments due to Vert. forces

	forces	arm	moments
S	59.600	× 6.1	= 364.000
T	730.000	× 3.35	= 2.445.000
Es	82.800	× 9.57	= 792.000
B	227.100	× 5.80	= 1.318.000
Sh	735.100	× 5.54	= 4.070.000
	ΣV = 1.834.600		− 8.989.000 '#

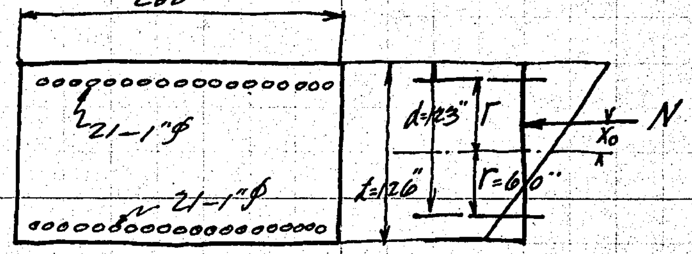
Resulting moments = −4.183.000

moment at Neutral axis of section CC'

$$M = -4.183.000 + 1.834.600 \times 5.25$$

$$= 5.447.000 \ '\#$$

$$\text{Eccentricity } x_0 = \frac{5.447.000}{1.834.600} = 2.97' = 35.7''$$



$42 - 1''\phi = 23^{\square''}$

$p_0 = 0.00102$

$$\frac{x_0}{t} = \frac{35.7}{126} = .284$$

To find the Neutral axis

$$k^3 - 3\left(\frac{1}{2} - \frac{x_0}{t}\right)k^2 + 6np_0k\frac{x_0}{t} = 3np_0\left(\frac{x_0}{t} + 2\frac{r^2}{t^2}\right)$$

$$k^3 - .648k^2 + 0.0261k = 0.03387$$

by trial $k = 0.69$ $j = 0.77$

$$L = \frac{np_0 r^2}{kt^2} + \frac{k}{12}(3-2k) = 0.098$$

$$f_c = \frac{M}{Lbt^2} = \frac{5.447.000 \times 12}{0.098 \times 258 \times 126^2} = 163 \ \#/\square'' \text{ OK}$$

$$f_s = nf_s\left(\frac{d}{kt} - 1\right) = 15 \times 163\left(\frac{123}{.69 \times 126} - 1\right)$$

$$= 1,014 \ \#/\square'' \quad \text{OK}$$

$$\text{Unit shear} = \frac{444.400}{258 \times 0.77 \times 123} = 18.2 \ \#/\square'' \text{ OK}$$

$$\text{Unit bond} = \frac{444.400}{3.142 \times 21 \times .77 \times 123} = 71 \ \#/\square'' \text{ OK}$$

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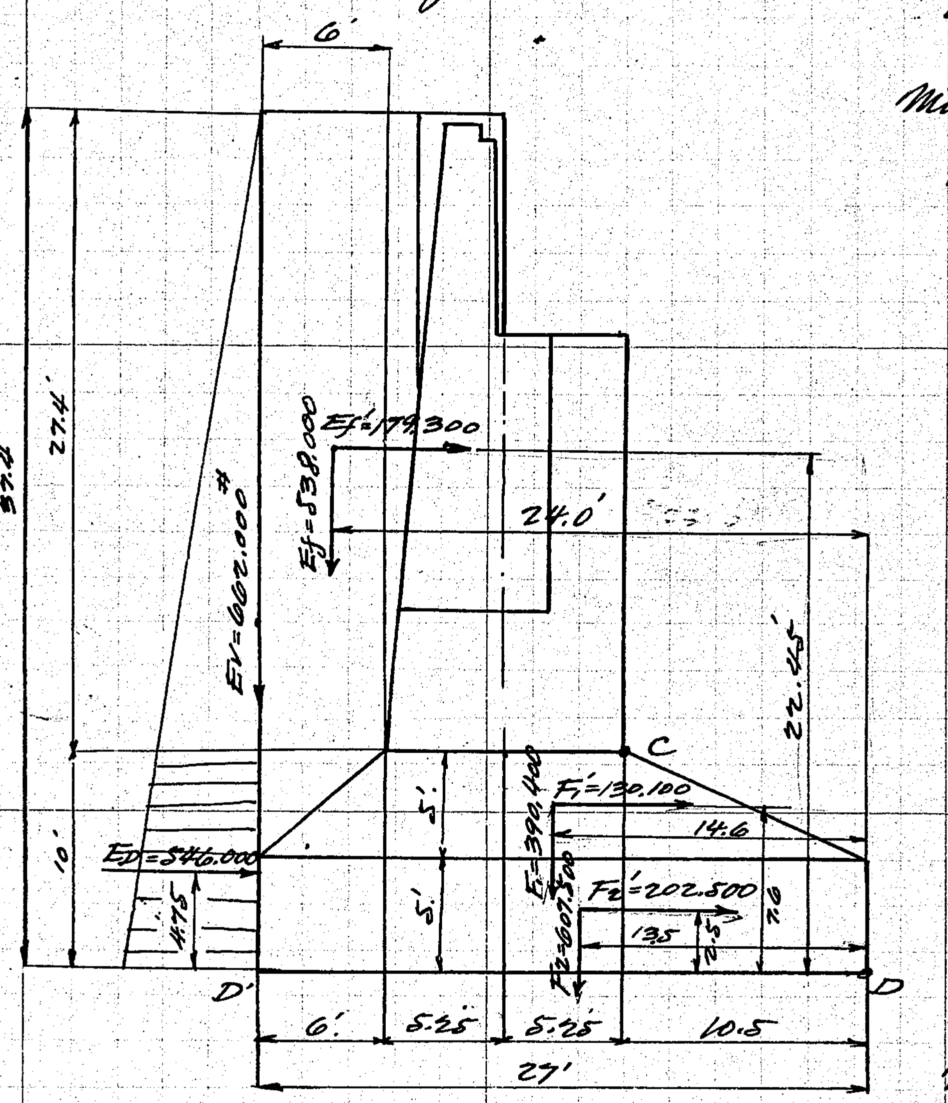
Shirahige Bashi for Tokyofu

Foundation

During earthquake Case A



Seismic forces due to earth fill and abutment structure acting at their center of gravity toward the river side.

Moment about C transmitted to foundation from shaft. due to Hor. forces

$\Sigma H = 1.036.700^{\#}$ $M_h = +11.576.000'^{\#}$

note! + sign shows clockwise direction

due to Vert. forces

$\Sigma V = 1.529,800$ $M_v = -7.815.000$

$E_f = 6' \times 29.9' \times 30 \times 100 = 538.000^{\#}$

$E_f' = 538.000 \times \frac{1}{3} = 179.500^{\#}$

$F_1 = \frac{1}{2}(22 \times 10.5 + 30 \times 27) \times 5 \times 150 = 390.400^{\#}$

$F_1' = 390.400 \times \frac{1}{3} = 130.100^{\#}$

$F_2 = 30 \times 27 \times 5 \times 150 = 607.500^{\#}$

$F_2' = 607.500 \times \frac{1}{3} = 202.500^{\#}$

$P_1 = 0.562 \times 100 \times 27.4 = 1.540^{\#}/\square'$

$P_2 = 0.562 \times 100 \times 37.4 = 2.100$ "

$E_D = \frac{1}{2}(1540 + 2.100) \times 10 \times 30 = 546.000^{\#}$

$E_V = \frac{1}{2} \times 0.562 \times 100 \times 37.4^2 \times 0.562 \times 30 = 662.000^{\#}$

(for $K = \frac{1}{3}$ $\beta = 29°20'$ $\tan\beta = 0.562$)

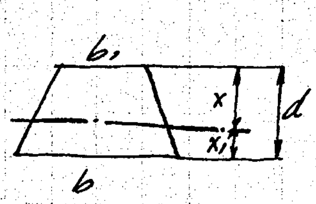
Moment about D due to Hor forces

moment from shaft $+11.576.000 + 1.036.700 \times 10$
$= +21,943,000'^{\#}$

E_f'	179.000	× 22.45	= + 4.018.000
F_1'	130.000	× 7.6	= + 998.000
F_2'	202.500	× 2.5	= + 506.000
E_D	546.000	× 4.75	= + 2.595.000
from shaft	1.036.700		
			+ 30,060,000

$\Sigma H = 2.094.300$

Width of foundation 30' assumed



$$x_1 = \frac{d(b + 2b_1)}{3(b + b_1)} = \frac{10(2100 + 3080)}{3(2100 + 1540)} = 4.75'$$

due to Vert. forces

moment from shaft $-7.815.000 - 1.529.800 \times 10.5$
$= -23,878,000'^{\#}$

Ef	538,000	× 24.0	= − 12,910,000
F1	390.400	× 14.6	= − 5,700,000
F2	607.500	× 13.5	= − 8.200.000
EV	662.000	× 27.0	= − 17.870.000
from shaft	1.529.800		
			− 68,558,000'#

$\Sigma V = 3,727.700$

Summ. $= -38,498,000'^{\#}$

Section modulus of bed area

$$Q = \frac{bd^2}{6} = \frac{30 \times 27}{6} = 3,644 \ (ft^3)$$

Bed area = 30 × 27 = 810 □'

Moment about center line of base

$M = -38,498,000 + 3.727.7000 \times 13.5$

$= +11.826.000'^{\#}$

Ecc. $\varepsilon = \frac{11.826.000}{3,727.700} = 3.18'$

Max toe pressure

$$= \frac{3.727.700}{810 \times 2240} \pm \frac{11.826.000}{3644 \times 2240}$$

$= 2.055 \pm 1.45 = 3.51$ tons/□' Comp. OK

or = 0.61 " "

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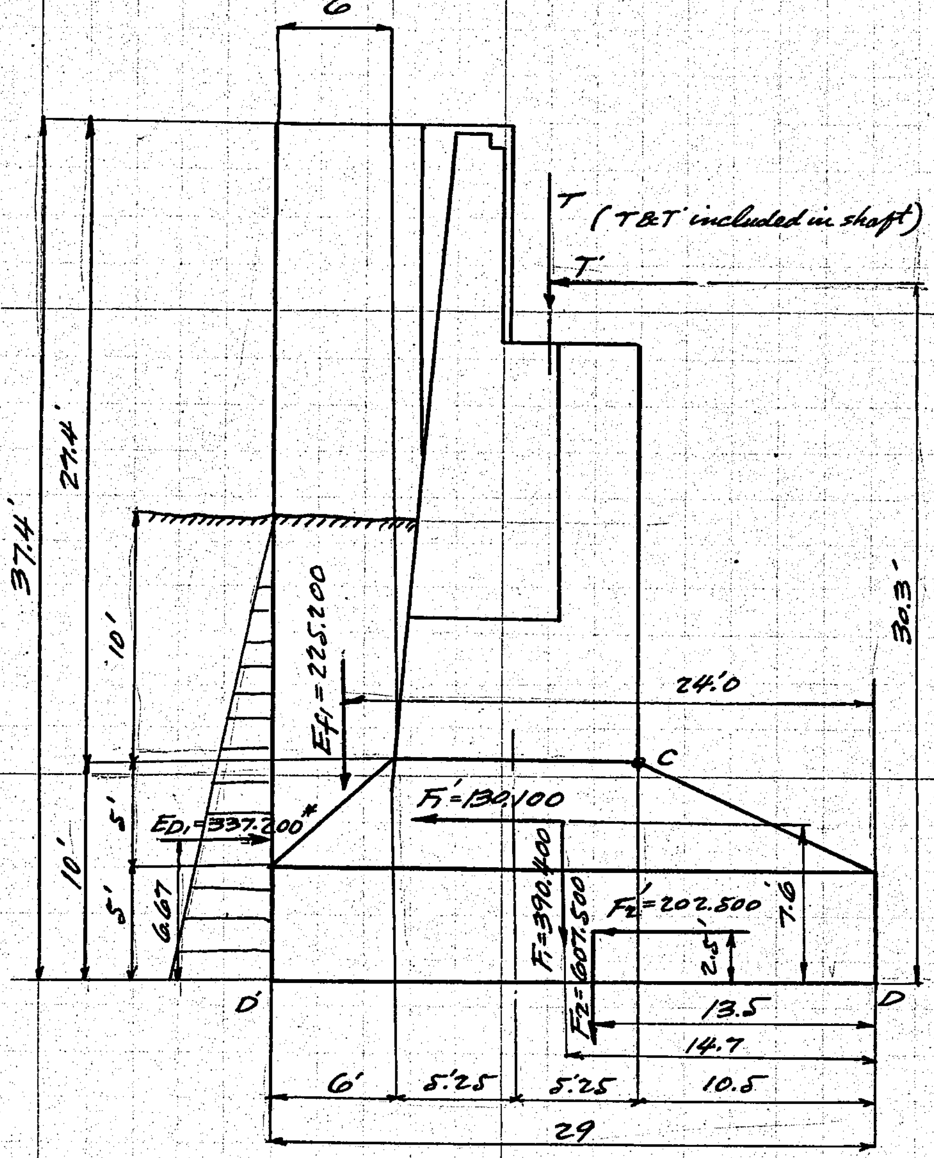
Shirahige Bashi for Tokyofu

During earthquake Case B



$E_{f_1} = 100 \times 12.5 \times 6 \times 30 = 225.200$ #

$E_{D_1} = \frac{.562 \times 100 \times 20^2}{2} \times 30 = 337.200$ #

$E_{DV} = 337.200 \times 0.562 = -189.700$ #

Seismic forces due to abutment structure and super structure acting at their center of gravity backward

Moment about C transmitted to foundation from shaft

due to Hor. forces

$\Sigma H = 482.800$ $M_h = +7.295.000'$ #

note: sign shows counter clockwise dir.

due to Vert. forces

$\Sigma V = 1.529.800$ $M_V = +7.818.000$

Moment about D

due to Hor. forces

moment from shaft		= +7.295.000 + 482.500 × 10	
ΣH =	482.500 ×		= + 12.120.000'#
T' =	130.000 ×	30.5	= + 3.965.000
F_1' =	130.100 ×	7.6	= + 989.000
F_2' =	202.500 ×	2.5	= + 506.000
E_{D_1} =	− 337.200 ×	6.67	= − 2.250.000
ΣH =	607.900		ΣM_h = + 15.330.000

due to Vert. forces

moment from shaft		= + 7.818.000 + 1.529.800 × 10.5	
from shaft ΣV =	1.529.800		= + 23.881.000
E_f	225.200 ×	24.0	= + 5.340.000
F_1	390.400 ×	14.7	= + 5.732.000
F_2	607.500 ×	13.5	= + 8.200.000
E_{DV}	− 189.700 ×	27'	= − 5.120.000
ΣV =	2.563.200		ΣM_V = +38.033.000'#

Summary of moment = +53,363.000'#

Moment about center line of base

$M = 53,363,000 - 2.563,200 \times 13.5$
$= +18,760,000$'#

Ecc. $e = \frac{18.760.000}{2.563.200} = 7.32$

Neglecting tension

$a = 13.5 - 7.32 = 6.18$

Max. toe pressure

$= \frac{2 \times 2.563.200}{3 \times 6.18 \times 30 \times 2240} = 4.11$ tons/□' OK

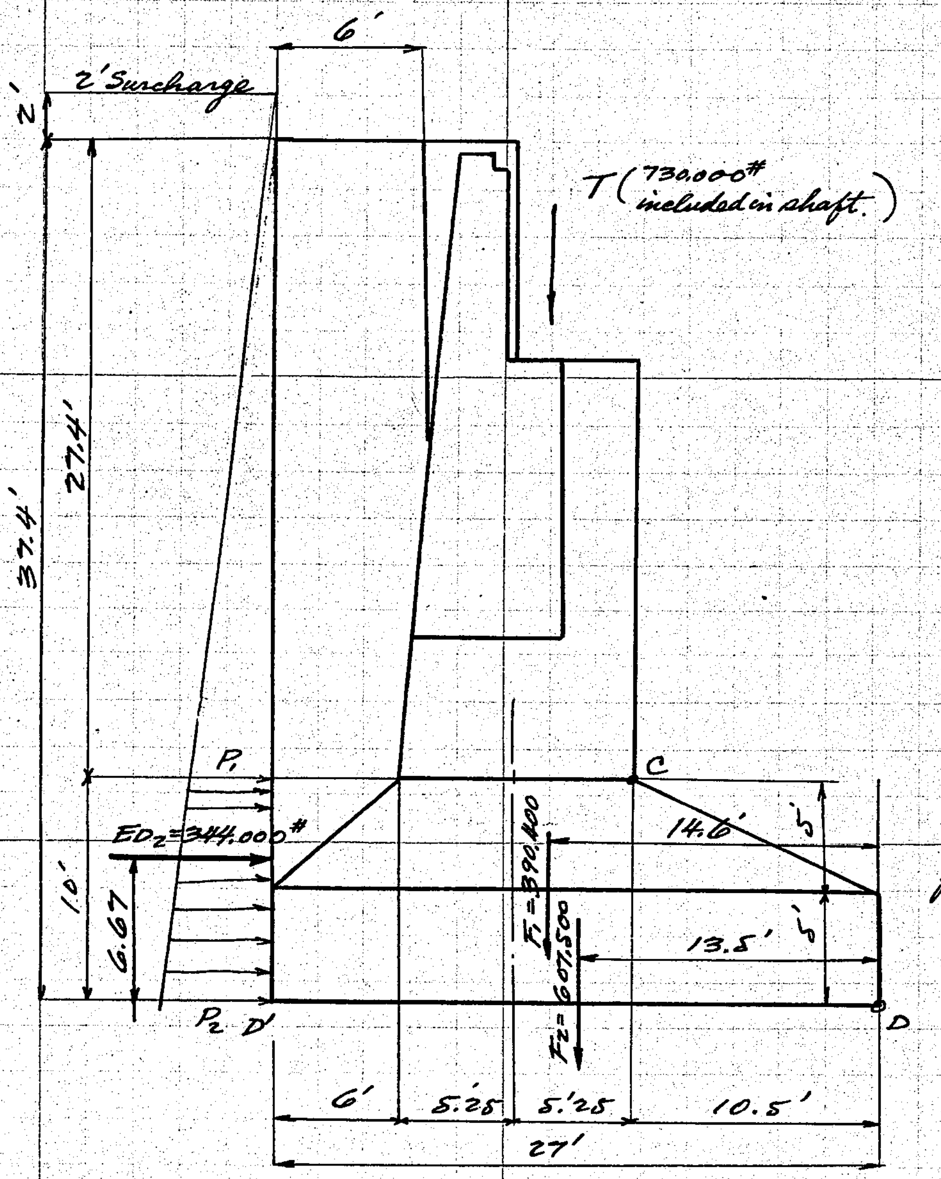
Tension at toe

$= \frac{2.563.200}{810 \times 2240} \pm \frac{18.760.000}{3644 \times 2240}$

$= 1.415 \pm 2.230 = +3.645$ tons/□' Comp.
or -0.885 " tension

Shirahige Bashi for Tokyofu 38

Stability of abutment for the case of Max. Live Load.



$P_1 = \frac{100 \times 29.4}{3} = 980^{\#/\square'} \times 30 = 29.400$

$P_2 = \frac{100 \times 39.4}{3} = 1.314^{\#/\square'} \times 30 = 39.400$

$ED_2 = \frac{29.400 + 39.400}{2} \times 10 = 344.000^{\#}$

moment about C transmitted for shaft

due to Hor. forces

$\Sigma H = 444.400^{\#}$ $M_h = +4.806.000'^{\#}$

note! + sign shows clockwise

due to Vert forces

$\Sigma V = 1.834.600^{\#}$ $M_V = -8.989.000'^{\#}$

Moment about D

due to Hor. forces

moment from shaft $= +4.806.000 + 444.400 \times 10$
$= +9.250.000'^{\#}$

ED_2 344.000 × 6.67 = +2.295.000

from shaft ΣH 444.400 — +11.545.000'#

ΣH = 788.400

due to Vert. forces

moment from shaft $= -8.989.000 - 1.834.600 \times 10.5$
$= -28.252.000$

F_1 390.400 × 14.6 = −5.700.000

F_2 607.500 × 13.5 = −8.205.000

from shaft 1.834.600 — −42,157.000

ΣV = 2,832.500

Summary of M. = −30.612.000'#

Moment about Center line of base

$M = -30.612.000 + 2.832,500 \times 13.5$
$= +7.628.000'^{\#}$

Ecc. $e = \frac{7.628.000}{2.832,500} = 2.69'$

Max. Toe press. $= \frac{2.832.500}{810 \times 2240} \pm \frac{7.628.000}{3644 \times 2240}$

$= 1.56 \pm .94$

$= 2.5$ tons/□' Comp.

Heel press. or $= 0.62$ " "

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MADE BY K Inaba DATE 2-6-17 FILE NO
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CALCULATIONS FOR

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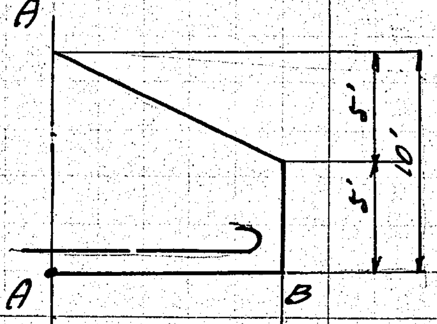
Shirahige Bashi for Tokyofu
Cantilever footing at toe

Moment at section A-A
During earthquake (case A) See page 66

Intencity of upward pressure at B = 3.51 tons/□' = 7,860 #/□'
" " " " A = $\frac{(3.51-0.61)(27-10.5)}{27}$ + .61
= 2.38 tons/□' = 5.330 #/□'
Intencity of downward press. due to dead wt at B = 5x150 = 750"
" " " " " A = 10x150 = 1.500"



Max. moment at A = 57.440 x 5.77 = 331.400'#/ft strip
" shear at A = 57,440#

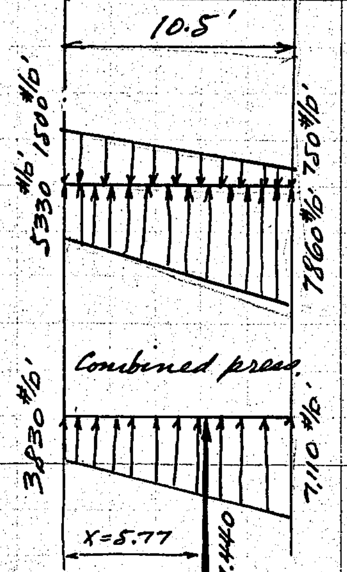
$p = \frac{1.57}{114 \times 12} = 0.0015$ k = 0.187 j = 0.938

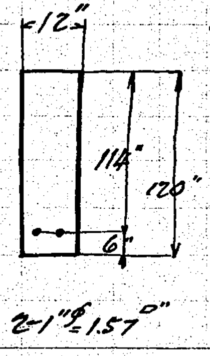
Stress in steel = $\frac{331.400 \times 12}{1.57 \times 0.938 \times 114}$ = 23.700 #/□" OK

Stress in conc = $\frac{23.700 \times 0.187}{15(1-0.187)}$ = 364 #/□" OK

Unit shear = $\frac{57.400}{12 \times 0.938 \times 114}$ = 44.7 #/□" OK

Unit bond = $\frac{57.400}{3.142 \times 2 \times .938 \times 114}$ = 85.4 #/□" OK





$x = \frac{10.5(3830+14220)}{3(3830+7.110)} = 5.77$

Resultant Press. = $\frac{7.110+3830}{2} \times 10.5$ = 57.440#

Case of Max. Live Load
Intencity of upward press. at B = 2.50 tons/□' = 5.600 #/□'
" A = $\frac{(2.50-.62)(27-10.5)}{27}$ + 0.62
= 1.67 tons/□' = 3.740 #/□'

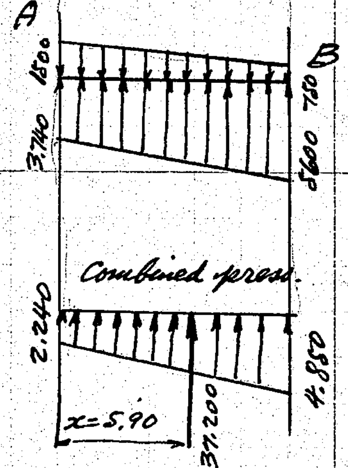
Moment at A = 37.200 x 5.90 = 219.500'#
Max. end shear = 37.200#

Stress in steel = $\frac{219.500 \times 12}{1.57 \times 0.938 \times 114}$ = 15.680 #/□" OK

Stress in Conc = $\frac{15.680 \times 0.187}{15(1-0.187)}$ = 241 #/□" OK

Unit shear = $\frac{37.200}{12 \times 0.938 \times 114}$ = 29 #/□" OK

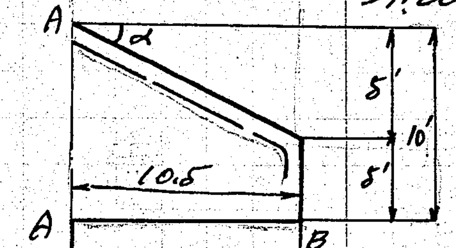
Unit bond = $\frac{37.200}{3.142 \times 2 \times .938 \times 114}$ = 55.4 #/□" OK



$x = \frac{10.5(2240+9700)}{3(2240+4880)} = 5.90$

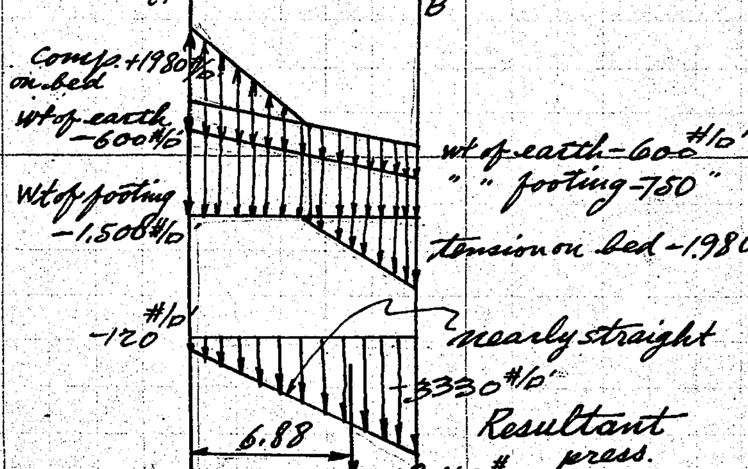
Resultant press = $\frac{4.880+2240}{2} \times 10.5$ = 37.200#

Moment at section A-A
During earthquake (case B) tension on toe.
Negative pressure on foundation at toe being neglected in the previous calcuration, but according to cohesion between the foundation and concrete piles and caissons, it should be taken into accoul for safty.



Tension at toe = 0.885 tons/□' = 1.980 #/□'
dist. of 0 point from B = 27 x $\frac{.885}{3.645+.885}$ = 5.28'

Compression at A approx. equal 1.980 #/□'
Max. negative moment at AA = -18.110 x 6.88
= -124.600'#
Max. shear at AA = 18.110#



$\cos\alpha = \frac{10.5}{\sqrt{10.5^2+5^2}} = 0.903$

Steel area reqd = $\frac{M}{f_s jd (\cos\alpha)}$ = $\frac{124.600 \times 12}{30600 \times 0.95 \times 118 \times 0.903}$
= 0.483 □"/ft strip

Use 1'-7/8"φ = 0.601

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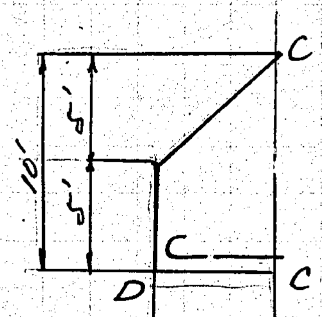
Shirahige Bashi for Tokyofu

Cantilever footing at heel

During earthquake (Case B)

Intencity of upward press. at D = 4.11 tons/□' = 9.200 #/□'

" " " at C = $\frac{4.11 \times 12.54}{18.54}$ = 2.78 tons/□' = 6.230 #/□'



Downward press. due to earth at D = 100 × 32.4 = 3.240 #/□'

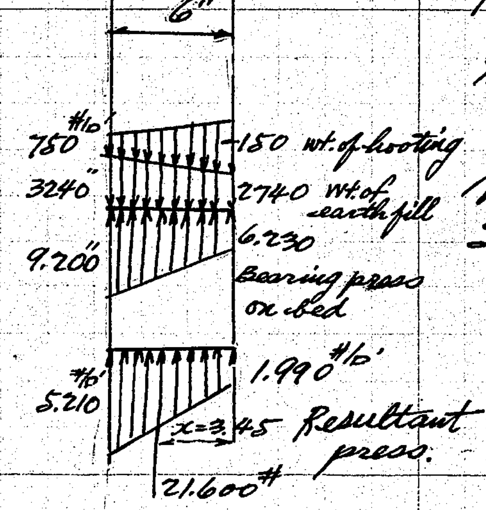
" C = 100 × 27.4 = 2.740 "

" " wt. of footing at D = 5 × 150 = 750 "

" " " " C = 10 × 150 = 1500 "

Resultant upward force = $\frac{5.210+1.990}{2}$ × 6' = 21.600 #

X = $\frac{6(1990+10.420)}{3(1990+5.210)}$ = 3.45



Max. Moment at C = 21.600 × 3.45 = 74.500 '#

Steel area req'd = $\frac{74.500 \times 12}{30600 \times .95 \times 116}$ = 0.265 □"

Use 1" ∅ 2'-0" c. to c. = 0.3927 □"

During earthquake (Case A)

Tension on foundation bed at D = 0.61 ton/□' = 1.370 #/□'

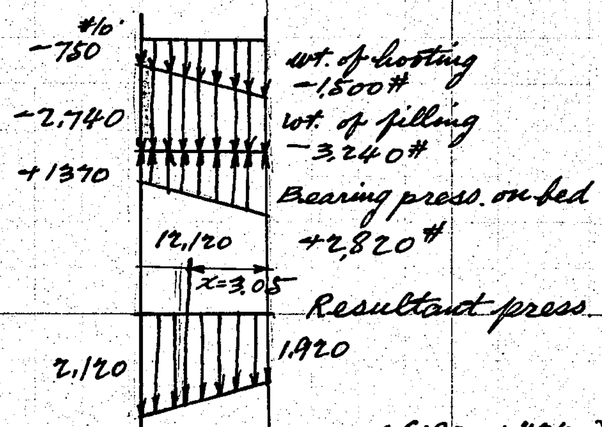
" C = 2,820

Weight of earth fill " D = 3.240

" " C = 2.740

" footing " D = 750

" " " C = 1,500



Resultant force = $\frac{2120+1.920}{2}$ × 6 = 12.120 #

Moment at C = 12.120 × 3.05 = 37.000 '#

Section modulus at section C-C

$\frac{12 \times 120^2}{6}$ = 28.800 $(")^3$

Extream fiber stress of concrete tension or comp.

= $\frac{37000 \times 12}{28.800}$ = 15.4 #/□" OK

x = $\frac{6(1920+4240)}{3(1920+2120)}$

= 3.05

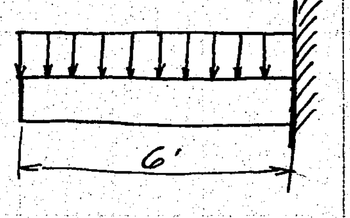
No reinforcement needed.

press. at C = $\frac{6(3.51-0.61)}{27}$ + 0.61

= 1.26 tons/□' = 2.820 #/□'

Wing wall Let us the figure the wing wall as a horizontal strip of cantilever beam spanning 7' at bottom and 8' at middle to top.

Earth fill 27.4



Outward earth press. on beam = $\frac{100 \times 27.4}{3}$ = 914 #/□'

Inward " " " = $\frac{100 \times 15}{3}$ = -500

414 #/□'

Moment at fixture = $\frac{1}{2}$ × 414 × 7² = 10.150 '#

End shear = 414 × 7 = 2.900 #

Steel area req'd = $\frac{10.150 \times 12}{17000 \times \frac{7}{8} \times 40}$ = 0.17 □"

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CALCULATIONS FOR

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Shirahige Bashi for Tokyofu

During earthquake

Earth pressure on beam
$= 0.562 \times 100 \times 27.4 = 1.540^{\#/\prime}$

Seismic force on beam
$= 4.0 \times 150 \times \frac{1}{3} = 200^{\#/\prime}$
total $1.740^{\#/\prime}$

Moment at fixture $= \frac{1740 \times 7^2}{2} = 42.600'^{\#}$
Steel req'd during earthquake $= \frac{42.600}{30600 \times \frac{7}{8} \times 40} = 0.477^{\square''}$

Use 2-5/8"ϕ = $0.614^{\square''}$
Use the same reinforcement on out side.

During earthquake

At 15' from top of wall
Earth press. on beam
$= 0.562 \times 100 \times 15 = 842^{\#/\prime}$
Seismic force $= 200''$
$1042^{\#/\prime}$

Moment at fixture $= \frac{1042 \times 7^2}{2} = 25.500'^{\#}$
Steel req'd $= \frac{25.500 \times 12}{30600 \times \frac{7}{8} \times 40} = 0.286^{\square''}$
Use 1-5/8"ϕ = $0.307^{\square''}$

At 9' from top of wall
Earth press. on beam
$= 0.562 \times 100 \times 9 = 506^{\#/\prime}$
Seismic force $= 200''$
$706^{\#/\prime}$

Moment at fixture $= \frac{706 \times 7^2}{2} = 17.300'^{\#}$
Steel req'd $= \frac{17.300 \times 12}{30600 \times \frac{7}{8} \times 40} = 0.194^{\square''}$
Use 5/8"ϕ 18" c. to c.

At 23' from top of wall
Earth press. on beam
$= 0.562 \times 100 \times 23 = 1290^{\#/\prime}$
Seismic force $= 200$
$1490^{\#/\prime}$

Moment at fixture $= \frac{1490 \times 7^2}{2} = 36.500'^{\#}$
Steel req'd $= \frac{36.500 \times 12}{30600 \times \frac{7}{8} \times 40} = 0.409^{\square''}$
Use 5/8"ϕ 9" c. to c. $= 0.409^{\square''}$

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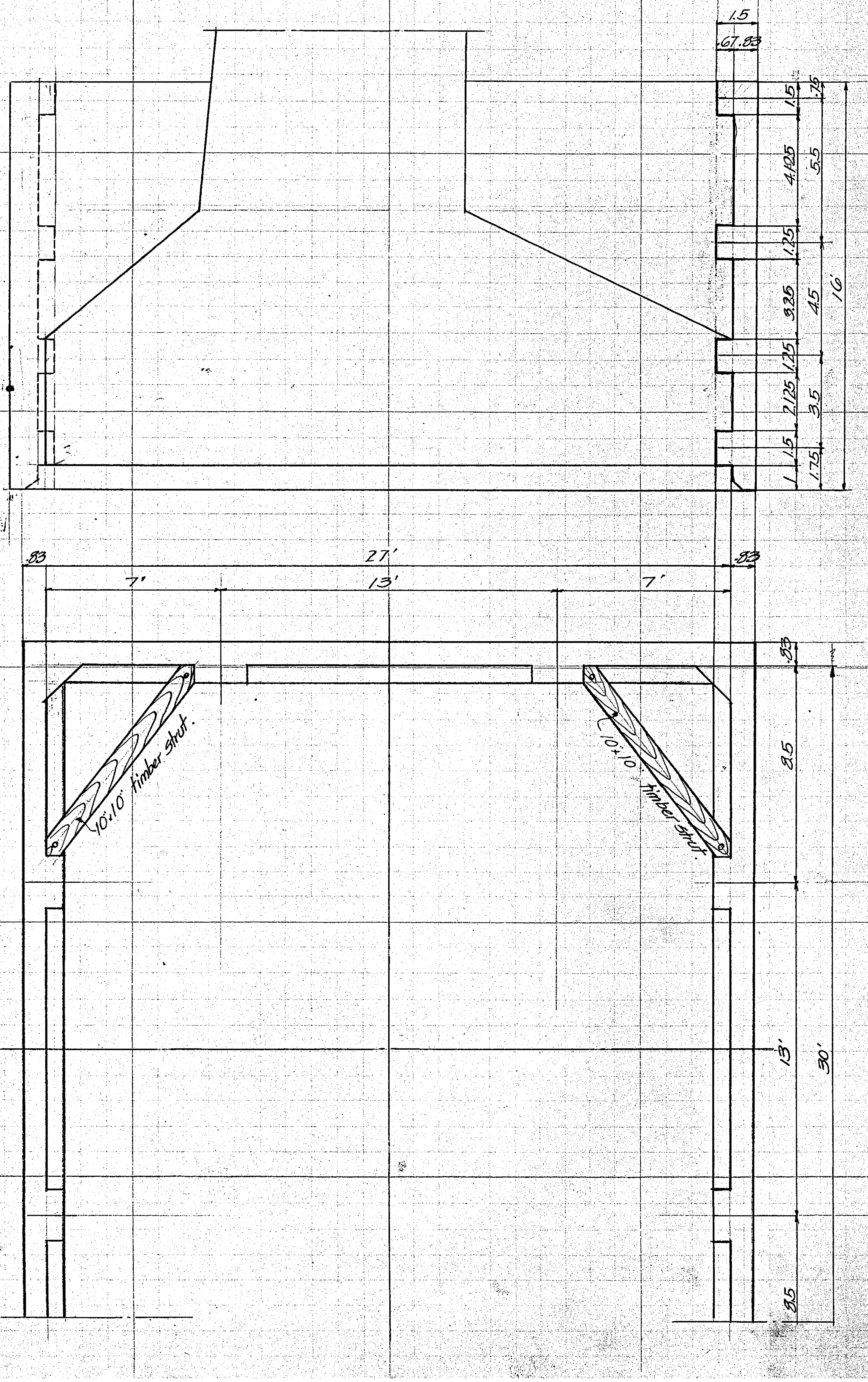
Shirahige Bashi for Tokyofu



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CALCULATIONS FOR

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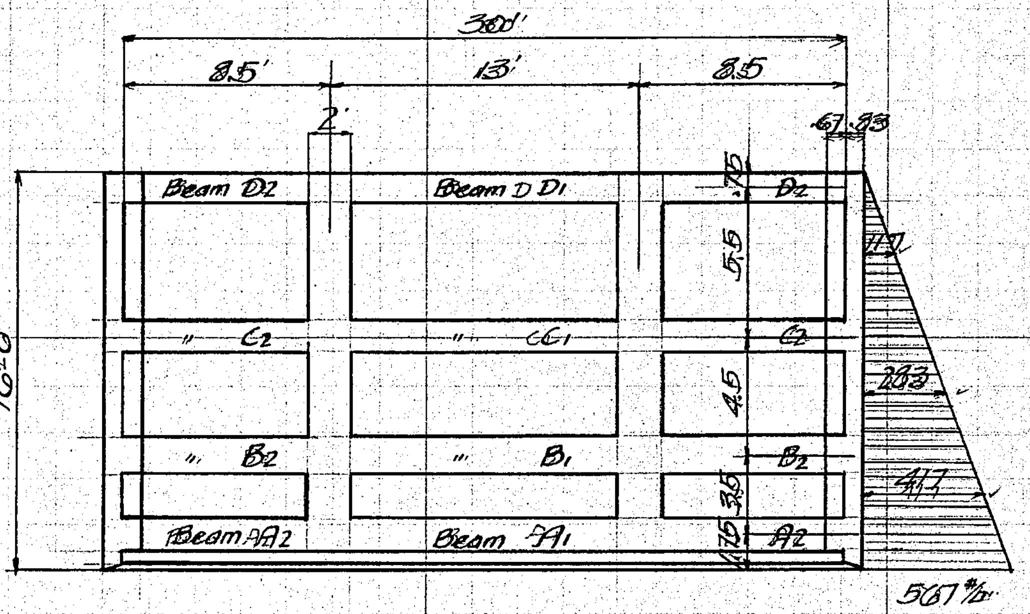
Shirahige Bashi for Tokyofu
Design of Open Caisson for foundation of abutment



Earth press. at bottom of caisson
$P_1 = \frac{100 \times 17}{3} = 567$ #/□'

at depth 14.25 $P_2 = \frac{100 \times 12.5}{3} = 417$ "
" 10.75 $P_3 = \frac{100 \times 8.5}{3} = 283$ "
" 6.25 $P_4 = \frac{100 \times 3.5}{3} = 117$ "

Slab between Beams A and B
Span 3'-6" thickness = 10"
Moment $= \frac{417 \times 3.5^2}{10} = 511$ '#/ft strip

End Shear = 417 × 1.75 = 730 #/"
Steel req'd $= \frac{511 \times 12}{17000 \times \frac{7}{8} \times 9} = 0.0458$ □"/ft

Use 1 - ½"φ = 0.196 □"
Unit shear $= \frac{730}{12 \times \frac{7}{8} \times 9} = 7.7$ #/□" Ok

Slab between Beams B and C
Span 4'-6", thickness 10"
Moment $= \frac{283 \times 4.5^2}{10} = 573$ '#

End shear = 283 × 2.25 = 637 #
Steel req'd $= \frac{573 \times 12}{17000 \times \frac{7}{8} \times 9} = 0.0513$ □"/ft

Use 1 - ½"φ = 0.196 □"

Slab between Beams C and D
Span 5'-6" thickness 10"
Moment $= \frac{117 \times 5.5^2}{10} = 354$ '#/ft strip

End shear = 117 × 2.75 = 322 #/"
Use 1 - ½"φ = 0.196 □"

Beam A1 span 13' Load $= \frac{417+567}{2} \times 3.5 = 1,720$ #/ft
moment $= \frac{1,720 \times 13^2}{10} = 29,100$ '#

End shear = 1720 × 6.5 = 11,200 #
Steel req'd $= \frac{29,100 \times 12}{17000 \times \frac{7}{8} \times 16} = 1.466$ □"

Use 4 - ¾"φ = 1.767 □"
Unit shear $= \frac{11,200}{15 \times \frac{7}{8} \times 16} = 53.4$ #/□" Ok

Unit bond $= \frac{11,200}{2.356 \times 4 \times \frac{7}{8} \times 16} = 84.8$ #/□" Ok

Beam A2 span 8.5' Load = 1,720 #/ft
moment $= \frac{1720 \times 8.5^2}{10} = 12,420$ '#

End shear = 1720 × 4.25 = 7,310 #
Steel req'd $= \frac{12,420 \times 12}{17000 \times \frac{7}{8} \times 16} = 0.625$ □"

Use 2 - ¾"φ = 0.8836 □"
Unit shear $= \frac{7,310}{15 \times \frac{7}{8} \times 16} = 34.8$ #/□" Ok

Unit bond $= \frac{7310}{2.356 \times \frac{7}{8} \times 16} = 110.7$ #/□" Ok

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Shirahige Bashi for Tokyofu

Beam B1

Span 13.0' Load $= \frac{283+417}{2} \times 4 = 1400^{\#}$

moment $= \frac{1400 \times 13^2}{10} = 23.660'^{\#}$

End shear $= 1400 \times 6.5 = 9.100^{\#}$

Steel req'd $= \frac{23.660 \times 12}{17000 \times \frac{7}{8} \times 16} = 1.192^{\square''}$

Use 3-3/4"ϕ = 1.325□"

Unit shear $= \frac{9100}{15 \times \frac{7}{8} \times 16} = 43.3^{\#/\square''}$ ok

Unit bond $= \frac{9100}{2.356 \times 3 \times \frac{7}{8} \times 16} = 92^{\#/\square''}$ ok

Beam B2

Span 8.5' Load $= 1400^{\#/I}$

moment $= \frac{1400 \times 8.5^2}{10} = 10.100'^{\#}$

End shear $= 1400 \times 4.25 = 5.950^{\#}$

Steel req'd $= \frac{10.100}{17000 \times \frac{7}{8} \times 16} = 0.425^{\square''}$

Use 2-3/4"ϕ = 0.883□"

Beam C1

Span 13' Load $= \frac{117+283}{2} \times 5 = 1.000^{\#/I}$

moment $= \frac{1000 \times 13^2}{10} = 16.900'^{\#}$

End shear $= 1000 \times 6.5 = 6.500^{\#}$

Steel req'd $= \frac{16900 \times 12}{17000 \times \frac{7}{8} \times 16} = 0.852^{\square''}$

Use 2-3/4 = 0.8836□"

Unit shear $= \frac{6.500}{15 \times \frac{7}{8} \times 16} = 32^{\#/\square''}$ ok

Unit bond $= \frac{6.500}{2 \times 2.356 \times \frac{7}{8} \times 16} = 95.5^{\#/\square''}$ ok

Beam C2

Use 2-3/4"ϕ = 0.8836□"

Beam D1

Use 2-3/4 = 0.8836□"

Beam D2

Use 2-3/4"ϕ

Diagonal strut

reaction from $A_1 = 11,200^{\#}$

" " $A_2 = 7.310$ "

$18,510^{\#}$

Angle of inclination $= 45^\circ$

Axial compression $= 18,510 \times \sec 45^\circ = 18510 \times 1.414 = 26.200^{\#}$

10"×10" wood strut

Unit compression $= \frac{26.200}{100} = 262^{\#/\square''}$ ok