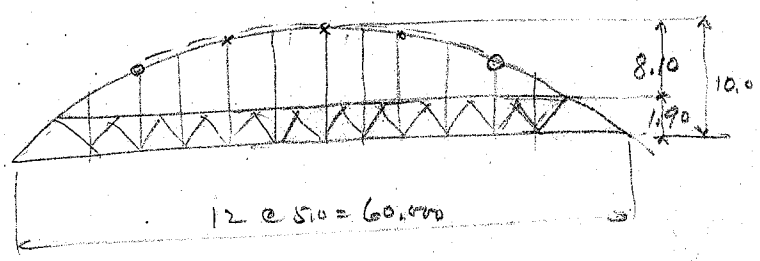
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CALCULATIONS FOR

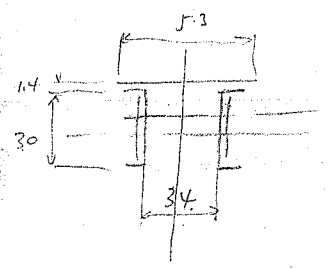
県道六十米メイトアーチ



Steel in one span. 126,390 tons (page 42)

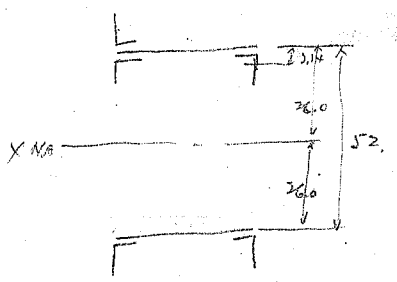
detail 38% (solid rib vertical transposing 横桁)

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1 cover pl.	530×14	= 74.20	
2 ⊏	300×90×9	= 97.14	
2 pls	240×14	= 67.20	
		238.54 kg	860

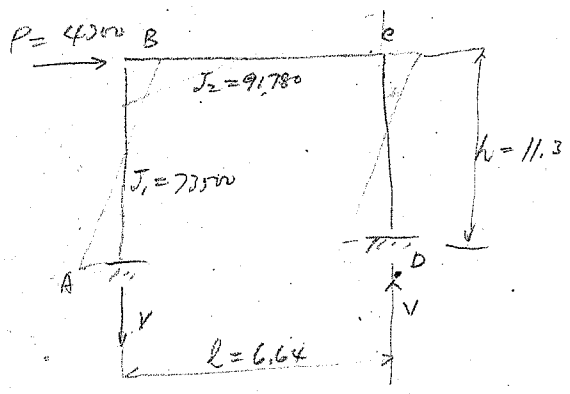
$\Sigma cc = 4.88$



2 ⊏ 90×75×9	= 28.08 × 52	=	1460
1 L "	= 14.04 × 48.86		686
1 web 300×8	= 24.00 × 52		1248
1 web 300×8	= 24.00 × 0		0
2 ⊏ 125×75×9	= 34.38 × (−4.59)		−158
	124.5 26.0		3236

I_x.

2 ⊏ 90·75×9	= 28.08 × 26.0² + 109×2	=	19220
1 L "	= 14.04 × 22.86² + 109	=	7450
2 web 300×8	= 48.00 × 26.0²	=	32450
− 2 ⊏ 125×75×9	= 34.38 × 30.59² + 269×2	=	3266
	124.5	I_x =	91,780 cm⁴

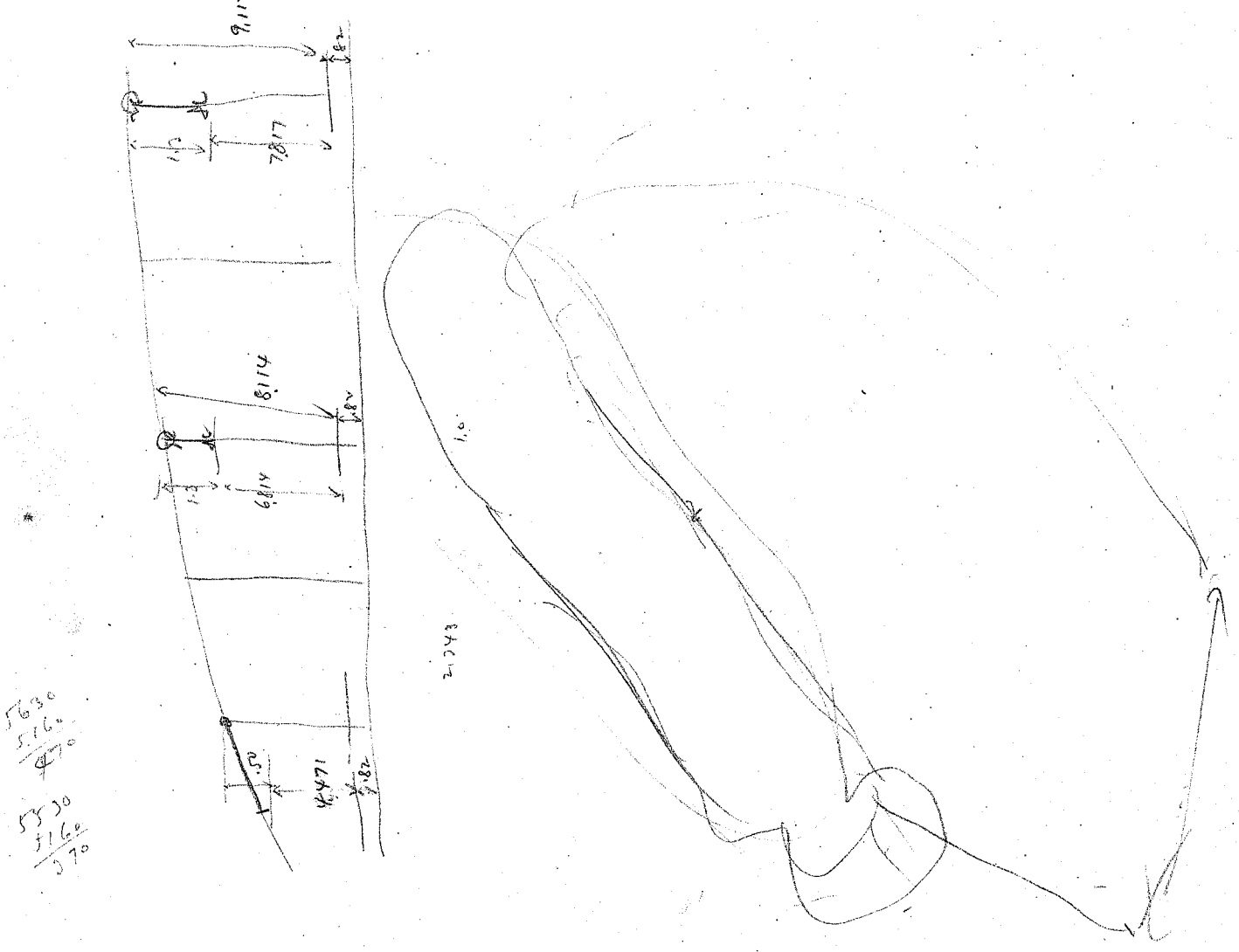


$$k = \frac{J_2}{J_1} \cdot \frac{h}{l} = \frac{91780}{7350} \times \frac{11.3}{6.64} = 2.125$$

3K = 6.38
3K+1 = 7.38
6K+1 = 13.75

$$V = \frac{3Phk}{l(6k+1)} = \frac{6.38 \times 4300 \times 11.3}{6.64 \times 13.75} = \pm 3400 \text{ kg}$$

$$M_A = -\frac{Ph}{2} \cdot \frac{3k+1}{6k+1} = -\frac{4300 \times 11.3}{2} \times \frac{7.38}{13.75} = 13030 \text{ kgm}$$



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Pratt 型 60m Curved warren Truss (Standard)

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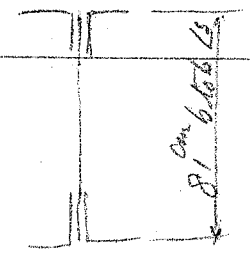
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CALCULATIONS FOR

Summary of Dead and Live Load moments and shears

	moment at S_1	moment at S_2	end shear
Dead load	15,765	11,815	7084
Live load	21,375	15,180	9930
	37,140 kgm	26,995 kgm	16,014 kg

Assumed section



Try web plate 800 × 9 — ½ web area = 9.0 cm² back to back of L° = 81 cm
Effective depth = 81.0 − 2.22×2 = 76.56 cm
flange stress = 37140 / .7656 = 48,500 kg T or C

Bottom flange area reqd = 48500 / 1200 = 40.41
½ web area = 9.00
31.41 cm² net
Use 2Ls 125×90×10 = 41.0 − 4.9 = 36.6 net ok (gross 38.08) 2 – 19φ rivet reduced.

Top flange area required = 48500 / 1100 = 9.410 gr
½ web 9.00
gr ok

Shear on web = 16024 / 72 = 223 kg/cm² ok

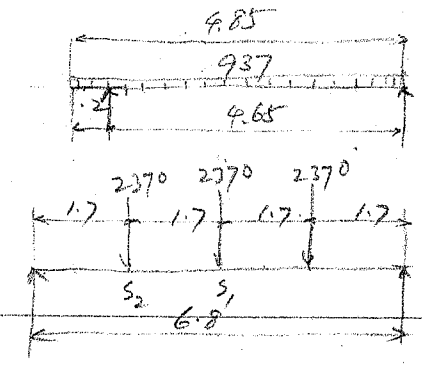
Approximate weight of intermediate floor beam.

web	1 PL	820 000 × 9	@ 56.52 kg	× 6.80	=	380
flange	4 Ls	125×90×10	@ 16.09	× 6.66	=	429
End conn.	2 Ls	125×90×11	@ 16.09	× 0.62	=	20
stiff.	14 Ls	100×75×9	@ 12.95	× 0.81	=	182
		Rivet heads and variations say		3½%	=	36
						1014 kg

weight of beam per line meter = 1010 / 6.8 = 149 kg/lin m ok

Design of End floor Beam. Span length = 6.80 meters, spacing 4.65 m

Dead Live :— Stringer concentration on floor beam.
Dead Load on stringer = 937 kg per lin. m. see page 4
937 × 4.85² / (2 × 4.65) = 2370 kg



Reaction 2370 × 1.5 = 3560 kg = end shear
moment at S_1 3560 × 3.4 = 12,100
2370 × 1.7 = − 4030
8070 kgm
moment at S_2 3560 × 1.70 = 6050 kgm

Dead load due to own wt of floor beam assumed 140 kg/m
moment at S_1 = ⅛ × 140 × 6.8² = 808 kgm
End shear = ½ × 140 × 6.8 = 476 kg
moment at S_2 140 × 1.7² / 2 = − 203
476 × 1.7 = 808
605 kgm

Summary of Dead load moments & shears

	moment at S_1	moment at S_2	end shear
Stringer concentration	8070	6050	3560
weight of beam	808	605	476
	8,878 kgm	6,655 kgm	4036 kg

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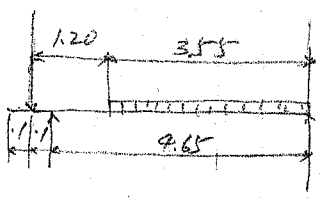
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CALCULATIONS FOR

Live Load :— Concentration on stringer due to motor truck wheels.

Rear wheel on S_1 $5{,}740 \times \frac{4.75}{4.65} = 5860$ kg S_2 $3670 \times \frac{4.75}{4.65} = 3{,}750$ kg

front wheel " $1910 \times 4 = 1{,}950$ " " $1220 \times 4 = 1{,}250$ "



Moment due to motor truck rear wheel.

Reaction $5860 \div 2 = 2930$

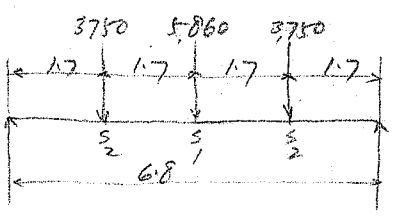
3750

6680 kg end shear

Moment at S_1 $6680 \times 3.4 = 22{,}700$

$3750 \times 1.7 = -\ 6910$

$15{,}790$ kgm



Moment at S_2 $6680 \times 1.7 = 11{,}400$ kgm

Uniform load

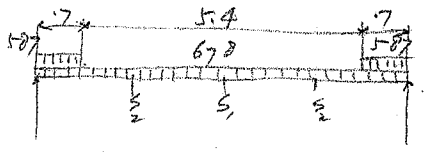
rear of motor truck

$\frac{400 \times 3.55^2}{2 \times 4.65} = 678 = w_1$

side of motor truck

$\frac{400 \times 4.85^2}{2 \times 4.65} = 1265$

diff $= 587 = w_2$

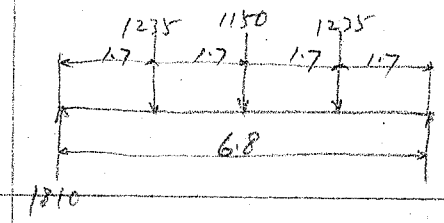


Stringer concentration $S_1 = 678 \times 1.7 = 1150$ kg

$S_2 = \frac{587 \times 1.7^2}{2 \times 1.7} = 85$

1150

1235 kg



Reaction $= \frac{1150}{2} + 1235 = 1810$ kg = end shear

Moment at $S_1 = 1810 \times 3.4 = 6150$

$1235 \times 1.7 = -\ 2100$

4050 kgm

" " $S_2 = 1810 \times 1.7 = 3080$ "

Summary of Live load moment

Rear wheel of truck	15,790	11,400	6680
unif. load	4050	3080	1810
	19,840 kgm	14,480 kgm	8490 kg

Summary for Dead and Live load moments and shears

Dead load	S_1	8,878	6,655	4036
Live load		19840	14480	8490
		28,718 kgm	21,135 kgm	12,526 kg

Assumed section.

Try web 800×9 $\frac{1}{8}$ web $= 9.0$ cm^2 b to b $\ell s = 81.0$ cm

flange stress $= \frac{28718}{0.761} = 37{,}750$ kg T or C

Bottom flange area reqd. $= \frac{37750}{1200} = 31.5$ cm^2 net

$\frac{1}{8}$ web area $= 9.0$

use 2 ℓs $\frac{125}{60} \times \frac{75}{40} \times \frac{9}{20}$ $= 3938 - 296 = 3642$ $= 31.6$ cm^2 ok 2-19ϕ rivet

top flange area reqd $= \frac{37250}{1100} = 343$

90

253 gr ok

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CALCULATIONS FOR

Approximate weight of end floor beam.

web	1 pc.	800 × 9	@ 56.52	× 6.80	=	384
flange	4 cs	100 × 25 × 10	@ 19.73	× 6.66	=	370
end conn	2 cs	125 × 90 × 10	@ 16.09	× 0.62	=	20
stiffners	16 cs	75 × 75 × 9	@ 9.96	× 0.81	=	113
		Rivet heads and variation say 3½ %			=	33
						920 kg

Weight per lin meter of beam = 920/6.8 = 135 kg ok

Design of Bottom Lateral Bearings

Dead load.

metal assumed

Stringers	3 @ 62.0	= 186
floor beam	1016 + 965	= 218
lateral bracing	assumed	= 65
main girder	assumed	= 850
		1319

Floor slab + pavement between girder

+15 × 6.8 = 3400

Over hanging slab coping with handrail = 920

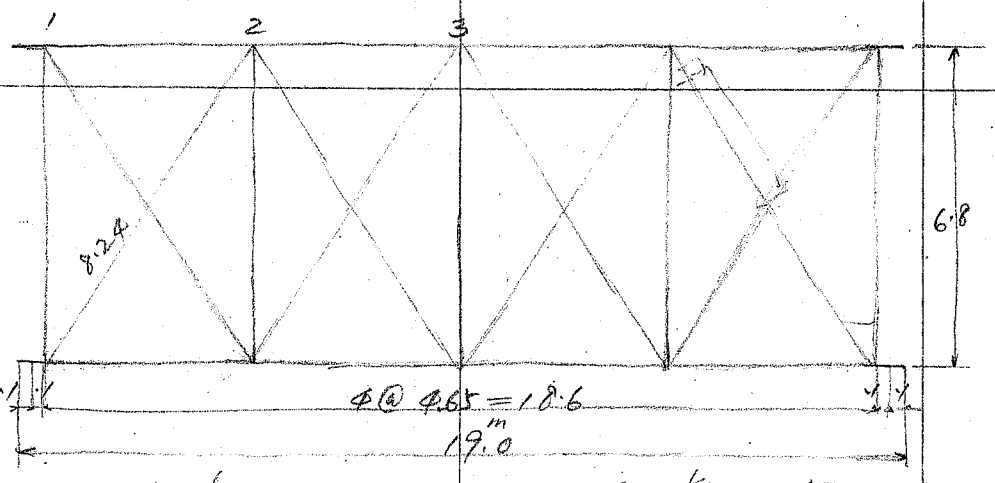
4420

5,739 kg per lin m for two girders

panel load = 5,739 × 9.65 = 26,680 kg

Seismic panel load = 26,680 × .2 = 5,340. kg

wind panel load = 600 × 9.65 = 2780 kg



Diagonal length

9.65^2 = 21.65

6.80^2 = 46.25

67.90

$\sqrt{67.90}$ = 8.24

Coeff. = 8.24/6.8 = 1.21

End shear = 5340 × 1.5 = 8010 kg stress 1-2 8010 × 1.21 = 9,690. kg T SR. = 5.09 cm² net

Shear 2-3 = 5340 × .5 = 2,670 " 2-3 2,670 × 1.21 = 3230 " " = 1.68 "

Max no of 22Φ rivets = 9690/2,851 × 1.6 = 2.1

Use 1L 100 × 100 × 10 = 19.0 − 2.5 = 16.5 cm² net OK

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CALCULATIONS FOR

Approximate weight of Lateral bracings

Diagonal	8 Ls	160 × 100 × 10 ~~150 × 90 × 9~~	@	~~16.32~~ 16.6	× 7.70	=	~~1255~~ 1280
Long Ls (end panel)	8 Ls	〃	@	〃	× 1.30	=	39
Center conn.	4 Pls	290 × 9	@	16.96	× 1.70	average =	48
Conn. to girder	6 〃	500 × 9	@	35.33	× 1.70	=	199
〃	4 〃	500 × 9	@	〃	× 1.50	=	71
		Rivet heads & variation say 15% add				=	18
							~~1330~~ 1630 kg

weight of lateral bracing per lin.m = ~~1330~~ 1630 / 1816 = ~~74.6~~ 91.5 kg/m. of span

Design of main girders.

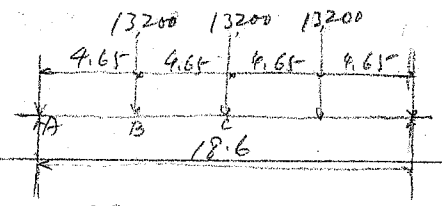
Span length = 19.60 c to c bearings 20 meters out to out.

Dead load.

Floor slab & pavement between main girders	515 × 6.8 =	3500
Handrails, copings and floor beyond girder	2 @ 460 =	920
		4420
Stringers 3 @ 620	=	186
floor beams 1,014 ÷ 4.65	=	218
lateral bracings	=	68
Main girder assumed	=	770
		1262
		5,662 kg/lin m

Intermediate panel load = $\frac{5662 \times 9.65}{2}$ = 13,200 kg for one girder

end 〃 〃 = $\frac{5662 \times 2.93}{2}$ = 6,900 〃



Reaction = 13200 × 1.5 = 19800 = end shear

moment at B 19,800 × 4.65 = 92,000 kgm

moment at C 19,800 × 9.3 = +184,000
13,200 × 4.65 = −61,400
122,600 kgm

Dead load on shoe 2,831 × 9.5 = 26900
weight of shoe say 150
27,050 kg

Live load

Uniform live load = 400 kg per sq meter

1 motor truck concentration 3000

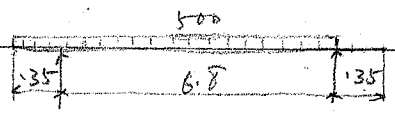
Impact $\frac{20}{60+186}$ = 25.5% 760

3760 kg rear wheel

front wheel with impact $\frac{3760}{3}$ = 1,250 kg

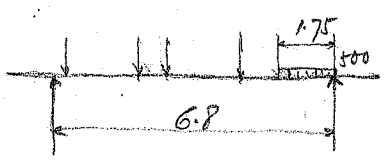
uniform live load on front & rear of truck assumed uniformly distributed over main girder

$\frac{500 \times 7.15^2}{2 \times 6.8}$ = 1880 kg/m



unif. load on side of truck

$= \frac{500 \times 1.75^2}{2 \times 6.8}$ = 110 kg/m = M_1



1880 − 110 = 1770 〃 = M_2

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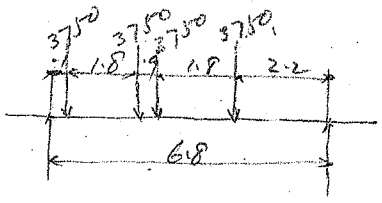
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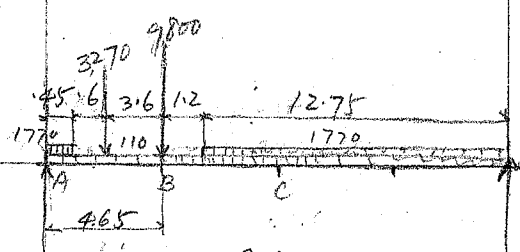
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CALCULATIONS FOR

Motor truck wheel concentration on main girder assumed as follows.



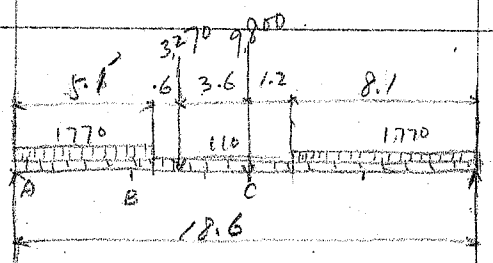
Reaction
3,750 × 6.70 = 25,120
〃 × 4.90 = 18,390
〃 × 4.00 = 15000
〃 × 2.20 = 8250
66760 ÷ 6.8 = 9800 kg rear wheel.
9800 ÷ 3 = 3270 kg front wheel.



19,720

Moment at B
Reaction 9800 × 3/4 = 7350
3270 × 17.55/18.6 = 3090
1770 × 12.75²/2×18.6 = 7,790
1.770 × 1.45 × 16.375/18.6 = 520
110 × 9.3 = 1,020
19,720 kg

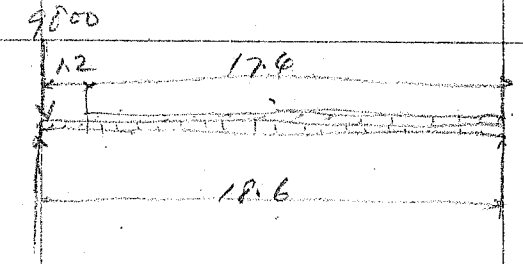
Moment 19,720 × 4.65 = 91,700
3,270 × 3.6 = − 11,800
110 × 4.65²÷2 = − 1,190
1770 × 1.45 × 4.93 = − 3560
75,150 kg.m



19,100

Moment at C
Reaction 9800 × 1/2 = 4900
3,270 × 12.9 ÷ 18.6 = 2,270
1,770 × 8.1²/2×18.6 = 3120
1,770 × 5.1 × 16.05/18.6 = 7,790
110 × 9.3 = 1,020
19,100 kg

Moment 19,100 × 9.3 = 177,500
3,270 × 3.6 = − 11,800
1770 × 5.1 × 6.75 = − 60,900
110 × 9.3²÷2 = − 2,900
100,000 kg.m



Max. End Shear
1770 × 17.9²/2×18.6 = 14,400
110 × 9.3 = 1,020
9,200
25,220 kg.

Summary of Dead & Live Load moments and end shear

	B	C	end shear	Load on shear
Dead Load	92,000	122,600	19,800	27,050
Live Load	75,150	100,000	25,220	25,600
	167,150 kgm	222,600 kgm	45,020 kg	52,650 kg

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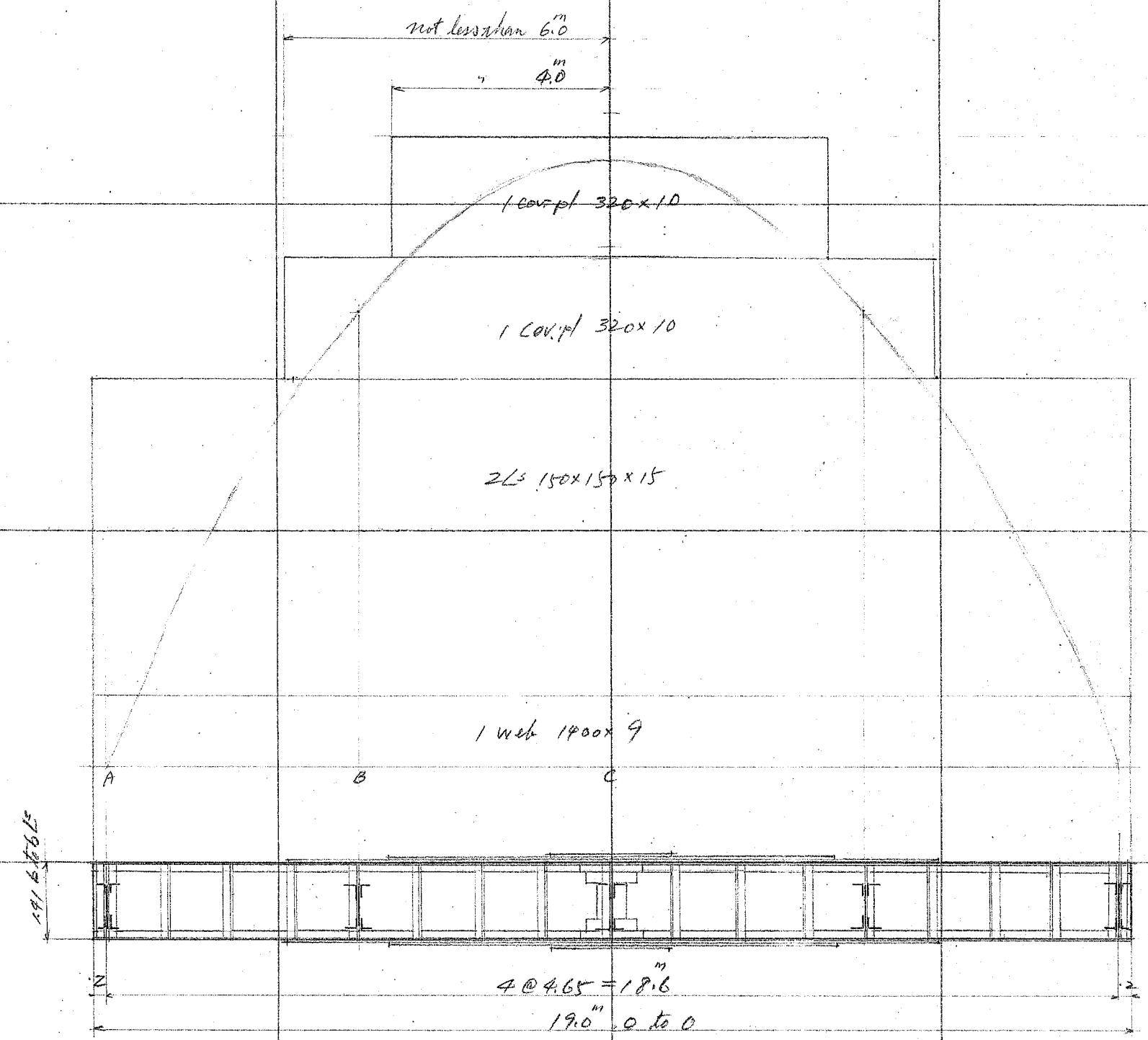
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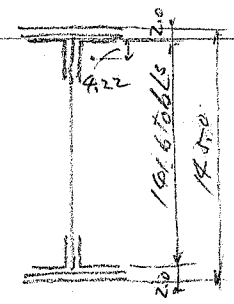
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CALCULATIONS FOR

Bending moment diagram for main girder
scale of span 1:100
scale of moment $\frac{1}{20}^{m}$ = 100,000 kg m



Section of main girder.



2 pl 32×1. = 64 ×1. = 64.
855 × 622 = 532
149.5 199 596

Try web pl. 1,400 × 9 = 126. ∴ web area = 15.8 cm² b.to.6L's 140.6 cm
effective depth say 145.0 − 3.99×2 = 137.0 cm
Flange stress = $\frac{22,600}{137.0}$ = 162.50 kg/cm²

Bottom flange area required = $\frac{162,500}{1200}$ = 135.50
∴ web area = 15.80
119.70 cm² net

Use 2L's 150×150×15 = 85.50 − 15.0 = 70.50
2 pl's 320×10 = $\frac{64.00}{199.50}$ − 10.00 = 54.00
124.50 cm² net ok

Top flange area req'd = $\frac{162,500}{11,000}$ = 198.00
∴ web 15.80
132.20 qr ok

unit shear on web = $\frac{45,020}{126.}$ = 357. kg/cm² ok

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CALCULATIONS FOR

Approximate weight of main girder.

Flange	4Pls	150 × 150 × 15	@	33.55	×	19.00	= 2,550
" Cov.pls	2pls	320 × 10	@	25.12	×	8.00	= 402
" "	2pls	"	@	"	×	12.00	= 603
web	1pls	1400 × 9	@	98.91	×	19.00	= 1879
end stiff	8Pls	125 × 90 × 10	@	16.09	×	1.38	= 178
fills	4pls	180 × 15	@	21.20	×	1.10	= 93
int. stiff	30Pls	125 × 90 × 10	@	16.09	×	1.40	= 676
Comm. Pls	5pls	150 × 100 × 9	@	17.02	×	0.60	= 51
Center splice	4Pls	150 × 150 × 15	@	33.55	×	1.20	= 161
" Cov.pls	2pls	320 × 10	@	25.12	×	1.10	= 61
" web.	4pls	220 × 12	@	20.72	×	0.65	= 56
" "	2pls	330 × 12	@	31.09	×	0.66	= 41
Fillers	2pls	75 × 18	@	10.60	×	1.15	= 29

Rivets heads and variation say 3½% = 237

7,010 Kg

Call this

7.010 ÷ 19.0 = 370 Kg/per 1m of span

Structural steel in one span

Stringer	19.0	@ 62. ×	3	= 3,534
Intermediat floor beam		3@	1,014.	= 3,042
End floor beams		2@	920	= 1840
Lateral bracings				= 1,330
main girder		2@	7,010	= 14,020
Expansion joint	say			= 760
Cast iron shoes	"			= 600
				25,126 Kg

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（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

国道 19 METER PLATE GIRDER SPAN

GIRDER G1L ~ G2L

							4 Reqd	
4	F/g ls.	150×150×15	✓	9.500	✓	33.55 ✓	1,274.9 ✓	
1	Web p/	1400 × 9.	✓	"	✓	98.91 ✓	939.6 ✓	
2	Cov. p/s.	320 × 10	✓	5920.	✓	25.12 ✓	297.4 ✓	
2	" "	"	✓	3450	✓	" ✓	173.3 ✓	
4	ls	125×90×13	✓	1380	✓	20.61 ✓		113.8 ✓
2	Filler	190× 15	✓	1.105	✓	22.373 ✓		49.4 ✓
14	ls.	125×90×9	✓	1.410.	✓	14.00 ✓		288.2 ✓
1	L	125×90×10	✓	240	✓	~~14.130~~ ✓	14.6	~~34~~ ✓ 3.5
1	"	"	✓	230	✓	" ✓	"	~~32~~ ✓ 3.4
1	"	"	✓	310	✓	" ✓	"	~~44~~ ✓ 4.5
1	Sole p/.	405 × 28	✓	465	✓	89.019 ✓		414 ✓ 3,109.9

2,685.2 ✓ + ~~5035~~ 5094.2 = 3,189.0 kg × 4 ✓ = ~~12,756.0~~ 12,775.6 kg

SPLICE AT CENTER — 2 Reqd

2	Spl p/s	320×10	✓	1840	✓	25.12 ✓	92.4 ✓	
2	" "	"	✓	1080	✓	" ✓	54.3 ✓	
4	ls	150×150×19	✓	1.080.	✓	41.91. ✓	181.1. ✓	
4	p/s.	235 ×12	✓	660	✓	22.137 ✓	58.4 ✓	
2	"	360 × 12	✓	630	✓	33.912 ✓	42.7 ✓	
2	ls	125×90×9	✓	1.342	✓	14.60 ✓	39.2 ✓	
2	Fills	90 × 22	✓	1.105	✓	15.543 ✓	34.4 ✓	
1	L	125×90×10	✓	230	✓	~~14.13~~ ✓	14.6	~~32~~ ✓ 3.4
1	"	"	✓	310	✓	" ✓	"	~~44~~ ✓ 4.5

~~510.1~~ 510.4 ✓ × 2 ✓ = ~~1,020.2~~ 1,020.8 kg

Summary of girders = ~~13,776.2~~ 13,778.4 kg

FLOOR BEAM FB1 — 2 Reqd

4	Flg ls	125×90×9	✓	6.500	✓	~~14.60~~ ✓	13.5	~~379.0~~ ✓ 351.0
1	Web p/	820 × 9	✓	6.740	✓	57.933 ✓		390.5 ✓
14	ls	90×75×9	✓	830	✓	11.02 ✓		128.1 ✓
2	Fills	120 × 9	✓	1.380	✓	8.478 ✓		23.4 ✓
2	"	"	✓	310	✓	" ✓		5.3 ✓
2	"	"	✓	240	✓	" ✓		4.1 ✓
3	p/s	235 × 9	✓	260	✓	16.603 ✓		130 ✓

~~944.0~~ 915.4 ✓ × 2 ✓ = ~~1,888.0~~ 1,830.8 kg

FB2 — 2 Reqd

2	Flg ls	125×90×9	✓	6.780	✓	14.60 ✓		198.0 ✓
2	"	"	✓	6.530	✓	" ✓		190.7 ✓
1	Web p/	820 × 9	✓	6.780	✓	57.933 ✓		392.8 ✓
2	ls	125×90×9	✓	640	✓	14.60 ✓		18.7 ✓
14	"	90×75×9	✓	830	✓	11.02 ✓		128.1 ✓
3	p/s	235 × 9	✓	260	✓	16.603 ✓		130 ✓

941.3 ✓ × 2 ✓ = 1,882.6 kg

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CALCULATIONS FOR

国道 19 METER PLATE GIRDER SPAN

FB3 ✓ / Reqd ✓

2	Flg ℓs	125×90×9 ✓	6710 ✓	14.60 ✓	195.9 ✓	
2	"	" ✓	6460 ✓	" ✓	188.6 ✓	
1	Web pl.	820 ×9 ✓	6710 ✓	57.933 ✓	388.7 ✓	
2	ℓs	125×90×9 ✓	555 ✓	14.60 ✓	16.2 ✓	
14	"	90×75×9 ✓	830 ✓	11.02 ✓	128.1 ✓	
3	pls	235 × 9 ✓	260 ✓	16.603 ✓	130 ✓	
						930.5 kgs

Summary of floor beams 4701 ✓ kgs / 4,643.9

STRINGERS S1, S2, S3 & S4 ✓ / Reqd ✓

6	ℓs.	✓ 350×150@58.5 kgs.	4845 ✓		1,702.1 ✓	
6	"	✓ " "	4640 ✓		1628.6 ✓	
18	pls	✓ 170 × 9	280 ✓	12.011 ✓	605 ✓	
						✓ 3,389.2 kgs

LATERAL BRACINGS ✓ / Reqd ✓

✓ 4	ℓs	100×100×10 ✓	~~7600~~ 7614 ✓	1491	~~4553~~ ✓ 4539	
✓ 8	"	" ✓	~~3715~~ 3725 ✓	"	~~443.1~~ ✓ 444.3	
✓ 4	pls	~~435~~ ✓ ~~410~~ × 9 ✓	470 ✓	~~28967~~ ✓ 30,733	~~545~~ ✓ 57.8	
✓ 4	"	~~440~~ ✓ ~~415~~ × 9 ✓	680 ✓	~~29320~~ ✓ 31,086	~~798~~ ✓ 84.6	
✓ 2	"	~~420~~ ✓ ~~395~~ × 9 ✓	660 ✓	~~27907~~ ✓ 29,673	~~368~~ ✓ 39.2	
✓ 4	"	~~280~~ ✓ ~~260~~ × 9 ✓	~~590~~ 600 ✓	~~18369~~ ✓ 19,782	~~434~~ ✓ 47.5	
✓ 4	"	~~180~~ ✓ ~~170~~ × 9 ✓	~~170~~ 180 ✓	~~12011~~ ✓ 12,717	~~82~~ ✓ 9.2	
✓ 4	ℓs	75×75×9 ✓	810 ✓	9.96 ✓	323 ✓	
✓ 4	"	150×100×9 ✓	190 ✓	17.02 ✓	12.9 ✓	
						✓ ~~1,1643~~ kgs / 1,181.7

SHOE & ANCHOR BOLTS

4	Cast iron shoes		@ 140.0	5600 ✓	
4	Pl.	70×30	160	16.486 ✓	10.6 ✓
16	Anchor bolts	25ϕ	700	@ 3.130	50.1 ✓
16	Washers.	100×9	100	7.065	11.3 ✓
8	Bolts	22ϕ	115	@ 0.650	5.2 ✓
					637.2 kgs

RIVET HEAD

~~4270~~ ✓	~~4220~~	22ϕ shop rivets	@ 0.0964	~~406.8~~ ✓	411.6
~~1650~~ ✓	~~1370~~	22ϕ Field "	@ "	~~122.4~~ ✓	159.1
~~1289~~ ✓	~~1930~~	19ϕ shop "	@ 0.06464	124.8 ✓	121.5
~~290~~ ✓	~~670~~	19ϕ Field "	@ "	~~43.5~~ ✓	18.7
					✓ ~~697.3~~ kgs / 719.9

Grand summary of 19 meter girder

Girder	~~13,7702~~ ✓	13,7784
Floor beams	~~4701.1~~ ✓	4643.9
Stringers	3389.2 ✓	
Lateral bracings	~~1.1643~~ ✓	1,181.7
Shoes & Anchor bolts	637.2 ✓	
	~~23,665.0~~ ✓	23,630.4
Rivet heads	~~697.3~~ ✓	719.9
	~~24,362.3~~ ✓ kgs or	~~24,350~~ kgs
	24,341.3	24.341

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CHECKED BY M. Matano DATE 5-12-4 PAGE NO 73

CALCULATIONS FOR

Materials of Jōnan Bashi for Kyoto Prefecture

第三款　高欄鋳鉄格子　ペンキ塗面積(全表面) 1枚分

表面	外縁(表裏)	2.76・.03・2	= .1656
〃	(外縁)	2.88・.03	= .0864
〃	(内縁)	2.49・.03	= .0747
中横桟延長		78・2+655+542 = 2757	497
〃縦〃〃		125・3・355・4+16・2 = 2115	
面積(表裏)		4.97・.015・2	= .1461
〃	(内外)	(4.97・2-.015・52)×.03	= .2688
			741 平方米

第四節　青銅製飾鋲 (Sheet No 13 参照)
青銅製飾鋲　20 枚

第五節　電灯 (Sheet No 13 参照)
青銅金具台付ランプ、ガラス及附属品共　10 基

注意 ヒンジ柱ノ伸縮目地用エラスタイトハ 76 頁床版第四節ニ記載ス

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CALCULATIONS FOR

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CALCULATIONS FOR

Design of truss span length 8@4.0 = 32.0 meters (Bowstring Type).

6.75^2 = 45.5625
4.25^2 = 18.0625
63.6250 — 7.98

Chord stress.

92.000 kgm.

48.0m
45m
1445.0

2.5
2.5
2.5
27.5
5.50

9.0m
4.5
40.5

5.5
5.5
60.5

9.7

4.25^2 = 18.0625
16.
34.0625

1010
89
921

193m	33.1m	414m	441m
74m	126m	158m	168m
267m	459.20	572m	609.90

5 102.33
49.50
151.83

4380
2700
1680

1818
534
1284

1.625

1000
15000

4.

3.5
14.0
6
8

111.5² x

111.5² x (4.0 x 3.5)

534

89

212
2
2.42

111.5 x 8 x 4.25 = 378000 = 95.5m
4.00.

198m = 49.5m
4.m

4500
149.500

580m
2920
259.0
3179.0

2190.0
222.0
2412.0

156m
1850.0
1745.0

1040m
148.0m
1188.0

6250
111.0
736.0

(41)

60.50
88.15
148.65

405

213m
18.90
231.90

366.00
32.40
398.40

457.0
40.50
497.50

48.70
4.32
53.02

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（独立行政法人 土木研究所蔵）

標準設計

貳拾壹米鈑桁橋（1等道）

（計算書、材料表）

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MADE BY K. Inaba DATE 5-7-29 FILE NO B&C
CHECKED BY M. K DATE 5-8-6 PAGE NO 1

CALCULATIONS FOR

Design of 18 meter and 12 meter Girder Spans for Mie-ken
Design of floor for girder span

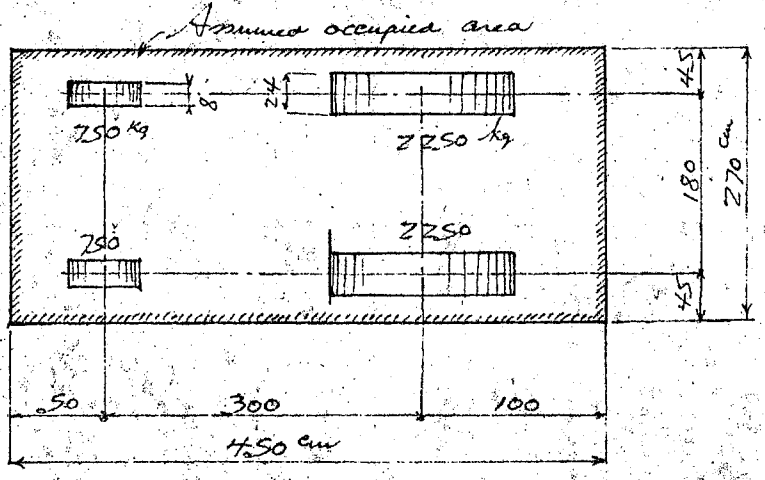
Clear width of Roadway = 5.40 meters
Pavement 3.8 cm asphaltic block on reinforced concrete slab.

Assumed loadings

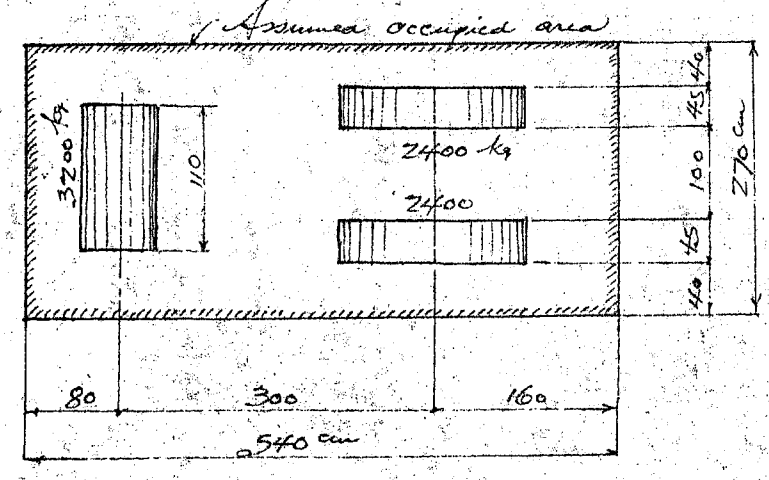
Uniform load on Roadway $w = \frac{100000}{170+l} \leq 500$ kg/m²

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

6 ton motor truck loading



8 ton road roller loading



Tow rows of motor traffic on roadway with occupied width of 270 cm ; Unoccupied space around the motor truck shall be filled with uniform load specified above.
One road roller on one span.
Impact for motor truck loading $\text{Coef} = \frac{20}{60+l}$ where l = loading length in meter
max. impact 30 %.
No impact for road roller and uniform live load.

Allowable working strength

Structural steel or Reinforcing bars.

Tension net	1200	kg/cm²
Extreme fibre stress net	1200	"
Shear of web, gross section	900	"
Compression member		
$1500\,(1-0.0055\,\frac{l}{r})$ not over	1000	"
where l = length of member in cm		
r = least radius of gyration in cm		
Compression flange of girder		
$1200\,(1-0.012\,\frac{l}{b})$ not over	1100	"
Shear on shop driven rivets (machine driven)	850	"
" " field " (" " and turned bolts	750	"
Shear on pin	900	"
Bearing on shop driven rivets (machine driven)	1700	"
" " field "	1500	"
" " pin	1800	"
Rollers $45\,d$ kg where d = diameter of roller in cm		

Concrete 1:2:4 mixture.

Direct compression	35	kg/cm²
Fibre stress due to bending	45	"
Combined stress direct and bending	35	"

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CHECKED BY M.K DATE 5-8-6 PAGE NO 2

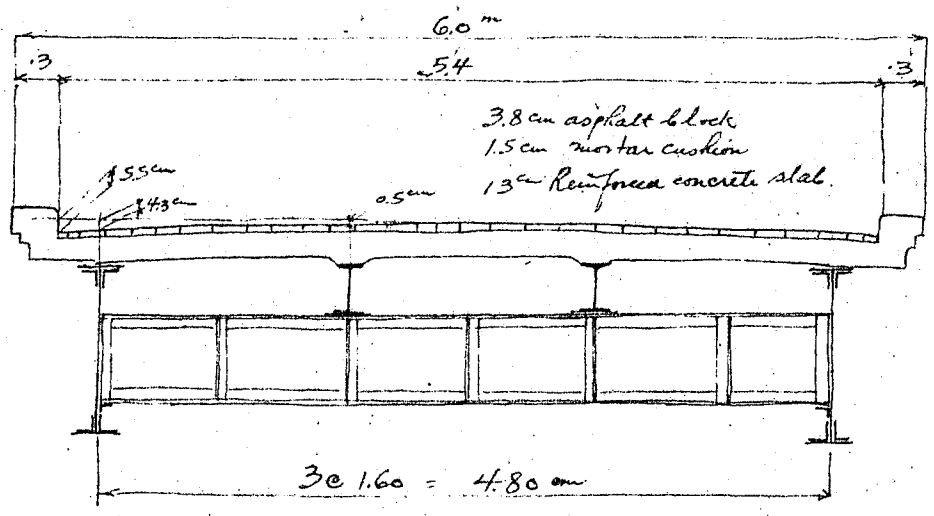
CALCULATIONS FOR

Design of 18 meter and 20 meter Girder Spans for Mid Span.
Concrete 1:2:4 mixture.

Punching shear of Concrete	9 kg/cm²
Shear of plain concrete	4 "
Bearing	45 "
Bond stress for plain bars	6 "
" " deformed "	9 "

For combined wind or temperature stress with dead, live, and impact stress, the allowable working strength shall be increased 25%. In case of earthquake, increase unit stress by 80%. Seismic acceleration assumed 3000 mm/sec² or k = 0.30

Design of 18.0 meter girder span.
Cross Section of Bridge as shown on sketch below.



Design of Floor slab. span length = 1.60 meters.
Dead load.

Asphalt block pavement	3.8 cm	@ 21 kg	=	80.
mortar cushion	1.5 cm	@ 17 "	=	26 "
Concrete slab	13.0 cm	@ 24 "	=	312 "
				418 "
misc concrete say			=	12 "
				430 kg/m²

Dead load moment = 1/10 × 430 × 1.60² = 110 kgm
Dead load shear = 1/2 × 430 × 1.60 = 345 kg

Live Load motor truck loading. Rear wheel concentration 2250
Impact 30% = 675
2925 kg
Front wheel concentration 2925/3 = 975 kg

Distribution of wheel concentration
Contact between wheel and pavement 20.0
distribution 2 × 5.3 = 10.6
Longitudinal distribution a = 30.6 cm
Transverse distribution b = 24 + 10.6 = 34.6 cm

Effective width E = 2/3 · l + a = where l = span length in meters.
= 2/3 × 1.60 + .306 = 1.37 meters.
Load per meter strip = 2925 ÷ 1.37 = 2140 kg

Uniform load 500 kg/sq. meter.

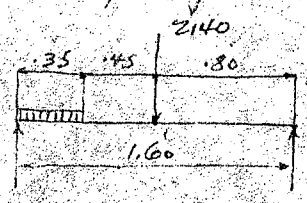
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CALCULATIONS FOR

Design of 18 meter and 20 meter Girder Spans for Rice Ku.



Uniform load $\frac{500 \times 0.35^2}{2 \times 1.60} = 19.1$ kg

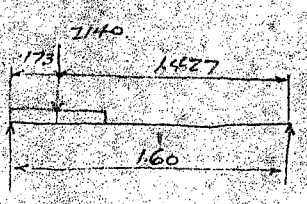
Moment at center of span

Due to uniform load $19.1 \times 0.8 = 15$

Due to motor truck $\frac{2140}{2} \times 0.8 = 856$ kgm. $\quad 871$ kgm.

For continuity of slab moment $= 0.8 \times 871 = 697$ kgm

Shear.



$2140 \times \frac{1.427}{1.60} = 1870$ kg

Summary for moments and shears.

	moments	Shears.
Dead Load	110	345
Live Load	697	1870
	807 kgm	2215 kg

Effective depth required for $f_s = 1200$ kg/cm² and $f_c = 45$ kg/cm²

$R = \frac{M}{bd^2}$, $d = \sqrt{\frac{M}{bR}}$, where $R = 7.18$ $\quad d = \sqrt{\frac{807 \times 100}{100 \times 7.18}} = 10.6$ cm

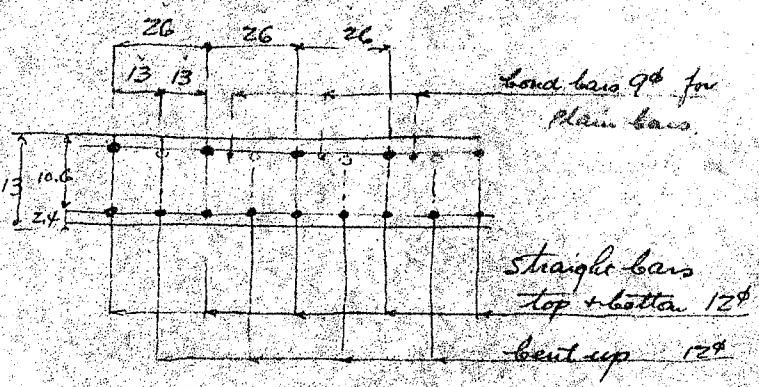
Use 13 cm Slab with insulation at bottom of 2.4 cm.

Steel area required $= \frac{807 \times 100}{1200 \times \frac{7}{8} \times 10.6} = 7.25$ cm² per meter strip.

use 12 mm ϕ bars at 13 cm c to c = 8.70 cm² ok.

Perimeter of reinforcing bars.

	Dia.	Circumf.	
12 mm	3.77	× 7.70	= 29.00 cm



Unit bond $= \frac{2215}{29.0 \times \frac{7}{8} \times 10.6} = 8.24$ kg/cm² ok for deformed bars.

if plain bars are used. add bond bars 9ϕ at 26 cm c to c

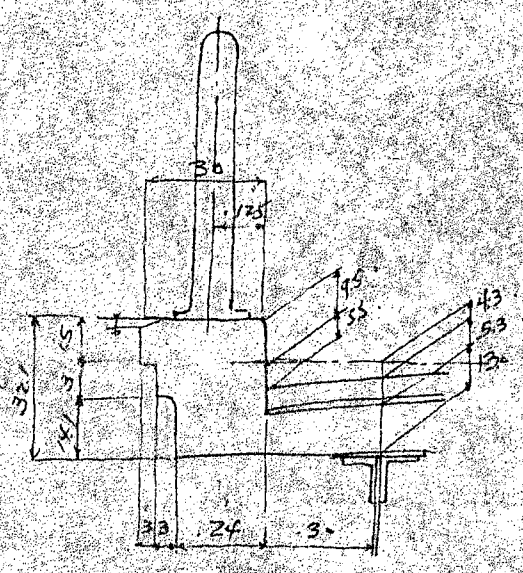
Perimeter of reinforcing bars.

12 mm	3.77	× 7.70	= 29.00
9 "	2.83	× 3.85	= 11.00
			40.00 cm

Unit bond $= \frac{2215}{40.0 \times \frac{7}{8} \times 10.6} = 5.96$ kg/cm² ok for plain bars.

Unit shear $= \frac{2215}{100 \times \frac{7}{8} \times 10.6} = 2.39$ kg/cm² ok.

Overhanging Slab beyond main girder.



Approximate weight of Handrail 70 kg per lin meter

" " " Coping 192 " " "

Dead load	Load.		
Handrail	70.	× 0.425	= 29.8
Coping	192	× say .44	= 84.5
Slab and pavement 1.30 × 3 =	129	× .15	= 19.4
	391 kg	.34	= 133.7

Live load motor truck rear wheel at 1.5 cm inside of curb line assumed

Distribution or effective width on main girder assumed thus

2 c 0.30 + 0.20 = 0.80 m

Transverse distribution 24 + 10.6 = 34.6 say 35 cm

Load on 30 cm = 2925 × $\frac{30}{35}$ = 2500 kg

for one meter strip = 2500 ÷ 8 = 3125 kg per meter strip.

Live load moment = 3125 × .15 = 467 kgm.

Shear assumed punching shear. sheared length along girder 80 / transverse length 2 × 30 = 60 / 140 cm

for one meter 3125 ÷ 1.40 = 2235 kg

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG TOKYO

MADE BY K. Inaba DATE 5-7-29 FILE NO B&C
CHECKED BY M. K DATE 5-8-6 PAGE NO 4

CALCULATIONS FOR

Design of 18 meter and 20 meter Girder Spans for Mui Ken.

Summary for moments and shears

	Moments	Shears
Dead Load	134	391
Live Load	467	2235
	601 kgm	2626 kg

Effective depth required = $\sqrt{\frac{601 \times 100}{100 \times 7.18}}$ = 9.15 cm

Use 13 cm slab, eff. depth 10.60 cm.

Unit shear = $\frac{2626}{100 \times \frac{7}{8} \times 106}$ = 2.83 kg/cm² ok.

Steel area required = $\frac{601 \times 100}{1200 \times \frac{7}{8} \times 10.6}$ = 5.40 cm² per meter strip.

use 12 m/m ϕ bars at 13 cm c to c = 8.70 cm² ok

for bond stress.

main bars 12 m/m ϕ	3.77 × 7.70	= 29.00
bond bars 9 m/m ϕ	2.83 × 7.70	= 21.80
		50.80 cm = total perimeter of bars.

Unit bond = $\frac{2626}{50.8 \times \frac{7}{8} \times 10.6}$ = 5.6 kg/cm² ok for plain bars.

For deformed bars use 2 - 9ϕ bond bars about, total perimeter 29.00

2c 2.83 = 5.66

34.66

Unit bond = $\frac{2626}{34.66 \times \frac{7}{8} \times 10.6}$ = 8.2 kg/cm² ok for deformed bars.

Temperature bars in slab.

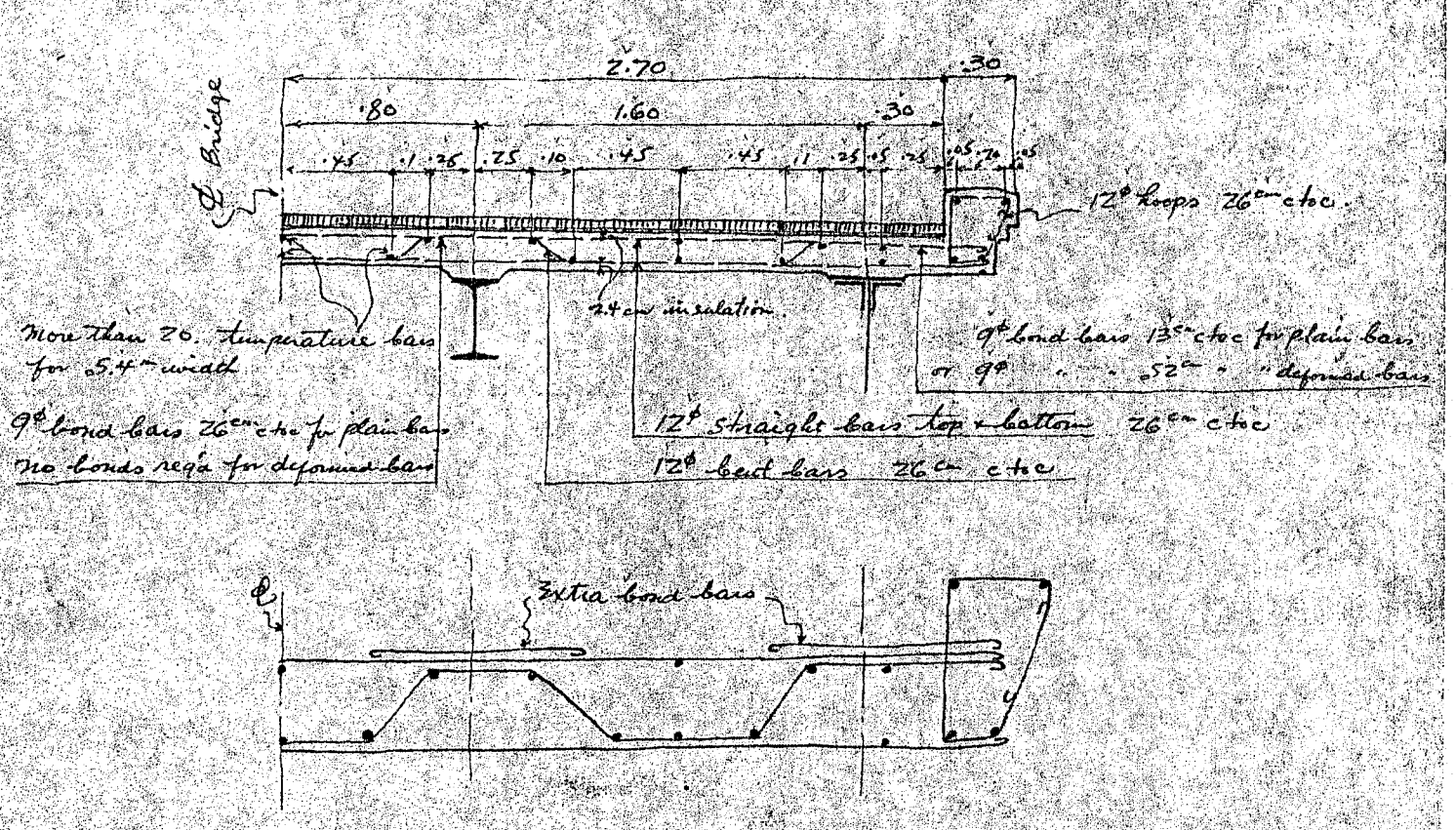
Steel area $\frac{3}{1000}$ of cross sectional area of slab concrete.

Sectional area of floor slab = 13 × 540 = 7020 cm²

Sectional area of temperature bars = $\frac{3}{1000} \times 7020$ = 21.0 cm²

12 m/m ϕ bars, required no = $\frac{21.0}{1.13}$ = 18.6 bars.

use more than 20 bars longitudinally.



General sketch for slab reinforcements.

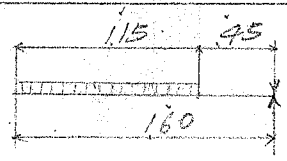
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CALCULATIONS FOR

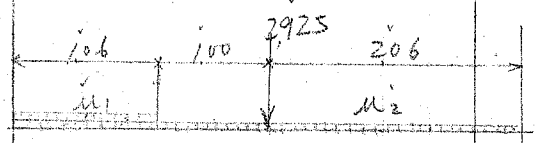
Standard 21 meters Girder

Design of I Beam stringers Span length 4.12 meters Spacing 1.60 meters
Dead load
floor slab and pavements 430 × 1.60 = 688
stringer assumed 50
738 call this 740 kg/lin. meter
Dead load moment = 1/8 × 740 × 4.12² = 1570 kgm
Dead load shear = 1/2 × 740 × 4.12 = 1525 kg
Live load
motor truck rear wheel concentration with impact = 2925 kg.
Uniform live load = 500 Kgs per sq meter

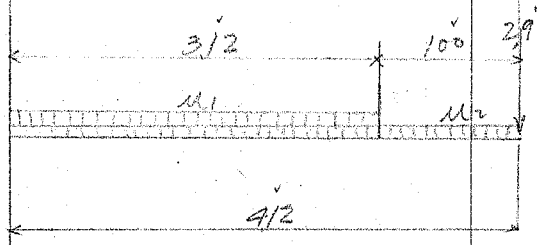


Load on Stringer = $\frac{500 \times 1.15^2}{2 \times 1.60}$ = 207 kg ..... u_2

Full uniform load = 500 × 1.60 = 800
less 207
593 kg ..... u_1



Moment
Due to motor truck $\frac{2925}{2} \times 2.06$ = 3,010
uniform load u_1 $\frac{593 \times 1.06}{2} \times 2.06$ = 166
uniform load u_2 $\frac{1}{8} \times 207 \times 4.12^2$ = 439
3615 Kgm



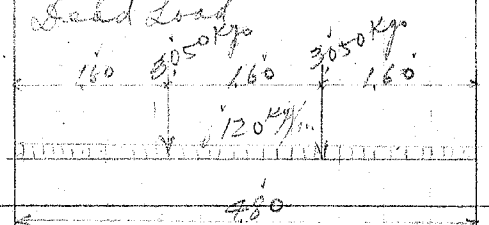
End Shear
Uniform load u_1 $\frac{593 \times 3.12^2}{2 \times 4.12}$ = 700
Uniform load u_2 207 × 4.12 ÷ 2 = 425
motor truck loading 2925
4050 kg

Summary for moments and shears

	moments	Shear
Dead Load	1570	1525
Live Load	3615	4050
	5185 Kgm	5575 kg

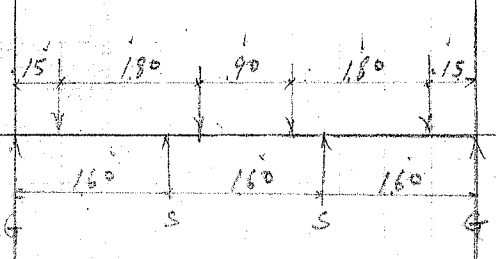
Section modulus required = $\frac{5185 \times 100}{1100}$ = 471.0 cm³
Use 300×150 I c 1834kg Sm = 6532 cm³ ok
Unit stress = $\frac{5185 \times 100}{6532}$ = 820 kg/cm² C or T ok
Unit shear = $\frac{5575}{30 \times .8}$ = 232 kg/cm² ok

Design of Intermediate Floor beam Span length 4.80 meters, spacing 4.12 meters
Dead Load
Concentration on stringer = 740 × 4.12 = 3,050 kg.
Floor beam assumed 120 Kgs. per lin. meter of span
Dead load moment
due to stringer concentration 3,050 × 1.60 = 4,880
due to weight of floor beam $\frac{1}{8} \times 120 \times 4.80^2$ = 345
5,225 Kgm



End shear from concentration = 3050
from beam wt. $\frac{1}{2} \times 120 \times 4.80$ = 288
3338 kg.

Live load motor truck loading, rear wheel concentration with impact = 2925 kg.
front wheel " " " = 975 "



Load on S $\frac{1.15}{1.60}$ = .094
1.000
1.094 × 2,925 = 3,200 kg rear wheel
1.094 × 975 = 1065 kg front wheel

Uniform live load 500 × 1.60 = 800 kg. on stringer

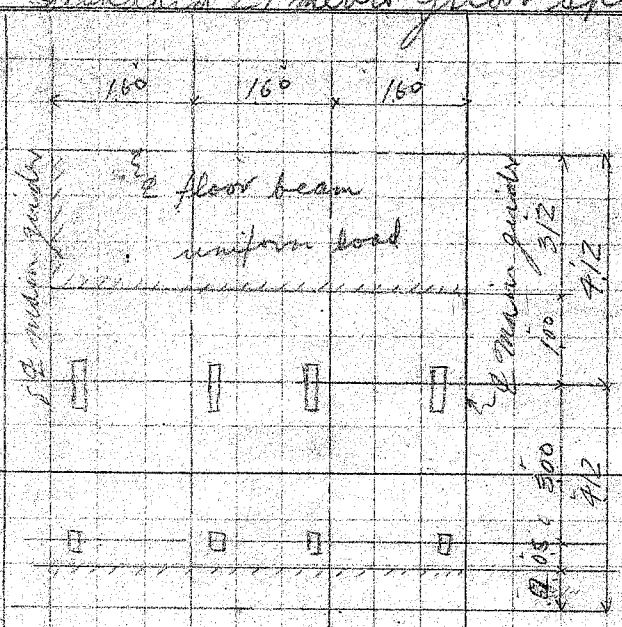
増田淳氏関係資料
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CALCULATIONS FOR

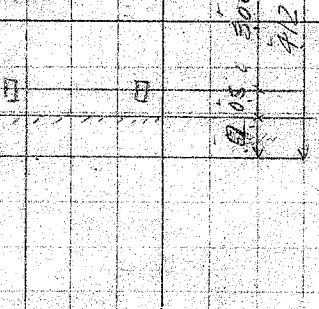
Standard 21 meter Girder span



Concentration on floor beam

Motor truck loading rear wheel 3200
front 1065 × 1/2 / 4/12 = 290
3490

uniform load 800 × 3.12 / 2 × 4.12 = 945
800 × 0.62² / 2 × 4.12 = 37
982
4472 Kgo



Moment 4.472 × 1.60 = 7.160 Kgm

Max End shear

Stringer concentration 4472
Rear wheel near main girder 2925 × 175/160 = 2,650
Front wheel near main girder 975 × 4/12 × 195/160 = 290
uniform load 982 × 500/800 × .80 = 490
7,852 Kgo

Summary for moments and shears

	moments	end shears
Dead Load	5225	2338
Live Load	7160	1852
	12385 Kgm	11790 Kgo

4472 7810

web assumed 870 × 8 = 6960 mm² 1/2 inch = 870 mm²
Back to back of L's 880 mm
Effective depth say 880 - 43 = 837 cm
Flange stress = 1238500 / 837 = 14800 Kg/cm² T
Flange area required = 14800 / 1200 = 12.39
- 1/2 web area - 8.70
3.69 net

Use 2 L's 75×75×9 = 25.38 - 3.96 = 21.42 cm² net
unit shear on web 11790 / 69.86 = 169 Kgo/cm²
7810 × 169 / 2640 = 3.0

Approximate weight of intermediate floor beam

Flange	4 L's	75×75×9	@	9.96	× 4.70	=	186
Web	1 Pl.	870×8	@	69.52	× 4.80	=	261 — 271
End conn.	2 L's	125×90×9	@	16.07	× .72	=	23 — 21
Stiffener	10 L's	75×65×9	@	8.28	× .88	=	73 — 81
web of stringer	2 Pl	290 × 9	@	20.5	× 3.1	=	19

56 — 572
19
573 Kgo
591

Rivet heads & variations say 3½ cut

578 = 48 = 120 Kg/lin mtr
591 123

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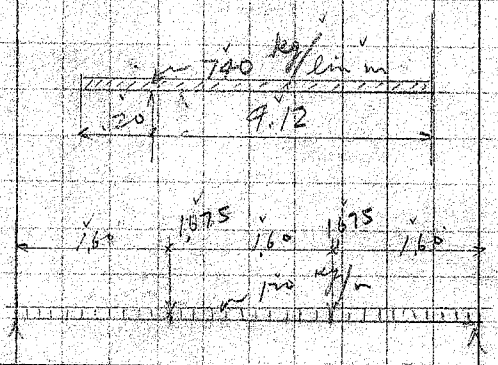
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CALCULATIONS FOR

Standard 21 metre Girder

Design of End floor beam Span length 4.80 metres, spacing 4.12 metres

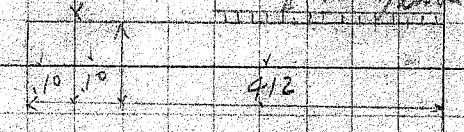


Stringer Concentration on floor beam
$\frac{740 \times 4.32^2}{2 \times 4.12} = 1675^{\vee}$ Kgs

Floor beam assumed 120 Kgs/sq lin metre
Dead load moment due to
Stringer concentration = $1675^{\vee} \times 1.80^{\vee} = 2,680^{\vee}$
weight of floor beam $\frac{1}{8} \times 120^{\vee} \times 4.80^{2} = 345^{\vee}$
3,025 Kgm

Max. End shear
from concentration
~~uniform load near main girder $\frac{120 \times 4.32^2}{2 \times 4.12} \times 0.5 = 780$~~ out
weight of floor beam $\frac{120}{2} \times 4.8^{\vee} = 290^{\vee}$
~~2785 Kgs~~
1,965

Live Load



Rear wheel concentration on stringer = 3,200 Kgs
front " " " = 1065 Kgs

Stringer concentration due to rear wheel $\frac{3,200 \times 4.22}{4.12} = 3,275^{\vee}$
" " unif load $\frac{740 \times 3.22^2}{2 \times 4.12} = 930^{\vee}$
4,205 Kgs



moment = $4,205^{\vee} \times 1.6^{\vee} = 6,720$ Kgm
End shear
Stringer concentration 4,205
Rear wheel near main girder $\frac{2,925 \times 4.22}{4.12} \times \frac{1.45}{1.6} = 2,710$ out
uniform load near main girder $\frac{500 \times 3.22^2}{2 \times 4.12} \times .80 = 500$
7,415 Kgs

Summary for moments and shears

	moments	shears
Dead Load	3,025	~~2785~~ 1,965
Live Load	6,720	~~2975~~ 4,205
	~~9,745~~ Kgm	~~6,160~~ Kgs 6,170

Use same details as for intermediate floor beam

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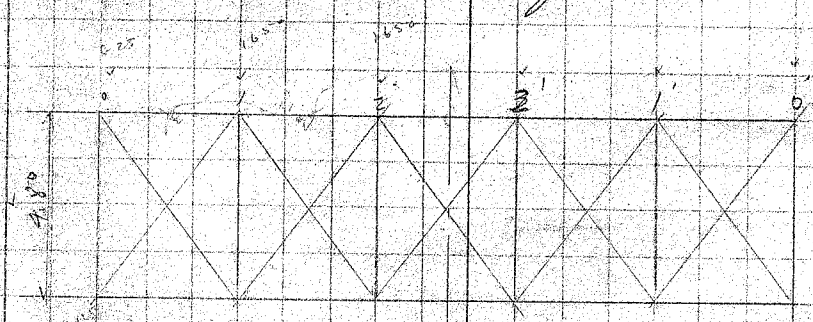
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CALCULATIONS FOR

Standard 21 meters girder

Design of lateral Bracings



Diagonal length
480² = 230400
412² = 169744
400144
Diagonal length = √400144 = 632 meters
Coefficient = 632/480 = 1.317

Wind load assumed 400 Kgs/lin. meter of bridge panel load = 400 × 4.12 = 1,650 Kgs.
Shear at panel o—1 1,650 × 2.0 = 3300 Kgs.
Stress in diagonal = 3,300 × 1.317 = 4,350 Kgs. T

Seismic stress acceleration assumed 2,500 mm/sec² or k = 0.250

Dead Load
Floor slab and pavement between main girder 4.5 × 930 = 2063
Handrails, coping, floor outside of " " 2 @ 391 = 782
2845
2 @ 50 = 100
575 ÷ 412 = 140
= 55
= 650
945
3790 Kgs per lin m

Stringers
floor beam
lateral bracing assumed
main girder assumed

panel load 3,790 × 4.12 = 15,600 Kgs
horizontal panel load 15,600 × .25 = 3900 Kgs
Shear in panel o—1 = 3,900 × 2.0 = 7800 Kgs
Stress in diagonal = 7800 × 1.317 = 10,250 Kgs. T

Seismic stresses govern the section of diagonal member
Sectional area required = 10250/1200 × 1.8 = 4.75 cm² net
19ϕ rivet no. required = 10250/4.26 × 1.8 = 2.68
use 1L. 125 × 75 × 9 = 17.19 − 2.0 = 15.19 cm² net ok
Radius of gyration about



x—x axis rx = 3.96 cm l/rx = 632/3.96 = 160 < 200 ok
y—y " ry = 2.07 l/ry = 316/2.07 = 153

Approximate weight of Lateral Bracings

Diagonal	10 Ls	125 × 75 × 9	c	13.50 × 4.91 × 5.95	=	887 → 803
Center conn.	5 P/s	270 × 9	c	1908 × 0.51	=	98
Conn. to girder	12 P/s	345 × 9	c	2437 × 0.60	=	175
						1160 → 1026
Rivets heads and variation say 1½% abt =						15
						1,175 Kgs
						1,041

1175 ÷ 20.60 = 55 kgs per lin meter
1041 49

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CALCULATIONS FOR
Design of Main girder
Standard 21 metre girder

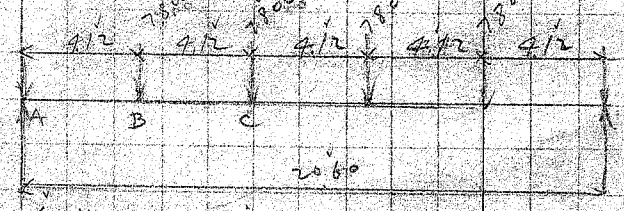
Dead Load

Floor and pavement between main girders	1.30	e 4.30	= 2,063
Handrail, copings and floor drain	2	e 391	= 782
			2,845
Stringers	2	e 50	= 100
floor beams	573 / 591	÷ 4.12	= 140 → 144
lateral bracing			= 53 → 51
main girder assumed			= 650
			945
			3,790 Kg. per lin m

For one girder 3790 ÷ 2 = 1895 Kg. per lin. meter

Let us assume the load concentrated at panel point

Intermediate panel concentration = 1895 × 4.12 = 7,800 Kgs. for one girder
End = 1895 × 2.06 = 3,900 Kg. " "



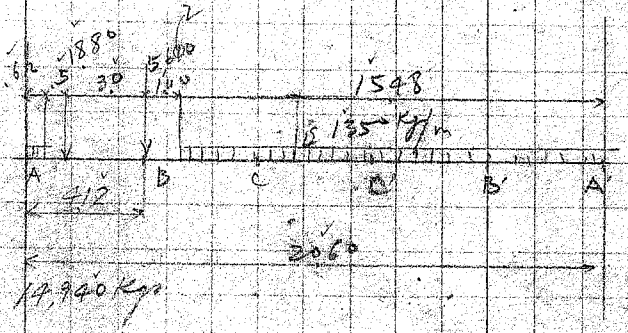
Reaction = 7,800 × 2.0 = 15,600 Kg.
moment at B 15,600 × 4.12 = 64,300 Kgm
moment at C 15,600 × 7.29 = 138,500 Kgm
7,800 × 4.12 = − 32,100
96,400 Kgm

max end shear = 15,600 Kg.
3,900 Kg. — out
19,500 Kg.

Dead load on shoe or bearing = 1,895 × 10.50 = 20000
add for shoe adj 100
20,100 Kgs. for one shoe

Live Load Uniform live load = 500 Kg. per sq. meter
Impact for motor truck loading 20 / 60 + 20.6 = 24.7%

motor truck rear wheel concentration = 2,250
24.7% impact 560
2,810 Kgs.
Front wheel with impact = 2,810 ÷ 3 = 940 Kgs.
uniform live load = 500 × 2.7 = 1350 Kg. per lin meter
motor truck concentration
Rear wheels 2 e 2,810 = 5,620 Kgs.
front " 2 e 940 = 1,880 Kgs.



moment at B
Reaction
5,620 × 4/5 = 4,490
1,880 × 1948/2060 = 1,780
1350 × 1.948²/2 × 2.060 = 7,850
1350 × 6.2 × 2.029/2.060 = 820
14,940 Kgs.

moment
14,940 × 4.12 = 61500
1,880 × 3.00 = − 5,640
1,350 × 6.2 × 3.81/2 = − 3190
52,670 Kgm

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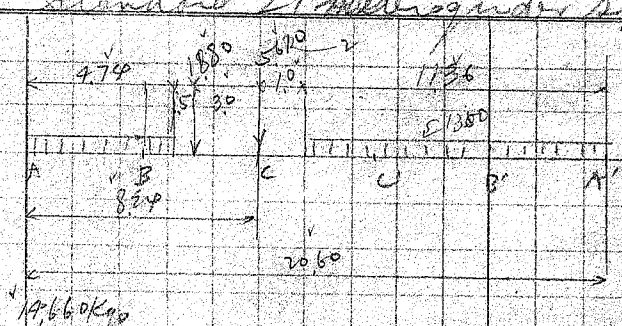
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CALCULATIONS FOR

Standard 21 meter girder span



moment at C

Reaction

5.60 × 3/11.36 = 3,370 ✓

1.88 × 5/11.36 = 1,400 ✓

1.350 × 20.6/11.36 = 4,230 ✓

1.350 × 4.74 × 18.23/(2×20.60) = 5,660 ✓

19,660 Kgo ✓

moment

19,660 × 8.24 = 120,800 ✓

1.88 × 3.00 = 5,640 ✓

1.350 × 4.74 × 5.87 = 37,550 ✓

77,610 Kgm ✓

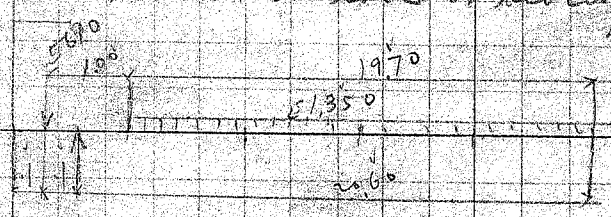
max. end shear. say rear wheel at C bearing

uniform load 1.350 × 19.60²/(2×20.60) = 12,590

motor truck 5,600

18,200 Kgo

Max. Load on shoe or bearing



Reaction 1.350 × 19.70²/(2×20.60) = 12,720

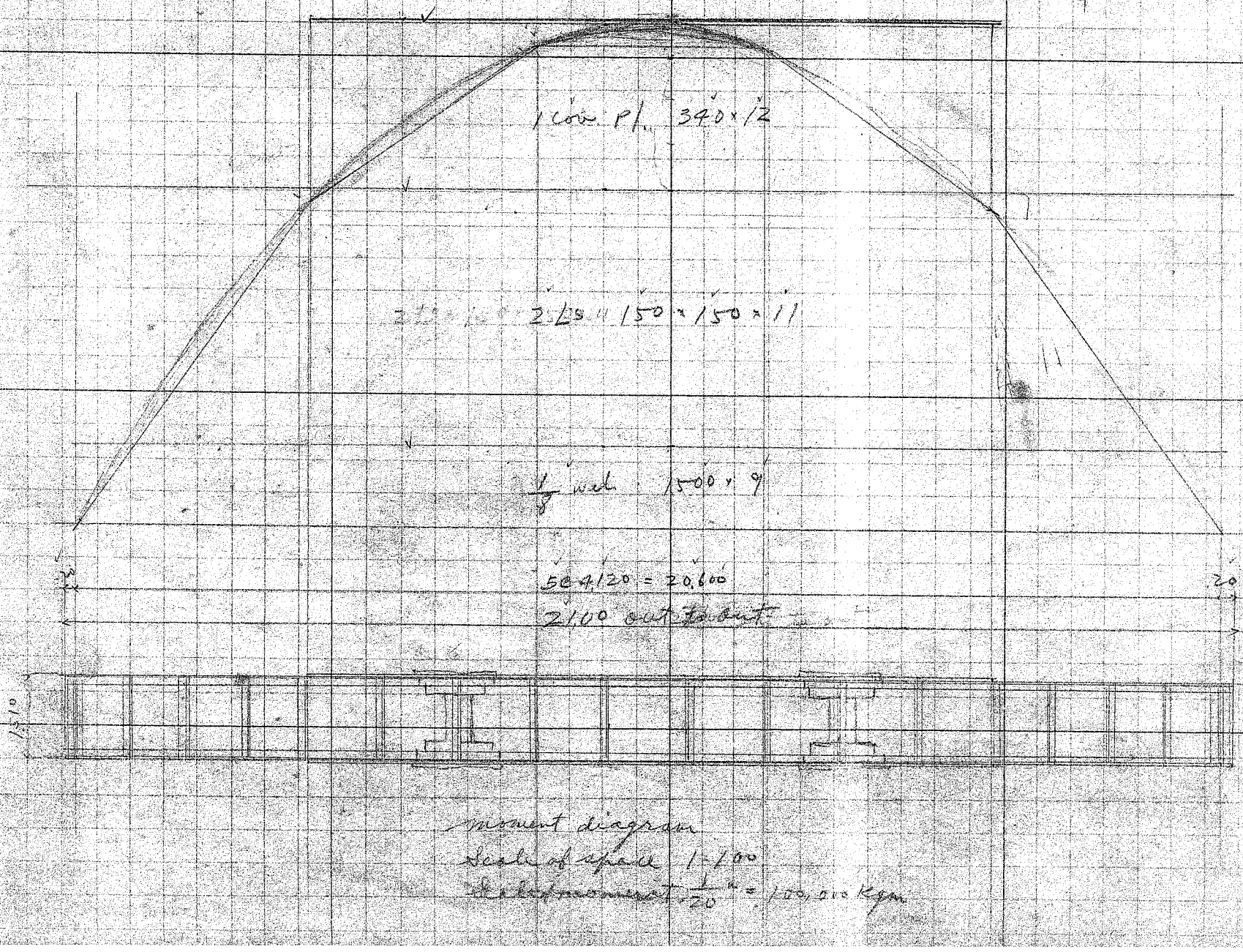
5,600

18,320 Kgo on one shoe

2 4

Summary for moments and shears

	moments B	moment C	end shear	Load on shoe
Dead load	64,300	96,400	9,500 15,600	20,100
Live load	52,670	77,610	18,200	18,320
	116,970 Kgm	174,010 Kgm	33,700 Kgo	38,420 Kgo
				4



moment diagram

Scale of space 1-100

Scale of moment 1-20 = 100,000 Kgm

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CALCULATIONS FOR

Standard 21 meter girder span

web plate 1500 × 9 web area = 150.9 = 1350 cm² 1/8 web area = 16.88 cm²
total depth 151 cm back to back of Ls Effective depth 1510 − 48 = 1462 cm
2 Ls 150×150×11 = 63.58 × 40.7 = 2585
1 Pl. 340×12 = 40.8 × 1.6 = 245
104.38 224 239
flange stress
= 174010 / 1462 = 118,700 Kgs T or C
flange area required 118,700 / 1200 = 99.00 cm² net
+ 1/8 web − 16.88
82.12 cm²
use 2 Ls 150×150×11 = 63.58 − 11.00 = 52.58
1 cov. Pl. 340×12 = 40.80 − 6.0 = 34.80
104.38 87.38 cm² net
unit tension = 118,700 / 87.38 + 16.88 = 1140 kg/cm² ok
unit compression = 118,700 / 104.38 + 16.88 = 980 kg/cm² ok
unit shear on web = 33,800 / 135.0 = 250 kg/cm² ok
Allowable unit compression on flange = 1200 (1 − 0.0002 × 41²/39) = 1040 kg/cm² C

Approximate weight of main girder

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CALCULATIONS FOR 標準橋梁

2,000 Girder Span

GIRDER G1 & G1E 4 Req'd.

4	Flg Ls	150×150×11	6,320	24.95		630.7	
1	Web Pl.	1,500 × 9	6,320	105.975		669.8	
2	Cov Pls.	340 × 12	1,990	32.028		127.5	
4	Ls	125×90×(10) 9	1,488	(16.09)	14.6		(95.8) 86.9
2	Fills	190 × 11	1,205	16.407			39.5
10	Ls	125×90×(10) 9	1,510	(16.09)	14.6		(243.0) 220.5
2	〃	150×100×9	230	17.02			7.8
1	〃	〃	305	〃			5.2
1	Sole Pl.	400 × 25	460	78.50			36.1

1,428.0 + (427.4) = 1,855.4 1,824.0
396.0 × 4
(742.6)
7,296.0

G2 2 Req'd.

4	Flg Ls	150×150×11	8,360	24.95		834.3	
1	Web Pl.	1,500 × 9	8,360	105.975		886.0	
2	Cov. Pls	340 × 12	8,360	32.028		535.5	
12	Ls	125×90×(10) 9	1,510	(16.09)	14.6		(291.6) 264.6
2	〃	150×100×9	230	17.02			7.8
2	〃	〃	305	〃			10.4

2,255.8 + (309.8) = (565.4) 2538.6
282.8 × 2
(513.2)
5,077.2

SPLICE 4 Req'd.

2	Pls.	340 × 12	1,070	32.028		68.5	
4	Ls	150×150×15	780	33.55		104.7	
4	Pls.	235 × (13) 14	630	(23.982)	25.827		(60.5) 65.1
2	〃	330 × 9	730	23.315			34.0

(267.0) 272.3
× 4
1070.4
1,089.2

Summary of girder (13,623.2) 13,462.4

FLOOR BEAM FB1 2 Req'd.

4	Flg Ls	75×75×9	4,510	9.96		179.7	
1	Web Pl.	900 × 8	4,750	56.52		268.5	
8	Ls	75×65×(8) 9	910	(8.28)	9.26		(60.3) 67.4
2	Fills	120 × 8	1,488	7.536			22.4
2	〃	〃	265	〃			4.0
2	〃	〃	315	〃			4.7
2	Pls	390 × 9	310	20.489			12.7

448.2 + (104.0) = (552.3) 559.4
111.2 × 2
(1102.6)
1,118.8

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CALCULATIONS FOR

21.m girder Span

FB2 4 Req'd.

2	Flg Ls	75 × 75 × 9	4,770	9.96		95.0		
2	"	"	4,530	"		90.2		
1	Web Pl.	900 × 8	4,770	56.52		269.6		
8	Ls	75 × 65 × 9	910	8.28	9.26		60.3 — 67.4	
2	"	125 × 75 × 10	720	14.91	14.60		21.5 — 21.0	
2	Pls.	290 × 9	310	20.489			12.7	
						454.8 +	194.5 = 649.3	655.9
							101.1	4
								2,197.2
								2,223.6

Summary of flor beams 3301.8 — 3342.4

STRINGER 1 Req'd.

6	I	300 × 150 @ 48.3#	4,110			1,192.1		
4	"	"	4,315			834.3		
16	Pls.	170 × 9	230	12.011			44.2	
						2,026.4 +	44.2	= 2,070.6

LATERAL BRACING 1 Req'd.

5	Ls	125 × 75 × 10	5,950	14.91	13.5	443.6	401.6	
10	"	"	2,920	"	"	435.4	394.2	
5	Pls	280 × 9	530	19.782			52.4	
4	Pls.	300 × 9	415	21.195			35.2	
8	"	290 × 9	645	20.489			105.7	
						879.0 +	193.3	= 1072.3
						795.8		989.1

RIVET HEADS

4,264	22φ	shop rivet heads	@.0639		411
1,840	"	field " "	@ "		178
1,590	19φ	shop " "	@.0464		103
620	"	field " "	@ "		40
					731

Grand Summary for 21.m girder Span

Girders	13,623.2	13,462.4
Flor beams	3,301.8	3342.4
Stringers	2,070.6	2070.6
Lateral bracings	1,022.3	989.1
	20,067.9	19,864.5
Rivet heads	731.0	+ 731.0
	20,798.9	20,595.5
Shoe Anchor bolts	637.2	+ 637.2
	21,436.1	21,232.7
		or 21,233 Kg tons

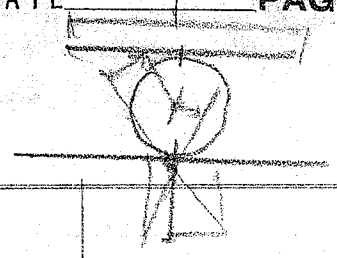
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CALCULATIONS FOR 桁主 標準橋梁

18.00 Girder span



GIRDER G1R & G1AR 4 Req'd.

4	L	150×150×11	8,000	24.95	798.4		
1	Web Pl.	1,200 × 9	8,000	84.78	678.2		
2	Cov. Pl.	320 × 12	5,060	30.144	305.1		
4	L	125×90×10→9	1,188	16.09→14.6		76.5→69.4	
2	Fill.	190 × 11	905	16.407		29.7	
14	L	125×75×10→9	1,210	14.91→13.5		252.6→241.3	
2	L	125→100×90×10→9	230	14.13→14.6		6.5→6.7	
1	L	〃	305	〃		4.3→4.5	
1	Sole Pl.	400 × 25	460	78.500		36.1	

1781.7 + 405.7 = 2187.4→2169.4

387.7 × 4 = 8749.6 → 8,677.6

SPLICE 2 Req'd.

2	Pls.	320 × 12	1,000	30.144	60.3	
4	L	150×150×15	810	33.55	108.5	
4	Pls	185 × 13→14	810	18.879→20.332	61.1→65.9	
2	〃	330 × 13→14	530	33.677→36.267	35.7→38.4	
2	L	125×75×10×9	1,188	14.91→13.5	35.5→27.0	
2	L	100×75×10	240	12.95	6.2	

307.3→306.3 × 2 = 614.6→612.6

Summary of main girder 9364→9,290.2

FLOOR BEAM FB1 2 Req'd.

4	L	75×75×9	4,510	9.96	179.3	
1	Web Pl.	690 × 8	4760	43.332	206.0	
10	L	75×65×8→9	700	8.28→9.26	58.0→64.8	
2	Fills	120 × 8	1,188	7.536	17.9	
4	〃	〃	245	〃	7.4	
2	Pls.	290 × 9	310	20.489	12.7	

385.3 + 96.0 = 481.3 → 488.1

102.8 × 2 = 962.0 → 976.2

FB2 2 Req'd.

2	L	75×75×9	4,780	9.96	95.2		
2	〃	〃	4,530	〃	90.2		
1	Web Pl.	690 × 8	4,780	43.332	207.0		
10	L	75×65×8→9	700	8.28→9.26		58.0→64.8	
2	〃	125×90×10→9	530	16.09→14.6		17.7→15.5	
2	Pls	290 × 9	310	20.489		12.7	

392.4 + 87.8 = 480.2→485.4

93.0 × 2 = 960.4→970.8

FB3 1 Req'd.

2	L	75×75×9	4,750	9.96	94.5		
2	〃	〃	4,500	〃	89.6		
1	Web Pl.	690 × 8	4,750	43.332	205.8		
10	L	75×65×8→9	700	8.28→9.26		58.0→64.8	
2	〃	125×75×10→9	530	14.91→13.5		15.5→14.3	
2	Pls.	290 × 9	310	20.489		12.7	

383.9 + 86.3 = 470.2→481.7

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CALCULATIONS FOR

18.00m Girder Span

STRINGERS 1 Req'd.

4	H	300×150 @ 48.34	4,390	✓ 850.6	
4	"	"	4,595 ✓	✓ 888.5	
12	Pl.	170 × 9	230	✓ 12.011	✓ 332
					✓ 1739.1 + 33.2 = ✓ 1,772.3

LATERAL BRACING 1 Req'd

4	L	125 × 75 × 10	9 ✓	6.140	✓ 14.90 135	✓ 367.0 331.6
8	"	"	" ✓	3,020	✓ " "	✓ 360.0 326.3
4	Pl.	310	× 9	410	✓ 21.902	✓ 36.0
4	"	300	× 9	630	✓ 21.195	✓ 53.4
2	"	305	× 9	645	✓ 21.548	✓ 27.8
4	"	280	× 9	520	✓ 19.782	✓ 41.0
						✓ 727.0 +✓ 158.2 = ✓ 585.2
						657.9 816.1

RIVET HEAD

3,536 ✓	22φ	shop	rivet heads	@ .09639	341 ✓
~~4~~ 8936	"	field	" "	@ "	90 ✓
~~1 3 0 4~~ 1264	19φ	shop	" "	@ .06464	~~84~~ 82 ✓
496 ✓	"	field	" "	@ "	32 ✓
					548 ~~5~~ ✓

Grand summary for girder span.

Summary of	main girders	~~9364.2~~ ✓	9290.2
"	floor beams	~~2399.4~~	2428.7
	stringers	1,772.3	1,772.3
	lateral bracings	~~885.2~~	816.1
		~~14.421.1~~	14,307.3
	Rivet head	~~540.05~~	545.0
		~~14.968.2~~	14,852.3
	Shoes & Anchor bolts	637.2	637.2
		~~15.603.3~~	15,489.5 kg
			≒ 15.490 kg ton

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CALCULATIONS FOR

37.5 Meters Truss

END POST Lo-U1 — 4 Req'd

✓	1	Cov. Pl.	550 × 9	4,830	38.858	187.7	
✓	2	E	250×90@34.59	5,335		369.1	
	2	L	90×75×9	280	1.102	6.2	
	2	〃	〃	270	〃	6.0	
	1	Pl.	550 × 9	630	38.858	24.5	
	1	〃	535 × 9	550	37.798	20.8	
	12	Lac bars	50 × 10	725	3.925	34.1	
	2	Pls	950 × 9	950	67.119	127.5	
	2	Pin Pls	480 × 9	645	33.912	43.7	
	2	〃	340 × 9	465	24.021	22.3	
	2	Fills	190 × 9	295	13.424	7.9	
	1	〃	〃	475	〃	6.4	
	1	Pl.	430 × 9	580	30.58	17.6	
	1	L	90×90×10	275	13.84	3.7	
	4	L	90×75×9	775	11.02	34.2	
	1	L	〃	300	〃	3.3	
	1	Pl.	310 × 8	765	19.468	14.9	
	1	Fills	75 × 9	145	5.299	.8	
	1	L	90×75×9	510	11.02	5.6	
	1	L	〃	270	〃	3.0	
	1	Bent Pls	550 × 9	690	38.858	26.8	
	1	〃	450 × 9	550	31.793	17.5	
	1	〃	90 × 10	280	7.065	2.0	
	2	Pls	290 × 9	550	20.489	22.5	
	2	〃	685 × 9	1,170	48.395	113.2	

556.8 + 564.5 = 1,121.3
× 4
4,485.2

TOP CHORD U1 - U3 — 4 Req'd.

✓	1	Cov Pl.	550 × 9	7,550	38.858	293.4	
✓	2	E	250×90@34.59	7.545		522.0	
	1	Pl.	535 × 9	550	37.798		20.8
	2	〃	370 × 9	535	26.141		28.0
	1	〃	295 × 9	535	20.842		11.2
	18	Lac bars	50 × 10	725	3.925		51.2
	1	Pl.	550 × 9	750	38.858		29.1
	2	Pls	90 × 15	750	10.598		15.9
	2	〃	570 × 9	1,090	40.271		87.9

815.4 + 244.0 = 1,059.4
× 4
4,237.6

U3 - U5 — 4 Req'd.

✓	1	Cov. Pl.	550 × 9	7.510	38.858	291.8	
✓	2	E	250×90@34.58	7.510		519.4	
✓	2	Side Pls	190 × 9	7.725	13.424	207.4	
	2	Pls.	370 × 9	535	26.141		28.0
	2	Pls	295 × 9	535	20.842		22.3
	20	Lac bars	50 × 10	725	3.925		56.9

1,018.6 + 107.2 = 1,125.8
× 4
4,503.2

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CALCULATIONS FOR

37.5 Metres Truss

SPLICE x GUSSET PLATE. 2 Req'd.

1	Pl.	550	× 9	750	38.853	29.1
2	"	90	× 15	750	10.593	15.9
2	Pls	480	× 9	1.070	33.912	72.6
						117.6 × 2 = 235.2

Summary of end post & Top chords = 13,461.2

BOTTOM CHORD L0-L2 4 Req'd.

2	E	250×90×34.59		6,390		442.1
2	Pls	340	× 9	675	24.021	32.4
1	Pl.	720	× 8	730	45.216	33.0
10	Pls	200	× 8	325	12.560	40.8
2	"	250	× 9	505	17.668	17.8
2	"	300	× 9	445	21.195	18.9
2	"	325	× 8	330	20.410	13.5
						442.1 + 156.4 = 598.5
						× 4
						2,394.0

L2-L4 4 Req'd.

2	E	250×90@40.16		7,375		592.4
2	Pls	250	× 11	7,375	21.588	318.5
2	"	675	× 9	1,060	47.689	101.0
2	"	340	× 9	675	24.021	32.4
1	Pl.	665	× 8	710	41.762	29.7
1	"	610	× 8	710	38.308	27.2
1	"	305	× 8	370	19.154	7.1
10	"	200	× 8	305	12.560	38.3
2	Pls	250	× 11	865	21.588	37.3
2	Pls	190	× 16	865	23.864	41.3
2	"	310	× 9	685	21.902	30.0
						910.7 + 344.3 = 1,255.0
						× 4
						5,020.0

L4-L4' 2 Req'd.

2	E	250×90@40.16		9,320		748.6
2	Pls	250	× 11	9,320	21.588	402.4
2	"	190	× 9	8,440	13.424	226.6
4	"	675	× 9	880	47.689	167.9
2	"	340	× 9	675	24.021	32.4
3	Pls	568	× 8	710	35.670	76.0
2	"	305	× 8	370	19.154	14.2
10	"	200	× 8	305	12.560	38.3
						1,377.6 + 328.8 = 1,706.4
						× 2
						3,412.8

Diaphragm at L1 4 Req'd.

4	L	90×75	× 9	190	11.02	8.4
1	Pl.	190	× 8	310	11.932	3.7
						12.1 × 4 = 48.4

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CALCULATIONS FOR

37.5 Metres Truss

Diaphragm at L2 & L3 8 Req'd.

4	L	90 × 75 × 9	190	11.02	8.4
1	Pl.	190 × 8	280	11.932	3.3
					11.7 × 8 = 93.6

L4 & L5 6 Req'd.

4	L	90 × 75 × 9	190	11.02	8.4
1	Pl.	190 × 8	260	11.932	3.1
					11.5 × 6 = 69.0

Summary of Bottom chords = 11,037.8

VERTICAL L1-U1 4 Req'd.

✓	2	L	90 × 75 × 9	3,600	11.02	79.5
✓	2	〃	〃	3,525	〃	77.7
	1	Pl.	325 × 8	470	20.410	9.5
	1	〃	〃	370	〃	7.6
	13	Lac bars	50 × 8	345	3.140	14.1
						157.0 + 31.2 = 188.2
						× 4
						752.8

L2-U2 & L4-U4 8 Req'd.

✓	4	L	90 × 75 × 9	3,920	11.02	172.8
	1	Pl.	325 × 8	470	20.410	9.6
	1	〃	310 × 8	325	19.468	6.3
	15	Lac bars	50 × 8	345	3.140	16.2
	2	Pls	190 × 9	315	13.424	8.5
						172.8 + 40.6 = 213.4
						× 8
						1,707.2

L3-U3 & L5-U5 6 Req'd.

✓	4	L	90 × 75 × 9	3,920	11.02	172.8
	1	Pl.	325 × 8	470	20.410	9.6
	1	〃	310 × 8	325	19.468	6.3
	15	Lac bars	50 × 8	345	3.140	16.2
						172.8 + 32.1 = 204.9
						× 6
						1,229.4

Summary of Verticals = 3,689.4

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CALCULATIONS FOR

37.5 Metres Truss

			DIAGONAL U1-L2	4 Req'd.			
2	L	125 x 90 x 10	4,945	16.09		159.1	
2	"	"	4,845	"		155.9	
2	P/s	310 x 8	325	19.468		12.7	
20	Lac bars	50 x 8	345	3.140		21.7	
							349.4
							x 4
							1,397.6
			L2-U3	4 Req'd.			
4	L	125 x 75 x 10	4,880	14.91		291.0	
2	P/s	310 x 8	325	19.468		12.7	
20	Lac bars	50 x 8	345	3.140		21.7	
							325.4
							x 4
							1,301.6
			U3-L4 & L4-U5	8 Req'd.			
2	L	100 x 75 x 10	4,945	12.95		128.1	
2	L	"	4,885	"		126.5	
2	P/s	310 x 8	315	19.468		12.3	
20	Lac. bars	50 x 8	345	3.140		21.7	
							288.6
							x 8
							2,308.8

Summary of Diagonals = 5,009.0

Same as 30 meter			FLOOR BEAM	2 Req'd.			
					678.5	+ 170.6	= 849.1
							x 2
							1,698.2
Same as 30 Meter			FLOOR BEAM	9 Req'd.			
					680.5	+ 170.6	= 851.1
							x 9
							7,659.9

Summary of floor beams 9,358.1

STRINGERS & BRACKETS

40	I	300x150 @ 48.34	3,710		7,173.7		
160	L	100 x100 x 10	224	14.91		534.4	
8	Tension Rd	150 x 8	430	9.42		32.4	
8	I	300x150@48.34	380		147.0		
16	L	100x100 x 10	224	14.91		53.4	
					7,320.7	+ 620.2	= 7,940.9

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CALCULATIONS FOR

37.5 Meter Truss

			LATERAL BRACINGS			
2	L	125 × 75 × 10	7,010	14.91	209.0	
2	"	"	3395	"	101.2	
2	"	"	3365	"	100.3	
2	"	"	6,980	"	208.1	
4	"	"	3,365	"	100,3	
6	"	75 × 75 × 9	6,980	9.96	417.1	
12	"	"	3,415	"	408.2	
2	PLs	300 × 8	890	18.840	33.5	
2	"	260 × 8	790	16.328	25.8	
2	"	200 × 8	650	12.560	16.3	
4	"	160 × 8	530	10.048	21.3	
40	L	75 × 75 × 9	370	9.96	147.4	
40	PLs	145 × 8	150	9.106	54.6	
40	L	125 × 75 × 9	225	13.50	121.5	
					1,544.2 + 420.4	= 1,964.6

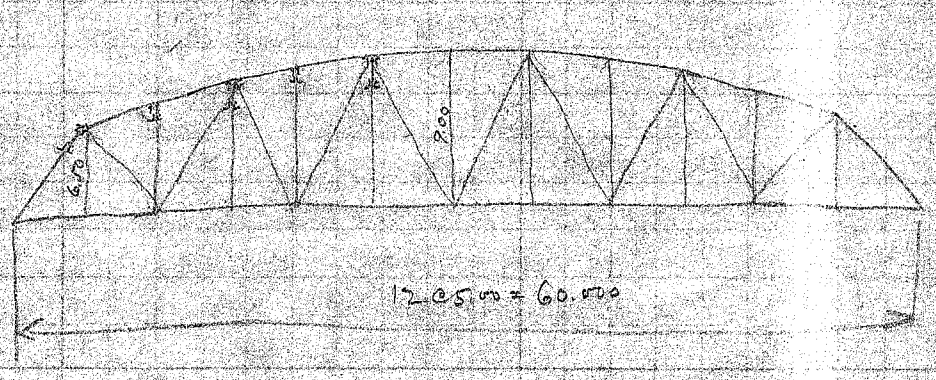
grand Summary

End posts & Top chords	13,461.2		
Bottom chords	11,037.8	33,196.4	(33,020)
Verticals	3,689.4		
Diagonals	5,008.0		
Floor beams	9,358.1		10,900
Stringers	7,940.9		7,620
Lateral bracings.	1,964.6		2,040
	52,460.0		
Rivet heads 3½ %	1,840.0		
	54,300.0		

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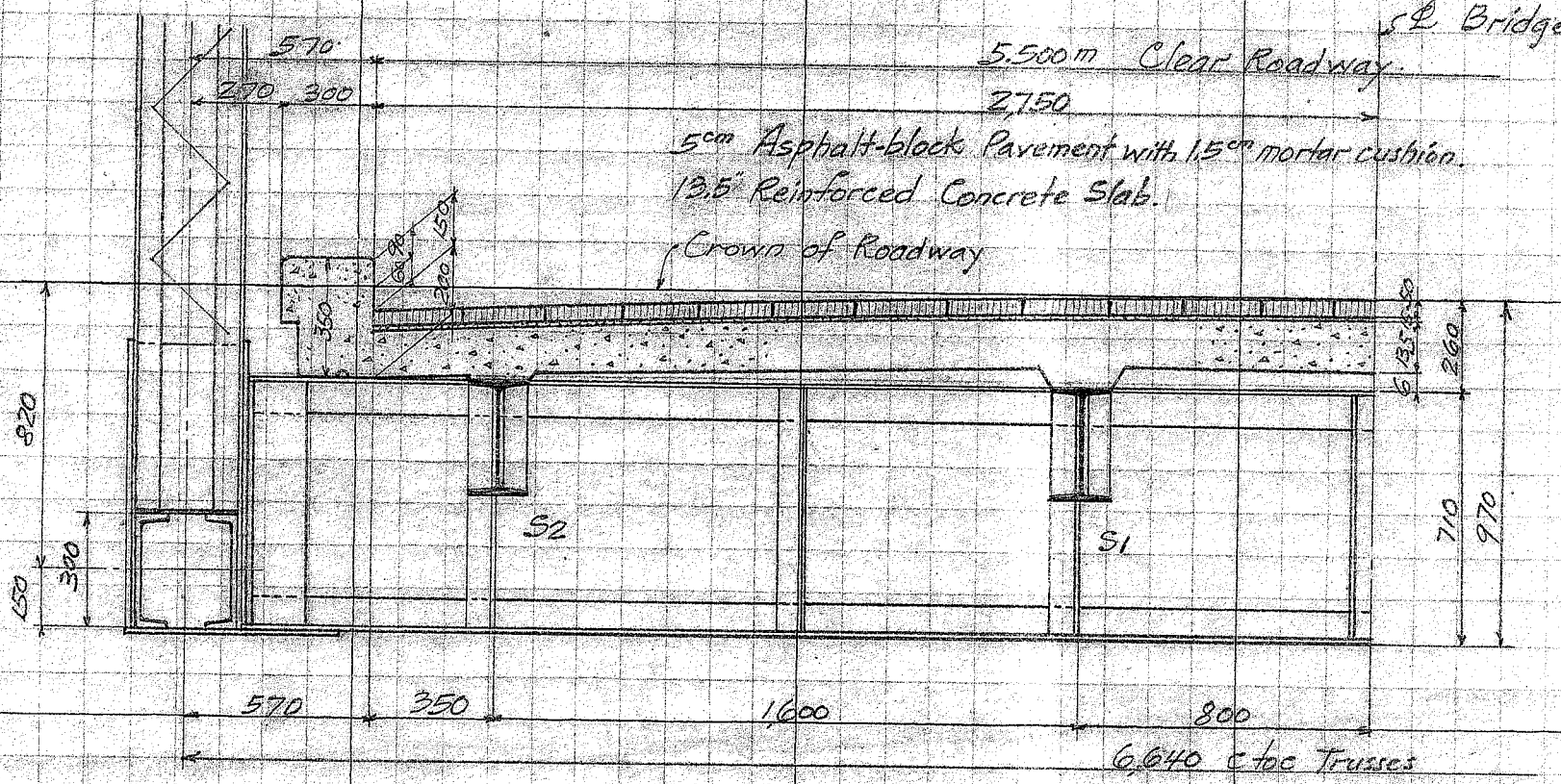
縣道六十米橋桁（ワーレン式）



Steel in one span = 25.00 ton. page 23

(detail 35%)

Cross section of the bridge assumed as shown on sketch below.



Scale 1:20.

Design of Floor Slab. Span length = 1.60 meters.

Dead load:-

Asphalt-block pavement	5cm @ 21 kg	=	105.0
Cement mortar cushion	1.5 @ 17	=	25.5
Reinforced concrete slab	13.5 @ 24	=	324.0
Fillets and misc. concrete say		=	10.5
			465.0 kg per sq. meter.

Dead load moment = 1/10 × 465 × 1.60² = 119 kgm.
Dead load shear = 1/2 × 465 × 1.60 = 372 kg.

Live load:- 6 ton motor truck loading, Rear wheel concentration = 2250
Impact coefficient 30% 675
2925 kg
Front wheel with impact say 2925 ÷ 3 = 975 kg.

Distribution of wheel concentration:

Contact between wheel and pavement	=	20.0
distributions 2@6.5	=	13.0
Longitudinal distribution	a =	33.0 cm
Transverse distribution 6 = 24 + 13	=	37.0 "

Effective width of slab. E = 3/3.l + a where l = span length in meters.
= 3/3 × 1.60 + 0.33 = 1.140 meters.
Load per meter strip = 2925 ÷ 1.140 = 2,090 kg

Uniform live load on roadway assumed as 500 kg per sq. meter.

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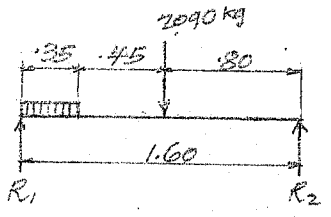
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CALCULATIONS FOR

Moment at center of span.



Reaction R_2

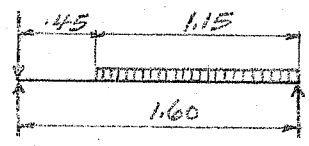
Uniform load	$\frac{500 \times 0.35^2}{2 \times 1.60}$ =	19
rear wheel	2090 ÷ 2 =	1045
		1064 kg

$1064 \times 0.80 = 852$

For continuity of slab, moment may be taken as 80% of the above value.

moment at center = $0.80 \times 852 = 681$ kgm

End shear.



Uniform load	$\frac{500 \times 1.15^2}{2 \times 1.60}$ =	207
rear wheel		2090
		2297 kg

Summary of moments and end shears.

	moment	shear
Dead load	119	372
Live load	681	2297
	800 kgm	2669 kg

Effective depth of slab required for
$f_s = 1200$, $f_c = 45$ kg/cm², $R = 7.13$.

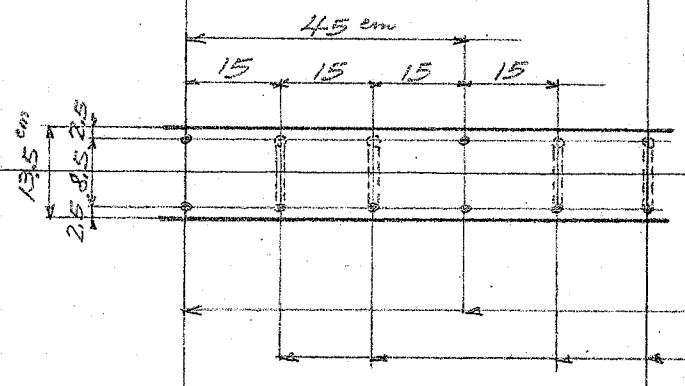
$d = \sqrt{\frac{800 \times 100}{100 \times 7.13}} = 10.6$ cm

Use 11 cm effective depth with 2.5 cm insulation
or 13.5 cm in total.

Steel area required = $\frac{800 \times 100}{1200 \times 7/8 \times 11.} = 6.94$ cm² per meter strip of slab.

Use 12mmϕ bars at 15 cm c to c = 7.54 " " " "

Unit shear = $\frac{2669}{100 \times 7/8 \times 11}$ = 2.77 kg/cm²



12ϕ Straight bars on top & bottom
12ϕ Bent bars (Bent up at each support

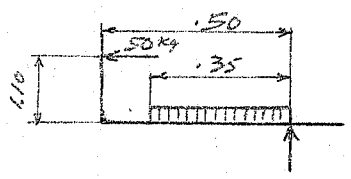
Overhanging slab beyond end stringer.

Dead load :-

Coping	0.35×0.27	@ 2400 =	227 × 0.50 =	114
floor + pav.	0.35	@ 465 =	163 × 0.18 =	29
handrail assumed as			80 × 0.50 =	40
			470 0.39	183

Dead load moment = 183 kgm per meter strip of slab.
Dead load shear = 470 kg " " " " "

Live load :-



moment due to
Uniform load $\frac{500 \times 0.35^2}{2}$ = 31

Thrust on handrail 50 × 1.10 = 55

86 kgm.

Shear = 500 × 0.35 = 175 kg.

Summary of moment & shear.

	moment	shear
Dead load	183	470
Live load	86	175
	269 kgm	645 kg

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CALCULATIONS FOR

Design of I beam stringers:- Span length = 5.00 m, Spacing 1.60 m.

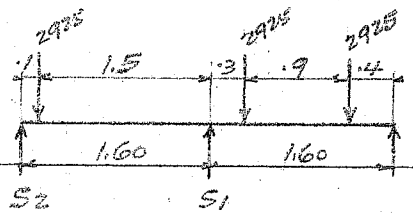
Dead Load:-

	Load on Inside stringer S1		Outside stringer S2.		
Floor slab + pavement	1.60 @ 465 =	744	0.80 @ 465 =	372	
Coping				227	see on page 4
Handrail				80	〃
overhanging slab+par.				163	〃
Overhanging effect	183 ÷ 1.60 =	- 114		114	〃
Beam assumed as		50		50	
		680 kg/m		1006 kg/m	

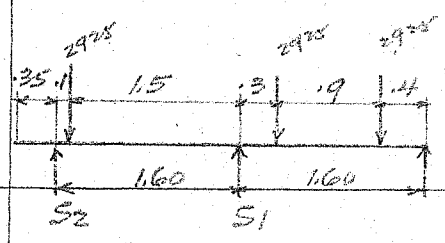
Dead load moment	1/8 × 680 × 5.0² =	2,125	1/8 × 1006 × 5.00² = 3,145 kgm
Dead load end shear	1/2 × 680 × 5.0 =	1,700	1/2 × 1006 × 5.00 = 2,515 kg.

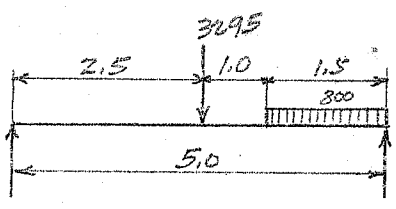
Live Load :— 6 ton motor truck rear wheel concentration with imp. = 2,925 kg

Maximum wheel concentration on stringer.



	Inside stringer S1	Outside stringer S2.
	2,925 × 0.10 = 293	
	2,925 × 0.40 = 1170	
	2,925 × 1.30 = 3800	
	5263	
	5263 ÷ 1.60 = 3295 kg	2925 × 1.5 ÷ 1.6 = 2,740 kg



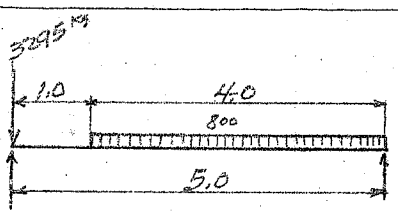


Uniform load on beams.

	500 × 1.60 = 800	500 × 1.15 = 575
		500 × 0.35² / 2 × 1.60 = 20
	800 kg/m	595 kg/m

moment.

	800 × 1.5² / 2 × 5.0 × 2.5 = 450	595 × 1.5² / 2 × 5.0 × 2.5 = 335
	3295 ÷ 2 × 2.5 = 4120	2740 ÷ 2 × 2.5 = 3425
	4,570 kgm	3760 kgm



End shear

	800 × 4.0² / 2 × 5.0 = 1,280	595 × 4.0² / 2 × 5.0 = 952
	3925	2740
	5,205 kg	3692 kg

Summary of moments and shears.

	Inside stringer S1		Outside stringer S2	
	moment	shear	moment	shear
Dead load	2,125	1,700	3,145	2,515
Live load	4570	5205	3760	3692
	6,695 kgm	6,905 kg	6,905 kgm	6,207 kg

Section modulus required: 6695 × 100 / 1100 = 608.5 cm³ ; 6905 × 100 / 1100 = 627.5 cm³

use 1 I beam 300 × 150 × 8 @ 48.3 kg, for both stringers S1 and S2.
Section modulus = 633.2 cm³

Shear on web gross section: 6905 / 30 × 0.8 = 288 kg/cm² ; 6207 / 30 × 0.8 = 259 kg/cm²

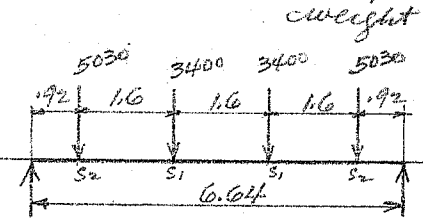
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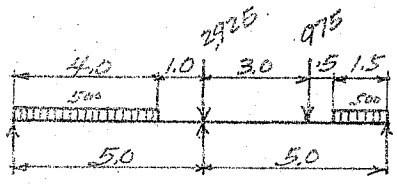
CALCULATIONS FOR

Design of Floor Beams. Span length = 6.64 metres, spacing = 5.00 metres.
Intermediate floor beam:-
Dead load:-
Stringer concentrations on floor beam.
Inside stringer S1. 2@1700 = 3400 kg } 8430
Outside " S2 2@2515 = 5030 "
weight of floor beam assumed as 150 kg per lin. metre.

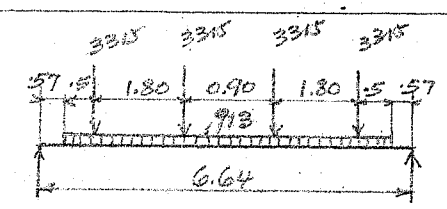


	Dead load moment		Dead load shear
8430 × 2.52	=	21,250	
5030 × 1.60	=	8050	
		13,200	8430
1/8 × 150 × 6.64²	=	830	1/2 × 150 × 6.64 = 500
		14030 kgm	8930 kg

Live load:-



max. wheel concentration on floor beam.
Front wheel 975 × 2.0 ÷ 5.0 = 390
rear " 2925
3315 kg
Uniform load
500 × 1.5² ÷ 2 = 560
500 × 4.0² ÷ 2 = 4000
4560 ÷ 5.0 = 913 kg/m



Live load moment.
6630 × 2.87 = 19020
3315 × 1.80 = - 5970
913 × 2.75 = 2510 × 3.32 = 8330
2510 × 1.375 = - 3450
17930 kgm.

Live load end shear.
wheels. 2@3315 = 6630
" eccentricity 132600 × 0.05 ÷ 6.64 = 100
uniform load 913 × 2.75 = 2510
9240 kg

Summary of moments and shears.

	moment	end shear
Dead load	14030	8930
Live load	17930	9240
	31,960 kgm	18,170 kg

unit shear on web gross section
= 18170 ÷ 63.00 = 288 kg/cm²

Try web pl. 700 × 9 = 63.00 cm², 1/8 wel = 7.88 cm²
Eff. depth say 71.00 - 2@2.22 = 66.56 cm
Flange stress = 31960 × 100 / 66.56 = 48050 kg T or C
Flange area req'd. = 48050 / 1200 = 40.00
1/8 web area, less 7.88
32.12 cm² nett.
or
48050 / 1100 = 43.70
7.88
35.82 cm² gross.

use 2 Ls 125 × 90 × 10 = 41.00 gr - 8.80 = 32.20 cm² net.

Approximate weight of Intermediate floor beam.

Flange	4 Ls	125×90×10	@	16.10	× 6.32	= 407
web	1 Pl.	700 × 9	@	49.46	× 6.32	= 313
Stiff. Ls	14 Ls	90 × 75 × 9	@	11.00	× 0.70 average	= 108
fills	8 Pls	75 × 10	@	5.89	× 0.32	= 15
"	8 Pls	150 × 10	@	11.78	× 0.20	= 19
end conn. Ls	4 Ls	150 × 100 × 12	@	22.40	× 0.69	= 62
fills	4 Pls.	150 × 10	@	11.78	× 0.52	= 24
Rivet heads say						32
						980 kg

980 ÷ 6.34 = 154 kg per lin. m. of beam.

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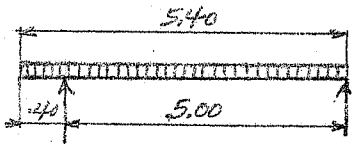
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CALCULATIONS FOR

Design of End Floor Beam. Span length = 6.64 metres, spacing = 5.00 metres c to c.

Dead load:-



Stringer concentration on floor beam.

Inside stringer S_1 $\frac{680 \times 5.4^2}{2 \times 5.0}$ = 1985 kg

Outside " S_2 $\frac{1006 \times 5.4^2}{2 \times 5.0}$ = 2935 " } 4920

Weight of floor beam assumed as 150 kg per lin. metre.

By the similar calculation as for intermediate floor beam (refer to page 6)

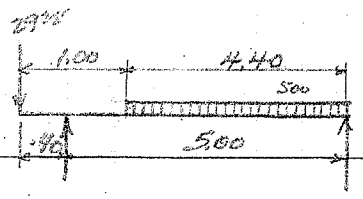
Dead load moment

4920 × 2.52 = 12400
2935 × 1.60 = — 4700
7700
1/8 × 150 × 6.64² = 830
8530 kgm

Dead load shear

4920
1/2 × 150 × 6.64 = 500
5420 kg

Live load:-



max. wheel concentration on floor beams.

Rear wheel 2925 × 5.40 ÷ 5.0 = 3160 kg

uniform load. 500 × 4.4 × 2.2 ÷ 5.0 = 970 kg per

live load moment.

6320 × 2.87 = 18150
3160 × 1.80 = — 5690
970 × 2.75 = 2670 × 3.52 = 8860
2670 × 1.375 = — 3670
17650 kgm

live load end shear.

wheels 2@ 3160 = 6320

+ eccentricity

12640 × 0.05 ÷ 6.64 = 95

Unif. load 970 × 2.75 = 2665

9080 kg.

Summary of moments + shears.

	moment	end shear.
Dead load	8530	5420
live load	17650	9080
	26180 kgm	14500 kg

Unit shear on web = $\frac{14500}{63.0}$ = 230 kg/cm² gross section

Dry web 700 × 9 = 63.00 cm², 1/8 web area = 7.88 cm²

Eff. depth = 71.00 − 4.36 = 66.64 cm

Flange stress = $\frac{26180 \times 100}{66.64}$ = 39260 kg T or C

Bottom flange area req'd. = 39260 ÷ 1200 = 32.71
1/8 web area, less 7.88
24.83 cm² net

Top flange area req'd. = 39260 ÷ 1100 = 35.70
1/8 web area, less 7.88
27.82 cm² gross.

Use 2 L 125 × 90 × 9 = 37.08 gr − 8.80 = 28.28 cm² net.

Approximate weight of End floor beam.

Flange	4 Ls	125 × 90 × 9 @	14.60	× 6.32	= 369
web.	1 Pl	700 × 9 @	49.46	× 6.32	= 313
Stiff Ls	14 Ls	90 × 75 − 9 @	11.00	× 0.70 average	= 108
fills	8 Pls	150 × 9 @	10.60	× 0.20	= 17
+	8 Pls	75 × 9 @	5.30	× 0.32	= 14
end conn. Ls	4 Ls	150 × 100 × 12 @	22.40	× 0.69	= 62
fills	4 Pls	150 × 9 @	10.60	× 0.52	= 22
Rivet heads say					32
					937 call this 940 kg

940 ÷ 6.34 = 148 kg per lin. metre of beam.

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CALCULATIONS FOR

Dead load of Bridge :-

Floor — Floor and pavement 5.50 @ 465 = 2,560
Copings 2 @ 227 = 454
handrails 2 @ 80 = 160
3174

Structural steel parts.

Stringers	4 @ 50 =	200	in which load on upper panel pt.
Floor beams	980 ÷ 5.0 =	196	
upper lateral bracing assumed as		70	70
lower " " "		85	
portals, sways + struts say		125	125
main trusses assumed as,	2 @ 650 =	1300	650
		1976	845
Electric wiring, and other fittings say		50	
		5200 kg per lin. m. of bridge.	

Dead panel load = $\frac{5200}{2} \times 5.00$ = 13,000 kg for one truss.

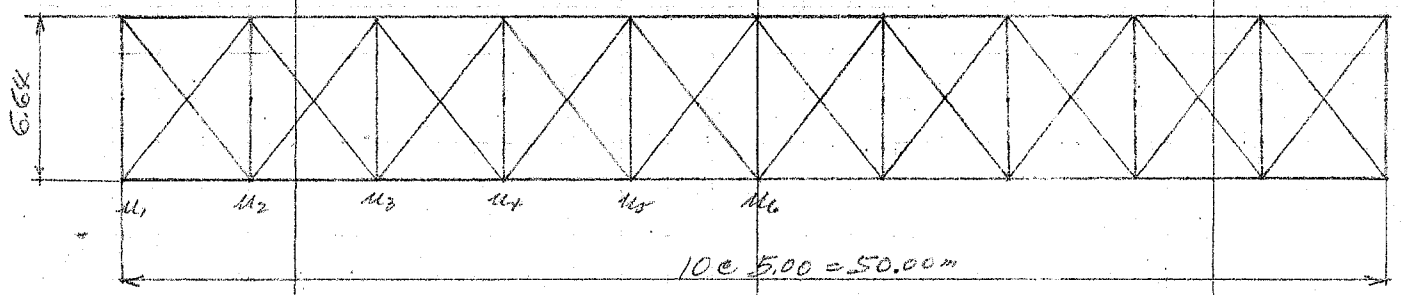
	panel load	Seismic thrust.
upper panel load =	$\frac{845}{2} \times 5.0$ = 2,110 kg	× 0.15 × 2 = 630 kg
lower " " =	$\frac{4355}{2} \times 5.0$ = 10,890 "	× 0.15 × 2 = 3270 "

Design of Lateral Bracings.

Wind load on upper lateral bracing. 5.0 @ 215 = 1075 kg on one panel point.
" " " lower " " 5,0 @ 415 = 2075 " " " " "

Upper lateral bracing.

Moving wind load 1075 kg on each panel point.
Diagonal length = $\sqrt{6.64^2 + 5.0^2}$ = 8.31 meters
Coefficient. = 8.31 ÷ 6.64 = 1.252



Panel			Shear	coef.	Stress		SR.	19φ rivets
U1–U2	1075+10 × 45	=	4840 kg	× 1.252 =	6060	T ÷ 1200 =	5.05 cm² net	7/8 125×75×9 = 34.38 − 7.92 = 26.46 cm²
U2–U3	" " 36		3870	"	4840		4.04	=
U3–U4	" " 28		3010	"	3770		3.14	=
U4–U5	" " 21		2260	"	2830		2.36	=
U5–U6	" " 15		1610	"	2020		1.68	=

use 6 − 19φ rivets for each connection.

Approximate weight of lateral bracing.

Diagonals	40 ls 125-75-9	@	13.50	× 7.65 =	4130
Side conn. say	20	@	15.0	=	300
Centre conn. "	10	@	20	=	200
Rivet heads	say				170
					4800 kg

weight of bracing per lin metre = $\frac{4800}{50}$ = 96 kg

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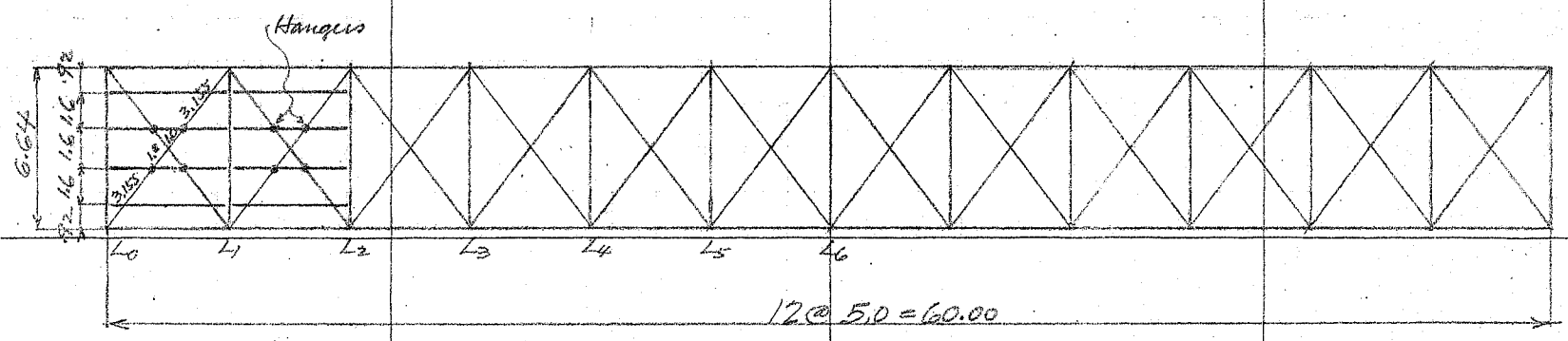
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CALCULATIONS FOR

Lower lateral bracing.

Moving wind panel load = 2,075 kg
Full seismic panel thrust = 3,270 kg

Diagonal length = 8.31 meters.
Coefficient = 1.252



Wind stress of diagonal members.

Panel.			Shear	Coef.	Stress		S.R.	19φ rivet no. req'd.	use.	
L_0-L_1	2,075 ÷ 12 ×	66 =	11,400	× 1.252 =	14,260 T	÷1200 =	11.88 cm²/rivet	6.7	10	4-19φ rivets 2L 125×75×9 = 34.38 - 7.92 = 26.46 cm²
L_1-L_2	"	55	9,520	"	11,910		9.92	5.6	10	"
L_2-L_3	"	45	7,780	"	9,740		8.12	4.6	8	"
L_3-L_4	"	36	6,230	"	7,800		6.50	3.7	6	"
L_4-L_5	"	28	4,840	"	6,060		5.05	2.9	5	L 125×90×9 = 18.54 - 1.98 = 16.56 cm²
L_5-L_6	"	21	3,630	"	4,540		3.79	2.1	4	"

Seismic stress of diagonal members.

Panel							
L_0-L_1	3,270 ×	5.5 =	18,000	× 1.252 =	22,540 T	÷1920 =	11.75 cm²/rivet
L_1-L_2	" ×	4.5	14,710	"	18,420		9.60
L_2-L_3	" ×	3.5	11,450	"	14,330		7.47
L_3-L_4	" ×	2.5	8,180	"	10,240		5.33
L_4-L_5	" ×	1.5	4,910	"	6,150		3.20
L_5-L_6	" ×	0.5	1,640	"	2,050		1.07

Approximate weight of lower lateral bracing.

Diagonals	32 L 125×75×9	@ 13.50 ×	7.80 =	3,370
"	8 L 125×90×9	@ 14.60 "	7.80 =	910
Side connections say	16	@ 20.0	=	320
" "	10	@ 15.0	=	150
Center connections "	8	@ 25.0	=	200
" "	4	@ 15.0	=	60
Hangers "	48	@ 10.0	=	480
Rivet heads etc say				210
				5,700 kg

weight per lin. meter of bracing = 5,700 ÷ 60 = 95. kg.

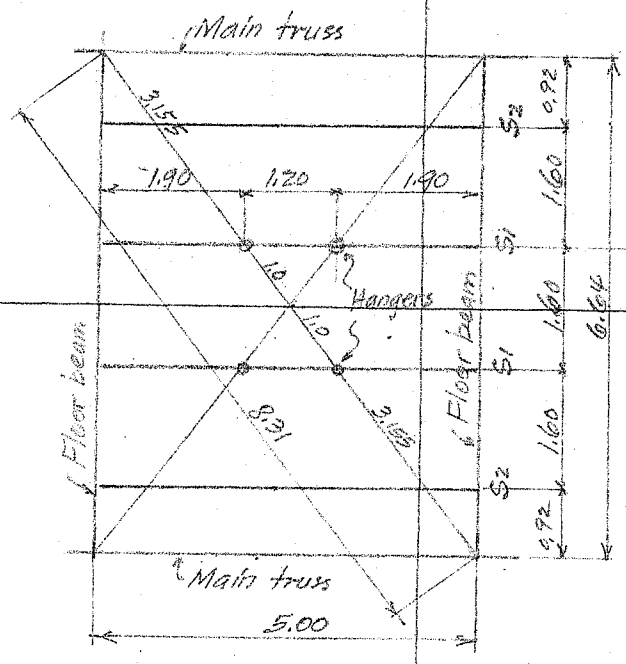
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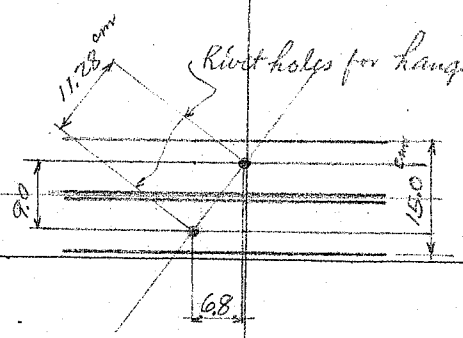
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CALCULATIONS FOR

Check for Strength of Inside stringer S1 at the connection point of lateral hanger.
Dead load :—



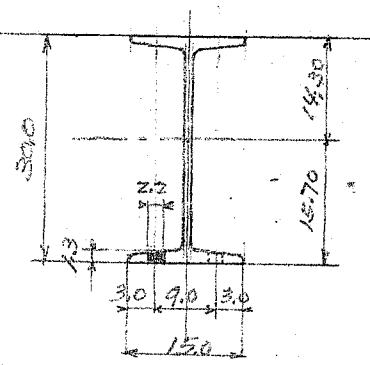


One rivet hole to be reduced from bottom flange of stringer S1.

Stringer S1. I I beam 300×150 @ 48.3 kg
moment of inertia Jx = 9499 cm⁴
Section modulus Zx = 633.2 cm³

for 19φ rivet, reduced area = 1.3×2.2 = 2.86 cm²

Center of gravity of reduced section.



gross section	61.58 × 15.00 =		923.7
rivet hole	− 2.86 × 0.65 =	−	1.9
	58.72	15.70 cm	921.8

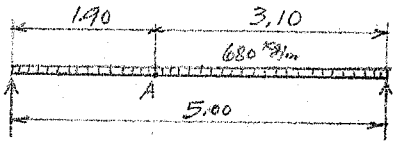
Moment of inertia Jx of reduced section.

gross section		9499
rivet hole	− 2.86 × 15.05² =	− 648
		8851 cm⁴

Section modulus for

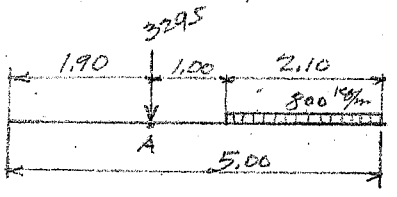
Compression fibre	=	8851 ÷ 14.3 = 619.0 cm³
tension fibre	=	8851 ÷ 15.7 = 564.0 "

Max. moment at point of hanger connection.



Dead load on beam = 680 kg/m (See on page 5)
Dead load moment at A = $\frac{680}{2}$ × 1.90 × 3.10 = 2,000 kgm

Live load :—



max. wheel concentration = 3,295 kg (refer to page 5)
uniform load = 800 kg/m (")
Reaction
wheel 3295 × 3.1 ÷ 5.00 = 2043
unif. load 800 × 2.10 × 1.05 ÷ 5.0 = 353
2,396 kg

Live load moment at A = 2396 × 1.90 = 4,550 kgm
D.L. L.L.
Total moment = 2000 + 4550 = 6,550 kgm at A.

Compression fibre stress = 6550 × 100 ÷ 619.0 = 1,058 kg/cm² < 1,100 ✓
Tension " " = 6550 × 100 ÷ 564.0 = 1,162 " < 1,200 ✓

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JIUN MASUDA
CONSULTING ENGINEER
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CALCULATIONS FOR

Dead load of Bridge:-

Floor.

Floor and pavement	5.50 @ 465 =	2560	
copings	2 @ 227 =	454	
handrails assumed as	2 @ 80 =	160	
			3174

Structural steel parts.

Stringers	4 @ 50 =	200	
floor beam	980 ÷ 5.0 =	196	
upper lateral bracing		96	
Lower lateral "		95	
portals, sways + struts		135	
main truss assumed as	2 @ 650 =	1300	
			2,012

Electric wiring, telephone piping, and other fillings say = 54.

5240 kg/m of bridge

Dead panel load = $\frac{5240}{2} \times 5.00 =$ 13,100 kg for one truss.

Live load for main truss.

Uniform load $w = \frac{100000}{170+60} =$ 435 kg per sq. metre.

or $435 \times 2.75 =$ 1200 kg per lin. meter for one truss.

6.0 ton motor truck rear wheel concentration = 2,250

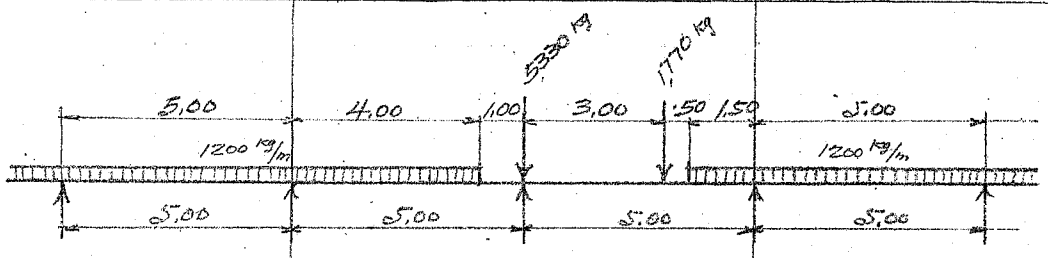
Impact coef. = $\frac{20}{60+60} =$ 16.67% 375

2,625 kg

max. wheel concentration on main truss

$2625 \times 4 \times 3.37 \div 6.64 =$ 5,330 kg

front wheel $5330 \div 3 =$ 1,770 "



Unif. load.	6,000	3,000		3000	6000
"		2,880	1920		
"			270	1530	
rear wheel.			5330		
front "			710	1060	
	6,000	5,880	8230	5,590	6000
		-120	+2,230	-410	

Now, let us assume the panel concentration for live load as 6000 kg on each panel point throughout and 2200 kg extra single concentration for motor truck loading.

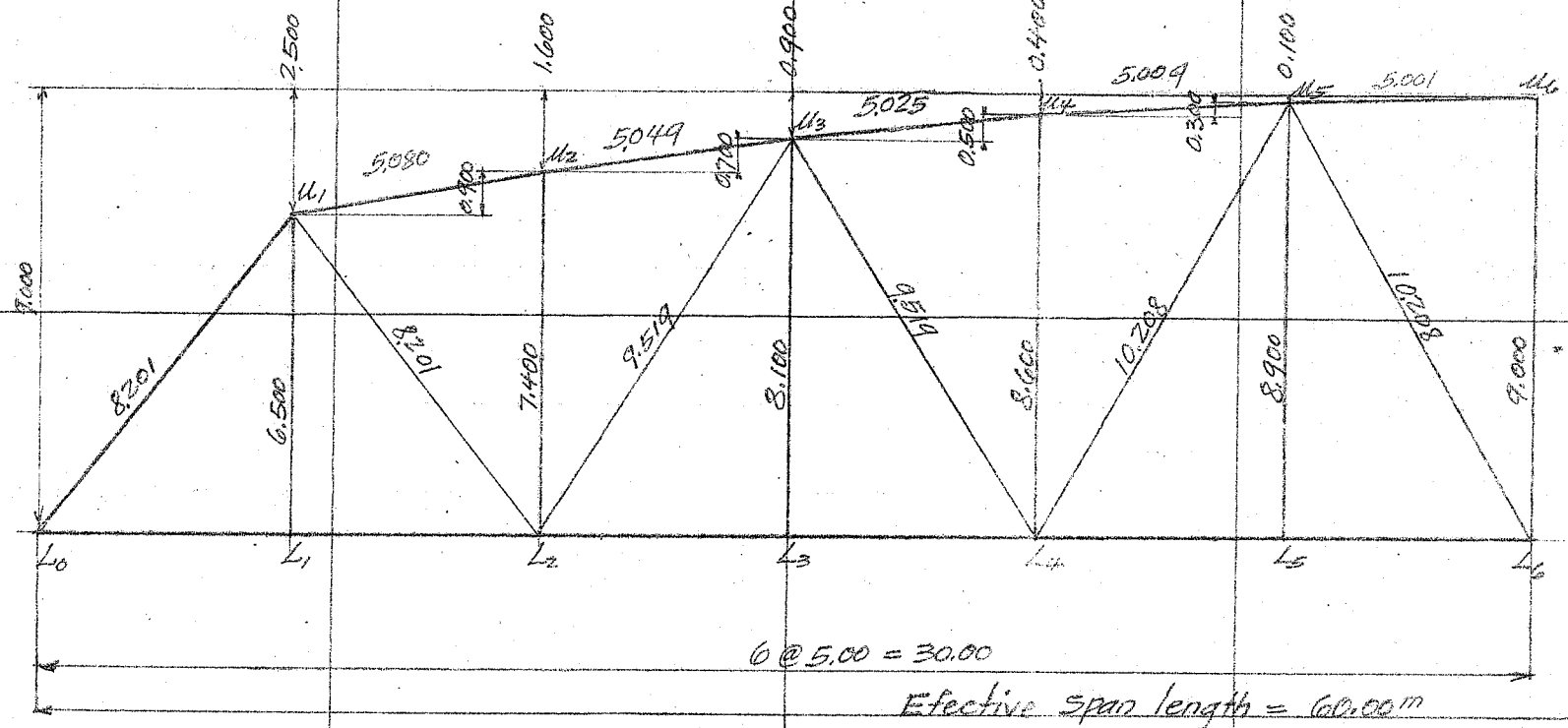
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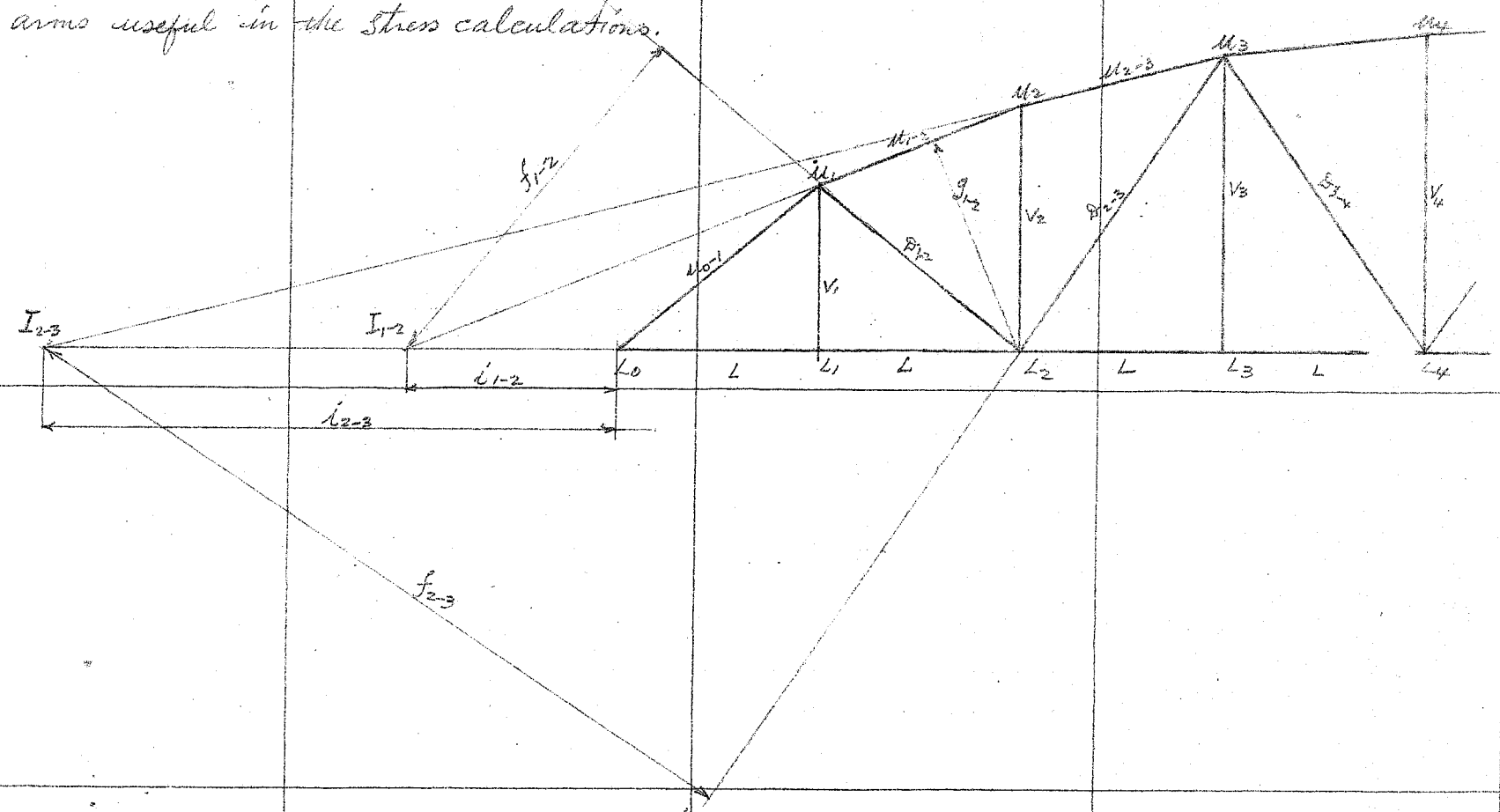
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Design of main truss.



Lever arms useful in the stress calculations.



Distances i

			i
1-2	5.000 × 6.50 ÷ 0.900 =	36.111 − 5.000 =	31.111 m.
2-3	5.000 × 7.40 ÷ 0.700 =	52.857 − 10.000 =	42.857
3-4	5.000 × 8.100 ÷ 0.30 =	81.000 − 15.000 =	66.000
4-5	5.000 × 8.600 ÷ 0.300 =	143.333 − 20.000 =	123.333
5-6	5.000 × 8.900 ÷ 0.100 =	445.000 − 25.000 =	420.000

Lever arms g.

1-2	7.400 × 5.000 ÷ 5.080 =	7.283
2-3	7.400 × 5.000 ÷ 5.049 =	7.328
3-4	8.600 × 5.000 ÷ 5.025 =	8.557
4-5	8.600 × 5.000 ÷ 5.009 =	8.585
5-6	9.000 × 5.000 ÷ 5.001 =	8.998

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CALCULATIONS FOR

Truss arms f.

1–2	31.111	+ 10.000	= 41.111	× 6.500	÷ 8.201	=	32.584
2–3	42.857	+ 10.000	= 52.857	× 8.100	÷ 9.519	=	44.978
3–4	66.000	+ 20.000	= 86.000	× 8.100	÷ 9.519	=	73.180
4–5	123.333	+ 20.000	= 143.333	× 8.900	÷ 10.208	=	124.967
5–6	420.000	+ 30.000	= 450.000	× 8.900	÷ 10.208	=	392.339

Stresses in Truss members.
Dead load stresses.
Reaction R_d = 5.50 @ 13100 = 72050 kg

Top chords.

L_0-U_1	72050 × 8.201 ÷ 6.500 =		90,900 C
U_1-U_2	72050 × 10.000 =	720,500	
	13100 × 5.000 = −	68,500	
		655,000 ÷ 7.283 =	89,900 C
		655000 ÷ 7.328 =	89,400 C
U_2-U_3			
U_3-U_4	72050 × 20.000 =	1441,000	
	13,100 × 30.000 = −	393,000	
		1048 000 ÷ 8.557 =	122,500 C
		1048,000 ÷ 8.585 =	122,200 C
U_4-U_5			
U_5-U_6	72050 × 30,000 =	2,161,500	
	13,100 × 75,000 = −	982,500	
		1179,000 ÷ 8.998 =	131,000 C

Bottom chords.

L_0-L_2	72050 × 5.000 =		360250 ÷ 6.500 = 55400 T
L_2-L_4	72050 × 15.000 =	1080,750	
	13100 × 15.000 = −	196500	
		884250 ÷ 8.100 =	109200 T
L_4-L_6	72050 × 25,000 =	1801,250	
	13,100 × 50.000 = −	655000	
		1146250 ÷ 8.900 =	128,800 T

Diagonals.

U_1-L_2	72050 × 31.111 =	2241,000	
	13,100 × 36.111 = −	473000	
		1768000 ÷ 32.584 =	54300 T
L_2-U_3	72,050 × 42.857 =	3086000	
	13,100 × 100.714 = −	1319000	
		1767000 ÷ 44978 =	39300 C
U_3-L_4	720 50 × 66.000 =	4755000	
	13100 × 228.000 = −	2985000	
		1770000 ÷ 73.180 =	24200 T
L_4-U_5	720,50 × 123.333 =	8890,000	
	13100 × 543.332 = −	7120000	
		1770,000 ÷ 124.967 =	14200 C
U_5-L_6	72050 × 420,000 =	30250,000	
	13,100 × 2175,000 = −	28500000	
		1750,000 ÷ 392.339 =	4,500 T

Hangers

U_1-L_1	Same as one panel load	=	13,100 T
U_3-L_3	〃		13,100 T
U_5-L_5	〃		13,100 T

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Verticals.

U_2-L_2	89,900 × 0.900 ÷ 5.080 =	15900	
	89,400 × 0.700 ÷ 5.049 = −	12400	
			3,500 T
U_4-L_4	122,500 × 0.500 ÷ 5.025 =	12,200	
	122,200 × 0.300 ÷ 5.009 = −	7300	
			4,900 T
U_6-L_6	131,000 × 0.100 ÷ 5.001 =	2600	
	131,000 × 0.100 ÷ 5.001 = +	2600	
			5,200 T

Live load stresses due to uniform panel load of 6000 kg
Ratio of this panel load to dead panel load = 6000 ÷ 13100 = 0.458

Top chords.	D.L. stress	load ratio		
L_0-U_1	90,900	× 0.458	=	41,600 C
U_1-U_2	89,900	"		41,200 C
U_2-U_3	89,400	"		41,000 C
U_3-U_4	122,500	"		56,100 C
U_4-U_5	122,200	"		56,000 C
U_5-U_6	131,000	"		60,000 C

Bottom chords.				
L_0-L_2	55400	× 0.458	=	25400 T
L_2-L_4	109,200	"		50,000 T
L_4-L_6	128,800	"		59,000 T

Live load reaction due to partial loading.

Loads on					R_L	R_R
L_1'	1	÷ 12 =	0.0833	× 6,000 =	500 kg	5,500
L_1' to L_2'	3	"	0.2500	"	1,500	10,500
L_1' to L_3'	6	"	0.5000	"	3000	15000
L_1' to L_4'	10	"	0.8333	"	5000	19,000
L_1' to L_5'	15	"	1.2500	"	7,500	22,500
L_1' to L_6	21	"	1.7500	"	10,500	25,500
L_1' to L_5	28	"	2.3333	"	14,000	28,000
L_1' to L_4	36	"	3.0000	"	18,000	30,000
L_1' to L_3	45	"	3.7500	"	22,500	31,500
L_1' to L_2	55	"	4.5833	"	27,500	32,500
L_1' to L_1	66	"	5.5000	"	33,000	33,000

Diagonals.

U_1-L_2	27,500	× 31.111 ÷	32.584	=	26,300 T
L_2-U_3	22,500	× 42.857 ÷	44.978		21,400 C
U_3-L_4	18000	× 66.000 ÷	73.180		16,200 T
L_4-U_5	14,000	× 123.333 ÷	124.967		13,800 C
U_5-L_6	10500	× 420.000 ÷	392.339		11,200 T
Counter stress.					
U_1-L_2	5,500 × 31.111 =	171,200			
	6000 × 36.111 = −	216700			
				− 45,500 ÷ 32.584 =	1,400 C
L_2-U_3	10,500 × 42.857 =	450,000			
	6000 × 100.714 = −	604000			
				− 154.000 ÷ 44.978 =	3,400 T
U_3-L_4	15,000 × 66.000 =	990,000			
	6000 × 228.000 = −	1368000			
				− 378.000 ÷ 73.180 =	5,200 C

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$L_4 - U_5$	19,000 × 123.333 = 2344000			
	6000 × 543.332 = − 3260000			
		− 916,000 ÷ 124.967 =	7,300 T	
$U_5 - L_6$	22,500 × 420.000 = 9450000			
	6000 × 2,175.000 = − 13050.000			
		− 3,600,000 ÷ 392.339 =	9,200 C	
Verticals.				
$U_2 - L_2$	41,200 × 0.900 ÷ 5,080 =	7,300		
	41,000 × 0.700 ÷ 5,049 =	− 5700		
			1600 T	
$U_4 - L_4$	56,100 × 0.500 ÷ 5,025 =	5600		
	56,000 × 0.300 ÷ 5,009 =	− 3300		
			2,300 T	
$U_6 - L_6$	60,000 × 0.100 ÷ 5,001 =	1,200		
	60,000 × 0.100 ÷ 5,001 =	+ 1,200		
			2400 T	
Hangers				
$U_1 - L_1$	Correspond to the max. floor beam concentration due to wheel loads + uniform load		9,200 T	(see on page 6)
$U_3 - L_3$			9,200 T	
$U_5 - L_5$			9,200 T	

Live load stresses due to extra single concentration of 2,200 kg.

Reaction due to extra concentration load on		R_L	R_R
L_1	2,200 ÷ 12 × 11 =	2020 kg	180 kg
L_2	" " 10	1830	370
L_3	" " 9	1650	550
L_4	" " 8	1470	730
L_5	" " 7	1280	920
L_6	" " 6	1100	1100

Top chords.							
$L_0 - U_1$	2020	× 8.201 ÷ 6.500 =				2,600 C	
$U_1 - U_2$	1830	× 10.000 ÷ 7.283 =				2,500 C	
$U_2 - U_3$	"	" ÷ 7.328 =				2,500 C	
$U_3 - U_4$	1470	× 20.000 ÷ 8.557 =				3,400 C	
$U_4 - U_5$	"	" ÷ 8.585 =				3,400 C	
$U_5 - U_6$	1100	× 30.000 ÷ 8.998 =				3,700 C	
Bottom chords							
$L_0 - L_2$	2020	× 5,000 ÷ 6.500 =				1,600 T	
$L_2 - L_4$	1,650	× 15,000 ÷ 8,100 =				3,100 T	
$L_4 - L_6$	1280	× 25,000 ÷ 8,900 =				3,600 T	
Diagonals						Counter stress.	
$U_1 - L_2$	1,830	× 31.111 ÷ 32,584 =	1800 T	180 × 91.111 ÷ 32.584 =		500 C	
$L_2 - U_3$	1,650	× 42.857 ÷ 44.978 =	1,600 C	370 × 102.857 ÷ 44.978 =		800 T	
$U_3 - L_4$	1470	× 66.000 ÷ 73,180 =	1,300 T	550 × 126.000 ÷ 73,180 =		1,000 C	
$L_4 - U_5$	1280	× 123.333 ÷ 124.967 =	1,300 C	730 × 183.333 ÷ 124.967 =		1,100 T	
$U_5 - L_6$	1,100	× 420.000 ÷ 392.339 =	1,200 T	920 × 480.000 ÷ 392.339 =		1,100 C	
Verticals							
$U_2 - L_2$	2500	× 0.900 ÷ 5,080 =	440				
	2,500	× 0.700 ÷ 5.049 = −	350				
			in round no.			100 T	

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Verticals

$M_4 - L_4$ 3400 × 0.500 ÷ 5.025 = 340
3400 × 0.300 ÷ 5.009 = − 200
in round no. 100 T

$M_6 - L_6$ 3700 × 0.100 ÷ 5.001 = 70
3700 × 0.100 ÷ 5.001 = + 70
in round no. 100 T

Summary of Dead Load and live load stresses.

Members.		Dead load Stress	Live Load stresses Unif. load	Extra con.	Total.	Summary of Stresses
Top chords	$L_0 - M_1$	90,900 C	41,600 C	2,600 C	44,200 C	135,100 C
	$M_1 - M_2$	89,900 C	41,200 C	2,500 C	43,700 C	133,600 C
	$M_2 - M_3$	89,400 C	41,000 C	2,500 C	43,500 C	132,900 C
	$M_3 - M_4$	122,500 C	56,100 C	3,400 C	59,500 C	182,000 C
	$M_4 - M_5$	122,200 C	56,000 C	3,400 C	59,400 C	181,600 C
	$M_5 - M_6$	131,000 C	60,000 C	3,700 C	63,700 C	194,700 C
Bottom chords	$L_0 - L_2$	55,400 T	25,400 T	1,600 T	27,000 T	82,400 T
	$L_2 - L_4$	109,200 T	50,000 T	3,100 T	53,100 T	162,300 T
	$L_4 - L_6$	128,800 T	59,000 T	3,600 T	62,600 T	191,400 T
Diagonals	$M_1 - L_2$	54,300 T	26,300 T	1,800 T	28,100 T	82,400 T
			1,400 C	500 C	1,900 C	—
	$L_2 - M_3$	39,300 C	21,400 C	1,600 C	23,000 C	62,300 C
			3,400 T	800 T	4,200 T	—
	$M_3 - L_4$	24,200 T	16,200 T	1,300 T	17,500 T	41,700 T
			5,200 C	1,000 C	6,200 C	—
	$L_4 - M_5$	14,200 C	13,800 C	1,300 C	15,100 C	29,300 C
			7,300 T	1,100 T	8,400 T	—
	$M_5 - L_6$	4,500 T	11,200 T	1,200 T	12,400 T	16,900 T
		2/3 × 4,500 T	9,200 C	1,100 C	10,300 C	7,300 C
Verticals	$M_2 - L_2$	3,500 T	1,600 T	100 T	1,700 T	5,200 T
	$M_4 - L_4$	4,900 T	2,300 T	100 T	2,400 T	7,300 T
	$M_6 - L_6$	5,200 T	2,400 T	100 T	2,500 T	7,700 T
Hangers	$M_1 - L_1$	13,100 T			9,200 T	22,300 T
	$M_3 - L_3$	13,100 T			9,200 T	22,300 T
	$M_5 - L_5$	13,100 T			9,200 T	22,300 T

Max. load on shoe.

Dead load 5.50 @ 13,100 = 72,100
Live load unif. 5.50 C 6000 = 33000
extra con. 2,200
35,200
107,300 kg on one shoe.

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CALCULATIONS FOR

Stresses and Sections of Truss members.

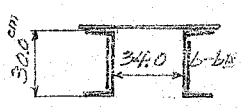
U_1-U_3.

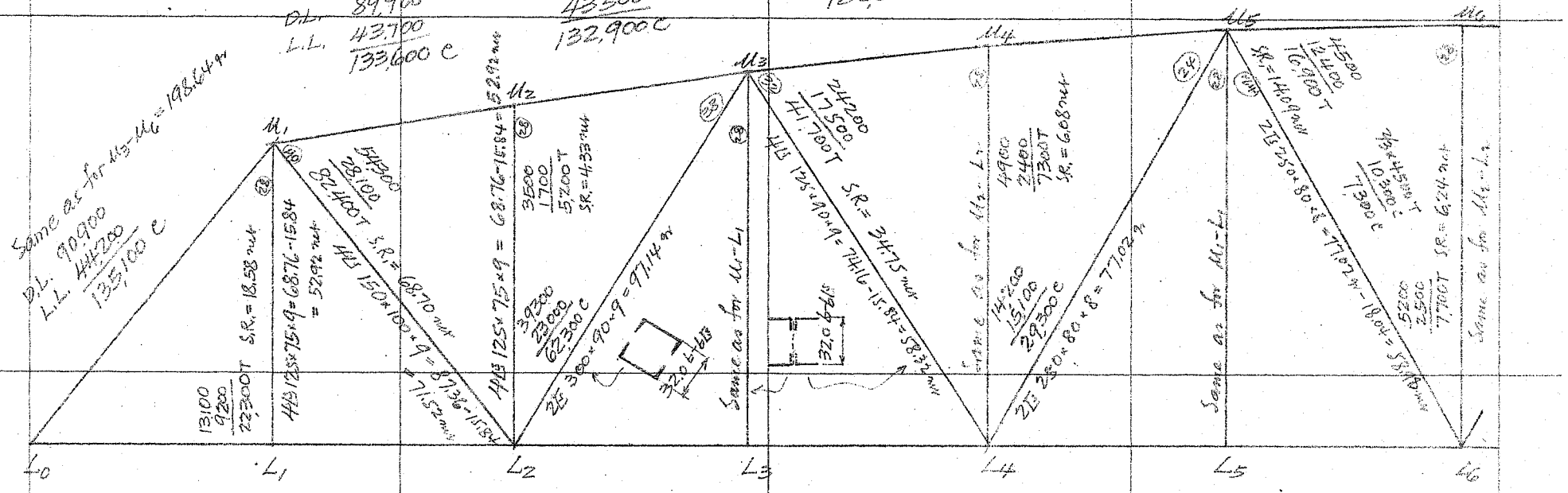
1 cov. pl.	530×11	= 58.30
2 L	300×90×9	= 97.14
		155.44 qr

Rivets 19 mm φ.
Guss. Pls. 9 mm thick

U_3-U_6

1 cov. pl.	530×11	= 58.30
2 L	300×90×9	= 97.14
2 Pls.	240×9	= 43.20
		198.64 qr





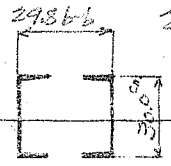
D.L. 55,400
L.L. 27,000
82,400 T S.R. = 68.70 mm

109,200
53,100
162,300 T S.R. = 135.25 mm

128,800
62,600
191,400 T. S.R. = 159.50 mm

L_0-L_2

2 L 300×90×9 = 97.14 − 19.36 = 77.78 mm

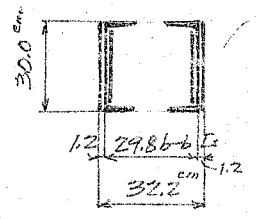


L_2-L_4

2 L 300×90×9 = 97.14 − 19.36 = 77.78
2 Pls 300 ×12 = 72.00 − 10.56 = 61.44
169.14 qr 139.22 mm

L_4-L_6

2 L 300×90×9 = 97.14 − 19.36 = 77.78
2 Pls 300 ×12 = 72.00 − 10.56 = 61.44
2 Pls. 240 × 9 = 43.20 − 7.92 = 35.28
212.34 qr 174.50 mm



Approximate weight of one truss.

members	gross section	length	no.	
L_0 – U_1	198.64	8.201	2	3,260
U_1 – U_3	155.44	10.129	2	3,145
U_3 – U_6	198.64	15.035	2	5,970
L_0 – L_2	97.14	10.000	2	1,943
L_2 – L_4	169.14	10.000	2	3,383
L_4 – L_6	212.34	10.000	2	4,247
U_1 – L_2	87.36	8.201	2	1,433
L_2 – U_3	97.14	9.519	2	1,850
U_3 – L_4	74.16	9.519	2	1,412
L_4 – U_5	77.02	10.208	2	1,573
U_5 – L_6	77.02	10.208	2	1,573
Verticals.	68.76	88.000	1	6050

35,839 @ 0.7850 = 28130
= 9850
37980
Call this 38,000 kg

Details say 35 %

weights of truss per lin. meter = 38000 ÷ 60 = 633.3 kg

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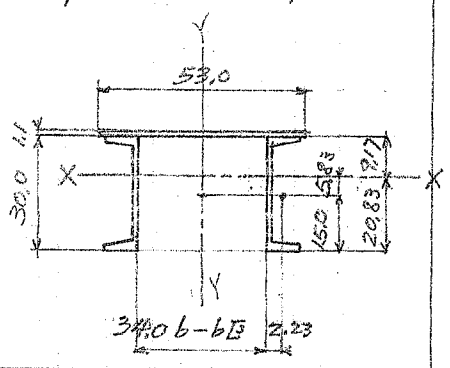
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CALCULATIONS FOR

Determination of section for each member.

Top chord U_1-U_3. 133,600 kg C



assumed section.

1 Cov. pl. 530×11	=	58.30
2□ 300×90×9	=	97.14
		155.44 cm² gr.

Eccentricity.

$\frac{58.30 \times 15.55}{155.44} = 5.83$ cm.

Moments of Inertia.

I_x	1 Pl.	58.30×9.72^2	= 5500
	2□	$97.14 \times 5.83^2 + 2 \times 6435$	= 16170
			I_x = 21670 cm⁴

I_y	1 Pl.	$1.1 \times 53.00^3 \div 12$	= 13650
	2□	$97.14 \times 19.23^2 + 2 \times 325$	= 36600
			I_y = 50250 cm⁴

Radii of Gyration.

$r_x = \sqrt{\frac{21670}{155.44}} = 11.82$ cm

$r_y = \sqrt{\frac{50250}{155.44}} = 17.98$ "

Slenderness ratio. Allowable stress.

$l/r_x = \frac{508}{11.82} = 43.0$, f = 1,000 kg/cm² C

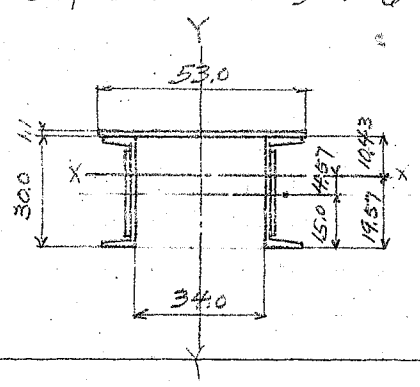
$l/r_y = \frac{508}{17.98} = 28.3$ "

Assumed section is good for compressive stress of

$155.44 \times 1000 = 155,440$ kg 73-19ϕ rivets

Top chord U_3-U_6

194,700 kg C



assumed section.

1 cov. pl. 530 × 11	=	58.30
2□ 300 × 90 × 9	=	97.14
2 Pls. 340 × 9	=	43.20
		198.64 cm² gr.

Eccentricity

$\frac{58.30 \times 15.55}{198.64} = 4.57$ cm

Moments of Inertia.

I_x	1 Pl	58.30×10.98^2	= 7040
	2□	$97.14 \times 4.57^2 + 2 \times 6435$	= 14900
	2 Pls	$43.20 \times 4.57^2 + 1.8 \times 24.0^3 \div 12$	= 2970
			I_x = 24910 cm⁴

I_y	1 Pl.	$1.1 \times 53.00^3 \div 12$	= 13650
	2□	$97.14 \times 19.23^2 + 2 \times 325$	= 36600
	2 Pls	43.20×18.35^2	= 14550
			I_y = 64800 cm⁴

Radii of Gyration.

$r_x = \sqrt{\frac{24910}{198.64}} = 11.20$ cm

Slenderness ratio. allowable stress.

$l/r_x = \frac{502.5}{11.2} = 44.9$, f = 1000 kg/cm² C

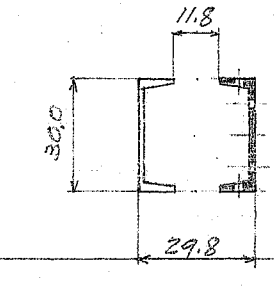
$r_y = \sqrt{\frac{64800}{198.64}} = 18.07$ "

$l/r_y = \frac{502.5}{18.07} = 27.8$ "

Good for $198.64 \times 1000 = 198,640$ kg C 94-19ϕ rivets.

Bottom chord L_0-L_2

82,400 kg T. Section required = 68.70 cm² net.



Assumed section.

2□ 300×90×9 = 97.14 − 19.36 = 77.78 cm² net

This section is good for 77.78 × 1200 = 93,300 kg T 94-19ϕ rivets.

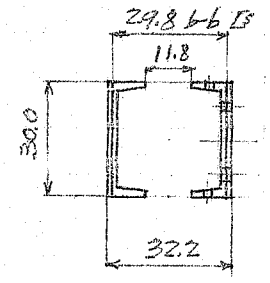
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CALCULATIONS FOR

Bottom chord L2–L4 — 162,300 kg T. — Section required = 135.25 cm² net



Assumed section.

2 Is 300×90×9	=	97.14 − 19.36 = 77.78	
2 Pls 300 ×12	=	72.00 − 10.56 = 61.44	
		169.14 gr	139.22 net

This section is good for 139.22 @ 1200 = 167,000 kg T, 79–19ᵠ rivets

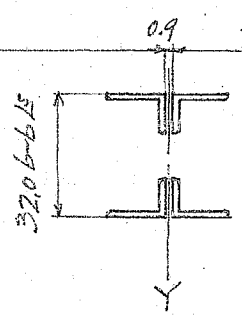
Bottom chord L4–L6 — 191,400 kg T — Section required = 159.50 cm² net

Assumed section.

2 Is 300×90×9	=	97.14 − 19.36 = 77.78	
2 Pls 300 ×12	=	72.00 − 10.56 = 61.44	
2 Pls 240 ×9	=	43.20 − 7.92 = 35.28	
		212.34 gr	174.50 net

This section is good for 174.50 @ 1200 = 209,200 kg T, 98–19ᵠ rivets.

Diagonal M1–L2 — 82,400 kg T — Section required = 68.70 cm² net.

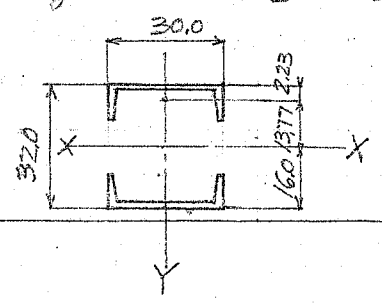


use
4 Ls 150×100×9 = 87.36 − 15.84 = 71.52 cm² net.
good for 71.52 @ 1200 = 85,800 kg T. Connection. 40–19ᵠ rivets
Least radius of gyration r_y = 7.05 cm
Slenderness ratio $l/r_y = \frac{820.1}{7.05} = 116.4 < 200$ ✓

Diagonal M3–L4 — 41,700 kg T. — Section required = 34.75 cm² net

use
4 Ls 125×90×9 = 74.16 − 15.84 = 58.32 cm² net.
good for 58.32 @ 1200 = 70000 kg T, Connection 32–19ᵠ rivets
Least radius of gyration r_y = 5.90 cm
Slenderness ratio $l/r_y = \frac{951.9}{5.90} = 161.3 < 200$ ✓

Diagonal L2–M3 — 62,300 kg C



use
2 Is 300×90×9 = 97.14 cm² gross.
moment of inertia I_x.
$97.14 \times 13.77^2 + 2 \times 325 = 19050$ cm⁴.
$r_x = \sqrt{\frac{190.50}{97.14}} = 14.01$ cm $\quad l/r_x = \frac{951.9}{14.01} = 67.9$
$r_y = 11.51$. $\quad l/r_y = \frac{951.9}{11.51} = 82.6 < 120$ ✓
allowable unit compression.
$f = 1500(1 - 0.0055 \times 82.6) = 818$ kg/cm² C

This section is good for 97.14 @ 818 = 79,400 kg C connection 38–19ᵠ rivets.

Diagonal L4–M5 and M5–L6 — 29300 kg C, 16,900 kg T – SR = 14.09 net
2 Is 250×80×8 = 77.02 − 18.04 = 58.98 cm² net
~~$l/r_y = \frac{1020.8}{9.71} = 105.3$, $f = 1500(1 - 0.0055 \times 105.3) = 631$ kg/cm² C~~

This section is good for 631 × 77.02 = 48,600 kg C connection 24–19ᵠ rivets
1200 × 58.98 = 70,800 " T

Verticals — 22300 kg T — Section required = 18.58 cm² net
4 Ls 125×75×9 = 68.76 − 15.84 = 52.92 net
good for 52.92 @ 1200 = 63,500 kg T. connection 28–19ᵠ rivets

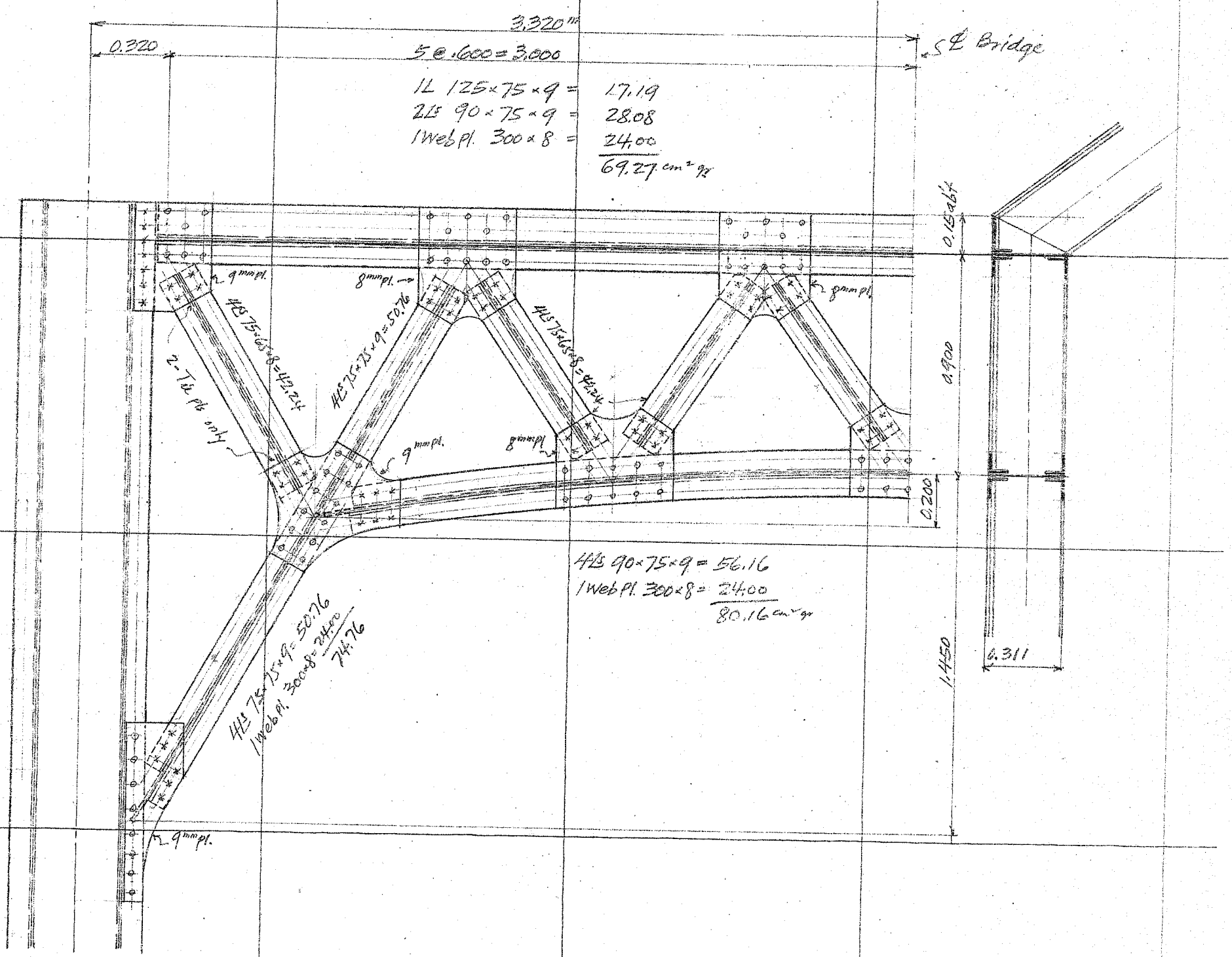
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CALCULATIONS FOR

Design of Portal Bracing



Approximate weight of Portal bracing.

Top chord	69.27	×	6.10		=	423.
bottom chord	80.16	×	4.80		=	385
diagonal	42.24	×	1.00	×2	=	84
"	"	×	0.82	×4	=	137
"	"	×	0.78	×2	=	66
"	50.76	×	1.10	×2	=	112
brace	74.76	×	1.35	×2	=	202
						1409 @ .785 = 1,105
Gusset pls.	8 @ 380×8	@	23.86	× 0.45	=	86.
"	6 @ 400×8	@	25.12	× 0.40	=	60
"	4 @ 330×9	@	23.32	× 0.45	=	42
"	4 @ 450×9	@	31.79	× 0.60	=	76
"	4 @ 230×9	@	16.25	× 0.75	=	49
Tie pls.	18 @ 250×8	@	15.70	× 0.30	=	85
Rivet heads. say						47
						445
						1,550 kg

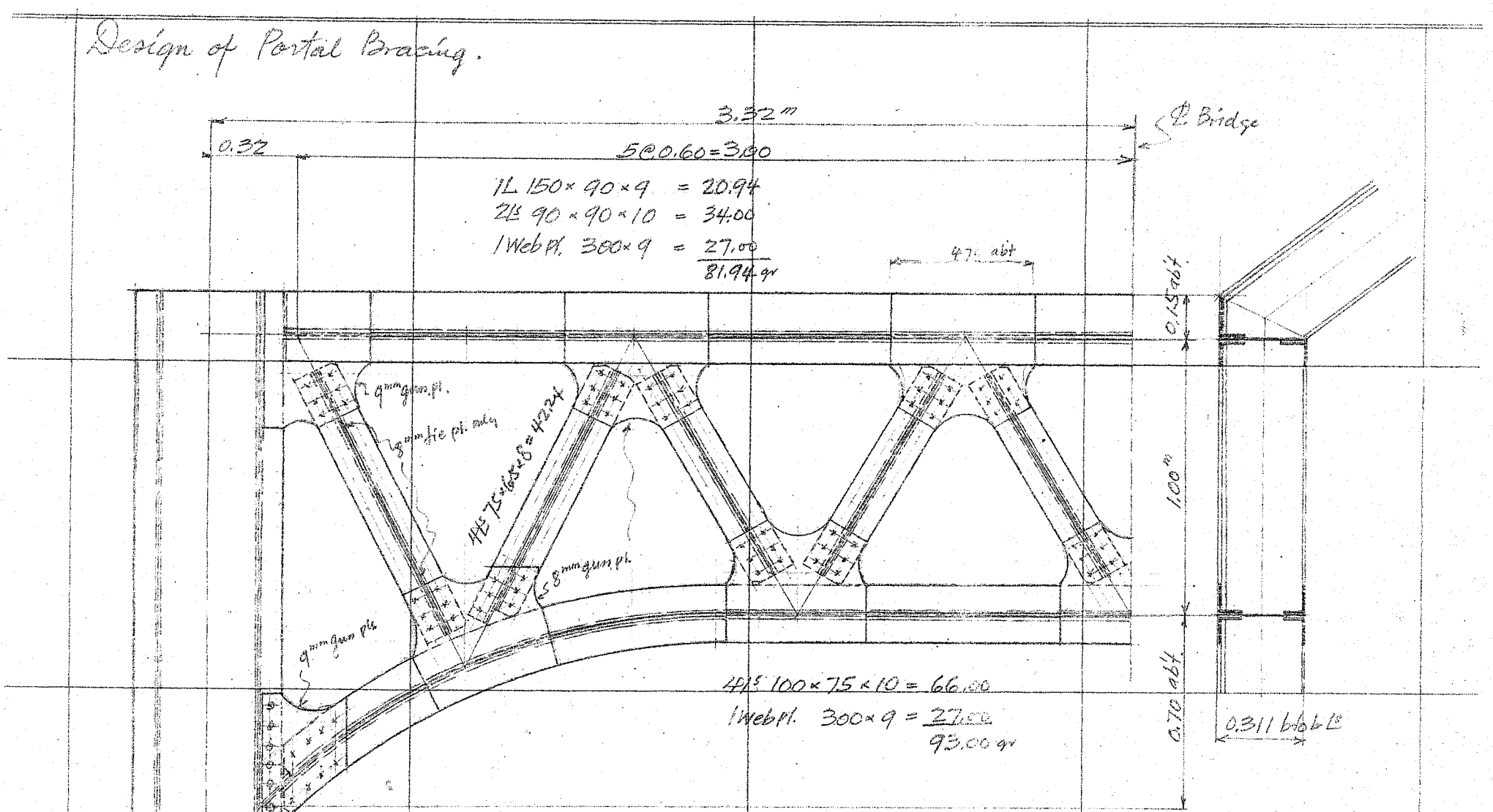
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CALCULATIONS FOR

Design of Portal Bracing.



Approximate weight of Portal Bracing.

Top chord.		81.94	×	6.10	=	500
Bottom chord.		93.00	×	6.50	=	605
Diagonals	2@	42.24	×	1.10	=	93
〃	2@	42.24	×	1.00	=	84
〃	6@	42.24	×	0.86	=	218
						1500 @.78 = 1,178
Details say 35%					=	412
						1590 kg

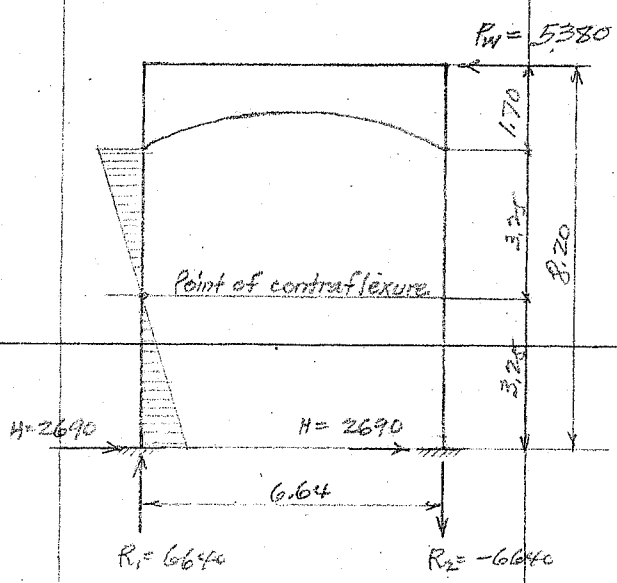
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CALCULATIONS FOR

Section of 3rd post.



Horizontal thrust on portal.

wind thrust $P_w = 5 @ 1075 = 5380$ kg

seismic thrust $P_s = 5 @ 630 = 3150$ "

$$R_1 = \frac{5380 \times 8.20}{6.64} = 6640 \text{ kg}$$

$R_2 = -6640$ "

Max. moment on end post

$= 2690 \times 3.25 = 8740$ kgm

Max. direct compression on end post

D.L.	90900
L.L.	44200
W.L.	6640
	141740 kg c

Direct stress $= \frac{141740}{198.64} = 713$

Moment stress $= \frac{8740 \times 100}{64800} \times \frac{53.0}{2} = 357$

1070 kg/cm²

Slenderness ratio $= \frac{821}{11.2} = 73.3$

Allowable unit compression

$f = 1500 (1 - 0.0055 \times 73.3) = 895$

add 25% = 224

1119 kg/cm² > 1070 ✓

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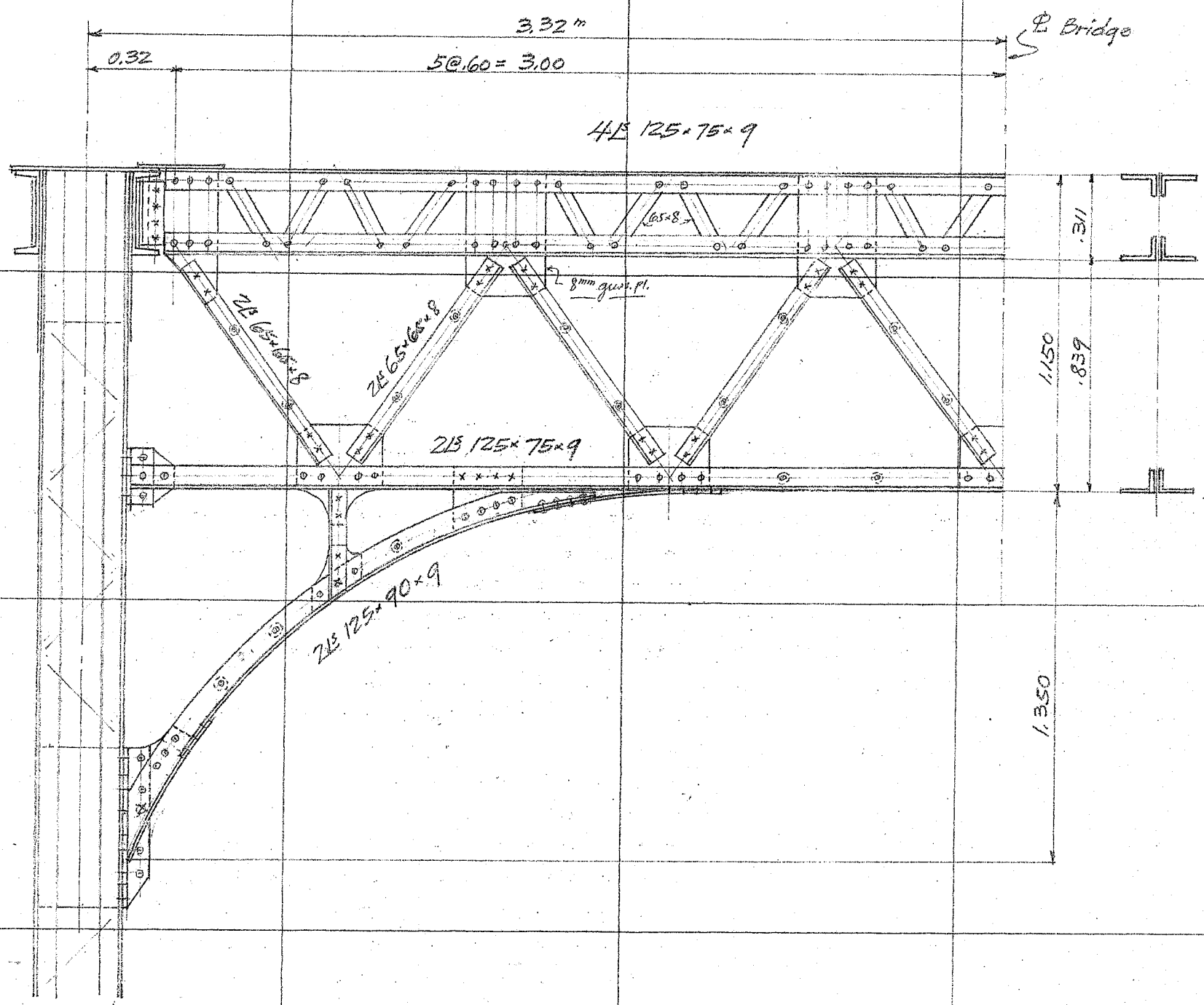
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CALCULATIONS FOR

Sketch of Sway Bracing and strut.



Approximate weight of Sway Bracing.

Top strut	4L	125×75×9	@ 13.50	× 6.07	=	328
Bottom Tie	2L	125×75×9	@ 13.50	× 6.30	=	170
Bracket	4L	125×90×9	@ 14.60	× 2.20	=	129
Diagonals	20L	65×65×8	@ 7.66	× 0.90	=	138
Tie	4L	"	@ "	× 0.40	=	12
	details say 35%				=	272
						1049 kg
			call this			1050 "

Approximate weight of Strut.

4L	125×75×9	@ 13.50	× 6.07	=	328
details say 30%				=	99
					427 kg
		call this			430 "

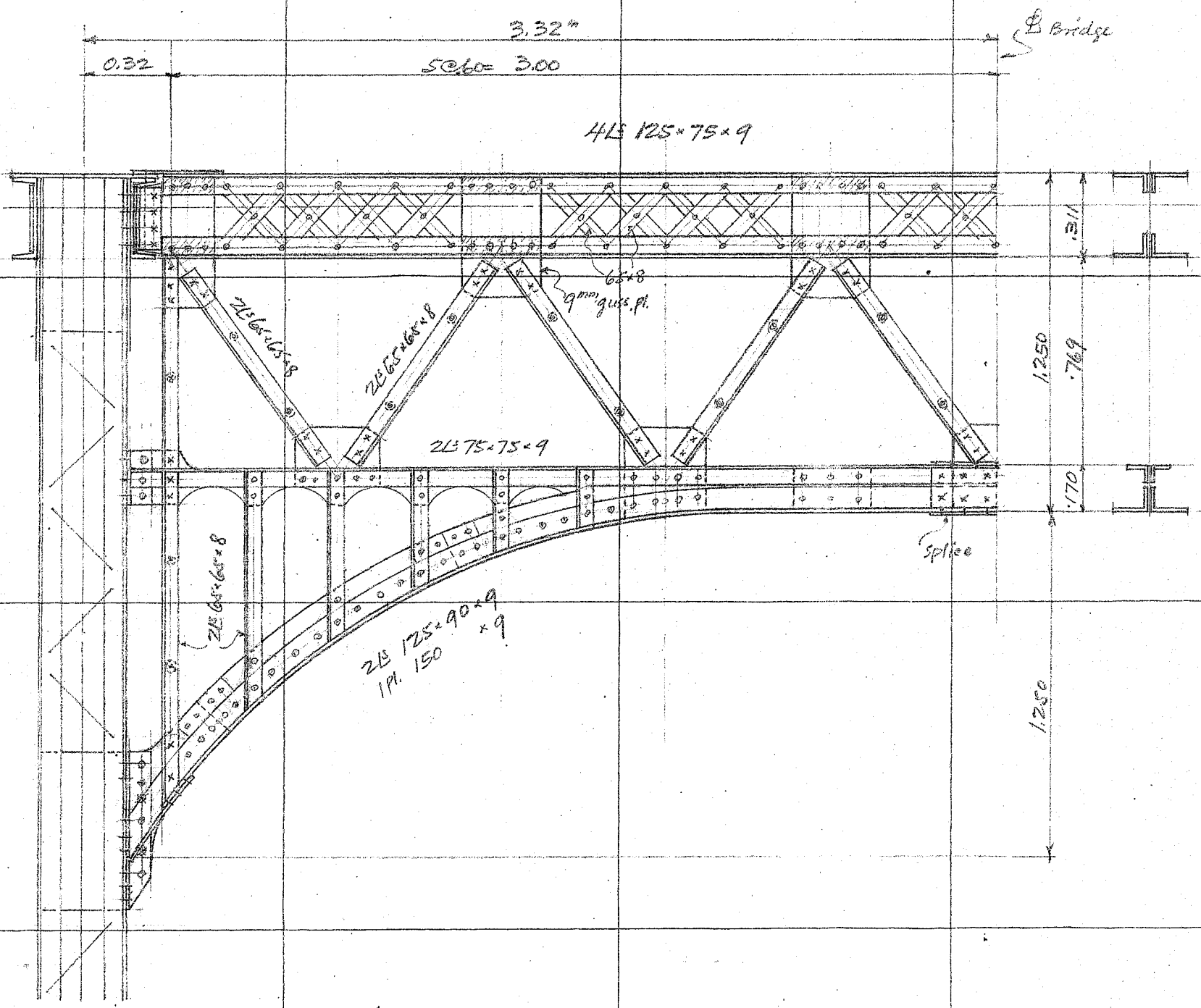
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CALCULATIONS FOR

Sketch of Sway Bracing and Strut.



Approximate weight of Sway Bracing.

Top strut.	4L	125×75×9	@	13.50	× 6.07 =	328.
Bottom "	2L	75×75×9	@	9.96	× 6.30 =	126
" "	2L	125×90×9	@	14.60	× 7.30 =	213
" "	2Pls	220 ×9	@	15.54	× 2.30 =	76
Diagonals	20L	65×65×8	@	7.66	× 0.90 =	138
verticals	4L	65×65×8	@	"	× 2.00 =	61
" average	20Pls	"	@	"	× 0.50 =	77
						1019
	details say 35%				=	357
						1376 call this 1,380 kg

Approximate weight of Strut.

	4L	125×75×9	@	13.50	× 6.07 =	328
	details say 37%				=	122
						450 kg.

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CALCULATIONS FOR

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Summary of Structural Steel in One span.

Stringers	4 @ 50 = 200	× 60.7	=	12,140
floor beams, intermediate	11	@ 980	=	10,780
" end	2	@ 940	=	1,880
upper lateral bracings complete.			=	4800
lower " "			=	5,700
main trusses	2	@ 38,000	=	76,000
portal bracings	2	@ 1,590	=	3,180
sway bracings	4	@ 1,050	=	4,200
struts on top.	5	@ 430	=	2,150
expansion metal say			=	800
shoes say	4	@ 850	=	3400
				125,030 kg
		Call this		125,000 tons.

See page 22A
See page 22A

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CALCULATIONS FOR

Deflection of the truss at center of span.
General equation of the deflection :-

$$\Delta = \Sigma \frac{SL}{EA} \cdot T \quad \text{or} \quad \frac{1}{E} \cdot \Sigma \frac{SL}{A} \cdot T$$

where Δ = The deflection of the truss at any panel point in centimeters;
S = the stress in each member in kilograms;
T = the stress in each member in kilograms, due to a unit load on the panel point at which the deflection is required, in the direction of the deflection;
L = the length of each member in centimeters;
A = the gross sectional area of each member in square centimeters;
E = the modulus of elasticity of steel in kg/cm² = 2,100,000.

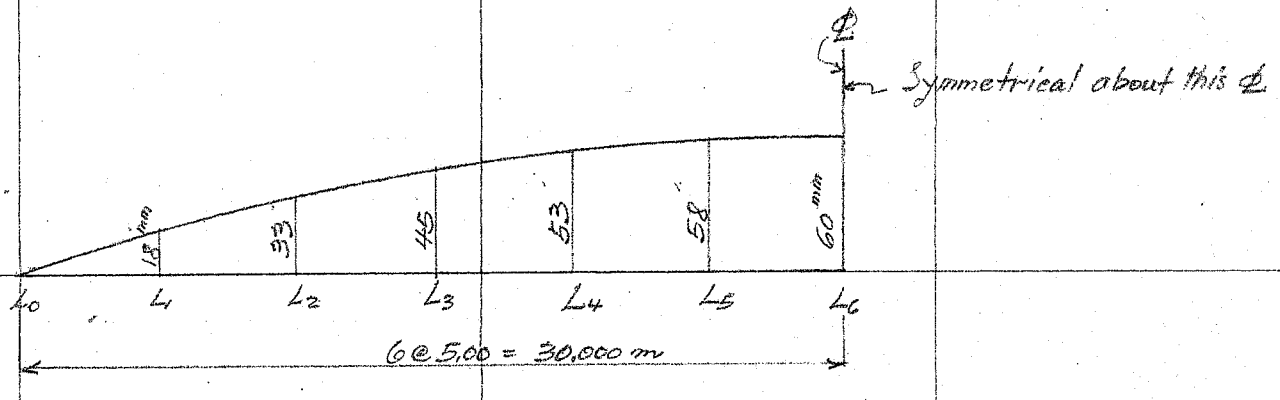
Members.		L	A.	L/A	S_D	T	$\frac{S_D L}{A} \cdot T$
Top chord	$L_0 - U_1$	820.1	198.64	4.131	- 90,900	- 0.631	237,000
	$U_1 - U_2$	508.0	155.44	3.268	- 89,900	- 0.687	201,800
	$U_2 - U_3$	504.9	"	3.250	- 89,400	- 0.683	198,500
	$U_3 - U_4$	502.5	198.64	2.531	-122,500	- 1.168	362,300
	$U_4 - U_5$	500.9	"	2.525	-122,200	- 1.165	359,200
	$U_5 - U_6$	500.1	"	2.520	-131,000	- 1.668	551,000
Bottom chord	$L_0 - L_2$	1000.0	97.14	10.295	55400	0.385	219,600
	$L_2 - L_4$	1000.0	169.14	5.915	109,200	0.926	598,000
	$L_4 - L_6$	1,000.0	212.34	4.712	128,800	1.404	853,000
diagonal	$U_1 - L_2$	820.1	87.36	9.390	54,300	0.478	243,800
	$L_2 - U_3$	951.9	97.14	9.795	- 39,300	- 0.477	183,700
	$U_3 - L_4$	951.9	74.16	12.830	24,200	0.451	140,200
	$L_4 - U_5$	1020.8	77.02	13.250	- 14,200	- 0.494	92,900
	$U_5 - L_6$	1020.8	77.02	13.250	4,500	0.535	31,900
vertical	$U_2 - L_2$	740.0	68.76	10.770	3,500	0.027	1000
	$U_4 - L_4$	860.0	"	12.510	4,900	0.047	2900
	$U_6 - L_6$	900.0 × ½	"	6.550	5,200	0.067	2300
							4279,100

Summary for the entire span = 8,558,200

Dead Load Deflection = 8,558,200 ÷ 2,100,000 = 4.08 cm
Uniform live load " = 4.08 × 0.458 = 1.87 "
5.95 cm.

Deflection ratio = 5.95 ÷ 6000 = $\frac{1}{1,008}$

6.0 cm camber shall be given to the truss.



Camber diagram.

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CALCULATIONS FOR

Dead Load of Bridge :-

Floor	Floor slab and pavement	550 @ 465 =	2560	
	Copings	2 @ 227 =	454	
	Handrails	2 @ 80 =	160	
			3174	
Structural steel parts.				
	Stringers	4 @ 50 =	200	in which load on upper panel pt.
	Floor beams	980 ÷ 5.0 =	196	
	upper lateral bracings assumed as		100	100
	lower lateral bracings "		100	
	portals, sways, struts &c "		140	140
	Main trusses assumed as	2 @ 600 =	1200	600
			1936	840
Electric wirings and other fittings say			50	
			5160 kg per lin. meter of bridge.	

Dead panel load = $\frac{5160}{2} \times 5.00$ = 12,900 kg for one truss.

Seismic thrust.

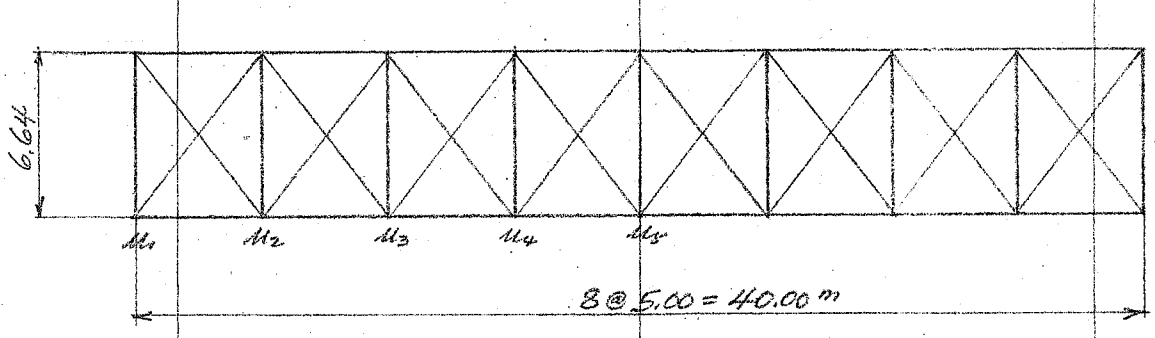
upper panel load	= 420 × 5.00 =	2,100 kg	× 0.15 × 2 =	630 kg
lower " "	= 2160 × 5.00 =	10800 "	× 0.15 × 2 =	3240 "
		12900		

Design of Lateral Bracings.

wind load on upper lateral bracing 5.00 @ 200 = 1000 kg on one panel point.
" " " lower " " 5.00 @ 400 = 2000 " " " "

Upper Lateral Bracings.

Moving wind load of 1,000 kg on each panel point.
Diagonal length = $\sqrt{6.64^2 + 5.00^2}$ = 8.31 meters.
Coefficient = 8.31 ÷ 6.64 = 1.252.



Panel			Shear	Coef.	Stress in diagonal		S.R
U_1-U_2	1,000 kg ÷ 8 ×	28 =	3,500 kg	× 1.252 =	4,380 kg T	÷ 1200 =	3.65 cm²/cut
U_2-U_3	" "	21	2,630	"	3,290 T	"	2.74
U_3-U_4	" "	15	1,880	"	2,350 T	"	1.96
U_4-U_5	" "	10	1,250	"	1,570 T	"	1.31

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撰

導型橋梁

府縣道

橋長三拾丈

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CALCULATIONS FOR

		Shop	Field
End post	L_0-U_1	1552 ~~1968~~	1448 ✓
Top chord	U_1-U_2	~~3360~~ ✗	920 ✓
	U_2-U_3	1184	696 ✓
	U_3-U_4	851	952 ✓
	U_4-U_5	548 ✓	348 ✓
	U_5-U_4	~~548~~ ✗ 420	476 ✓
Bottom chord	L_0-L_1	1,451 ✓	~~760~~ ~~654~~
	L_1-L_2	1,152 ✓	800
	L_2-L_4	1,680 ✓	64 ✓
	L_3-L_4	1,440 ✓	832 ✓
	L_4-L_5	1,632 ✓	64 ✓
Vertical	U_1-L_1	528 ✓	384 ✓
	U_2-L_2	624 ✓	288 ✓
	U_3-L_3	~~592~~	384 ✓
	U_4-L_4	624 ✓	288 ✓
	U_5-L_5	296 ✓	192 ✓
Diagonal	U_1-L_2	~~720~~ ✓	576 ✓
	L_2-U_3	~~720~~ 700	576 ✓
	U_3-L_4	464	384 ✓
	L_4-U_5	464	384 ✓
Diaphragm	DM1	184 ✓	
	DM2	192 ✓	
	DM3	108 ✓	
		17,904	~~10,712~~
		17,772 ✓	11,032 ✓

main section

4e	642.9 =	2,571.6
4e	927.6 =	3,710.4
4e	922.0 =	3,688.0
4e	592.3 =	2,369.2
4e	922.4 =	3,689.6
2e	1,266.0 =	2,532.0
4e	285.0 =	1,140.0
4e	263.5 =	1,054.0
8e	252.7 =	2,021.6
4e	218.0 =	872.0
8e	246.5 =	1,972.0
6e	246.5	1,479.0
		27,094

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CALCULATIONS FOR

Materials of

Structural Steel

End Post $L_0 - U_1$ — Required for one span — 4 - Required

Main section	1 -	Cov. Pl -	540 × 11 - c	46629	×	5120 -	2387		
" "	2 -	E -	300 × 90 - c	38.1	×	5305 -	4042		
Connection at L_0	2 -	L_3 -	150 - 100 - 12 - c	22.4	×	745 -			33.4
" "	2 -	Fills -	145 × 9 - c	10.244	×	370 -			7.6
Splice at U_1	2 -	L_3 -	100 - 100 - 10 - c	14.9	×	610 -			18.2
" " "	2 -	P_1 -	725 × 9 - c	51221	×	1.135 -			1163
" " "	2 -	" -	360 × 9 - c	25434	×	650 -			331
Tie plate	2 -	" -	510 × 9 - c	36032	×	595 -			429
Lacing bar	12 -	bars -	60 × 9 - c	4239	×	710 -			361
Washer	2 -	Wash. -	60$^{\phi}$ - c	222	×	9 -			4
							6429	+	2880 = 9309

Top chord $U_1 - U_3$ — 4 - Required

Main section	1 -	Cov. Pl -	540 × 11 - c	46629	×	7555 -	3523		
" "	2 -	E -	300 × 90 - c	38.1	×	7550 -	5753		
Splice at U_1	1 -	P_1 -	440 × 9 - c	31.086	×	540 -			16.8
" " "	2 -	L_3 -	100 × 100 - 10 - c	14.9	×	610 -			18.2
" " "	2 -	P_{1_3} -	85 × 9 - c	6.005	×	305 -			3.7
" " U_3	1 -	P_1 -	540 × 11 - c	46.029	×	850 -			396
" " "	2 -	P_{1_3} -	85 × 12 - c	8.007	×	850 -			136
" " "	2 -	" -	240 × 9 - c	16956	×	560 -			190
" " "	2 -	" -	665 × 9 - c	46982	×	1.185 -			1113
Tie plate	2 -	" -	510 × 9 - c	36032	×	670 -			483
" "	2 -	" -	370 × 9 - c	26141	×	510 -			267
Lacing bar	16 -	bars -	60 × 9 - c	4239	×	710 -			482
Washer	4 -	Wash. -	60$^{\phi}$ - c	222	×	9 -			8
							9276	+	3462 = 12738

$U_3 - U_5$ — 4 - Required

Main section	1 -	Cov. Pl -	540 × 11 - c	46629	×	7506 -	3500		
" "	2 -	E -	300 × 90 - c	38.1	×	7506 -	5720		
Tie plate	2 -	P_{1_3} -	510 × 9 - c	36032	×	520 -			375
" "	2 -	" -	295 × 9 - c	20842	×	510 -			213
Lacing bar	20 -	bars -	60 × 9 - c	4239	×	685 -			581
Washer	4 -	Wash. -	60$^{\phi}$ - c	222	×	9 -			8
							9220	+	1177 = 10397

Splice at U_5 — 2 - Required

	1 -	P_1 -	540 × 11 - c	46629	×	850 -			396
	2 -	P_{1_3} -	85 × 12 - c	8007	×	850 -			136
	2 -	" -	240 × 9 - c	16956	×	560 -			190
	2 -	" -	635 × 9 - c	44863	×	1.145 -			1027
									1749

Bottom chord $L_0 - L_2$ — 4 - Required

Main section	2 -	L_2 -	100 - 100 - 13 - c	19.1	×	6450 -	2464		
" "	2 -	P_{1_3} -	300 × 12 - c	28260	×	6120 -	3459		
Gusset at L_0	2 -	" -	935 × 9 - c	66058	×	1020 -			1348
" " "	2 -	" -	615 × 12 - c	57933	×	745 -			863
Pin plate	2 -	" -	360 × 11 - c	31.086	×	405 -			252

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SHOWA BLDG, TOKYO

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CALCULATIONS FOR

Materials of

Bent plate	1	Pl	540	9	38.151	755		28.8
Bent plate conn.	1	L	90 90	10	13.0	580		7.5
" " "	1	"	"	"	"	370		4.8
Panel pt. 0	1	Pl	275	11	23.746	510		12.1
" " "	2	Fills	210	12	19.782	270		10.7
" " "	2	L	90 90	10	13.0	325		8.5
Splice	2	Pls	300	12	28.260	505		28.5
"	2	"	85	13	8.674	635		11.0
"	1	Pl	290	9	20.489	805		16.5
Gusset	1	"	340	9	24.021	775		18.6
"	1	"	340	9	"	600		14.4
Filler of diaph.	6	Fills	190	13	19.390	200		23.3
Tie plate	4	Pls	200	9	14.130	290		16.4
" "	1	Pl	290	9	20.489	300		6.1
Spacer	4	Bars	75	9	5.299	298		6.3
Gusset of lateral	1	Pl	325	9	22.961	720		16.5
" " "	1	"	390	9	27.554	580		16.0
connection	1	L	90 75	9	11.0	445		4.9
							5923 +	4972 = 10895

L2 - L4 — 4 - Required

Main section	2	L	100 100	13	19.1	7.500	2865	
" "	2	Pls	300	12	28.260	7.500	4239	
" " "	2	"	200	9	14.130	6815	1926	
Gusset	2	"	775	9	54.754	1.210		1325
"	1	Pl	340	9	24.021	775		18.6
"	1	"	"	"	"	600		14.4
Splice	2	Pls	300	12	28.260	625		353
"	2	"	290	13	29.595	625		370
"	2	Fills	200	4	6.280	310		3.9
"	1	Pl	290	9	20.489	805		16.5
Filler of diaph.	6	Fills	190	4	5.966	200		7.2
Tie plate	4	Pls	200	9	14.130	290		16.4
Spacer	4	Bars	75	9	5.299	280		5.9
Gusset of lateral	1	Pl	580	9	40.977	1.020		41.8
" " "	1	"	390	9	27.554	580		16.0
							9030 +	3455 = 12485

L4 - L4' — 2 - Required

Main section	2	L	100 100	13	19.1	9340	3568	
" "	2	Pls	300	12	28.260	9340	5279	
" "	2	"	200	13	20.410	9340	3813	
Gusset	4	"	775	9	54.754	1.060		2322
"	1	Pl	340	9	24.021	775		18.6
"	1	"	"	"	"	600		14.4
Spacer	4	Pls	75	9	5.299	272		5.8
Tie plate	4	"	200	9	14.130	290		16.4
Gusset	2	"	580	9	40.977	1.020		83.6
"	1	Pl	390	9	27.554	580		16.0
							12660 +	3870 = 16530

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Materials of

		Diagonal $U_1 - L_2$	4 - Required			
Main section	4	L_2 - 125 × 90 × 9 - c	14.6 ' × 4880 '	2850 '		
Tie plate	2	Pl 3 - 310 × 9 - c	21.902 ' × 370 '		162 '	
Lacing bar	19	bars - 60 × 9 - c	4239 ' × 315 '		254 '	
				2850 ' +	41.6 ' =	3266 '
		$L_2 - U_3$	4 - Required			
Main section	4	L_2 - 125 × 75 × 9 - c	13.5 ' × 4880 '	2635 '		
Tie plate	2	Pl 3 - 310 × 9 - c	21.902 ' × 370 '		162 '	
Lacing bar	17	bars - 60 × 9 - c	4239 ' × 350 '		252 '	
				2635 ' +	41.4 ' =	3049 '
		$U_3 - L_4$ & $L_4 - U_5$	8 - Required			
Main section	4	L_3 - 100 × 75 × 10 - c	13.0 ' × 4860 '	2527 '		
Tie plate	2	Pl 3 - 310 × 9 - c	21.902 ' × 370 '		162 '	
Lacing bar	17	bars - 60 × 9 - c	4239 ' × 350 '		252 '	
				2527 ' +	41.4 ' =	2941 '
		Vertical $U_1 - L_1$	4 - Required			
	4	L_3 - 90 × 75 × 9 - c	11.0 ' × 3435 '	151.1 '		
	1	Pl - 310 × 8 - c	19.468 ' × 3435 '	66.9 '		
	2	Pl 3 - 290 × 9 - c	20.489 ' × 305 '		12.5 '	
	2	" - 160 × 9 - c	11.304 ' × 305 '		6.9 '	
				2180 ' +	19.4 ' =	2374 '
		$U_2 - L_2$ & $U_4 - L_4$	8 - Required			
	4	L_3 - 90 × 75 × 9 - c	11.0 ' × 3885 '	170.9 '		
	1	Pl - 310 × 8 - c	19.468 ' × 3885 '	75.6 '		
	2	Pl 3 - 290 × 9 - c	20.489 ' × 305 '		12.5 '	
	2	" - 160 × 9 - c	11.304 ' × 305 '		6.9 '	
	2	" - 190 × 9 - c	13.424 ' × 350 '		9.4 '	
				2465 ' +	28.8 ' =	2753 '
		$U_3 - L_3$ & $U_5 - L_5$	6 - Required			
	4	L_3 - 90 × 75 × 9 - c	11.0 ' × 3885 '	170.9 '		
	1	Pl - 310 × 8 - c	19.468 ' × 3885 '	75.6 '		
	2	Pl 3 - 290 × 9 - c	20.489 ' × 305 '		12.5 '	
	2	" - 160 × 9 - c	11.304 ' × 305 '		6.9 '	
				2465 ' +	19.4 ' =	2659 '
		Diaphragm DM 1	4 - Required			
	4	L_3 - 90 × 75 × 12 - c	14.4 ' × 835 '		48.1 '	
	1	Pl - 290 × 9 - c	20.489 ' × 830 '		17.0 '	
	1	L - 90 × 75 × 9 - c	11.0 ' × 274 '		3.0 '	
	1	Wash. - 60# - c	22.2 ' × 12 '		3 '	
						684 '

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CALCULATIONS FOR

Materials of

	Diaphragm DM2	12 - Required		
4 -	L - 90 × 75 × 9 - c	11.0 "	292 "	12.8 "
1 -	Pl - 260 × 9 - c	18.369 "	292 "	5.4 "
				18.2 "
	Diaphragm DM3	18 - Required		
4 -	L - 90 × 90 × 10 - c	13.3 "	292 "	15.5 "
1 -	Pl - 260 × 9 - c	18.369 "	292 "	5.4 "
2 -	L - 100 × 100 × 10 - c	14.9 "	252 "	7.5 "
2 -	Wash. 60# - c	22.2 "	13 "	6 "
2 -	" - 60# - c	" "	10 "	4 "
				29.4 "
Rivet heads	19# Shop rivet	17,780 @ 0.049 =		871.2 "
	" field rivet	11,040 @ 0.049 =		541.0 "
				1,412.2 "
Summary of main truss for one Span				
End post	$L_0 - U_1$	4 @ 9309 " =		37236 "
Top chord	$U_1 - U_3$	4 @ 12738 " =		50952 "
" "	$U_3 - U_5$	4 @ 10397 " =		41588 "
Splice at U_5		2 @ 1749 " =		3498 "
				133274 "
Bottom chord	$L_0 - L_2$	4 @ 10895 " =		43580 "
" "	$L_2 - L_4$	4 @ 12485 " =		49940 "
" "	$L_4 - L_4$	2 @ 16530 " =		33060 "
				126580 "
Diagonal	$U_1 - L_2$	4 @ 3266 " =		13064 "
"	$L_2 - U_3$	4 @ 3049 " =		12196 "
"	$U_3 - L_4$ & $L_4 - U_5$	8 @ 2941 " =		23528 "
				48788 "
Vertical	$U_1 - L_1$	4 @ 2374 " =		9496 "
"	$U_2 - L_2$ & $U_4 - L_4$	8 @ 2753 " =		22024 "
"	$U_3 - L_3$ & $U_5 - L_5$	6 @ 2659 " =		15954 "
				47474 "
Diaphragm	DM 1	4 @ 68.4 " =		273.6 "
"	DM 2	12 @ 18.2 " =		218.4 "
"	DM 3	18 @ 29.4 " =		529.2 "
				1021.2 "
Rivet heads				1412.2 "
				380450 "

Actual 35.5%
Rivet head 3.8%

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CALCULATIONS FOR

Materials of

		Structural steel End Post L_0-U_1				Required for one span 4-Required				
Main section	1	Cov. Pl	530	×	11	c	45.766	× 7.855	3595	
" "	2	E	300	×	90	c	38.1	× 7.925	603.9	
" "	2	Pl	240	×	9	c	16.956	× 7.915	268.4	
Connection at L_0	2	L	150×100×		9	c	17.1	× 685		23.4
" " "	2	Fills	145	×	12	c	13.659	× 310		11.6
Splice at U_1	2	L	100×100×		10	c	14.9	× 730		21.8
" " "	2	Pl	330	×	9	c	23.315	× 540		25.2
" " "	1	Pl	85	×	14	c	9.342	× 590		5.5
" " "	1	"	530	×	16	c	66.568	× 465		31.0
" " "	2	Pl	870	×	9	c	61.466	× 1.350		166.0
Tie plate	1	Pl	510	×	9	c	36.032	× 630		22.7
" "	1	"		"		c	"	× 595		21.4
Lacing bar	20	bars	65	×	10	c	5.103	× 707		72.2
Washer	4	Wash.	65φ			c	26.0	× 10		1.0
Filler	1	Fill.	75	×	16	c	9.420	× 770		7.3
"	1	"		"		c	"	× 185		1.7
									1231.8	+ 410.8 = 1,642.6
		Top Chord U_1-U_2				4-Required				
Main section	1	Cov. Pl	530	×	11	c	45.766	× 5.140	2352	
" "	2	E	300	×	90	c	38.1	× 5.135	391.3	
Splice at U_1	2	L	100×100×		10	c	14.9	× 730		21.8
" " "	2	Fills	240	×	9	c	16.956	× 355		12.0
" " U_2	2	Pl	550	×	9	c	38.858	× 640		49.7
" " "	2	"	250	×	9	c	17.663	× 510		18.0
" " "	1	Pl	530	×	16	c	66.568	× 1.015		67.6
" " "	2	Pls	85	×	14	c	9.342	× 655		12.2
Tie plate	2	"	510	×	9	c	36.032	× 630		45.4
Lacing bar	12	bars	65	×	10	c	5.103	× 707		43.3
Washer	2	Wash.	65φ			c	26.0	× 10		.5
Filler	1	Fill	75	×	16	c	9.420	× 185		1.7
									626.5	+ 272.2 = 898.7
		U_2-U_3				4-Required				
Main section	1	Cov. Pl	530	×	11	c	45.766	× 5.065	231.8	
" "	2	E	300	×	90	c	38.1	× 5.065	386.0	
Splice at U_3	2	Pls	830	×	9	c	58.640	× 1.100		129.0
" " "	2	"	250	×	9	c	17.663	× 500		17.7
" " "	2	Fills	240	×	12	c	22.608	× 325		14.7
" " "	2	Pls	85	×	14	c	9.342	× 660		12.3
" " "	2	"	530	×	16	c	66.568	× 1.010		134.5
Tie plate	2	"	460	×	9	c	32.499	× 510		33.1
Lacing bar	14	bars	65	×	10	c	5.103	× 707		50.5
Washer	1	Wash.	65φ			c	26.0	× 10		.3
									617.8	+ 392.1 = 1,009.9
		U_3-U_4				4-Required				
Main section	1	Cov. Pl	530	×	11	c	45.766	× 5.310	243.0	
" "	2	E	300	×	90	c	38.1	× 5.310	404.6	
" "	2	Pls	240	×	9	c	16.956	× 5.310	180.1	
Splice at U_4	2	"	245	×	9	c	17.309	× 490		17.0

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CALCULATIONS FOR

Materials of

Splice at U4	2	Pls	540	×	9	℄	38.151	× 630		48.1
" " "	2	"	85	×	14	℄	9.342	× 660		12.3
" " "	1	Pl	530	×	16	℄	66.568	× 1,010		67.2
Tie plate	2	Pls	490	×	9	℄	34.619	× 510		35.3
Lacing bar	14	bars	65	×	10	℄	5.103	× 707		50.5
Washer	2	Wash.	65#			℄	260	× 10		5
									827.7 +	230.9 = 1,058.6
		U4 - U5					4-Required			
Main section	1	cov. Pl	530	×	11	℄	45.766	× 5,025	230.0	
" "	2	∠	300	×	90	℄	38.1	× 5,025	382.9	
" "	2	Pls	240	×	9	℄	16.956	× 5,025	170.4	
Splice at U5	2	"	775	×	9	℄	54.754	× 980		107.3
" " "	2	"	245	×	9	℄	17.307	× 485		16.8
" " "	2	"	85	×	14	℄	9.342	× 660		12.3
" " "	1	Pl	530	×	16	℄	66.568	× 1,010		67.2
Tie plate	2	Pls	490	×	9	℄	34.619	× 510		35.3
Lacing bar	14	bars	65	×	10	℄	5.103	× 707		50.5
Washer	2	Wash.	65#			℄	260	× 10		5
									783.3 +	289.9 = 1,073.2
		U5 - U6					4-Required			
Main section	1	cov. Pl	530	×	11	℄	45.766	× 5,015	229.5	
" "	2	∠	300	×	90	℄	38.1	× 5,015	382.1	
" "	2	Pls	240	×	9	℄	16.956	× 5,015	170.1	
Tie plate	2	"	490	×	9	℄	34.619	× 510		35.3
Lacing bar	14	bars	65	×	10	℄	5.103	× 707		50.5
Washer	2	Wash.	65#			℄	260	× 10		5
									781.7 +	86.3 = 868.0
		Splice at U6					2-Required			
	2	Pls	250	×	9	℄	17.663	× 480		17.0
	2	"	530	×	9	℄	37.445	× 615		46.1
	2	"	85	×	14	℄	9.342	× 660		12.3
	1	Pl	530	×	16	℄	66.568	× 1,010		67.2
										142.6
		Bottom Chord		Lo - L2			4-Required			
Main section	2	∠	300	×	90	℄	38.1	× 8,100	617.2	
Pin plate	2	Pls	905	×	9	℄	63.938	× 1,150		147.1
" "	2	"	590	×	12	℄	55.578	× 855		95.0
" "	2	"	520	×	14	℄	57.148	× 625		71.4
Filler	2	Fills	175	×	12	℄	16.485	× 260		8.6
"	1	Fill	260	×	14	℄	28.574	× 260		7.4
Cover plate	1	Pl	530	×	9	℄	37.445	× 570		21.3
Connection	1	∠	90 × 90 × 10			℄	133	× 400		53
Filler	2	Pls	300	×	12	℄	28.260	× 500		28.3
Gusset	2	"	520	×	9	℄	76.738	× 700		107.4
Filler	2	Fills	300	×	12	℄	28.260	× 520		29.4
Splice	2	Pls	300	×	11	℄	25.905	× 505		26.2
"	2	"	290	×	9	℄	20.489	× 565		23.2
Tie plate	12	"	190	×	9	℄	13.424	× 290		46.7

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Materials of

Tie plate	2	Pl3	290	× 9 ℄	20.489	× 310		12.7
Connection	1	L	90 × 75 × 9 ℄		11.0	× 395		4.3
〃	1	Fill	70	× 14 ℄	7.693	× 200		1.5
Gusset	1	Pl	370	× 9 ℄	26.141	× 655		17.1
〃	1	〃	645	× 9 ℄	45.569	× 810		36.9
							6172	6898 = 13070
		L2 - L3			4 - Required			
Main section	2	I3	300	× 90 ℄	38.1	× 8,700	6629	
〃 〃	2	Pl3	300	× 12 ℄	28.260	× 7,760	4386	
Gusset at L2	2	〃	860	× 9 ℄	60.759	× 1,070		130.0
〃 〃 L3	2	〃	520	× 9 ℄	36.738	× 700		51.4
Splice	2	〃	300	× 14 ℄	32.970	× 755		49.8
〃	2	〃	240	× 14 ℄	26.376	× 625		33.0
〃	2	〃	290	× 9 ℄	20.489	× 565		23.2
Tie plate	12	〃	190	× 9 ℄	13.424	× 290		46.7
Gusset	1	Pl	645	× 9 ℄	45.569	× 810		36.9
〃	1	〃	630	× 9 ℄	44.510	× 755		33.6
							11015	4046 = 15061
		L3 - L5			4 - Required			
Main section	2	I3	300	× 90 ℄	38.1	× 8,695	6626	
〃 〃	2	Pl3	300	× 12 ℄	28.260	× 3,440	4770	
〃 〃	2	〃	240	× 9 ℄	16.956	× 6,160	2089	
Gusset at L4	2	〃	845	× 9 ℄	59.699	× 1,005		120.0
〃 〃 L5	2	〃	520	× 9 ℄	36.738	× 700		51.4
Splice	2	〃	300	× 18 ℄	42.390	× 1,265		107.2
〃	2	〃	300	× 12 ℄	28.260	× 505		28.5
〃	2	〃	240	× 12 ℄	22.608	× 1,015		45.9
〃	2	〃	290	× 9 ℄	20.489	× 565		25.4
Tie plate	11	〃	190	× 9 ℄	13.424	× 290		42.8
Gusset	1	Pl	585	× 9 ℄	41.330	× 700		28.9
〃	1	〃	560	× 9 ℄	39.564	× 675		26.7
							13485	4768 = 18253
		L5 - L5'			2 - Required			
Main section	2	I3	300	× 90 ℄	38.1	× 8,190	6241	
〃 〃	2	Pl3	300	× 12 ℄	28.260	× 7,680	4341	
〃 〃	2	〃	240	× 9 ℄	16.956	× 8,190	2777	
Gusset at L6	2	〃	825	× 9 ℄	58.286	× 1,000		116.6
Tie plate	7	〃	190	× 9 ℄	13.424	× 290		27.3
Gusset	1	Pl	550	× 9 ℄	38.858	× 590		22.9
							13359	1668 = 15027
		Virtical U1 - L1			4 - Required			
Main section	4	I3	125 × 75 × 9 ℄		13.5	× 5,880	3175	
Tie plate	2	Pl3	310	× 9 ℄	21.902	× 390		17.1
Lacing bars	22	bars	60	× 9 ℄	4.239	× 350		32.6
							3175	497 = 3672

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Materials of

		$U_2 - L_2$			4 - Required			
Main section	4	L	125 × 75 × 9	c	13.5 × 7,335	396.1		
Tie plate	2	Pl	310 × 9	c	21.902 × 390		17.1	
Lacing bar	28	bars	60 × 9	c	4.239 × 350		41.5	
						396.1	+ 58.6	= 454.7
		$U_3 - L_3$			4 - Required			
Main section	4	L	125 × 75 × 9	c	13.5 × 8,035	433.9		
Tie plate	2	Pl	310 × 9	c	21.902 × 390		17.1	
" "	1	Pl	"	c	" × 495		10.8	
Lacing bar	29	bars	60 × 9	c	4.239 × 350		43.0	
						433.9	+ 70.9	= 504.8
		$U_4 - L_4$			4 - Required			
Main section	4	L	125 × 75 × 9	c	13.5 × 8,530	460.6		
Tie plate	2	Pl	310 × 9	c	21.902 × 390		17.1	
Lacing bar	34	bars	60 × 9	c	4.239 × 350		50.4	
						460.6	+ 67.5	= 528.1
		$U_5 - L_5$			4 - Required			
Main section	4	L	125 × 75 × 9	c	13.5 × 8,825	476.6		
Tie plate	2	Pl	310 × 9	c	21.902 × 390		17.1	
" "	1	Pl	"	c	" × 495		10.8	
Lacing bar	33	bars	60 × 9	c	4.239 × 350		49.0	
						476.6	+ 76.9	= 553.5
		$U_6 - L_6$			2 - Required			
Main section	4	L	125 × 75 × 9	c	13.5 × 8,925	482.0		
Tie plate	2	Pl	310 × 9	c	21.902 × 390		17.1	
Lacing bar	35	bars	60 × 9	c	4.239 × 350		51.9	
						482.0	+ 69.0	= 551.0
		Diagonal $U_1 - L_2$			4 - Required			
Main section	2	L	150 × 100 × 9	c	17.1 × 7,565	258.7		
" "	2	L	" "	c	" × 7,655	261.8		
Tie plate	2	Pl	310 × 9	c	21.902 × 470		20.6	
Lacing bar	32	bars	60 × 9	c	4.239 × 303		41.1	
						520.5	+ 61.7	= 582.2
		$L_2 - U_3$			4 - Required			
Main section	2	L	300 × 90	c	38.1 × 8,645	658.7		
Tie plate	4	Pl	280 × 9	c	19.782 × 370		29.3	
Lacing bar	72	bars	60 × 9	c	4.239 × 314		95.8	
						658.7	+ 125.1	= 783.8
		$U_3 - L_4$			4 - Required			
Main section	4	L	125 × 90 × 9	c	14.6 × 8,790	513.3		
Tie plate	2	Pl	310 × 9	c	21.902 × 310		13.6	
Lacing bar	38	bars	60 × 9	c	4.239 × 314		50.6	
						513.3	+ 64.2	= 577.5

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Materials of

		L4-U5, U5-L6		8-Required		
Main section	2	E	250 × 80 ℄	30.2 × 9330	5635	
Tie plate	4	Pb	310 × 9 ℄	21.902 × 310	272	
Lacing bar	74	bars	60 × 9 ℄	4239 × 339	1063	
					5635 + 1335 =	6970
		Diaphragm DM1		4-Required		
	4	L	125 × 75 × 9 ℄	13.5 × 855	462	
	1	Pl	290 × 9 ℄	20.489 × 855	175	
	1	L	125 × 90 × 9 ℄	14.6 × 855	125	
	1	Wash.	65# ℄	26.0 × 9	3	
						765
		DM2		12-Required		
	4	L	125 × 75 × 9 ℄	13.5 × 240	130	
	1	Pl	240 × 9 ℄	16.956 × 270	46	
						17.6
		DM3		10-Required		
	4	L	125 × 75 × 9 ℄	13.5 × 240	130	
	1	Pl	240 × 9 ℄	16.956 × 152	26	
						15.6

Summary of main truss for one span

$L_0 - U_1$	4	℄	1,642.6 =	6,570.4
$U_1 - U_2$	4	℄	899.7 =	3,598.8
$U_2 - U_3$	4	℄	1,009.9 =	4,039.6
$U_3 - U_4$	4	℄	1,058.6 =	4,234.4
$U_4 - U_5$	4	℄	1,073.2 =	4,292.8
$U_5 - U_6$	4	℄	686.0 =	2,744.0
Splice at U_6	2	℄	142.6 =	285.2
$L_0 - L_2$	4	℄	1,307.0 =	5,228.0
$L_2 - L_3$	4	℄	1,506.1 =	6,024.4
$L_3 - L_5$	4	℄	1,825.3 =	7,301.2
$L_5 - L_5$	2	℄	1,502.7 =	3,005.4
$U_1 - L_1$	4	℄	367.2 =	1,468.8
$U_2 - L_2$	4	℄	454.7 =	1,818.8
$U_3 - L_3$	4	℄	504.8 =	2,019.2
$U_4 - L_4$	4	℄	528.1 =	2,112.4
$U_5 - L_5$	4	℄	553.5 =	2,214.0
$U_6 - L_6$	2	℄	551.0 =	1,102.0
$U_1 - L_2$	4	℄	582.2 =	2,328.8
$L_2 - U_3$	4	℄	783.8 =	3,135.2
$U_3 - L_4$	4	℄	577.5 =	2,310.0
L4-U5, U5-L6	8	℄	697.0 =	5,576.0
Diaphragm DM1	4	℄	76.5 =	306.0
" DM2	12	℄	17.6 =	211.2
" DM3	10	℄	15.6 =	156.0
				72,078.6

54,996.6 17,082.0 31%

43.4

72,510.0

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Materials of

	End floor beam FB1			2 - Required		
Flange angle	4	L	125 × 90 × 9	c	146 × 6272	3663
Web plate	1	Pl	700 × 9	c	49455 × 6260	3096
End connection	4	L	150 × 100 × 12	c	224 × 692	620
" "	4	Fills	145 × 9	c	10244 × 520	213
Stiffener	8	L	90 × 75 × 9	c	110 × 692	609
"	8	Fills	70 × 9	c	4946 × 520	206
"	6	L	90 × 75 × 9	c	110 × 710	469
Stringer conn.	8	Fills	70 × 9	c	4946 × 270	107
						8983

	Intermediate floor beam FB2 & FB3			11 - Required		
Flange angle	4	L	125 × 90 × 10	c	161 × 6300	4057
Web plate	1	Pl	700 × 9	c	49455 × 6290	3111
End connection	4	L	150 × 100 × 12	c	224 × 690	618
" "	4	Fills	145 × 10	c	11383 × 520	237
Stiffener	8	L	90 × 75 × 9	c	110 × 690	607
"	8	Fills	70 × 10	c	5495 × 520	229
"	6	L	90 × 75 × 9	c	110 × 710	469
Stringer conn.	8	Fills	70 × 10	c	5495 × 270	119
						9447

Summary of floor beam for one span

FB 1	2	c	8983 =	17966
FB 2	11	c	9447 =	103917
				121883

	Stringers S1; $S2_L^R$, S3, $S4_L^R$ & $S5_L^R$			1 - Required		
	48	I	300 × 150	c	483 × 4960	114993
	48	L	90 × 90 × 10	c	133 × 190	1213
	96	"	90 × 75 × 9	c	110 × 240	2534
						118740

	Bracket BR1			1 - Required		
	8	I	300 × 150	c	483 × 355	1372
	8	Pl	150 × 9	c	10598 × 540	458
	8	L	90 × 75 × 9	c	110 × 240	211
						2041

Summary of Stringers and Brackets for one span

S1, $S2_L^R$, S3, $S4_L^R$ & $S5_L^R$	118740	
BR1	2041	
		120781

	Bottom Lateral Bracings			1 - Required		
BL 1	2	L	125 × 75 × 9	c	135 × 7570	2044
"	2	"	"	c	" × 7615	2056
BL 2	2	"	"	c	" × 3630	1016
"	2	"	"	c	" × 3585	968
BL 3	12	"	"	c	" × 3675	5954

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Materials of

BL4	8	L	125 × 75 × 9	13.5 × 7,660		827.3
BL5, BL7	8	〃	〃	〃 × 7,660		827.3
BL6, BL8	16	〃	〃	〃 × 3,675		793.8
BL9, BL12	4	〃	125 × 90 × 9	14.6 × 7,755		452.9
BL10, BL13	4	〃	〃	〃 × 3800		221.9
BL11, BL14	4	〃	〃	〃 × 3,800		221.9
	4	Pls	510 × 9	36.032 × 810		116.7
	2	〃	380 × 9	26.847 × 720		38.7
	2	〃	〃	〃 × 630		33.8
	2	〃	345 × 9	24.374 × 655		31.9
	2	〃	〃	〃 × 565		27.5
	48	〃	190 × 9	13.424 × 390		251.3
						5,048.8

Summary of Bottom Lateral Bracings

Top Lateral Bracings 1-Required

TL1	2	L	125 × 75 × 9	13.5 × 7,655		206.7
〃	2	〃	〃	〃 × 7,715		208.3
TL2	4	〃	〃	〃 × 3,800		205.2
TL3	2	〃	〃	〃 × 3,725		100.6
〃	2	〃	〃	〃 × 3,665		99.0
TL4	4	〃	〃	〃 × 7,470		403.4
TL5	2	〃	〃	〃 × 3,580		96.7
〃	2	〃	〃	〃 × 3,640		98.3
TL6	2	〃	〃	〃 × 3,700		99.9
〃	2	〃	〃	〃 × 3,640		98.3
TL7	4	〃	〃	〃 × 7,460		412.6
TL8	2	〃	〃	〃 × 3,585		96.8
〃	2	〃	〃	〃 × 3,645		98.4
TL9	2	〃	〃	〃 × 3,685		99.5
〃	2	〃	〃	〃 × 3625		97.9
TL10	4	〃	〃	〃 × 7,450		402.3
TL11	2	〃	〃	〃 × 3,590		96.9
〃	2	〃	〃	〃 × 3,650		98.6
TL12	2	〃	〃	〃 × 3,670		99.1
〃	2	〃	〃	〃 × 3,610		97.5
TL13	4	〃	〃	〃 × 7,445		402.0
TL14	2	〃	〃	〃 × 3595		97.1
〃	2	〃	〃	〃 × 3,655		98.7
TL15	2	〃	〃	〃 × 3,660		98.8
〃	2	〃	〃	〃 × 3,600		97.2
	10	Pls	355 × 9	25.081 × 570		1430
						4,152.8

Lateral Plate 1-Required

$LP1^R_L$	4	Pls	460 × 9	32.499 × 755		98.1
$LP2^R_L$	4	〃	455 × 9	32.146 × 755		97.1
$LP3^R_L$	4	〃	445 × 9	31.439 × 755		94.9
$LP4^R_L$	4	〃	〃	〃 × 755		94.9
LP5	2	〃	435 × 9	30.733 × 755		464
						431.4

Summary of Top Lateral Bracings & Lateral Plate

Lateral bracings	4,152.8
Lateral Plate	431.4
	4,584.2

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CALCULATIONS FOR

Materials of

		Portal Bracings PB1			2-Required		
Top strut	1	L	125 × 75 × 9	c	13.5	× 6,100	82.4
" "	2	L	90 × 75 × 9	c	11.0	× 6,100	134.2
" "	1	Pl	317 × 8	c	19.908	× 6,100	121.4
Bottom strut	4	L	90 × 75 × 9	c	11.0	× 4,560	200.6
" "	1	Pl	317 × 8	c	19.908	× 4,860	96.8
PB2	4	L	75 × 75 × 9	c	9.96	× 2,350	93.6
"	4	"	"	c	"	× 2,290	91.2
"	2	Pls	317 × 8	c	19.908	× 1,497	59.6
PB1	8	L	75 × 65 × 8	c	8.29	× 970	64.3
PB3	8	"	"	c	"	× 830	55.0
PB4	8	"	"	c	"	× 810	53.7
PB5	8	"	"	c	"	× 765	50.7
Tie plate	18	Pls	190 × 8	c	11.932	× 317	68.1
Gusset	2	"	295 × 9	c	20.842	× 770	32.1
"	2	"	740 × 9	c	52.281	× 770	90.5
"	2	"	330 × 9	c	23.315	× 850	39.6
"	2	"	345 × 9	c	24.374	× 590	28.8
"	2	"	410 × 8	c	25.748	× 420	21.6
"	2	"	270 × 8	c	16.956	× 420	14.2
"	2	"	410 × 8	c	25.748	× 440	22.7
"	2	"	270 × 8	c	16.956	× 440	14.9
"	4	"	450 × 8	c	28.260	× 590	66.7
"	4	"	435 × 8	c	27.318	× 495	54.1
"	2	"	380 × 8	c	23.864	× 455	21.7
							1568.5
		Sway Bracings SB1			2-Required		
Top strut	4	L	125 × 75 × 9	c	13.5	× 6,100	3294
" "	2	Pls	275 × 9	c	19.429	× 460	17.9
" "	4	"	280 × 8	c	17.584	× 460	32.4
" "	20	bars	60 × 8	c	3.768	× 357	26.9
Bottom strut	2	L	125 × 75 × 9	c	13.5	× 6,318	170.6
" "	2	Pls	155 × 9	c	10.951	× 220	4.8
" "	2	"	290 × 8	c	18.212	× 350	12.7
" "	3	"	215 × 8	c	13.502	× 290	11.7
Knee bracing	4	L	125 × 90 × 9	c	14.6	× 2,260	132.0
" "	2	Pls	260 × 9	c	18.369	× 555	20.4
" "	2	L	65 × 65 × 8	c	7.66	× 400	6.1
" "	2	Pls	205 × 8	c	12.874	× 275	7.1
" "	2	"	250 × 8	c	15.700	× 260	8.2
" "	2	"	258 × 9	c	18.228	× 610	22.2
Diagonal	4	L	65 × 65 × 8	c	7.66	× 905	27.7
"	16	"	"	c	"	× 895	109.7
End connection	4	"	90 × 90 × 10	c	13.3	× 495	26.3
" "	4	"	125 × 75 × 9	c	13.5	× 220	11.9
" "	4	"	"	c	"	× 260	14.0 * Special for SB1
	26	Wash.	60ϕ	c	22.2	× 8	4.6
							996.6
		Sway Bracings SB2			2-Required		
Same parts for SB1							982.6
End connection	4	L	125 × 75 × 9	c	13.5	× 250	13.5
							996.1

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Materials of

Struts ST1, ST2, & ST3 1-Required

	20	L's	125	75	9 ℄	135 × 6,100	1,647.0
	20	P/s	250	"	8 ℄	15,700 × 315	989
	100	bars	60	"	8 ℄	3768 × 375	141.3
	10	P/s	275	"	9 ℄	19429 × 315	61.2
For ST1	8	L's	100	75	10 ℄	130 × 260	27.0
For ST2	8	"		"	℄	" × 255	26.5
For ST3	4	"		"	℄	" × 245	12.7
							2,014.6

Summary of Portal & Sway bracings & Struts

PB1	2	℄	1,568.5	= 3,137.0
SB1	2	℄	996.6	= 1,993.2
SB2	2	℄	996.1	= 1,992.2
Struts	1	℄	2,014.6	= 2014.6
				9,137.0

Rivet heads for one span (all rivet 19ϕ)

Main trusses

Shop rivet	36,176	× 0.048	=	1,736.4
Field rivet	16,016	× 0.048	=	768.8
				2,505.2

3.46 %

Portal bracings, Sway bracings & struts

Shop rivet	4,208	× 0.048	=	202.0
Field rivet	2,040	× 0.048	=	97.9
				299.9

Bottom lateral bracings

Shop rivet	1,112	× 0.048	=	53.4
Field rivet	1,392	× 0.048	=	66.8
				120.2

Top lateral bracings

Shop rivet	880	× 0.048	=	42.2
Field rivet	720	× 0.048	=	34.6
				76.8

Floor beams

Shop rivet	6,110	× 0.048	=	293.3
Field rivet	1,556	× 0.048	=	74.7
				368.0

Stringers & brackets

Shop rivet	256	× 0.048	=	12.3
Field rivet	64	× 0.048	=	3.1
				15.4

Summary of rivet heads 33855

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CALCULATIONS FOR

Materials of

		Roller shoe (one set)			2- Required	
Top casting	RS1		1 ℄	358	=	358
Bed plate	BP1		1 ℄	1965	=	196.5
Roller	RN1		5 ℄	37	=	185
Pin	PN1		1 ℄	65.6	=	65.6
Plate		2 ℄ 70 × 14 ℄ 7.693	×	510	=	7.8
Dust guard	DG1		2 ℄	27.6	=	55.2
" "	DG2	2 ℄ 133 × 10 ℄ 10.441	×	630	=	13.2
Pin	P 1		6 ℄	0.15	=	.9
"	P 2		4 ℄	0.16	=	.6
Tapped bolt		6" × 25	10 ℄	0.01	=	.1
" "		22" × 50	4 ℄	0.28	=	1.1
" "		9" × 25	2 ℄	0.03	=	.1
Anchor bolt	AB1	40" × 700	4 ℄	8.3	=	33.2
Washer		4 ℄ 100 × 10 ℄ 7.850	×	100	=	31
						920.4
		Fixed shoe (one set)			2- Required	
Shoe	FS1		1 ℄	403	=	403
Pin	PN1		1 ℄	65.6	=	65.6
Anchor bolt	AB1	40" × 700	4 ℄	8.3	=	33.2
Washer		4 ℄ 100 × 10 ℄ 7.850	×	100	=	31
						504.9
Summary of Shoes						
Roller shoe			2 ℄	920.4	=	1,840.8
Fixed shoe			2 ℄	504.9	=	1,009.8
						2,850.6
Grand summary for one span						
Main trusses				72,078.6		
Floor beams				12,188.3		
Stringers				12,078.1		
Bottom lateral bracings				5,048.8		
Top lateral bracings				4,584.2		
Portal, sway bracings & struts				9,137.0		
Rivet heads				3,385.5		
				118,500.5		
Shoes				2,850.6		
				121,351.1 kgs		

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SHOP

L_0-U_1
top ~~164~~ 167
bottom 92
side 122

U_1-U_2
top ~~60~~ 91
bottom 56
side 8

U_2-U_3
top 80
bottom 51
side 40

U_3-U_4
T 80
B 55
S 176

U_4-U_5
T 80
B 55
S 200

U_5-U_6
T 80
B 55
S 172

1860

FIELD

top 44
bottom 10
side 104

U_1-U_2
top 74
bottom 18
side 108

U_2-U_3
top 80
bottom 20
side 50

U_3-U_4
T 80
B 20
S 80

U_4-U_5
T 80
B 20
S 56

U_5-U_6
T 80
B 20
S 84

1,028

3

L_0-L_1
T+B 78
S 178

L_1-L_2
T+B 51
S 62

L_2-L_3
T+B 58
S 118

L_3-L_4
T+B 52
S 116

L_4-L_5
T+B 64
S 214

L_5-L_6
T+B 18
S 162

136

7

L_0-L_1
T+B
S 28

L_1-L_2
T+B 32
S 62

L_2-L_3
T+B
S 6

L_3-L_4
T+B 32
S 78

L_4-L_5
T+B
S 6

L_5-L_6
T+B 32
S 134

410

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CALCULATIONS FOR

4455 / 5

Truss
shop 4,455 × 8 = 35,640
field 2,002 × 8 = 16,016 ——— 36176

Diaphragm
DM1 23 × 8 = 184
DM2 8 × 24 = 192
DM3 8 × 20 = 160 } 536

floor beam
FB1
S 235 × 4 = 940
F 64 × 4 = 256

FB2
S 235 × 12 = 2,820
F 60 × 12 = 720

FB3
S 235 × 10 = 2,350
F 58 × 10 = 580

S = 6,110
F = 1,556

stringer + bracket
S 128 × 2 = 256
F 32 × 2 = 64

bottom lateral
S 278 × 4 = 1,112
F 348 × 4 = 1,392

top lateral
S 44 × 20 = 880
F 36 × 20 = 720

Portal
S 492 × 4 = 1,968
F 224 × 4 = 896

sway
S 175 × 8 = 1,400
F 108 × 8 = 864

strut
S 84 × 10 = 840
F 28 × 10 = 280

S 4208
F 2,040

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CALCULATIONS FOR

	SHOP	FIELD	
U_1-L_2	88	U_1-L_2	80
L_2-U_3	192	L_2-U_3	+0
U_3-L_4	92.	U_3-L_4	64
L_4-U_5	188	L_4-U_5	48
U_5-L_6	188	U_5-L_6	48
U_1-L_1	64	U_1-L_1	+8
U_2-L_2	76	U_2-L_2	+4
U_3-L_3	90	U_3-L_3	66
U_4-L_4	88	U_4-L_4	+4
U_5-L_5	98	U_5-L_5	62
U_6-L_6	45	U_6-L_6	20
	1209		564
Diaphragm			
DM1	23		
DM2	8		
DM3	8		
Floor beam			
FB1	235	FB1	47
			20
			64
FB2	235		44
			16
			60
FB3	235		44
			12
			56
stringer	96		
bracket	32	bracket	32

	S	F
bottom lateral (T.S)		
L_0-L_1	61	60
		12
		72
L_1-L_2	61	72
L_2-L_3	61	+8
		60
L_3-L_4	61	48
		12
		60
L_4-L_5	17	42
L_5-L_6	17	42
	78	348
top lateral		
	44	36
portal	122	16
	40	8
	20	80
	60	64
	94	24
	156	32
	492	224
sway	84	20
	70	8
	37	40
	34	40
	175	108
strut		
	84	20
		8
		28

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CALCULATIONS FOR

Dead load of Bridge:

				on lower Panel points	on upper panel points
Floor.					
Floor and pavement	5.50 @	465	=	2560	—
Copings	2 @	227	=	454	—
handrails assumed as	2 @	85	=	170	—
				3184	
Structural steel parts.					
Stringers	4 @	50	=	200	—
floor beams	980 ÷	5.0	=	196	—
upper lateral bracings assumed as				—	100
lower " " "				100	—
portals, sways, struts etc say				—	140
main trusses assumed as				650	650
				1146	
Electric wiring and other fittings say				60	
				4390	890 kg/m of bridge

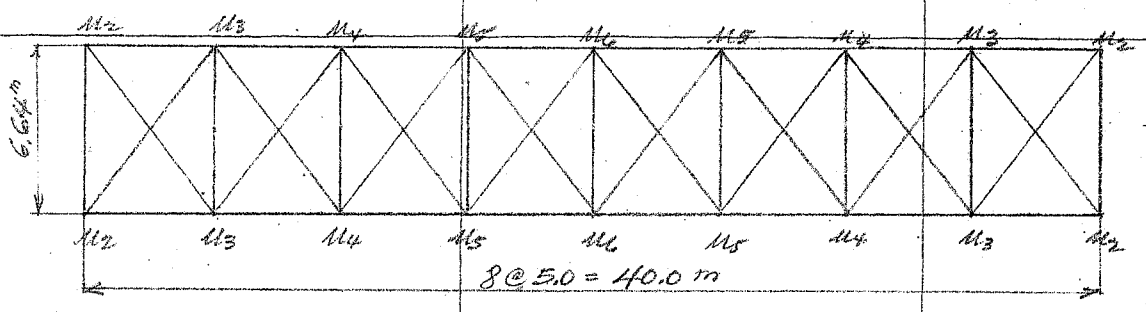
Upper panel load = 890 × 5.0 = 4450 kg for 2 trusses
lower " " = 4390 × 5.0 = 21950 " " "
26400 " " "

Design of lateral Bracings.

Wind load on upper lateral bracing 5.0"@215 = 1075 kg on one panel point
" " " lower " " 5.0"@415 = 2075 " " " "

Seismic thrust on upper lateral bracing 4450 × 0.15 = 670 kg " " "
" " " lower " " 21950 × 0.15 = 3290 " " "

Upper lateral bracing,



Diagonal length
$= \sqrt{6.64^2 + 5.0^2} = 8.31$ meters.
coefficient $= \frac{8.31}{6.64} = 1.252$

Stress due to moving wind load of 1,075 kg on each panel point. $l = 8.31 \times \frac{6.12}{6.64} = 7.66^{cm}$

Panel		max. shear	coef.	Stress	Section required	$r = 3.96^{cm}$ $\frac{l}{r} = \frac{766}{3.96} = 193.5 < 200$
U_2-U_3	1,075 ÷ 8 × 28	= 3770 ×	1.252 =	4,720 T ÷ 1200 =	3.93 cm² net.	2L 125×75×9 = 34.38 − 7.92 = 26.46 cm² net
U_3-U_4	" 21	2820 "		3,530 T "	2.94	"
U_4-U_5	" 15	2020 "		2,530 T "	2.11	"
U_5-U_6	" 10	1350 "		1690 T "	1.41	"

use 6-19φ rivets for each connection.

Approximate weight of upper lateral bracing.

Diagonals	32L 125×75×9	@ 13.50 ×	7.65 =	3300
Side connection say	16	@ 15.00	=	240
Center " "	8	@ 20.00	=	160
rivet heads say				130
				3830 kg

weight of bracing per lin. meter = 3830 ÷ 40. = 96 kg

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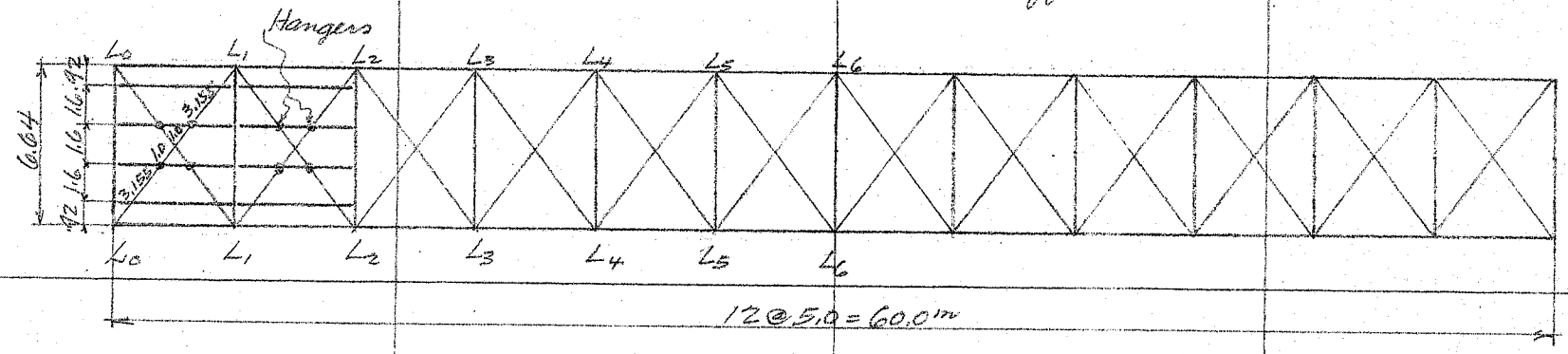
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CALCULATIONS FOR

Lower lateral bracing.

moving wind panel load = 2075 kg
full seismic panel thrust = 3290 "

Diagonal length = 8.31 meters
Coefficient = 1.252



Wind stress in diagonal members.

Panel				Shear	coef.	Stress	Section required	19mm rivet no. reqd.	19mm rivet no. use	
L0–L1	2,075÷12×	66	=	11,400	× 1.252 =	14260 T÷1200 =	11.88 net	6.7	10	2L 125×75×9 = 34.38 − 7.92 = 26.46 net
L1–L2	"	55	=	9,520	"	11,910	9.92	5.6	10	"
L2–L3	"	45	=	7,780	"	9,740	8.12	4.6	8	"
L3–L4	"	36	=	6,230	"	7,800	6.50	3.7	6	"
L4–L5	"	28	=	4,840	"	6,060	5.05	2.9	5	1L 125×90×9 = 18.54 − 1.98 = 16.56 net
L5–L6	"	21	=	3,630	"	4,540	3.79	2.1	4	"

4-19mm

Seismic stresses in diagonal members.

Panel				Shear	coef.	Stress	Section required	19mm rivet no. reqd.	19mm rivet no. use
L0–L1	3290 ×	5.5	=	18,100	× 1.252 =	22,650 T÷1920 =	11.80 net	6.7	10
L1–L2	"	4.5		14,800	"	18,520	9.64	5.5	10
L2–L3	"	3.5		11,510	"	14,400	7.50	4.2	8
L3–L4	"	2.5		8,220	"	10,300	5.37	3.0	6
L4–L5	"	1.5		4,930	"	6,170	3.21	1.8	5
L5–L6	"	0.5		1,650	"	2,070	1.08	0.6	4

Approximate weight of lower lateral bracing.

Diagonals	32L 125×75×9	@ 13.50 × 7.80	=	3,370
"	8L 125×90×9	@ 14.60 × 7.80	=	910
Side connections say	16	@ 20.0	=	320
" "	10	@ 15.0	=	150
Centre connections say	8	@ 25.0	=	200
"	4	@ 15.0	=	60
Hangers say	48	@ 10.0	=	480
rivet heads etc say				210
				5,700 kg

weight of bracing per lin. meter = $\frac{5700}{60}$ = 95 kg

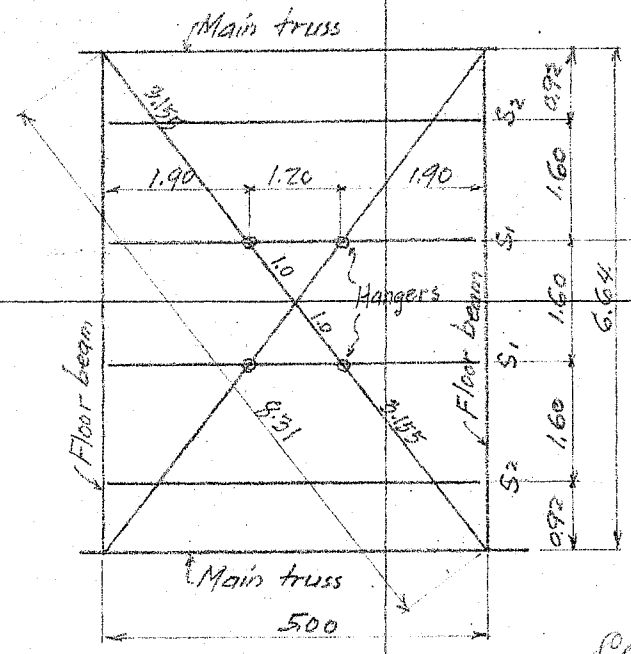
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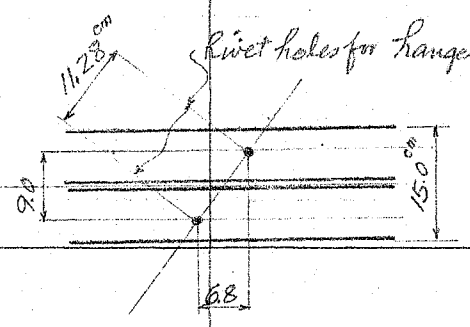
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CALCULATIONS FOR

Check for Strength of Inside stringers S1 at the connection point of lateral hangers.
Dead load :-

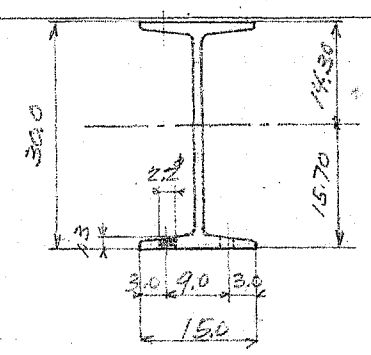




One rivet hole to be reduced from bottom flange of Stringer S1.

Stringer S1. Ibeam 300×150@48.3 kg
Moment of inertia I_x = 9499 cm⁴
Section modulus Z_x = 633.2 cm³
reduced area = 1.3×2.2 = 2.86 cm²

for 19φ rivet



Center of gravity of reduced area.

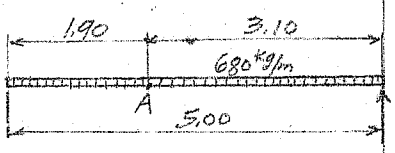
Gross section	61.58 × 15.00 =	923.7	
rivet hole	− 2.86 × 0.65 =	− 1.9	
	58.72	15.7 cm	921.8

Moment of inertia I_x of reduced section.

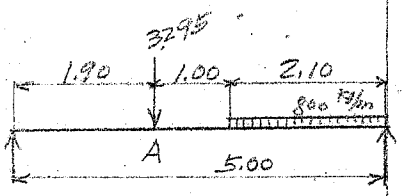
Gross section		9499
rivet hole	− 2.86 × 15.05² =	− 648
		8851 cm⁴

Section modulus for
Compression fibre = 8851 ÷ 14.30 = 619.0 cm³
Tension fibre = 8851 ÷ 15.70 = 564.0 "

Max. moment at point of hanger connection.
Dead load on beam = 680 kg/m (See on page 5)
Dead load moment at A = $\frac{680}{2}$ × 1.90 × 3.10 = 2,000 kgm



Live load
max. wheel concentration = 3295 kg (refered to page 5)
uniform load = 800 kg/m (")



Reaction
wheel 3295 × 3.1 ÷ 5.0 = 2,043
unif. load 800 × 2.10 × 1.05 ÷ 5 = 353
2,396 kg

Live load moment at A = 2396 × 1.9 = 4,550 kgm
D.L. L.L.
Total moment = 2,000 + 4,550 = 6,550 kgm at A

Compression fibre stress = 6550 × 100 ÷ 619.0 = 1,059 kg/cm² < 1100 "
Tension " " = 6550 × 100 ÷ 564.0 = 1,162 " < 1200 "

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CALCULATIONS FOR

Design of Main Truss. Span length = 12 @ 5.0 = 60.0 metres, Spacing = 6.64 m c to c

Dead load:

Deck construction

Floor slab and pavement	5.50	@ 465	=	2560
copings	2	@ 227	=	454
handrails assumed as	2	@ 85	=	170
				3184

Structural steel parts

Stringers	4	@ 50	=	200
floor beams	980	÷ 50	=	196
Upper lateral bracing assumed as				100
Lower " " "				100
portals, sways, struts &c say				140
main trusses assumed as	2	@ 650	=	1300
				2,036
Electric wiring and other fittings say				60
				5,280 kg/m of bridge.

Dead panel load = $\frac{5280}{2} \times 5.00 =$ 13,200 kg for one truss.

Live load:

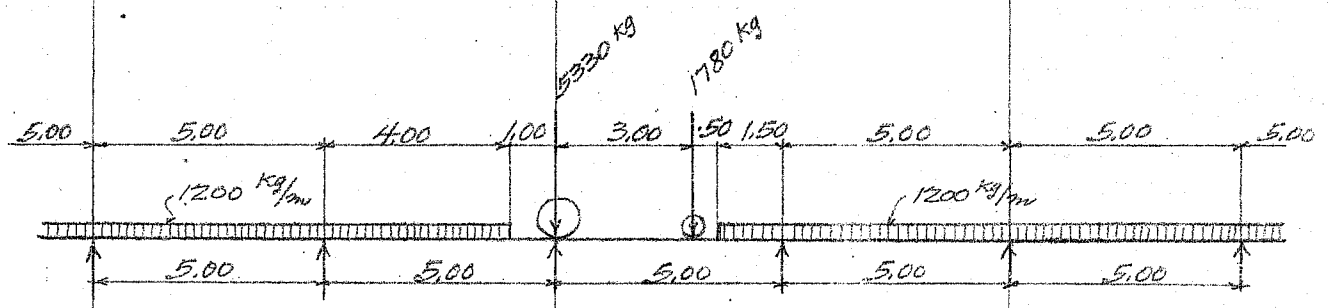
Uniform live load $w = \frac{100,000}{170+60} =$ 435 kg per square meter.

Uniform load on one truss = 435 × 2.75 = 1,195 call this 1200 kg/lin. metre.

6 ton motor truck rear wheel concentration = 3250
Impact coefficient = $\frac{20}{60+60} = 16.67\%$ = 375
2625 kg.

max. wheel concentration on main truss.

2625 × 4 × 3.37 ÷ 6.64 = 5330 kg
front wheel concentration say 5330 ÷ 3 = 1780 "



Uniform load	6,000	3,000			3,000	6,000	6,000
" "		2880		1920			
" "				270	1,530		
rear wheel load				5330			
front " "				710	1,070		
Summary	6,000	5,880		8230	5,600	6,000	6,000
		− 120		+ 2230	− 400		
assume	6000	6,000		{6,000 / 2,200}	6,000	6000	6000

Now let us assume the panel concentration for live load as 6000 kg on each panel point throughout and an extra concentration for motor truck loading as 2200 kg.

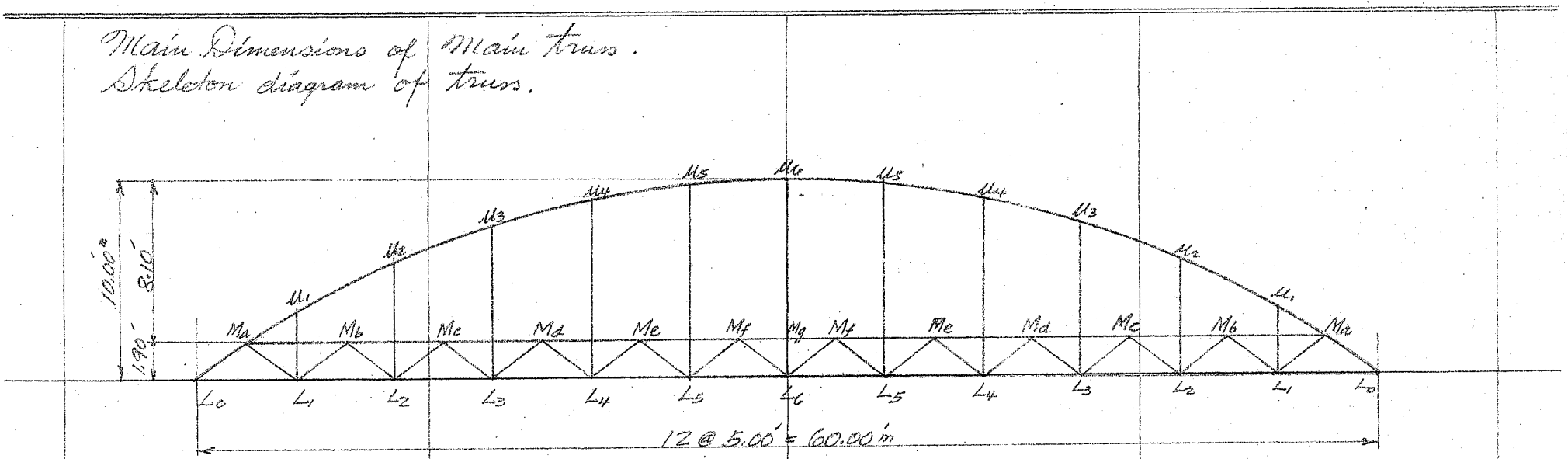
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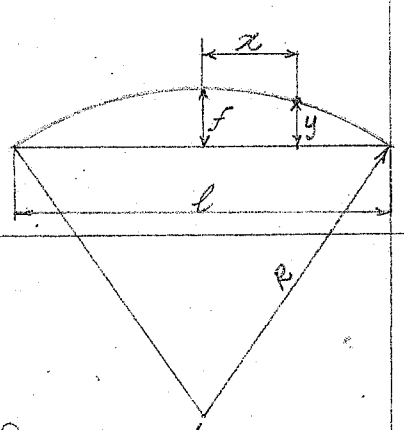
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CALCULATIONS FOR

Main Dimensions of Main truss.
Skeleton diagram of truss.



Radius of Curvature of Top chord.



Radius $R = \frac{l^2}{8f} + \frac{f}{2}$, where $l = 60.00$ meters;
$f = 10.00$ " "

$= \frac{3600}{80} + \frac{10}{2}$,

$= 50.000$ meters.

Depth of truss at each panel point.

Depth $y = f - R + \sqrt{R^2 - x^2}$, where $f = 10.00$ meters;
$R = 50.00$ " "

$= \sqrt{2{,}500 - x^2} - 40.00$ in meters.

Depth of	x	x^2	$2{,}500 - x^2$	$\sqrt{2500 - x^2} - 40.000$ =	Depth in meters.	Difference
$U_6 - L_6$	0.00 '	0.0 '	2,500 '	50.000 ' =	10.000 '	0.251 '
$U_5 - L_5$	5.00 '	25.0 '	2,475 '	49.749 ' =	9.749 '	0.759 '
$U_4 - L_4$	10.00 '	100.0 '	2400 '	48.990 ' =	8.990 '	1.293 '
$U_3 - L_3$	15.00 '	225.0 '	2,275 '	47.697 ' =	7.697 '	1.871 '
$U_2 - L_2$	20.00 '	400.0 '	2,100 '	45.826 ' =	5.826 '	2.525 '
$U_1 - L_1$	25.00 '	625.0 '	1,875 '	43.301 ' =	3.301 '	

Length of chord members.

Members	Vert. proj	Square	+ (Hor. proj)^2		Length of members		Coef. $L/5.000$
$L_0 - U_1$	3.301 '	10.8966 '	+ 25.000 '	= 35.8966 '	5.991 m.		1.198 '
$U_1 - U_2$	2.525 '	6.3756 '	" '	31.3756 '	5.601 '		1.120 '
$U_2 - U_3$	1.871 '	3.5006 '	" '	28.5006 '	5.339 '		1.068 '
$U_3 - U_4$	1.293 '	1.6718 '	" '	26.6718 '	5.164 '		1.033 '
$U_4 - U_5$	0.759 '	0.5761 '	" '	25.5761 '	5.057 '		1.011 '
$U_5 - U_6$	0.251 '	0.0630 '	" '	25.0630 '	5.006 '		1.001 '

Length of Diagonal members.

$\sqrt{1.900^2 + 2.500^2}$ ' = 3.140 m ' for $L_1 - M_b$ to $M_f - L_6$ inclusive. Coef. $= \frac{3.140}{1.900} = 1.653$ '

$5.00 \times \frac{1.401}{3.301}$ = 2.122 m '

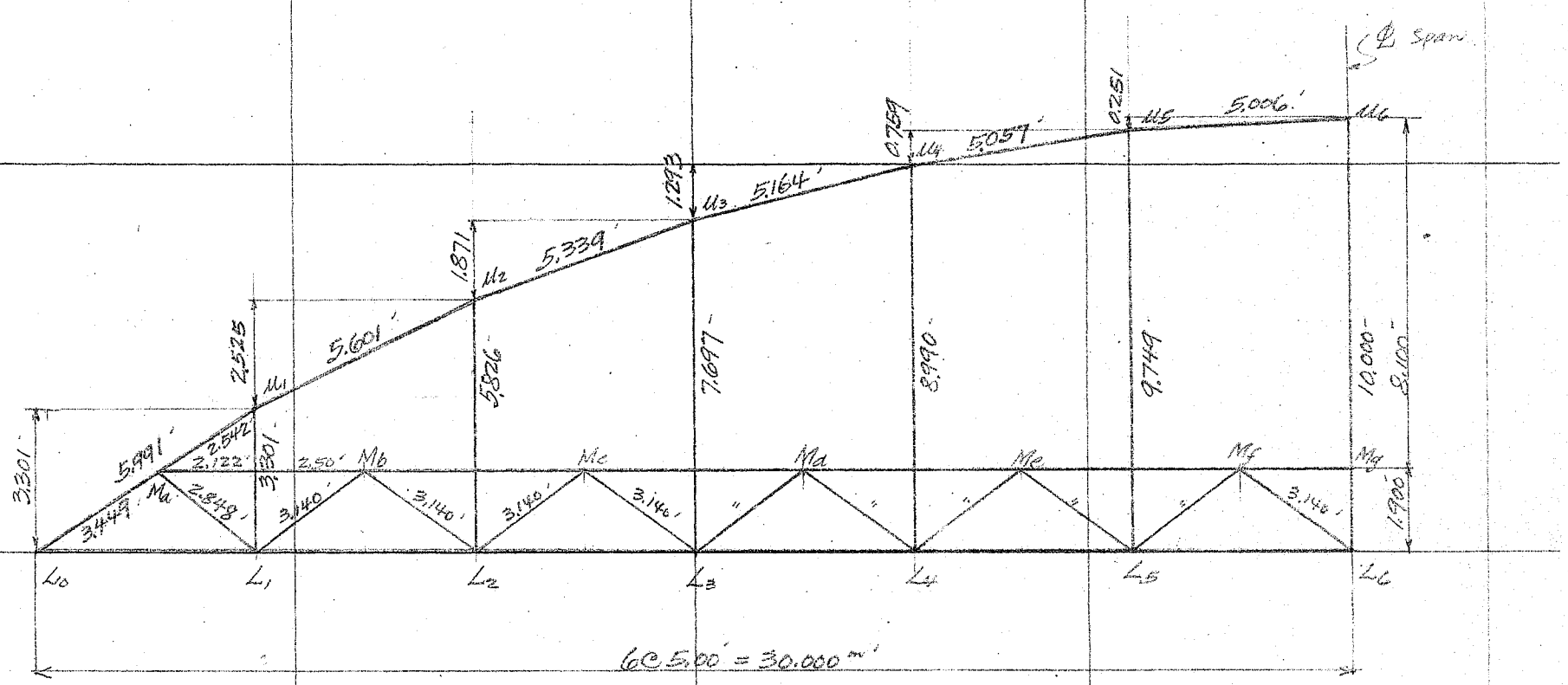
$\sqrt{1.900^2 + 2.122^2}$ ' = 2.848 m ' for $M_a - L_1$ Coef. = 2.848 ÷ 1.900 = 1.499 '

$\sqrt{1.900^2 + 2.878^2}$ = 3.449 m ' for $L_0 - M_a$ " = 3.449 ÷ 1.900 = 1.815 '

5.991 − 3.449 = 2.542 m ' for $M_a - U_1$

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Length of Truss members.



Horizontal Thrust due to a unit load at any panel point.

General Equation to Thrust.

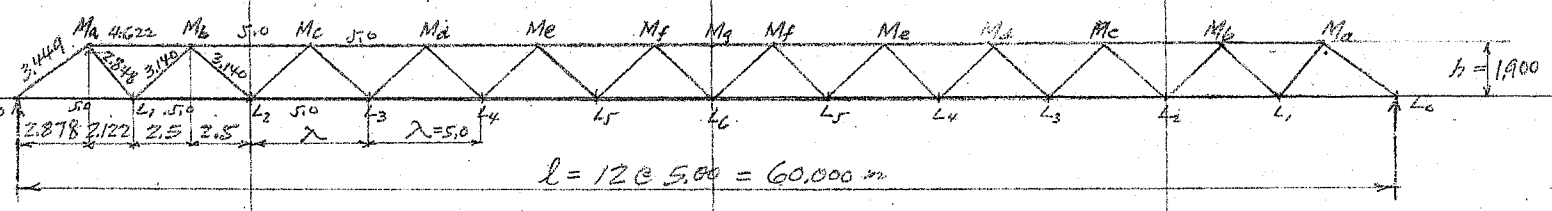
$$H = \frac{\Sigma\, So\, Sa\, L/A}{\Sigma\, Sa^2\, L/A}$$

where H = Horizontal thrust due to a unit load at any panel point;
So = Stress in each member due to a unit load, when the redundancy being removed;
Sa = Stress in each member when $H = -1$ applied;
L = length of each member;
A = gross sectional area of each member.

Stress So. (in the above equation of H)

Top chords. So for all top chords are zero.

So



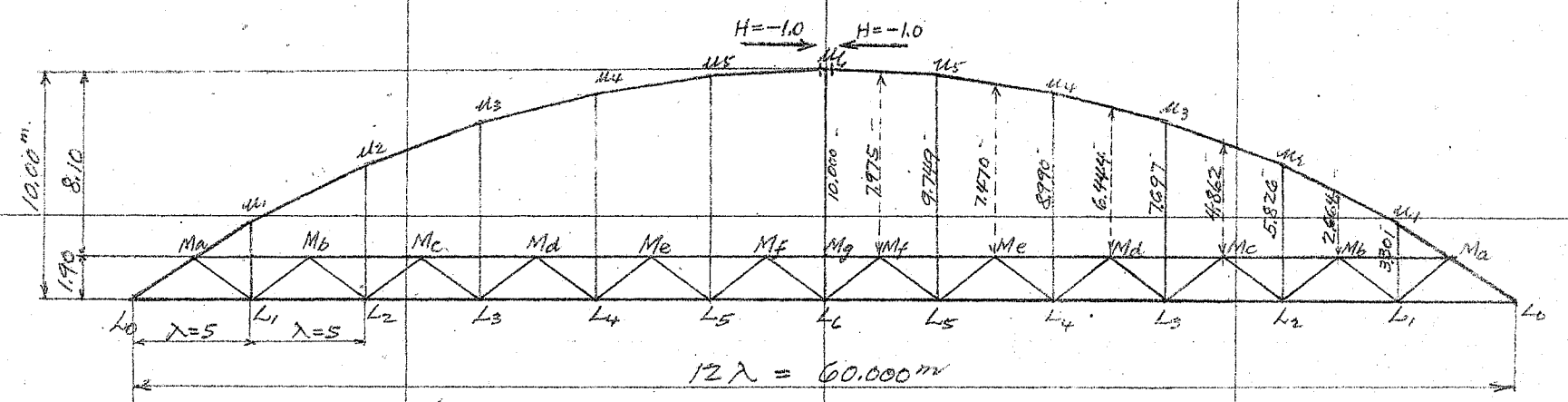
Middle chords. So

Ma–Mb	$-\frac{11}{12}\cdot\frac{\lambda}{h}' = -\frac{11}{12}\cdot\frac{5}{1.9}' =$	$-11 \times 0.2193'$
Mb–Mc	$-\frac{10}{12}\cdot\frac{2\lambda}{h}' = -\frac{10}{12}\cdot\frac{10}{1.9} =$	-10×0.4386;
Mc–Md	$-\frac{9}{12}\cdot\frac{3\lambda}{h}' = -\frac{9}{12}\cdot\frac{15}{1.9} =$	$-9 \times 0.6579'$
Md–Me	$-\frac{8}{12}\cdot\frac{4\lambda}{h}' = -\frac{8}{12}\cdot\frac{20}{1.9} =$	$-8 \times 0.8772'$
Me–Mf	$-\frac{7}{12}\cdot\frac{5\lambda}{h}' = -\frac{7}{12}\cdot\frac{25}{1.9} =$	$-7 \times 1.0965'$
Mf–Mf	$-\frac{6}{12}\cdot\frac{6\lambda}{h}' = -\frac{6}{12}\cdot\frac{30}{1.9} =$	$-6 \times 1.3158'$

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Bottom chords. S_o	L_0-L_1	$\frac{11}{12}\times\frac{2.878}{1.90}$ =		11×0.1262
	L_1-L_2	$\frac{10}{12}\times\frac{1.5\lambda}{h}$ =	$\frac{10}{12}\times\frac{1.5\times5}{1.9}$ =	10×0.3289
	L_2-L_3	$\frac{9}{12}\times\frac{2.5\lambda}{h}$ =	$\frac{9}{12}\times\frac{2.5\times5}{1.9}$ =	9×0.5482
	L_3-L_4	$\frac{8}{12}\times\frac{3.5\lambda}{h}$ =	$\frac{8}{12}\times\frac{3.5\times5}{1.9}$ =	8×0.7675
	L_4-L_5	$\frac{7}{12}\times\frac{4.5\lambda}{h}$ =	$\frac{7}{12}\times\frac{4.5\times5}{1.9}$ =	7×0.9868
	L_5-L_6	$\frac{6}{12}\times\frac{5.5\lambda}{h}$ =	$\frac{6}{12}\times\frac{5.5\times5}{1.90}$ =	6×1.2061
Diagonals. S_o	L_0-M_a	$-\frac{11}{12}\times 1.815$ =		$= -11\times 0.1512$
	M_a-L_1	$+\frac{11}{12}\times 1.449$ =		$= +11\times 0.1207$
	L_1-M_b	$-\frac{10}{12}\times 1.653$ =		$= -10\times 0.1378$
	M_b-L_2	+ " " =		$= +10\times 0.1378$
	L_2-M_c	$-\frac{9}{12}\times 1.653$ =		$= -9\times 0.1378$
	M_c-L_3	+ " " =		$= +9\times$ "
	L_3-M_d	$-\frac{8}{12}\times$ " =		$= -8\times$ "
	M_d-L_4	+ " " =		$= +8\times$ "
	L_4-M_e	$-\frac{7}{12}\times$ " =		$= -7\times$ "
	M_e-L_5	+ " " =		$= +7\times$ "
	L_5-M_f	$-\frac{6}{12}\times$ " =		$= -6\times$ "
	M_f-L_6	+ " " =		$= +6\times 0.1378$
Verticals S_o	So for all vertical members are Zero.			

Stress S_o. (in the equation of Thrust H)



note. Horizontal component of top chord stress = -1.0

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Top chords. Sa	L_0-M_a						=	0.0000		
	M_a-M_1	1.0	×	1.198			=	1.1980		
	M_1-M_2	1.0	×	1.120			=	1.1200		
	M_2-M_3	1.0	×	1.068			=	1.0680		
	M_3-M_4	1.0	×	1.033			=	1.0330		
	M_4-M_5	1.0	×	1.011			=	1.0110		
	M_5-M_6	1.0	×	1.001			=	1.0010		
Middle chords. Sa	M_a-M_b	− 1.0	÷	1.9	×	3.301	=	− 1.7374		
	M_b-M_c	− 〃	〃	5.826			=	− 3.0663		
	M_c-M_d	− 〃	〃	7.697			=	− 4.0511		
	M_d-M_e	− 〃	〃	8.996			=	− 4.7316		
	M_e-M_f	− 〃	〃	9.749			=	− 5.1311		
	M_f-M_g	− 〃	〃	10.000			=	− 5.2632		
Bottom chords Sa	L_0-L_1						=	0.0000		
	L_1-L_2	1.0	÷	1.9	×	2.664	=	1.4021		
	L_2-L_3	〃	〃			4.862	=	2.5589		
	L_3-L_4	〃	〃			6.444	=	3.3916		
	L_4-L_5	〃	〃			7.470	=	3.9316		
	L_5-L_6	〃	〃			7.975	=	4.1974		
Verticals Sa	M_1-L_1	− 1/5 × (3.301 − 2.525)					=	− 0.1552		
	M_2-L_2	− 〃 × (2.525 − 1.871)					=	− 0.1308		
	M_3-L_3	− 〃 × (1.871 − 1.293)					=	− 0.1156		
	M_4-L_4	− 〃 × (1.293 − 0.759)					=	− 0.1068		
	M_5-L_5	− 〃 × (0.759 − 0.251)					=	− 0.1016		
	M_6-L_6	− 〃 × (0.251 + 0.251)					=	− 0.1004		
Diagonals Sa	L_0-M_a						=	0.0000		
	M_a-L_1	1/5 × 3.301	×	1.499			=	0.9896		
	L_1-M_b	〃 × 2.525	×	1.653			=	− 0.8348		
	M_b-L_2	〃 × 〃	×	〃			=	0.5348		
	L_2-M_c	〃 × 1.871	×	〃			=	− 0.6186		
	M_c-L_3	〃 × 〃	×	〃			=	0.6186		
	L_3-M_d	〃 × 1.293	×	〃			=	− 0.4275		
	M_d-L_4	〃 × 〃	×	〃			=	0.4275		
	L_4-M_e	〃 × 0.759	×	〃			=	− 0.2509		
	M_e-L_5	〃 × 〃	×	〃			=	0.2509		
	L_5-M_f	〃 × 0.251	×	〃			=	− 0.0830		
	M_f-L_6	〃 × 〃	×	〃			=	0.0830		

Values of S_0 S_a L and $S_a^2 L$ for each member.

		S_{01}	S_a	L	$S_{01}S_a$	S_a^2	$S_{01}S_aL$	S_a^2L
Top chords	M_a-M_1	0.0000	1.1980	2.542	0.0000	1.43522	0.0000	3.6483
	M_1-M_2	〃	1.1200	5.601	〃	1.25544	〃	7.0259
	M_2-M_3	〃	1.0680	5.339	〃	1.14062	〃	6.0897
	M_3-M_4	〃	1.0330	5.164	〃	1.06711	〃	5.5105
	M_4-M_5	〃	1.0110	5.057	〃	1.02221	〃	5.1688
	M_5-M_6	〃	1.0010	5.006	〃	1.00220	〃	5.0160
	M_6-M_6'	〃	1.0010	5.006	〃	1.00220	〃	5.0160
	$M_6'-M_4'$	〃	1.0110	5.057	〃	1.02221	〃	5.1688
	$M_4'-M_3'$	〃	1.0330	5.164	〃	1.06711	〃	5.5105
	$M_3'-M_2'$	〃	1.0680	5.339	〃	1.14062	〃	6.0897
	$M_2'-M_1'$	〃	1.1200	5.601	〃	1.25544	〃	7.0259
	$M_1'-M_a$	〃	1.1980	2.542	〃	1.43522	〃	3.6483

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		S_{oi}	S_a	L	$S_{oi}S_a$	S_a^2	$S_{oi}S_aL$	S_a^2L
Middle chords	$M_a - M_b$	− 0.2193	− 1.7374	4.622	0.3810	3.0186	1.7610	13.9520
	$M_b - M_c$	− 0.4386	− 3.0663	5.000	1.3449	9.4022	6.7245	47.0110
	$M_c - M_a$	− 0.6579	− 4.0511	5.000	2.6652	16.4114	13.3260	82.0570
	$M_d - M_e$	− 0.8772	− 4.7316	5.000	4.1506	22.3880	20.7530	111.9400
	$M_e - M_f$	− 1.0965	− 5.1311	5.000	5.6263	26.3282	28.1315	131.6410
	$M_f - M_{f'}$	− 1.3158	− 5.2632	5.000	6.9253	27.7013	34.6265	138.5065
	$M_{f'} - M_{e'}$	− 1.5351	− 5.1311	5.000	7.8765	26.3282	39.3840	131.6410
	$M_{e'} - M_{d'}$	− 1.7544	− 4.7316	5.000	8.3011	22.3880	41.5055	111.9400
	$M_{d'} - M_{c'}$	− 1.9737	− 4.0511	5.000	7.9957	16.4114	39.9785	82.0570
	$M_{c'} - M_{b'}$	− 2.1930	− 3.0663	5.000	6.7244	9.4022	33.6220	47.0110
	$M_{b'} - M_{a'}$	− 2.4123	− 1.7374	4.622	4.1911	3.0186	19.3713	13.9520
Bottom chords	$L_o - L_1$	0.1262	0.0000	5.000	0.0000	0.0000	0.0000	0.0000
	$L_1 - L_2$	0.3289	1.4021	5.000	0.4612	1.9659	2.3060	9.8295
	$L_2 - L_3$	0.5482	2.5589	5.000	1.4028	6.5480	7.0140	32.7400
	$L_3 - L_4$	0.7675	3.3916	5.000	2.6031	11.5030	13.0155	57.5150
	$L_4 - L_5$	0.9868	3.9316	5.000	3.8797	15.4575	19.3985	77.2875
	$L_5 - L_6$	1.2061	4.1974	5.000	5.0625	17.6182	25.3125	88.0910
	$L_6 - L_{6'}$	1.4254	4.1974	5.000	5.9820	17.6182	29.9150	88.0910
	$L_{5'} - L_{4'}$	1.6447	3.9316	5.000	6.4663	15.4575	32.3315	77.2875
	$L_{4'} - L_{3'}$	1.8640	3.3916	5.000	6.3219	11.5030	31.6095	57.5150
	$L_{3'} - L_{2'}$	2.0833	2.5589	5.000	5.3310	6.5480	26.6550	32.7400
	$L_{2'} - L_{1'}$	2.3026	1.4021	5.000	3.2285	1.9659	16.1425	9.8295
	$L_{1'} - L_{0'}$	1.3882	0.0000	5.000	0.0000	0.0000	0.0000	0.0000
Diagonals	$L_o - M_a$	− 0.1512	− 0.0000	3.449	0.0000	0.0000	0.0000	0.0000
	$M_a - L_1$	0.1207	0.9896	2.848	0.1194	0.9793	0.3401	2.7890
	$L_1 - M_b$	− 0.1378	− 0.8348	3.140	0.1150	0.6969	0.3611	2.1883
	$M_b - L_2$	0.1378	0.8348	3.140	0.1150	0.6969	0.3611	2.1883
	$L_2 - M_c$	− 0.1378	− 0.6186	3.140	0.0852	0.3827	0.2675	1.2017
	$M_c - L_3$	0.1378	0.6186	3.140	0.0852	0.3827	0.2675	1.2017
	$L_3 - M_d$	− 0.1378	− 0.4275	3.140	0.0589	0.1828	0.1849	0.5740
	$M_d - L_4$	0.1378	0.4275	3.140	0.0589	0.1828	0.1849	0.5740
	$L_4 - M_e$	− 0.1378	− 0.2509	3.140	0.0346	0.0630	0.1086	0.1978
	$M_e - L_5$	0.1378	0.2509	3.140	0.0346	0.0630	0.1086	0.1978
	$L_5 - M_f$	− 0.1378	− 0.0830	3.140	0.0114	0.0069	0.0358	0.0217
	$M_f - L_6$	0.1378	0.0830	3.140	0.0114	0.0069	0.0358	0.0217
	$L_6 - M_{f'}$	− 0.1378	0.0830	3.140	− 0.0114	0.0069	− 0.0358	0.0217
	$M_{f'} - L_{5'}$	0.1378	− 0.0830	3.140	− 0.0114	0.0069	− 0.0358	0.0217
	$L_{5'} - M_{e'}$	− 0.1378	0.2509	3.140	− 0.0346	0.0630	− 0.1086	0.1978
	$M_{e'} - L_{4'}$	0.1378	− 0.2509	3.140	− 0.0346	0.0630	− 0.1086	0.1978
	$L_{4'} - M_{d'}$	− 0.1378	0.4275	3.140	− 0.0589	0.1828	− 0.1849	0.5740
	$M_{d'} - L_{3'}$	0.1378	− 0.4275	3.140	− 0.0589	0.1828	− 0.1849	0.5740
	$L_{3'} - M_{c'}$	− 0.1378	0.6186	3.140	− 0.0852	0.3827	− 0.2675	1.2017
	$M_{c'} - L_{2'}$	0.1378	− 0.6186	3.140	− 0.0852	0.3827	− 0.2675	1.2017
	$L_{2'} - M_{b'}$	− 0.1378	0.8348	3.140	− 0.1150	0.6969	− 0.3611	2.1883
	$M_{b'} - L_{1'}$	0.1378	− 0.8348	3.140	− 0.1150	0.6969	− 0.3611	2.1883
	$L_{1'} - M_{a'}$	1.3277	0.9896	2.848	+ 1.3139	0.9793	3.7420	2.7890
	$M_{a'} - L_{0'}$	− 1.6632	− 0.0000	3.449	+ 0.0000	0.0000	0.0000	0.0000

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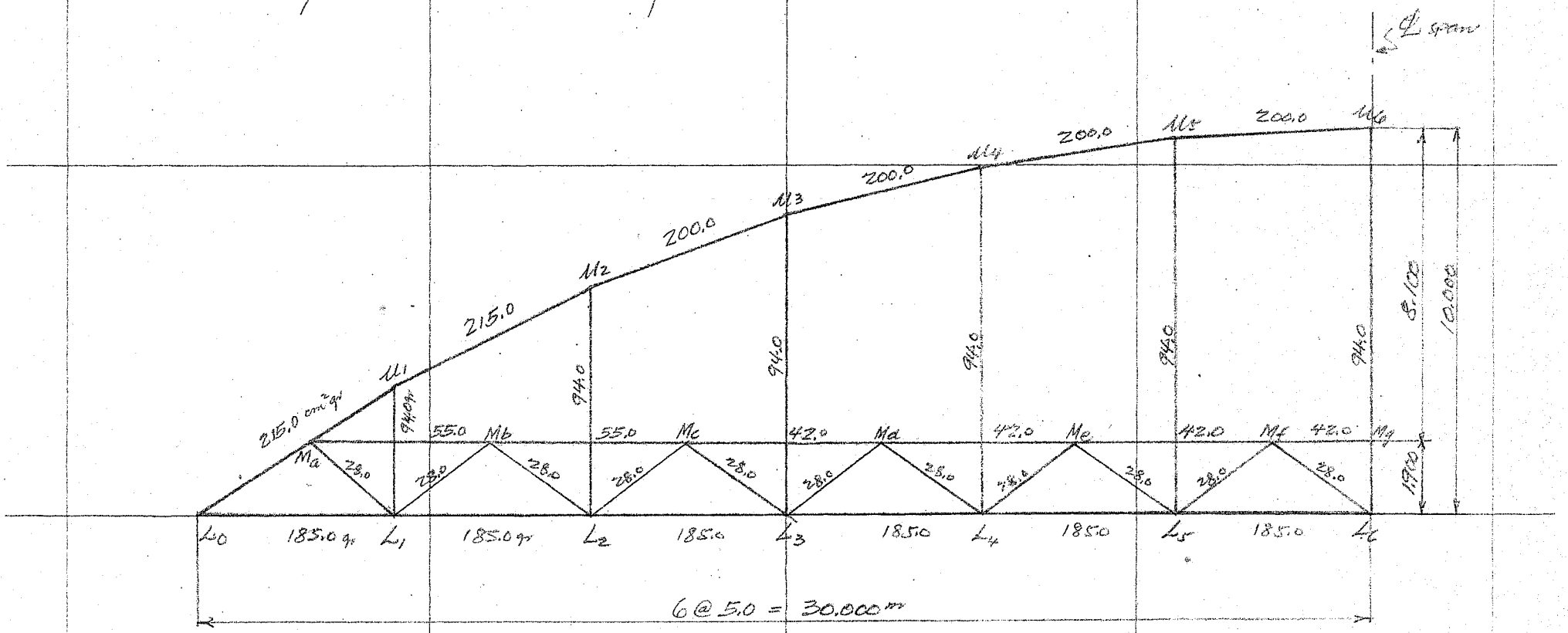
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CALCULATIONS FOR

		S_{oi}	S_a	L	$S_{oi}S_a$	S_a^2	$S_{oi}S_aL$	S_a^2L
Verticals	M_1-L_1	0.0000	−0.1552	3301	0.0000	0.0241	0.0000	0.0796
	M_2-L_2	"	−0.1308	5826	"	0.0171	"	0.0996
	M_3-L_3	"	−0.1156	7697	"	0.0134	"	0.1031
	M_4-L_4	"	−0.1068	8990	"	0.0114	"	0.1025
	M_5-L_5	"	−0.1016	9749	"	0.0103	"	0.1004
	M_6-L_6	"	−0.1004	10.000	"	0.0101	"	0.1010
	$M_5'-L_5'$	"	−0.1016	9749	"	0.0103	"	0.1004
	$M_4'-L_4'$	"	−0.1068	8990	"	0.0114	"	0.1025
	$M_3'-L_3'$	"	−0.1156	7697	"	0.0134	"	0.1031
	$M_2'-L_2'$	"	−0.1308	5826	"	0.0171	"	0.0996
	$M_1'-L_1'$	0.0000	−0.1552	3301	0.0000	0.0241	0.0000	0.0796

Assumed Gross sectional area of members.



$S_{oi}S_aL/A$ and S_a^2L/A

		Gross area A	$S_{oi}S_aL$	S_a^2L	$S_{oi}S_a^2L/A$	S_a^2L/A
Top Chords	M_a-M_1	215.0 cm²	0.0000	3.6483	0.0000	1.697
	M_1-M_2	215.0	"	7.0259	"	3.268
	M_2-M_3	200.0	"	6.0897	"	3.045
	M_3-M_4	200.0	"	5.5105	"	2.755
	M_4-M_5	200.0	"	5.1688	"	2.584
	M_5-M_6	200.0	"	5.0160	"	2.508
	M_6-M_5'	200.0	"	5.0160	"	2.508
	$M_5'-M_4'$	200.0	"	5.1688	"	2.584
	$M_4'-M_3'$	200.0	"	5.5105	"	2.755
	$M_3'-M_2'$	200.0	"	6.0897	"	3.045
	$M_2'-M_1'$	215.0	"	7.0259	"	3.268
	$M_1'-M_a$	215.0	0.0000	3.6483	0.0000	1.697
					Summary	31.714

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		Gross area A	$S_0 S_a L$	$S_a^2 L$	$S_0 S_a L/A$	$S_a^2 L/A$	
Middle chords	$M_a - M_b$	55.0 cm²	1.7610	13.9520	3.202	25.367	
	$M_b - M_c$	55.0	6.7245	47.0110	12.226	85.475	
	$M_c - M_d$	42.0	13.3260	82.0570	31.729	195.374	
	$M_a - M_e$	42.0	20.7530	111.9400	49.412	266.524	
	$M_e - M_f$	42.0	28.1315	131.6410	66.980	313.431	
	$M_f - M_{f'}$	42.0	34.6265	138.5065	82.444	329.777	
	$M_{f'} - M_{e'}$	42.0	39.3840	131.6410	93.771	313.431	
	$M_{e'} - M_{a'}$	42.0	41.5055	111.9400	98.823	266.524	
	$M_{a'} - M_{c'}$	42.0	39.9785	82.0570	95.187	195.374	
	$M_{c'} - M_{b'}$	55.0	33.6220	47.0110	61.131	85.475	
	$M_{b'} - M_{a'}$	55.0	19.3713	13.9520	35.221	25.367	
					Summary		2102.119
Bottom chords	$L_0 - L_1$	185.0	0.0000	0.0000	0.000	0.000	
	$L_1 - L_2$	185.0	2.3060	9.8295	1.246	5.313	
	$L_2 - L_3$	185.0	7.0140	32.7400	3.791	17.697	
	$L_3 - L_4$	185.0	13.0155	57.5150	7.035	31.089	
	$L_4 - L_5$	185.0	19.3985	77.2875	10.486	41.777	
	$L_5 - L_6$	185.0	25.3125	88.0910	13.682	47.617	
	$L_6 - L_{5'}$	185.0	29.9150	88.0910	16.170	47.617	
	$L_{5'} - L_{4'}$	185.0	32.3315	77.2875	17.476	41.777	
	$L_{4'} - L_{3'}$	185.0	31.6095	57.5150	17.086	31.089	
	$L_{3'} - L_{2'}$	185.0	26.6550	32.7400	14.408	17.697	
	$L_{2'} - L_{1'}$	185.0	16.1425	9.8295	8.726	5.313	
	$L_{1'} - L_{0'}$	185.0	0.0000	0.0000	0.000	0.000	
					Summary		286.986
Diagonals	$L_0 - M_a$	28.0	0.0000	0.0000	0.000	0.000	
	$M_a - L_1$	28.0	0.3401	2.7890	1.215	9.961	
	$L_1 - M_b$	28.0	0.3611	2.1883	1.290	7.815	
	$M_b - L_2$	28.0	0.3611	2.1883	1.290	7.815	
	$L_2 - M_c$	28.0	0.2675	1.2017	0.955	4.292	
	$M_c - L_3$	28.0	0.2675	1.2017	0.955	4.292	
	$L_3 - M_d$	28.0	0.1849	0.5740	0.660	2.050	
	$M_d - L_4$	28.0	0.1849	0.5740	0.660	2.050	
	$L_4 - M_e$	28.0	0.1086	0.1978	0.388	0.706	
	$M_e - L_5$	28.0	0.1086	0.1978	0.388	0.706	
	$L_5 - M_f$	28.0	0.0358	0.0217	0.128	0.077	
	$M_f - L_6$	28.0	0.0358	0.0217	0.128	0.077	
	$L_6 - M_{f'}$	28.0	−0.0358	0.0217	−0.128	0.077	
	$M_{f'} - L_{5'}$	28.0	−0.0358	0.0217	−0.128	0.077	
	$L_{5'} - M_{e'}$	28.0	−0.1086	0.1978	−0.388	0.706	
	$M_{e'} - L_{4'}$	28.0	−0.1086	0.1978	−0.388	0.706	
	$L_{4'} - M_{d'}$	28.0	−0.1849	0.5740	−0.660	2.050	
	$M_{d'} - L_{3'}$	28.0	−0.1849	0.5740	−0.660	2.050	
	$L_{3'} - M_{c'}$	28.0	−0.2675	1.2017	−0.955	4.292	
	$M_{c'} - L_{2'}$	28.0	−0.2675	1.2017	−0.955	4.292	
	$L_{2'} - M_{b'}$	28.0	−0.3611	2.1883	−1.290	7.815	
	$M_{b'} - L_{1'}$	28.0	−0.3611	2.1883	−1.290	7.815	
	$L_{1'} - M_{a'}$	28.0	3.7420	2.7890	13.364	9.961	
	$M_{a'} - L_{0'}$	28.0	0.0000	0.0000	0.000	0.000	
					Summary		79.682

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CALCULATIONS FOR

		Gross area A	$S_o S_a L$	$S_a^2 L$	$S_o S_a^2/A$	S_a^2/A
Verticals	$U_1 - L_1$	94.0 cm²	0.0000	0.0769	0.0000	0.082
	$U_2 - L_2$	94.0	=	0.0996	=	0.106
	$U_3 - L_3$	94.0	=	0.1031	=	0.110
	$U_4 - L_4$	94.0	=	0.1025	=	0.109
	$U_5 - L_5$	94.0	=	0.1004	=	0.107
	$U_6 - L_6$	94.0	=	0.1010	=	0.107
	$U_5' - L_5'$	94.0	=	0.1004	=	0.107
	$U_4' - L_4'$	94.0	=	0.1025	=	0.109
	$U_3' - L_3'$	94.0	=	0.1031	=	0.110
	$U_2' - L_2'$	94.0	=	0.0996	=	0.106
	$U_1' - L_1'$	94.0	0.0000	0.0796	=	0.082
					Dummy	1.135

Dummary of S_a^2/A for the entire truss.

	$\Sigma S_a^2/A$
Top chords	31.714
middle chords	2,102.119
bottom "	286.986
diagonals	79.682
verticals	1.135
$\Sigma S_a^2/A$ =	2501.636

Dummary of $S_o S_a/A$

Middle chords.

	$S_o S_a/A$	Summary	Loaded point.				$\Sigma S_o S_a/A$
1	3.202	3.202	1				630.126
2	12.226	15.428					
3	31.729	47.157	2	2@533.774 + 10@	15.428 =	1,067.548 + 154.280 =	1,221.828
4	49.412	96.569					
5	66.980	163.549	3	3@438.587 + 9@	47.157 =	1315.761 + 424.413 =	1740.174
6	82.444	245.993					
7	93.771	339.764	4	4@339.764 + 8@	96.569 =	1359.056 + 772.552 =	2131.608
8	98.823	438.587					
9	95.187	533.774	5	5@245.993 + 7@	163.549 =	1229.965 + 1144.843 =	2374.808
10	61.131	594.905					
11	35.221	630.126	6	6@163.549 + 6@	245.993 =	981.294 + 1475.958 =	2457.252

Bottom chords.

1	0.000	0.000	1				110.106
2	1.246	1.246					
3	3.791	5.037	2	2@101.380 + 10@	1.246 =	202.760 + 12.460 =	215.220
4	7.035	12.072					
5	10.486	22.558	3	3@86.972 + 9@	5.037 =	260.916 + 45.333 =	306.249
6	13.682	36.240					
7	16.170	52.410	4	4@69.886 + 8@	12.072 =	279.544 + 96.576 =	376.120
8	17.476	69.886					
9	17.086	86.972	5	5@52.410 + 7@	22.558 =	262.050 + 157.906 =	419.956
10	14.408	101.380					
11	8.726	110.106	6	6@36.240 + 6@	36.240 =	217.440 + 217.440 =	434.880
12	0.000	110.106					

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CALCULATIONS FOR

Diagonals

	$So\,S_A/A$		Summary	Loaded point.				$\Sigma\, So\,S_A/A$
1	0.000	=	0.000	1				14.579
2	1.215	=	1.215					
3	1.290	=	2.505	2	2@	3.795 + 10@ 3.795 =		45.540
4	1.290	=	3.795					
5	0.955	=	4.750	3	3@	5.705 + 9@ 5.705 =		68.460
6	0.955	=	5.705					
7	0.660	=	6.365	4	4@	7.025 + 8@ 7.025 =		84.300
8	0.660	=	7.025					
9	0.388	=	7.413	5	5@	7.801 + 7@ 7.801 =		93.612
10	0.388	=	7.801					
11	0.128	=	7.929	6	6@	8.057 + 6@ 8.057 =		96.684
12	0.128	=	8.057					
13	−0.128	=	7.929					
14	−0.128	=	7.801					
15	−0.388	=	7.413					
16	−0.388	=	7.025					
17	−0.660	=	6.365					
18	−0.660	=	5.705					
19	−0.955	=	4.750					
20	−0.955	=	3.795					
21	−1.290	=	2.505					
22	−1.290	=	1.215					
23	13.364	=	14.579					
24	0.000	=	14.579					

$\Sigma\, So\,S_A/A$ for Top chords and Verticals are zero for all points.

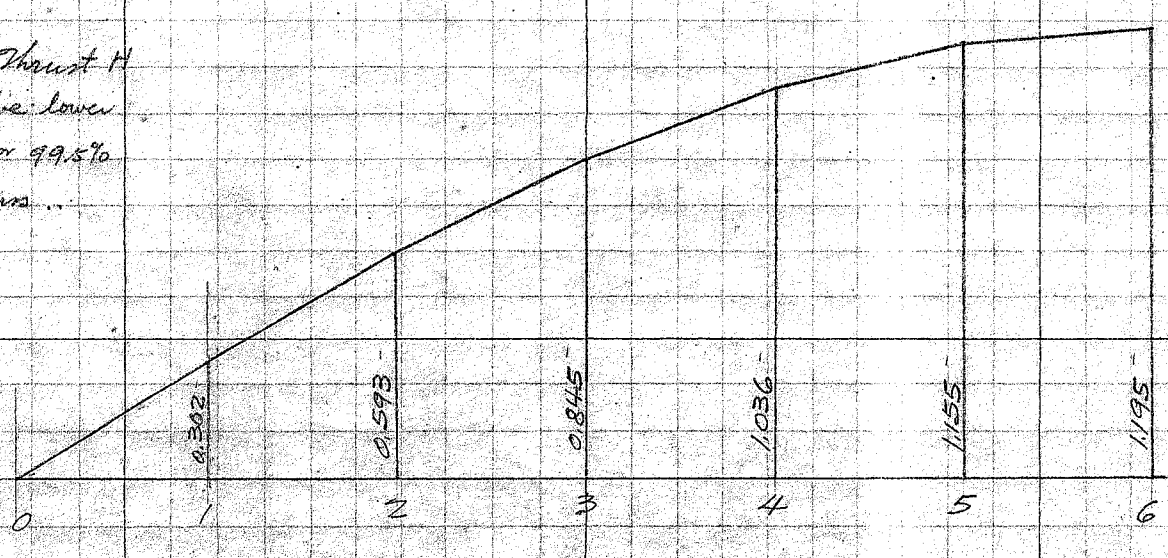
Influence values of Horizontal Thrust H.

Loaded Points	Top chords	Middle chords	Bottom chords	Diagonals	Verticals	Summary of $So\,S_A/A$	Summary of S_A^2/A	Thrust H
1	0.000 +	630.126 +	110.106 +	14.579 +	0.000 =	754.811 ÷	2501.636 =	0.302 H_1
2	〃	1221.828	215.220	45.540	〃	1482.588	〃 =	0.593 H_2
3	〃	1740.174	306.249	68.460	〃	2114.883	〃 =	0.845 H_3
4	〃	2131.608	376.120	84.300	〃	2592.028	〃 =	1.036 H_4
5	〃	2374.808	419.956	93.612	〃	2888.376	〃 =	1.155 H_5
6	〃	2457.252	434.880	96.684	〃	2988.816	〃 =	1.195 H_6

Summary for the entire span ΣH = 9.057

Line of action of Thrust H for full load, above lower chord by 9.95m, or 99.5% of the depth of truss...

$\frac{9.000}{9.057} = 0.995$



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Influence surface of Stress in Truss members.

Members	Values of Sa	Load on panel point 1	2	3	4	5	6
		H= 0.302	0.593	0.845	1.036	1.155	1.195
-SaH for Top chords.							
Ma - M1	1.1980	-0.362	-0.710	-1.012	-1.241	-1.384	-1.432
M1 - M2	1.1200	-0.338	-0.664	-0.946	-1.160	-1.294	-1.338
M2 - M3	1.0680	-0.323	-0.633	-0.902	-1.106	-1.234	-1.276
M3 - M4	1.0330	-0.312	-0.613	-0.873	-1.070	-1.193	-1.234
M4 - M5	1.0110	-0.305	-0.600	-0.854	-1.047	-1.168	-1.208
M5 - M6	1.0010	-0.302	-0.594	-0.846	-1.037	-1.156	-1.196
-SaH for Middle chords							
Ma - Mb	-1.7374	0.525	1.030	1.468	1.800	2.007	2.076
Mb - Mc	-3.0663	0.926	1.818	2.591	3.177	3.542	3.664
Mc - Md	-4.0511	1.223	2.402	3.423	4.197	4.679	4.841
Ma - Me	-4.7316	1.429	2.806	3.998	4.902	5.465	5.654
Me - Mf	-5.1311	1.550	3.043	4.336	5.316	5.926	6.132
Mf - Mf'	-5.2632	1.589	3.121	4.447	5.453	6.079	6.290
-SaH for Bottom chords							
L0 - L1	0.0000	0.000	0.000	0.000	0.000	0.000	0.000
L1 - L2	1.4021	-0.423	-0.831	-1.185	-1.453	-1.619	-1.676
L2 - L3	2.5589	-0.773	-1.517	-2.162	-2.651	-2.956	-3.058
L3 - L4	3.3916	-1.024	-2.011	-2.866	-3.514	-3.917	-4.053
L4 - L5	3.9316	-1.187	-2.331	-3.322	-4.073	-4.541	-4.698
L5 - L6	4.1974	-1.268	-2.489	-3.547	-4.349	-4.848	-5.016
-SaH for Diagonals.							
L0 - Ma	-0.0000	0.000	0.000	0.000	0.000	0.000	0.000
Ma - L1	0.9896	-0.299	-0.587	-0.836	-1.025	-1.143	-1.183
L1 - Mb	-0.8348	0.252	0.495	0.705	0.865	0.964	0.998
Mb - L2	0.8348	-0.252	-0.495	-0.705	-0.865	-0.964	-0.998
L2 - Mc	-0.6186	0.187	0.367	0.523	0.641	0.714	0.739
Mc - L3	0.6186	-0.187	-0.367	-0.523	-0.641	-0.714	-0.739
L3 - Md	-0.4275	0.129	0.254	0.361	0.443	0.494	0.511
Md - L4	0.4275	-0.129	-0.254	-0.361	-0.443	-0.494	-0.511
L4 - Me	-0.2509	0.076	0.149	0.212	0.260	0.290	0.300
Me - L5	0.2509	-0.076	-0.149	-0.212	-0.260	-0.290	-0.300
L5 - Mf	-0.0830	0.025	0.049	0.070	0.086	0.096	0.099
Mf - L6	0.0830	-0.025	-0.049	-0.070	-0.086	-0.096	-0.099
-SaH for Verticals.							
M1 - L1	-0.1552	0.047	0.092	0.131	0.161	0.179	0.185
M2 - L2	-0.1308	0.040	0.078	0.111	0.136	0.151	0.156
M3 - L3	-0.1156	0.035	0.069	0.098	0.120	0.134	0.138
M4 - L4	-0.1068	0.032	0.063	0.090	0.111	0.123	0.128
M5 - L5	-0.1016	0.030	0.060	0.086	0.105	0.117	0.121
M6 - L6	-0.1004	0.030	0.060	0.085	0.104	0.116	0.120

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CALCULATIONS FOR

Stress Influence surface of Top chords.

	Ma − U1 So	Ma − U1 −SaH	Ma − U1 Final	U1 − U2 So	U1 − U2 −SaH	U1 − U2 Final	U2 − U3 So	U2 − U3 −SaH	U2 − U3 Final	U3 − U4 So	U3 − U4 −SaH	U3 − U4 Final
1	0.000	−0.362	−0.362	0.000		−0.338	0.000		−0.323	0.000		−0.312
2	〃	−0.710	−0.710	〃		−0.664	〃		−0.633	〃		−0.613
3	〃	−1.012	−1.012	〃		−0.946	〃		−0.902	〃		−0.873
4	〃	−1.241	−1.241	〃		−1.160	〃		−1.106	〃		−1.070
5	〃	−1.384	−1.384	〃		−1.294	〃		−1.234	〃		−1.193
6	〃	−1.432	* −1.432	〃		* −1.338	〃		* −1.276	〃		* −1.234
5	〃	−1.384	−1.384	〃		−1.294	〃		−1.234	〃		−1.193
4	〃	−1.241	−1.241	〃		−1.160	〃		−1.106	〃		−1.070
3	〃	−1.012	−1.012	〃		−0.946	〃		−0.902	〃		−0.873
2	〃	−0.710	−0.710	〃		−0.664	〃		−0.633	〃		−0.613
1	〃	−0.362	−0.362	〃		−0.338	〃		−0.323	〃		−0.312
	Summary		−10.850			−10.142			−9.672			−9.356

	Mp − M5			M5 − M6		
1	0.000		−0.305	0.000		−0.302
2	〃		−0.600	〃		−0.594
3	〃		−0.854	〃		−0.846
4	〃		−1.047	〃		−1.037
5	〃		−1.168	〃		−1.156
6	〃		* −1.208	〃		* −1.196
5	〃		−1.168	〃		−1.156
4	〃		−1.047	〃		−1.037
3	〃		−0.854	〃		−0.846
2	〃		−0.600	〃		−0.594
1	〃		−0.305	〃		−0.302
	Summary		−9.156			−9.066

Stress Influence surface of Middle chords.

	Ma − Mb So	Ma − Mb −SaH	Ma − Mb Final	Mb − Mc So	Mb − Mc −SaH	Mb − Mc Final	Mc − Md So	Mc − Md −SaH	Mc − Md Final	Md − Me So	Md − Me −SaH	Md − Me Final
1	−0.219	0.525	0.306	−0.439	0.926	0.487	−0.658	1.233	0.575	−0.877	1.429	0.552
2	−0.439	1.030	0.591	−0.877	1.818	0.941	−1.316	2.402	1.086	−1.754	2.806	1.052
3	−0.658	1.468	0.810	−1.316	2.591	1.275	−1.974	3.423	1.449	−2.632	3.998	1.366
4	−0.877	1.800	* 0.923	−1.754	3.177	* 1.423	−2.632	4.197	* 1.565	−3.509	4.902	* 1.393
5	−1.097	2.007	0.910	−2.193	3.542	1.349	−3.290	4.679	1.389	−4.386	5.465	1.079
6	−1.316	2.076	0.760	−2.632	3.664	1.032	−3947	4.841	0.894	−5.263	5.654	0.391
5	−1.535	2.007	0.472	−3.070	3.542	0.472	−4.605	4.679	0.074	−6.140	5.465	−0.675
4	−1.754	1.800	0.046	−3.509	3.177	−0.332	−5.263	4.197	−1.066	−7.018	4.902	* −2.116
3	−1.974	1.468	−0.506	−3947	2.591	−1.356	−5.921	3.423	* −2.498	−5.263	3.998	−1.265
2	−2.193	1.030	−1.163	−4.386	1.818	* −2.568	−3.947	2.402	−1.545	−3.509	2.806	−0.703
1	−2.412	0.525	* −1.887	−2.193	0.926	−1.267	−1.974	1.223	−0.751	−1.754	1.429	−0.325
	Summary		4.818			6.979			7.032			5.833
			−3.556			−5.523			−5.860			−5.084
			1.262			1.456			1.172			0.749

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CALCULATIONS FOR

	Me-Mf So	-Sa.H	Final	Mf-Mf' So	-Sa.H	Final	So	-Sa.H	Final	So	-Sa.H	Final
1	-1.097	1.550	0.453	-1.316	1.589	0.273						
2	-2.193	3.043	0.850	-2.632	3.121	0.489						
3	-3.290	4.336	* 1.046	-3.947	4.447	* 0.500						
4	-4.386	5.316	0.930	-5.263	5.453	0.190						
5	-5.483	5.926	0.443	-6.579	6.079	-0.500						
6	-6.579	6.132	-0.447	-7.895	6.290	* -1.605						
5	-7.676	5.926	* -1.750	-6.579	6.079	-0.500						
4	-6.140	5.316	-0.824	-5.263	5.453	0.190						
3	-4.605	4.336	-0.269	-3.947	4.447	* 0.500						
2	-3.070	3.043	-0.027	-2.632	3.121	0.489						
1	-1.535	1.550	0.015	-1.316	1.589	0.273						
	Summary		3.737			2.904						
			-3317			-2.605						
			0.420			0.299						

Stress Influence Surface of Bottom chords.

	Lo-L1			L1-L2			L2-L3			L3-L4		
1	0.126	0.000	0.126	0.329	-0.423	-0.094	0.548	-0.773	-0.225	0.768	-1.024	-0.256
2	0.252	=	0.252	0.658	-0.831	-0.173	1.096	-1.517	-0.421	1.535	-2.011	-0.476
3	0.379	=	0.379	0.987	-1.185	* -0.198	1.645	-2.162	* -0.517	2.303	-2.866	* -0.563
4	0.505	=	0.505	1.316	-1.453	-0.137	2.193	-2.651	-0.458	3.070	-3.514	-0.444
5	0.631	=	0.631	1.645	-1.619	0.026	2.741	-2.956	-0.215	3.838	-3.917	-0.079
6	0.757	=	0.757	1.973	-1.676	0.297	3.289	-3.058	0.231	4.605	-4.053	0.552
5	0.883	=	0.883	2.302	-1.619	0.683	3.837	-2.956	0.881	5.373	-3.917	1.456
4	1.010	=	1.010	2.631	-1.453	1.178	4.386	-2.651	1.735	6.140	-3.514	2.626
3	1.136	=	1.136	2.960	-1.185	1.775	4.934	-2.162	* 2.772	5.592	-2.866	* 2.726
2	1.262	=	1.262	3.289	-0.831	* 2.458	4.167	-1.517	2.650	3.728	-2.011	1.717
1	1.388	=	* 1.388	2.302	-0.423	1.879	2.083	-0.773	1.310	1.864	-1.024	0.840
	Summary		8.329			8.296			9.579			9.918
			-0.000			-0.602			-1.836			-1.819
			8.329			7.694			7.743			8.099

	L4-L5			L5-L6		
1	0.987	-1.187	-0.200	1.206	-1.268	-0.062
2	1.974	-2.331	-0.357	2.412	-2.489	* -0.077
3	2.960	-3.322	* -0.362	3.618	-3.547	0.071
4	3.947	-4.073	-0.126	4.824	-4.349	0.475
5	4.934	-4.541	0.393	6.031	-4.848	1.183
6	5.921	-4.698	1.223	7.237	-5.016	2.221
5	6.908	-4.541	2.367	7.127	-4.848	* 2.279
4	6.579	-4.073	* 2.506	5.702	-4.349	1.353
3	4.934	-3.322	1.612	4.276	-3.547	0.729
2	3.289	-2.331	0.958	2.851	-2.489	0.362
1	1.645	-1.187	0.458	1.425	-1.268	0.157
	Summary		9.517			8.830
			-1.045			-0.139
			8.472			8.691

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CALCULATIONS FOR

Stress Influence surface of Diagonal members.

	L_0-M_a So	L_0-M_a -SaH	L_0-M_a Final	M_a-L_1 So	M_a-L_1 -SaH	M_a-L_1 Final	L_1-M_b So	L_1-M_b -SaH	L_1-M_b Final	M_b-L_2 So	M_b-L_2 -SaH	M_b-L_2 Final
1	-0.151	0.000	-0.151	0.121	-0.299	-0.178	-0.138	0.252	0.114	0.138	-0.252	-0.114
2	-0.302	〃	-0.302	0.241	-0.587	-0.346	-0.276	0.495	0.219	0.276	-0.495	-0.219
3	-0.454	〃	-0.454	0.362	-0.836	-0.474	-0.413	0.705	0.292	0.413	-0.705	-0.292
4	-0.605	〃	-0.605	0.483	-1.025	*-0.542	-0.551	0.865	0.314	0.551	-0.865	-0.314
5	-0.756	〃	-0.756	0.604	-1.143	-0.539	-0.689	0.964	0.275	0.689	-0.964	-0.275
6	-0.907	〃	-0.907	0.724	-1.183	-0.459	-0.827	0.998	0.171	0.827	-0.998	-0.171
5	-1.058	〃	-1.058	0.845	-1.143	-0.298	-0.965	0.964	-0.001	0.965	-0.964	0.001
4	-1.210	〃	-1.210	0.966	-1.025	-0.059	-1.102	0.865	-0.237	1.102	-0.865	0.237
3	-1.361	〃	-1.361	1.086	-0.836	0.250	-1.240	0.705	-0.535	1.240	-0.705	0.535
2	-1.512	〃	-1.512	1.207	-0.587	0.620	-1.378	0.495	*-0.883	1.378	-0.495	*0.883
1	-1.663	〃	* -1.663	1.328	-0.299	* 1.029	+0.138	0.252	* 0.390	-0.138	-0.252	*-0.390
			+0.000			1.899			1.775			1.656
	Summary		-9.979			-2.895			-1.656			-1.775
			-9.979			-0.996			0.119			-0.119

	L_2-M_c			M_c-L_3			L_3-M_d			M_d-L_4		
1	-0.138	0.187	0.049	0.138	-0.187	-0.049	-0.138	0.129	-0.009	0.138	-0.129	0.009
2	-0.276	0.367	0.091	0.276	-0.367	-0.091	-0.276	0.254	-0.022	0.276	-0.254	0.022
3	-0.413	0.523	0.110	0.413	-0.523	-0.110	-0.413	0.361	-0.052	0.413	-0.361	0.052
4	-0.551	0.641	0.090	0.551	-0.641	-0.090	-0.551	0.443	-0.108	0.551	-0.443	0.108
5	-0.689	0.714	0.025	0.689	-0.714	-0.025	-0.689	0.494	-0.195	0.689	-0.494	0.195
6	-0.827	0.739	-0.088	0.827	-0.739	0.088	-0.827	0.511	-0.316	0.827	-0.511	0.316
5	-0.965	0.714	-0.251	0.965	-0.714	0.251	-0.965	0.494	-0.471	0.965	-0.494	-0.471
4	-1.102	0.641	-0.461	1.102	-0.641	0.461	-1.102	0.443	*-0.659	1.102	-0.443	* 0.659
3	-1.240	0.523	*-0.717	1.240	-0.523	* 0.717	0.413	0.361	* 0.774	-0.413	-0.361	*-0.774
2	0.276	0.367	* 0.643	-0.276	-0.367	*-0.643	0.276	0.254	0.530	-0.276	-0.254	-0.530
1	0.138	0.187	0.325	-0.138	-0.187	-0.325	0.138	0.129	0.267	-0.138	-0.129	-0.267
			1.333			1.517			1.571			1.832
	Summary		-1.517			-1.333			-1.832			-1.571
			-0.184			0.184			-0.261			0.261

	L_4-M_e			M_e-L_5			L_5-M_f			M_f-L_6		
1	-0.138	0.076	-0.062	0.138	-0.076	0.062	-0.138	0.025	-0.113	0.138	-0.025	0.113
2	-0.276	0.149	-0.127	0.276	-0.149	0.127	-0.276	0.049	-0.227	0.276	-0.049	0.227
3	-0.413	0.212	-0.201	0.413	-0.212	0.201	-0.413	0.070	-0.343	0.413	-0.070	0.343
4	-0.551	0.260	-0.291	0.551	-0.260	0.291	-0.551	0.086	-0.465	0.551	-0.086	0.465
5	-0.689	0.290	-0.399	0.689	-0.290	0.399	-0.689	0.096	-0.593	0.689	-0.096	0.593
6	-0.827	0.300	-0.527	0.827	-0.300	0.527	-0.827	0.099	*-0.728	0.827	-0.099	* 0.728
5	-0.965	0.290	*-0.675	0.965	-0.290	* 0.675	0.689	0.096	* 0.785	-0.689	-0.096	*-0.785
4	0.551	0.260	* 0.811	-0.551	-0.260	*-0.811	0.551	0.086	0.637	-0.551	-0.086	-0.637
3	0.413	0.212	0.625	-0.413	-0.212	-0.625	0.413	0.070	0.483	-0.413	-0.070	-0.483
2	0.276	0.149	0.425	-0.276	-0.149	-0.425	0.276	0.049	0.325	-0.276	-0.049	-0.325
1	0.138	0.076	0.214	-0.138	-0.076	-0.214	0.138	0.025	0.163	-0.138	-0.025	-0.163
			2.075			2.282			2.393			2.469
	Summary		-2.282			-2.075			-2.469			-2.393
			-0.207			0.207			-0.076			0.076

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CALCULATIONS FOR

Stress Influence surface of Vertical members.

	M_1-L_1 S_o	-S_aH	Final	M_2-L_2 S_o	-S_aH	Final	M_3-L_3 S_o	-S_aH	Final	M_4-L_4 S_o	-S_aH	Final
1	0.000	0.047	0.047	0.000		0.040	0.000		0.035	0.000		0.032
2	"	0.092	0.092	"		0.078	"		0.069	"		0.063
3	"	0.131	0.131	"		0.111	"		0.098	"		0.090
4	"	0.161	0.161	"		0.136	"		0.120	"		0.111
5	"	0.179	0.179	"		0.151	"		0.134	"		0.123
6	"	0.185	* 0.185	"		* 0.156	"		* 0.138	"		* 0.128
5	"	0.179	0.179	"		0.151	"		0.134	"		0.123
4	"	0.161	0.161	"		0.136	"		0.120	"		0.111
3	"	0.131	0.131	"		0.111	"		0.098	"		0.090
2	"	0.092	0.092	"		0.078	"		0.069	"		0.063
1	"	0.047	0.047	"		0.040	"		0.035	"		0.032
	Summary		1.405			1.188			1.050			0.966

	M_5-L_5		M_6-L_6	
1	0.000	0.030	0.000	0.030
2	"	0.060	"	0.060
3	"	0.086	"	0.085
4	"	0.105	"	0.104
5	"	0.117	"	0.116
6	"	* 0.121	"	* 0.120
5	"	0.117	"	0.116
4	"	0.105	"	0.104
3	"	0.086	"	0.085
2	"	0.060	"	0.060
1	"	0.030	"	0.030
	Summary	0.917		0.910

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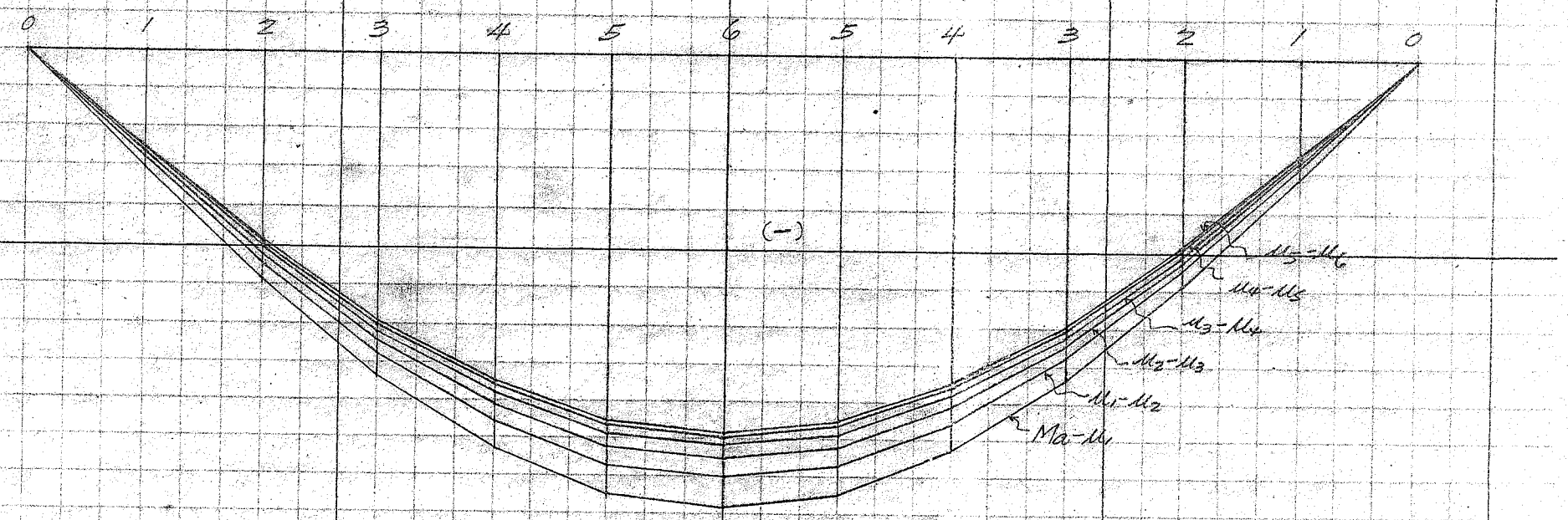
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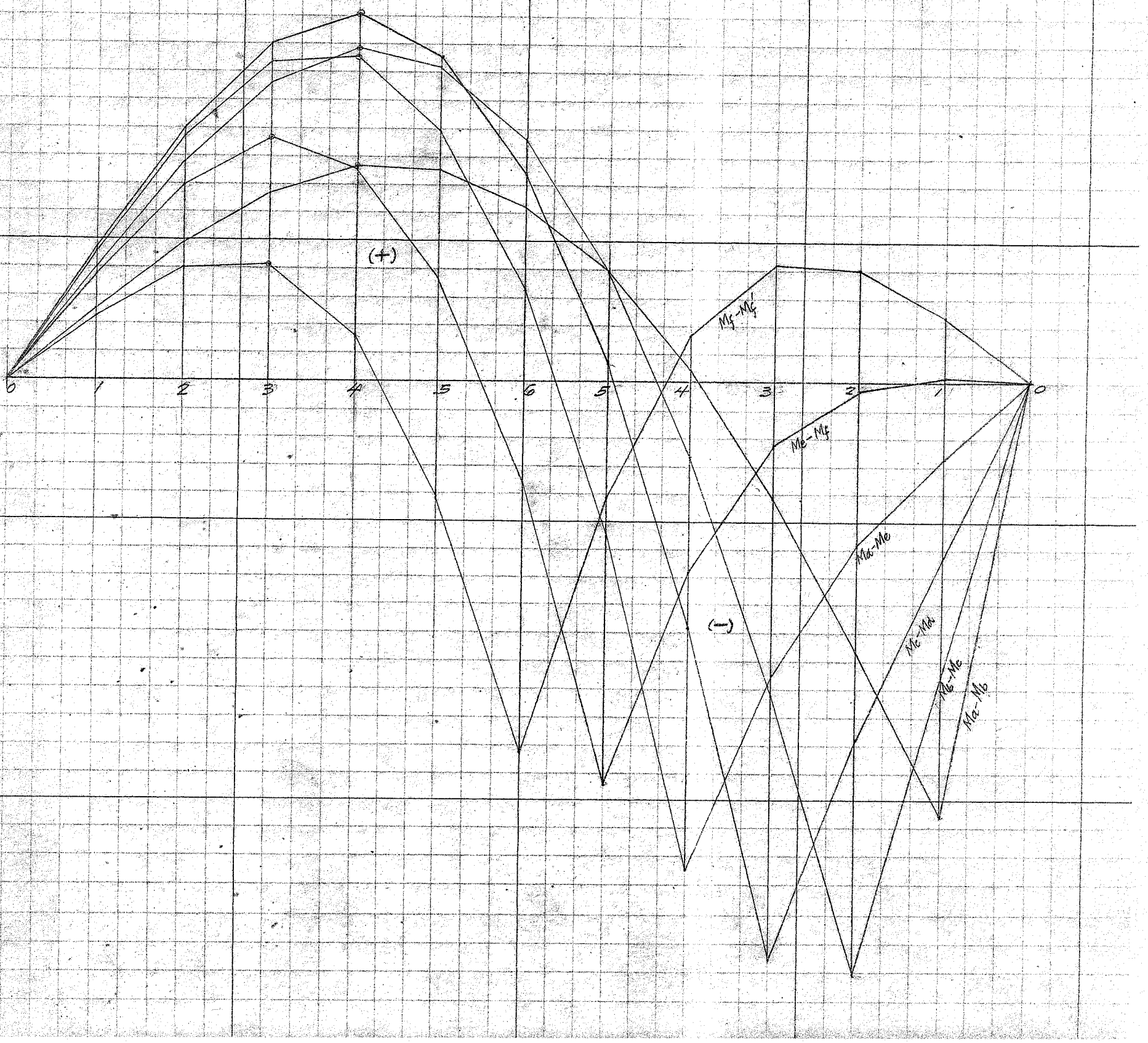
CALCULATIONS FOR

Influence Diagram of Top chord stresses.

− Compressive Stress.
+ Tensile "
Scale of Stress $\frac{1}{20}$ m = 1,000 kg



Influence Diagram of Middle chord stresses.



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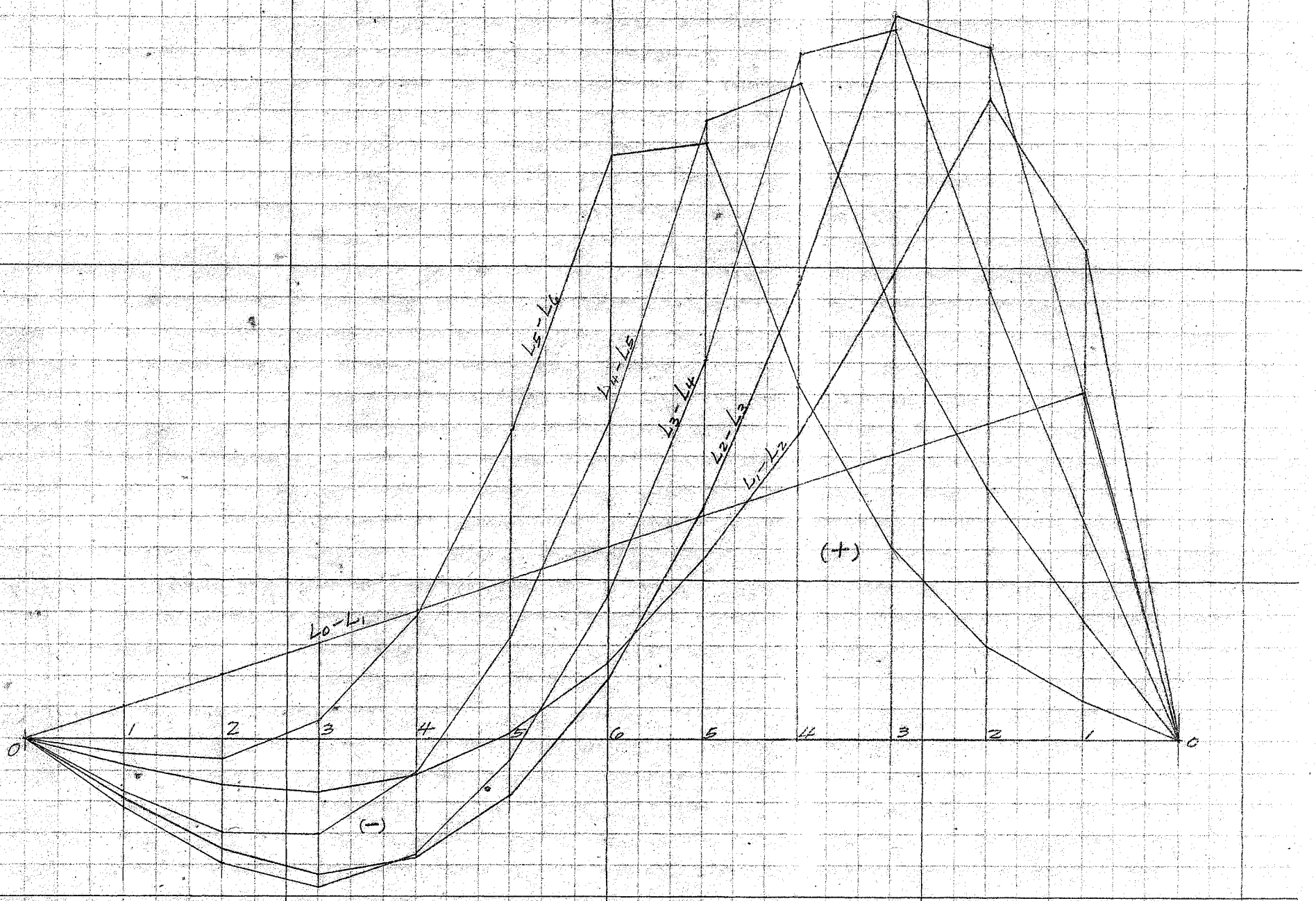
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CALCULATIONS FOR

Influence Diagram of Bottom Chord stresses.

note: − Sign show Compressive stress.
+ " " Tensile "

Scale of Stress 20^m = 1.000 kg.



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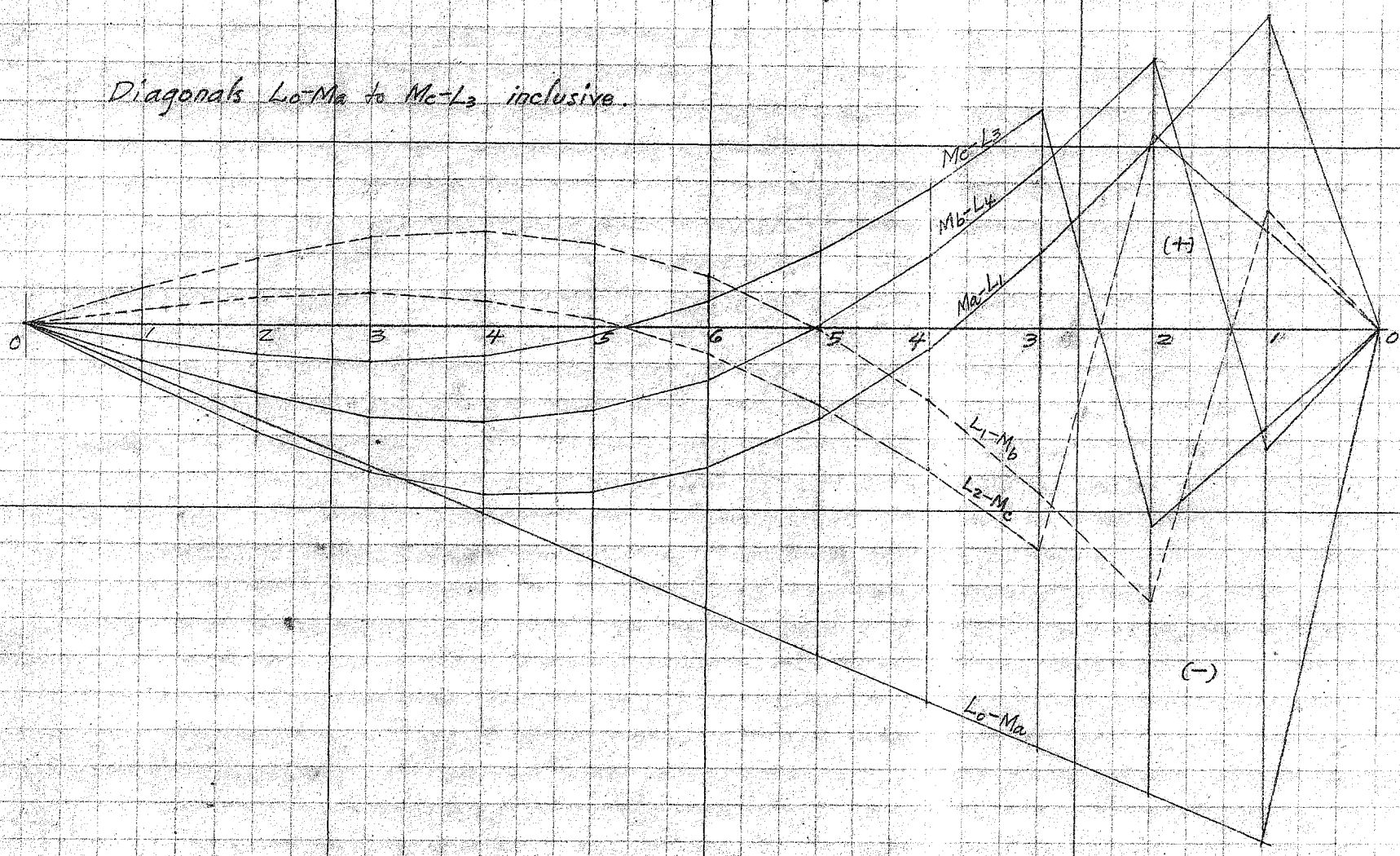
CALCULATIONS FOR

Influence Diagram of Diagonal stresses.

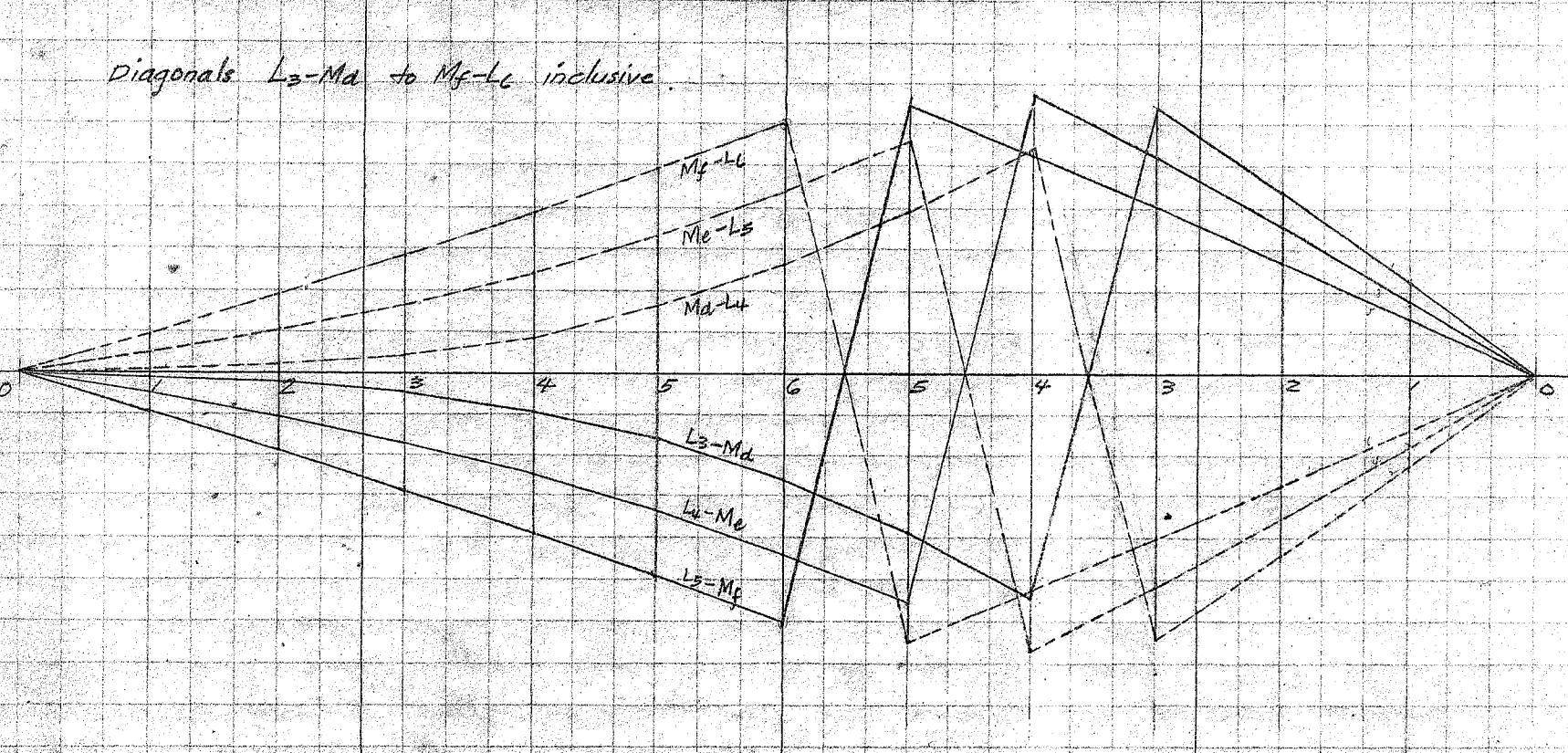
Note: + Sign show Tensile stress.
− " " Compressive "

Scale of stress 1/20 m = 1,000 kg.

Diagonals L_0-M_a to M_c-L_3 inclusive.



Diagonals L_3-M_d to M_f-L_6 inclusive.



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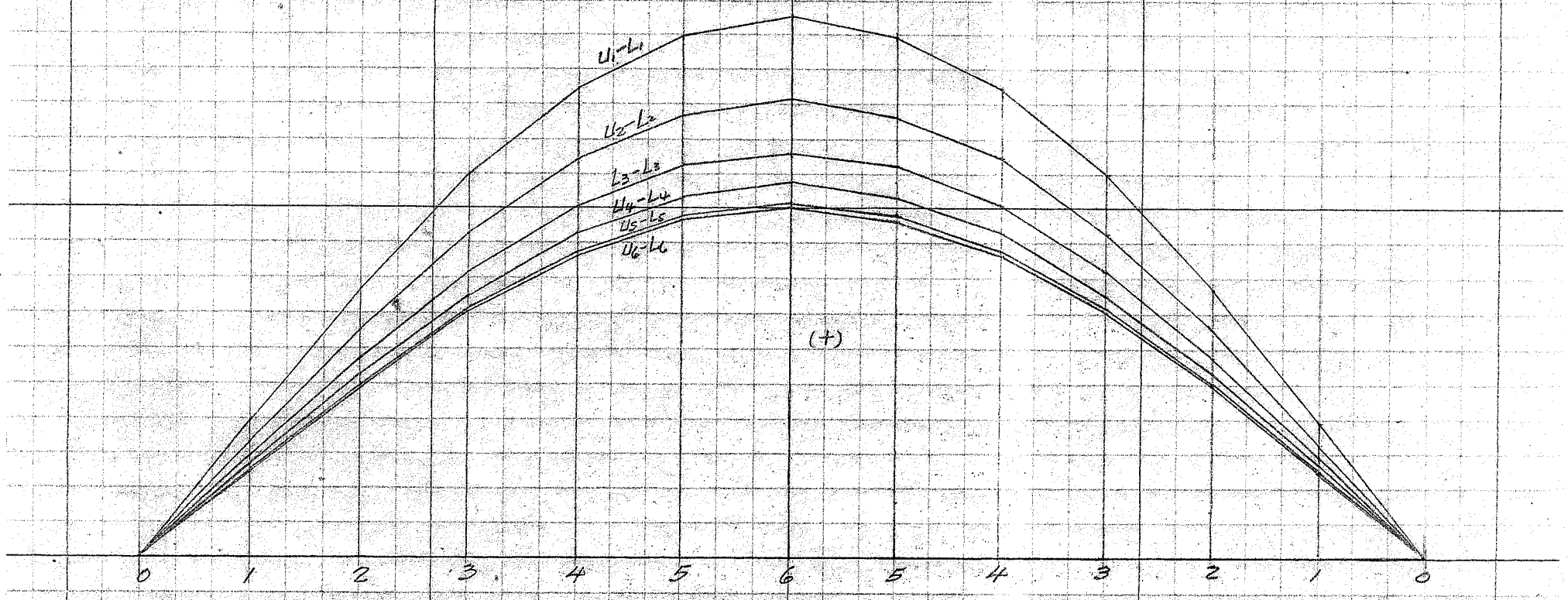
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CALCULATIONS FOR

Influence Diagram of Vertical Stresses.

Note. − Sign show Compressive stress.
+ " " Tensile " "

Scale of stress 1/20 m = 0.100 Kg.



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CALCULATIONS FOR

Stresses in Truss Members.

Top chord members.

	Σ pos. Ordinates	Σ Neg. Ordinates	Summary	Max. Ordinates	D.L. stress 13200 kg	L.L. stress 6000 kg	L.L. stress 2200 kg	Total L.L. stress	Total stress D.L.S.+L.L.S.	
$M_a - U_1$		-10.850		-1.432	-143,200	-65,100	-3200	-68300	-211,500	
$U_1 - U_2$		-10.142		-1.338	-133,900	-60,900	-2,900	-63,800	-197,700	
$U_2 - U_3$		-9.672		-1.276	-127,700	-58,000	-2,800	-60,800	-188,500	
$U_3 - U_4$		-9.356		-1.234	-123,500	-56,100	-2,700	-58,800	-182,300	
$U_4 - U_5$		-9.156		-1.208	-120,800	-54,900	-2,700	-57,600	-178,400	
$U_5 - U_6$		-9.066		-1.196	-119,700	-54,400	-2,600	-57,000	-176,700	

Middle chord members.

$M_a - M_b$	4.818		1.262	0.923	16,650	28,900	2,030	30,930	47,580	
		-3.556		-1.887	(11,100)	-21,350	-4,150	-25,500	-14,400	3/3 D.L.+L.L.
$M_b - M_c$	6.979		1.456	1.423	19,230	41,900	3,130	45,030	64,260	
		-5.523		-2.568	(17,870)	-33,150	-5,650	-38,800	-25,980	〃
$M_c - M_d$	7.032		1.172	1.565	15,460	42,200	3,440	45,640	61,100	
		-5.860		-2.498	(10,300)	-35,200	-5,490	-40,690	-30,390	〃
$M_d - M_e$	5.833		0.749	1.393	9,880	35,000	3,070	38,070	47,950	
		-5.084		-2.116	(6,590)	-30,500	-4,660	-35,160	-28,570	〃
$M_e - M_f$	3.737		0.420	1.046	5,540	22,420	2,300	24,720	30,260	
		-3.317		-1.750	(3,690)	-19,900	-3,850	-23,750	-20,060	〃
$M_f - M_f'$	2.904		0.299	0.500	3,950	17,430	1,100	18,530	22,480	
		-2.605		-1.605	(2,630)	-15,630	-3,530	-19,160	-16,530	

Bottom chord members.

$L_0 - L_1$	8.329		8.329	1.388	110,000	49,400	3,050	52,450	162,450	
$L_1 - L_2$	8.296		7.694	2.458	101,500	49,750	5,410	55,160	156,660	
		-0.602		-0.198		-3,610	-440	-4,050	—	
$L_2 - L_3$	9.579		7.743	2.772	102,300	57,500	6,100	63,600	165,900	
		-1.836		-0.517		-11,020	-1,140	-12,160	—	
$L_3 - L_4$	9.918		8.099	2.726	106,800	59,500	6000	65,500	172,300	
		-1.819		-0.563		-10,910	-1,240	-12,150	—	
$L_4 - L_5$	9.517		8.472	2.506	111,800	57,100	5510	62,610	174,410	
		-1.045		-0.362		-6270	-800	-7,070	—	
$L_5 - L_6$	8.830		8.691	2.279	114,700	53000	5,010	58,010	172,710	
		-0.139		-0.077		-830	-170	-1,000	—	

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CALCULATIONS FOR

Diagonal members.

	Σ Pos. Ordinates	Σ Neg. Ordinates	Summary	Max. Ordinates	D.L. Stress 13,200 kg	L.L. Stress 6000 kg	2,200 kg	Total L.L. Stress	Total Stress D.L.s+L.L.s.	
L_0-M_a		-9.979	-9.979	-1.663	-13,800	-59,900	-3,660	-63,560	-195,360	
M_a-L_1	1.899			1.029	(-8,770)	11,400	2,260	13,660	4890	3/3 D.L.+L.L
		-2.895	-0.996	-0.542	-13,150	-17,370	-1,190	-18,560	-31,710	
L_1-M_b	1.775		0.119	0.390	1,570	10,650	860	11,510	13,080	
		-1.656		-0.883	(1,050)	-9,940	-1,940	-11,880	-10,830	=
M_b-L_2	1.656			0.883	(-1,050)	9,940	1,940	11,880	10,830	=
		-1.775	-0.119	-0.390	-1,570	-10,650	-860	-11,510	-13,080	
L_2-M_c	1.333			0.643	(-1,620)	8,000	1,410	9,410	7,790	=
		-1.517	-0.184	-0.717	-2,430	-9,100	-1,580	-10,680	-13,110	
M_c-L_3	1.517		0.184	0.717	2,430	9,100	1,580	10,680	13,110	
		-1.333		-0.643	(1,620)	-8,000	-1,410	-9,410	-7,790	=
L_3-M_d	1.571			0.774	(-2,300)	9,430	1,700	11,130	8,830	=
		-1.832	-0.261	-0.659	-3,450	-11,000	-1,450	-12,450	-15,900	
M_d-L_4	1.832		0.261	0.659	3,450	11,000	1,450	12,450	15,900	
		-1.571		-0.774	(2,300)	-9,430	-1,700	-11,130	-8,830	=
L_4-M_e	2.075			0.811	(-1,820)	12,450	1,790	14,240	12,420	=
		-2.282	-0.207	-0.675	-2,730	-13,700	-1,490	-15,190	-17,920	
M_e-L_5	2.282		0.207	0.675	2,730	13,700	1,490	15,190	17,920	
		-2.075		-0.811	(1,820)	-12,450	-1,790	-14,240	-12,420	=
L_5-M_f	2.393			0.785	(-670)	14,350	1,730	16,080	15,410	=
		-2.469	-0.076	-0.728	-1,000	-14,800	-1,600	-16,400	-17,400	
M_f-L_6	2.469		0.076	0.728	1,000	14,800	1,600	16,400	17,400	
		-2.393		-0.785	(670)	-14,350	-1,730	-16,080	-15,410	=

Vertical members

U_1-L_1	1.405	0.185	18,550	8,430	410	8,840	27,390
U_2-L_2	1.188	0.156	15,680	7,130	340	7,470	23,150
U_3-L_3	1.050	0.138	13,850	6,300	300	6,600	20,450
U_4-L_4	0.966	0.128	12,750	5,800	280	6,080	18,830
U_5-L_5	0.917	0.121	12,100	5,500	270	5,770	17,870
U_6-L_6	0.910	0.120	12,000	5,460	260	5,720	17,720

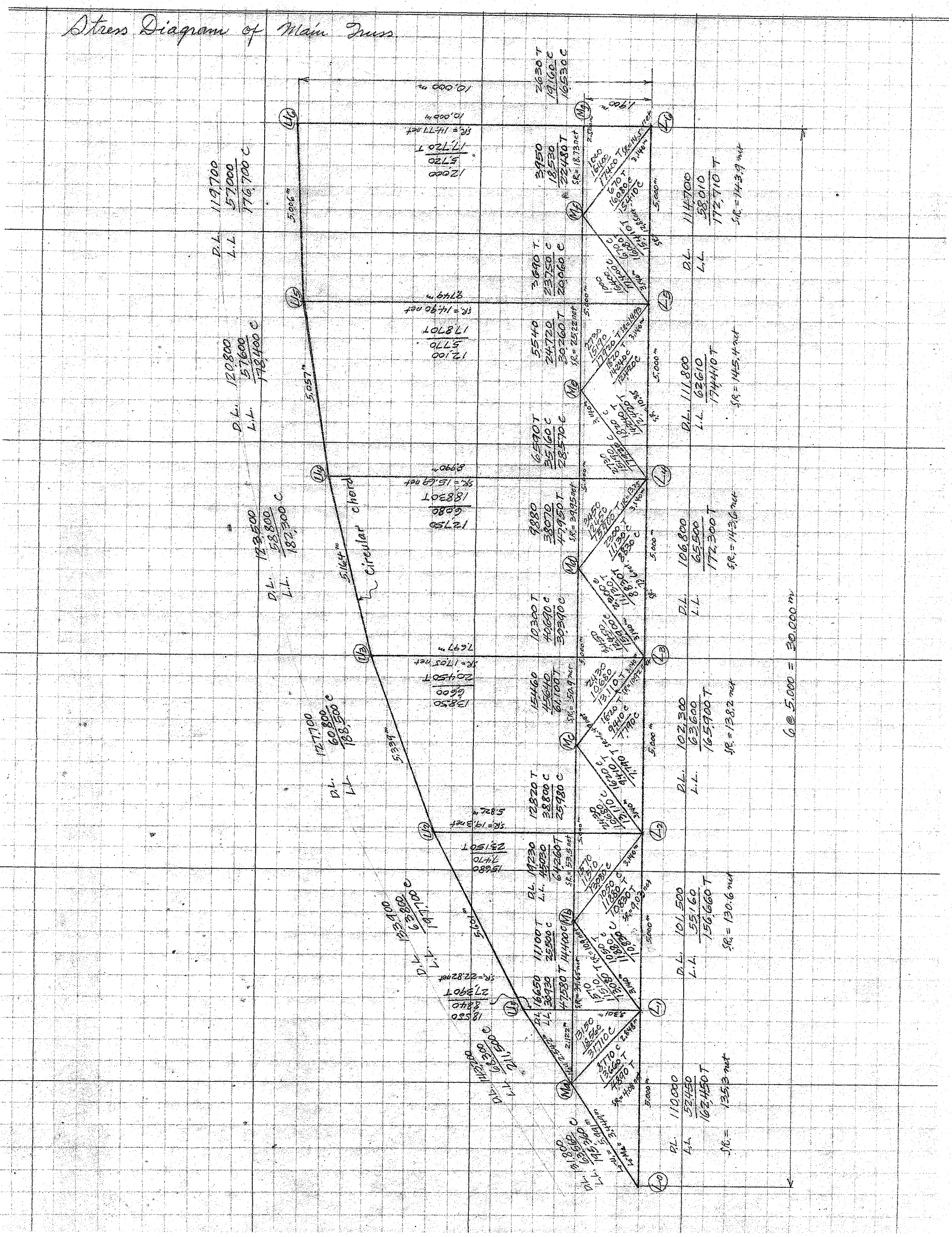
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CALCULATIONS FOR

Stress Diagram of Main Truss.


L0
L1
L2
L3
L4
L5
L6
L7
M1
M2
M3
M4
M5
M6
M7
M8
M9
M10
M11
5.057"
5.000"
5.334"
5.826"
5.000"
5.000"
5.000

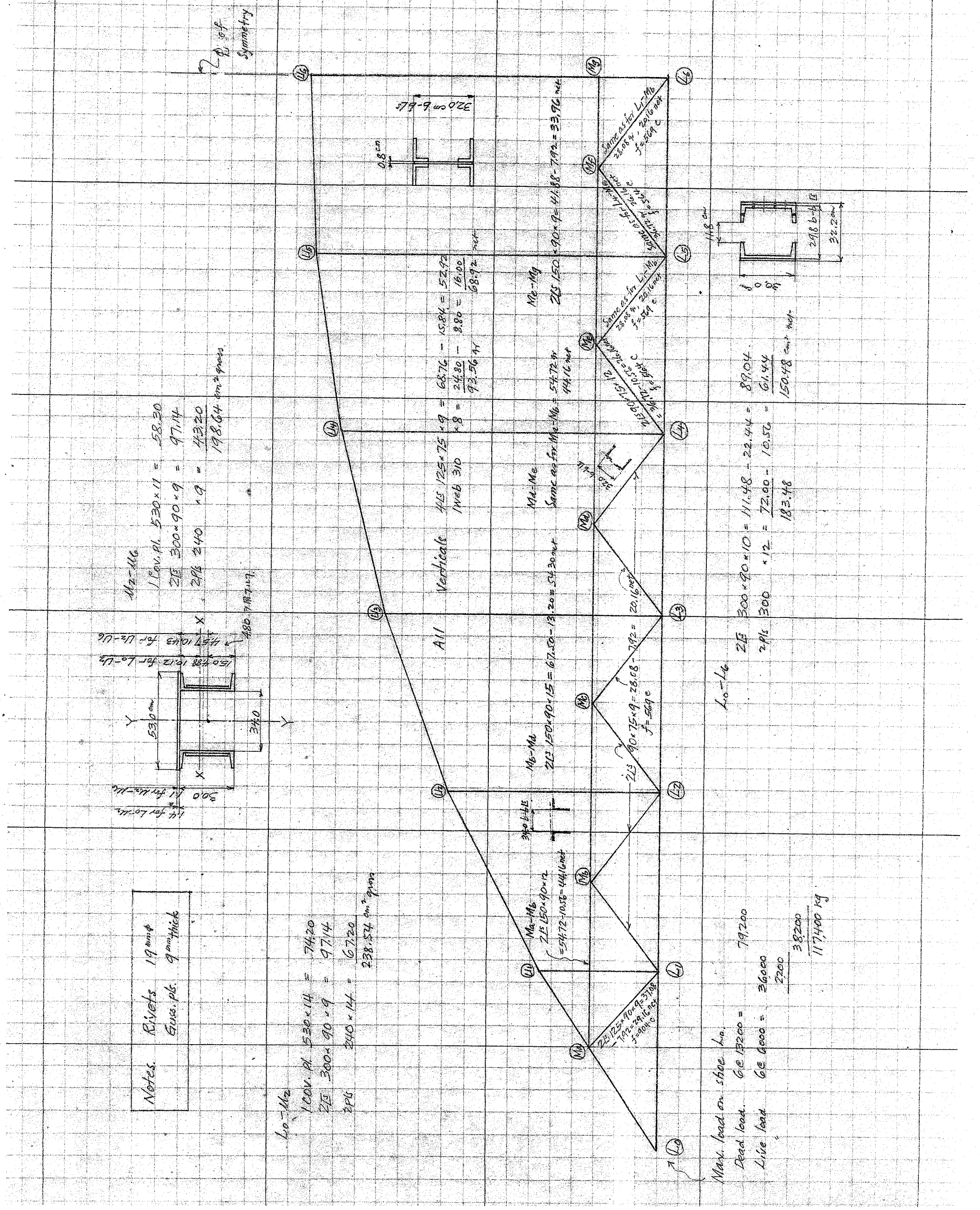
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CALCULATIONS FOR

Cross section Diagram of Truss Members.



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CALCULATIONS FOR

Approximate weight of Main truss.

Members	gross section	length	no. of members	product.
$L_0 - M_2$	238.54	11,592	2	5,530
$M_2 - M_6$	198.64	20,566	2	8,170
$L_0 - L_6$	183.48	30,000	2	11,000
$M_a - M_b$	54.72	4,622	2	506
$M_b - M_d$	67.50	10,000	2	1350
$M_d - M_e$	54.72	5,000	2	547
$M_e - M_g$	41.88	7,500	2	628
$M_a - L_1$	37.08	2,848	2	212
$L_1 - M_b$ to $M_d - L_4$	28.08	3,140	12	1,058
$L_4 - M_e$ & $L_5 - M_f$	36.72	3,140	4	461
$M_e - L_5$ & $M_f - L_6$	28.08	3,140	4	353
$U_1 - L_1$	93.56	3,301	2	618
$U_2 - L_2$	"	5,826	2	1,090
$U_3 - L_3$	"	7,697	2	1,440
$U_4 - L_4$	"	8,990	2	1,683
$U_5 - L_5$	"	9,749	2	1,825
$U_6 - L_6$	"	10,000	1	936

37,407 @ 0.7850 = 29350

Details say 38 % = 11,150

40,500 kg

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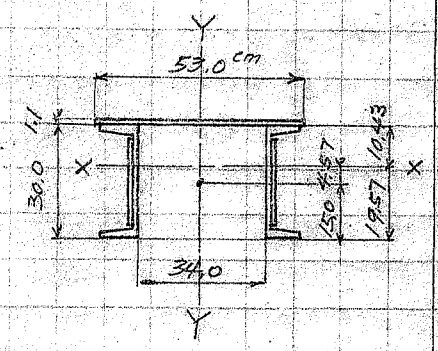
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CALCULATIONS FOR

Determination of Sections for each member.

Top chord. U_2-U_4. max. stress = 188500 kg c for which unsupported length = 5.339 meters.



assumed section

			eccentricity
1 cov. pl.	530 × 11	= 58.30	$\frac{58.30 \times 15.33}{198.64} = 4.57$ cm
2 I s	300 × 90 × 9	= 97.14	
2 Pls	240 × 9	= 43.20	
		198.64 cm² gross.	

Moments of Inertia.

I_x 1 Pl.	58.30 × 10.98²		= 7040
2 I s	97.14 × 4.57² + 2 @ 6435		= 14900
2 Pls	43.20 × 4.57² + 1.8 × 24.0³ ÷ 12		= 2970
		I_x =	24910 cm⁴
I_y 1 Pl.	1.10 × 53.00³ ÷ 12		= 13650
2 I s	97.14 × 19.23² + 2 @ 325		= 36600
2 Pls	43.20 × 18.35²		= 14550
		I_y =	64800 cm⁴

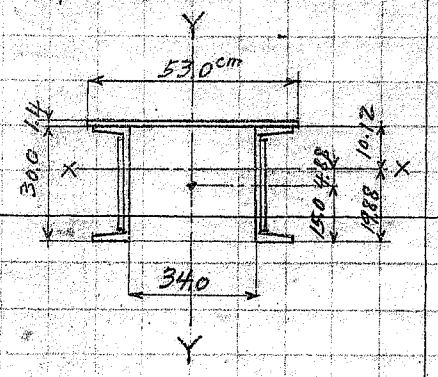
Radii of gyration. slenderness ratio. allowable unit stress.

$r_x = \sqrt{\frac{24910}{198.64}} = 11.20$ cm, $\frac{l}{r_x} = \frac{533.9}{11.2} = 47.7$ $f = 1000$ kg/cm² c

$r_y = \sqrt{\frac{64800}{198.64}} = 18.07$ ", $\frac{l}{r_y} = \frac{533.9}{18.07} = 29.6$ "

Assumed section is good for 198.64 @ 1000 = 198640 kg c.

Top chord $L_0-U_1-U_2$ max. stress = 211500 kg c



assumed section.

			eccentricity
1 cov. pl.	530 × 14	= 74.20	$\frac{74.20 \times 15.70}{238.54} = 4.88$ cm
2 I s	300 × 90 × 9	= 97.14	
2 Pls	240 × 14	= 67.20	
		238.54 cm² gross	

Moments of Inertia.

I_x 1 Pl.	74.20 × 10.82²		= 8680
2 I s	97.14 × 4.88² + 2 @ 6435		= 15180
2 Pls	67.20 × 4.88² + 2.8 × 240³ ÷ 12		= 4830
		I_x =	28690 cm⁴
I_y 1 Pl.	1.40 × 53.00³ ÷ 12		= 17370
2 I s	97.14 × 19.23² + 2 @ 325		= 36600
2 Pls	67.20 × 18.60²		= 23350
		I_y =	77220 cm⁴

unsupported length $l = 560.1$ cm

unsupported length $l = 1159.2 - 200 = 959.2$ cm (Portal depth)

Radii of gyration. Slenderness ratio. allowable unit stress.

$r_x = \sqrt{\frac{28690}{238.54}} = 10.97$ cm, $\frac{l}{r_x} = \frac{560.1}{10.97} = 51.1$ $f = 1000$ kg/cm² c

$r_y = \sqrt{\frac{77220}{238.54}} = 18.00$ cm, $\frac{l}{r_y} = \frac{959.2}{18.00} = 53.3$ "

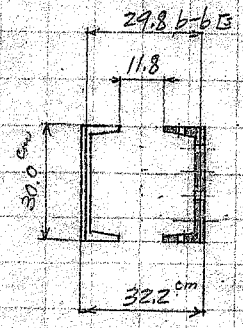
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CALCULATIONS FOR

Bottom chord L0-L6, max. stress = 174410 kg T, Section required = 145.44 cm² net



Assumed section.

2 I 300×90×10	= 111.48	− 22.44	= 89.04
2 Pl 300 ×12	= 72.00	− 10.56	= 61.44
	183.48 cm² gr		150.48 cm² net

This section is good for 150.48 @1200 = 180,500 kg T

Middle chord



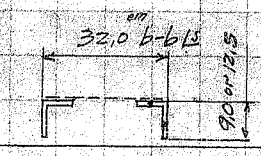
Assumed sections.

	gross area		net area	l/r	
2 L 150×90×9	= 41.88	− 7.92	= 33.96	250÷4.75 = 52.6	f = 1000 kg/cm² C
2 L 150×90×12	= 54.72	− 10.56	= 44.16	250÷4.75 = 52.6	f = 1,200 " T
2 L 150×90×15	= 67.50	− 13.20	= 54.30	250÷4.73 = 52.8	

Safe stress for each member.

2 L 150×90×9	41.88 @1000 = 41,880 kg C	Me-Mf and Mf-Mg
	33.96 @1200 = 40800 " T	
2 L 150×90×12	54.72 @1000 = 54,720 " C	Ma-Mb and Md-Me
	44.16 @1200 = 53,000 " T	
2 L 150×90×15	67.50 @1000 = 67,500 " C	Mb-Mc and Mc-Md
	54.30 @1200 = 65,200 " T	

Diagonals



Assumed section

	gross area		net area	l/r		allowable unit compression
2 L 90×75×9	= 28.08	− 7.92	= 20.16	314.0 ÷ 2.78 = 113.0	f = 568 kg/cm² C	
2 L 90×75×12	= 36.72	− 10.56	= 26.16	314.0 ÷ 2.77 = 113.3	f = 565 " C	
2 L 125×90×9	= 37.08	− 7.92	= 29.16	284.8 ÷ 3.94 = 72.2	f = 905 " C	

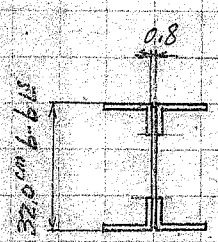
Safe stress for each member.

2 L 90×75×9	28.08 @568 = 15,950 kg C	L1-Mb, Mb-L2, L2-Mc, Mc-L3, L3-Md, Md-L4, Me-L5, & Mf-L6
	20.16 @1200 = 24,200 " T	
2 L 90×75×12	36.72 @565 = 20,750 " C	L4-Me and L5-Mf
	26.16 @1200 = 31,400 " T	
2 L 125×90×9	37.08 @905 = 33,550 " C	Ma-L1
	29.16 @1200 = 35,000 " T	

Verticals

max. stress 27390 T, Section required = 22.82 cm² net, max. length = 8.10 m

assumed section



4 L 125×75×9	= 68.76	− 15.84	= 52.92	
1 web 310 ×8	= 24.80	− 8.80	= 16.00	note. web plate is used to give a good appearance.
	93.56 cm² gr		68.92 cm² net	

Least radius of gyration

$I_y = 68.76 \times 4.59^2 + 4 @ 269 = 2524 \text{ cm}^4$

$r_y = \sqrt{\frac{2524}{93.56}} = 5.19 \text{ cm}$

$l_y/r_y = 810 \div 5.19 = 156 < 200$ ✓

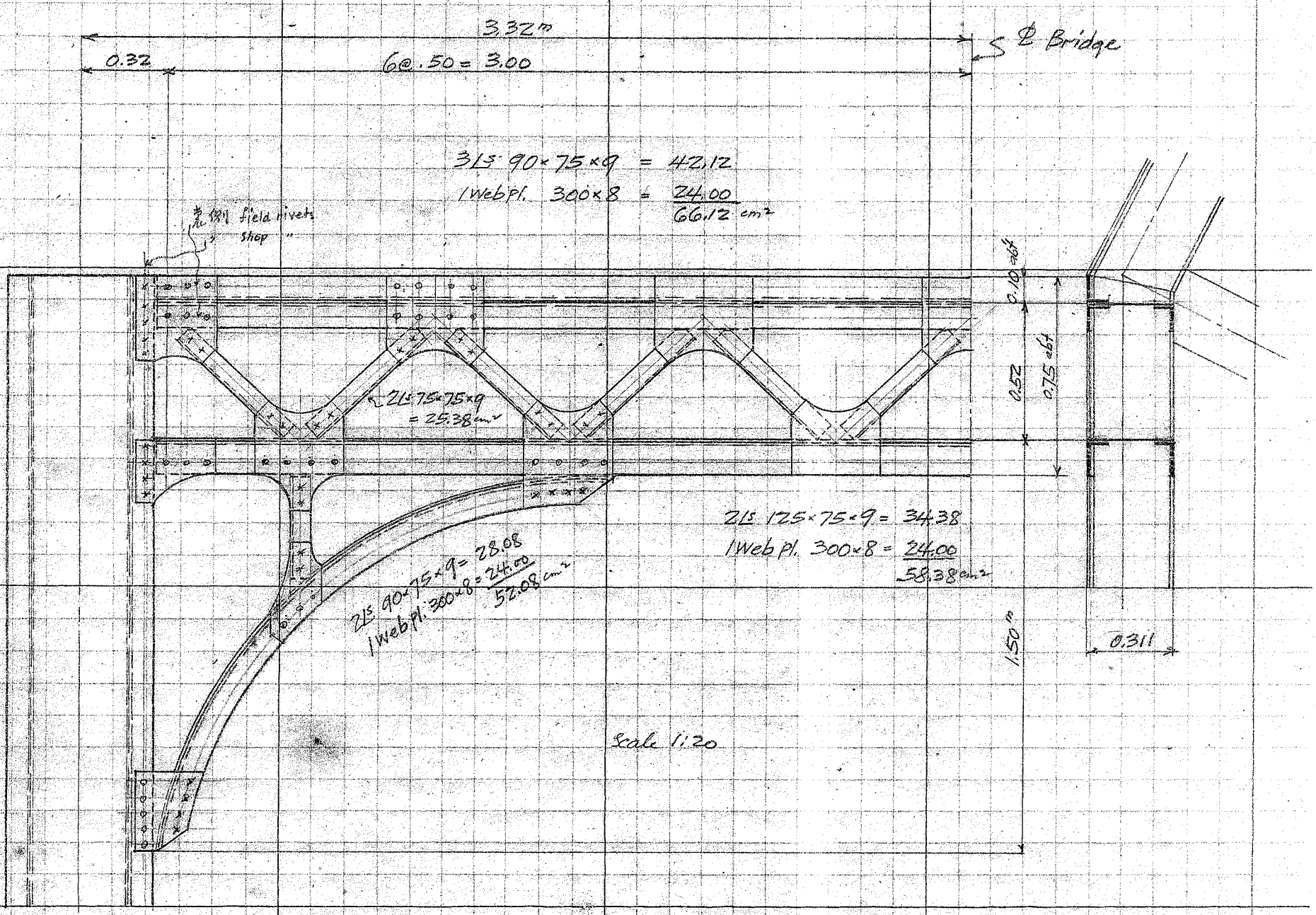
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CALCULATIONS FOR

Sketch of Portal Bracing



Approximate weight of Portal Bracing

Top chord		66.12 ×	6.10	=	404
bottom "		58.38 ×	6.10	=	356
brackets	2 @	52.08 ×	2.45	=	255
diagonals	12 @	25.38 ×	0.55	=	168
verticals	2 @	25.38 ×	0.40	=	20
gus pls	10 @	350×0.9 ×	0.38	=	120
"	8 @	300×0.9 ×	0.35	=	76
"	8 @	350×0.9 ×	0.42	=	106
"	4 @	200×0.9 ×	0.40	=	29
"	4 @	250×0.9 ×	0.30	=	27
"	2 @	250×0.9 ×	0.31	=	14
"	2 @	310×0.9 ×	0.73	=	41
					1616 @.7850 = 1270
rivet heads say 4%					= 50
					1320 kg

details 40% of main section.

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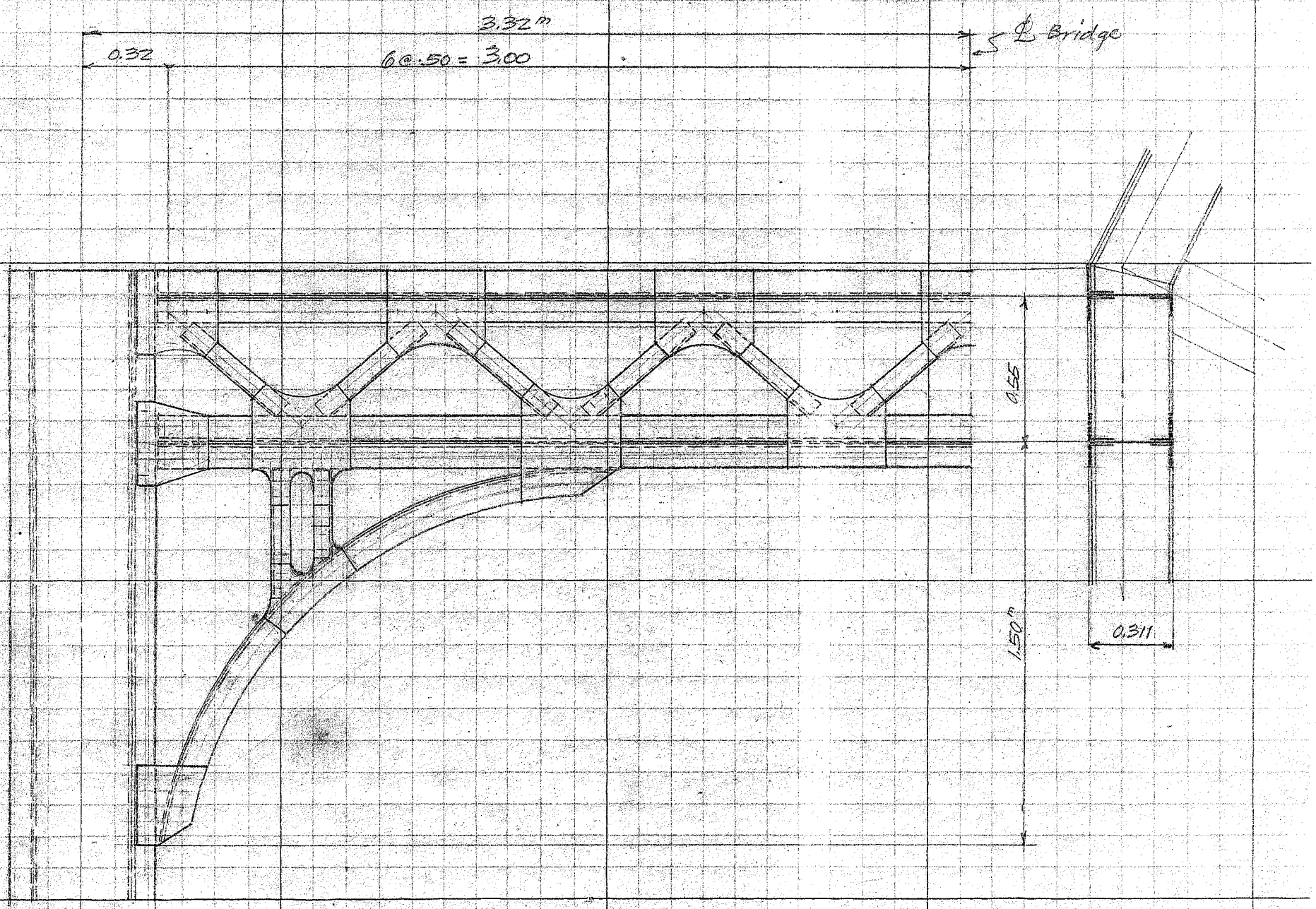
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Sketch of Portal Bracing.



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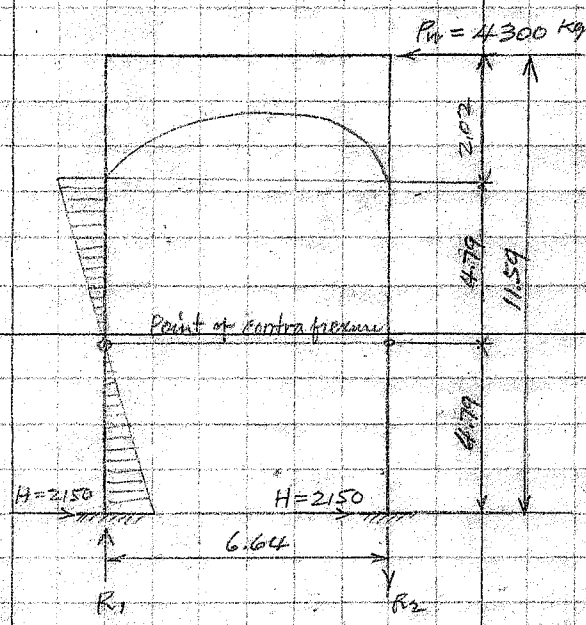
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CALCULATIONS FOR

Section of End Post Lo-U1-U2



Horizontal thrust on portal.

wind thrust 4@1075 = 4,300 kg

seismic " 4@670 = 2,680 "

$R_1 = \frac{4300 \times 1.59}{6.64} = 7,500$ kg

$R_2 = -7500$

max. moment on end post.

$= 2,150 \times 4.79 = 10,300$ kgm

max. direct compression on end post. (part U1-U2)

D.L.	133900
L.L.	63800
W.L.	7500
	205200 kg C

Direct stress $= \frac{205200}{238.54} = 860$

moment stress $= \frac{10300 \times 100}{77220} \times \frac{53.0}{2} = \frac{353}{1213}$ kg/cm² C

Slenderness ratio $= \frac{958}{18.00} = 53.2$

or $\frac{560.1}{10.97} = 51.1$

allowable unit compression

$f = 1,500(1 - 0.0033 \times 53.2) = 1062$ use 1,000 kg/cm²

add 25% allowance for wind $\frac{250}{1250} \cdot 71213$ ✓

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CALCULATIONS FOR

Deflection of the truss at center of span.

General equation to the deflection:

$$\Delta = \Sigma \frac{SL}{EA} \cdot T \quad \text{or} \quad \frac{1}{E} \cdot \Sigma \frac{SL}{A} T$$

where Δ = The deflection of the truss at any panel point in centimeters;
S = the stress in each member in kgs;
T = the stress in each member in kgs., due to a unit load on the panel point at which the deflection is desired, in the direction of the deflection;
L = the length of each member in cms;
A = the gross sectional area of each member in sq. cms;
E = the modulus of elasticity of steel in kg/cm^2 = 2,100,000.

Members		L	A	L/A	S_D	T	$\frac{S_D L}{A} T$
Top chord	$L_0 - M_a$	3,449 m/100	238.54 cm²	1.445	−131,800	−0.907	173,000
	$M_a - M_1$	2,542	238.54	1.065	−143,200	−1.432	218,500
	$M_1 - M_2$	5,601	238.54	2.350	−133,900	−1.338	421,000
	$M_2 - M_3$	5,339	198.64	2.690	−127,700	−1.276	438,000
	$M_3 - M_4$	5,164	198.64	2.603	−123,500	−1.234	396,500
	$M_4 - M_5$	5057	198.64	2.545	−120,800	−1.208	371,500
	$M_5 - M_6$	5,006	198.64	2.523	−119,700	−1.196	361000
Middle chord	$M_a - M_b$	4,622	54.72	8.450	16,650	0.760	106,800
	$M_b - M_c$	5,000	67.50	7.400	19,230	1.032	147,000
	$M_c - M_d$	5,000	67.50	7.400	15,460	0.894	102,300
	$M_d - M_e$	5,000	54.72	9.130	9,880	0.391	35,300
	$M_e - M_f$	5,000	41.88	11.940	5,540	−0.447	−29,600
	$M_f - M_g$	2,500	41.88	5.970	3,950	−1.605	−37,900
bottom chord	$L_0 - L_1$	5,000	183.48	2.728	110,000	0.757	227,000
	$L_1 - L_2$	5000	183.48	2.728	101,500	0.297	82,200
	$L_2 - L_3$	5,000	183.48	2.728	102,300	0.231	64,400
	$L_3 - L_4$	5,000	183.48	2.728	106,800	0.552	160,800
	$L_4 - L_5$	5,000	183.48	2.728	111,800	1.223	373,000
	$L_5 - L_6$	5,000	183.48	2.728	114,700	2.221	695,000
diagonal	$M_a - L_1$	2,848	37.08	7.680	−13,150	−0.459	45,900
	$L_1 - M_b$	3,140	28.08	11.180	1,570	0.171	3,000
	$M_b - L_2$	3,140	28.08	11.180	−1,570	−0.171	3000
	$L_2 - M_c$	3,140	28.08	11.180	−2,430	−0.088	2400
	$M_c - L_3$	3,140	28.08	11.180	2,430	0.088	2400
	$L_3 - M_a$	3,140	28.08	11.180	−3,450	−0.316	12,200
	$M_a - L_4$	3,140	28.08	11.180	3,450	0.316	12,200
	$L_4 - M_e$	3,140	36.72	8.550	−2,730	−0.527	12,300
	$M_e - L_5$	3,140	28.08	11.180	2,730	0.5277	16,100
	$L_5 - M_f$	3,140	36.72	8.550	−1,000	−0.728	6,200
	$M_f - L_6$	3,140	28.08	11.180	1,000	0.728	8,100
Vertical	$M_1 - L_1$	3,301	93.56	3.530	18,550	0.185	12,100
	$M_2 - L_2$	5,826	93.56	6.230	15,680	0.156	15,200
	$M_3 - L_3$	7,697	93.56	8.230	13,850	0.138	15,700
	$M_4 - L_4$	8,990	93.56	9.610	12,750	0.128	15,700
	$M_5 - L_5$	9,749	93.56	10.420	12,100	0.121	15,300
	$M_6 - L_6$	10.00'0 × 1/2	93.56	5.340	12,000	0.120	7700
							4511,300

Summary for the entire span 4511,300 × 2 = 9022,600

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CALCULATIONS FOR

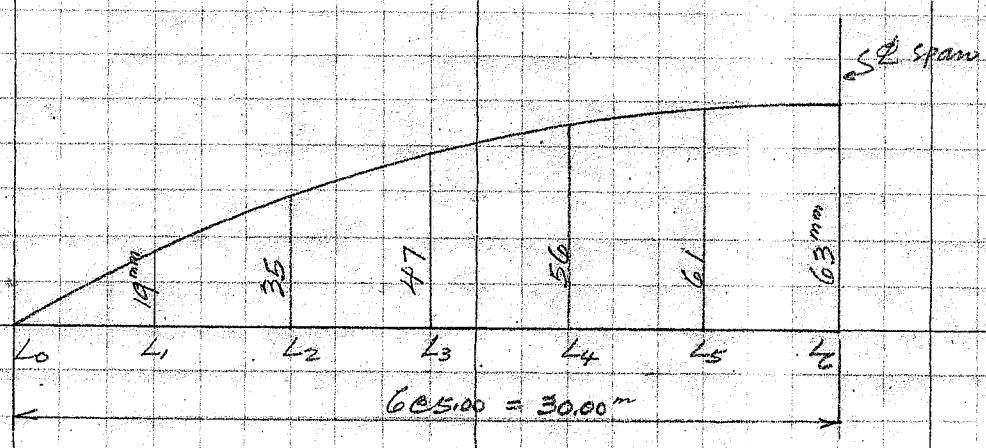
Dead load Deflection $= \frac{9022600}{2,100,000} = 4.30$ cm

Uniform live load $= 4.30 \times \frac{6000}{13200} = 1.96$

6.26 cm

Deflection ratio $= \frac{6.26}{6000.0} = \frac{1}{960}$

63 mm camber shall be given to the truss.



Camber diagram.

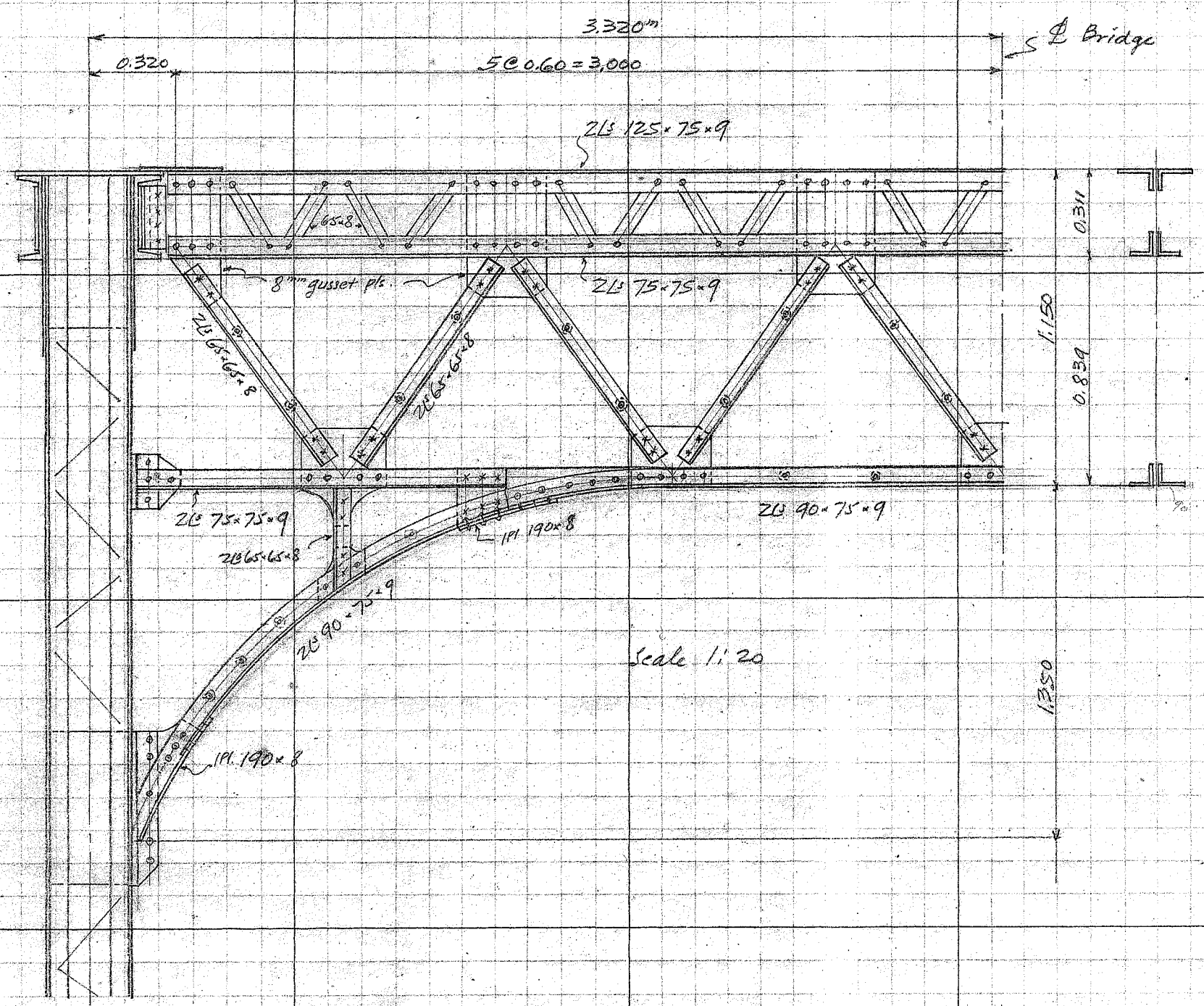
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CALCULATIONS FOR

Sketch of Sway Bracing.



Scale 1:20

Approximate weight of Sway Bracing.

Top strut	2L 125×75×9	@	13.50	× 6.07	=	164
"	2L 75×75×9	@	9.96	× 6.07	=	121
bottom chord	2L 90×75×9	@	11.00	× 7.40	=	163
"	4L 75×75×9	@	9.96	× 1.35	=	54
diagonals	20L 65×65×8	@	7.66	× 0.90	=	138
verticals	4L 65×65×8	@	7.66	× 0.37	=	11
						651
gusset pls.	4Pls 300×8	@	18.84	× 0.45	=	34
"	3Pls " "	@	"	× 0.24	=	14
"	2Pls 250×8	@	15.70	× 0.50	=	16
"	2Pls 300×8	@	18.84	× 0.36	=	14
"	2Pls 70×8	@	4.40	× 0.20	=	2
"	2Pls 200×8	@	12.56	× 0.63	=	16
"	2Pls 150×8	@	9.42	× 0.20	=	4
"	2Pls 200×8	@	12.56	× 0.60	=	15
Spl. pl. (leg)	2Pls 190×8	@	11.93	× 0.40	=	10
"	2Pls "	@	"	× 0.53	=	13
Conn. L	4Ls 90×90×10	@	13.30	× 0.24	=	13
"	4Ls "	@	"	× 0.20	=	11
"	4Ls "	@	"	× 0.55	=	29
lacing bars	20Pls 65×8	@	4.08	× 0.34	=	28
"	rivet head say					219
						30
						900 kg

Strut say 380 kg

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CALCULATIONS FOR

Approximate weight of Structural steel in one span.

Stringers	4@50 =	200 × 60.7	=	12,140
floor beams, intermediate		11 @ 980	=	10780
" , end		2 @ 940	=	1,880
upper lateral bracings				3,830
lower " "				5,700
portal bracings		2 @ 1,320	=	2,640
Sway "		3 @ 900	=	2,700
Struts		4 @ 380	=	1520
main trusses		2 @ 40,500	=	81,000
expansion metals say				800
shoes say		4 @ 850	=	3,400
				126,390

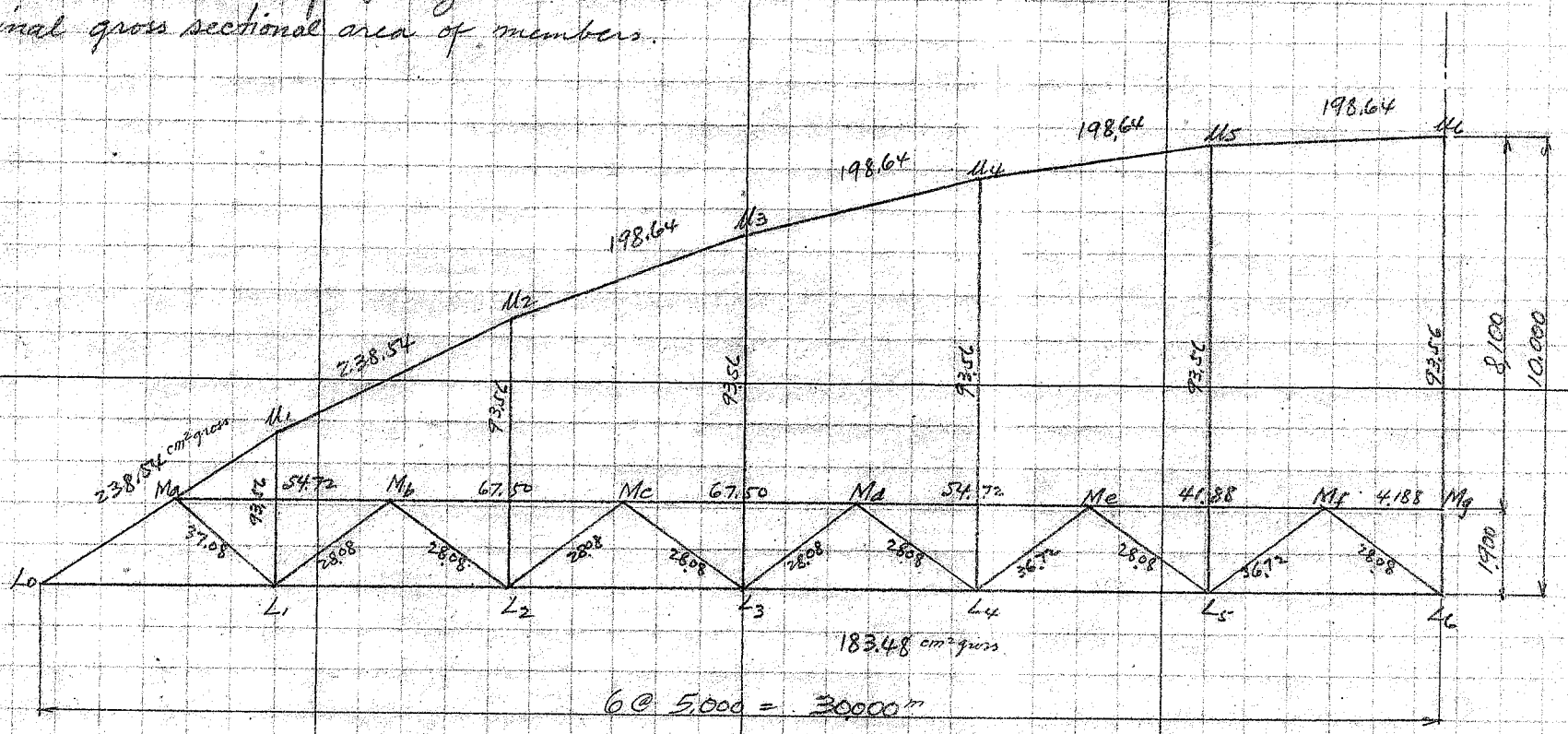
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CALCULATIONS FOR

Final Calculation of Horizontal Thrust.
Final gross sectional area of members.



$S_0 S_a^4/A$ and $S_a^2 4/A$

	Members	Gross area	$S_0 S_a L$	$S_a^2 L$	$S_0 S_a^4/A$	$S_a^2 4/A$
Top chords	Ma – M1	238.54 cm²	0.0000	3.6483	0.000	1.529
	M1 – M2	238.54	"	7.0259	"	2.945
	M2 – M3	198.64	"	6.0897	"	3.066
	M3 – M4	198.64	"	5.5105	"	2.774
	M4 – M5	198.64	"	5.1688	"	2.602
	M5 – M6	198.64	"	5.0160	"	2.525
	M6 – M5'	198.64	"	5.0160	"	2.525
	M5' – M4'	198.64	"	5.1688	"	2.602
	M4' – M3'	198.64	"	5.5105	"	2.774
	M3' – M2'	198.64	"	6.0897	"	3.066
	M2' – M1'	238.54	"	7.0259	"	2.945
	M1' – Ma'	238.54	"	3.6483	"	1.529
				Summary		30.882
Middle chords.	Ma – Mb	54.72	1.7610	13.9520	3.218	25.497
	Mb – Mc	67.50	6.7245	47.0110	9.962	69.646
	Mc – Md	67.50	13.3260	82.0570	19.742	121.566
	Md – Me	54.72	20.7530	111.9400	37.926	204.569
	Me – Mf	41.88	28.1315	131.6410	67.172	314.329
	Mf – Mf'	41.88	34.6265	138.5065	82.680	330.722
	Mf' – Me'	41.88	39.3840	131.6410	94.040	314.329
	Me' – Md'	54.72	41.5055	111.9400	75.851	204.569
	Md' – Mc'	67.50	39.9785	82.0570	59.227	121.566
	Mc' – Mb'	67.50	33.6220	47.0110	49.810	69.646
	Mb' – Ma'	54.72	19.3713	13.9520	35.401	25.497
				Summary		1801.936

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CALCULATIONS FOR

	Member	Gross area	$\Sigma_0 S_a L$	$S_a^2 L$	$\Sigma_0 S_a L/A$	$S_a^2 L/A$
Bottom chords	$L_0 - L_1$	183.48 cm²	0.0000	0.0000	0.000	0.000
	$L_1 - L_2$	"	2.3060	9.8295	1.257	5.357
	$L_2 - L_3$	"	7.0140	32.7400	3.823	17.844
	$L_3 - L_4$	"	13.0155	57.5150	7.094	31.347
	$L_4 - L_5$	"	19.3985	77.2875	10.573	42.123
	$L_5 - L_6$	"	25.3125	88.0910	13.796	48.011
	$L_6 - L_5'$	"	29.9150	88.0910	16.304	48.011
	$L_5' - L_4'$	"	32.3315	77.2875	17.621	42.123
	$L_4' - L_3'$	"	31.6095	57.5150	17.228	31.347
	$L_3' - L_2'$	"	26.6550	32.7400	14.527	17.844
	$L_2' - L_1'$	"	16.1425	9.8295	8.798	5.357
	$L_1' - L_0'$	"	0.0000	0.0000	0.000	0.000
				Summary		289.364
Diagonals	$L_0 - M_a$	238.54	0.0000	0.0000	0.000	0.000
	$M_a - L_1$	37.08	0.3401	2.7890	0.917	7.522
	$L_1 - M_b$	28.08	0.3611	2.1883	1.286	7.793
	$M_b - L_2$	"	0.3611	2.1883	1.286	7.793
	$L_2 - M_c$	"	0.2675	1.2017	0.953	4.280
	$M_c - L_3$	"	0.2675	1.2017	0.953	4.280
	$L_3 - M_d$	"	0.1849	0.5740	0.658	2.044
	$M_d - L_4$	28.08	0.1849	0.5740	0.658	2.044
	$L_4 - M_e$	36.72	0.1086	0.1978	0.296	0.539
	$M_e - L_5$	28.08	0.1086	0.1978	0.387	0.704
	$L_5 - M_f$	36.72	0.0358	0.0217	0.097	0.059
	$M_f - L_6$	28.08	0.0358	0.0217	0.127	0.077
	$L_6 - M_f'$	28.08	−0.0358	0.0217	−0.127	0.077
	$M_f' - L_5'$	36.72	−0.0358	0.0217	−0.097	0.059
	$L_5' - M_e'$	28.08	−0.1086	0.1978	−0.387	0.704
	$M_e' - L_4'$	36.72	−0.1086	0.1978	−0.296	0.539
	$L_4' - M_d'$	28.08	−0.1849	0.5740	−0.658	2.044
	$M_d' - L_3'$	"	−0.1849	0.5740	−0.658	2.044
	$L_3' - M_c'$	"	−0.2675	1.2017	−0.953	4.280
	$M_c' - L_2'$	"	−0.2675	1.2017	−0.953	4.280
	$L_2' - M_b'$	"	−0.3611	2.1883	−1.286	7.793
	$M_b' - L_1'$	28.08	−0.3611	2.1883	−1.286	7.793
	$L_1' - M_a'$	37.08	3.7420	2.7890	10.092	7.522
	$M_a' - L_0'$	238.54	0.0000	0.0000	0.000	0.000
				Summary		74.270
Verticals	$M_1 - L_1$	93.56	0.0000	0.0769	0.000	0.082
	$M_2 - L_2$	"	"	0.0996	"	0.106
	$M_3 - L_3$	"	"	0.1031	"	0.110
	$M_4 - L_4$	"	"	0.1025	"	0.110
	$M_5 - L_5$	"	"	0.1004	"	0.107
	$M_6 - L_6$	"	"	0.1010	"	0.108
	$M_5' - L_5'$	"	"	0.1004	"	0.107
	$M_4' - L_4'$	"	"	0.1025	"	0.110
	$M_3' - L_3'$	"	"	0.1031	"	0.110
	$M_2' - L_2'$	"	"	0.0996	"	0.106
	$M_1' - L_1'$	"	"	0.0769	"	0.082
				Summary		1.138

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CALCULATIONS FOR

Summary of $\Sigma Sa^2 4/A$ for the entire span.

	$\Sigma Sa^2 4/A$
Top chords	30.882
middle chords	1,801.936
bottom "	289.364
diagonals	74.270
verticals	1.138
$\Sigma Sa^2 4/A$ =	2,197.590

Summary of $So\,Sa\,4/A$

Middle chords

	$So\,Sa\,4/A$	Summary
1	3.218	3.218
2	9.962	13.180
3	19.742	32.922
4	37.926	70.848
5	67.172	138.020
6	82.680	220.700
7	94.040	314.740
8	75.851	390.591
9	59.227	449.818
10	49.810	499.628
11	35.401	535.029

Loaded point					$\Sigma So\,Sa\,4/A$
					535.029
2	2@ 449.818 + 10@ 13.180 =	899.636	+ 131.800	= 1,031.436	
3	3@ 390.591 + 9@ 32.922 =	1,171.773	+ 296.298	= 1468.071	
4	4@ 314.740 + 8@ 70.848 =	1258.960	+ 566.784	= 1825.744	
5	5@ 220.700 + 7@ 138.020 =	1,103.500	+ 966.140	= 2069.640	
6	6@ 138.020 + 6@ 220.700 =	828.120	+ 1324.200	= 2152.320	

Bottom chords.

1	0.000	0.000
2	1.257	1.257
3	3.823	5.080
4	7.094	12.174
5	10.573	22.747
6	13.796	36.543
7	16.304	52.847
8	17.621	70.468
9	17.228	87.696
10	14.527	102.223
11	8.798	111.021
12	0.000	111.021

Loaded point					$\Sigma So\,Sa\,4/A$
					111.021
2	2@ 102.223 + 10@ 1.257 =	204.446	+ 12.570	= 217.016	
3	3@ 87.696 + 9@ 5.080 =	263.088	+ 45.720	= 308.808	
4	4@ 70.468 + 8@ 12.174 =	281.872	+ 97.392	= 379.264	
5	5@ 52.847 + 7@ 22.747 =	264.235	+ 159.229	= 423.464	
6	6@ 36.543 + 6@ 36.543 =	219.258	+ 219.258	= 438.516	

Diagonals

1	0.000	0.000
2	0.917	0.917
3	1.286	2.203
4	1.286	3.489
5	0.953	4.442
6	0.953	5.395
7	0.658	6.053
8	0.658	6.711
9	0.296	7.007
10	0.387	7.394
11	0.097	7.491
12	0.127	7.618
13	−0.127	7.491
14	−0.097	7.394

Loaded point					$\Sigma So\,Sa\,4/A$
					11.009
2	2@ 3.489 + 10@ 3.489 =	6.978	+ 34.890	= 41.868	
3	3@ 5.395 + 9@ 5.395 =	16.185	+ 48.555	= 64.740	
4	4@ 6.711 + 8@ 6.711 =	26.844	+ 53.688	= 80.532	
5	5@ 7.394 + 7@ 7.394 =	36.970	+ 51.758	= 88.728	
6	6@ 7.618 + 6@ 7.618 =	45.708	+ 45.708	= 91.416	

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9-1-6 (4)

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CALCULATIONS FOR

15	− 0.387	7.007
16	− 0.296	6.711
17	− 0.658	6.053
18	− 0.658	5.395
19	− 0.953	4.442
20	− 0.953	3.489
21	− 1.286	2.203
22	− 1.286	0.917
23	10.092	11.009
24	0.000	11.009

$\Sigma S_o S_a {}^4/_A$ for Top chords and Verticals are zero for all points.

Influence values of Horizontal Thrust H.

Loaded Points	Top chords	Middle chords	Bottom chords	Diagonals	Verticals	Summary of $S_o S_a {}^4/_A$	Summary of $S_a^2 {}^4/_A$	Hor. Thrust H.
1	0.000 +	535.029 +	111.021 +	11.009 +	0.000 =	657.059 ÷	2,197.590 =	0.299
2	"	1,031.436	217.016	41.868	"	1290.320	"	0.587
3	"	1468.071	308.808	64.740	"	1,841.619	"	0.838
4	"	1,825.744	379.264	80.532	"	2285.540	"	1.040
5	"	2069.640	423.464	88.728	"	2,581.832	"	1.175
6	"	2,152.320	438.516	91.416	"	2682.252	"	1.221

Summary for the entire span $\Sigma H =$ 9.099

Error $= \frac{9.057}{9.099} - 1.0 = -0.00446$ or -0.46%

The first calculation is accurate enough in practice.

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9-11-6

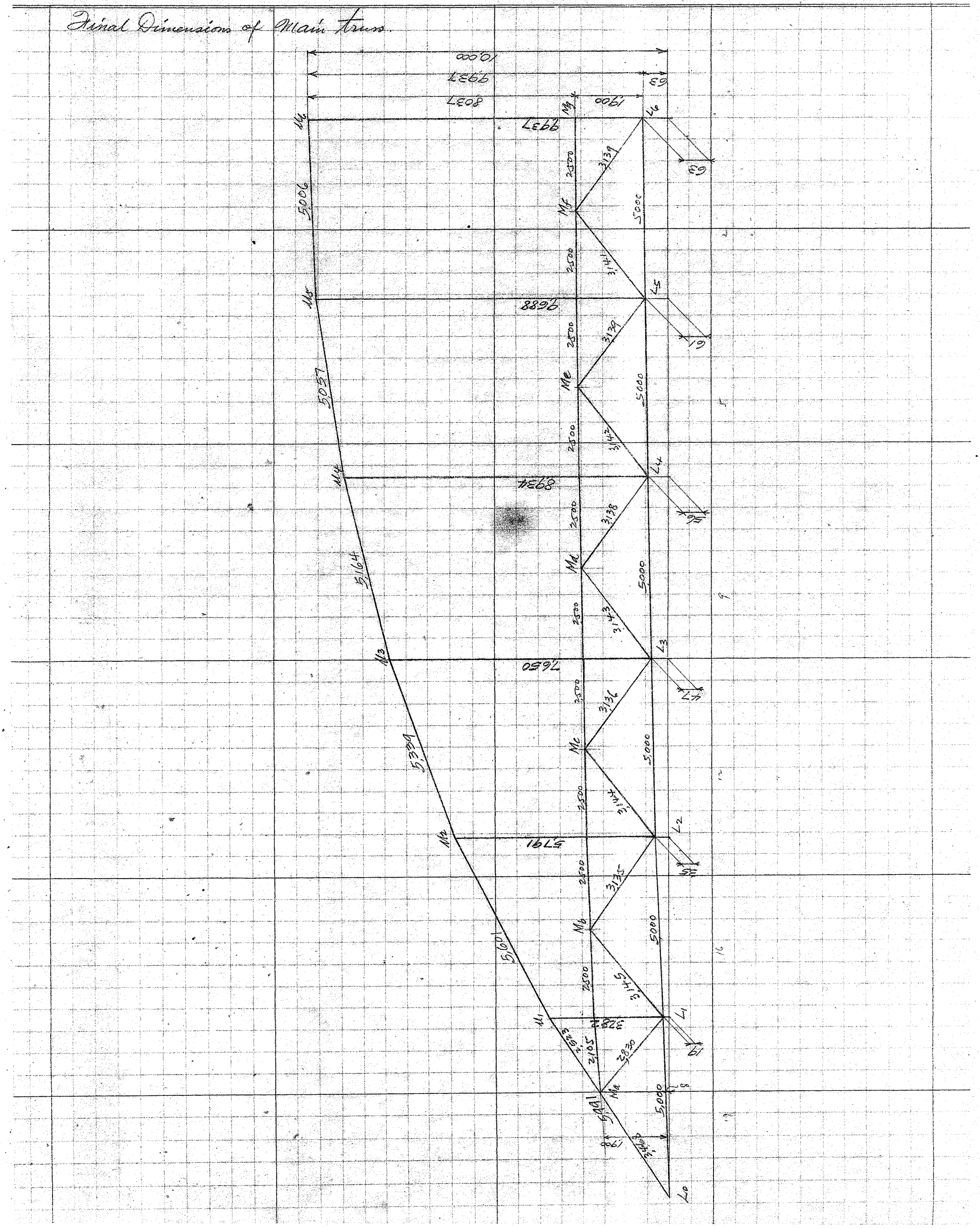
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CALCULATIONS FOR

Final Dimensions of Main truss.

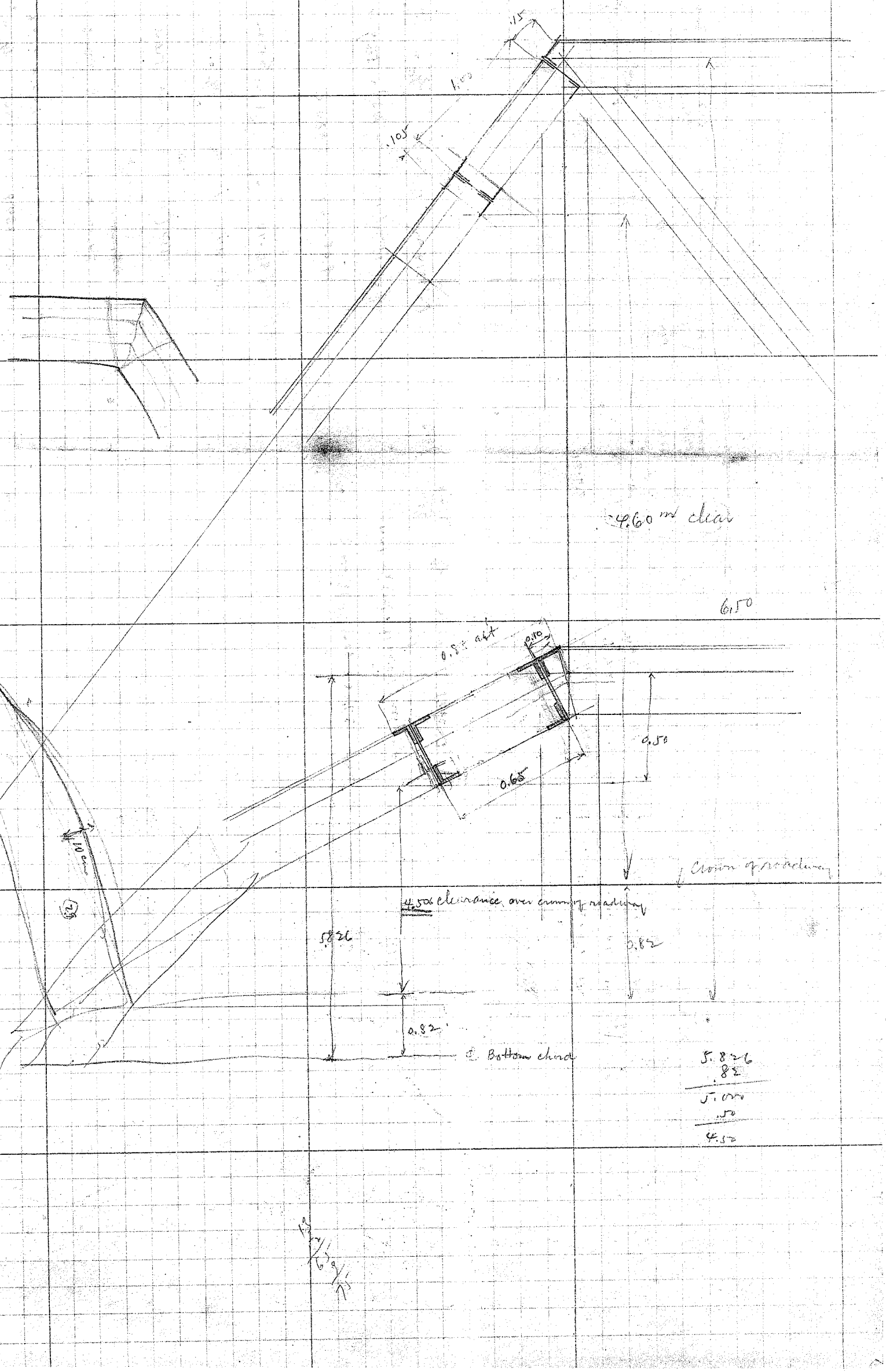


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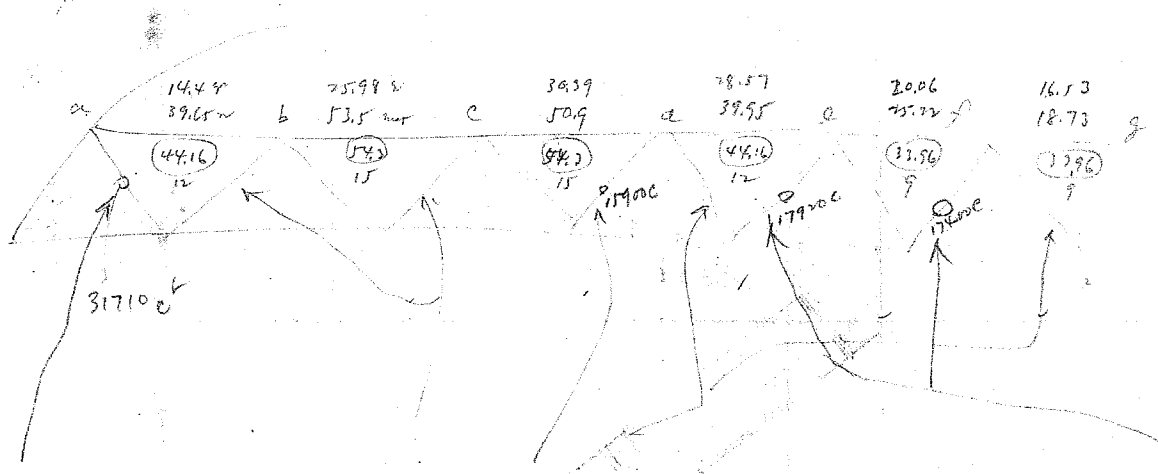
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CALCULATIONS FOR



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2C 150×90×9 = 41.88 − 7.92 = 33.96

2C 150×90×12 = 54.72 − 10.56 = 44.16

2C 150×90×15 = 67.50 − 13.20 = 54.30

2C 125×90×9 = 37.08 − 7.92 = 29.16

ℓ = 284.8 r = 3.94 $\lambda = \frac{284.8}{3.94} = 72.3$

$f = 1500\left(1 - 0.0055 \times 72.3\right) = 904$

37.08 @ 904 = 33520 ㎏

2C 90×75×9 = 28.08 − 7.92 = 20.16

ℓ = 314.0㎝ r_x = 2.78 $\lambda = \frac{314}{2.78} = 113$

$f = 1500\left(1 - 0.0055 \times 113\right) = 576$ ㎏/㎠

28.08×576 = 15980 ㎏

2C 90×90×10 = 34.60 − 8.8 = 25.80㎠

ℓ = 314 r = 2.71 $\lambda = \frac{314}{2.71} = 116$

$f = 1500\left(1 - 0.0055 \times 116\right) = 544$

34.60 @ 544 = 18830 ㎏

2C 125×90×10 = 52.52 − 11.44 = 41.08

ℓ = 284.8 r = 3.92 $\lambda = \frac{284.8}{3.92} = 72.7$

$f = 1500\left(1 - 0.0055 \times 72.7\right) = 900$ ㎏

52.52×900 = 47268㎏

2C 125×90×10 = 41.00 − 8.8 = 32.20㎠

$\lambda = \frac{284.8}{3.93} = 72.5$

$f = 1500\left(1 - 0.0055 \times 72.5\right) = 902$

41.0 @ 902 = 37000 ㎏

2C 90×75×12 = 36.72 − 10.56 = 26.16

ℓ = 314. r = 2.77 $\lambda = \frac{314}{2.77} = 113.3$

$f = 1500\left(1 - 0.0055 \times 113.3\right) = 564$

36.72 @ 564 = 20720 ㎏

110 565

1580

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BOLT CONNECTION DETAIL DATE FILE NO

(1) $\Sigma H=0$ $\Sigma V=0$ $\Sigma M=0$

(2) 支点 ○M

(3) Reaction, Reaction force to, Sectional force, bending stress

(4) Dead load stress (analytical method)

Parallel, Howe, pratt, warren, Double warren, whipple, K,
Baltimore, ~~petit~~, compound Triangular,
(graphical method)
~~Truss load stress~~

curved chord — Uniload stress = 荷重
warren, pratt, petit — loading manner.

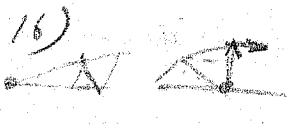
cantilever truss, continuous truss
conventional load system
skew truss, portal bracing, sway bracing, wind bracing
Deflection,

snow load, ~~Seismic~~ force centrifugal brake force

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CALCULATIONS FOR

Chapter (I)
Section (1) The truss and its elements. (Johnson 4)
Section (2) resultant and equilibrium of concurrent forces (p. 16)
$\Sigma H = 0$, $\Sigma V = 0$ $\Sigma M = 0$. ($\Sigma M = 0$ moment at intersection)
algebraic solution, graphical solution



section (3) Resultant and equilibrium of non concurrent forces
$\Sigma H = 0$, $\Sigma V = 0$ $\Sigma M = 0$
algebraic solution, graphical solution.

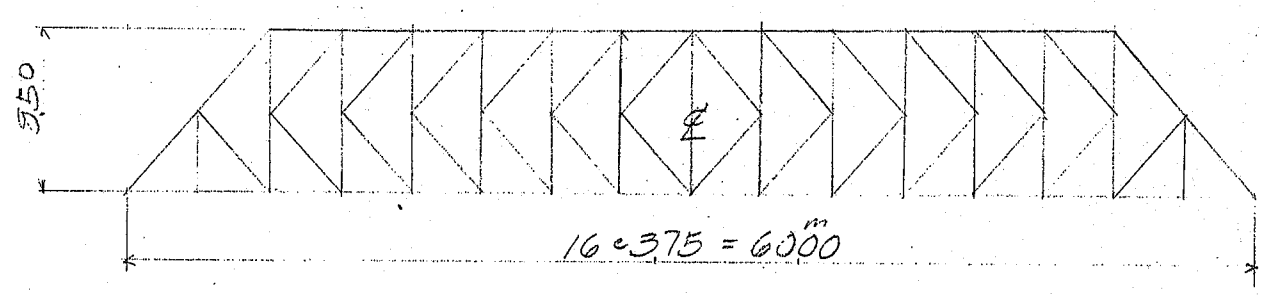
section (4) parallel force
algebraic solution, graphical solution.

section (5) application to the structure (determined to reaction) stress.
chapter (II) Dead load stress
section (6) Type of structure
section (7) analytical solution of parallel chord.
section (8) graphical
section (9) curved chord (analytical)
section (10) (graphical)
chapter (III) Live load stress (uniform & concentration)
section (11) algebraic solution (parallel chord)
section (12) graphical solution (parallel chord)
section (13) algebraic solution (curved chord)
section (14) graphical solution (curved chord)

chapter IIII Solution for coefficient
section (15) Dead Load stress { parallel chord / curved chord
section (16) Live load stress { parallel chord / curved chord

chapter V Lateral, portal, sway bracing (wind & seismic)
chapter VI Tower & column
chapter VII Deflection
chapter VIII Secondary stress

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府縣道標準型橋梁

橋長六拾メ 応力計算書

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CALCULATIONS FOR

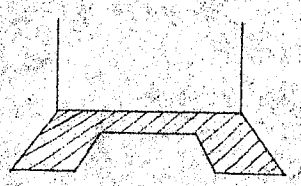
Materials of Kisogawa-Bashi for Aichi-Prefecture

Materials of River caisson

Concrete 1:3:6 Mixture

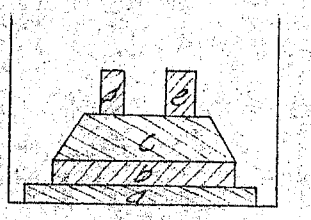
Floating caisson for 7.25m

Base (1.5m depth)



(140·54+164·78)×½×1.5 = 152.640
less (106·20+124·38)×½×1.35 = -46.116
106.524 cub.m

On working chamber (2.0m depth) (1:3:6 mix)

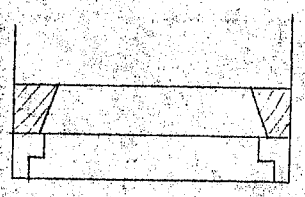


a 13.5·49·3 = 19.845
less (.12·2·24+.12·.12·4)·.24 = -0.152 (木材)
〃 3·3·49·4 = -1.764 (〃)
b 12.6·40·3 = 15.120
c (12.6·40+11.1·25)·½·1.4 = 54.705
less .2·.2·.25·4 = -.400 (木材 F7)
d 122φ × 1.35 = 2.163
e 91φ × 1.35 = 1.203
90.72 cub.m

Concrete 1:2:4 mixture

Summary 197.244 cub.m.

Wall of working chamber (14.5m depth) (1:2:4 mix)



13.66·5.06·14.5 = 100.223
less (13.0·4.4+11.5·2.9)·½·1.45 = -65.649 (作業室)
〃 .2·.2·1.45·56 = -3.249 (木材)
31.326 cub.m

Slab (1.8m depth)

			sectional Area	
	(13.9×5.3)·1.8	= 73.670 sqm	1.8 = 132.606	
less	.2·.2·1.8·28	= -2.016		(側柱)
〃	〃 〃 13.26·4	= -2.122		(切張 F1, F2)
〃	〃 〃 2.68·4	= -.429		(〃 F6)
〃	〃 〃 4.65·18	= -3.348		(〃 F5)
〃	.075·.2·4·96	= -.576		(切張踹継材 I1)
〃	.2·.2·8·23	= -.736		(竪材 E1)
〃	〃 〃 .2·14	= -.112		(〃 〃)
〃	〃 〃 .25·6	= -.060		(斜材 C1, 上部)
〃	.075·.2·1.1·12	= -.198	-15.729	(〃 C2)
〃	〃 〃 2.0·36	= -1.080		(〃 C5)
〃	〃 〃 .3·24	= -.108		(猫木 C3)
〃	〃 〃 .5·12	= -.090		(筋違 H1, 下部)
〃	.12·.65·.53·2	= -.827		(側壁横板 PL16)
〃	〃 〃 13.66·2	= -2.131		(〃 PL16, 17, 15)
〃	〃 .2·.53·2	= -.254		(〃 布木 G1)
〃	〃 〃 13.9·2	= -.667		(〃 〃 G2, G1)
〃	.075·.2·.8·10	= -.120		(筋違 H3, 下部)
〃	.12·.26·11.1·2	= -.693		(作業室側壁横板 PL1)
〃	〃 〃 .26·2	= -.162		(〃 〃 PL9)
				116.877 cub.m

Shell (1 Lot 34m depth)

Sectional area

side wall 13.5×(13.9·2+2.6×2) = 44.55
partition wall .7·2.6·2 = 3.64
48.19 sq.m

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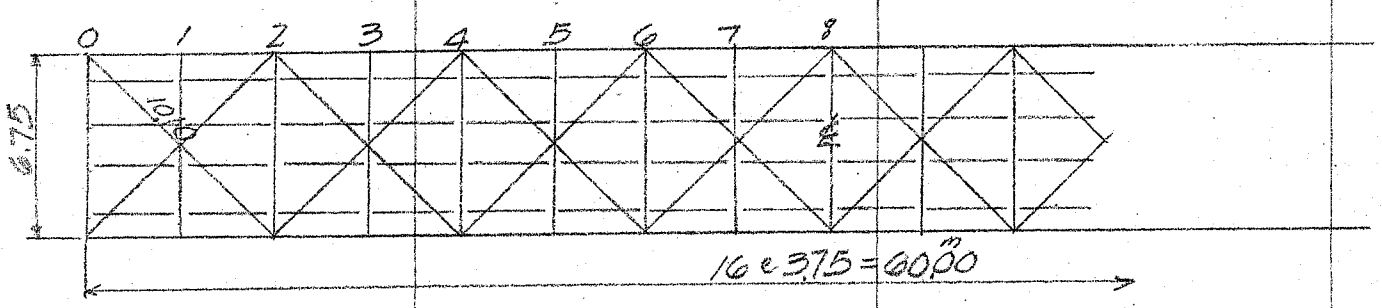
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CALCULATIONS FOR

府縣道標準型橋梁六拾米

Design of Lower lateral bracings



Wind load Loaded chord 400 Kgs per lin meter $\sec\theta = \frac{10.10}{6.75} = 1.50$

Panel load $400 \times 7.50 = 3{,}000$ Kgs $\tan\theta = \frac{7.50}{6.75} = 1.11$

Diagonal stresses

Panel		Shear		stress		SR	19°R
0-2	$3{,}000 \times \frac{1}{8} \times 38$	$= 14{,}250 \times 1.50$	$=$	21,400	$\div 1{,}200$	$= 17.83$	10.1
2-4	$\times 21$	$= 7{,}870$	$=$	11,800		$= 9.84$	5.6
4-6	$\times 15$	$= 5{,}620$	$=$	8,430		$= 7.03$	4.0
6-8	$\times 10$	$= 3{,}750$	$=$	5,620		$= 4.69$	2.6

Chord stresses

Panel				stress
0-2	$3{,}000 \times 3.5 \times 1.11$	$=$		11,650
2-4	$\times 6.0 \times$ "	$=$		20,000
4-6	$\times 7.5 \times$ "	$=$		25,000
6-8	$\times 8.0 \times$ "	$=$		26,600

Stresses during earthquake acceleration assumed 1,500 m/m/sec² or K = 0.15

approximate weight of structures

Floor slab and pavement	550×450	$= 2{,}475$
coping and handrails	$2 \text{ c } 270$	$= 540$
Stringers	$4 \text{ c } 50$	$= 200$
Floor beam		$= 240$
Lower laterals say		$= 60$
Trusses say	$2 \text{ c } \frac{2}{3} \times 750$	$= 1{,}000$
Miscellaneous		$= 55$

4,570 Kgs per lin meter of span

Panel load $4{,}570 \times 7.50 = 34{,}300$ Kgs
Seismic force $34{,}300 \times 0.15 = 5{,}140$ Kgs

Diagonal stresses

Panel			shear		stress		SR	19°R
0-2	$5{,}140 \div \frac{1}{8} \times$	$38 =$	$24{,}400 \times 1.5$	$=$	36,600	$\div 1{,}920 =$	19.05	10.8
2-4		$21 =$	$13{,}500 \times$	$=$	20,250	$=$	10.55	6.0
4-6		$15 =$	$9{,}640 \times$	$=$	14,450		7.53	4.3
6-8		$10 =$	$6{,}430 \times$	$=$	9,650		5.03	2.8

0-2	use	1L $130 \times 130 \times 9 = 22.59 - 1.98 = 20.61$	$r = 2.52$	$l/r = \frac{370}{2.52} = 147$	
2-4, 4-6, 6-8		1L $100 \times 100 \times 10 = 19.00 - 2.20 = 16.80$	$r = 1.92$	$l/r = \frac{370}{1.92} = 193$	

Approximate weight of Lateral bracings

8 L's	$130 \times 130 \times 9 \times 17.7 \text{ c } 4.45$	$=$	630
24 L's	$100 \times 100 \times 10 \times 14.9 \text{ c } 4.45$	$=$	1,590
Conn. 8 P/s	$1{,}000 \times 9 \times 7.065 \text{ c } 1.10$	$=$	620
			2,840 Kgs

$2{,}840 \div 60 = 47.4$ Kgs per lin meter of span

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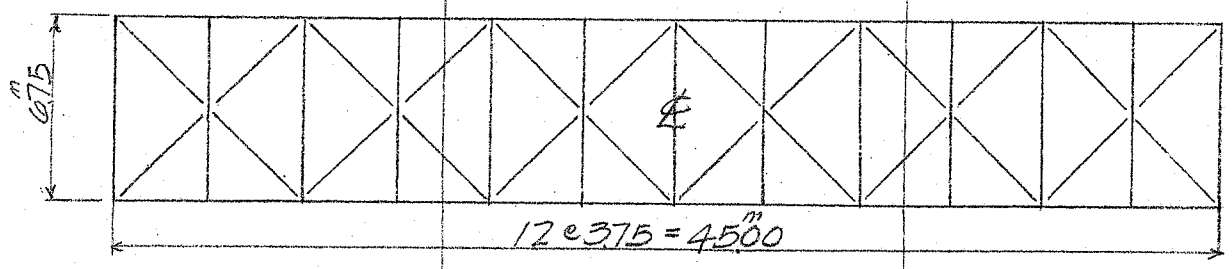
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CALCULATIONS FOR

府縣道標準型橋梁六拾米

Design of Top lateral bracings



wind load unloaded chord
200 kgs per lin meter
panel load 200 × 7.50 = 1,500 kgs
sec θ = 1.50
tan θ = 1.11

Diagonal stresses

Panel			shear		stress		S.R	19 R
2-4	1,500 × 1/6	× 1.5 =	3,750	× 1.50 =	5,620	÷ 1,200 =	4.69	2.6
4-6		× 1.0 =	2,500	=	3,750	=	3.13	1.8
6-8		× 0.6 =	1,500	=	2,250	=	1.88	1.1

Chord stresses

Panel				stress
2-4	1,500	× 2.5 × 1.11	=	4,120
4-6		× 4.0 × "	=	6,600
6-8		× 4.5 × "	=	7,420

Stresses during earthquake acceleration 1,500 mm/sec² or k = 0.15
approximate weight of structures

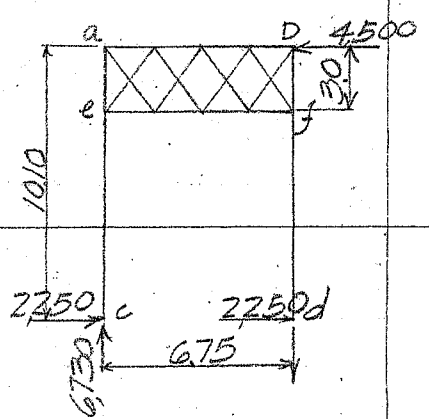
Upper laterals	say		= 50
portal + sway			= 110
Struts	say		= 35
Trusses	say	2 × 1/3 × 750	= 500
miscellaneous			= 55
			750 kgs per lin. meter of span

Panel load 750 × 7.5 = 5620 Kgs
Seismic force 5,620 × 0.15 = 844 Kgs
Seismic force smaller than wind load

use 1L 100×100×10 = 19.00 − 2.20 = 16.8 cm²
r = 1.92 l/r = 370/1.92 = 193

Approximate weight of upper lateral bracings
24 L 100×100×10 × 14.9 @ 4.50 = 1,010
Conn. 6 P/s 900 × 9 × 63.59 @ 1.00 = 380
1,990 kgs
1,990 ÷ 45.0 = 44.3 kgs per lin meter

Design of portal bracing



Wind panel load 1,500 × 3.0 = 4,500 kgs
Seismic force 844 kgs

Max. stress for a-b = (6.730 × 6.75 − 2.250 × 7.10)/3.0 = −9,800 kgs
for e-f = (6.730 × 6.75 − 2.250 × 10.10)/3.0 = +7,570 kgs

Stresses of lattice members = (4,500 × 344)/(2 × 1.69) = 4,580 kgs ten. + comp.

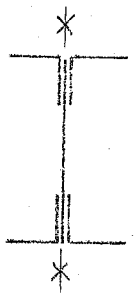
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CALCULATIONS FOR

府縣道標準型橋梁六拾米



use 2L 125×90×9 = 37.08 − 7.92 = 29.16
rx = 5.87 $\ell/r_x = \frac{600}{587} = 102$

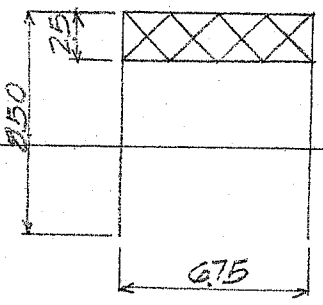
allowable fibre stress for compression flange $1{,}200\times(1-0.012\times\frac{600}{19.0}) = 745$ kg/cm²
9,800 ÷ 74.5 = 131.5 cm²

Web members 4,580 ÷ 1,000 = 4.58 cm²
use 1L 75×75×9 = 12.69 cm²

Approximate weight of portal bracing
4L 125×90×9×14.6 @ 6.15 = 359
8L 75×75×9×9.96 @ 3.25 = 259
Connection P/s = 102
720 kgs

Sway bracing

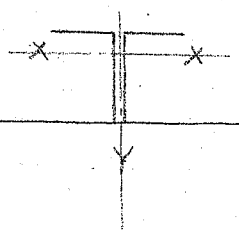
use 2L 100×75×10 = 33.00 − 8.9 = 24.2
rx = 4.77 $\ell/r_x = \frac{600}{4.77} = 126$



web members use 1L 75×75×9 = 12.69 cm²

Approximate weight of sway bracings
4L 100×75×10×13.0 @ 6.15 = 320
8L 75×75×9×9.96 @ 3.25 = 259
Conn. P/s. = 107
650 kgs

Strut



use 2L 150×90×9 = 41.58 cm²
rx = 4.75 $\ell/r_x = \frac{600}{4.75} = 126$

Approximate weight of strut
2L 150×98×9 = 163 × 6.10 = 199
misc. = 11
210 kgs

Design of truss Span length 16 @ 3.75 = 60.00 meters
Dead Load
Upper panel concentration
upper laterals = 25
portal + sway = 55
strut = 15
Truss say 1/3 × 750 = 250
misc. = 25
370 Kgs per lin meter of truss

Upper panel concentration 370 × 3.75 = 1,390 Kgs = P1

Lower panel concentration
Floor slab + pavement 2.75 × 450 = 1,230
Coping and handrails = 270
Stringers 2 @ 50 = 100
Floor beam = 120
Lower laterals = 30
Truss say 2/3 × 750 = 500
misc. = 20
2,270 Kgs per lin meter of truss

Lower panel concentration 2,270 × 3.75 = 8,500 Kgs = P2

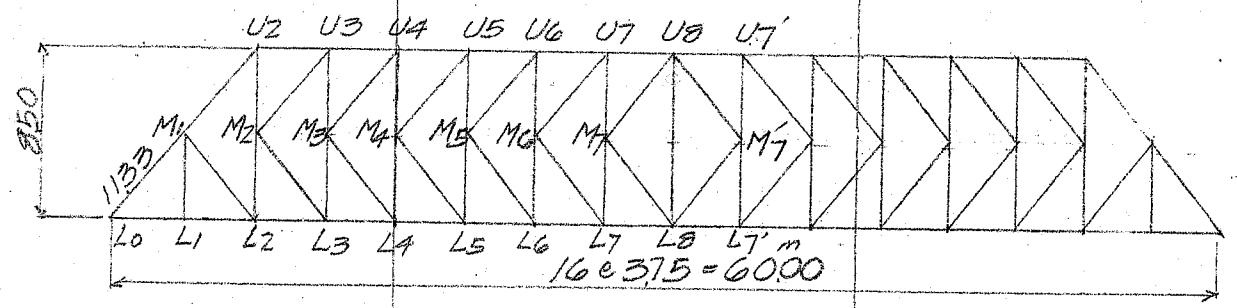
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CALCULATIONS FOR

府縣道標準型橋梁六拾米



$\sec\theta = \frac{1133}{950} = 1.34$

$\cos\theta = \frac{950}{1133} = 0.75$

$\tan\theta = \frac{750}{950} = 0.88$

Dead load stresses

Chord stress $3.75 \div 9.50 = 0.44$ $P_1 = 1{,}390$ $P_2 = 9{,}500$

U2-U3	$\{(6.5P_1+7.5P_2)\times 2 - 1P_2\}\times 0.44$	$=-5.72P_1-6.16P_2$	$=-7{,}950-52{,}350$	$=-60{,}300$
U3-U4	$\{\quad\quad\quad\quad 3-(1P_1+3P_2)\}\times$ 〃	$=-8.14P_1-8.58P_2$	$=-11{,}310-72{,}900$	$=-84{,}210$
U4-U5	$\{\quad\quad\quad\quad 4-(3P_1+6P_2)\}\times$ 〃	$=-10.12P_1-10.56P_2$	$=-14{,}050-89{,}700$	$=-103{,}750$
U5-U6	$\{\quad\quad\quad\quad 5-(6P_1+10P_2)\}\times$ 〃	$=-11.65P_1-12.10P_2$	$=-16{,}200-102{,}750$	$=-118{,}950$
U6-U7	$\{\quad\quad\quad\quad 6-(10P_1+15P_2)\}\times$ 〃	$=-12.75P_1-13.20P_2$	$=-17{,}720-112{,}100$	$=-129{,}820$
U7-U8	$\{\quad\quad\quad\quad 7-(15P_1+21P_2)\}\times$ 〃	$=-13.40P_1-13.85P_2$	$=-18{,}630-117{,}600$	$=-136{,}230$
L0-L2	$\{(\quad\quad\quad\quad)\times 0.88$	$=+5.72P_1+6.60P_2$	$=+7{,}950+56{,}100$	$=+64{,}050$
L2-L3		$=$	$=$	$=+60{,}300$
L3-L4		$=$	$=$	$=+84{,}210$
L4-L5		$=$	$=$	$=+103{,}750$
L5-L6		$=$	$=$	$=+118{,}950$
L6-L7		$=$	$=$	$=+129{,}820$
L7-L8		$=$	$=$	$=+136{,}230$

Web members stresses

M1-L1	$1.0P_2$	$=+1.00P_2$	$=+8{,}500$	$=+8{,}500$
M1-L2	$0.5P_2 \times \frac{1.34}{1.34}$	$=-0.67P_2$	$=-5{,}690$	$=-5{,}690$
M2-U3	$\{(6.5P_1+7.5P_2)-(1P_1+2P_2)\}\times\frac{1.34}{2}$	$=-3.69P_1-3.69P_2$	$=-5{,}130-31{,}350$	$=-36{,}480$
M2-L3		$=$	$=+5{,}130+31{,}350$	$=+36{,}480$
M3-U4	$\{(\quad\quad\quad\quad)-(2P_1+3P_2)\}\times$ 〃	$=-3.02P_1-3.02P_2$	$=-4{,}200-25{,}650$	$=-29{,}850$
M3-L4		$=$	$=+4{,}200+25{,}650$	$=+29{,}850$
M4-U5	$\{(\quad\quad\quad\quad)-(3P_1+4P_2)\}\times$ 〃	$=-2.35P_1-2.35P_2$	$=-3{,}270-19{,}950$	$=-23{,}220$
M4-L5		$=$	$=+3{,}270+19{,}950$	$=+23{,}220$
M5-U6	$\{(\quad\quad\quad\quad)-(4P_1+5P_2)\}\times$ 〃	$=-1.67P_1-1.67P_2$	$=-2{,}320-14{,}200$	$=-16{,}520$
M5-L6		$=$	$=+2{,}320+14{,}200$	$=+16{,}520$
M6-U7	$\{(\quad\quad\quad\quad)-(5P_1+6P_2)\}\times$ 〃	$=-1.00P_1-1.00P_2$	$=-1{,}390-8{,}500$	$=-9{,}890$
M6-L7		$=$	$=+1{,}390+8{,}500$	$=+9{,}890$
M7-U8	$\{(\quad\quad\quad\quad)-(6P_1+7P_2)\}\times$ 〃	$=-0.34P_1-0.34P_2$	$=-470-2{,}900$	$=-3370$
M7-L8		$=$	$=+470+2{,}900$	$=+3370$
U2-M2	$8.70P_1\times 0.75 - P_1 + 9.38P_2\times 0.75$	$=+5.50P_1+7.00P_2$	$=+7{,}640+59{,}500$	$=+67{,}140$
M2-L2	$0.67P_2\times 0.75 + 1.00P_2$	$=+1.50P_2$	$=+12{,}750$	$=+12{,}750$
U3-M3	$3.69P_1\times 0.75 - P_1 + 3.69P_2\times 0.75$	$=+1.77P_1+2.77P_2$	$=+2460+23500$	$=+25960$
M3-L3	$3.69P_1\times 0.75 + 3.69P_2\times 0.75 - 1.00P_2$	$=-2.77P_1-1.77P_2$	$=-3950-15{,}000$	$=-18850$
U4-M4	$3.02P_1\times 0.75 - P_1 + 3.02P_2\times 0.75$	$=+1.26P_1+2.26P_2$	$=+1{,}750+19{,}200$	$=+20950$
M4-L4	$3.02P_1\times 0.75 + 3.02P_2\times 0.75 - 1.00P_2$	$=-2.26P_1-1.26P_2$	$=-3140-10{,}700$	$=-13840$
U5-M5	$2.35P_1\times 0.75 - P_1 + 2.35P_2\times 0.75$	$=+0.76P_1+1.76P_2$	$=+1055+14950$	$=+16005$
M5-L5	$2.35P_1\times 0.75 + 2.35P_2\times 0.75 - 1.00P_2$	$=-1.76P_1-0.76P_2$	$=-2450-6460$	$=-8910$
U6-M6	$1.67P_1\times 0.75 - P_1 + 1.67P_2\times 0.75$	$=+0.25P_1+1.25P_2$	$=+350+10600$	$=+10950$
M6-L6	$1.67P_1\times 0.75 + 1.67P_2\times 0.75 - 1.00P_2$	$=-1.25P_1-0.25P_2$	$=-1740-2125$	$=-3865$
U7-M7	$1.00P_1\times 0.75 - P_1 + 1.00P_1\times 0.75$	$=-0.25P_1+0.75P_2$	$=-350+6370$	$=+6020$
M7-L7	$1.00P_1\times 0.75 + 1.00P_2\times 0.75 - 1.00P_2$	$=-0.75P_1+0.25P_2$	$=-1040+2125$	$=+1085$
U8-L8	$2\times 0.34P_1\times 0.75 - P_1 + 2\times 0.34P_2\times 0.75$	$=-0.49P_1+0.51P_2$	$=-680+4335$	$=+3655$
L0-M1	$\{(6.5P_1+7.50P_2)\times 1.34$	$=-8.70P_1+10.05P_2$	$=-12{,}100-85400$	$=-97{,}500$
M1-U2	$\{(\quad\quad\quad\quad)-(1P_2-0.5P_2)\}\times$ 〃	$=-8.70P_1+9.38P_2$	$=-12{,}100-79{,}700$	$=-91{,}800$

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CALCULATIONS FOR

府縣道標準型橋梁六拾米

Live Load stresses

Live Load on truss — uniform live load $w = \frac{100,000}{170+60} = 435$ kgs/m²

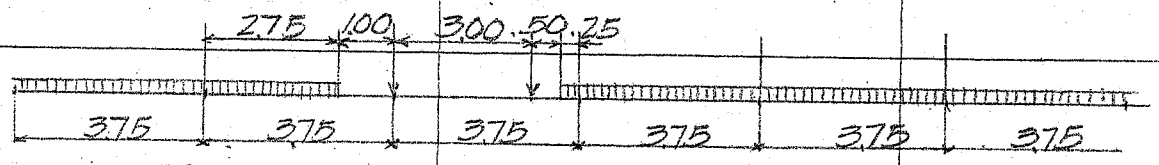
motor truck loading

$\text{Impact} = \frac{20}{60+60} = 16.7\%$

Rear wheel	2,250
16.7% impact	375
	2,625 kgs

Front wheel 2,625 ÷ 3 = 875 kgs including impact

Panel concentration — one motor truck on one truss assumed



2,250			
2,090	1,210		
	5,250		
	350	1,400	
	10	290	
		2,250	4,500
4,340	6,820	3,940	4,500

uniform load 435 × 2.75 = 1,200 kgs

assume the panel concentration for live load 4,500 kgs for panel throughout as full unif. load and 6,820 − 4,500 = 2,320 kgs or say 2,300 kgs concentration for panel load of moto truck

Stresses due to 4,500 kgs concentration

chord stresses

U2-U3	−6.16	× 4,500	=	−27,700 kgs
U3-U4	−8.58	〃	=	−38,600
U4-U5	−10.56	〃	=	−47,500
U5-U6	−12.10	〃	=	−54,400
U6-U7	−13.20	〃	=	−59,400
U7-U8	−13.85	〃	=	−62,300
L0-L2	+ 6.60	〃	=	+29,700
L2-L3	+ 6.16	〃	=	+27,700
L3-L4	+ 8.58	〃	=	+38,600
L4-L5	+ 10.56	〃	=	+47,500
L5-L6	+ 12.10	〃	=	+54,400
L6-L7	+ 13.20	〃	=	+59,400
L7-L8	+ 13.85	〃	=	+62,300

Web members stresses

M1-L1			=	+ 1.00	× 4,500 =	+ 4,500
M1-L2	0.5	× 1.34	=	− 0.67	〃 =	− 3020
M2-U3	(1~13) ÷ 16	× 1.34/2	=	− 3.81	〃 =	− 17,150
M2-L3			=	+ 3.81	〃 =	+ 17,150
M3-U4	(1~12) ÷ 〃	〃	=	− 3.27	〃 =	− 14,700
M3-L4			=	+ 3.27	〃 =	+ 14,700
M4-U5	(1~11) ÷ 〃	〃	=	− 2.76	〃 =	− 12,400
M4-L5			=	+ 2.76	〃 =	+ 12,400
M5-U6	(1~10) ÷ 〃	〃	=	− 2.30	〃 =	− 10,350
M5-L6			=	+ 2.30	〃 =	+ 10,350
M6-U7	(1~9) ÷ 〃	〃	=	− 1.88	〃 =	− 8,460
M6-L7			=	+ 1.88	〃 =	+ 8,460
M7-U8	(1~8) ÷ 〃	〃	=	− 1.51	〃 =	− 6,790
M7-L8			=	+ 1.51	〃 =	+ 6,790
U2-M2	9.38	× 0.75	=	+ 7.00	〃 =	+ 31,500
M2-L2	1.00 + 0.67	〃	=	+ 1.50	〃 =	+ 6,750
U3-M3	(1~13) ÷ 32		=	+ 2.85	〃 =	+ 12,800
M3-L3	(1~12) ÷ 〃		=	− 2.44	〃 =	− 11,000
U4-M4	(1~12) ÷ 32		=	+ 2.44	〃 =	+ 11,000
M4-L4	(1~11) ÷ 〃		=	− 2.06	〃 =	− 9,270

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CALCULATIONS FOR

府縣道標準型橋梁六拾米

U_5-M_5	(1~11) ÷ 32		=	+	2.06	×	4,500	=	+	9,270
M_5-L_5	(1~10) ÷ 〃		=	−	1.72	×	〃	=	−	7,740
U_6-M_6	(1~10) ÷ 〃		=	+	1.72	×	〃	=	+	7,740
M_6-L_6	(1~9) ÷ 〃		=	−	1.41	×	〃	=	−	6,340
U_7-M_7	(1~9) ÷ 〃		=	+	1.41	×	〃	=	+	6,340
M_7-L_7	(1~8) ÷ 〃		=	−	1.13	×	〃	=	−	5,080
U_8-L_8	(1~8) ÷ 32 − {(1~7) ÷ 16 − 1} × ½		=	+	0.75	×	〃	=	+	3,375
L_0-M_1	7.50	× 1.34	=	−	10.05	×	〃	=	−	45,200
M_1-U_2	7.50 − 1.00 + 0.5	× 〃	=	−	9.38	×	〃	=	−	42,200
U_8-M_7'	(1~7) ÷ 32	× 1.34	=	+	1.17	×	〃	=	+	5,260
M_7-L_8	〃	〃	=	−	1.17	×	〃	=	−	5,260

Stresses due to 2,300 kgs single concentration

Chord stresses

U_2-U_3	1/16	× 14 × 2 × 0.44	=	−	0.77	×	2,300	=	−	1,770
U_3-U_4	〃	× 13 × 3 〃	=	−	1.07	×	〃	=	−	2,460
U_4-U_5	〃	× 12 × 4 〃	=	−	1.32	×	〃	=	−	3,035
U_5-U_6	〃	× 11 × 5 〃	=	−	1.51	×	〃	=	−	3,470
U_6-U_7	〃	× 10 × 6 〃	=	−	1.65	×	〃	=	−	3,790
U_7-U_8	〃	× 9 × 7 〃	=	−	1.73	×	〃	=	−	3,980
L_0-L_2	〃	× 15 × 2 × 〃	=	+	0.83	×	〃	=	+	1,910
L_2-L_3			=	+	0.77	×	〃	=	+	1,770
L_3-L_4			=	+	1.07	×	〃	=	+	2,460
L_4-L_5			=	+	1.32	×	〃	=	+	3,035
L_5-L_6			=	+	1.51	×	〃	=	+	3,470
L_6-L_7			=	+	1.65	×	〃	=	+	3,790
L_7-L_8			=	+	1.73	×	〃	=	+	3,980

Web members stresses

M_1-L_1	1.00		=	+	1.00	×	2,300	=	+	2,300
M_1-L_2	0.50	× 1.34	=	−	0.67	×	〃	=	−	1,540
M_2-U_3	1/32	× 13 × 1.34	=	−	0.54	×	〃	=	−	1,240
M_2-L_3			=	+	0.54	×	〃	=	+	1,240
M_3-U_4	〃	× 12 × 〃	=	−	0.50	×	〃	=	−	1,150
M_3-L_4			=	+	0.50	×	〃	=	+	1,150
M_4-U_5	〃	× 11 × 〃	=	−	0.46	×	〃	=	−	1,060
M_4-L_5			=	+	0.46	×	〃	=	+	1,060
M_5-U_6	〃	× 10 × 〃	=	−	0.42	×	〃	=	−	970
M_5-L_6			=	+	0.42	×	〃	=	+	970
M_6-U_7	〃	× 9 × 〃	=	−	0.38	×	〃	=	−	870
M_6-L_7			=	+	0.38	×	〃	=	+	870
M_7-U_8	〃	× 8 × 〃	=	−	0.34	×	〃	=	−	780
M_7-L_8			=	+	0.34	×	〃	=	+	780
U_2-M_2	1/16	× 14	=	+	0.88	×	〃	=	+	2,020
M_2-L_2			=	+	1.00	×	〃	=	+	2,300
U_3-M_3	1/32	× 13	=	+	0.41	×	〃	=	+	940
M_3-L_3	〃	× 12	=	−	0.38	×	〃	=	−	870
U_4-M_4	〃	× 12	=	+	0.38	×	〃	=	+	870
M_4-L_4	〃	× 11	=	−	0.34	×	〃	=	−	780
U_5-M_5	〃	× 11	=	+	0.34	×	〃	=	+	780
M_5-L_5	〃	× 10	=	−	0.31	×	〃	=	−	710
U_6-M_6	〃	× 10	=	+	0.31	×	〃	=	+	710
M_6-L_6	〃	× 9	=	−	0.28	×	〃	=	−	640
U_7-M_7	〃	× 9	=	+	0.28	×	〃	=	+	640
M_7-L_7	〃	× 8	=	−	0.25	×	〃	=	−	580
U_8-L_8	〃	× 8 × 2	=	+	0.50	×	〃	=	+	1,150

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CALCULATIONS FOR

府縣道標準型橋梁天拾米

L_0-M_1	$\frac{1}{16}$ ×	15 × 1.34	= −	1.26	× 2,300	= −	2,900
M_1-U_2	〃 ×	14 × 〃	= −	1.17		= −	2,690
U_8-M_7'	$\frac{1}{32}$ ×	7	= +	0.22		= +	505
$M_7'-L_8$	〃	〃	= −	0.22		= −	505
Stresses due to 4,500 kg. concentration							
$U_7'-M_6'$	(1~6) ÷ 32 × 1.34		= +	0.88	× 4,500	= +	3,960
$L_7'-M_6'$			= −	0.88	〃	= −	3,960
$U_6'-M_5'$	(1~5) 〃	〃	= +	0.63	〃	= +	2,835
$L_6'-M_5'$			= −	0.63	〃	= −	2,835
$U_7'-M_7'$	(1~0) ÷ 32		= −	0.66	〃	= −	2,970
$M_7'-L_7'$	{(1~7) ÷ 16 − 1} ÷ 2.0 + 1.0		= +	1.38	〃	= +	6,200
$U_6'-M_6'$	(1~5) ÷ 32		= −	0.47	〃	= −	2,110
$M_6'-L_6'$	{(1~6) ÷ 16 − 1} ÷ 2.0 + 1.0		= +	1.16	〃	= +	5,220
$U_5'-M_5'$	(1~4) ÷ 32		= −	0.31	〃	= −	1,395
$M_5'-L_5'$	{(1~5) ÷ 16 − 1} ÷ 2.0 + 1.0		= +	0.97	〃	= +	4,360
Stress due to 2,300 kg. single concentration							
$U_7'-M_6'$	$\frac{1}{32}$ × 6	× 1.34	= +	0.25	× 2,300	= +	575
$L_7'-M_6'$			= −	0.25		= −	575
$U_6'-M_5'$	× 5	× 〃	= +	0.21		= +	480
$L_6'-M_5'$			= −	0.21		= −	480
$U_7'-M_7'$	$\frac{1}{32}$ × 6		= −	0.19		= −	440
$M_7'-L_7'$	$\frac{1}{32}$ × 7 − 1.00		= +	0.78		= +	1,795
$U_6'-M_6'$	〃 × 5		= −	0.16		= −	370
$M_6'-L_6'$	〃 × 6 − 1.00		= +	0.81		= +	1,860
$U_5'-M_5'$	〃 × 4		= −	0.13		= −	300
$M_5'-L_5'$	〃 × 5 − 1.00		= +	0.84		= +	1,930

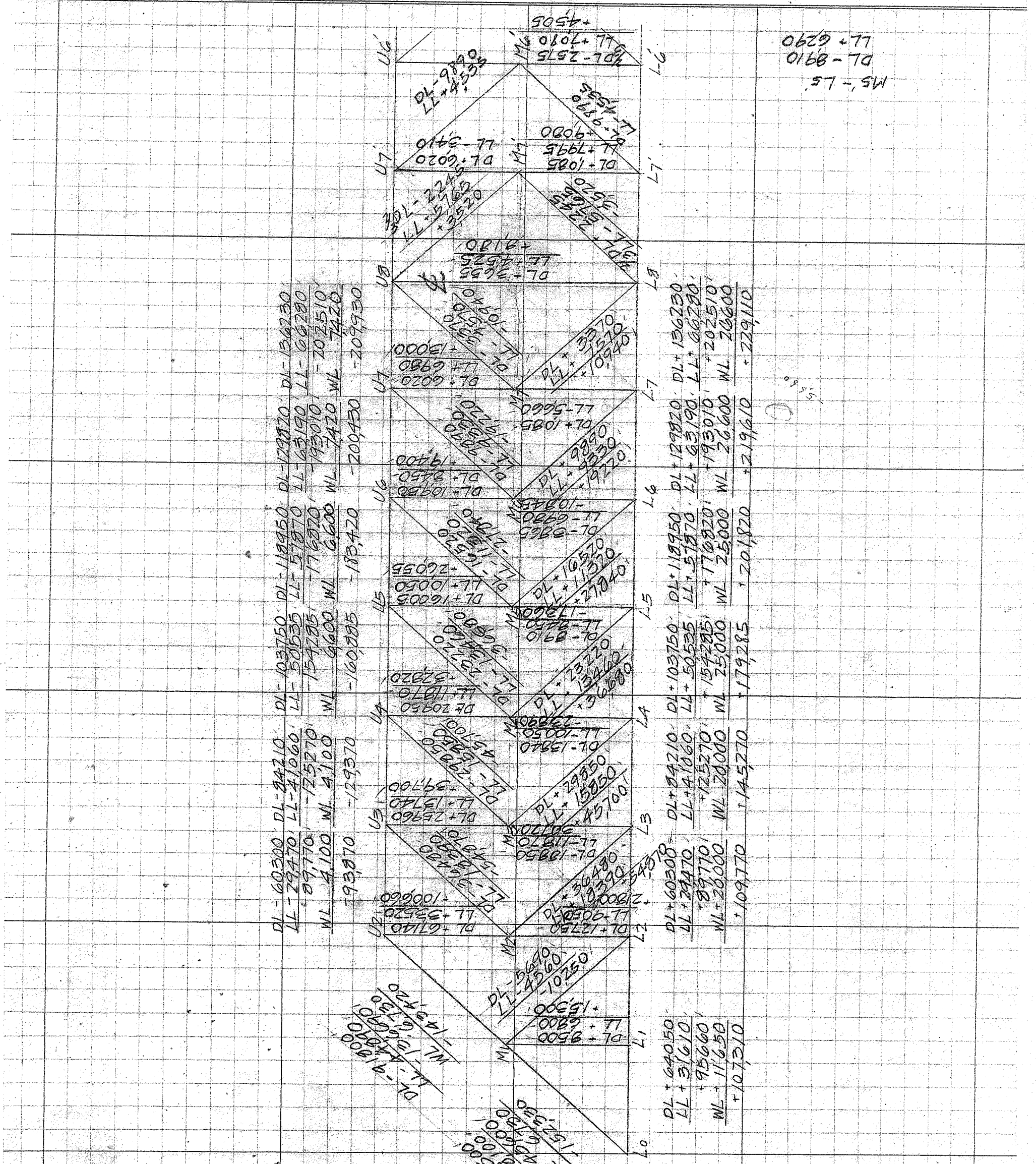
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CALCULATIONS FOR

府縣道標準型橋梁六拾米



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CALCULATIONS FOR

Materials of Kisogawa-Bashi for Aichi-Prefecture

Forms for shaft

Projection under shoe for P1 only

2 @ .13 × 4.30 = 1.12 sq. m.

Coping

Side	2 @ 60	× 9.20	= 11.04
Circular end	1 @ 240°	× 1.60	= 4.52
			15.56 sq.m.

Shaft. Total length of perimeter at top of Shaft

Side	2 @	9.20	= 18.40
Circular end	1 @	210°	= 6.60
			25.00 m

Total length of perimeter at bottom of Shaft

Pier	Circular end	Perimeter		Side	Total perimeter
P1	2.985 φ	9.38	+	18.40	= 27.78
P2	3.020 φ	9.49	+	〃	= 27.89
P3	3.049 φ	9.58	+	〃	= 27.98
P4	3.073 φ	9.65	+	〃	= 28.05
P5	3.090 φ	9.71	+	〃	= 28.11
P6	3.102 φ	9.75	+	〃	= 28.15
P7	3.108 φ	9.76	+	〃	= 28.16

Top length of hollow

4 @ 2.45 + 4 @ 1.11 = 14.24 m

Bottom length of hollow

P1	4 @ 2.45	+ 4 @ 1.785	=	16.94
P2	〃	+ 4 @ 1.820	=	17.08
P3	〃	+ 4 @ 1.849	=	17.20
P4	〃	+ 4 @ 1.872	=	17.29
P5	〃	+ 4 @ 1.890	=	17.36
P6	〃	+ 4 @ 1.902	=	17.41
P7	〃	+ 4 @ 1.908	=	17.43

Total area of forms

Pier	Top l	Bottom l	Mean l	Height	area	Top l	Bottom l	Mean l	Height	area	Coping	Shoe bed	Total area
P1	25.00	27.78	26.39	44.73	116.72	14.24	16.94	15.59	33.73	52.59	15.56	1.12	185.99 sq.m
P2	〃	27.89	26.45	45.99	121.64	〃	17.08	15.66	35.49	55.58	〃		192.78
P3	〃	27.98	26.49	47.46	125.72	〃	17.20	15.72	36.96	58.10	〃		199.38
P4	〃	28.05	26.53	48.63	129.02	〃	17.29	15.77	38.13	60.13	〃		204.71
P5	〃	28.11	26.56	49.51	131.50	〃	17.36	15.80	39.01	61.64	〃		208.70
P6	〃	28.15	26.58	50.10	133.17	〃	17.41	15.83	39.60	62.69	〃		211.42
P7	〃	28.16	26.58	50.39	133.94	〃	17.43	15.84	39.89	63.19	〃		212.69

埋込型枠

Top of hollow	2	@ 1.11 × 2.45	=	5.44
Bottom of slab	3	@ 3.60 × 3.60	=	38.88
less fillets	6	@ .50 × .50	=	-1.50
				42.82 sq.m

Reinforcements, plain bars

P1	60.97 Kg tons
P2 & P3	62.20 〃
P4, P5, P6 & P7	63.32 〃

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CALCULATIONS FOR

Formula's for integration

$\int x^n dx = \frac{x^{n+1}}{n+1} + C$ but except $x=-1$

$\int (a+bx)^n dx = \frac{(a+bx)^{n+1}}{(n+1)b} + C$

$\int \{f(x)\}^n df(x) = \frac{\{f(x)\}^{n+1}}{n+1} + C$

$\int (xy^{x-1}dy + y^x \log_e dx) = y^x + C$

$\int \frac{dx}{x} = \log_e x + C$

$\int \frac{1}{x^2} dx = -\frac{1}{x} + C$

$\int \frac{df(x)}{f(x)} = \log_e f(x) + C$

$\int a^x dx = \frac{a^x}{\log_e a} + C$

$\int x^n dx = \frac{x^{n+1}}{n+1}$ $n \neq -1$ ··· Example: $\int (a+bx)^5 dx = \frac{(a+bx)^6}{6 \cdot b}$ (1; 2; 8)

$\int a^{f(x)} df(x) = \frac{a^{f(x)}}{\log_e a} + C$

$\int a\,dx = ax + C$

$\int a\,df(x) = af(x) + C$

$\int e^x dx = e^x + C$

$\int e^{f(x)} df(x) = e^{f(x)} + C$

$\int \sin x\,dx = -\cos x + C$

$\int \cos mx\,dx = \frac{1}{m} \sin mx + C$

$\int \sin mx\,dx = -\frac{1}{m} \cos mx$,

$\int \tan x\,dx = \int \frac{\sin x}{\cos x} dx = -\log_e \cos x$

$\int \cot x\,dx = \log_e \sin x$

$\int \sin f(x)\,df(x) = -\cos f(x) + C$

$\int \cos x\,dx = \sin x + C$

$\int \cos(x+l)\,dx = \sin(x+l)$

$\int \frac{dx}{\sin x} = \log_e \tan \frac{x}{2}$

$\int \frac{dx}{\cos x} = \log_e \tan\left(\frac{x}{2} + \frac{\pi}{4}\right)$

$\int \cos f(x)\,df(x) = \sin f(x) + C$

$\int \frac{dx}{\sin^2 x} = -\cot x + C$

$\int \frac{\sin x\,dx}{\cos^2 x} = \frac{1}{\cos x}$, $\int \frac{\cos x\,dx}{\sin^2 x} = -\frac{1}{\sin x}$

$\int \frac{df(x)}{\sin^2 f(x)} = -\cot f(x) + C$

$\int \frac{dx}{\sin x \cos x} = \int \left(\frac{1}{\cos^2 x} \times \frac{\sin x}{\cos x}\right) dx = \log_e \tan x$

$\int \frac{dx}{\cos^2 x} = \tan x + C$

$\int \frac{df(x)}{\cos^2 f(x)} = \tan x + C$ ← (D)

$\int \frac{dx}{\sqrt{a^2 - x^2}} = \sin^{-1} \frac{x}{a} + C$ ---- $\int \frac{dx}{\sqrt{1-x^2}} = \sin^{-1} x$ or $-\cos^{-1} x$

$\int \frac{df(x)}{\sqrt{a^2 - \{f(x)\}^2}} = \sin^{-1} \frac{f(x)}{a} + C$

$\int \frac{dx}{a^2 + x^2} = \frac{1}{a} \tan^{-1} \frac{x}{a} + C$ ---- $\int \frac{dx}{1+x^2} = \tan^{-1} x$ or $-\cot^{-1} x$ ← (C)

$\int \frac{df(x)}{a^2 + \{f(x)\}^2} = \frac{1}{a} \tan^{-1} \frac{f(x)}{a} + C$

$\int \sec x \tan x\,dx = \sec x + C$

$\int \sec f(x) \tan f(x)\,df(x) = \sec f(x) + C$

$\int \frac{dx}{1+\cos x} = \tan \frac{x}{2}$

$\int \frac{dx}{1-\cos x} = -\cot \frac{x}{2}$

$\int \operatorname{cosec} x \cot x\,dx = -\operatorname{cosec} x + C$

$\int \operatorname{cosec} f(x) \cot f(x)\,df(x) = -\operatorname{cosec} f(x) + C$

$\int \frac{dx}{x\sqrt{x^2 - a^2}} = \frac{1}{a} \sec^{-1} \frac{x}{a} + C$

$\int \sin^2 x\,dx = \int \sin x\,d(-\cos x) = \frac{1}{2}(-\sin x \cos x + x)$

$\int \cos^2 x\,dx = \int \cos x\,d(\sin x) = \sin x \cos x + \int \sin^2 x\,dx = \frac{1}{2}(\sin x \cos x + x)$

$\int \sin^n x\,dx = -\frac{1}{n} \cos x \sin^{n-1} x + \frac{n-1}{n} \int \sin^{n-2} x\,dx$

$\int \cos^n x\,dx = \frac{1}{n} \sin x \cos^{n-1} x + \frac{n-1}{n} \int \cos^{n-2} x\,dx$

$\int \tan^n x\,dx = \int \frac{y^n}{1+y^2} dy$ $y = \tan x$

$\int \cot^n x\,dx = -\int \frac{y^n}{1+y^2} dy$ $y = \cot x$

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$\int \sin^2 x\,dx = -\frac{1}{4} \sin 2x + \frac{1}{2} x + C$

$\int \cos^2 x\,dx = \frac{1}{4} \sin 2x + \frac{1}{2} x + C$

$\int \sin^n x\,dx = -\frac{\cos \sin^{n-1} x}{n} + \frac{n-1}{n} \int \sin^{n-2} x\,dx$

$\int \cos^n x\,dx = \frac{\sin x \cos^{n-1} x}{n} + \frac{n-1}{n} \int \cos^{n-2} x\,dx$

$\int \tan^n x\,dx = \frac{\tan^{n-1} x}{n-1} - \int \tan^{n-2} x\,dx$

$\int \cot^n x\,dx = -\frac{\cot^{n-1} x}{n-1} - \int \cot^{n-2} x\,dx$

$\int \frac{dx}{\sin^n x} = -\frac{\cos x}{(n-1)\sin^{n-1} x} + \frac{n-2}{n-1} \int \frac{dx}{\sin^{n-2} x}$

$\int \frac{dx}{\cos^n x} = \frac{\sin x}{(n-1)\cos^{n-1} x} + \frac{n-2}{n-1} \int \frac{dx}{\cos^{n-2} x}$

$\int \sin mx \cos nx\,dx = -\frac{\cos(m+n)x}{2(m+n)} - \frac{\cos(m-n)x}{2(m-n)} + C$

$\int \sin mx \sin nx\,dx = \frac{\sin(m-n)x}{2(m-n)} - \frac{\sin(m+n)x}{2(m+n)} + C$

$\int \cos mx \cos nx\,dx = \frac{\sin(m-n)x}{2(m-n)} + \frac{\sin(m+n)x}{2(m+n)} + C$

$\int \sin x \cos x\,dx = \frac{1}{4} \int \sin 2x\,d(2x) = -\frac{1}{4} \cos 2x$

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JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

CALCULATIONS FOR

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$$\int \frac{df(x)}{f(x)\sqrt{\{f(x)\}^2-a^2}} = \frac{1}{a}\sec^{-1}\frac{f(x)}{a} + C$$

$$\int\{f(x)+F(x)+\cdots\}dx = \int f(x)dx + \int F(x)dx + \cdots$$

$$\int m f(x)\,dx = m\int f(x)dx \quad \text{but } m = \text{恒数}$$

$$\int u\,dv = uv - \int v\,du$$

$$\int u\frac{dv}{dx}dx = uv - \int v\frac{du}{dx}dx$$

$$\int \frac{dx}{a+bx} = \frac{1}{b}\log_e(a+bx)$$

$$\int \frac{dx}{(a+bx)^2} = -\frac{1}{b(a+bx)}$$

$$\int \frac{dx}{(a+bx)^3} = -\frac{1}{2b(a+bx)^2}$$

$$\int \frac{x\,dx}{a+bx} = \frac{1}{b^2}\{a+bx-a\log(a+bx)\}$$

$$\int \frac{x\,dx}{(a+bx)^2} = \frac{1}{b^2}\left\{\log(a+bx)+\frac{a}{a+bx}\right\}$$

$$\int \sqrt{a+bx}\,dx = \frac{2}{3b}\sqrt{(a+bx)^3}$$

$$\int x\sqrt{a+bx}\,dx = -\frac{2(2a-3bx)\sqrt{(a+bx)^3}}{15b^2}$$

$$\int \frac{dx}{\sqrt{a+bx}} = \frac{2\sqrt{a+bx}}{b}$$

$$\int \frac{x\,dx}{\sqrt{a+bx}} = -\frac{2(2a-bx)\sqrt{a+bx}}{3b^2}$$

$$\int \frac{x^2dx}{\sqrt{a+bx}} = \frac{2(8a^2-4abx+3b^2x^2)\sqrt{a+bx}}{15b^3}$$

$\int x^n\sqrt{a+bx}\,dx$ and $\int \frac{x^n dx}{\sqrt{a+bx}}$ desgl.

$\int \frac{dx}{\alpha+\beta x\sqrt{a+bx}}$ } zu lösen durch die substitution $y=\sqrt{a+bx}$

$$\int \frac{dx}{\sqrt{a-bx^2}} = \frac{1}{\sqrt{b}}\sin^{-1}\left(\sqrt{\frac{b}{a}}x\right)$$

$$\int \frac{dx}{\sqrt{q\pm 2px-x^2}} = \sin^{-1}\frac{x\mp p}{\sqrt{q+p^2}}$$

$$\int \frac{x\,dx}{\sqrt{q+2px-x^2}} = -\sqrt{q+2px-x^2} + p\sin^{-1}\frac{x-p}{\sqrt{p^2+q}}$$

$$\int \frac{x^2dx}{\sqrt{a^2-x^2}} = \frac{a^2}{2}\sin^{-1}\frac{x}{a} - \frac{x}{2}\sqrt{a^2-x^2}$$

$$\int \sqrt{a^2-x^2}\,dx = \frac{x}{2}\sqrt{a^2-x^2} + \frac{a^2}{2}\sin^{-1}\frac{x}{a}$$

$$\int x\sqrt{a^2-x^2}\,dx = -\frac{1}{3}\sqrt{(a^2-x^2)^3}$$

$$\int x^2\sqrt{a^2-x^2}\,dx = \frac{x}{4}\left(x^2-\frac{a^2}{2}\right)\sqrt{a^2-x^2} + \frac{a^4}{16}\sin^{-1}\frac{2x^2-a^2}{a^2}$$

$$\int \frac{dx}{\sqrt{q+2px+x^2}} = \log_e(p+x+\sqrt{q+2px+x^2})$$

$$\int \frac{x\,dx}{\sqrt{x^2\pm a^2}} = \sqrt{x^2\pm a^2}$$

$$\int \frac{x\,dx}{\sqrt{q+2px+x^2}} = \sqrt{q+2px+x^2} - p\log_e(p+x+\sqrt{q+2px+x^2})$$

$$\int \frac{x^2dx}{\sqrt{x^2\pm a^2}} = \frac{x}{2}\sqrt{x^2\pm a^2} \mp \frac{a^2}{2}\log_e(x+\sqrt{x^2\pm a^2})$$

$$\int \frac{dx}{x^2-1} = \frac{1}{2}\frac{x-1}{x+1} \text{ or } \operatorname{Ar}\coth x \quad \text{when } x>1$$

$$\int \frac{dx}{1-x^2} = \frac{1}{2}\log_e\frac{1+x}{1-x} \text{ or } \operatorname{Ar}\tanh x \quad \text{when } x<1$$

$$\int \frac{dx}{a+bx^2} = \frac{1}{\sqrt{ab}}\tan^{-1}\left(x\sqrt{\frac{b}{a}}\right)$$

$$\int \frac{dx}{x^2+2px+q} = \int \frac{dx}{(x+p)^2+(q-p^2)} = \frac{1}{\sqrt{q-p^2}}\tan^{-1}\frac{x+p}{\sqrt{q-p^2}} \quad \text{when } q>p^2$$

$$\int \frac{dx}{a-bx^2} = \frac{1}{2\sqrt{ab}}\log_e\frac{\sqrt{a}+x\sqrt{b}}{\sqrt{a}-x\sqrt{b}} = \frac{1}{\sqrt{ab}}\operatorname{Ar}\tanh\left(x\sqrt{\frac{b}{a}}\right)$$

$$\int \frac{dx}{x^2+2px+q} = \frac{1}{2\sqrt{p^2-q}}\log_e\frac{x+p-\sqrt{p^2-q}}{x+p+\sqrt{p^2-q}} \quad \text{when } p^2>q$$

$$\int \frac{(x+b)dx}{x^2+2px+q} = \frac{1}{2}\log_e(x^2+2px+q) + (b-q)\int \frac{dx}{x^2+2px+q}$$

Example $\int \frac{3x+4}{x^2+4x+8}dx = 3\int \frac{x+\frac{4}{3}}{x^2+4x+8}dx = \frac{3}{2}\log_e(x^2+4x+8)$

$$-3\times\frac{2}{3}\int \frac{dx}{x^2+4x+8} = \frac{3}{2}\log_e(x^2+4x+8) - \tan^{-1}\frac{x+2}{2}$$

$\int \frac{f(x)dx}{x^2+2px+q}$ ist $f(x)$ eine algebraische Funktion höheren Grades, so dividiere man mit dem Nenner in $f(x)$ und integriere die entstehende Summe.

Example $\int \frac{2x^3-7x^2-6x+8}{x^2-6x+7}dx = \int(2x+5)dx + \int \frac{10x-27}{x^2-6x+7}dx$

$\int \frac{f(x)dx}{g(x)}$ ist der Quotient eine echt gebrochene, rationale Funktion, so zerlegt man ihn in eine Summe von Teilbrüchen.

$$\int \frac{dx}{(a+2bx+cx^2)^p} = \frac{1}{2(ac-b^2)(p-1)}\frac{b+cx}{(a+2bx+cx^2)^{p-1}} + \frac{(2p-3)c}{2(ac-b^2)(p-1)}\int \frac{dx}{(a+2bx+cx^2)^{p-1}}$$

$$\int \frac{(\alpha-\beta x)dx}{(a+2bx+cx^2)^p} = -\frac{\beta}{2c(p-1)}\frac{1}{(a+2bx+cx^2)^{p-1}} + \frac{\alpha c-\beta b}{c}\int \frac{dx}{(a+2bx+cx^2)^p}$$

$$\int x^{m-1}(a+bx)^n dx = \frac{x^{m-1}(a+bx)^{n+1}}{(m+n)b} - \frac{(m-1)a}{(m+n)b}\int x^{m-2}(a+bx)^n dx$$

$$= \frac{x^m(a+bx)^n}{m+n} + \frac{na}{m+n}\int x^{m-1}(a+bx^{n-1})\,dx$$

$$\int \frac{f(x)}{F(x)}dx = \int \frac{Ax^m+Bx^{m-1}+\cdots+Px+Q}{x^n+ax^{n-1}+\cdots Px+q}dx \quad m>n$$

$$\int \frac{(\alpha+\beta x)dx}{\sqrt{a+bx}} = \frac{2}{3b^2}(3\alpha b-2a\beta+\beta bx)\sqrt{a+bx}+C$$

$$\int \frac{dx}{\sqrt{a^2-x^2}} = \sin^{-1}\frac{x}{a}+C = -\cos^{-1}\frac{x}{a}+C' = 2\tan^{-1}\sqrt{\frac{a+x}{a-x}}+C$$

$$\int \frac{dx}{\sqrt{a^2+x^2}} = \log_e\{x+\sqrt{a^2+x^2}\}+C = \frac{1}{2}\log_e\left\{\frac{x+\sqrt{a^2+x^2}}{-x+\sqrt{a^2+x^2}}\right\}+C'$$

$$= \operatorname{Ar}\sinh\frac{x}{a}+C''$$

$\int x^{m-1}$ —

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JIJI BLDG, TOKYO

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$\int \sqrt{x^2 \pm a^2}\,dx = \frac{x}{2}\sqrt{x^2 \pm a^2} \pm \frac{a^2}{2}\log_e\left(x + \sqrt{x^2 \pm a^2}\right)$

$\int x\sqrt{x^2 \pm a^2}\,dx = \frac{1}{3}\sqrt{(x^2 \pm a^2)^3}$

$\int x^2\sqrt{x^2 \pm a^2}\,dx = \frac{1}{4}\left(x^2 \pm \frac{a^2}{2}\right)x\sqrt{x^2 \pm a^2} - \frac{a^4}{16}\log_e\left[\left(x^2 \pm \frac{a^2}{2}\right) + x\sqrt{x^2 \pm a^2}\right]$

$\int \frac{(\alpha + \beta x)\,dx}{\sqrt{(a + 2bx + cx^2)^3}} = \frac{(a\beta - b\alpha) + (b\beta - c\alpha)x}{(b^2 - ac)\sqrt{a + 2bx + cx^2}}$

$\int x e^x dx = (x-1)e^x$

$\int x^m e^x\,dx = x^m e^x - m\int e^x x^{m-1}\,dx$

$\int \frac{e^x}{x}\,dx = \int\left(\frac{1}{x} + 1 + \frac{x}{1\cdot 2} + \frac{x^2}{1\cdot 2\cdot 3} + \cdots\right)dx = \log_e x + x + \frac{x^2}{4} + \frac{x^3}{18} + \cdots$

$\int \log_e x = x(\log_e x - 1)$

$\int (\log_e x)^n dx = x(\log_e x)^n - n\int (\log_e x)^{n-1}\,dx$

$\int x^n \log_e x\,dx = \frac{x^{n+1}}{n+1}\left(\log_e x - \frac{1}{n+1}\right)$

$\int x^{-n}\log_e x\,dx = \frac{x^{-(n-1)}}{-(n-1)}\left(\log x - \frac{1}{-(n-1)}\right)$

$\int x^n e^{ax}\,dx = \frac{x^n e^{ax}}{a}\left\{1 - \frac{n}{ax} + \frac{n(n-1)}{a^2x^2} - \cdots \pm \frac{n!}{a^n x^n}\right\} + C$

Ⓐ

$\int \frac{dx}{\sin^p x} = -\frac{\cos x}{(p-1)\sin^{p-1}x} + \frac{p-2}{p-1}\int \frac{dx}{\sin^{p-2}x}$

$\int \frac{dx}{\cos^p x} = \frac{\sin x}{(p-1)\cos^{p-1}x} + \frac{p-2}{p-1}\int \frac{dx}{\cos^{p-2}x}$

$\int \frac{dx}{a + \cos x} = \frac{2}{\sqrt{a^2-1}}\tan^{-1}\left(\sqrt{\frac{a-1}{a+1}}\tan\frac{x}{2}\right)$ when $a^2 > 1$

$= \frac{2}{\sqrt{1-a^2}}\,\mathrm{Ar}\tan\left(\sqrt{\frac{1-a}{1+a}}\tan\frac{x}{2}\right)$ when $a^2 < 1$

$\int \frac{dx}{a + \sin x} = \frac{2}{\sqrt{a^2-1}}\tan^{-1}\left\{\sqrt{\frac{a+1}{a-1}}\tan\left(45 + \frac{x}{2}\right)\right\}$ when $a^2 > 1$

$= \frac{-2}{\sqrt{1-a^2}}\,\mathrm{Ar}\tan\left\{\sqrt{\frac{1+a}{1-a}}\tan\left(45 + \frac{x}{2}\right)\right\}$ when $a^2 < 1$

$\int \sin^{-1}x\,dx = x\sin^{-1}x + \sqrt{1-x^2}$

$\int \cos^{-1}x\,dx = x\cos^{-1}x - \sqrt{1-x^2}$

$\int \tan^{-1}x\,dx = x\tan^{-1}x - \frac{1}{2}\log_e(1+x^2)$

$\int \cot^{-1}x\,dx = x\cot^{-1}x + \frac{1}{2}\log_e(1+x^2)$

$\int x^m \sin x\,dx = -x^m\cos x + m\int x^{m-1}\cos x\,dx$

$\int x^m \cos x\,dx = x^m \sin x - m\int x^{m-1}\sin x\,dx$

$\int \frac{dx}{\sin x \cos x} = \log_e \tan x + c$

$\int \sin^p x \cos^q x\,dx = \frac{\sin^{p+1}x\cos^{q-1}x}{p+q} + \frac{q-1}{p+q}\int \sin^p x\cos^{q-2}x\,dx$

$= -\frac{\sin^{p-1}x\cos^{q+1}x}{p+q} + \frac{p-1}{p+q}\int \sin^{p-2}x\cos^q x\,dx$

$\int \sin^p x\cos^q x\,dx = \frac{\sin^{-p+1}x\cos^{q+1}x}{p-1} + \frac{p-q-2}{p-1}\times$

$\times \int \sin^{-p+2}x\cos^q x\,dx$

$\int \sin^p x\cos^{-q}x\,dx = \frac{\sin^{p+1}x\cos^{-q+1}x}{q-1} + \frac{q-p-2}{q-1}\times$

$\int \sin^p x\cos^{-q+2}x\,dx$

$\int \frac{dx}{a + b\cos x} = \frac{2}{\sqrt{a^2-b^2}}\tan^{-1}\left(\sqrt{\frac{a-b}{a+b}}\tan\frac{1}{2}x\right) + C$ when $a^2 > b^2$

$= \frac{1}{\sqrt{b^2-a^2}}\log_e\frac{b + a\cos x + \sin x\sqrt{b^2-a^2}}{} + C$

$= \frac{2}{\sqrt{b^2-a^2}}\,\mathrm{Ar}\tan\left(\sqrt{\frac{b-a}{b+a}}\tan\frac{1}{2}x\right) + C$ } when $a^2 < b^2$

$\int \frac{\cos x\,dx}{a + b\cos x} = \frac{x}{b} - \frac{a}{b}\int \frac{dx}{a + b\cos x} + C$

$\int \frac{\sin x\,dx}{a + b\cos x} = -\frac{1}{b}\log_e(a + b\cos x) + C$

$\int \frac{A + B\cos x + C\sin x}{a + b\cos x + c\sin x}\,dx = A\int \frac{d\varphi}{a + p\cos\varphi} + (B\cos\alpha + C\sin\alpha)\int \frac{\cos\varphi\,d\varphi}{a + p\cos\varphi}$

$- (B\sin\alpha - C\cos\alpha)\int \frac{\sin\varphi\,d\varphi}{a + p\cos\varphi}$

wenn man $b = p\cos\alpha$, $c = p\sin\alpha$ und $x - \alpha = \varphi$ setzt.

$\int e^{ax}\sin bx\,dx = \frac{a\sin bx - b\cos bx}{a^2 + b^2}e^{ax} + C$

$\int e^{ax}\cos bx\,dx = \frac{a\cos bx + b\sin bx}{a^2 + b^2}e^{ax} + C$

$\frac{1}{1+x} = 1 - x + x^2 - x^3 + \cdots$, mit $\int_0^x \frac{dx}{1+x} = x - \frac{x^2}{2} + \frac{x^3}{3} - \frac{x^4}{4} + \cdots = \log_e(1+x)$ Konvergenzbereich -1 bis $+1$

$\frac{1}{1+x^2} = 1 - x^2 + x^4 - x^6 + \cdots$, $\int_0^x \frac{dx}{1+x^2} = x - \frac{x^3}{3} + \frac{x^5}{5} - \frac{x^7}{7} + \cdots = \tan^{-1}x$

$\int f(x)\,dx = f(x)$, $\int_a^b f(x)\,dx = f(b) - f(a)$. $\int_a^b = -\int_b^a$, $\int_a^b = \int_a^c + \int_c^b$

$\int_0^\infty \frac{dx}{a + bx^2} = \frac{1}{\sqrt{ab}}(\tan^{-1}\infty - \tan^{-1}0) = \frac{1}{\sqrt{ab}}\cdot\frac{\pi}{2}$

$\int_0^a \frac{dx}{a + bx^2} = \frac{1}{\sqrt{ab}}(\tan^{-1} - \tan^{-1}0) = \frac{1}{\sqrt{ab}}\frac{\pi}{4}$ for $n = \sqrt{\frac{a}{b}}$

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$$\int_0^u \frac{dx}{\sqrt{a-hx^2}} = \frac{1}{\sqrt{h}}(\sin^{-1}1 - \sin^{-1}0) = \frac{1}{\sqrt{h}}\frac{\pi}{2} \quad \text{for } u=\sqrt{\frac{a}{h}}$$

$$\int_0^\infty e^{-x}dx = \left[-e^{-x}\right]_0^\infty = -\left(\frac{1}{e^\infty} - \frac{1}{e^0}\right) = +1$$

$$\int_0^\infty e^{-x^2}dx = \frac{1}{2}\sqrt{\pi}$$

(B)
$$\int \frac{(\alpha+\beta x)dx}{\sqrt{a+2hx+cx^2}} = \frac{\beta}{c}\sqrt{a+2hx+cx^2} + \frac{\alpha c-\beta b}{c}\int \frac{dx}{\sqrt{a+2hx+cx^2}} + C$$

$$\int \frac{x^m dx}{\sqrt{a+2hx+cx^2}} = \frac{x^{m-1}x}{mc} - \frac{(m-1)a}{mc}\int \frac{x^{m-2}dx}{x} - \frac{(2m-1)h}{mc}\int \frac{x^{m-1}dx}{x} \quad \because x=\sqrt{a+2hx+cx^2}$$

$$\int_0^{\sqrt{\frac{a}{h}}} \frac{dx}{a+hx^2} = \int_{\sqrt{\frac{a}{h}}}^\infty \frac{dx}{a+hx^2} = \frac{\pi}{4\sqrt{ah}}$$

$$\int_0^{\sqrt{\frac{a}{h}}} \frac{dx}{\sqrt{a-hx^2}} = \frac{\pi}{2\sqrt{h}}$$

$$\int_0^\infty \frac{\sin hx}{x}dx = \frac{\pi}{2} \quad \text{when } h>0$$

$$\int_0^\infty \frac{\cos hx}{x}dx = \infty$$

$$\left.\begin{aligned}\int_0^{\frac{\pi}{2}} \sin^{2n+1}x\,dx &= \int_0^{\frac{\pi}{2}} \cos^{2n+1}x\,dx = \frac{2\cdot4\cdot6\cdots 2n}{3\cdot5\cdot7\cdots(2n+1)}\\ \int_0^{\frac{\pi}{2}} \sin^{2n}x\,dx &= \int_0^{\frac{\pi}{2}} \cos^{2n}x\,dx = \frac{1\cdot3\cdot5\cdots(2n-1)}{2\cdot4\cdot6\cdots 2n}\frac{\pi}{2}\end{aligned}\right\} \text{wenn } n \text{ eine positive ganze gahl ist.}$$

$$\int_0^\infty x^n e^{-ax}dx = \frac{n!}{a^{n+1}} \quad \text{wenn } a \text{ eine positive, } n \text{ eine positive ganze gahl ist.}$$

$$\int_0^\infty \frac{x^{n-1}}{x+1}dx = \frac{\pi}{\sin n\pi}, \quad \text{wenn } 0<n<1$$

(C)
$$\int \frac{2a\,dx}{a^2-x^2} = \log\left(\frac{a+x}{a-x}\right)+C$$

$$\int \frac{2a\,dx}{x^2-a^2} = \log\left(\frac{x-a}{x+a}\right)+C$$

$$\int \frac{2a\,dx}{x\sqrt{a^2+x^2}} = \log\left(\frac{\sqrt{a^2+x^2}-a}{\sqrt{a^2+x^2}+a}\right)+C$$

$$\int \frac{2a\,dx}{x\sqrt{a^2-x^2}} = \log\left(\frac{a-\sqrt{a^2-x^2}}{a+\sqrt{a^2-x^2}}\right)+C$$

$$\int \frac{x^{-2}dx}{\sqrt{x-x^{-2}}} = -\log\left(\frac{1+\sqrt{1+a^2x^2}}{x}\right)+C$$

(D)
$$\int \frac{dy}{\sqrt{1-y^2}} = \sin^{-1}y + C \qquad \int \frac{r\,dv}{\sqrt{2rv+v^2}} = \text{versin}^{-1}v + C$$

$$\int \frac{-dx}{\sqrt{1-x^2}} = \cos^{-1}x + C \qquad \int \frac{v^2dt}{v^2+t^2} = \tan^{-1}t + C$$

$$\int \frac{dv}{\sqrt{2v-v^2}} = \text{versin}^{-1}v + C \qquad \int \frac{du}{\sqrt{a^2-u^2}} = \sin^{-1}\frac{u}{a} + C$$

$$\int \frac{dt}{1+t^2} = \tan^{-1}t + C \qquad \int \frac{-du}{\sqrt{a^2-u^2}} = \cos^{-1}\frac{u}{a} + C$$

$$\int \frac{r\,dy}{\sqrt{r^2-y^2}} = \sin^{-1}y + C \qquad \int \frac{du}{\sqrt{2au-u^2}} = \text{versin}^{-1}\frac{u}{a} + C$$

$$\int \frac{-r\,dx}{\sqrt{r^2-x^2}} = \cos^{-1}x + C \qquad \int \frac{a\,du}{a^2+u^2} = \tan^{-1}\frac{u}{a} + C$$

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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

Formulas for differentiation

$$\frac{d(ax^n)}{dx} = nax^{n-1}$$

$$\frac{d(a+x)}{dx} = 1 \quad , \quad \frac{d(ax)}{dx} = a$$

$$\frac{d(u+v+w)}{dx} = \frac{du}{dx} + \frac{dv}{dx} - \frac{dw}{dx}$$

$$\frac{d(uv)}{dx} = v\frac{du}{dx} + u\frac{dv}{dx}$$

$$\frac{d(uvw)}{dx} = vw\frac{du}{dx} + uw\frac{dv}{dx} + uv\frac{dw}{dx}$$

$$d(x, y, z, u\cdots) = \left(\frac{dx}{x} + \frac{dy}{y} + \frac{dz}{z} + \frac{du}{u} + \cdots\right) xyzu\cdots$$

$$\frac{d\left(\frac{u}{v}\right)}{dx} = \frac{v\frac{du}{dx} - u\frac{dv}{dx}}{v^2}$$

$$\frac{d\sqrt{x}}{dx} = \frac{1}{2\sqrt{x}} \quad , \quad \frac{d\frac{1}{x}}{dx} = -\frac{1}{x^2}$$

$$\frac{d(a^x)}{dx} = a^x \log_e a$$

$$\frac{d(e^x)}{dx} = e^x$$

$$\frac{d(e^u)}{dx} = e^u\frac{du}{dx} \cdots u = f(x)$$

$$\frac{d(e^{ax})}{dx} = ae^{ax}$$

$$\frac{d(\log_a x)}{dx} = \frac{1}{x\log_e a}$$

$$\frac{d(\log_e x)}{dx} = \frac{1}{x}$$

$$\frac{d(\log_e u)}{dx} = \frac{1}{u}\frac{du}{dx} \cdots u = f(x)$$

$$\frac{d\{\log_e(a+bx)\}}{dx} = \frac{b}{a+bx}$$

$$\frac{d\left(\log\frac{e^x}{1+e^x}\right)}{dx} = \frac{1}{1+e^x}$$

$$\frac{d(\sin x)}{dx} = \cos x$$

$$\frac{d\frac{1}{\sin x}}{dx} = -\frac{\cos x}{\sin^2 x}$$

$$\frac{d(\cos x)}{dx} = -\sin x$$

$$\frac{d\frac{1}{\cos x}}{dx} = \frac{\sin x}{\cos^2 x}$$

$$\frac{d(\tan x)}{dx} = \frac{1}{\cos^2 x}$$

$$\frac{d(\cot x)}{dx} = -\frac{1}{\sin^2 x}$$

$$\frac{d(\sec x)}{dx} = \tan x \sec x$$

$$\frac{d(\operatorname{cosec} x)}{dx} = -\cot x \operatorname{cosec} x$$

$$\frac{d(\sin^{-1}x)}{dx} = \pm\frac{1}{\sqrt{1-x^2}}$$ (+ プラスマイナス)

$$\frac{d(\cos^{-1}x)}{dx} = -\frac{1}{\sqrt{1-x^2}}$$

$$\frac{d(\tan^{-1}x)}{dx} = \frac{1}{1+x^2}$$

$$\frac{d(\cot^{-1}x)}{dx} = -\frac{1}{1+x^2}$$

$$\frac{d(\sec^{-1}x)}{dx} = \frac{1}{x\sqrt{x^2-1}}$$

$$\frac{d(\operatorname{cosec}^{-1}x)}{dx} = -\frac{1}{x\sqrt{x^2-1}}$$

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

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$$\frac{d\log_e \sin x}{dx} = \cot x$$

$$\frac{d\log_e \cos x}{dx} = -\tan x$$

$$\frac{d\log \tan x}{dx} = \frac{2}{\sin 2x}$$

$$\frac{d\log_e \cot x}{dx} = -\frac{2}{\sin 2x}$$

$$dx^y = x^{y-1}(x\log_e x\,dy + y\,dx)$$

$$\begin{cases} d\sin x = \cos x\,dx \\ d\tan x = \dfrac{dx}{\cos^2 x} \end{cases}$$

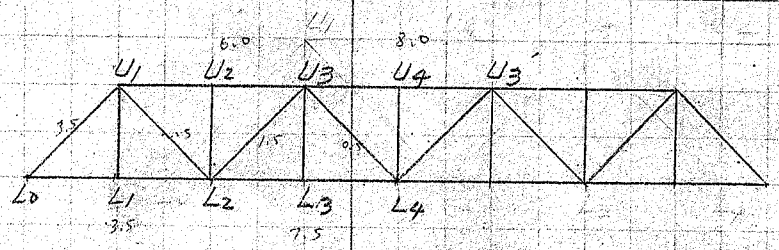
$$\begin{cases} d\cos x = \sin x\,dx \\ d\cot x = -\dfrac{dx}{\sin^2 x} \end{cases}$$

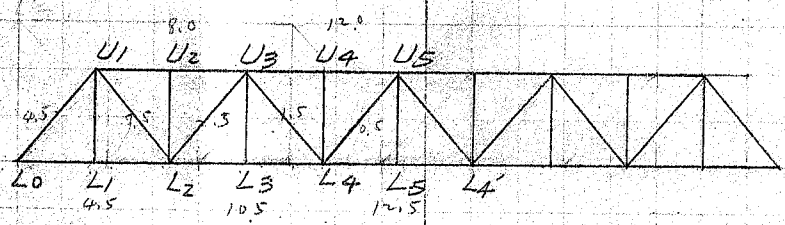
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CALCULATIONS FOR

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Span 8 @ 3.75 = 30.0 height = 3.5 sec = 1.466 tan. = 1.071

Members	Length	D.L. stress	Live Load stress Unif.L.stress	Single Con.
U1 - U2	3.750	6.426 C	6.426 C	1.607 C
U2 - U3	〃	6.426 C	6.426 C	1.607 C
U3 - U4	〃	8.568 C	8.568 C	2.142 C
L0 - L1	〃	3.749 T	3.749 T	.937 T
L1 - L2	〃	3.749 T	3.749 T	.937 T
L2 - L3	〃	8.033 T	8.033 T	2.008 T
L3 - L4	〃	8.033 T	8.033 T	2.008 T
L0 - U1	5.130	5.131 C	5.131 C	1.283 C
U1 - L2	〃	3.665 T	3.848 T	1.100 T
L2 - U3	〃	2.199 C	2.749 C	.916 C
U3 - L4	〃	.733 T	1.833 T	.733 T
L4 - U3'	〃	.733 T	1.100 C	.550 C
U1 - L1	3.500	1.000 T	1.000 T	1.000 T
U3 - L3	3.500	1.000 T	1.000 T	1.000 T

Span 10 @ 3.00 = 30.0 height = 3.5 sec = 1.317 tan. = .857

Members	Length	D.L stress	Live Load stress Unif.Lstress	Single Con.
U1-U2	3.000	6.856 C	6.856 C	1.371 C
U2-U3	3.000	6.856 C	6.856 C	1.371 C
U3-U4	3.000	10.284 C	10.284 C	2.057 C
U4-U5	3.000	10.284 C	10.284 C	2.057 C
L0-L1	3.000	3.857 T	3.857 T	.771 T
L1-L2	3.000	3.857 T	3.857 T	.771 T
L2-L3	3.000	8.999 T	8.999 T	1.800 T
L3-L4	3.000	8.999 T	8.999 T	1.800 T
L4-L5	3.000	10.713 T	10.713 T	2.143 T
L0-U1	4.610	5.927 C	5.927 C	1.185 C
U1-L2	4.610	4.610 T	4.741 T	1.054 T
L2-U3	4.610	3.293 C	3.688 C	.922 C
U3-L4	4.610	1.976 T	2.766 T	.790 T
L4-U5	4.610	.659 C	1.976 C	.659 C
U5-L4'	4.610	.659 C	1.317 T	.527 T
U1-L1	3.500	1.000 T	1.000 T	1.000 T
U3-L3	3.500	1.000 T	1.000 T	1.000 T
U5-L5	3.500	1.000 T	1.000 T	1.000 T

Span 8 @ 4.00 = 32.0 height = 3.9 sec = 1.433 Tan = 1.026

Members	Length	D.L stress	Live Load stress Unif.Lstress	Single Con.
U1-U2	4.000	6.156 C	6.156 C	1.539 C
U2-U3	4.000	6.156 C	6.156 C	1.539 C
U3-U4	4.000	8.208 C	8.208 C	2.052 C
L0-L1	4.000	3.591 T	3.591 T	.898 T
L1-L2	4.000	3.591 T	3.591 T	.898 T
L2-L3	4.000	7.695 T	7.695 T	1.924 T
L3-L4	4.000	7.695 T	7.695 T	1.924 T
L0-U1	5.587	5.016 C	5.016 C	1.254 C
U1-L2	5.587	3.583 T	3.762 T	1.075 T
L2-U3	5.587	2.150 C	2.687 C	.896 C
U3-L4	5.587	.717 T	1.791 T	.717 T
L4-U3'	5.587	.717 T	1.075 C	.537 C
U1-L1	3.900	1.000 T	1.000 T	1.000 T
U3-L3	3.900	1.000 T	1.000 T	1.000 T

Span 10 @ 3.20 = 32.0 height = 3.9 sec = 1.284 Tan = .821

Members	Length	D.L stress	Live Load stress Unif.L.st.	Single Con.
U1-U2	3.20	6.568 C	6.568 C	1.314 C
U2-U3	〃	6.568 C	6.568 C	1.314 C
U3-U4	〃	9.852 C	9.852 C	1.970 C
U4-U5	〃	9.852 C	9.852 C	1.970 C
L0-L1	〃	3.695 T	3.695 T	.739 T
L1-L2	〃	3.695 T	3.695 T	.739 T
L2-L3	〃	8.621 T	8.621 T	1.724 T
L3-L4	〃	8.621 T	8.621 T	1.724 T
L4-L5	〃	10.263 T	10.263 T	2.053 T
L0-U1	5.045	5.778 C	5.778 C	1.156 C
U1-L2	〃	4.494 T	4.622 T	1.027 T
L2-U3	〃	3.210 C	3.595 C	.899 C
U3-L4	〃	1.926 T	2.696 T	.770 T
L4-U5	〃	.642 C	1.926 C	.642 C
U5-L4'	〃	.642 C	1.284 T	.514 T
U1-L1	3.9	1.000 T	1.000 T	1.000 T
U3-L3	〃	1.000 T	1.000 T	1.000 T
U5-L5	〃	1.000 T	1.000 T	1.000 T

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CALCULATIONS FOR

Span 8 @ 4.25 = 34.0 height = 4.00 Sec. = 1.459 Tan = 1.063

Members	Length	D.L. stress	Live Load Stress Unif. L. stress	Live Load Stress single con.
U1-U2	4250	6.378 C	6.378 C	1.595 C
U2-U3	〃	6.378 C	6.378 C	1.595 C
U3-U4	〃	8.504 C	8.504 C	2.126 C
L0-L1	〃	3.721 T	3.721 T	.930 T
L1-L2	〃	3.721 T	3.721 T	.930 T
L3-L3	〃	7.973 T	7.973 T	1.993 T
L3-L4	〃	7.973 T	7.973 T	1.993 T
L0-U1	5.836	5.107 C	5.107 C	1.277 C
U1-L2	〃	3.648 T	3.830 T	1.094 T
L2-U3	〃	2.189 C	2.736 C	.912 C
U3-L4	〃	.730 T	1.824 T	.730 T
L4-U3	〃	.730 T	1.094 C	.547 C
U1-L1	4.000	1.000 T	1.000 T	1.000 T
U3-L3	4.000	1.000 T	1.000 T	1.000 T

Span 10 @ 3.40 = 34.0 height = 4.00 Sec. = 1.313 Tan = .850

Members	Length	D.L. stress	Live Load stress Unif. L. stress	Live Load stress single Con.
U1-U2	3.400	6.800 C	6.800 C	1.360 C
U2-U3	〃	6.800 C	6.800 C	1.360 C
U3-U4	〃	10.200 C	10.200 C	2.040 C
U4-U5	〃	10.200 C	10.200 C	2.040 C
L0-L1	〃	3.825 T	3.825 T	.765 T
L1-L2	〃	3.825 T	3.825 T	.765 T
L2-L3	〃	8.925 T	8.925 T	1.785 T
L3-L4	〃	8.925 T	8.925 T	1.785 T
L4-L5	〃	10.625 T	10.625 T	2.125 T
L0-U1	5.250	5.909 C	5.909 C	1.182 C
U1-L2	〃	4.596 T	4.727 T	1.050 T
L2-U3	〃	3.283 C	3.676 C	.919 C
U3-L4	〃	1.970 T	2.757 T	.788 T
L4-U5	〃	.657 C	1.970 C	.657 C
U5-L4	〃	.657 C	1.313 T	.525 T
U1-L1	4.000	1.000 T	1.000 T	1.000 T
U3-L3	〃	1.000 T	1.000 T	1.000 T
U5-L5	〃	1.000 T	1.000 T	1.000 T

Span 8 @ 4.50 = 36.0 height = 4.35 sec = 1.439 Tan = 1.034

Members	Length	D.L. stress	Live Load stress Unif. L. stress	Live Load stress single con.
U1-U2	4.500	6.204 C	6.204 C	1.551 C
U2-U3	〃	6.204 C	6.204 C	1.551 C
U3-U4	〃	8.272 C	8.272 C	2.068 C
L0-L1	〃	3.619 T	3.619 T	.905 T
L1-L2	〃	3.619 T	3.619 T	.905 T
L2-L3	〃	7.755 T	7.755 T	1.939 T
L3-L4	〃	7.755 T	7.755 T	1.939 T
L0-U1	6.259	5.037 C	5.037 C	1.259 C
U1-L2	〃	3.598 T	3.777 T	1.079 T
L2-U3	〃	2.159 C	2.698 C	.899 C
U3-L4	〃	.720 T	1.799 T	.720 T
L4-U3	〃	.720 T	1.079 C	.540 C
U1-L1	4.35	1.000 T	1.000 T	1.000 T
U3-L3	〃	1.000 T	1.000 T	1.000 T

Span 10 @ 3.60 = 36.0 height = 4.35 sec = 1.298 Tan = .829

Members	Length	D.L. stress	Live Load stress Unif. L. stress	Live Load stress single Con.
U1-U2	3.600	6.624 C	6.624 C	1.325 C
U2-U3	〃	6.624 C	6.624 C	1.325 C
U3-U4	〃	9.936 C	9.936 C	1.987 C
U4-U5	〃	9.936 C	9.936 C	1.987 C
L0-L1	〃	3.726 T	3.726 T	.745 T
L1-L2	〃	3.726 T	3.726 T	.745 T
L2-L3	〃	8.694 T	8.694 T	1.739 T
L3-L4	〃	8.694 T	8.694 T	1.739 T
L4-L5	〃	10.350 T	10.350 T	2.070 T
L0-U1	5.646	5.841 C	5.841 C	1.168 C
U1-L2	〃	4.543 T	4.673 T	1.038 T
L2-U3	〃	3.245 C	3.634 C	.909 C
U3-L4	〃	1.947 T	2.726 T	.779 T
L4-U5	〃	.649 C	1.947 C	.649 C
U5-L4	〃	.649 C	1.298 T	.519 T
U1-L1	4.35	1.000 T	1.000 T	1.000 T
U3-L3	〃	1.000 T	1.000 T	1.000 T
U5-L5	〃	1.000 T	1.000 T	1.000 T

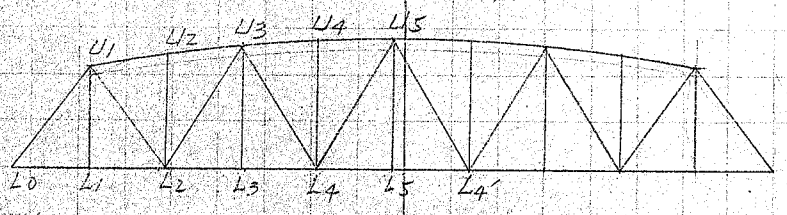
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JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

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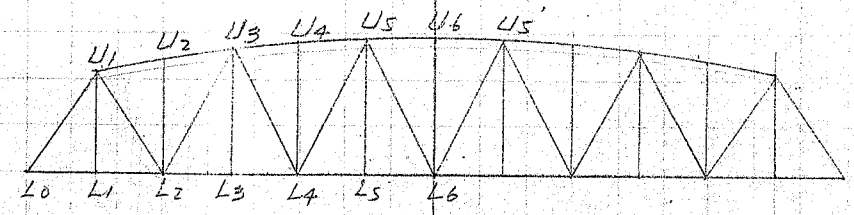
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Span 10 @ 5.0 = 50.0 height at center = 9.30

Members	Length	D.L. stress	Live Load stress Unif. L. stress	Single Con.
U1-U2	5130	5229 C	5229 C	1046 C
U2-U3	5065	5163 C	5163 C	1032 C
U3-U4	5023	6595 C	6595 C	1319 C
U4-U5	5003	6568 C	6568 C	1314 C
L0-L1	5000	3358 T	3358 T	672 T
L1-L2	〃	3358 T	3358 T	672 T
L2-L3	〃	6064 T	6064 T	1213 T
L3-L4	〃	6064 T	6064 T	1213 T
L4-L5	〃	6720 T	6720 T	1344 T
L0-U1	8360	5615 C	5615 C	1123 C
U1-L2	〃	2907 T	3178 T	706 T
L2-U3	9998	1933 C	2566 C	641 C
U3-L4	〃	1001 T	1913 T	547 T
L4-U5	10559	329 C	1584 C	528 C
U5-L4'	〃	329 C	1255 T	502 T
U2-L2	7848	344 T	344 T	069 T
U4-L4	9140	423 T	423 T	085 T
U1-L1	6700	1000 T	1000 T	1000 T
U3-L3	8658	1000 T	1000 T	1000 T
U5-L5	9300	1000 T	1000 T	1000 T

reduced top panel load for dead load



Span 12 @ 4.167 = 50.0 height at center = 9.30

Members	Length	D.L. stress	Live Load stress Unif. L. stress	Single Con.
U1-U2	4273	5590 C	5590 C	932 C
U2-U3	4230	5533 C	5533 C	922 C
U3-U4	4199	7558 C	7558 C	1260 C
U4-U5	4178	7520 C	7520 C	1253 C
U5-U6	4168	8067 C	8067 C	1345 C
L0-L1	4167	3421 T	3421 T	570 T
L1-L2	〃	〃 T	〃 T	〃 T
L2-L3	〃	6719 T	6719 T	1120 T
L3-L4	〃	〃 T	〃 T	〃 T
L4-L5	〃	7928 T	7928 T	1321 T
L5-L6	〃	〃 T	〃 T	〃 T
L6-U5	7890	6477 C	6477 C	1079 C
U1-L2	〃	3844 T	4063 T	739 T
L2-U3	9353	2846 C	3391 C	678 C
U3-L4	〃	1895 T	2573 T	572 T
L4-U5	10098	1036 C	2205 C	551 C
U5-L6	〃	332 T	1795 T	513 T
L6-U5'	〃	332 T	1463 C	488 C
U2-L2	7645	284 T	284 T	047 T
U4-L4	8889	373 T	373 T	062 T
U6-L6	9300	394 T	394 T	066 T
U1-L1	6700	1000 T	1000 T	1000 T
U3-L3	8373	1000 T	1000 T	1000 T
U5-L5	9198	1000 T	1000 T	1000 T

reduced top panel load

Span 10 @ 5.2 = 52.0 height at center = 9.50

Members	length	D.L. stress	Live load stress Unif. L. stress	Single conc.
U1-U2	5335	5340 C	5340 C	1068 C
U2-U3	5268	5273 C	5273 C	1055 C
U3-U4	5224	6716 C	6716 C	1343 C
U4-U5	5203	6689 C	6689 C	1338 C
L0-L1	5200	3441 T	3441 T	688 T
L1-L2	〃	3441 T	3441 T	688 T
L2-L3	〃	6181 T	6181 T	1236 T
L3-L4	〃	6181 T	6181 T	1236 T
L4-L5	〃	6842 T	6842 T	1368 T
L0-U1	8560	5665 C	5665 C	1133 C
U1-L2	〃	2904 T	3179 T	707 T
L2-U3	10250	1924 C	2565 C	641 C
U3-L4	〃	992 T	1913 T	547 T
L4-U5	10830	327 C	1588 C	529 C
U5-L4'	〃	327 C	1262 T	505 T
U2-L2	7992	351 T	351 T	071 T
U4-L4	9334	431 T	431 T	086 T
U1-L1	6800	1000 T	1000 T	1000 T
U3-L3	8833	1000 T	1000 T	1000 T
U5-L5	9500	1000 T	1000 T	1000 T

Span 12 @ 4.333 = 52.0 height at center = 9.50

Members	length	D.L. stress	Live Load stress Unif. L. stress	Single Conc.
U1-U2	4443	5710 C	5710 C	952 C
U2-U3	4398	5651 C	5651 C	942 C
U3-U4	4366	7703 C	7703 C	1283 C
U4-U5	4345	7665 C	7665 C	1277 C
U5-U6	4334	8213 C	8213 C	1369 C
L0-L1	4333	3505 T	3505 T	584 T
L1-L2	〃	〃 T	〃 T	〃 T
L2-L3	〃	6851 T	6851 T	1142 T
L3-L4	〃	〃 T	〃 T	〃 T
L4-L5	〃	8072 T	8072 T	1345 T
L5-L6	〃	〃 T	〃 T	〃 T
L0-U1	8063	6522 C	6522 C	1087 C
U1-L2	〃	3840 T	4063 T	739 T
L2-U3	9575	2836 C	3388 C	678 C
U3-L4	〃	1744 T	2569 T	571 T
L4-U5	10345	1030 C	2207 C	552 C
U5-L6	〃	329 T	1798 T	514 T
L6-U5'	〃	329 T	1469 C	490 C
U2-L2	7782	291 T	291 T	048 T
U4-L4	9074	381 T	381 T	064 T
U6-L6	9500	402 T	402 T	066 T
U1-L1	6800	1000 T	1000 T	1000 T
U3-L3	8538	1000 T	1000 T	1000 T
U5-L5	9394	1000 T	1000 T	1000 T

reduced top panel load

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CALCULATIONS FOR

Span 10 c 5.4 = 54.0 height at center = 9.75

Members	Length	D.L. stress	Live Load stress Unif. L. stress	Live Load stress Single conc
U_1-U_2	5.540	5.414 C	5.414 C	1.083 C
U_2-U_3	5.470	5.345 C	5.345 C	1.069 C
U_3-U_4	5.425	6.799 C	6.799 C	1.359 C
U_4-U_5	5.403	6.769 C	6.769 C	1.354 C
L_0-L_1	5.400	3.496 T	3.496 T	.699 T
L_1-L_2	〃	3.496 T	3.496 T	.699 T
L_2-L_3	〃	6.259 T	6.259 T	1.256 T
L_3-L_4	〃	6.259 T	6.259 T	1.256 T
L_4-L_5	〃	6.923 T	6.923 T	1.385 T
L_0-U_1	8.801	5.699 C	5.699 C	1.140 C
U_1-L_2	〃	2.902 T	3.181 T	.707 T
L_2-U_3	10.546	1.918 C	2.565 C	.641 C
U_3-L_4	〃	.989 T	1.915 T	.547 T
L_4-U_5	11.146	.325 C	1.592 C	.531 C
U_5-L_4'	〃	.325 C	1.266 T	.507 T
U_2-L_2	8.187	.357 T	.357 T	.072 T
U_4-L_4	9.578	.435 T	.435 T	.087 T
U_1-L_1	6.950	1.000 T	1.000 T	1.000 T
U_3-L_3	9.059	1.000 T	1.000 T	1.000 T
U_5-L_5	9.750	1.000 T	1.000 T	1.000 T

Span 12 c 4.50 = 54.00 height at center = 9.75

Members	Length	D.L. stress	Live Load stress Unif. L. stress	Live Load stress Single conc
U_1-U_2	4.610	5.799 C	5.799 C	.966 C
U_2-U_3	4.571	5.749 C	5.749 C	.958 C
U_3-U_4	4.534	7.794 C	7.794 C	1.299 C
U_4-U_5	4.512	7.756 C	7.756 C	1.293 C
U_5-U_6	4.501	8.309 C	8.309 C	1.385 C
L_0-L_1	4.500	3.561 T	3.561 T	.594 T
L_1-L_2	〃	〃 T	〃 T	〃 T
L_2-L_3	〃	6.941 T	6.941 T	1.157 T
L_3-L_4	〃	〃 T	〃 T	〃 T
L_4-L_5	〃	8.170 T	8.170 T	1.362 T
L_5-L_6	〃	〃 T	〃 T	〃 T
L_0-U_1	8.280	6.553 C	6.553 C	1.092 C
U_1-L_2	〃	3.863 T	4.087 T	.743 T
L_2-U_3	9.841	3.053 C	3.366 C	.673 C
U_3-L_4	〃	1.736 T	2.567 T	.571 T
L_4-U_5	10.638	1.028 C	2.209 C	.552 C
U_5-L_6	〃	.326 T	1.799 T	.514 T
L_1-U_5'	〃	〃 T	1.474 C	.491 C
U_2-L_2	7.950	.249 T	.249 T	.042 T
U_4-L_4	9.308	.387 T	.387 T	.064 T
U_6-L_6	9.750	.410 T	.410 T	.068 T
U_1-L_1	6.950	1.000 T	1.000 T	1.000 T
U_3-L_3	8.752	1.000 T	1.000 T	1.000 T
U_5-L_5	9.639	1.000 T	1.000 T	1.000 T

Span 10 c 5.6 = 56.0 height at center 10.00

Members	Length	D.L. stress	Live load stress Unif. L. stress	Live load stress Single Conc.
U_1-U_2	5.745	5.484 C	5.484 C	1.097 C
U_2-U_3	5.672	5.414 C	5.414 C	1.083 C
U_3-U_4	5.626	6.874 C	6.874 C	1.375 C
U_4-U_5	5.603	6.846 C	6.846 C	1.369 C
L_0-L_1	5.600	3.549 T	3.549 T	.710 T
L_1-L_2	〃	〃 T	〃 T	〃 T
L_2-L_3	〃	6.333 T	6.333 T	1.267 T
L_3-L_4	〃	〃	〃 T	〃
L_4-L_5	〃	7.000 T	7.000 T	1.400 T
L_0-U_1	9.043	5.731 C	5.731 C	1.146 C
U_1-L_2	〃	2.900 T	3.184 T	.707 T
L_2-U_3	10.842	1.913 C	2.565 C	.641 C
U_3-L_4	〃	.985 T	1.916 T	.547 T
L_4-U_5	11.461	.322 C	1.594 C	.531 C
U_5-L_4'	〃	.322 C	1.271 T	.509 T
U_2-L_2	8.381	.361 T	.361 T	.073 T
U_4-L_4	9.821	.437 T	.437 T	.087 T
U_1-L_1	7.100	1.000 T	1.000 T	1.000 T
U_3-L_3	9.284	1.000 T	1.000 T	1.000 T
U_5-L_5	10.000	1.000 T	1.000 T	1.000 T

Span 12 c 4.666 = 56.00 height at center = 10.00

Members	Length	D.L. stress	Live load stress Unif. L. stress	Live load stress Single Conc.
U_1-U_2	4.784	5.867 C	5.867 C	.978 C
U_2-U_3	4.736	5.808 C	5.808 C	.968 C
U_3-U_4	4.701	7.883 C	7.883 C	1.314 C
U_4-U_5	4.681	7.849 C	7.849 C	1.308 C
U_5-U_6	4.667	8.400 C	8.400 C	1.400 C
L_0-L_1	4.666	3.830 T	3.830 T	.602 T
L_1-L_2	〃	〃 T	〃 T	〃 T
L_2-L_3	〃	7.523 T	7.523 T	1.176 T
L_3-L_4	〃	〃 T	〃 T	〃 T
L_4-L_5	〃	8.877 T	8.877 T	1.377 T
L_5-L_6	〃	〃 T	〃 T	〃 T
L_0-U_1	8.496	6.581 C	6.581 C	1.097 C
U_1-L_2	〃	3.838 T	4.067 T	.739 T
L_2-U_3	10.107	2.823 C	3.385 C	.677 C
U_3-L_4	〃	1.729 T	2.565 T	.570 T
L_4-U_5	10.932	1.021 C	2.208 C	.552 C
U_5-L_6	〃	.326 T	1.803 T	.515 T
L_6-U_5'	〃	.326 T	1.477 C	.492 C
U_2-L_2	8.154	.297 T	.297 T	.049 T
U_4-L_4	9.542	.387 T	.387 T	.065 T
U_6-L_6	10.000	.410 T	.410 T	.068 T
U_1-L_1	7.100	1.000 T	1.000 T	1.000 T
U_3-L_3	8.966	1.000 T	1.000 T	1.000 T
U_5-L_5	9.886	1.000 T	1.000 T	1.000 T

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JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

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Span 10 @ 5.8 = 58.0 Height at center = 10.25

Members	Length	D.L. stress	Live load stress Unif. L. stress	Live load stress Single Conc.
U_1-U_2	5.954	5.573 C	5.573 C	1.115 C
U_2-U_3	5.877	5.501 C	5.501 C	1.100 C
U_3-U_4	5.827	6.950 C	6.950 C	1.390 C
U_4-U_5	5.803	6.921 C	6.921 C	1.384 C
L_0-L_1	5.800	3.625 T	3.625 T	.725 T
L_1-L_2	〃	〃 T	〃 T	〃 T
L_2-L_3	〃	6.360 T	6.360 T	1.283 T
L_3-L_4	〃	〃 T	〃 T	〃 T
L_4-L_5	〃	7.073 T	7.073 T	1.415 T
L_0-U_1	9.246	5.779 C	5.779 C	1.156 C
U_1-L_2	〃	2.879 T	3.169 T	.704 T
L_2-U_3	11.128	1.887 C	2.552 C	.638 C
U_3-L_4	〃	.969 T	1.909 T	.546 T
L_4-U_5	11.777	.317 C	1.595 C	.532 C
U_5-L_4'	〃	.317 C	1.278 T	.511 T
U_2-L_2	8.547	.372 T	.372 T	.074 T
U_4-L_4	10.062	.450 T	.450 T	.090 T
U_1-L_1	7.200	1.000 T	1.000 T	1.000 T
U_3-L_3	9.497	1.000 T	1.000 T	1.000 T
U_5-L_5	10.250	1.000 T	1.000 T	1.000 T

Span 12 @ 4.833 = 58.00 Height at center = 10.25

Members	Length	D.L. stress	Live Load stress Unif. L. stress	Live Load stress Single Conc.
U_1-U_2	4.959	5.968 C	5.968 C	.995 C
U_2-U_3	4.908	5.907 C	5.907 C	.984 C
U_3-U_4	4.871	7.979 C	7.979 C	1.330 C
U_4-U_5	4.847	7.939 C	7.939 C	1.323 C
U_5-U_6	4.834	8.489 C	8.489 C	1.415 C
L_0-L_1	4.833	3.692 T	3.692 T	.615 T
L_1-L_2	〃	〃 T	〃 T	〃 T
L_2-L_3	〃	7.121 T	7.121 T	1.187 T
L_3-L_4	〃	〃 T	〃 T	〃 T
L_4-L_5	〃	8.349 T	8.349 T	1.392 T
L_5-L_6	〃	〃 T	〃 T	〃 T
L_0-U_1	8.672	6.629 C	6.624 C	1.104 C
U_1-L_2	〃	3.812 T	4.047 T	.736 T
L_2-U_3	10.359	2.795 C	3.368 C	.674 C
U_3-L_4	〃	1.706 T	2.551 T	.567 T
L_4-U_5	11.224	1.005 C	2.202 C	.550 C
U_5-L_6	〃	.321 T	1.803 T	.515 T
L_6-U_5'	〃	.321 T	1.482 C	.494 C
U_6-L_6	8.309	.307 T	.307 T	.052 T
U_4-L_4	9.768	.398 T	.398 T	.066 T
U_6-L_6	10.250	.422 T	.422 T	.070 T
U_1-L_1	7.200	1.000 T	1.000 T	1.000 T
U_3-L_3	9.163	1.000 T	1.000 T	1.000 T
U_5-L_5	10.130	1.000 T	1.000 T	1.000 T

Span 10 @ 6.0 = 60.0 Height at center = 10.50

Members	Length	D.L. stress	Live load stress Unif. L. stress	Live load stress Single Conc.
U_1-U_2	6.159	5.636 C	5.636 C	1.127 C
U_2-U_3	6.080	5.564 C	5.564 C	1.113 C
U_3-U_4	6.028	7.019 C	7.019 C	1.404 C
U_4-U_5	6.003	6.990 C	6.990 C	1.398 C
L_0-L_1	6.000	3.673 T	3.673 T	.735 T
L_1-L_2	〃	3.673 T	3.673 T	.735 T
L_2-L_3	〃	6.479 T	6.479 T	1.296 T
L_3-L_4	〃	6.479 T	6.479 T	1.296 T
L_4-L_5	〃	7.143 T	7.143 T	1.429 T
L_0-U_1	9.488	5.809 C	5.809 C	1.162 C
U_1-L_2	〃	2.873 T	3.166 T	.704 T
L_2-U_3	11.425	1.883 C	2.552 C	.638 C
U_3-L_4	〃	.965 T	1.909 T	.545 T
L_4-U_5	12.093	.313 C	1.582 C	.527 C
U_5-L_4'	〃	.313 C	1.270 T	.508 T
U_2-L_2	8.742	.376 T	.376 T	.075 T
U_4-L_4	10.306	.453 T	.453 T	.091 T
U_1-L_1	7.350	1.000 T	1.000 T	1.000 T
U_3-L_3	9.723	1.000 T	1.000 T	1.000 T
U_5-L_5	10.500	1.000 T	1.000 T	1.000 T

Span 12 @ 5.00 = 60.000 Height at center = 10.50

Members	Length	D.L. stress	Live Load stress Unif. L. stress	Live Load stress Single Conc.
U_1-U_2	5.130	6.038 C	6.038 C	1.006 C
U_2-U_3	5.077	5.976 C	5.976 C	.996 C
U_3-U_4	5.039	8.060 C	8.060 C	1.343 C
U_4-U_5	5.015	8.022 C	8.022 C	1.337 C
U_5-U_6	5.002	8.575 C	8.575 C	1.429 C
L_0-L_1	5.000	3.741 T	3.741 T	.624 T
L_1-L_2	〃	〃 T	〃 T	〃 T
L_2-L_3	〃	7.200 T	7.200 T	1.200 T
L_3-L_4	〃	〃 T	〃 T	〃 T
L_4-L_5	〃	8.433 T	8.433 T	1.406 T
L_5-L_6	〃	〃 T	〃 T	〃 T
L_0-U_1	8.889	6.652 C	6.652 C	1.109 C
U_1-L_2	〃	3.811 T	4.048 T	.736 T
L_2-U_3	10.625	2.794 C	3.371 C	.674 C
U_3-L_4	〃	1.695 T	2.546 T	.566 T
L_4-U_5	11.518	1.003 C	2.204 C	.511 C
U_5-L_6	〃	.319 T	1.805 T	.516 T
L_6-U_5'	〃	.319 T	1.486 C	.495 C
U_2-L_2	8.496	.314 T	.314 T	.053 T
U_4-L_4	10.003	.408 T	.408 T	.068 T
U_6-L_6	10.500	.426 T	.426 T	.070 T
U_1-L_1	7.350	1.000 T	1.000 T	1.000 T
U_3-L_3	9.375	1.000 T	1.000 T	1.000 T
U_5-L_5	10.376	1.000 T	1.000 T	1.000 T

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
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CALCULATIONS FOR

MADE BY M.Jmda DATE 6-2-27 FILE NO 6
CHECKED BY ____ DATE ____ PAGE NO ____

Span 10 @ 62 = 620 height at center = 10.75

Members	Length	D.L. stress	Live Load stress unif L. stress	Live Load stress Single conc.
U_1-U_2	6369	5720 C	5720 C	1144 C
U_2-U_3	6285	5644 C	5644 C	1129 C
U_3-U_4	6230	7088 C	7088 C	1418 C
U_4-U_5	6204	7059 C	7059 C	1412 C
L_0-L_1	6200	3745 T	3745 T	749 T
L_1-L_2	〃	〃 T	〃 T	〃 T
L_2-L_3	〃	6552 T	6552 T	1310 T
L_3-L_4	〃	〃 T	〃 T	〃 T
L_4-L_5	〃	7209 T	7209 T	1442 T
L_0-U_1	9692	5854 C	5854 C	1171 C
U_1-L_2	〃	2850 T	3150 T	700 T
L_2-U_3	11712	1859 C	2539 C	635 C
U_3-L_4	〃	949 T	1902 T	543 T
L_4-U_5	12410	297 C	1592 C	531 C
U_5-L_4'	〃	297 C	1294 T	518 T
U_2-L_2	8908	386 T	386 T	077 T
U_4-L_4	10547	452 T	452 T	091 T
U_1-L_1	7450	1000 T	1000 T	1000 T
U_3-L_3	9936	1000 T	1000 T	1000 T
U_5-L_5	10750	1000 T	1000 T	1000 T

Span 12 @ 5167 = 6200 height at center = 10.75

Members	Length	D.L. stress	Live Load stress unif L. stress	Live Load stress Single conc.
U_1-U_2	5305	6133 C	6133 C	1022 C
U_2-U_3	5249	6068 C	6068 C	1011 C
U_3-U_4	5208	8147 C	8147 C	1358 C
U_4-U_5	5182	8106 C	8106 C	1351 C
U_5-U_6	5169	8655 C	8655 C	1442 C
L_0-L_1	5167	3815 T	3815 T	636 T
L_1-L_2	〃	〃 T	〃 T	636 T
L_2-L_3	〃	7286 T	7286 T	1214 T
L_3-L_4	〃	〃 T	〃 T	1214 T
L_4-L_5	〃	8514 T	8514 T	1419 T
L_5-L_6	〃	〃 T	〃 T	1419 T
L_0-U_1	9066	6693 C	6693 C	1116 C
U_1-L_2	9066	3788 T	4030 T	733 T
L_2-U_3	10879	2763 C	3351 C	670 C
U_3-L_4	10879	1677 T	2536 T	564 T
L_4-U_5	11810	988 C	2198 C	549 C
U_5-L_6	11810	314 T	1805 T	516 T
L_6-U_5'	11810	314 T	1491 C	497 C
U_2-L_2	8650	319 T	319 T	053 T
U_4-L_4	10229	413 T	413 T	069 T
U_6-L_6	10750	436 T	436 T	072 T
U_1-L_1	7450	1000 T	1000 T	1000 T
U_3-L_3	9574	1000 T	1000 T	1000 T
U_5-L_5	10620	1000 T	1000 T	1000 T

Span 10 @ 64 = 640 height at center = 11.000

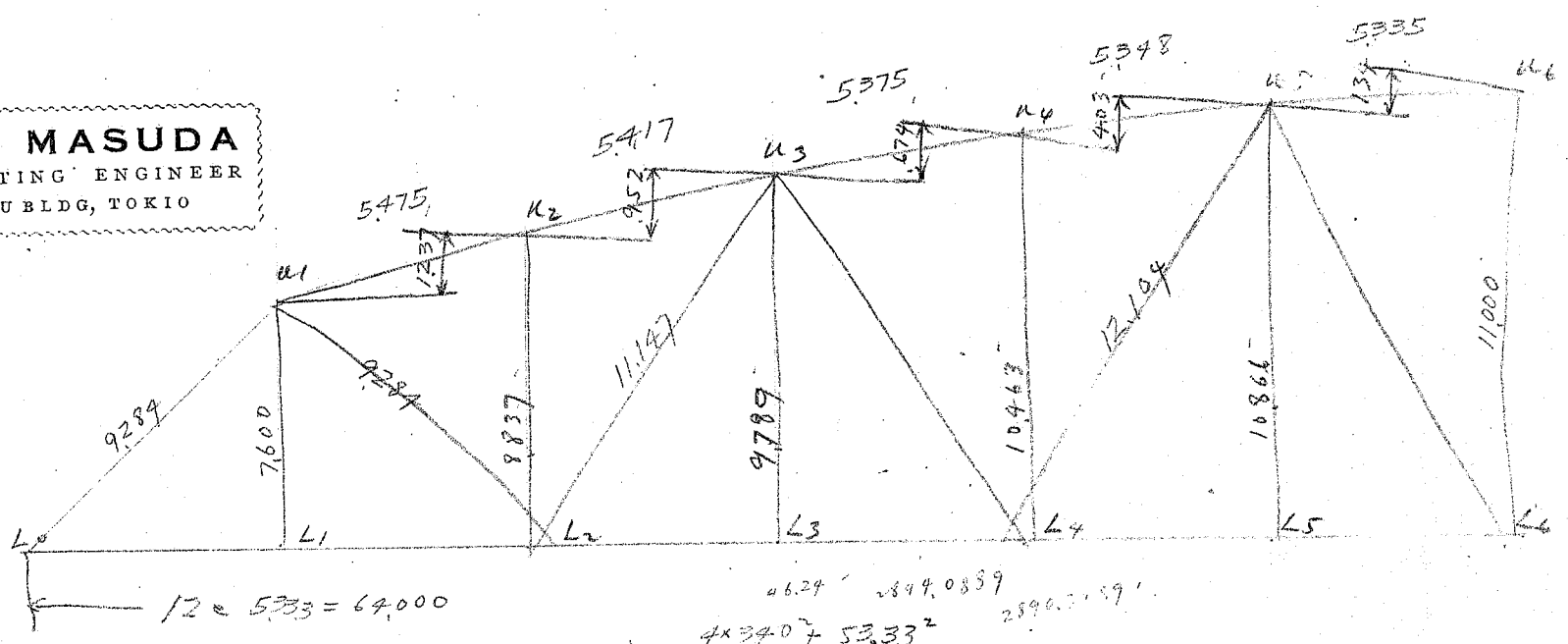
Members	Length	D.L. stress	Live Load stress unif L. stress	Live Load stress Single conc.
U_1-U_2	6574	5778 C	5778 C	1156 C
U_2-U_3	6487	5702 C	5702 C	1140 C
U_3-U_4	6431	7152 C	7152 C	1430 C
U_4-U_5	6403	7120 C	7120 C	1424 C
L_0-L_1	6400	3789 T	3789 T	758 T
L_1-L_2	〃	〃 T	〃 T	〃 T
L_2-L_3	〃	6614 T	6614 T	1323 T
L_3-L_4	〃	〃 T	〃	〃 T
L_4-L_5	〃	7273 T	7273 T	1455 T
L_0-U_1	9936	5883 C	5883 C	1177 C
U_1-L_2	〃	2850 T	3153 T	701 T
L_2-U_3	12009	1855 C	2539 C	635 C
U_3-L_4	〃	945 T	1902 T	544 T
L_4-U_5	12726	310 C	1601 C	534 C
U_5-L_4'	〃	310 C	1291 T	517 T
U_2-L_2	9102	389 T	389 T	078 T
U_4-L_4	10791	469 T	469 T	094 T
U_1-L_1	7600	1000 T	1000 T	1000 T
U_3-L_3	10161	1000 T	1000 T	1000 T
U_5-L_5	11000	1000 T	1000 T	1000 T

Span 12 @ 5333 = 6400 height at center = 11.000

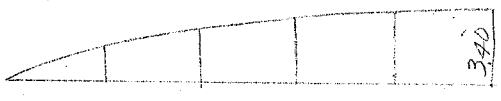
members	Length	D.L. stress	Live Load stress unif L. stress	Live Load stress Single conc.
U_1-U_2	5475			
U_2-U_3	5417			
U_3-U_4	5375			
U_4-U_5	5348			
U_5-U_6	5335			
L_0-L_1	5333			
L_1-L_2	〃			
L_2-L_3	〃			
L_3-L_4	〃			
L_4-L_5	〃			
L_5-L_6	〃			
L_0-U_1	9.284			
U_1-L_2	〃			
L_2-U_3	11.147			
U_3-L_4	〃			
L_4-U_5	12.104			
U_5-L_6	〃			
L_6-U_5'	〃			
U_2-L_2	8.837			
U_4-L_4	10.463			
U_6-L_6	11.000			
U_1-L_1	7.600			
U_3-L_3	9789			
U_5-L_5	10.866			

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JIUN MASUDA
CONSULTING ENGINEER
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$$\text{Radius} = \frac{4\times 3.40^2 + 5.333^2}{8\times 3.40} = 106.$$



L_0–u_1 $5.333^2 = 28.4409$
$7.600^2 = 57.7600$
$86.2009 = 9.284$

u_1–u_2 $5.333^2 = 28.4409$
$1.237^2 = 1.5302$
$29.9711 = 5.475$

u_2–u_3 $5.333^2 = 28.4409$
$.952^2 = .9063$
$29.3472 - 5.417$

u_3–u_4 $5.333^2 = 28.4409$
$.674^2 = .4543$
$28.8952 - 5.375$

u_4–u_5 $5.333^2 = 28.4409$
$4.03^2 = 1.624$
$28.6053 - 5.348$

u_5–u_6 $5.333^2 = 28.4409$
$.134^2 = .0180$
$28.4589 - 5.335$

$y = 3.400 - 106.26 + \sqrt{106.26^2 - 5.333^2} =$

$-10.286 + \sqrt{106.26^2 - 10.666^2} =$ $11,291.1876$ $11,177.4740 - 105.723$ 2.863

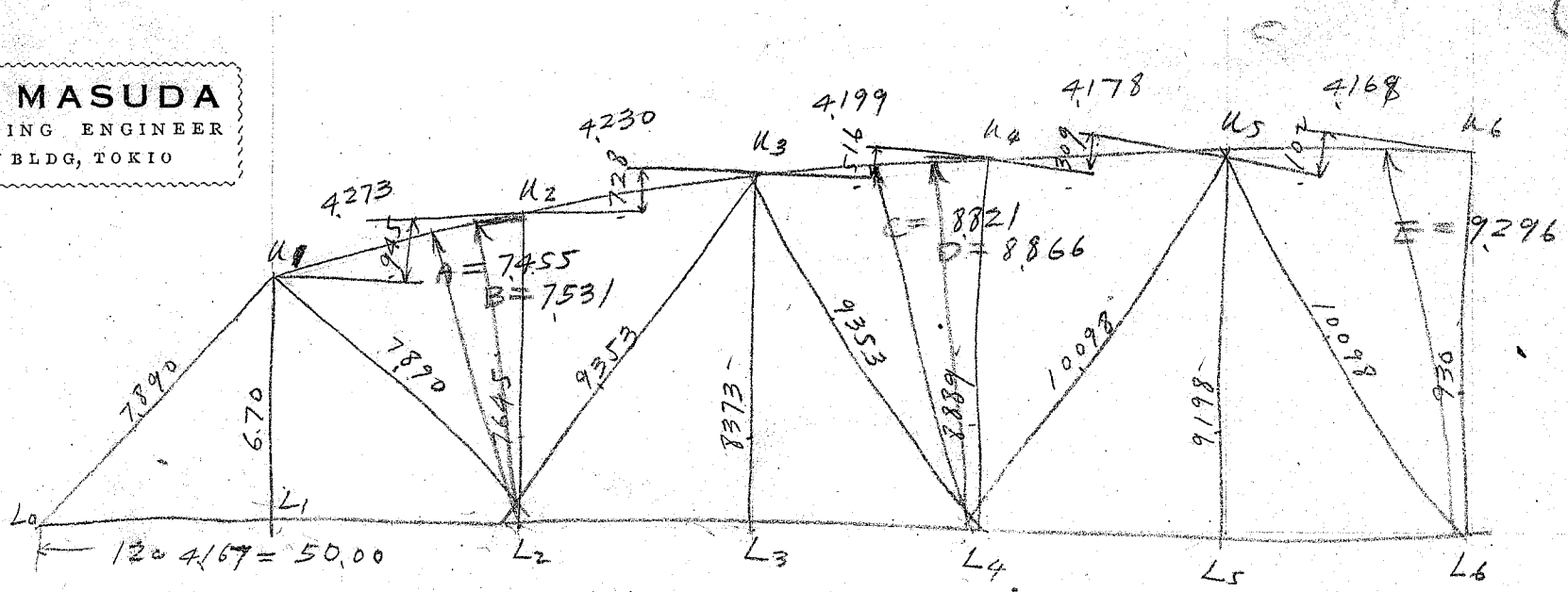
$+ \sqrt{106.26^2 - 15.999^2} =$ $11,291.1876 - 455.0580$ $11,035.2196 - 105.049$ 2.189

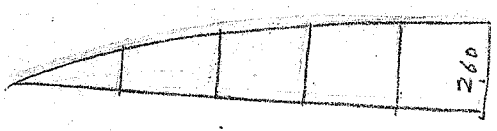
$+ \sqrt{106.26^2 - 21.332^2} =$ $11,291.1876 - 455.0542$ $10,836.1334 - 104.097$ 1.237

L_2–u_3 $5.333^2 = 28.4409$
$9.789^2 = 95.8245$
$124.2654 - 11.147$

L_4–u_5 $5.333^2 = 28.4409$
$10.866^2 = 118.0700$
$146.5109 - 12.104$

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Radius $= \frac{4\times 260^2 + 41.67^2}{8\times 260} = \frac{27.04\ \ 17361.3889}{208\ \ \ 1763.0289} = 84.780$

7,187.6484

$y = 2.60 - \frac{17.3639}{82.180}84.780 + \sqrt{84.780^2 - 41.67^2}$ $\quad 7170.2845 / 84.678 = 2498$

$+ \sqrt{84.478^2 - 83.34^2}$ $\quad 7118.1928 / 84.369 = 2189$

$+ \sqrt{84.478^2 - 12.501^2}$ $\quad 7031.3734 / 83.853 = 1673$

$+ \sqrt{84.478^2 - 16.668^2}$ $\quad 6909.8262 / 83.125 = 945$

$L_0 - U_1$ $\quad 6.7 — 44.89$
$\quad 4.167 — 17.3639$
$\quad 62.2539 — 7890$

$U_2 - U_3$ $\quad 8.373 — 70.1071$
$\quad 4.167 — 17.3639$
$\quad 87.4710 — 9353$

$U_4 - U_5$ $\quad 9.198 — 84.6032$
$\quad 4.167 — 17.3639$
$\quad 101.9671 — 10098$

$U_4 - U_5$ $\quad 4167 — 17.3639$
$\quad 102 — 0144$
$\quad 17.3779 — 4169$
$\quad 17.3743$

$U_1 - U_2$ $\quad .945 — .8930$
$\quad 4.167 — 17.3639$
$\quad 18.2569 — 4273$

$U_2 - U_3$ $\quad .728 — .5300$
$\quad 4.167 — 17.3639$
$\quad 17.8939 — 4230$

$U_3 - U_4$ $\quad .516 — .2663$
$\quad 17.3639$
$\quad 17.6302 — 4199$

$U_4 - U_5$ $\quad .309 — 0.0955$
$\quad 17.3639$
$\quad 17.4594 — 4178$

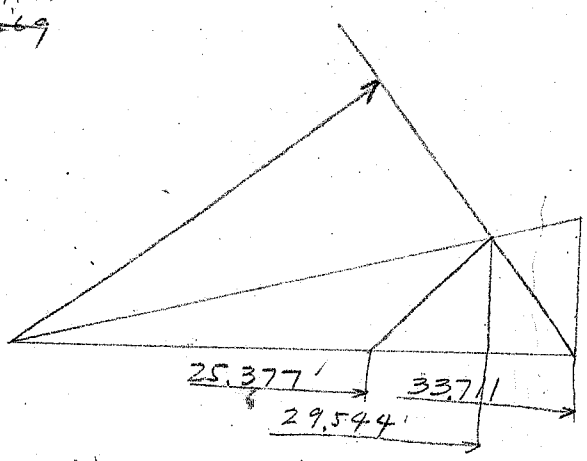
$A = \frac{4167\times 7.645}{4.273} = 7.455$

$B = \frac{31.3567}{4.230} = 7.531$

$C = \frac{4.167\times 8.889}{4.199} = 8.821$

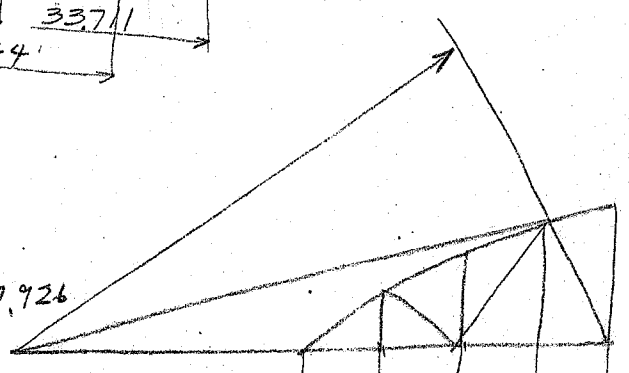
$D = \frac{37.0465}{4.178} = \frac{}{38.7531}\ 8.866$

$E = \frac{4167\times 9.30}{4.168} = 9.296$



$\frac{4167\times 7.645}{.945} = 33.711$

$d = \frac{6.70\times 33711}{7.890} = 28.627$

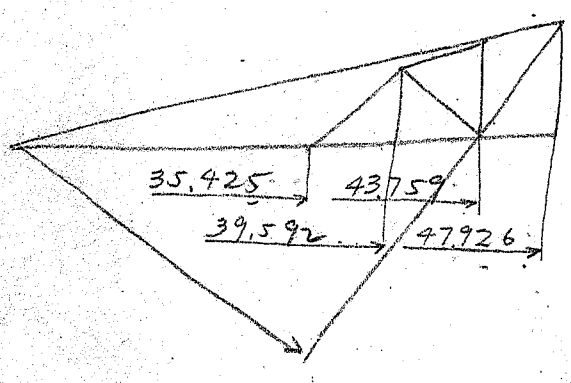


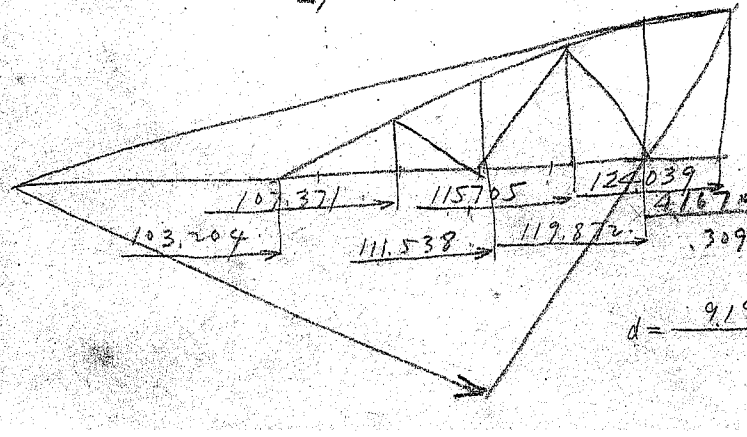
$\frac{4167\times 8889}{516} = 71.754$

$\frac{4167\times 8373}{.729} = 47.926$

$d = \frac{8373\times 71784}{9.353} = 64.262$

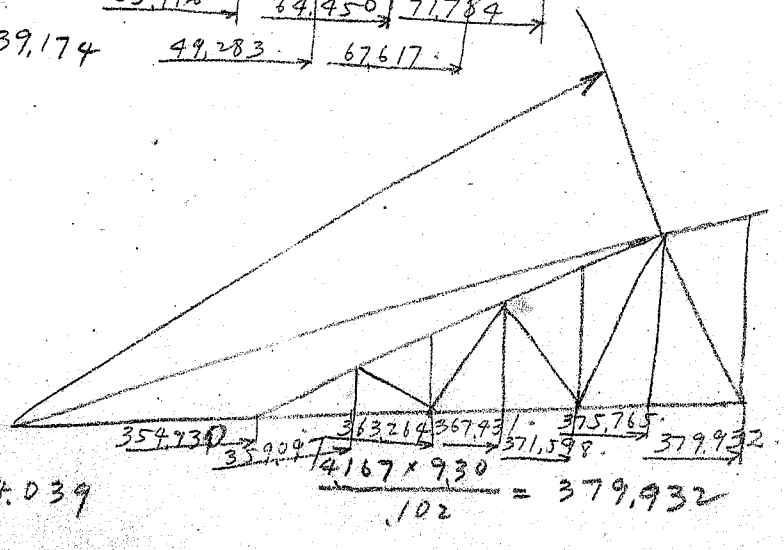
$d = \frac{8373\times 43759}{9.353} = 39.174$





$\frac{4167\times 9.198}{.309} = 124.039$

$d = \frac{9.198\times 119.872}{10.098} = 109.188$



$\frac{4167\times 930}{.102} = 379.932$

$d = \frac{9.198\times 379.932}{10.098} = 346.070$

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

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Dead Load

u_1-u_2 $\frac{5.5\times 8334-4167}{7455}=5.590$

u_2-u_3 $\frac{}{7531}=5.533$

u_3-u_4 $\frac{5.5\times 16668-6\times 4167}{8821}=7.558$

u_4-u_5 $\frac{5.5}{8866}=7520$

u_5-u_6 $\frac{5.5\times 25.00-15\times 4167}{9296}=8.067$

u_6-u_5 $\frac{5.5\times 103.204-107.371-111.538-115.705-119.872}{109.188}=1.036$

L_0-L_2 $\frac{5.5\times 4167}{6.7}=3.421$

L_2-L_4 $\frac{5.5\times 12501-3\times 4167}{8373}=6.719$

L_4-L_6 $\frac{5.5\times 20835-10\times 4167}{9198}=7.928$

L_0-u_1 $\frac{5.5\times 7890}{670}=6.477$

u_1-L_2 $\frac{5.5\times 25.377-29.544}{28.627}=3.844$

L_2-u_3 $\frac{5.5\times 35.425-39.592-43.759}{39.174}=2.846$

u_3-L_4 $\frac{5.5\times 55.116-49.283-54.450-67.617}{64.262}=1.895$

u_5-L_6 $\frac{5.5\times 354.930-359.097-363.264-367.431-371.598-375.765}{346.070}=332$

u_1-L_1 $\frac{5.5\times 4167}{8334}=2.750$

u_2-L_2 $\frac{5590\times 945}{4273}=1236$

$\frac{5533\times 728}{423}=952$

$\frac{}{284}$

u_4-L_4 $\frac{7558\times 516}{4199}=929$

$\frac{7520\times 309}{4178}=556$

$\frac{}{373}$

u_6-L_6 $\frac{8064\times 102}{4168}=197$

$\frac{}{394}$

Single $\frac{8333}{5000}\times 2=1.6667$

u_1-u_2 $\frac{10\times 8334}{12\times 7455}=932$

u_2-u_3 $\frac{}{7531}=922$

u_3-u_4 $\frac{8\times 16668}{12\times 8821}=1260$

u_0-u_5 $\frac{}{8866}=1253$

u_5-u_5 $\frac{6\times 25.00}{12\times 9296}=1345$

Uniform load

u_1-L_1 $\frac{(1\sim 10)\times 25.377}{12\times 28.627}=4.063$

L_1-u_2 $\frac{(1\sim 9)\times 35.425}{12\times 39.174}=3.391$

u_3-L_4 $\frac{(1\sim 8)\times 55.116}{12\times 64.262}=2.573$

L_4-u_5 $\frac{(1\sim 7)\times 103.204}{12\times 109.188}=2.205$

u_5-L_6 $\frac{(1\sim 6)\times 354.930}{12\times 346.070}=1.795$

u_0-L_2 $\frac{11\times 4167}{12\times 6670}=570$

L_2-L_4 $\frac{9\times 12501}{12\times 8373}=1.120$

L_4-L_6 $\frac{7\times 20835}{12\times 9198}=1.371$

u_2-L_2 $\frac{932\times 945}{4273}=206$

$\frac{922\times 729}{4230}=159$

$\frac{}{047}$

u_4-L_4 $\frac{1260\times 516}{4199}=155$

$\frac{1253\times 309}{4178}=093$

$\frac{}{62}$

u_6-L_6 $\frac{1345\times 102}{4168}=033$

L_0-u_1 $\frac{11\times 7890}{12\times 670}=1.079$

u_1-L_2 $\frac{10\times 25377}{12\times 28627}=739$

L_2-u_3 $\frac{9\times 35425}{12\times 39174}=678$

u_3-L_4 $\frac{8\times 55116}{12\times 64262}=572$

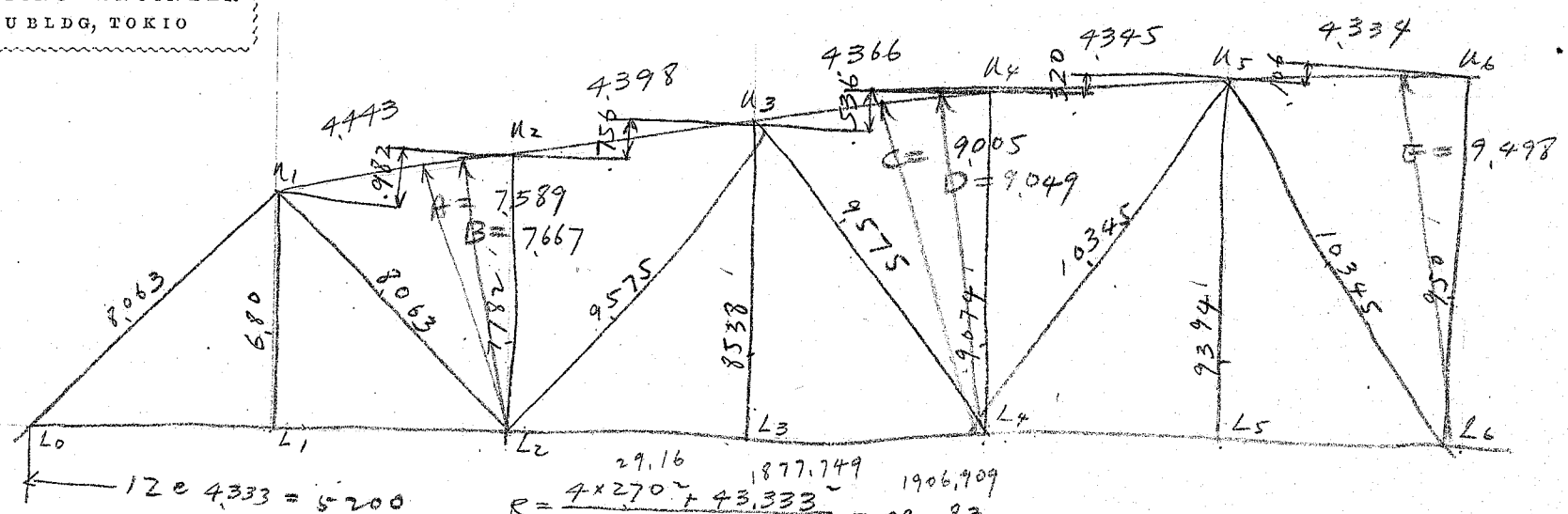
L_4-u_5 $\frac{7\times 103204}{12\times 109188}=551$

u_5-L_6 $\frac{6\times 354930}{12\times 346070}=513$

L_6-u_5 $\frac{5\times 404930}{12\times 346070}=488$

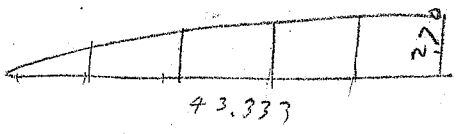
増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO



$12 \text{ e } 4,333 = 52.00$

$$R = \frac{4 \times 270^2 + 43,333^2}{8 \times 270} = 88.283$$



$$y = 270 - 88,283 + \sqrt{88,283^2 - 4,333^2} = 85.583 + \sqrt{88,283^2 - 8,666^2} + \sqrt{88,283^2 - 12,999^2} + \sqrt{88,283^2 - 17,332^2}$$

$L_0 - u_1$: $4,333 : 18,7749 = 6,80 : 46,2406$; $65,0149 : 8,063$

$L_2 - u_3$: $18,7749 : 8,538 = 72,8974 : 91,6723$; $9,575$

$L_4 - u_5$: $18,7749 : 9394 = 88,2472 : 107,0221$; $10,345$

$u_1 - u_2$: $18,7749 : ,982 = ,9643 : 19,7592$; $4,443$

$u_2 - u_3$: $18,7749 : ,756 = ,5715 : 19,3464$; $4,398$

$u_3 - u_4$: $18,7749 : ,536 = ,2873 : 19,0622$; $4,366$

$u_6 - u_5$: $18,7749 : ,106 = ,1012 : 18,8761$; 4334

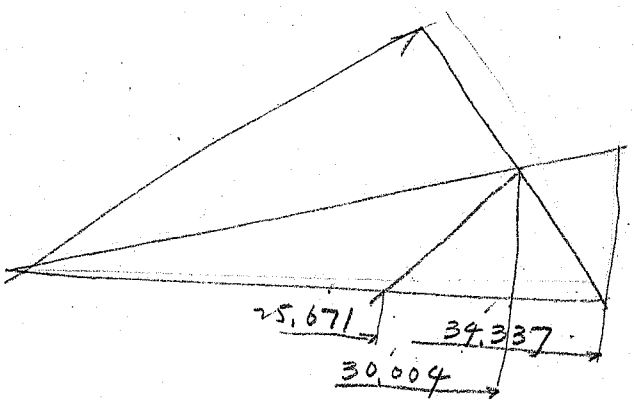
$$A = \frac{4333 \times 7,782}{4,443} = 7.589$$

$$B = \frac{4333 \times 7,782}{4,398} = 7.667$$

$$C = \frac{4,333 \times 9,074}{4,366} = 9,005$$

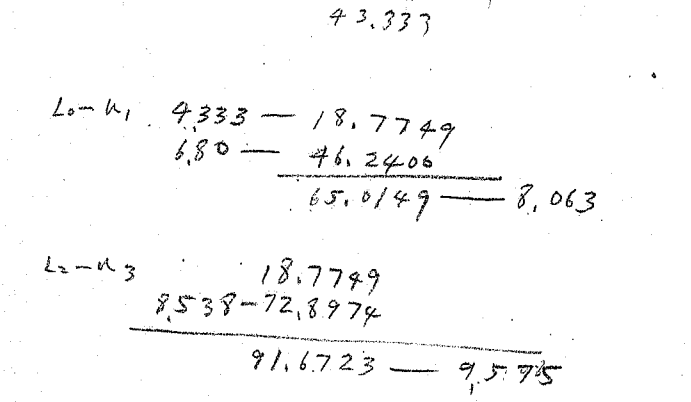
$$D = \frac{}{4,345} = 9,049$$

$$E = \frac{4,333 \times 950}{4,334} = 9,498$$



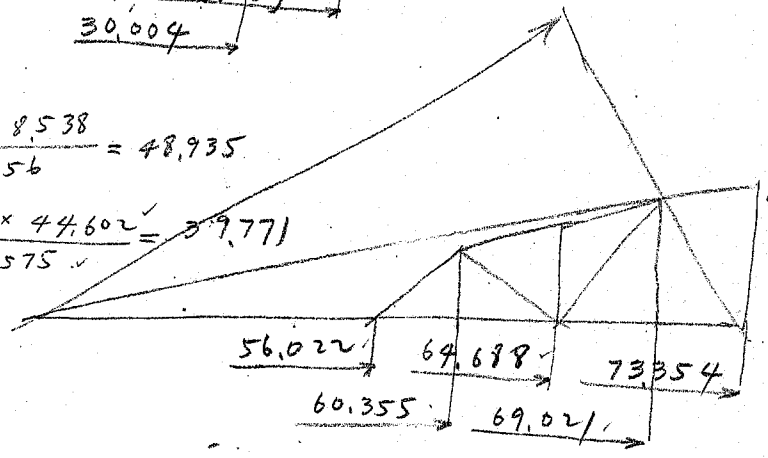
$$\frac{4,333 \times 7,782}{,982} = 34,337$$

$$d = \frac{6,80 \times 34,337}{8,063} = 28,958$$



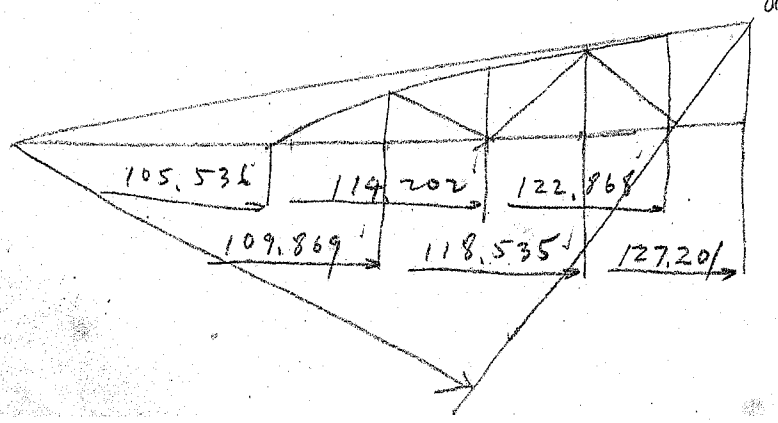
$$\frac{4,333 \times 8,538}{,756} = 48,935$$

$$d = \frac{8538 \times 44,602}{9,575} = 39,771$$



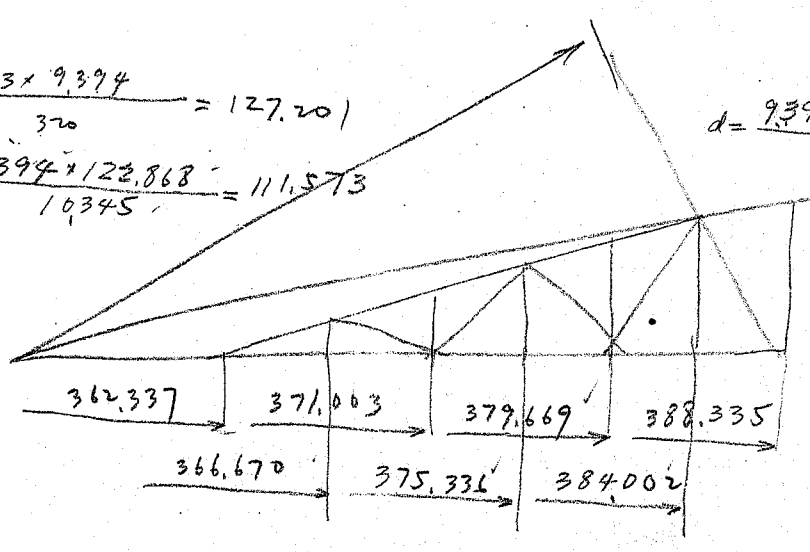
$$\frac{4333 \times 9,074}{,536} = 73,354$$

$$d = \frac{8,538 \times 73,354}{9,575} = 65,409$$



$$\frac{4,333 \times 9,394}{,320} = 127,201$$

$$d = \frac{9394 \times 122,868}{10345} = 111,573$$



$$\frac{4,333 \times 950}{,106} = 388,335$$

$$d = \frac{9,394 \times 388,335}{10,345} = 352,636$$

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Dead load

$U-U_2 \quad \frac{5.5\times 8666-4333}{7589} = 5.710$

$U_2-U_3 \quad \frac{5.5\times}{7667} = 5.651$

$U_3-U_4 \quad \frac{5.5\times 17.332-6\times 4333}{9005} = 7.703$

$U_4-U_5 \quad \frac{}{9049} = 7.665$

$U_5-U_6 \quad \frac{5.5\times 26.0-15\times 4333}{9498} = 8.213$

$U_2-L_2 \quad \frac{5710\times 982}{4443} = 1.262$

$\frac{5651\times 756}{4398} = 971$

2911

$U_4-L_4 \quad \frac{7703\times 536}{4366} = 946$

$\frac{7665\times 320}{4345} = 565$

1511

$U_6-L_6 \quad \frac{8213\times 106}{4334} = 201$

402

Unif.

$U_1-L_2 \quad \frac{4.58333\times 25.671}{28.958} = 4.063$

$L_2-U_3 \quad \frac{3.75\times 35.936}{39.771} = 3.388$

$U_3-L_4 \quad \frac{3.00\times 56.022}{65.409} = 2.569$

$L_4-U_5 \quad \frac{2.333\times 105.536}{111.573} = 2.207$

$U_5-L_6 \quad \frac{1.75\times 362.337}{352.636} = 1.798$

$L_6-U_6 \quad \frac{1.25\times 414.337}{352.636} = 1.469$

Single

$U_1-U_2 \quad \frac{.8333\times 8.666}{7589} = .952$

$U_2-U_3 \quad \frac{}{1667} = .942$

$U_3-U_4 \quad \frac{.6666\times 17.332}{9005} = 1.283$

$U_4-U_5 \quad \frac{}{9049} = 1.277$

$U_5-U_6 \quad \frac{0.5\times 26.0}{9498} = 1.369$

$U_2-L_2 \quad \frac{952\times 982}{4443} = 210$

$\frac{942\times 756}{4398} = 162$

048

$U_4-L_4 \quad \frac{1283\times 536}{4366} = 158$

$\frac{1277\times 320}{4345} = 094$

064

$L_2-L_3 \quad \frac{5.5\times 4333}{6.8} = 3.505$

$L_2-L_4 \quad \frac{5.5\times 12999-3\times 4333}{8538} = 6.851$

$L_4-L_6 \quad \frac{5.5\times 21.665-10\times 4333}{9394} = 8.072$

$L_0-U_1 \quad \frac{5.5\times 8063}{6.80} = 6.522$

$U_1-L_2 \quad \frac{5.5\times 25.671-30.004}{28.958} = 3.840$

$L_2-U_3 \quad \frac{5.5\times 35.936-40.269-44.602}{39.771} = 2.836$

Single

$U_1-U_2 \quad \frac{}{7589} =$

$U_2-U_3 \quad \frac{}{7667} =$

$U_3-U_4 \quad \frac{.6666\times 17.332}{9005} = 11.555$

$U_4-U_5 \quad \frac{}{9049} =$

$U_5-U_6 \quad \frac{0.5\times 26.0}{9498} = 13.$

$L_0-L_2 \quad \frac{.9166\times 4333}{6.8} = 3.972$

$L_2-L_4 \quad \frac{.75\times 12.999}{8538} = 1.142$

$L_4-L_6 \quad \frac{5833\times 21.665}{9394} = 1.345$

$U_3-L_4 \quad \frac{1283\times 536}{4366} = 158$

$\frac{1277\times 320}{4345} = 094$

064

$U_4-L_4 \quad \frac{}{4366} =$

$\frac{}{4345} =$

$L_4-L_6 \quad \frac{}{4334} =$

$\frac{}{4334} =$

$U_3-L_4 \quad \frac{3.00\times 56.022}{65.409} = 2.569$

$L_4-U_5 \quad \frac{2.333\times 105.536}{111.573} = 2.207$

$U_5-L_6 \quad \frac{1.75\times 362.337}{352.636} = 1.798$

$L_6-U_6 \quad \frac{1.25\times 414.337}{352.636} = 1.469$

$U_4-L_4 \quad \frac{}{4366} =$

$\frac{}{4345} =$

$L_4-L_6 \quad \frac{}{4334} =$

$\frac{}{4334} =$

$U_3-L_4 \quad \frac{308.121}{5.5\times 56.022-60.355-64.688-69.021} = 1.744$

65.409

$U_4-U_5 \quad \frac{580.448}{5.5\times 105.531-109.869-114.202-118.535-122.868} = 1.030$

111.573

$U_5-L_6 \quad \frac{1992.854}{5.5\times 362.337-366.670-371.003-375.336-379.669-584.002} = .329$

352.636

$L_0-L_2 \quad \frac{.9166\times 4333}{6.8} =$

$L_2-L_4 \quad \frac{.75\times 12.999}{8538} =$

$L_4-L_6 \quad \frac{.5833\times 21.665}{9394} =$

$L_0-U_1 \quad \frac{.9166\times 8063}{680} = 1.087$

$U_1-L_2 \quad \frac{.8333\times 25.671}{28.958} = .739$

$L_2-U_3 \quad \frac{.75\times 35.936}{39.771} = .678$

$U_3-L_4 \quad \frac{.6666\times 56.022}{65.409} = .571$

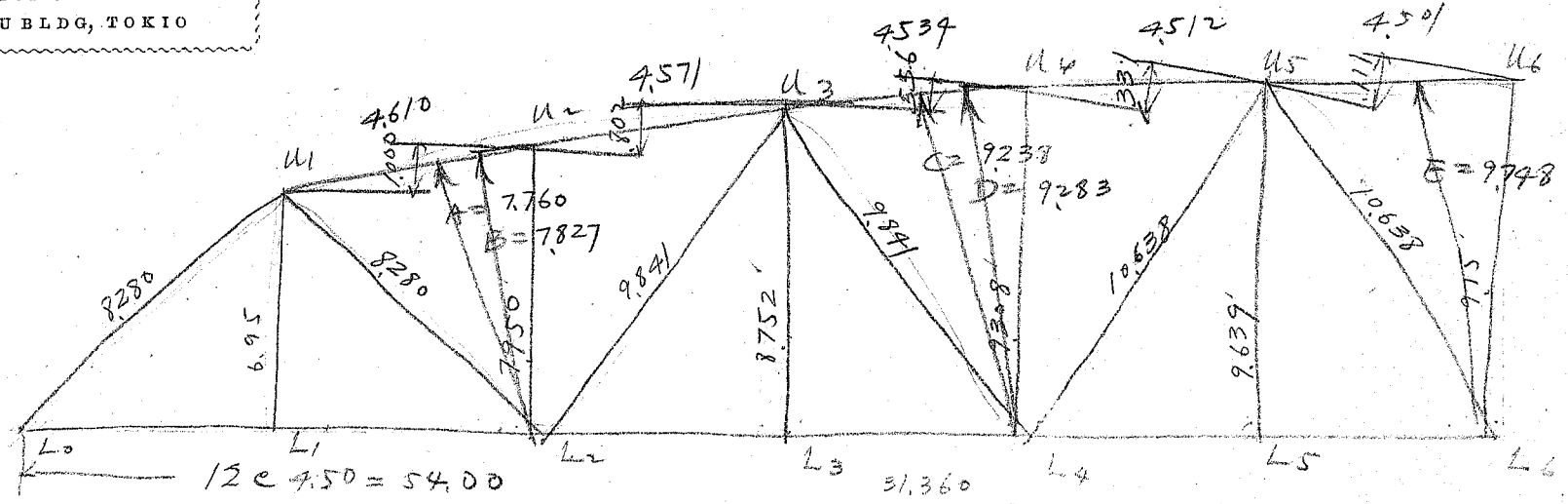
$L_4-U_5 \quad \frac{.5833\times 105.536}{111.573} = .552$

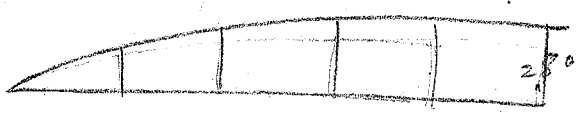
$U_5-L_6 \quad \frac{0.5\times 362.337}{352.636} = .514$

$L_6-U_6 \quad \frac{.4166\times 414.337}{352.636} = .490$

$\frac{.1369\times 106}{4334} = .033$

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$$R=\frac{4\times280^2+45.00^2}{8\times280}=\frac{20256.360}{91.802}$$

$$y=280-91.802+\sqrt{91.802^2-45^2}=89.002'+\sqrt{91.802^2-9.0^2}+\sqrt{91.802^2-13.5^2}+\sqrt{91.802^2-18.0^2}$$

8427.607 / 20.25 = 8907.357 → 91.191 2.689'
81.0 → 8346.607 → 91,360 2.358'
18.75 → 8245.357 → 90.809 1.802'
324 → 8103.607 → 90.002 1.902'

U_1-U_2
4.5 — 20.25
1.000 — 1.00
21.25 — 4.610

U_2-U_3 4.5 — 20.25
.802 — .643
20.893 — 4.571

U_3-U_4 4.5 — 20.25
.556 — .309
20.559 — 4.534

U_4-U_5 4.5 — 20.25
.331 — .110
20.360 — 4.512

U_5-U_6 4.5 — 20.25
.111 — .012
20.262 — 4.501

L_0-U_1 4.50 — 20.25
6.95 — 48.303
68.553 — 8.280

L_2-U_3 4.5 — 20.25
8.752 — 76.598
96.848 — 9.841

L_4-U_5 4.5 — 20.25
9.639 — 92.910
113.160 — 10.638

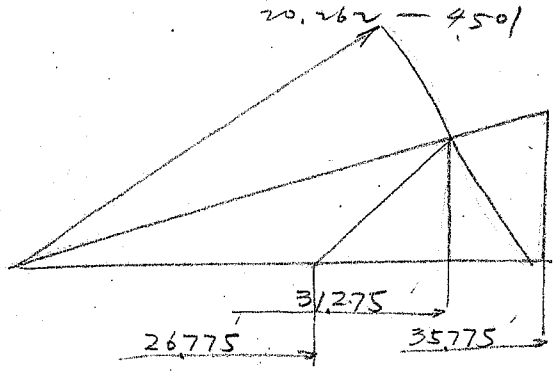
$A=\frac{4.50\times7.950}{4.610}=\frac{35.775}{}=7.760$

$B=\frac{4.50\times7.950}{4.571}=7.827$

$C=\frac{4.5\times9.308}{4534}=\frac{41.886}{}=9.238$

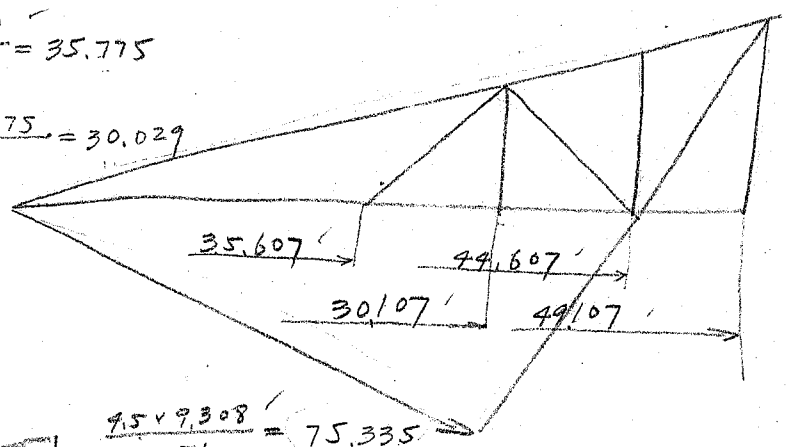
$D=\frac{4.5\times9.308}{4.512}=9.283$

$E=\frac{4.5\times9.75}{4.501}=\frac{43.875}{}=9.748$



$\frac{4.50\times7.950}{1.000}=35.775$

$\frac{6.95\times35.775}{8.280}=30.029$

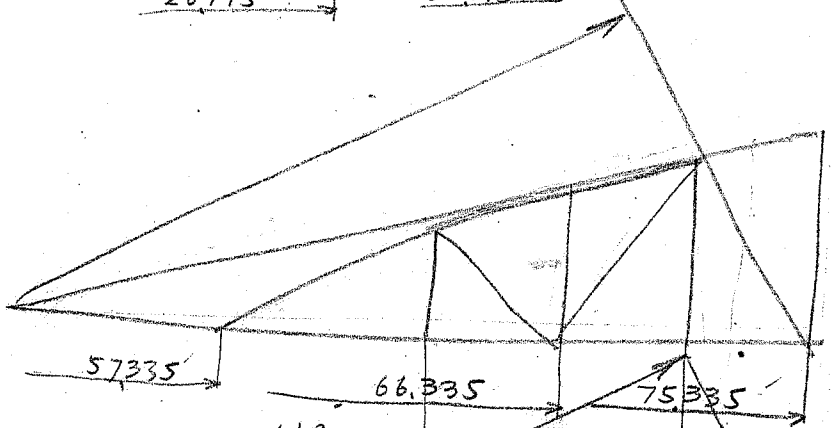


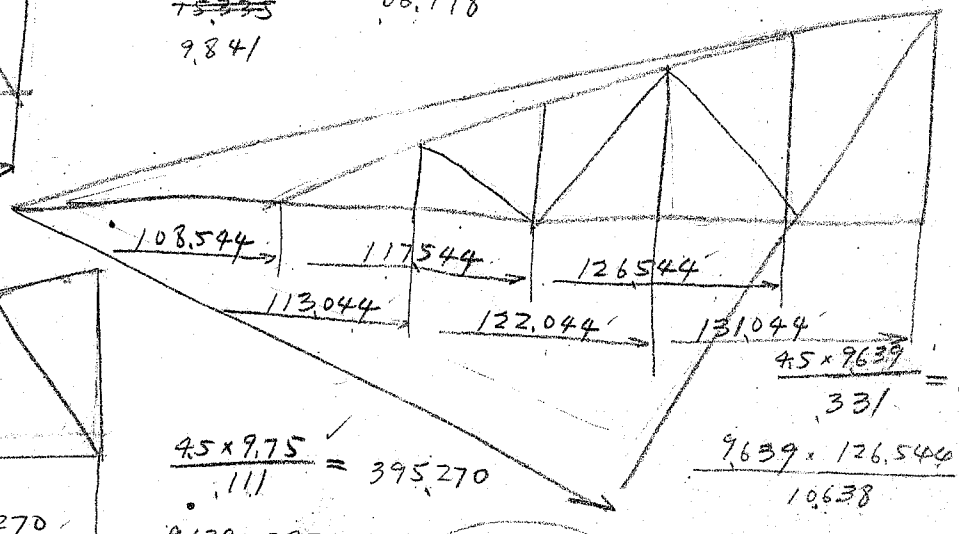
$\frac{4.5\times8.752}{1.802}=49.107$

$\frac{8.752\times49.607}{9.841}=\frac{39.671}{}$

$\frac{9.5\times9.308}{.556}=75.335$

$\frac{8.752\times75.335}{9.841}=66.998$

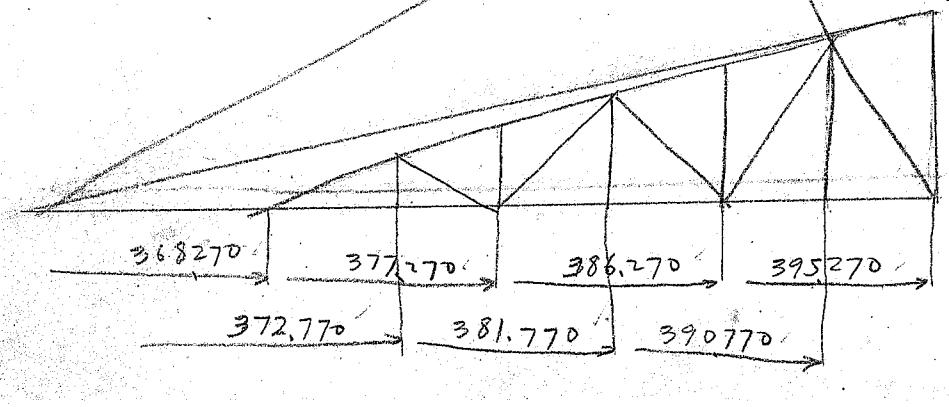




$\frac{4.5\times9.639}{.331}=131.044$

$\frac{4.5\times9.75}{.111}=395.270$

$\frac{9.639\times126.544}{10.638}=114.660$



$\frac{9.639\times395.270}{10.638}=\frac{358.151}{}$

Dead Load stresses

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U_1-U_2 49.5 $\frac{5.5\times 9.0-4.5}{7.760}=5.799$

U_2-U_3 $\frac{45}{7.827}=5.749$

U_3-U_4 99.0 −7.0 $\frac{5.5\times 18.0-4.5(6.0)}{9.238}=7.794$

U_4-U_5 $\frac{72}{9.283}=7.756$

U_5-U_6 148.5 67.5 $\frac{5.5\times 27.0-4.5\times 15}{9.748}=8.309$

L_0-L_2 24.75 $\frac{5.5\times 4.5}{6.95}=3.561$

L_2-L_4 74.25 13.5 60.75 $\frac{5.5\times 13.5-4.5\times 3}{8.752}=6.941$

L_4-L_6 123.75 45 78.75 $\frac{5.5\times 22.50-4.5\times 10}{9.639}=8.170$

L_0-U_1 45.540 $\frac{5.5\times 8.280}{6.95}=6.553$

U_1-L_2 147.263 115.988 $\frac{5.5\times 26.775-31.275}{30.029}=3.863$

L_2-U_3 195.839 121.125 $\frac{5.5\times 35.607-30.107-44.607}{39.671}=3.053$

U_3-L_4 315.343 116.338 $\frac{5.5\times 57.335-61.835-66.335-70.835}{66.998}=1.736$

L_4-U_5 596.992 117.516 $\frac{5.5\times 108.564-173.044-117.544-122.044-126.544}{114.660}=1.028$

U_5-L_6 2025.485 116.635 $\frac{5.5\times 368.270-372.770-377.270-381.770-386.270-390.770}{358.151}=.326$

U_2-L_2 $\frac{5.799\times 1.080}{4.610}=1.258$ $\frac{5.749\times .802}{4.571}=1.009$ 2.49 T

U_4-L_4 $\frac{7.794\times .556}{4.534}=.956$ $\frac{7.756\times .331}{4.512}=.569$ .387 T

U_6-L_6 $\frac{8.309\times .111}{4.501}=.205$ .410

Unif. Load (Diagonal)

U_1-L_2 $\frac{(1\sim 10)\times 26.775}{12\times 30.029}=4.087$ 122.719

L_2-U_3 $\frac{(1\sim 9)\times 35.607}{12\times 39.671}=3.366$ 133.521

U_3-L_6 $\frac{(1\sim 8)\times 57.335}{12\times 66.998}=2.567$ 172.005

L_4-U_5 $\frac{(1\sim 7)\times 108.564}{114.660}=2.209$ 253.269

U_5-L_6 $\frac{(1\sim 6)\times 368.270}{358.151}=1.799$ 644.479

L_6-U_5' $\frac{(1\sim 5)\times 422.270}{358.151}=1.474$ 527.838

Single cone.

U_1-U_2 $\frac{10\times 9.0}{12\times 7.760}=.966$ 7.5000

$U_2-U_3=\frac{}{7.827}=.958$

$U_3-U_4=\frac{8\times 18.0}{10\times 9.238}=1.299$ 12.000

$U_4-U_5=\frac{}{9.283}=1.293$

$U_5-U_6=\frac{6\times 27.0}{10\times 9.748}=1.385$ 13.5

L_0-L_2 $\frac{11\times 4.5}{12\times 6.95}=.594$ 4.125

L_2-L_4 $\frac{9\times 3\times 4.5}{12\times 8.752}=1.157$ 10.125

$L_4-L_6=\frac{9\times 5\times 9.5}{12\times 9.639}=1.362$ 13.15

L_0-U_1 $\frac{11\times 8.280}{12\times 6.95}=1.092$

U_1-L_2 $\frac{10\times 26.775}{12\times 30.029}=.743$

L_2-U_3 $\frac{9\times 35.607}{12\times 39.671}=.673$

U_3-L_4 $\frac{8\times 57.335}{12\times 66.998}=.571$

$L_4-U_5=\frac{7\times 108.564}{12\times 114.660}=.552$

$U_5-L_6=\frac{6\times 368.270}{12\times 358.151}=.514$

$L_6-U_5'=\frac{5\times 422.270}{12\times 358.151}=.491$

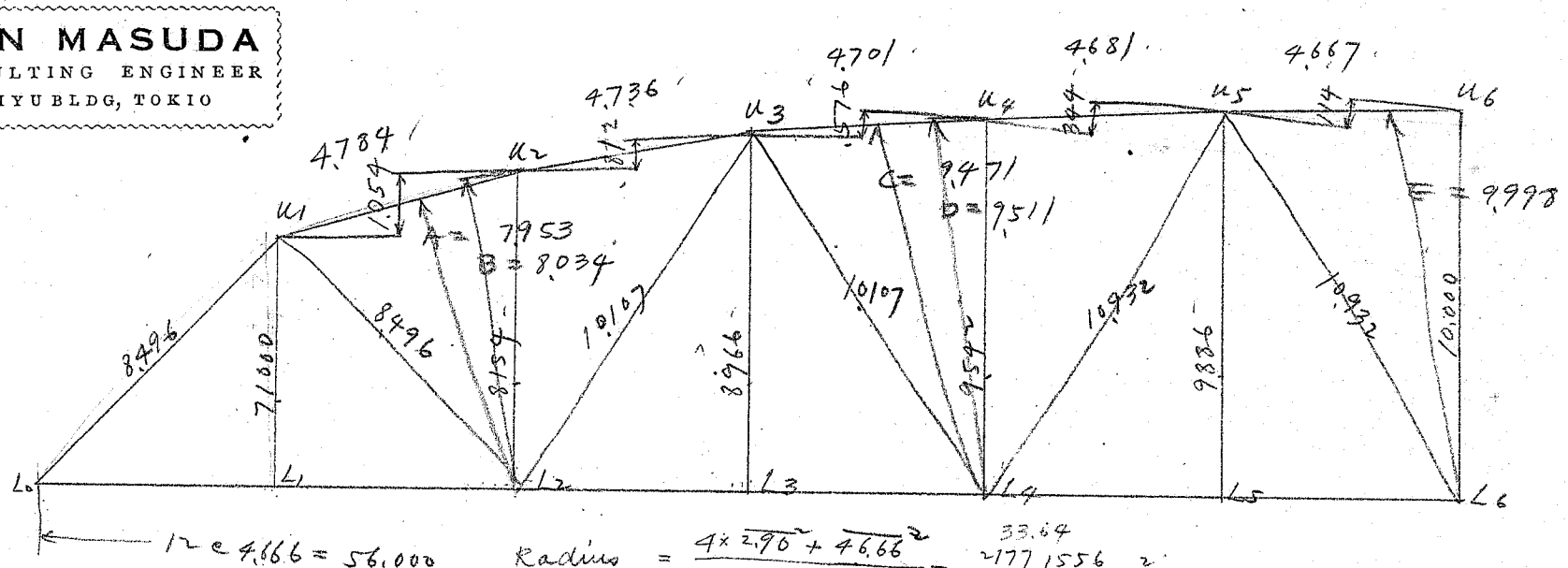
U_2-L_2 $\frac{.966\times 1.000}{4.61}=.210$ $\frac{.958\times .802}{4.571}=.168$ .042

U_4-L_4 $\frac{1.299\times .556}{4.534}=.159$ $\frac{1.293\times .331}{4.512}=.095$ .064

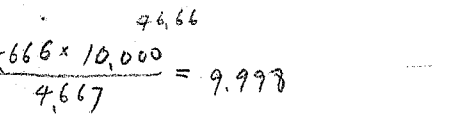
U_6-L_6 $\frac{1.385\times .111}{4.501}=.034$ .068

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO



Radius $= \frac{4\times\overline{2.90}^2 + \overline{46.66}^2}{8\times 2.90} = \frac{33.64 + 2177.1556}{23.2} = \frac{2210.7956}{23.2} = 95.293$



$y = 29.00 - 95.293 + \sqrt{95.293^2 - 4.666^2} = 9058.8842 = 2786$
(21.7716; 87.0868; 95.179)

$- 92.393 \quad + \sqrt{95.293^2 - 9.332^2} = 8993.1696 = 2442$
(95.835)

$9080.7558 \quad + \sqrt{95.293^2 - 13.998^2} = 8854.8118 = 1866$
(195.9440; 94.259)

$+ \sqrt{95.293^2 - 18.664^2} = 8732.4109 = 1054$
(345.3749; 93.447)

$L_0 - u_1$ $7.100^2 = 50.4100$ ✓
$4.666^2 = 21.7716$ ✓
$72.1816 \quad 8.496$ ✓

$L_0 - u_3$ $8.966^2 = 80.3892$
21.7716
$102.1608 - 10107$

$L_0 - u_5$ $9.886^2 = 97.7330$
21.7716
$119.5046 - 10932$

$A = \frac{4.666\times 8.154}{4.784} = 7.953$

$B = \frac{38.047}{4.736} = 8.034$

$C = \frac{4.666\times 9.542}{4.701} = 9.471$

$D = \frac{44.503}{4.681} = 9.511$

$E = \frac{4.666\times 10.000}{4.667} = 9.998$

$u_1 - u_2$
$4.661^2 = 21.7716$
$1.054^2 = 1.1109$
$22.8825 - 4.784$

$u_2 - u_3$
$4.666^2 = 21.7716$
$.812^2 = .6593$
$22.4309 - 4.736$

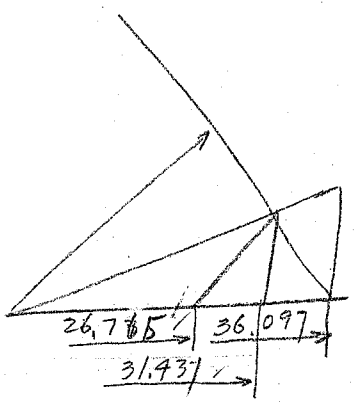
$u_3 - u_4$
21.7716
$.576^2 = .3318$
$22.1034 - 4.701$

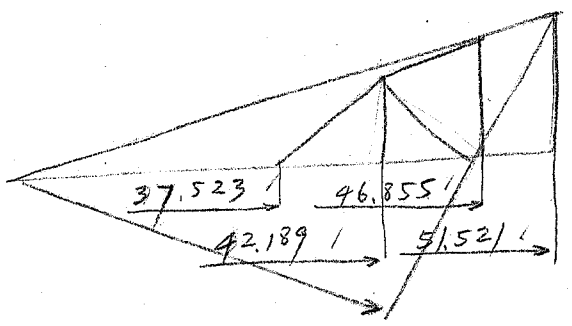
$u_4 - u_5$ 21.7716
$.344^2 = .1183$
$21.8899 - 4.681$

$u_5 - u_6$ 21.7716
$.114^2 = .0130$
$21.7846 - 4.667$

$\frac{4.666\times 8.154}{1.054} = 36.097$ ✓

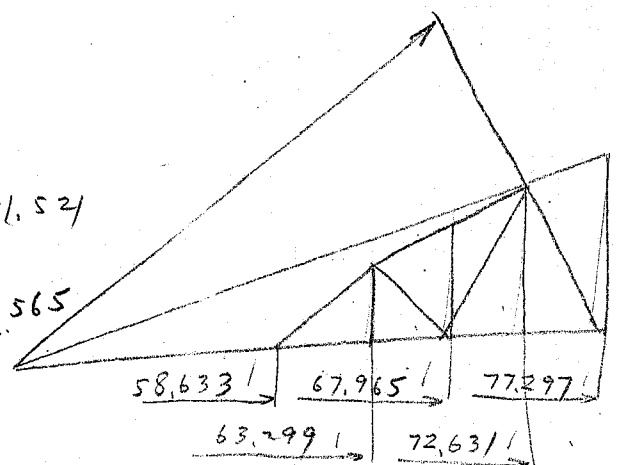
$\frac{7.100\times 36.097}{8.496} = 30.166$ ✓





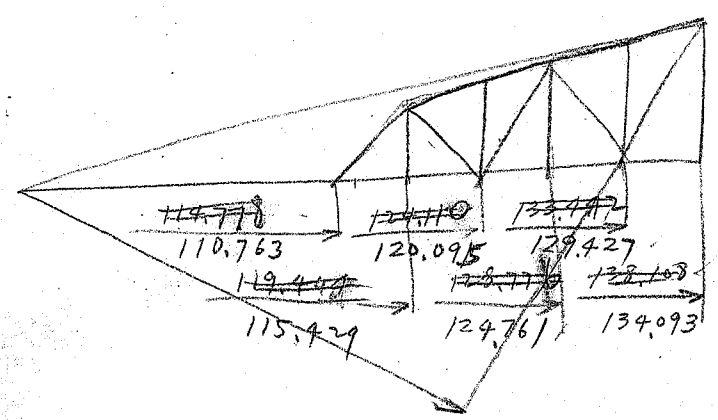
$\frac{4.666\times 8.966}{.812} = 51.521$

$\frac{8.966\times 46.855}{10.107} = 41.565$



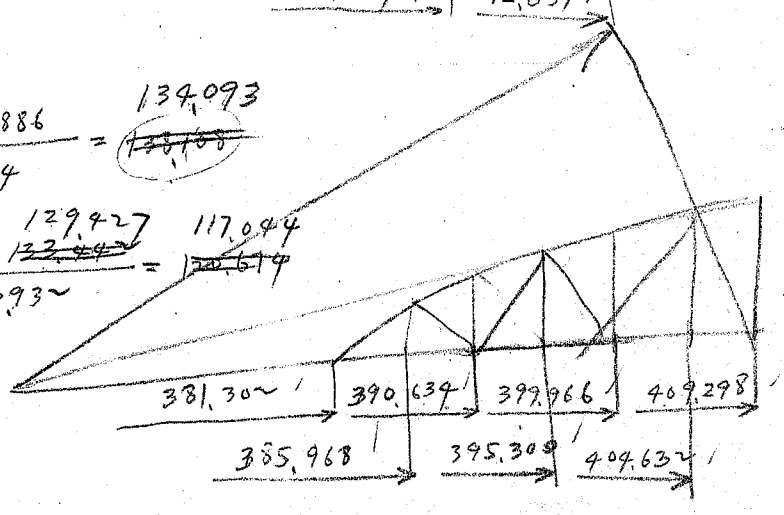
$\frac{4.666\times 9.542}{.576} = 77.297$

$\frac{8.966\times 77.297}{10.107} = 68.571$



$\frac{4.666\times 9886}{.344} = 134.093$

$\frac{9.886\times 129.427}{10.932} = 117.044$



$\frac{4.666\times 10.000}{.114} = 409.298$

$\frac{9.886\times 409.298}{10.932} = 370.135$

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U_1-U_2 $\frac{5.5\times 2\times 4666 - 4.666}{7,953} = 5,867$

U_2-U_3 $\frac{\quad}{8,034} = 5808$

U_3-U_4 $\frac{5.5\times 4\times 4.666 - 6\times 4666}{9,471} = 7.883$

U_4-U_5 $\frac{\quad}{9511} = 7849$

U_5-U_6 $\frac{5.5\times 6\times 4.666 - 15\times 4666}{9,998} = 8,400$

L_0-L_2 $\frac{5.5\times 4666}{67} = 3,830$

L_2-L_4 $\frac{5.5\times 3\times 4.666 - 4.666\times 3}{8373} = 7523$

L_4-L_6 $\frac{5.5\times 5\times 4666 - 4.666\times 10}{9198} = 8.877$

L_0-U_1 $\frac{5.5\times 8.496}{71} = 6.581$

U_1-L_2 $\frac{5.5\times 26.765 - 31.431}{30166} = 3,838$

L_2-U_3 $\frac{5.5\times 37.523 = 42.189 - 46.855}{41,565} = 2.823$

U_3-L_4 $\frac{5.5\times 58.633 - 63.299 - 67.965 - 72.631}{68.571} = 1.729$

L_4-U_5 $\frac{5.5\times 1.14.778 - 119.444 - 124,110 - 129.776 - 133.442}{120,674} = 1.040$

U_2-L_2 $\frac{5867\times 1054}{4784} = 1.293$

$\frac{5808\times 812}{4736} = .996$

$\frac{\quad}{.97}$

U_4-L_4 $\frac{7883\times 576}{4701} = .966$

$\frac{7849\times 344}{4681} = .577$

$.389$

U_6-L_6 $\frac{8400\times 114}{4667} = .205$

$.410$

U_5-L_6 $\frac{5.5\times 38,302 - 385.968 - 390.634 - 395.300 - 399.966 - 409.632}{370.135} = .326$

uniform. Diagonal

$\frac{(1\sim 10)\times 26,765}{1\sim \times 30166} = 4.067$

$\frac{(1\sim 9)\times 37523}{1\sim \times 41,565} = 3.385$

$\frac{(1\sim 8)\times 58.633}{1\sim \times 68571} = 2.565$

$\frac{(1\sim 7)\times 110.763}{1\sim \times 117.044} = 2.208$

$\frac{(1\sim 6)\times 38,302}{1\sim \times 370.135} = 1803$

$\frac{(1\sim 5)\times 437.302}{1\sim \times 370.135} = 1.477$

Single concentration

U_1-U_2 $\frac{10\times 2\times 4.666}{12\times 7,953} = .978$

U_2-U_3 $\frac{\quad}{8,034} = .968$

U_3-U_4 $\frac{8\times 4\times 4666}{12\times 9,471} = 1.314$

U_4-U_5 $\frac{\quad}{9511} = 1.309$

U_5-U_6 $\frac{6\times 6\times 46.66}{1\sim \times 9,998} = 1.400$

L_0-L_2 $\frac{11\times 4.666}{12\times 7,100} = .602$

L_2-L_4 $\frac{9\times 3\times 4666}{12\times 8,966} = 1.176$

L_4-L_6 $\frac{7\times 5\times 4.666}{1\sim \times 8,886} = 1.377$

$609.197 \quad 119.485$

$5.5\times 110,763 - 115,429 - 120,095 - 124,761 - 129.427 = 1021$

117.044

Diagonal

L_0-U_1 $\frac{11\times 8496}{12\times 7,100} = 1.097$

U_1-L_2 $\frac{10\times 26765}{12\times 30166} = .739$

L_2-U_3 $\frac{9\times 37523}{1\sim \times 41565} = .677$

U_3-L_4 $\frac{8\times 58.633}{1\sim \times 69,571} = .570$

L_4-U_5 $\frac{7\times 110.763}{12\times 120,674} = .555$

117.044

U_5-L_6 $\frac{6\times 38,302}{1\sim \times 370.135} = .515$

L_6-U_5' $\frac{5\times 437.302}{1\sim \times 370.135} = .49\sim$

U_2-L_2 $\frac{978\times 1054}{4784} = .215$

$\frac{968\times 812}{4736} = .166$

$.049$

U_4-L_4 $\frac{1314\times 576}{4701} = .161$

$\frac{1308\times 344}{4681} = .096$

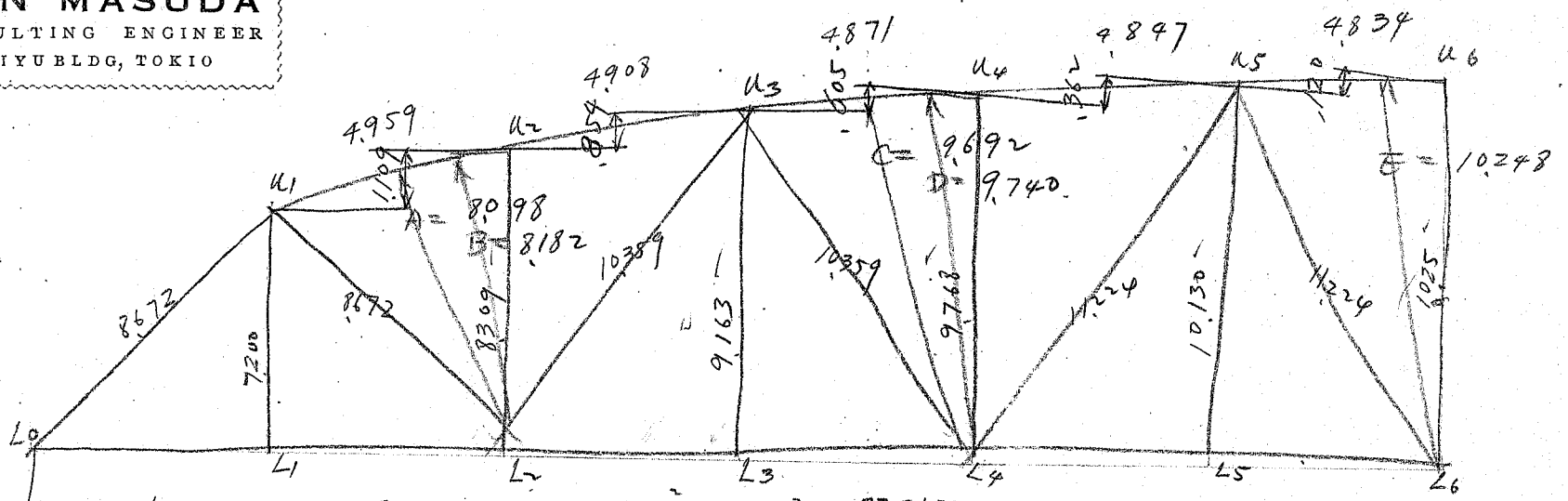
$.065$

U_6-L_6 $\frac{1400\times 114}{4667} = .034$

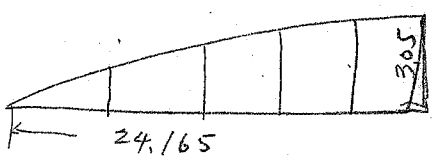
$.068$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO



← 1/2 e 4.833 = 58.00



$R = \frac{4 \times 3.05^2 + 48.33^2}{8 \times 3.05} = \frac{37.2100}{2335.7889} = 97.254$

$y = 3.05 - 97.254 + \sqrt{97.254^2 - 4.833^2} = 2.930$ ✓

$+ \sqrt{97.254^2 - 9.666^2} = 96.772 = 2.568$ ✓

$+ \sqrt{97.254^2 - 14.499^2} = 96.167 = 1.963$ ✗

$+ \sqrt{97.254^2 - 19.332^2} = 95.313 \approx 1.109$

L0–U1 $4.833^2 = 23.3579$ ✓
$7.200^2 = 51.8400$
75.1979 ✓ — 8.672 ✓

L2–U3 $= 23.3579$
$9.163^2 = 83.9606$
107.3185 — 10.359

U1–U2 $= 23.3579$
1.109 = 1.2299
24.5878 — 4.959

U2–U3 $= 23.3579$
.854 = .7293
24.0872 — 4.908

U3–U4 $= 23.3579$
.605 = .3660
23.7239 — 4.871

U4–U5 $= 23.3579$
.362 = .1310
23.4889 — 4.847

U5–U6 $= 23.3579$
.120 = .0144
23.3723 — 4.834

U0–U5 $= 23.3579$
$10.130^2 = 102.6169$
125.9748 — 11.224

$A = \frac{4.833 \times 8.309}{4.959} = 8.098$ (40.1574)

$B = \frac{4.833 \times 8.309}{4.908} = 8.18$ ~

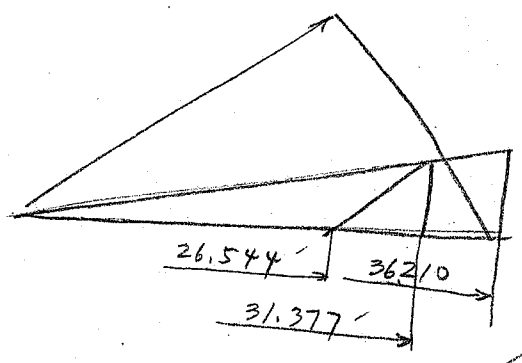
$C = \frac{4.833 \times 9.768}{4.871} = 9.692$ ~ (47.2087)

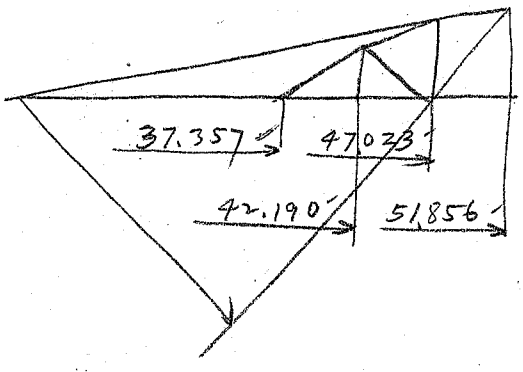
$D = \frac{4.833 \times 9.768}{4.847} = 9.740$

$E = \frac{4.833 \times 10.25}{4.834} = 10.248$ (49.5383)

$\frac{4.833 \times 8.309}{1.109} = 36.210$ ✓

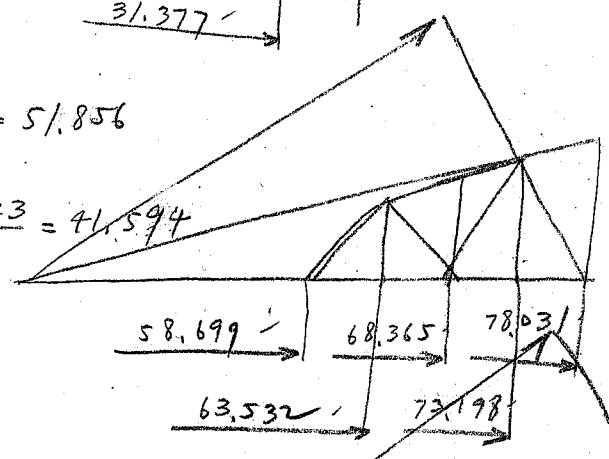
$\frac{7.200 \times 36.210}{8.672} = 30.064$ ✓





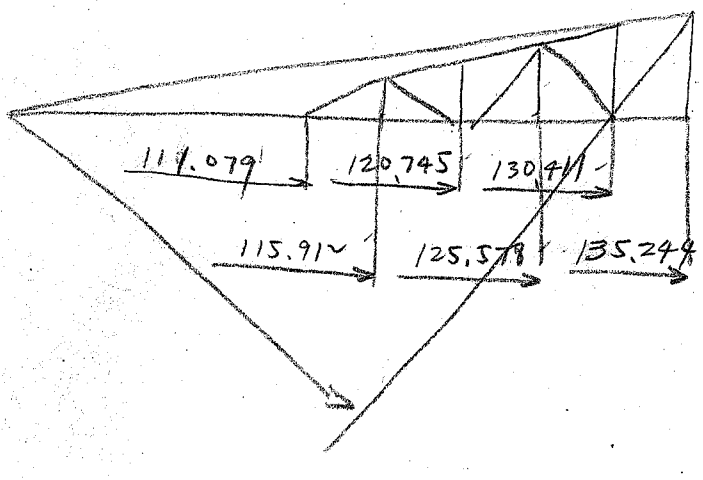
$\frac{4.833 \times 9.163}{.854} = 51.856$

$\frac{9.163 \times 47.023}{10.359} = 41.594$



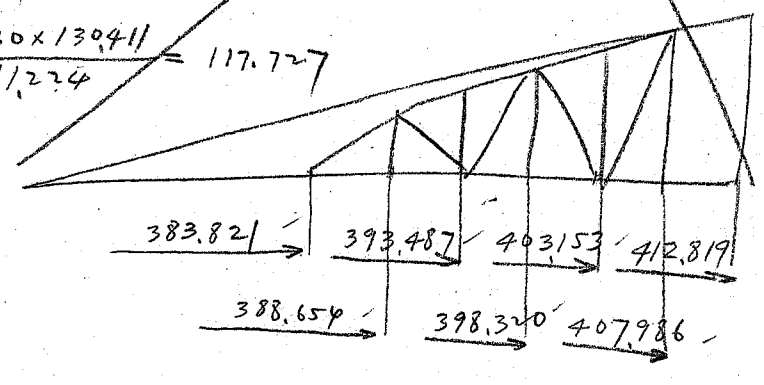
$\frac{4.833 \times 9.768}{.605} = 78.031$

$\frac{9.163 \times 78.031}{10.359} = 69.02$ ~



$\frac{4.833 \times 10.130}{.362} = 135.244$

$\frac{10.130 \times 130.411}{11.224} = 117.727$



$\frac{4.833 \times 10.25}{.120} = 412.819$

$\frac{10.130 \times 412.819}{11.224} =$
372.58 ~

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

Dead Load stresses

U_1-U_2 $\frac{5.5\times2\times4,833-4,833}{8,098}=5.968$

U_2-U_3 $\frac{}{8,182}=5907$

U_3-U_4 $\frac{5.5\times4\times4833-6\times4,833}{9,692}=7.979$

U_4-U_5 $\frac{}{9,740}=7939$

U_5-U_6 $\frac{5.5\times6\times4833-15\times(4833)}{10,248}=8489$

L_0-U_5 $\frac{5.5\times111,079-115,912-120,745-125,578-130,411}{117,727}=1,005$

U_2-L_2 $\frac{5968\times1109}{4959}=1335$

$\frac{5,907\times.854}{4908}=\frac{1,028}{307}$

U_4-L_4 $\frac{7,979\times.605}{4871}=991$

$\frac{7939\times.362}{4847}=\frac{593}{398}$

U_6-L_6 $\frac{8,489\times.120}{4834}=211$

$.422$

U_1-U_2 $\frac{10\times2\times4833}{12\times8,098}=.995$

U_2-U_3 $\frac{}{8182}=984$

U_3-U_4 $\frac{8\times4\times4833}{10\times9,692}=1330$

U_6-U_5 $\frac{}{9,740}=1323$

U_5-U_6 $\frac{6\times6\times4833}{10\times10248}=1415$

U_3-L_4 $\frac{8\times58,699}{12\times69,022}=567$

L_4-U_5 $\frac{7\times111,079}{12\times117,727}=550$

U_5-L_6 $\frac{6\times383,821}{12\times372,582}=515$

L_6-U_5 $\frac{5\times441,821}{12\times372,582}=494$

L_0-L_2 $\frac{5.5\times4,833}{7.2}=3.692$

L_2-L_4 $\frac{5.5\times3\times4833-3\times4833}{9163}=7,172$

L_4-L_6 $\frac{5.5\times5\times4833-10\times4833}{10,130}=8.349$

Diagonal

L_2-U_1 $\frac{5.5\times867\sim}{7.2}=6.624$

U_1-L_2 $\frac{5.5\times26.544-31,377}{30,064}=3812$

U_5-L_6 $\frac{5.5\times383,821-388,656-393,487-398,320-403,153-407,986}{372,582}=.324$

U_1-L_2 $\frac{(1\sim10)\times26544}{12\times30,064}=4047$

L_2-U_3 $\frac{(1\sim9)\times37.357}{12\times41.594}=3.368$

U_3-L_4 $\frac{(1\sim8)\times58,699}{12\times69.022}=2551$

L_4-U_5 $\frac{(1\sim7)\times111,079}{12\times117,727}=2202$

L_0-L_2 $\frac{11\times4833}{12\times7200}=.615$

L_2-L_4 $\frac{9\times3\times4833}{12\times9,163}=1187$

L_4-L_6 $\frac{7\times5\times4,833}{12\times10130}=1,392$

L_2-U_3 $\frac{9\times37.357}{12\times41,594}=.674$

U_2-L_2

$\frac{995\times1109}{4959}=223$

$\frac{984\times.854}{4908}=\frac{171}{.057}$

U_4-L_4

$\frac{1330\times.605}{4871}=165$

$\frac{1323\times.362}{4847}=\frac{099}{.064}$

L_0-U_4 $\frac{11\times8672}{12\times7,200}=1.104$

U_1-L_2 $\frac{10\times26,544}{12\times30,064}=736$

U_6-L_6 $\frac{1415\times120}{4834}=035$

$.070$

L_0-U_2 $\frac{5.5\times37,357-42,190-47,023}{41,594}=2795$

U_3-L_4 $5.5\times58.699-63,532-68,365-73,198=1706$

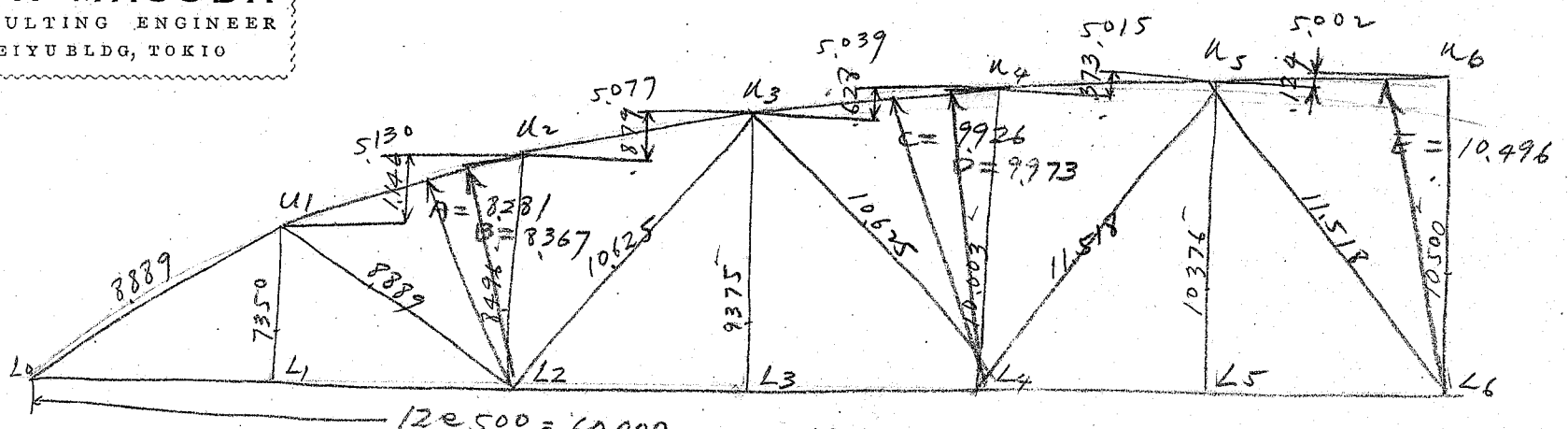
$69,022$

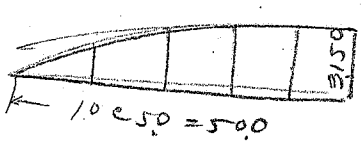
U_5-L_6 $\frac{(1\sim6)\times383,821}{372,582}=1803$

L_6-U_5 $\frac{(1\sim5)\times441,821}{372,582}=1482$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO





$$\text{Radius} = \frac{4\times 3{,}150^2 + 50.0^2}{8\times 3{,}150} = \frac{39{,}690}{25{,}200} = \frac{2500.000}{539.690} = 100.781$$

$$10{,}156{,}8100 \div 25$$

$L_0 - U_1$

$7.35^2 = 54.0225$
$5.00^2 = 25.0000$
$79.0225 - 8.889$

$L_2 - U_3$

$9.375^2 = 87.8906$
$= 25.$
$112.8906 - 10.625$

$L_4 - U_5$

$10.376^2 = 107.6614$
25
$132.6614 - 11.518$

$$A = \frac{5.0\times 8.496}{5.130} = 8.281 \qquad B = \frac{42.48}{5.077} = 8.367$$

$$C = \frac{5.0\times 10.003}{5.039} = 9.926 \qquad D = \frac{5.0015}{5.015} = 9.973 \qquad E = \frac{5.0\times 10.500}{5.002} = 10.496$$

$$l = 3{,}150 - 100.781 + \sqrt{100.781^2 - 5.0^2} = \frac{10{,}131.81}{100.657} = 1 = 3{,}026.$$

$$97.631 \qquad +\sqrt{100.781^2 - 10.0^2} = \frac{10056.81}{100.284} = 2{,}653.$$

$$+\sqrt{100.781^2 - 15.0^2} = \frac{9931.81}{99.656} = 2{,}025.$$

$$+\sqrt{100.781^2 - 20.0^2} = \frac{9756.81}{98.777} = 1146$$

$U_1 - U_2$

$5.00^2 = 25.00$
$1.146^2 = 1.3133$
$26.3133 - 5.130$

$U_1 - U_3$

$.879 = \frac{25.000}{25.7726}$
$25.7726 - 5.077$

$U_3 - U_4$

$.628 = \frac{25.00}{.3944}$
$25.3944 - 5.039$

$U_4 - U_5$

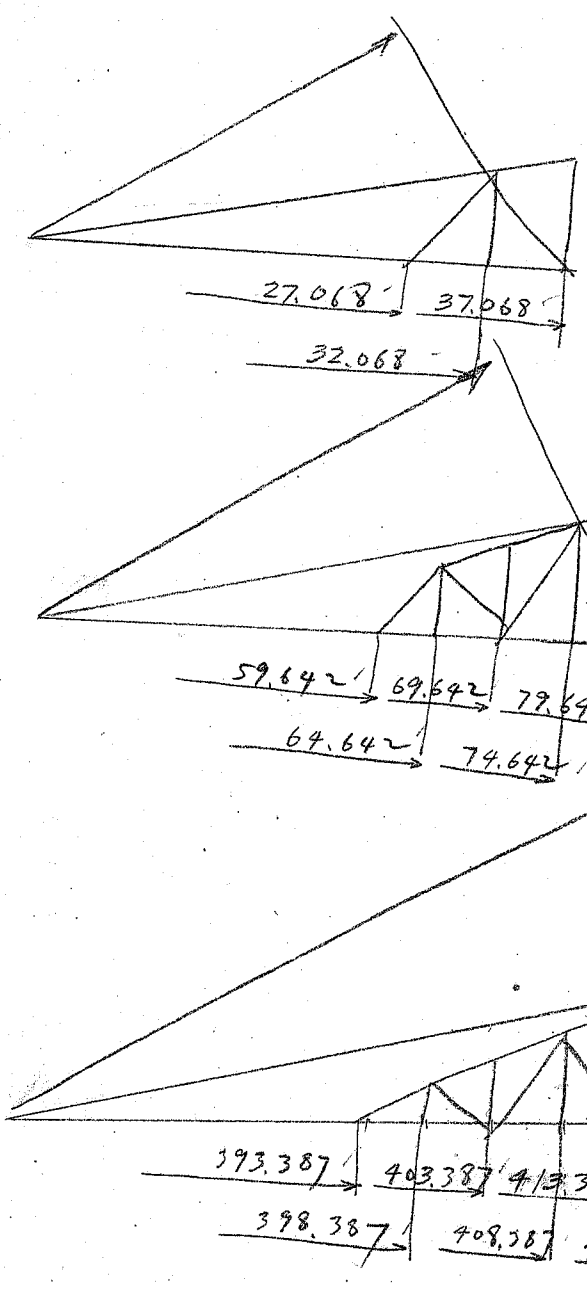
$.373 = \frac{25.0000}{.1391}$
$25.1391 - 5.015$

$U_5 - U_6$

$.124 = \frac{25.0000}{.0154}$
$25.0154 - 5.002$

$$\frac{5.0\times 8.496}{1.146} = 37.068 \qquad \frac{42.48}{}$$

$$\frac{7350\times 37.068}{8889} = 30.650 \qquad \frac{272.44}{}$$



$$\frac{5.0\times 10.003}{.628} = 79.642 \qquad \frac{50.015}{}$$

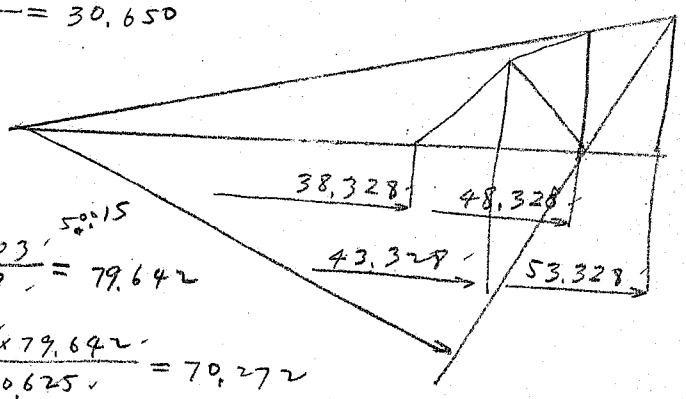
$$\frac{9375\times 79.642}{10625} = 70.272$$

$$\frac{5.0\times 10.500}{.124} = 423.387 \qquad \frac{52.5}{}$$

$$\frac{10.376\times 423.387}{11.518} = 381.409$$

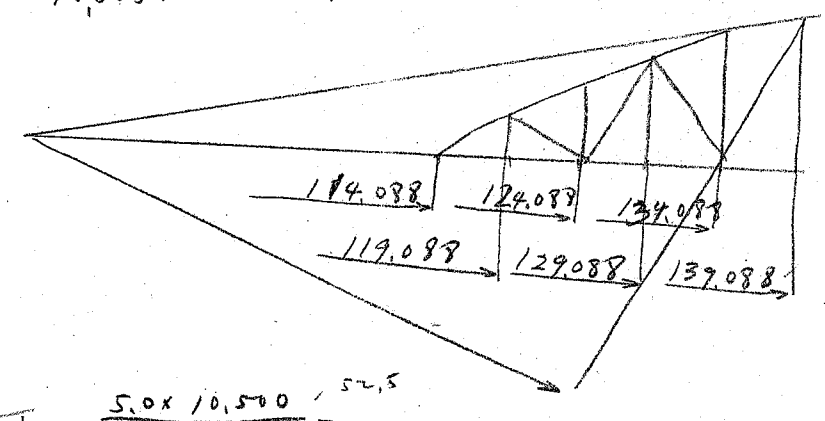
$$\frac{5.0\times 9.375}{.879} = 53.328 \qquad \frac{46.875}{}$$

$$\frac{9375\times 48.328}{10625} = 42.642$$



$$\frac{5.0\times 10376}{.373} = \frac{51.88}{139.088}$$

$$\frac{10.376\times 134.088}{11.518} = 120.793$$



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

U_1-U_2 $\frac{5.5\times2\times5.0-5.0}{8281}=6,038$

U_2-U_3 $\frac{5.5}{8,367}=5.976$

U_3-U_4 $\frac{5.5\times4\times5.0-6.0\times5.0}{9926}=8,060$

U_4-U_5 $\frac{}{9,973}=8,022$

U_5-U_6 $\frac{5.5\times6\times5.0-15\times5.0}{10,496}=8.575$

U_5-L_6 $5.5\times393.387-398.787-403.387-408.387-413.787-418.387=319$
381.409

L_0-L_2 $\frac{5.5\times5.0}{7.35}=3.741$

L_2-L_4 $\frac{5.5\times3\times5.0-3\times5.0}{9375}=7.200$

L_4-L_5 $\frac{5.5\times5\times5.0-10.\times5.0}{10,376}=8433$

L_0-U_1 $\frac{5.5\times9889}{7350}=148.874$ 6.652

U_1-L_2 $\frac{5.5\times27.068-32.068}{30.650}=3.811$

U_2-L_2 $\frac{6038\times1146}{5130}=1.349$

$\frac{5.976\times879}{5077}=1035$
314

U_1-U_2 $\frac{10\times2\times5.0}{12\times8281}=1006$

U_2-U_3 $\frac{10\times2\times5.0}{12\times8367}=996$

U_3-U_4 $\frac{8\times4\times5.0}{12\times9926}=1343$

U_4-U_5 $\frac{8\times4\times5.0}{12\times9973}=1.337$

U_5-U_6 $\frac{6\times6\times5.0}{12\times10,496}=$ ~~.858~~ 1.429

L_2-U_3 $5.5\times38.328-43.328-48.328=2794$
42.642

U_3-L_4 $5.5\times59.642-64.642-69.642-74.642=1.695$
70.272

U_4-U_5 $5.5\times114.088-119.088-124.088-129.088-134.088=1003$
120.793

U_4-L_4 $\frac{8.060\times639}{5039}=1005$

$\frac{8022\times377}{5015}=597$
1408

U_6-L_6 $\frac{8575\times124}{5002}=213$
426

L_0-L_2 $\frac{11\times5.0}{12\times7350}=1624$

L_2-L_4 $\frac{9\times3\times5.0}{12\times9375}=1.2$

L_6-L_6 $\frac{7\times5\times5.0}{12\times10376}=1406$

L_0-U_1 $\frac{11\times8.889}{12\times735}=1109$

$\frac{10.\times27.068}{12\times30.650}=736$

$\frac{9\times38.328}{12\times42.642}=674$

$\frac{8\times59.642}{12\times70.272}=566$

$\frac{7\times114.087}{12\times120.793}=551$

$\frac{6\times393.387}{12\times381.409}=516$

$\frac{5\times453.387}{12\times381.409}=495$

L_2-L_2 $\frac{1006\times1146}{513}=1225$

$\frac{998\times879\times8755}{5077}=17$
1053

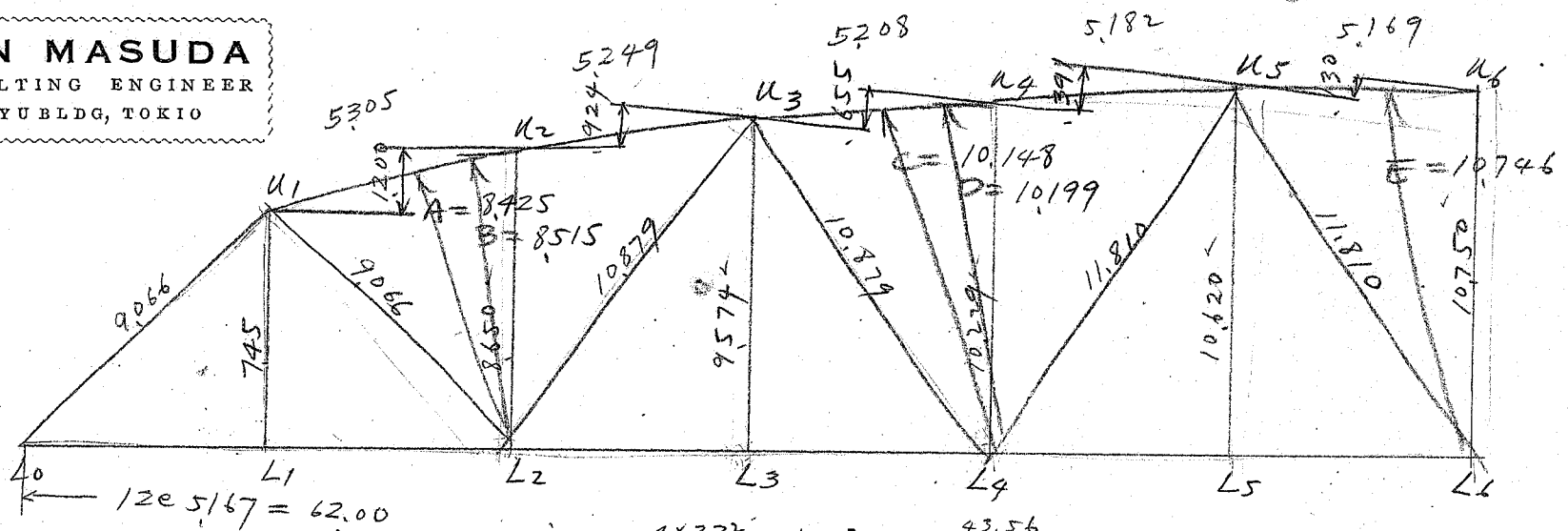
U_4-L_4 $\frac{1343\times628\times8439}{5039}=167$

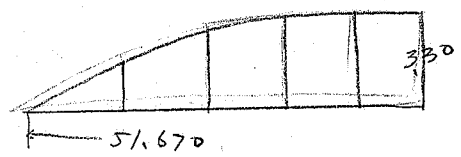
$\frac{1337\times373\times4987}{5015}=099$
1068

U_6-L_6 $\frac{1429\times124}{5002}=1772$
1035
1070

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JIUN MASUDA
CONSULTING ENGINEER
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Radius $= \frac{4\times 3.3^2 + 5.167^2}{8\times 3.3} = \frac{43.56}{26.4} = \frac{2669.7389}{2713.3489} = 102.778$

$y = 3.30 - 102.778 + \sqrt{102.778^2 - 5.167^2} = \frac{26.6979}{10,563.3173} = \frac{10,536.6194}{102.648} = 3.170$

$+\sqrt{102.778^2 - 10.334^2} = \frac{106.7916}{10,456.5257} = \frac{10,456.5257}{102.25} = 2.779$

$+\sqrt{102.778^2 - 15.501^2} = \frac{240.2810}{10323.0363} = \frac{10323.0363}{101.602} = 2.124$

$+\sqrt{102.778^2 - 20.668^2} = \frac{427.166}{10136.1511} = \frac{10136.1511}{100.678} = 1.200$

u_1-u_2
$5.167^2 = 26.6979$
$1.200^2 = 1.4400$
$28.1379 \leftarrow 5.305$

u_2-u_3
$.924 = \frac{26.6979}{.8538}$
$27.5517 \leftarrow 5.249$

u_3-u_4
$.655 = \frac{26.6979}{.4290}$
$27.1269 \leftarrow 5.208$

u_4-u_5
$.391 = \frac{26.6979}{.1529}$
$26.8508 \leftarrow 5.182$

u_5-u_6
$.130 = \frac{26.6979}{.0169}$
$26.7148 \leftarrow 5.169$

L_0-u_1
$5.167^2 = 26.6979$
$7.45 = 55.5025$
$82.2004 \leftarrow 9.066$

u_2-u_3
$9.574 = \frac{26.6979}{91.6615}$
$118.3594 \leftarrow 10.879$

L_0-u_5
$10.62 = \frac{26.6979}{112.7844}$
$139.4823 \leftarrow 11.810$

$A = \frac{5.167\times 8.650}{5.305} = 8.425$

$B = \frac{44.695}{5.249} = 8.515$

$C = \frac{5.167\times 10.229}{5.208} = 10.148$

$D = \frac{52.853}{5.182} = 10.199$

$E = \frac{5.167\times 10.750}{5.169} = 10.746$

$\frac{5.167\times 8.650}{1.200} = 37.246$

$\frac{745\times 37.246}{9.066} = 30.607$

$\frac{5.167\times 10.229}{.655} = 80.692$

$\frac{9.574\times 80.692}{10.879} = 71.013$

$\frac{5.167\times 10.750}{.130} = 427.269$

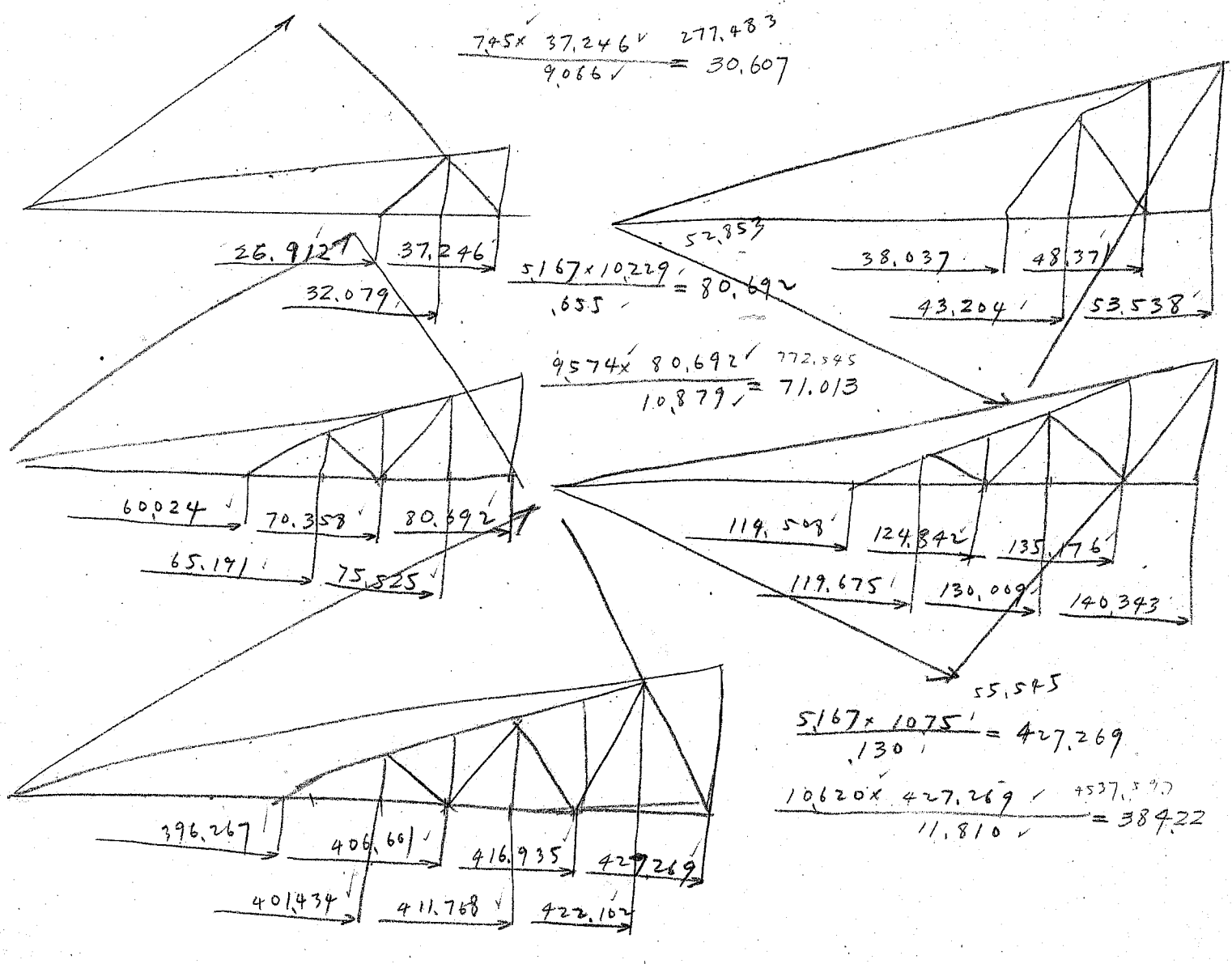
$\frac{10.620\times 427.269}{11.810} = 384.22$

$\frac{5167\times 9.574}{.924} = 53538$

$\frac{9.574\times 48.371}{10.879} = 42.569$

$\frac{5167\times 10.620}{.391} = 140.343$

$\frac{10.620\times 135.176}{11.810} = 121.56$



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Dead Load stress

L_0-L_2

$\frac{5.5\times 5{,}167}{7{,}450} = 3.815$

L_2-L_4 16.5 85.256 15.501 69.755

$\frac{5.5\times 3\times 5{,}167 - 3\times 5{,}167}{9{,}574} = 7.286$

L_4-L_6 27.5 142.093 51.670 90.423

$\frac{5.5\times 5\times 5{,}167 - 10\times 5{,}167}{10{,}620} = 8.514$

u_2-L_2

$\frac{6{,}133\times 1{,}200}{5{,}305} = 1.387$

$\frac{6068\times 1{,}924}{5{,}249} = 1{,}068$

.319

u_4-L_4 $\frac{8147\times 655}{5{,}208} = 1025$

$\frac{8{,}106\times 391}{5182} = .612$

.413

u_6-L_6

$\frac{8{,}655\times 130}{5169} = .218$

.436

u_1-u_2

$\frac{10\times 2\times 5167}{12\times 8{,}425} = 1.022$ 8.617

u_2-u_3

$\frac{}{8{,}515} = 1.011$

u_3-u_4

$\frac{8\times 4\times 5167}{12\times 10{,}148} = 1.358$ 13.779

u_4-u_5

$\frac{}{10{,}199} = 1351$

u_5-u_6 15.501

$\frac{6\times 6\times 5167}{12\times 10{,}746} = 1.442$

u_1-u_2 56.837

$\frac{5.5\times 2\times 5{,}167 - 5{,}167}{8{,}425} = 6.133$

u_2-u_3 51.670

$\frac{}{8{,}515} = 6.068$

L_0-u_1 49.863

$\frac{5.5\times 9{,}066}{745} = 6.693$

u_1-L_2 148.0 115.941

$\frac{5.5\times 26{,}912 - 32{,}079}{30{,}607} = 3.788$

L_2-u_3 209.204

$\frac{5.5\times 38{,}037 - 43{,}204}{42{,}569}$ 48.371 117.629

$= 2.763$

u_3-L_4 330.132

$\frac{5.5\times 60{,}024 - 65{,}191}{71{,}013}$ 70.358 119.058

75.525

$= 1.677$

u_1-L_2 123.347

$\frac{(1\sim 10)\times 26{,}912}{12\times 30{,}607} = 4.030$

L_2-u_3 $\frac{(1\sim 9)\times 38{,}037}{12\times 42{,}569} = 3.351$ 142.639

u_3-L_4 $\frac{(1\sim 8)\times 60{,}024}{12\times 71{,}013} = 2.536$ 180.072

L_4-u_5 $\frac{(1\sim 7)\times 114{,}508}{12\times 121.56} = 2.198$ 267.185

u_5-L_6 $\frac{(1\sim 6)\times 396.267}{12\times 384.22} = 1.805$ 693.467

L_0-L_2

$\frac{11\times 5167}{12\times 7{,}45} = .636$ 4.736

L_2-L_4

$\frac{3\times 9\times 5167}{12\times 9{,}574} = 1214$ 11.626

L_4-L_6

$5\times\frac{7\times 5167}{12\times 10{,}620} = 1419$ 15.072

u_5-L_6

$\frac{}{12\times 10{,}746} = 1442$ 15.501

L_4-u_5 629.794

$5.5\times 114{,}508 - 119.675$

$- 124.842$

$- 130.009$ 120.092

$- 135.176$

$\frac{}{121.56} = .988$

u_5-L_6 179.469

$5.5\times 396.267 - 401.434$

406.601

411.768 120.629

416.935

422.102

$\frac{}{384.22} = .314$

$u_5'-L_6$ $\frac{(1\sim 5)\times 458{,}267}{12\times 384.22} = 1491$ 572.834

u_1-L_2

$\frac{}{12\times 7{,}45} = 1116$ 8.311

u_4-L_2

$\frac{10\times 26{,}912}{12\times 71{,}013}$ 30,607 $= .733$ 22.427

L_2-u_3 8.598

$\frac{9\times 38{,}037}{12\times 42{,}569} = .670$

u_3-L_4

$\frac{8\times 60024}{12\times 71{,}013} = .564$ 40.016

L_4-u_5

$\frac{7\times 114{,}508}{12\times 121{,}56} = .549$ 66.796

u_5-L_6

$\frac{6\times 396{,}267}{12\times 384{,}22} = .516$ 198.134

L_1-u_5'

$\frac{5\times 458{,}267}{12\times 384{,}22} = .497$ 190.945

u_1-L_2

$\frac{1.022\times 1{,}200}{5{,}305} = .231$

$\frac{1{,}011\times 924}{5{,}249} = \frac{.178}{.053}$

u_4-L_4

$\frac{1{,}358\times 655}{5{,}208} = .171$

$\frac{1351\times 391}{5182} = \frac{.102}{.069}$

u_6-L_6

$\frac{1442\times 130}{5169} = .036$

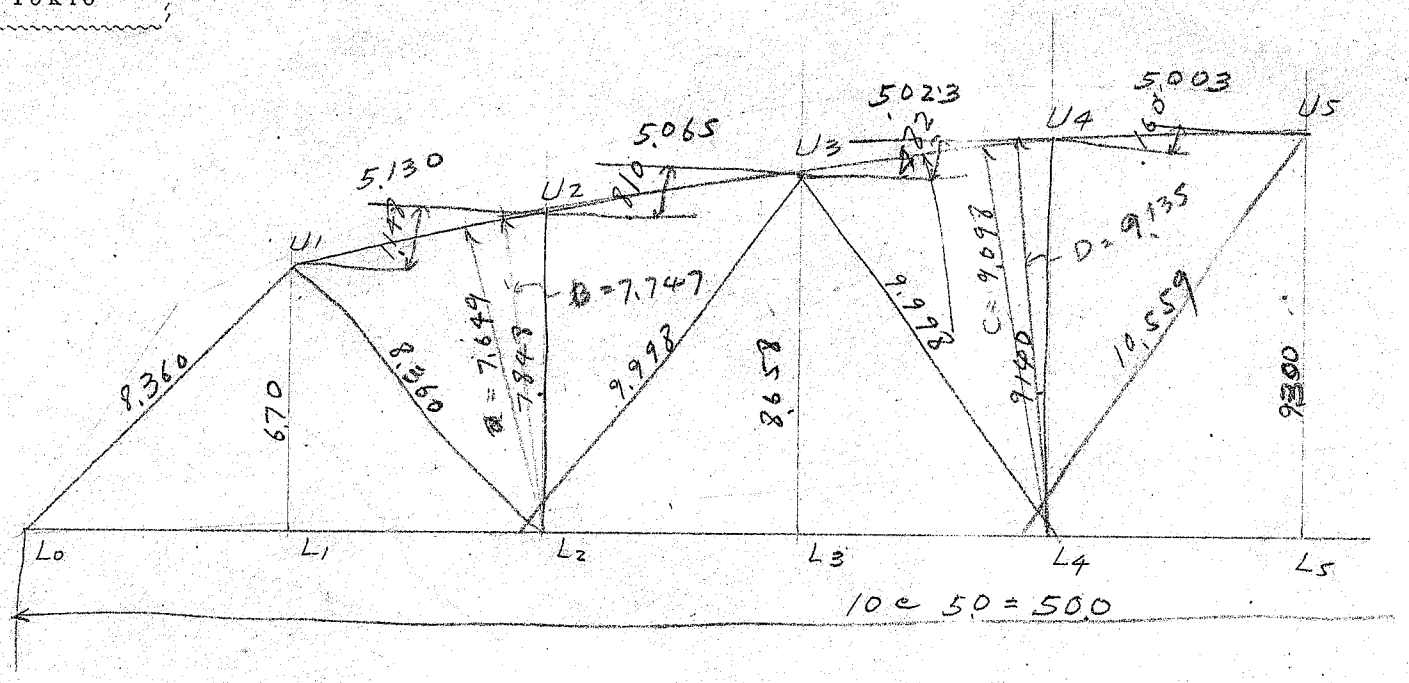
.072

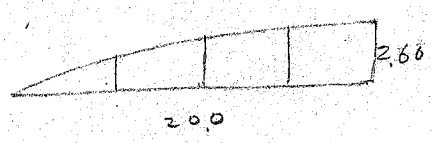
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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

(1)

(1)





$$\text{Radius} = \frac{4h^2+c^2}{8h} = \frac{4\times 2.60^2+40^2}{8\times 2.60} = \underline{78228}$$

$$f = h - r + \sqrt{r^2 - x^2} = 2.60 - 78.228 + \sqrt{78.228^2 - 50^2} = \underline{2.440}$$

$$+\sqrt{78.228^2 - 10^2} = \underline{1958}$$

$$+\sqrt{78.228^2 - 15^2} = \underline{1.148}$$

$L_0 - U_1 = \sqrt{6.70^2 + 5.00^2} = 69.8900 = \underline{8.360}$

$L_2 - U_3 = \sqrt{8.658^2 + 5.000^2} = 99.960964 = 9.998$

$L_4 - U_5 = \sqrt{9.30^2 + 5.00^2} = 111.4900 = \underline{1.559}\ 5$

$U_1 - U_2 = \sqrt{1.148^2 + 5.00^2} = 26.317904 = 5.130$

$U_2 - U_3 = \sqrt{.810^2 + 5.00^2} = 25.656\,1 = 5.065$

$U_3 - U_4 = \sqrt{.482^2 + 5.00^2} = 25.232324 = 5.023$

$U_4 - U_5 = \sqrt{.160^2 + 5.00^2} = 25.0256 = 5.003$

$$A = \frac{500\times 7.848}{5.130} = 7.649$$

$$B = \frac{39.240}{5.065} = 7.747$$

$$C = \frac{500\times 9.140}{5.023} = 9.098$$

$$D = \frac{\quad}{5.003} = 9.135$$

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(2)

Diagonal

$L_0 - u_1 = \dfrac{4.5 \times 8360}{6,70} = 5,615$ C

(2)

$u_1 - L_2 = 3.5 \times \left(1 - \dfrac{1,148}{7,848}\right) \times \dfrac{8360}{6,70} = .854 \quad 3,730$ T

$u_2 - u_3 = 2.5 \left(1 + \dfrac{.810}{8658}\right) \times \dfrac{19,998}{8658} = 1.094 \quad 3,158$ C

$u_3 - L_4 = 1.5 \left(1 - \dfrac{.482}{9140}\right) \times \dfrac{9998}{8658} = .947 \quad 1,640$ T

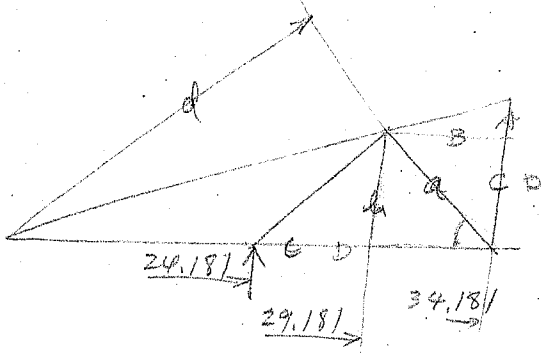
$u_4 - u_5 = 0.5 \left(1 + \dfrac{.160}{9,300}\right) \times \dfrac{10,559}{930} = 1.017 \quad ,577$ C

$\dfrac{108.815 \quad 79,630}{4.5 \times 24,181 - 29,181} = 2,907$
$27,393$

$\dfrac{172,998 \quad 81,110}{4.5 \times 38,444 - 43,444 - 48,444} = 1,933$
$41,951$

$\dfrac{336,659 \quad 82,22}{4.5 \times 74,813 - 79,813 - 84,813 - 89,813} = 1,001$
$82,106$

$\dfrac{1,195,313 \quad 82,513}{4.5 \times 265,625 - 270,625 - 275,625 - 280,625 - 285,625} = \dfrac{\cancel{.324}}{329}$
$\cancel{255,972}$ $251,568$

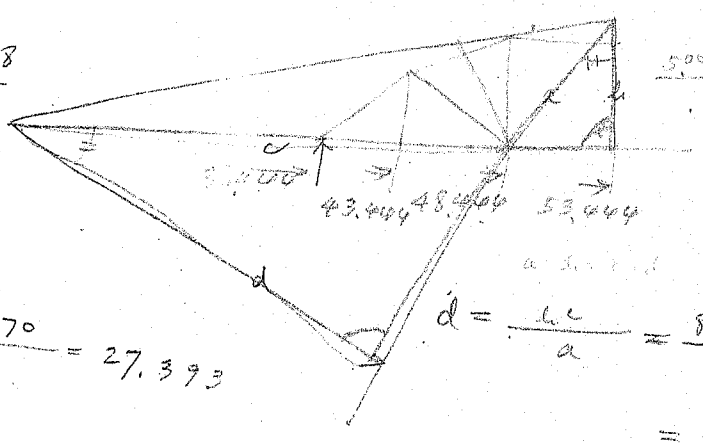


A : B = C : D

$D = \dfrac{Bc}{A} = \dfrac{5,00 \times 7,848}{1,148} = 34,181$

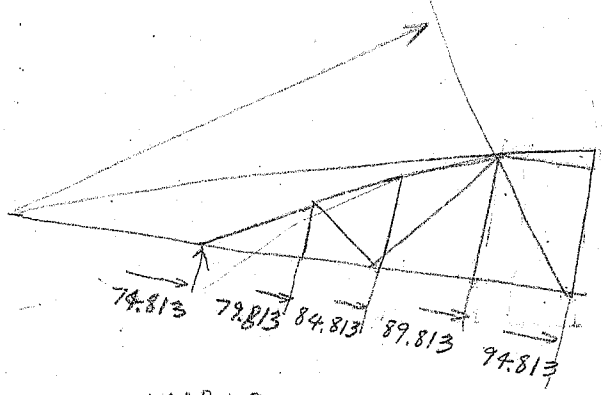
A = b = c = d

$d = \dfrac{b.c}{a} = \dfrac{34,181 \times 670}{8360} = 27,393$



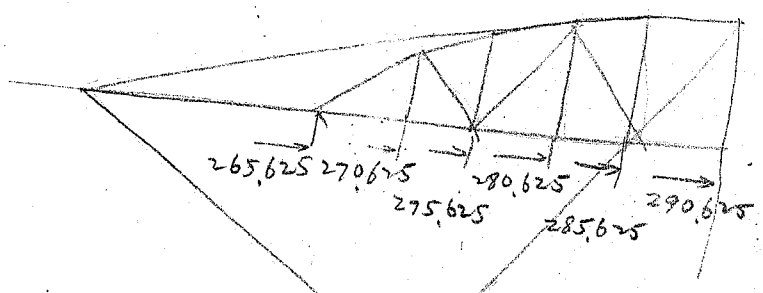
$\dfrac{5,00 \times 8658}{,810} = 59,444$

$d = \dfrac{b.c}{a} = \dfrac{8,658 \times 48,444}{9,998} = 41,951$



$\dfrac{5,00,0 \times 9140}{,482} = 94,813$

$d = \dfrac{8,658 \times 94,813}{9,998} = 82,106$



$\dfrac{5,00 \times 9300}{,160} = 290,625$

$d = \dfrac{9300 \times \cancel{290,625}\ 285,625}{10,559} = \cancel{255,972}\ 251,568$

$u_2 - L_2 = \dfrac{529 \times 1148}{5130} = 1,170$
$\dfrac{5163}{\cancel{5,037} \times ,810} = \dfrac{826}{\cancel{815}}$
5065
$1,\cancel{355}\ T$
344

$u_4 - L_\phi = \dfrac{6595 \times ,482}{5023} = \dfrac{6,33}{\cancel{937}}$

$\dfrac{6568 \times ,160}{5003} = ,216$
$\cancel{,727}\ T$
$,423$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

Diagonal for unif. line load

u_1-L_2 $\frac{\times 24181(1+2+3\approx 8)}{10\times 27.393} = 3.178$

$u_2-u_3\ \frac{(1-7)\times 38.404}{10\times 41.951} = 2.566$

$u_3-L_4\ \frac{(1-6)\times 74813}{10\times 82.106} = 1.913$

$u_4-u_5\ \frac{(1-5)\times 265.625}{10\times 251.569} = 1.584$

$u_5-L_4'\ \frac{(1-4)\times 315.625}{10\times 251.569} = 1.255$

for sigh conc.

$L_0-u_1\ \frac{9\times 8.360}{10\times 670} = 1.123$

$u_1-L_2\ \frac{8\times 24181}{10\times 27.393} = .706$

$L_2-u_3\ \frac{7\times 38.404}{10\times 41.951} = .641$

$u_3-L_4\ \frac{6\times 74.813}{10\times 82.106} = .547$

$L_4-u_5\ \frac{5\times 265.625}{10\times 251.569} = .528$

$u_5-L_4'\ \frac{4\times 315.625}{10\times 251.569} = .502$

line load stress (single conc.)

$u_1-u_2\ \frac{8\times 10}{10\times 7.649} = 1.046$ $L_0-L_2\ \frac{9\times 5}{10\times 6.70} = .672$

$u_2-u_3\ \frac{8\times 10}{10\times 7.749} = 1.032$ $L_2-L_4\ \frac{7\times 15}{10\times 8.658} = 1.213$

$u_3-u_4\ \frac{6\times 20}{10\times 9.098} = 1.319$ $L_4-L_5\ \frac{5\times 25}{10\times 9.30} = 1.344$

$u_4-u_5\ \frac{6\times 20}{10\times 9.135} = 1.314$

u_2-L_2 $\frac{1.046\times 1.148}{5.130} = .234$

$\frac{1.032\times .810}{5.065} = .165$

.069

u_0-L_4 $\frac{1.319\times .482}{5.023} = .127$

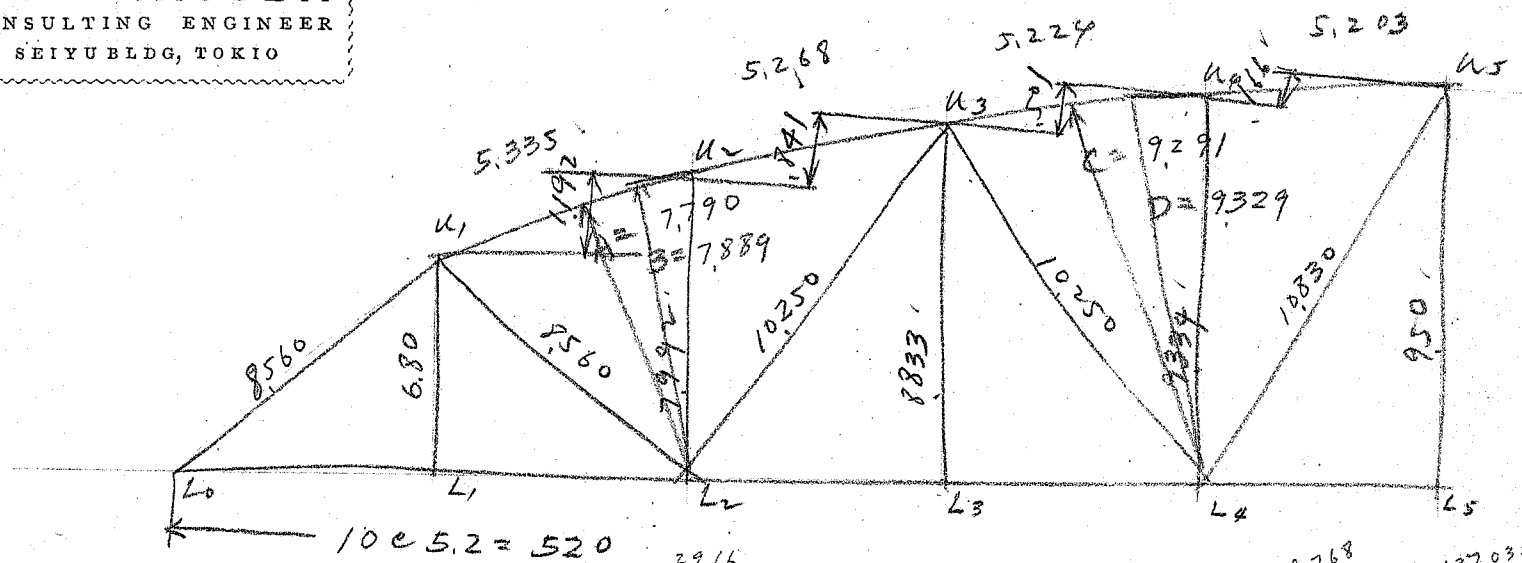
$\frac{1.314\times .160}{5.003} = .042$

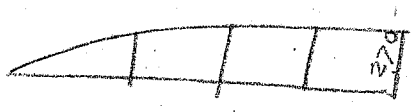
.085

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

(4)





$$R = \frac{4 \times 2.7^2 + 47.6^2}{8 \times 2.70} = 81.468$$

$$y = 270 - 81.468 + \sqrt{81.468^2 - 5.2^2} + \sqrt{81.468^2 - 10.4^2} + \sqrt{81.468^2 - 15.6^2}$$

$L_0 - u_1$ $5.2^2 = 27.04$, $6.8^2 = 46.24$, $73.28 = 8.560$

$L_2 - u_3$ $5.2^2 = 27.04$, $8833^2 = 78.0219$, 105.0619, 10.250

$L_0 - u_5$ $5.2^2 = 27.04$, $9.5^2 = 90.25$, 117.2900, 10.817, 10.830

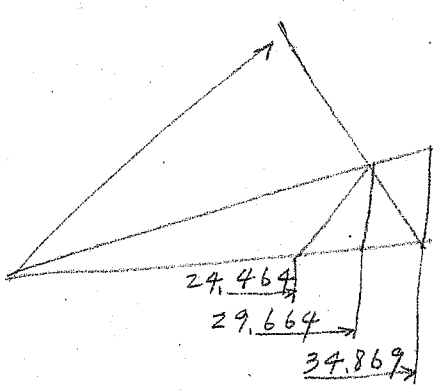
$u_1 - u_2$ $5.2 = 27.04$, $1192 = 1.4209$, $28.4609 = 5.335$

$u_2 - u_3$ $5.2 = 27.04$, $.841 = .7073$, $27.7473 = 5.268$

$u_3 - u_4$ $5.2 = 27.04$, $.501 = .2510$, $27.2910 = 5.224$

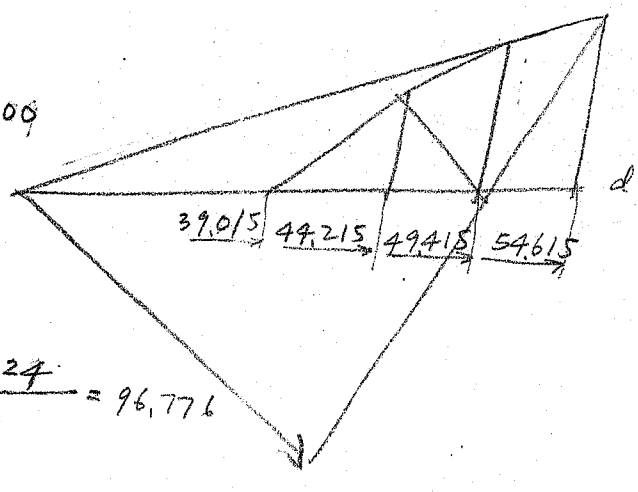
$u_4 - u_5$ $5.2 = 27.04$, $.166 = .0276$, $27.0676 = 5.203$

$A = \frac{5.2 \times 7992}{5335} = 7.790$ $B = \frac{5.2 \times 7992}{5268} = 7.889$ $C = \frac{5.2 \times 9334}{5.224} = 9.291$ $D = \frac{5.2 \times 9334}{5.203} = 9.329$



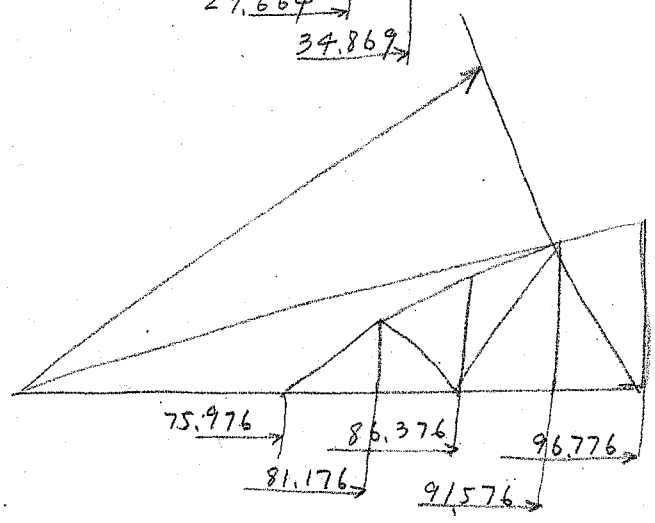
$\frac{5.2 \times 7992}{1192} = 34.864$

$\alpha = \frac{6.8 \times 34.869}{8.560} = 27.709$



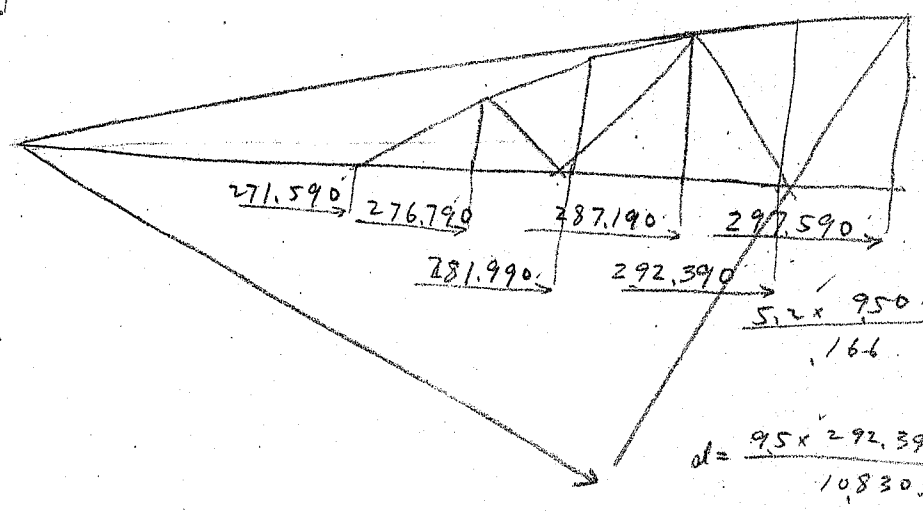
$\frac{5.2 \times 8833}{.841} = 54.615$

$d = \frac{8,833 \times 49.415}{10,250} = 42.584$



$\frac{5.2 \times 9324}{.501} = 96.776$

$d = \frac{8833 \times 96.776}{10,250} = 83.397$



$\frac{5.2 \times 950}{.166} = 297.590$

$d = \frac{95 \times 292.390}{10,830} = 256.453$

JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO

Dead load

u_1-u_2 $\frac{4.5\times104-5.2}{7790}=5340$ (43.80, 41.6)

u_2-u_3 $\frac{4.5\times10.4-5.2}{7889}=5.273$ (41.6)

u_3-u_4 $\frac{4.5\times208-6.0\times5.2}{9291}=6.716$ (93.6, 31.2, 62.4)

u_3-u_4 $\frac{\quad}{9329}=6,689$

L_0-L_2 $\frac{4.5\times5.2}{680}=3,441$ (23.4)

L_2-L_4 $\frac{4.5\times156-3\times5.2}{8,833}=6,181$ (70.2, 15.6, 54.6)

L_4-L_5 $\frac{4.5\times26-10\times5.2}{9.50}=6,842$ (117.0, 65)

L_0-u_4 $\frac{4.5\times8.56}{680}=5.665$ (38.52)

u_2-L_2 $\frac{5340\times1192}{5335}=1.193$

$\frac{5273\times841}{5268}=,842$

$,351$

u_1-L_2 $\frac{4.5\times29464-29664}{27.700}=2.904$ (110,088, 80,424)

u_4-L_4 $\frac{6716\times501}{5224}=,644$

$\frac{6689\times166}{5203}=,213$

$,431$

L_2-u_3 $4.5\times39015-44415=1.924$ (175.568, 81,936, 49,415, 42,584)

u_3-L_4 $4.5\times75.976-81.176-86,376-91,576=,992$ (341,892, 82,764, 83,397)

L_4-u_5 $4.5\times271.590-276,790-281,990-287,190-292,390=,327$ (1222,155, 83,795, 256,483)

Live load (unif)

u_1-L_2 $\frac{(1-\frac{36}{8})\times24.464}{10\times27.700}=3.179$

L_2-u_3 $\frac{(1-\frac{28}{7})\times39.015}{10\times42584}=2.565$

u_3-L_4 $\frac{(1-\frac{21}{6})\times75976}{10\times83397}=1913$

L_4-u_5 $\frac{(1-\frac{15}{5})\times271.590}{10\times256.483}=1.588$

u_5-L_4 $\frac{(1-\frac{10}{4})\times323.590}{10\times256.483}=$ ~~1.262~~ ~~,759~~

(Single)

u_1-u_2 $\frac{8\times10.4}{10\times7790}=1068$

u_2-u_3 $\frac{8\times10.4}{10\times7889}=1055$

u_3-u_4 $\frac{8\times208}{10\times9291}=$ $\frac{1248}{1343}$

u_4-u_5 $\frac{6\times208}{9329}=1338$

L_0-L_2 $\frac{9\times5.2}{10\times680}=,688$

L_2-L_4 $\frac{7\times15.6}{10\times8833}=1.236$

L_4-L_5 $\frac{5.\times26.}{10\times9.5}=1.367$

L_0-u_1 $\frac{9\times8.56}{10\times680}=1133$

u_1-L_2 $\frac{8\times24.464}{10\times27.70}=,707$

L_2-u_3 $\frac{7\times39.015}{10\times42584}=,641$

u_3-L_4 $\frac{6\times75.976}{10\times83.397}=,597$

L_4-u_5 $\frac{5\times271.590}{10\times256.483}=,529$

u_5-L_4 $=\frac{4\times323.590}{10\times256.483}=$ ~~,505~~ ~~,759~~

u_2-L_2

$\frac{1068\times1192}{5335}=,239$

$\frac{1055\times841}{5268}=,168$

$,071$

u_4-L_4 $\frac{1343\times501}{5224}=,129$

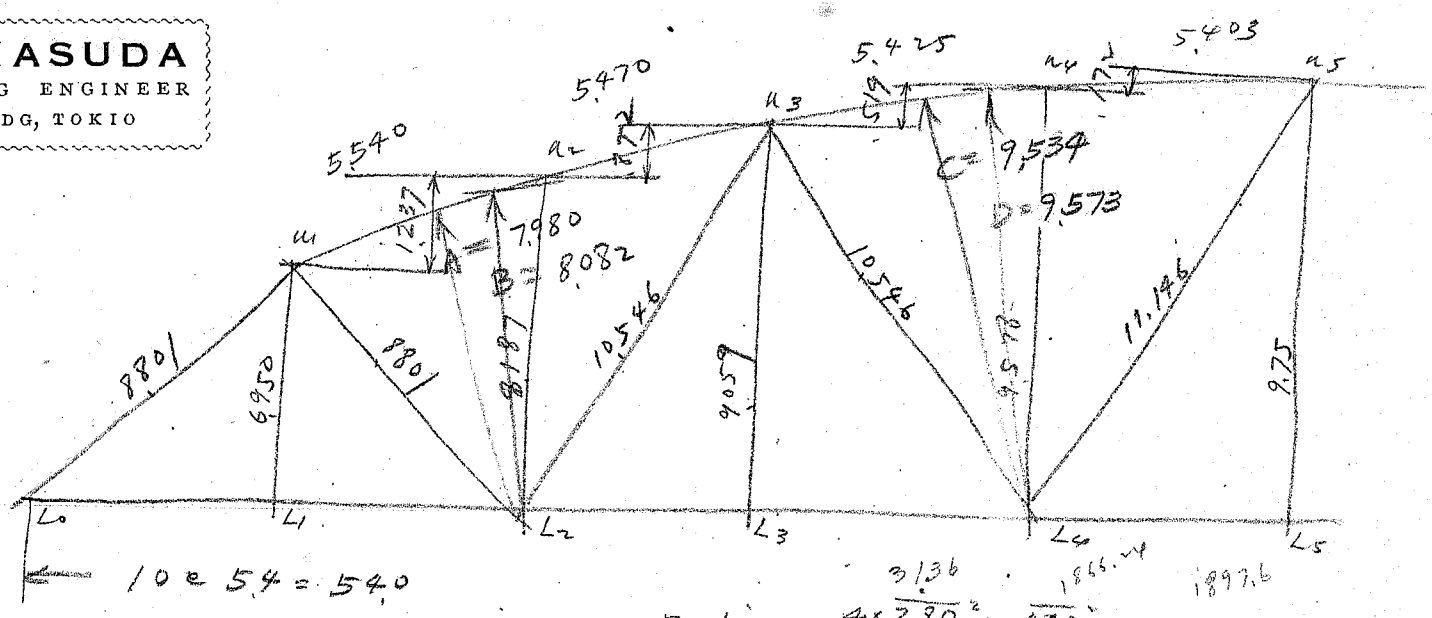
$\frac{1338\times166}{5202}=0,43$

$,086$

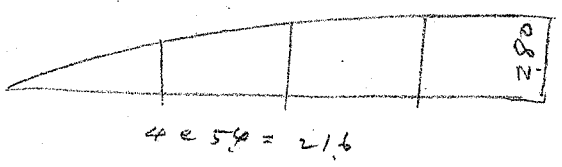
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$$\text{Radius} = \frac{4 \times 2.80^2 + 43.2^2}{8 \times 2.80} = 84.714$$



4 @ 5.4 = 21.6

$$y = b - r + \sqrt{r^2 - x^2} = 2.80 - 84.714 + \sqrt{84.714^2 - 5.4^2} = 84.542 = 2.628$$

$$+ \sqrt{84.714^2 - 10.8^2} = 84.023 \quad 2.109$$

$$+ \sqrt{84.714^2 - 16.2^2} = 83.151 \quad 1.237$$

$L_0 - u_1 = 6.95^2 = 48.3025$
$5.40^2 = 29.1600$
$77.4625 = 8.801$

$L_2 - u_3 \quad 9.059^2 = 82.0655$
$5.40^2 = 29.1600$
$111.2255 = 10.546$

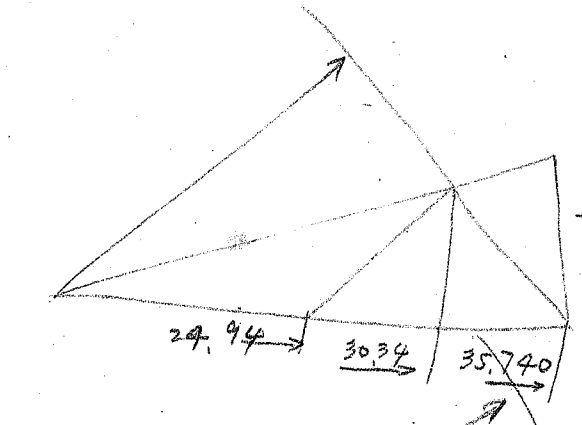
$L_0 - u_5 = 9.75^2 = 95.0625$
29.16
$124.2225 = 11.146$

$u_1 - u_2 \quad 5.4^2 = 29.1600$
$1.237 = 1.5302$
$30.6902 \text{ — } 5.540$

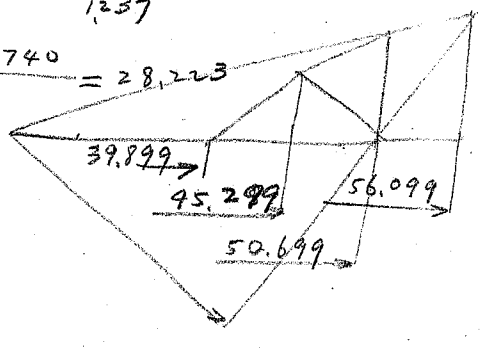
$u_2 - u_3 \quad 29.1600$
$.872 = .7604$
$29.9204 \text{ — } 5.470$

$u_3 - u_4 \quad 29.1600$
$.519 \quad .2694$
$29.4294 \text{ — } 5.425$

$u_4 - u_5 \quad 29.1600$
$.172 \quad .0296$
$29.1896 \text{ — } 5.403$



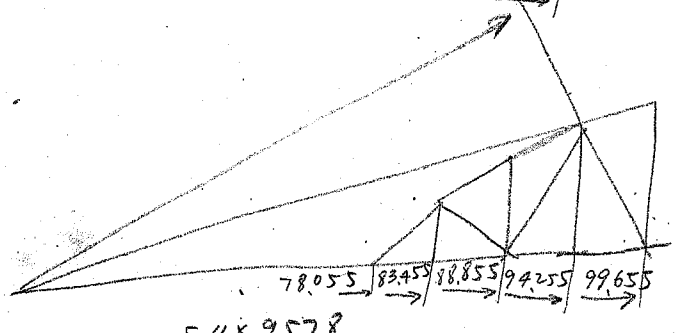
$$\frac{5.4 \times 8.187}{1.237} = 35.740$$



$$\frac{6.95 \times 35.740}{8.801} = 28.223$$

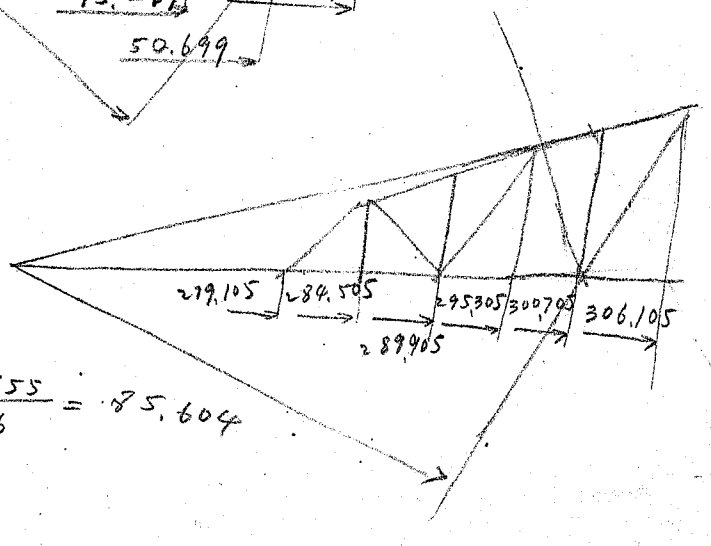
$$\frac{54 \times 9059}{.872} = 56.099$$

$$\frac{9.059 \times 50.699}{10.546} = 43.550$$



$$\frac{5.4 \times 9.578}{.519} = 99.655$$

$$\frac{9.059 \times 99655}{10.546} = 85.604$$



$$\frac{5.4 \times 9.75}{.172} = 306.105$$

$$\frac{9.75 \times 300.705}{11.146} = 263.042$$

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L_0

$\frac{54\times 8{,}187}{5540} = 7{,}980$ $\quad \frac{5.4\times 9578}{5425} = 9{,}534$ $\quad L_0-u_1 = \frac{4.5\times 8801}{695} = 5{,}699$

$\frac{5.4\times 8187}{5470} = 8{,}082$ $\quad \frac{5.4\times 9578}{5403} = 9{,}573$ $\quad u_1-L_2 = \frac{4.5\times 24.94 - 30340}{28{,}223} = 2902$

$L_2-u_3 = \frac{4.5\times 39.899 - 43.299 - 50{,}699}{43{,}550} = 1.918$

Dead Load stress

$u_0-u_2 = \frac{4.5\times 1.08 - 5.4}{7980} = 5{,}414$ $\quad L_0-L_2 \ \frac{4.5\times 5.4}{695} = 3.496$ $\quad U_3-L_4 = \frac{4.5\times 78{,}055 - 83{,}455 - 88{,}855}{85{,}604} = .989$

$u_2-u_3 = \frac{\qquad}{8082} = 5{,}345$ $\quad L_2-L_4 \ \frac{4.5\times 16.2 - 5.4\times 3}{9059} = 6.259$

$u_3-u_4 = \frac{4.5\times 21.6 - 5.4\times 6.0}{9539} = 6.799$ $\quad L_4-L_5 \ \frac{4.5\times 27.0 - 5.4\times 10}{9.75} = 6.923$ $\quad u_4-u_5 = \frac{4.5\times 279{,}105 - 284{,}505 - 289{,}905 - 295{,}305 - 300{,}705}{263{,}042} = .325$

$u_5-a_5 = \frac{\qquad}{9573} = 6.769$

$u_2-L_2 \ \frac{5414\times 1237}{5540} = 1{,}209$ $\quad u_4-L_4 \ \frac{6799\times 519}{5425} = .650$

$\frac{5345\times 872}{5470} = .852$ $\quad \frac{6769\times 172}{5403} = .215$

357 $\qquad$ 435

Live load (uniform)

$u_1-L_2 \ \frac{(1\sim 8)\times 24.94}{10\times 28{,}223} = 3.191$

$L_2-u_3 \ \frac{(1\sim 7)\times 39.899}{10\times 43{,}550} = 2.565$

$u_3-L_4 = \frac{(1-6)\times 78.055}{10\times 85{,}604} = 1.915$

$L_4-u_5 \ \frac{(1-5)\times 279.105}{10\times 263.042} = 1.592$

$u_5-L_6' \ \frac{(1-6)\times 333105}{10\times 263{,}042} = 1.266$

(single)

$u_1-u_2 = \frac{8\times 1.08}{10\times 7.980} = 1.083$ $\quad L_0-L_2 = \frac{9\times 5.4}{10\times 6.95} = .699$

$u_2-u_3 = \frac{8\times 108}{10\times 8082} = 1.069$ $\quad L_2-L_4 = \frac{7\times 162}{10\times 9059} = 1.256$

$u_3-u_4 = \frac{6\times 216}{10\times 9534} = 129.6 = 1.359$ $\quad L_4-L_5 = \frac{5\times 27.0}{10\times 9.75} = 1.385$

$u_4-u_5 = \frac{8\times 216}{10\times 9573} = 1.354$

(single)

$L_0-u_1 = \frac{9\times 8801}{10\times 6950} = 1.140$

$u_1-L_2 = \frac{8\times 24.94}{10\times 28223} = .707$

$L_2-u_3 = \frac{7\times 39.899}{10\times 43.550} = .641$

$u_3-L_4 = \frac{6\times 78055}{10\times 85.604} = .547$

$L_4-u_5 = \frac{5\times 279.105}{10\times 263.042} = .531$

$u_5-L_4' = \frac{4\times 333105}{10\times 263042} = .507$

u_2-L_2

$\frac{1083\times 1237}{5540} \quad .242$

$\frac{1069\times 872}{5470} \quad .170$

.072

u_4-L_4

$\frac{1359\times 519}{5425} \quad .130$

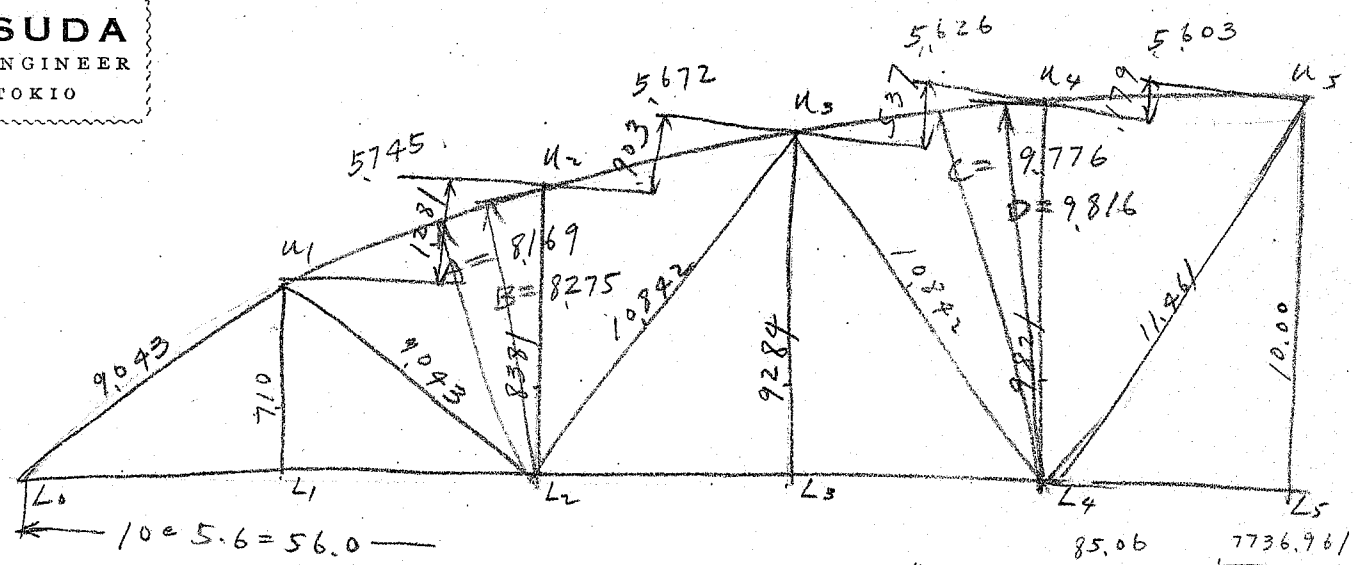
$\frac{1354\times 172}{5403} \quad .043$

.087

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$R = \frac{4\times 2.90^2 + 4.48^2}{8\times 2.90} = 87.960$

$y = 2.90 - 87.960 + \sqrt{87.960^2 - 5.6^2} + \sqrt{87.960^2 - 11.2^2} + \sqrt{87.960^2 - 16.8^2} = 87.781 + 87.244 + 86.341$

$L_0 - U_1$: $5.6^2 = 31.36$, $7.1^2 = 50.41$, $81.77 = 9.043$

$L_2 - U_3$: 5.6×31.36, 9.284×86.1927, $117.5527 = 10.842$

$L_4 - U_5$: 5.6×31.36, $10. \times 10.0$, $131.36 = 11.461$

$U_1 - U_2$: 5.6×31.36, 1.281×1.6410, 33.0010, 5.745

$U_2 - U_3$: 31.36, $.903 \times .8154$, 32.1754, 5.672

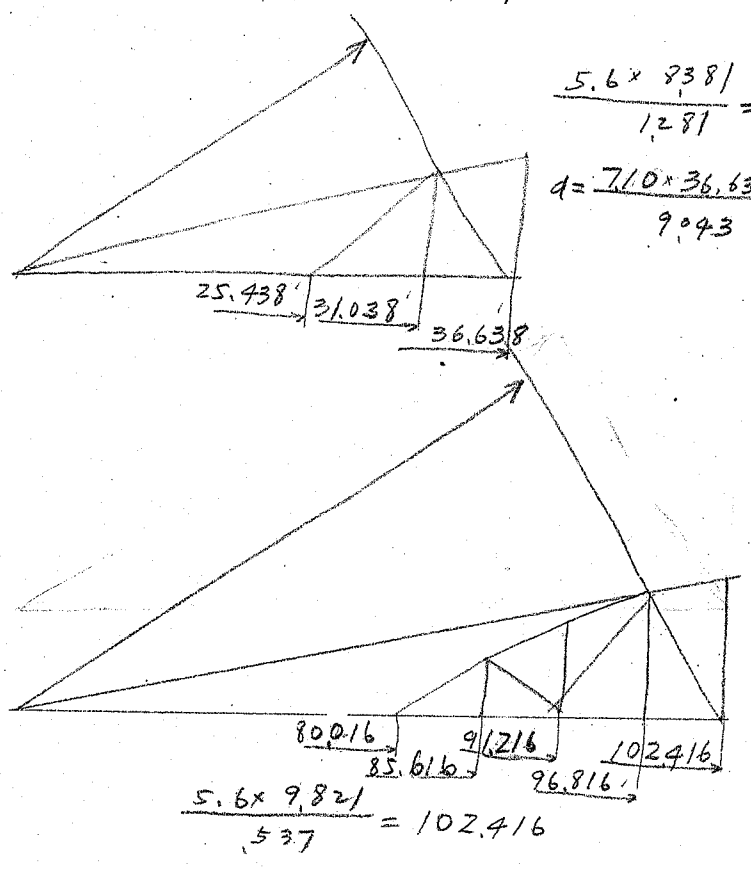
$U_3 - U_4$: 31.36, $.537 \times .2884$, 31.6484, 5.626

$U_4 - U_5$: 31.36, $.179 \times .0320$, 31.3920, 5.603

$A = \frac{5.6\times 8.381}{5.745} = 8.169$ (46.9336)

$B = \frac{5.6\times 8.381}{5.672} = 8.275$

$C = \frac{5.6\times 9.821}{5.626} = 9.776$ (54.9976) $D = \frac{5.6\times 9.821}{5.603} = 9.816$

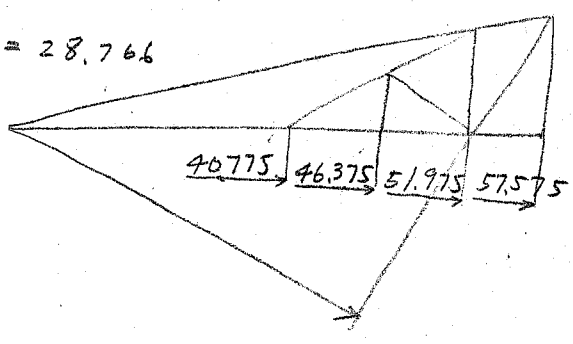


$\frac{5.6\times 8.381}{1.281} = 36.638$

$d = \frac{7.10\times 36.638}{9.043} = 28.766$

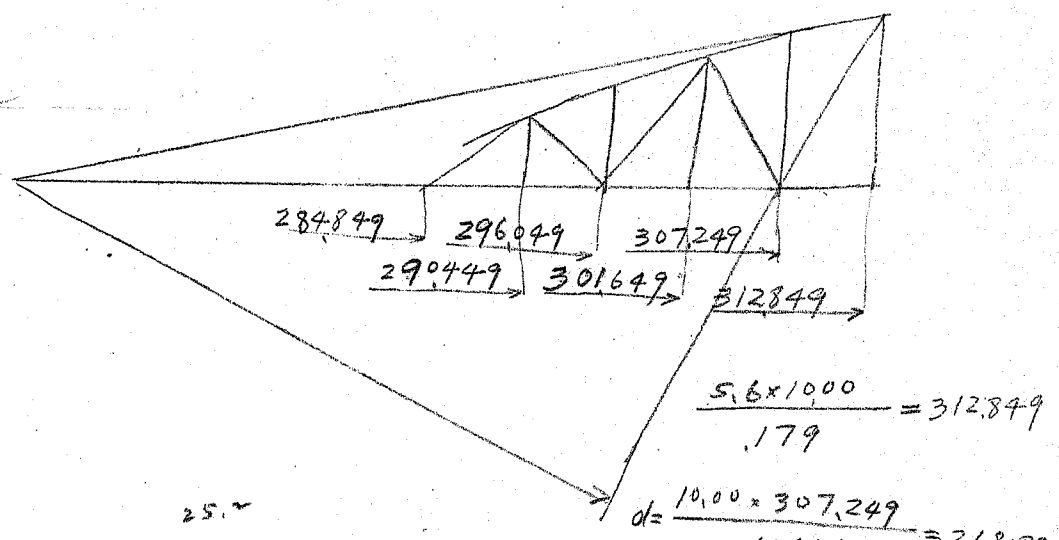
$\frac{5.6\times 9.821}{.537} = 102.416$

$d = \frac{9.284\times 102.416}{10.842} = 87.699$



$\frac{5.6\times 9.284}{.903} = 57.575$

$d = \frac{9.284\times 51.975}{10.842} = 44.506$



$\frac{5.6\times 1000}{.179} = 312.849$

$d = \frac{10.00\times 307.249}{11.461} = 268.032$

Dead load stresses

$U_1 - U_2$ $\frac{4.5\times 11.2 - 5.6}{8.169} = 5.484$ (50.40, 44.8)

$U_2 - U_3$ $\frac{\quad}{8.275} = 5.414$

$U_3 - U_4$ $\frac{4.5\times 2.24 - 6\times 5.6}{9.776} = 6.874$ (100.80, 33.6, 67.2)

$U_4 - U_5$ $\frac{\quad}{9.816} = 6.846$

$L_0 - L_2 = \frac{4.5\times 5.6}{7.10} = 3.549$ (25.2)

$L_2 - L_4 = \frac{4.5\times 16.8 - 3\times 5.6}{9.284} = 6.333$ (75.6, 16.8, 58.8)

$L_4 - L_5 = \frac{4.5\times 28.0 - 10\times 5.6}{10.0} = 7.000$ (126, 70)

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Dead Load stress

L_0-U_1 $\frac{4.5\times 9{,}043}{7{,}10} = 5.731$

U_1-L_2 $\frac{4.5\times 25{,}438 - 31.038}{28{,}766} = 2{,}900$ (114.471; 83.433; 55.138)

L_2-U_3 $\frac{4.5\times 40{,}775 - 51{,}975}{44{,}506} = 1{,}913$ (183,488; 46375)

U_3-L_4 $\frac{4.5\times 80{,}016 - 85.611 - 91.216 - 96.816}{87.699} = .985$ (360.072; 86.424)

L_4-U_5 $\frac{4.5\times 284{,}849 - 290{,}449 - 296{,}049 - 301.649 - 307249}{268{,}082} = 322$ (181.821; 86.425)

Live Load stress (unif)

U_1-L_2 $\frac{36.\times 25.438}{10\times 28.766} = 3{,}184$

L_2-U_3 $\frac{28.\times 40{,}775}{10\times 44{,}506} = 2{,}565$

U_3-L_4 $\frac{21\times 80016}{10\times 87{,}699} = 1.916$

L_4-U_5 $\frac{15\times 284.849}{10\times 268.082} = 1{,}594$

U_5-L_6' $\frac{10\times 340{,}849}{10\times 268.082} = 1.271$ ~~1.065~~

Live Load (single)

U_1-U_2 $\frac{7\times 112}{10\times 8169} = 1.097$ (89.6)

U_2-U_3 $\frac{8\times 112}{10\ 8{,}275} = 1083$

U_3-U_4 $\frac{6\times 224}{10\times 9776} = 1{,}375$ (134.4)

U_4-U_5 $\frac{6\times 224}{10\times 9816} = 1.369$

L_0-L_2 $\frac{9\times 5.6}{10\times 7.1} = .710$ (50.4)

L_2-L_4 $\frac{7\times 168}{10\times 9284} = 1.267$ (117.6)

L_4-L_5 $\frac{5\times 28.0}{10\times 10.00} = 1.400$ (140)

U_2-L_2 $\frac{5484\times 1281}{5745} = 1.223$

$\frac{5414\times 903}{5672} = .862$

.361

U_4-L_4 $\frac{6846\times 537}{5626} = .656$

$\frac{6846\times 179}{5603} = .219$

.437

L_0-U_1 $\frac{9\times 9043}{10\times 7.1} = 1.146$

U_1-L_2 $\frac{8\times 25438}{10\times 28.766} = .707$

L_2-U_3 $\frac{7\times 40775}{10\times 44506} = .641$

U_3-L_4 $\frac{6\times 80016}{10\times 87.699} = .547$

L_4-U_5 $\frac{105\times 284.849}{10\times 268.082} = 531$

U_5-L_6' $\frac{4\times 340.849}{10\times 268.082} = .509$ ~~.445~~

U_2-L_2 $\frac{1097\times 1281}{5745} = .245$

$\frac{1083\times 903}{5672} = .172$

.073

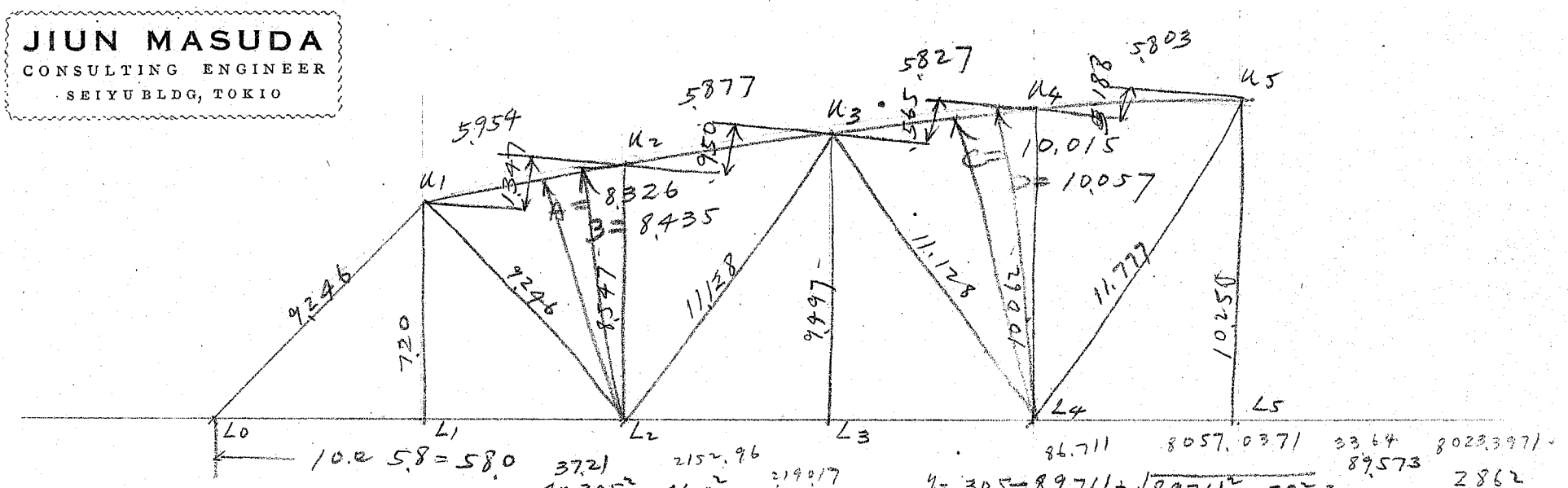
U_4-L_4 $\frac{1375\times 537}{5626} = .131$

$\frac{1369\times 179}{5603} = .044$

.087

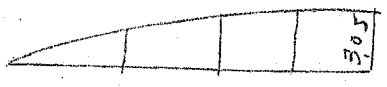
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CONSULTING ENGINEER
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$10.2\ 5.8 = 58.0$

$R = \dfrac{4\times 3.05^2 + 46.4^2}{8\times 3.05} = 89.761$



$y = 3.05 - 89.761 + \sqrt{89.761^2 - 5.8^2} = 89.573$

$+\sqrt{89.761^2 - 11.6^2} = 89.008$

$+\sqrt{89.761^2 - 17.4^2} = 88.058$

L_0–U_1
7.2 — 51.84
5.8 — 33.64
8.548 — 9.246

U_1–U_2
5.8 — 33.64
1.347 — 1.8144
35.4544 — 5.954

L_2–U_3
9.497 — 90.1930
5.800 — 33.6400
123.8330 — 11.128

U_2–U_3
5.8 — 33.6400
.25 — .9025
34.5425 — 5.877

L_4–U_5
10.25 — 105.0625
5.8 — 33.64
138.7025 — 11.777

U_3–U_4
5.8 — 33.6400
.565 — .3192
33.9592 — 5.827

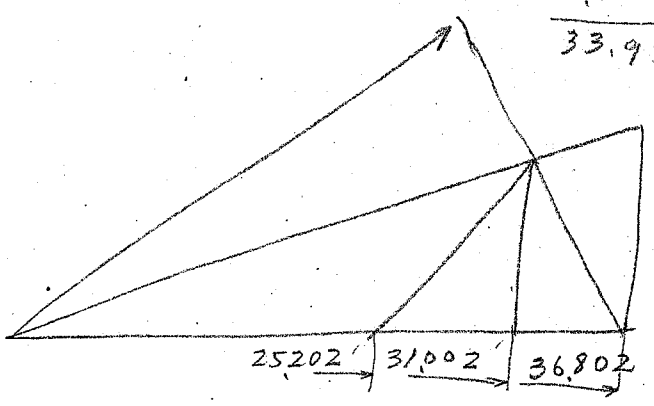
U_4–U_5
5.8 — 33.6400
.188 — 0.0353
33.6753 — 5.803

$A = \dfrac{5.8\times 8.547}{5.954} = 8.326$

$B = \dfrac{5.8\times 8.547}{5.877} = 8.435$

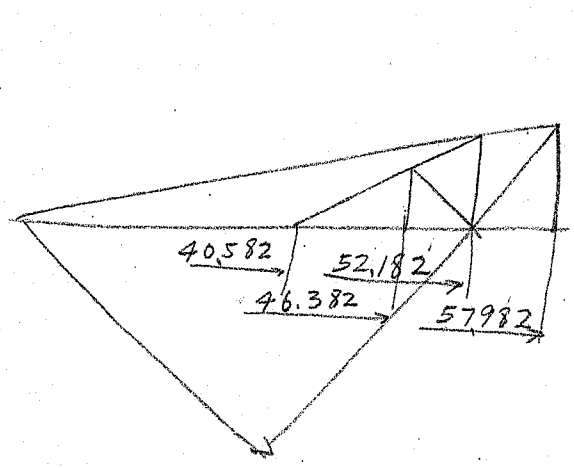
$C = \dfrac{5.8\times 10.062}{5.827} = 10.015$

$D = \dfrac{5.8\times 10.062}{5.803} = 10.057$



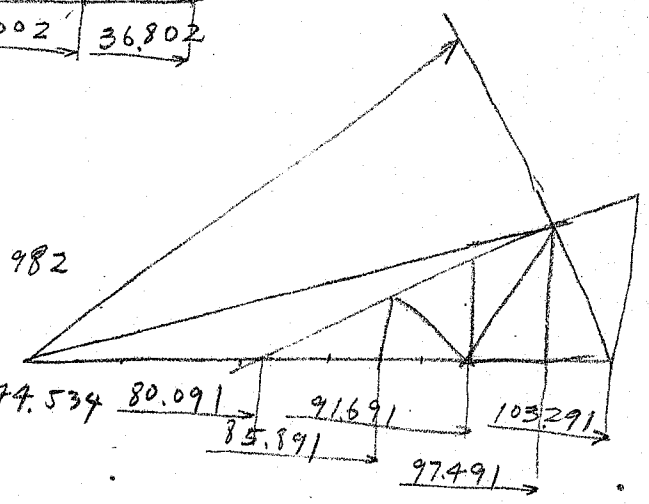
$\dfrac{5.8\times 8547}{1.347} = 36.802$

$d = \dfrac{7.20\times 36.802}{9.246} = 28.628$



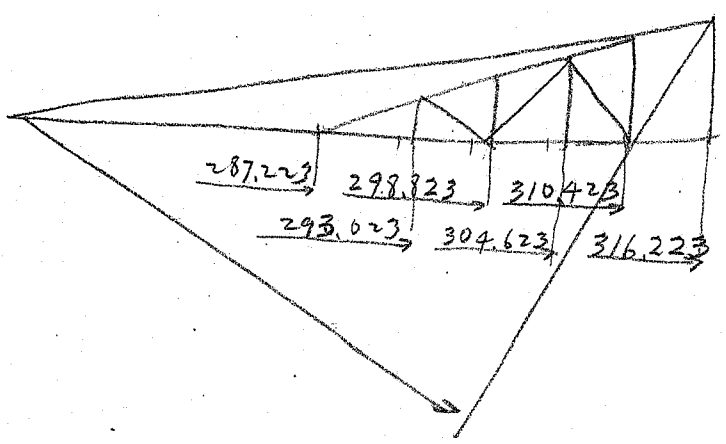
$\dfrac{5.8\times 9497}{.950} = 57.982$

$\dfrac{9.497\times 52.182}{11.128} = 44.534$



$\dfrac{5.8\times 10.062}{.565} = 103.291$

$\dfrac{9.497\times 103.291}{11.128} = 88.098$



$\dfrac{5.8\times 10250}{.188} = 316.223$

$\dfrac{10.250\times 310.423}{11.777} = 270.174$

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U_1-U_2 $\frac{4.5\times 116-5.8}{8,326}=5.573$ (52.2, 46.4)

U_2-U_3 $\frac{}{8435}=5501$

U_3-U_4 $\frac{4.5\times 232-6\times 5.8}{10,015}=6.950$ (104.4, 34.8, 69.6)

U_4-U_5 $\frac{}{10057}=6,921$

L_0-L_2 $\frac{4.5\times 5.8}{7.20}=3,625$ (26.1)

L_2-L_4 $\frac{4.5\times 17.4-3\times 5.8}{9,497}=6,360$ (78.3, 17.4, 60.4)

L_4-L_5 $\frac{4.5\times 29.0-10\times 5.8}{10,250}=7,073$ (130.5, 58., 72.5)

L_0-U_1 $\frac{4.5\times 9,246}{7,20}=5.779$ (41.607)

U_1-L_2 $\frac{4.5\times 25.202-31,002}{28,628}=2.879$ (113,409, 82,407)

$L_2-U_3=\frac{4.5\times 40582-46,382-52,182}{44.534}=1,887$ (182,619, 84,055)

$U_3-L_4=\frac{4.5\times 80,091-85,891-91,691-97,491}{88,090}=,969$ (360,410, 85,537)

$L_4-U_5=\frac{4.5\times 287,223-293,023-298,823-304,623-310,423}{270,174}=,317$ (1292.504, 85,617)

U_1-L_1 $\frac{5573\times 1347}{5954}=1,261$

$\frac{5501\times,950}{5,877}=\frac{,889}{,372}$

U_0-L_4 $\frac{6950\times,565}{5827}=,674$

$\frac{6921\times,188}{5803}=\frac{,224}{,450}$

unit. U_1-L_2 $\frac{36\times 25,202}{10\times 28,628}=3,169$

L_2-U_3 $\frac{28\times 40582}{10\times 44,534}=2,552$

U_3-L_4 $\frac{21\times 80091}{88,090}=1,909$

L_4-U_5 $\frac{15\times 287,223}{270,174}=1,595$

$L_4'-U_5$ $\frac{10\times 345,223}{10\ 270174}=1,278$

U_1-U_2 $\frac{8\times 116}{10\times 8,326}=1,115$

U_2-U_3 $\frac{92.9}{10\times 8,435}=1,100$

U_3-U_4 $\frac{6\times 23,2}{10\times 10,015}=1,390$

U_4-U_5 $\frac{6\times 232}{10\times 10,057}=1,384$ (139.2)

L_0-L_2 $\frac{9\times 5.8}{10\times 7,2}=,725$ (52.2)

L_2-L_4 $\frac{7\times 17,4}{10\times 9,497}=1,283$ (121.8)

L_4-L_5 $\frac{5\times 29,0}{10\times 10,25}=1,415$ (145)

L_0-U_1 $\frac{9\times 9,246}{10\times 7,20}=1,156$

U_1-L_2 $\frac{8\times 25,202}{10\times 28,628}=,704$

L_2-U_3 $\frac{7\times 40,582}{10\times 44,534}=,638$

U_3-L_4 $\frac{6\times 80,091}{10\times 88,090}=,546$

L_4-U_5 $\frac{5\times 287223}{10\times 270,174}=5,32$

$L_4'-U_5$ $\frac{4\times 345,223}{10\times 270.174}=1,511$

U_2-L_2 $\frac{1,115\times 1,347}{5,954}=,252$

$\frac{1110\times,950}{5,877}=\frac{,178}{,074}$

U_4-L_4 $\frac{1,390\times,565}{5827}=,135$

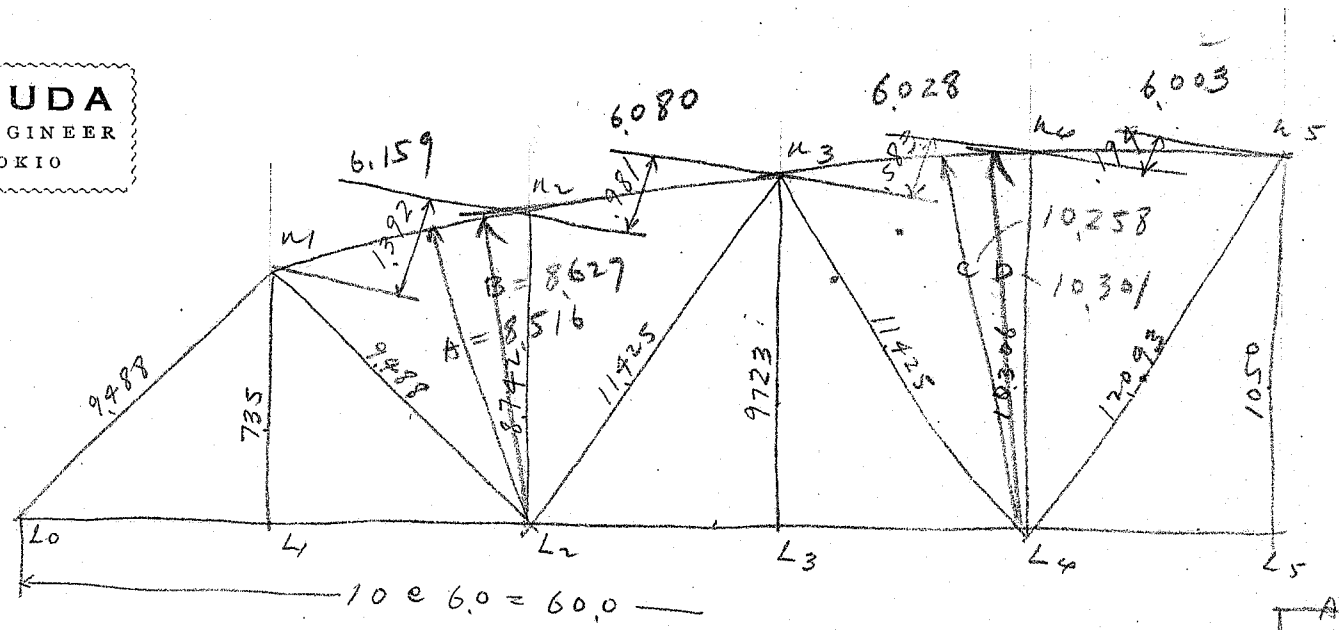
$\frac{1384\times,188}{5803}=\frac{,045}{,090}$

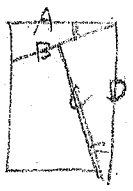
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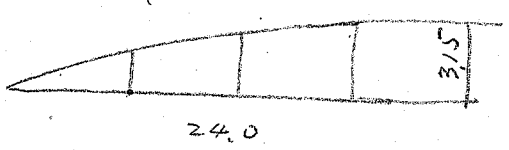
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A = B = C = D

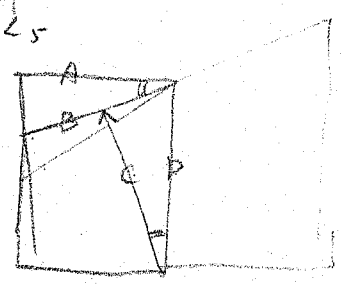
$\frac{A \times D}{B} = C$



$$\text{Radius} = \frac{4 \times 3{,}15^2 + 48.0^2}{8 \times 3{,}15} = 93.004$$

39.69 2304 2343.69

25.2



A = B = C = D

$C = \frac{A \times D}{B}$

$$f = 3{,}15 - 93{,}004 + \sqrt{93{,}004^2 - 6.0^2} = 92.810 = 2{,}956.$$

89854 8649.744

36. 8613.744

$$+ \sqrt{93{,}004^2 - 72.0^2} = 92.227 = 2{,}373.$$

144 8505.744

$$+ \sqrt{93{,}004^2 - 18.0^2} = 91.246 = 1{,}392.$$

324 8325.744

$L_0 - u_1$

$6.0^2 = 36{,}0000$
$7{,}35^2 = 54.0225$
$90.0225 \quad 9.488$

$L_2 - u_3$

$6.0^2 = 36{,}0000$
$9{,}723^2 = 94.5367$
$130.5367 \quad 11.425$

$L_4 - u_5 =$

$6.0^2 = 36{,}0000$
$10{,}50^2 = 110.2500$
$146.2500 \quad 12.093$

$u_1 - u_2$

$6.0^2 = 36{,}0000$
$1{,}392^2 = 1.9377$
$37.9377 \leftarrow 6.159$

$u_2 - u_3$

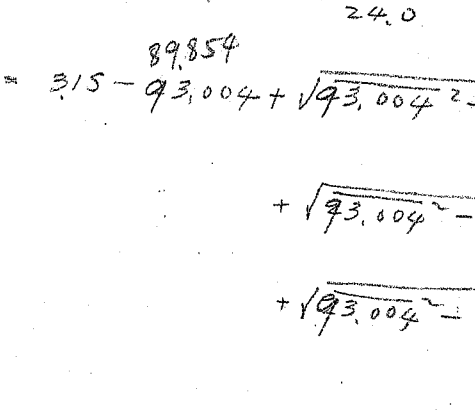
$6.0^2 = 36{,}0000$
$.981^2 = .9624$
$36.9624 \quad 6.080$

$u_3 - u_4$

$6.0^2 = 36{,}0000$
$.583^2 = .3399$
$36.3399 \quad 6.028$

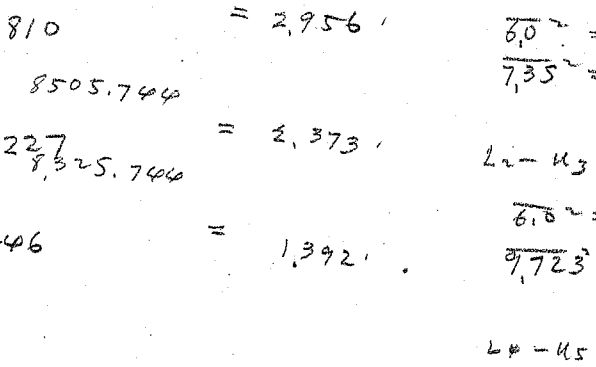
$u_4 - u_5$

$6.0^2 = 36{,}0000$
$.194^2 = .0376$
$36.0376 \quad 6.003$



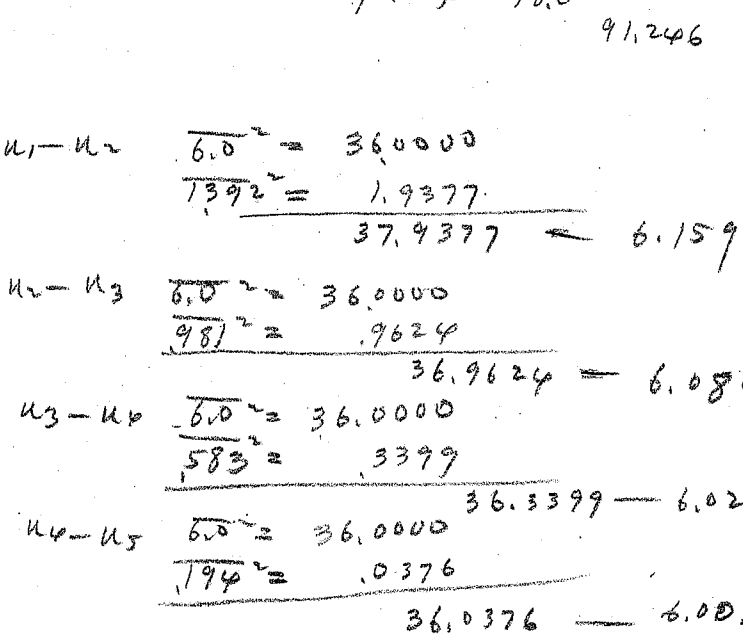
$d = \frac{7.35 \times 37.681}{9.488} = 29.20$

$\frac{6.0 \times 8742}{1.392} = 37.681$



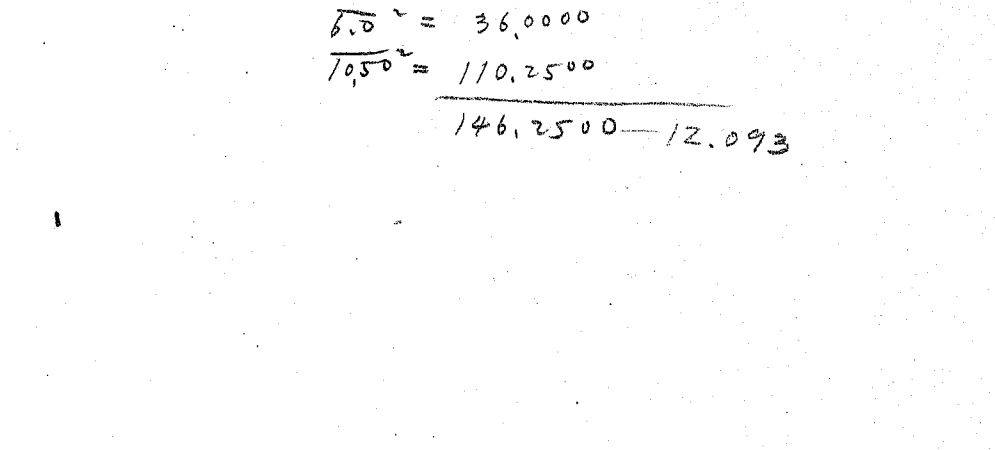
$d = \frac{9.723 \times 53.468}{11.425} = 45.503$

$\frac{6.0 \times 9.723}{.981} = 59.468$



$d = \frac{9723 \times 106.065}{11.425} = 90.264$

$\frac{6.0 \times 10306}{.583} = 106.065$



$d = \frac{105^0 \times 318.742}{12.093} = 279.390$

$\frac{6.0 \times 1050}{.194} = 324.742$

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$A = \frac{600 \times 8,742}{6,159} = 8,516$

$B = \frac{600 \times 8742}{6080} = 8,627$

$C = \frac{600 \times 10,306}{6028} = 10,258$

$D = \frac{600 \times 10306}{6003} = 10,301$

Top chord (dead)

$U_1 - U_2 \quad \frac{4.5 \times 12.0 - 6.0}{8,516} = 5.636$

$U_2 - U_3 \quad \frac{4.5 \times 12.0 + 6.0}{8627} = 5.564$

$U_3 - U_4 \quad \frac{4.5 \times 24.0 - (1+2+3) \times 6}{10,258} = 7.019$

$U_4 - U_5 \quad \frac{}{10,301} = 6.990$

Bottom chord (dead)

$L_0 - L_2 \quad \frac{4.5 \times 6.0}{7.35} = 3.673$

$L_2 - L_4 \quad \frac{4.5 \times 18.0 - 3.0 \times 6.0}{9723} = 6.479$

$L_4 - L_5 \quad \frac{4.5 \times 30.0 - 10 \times 6.0}{1050} = 7.143$

Diagonal (dead)

$L_0 - U_1 \quad \frac{4.5 \times 9.488}{735} = 5.809$ ✓

$U_1 - L_2 \quad \frac{4.5 \times 25.681 - 31.681}{29.201} = 2.873$

$L_2 - U_3 \quad \frac{4.5 \times 41.468 - 47.468 - 53468}{45.503} = \frac{1.833}{2.413}$

$U_3 - L_4 \quad \frac{4.5 \times 82.065 - 88065 - 94065 - 100.065}{90.264} = .965$ ✓

$L_4 - U_5 \quad \frac{4.5 \times 294.742 - 300.742 - 306.742 - 312.742 - 318.742}{279,390} = 313$ ✓

Vert. (dead)

$U_2 - L_2 \quad \frac{5.636 \times 1.392}{6159} = 1.274$

$\frac{5564 \times .981}{6080} = \frac{898}{+376}$

$U_0 - L_4 \quad \frac{7019 \times .583}{6028} = .679$

$\frac{6990 \times .194}{6003} = \frac{226}{453}$

Diagonal (single)

$L_0 - U_1 \quad \frac{6 \times 9.488}{10 \times 7.35} = 1.162$

$U_1 - L_2 \quad \frac{8 \times 25.681}{10 \times 29.201} = .704$

$L_2 - U_3 \quad \frac{7 \times 41.468}{10 \times 45.503} = .638$

$U_3 - L_4 \quad \frac{6 \times 82.065}{10 \times 90.264} = .545$

$L_4 - U_5 \quad \frac{5 \times 294.742}{10 \times 279,390} = .527$

$U_5 - L_4' \quad \frac{4 \times 294.742}{10 \times 279,390} = .422$

Diagonal

$U_1 - L_2 \quad \frac{36 \times 25.681}{10 \times 29.201} = 3.166$

$L_2 - U_3 \quad \frac{28 \times 41.468}{10 \times 45.503} = 2.552$

$U_3 - L_4 \quad \frac{21 \times 82.065}{10 \times 90.264} = 1.909$

$L_4 - U_5 \quad \frac{15 \times 294.742}{10 \times 279.390} = 1.582$

$U_5 - L_4' \quad \frac{10 \times 294.742}{10 \times 279.390} = 1.055$

Top. chord. (single)

$U_1 - U_2 \quad \frac{8 \times 12}{10 \times 8.516} = 1.127$

$U_2 - U_3 \quad \frac{8 \times 12}{10 \times 8.627} = 1.113$

$U_3 - U_4 \quad \frac{6 \times 24}{10 \times 10258} = 1.404$

$U_4 - U_5 \quad \frac{6 \times 24}{10 \times 10.301} = 1.398$

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Bottom chord (single)

L_0-L_2 $\frac{9\times 6}{10\times 7.35} = .735$

L_2-L_4 $\frac{7\times 18}{10\times 9.725} = 1.296$

L_4-L_5 $\frac{5\times 30}{10\times 1.050} = 1.429$

vert. (single)

d_2-L_2 $\frac{1127\times 1392}{6159} = .255$

$\frac{1113\times 981}{6080} = .180$

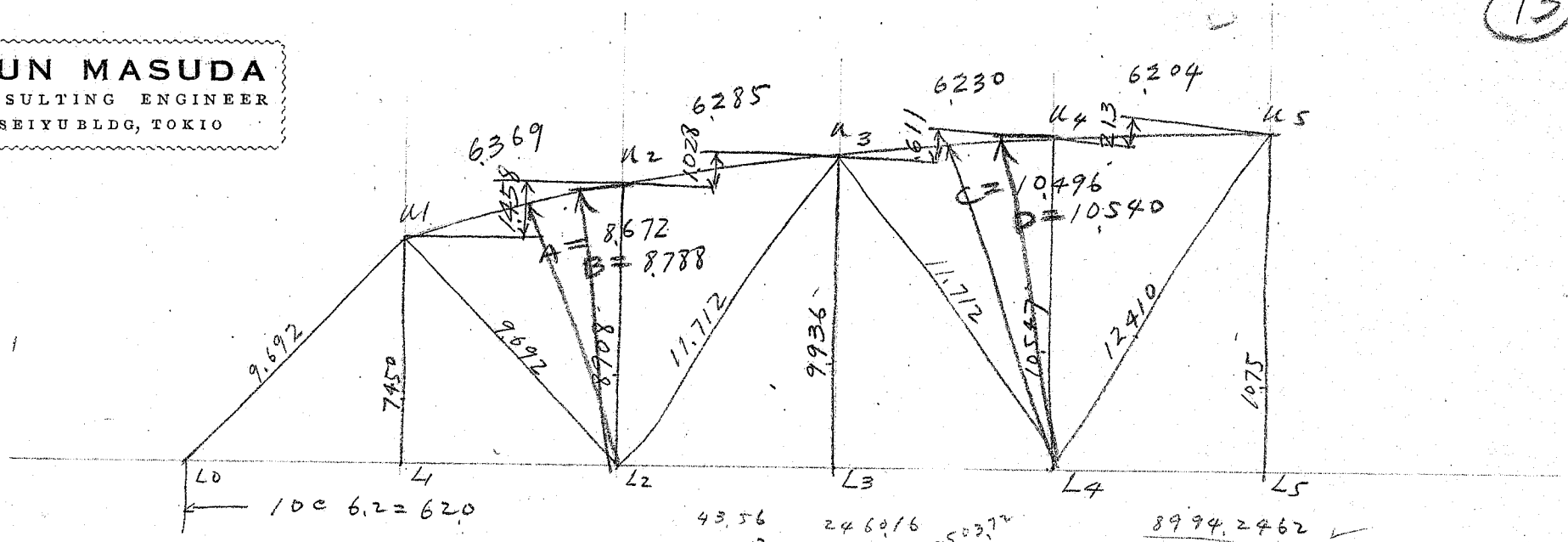
$.075$

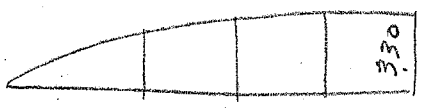
U_4-L_4 $\frac{1406\times 583}{6028} = .136$

$\frac{1398\times 194}{6003} = .045$

$.091$

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$$Radius = \frac{4\times 330^2 + 496^2}{8\times 330} = 94.838$$

$$y = 330 - 94.838 + \sqrt{94.838^2 - 6.2^2} = 91.538'$$
$$+ \sqrt{94.838^2 - 12.4^2} =$$
$$+ \sqrt{94.838^2 - 18.6^2} =$$

$L_0 - u_1$
6.2 — 38.4400
7.45 — 55.5025
93.9425 — 9.692

$L_2 - u_3$
6.2 — 38.4400
9.936 — 98.7241
137.1641 — 11.712

$L_4 - u_5$
6.2 — 38.4400
10.75 — 115.5625
154.0025 — 12.410

$u_1 - u_2$
6.2 — 38.4400
1.458 — 2.1258
405658 — 6.369

$u_2 - u_3$
6.2 — 38.4400
1.028 — 1.0568
39.4968 — 6.285

$u_3 - u_4$
6.2 — 38.4400
.611 — .3733
38.8133 — 6.230

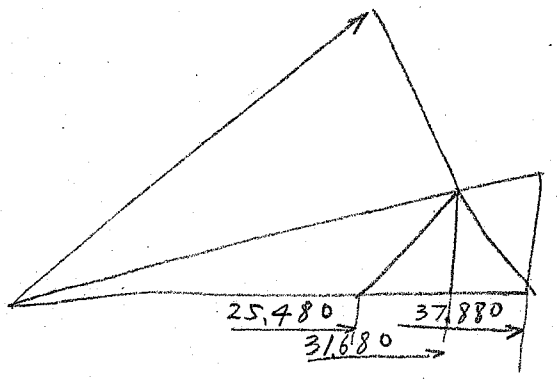
$u_4 - u_5$
6.2 — 38.4400
.213 — 0.0454
38.4854
— 6.204

$$A = \frac{6.2\times 8.908}{6.369} = 8.672$$

$$B = \frac{6.2\times 8.908}{6.285} = 8.788$$

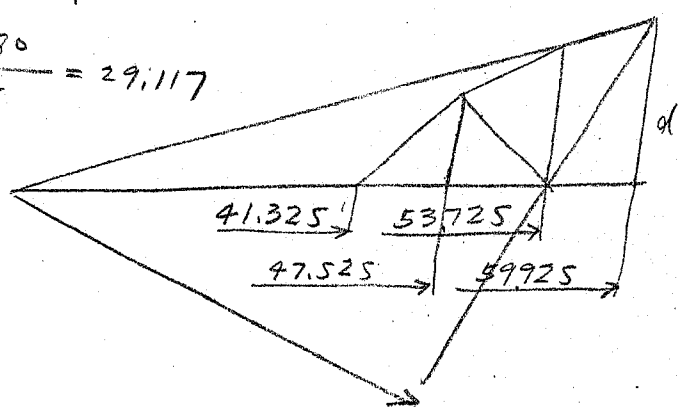
$$C = \frac{6.2\times 10.547}{6.230} = 10.496$$

$$D = \frac{6.2\times 10.547}{6.204} = 10.540$$



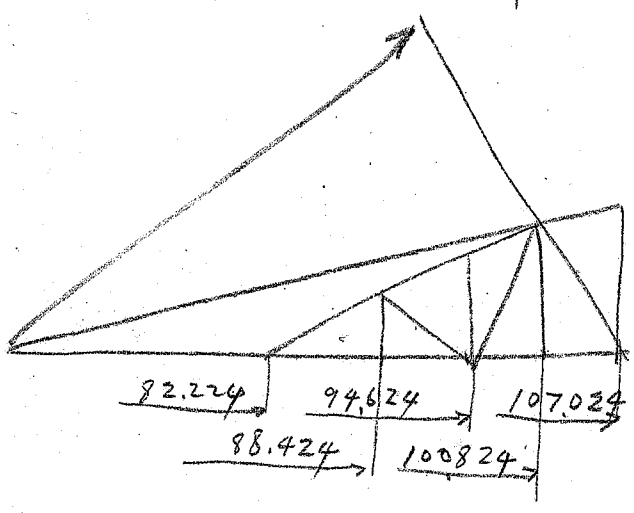
$$\frac{6.2\times 8.908}{1.458} = 37.880$$

$$d = \frac{7.45\times 37.880}{9.692} = 29.117$$



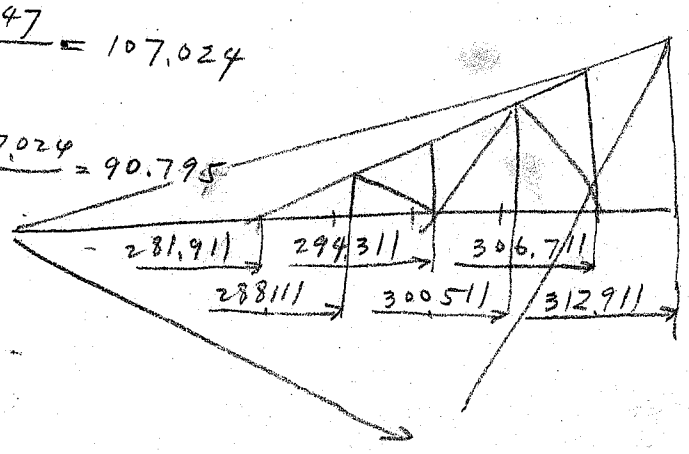
$$\frac{6.2\times 9.936}{1.028} = 59.925$$

$$d = \frac{9.936\times 53.725}{11.712} = 45.578$$



$$\frac{6.2\times 10.547}{.611} = 107.024$$

$$d = \frac{9.936\times 107.024}{11.712} = 90.795$$



$$\frac{6.2\times 10.75}{.213} = 312.911$$

$$d = \frac{10.75\times 306.711}{12.410} = 265.654$$

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CONSULTING ENGINEER
SEIYU BLDG, TOKIO

Dead Load stress

U_1-U_2 $\frac{4.5\times 12.4-6.2}{8672}=.5720$ (55.80, 49.6)

U_2-U_3 $\frac{\qquad\qquad}{8788}=.5644$

U_3-U_4 $\frac{4.5\times 24.8-6\times 6.2}{10496}=.7088$ (111.60, 37.2, 74.4)

U_4-U_5 $\frac{4.5\times}{10540}=.7059$

U_3-L_4 $\frac{4.5\times 82.224-88.424-94.624-100.824}{90795}=.949$ (370.009, 86.136)

L_4-U_5 $\frac{4.5\times 281.911-288.111-294.311-300.511-306.711}{265.684}=.297$ (1268.600, 78.952)

L_0-L_2 $\frac{4.5\times 6.2}{7.45}=3.745$ (27.9)

L_2-L_4 $\frac{4.5\times 18.6-3\times 6.2}{9.936}=6.552$ (83.70, 18.6, 65.1)

L_4-L_5 $\frac{4.5\times 31.0-10\times 6.2}{10.75}=7.209$ (139.5, 77.5)

L_0-U_1 $\frac{9\times 9.692}{10\times 7.450}=$ ~~.883~~ 1.171

U_1-L_2 $\frac{8\times 25.48}{10\times 29.117}=.700$

L_2-U_3 $\frac{7\times 41.325}{10\times 45.578}=.635$

U_3-L_4 $\frac{8\times 82.224}{10\times 90.795}=.543$

L_4-U_5 $\frac{5\times 281.911}{10\times 265.684}=.531$

U_5-L_4 $\frac{4\times 343.911}{10\times 265.684}=.518$

U_2-L_2 $\frac{5.720\times 1.458}{6.369}=1.309$

$\frac{5.644\times 1.028}{6.285}=.923$

3.81

U_4-L_4 $\frac{7.088\times .611}{6.230}=.695$

$\frac{7.059\times .213}{6.204}=.243$

.452

L_0-L_2 $\frac{4.5\times 25.480-3168}{29.117}=2.850$ (114.660, 82.98)

L_2-U_3 $\frac{4.5\times 41.325-47.525-53.725}{45.578}=1.859$ (185.963, 84.713)

U_1-U_2 $\frac{8\times 124}{10\times 8.672}=1.144$ (99.2)

U_2-U_3 $\frac{\qquad\qquad}{8.788}=1.129$

U_3-U_4 $\frac{\cancel{8}\times 248}{10\times 10.496}=1.418$

U_4-U_5 $\frac{148.8}{10540}=1.412$

L_0-L_2 $\frac{9\times 6.2}{10\times 7.45}=.749$ (55.8)

L_2-L_4 $\frac{7\times 18.6}{10\times 9.936}=1.310$ (130.2)

L_4-L_5 $\frac{5\times 31.0}{10\times 10.75}=1.442$ (155)

U_1-L_2 $\frac{8\times 2548}{10\times 29.117}=.700$

L_2-U_3 $\frac{7\times 41.325}{10\times 45.578}=.635$

U_3-L_4 $\frac{8\times 82.224}{10\times 90.795}=.543$

L_4-U_5 $\frac{5\times 281.911}{10\times 265.684}=.531$

U_5-L_4 $\frac{4\times 343.911}{10\times 265.684}=.518$

U_2-L_2 $\frac{1.144\times 1.458}{6.369}=.262$

$\frac{1.129\times 1.028}{6.285}=.185$

.077

U_4-L_4 $\frac{1.418\times .611}{6230}=.139$

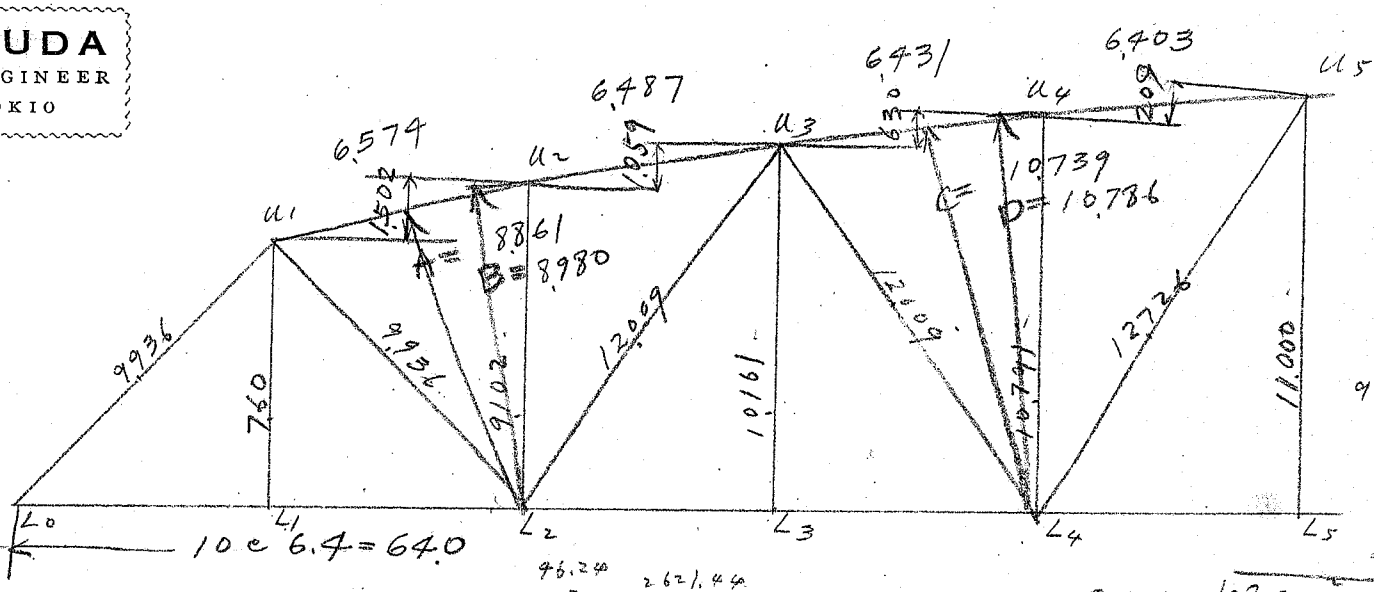
$\frac{1.412\times .213}{6204}=.048$

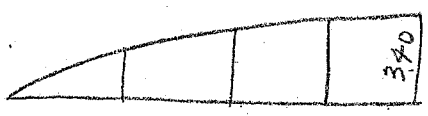
.091

増田淳氏関係資料
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$$\text{Radius} = \frac{4\times 3.4^2 + 5.12^2}{8\times 3.4} = 98.076$$

$$j = 3.40 - 98.076 + \sqrt{98.076^2 - 6.4^2} = 3.191$$

$$+\sqrt{98.076^2 - 12.8^2} = 97.257$$

$$+\sqrt{98.076^2 - 19.2^2} = 96.178$$

$L_0 - u_1$ 6.4 — 40.96
7.6 — 57.76
98.72 — 9.936

$u_1 - u_2$ 6.4 — 40.96
1.502 — 2.2560
43.2160 — 6.574

$L_2 - u_3$ 6.4 — 40.96
10.161 — 103.2459
144.2059 — 12.009

$u_2 - u_3$ 6.4 — 40.96
1.059 — 1.1215
42.0815 — 6.487

$L_4 - u_5$ 6.4 — 40.96
11.000 — 12.100
161.9600 — 12.726

$u_3 - u_4$ 6.4 — 40.96
.630 — .3969
41.3569 — 6.431

$u_4 - u_5$ 6.4 — 40.96
.209 — .0437
41.0037 — 6.403

$$A = \frac{6.4\times 9.102}{6.574} = 8.861$$

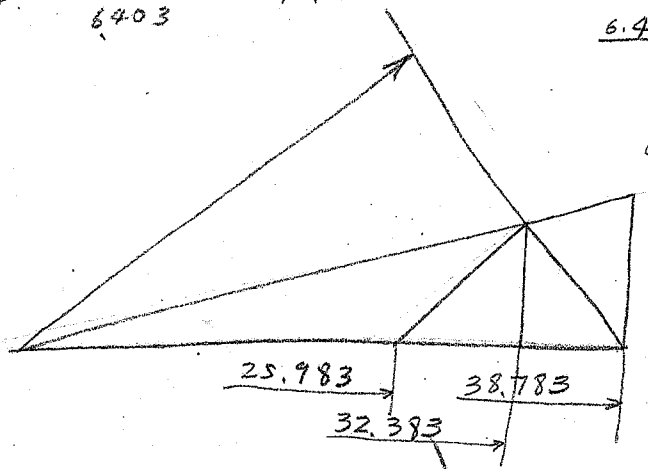
$$B = \frac{58.25\sqrt{3}}{6.487} = 8.980$$

$$C = \frac{6.4\times 10.791}{6.431} = 10.739$$

$$D = \frac{69.0624}{6.403} = 10.786$$

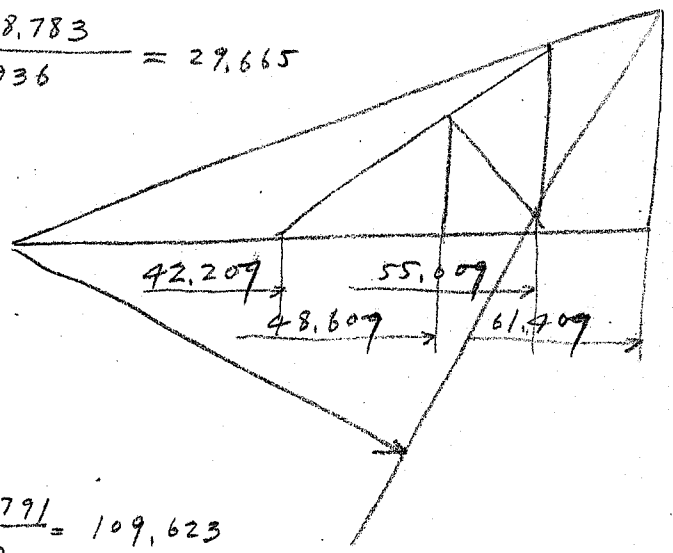
$$\frac{6.4\times 9102}{1502} = 38.783$$

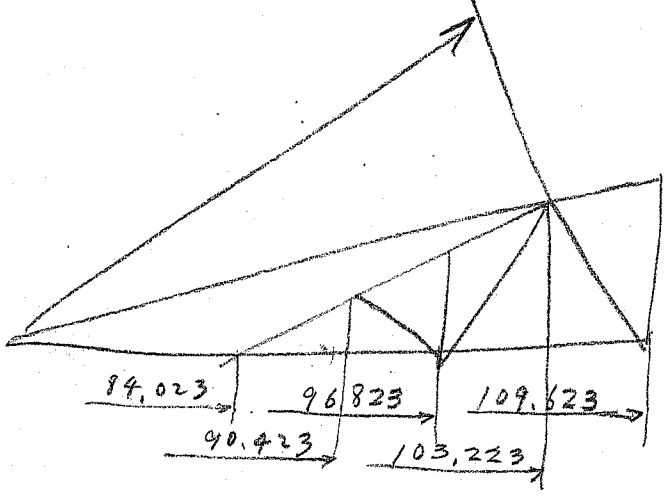
$$d = \frac{7.60\times 38.783}{9.936} = 29.665$$



$$\frac{6.4\times 10161}{1059} = 61.407$$

$$d = \frac{10.161\times 55.007}{12.009} = 46.542$$



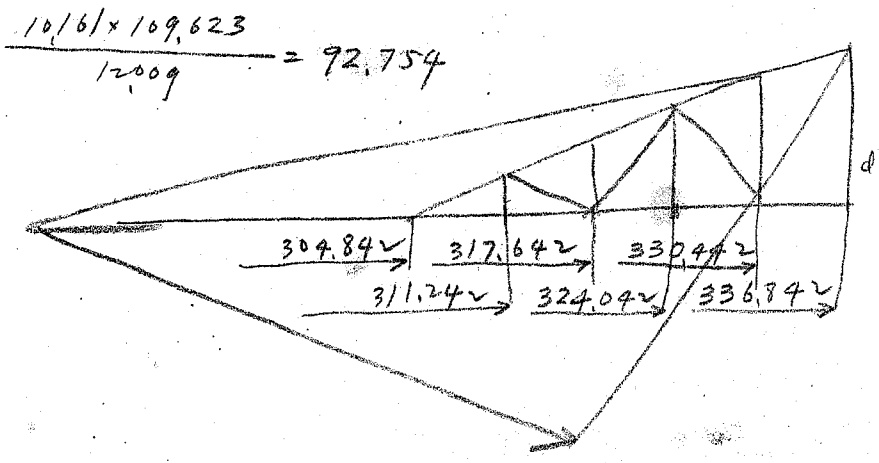


$$\frac{6.4\times 10.791}{.630} = 109.623$$

$$d = \frac{10.161\times 109.623}{12.009} = 92.754$$

$$\frac{6.4\times 11.000}{.209} = 336.842$$

$$d = \frac{11.00\times 330.442}{12.726} = 285.625$$



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u_1-u_2 $\frac{4.5\times 128-6.4}{8861}=5.778$ (57.6)

u_2-u_3 $\frac{5.1\sim}{8980}=5.702$

u_3-u_4 $\frac{4.5\times 256-6\times 6.4}{10739}=7.152$ (115.2, 38.4)

u_4-u_5 $\frac{76.8}{10786}=7.120$

L_3-L_4 $4.5\times 84.023-90423-96823-103.223$ (378.104, 87.655✓) $=.945$
92.754

L_4-u_5 $45\times 304.842-311.242-317.642-324.042-330.442$ (137.1789, 88.4✓) $=.310$
285.625

L_0-L_2 $\frac{4.5\times 6.4}{7.6}=3.789$ (8.8)

L_2-L_4 $\frac{4.5\times 19.2-19.2}{10.161}=6.614$ (86.4, 67.2)

L_4-L_5 $\frac{4.5\times 32.0-10\times 6.4}{11.00}=7.273$ (144, 80)

u_2-L_2 $\frac{5.778\times 1502}{6574}=1.320$

$\frac{5702\times 1059}{6487}=.931$
$.389$

u_4-L_4 $\frac{.7152\times .630}{6431}=.701$

$\frac{.7120\times .209}{6403}=.232$
$.469$

L_0-u_1 4.5×9.936 (44.712) $/7.6=5.883$

u_1-L_2 $4.5\times 25.983-32.383$ (116.924, 84.541) $/29.665=2.850$ (189.932)

u_2u_3 $45\times 42.207-48.607-55.007$ (36.318) $/46.542=1.855$

$u-L_2$ $\frac{36\times 25.983}{10\times 29.665}=3.153$

L_2-u_3 $\frac{28,42.209}{10\times 46.542}=2.539$

u_3-L_4 $\frac{21\times 84.023}{10\times 92.754}=1.902$

L_4-u_5 $\frac{15\times 304.842}{10\times 285.625}=1.601$

u_5-L_6 $\frac{10\times 368.842}{10\times 285.625}=1.291$

u_1-u_2 $\frac{8\times 12.8}{10\times 8.861}=1.156$

u_2-u_3 $\frac{10.14}{10\times 8.980}=1.140$

u_3-u_6 $\frac{6\times 25.6}{10\times 10.739}=1.430$

L_0-L_2 $\frac{9\times 6.4}{10\times 7.6}=.758$ (57.6)

L_2-L_4 $\frac{7\times 19.2}{10\times 10.161}=1.323$ (134.4)

L_4-L_5 $\frac{5\times 32.0}{10\times 11.00}=1.455$ (160)

u_6-u_5 $\frac{15.36}{10.786}=1.424$

L_0-u_1 $\frac{9\times 9.936}{10\times 7.6}=1.177$

u_1-L_2 $\frac{8\times 25.983}{10\times 29.665}=.701$

L_2-u_3 $\frac{7\times 42.207}{10\times 46.542}=.635$

u_3-L_4 $\frac{6\times 84.023}{10\times 92.754}=.544$

L_4-u_5 $\frac{5\times 304.842}{10\times 285.625}=.534$

u_5-L_4 $\frac{4\times 368.842}{10\times 285.625}=.517$

u_2-L_2 $\frac{1.156\times 1.502}{6574}=.264$

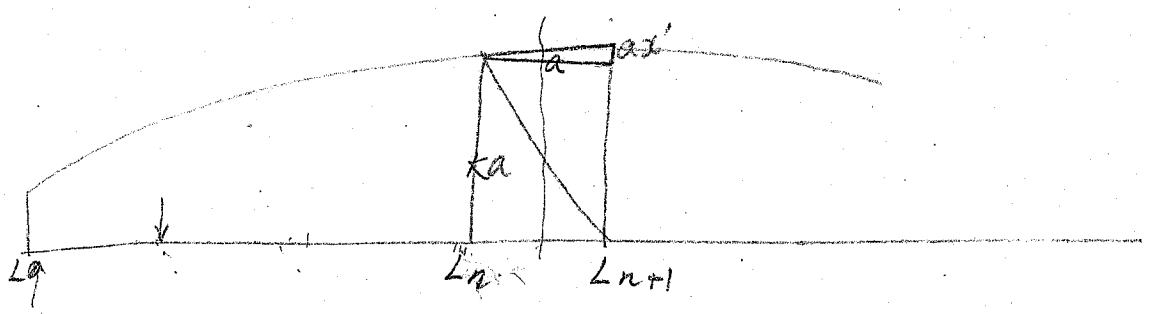
$\frac{1.140\times 1059}{6487}=.186$
$.078$

u_4-L_4 $\frac{1.430\times .630}{6431}=.140$

$\frac{1.424\times .209}{6403}=.046$
$.094$

増田淳氏関係資料
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JIUN MASUDA
CONSULTING ENGINEER
SEIYU BLDG, TOKIO



$$R\frac{Ka}{x'}-P\left(\frac{Ka}{x'}+a\right)-P\left(\frac{Ka}{x'}+2a\right)\cdots\cdots P\left(\frac{Ka}{x'}+na\right)\doteqdot\frac{(K+x')aK}{x'\times\sqrt{1+K^2}}$$

$$=\frac{R\frac{Ka}{x'}-P\left(\frac{Ka}{x'}\right)\times n-Pa\times n}{\frac{(K+x')aK}{x'\times\sqrt{1+K^2}}}=\frac{(RKa-PKan-Pax'n)\times x'\times\sqrt{1+K^2}}{x'\times(K+x')aK}$$

$$=\boxed{\frac{(RK-PKn-Px'n)\times\sqrt{1+K^2}}{(K+x')K}}$$

$x'=\frac{375}{375}=0.1$

$K=\frac{300}{375}=0.8$

$n=2$

$\sqrt{1+K^2}=\sqrt{1.64}=1.28$

23,700　15,250　8,450

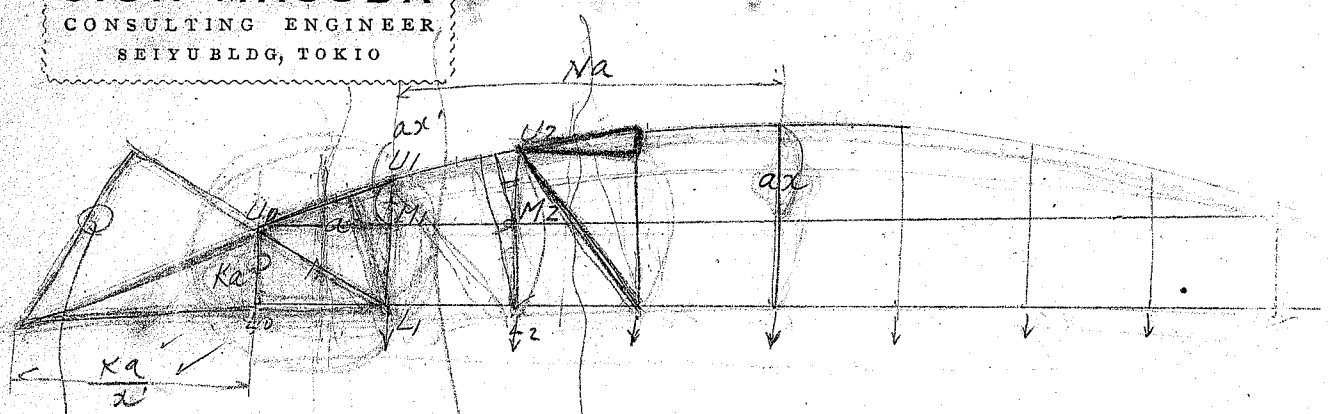
$$\frac{\{29,650\times0.8-8,470(0.9)\times2\}\times1.28}{0.9\times0.9}=14,620$$

$$R\left(\frac{Ka}{x'}-na\right)-P\left(\frac{Ka}{x'}-(n-1)a\right)-P\left(\frac{Ka}{x'}-(n-2)a\right)\cdots\cdots P\left(\frac{Ka}{x'}\right)\doteqdot\frac{(K+x')aK}{x'\times\sqrt{1+K^2}}$$

$$=\frac{R(Ka-nax')-P(}{x'}$$

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$\frac{(K+\alpha)ax}{x\times\sqrt{1+K^2}}$

$U_0-L_1=a\sqrt{1+K^2}$

$U_0-U_1=a\sqrt{1+x'^2}$

$ax'=ax-r+\sqrt{r^2-Na}$

$\frac{a\sqrt{1+K^2}}{Ka}$ $\frac{Ka}{\left(\frac{Ka}{x}+a\right)x}$

$R\times\frac{a\sqrt{1+x'}}{ax'}=R\times\frac{\sqrt{1+x'^2}}{x'}$

$W\left\{R-\frac{ax'}{a\sqrt{1+x'^2}}-\frac{Ka}{a\sqrt{1+K^2}}\right\}\times\frac{a\sqrt{1+K^2}}{Ka}$

$W\left\{R-\frac{x'}{\sqrt{1+x'^2}}-\frac{K}{\sqrt{1+K^2}}\right\}\times\frac{\sqrt{1+K^2}}{K}$

$\frac{Ka\times\left(\frac{Ka}{x}+a\right)}{a\sqrt{1+K^2}}dia=W\left\{R-\frac{ax'}{a\sqrt{1+x'^2}}\right\}\times\frac{a\sqrt{1+K^2}}{Ka}=W\left\{R-\frac{x'}{\sqrt{1+x'^2}}\right\}\times\frac{\sqrt{1+K^2}}{K}$

$8470\left\{3.5-\frac{.2335}{\sqrt{1+.2335^2}}\right\}\times\frac{\sqrt{1+0.4^2}}{0.4}$ (1.076 / 1.0546)

$8470\left\{3.5-\frac{.2275}{1.225}\right\}\times 2.69=27900$

$\frac{.2335}{\sqrt{1+2335^2}}=.2277$ (1.0545) $\qquad \frac{\sqrt{1+0.4^2}}{0.4}=2.69$ (1.076 / 1.16)

$dia.=W\left\{R-\frac{ax'}{a\sqrt{1+x'^2}}\right\}\times\frac{a\sqrt{1+K^2}}{aK}=8470\left\{3.5-.2277\right\}\times 2.69=74500$

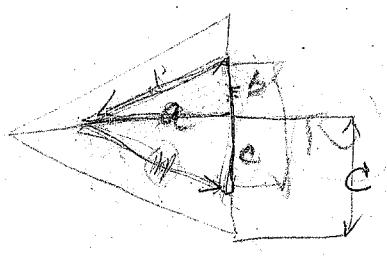
$Ka\times\frac{dx'}{ax}=\frac{Ka}{x'}$

$\frac{WR\times\frac{Ka}{x'}}{\frac{K\times\left(\frac{Ka}{x'}+a\right)}{\sqrt{1+K^2}}}=\frac{WR\times\sqrt{1+K^2}}{K+ax'}$

$\frac{R\times\frac{Ka}{x'}}{\frac{(K+x')aK}{x'\times\sqrt{1+K^2}}}=\frac{R\times\sqrt{1+K^2}}{(K+x')}$

$P\times 3.5\left(1-.2278\right)\times 2.69=$

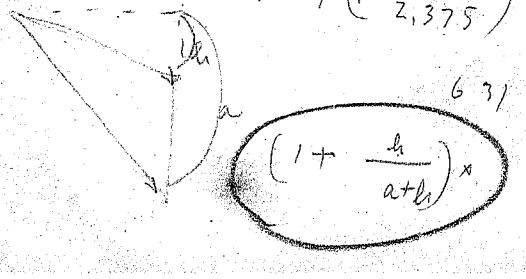
$P\times 3.5(0.7730)\times 2.69=$



Diagonal stress $=\sqrt{a^2+C^2}=K$

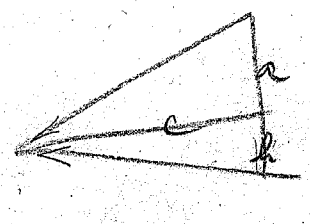
$\frac{R\times C}{b+C}=C$

Diagonal stress $=CK$



$3.5\times 8470\left(1-\frac{875}{2,375}\right)\times 2.69=$

$\left(1+\frac{h}{a+h}\right)\times$



$R\times\left(1-\frac{a}{a+h}\right)\times\frac{\sqrt{K^2+C^2}}{K}$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

標準二十一本鈑桁橋（国道）

（計算書、鉄材料表）

JIUN MASUDA
CONSULTING ENGINEER
IIJI BLDG. TOKYO

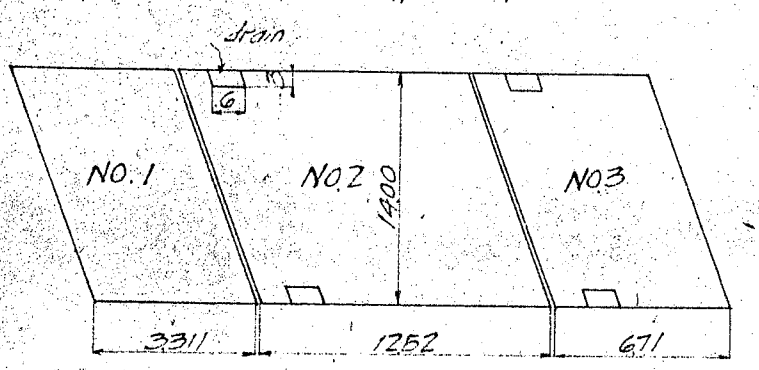
MADE BY M. Kojima DATE 5-11-28 FILE NO
CHECKED BY I. Matano DATE 5-12-3 PAGE NO 74

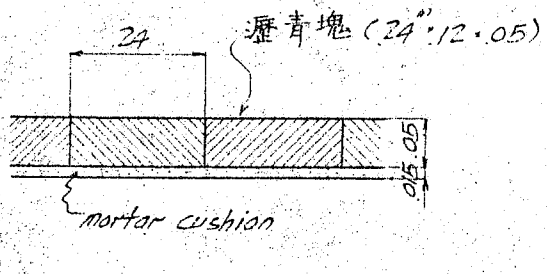
CALCULATIONS FOR

Materials of Jōnan Bashi for Kyoto Prefecture

第九編　橋面鋪裝　(Sheet no. 5 参照)

第一節　瀝青塊鋪裝 (車道)





面積 NO.1 3311 · 1400 = 464 m²

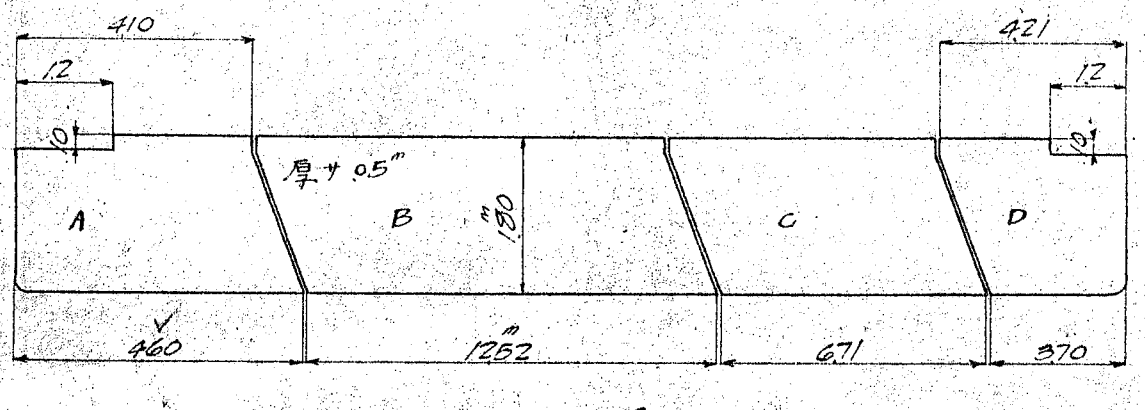
NO.2 1252 · 1400 = 1753
less .6 · .3 · 2 = - .4
1749 m²

NO.3 671 · 1400 = 939
- 4
935 m²

面積合計 NO.1 2 @ 464 = 928
NO.2 6 @ 1749 = 10494
NO.3 5 @ 935 = 4675
16097 平方米

第二節　混凝土鋪裝 (歩道)

配合 1:1:2



A ½ · (460 + 410) · 180 = 783 m²
less 12 · 01 = - 12
771

B 1252 · 180 = 2254

C 671 · 180 = 1208

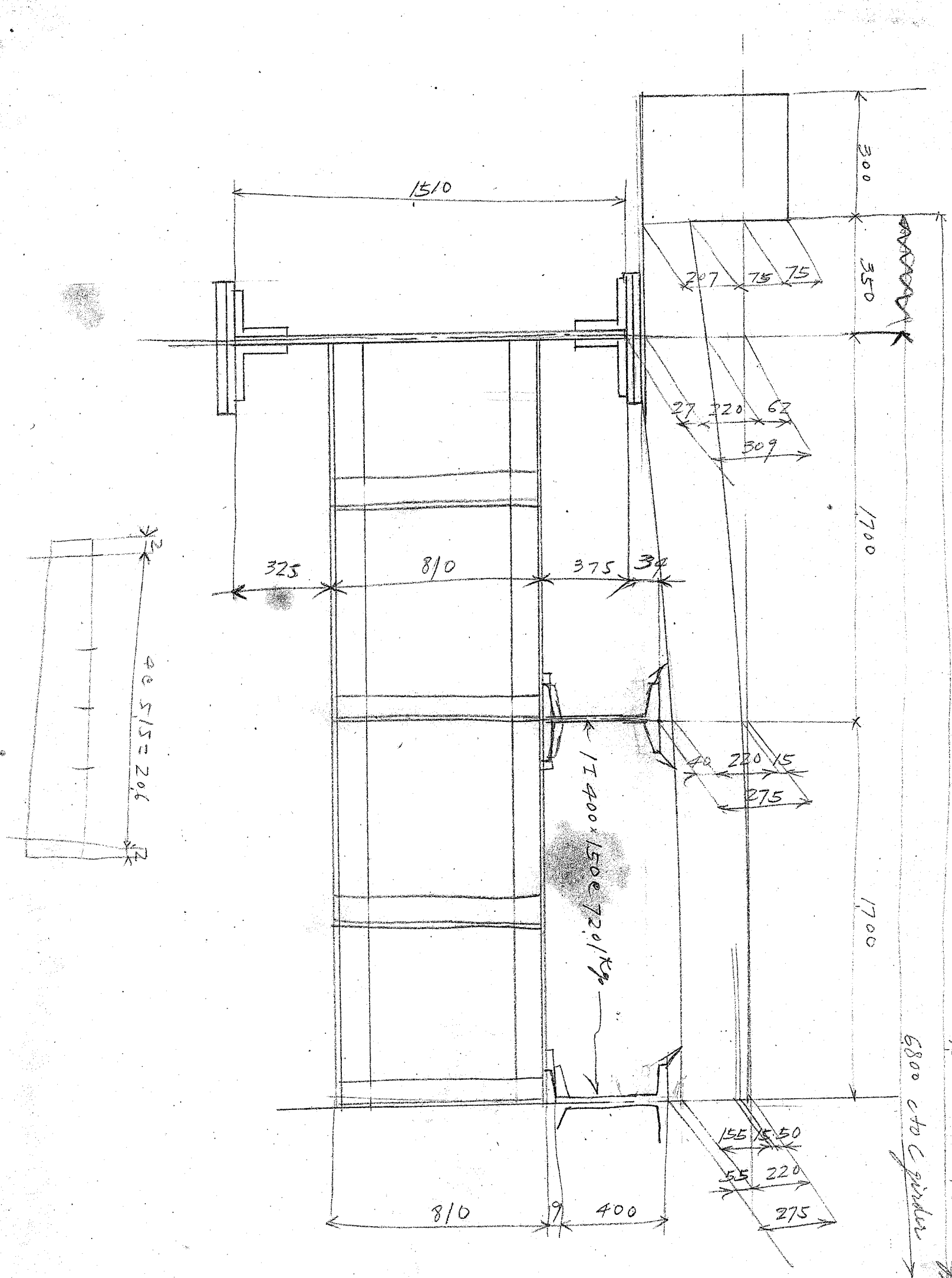
D ½ · (421 + 370) · 180 = 712
less - 12
700

面積合計
A 2 @ 771 = 1542
B 12 @ 2254 = 27048
C 10 @ 1208 = 12080
D 2 @ 700 = 1400
42070 平方米

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CONSULTING ENGINEER
SEIYU BLDG, TOKIO

図3-21



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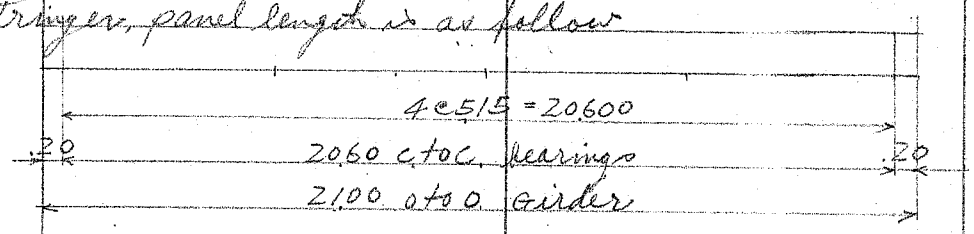
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CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

Design of 21 meters girder span

Design of I beam stringer, panel length is as follow



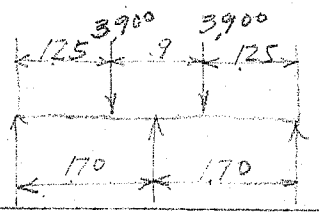
Stringer S1, span length = 5.15 meters, spacing 1.70 meters

Dead load
floor slab and pavement 5.15 × 1.7 = 8.75
Beam assumed .75
9.50 Kg. per lin. meter

Dead load moment = $\frac{1}{8} \times 950 \times 5.15^2$ = 3,150 Kgms
Dead load shear = $\frac{1}{2} \times 950 \times 5.15$ = 2,440 Kgs.

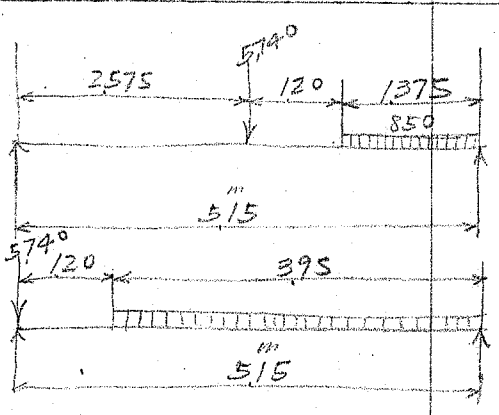
Live load

Motor truck loading rear wheel $2 \times 3900 \times \frac{125}{170}$ = 5,740 Kgs.
Uniform live load 500×1.70 = 850 Kgs.



Moment

reaction rear wheel 5,740 ÷ 2 = 2,870
unif. load $\frac{850 \times 1.375^2}{2 \times 5.15}$ = 156
3,026 Kgs.



moment 3,026 × 2.575 = 7,800 Kgms.

End shear

unif. load $\frac{850 \times 3.95^2}{2 \times 5.15}$ = 1,290
Rear wheel = 5740
7,030 Kgs.

one hole one 5m = 1080 m/m

Summary of moments and shear

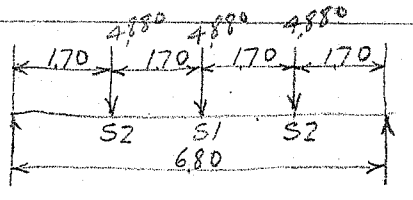
	moments	Shear
Dead load	3,150	2,440
Live load	7,800	7030
	10,950 Kgm.	9,470 Kgs.

Section modulus required = $\frac{10,950 \times 100}{1,100}$ = 995 cm³

use 400 × 150 @ 72.1 Kgs.
whose section modulus 1199 cm³ ok
Shear on web = $\frac{9,470}{40 \times 1.0}$ = 237 Kg/cm² ok

Stringers S2 use same section as for S1.

Design of Intermediate floor beam Span length = 6.80 meters spacing 5.15 meters



Stringer concentration on floor beam = 2440 Kgs × 2 = 9,880 Kgs.
Overhanging effect of slab, on S2 neglected
Reaction

1.5 × 4,880 = 7,330 Kgs = End shear

moment at S1

7,330 × 3.4 = 24,950
4,880 × 1.7 = − 8,300
16,650 Kgms

moment at S2

7,330 × 1.7 = 12,450 Kgms

weight of floor beam assumed 160 Kg/m
moment at S1 = $\frac{1}{8} \cdot 160 \times 6.80^2$ = 925 Kgms
End shear = $\frac{1}{2} \times 160 \times 6.80$ = 545 Kgs.
Moment at S2 = 545 × 1.7 = 925
$\frac{1}{2} \times 160 \times 1.70^2$ = − 230
695 Kgm

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JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

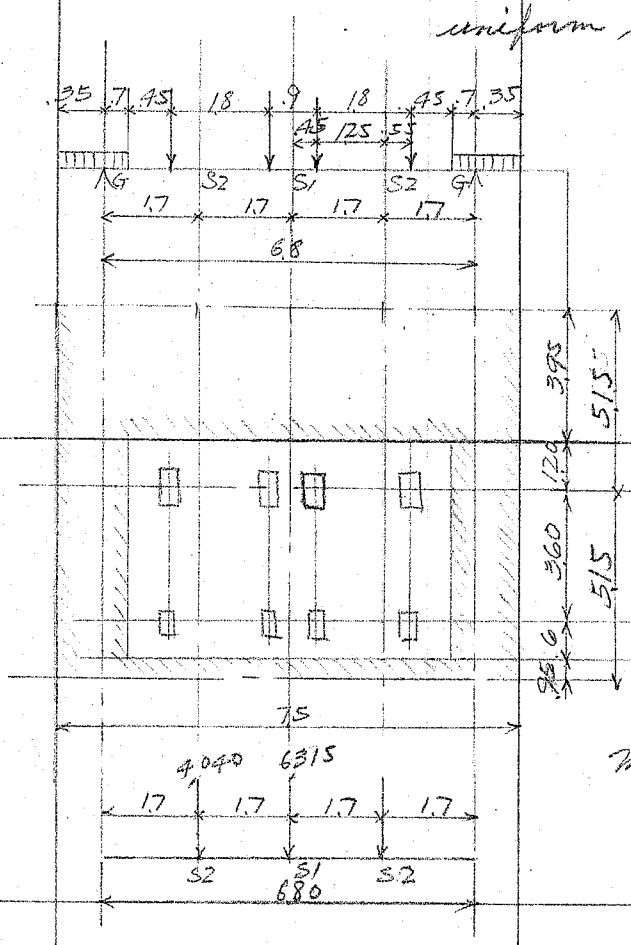
Design of 21 meters girder span

Summary of Dead load moment and shear

	moment at S1	S2	end shear
Stringer concentration	16,650	12,450	7330
weight of floor beam	925	695	545
	17,575 Kgm	13,145 Kgm	7,875 Kgo

Live load :— motor truck loading rear wheel concentration = 3000
Impact = $\frac{20}{60+6.8}$ = 29.95 % say 30 % 900
3,900 Kgo

Front wheel concentration with impact = $\frac{1}{3} \times 3,900 = 1,300$ Kgo
uniform live load = 500 Kgo / sq meter



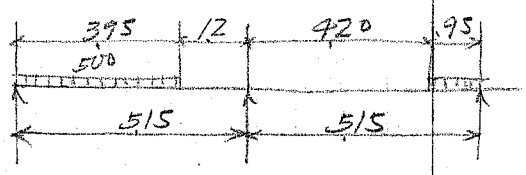
Reaction of stringer due to rear wheel of motor truck
on stringer S1 $3,900 \times \frac{125}{170} \times 2 = 5,740$ Kgo, front wheel $\frac{5,740}{3} = 1,910$ Kgo
on stringer S2 $3,900 \times \frac{45}{170} = 1,030$
$3,900 \times \frac{\frac{170}{155}}{170} = 2,640$ front wheel $\frac{3670}{3} = 1,220$ Kgo
3,670 Kgo

Stringer concentration due to front wheel
on S1 $1,910 \times \frac{155}{515} = 575$ Kgo
on S2 $1,220 \times \frac{155}{515} = 370$ Kgo

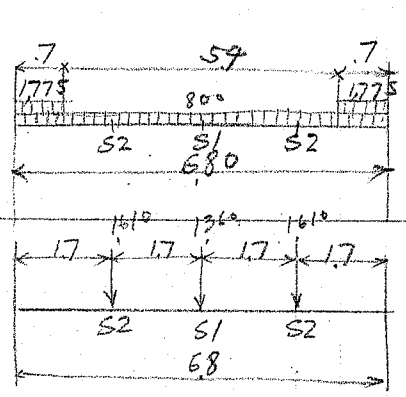
Total wheel concentration on floor beam
on S1 5,740 + 575 = 6315 Kgo
on S2 3,670 + 370 = 4040 Kgo

Moment due to motor truck wheel concentration
Reaction $4040 + \frac{6315}{2} = 7,200$ Kgo = end shear
moment at S1 $7,200 \times 3.40 = 24,500$
$4,040 \times 1.70 = -6860$
17,640 Kgm

moment at S2 $7,200 \times 1.70 = 12,230$ Kgm



Uniform load
front and rear of truck $\frac{500 \times 395^2}{2 \times 515} = 760$
$\frac{500 \times 95^2}{2 \times 515} = 40$
900 = W1



Side of motor truck $500 \times 515 = 2575$
1,775 = W2

Moment due to uniform load
Stringer concentration S1 $800 \times 1.70 = 1,360$ Kgo
S2 $\frac{1,775 \times 1.7^2}{2 \times 1.7} = 250$
1,360
1,610 Kgo

Reaction = $1,610 + \frac{1360}{2} = 2,290$ Kgo = end shear
moment at S1 $2,290 \times 3.4 = 7,800$
$1,610 \times 1.7 = -2,740$
5,160 Kgm

moment at S2 $2,290 \times 1.7 = 3,890$ Kgm

Summary of Live load moments and shears

	moment at S1	moment at S2	end shear
motor truck wheels	17,640	12,230	7200
uniform load	5,160	3,890	2290
	22,800 Kgm	16,120 Kgm	9,490 Kgo

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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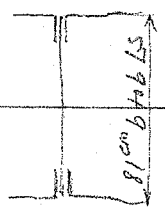
CALCULATIONS FOR

Design of 21 meters Girder Span

Summary of dead and live load moments and shears

	moment at S1	moment at S2	End shear
Dead load	17,575	13,145	7,875
Live load	22,800	16,120	9,490
	40,375 Kgm	29,265 Kgm	17,365 Kgo

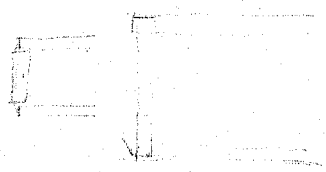
assumed section



Dry web plate 800 × 9 ⅛ web area = 9.0 cm² back to back of Ls = 81 cm
Effective depth - 81.0 - 2 × 2.22 = 76.56 Kgo cm
flange stress = 40375 / 76.56 = 52,700 Kgo T or C
Bottom flange area required = 52700 / 1,200 = 44.9 - 6.0 = 38.9 cm² net ok. out
use 2 Ls 125 × 90 × 10 = 91.0 - 4.4 = 36.6 cm² net ok
Top flange area required = 52,700 / 1100 = 47.9 - 9.0 = 38.9 cm² gr. ok
Shear on web = 17365 / 72 = 241 Kgo/cm² ok

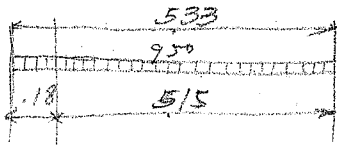
Approximate weight of Intermediate floor beam

web 1 Pl.	800 × 9	@	56.52 Kgo	× 6.80	=	384
flange 4 Ls	125 × 90 × 10	@	16.09	× 6.66	=	429
End conn. 2 Ls	125 × 90 × 10	@	16.09	× 0.62	=	20
Stiff 14 Ls	100 × 75 × 10	@	12.95	× 0.81	=	147
Rivet heads and variation say				3½ %	=	35
						1,015

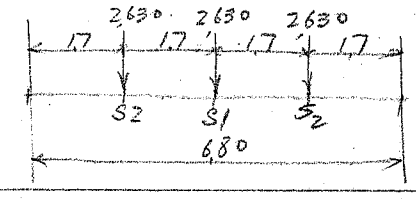


Weight of beam per lin meter = 1,015 / 6.8 = 150 Kgo/lin. m.

Design of End floor beam Span length = 6.80 meters spacing 5.15 meters
Dead load :— Stringer concentration on floor beam



Dead Load on stringer = 950 Kgo per lin. meter
950 × 5.33² / 2 × 5.15 = 2630 Kgo



Reaction	2,630 × 1.5	= 3,940	= End shear
moment at S1	3,940 × 3.4	= 13,400	
	2,630 × 1.7	= - 9,470	
		8,930 Kgm	
moment at S2	3,940 × 1.7	= 6,700 Kgm	

Dead load due to own wt. of floor beam assumed 135 Kg/m

moment at S1	⅛ × 135 × 6.8²	= 780 Kgm
End shear	½ × 135 × 6.8	= 460 Kgo
moment at S2		
	460 × 1.70	= 780
	135 × 1.7² / 2	= - 195
		585 Kgm

Summary of dead load moment and shear

	moment at S1	moment at S2	end shear
Stringer concentration	8,930	6,700	3,940
weight of beam	780	585	460
	9,710 Kgm	7,285 Kgm	4,400 Kgo

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

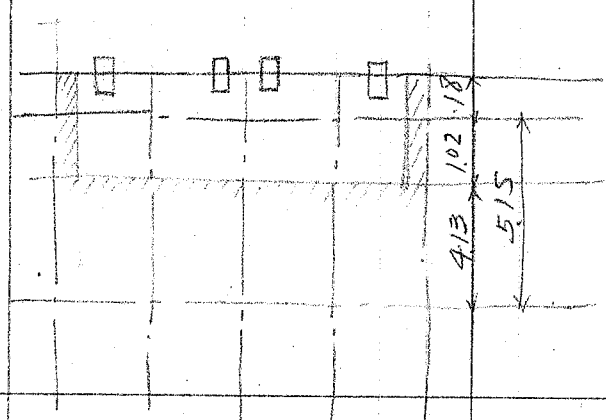
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CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

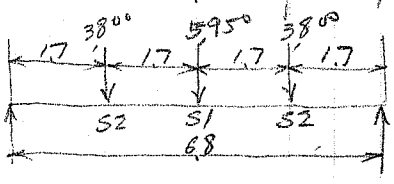
Design of 21 meters girder span

Live load



Concentration on stringer due to motor truck wheels
Rear wheel on S1 5,740 Kgs on S2 3,670 Kgs

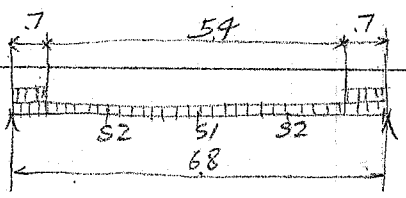
Stringer concentration due to rear wheel
on S1 $5{,}740 \times \frac{5.33}{5.15} = 5{,}950$ Kgs
on S2 $3{,}670 \times \frac{5.33}{5.15} = 3{,}800$ Kgs



moment due to motor truck rear wheel
Reaction 5,950 ÷ 2 = 2,975
3,800
6775 Kgs end shear

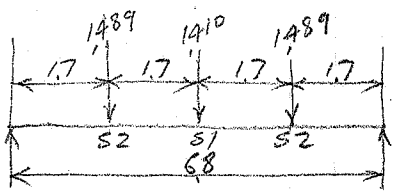
moment at S1 6,775 × 3.4 = 23,000
3,800 × 1.7 = − 6,460
16,540 Kgm

moment at S2 6,775 × 1.7 = 11,500 Kgm



Uniform load
rear of motor truck
$\frac{500 \times 4.13^2}{2 \times 5.15} = 830 = u_1$

side of motor truck
$\frac{500 \times 5.33^2}{2 \times 5.15} = 1380$
diff. = 550 = u_2



Stringer concentration S1 = 830 × 1.7 = 1,410 Kgs

S2 = $\frac{550 \times 1.7^2}{2 \times 1.7}$ = 79
1,410
1,489 Kgs

Reaction = $\frac{1{,}410}{2}$ + 1,489 = 2,194 Kgs = end shear

moment at S1 = 2,194 × 3.4 = 7,460
1,489 × 1.7 = − 2,530
4,930 Kgm

at S2 2,194 × 1.7 = 3,730 Kgm

Summary for Live load moments & shear

	moment at S1	at S2	end shear
Rear wheel of motor truck	16540	11,500	6,775
Unif. load	4,930	3,730	2,194
	21,470 Kgm	15,230 Kgm	8,969 Kgs

Summary for Dead and Live load moments and shear

	moment at S1	S2	end shear
Dead load	9,710	7,285	4,400
Live load	21,470	15,230	8,969
	31,180 Kgm.	22,515 Kgm	13,369 Kgs

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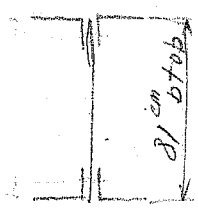
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JIJI BLDG, TOKYO

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CALCULATIONS FOR

Design of 21 meters girder span

assumed section



Dry web 800 × 9 = 72.0 cm² 1/8 web area = 9.0 cm² b to b Ls = 81.0 cm

Flange stress = 31,180 / .7712 = 40,400 kgs T or C

Bottom flange area required = 40,400 / 1,200 = 33.7

1/8 web = -9

use 2Ls (125 or 100) × 75 × (9 or 10) = 36.38 (33.0) − 3.96 (4.8) = 24.7 (28.2) cm² net ok (30.92)

Top flange area required = 40,400 / 1,100 = 36.7

1/8 web −9.0

27.7 cm² gross ok.

approximate weight of end floor beam

web	1Pl	800 × 9	c	56.52 kgs	× 6.80	= 384
flange	4Ls	(100) × 75 × (10)	c	1295	6.66	= (345)
end conn.	2Ls	125 × 90 × (10)	c	1609	0.62	= (20)
stiffener	14Ls	(75) × 75 × 9	c	996	0.81	= 113
		90	rivet heads & variation abt. 3 1/3 %			33
						(895) kgs

weight per lin. meter of beam = (893) / 6.8 = (132) kgs ok

Design of bottom Lateral bracings

Dead load metal assumed

Stringers	3 c 75.0	= 225
floor beam	1015 ÷ 5.15	= 198
lateral bracing assumed		= 70
main girder assumed		= 850
		1343

Floor slab + pavement between girder

5.15 × 6.8 = 3,500

overhanging slab coping with handrail = 920

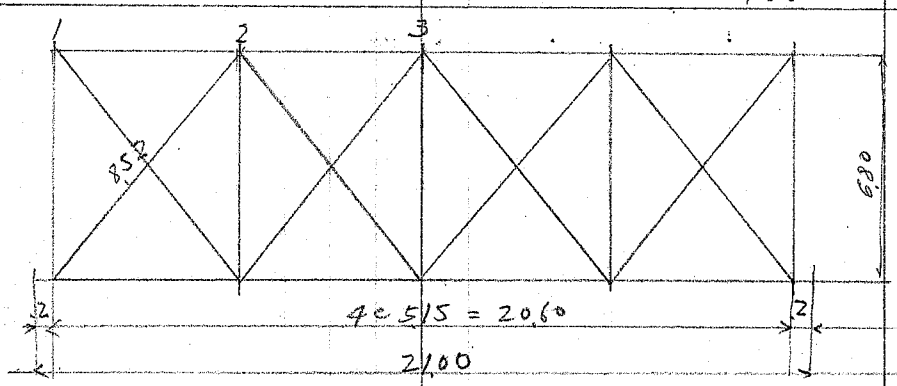
4420

5,763 kgs per lin. meter

Panel load = 5,763 × 5.15 = 29,700 kgs for two girders

Seismic panel load = 29,700 × .2 = 5,940 kgs

Wind panel load = (600 / 400) × 5.15 = 3,090 / 2,060



Diagonal length

5.15² = 26.50

6.80² = 46.25

72.75

√72.75 = 8.52

coeff. = 8.52 / 6.80 = 1.25

End shear = 3,090 × 1.5 = 4,640 kgs stress 1-2 4640 × 1.25 = 5,800 kgs T SR = (4.84) cm² net

shear 2-3 = 3,090 × .5 = (1,545) kgs stress 2-3 1545 × 1.25 = 1,930 kgs " (1.61) cm² "

max. no of 19φ rivet = 5,800 / 2,126 = 2.7

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JIUN MASUDA
CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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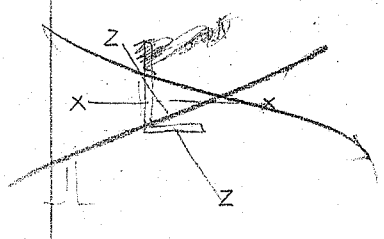
CALCULATIONS FOR

Design of 21 meter girder span

Seismic stress
End shear = 5,940 × 1.5 = 8,900 Kgo stress 1–2 8,900 × 1.25 = 11,120 Kgo T
Shear 2–3 = 5,940 × 0.5 = 2,970 Kgo stress 2–3 2,970 × 1.25 = 3,680 Kgo T

Seismic stresses govern the section of diagonal members
sectional area required = $\frac{11,120}{1,200 \times 0.8 - 1.6}$ = 515 cm² net
required no of 22 rivet = $\frac{11,120}{2,851 - 2,120 \times 1.8}$ = 2.4

Use L 150 × 100 × 9 = 21.69 − 3.96 = 17.73 cm² net
L 100 × 100 × 10 = 19.0 − 2.5 = 16.5 cm² net



Radius of gyration length l/r
rx = 4.79 152 158.5 ok
rz = 2.09 3.96 184.5 ok

Approximate weight of lateral bracing

Diagonals	8 Ls	150 × 100 × 9	e	17.02	× 8.00	= 4,090
Center conn.	4 Pls	240 × 9	e	16.96	× .70	= 48
Conn. to girder	6 Pls	500 × 9	e	35.33	× .70	= 148
〃	4 〃	500 × 9	e	35.33	× .50	= 71
						1357
	Rivets heads and variation 1½% abt					23
						1380 Kg

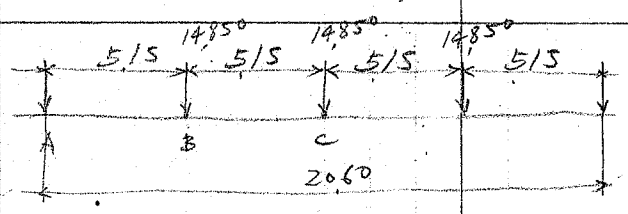
weight of lateral bracing per lin meter = $\frac{1380}{20.6}$ = 67.0 Kg/m

Design of Main girders. Span length = 20.60 meters c to c bearing. 21 meters out to out

Dead load

Floor slab and pavement between main girders	515 × 6.80	= 3500
Handrails, copings, and floor beyond girders	2 e 460	= 920
		4,420
Stringers	3 × 75	= 225
floor beams	1,015 ÷ 5.15	= 198
Lateral bracing		= 67
Main girders Assumed		= 850
		1340
		5,760 Kgo/m

Intermediate floor beam = $\frac{5,760 \times 5.15}{2}$ = 14,850 Kgo for one girder
End = $\frac{5760 \times \frac{2}{2} \times 2.78}{2}$ = 8,020 〃 〃 〃



Reaction = 14,850 × 1.5 = 22,270 Kgo = end shear
moment at B 22,270 × 5.15 = 114,800 Kgm

moment at C 22,270 × 10.30 = 230,000
14,850 × 5.15 = − 76,500
153,500 Kgm

Dead load on shoe 22,270
8,020
wt. of shoe say 170
30,460 Kgo

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTIEG ENGINEER
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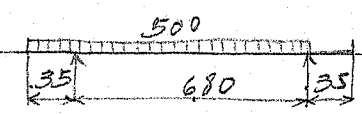
CALCULATIONS FOR

Design of 21 meters girder span

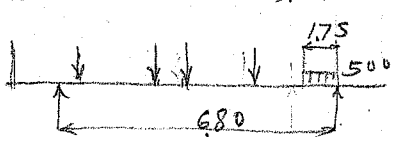
Live Load

uniform live load = 500 Kgs per sq. meter
motor truck concentration 3,000
Impact $\frac{20}{60+20,60} = 24.8\%$ 745
3,745 Kgs rear wheel
front wheel with impact $\frac{3,745}{3} = 1,250$ Kgs

Uniform live load on front and rear of truck assumed uniformly Distribution over main girder



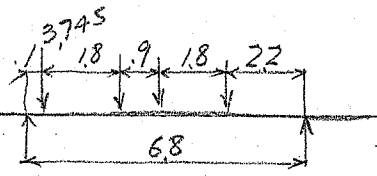
$\frac{500 \times 7.15^2}{2 \times 6.8} = 1,880$ Kg/m



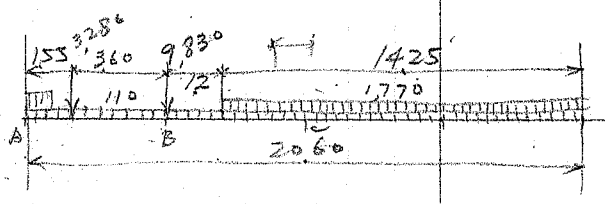
uniform load on side of truck
$= \frac{500 \times 175^2}{2 \times 6.8} = 110$ Kg/m $= u_1$

$1,880 - 110 = 1,770$ " $= u_2$

motor truck wheel concentration on main girder assumed as follows

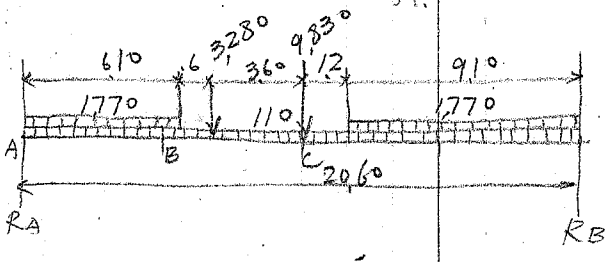


Reaction
3745 × 6.70 = 25,100
3745 × 4.90 = 18,380
3745 × 4.00 = 15,000
3745 × 2.20 = 8,250
66,730 ÷ 6.8 = 9,830 Kgs rear wheel
9,830 ÷ 3 = 3,280 Kgs front wheel



DL ____ 22,275

$\frac{9830 \times 15.45}{20.6} = 7,370$
$\frac{3280 \times 11.85}{20.6} = 3,890$
$\frac{110 \times 15.45^2}{2 \times 20.6} = 640$
$\frac{1770 \times 11.75^2}{2 \times 20.6} = 5,450$
39,650



front wheel 3,280
rear " 9830
u_1 $110 \times 20.60 = 2,265$
u_2 $1,770 \times 6.10 = 10,800$
" $1,770 \times 9.10 = 16,100$
42,275
RA = −20,410
RB 21,865

moment at B reaction
front wheel $\frac{3280 \times 19.05}{20.6} = 3,035$
rear wheel $\frac{9830 \times 15.45}{20.6} = 7,380$
u_2 $\frac{1770 \times 9.5}{20.60} \times 20.125 \frac{20.60}{2} = 1,640$
u_1 $110 \times 10.30 = 1,130$
u_2 $\frac{1,770 \times 14.25^2}{2 \times 20.60} = 8,740$
21,925 Kgs

moment = $21,925 \times 5.15 = 113,000$
$3,280 \times 3.60 = -11,800$
$\frac{110 \times 5.15^2}{2} = -1,460$
$1,770 \times 9.5 \times 4.675 = -5,850$
93,890 Kgm

Moment at C
Reaction
front wheel $\frac{3,280 \times 13.90}{20.60} = 1,590$
rear wheel $\frac{9830 \times 10.30}{20.60} = 4,920$
u_1 $110 \times 10.30 = 1,130$
u_2 $\frac{1770 \times 6.10 \times 17.55}{20.60} = 9,210$
u_2 $\frac{1,770 \times 9.10^2}{2 \times 20.60} = 3,560$
20,410 Kgs

moment $21,865 \times 10.30 = 225,000$
u_1 $\frac{110 \times 10.30^2}{2} = -5,830$
u_2 $1,770 \times 9.10 \times 5.75 = -92,600$
126,570 Kgm

93,540

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（独立行政法人 土木研究所蔵）

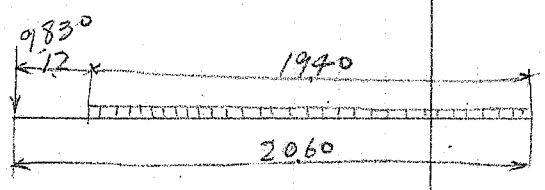
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CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

Design of 21 metres girder span

Max. End shear

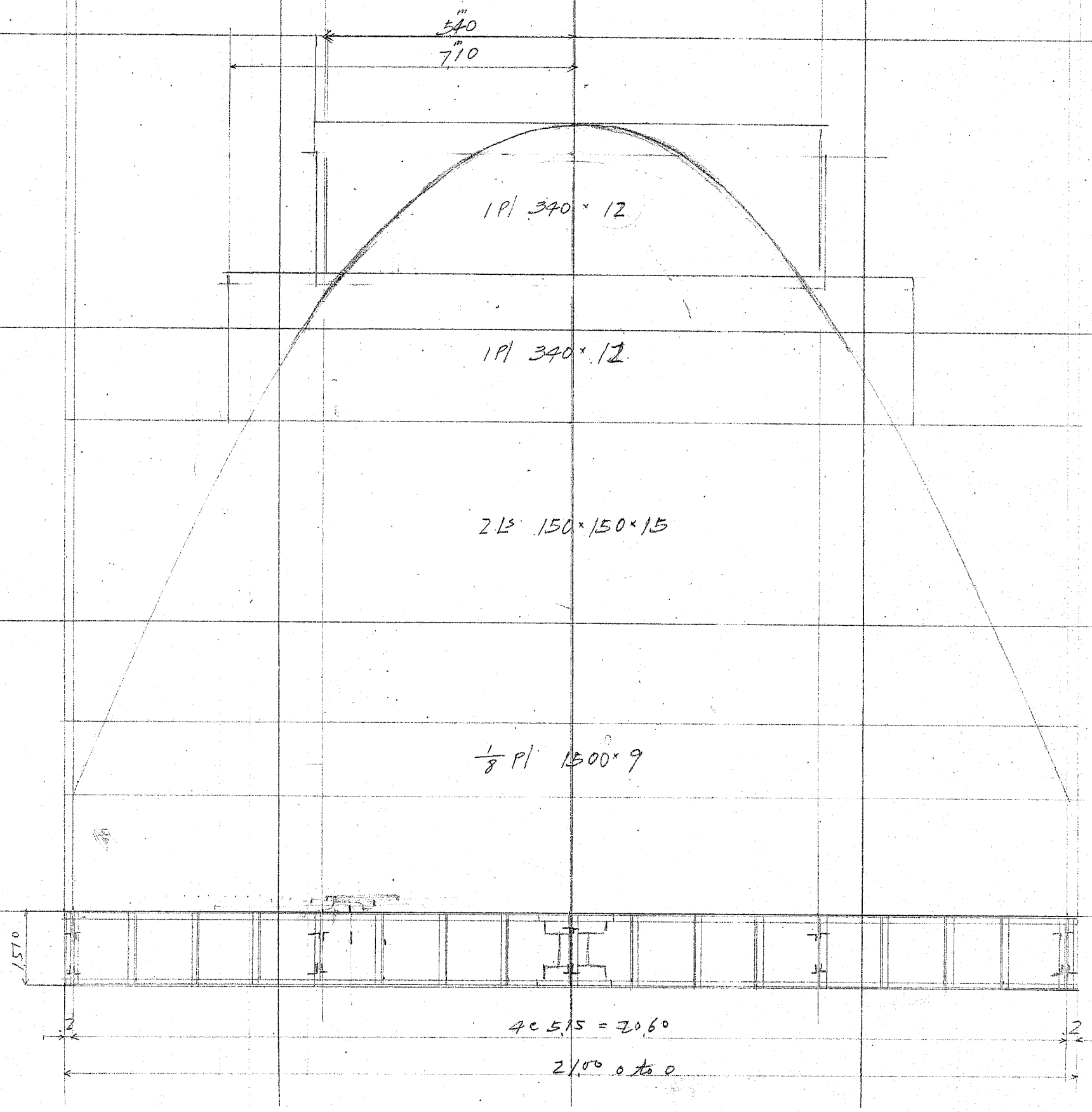


$110 \times 1030 = 1,130$
$\frac{1770 \times 1990^2}{2 \times 2060} = 16,180$
9830
$27,140$ Kg

Summary of Dead and Live Load moments and end shear

	moment at B	at C	end shear	Load on shoe	Shear at B
Dead Load	114800	153500	22270	30460	
Live Load	93890	126570	27140	27520	39,630
	208690 Kgm	280070 Kgm	49410 Kg	57980	

Bending Moment Diagram for main girder



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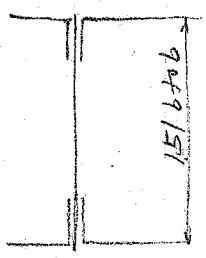
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CONSULTIEG ENGINEER
JIJI BLDG, TOKYO

MADE BY m. Jinda DATE 5-9-23 FILE NO b
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CALCULATIONS FOR

Design of 21 meters girder span

Section of main girder



Dry web pl. 1500×9 = 135. cm² ⅛ web area = 17.0 cm² b.to b. L= 150.0 cm
Effective depth 151 − 1.25 × 2 = 148.5 cm
Flange stress = 280070 / 1,485 = 188,500 Kg. T or C

Bottom flange area required = 188,500 / 1,200 = 157.0
⅛ web area 17.9
140.0 cm² net

approx.
2 P/s 34 × 2.4 = 81.5 × 1.2 = 97.8
2 L's 85.5 × 4.22 = 306.7
167.9 1.25 209

use 2 Ls 150 × 150 × 15 = 85.50 − 15.0 = 70.50
2 P/s 340 × 12 = 81.60 − 12.9 = 69.60
167.1 cm² gr 140.1 cm² net ok

Top flange area required 188500 / 1100 = 171.5
⅛ web = 17.6
154.5 cm² gr ok

Unit shear on web 49410 / 135 = 366.0 Kg/cm²

Approximate weight of main girder

Flange	4 Ls	150×150×15	c	33.55	×	21.00	=	2,820	✓
, Cov. pl.	2 P/s	340 × 10	c	29.38	×	14.20	=	834	
, ,	2 P/s	340 × 6	c	29.36	×	10.80	=	634	
web pl.	1 P/s	1,500 × 9	c	105.98	×	21.00	=	2220	✓
End stiff	8 Ls	125×90×10	c	16.09	×	1.48	=	190	✓
Fillo	4 P/s	190 × 15	c	22.37	×	1.21	=	108	✓
Int. stiff	24 Ls	125×75×10	c	14.91	×	1.51	=	541	✓
panel stiff	6 Ls	125×75×10	c	14.91	×	1.51	=	135	✓
Connection Ls	3 Ls	125×90×10	c	16.09	×	.62	=	30	✓
Center splice	4 Ls	150×150×15	c	33.55	×	1.40	=	188	
Cov. p/splice	2 P/s	340 × 12	c	32.03	×	1.40	=	90	
web splice	4 P/s	240 × 14	c	26.38	×	1.20	=	127	
, ,	2 P/s	360 × 14	c	39.56	×	.77	=	61	
Fillo	2 P/s	75 × 16	c	9.42	×	1.21	=	22	
		Rivets heads and variation 3.5%						280	
								8,280 Kg.	

8,380 ÷ 21.00 = 400 Kg. per lin m of span

Structural steel in one span

Stringer	21.0 c	75 × 3 =	4,725
Intermediate floorbeam		3 × 1,015 =	3,040
End floorbeam		2 × 895 =	1,790
Lateral bracing		=	1,380
main girder		2 × 8,280 =	16,560
Expansion joint			760
Castion shoe			650
			28,905 Kg.

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CALCULATIONS FOR

陸道 21 Meter plate girder SOFN

	NO.	Discription	section	Length	Wt. of one meter	Wt of main section	Wt of Details	Total Wt	Remarks
				GIRDER		4 Reqd			
✓ ✓	4	F/g ls	✓ 150×150×15	✓ 6.020	3355 ✓	807.9 ✓			
✓ ✓	1	Web pl	✓ 1500× 9	✓ 6.020	105975 ✓	638.0 ✓			
✓	2	Cov. pls	✓ 340× (11) 12	✓ 2.700	32.028 ~~29359~~	~~158.5~~ 173.0			
✓ ✓	4	ls	✓ 125×90×13	✓ 1.480	2061 ✓		122.0 ✓		
✓ ✓	2	Fills.	✓ 190 × 15	✓ 1.205	22373 ✓		53.9 ✓		
✓ ✓	8	ls	✓ 125×90×9	✓ 1.510	1460 ✓		176.4 ✓		
✓ ✓	2	Conn. ls	✓ 150×100×9	✓ 230	1702 ✓		7.8 ✓		
✓ ✓	1	" L	✓ "	✓ 310	" ✓		5.3 ✓		
✓	1	Sole pl	405×28 ✓	✓ 465	89019 ✓		41.4 ✓		

~~40044~~ ✓ + 406.8 ✓ = ~~2011.2~~ ✓ 2025.7
1618.9
× 4 ✓
~~80448~~
81028

G2 R/L ✓ 2 Reqd

✓ ✓	4	F/g ls	✓ 150×150×15	✓ 8960	33.55 ✓	1202.4 ✓			
✓ ✓	1	Web pl	✓ 1500× 9	✓ 8960	105975 ✓	949.5 ✓			
✓	2	Cov. pls	✓ 340× (11) 12	✓ 8960	32.028 ~~29359~~ ✓	~~526.1~~ ✓ 573.9			
✓	2	"	✓ "	✓ 7940	" ✓	~~466.2~~ ✓ 508.6			
✓ ✓	14	ls	✓ 125×90×9	✓ 1510	1460 ✓		308.6 ✓		
✓	1	Conn. L	✓ 150×100×9	✓ 230	1702 ✓		3.9 ✓		
✓	1	" "	"	✓ 310	"		5.3 ✓		

~~3144.2~~ + 317.8 ✓ = ~~3462.0~~ ✓ 3552.2
3234.4
× 2 ✓
~~6924.0~~
7104.4

SPLICE ✓ 4 Reqd

✓ ✓	2	Spl. pls	✓ 340× (11) 12	✓ 1655	~~29359~~ ~ 32.028	~~97.2~~ ✓ 106.0			
✓ ✓	2	" "	✓ "	✓ 1975	" ✓	~~116.0~~ ✓ 126.5			
✓ ✓	4	pls	✓ 235×14	✓ 630	25827 ✓	65.1 ✓			
✓	2	"	✓ 330×9	✓ 730	23315 ✓	34.0 ✓			
✓ ✓	4	ls	✓ 150×150×19	✓ 1020	4191 ✓	17.0 ✓			

17,217.6
~~15902.0~~
15452 kgs.

502.6 ✓ × 4 = 2010.4
~~493.3~~ ✓ × 4 = ~~1933.2~~

Summary of girders

✓ 125×75×9 FLOOR BEAM FB1 ✓ 2 Reqd

✓	4	F/g ls	✓ 125×90×10	✓ 6500	~~1609~~ ✓ 13.50	~~418.3~~ ✓ 351.0			
✓	1	Web pl	✓ 800 × 9	✓ 6740	56.52 ✓	380.9 ✓			
✓	14	ls	✓ 90×75×9	✓ 810	11.02 ✓	125.0 ✓			
✓	2	Fills.	✓ 120 × 9	✓ 360	8.478 ✓	6.1 ✓			
✓	2	"	✓ "	✓ 310	" ✓	5.3 ✓			
✓	2	"	✓ "	✓ 1480	" ✓	25.1 ✓			
✓	3	pls	✓ 235×9	✓ 260	16.603 ✓	13.0 ✓			

~~973.7~~ ✓ 906.4
× 2 ✓
~~1947.4~~ ✓
1812.8

FB2 ✓ 3 Reqd

✓	2	F/g ls	✓ 125.90×10	✓ 6780	1609 ✓	218.2 ✓			
✓	2	"	✓ "	✓ 6530	" ✓	210.1 ✓			
✓	1	Web pl	✓ 800 ×9	✓ 6780	56.52 ✓	383.2 ✓			
✓	2	ls	✓ 125.90×9	✓ 620	1460 ✓	18.1 ✓			
✓	14	"	✓ 90×75×9	✓ 810	11.02 ✓	125.0 ✓			
✓	3	pls	✓ 235 × 9	✓ 260	16.603 ✓	13.0 ✓			

967.6 ✓
× 3 ✓ 2902.8
2902.8 ✓

Summary of Floor beams ~~4.8503~~ kgs ✓ 4,7156

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CALCULATIONS FOR

STRINGERS S1, S2, S3 & S4 1-Req'd

6	Is	400	150 @ 72.01 kg	5.345			23094	
6	Is		"	5.140			22209	
18	pls.	170	9	310	12.011		670	
								45972

LATERAL BRACINGS 1-Req'd.

4	ls	100×100×10	7940		14.91		4735	474.1
8	"	"	3885		"		4646	465.8
4	pls.	280×9	570		18.360	18.782	417	45.9
4	"	170×9	470		12.011	12.717	92	9.2
4	ls	75×75×9	290		9.96		315	31.9
4	"	150×100×9	190		17.02		129	
4	pls	380×9	460		26.847	28.613	494	52.6
6	"	"	680		"		1095	116.7
								1915
								1,209.1

SHOES & ANCHOR BOLTS

4	cast iron shoes.		@ 1400.	560.0	
4	pls. 70×30	160	10.486	10.0	
16	Anchor bolts 25 ϕ	700.	@ 3.13.	50.1	
10	washers 100×9	100	7.065	11.3	
8	Bolts 22 ϕ	115	@ 0.650	5.2	
					637.2

RIVET HEADS.

4430	22 ϕ shop rivets heads		@ 0.0964	427	431.9
2270	22 ϕ Field " "		@ "	2188	256.4
4930	19 ϕ shop " "		@ 0.06464	1243	121.5
670	19 ϕ Field " "		@ "	433	18.1
					814.0
					827.9

Total wt.

Grand summary of 21 Meter plate girder

Girder's	15452.1 kgs	16.902.0	17217.6	
Floor beams	4850.2	4,715.6		
Stringers	4597.2			
Lateral bracings	1191.5	1,209.1		
Shoe & Anchor bolts	637.2			
	27282	28,178.1	28492.7	28,376.7
Rivet heads	914.0	827.9		
	27,542.2	or 27542	Kgs. tons	
	28.992.1		28.992	
	29,307.7		29,208	
	29,204.6		29,204	

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CHECKED BY M. Jinda DATE 5-12-3 PAGE NO 70

CALCULATIONS FOR

Materials of Jonan Bashi for Kyoto Prefecture

地覆石(下部)(円形)	.109	.30	.16	.052	1	.052
〃 〃 〃	.10	.29	.16	.046	2	.092
〃 〃 〃	.98	.27	.16	.042	1	.042
〃 〃	.95	.24	.16	.036	3	.108
〃 〃	.475	.24	.16	.018	1	.018
袖 柱	1.36	.60	.47	.384	1	.384
						1.649 立方米

石材立積合計
長 3.5m 分 1.193
〃 6.5m 分 1.649
2.842 立方米

第貳節 石材仕上計算

第一款 表面及合端仕上計算(小叩仕上)

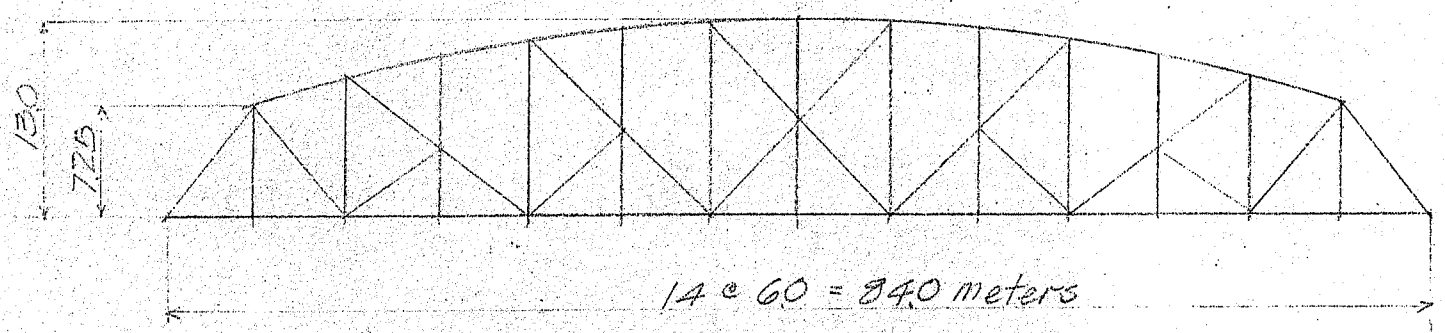
長廿三五米分

笠石		.58	× 4.212	=	2.44	表面
窓内面	9 @	.10	× .75	=	.68	〃
a	2 @	.58	× 2.107	=	2.44	〃
b	2 @	.38	× 2.105	=	1.60	〃
c	8 @	.11	× .20	=	.18	〃
袖 柱	2 @	1.07	× 1.36	=	2.91	〃
〃		.47	× .60	=	.28	〃
地覆石		.68	× 4.212	=	2.86	〃
笠石	10 @	.03	× .58	=	.17	合端
窓内面	9 @	.02	× .77	=	.14	〃
a	4 @	.03	× 3.64	=	.44	〃
〃(親柱寄り)		.03	× 3.00	=	.09	〃
〃(袖柱寄り)		.03	× 2.40	=	.07	〃
b	4 @	.03	× 2.60	=	.31	〃
c	16 @	.03	× .28	=	.13	〃
地覆石	10 @	.03	× .30	=	.09	〃
〃	10 @	.03	× .35	=	.11	〃
〃	6 @	.03	× 4.212	=	.76	〃
笠石		.03	× 4.814	=	.14	〃
					15.94 平方米	

長廿六五米分

笠石		.58	× 7.212	=	4.18	表面
窓内面	15 @	.10	× .75	=	1.13	〃
a	12 @	.58	× 3.667	=	4.25	〃
b	2 @	.38	× 3.545	=	2.69	〃
c	14 @	.11	× .20	=	.31	〃
袖 柱	2 @	1.07	× 1.36	=	2.91	〃
〃		.47	× .60	=	.28	〃
地覆石		.68	× 7.212	=	4.90	〃
笠石	16 @	.03	× .58	=	.28	合端
窓内面	15 @	.02	× .77	=	.23	〃
a	7 @	.03	× 3.64	=	.76	〃
〃(親柱寄り)		.03	× 3.00	=	.09	〃
〃(袖柱寄り)		.03	× 3.00	=	.09	〃
b	7 @	.03	× 2.60	=	.55	〃
c	28 @	.03	× .28	=	.24	〃
地覆石	16 @	.03	× .66	=	.32	〃
〃	6 @	.03	× 7.212	=	1.30	〃
笠石(下端)		.03	× 8.30	=	.25	〃
					24.76 平方米	

増田淳氏関係資料
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國道標準型橋梁
八拾四米桁

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CALCULATIONS FOR
常願寺川水路橋材料表

HANGER HR1 R 4 Reqd.

1	L	90×90×10	×	2.440	13.30	32.5	12
1	PI	90×9	×	270	6.36	1.7	
1	"	160×9	×	1.60	11.30	1.8	
						36.0	
						× 4	
						144.0	

Summary for hangers = 144.0 kgs

CAST STEEL SHOES

			kgs		
4	Cast steel shoes	@	1,180.0	4720.0	13
4	Pins & Nuts	@	84.0	336.0	
24	Anchor bolts	@	8.4	201.6	

CAST STEEL BEARING BLOCK FOR PIN AT L0

4	Cast steel	@	139.0 kgs	556.0	14

CAST STEEL SHEAR BLOCK AT T5

4 Sets	Cast steel	@	40.0 kgs	160.0	24

CAST STEEL BRACKETS FOR STRINGERS AT M5

8	Cast steel	@	13.6 kgs	108.8	12

Summary for Cast steel 6082.4 kgs

TOTAL SUMMARY (Rivet heads ⇒ 含マズ)

		Kg.	Tons.		
×	Summary for End posts and Top chords	×	46.9653		
×	Lower chords	×	54.5462	Kg. tons	
	Diagonals	×	13.8592	150.3609 (Trusses)	×
×	Verticals and Hangers	×	25.4450		
×	Ties	×	9.5452		
	Floor beams	×	5.6226		
×	Floor frames	×	15.1248	34.7724 (Floor System)	×
	Stringers	×	14.0250		
	Top lateral bracings	×	3.8314		
	Portal & Sway bracings	×	9.3314		
	Thrust frames and longitudinal ties	×	7.1092		
	Longitudinal & transverse Struts on water way deck	×	6.3703	40.8422 (Bracings etc.)	
	Bottom lateral bracings	×	8.5925		
	Sway bracings under water way deck	×	5.4634		
	Hangers HR1R	×	0.1440		
×			225.9755		
	Cast steel	×	6.0824		
×		×	232.0579 (Truss span complete)		

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CALCULATIONS FOR

國道標準型橋梁八拾四米

Design of truss Span length 14 @ 6.0 = 84.0 meters

Lower panel concentration

Floor slab + pavement	3.75	× 520	=	1,950
Coping and handrails			=	300
Stringers	2.50	× 75	=	188
Floor beam			=	150
Lower laterals			=	75
Truss say	2/3	× 1,100	=	735
misc.			=	32

3,430 kgs per lin meter of truss

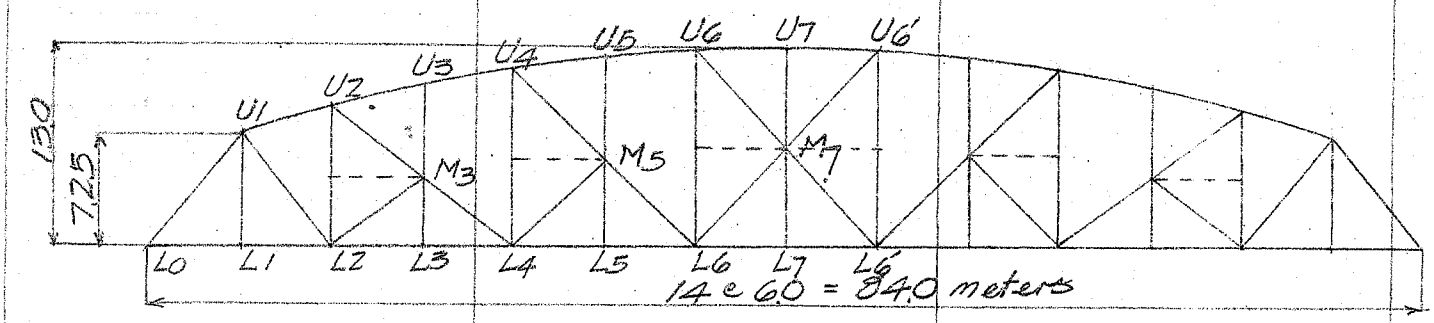
Lower panel concentration 3,430 × 6.0 = 20,600 kgs = P2

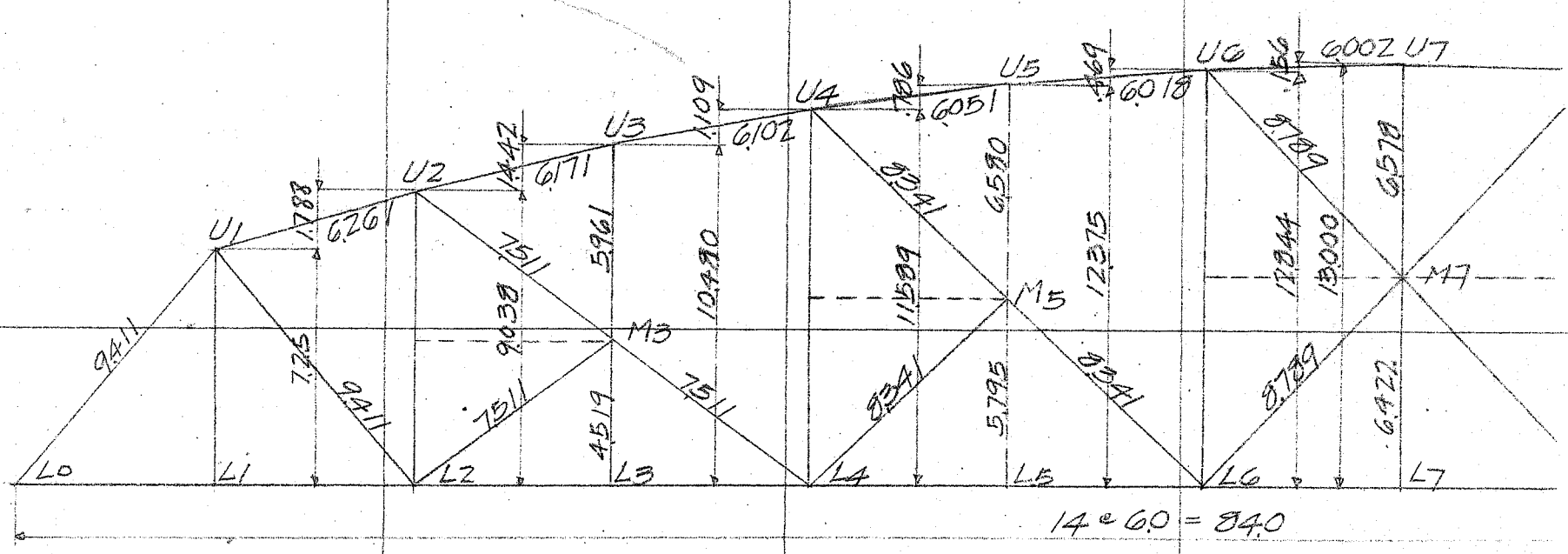
Upper panel concentration

upper laterals		=	70
Sway and portal		=	90
Truss say	1/3 × 1,100	=	370
misc		=	30

560 kgs per lin meter of truss

upper panel concentration 560 × 6.0 = 3,360 kgs = P1





Dead Load stresses

$L_0 - U_1$ $6.5(P_1+P_2) \times \frac{9.411}{7.25} = 8.43(P_1+P_2) = 8.43 \times 23{,}960 = -202{,}000$ kgs

$L_0 - L_2$ $6.5(P_1+P_2) \times \frac{6.00}{7.25} = 5.38(P_1+P_2) = 5.38 \times 23{,}960 = +128{,}700$ kgs

$U_1 - U_2$ Hor. Comp U_1-U_2 $\{6.5(P_1+P_2) \times 2 - (P_1+P_2)\} \times \frac{6.00}{9.038} = 7.97(P_1+P_2)$

Stress $7.97(P_1+P_2) \times \frac{6.261}{6.0} = 8.32(P_1+P_2) = -199{,}200$ kgs

$U_2 - U_4$ Hor. Comp. U_2-U_4 $\{6.5(P_1+P_2) \times 4 - 6(P_1+P_2)\} \times \frac{6.00}{11.599} = 10.35(P_1+P_2)$

Stress U_2-U_3 $10.35(P_1+P_2) \times \frac{6.171}{6.00} = 10.65(P_1+P_2) = -255{,}000$ kgs

U_3-U_4 " $\times \frac{6.102}{6.00} = 10.53(P_1+P_2) = -252{,}300$ kgs

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CALCULATIONS FOR

國道標準型橋梁八拾四米

	vert. Comp U2-U3	$10.35(P_1+P_2)\times\frac{1442}{900}=2.49(P_1+P_2)$
	〃 U3-U4	$\times\frac{1109}{600}=1.91(P_1+P_2)$
U3-M3		$(2.49-1.91)(P_1+P_2)-P_1=0.58(P_1+P_2)-P_1=-0.42P_1+0.58P_2=+10,540$ kgs
L2-L4		$\{6.5(P_1+P_2)\times 2-(P_1+P_2)+P_2+0.42P_1-0.58P_2\}\frac{6.00}{9.038}=8.25(P_1+P_2)$
	8.25 × 23,960	$=+197,500$ kgs
U4-U6	Hor. comp U4-U6	$\{6.5(P_1+P_2)\times 6-15(P_1+P_2)\}\times\frac{6.00}{12.844}=11.22(P_1+P_2)$
	stress U4-U5	$11.22(P_1+P_2)\times\frac{6051}{600}=11.32\times 23,960=-271,000$ kgs
	U5-U6 〃	$\times\frac{6.018}{6.00}=11.25\times 23,960=-269,500$ kgs
	vert. comp. U4-U5	$11.22(P_1+P_2)\times\frac{786}{600}=1.47(P_1+P_2)$
	U5-U6	$\times\frac{469}{600}=0.88(P_1+P_2)$
U5-M5		$(1.47-0.88)(P_1+P_2)-P_1=-0.41P_1+0.59P_2=+10,770$ kgs
L4-L6		$\{6.5(P_1+P_2)\times 4-6(P_1+P_2)+P_2+0.40P_1-0.60P_2\}\frac{6.00}{11.589}=10.56(P_1+P_2)$
		$10.56\times 23,960=+253,000$ kgs
U6-U7	Hor. comp.	$\{6.5(P_1+P_2)\times 8-28(P_1+P_2)\}\times\frac{6.00}{12.844}=11.22(P_1+P_2)$
	stress U6-U7	$11.22\times(P_1+P_2)\times\frac{6002}{600}=11.23\times 23,960=-269,200$ kgs
U7-M7	vert comp U6-U7	$11.22(P_1+P_2)\times\frac{156}{600}=0.29(P_1+P_2)$
	stress U7-M7	$0.29(P_1+P_2)\times 2-P_1=-0.42P_1+0.58P_2=+10,540$ kgs
L6-L7		$\{6.5(P_1+P_2)\times 6-15(P_1+P_2)+P_2+0.42P-0.58P_2\}\frac{6.00}{12.844}=11.41(P_1+P_2)$
		$11.41\times 23,960=+273,400$ kgs
U1-L1	P_2	$=+20,600$ kgs
U1-L2	shear L1~L2	$\{6.5(P_1+P_2)-(P_1+P_2)\}=5.5(P_1+P_2)$
	vert. comp. U1-U2	$7.97(P_1+P_2)\times\frac{1788}{6.00}=2.37(P_1+P_2)$
	stress U1-L2	$(5.5-2.37)(P_1+P_2)\times\frac{9411}{725}=4.06\times 23,960=+97,300$ kgs
M3-L3		$=+20,600$ kgs
M3-L2	Load act M3	$P_2+0.42P_1-0.58P_2=0.42(P_1+P_2)$
	stress M3-L2	$\frac{1}{2}\times 0.42(P_1+P_2)\times\frac{7511}{4519}=0.35\times 23,960=-8,380$ kgs
U2-M3	shear L2~L3	$\{6.5(P_1+P_2)-2(P_1+P_2)\}=4.50(P_1+P_2)$
	vert. comp U2-U3	$=2.49(P_1+P_2)$
	〃 L2-M3	$=0.21(P_1+P_2)$
	stress U2-M3	$(4.50-2.49-0.21)(P_1+P_2)\times\frac{7511}{4519}=2.99\times 23,960=+71,600$ kgs
U2-L2	vert. comp. U1-L2	$=3.13(P_1+P_2)$
	〃 L2-M3	$=0.21(P_1+P_2)$
	stress U2-L2	$(3.13-0.21)(P_1+P_2)-P_2=2.92P_1+1.92P_2=-49,300$ kgs
M3-L4	vert. comp. U3-U4	$=1.91(P_1+P_2)$
	shear L3~L4	$\{6.5(P_1+P_2)-3(P_1+P_2)\}=3.50(P_1+P_2)$
	vert. comp M3-L4	$=0.21(P_1+P_2)$
	stress M3-L4	$\{3.50(P_1+P_2)-1.91(P_1+P_2)-0.21(P_1+P_2)\}\times\frac{7511}{4519}=2.29\times(P_1+P_2)$
		$=+54,900$ kgs
M5-L5		$=+20,600$ kgs

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

國道標準型橋梁八拾四米

M5-L4 Load act M5 $P_2 + 0.40P_1 - 0.60P_2 = 0.40(P_1+P_2)$
Stress M5-L4 $\frac{1}{2} \times 0.40(P_1+P_2) \times \frac{8341}{5795} = 0.29(P_1+P_2) = -6,940$ Kgs

U4-L4 vert. comp M3-L4 $= 1.39(P_1+P_2)$
vert comp L4-M5 $= 0.20(P_1+P_2)$
Stress U4-L4 $1.39(P_1+P_2) - P_2 - 0.20(P_1+P_2) = 1.19P_1 + 0.19P_2 = -7,670$ Kgs

U4-M5 vert comp U4-U5 $11.22(P_1+P_2) \times \frac{786}{600} = 1.47(P_1+P_2)$
" M5-L4 $= 0.20(P_1+P_2)$
Shear L4~L5 $= \{6.5(P_1+P_2) - 4(P_1+P_2)\} = 2.50(P_1+P_2)$
Stress U4-M5 $\{2.50(P_1+P_2) - 1.47(P_1+P_2) - 0.20(P_1+P_2)\} \times \frac{8341}{5795} = 1.20 \times 23,960$
$= +28,750$ Kgs

M5-L6 vert. comp U5-U6 $11.22 \times (P_1+P_2) \times \frac{469}{600} = 0.88(P_1+P_2)$
vert. comp M5-L6 $= 0.20(P_1+P_2)$
Shear L5~L6 $6.5(P_1+P_2) - 5.0(P_1+P_2) = 1.5(P_1+P_2)$
Stress M5-L6 $= (1.5 - 0.88 - 0.20)(P_1+P_2) \times \frac{8341}{5795} = .60(P_1+P_2) = +14,360$ Kgs

M7-L7 $= +20,600$ Kgs

M7-L6 Load act M7 $P_2 + 0.42P_1 - 0.58P_2 = 0.42(P_1+P_2)$
stress M7-L6 $\frac{1}{2} \times 0.42(P_1+P_2) \times \frac{3789}{6422} = 0.29(P_1+P_2) = -6,940$ Kgs

U6-L6 vert comp. M5-L6 $= 0.42(P_1+P_2)$
vert comp L6-M7 $= 0.21(P_1+P_2)$
stress U6-M6 $= 0.42(P_1+P_2) - P_2 - 0.21(P_1+P_2) = 0.21P_1 - 0.79P_2 = +15,550$ Kgs

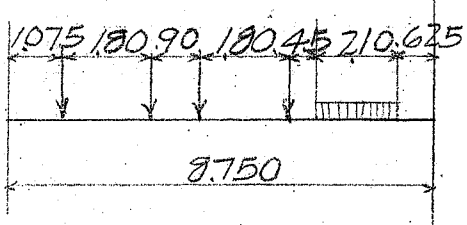
U6-M7 vert comp. U6-U7 $= 0.29(P_1+P_2)$
" L6-M7 $= 0.21(P_1+P_2)$
Shear L6~L7 $6.50(P_1+P_2) - 6.0(P_1+P_2) = 0.50(P_1+P_2)$
stress U6-M7 $= \{0.50(P_1+P_2) - 0.29(P_1+P_2) - 0.21(P_1+P_2)\} \times \frac{3789}{6422} = 0$

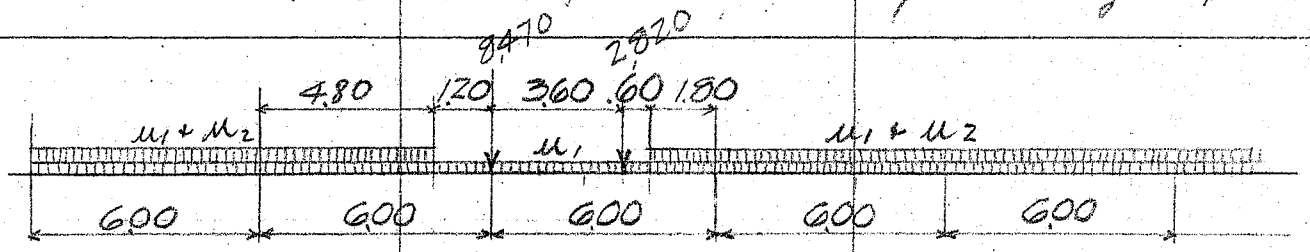
Live Load stresses
Live Load on truss uniform live load $w = \frac{100000}{170+84} = 394$ Kgs/cm²

Impact $= \frac{20}{60+84} = 13.9\%$.

motor truck loading Rear wheel 3,000 Kgs
13.9% impact 417
3,417 Kgs
Front wheel $3,417 \div 3$ = 1,138 Kgs including impact

Panel concentration





	948	948	948	948
$\frac{394 \times 210 \times 1675}{875} = 158$ Kgs $= M_1$	3,960			
	3,900	2,530		
$394 \times 3.75 = 1,477$ Kgs		356	2,018	
-158			3,960	7,920
$1,319$ Kgs $= M_2$		8,470		
		1,130	1,690	
Rear wheel $4 \times 3,417 \times \frac{5425}{875} = 8,470$ Kgs	8,708	13,434	8,616	8,868

Front wheel $8,470 \div 3 = 2,820$ Kgs

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（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

國道標準型橋梁八拾四米

assume the panel concentration for live load 9,870 Kgs for panel throughout as full uniform load and 13,434 - 9,870 = 4,564 Kgs or say 4,560 Kgs concentration for panel load of motor truck.

Stresses due to 9,870 Kgs concentration

$L_0 - U_1$		8.43 × 9,870	= − 74,900 Kgs
$L_0 - L_2$		5.38 × "	= + 47,700
$U_1 - U_2$		8.32 × "	= − 73,900
$U_2 - U_3$		10.65 × "	= − 94,500
$U_3 - U_4$		10.53 × "	= − 93,400
$L_2 - L_4$		8.25 × "	= + 73,200
$U_4 - U_5$		11.32 × "	= − 100,300
$U_5 - U_6$		11.25 × "	= − 99,800
$L_4 - L_6$		10.56 × "	= + 93,600
$U_6 - U_7$		11.23 × "	= − 99,600
$L_6 - L_7$		11.41 × "	= + 101,200
$U_1 - L_1$		1.00 × "	= + 9,870
$M_3 - L_3$		1.00 × "	= + 9,870
$M_5 - L_5$		1.00 × "	= + 9,870
$M_7 - L_7$		1.00 × "	= + 9,870
$U_3 - M_3$		0.58 × "	= + 5,140
$U_5 - M_5$		0.59 × "	= + 5,240
$U_7 - M_7$		0.58 × "	= + 5,140
$L_2 - M_3$	$(0.50P - 0.03P) \times \frac{7511}{4519}$	= 0.78 × "	= − 6,920
	$\frac{3P}{14} \times 10 \times 60 \div 11,589 \times (1442 - 1109) \div 6 = 0.06P$		
$L_4 - M_5$	$(0.50P - 0.04P) \times \frac{8341}{5795}$	= 0.66 × "	= − 5,850
	$\frac{5P}{14} \times 8 \div 12,844 \times (786 - 469) = 0.07P$		
$L_6 - M_7$	$(0.50P - 0.04P) \times \frac{8789}{6422}$	= 0.63 × "	= − 5,590
	$\frac{7P}{14} \times 6 \div 12,844 \times (2 \times 156) = 0.07P$		

$U_1 - L_2$

vert comp $U_1 - U_2 = \frac{(1 \sim 12)P \times 2}{14} \times \frac{60 \times 1,788}{9,038 \times 6.0} = 2.21P$

shear $L_1 \sim L_2 = \frac{(1 \sim 12)P}{14} = 5.57P$

stress $U_1 - L_2$ $(5.57 - 2.21)P \times \frac{9411}{725} = 4.36 \times 9,870 = +$ 39,700

$U_2 - L_2$

shear $L_1 \sim L_2 = \frac{(1 \sim 11)P}{14} = 4.72P$

vert. comp $U_1 - U_2 = \frac{(1 \sim 11)P \times 2 \times 1,788}{14 \times 9,038} = 1.87P$

" " $M_3 - L_2 = 0.5P$

Horiz. comp $U_2 - U_4 = \{\frac{(1 \sim 11)P \times 4}{14} - P\} \times \frac{60}{11,589} = 9.25P$

stress $U_3 - M_3 = 9.25P \times (1,442 - 1,109) \div 6.00 = 0.51P$

Stress $U_2 - L_2 = (4.72 - 1.87 - 0.50 + 0.26)P = 2.61 \times 9,870 = -$ 23,150

$U_2 - M_3$

shear $L_2 \sim L_3 = 4.72P$

vert comp $U_2 - U_3 = 9.25P \times 1442 \div 6.0 = 2.22P$

Stress $U_2 - M_3 = (4.72 - 2.22 - 0.50 + 0.26)P \times \frac{7511}{4519} = 3.76P$

$= 3.76 \times 9,870 = +$ 33,300

$M_3 - L_4$

vert comp $U_3 - U_4$ $\frac{(1 \sim 10)P \times 4 \times 60 \times 1109}{14 \times 11,589 \times 6.0} = 1.51P$

shear $L_3 \sim L_4 = \frac{(1 \sim 10)P}{14} = 3.93P$

Stress $M_3 - L_4$ $(3.93 - 1.51 + 0.45)P \times \frac{7511}{4519} = 4.77 \times 9,870 = +$ 47,300

Stress $M_3 - U_3$ $1.51P \times \frac{(1442 - 1109)}{1109} = 0.45P$

$U_4 - L_4$

shear $L_3 \sim L_4 = \frac{(1 \sim 9)P}{14} = 3.21P$

vert. comp $U_3 - U_4 = \frac{(1 \sim 9)P}{14} \times 4 \times \frac{1109}{11,589} = 1.23P$

" " L_4 $M_5 = 0.5P$

Horiz. comp $U_4 - U_6 = \{\frac{(1 \sim 9)P}{14} \times 6 - P\} \times \frac{60}{12,844} = 8.55P$

stress $U_5 - M_5 = 8.55P \times (786 - 469) \div 6.00 = 0.45P$

stress $U_3 - M_3 = 1.23P \times \frac{(1442 - 1,109)}{1,109} = 0.37P$

Stress $U_4 - L_4$ $(3.21 - 1.23 - 0.5 + 0.23 + 0.19)P = 1.90 \times 9,870 = -$ 16,850

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

國道標準型橋梁八拾四米

U4 - M5
shear L4~L5 = 3.21P
vert comp U4-U5 8.55P × .786 ÷ 6.00 = 1.12P
stress U4-M5 = (3.21 - 1.12 - 0.50 + 0.23)P × 2341/5795 = 2.62P × 8,870 = + 23,200

M5 - L6
vert comp U5-U6 (1~8)P × 6 × 60 × 469 / 4 × 12844 × 60 = 0.56P
shear L5~L6 (1~8)P = 2.57P
stress U5-M5 0.56P × (.786-.469)/4 = 0.38P
stress M5-L6 (2.57 - 0.56 + 0.19)P × 2341/5795 = 3.17 × 8,870 = + 28,100

U6-L6
shear L5~L6 = (1~7)P/4 = 2.0P
vert comp U5-U6 2.0P × 6 × 469/12844 = 0.44P
vert comp L6-M7 = 0.5P
Horiz. comp U6-U6' {(1~7)P 8.0-P} × 6.0/12.844 = 7.02P
stress U7 - M7 = 7.02 × 14.156 × 2 = 0.37P
stress U5 - M5 = 0.44P × 6.0(.786-.469) = 0.30P
stress U6-L6 (2.0 - 0.44 - 0.5 + 0.18 + 0.15)P = 1.39 × 8,870 = - 12,320

U6 - M7
shear L6~L7 = 2.0P
vert comp U6-U7 7.02 × 1.56/6.00 = 0.18P
stress U6-M7 (2.00 - 0.18 - 0.50 + 0.18)P × 8.789/6.422 = 2.05 × 8,870 = + 18,170

M7 - L6'
shear L7 ~ L6' = (1~6)P/14 = 1.5P
Hor. comp. U7-U6' 1.5P × 8 × 6.0/12.944 = 5.61P
vert. comp U7-U6' 5.61P × 1.56/60 = 0.15P
stress M7-L6' (1.5 + 0.14 + 0.14)P × 8.789/6.422 = 2.44 × 8,870 = + 21,600

U6'-M5'
shear L6'~L5' = (1~4)P/14 = 0.71P
Hor. comp U6-U5 (1~4)P × 60 ÷ 11589 = 3.70P
stress U5-M5 = 3.70P × (.786-.469) ÷ 60 = 0.20P
vert comp U6-U5 = 3.70P × 469 ÷ 60 = 0.29P
stress U6-M5' (0.71 + 0.29 + 0.10)P × 9.257/7.049 = 1.44 × 8,870 = + 12,750

Stress due to 4,560 Kgs single concentration
L0 - U1 13P/14 × 9.411 ÷ 7.25 = 1.21 × 4,560 = - 5,520
U1 - U2 12P × 2 × 60 × 6261 / 14 × 9.038 × 6.0 = 1.19 × 4,560 = - 5,420
U2 - U3 Hor. comp U2~U4 10P × 4 × 60 / 14 × 11589 = 1.48P
stress U2 - U3 1.48P × 6.17/6.0 = 1.52 × 4,560 = - 6,930
U3 - U4 1.48P × 6.102/6.0 = 1.50 × 4,560 = - 6,840
U4 - U5 Hor. comp U4~U6 8P × 6 × 60 / 14 × 12844 = 1.60P
stress U4-U5 1.60P × 6.051/6.0 = 1.61 × 4,560 = - 7,340
U5 - U6 1.60P × 6.018/6.0 = 1.60 × 4,560 = - 7,300
U6 - U7 Hor. comp U6~U6' 6P × 8 × 6.0 / 14 × 12844 = 1.60P
stress U6 - U7 1.60P × 6.062/6.0 = 1.60 × 4,560 = - 7,300

L0 - L2 13P/14 × 6.00/7.25 = 0.77 × 4,560 = + 3,510
L2 - L4 (10/14 × 2 - 0.08) × 6P ÷ 9.038 = 0.90 × 4,560 = + 4,100
L4 - L6 (8/14 × 4 - 0.08) × 6P ÷ 11589 = 1.15 × 4,560 = + 5,240
L6 - L7 (6/14 × 6 - 0.08) × 6P ÷ 12844 = 1.16 × 4,560 = + 5,290

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

國道標準型橋桁八拾四米

U_1-L_1			P = + 4,560 kg.	1
U_1-L_2	shear L_1-L_2	$\frac{12P}{14} = 0.86P$		2
	vert comp U_1-U_2	$\frac{12P \times 2 \times 60 \times 1.788}{14 \times 9.038 \times 60} = 0.34P$		
	stress U_1-L_2	$(0.86-0.34)P \frac{9.411}{7.25} = 0.68 \times 4,560$	= + 3,100	
U_2-L_2	shear L_0-L_4	$\frac{10P}{14} = 0.71P$		4
	vert comp U_1-U_2	$\frac{10P \times 2 \times 60 \times 1.788}{14 \times 9.038 \times 60} = 0.28P$		
	Hor comp U_2-U_4	$1.48P$		
	stress U_3-M_3	$0.08P$		
	vert comp L_2-M_3	$0.04P$		
	stress U_2-L_2	$(0.71-0.28+0.04)P = 0.47 \times 4,560$	= − 2,140	
U_3-M_3	$1.48P \times (1.442-1.109) \div 6.0$	$= 0.08 \times 4,560$	= + 365	4
U_5-M_5	$1.60P \times (.786-.469) \div 6.0$	$= 0.08 \times 4,560$	= + 365	6
U_7-M_7	$1.60P \times 2 \times .156 \div 6.0$	$= 0.08 \times 4560$	= + 365	8
U_4-L_4	shear L_0-L_6	$\frac{8P}{14} = 0.57P$		6
	vert comp U_3-U_4	$\frac{8P \times 4 \times 60 \times 1.109}{14 \times 11.589 \times 60} = 0.22P$		
	vert comp L_4-M_5	$0.04P$		
	stress U_4-L_4	$(0.57-0.22+0.04)P = 0.39 \times 4,560$	= − 1,780	
U_6-L_6	shear L_0-L_8	$\frac{6P}{14} = 0.43P$		6
	vert comp U_5-U_6	$\frac{6P \times 6 \times 60 \times .469}{14 \times 12.844 \times 60} = 0.09P$		
	vert comp L_6-M_7	$0.04P$		
	stress U_6-L_6	$(0.43-0.09+0.04)P = 0.38 \times 4,560$	= − 1,730	
M_3-L_3		$= 1.00 \times 4,560$	= + 4,560	3
M_5-L_5		$= 1.00 \times 4,560$	= + 4,560	5
M_7-L_7		$= 1.00 \times 4,560$	= + 4,560	7
L_2-M_3	Hor. comp U_2-U_4	$\frac{3P \times 10 \times 60}{14 \times 11.589} = 1.11P$		3
	stress U_3-U_3	$1.11P(1.442-1.109) = 0.06P$		
	stress L_2-M_3	$P(0.50 \frac{6.0}{60} - 0.03) \times \frac{7.511}{4.519} = 0.78 \times 4,560$	= − 3,555	
L_4-M_5	Hor. comp U_4-U_6	$\frac{5P \times 8 \times 60}{14 \times 12.844} = 1.34P$		5
	stress U_5-M_5	$\frac{1.34P(.786-.469)}{60} = 0.07P$		
	stress L_4-M_5	$(0.50-0.04)P \times \frac{4.341}{5.795} = 0.66 \times 4,560$	= − 3,010	
L_6-M_7	Hor. comp U_6-U_6	$\frac{7P \times 6 \times 60}{14 \times 12.844} = 1.40P$		7
	stress U_7-M_7	$\frac{1.40P \times 2 \times .156}{60} = 0.07P$		
	stress L_6-M_7	$(0.50-0.04)P \times \frac{3.784}{6.422} = 0.63 \times 4,560$	= − 2,870	
U_2-M_3	shear L_0-L_4	$\frac{10P}{14} = 0.71P$		4
	Hor. comp	$U_2-U_4 = 1.48P$		
	vert comp	U_2-U_3 $1.48P \times 1.442 \div 6.0 = 0.36P$		
	stress U_3-M_3	$= 0.08P$		
	stress U_2-M_3	$(0.71-0.36+0.04)P \times \frac{7.511}{4.519} = 0.65 \times 4,560$	= + 2,960	
M_3-L_4	vert. comp U_3-U_4	$1.48P \times 1.109 \div 6.0 = 0.27P$		8
	stress M_3-L_4	$(0.71-0.27+0.04)P \times \frac{7.511}{4.519} = 0.80 \times 4560$	= + 3,650	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

國道標準型橋梁 A 拾四米

U4 - M5 Shear L0-L6 $\frac{8P}{14}$ = 0.57P
Hor. comp U4-U5 = 1.60P
vert. comp U4-U5 = $1.60P \times \frac{.786}{6.0}$ = 0.21P
stress U5-M5 = 0.08P
stress U4-M5 $(0.57 - 0.21 + 0.04)P \times \frac{8341}{5795}$ = 0.58P × 4,560 = + 2,640

M5-L6 vert. comp U5-U6 = $1.60P \times \frac{.469}{6.0}$ = 0.13P
stress M5-L6 $(0.57 - 0.13 + 0.04)P \times \frac{8341}{5795}$ = 0.69P × 4,560 = + 3,150

U6-M7 Shear L0-L6 $\frac{6P}{14}$ = 0.43P
Hor. comp U6-U7 = 1.60P
vert. comp U4-U5 = $1.60P \times \frac{.156}{6.0}$ = 0.04P
stress U7-M7 = 0.08P
stress U6-M7 $(0.43 - 0.04 + 0.04)P \times \frac{8789}{6422}$ = 0.59P × 4560 = + 2,690

M7-L6' vert comp U7-U6' = 0.04P
stress M7-L6' $(0.43 - 0.04 + 0.04)P \times \frac{8789}{6422}$ = 0.59P × 4,560 = + 2,690

U6'-M5' Shear L0-L4' $\frac{4P}{14}$ = 0.29P
Hori. comp U6'-U4' 0.29P × 4 × 60 = 11.589 = 0.60P
vert. comp U6'-U5' 0.60P × .469 ÷ 60 = 0.05P
vert comp U5'-M4' 0.60P × .786 ÷ 60 = 0.09P
vert comp U5'-M5' 0.04P
stress U6'-M5' $(0.29 - 0.05 + 0.02)P \times \frac{9257}{7049}$ = 0.34P × 4,560 = + 1,550

M5'-L4' stress M5'-L4' $(0.29 - 0.09 + 0.02)P \times \frac{8341}{5795}$ = 0.32P × 4,560 = + 1,460

uniform live load Load on L4'-L6'
M5'-L6' Shear $\frac{(1\sim4)P}{14}$ = 0.71P
Hori. comp. U6'-U4' (.71P × 10) × 6 ÷ 11.589 = 3.68P
vert. comp. U6'-U5' 3.68P × .469 ÷ 6.0 = 0.29P
Vert. comp U5'-M5' 3.68 × (.786 - .469)P ÷ 60 = 0.19P
stress M5'-L6' (0.71 + 0.29 - 0.10)P × $\frac{8341}{5795}$ = 1.30P
1.30 × 8,870 = - 11,500

Single Live Load
M5'-L6' Shear $\frac{4P}{14}$ = 0.29P
Hori. comp. U6'-U4' 0.29P × 10 × 6 ÷ 11.589 = 1.50P
vert comp U6'-U5' 1.50P × .469 ÷ 6.0 = 0.12P
vert comp U5'-M5' 1.50P × (.786 - .469) ÷ 6.0 = 0.08P
stress M5'-L6' (0.29 + 0.12 - 0.04)P × $\frac{8341}{5795}$ = 0.53P
0.53 × 4,560 = - 2,420

uniform live load
U4'-M5' vert comp U5'-U4' 3.68P × .786 ÷ 6.0 = 0.48P
stress U4'-M5' (0.71 + 0.48 - 0.10)P × $\frac{8341}{5795}$ = 1.57P
1.57 × 8,870 = - 13,920

Single live load
U4'-M5' vert comp U5'-U4' 1.50P × (.786) ÷ 6.0 = 0.20P
stress U4'-M5' (0.29 + 0.20 - 0.04)P × $\frac{8341}{5795}$ = 0.65P
0.65 × 4,560 = - 2,960

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（独立行政法人 土木研究所蔵）

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CONSULTING ENGINEER
JIJI BLDG, TOKYO

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CALCULATIONS FOR

國道標準型橋梁八拾四米

L6'-M5'

Uniform live load Load on L5'-L4'

Shear $= \frac{(1\sim5)P}{14} = 1.07P$

Horiz. comp. U6-U4' $(1.07P \times 10 - P) \times 6 \div 11.589 = 5.02P$

Vert. comp. U6-U5' $5.02P \times .469 \div 6.0 = 0.39P$

Vert. Comp U5-M5' $5.02P \times (.786 - .469) \div 6.0 = 0.27P$

Stress M5'-L6' $(1.07 - 0.5 - 0.27 + 0.39)P \times \frac{3341}{5795} = 0.995P$

$0.995 \times 8,870 = -8,830 kg$

Single concentration Load on L5'

Shear $\frac{5P}{14} = 0.36P$

Horiz. Comp. U6-U4' $\frac{9P}{14} \times 6 \times 4 \div 11.589 = 0.88P$

Vert. comp U6-U5' $0.88P \times .469 \div 6.0 = 0.07P$

Vert comp U5'-M5' $0.88P \times (.786 - .469) \div 6.0 = 0.05P$

stress M5'-L6' $(0.36 - 0.5 - 0.05 + 0.07)P \times \frac{3341}{5795} = +0.17P$

$+ 0.17 \times 4,560 = + 775 kg$

Total Live Load stress $-8,830 + 775 = -8,055 kg$.

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CALCULATIONS FOR

U2 – L1
1 Cov. Pl 680×10 = 68.0
2 Web Pls 550×10 = 110.0
2 Web Pls 550×10 = 110.0
2 Ls 90×90×13 = 43.42
2 Ls 125×90×13 = 52.52
383.94
DL – 269,200
LL – 106,900
– 376,100
DL – 271,000
LL – 107,640
– 378,640
DL – 269,500
LL – 107,140
– 376,640
DL – 255,000
LL – 101,430
– 356,430
L0 – L1 – L2
1 Cov. Pl 680×10 = 68.0
2 Web Pls 550×10 = 110.0
2 Web Pls 370×10 = 74.0
2 Ls 90×90×10 = 34.0
2 Ls 125×90×10 = 41.0
327.0
DL – 252,300
LL – 100,240
– 352,540
DL – 255,000
LL – 101,430
– 356,430
DL + 273,400
LL + 106,490
379,890
DL + 253,000
LL + 98,840
351,840
DL + 197,500
LL + 77,300
+ 274,800
DL + 129,700
LL + 51,210
+ 179,910
SR = 317
SR = 293
SR = 229
SR = 150
4 Bars 20×4 = 320.0
4 Bars 20×4 = 320.0
2 Bars 20×3 = 240.0
4 Bars 20×3 = 240.0
2 Bars 20×4 = 160.0
2 Bars 20×3 = 120.0
2 Bars 20×3 = 120

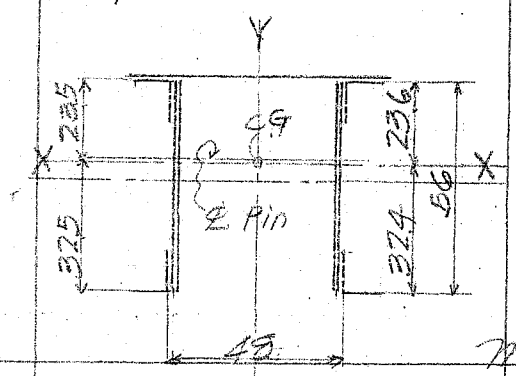
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CALCULATIONS FOR

Section of truss members

Top chord members $U_2 \sim U_3 \sim U_4 \sim U_5 \sim U_6 \sim U_7$ Max. stress – 378,640 kgs



1 Cov. Pl	680 × 10 =	68.0	×	56.5	=	3,840
2 Web Pls	550 × 10 =	110.0	×	28.0	=	3,080
2 Web Pls	550 × 10 =	110.0	×	28.0	=	3,080
2 Ls	90×90×13 =	43.42	×	53.34	=	2,310
2 Ls	125×90×13 =	52.52	×	2.33	=	120
		383.94		32.4		12,430

Moment of inertia X–X axis

1 Cov. Pl	680 × 10 =	68.00×24.1^2	=	39,500
2 Web Pls	550 × 10 =	$110.00 \times 4.4^2 + \frac{2 \times 55^3}{12}$	=	29,880
2 Web Pls	550 × 10 =	$110.00 \times 4.4^2 + \frac{2 \times 55^3}{12}$	=	29,880
2 Ls	90×90×13 =	$43.42 \times 20.94^2 + 315$	=	19,320
2 Ls	125×90×13 =	$52.52 \times 30.07^2 + 348$	=	47,750
				166,330

Radius of gyration $R_x = \sqrt{\frac{166330}{383.94}} = 20.8^{cm}$ $\frac{l}{r} = \frac{617.1}{20.8} = 29.7$

moment of inertia Y–Y axis

1 cov. Pl	680 × 10 =	$68.00 \times \frac{68^2}{12}$	=	26,200
2 Web Pls	550 × 10 =	110.00×22.5^2	=	55,800
2 Web Pls	550 × 10 =	110.00×23.5^2	=	60,700
2 Ls	90×90×13 =	$43.42 \times 26.66^2 + 315$	=	30,020
2 Ls	125×90×13 =	$52.52 \times 28.06^2 + 805$	=	42,210
				214,930

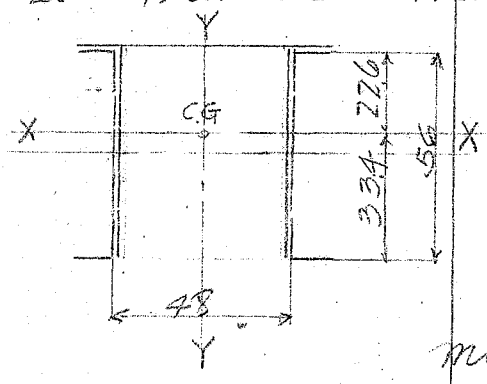
allowable unit compression = $1500(1 - 0.0055\frac{l}{r}) = 1,360$ kgs use 1000 kg/cm²

Direct compression $378640 \div 383.94 =$ 988

Bending stress $\frac{378,640 \times 9.1 \times 32.5}{138,530} =$ 9

997 kgs/cm²

$L_0 \sim U_1$, $U_1 \sim U_2$ max. stress – 282,320 kgs



1 Cov. Pl	680 × 10 =	68.0	×	56.5	=	3,840
2 Web Pls	550 × 10 =	110.0	×	28.0	=	3,080
2 Web Pls	370 × 10 =	74.0	×	28.0	=	2,070
2 Ls	90×90×10 =	34.0	×	53.44	=	1,820
2 Ls	125×90×10 =	41.0	×	2.22	=	90
		327.0		33.4		10,900

Moment of inertia X–X axis

1 cov. Pl	680 × 10 =	68.00×23.1^2	=	36,200
2 Web Pls	550 × 10 =	$110.00 \times 5.4^2 + \frac{2 \times 55^3}{12}$	=	30,910
2 Web Pls	370 × 10 =	$74.0 \times 5.4^2 + \frac{2 \times 37^3}{12}$	=	10,590
2 Ls	90×90×10 =	$34.0 \times 20.04^2 + 249$	=	13,950
2 Ls	125×90×10 =	$41.0 \times 31.18^2 + 276$	=	40,080
				131,730

Radius of gyration $R_x = \sqrt{\frac{131,730}{327.0}} = 20.0$ $\frac{l}{r} = \frac{941.1}{20.0} = 47.0$

Moment of inertia Y–Y axis

1 Cov. Pl	680 × 10 =	$68.00 \times \frac{68^2}{12}$	=	26,200
2 Web Pls	550 × 10 =	110.00×23.5^2	=	60,900
2 Web Pls	370 × 10 =	74.00×24.5^2	=	44,500
2 Ls	90×90×10 =	$34.00 \times 26.56^2 + 249$	=	24,250
2 Ls	125×90×10 =	$41.00 \times 27.95^2 + 635$	=	32,740
				188,590

allowable unit compression = $1500(1 - 0.0055\frac{l}{r}) = 1,110$ kgs use 1000 kg/cm²

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CALCULATIONS FOR

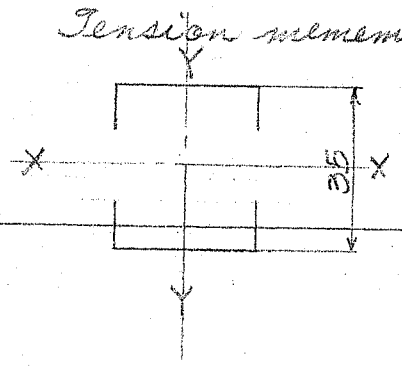
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Direct stress $282320 \div 327.0 = 865$

Bending stress $\frac{282320 \times 0.9 \times 23.5}{131,730} = \frac{46}{911}$

Vertical members

Tension memembers max. stress +34,030kgs SR = 29.3 cm²



2 ⊏ 300×90×38.1 kgs = 97.13 cm²

use 17 cm φ Pin Reduction holes − 40.1 = 57.03 cm²

Moment of inertia X−X axis

2 ⊏ 300·90·38.1 = 97.13 × 15.27² + 649 = 23,250

Radius of gyration $r_x = \sqrt{\frac{23250}{97.13}} = 15.5$

Radius of gyration $r_y = 11.51$ $\frac{l}{r} = \frac{1284.4}{11.55} = 111$

Compression members U4 − L4, max. stress − 26,300kgs
use same as tension members

allowable unit compression $1,500\left(1-0.0055\frac{l}{r}\right) = 1,500\left(1-0.0055\frac{1158.9}{11.55}\right) = 675\ kg/cm^2$

$S.R = \frac{26,300}{675} = 39\ cm^2$

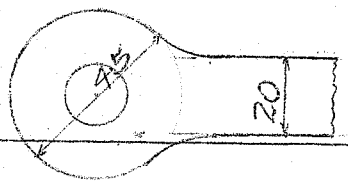
U2 − L2 max. stress − 74,590kgs
use same as tension members and 9 mm pin P/s

allowable unit compression $1,500\left(1-0.0055\frac{l}{r}\right) = 1,500\left(1-0.0055\frac{903.8}{11.55}\right) = 855\ kg/cm^2$

$S.R = \frac{74,590}{855} = 87.3\ cm^2$

Diagonal members

U1 ~ L2 max. stress + 139,100kgs SR = 116. cm²



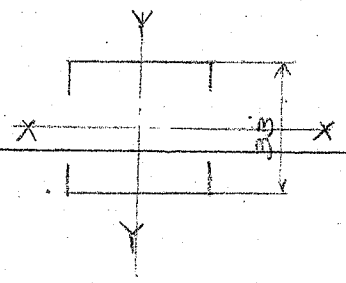
use eye bars 2 − 20 × 30 cm

U2 − M3, M3 − L4 max. stress + 107,960kgs SR = 90 cm²

use eye bars 2 − 20 × 25 cm

U4 − M5, M5 − L6, U6 − M7, M7 ~ L6'
same as U2 − M3

M3 ~ L2, M5 ~ L4 max. stress − 18,855kgs



2 ⊏ 250·90·34.6 kgs = 88.14

Moment of inertia X−X axis

2 ⊏ 250·90·34.6 = 88.14 × 14.08² + 612 = 18,060

$r_x = \sqrt{\frac{18,060}{88.14}} = 14.2$

$r_y = 9.7$ $\frac{l}{r} = \frac{878.9}{9.7} = 90.5$

allowable unit compression $1,500(1-0.0055\times 90.5) = 746\ kg/cm^2$

SR = 25.3 cm²

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CONSULTING ENGINEER
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CALCULATIONS FOR

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Lower chords section

L6-L7 max. stress + 379,890 Kgs SR = 317
use eye bars 4 × 20 × 4 cm = 320

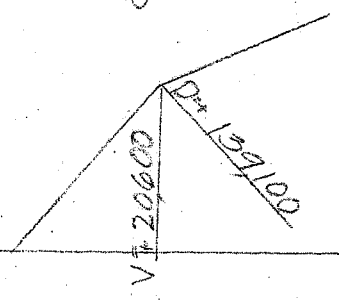
L4-L5, L5-L6 use same as L6-L7

L2-L3, L3-L4 max. stress + 274,800 Kgs SR = 229
use eye bars 4 × 20 × 3 = 240

L0-L1, L1-L2 max. stress + 179,910 Kgs SR = 150
use eye bars 2 × 20 × 4 = 160

Design of pin

Shearing stress max. shearing stress of upper panel point occured at U1



vertical component of D = VD = $139100 \times \frac{725}{9.411} = +107{,}200$

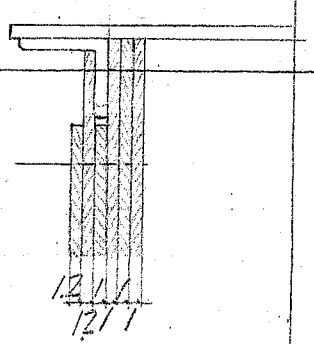
vert. shear $\frac{20{,}600 + 107{,}200}{2} = 63{,}900$ Kgs

shear direction of D $20{,}600 \times \frac{725}{9.411} = 15{,}900$

shear $\frac{1}{2}(139{,}100 + 15{,}900) = 77{,}500$ Kg.

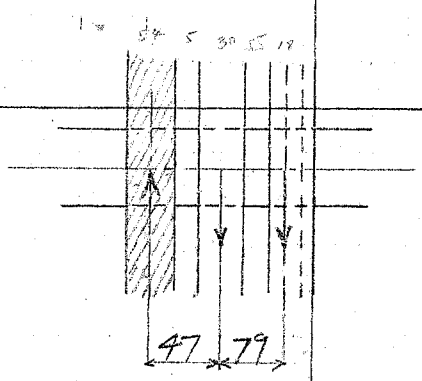
shearing stress = $\frac{77500}{9.5^2 \times \pi} = 342$ Kgs

Bearing stress max. Bearing stress of upper panel point occured at U4
stress - 378,640 Kgs



Bearing stress $\frac{378{,}640}{17 \times 6.4 \times 2} = 1{,}765$ Kgs/cm²

Bending stress Bending stress at U1



Reaction = 77,500 Kgs
max. moment = 77,500 × 12.6 − 69,550 × 7.9 = 427,000 Kg cm

Resisting moment of 17cm/m pin = $\frac{\pi \times 1800 \times 17^3}{32} = 868{,}200$ Kg cm

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JIUN MASUDA
CONSULTING ENGINEER
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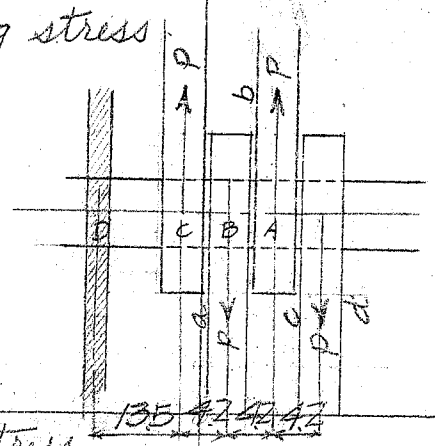
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CALCULATIONS FOR

Design of Lower chord Pin.

Shearing stress

L7



Shear at a 95,000
" b 0
" c 95,000
" d 0

Shearing stress $= \frac{95,000}{\pi \cdot 9.5^2} = 418 \text{ kgs/cm}^2$

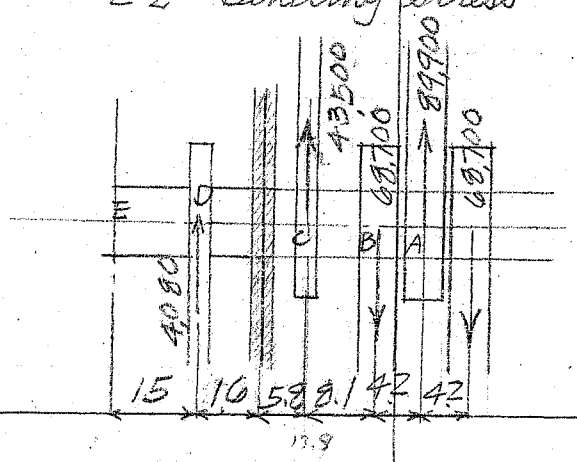
Bearing stress $\frac{95000 \cdot}{17 \cdot 4} = 1,395 \text{ kgs/cm}^2$

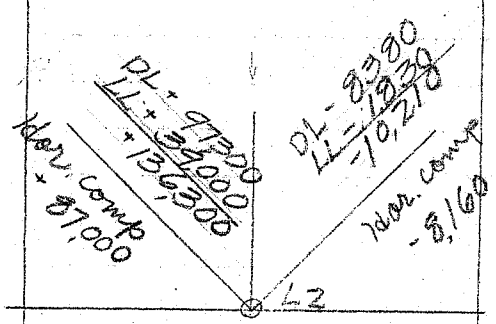
Bending stress

moment at A $95,000 \times 4.2 = 399,000$
" B $95,000 \times (9.4 - 4.2) = 399,000$
" C $95,000 \times (12.6 - 8.4 + 4.2) = 798,000$
" D $95,000 \times (26.1 - 21.9 + 17.7 - 13.5) = 798,000$

Resisting moment of 17cm ϕ Pin = 868,200 kg cm

L2 Bending stress





DL + 128,700
LL + 50,940
+ 179,640

DL + 197,500
LL + 77,300
+ 274,800

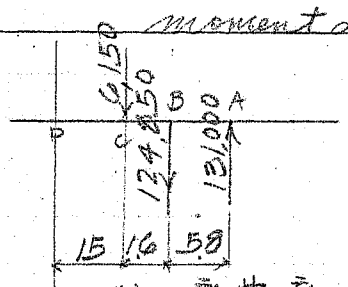
L1-L2
uniform live load stress +47,700
live load single concentration act at L2 $3,510 \times \frac{12}{13} = +3,240$
+50,940

U1-L2 uniform live load stress load act all panels
vert. comp $U_1 - U_2 = (6.5P \times 2 - P) \times \frac{60 \times 1.788}{9.038 \times 60} = 2.38P$
shear L1-L2 = $= 5.50P$
stress U1-L2 $(5.50 - 2.38)P \times \frac{94.11}{7.25} = 4.05 \times 8,870 = 35,900 \text{ kgs}$

live load single concentration 3,100 kgs

moment at A $68,700 \times 4.2 = +288,000$
B $68,700 \times 8.4 - 89,900 \times 4.2 = +200,000$
C $68,700 \times (16.5 + 8.1) - 89,900 \times 12.3 = +595,000$
D $68,700 \times (23.9 + 15.5) - 89,900 \times 19.7 - 43,500 \times 7.4 = +618,000$
E $68,700 \times (38.9 + 30.5) - 89,900 \times 34.7 - 43,500 \times 22.4 - 4,080 \times 15 = +623,000$

moment due to vertical component



moment at A $\frac{1}{2}(131,000 \times 5.8) = 380,000$
" B $\frac{1}{2}(131,000 \times 7.4 - 124,850 \times 1.6) = 385,000$
" C $\frac{1}{2}(131,000 \times 22.4 - 124,850 \times 16.6 - 6,150 \times 1.5) = 341,000$

鉄ノ弾性率ヘ chord ノ max stress ノ起キタ時ニ対シ Diagonal ノ max stress ノ起キタ時ヨリ大キク

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JIUN MASUDA
CONSULTING ENGINEER
JIJI BLDG, TOKYO

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Pin at Lo

shearing stress direction : Lo-L1 = 90,000Kgs

vertical DL 83,800
LL.(unif) 30,100
" (single) 2,280 } = 116,180

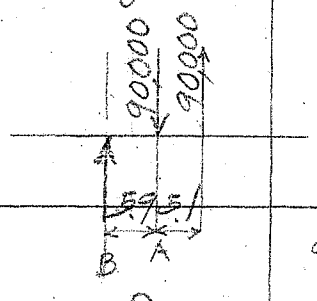
shearing stress of 17cm ϕ pin = $\frac{116,180}{\pi \times 8.5^2}$ = 512 Kgs/cm²

Bearing stress direction Lo-L1 = $\frac{90,000}{17 \times 4}$ = 1,320 "

vertical = $\frac{116,180}{17 \times 5.4}$ = 1,270 "

Bending stress

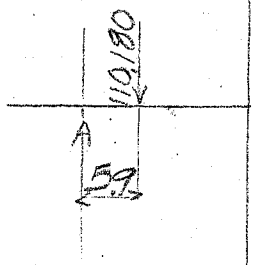
due to horizontal component



moment at A 90,000 × 5.1 = 459,000 Kg cm
" B 90,000 × (11.0 - 5.9) = 459,000 "

due to vertical component

moment at A 116,180 × 5.9 = 685,000 Kg cm



Load on shoe 2c 116,180 = 232360 Kgs
Dia of roller 15cm shearing strength 45 × 15 = 675 Kgs/cm²
Required length = $\frac{232360}{675}$ = 344cm
2 roller - 70cm length

approximate weight of truss

Lo-U1	327.0	×	.785	× 9.411	=	2,410
U1-U2	327.0	×	〃	× 6.261	=	1,608
U2-U3	383.94	×	〃	× 6.171	=	1,860
U3-U4	〃	×	〃	× 6.102	=	1,835
U4-U5	〃	×	〃	× 6.051	=	1,820
U5-U6	〃	×	〃	× 6.019	=	1,810
U6-U7	〃	×	〃	× 6.002	=	1,805
U1-L1	97.13	×	〃	× 7.250	=	.553
U2-L2	〃	×	〃	× 9.038	=	,688
U3-L3	〃	×	〃	× 10.480	=	,798
U4-L4	〃	×	〃	× 11.599	=	,883
U5-L5	〃	×	〃	× 12.375	=	,945
U6-L6	〃	×	〃	× 12.844	=	,980
U7-L7	4857	×	〃	× 13.000	=	,495
U1-L2	120.0	×	〃	× 9.411	=	,887
U2-M3	100.0	×	〃	× 7.511	=	,588
M3-L4	〃	×	〃	× 7.511	=	,588
U4-M5	〃	×	〃	× 8.341	=	,655
M5-L6	〃	×	〃	× 8.341	=	,655
U6-M7	〃	×	〃	× 8.789	=	,689
M7-L6	〃	×	〃	× 8.789	=	,689

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

L2 - M3	240.0 × .785 × 7.511 = 1415	
L4 - M3	" × " × 8.341 = 1570	
L0 - L2	160.0 × " × 12.00 = 1510	
L2 - L4	240.0 × " × 12.0.0 = 2260	
L4 - L7	320.0 × " × 18.00 = 4530	
	34,526 × 2 =	69,052
Details say 40%		27,608
		96,660 kgs

96660 ÷ 84.0 = 1150 kgs/m per truss

For 2 trusses 2 @ 96,660 = 193,320 kgs

193,320 ÷ 84.0 = 2,300 kgs/m of span

Structural steel in one span Span length = 84.0 meters

Stringers
Intermediate Floor Beam
End Floor Beam
upper laterals
Lower laterals
portal Bracings
Sway Bracings
Trusses
Shoes

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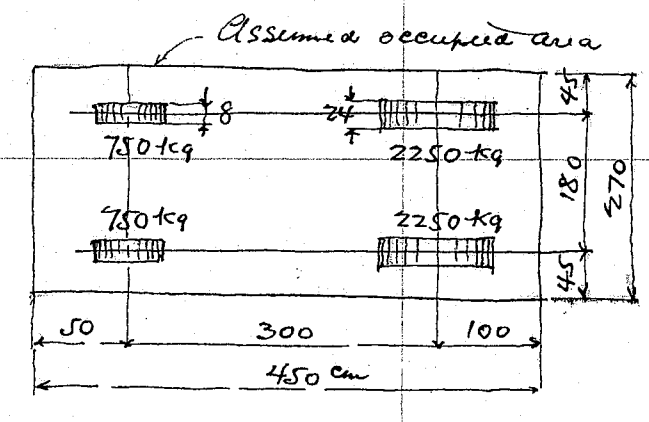
CALCULATIONS FOR

Assumed loadings

Uniform load on roadway $w = \frac{100.000}{170+l} = 500$ kg/m²

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

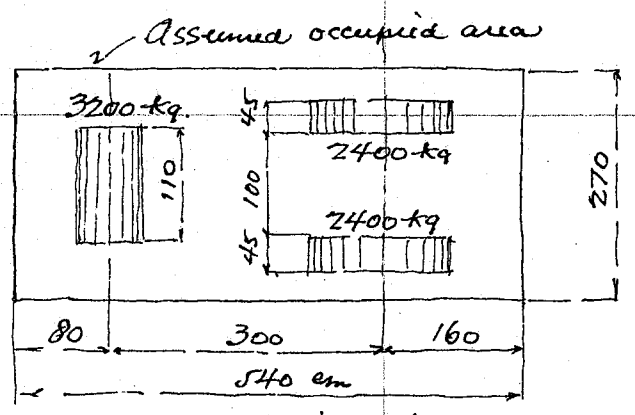
Ston motor truck loading.



2 rows of motor traffic on roadway with occupied swidth of 270 cm each Unoccupied space of front and rear and also sides of motor truck shall be filled with uniform load specified above.

One road roller on span

8 ton road roller



Impact for motor truck loading

$$\frac{20}{60+l}$$

where l = span length in meter (loaded length)
max impact 30%

No impact for roadroller and uniform live load.

Allowable Working strength

Structural Steel or Reinforcing Bars

Tension	1200 kg/cm²	net
Compression	1200 "	gross
Compression member	$1500\left(1-0.0055\frac{l}{r}\right) \leqq 1.000$ kg/cm²	

where l = length of member in centimeter
r = radius of gyration of section (least)

Bending stress	Tension	1200 kg/cm² net
	Comp	$1200\left(1-0.012\frac{l}{b}\right) \leqq 1.100$ kg/cm²

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

For Pin Fibre	1800 kg/cm²	net.
Shearing strength		
For plate	900 kg/cm²	
" pin	do	
" Reinforcing bars	do	
Machine driven shop rivet	850 kg/cm²	
" " field rivet	750 "	
Bearing Values		
For pin	1800 kg/cm²	
machine driven shop rivet	1700	
" " field rivet	1500	
Roller	45 d where d = diameter in centimeter	

Reinforced Concrete 1:2:4 mixture

Direct Compression	35 kg/cm²	Bearing	45 kg/cm²
Compressive fibre stress	54 "		
Combined stress	35 "		
Punching shear	9 "		
Shear plain concrete	4 "		
Bond stress plain bar	6 "	for deformed	9.0 kg/cm²

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CALCULATIONS FOR

slab 1

Design of floor slab. Span length 1.60 meters.

Dead Load

5c	Asphalt Block pavement	@ 21 kg	=	105
1.5c	mortar cushion	@ 22 kg.	=	33
15.0c	Concrete slab	@ 24 "	=	360
	mise concrete say			12
				510 kg. per sq meter

Dead Load moment = $\frac{1}{10} \times 510 \times 1.6^2$ = 131.0 kgm
Dead Load shear = $\frac{1}{2} \times 510 \times 1.6$ = 408.0 kg.

Live Load motor truck loading

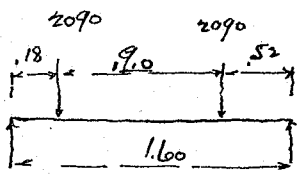
Rear wheel Concentration	2250	Front wheel Concentration	750
30% impact	675	30% impact	225
	2925 kg.		975 kg

Distribution of wheel Concentration on floor slab.
Thickness of pavement and mortar cushion 6.5 cm
Longitudinal distribution a Contact between wheel & pavement 20
distribution 2@ 6.5 13
a = 33
Transverse distribution b 24 + 13 = 37

Effective width $E = \frac{2}{3}l + a$ where l = span length in meter
$= \frac{2}{3} \cdot 1.60 + .33 = 1.40$ meter

Load per meter strip 2925 ÷ 1.40 = 2090 kg.
Moment per meter strip 1045 · $\frac{1.6}{2}$ = 835 kgm
For continuity of slab 0.8 · 835 = 668 kg meter

End shear



2090 · $\frac{.52}{1.60}$ = 680
2090 · $\frac{1.42}{1.60}$ = 1850
2530 kg.

Summary for moments and shears

	moment	shear
Dead Load	131.	408
Live Load	668	2530
	799 kgm	2938 kg.

Effective depth required for slabs.
f_s = 1200 kg/cm² f_c = 45 kg/cm²

$R = \frac{M}{bd^2}$ $d = \sqrt{\frac{M}{bR}}$

where R = 7.18

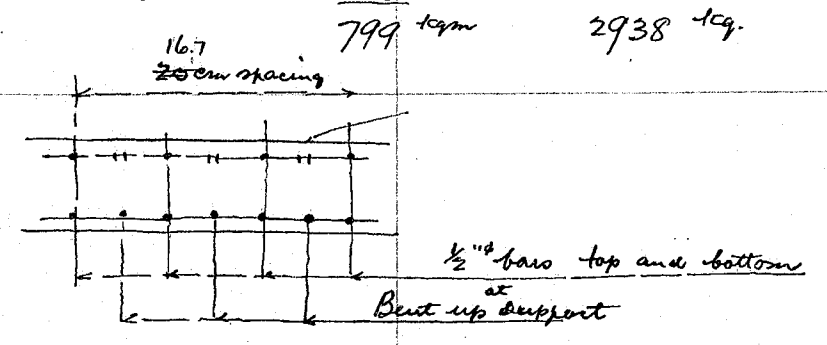
$d = \sqrt{\frac{799 \times 100}{100 \times 7.18}}$ = 10.56 cm

Use 15 cm floor slab as assumed
Effective depth say 12.5 cm

Steel area = $\frac{799 \times 100}{7/8 \times 12.5 \times 1200}$ = 5.32 cm² per lin. meter.

½"d bars $\frac{1.27 \times 100}{5.32}$ = 23.9 cm spacing

Use 16.7 20 cm spacing



perimeter of ½"d bars = 12.7 mm
circumference 4.0 cm
9 bars for 1 meter strip 36.0 cm

Unit Bond stress = $\frac{2938}{7/8 \times 12.5 \times 36.0}$ = 7.45 kg/cm²

Use deformed bars in slab on account of lack of bond stress.

増田淳氏関係資料
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CONSULTIEG ENGINEER
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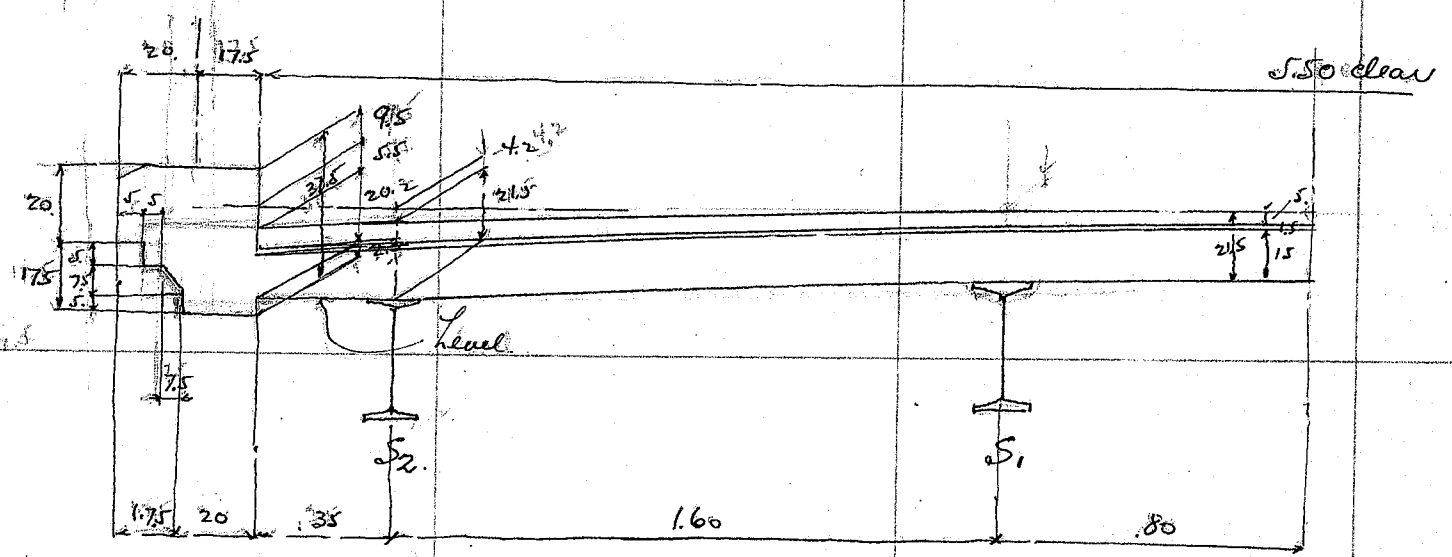
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CALCULATIONS FOR

Slab 2

Design of floor slab.
Overhanging slab beyond end stringer



Volume of Coping

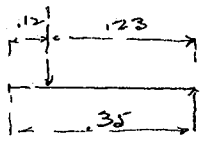
.375	. .20	=	.0750
.20	. 17.5	=	.0350
.5	. 12.5	=	62
7.5	. 7.5/2	=	28
			.1190 @ 2400 kg = 286 ~~262~~ kg per meter strip

Handrail assumed 100 kg per lin. meter. = 67# per lin ft.

Dead Load moment

Handrail	100	. 5.25	=	52.5
Coping	286	. 5.25	=	150.0
Slab and pavement 5.10-.35 = 1.78		. .175	=	31.2
	564 kg			233.7 kg meter.

Live Load. motor truck loading



Distribution of wheel Concentration assumed
2 @ .35 + .20 = .90 meter
Load per meter strip = 2925 ÷ .90 = 3260 kg.
Live Load Moment = 3260 . 0.23 = 750 kg meter

Summary for moments and shears

	moment	shear
Dead Load	233.7	564
Live Load	750.0	3260
	983.7 kgm	3824 kg

Depth of slab 12.5 cm in effect

Steel area req'd = $\frac{983.7 \times 100}{7/8 \times 12.5 \times 1200}$ = 7.50 cm

½"ϕ bars. $\frac{1.27 \times 100}{7.50}$ = 17.0 spacing.

use 1.67 cm spacing

4 @ 12 = 48.0 cm

Unit. shear = $\frac{3824}{7/8 \times 12.5 \times 100}$ = 3.5 kg/cm²

For Bond stress. use 12 - ½"ϕ bars

Unit Bond = $\frac{3824}{7/8 \times 12.5 \times 48}$ = 7.3 kg/cm².

Load on stringer S_1. 510 . 1.60 = 815.
stringer say 50
865 kg per lin meter

Load on stringer S_2 815 ÷ 2 = 407
Handrail, coping & Slab. 564
stringer say 50
1021 kg/m

For max load 233.7 ÷ 1.6 = 146
1021
1167 kg/m

増田淳氏関係資料
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CALCULATIONS FOR

3 stringers
3.75

Design of stringer S1 span length 3.75 meters.

Dead Load 865 kg/m.

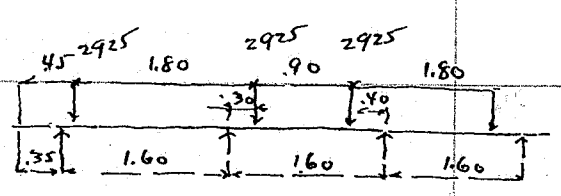
Dead Load moment = 1/8 · 865 · 3.75² = 1520.0 kgm

Dead Load shear = 1/2 · 865 · 3.75 = 1620. kg.

Live Load motor truck rear wheel concentration with impact = 2925

front wheel " " " = 975

max load on stringer.

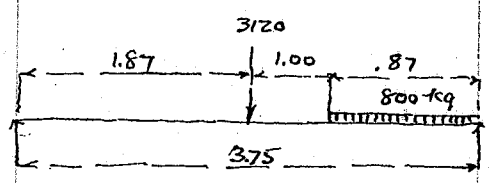


Load on stringer 2925 × 1.70/1.60 = 3120 kg.

Front wheel 3120 ÷ 3 = 1040 kg

Uniform Live load 500 × 1.60 = 800 kg per sq meter.

Live Load Moment



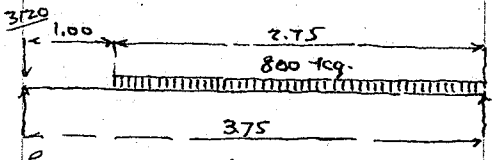
uniform load 800 × .87 = 695

Moment due to motor truck 1560 · 1.87 = 2920

Unif. load 695 · .435/3.75 · 1.87 = 151

3071 kgm

max End shear



uniform load 800 · 2.75 · 1.37/3.75 = 804 kg.

Due to motor concentration 3120

3924 "

Summary for moments and shears

	moment	shear
Dead Load	1520	1620
Live Load.	3071	3924
	4591 kgm	5544 kg

Section modulus req'd = 4591 × 100/1100. = 417.

Use 300 · 150 I @ 48.34 kg Sm = 633.2

Unit stress = 4591 × 100/633.2 = 726 kg/cm².

Design of stringer S2 Span length 3.75 meters.

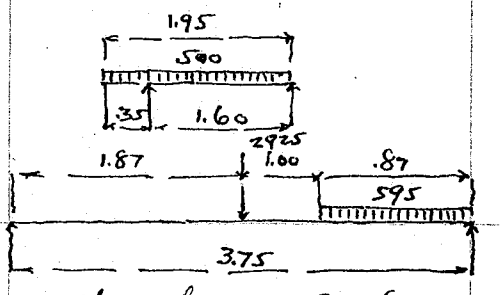
Dead Load max. 1167 kg per meter

Dead Load moment = 1/8 · 1167 · 3.75² = 2050 kgm

Dead Load shear = 1/2 · 1167 · 3.75 = ~~445~~ 2190 kg.

Live Load motor truck loading rear wheel Conc. with impact = 2925 kg.

front wheel " " " = 975 "



Uniform live load 500 · 1.95 × .975/1.60 = 595 kg per meter

Due to motor truck 1462 · 1.87 = 2740

Due to unf. 517 · .435/3.75 · 1.87 = 112

2852 kgm

uniform load 595 × .87 = 517 kg.

Max End shear Uniform load 517 · 2.75 · 1.37/3.75 = 520

motor truck loading = 2925

3445 kgm

Summary for moments and shears

	moment	shears
Dead Load	2050	2190
Live Load	2852	3445
	4902 kgm	5635 kg

Use 300 · 150 I @ 48.34 kg. Sm = 633.2

Unit stress = 4902 × 100/633.2 = 775.0 kg/cm².

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTIEG ENGINEER
SEIYU BLDG, TOKIO

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CALCULATIONS FOR



Design of stringer S1. Span length 4.0 metres

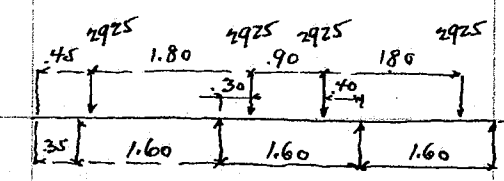
Dead load 865 kg per metre

Dead load moment = 1/8 · 865 · 4.0² = 1730 kgm

Dead load shear = 1/2 · 865 · 4.0 = 1730 kg.

Live load motor truck rear wheel concentration with impact = 2925

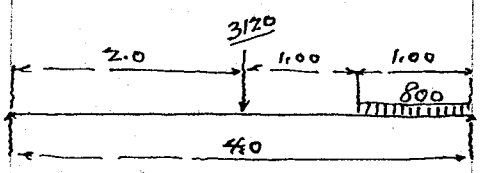
front " " " " " = 975



Load on stringer S1 2925 · 1.70/1.60 = 3120 kg.

Front wheel 3120 ÷ 3 = 1040 "

Uniform live load = 500 · 1.60 = 800 kg per lin. metre

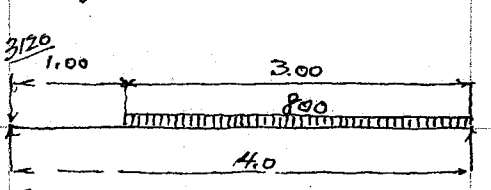


Moment due to motor truck = 1560 · 2.0 = 3120 kgm

Unif. load 800 · 0.5/4.0 · 2.0 = 200

3320

Unif. load 800 · 1.0 = 800



Max End shear

Unif. load 2400 · 1.5/4.0 = 900

motor truck loading 3120

4020 kg.

Summary for moments and shears

	moments	shears
Dead Load	1730	1730
Live Load	3320	4020
	5050 kgm	5750 kg.

Section modulus req'd 5050 · 100/11.00 = 459.

else 300 × 150 I @ 48.34 kg Sm = 633.2

Unit stress = 5050 · 100/633.2 = 797.0 kg/cm²

Design of stringer S2 Span length 4.0 metres

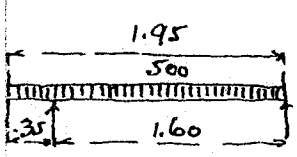
Dead Load max 1167 kg per metre

Dead Load moment = 1/8 · 1167 · 4.0² = 2334 kgm

Dead Load shear = 1/2 · 1167 · 4.0 = 2334 kg.

Live load motor truck loading rear wheel conc. with impact = 2925

front " " " " = 975



Uniform live load 500 · 1.95 · 975/1.60 = 595 kg per metre.

Manner of loading same as for S1

Uniform load 595 · 1.0 = 595 kg.

moment

motor truck loading 1462 · 2.0 = 2925

Unif. loading 595 · 50/4.0 · 2.0 = 149

3074 kgm

max End shear Unif. load 595 · 3.0 · 1.5/4.0 = 670

motor truck 2925

3595 kg.

Summary for moments and shears

	moments	shears
Dead Load	2334	2334
Live Load	3074	3595
	5408 kgm	5929 kg.

Use 300 · 150 I @ 48.34 kg Sm = 633.2

Unit stress = 5408 · 100/633.2 = 855.0 kg/cm²

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTIEG ENGINEER
SEIYU BLDG, TOKIO

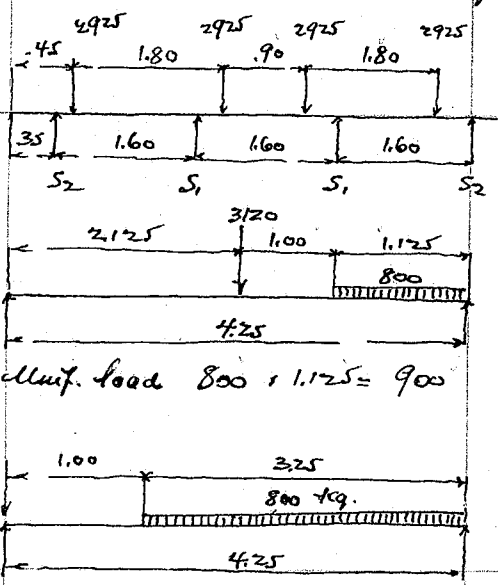
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CALCULATIONS FOR

Design of stringer S1 Span length 4.25 meters
Dead Load 865 kg per meter
Dead Load Moment = 1/8 · 865 · 4.25² = 1955 kgm
Dead Load shear = 1/2 · 865 · 4.25 = 1840 kg

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



Load on stringer S1 2925 × 1.70/1.60 = 3120 kg

Unif. load 800 × 1.125 = 900

Uniform load = 500 × 1.60 = 800 kg per lin. meter
Live Load moment
Due to motor truck = 1560 × 2.125 = 3315
Uniform load 900 × .562/4.25 × 2.125 = 253
3567 kgm

max End shear
Uniform load 800 · 3.25 · 1.62/4.25 = 992
motor truck loading 3120
4112 kg.

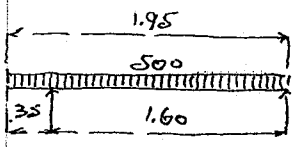
Summary for moments and shears

	moment	shear
Dead Load	1955	1840
Live Load	3567	4112
	5522 kgm	5952 kg.

Section modulus req'd = 5522 · 100/1100 = 502.
Use 300 · 150 I @ 48.34 kg Sm = 633.2
Unit stress = 5522 · 100/633.2 = 8720 kg/cm²

Design of Stringer S2 Span length 4.25 meters
Dead Load max 1167 kg per meter
Dead Load moment = 1/8 · 1167 · 4.25² = 2630 kgm
Dead Load shear = 1/2 · 1167 · 4.25 = 2480 kg.

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



Uniform live load 500 · 1.95 · 975/1.60 = 595 kg per meter.

Loading manner same as for S1

Moment Uniform load 595 · 1.125 = 670 kg
motor truck loading 1462 · 2.125 = 3110
Uniform load 670 · .562/4.25 · 2.125 = 188
3298 kgm

Max End Shear Uniform load 670 · 3.25 · 1.62/4.25 = 830
motor truck loading 2925
3755 kg

Summary for moments and shears

	moments	shears
Dead Load	2630	2480
Live Load	3298	3755
	5928 kgm	6235 kg

Use 300 · 150 I @ 48.34 kg Sm = 633.2
Unit stress = 5928 · 100/633.2 = 935.0 kg/cm²

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTIEG ENGINEER
SEIYU BLDG, TOKIO

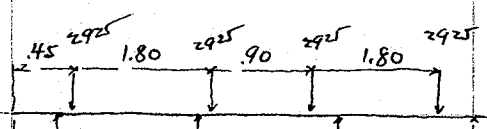
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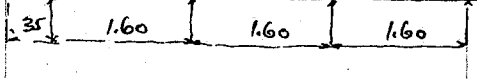
CALCULATIONS FOR

Design of stringer S1 span length 4.50 meters
Dead Load 865 kg per meter
Dead Load moment = 1/8 · 865 · 4.5² = 2190 kgm
Dead Load shear = 1/2 · 865 · 4.5 = 1950 kg

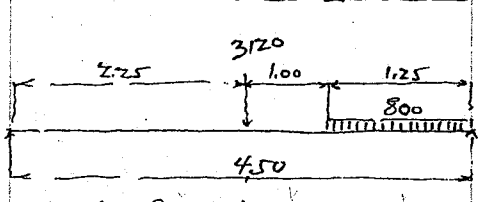
Live Load motor truck rear wheel Concentration with impact = 2925 kg
front " " " " = 975 "



Load on stringer S1 2925 · 1.70/1.60 = 3120 kg

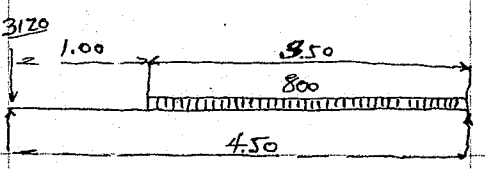


Uniform load 500 · 1.60 = 800 kg per live meter



Live Load Moment
Due to motor truck 1560 · 2.25 = 3510
Uniform load 1000 · .625/4.5 · 2.25 = 312
3822 kgm

Unif. 800 · 1.25 = 1000



Max End shear
Uniform load 800 · 3.50 · 1.75/4.50 = 1090
motor truck loading 3120
4210 kg.

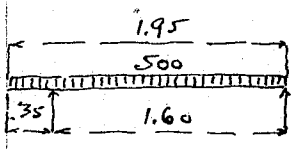
Summary for moments and shears

	moment	shear
Dead Load	2190	1950
Live Load	3822	4210
	6012 kgm	6160 kg

Section modulus req'd = 6012 × 100/1100 = 545.0
Use 300 · 150 I @ 48.34 kg Sm = 633.2
Unit stress = 6012 · 100/633.2 = 950.0 kg/cm²

Design of stringer S2 span length 4.50 meters
Dead Load max 1167 kg per meter
Dead Load moment = 1/8 · 1167 · 4.5² = 2960 kgm
Dead Load shear = 1/2 · 1167 · 4.5 = 2625 kg.

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



Uniform live load 500 · 1.95 · 975/1.60 = 595 kg per meter.

Loading manner same as for stringer S1

moment
Uniform load 595 · 1.25 = 744 kg.
motor truck loading 1462 · 2.25 = 3300
Unif. load 744 · .625/4.5 · 2.25 = 232
3532

Max End Shear
Uniform load 744 · 3.50 · 1.75/4.50 = 1012
motor truck loading 2925
3937

Summary for moments and shears

	moment	shear
Dead Load	2960	2625
Live Load	3532	3937
	6492 kgm	6562 kg.

Use 300 × 150 I @ 48.34 kg. Sm = 633.2
Unit stress = 6492 × 100/633.2 = 1028 kg/cm².

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTIEG ENGINEER
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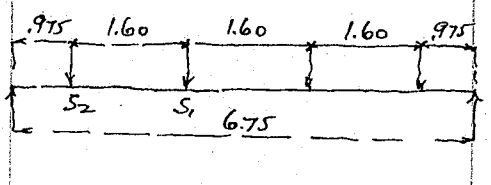
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CALCULATIONS FOR

3.75 21.3 1

Design of Intermediate Floor Beam. Span length 6.75 m spacing 3.75 meters.

Dead Load



D.L. Concentration

S_1	865 × 3.75	=	3240
S_2	1021 × 3.75	=	3830
			7070 kg.

Moment at S_1

7070 × 2.575	=	18200
3830 × 1.60	=	6120
		12080 kgm.

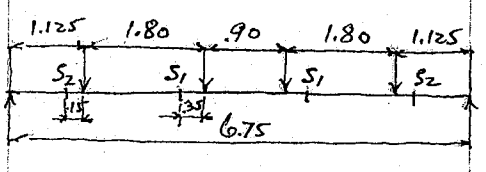
Dead Load beam assumed 200 kg per lin. meter $m = \frac{1}{8} \cdot 200 \cdot 6.75^2 =$ 1140

13220 "

Live Load End shear = 7070 + 675 = 7745 kg.

motor truck loading rear wheel concentration with impact = 2925 kg.

Front " " " " = 975 "



Rear wheel direct on floor beam

Front wheel of motor truck and uniform live load through stringers connected to floor beam.

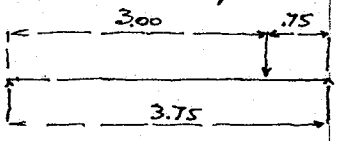
Reaction on stringer due to front wheel of motor truck

On S_1 $975 \times \frac{.15}{1.60} = 91$ On S_2 $975 - 91 = 884$.

975

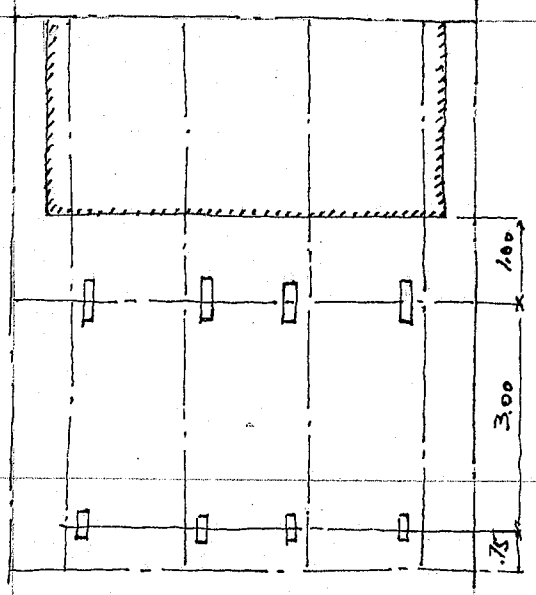
1066 kg.

Reaction on floor beam



S_1 $1066 \times \frac{.75}{3.75} =$ 213 kg

S_2 $884 \times \frac{.75}{3.75} =$ 177 kg.

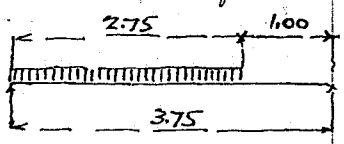


Uniform live load 500 kg/m²

On stringer S_1 500 × 1.60 = 800 kg per meter

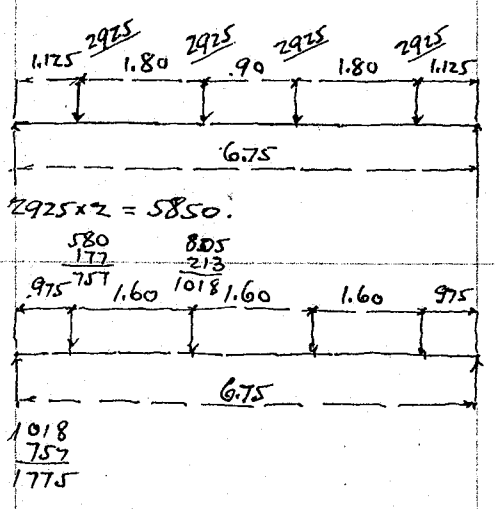
On " S_2 500 × 1.15 = 575 " " "

Reaction on floor beam



from S_1 $800 \times 2.75 \times \frac{1.375}{3.75} =$ 805

from S_2 $575 \times 2.75 \times \frac{1.375}{3.75} =$ 580



Moment due to motor truck rear wheel.

m	5850 × 2.925	=	17100
	2925 × 1.800	=	− 5260
			11840 kgm

Moment due to motor truck front wheel and uniform load.

m	1775 × 2.575	=	4570
	757 × 1.60	=	− 1210
			3360 kgm

Total moment 11840 + 3360 = 15200 kgm

Max End shear say motor truck rear wheel 5850

front wheel and unif. load 1775

7625 kg.

Summary for moments and shears

	Moment	Shear
Dead Load	13220	7745
Live Load	15200	7625
	28420 kgm	15370 kg

web assumed 67.0 × .8 = 53.6 ○cm 1/8 web = 6.70 ○cm

b to b of Ls 68.0 cm Effective depth = 63.6

flange stress = $\frac{2842000}{63.6}$ = 44700 kg.

flange section req'd = $\frac{44700}{1200}$ = 37.3

6.7

30.6 ○cm net

Try 2Ls 125 × 90 × 10 = 40.50 gross − 4.4 = 36.10 ○cm net.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTIEG ENGINEER
SEIYU BLDG, TOKIO

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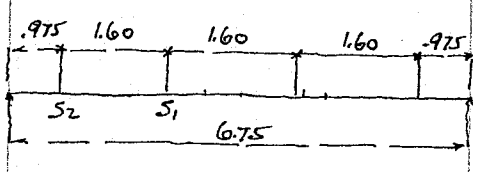
CALCULATIONS FOR

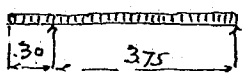
375 152

Design of End Floor Beam

Span length 6.75 m Projection .30

Dead Load





Dead Load concentration

S1	865	× 4.05 ×	2.025 / 3.75	=	1885
S2	1021	× 4.05 ×	2.025 / 3.75	=	2220
					4105

Moment at S1

4105	×	2.575	=	10600
2220	×	1.60	=	-3550
				7050

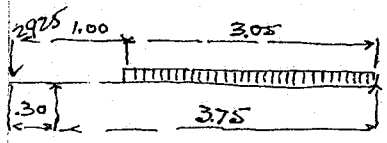
Dead Load beam assumed 200 kg/m $m = \frac{1}{8} \times 200 \times 6.75^2 =$ 1140

8190 kgm

End shear 4105 + 675 = 4780 kg.

Live Load motor truck loading rear wheel concentration with impact = 2925 kg
motor truck rear wheel direct on floor beam, uniform live load through stringer to floor beam assumed.

motor truck loading $2925 \times \frac{4.05}{3.75} = 3160$ on floor beam



Due to uniform live load.

S1	800 × 3.05 ×	1.525 / 3.75	=	992	kg
S2	575 × 3.05 ×	1.525 / 3.75	=	713	
				1705	

Moment due to motor truck

3160 × 2	×	2.925	=	18500
3160	×	1.800	=	5690
				12810

Moment due to unif. load.

1705	×	2.575	=	4390
713	×	1.60	=	1140
				3250

Total moment 16060 kgm

Max End Shear motor truck 6320
unif. load. 1705
8025 kg.

Summary for moments and shears

	moment	shear
Dead Load	8190	4780
Live Load	16060	8025
	24250 kgm	12805 kg.

web assumed 67.0 × .8 = 53.6 ⅛ web = 6.700 cm
Back to b of Ls 68.0 cm Effective depth = 63.6
flange stress = $\frac{24250 \times 100}{63.6}$ = 38100 kg
flange section req'd = $\frac{38100}{1200}$ = 31.80
6.70
25.100 cm net

Try 2Ls 125 × 90 × 10 = 405 gross - 4.4 = 36.100 cm net

Approximate weight of floor beam

flange	4Ls	125 × 90 × 10	@	160.9 kg	× 6.43	=	414.0
web.	1Pl	67.0 × .8	@	42.1	× 6.43	=	271.0
End conn	4Ls	125 × 90 × 10	@	16.09	× .68	=	43.7
fills	4Pls.	125 × 10	@	9.81	× .150	=	19.6
Int. stiffs	6Ls	75 × 75 × 9	@	9.96	× .68	=	40.7
Stringer conn	8Ls	125 × 90 × 10	@	16.09	× .68	=	87.3
fills	8Pls.	90 × 10	@	7.07	× .129	=	16.4
"	8Pls.	190 × 10	@	14.9	× .121	=	25.0
Stringer conn	8Ls	125 × 90 × 10	@	16.09	× .127	=	34.8
							952.5
Rivet heads say 3½ %							33.3
							985.8

call this 990 kg.

990 ÷ 6.43 = 154 kg per lin meter of girder
or 990 ÷ 3.75 = 264 kg per lin meter of truss span

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTIEG ENGINEER
SEIYU BLDG, TOKIO

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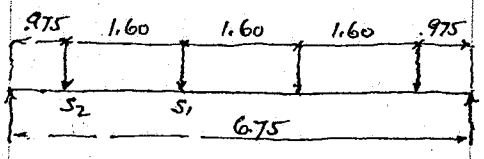
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CALCULATIONS FOR

40. FB 1.

Design of Intermediate Floor Beam span length 6.75 m spacing 4.0 m

Dead Load



Dead Load concentration

S_1	865	· 4.0	=	3460
S_2	1021	· 4.0	=	4090
				7550

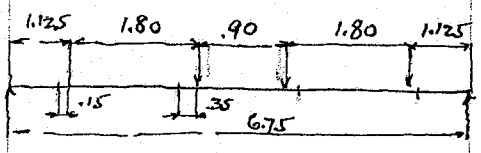
Moment at S_1

7550	· 2.575	=	19450
4090	· 1.60	=	- 6550
			0 12900
			1140
			14040 kgm

Dead Load beam assumed 200 kg per lin. meter $m = \frac{1}{8} \cdot 200 \cdot 6.75^2 =$

End shear 7550 + 675 = 8225 kg.

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



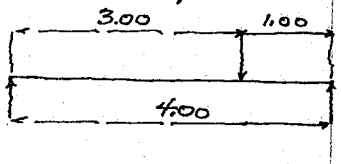
Rear wheel direct on floor beam ; front wheel of motor truck and uniform live load through stringers.

Reaction on stringer due to front wheel of motor truck

On S_1 $975 \cdot \frac{.15}{1.60} = 91$ On S_2 $975 - 91 = 884$

975
1066

Reaction on floor beam



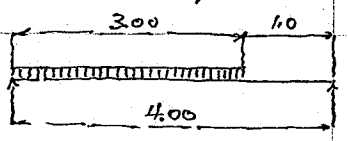
S_1 $1066 \cdot \frac{1}{4}$ = 266 kg.

S_2 $884 \cdot \frac{1}{4}$ = 221 "

Uniform Live Load

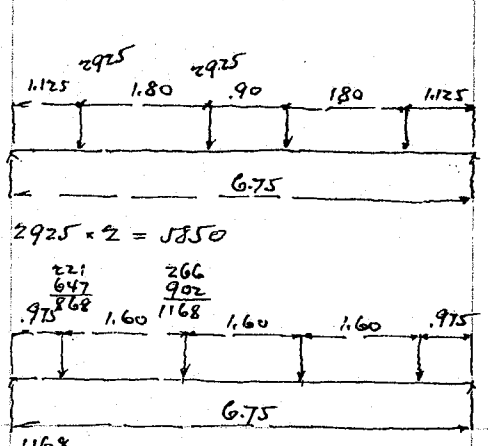
S_1	500 · 1.60	=	800 kg per lin. metre
S_2	500 · 1.15	=	575 " " " "

Reaction on floor beam



S_1 $800 \cdot 3.00 \cdot \frac{1.5}{4.0}$ = 902 kg.

S_2 $575 \cdot 3.00 \cdot \frac{1.5}{4.0}$ = 647 "



Moment due to motor truck rear wheel.

Moment	5850 ·	2.925	= 17100
	2925 ·	1.800	= - 5260
			11840 kgm

Moment due to motor truck front wheel and uniform L Load.

Moment	2036 ·	2.575	= 5250
	868 ·	1.600	= 1390
			3860 kgm

Total moment 11840 + 3860 = 15700 kgm

Max End shear say motor truck rear wheel 5850
front wheel and unif. load 2036
7886 kg

Summary for moments and shears

	moment	shear
Dead Load	14040	8225
Live Load	15700	7886
	29740 kgm	16111 kg

web assumed 67.0 · .8 = 53.60 cm 1/8 web = 6.700 cm
b to b of 1s 68.0 cm Effective depth 63.6 cm

Flange stress = $\frac{29740 \times 100}{63.6}$ = 46700 kg.

Flange section req'd = $\frac{46700}{1200}$ = 38.9
6.70
37.200 cm net

Try 2ls 125 × 90 · 10 = 40.5 gross - 4.4 = 36.10 cm net

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

JIUN MASUDA
CONSULTIEG ENGINEER
SEIYU BLDG, TOKIO

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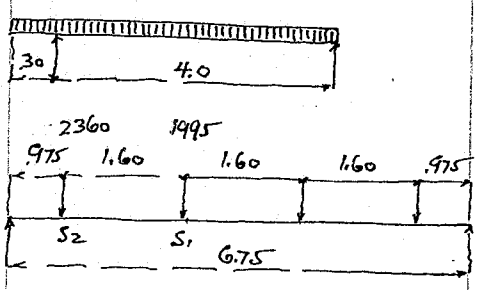
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CALCULATIONS FOR

4.0 FB2

Design of End floor beam span length 6.75 m Projection .30

Dead Load D1. Concentration



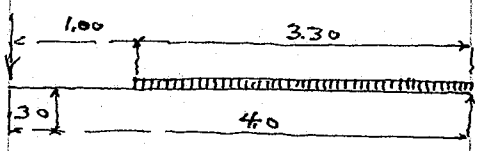
S_1	865 · 4.30 ·	$\frac{2.15}{4.0}$	=	1995
S_2	1021 · 4.30 ·	$\frac{2.15}{4.0}$	=	2360
				4355

Moment at S_1	4355 ·	2.575	=	11200
	2360 ·	1.600	=	3780
				7420
Dead Load Beam	1/8 · 200 ·	6.75^2	=	1140
				8560 · kgm

End shear 4355 + 675 = 5030 kg.

Live Load motor truck loading rear wheel concentration with impact = 2925 kg.

Motor truck rear wheel direct on floor beam; uniform live load through stringers to FB assumed



Motor truck loading $2925 \cdot \frac{4.30}{4.00}$ = 3150 · kg.

Uniform load

S_1	800 ·	3.30 ·	$\frac{1.65}{4.00}$	=	1090
S_2	575 ·	3.30 ·	$\frac{1.65}{4.00}$	=	783
					1873 · kg.

Moment due to motor truck	3150 · 2 ·	2.925	=	18400
	3150 ·	1.800	=	5670
				12730 kgm
Moment due to unif. load	1873 ·	2.575	=	4820
	783 ·	1.600	=	1250
				3570
				16300 kgm

Max End Shear motor truck loading 6300

uniform load. 1873

8173 kg.

Summary for moments and shears Web assumed 67.0 · .8 = 53.6 1/8 web = 6.700 cm

b to b of LS 68.0 cm Effective depth = 63.6

	Moment	Shear
Dead Load	8560	5030
Live Load	16300	8173
	24860 kgm	13203 kg.

flange stress = $\frac{24860 \times 100}{63.6}$ = 39100 kg

flange section = $\frac{39100}{1200}$ = 32.60

670

25.900 cm net

Try 2LS 125 · 90 · 10 = 40.5 gross − 4.4 = 36.100 cm net

Approximate weight of Floor Beam

flange	4LS	125 · 90 · 10	@	16.09 kg	·	6.43	= 414.0
web	1Pl.	67 · .8	@	42.1	·	6.43	= 271.0
End Conn	4LS	125 × 90 × 10	@	16.09	·	.68	= 43.7
fills	4Pls.	125 × 10	@	9.81	·	.150	= 19.6
Int stiffs	6LS	75 × 75 × 9	@	9.96	·	.68	= 40.7
stringer conn	8LS	125 × 90 × 10	@	16.09	·	.68	= 87.3
fill	8Pls.	90 · 10	@	7.07	·	.29	= 16.4
"	8Pls.	190 · 10	@	14.9	·	.21	= 25.0
stringer conn	8LS	125 · 90 · 10	@	16.09	·	.27	= 34.8
							952.5
Rivet Heads 3½%							33.3
							985.8 call this 990 kg.

990 ÷ 6.43 = 154 kg. per meter of girder

or 990 ÷ $\frac{3.75}{4.00}$ = ~~264~~ 248 kg per meter of truss span

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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