Kakogawa Bridge, Kakogawa Machi, Hyogoken.

The proposed new bridge is to be designed for new loading issued by the department of the Home Affairs. To replace the present bridge which was designed for light loading. The established new embankments for Kakogawa will have a skew of 8° on the left bank; 19° on the right bank to the ℄ of new bridge. The amount of skew of bridge was discussed and finally we agreed to make the bridge 15° skew to the give minimum obstruction during flood time. At 400 Ken up stream there is Imperial Railway bridge which is composed of 18 spans of 70' clear girder spans between piers. The piers are built square to the bridge tangent, and they will give considerable obstruction on account of skew to the stream line.

The new bridge will have 69'-11" ~~70'-7"~~ about clear between piers at mid-height of bridge shaft and the piers parallel to stream line, which is more faborable than the Imperial Railway bridge during flood time. And the new layout was approved by the official, Mr. Otazaki, of the Osaka Branch Office of the Naimucho and also by Mr. Yamanouchi, resident Engineer of Kakogawa Improvement Office.

The new bridge is composed of 17 spans of 72'-10½" between ℄ of bearings and the total length of 1259'-10½" between center to center of end bearings of end girders. The spans will be deck girders with concrete slab; the roadway is 21' clear between curbs; the "handrail" are made of concrete; pavement will be the creosoted wood block with ½" asphaltic cement sand mortar cushion under the pavement.

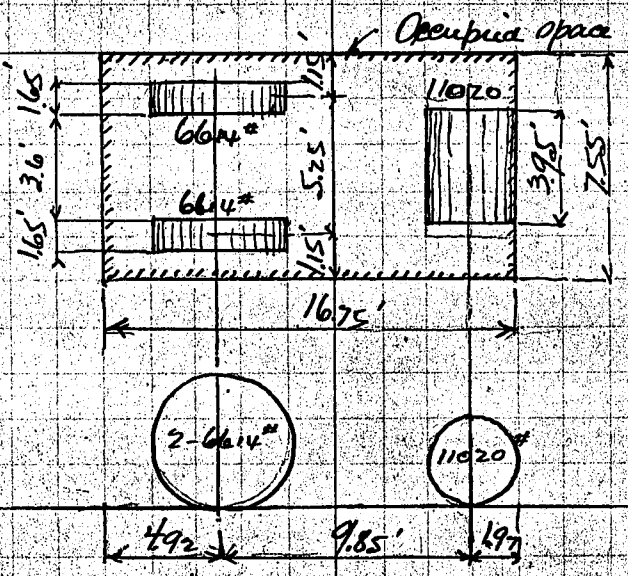
Assumed loadings.

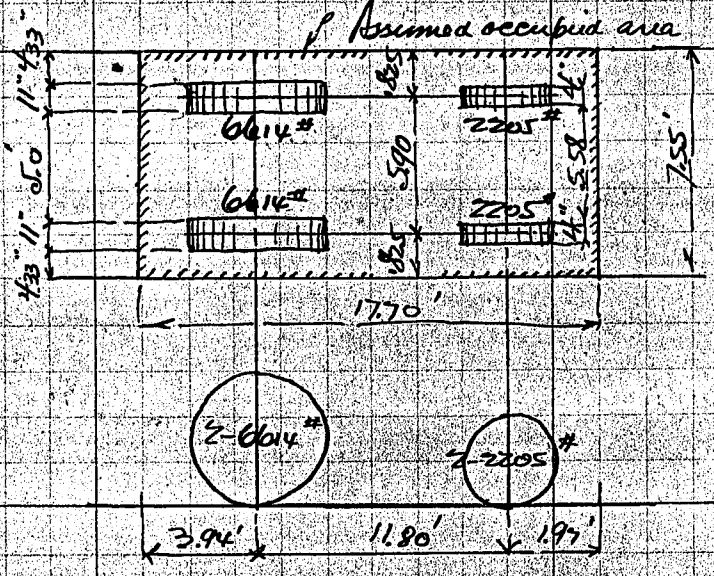
Uniform live load $q\ kg/m^2 = \frac{100.000}{170+l}$ with max of 500 kg/m².
where l = the span length of bridge in meter.

Equivalent formula for ft-lbs units $\#/0' = \frac{2048}{170+\frac{l}{3.28}}$ where l = span length in ft.

The max. are give 102.4 #/0' for spans under 30 meters.

Load Roller Concentration 11000 kg (24250#)





Single road roller is assumed on one span; consider no impact.

Outside of these occupied space of road roller and trucks, the uniform load is to be added to cover whole width of roadway.

When trucks are running in series with more than 2 trucks, consider no impact, unless otherwise with max. impact of 25%. Impact formula is as follows

$I = \frac{L}{L+D}$ L = Live load, D = Dead Load.

Assumed Working Stresses and Constants.

Constants

$fs/fc = n = 15$ Temperature change ±40°F for concrete
Expansion Coefficient for Concrete = 0.00000055
" " " Steel = 0.00000060

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Working strength of 1:2:4 concrete

		I	II
Bearing		46 kg/cm² = 654.3 #/□"	56 kg/cm² = 796.5 #/□"
Direct Compression		35 — 497.8	42 — 597.4
Compression due to moment only	positive moment	46 — 654.3	56 — 796.5
	negative moment	53 — 753.8	56 — 796.5
Compression due to moment & direct comp combined	Slab	35 — 497.8	42 — 597.4
	Arch	46 — 654.3	56 — 796.5
Punching shear		9 — 128.0	10.5 — 149.3
Shear	without web reinforcement	3 — 42.7	3.5 — 49.8
	with web reinforcement	9 — 128.0	10.5 — 149.3
Bond Stress (plain bar).		5.5 — 78.2	6.5 — 92.5

Working Stength of Reinforcing Bars

	I	II
Tension	1200 kg/cm² 17067 #/□"	1500 kg/cm² 21335 #/□"
Compression	1200 — 17067	1500 — 21335
Shearing	800 — 11378	1000 — 14223

Note Group I not considering wind, temperature and brake
Group II Considering one or all of wind, temperature and brake.

The above working stresses are to be issued by the department of the Home Affairs, at near feture; those will be the limiting strengths for concrete and steel; To take on side of safty, and to make the figures in round number of inch-lbs units, let us assumed the following working stress;

Concrete stress
- Compresion due to moment: positive 600 #/□"; negative 750 #/□"
- Shear without web reinforcement 40 #/□"
- " with " " 120 #/□"

Steel Stress
- Tensile strength 16000 #/□"
- Compression 600 × 15 = 9000 #/□"
- 750 × 15 = 11250 #/□"
- Shear 10.000 #/□"

Working Strength of Structural Steel

Tension	Axial tension on net section	16.000 #/□"
Compression	Axial compresion on gross section of cols	= 16000 − 70 l/r
	with a max of	14000
	Direct Compression on Steel Casting	16000
Bending	Bending on extreme fibre of rolled shapes and built sections & Girders & Steel casting; net	16000
	Extreme fibre of pins	24.000
Shearing	Shearing on shop driven rivets and pins	12.000
	Shearing on field driven rivets and turned bolts	10.000
	Shearing on plate girder webs gross section	10.000
Bearing	Shop rivets & pins	24.000
	field rivets & turned bolts	20.000
	Expansion rollers per lin. inch, d- dia. of roller	600 d

The above working stresses are adopted from AREA specification 1910. Let us follow the same specification for the general design with slight modification on account of highway ~~bridge~~ instead of railway bridge.

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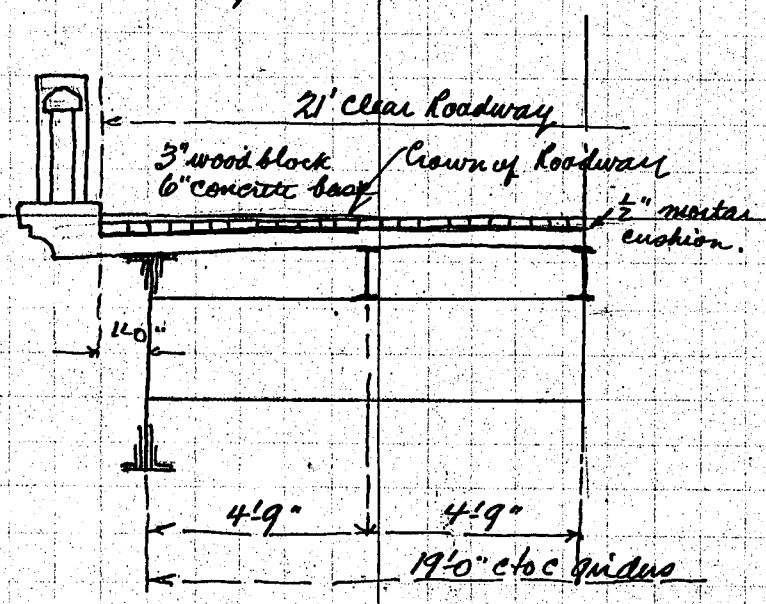
Kakogawa Bridge, Kakogawa Machi, Hyogoken

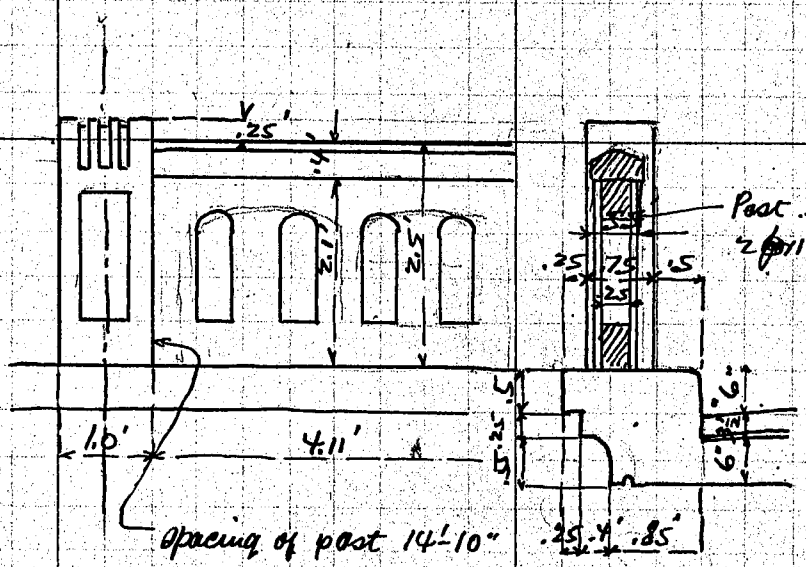
General description of Girder Spans.

The spacing of main girders 19'-0" Skew at end = 5'-1⅛" for 15° Dividing the span into 5 panels @ 13'-6⅝" = 67'-9⅛" @ skew @ 5'-1⅛" Total length 72'-10¼" center to center of End Bearings.

Cross Section of Structure.



Details of Handrail.



Weight of Handrail.

Panel Post	1.0 × .75 × 2.75	=	206
Top rail	.55 × .37 × 13.83'	=	2.82'
Int. Posts	2 - .75 × .55 × 2.10	=	1.73'
Panel	3 - .25 × 2.1 × 4.11	=	6.50'
less	16 - .25 × 1.0 × .5		

1.311'
2.00
11.11 @ 150 = 1670 #'

1670 ÷ 14.83 = 113 #
call this 115 # per lin ft (average)

Weight of Coping.

	.5 × 2.0	=	1.00
	.25 × 1.75	=	.44
	.85 × .55	=	.47'
	1.91 @ 150	=	286 # per lin. ft

Overhanging Arm of Slab

max. moment at ℄ girder.

Pavement 3" wood block	15 #	
½" Sand mortar cushion	7	
	22 #	
Concrete Slab assumed	75	
	97 # per sq ft.	

Live load moment

Motor truck Rear wheel with 25% impact

	6614
25%	1655
	8269 #

moment 8269 × .54 = 4460 '#

This load distributed over 2' longitudinally

moment per ft = 2230 '#

Bending moment due to hor. pressure to Handrail 40 × 3.55 abt = 142 '#

Dead Load Moment

Roadway	97 #	× 0.5	=	49
Coping	286	× 1.75 abt	=	500
Handrail	.115	× 1.88 o	=	216
	498 #	1.54		765

Summary of moments

Dead Load moment	--	765
Live Load moment		2230
Horizontal moment		142
		3137 '#

Shear - Dead Load say 500
Live Load 4135
4635 #

Effective Depth required for negative moment = $\sqrt{\frac{3137}{1335}}$ = .485' fc = 750 #/ fs = 16000 #/

Make slab 6" with effective depth 5" Steel Area = $\frac{3137 \times 12}{\frac{7}{8} \times 5 \times 16000}$ = .535 □"

spacing of ½" ϕ bars $\frac{.196}{.535}$ = .366'

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Four span Continuous Beams with overhanging arms at both ends.
Overhanging Arm.

M_1 given $M_5 = 0$ assumed

By the theorem of three moments,

$$M_1 + 4M_2 + M_3 = 0$$
$$M_2 + 4M_3 + M_4 = 0$$
$$M_3 + 4M_4 + M_5 = 0$$

$M_5 = 0$ $\quad M_4 = -\frac{M_3}{4}$

$M_2 + 4M_3 - \frac{M_3}{4} = 0 \qquad M_2 + \frac{15}{4} M_3 = 0 \qquad M_3 = \frac{4}{15} M_2$

$M_1 + 4M_2 - \frac{4}{15} M_2 = 0 \qquad M_1 + \frac{56}{15} M_2 = 0 \qquad M_2 = -\frac{15}{56} M_1$

$M_3 = -\frac{4}{15} \times -\frac{15}{56} M_1 = +\frac{4}{56} M_1$.

Summary

Load on left arm = P	neg. M_1	pos. M_2	neg M_3	pos M_4	neg. M_5	
	$\frac{56}{56} M_1$	$\frac{15}{56} M_1$	$\frac{4}{56} M_1$	$\frac{1}{56} M_1$	0	
	0	$\frac{1}{56} M_1$	$\frac{4}{56} M_1$	$\frac{15}{56} M_1$	$\frac{56}{56} M_1$	← load on right arm
	$-\frac{56}{56} M_1$	$+\frac{16}{56} M_1$	$-\frac{8}{56} M_1$	$+\frac{16}{56} M_1$	$-\frac{56}{56} M_1$	← Summary - Symmetrical loading.

Reactions.

R_1' $\quad P$

R_2'' $\quad \frac{-\frac{15}{56}(-M_1) - \frac{56}{56}(M_1)}{l} = \frac{71}{56}\frac{M_1}{l}$ $\Big\}$ $P + \frac{71}{56}\frac{M_1}{l}$

R_2' $\quad \frac{\frac{56}{56}(-M_1) + \frac{15}{56}(-M_1)}{l} = -\frac{71}{56}\frac{M_1}{l}$ $\Big\}$

R_2'' $\quad \frac{-\frac{4}{56} M_1 - \frac{15}{56} M_1}{l} = -\frac{19}{56}\frac{M_1}{l}$ $\Big\}$ $-\frac{90}{56}\frac{M_1}{l}$

R_3' $\quad \frac{+\frac{15}{56} M_1 + \frac{4}{56} M_1}{l} = +\frac{19}{56}\frac{M_1}{l}$ $\Big\}$

R_3'' $\quad \frac{\frac{1}{56} M_1 + \frac{4}{56} M_1}{l} = +\frac{5}{56}\frac{M_1}{l}$ $\Big\}$ $+\frac{24}{56}\frac{M_1}{l}$

R_4 $\quad \frac{-\frac{4}{56} M_1 - \frac{1}{56} M_1}{l} = -\frac{5}{56}\frac{M_1}{l}$ $\Big\}$

R_4 $\quad -\frac{1}{56}\frac{M_1}{l} = -\frac{1}{56}\frac{M_1}{l}$ $\Big\}$ $-\frac{6}{56}\frac{M_1}{l}$

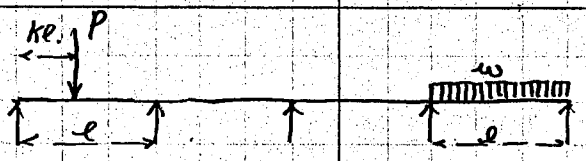
R_5 $\quad +\frac{1}{56}\frac{M_1}{l} = +\frac{1}{56}\frac{M_1}{l} + \frac{1}{56}\frac{M_1}{l}$

Summary for Symmetrical loading

R_1	R_2	R_3	R_4	R_5
+71	−90	+24	−6	+1
−1	−6	+24	−90	+71
$+\frac{72}{56}$	$-\frac{96}{56}$	$+\frac{48}{56}$	$-\frac{96}{56}$	$+\frac{72}{56}$

These will be the coef. of $\frac{M_1}{l}$
For R_1 and R_5, loads on arms to be added to get reaction.

Load on First Span



M_1 & M_5 assumed zero

$A = (2K - 3K^2 + K^3) \qquad B = (K - K^3)$.

	For conc. load	For unif. load
M_2	$-\frac{15}{56} Pl\,(B)$.	$-\frac{15}{224} wl^2$
M_3	$+\frac{4}{56} Pl\,(B)$	$+\frac{4}{224} wl^2$
M_4	$-\frac{1}{56} Pl\,(B)$.	$-\frac{1}{224} wl^2$

Reactions

	For conc. load.	For unif. load.
R_1	$P(1-K) - \frac{15}{56} P(B)$	$R_1 \quad +\frac{97}{224} wl$
R_2	$PK + \frac{34}{56} P(B)$	$R_2 \quad +\frac{146}{224} wl$
R_3	$-\frac{24}{56} P(B)$	$R_3 \quad -\frac{24}{224} wl$
R_4	$+\frac{6}{56} P(B)$	$R_4 \quad +\frac{6}{224} wl$
R_5	$-\frac{1}{56} P(B)$.	$R_5 \quad -\frac{1}{224} wl$.

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Load on Second Span.

	For conc. load.	For unif. load
M_2	$= -\frac{15}{76} Pl\,(A) + \frac{4}{76} Pl\,(B)$	$M_2 - \frac{11}{224} wl^2$
M_3	$= +\frac{4}{76} Pl\,(A) - \frac{16}{76} Pl\,(B)$	$M_3 - \frac{12}{224} wl^2$
M_4	$= -\frac{1}{76} Pl\,(A) + \frac{4}{76} Pl\,(B)$	$M_4 + \frac{3}{224} wl^2$
R_1	$= -\frac{15}{76} P\,(A) + \frac{4}{76} P\,(B)$	$R_1 - \frac{11}{224} wl$
R_2	$= +\frac{34}{76} P\,(A) + \frac{24}{76} P\,(B)$	$R_2 + \frac{122}{224} wl$
R_3	$= -\frac{24}{76} P\,(A) + \frac{40}{76} P\,(B)$	$R_3 + \frac{128}{224} wl$
R_4	$= +\frac{6}{76} P\,(A) - \frac{24}{76} P\,(B)$	$R_4 - \frac{18}{224} wl$
R_5	$= -\frac{1}{76} P\,(A) + \frac{4}{76} P\,(B)$.	$R_5 + \frac{3}{224} wl$

Summary for moment (Uniform load).

	M_2	M_3	M_4
1st span loaded	$-\frac{15}{224} wl^2$	$+\frac{4}{224} wl^2$	$-\frac{1}{224} wl^2$
2nd span loaded	$-\frac{11}{224} wl^2$	$-\frac{12}{224} wl^2$	$+\frac{3}{224} wl^2$
	$-\frac{26}{224} wl^2$	$-\frac{8}{224} wl^2$	$+\frac{2}{224} wl^2$
	$+\frac{2}{224} wl^2$	$-\frac{8}{224} wl^2$	$-\frac{26}{224} wl^2$
For full unif load	$-\frac{24}{224} wl^2$	$-\frac{16}{224} wl^2$	$-\frac{24}{224} wl^2$
	$-0.107\ wl^2$	$-.0715\ wl^2$	$-.107\ wl^2$

Summary for Reactions (Uniform load).

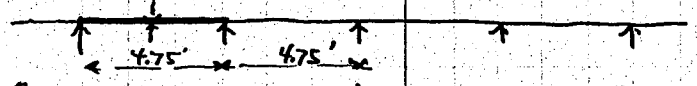
	R_1	R_2	R_3	R_4	R_5
1st Span loaded	$+\frac{97}{224} wl$	$+\frac{146}{224} wl$	$-\frac{24}{224} wl$	$+\frac{6}{224} wl$	$-\frac{1}{224} wl$
2nd Span loaded	$-\frac{11}{224} wl$	$+\frac{122}{224} wl$	$+\frac{128}{224} wl$	$-\frac{18}{224} wl$	$+\frac{3}{224} wl$
	$+\frac{86}{224} wl$	$+\frac{268}{224} wl$	$+\frac{104}{224} wl$	$-\frac{12}{224} wl$	$+\frac{2}{224} wl$
	$+\frac{2}{224} wl$	$-\frac{12}{224} wl$	$+\frac{104}{224} wl$	$+\frac{268}{224} wl$	$+\frac{86}{224} wl$
For full unif. load.	$+\frac{88}{224} wl$	$+\frac{256}{224} wl$	$+\frac{208}{224} wl$	$+\frac{256}{224} wl$	$+\frac{88}{224} wl$
	$.393\ wl$	$1.142\ wl$	$.928\ wl$	$1.142\ wl$	$.393\ wl$

Stresses in Roadway Slab.

Roadway Pavement
- 3" wood block 105
- ½" Sand mortar 7
- 22
- 75
- 97# per sq ft.

Concrete assumed

Dead Load Pos. moment at center of 2nd span



For symmetrical loading

$R_1' = 1.269 \times \frac{765}{4.75} = 204\#$

$R_2 = 1.608 \times \text{"} = 259\#$

$R_3 = .428 \times \text{"} = 69\#$

$R_4 = .107 \times \text{"} = 17.2\#$

$R_5 = .018 \times \text{"} = 2.9\#$

R_1'	R_2	R_3	R_4	R_5
204	259	69	17.2	2.9
2.9	17.2	69	259.0	204.0
498.	276.2	138	276.2	498.0
704.9#				704.9#

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Reaction due to uniform load on 4 Spans

$R_1 = 0.393 \times 97^{\#} \times 4.75 = 181.0$

Reaction due to load on one hanging brm $\underline{705.0}$

$886.^{\#}$

Moment at center of End Span

$886 \cdot \frac{4.75}{2} = 2104$

Less $498 \times 3.92 = 1954$

$97 \cdot \frac{2.38^2}{2} = \underline{275}$

-2229

$-125^{\#\#}$

Moment M_2 at Second Support

$886 \cdot 4.75 = 4208$

$498 \times 6.29 = 3132$

$97 \times \frac{4.75^2}{2} = \underline{1092}$

-4224

$-16^{\#\#}$

Moment at Center of Second Span.

R_1 $886 \cdot 7.13 = 6317$

R_2 $2762 \cdot 2.38 = \underline{657}$

$498 \cdot 8.67 = 4325$ 6974

$97 \cdot \frac{7.13^2}{2} = \underline{2470}$

-6795

$+179^{\#\#}$

Moment M_3 at center Support

$886 \cdot 9.50 = 8417$

$2762 \cdot 4.75 = \underline{1312}$

9729

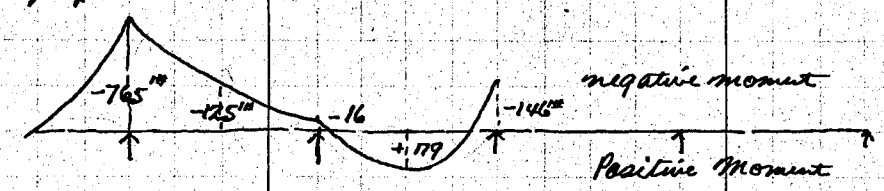
$498 \cdot 11.04 = 5500$

$97 \cdot \frac{9.52^2}{2} = \underline{4375}$

-9875

-146

Summary of Dead load Moments



Live Load Moment

Maximum moment due to concentration will be found by trials; the figures below will not be an absolute max but one of the max. moments. We don't need the exact figure for moment because of approx. design of concrete slabs which will be detailed for the possible max. moment under judgement of designer.

Max. Positive Moment at center of 2nd span.

8269# wheel load with impact

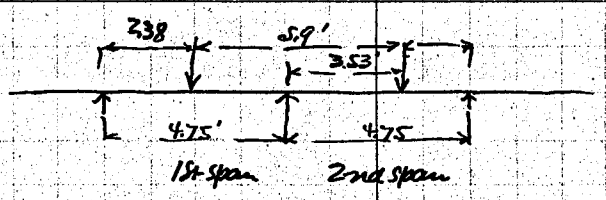
R_1 $(.400 - 0.04) \times 8269 \times 2.38 = 7090^{\#\#}$

This wheel load assumed distributed over 2'×2'

For 1' strip $m = 3545$

Less $\frac{8269^{\#}}{2} \cdot 0.5 = -\underline{1030}$

$2515^{\#\#}$



Uniform load on other span which will add the positive moment neglected.

Moment M_2 at Second Support

Loading same as for moment at center of 2nd Span.

1st wheel load at center of end span

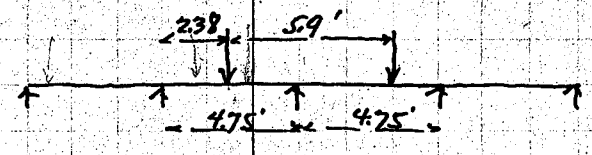
R_1 $(.400 - 0.04) \times 8269 \times .475 = 14100$

$8269 \times 2.38 = -\underline{19700}$

-5600

For 1' Strip $5600 \div 2 = 2800^{\#\#}$

Uniform load on 4th span neglected.



Positive Moment at center of 2nd span

1st load	R_1	-0.074	$\times 8269$	$= -610$	
2nd load	R_1	$+0.010$	$\times 8269$	$= +\underline{82}$	
				-528	$\times 7.13 = -3760$
1st load	R_2	$.565$	$\times 8269$	$= 4670$	
2nd load	R_2	$-.072$	$\times 8269$	$= -\underline{595}$	
				4085	$\times 2.38 = +\underline{9730}$
					$5970^{\#\#}$

wheel load distributed on 2'×2' less $m = \frac{8269}{2} \cdot 0.5 = -\underline{2065}$

$+3905$

For one ft strip $1952^{\#\#}$

Note: Uniform load neglected.

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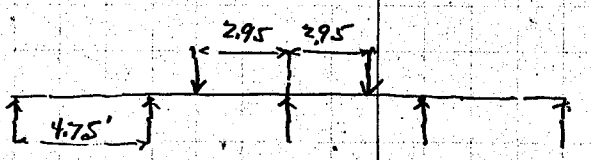
Moment M_3 at center support.

(R_1) $-0.08 \times 8269 \times 9.5 = -6290$

(R_2) $0.715 \times 8269 \times 4.75 = +28100$

$8269 \times 2.95 = -24400$

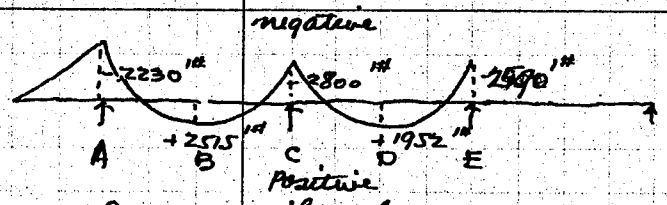
2590



For two wheels 5180 #

For 1' strip 2590 Uniform load neglected

Summary of Live Load Moments



Summary of Dead and Live Load Moments

	(A)	(B)	(C)	(D)	(E)
Dead Load moment	− 765	−125	− 16	+ 179	− 146
Live Load Moments	− 2230	+ 2515	− 2800	+ 1952	− 2590
	2995	+ 2390	2816	+ 2131	− 2736

Design the roadway slab for negative moment of 3137 # see p3

" positive moment of 2500 #

Effective Depth required for positive moment

$= \sqrt{\frac{2500}{95}} = \sqrt{26.3} = 5.1"$ make slab 6" thick $f_c = 600^{\#}/_{\square"}$ $f_s = 16000^{\#}/_{\square"}$

$d_{act} = \frac{2500 \cdot 12}{\frac{7}{8} \cdot 5 \cdot 16000} = .430"$ $\frac{1}{2}$"ϕ bars $d_{req} \frac{.196}{.43} = .455'$

For reinforcement A_c for negative moments see p3.

Reinforcing bars $\frac{1}{2}$"ϕ spaced .33' ctrs Every third bars straight top and bottom

2 bars bent up at support

Approximate shear for Intermediate Spans

neglect continuity of slab and figure shear for simple beam.

Dead Load shear $= 97 \cdot \frac{4.75}{2} = 230^{\#}$

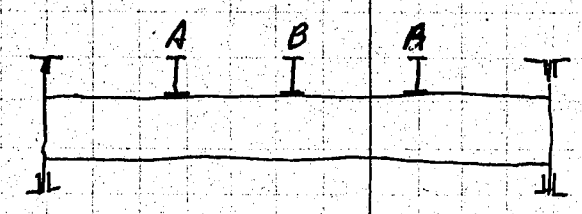
Live Load shear $\frac{8269}{2} \cdot \frac{3.75}{4.75} = \frac{3265}{3495}$ wheel load assumed distributed over 2'×2'

Unit shear $= \frac{3495}{\frac{7}{8} \times 5 \times 12} = 66.5^{\#}/_{\square"}$ Use web reinforcement

Intermediate Stringers. Span length $13' 6\frac{7}{8} = 13.55'$

max dead load reaction $= 1.14 \times 97 \times 4.75 = 526^{\#}$

$-\frac{96}{56} \times \frac{765}{4.75} = -\frac{281}{245^{\#}}$ load for stringer A.



$.93 \times 97 \times 4.75 = 428$

$+\frac{48}{56} \times \frac{765}{4.75} = \frac{138}{566^{\#}}$ load for stringer B.

Concrete fillet say 10

Stringer assumed $\frac{35}{611^{\#}}$ per ft

Design stringer B and use same section for stringers A

D.L. $m = \frac{1}{8} \times 611 \times 13.55^2 = 14,020^{\#}$

D.L. shear $= 611 \times \frac{13.55}{2} = 4140^{\#}$

Live load moment

Use ~~two~~ three wheel loads at center of span − 8269 # including 25% Impact spaced 4' centers

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Intermediate Stringers (continued).

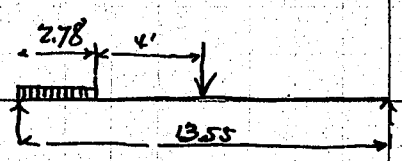
Neglecting continuity of slab & taking reactions as simple Beam,

Roadway Pavement & slab 97 × 4.75 = 460
Concrete Fillet say 12
Stringer say 40
512#

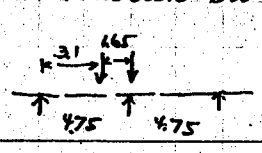
$M = \frac{1}{8} \times 512 \times 13.55^2 = 11750$ 1#

Shear = $512 \times \frac{13.55}{2} = 3470$ #

Live Load.



Reaction on Stringer due to wheel loads (with impact).



8269 × $\frac{3.1}{4.75}$ = 5400
8269
13669#

Distribution of wheel load = 2(3½"+6") + 1.0' = 2.57'

moment at center of span

$\frac{13669}{2} \times 6.78 = 46200$

$\frac{13669}{2} \times \frac{2.57}{4} = -4400$

41800#

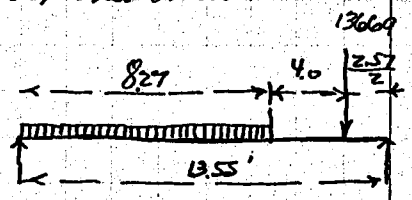
Moment due to unif. load $50 \times 6.78 =$ 340
42140

Summary of moments

Dead Load M = 11750
Live load m = 42140
53890 1#

Section modulus req'd = $\frac{53890 \times 12}{16000} = 40.3$ Use 12" × 40# I = Sm = 448

Max. End Shear



Uniform load 102.4 × 4.75 = 486#

Live Load shear conc. $13669 \times \frac{12.27}{13.55} = 12380$

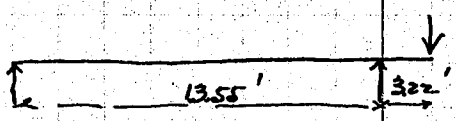
Unif. load $\frac{486 \times 8.27^2}{2 \times 13.55} = 1230$

13610#

Dead Load shear 3470

17080#

Cantilever at the End of Span.



Amount of Skew = 5' 1⅛"
Space bet. bearings = 1' 4"
6' 5⅛" ÷ 2 = 3' 2 9/16" = 3.22'

Dead load 512#
moment = ½ × 512 × 3.22² = 26600 1#
Live load, wheel conc = 13669#
moment = 13669 × 3.22 = 44000
Total moment 46660 1#

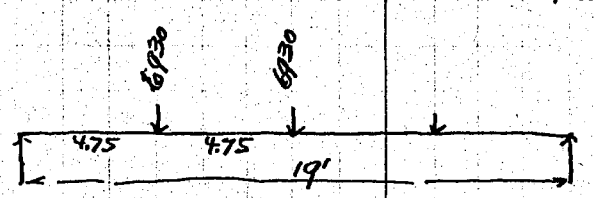
Sm req'd = $\frac{46660 \times 12}{16000} = 35.0$ 12" × 40# I OK

End Stringer with overhang : Use same section as for intermediate Stringer.

Intermediate Floor Beam

Span length 19'-0" Spacing of floor beam 13.55'

Roadway 512# conc = 512 × 13.55 = 6930#
Reaction = 6930 × 1.5 = 10400#



Dead load moment

10400 × 9.5 = 98700
less 6930 × 4.75 = −32900
65800

10400
855
11255

Dead Load Beam assumed = 90# per ft

Reaction = 90 × 19/2 = 855#

$M = \frac{1}{8} \times 90 \times 19^2 =$ 4070
69870 1#

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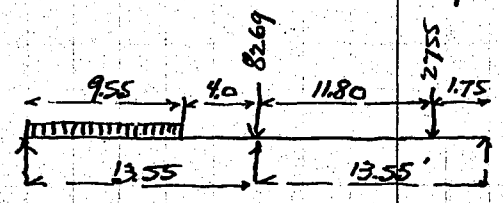
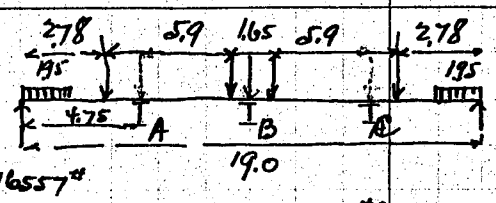
Live Load.

Wheel concentration with impact, Uniform load at front and rear and also at sides of trucks



uniform load = 102.4 × 4.75 = 486#



uniform load 102.4 #/o'

Moment at conc. point near center;

16557 × 9.59 = 158800 ft#

Less 8607 × 4.75 = 40900

116300

Reaction 2755 × 1.75/13.55 = 356

8269

8625

Unif load per panel 486 × 9.55²/2 × 13.55 = 1640 #

or 1640 ÷ 4.75 = 346# per ft

Uniform load on sides,

102.4 × 4 × 11.55/13.55 = 350

102.4 × 13.55/2 = 695

1045#

Concentration on A 8625 × 2.78/4.75 = 5050

1045 × 1.95²/2 × 4.75 = 417

Unif. load ---- 1640

Conc --- 8625 × 825/4.75 = 1500 ⎫ 8607#

Concentration on B

8625 × 393/4.75 = 7130

7130

Uniform load 1640

15900#

Concentration on C same as for A.

Reaction ---- 8607

7950

16557 #

Summary of moments

Dead Load moment = 69870

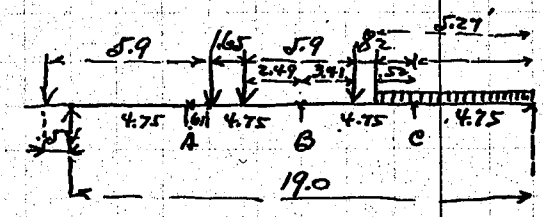
Live Load Moment = 116300

186170 #

Dm reqd = 186170 × 12/16000 = 140.0

24" × 80# I Sm. 173.9

Maximum End Shear. Live Load



Uniform load 1045# × 5.27²/2 × 19.0 = 765

Conc. 8625 × 609

11.99

13.64

31.72 ÷ 1.90 = 14400

Unif. load 1640 × 1.5 = 2460

Total Live load Shear 17625 #

Dead load Shear 11255

28880 #

Built Section for Floor Beam

Assume web = 27 × 3/8 = 10.10" ⅜ web = 1.270" 27½" b to b of Ls = 2.29'

Effective depth = 2.29 − 0.19 = 2.1' Stress = 186170 ÷ 2.1 = 88600#

Section Required = 88600 ÷ 16000 = 5.41

less 1.27

4.140" net Use 3½ × 3½ × ⅜" Ls = 4.96" gr = 4.210" net

End Shear = 28880# No of Rivet for end connection ⅞" rivet 28880 ÷ 6010 = 5.0

End spacing of Rivets ⅞" Rivet bearing on ⅜" plate = 7880

= 7880 × 23½/28880 = 6.4"

Use 3" to 4" spacing at end section + 4" to 6" Spacing at center section.

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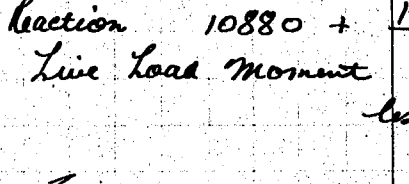
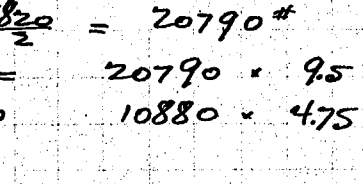
Weight of Intermediate Floor Beam

1 web	27 × 3/8	@ 34.43 × 18.95 =	652	1687 ÷ 19 = 88.5# per ft
Flanges 4 L's	3½ × 3½ × 3/8	@ 8.5 × 18.95 =	644	
Int. Stiffs 14 L's	3 × 3 × 5/16	@ 6.1 × 2.25 =	192	1678 ÷ 13.55 = 124# per ft
End Stiffs 4 L's	3½ × 3½ × 3/8	@ 8.5 × 2.23 =	76	
End Fills 4 PL's	3½ × 3/8	@ 4.46 × 1.83 =	33	
	Rivet heads + variation say		90	
			1687#	

End Floor Beam

Dead Load





7965 / 855 } 8820

Dead Load Beam 90# per ft assumed

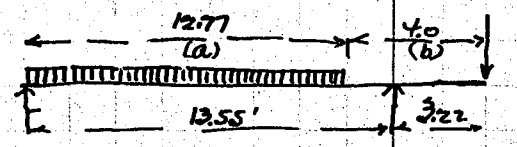
Shear = ½ × 90 × 19. = 855

Uniform Dead load .512 × 16.77²/(2 × 13.55) = .5310#

Dead Load moment

	7965 × 9.5	=	75700
Less	5310 × 4.75	= −	25250
			50450
Moment beam	⅛ × 90 × 19² =		4070
Total Dead Load moment			54520#

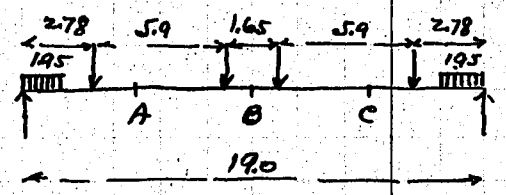
Live Load — Wheel load with impact 8269#



Reaction = 8269 × 16.77/13.55 = 10230#

Uniform load = 102.4 × 4.75 = 486#

Reaction = 486/2 × 12.77²/13.55 = 2920#



Uniform load on side of truck 1' strip (b).

102.4 × 4 × 14.77/13.55 = 446# per ft strip.

Conc. A	Conc 10230 × 2.78/4.75	=	6000
	Unif. (b) 446 × 1.95²/(2 × 4.75)	=	180
	Conc 10230 × .825/4.75	=	1780
	Unif. (a)		2920
			10880#

Conc. B 10230 × 3.93/4.75 = 8450

8450

Unif. load (a) 2920

19820#

Reaction 10880 + 19820/2 = 20790#

Live Load Moment	= 20790 × 9.5	=	197200
	less 10880 × 4.75	=	51700
			145500#

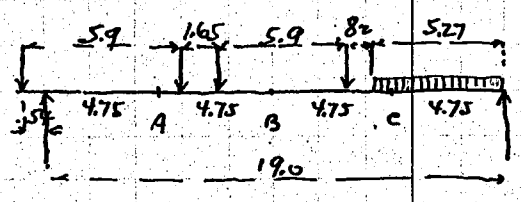
Summary of moments

Dead Load m = 54520

Live Load m = 145500

200020#

Maximum End Shear



Live Load

Uniform load (b) 446 × 5.27²/(2 × 19.0) = 326.

Concentration 10230 × 6.09

11.99

13.64

31.72 ÷ 19 = 1.7100

Uniform load (a) 2920

Total live load shear	20346
Dead load shear	8820
Total shear	29166#

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End Floor Beam

Assume web 27±3/8" = 10.10 □" 1/8 web = 1.27 □" 27½" b.to b. of Ls = 2.29 Effective d = 2.10'

Stress = 200020 ÷ 2.10 = 95200# SR = 95200 ÷ 16.000 = 5.95

1/8 web 1.27

4.68 □" net

Use 2Ls 3½" × 3½" × 7/16" = 5.14 gr □" 4.87 □" net

End shear is practically same as for intermediate floor beam; Use same detail as for Int. F.B.

Weight of End Floor Beam

web	1Pl	27±3/8" @ 34.43	×	18.95	=	652
Flange Ls	4Ls	3½ 3½ × 7/16 @ 9.8	×	18.95	=	743
Int. Stiffo	14Ls	3 × 3 × 9/16 @ 6.1	×	2.25	=	192
End Stiffo	4Ls	3½ 3½ × 3/8 @ 8.5	×	2.23	=	76
End fills.	4Pls	3½ × 7/16 @ 5.21	×	1.83	=	38
		Rivet heads & variation				90
						1791#

1791 ÷ 19 = 94.3# per ft.

Weight of Floor Beams.

Intermediate Floor Beams	4 @ 1687	= 5748
End Floor Beams	2 @ 1791	= 3582
		9330#

Weight of Stringers

Intermediate Stringers	9 @ 12" × 40# I × 135.0	=	4860#
End Stringers	6 @ 12" × 40# I × 16.73	=	4020
Details. splice	24 Pls 9" × 3/8" @ 11.48# × 1.0	=	275
	Rivet heads & variation say		75
			9230#

9230 ÷ 74.17 = 125# per ft

Bottom Lateral Bracing

The Structure has solid concrete floor to take ~~carry~~ effect of wind stress; However design bottom lateral to take full wind load & to carry to the end bearing.

wind load 30#/□' of Exposed Surface.

Exposed surface Height of girder . 4.50

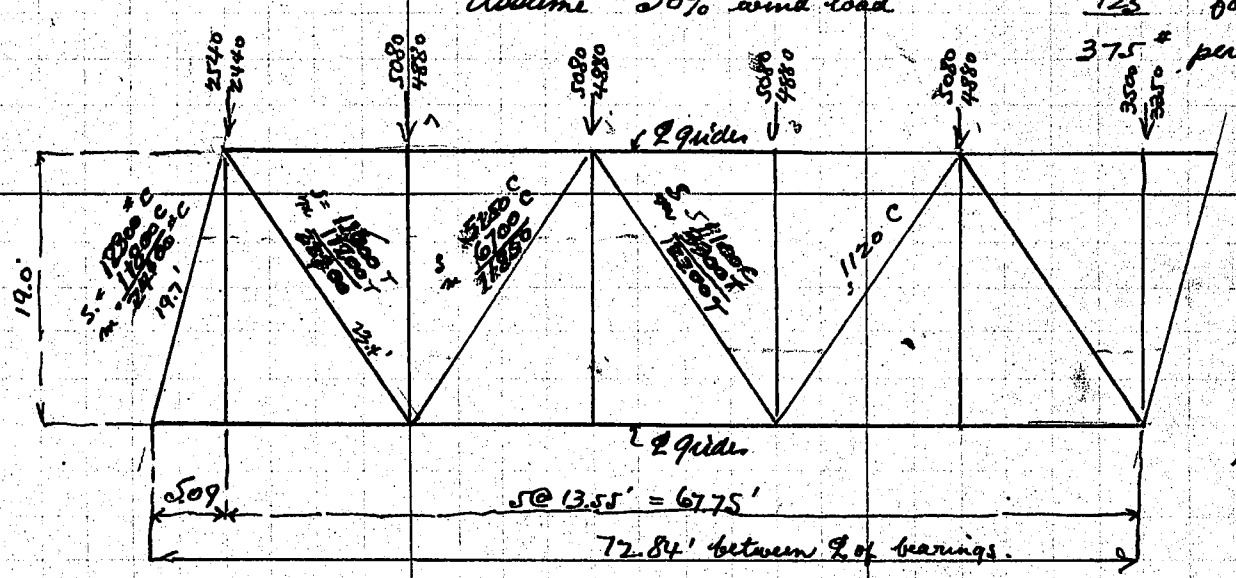
Concrete coping 1.30

Handrail 2.50

8.30' × 30# = say 250#

Assume 50% wind load 125 for leward

375# per lin. ft. of girder.



Reaction 4880 × 13.55/72.84 = 908#

" 908 × 3 = 2760#

" 908 × 6 = 8450#

values of Secant

23.4 ÷ 19.0 = 1.23

19.7 ÷ 19.0 = 1.037

Note S stands for Static wind load

m " " moving " "

Panel concentration	375 × 13.55	=	5080#	Reaction	5080 × 10 × 13.55/72.84	= 9450
"	375 × 9.32	=	say 3500#		2540 × 5 × 13.55/72.84	= 2360
	375 × 13.55/2	=	2540#			11810#
Moving wind load	12 × 30#	=	360#			
Panel concentration	360 × 13.55	=	4880#	Reaction	4880 × 10 × 13.55/72.84	= 9100
	360 × 13.55/2	=	2440#		2440 × 5 × 13.55/72.84	= 2270
	360 × 9.32	=	3350			11370#

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Section of Bottom Lateral Bracing.

l/r limited to 150. Length between end connection say 20.4 × 12 = 245"

Try 2 L 6×4×3/8 3/8" b to b $r = 1.62$ $l/r = 151.0$

Unit stress = 16000 − 70 × 151 = 5400

Add 30% 1620

7020#

Stress 22700# Tor C $K = 22700 \div 7020 = 3.240"$ 2 L 6×4×3/8 = 7.220"

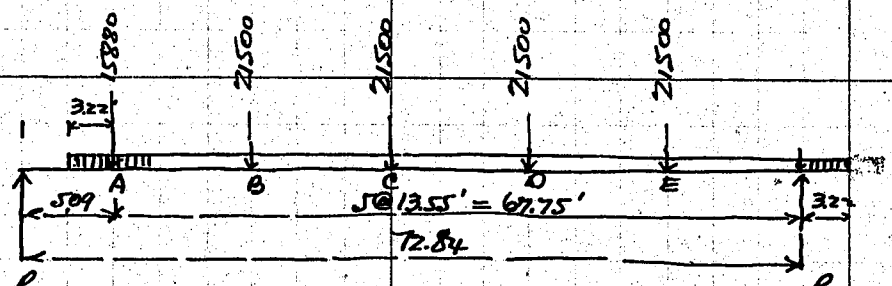
Use 6 rivets for lateral connection.

Weight of Lateral Bracing.

10 L 6×4×3/8	@ 12.3	= 21.4 =	2630
4 L 6×4×3/8	@ 12.3	= 19.4 =	955
3/8" Pl.	4 @ 75#	=	300
" "	2 @ 55	=	110
Misc. Connections + rivet heads		say	120
			4115#

4115 ÷ 72.83 = 57# per ft

Design of Girder.



Concentration 1588 × 13.55 = 21500#

" assumed (A) 1588 × 10.0 = 15880#

Reaction R_1 $1588 \times 74.19 \times \frac{33.88}{72.84} = 54700$#

Dead Load

woodblock pavement	15	
½" mortar cushion	7	
concrete 6" thick	75	
	97 × 21 =	2037
Concrete filler over girder flange		31
Copings	2 @ 286	572
Handrails	2 @ 115	230
		2870#

Roadway complete	2870
Stringers	125
Floor Beams	124
Lateral Bracing	57
	3176 ÷ 2 = 1588# per ft per girder.

Dead load moments

Moment at A.	54700 × 5.09 =	278000#
Moment at B.	54700 × 18.64 =	102.0000
	− 15880 × 13.55 =	− 215000
		805000
		805.000#
Moment at C	54700 × 32.19 =	1760.000
	− 15880 × 27.10 =	− 431.000
	− 21500 × 13.55 =	− 291.000
		1038.000#
Moment at D	54700 × 45.74 =	2500.000
	− 15880 × 40.65 =	− 646.000
	− 21500 × 40.65 =	− 874.000
		980.000#
Moment at E.	54700 × 59.29 =	3240.000
	− 15880 × 54.20 =	− 861.000
	− 21500 × 6 × 13.55 =	− 1748.000
		631.000#

End shear = 54700#

Shear A to B = 38820

Shear B to C = 17320

Shear C to D = 4180

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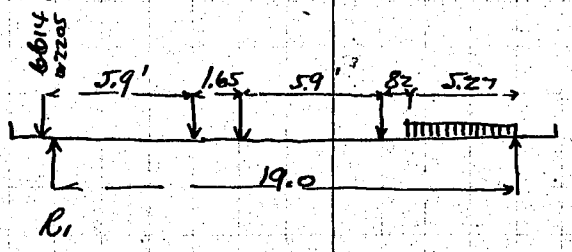
Live Load



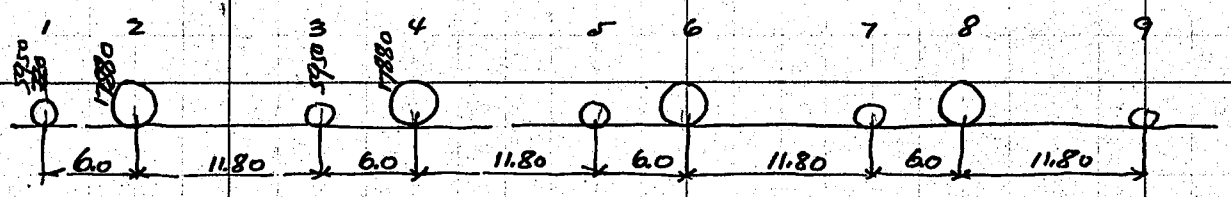
Rear wheel $R_1 = 6614 \times \frac{51.26}{19.0} = 17.880^{\#}$

Front wheel $R_1 = 2205 \times \frac{51.26}{19.0} = 5950^{\#}$

Uniform load $102.4 \times 5.27 = 540^{\#}$

Reaction $R_1 = 540 \times \frac{5.27}{2 \times 19.0} = 75^{\#}$ per lin. ft.

Live Load wheel concentrations spaced as shown below.



Moment at A. (wheel no 2 at A)

	wheel	
Reaction	2	67.75
	4	49.95
	6	32.15
	8	14.35
		164.20

$17880 \times \frac{164.20}{72.84} = 40260$

	3	53.95
	5	36.15
	7	18.35
	9	6.55
		115.00

$5950 \times \frac{115.00}{72.84} = 9390$

49650

Moment $= 49650 \times 5.09 = 252.700^{\#}$

Moment at B wheel no 3 at B.

	wheel	
Reaction	3	54.20
	5	36.40
	7	18.60
	9	.80
		110.00

$5950 \times \frac{110.00}{72.84} = 9000$

	2	66.00
	4	48.20
	6	30.40
	8	12.60
		157.20

$17880 \times \frac{157.20}{72.84} = 38580$

$47580^{\#}$

Moment $47580 \times 18.64 = 886500$

$17880 \times 11.80 = -211000$

$675500^{\#}$

Moment at C; wheel no 5 at C.

	wheel	
Reaction	3	58.45
	5	40.65
	7	22.85
	9	3.05
		125.00

$5950 \times 125.00 \div 72.84 = 10200$

	2	7025
	4	52.45
	6	34.65
	8	16.85
		174.20

$17880 \times 174.20 \div 72.84 = 42750$

$52950^{\#}$

moment

$52950 \times 32.19 = 1704000$

$17880 \times 41.40 = -734500$

$5950 \times 17.80 = -105900$

$863600^{\#}$

Moment at D; wheel no 6 at D.

Reaction	8	63.54
	6	45.74
	4	27.94
	2	10.14
		147.36

$17880 \times 147.36 \div 72.84 = 36150$

	9	705.34
	7	57.54
	5	39.74
	3	21.94
	1	4.14
		198.70

$5950 \times 198.70 \div 72.84 = 16230$

52380

moment

$52380 \times 27.1 = 1416,000$

$-5950 \times 41.4 = -246500$

$-17880 \times 17.80 = -318500$

$851000^{\#}$

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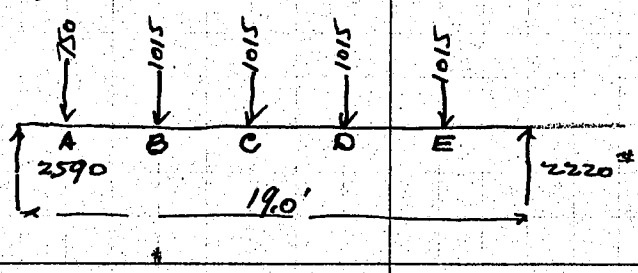
Moment at E; wheel no 8 at E.

Reaction	8	59.29	Moment	46520 · 13.55 =	630.000
	6	41.49		5950 · 11.80 −	70200
	4	23.69			559800#
	2	5.89			
	17880 ·	130.36 ÷ 72.84 =	32000		
		71.09			
		53.29			
		35.49			
		17.69			
	5950 ·	177.56 ÷ 72.84 =	14520		
			46520#		

Uniform live load moments.



Uniform load 75# panel conc 75 · 13.55 = 1015#
75 · 10.0 = 750#

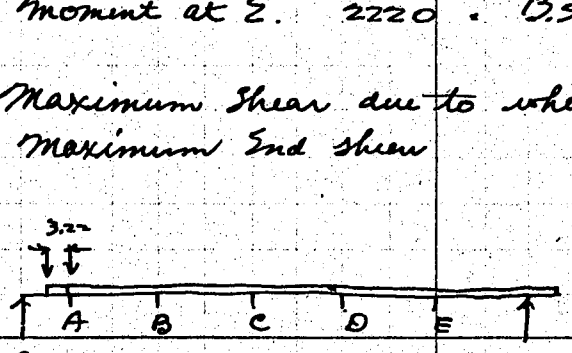
Reaction 1015 · 10 · 13.55 ÷ 72.84 = 1890#
750 · 5 · 13.55 ÷ 72.84 = 700
2590

Moment at A	2590 · 5.09 =	13200#	
Moment at B	2590 · 18.64 =	48300	
	750 · 13.55 = −	10150	
		38150#	
moment at C	2590 · 32.19 =	83300	
	1015 · 13.55 = −	13800	
	750 · 27.10 = −	20400	
		49100#	
Moments at D	2220 · 27.10 =	60000	
	1015 · 13.55 = −	13800	
		46200#	
moment at E.	2220 · 13.55 =	30100#	

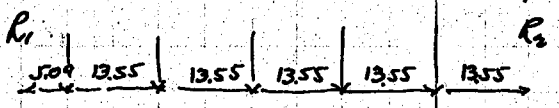
Shear at end 2590#
A to B 1890
B to C 1835
C to D 565

Maximum Shear due to wheel loads.

Maximum End shear



R_1	8	70.97		
	6	53.17		
	4	35.37		
	2	17.57		
17880 ·	177.08 ÷ 72.84 =		43400	



	7	64.97		
	5	57.17		
	3	29.37		
	1	11.57		
5950 ·	163.08 ÷ 72.84 =		13350	
			56750#	

Shear at B.

	54.20		
	36.40		
	18.60		
	.80		
17880 ·	110.00 ÷ 72.84 =		27000
	~~48.20~~	42.40	
	~~30.40~~	~~24.60~~	
	~~12.60~~	6.680	
5950 ·	~~91.20~~ ÷ 72.84 =		~~7450~~ 6000
	73.80		33000
			34450#

Shear at C.

	40.65			
	22.85			
	305			
17880 ·	66.55 ÷ 72.84 =		16300	
	~~34.65~~	28.85		
	~~16.85~~	11.05		
5950 ·	~~51.50~~ ÷ 72.84 =		~~4200~~ 3260	
	39.90		19560	
			20500#	

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Shear at D

45.74
27.94
10.14
17880 × 83.82 ÷ 72.84 = 20600
33.94
16.14
5950 × 50.08 ÷ 72.84 = 4100
24700#

Shear at E

59.29
41.49
23.69
5.89
17880 × 130.36 ÷ 72.84 = 32100
47.49
29.69
11.89
5950 × 89.07 ÷ 72.84 = 7300
39400#

End Shear

72.84
55.04
37.24
19.44
1.64
17880 × 186.20 ÷ 72.84 = 45800
66.84
49.04
31.24
13.44
5950 × 160.56 ÷ 72.84 = 13100
58900

Less

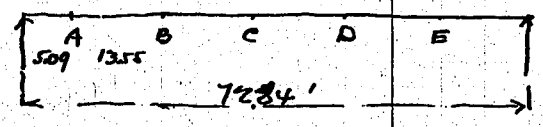
5950 × 7.55/13.55 = 3300
17880
58900 − 21180 = 37720

5950 × 186.20 ÷ 72.84 = 15200
17880 × 160.56 ÷ 72.84 = 39400
54600

5950
17880 × 7.55/13.55 = 9950
−15900
38700#

First wheel at E will give max shear.

Dead Load Moment of main Girder.



Dead load of girder assumed 340# per lin ft

Moment at

A 340/2 × 5.09 × 67.75 = 58500#
B " 18.64 × 94.20 = 172000
C " 32.19 × 40.65 = 222000
D " 45.74 × 27.10 = 210000
E " 59.29 × 13.55 = 136000

Shear at

A 12400 − 1720 = 10680
B " 6350 = 6050
C " 10900 = 500
D " 15500 = 3100
E " 20100 = 7700

Reaction 340/2 × 72.84 = 12400#

Summary of moments

	A	B	C	D	E
Dead load moment	278000	805000	1038000	980000	631000
Live Load conc.	252700	675500	863600	851000	559800
" " unif.	13200	38150	49100	46200	30100
Dead load girder	58500	172000	222000	210000	136000
	602400#	1690650#	2172700	2087200	1356900#

Summary of Shears

	End Shear	A to B	B to C
Dead Load	54700	38820	17320
Live Load conc	56750	33000	19560
" " unif.	2590	1890	1130
D.L. girder	12400	10680	6050
	126440	84390	44060

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Section of main girder.

End shear say 130.000# Section required for web 13.0 □" gross.

Assume 60 × 3/8" plate = 22.50 □" ℄ web = 2.81 □" 60½" back to back of Ls.

Flange assumed

			q.	centre of gravity			
2 Ls 6 × 6 × 3/4"	@	16.44	×	1.78	=	29.20	$\frac{1.633 \times 2}{12} = .21'$
1 cov Pl. 14 × 5/8	@	8.75	×	-.312	=	- 2.74	
1 cov Pl. 14 × ½	@	7.00	×	-.87	=	- 6.10	5.04' - .21 = 4.83'
		32.19		.633		20.36	

Total moment = 2172700 ft# Effective depth = 4.83' Stress = 449.000 #

Section Required = 449000 ÷ 16000 = 28.00 □" net - 2.81 = 25.19 □"

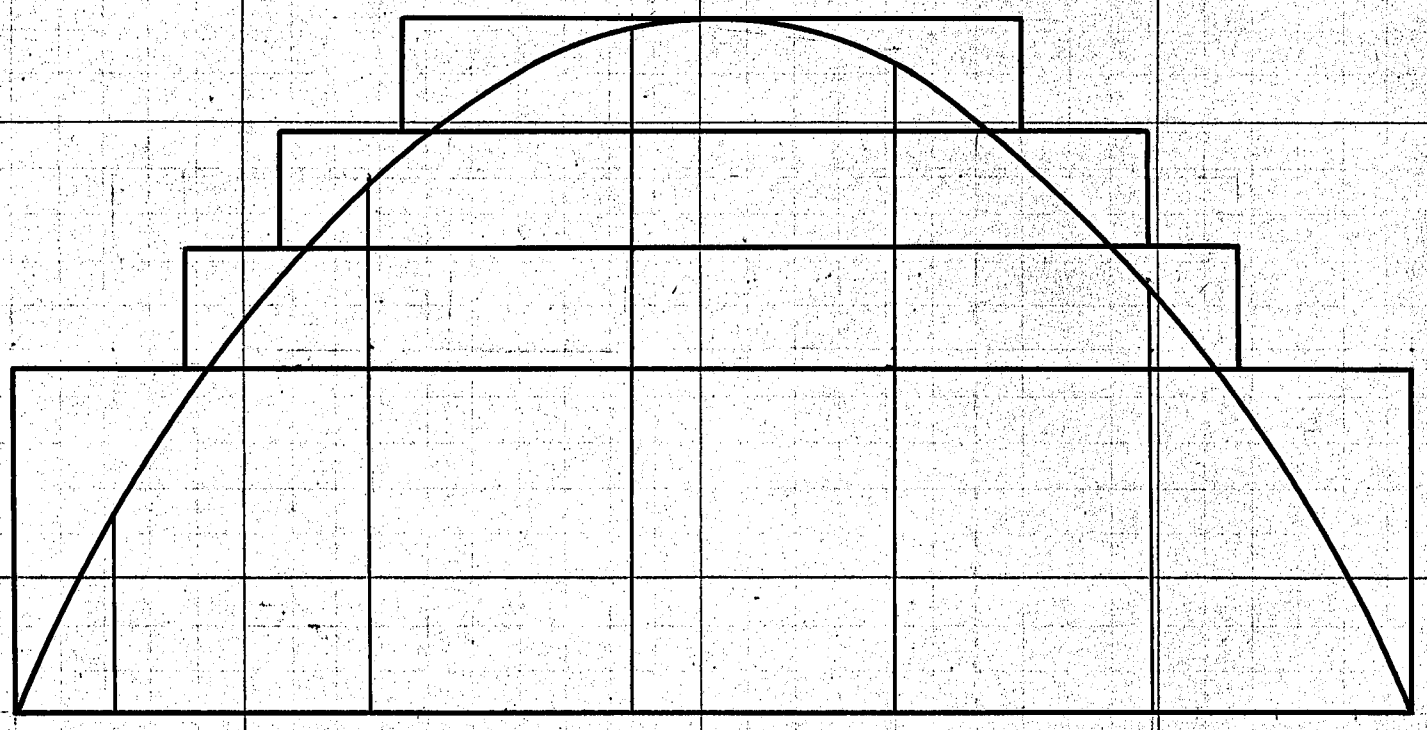
Try the following section

2 Ls 6 × 6 × 3/4	=	16.44 q.	13.44 □" net
1 cov pl 14 × 5/8	=	8.75	7.50
1 cov pl. 14 × ½	=	7.00	6.00
			26.94 □" net

Let us figure the fibre stress by moment of inertia method.

		net		For both flanges	arm		M of I.
2 Ls 6 × 6 × 3/4	=	16.44		13.44	26.88	× 28.47²	= 21800 + 113
1 cov Pl, 14 × 5/8	=	8.75	2	7.50	15.00	× 30.56²	= 14000
1 Cov Pl. 14 × ½	=	7.00	2	6.00	12.00	× 31.12²	= 11630
1 web, 60 × 3/8	=	22.50	60 × .28 net ave.		16.80	×	5065
		3249	64.38				52475 / 113 / 52588

$$\text{Fibre stress} = \frac{2172700 \times 12 \times 31.37}{52475} = 15600 \#/\square" \text{ OK}$$



End Rivet Spacing. to carry 130,000# Shear.

Bearing pressure of 7/8" Rivet on 3/8" plate = 7880#

without direct load Rivet spacing = $\frac{7880 \times 53.25}{130.000} = 3.22"$ For 1' Strip $\frac{12}{3.22} = 3.72$ Rivets

Direct load; Live load reaction on girder

$8269 \times \frac{5.29}{4.75} = 9200\#$ distributed over 2' 4600# per one ft Strip

Dead load 705

5305

No of Rivet regd for 1' strip = $\frac{5305}{7880} = .673^R$

Combined $\sqrt{3.72^2 + .67^2} = 3.78$ Rivets per ft

Rivet spacing 12 ÷ 3.78 = 3.18"

Use 3" spacing at end.

Panel next to the End shear 85.000# Rivet gp = $\frac{7880 \times 53.25}{85.000} = 4.94"$

use 4½" spacing For other panel 6" max. rivet spacing

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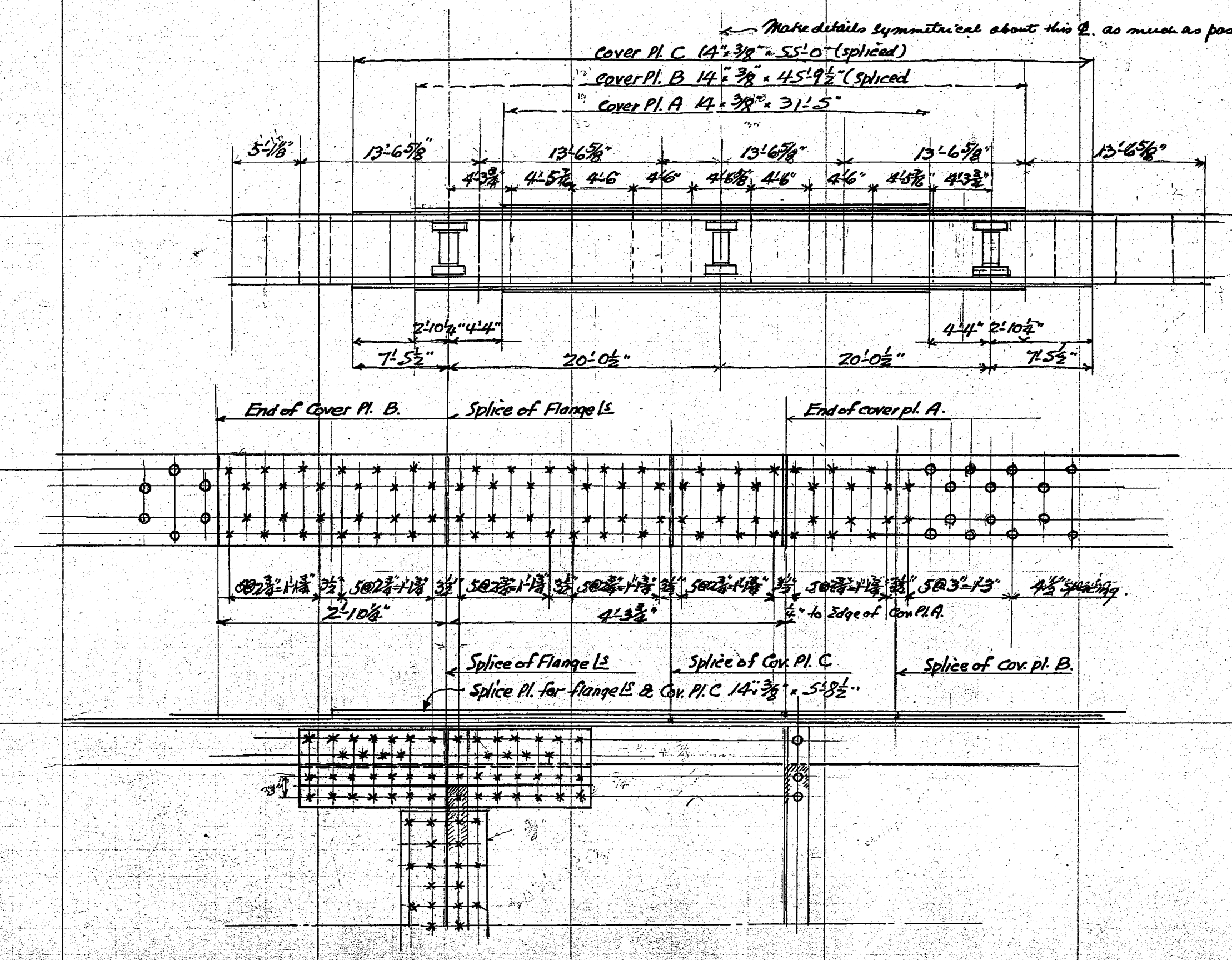

← Make details symmetrical about this ℄ as much as possible.
Cover Pl. C 14" × 3/8" × 55'-0" (spliced)
Cover Pl. B 14" × 3/8" × 45'-9 1/2" (spliced
Cover Pl. A 14" × 3/8" × 31'-5"
5'-1 1/8"
13'-6 5/8"
13'-6 5/8"
13'-6 5/8"
13'-6 5/8"
13'-6 5/8"
4'-3 3/4"
4'-5 7/16"
4'-6"
4'-6"
4'-6 1/8"
4'-6"
4'-6"
4'-5 7/16"
4'-3 3/4"
2'-10 1/4"
4'-4"
4'-4"
2'-10 1/4"
7'-5 1/2"
20'-0 1/2"
20'-0 1/2"
7'-5 1/2"
End of Cover Pl. B.
Splice of Flange ℓs
End of cover pl. A.
5@3"=1'-3"
3 1/2"
5@3"=1'-3"
3 1/2"
5@3"=1'-3"
3 1/2"
5@3"=1'-3"
3 1/2"
5@3"=1'-3"
3 1/2"
5@3"=1'-3"
4 1/2" Spacing.
2'-10 1/4"
4'-3 3/4"
1/2" to Edge of Cov Pl. A.
Splice of Flange ℓs
Splice of Cov. Pl. C
Splice of cov. pl. B.
Splice Pl. for flange ℓs & Cov. Pl. C 14" × 3/8" × 5'-8 1/2"
3 1/2"

Detail design of splice.

Web splice.

Splice web for shear and for moment considering ⅛" web as flange area.

Shear = 85000# Bending Stress 2.81 × 16000 = 45.000#

For moment Unit stress to be reduced to $16000 \times \frac{3.5}{4.8_2} = 11600$#/□"

Section required = 45000 ÷ 11600 = 3.880" net

Try 2 Pls 6" × ½" = 6.0 gr or 4.00" net

Field Rivet ⅞" bearing 6560# Reduced Strength $6560 \times \frac{3.5}{4.8_2} = 4770$#

No of Rivets 45000 ÷ 4770 = 10.0

For shop rivets reduce number by 20%

For shearing — 85000 ÷ 6010 = 14 Rivets

Splice of Cover Plate

14 × ⅜" = 5.25 gr or 4.50" net Stress 4.5 × 16000 = 72000#

No of Rivets required 72.000 ÷ 6010 = 12 for field conn

72.000 ÷ 7220 = 10 " shop "

Splice of flange Angles.

Net Section of Ls = 13.440" Stress 13.44 × 16000 = 215000

Splice one flange leg by 14 × ⅜" Pl. — 72000

143000 to be spliced by side pls.

Unit Stress reduced to $16000 \times \frac{4.25}{4.8_2} = 14100$#/□"

Section required 143.000 ÷ 14100 = 10.150" net

Use 2 Pls 8¼" × ½" = 8.25 gr 6.25 net

2 Pls 8¼ × ⅜ = 6.12 " 4.70 "

10.95 □" net

No of Rivet For double shear $12020 \times \frac{4.25}{4.8_2} = 10600$

For ⅞" bearing $6560 \times \frac{4.25}{4.8_2} = 5800$

11 Rivets @ 106000 = 143000 / 117000

26000 ÷ 5800 = 4.5 Rivets

All details as shown on pp 17

Stiffeners at flange splice 2Ls 3½ × 3½ × ⅜"

All other intermediate stiffeners to be 2Ls 5 × 3½ × ⅜ Crimped.

Max End Reaction 130.000#

Section of End stiffeners 10.00" gross. taking 13,000#/□" unit stress.

Use 4Ls 5 × 3½ × ⅜ = 12.20 □"

No of rivets required each angle $\frac{130.000}{4 \times 7220} = 4.5$

Use ¾" × 3½" fills under end stiffeners.

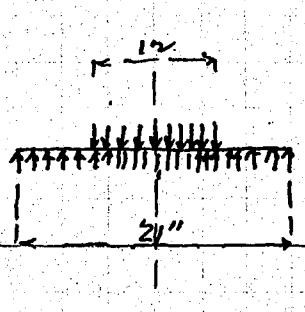
Bearing on Masonry

Bearing area assumed 15" × 21"

Unit pressure = $\frac{130.000}{15 \times 21} = 414$#/□"

Superimposed load assumed distributed over 15 × 12

Unit pressure = $\frac{130.000}{15 \times 12} = 722$#/□"



Moment at center

Upward moment ½ × 414 × 10.5² = 22900 "#

downward " ½ × 722 × 6.0² = − 13000

9900 "#

Assume 3 - 1" plates for base Flange Ls 3½"

Share of moment by 1" plate 9900 ÷ 3.75 = 2640 "#

Effective depth = $\sqrt{\frac{2640 \times 6}{16000}} = 1.0$"

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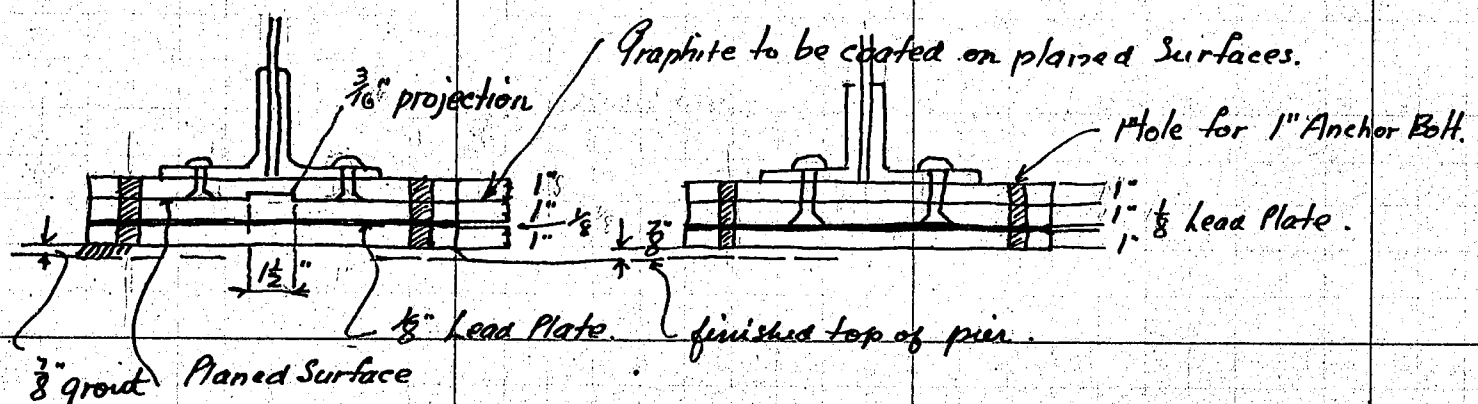
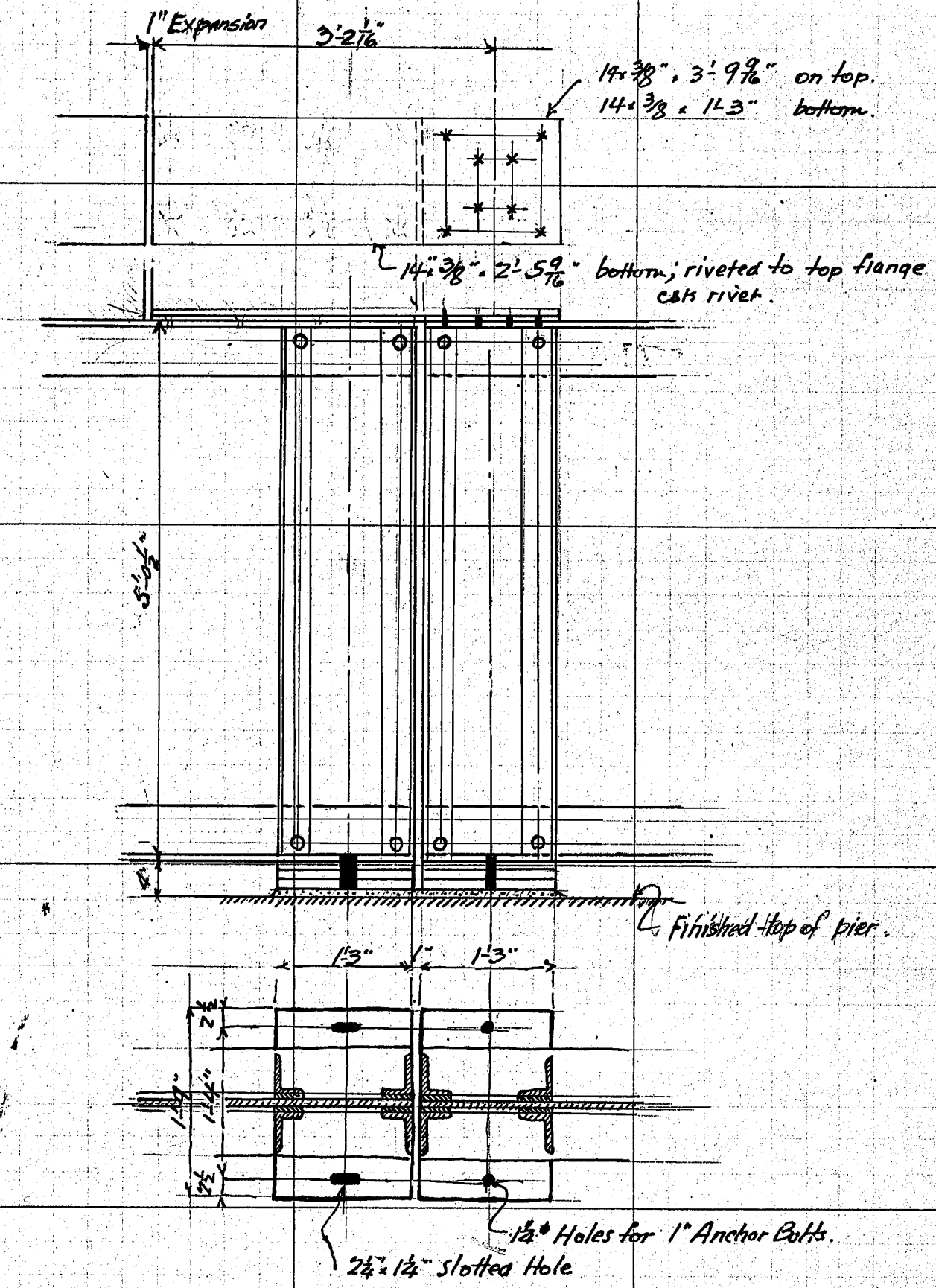
Details of Shoes.



Details of Exp. Shoe.

Details of Fixed Shoe.



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Weight of main Girder.						
web plate	1 web	60 × 3/8	@ 76.5#	× 74.08	=	5675 ✓
Flanges	4 Ls	6 × 6 × 3/4	@ 28.7	× 74.08	=	8510 ✓
Cover plates A	2 Pls	14 × 3/8	@ 17.85	× 31.42	=	1125
Cover plates B	2 Pls	14 × 3/8	@ 17.85	× 45.79	=	1640
Cover plates C	2 Pls	14 × 3/8	@ 17.85	× 55.00	=	1969 ✓
Flange splice top	4 Pls	14 × 3/8	@ 17.85	× 5.71	=	408 ✓
" " sides	8 Pls	8¼ × 1/2	@ 14.03	× 3.54	=	398
" " "	8 Pls	8¼ × 3/8	@ 10.52	× 3.54	=	298 ✓
web splice near end	8 Pls	6 × 3/4	@ 15.30	× 3.54	=	434 ✓
web splice at center	4 Pls	6 × 1/2	@ 10.20	× 2.54	=	104 ✓
" " on web	6 Pls	12½ × 3/8	@ 15.94	× 3.00	=	287 ✓
Intermediate Stiffs	28 Ls	5 × 3½ × 3/8	@ 10.40	× 5.04	=	1470
" " at splice Pt.	4 Ls	3½ × 3½ × 3/8	@ 8.50	× 4.92	=	167
" " " " fills	4 Pls	3½ × 3/8	@ 4.46	× 3.00	=	53
" "	4 Pls	3½ × 3/8	@ 4.46	× 3.50	=	62
" "	4 Pls	3½ × 1/2	@ 5.95	× 3.50	=	83
End Stiffs	8 Ls	5 × 3½ × 3/8	@ 10.40	× 4.92	=	409 ✓
fills	8 Pls	3½ × 9/16	@ 8.93	× 4.00	=	286 ✓
Expansion End	1 Pl.	14 × 3/8	@ 17.85	× 3.80	=	68 ✓
" "	1 Pl	14 × 3/8	@ 17.85	× 1.25	=	22
" "	1 Pl	14 × 3/8	@ 17.85	× 2.45	=	44
Shelf Ls	6 Ls	3½ × 3½ × 3/8	@ 8.50	× 0.8	=	41
" Ls & Lateral conn.	6 Ls	"	@ 8.50	× 2.25	=	115
Shoes	6 Pls	15" × 1"	@ 51.0	× 1.75	=	536 ✓
Anchor Bolts &c say						50 ✓
		Rivet heads & variation 5%				1213
						25467 #
				Call this		25500 # per girder.
				For 2 girders		51000 #

Weight of metal in one span.

Stringers	9230
Floor Beams	9330
Laterals	4115
2 Girders	51000
	73675
Misc structural steel say	725
	74400 ÷ 2240 = 33.2 tons per span.

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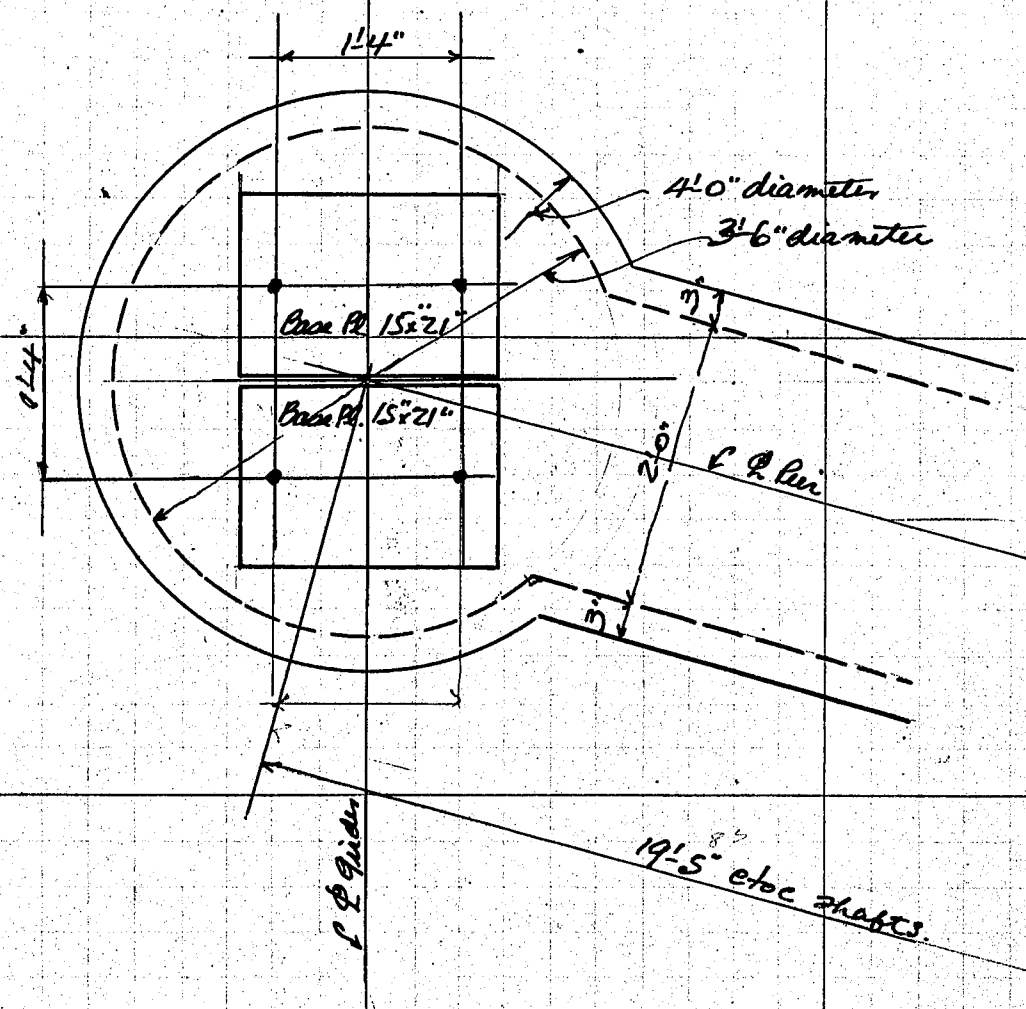
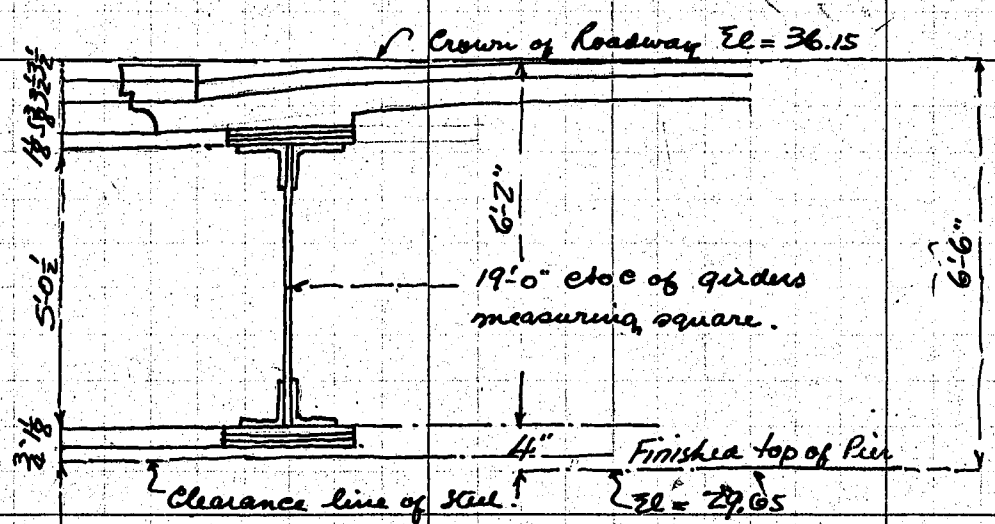
Design of Piers.



Amount of Skew 5'-1⅛" for 19'-0"
Length between ℄ shafts.

19'-0'² = 361.0000
5'-1⅛"² = 25.9463
386.9463 - 19'-4 3/32"
call this 19'-8"

Relative dimensions between super and sub structures.



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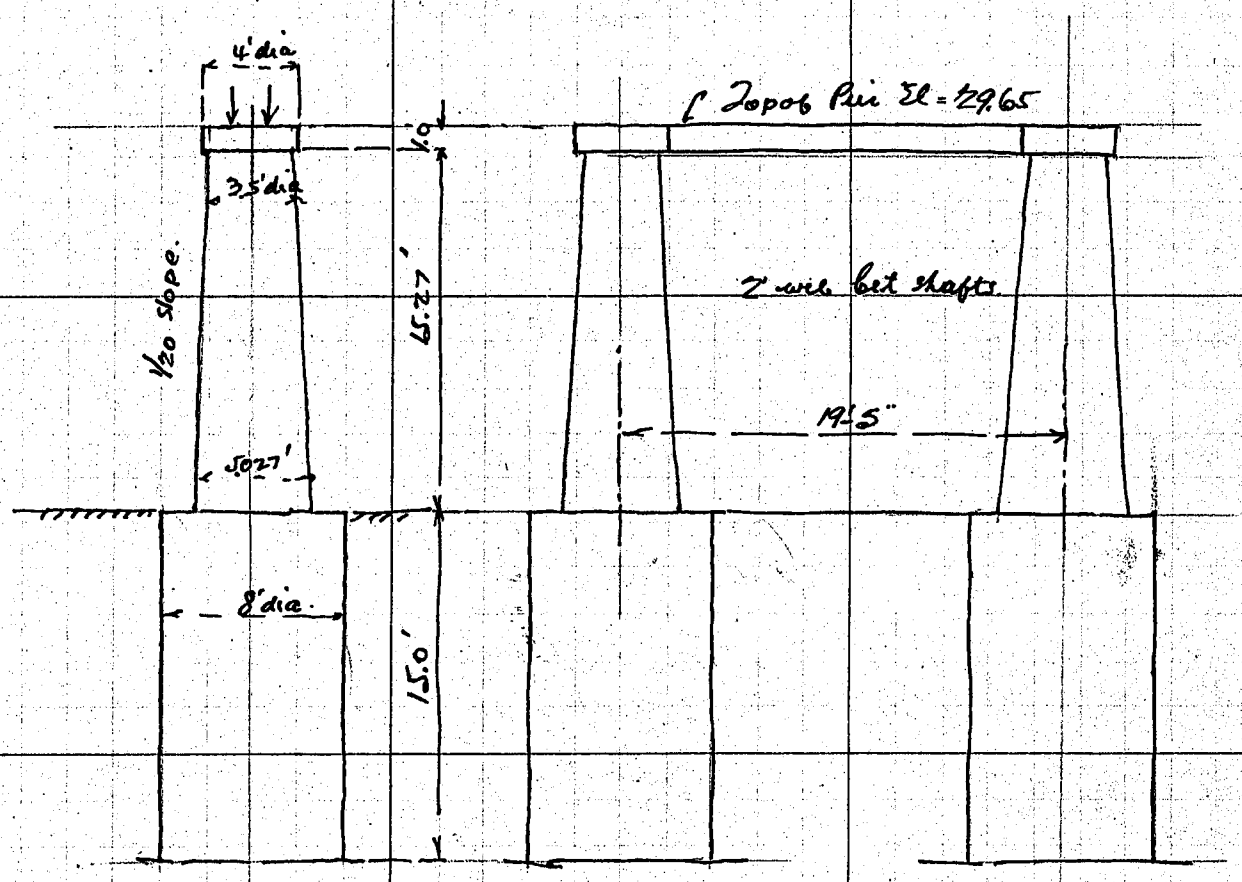
Pier nos 1 to 10 inclusive.



Coping
2 @ 4'dia = 2 × 12.57 = 25.14
web 2.5' × 15.42 = 38.60
63.74
volume = say 64 cubic ft

Shaft
Top 3.5'dia 2 × 9.6 = 19.20
2 × 15.92 = 31.84
51.04
Bott 5.02'dia 2 × 19.8 = 39.60
2 × 14.4 = 28.8
68.40
119.44
Average 59.72 sq ft
vol = 59.72 × 15.27 = 911 cub ft

volume of shaft + coping
911 + 64 = 975 = 4.51 ℄
weight = 975 @ 150 = 146.000#

Base 8'dia 2 @ 50.3 = 100.6 sq ft
volume = 100.6 × 15 = 1509 cubic ft or 6.97 ℄
weight = 1509 × 150 = 226.000#

Superimposed Load
Uniform load span 2 @ 74'-2" = 148'-3
From formula $\frac{20480}{170 + \frac{148.3}{3.28}}$ = 95# Load per ft = 95 × 21 = say 2000#

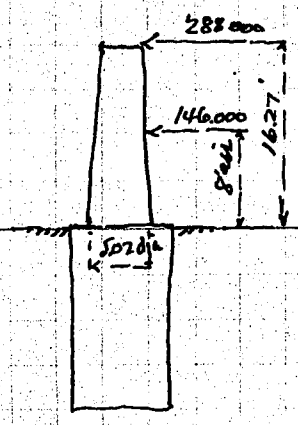
Total Live Load on pier = 2000 × 74.17 = 148.000
Dead Load 288.000
436 000#

Qu. floor 3176
girder 700
3876
For one span 3876 × 74.17 = 288000#

Summary of loads
Superimposed load 436.000
Shaft 146000
Base 226.000
808000#

Unit Bearing Pressure = $\frac{808000}{100.6}$ = 8030 #/' or 3.6 tons per sqft.

Resistance to Earthquake.



Acceleration of falling body g = 32.2' per sec².
Assumed Acc of Earth quake 2500 mm/sec² = 8.2' per sec².

Strength at the top of Base.
Moment due to acc. $M = \frac{W}{g} a y$
where W - weight
g - acceleration of gravity
a - " " Earthquake motion.
y - lever arm to extreme fibre.

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Moment deck $\frac{288000}{32.2} \times 8.2 \times 16.27 = 1.192.000$

Shaft $\frac{146.000}{32.2} \times 8.2 \times 8.0 = \underline{298.000}$

1.490.000

745.000 # per base. or per shaft.

Moment of Inertia of shaft at bottom $I = 0.049 \times 60^{"4} = 635000$ (inch)^4

Fibre stress $= \frac{745000 \times 12 \times 30}{635000} = 422$ #/o" Comp or Tension.

Direct load Deck 288000

Shaft 148.000

436000 # Area of shaft 2@2830 = 5660 □"

Unit pressure 436000 ÷ 5660 □" = 77.0 obs

Summary of stress

422 C	422 T
77	77 C
499 # C	345 T

Fibre stress on tension side is too big for concrete, let us reinforce the shaft to resist the tension. Round shaft is assumed to be a square beam of 36".57"

Assume Reinforcing Bars 11-7/8"□ @ 6.0 = 6.60" % of steel 66/36.57 = 0.0032

Eccentricity 1.490.000 ÷ 43600 = 3.41' or 41"

e/h = 41 : 57 = 0.72 k = .110 $f_c = \frac{745.000 \times 12}{.110 \times 36 \times 57^2} = 695$ #/o"

Steel Stress $= \frac{745.000 \times 12}{\frac{7}{8} \times 57 \times 6.6} = 27.000$ #/□" Ok. Factor of safety = 2.0.

Strength of Shaft 8' below from top of coping.

weight of shaft above section AA

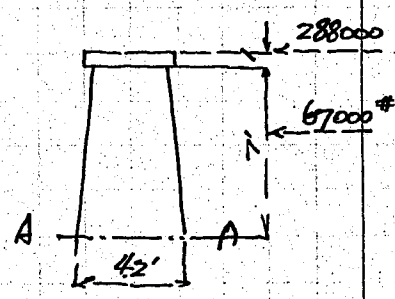
Bottom Area 4.2' dia 2@ 1385 = 27.70

2' × 1522 = 30.44

58.14

51.04

109.18 ÷ 2 = 54.59



Top Area

Average

volume 54.59 × 7 = 382.0

coping

64

446 @150 = 67000 #

Moment Deck $\frac{288000}{32.2} \times 8.2 \times 8.0 = 600.000$

Shaft $\frac{67000}{32.2} \times 8.2 \times$ say 40 $= 68200$

669200 ÷ 2 = 334600 per shaft

Weight deck 288000

Shaft 67000

355000

Eccentricity 669.200 ÷ 355000 = 1.885' = 22.6"

Round shaft assumed as a square beam of 30".47"

e/h = 22.6 ÷ 47 = 0.48

Reinforcing bars 11-7/8"□ = 6.6 Steel % $= \frac{6.6}{30 \times 47} = 0.47\%$ k = .129 for double reinf.

$f_c = \frac{334600 \times 12}{.129 \times 30 \times 47^2} = 470$ #/o" Steel stress $= \frac{334600 \times 12}{\frac{7}{8} \times 47 \times 6.6} = 14800$ #/o"

Reinforce shaft as shown in Sketch on pp 24

Reinforcement in web.

Assuming the web to carry lateral force due to Earthquake, the web to be treated as column fixing two shafts. As column the reduction of unit stress = 20%

where l = length between shafts = 15.4' b = breadth = 2.0; Then 20% = 154 #/o"

This will be the fibre stress due to bending; Let us figure the reinforcement in the web for the moment which will produce fibre stress of 154 #/o"

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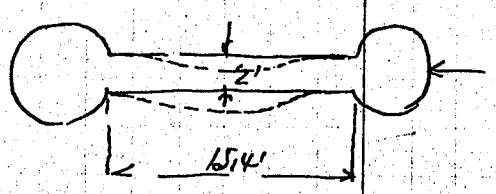


Moment for fixed Beam = $\frac{1}{12} wl^2$ fibre stress = $\frac{6M}{bd^2}$

$w = \frac{fbd^2}{6l^2}$ where $b = 12"$ $d = 22"$ $l = 154'$ $f = 15\,\%$.

$w = 750$ $M = \frac{1}{12} \times 750 \times 154^2 = 14800^{tt}$

Design reinforcement for this moment

$As = \frac{14800 \times 12}{7/8 \times 22 \times 27000} = .34.0"$ per ft strip

Use 5/8"ϕ bars 1'-0" ctrs both faces.

Lateral force at top.

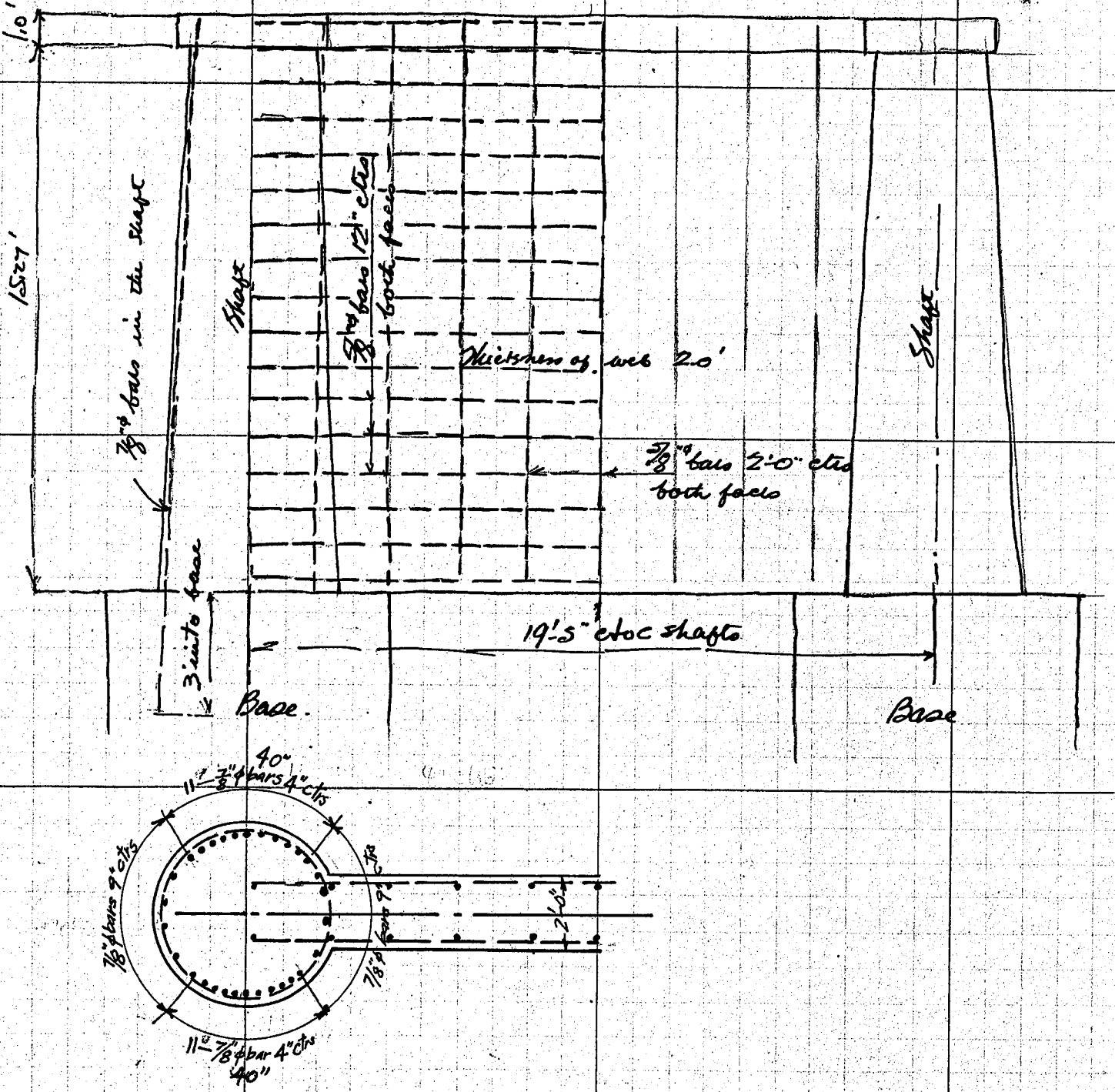
$\frac{288000}{32.2} \times 8.2 = 73400^{\#}$

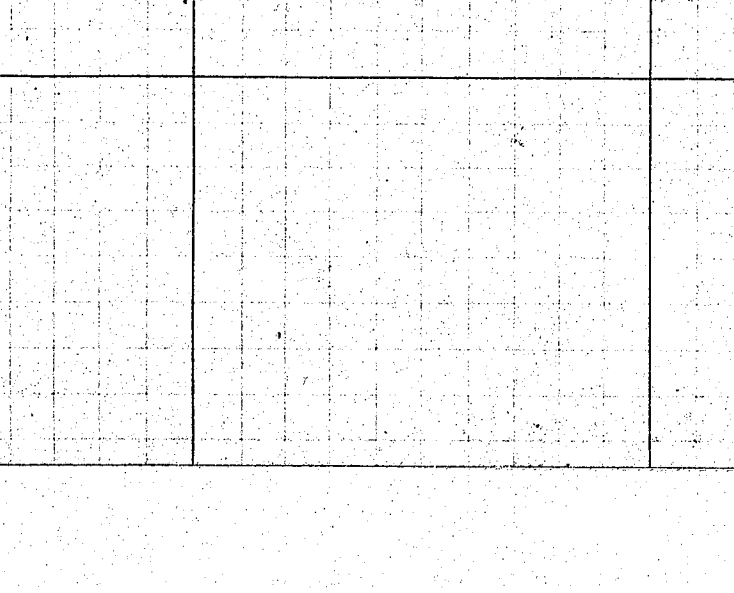
Section required to Carry this lateral force $\frac{73400}{500} = 147.0"$

2' Web is strong enough to resist this lateral force.

vertical bars 5/8"ϕ bars 2'-0" ctrs.

Sketch of Reinforcement in Pier.





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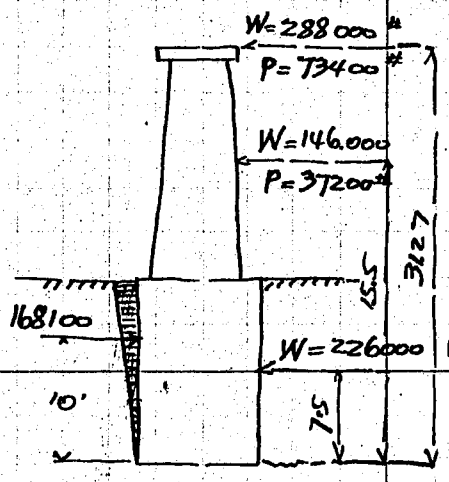
Dail Pressure at base due to Earthquake



		P.	arm	moment
$\frac{288000}{32.2} \times 8.2$	=	73400	31.27	= 2.295.000
$\frac{146000}{32.2} \times 8.2$	=	37200	15.5	= 576.000
$\frac{226000}{32.2} \times 8.2$	=	57500	7.5	= 431.000
		168100 #		3,302.000
Less 168100×10.0	=			− 1681.000
				1,621,000 #

M = 810500 # per cylinder

Moment of Inertia of cylinder base 8' dia

$I = 0.049 \times 96^4 = 4,160,000$ (inch)^4

Extreme fibre stress $= \frac{810500 \times 12 \times 48}{4,160.000} = 78$ #/o" or 11,200 #/o'

Total Downward load

Deck	288000
shaft	146000
base	226000
	660,000 #

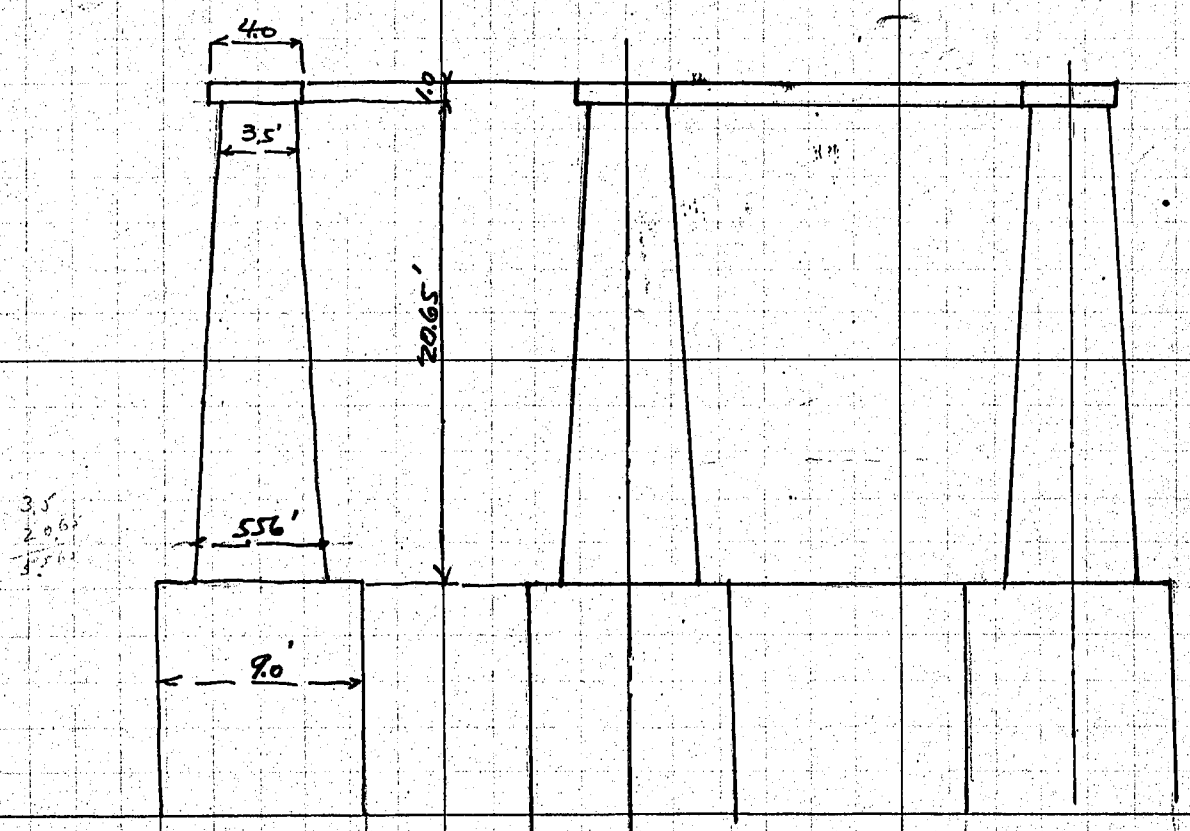
Unit pressure $= \frac{660.000}{100.6} =$ say 6600 #/o'

	6600
	17800 #/o'
or	7.95 tons/o'

Tensile stress $78 - \frac{6600}{144} = 32.2$ #/o"

OK without reinforcement.

Pier Shaft for Piers Nos 11 to 16 inclusive.



Coping 2 @ 4' dia $2 \times 12.57 = 25.14$

web $2.5 \times 15.42 = 38.60$

63.94

volume say 64 cubic ft

Total volume of coping + shaft $\frac{1315}{64} + 216 = 6.37$ say 7

weight = 1379 @ 150 = 207,000 #

Dead load on pier = 288.000 #

shaft Top 3.5' dia $2 \times 9.6 = 19.20$

web $2 \times 15.92 = 31.84$

51.04

Bott. 5.56' dia $2 \times 24.3 = 48.60$

web $2 \times 13.86 = 27.72$

76.32

127.36

Average 63.68

volume = $63.68 \times 20.65 = 1315$ cubic ft

or 6.1 cubic tsubo

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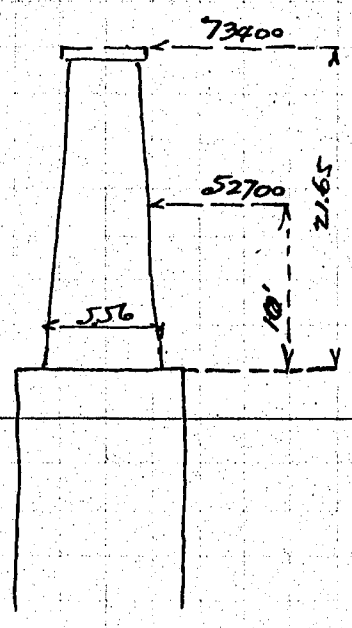


		Lateral Force	Arm	Moment
$\frac{288000}{32.2} \times 8.2$	=	73400	× 21.65	= 1590.000 ft#
$\frac{207000}{32.2} \times 8.2$	=	52700	× 10.0	= 527.000 ft#
				2117000 ft#
				1058500 ft# per shaft

Moment of Inertia at the bottom of shaft
dia say 66" $I = 0.049 \times 66^4 = 930.000$ (inch)^4

$$\text{Fibre Stress} = \frac{1058500 \times 12 \times 33}{930.000} = 450\ \#/0" \text{ Comp or Tension.}$$

Direct Load Deck 288000
shaft 207.000
495000#

Area of shaft at base –
66" dia 3421 0"
2@ 3421 = 6842 0"

Unit Stress = 495000 ÷ 6842 = Say 73 #/0"

Summary of Stress –

450 C	450 T
73 c	73 c
523	377 T

Let us reinforce shaft to resist tension
Round shaft assumed to be a square beam of 63" × 40"

Eccentricity $\frac{h}{h}$ 2117.000 ÷ 495000 = 4.275' $\quad e/h = 51/63 = .81$

Reinforcement on one side – 13 – 7/8"ϕ @ 0.6 = 7.8

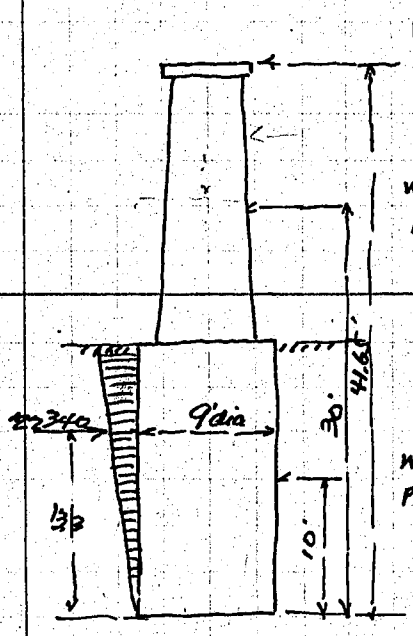
Steel % = $\frac{7.8}{40 \times 63} = 0.0031$ $\quad k = .11$

$f_c = \frac{1058500 \times 12}{.11 \times 40 \times 63^2} = 725\ \#/0"$ $\quad f_s = \frac{1058500 \times 12}{\frac{7}{8} \times 63 \times 7.8} = 29500\ \#/0"$

Use – 15 bars – 7/8"ϕ = 9.0 $\quad f_s = 29500 \times \frac{7.8}{9.0} = 25600\ \#/0"$ ok

Web to be carried down to top of base; Reinforcement same as for Pier No 1
Horizontal bars 7/8"ϕ bars 12" ctrs both sides vertical bars 7/8"ϕ – 24" ctrs

Stability of Pier nos 13 × 16
Height of base = 20'



	P	W
Deck	73400	288000
Shaft	52700	207000
base	97300	382000
	223400	877000

Weight of base 9' dia – 2 × 63.6 = 127.2
volume 127.2 × 20 = 2544 cubic ft
weight = 2544 × 150 = 382000#
P = $\frac{382000}{32.2} \times 8.2$ = 97300

Moment at bottom of base

73400	× 41.65	=	3060.000
52700	× 30.	=	1581000
97300	× 10.	=	973000
			5,614,000
Less 22340 × 13.3			− 2970.000
			2,644,000 ft#

Moment of Inertia of cylinder base d = 20' = 108"
$I = 0.049 \times 108^4 = 6.660.000$ (inch)^4

Extreme Fibre Stress = $\frac{1322,000 \times 12 \times 54}{6.660.000} = 129$

Unit pressure $\frac{877000}{223400} \div 127.2$

18600
6900
25500 #/0'
or 11.4 tons /0'

Tensile Stress = $129 - \frac{6900}{144} = 81\ \#/0"$ ok.

The bearing pressure is too high, let us figure the pier as long column and assume the point of pressure at 2/3 of pier height. The moment at this point will give maximum to pier body.

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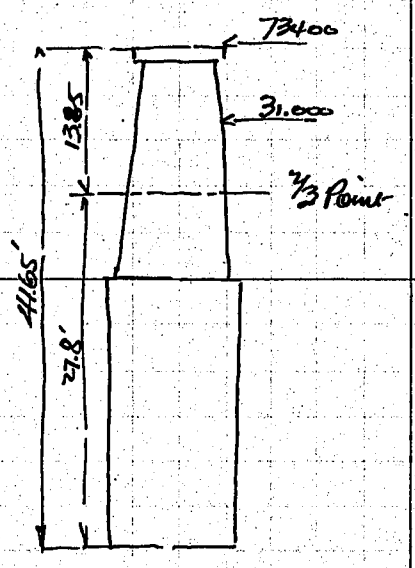
Bearing Pressure as long column



Area under coping 51.04
At 2/3 point 65.08
116.12 ÷ 2 = 58.06

58.06 × 12.85 = 747
64
811 @ 150 = 122000

cwt

$P = \frac{122000}{32.2} \times 8.2 = -31.000^{\#}$

Moment
73400 × 13.85 = 1,018.000
31000 × 6.8 abt = 211.000
1,229.000
614500 per base

Fibre stress = $\frac{614500 \times 12 \times 54}{6660.000}$ = 60#/o.
8630 #/o'
6900
15530 #/o' 6.93 tons/o'

Eccentricity 1229.000 ÷ 877000 = 1.4' Ok

Total load
Deck 288000
Shaft 207000
base 382000
877000 #

Unit pressure at bottom

Summary Remarks for Piers.

Piers nos 1 to 10 inclusive (10 piers)
Coping of shaft 4.0'
Under coping 3.5'
Height of shaft 15.27'
Batter of shaft 1/20
Pier base 2 @ 8' dia 15' high
Solid web 2' thick between shafts
shafts & web reinforced

Pier nos 11 to 16 inclusive 6 piers.
Same details of shaft for all piers.
Shaft
Coping of shaft 4.0'
Under coping 3.5
Height of shaft 20.65'
Batter of shaft 1/20
Solid web 2' thick between shafts
Shafts & web reinforced

Base
Piers nos 11 + 12 2-9' dia 15' high.
" " 13 + 16 2-9' dia 20' high
" " 14 + 15 2-9' dia 25' high.

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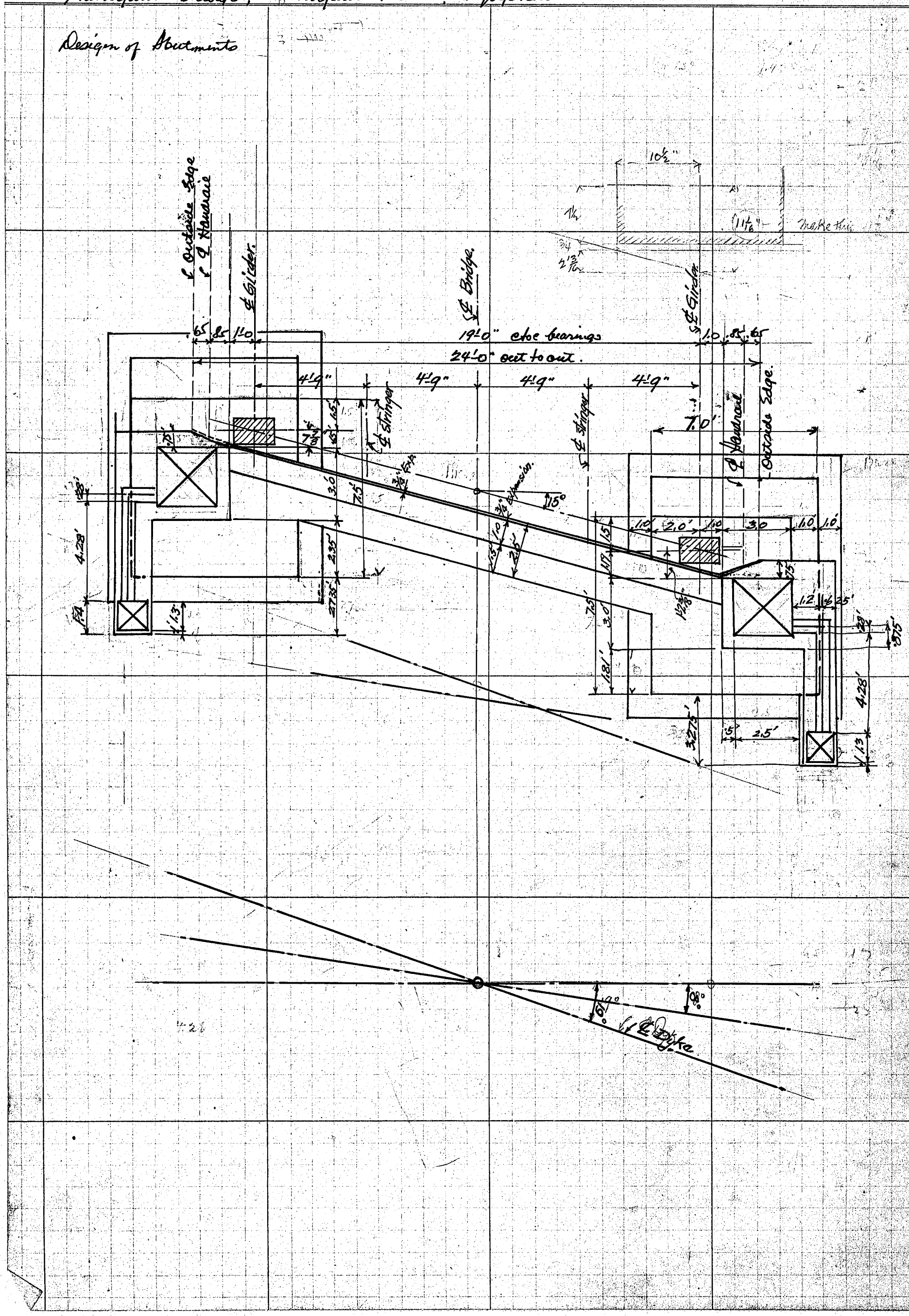
Design of Abutments



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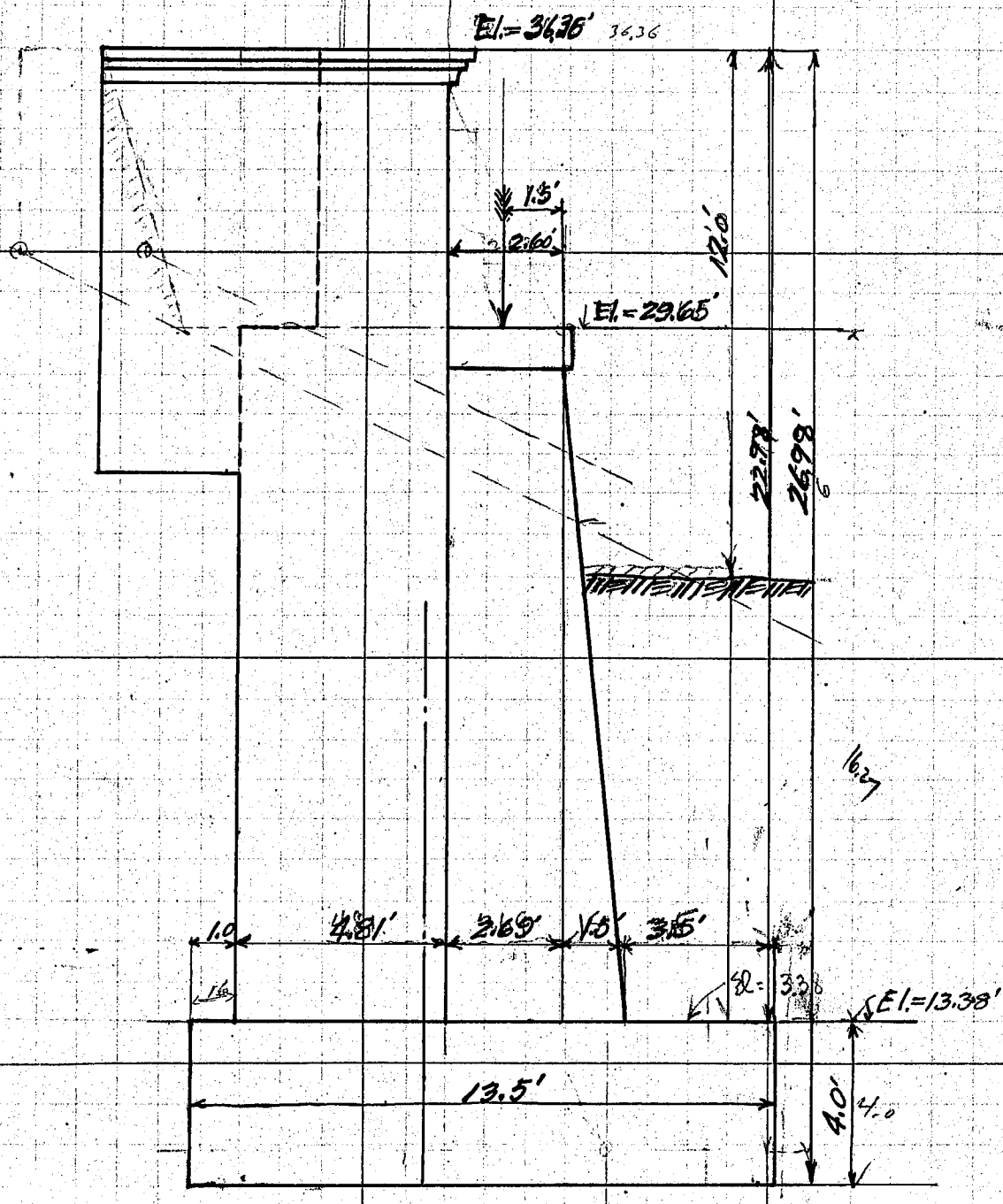

Approximate weight of structure above girder seat

						Arm Approx		
End Pedestal say	2.5 × 2.5 × say 4'	=	25.0	@150	= 3750#	× 1.25	=	4700
Base for pedestal	3.0 × 4.0 ×	6.5	= 78.0	@150	= 11600	× 1.50	=	17400
Wing wall	1.0 × 5.0 ×	6.5	= 32.5	@ "	= 4870	× 5.5	=	26800
" "	3.3 × 1.0 ×	3.8	= 10.9	@ "	= 1640	× 6.5	=	10700
Handrail say		5 @ 160			750	× 5.0	=	3700
Front wall	1.0 × 11.0 ×	6.5	= 71.5	@ 150	= 10700	× 0.5	=	5300
Earth fill on top	3.0 × 4.0 ×	6.5	= 78.0	@ 100	= 7800	× 3.0	=	23400
" " "	2.0 × 3.0 ×	6.5	= 39.0	@ 100	= 3900	× 4.0	=	15600
					45010	× 2.4	=	107600
Wall between shafts	2.5 × 7.0 × 16.0	=	280	@ 150	42000	× 0.7 adj	=	29400
					87010	1.57		137000

Call this 90.000# ×
Arm at toe 9.26
moment at toe = 90.000 × 9.26 = 835.000 ft#

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Approximate load on pier shaft

				arm		Moment at toe
Shaft	$\frac{1.5}{2} \cdot 5.925 \times 16.27$	=	72.2	× 4.5	=	325
	$2.69 \times 5.925 \times 16.27$	=	259.0	× 6.35	=	1645
	$4.81 \times 7.0 \times 16.27$	=	547.0	× 10.09	=	5520
Base	$9.0 \times 4.0 \times 13.5$	=	485.0	× 6.75	=	3280
			1363.2	7.90		10770

wt = 1363.2 @ 150 = 204.000 #

Earth fill approximate only

				arm		
At Front	$3 \times 5 \cdot 9 \cdot 10$	=	3.15	× say 1.75	=	551
Rear	$1.0 \cdot 9 \times 23$	=	207	× 13.0	=	2690
sides	$2.0 \cdot 9 \times 13.5$	=	243	× 8.0	=	1940
			765	6.76		5181

wt = 765 × 100 = 76500 #

Moment at Toe					
shaft & base	204.000	×	7.90	=	1610.000
Earth fill	76500	×	6.76	=	518.000
Above bearing	90.000	×	9.26	=	835.000
superimposed load	140.000	×	6.5	=	910.000
	510500				3873.000 ft#

Horizontal Earth Pressure

At shaft	$P = \frac{1}{6} \times 100 \cdot 26.77^2$	=	11920 #	×	7.0	=	83500	× $\frac{26.77}{3}$	= 745000
	$P' = \frac{1}{6} \times 100 \cdot 14.77^2$	=	3640	×	7.0	=	25450	× $\frac{14.77}{3}$	= −125000
									620,000 ft#
At base	$\frac{1}{6} \times 100 \times 22.77^2$	=	8600	×	7.5	=	64500	× $\frac{22.77}{3}$	= 488.000
	$\frac{1}{6} \times 100 \times 10.77^2$	=	1935	×	7.5	=	14500	× $\frac{10.77}{3}$	= −52.000
									436000
						Total Horizontal moment			1056000

Point of resultant at base

vertical load 510500 #

vert. m 3873.000

Hor. m − 1056.000

2817.000

arm = 2817.000 ÷ 510500 = 5.52

Eccentricity 6.75 − 5.52 = 1.23

max Toe Pressure

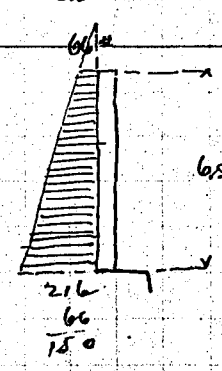
$\frac{510500}{9 \cdot 13.5} \left(1 \pm \frac{6 \cdot 1.23}{13.5}\right)$ = 6500 #/ft² max

2.9 tons per sq ft.

say 6 tons.

Reinforcement in vertical walls.

1.0 × 6.5' 2' surcharge assumed



moment $66 \cdot 6.5 \cdot \frac{6.5}{2}$ = 1395

$\frac{1}{3} \times 100 \cdot 6.5 = 216$ #

$\frac{150}{2} \cdot 6.5 \cdot \frac{6.5}{3}$ = 1060

2455 ft# per ft

Effective depth required for 600 × 16000 stresses

$\sqrt{\frac{2455}{955}}$ = 16" 4.9" steel = $\frac{2455 \times 12}{7/8 \cdot 10" \cdot 16000}$ = .210" per ft

make wall 12" thick use ½" ϕ bars 12" ctrs at rear

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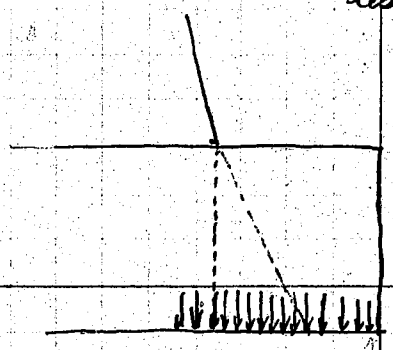
Reinforcement at front (ver) ½"φ bars 24" ctrs about
(Hor) ½"φ bars 24" ctrs ..

Reinforcement in wall between shafts.
2.5' thick. Use same reinforcement as for the wall between Pier shafts.

Reinforcement in wing wall.
make main reinforcement ½"φ bars 12" ctrs unless otherwise 24" ctrs.

Stresses at Toe Approximate only
Projection 3.5' upward pressure say 6500 #/□'
less 150 × 4 600
5900 #/□'.



Uniformly distributed load
$\frac{51,500}{9.135} = 4200$ #/□'.

Upward moment $5900 \times \frac{3.5^2}{2} = 36,100$ ft#

Assuming the load distributed over base.
upward pressure say 6500
less 4200
2300 #

Upward moment $2300 \times \frac{3.5^2}{2} = 14100$ ft#

Fibre stress in concrete $= \frac{6 \times 14100}{12 \times 48 \times 48} = 37$ #/□.

OK without reinforcement.

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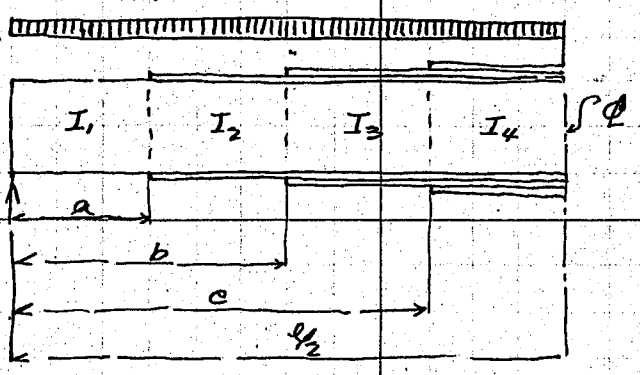
Katsogawa Bridge, Katsogawa Machi, Hyogoken

Deflection of girder

General formula of deflection at the center of span with varying moment of inertia under uniform load.

w pounds per linear unit.



E = Modulus of Elasticity
I = moment of Inertia of section
w = uniform load on girder

$$\text{Deflection} = \frac{w}{2E}\int_0^a \frac{(lx^2 - x^3)}{I_1}dx + \frac{w}{2E}\int_a^b \frac{(lx^2 - x^3)}{I_2}dx + \frac{w}{2E}\int_b^c \frac{(lx^2 - x^3)}{I_3}dx + \frac{w}{2E}\int_c^{\frac{l}{2}} \frac{(lx^2 - x^3)}{I_4}dx$$

$$= \frac{w}{2E}\left\{ \frac{la^3}{3I_1} - \frac{a^4}{4I_1} + l\frac{(b^3 - a^3)}{3I_2} - \frac{b^4 - a^4}{4I_2} + \frac{l(c^3 - b^3)}{3I_3} - \frac{c^4 - b^4}{4I_3} + \frac{l(l^3 - 8c^3)}{24I_4} - \frac{l^4 - 16c^4}{64I_4} \right.$$

$$= \frac{5}{384}\frac{wl^4}{EI_4} + \frac{we}{6E}\left(\frac{a^3}{I_1} - \frac{a^3}{I_2} + \frac{b^3}{I_2} - \frac{b^3}{I_3} + \frac{c^3}{I_3} - \frac{c^3}{I_4}\right) - \frac{w}{8E}\left(\frac{a^4}{I_1} - \frac{a^4}{I_2} + \frac{b^4}{I_2} - \frac{b^4}{I_3} + \frac{c^4}{I_4} - \frac{c^4}{I_4}\right)$$

A B

Moment of inertia of girder section. (Gross Area).

			For both Flgs	arm		Moment of Inertia	83%
1 web plate	60 × 3/8	= 22.5	22.50			6750	5600
Flange angles	2 ⌞ 6 × 6 × 3/4	= 16.44	32.88	28.47²	+ 113	= 26730	22180
1 cov. plate	14 × 3/8	= 5.25	10.50	30.44²		= 9725	8060
1 Cov. plate	14 × 3/8	= 5.25	10.50	30.82²		= 9975	8270
1 Cov. plate	14 × 3/8	= 5.25	10.50	31.20²		= 10220	8480
						63400	52590

Reduce moment of inertia for net section by $\frac{52588}{63400} = 83\%$

Moment of Inertia $I_4 = 52590$ (inch)$^4 = 45.13$ (ft)^4

" " " $I_3 = 44110 = 14.50$

" " " $I_2 = 35840 = 13.76$

" " " $I_1 = 27780 = 12.90$

				$\frac{1}{I_1}$	$\frac{1}{I_2}$	$\frac{1}{I_3}$
a	8.92′	cube	710	= 0.0776	0.0727	0.0690
b	13.52	cube	2471			
c	20.71	cube	8882	$-\frac{1}{I_2}$ = −0.0727	$\frac{1}{I_3}$ = −0.0690	$\frac{1}{I_4}$ = −0.06611
l	72.85	quad	28,150,000	0.0049	0.0037	0.0029

Coefficient A

0.0049	× 710	=	3.48
0.0037	× 2471	=	9.15
0.0029	× 8882	=	25.75
			38.38

Coefficient B.

3.48	× 8.92	=	31.0
9.15	× 13.52	=	123.6
25.75	× 20.71	=	534.0
			688.6

Dead load uniform load 1935# per lin ft

Live load " " say 1200 " " "

Deflection due to dead load

$$\frac{5}{384} \times \frac{1935 \times 28.150.000}{144 \times 30.000.000 \times 15.13} = 0.010796$$

$$\frac{1935 \times 72.85 \times 38.38}{6 \times 144 \times 30.000.000} = 0.000071$$

$$\frac{1935 \times 688.6}{8 \times 144 \times 30.000.000} = -0.000004$$

$$0.011103'$$

Deflection due to live load

$$0.0110 \times \frac{1200}{1935} = 0.0068$$

0.0110

Total Deflection 0.0178′

1/4" about.

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Estimate of Cost.

1.	Structural Steel (superstructure only).	560.196 tons	@	230.00 =	128845.00
2.	Structural Steel erected	560.196 "	@	65.00 =	36412.00
3.	Structural steel painted	560.196 "	@	11.00 =	6162.00
4.	Cement mortar finish complete		@		2100.00
5.	Wood block pavement with cushion	735.3 坪	@	40.00 =	29412.00
6	Concrete in floor slab.	79.77 立坪	@	170.00 =	13561.00
7	Forms for concrete floor slab	855.5 坪	@	5.00 =	4277.00
8	Handrails & pedestals complete		@	=	5222.00
9	Reinforcing steel in floor slab.	53.216 tons	@	185.00 =	9845.00
10	Concrete in pier shafts (1:2:4)	82.74 立坪	@	170.00 =	14066.00
11	Concrete in bases (1:3:6).	82.98 "	@	130.00 =	10760.00
12	Concrete in bases concrete blocks 1:2:4	61.02 "	@	250.00 =	15255.00
13	Cement mortar for joints of Concrete blocks	648 立尺	@	.80 =	520.00
14	Excavation Complete for piers 1 to 10	70 立坪	@	100.00 =	7000.00
15	Excavation Complete for piers 11 to 16	89.46 立坪	@	200.00 =	17892.00
16	Reinforcing Steel complete in piers	36.70 tons	@	185.00 =	6789.00
17	Forms complete for piers	445 坪	@	8.00 =	3560.00
18	Concrete in abutments 1:2:4	34.22 立坪	@	170.00 =	5817.00
19	Reinforcing Steel in abutments	1.56 坪	@	185.00 =	289.00
20	Forms complete	149 坪	@	12.00 =	1790.00
21	Excavation Complete for abutments	98 立坪	@	7.50 =	735.00
22	Electric wiring & lighting Complete		@		5000.00
23	Miscellaneous works		@		2000.00
24	Structural steel in shoes	5.6 tons	@	340.00 =	1900.00
					329209.00

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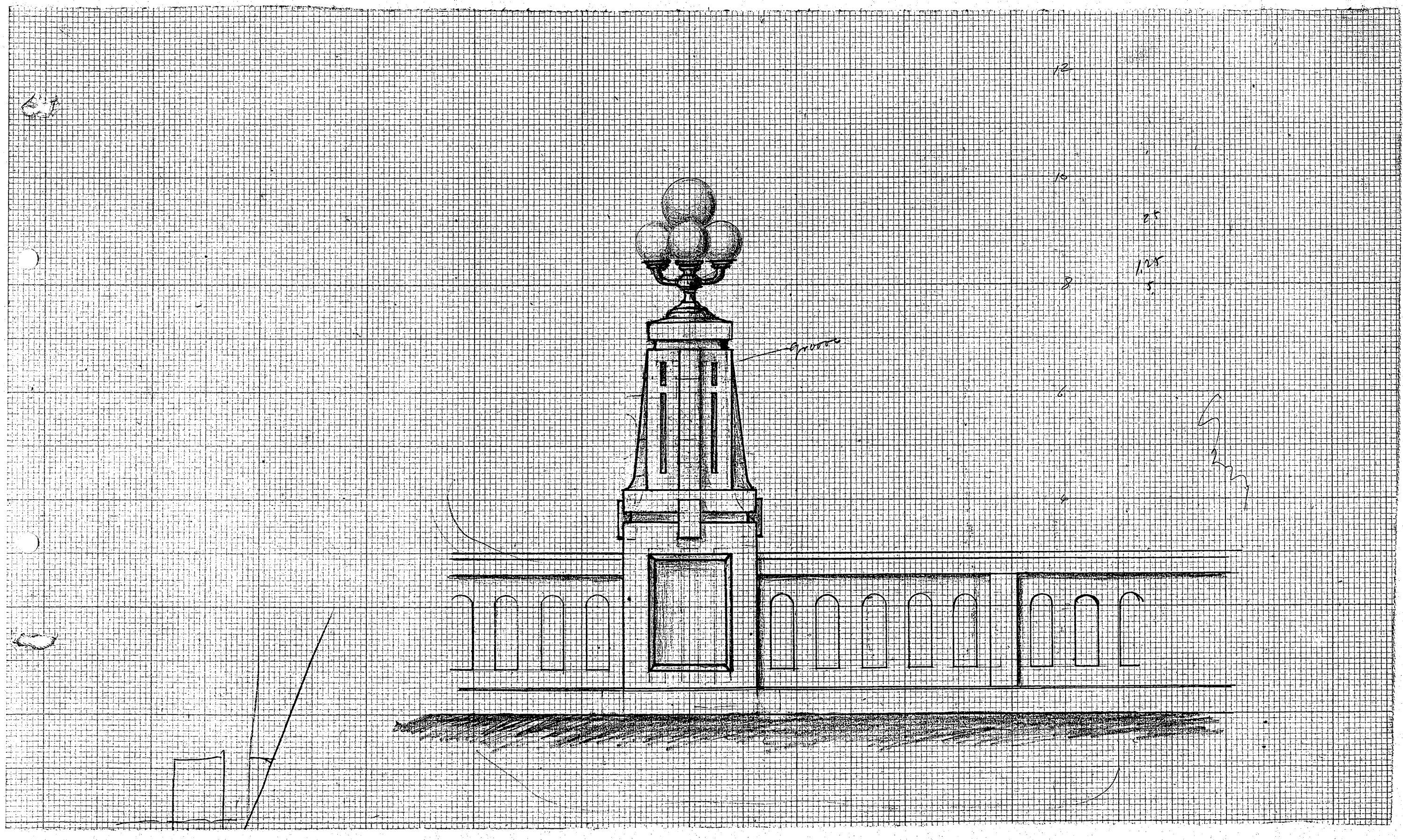
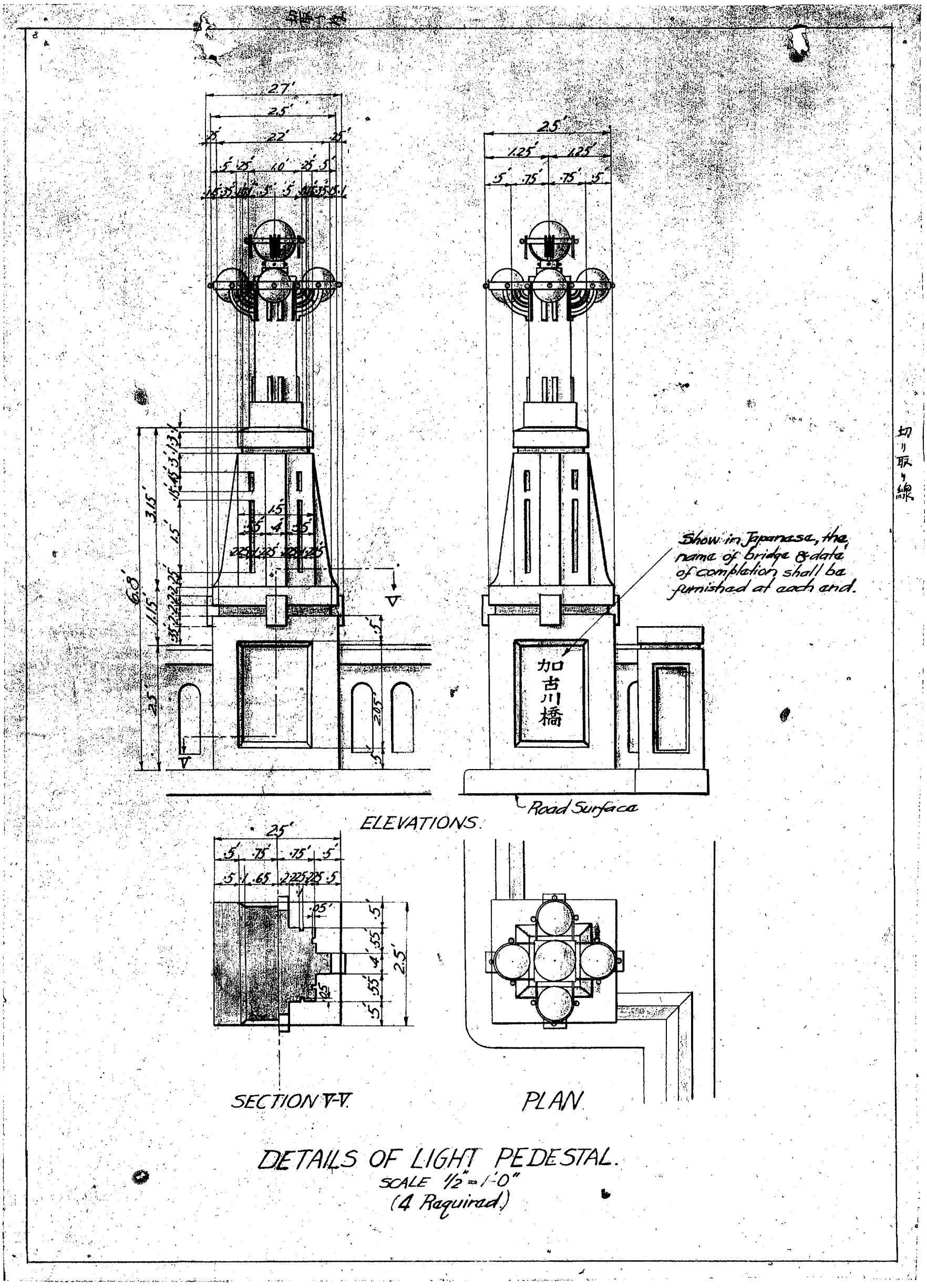
(独立行政法人 土木研究所蔵)





増田淳氏関係資料
（独立行政法人 土木研究所蔵）

CALCURATION PAPERS

FOR

MUKO-O-HASHI.

Designed by K. Inaba.
Consulted by J. Masuda.

1925.

HYOGO-KEN.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Inaba DATE 13 8 25 FILE NO
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CALCULATIONS FOR

Mutsu-Ohashi over Mutsugawa, Hyogoken.

The Bridge is located at the crossing of Mutsugawa for the new Han-Shin National Road which is in the course of execution. The bridge site is just at the down stream of the Imperial Government Railway Bridge (about ken). The centre line of the new bridge is square to the stream line and also to the banks of the river.

The electric railway is 17.9 ft. (3.0 ken) wide and is located at the centre of the bridge, both sides of which are trafficways each 17.9 ft. (3.0 ken) wide. The out side of the trafficways, there are 2 sidewalks each 5.97 ft. (1.0 ken) wide. Now the clear width of the roadway is 65.64 ft. (11.0 ken) in total.

The structure will have 6 – arch spans 66' each at the centre and reinforced concrete girder spans 27 ft each at both approaches; 6 spans for the east and 3 for the west. The total length of the bridge proper will be ft long end to end.

The grade shall be parabolic for the portion of arch spans and straight lines which are nearly tangent to the previous curve for girder spans giving the total camber of 5 ft about.

Assumed Loadings:-

On Electric Railway, double lines of 33 ton (66,000 #) bogie Cars in series, or 12 ton motor trucks running side by side in a single line transversely, for both cases, impact allowance shall be taken at $I = \frac{20}{60+l} \leqq 0.3$. The out side of the occupied area of cars or trucks shall be filled with a uniform live load of $w = \frac{120{,}000}{170+l} \leqq 600\ kg/m^2 = 125\ \#/ft^2$.

On Trafficways, 2 – 12 ton motor trucks running side by side singly with the same impact as before. All around the area occupied by trucks shall be loaded as before.

On sidewalks, a uniform live load of $w = \frac{100{,}000}{170+l} \leqq 500\ kg/m^2 = 100\ \#/ft^2$

As a bridge whole, the following 3 cases shall be considered and taken the greater value.

Case 1. 2 lines of 33 ton bogie cars in series and a 12 ton motor truck on both sides with the same impact as before. For uniform live load, the same as before.

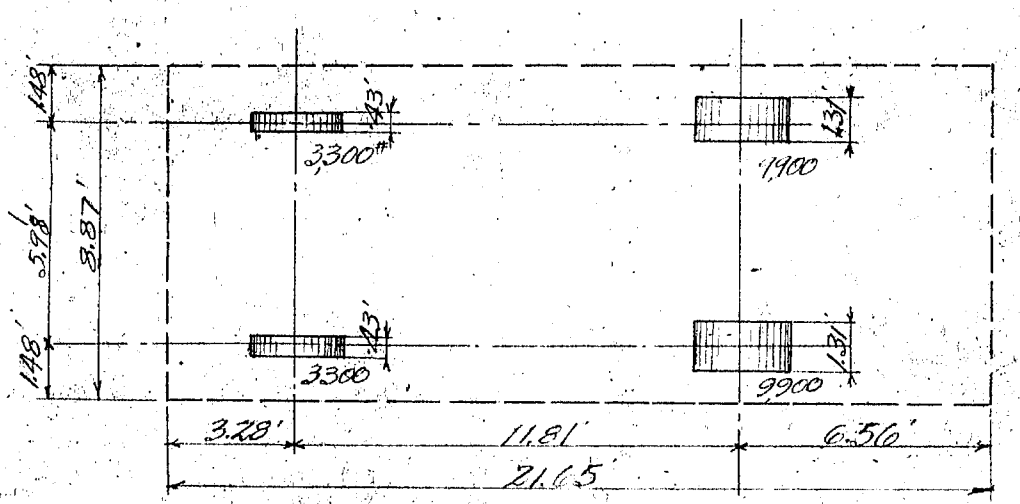
Case 2. 4 lines of 12 ton motor trucks running side by side in a single line transversely. For impact allowance and uniform live load, all the same as before.

Case 3. Combination of one 14 ton road roller with no impact and either of above two cases but for the case of motor truck only one truck shall be considered

Seismic Load.

Earthquake coefficient $K = 0.30$ or acceleration of earthquake = 3000 mm/sec².

12 ton Motor Truck Loading.



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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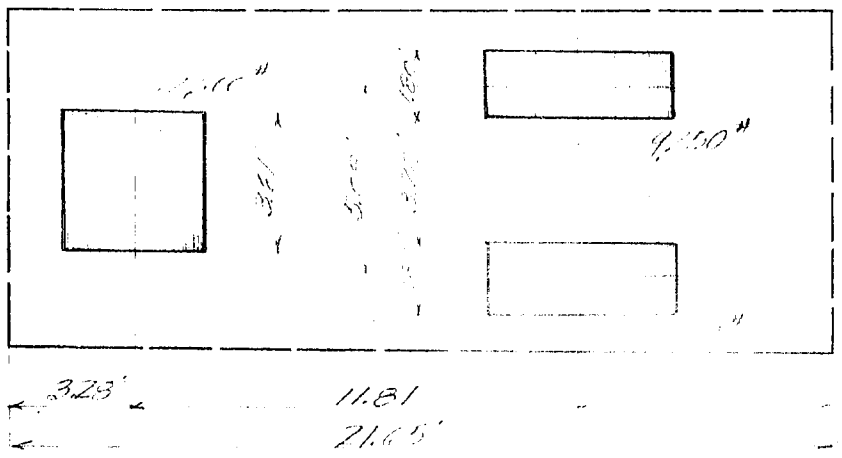
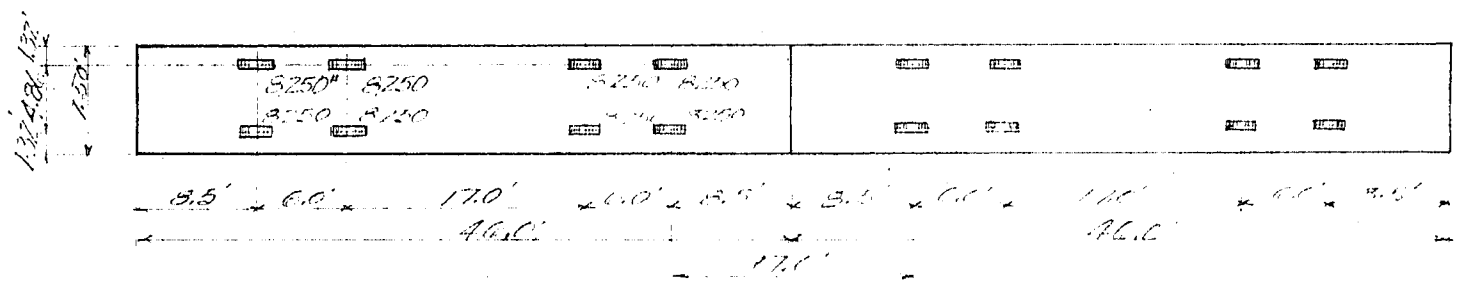
CALCULATIONS FOR

Mube Ohashi over Mukogawa, Hyogoken.

14 ten Road Roller Loading.



33 ton (66,000 lbs) Electric Car Loading (full Load.)



Allowable Unit Stresses.

	Case I. dead load & live load with Impact.		Case II. Temperature included in case I.	
	C.G.S. Unit.	F.P.S. Equivalents.	C.G.S. Unit.	F.P.S. Equivalents.
Tension in Steel	1,200 kg/cm²	17070 #/□"	1500 kg/cm²	21340 #/□"
Compression in Steel	1,200 "	17070 "	1500 "	21340 "
Tension in 1:2:4 Concrete	neglected	neglected	neglected	neglected.
Compression in 1:2:4 Concrete				
due to direct stress	35	500	43.8	620
" " bending	45	640	56.3	800
" " direct stress & bending	32	500	43.8	620
for deck ring	45	640	56.3	800
Shear pedia	1	57	5	71
punching	9	128	11.3	160
Bearing	40	570	56.3	800
Bending	6	85.0	7.5	107

In case of including the effect of earthquake, add 50 % more to the allowable unit stresses given in Case I.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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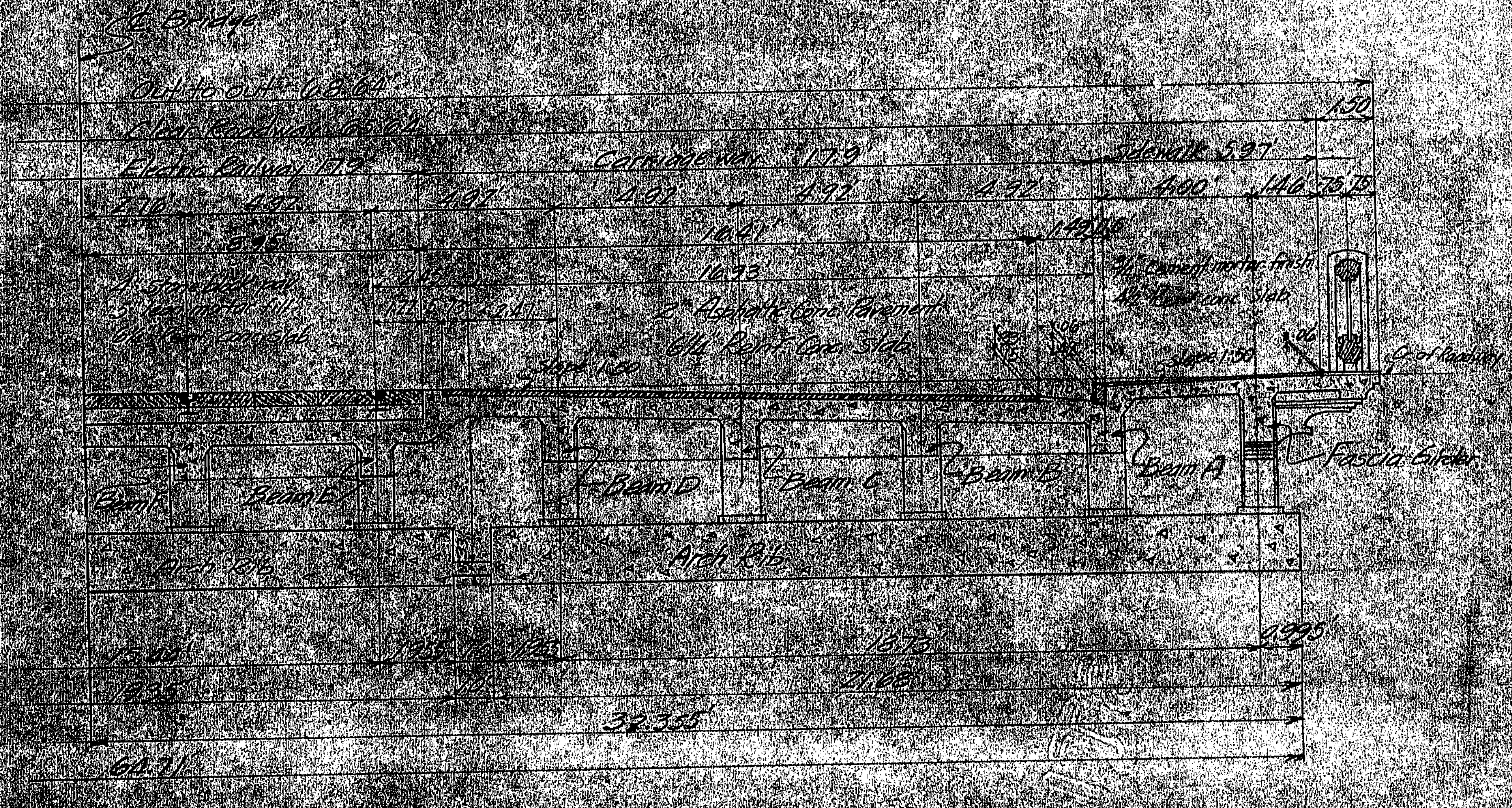
CALCULATIONS FOR

Muko Ohashi over Mukogawa, Hyogoken
Cross Section of Super Structure
for Arch Spans.



Floor System on 66' Arch Span
The span will be divided into 12 equal panels of 5.5' each. Construction and details of floor system as shown in the above figure.

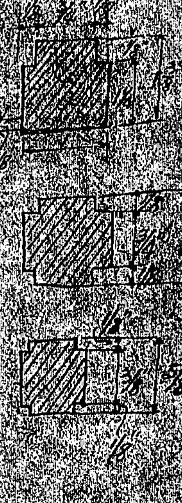
Dead Weight

Handrail: see details in next page.

Casting
[illegible] $= 0.1154$
Sectional area $1.25 \times 1 = 1.25^{\square''}$, $.75 \times 1.25 = 0.09$, $1.34^{\square''}$

[illegible] $= 0.0219$
[illegible] $= 0.1175$
[illegible] $= 0.0313$
$0.75 \times 1 = 0.750^{\square''}$, $2 \times 0.75 \times 1.25 = 0.19$, $0.94^{\square''}$

Diagonals [illegible] $= 0.0112$
$= 0.328^{\square''}$
Total $= 0.2973^{cu.ft} @ 453^{\#} = 134.7^{\#}$



Granite
Rail post [illegible] $= 1.349^{cu.ft}$
Int. post [illegible] $= 0.506$
Top rail [illegible] $= 1.785$
[illegible] $= 0.122$
Bottom rail [illegible] $= 0.616$
[illegible] $= 0.384$
Moulding [illegible] $= 0.030$
[illegible] $= 0.312$
[illegible] $= 0.296$

Total $5.388^{cu.ft} @ 160^{\#} = 862.1^{\#}$
Total weight of hand rail per 5.5 ft $= 996.8^{\#}$
per lin. ft $= 181.2^{\#}$

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTING ENGINEER
CALCULATIONS FOR

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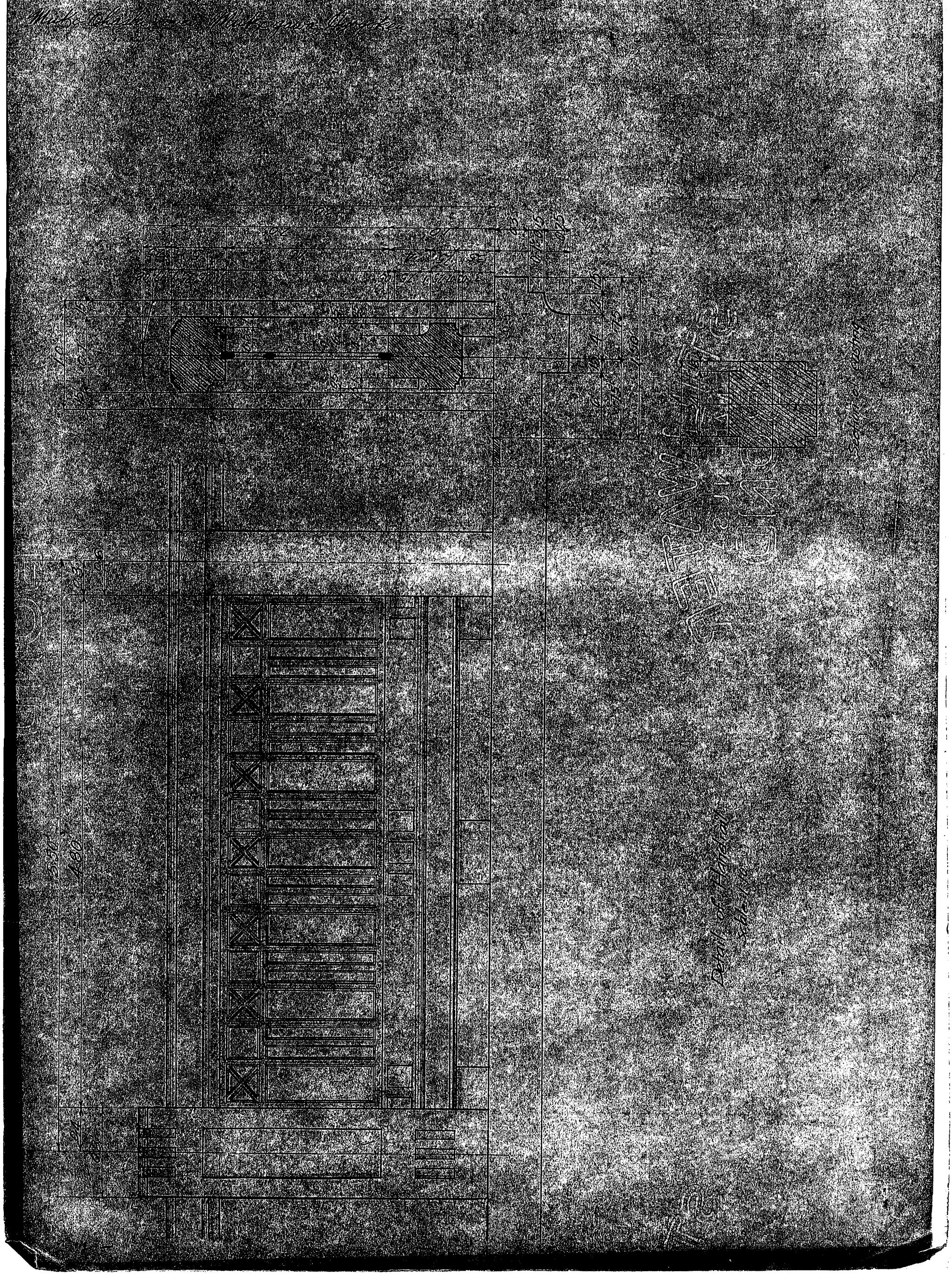

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Inaba DATE 13-9-9 FILE NO
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CALCULATIONS FOR

Mukō-Ohashi over Mukōgawa, Hyogo-ken.
Concrete Floor Slab for Sidewalk.

Sidewalk slab, cantilever arm 2.21' between ℄ of fascia girder and ℄ of handrail.

Dead Load Moment.

Name	Volume	unit weight	total wt.	lever arm		cantilever moment.
Pavement ¾" mortar	1.46×.006=.0088@	110#	9.7#	× 0.73'	=	7'#
4½" Slab assumed	1.46×.37=.540@	150	85.0	× 0.73	=	62
Handrail			181.2	× 2.21	=	400
Coping	1.5×.3 =.45@	150	67.5	× 2.21	=	149
Coping "	1.35×.15=.203@	150	30.5	× 2.14	=	65
"	.6×.25=.150@	150	22.5	× 2.36	=	53
		Shear =	396.4#	5.35		Total m = 736'#

Live Load Moment.
uniform live load of 100 #/0'

m = 100×1.46×0.73 = −107'# End shear = 100×1.46 = 146#

hor. pressure on top of handrail 45#/1' of handrail

M = 45×3.0' = −135

total m = −242'#

Summary of moment and shear.

	Moment	Shear.
Dead Load	−736	396
Live Load	−242	146
Total	−978'#	542#

Effective depth of slab for steel 17,000 #/0", concrete 640 #/0", $d = \sqrt{\frac{978}{102}} = 3.1$"

Steel required = $\frac{978\times12}{7/8\times3.5\times17{,}000} = 0.226$ 0"/ft. Make slab 4½" over all with 1" insulation.

use ⅜"ϕ bars 4" C to C = 0.332 0"

unit shear = $\frac{542}{12\times7/8\times3.5} = 14.8$ #/0" OK

unit bond = $\frac{542}{1.178\times3\times7/8\times3.5} = 5.10$ #/0" OK.

Sidewalk slab between outer 2 girders. Span length 40'

¾" mortar finish .06×110 = 6.6 #/0'
4½" Conc. slab assumed .37×150 = 55.5
62.1 #/0' Call this 65 #/0' including fillets over girders.

Dead Load moment = $\frac{65\times4^2}{10} = 104$'# Dead Load shear = 65×2 = 130#
Live Load moment = $\frac{100\times4^2}{10} = 160$ Live Load shear = 100×2 = 200
Total m = 264'# Total V = 330#

Neg. moment at the centre of span = −978 + 264 = −714'# say.
Moments and shear, smaller that of cantilever slab, so use the same reinforcement (Main reinforcement at top of slab)

Motor Trafficway Slab. 4.92' span, pavement 2" asphaltic concrete.
2" asphaltic concrete @ 140/12 = 22 #/0'
6½" Concrete slab assumed .54@150 = 81
103 #/0' Call this 105 #/0' including fillets on girders.

Dead Load moment = $\frac{105\times4.92^2}{10} = 253$'#
Dead Load shear = $\frac{105\times4.92}{2} = 258$#

Live Load Rear wheel concentration 9,900#
Impact = $\frac{20}{60+\frac{4.92}{3.28}} = 0.325$ Impact assumed 30% 2970
12,870# including impact.

Distribution of wheel concentration.
Dimension of contact between wheel and pavement assumed 20 cm = 0.66'
Thickness of asphaltic pav. 2"×2 = 0.34'
1.00' = a.

Effective width E = ⅔(l+b)+a where l = span length = 4.92'
= ⅔(4.92+1.65)+1.0 = 5.37 ft. t = 1.31
0.34'
1.65' = b

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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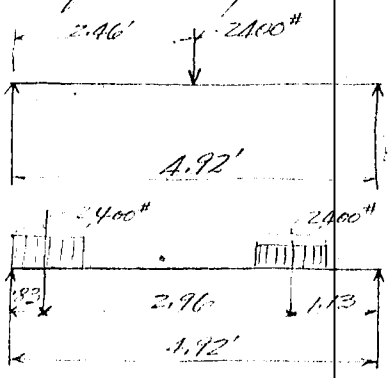
CALCULATIONS FOR

Mukyo-Chashi over Mukogawa, Hyogoken.
Motor Trafficway Slab Continued.
Load for 1' strip = 12570÷5.37' = 2,400#



Live Load moment = 1,200×2.46 = 2,940'# as a simple beam.
0.8×2,940 = 2,350'# as a continuous beam.
Max. End shear = $\frac{2400\times(4.09+1.13)}{4.92}$ = 2,550#

Summary for moment and shear.

	moment	shear.
Dead Load	253	1258
Live Load	2350	2550
Total	2,603'#	2,808#

Effective depth of slab for steel 17,000#/□", concrete 640#/□", $d=\sqrt{\frac{2603}{102}}$ = 5.07"
make slab 6¼" thick over all with 1" insulation.

Steel required = $\frac{2603\times12}{7/8\times5.125\times17,000}$ = 0.401□" per ft.

use ½"ϕ bars 4" ctrs = 0.472□" per ft.

unit shear $v=\frac{2808}{12\times7/8\times5.125}$ = 50.9#/□" o.k.

Unit bond $u=\frac{2,808}{1.571\times3\times7/8\times5.125}$ = 128#/□" Use deformed bars.

Floor slab under Electric Railway tracks. Span length 5.0'

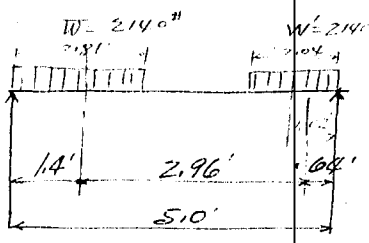
4" stone block pavement	@	16/12	= 54#/□'
5" lean mortar filling	@	110/12	= 46
6½" concrete slab assumed	@	15/12	= 81
			181#/□'

Dead load moment = $\frac{1}{10}\times181\times5^2$ = 452'#
Dead load shear = $181\times5\times\frac{1}{2}$ = 452'#

Live Load motor truck rear wheel concentration with 30% impact = 12,870#
Distribution of wheel load Lengthwise a = .66 + 1.50 = 2.16'
Crosswise b = 1.31 + 1.50 = 2.81'
Span length l = 5.0'

E = ⅔(5.00+2.81)+2.16 = 7.36' Assume distribution 6.0' wide.
Load for one ft. strip of slab = $\frac{12870}{6}$ = 2,140#

Live Load moment, concentration at centre 1070×2.5 = 2680'# as a simple beam.
2680×0.8 = 2,140'# as a continuous beam.



Max End Shear 2140×3.6/5 = 1540#
1530×1.02/5 = 320
1,860#

Summary of moment and shear.

	moment	shear.
Dead Load	452'#	452#
Live Load	2140	1860
Total	2,592'#	2,312#

Effective depth of slab slab for steel 17,000#/□" Concrete 640#/□" $d=\sqrt{\frac{2592}{102}}$ = 5.04"
Make slab 6¼" over all with 1" insulation.
Use the same reinforcements as for motor traffic way slab.
unit shear $v=\frac{2312}{12\times7/8\times5.25}$ = 42#/□" o.k.

Unit bond $u=\frac{2,312}{1.571\times3\times7/8\times5.25}$ = 107#/□" Use deformed bars.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Mutsu-Ohashi over Mutsugawa, Hyogoken.

Floor Slab at Centre. Span 5.44'

Dead Load per sq. ft = 181.

Dead Load moment = $\frac{1}{10} \times 181 \times 5.44^2 = 535'^{\#}$

Dead Load Shear = $181 \times 5.44 \times \frac{1}{2} = 493^{\#}$

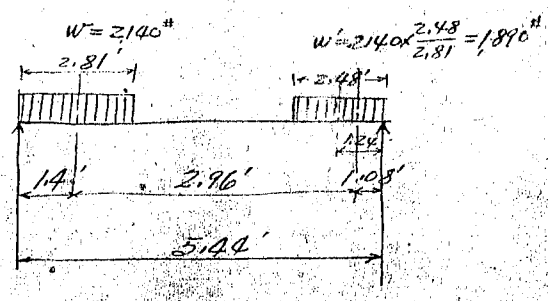
Live Load motor truck rear wheel concentration with 30% impact = 12,870#

Distribution of load, same as before.

Load per 1 ft. strip of slab = 2140#

Live Load moment, concentration at centre $1,070 \times 2.72 = 2,910'^{\#}$ as a simple beam.

$2,910 \times 0.8 = 2,330'^{\#}$ as a continuous beam.



Max. End Shear $2140 \times \frac{4.04}{5.44} = 1,590^{\#}$

$1890 \times \frac{1.24}{5.44} = \underline{431}$

$2,021^{\#}$

Summary of moment and shear.

	moment	shear.
Dead Load	535'#	493#
Live Load	2330	2021
Total	2,865	2,514

Effective depth of slab for steel 17000 #/□". concrete 640 #/□". $d = \sqrt{\frac{2865}{102}} = 5.3"$

make slab 6¼" over all with 1" insulation.

Steel required = $\frac{2865 \times 12}{7/8 \times 5.25 \times 17000} = 0.440"$ per ft.

use ½"ϕ bars 4" c/c. = 0.472"

unit shear $v = \frac{2514}{12 \times 7/8 \times 5.25} = 45.6$ #/□" Ok.

unit bond $u = \frac{2514}{5 \times 1.571 \times 7/8 \times 5.25} = 116$ #/□" require web reinforcement.

Design of Beams under Track. continuous beams of 5.5' span (Beam F).

Comparing the effects on girders due to Electric Ry. trucks and motor trucks, the case of max. moment and shear is taken as follows.

Dead Load slab. $181 \times \frac{5.44+5.0}{2} = 945$ #/ft run

Stem of beam $.83 \times .92 \times 150 = 115$

rail and accessories 30

1,090 #/ft run

Dead Load moment = $\frac{1090 \times 5.5^2}{10} = 3,300^{\#}$

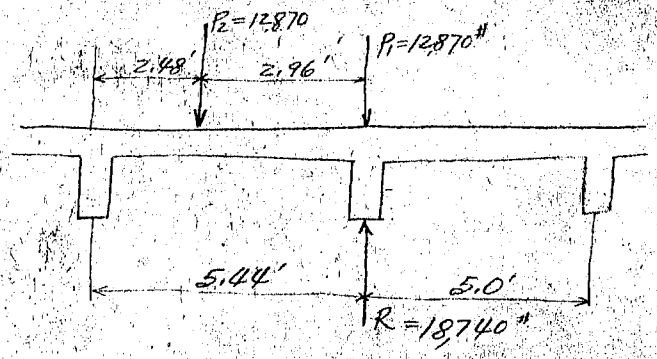
Dead Load Shear = $1090 \times \frac{5.5}{2} = 3,000^{\#}$

weight of one rail $\frac{85}{3} \times 1.2 = 28.4$

accessories $\frac{2.8}{31.2}$ #/ft.

Call this 30#/ft additional to flooring.

Live Load. Rear wheel concentration of motor truck 12,870# including impact.



Concentration on girder $12870 \times \frac{2.48}{5.44} = 5,870^{\#}$

$12,870$

$18,740^{\#}$

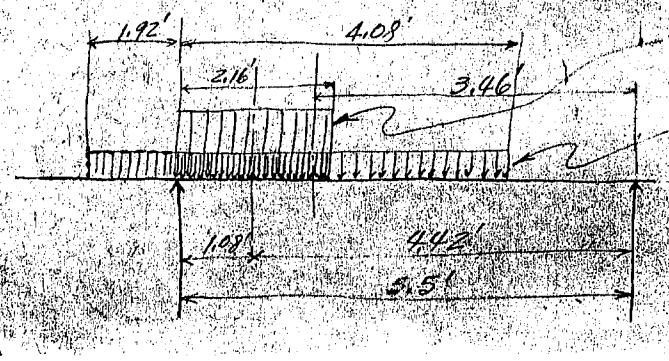
Live Load moment = $\frac{18,740 \times 5.5}{4} = 25,800'^{\#}$ as a simple beam.

$25,800 \times 0.8 = 20,600'^{\#}$ as a continuous beam.

max End Shear = $12870 \times \frac{4.42}{5.5} = 10,350^{\#}$

$5870 \times \frac{4.08}{6.0} \times \frac{3.46}{5.5} = 2,510^{\#}$

total = $12,860^{\#}$



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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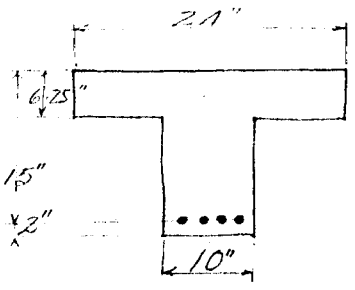
CALCULATIONS FOR

Mukyo-Ohashi over Mukogawa, Hyogoken.
Beams under Tracks Continued.
Summary of moment and shear.

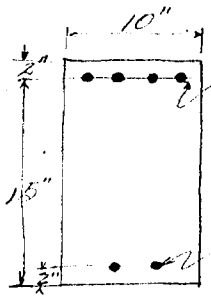
	Moment	Shear
Dead Load	3,300'#	3000 #
Live Load	20600	12860
Total	23,900'#	15860 #



$t/d = 6.25/15 = 0.417$ Neutral axis in the flange.
Effective depth $= \sqrt{\frac{23900\times12}{24\times102}} = 10.3"$
Effective depth necessary for 120#/□" shear $= \frac{15860}{10\times7/8\times120} = 15.1"$

Make depth of beam 17" over all with 2" insulation.
Steel required $= \frac{23900\times12}{7/8\times15\times17000} = 1.283\,□"$
use 4 - 3/4"ϕ $= 1.765\,□"$
unit shear $v = \frac{15860}{10\times7/8\times15} = 121$ #/□" require web reinforcement.
unit bond $u = \frac{15860}{\pi\times3.55\times4\times7/8\times15} = 128$ #/□" Use deformed bars OK.

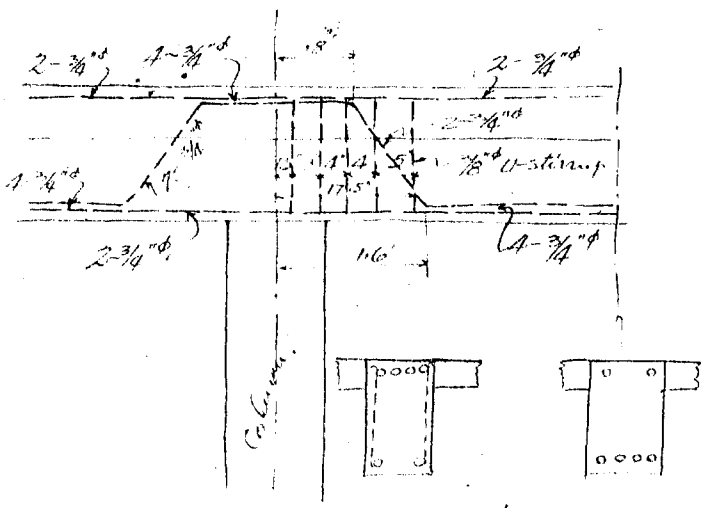
Negative moment at support. Say same as for positive moment $= -23,900'$#
Double reinforced rectangular beam



$A_s = 4 - 3/4"\phi = 1.765\,□"$ $p = 0.0118$
$A_s' = 2 - 3/4"\phi = 0.883\,□"$ $p' = 0.0059$ $p' = 0.5p$.
From prepared diagram,
$K = 0.408$ .. $j = 0.862$
$f_s = \frac{M}{A_s j d} = \frac{23900\times12}{1.165\times0.862\times15} = 12,560$ #/□" OK.
$f_c = f_s \frac{K}{n(1-k)} = 12560\times\frac{0.408}{15(1-0.408)} = 578$ #/□" OK.
$f_s' = 578\times15 = 8680$ #/□" Compression OK.

Web Reinforcements.

Unit shear at support $= 121$ #/□"
max. end shear $= 15,860$#
diam. of stirrup $i = (2.4\frac{u}{f_s})d = 2.4\times\frac{8.5}{12,000}\times15 = 0.255"$
use 3/8"ϕ plain bar - U-stirrup.
Distance from support, between which stirrups are necessary
$x = \frac{l}{2}(1-\frac{v_1}{v}) = \frac{5.5}{2}(1-\frac{57}{121}) = 1.46' = 17.5"$



Section at end. Section at centre.

Max. spacing of U-stirrups (at support)
$S = \frac{3}{2}\cdot\frac{A_s f_s j d}{V} = \frac{3}{2}\cdot\frac{2\times0.1105\times12000\times7/8\times15}{15860} = 3.3"$

Make spacing of stirrups as follows.
$S_1 = 1.5"$, $S_2 = 3"$, $S_3 = 4"$, $S_4 = 4"$, $S_5 = 5"$
total 17.5', 5- stirrups.

Bent up 2 of the main reinforcements at 1.6' from the support.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Mutsu-Ohashi over Mutsugawa, Hyogoken.

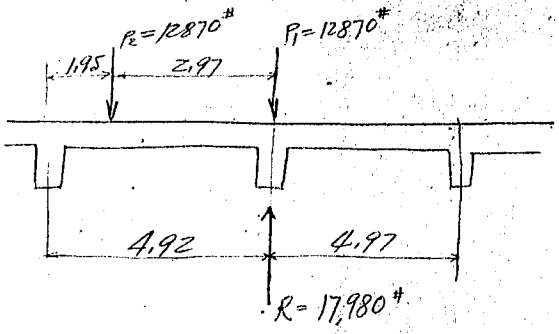
Design of Beams under Traffic way.

Beam B. Continuous beam of 5.5' span.

Dead Load slab 105×4.91 = 526
stem of beam .83×.92×150 = 115
641 #/ft run.

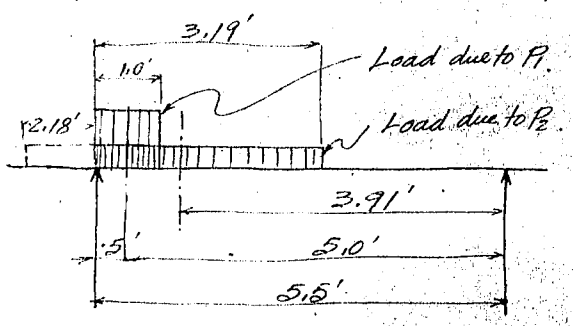
Dead Load moment = $\frac{641\times5.5^2}{10}$ = 1,940 '#
Dead Load shear = 641×2.75 = 1760 #

Live Load Rear wheel concentration of motor truck 12,870 # including impact.
Concentration on girder $12870\times\frac{1.95}{4.92}$ = 5,110 #
12,870
17,980 #



Live Load moment = $\frac{17980\times5.5}{4}$ = 24700 '# as a simple beam.
24700 × 0.8 = 19,750 '# as a continuous beam.

max. end shear $12870\times\frac{5.0}{5.5}$ = 11,700 #
$5110\times\frac{3.19}{5.37}\times\frac{3.91}{5.5}$ = 2160 #
Total = 13860 #



Summary of moments and shears.

Summary	moment	Shear.
Dead Load	1,940 '#	1,760 #
Live Load	19,750 "	13860
Total	21,690 '#	15,620 #

The moment and shear are smaller than, but very near to those for Beams under R_2 (Beam F)
All dimensions and details shall be the same as for Beam F.
Beam B, C & D shall be the same dimensions and details also.

Fascia Girder.

Dead Load.

Slab, handrail etc. (Cantilever) 396 + (minimum span) 130 = 526 #/ft run.
stem of girder .83×1.3×150 = 162
688 #/ft run

Dead Load M = $\frac{688\times5.5^2}{10}$ = 2,080 '#
Dead Load V = 688×2.75 = 1,890 #

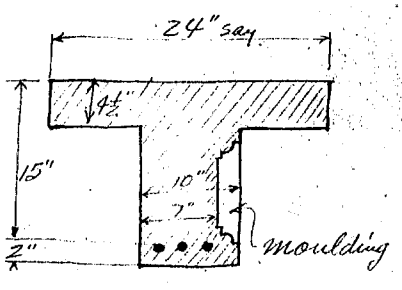
Live Load.

Uniform load of 100 #/ft² 100×3.46 = 346 #/ft run

Live Load moment = $\frac{346\times5.5^2}{10}$ = 1,050 '#
" " end shear = 346×2.75 = 950 #

Summary for moment and shear.

Summary	moment	shear.
Dead Load	2,080 '#	1890 #
Live Load	1,050 '#	950
Total	3,130 '#	2,840 #



t/d = 4.5/15 = 0.30
Assume 3 - ½"φ = 0.59 0" p = $\frac{0.59}{24\times15}$ = 0.0016
neutral axis in flange.
steel required = $\frac{3130\times12}{7/8\times15\times17000}$ = 0.1685 0"
use ½"φ 3 bars. = 0.59 0"
unit shear v = $\frac{2840}{7\times7/8\times15}$ = 31 #/0" OK.
unit bond u = $\frac{2840}{3\times1.571\times7/8\times15}$ = 46 #/0" OK.

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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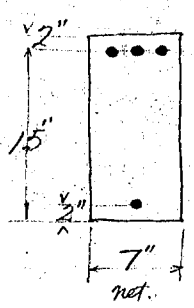
CALCULATIONS FOR

Mukō-Ohashi over Mukōgawa, Ibyogoken.
Fascia Girder Continued.

Neg. moment at support, say same as positive moment = −3,150'#
double reinforced rectangular beam.



$A_s = 3-\frac{1}{2}"\phi = 0.39\square"$ $A_s' = 1-\frac{1}{2}"\phi = 0.196\square"$ $A_s' = 0.33A_s$.

$p = 0.0056$; $p' = 0.0019$, $p' = 0.33p$.

$K = 0.31$.

$f_s = \frac{M}{A_s jd} = \frac{3,150\times12}{0.89\times7/8\times15} = 4,850$ #/□" ok.

$f_c = f_s \frac{K}{n(1-K)} = 4850\times\frac{0.31}{15(1-0.31)} = 145$ #/□" ok.

Beam A.

Dead Load.

Motor truck ways, Slab etc	= 150×2.46 =	369
sidewalk slab	= 65×2 =	130
Curb stone .5×.5 @ 160	= .25×160 =	40
Stem of beam .83×1.2@150	=	150
Chamfer etc		21
		710 #/ft run.

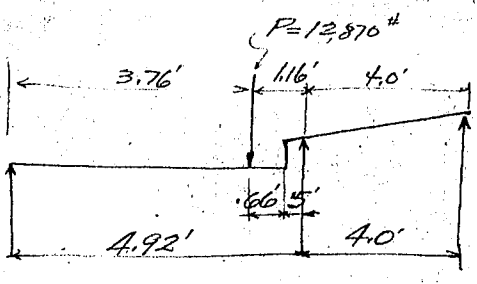
Dead load moment = $\frac{710\times5.5^2}{10}$ = 2,150'#
Dead load end shear = 710×2.75 = 1,950#

Live Loads.

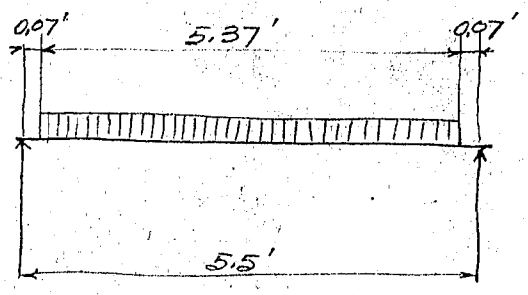
Live load on sidewalk 100×2 = 200 #/ft run

Unif. Live Load moment = $\frac{200\times5.5^2}{10}$ = 610'#
Max. End shear = 200×2 = 400#

Motor truck rear wheel concentration = 12,870# including impact.



Concentration on beam $12870\times\frac{3.76}{4.92}$ = 9,850#
Live Load moment, say full loaded.
$M = \frac{1840\times5.5^2}{10}$ = 5,560'#
end shear $V = 1840\times2.75$ = 5,060#



Summary for moment and shear.

Summary	moment	shear.
Dead Load	2,150	1,950
Live unif load	610	400
Live concent. load	5560	5060
	8320	7,410

T-Beam Neutral axis in the flange. d = 12", b = 24"
Effective depth req'd. for + moment, $d = \sqrt{\frac{8320\times12}{24\times102}}$ = 6.4"
" " " − moment, $d = \sqrt{\frac{8320\times12}{10\times102}}$ = 10.0"
Make 14" deep over all with 2" insulation.

steel required = $\frac{8320\times12}{7/8\times12"\times17,000}$ = 0.56□" use 3−¾"φ = 1.33□"

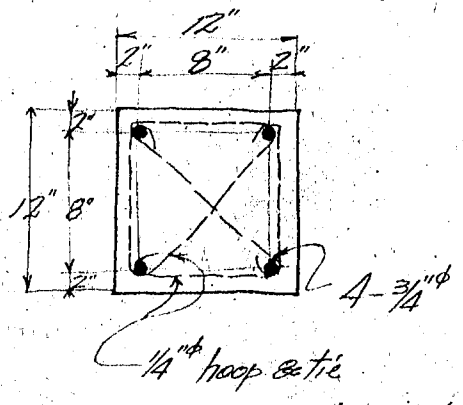
unit shear = $\frac{7,410}{10\times7/8\times14}$ = 60 #/□" ok
for safty use some stirrups as additional.

bond = $\frac{7,410}{3\times2.35\times7/8\times14}$ = 85 #/□" ok.

Design of Columns.

max. length of col. 6.3' both end fixed assumed. Bending moment neglected.

Live load concentration on column = 18,740#
Dead Load on column 3000×2 = 6,000#
Column's own weight = 950#
Total 25,690#



Transformed sectional area of col. = 12×12 + 17.67×14 = 168.7□"
Average compression on concrete = $\frac{25,690}{168.7}$ = 153 #/□" ok
" " " steel = 153×15 = 2,300 " ok.
¼"φ looping and ties shall be arranged 6" spacing.

6" spacing at top and bottom
12" " " intermediate portion.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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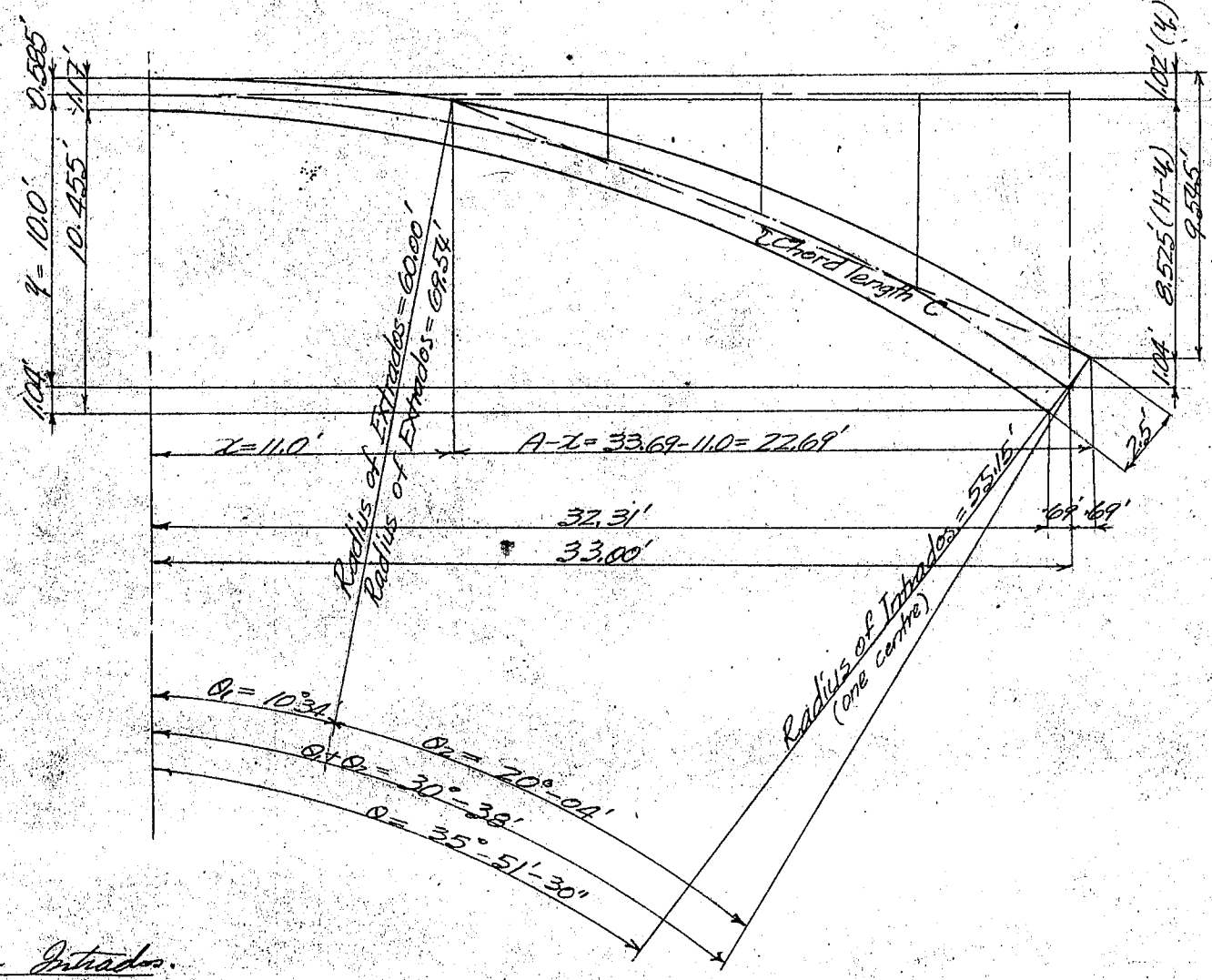
CALCULATIONS FOR

Mutsu-Akashi over Mutsugawa, Aizu-Wakamatsu.
Curves for Arch Ring.



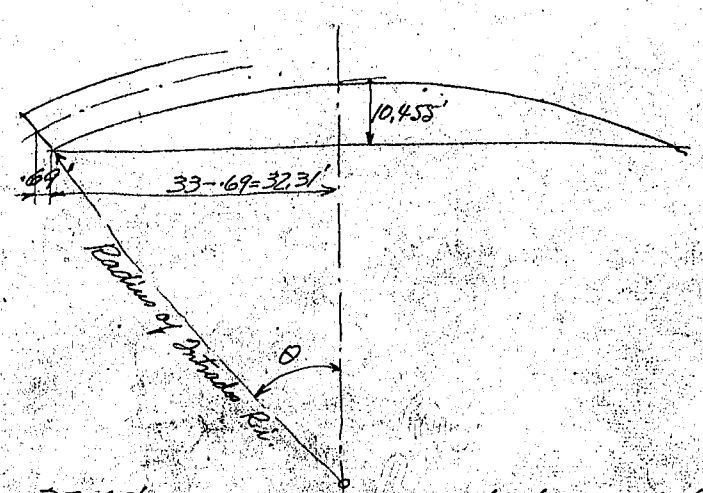
Curves for Intrados.

$l = 64.62'$, $f = 10.455'$

$$Ri = \frac{l^2}{8f} + \frac{f}{2} = \frac{64.62^2}{8\times 10.455} + \frac{10.455}{2}$$

$$= 55.1527 \text{ ft.} \quad \text{say } \underline{55.15'}$$

$$\theta = \sin^{-1} \frac{32.31}{55.1527} = 35°-51'-30"$$



Curves for Extrados.

Let A = Half span length
H = Rise
θ_1 = Angle subtended by half arc at centre.
θ_2 = Angle subtended by each end arc.
C = Length of chord for θ_2

$$y = R_1 - \sqrt{R_1^2 - x^2}$$

$$\theta_1 = \sin^{-1} \frac{x}{R_1}, \quad \frac{\theta_2}{2} = \phi - \theta_1$$

$$\phi = \tan^{-1} \frac{H-y}{A-x}, \quad C = \frac{H-y}{\sin\phi}$$

$$R_2 = \frac{C/2}{\sin \theta_2/2}$$

$A = 33.69'$, $H = 9.595'$, $R = \underline{60'}$, $x = 11.0'$, $y = 1.02'$

$\theta_1 = 10° 34'$

$\tan\phi = \frac{8.525}{22.69} = 0.37572$, $\phi = 20°-36'$

$\theta_2/2 = \phi - \theta_1 = 10°-02'$

$\theta_2 = 20°-04'$

$C = \frac{8.525}{0.35183} = 24.230'$, $\frac{C}{2} = 12.115'$

$\sin \theta_2/2 = 0.17422$

$R_2 = \frac{12.115}{0.17422} = \underline{69.54'}$

Neutral axis runs between Extrados and Intrados curves bisecting the distance between the 2 curves at any point and has no equation for it, but by a graphic method, its curve is found to be very near to a circular curve whose radius is 59.51 ft. about.

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Mutsu-Ohashi Over Mutsugawa Kyotsugoken

Arch Ring Section. The arch ring will consists of 2- 21.68' each for carriageway and sidewalks and 1- 19.35' for Electric Railway.

Reinforcements for crown, 1"ϕ bars 12" centres top and bottom = 2@.785=1.57°"/ft.

do. for springing, 1"ϕ bars 6" ctrs top and bottom = 4@.785=3.14°"/ft.

Moment of inertia for 1 ft. strip.

Moment of inertia of concrete $= \frac{1}{12}d^3$

Crown to R.11, Moment of inertia of steel $= \frac{15.7\times15}{144}(\frac{d}{2}-0.12)^2 = 0.1635(\frac{d}{2}-0.12)^2$

at Springing, moment of inertia of steel $= \frac{3.14\times15}{144}(\frac{d}{2}-0.12)^2 = 0.327(\frac{d}{2}-0.13)^2$

Division	Depth	d^3	$\frac{1}{12}d^3$	$(\frac{d}{2}-0.12)^2$	$0.1635(\frac{d}{2}-0.12)^2=I_s$	I_c+I_s	Steel Area equivalents	Concrete area.
0 Crown	1.170	1.602	0.134	0.2162	0.0354	0.1694	0.1635	1.170
1	1.190	1.643	0.137	0.2209	0.0361	0.1731	"	1.180
2	1.195	1.706	0.142	0.2280	0.0373	0.1793	"	1.195
3	1.220	1.816	0.151	0.2401	0.0393	0.1903	"	1.220
4	1.250	1.953	0.163	0.2550	0.0417	0.2047	"	1.250
5	1.295	2.172	0.181	0.2783	0.0455	0.2265	"	1.295
6	1.370	2.571	0.214	0.3192	0.0522	0.2662	"	1.370
7	1.480	3.242	0.270	0.3844	0.0628	0.3328	"	1.480
8	1.620	4.252	0.354	0.4761	0.0778	0.4318	"	1.620
9	1.775	5.592	0.466	0.5852	0.0957	0.5617	"	1.775
10	1.970	7.645	0.637	0.7482	0.1223	0.7593	"	1.970
11	2.200	10.648	0.887	0.9604	0.1570	1.0440	"	2.200
12 Springing	2.500	15.625	1.302	1.2769	0.4175	1.7195	0.3270	2.500

Division	x	x^2	y	y^2	ds	I	$\frac{ds}{I}$	$\frac{x\,ds}{I}$	$\frac{x^2ds}{I}$	$\frac{y\,ds}{I}$	$\frac{y^2ds}{I}$
0 Crown	0	0	0	0	1.375	0.1694	8.122	0	0	0	0
1	2.75	7.6	0.070	0.005	2.755	0.1731	15.916	43.77	121.37	1.11	0.08
2	5.50	30.3	0.260	0.068	2.765	0.1793	15.421	84.82	466.51	4.01	1.04
3	8.25	68.1	0.575	0.331	2.780	0.1903	14.609	120.52	994.29	8.40	4.83
4	11.00	121.0	1.050	1.103	2.810	0.2047	13.727	151.00	1661.00	14.42	15.14
5	13.75	189.1	1.650	2.723	2.830	0.2265	12.494	171.79	2362.11	20.60	34.00
6	16.50	272.3	2.380	5.664	2.865	0.2662	10.763	177.59	2930.24	25.60	60.92
7	19.25	370.6	3.250	10.563	2.905	0.3328	8.729	168.03	3,234.58	28.36	92.22
8	22.00	484.0	4.260	18.148	2.965	0.4318	6.867	151.07	3,323.54	29.28	124.80
9	24.75	612.6	5.440	29.594	3.030	0.5617	5.394	133.50	3304.13	29.35	159.50
10	27.50	756.3	6.780	45.968	3.095	0.7593	4.076	112.09	3,082.48	27.64	187.30
11	30.25	915.1	8.300	68.890	3.185	1.0440	3.051	92.29	2,771.77	25.33	210.20
12 Springing	33.00	1089.0	10.000	100.000	1.630	1.7195	0.947	31.09	1,025.97	9.42	94.20
					34.980		120.101	1,437.56	25,277.99	223.52	984.23

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Mukō-Ohashi over Mukōgawa, Nyogotsu.
Computations of Ho, Mo and Vo at Crown.

	Point #0					Point # 1				Point # 2			
Point	x	m	m·ds/I	m·x·ds/I	m·y·ds/I	m	m·ds/I	m·x·ds/I	m·y·ds/I	m	m·ds/I	m·x·ds/I	m·y·ds/I
0	0	0	0	0	0								
1	2.75	2.75	43.77	121.37	3.05	0							
2	5.50	5.50	84.82	466.51	22.07	2.75	42.42	233.40	11.03	0			
3	8.25	8.25	120.52	994.29	69.30	5.50	80.35	662.80	46.40	2.75	40.20	331.70	23.11
4	11.00	11.00	151.00	1,661.00	158.60	8.25	113.25	1,246.00	119.00	5.50	75.50	830.50	79.28
5	13.75	13.25	171.79	2,362.11	283.20	11.00	137.45	1,890.00	226.90	8.25	102.40	1408.00	169.00
6	16.50	16.50	177.59	2930.24	422.30	13.75	148.00	2,443.00	352.40	11.00	118.40	1,954.00	282.00
7	19.25	19.25	168.03	3234.38	546.00	16.50	144.00	2,772.00	468.00	13.75	120.00	2,310.00	390.00
8	22.00	22.00	151.07	3323.54	644.00	19.25	132.10	2,906.00	563.00	16.50	113.30	2,494.00	483.00
9	24.75	24.75	133.50	3304.13	726.20	22.00	118.60	2,935.00	645.50	19.25	103.85	2,570.00	565.00
10	27.50	27.50	112.09	3,082.48	760.00	24.75	100.80	2,774.00	683.90	22.00	89.70	2,467.00	608.00
11	30.25	30.25	92.29	2,771.77	766.00	27.50	83.90	2,538.00	696.50	24.75	75.50	2,283.00	626.60
12	33.00	33.00	31.09	1025.97	310.90	30.25	28.50	940.60	285.00	27.50	25.91	855.50	259.10
			1437.56	25277.99	4711.62		1129.37	21,340.80	4097.63		864.76	17,503.70	3485.09

	Point # 3					Point # 4				Point # 5			
Point	x	m	m·ds/I	m·x·ds/I	m·y·ds/I	m	m·ds/I	m·x·ds/I	m·y·ds/I	m	m·ds/I	m·x·ds/I	m·y·ds/I
3	8.25	0											
4	11.00	2.75	37.76	415.00	39.64	0	∫34.36						
5	13.75	5.50	68.72	944.50	113.40	2.75	∫58.70	472.20	56.66	0			
6	16.50	8.25	88.81	1465.50	211.50	5.50	∫72.00	968.00	139.75	2.75	29.60	488.30	70.42
7	19.25	11.00	96.04	1848.50	312.10	8.25	∫75.55	1,386.00	238.00	5.50	48.00	924.00	156.00
8	22.00	13.75	94.40	2077.00	402.20	11.00	∫74.12	1,661.00	323.00	8.25	56.62	1,246.00	241.40
9	24.75	16.50	89.00	2,203.00	484.20	13.75	∫67.22	1,835.00	403.40	11.00	59.34	1,468.00	325.90
10	27.50	19.25	78.40	2156.00	531.50	16.50	∫58.74	1,849.00	455.70	13.75	56.05	1,542.20	380.10
11	30.25	22.00	67.12	2,030.00	536.80	19.25	∫20.72	1,777.00	487.50	16.50	50.33	1,522.50	417.70
12	33.00	24.75	23.30	769.00	233.00	22.00		683.90	207.20	19.25	18.13	598.15	181.30
			643.55	13988.50	2,884.34		461.41	10632.10	2311.21		318.07	7,789.18	1772.82

	Point # 6					Point # 7				Point # 8			
Point	x	m	m·ds/I	m·x·ds/I	m·y·ds/I	m	m·ds/I	m·x·ds/I	m·y·ds/I	m	m·ds/I	m·x·ds/I	m·y·ds/I
6	16.50	0											
7	19.25	2.75	23.79	457.70	77.30	0	18.90						
8	22.00	5.50	37.77	831.20	160.90	2.75	29.38	415.70	80.57	0			
9	24.75	8.25	44.50	1,101.50	242.10	5.50	33.62	734.10	161.50	2.75	14.83	367.10	80.70
10	27.50	11.00	44.83	1233.00	304.00	8.25	33.57	925.00	228.20	5.50	22.43	608.45	152.15
11	30.25	13.75	41.95	1269.50	348.00	11.00		1,015.50	278.70	8.25	25.18	761.80	209.00
12	33.00	16.50	15.55	512.80	155.50	13.75	12.95	427.50	129.50	11.00	10.37	342.00	103.70
			208.39	5,405.70	1287.80		128.39	3517.80	878.41		72.81	2079.35	545.55

	Point # 9					Point # 10				Point # 11			
Point	x	m	m·ds/I	m·x·ds/I	m·y·ds/I	m	m·ds/I	m·x·ds/I	m·y·ds/I	m	m·ds/I	m·x·ds/I	m·y·ds/I
9	24.75	0											
10	27.50	2.75	11.21	308.40	76.02	0							
11	30.25	5.50	16.79	507.90	139.35	2.75	8.39	253.80	69.62	0			
12	33.00	8.25	7.77	256.50	77.70	5.50	5.09	171.10	50.90	2.75	2.59	85.48	25.90
			35.77	1072.80	293.07		13.48	424.90	120.52		2.59	85.48	25.90

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CALCULATIONS FOR

Mutsu-Ohashi over Mutsugawa, Hyogoken.
Computation of H_o, M_o and V_o at Crown continued.

$$H_o = \frac{\int\frac{ds}{I}\int\frac{m.y\,ds}{I} - \int\frac{m\,ds}{I}\int\frac{y\,ds}{I}}{2\left[\int\frac{ds}{I}\int\frac{y^2 ds}{I} - \left(\int\frac{y\,ds}{I}\right)^2\right]} = \frac{A}{B}, \quad M_o = \frac{-H_o\,2\int\frac{y\,ds}{I} + \int\frac{m\,ds}{I}}{2\int\frac{ds}{I}} = \frac{C}{D}, \quad V_o = \frac{\int\frac{m.x\,ds}{I}}{2\int\frac{x^2 ds}{I}} = \frac{E}{F}$$

$$B = 2(120.101 \times 984.23 - 223.52^2) = 136,492.$$

Point Loaded		A		B		H_o
crown	0	244,546.9	÷	136,492.	=	1.79166
	1	239,692.7		"	=	1.75610
	2	225,271.6		"	=	1.65044
	3	202,565.8		"	=	1.48408
	4	174,444.2		"	=	1.27833
	5	141,822.5		"	=	1.03906
	6	108,086.8		"	=	0.791891
	7	76,800.2		"	=	0.562672
	8	49,246.6		"	=	0.360802
	9	27,202.7		"	=	0.199290
	10	11,461.6		"	=	0.083973
	11	2,531.7		"	=	0.0185483
Springing	12					

Point Loaded		$-2H_o\int\frac{y\,ds}{I}$	+	$\int\frac{m\,ds}{I}$		C	*	$2\int\frac{ds}{I}$	=	D		M_o
crown	0	− 800.943	+	1437.56	=	+636.617	÷	240.202	=	+		2.6503
	1	− 785.044	+	1,129.37	=	+344.326		"	=	+		1.4335
	2	− 737.822	+	864.76	=	+126.938		"	=	+		0.5287
	3	− 663.446	+	643.55	=	− 19.896		"	=	−		0.0828
	4	− 571.466	+	461.41	=	− 110.056		"	=	−		0.4582
	5	− 464.499	+	318.07	=	− 146.429		"	=	−		0.6096
	6	− 354.010	+	208.39	=	− 145.620		"	=	−		0.6062
	7	− 251.537	+	128.39	=	− 123.147		"	=	−		0.5127
	8	− 161.293	+	72.81	=	− 88.483		"	=	−		0.3684
	9	− 89.095	+	35.77	=	− 53.325		"	=	−		0.2220
	10	− 37.539	+	13.48	=	− 24.059		"	=	−		0.1002
	11	− 8.292	+	2.59	=	− 5.702		"	=	−		0.0237
Springing	12	0		0		0						

Point Loaded		E		F		V_o
crown	0	25,277.99	÷	50,555.98	=	0.5000
	1	21,340.80		"	=	0.4221
	2	17,503.70		"	=	0.3462
	3	13,908.50		"	=	0.2751
	4	10,632.10		"	=	0.2103
	5	7,789.15		"	=	0.1541
	6	5,405.70		"	=	0.1070
	7	3,577.80		"	=	0.0696
	8	2,079.35		"	=	0.0411
	9	1,072.80		"	=	0.0212
	10	424.90		"	=	0.0084
	11	85.48		"	=	0.0017
Springing	12	0		"	=	0

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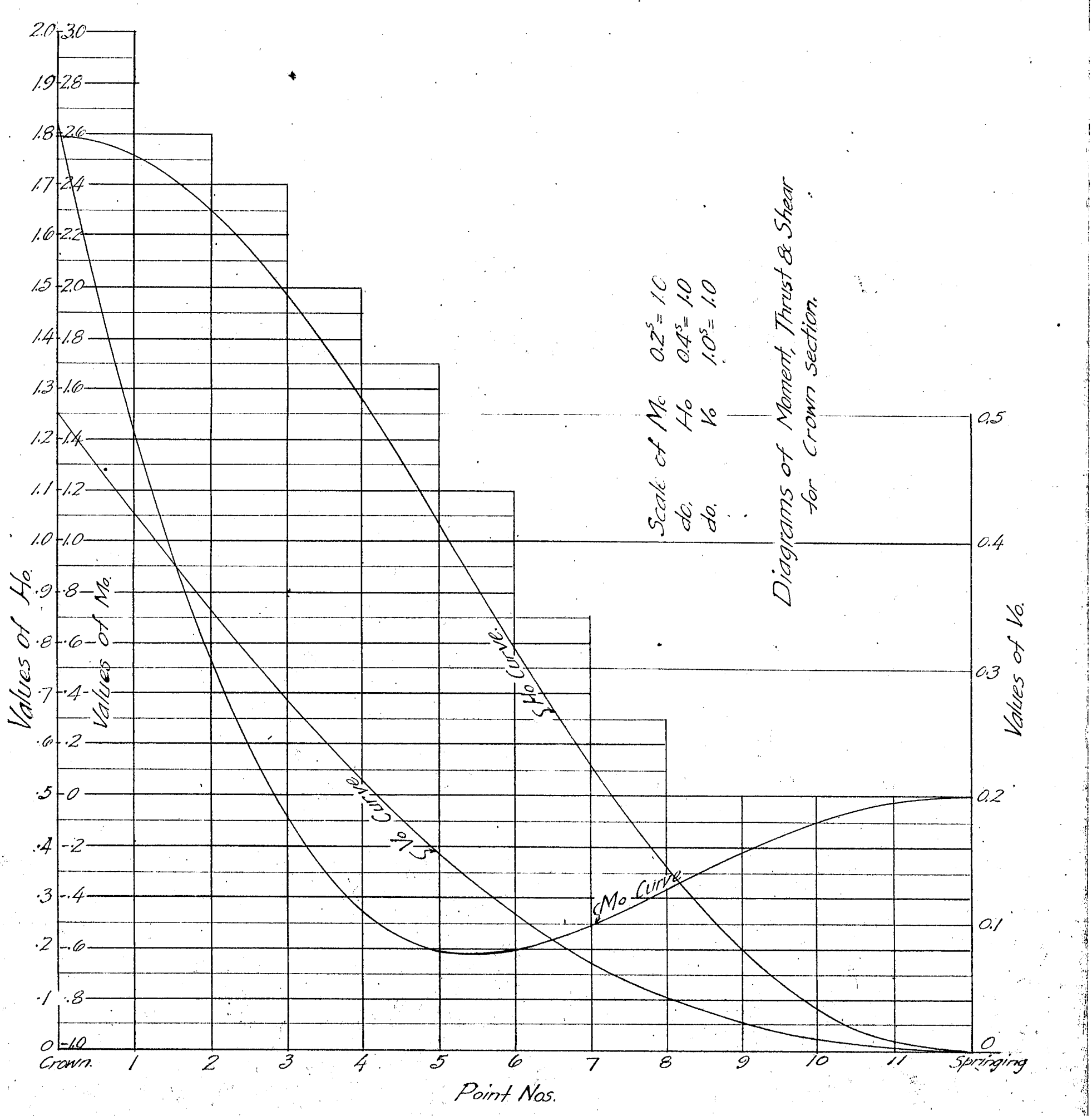
CALCULATIONS FOR



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MADE BY K. Iraba DATE 13-7-5 / 13-9-6 FILE NO

CHECKED BY DATE PAGE NO 16

CALCULATIONS FOR

Maiso-Ohashi over Mukogawa, Hyogoken.

Moments at various points for Unit Load.

Let x and y be co-ordinates of centre of sections, origin at crown.
and d' be lever arm of unit load about centre of sections,

For left hand sections, $M_L = M_0 + H_0 y + V_0 x - d'$

" right " " . $M_R = M_0 + H_0 y - V_0 x$.

Moments at various sections.

Points	Springing x=33, y=10						Point #11 x=30.25 y=8.30					
	M_0	$H_0 y$	$V_0 x$	d'	M_L	M_R	M_0	$H_0 y$	$V_0 x$	d'	M_L	M_R
0	+2.650	17.917	16.500	33.00	+4.067	+4.067	+2.650	14.871	15.125	.3025	123916	+23396
1	+1.434	17.561	13.929	30.25	+2.674	+5.066	+1.434	14.576	12.769	27.50	+1.275	+3.241
2	+0.529	16.504	11.425	27.50	+0.758	+5.608	+0.529	13.698	10.473	24.75	-0.050	+3.754
3	-0.083	14.841	9.078	24.75	-0.914	+5.680	-0.083	12.318	8.322	22.00	-1.443	+3.913
4	-0.458	12.783	6.940	22.00	-2.735	+5.385	-0.458	10.610	6.362	19.25	-2.736	+3.790
5	-0.610	10.391	5.085	19.25	-4.384	+4.696	-0.610	8.625	4.662	16.50	-3.823	+3.353
6	-0.606	7.919	3.531	16.50	-5.656	+3.782	-0.606	6.573	3.237	13.75	-4.546	+2.730
7	-0.513	5.627	2.297	13.75	-6.339	+2.817	-0.513	4.670	2.105	11.00	-4.738	+2.052
8	-0.368	3.608	1.356	11.00	-6.404	+1.884	-0.368	2.995	1.243	8.25	-4.380	+1.384
9	-0.222	1.993	0.700	8.25	-5.779	+1.071	-0.222	1.654	0.641	5.50	-3.427	+0.191
10	-0.100	0.840	0.277	5.50	-4.483	+0.463	-0.100	0.697	0.254	2.75	-1.899	+0.343
11	-0.024	0.185	0.056	2.75	-2.533	+0.105	-0.024	0.154	0.051	0	+0.181	+0.079
12	0			0			0				0	

Points	Point #10, x=27.5, y=6.78						Point #9 x=24.75 y=5.44					
	M_0	$H_0 y$	$V_0 x$	d'	M_L	M_R	M_0	$H_0 y$	$V_0 x$	d'	M_L	M_R
0	+2.650	12.148	13.750	27.50	+1.048	+1.048	+2.650	9.747	12.375	24.75	+0.022	+0.022
1	+1.434	11.906	11.684	24.75	+0.198	+1.732	+1.434	9.553	10.447	22.00	-0.566	+0.540
2	+0.529	11.190	9.521	22.00	-0.760	+2.198	+0.529	8.978	8.568	19.25	-1.175	+0.939
3	-0.083	10.062	7.565	19.25	-1.706	+2.414	-0.083	8.074	6.809	16.50	-1.700	+1.182
4	-0.458	8.667	5.783	16.50	-2.508	+2.436	-0.458	6.954	5.206	13.75	-2.048	+1.291
5	-0.610	7.045	4.238	13.75	-3.077	+2.197	-0.610	5.653	3.814	11.00	-2.143	+1.229
6	-0.606	5.369	2.943	11.00	-3.294	+1.820	-0.606	4.308	2.648	8.25	-1.900	+1.054
7	-0.513	3.815	1.914	8.25	-3.034	+1.388	-0.513	3.061	1.723	5.50	-1.229	+0.825
8	-0.368	2.446	1.130	5.50	-2.292	+0.948	-0.368	1.963	1.017	2.75	-0.138	+0.578
9	-0.222	1.351	0.583	2.75	-1.038	+0.546	-0.222	1.084	0.525	0	+1.387	+0.337
10	-0.100	0.570	0.231	0	+0.701	+0.239	-0.100	0.457	0.208		+0.565	+0.149
11	-0.024	0.125	0.047		-0.148	+0.054	-0.024	0.101	0.042		+0.119	+0.035
12	0				0			0			0	

Point	Point #8 x=22.0, y=4.26						Point #7 x=19.25, y=3.25					
	M_0	$H_0 y$	$V_0 x$	d'	M_L	M_R	M_0	$H_0 y$	$V_0 x$	d'	M_L	M_R
0	+2.650	7.633	11.000	22.00	-0.717	-0.717	+2.650	5.823	9.625	19.25	-1.152	-1.152
1	+1.434	7.481	9.286	19.25	-1.049	-0.371	+1.434	5.707	8.125	16.50	-1.234	-0.984
2	+0.529	7.031	7.616	16.50	-1.324	-0.056	+0.529	5.364	6.664	13.75	-1.193	-0.771
3	-0.083	6.322	6.052	13.75	-1.459	+0.208	-0.083	4.823	5.296	11.00	-0.964	-0.556
4	-0.458	5.446	4.627	11.00	-1.385	+0.361	-0.458	4.154	4.049	8.25	-0.505	-0.353
5	-0.610	4.427	3.390	8.25	-1.043	+0.427	-0.610	3.377	2.966	5.50	+0.233	-0.199
6	-0.608	3.373	2.354	5.50	-0.379	+0.413	-0.606	2.574	2.060	2.75	+1.278	-0.092
7	-0.513	2.397	1.531	2.75	+0.665	+0.354	-0.513	1.829	1.340	0	+2.656	-0.024
8	0.368	1.537	0.904	0	+2.073	+0.265	-0.368	1.173	0.791		+1.596	-0.014
9	-0.222	0.849	0.466		+1.093	+0.161	-0.222	0.648	0.408		+0.834	+0.018
10	-0.100	0.358	0.185		+0.443	+0.073	-0.100	0.273	0.162		+0.335	+0.011
11	-0.024	0.079	0.037		+0.092	+0.018	-0.024	0.060	0.033		+0.069	+0.003
12	0	0					0	0			0	

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CALCULATIONS FOR

Moments at Various Sections Continued.

	Point #6 x=16.5, y=2.38						Point #5, x=13.75, y=1.65					
Point	Mo	Hoy	Vox	d'	ML	MR	Mo	Hoy	Vox	d'	ML	MR
0	+2.650	4.264	8.250	16.50	-1.336	-1.336	+2.650	2.956	6.875	13.75	-1.269	-1.269
1	+1.434	4.180	6.965	13.75	-1.171	-1.351	+1.434	2.898	5.804	11.00	-0.864	-1.472
2	+0.529	3.928	5.712	11.00	-0.831	-1.255	+0.529	2.723	4.760	8.25	-0.238	-1.508
3	-0.083	3.532	4.539	8.25	-0.262	-1.090	-0.083	2.449	3.783	5.50	+0.649	-1.417
4	-0.458	3.042	3.470	5.50	+0.554	-0.886	-0.458	2.109	2.891	2.75	+1.792	-1.240
5	-0.610	2.473	2.543	2.75	+1.656	-0.680	-0.610	1.715	2.119	0	+3.224	-1.014
6	-0.606	1.885	1.766	0	+3.045	-0.487	-0.606	1.307	1.471		+2.172	-0.770
7	-0.513	1.334	1.148		+1.974	-0.322	-0.513	0.928	0.957		+1.372	-0.542
8	-0.368	0.859	0.678		+1.169	-0.187	-0.368	0.595	0.565		+0.792	-0.338
9	-0.222	0.474	0.350		+0.602	-0.098	-0.222	0.329	0.292		+0.399	-0.185
10	-0.100	0.199	0.144		+0.248	-0.050	-0.100	0.139	0.116		+0.155	-0.077
11	-0.024	0.044	0.028		+0.048	-0.008	-0.024	0.031	0.023		+0.030	-0.030
12												

	Point #4 x=11.00, y=1.05						Point #3 x=8.25 y=0.575					
Point	Mo	Hoy	Vox	d'	ML	MR	Mo	Hoy	Vox	d'	ML	MR
0	+2.650	1.881	5.500	11.00	-0.969	-0.969	+2.650	1.030	4.125	8.25	-0.445	-0.445
1	+1.434	1.844	4.643	8.25	-0.329	-1.365	+1.434	1.010	3.482	5.50	+0.426	-1.038
2	+0.529	1.733	3.808	5.50	-0.570	-1.546	+0.529	0.944	2.856	2.75	+1.584	-1.378
3	-0.083	1.538	3.026	2.75	+1.751	-1.551	-0.083	0.853	2.270	0	+3.040	-1.500
4	-0.458	1.342	2.313	0	+3.197	-1.429	-0.458	0.735	1.735		+2.012	-1.459
5	-0.610	1.091	1.695		+2.176	-1.214	-0.610	0.597	1.271		+1.258	-1.284
6	-0.606	0.831	1.177		+1.402	-0.952	-0.606	0.455	0.883		+0.732	-1.034
7	-0.513	0.591	0.766		+0.844	-0.688	-0.513	0.324	0.574		+0.385	-0.783
8	-0.368	0.379	0.452		+0.463	-0.441	-0.368	0.207	0.339		+0.178	-0.500
9	-0.222	0.209	0.233		+0.220	-0.246	-0.222	0.115	0.175		+0.068	-0.282
10	-0.100	0.088	0.092		+0.080	-0.104	-0.100	0.048	0.069		+0.017	-0.121
11	-0.024	0.019	0.019		+0.014	-0.024	-0.024	0.011	0.014		+0.001	-0.027
12	0	0					0					

	Point #2 x=5.50 y=026						Point #1, x=2.75' y=0.07'					
Point	Mo	Hoy	Vox	d'	ML	MR	Mo	Hoy	Vox	d'	ML	MR
0	+2.650	0.466	2.750	5.50	+0.366	+0.366	+2.650	0.125	1.375	2.75	+1.400	+1.400
1	+1.434	0.457	2.322	2.75	+1.463	-0.431	+1.434	0.123	1.161	0	+2.718	+0.396
2	+0.529	0.429	1.904	0	+2.862	-0.946	+0.529	0.115	0.952		+1.596	-0.308
3	-0.083	0.386	1.513		+1.816	-1.210	-0.083	0.104	0.757		+0.778	-0.736
4	-0.458	0.332	1.157		+1.031	-1.283	-0.458	0.089	0.578		+0.209	-0.947
5	-0.610	0.270	0.848		+0.508	-1.188	-0.610	0.073	0.424		-0.113	-0.961
6	-0.606	0.206	0.589		+0.189	-0.989	-0.606	0.055	0.294		-0.357	-0.845
7	-0.513	0.146	0.383		+0.016	-0.750	-0.513	0.039	0.191		-0.283	-0.665
8	-0.368	0.094	0.226		-0.048	-0.500	-0.368	0.025	0.113		-0.230	-0.456
9	-0.222	0.052	0.117		-0.053	-0.287	-0.222	0.014	0.058		-0.150	-0.266
10	-0.100	0.022	0.046		-0.032	-0.124	-0.100	0.006	0.023		-0.071	-0.117
11	-0.024	0.005	0.009		-0.010	-0.028	-0.024	0.001	0.005		-0.018	-0.028
12	0						0	0				

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CALCULATIONS FOR

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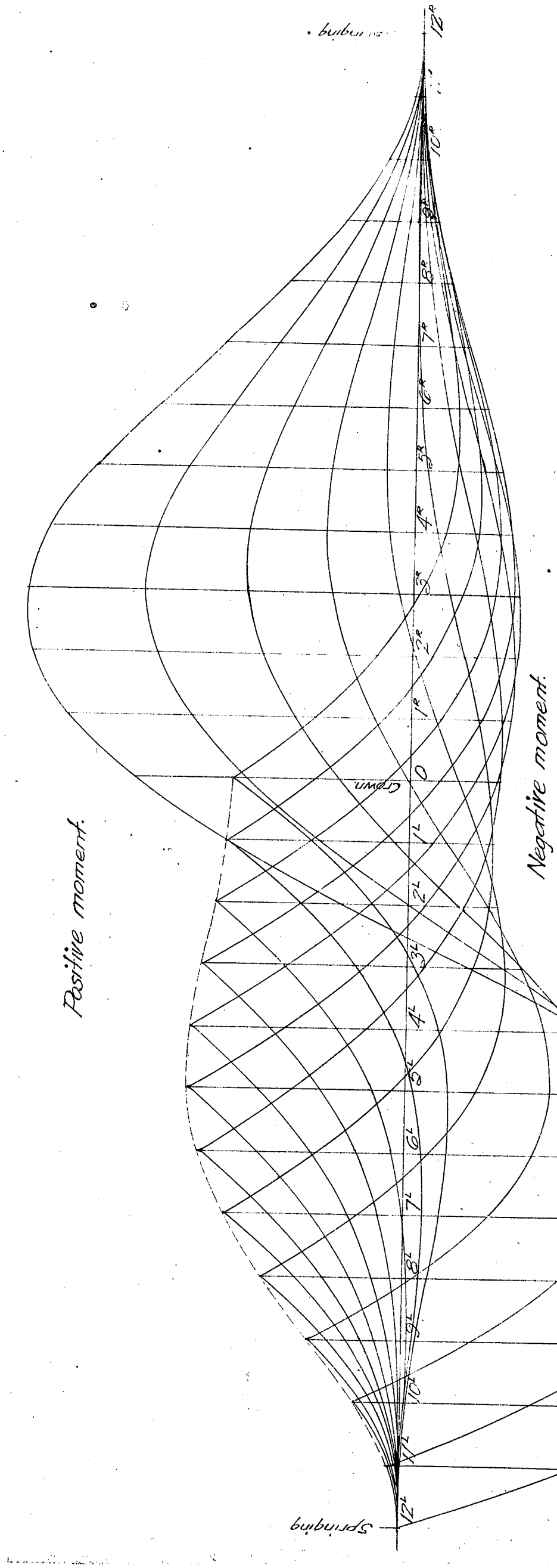

Scale for moment. $0.1^S = 2.0$
Scale for space. $0.1^S = 60^S$

INFLUENCE DIAGRAM OF MOMENT AT VARIOUS POINTS FOR ARCH RIB.

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MADE BY M. Iwata DATE 13-1-12 FILE NO
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CALCULATIONS FOR

Mieko-Ohashi over Mutsugawa, Nagoya-shi.
Stresses of Arch Ring for Traffic way and Sidewalks.
Dead Load Moments at Crown Section.
Dead Load on each rib per lin. ft. (21.68' wide).

Sidewalk floor.

Handrail			= 181.2	#/ft
Pavement	3/4" cement mortar finish	5.75 @ 4.7	= 35.5	
Slab	4½" reinforced concrete	6 @ 55.5	= 333.0	
Coping			120.3	
Bracket	220 + 55 mean		= 44.0	
Fascia girder	8'x14½" @ 150		= 173.00	
Gas pipe			= 40.00	
			Total 947.5	#/lin. ft.

Motor trackway floor.

Pavement	2" Asphaltic concrete	15.2 @ 22	= 335.0	
Slab	6½" reinf. concrete	17.18 @ 78.75	= 1,352.0	
Beams	B, C and D	3 @ 118	= 354.0	
Beam	A including chamfers		= 120.0	
Curb stone			40.0	
		total	2201.0 #/lin. m	
			3,148 #/lin. m	

Dead load = 3,148 x 5.5 = 17,300 #.

Dead Panel Load.

Panel Point	Weight of Columns	floor load	Wt. of Arch rib	fill and Cor beam	total Load
Crown 0	None	8,700	5,200	fill 1,600	15,500
1			10,600	3,600	14,200
2	None	17,300	10,700	2,100	30,100
3			10,800		10,800
4	.51x5@150=400	17,300	11,300		29,000
5			11,500		11,500
6	1.76x8@150=1,300	17,300	12,300		30,900
7			13,200		13,200
8	3.51x5@150=2,600	17,300	14,500	Cor beam 1,500	35,900
9			15,800		15,800
10	5.79x5@150=4,300	17,300	17,600		39,200
11			19,700		19,700
Springing 12	8.8x5@75=3,600	8,700	11,200		23,500

Note - Haunches at Crown shall be cinder concrete. (Unit weight equals 110 #/cu. ft.)

Dead Load Moments at Crown section.

Point	Load	Mo	+ M	- M
0	15,500 #	+2.650	+41,100	
1	14,200	+1.434	+20,400	
2	30,100	+0.529	+15,900	
3	10,800	-0.083		- 900
4	29,000	-0.458		-13,300
5	11,500	-0.610		- 7,000
6	30,900	-0.606		-18,700
7	13,200	-0.513		- 6,800
8	35,900	-0.368		-13,200
9	15,800	-0.222		- 3,500
10	39,200	-0.100		- 3,900
11	19,700	-0.024		- 500
12				
	265,800		+77,400	-67,800 = +9600 '# for one half span.
	23,500		for Entire span, 2 @ 9600 = +19,200 '#	
	289,300			

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JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Ouchi DATE 3-9-12 FILE NO
CHECKED BY DATE PAGE NO 20

CALCULATIONS FOR

Mutsu-Chashi ore, Mutsugawa, Nyogoka.
Dead Load Moments at Various Points.

Point	Load	Point # 1 M.Unit	+M	−M	Point # 2 M.Unit	+M	−M	Point # 3 M.unit	+M	−M
11R	19,700	-0.028		600	-0.028		600	-0.027		500
10	39,200	-0.117		4,600	-0.124		4,900	-0.121		4,700
9	15,800	-0.266		4,200	-0.287		4,500	-0.282		4,500
8	35,900	-0.456		16,800	-0.500		18,000	-0.500		18,000
7	13,200	-0.665		8,800	-0.750		9,900	-0.783		10,300
6	30,900	-0.845		26,100	-0.989		30,300	-1.034		32,000
5	11,500	-0.961		11,000	-1.188		13,700	-1.284		14,800
4	29,000	-0.947		27,500	-1.283		37,200	-1.459		42,300
3	10,800	-0.736		8,000	-1.210		13,100	-1.500		16,200
2	30,100	-0.308		9,300	-0.946		28,500	-1.378		41,500
1R	14,200	+0.396	5,600		-0.431		6,100	-1.038		14,800
0	31,000	+1.400	43,400		+0.366	11,300		-0.445		13,800
1L	14,200	+2.718	38,600		+1.463	20,400		+0.426	6,100	
2	30,100	+1.596	48,000		+2.862	86,200		+1.584	47,700	
3	10,800	+0.778	8,400		+1.816	19,600		+3.040	32,800	
4	29,000	+0.209	6,100		+1.031	30,000		+2.012	58,300	
5	11,500	-0.113		1,500	+0.508	5,400		+1.258	14,500	
6	30,900	-0.257		7,900	+0.189	5,800		+0.732	22,600	
7	13,200	-0.283		3,700	+0.016	200		+0.385	5,100	
8	35,900	-0.230		8,300	-0.048		1,700	+0.178	6,400	
9	15,800	-0.150		2,400	-0.053		800	+0.068	1,100	
10	39,200	-0.071		2,800	-0.032		1,200	+0.017	700	
11L	19,700	-0.018		400	-0.010		200	+0.001	0	
			+150,100	-143,900		+178,900	-170,700		+195,300	-213,400
			+ 6,200 '#			+ 8,200 '#			-18,100 '#	

Point	Load	Point # 4 M.unit	+M	−M	Point # 5 M.unit	+M	−M	Point # 6 M.unit	+M	−M
11R	19,700	-0.024		400	-0.030		600	-0.008		200
10	39,200	-0.104		4,100	-0.077		3,100	-0.050		2000
9	15,800	-0.246		3,900	-0.185		2,900	-0.098		1,600
8	35,900	-0.441		15,800	-0.338		12,100	-0.187		6,700
7	13,200	-0.688		9,100	-0.542		7,200	-0.322		4,300
6	30,900	-0.952		29,400	-0.770		23,800	-0.487		15000
5	11,500	-1.214		14,000	-1.014		11,700	-0.680		7,800
4	29000	-1.429		41,400	-1.240		36,000	-0.886		25,700
3	10,800	-1.551		16,800	-1.417		15,300	-1.090		11,800
2	30,100	-1.546		46,600	-1.508		45400	-1.255		37,800
1R	14,200	-1.365		19,800	-1.472		20,900	-1.351		19,200
0	31,000	-0.969		30,000	-1.269		39,000	-1.336		41,400
1L	14,200	-0.329		4,700	-0.864		12,300	-1.171		16,600
2	30,100	+0.570	17,100		-0.238		7,200	-0.831		25,000
3	10,800	+1.751	18,900		+0.649	7,000		-0.262		2,800
4	29,000	+3.197	92,700		+1.792	52,000		+0.554	16,100	
5	11,500	+2.176	25,000		+3.224	37,100		+1.656	19,100	
6	30,900	+1.402	43,300		+2.172	67,100		+3.045	94,200	
7	13,200	+0.844	11,100		+1.372	18,100		+1.947	26,200	
8	35,900	+0.463	16,600		+0.792	28,400		+1.169	42,000	
9	15,800	+0.220	3,500		+0.399	6,300		+0.602	9,500	
10	39,200	+0.080	3,100		+0.125	6,800		+0.248	9,700	
11L	19,700	+0.014	300		+0.030	600		+0.048	906	
			+231,600	-236,000		+223,400	-237,500		+217,700	-217,900
			-4,400 '#			-14,100 '#			-200 '#	

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Imada. DATE 13-9-12 13-913 FILE NO
CHECKED BY DATE PAGE NO 21

CALCULATIONS FOR

Mutsu-Ohashi over Mutsugawa, Hyogoken.
Dead Load Moments at Various Points Continued.

Point	Load	Point #7 M.unit	+M	−M	Point #8 M.unit	+M	−M	Point #9 M.unit	+M	−M
11R	19,700	+0.003	100		+0.018	400		+0.035	700	
10	39,200	+0.011	400		+0.073	2,900		+0.149	5,800	
9	15,800	+0.018	300		+0.161	2,500		+0.337	5,300	
8	35,900	−0.014		500	+0.265	9,500		+0.578	20,700	
7	13,200	−0.024		300	+0.354	4,700		+0.825	10,900	
6	30,900	−0.092		2800	+0.413	12,800		+1.054	32,600	
5	11,500	−0.199		2300	+0.427	4,900		+1.229	14,100	
4	29,000	−0.353		10,200	+0.361	10,500		+1.291	37,500	
3	10,800	−0.556		6000	+0.208	2,200		+1.182	12,800	
2	30,100	−0.771		23200	±0.056		1,700	+0.939	28,200	
1R	14,200	−0.984		14,000	−0.371		5300	+0.540	7,700	
0	31,000	−1.152		35,800	−0.717		22,200	+0.022	700	
1L	14,200	−1.234		17,500	−1.049		14,900	−0.566		8,000
2	30,100	−1.193		35,900	−1.324		39,800	−1.175		35,400
3	10,800	−0.964		10,300	−1.459		15,800	−1.700		18,400
4	29,000	−0.505		16,000	−1.385		40,200	−2.048		59,400
5	11,500	+0.233	2700		−1.043		12,000	−2.143		24,700
6	30,900	+1.278	39,500		−0.379		11,700	−1.900		58,700
7	13,200	+2.656	35,000		+0.665	8,800		−1.229		16,200
8	35,900	+1.596	57,300		+2.073	74,500		−0.138		5,000
9	15,800	+0.834	13,200		+1.093	17,300		+1.387	21,900	
10	39,200	+0.335	13,100		+0.443	17,400		+0.565	22,200	
11L	19,700	+0.069	1,400		+0.092	1.800		+0.119	2300	
		+163,000 −174,800 −11,800'#			+170,200 −163,600 +6600			+223,400 +225,800 −2,400'#		

Point	Load	Point # 10 M.unit	+M	−M	Point # 11 M.unit	+M	−M	Point # 12 (Spr) M.unit	+M	−M
11R	19,700	+0.054	1,100		+0.079	1,600		+0.105	2,400	
10	39,200	+0.239	9,400		+0.343	13,400		+0.463	18,100	
9	15,800	+0.546	8,600		+0.791	12,500		+1.071	16,900	
8	35,900	+0.948	34,000		+1.384	49,700		+1.884	67,600	
7	13,200	+1.388	17,700		+2.052	27,100		+2.817	37,200	
6	30,900	+1.820	56,300		+2.730	84,400		+3.782	116,800	
5	11,500	+2.197	25,300		+3.353	38,500		+4.696	54,000	
4	29,000	+2.436	70,500		+3.790	109,800		+5.385	156,000	
3	10,800	+2.414	26,100		+3.913	42,200		+5.680	61,500	
2	30,100	+2.198	63,100		+3.754	113,000		+5.608	171,000	
1R	14,200	+1.732	24,600		+3.241	46,000		+5.066	72,000	
0	31,000	+1.048	32,500		+2.396	74,400		+4.067	126,000	
1L	14,200	+0.198	2600		+1.275	18,100		+2.674	37,900	
2	30,100	−0.760		22900	−0.050		1,500	+0.958	28,800	
3	10,800	−1.706		18400	−1.443		15,600	−0.914		9,900
4	29,000	−2.508		72,800	−2.736		79,300	−2.735		79,300
5	11,500	−3.077		35,400	−3.823		44,100	−4.384		50,500
6	30,900	−3.24		101,800	−4.546		140,400	−5.656		174,900
7	13,200	−3.034		40,100	−4.738		62,500	−6.339		83,600
8	35,900	−2.292		82,400	−4.380		157,500	−6.404		230,000
9	15,800	−1.038		16,400	−3.427		54,200	−5.779		91,400
10	39,200	+0.701	27,500		−1.899		74500	−4.483		175,800
11L	19,700	+0.148	2900		+0.181	3,600		−2.533		49,800
		+402,200 −390,200 +12,000'#			+634,300 −629,600 +4,700'#			+966,200 −945,200 +21,000'#		

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（独立行政法人 土木研究所蔵）

JUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Inaba DATE 13-9-43 FILE NO
CHECKED BY DATE PAGE NO 22

CALCULATIONS FOR

Mutsu-Ebashi over Mutsugawa, Hyogoken.
Dead Load Thrusts.

Point	Load	Ho Unit load	Ho	Vert. load: Vert. shear	Vert. load: Unit load	Vert. load: Normal T	Hor. Thrust: H	Hor. Thrust: Unit load	Hor. Thrust: Normal T
0 crown	15,500	1.7917	27,800	0	0	1,000	438000	1,000	438,000
1	14,200	1.7561	24,900	22,600	0.043	4,100		0.999	437,600
2	30,100	1.6504	49,700	44,750	0.091	9,100		0.996	436,200
3	10,800	1.4841	16,000	65,200	0.140	15,400		0.991	434,100
4	39,000	1.2783	37,100	85,100	0.181	24,200		0.982	430,100
5	11,500	1.0391	11,900	105,400	0.230	34,800		0.972	425,700
6	30,900	0.7919	24,500	126,550	0.275	48,100		0.960	420,500
7	13,200	0.5627	7,400	148,600	0.324	64,100		0.945	413,900
8	35,900	0.3608	13,000	173,200	0.370	83,000		0.928	406,500
9	15,800	0.1993	3,100	199,000	0.417	105,500		0.910	398,600
10	39,200	0.0840	3,300	226500	0.466	130,300		0.887	388,500
11	19,700	0.0185	400	256000	0.509	130,300		0.862	377,600
12 Spr.	11,200 counterweight only	0	0	277,000	0.556	151,400		0.834	365,300
	277,000#		219000						

H for entire span 438,000#

Summary of Dead Load Normal Thrusts, Moments and Eccentricities.

Points	Normal Thrusts: due to vert. load	due to hor. load	Sum	moments	eccentricity
0 crown	0	438,000	438,000	+ 19,200'#	+ 0.044
1	1,000	437,600	438,600	+ 6,200	+ 0.014
2	4,100	436,200	440,300	+ 8,200	+ 0.019
3	9,100	434,100	443,200	− 18,100	− 0.041
4	15,400	430,100	445,500	− 4,400	− 0.010
5	24,200	425,700	449,900	− 14,100	− 0.031
6	34,800	420,500	455,300	− 200	− 0.000
7	48,100	413,900	462,000	− 11,800	− 0.026
8	64,100	406,500	470,600	+ 6600	+ 0.014
9	83,000	398,600	481,600	− 2,400	− 0.005
10	105,300	388,500	494,000	+ 12,000	+ 0.024
11	130,300	377,600	507,900	+ 4,700	+ 0.009
12 Springing	151,400	365300	516,700	+ 11,900 *	+ 0.023

Note:— Resultant acts above of Neutral axis for 4 Spr. of Eccentricity and over crown.

* Add moment due to wt. of brick
— 11,200 × 0.81 = + 9,100'#
M1/2 = 21,000 − 9,100 = 11,900'#

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Inaba. DATE 13-9-14 FILE NO
CHECKED BY DATE PAGE NO 23

CALCULATIONS FOR

Mutsu-Ohashi over Mutsugawa, Nyugotsen.

Live Load Stresses of Arch Rib under Trafficway.

Concentrated Loads:—

12 ton motor truck concentration.

$$\text{Impact } I = \frac{20}{60+l} = \frac{20}{60+\frac{66}{3.28}} = 25\,\%$$

12 ton motor truck Rear wheel concentration = 9,900 × 1.25 = 12,400# for 2 lines, 49,600#
Front wheel concentration = 4,300 × 1.25 = 4,100# " " 16,400#

14 ton road roller concentration

one roller on one span with no impact. Front wheel concentration 12,300#
Rear wheel concentration 9,250 × 2 = 18,500#

Uniform Live Load.

Motor truckway $w = \frac{120,000}{170+l} = \frac{120,000}{170+\frac{66}{3.28}} = 630\ kg/m^2$ Assume $w = 600\ kg/m^2 = 125\ \#/ft^2$

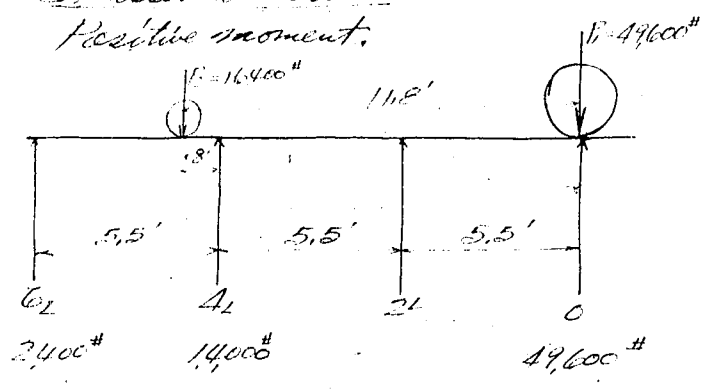
Sidewalk $w = \frac{100,000}{170+l} = \frac{100000}{170+\frac{66}{3.28}} = 527\ kg/m^2$ Assume $w = 500\ kg/m^2 = 100\ \#/ft^2$

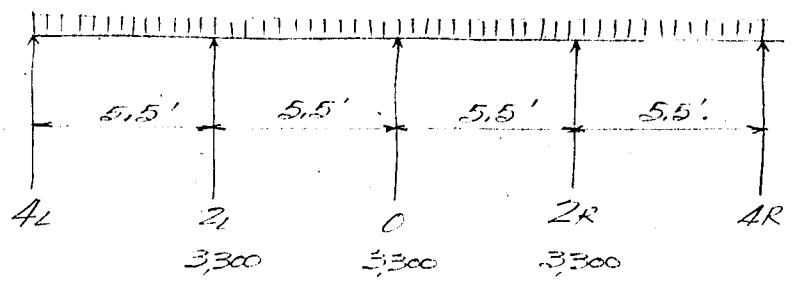
uniform live load per lin. ft. of Motor truckway 125 × 16.93' = 2,120#/ft. panel load = 2,120 × 5.5 = 11,600#
Sidewalk 100 × 5.97 = 600#/ft. " " = 600 × 5.5 = 3,300#

Assumption: 2 Motor trucks side by side and uniform live load around them on roadway, and uniform live load on sidewalk assumed. (There is no space for uniform live load, on both sides of the trucks.)

Stresses at Crown.

Positive moment.





Moto-truck.

Point	load	M. unit	moment	H. unit load	Hor. thrust	Shear unit load	vert shear.
0	49,600	+2.650	+131,500	1.7917	87,100	−0.5000	−24,800
2L							
4L	14,000	−0.458	−6,400	1.2783	17,900	−0.3462	−2,900
6L	3400	−0.606	−1,500	0.7917	1,900	−0.1070	−300
			+123,600'#		106,900#		−28,000#

Uniform Live Load.

(Loads on sidewalk only)

Point	load	M. unit load	moment	H. unit load	Hor. thrust.	Shear unit load.	vert shear.
2R	3,300	+0.529	+1,700	1.6504	5,400	+0.346	+1,100
0	3,300	+2.650	+8,800	1.7917	5,900	−0.500	−1,600
2L	3,300	+0.529	+1,700	+1.6504	5,400	−0.346	1,100
			+12,200'#		16,700		1,600

Summary of + moments, thrusts and shears at Crown section.

	moment	Thrust	vert shear	Normal thrust
Motor truck	+123,600	106,900	−28,000	123,600 × 1.0 = 123,600
uniform live load	+12,200	16,700	1,600	−29,600 × 0.0 = 0
	+135,800'#	123,600	−29,600	123,600#

Neg. moment at crown section is small compared to positive moment. It is unnecessary to investigate.

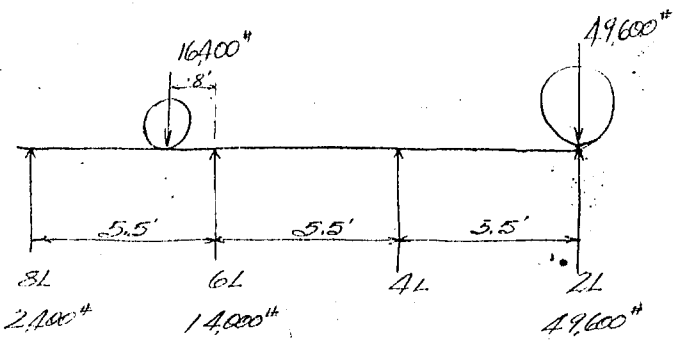
増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Iraba DATE 13-9-14 FILE NO
CHECKED BY DATE PAGE NO 24

CALCULATIONS FOR

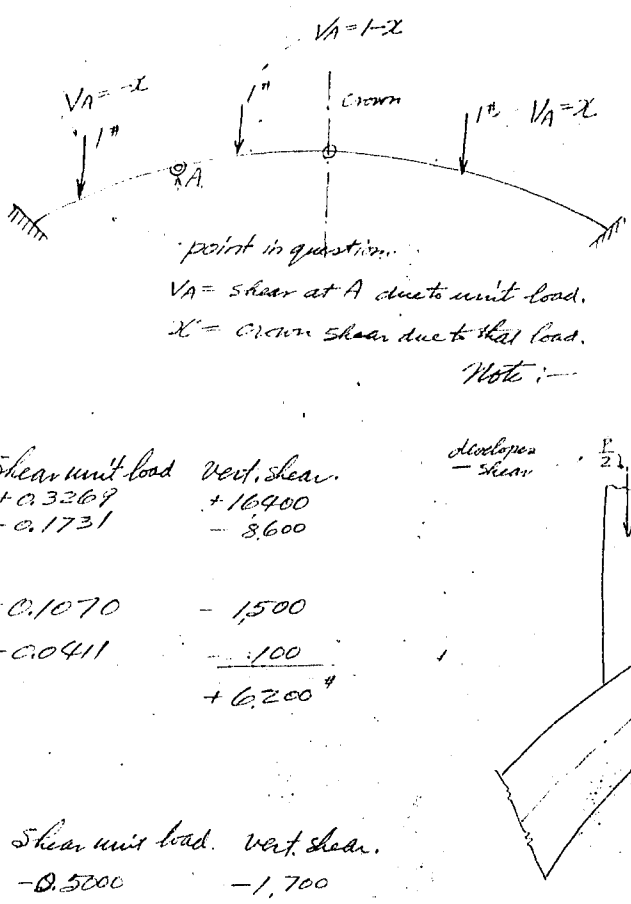
Mukō-Ohashi over Mukōgawa, Hyōgoken.
Stresses at Panel Point #2L.
Positive Moments
Motor Truck.



Point	Load	M. unit load	Moment	H. unit load	Hor. Thrust	Shear unit load	Vert. Shear
2L	49,600	+0.2862	+142,000	1.6504	81,900	+0.3269 -0.1731	+16400 -8,600
4L							
6L	14,000	+0.189	+2,600	0.7919	11,100	-0.1070	-1,500
8L	2,400	-0.048	-100	0.3608	900	-0.0411	-100
			+144,500#		93,900#		+6,200#

Uniform Live Load.

Point	Load	M unit load	moment	H. unit load	Hor. Thrust	Shear unit load	Vert. Shear.
0	3,300	+0.366	+1,200	1.7917	5,900	-0.5000	-1,700
2L	3,300	+2.862	+9,500	1.6504	5,500	+0.3269 -0.1731	+1,100 -600
4L	3,300	+1.031	+3,400	1.2783	4,200	-0.2103	-700
6L	3,300	+0.189	+600	0.7919	2,600	-0.1070	-400
			+14,700		18,200		-2,300



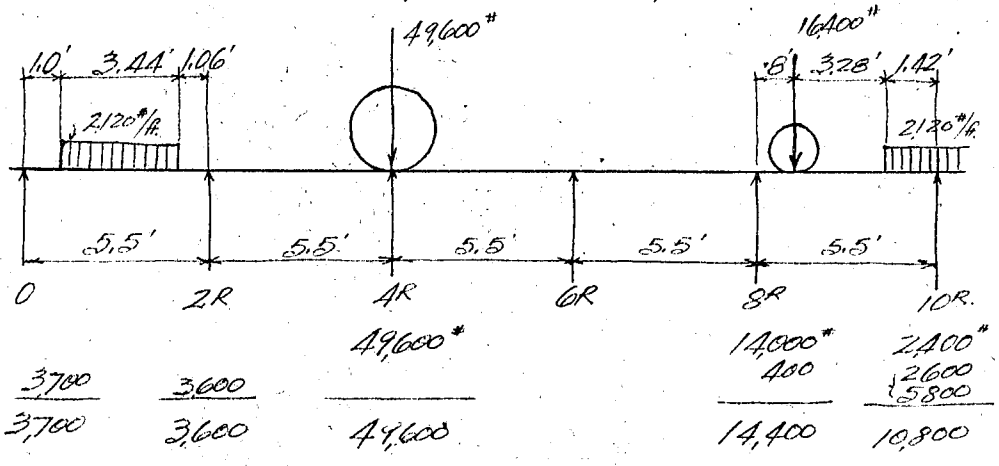
point in question.
V_A = shear at A due to unit load.
x = crown shear due to that load.
Note :—
Half load on both sides of the section. Right half developes + shear and left half — shear, is assumed.

Summary of Positive moments and thrusts for Section #2.

	Moment	Thrust	Vert. Shear.
Motor truck	+144,500	93,900	+6,200
unif. load	+14,700	18,200	-2,300
	+159,200	112,100	+3,900

Normal thrust.
112,100 × 0.996 = 111,700#
3900 × 0.091 = 3,500
115,200

Neg. Moment at Panel Point #2L.
Motor truck and uniform live load before and behind it.



Point	Load	M. unit load	Moment	H. unit load	Hor. Thrust	Shear unit load	Vert. Shear.
0	3,700	+0.366	+1,300	1.7917	6,600	+0.5000	+1,900
2R	3,600	-0.946	-3,400	1.6504	5,900	+0.3462	+1,200
4R	49,600	-1.283	-63,700	1.2783	63,400	+0.2103	+10,400
6R		-0.989		0.7919		+0.1070	
8R	14,400	-0.500	-7,200	0.3608	5,200	+0.0411	+600
10R	10,800	-0.124	-1,300	0.0840	900	+0.0084	+100
			-74,300		82,000		+14,200

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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MARUNOUCHI, TOKIO

MADE BY K. Iraba DATE 15 7-14 FILE NO
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CALCULATIONS FOR

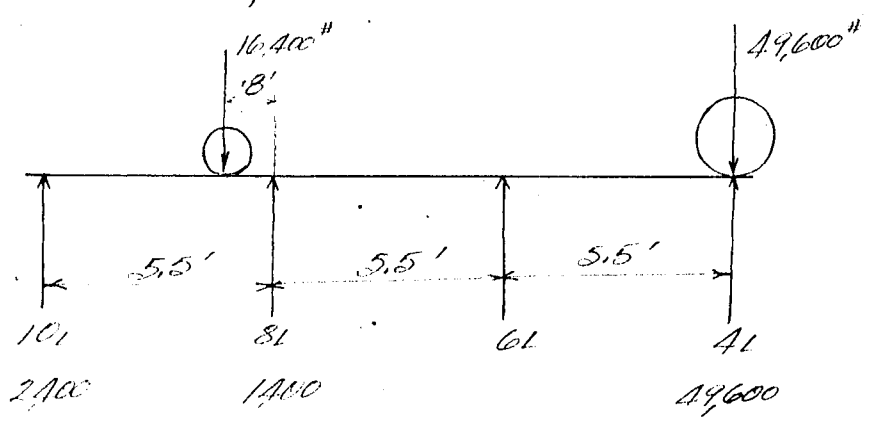
Mutsu-Ohashi over Mutsugawa, Nyugoken.
Neg. Moment at Panel Pt. #2L Continued.
Uniform Load on Sidewalks.

Point	Load	M unit load	Moment	H unit load	Hor Thrust	V unit load	Vert. Shear
10L	3,300	- 0.032	- 100	0.0840	300	-0.0084	-0
8L	3,300	- 0.048	- 200	0.3608	1,200	-0.0411	-100
2R	3,300	-0.946	- 3,100	1.6504	5,500	+0.3462	-1,100
4R	3,300	-1.283	- 4,200	1.2783	4,200	+0.2103	+ 700
6R	3,300	-0.989	- 3,100	0.7919	2,600	+0.1070	+ 400
8R	3,300	-0.500	- 1,700	0.3608	1,200	+0.0411	+100
10R	3,300	-0.124	- 400	0.0840	300	+0.0084	0
			-12,800		15,300		+2,200

Summary of Negative moments and Thrusts for Section #2L.

	Moment	Horiz. thrust	vert. shear	Normal thrusts.
[illegible]	[illegible]	82,000	[illegible]	[illegible]
Unif. load	-12,800	[illegible]	[illegible]	-500
[illegible]	[illegible]	71,300	+16,400	98,400

Stresses at Panel Point #1
Positive moments
Motor truck.



Point.	Load.	M unit load.	Moment.	H. unit load.	Hor. thrust.	Shear unit load	Vert. Shear.
4L	49,600	+3,197	+158,400	1.2783	63,400	+0.3949 -0.1052	+19,600 - 5,300
6L		+1,402		0.7919		-0.1070	
8L	14,000	+0.463	+ 6,500	0.3608	5,100	-0.0411	- 600
10L	2,400	+0.080	+ 200	0.0840	200	-0.0084	0
			+165,100		68,700		+ 13,700

Uniform Loading (Sidewalks only).

Point	Load	M. unit load	Moment	H. unit load	Hor. thrust.	V unit load	Vert. Shear.
2L	3,300	+ 0.570	+1,900	1.6504	5,400	+0.6538	+ 2,200
4L	3,300	+ 3,197	+10,500	1.2783	4,200	+0.3949 -0.1052	+1,300 - 400
6L	3,300	+1.402	+ 4,600	0.7919	2,600	-0.1070	- 400
8L	3,300	+0.463	+1,500	0.3608	1,200	-0.0411	- 100
10L	3,300	+0.080	1 300	0.084	300	-0.0084	0
			+18,800		13,700		+2,600

Summary of Positive Moments and Thrusts for Panel Point #4.

	Moment	Thrust	vert. shear	Normal thrust.
Motor truck	+165,100	68,700	+13,700	82,400 × 0.982 = 80,800#
Unif. live load	+ 18,800	13,700	+ 2,600	16,300 × 0.181 = 3,000
Total	+183,900	82,400	+16,300	83,800#

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（独立行政法人 土木研究所蔵）

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MARUNOUCHI, TOKIO

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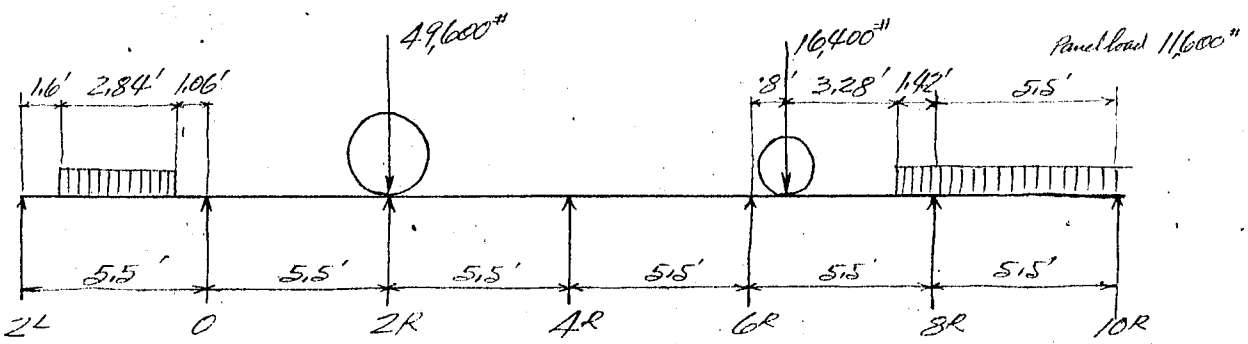
CALCULATIONS FOR

Mutsu-Ohashi over Mutsugawa, Hyogoken.
Negative Moment at Panel Point # 4.
Motor truck and unif. live load on carriageway.



			49,600	14,000	2,400		
2,700	3,300			400	2,600 / 5,800	11,600	
2,700	3,300		49,600	14,400	10,800	11,600	

Point	Load	M. unit load	Moment	H. unit load	Hor. thrust	V. unit load	Vert. shear.
2L	2,700	+ 0.570	+1,500	1.6504	4,500	+ 0.6538	+1,800
0	3,300	− 0.969	−3,200	1.7917	5,900	+ 0.5000	+1,700
2R	49,600	− 1.546	−76,700	1.6504	82,000	+0.3462	+17,200
4R							
6R	14,400	− 0.952	−13,700	0.7919	11,400	+0.1070	+1,500
8R	10,800	−0.441	− 4,800	0.3608	3,900	+0.0411	+ 400
10R	11,600	−0.104	−1,200	0.0840	1,000	+0.0084	+ 100
			−98,100		108,700		+22,700

Uniform load on Sidewalks.

Point	Load	M. unit load	Moment	H unit load	Hor. thrust	V unit load	Ver. shear.
0	3,300	−0.964	−3,200	1.7917	5,900	+ 0.5000	+ 1700
2R	3,300	−1.546	−5,100	1.6504	5,400	+ 0.3462	+ 1100
4R	3,300	−1.429	−4,700	1.2783	4,200	+0.2103	+ 700
6R	3,300	−0.952	−3,100	0.7919	2,600	+0.1070	+ 400
8R	3,300	−0.441	−1,400	0.3608	1,200	+0.0411	+1,300
10R	3,300	−0.104	− 300	0.0840	300	+0.0084	+ 0
			−17,800		19,600		+5,200

Summary for Negative moment and thrust for Panel Point # 4.

	Moment	Thrust	Vert. shear	Normal Thrust.
Motor truck	−98,100	108,700	+22,700	128,300×0.982 = 126,100 #
Unif. live load	−17,800	19,600	+ 5200	27,900×0.181 = 5,100
Total	−115,900	128,300	+27,900	131,200

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（独立行政法人 土木研究所蔵）

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CONSULTING ENGINEER
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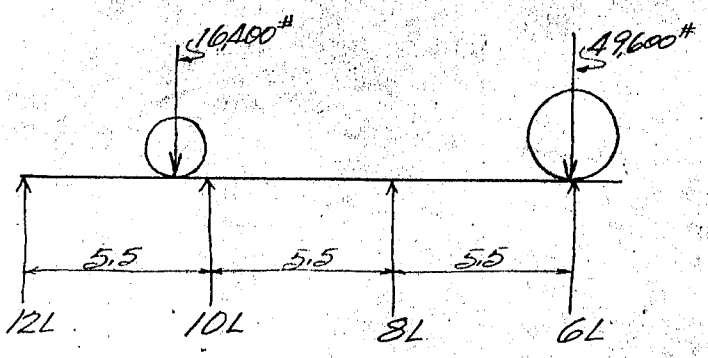
CALCULATIONS FOR

Mukō-Ohashi over Mukōgawa, Hyogoken.
Stresses at Panel Point # 6.
Positive moment.
Motor truck concentration (no unif. load on carriageway)



2,400 14,000 49,600

Point	Load	M. unit load	Moment	H. unit load	Hor. thrust.	Shear unit load	Vert. shear.
6L	49,600	+ 3.045	+151,000	0.7919	39,200	+ 0.4465 − 0.0535	+ 22,100 − 2,700
8L							
10L	14,000	+0.248	+ 3,500	0.0840	1,200	− 0.0084	− 100
			+154,500#		40,400		+ 19,300

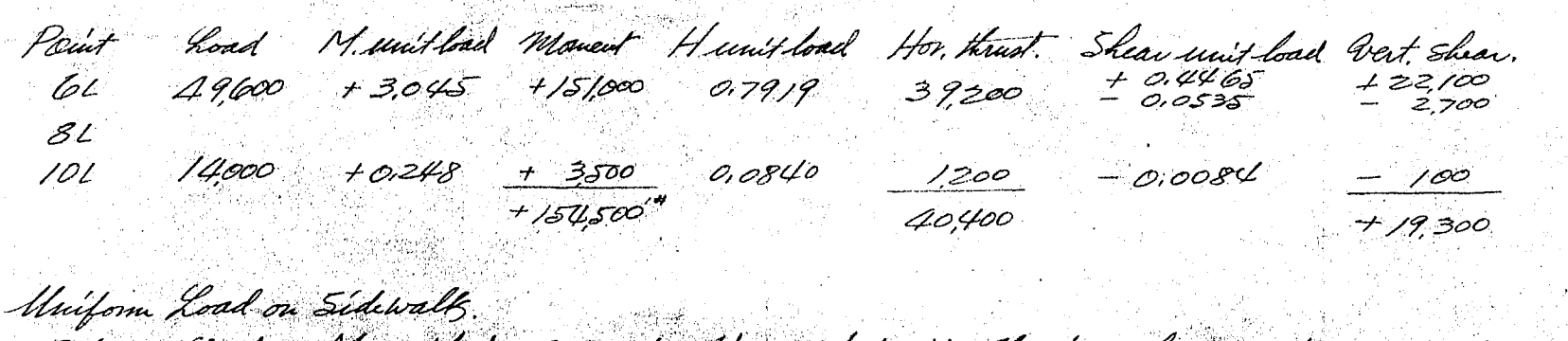
Uniform Load on Sidewalk.

Point	Load	M. unit load	Moment	H. unit load	Hor. thrust	Shear unit load	Vert. shear.
4L	3,300	+0.554	+1,800	1.2783	4,200	+ 0.7897	+ 2,600
6L	3,300	+ 3.054	+10,100	0.7919	2,600	+ 0.4465 − 0.0535	+ 1,500 200
8L	3,300	+ 1.169	+ 3,900	0.3608	1,200	− 0.0411	− 100
10L	3,300	+0.248	+ 800	0.0840	300	− 0.0084	− 0
			+16,600		8,300		+ 3,800

Summary of Positive Moments and thrust for Panel Point # 6.

	Moment	Thrust	Vert. Shear.	Nominal thrust
Motor trucks	+ 154500	40,400	+19,300	48,700 × 0.960 = 46,700#
Unif. load	+ 16,600	8,300	+ 3,800	23,100 × 0.275 = 6,400
Total	+ 171,100	48,700	+23,100	53,100#

Negative Moment and Thrust at Panel Point # 6



49,600
14,000 2,400
400 2,600
5,800
14,400 10,800

2900 3,300 11600 11600
2,900 3,300 49,600 11,600 11,600

Point	Load	M. unit load	Moment	H. unit load	Hor. thrust.	Shear unit load	Vert. shear.
4L	2,900	+0.554	+1,600	1.2783	3,700	+ 0.7897	+ 2,300
2L	3,300	−0.831	−2,700	1.6504	5,500	+ 0.6538	+ 2,200
0	49,600	−1.336	−66,300	1.7917	88,900	+ 0.5000	+ 24,800
4R	14,400	−0.886	−12,800	1.2783	18,400	+ 0.2103	+ 3,000
6R	10,800	−0.487	− 5,300	0.7919	8,600	+ 0.1070	+ 1,200
8R	11,600	−0.187	− 2,200	0.3608	4,200	+ 0.0411	+ 500
10R	11,600	−0.050	− 600	0.084	1,000	+ 0.0084	+ 100
			−88,300		130,300		+ 34,100

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

Mutsu-Ohashi over Mutsugawa, Nyogotsu.

Uniform Load on Sidewalk.

Point	Load	M. unit load	Moment	H. unit load	Hor. thrust	Shear unit load	Vert. Shear.
2L	3,300	− 0.831	−2,700	1.6504	5,500	+ 0.7897	+ 2,600
0	3,300	−1.336	−4,400	1.7917	5900	+ 0.5000	+ 1,700
2R	3,300	−1.255	−4,100	1.6504	5,500	+ 0.3462	+ 1,100
4R	3,300	−0.886	−2,900	1.2783	4,200	+0.2103	+ 700
6R	3,300	−0.487	−1,600	0.7919	2,600	+0.1070	+ 400
8R	3,300	−0.187	− 600	0.3608	1,200	+0.0411	+ 100
10R	3,300	−0.050	− 200	0.0840	300	+0.0084	+ 0
			−16,500		25,200		+ 6,600

Summary of Negative Moments and Thrusts for Panel Point #6.

	Moment	Thrust	Vert. shear
Motor truck	−88,300	130,300	34,100
Unif. load	− 16,500	25,200	6,600
Total	−104,800	155,500	40,700

Stresses at Panel Point #8.

Positive moment.

Motor truck and uniform load on carriageway

Point	Load	M. unit load	Moment	H. unit load	Hor Thrust	Shear unit load	Vert. Shear.
8L	49,600	+2,073	+102,800	0.3608	17,900	{−0.0206 +0.4795	{−1,000 +23,800
4R	11,600	+0.361	+ 4,200	1.2783	14,800	+0.2103	+ 2,300
6R	11,600	+0.413	+ 4,800	0.7919	9,200	+0.1070	+ 1,200
8R	11,600	+0.265	+ 3,100	0.3608	4,200	+0.0411	+ 500
10R	11,600	+0.073	+ 800	0.0840	1,000	+0.0084	+ 100
			+115,700		47,100		+26,900

Uniform Load on Sidewalk.

Point	Load	M. unit load	moment	H unit load	Hor. thrust	Shear unit load	Vert. Shear.
10L	3,300	+0.443	+1,400	0.0840	300	−0.0084	− 0
8L	3,300	+2.073	+ 6,800	0.3608	1,200	{−0.0206 +0.4795	−100 +1,600
4R	3,300	+0.361	+1,200	1.2783	4,200	+0.2103	+ 700
6R	3,300	+0.413	+1,400	0.7919	2,600	+0.1070	+ 400
8R	3,300	+0.265	+ 900	0.3608	1,200	+0.0411	+100
10R	3,300	+0.073	+ 200	0.0840	300	+0.0084	+ 0
			+11,900		9,800		+2,700

Summary of Positive Moment and Thrusts for Panel Point #8.

	Moment	Thrust	Vert. Shear.	Normal thrust.
Motor truck	+115,700	47,100	+26,900	56,900×0.928 = 52,800
Unif. load on sidewalk.	+ 11,900	9,800	+ 2,700	29,600×0.370 = 10,900
	+127,600	56,900	+ 29,600	63,700

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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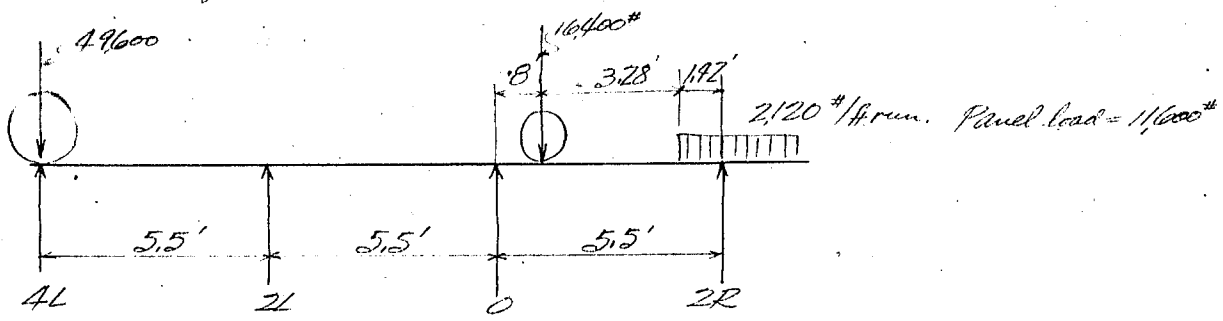
CALCULATIONS FOR

Mukō-Okashi over Mukōgawa, Hyōgoken.
Negative Moments and Thrusts for Panel Point #8.
Motor truck and uniform live load on trafficway.



	49,600		14,000		2,400		
			400		2,600		
					5,800		
	49,600		14,400		10,800		

Point.	Load	M. unit load	Moment	H. unit load	Hor. thrust.	Shear unit load	Vertical Shear.
4L	49,600	−1.385	−68,700	1.2783	58,700	+0.7897	+39,100
2L							
0	14,400	−0.717	−10,300	1.7917	25,800	+0.5000	+7,200
2R	10,800	−1.324	−14,300	1.6504	17,800	+0.3462	+3,700
			−93,300		102,300		+50,000

Uniform Load on Sidewalk.

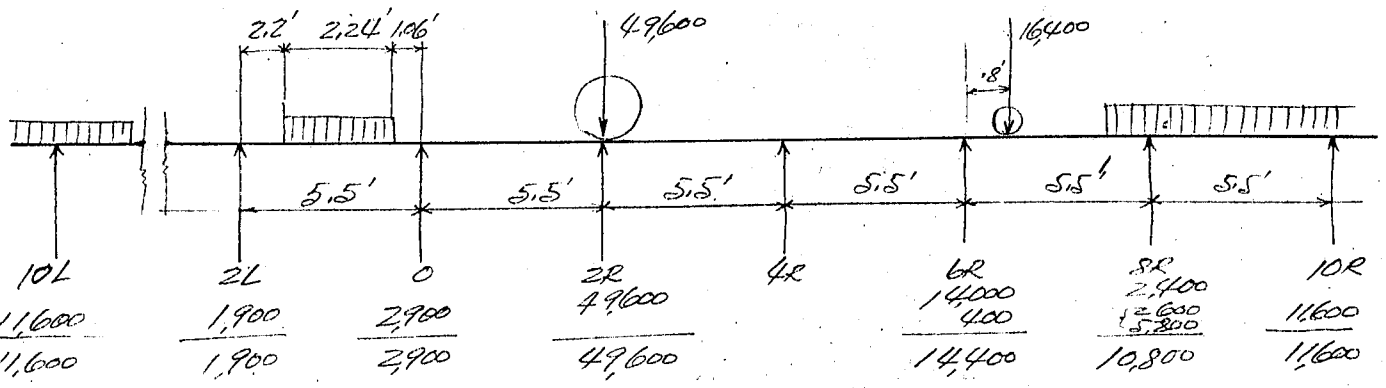
Point	Load	M. unit load	Moment	H. unit load	Hor. thrust	Shear unit load	Vertical Shear.
6L	3,300	−0.379	−1,300	0.7919	2,600	+0.8930	+2,900
4L	3,300	−1.385	−4,600	1.2783	4,200	+0.7897	+2,600
2L	3,300	−1.324	−4,400	1.6504	5,400	+0.6538	+2,200
0	3,300	−0.717	−2,400	1.7917	5,900	+0.5000	+1,700
2R	3,300	−0.056	−200	1.6504	5,400	+0.3462	+1,100
			−12,900		23,500		+10,500

Summary of Negative Moment and Thrust for Panel Point #8.

	Moment	thrust	Vert. Shear
Motor truck.	−93,300	102,300	+50,000
Unif. live load on Sidewalk	−12,900	23,500	+10,500
	−106,200	125,800	+60,500

Normal thrust.
125,800 × 0.928 = 116,800
60,500 × 0.370 = 22,400
139,200

Stresses at Panel Point #10.
Positive moment.
Motor trucks.



Point	Load	M. unit load	Moment	H. unit load	Hor. thrust	V. unit load.	Vert. Shear.
10L	11,600	+0.701	+8,100	0.0840	1,000	+0.0084	+100
2L	1,900	−0.760	−1,400	1.6504	3,100	+0.6538	+1,200
0	2,900	+1.048	+3,000	1.7917	5,200	+0.5000	+1,500
2R	49,600	+2.198	+108,800	1.6504	81,900	+0.3462	+17,200
4R		+2.436		1.2783		+0.2103	
6R	14,400	+2.820 +2.474	+26,200	0.7919	11,400	+0.1070	+1,500
8R	10,800	+0.948	+10,300	0.3608	3,800	+0.0411	+400
10R	11,600	+0.239	+2800	0.0840	1,000	+0.0084	+100
			+157,800		107,400		122,000

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

Mutsu-Ohashi ove Mutsugawa, Hyogo Ken.
Stresses at Panel Point #10 continued.
Uniform load on sidewalk.

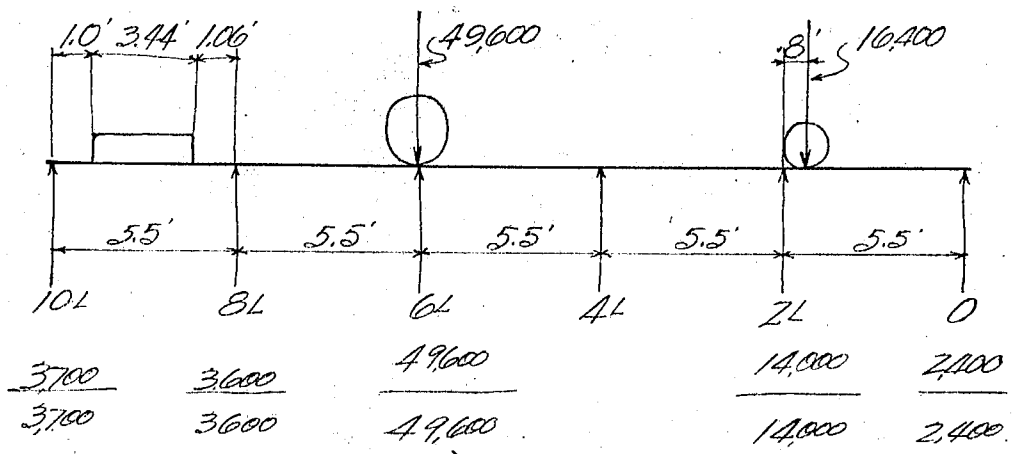
Point	Load	M. unit load	Moment	H. unit load	Hor. Thrust	V. unit load	Vert. Shear.
0	3,300	+1.048	+3,500	1.7917	5,900	+0.5000	+1,700
2R	3,300	+2.8198	+7,300	1.6504	5,400	+0.3462	+1,100
4R	3,300	+2.436	+8,100	1.2783	4,200	+0.2103	+700
6R	3,300	+1.820	+6,000	0.7919	2,600	+0.1070	+400
8R	3,900	+0.948	+3,100	0.3608	1,200	+0.0411	+100
10R	3,300	+0.239	+ 800	0.0840	300	+0.0084	+ 0
			+28,800		19,600		+4,000

Summary of Positive moments and thrusts for Panel Point #10.

	Moment	Thrust	Vert. Shear.
Motor truck	+ 157,800	107,400	+ 22,000
Unif. load on sidewalk	+ 28,800	19,600	+ 4000
	+ 186,600	127,000	+26,000

Hor. thrust:
127,000 × 0.887 = 112,600
26,000 × 0.466 = 12,100
124,700

Negative Moments and Thrusts at Panel Point #10
Motor trucks and Unif. load on carriageway.



3,700 / 3,700
3,600 / 3600
49,600 / 49,600
14,000 / 14,000
2,400 / 2,400

Point	Load	M. unit load	Moment	H. unit load	Hor. thrust.	V. unit load	Vert. Shear.
10L	3,700	+0.701	+ 2,600	0.0840	300	−0.0042 / +0.4958	−0
8L	3,600	−2.292	− 8,200	0.3608	1,300	+0.9589	+1,800
6L	49,600	−3.294	−163,000	0.7919	39,200	+0.8930	+3,400
4L		−2.508		1.2783		+0.7897	+44200
2L	14,000	−0.760	−10,600	1.6504	23,100	+0.6538	+ 9,200
0	2,400	+1.048	+ 2,500	1.7917	4,300	+0.5000	+ 1,200
			−176,700		68,200		+59,800

Unif. load on Sidewalk.

Point	Load	M. unit load	Moment	H. unit load	Hor. thrust	V unit load	Vert. Shear.
8L	3,300	−2.292	−7,600	0.3608	1,200	+0.9589	+ 3,200
6L	3,300	−3.294	−10,900	0.7919	2,600	+0.8930	+ 2,900
4L	3,300	−2.508	− 8,300	1.2783	4,200	+0.7897	+ 2,600
2L	3,300	−0.760	− 2,500	1.6504	5,500	+0.6538	+ 2,200
			−29,300		13,500		+10,900

Summary of Negative Moments and Thrusts at Panel Point #10

	Moment	Thrust	Vert. Shear	Hor. thrust
Motor truck	−176,700	68,200	+ 59,800	81,700 × 0.887 = 72,500
Unif. load on side walk.	− 29,300	13,500	+ 10,900	70,700 × 0.446 = 31,500
	−206,000	81,700	+ 70,700	104,000

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CONSULTING ENGINEER
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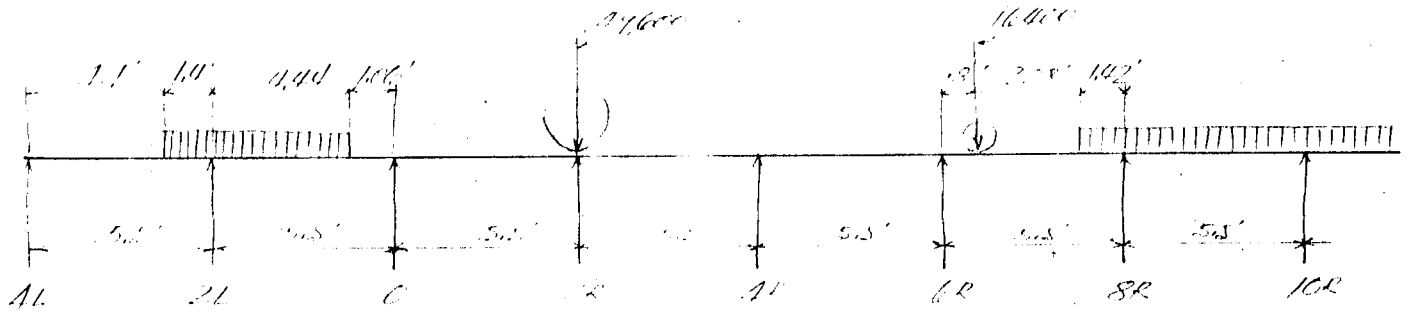
CALCULATIONS FOR

Motor Trucks on Motorgaun, Yokohama.
Stresses at Springing. (Road Act "D")
Positive Moments and Thrusts.
Motor trucks and Non Uniform load on Trafficway.



400	16,000	3,500		400	5,600	11,600	
400	9,200	3,800	49,600	14,000	10,800	4,600	

Point	Load	M. unit load	Moment	H. unit load	Hor. Thrust	V. unit load	Vert. Shear.
A1	400	−2.735	− 1,100	1.2183	500	+0.7897	+ 300
21	1,200	+0.758	+ 7,900	1.6504	13,500	+0.6538	+ 5,400
C	3,800	+4.067	+15,400	1.7917	6,800	+0.5000	+ 1,900
2C	49,600	+5.608	+278,000	1.6504	81,900	+0.3462	+17,200
4L		+5.385		1.2783		+0.2103	
6L	14,000	+3.782	+ 54,000	0.7919	11,400	+0.1070	+ 1,500
8R	10,800	+1.884	+ 20,300	0.3608	3,900	+0.0411	+ 400
10R	11,600	+0.463	+ 5,400	0.0540	1,000	+0.0054	+ 100
			+379,900		119,000		+26,800

Unif. load on Sidewalk.

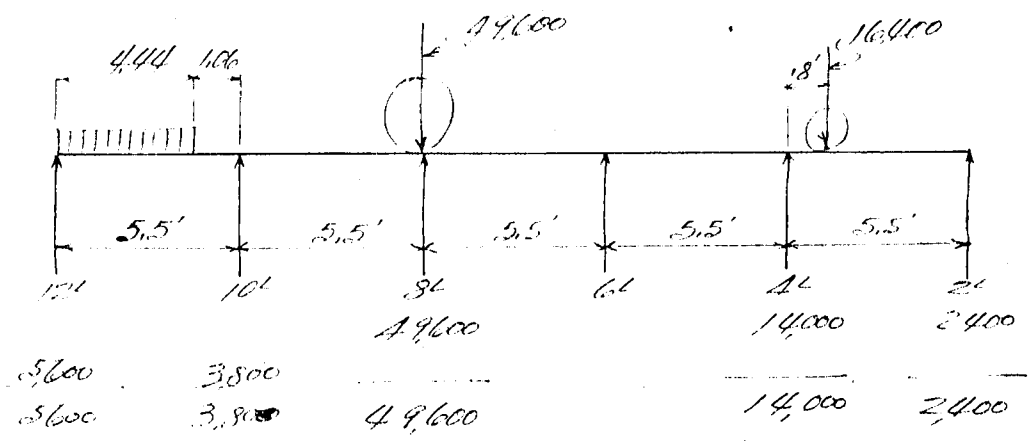
Point	Load	M. unit load	Moment	H. unit load	Hor. Thrust	V. unit load	Vert. Shear.
21	3,300	+0.958	+ 3,200	1.6504	5,400	+0.6538	+ 2,100
C	3,300	+4.067	+13,400	1.7917	5,900	+0.5000	+ 1,700
2C	3,300	+5.608	+18,500	1.6504	5,400	+0.3462	+ 1,100
4R	3,300	+5.385	+17,800	1.2783	4,200	+0.2103	+ 700
6R	3,300	+3.782	+12,500	0.7919	2,600	+0.1070	+ 400
8L	3,300	+1.884	+ 6,200	0.3608	1,200	+0.0411	+ 100
10R	3,300	+0.463	+ 1,500	0.0540	300	+0.0054	+ 0
			+87,200		25,000		+ 6,100

Summary of Positive Moments and Thrusts at Springing.

	Moments	Thrust	Vert. Shear.
Motor trucks with uniform load on Trafficway	+ 379,900	119,000	26,800
Uniform load on Sidewalk	+ 87,200	25,000	+ 6,100
	+ 467,100	144,000	32,900

Normal Thrust.
144,000 × 0.834 = 120,000
32,900 × 0.558 = 18,400
138,400

Negative Moments and Thrusts for Springing.
Motor trucks with uniform load on Trafficway.



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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MARUNOUCHI, TOKIO

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CHECKED BY DATE PAGE NO 32

CALCULATIONS FOR

Mutsu-Ohashi over Mutsugawa, Hyogoken.

Neg. Moments and Thrusts for Springing continued.

Point	Load	M. unit load	Moment	H. unit load	Hor. Thrust	Shear unit load	Vert. Shear.
12L	5,600	0	-17,000	0		+0.5000	+2,800
10L	3,800	-4.483	-317,600	0.0840	300	+0.9916	+3,800
8L	49,600	-6.404		0.3608	17,900	+0.9589	+47,600
6L		-5.656		0.7919		+0.8930	
4L	14,000	-2.735	-38,300	1.2783	17,900	+0.7897	+11,100
2L	2,400	+9.958	+2,300	1.6504	4,000	+0.6538	+1,600
			-370,600		40,100		+66,900

Unif. Load on Sidewalks.

Point	Load	M. unit load	Moment	H. unit load	Hor. Thrust	Shear unit load	Vert. Shear.
12L	3,300	0		0		+0.5000	1,700
10L	3,300	-4.483	-14,800	0.0840	300	+0.9916	3,300
8L	3,300	-6.404	-21,100	0.3608	1,200	+0.9589	3,200
6L	3,300	-5.656	-18,700	0.7919	2,600	+0.8930	2,900
4L	3,300	-2.735	-9,000	1.2783	4,200	+0.7897	2,600
			-63,600		8,300		13,700

Summary of Negative Moments and Thrusts for Springing.

	Moment	Thrust	Vert. Shear	Hor. Thrust.
Motor trucks and unif. load on trafficway.	-370,600	40,100	+66,900	48,400 × 0.834 = 40,400
Unif. load on Sidewalks.	-63,600	8,300	+13,700	80,600 × 0.558 = 45,000
	-434,200	48,400	+80,600	85,400

Temperature Stresses.

Crown Section.

$$H_0 = \frac{E.w.t.l.\int \frac{ds}{I}}{2\left[\int \frac{ds}{I}\int \frac{y^2 ds}{I} - \left(\int \frac{y\,ds}{I}\right)^2\right]} = \frac{G}{B}$$

$E = 288,000,000$ #/0', $B = 136,492.$
$w = 0.000,0067$, $t = 27°F$
$l = 66.0'$, $E.w.t = 52,100.$

for fall of 27°F

$$H_0 = \frac{-52,100 \times 66 \times 120.101}{136,492.} = -3,028 \text{ \# per 1\# strip.}$$

For 21.68' wide $-3,028 \times 21.68 = -65,600$ #

$$M_0 = -\frac{H_0 \int \frac{y\,ds}{I}}{\int \frac{ds}{I}} = +65,600 \times \frac{223.52}{120.101} = +122,100 \text{ '\#}$$

Panel Point		Moment	Hor. Thrust.
Crown		+122,100 '#	-65,600 #
2	M = 122,100 - 65,600 × 0.26 =	+105,000 '#	-65,600 × 0.996 = -65,300 #
4	M = 122,100 - 65,600 × 1.05 =	+53,200	" × 0.982 = -64,300
6	M = 122,100 - 65,600 × 2.38 =	-33,900	" × 0.960 = -62,900
8	M = 122,100 - 65,600 × 4.26 =	-156,600	" × 0.928 = -60,900
10	M = 122,100 - 65,600 × 6.78 =	-322,900	" × 0.887 = -58,200
Springing	M = 122,100 - 65,600 × 10.00 =	-533,900	" × 0.834 = -54,700

for M = 0, y = 1.865'

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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MADE BY K. Baba DATE 13-9-13 FILE NO
CHECKED BY DATE PAGE NO 33

CALCULATIONS FOR

Mutsu-Chaski over Mukogawa, Hyogo Ken.

Average Stresses.

Coefficients for Average Stresses due to live load for entire span at intermediate panel are only approximate.

Conc. Area (Concrete) $1.17 \times 21.68 = 25.37^{\square'}$ $M_S = 2.5/1.17 = 2.14$

10 lines of Steel area $\frac{1.785 \times 22 \times 2 \times 14}{144} = 3.36$

Equiv. total $= 28.73^{\square'}$ Rise ratio $= 10 \div 66 = 0.1515$

Dead Load Average Stresses.

Point	Normal thrust		Sectional area		Length of Neutral Axis		
Crown	438,000	÷	28.73	×	2.75	=	42,000
2	440,300	÷	29.27	×	5.53	=	83,700
4	445,500	÷	30.46	×	5.62	=	82,300
6	455,300	÷	33.06	×	2.73	=	79,000
8	470,600	÷	38.46	×	2.93	=	72,500
10	494,000	÷	45.06	×	6.20	=	68,000
springing	516,700	÷	57.56	×	3.21	=	28,800
					35.00		456,300 ÷ 35 = 13000 #/□'

Live Load Average Stress.

		Hor. thrust at Crown.			Average Stress.
Panel Point #0	Pos. m.	123,600	÷ 28.73 =	4,300	4,300 × .88 = 3,780 #/□'
	Neg. m.				
Panel Point #2	Pos. m.	112,100	÷ 28.73 =	3,910	3910 × .88 = 3,440 #/□'
	Neg. m.	97,300	÷ 28.73 =	3390	3390 × .92 = 3,120
Panel Point #4	Pos. m.	82,400	÷ 28.73 =	2,870	2,870 × .88 = 2,520
	Neg. m.	128,300	÷ 28.73 =	4,470	4,470 × .92 = 4,110
Panel Point #6	Pos. m.	48,700	÷ 28.73 =	1,690	1,690 × .88 = 1,490
	Neg. m.	153,500	÷ 28.73 =	5,420	5,420 × .92 = 4,980
Panel Point #8	Pos. m.	56,900	÷ 28.73 =	1,980	1,980 × .88 = 1,740
	Neg. m.	125,800	÷ 28.73 =	4,380	4,380 × .92 = 4,030
Panel Point #10	pos. m.	127,000	÷ 28.73 =	4,420	4,420 × .88 = 3,890
	neg. m.	81,700	÷ 28.73 =	2,840	2,840 × .92 = 2,610
Springing	pos. m.	144,000	÷ 28.73 =	5,020	5,020 × .88 = 4,420
	neg. m.	48,400	÷ 28.73 =	1,680	1,680 × .92 = 1,550
Temperature		65,600	÷ 28.73 =	2280	2,280 × .81 = 2,100

Rib Stresses at Crown.

Positive Moment, Compression in upper fibre —

	Thrust	Moment	Average Stress.	
Dead Load	438,000#	+19,200	13,000 #/□'	16,130 ÷ 52,100 = 0.31 $E_{cc} = 0.356$, $\varepsilon/h = \frac{0.356}{1.17} = 0.304$
Live Load	123,600	+135,800	3,780	$d'/h = 0.12/1.17 = 0.103$, $p = \frac{120}{1.17 \times 144} = 0.00714$
Rib Shortening	−20,300	+37,900	−650	From prepared diagram, $K = 0.714$, $L = 0.185$
	541,300	+192,900	16,130	$f_c = \frac{M}{L\,b\,h^2} = \frac{192,900}{0.1185 \times 21.68 \times 1.17^2} = 5480$ #/□' OK.
				$f_s = n f_c \left(\frac{d}{kh} - 1\right) = 15 \times 380 \left(\frac{1.05}{.714 \times 1.17} - 1\right) = 1470$ #/□' OK.
Dead Load	438,000	+19,200	13,000	14,110 ÷ 52,100 = 0.271, $E_{cc} = 0.648$, $\varepsilon/h = \frac{0.648}{1.17} = 0.554$
Live Load	123,600	+135,800	3,780	$d'/h = 0.12/1.17 = 0.103$, $p = 0.00714$, $K = 0.42$, $L = 0.128$
Rib Shortening	−17,800	+32,400	−570	$f_c = \frac{309,500}{0.118 \times 21.68 \times 1.17^2} = 8840$ #/□' = 614 #/□' OK.
Temperature 27°F. fall	−65,600	+122,100	−2,100	$f_s = 15 \times 614 \left(\frac{1.05}{.42 \times 1.17} - 1\right) = 10500$ #/□' OK.
	478,200	+309,500	14,110	

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Minto-Ohashi over Mukogawa, Nyogoten.

Fibre Stresses at Panel Point #6 (Positive Moment)

	Mer. Thrust	Moment	Axial Stress
Dead Load	453000	- 200	13000
Live Load	53,100	+171,100	1490
Rib Shortening	- 16900	- 9100	- 560
	491,500	= 161,800	13930

13930÷52100=0.268, $\frac{E}{E_{cc}}=\frac{161,800}{491,500}=0.33$, $\frac{E}{f_t}=\frac{0.33}{737}=0.241$

$\frac{d}{h}=\frac{12}{137}=.088$, $p=\frac{12}{137\times144}=.00061$, $\chi=0.1109$

$f_c=\frac{161,800}{1.10\times2168\times137^2}=35900\,\#/\square''=256\,\#/\square''$ OK

Dead Load	455,380	200	13000
Live Load	53,100	+171,100	1490
Rib Shortening	- 16,200	- 10,400	- 686
Temperature	- 62,900	- 33,900	- 2,400
27° Rise	559,100	= 127,100	12,350

15930÷52100=0.306, $E_{cc}=\frac{127,100}{559,100}=0.227$, $\frac{E}{f_t}=\frac{0.23}{737}=0.0312$

$\frac{d}{h}=0.088$, $p=.0061$, $\chi=.0115$

$f_c=\frac{127,100}{1.10\times2168\times137^2}=41.5\,\#/\square''=298\,\#/\square''$ OK

Fibre Stresses at Panel Point #8 (Negative Moment)

Dead Load	455,380	200	13000
Live Load	160,200	-148,800	4480
Rib Shortening	19,400	9000	610
Temperature	- 62,900	- 33,900	- 2400
27° Fall	534,200	= 166,500	15,790

15770÷52100=0.2915, $E_{cc}=\frac{166,500}{534,200}=0.288$, $\frac{E}{f_t}=\frac{0.288}{737}=0.0204$

$\frac{d}{h}=0.088$, $p=.0061$, $\chi=.107$

$f_c=\frac{166,500}{1.07\times2168\times137^2}=34,460\,\#/\square''=238\,\#/\square''$ OK

Fibre Stresses at Panel Point #3 (Positive Moment)

	Mer. Thrust	Moment	Axial Stress
Dead Load	470,600	+ 6,600	13000
Live Load	123,900	+147,600	+ 1940
Rib Shortening	- 18,900	- 11,600	- 850
Temperature	- 60,700	- 36,600	- 2700
27° Rise	513,900	+ 105,000	+ 11390

$\frac{14,770}{52,100}=0.282$, $E_{cc}=\frac{91,600}{517,700}=0.177$, $\frac{E}{f_t}=\frac{0.177}{737}=0.24$

$\frac{d}{h}=\frac{12}{162}=.074$, $p=.00054$, $\chi=.080$

$f_c=\frac{91,600}{1.08\times2168\times162^2}=20,400\,\#/\square''=140\,\#/\square''$ OK

Dead Load	470,600	6,600	13000
Live Load	123,900	+147,600	+ 1940
Rib Shortening	- 18,900	- 11,600	- 850
Temperature	- 60,700	- 36,600	- 2700
27° Rise	513,900	+ 105,000	+ 11390

$\frac{14,770}{52,100}=.282$, $E_{cc}=\frac{239,200}{513,900}=.465$, $\frac{E}{f_t}=\frac{0.465}{737}=0.631$

$\frac{d}{h}=.074$, $p=.00540$, $\chi=.1062$

$f_c=\frac{239,200}{1.062\times2168\times162^2}=33,100\,\#/\square''=228\,\#/\square''$ OK

Fibre Stresses at Panel Point #3 (Negative Moment)

Dead Load	470,600	6,600	13000
Live Load	139,200	-106,200	4030
Rib Shortening	- 19,100	- 10,200	- 660
	590,700	= 145,800	16,370

$\frac{16,370}{52,100}=0.314$, $E_{cc}=\frac{145,800}{590,700}=0.252$, $\frac{E}{f_t}=\frac{0.252}{737}=0.341$

$\frac{d}{h}=.074$, $p=.00574$

Dead Load	470,600	6600	13000
Live Load	139,230	-106,200	4030
Rib Shortening	- 16,800	- 9,320	- 580
Temperature	- 60,900	- 33,600	- 2100
27° Fall	522,100	= 260,500	14,350

$\frac{14,350}{52,100}=.278$, $E_{cc}=\frac{260,500}{522,100}=.499$, $\frac{E}{f_t}=\frac{0.49}{737}=.303$

$\frac{d}{h}=.074$, $p=.00574$, $\chi=.1128$

$f_c=\frac{260,500}{1.128\times2168\times162^2}=40,720\,\#/\square''=282\,\#/\square''$ OK

$f_s=15\times282\left(\frac{1.5}{.074\times162}-1\right)=1890\,\#/\square''$ OK

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Mukō-Okaski over Mukōgawa, Hyogoken.

Fibre Stresses at Panel Point #10 (Positive Moment)

	Nor. Thrust.	Moment	Aver. Stress.
Dead Load	494,000	+ 12,000	13,000
Live Load	124,700	+186,600	3,890
Rib Shortening	− 18,200	−100,900	− 650
	600,500	97,700	16,240

$\frac{16240}{52100}=0.312$, $e_{cc}=\frac{97700}{600.500}=0.162$, $e/h=\frac{0.162}{1.97}=0.082$
$d'/h=\frac{12}{1.97}=0.061$, $p=\frac{4.2}{1.97\times144}=0.0042$

Dead Load	494,000	+ 12,000	13,000
Live Load	124,700	+ 186,600	3,890
Rib Shortening	− 20,300	−113,000	− 740
Temperature	58,200	+ 322,900	+ 2,100
27° rise	656,600	+408,500	18,250

$\frac{18250}{52,100}=0.35$, $e_{cc}=\frac{408500}{656600}=0.62$, $e/h=\frac{0.62}{1.97}=0.314$
$d'/h=.061$, $p=.0042$, $K=.6865$, $L=.1106$
$f_c=\frac{400.400}{.1106\times21.68\times1.97^2}=43,000\#/'=300\#/"$ OK.

Fibre Stresses at Panel Point #10 (Negative moment)

	Nor Thrust	Moment	Aver. Stress.
Dead Load	494,000	+ 12,000	13,000
Live Load	104,000	+206,000	2,610
Rib Shortening	− 16,700	− 92,900	− 610
	581,300	+125,100	15,000

$\frac{15000}{52100}=.288$, $e_{cc}=\frac{125100}{581,300}=.215$, $e/h=\frac{.215}{1.97}=0.109$

Dead Load	494,000	+ 12,000	13,000
Live Load	104,000	−206,000	2,610
Rib Shortening	−14,500	− 80,300	− 520
Temperature;	− 58,200	− 322,900	− 2,100
25°F fall.	525,300	−597,200	12,990

$\frac{12990}{52100}=0.249$, $e_{cc}=\frac{597200}{525,300}=1.135$, $e/h=\frac{1.135}{1.97}=0.575$
$d'/h=.061$, $p=.0042$, $K=.59$, $L=.1067$
$f_c=\frac{597200}{0.1067\times21.68\times1.97^2}=66,600\#/'=463\#/"$ OK.
$f_s=15\times463\left(\frac{1.85}{.59\times1.97}-1\right)=4,100\#/"$ OK

Fibre Stresses at Springing (Positive Moment).

	Nor. Thrust	Moment	Aver. Stress
Dead Load	516,700	+11,900	13,000
Live Load	138,400	+467,100	4,420
Rib Shortening	− 17,600	−173,500	− 670
	637,500	+405,500	16,750

$\frac{16750}{52100}=0.322$ $e_{cc}=\frac{405500}{637,500}=0.637$, $e/h=\frac{.637}{2.5}=.255$
$d'/h=.048$, $p=.0033$; $K=.4444$, $L=.1005$
$f_c=\frac{405500}{.1005\times21.68\times2.5^2}=29800\#/'=207\#/"$ OK
$f_s=15\times207\left(\frac{2.38}{.4444\times2.5}-1\right)=3,540\#/"$ OK

Dead Load	516,700	+ 11,900	13,000
Live Load	138,400	+ 467,100	4,420
Rib Shortening	− 19,800	− 193,000	− 760
Temperature;	+ 54,700	+ 533,900	+ 2,100
27° rise	+ 735,000	+ 819,900	18,760

$\frac{18760}{52100}=0.36$, $e_{cc}=\frac{819.900}{735000}=1.15$, $e/h=\frac{1.15}{2.5}=0.460$
$d'/h=0.048$, $p=0.0033$, $K=.4442$, $L=.1006$
$f_c=\frac{819.900}{.1006\times21.68\times2.5^2}=60,200\#/'=418\#/"$ OK.
$f_s=15\times418\left(\frac{2.38}{.442\times2.5}-1\right)=7,280\#/"$ OK

Fibre Stresses at Springing (Negative Moment)

	Nor. Thrust.	Moment	Aver. Stress.
Dead Load	516,700	+ 11,900	13,000
Live Load	85,400	−434,200	1,550
Rib Shortening	− 14,700	−143,500	− 560
	587,400	−565,800	13,990

$\frac{13990}{52100}=0.269$, $e_{cc}=\frac{565800}{587400}=0.965$, $e/h=\frac{.965}{2.5}=.386$
$d'/h=0.048$, $p=0.0033$, $K=0.498$, $L=0.103$
$f_c=\frac{565800}{.103\times21.68\times2.5^2}=40600\#/'=282\#/"$ OK
$f_s=15\times282\left(\frac{2.38}{.498\times2.5}-1\right)=9,600\#/"$ OK

Dead Load	516,700	+ 11,900	13,000
Live Load	85,400	−434,200	1,550
Rib Shortening	− 12,600	−122,600	− 480
Temperature;	− 54,700	− 533,900	− 2,100
27°F fall)	534,800	−1,078,800	11,970

$\frac{11970}{52100}=0.23$, $e_{cc}=\frac{1078800}{534,800}=2.02$, $e/h=\frac{2.02}{2.50}=0.808$
$d'/h=12/2.5=0.048$, $p=0.0033$, $K=0.291$, $L=0.0925$
$f_c=\frac{1,078,800}{0.0925\times21.68\times2.5^2}=86,300\#/'=\underline{600}\#/"$ OK
$f_s=15\times600\left(\frac{2.38}{.291\times2.5}-1\right)=\underline{20,400}\#/"$ OK

Note.— In the above calculations, the assumption for reinforcements was somewhat corrected as follows:—
Assumption for steel in calculation of moment of inertia — 1" ϕ bars 12" c to c for Crown to Panel Pt. #10,
and 6" c to c at Springing on top and bottom.
Above assumptions are corrected in calculating fibre stresses as 7/8" ϕ bars 12" c to c throughout Crown to Springing on top and bottom.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Mutsu-Ohashi over Mutsugawa, Nyugoku-sen.
Stresses of Arch Rib for Electric Railway.

Dead Load Moments at Crown Section.

Pavement, 1" stone-block @ $\frac{160}{12}$ = 54 #/□'
clean mortar filling 5" @ $\frac{110}{12}$ = 46
reinforced conc. slab 6¼ @ $\frac{150}{12}$ = 78
total 178 × 17.9 = 3,180 #
Rails and accessories $\frac{85}{5}$ × 4 × 1.1 (10%) = 125
Steel sleepers etc. Say = 60
longitudinal conc. beam 4 × .83 × .9 × 150 = 408
Part of traffic way
Slab 78 × 2.46 = 192
pavement 22 × 2.46 = 54
fillet concrete .5 × .5 @ 150 = 38
" .1 × .1 × 4 @ 150 = 6
4,103 #/lin. ft. of bridge.
Panel load = 4,103 × 5.5 = 22,600 #

Dead Panel Loads

Panel Point	Wt. of Columns	floor load	wt. of arch-rib	Cross-beam and fill	total load	Remarks.
0 crown	none	11,300	4,700	500	16,500	
1			9,600	1,100	10,700	
2	none	22,600	9,700	800	33,100	
3			9,700		9,700	
4	.2×4@150=100#	22,600	10,200		32,900	
5			10,400		10,400	
6	1.45×4@150=900	22,600	11,100		34,600	
7			11,900		11,900	
8	3.2×4@150=1,900	22,600	13,000	1,800	39,300	
9			14,200		14,200	
10	5.48×4@150=3,300	22,600	15,800		41,700	
11			17,100		17,100	
12 Springing	8.5×4@75=2,600	11,300	10,100		24,000	

Dead Load Moments at Crown Section.

Point	Load	M_o	+M	−M
0 crown	16,500	+2,650	+43,700	
1	10,700	+1,434	+15,300	
2	33,100	+0.529	+17,500	
3	9,700	−0.083		−800
4	32,900	−0.458		−15,100
5	10,400	−0.610		−6,400
6	34,600	−0.606		−21,000
7	11,900	−0.513		−6,100
8	39,300	−0.368		−14,500
9	14,200	−0.222		−3,200
10	41,700	−0.100		−4,200
11	17,100	−0.024		−300
	272,100		+76,500	−71,600 = +4,900 '#
	24,000			
	296,100 #		For Entire span, 2@4,900 =	+9,800 '#

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JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Iraba DATE 13-9-21 FILE NO
CHECKED BY DATE PAGE NO 38

CALCULATIONS FOR

Mutsu-Ohashi over Mutsugawa, Myogotsu.
Dead Load Moments at Various Panel Points.

		Point #1			Point #7			Point #13		
Point	Load	M unit	+M	−M	M unit	+M	−M	M unit	+M	−M
11R	17,100	−0.028		500	−0.028		500	−0.027		500
10	41,700	−0.117		4,900	−0.124		5,200	−0.121		5,000
9	14,200	−0.266		3,800	−0.287		4,100	−0.282		4,000
8	39,300	−0.456		17,900	−0.500		19,700	−0.500		19,600
7	11,900	−0.665		7,900	−0.750		8,900	−0.753		9,300
6	34,600	−0.845		29,200	−0.989		34,200	−1.034		35,800
5	10,400	−0.961		10,000	−1.188		12,400	−1.284		13,300
4	32,900	−0.947		31,200	−1.283		42,300	−1.454		48,000
3	9,700	−0.736		7,100	−1.210		11,800	−1.500		14,500
2	33,100	−0.308		10,200	−0.946		31,300	−1.378		45,600
1R	10,700	+0.396	4,200		−0.431		4,600	−1.038		11,100
0	33,000	+1.400	46,200		−0.366	12,100		−0.445		14,700
1L	10,700	+2.718	29,100		+1.463	15,700		+0.426	4,600	
2	33,100	+1.596	52,800		+2.862	94,800		+1.584	52,500	
3	9,700	+0.778	7,500		+1.816	17,600		+3.040	29,500	
4	32,900	+0.209	6,900		+1.031	33,800		+2.012	66,200	
5	10,400	−0.113		1,200	+0.508	5,300		+1.258	13,100	
6	34,600	−0.257		8,400	+0.184	6,500		+0.732	25,300	
7	11,900	−0.283		3,400	+0.016	200		+0.385	4,600	
8	39,300	−0.230		9,000	−0.048		1,900	+0.178	7,000	
9	14,200	−0.150		2,100	−0.053		800	+0.068	1,000	
10	41,700	−0.071		3,000	−0.032		1,300	+0.017	700	
11L	17,100	−0.018		300	−0.010		200	+0.001	0	
		+146,700 − 150,600 = − 3,900'#			+ 186,000 − 179,200 + 7,800'#			+204,500 −221,400 −16,900'#		

		Point # 4			Point # 5			Point # 6		
Point	Load	M unit	+M	−M	M unit	+M	−M	M unit	+M	−M
11R	17,100	−0.024		400	−0.030		500	−0.008		100
10	41,700	−0.104		4,300	−0.077		3,200	−0.050		2,100
9	14,200	−0.246		3,500	−0.185		2,600	−0.098		1,400
8	39,300	−0.441		17,300	−0.338		13,300	−0.187		7,400
7	11,900	−0.688		8,200	−0.542		6,500	−0.322		3,800
6	34,600	−0.952		32,900	−0.770		26,600	−0.487		16,800
5	10,400	−1.214		12,600	−1.014		10,600	−0.690		7,100
4	32,900	−1.429		47,000	−1.240		40,800	−0.886		29,200
3	9,700	−1.551		15,000	−1.417		13,800	−1.090		10,600
2	33,100	−1.546		51,200	−1.508		49,900	−1.255		41,600
1R	10,700	−1.365		14,600	−1.472		15,800	−1.351		14,500
0	33,000	−0.969		32,000	−1.269		41,900	−1.336		44,100
1L	10,700	−0.329		3,500	−0.864		9,300	−1.171		12,600
2	33,100	+0.570	18900		−0.238		7,900	−0.831		27,500
3	9,700	+1.751	17,000		+0.649	6300		−0.262		2,500
4	32,900	+3.197	105,000		+1.792	59,000		+0.554	18200	
5	10,400	+2.176	22,600		+3.224	33,600		+1.656	17,200	
6	34,600	+1.402	48,500		+2.172	75,200		+3.045	105,400	
7	11,900	+0.844	10,100		+1.372	16,400		+1.974	23,600	
8	39,300	+0.463	18,200		+0.792	31,100		+1.169	46000	
9	14,200	+0.220	3,100		+0.399	5,700		+0.602	8,500	
10	41,700	+0.080	3,300		+0.155	6,500		+0.248	10,300	
11L	17,100	+0.014	200		+0.030	500		+0.048	800	
		+246,900 −242,500 +4,400'#			+234,300 −242,700 −8,400'#			+230,000 −22,300 +8,700'#		

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER.
MARUNOUCHI, TOKIO

MADE BY K. Inaba DATE 13-9-21 FILE NO

CHECKED BY DATE PAGE NO 39

CALCULATIONS FOR

Dead Load Moment at various points continued.

		Point #7			Point #8			Point #9		
Point	Load	M. unit	+M	−M	M. unit	+M	−M	M. unit	+M	−M
11R	17,100	+0.003	100		+0.018	300		+0.035	600	
10	41,700	+0.011	500		+0.073	3,000		+0.149	6,200	
9	14,200	+0.018	300		+0.161	2,300		+0.337	4,800	
8	39,300	−0.014		600	+0.265	10,400		+0.578	22,700	
7	11,900	−0.024		300	+0.354	4,200		+0.825	9,800	
6	34,600	−0.092		3,200	+0.413	14,300		+1.054	36,400	
5	10,400	−0.199		2,100	+0.427	4,400		+1.229	12,800	
4	32,400	−0.353		11,600	+0.361	11,900		+1.291	42,500	
3	9,700	−0.556		5,400	+0.208	2,000		+1.182	11,500	
2	33,100	−0.771		25,500	−0.056		1,900	+0.939	31,100	
1R	10,700	−0.984		10,500	−0.371		4,000	+0.540	5,800	
0	33,000	−1.152		38,100	−0.717		23,600	+0.022	700	
1L	10,700	−1.234		13,200	−1.049		11,200	−0.566		6,100
2	33,100	−1.193		39,500	−1.324		43,800	−1.175		38,900
3	9,700	−0.960		9,400	−1.459		14,200	−1.700		16,500
4	32,400	−0.505		16,600	−1.385		45,600	−2.048		67,400
5	10,400	+0.233	2400		−1.043		10,900	−2.143		22,300
6	34,600	+1.278	44,200		−0.379		13,100	−1.900		64,800
7	11,900	+2.656	31,600		+0.665	7,900		−1.229		14,600
8	39,300	+1.596	62,700		+2.073	81,500		−0.138		5,400
9	14,200	+0.834	11,800		+1.093	15,500		+1.387	19,700	
10	41,700	+0.335	14,000		+0.443	18,500		+0.565	23,600	
11L	17,100	+0.069	1,200		+0.092	1,600		+0.119	2000	
		+168,500 −176,000 −7,200 "#			+177,500 −168,300 +9,500 "#			+230,200 −236,000 −5800 "#		

		Point #10			Point #11			Point #12 Springing.		
Point	Load	M. unit	+M	−M	M. unit	+M	−M	M. unit	+M	−M
11R	17,100	+0.054	900		+0.079	1,400		+0.105	1,800	
10	41,700	+0.234	10,000		+0.343	14,300		+0.463	19,300	
9	14,200	+0.546	7,800		+0.791	11,200		+1.071	15,200	
8	39,300	+0.948	37,100		+1.384	54,400		+1.884	74,000	
7	11,900	+1.388	16,500		+2.052	24,400		+2.817	33,500	
6	34,600	+1.820	62,900		+2.730	94,500		+3.782	130,900	
5	10,400	+2.197	22,800		+3.352	34,800		+4.696	48,800	
4	32,900	+2.436	80,100		+3.790	124,500		+5.385	177,000	
3	9,700	+2.444	23,400		+3.913	38,000		+5.680	55,000	
2	33,100	+2.198	69,500		+3.754	124,200		+5.608	185,800	
1R	10,700	+1.732	18,600		+3.241	34,700		+5.066	54,200	
0	33,000	+1.048	34,600		+2.396	79,100		+4.067	134,100	
1L	10,700	+0.198	2,100		+1.275	13,700		+2.674	28,600	
2	33,100	−0.760		25,200	−0.050		1,700	+0.958	31,700	
3	9,700	−1.706		16,500	−1.443		14000	−0.914		8,900
4	32,900	−2.508		82,500	−2.736		89,900	−2.735		89,800
5	10,400	−3.077		32,000	−3.826		39,800	−4.384		45,600
6	34,600	−3.294		114,000	−4.546		157,200	−5.656		195,800
7	11,900	−3.034		36,100	−4.738		56,300	−6.339		75,400
8	39,300	−2.292		90,000	−4.380		172,000	−6.404		251,500
9	14,200	−1.038		14800	−3.427		48,700	−5.779		82,100
10	41,700	+0.701	29,200		−1.899		79,100	−4.483		186,800
11L	17,100	+0.148	2,500		+0.181	3,100		−2.533		43,300
		+418,000 −411,100 +6,900 "#			+652,300 −658,700 −6,400 "#			+989,900 −979,200 +10,700 "#		

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JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Ikeda. DATE 13/9/21 FILE NO
CHECKED BY DATE PAGE NO 40

CALCULATIONS FOR

Mukō Okashi over Mukōgawa, Hyogoken.
Dead Load Thrusts (For Electric Railway)

Point	Load	Ho unit Load	Ho	Vert Load: Vert Shear	Vert Load: unit Load	Vert Load: Hor. Thrust	H	Hor. Thrust: unit Load	Hor. Thrust: Normal Thr.
0 Crown	16500	1.7917	29,600	0	0		450,400	1.000	450,400
1	10,700	1.7561	18,800	21,900	0.043	900	"	0.999	449,900
2	33,100	1.6504	54,600	43,800	0.091	4,000	"	0.996	448,600
3	9,700	1.4841	14,400	65,200	0.140	9,100	"	0.991	446,300
4	32,900	1.2783	42,000	86,500	0.181	15,700	"	0.982	442,300
5	10,400	1.0391	10,800	108,100	0.230	24,900	"	0.972	437,800
6	34,600	0.7919	27,400	130,600	0.275	35,900	"	0.960	432,400
7	11,900	0.5627	6700	153,900	0.324	49,900	"	0.945	425,600
8	39,300	0.3608	14,300	179,500	0.370	66,400	"	0.928	418,000
9	14,200	0.1993	2,800	206,200	0.417	86,000	"	0.910	409,900
10	41,700	0.0840	3,500	234,200	0.466	109,100	"	0.887	399,500
11	17,100	0.0185	300	263,600	0.509	134,000	"	0.862	388,200
12 Springing	10,100	0	0	282,200	0.558	157,500	"	0.834	375,600
	282,200	arching only	225200						

H for Entire Span = 450,400 #

Summary of Dead Load Normal Thrusts, Moments and Eccentricities.

Points	Normal Thrusts: due to vert. load	Normal Thrusts: due to hor. load	Normal Thrusts: Sum	Moments: Moments	Moments: eccentricities
0 crown	0	450,400	450,400	+ 9,800	+ 0.022'
1	900	449,900	450,800	− 3,900	− 0.009'
2	4000	448,600	452,600	+ 7,800	+ 0.017
3	9,100	446,300	455,400	−16,900	− 0.037
4	15,700	442,300	458,000	+ 4,400	+ 0.010
5	24,900	437,800	462,700	− 8,400	− 0.018
6	35,900	432,400	468,300	+ 8,700	+ 0.019
7	49,900	425,600	475500	− 7,200	− 0.015
8	66,400	418,000	484,400	+ 9,500	+ 0.020
9	86,000	409,900	495,900	− 5,800	− 0.012
10	109,100	399,500	508,600	+ 6900	+ 0.014
11	134,000	388,200	522,200	− 6,400	− 0.012
12 Springing	157,500	375,600	533,100	+ 2,600 ※	+ 0.005

Note:— Reaction acts above of neutral axis for + sign of eccentricity, below —

※ The moment due to weight of arching for springing into consideration.
Moment due to wt. of arching = −10,080 × 0.81 = − 8100 #
Dead Load moment = + 10700
Total moment at springing = + 2,600 #

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（独立行政法人 土木研究所蔵）

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MADE BY K. Inaba DATE 13-9-24 FILE NO
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CALCULATIONS FOR

Mukogawa Bridge over Mukogawa, Hyogo-ken.

Live Load Stresses.

Concentrated Loads.

12 ton Motor truck concentration.

Impact allowance $I = \frac{20}{60+l} = \frac{20}{60+\frac{66}{3.28}} = 25\%$.

12 ton motor truck rear wheel concentration = 9900×1.25 = 12,400#, for 2 lines 49,600# (mark M.T.)
" " " front wheel concentration = 3,300×1.25 = 4,100, " 2 lines 16,400# "

The effects of 14 ton road roller is smaller than that of 12 ton motor truck and it is unnecessary to investigate.

33 ton Electric Railway Truck Concentration (66,000# full loaded)
Concentration of one wheel = 8,250×1.25 = 10,300#, for 2 lines 41,200# (mark R.T.)

Uniform Live Load

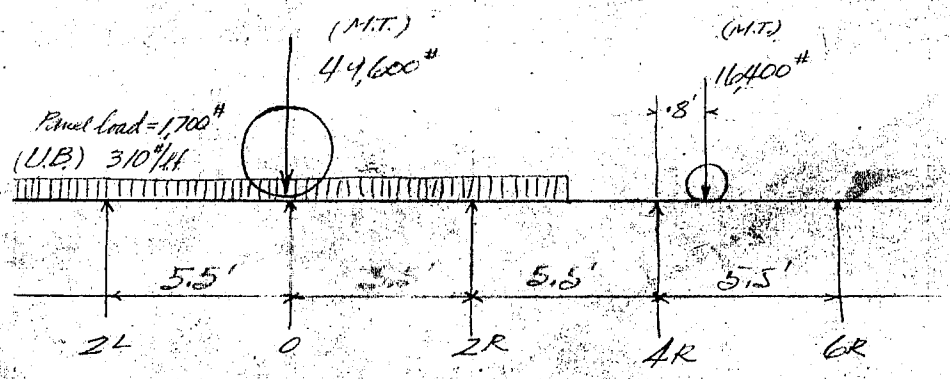
$w = \frac{120,000}{170+l} = \frac{120,000}{170+\frac{66}{3.28}} = 630^{kg}/_{m^2}$ assume $w = 600^{kg}/_{m^2} = 125^{\#}/_{ft}$

Uniform load per lin. ft of electric railway 125×20.35 = 2,550#/ft. Panel load = 2,550×5.5 = 14,000# (U.L)
Unif. load on both sides of trucks 125×2.5 = 310 " " " = 310×5.5 = 1,700# (U.B)

Assumption: 2 electric railway trucks or 2 motor trucks side by side and uniform live load all around them assumed.

Stresses at Crown

Positive Moment. (Max. Moment occurs in the following case which is selected after several investigations)



(M.T.)		49,600			14,000	2,400		
(U.B.)	1,700	1,700	1,700					
	1,700	51,300	1,700		14,000	2,400		

Load (M.T.) and (U.B.)

Point	Load	M. unit load	Moment	H. unit load	Hor. Thrust	V unit load	Vert shear.	Mom. Thrust.
2L	1,700	+ 0.5287	+ 900	1.6504	2,800	−0.3462	− 600	
0	51,300	+ 2.6503	+136,000	1.7917	88,200	+0.5000	+25,700	113,600#
2R	1,700	+ 0.5287	+ 900	1.6504	2,800	+0.3462	+ 600	
4R	14,000	− 0.4582	− 6,400	1.2783	17,900	+0.2103	+ 2,900	
6R	2,400	− 0.6062	− 1,500	0.7919	1,900	+0.1070	+ 300	
			+129,900		113,600		+29,500	

Negative moment at this section is small compared with positive moment and it is unnecessary to investigate.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
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MADE BY K. Iwaba DATE 19-7-21 FILE NO ____
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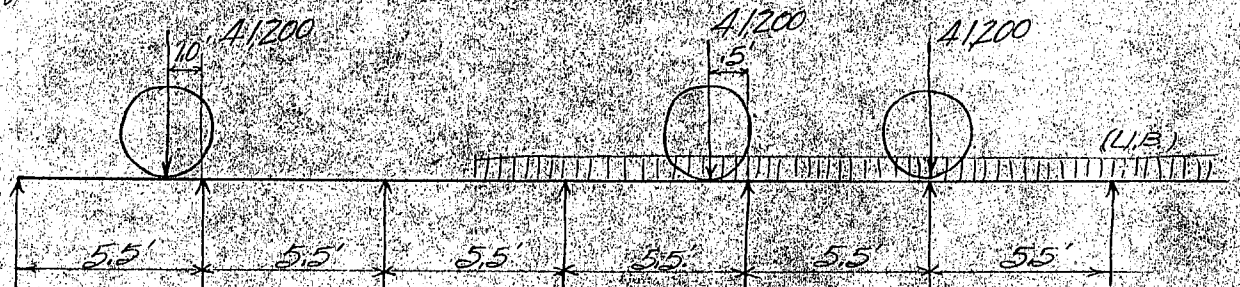
CALCULATIONS FOR

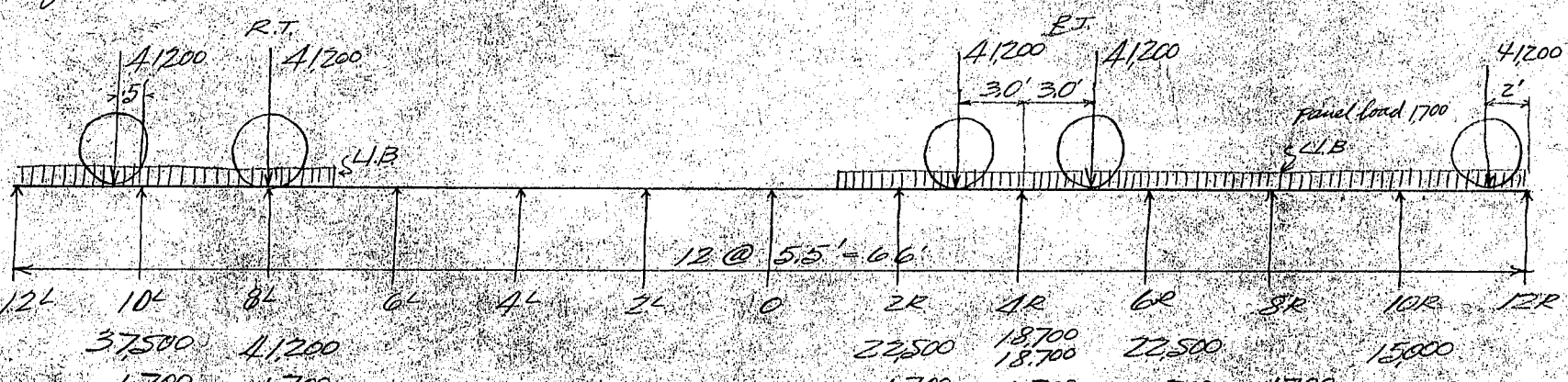
Mukō-Gawa Bridge, Mukogawa, Hyogoken.
Stresses at Panel Point #2. (Electric Ry. Trucks.)
Positive Moment.



(R.T.) 7,500 33,700 3,700 37,500 42,100
(U.B.) 1,700 1,700 1,700
7,500 33,700 5,400 39,200 43,800 1,700

Point	Load	M. unit load	Moment	H. unit load	Hor. thrust	V. unit load	Vert. Shear	Mor. Thrust.
0	1,700	+0.366	+ 600	1.7917	3,100	-0.5000	- 900	
2L	43,800	+2.862	+125,500	1.6504	72,500	+0.6538	+28,600	132,800×0.996=132,100
4L	39,200	+1.031	+40,500	1.2783	50,100	-0.2103	- 8,200	+18,600×0.091=+ 1,700
6L	5,400	+0.189	+ 1,000	0.7919	4,300	-0.1070	- 600	133,800
8L								
10L	33,700	-0.032	- 1,100	0.084	2800	-0.0084	- 300	
			+166,500		132,800		18,600	

Negative Moment at Panel Point #2.



37,500 41,200 22,500 18,700 22,500 15,000
1,700 1,700 1,700 18,700 1,700 1,700 1,700
39,200 42,900 24,200 39,100 24,200 1,700 16,700

Point	Load	M. unit load	Moment	H. unit load	Hor. thrust	V. unit load	Vert. Shear	Mor. Thrust.
10R	16,700	-0.124	- 2,100	0.0846	1,400	+0.0084	+ 100	
8R	1,700	-0.500	- 900	0.3608	600	+0.0411	+ 100	129,900×0.996=129,300
6R	24,200	-0.989	-23,900	0.7919	19,200	+0.1070	+ 2,600	+17,300×0.091= 1,600
4R	39,100	-1.283	-50,200	1.2783	50,000	+0.2103	+ 8,200	130,900
2R	24,200	-0.946	-22,900	1.6504	39,900	+0.3462	+ 8,400	
8L	42,900	-0.048	- 2,100	0.3608	15,500	-0.0411	- 1,800	
10L	39,200	-0.032	- 1,300	0.0840	3,300	-0.0084	- 300	
			-103,400		129,900		+17,300	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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MADE BY K. Inaba. DATE 12-9-22 FILE NO
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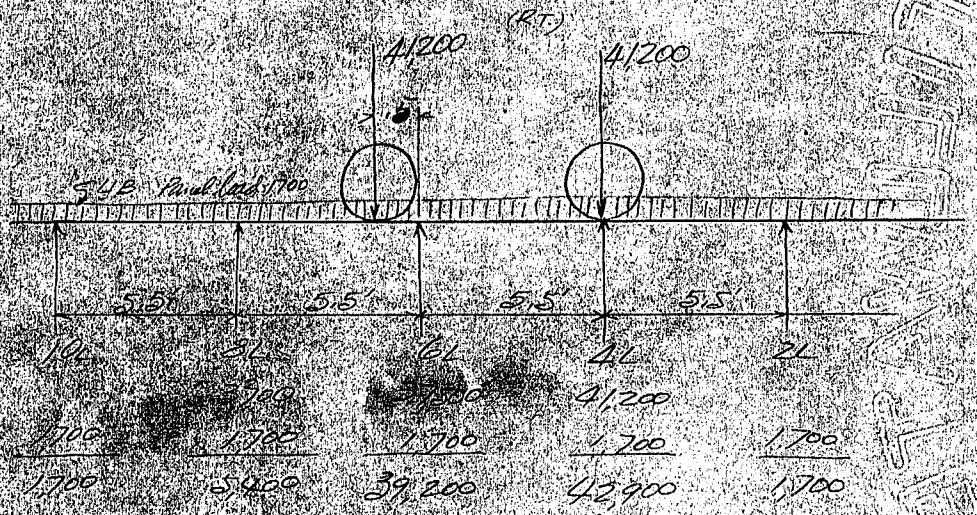
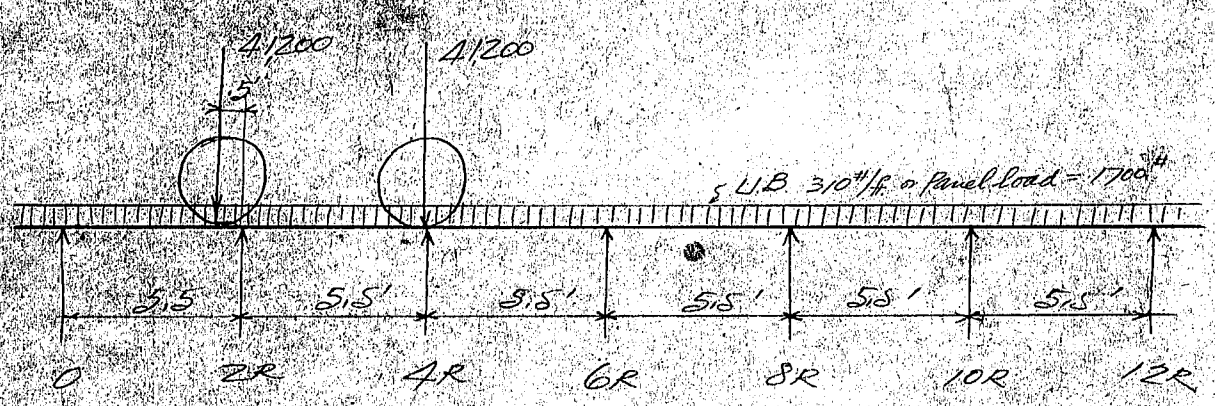
CALCULATIONS FOR

Mukō-Ohashi over Mukōgawa, Hyōgoken.
Stream at Panel Point #4
Positive Moment.



Point	Load	M. unit load	Moment	H. unit load	Hor. thrust	V. unit load	Vert. shear	Net. thrust.
2L	1,700	+0.570	+1,000	1,6504	2,800	+0.6538	+1,100	
4L	42,900	+3.197	+137,100	1,2783	54,900	+0.3949 / −0.105 =	+17,000 / −4,500	90,900×0.982 = 89,200
6L	39,200	+1.402	+55,000	0.7919	31,200	−0.1070	−4,200	+9,200×0.181 = 1,700
8L	5,400	+0.463	+2,500	0.3608	1,900	−0.0411	−200	90,900
10L	1,700	+0.080	+100	0.0840	100	−0.0084	0	
			+195,700		90,900		+9,200	

Negative Moment at Panel Point #4



Point	Load	M. unit load	Moment	H. unit load	Hor. thrust	V. unit load	Vert. shear	Net. thrust.
0	5400	−0.969	−5,200	1,7917	9,700	+0.5000	2,700	
2R	39,200	−1.546	−60,600	1,6504	64,900	+0.3462	13,600	131,400×0.982 = 129,000
4R	42,900	−1.429	−61,300	1,2783	54,800	+0.2103	9,000	25,600×0.181 = 4,600
6R	1,700	−0.952	−1,600	0.7919	1,300	+0.1070	200	133,600
8R	1,700	−0.441	−800	0.3608	600	+0.0411	100	
10R	1,700	−0.104	−200	0.0840	100	+0.0081	0	
			−129,700		131,400		+25,600	

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（独立行政法人 土木研究所蔵）

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CONSULTING ENGINEER
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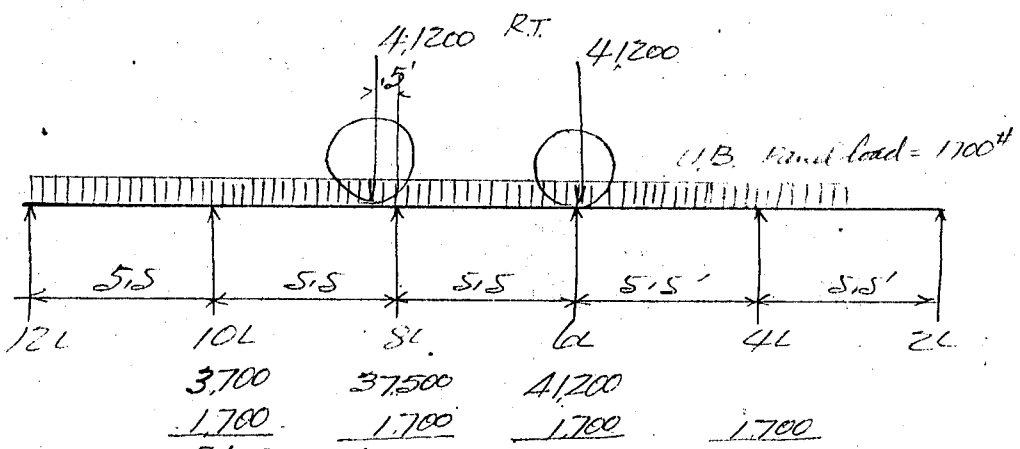
MADE BY K. Inaba DATE 13-9-22 FILE NO
CHECKED BY DATE PAGE NO 44

CALCULATIONS FOR

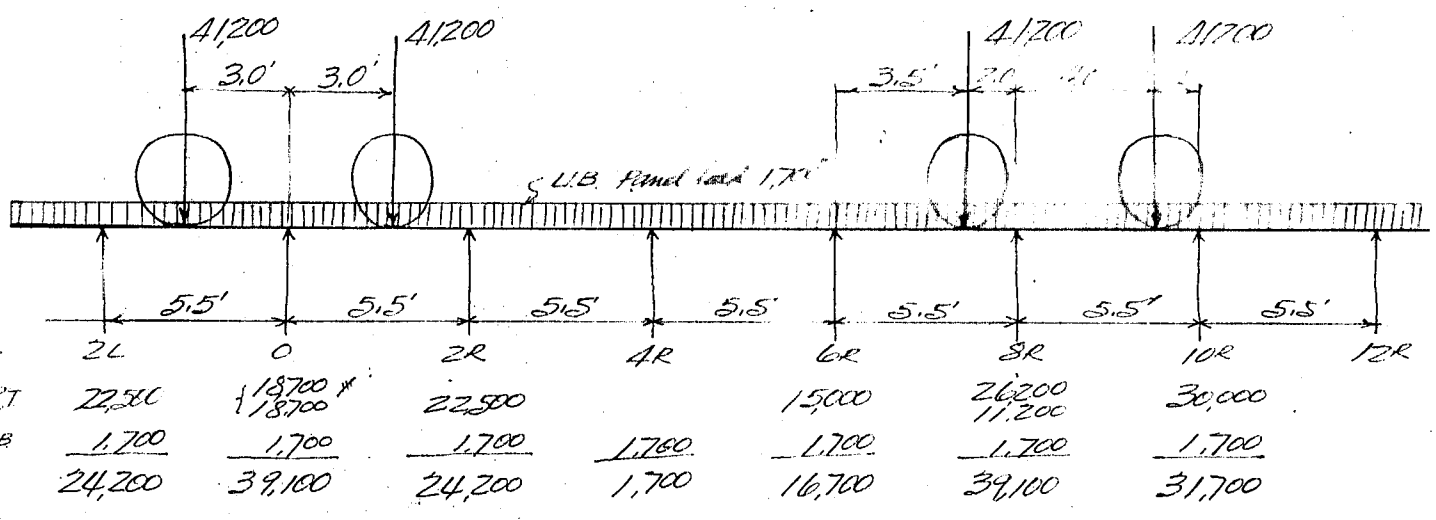
Mukko-Ohashi over Mukogawa, Nyogo-ken.
Stresses at Panel Point #6. (Electric Ry)
Positive Moment.



3,700 37,500 41,200
1,700 1,700 1,700 1,700
5,400 39,200 42,900 1,700

Point	Load	M.unitload	Moment	H.unitload	Hor.Thrust.	V.unitload	Vert.Shear	Hor. Thrust.
2L	1,700							
4L	42,900	+0.554	+ 900	1.2783	2,200	+ 0.7897	-1,300	178,800×0.960=171,600
6L	39,200	+3.045	+130,800	0.7919	33,900	+ 0.4470 / - 0.0535	+19,200 / -2,300	+16,600×0.275= 4,600
8L		+1.169	+ 45,800	0.3608	14,100	- 0.0411	- 1,600	176,200
10L	5,400	+0.248	+ 1,300	0.0840	500	- 0.0084	0	
			+178,800		50,700		+16,600	

Negative Moments at Panel Point #6.



RT	22,500	18,700 / 18,700	22,500		15,000	26,200 / 11,200	30,000
UB	1,700	1,700	1,700	1,700	1,700	1,700	1,700
	24,200	39,100	24,200	1,700	16,700	39,100	31,700

Point	Load	M.unitload	Moment	H.unitload	Hor.Thrust	V.unitload	Vert.Shear	Hor. Thrust.
2L	24,200	-0.831	-29,100	1.6504	40,000	+0.6538	15,800	
0	39,100	-1.336	-52,300	1.7917	70,200	+0.5000	19,600	182,400×0.960=175,000#
2R	24,200	-1.225	-29,700	1.6504	40,000	+0.3462	8,400	47,900×0.275= 13,200
4R	1,700	-0.886	-1,500	1.2783	2,200	+0.2103	400	188,200
6R	16,700	-0.487	- 8,100	0.7919	13,200	+0.1070	1,800	
8R	39,100	-0.187	- 7,300	0.3608	14,100	+0.0411	1,600	
10R	31,700	-0.050	- 1,600	0.084	2,700	+0.0084	300	
			-120,600		182,400		+47,900	

Note: Concentrated loads between panel points 2L to 2R are transmitted directly to the arch rib by the crown fill, but, for the simplicity of calculations, they are assumed to be transmitted as panel loads on points 2L, crown and 2R and the results is approximately correct.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Inaba. DATE 13-9-22/33 FILE NO
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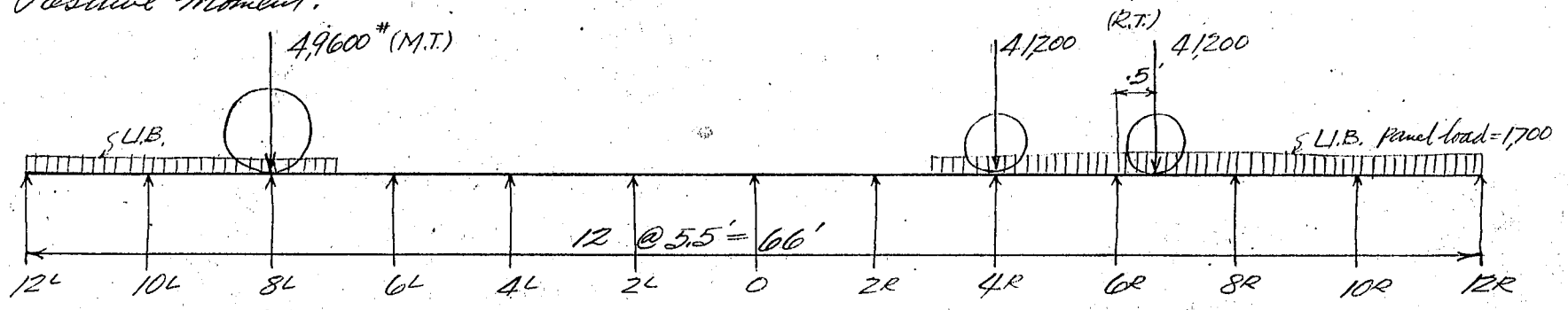
CALCULATIONS FOR

Mutsu-Ohashi over Mutsugawa, Myogo-ken.
Stresses at Panel Point #8. (Electric Ry.)
Positive Moment.



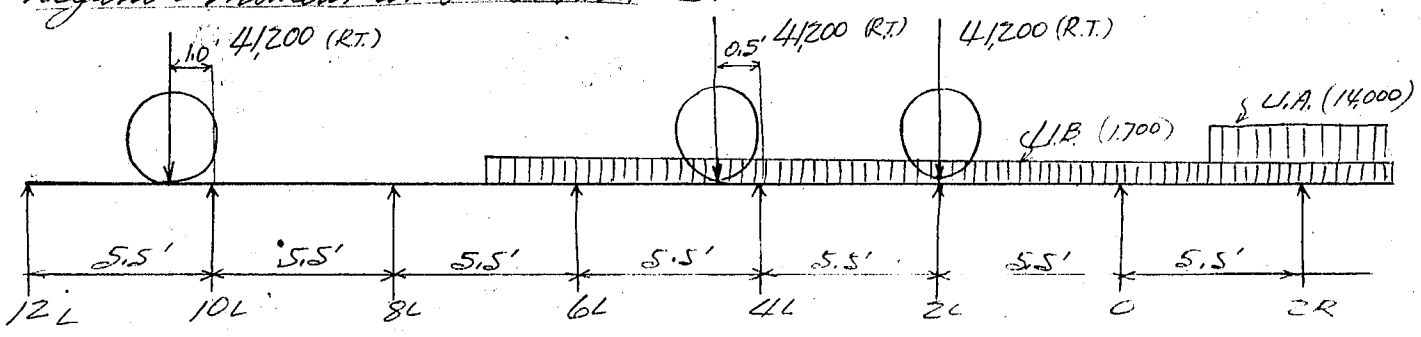
12L	10L	8L	6L	4L	2L	0	2R	4R	6R	8R	10R	12R
		49,600						41,200	37,500	3,700		
	1,700	1,700						1,700	*1,700	1,700	1,700	
	1,700	51,300						42,900	39,200	5,400	1,700	

Point	Load	M. Unit-load	Moment	H. unit-load	Hor. Thrust	V. unit-load	Vert. Shear
10R	1,700	+0.073	+ 100	0.0840	100	+0.0084	0
8R	5,400	+0.265	+ 1,400	0.3608	2,000	+0.0411	+ 200
6R	39,200	+0.413	+16,200	0.7919	31,100	+0.1070	+ 4,200
4R	42,900	+0.361	+15,500	1.2783	54,800	+0.2103	+ 9,000
8L	51,300	+2.073	+106,300	0.3608	18,500	±0.4845	+24,000
						±0.0206	− 1,100
10L	1,700	+0.443	+ 800	0.0840	100	−0.0084	0
			+140,300		106600		+37,100

Nor. Thrust.

106,600×0.928 = 99000
37,100×0.370 = 13700
112,700

Negative Moment at Panel Point #8.



12L	10L	8L	6L	4L	2L	0	2R
33,700		3,700	37,500	41,200			14,000
		1,700	1,700	1,700	1,700	1,700	1,700
33,700		5,400	39,200	42,900	1,700		15,700

Point	Load	M. Unit-load	Moment	H. unit-load	Hor. Thrust.	V. unit-load	Vert. Shear
2R	15,700	−0.056	− 900	1.6504	25,900	+0.3462	+ 5,400
0	1,700	−0.717	− 1,200	1.7917	3,000	+0.3000	+ 900
2L	42,900	−1.324	−56,800	1.6504	70,900	+0.6538	+28,100
4L	39,200	−1.385	−54,300	1.2783	50,200	+0.7897	+30,900
6L	5,400	−0.379	− 2000	0.7919	4,300	+0.8930	+ 4,800
8L							
10L	33,700	+0.443	+15,000	0.0840	2,800	−0.0084	− 300
			−100,200		157,100		+69,800

Nor. Thrust.

157,100×0.928 = 145,900
69,800×0.370 = 25800
171,700

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（独立行政法人 土木研究所蔵）

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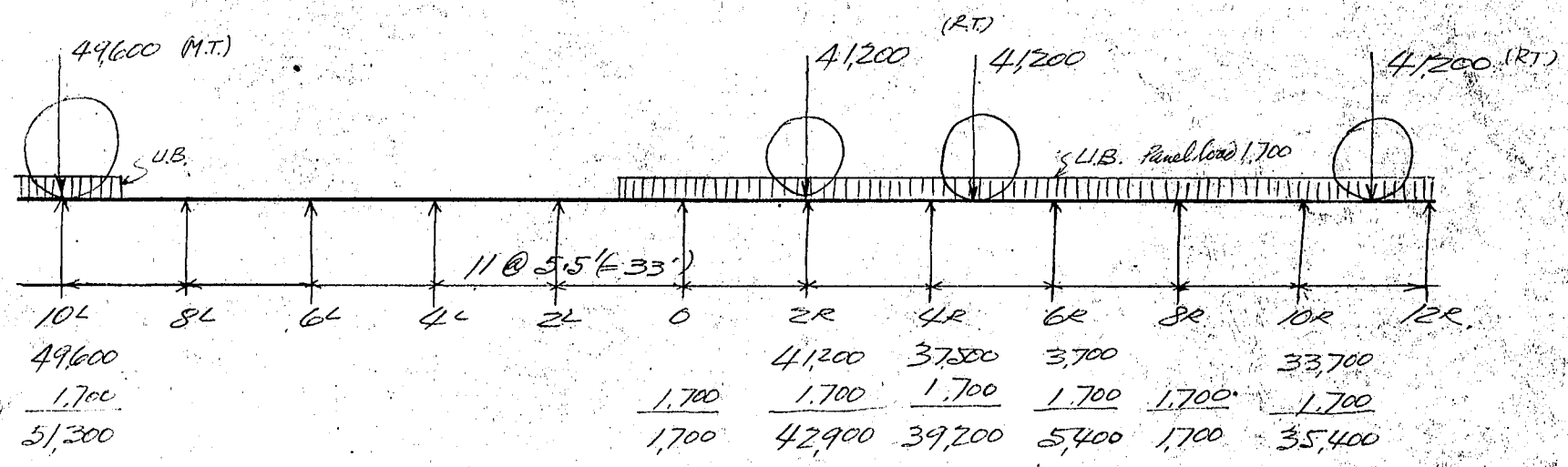
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Inaba DATE 13-9-23 FILE NO ____
CHECKED BY ____ DATE ____ PAGE NO 46

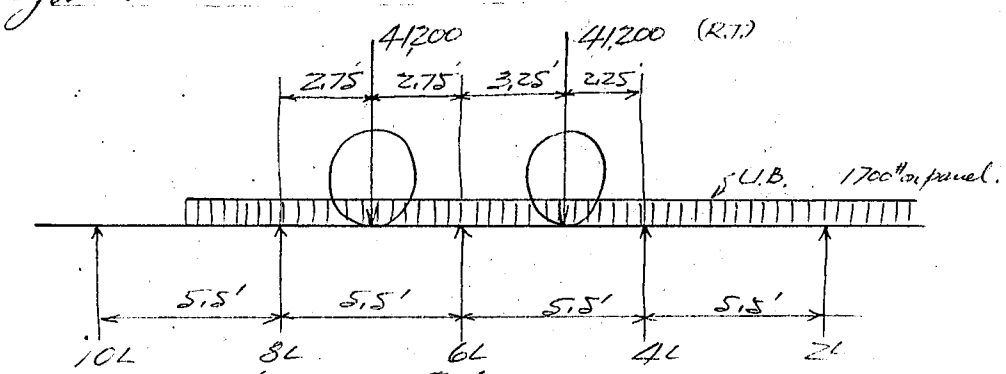
CALCULATIONS FOR

Mutsu-Ohashi over Mutsugawa, Kyogoku.
Stresses at Panel Point #10.
Positive Moments.



Point	Load	M. unit load	Moment	H. unit load	Hor. Thrust	V. unit load	Vert. Shear	Hor. Thrust
10R	35,400	+0.239	+ 8,500	0.0840	3,000	+0.0084	+ 300	
8R	1,700	+0.948	+ 1,600	0.3608	600	+0.0411	+ 100	136200×0.887=120,800
6R	5,400	+1.820	+9,800	0.7919	4,300	+0.1070	+ 600	50,300×0.466= 23400
4R	34,200	+2.436	+85,500	1.2783	50,100	+0.2103	+8,200	144,200
2R	42,900	+2.198	+94,300	1.6504	70,800	+0.3462	+14,900	
0	1,700	+1.048	+ 1,800	1.7917	3,100	+0.5000	+ 900	
10R	51,300	+0.701	+36,000	0.0840	4,300	{+0.4958 −0.0042	{+25500 − 200	
			+247,500		136,200		+50,300	

Negative Moment at Panel Point #10.



(R.T.)		20,600	20600 16,900	24,300				
(U.B.)		1,700	1,700	1,700	1,700			
		22,300	39,200	26,000	1,700			

Point	Load	M. unit load	Moment	H. unit load	Hor. Thrust	V. unit load	Vert. Shear	Hor. Thrust
2ℓ	1,700	−0.760	− 1,300	1.6504	2,800	+0.6538	+1,100	
4ℓ	26,000	−2.508	−65,300	1.2783	33,200	+0.6897	+17,900	75,000×.887=66,500
6ℓ	39,200	−3.294	−129,100	0.7919	31,000	+0.8930	+35,000	75,400×.466=35,100
8ℓ	22,300	−2.292	− 51,200	0.3608	8000	+0.9589	+21,400	101,600
			−246,900		75,000		+25,400	

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（独立行政法人 土木研究所蔵）

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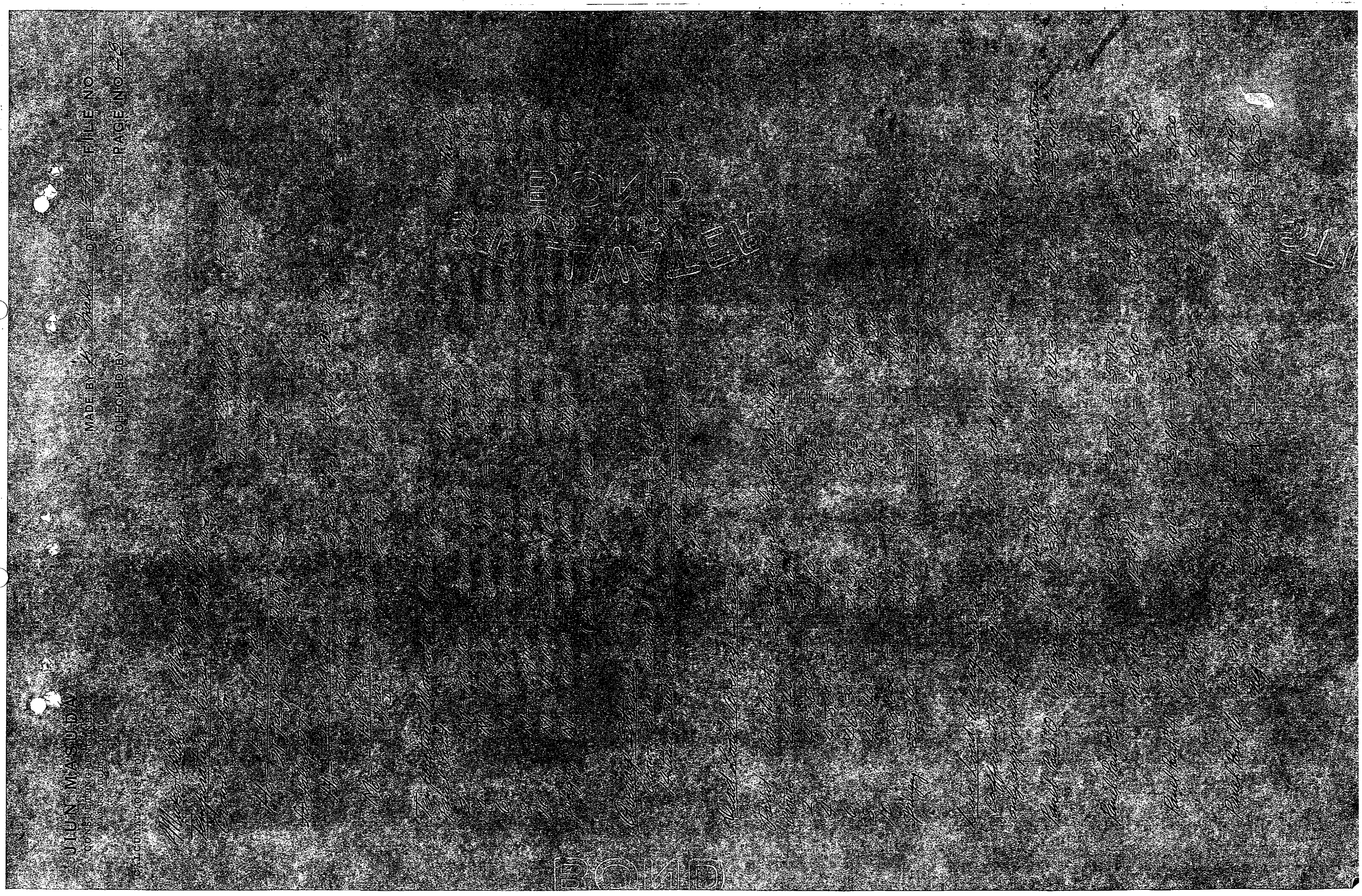

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MADE BY H. Iwata DATE 8/1933 FILE NO.
CHECKED BY DATE PAGE NO. 49

CALCULATIONS FOR

Web Stress at Panel Point No. 2
Concrete Stress Neglected

		Shear	Moment	Average Stress
Panel Pt. 2	Dead Load	126,600 ÷ 25.69	= 4,150 ℔	4,150 × 0.88 = 3650 ℔
	Live Load	157,100 ÷ 25.69	= 6120	6120 × 0.88 = 5,630
Panel Pt. 16	Dead Load	136,200 ÷ 25.69	= 5310	5310 × 0.88 = 4680
	Live Load	75,058 ÷ 25.69	= 2920	2,920 × 0.92 = 2,690
Spring 16	Dead Load	156,200 ÷ 25.69	= 6090	6060 × 0.88 = 5330
	Live Load	34,700 ÷ 25.69	= 1,350	1,350 × 0.92 = 1,240
Temperature		55,600 ÷ 25.69	= 2,280	2280 × 0.81 = 2,100

Web Stress at Panel Pt. #2
Positive moment (Compression in Upper Tube)

	Shear	Moment	Average Stress
Dead Load	+ 452,600	+ 7800	+ 15030
Live Load	+ 133,800	+ 166,500	+ 4,550
Rib Shortening	− 21,200	+ 30,000	− 760
	+ 565,200	+ 208,300	+ 18820

Web Stress at Panel Pt. #2
Negative moment

	Shear	Moment	Average Stress
Dead Load	+ 452,600	+ 7800	+ 15030
Live Load	+ 133,800	+ 166,500	+ 4,550
Rib Shortening	− 18,900	+ 30,300	− 680
Temperature	− 58,400	+ 93,900	− 2,100
27° Fall	+ 509,100	+ 298,500	+ 16,800

Web Stress at Panel Pt. #2

	Shear	Moment	Average Stress
Dead Load	+ 452,600	+ 7800	+ 15030
Live Load	+ 130,100	− 103,400	+ 3,060
Rib Shortening	− 21,600	+ 34,600	− 780
	+ 561,100	− 60,800	+ 17,310

Web Stress at Panel Pt. #2

	Shear	Moment	Average Stress
Dead Load	+ 452,600	+ 7800	+ 15030
Live Load	+ 130,400	− 103,400	+ 3,060
Rib Shortening	− 20,300	+ 33,500	− 860
Temperature	− 58,400	+ 93,900	− 2,100
27° Rise	+ 504,300	− 68,200	+ 15,130

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MADE BY K. Inaba DATE 13-9-24 FILE NO
CHECKED BY DATE PAGE NO 50

CALCULATIONS FOR

Mukto Ohashi over Mukogawa, Hyogo Ken.

Stresses at Panel Pt. #4.

Positive Moment.

	Thrust	Moment	aver. Stress
Dead Load	+ 458,000	+ 4,400	+ 15,030
Live Load	+ 90,900	+ 195,700	+ 3,120
Rib Shortening	− 19,300	+ 16,000	− 700
	+529,600	+216,100	+ 17,450

$\frac{17450}{52100}=0.335$, Ecc.=0.41, $e/h=\frac{0.41}{1.25}=0.328$

$d'/h=12/125=0.096$, $p=\frac{12}{125\times144}=.0067$, $K=.667$, $L=.1115$

$f_c=\frac{216,100}{.1115\times19.35\times1.25}=64,200\#/0'=446\#/0"$

$f_s=15\times446\left(\frac{1.05}{.667\times.117}-1\right)=2,310\#/0"$ Ok.

Dead Load	+ 458,000	+ 4,400	+ 15,030
Live Load	+ 90,900	+ 195,700	+ 3,120
Rib Shortening	− 17,000	+ 14,100	− 620
Temperature	*−57,500	+ 47,600	− 2,100
27° fall	+474,000	+261,800	+15,430

$\frac{154800}{52100}=.296$, Ecc=.552, $e/h=\frac{.552}{1.25}=.443$

$d'/h=.096$, $p=.0067$, $K=.512$, $L=.0999$

$f_c=\frac{261,800}{.0999\times19.35\times1.25}=86,500\#/0'=600\#/0"$ Ok.

$f_s=15\times600\left(\frac{1.05}{.512\times1.125}-1\right)=6,880\#/0"$ Ok.

Negative Moment at Panel Point #4.

	Nor. Thrust	Moment	average Stress.
Dead Load	+ 458,000	+ 4,400	+ 15,030
Live Load	+ 133,600	− 129,700	+ 4,710
Rib Shortening	− 20,900	+ 17,300	− 770
	+570,700	−108,000	+18,970

$\frac{18,970}{52,100}=.364$, Ecc=.189, $e/h=\frac{.189}{1.25}=.151$

$d'/h=.096$, $p=.0067$, $K=$ $L=$

Dead Load	+ 458,000	+ 4,400	+ 15,030
Live Load	+ 133,600	− 129,700	+ 4,710
Rib Shortening	− 23,200	+ 19,200	− 850
Temperature	+ 57,500	− 47,600	+ 2,100
27° rise	+685,900	−153,700	+ 20,990

$\frac{20,990}{52,100}=.403$, Ecc.=.246 $e/h=\frac{.246}{1.25}=.197$

$d'/h=.096$, $p=.0067$, $K=$ $L=$

Stresses at Panel Pt. #6

Positive moment.

	Thrust	Moment	average Stress.
Dead Load	468,300	+ 8,700	+ 15,030
Live Load	176,200	+ 178,800	+ 1,790
Rib Shortening	− 16,000	− 8,600	− 650
	+628,500	+178,900	+16,170

$\frac{1563}{52100}\frac{16170}{}=.\frac{31}{284}$ Ecc=.304 $e/h=\frac{.304}{1.37}=.222$

$d'/h=\frac{12}{1.37}=.0875$, $p=\frac{12}{1.37\times144}=.0061$, $K=.933$ $L=0.1037$

$f_c=\frac{178,900}{0.1037\times19.35\times1.37^2}=47,400\#/0'=329\#/0"$ Ok

$f_s=15\times329\left(\frac{1.25}{.933\times1.37}-1\right)=$ Negative.

Dead Load	+468,300	+ 8,700	+ 15,030
Live Load	+176,200	+ 178,800	+ 1,790
Rib Shortening	− 19,700	− 10,600	− 730
Temperature	+ 56,300	+ 30,400	+ 2,100
27° rise	+681,100	+207,300	+18,190

$\frac{18190}{52100}=.349$, Ecc=.304, $e/h=\frac{.304}{1.37}=.222$

$d'/h=.0875$, $p=.0061$, $K=.891$, $L=.1099$

$f_c=\frac{207,300}{.1099\times19.35\times1.37^2}=52,000\#/0'=361\#/0"$ Ok.

$f_s=15\times361-\left(\frac{1.25}{.891\times1.37}-1\right)=135\#/0"$ Ok.

Negative Moment at Panel Pt. #6.

	Thrust	Moment	average stress.
Dead Load	468,300	+ 8,700	+ 15,030
Live Load	188,200	− 120,600	+ 6530
Rib Shortening	− 22,400	− 12,100	− 840
	+634,100	−124,000	+ 20,720

$\frac{20720}{52100}=.398$, Ecc=.196 $e/h=\frac{.196}{1.37}=.143$

$d'/h=.0875$, $p=.0061$, $K=$ $L=$

Dead Load	+ 468300	+ 8,700	+ 15,030
Live Load	+ 188,200	− 120,600	+ 6530
Rib Shortening	− 20,200	− 10,900	− 750
Temp. 27° fall	− 56,300	− 30,400	− 2,100
	+580,000	−153,200	+18,710

$\frac{18,710}{52,100}=.359$, Ecc=.264, $e/h=\frac{.264}{1.37}=.193$

$d'/h=.0875$, $p=.0061$, $K=$ $L=$

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Ueda DATE 12-7-25 FILE NO
CHECKED BY DATE PAGE NO 51

CALCULATIONS FOR

Mitsu-Chashi, over carriageway, Hyogo-ken.
Steven at Panel Point #5
Positive Moment

	Moment	Moment	Average Stress	
Dead Load	+ 484,400	+ 9,500	+ 15030	17960/52100 = .345; Ec = .176; s/n = 176/1.62 = .108
Live Load	+ 112,700	+ 160,300	+ 3650	
Pet Shortening	- 18,800	- 4838.00	- 720	
	+ 578,300	101,300	+ 17960	
Dead Load	+ 484,400	+ 9500	+ 15030	17970/52100 = .345; Ec = .175; s/n = .231
Live Load	+ 112,700	+ 160,300	+ 3650	
Pet Shortening	- 20800	- 5338.00	- 810	
Temperature	+ 54,400	+ 140,500	+ 2100	
27°-Fall	+ 630,700	+ 236,800	+ 14,970	

Negative Moment at Panel Point #5

	Moment	Moment	Average Stress	
Dead Load	+ 484,400	+ 9500	+ 15030	17960/52100 = .345; Ec = .176; s/n = .166
Live Load	+ 171,700	- 180,200	+ 5630	
Pet Shortening	- 20,700	- 5338.00	- 800	
	+ 535,400	185,200	+ 19860	
Dead Load	+ 484,400	+ 9500	+ 15030	17860/52100 = .342; Ec = .168; s/n = .216
Live Load	+ 191,700	- 180,200	+ 5630	
Pet Shortening	- 18,600	- 5338.00	- 720	
Temperature	- 54,400	- 140,500	- 2100	
27°-Fall	+ 583,100	- 334,800	+ 17860	

Steven at Panel Point #7
Positive Moment

	Moment	Moment	Ave. Stress	
Dead Load	+ 388,600	+ 6,900	+ 15030	19970/52100 = .383; Ec = .235; s/n = .231
Live Load	+ 142,200	+ 247,500	+ 4680	
Pet Shortening	- 20,900	- 4838.00	- 850	
Temperature	+ 52,000	+ 288,200	+ 2100	
27°-Fall	+ 683,900	+ 426,800	+ 20,960	

Negative Moment at Panel Point #7

	Moment	Moment	Ave. Stress	
Dead Load	+ 388,600	+ 6,900	+ 15030	20960/52100 = .402; Ec = .235; s/n = .317
Live Load	+ 142,200	+ 247,500	+ 4680	d/n = .061; p = .00424; k = .677; L = 1107
Pet Shortening	- 20,900	- 4838.00	- 850	$f_c = \frac{426800}{1107 \times 1935 \times 197^2} = 31500$ #/□" $f_s = 357$ #/□" ok.
Temperature	+ 52,000	+ 288,200	+ 2100	
27°-Fall	+ 683,900	+ 426,800	+ 20,960	$f_s = 15 \times 35.7 \times \left(\frac{1685}{627 \times 197} - 1\right) = 2090$ #/□" ok.

Negative Moment at Panel Point #10

	Moment	Moment	Ave. Stress	
Dead Load	+ 388,600	+ 6,900	+ 15030	17030/52100 = .327; Ec = .164; s/n = .286
Live Load	+ 101,600	- 246,900	+ 2690	d/n = 1/1.97 = .061; p = .00424; k = .735; L = 1101
Pet Shortening	- 17,000	- 4838.00	- 690	$f_c = \frac{334300}{1101 \times 1935 \times 197^2} = 40,800$ #/□" $= 281$ #/□" ok.
	+ 573,200	- 334,800	+ 17030	$f_s = 281 \times 15 \left(\frac{1685}{735 \times 197} - 1\right) = 1480$

Positive Moment at Panel Point #10

	Moment	Moment	Ave. Stress	
Dead Load	+ 388,600	+ 6,900	+ 15030	15010/52100 = .288; Ec = .145; s/n = .172
Live Load	+ 101,600	- 246,900	+ 2690	d/n = .061; p = .00424; k = .792; L = 1036
Pet Shortening	- 15,000	- 4838.00	- 610	$f_c = \frac{611300}{1036 \times 1935 \times 197^2} = 7860$ #/□" $= 546$ #/□" ok.
Temperature	- 52,000	- 288,200	- 2100	
27°-Fall	+ 484,200	- 311,300	+ 15010	$f_s = 15 \times 45.46 \times \left(\frac{1685}{792 \times 197} - 1\right) = 1450$ #/□" ok.

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CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Mabe DATE 13-425 13-426 FILE NO
CHECKED BY DATE PAGE NO 52

CALCULATIONS FOR

Mutsu Chuo-ku, Mutsugawa, Aomori-ken.
Stresses at Springing.
Positive Moment

	Thrust	Moment	average Shear
Dead Load	+ 533,100	+ 2600	+ 15,030
Live Load	+ 149,300	+ 511,400	+ 5330
Rib Shortening	— 18,700	— 178,900	— 790
	+ 663,700	+ 335,100	+ 19,570

$\frac{19570}{52100} = .375$, $E_c = \frac{335100}{663700} = .505$, $E/h = .202$
$d'/h = .048$, $p = .0033$, $k = .939$, $L = .0976$.
$f_c = \frac{335,100}{.0976 \times 19.35 \times 2.5^2} = 28400$ #/0' = 197 #/0" Ok.
$f_s = 15 \times 197 \left(\frac{2.38}{.939 \times 2.5} - 1\right) = 45$ #/0"

Dead Load	+ 533,100	+ 2,600	+ 15,030
Live Load	+ 149,300	+ 511,400	+ 5,330
Rib Shortening	— 20,600	— 200,500	— 870
Temperature	+ 48,900	+ 476,900	+ 2,100
27° fall	+ 710,700	+ 790,400	+ 21,590

$\frac{21590}{52100} = .421$, $E_c = \frac{790400}{710,700} = 1.11$, $E/h = 0.445$
$d'/h = .048$, $p = .0033$, $k = .452$, $L = .101$
$f_c = \frac{790400}{.101 \times 19.35 \times 2.5^2} = 64800$ #/0' = 450 #/0" Ok.
$f_s = 15 \times 450 \left(\frac{2.38}{.452 \times 2.5} - 1\right) = 7500$ #/0" Ok.

Negative Moment at Springing.

	Thrust	Moment	average Shear
Dead Load	+ 533100	+ 2600	+ 15030
Live Load	+ 76300	— 523800	+ 1240
Rib Shortening	— 14,700	— 143,000	— 630
	+ 594,700	— 664,200	+ 15,640

$\frac{15640}{52100} = .300$, $E_c = \frac{664200}{594,700} = 1.22$, $E/h = \frac{1.22}{2.5} = .149$
$d'/h = .048$, $p = .0033$, $k = .416$, $L = .0942$
$f_c = \frac{664,200}{.0942 \times 19.35 \times 2.5^2} = 58,300$ #/0' = 405 #/0" Ok.
$f_s = 15 \times 405 \left(\frac{2.38}{.416 \times 2.5} - 1\right) = 7840$ #/0" Ok.

Dead Load	+ 533100	+ 2600	+ 15030
Live Load	+ 76,300	— 523,800	+ 1,240
Rib Shortening	— 12,800	— 125000	— 550
Temperature	— 48,900	— 476900	— 2,100
27° fall	+ 547,700	— 1,123,100	+ 13,620

$\frac{13620}{52100} = .262$, $E_c = \frac{1123,100}{547,700} = 2.05$, $E/h = \frac{2.05}{2.5} = 0.82$
$d'/h = .048$, $p = .0033$, $k = .300$, $L = .0935$
$f_c = \frac{1123,100}{.0935 \times 19.35 \times 2.5^2} = 99200$ #/0' = 687 #/0" Ok.
$f_s = 15 \times 687 \left(\frac{2.38}{.300 \times 2.5} - 1\right) = 22,350$ #/0"

Use 20 — 7/8" ϕ bars for 19.35' rib.
$f_s = 22,350 \times \frac{19.35}{20} = 21600$ #/0" Ok.

Unit Shear $v = \frac{84,900 \times 0.833}{19.35 \times 12 \times .9 \times 2.38 \times 12} = 11.9$ #/0" Ok.

Unit bond $u = \frac{84,900 \times .833}{2.75 \times 20 \times 0.9 \times 2.38 \times 12} = 50$ #/0" Ok.

If the reinforcement at Springing were use 1"ϕ at 1.0' c to c on top and bottom.
for D.L. + L.L + R.S

$p = \frac{.7854 \times 2}{2.5 \times 144} = .00436$, $d'/h = .048$, $E/h = .149$, $k = .435$, $L = .108$
$f_c = \frac{664,200}{.108 \times 19.35 \times 2.5^2} = 50800$ #/0' = 353 #/0" Ok.
$f_s = 15 \times 353 \left(\frac{2.38}{.435 \times 2.5} - 1\right) = 5,800$ #/0" Ok.

for D.L + L.L + R.S + T.C.
$p = .00436$, $d'/h = .048$, $E/h = .82$, $k = .326$, $L = .1045$
$f_c = \frac{1123,100}{.1045 \times 19.35 \times 2.5^2} = 89000$ #/0' = 618 #/0" Ok.
$f_s = 15 \times 618 \times \left(\frac{2.38}{.326 \times 2.5} - 1\right) = 17,800$ #/0" Ok.

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MARUNOUCHI, TOKIO

MADE BY K. Inaba DATE 13-9-26 FILE NO
CHECKED BY DATE PAGE NO 53

CALCULATIONS FOR

Mukoohashi over Mukogawa Ryogoku.
Erection Stresses of Arch Ring.
Motor Trafficway.
Moments.

Point	Load	Crown		Point #2		Point #4		Point #6		Point #8		Point 10		Springing	
12R	11,200	0													
11	19,700	-.024	- 500	-.028	-600	-.024	- .500	-.008	- 200	+.018	+ 400	+.054	+ 1,100	+.105	+ 2,100
10	17,600	-.100	- 1,800	-.124	-2,200	-.0104	- 200	-.005	- 100	+.073	+ 1,300	+.239	+ 4,200	+.463	+ 8,100
9	15,800	-.222	-3,500	-.287	-4,500	-.246	-3,900	-.098	- 1,600	+.161	+.2,500	+.546	+ 8,600	+1.071	+17,000
8	14,500	-.368	-5,300	-.500	-7,300	-.441	-6,400	-.187	- 2,700	+.265	+ 3,800	+.948	+13,800	+1.884	+27,300
7	13,200	-.513	-6,800	-.750	-9,900	-.688	-9100	-.322	- 4,300	+.354	+4,700	+1.388	+18,300	+2.817	+37,200
6	12,300	-.606	-7,500	-.989	-12,200	-.952	-11,700	-.487	-6,000	+.413	+5,100	+1.820	+22,400	+3.782	+46,500
5	11,500	-.610	-7,000	-1.188	-13,700	-1.214	-14000	-.680	- 7,800	+.427	+4,900	+2.197	+25,100	+4696	+54,000
4	11,300	-.458	-5,200	-1.283	-14,500	-1.429	-16,100	-.886	10,100	+.361	+4,100	+2436	+27,500	+5.385	+60,800
3	10,800	-.083	- 900	-1.210	-13,200	-1.551	-16,800	-1.090	-11,800	+.208	+2,200	+2.414	+26,000	+5.680	+61,300
2	10,700	+.529	-5,700	-.946	-10,100	-1.546	-16,600	-1.255	-13,400	+.056	- 600	+2.198	+23,500	+5.608	+60,100
1R	10,600	+1.434	+15,200	-.431	-4600	-1.365	-14,500	-1.351	-14,300	-.371	-3,900	+1.732	+18,400	+5.066	+53,800
0	10,400	+2.650	+27,500	+.366	+3,800	-.969	-10,100	-1.336	-13,900	-.717	-7,500	+1.048	+10,900	+4.067	+42,300
1L	10,600	+1.434	+15200	+1.463	+15500	-.329	- 3,500	-1.171	-12,400	-1.049	-11,100	+0.198	+ 2,100	+2.674	+28400
2	10,700	+.529	+5,700	+2.862	+30,700	+.057	+ 600	-.831	- 8900	-1.324	-14,200	-0.760	-8,100	+0.958	+10,300
3	10,800	-.083	- 900	+1.816	+19,600	+1.751	+18,900	-.262	-2,800	-1.454	-15,800	-1.706	-18,400	-0.914	-9,900
4	11,300	-.458	-5,200	+1.031	+11,700	+3.197	+36,100	+.554	+6,300	-1.385	-15,700	-2.508	-27,400	-2.735	-30,900
5	11,500	-.610	-7,000	+.508	+5,900	+2.176	+25,000	+1.656	+19,100	-1.043	-12,000	-3.077	-35,400	-4.334	-50,500
6	12,300	-.606	-7,500	+.189	+2,300	+1.402	+17,200	+3.045	+37,400	-0.379	-4,700	-3.294	-40,500	-5.656	-69600
7	13,200	-.513	-6,800	+.016	+ 200	+.844	+11,100	+1.974	+26,100	+.665	+8,800	-3.034	-40,100	-6.339	-83600
8	14,500	-.368	-5,300	-.048	- 700	+.463	+6,700	+1.169	+17000	+2.073	+31,000	-2.292	-33,300	-6.404	-92900
9	15,800	-.222	-3,500	-.053	- 800	+.220	+3,500	+.602	+9500	+1.093	+17300	-1.038	-16,400	-5.779	-91,300
10	17,600	-.100	-1,800	-.032	-600	+.080	+1,400	+.248	+4,400	+0.443	+7,800	+0.701	+12,300	-4.483	-79000
11	19,700	-.024	- 500	-.010	-200	+.014	+ 300	+.048	+ 900	+0.092	+1800	+0.148	+ 2900	-2.533	-49900
12L	11,200	0													
+ Moment			+69,300		+39,700		+120,800		+120,700		+95,700		+217,100		+509,200
- Moment			- 77,000		-95,100		-123,400		- 110,300		-85,500		- 219,600		-557,600
Total Moment			- 7,700		- 5,400		- 2,600		+ 10,400		+10,200		- 2,500		- 48,400

Horizontal Thrust due to Arch Ring.

Point	Load	H unit load	Hor. Thrust.	Vert. load	Coeff.	Hor. Thrust.	Hor. Thrust.		coefficient.		Hor. Thrust.
0.cr	5,200	1.7918	9,300	0	0	0	230,800	×	1,000	=	230,800
1	10,600	1.7561	18,700	10,500	.043	500	230,800	×	.999	=	230,600
2	10,700	1.6504	17,700	21,200	.091	1,900	"	×	.996	=	229,900
3	10,800	1.4841	16,000	31,900	.140	4,500	"	×	.991	=	228,800
4	11,300	1.2783	14,500	43,000	.181	7,800	"	×	.982	=	226,700
5	11,500	1.0391	11,900	54,400	.230	12,500	"	×	.972	=	224,200
6	12,300	0.7919	9,700	66,300	.275	18,200	"	×	.960	=	221,500
7	13,200	0.5627	7,300	79,000	.324	25,600	"	×	.945	=	218,100
8	14,500	0.3608	5,200	92,900	.370	34,400	"	×	.928	=	214,100
9	15,800	0.1993	3,200	108,000	.417	45,000	"	×	.910	=	210,000
10	17,600	0.084	1,500	124,700	.466	58,200	"	×	.887	=	204,800
11	19,700	0.0185	400	143,400	.509	73,000	"	×	.862	=	199,000
12L	11,200	0	0	164,400	.558	91,700	230,800	×	.834	=	192,500
			115,400								

H = Σ @ 115,400 = 230,800#

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MADE BY K. Iruba DATE 13-7-26 FILE NO
CHECKED BY DATE PAGE NO 54

CALCULATIONS FOR

Muko-Okashi over Mukogawa, Hyogoken.
Summary of Erection Stresses due to Weight of Arch Ring.

Point.	Normal Thrust: due to vert. load	due to Hor. load	Sum	Moment	Eccentricity	
0 cr.	0	230,800	230,800#	−7,700#	−0.033'	
1	500	230,600	231,300			
2	1,900	229,900	231,800	−5,400	−0.023	
3	4,500	228,800	233,300			
4	7,800	226,700	234,500	−2,600	−0.011	
5	12,500	224,200	236,700			
6	18,200	221,500	239,700	+10,400	+0.043	
7	25,600	218,100	243,700			
8	34,400	214,100	248,500	+10,200	+0.041	
9	45,000	210,000	255,000			
10	58,200	204,800	263,000	−2,500	−0.010	
11	73,000	199,000	272,000			
12 Sp.	91,700	192,500	284,200	−48,400	−0.170	

Average Stress 230,800 ÷ 28.73° = 8040 #/ft² 8040 × 0.91 = 7,320 #/ft²

Fibre Stress at Crown.

	Nor. Thrust	Moment	Average Stress
Dead Load	+230,800	−7,700	+7,320
Rib Shortening	−8,900	+16,500	−280
	+221,900	+8,800	+7,040
Dead Load	+230,800	−7,700	+7,320
Rib Shortening	−11,400	+21,300	−360
Temperature 27° rise	+65,600	−122,100	+2,100
	+285,000	−108,500	+9,060

7040/52100 = .135, Ecc = .04, $\varepsilon/h = \frac{.04}{1.17} = .034$

9060/52100 = .174, Ecc = .381, $\varepsilon/h = \frac{.381}{1.17} = .325$
$d/h = 1/1.17 = .103$, $p = .00714$, $K = .1675$, $L = .1182$
$f_c = \frac{108500}{.1182 \times 21.68 \times 1.17^2} = 30,900$ #/ft² = 215 #/in². OK.
$f_s = 15 \times 215 \left(\frac{.105}{.1675 \times 1.17} - 1\right) =$ 1,070 #/in². OK.

Fibre Stress at Springing.

	Nor. Thrust	Moment	Average Stress
Dead Load	+284,200	−48,400	7,320
Rib Shortening	−5,200	−51,200	−220
Temperature 27° fall	−54,700	−533,900	−2,100
	+224,300	−633,500	+5,000

5000/52100 = .096, Ecc = 2.82, $\varepsilon/h = 1.128$
$d/h = .1048$, $p = .0033$, $K = .259$, $L = .0926$
$f_c = \frac{633,500}{.0926 \times 21.68 \times 2.5^2} = 50,500$ #/ft² = 350 #/in². OK.
$f_s = 15 \times 350 \left(\frac{2.38}{.259 \times 2.5} - 1\right) =$ 14,100 #/in². OK.

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MADE BY K. Ikeda DATE 13-7-26 / 13-4-27 FILE NO ____
CHECKED BY ____ DATE ____ PAGE NO 55

CALCULATIONS FOR

Muko-Ohashi over Matsugawa, Gyoda-machi.
Erection & Stress during Construction.
Dead Load.
No pavement and no handrail.

Sidewalks
Slab	333.0
Coping	120.5
Bracket	44.0
Fascia Girder	173.0
	671 #/ft. of span.

Motor traffic way.
Slab	1,352.0
Beam B, C & D	354.0
" A	120.0
	1,826 #/ft. of span.
Forms say	300 #/ "
Total =	2,800 #/ft. of span.

Panel Load = 2800 × 5.5 = 15,400 #

Panel Load on Arch Ring.

Panel Pt.	wt. of Column	floor load.	Crown fill and Cross beam	Total.
0 Cr.	—	7,700	1,600	9,300
1			3,600	3,600
2	—	15,400	2,100	17,500
3				
4	400	15,400		15,800
5				
6	1,300	15,400		16,700
7				
8	2,600	15,400	1,500	19,500
9				
10	4,300	15,400		19,700
11				
12 Sp.	3,600	7,700		11,300

Stresses at Springing (Negative moment controls)
4 Panels of one end poured.

Point	Load	M unit load	Moment	H. unit load	Hor. Thrust	V. unit load	Vert Shear	Nor Thrust.
10c	19,700	−4.483	−88,400	.0840	1,700	+.9916	+19,600	32,000×.836=26,700#
8c	19,500	−6.404	−125,000	.3608	7,000	+.9589	+18,700	59,500×.558=33,200
6c	16,700	−5.656	−94,500	.7919	13,200	+.8930	+14,900	59,900
4c	7,900	−2.735	−21,600	1.2783	10,100	+.7897	+6,300	Aver. Stress = $\frac{32000}{28.73}$ × .92
			−329,500		32,000		+59,500	= 1,030 #/□'

Stresses at Panel Point #10 (Neg. moment controls)
4 Panels of one end being poured.

Point	Load	M unit load	moment	H unit load	Hor. Thrust	V unit load	Vert shear	Nor thrust.
10c	19,700	+0.701	+13,800	.0840	1,700	−.0420 / +.4958	−800 / +9,800	32,000×.887=28,400
8c	19,500	−2.292	−44,700	.3608	7,000	+.9589	+18,700	48,900×.466=22,800
6c	16,700	−3.294	−55,000	.7919	13,200	+.8930	+14,900	51,200
4c	7,900	−2.508	−19,800	1.2783	10,100	+.7897	+6,300	aver. stress = $\frac{32000}{28.73}$ × .92
			−105,700		32,000		+48,900	= 1,030 #/□'

Stresses at Crown (Pos. moment controls).
2 panels at centre being poured.

2c	17,500	+.5287	+9,300	1.6504	28,900	+.6538	11,400	Nor. Thrust.
0	9,300	+2.6503	+24,600	1.7917	16,700	+.5000	4,700	74,500×1 = 74,500
2R	17,500	+.5287	+9,300	1.6504	28,900	+.3462	6,100	22,200×0 = 0
			+43,200		74,500		22,200	74,500

Average Stress = $\frac{74,500}{28.73}$ × .88
= 2,280 #/□'

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MADE BY K. Iwata DATE 13-9-27 FILE NO
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CALCULATIONS FOR

Mutsu-Ohashi over Mutsugawa, Hyogo-Ken.
Fibre Stresses of Arch Ring during Construction Continued.
Springing.

	Nor. Thrust	Moment	average Stress
Wt. Arch rib	+ 284,200	- 48,400	+ 7,320
Wt. floor	+ 59,900	-329,500	+ 1,030
Rib Shortening	- 8,400	- 82,400	- 320
	+ 335,700	-460,300	+ 8,030

$\frac{8030}{52100} = .154$, $Ecc = 1.38$, $\varepsilon/h = \frac{1.38}{2.5} = .552$
$d'/h = .048$, $p = .0033$, $k = .425$, $L = .100$
$f_c = \frac{460,300}{.100 \times 21.68 \times 2.5^2} - 34000\#/0' = \underline{236\#/0".0K.}$
$f_s = 15 \times 236 \left(\frac{2.38}{.425 \times 2.5} - 1\right) = \underline{4380\#/0".0K.}$

Wt. Arch rib	+ 284,200	- 48,400	+ 7,320
wt. floor	+ 59,900	-329,500	+ 1,030
Rib Shortening	- 6,300	- 61,500	- 240
Temperature 27° Fall	- 54,700	-533,900	- 2,100
	+283,100	-973,300	+ 6,010

$\frac{6010}{52,100} = .115$, $Ecc = 3.44$, $\varepsilon/h = \frac{3.44}{2.5} = 1.37$
$d'/h = .048$, $p = .0033$, $k = .275$, $L = .932$
$f_c = \frac{973,300}{.932 \times 21.68 \times 2.5^2} = 77,100\#/0' = 538\#/0".0K.$
$f_s = 15 \times 538 \left(\frac{2.38}{.275 \times 2.5} - 1\right) = \underline{19,700\#/0".0K.}$

Fibre Stress at Panel Pt #10.

	Nor. Thrust	Moment	Aver. Stress.
Wt. arch rib.	+ 263,000	- 2,500	+ 7,320
wt. Floor	+ 51,200	-105,700	+ 1,030
Rib Shortening	- 9,000	- 51,300	- 320
	+305,200	-159,500	+ 8,030

$8030/52100 = .154$, $Ecc = .523$, $\varepsilon/h = \frac{.523}{1.97} = .28$
$d'/h = .061$, $p = .00424$, $k = .751$, $L = 0.1099$
$f_c = \frac{159,500}{.1099 \times 21.68 \times 1.97^2} = 17,250\#/0' = 120\#/0".0K.$
$f_s = 15 \times 120 \left(\frac{1.85}{.751 \times 1.97} - 1\right) = 450\#/0".0K.$

Wt. arch rib	+ 263,000	- 2,500	+ 7320
wt. Floor	+ 51,200	-105,700	+ 1,030
Rib Shortening	- 6,700	- 37,100	- 240
Temperature 27° Fall	- 58,200	-322,900	- 2,100
	+249,300	-468,200	+ 6,010

$6,010/52100 = .115$, $Ecc = 1.88$, $\varepsilon/h = \frac{1.88}{1.97} = .955$
$d'/h = .061$, $p = .00424$, $k = .292$, $L = .1005$
$f_c = \frac{468,200}{.1005 \times 21.68 \times 1.97^2} = 55,400\#/0' = \underline{384\#/0".0K.}$
$f_s = .15 \times 384 \left(\frac{1.85}{1.97 \times .292} - 1\right) = \underline{12,780\#/0".0K.}$

Fibre Stress at Crown.

	Nor. Thrust	Moment	Average Stress
Wt. arch rib	+ 230,800	- 7,700	+ 7,320
Wt. floor	+ 74,500	+ 43,200	+ 2,280
Rib shortening	- 11,600	+ 21,600	- 370
	+293,700	+ 57,100	+ 9,230

$9230/52100 = .177$, $Ecc = .195$, $\varepsilon/h = \frac{.195}{1.17} = .167$
$d'/h = 1.2/1.17 = .1025$, $p = .00714$, $k =$ $L =$
$f_c =$ very Small
$f_s =$ " "

Wt. Arch rib	+ 230,800	- 7,700	+ 7,320
Wt. Floor	+ 74,500	+ 43,200	+ 2,280
Rib Shortening	- 9,100	+ 16,800	- 290
Temperature 27° Fall	- 65,600	+ 122,100	- 2100
	+230,600	+174,400	+ 7,210

$7210/52100 = .138$, $Ecc = .757$, $\varepsilon/h = .647$
$d'/h = .1025$, $p = .00714$, $k = .409$, $L = .1158$
$f_c = \frac{174,400}{.1158 \times 21.68 \times 1.17^2} = 50,800\#/0' = \underline{352\#/0".0K.}$
$f_s = 15 \times 352 \times \left(\frac{1.05}{.409 \times 1.17} - 1\right) = \underline{6,300\#/0".0K.}$

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CALCULATIONS FOR

Mukō-Okachi over Mukōgawa, Hyogoken.
Deflection of Arch Ring due to Moment. (Motor traffic way)

The vertical deflection of any point is given by the general equations

$$\Delta y = \int \frac{M.x.ds}{E.I.}$$

Where M is the true bending moment at any section due to the given loads. The abecissa x is to be measured from the point where the deflection is desired.

Deflection at Crown.

$$\Delta y = \int \frac{M.x.ds}{E.I.}$$

$$M_R = M_0 + H_0 y - V_0 x$$
$$M_L = M_0 + H_0 y + V_0 x - m$$
$$M_R \frac{x ds}{I} = M_0 \frac{x ds}{I} + H_0 xy \frac{ds}{I} - V_0 \frac{x^2 ds}{I}$$
$$M_L \frac{x ds}{I} = M_0 x \frac{ds}{I} + H_0 xy \frac{ds}{I} + V_0 \frac{x^2 ds}{I} - mx \frac{ds}{I}.$$
$$\int_0^{1/2} M_R \frac{x ds}{I} = M_0 \int \frac{x ds}{I} + H_0 \int xy \frac{ds}{I} - V_0 \frac{x^2 ds}{I}$$
$$\int_0^{1/2} M_L \frac{x ds}{I} = M_0 \int \frac{x ds}{I} + H_0 \int xy \frac{ds}{I} + V_0 \int \frac{x^2 ds}{I} - \int mx \frac{ds}{I}$$
$$\int_0^{l} Mx \frac{ds}{I} = 2M_0 \int \frac{x ds}{I} + 2H_0 \int xy \frac{ds}{I} - \int mx \frac{ds}{I}.$$

$$\Delta y = \frac{1}{E}\left[2M_0 \int \frac{x ds}{I} + 2H_0 \int \frac{xy ds}{I} - \int mx \frac{ds}{I}\right]$$

Crown deflection due to dead load.

Summation for one half span.

$$\int \frac{x ds}{I} = 1{,}437.56 \;,\; \int \frac{xy ds}{I} = 4{,}706.431.$$

Point	M_0	H_0	$2M_0\int\frac{x ds}{I}$	$2H_0\int\frac{xy ds}{I}$	$-\int\frac{mx ds}{I}$	Sum.	Load	Product.
0 cr.	+2.650	1.7917	+7,619	16,868	−25,278	−791	15,500	−12,260,000
1	+1.434	1.7561	+4,123	16,529	−21,341	−689	14,200	−9,780,000
2	+0.539	1.6504	+1,521	15,531	−17,504	−452	30,100	−13,610,000
3	−0.083	1.4841	−239	13,969	−13,909	−179	10,800	−1,930,000
4	−0.458	1.2783	−1,317	12,030	−10,632	+81	29,000	+2,350,000
5	−0.610	1.0391	−1,754	9,780	−7,789	+237	14,500	+2,730,000
6	−0.606	0.7919	−1,742	7,455	−5,406	+307	30,900	+9,490,000
7	−0.513	0.5627	−1,475	5,300	−3,518	+307	13,200	+4,050,000
8	−0.368	0.3608	−1,058	3,398	−2,079	+261	35,900	+9,370,000
9	−0.222	0.1993	−638	1,873	−1,073	+162	15,800	+2,560,000
10	−0.100	0.0840	−288	791	−425	+78	39,200	+3,060,000
11	−0.024	0.0185	−69	174	−85	+20	19,700	+390,000
12 Sp.	0	0	0					−37,580,000
								+34,000,000
								−3,580,000

$$\text{Deflection of Crown} = \frac{-3{,}580{,}000}{288{,}000{,}000 \times 2{,}168} = -0.000574' = -\underline{0.00688}\text{" Fall.}$$

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CALCULATIONS FOR

Mutsu-Okachi over Mukogawa, Ogogoken.

Deflection due to temperature change:

Deflection at crown due to temperature change is obtained by the following formula.

$$\Delta y = \frac{E}{}\left[2M_0 \int \frac{x\,ds}{I} + 2H_0 \int \frac{xy\,ds}{I} - \int mx \frac{ds}{I} \right] \quad \text{------ (A)}$$

where $H_0 = \dfrac{E w t l \int \frac{ds}{I}}{2\left[\int \frac{ds}{I} \int \frac{y^2 ds}{I} - \left(\int \frac{y\,ds}{I}\right)^2\right]}$

$$M_0 = -\frac{2H_0 \int \frac{y\,ds}{I}}{\int \frac{ds}{I}} = -\frac{2E w t l \int \frac{ds}{I} \int \frac{y\,ds}{I}}{2\left[\int \frac{ds}{I} \int \frac{y^2 ds}{I} - \left(\int \frac{y\,ds}{I}\right)^2\right] \int \frac{ds}{I}}$$

$$\int \frac{mx\,ds}{I} = 0$$

Substituting these values to equation (A), we have the following result.

$$\Delta y = \frac{2E w t l}{E} \cdot \frac{\left(\int \frac{ds}{I} \int \frac{xy\,ds}{I} - \int \frac{x\,ds}{I} \int \frac{y\,ds}{I}\right)}{2\left[\int \frac{ds}{I} \int \frac{y^2 ds}{I} - \left(\int \frac{y\,ds}{I}\right)^2\right]}$$

For Fall of temperature $\Delta y = -\dfrac{2 w t l \left(\int \frac{ds}{I} \int \frac{xy\,ds}{I} - \int \frac{x\,ds}{I} \int \frac{y\,ds}{I}\right)}{2\left[\int \frac{ds}{I} \int \frac{y^2 ds}{I} - \left(\int \frac{y\,ds}{I}\right)^2\right]}$

Where $w = 0.000,0067$, $t = 27°F$ fall, $l = 66'$

$2\left[\int \frac{ds}{I} \int \frac{y^2 ds}{I} - \left(\int \frac{y\,ds}{I}\right)^2\right] = B = 136,492$

$\int \frac{ds}{I} \int \frac{xy\,ds}{I} - \int \frac{x\,ds}{I} \int \frac{y\,ds}{I} = A$ at Crown $= 244,547$

$A : B = 1.7917$ (Same as H_0 at crown always.)

Deflection of crown for 27° fall of temperature.

$\Delta y = -2 \times 0.000,0067 \times 27 \times 66 \times 1.7917 = -0.0428' = -0.513''$ Fall.

Deflection ~~due to~~ of Crown due to Rib Shortening -

Average Stress due to dead load = 13,000 #/'

Rib Shortening	=	− 420
Temp. 27° fall	=	− 2100
		+10,480 #/'

Ratio $\dfrac{\text{average Stress}}{E.w.t} = \dfrac{10480}{52100} = 0.21$

Deflection due to rib shortening for crown $= -0.513 \times 0.21 = -0.103''$ fall.

Neglecting dead load moment deflection

Deflection of Crown due to, Temperature 27°F fall	=	−0.513"
do , Rib Shortening	=	−0.103"
		−0.616" fall.

For Normal Temperature during construction, 3/4" camber to be given to Arch Ring at crown of arch.

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CALCULATIONS FOR

Muko-Gawa over Mukogawa, Hyogoken.
Design of Piers for Arch Span.
Centre of gravity of Super structure. (Taking moment at Springing)
For Moto traffic way (one half span).

Names		weights	lever arms	Moments	Remarks
Handrails		6,000 #	13.65'	81,800 '#	
Sidewalks	pavement	1,800	12.06	21,700	
	Curb.	1,300	11.78	15,300	
	Slab	11,000	11.87	130,500	
	Coping	4,000	11.80	47,200	
	girders	5,700	10.90	62,100	
	brackets	1,500	11.32	17,000	
	gas pipe	1,300	10.90	14,200	
		32,600		389,800	
Trafficway, pavement		11,100	11.84	131,400	
	Slab	44,600	11.50	513,000	
	girders	15,600	10.78	168,200	
		71,300		812,600	
Columns, fills and Cross beam					
panel Pt. #	0 fill	3,400	10.84	36,800	
	2 "	3,900	10.70	41,700	
	4 col.	400	9.84	3,900	
	6 "	1,300	9.24	12,000	
	8 "	1,500	10.80	16,200	
	cross beam	2,600	8.34	21,700	
	10 Column	4,300	7.20	30,900	
	12 "	3,600	5.85	21,200	
		21,000		184,400	
Arch ring					
panel Pt. # 0		5,200	10.00	52,000	
1		10,600	9.93	105,100	
2		10,700	9.74	104,000	
3		10,800	9.43	101,900	
4		11,300	8.95	101,100	
5		11,500	8.35	96,000	
6		12,300	7.62	93,800	
7		13,200	6.75	89,200	
8		14,500	5.74	81,800	
9		15,800	4.56	72,100	
10		17,600	3.22	56,700	
11		19,700	1.70	33,500	
12		11,200	0.85	9,500	
		164,400		996,700	

Total for one half span = 289,300 × 8.25 = 2,383,500
For one span 578,600 # × 8.25'
On one pier 1,157,200 # × 8.25'

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CALCULATIONS FOR

Miuko-Ohashi over Mutsugawa, Nyogoken.

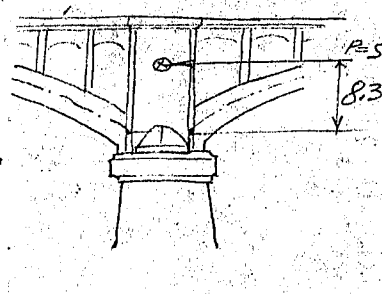
Centre of Gravity of Super Structure continued (Electric Railway)

Names		Weights	Lever arms	Moments about springing
Pavements		31,900	11.92	382,000
Slab		46,100	11.08	511,000
Rails and accessories		4,100	11.90	48,800
Sleepers etc		2,000	11.60	23,200
Longitudinal conc Beam		14800	10.37	168,000
		98,900		1,133,000
Part of Trafficway				
Slab		6,300	11.65	73,400
pavement		1,800	12.00	21,600
fillet		1,500	11.20	16,800
beam mortar filling		27,200	11.55	314,000
		36,800		425,800
Columns, fills and Cross beams				
Panel Pt. #0	fill	1,000	10.70	10,700
2	"	1,400	10.47	14,700
4	cols.	100	9.68	1,000
6	"	900	9.06	8,200
8	cross beam	1,800	7.20	12,600
8	cols	1,900	6.74	12,800
10	"	3,300	7.04	23,200
12	"	2,600	5.90	14,800
		13,000		98,000
Arch Ring				
Panel Pt. #0		4,700	10.00	47,000
1		9,600	9.93	95,300
2		9,700	9.74	94,400
3		9,700	9.43	91,500
4		10,200	8.95	91,300
5		10,400	8.35	86,800
6		11,100	7.62	84,600
7		11,900	6.75	80,300
8		13,000	5.74	74,600
9		14,200	4.56	64,800
10		15,800	3.22	51,000
11		17,100	1.70	29,100
12		10,100	0.85	8,600
		147,500		899,300

Total for one half span: 296,200# × 8.63' = 2,556,100
On one pier: 592,400# × 8.63' = 5,112,200 #

Summary for Superstructure

$$\text{Mean lever arm} = \frac{2,383,500 \times 4 + 5,112,200}{1,157,200 + 592,400} = \frac{14,646,200}{1,749600} = \underline{8.38'} \text{ above springing.}$$



Seismic force of Superstr.
= 1,749,600# × 0.3 = 525,000 #

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CALCULATIONS FOR

Muko-Okaki over Mukogawa, Hyogoken.
Design of Pier Continued.
Loads on Pier.

	Trafficway and Sidewalk		Electric Railway		Summary for whole width of Bridge.	
	Hor. Thrust	Vertical load	Hor. Thrust	Vert. load	Hor. Thrust	Vert. Load.
Dead load	438,000	556,000	450,400	564,400	1,326,400	1,672,400
Live load	* 155,500	40,700	Δ 182,400	47,900	493,400	129,300
Rib Shortening	− 7,600	—	− 6,800	—	− 22,000	—
Temp. Change 27°	± 65,600	—	± 58,600	—	± 189,800	—

Note :- * Neg. moment at Panel Pt. #6, Δ Neg. moment at Panel Pt. #6.

Centre of Gravity of Pier (Centre of gravity of Pier for the Part above top of base, call this C.g.1.)

Names	Dimensions	No.	weight	total wt.	lever arm	moment
Lamp post		2	say 800#	1,600	42.0	67,200#
Pedestal	2.0×3.5×4@160#	2	4,500	9,000	34.2	308,000
Top Shaft.	2.0×4.5×13.9@140	2	17,500	35,000	25.0	875,000
Wall (Top)	3.0×3.2×12.@150	2	1,700	3,400	30.2	102,500
" "	4.5×2.4×12@150	1	1,900	1,900	29.9	56,800
" "	3.9×2.4×12@150	2	1,700	3,400	29.9	102,000
" "	3.9×3.0×12@"	8	2,100	16,800	30.0	504,000
Columns	1.0×2.0×9.0@150	14	2,700	37,800	24.5	925,000
Granite block pav.	5.0×3.3×1.9@160	1	4,700	4,700	31.6	148,000
Lean mortar filling	5.0×4.1?×1.?@110	1	4,100	4,100	31.3	128,000
Rails and accessories	3.0×5.5×4	2	700	1,400	31.5	44,000
Asphalt pavement	16×5.5 @22	2	1,900	3,800	31.5	120,000
Curb Stone	5.5 @40	2	200	400	31.6	12,600
Slab	$\frac{6.25}{12}$×5.5×6.5@150	1	28,000	28,000	31.2	875,000
Top coping	1.5×1.7×1.7@150	2	1,100	2,200	31.7	69,800
Wall (bottom)	49.4×5.5×3.5×1.8@150	1	114,000	114,000	19.7	2,250,000
Coping	6.8×1.5×73.4@150	1	112,000	112,000	17.35	1,945,000
Shaft.	7.65×16.5×6.5@150	1	1,232,000	1,232,000	7.75	9,570,000
"	7×3.83×16.5@150	2	66,300	132,600	say 6.0'	795,000
Mouldings.	3.5×2.7×1.2 @140	2	1,600	3,200	+ 1.85	5,900
				1,747,300	× 10.97 =	19,156,900

Centre of gravity of Whole body of pier (Call this C.g.2.)

Name	dimension	no.	weight	total wt.	lever arm	moment.
Part above top of base		1	1,747,300	1,747,300	× 17.97 =	31,400,000
Upper base	14×7.9×3@150	1	498000	498,000	× 5.5 =	2,740,000
Lower base	18×8.2×4@150	1	885,000	885,000	× 2.0 =	1,770,000
				3130,000#	× 11.48 =	35,910,000

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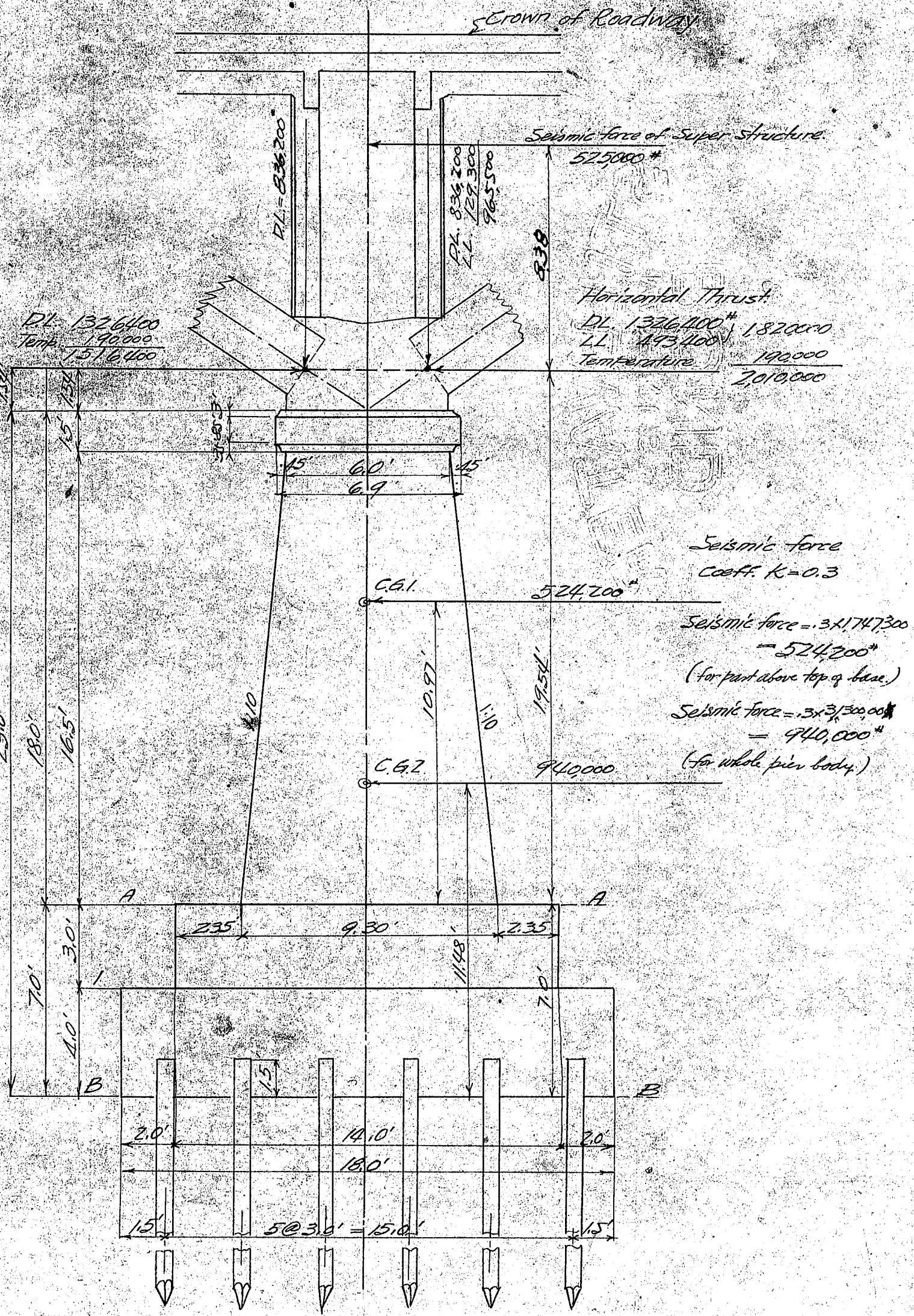
CALCULATIONS FOR

Mukō-Okachi over Mukogawa, Myogoku

Design of Pier Continued



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CALCULATIONS FOR

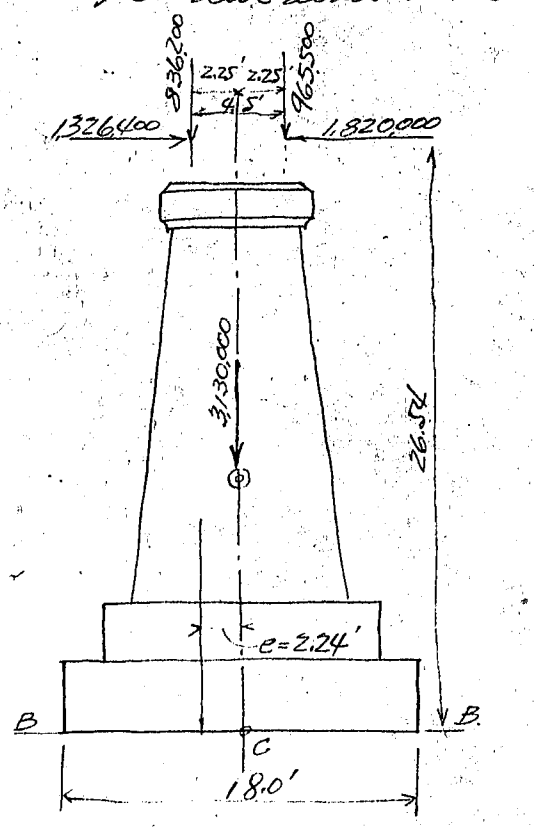
Muko-Ohashi over Mukogawa, Hyogoken.

Design of Pier Continued.

Stability of Pier.

For Live Load and Dead Load (Temperature thrusts cancels each other)

Stability at B-B line.



	load	lever arm	moment abt. C.
Hor. Thrust (right)	+1,820,000	× 26.54'	= +46,500,000
" " (left)	−1,326,400	× 26.54	= −35,200,000
Vert. load (right)	+965,500	× −2.25	= − 2,170,000
" " (left)	+836,200	× 2.25	= + 1,880,000
Wt. of pier body	+3,130,000	× 0	= 0
Sum of vert. loads	= 4,930,000		11,010,000

$$\text{Eccentricity} = \frac{11,010,000}{4,930,000} = 2.24'$$

$$S = \frac{P}{A} \pm \frac{6M}{bh^2} = \frac{4,930,000}{18\times82} \pm \frac{6\times11,010,000}{82\times18^2} = 3340 \pm 2490 = 5830\ \#/\text{ft}^2$$

$$= 2.60\ \text{tons/ft}^2 \text{ OK.}$$

No tension occurs in the heel.

Stability for Dead Load and Seismic forces (at Bottom of base).

Moment due to Seismic force Super structure = 525,000 × 34.92 = 18,300,000

" " " " Sub Structure = 940000 × 11.48 = 10,800,000

29,100,000

$$\text{Eccentricity} = \frac{29,100,000}{4,930,000} = 5.90'$$

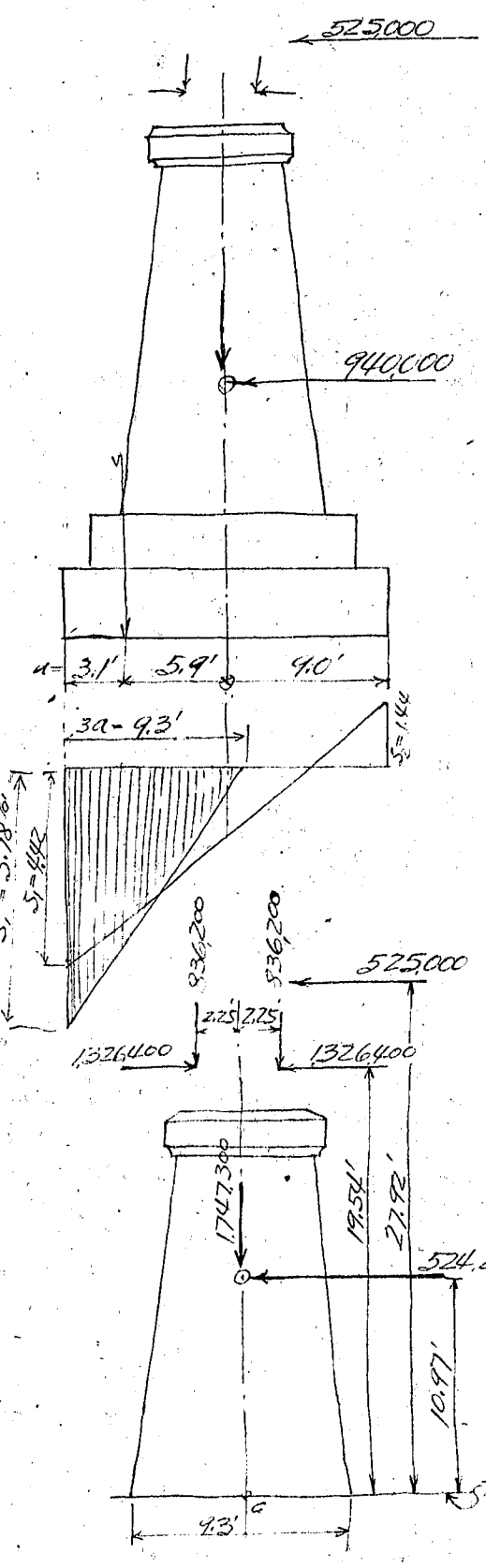
$$S = \frac{4930,000}{18\times82} \pm \frac{6\times29100,000}{82\times18^2} = 3340 \pm 6570 = 9910\ \#/\text{ft}^2 = 4.42\ \text{tons/ft}^2$$

" = 3230 #/ft² = 1.44 " tension.

Neglecting tension,

$$S_1' = \frac{2V}{3ab} = \frac{2\times4930,000}{3\times3.1\times\#82} = 12,940\ \#/\text{ft}^2 = 5.78\ \text{tons/ft}^2 \text{ Compression.}$$

(3.24×1.8) OK.



Stability for Dead Load & Seismic forces (at top of base).

	Load.	Lever arm.	Moment about C.
Hor Thrust (right)	1,326,400	19.54	25,950,000
" " (left)	−1326,400	19.54	−25,950,000
Vert. load (right)	836,200	2.25	− 1,880,000
" " (left)	836,200	2.25	1,880,000
Weight of Pier	1747,300	0.00	0
Seismic force, Pier	524,200	10.47	5,750,000
" " Super str.	525,000	27.92	14,650,000
Σ Vert. forces	= 3,419,700 #		20,400,000 '#'

$$\text{Eccentricity} = \frac{20400,000}{3419,700} = 5.98'$$

Assume 7/8" ϕ bars 1.0' c to c on both sides = 1.20 □"/ft.

$$e/h = 5.98/9.3 = .644,\quad d'/h = .05,\quad p = .0011;\ k = .23,\ j = .854$$

$$f_c = \frac{M}{jbh^2} = \frac{20,400,000}{.854\times72\times9.3^2} = 60,700\ \#/\text{ft}^2 = 422\ \#/\text{in}^2 \text{ OK.}$$

$$f_s = nf_c\left(\frac{d}{kh} - 1\right) = 15\times42.2\left(\frac{9.0}{.23\times9.3} - 1\right) = 20,200\ \#/\text{in}^2 \text{ OK.}$$

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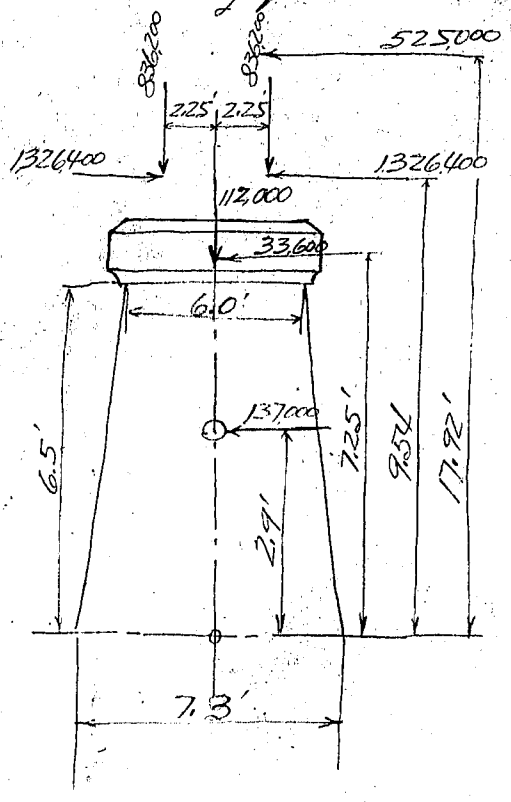
CALCULATIONS FOR

Muko-Okaki over Mukogawa Myojoteu.
Design of Pier Continued.
Stability for Dead Load and Seismic force at 10 ft. above top of base.



	Load.	lever arm	Moment about C
Hor. Thrust (right)	1,326,400	9.54	12,650,000
" " (left)	−1,326,400	9.54	−12,650,000
Vert. load (right)	836,200	−2.25	−1,880,000
" " (left)	836,200	2.25	1,880,000
Seismic force Super Str.	525,000	17.92	9,330,000
" " Coping	33,600	7.25	243,000
" " Shaft.	137,000	2.9	397,000
Wt. of Coping	112,000	0	0
" " Shaft.	456,000	0	0
Sum of vert. load.	= 2,240,300		9,970,000.

$$\text{Eccentricity} = \frac{9970000}{2240300} = 4.44' \qquad e/h = \frac{4.44}{7.3} = 0.61$$

$d/h = .05$,

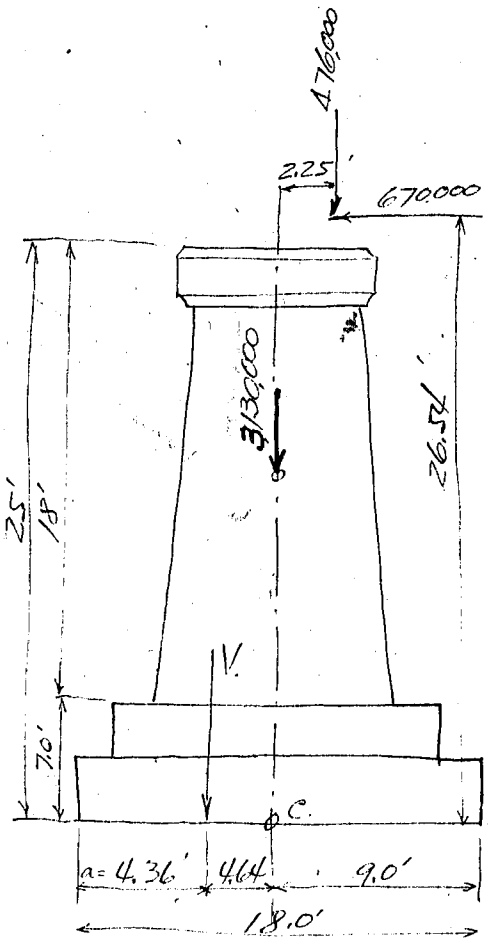
Steel same as before.

$p_0 = 0.0013$, $K = .24$, $L = .067$

$$f_c = \frac{9,970,000}{.067 \times 70 \times 7.3^2} = 49900\ \#/0' = \underline{277\ \#/0''}\ \text{Ok.}$$

$$f_s = 15 \times 277 \times \left(\frac{7}{.24 \times 7.3} - 1\right) = \underline{12,450\ \#/0'}\ \text{Ok}$$

Stability of Abutment Pier.
Arch ring on one side of the pier and none on the other.



	load	lever arm.	moment.
Hor. Thrust.	670,000	26.54	17,800,000
Vert. load	476,000	−2.25	−1,070,000
Wt. of pier	3,130,000	0	0
			16,730,000

Sum of vert. load. = 3,606,000

$$\text{Eccentricity} = \frac{16,730,000}{3,606,000} = 4.64'$$

$$S = \frac{3,606,000}{18 \times 82} \pm \frac{16,730,000 \times 6}{82 \times 18^2} = 2440 \pm 3780$$

$$= 6220\ \#/0' = 2.78\ \text{tons/0' comp.}$$

$$\text{or} = -1,340\ \#/0' = 0.60\ \text{tons/0' tension.}$$

Neglecting tension;

$$a = 9 - 4.64 = 4.36'$$

$$S' = \frac{2 \times 3,606,000}{3 \times 4.36 \times 82} = 6730\ \#/0' = \underline{3.01\ \text{tons/0'}}\ \text{Ok.}$$

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HYOGO KEN

MADE BY K. Inab. DATE 15-7-5 FILE NO

CHECKED BY K. Inab. DATE 15-7-7 PAGE NO 65

CALCULATIONS FOR

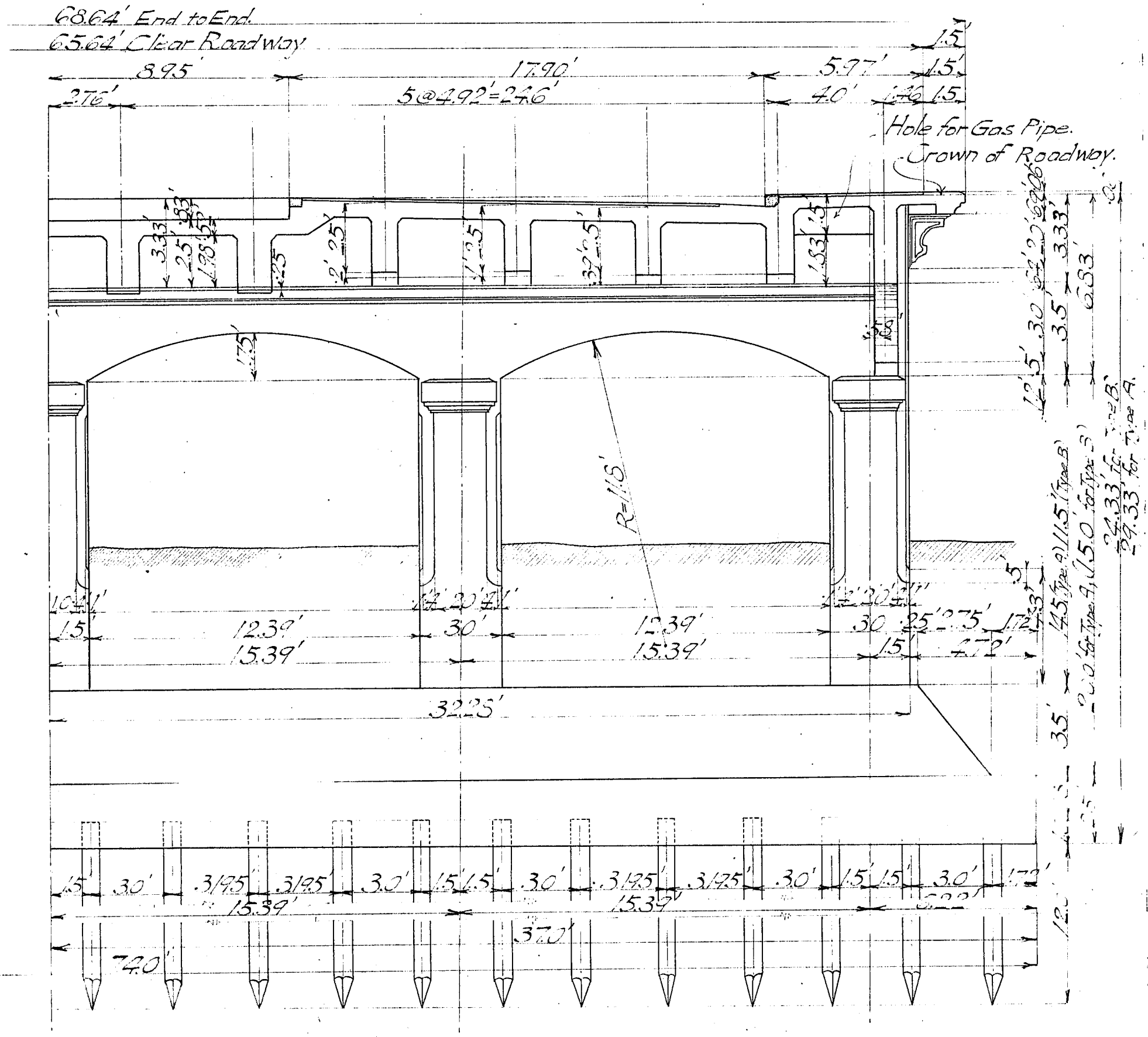
Muko-Ohashi over Mukogawa, Hyogo Ken.

Design of Approach Girder Span.

Floor Slab.

All details for floor slab shall be the same as for arch span.

Span numbers of approach girders are 3 at right bank and 6 at left bank.



ELEVATION OF PIER.

The girders shall be calculated as a 3 span continuous beam of 27 ft. span (effective) with one end fixed and the other free, (fixed ends near to the centre of the river.).

For the appearance sake, the bottom of the fascia girders shall be cambered 3 in. while other beams shall be straight.

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CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Iwata DATE 12-10-5 FILE NO
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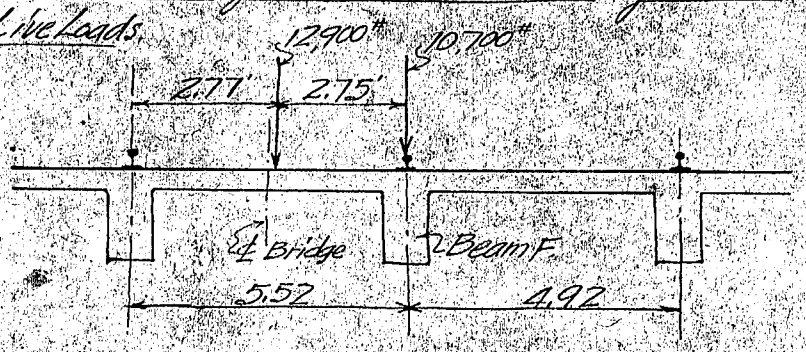
CALCULATIONS FOR

Muko-Gawa over Nukegawa Myogoku
Design of Approach girder Span, continued
Main Girders under Railway (Beam F)
3 Span continuous beam with two end span the other ends continuous. Effective span = 27.0'
The span length next to the free end girder 26' 0"

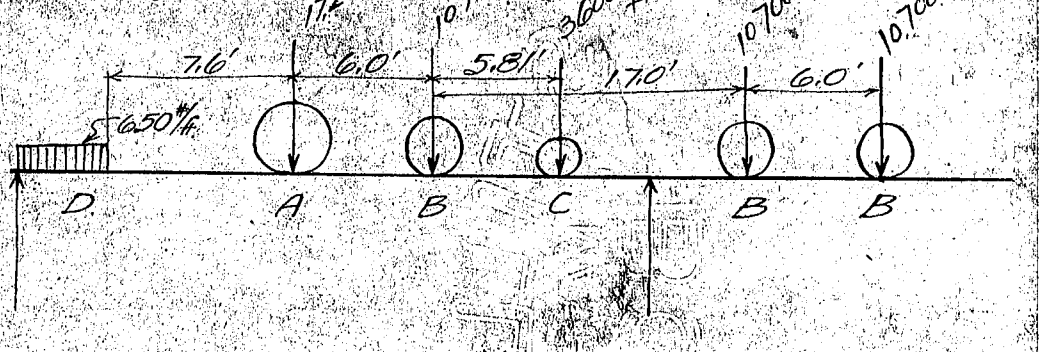
Dead Load.
Weight of floor. 181 #/ft² × 5.33+4.92/2 = 980 #/ft run.
" Stem of beam 1.0×2.0@150 = 300
railband & connection 30
Total dead load per ft of girder = 1280 #/ft.

Live Load.
Impact allowance I = 20/(60+l) = 20/(60+27/328) = 0.29 ℄
12 ton motor truck rear wheel concentration = 9,900×1.29 ℄ = 12,900 #
" " front wheel " = 3,300×1.29 ℄ = 4,300
33 ton Electric car wheel Concentration = 8,250×1.29 ℄ = 10,700
Uniform live load for Railway W = 120,000/(170+l) = 120,000/(170+27/328) = 674 kg/m² take this at 600 kg/m² = 125 #/ft.
" " " Sidewalk. W = 100,000/(170+l) = 100,000/(170+27/328) = 560 " " = 500 " = 100 #/ft.

Inside Main Girders under Railway truck (Beam F)
Live Loads.

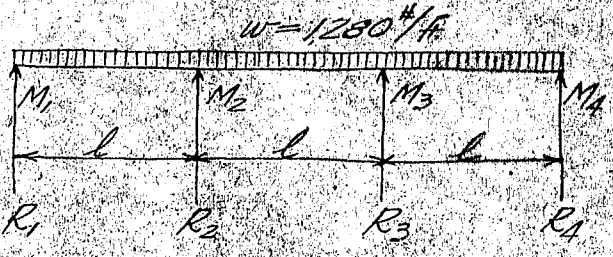


R₂ 6.400 R₁ 10,700 / 6,500 / 17,200



Assume the intermediate supports being continuous and both ends supports be free on account of the poor Continuity of fixed end.

Dead Load Moments.
Negative moments.



$R_1 = R_4 = \frac{wl}{2} - \frac{wl^2}{10l} = \frac{4wl}{10}$

$R_2' = R_3'' = \frac{wl}{2} + \frac{wl}{10} = \frac{6wl}{10}$

$R_2'' = R_3' = \frac{wl}{2} + \frac{wl}{10} - \frac{wl}{10} = \frac{5wl}{10}$

$R_2 = R_3 = \frac{11wl}{10}$

By theory of Three Moments.

$M_1 + 4M_2 + M_3 = -\frac{wl^2}{2}$

we have.

$0 + 4M_2 + M_3 = -\frac{wl^2}{2}$

$M_2 + 4M_3 + 0 = -\frac{wl^2}{2}$

Solving these formulas.

$M_2 = -\frac{wl^2}{10}$

$M_3 = -\frac{wl^2}{10}$

for $l = 27'$, $w = 1,280$ #/ft.

$M_2 = M_3 = -\frac{1280 \times 27^2}{10} = -93,500$ #

$R_1 = R_4 = \frac{4 \times 1280 \times 27}{10} = 13,800$ #

$R_2 = R_3 = \frac{11 \times 1280 \times 27}{10} = 38,000$ #

Max. End Shear $V_{2L} = \frac{6wl}{10} = -20,800$ #

$V_{1R} = +13,800$ #

$V_{2R} = +17,300$ #

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（独立行政法人 土木研究所蔵）
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JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Inaba DATE 13-10-6 FILE NO
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CALCULATIONS FOR

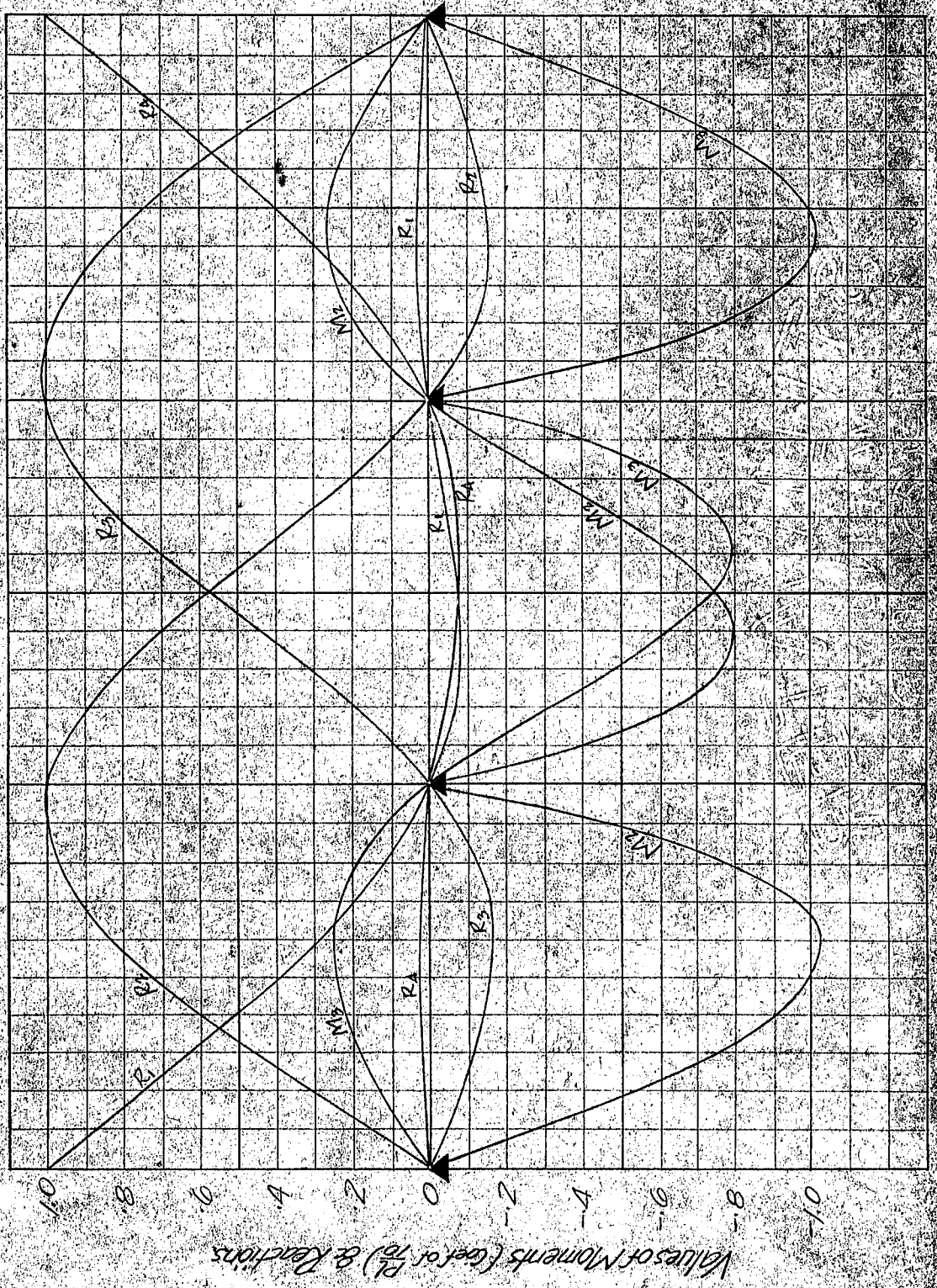
Muko-Okashi over Mukogawa, Hyogoken.

THREE SPAN CONTINUOUS BEAM

Supported Ends.

Influence Lines for Neg. Moments & Reactions.

(1)



1st. Span loaded.

$M_2 = -\frac{4}{15}P\ell(B)$; $M_3 = -\frac{P\ell}{15}(B)$.

$R_1 = P(1-K) - \frac{4}{15}P(B)$; $R_2 = PK + \frac{4}{15}P(B)$, $R_3 = -\frac{2}{5}P(B)$.

$R_4 = \frac{P}{15}(B) = (K^2 - 3K + 3)$

where $B = (K^2 - 3K + 3)$

$A = (2K - 3K^2 + K^3)$

2nd Span loaded.

$M_2 = -\frac{P\ell}{15}(4A-B)$, $M_3 = \frac{P\ell}{15}(A-4B)$.

$R_1 = -\frac{P}{15}(4A-B)$, $R_2 = P(1+K) + \frac{4}{15}P(3A-2B)$

$R_3 = PK - \frac{P}{5}(2A-3B)$, $R_4 = -\frac{P}{15}(A-4B)$

A & B as before.

(2)

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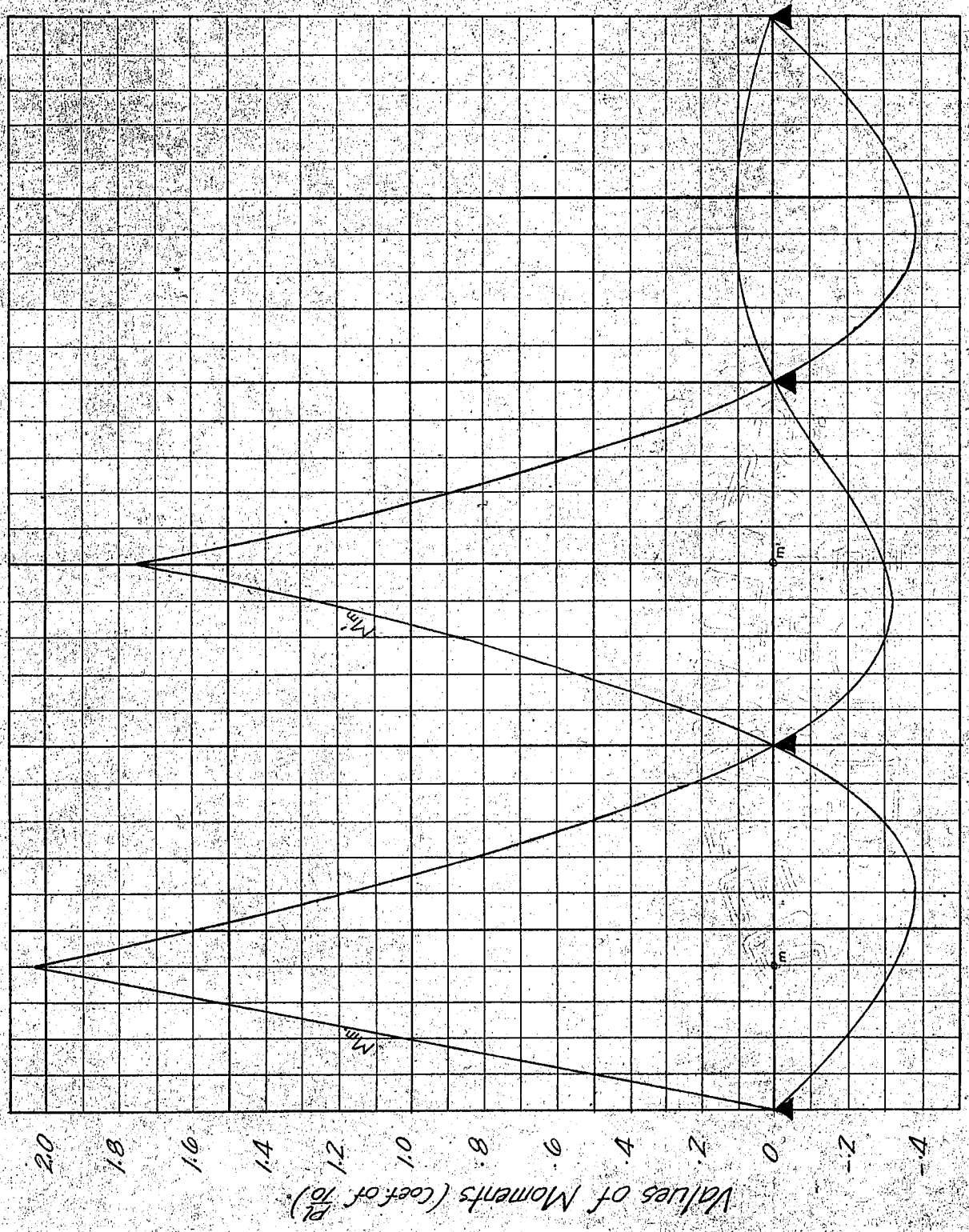
CALCULATIONS FOR

Mukoohashi on Mukogawa, Hyogoken.

3 SPAN CONTINUOUS BEAM WITH SUPPORTED ENDS.
Influence Lines for Max. Pos. Moments.
for end spans, max. pos. M occurs at K=0.4 (for Live load only, K=0.45; for dead only, K=0.44)
" Centre " " K=1.5



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CALCULATIONS FOR

Muko-Okashi over Mukogawa, Nyogoku-sen.
Moments and Sheans for Beam F Continued.

Dead Load Positive Moments.

Dead load max. pos. moment occurs at 0.4 l from the end support in the end span
and its value = $0.08\,wl^2$

$+M = 0.08\,w\,l^2 = 0.08 \times 1{,}280 \times 27^2 = +74{,}600$ '#

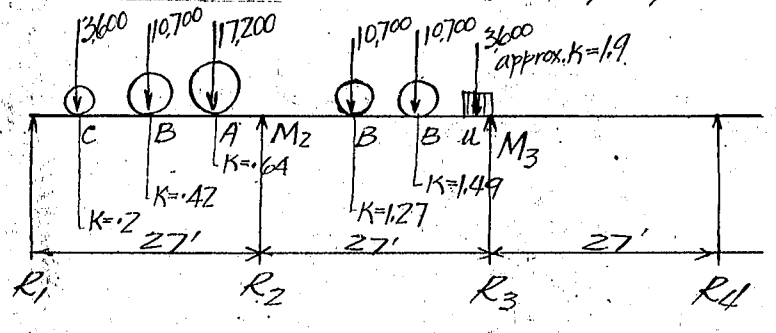
In centre span, max. pos. moment occurs at centre.

$+M = 0.025\,w\,l^2 = 0.025 \times 1{,}280 \times 27^2 = +23{,}300$ '#

Live Load Moments.

Max. Neg. Moment. $-M_2$ & $-M_3$.

All Load manners for max. moments and reactions are desided by the prepaired diagrams.

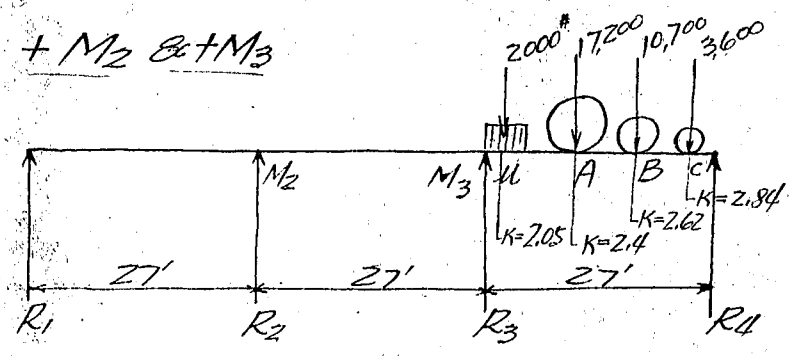


load no.	load.	M_2 unit-load	$10M_2/l$	K.
C	3,600	−0.51	−1,840	0.20
B	10,700	−0.92	−9,850	0.42
A	17,200	−1.01	−17,380	0.64
B	10,700	−0.74	−7,920	1.27
B	10,700	−0.76	−8,130	1.49
u	3,600	−0.15	−540	1.90
			−45,660	

$\therefore -M_2 = -45660 \times \frac{27}{10} = -123{,}200$ '#

$-M_3 = -123{,}200$ '# (for reverse load manner)

$+M_2$ & $+M_3$



Poo. M_2 & M_3.

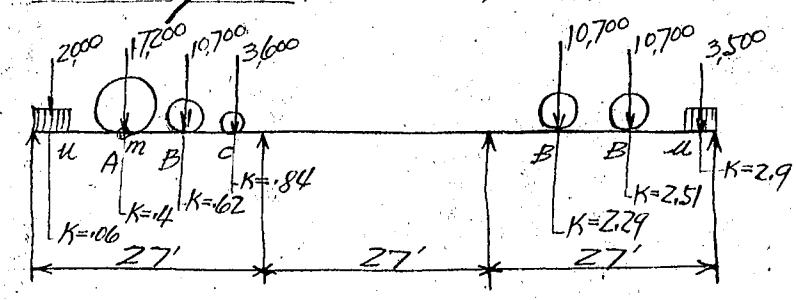
load no.	load.	M_2 unit-load	$10M_2/l$	K
A	17,200	+0.26	+4,470	2.40
B	10,700	+0.22	+2,350	2.62
C	3,600	+0.11	+400	2.84
u	2,000	+0.06	+120	2.05
			+7,340	

$+M_2 = 7{,}340 \times 2.7 = +19{,}800$ '#

$+M_3 =$ " $= +19{,}800$ '# (for reverse load manner)

Max. Positive Moments.

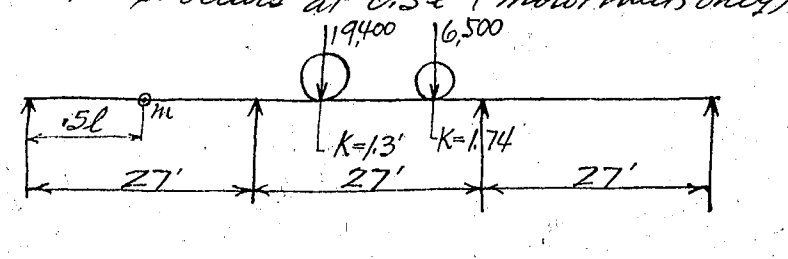
Side Spans. $+M_{max} = M_m$ at 0.4 l.



load no.	load.	M_2 unit-load	$10M_2/l$	K.
A	17,200	+2.04	+35,100	.40
B	10,700	+1.10	+11,800	.62
C	3,600	+0.40	+1,400	.84
B	10,700	+0.95	+10,200	2.29
B	10,700	+0.97	+10,400	2.51
u	2,000	+0.06	+120	.06
u	3,500	+0.06	+110	2.9
			+69,130	

$+M_{m.4} = +186{,}800$ '#

−M max. occurs at 0.5 l (Motor truck only).



Motor truck rear wheel	19,400	−0.382	−7,410	K=1.3
front wheel	6,500	−0.210	−1,370	K=1.74
			−8,780	

$-M_m = -8{,}780 \times \frac{27}{10} = -23{,}700$ '#

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CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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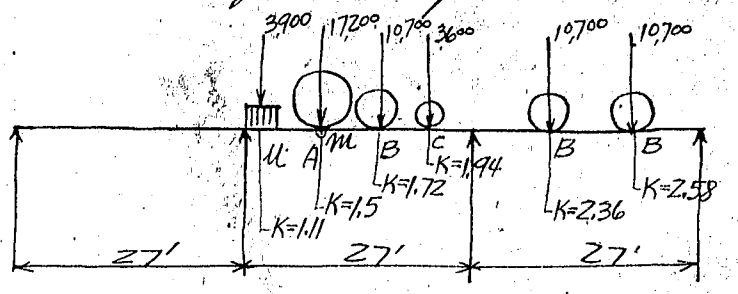
CALCULATIONS FOR

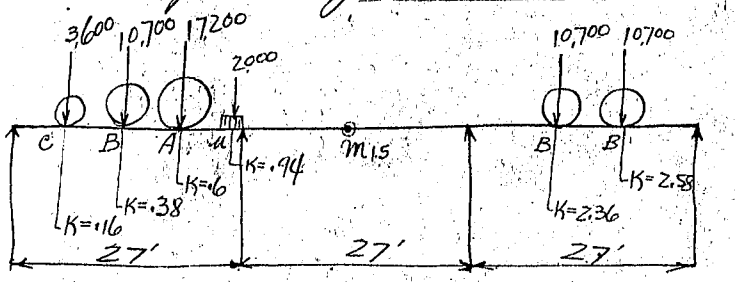
Muko-Ohashi over Mukogawa, Hyogoken.
Moments and Shears for Beams F. Continued.
Pos. Moment for Centre span. + Mmax. at centre of centre span.



load no.	load	Mm unit load	10 Mm/l	K
A	17,200	+1.75	+30,100	1.5
B	10,700	+0.78	+8,350	1.72
C	3,600	+0.13	+470	1.94
B	10,700	−0.375	−4010	2.36
B	10,700	−0.34	−3,640	2.58
u	3900	+0.26	+1.010	1.11
			+32,280	

$+Mm = +32{,}280 \times \frac{27}{10} = +87{,}200'\#$

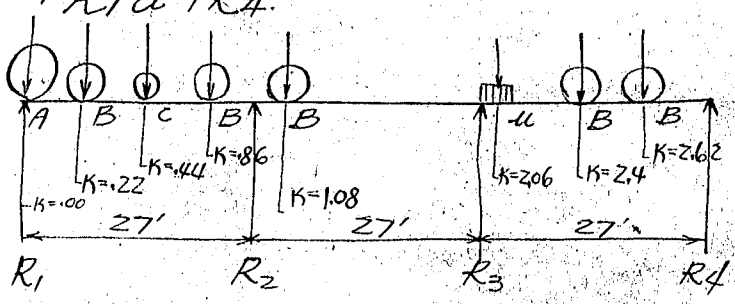
Centre Span. Negative Moment.



load no.	load	Mm unit load	Mm × 10/l	K
C	3600	−0.16	−580	0.16
B	10,700	−0.32	−3,420	0.38
A	17,200	−0.382	−6,570	0.60
B	10,700	−0.375	−4,010	2.36
B	10,700	−0.34	−3640	2.58
u	2,000	−0.11	−220	0.94
			−18,440	

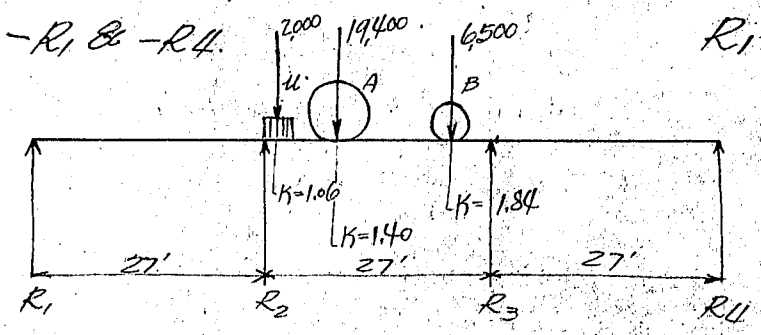
$-Mm_{1.5\ max.} = -18{,}440 \times \frac{27}{10} = -49800'\#$

Shears and Reactions.
$+R_1$ & $+R_4$.



load no.	load	R_1 unit load	R_1	K
A	17,200	+1.00	+17,200	0.0
B	10,700	+0.72	+7,700	.22
C	3,600	+0.463	+1,670	.44
B	10,700	+0.082	+880	.86
B	10,700	−0.03	−320	1.08
u	2,300	+0.01	+20	2.06
B	10,700	+0.025	+270	2.4
B	10,700	+0.022	+240	2.62
			+27,660	

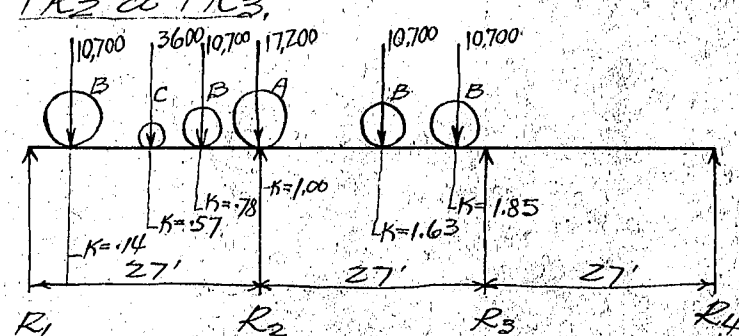
$R_1 = +27660\#$ Max. end shear at 1st. support. $V_{1R} = +27660$



u	2,000	−0.022	−40	1.06
A	19,400	−0.08	−1,550	1.40
B	6,500	−0.022	−140	1.84
			−1,730	

$-R_1 = -1{,}730\#$

$+R_2$ & $+R_3$,



load no.	load	$+R_2$ unit load	R_2	
B	10,700	+0.22	+2,360	Max. Shear at 2nd. Support, left side.
C	3,600	+0.80	+2,880	
B	10,700	+0.96	+10,260	$V_{2L} = -32{,}700\#$
A	17,200	+1.00	+17,200	
B	10,700	+0.41	+4,390	
B	10,700	+0.157	+1,610	
			+38,700	

$+R_2 = +38{,}700\#$ = $+R_3$ max.

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CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Inaba DATE 8-10-8 FILE NO ____
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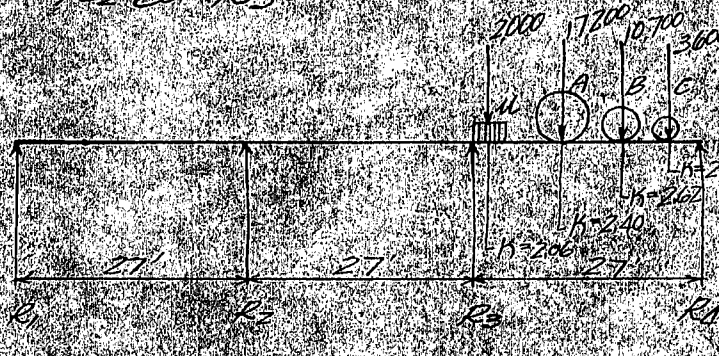
CALCULATIONS FOR

Meiko-Shoshi ore, Mukogawa, Nyogoken.

Moments and Shears for Beam F continued.

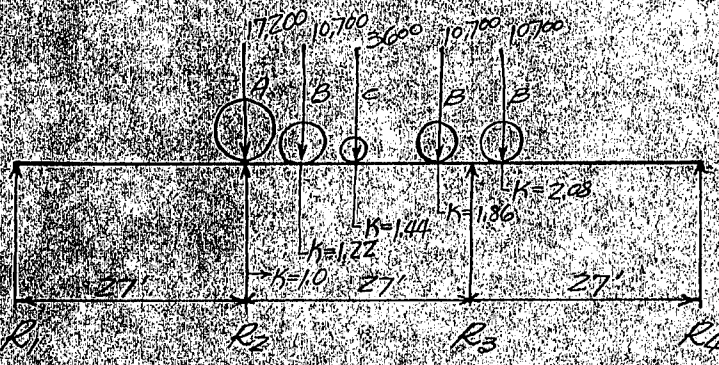
$-R_2$ & $-R_3$



Load no	Load	$-R_2$ unit load	$-R_2$
M	2000	−0.04	−80
A	17200	−0.158	−2720
B	10700	−0.13	−1390
C	3600	−0.06	−220
			−4410

$-R_2 = -4400$ #

Max. Shear at 2 right side.



Load no	Load	$+V_{2R}$ unit load	V_{2R}
A	17200	+1.00	+17200
B	10700	+0.88	+9420
C	3600	+0.65	+2340
B	10700	+0.13	+1390
B	10700	−0.055	−590
			+29760

$+V_{2R} = 29760$ #

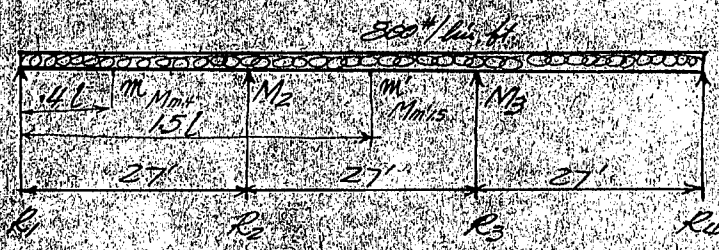
Summary for Moments and Reactions. (For Beam F under Railway)

	Moments						Reactions				Shears			
	At support		Side span k=.4		Centre span k=.5		Side span		Centre span		Side span		Centre span	
	$+M_2$ & M_2	$-M_2$ & $-M_3$	$+M_{0.4}$	$-M_{0.4}$	$+M_{1.5}$	$-M_{1.5}$	$+R_2$ & $+R_4$	$-R_2$ & $-R_4$	$+R_2$ & $+R_4$	$-R_2$ & $-R_3$	V_{1R}	V_{2L}	V_{2R}	V_{3L}
Dead Load	—	−93500	+74600	—	+23300	—	+13800	—	+38000	—	+13800	−20800	+17300	+17300
Live Load	+19800	−123700	+186800	−23700	+87200	−49800	+27660	−1730	+38700	−4480	+27660	−32700	+29760	+29760
Total	+19800	−216700	+216400	−23700	+110500	−49800	+41460	−1730	+76700	−4400	+41460	−53500	+47060	+47060

Beam B, C and A for Trafficway.

Moments and Reactions.

Dead Load Moments & Reactions.



Dead Load:—

2" asphaltic pavement @ $\frac{13}{12}$ = 22 #/ft.

6½" Reinf. conc. slab .52@150 = 78

100 #/ft.

Dead Load per lin. ft of beam = 100×4.92 = 500 # say

Stem of beam = 10×20@150 = 300

800 #/ft. run

$-M_2 = -M_3 = -\frac{wl^2}{10} = -\frac{800\times27^2}{10} = -58400$ '#

$+M_{m.4} = 0.08wl^2 = +.08\times800\times27^2 = +46600$ '#

$+M_{m.5} = 0.025wl^2 = +.025\times800\times27^2 = +14600$ '#

$V_{1R} = +8630$ #

$V_{2L} = -0.6wl = -12940$

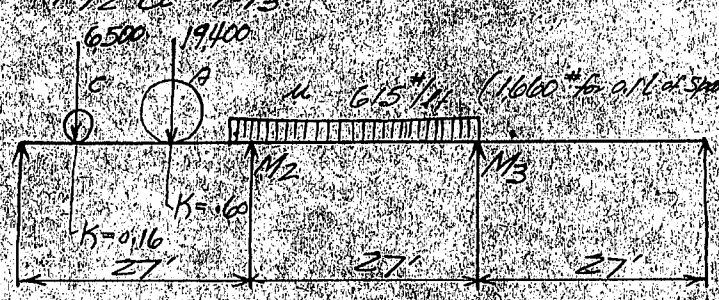
$V_{2R} = +0.5wl = +10780$

Reactions R_1 & R_4 = 0.4wl = +8630 #

R_2 & R_3 = 1.1wl = +23800

Live Load Moments.

$-M_2$ & $-M_3$



Load no	Load	$-M_2$ unit load	$-M_2$ '#
C	6500	−0.42	−2730 '#
A	19400	−1.024*	−19800
M	1660	*−5.31	−8820
			−31430 '#

$-M_2 = -31430\times2.7 = -85000$ '# $= -M_3$

* Sum of ordinate of influence line for M_2.

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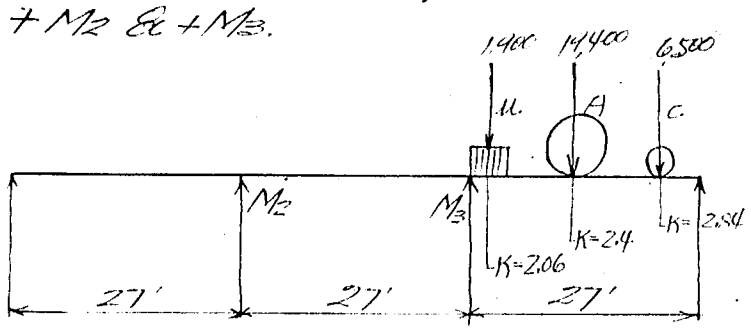
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CIVIL ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Iraba DATE 13-10-2 FILE NO
CHECKED BY DATE PAGE NO 72

CALCULATIONS FOR

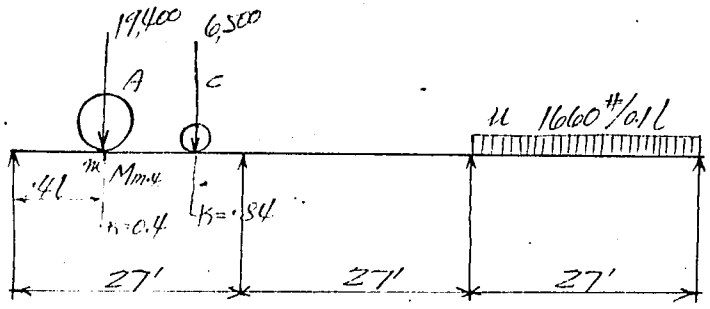
Mutsu-Ohashi over Mutsugawa, Nyugoka.
Moments and Reactions for Beam B, C and D Continued.
$+M_2$ & $+M_3$.



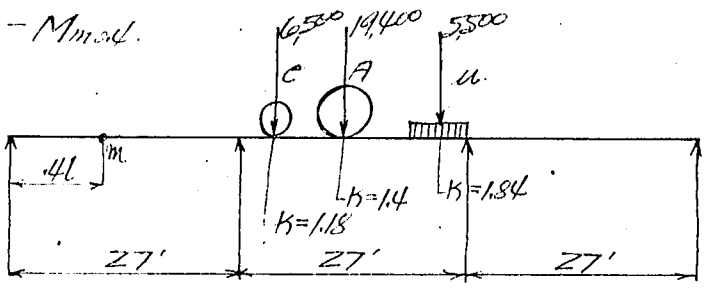
load no.	load	$+M_2$ unit load	$+M_2$ load
A	14,400	+0.26	+5,050
C	6,500	+0.11	+ 715
u	1,900	+0.07	+ 130
			5,895

$+M_2 = +M_3 = +5,895 \times 27 = +159,400$ '#

Positive Mal. Moment. Side Span. ($M_{m.4}$)



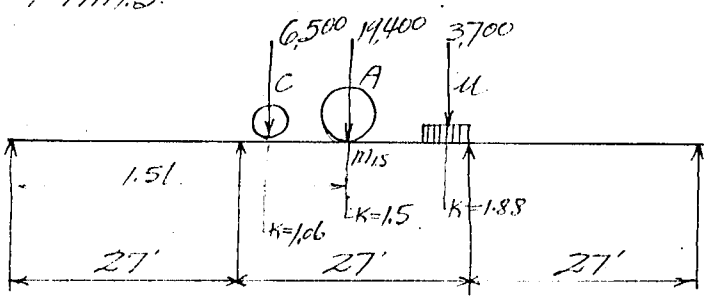
$-M_{m.4}$.



load no.	load	$-M_{m.4}$ unit load	$-M_{m.4}$ load
C	6,500	−0.24	− 1,560
A	19,400	−0.32	− 6,210
u	5,500	−0.10	− 550
			−8,320

$-M_{m.4} = -8,320 \times 27/0 = -224,400$ '#

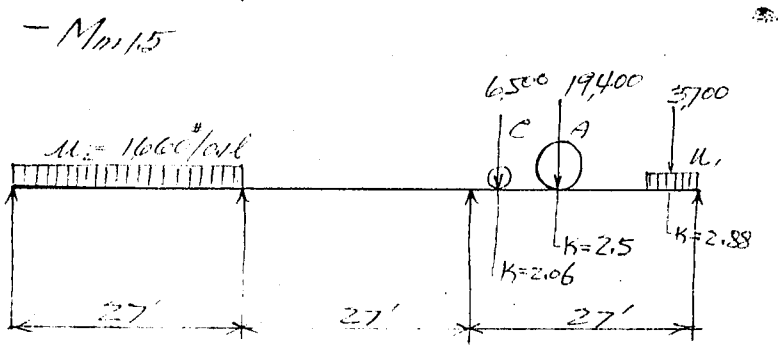
Positive Mal. Moment. Centre Span.
$+M_{m1.5}$.



load no.	load	$+M_{m1.5}$ unit load	$+M_{m1.5}$ load
C	6,500	+ 0.14	+ 910
A	19,400	+ 1.75	+34,000
u	3,700	+0.19	+ 700
			+35,610

$+M_{m1.5} = +35,610 \times 27/0 = +496,200$ '#

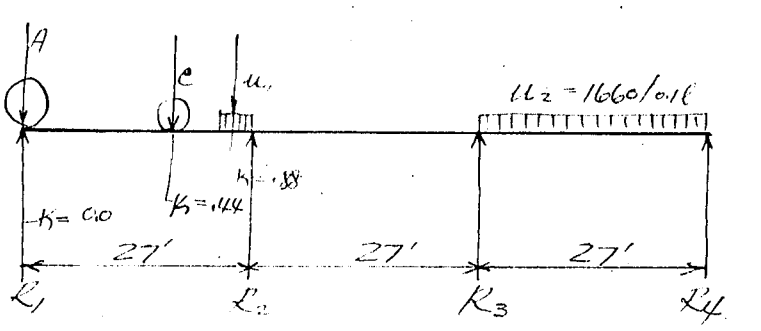
$-M_{m1.5}$



load no.	load	$-M_{m1.5}$ unit load	$-M_{m1.5}$ load
u	1,660	− 2.522	− 4,190
c	6500	− 0.34	− 2210
A	19,400	− 0.375	− 7,280
u	3,700	− 0.12	− 440
			− 14,120

$-M_{m1.5} = -14,120 \times 27/0 = -381,200$ '#

Shears and Reactions.
$+R_1$



load no.	load	$+R_1$ unit load	$+R_1$
A	19,400	+1.00	+19,400
C	6,500	+0.465	+3,020
u_1	4,300	+0.07	+ 300
u_2	1,660	+0.177	+ 290
			+23,010

$+R_1 = +23,010$ '# $V_1R = +23,010$ '#

$-R_1$ is very small and it is not necessary to investigate.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Iraba DATE 13-10-8 FILE NO
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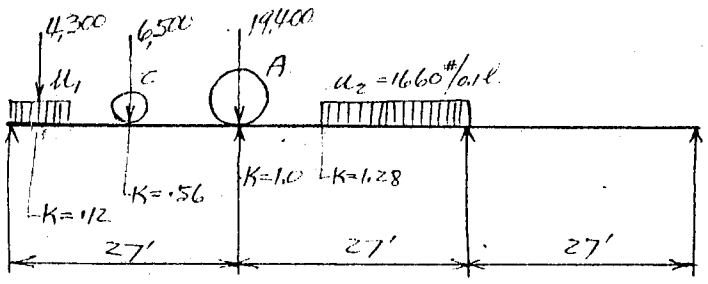
CALCULATIONS FOR

Mukō-bashi over Mukōgawa, Hyōgoken.

Shears and Reactions for Beam B, C and D Continued. (Trafficway.)

$+R_2$

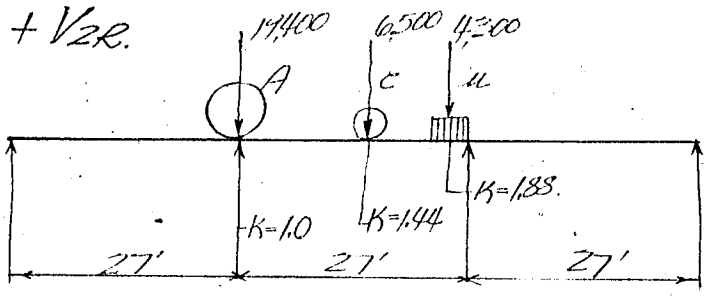


load no.	load	$+R_2$ Unit load	$+R_2$
u_1	4,300	+0.22	+ 950
C	6,500	+0.79	+5,130
A	19,400	+1.00	+19,400
u_2	1,660	+2.71	+ 4,500
			+29,980#

$V_{2L} = -25,480$#

$+R_2 = +29,980$#

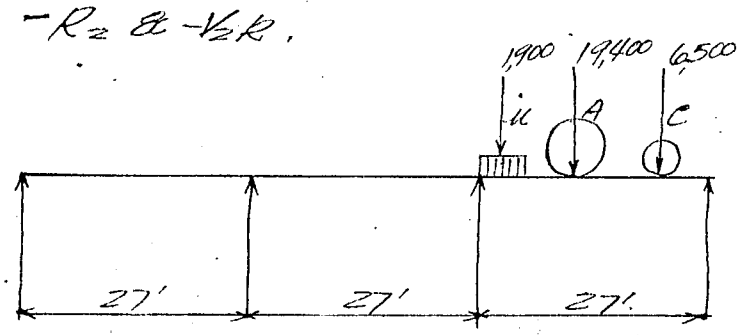
$+V_{2R}$.



A	19,400	+ 1.00	+ 19,400
C	6,500	+0.65	+ 4,220
u	4,300	+0.11	+ 470
			+24,090

$+V_{2R} = +24,090$#

$-R_2$ & $-V_{2R}$.



u	1,900	−0.04	− 80
A	14,400	−0.158	−3,070
C	6,300	−0.06	− 390
			−3,540

$-R_2$ & $-V_{2R} = -3,540$#

Summary for Moments and Shears (Beams under Carriageway).

	Moments						Reactions				Shears.			
	At support		Side span k=.4		Centre span k=1.5		Side span		Centre span		Side span		Centre span.	
	$+M_2$ & $+M_3$	$-M_2$ & $-M_3$	$+M_{m.4}$	$-M_{m.4}$	$+M_{m1.5}$	$-M_{m1.5}$	$+R_1$ & $+R_4$	R_1 & $-R_4$	$+R_2$ & $+R_3$	$-R_2$ & $-R_3$	V_{1R}	V_{2L}	V_{2R}	V_{3L}
Dead Load	-	−58,400	+46,600	—	+11,600	—	+ 3,630	—	+23,900	—	+8,630	−12,440	+10,790	−10,780
Live Load	+15,900	−85,000	+116,500	−22,400	+46,200	−38,100	+23,010	Very small	+29,980	−3,540	+23,010	−25,480	+24,090	−24,090
Total	+15,900	−143,400	+163,100	−22,400	+110,800	−38,100	+31,640	"	+53,780	−3540	+31,640	−38,420	+34,870	−34,870

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Inaba DATE 13-10-9 FILE NO
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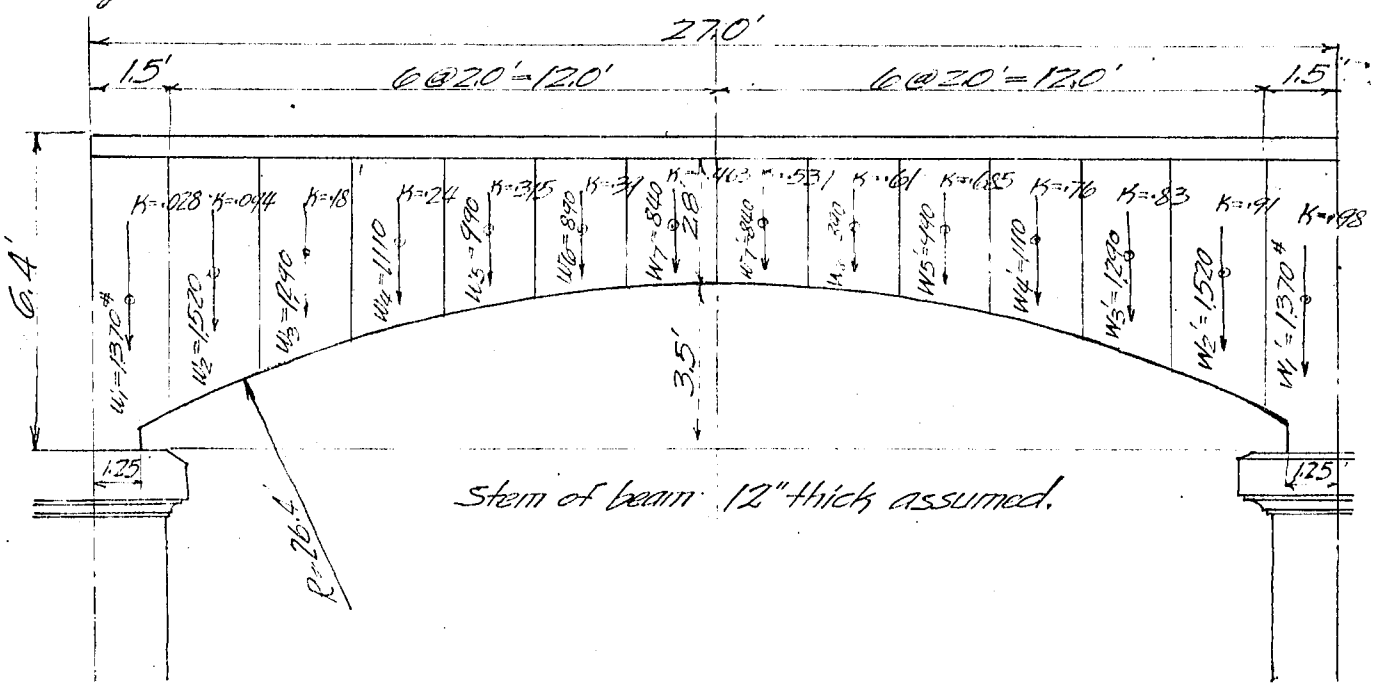
CALCULATIONS FOR

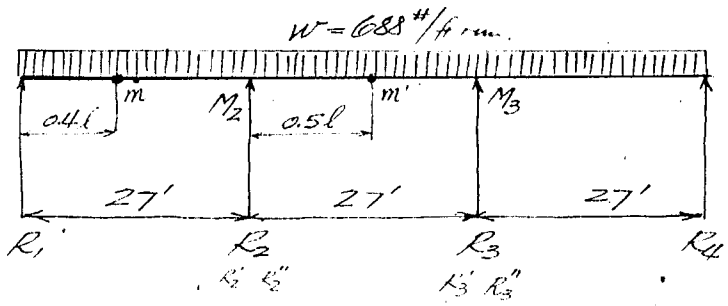
'Muko-Okashi' over Mukogawa, Hyogoken.
Fascia Girder.



Moments, Reactions and Shears.
Dead Load on Fascia Girder, except wt of web of beam.
Slab, handrail etc. 3.46 × [illegible] = +526 #/ft. run.
Load due to dead load moment 736" ÷ 4 = +184
87 ÷ 4 = −22
Total. = +688#/ft. run

Live Load on Fascia Girder.
Uniform live load = 100 × 3.46 = +346#/ft run
live load due to moment = 242 ÷ 4 = +60
" " = 133 ÷ 4 = −33
Total = +373#/ft run

Dead Load Moments, Shears and Reactions (except due to wt. of stem).



$M_2 = M_3 = -\frac{1}{10} wl^2 = -50,200'^{\#}$
$M_{m.4} = +\frac{8}{100} wl^2 = +40,100$
$M_{m.1.5} = +\frac{2.5}{100} wl^2 = +12,500$
$R_1 = R_4 = +\frac{4}{10} wl = +7,400^{\#}$
$R_2' = R_3'' = +\frac{6}{10} wl = +11,100^{\#}$ Max end shear.
$R_1'' = R_3' = +\frac{5}{10} wl = +7,300$
$\therefore R_2 = R_3 = 11,100 + 9,300 = +20,400^{\#}$

Moments, Reactions and Shears due to weight of web.
$-M_2$ and $-M_3$.

Load no.	load #	1st span $-M_2$ unit load	2nd span $-M_2$ unit load	3rd span $-M_2$ unit load	Total	$-M_2$ ft/l.
W_1	1,370	−0.08	−0.13	+1.035	−0.175	− 240
W_2	1,520	−0.24	−0.36	+0.10	−0.500	− 760
W_3	1,290	−0.465	−0.60	+0.175	−0.890	−1,150
W_4	1,110	−0.60	−0.71	+0.21	−1.100	−1,220
W_5	990	−0.75	−0.78	+0.24	−1.290	−1,280
W_6	890	−0.88	−0.80	+0.255	−1.455	−1,300
W_7	840	−0.975	−0.78	+0.26	−1.495	−1,250
W_7'	840	−1.02	−0.715	+0.245	−1.490	−1,250
W_6'	890	−1.02	−0.62	+0.225	−1.415	−1,260
W_5'	990	−0.975	−0.52	+0.20	−1.245	−1,280
W_4'	1,110	−0.84	−0.40	+0.16	−1.080	−1,200
W_3'	1,290	−0.68	−0.275	+0.115	−0.840	−1,080
W_2'	1,520	−0.41	−0.13	+0.07	−0.470	720
W_1'	1,370	−0.11	−0.03	+0.03	−0.110	− 150
	16,020					−14,140

Mean wt. of web = 16,020 / 27 = 500#/ft. of span.

$-M_2 = -M_3 = -14,140 \times 27/10 = -38200'^{\#}$

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Ohata DATE 13-10-1 FILE NO.
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CALCULATIONS FOR

Muko-Okashi over Mukogawa, Hyogoken.
Fascia Girder continued.
Positive Moments. (due to wt. of Stem of beam only)

+ Mm 0.4.

Load no.	Load	1st. Span +Mm.4 unit	2nd Span Mm.4 unit	3rd Span +Mm.4 unit	Total	+Mm.4 10%
W1	1,370	+.14+.06	-.04-.01	+.015+.01	+.175	+240
W2	1,520	+.44+.21	-.14-.06	+.04+.03	+.52	+790
W3	1,290	+.71+.44	-.24-.12	+.07+.05	+.91	+1,170
W4	1,110	+1.23+.62	-.28-.16	+.085+.06	+1.555	+1,730
W5	940	+1.6+.86	-.31-.20	+.095+.08	+2.125	+2,100
W6	890	+1.48+1.15	-.32-.25	+.10+.09	+2.75	+2,450
W7	840	+1.74+1.46	-.31-.29	+.10+.095	+2.795	+2350
						+10,830

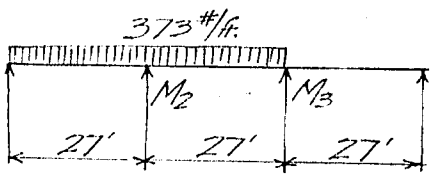
+ Mm.4 = + 10,830 × 27/10 = +29200 '#

+ Mm 1.5

		1st. Span +Mm1.5 unit	2nd. Span Mm1.5 unit	3rd. Span Mm1.5 unit	Total	+Mm1.5 10%
W1	1,370	-.03-.05	+.06+.06	-.05-.03	-.04	- 50
W2	1,520	-.045-.16	+.22+.22	-.16-.045	-.07	- 110
W3	1,290	-.175-.27	+.46+.46	-.27-.175	+.03	+ 40
W4	1,110	-.27-.32	+.66+.66	-.32-.22	+.24	+270
W5	940	-.29-.36	+.92+.92	-.36-.29	+.54	+530
W6	890	-.33-.35	+1.22+1.22	-.35-.33	+1.02	+910
W7	840	-.365-.38	+1.58+1.58	-.38-.365	+1.67	+1400
						+2940

+ Mm1.5 = + 2,940 × 27/10 = +5,080 '#

Live Load Moments, Shears and Reactions of Fascia Girder.
- M2 and - M3.



Neg. M2 = M3 = -0.1 l w × (Neg. area of influence diagram for moment (at 10 'span)) × 6/10
= -27 × 373 × (.26+.51+.73+.9+1.0+1.02+.95+.77+.45
+.39+.64+.77+.80+.77+.64+.49+.32+.15) × 27/10
= -1,008 × (6.58+4.97) × 2.7 = -31,600 '#

+ M2 and + M3.
3rd span loaded.

Pos. M2 = M3 = +1,008 × (.11+.19+.24+.26+.25+.23+.19+.13+.07) × 6/10
= +1,008 × 1.67 × 2.7 = + 4,600 '#

Pos. moment at 0.4 l in the end span + Mm.4 = +1,008 × (.49+1.0+1.51+2.04+1.6+1.19+.82+.49+.21
+ Mm .4; 1st. and 3rd span loaded. +.04+.08+.09+.1+.1+.09+.07+.06+.03) × 2.7
= +1,008 × 10.01 × 2.7 = + 27300 '#

Neg. Moment at 1.5 l in the centre span. -Mm 1.5 = -1,008 × (.1+.19+.27+.33+.37+.38+.36+.29+.17) × 2 × 2.7
- Mm 1.5. loading same as before. = -1,008 × 4.92 × 2.7 = - 13,400 '#

Pos. Moment at 1.5 l in the centre span + Mm1.5 = +1,008 × (.23+.52+.87+1.28+1.75+1.28+.87+.52+.23) × 2.7
+ Mm1.5; Centre span loaded. = +1,008 × 7.55 × 2.7 = +20,600 '#

Neg. moment at 0.4 l in the end span -Mm.4 = -1,008 × (.16+.26+.31+.32+.3+.25+.19+.13+.06) × 2.7
- Mm.4 Centre span loaded = -1,008 × 1.98 × 2.7 = - 5390 '#

Shears and Reactions.
+ R1, R4 and V1R, V4L
1st and 3rd span loaded. + R1 = R4 = + V1R = V2L = +1,008 × (.5+.87+.75+.62+.5+.4+.3+.21+.13+.06
+.01+.02+.025+.03+.03+.02+.02+.018+.01)
= +1,008 × 4.53 = + 4,600 #

+ R2 and V2L, V2R, V3R, V3L
1st and 2nd span loaded + R2 = R3 = +1,008 × (.16+.32+.46+.60+.72+.83+.92+.97+1.0+.5
+.5+.96+.9+.81+.7+.58+.45+.32+.2+.09)
= +1,008 × (6.48+5.51) = +12,100 #

* V2L = -V3R = -1,008 × 6.48 = -6,500 #
V2R = -V3L = +1,008 × 5.51 = + 5,600 #

- R2 3rd span loaded. - R2 = -1,008 × (.07+.11+.14+.16+.15+.14+.11+.07+.04)
= -1,008 × 0.99 = -1,000 #

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Iraba DATE 13-10-9 FILE NO
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CALCULATIONS FOR

Mukko-Okashi over Mukogawa Hyogoken.
Garcia Girder continued.
Shears and Reactions due to weight of stem.
$+R_1$, $+R_4$, $+V_{1R}$ and $-V_{4L}$

		1st. span		2nd span		3rd span.			
load no.	load	$+R_1$	Unit load	$+R_1$	unit load	$+R_1$	unit load	Total	$+R_1$
W1	1,370	+.96	+.015	−.01	−.005	+.005	−.002	+.967	+1,320
W2	1,520	+.88	+.055	−.035	−.010	+.01	+.008	+.908	+1,380
W3	1,290	+.775	+.11	−.06	−.025	+.02	+.015	+.835	+1,080
W4	1,110	+.70	+.125	−.07	−.04	+.024	+.02	+.759	+840
W5	940	+.61	+.22	−.075	−.055	+.025	+.022	+.747	+740
W6	840	+.52	+.29	−.08	−.065	+.025	+.025	+.715	+640
W7	840	+.44	+.37	−.075	−.07	+.025	+.025	+.715	+600
	8,010								+6600

$+R_1 = +R_4 = +V_{1R} = -V_{4L} = 6600$ #

$V_{2L} = -8,010 + \frac{-38,200}{27} = -9,430^{\#} = -V_{3R}$

$V_{2R} = +8,010^{\#} = -V_{3L}$

$+R_2$ and $+R_3$

	1st. span		2nd. span		3rd. span			
	R_2 unit load.		R_2 unit load.		R_2 unit load		Total	R_2.
W1	+.05	+1.00	+.99	+.01	−.02	−.015	+2.015	+2,760
W2	+.15	+1.00	+.965	+.09	−.06	−.035	+2.11	+3,210
W3	+.28	+.98	+.91	+.18	−.105	−.07	+2.175	+2,810
W4	+.37	+.95	+.86	+.30	−.13	−.09	+2.26	+2,510
W5	+.49	+.90	+.785	+.34	−.17	−.11	+2.235	+2,210
W6	+.59	+.84	+.71	+.43	−.175	−.13	+2.265	+2,020
W7	+.69	+.76	+.61	+.53	−.175	−.15	+2.265	+1,910
								+17430

$+R_2 = +R_3 = +17430$ #

Summary for Moments, Reactions and Shears (Garcia Girder).

	Moments						Reactions				Shears.			
	At support		Side span K=.4		Centre span K=1.5		Side span		centre span		Side span		Centre span	
	$+M_2+M_3$	$-M_2,-M_3$	$+M_{m.4}$	$-M_{m.4}$	$+M_{1.5}$	$-M_{m.15}$	$+R_4,+R_4$	$-R_1,-R_4$	$+R_2,+R_3$	$-R_2,-R_3$	V_{1R}	V_{2L}	V_{2R}	V_{3L}
Dead Load except wt. of web.	—	−50,200	+40,100	—	+12,500	—	+7,400	—	+20,400	—	+7,400	−11,100	+9,300	−9,300
wt. of web.	—	−38,200	+29,200	—	+8,080	—	+6,600	—	+17430	—	+6,600	−9,430	+8,010	−8,010
Live Load	+4,600	−31,600	+27,300	−5,390	+20,600	−13,400	+4,600	Very small	+12,100	−1,000	+4,600	−6,500	+5,600	−5,600
Total	+4,600	−120,000	+96,600	−5,390	+41,200	−13,400	+18,600	"	+49,900	−1,000	+18,600	−27,000	+22,900	−22,900

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MADE BY K. Inaba DATE 13-IX-9 10-10 FILE NO

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CALCULATIONS FOR

Mutsu-Okashi over Mutsugawa, Hyogoken.

Equivalent Uniform Loads.

Dead Load.

Beam under railway. $g_r = 1,280$ #/ft. of span $g_r l^2 = 934,000$ ft#
Beam under trafficway. $g_t = 800$ " " $g_t l^2 = 583,000$
Fascia girder $g_f = 593+11 = 540 + 688 = 1228$ $g_f l^2 = 895,000$

Live Load.

Beam under railway.
Side span neg. moment. Concentration { 17200 # / 10700 / 3600

$(-)M_2 = -123,200$ ft#
$(+)M_2 = +19,800$ "

for Equivalent unif. load $M = 0.11667\, pl^2$
∴ Equiv. unif. load for $(-)M_2$, $P = \frac{123200}{0.11667 \times 27^2} = 1,450$ #/ft.run
" " " $(+)M_2$, $P'' = \frac{19800}{.11667 \times 27^2} = 1,630$ "

Positive moment at 0.4l from left support.
$+)M_{m0.4} = +186,800$ ft#
for equiv. unif. load $M = 0.10\, pl^2$
∴ Equiv. unif. load $P = \frac{186800}{0.10 \times 27^2} = 2,560$ #/ft.run

Neg. moment at 0.4l from left support.
$(-)M_{m0.4} = -23,700$
for Equiv. unif. load $M = 0.02\, pl^2$
∴ Equiv. unif. load $P'' = 23700/.02 \times 27^2 = 1,625$ #/ft.run

Centre span negative moment at 1.5 l from left End support.
$(-)M_{m1.5} = -49,800$ for equivalent unif. load $M = 0.05\, pl^2$
∴ Equivalent unif. load $P' = \frac{49,800}{0.05 \times 27^2} = 1,367$ #/ft. of span.

Positive moment at 1.5 l from left end support.
$(+)M_{m1.5} = +87,200$ ft#
for equiv. unif. load, $M = 0.075\, pl^2$
∴ Equivalent unif. load $P' = \frac{87200}{.075 \times 27^2} = 1,595$ #/ft. run of span.

Beam under trafficway.
End span negative moment.
$(-)M_2 = -85,000$ ft# Concentration { 19400 # / 6,500
End span M_2 position.
$(+)M_2 = +15,900$ ft#
for Equivalent unif. load, $M = 0.11667\, pl^2$
∴ Equiv. unif. load for $(-)M_2$, $P = \frac{85000}{0.11667 \times 27^2} = 998$ #/ft. run.
" " " $(+)M_2$, $P'' = \frac{15900}{.11667 \times 27^2} = 1,308$ "

Pos. moment at 0.4 l from left support.
$(+)M_{m0.4} = +116,500$ ft# for equiv. unif. load $M = 0.10\, pl^2$
∴ Equiv. unif. load $P = \frac{116500}{0.10 \times 27^2} = 1,598$ #/ft.

$(-)M_{m.4} = -22,400$ ft# for equiv. unif. load $M = 0.02\, pl^2$
∴ Equiv. unif. load $P'' = \frac{22,400}{0.02 \times 27^2} = 1,540$ #/ft.run.

Center span neg. moment at 1.5 l from left end support.
$(-)M_{m1.5} = -38,100$ ft# for equiv unif. load, $M = 0.05\, pl^2$
∴ Equiv. unif. load $P' = 38100/0.05 \times 27^2 = 1,045$ #/ft.run.
Pos. moment at 1.5 l from left end support. $(+)M_{m1.5} = +96,200$ ft# for equiv unif. load, $M = 0.075\, pl^2$
∴ Equiv. unif. load $P' = 96,200/0.075 \times 27^2 = 1,760$ #/ft. run.

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（独立行政法人 土木研究所蔵）

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ALCULATIONS FOR

Mutsu-Okashi over Mutsugawa, Aomori-ken.
Equivalent Uniform Load Continued.
Fascia Girders.

Uniform live load of 373 #/ft. of span.

Summary of Equivalent Uniform Loads.

	Equivalent unif load. End Span			Centre Span			End Span			Centre Span.		
	$-M_2$	$+M_2$	$+M_{m.4}$	$-M_{m.4}$	$-M_{m.5}$	$+M_{m.5}$	$P\cdot l^2$	$P''l^2$	$P+l^2$	$P''l^2$	$P'l^2$	$P+l^2$
Beam under railway.	1,450	1,630	2,560	1,625	1,367	1,595	1,058,000	1,189,000	1,866,000	1,185,000	997,000	1,163,000
Beam under Trafficway.	998	1,308	1,548	1,540	1,045	1,760	728,000	955,000	1,165,000	1,123,000	763,000	1,283,000
Fascia girder.	373	373	373	373	373	373	272,000	272,000	272,000	272,000	272,000	272,000
	P	P''	P+	P''	P'	P+						

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CALCULATIONS FOR

Mukō-Okashi over Mukōgawa, Ikyogoku.
Moment at several Points throughout Beams (Calculated by Equiv. uniform. load.)

		Dead Load Moment			Live Load Negative M.				Live Load Positive M.			
	x/l	Coef. of gl^2	y_f=1,260	y_f=800	y_f=1,228	Coef. of $P''l^2$	P=1450 P'=1,367 P''=1625	P=998 P'=1045 P''=1540	P=373	Coef. of $P_f l^2$	P_f=2560 P_f'=1,595 P_f''=1630	P_f=1,598 P_f'=1,760 P_f''=1308	P_f=373 P_f'=373 P_f''=373
End Span.	0.10	+0.0359	32,700	20,400	31,300	0.005	− 5,900	− 5,600	− 1,400	0.040	74,600	46,600	10,900
	0.20	+0.060	56,000	35,000	53,700	0.010	−11,900	−11,200	− 2,700	0.070	131,000	81,600	19,100
	0.30	+0.075	70,000	43,700	67,100	0.015	−17,500	−16,400	− 4,100	0.090	168,000	104,900	24,500
	0.40	+0.080	74,700	46,700	71,600	0.020	−23,700	−22,400	−5,400	0.100	186,800	116,500	27,300
	0.50	+0.075	70,000	43,700	67,100	0.025	−31,600	−28,100	−6,800	0.100	186,800	116,500	27,300
	0.60	+0.060	56,000	35,000	53,700	0.030	−35,500	−33,700	−8,200	0.090	168,000	104,900	24,500
	0.70	+0.035	32,700	20,400	31,300	0.035	−41,500	−39,300	−9,500	0.070	131,000	81,600	19,100
	0.80	0.000	0	0	0	$0.0422\,Pl^2$	−42,500	−29,200	−10,400	0.0422	75,000	46,800	10,900
	0.85	−0.0125	−19,400	−12,400	−14,000	0.0488	−51,800	−35,600	−13,300	$0.02773\,P_f''l^2$	33,000	26,500	7,600
	0.90	−0.0450	−42,000	−26,200	−40,300	0.06542	−69,200	−47,600	−17,500	0.03042	24,300	14,800	5,600
	0.95	−0.07125	−66,500	−41,500	−63,800	0.08531	−93,400	−64,300	−24,500	0.01706	20,300	16,300	4,600
	1.00	−0.1000	−93,400	−58,300	−89,500	0.11667	−123,200	−85,000	−31,600	0.01667	19,800	15,900	4,600
Centre Span.	1.05	−0.07625	−71,200	−44,500	−68,200	0.04033	−95,500	−65,700	−24,300	0.01408	16,700	13,400	3,800
	1.10	−0.0550	−51,300	−32,100	−49,200	0.07014	−74,100	−51,000	−19,100	$0.01514\,P_f'l^2$	17,600	19,400	4,100
	1.15	−0.03625	−34,900	−21,200	−32,400	0.05018	−60,100	−41,300	−15,400	0.02053	23,900	26,300	5,600
	1.20	−0.0200	−19,700	−11,700	−17,900	$0.0500\,P'l^2$	−49,800	−35,100	−13,400	0.0300	34,900	38,500	8,200
	1.2764	0.0000	0	0	0	0.0500	−49,800	−35,100	−13,400	0.0500	55,200	64,200	13,600
	1.30	+0.0050	+4,700	+2,900	+4,500	0.0500	−49,800	−35,100	−13,400	0.0550	64,000	70,600	15,000
	1.40	+0.0300	+18,700	+11,700	+17,900	0.0500	−49,800	−35,100	−13,400	0.0700	81,500	84,800	19,100
	1.50	+0.0250	+23,400	+14,600	+22,400	0.0500	−49,800	−35,100	−13,400	0.0750	87,200	96,200	20,600
			gl^2=934,000	gl^2=583,000	gl^2=895,000		$P'l^2$=1,058,000 $P_f'l^2$=997,000 $P_f''l^2$=1,185,000	$P''l^2$=763,000 $P''l^2$=1123,000	Pl^2=272,000		$P_f l^2$=186,000 $P_f'l^2$=116,300 $P_f''l^2$=1184,000	$P_f l^2$=116,500 $P_f'l^2$=1,082,000 $P_f''l^2$=925,000	$P_f l^2$=272,000 $P_f'l^2$ = $P_f''l^2$ =
			Railway	Trafficway	Fascia girder		Railway	Trafficway	Fascia girder		Railway	Trafficway	Fascia girder

Summary of Dead and Live Load Moment.

x/l	Beams under Electric Ry.		Beams under Trafficway		Fascia Girders.	
	Negative	Positive	Negative	Positive	Negative	Positive
0.10	−26,800	107,300	14,800	67,000	−29,900	42,200
0.20	44,100	187,000	23,800	116,600	−51,000	72,800
0.30	−52,200	238,000	26,500	148,600	−63,000	91,600
0.40	−51,000	261,500	24,700	163,700	−66,200	98,900
0.50	−40,400	256,500	13,600	160,700	−60,300	94,400
0.60	−20,500	224,000	1,300	134,900	−45,500	75,200
0.70	− 8,800	163,700	−7,800	102,600	−21,800	50,400
0.80	−42,500	75,000	−29,200	46,800	−10,900	10,900
0.85	− 71,700	13,100	−48,000	14,100	−32,300	−11,400
0.90	−111,200	− 17,700	−73,800	− 6,700	−58,100	−34,700
0.95	−159,900	−46,200	−105,800	−25,200	−87,500	−59,200
1.00	−216,600	−73,600	−143,300	−42,400	−121,100	−84,400
1.05	−166,700	−74,500	−110,200	−31,100	−97,500	−64,400
1.10	−125,400	−33,700	− 83,100	−12,700	− 68,300	−45,100
1.15	− 95,000	−11,000	− 62,500	5,100	−47,500	−26,500
1.20	− 68,500	+16,200	− 49,800	26,800	−31,300	− 7,700
1.2764	− 49,800	+58,200	− 35,100	64,200	−13,400	+13,600
1.30	− 45,100	+65,700	− 35,200	73,500	− 8,400	+19,500
1.40	− 31,300	+100,200	−26,400	101,500	+ 4,500	+37,000
1.50	− 26,400	+110,600	−23,500	110,500	+ 4,000	+43,000

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（独立行政法人 土木研究所蔵）

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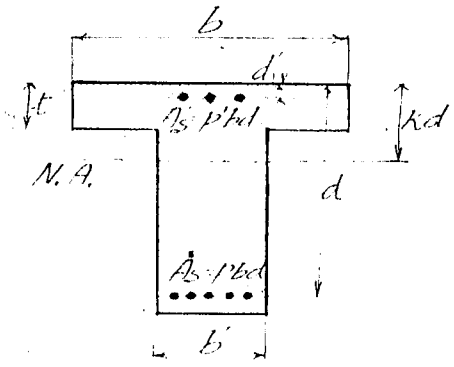
CALCULATIONS FOR

Mishi-Okashi over the Mukogawa Myogi-kau.
Cross Sections for Main Girders.
Formulas for Reinforced Concrete T-Beams



T-Beam with Steel in Top and Bottom.

$$\Delta = t/d.$$

$$K = \frac{p + p'\left(\frac{d'}{d}\right) + \frac{\Delta^2}{2n}}{p + p' + \frac{\Delta}{n}}$$

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

$$f_s = \frac{M}{A_s j d} = \frac{M}{pjbd^2}$$

$$f_c = \frac{f_s K}{n(1-K)}$$

$$f_s' = f_s \frac{K - \frac{d'}{d}}{1-K}$$

$$f_s = \frac{f_c n(1-K)}{K}$$

$$M_s = bd^2 f_s p j'$$

Formulas for determining percentages of Steel in Double-Reinforced concrete T-Beams.

$$\Delta = t/d.$$

$$K = \frac{1}{1 + \frac{f_s}{n f_c}}$$

$$p_1 = \Delta \frac{f_c}{f_s}\left(1 - \frac{\Delta}{2}\right) - \frac{\Delta^2}{2n}$$

$$j' = \frac{6 - 6\Delta + 3\Delta^2 + \Delta^3\left(\frac{1}{3n}\right)}{6 - 3\Delta}$$

$$M_1 = f_s \cdot p_1 j' b d^2$$

$$M_2 = M - M_1$$

$$p_2 = \frac{M_2}{f_s\left(1 - \frac{d'}{d}\right) b d^2}$$

$$p = p_1 + p_2$$

$$p' = p_2 \frac{1-K}{K - \frac{d'}{d}}.$$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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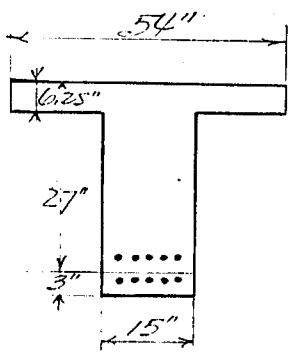
CALCULATIONS FOR

Mukko-Okashi over the Bun-kogawa, Kyogoku.
Beams under Electric Railway.



Section at 0.4 l.
Max. moment.

Max. moments and Shears.

	at C.	at 0.4 l	at 1.0 l	at 1.5 l.
Moment	—	+261,500	−216,600	+110,600
Shear.	+41,500	—	−53500, 40700	—

$f_s = 17{,}000$ #/□", $f_c = 640$ #/□", $n = 15$

$\Delta = t/l = 6.25/27 = 0.231$

$$K = \frac{1}{1+\frac{f_s}{n f}} = \frac{1}{1+\frac{17000}{15\times640}} = 0.361$$

$$p = \Delta \frac{f_c}{f_s}\left(1-\frac{\Delta}{3}\right) - \frac{\Delta^2}{2n} = 0.231\times\frac{640}{17000}\left(1-\frac{0.231}{2}\right) - \frac{0.231^2}{2\times15} = 0.0059$$

$$j' = \frac{6-6\Delta+2\Delta^2+\Delta^3\left(\frac{1}{2pn}\right)}{6-3\Delta} = \frac{6-6\times0.231+2\times0.231^2+0.231^3\left(\frac{1}{2\times0.0059\times15}\right)}{6-3\times0.231} = 0.903$$

$M_1 = f_s p_1 j' bd^2 = 17000\times0.0059\times0.903\times54\times27^2 = 3{,}570{,}000$

$M_2 = M - M_1 = 1{,}500\times12 - 3{,}570{,}000 = -430{,}000$

$$p_2 = \frac{M_2}{f_s(1-\frac{d'}{d})bd^2} = \frac{-430000}{17000(1-\frac{3}{27})54\times27^2} = -0.00058$$ no top reinf.

$p' = p_1 + p_2 = 0.0059 - 0.0006 = 0.0053$

$$p' = p_2\frac{1-K}{K-\frac{d'}{d}} = -0.00058\frac{1-0.36}{0.36-\frac{3}{27}} = -0.0015$$

$A_s = 54"\times27"\times0.0053 = 7.72$ □" use $10-1"\phi = 7.854$ □"

$$f_s = \frac{M}{A_s jd} = \frac{261500\times12}{7.854\times0.903\times27} = 16400$$ #/□" OK.

$$f_c = \frac{f_s K}{n(1-K)} = \frac{16400\times0.36}{15(1-0.36)} = \frac{16400\times0.36}{15\times0.64} = 615$$ #/□" OK.

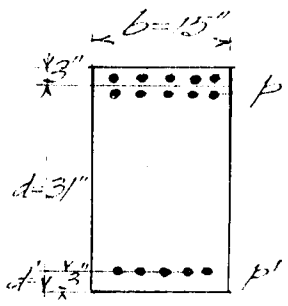
Max. Shear $V_2 l = -53500$# take d = 31" (4" chamfer)

Unit Shear $v = \frac{V}{bjd} = \frac{53500}{15\times0.903\times31} = 127$ #/□" Web reinf. required.

Unit bond $u = \frac{V}{\Sigma ojd} = \frac{53500}{3.1416\times10\times0.903\times31} = 61$ #/□" OK.

required no. of bars for bond $= 10\times\frac{6'}{25} = 7.2$ bars for plain bar.
$= 10\times\frac{6'}{25} = 4.8$ " " deformed bar.

Section at 1.0 l.
Negative moment.



$M_2 = -216{,}700$#

Double reinforced rectangular beam.

$d'/d = 3/31 = 0.097$ assume $p' = 0.5p$.

assume $10-1"\phi$ bars on top and $5-1"\phi$ bars on bottom.

$p = \frac{7.854}{15\times31} = 0.0169$

$p' = \frac{3.93}{15\times31} = 0.0085$

$K = 0.436$, $j' = 0.869$

$$f_s = \frac{M}{A_s jd} = \frac{216700\times12}{7.854\times0.869\times31} = 12{,}300$$ #/□" OK.

$$f_c = \frac{f_s K}{n(1-K)} = \frac{12300\times0.436}{15(1-0.436)} = 634$$ #/□" OK.

At end support.

$V = 41{,}500$#

unit $v = \frac{41500}{15\times7/8\times31} = 102$ #/□" Web reinforcement required.

Unit $u = \frac{41500}{3.1416\times7/8\times31} = 111$ #/□" for one bar.

required no. of bars for bond $= 4.8/.85 = 5.75$ for plain bar.
$= 4.8/1.28 = 3.81$ " deformed bar
(at end support)

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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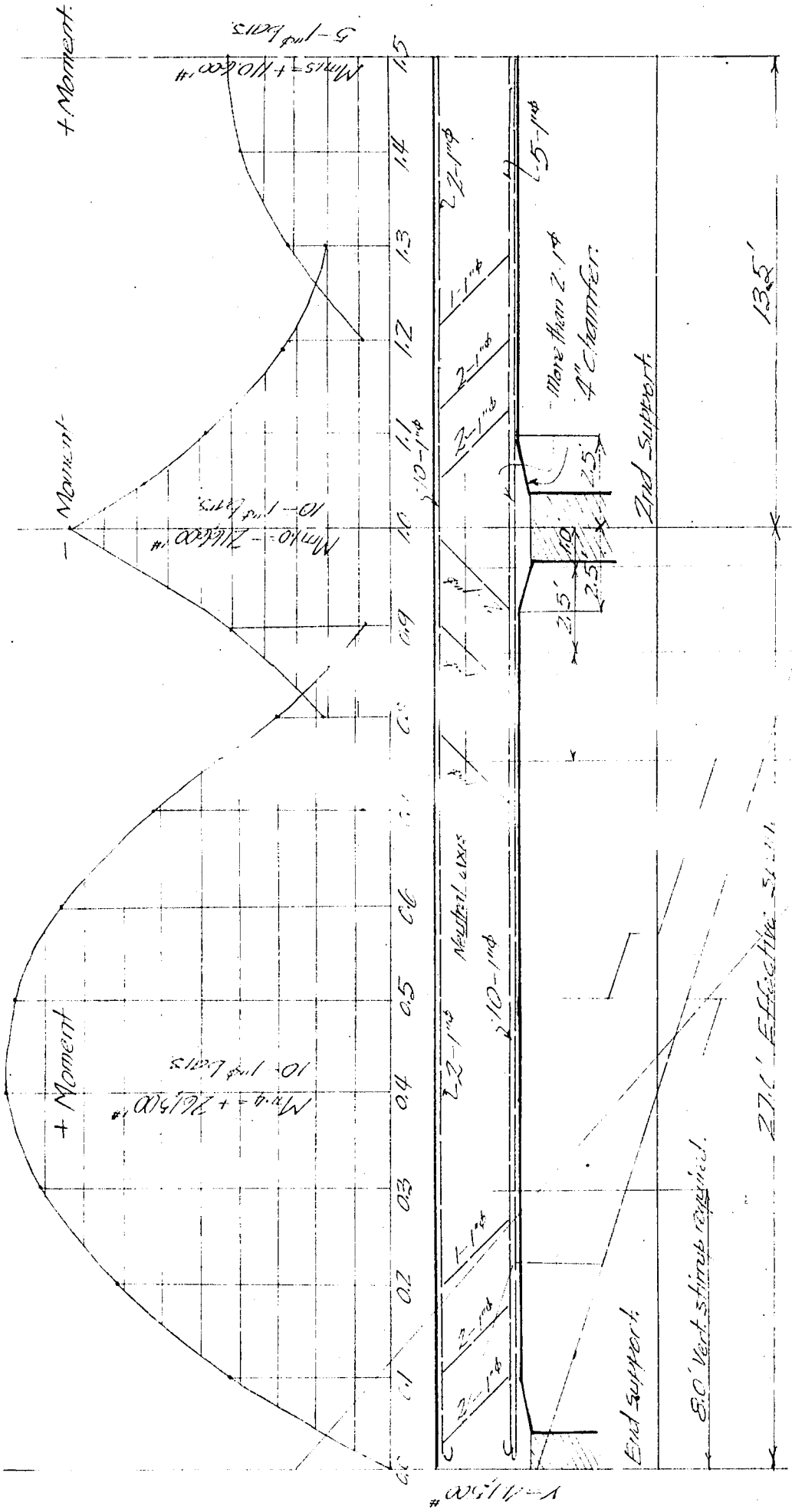
CALCULATIONS FOR

Muko-Gawa Bridge over the Mukogawa, Hyogo-Ken.
Details of Reinforcements for Beams under Railway (Beam E & F).



Bent up bars.
(1) 2-1" bars S=2.0'
$6Z \times 1.5 = 930$ #/sq.in. per sq. in. bar.
$P = 0.7 \frac{V}{S} = 0.7 \frac{53.5 \times 0.7}{2.0} = 15.00$ #
for 1 sq.in. 7500 #.
(2) 2-1" bars S=3.0'
$P = 4 \times 1.5 \times 36 \times 0.7 \times \frac{1}{2} = 7750$ # in one bar.
(3) 1-1" bars S=3.3'
$P = 14 \times 1.5 \times 40 \times 0.7 = 5900$ #

For 1" bars Max. Section on the bar $= \frac{2800}{3.85 \times} = 995$ #/sq.in. OK

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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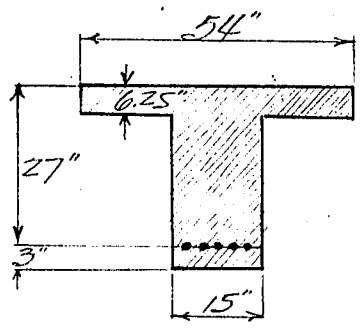
CALCULATIONS FOR

Mutsu-Ohashi over the Mutsugawa, Hyogo-Ken.
Beams under Electric Railway continued.
Section at centre of the centre span. (1.5 l.)
M = +110,600'# t/d = 6.25/27 = 0.232



Steel area required $A_s = \frac{110600 \times 12}{17000 \times 7/8 \times 27} = 3.32^{0"}$

Assume 5-1"ϕ bars = $3.93^{0"}$

$p = \frac{3.93}{54 \times 27} = 0.0027$

$K = 0.245$, $j = 0.918$

$f_s = \frac{110,600 \times 12}{3.93 \times 0.918 \times 27} = 13,650^{\#/0"}$ Ok

$f_c = \frac{f_s k}{n(1-k)} = \frac{1350 \times 0.245}{15(1-0.245)} = 295^{\#/0"}$ Ok.

Vertical Stirrups
At 2nd Support.
V = 41,500 # x = 8.0'; Distance from support between which stirrups are necessary.
Assuming the web reinforcement to carry #/3 of the total shear and the remaining 1/3 to be carried by the concrete.
Use 1/2"ϕ U shaped vertical stirrups. $A_s = 2 @ 0.1963 = 0.393^{0"}$

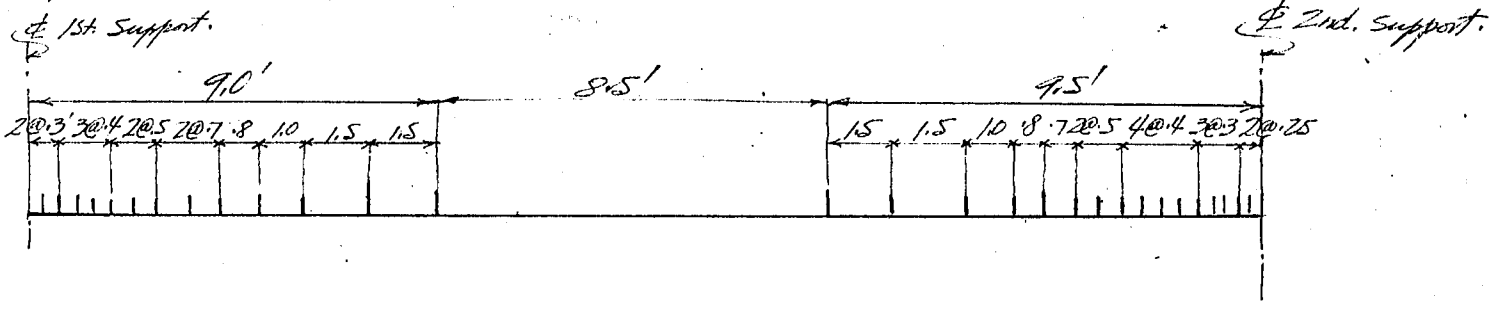
Min. spacing $S = \frac{3}{2} \frac{A_s f_s j d}{V} = \frac{3 \times 0.393 \times 11000 \times 7/8 \times 31}{2 \times 41,500} = 4.25"$ say 4" spacing

Dist. from support at which
S = 5", $x' = 8 - 8 \times \frac{4.25}{5} = 8 - 8 \times \frac{4.25}{5} = 1.20'$
S = 6", $x'' = 8 - 8 \times \frac{4.25}{6"} = 2.50'$
S = 9" x" = 4.20'
S = 12" x"' = 5.2'
S = 18" x"" = 6.1'

At 2nd Support.
V = 53,500 # x = 8.3'.
use 1/2"ϕ U-stirrup.
Min. spacing $S = \frac{3 \times 0.393 \times 11,000 \times 7/8 \times 31}{2 \times 53,500} = 3.35"$ say 3" spacing

for S = 4" $x' = 8.3 - 8.3 \times \frac{3.35}{4} = 1.35'$
S = 5" x" = 2.25'
S = 6" x"' = 3.65'
S = 9" x"" = 5.2'
S = 12" x""' = 6.0'
S = 18" x"" = 6.75'

Stirrup Spacing.



Negative moment at 1.5 l
M = − 26,400 '#
Steel area required = $\frac{26,400 \times 12}{17,000 \times 7/8 \times 27} = 0.79^{0"}$
use 2-1"ϕ = $1.57^{0"}$ Ok.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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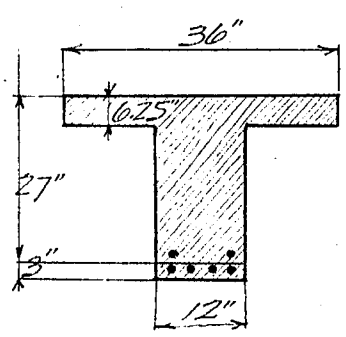
CALCULATIONS FOR

Mukko-Gawa over the Mukogawa, Hyogoken.

Section of Beams under Trafficway (Beams A, B, C and D)

Maximum Stresses:—



	at 0.0	0.4 l.	1.0 l.	1.5 l.
Moment	—	+163,200'#	−143,300'#	+110,800'#
Shear	+31,600#	—	−38,400, +34,900	—

Pos. moment at 0.4 l.

$M = +163{,}200'^{\#}$ $t/d = 6.25/27 = 0.232$

try 6–1"φ bars = 4.720"

$p = \frac{4.72}{36\times27} = 0.00485$

$K = 0.326$, $j = 0.906$

$f_s = \frac{163{,}200\times12}{4.72\times0.906\times27} = \underline{16{,}980}\ ^{\#}/_{\square"}$ Ok.

$f_c = \frac{16{,}980\times0.326}{15(1-0.326)} = \underline{548}\ ^{\#}/_{\square"}$ Ok.

Pos. moment at 1.5 l

$M = +110{,}800'^{\#}$

steel required $= \frac{110{,}800}{17{,}000\times7/8\times27"} = 3.30"$

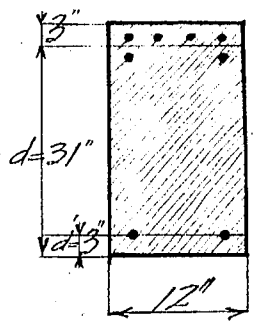
use 4–1"φ bars = 3.142"

$p = \frac{3.142}{36\times27} = 0.00324$

$K = 0.268$, $j = 0.915$

$f_s = \frac{110{,}800\times12}{3.142\times0.915\times27} = \underline{17{,}070}\ ^{\#}/_{\square"}$ Ok.

$f_c = \frac{17{,}070\times0.268}{15(1-0.268)} = \underline{418}\ ^{\#}/_{\square"}$ Ok



Neg. moment at 2nd. support (1.0 l)

$M = -143{,}300'^{\#}$

double reinforced rectangular beam.

Effective depth req'd for shear $= \frac{38400}{12\times7/8\times120^{\#}/_{\square"}} = 30.5"$ say 31"

steel required $= \frac{143{,}300\times12}{17000\times7/8\times32} = 3.61'^{\square"}$

use 6–1"φ bars = 4.72"

$d'/d = 3/31 \div 0.1$, $p' = \frac{1}{3}p$. $p = \frac{4.72}{12\times31} = 0.0127$, $p' = 0.00424$

$K = 0.404$, $j = 0.872$

$f_s = \frac{143300\times12}{4.72\times0.872\times31} = \underline{13{,}500}\ ^{\#}/_{\square"}$ Ok.

$f_c = \frac{13{,}500\times0.404}{15(1-0.404)} = \underline{610}\ ^{\#}/_{\square"}$ Ok.

Unit shear $v = \frac{38.400}{12\times.872\times31} = \underline{118}\ ^{\#}/_{\square"}$ Web reinf. required.

Unit bond $u = \frac{38.400}{3.142\times6\times0.872\times31} = \underline{75.4}\ ^{\#}/_{\square"}$ Ok.

Shear at End support.

$V = +31{,}600^{\#}$

unit shear $v = \frac{31600^{\#}}{12\times0.872\times31} = \underline{97.5}\ ^{\#}/_{\square"}$ Web reinf. required.

No. of bars req'd for bond at end support $= \frac{31600}{3.142\times.872\times3\times.85} = 4.4$ bars for plain bar.

$= \frac{31600}{3.142\times.872\times3\times128} = 2.9$ bars. for deformed bars.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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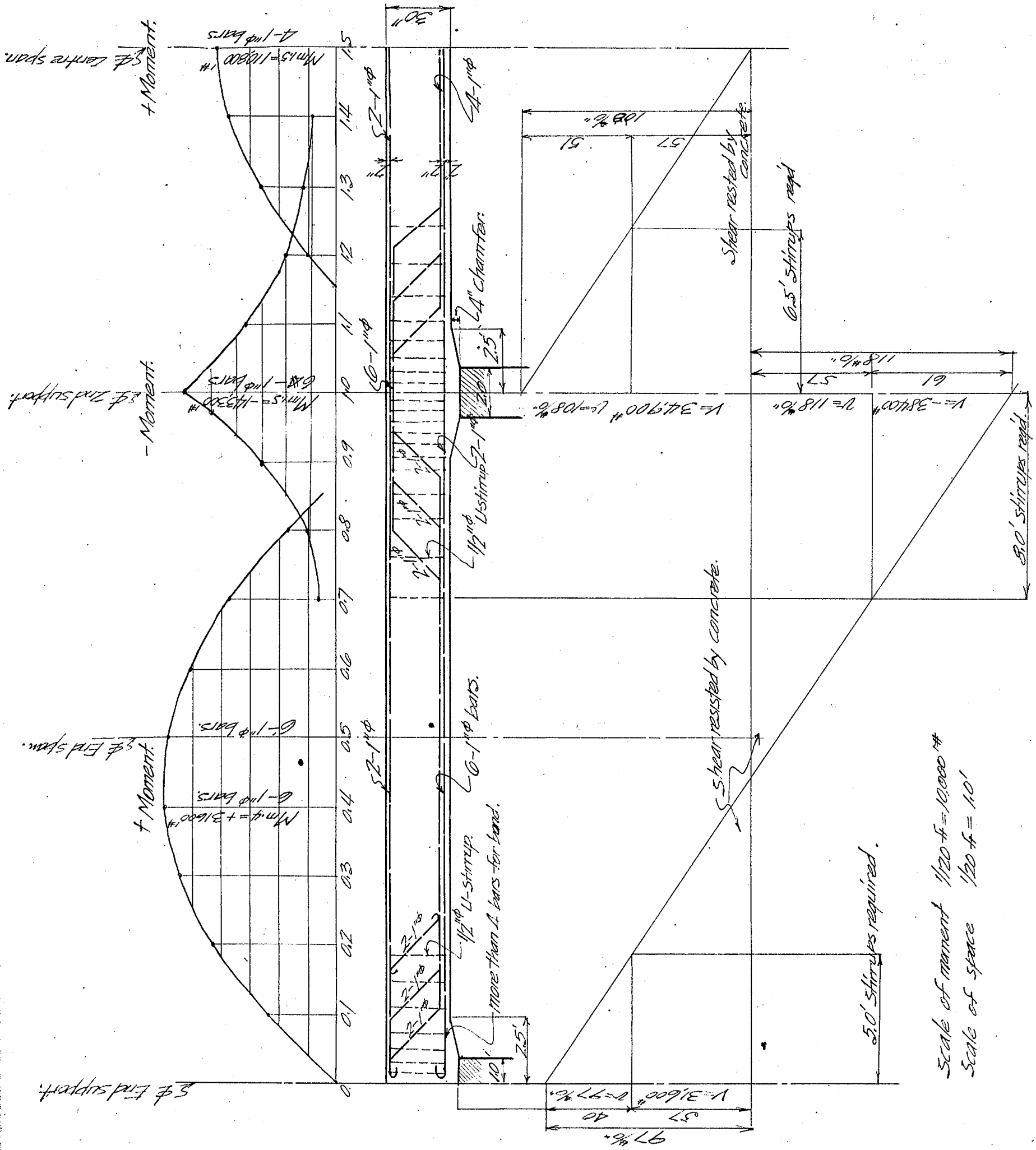
CALCULATIONS FOR

Muko-Ohashi over the Mukogawa, Hyogoken.
Beams under Trafficway continued.
Reinforcements.



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（独立行政法人 土木研究所蔵）

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CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Mutsu-Okashi over the Mutsugawa, Ajigotoku.

Vertical Stirrups for Beams under Traffic way.

At 2nd support. (left side)

$V = -38{,}400^{\#}$ $x = 8.0'$ distance between which vertical stirrups are necessary.

With the same assumptions as before, shear to be carried by steel equals $\frac{2}{3} \times 118 = 79^{\#}/0"$

use 1/2"ϕ U stirrups, $A_s = 2 @ 0.1963 = 0.393\,0"$

minimum spacing of stirrups $= s = \frac{3}{2} \cdot \frac{A_s f_s jd}{V} = \frac{3 \times 0.393 \times 11{,}000 \times 7/8 \times 31}{2 \times 38{,}400} = 4.58"$

Distance from support at which
$s = 5"$ $x' = 8 - 8 \times \frac{4.58}{5} = 0.7'$
$s = 6"$ $x'' = 8 - 8 \times \frac{4.58}{6} = 1.9'$
$s = 9"$ $x''' = \quad = 4.0'$
$s = 12"$ $x'''' \quad = 5.0'$
$s = 18"$ $x''''' \quad = 6.0'$

At 2nd support. (right hand side)

$V = +34{,}900^{\#}$, $x = 6.5'$

use 1/2"ϕ U stirrups $A_s = 2 @ 0.1963 = 0.393\,0"$

Min. spacing of stirrups $s = \frac{3 \times 0.393 \times 11{,}000 \times 7/8 \times 31}{2 \times 34{,}900} = 5.04"$

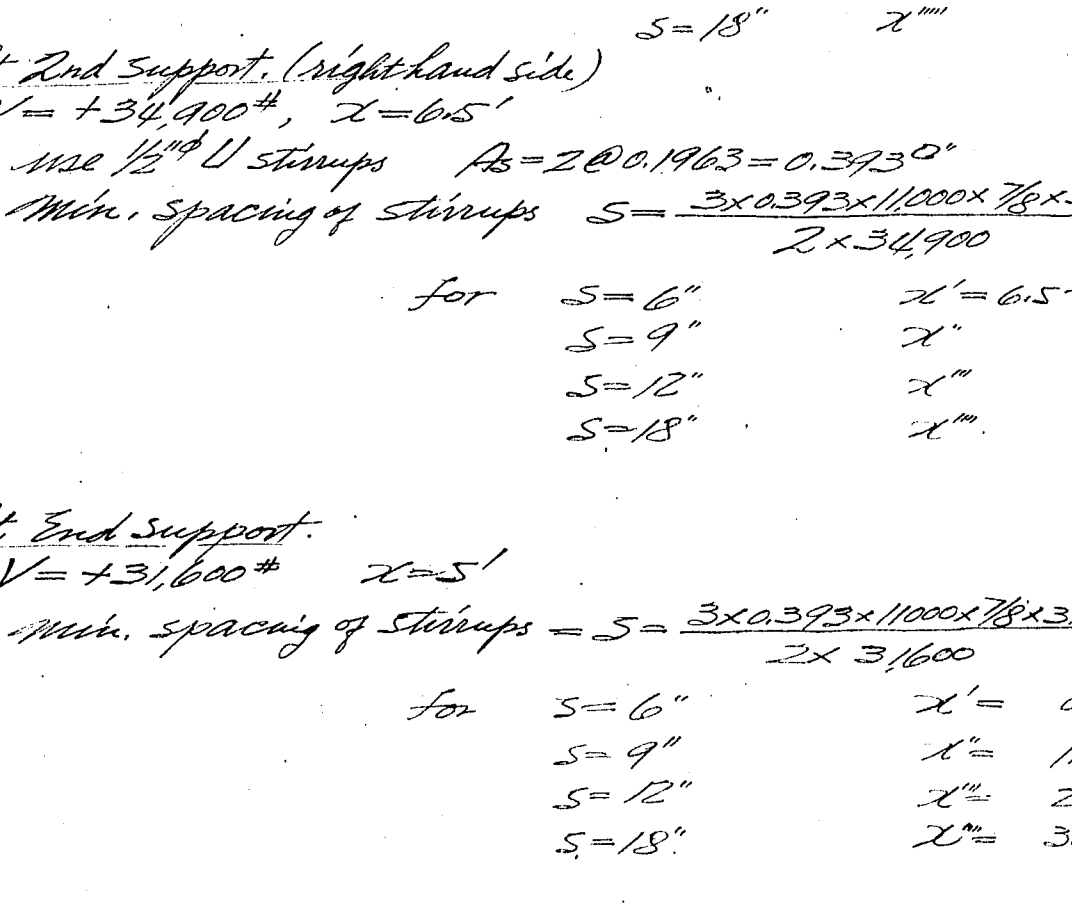
for
$s = 6"$ $x' = 6.5 - 6.5 \times \frac{5.04}{6} = 1.0'$
$s = 9"$ $x'' \quad = 1.9'$
$s = 12"$ $x''' \quad = 3.8'$
$s = 18"$ $x'''' \quad = 4.7'$

At. End support.

$V = +31{,}600^{\#}$ $x = 5'$

min. spacing of stirrups $= s = \frac{3 \times 0.393 \times 11000 \times 7/8 \times 31}{2 \times 31{,}600} = 5.56"$

for
$s = 6"$ $x' = 0.4'$
$s = 9"$ $x'' = 1.9'$
$s = 12"$ $x''' = 2.7'$
$s = 18"$ $x'''' = 3.5'$



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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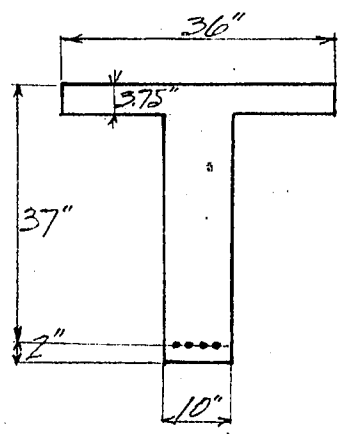
CALCULATIONS FOR

Muko-Ohashi over the Mukogawa, Nyogoku.
Section of Fascia Girder.



Maximum Stresses.

	at 0.0 l	at 0.4 l	at 1.0 l	at 1.5 l
Moments.	——	+98,900 '#	−121,100 '#	+43,000 '#
Shears.	+18,600 #	——	−27,000 #, +22,900 #	——

At 0.4 l.

$M = +98{,}900$ '#

Steel required $= \frac{98900 \times 12}{17{,}000 \times 7/8 \times 37"} = 2.160"$

Use 4 − 7/8"ϕ = 2.40 0"

$t/d = 3.75/37 \fallingdotseq 0.1$

$p = \frac{2.40}{36 \times 37} = 0.00206$

$K = 0.27$, $j = 0.956$

$f_s = \frac{98{,}900 \times 12}{2.40 \times 0.956 \times 37} = 14200$ #/0" Ok.

$f_c = \frac{14200 \times .27}{15(1-.27)} = 350$ #/0" Ok.

At 1.5 l.

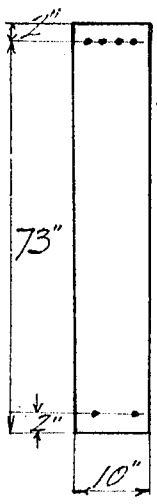
$M = +43000$ '#

Steel req'd $= \frac{43000 \times 12}{17{,}000 \times 7/8 \times 37} = 0.94 0"$

Use 4 − 3/4"ϕ = 1.77 0"

$f_s = \frac{43000 \times 12}{1.77 \times .956 \times 37} = 8270$ #/0" Ok

$f_c = \frac{8270 \times 0.27}{15(1-.27)} = 204$ #/0" Ok.



At 1.0 l

$M = -121{,}100$ '#

Use 4 − 7/8"ϕ = 2.40 0"

$f_s = \frac{121100 \times 12}{2.40 \times 7/8 \times 73} = 9700$ #/0" Ok

$f_c = \frac{9700 \times .437}{15(1-.37)} = 380$ #/0" Ok.

Unit shear $v = \frac{27000}{10 \times 7/8 \times 73} = 42$ #/0" Ok.

Unit bond $u = \frac{27000}{3.75 \times 4 \times 7/8 \times 73} = 39$ #/0" Ok.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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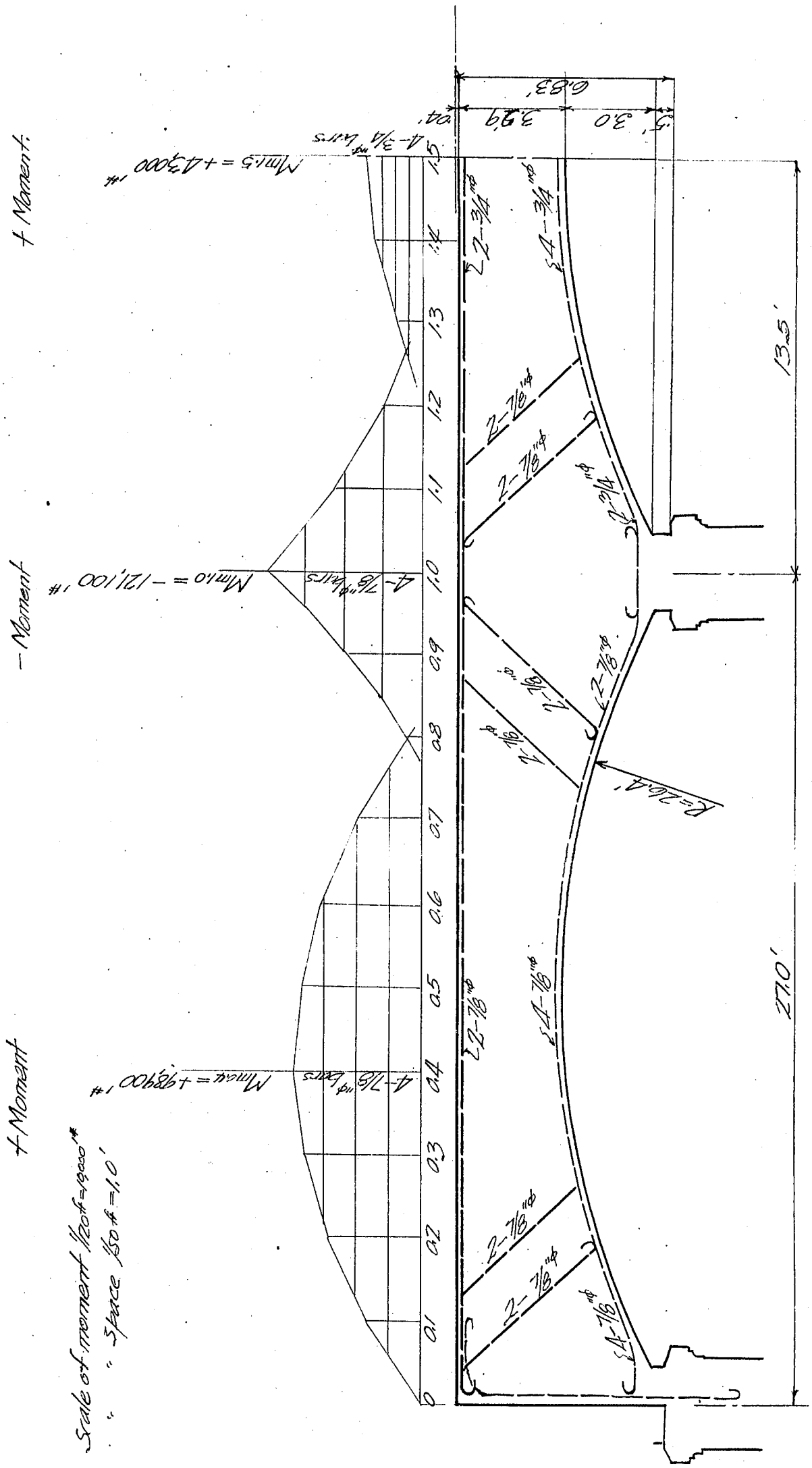
CALCULATIONS FOR

Mukko-Ohashi over the Mukogawa, Hyogo-ken.
Details of Reinforcements for Fascia Girder.



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（独立行政法人 土木研究所蔵）

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MADE BY K. Ikeda DATE 25-1-15 FILE NO
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CALCULATIONS FOR

Mukō-Ohashi on the Meishogawa, Nagoya-shi

Piers for Girder Span.

Centre of gravity of Superstructure (Taking moment about top of Coping of Pier)

Names	Weight	Lever arm	Moment	Remarks
Footpathway & Sidewalk	9800	8.32	53,700	54 @ 181.2 = 9780
Sidewalk pavements	3600	6.73	20,200	" @ 55.8 = 3020
Curbs	2280	6.45	14,200	" @ 40 = 2460
Slab	19000	6.55	120,200	" @ 351 = 18,150
Copings	6500	6.50	42,300	" @ 1205 = 6500
Fascia girders	32,000	4.30	137,500	2×12 @ 573 = 32000
Brackets	3000	5.50	16,600	10 @ 300 = 3000
Gas pipes	2700	5.60	12,300	54 @ 40 = 2160
	77,700		450,400	
Trafficway pavement	18,900	6.5	122,800	54 @ 1×179 @ 22 = 18900
Slab	75,400	6.2	467,000	54×179 @ 78 = 75400
Girders	64,100	4.9	317,000	27×8 @ 300 = 64,700
	158,400		906,800	
Elec. Ry. pavements	26,100	6.6	172,000	27×179 @ 54 = 26100
Slabs	37,700	5.55	209,000	27×179 @ 78 = 37,700
Rails and accessories	4,100	6.37	26,100	27 @ 125 = 4050
Steel chair etc	1,600	6.10	9,800	27 @ 60 = 1,640
Girders	40,000	4.55	182,000	27×4 @ 370 = 40,000
Ecru mortar fill.	22,200	6.0	133,200	27×179 @ 46 = 22,200
Fillets	200	5.7	1,100	27 @ 6 = 160
	131,900		733,200	

Total wt. = 368,600 lbs Total m = 2,090,400 lbs

Dist of centre of gravity of Superst. above top of ~~coping~~ Pier

$= \frac{2090400}{368600} = 5.67'$

Centre of gravity of Pier above line x-x (moment about x-x line)

Names	Weight	Lever arm	Moment	Remarks
Girders	48,000	16.9	812,000	15.34×26×2 @ 150 × ½ = 48,000
Columns upper 10.4'	30000	9.6	288,000	(2×3 + 2×1) 5.5 @ 150 = 30,000
" Unit 4.5'	30,400	2.75	68,400	3×3×4.575 @ 150 = 30,400
	108,400		1168,400	

Dist. of centre of gravity of pier above x-x line from x-x line = 1168,400 ÷ 108,400

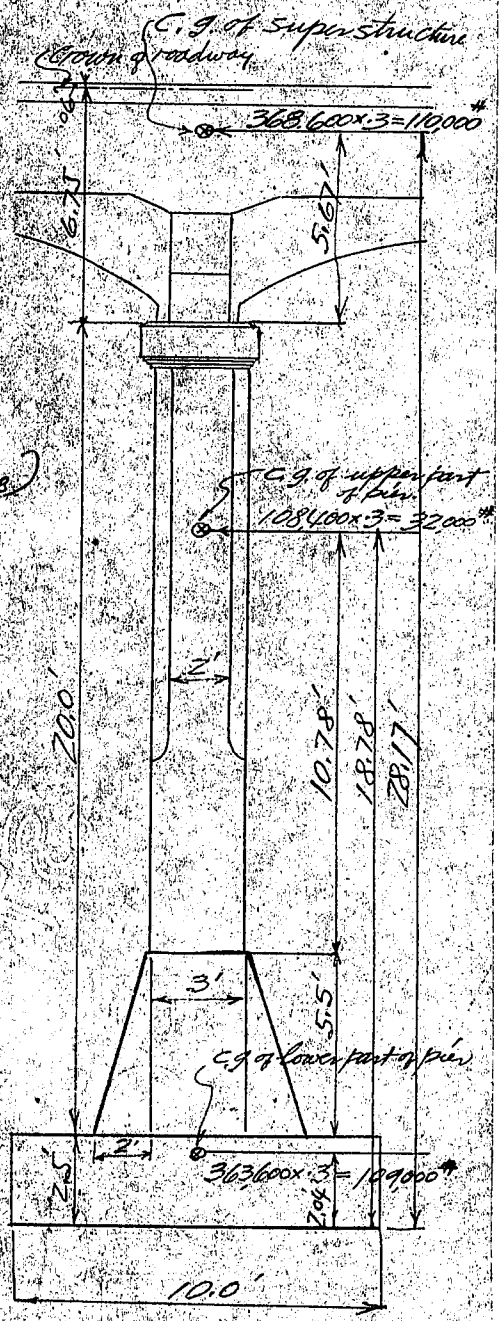
$= 10.78'$

Centre of gravity of pier below line x-x (moment about bottom of base)

Colo. lower 55'	37,100	5.25	194,800	3×3×5.5×5 @ 150 = 37100
Brackets	49,500	4.33	214,300	3×2×5.5×½×4×5 @ 150 = 49,500
Base	27,000	1.25	34,000	10×7.4×2.5 @ 150 = 27,000
	363,600		756,100	

Dist. of C. of G of pier below x-x line from bottom of base

$= \frac{756100}{363600} = 2.04'$



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARLINGTON, MASS.

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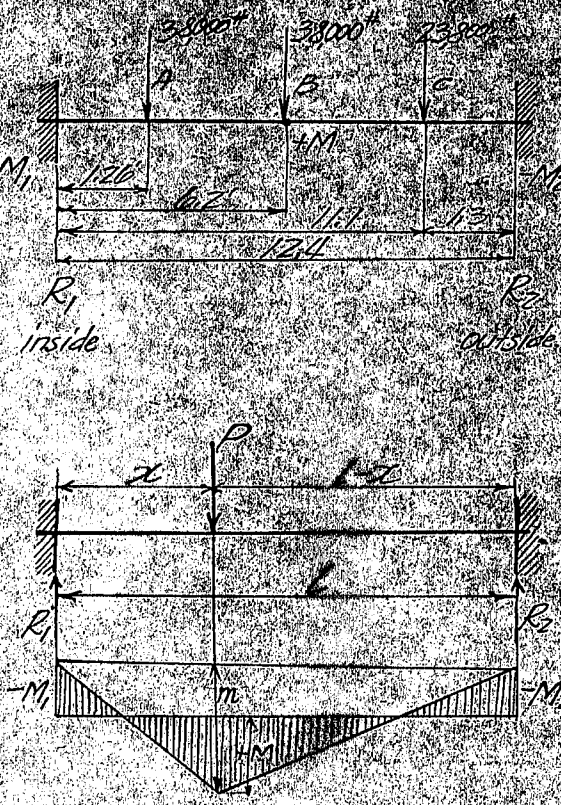
CALCULATIONS FOR

Muko-Gawa over the Motegawa, Hokkaido.
Strength of Girder for girder-span
Beams under Railway. (Ballast floor)
Dead load moments and Shears.



$$-M_1 = -\frac{P x (L-x)^2}{L^2}$$

$$-M_2 = -\frac{x^2}{L^2} P (L-x)$$

$$R_1 = \frac{L-x}{L} P + \frac{M_2 - M_1}{L}$$

$$R_2 = P - R_1$$

$$+M = \frac{L-x}{L} P x$$

Max. reaction for girder under railway = 38000# + 23800

Moment due to its own wt. span 12.4'

w = 780 #/ft of span

For moment at centre of span

$+M = \frac{1}{24} w L^2 = \frac{1}{24} \times 780 \times 12.4^2 = 5{,}300$ ft#

Neg. moment at ends

$-M = \frac{1}{12} w L^2 = \frac{1}{12} \times 780 \times 12.4^2 = 10{,}600$ ft#

Reaction or max. end shear $= \frac{1}{2} \times 780 \times 12.4 = 4800$ #

Moment due to Dead Load of two girders

For moment at centre of span

$-M_{1A} = -\frac{3800 \times 1.26 \times 11.14}{12.4} = -4{,}300$ ft#

$-M_{1B} = -\frac{3800 \times 6.2 \times 6.2^2}{12.4^2} = -58{,}500$

$-M_{1C} = -\frac{23800 \times 11.1 \times 1.3^2}{12.4^2} = -2{,}400$

Total $= -102{,}200$ ft# $= M_1$

$-M_{2A} = -\frac{1.26^2 \times 3800 \times 11.14}{12.4^2} = -400$ ft#

$-M_{2B} = -\frac{3800 \times 6.2 \times 6.2^2}{12.4^2} = -58{,}500$

$-M_{2C} = -\frac{11.1^2 \times 23800 \times 1.3}{12.4^2} = -38{,}200$

Total $= -97{,}400$ ft# $= M_2$

$+M_A = \frac{11.14 \times 3800 \times 1.26}{12.4} = +4{,}300$ ft#

$+M_B = \frac{3800 \times 12.4}{4} = +11{,}800$

$+M_C = \frac{1.3 \times 23800 \times 11.1}{12.4} = +27{,}700$

Total $= +119{,}700$ ft# $= M$

$\frac{1}{2}(M_1 + M_2) = \frac{-102{,}200 - 98{,}400}{2} = -100{,}300$

$+ 90{,}400$ ft# $= M_2$

$R_1 = \frac{11.14 \times 3800}{12.4} + \frac{3800}{2} + \frac{1.3 \times 23800}{12.4} + \frac{90{,}400 - 102{,}200}{12.4}$

$= 34{,}100 + 19000 + 2{,}500 + 300 = 55{,}900$ # $= V_1$

$R_2 = 3800 \times 2 + 23{,}800 - 55{,}900 = 43{,}900$ # $= V_2$

Moment due to Live Loads on Main Girders

Neg. $M_1 = -40500 \times \frac{38700}{38000} - 58800 \times \frac{38700}{38000} - 2700 \times \frac{23400}{23800} = -103{,}800$ ft# $= M_1$

Neg. $M_2 = -4400 \times \frac{38700}{38000} - 58800 \times \frac{38700}{38000} - 37200 \times \frac{23400}{23800} = -98{,}400$ ft#

Pos. $M = M - \frac{1}{2}(M_1 + M_2)$

$= 44000 \times \frac{38700}{38000} + 11900 \times \frac{38700}{38000} + 27700 \times \frac{23400}{23800} - \frac{-98{,}400 - 103{,}800}{2}$

$= 192{,}700 - 101{,}100$

$= +91{,}600$ ft#

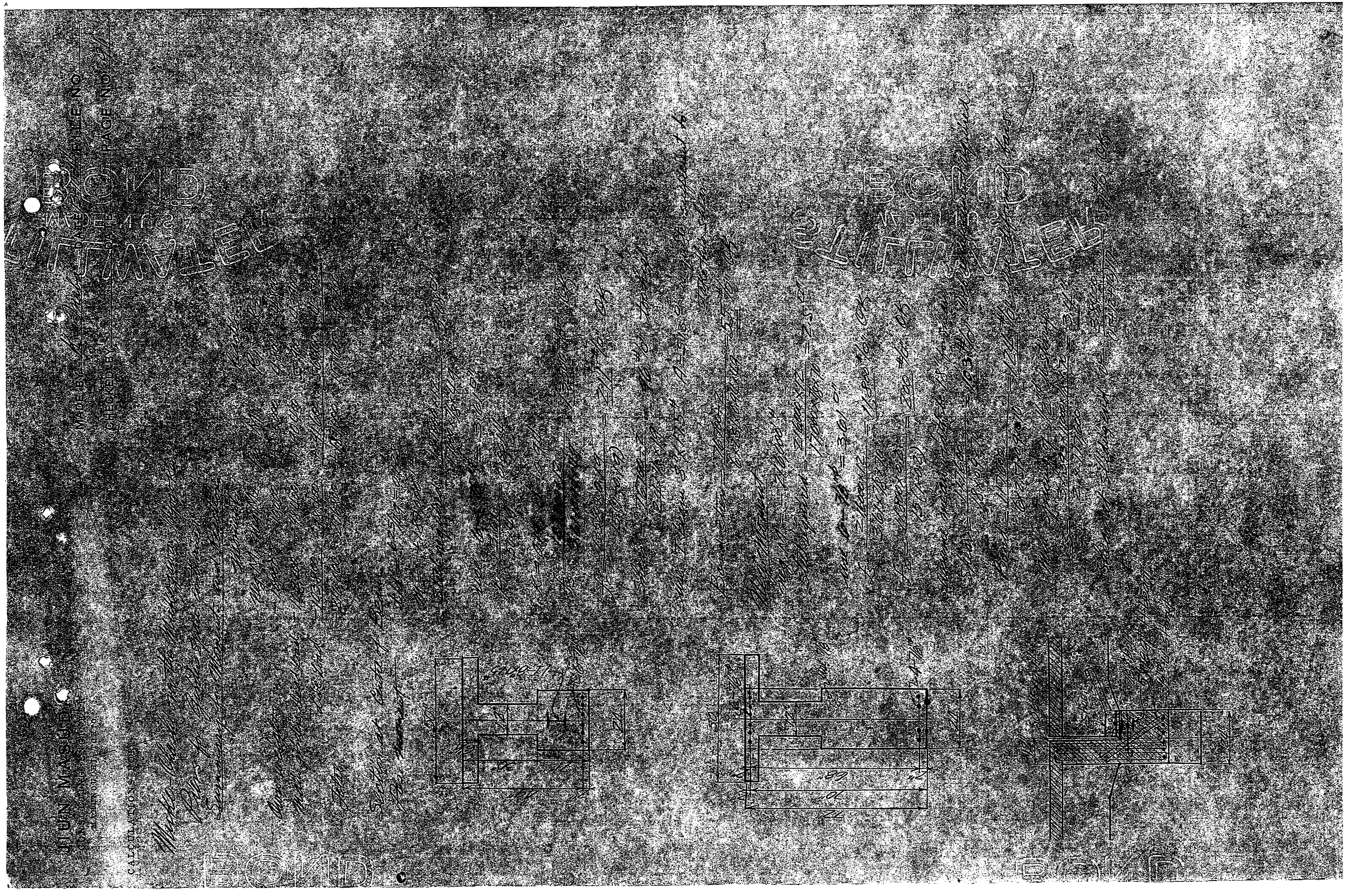
$R_1 = 34{,}100 \times \frac{38700}{38000} + 19000 \times \frac{38700}{38000} + 2{,}500 \times \frac{38700}{23800} + \frac{-98400 - 103800}{12.4}$

$= -56{,}900 = V_1$

$R_2 =$ $= 43{,}900 = V_2$

As the position of loads, same as for dead loads, all moments and reactions for live load can be got by multiplying the ratio of Live load to dead load.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）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WATERTIGHT
JUAN MASUDA
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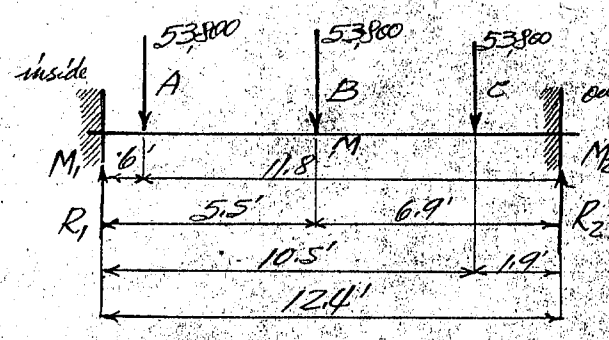
CALCULATIONS FOR

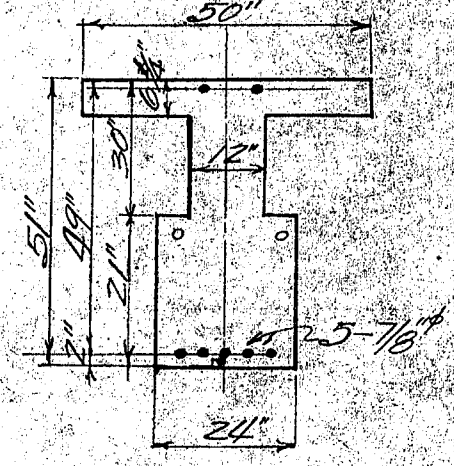
Muko-Okashi over the Mukogawa, Hyogoken.
Pier for Girder span continued.
Beams under Trafficway Girders. (Both ends fixed. l=12.4')



Moments due to it's own wt.
$M_1 = M_2 = -10,000$ '#
$M = +5000$ '#
$R_1 = R_2 = 4800$ #

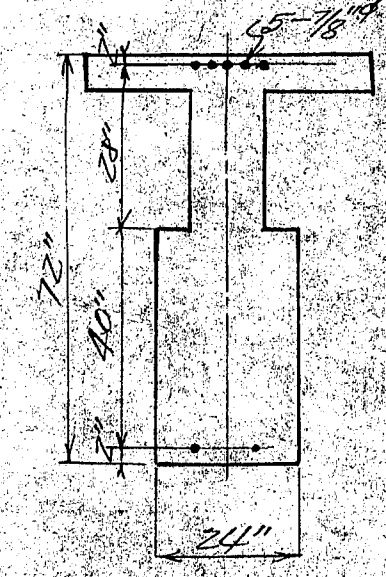
Moments due to main girders max. reactions. (dead and live)
$M_1 = \frac{-53,800}{12.4^2}(0.6\times11.8^2+5.5\times6.9^2+10.5\times1.9^2) = -134,200$ '#
$M_2 = -\frac{53,800}{12.4^2}(0.6^2\times11.8+5.5^2\times6.9+10.5^2\times1.9) = -148,000$
$M = \frac{-53,800}{12.4}(0.6\times11.8+5.5\times6.9+10.5\times1.9) = 282,000$
$\frac{M_2+M_1}{2} = \frac{134,200+148,000}{2} = 141,100$
$M = m - \frac{1}{2}(M_2+M_1) = +140,900$ '#
$R_1 = \frac{53,800}{12.4}(11.8+6.9+1.9) + \frac{+148,800-134,200}{12.4} = 73,100$ #
$R_2 = 53,800\times3 - 88,100 = 88,300$ #

Section at Centre of Beam.



Pos. moment = +140,900 '#
Steel reqd. = $\frac{140,900\times12}{17,000\times7/8\times49} = 2.32$ □"
Use 5-7/8"ϕ = 3.01 □"
t/d = 6.25/49 = 0.128, $p = \frac{3.01}{50\times49} = 0.00123$
k = 0.19, j = 0.944
$f_s = \frac{140,900\times12}{3.01\times0.944\times49} = 12,200$ #/□" ok.
$f_c = \frac{12,200\times0.19}{15(1-0.19)} = 191$ #/□" ok.
Shear at load B = $\frac{53,800\times2}{24\times7/8\times49} = 67.4$ #/□" web reinf. reqd.
Bond " " = $\frac{53,800\times2}{5\times2.51\times7/8\times49} = 46$ #/□" ok.
Min spacing of 3/8"ϕ U stirrups $s = \frac{3\times0.393\times17,000\times7/8\times70}{2\times53,800} = 7.4$"

Section at Support.



Neg. moment = −148,000 '#
Steel reqd. = $\frac{148,000\times12}{17,000\times7/8\times70} = 1.71$ □"
Use 5-7/8"ϕ = 3.01 □"
$f_s = \frac{148,000\times12}{3.01\times7/8\times70} = 9,700$ #/□" ok.
$f_c = \frac{9,700\times1.2}{45\times1.8} = 161$ #/□" ok.
Shear = $\frac{88,300}{18\times7/8\times70} = 80$ #/□" web reinf required.
Bond = $\frac{88,300}{5\times5.21\times0.9\times70} = 42$ #/□" use deformed bars.

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Ibuka. DATE 13-10-15 FILE NO
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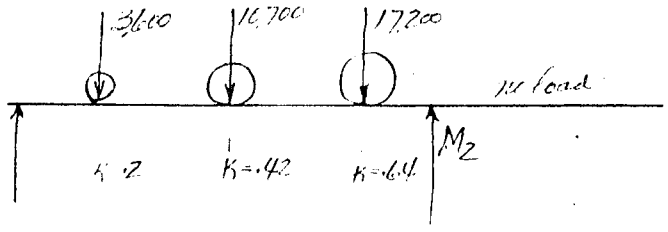
CALCULATIONS FOR

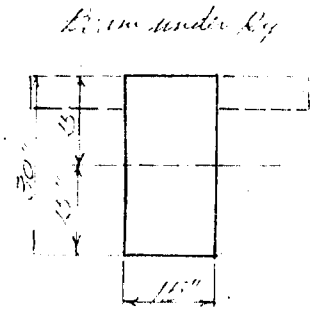
Mukō Bashi over the Mukōgawa, Ayagoku.
Pier for Girder-span continued.
Column under Railway.
Negative max. moment at support due to one side span full loaded and on the other span no live load.

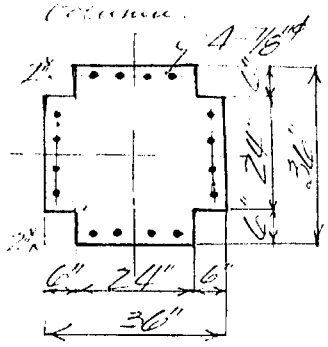


Unit no.	Load	M_2 unit load	M_2
C	3600	-0.51	-1840
B	10,700	-0.97	-10,380
A	17,200	-1.01	-17,380
			-29,070

Neg. M_2 = $-29070 \times 27/10 = -78,500$ ft

for 2 girders $2M_2 = -157,000$ ft





Moment of inertia of girder, Slab neglected.
$I_g = \frac{16 \times 30^3}{12} = 27000$ (in)^4

2 girders on center column
$2I_g = 2 \times 27,000 = 54,000$ (in)^4 for both sides $I_G = 2 \times 54,000 = 108,000$ (in)^4

Moment of inertia of column.
$I_c = \frac{24 \times 36^3}{12} + \frac{12 \times 24^3}{12} = 93,200 + 13,800 = 107,000$ (in)^4

Bending moment taken by column $= 2M_2 \times \frac{I_c}{I_G + I_c}$
$= -157,000$ '# $\times \frac{107,000}{108,000 + 107,000} = -78,000$ '#

Vertical load on column.

Dead Load. approximately, total load from 3 beams under Ry.
$38,000 \times 3 = 114,000$ #

Live Load.	load	R_2 unit load	R_2
C	3600	0.32	1,150
B	10,700	0.66	7,070
A	17,200	0.90	15400
			23,600 # = R_2 from one girder

for 3 girders, $R_2 = 3 \times 23,600 = 70,700$ '#

Weight of Pier above X-X line.
$108,400 \div 5 = 21,700$
Total dead load on column $= 114,000 + 21,700 = 135,700$
" live load " " $= 70,700$
Total $= 206,400$ #

Live load moment on column, (dead load moment neglected)
$= -78,000$ '#

Eccentricity $= \frac{78,000 \times 12}{206,400} = 4.53$"

$e/h = 4.53/36 = 0.126$, $d'/h = 3/36 = 0.083$ $p = 0.007$
$K = 1.55$

$f_c = \frac{NK}{bh} = \frac{206400 \times 1.55}{24 \times 36} = 371$ #/0" Ok.

$f_s = 371 \times 15 = 5600$ #/0" Ok.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Inaba DATE 13-10-15 FILE NO
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CALCULATIONS FOR

Muko-Gawa over the Mukogawa, Hyogoken.
Pier for Girder Span continued.
Stability of Pier for full live load.
Moment due to unsymmetrical loading is very small and neglected.
Loads on Pier.
Dead Load.

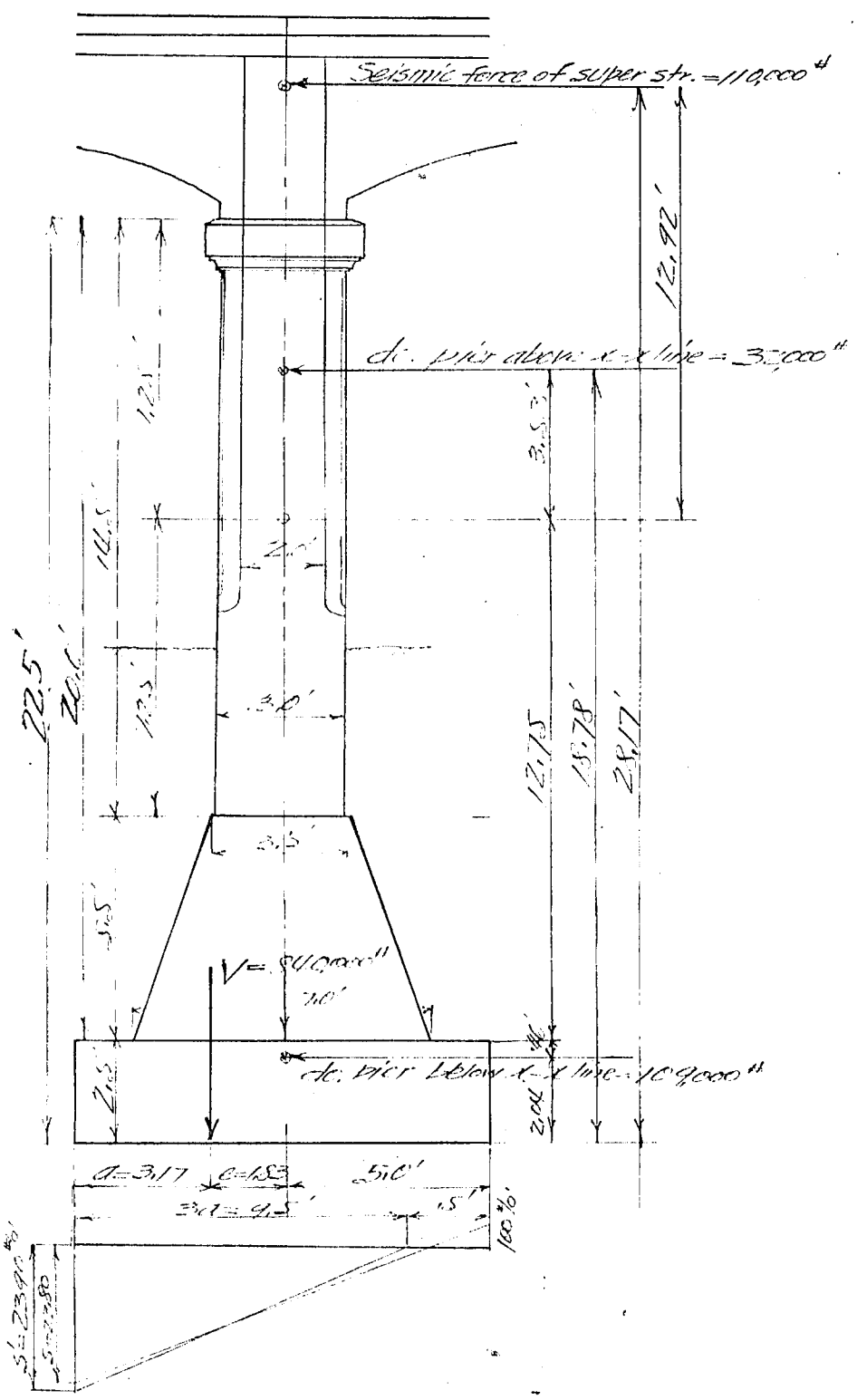
wt. of Super Structure	= 363600#
" " pier above x-x line	= 108400
" " " below " "	= 363600
	840,000 #

Live Load

Railway 35700×4	= 143,000#
Traffic way 30,000×8	= 240,000
Footway load 12,100×2	= 24,000
	414,000 #
Total Loads	= 1,254,000 #

Max. Pressure on foundation
$= \frac{1259,000}{10\times74} = 1700\,\#/\square' = 0.76$ tons/□' OK.

Seismic Stability of Pier. (K=0.3)

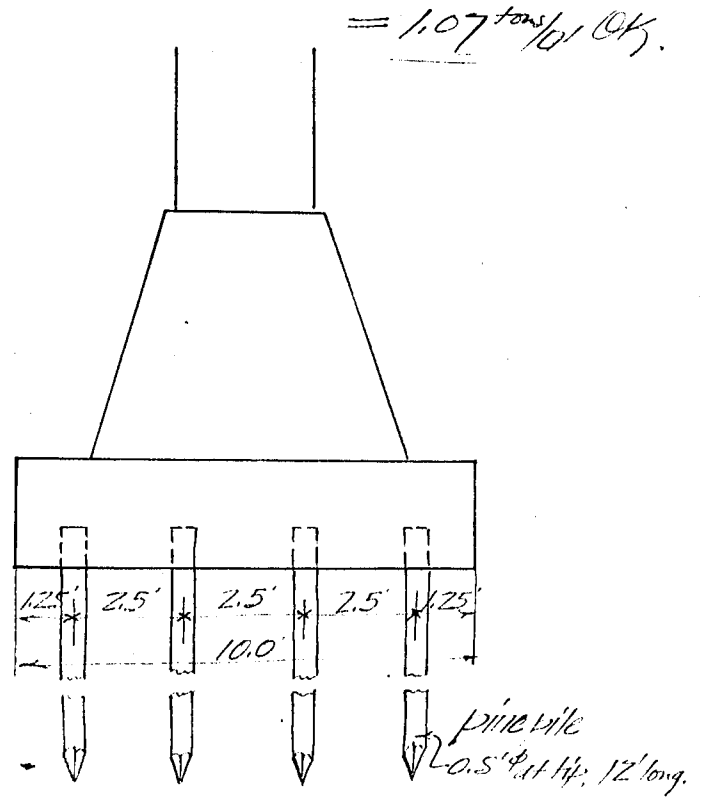


Max toe pressure.
Moment of Seismic forces about C (inflection point)
$110,000\times12.92 = 1,420,000$ #
$32,000\times3.53 = 113,000$
Total m $= 1,533,000$ #
Eccentricity $= \frac{1533000}{840,000} = 1.83'$

$$S = \frac{P}{A} \pm \frac{6M}{bh^2} = \frac{840,000}{10\times74} \pm \frac{6\times1,533,000}{74\times10^2}$$
$$= 1,140 \pm 1240$$
$= 2,380$ #/□' compression & 100 # tension.

Neglecting tension

$$S' = \frac{2V}{3ab} = \frac{2\times840000}{3\times3.17\times74} = 2370\,\#/\square'$$
$= 1.07$ tons/□' OK.



piles 3 ft. spacing longitudinally.

load on one pile $= 0.76\times2.5\times3 = 5.70$ tons
Seismic load $= \left(\frac{1.07\times7.9}{7.5} + 1.07\right)\frac{1}{2}\times2.5\times3 = 7.0$ tons.

Specify the safe bearing power of pile = 5.7 tons
by the Engineering News Formula.

$$P = \frac{2WH}{S+1} \geq 5.7 \text{ tons.}$$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Iraba DATE 13-10-15 FILE NO
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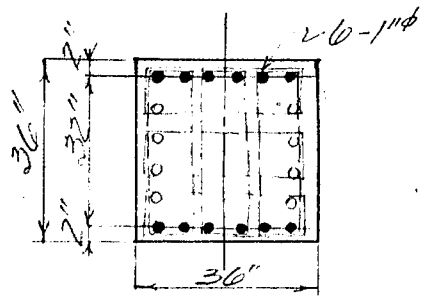
CALCULATIONS FOR

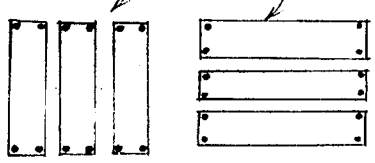
Mutsu-Ohashi over the River Mutsu, Hyogoken.
Pier for Girder span continued
Section of column at x-x line. (Centre column).



Moment = 1,533,000 1# vert-load = 368,600 + 108,400 = 477,000 #
For one column $M = \frac{1,533,000}{5} = 307,000$ 1# $N = \frac{477,000}{5} = 95,500$ #
For centre column, add 50 %. say.
$M' = 460,000$ 1# $N' = 143,000$ #
use 6 - 1"φ bars on one side = 4.72 0" $p = p' = \frac{4.72}{36\times36} = 0.0037$, $p_0 = 0.007$
$d/h = 3/36 = 0.055$, $e_{cc} = \frac{460,000}{143,000} = 3.22' = 38.8"$ $e/h = \frac{38.8}{36} = 1.07$
$K = 0.342$, $L = 0.131$

$f_c = \frac{M}{Lbh^2} = \frac{460000\times12}{0.131\times36\times36^2} = 905$ #/0" OK (allowable f_c in this case $= 640\times1.8 = 1,150$ #/0")

3/8" 1100p. group(1) group(2)



$f_s = n f_c \left(\frac{d}{Kh} - 1\right) = 15\times905\left(\frac{36}{0.342\times36} - 1\right) = 23900$ #/0" O.K. (f_s allowable = 30,600)

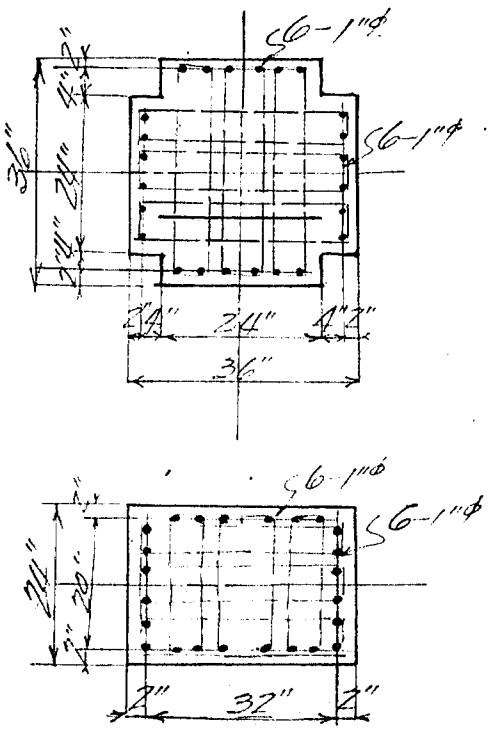
Shear $V = 110,000 + 32,000 = 142,000$ # for one col. $V' = \frac{142,000\times1.5}{5} = 42,600$ #
$v = \frac{42600}{36\times7/8\times34} = 40$ #/0" OK.

use 3/8"φ hoops group 1 & 2 alternately. 6" c to c.
for top and bottom 2' use 4" spacing.

bond $u = \frac{42600}{3.142\times6\times7/8\times34} = 76$ #/0" OK.

Section of Column at Top.

$M = 460,000$ 1#, $N = \frac{368600\times1.5}{5} = 110,500$ #
use 6-1"φ = 4.72 0", $p = p' = \frac{4.72}{24\times36} = 0.0055$, $p_0 = 0.011$
$d'/h = 0.055$, $e_{cc} = \frac{460000}{110,500} = 4.15' = 49.8"$, $e/h = \frac{49.8}{36} = 1.38$
$K = 0.359$, $L = 0.162$



$f_c = \frac{460000\times12}{0.162\times24\times36^2} = 1095$ #/0" OK. (allowable $f_c = 1150$ #/0")

$f_s = 1095\times15\left(\frac{34}{0.359\times36} - 1\right) = 26800$ #/0" OK. (allowable $f_s = 30,600$ #/0")

Shear $V = \frac{110,000}{5}\times1.5 = 33,000$ #
$v = \frac{33000}{24\times7/8\times34} = 46$ #/0" OK.

use 3/8"φ hoops 6" spacing at top and bottom for 3 ft.
for other portions 1.0' spacing.

bond $u = \frac{33000}{3.142\times6\times7/8\times34} = 59$ #/0" OK.

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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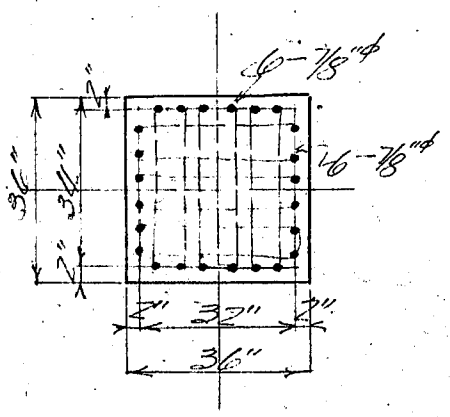
CALCULATIONS FOR

Mutsu-Ohashi over the Mutsugawa, Hyogoken.
Pier for Girder span continued.
Column under Trafficway.
Section at X-X line.

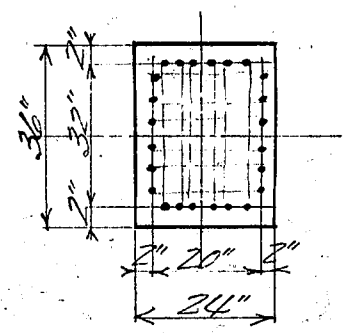
Seismic moment for one column = 307,000 '#
Vertical load N = 95,500 #
shear V = 28,400 #



Arrangement of hoops, same as before.

Eccentricity = 3.11' $\varepsilon/h = 3.11/3.0' = 1.03$
$d'/h = 2/36 = 0.055$
try 6-7/8"φ on both sides, $p = p' = \frac{3.61}{36\times36} = 0.0028$, $p_0 = 0.0056$
$K = 0.32$, $L = 0.115$
$f_c = \frac{307000\times12}{0.115\times36\times36^2} = \underline{690}$ #/0" Ok.
$f_s = -690\times15\left(\frac{34}{32\times36}-1\right) = 20,300$ #/0" Ok
$v = \frac{28400}{36\times\frac{7}{8}\times34} = 26.5$ #/0" Ok.
use 3/8"φ hoops 6" c to c.
bond $u = \frac{28400}{251\times6\times\frac{7}{8}\times34} = \underline{63.5}$ #/0" Ok.

Section at Top of coping



Seismic moment M = 307,000 '# for one column
vert load N = 74,000 #
Shear V = 22,000 #

eccentricity = $\frac{307000}{74,000} = 4.15'$ $\varepsilon/h = 4.15/3.0 = 1.38$
$d'/h = 0.055$
use 6-7/8"φ = 3.61 0" on both sides, $p = p' = \frac{3.61}{36\times24} = 0.0042$, $p_0 = 0.0084$
$K = 0.33$, $L = 0.142$
$f_c = \frac{307000\times12}{0.142\times24\times36^2} = \underline{835}$ #/0" Ok. (fc allowable = 1150 for tie
$f_s = 15\times835\left(\frac{34}{133\times36}-1\right) = \underline{23,300}$ #/0" Ok. (fs = 30600
$v = \frac{22000}{24\times\frac{7}{8}\times34} = \underline{31}$ #/0" Ok
use 3/8"φ hoop 6" c to c.
$u = \frac{22,000}{251\times6\times\frac{7}{8}\times34} = 49$ #/0" Ok

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(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTING ENGINEER
NEW YORK, N.Y.

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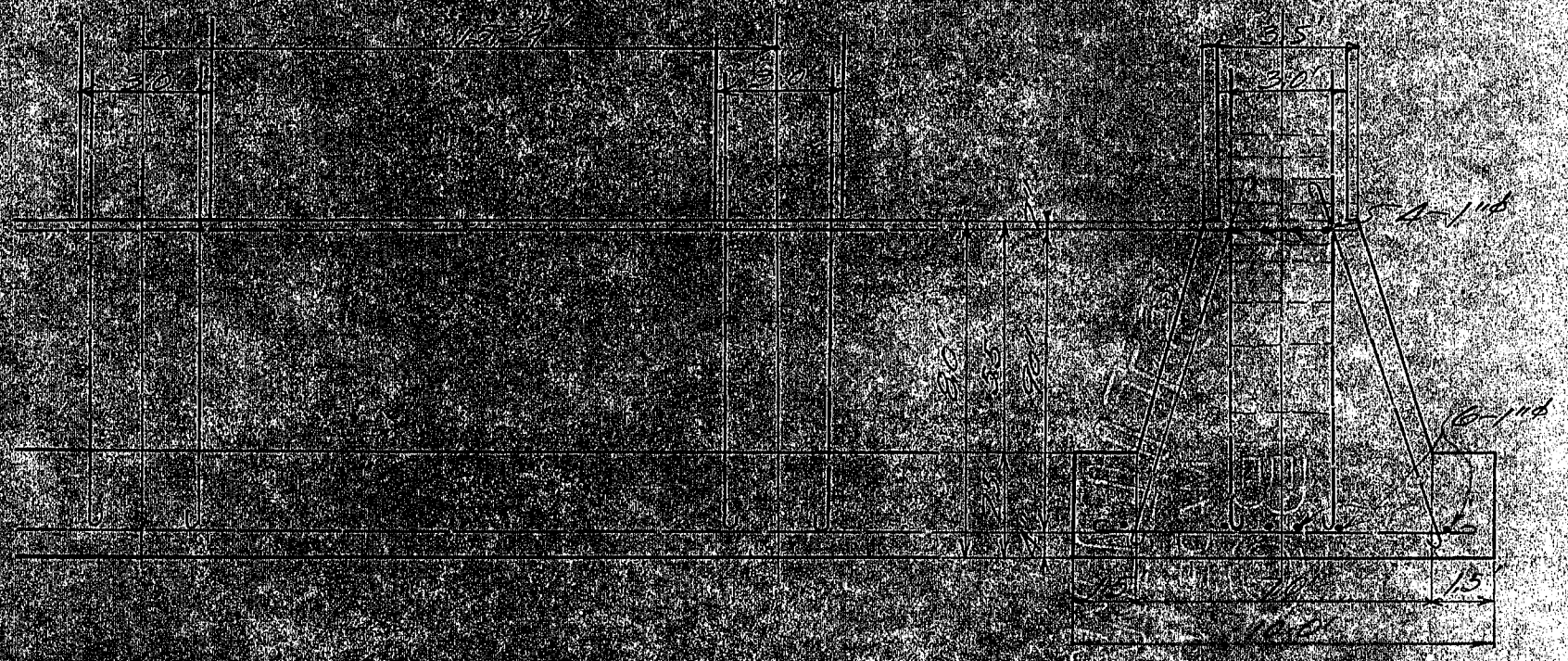
CALCULATIONS FOR



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（独立行政法人 土木研究所蔵）

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CONSULTING ENGINEER
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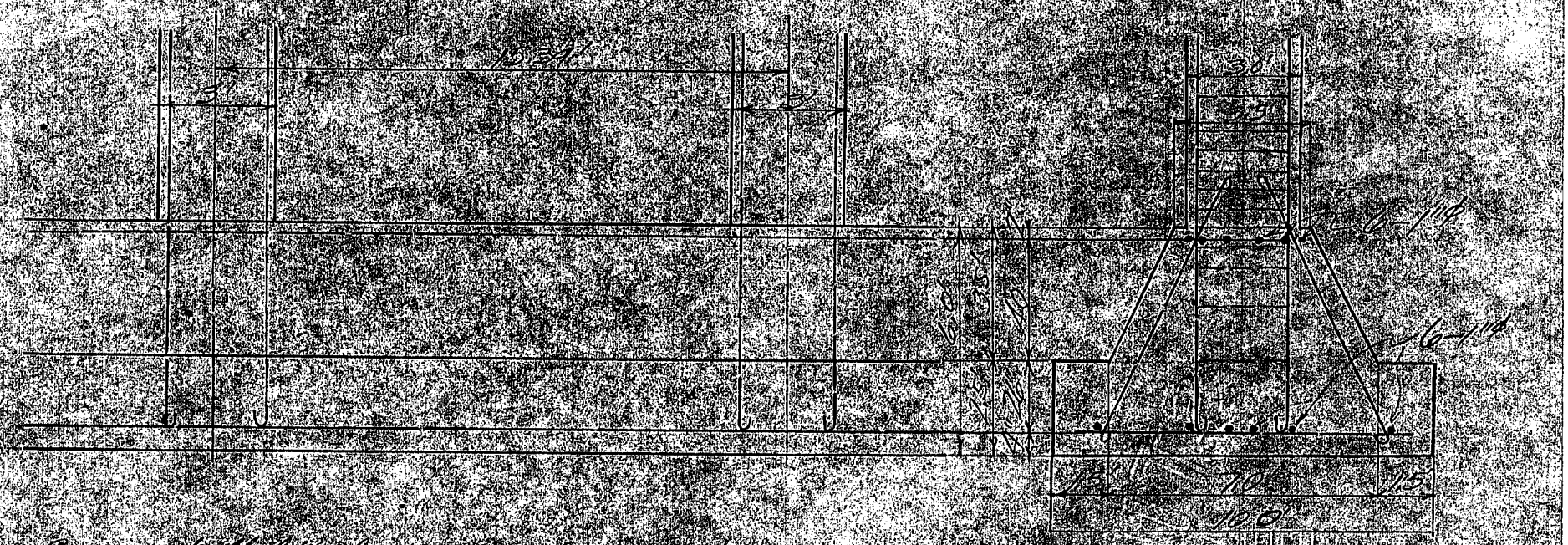
CALCULATIONS FOR

Mukō-Okachi-ryō Ikebukuro, Nagasaki
Pier for Girder Span, continued
Foundation (Pier Type B)



Case of full live load.

Upward pressure = 1,700 #/□ft

Moment at Centre of Span. $M = \frac{wl^2}{10} = \frac{1{,}700 \times 15.34^2}{10} = 40{,}300$ #

Shear. $V = \frac{w}{2} = \frac{40{,}300}{2} = 20{,}150$ #

At Centre of Span. $p = \frac{M}{fbd^2} = \frac{40{,}300 \times 12}{16{,}000 \times 120 \times 56^2} = 0.00056$ 6-1"ϕ assumed.

$k = \sqrt{2np + (np)^2} - np = \sqrt{2 \times 15 \times 0.00056 + (15 \times 0.00056)^2} - 15 \times 0.00056 = 0.111$

$j = 1 - \frac{1}{3}k = 1 - \frac{0.111}{3} = 0.963$

$f_s = \frac{40{,}300 \times 12}{4.7 \times 0.963 \times 56} = 1{,}530$ #/□in OK

$f_c = \frac{1{,}530 \times 0.111}{15(1 - 0.111)} = 142$ #/□in OK

At Column. $p = \frac{4.72}{42 \times 65} = 0.00173$ 6-1"ϕ assumed.

$k = 0.176$, $j = 0.941$

$f_s = \frac{40{,}300 \times 12}{4.7 \times 0.941 \times 65} = 1{,}610$ #/□in OK

$f_c = \frac{1{,}610 \times 0.176}{15(1 - 0.176)} = 234$ #/□in OK

Max. Bed Shear $V = \frac{1{,}700 \times 10 \times 15.34}{2} = 131{,}000$ #

Max. Shear $v = \frac{131{,}000}{0.941 \times 42 \times 65} = 34$ #/□in OK

Unit bond $u = \frac{131{,}000}{3.1416 \times 0.98 \times 65} = 109$ #/□in OK for deformed bars

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
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CALCULATIONS FOR

Muko-Gawa Bridge over the Muko-gawa, Hyogo-ken.
Design of Abutment for arch span.
Normal Thrusts.—

Dead Load + Live Load + Rib Shortening.

	Traffic-way	Elect. Ry.	Total	Eccentricity: Traffic-way	Eccentricity: Elect. Ry.	Resultant Ecc.
Nor. thrust for pos. moment	637,500 × 2 = 1,275,000 #	663,700 #	1,938,700 #	+0.637'	+0.505'	+0.59'
do. for neg. moment	587,400 × 2 = 1,174,800 #	594,700	1,769,500	−0.965	−1.22	−1.05

Dead Load + Live Load + Temperature and Rib Shortening.

Nor. thrust for	Traffic-way	Elect. Ry.	Total	Eccentricity: Traffic-way	Eccentricity: Elect. Ry.	Resultant Ecc.
Positive M.	735,000×2 = 1,470,000	710,700	2,180,700 #	+1.15'	+1.11'	+1.14'
negative M.	534,800×2 = 1,069,600	547,700	1,617,300 #	−2.02'	−2.05'	−2.03'

Shear at Springing normal to Neutral Axis.
For Live Load Pos. moment

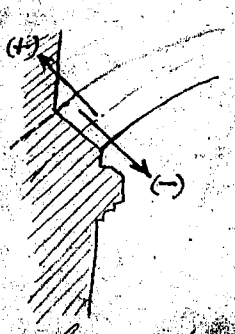
	Shear due to hor. thrust: Traf. thrust	Ry. thrust	total thrust	coef.	Shear	Shear due to vert. shear: Traf. Shear	Ry. Shear	total	coef.	Shear	Total Shear
Dead Load	876,000	450,400	1,326,400	0.558	740,000	554,000	282,200	836,200	−0.834	−697,000	43,000 #
Live Load	288,000	156,200	444,200	0.558	247,500	65,800	34,300	100,100	−.834	−83,500	164,000
Rib Shortening	−42,000	−22,000	−64,000	0.558	−35,700	—	—	—	−.834	—	− 35,700
											171,300 #
Dead Load	876,000	450,400	1,326,400	.558	740,000	554,000	282,200	836,200	−.834	−697,000	43,000
Live Load	288,000	156,200	444,200	.558	247,500	65,800	34,300	100,100	−.834	−83,500	164,000
Temp. Change	+131,200	+58,600	189,800	.558	−105,800	—	—	—	—	—	−105,800
Rib Shortening	−47,500	−24,300	−71,800	.558	−40,000	—	—	—	—	—	− 40,000
											61,200 #

For Live Load neg. Moment.

	Shear due to hor. thrust: Traf. thrust	Ry. thrust	total thrust	coef.	Shear	Shear due to vert. Shear: Traf. Shear	Ry. Shear	total	coef.	Shear	Total Shear
Dead Load	876,000	450,400	1,326,400	0.558	740,000	554,000	282,200	836,200	0.834	−697,000	43,000
Live Load	96,800	34,700	131,500	0.558	73,400	161,200	84,900	246,100	.834	−205,500	−132,100
Rib Shortening	−35,000	−17,600	−52,600	0.558	−29,300	—	—	—	—	—	− 29,300
											−118,400
Dead Load	876,000	450,400	1,326,400	0.558	740,000	554,000	282,200	836,200	.834	−697,000	43,000
Live Load	96,800	34,700	131,500	0.558	73,400	161,200	84,900	246,100	.834	−205,500	−132,100
Temp. change	−131,200	−58,600	−189,800	0.558	−105,800	—	—	—	—	—	−105,800
Rib Shortening	−30,000	−15,400	−45,400	0.558	−25,300	—	—	—	—	—	− 25,300
											−220,200

Summary of External Forces on Abutment.

	Hor. thrust	Eccentricity	Shear.
(1) In case Live pos. moment to max. without Temp. change	1,938,700 #	+0.59'	171,300 #
(2) " " " " " max. with temp change	2,180,700	+1.14'	61,200
(3) In case Live neg. moment max. without temp change	1,769,500 #	−1.05'	−118,400 #
(4) " " " " " with Temp change	1,617,300	−2.03'	−220,200



Sign of Shear and eccentricity.

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY H. Ihaba DATE 13-10-20 FILE NO
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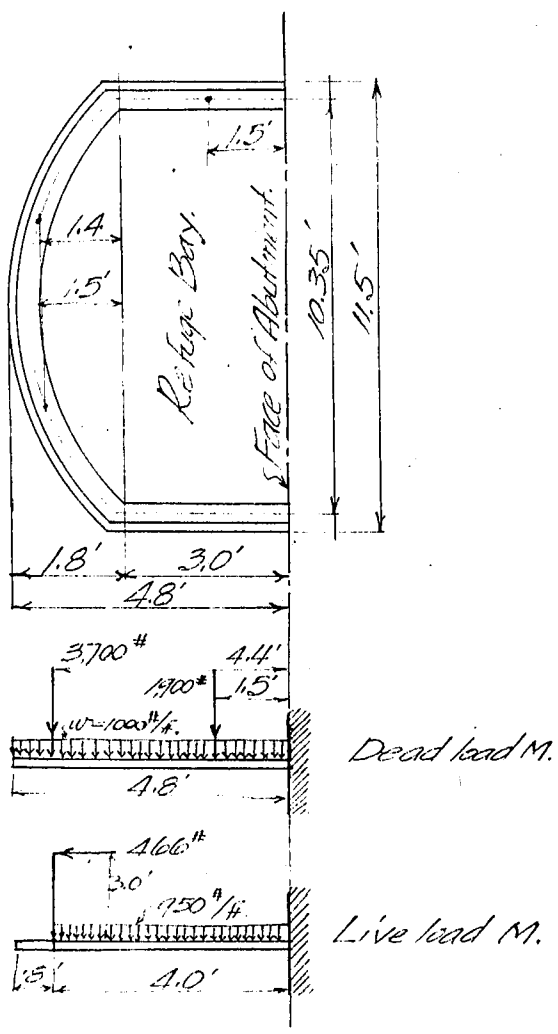
CALCULATIONS FOR

Muko-Gawa bridge over the Mukogawa, Hyogoken.
Design of Abutment between Arch Span and Girder Span.
Refuge Bay



Dead Load.—

Handrail, toprail (0.5×0.7 − 1×1.2×2) ×160 = 56 #/ft.
post .5×1.4×2.2 @160 = 70
coping .3×.8×150 = 36
fascia girder 1.4×.7 @ 150 = 148
total = 310 #/ft.

7" Slab assumed. total width = 11.5'
.583×11.5@150 = 1,000 #/ft.

Load due to front rail 12×310 = 3,700 #
" " " side rails 2×3×310 = 1900

Dead Load moment:—
due to wt. of slab = $\frac{1000\times4.8^2}{2}$ = 11,500 '#
" " " handrail = 3700×4.4 = 16,300
1400×1.5 = 2,900
total dead load M. = 30,700 '#

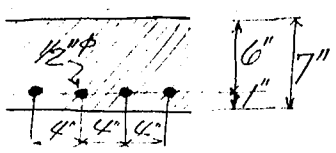
Live Load moment: 110 #/ft² = 100×4.5 = 950 #/ft.
due to unif. load $\frac{950\times4^2}{2}$ = 7,600 '#
hor. pressure on top rail = 45×10.35 = 466 # on front rail.
moment = 466×3 = 1,400 '#
total live load M = 9,000 '#

Total moment = 30,700 + 9000 = 39,700 '#
moment for 1 ft strip of slab = $\frac{39,700}{11.5}$ = 3,450 '#

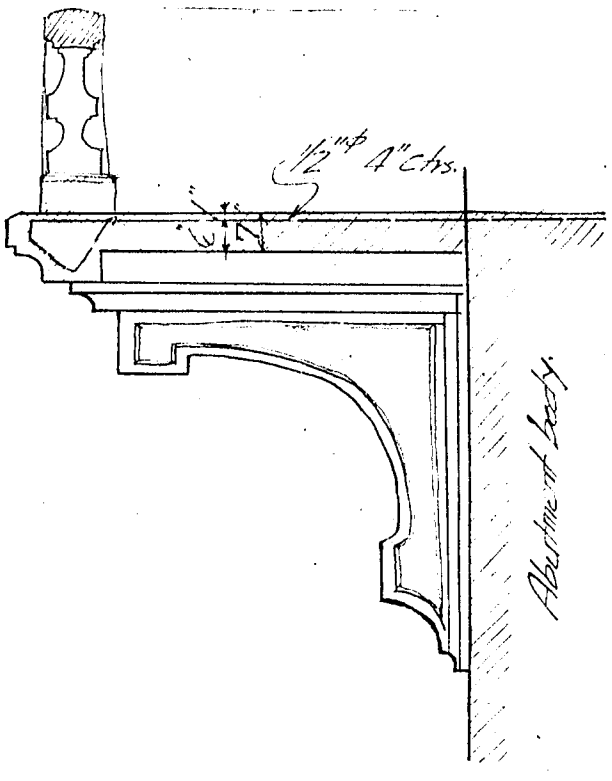
Steel required = $\frac{3450\times12}{17000\times7/8\times6}$ = 0.557 0"
use ½"ϕ — 4" spacing = 0.588 0"
$p=\frac{.588}{12\times6}$ = 0.00776, k = .38, j = .873

$f_s=\frac{3450\times12}{0.588\times0.873\times6}$ = 13,420 #/0" Ok

$f_c=\frac{2f_sp}{k}=\frac{2\times13420\times0.00776}{0.38}$ = 544 #/0" Ok



Slab thickness 7" overall
with 1" insulation.



Max. end shear.
wt of slab 1000×4.8 = 4,800
front rail = 3,700
side rails = 1,400
live unif load 950×4 = 3,800
total = 14,200 # ÷ 11.5' = 1270 #/ft strip of slab.

$v=\frac{1270}{12\times.873\times6}$ = 20.2 #/0" Ok

$u=\frac{1270}{1.57\times.3\times.873\times6}$ = 51.4 #/0" Ok.

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（独立行政法人 土木研究所蔵）

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CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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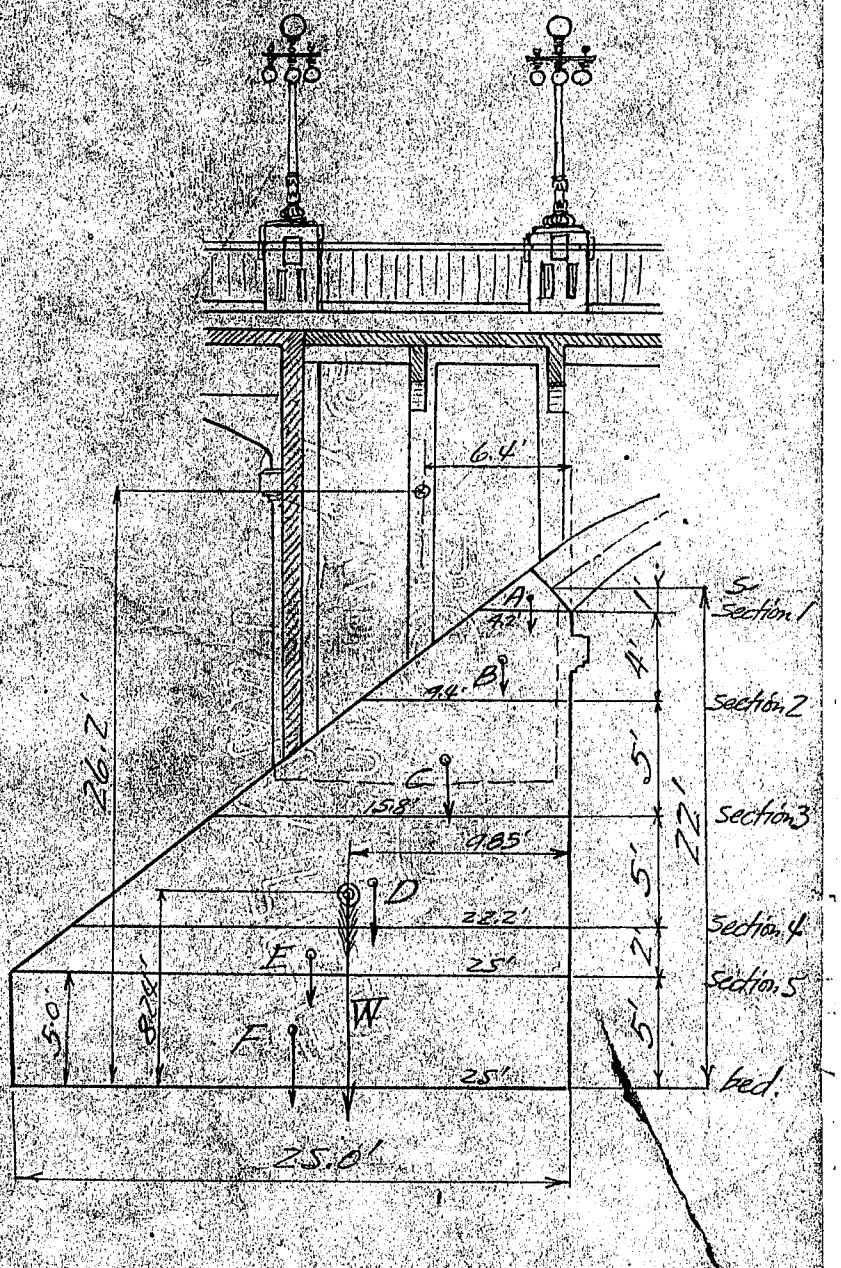
CALCULATIONS FOR

Muko-Gawa over the Mukogawa, Hyogoken.
Abutment for Arch span continued.
Weight and Centre of gravity of Structure of Abutment body.

Names.	Weight	Lever arm	moment (about bed of abutment body.)
Light pole 2@ say 500#	1000 #	43.5'	43,500
Light pedestal 2×2.5×2.5×4.4×150	8,300	36.5	315,000
Range bars 2@ 14.200	28,400	34.0	966000
Brackets 2@ 3.5@150	1,100	31.8	35000
Sidewalk			
Pavement 55.2×0.6×1@150	1,100	34.2	37,600
Slab 39×5.5×2×1@150	7,180	34.0	244,500
Web of beam 1.5×8.3×1×2@150	4,180	33.0	135 200
curb stone 1×2@ 40	900	33.9	30,300
Trafficway			
Pavement 15.8×2×1 @ 22	7,700	33.9	261,000
Slab 17.9×2×1 @ 78	30,700	33.6	1030,000
Web of beam 1×9.83×8@150	4900	32.8	325,000
Electric Ry.			
Pavement 11×17.9@ 56	10,600	33.9	359,000
beam and curb 1×17.9 @46	9,100	33.5	305,000
Slab 1.1×17.4@ 78	15400	33.0	508,000
Web of beam 4×9×8.3×1×2@150	4,400	32.3	158,000
rails & accen. 4.1 @ 30	1 300	33.9	44,000
sleepers etc. 4×1@54×20	900	33.6	30,200
Wall 83×22×50 @150	13700	31.0	425,000
Columns 1.0×½×7.0×14 @150	7,400	27.2	201,000
" 1.0×1.0×11.5×16@150	24,200	26.1	631,000
" 2×2.7×12×5 @150	48,600	21.4	1040,000
Wall 1.0×54×12 @150	97,300	21.4	2080,000
Shaft 11.5×38×20×2@150	262,000	24.5	6420,000
Total =	595,700#		15,621,300#

Distance of C.G. from bed = 15621300 / 595,700 = 26.2 ft



Weight and Centre of Gravity of Abutment body. (width = 64.7')

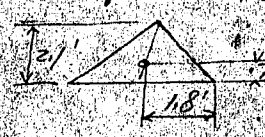
Mass.	Weight	lever arm	moment
A	28,500	21.7	618000
B	246,900	18.8	4625,000
C	572,000	14.25	8150,000
D	860,000	9.40	8080,000
E	427,000	6.04	2575,000
F	1130000	2.5	2830,000
	3,263,500		26,878,000

Dist. distance of C.g. from bed = 26878000 / 3263,500 = 8.24'

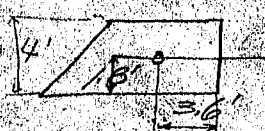
A	28,500#	1.8	51,000
B	246,000	3.6	876,000
C	572,000	6.45	3690,000
D	860,000	9.65	8300,000
E	427,000	11.9	5080,000
F	1130,000	12.5	14120000
	3,263,500#		32,117000

Hor. distance of C.g. from abutment face
= 32,117000 / 3263,500 = 9.85'

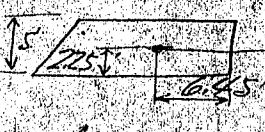
Wt. of mass A = 21' × 64.7' × 140 = 28,500#



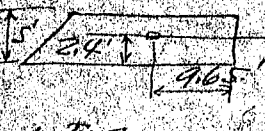
Wt. of mass B = (4.2+9.4)/2 × 4 × 64.7 × 140 = 246000#



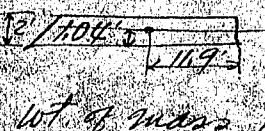
Wt. of mass C = (9.4+15.8)/2 × 5 × 64.7 × 140 = 572,000



Wt. of mass D = (15.8+22.2)/2 × 5 × 64.7 × 140 = 860,000



Wt. of mass E = (22.2+25)/2 × 2 × 64.7 × 140 = 427,000



Wt. of mass F = 25 × 5 × 64.7 × 140 = 1,130,000#

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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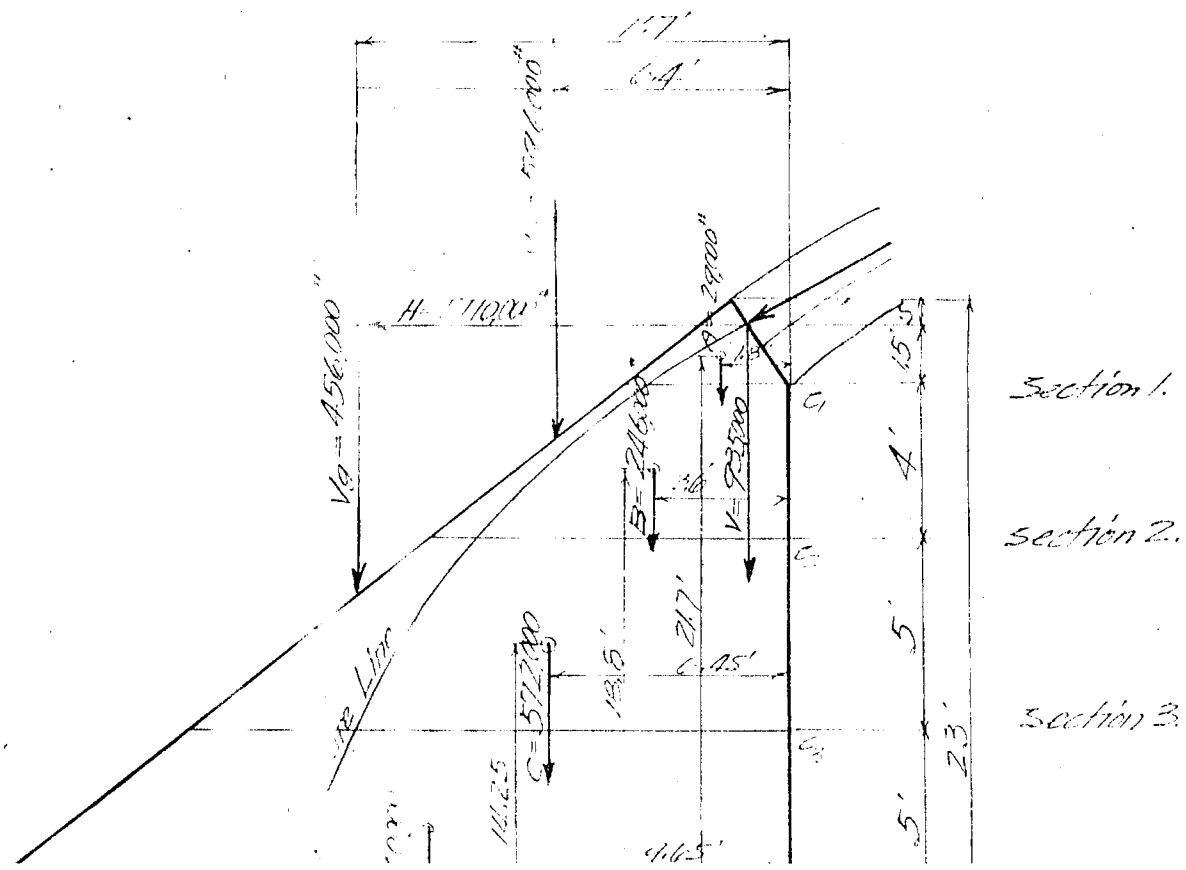
CALCULATIONS FOR

Muko-Okawa over the Yunokawa, Ugoyoshen.
Abutment for Arch Span continued.
Stability of abutment body.
Case I. Positive moment mod. Dead Load + Live Load + Rib Shortening



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MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Mucho-Chuki over the Mukogawa, Upper the Estuary of Arakawa Loop, combined

Case I (Live Load for Engineer's Train + Wind + Temp. Change)

Forces				
H	(Hor. Thrust)	1,710,000#	21.50	36,450,000#
V	(Vert. Load)	23,500	1.00	240,000
Va	(Vert. load, ballast)	596,000	6.40	3,810,000
Vg	(" girder)	456,000	11.70	5,340,000
A	(" Truss A)	24,000	7.80	520,000
B	(" " B)	246,000	3.60	890,000
C	(" " C)	572,000	6.45	3,690,000
D	(" " D)	860,000	9.65	8,300,000
E	(" " E)	427,000	11.90	5,080,000
F	(" " F)	1,120,000	12.50	14,250,000
	Total vert. load	=5,251,000#		80,820,000#

Distance of the application of the resultant force after from C6 $= \frac{80,820,000}{5,251,000} = 15.40\text{ft} = d_6$

Eccentricity $= 15.4 - \frac{25}{2} = 2.90\text{ft}$

Max. Pressure $S = \frac{V}{A} \pm \frac{6Ve}{bh^2} = \frac{5,251,000}{25\times 64.7} \pm \frac{6\times 5,251,000\times 2.9}{64.7\times 25^2} = 3280 \pm 2760 = 5510\#/\square' = 2.46\ \text{tons/}\square'\text{ Comp.}$
$\sigma = -790\#/\square' = 0.44\ \text{tons/}\square'\text{ Comp.}$
OK.

At Section 5

$V = 5,251,000 - 1,130,000 = 4,121,000\#$

$M_5 = 80,820,000 - 14,250,000 - 1,710,000\times 5 = 58,020,000$

$d_5 = \frac{58,020,000}{4,121,000} = 14.10\text{ft}$ $\quad e_{cc} = 14.1 - 12.5 = 1.6\text{ft}$

At Section 3

$V = 4,121,000 - (427,000 + 860,000) = 2,834,000$

$M_3 = 58,020,000 - (5,080,000 + 8,300,000) - 1,710,000\times 7 = 32,670,000$

$d_3 = \frac{32,670,000}{2,834,000} = 11.54\text{ft}$ $\quad e_{cc} = 11.54 - 7.9 = 3.64'$

At Section 2

$V = 2,834,000 - 572,000 - 456,000 = 1,806,000$

$M_2 = 32,670,000 - 3,690,000 - 1,710,000\times 5 = 15,100,000$

$d_2 = \frac{15,100,000}{1,806,000} = 8.36\text{ft}$ $\quad e_{cc} = 8.36 - 4.7 = 3.66\text{ft}$

At Section 1

$V = 1,806,000 - 240,000 - 596,000 = 964,000$

$M_1 = 15,100,000 - 596,000 - 3,810,000 - 1,710,000\times 4 = 3,560,000$

$d_1 = \frac{3,560,000}{964,000} = 3.69\text{ft}$ $\quad e_{cc} = 3.69 - 2.1 = 1.59\text{ft}$

Stress at Section 1

$V = 964,000\#$ $\quad M = 1,710,000 + 1,130,000 + 3,560,000 = 6,400,000$

Stress $f = \frac{V}{bh} + \frac{6M}{bh^2} = \frac{964,000}{64.7\times 12} + \frac{6\times 6,400,000}{64.7\times 12^2} = 1240 + 4630 = 5870\#/\square'$

$\frac{e}{h} = \frac{1.59}{12} = 0.132$ $\quad \frac{d}{h} = \frac{2.5}{12} = 0.208$

$M = 964,000\times 2.5 + 1,130,000\times 1.5 = 4,100,000\text{ in-lbs}$

$R = 0.57$, $L = 0.057$

$f = \frac{M}{Lbh^2} = \frac{4,100,000}{2,442\times 64.7\times 144} = 27,100\#/\square' = 25.7\ \text{tons/}\square'\text{ Comp.}$

$f' = 15\times 25.7\left(\frac{0.208}{0.208} - 0.132\right) = 29.90\ \text{tons/}\square'\text{ Comp.}$

$\text{Area} = \frac{12}{0.6}\ \text{sq.ft.}$ $\quad P = 29.90\times\frac{12}{0.6} = 448.5\ \text{tons/ft.}$

Sum. Pile shall be executed for Safty. (3×3 Cbs. about)

Check on Sliding

Max. hor. force on foundation
$= 1,710,000\#$

Sum. of vert. forces
$= 5,251,000\#$

Coef. of friction for concrete on gravel foundation $= 0.5$ say

Resistance to hor. force
$= 5,251,000\#\times 0.5$
$= 2,626,000\#$

Factor of Safty $= \frac{2,626,000}{1,710,000} = 1.53$

To increase the resistance to this force, give some inclination to the piles near the toe.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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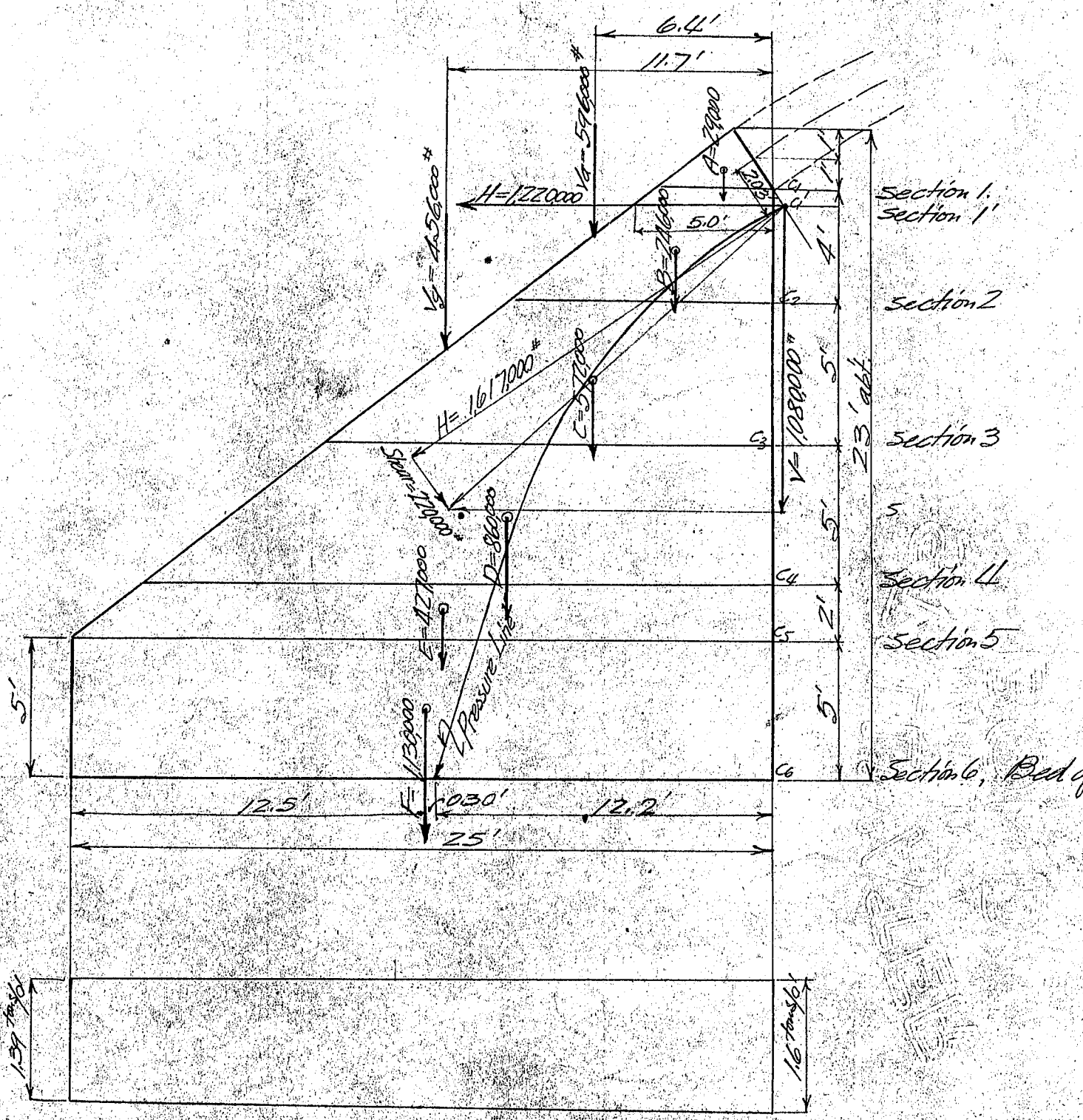
CALCULATIONS FOR

Muko-Chashi over the Mukogawa, Hyogoken.
Stability of Abutment body continued.
Case 4 Neg. moment max. with Temp. change. (max. pressure on heel.)



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（独立行政法人 土木研究所蔵）

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CONSULTING ENGINEER
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CALCULATIONS FOR

Muko-Ohashi over the Mukogawa, Hyogoken.
Stability of Abutment body for Arch span continued
Abutment Body.
Case 4. Neg. moment max. with Temperature change (heel pressure max.)
Stability at Section 1' (moment about C,')

Forces	Magnitude	lever arm	moment about C.
H	1,220,000 #	0	0
V	1,080,000	-0.4	-432,000
A	29,000	1.8	52,000
			-380,000

$\Sigma V = 1{,}109{,}000$ #

Hor. distance of pt. of application of resultant from pt. C,' $= \frac{-380{,}000}{1{,}109{,}000} = -0.34'$

$Ecc = -(\frac{5}{2} + .34) = -2.84'$

At Section 2.

$M = -380{,}000 + 890{,}000 + 3{,}810{,}000 + 1{,}220{,}000 \times 3.3 = 8{,}346{,}000$ #

$V = 1{,}109{,}000 + 246{,}000 + 596{,}000 = 1{,}951{,}000$ #

$d_2 = \frac{8{,}346{,}000}{1{,}951{,}000} = 4.27'$ $\quad Ecc. = -(4.7 - 4.27) = -0.43$

at Section 3.

$M = 8{,}346{,}000 + 3{,}680{,}000 + 5{,}340{,}000 + 1{,}220{,}000 \times 5 = 23{,}466{,}000$

$V = 1{,}951{,}000 + 572{,}000 + 456{,}000 = 2{,}979{,}000$ #

$d_3 = \frac{23{,}466{,}000}{2{,}979{,}000} = 7.9'$ $\quad Ecc. = 7.9 - 7.9 = 0.0'$

At Section 5

$M = 23{,}466{,}000 + 8{,}300{,}000 + 5{,}080{,}000 + 1{,}220{,}000 \times 7 = 45{,}386{,}000$

$V = 2{,}979{,}000 + 860{,}000 + 427{,}000 = 4{,}266{,}000$ #

$d_5 = \frac{45{,}386{,}000}{4{,}266{,}000} = 10.63'$ $\quad Ecc. = 10.63 - 12.5 = -1.97'$

At Section 6 (Bed of foundation)

$M = 45{,}386{,}000 + 14{,}250{,}000 + 1{,}220{,}000 \times 5 = 65{,}736{,}000$

$V = 4{,}266{,}000 + 1{,}130{,}000 = 5{,}396{,}000$

$d_6 = \frac{65{,}736{,}000}{5{,}396{,}000} = 12.2'$ $\quad Ecc. = 12.2 - 12.5 = -0.30'$

Max. heel Pressure

$S = \frac{5{,}396{,}000}{25 \times 647} \pm \frac{5{,}396{,}000 \times 0.3 \times 6}{64.7 \times 25^2} = 3{,}340 \pm 240 = 3{,}580$ #/0' $= 1.60$ tons/0' Comp.

or $= 3{,}100$ #/0' $= 1.39$ tons/0' Comp.

OK.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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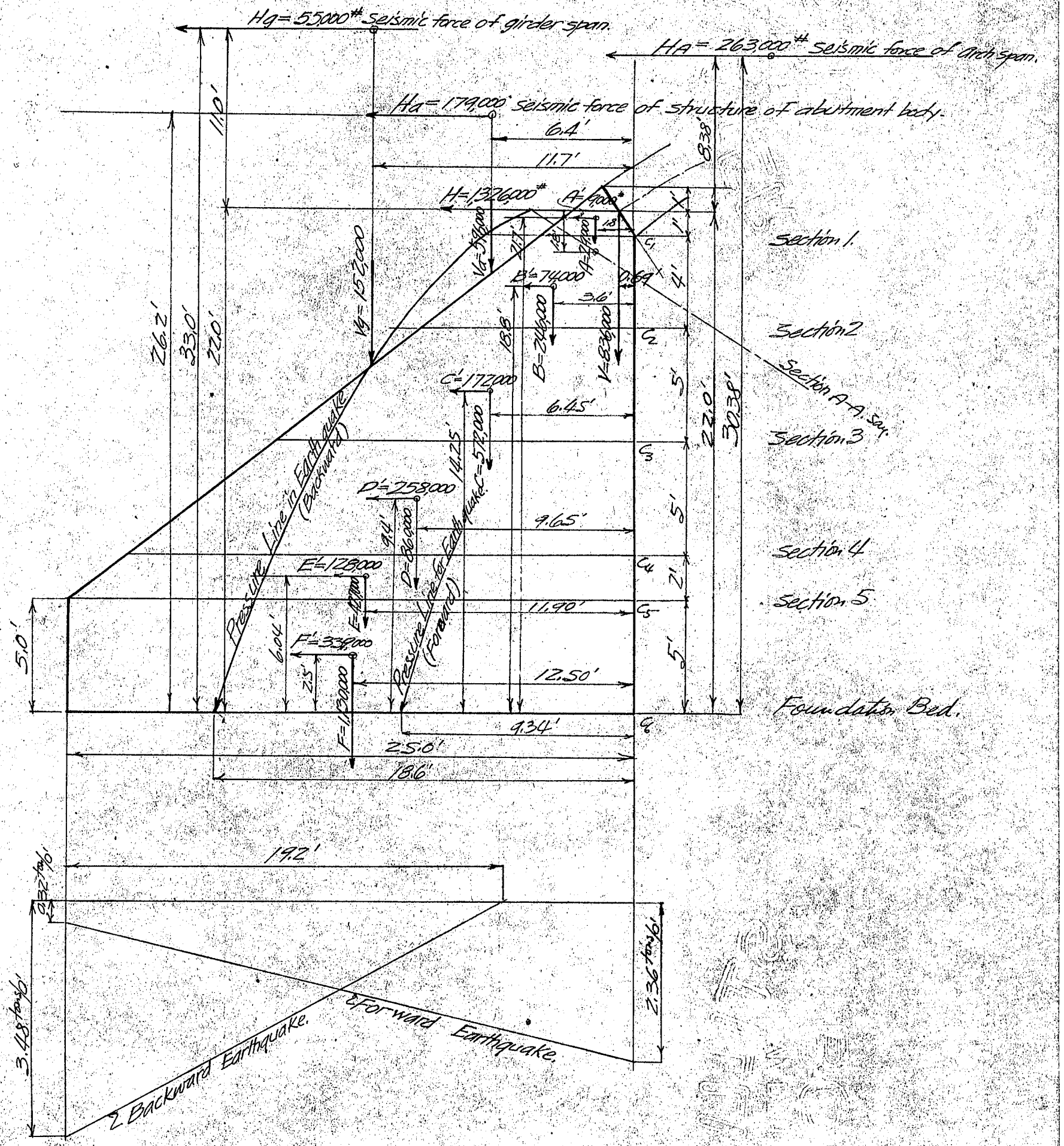
CALCULATIONS FOR

Muko-Gawa Bridge over the Mukogawa, Hyogoken.
Stability of Abutment for Arch span continued.
Seismic Stability of Abutment Body. Dead Load only (Temp change and rib shortening neglected)



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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Forces	Magnitude	Lever Arm	Moments about
			Centroid of Section
Hy	35000	33.20	+ 1,162,000
HA	263000	30.18	+ 7,936,000
Hx	199,000	26.80	+ 5,330,000
H	1,224,000	23.00	+ 28,150,000
V	450,000	1.80	+ 810,000
A	2,000	1.80	+ 3,600
A'	9,000	34.70	+ 312,000
V'	36,000	1.80	+ 64,800
B	24,000	3.60	+ 86,400
B'	11,000	18.80	+ 206,800
Vy	102,000	11.70	+ 1,193,400
E	52,000	6.45	+ 335,400
C	112,000	19.25	+ 2,156,000
D	860,000	9.65	+ 8,299,000
D'	256,000	9.40	+ 2,406,400
E	1,27,000	19.90	+ 5,280,000
E'	10,000	6.40	+ 70,400
F	1,30,000	12.50	+ 1,625,000
F'	33,200	2.30	+ 76,360
Total	Σ Vert. Forces = 4,838,000		+ 70,100,000
			+ 6,162,060
			- 27,462,060
			+ 45,140,000

Case of Seismic forces acting Eastward

Case of Seismic forces acting Westward

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKYO

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CALCULATIONS FOR

Moto-Machi on the Mitsugawa, Nagaoka

Section Stability of Retaining Wall Type Continued

Stability at Section 3 (Retaining Type originally Gravity Section Continued)

Hg	55000	11.00	610000
Hw	263000	16.33	4340000
Ha	179000	14.33	2560000
H	1326000	12.00	15900000
V	336000	9.67	3250000
A	29000	1.00	30000
A'	5000	3.00	15000
Va	579000	6.40	3700000
B	246000	3.50	860000
B'	74000	6.83	500000
Vg	152000	12.33	1870000
C	112000	6.83	760000
C'	138000	4.25	590000
ΣV = 2431000			33040000

$d_3 = \frac{33040000}{2431000} = 13.6'$

Eccentricity = 6.7'

ΣH = 2078000

Present Line of Section 3

Hg	55000	11.00	610000
HA	263000	9.83	2590000
Ha	179000	4.00	720000
H	1326000	3	
V	363000	0	
ΣV = 3561000			3560000

$Ecc. = \frac{3560000}{3561000} = 1.00'$

Stresses at Section 3-3

b = 64.7', b_1 = 4.0', b_2 = 3.5', $S_h = \frac{3.5}{64.7} = 0.075$

$d/h_2 = 1/4.0 = 0.25$

M_3 at 1/4 = 1326000 × 4.0 = 5304000

$p_3 = \frac{2M}{bh^2} = \frac{2 \times 5304000}{64.7 \times 16^2} = 0.0025$

K = 0.265, L = 0.975

Moment at Section 3-3

Hg	55000	12.80	640000
Hw	263000	10.18	2680000
Ha	179000	6.00	1070000
H	1326000	1.80	2390000
V	336000	−1.10	−370000
A	29000	0	
A'	5000	1.50	7500
			5377500

Allowed $f_{max} = \frac{5377500}{3.5^2} = 1680000$ #

$e_2 = \frac{5377500}{2431000 \times 0.975} = 2.26' = 27.2''$

$f_{max} = \frac{2431000}{64.7 \times 12} \left(1 + \frac{6 \times 27.2}{64.7}\right) = 4.15 \times 1.42 = 5.90$ #/□"

Stability at Section 4

M = 33040000 + 2878000 × 5 + 860000 × 9.67 + 6800000 × 11.4
= 33040000 + 10370000 + 5500000 + 6200000
= 55110000 #

ΣV = 2431000 + 860000 = 3291000 #

$d_4 = \frac{55110000}{3291000} = 16.8'$

Eccentricity = 16.8 − 12.5 = 4.3'

ΣH = 2078000 + 258000 = 2336000 #

Stability at Section 5

M = 55110000 + 2336000 × 12 + 427000 × 11.9 + 128000 × 10.67
= 55110000 + 28032000 + 5082000 + 1365000
= 89589000 #

ΣV = 3291000 + 427000 = 3718000

$d_5 = \frac{89589000}{3718000} = 24.1'$

Ecc. = 24.1 − 12.5 = 11.6'

ΣH = 2336000 + 128000 = 2464000 #

Check for Sliding

Max. horizontal force acting on foundation in earthquake = 2800000 #

$\left(\text{or } \frac{2800000}{64.7 \times 25.0}\right) = 0.70$ #/□" = 0.70 ton/□'

Sum of vertical forces = 4808000 #

Coefficient of friction for concrete on gravel foundation = say 0.6 + earthquake,

Resistance to horizontal force

= 0.6 × 4808000 = 2900000 #

To increase this resistance, give some inclination to the foundation soil as near the toe as (1/3 = 0.1)

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY [illegible] DATE 13-10-2[illegible] FILE NO
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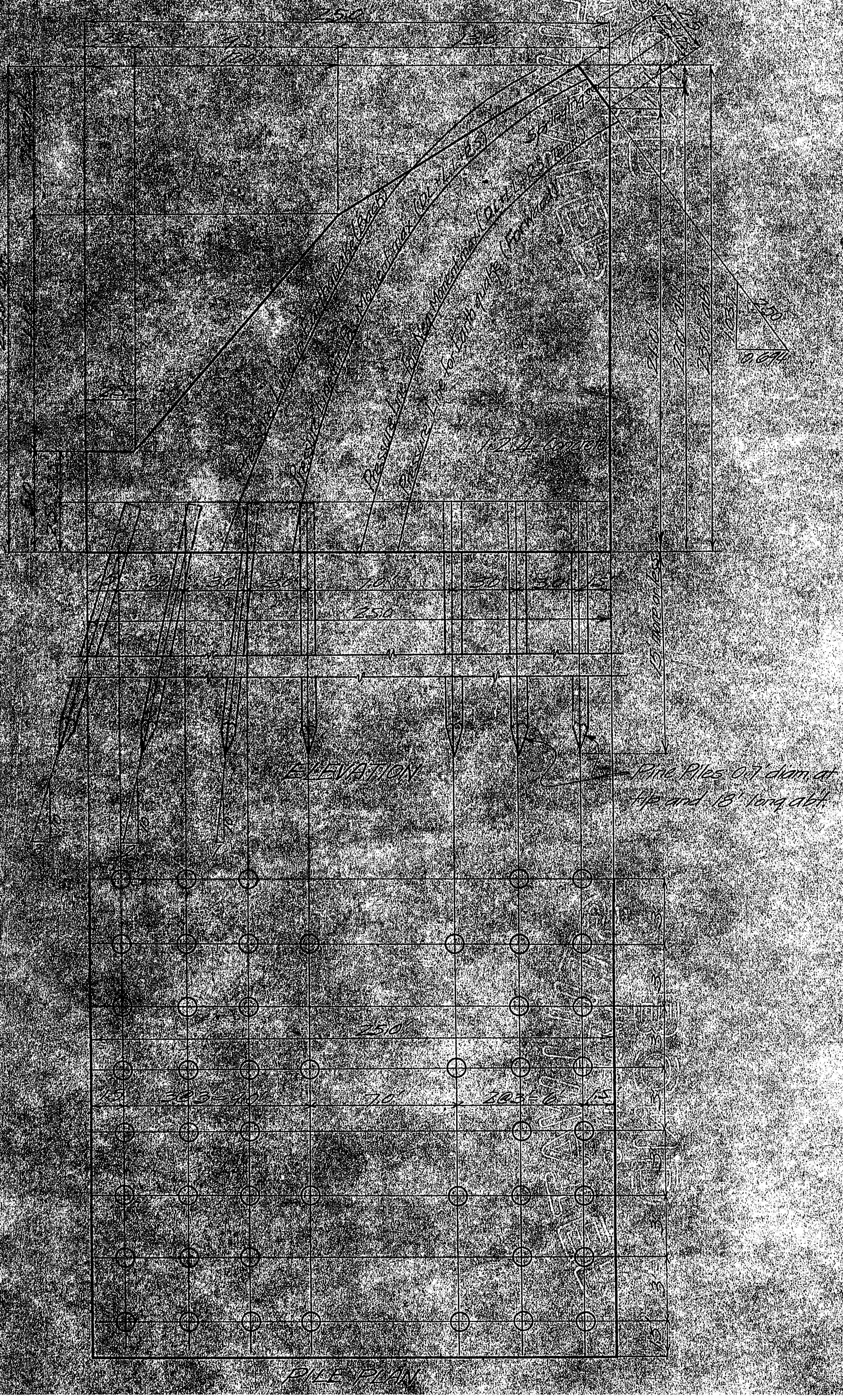
CALCULATIONS FOR

[illegible]
Summary of [illegible] Lines and [illegible] Abutment Body



增田淳氏関係資料
(独立行政法人 土木研究所蔵)

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Inaba DATE 13-10-26 FILE NO

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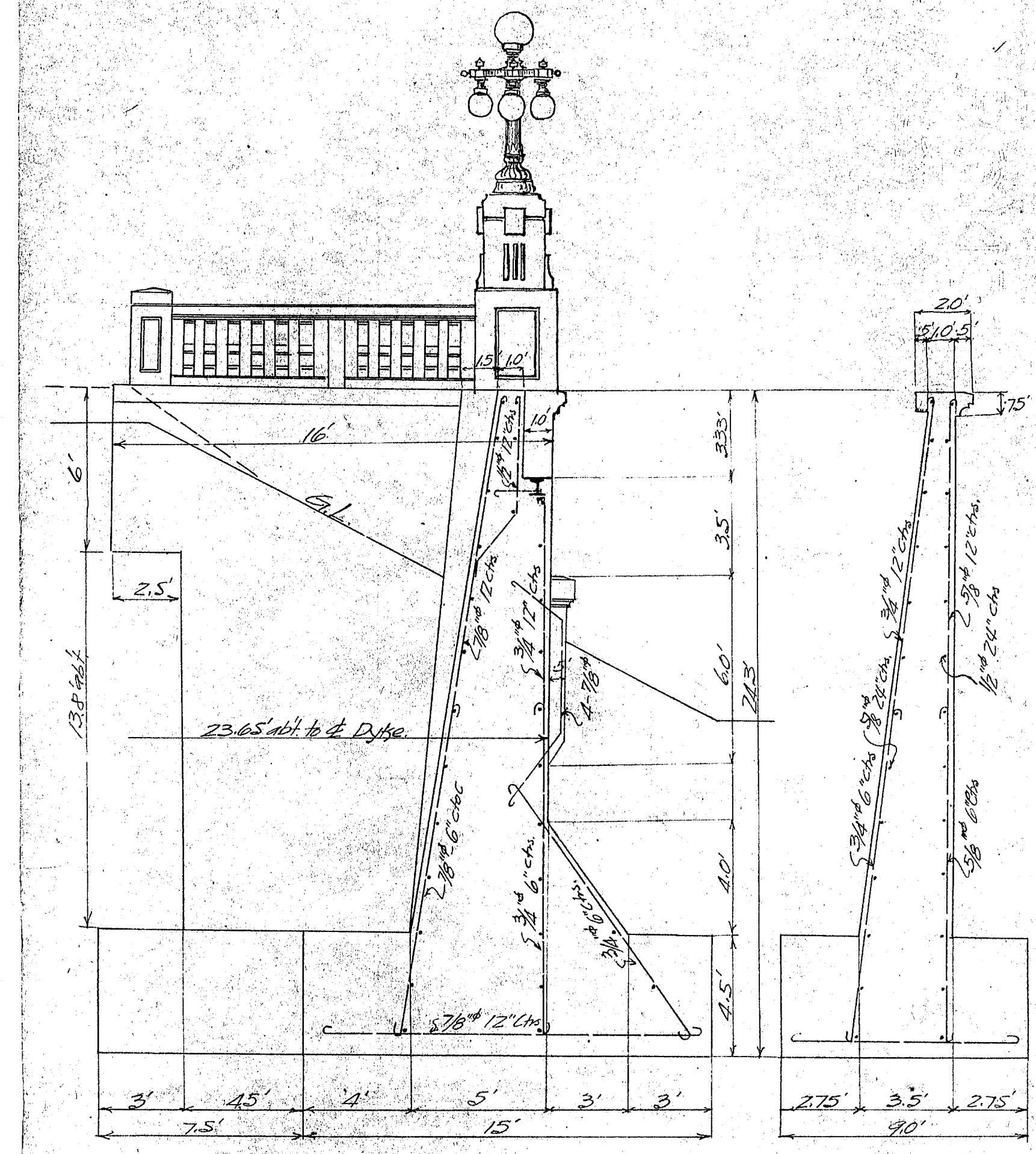
CALCULATIONS FOR

Muko-Ohashi over the Mukogawa, Hyogoken.
Design of Abutment for Approach Girder Span.



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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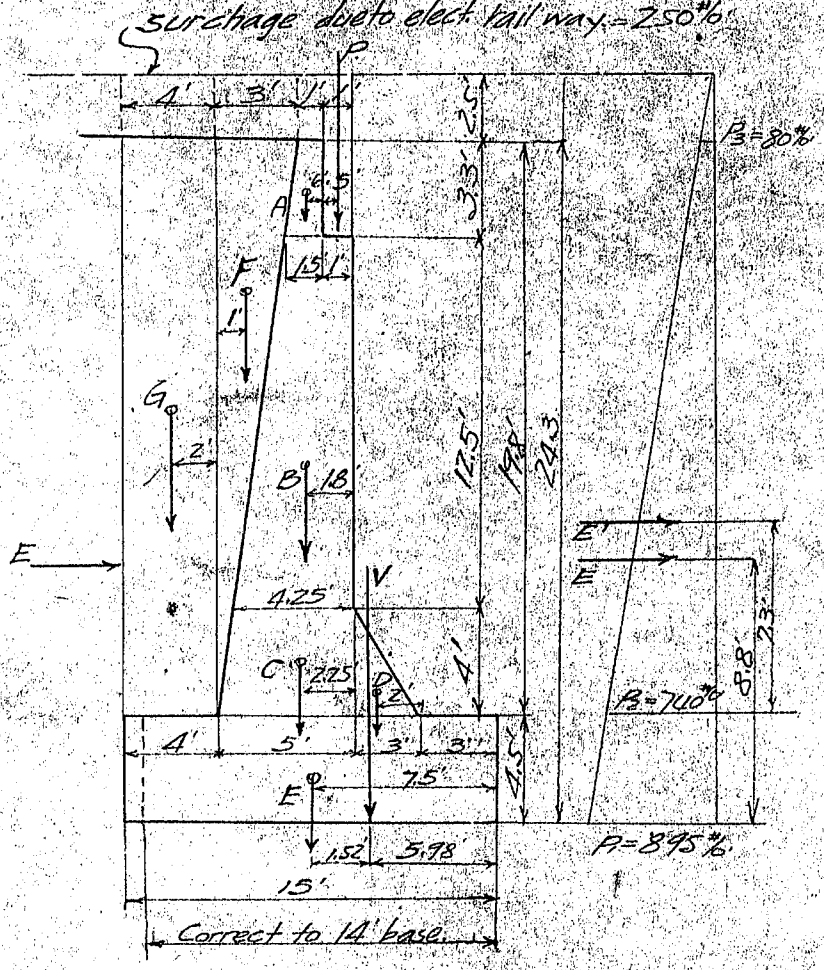
CALCULATIONS FOR

Muko-Ohashi over the Mukogawa, Hyogoken.
Abutment for Approach Girder span continued.
Stability of Abutment body



Earth Pressure.

$P_1 = \frac{100 \times 26.8}{3} = 895$ #/f' $E = \frac{895+80}{2} \times 24.3 = 11{,}800$ #

$P_2 = \frac{100 \times 19.8}{3} = 740$ $E' = \frac{740+80}{2} \times 19.8 = 8{,}100$ #

$P_3 = \frac{100 \times 2.5}{3} = 80$

Max. toe pressure for full live load.

mass	wt	lever arm	moment about toe	
A	620#	7.6'	4,700 ft#	12.5×33@150
B	6340	7.8	48,700	$\frac{2.5\times2.5}{2}$ 12.5@150
C	2770	8.25	22,800	$\frac{5+4.25}{2}\times4$@150
D	900	5.0	4,500	$\frac{3\times4}{2}$@150
E	10,100	7.5	75,700	4.5×15@150
F	2970	10.0	29,700	$\frac{3\times19.8}{2}$@100
G	7920	13.0	103,000	4×19.8@100
P	7100	6.5	46,100	$\frac{3\times16.4\times4}{17.9}=7100$#/f'
E	11,800	−8.8	−103,800	
			231,400 ft#	

(19820#)

Σ vert force = 38,720#

d = 5.98' from toe.

Eccentricity = 7.5 − 5.98 = 1.52'

max. toe pressure for full live load.

$S = \frac{38{,}720}{15} \pm \frac{38{,}720 \times 1.52 \times 6}{15^2} = 2580 \pm 1570$

$= 4{,}150$ #/f' $= 1.85$ tons/f' OK.

or $= 1{,}010$ " $= 0.45$ "

Correct the base to be 14' wide cutting off 1ft at heel approx. max. toe pressure. Ecc = 7 − 5.98 = 1.02'

$S = \frac{38{,}720}{14} \pm \frac{38{,}720 \times 1.02 \times 6}{14^2} = 2{,}760 \pm 1{,}210$

$= 3{,}970$ #/f' $= 1.78$ tons/f' OK.

or $= 1{,}550$ " $= 0.70$ "

Stresses at Bottom of Wall.

mass	wt.	lever arm	moment abt. toe of wall.
A	620 #	4.6'	2,900 ft#
B	6340	4.8	30,400
C	2,770	5.25	14,500
D	900	2.0	1,800
F	2970	7.0	20,800
P	7100	3.5	24,800
E'	8,100	−7.3	−59,100
ΣV =	20,700#		36,100 ft#

$Ecc. = \frac{8}{2} - \frac{36100}{20700} = 4.0 - 1.75$

$= 2.25'$

$M = 28800 \times 2.25 = 47{,}000$ ft#

steel req'd. $= \frac{47000 \times 12}{17000 \times 7/8 \times 94} = 0.405$ "

For 7/8"φ 6" c to c = 1.20 "/f', $p = 0.00106$, $k = 0.20$

$f_s = \frac{47000 \times 12}{1.2 \times 7/8 \times 94} = 5{,}700$ #/" OK.

$f_c = \frac{2 \times 5700 \times 0.00106}{0.2} = 61$ #/" OK

Stresses of wall at 4 f from bottom of wall.

mass	wt.	lever arm	moment at centre of Section.
A	620#	−0.9'	−560
B	6340	−0.7	−4,430
F	2,050	1.1	2,260
P	7,100	−2.0	−14,200
E''	5500	−6.1	−33,500
ΣV =	16,110#		−54,950 ft#

$Ecc. = \frac{-54950}{16110} = -3.4'$

Steel req'd. $= \frac{54950 \times 12}{17000 \times 7/8 \times 49} = 0.905$ "/f'

use 7/8"φ 6" c to c = 1.20 "/f'. $p = 0.00204$, $k = 0.22$

$f_s = \frac{54950 \times 12}{1.2 \times 7/8 \times 49} = 12{,}800$ #/" OK

$f_c = \frac{2 \times 12800 \times 0.00204}{0.22} = 238$ #/" OK.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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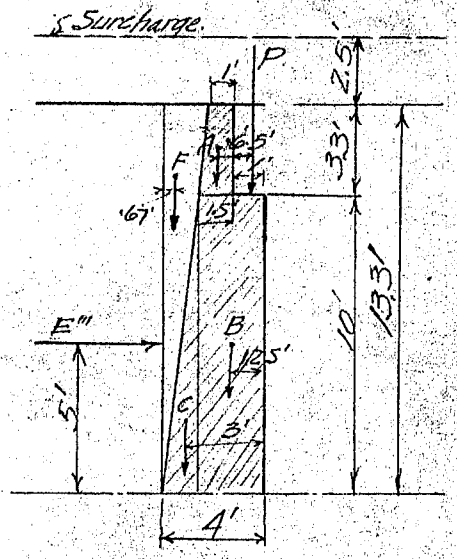
CALCULATIONS FOR

Muko-Okashi over the Mukogawa, Hyogo-ken.
Stability of Abutment body continued.
At 13.3 ft from top of wall.



Earth pressure $E''' = \frac{15.8^2}{6} \times 10 = 4,160^{\#}$

	mass, wt.	leverarm	moment about ℄ of ~~base~~ Section.
A	620#	−0.4'	−250
B	3,750	−0.75	−2,810
C	1,120	+1.00	+1,120
F	1,330	+1.33	+1,770
P	7,100	−1.5	−10,650
E'''	4,160	−5.0'	−20,800
ΣV = 13,920#			−31,620'#

$Ecc. = \frac{-31,620}{13,920} = -2.27'$

Steel req'd $= \frac{31,620 \times 12}{17,000 \times \frac{7}{8} \times 46} = 0.555$ □"/ft.

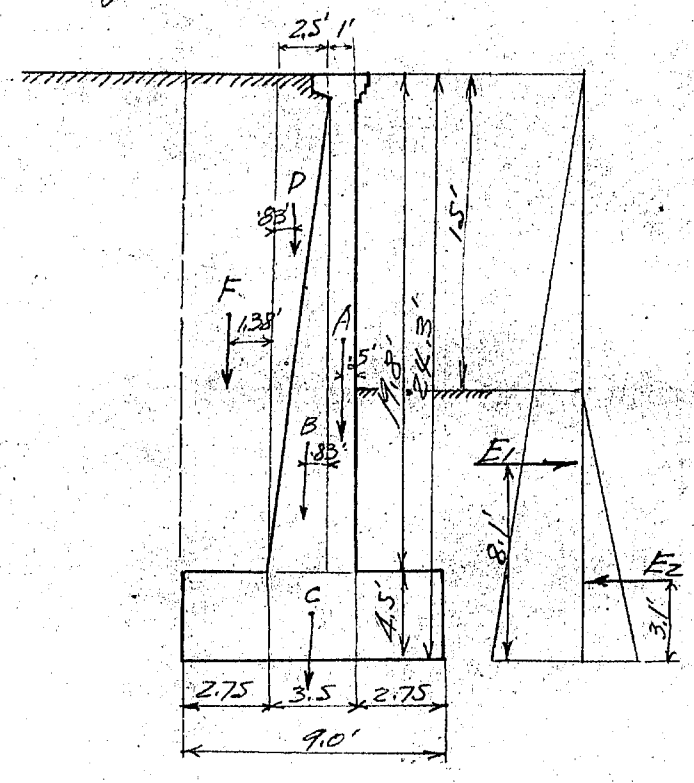
use 7/8" ϕ bars at 12" ctrs = 0.601 □"

$p = \frac{.601}{12 \times 46} = .0011$ $K = 0.2$ $j = 0.93$

$f_s = \frac{31620 \times 12}{0.601 \times .93 \times 46} = 14,800^{\#}/□"$ Ok

$f_c = \frac{2 f_s p}{K} = \frac{2 \times 14,800 \times 0.0011}{0.2} = 163^{\#}/□"$ Ok.

Wing wall.



Max diff. of earth fills causing earth pressure = say 15'

$E_1 = \frac{24.3^2 \times 100}{6} = 9,800^{\#}$

$E_2 = \frac{9.3^2 \times 100}{6} = 1,400^{\#}$

	mass	force	lever arm	moment abt. centre of base.
A	3,000#		−1.25'	−3,800'#
B	3,700		+0.08	+ 300
C	6,100		0	0
D	2,500		+0.92	+ 2,300
F	5,400		+3.13	+16,900
E_1	9,800		−8.1	−79,400
E_2	1,400		+3.1	+ 4,300
	ΣV = 20,700			−59,400'#

$Ecc. = \frac{-59,400}{20,700} = -2.86'$ $a = 4.5 - 2.86 = 1.64$, $3a = 4.92'$

Max. toe pressure neglecting tension,

$S = \frac{2V}{3ab} = \frac{2 \times 20,700}{4.92 \times 1'} = 8,420^{\#}/□' = 3.75^{\text{tons}}/□'$ Ok.

At Bottom of Wall. (top of base).

	mass	wt.	leverarm	moment abt ℄ of section.
A	3,000#		−1.25'	−3,800'#
B	3,700		+0.08	+ 300
D	2,500		+0.92	+ 2,300
E_1'	6,500		−6.6	+42,900
E_2'	400		+1.6	+ 600
	ΣV = 9,200#			−43,500#

Some piles req'd.

$E_1' = \frac{19.8^2 \times 100}{6} = 6,500^{\#}$

$E_2' = \frac{4.8^2 \times 100}{6} = 400$

$Ecc. = \frac{43,500}{9,200} = 4.73'$

Steel req'd $= \frac{43500 \times 12}{17000 \times \frac{7}{8} \times 40} = 0.88$ □"/ft.

use 3/4" ϕ base at 6" ctrs. = 0.884 □"/ft $p = 0.00184$

$K = 0.215$, $j = 0.93$

$f_s = \frac{43500 \times 12}{0.884 \times 0.93 \times 40} = 15,900^{\#}/□"$ Ok.

$f_c = \frac{2 \times 15,900 \times 900/84}{0.215} = 272^{\#}/□"$ Ok.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

MADE BY K. Inaba DATE 13-11-3 FILE NO
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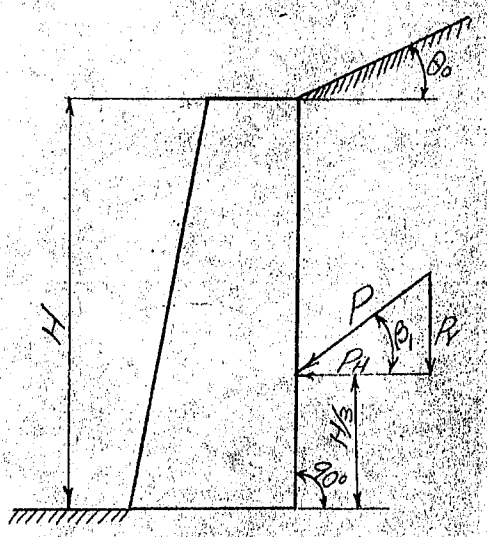
CALCULATIONS FOR

Muko-Gawa over the Mukogawa, Hyogoken.
Seismic Stability of Abutment for Approach girder span.
Earth Pressure in Earthquake.



Let φ = Angle of repose of earth.
θ_0 = Surcharge angle of earth fill.
H = Height of fill wall.
W = Unit weight of earth fill.
Then, $\theta = \tan^{-1} K$

Earth pressure in quake.

$$P = \frac{WH^2}{2}\sqrt{1+K^2}\,\frac{\cos\theta_0\{[\cos(\theta_0-\theta)-\sqrt{\cos^2(\theta_0+\theta)-\cos^2\varphi}]^2+[\sin(\theta_0+\theta)-\sin(\theta_0-\theta)]^2\}^{\frac{1}{2}}}{\cos^2\theta\{\cos(\theta_0+\theta)+\sqrt{\cos^2(\theta_0+\theta)-\cos^2\varphi}\}}$$

for $\theta_0=0$

$$P = \frac{WH^2}{2}\sqrt{1+K^2}\,\frac{\{[\cos\theta-\sqrt{\cos^2\theta-\cos^2\varphi}]^2+[\sin\theta+\sin\theta]^2\}^{\frac{1}{2}}}{\cos^2\theta\{\cos\theta+\sqrt{\cos^2\theta-\cos^2\varphi}\}} = \frac{WH^2}{2}A\frac{B}{C}$$

for $K=0.30$, $\varphi=30°$;
$\theta=\tan^{-1}K=\tan^{-1}0.30=16°42'$
$\cos\theta=0.9578$, $\sin\theta=0.2874$, $\cos^2\theta=0.9174$, $\sin^2\theta=0.0826$
$\cos\varphi=0.8660$, $\cos^2\varphi=0.7500$

$A=\sqrt{1+0.3^2}=\sqrt{1.09}=1.044$
$B=\{[0.9578-\sqrt{0.9174-0.7500}]^2+4\times0.0826\}^{\frac{1}{2}}=\sqrt{0.5487^2+0.3304}=\sqrt{0.6315}=0.7947$
$C=0.9174\{0.9578+\sqrt{0.9174-0.7500}\}=0.9174(0.9578+0.4091)=1.2540$

$$P=\frac{WH^2}{2}A\frac{B}{C}=\frac{WH^2}{2}\times1.044\times\frac{0.7947}{1.2540}=\frac{WH^2}{2}\times0.6616$$

$$\tan\beta=\tan(\theta_0+\theta)+\frac{\sin\theta}{\cos\theta_0\cos(\theta_0+\theta)+\dfrac{\cos(\theta_0+\theta)-\sqrt{\cos^2(\theta_0+\theta)-\cos^2\varphi}}{\cos(\theta_0+\theta)+\sqrt{\cos^2(\theta_0+\theta)-\cos^2\varphi}}}$$

for $\theta_0=0$

$$\tan\beta=\tan\theta+\frac{\sin\theta}{\cos\theta+\dfrac{\cos\theta-\sqrt{\cos^2\theta-\cos^2\varphi}}{\cos\theta+\sqrt{\cos^2\theta-\cos^2\varphi}}}$$

$$=0.3000+\frac{0.2874}{0.9578+\dfrac{0.5487}{1.3669}}=0.3000+\frac{0.2874}{1.3592}=0.3000+0.2115=0.5115$$

$\therefore \beta = 27°-05'$ $\cos\beta=0.8903$, $\sin\beta=0.4553$

$$P_V=P\sin\beta=\frac{WH^2}{2}\times0.6616\times0.4553=\frac{WH^2}{2}\times0.302=0.151\,WH^2$$

$$P_H=P\cos\beta=\frac{WH^2}{2}\times0.6616\times0.8903=\frac{WH^2}{2}\times0.589=0.295\,WH^2$$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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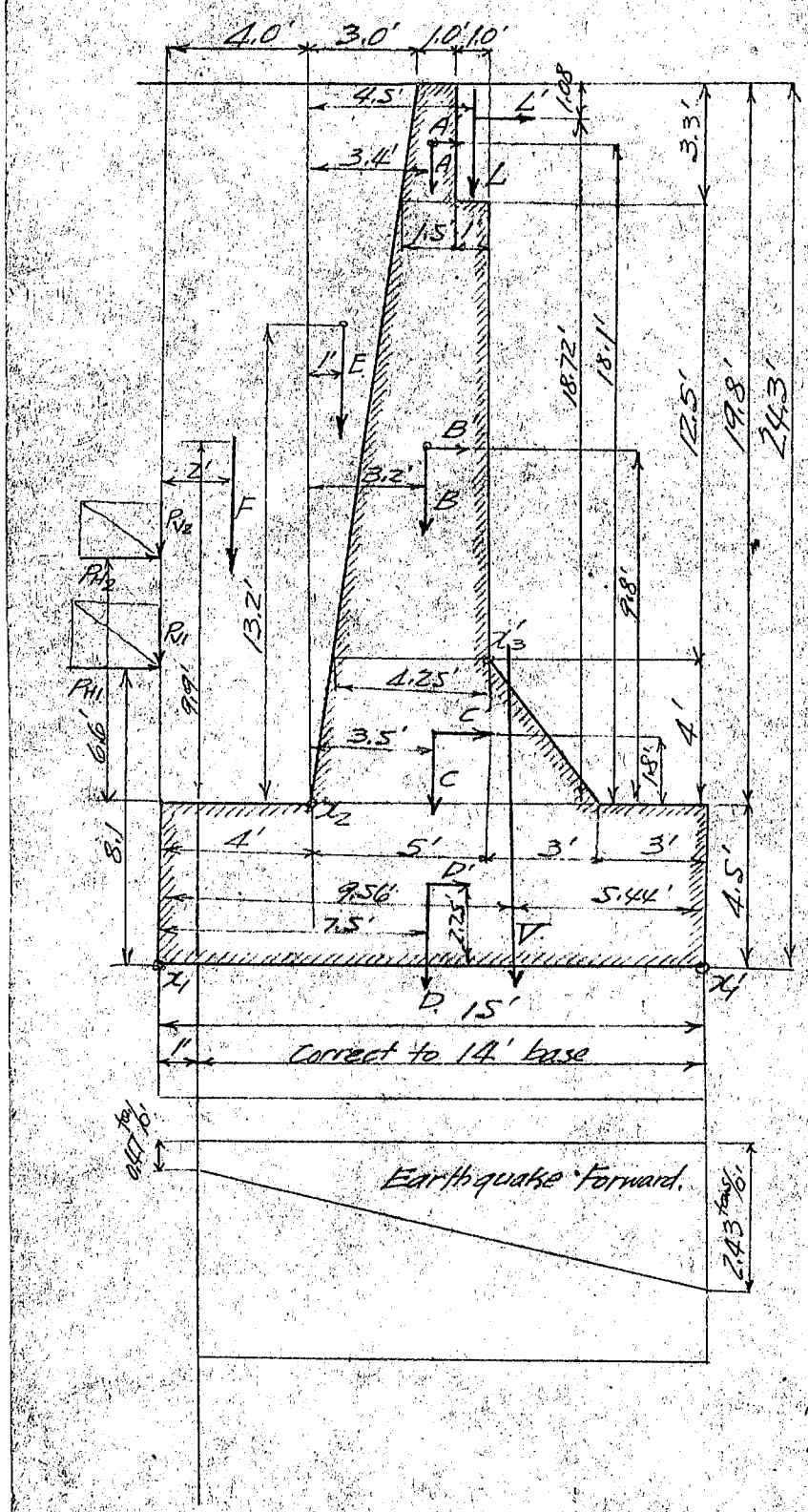
CALCULATIONS FOR

Mukō-Gawa Bridge over the Mukōgawa, Hyogo-ken.
Seismic Stability of Abutment continued.



Weights and seismic forces of several masses,
K=0.30 for seismic force.

mass	wt.	K	Seismic force	Dead Load due to girder, under Ry.
L	2,800# × 0.3		800# = L'	13800# ÷ 4.92 = 2,800#/Aq wall
A	600	"	200 = A'	$\frac{1.5+1.0}{2}$ × 3.3 @ 150 = 600
B	6,300	"	1,900 = B'	$\frac{4.25+2.5}{2}$ × 12.5 @ 150 = 6,300
C	3,700	"	1,100 = C'	$\frac{4.25+8}{2}$ × 4 @ 150 = 3,700
D	9,500	"	2,800 = D'	4.5 × 15 @ 140 = 9,500
E	3,000	"	none	$\frac{3 \times 19.8}{2}$ @ 100 = 3,000
F	7,900	"	none.	4 × 19.8 @ 100 = 7,900
	33,800#		6,800#	

for Bed of foundation,

$P_{V1} = 0.151\,WH^2 = 0.151 \times 100 \times 24.3^2 =$ 8,900#
$P_{H1} = 0.295\,WH^2 = 0.295 \times 100 \times 24.3^2 =$ 17,400
$\Sigma V = 33,800 + 8,900 = 42,700$#
$\Sigma H = 6800 + 17,400 = 24,200$

for bottom of wall.

$P_{V2} = 0.151 \times 100 \times 19.8^2 =$ 5,900
$P_{H2} = 0.295 \times 100 \times 19.8^2 =$ 11,600
$\Sigma V = 33,800 + 5900 = 39,700$#
$\Sigma H = 6800 + 11,600 = 18,400$

Stability at bed of foundation. (Forward)

forces	magnitude	lever arm	moment abt x_1	Earth fill in front of abutment neglected.
L	2,800#	8.5'	23,800'#	
L'	800	23.22	18,600	
A	600	7.4	4,400	
A'	200	22.6	4,500	
B	6,300	7.2	45,300	
B'	1,900	14.3	27,200	
C	3,700	7.5	27,700	
C'	1,100	6.3	6,900	
D	9,500	7.5	71,200	
D'	2,800	2.25	6,300	
E	3,000	5.0	15,000	
F	7,900	2.0	15,800	
P_{V1}	8,900	0	0	
P_{H1}	17,400	8.1	141,000	

$\Sigma M = 407,700$'#

dist. of pt. of application of resultant force from x_1,

$d_1 = \frac{407,700}{42,700} = 9.56'$

$Ecc_1 = 7.5 - 9.56 = -2.06'$

max. toe pressure

$S = \frac{42700}{15} \pm \frac{42700 \times 2.06 \times 6}{15^2} = 2843 \pm 2340$

$= 5183$ #/0' $= 2.32$ tons/0'
$\sigma_2 = 503$ #/0' $= 0.22$ "

try for 14' base cutting 1ft at heel, approx. max. toe pressure

$Ecc. = 9.56 - 1.0 - 7.0 = 1.56'$

$S = \frac{42700}{14} \pm \frac{42700 \times 1.56 \times 6}{14^2} = 3250 \pm 2200$

$= 5450$ #/0' $= 2.43$ tons/0' ok.
$= 1050$ " $= 0.47$ "

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
MARUNOUCHI, TOKIO

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CALCULATIONS FOR

Muko-Ohashi over the Mukogawa, Hyogoken,
Seismic stability of Abutment continued.
Backward earthquake.

forces	magnitude	lever arm	moment abt. x_1'
L	2,800#	6.5'	18,200'#
L'	800	23.22	18,600
A	600	7.6	4,600
A'	200	22.6	4,500
B	6,300	7.8	49,100
B'	1,900	14.3	27,200
C	3,700	7.5	27,800
C'	1,100	6.3	6,900
D	8,800	8.0	70,300
D'	2,600	2.25	5,900
E	3,000	10.0	30,000
F	5,900	12.5	74,000 (for 16' base)
P_{H1}'	7,600	5.33	40,500
P_{V1}'	3,900	0	0
	$\Sigma V = 35,000$		377,600'#

$P_{H1}' = 0.295 \times 100 \times 16^2 = 7,600\#$
$P_{V1}' = 0.151 \times 100 \times 16^2 = 3,900$

$d = \frac{377,600}{35,000} = 10.75'$ from x_1'

$Ecc. = 10.75 - 7 = 3.75'$

Tension occurs.
neglecting tension,

$S = \frac{2 \times 35000}{3(7-3.75)\times 1} = 7,200\#/\square' = 3.20$ tons/$\square'$ Ok.
$(= 1.75 \times 1.8)$

Stresses at Bottom of Wall. (Forward quake)

forces	magnitude	lever arm	moment abt. x_2
L	2,800#	4.5'	12,600'#
L'	800	18.72	15,000
A	600	3.4	2,000
A'	200	18.1	3,600
B	6,300	3.2	20,200
B'	1,900	9.8	18,600
C	3,700	3.5	13,000
C'	1,100	1.8	2,000
P_{H2}	11,600	6.6	76,600
P_{V2}	5,900	−4.0	−23,600
	$\Sigma V = 19,300\#$		140,000'#

$d = \frac{140,000}{19,300} = 7.26'$ from x_2

Correct the bottom of wall to be 7' thick as shown

$Ecc. = 7.26 - 3.5 = 3.76'$

$M = 19,300 \times 3.76 = 72,500'\#$

Steel req'd $= \frac{72,500 \times 12}{30000 \times 7/8 \times 80} = 0.415\,\square''$

Steel used for full live load $= 1.20\,\square''$ Ok.

Stresses at Section 4' from bottom.

In case, the Seismic force acts backward the most dangerous section is at 4' above bottom of wall.

Stresses at Section 4' above bottom of wall (backward quake).

forces	magnitude	lever arm	moment abt. x_3'
L	2,800#	0.5'	1,400'#
L'	800	14.72	11,800
A	600	1.6	1,000
A'	200	14.1	2,800
B	6,300	1.8	11,300
B'	1,900	5.8	11,000
P_{H3}'	1,900	2.67	5,100
	$\Sigma V = 9,700\#$		44,400

$d = \frac{44,400}{9,700} = 4.58'$ from x_3'

$Ecc. = 4.58 - 2.13 = 2.45'$

$M = 9,700 \times 2.45 = 23,800'\#$

Steel req'd $= \frac{23800 \times 12}{30000 \times 7/8 \times 48} = 0.226\,\square''/ft.$

use 3/4"ϕ bars at 12" c to c $= 0.442\,\square''$

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HYOGO KEN

MADE BY K. Inaba. DATE 47-6-17 FILE NO

CHECKED BY I.N. DATE 47-6-17 PAGE NO 116'

CALCULATIONS FOR

Muko-o-Hashi over the Mukogawa.

Index of Marks for Reinforcement.

F.S.	Reinforcements for Floor Slab.	
F.B.	" " Floor Beams.	
F.B.U.	" " " " U-Stirrups.	
F.C.2.S.	" " Floor Columns at Panel Pt. #2	under Side walk.
F.C.3.T.	" " " " " " #3	" Traffic way
F.C.5.R.	" " " " " " #5	" Electric Railway
F.C.h.	" " " " , hoops.	
B.R.A.	" " Brackets for Arch Span.	
P.A.V.	" " Pier for Arch Span, Vertical bars.	
P.A.H.	" " " " " , Horizontal " .	
C.B.	" " Column base.	
P.A.I.	" " Pier for Arch Span, Inclined bars.	
P.A.S.	" " " " " , Stirrups.	
A.A.H.	" " Abutment for Arch Span, Horizontal bars.	
A.A.V.	" " " " " , Vertical "	
A.A.I	" " " " " , Inclined "	
A.A.S.	" " " " " , Stirrups.	
A.A.C.H.	" " " " " , hoops of Column AAC2.	
A.A.B.	" " " " " , Bracket.	
R.F.	" " Refuge Bay.	
G.f.	" " Girders under Side walk for Girder Span. (Fascia girder)	
G.r.	" " " Electric Railway for Girder Span.	
G.t.	" " " Traffic way " " "	
G.S.	" " Girders, Stirrups.	
F.g.B	" " Brackets of Floor for Girder Span.	
P.g.V.	" " Pier for Girder Span, Vertical bars.	
P.g.S.	" " " " " , Stirrups.	
P.g.h.	" " " " " , Horizontal bars.	
G.A.V.	" " Abutment for Girder Span, Vertical bars.	
G.A.H.	" " " " , Horizontal " .	
H.	" " Handrail (Bottom rails and Intermediate posts.).	
C.T.P.	" " Centre Pole base.	

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HYOGO KEN

MADE BY K. Inaba. DATE 14-5-13 FILE NO

CHECKED BY T.N. DATE 14-5-21 PAGE NO 116.

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

Material List.

Reinforcements for Arch Span.

Slab Reinforcements. Sheet No. 5.

End Span. 2 required.

Mark	req'd no	dia	length	wt/ft.	wt/piece	total wt.	remarks
FS 1	71.5	1/2"	21.0'	0.668#	14.03#	1,003.1#	
FS 2	132	1/2	25.5	"	17.03	2,248.0	
FS 3	63.5	1/2	24.9	"	16.63	1,056.0	
FS 4	143	1/2	18.9	"	12.63	1,806.1	
FS 5	264	1/2	22.7	"	15.16	4,002.2	
FS 6	127	1/2	22.1	"	14.76	1,874.5	
FS 7	116	3/8	10.5	0.376	3.95	458.2	
FS 8	216	3/8	9.1	"	3.42	738.7	
FS 9	104	3/8	7.3	"	2.74	285.0	
FS 10	132	1/2	17.0	0.668	11.36	1,499.5	
FS 11	36	3/8	17.0	0.376	6.39	230.0	
FS 12	66	1/2	14.0	0.668	9.35	617.1	
FS 13	18	3/8	14.0	0.376	5.26	94.7	
FS 14	5	3/8	12.0	"	4.51	22.6	
FS 15	8	3/8	10.6	"	3.99	31.9	
FS 16	3	3/8	8.8	"	3.31	9.9	
FS 17	1	3/8	8.0	"	3.01	3.0	
FS 18	1	3/8	9.4	"	3.53	3.5	
FS 19	1	3/8	11.4	"	4.29	4.3	
FS 20	66	1/2	21.2	0.668	14.16	934.6	
FS 21	18	3/8	21.2	0.376	7.97	143.5	
				Total	=	17,066.4#	= 7.619 tons.

Intermediate Span. 4 required.

Mark	req'd no	dia	length	wt/ft.	wt/piece	total wt.	remarks
FS 1	63	1/2"	21.0	0.668#	14.03#	883.9#	
FS 2	116	1/2	25.5	"	17.03	1,975.5	
FS 3	55	1/2	24.9	"	16.63	914.7	
FS 4	126	1/2	18.9	"	12.63	1,591.4	
FS 5	232	1/2	22.7	"	15.16	3,517.1	
FS 6	110	1/2	22.1	"	14.76	1623.6	
FS 7	116	3/8	10.5	0.376	3.95	458.2	
FS 8	216	3/8	9.1	"	3.42	738.7	
FS 9	104	3/8	7.3	"	2.74	285.0	
FS 10	132	1/2	17.0	0.668	11.36	1,499.5	
FS 11	36	3/8	17.0	0.376	6.39	230.0	
FS 12	132	1/2	14.0	0.668	9.35	1,234.2	
FS 13	36	3/8	14.0	0.376	5.26	189.4	
FS 14	10	3/8	12.0	"	4.51	45.1	
FS 15	16	3/8	10.6	"	3.99	63.8	
FS 16	6	3/8	8.8	"	3.31	19.9	
FS 17	2	3/8	8.0	"	3.01	6.0	
FS 18	2	3/8	9.4	"	3.53	7.1	
FS 19	2	3/8	11.4	"	4.29	8.6	
				Total	=	15,291.7#	= 6.827 tons.

Beam Reinforcements. Sheet No. 7

End Span. 2 required.

Mark	req'd no	dia	length	wt/ft.	wt/piece	total wt.	remarks
FB 1	40	3/4"	14.6	1.502#	21.93#	877.2#	Beam B.C.D.E & F
FB 2	40	3/4	16.8	"	25.23	1,009.2	"
FB 3	44	3/4	12.4	"	18.62	819.3	Beam A.B.C.D.E & F
FB 4	40	3/4	16.5	"	24.78	991.2	" B.C.D.E & F
FB 5	44	3/4	14.0	"	21.03	925.3	" A.B.C.D.E & F
FB 6	56	3/4	14.3	"	21.48	1,202.9	" "
FB 7	56	3/4	16.6	"	24.93	1396.1	" "
FB 8	4	3/4	16.8	"	25.23	100.9	" A
FB 9	8	1/2	14.0	0.668	9.35	74.8	Fascia girder
FB 10	8	1/2	15.2	"	10.15	81.2	"
FB 11	4	1/2	12.4	"	8.28	33.1	"
FB 12	4	1/2	17.0	"	11.36	45.4	"
FB 13	4	1/2	12.2	"	8.15	32.6	"
FB 14	8	1/2	13.8	"	9.22	73.8	"
FB 15	8	1/2	15.0	"	10.02	80.2	"
FB 16	48	3/4	9.9	1.502	14.87	713.8	Beam A.B.C.D.E & F on abutment.
FB 17	22	3/4	11.5	"	17.27	379.9	
FBU 1	880	3/8	3.4	0.376	1.28	1,126.4	Beam B.C.D.E & F
FBU 2	88	3/8	3.3	"	1.24	109.1	" A
FB 19	24	1/2	5.5	0.668	3.67	88.1	Beam E.F.& Fas. gir.
FB 20	40	1/2	6.5	"	4.34	173.6	" A.B.C.D.& Fas. gir.
FB 18	48	3/4	18.0	1.502	27.04	1297.9	Cross beam
FB 21	48	3/8	3.2	0.376	1.20	57.6	"
FB 22	36	3/8	4.6	"	1.73	62.3	"
FB 23	48	3/8	4.0	"	1.50	72.0	"
FB 24	20	3/8	2.6	"	0.98	19.6	"
				Total	=	11,843.5#	= 5.287 tons.

Intermediate Span. 4 required.

Mark	req'd no	dia	length	wt/ft.	wt/piece	total wt.	remarks
FB 1	40	3/4"	14.6	1.502#	21.93#	877.2#	Beam B.C.D.E & F
FB 2	40	3/4	16.8	"	25.23	1,009.2	"
FB 3	44	3/4	12.4	"	18.62	819.3	Beam A.B.C.D.E & F.
FB 4	40	3/4	16.5	"	24.78	991.2	" B.C.D.E & F.
FB 5	44	3/4	14.0	"	21.03	925.3	" A.B.C.D.E & F.
FB 6	56	3/4	14.3	"	21.48	1,202.9	"
FB 7	56	3/4	16.6	"	24.93	1396.1	"
FB 8	4	3/4	16.8	"	25.23	100.9	Beam A
FB 9	8	1/2	14.0	0.668	9.35	74.8	Fascia girder
FB 10	8	1/2	15.2	"	10.15	81.2	"
FB 11	4	1/2	12.4	"	8.28	33.1	"
FB 12	4	1/2	17.0	"	11.36	45.4	"
FB 13	4	1/2	12.2	"	8.15	32.6	"
FB 14	8	1/2	13.8	"	9.22	73.8	"
FB 15	8	1/2	15.0	"	10.02	80.2	"
FBU 1	720	3/8	3.4	0.376	1.28	921.6	Beam B.C.D.E & F.
FBU 2	72	3/8	3.3	"	1.24	89.3	" A
FBU 3	32	3/8	3.5	"	1.32	42.2	Fascia girder
FB 19	24	1/2	5.5	0.668	3.67	88.1	Beam E.F.& Fas. gir.
FB 20	40	1/2	6.5	"	4.34	173.6	" A.B.C.D.& Fas. gir.
FB 18	48	3/4	18.0	1.502	27.04	1297.9	Cross beam
FB 21	48	3/8	3.2	0.376	1.20	57.6	"
FB 22	36	3/8	4.6	"	1.73	62.3	"
FB 23	48	3/8	4.0	"	1.50	72.0	"
FB 24	20	3/8	2.6	"	0.98	19.6	"
				Total	=	10,567.4#	= 4.718 tons.

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HYOGO KEN

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CHECKED BY T.N. DATE 15-5-2 PAGE NO. 17

CALCULATIONS FOR

Muko-o-Hashi over the Mukogawa.

List of Reinforcements for Arch Span continued.

Column.

Column Reinforcement for one Span. Sheet No. 7.

6 required

Mark	req'd no.	dia	length	wt/ft	wt/piece	total wt	Remarks
FC0S	8	3/4	2.7	1.502	4.06	32.5#	Panel P.O. Sidewalk
FC2S	16	3/4	3.0	"	4.51	72.2	" 2 "
FC4S	16	3/4	3.8	"	5.71	91.4	" 4 "
FC6S	16	3/4	5.2	"	7.81	125.0	" 6 "
FC8S	16	3/4	7.2	"	10.81	173.0	" 8 "
FC10S	16	3/4	10.0	"	15.02	240.3	" 10 "
FC0T	8	3/4	2.3	"	3.45	27.6	" 0 Traff. Way
FC2T	16	3/4	2.6	"	3.91	62.6	" 2 "
FC4T	64	3/4	3.4	"	5.11	327.0	" 4 "
FC6T	64	3/4	4.8	"	7.21	461.4	" 6 "
FC8T	64	3/4	6.8	"	10.21	653.4	" 8 "
FC10T	64	3/4	9.5	"	14.27	913.3	" 10 "
FC4E	32	3/4	3.0	"	4.51	144.3	" 4 Elect. Ry.
FC6E	32	3/4	4.4	"	6.61	211.5	" 6 "
EC8E	32	3/4	6.4	"	9.61	307.5	" 8 "
FC10E	32	3/4	9.2	"	13.82	442.2	" 10 "
FCh	1192	1/4	3.0	0.167	0.50	596.0	Column hoops
BRA	44	1/2	3.7	0.668	2.47	108.7	Brackets
				Total =		4,984.9#	= 2.227 tons.

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HYOGO KEN

MADE BY K. Ohata DATE 4-5-24 FILE NO

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CALCULATIONS FOR

Mukō-o-Hashi over the Mukōgawa.

Lists of Reinforcements for Arch Span continued.

Arch Ring Reinforcements.

Arch Ring Reinforcements. Sheet No. 7.

End Span 2 required.								Intermediate Span 4 required.							
Mark	Reqd. no.	dia.	length	Wt. #	Wt. piece	total wt.	remarks.	Mark	Reqd. no.	dia.	length	Wt. #	Wt. piece	total wt.	remarks
R 1	64	5/8	32.5	2.02#	65.73#	4205.5#	Longi. bars.	R 1	64	5/8	32.5	2.02#	65.73#	4205.5#	Longi. bars.
R 2	128	5/8	18.0	"	36.39	4659.1	"	R 2	128	5/8	18.0	"	36.39	4659.1	"
R 3	28	5/8	24.0	"	48.66	1362.7	"	R 3	128	5/8	24.0	"	48.66	6229.7	"
R 4	32	1	16.5	2.670	44.05	1409.9	" pier side	R 4	64	1	16.5	2.670	44.05	2819.5	"
R 5	32	1	12.5	"	33.38	1068.2	" "	R 5	64	1	12.5	"	33.38	2136.3	"
R 6	32	1	24.8	"	66.22	2119.0	" "	R 6	64	1	24.3	"	64.89	4152.9	"
R 7	32	1	20.8	"	55.54	1777.3	" "	R 7	64	1	20.8	"	55.54	3554.6	"
R 8	32	1	21.5	"	57.41	1837.1	" about	R 12	74	5/8	21.5	1.043	22.42	1659.1	Cross bars Top
R 9	32	1	12.5	"	33.38	1068.2	" "	R 13	37	5/8	19.1	"	19.92	737.0	" "
R 10	32	1	29.3	"	79.57	2546.2	" "	R 14	74	5/8	17.2	"	17.94	1327.6	" Bott
R 11	32	1	22.3	"	59.52	1905.3	" "	R 15	74	5/8	18.0	"	18.77	1389.0	" "
R 12	74	5/8	21.5	1.043	22.42	1659.1	Cross bars Top	R 16	32	3/4	18.0	1.502	27.04	865.3	Column base Bott.
R 13	37	5/8	19.1	"	19.92	737.0	" "	R 17	32	3/4	17.2	"	25.83	826.6	" "
R 14	36	5/8	17.2	"	17.94	1423	" Bott	R 18	32	3/4	21.5	"	32.29	1033.3	" Top
R 15	36	5/8	18.0	"	18.77	1514.2	" "	R 19	16	3/4	19.1	"	28.69	459.0	" "
R 16	32	3/4	18.0	1.502	27.04	865.3	Column base Bott.	R 20	24	3/4	6.0	"	9.01	216.2	" Top & Bott.
R 17	32	3/4	17.2	"	25.83	826.6	" "	R 5	32	1/2	1.8	0.668	1.20	38.4	Stirrups
R 18	32	3/4	21.5	"	32.29	1033.3	" " Top	R 5-1	64	1/2	1.8	"	1.20	76.8	"
R 19	16	3/4	19.1	"	28.69	459.0	" " Top	R 5-2	64	1/2	1.8	"	1.20	76.8	"
R 20	24	3/4	6.0	"	9.01	216.2	" Top & Bott.	R 5-3	64	1/2	1.8	"	1.20	76.8	"
R 5-1	32	1/2	1.8	0.668	1.20	38.4	Stirrups	R 5-4	64	1/2	1.8	"	1.20	76.8	"
R 5-2	64	1/2	1.8	"	1.20	76.8	"	R 5-5	64	1/2	1.8	"	1.20	76.8	"
R 5-3	64	1/2	1.8	"	1.20	76.8	"	R 5-6	64	1/2	1.8	"	1.20	76.8	"
R 5-4	64	1/2	1.8	"	1.20	76.8	"	R 5-7	64	1/2	1.8	"	1.20	76.8	"
R 5-5	64	1/2	1.8	"	1.20	76.8	"	R 5-8	64	1/2	2.0	"	1.34	85.8	"
R 5-6	64	1/2	1.8	"	1.20	76.8	"	R 5-9	64	1/2	2.0	"	1.34	85.8	"
R 5-7	64	1/2	1.8	"	1.20	76.8	"	R 5-10	64	1/2	2.0	"	1.34	85.8	"
R 5-8	64	1/2	2.0	"	1.34	85.8	"	R 5-11	64	1/2	2.2	"	1.47	94.1	"
R 5-9	64	1/2	2.0	"	1.34	85.8	"	R 5-12	64	1/2	2.2	"	1.47	94.1	"
R 5-10	64	1/2	2.0	"	1.34	85.8	"	R 5-13	64	1/2	2.4	"	1.60	102.6	"
R 5-11	64	1/2	2.2	"	1.47	94.1	"	R 5-14	64	1/2	2.6	"	1.74	111.4	"
R 5-12	64	1/2	2.2	"	1.47	94.1	"	R 5-15	64	1/2	2.6	"	1.74	111.4	"
R 5-13	64	1/2	2.4	"	1.60	102.6	"	R 5-16	64	1/2	3.0	"	2.00	128.0	"
R 5-14	64	1/2	2.6	"	1.74	111.4	"	R 5-17	64	1/2	3.0	"	2.00	128.0	"
R 5-15	64	1/2	2.6	"	1.74	111.4	"	R 5-18	64	1/2	3.2	"	2.14	137.0	"
R 5-16	64	1/2	3.0	"	2.00	128.0	"	R 5-19	64	1/2	3.2	"	2.14	137.0	"
R 5-17	64	1/2	3.0	"	2.00	128.0	"								
R 5-18	64	1/2	3.2	"	2.14	137.0	"								
R 5-19	64	1/2	3.2	"	2.14	137.0	"								
R 5-20	32	1/2	6.0	"	4.01	128.3	"								
R 5-21	32	1/2	8.5	"	5.68	181.8	"								
R 5-22	32	1/2	11.0	"	7.35	235.2	"								
				Total =		20443 =	19.05 tons					Total =		38,3027# =	17.094 tons

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HYOGO KEN

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CHECKED BY T.Y. DATE 4-5-21 PAGE NO 157

CALCULATIONS FOR

Miki-o-Hashi over the Mukogawa

List of Reinforcements for Arch Span continued.

Handrail Reinforcements.

Handrail Reinforcements for Arch Span. Sheet No.10.

End Span 2 required								Intermediate Span 7 required.							
Mark	Radius	dia	Length	Wt/ft	Wt/piece	total Wt	Remarks	Mark	Radius	dia	Length	Wt/ft	Wt/piece	tot'l Wt	Remarks
H8	5	1/2	5.6	0668	3.74#	22.4#	2 Bott rails	H1	72	1/2	5.0	0668	3.34#	23.38#	24 Bott rails
H2	22	1/2	1.5	0167	0.25	5.5	Bd	H2	240	1/2	1.5	0167	0.25	5.0	Ba
H1	66	1/2	5.0	0668	3.34	220.4	22 Bott rails	H5	24	1/2	2.0	0668	1.34	32.2	12 Interm.
H2	220	1/2	1.5	0167	0.25	5.50	Ba	H7	24	1/2	2.5	0668	1.67	40.1	posts.
H5	24	1/2	2.0	0668	1.34	32.2									
H6	24	1/2	2.0	0167	0.33	7.7	12 Intermediate								
H7	24	1/2	2.5	0668	1.67	40.1	posts.								
				Total	=	472.5#	= 0.23 tons					Total	=	472.7#	= 0.23 tons

Pier Reinforcements

Reinforcements for Pier 5 required

Sheet No.10.

Mark	Radius	dia	length	Wt/ft	Wt/piece	tot'l Wt	Remarks
PAH1	72	7/8	18.0	2.044	36.79	183.96	Hor. bars of
PAH2	72	7/8	18.0	"	36.79	183.96	Pier Shaft
PAH3	4	5/8	12.3	1.043	12.83	64.15	
PAH4	4	5/8	13.2		13.77	68.85	
PAH5	4	5/8	14.0		14.60	73.0	
PAH6	4	5/8	14.1		15.54	77.7	
PAH7	4	5/8	15.8		16.48	82.4	
PAH8	4	5/8	16.6		17.31	86.55	
PAH9	4	5/8	17.5		18.25	91.25	
PAH10	4	5/8	18.4		19.19	95.95	
PAH11	15	1	18.0	2.670	48.06	240.3	Cross Wall
PAH12	8	1	5.0		13.35	106.8	Hor. bars of
PAH13	28	1/2	5.5	0.668	3.67	102.8	end column.
PAH14	28	1/2	10.0		6.68	187.0	Ver. bars of Shaft
PAV1	124	7/8	12.0	2.044	24.53	3041.7	"
PAV2	124	7/8	18.0	"	36.79	4560	
PAV3	26	7/8	12.0	"	24.53	637.8	
PAV4	26	7/8	14.0	"	28.62	744.1	
PAV5	40	5/8	4.5	1.043	4.69	187.6	
FC12	112	3/4	13.5	1.502	20.28	2271.4	Ver. bar of col.
FC4	550	1/4	3.0	0.167	0.50	275.0	Hoops of col.
C5	112	1/2	2.5	0.668	1.67	187.0	Column base
PAI	104	5/8	3.5	1.043	3.65	379.5	Cross Wall
PAS	50	3/8	5.0	0.376	1.88	94.0	" Stirrup
PAB	12	5/8	8.0	1.043	8.32	100.1	Bracket
				Total	=	17891.6#	= 8.95 tons

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

HYOGO KEN

MADE BY K. Inaba DATE 14-5-15 FILE NO

CHECKED BY T.N DATE 14-5-21 PAGE NO 120'

CALCULATIONS FOR

Mukō-O-Hashi over the Mukōgawa.

Reinforcements for Arch Span continued.

Reinforcements for Center Pole base.

Reinforcements for Center Pole base. Sheet No. 21.

Center Pole on Pier 5 required.

Mark	reqd no	dia	length	wt/ft.	wt/piece	total wt.	remarks
CTP1	20	½"	6.3	0.668#	4.21#	84.2#	Ver. bar
CTP2	8	¼	12.0	0.167	2.00	16.0	outer hoop
CTP3	7	¼	12.0	"	2.00	14.0	Inner "
CTP4	3	½	8.0	0.668	5.34	16.0	Bent up bar
CTP5	3	½	9.0	"	6.01	18.0	" " "
CTP6	15	½	5.0	"	3.34	50.1	Longi. bar
CTP7	11	½	6.5	"	4.34	47.7	Cross "
				Total =		246.0#	= 0.110 ton.

Center Pole on Abutment 2 required.

Mark	reqd no	dia	length	wt/ft.	wt/piece	total wt.	remarks
CTP1	20	½	6.3	0.668#	4.21#	84.2#	Ver. bar.
CTP2	8	¼	12.0	0.167	2.00	16.0	Outer hoop.
CTP3	7	¼	12.0	"	2.00	14.0	Inner " .
CTP5	6	½	9.0	0.668	6.01	36.1	Bent up bar.
CTP7	30	½	6.5	"	4.34	130.2	Hor. bar.
				Total =		280.5#	= 0.125 ton.

増田淳氏関係資料
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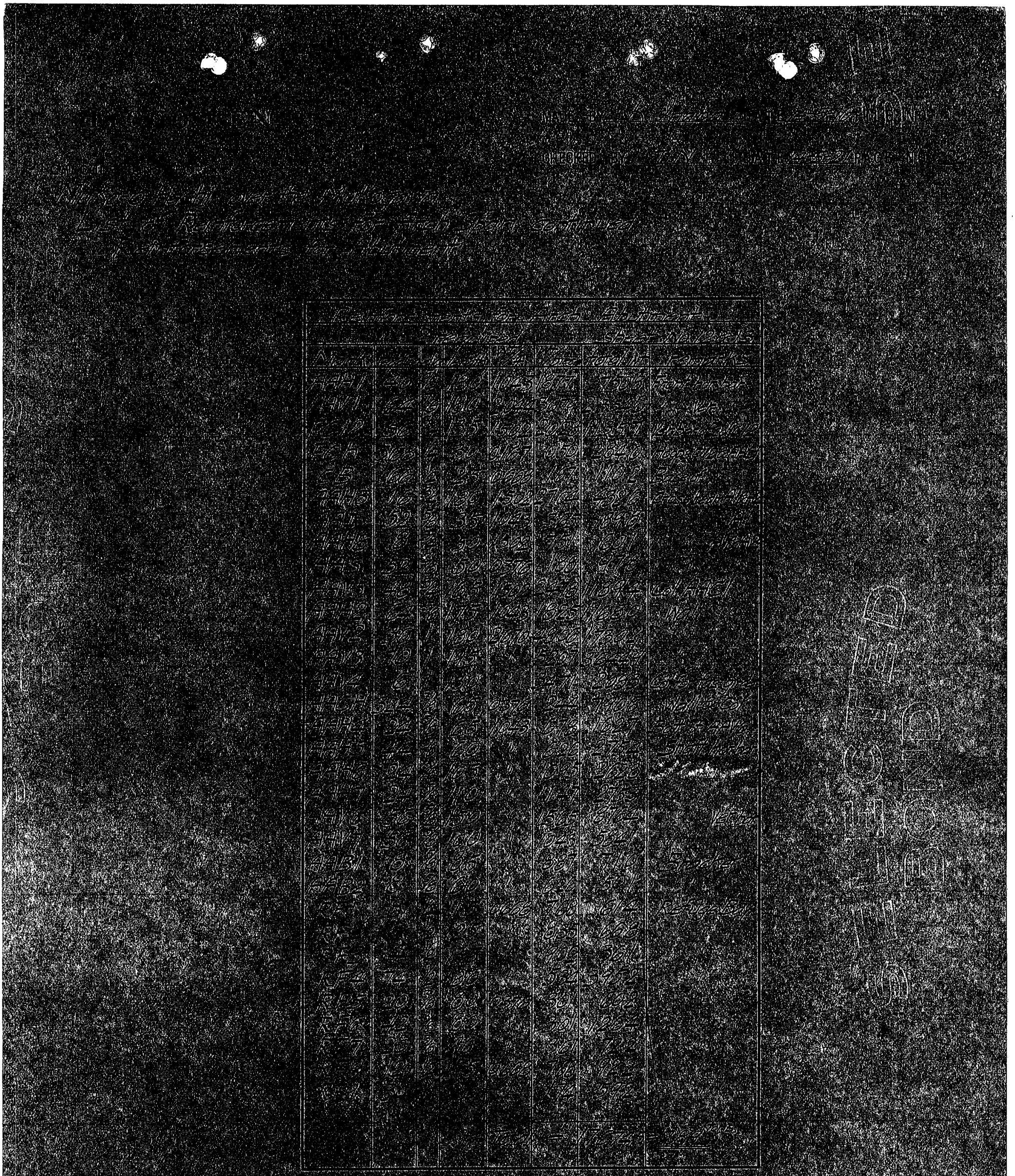

HYOGO KEN

MADE BY K. Inaba. DATE 14-5-16 FILE NO

CHECKED BY T.N. DATE 14-5-21 PAGE NO 121

CALCULATIONS FOR

Muko-o-Hashi over the Mukogawa.
List of Reinforcements for Girder Span.

Slab Reinforcements.

Slab Reinforcements for 3 Continous Spans.
3 required Sheet No. 6

Mark	reqd no.	dia.	length	wt./ft.	wt./piece	total wt.	remarks
FS1	81	1/2"	21.0	0.668#	14.03#	1,136.4#	Cross bar.
FS2	162	1/2	25.5	"	17.03	2,758.9	"
FS3	81	1/2	24.9	"	16.63	1,347.0	"
FS4	162	1/2	18.9	"	12.63	2,046.1	"
FS5	324	1/2	22.7	"	15.16	4,911.8	"
FS6	162	1/2	22.1	"	14.76	2,391.1	"
FS7	162	3/8	10.5	0.376	3.45	639.9	"
FS8	324	3/8	9.1	"	3.42	1,108.1	"
FS9	162	3/8	7.3	"	2.74	443.9	"
FS22	264	1/2	20.0	0.668	13.36	3,527.0	longi. bar.
FS23	66	1/2	9.0	"	6.01	396.7	"
FS24	72	3/8	19.5	0.376	7.33	527.8	"
FS25	18	3/8	8.5	"	3.20	57.6	"
				Total	=	21,242.3#	= 9.505 tons.

Reinforcements for base of Light Pedestal.
1 required Sheet No. 6.

Mark	reqd no.	dia.	length	wt./ft.	wt./piece	total wt.	remarks
RS26	8	1/2"	5.0'	0.668#	3.34#	26.7#	
RS27	2	1/2	7.6	"	5.08	10.2	
RS28	4	1/2	2.8	"	1.87	7.5	
				Total	=	44.4#	= 0.020 ton

Handrail Reinforcements for 9 Girder Spans.
1 required Sheet No. 13 & 14

Mark	reqd no.	dia.	length	wt./ft.	wt./piece	total wt.	remarks
H3	12	1/2"	5.7	0.668#	3.81#	45.7#	4 Bott. rail Bb
H2	48	1/4	1.5	0.167	0.25	12.0	" " "
H4	180	1/2	6.2	0.668	4.14	745.2	60 Bott. rail Bc
H2	780	1/4	1.5	0.167	0.25	195.0	" " "
H9	24	1/2	5.9	0.668	3.94	94.6	8 Bott. rail Be
H2	96	1/4	1.5	0.167	0.25	24.0	" " "
				Total	=	1,116.5#	= 0.498 ton

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

HYOGO KEN

MADE BY K. Inaba DATE 14-5-16 FILE NO.

CHECKED BY TN DATE 14-5-21 PAGE NO 122

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.
Reinforcements for Girder Spans continued.

Beam Reinforcements.

Beam Reinforcements for 3 continous Spans.
3 required. Sheet No.8.

Mark	reqd no	dia	length	wt/ft	wt/piece	total wt	remarks
Gf 1	8	1⅛"	30.5'	2.044#	62.34#	498.7#	2 fascia girders
Gf 2	4	¾	30.5	1.502	45.81	183.2	"
Gf 3	8	⅞	8.5	2.044	17.37	139.0	"
Gf 4	8	⅞	16.1	"	32.91	263.3	"
Gf 5	8	⅞	25.4	"	51.92	415.4	"
Gf 6	4	¾	19.5	1.502	29.29	117.2	"
Gf 7	8	⅞	19.5	2.044	39.86	318.9	"
Gf 8	16	⅞	19.3	"	39.45	631.2	"
Gf 9	4	¾	21.9	1.502	32.89	131.6	"
Gf 10	8	¾	8.0	"	12.02	96.2	"
GS 1	24	½	7.5	0.668	5.01	120.2	"
GS 2	24	½	9.1	"	6.08	145.9	"
GS 3	24	½	11.2	"	7.48	179.5	"
GS 4	48	½	12.6	"	8.42	404.2	"
FgB 1	36	⅝	2.5	1.043	2.61	94.0	"
FgB 2	36	⅝	5.0	"	5.22	187.9	"
Gr 1	16	1	30.0	2.670	80.10	1,281.6	4 girders under rail way
Gr 2	12	1	32.0	"	85.44	1,025.3	"
Gr 3	16	1	26.2	"	69.95	1,119.2	"
Gr 4	16	1	14.3	"	38.18	610.9	"
Gr 5	8	1	30.9	"	82.50	660.0	"
Gr 6	8	1	10.7	"	28.57	228.6	"
Gr 7	16	1	33.6	"	89.71	1,435.4	"
Gr 8	8	1	28.5	"	76.10	608.8	"
Gr 9	16	1	21.1	"	56.34	901.4	"
Gr 10	16	1	22.8	"	60.88	974.1	"
Gr 11	8	1	14.1	"	37.65	301.2	"
Gr 12	8	1	27.3	"	72.89	583.1	"
Gr 13	32	1	20.0	"	53.40	1,708.8	"
Gr 14	8	1	22.0	"	58.74	469.9	"
Gr 15	16	¾	6.5	1.502	9.76	156.1	"
GS 5	404	½	6.0	0.668	4.01	1,620.0	"
dowel	48	¾	3.0	1.502	4.51	216.5	"
Gt. 1	32	1	30.0	2.670	80.10	2,563.2	8 girders under traffic way.
Gt 2	16	1	32.0	"	85.44	1,367.0	"
Gt 3	32	1	5.2	"	13.88	444.2	"
Gt 4	32	1	9.3	"	24.83	794.6	"
Gt 5	32	1	20.2	"	53.93	1,725.8	"
Gt 6	16	1	18.4	"	49.13	786.1	"
Gt 7	32	1	23.3	"	62.21	1,990.7	"
Gt 8	32	1	25.1	"	67.02	2,144.6	"
Gt 9	32	1	18.3	"	48.86	1,563.5	"
Gt 10	64	1	20.0	"	53.40	3,417.6	"
Gt 11	16	1	22.0	"	58.74	939.8	"
Gt 12	32	¾	6.5	1.502	9.76	312.3	"
GS 6	520	½	5.8	0.668	3.87	2,012.4	"
GS 7	96	½	6.6		4.41	423.4	"
dowel	96	¾	3.0	1.502	4.51	433.0	"
					Total =	38,745.5#	= 17.296 tons.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

HYOGO KEN

MADE BY K. Inaba DATE 14-5-18 FILE NO.

CHECKED BY T.N. DATE 14-5-21 PAGE NO 123

CALCULATIONS FOR

Muko-o-Hashi over the Mukogawa.

Reinforcements for Girder Span continued.

Reinforcements for Pier Type A (Pier Nos. GP3 to GP7 inclusive)

Reinforcements for Pier Type A.

For GP4,5,6 & 7 4 required Sheet No.15

Mark	reqd no	dia	length	wt/ft	wt/piece	total wt	remarks
PgV1	60	1"	16.0	2.670	42.72#	2563.2#	Ver. reinf. column
PgV2	60	1	21.0	"	56.07	3364.2	" "
PgV3	40	1	12.5	"	33.38	1335.2	" "
PgV4	20	1	15.5	"	41.39	827.8	" "
PgV5	60	1	8.5	"	22.70	1362.0	" "
PgV6	64	1	8.5	"	22.70	1452.8	Inclined reinf. "
PgV11	64	7/8	9.5	2.044	19.42	1242.9	Ver. reinf. Wall
Pgh1	8	7/8	10.0	"	20.44	163.5	Hor. bar Beam top
Pgh2	4	7/8	23.0	"	47.01	188.0	" " "
Pgh3	4	7/8	6.5	"	13.29	53.2	" " "
Pgh4	2	7/8	16.0	"	32.70	65.4	Neg. reinf. Beam
Pgh5	4	7/8	16.5	"	33.73	134.9	" " "
Pgh6	4	7/8	9.0	"	18.40	73.6	" " "
Pgh7	40	7/8	18.0	"	36.79	1471.6	Hor. bar Beam Bott.
Pgh8	6	7/8	14.0	"	28.62	171.7	Haunch bott.
Pgh8'	4	7/8	8.5	"	17.37	69.5	" "
Pgh9	24	7/8	18.5	"	37.81	907.4	Base top.
Pgh10	12	7/8	18.0	"	36.79	441.5	" bott.
Pgh11	18	7/8	16.0	"	32.70	588.6	" "
Pgh12	20	7/8	9.0	"	18.40	368.0	" "
Pgh13	660	3/8	6.5	0.376	2.44	1610.4	Hoop
Pgh14	60	3/8	4.2	"	1.58	94.8	"
Pgh15	12	7/8	33.0	2.044	67.45	809.4	Wall top
PgS1	24	3/8	14.0	0.376	5.26	126.2	Stirrup
PgS2	28	3/8	12.0	"	4.51	126.3	"
PgS3	48	3/8	9.5	"	3.57	171.4	"
PgS4	28	3/8	11.0	"	4.14	115.9	"
PgS7	36	1/2	7.5	0.668	5.01	180.4	"
PgB1	4	5/8	2.5	1.043	2.61	10.4	Bracket
PgB2	4	5/8	5.0	"	5.22	20.9	"
				Total =		20,111.1#	= 8.978 tons.

For Pier GP3 1 required Sheet No.15,16&6.

Mark	reqd no	dia	length	wt/ft	wt/piece	total wt	remarks
PgV1	60	1"	16.0	2.670	42.72#	2563.2#	Ver. reinf. column
PgV2	60	1	21.0	"	56.07	3364.2	" "
PgV3	40	1	12.5	"	33.38	1335.2	" "
PgV4	20	1	15.5	"	41.39	827.8	" "
PgV5	60	1	8.5	"	22.70	1362.0	" "
PgV6	64	1	8.5	"	22.70	1452.8	Inclined reinf.
PgV11	64	7/8	9.5	2.044	19.42	1242.9	Ver. reinf. Wall
Pgh1	8	7/8	10.0	"	20.44	163.5	Hor. bar Beam top
Pgh2	4	7/8	23.0	"	47.01	188.0	" " "
Pgh3	4	7/8	6.5	"	13.29	53.2	" " "
Pgh4	2	7/8	16.0	"	32.70	65.4	Neg. reinf. Beam.
Pgh5	4	7/8	16.5	"	33.73	134.9	" " "
Pgh6	4	7/8	9.0	"	18.40	73.6	" " "
Pgh7	40	7/8	18.0	"	36.79	1471.6	Hor. bar Beam bott.
Pgh8	6	7/8	14.0	"	28.62	171.7	Haunch Bott.
Pgh8'	4	7/8	8.5	"	17.37	69.5	" "
Pgh9	24	7/8	18.5	"	37.81	907.4	Base top.
Pgh10	12	7/8	18.0	"	36.79	441.5	" bott.
Pgh11	18	7/8	16.0	"	32.70	588.6	" "
Pgh12	20	7/8	9.0	"	18.40	368.0	" "
Pgh13	660	3/8	6.5	0.376	2.44	1610.4	Hoop column
Pgh14	60	3/8	4.2	"	1.58	94.8	" "
Pgh15	12	7/8	33.0	2.044	67.45	809.4	Wall top
PgS1	24	3/8	14.0	0.376	5.26	126.2	U-stirrup
PgS2	28	3/8	12.0	"	4.51	126.3	"
PgS3	48	3/8	9.5	"	3.57	171.4	"
PgS4	28	3/8	11.0	"	4.14	115.9	"
PgS5	44	1/2	6.0	0.668	4.01	176.4	Inclined Stirrup
PgS6	44	1/2	5.5	"	3.67	161.5	Square Stirrup
Pgh16	10	1/2	8.0	"	5.34	53.4	End Col. top
PgV12	6	3/4	6.0	1.502	9.01	54.1	" "
PgB1	4	5/8	2.5	1.043	2.61	10.4	Bracket
PgB2	4	5/8	5.0	"	5.22	20.9	"
				Total =		20,376.1#	= 9.096 tons.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

HYOGO KEN

MADE BY K. Yazaki DATE 4-5-22 FILE NO

CHECKED BY T.N. DATE 4-5-22 PAGE NO 124

CALCULATIONS FOR

Mukō-o-Hashi over the Mukōgawa.

Reinforcements for Girder Span continued.

Reinforcements for Pier Type 5. (Pier #GP1 & GP2)

Reinforcements for Pier Type B (GP1 & 2)

2 required Sheet No. 16.

Mark	req'd no.	dia.	length	Wt./ft.	wt./piece	total wt.	remarks
PgV5	60	1"	8.5	2.670	22.70#	1362.0#	Ver. reinf. column
PgV7	60	1	16.0	"	42.72	2563.2	" "
PgV8	40	1	20.0	"	53.40	2136.0	" "
PgV9	20	1	23.0	"	61.41	1228.2	" "
PgV10	64	1	7.0	"	18.69	1196.2	Inclined reinf."
PgH1	8	7/8	10.0	2.044	20.44	163.5	Hor. bar Beam top
PgH2	4	7/8	23.0	"	47.01	188.0	" "
PgH3	4	7/8	6.5	"	13.29	53.2	" "
PgH4	2	7/8	16.0	"	32.70	65.4	Neg. bar Beam
PgH5	4	7/8	16.5	"	33.73	134.9	" "
PgH6	4	7/8	9.0	"	18.40	73.6	" "
PgH7	40	7/8	18.0	"	36.74	1471.6	Hor. bar Beam bott.
PgH8	6	7/8	14.0	"	28.62	171.7	Haunch bott.
PgH8'	4	7/8	8.5	"	17.37	69.5	" "
PgH9	24	7/8	18.5	"	37.81	907.4	Base top.
PgH10	12	7/8	18.0	"	36.79	441.5	" bott.
PgH11	18	7/8	16.0	"	32.70	588.6	" "
PgH12	20	7/8	9.0	"	18.40	368.0	" "
PgH13	555	3/8	6.5	0.376	2.44	1354.2	Hoop Column
PgH14	60	3/8	4.2	"	1.58	94.8	" "
PgS1	24	3/8	14.0	"	5.26	126.2	Stirrup
PgS2	28	3/8	12.0	"	4.51	126.3	"
PgS3	48	3/8	9.5	"	3.57	171.4	"
PgS4	28	3/8	11.0	"	4.14	115.9	"
PgS7	36	1/2	7.5	0.668	5.01	180.4	"
PgB1	4	5/8	2.5	1.043	2.61	10.4	Bracket
PgB2	4	5/8	5.0	"	5.22	20.9	"
				Total	=	15383.0	= 6.867 tons.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

HYOGO KEN

MADE BY K. Inaba DATE 14-5-18 FILE NO

CHECKED BY T.N DATE 14-5-22 PAGE NO 125.

CALCULATIONS FOR

Mukō-o-Hashi over the Mukōgawa.

Reinforcements for Girder Span continued.

Reinforcements for Abutments (GA1 & GA2).

Reinforcements for Abutments (GA1 & GA2) Sheet No. 17 & 20.

East Abutment (GA1) 1 required

Mark	reqd. no.	dia.	length	Wt./ft.	Wt./piece	total wt.	remarks
GAV1	80	7/8	13.3'	2.044#	27.19#	2,175.2#	Ver. bar shaft.
GAV2	82	7/8	17.3	"	35.36	2,899.5	" "
GAV2'	82	7/8	11.3	"	23.10	1,894.2	" "
GAV3	64	1/2	8.5	0.668	5.68	363.5	" "
GAV4	80	3/4	13.0	1.502	19.53	1,562.4	" "
GAV5	64	3/4	21.0	"	31.54	2018.6	" "
GAV6	18	3/4	17.0	"	25.53	459.5	" "
GAV6'	18	3/4	11.0	"	16.52	297.4	" "
GAV7	82	3/4	12.0	"	18.02	1477.6	Inclined bar "
GAV8	8	7/8	11.5	2.044	23.51	188.1	Supp. of Fas. Gir.
GAh1	48	5/8	18.4	1.043	19.19	921.1	Hor. bar shaft.
GAh2	26	5/8	19.4	"	20.23	526.0	" "
GAh3	26	5/8	10.0	"	10.43	271.2	" "
GAh4	6	1/2	18.0	0.668	12.02	72.1	" "
GAh5	6	1/2	19.0	"	12.69	76.1	" "
GAh6	112	7/8	14.3	2.044	29.23	3,273.8	" base bott.
GAh7	22	5/8	21.0	1.043	21.90	481.8	" Shaft.
GAh8	26	5/8	9.5	"	9.91	257.7	" "
GAV9	30	3/4	13.0	1.502	19.53	585.9	Ver. bar Wing
GAV10	34	3/4	17.0	"	25.53	868.0	" "
GAV10'	34	3/4	11.0	"	16.52	561.7	" "
GAV11	6	1/2	6.0	0.668	4.01	24.1	" "
GAh9	18	1/2	10.0	"	6.68	120.2	Hor. bar Wing
GAh10	8	1/2	13.0	"	8.68	69.5	" "
GAh11	8	1/2	5.0	"	3.34	26.7	" "
GAh12	18	3/4	9.0	1.502	13.52	243.4	" base bott.
GAh13	26	1/2	7.0	0.668	4.68	121.7	" Wing
				Total	=	21,837.0#	= 9.749 tons.

West Abutment (GA2) 1 required

Mark	reqd. no.	dia.	length	Wt./ft.	Wt./piece	total wt.	remarks
GAV1	80	7/8	13.3'	2.044#	27.19#	2,175.2#	Ver. bar shaft.
GAV2	82	7/8	17.3	"	35.36	2,899.5	" "
GAV2'	82	7/8	11.3	"	23.10	1,894.2	" "
GAV3	64	1/2	8.5	0.668	5.68	363.5	" "
GAV4	80	3/4	13.0	1.502	19.53	1,562.4	" "
GAV5	64	3/4	21.0	"	31.54	2,018.6	" "
GAV6	18	3/4	17.0	"	25.53	459.5	" "
GAV6'	18	3/4	11.0	"	16.52	297.4	" "
GAV7	82	3/4	12.0	"	18.02	1,477.6	Inclined bar "
GAV8	8	7/8	11.5	2.044	23.51	188.1	Supp. of Fas. Gir.
GAh1	48	5/8	18.4	1.043	19.19	921.1	Hor. bar shaft.
GAh2	26	5/8	19.4	"	20.23	526.0	" "
GAh3	26	5/8	10.0	"	10.43	271.2	" "
GAh4	6	1/2	18.0	0.668	12.02	72.1	" "
GAh5	6	1/2	19.0	"	12.69	76.1	" "
GAh6	112	7/8	14.3	2.044	29.23	3,273.8	" base bott.
GAh7	22	5/8	21.0	1.043	21.90	481.8	" Shaft.
GAh8	26	5/8	9.5	"	9.91	257.7	" "
GAV9	36	3/4	13.0	1.502	19.53	703.1	Ver. bar Wing.
GAV10	40	3/4	17.0	"	25.53	1,021.3	" "
GAV10'	40	3/4	11.0	"	16.52	660.8	" "
GAV11	6	1/2	6.0	0.668	4.01	24.1	" "
GAh9	44	1/2	10.0	"	6.68	302.7	Hor. bar wing
GAh10	8	1/2	13.0	"	8.68	69.5	" "
GAh11	8	1/2	5.0	"	3.34	26.7	" "
GAh12	22	3/4	9.0	1.502	13.52	297.4	" base bott.
				Total	=	22,321.4#	= 9.965 tons.

増田淳氏関係資料
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HYOGO KEN

MADE BY N. Yabe DATE 4-5-1 FILE NO

CHECKED BY T.N DATE 4-5-1 PAGE NO 127

CALCULATIONS FOR

Mukjo-o-Hashi over the Mukjogawa.

Reinforcements for Girder Span continued.

Center Pole base.

Reinforcements for Center Pole base.

1 required Sheet No 21

Mark	reinforcing	dia	length	Wt #	pieces	total Wt	remarks
CTP3	10	½"	11.5'	0668"	4.3"	43.0"	4-5 bar
CTP1	10	½	7.7	"	5.14	5.14	
CTP10	20	½	5.7	"	4.3	34.0	Ver. bar
CTP11	9	¼	12.0	0167	2.00	18.0	Outer hoop
CTP12	7	¼	12.0		2.00	14.0	Inner hoop
				Total =		216#	= 0.09 Ton

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HYOGO KEN

MADE BY K. Inaba DATE 14-5-16 FILE NO

CHECKED BY T.N. DATE 14-5-22 PAGE NO 127

CALCULATIONS FOR

MuKo-O-Hashi over the MuKogawa.
List of Reinforcements Continued.
Summary of Reinforcements.

Reinforcements for Arch Span.				tons	tons.
Slab.	End Span.	2@	7.619	=15.238	
	Interm. Span.	4@	6.827	=27.308	
					42.546 tons.
Beam.	End Span	2@	5.287	=10.574	
	Interm. Span	4@	4.718	=18.872	
					29.446
Column.	All Spans.	6@	2.227	=13.362	
					13.362
Arch Ring.	End Span.	2@	18.054	=36.108	
	Interm. Span.	4@	17.099	=68.396	
					104.504
Center Pole base on Abut'		2@	0.125	= 0.250	
	Pier.	5@	0.110	= 0.550	
					0.800
Handrail.	End Span.	2@	0.210	= 0.420	
	Interm. Span.	4@	0.204	= 0.836	
					1.256 tons.
Total Steel req'd. for supper structure of arch spans =					191.914 tons.
Pier.		5@	8.876	=	44.380
Abutment.		2@	7.890	=	15.780 tons.
Total steel req'd. for sub-structure of arch spans =					60.160
Total steel req'd. for arch spans					252.074 tons.
Reinforcements for Girder Span.					
Slab.		3@	9.505	=28.515	28.515
Base of Light Pedestal.		1@	0.020	= 0.020	0.020
Handrail.		1@	0.498	= 0.498	0.498
Center Pole base.		1@	0.097	= 0.097	0.097
Beams.		3@	17.296	=51.888	51.888
Pier Type A(GP4.5&6.7).		4@	8.978	=35.912	
" " (GP3.).		1@	9.096	= 9.096	
" " B(GP1&GP2).		2@	6.867	=13.734	
					58.742
Abutment.	(East)	1@	9.749	= 9.749	
"	(West)	1@	9.965	= 9.965	
				19.714	
Total steel req'd for girder spans.					159.474 tons.
Total steel required for the whole bridge. =					411.548 tons.

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HYOGO KEN

MADE BY K. Inaba DATE 4-5-21 FILE NO

CHECKED BY T.N. DATE 4-5-21 PAGE NO 128

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

Concrete.

Concrete for Arch Span.

Concrete for Slab.

Concrete for Slab — Sheet No. 5.6.13.8.14.

End Span — 2 required

Name	section	length	Vol.	No.	total Vol	Remarks
Slab	17.9x0.52	78.63	731.9	1	731.9	Elect. Sy. Slab
"	16.41x.52	78.63	671.0	2	1,342.0	Traff. Way "
Chamfer	.63x1.0÷2	78.63	24.8	2	49.6	"
Slab	.73x.52	78.63	29.8	2	59.6	"
"	.5 x.4	78.63	15.7	2	31.4	"
"	1.99x.63	78.63	98.6	2	197.2	" "
"	5.47x.39	78.63	167.7	2	335.4	Side walk "
Chamfer	.2x.2÷2	78.63	1.6	2	3.2	" "
Coping	1.38x.05	69.75	4.8	2	9.6	" "
"	.39x.5	69.75	13.6	2	27.2	" "
"	.35x1.0	69.75	24.4	2	48.8	" "
"	.1x.9	69.75	6.3	2	12.6	" "
"	.65x.25	69.75	11.3	2	22.6	" "
"	.1x.1÷2	69.75	0.3	2	0.6	" "
Slab	.39x1.5	2.50	1.5	2	3.0	" "
Ref. bay	3.0x.39	11.50	13.5	2	27.0	Refuge bay Slab
"	3.0x.58	8.70	15.1	2	30.2	" "
"	.7x.58	8.70	3.5	2	7.0	" "
Coping	2.1x.4	5.50	4.6	4	18.4	" Coping
"	1.45x.1	5.35	1.0	4	4.0	" "
"	1.7x.25	5.10	2.2	4	8.8	" "
"	1.55x.35	5.00	2.7	4	10.8	" "
"	1.5x.4	10.80	6.5	2	13.0	" "
"	1.35x.1	10.80	1.5	2	3.0	" "
"	1.1x.25	10.50	2.9	2	5.8	" "
"	.95x.35	10.30	3.4	2	6.8	" "
			Total	=	3,009.5 ㎥	= 13.933 tsubo

Intermediate Span — 4 required

Name	Section	Length	Vol.	No.	Total Vol	Remarks
Slab	17.9x.52	70.42	6.555	1	6.555	Elect. Sy. Slab
"	16.41x.52	"	600.4	2	1201.8	Traff. Way "
Chamfer	.63x1.0÷2	"	22.2	2	44.4	" "
Slab	.73x.52	"	26.7	2	53.4	" "
"	.5x.4	"	14.1	2	28.2	" "
"	1.99x.63	"	88.3	2	176.6	" "
"	5.47x.39	"	150.2	2	300.4	Side walk "
Chamfer	.2x.2÷2	"	1.4	2	2.8	" "
Coping	1.38x.05	73.2	.51	2	10.2	"
"	.39x.5	"	14.3	2	28.6	"
"	.35x1.0	"	25.7	2	51.4	"
"	.1x.9	"	6.6	2	13.2	"
"	.65x.25	"	11.9	2	23.8	"
"	.1x.1÷2	"	3.7	2	7.4	"
Slab	.34x1.5	5.00	2.9	2	5.8	On pier
			Total	=	2603.5 ㎥	= 12.053 tsubo

Concrete for Hand rail — Sheet No. 19.

End Span — 2 required.

Name	Section	Length	Vol.	No.	Total Vol	Remarks
Bott. rail	.4x.4	4.70	.75	22	16.5	22 Bott. rail Ba
"	.6x.3	"	.85	22	18.7	" " "
"	.3x.3	.40	.04	44	1.8	" " "
"	.3x.3	.60	.05	22	1.1	" " "
"	.4x.4	5.35	.86	2	1.7	2 " Bd
"	.6x.3	"	.96	2	1.9	" " "
"	.3x.3	.40	.04	4	.2	" " "
"	.3x.3	.60	.05	2	.1	" " "
Int. post	.6x.7	2.50	1.05	12	12.6	12 Int. post
			Total	=	54.6 ㎥	= 0.253 tsubo

Intermediate Span — 4 required

Name	Section	Length	Vol.	No.	Total Vol	Remarks
Bott. ra.	.4x.4	4.7	.75	24	18.0	24 Bott. rail Ba
"	.6x.3	4.7	.85	24	20.4	" " "
"	.3x.3	.4	.04	48	1.9	" " "
"	.3x.3	.6	.05	24	1.2	" " "
post	.6x.7	2.5	1.05	12	12.6	12 Int. post
			Total	=	54.1 ㎥	= 0.250 tsubo

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HYOGO KEN

MADE BY K. Inaba DATE 12-5-22 FILE NO

CALCULATIONS FOR

CHECKED BY T. Iwa DATE 12-5-22 PAGE NO 129

Muko-O-Hashi over the Mukogawa.

Concrete for Arch Span continued

Concrete for Crown Fill

Sheet No 4 Set

6 required

Name	Section	Length	Vol	No	Total Vol	Remarks
Fill	28×16.6	12.8	32.52	2	65.0	Under Elect Rly
"	5.4×10.9	"	7.08	2	14.2	Footway
"	7.4×16.26	"	14.439	2	28.8.8	"
"	7.5×10.0	"	8.76	2	17.5	"
			Total	=	3.855	= 1.785 tsubo

Concrete for Beams Sheet No. 7.

End Span 2 required

Name	Section	Length	Vol	No	Total Vol	Remarks
Stem	.83×.9	66.75	49.86	4	199.2	Beam E & F
"	.83×.9	66.75	49.86	6	299.2	" B C & D
"	.83×1.11	78.75	72.55	2	145.1	" A
"	.7×1.53	65.00	69.62	2	139.2	Fascia girder
Chamfer	1×1÷2	66.75	033	22	7.3	
"	1×1÷2	78.75	039	2	0.8	
"	1×1÷2	65.00	033	2	0.7	
"	.3×.3÷2	78.75	.356	2	7.1	
Crossbeam	.9×.83	1.230	.423	2	1.85	
"	1.37×.83	15.66	1.703	4	7.13	
"	.75×.83	3.00	1.87	4	7.5	
Chamfer	1×1÷2	8.740	044	1	0.4	
			Total	=	8.965	= 4.150 tsubo

Intermediate Span 4 required

Name	Section	Length	Vol	No	Total Vol	Remarks
Stem	.83×.9	58.50	43.80	4	175.2	Beam E & F
"	.83×.9	58.50	43.80	6	262.8	" B C & D
"	.83×1.11	70.50	64.95	2	129.9	" A
"	.7×1.53	70.50	75.51	2	151.0	Fascia girder
Chamfer	1×1÷2	58.50	029	22	6.4	
"	1×1÷2	70.50	035	4	1.4	
"	.3×.3÷2	7050	317	2	6.3	
Crossbeam	.9×.83	16.36	1.22	2	2.44	
"	1.37×.83	13.68	15.56	4	62.2	
"	.75×.83	3.06	1.87	4	7.5	
Chamfer	1×1÷2	8.740	044	1	0.4	
			Total	=	827.5	= 3.831 tsubo

Concrete for Columns Sheet No 7

6 required

Name	Section	Length	Vol	No	Total Vol	Remarks
Panel P.O	.17×1.0	1.00	0.17	3	0.5	
" 2	.17×1.0	1.20	0.20	3	0.6	
" 4	.17×1.0	1.50	0.26	14	3.6	
" 6	.17×1.0	.90	0.15	14	2.1	Col. under beam
" 6	1.0×1.0	1.50	1.50	14	21.0	Col. base
" 6	1.3×1.3	.40	0.68	14	9.5	
" 8	.17×1.0	.90	0.15	14	2.1	
" 8	1.0×1.0	3.06	3.06	14	42.8	Col. under beam
" 8	1.3×1.3	1.50	0.85	14	11.9	Col. base
" 10	.17×1.0	.90	0.15	14	2.1	
" 10	1.0×1.0	5.20	5.20	14	72.8	Col. under beam
" 10	1.3×1.3	0.60	1.01	14	14.1	Col. base
Bracket	.7×.5	1.70	0.60	22	13.2	
			Total	=	196.3	= 0.909 tsubo

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HYOGO KEN

MADE BY K. Inaba DATE 14-5-25 FILE NO

CHECKED BY T.N. DATE 14-5-26 PAGE NO 131

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

Concrete for Arch Span Continued

Arch Ring.

Concrete for Arch Ring of Intermediate Span. Sheet No. 4 & 9.

Arch Ring under Electric Ry. 4 required.

Name	Section	Length	Vol.	No.	Total Vol.	Remarks
Panel Pt. 0-1	1.175×1.935	2.75	62.52 ft³	2	125.0 ft³	
1-2	1.19×1.935	2.76	63.55	2	127.1	
2-3	1.21×1.935	2.77	64.86	2	129.7	
3-4	1.235×1.935	2.795	66.79	2	133.6	
4-5	1.27×1.935	2.82	69.30	2	138.6	
5-6	1.33×1.935	2.85	73.35	2	146.7	
6-7	1.425×1.935	2.885	79.55	2	159.1	
7-8	1.55×1.935	2.935	88.03	2	176.1	
8-9	1.7×1.935	3.00	98.69	2	197.4	
9-10	1.87×1.935	3.06	110.72	2	221.4	
10-11	2.085×1.935	3.14	126.68	2	253.4	
11-12	2.35×1.935	3.21	145.97	2	291.9	
0-10	.3×1.0	28.63	8.59	2	17.2	Between 2 rings
10-12	1.25×1.0	6.35	7.94	2	15.9	" "
On pier	.8×3.3	20.35	53.72	2	107.4	
"	.2×1.4	20.35	5.70	2	11.4	
			Total	=	2251.9 ft³	= 10.425 tsubo

Arch Ring under Trafficway and Side walk 4 required.

Name	Section	Length	Vol.	No.	Total Vol.	Remarks
Panel Pt. 0-1	1.175×2.168	2.75	70.05 ft³	4	280.2 ft³	
" 1-2	1.19×2.168	2.76	71.21	4	284.8	
" 2-3	1.21×2.168	2.77	72.66	4	290.6	
" 3-4	1.235×2.168	2.795	74.84	4	299.4	
" 4-5	1.27×2.168	2.82	77.64	4	310.6	
" 5-6	1.33×2.168	2.85	82.18	4	328.7	
" 6-7	1.425×2.168	2.885	89.13	4	356.6	
" 7-8	1.55×2.168	2.935	98.63	4	394.5	
" 8-9	1.7×2.168	3.00	110.57	4	442.3	
" 9-10	1.87×2.168	3.06	124.06	4	496.2	
" 10-11	2.085×2.168	3.14	141.94	4	567.8	
" 11-12	2.35×2.168	3.21	163.54	4	654.2	
" 0-10	.30×1.0	28.63	8.59	2	17.2	Between 2 rings.
" 10-12	1.25×1.0	6.35	7.94	2	15.9	"
On pier	.8×3.3	22.18	58.56	4	234.2	
"	.2×1.4	22.18	6.21	4	24.8	
			Total	=	4998.0 ft³	= 23.139 tsubo

Total concrete for one intermediate span = 33.564 tsubo.

Concrete for Arch Ring of End Span. Sheet No. 4 & 9.

Arch Ring under Electric Ry. 2 required.

Name	Section	Length	Vol.	No.	Total vol.	Remarks
Panel Pt. 0-12		34.975	1066.54 ft³	2	2133.1 ft³	See Interm. Span
On pier	.8×3.3	20.35	53.72	1	53.7	
"	.2×1.4	20.35	5.70	1	5.7	
On abut.	.8×2.8	20.35	45.58	1	45.6	
			Total	=	2238.1 ft³	= 10.362 tsubo

Arch Ring under Trafficway and Side walk 2 required.

Name	Section	Length	Vol.	No.	Total Vol.	Remarks
Panel Pt. 0-12		34.975	1,192.96 ft³	4	4771.9 ft³	See Interm. Span
On pier	.8×3.3	22.18	58.56	2	117.1	
"	.2×1.4	22.18	6.21	2	12.4	
On abut.	.8×2.8	22.18	49.68	2	99.4	
			Total	=	5,000.8 ft³	= 23.152 tsubo

Total concrete for one end span = 33.514 tsubo

Centre Pole Base.

Concrete for Centre Pole Base. Sheet No 21.

On Pier. 5 required.

Name	Section	Length	Vol.	No.	Total Vol.	Remarks
	3.5×1.75	6.36	38.96 ft³	1	39.0 ft³	Except Slab
	2.5×2.5	.83	5.19	1	5.2	
Less	1.5' ϕ	2.25	-3.96	1	- 4.0	
			Total	=	40.2 ft³	= 0.186 tsubo

On Abutment. 2 required.

Name	Section	Length	Vol.	No.	Total Vol.	Remarks
	4.67×1.75	6.36	51.98 ft³	1	52.0 ft³	Except Slab
	2.5×2.5	.83	5.19	1	5.2	
Less	1.5' ϕ	2.25	-3.96	1	-4.0	
			Total	=	53.2 ft³	= 0.246 tsubo

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HYOGO KEN

MADE BY K. Inaba DATE 14-5-26 FILE NO

CHECKED BY T.N. DATE 14-5-24 PAGE NO 137

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa

Concrete for Girder Span.

Concrete for Slab and Stem of Beams.

Concrete for Slab of 3 continuous Spans — 3 required — Sheet No. 6

Name	Section	Length	Vol.	No.	Total Vol.	Remarks
Slab	17.9×5.2	80.94	753.3 ft³	1	753.4	Elect. Ry. Slab
"	16.41×5.2	80.94	69,068	2	1381.4	Traffic Way "
Chamfer	.63×1.0÷2	80.94	25.50	2	51.0	" " "
Slab	.73×5.2	80.94	30.72	2	61.4	" " "
"	.5×.4	80.94	16.19	2	32.4	
"	1.99×.63	80.94	101.47	2	202.9	
"	.547×.39	80.94	17.267	2	34.53	
Chamber	.2×.2÷2	80.94	1.62	2	3.2	
Coping	1.38×.05	80.94	5.58	2	11.2	
"	.34×.5	80.94	15.78	2	31.6	
"	.35×1.0	80.94	28.33	2	56.7	
"	.1×.9	80.94	7.28	2	14.6	
"	.65×.25	80.94	13.15	2	26.3	
"	.1×.1÷2	80.94	.40	2	.8	
			Total	=	2,972.2 ft³	= 13.760 tsubo

Concrete for Stem of Beams for 3 continuous Spans — 3 required — Sheet No. 8.

Name	Section	Length	Vol.	No.	Total Vol.	Remarks
Fas. girder	5.9×.83	24.50	119.78 ft³	6	719.4	Fascia girder
Less	3×.675×.83	24.50	-41.48	6	-248.9	Area of segment
Fas. girder	5.9×.83	2.50	12.24	4	49.0	Stem on pier
"	.59×.83	1.22	5.97	4	23.4	"
Beam E&F	1.98×1.25	80.94	200.33	4	801.3	Stem
Chamfer	.08×1.2	1.25	0.12	20	2.4	Cross wise
Beam BCD	1.98×1.0	80.94	160.26	6	961.6	Stem
Chamfer	.25×1.0	1.20	0.30	36	10.8	Cross wise
Beam A	2.18×1.0	80.94	176.45	2	352.9	Stem
Chamfer	.2×1.0	1.20	0.24	12	2.9	Cross wise
"	.15×1.5÷2	77.94	0.88	26	22.9	Longitudinal
"	.3×.3÷2	77.94	3.51	2	7.0	"
Cross walk	1.98×1.0	11.61	22.99	3	69.0	under Elect. Ry
"	2.45×1.0	15.66	38.37	6	230.2	" Traff. Way
"	1.33×2.0	3.08	11.27	6	67.6	" Sidewalk
Bracket	.9×0.8	1.95	1.40	18	25.2	for Fas. girder.
			Total	=	3,097.7 ft³	= 14.341 tsubo

Projection of Slab for Light pedestal on GE3 — 1 required

Name	Section	Length	Vol.	No.	Total Vol.	Remarks
Projection	.75×1.15	2.5	2.16 ft³	2	4.3 ft³	
Less	.75×.5	1.15	-0.43	2	-0.9	
			Total	=	3.4 ft³	= 0.016 tsubo

Concrete for Center Pole base — 1 required — Sheet No. 21

Name	Section	Length	Vol.	No.	Total Vol.	Remarks
	3.5×1.98	4.27	29.54 ft³	1	29.6 ft³	Between Beams
	2.5×.83	2.50	5.18	1	5.2	On Slab
Less	1.5 ft ϕ	2.25	-3.98	1	- 4.0	Hole
			Total	=	30.8 ft³	= 0.143 tsubo

Concrete for Handrail — Sheet No. 13 — 1 required

Name	Section	Length	Vol.	No.	Total Vol.	Remarks
Bottom rail	.4×.4	5.4	0.86 ft³	4	3.4 ft³	4 Bott. rail B6
" "	.6×.3	5.4	0.47	4	3.4	" " "
" "	.3×.3	.4	0.04	8	0.3	" " "
" "	.3×.3	.6	0.05	4	0.2	" " "
" "	.4×.4	5.45	0.95	60	57.0	60 Bott. rail B5
" "	.6×.3	5.45	1.07	60	64.2	" " "
" "	.3×.3	.4	0.04	120	4.8	" " "
" "	.3×.3	.6	0.05	60	3.0	" " "
" "	.4×.4	5.6	0.90	8	7.2	8 Bott. rail B2
" "	.6×.3	5.6	1.01	8	8.1	" " "
" "	.3×.3	.4	0.04	16	0.6	" " "
" "	.3×.3	.6	0.05	8	.4	" " "
Int. post	.6×.7	2.5	1.05	36	37.8	36 Int. post
			Total	=	190.9 ft³	= 0.884 tsubo

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MADE BY K. Inaba DATE 14-5-27 FILE NO

CHECKED BY T.N. DATE 14-5-24 PAGE NO 133

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

Concrete for Girder Span continued.

Concrete for Piers.

Concrete for Piers — Sheet No. 15 & 16.

Pier Type A (GP 3.4.5.6 & 7) — 5 required

Name	Section	Length	Vol.	No.	Total Vol.	Remarks
Beam	3.5×2.0	61.88	433.16	1	433.2	Face to Face of Fas.gr.
Less	2×1.75×6.7	12.37	-29.36	4	-117.4	Segment
Coping	.65×.2	61.88	8.04	2	16.1	Top of Pier
Column	2.0×3.0	9.50	57.00	5	285.0	
"	2.0×.5	9.50	9.50	10	95.0	
"	2.0×.5	6.50	6.50	2	13.0	
Coping	.3×1.2	4.20	1.51	10	15.1	Top of col.
Column	3.0×3.0	5.00	45.00	5	225.0	
Wall	2.0×4.0	12.39	99.12	4	396.5	Between cols.
Bracket	.8×2.0	1.45	2.32	2	4.6	
Total conc. above foundation = 1366.1 ≐ 6.325 tsubo						
Foundation	5.5×5.25	65.6	1,894.2	1	1,894.2	Upper layer
"	5.5×5.25	2.75÷2	39.7	2	79.4	" "
"	2.5×10.0	74.0	1850.0	1	1,850.0	Lower "
Pile head	.7 φ	1.0	-.38	96	-36.5	Less
Total concrete for foundation = 3787.1 ≐ 17.533 tsubo						
Summary of concrete for one pier = 23.858 tsubo						

Pier Type B (GP 1 & 2) — 2 required

Name	Section	Length	Vol.	No.	Total Vol.	Remarks
Beam	3.5×2.0	61.88	433.16	1	433.2	Face to Face of Fas.gr.
Less	2×1.75×6.7	12.39	-29.36	4	-117.4	Segment
Coping	.65×.2	61.88	8.04	2	16.1	Top of pier
Column	2.0×3.0	7.50	45.00	5	225.0	
Column	2.0×.5	7.50	7.50	10	75.0	
Column	2.0×.5	6.50	6.50	2	13.0	
Coping	.3×1.2	4.20	1.51	10	15.1	Top of col.
Column	3.0×3.0	4.00	36.00	5	180.0	
Bracket	.8×2.0	1.45	2.32	2	4.6	
Total Conc. above foundation = 844.6 ≐ 3.911 tsubo						
Foundation	3.5×5.25	65.6	1,205.4	1	1,205.4	Upper layer
"	3.5×5.25	2.75÷2	25.27	2	50.5	" "
"	2.5×10.0	74.0	1850.0	1	1,850.0	Lower "
Pile head	.7 φ	1.0	-.38	96	-36.5	Less
Total concrete for foundation = 3,069.4 ≐ 14.210 tsubo						
Summary of concrete for one pier = 18.121 tsubo						

Concrete for Abutment.

Concrete for Abutment. — Sheet No. 17 & 20.

West Abutment — 1 required

Name	Section	Length	Vol.	No.	Total Vol.	Remarks
Shaft	2.5×1.34	17.90	59.97	1	60.0	Parapet Wall
"	2.6×1.33	17.90	61.90	2	123.8	" "
"	3.26×1.24	5.06	20.54	2	41.1	" "
"	3.75×16.5	63.86	395.33	1	395.3	Main Wall
"	4.0×1.0	71.64	286.56	1	286.6	Haunch
"	6.0×1.5	5.50	49.50	2	99.0	"
Coping	.65×.2	61.88	8.04	1	8.0	
Support	2.0×.5	6.50	6.50	2	13.0	Support for Fas.gr.
Coping	.3×1.2	4.20	1.51	2	3.0	"
Shaft	3.5×19.83	0.39	27.07	2	54.1	Under pedestal
"	3.5×1.0	19.89	69.62	2	139.2	Both ends
"	3.25×19.89	9.00	581.78	2	1163.6	Wing wall
Wing	2.25×19.14	9.25	398.35	2	796.7	Down stream
"	1.34×5.25	2.50	17.60	1	17.6	Upstream
"	1.34×5.25	0.50	3.52	1	3.5	
Coping	.4×.75	22.90	6.87	1	6.9	Down stream
"	.4×.75	20.90	6.27	1	6.3	Upstream
"	.7×1.5	23.00	24.15	1	24.2	Both Sides
Total conc. above foundation = 6,797.9 ≐ 31.472 tsubo						
Foundation	14.0×4.5	88.14	5552.82	1	5,552.8	Under main wall
"	9.0×4.5	8.50	344.25	2	688.5	Wing
Pile head	.8 φ	1.50	-.750	124	-93.0	Less
Total concrete for foundation = 6,148.3 ≐ 28.464 tsubo						
Total Conc. for West Abutment = 59.936 tsubo						

East Abutment — 1 required.

Name	Section	Length	Vol.	No.	Total Vol.	Remarks
Shaft	2.5×1.34	17.90	59.97	1	60.0	Parapet Wall
"	2.6×1.33	17.90	61.90	2	123.8	" "
"	3.26×1.24	5.06	20.54	2	41.1	" "
"	3.75×16.5	63.86	395.33	1	395.3	Main Wall
"	4.0×1.0	71.64	286.56	1	286.6	Haunch
"	6.0×1.5	5.50	49.50	2	99.0	"
Coping	.65×.2	61.88	8.04	1	8.0	
Support	2.0×.5	6.50	6.50	2	13.0	Support for Fas.gr.
Coping	.3×1.2	4.20	1.51	2	3.0	"
Shaft	3.5×19.83	0.39	27.07	2	54.1	
"	3.5×1.0	19.89	69.62	2	139.2	Under pedestal
"	3.25×19.89	9.00	581.78	2	1163.6	Both ends
Wing	2.25×19.14	9.25	398.35	1	398.4	Down stream
"	1.34×5.25	2.50	17.60	1	17.6	" "
"	2.25×19.14	5.75	247.62	1	247.6	Upstream
Coping	.4×7.5	22.90	6.87	1	6.9	Down stream
"	.4×7.5	16.90	.507	1	5.1	Upstream
"	.7×1.5	19.00	19.95	1	20.0	Both sides
Total conc. above foundation = 6,638.3 ≐ 30.733 tsubo						
Foundation	14.0×4.5	88.14	5552.82	1	5,552.8	Under main wall
"	9.0×4.5	8.50	344.25	1	344.3	Down stream Wing
"	9.0×4.5	3.00	121.50	1	121.5	Upstream Wing
Pile head	.8 φ	1.50	-.075	118	-88.5	Less
Total conc. for foundation = 5430.1 ≐ 27.454 tsubo						
Total Conc. for East Abutment = 58.187 tsubo						

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CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

Summary for Concrete.

Concrete for Arch Span.					
Slab	End Span	2@	13.933	=27.866	tsubo
Slab	Int. Span	4@	12.053	=48.212	tsubo
				76.078	
Handrail	End Span	2@	0.253	= 0.506	
"	Int. Span	4@	0.250	= 1.000	
				1.506	
Crown fill		6@	1.785	=10.710	
				10.710	
Beams	End Span	2@	4.150	= 8.300	
"	Int. Span	4@	3.831	=15.324	
				23.624	
Columns		6@	0.909	= 5.454	
				5.454	
Pier	Structure	5@	5.843	=29.215	} (242.515)
	Shaft	5@	42.660	=213.300	
	Foundation	5@	42.037	=210.185	(210.185)
				452.700	
Abutment	Structure	2@	17.269	= 34.538	
	Body	2@	72.684	=145.368	
	Foundation	2@	39.697	= 79.394	
				259.300	
Arch Ring	End Span	2@	33.514	= 67.028	
	Int. Span	4@	33.564	=134.256	
				201.284	
Centre Pole base.	On Pier	5@	0.186	= 0.930	
"	On Abutment	2@	0.246	= 0.492	
				1.422	tsubo

Total concrete for arch span = 1,032.078 tsubo

Concrete for Girder Span.					
Slab	3-continuous spans	3@	13.760	=41.280	
Projection	for GP3	1@	0.016	= 0.016	
				41.296	
Handrail		1@	0.884	= 0.884	
				0.884	
Beams	3-continuous Spans	3@	14.341	=43.023	
				43.023	
Centre Pole Base		1@	0.143	= 0.143	
				0.143	
Piers	Type A	5@	23.858	= [illegible]	
"	Type B	2@	18.121	= [illegible]	
				155.532	above foundation (39.45) / foundation (116.09)
Abutment	West Abut.	1@	[illegible]	= [illegible]	
"	East Abut.	1@	[illegible]	= [illegible]	
				18.123	tsubo

Total concrete for Girder Span = 359.001

Total concrete for one whole bridge = 1391.079 tsubo

Concrete for flood water bed.	36.26+18.45	=	55.21
Pavement for sidewalk on abutment for gir. span	0.11×1.10		1.22
低×護岸工事	2×1.35		2.70
			9.13 tsubo
Total			1,403.209 tsubo

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CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

Forms.

Forms for Arch Span.

Forms for Floor.

Forms for Floor (Slab, Beam, Column etc.)									
End Span 2 required					Intermediate Span 4 required				
Name	Width	Length	Area	Remerks	Name	Width	Length	Area	Remarks
Slab	49.72	66.75	3318.8	Bottom of Slab	Slab	49.72	58.50	2908.6	Bottom of Slab
"	2.92	68.25	199.3	" " "	"	2.92	70.50	205.9	" "
"	(66+5)2	78.75	182.7	Curbs	"	(66+5)2	70.50	163.6	Curbs
"	(06+17)2	78.75	36.2	End of pavement	"	(06+17)2	70.50	32.4	End of pavement
Coping	1.88x2	68.25	256.6	Without refuge bay	Coping	1.88x2	72.90	273.7	
Exp. jt.	.52x3	68.64	107.1	End of Slab	Exp. jt.	.52x2	68.64	71.4	End of Slab
"	.83x14	0.90	10.5	" " Beam	Beam Side	1.8x10	58.50	1,053.0	Beam B,C,D,E&F
Beam Side	1.8x10	66.75	1,201.5	Beam B,C,D,E&F	" Bott.	1.0x10	39.50	395.0	" " " " " "
" Bott.	1.0x10	48.25	482.5	" " " " " "	" Side	.81x2	58.50	94.8	Beam A
" Side	.81x2	66.75	108.1	Beam A	" "	1.35x2	70.50	190.4	" "
" "	1.35x2	78.75	212.6	" "	" Bott.	1.0x2	39.50	79.0	" "
" Bott.	1.0x2	48.25	96.5	" "	" Side	3.06x2	70.50	431.5	Fascia girder
" Side	3.06x2	68.25	417.7	Fascia girder	" Bott.	.75x2	36.00	54.0	" "
" Bott.	.75x2	36.00	54.0	" "	Fill	.8x2	21.90	35.0	End of Fill
Fill	.8x2	21.90	35.0	End of Fill	Column	.66x3	1.00	2.0	Panel Pt # 0
Column	.66x3	1.00	2.0	Panel Pt # 0	"	.66x3	1.30	2.6	" " 2
"	.66x3	1.30	2.6	" " 2	"	.66x14	0.90	8.3	" " 4
"	.66x14	0.90	8.3	" " 4	"	4.0x14	0.67	37.5	" " 4
"	4.0x14	0.67	37.5	" " 4	"	.66x14	0.90	8.3	" " 6
"	.66x14	0.90	8.3	" " 6	"	4.0x14	1.44	80.6	" " 6
"	4.0x14	1.44	80.6	" " 6	"	5.2x14	0.40	29.1	" " 6
"	5.2x14	0.40	29.1	" " 6	"	.66x14	0.90	8.3	" " 8
"	.66x14	0.90	8.3	" " 8	"	4.0x14	3.04	170.2	" " 8
"	4.0x14	3.04	170.2	" " 8	"	5.2x14	0.50	36.4	" " 8
"	5.2x14	0.50	36.4	" " 8	"	.66x14	0.90	8.3	" " 10
"	.66x14	0.90	8.3	" " 10	"	4.0x14	5.23	292.9	" " 10
"	4.0x14	5.23	292.9	" " 10	"	5.2x14	0.60	43.7	" " 10
"	5.2x14	0.60	43.7	" " 10	Cross beam	3.3x2	43.70	288.4	On Span, both sides
Cross beam	3.3x2	43.70	288.4	On Span, Both Sides	"	3.0x4	3.00	36.0	" " "
"	3.0x4	3.00	36.0	" " "	"	3.8x2	43.70	332.1	On Pier,
"	3.8	43.70	166.1	On Abutment,	"	1.58x3x2	0.60	24.6	" B.H.
"	1.58x13	0.60	12.3	" Bott.	"	4.7x2	3.00	56.4	" Top & Side
"	4.1x2	3.00	24.6	" " Top & Bott.	Column	4.0x2x2	5.60	537.6	On Pier,
"	3.4	43.70	148.6	" " Both Sides	"	2.8x12x2	2.30	154.6	" "
"	5.0x2	3.00	30.0	" " "	"	5.2x24	0.30	37.4	" "
Slab	2.8x2	8.80	49.3	Refuge bay	Bracket	1.8x22	1.86	73.7	" "
"	1.0x2	8.80	17.6	" "	Projection	1.15x2	4.00	9.2	Slab under pedestal
Coping	2.84x2	23.30	132.3	" "					
Less	1.5x2	4.50	- 13.5	Top of Bracket			Total =	8,266.5	= 229.6 tsubo.
Column	4.0x12	.560	268.8	On Abutment C12					
"	2.3x12	1.20	33.1	" "					
"	5.2x12	0.60	37.4	" "					
"	4.0x12	10.50	504.0	" AAC1					
"	2.3x12	0.70	19.3	" "					
"	5.2x12	0.60	37.4	" "					
		Total =	9239.0	= 256.6 tsubo.					

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CALCULATIONS FOR

MuKo-O-Hashi over the MuKogawa.
Forms for Arch Span continued.
Form for Pier, Abutment and Handrail.

Form for Pier

5 required

Name	Width	Length	Area	Remarks
Coping	2.36x2	63.95	301.5	
Shaft.	16.5x2	63.45	2110.4	
"	16.5x2	10.30	340.0	Cut water nose
Foundation	3.0	185.60	556.8	Upper layer
"	4.0	199.60	798.4	Lower "
Face column	17.1x2	9.73	332.8	
" "	7.4x2	3.40	50.3	
Bracket	0.5x6	1.36	4.1	
"	0.8x3	2.10	5.0	
		Total =	4,499.6 m²	= 1250 tsubo

Form for Handrail

End Span 2 required

Name	Width	Length	Area	Remarks
Bottom rail	2.5	4.7x22	258.5 m²	22 Bott. rail Ba
"	0.3	4.0x22	26.4	" " "
"	2.5	5.35x2	26.8	2 " Bd
"	0.3	4.0x2	2.4	" "
Int. post	2.6	2.5x12	78.0	12 Int. post
		Total =	342.1 m²	= 10.9 tsubo

Form for Handrail

Intermediate Span 4 required

Name	Width	Length	Area	Remarks
Bottom rail	2.5	4.7x24	282.0 m²	24 Bott. rail Ba
"	0.3	4.0x24	28.8	" " " "
Int post	2.6	2.5x12	78.0	12 Inp. post
		Total =	388.8 m²	= 10.9 tsubo

Form for Abutment

2 required

Name	Width	Length	Area	Remarks
Column	3.5x12	14.3	600.1 m²	Column AAC2
Wall	63.66	13.3	846.7	Girder side
Coping	62.00	0.8	49.6	
Support	1.00	11.0	11.0	Support for Fas.gr.
Wall	47.70	14.5	691.7	Arch side
Face col.	21.42	7.58	162.4	Under coping
Less	0.50	3.0	- 1.5	" "
Moulding	2.80	7.5	21.0	Side
"	1.70	5.5	9.4	Top
Coping	1.93	11.7	22.6	
Side	4.05	3.96x2	32.1	
Front	3.30	8.9	29.4	Face
"	1.00	7.0	7.0	Bott. plane of arch
Moulding	2.10	8.9	18.7	Top moulding
Bracket	1.6x4	3.15	20.2	Side
"	1.2x2	6.5	15.6	Face
Face col.	14.50	13.7	198.7	Back of Face col.
Projection	3.47	11.5	39.9	Bottom
"	1.20	11.5	13.8	"
"	2.00	19.1	38.2	Side
"	7.50	16.8	126.0	"
Body	14.00	64.71	905.9	Front face
Coping	2.36	65.8	155.3	"
Body	0.695x2	14.0	19.5	Side plane
"	6.0x2	11.5	138.0	" "
"	5.8x2	10.3	119.5	" "
"	14.5	64.71	938.3	Back
Foundation	5.0	186.0	930.0	
		Total =	6,159.1 m²	= 171.1 tsubo

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CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.
Forms for Girder Span.
Forms for Floor, Pier and Handrail.

Forms of Floor for 3 continuous Spans

3 required

Name	Width	Length	Area	Remarks
Slab	48.99	90.0	3,910.7	Bottom of Slab
"	3.03	81.0	249.5	" " "
"	3.03×2	1.0	6.2	" " "
"	(.66+5)2	31.0	187.9	Curb
"	(.96+17)2	81.0	37.3	End of pavement
Coping	1.33×2	31.0	304.6	
Wall	2.25×2	42.7×3	576.5	3 walls
"	1.83×2	3.08×6	67.6	6 "
Ex. jt.	.52×2	68.64	71.4	End of Slab
"	2.25×2	13.00	58.5	End of beams
"	6.4×2	.83×2	21.3	" Fas. gr.
Beams	5.21×4	78.0	1,625.5	Beam E & F
"	4.96×8	78.0	3,045.0	" A, B, C & D
"	4.93×4	77.2	1,522.4	Fascia girder
"	.83×2	78.9	131.0	" "
Bracket	2.0×18	1.95	70.2	
		Total =	11,935.3 □'	= 331.5 tsubo

Forms for Handrail

1 required

Name	Width	Length	Area	Remarks
Both. rail	2.5	54×2	54.0	4 Both. rail B6
" "	.3	40×	4.8	" " " "
" "	.5	195×2	342.5	60 " " Bc
" "	.3	120	72.0	" " " "
" "	2.5	56×8	112.0	8 " " "
" "	.3	10×8	7.6	" " " "
Int. post	2.5	2.5×36	234.0	36 Int. post
		Total =	1,378.9 □'	= 38.3 tsubo

Forms for Pier

Pier Type A (GP. 3. 4. 5. 6 & 7)

5 required

Name	Width	Length	Area	Remarks
Beam	6.56	12.39×4	325.1	Beam on span
"	26.70	2.85	76.1	Beam on 5 Cols.
Coping	0.85	61.88×2	104.6	Top of beam
Column	12.00	14.5×5	870.0	Under coping
"	2×2	6.1	24.4	Above "
Bracket	.75×4	2.0	6.0	Side plane
"	.8×2	3.0	4.8	Face "
Wall	10	49.6	496.0	Between bott. of "A"
Less	2×8	4.0	−64.0	
Foundation	5.8×2	135.6	1,573.0	Side plane
"	.525×2	5.5	57.8	End plane
"	2.5	168.0	420.0	
		Total =	3,833.3	= 103.2 tsubo

Pier Type B (GP. 1 & 2)

2 required

Name	Width	Length	Area	Remarks
Beam	6.56	12.39×4	325.1	Beam on span
"	26.70	2.85	76.1	" " 5 Cols.
Coping	0.85	61.88×2	105.2	Top of Beam
Column	12.00	11.5×5	690.0	Under Coping
"	2.6×2	6.1	24.4	Above "
Bracket	.75×4	2.0	6.0	Side plane
"	.6×2	3.0	4.8	Face "
Foundation	4.0×2	135.6	1,084.8	Side plane
"	.525×2	3.5	36.8	End plane
"	2.5	168.0	420.0	
		Total =	2,773.2 □'	= 77.0 tsubo

増田淳氏関係資料
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HYOGO KEN

MADE BY K. Inaba DATE 14-5-30 FILE NO

CHECKED BY T.N. DATE 14-6-3 PAGE NO 138

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.
Forms for Girder Span continued.
Forms for Abutment.

Forms for Abutment

West Abutment

1 required

Name	Width	Length	Area	Remarks
Parapet wall	3.48	63.86	222.2	Front side
Main wall	15.85	63.86	1,012.2	"
"	0.85	63.86	54.3	Coping
"	1.0×2	7.00	14.0	Support for Fas.gr.
"	1.64×2	13.14	253.3	Both ends
Chamfer	4.4	71.64	315.2	Center
"	6.7	5.5×2	73.7	Both ends
Main Wall	1.0	14.14×2	28.3	Side
Wing Wall	1.5	6.0×2	18.0	Side of chamfer
"	19.14	12.5×2	478.5	Both wing out side
"	5.25	3.00	15.8	" " "
Coping	1.08	44.00	47.5	out side
Wing Wall	2.25	44.20	99.5	End of wing
Main "	19.60	77.64	1521.7	Back Side
Wing wall	9.25×2	19.14	354.1	" "
"	3.0	5.25	15.8	" "
Coping	1.1	23.00	25.3	" "
Foundation	4.5	238.3	1,072.4	
		Total =	5621.8 □'	= 156.2 tsubo

East Abutment

1 required

Name	Width	Length	Area	Remarks
Parapet wall	3.48	63.86	222.2	Front side
Main wall	15.85	63.86	1,012.2	"
"	0.85	63.86	54.3	Coping
"	1.0×2	7.00	14.0	Support for Fas.gr.
"	9.64×2	13.14	253.3	Both ends
Chamfer	4.40	71.64	315.2	Center
"	6.70	5.5×2	73.7	Both ends
Main wall	1.00	14.14×2	28.3	Side
Wing wall	1.50	6.0×2	18.0	Side of chamfer
"	19.14	21.50	411.5	Both wing out side
"	5.25	2.50	13.1	" "
Coping	1.08	40.00	43.2	Out side
Wing wall	2.25	43.70	98.3	End of wing
Main wall	19.60	77.64	1521.7	Back side
Wing Wall	15.00	19.14	287.1	" "
"	2.50	5.25	13.1	" "
Coping	1.10	19.00	20.9	" "
Foundation	4.50	227.30	1,022.8	
		Total =	5422.9 □'	= 150.6 tsubo

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HYOGO KEN

MADE BY K. Omata DATE 14-5-30 FILE NO

CHECKED BY T.Y. DATE 14-6-3 PAGE NO 134

CALCULATIONS FOR

Miuko-O-Hashi over the Mukogawa.

Forms.

Summary of Forms.

Arch Span.					
Floor	End Span	2@	255.6	=513.2	tsubo
"	Int Span	4@	229.6	=918.4	
				1431.6	tsubo
Pier		5@	1250	=6250	
				6250	
Abutment		2@	171.1	=342.2	
				342.2	
Handrail	End Span	2@	10.9	= 21.8	
	Int. Span	4@	10.8	= 43.2	
				65.0	tsubo
			Total for Arch Span		=2,063.0
Girder Span.					
Floor	3 continuous Span	3@	331.5	=994.5	
				994.5	
Pier	Type A	5@	138.2	=541.0	
	Type B	2@	77.0	=154.	
				695.0	
Abutment	West abut.	1@	156.2	= 156.2	
	East abut.	1@	150.6	=150.6	
				306.8	
Handrail		1@	38.3	= 38.3	
				38.3	tsubo
			Total for Girder Span		=2034.6

Summary for the whole bridge 4498.4 Tsubo

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HYOGO KEN

MADE BY K. Inaba DATE 14-5-30 FILE NO

CHECKED BY T.N DATE 14-6-6 PAGE NO 140

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

List of Artificial Granite Finish.

Arch Span

Handrail

Artificial Granite Finish for Handrail

End Span 2 required

Name	Width	Length	Area	No	Total Area	Remarks
Bottom rail Ba	.204	4.70'	9.59□'	22	211.0□'	Both sides
"	0.05	5.40	027	22	5.9	Side of moulding
"	040	3.30	1.32	22	29.0	Bottom
"	030	4.40	1.32	22	29.0	Sides of legs
" Bd	204	5.35	10.91	2	21.8	Both sides
"	005	5.40	027	2	0.5	Side of moulding
"	040	3.95	1.58	2	3.2	Bottom
"	030	4.40	1.32	2	2.6	Sides of legs
Intermediat posts	2.60	2.50	6.50	12	78.0	Sides of post
			Total	=	381.0□'	= 10.58 tsubo

Intermediate Span 4 required

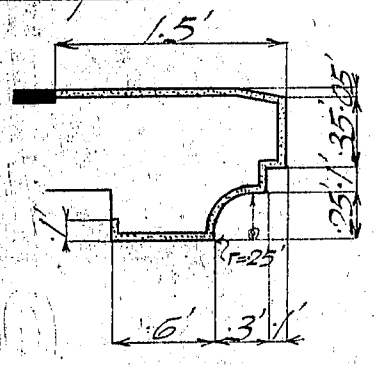
Bottom rail Ba	204	4.7'	9.59□'	24	230.2□'	Both sides
" "	005	5.4	027	24	6.5	Side of moulding
" "	040	3.3	1.32	24	31.7	Bottom
" "	030	4.4	1.32	24	31.7	Sides of legs
Intermediate posts	2.60	2.5	6.50	12	78.0	Sides of post
			Total	=	378.1□'	= 10.50 tsubo

Coping

Artificial Granite Finish for Coping

End Span 2 required

Name	Width	Length	Area	No	Total Area	Remarks
Top	1.50'	65.85'	98.78□'	2	197.6□'	
Less batt. of panel post	1.00	1.00	- 1.00	10	- 10.0	
" " " Int. post	060	0.70	- 0.42	12	- 5.0	
" " Bott. rail	040	1.40	- 0.56	24	- 13.4	
" " " End post	035	1.00	- 0.35	2	- 0.7	
Side	099	65.20	64.55	2	129.1	
Bottom	060	65.20	39.12	2	78.2	
Back	010	65.20	6.52	2	13.0	
Less top of Bracket	040	0.90	- 0.36	22	- 7.9	
			Total	=	380.9□	= 10.58 tsubo



Intermediate Span 4 required

Top	1.50'	65.70'	98.55□'	2	197.1□'	
Less batt. of Panel post	1.00	1.00	- 1.00	10	- 10.0	
" " " Int. post	060	0.70	- 042	12	- 5.0	
" " " Bott. rail	040	1.40	- 0.56	24	- 13.4	
" " End post	035	1.00	- 035	4	- 1.4	
Side	099	65.25	64.60	2	129.2	
Bottom	060	65.25	39.15	2	78.3	
Back	010	65.25	6.53	2	13.1	
Less top of Bracket	040	0.90	- 0.36	22	- 7.9	
			Total	=	380.0□'	= 10.56 tsubo

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

HYOGO KEN

MADE BY K. Inaba DATE 12-5-3 FILE NO

CHECKED BY T.N DATE 12-6-4 PAGE NO 141

CALCULATIONS FOR

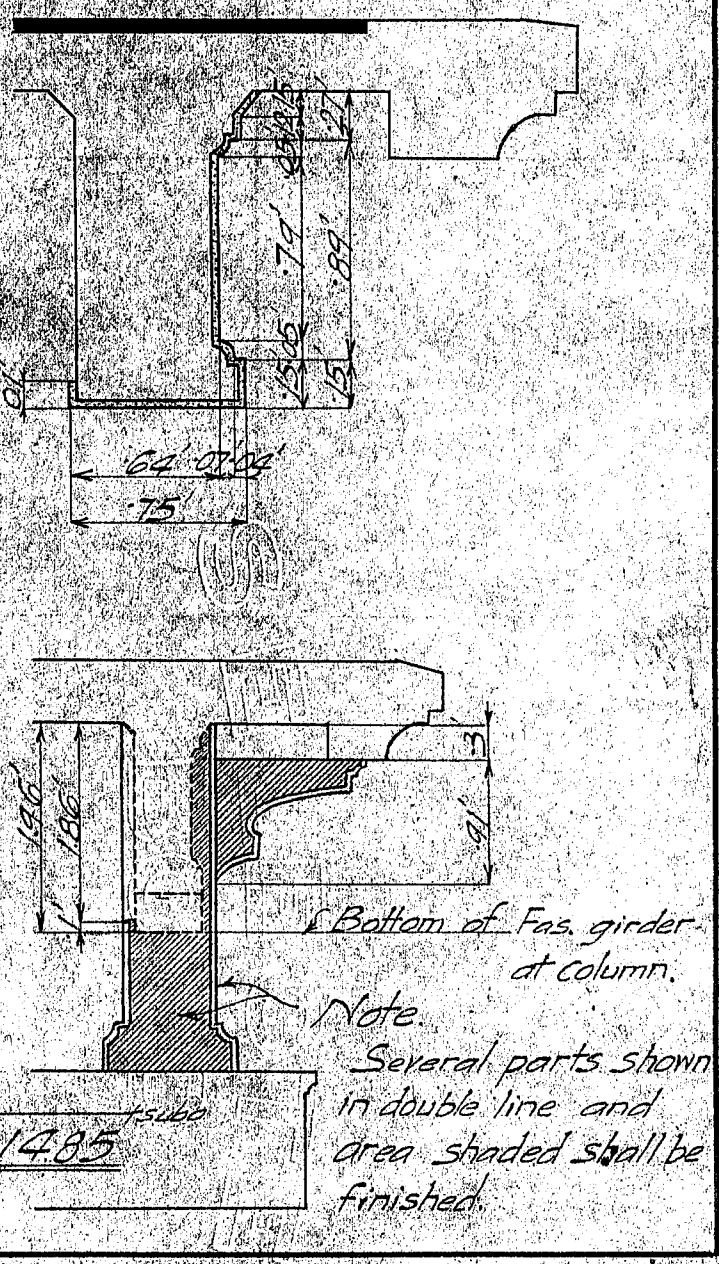
Muko-O-Hashi over the Mukogawa

Artificial Granite Finish for Arch Span continued.

Fascia Girder, Column and Bracket.

Artificial Granite Finish for Fascia Girder Column & Bracket. 6 required.

Name	Width	Length	Area	No	Total Area	Remarks
Fas. girder, side	1.75'	4.50	7.88"	16	126"	
" " Bott.	0.75	4.70	3.53	16	56.5	
" " Back	0.10	4.70	0.47	16	7.5	
" " Side	1.74	4.50	7.83	4	31.3	
" " "	1.24	4.50	5.58	4	22.3	
Bracket coping	0.21	2.40	0.92	22	20.2	
" Side	0.35	3.42	1.20	22	26.4	
" Face	0.70	2.45	1.72	22	37.8	
Column 10	4.00	4.94	19.76	4	79.0	
" " "	1.25	1.86	2.33	4	9.3	
" " "	5.20	0.60	3.12	4	12.5	
" " "	0.05	4.00	0.20	4	0.8	
Column 8	4.00	2.70	10.80	4	43.2	
" " "	1.25	1.86	2.33	4	9.3	
" " "	5.20	0.50	2.60	4	10.4	
" " "	0.05	4.00	0.20	4	0.8	
Column 6	4.00	1.05	4.20	4	16.8	
" " "	1.25	1.86	2.33	4	9.3	
" " "	5.20	0.40	2.08	4	8.3	
" " "	0.05	4.00	0.20	4	0.8	
Column 4	1.25	2.12	2.65	4	10.6	
" 2	1.25	1.36	1.70	4	6.8	
" 0	1.25	1.11	1.39	2	2.8	
Less Bracket area	0.70	0.91	- 0.64	22	- 14.1	
			Total	=	534.7"	= 14.85 tsubo

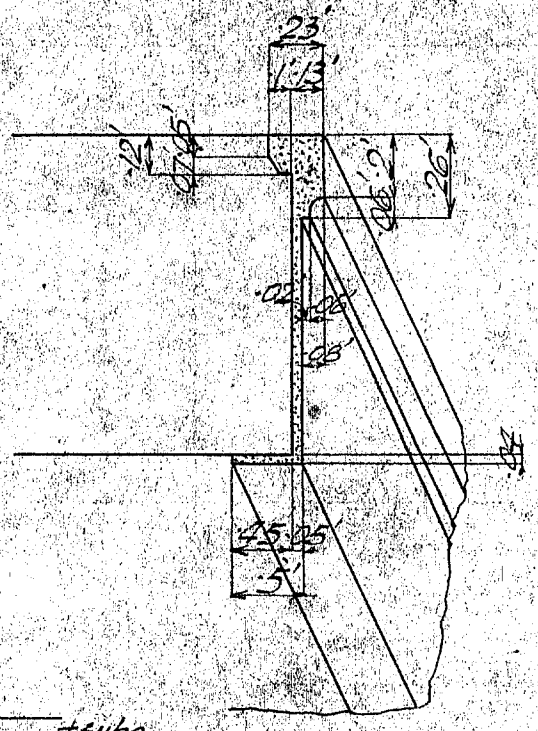


Note.
Several parts shown in double line and area shaded shall be finished.

Arch Ring.

Artificial Granite Finish for Arch Ring 6 required.

Name	Width	Length	Area	No	Total Area	Remarks
Extrad & Moulding	0.54'	69.80'	37.69"	2	75.4"	
Intradose	0.50	70.20	35.10	2	70.2	
0 - 1	0.955	2.75	2.63	4	10.5	
1 - 2	0.97	2.76	2.68	4	10.7	
2 - 3	0.99	2.77	2.74	4	11.0	
3 - 4	1.015	2.795	2.84	4	11.4	
5 - 6 ← 4 - 5	1.05	2.82	2.96	4	11.8	
6 - 7 ↘	1.11	2.85	3.16	4	12.6	
7 - 8 ↘	1.205	2.885	3.48	4	13.9	
8 - 9 ↘	1.33	2.935	3.90	4	15.6	
9 - 10 ↘	1.48	3.00	4.44	4	17.8	
	1.65	3.06	5.05	4	20.2	
10 - 11	1.865	3.14	5.86	4	23.4	
11 - 12	2.13	3.21	6.84	4	27.4	
End	0.595	1.02	0.61	4	2.4	
Less End	0.70	0.90	- 0.63	4	- 2.5	
			Total	=	331.8"	= 9.22 tsubo



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

HYOGO KEN,

MADE BY K. Iraba DATE 14-6-1 FILE NO

CHECKED BY T.Y. DATE 14-6-4 PAGE NO 142

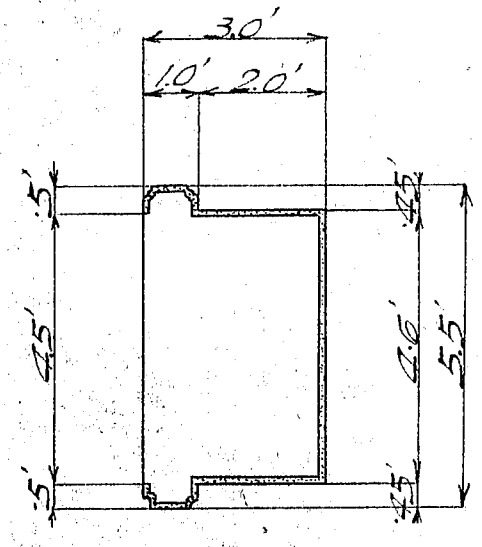
CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

Artificial Granite Finish for Arch Span continued.

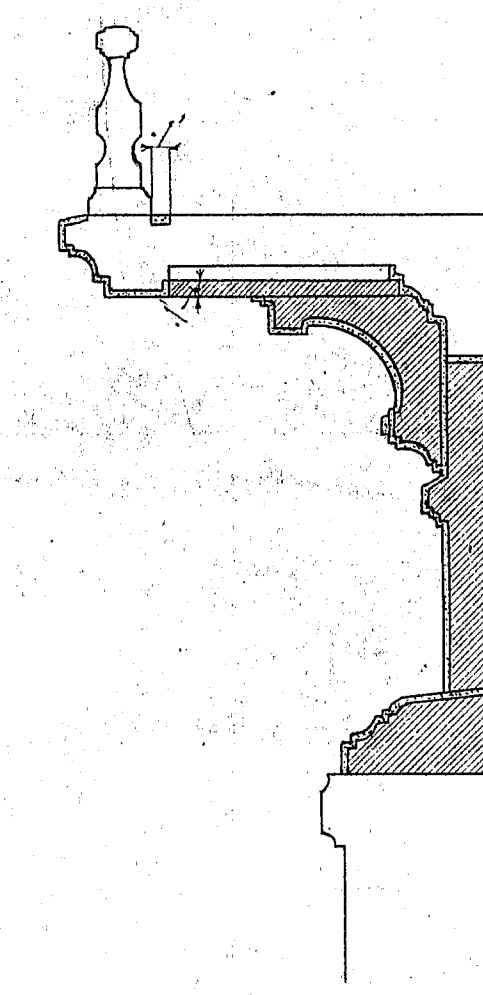
Pier

Artificial Granite Finish for Pier — 5 required						
Name	Width	Length	Area	No.	Total Area	Remarks
Coping, Top	3.00'	6.00'	18.00□'	2	36.0□'	
Less Bott. of Pedestl	2.20	4.80	− 10.56	2	− 21.0	
" " "	0.50	3.00	− 1.50	2	− 3.0	
Coping, Side	0.94	8.70	8.18	2	16.4	
" Bott.	0.60	8.20	4.92	2	9.8	
Less Bracket area	0.40	1.20	− 0.48	6	− 2.9	
Back	0.10	7.00	0.70	2	1.4	
Face column	4.60	13.33	61.32	2	122.6	
Less Bracket area	0.90	0.90	− 0.81	6	− 4.9	
" Stone Y	1.70	4.40	− 7.48	2	− 15.0	
Face column	2.00	10.28	20.56	4	82.2	
" "	1.40	3.05	4.27	4	17.1	
Column C12	2.00	10.28	20.56	4	82.2	
Less Fas. gr. area	0.70	1.96	− 1.37	4	− 5.5	
" back	0.50	1.86	− 0.93	4	− 3.7	
Coping of shaft.	2.17	639.5	1387.7	2	277.5	
			Total	=	589.0□' =	16.36 tsubo.



Abutment.

Artificial Granite Finish for Abutment — 2 required						
Name	Width	Length	Area	No.	Total Area	Remarks
Coping Top	0.70'	1.70'	1.19□'	4	4.8□'	
" "	1.10	3.85	4.24	4	17.0	
" "	0.10	2.10	0.21	4	0.8	
" "	0.10	8.70	0.87	2	1.7	
" "	0.40	11.60	4.64	2	9.3	
" Side	1.49	5.40	8.05	4	32.2	
" "	1.49	13.40	19.97	2	39.9	
" Bott.	0.90	12.20	10.98	2	22.0	
" Back	0.10	18.20	1.82	2	3.6	
Moulding	1.20	8.90	10.68	2	21.4	
"	0.30	8.90	2.67	2	5.3	
"	0.30	17.80	5.34	2	10.7	
Face	2.96	8.90	26.34	2	52.7	
"	1.00	7.20	7.20	2	14.4	
"	5.50	7.00	38.50	2	77.0	
Coping, Column	1.45	3.80	5.51	4	22.0	
Less. Bracket area	1.00	1.20	− 1.20	4	− 4.8	
Coping Side	1.37	6.30	8.63	4	34.5	
Column Inside	1.00	6.86	6.86	4	27.4	
" Face	3.00	7.58	22.74	4	91.0	
Less moulding area	1.00	1.30	− 1.30	4	− 5.2	
Moulding, Top	1.60	5.50	8.80	2	17.6	
" Side	2.70	7.50	20.30	2	40.6	
Projection Top	0.50	1.70	0.85	4	3.4	
Column Side	0.30	1.50	0.45	4	1.8	
Coping, ...	0.60	3.96	2.38	4	9.5	
Column Side	3.50	3.96	13.86	4	55.4	



Note. Several parts shown in double line and area shaded shall be finished.

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HYOGO KEN

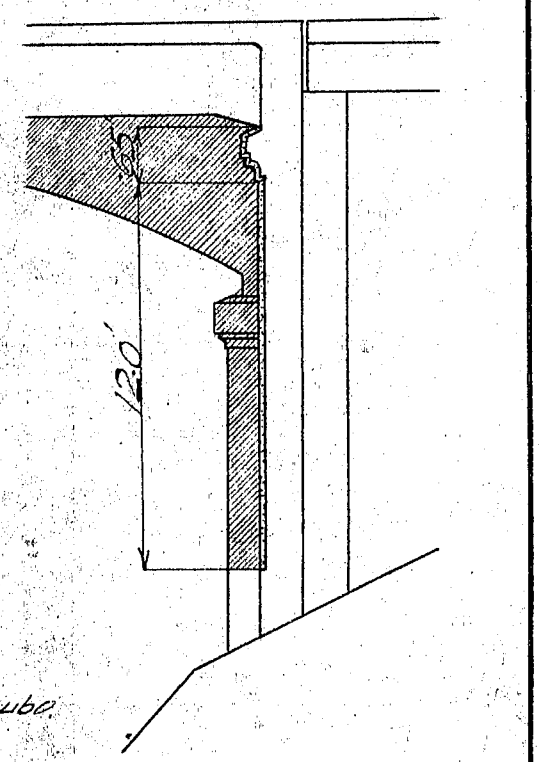
MADE BY K Inaba DATE 14-6-1 FILE NO

CHECKED BY T.Y. DATE 14-6-4 PAGE NO 143

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.
Abutment continued.

Coping, Side	1.45'	4.36'	6.32 0'	4	25.3 0'
Column, arch side	3.96	4.63	18.33	2	36.7
" "	3.46	3.05	10.55	2	21.1
" air. side	3.96	7.48	29.62	2	59.2
Bracket side	0.90	3.15	2.84	8	22.7
" Face	1.20	8.00	9.60	4	38.4
Coping of wall	1.08	61.88	66.83	1	66.8
Support of Fas gr.	0.90	2.80	2.52	2	5.0
" Coping Side	1.42	4.20	5.96	2	11.9
" Column	3.00	9.50	28.50	2	57.0
Column C12	2.00	10.00	20.00	2	40.0
Less Fas.gr. area	0.70	1.96	- 1.37	2	- 2.7
" Back	1.25	1.86	- 2.33	2	- 4.7
Wall girder side	12.00	59.56	714.72	1	714.7
" " "	.17	2.80	.328	2	6.6
			Total	=	1,704.0 m² = 4733 +sub.



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HYOGO KEN

MADE BY K. Inaba DATE 14-6-2 FILE NO

CHECKED BY T.N. DATE 14-6-4 PAGE NO 144

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

Artificial Granite Finish for Girder Span.

Handrail

Artificial Granite Finish for Handrail — 1 required.

Name	Width	Length	Area	No.	Total Area	Remarks
Bottom rail Bb	2.04'	5.40'	11.02 □'	4	44.1 □'	
" " "	0.05	5.40	0.27	4	1.1	
" " "	0.40	3.85	1.54	4	6.2	
" " "	0.30	4.40	1.32	4	5.3	
" " Bc	2.04	5.95	12.14	60	728.4	
" " "	0.05	5.40	0.27	60	16.2	
" " "	0.40	4.55	1.82	60	109.2	
" " "	0.30	4.40	1.32	60	79.2	
" " Be	2.04	5.60	11.42	8	91.4	
" " "	0.05	5.40	0.27	8	2.2	
" " "	0.40	4.20	1.68	8	13.4	
" " "	0.30	4.40	1.32	8	10.6	
Intermediate post.	2.60	2.50	6.50	36	234.0	
			Total	=	1341.3 □'	= 37.26 tsubo

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

HYOGO KEN

MADE BY K. Inaba DATE 14-6-2 FILE NO

CHECKED BY T.N. DATE 14-6-4 PAGE NO 145

CALCULATIONS FOR

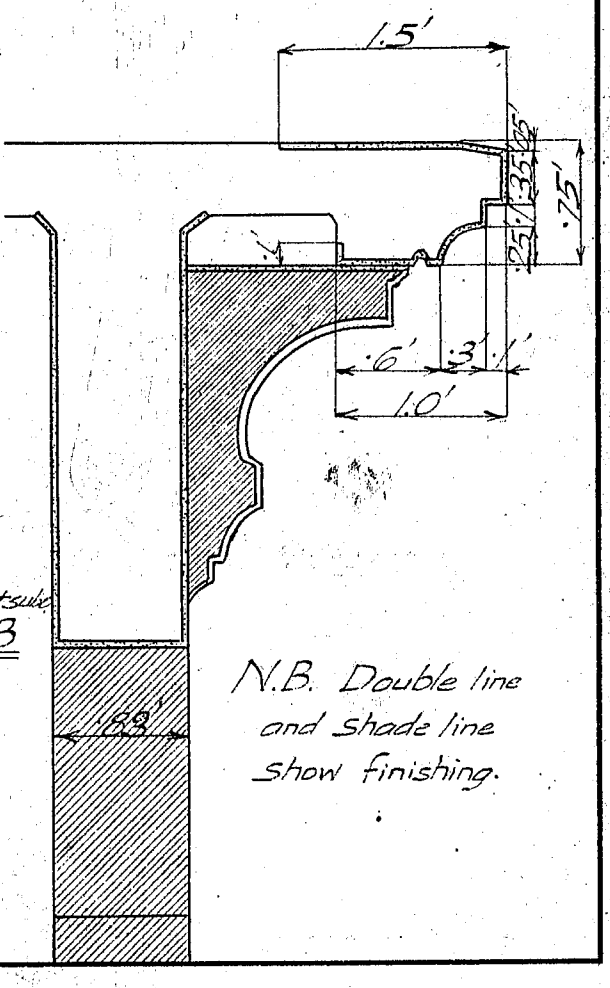
Muko-O-Hashi over the Mukogawa.

Artificial Granite Finish for Girder Span continued.

Floor.

Artificial Granite Finish for Floor (3-continuous span) 3 required.

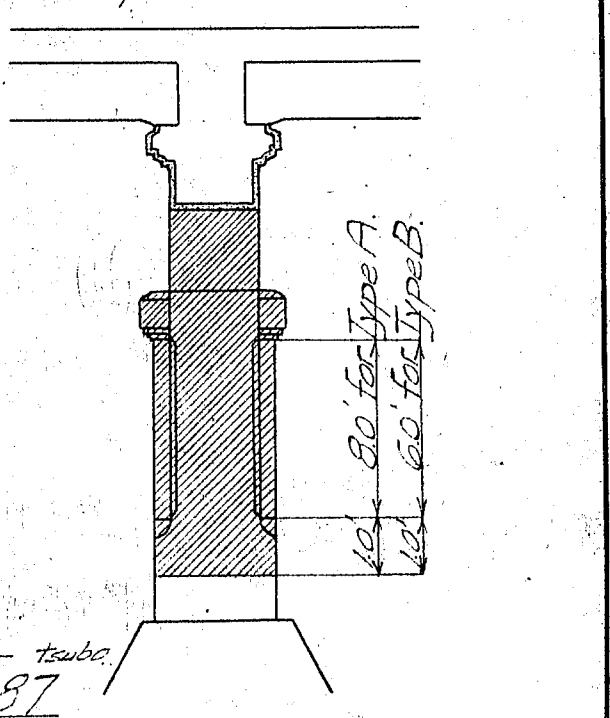
Name	Width	Length	Area	No.	Total Area	Remarks
Top	1.50'	80.0'	120.00□'	2	240.0□'	
Less bott. of Panel post	1.00	1.0	− 1.00	10	− 10.0	
" " " Int. post	0.60	0.7	− 0.42	12	− 5.0	
" " " Bott. rail	0.40	1.4	− 0.56	24	− 13.4	
Side	0.99	80.0	79.20	2	158.4	
Bott.	0.60	80.0	48.00	2	96.0	
Back	0.10	80.0	8.00	2	16.0	
Less Top of Bracket	0.40	1.1	− 0.44	18	− 7.9	
Fas. girder Out side	5.00	25.0	125.00	6	750.0	
" " Bottom	0.83	26.3	21.83	6	131.0	
" " Back	5.00	25.0	125.00	6	750.0	
Bracket coping	0.26	8.6	2.24	18	40.3	
" Side	0.75	1.65	1.24	36	44.6	
" Face	0.80	3.0	2.40	18	43.2	
Less Bracket area.	1.10	2.0	− 2.20	18	−39.6	
			Total		= 2,193.6□'	= 60.93 tsubo



Pier Type A

Artificial Granite Finish for Pier Type A 5 required (GP 3.4.5.6 & 7)

Name	Width	Length	Area	No.	Total Area	Remarks
Coping	1.24'	61.88'	76.73□'	2	153.5□'	
Beam on Span	2.27	12.39	28.13	8	225.0	
" " "	2.00	13.00	26.00	4	104.0	
" on Col.	2.85	3.00	8.55	6	51.3	
" " "	2.85	1.66	4.73	4	18.9	
Column	3.00	6.44	19.32	2	38.6	
Less Bracket area	1.10	− 2.00	− 2.20	2	− 4.4	
Bracket coping	0.26	7.60	1.98	2	4.0	
" Side	0.75	1.15	0.86	4	3.4	
" Face	0.80	2.50	2.00	2	4.0	
Coping top	0.90	2.80	2.52	10	25.2	
" Side	1.43	4.40	6.29	10	62.9	
Column	4.80	1.20	5.76	5	28.8	
"	12.00	9.00	108.00	5	540.0	
			Total		= 1,255.2□'	= 34.87 tsubo



Pier Type B

Artificial Granite Finish for Pier Type B 2 required (GP 1 & 2)

For Pier Type B, less 12.0×2.0×5 = 120□' from that of Pier Type A

Total area Finished = 1,135.2□' = 31.53 tsubo

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HYOGO KEN

MADE BY K. Inaba DATE 14-6-2 FILE NO

CHECKED BY T.N. DATE 15-6-4 PAGE NO 146

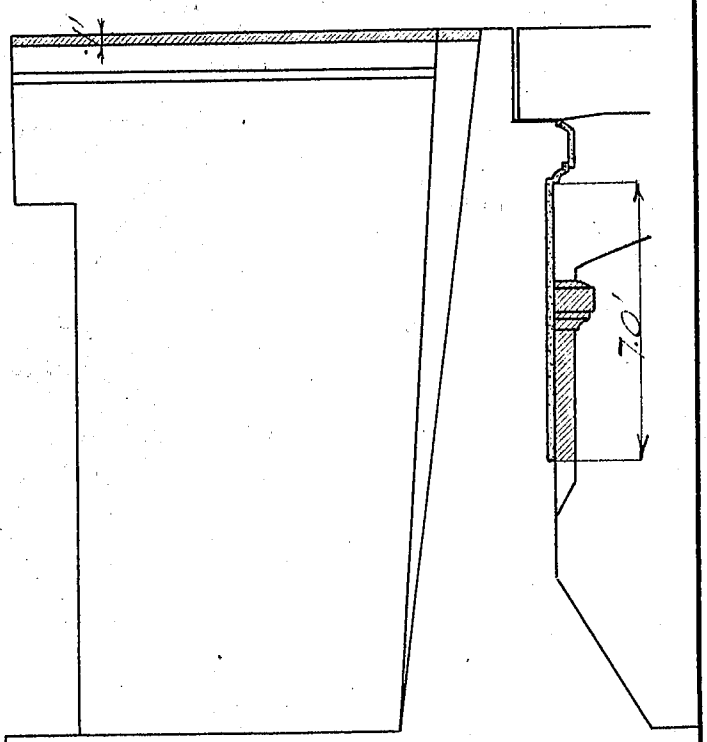
CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

Artificial Granite Finish for Abutment.

Artificial Granite Finish for West Abutment 1 required

Name	Width	Length	Area	No	Total area	Remarks
Main wall front	7.00'	59.56'	416.92□'	1	416.9□'	
" " "	1.17	2.80	3.28	2	6.6	
Support for Fas.gr.	0.90	2.80	2.52	2	5.0	
" Coping side	1.42	4.20	5.96	2	11.9	
" Column	3.00	3.00	9.00	2	18.0	
Front Face Both ends	10.30	10.50	108.15	2	216.3	
Down stream wing	6.00	16.00	96.00	1	96.0	
Up stream "	6.00	14.00	84.00	1	84.0	
Coping, wing top	2.00	23.00	46.00	1	46.0	
" top	3.00	5.50	16.50	2	33.0	
Under Pedestal	4.00	4.00	16.00	2	32.0	
Less Post area	1.80	1.80	- 3.24	4	- 13.0	
" " "	0.60	1.00	- 0.60	2	- 1.2	
" Bott. rail	0.80	28.00	-22.40	1	-22.4	
" Light Pedestal	2.50	3.50	- 8.75	2	-17.5	
" " "	0.50	2.50	- 1.25	2	- 2.5	
Coping out side	1.09	47.00	51.23	1	51.2	
" Inside	0.10	41.00	4.10	1	4.1	
			Total	=	964.4□'	= 26.79 tsubo.



Artificial Granite Finish for East Abutment 1 required

Name	Width	Length	Area	No	Total area	Remarks
Main wall front	7.00'	59.56'	416.92□'	1	416.9□'	
" " "	1.17	2.80	3.28	2	6.6	
Support for Fas.gr.	0.90	2.80	2.52	2	5.0	
" Coping, Side	1.42	4.20	5.96	2	11.9	
" Column	3.00	3.00	9.00	2	18.0	
Front face, Both ends	10.30	10.50	108.15	2	216.3	
Downstream wing	6.00	16.00	96.00	1	96.0	
Upstream "	7.00	10.00	70.00	1	70.0	
Coping, wing top	2.00	19.00	38.00	1	38.0	
" top	3.00	5.50	16.50	2	33.0	
Under Pedestal	4.00	4.00	16.00	2	32.0	
Less. Post area	1.80	1.80	- 3.24	4	- 13.0	
" " "	0.60	1.00	- 0.60	2	- 1.2	
" Bott. rail	0.80	24.00	- 19.20	1	- 19.2	
" Light Pedestal	2.50	3.50	- 8.75	2	- 17.5	
" " "	0.50	2.50	- 1.25	2	- 2.5	
Coping, out Side	1.09	43.00	46.88	1	46.9	
" Inside	0.10	37.00	3.70	1	3.7	
			Total	=	940.9□'	= 26.14 tsubo.

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HYOGO KEN

MADE BY K. Inaba DATE 4-6-[illegible] FILE NO

CHECKED BY T.N. DATE 4-6-74 PAGE NO 147

CALCULATIONS FOR

Muko-C-Hashi over th[illegible]wa.

Summary for Artificial Granite Finish.

Arch Span.					
Handrail	End Span	2@	10.58	=21.16	
	Int. Span	4@	10.50	=42.00	
				63.16	tsubo
Coping	End Span	2@	10.58	=21.16	
	Int. Span	4@	10.56	=42.24	
				63.40	
Fascia girder, column and bracket		6@	14.85	=89.10	
				89.10	
Arch Ring		6@	4.22	=25.32	
				25.32	
Total for Super Structure				=270.98	tsubo
Pier		5@	16.36	=81.80	
				81.80	
Abutment		2@	47.33	=94.66	
				94.66	
Total for Sub Structure				=176.46	
Total for Arch Span = 447.44					tsubo

Girder Span.

Handrail		1@	37.26	=37.26	
				37.26	
Floor		3@	60.93	=182.79	
				182.79	
Total for Super Structure				=220.05	tsubo
Pier	Type A	5@	34.87	=174.35	
	Type B	2@	31.53	=63.06	
				237.41	
Abutment	West abut.	1@	26.79	=26.79	
	East abut.	1@	26.14	=26.14	
				52.93	
Total for Sub Structure				=290.34	
Total for Girder Span = 510.39					

Summary for the Whole Bridge = 957.83 tsubo.

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HYOGO KEN

MADE BY K. Inaba DATE 14-6-3 FILE NO

CHECKED BY T.N. DATE 14-6-8 PAGE NO 148

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.
List for Granite
Granite for Sub Structure.

Pier for Arch Span — 5 required

Coping Stone

Name	Section	Length	Volume	No.	Total volume	Remarks
Coping Cp	1.2×1.5	0.70'	1.26 ft³	2	2.52 ft³	
Cp1	1.2×1.5	1.15	2.07	16	33.12	
Cp2	1.2×1.5	1.25	2.25	4	9.00	
Cap Y	2.0×2.42	4.40	21.30	2	42.60	
			Total	=	87.24 ft³	

Facing Stone

P	1.1×1.0	1.50'	1.65 ft³	2	3.30 ft³
P_1	1.0×1.0	1.50	1.50	16	24.00
P_2	" "	1.50	1.50	4	6.00
O	1.3×1.0	2.70	3.51	2	7.02
O_1	1.0×1.0	1.47	1.47	16	23.52
O_2	" "	1.49	1.49	4	5.96
N	1.1×1.0	1.35	1.49	2	2.98
N_1	1.0×1.0	1.44	1.44	16	23.36
N_2	" "	1.46	1.46	4	5.84
M	1.3×1.0	2.50	3.25	2	6.50
M_1	1.0×1.0	1.42	1.42	16	22.72
M_2	" "	1.48	1.48	4	5.92
L	1.1×1.0	1.20	1.32	2	2.64
L_1	1.0×1.0	1.39	1.39	16	22.24
L_2	" "	1.42	1.42	4	5.68
K	1.25×1.0	2.30	2.88	2	5.76
K_1	1.0×1.0	1.36	1.36	16	21.76
K_2	" "	1.47	1.47	4	5.88
J	1.1×1.0	1.21	1.21	2	2.42
J_1	1.0×1.0	1.34	1.34	16	21.44
J_2	" "	1.38	1.38	4	5.52
I	1.3×1.0	2.20	2.86	2	5.72
I_1	1.0×1.0	1.31	1.31	16	20.96
I_2	" "	1.45	1.45	4	5.80
H	1.0×1.0	0.95	0.95	2	1.90
H_1	" "	1.28	1.28	16	20.48
H_2	" "	1.34	1.34	4	5.36
G	1.2×1.0	2.00	2.40	2	4.80
G_1	1.0×1.0	1.25	1.25	16	20.00
G_2	" "	1.44	1.44	4	5.76
F	1.0×1.0	0.80	0.80	2	1.60
F_1	" "	1.23	1.23	16	19.68
F_2	" "	1.30	1.30	4	5.20
E	1.2×1.0	2.16	2.16	2	4.32
E_1	1.0×1.0	1.20	1.20	16	19.20
E_2	" "	1.43	1.43	4	5.72
D	1.0×1.0	0.70	0.70	2	1.40
D_1	" "	1.17	1.17	16	18.72
D_2	" "	1.26	1.25	4	5.04
C	1.2×1.0	1.60	1.92	2	3.84
C_1	1.0×1.0	1.15	1.15	16	18.40
C_2	" "	1.40	1.40	4	5.60

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MADE BY K. Inaba DATE 14-6-3 FILE NO

CHECKED BY IN DATE 14-6-5 PAGE NO 1-

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.
Granite for Sub Structure continued

B	1.0×1.0'	0.60'	0.60 ℄	2	1.20 ℄	
B_1	" "	1.12	1.12	16	17.92	
B_2	" "	1.21	1.21	4	4.84	
A	1.2×1.0	1.50	1.80	2	3.60	
A_1	1.0×1.0	1.09	1.09	16	17.44	
A_2	" "	1.40	1.40	4	5.60	
			Total	=	480.56	℄

Abutment for Arch Span 2 required

Coping Stone

Name	Section	Length	Volume	No.	Total Vol.	Remarks
CA1	1.1×1.5	1.90'	3.14 ℄	2	6.28 ℄	
CA2	1.65×1.5	2.60	6.44	4	25.76	
CA3	1.1×1.5	2.06	3.40	10	34.00	
CA4	1.1×1.5	2.49	4.11	2	8.22	
			Total	=	74.26	℄

Facing Stone

a	0.96×0.6'	0.97'	0.56 ℄	126	70.56 ℄	
b	0.96×0.83	1.50	1.20	24	28.80	
c	0.96×0.6	0.97	0.56	42	23.52	
d	0.96×0.6	1.29	0.74	6	4.44	
e	0.96×0.6	0.78	0.45	6	2.70	
f	0.96×0.6	0.87	0.50	6	3.00	
g	0.96×0.6	1.37	0.79	6	4.74	
			Total	=	137.76	℄

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MADE BY K. Suzuki DATE 4-4-62 FILE NO.
CHECKED BY T. Y. DATE 4-22-62 PAGE NO. 25

CALCULATIONS FOR

HOOKEN

Wall Type Bridge Abutment

Design Data

Soil Properties: $\gamma_s = 120$ pcf, $\phi = 30°$, $\mu = 0.5$

Concrete: $f'_c = 3000$ psi, $n = 10$, $k = 0.333$, $j = 0.889$, $f_c = 1350$ psi

Steel: $f_s = 20000$ psi

Live Load Surcharge: 2 ft

Earthquake Coefficient: 0.2

Allowable Soil Pressure: 4000 psf

Assumed Dimensions

Top Wall: 1'-6"

Base: 1'-6" × 6'-0"

Stem: 1'-6" × 11'-0"

Total Height: 12'-6"

Toe: 2'-0"

Heel: 4'-0"

Base Width: 6'-0"

Coefficient of Active Earth Pressure

$$C_a = \frac{\cos^2\phi}{\cos\delta\left(1+\sqrt{\frac{\sin(\phi+\delta)\sin\phi}{\cos\delta}}\right)^2} = \frac{\cos^2 30°}{\cos 0°\left(1+\sqrt{\frac{\sin 30° \sin 30°}{\cos 0°}}\right)^2} = \frac{0.75}{4} = 0.333$$

Coefficient of Passive Earth Pressure

$$C_p = \frac{\cos^2\phi}{\cos\delta\left(1-\sqrt{\frac{\sin(\phi+\delta)\sin\phi}{\cos\delta}}\right)^2} = \frac{\cos^2 30°}{\cos 0°\left(1-\sqrt{\frac{\sin 30° \sin 30°}{\cos 0°}}\right)^2} = \frac{0.75}{0.25} = 3.0$$

Coefficient of Friction between Concrete and Soil

$$\mu = \tan\phi = \tan 30° = 0.5$$

Weight of Concrete: 150 pcf

Weight of Soil: 120 pcf

Weight of Water: 62.5 pcf

Unit Stresses

$$f_c = 0.4 f'_c = 0.4 \times 3000 = 1200 \text{ psi}$$

$$f_s = 20000 \text{ psi}$$

$$n = \frac{E_s}{E_c} = \frac{29 \times 10^6}{2 \times 10^6} = 10$$

$$k = \frac{n}{n + \frac{f_s}{f_c}} = \frac{10}{10 + \frac{20000}{1200}} = 0.375$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.375}{3} = 0.875$$

$$R = \frac{1}{2} f_c k j = \frac{1}{2} \times 1200 \times 0.375 \times 0.875 = 197 \text{ psi}$$

Earth Pressure

$$P_a = \frac{1}{2} C_a \gamma_s H^2 = \frac{1}{2} \times 0.333 \times 120 \times 12.5^2 = 3125 \text{ lb/ft}$$

Surcharge Load

$$P_s = C_a \gamma_s h H = 0.333 \times 120 \times 2 \times 12.5 = 1000 \text{ lb/ft}$$

Passive Earth Pressure

$$P_p = \frac{1}{2} C_p \gamma_s h_p^2 = \frac{1}{2} \times 3.0 \times 120 \times 1.5^2 = 405 \text{ lb/ft}$$

Earthquake Load

$$P_e = C \gamma_s H^2 = 0.2 \times 120 \times 12.5^2 = 3750 \text{ lb/ft}$$

Weight of Concrete

$$W_c = 150 \times 1.5 \times 11.0 = 2475 \text{ lb/ft}$$

$$W_b = 150 \times 6.0 \times 1.5 = 1350 \text{ lb/ft}$$

Weight of Soil

$$W_s = 120 \times 4.0 \times 11.0 = 5280 \text{ lb/ft}$$

Weight of Water

$$W_w = 62.5 \times 2.0 \times 11.0 = 1375 \text{ lb/ft}$$

Overturning Moment

$$M_o = P_a \frac{H}{3} + P_s \frac{H}{2} + P_e \frac{H}{2} = 3125 \times \frac{12.5}{3} + 1000 \times \frac{12.5}{2} + 3750 \times \frac{12.5}{2} = 39063 \text{ ft-lb/ft}$$

Resisting Moment

$$M_r = W_c \times 3.0 + W_b \times 3.0 + W_s \times 5.0 + W_w \times 5.0 + P_p \frac{h_p}{3} = 2475 \times 3.0 + 1350 \times 3.0 + 5280 \times 5.0 + 1375 \times 5.0 + 405 \times \frac{1.5}{3} = 47252 \text{ ft-lb/ft}$$

Factor of Safety against Overturning

$$F.S. = \frac{M_r}{M_o} = \frac{47252}{39063} = 1.21 > 1.2 \quad \text{O.K.}$$

Total Vertical Load

$$V = W_c + W_b + W_s + W_w = 2475 + 1350 + 5280 + 1375 = 10480 \text{ lb/ft}$$

Total Horizontal Load

$$H = P_a + P_s + P_e - P_p = 3125 + 1000 + 3750 - 405 = 7470 \text{ lb/ft}$$

Factor of Safety against Sliding

$$F.S. = \frac{\mu V}{H} = \frac{0.5 \times 10480}{7470} = 0.70 < 1.2 \quad \text{N.G.}$$

Resultant Force

$$R = \sqrt{V^2 + H^2} = \sqrt{10480^2 + 7470^2} = 12870 \text{ lb/ft}$$

Eccentricity

$$e = \frac{M_r - M_o}{V} = \frac{47252 - 39063}{10480} = 0.78 \text{ ft}$$

Distance from Toe to Resultant

$$x = \frac{M_r - M_o}{V} + \frac{B}{2} = 0.78 + \frac{6.0}{2} = 3.78 \text{ ft}$$

Pressure at Toe

$$q_t = \frac{V}{B}\left(1 + \frac{6e}{B}\right) = \frac{10480}{6.0}\left(1 + \frac{6 \times 0.78}{6.0}\right) = 2620 \text{ psf}$$

Pressure at Heel

$$q_h = \frac{V}{B}\left(1 - \frac{6e}{B}\right) = \frac{10480}{6.0}\left(1 - \frac{6 \times 0.78}{6.0}\right) = 1310 \text{ psf}$$

Pressure at Footing Edge

$$q_e = \frac{V}{B}\left(1 - \frac{3e}{B}\right) = \frac{10480}{6.0}\left(1 - \frac{3 \times 0.78}{6.0}\right) = 1965 \text{ psf}$$

Pressure at Footing Edge

$$q_e = \frac{V}{B}\left(1 + \frac{3e}{B}\right) = \frac{10480}{6.0}\left(1 + \frac{3 \times 0.78}{6.0}\right) = 2185 \text{ psf}$$

Pressure at Footing Edge

$$q_e = \frac{V}{B}\left(1 - \frac{3e}{B}\right) = \frac{10480}{6.0}\left(1 - \frac{3 \times 0.78}{6.0}\right) = 1965 \text{ psf}$$

Pressure at Footing Edge

$$q_e = \frac{V}{B}\left(1 + \frac{3e}{B}\right) = \frac{10480}{6.0}\left(1 + \frac{3 \times 0.78}{6.0}\right) = 2185 \text{ psf}$$

Pressure at Footing Edge

$$q_e = \frac{V}{B}\left(1 - \frac{3e}{B}\right) = \frac{10480}{6.0}\left(1 - \frac{3 \times 0.78}{6.0}\right) = 1965 \text{ psf}$$

Pressure at Footing Edge

$$q_e = \frac{V}{B}\left(1 + \frac{3e}{B}\right) = \frac{10480}{6.0}\left(1 + \frac{3 \times 0.78}{6.0}\right) = 2185 \text{ psf}$$

Pressure at Footing Edge

$$q_e = \frac{V}{B}\left(1 - \frac{3e}{B}\right) = \frac{10480}{6.0}\left(1 - \frac{3 \times 0.78}{6.0}\right) = 1965 \text{ psf}$$

Pressure at Footing Edge

$$q_e = \frac{V}{B}\left(1 + \frac{3e}{B}\right) = \frac{10480}{6.0}\left(1 + \frac{3 \times 0.78}{6.0}\right) = 2185 \text{ psf}$$

Pressure at Footing Edge

$$q_e = \frac{V}{B}\left(1 - \frac{3e}{B}\right) = \frac{10480}{6.0}\left(1 - \frac{3 \times 0.78}{6.0}\right) = 1965 \text{ psf}$$

Pressure at Footing Edge

$$q_e = \frac{V}{B}\left(1 + \frac{3e}{B}\right) = \frac{10480}{6.0}\left(1 + \frac{3 \times 0.78}{6.0}\right) = 2185 \text{ psf}$

HYOGO KEN

MADE BY K. Inaba DATE 14-6-3 FILE NO ____

CHECKED BY T. N. DATE 14-6-8 PAGE NO 151

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.
List for Granite continued.
Granite for Super Structure.
Girder Span.
Handrail and Light Pedestal.

Name	Section	Length	Volume	No	Total vol	Remarks
Granite for Handrail and Light Pedestal						
Handrail on Span						1 required.
Top rail Tb	0.5x0.7'	5.80'	2.03 ft	4	8.12 ft	
" " Tc	0.5x0.7	6.35	2.22	60	133.20	
" " Tf	0.5x0.7	6.00	2.10	8	16.80	
Panel Post	1.0x1.0	3.20	3.20	30	96.00	
			Total	=	254.12	ft
Handrail on Abutment						1 required.
Top rail T5	0.5x0.7'	5.85'	2.05 ft	4	8.20 ft	Down stream Handrails
" " T6	0.5x0.7	4.85	1.70	2	3.40	Up stream " of West abutment
" " T7	0.5x0.7	4.95	1.73	1	1.73	" " " " East abutment.
" " T8	0.5x0.7	4.35	1.53	4	6.12	
Post P1	0.45x0.6	2.20	0.60	16	9.60	Front
" P3	0.5x0.6	2.20	0.66	32	21.12	Side
" P4	0.6x1.0	2.50	1.50	3	4.50	
Bottom rail B5	0.5x0.8	5.30	2.12	4	8.48	Down stream Handrails
" " B6	0.5x0.8	4.30	1.72	2	3.44	Up stream " of West abutment.
" " B7	0.5x0.8	4.15	1.66	1	1.66	" " " " East abutment
" " B8	0.5x0.8	3.85	1.54	4	6.16	Front
			Total	=	74.41	ft
Light Pedestal on Pier						2 required
Cap stone	0.7x2.6	2.6	4.73 ft	1	4.73 ft	
Column	2.1x2.1	2.7	11.91	1	11.91	
Base	0.6x2.4	2.4	3.46	1	3.46	
			Total	=	20.10	ft
Light Pedestal on Abutment						4 required.
Cap stone	0.6x3.7'	3.7'	8.21 ft	1	8.21 ft	
Column	2.7x2.9	3.2	25.06	1	25.06	
Base	0.5x3.0	3.5	5.25	1	5.25	
			Total	=	38.52	ft
End Posts on Abutment						8 required.
Cap stone	0.6x2.0'	2.0'	2.40 ft	1	2.40 ft	
Column	1.5x1.5	2.5	5.63	1	5.63	
Base	0.5x1.8	1.8	1.62	1	1.62	
			Total	=	9.65	ft

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

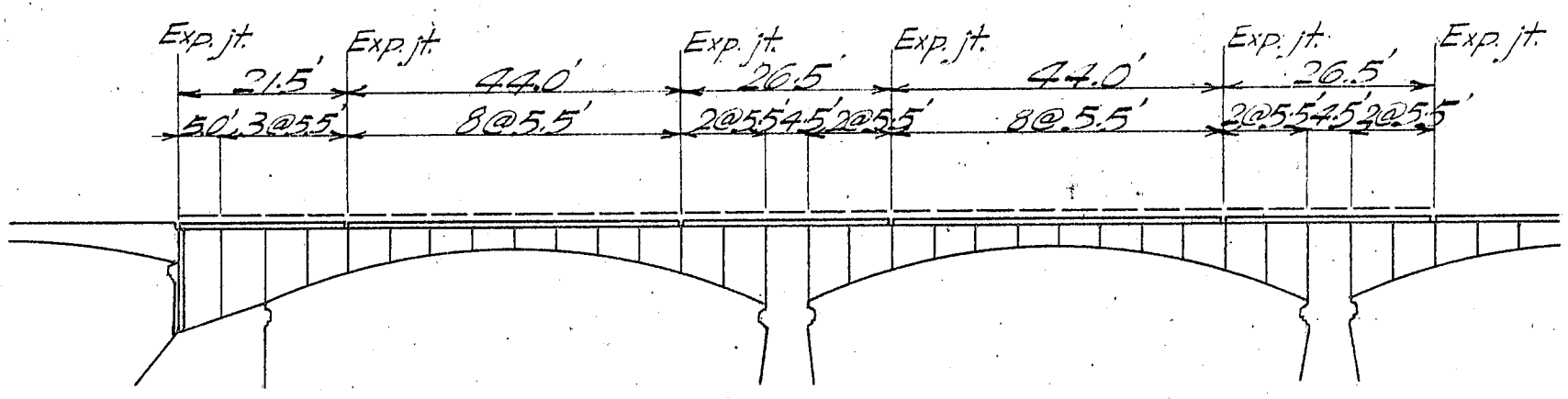
HYOGO KEN

MADE BY K. Inaba DATE 14-6-5 FILE NO

CHECKED BY T.N. DATE 14-6-8 PAGE NO 152

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.
List for Granite continued.
Curb Stone
Arch Span.



Arrangement of Curb Stones.

Name	Section	Length	Volume	No.	Total Volume	Remarks
End Span						2 required
Curb Stone	0.4×0.5'	5.5'	1.10 ft	52	57.2 ft	for 4 rows
" "	0.4×0.5	5.0	1.00	4	4.0	" " "
" "	0.4×0.5	4.5	0.90	2	1.8	" " "
				Total=	63.0 ft	
Intermediate Span						4 required
Curb Stone	0.4×0.5'	5.5'	1.10 ft	48	52.8 ft	for 4 rows
" "	0.4×0.5	4.5	0.90	4	3.6	" " "
				Total=	56.4 ft	

Girder Span.

Name	Section	Length	Volume	No.	Total Volume	Remarks
Girder Span (3 continuous spans)						3 required
Curb Stone	0.4×0.5'	5.5'	1.10 ft	56	61.6 ft	for 4 rows
" "	0.4×0.5	4.0	0.80	4	3.2	" " "
				Total=	64.8 ft	
On Abutment for Girder Span						4 required
Curb stone	0.4×0.5	3.0	0.60	7	4.2 ft	
Stone	0.4×1.0	7.5	3.00	1	3.0	
				Total =	7.20 ft	Revised 14-6-16.

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HYOGO KEN

MADE BY K. Inaba DATE 14-6-5 FILE NO

CHECKED BY T.N. DATE 14-6-8 PAGE NO 153

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

List for Granite continued.

Summary for Granite.

Facing Stone.

Pier for Arch Span	5@	480.56	= 2402.80 ft
Abutment " " "	2@	137.76	= 275.52
		Total Fasing stone	= 2678.32 ft

Coping stone

Pier for Arch Span	5@	87.24	= 436.20
Abutment " "	2@	74.26	= 148.52
		Total Coping stone	= 584.72
		Total granite for Sub structure	= 3263.04 ft

Handrail

End Arch Span	2@	78.24	= 156.48
Int. " "	4@	80.64	= 322.56
Refuge Bay	2@	68.28	= 136.56
			615.60
Girder Span	1@	254.12	= 254.12
Abutment for Girder Span	1@	74.41	= 74.41
			328.53
		Total granite for Handrail	= 944.13 ft

Light Pedestals & End Posts

Pier for Arch Span	10@	46.04	= 460.40
Abutment " " "	8@	26.43	= 211.44
			671.84
Pier for Girder Span	2@	20.10	= 40.20
Abutment " " "	4@	38.52	= 154.08
			194.28
End Post on abutment for girder span	8@	9.56	= 76.48
		Total granite for Light Pedestals and End Posts	= 942.60 ft

Curb Stone

End Arch Span	2@	63.00	= 126.00
Int. " "	4@	56.40	= 225.60
			351.60
Girder Span	3@	64.80	= 194.40 ft
		Total granite for Curb stone	= 546.00
		Total granite for Super structure	= 2432.73 ft

Total granite for the Whole Bridge = 5695.77 ft

Curb stone etc. on abutment 4@7.2 = 28.80 ft

Total granite for the Whole Bridge = 5724.57 ft

Revised 14-6-16

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HYOGO KEN

MADE BY K. Inaba. DATE 14-6-6 FILE NO

CHECKED BY T.N. DATE 14-6-9 PAGE NO 154

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

List of Cement Mortar for Stone facings and Handrail.

Stone facing. 1:2 mortar

Pier for Arch Span. 5 required.

11.0×18.0×0.15×2	=	59.4 ¢	Back of Stone
13.0×1.0×0.03×17×2	=	13.3	Hor. joint
0.95×0.97×0.03×12×17×2	=	11.3	Ver. "
4.4×2.4×⅔×0.03×2	=	0.4	for stone Y
4.4×2.0×⅔×0.05×2	=	0.6	
Total	=	85.0 ¢	

Abutment for Arch Span. 2 required

18.1×7.5×0.1×2.	=	27.2 ¢	Back of stone
19.5×0.6×0.04×7×2	=	6.6	Hor. jt.
0.6×0.96×0.03×19×2	=	0.7	¢ Ver. jt.
Total	=	34.5	

Handrail 1:2 mortar

Arch Span, one span 6 required

24×(0.4×0.6+0.4×0.4×2)×0.03	=	0.4 ¢	Bottom rails bottom
1.0×1.0×0.03×10.0	=	0.3	Panel post.
0.6×0.7×0.03×12×2	=	0.3	Int. " Top & bott.
0.03×0.08×12.8×24	=	0.7	Casting
0.7×0.5×0.03×26	=	0.3	Top rail
0.6×0.7×0.03×24×2	=	0.6	Bott. rail & Panel post
		2.6 ¢	
4.5×2.4×0.02	=	0.4 ¢	Light pedestal, cap (joint 0.02 thick)
4.5×2.4×0.04	=	0.4	" " base "
4.8×2.7×0.04	=	0.5	" " "
0.6×4.8×0.04	=	0.1	" " "
		1.4 ¢	
	=	4.0 ¢/span	

Girder Span 9 required

0×0.4×0.6+0.4×0.4×2)×0.03	=	0.1 ¢	Bottom rail bottom
1.0×1.0×0.03×4	=	0.1	Panel post
0.6×0.7×0.03×4×2	=	0.1	Int. post
0.03×0.08×153×8	=	0.3	Casting
0.7×0.5×0.03×12	=	0.1	Top rail
0.6×0.7×0.03×16	=	0.2	Bott. rail & Panel post
Total	=	0.9 ¢	

Refuge Bay, for one abutment 2 required

0.8×17×0.03×2	=	0.8 ¢	Bottom rail
0.5×0.8×0.03×8.1	=	0.2	" "
0.7×0.5×0.03×8×2	=	0.2	Top "
0.25×0.6×0.03×2×2×4	=	0.9	Post P1
(1.0×1.8+0.6×1.6)×0.03×4×2	=	0.7	" P2
		2.8 ¢	
2.7×2.2×0.03×4	=	0.7 ¢	Light Pedestal Cap
2.7×2.2×0.03×4	=	0.7	" " base
3.0×2.5×0.03×4	=	0.9	" " "
		2.3 ¢	
Total	=	5.1 ¢	

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HYOGO KEN

MADE BY K. Inaba DATE 14-6-8 FILE NO.

CHECKED BY T.N. DATE 14-6-10 PAGE NO. 155

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

Cement Mortar continued.

Handrail on abutment for girder span (1:2 mortar) 1 required

(0.8×0.03)(5.2+4.2)×2	=	0.5 ft	Bott. rail	West abutment
0.8×0.5×0.03×8	=	0.1	"	
0.6×0.5×0.03×4×18	=	0.6	Post P3	
0.6×(1.0+0.7)×0.03×2	=	0.1	" P4	
0.7×0.5×0.03×6	=	0.1	Top rail	
(0.8×0.03)(5.2×2+4.1)	=	0.3	Bott. rail	East abutment
0.8×0.5×0.03×6	=	0.1	"	
0.6×0.5×0.03×4×14	=	0.5	Post	
0.6×(1.0+0.7)×0.03	=	0.0	"	
0.7×0.5×0.03×5	=	0.1	Top rail	
		2.4 ft		
1.5×1.5×0.03×8	=	0.5 ft	End post Cap & base	
1.8×1.8×0.03×8	=	0.8	" base	
2.7×2.7×0.03×4×2	=	1.7	Light Pedestal Cap&base	
3.0×3.0×0.03×4	=	1.1	" " base	
		4.1 ft		
Total	=	6.5 ft		

Summary for 1:2 mortar

Stone facing	Pier for arch span	5@	8.50 = 42.50 ft	
	Abutment " "	2@	3.45 = 6.90	
			49.40 ft	
Handrail	Arch span	6@	4.0 ft = 24.0 ft	
	Girder Span	9@	0.9 = 8.1	
	Refuge bay	2@	5.1 = 10.2	
	Abutment for arch span	1@	6.5 = 6.5	
			48.8 ft	
Total 1:2 mortar	=		542.8 ft	

Cement Mortar (1:3) for Curb Stone

(0.4+0.5)×0.03×582.5×4	=	73.7 ft
0.4×1.25×0.03×504	=	13.0
76.7×2.0	=	153.4 ft

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HYOGO KEN

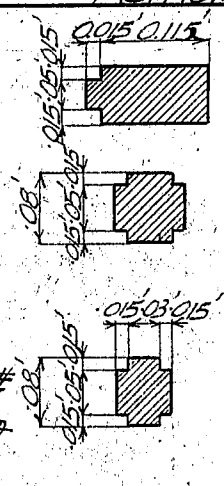
MADE BY K Inaba DATE 14-6-5 FILE NO

CHECKED BY T.N. DATE 14-6-10 PAGE NO 156

CALCULATIONS FOR
Muko-O-Hashi over the Mukogawa.
List for Casting of Handrail

Casting Ca (Ca1, Ca2 & Ca3) 48 Ca1, 48 Ca2, 44 Ca3 required (Total 140)

Name	Section	Length	Volume	No.	Total volume	Remarks
Top and Bott.	0.00995□'	4.80'	0.04776	2	0.09552	
Sides	0.00995	1.34	0.01333	2	0.02666	
Vertical	0.0055	0.99	0.00545	18	0.09810	
"	0.0055	0.27	0.00149	12	0.01788	
Horizontal	0.0055	4.54	0.02497	1	0.02497	
Diagonal	0.0039	0.35	0.00137	7	0.00959	
"	0.0039	0.15	0.00059	14	0.00826	
			Total	=	0.28098	



Sectional area
= 0.115×0.08 + 0.015×0.05
= 0.00995□'

Sectional area
= 0.08×0.05 + 0.05×0.03
= 0.00055□'

Sectional area
= 0.08×0.03 + 0.05×0.03
= 0.00039□'

Weight of Casting Ca = 0.28098×450 = 126.44#
= 15.30貫

Casting Cb 4 required

Name	Section	Length	Volume	No.	Total volume	Remarks
Top and Bott.	0.00995□'	5.50'	0.05473	2	0.10946	
Sides	0.00995	1.34	0.01333	2	0.02666	
Vertical	0.0055	0.99	0.00545	21	0.11445	
"	0.0055	0.27	0.00149	14	0.02086	
Horizontal	0.0055	5.24	0.02882	1	0.02882	
Diagonal	0.0039	0.35	0.00137	8	0.01096	
"	0.0039	0.15	0.00059	16	0.00944	
			Total	=	0.32065×450 = 144.29#	
					= 17.46貫	

Casting Cc 60 required

Name	Section	Length	Volume	No.	Total volume	Remarks
Top and Bott.	0.00995□'	6.05'	0.06020	2	0.12040	
Sides	0.00995	1.34	0.01333	2	0.02666	
Vertical	0.0055	0.99	0.00545	24	0.13080	
"	0.0055	0.27	0.00149	16	0.02384	
Horizontal	0.0055	5.79	0.03185	1	0.03185	
Diagonal	0.0039	0.36	0.00140	9	0.01260	
"	0.0039	0.15	0.00059	18	0.01062	
			Total =		0.35677×450 = 160.55#	
					= 19.43貫	

Casting Cd 4 required

Name	Section	Length	Volume	No.	Total volume	Remarks
Top and Bott.	0.00995□'	5.45'	0.05423	2	0.10846	
Sides	0.00995	1.34	0.01333	2	0.02666	
Vertical	0.0055	0.99	0.00545	21	0.11445	
"	0.0055	0.27	0.00149	14	0.02086	
Horizontal	0.0055	5.19	0.02855	1	0.02855	
Diagonal	0.0039	0.35	0.00137	8	0.01096	
"	0.0039	0.15	0.00059	16	0.00944	
			Total =		0.31938×450 = 143.72#	
					= 17.39貫	

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MADE BY K. Inaba DATE 14-6-6 FILE NO

CHECKED BY T.N DATE 14-6-10 PAGE NO 157

CALCULATIONS FOR

Muko-O-Ha[illegible] over the Mukogawa.

Casting [illegible] Handrail continued.

Casting Ca. 8 required

Name	Section	Length	Volume	NO	Total Volume	Remarks
Top and Bott.	0.0945	5.70	0.05672	2	0.11344	
Sides	0.0945	1.34	0.01333	2	0.02666	
Vertical	0.0055	0.99	0.00545	23	0.12535	
"	0.0055	0.27	0.00149	15	0.02235	
Horizontal	0.0055	5.44	0.02992	1	0.02992	
Diagonal	0.0039	0.35	0.00137	8	0.01096	
"	0.0039	0.15	0.00059	16	0.00944	
			Total	=	0.33812 ×450 = 152.15	
					= 1.841	

Summary of Castings for Handrail.

Casting Ca.	Ca1	48@	1530	= 73440
"	Ca2	48@	1530	= 73440
"	Ca3	44@	1530	= 67320
				214200
"	Cb	4@	1746	= 6984
"	Cc	60@	1943	= 116580
"	Cd	4@	1739	= 6956
"	Ce	8@	1841	= 14728

Total No. = 216 Total casting for the Whole Bridge = 359448 kgs.
= 13.264 tons.

Painted Area = 95.56 ㎡ for 216 castings.

List of Lead Dovetails for Handrail

Weight of dovetail = $\frac{0.12+0.2}{2} \times 0.4 \times 0.1 \times 706^{\#} = 0.0064 \times 706 = 4.52^{\#}$
= 0.55 kg

Required number:

Arch Span	6@12	= 72
Gider Span	9@4	= 36
West Abutment		2
East		1
		111 dovetails required

Total wt. of dovetails required for the Whole Bridge
= 111@0.55
= 61.05 kg

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HYOGO KEN

MADE BY K. Inaba DATE 14-6-6 FILE NO

CHECKED BY T.N. DATE 14-6-10 PAGE NO 158

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.
Expansion Shoes for Girders.
Shoes for girders under Electric Railway.

Shoes Sr for girders under Electric Ry. 3 required.

Name	Section	Length	Wt./ft.	Wt./piece	No.	Total wt.	Remarks
H-beam	5"×18.9#	2'-0"	18.90#	396.9#	1	396.9#	
Angles	4"×4"×1/4"	1'-5 1/8"	6.60	6.4	8	51.2	
Stay	2 1/2"×3/8"	3'-3"	3.19	10.4	4	41.6	
Sole plate	11 5/8"×1/2"	1'-3 1/2"	19.77	25.5	4	102.0	
Bed "	12 3/8"×1 1/2"	1'-3 1/2"	21.04	27.1	4	108.4	
Bolts	7/8" φ	1.9'		4.48	17	76.2	with 2 nuts
Rivets	5/8" φ	grip 7/8"		0.28	10	2.8	
"	5/8" φ	" 5/8"		0.26	6	1.6	
				Total =		780.7#	

Shoes St for girders under Traffic way 6 required.

Name	Section	Length	Wt./ft.	Wt./piece	No.	Total wt.	Remarks
H-beam	5"×18.9#	1'-9"	18.90#	340.2#	1	340.2#	
Angles	4"×4"×1/4"	1'-5 1/8"	6.60	6.4	8	51.2	
Stay	2 1/2"×3/8"	3'-3"	3.19	10.4	4	41.6	
Sole plate	11 5/8"×1/2"	1'-3 1/2"	19.77	20.6	4	82.4	
Bed "	12 3/8"×1 1/2"	1'-0 1/2"	21.04	21.9	4	87.6	
Bolts	7/8" φ	1.9'		4.48	15	67.2	
Rivets	5/8" φ	grip 7/8"		0.28	10	2.8	
"	5/8" φ	" 5/8"		0.26	6	1.6	
				Total =		674.6#	

Shoes Sf for Fascia Girders 6 required

Name	Section	Length	Wt./ft.	Wt./piece	No.	Total wt.	Remarks
H-beam	5"×18.9#	2'-6"	18.90#	47.3#	1	47.3#	
Angles	4"×4"×1/4"	1'-1 5/8"	6.10	6.9	2	13.8	
Stay	2 1/2"×3/8"	3'-0"	3.19	9.6	1	9.6	
Conn. plate	10"×3/8"	1'-1 3/8"	12.75	14.2	1	14.2	
Sole "	12 1/2"×1/2"	1'-2 5/8"	21.25	25.4	1	25.4	
Bed "	12 1/2"×1 1/2"	1'-4 5/8"	21.25	29.3	1	29.3	
Bolts	7/8" φ	2.5'		5.68	3	17.0	
Rivets	5/8" φ	grip 7/8"		0.28	10	2.8	
"	5/8" φ	" 5/8"		0.26	6	1.6	
"	5/8" φ	" 3/4"		0.27	2	0.5	
				Total		162.0#	

Summary of Shoes.

Sr 3@ 780.7# = 2,342.1#
St 6@ 674.6 = 4,047.6
Sf 6@ 162.0 = 972.0
7,361.7# = 3.286 tons.

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HYOGO KEN

MADE BY K. Inaba DATE 14-6-6 FILE NO

CHECKED BY T.N. DATE 14-6-10 PAGE NO 159

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

List of I-beam for Pier for Girder Span.

I-beam 8"x17.5#x24.0' Weight for one piece = 17.5x24 = 420#

7 beams required for 7 piers.

Total wt. req'd for the whole bridge = 7@420# = 2,940#

= 1.313 tons.

List of Angles for the Support of Gas pipes.

3"x3"x1/4"x4.0' Wt./piece = 4.9x4 = 19.6#

Required number.

Arch Span	End Span	2@	28	= 56
	Int. Span	4@	26	= 104
				160 angles
Girder Span		9@	6	= 54

Total no. req'd = 214 angles

Total wt. of angle req'd. for the whole bridge

= 214@19.6# = 4,194.4#

= 1.873 tons.

List of Expansion Feed. 17 required.

Copper plate	0.3'x7.6'x2	= 4.2 □'@0.12貫	= 0.504貫	
1/8" lead "	0.96x53.7'	= 51.55 □'@0.85 "	= 43.82 "	
Asphalt.		1@ 3.50	= 3.50 "	
Asbestus and the likes.	2@0.75x1.5		= 2.25 □'	

List for Drain Pipes.

Drain at Crown of Arch. 12 required.

Grate 0.6'x0.6'x0.1 Cast iron.

Wt. = {(0.6x0.6x0.1)-(0.05x0.05x25x0.1)}x450# = 13.4#/piece

= 1.62貫/piece.

Pipe D.A. Total area = 0.1x0.69x4 + 0.075x0.6x4 + 0.3x0.54x4

+ (0.54x4 + 0.37xπ) x 1/2 x 0.21 + 0.37π x 2.12

= 3.92 □'

Total vol. of cast iron = 3.92 x 0.04 = 0.157 ℄

Total wt. of D.A. = 0.157x450 = 70.65# = 8.55貫

Total = 10.17貫

Drain for Pier and Abutment (arch span) 14 required

Grate	0.6x0.6x0.1	1@	1.62貫	= 1.62貫	
Casting D.A.		1@	8.55	= 8.55	
Gas pipe	4"φ x 4.62	1@	10.07	= 10.07	18#/ft. for 4"φ pipe
" "	" x 2.00	1@	4.36	= 4.36	
" "	" x 3.20	1@	6.97	= 6.97	15° Bend
" "	" x 8.00	1@	17.42	= 17.42	
" "	" x 1.90	1@	4.14	= 4.14	60° Bend
" "	" x 1.60	1@	3.48	= 3.48	75° "
Flange bolt.	1/2"φ x 0.25	12@	0.44	= .528	
				Total 61.89貫	
Lead for joint		4 joints@	0.85	= 3.40"	

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CHECKED BY T.O. DATE 14-6-10 PAGE NO 60

ATIONS FOR

...-O-Hashi over the Mukogawa.

Drain for Girder Span.

22 required.

Grate 0.6'x0.6'x0.1'	1@	1.62 lb	= 1.62 lb
Casting D.G.	1@	4.48	= 4.48
		Total	= 6.10 lb

Summary for Drain Pipes.

		Casting	Lead for jt.
Arch span			
at crown	12@	10.17 lb	= 122.04 lb
Pier and abutment	14@	61.84	= 866.76 14@ 3.4 = 47.60 lb
Girder span	22@	6.10	= 134.20
Total			= 1,122.70 lb Total = 47.60 lb

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HYOGO KEN

MADE BY K. Inaba. DATE 14-6-8 FILE NO

CHECKED BY T.D. DATE 14-6-10 PAGE NO 161

CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

Excavation.

Pier for Arch Span

Area of foundation bed = 15×81.82' = 1,473 □'

Pier AP1	1,473 × 16.3' =	24,010 ㎥	= 111.2	±千
〃 AP2	1,473 × 16.0 =	23,568	= 109.1	
〃 AP3	1,473 × 15.6 =	22,974	= 106.4	
〃 AP4	1,473 × 14.2 =	20,917	= 96.8	
〃 AP5	1,473 × 16.3 =	24,010	= 111.2	
	Total excavation for Pier		= 534.7	±千

Abutment for Arch Span

Area of foundation bed = 26' × 67' = 1,742 □'

Abutment AA1	1,742 □' × 17.6' =	30,659 ㎥	= 141.9	±千
〃 AA2	1,742 × 20.2 =	35,188	= 162.9	
	Total		= 304.8	±千

Pier for Girder Span.

Area of foundation bed = 10' × 74' = 740 □'

Pier GP1	740 □' × 12.5' =	9,250 ㎥	= 42.8	±千
GP2	740 × 11.5 =	8,510	= 39.4	
GP3	740 × 15.6 =	11,544	= 53.4	
GP4	740 × 14.5 =	10,730	= 49.7	
GP5	740 × 14.0 =	10,360	= 48.0	
GP6	740 × 17.0 =	12,580	= 58.2	
GP7	740 × 18.0 =	13,320	= 61.7	
	Total		= 353.2	±千

Abutment for Girder Span.

Area of foundation bed for West abutment = 88.14×14 + 9×8.5×2 = 1387 □'

〃 〃 〃 〃 〃 East 〃 = 88.14×14 + 9×8.5 + 9×3 = 1337

West abutment	1,387 □' × 19.0 =	26,353 ㎥	= 122.0	±千
East 〃	1,337 × 18.2 =	24,333	= 112.7	
	Total		= 334.7	±千

増田淳氏関係資料
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CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.

List of Foundation Piles.

Arch Span

Japanese Pine Pile 0.7'$^{\phi}$ at Tip 18' long for Arch Span.

Arch Span	Pier	5@	162	= 810 piles
"	"	Abutment	2@	134 = 268
			Total	= 1078 piles.

Girder Span

Japanese Pine Pile 0.6'$^{\phi}$ at Tip 14' long for Girder Span.

Girder Span	Pier		7@	96 = 672 piles.
"	West	abutment	1@	124 = 124 "
"	East	"	1@	118 = 118 "
			Total	= 914 piles

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CALCULATIONS FOR

Muko-O-Hashi over the Mukogawa.
List of Centering for Arch Ring.

Centering for Arch Span required Sheet No. 18.

Wood members

Name	Section	Length	Volume	No.	Total vol.	Kind of wood	Remarks
Lagging	0.3'×0.3'	15.0'	1.35 ft	912	1,231.2 ft	American pine	
〃	0.3×0.3	10.0	0.90	342	307.8	〃	
〃	1.0×0.9	13.5	12.15	10	121.5	〃	
Segment	1.0×0.6	9.2	5.52	28	154.6	〃	Truss T2
〃	1.0×0.6	8.7	5.22	28	146.2	〃	〃 〃
〃	1.0×0.6	7.8	4.68	28	131.0	〃	〃 T1
〃	1.0×0.6	8.0	4.80	28	134.4	〃	〃 〃
Block	0.9×0.6	4.0	2.16	28	60.5	〃	〃 T2
〃	0.8×0.6	4.0	1.92	28	53.8	〃	〃 T1
〃	1.4×1.3	0.6	1.09	28	30.5	〃	〃 T2
Vertical	0.6×0.6	4.5	1.62	28	45.4	〃	〃 〃
〃	0.6×0.6	8.0	2.88	56	161.3	〃	〃 T1 & T2
〃	0.6×0.6	10.0	3.60	28	100.8	〃	〃 T1
Diagonal	0.6×0.5	4.5	1.35	28	37.8	〃	〃 T2
〃	0.6×0.6	7.7	2.77	28	77.6	〃	〃 〃
〃	0.6×0.6	10.6	3.82	28	107.0	〃	〃 T1
〃	0.6×0.6	11.0	3.96	28	110.9	〃	〃 〃
Brace	0.5×0.3	22.0	3.30	28	82.4	〃	Connection of T1 & T2
Bottom chord	0.8×0.6	17.0	8.16	28	228.5	〃	Truss T2
〃	0.8×0.6	15.2	7.30	28	204.4	〃	〃 T1
Cap	0.6×0.6	16.5	5.94	24	142.6	〃	Cross member
〃	0.6×0.6	14.5	5.22	8	41.8	〃	〃 〃
〃	0.6×0.6	9.5	3.42	8	27.4	〃	〃 〃
Wedge	0.6×0.25	1.5	1.88 t	140	263.2 t	Oak or Chestnut	Under cap
〃	0.5×0.3	1.0	1.25	28	35.0	〃	Between bott. chord & pier
〃	0.7×0.6	1.5	5.25	28	147.0	〃	〃 stringer 〃 〃
Stringer	0.7×0.7	12.0	5.88 ft	28	164.6 ft	American pine	End
〃	0.7×0.7	14.5	7.11	28	199.1	〃	Intermediate
〃	0.7×0.7	17.0	8.33	14	116.6	〃	Center
Block	0.7×0.7	3.0	1.47	70	102.9	〃	on top of pile
Sand Box	0.85'φ	1.5		70			See next page
Pile	0.6'φ at tip	14.0	42.00 t	28	1,176.0 t	Japanese pine	Both ends
〃	0.6' 〃	18.0	54.00	42	2,268.0	〃 〃	Intermediate
〃 〃	0.6' 〃	21.0	63.00	5	315.0	〃 〃	Protection bent
Brace	0.5×0.3	10.0	1.50 ft	26	39.0 ft	American pine	Upper brace at center
〃	0.5×0.3	8.5	1.28	52	66.6	〃 〃	〃 〃 〃 both sides
〃	0.5×0.3	12.0	1.80	60	108.0	〃 〃	Lower brace except right end
〃	0.5×0.3	6.5	0.98	10	9.8	〃 〃	〃 〃 right end
〃	0.5×0.3	16.0	2.40	30	72.0	〃 〃	〃 〃 horizontal
〃	0.5×0.3	11.0	1.65	20	33.0	〃 〃	〃 〃 〃
Edge of Lagging	0.6×0.2	12.0	1.44	24	34.6	〃 〃	
End beam	0.6×0.4	9.0	2.16	16	34.6 t	〃 〃	
Timber	0.4'φ	13.0	17.33 t	10	173.3	Ceder	
〃	0.4'φ	10.0	13.33	12	160.0	〃	
Board	0.1' thick				31.0 坪	Pine	End planes of arch ring
〃	0.15' 〃				7.8 坪	〃	
Mescellaneous	0.15×0.15	12.0	2.25 t	15	33.8 t	〃	
					米松合計 = 4720.2 立方尺		
					樫 〃 = 445.2 t		

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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