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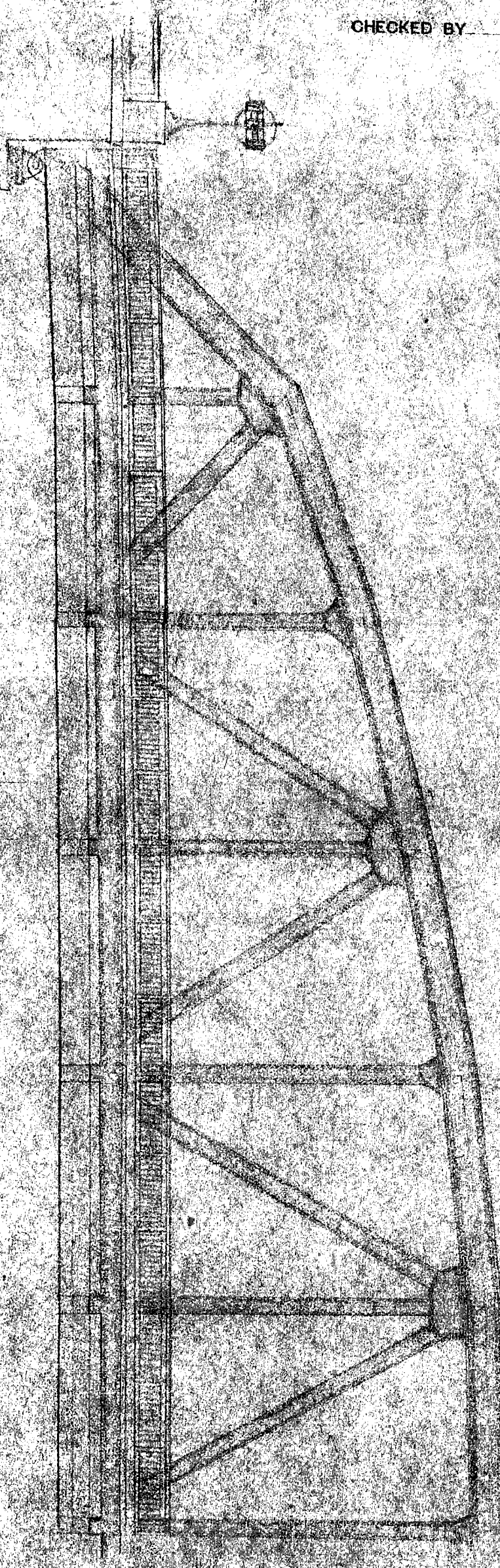
CALCULATIONS FOR



27.

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 43-B Bridge for city of Tokyo

☐ Loading (First class loadings)

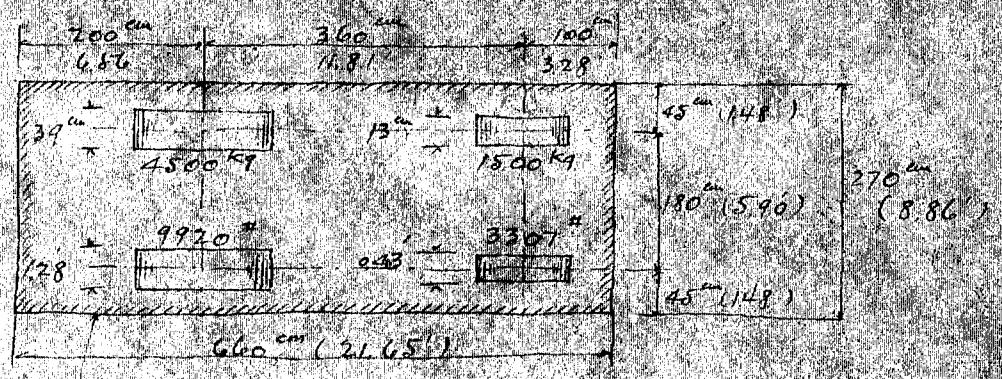
uniform load

for carriage way $w = \frac{120,000}{170+L} = 600$ kg/m² or 125 #/ft²

for footway $w = \frac{100,000}{170+L} = 500$ kg/m² or 100 #/ft²

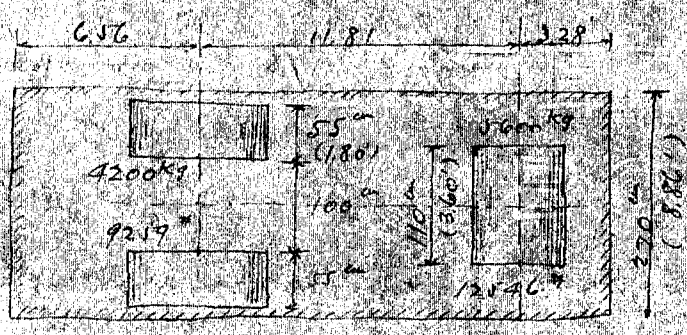
where L = span length in meters

motor car loadings (12.0 tons)



Occupied area

Road roller



Impact.

$I = \frac{20}{60+L}$ where L = loaded length in meters

max impact limited to 30%.

No impact for uniform load and road roller concentrations

☐ Weight of materials.

	lbs/cu ft
creosoted wooden block pavement	60
mortar	110
plain concrete	144
Reinforced concrete	150
structural steel	490
cast steel	491
cast iron	450
wrought iron	487
masonry granite	160
earth	100
sand and gravel	110

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CALCULATIONS FOR 42-B Bridge for city of Tokyo

☐ Allowable working Strength of materials

Structural Steel or reinforced bars

Tension	1200 kg/cm²	or	17,000 #/□"
extreme fiber stress (net)	1200 "	or	17,000 "
Shearing (gross section)	900 kg/cm²	or	12,800 "

Compression members $1500\left(1-0.0053\frac{\ell}{r}\right) \geqq 1{,}000$ kg/cm²

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

Equivalent formula for inch lbs unit.

$$21300\left(1-0.0053\frac{\ell}{r}\right) \geqq 14{,}000 \text{ \#/□"}$$

Compression flange of plate girder

$$1200\left(1-0.012\frac{\ell}{b}\right) \geqq 1{,}100 \text{ kg/cm}^2$$

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

Equivalent formula for inch-lbs unit

$$17{,}000\left(1-0.017\frac{\ell}{b}\right) \geqq 15{,}400 \text{ \#/□"}$$

Shearing on shop driven rivets (machine rivets)		17,000	#/□"
Shearing on field driven rivets and turned bolts (machine)		10,000	#/□"
Extreme fiber of pin	18,000 kg/cm²	or 25,600	#/□"
Bearing on shop rivets		24,000	"
Bearing on field rivets & turned bolts		20,000	"
Expansion roller	450d kg/cm	where d = diameter of roller in cm.	
or inch lbs unit	640d lbs/in	where d = " " " in inches	
Bearing on masonry 1:2:4 concrete		45 kg/cm² or 640	#/□"

l/r for compression member not over 120 for truss members 140 for wind bracings

l/r for riveted tension members limitted to 200

Strength of concrete 1:2:4 mixture

Bended compression for column	35 kg/cm²	or 500 lbs/□"
Compression fiber stress	45	640
Combined stress for column	35	500
Combined stress for arch	45	640
Bending shear	9	128
Shear plain concrete	4	58
Bond stress for plain bar	6	85
Bond stress for deformed bar	9	130
Shear for reinforced concrete with web reinforcement		128

l/r for reinforced concrete compression members limitted to 80

Considering wind, traction, and temperature stress in addition to dead, live and impact stresses, the allowable working stresses shall be increased 25% and proportion the section of members.

In case of figuring earthquake effect the working stress shall be increased 80%.

☐ Miscellaneous data

Ratio between modulii of elasticities of steel and concrete $n = 15$

Expansion coefficient of concrete 0.000 0011 per 1° c

" " steel 0.000 0012 "

Temperature change for concrete structure ± 15° c

" " steel structure ± 30° c

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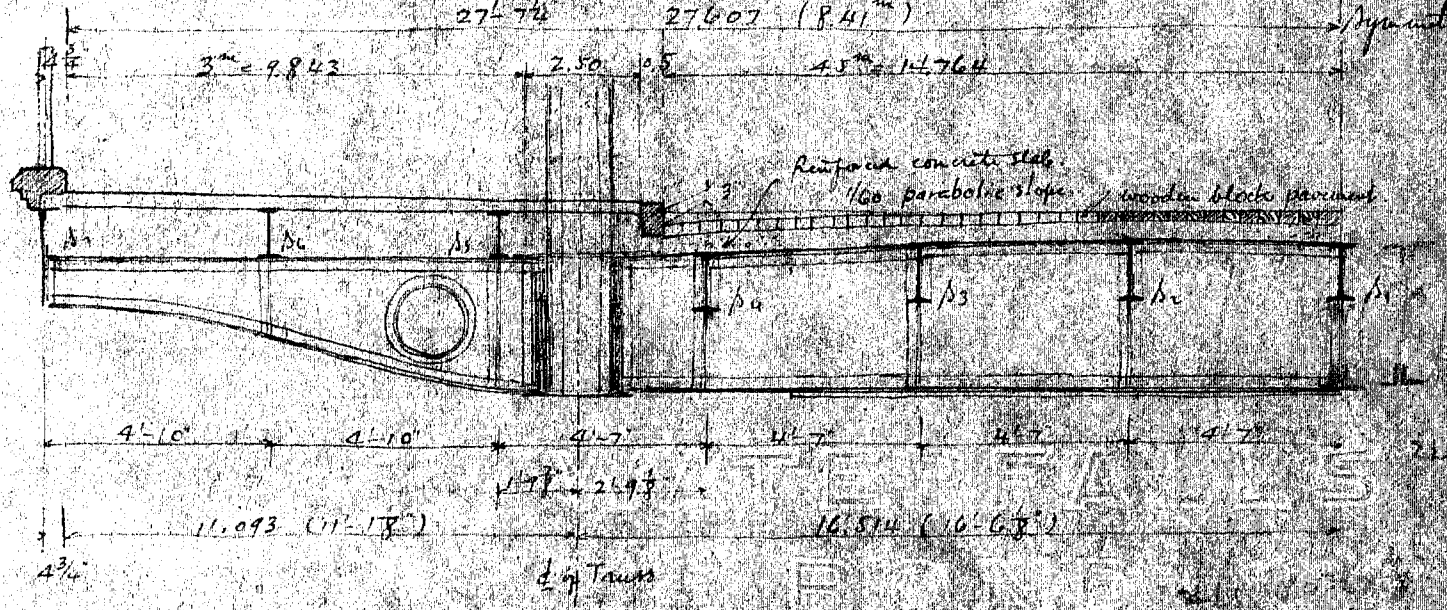
CALCULATIONS FOR 42-B Bridge for city of Tokyo

Standard temperature assumed 20° C
Accelleration of earth quake 3300 mm/sec²

☐ Assumed cross section of bridge

Road way 9 meters = 29.528' Clear
Side walks 3 meters = 9.843' Clear
Span length 12 @ 15' 3½" = 183' 6"



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CALCULATIONS FOR 42-B Bridge for city of Tokyo

☐ Design of floor slab. under carriage way. Span length 14'7" = 4.58m

Dead load 3" wooden block pavement 15 *
1" mortar cushion $\frac{40}{12}$ say 10
floor slab assumed 75
½" allowance 6
106 */ft

Dead load moment = $\frac{1}{10} \times 106 \times 4.58^2 =$ 223 ft*
" " shear = $\frac{1}{2} \times 106 \times 4.58 =$ 243 #

Live load.

Motor truck loading. wheel conc 9920 #
30% impact 2980
12,900 #

Front wheel carry 12,900 ÷ 3 = 4300 #

Live load distribution

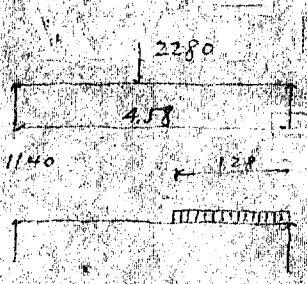
Transverse thickness 64" .66
20" .66
Longitudinal $a = 1.32$

Transverse distribution $b = 1.28 + 0.66 = 1.94$

Effective width = ⅔ (4.58 + 1.94) + 1.32 = 5.67

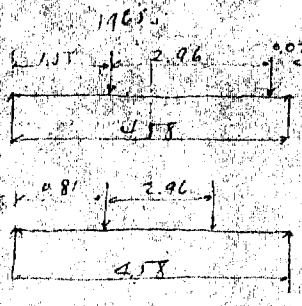
Load per ft 12900 ÷ 5.67 = 2280 #

Live load moment

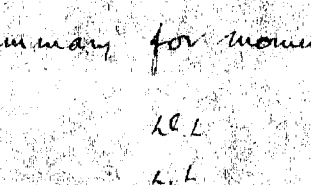


M = 1140 × 2.29 = 2610 ft#
for continuity of slab M = 2610 × 0.8 = 2090 ft#
End shear = $2280 \cdot \frac{394}{458} =$ 1960 #

For double wheel $b = 0.9^m + t + 2d = 2.95 + 1.28 + 0.66 = 4.89$
Effective width = ⅔ (4.58 + 4.89) + 1.32 = 7.63 > 2m = 6.56'
load per ft strip = 12900 ÷ 6.56 = 1965



Live load moment = $1965 \times \frac{303}{2 \cdot 4.58} \times 1.35 =$ 1007 ft#



Live load moment = 1965 × 0.81 = 1592 ft#
for continuity 1592 × 0.8 = 1275 ft# say

Summary for moment and shear.

	moment	shear
D.L	223	243
L.L	2090	1960
	2313 ft#	2203 #

Effective depth of slab for steel 17,000 #/□" concrete 640 #/□" mix 1:3:6

$d = \sqrt{\frac{M}{bk}} = \sqrt{\frac{2313}{102}} =$ 4.75"

make slab 6" thick with 1¼" insulation.

steel area req'd = $\frac{2313 \times 12}{7/8 \times 4.75 \times 17,000} =$ 0.393 □"

use ½" ϕ bar 6" spacing its area = 0.39 □"
perimeter 2@ 1.57 = 3.14"

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CALCULATIONS FOR 42-B Bridge for city of Tokyo

Bond stress = $\frac{2203}{7/8 \times 475 \times 3.14} = 169$ #/p" < 130 #

use ½" ϕ 4" spacing

1ft area = 3 e 0.1963 = 0.5889 □"

perimeter 3@1.57 = 4.71"

Bond stress = $\frac{2203}{7/8 \times 475 \times 4.71} = 113$ #/p" < 130 #

deformed bar used

1.0' strip

4½" 6"

1¾"

½" ϕ deformed bar

☐ Design of floor slab under footway span length = 4'10" = 4.83'

Dead load

1" mortar finished 10 #

4" slab $\frac{50}{60}$ #

60 #/sq. ft run

Dead load moment $\frac{1}{8} \times 60 \times 4.83^2 = 175$ '#

" " shear $\frac{1}{2} \times 60 \times 4.83 = 145$ #

Live load. uniform load intensity = 100 #/sq'

Live load moment $\frac{1}{8} \times 100 \times 4.83^2 = 292$ '#

Live load shear $\frac{1}{2} \times 100 \times 4.83 = 242$ #

Summary for moment and shear

	moment	shear
Dead load	175	145
Live load	292	242
	467 '#	387 #

Section of slab.

effective depth of slab $d = \sqrt{\frac{467}{102}} = 2.2$" make 4" thick with 1" insulation

steel area req'd = $\frac{467 \times 12}{7/8 \times 3 \times 17,000} = 0.126$ □"

1.0' strip

3" 4"

½" ϕ deformed bar

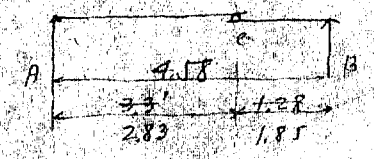
use ½" ϕ 1.0' spacing 1ft area = 0.196 □"

1ft perimeter = 1.57'

Bond stress = $\frac{387}{0.875 \times 3 \times 1.57} = 94$ # deformed bar used.

use ¾ ϕ 6¼ p.f.

☐ Design of floor slab span length 4'7" = 4.58'



Dead load 60 #/□'

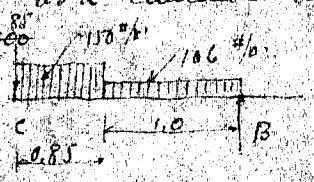
Live load 100

Reaction on C due to D.L = $60 \times \frac{2.83}{2} \times \frac{3.3}{2} = 100$ # 85 □

" " " " L.L = ~~$100 \times \frac{2.83}{2} = 165$ #~~ ~~100~~

$100 \cdot \frac{3.58^2}{2 \times 2.83} = 225$ # req'

BC as a cantilever arm

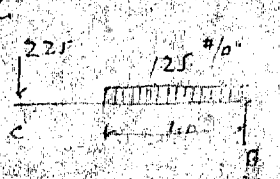


D.L = 106 #/□'

D.L moment = $106 \times \frac{1.0}{2} + 130 \times 0.85 \times 1.43 + 85 \times 1.85 =$

$= 53 + 182 + 157 = 392$ '#

L.L



L.L moment = $225 \times 1.85 + 125 \times \frac{1}{2} = 417 + 63 = 480$ '#

Total mt = 392 + 480 = 872 '# < 2313 '#

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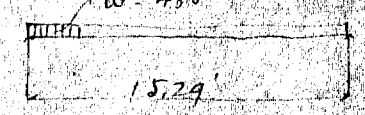
CALCULATIONS FOR

☐ Design of Stringer under ℄ carriage way. Span length 15'3½" = 15.29

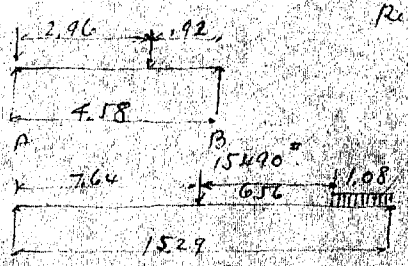
Dead load 106 ℔ × 4.58 = 485 ℔



wt of stringer assumed 50 ℔
total load = 485 + 50 = 535 ℔

Dead load moment = $\frac{1}{8} \times 535 \times 15.29^2 = 15{,}700$ ℔'
Dead load shear = $\frac{1}{2} \times 535 \times 15.29 = 4100$ ℔

Live Load.

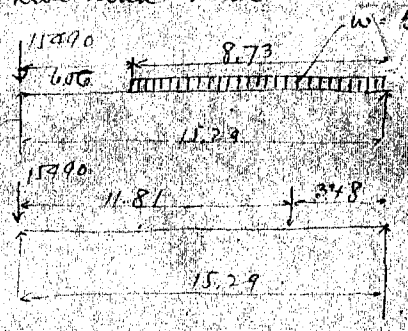


Reaction on A
$12{,}900 \times \frac{9.2}{4.58} = 2590$
12,900
15,490 ℔

125 × 4.58

moment → 7.745 × 7.64 = 59200 ℔'
125 × 4.58 = 573
moment → $\frac{573 \times 1.08^2}{2 \times 15.29} \times 7.64 = 167$
6087
59400 say

Live load shear



$\frac{8.73^2 \times 573}{2 \times 15.29} = 1430$
15,490
16,920 ℔

$15{,}490 \times \frac{1}{3} \times \frac{3.48}{15.29} = 1180$ ℔

Summary for moment and shear

	moment	shear
Dead load	15,700	4100
Live load	59,400	16,920
	75100 ℔	21,020 ℔

Section modulus req'd = $\frac{75100 \times 12}{15600} = 57.8$ in.³ use 14"×6" I @ 46.01 ℔ S.m = 62.946

☐ Design of stringer under sidewalk. Span length 15'3½" = 15.29' Spacing = 4'10" = 4.83'

Dead load slab 60 ℔/ft × 4.83 = 290 ℔ say
stringer 40
330 ℔

Dead load moment = $\frac{1}{8} \times 330 \times 15.29^2 = 9650$ ℔'
Dead load shear = $\frac{1}{2} \times 330 \times 15.29 = 2530$ ℔
Live Load 100 × 4.83 = 483 ℔
Live Load moment = $\frac{1}{8} \times 483 \times 15.29^2 = 14{,}130$ ℔'
Live Load shear = $\frac{1}{2} \times 483 \times 15.29 = 3700$ ℔

Summary for moment and shear

	moment	shear
D.L	9650	2,530
L.L	14,130	3,700
	23,780 ℔'	6,230 ℔

Section modulus req'd = $\frac{23{,}780 \times 12}{15600} = 18.3$ in.³ use 12"×5" I @ 31.99 ℔ its S.m = 36.69 in.³

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CALCULATIONS FOR 42,13 Bridge for city of Tokyo

☐ Design of Fascia Stringer

Dead load per 60 #/□' × 4.83 ÷ 2 = 145 #
Handrail including ~~coping~~ coping 250
stringer its own wt 50
445 say 450 #

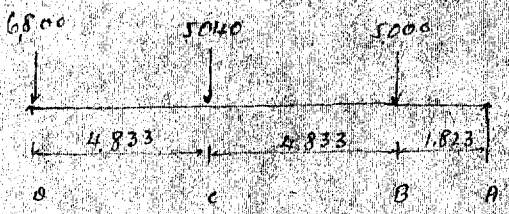
Dead load moment $\frac{1}{8} \cdot 450 \cdot 15.29^2$ = 13150 in#
Live load moment taken 7050
20200 in#

Section. req'd = $\frac{20200 \times 12}{19,000 \times 12}$ = 1.2 in²

3 L's 3×3×3/8 @ 7.2 = 21.6
1 Pl 15×5/16" 15.94
37.54
allow 12.46
50.00 #

☐ Design of Cantilever beam Span length 9.843+1.25+
2@2'-10", 1'-9 7/8" = 11'-6 5/8" take 11'-6"

Dead load Floor 60 × 1/12' × 4.83 = 290 # 290 × 15.29 = 4430 in#
60 × 4.58 ÷ 2 = 140 # 140 × 15.29 = 2140
Stringer 40 × 15.29 = 610 #

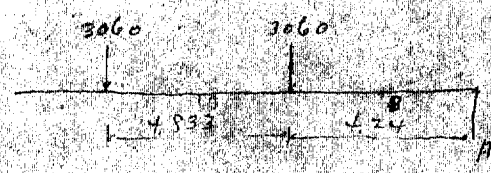


Stringer reaction 4430 × 610 = 5040 #
4430 × 1/2 + 2140 × 610 = 4961 say 5000 #
4430 × 1/2 = 2215
Handrail, coping 250 × 15.29 = 3825
stringer at fascia 50 × 15.29 = 768
6800 #

Dead load mt at A 5,000 × 1.823 = 9115
5040 × 6.656 = 33526
6,800 × 11.489 = 78,125
120,786 in#

at B 5040 × 4.833 = 24358
6800 × 9.666 = 65729
90087 in#

Moment due to water main and conduit



water main 200 × 15.29 = 3060 #
conduit 200 × 15.29 = 3060

moment at A 3060 × 4.240 = 12,974
3060 × 9.073 = 27,763
40,737 in#

moment at B 3060 × 2.417 = 7396
3060 × 7.25 = 22,191
29,587 in#

Shear due to dead load at A 6,800 + 5040 + 5,000 = 16,840
B 6,800 + 5040 = 11,840

due to water main conduit at A 3060 + 3060 = 6120
at B 6120

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CALCULATIONS FOR 42-B Bridge for City of Tokyo.

Live Load 100 #/p' × 15.29 = 1529

1529 · 4.83 = 7385

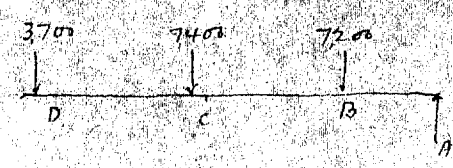
1529 · 4.58 × ½ = 3.500

Stringer reaction

" at B 7385 · ½ + 3500 = 7.200 #

at C 7385 = 7,400 #

at D = 3.700 "



moment at A 7200 × 1.823 = 13.126

7.400 × 6.656 = 49.254

3.700 × 11.489 = 42.510

104.890 1#

at B 7.400 × 4.833 = 35.774

3.700 × 9.666 = 35.764

71.538 1#

Shear due to live load

at A 7,200 + 7.400 + 3.700 = 18.300

B 7,400 + 3,700 = 11,100

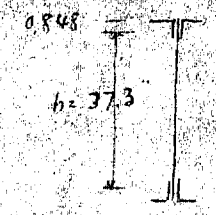
Summary for moment and shear at A

	moment	shear
dead load	120,786	16,840
	40,737	6,120
Live Load	104890	18300
	266413 1#	41.260 #

Section req'd

$\frac{266413}{17.000 \times 3.0} = 5.2^{\square"}$ net

$\frac{266413}{14.000 \times 3.0} = 6.34^{\square"}$ gross



2 L's 5 × 3½ × ⅞ — 2 @ 3.05 = 6.10 □"

1 web 39 × 7/16 12.19 □"

⅛ × 12.19 = 1.53

39 − 2 @ 0.848 = 37.304 I.R = $\frac{266.413 \times 12}{14.000 \times 37.3} = 6.1^{\square"}$

I.R = $\frac{266.413 \times 12}{17.000 \times 37.3} = 5.04^{\square"} \cdot 1.13 = 3.51^{\square"}$

Net flange area: 6.10 − 4 × ⅞ = 4.6 □"

for tension plate 12" × ½ = 6.00 □" net area = 5.00 □".

Approximate weight of side floor beam

4 L's 5 × 3½ × ⅜ @ 30.5 = 12.20 □"

1 web 32 × 7/16 10.00

22.20 × 3.4 = 75.5

30% details 22.7

98.2 call this 100 #

100 × 10.75 = 1075 # per piece.

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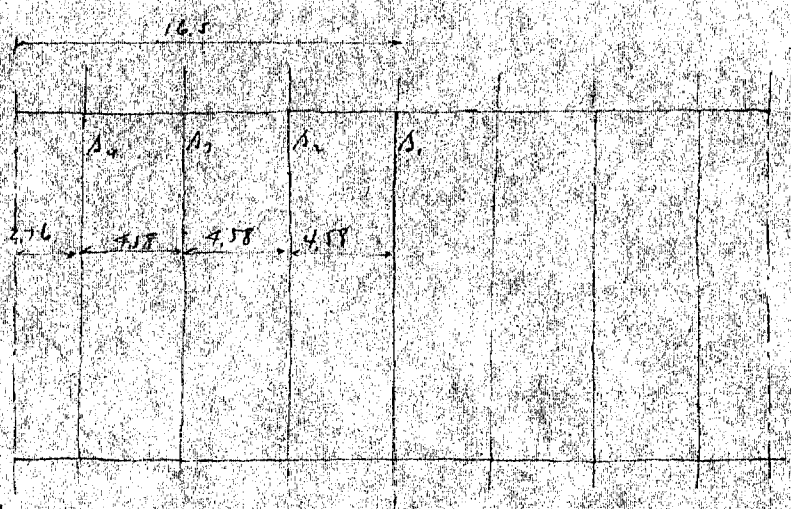
CALCULATIONS FOR 42-B Bridge for City of Tokyo.

☐ Design of main floor beam. Span length 2@ 16'6⅝" = 33' 0¼" take 33'0"



Dead load $106 \times 4.58 \times 15.29 = 7420$ # ← floor slab
$50 \times 15.29 = 765$ ← stringer
$8,185$ say $8,200$ # for S_2-S_3

$110 \times 15.29 = 1680$ — stringer
$9,100$ for S_1

$106 \times (2.29 + 1.0) \times 15.29 = 5350$ #
$150 \times 1.0 \times 15.29 = 2290$
$7,640$ # for S_4

Dead load moment:

Reaction $9,100 \times \frac{1}{2} + 2 \times 8,200 + 7,640 = 28,590$ #

moment at S_1

$28,590 \times 16.5 - (8,200 \times 4.58 + 8,200 \times 9.16 + 7,640 \times 13.74)$

$= 471,735 - (37,556 + 75,112 + 104,974) = 254,093$ # say 254,100 #

moment at S_2

$28,590 \times 10.92 - (8,200 \times 4.58 + 7,640 \times 8.16)$

$= 340,793 - (37,556 + 62,342) = 340,793 - 99,898 = 240,895$ say 240,900 #

moment at S_3

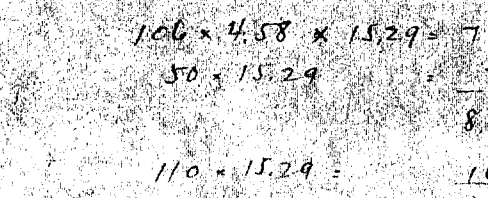
$28,590 \times 6.34 - 7,640 \times 4.58 = 181,261 - 34,991 = 146,270$ say 146,300 #

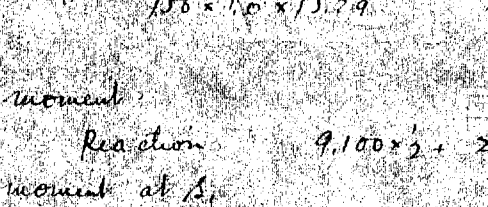
moment at S_4 $28,590 \times 2.76 = 78,910$ #

Negative moment = 120,800 #

Actual mt at S_1 = 254,100 − 120,800 = 133,300 #
S_2 = 240,900 − 120,800 = 120,100
S_3 = 212,300 − 120,800 = 91,500
S_3 = 146,300 − 120,800 = 25,500
174,860 54,100
S_4 = 78,900 − 120,800 = −41,900

Live load:





motor truck loading

$4300 \times 3.48 - 15.29 = 980$
12900
$13,880$ #

Uniform load

$125 \times 8.73 = 1090$ #
$1090 - 437 - 15.29 = 312$ #/lin ft.

$312 \times 4.58 = 1430$ # say for S_1-S_3

$312 \times \frac{1.01^2}{2.76 \times 2} + 715 = 577 + 715 = 1292$ say 1290 #
for S_4

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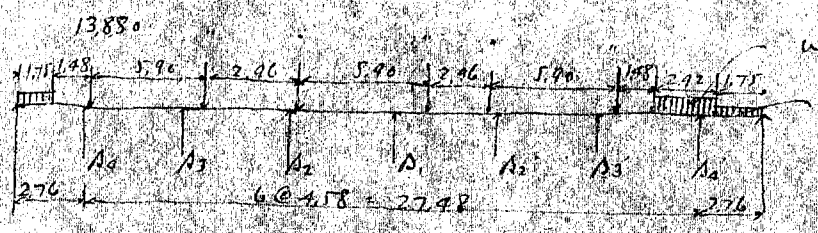
CALCULATIONS FOR 42 B. Bridge for city of Tokyo

Case 1.



$\omega = 125 \times \frac{21.85^2}{2 \times 15.29} = 1950$ #

$\omega' = 100 \times 15.29 = 1,530$ #

Reaction at A_1 $13,880 \left(\frac{3.09 + 0.17 + 0.13}{4.58} \right) = 13,880 \times \frac{3.39}{4.58} = 10,270$ #

at A_2' $13,880 \left(\frac{1.49 + 4.95}{4.58} \right) = 18,000$

at A_3' $13,880 \times \frac{3.4}{4.58} + 1950 \times \frac{1.91^2}{2 \times 4.58} = 10,300 + 780 = 11,070$

at A_4' $(1950 \times 1.91 - 780) + 1950 \times \frac{1.01 \times 2.25}{4.58 \times 2.76} + \frac{1.75^2 \times 1530}{2 \times 7.76} =$

$= 2,940 + 1,600 + 850 = 5,390$

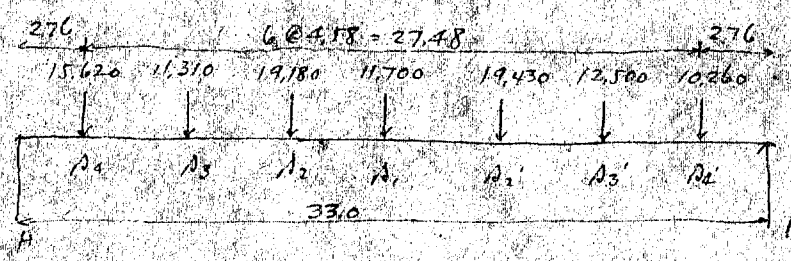
$13880 \cdot \frac{4.18}{4.58} = \frac{3,580}{8,970}$ #

at A_2 $13880 \times \frac{4.41 + 1.45}{4.58} = 17,750$

at A_3 $17,880 \times \frac{3.13 + 0.13}{4.58} = 9,880$

at A_4 $13,880 \times \frac{4.41}{4.58} + \frac{1.75^2 \times 1530}{2 \times 2.76} = 13,480 + 850 = 14,330$

Moment due to live load.



Reaction on A $\frac{11700}{2} + (10,260 \times 2.76 + 12,580 \times 7.34 + 19,430 \times 11.92 + 19,180 \times 21.08 + 11,310 \times 25.66 + 15,620 \times 30.24) \div 33 =$

$5,850 + (28,320 + 91,750 + 231,600 + 404,310 + 290,210 + 372,350) \div 33$

$= 5,850 + 1418,540 \div 33 = 5,850 + 42,986 = 48,836$ say 48,850 #

Reaction on B = 51,150 #

Moment at A_1 $48,850 \times 16.5 - (19,180 \times 4.58 + 11,310 \times 9.16 + 15,620 \times 13.74)$

$= 806,025 - (87,840 + 103,600 + 214,620) = 400,000$ 1#

at A_2 $48,850 \times 11.92 - (11,310 \times 4.58 + 15,620 \times 9.16)$

$= 582,290 - (51,800 + 143,080) = 387,410$ 1#

at A_3 $48,850 \times 7.34 - 15,620 \times 4.58 = 358,560 - 71,540 = 287,020$ 1#

at A_4 $= 48,850 \times 2.76 = 134,830$ 1#

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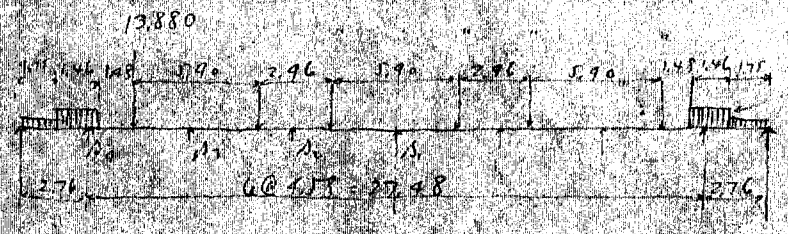
CALCULATIONS FOR 42. B. Bridge for city of Tokyo.

Case 2.



w = 2000 #/ft

Reaction at A_1 $13880 \times \frac{3.26}{4.58} = 9,870$ #

at A_2 ~~13,880 × 2 = 4,440~~ =

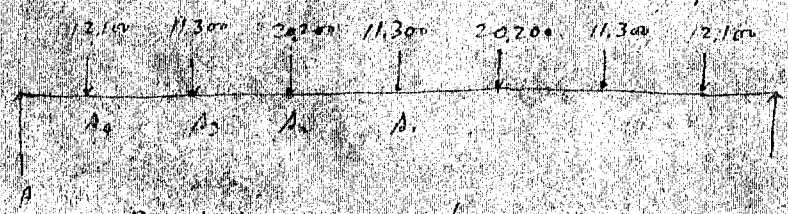
$13,880 \times \frac{2.95 + 3.25}{4.58} = 13,880 \times \frac{6.20}{4.58} = 18,780$ #

at A_3 $13,880 \times \frac{1.33 + 1.93}{4.58} = 13,880 \times \frac{3.26}{4.58} = 9,870$ #

at A_4 $13,880 \times \frac{2.65}{4.58} = 8,020$ #

$2,000 \times 2.76 \times \frac{1}{2} = 2760$

10,780



Reaction on A = $\frac{1}{2} \times 11,300 + 12,100 + 11,300 + 20,200 = 49,250$ #

Moment at A_1 $49,250 \times 1.65 - (20,200 \times 4.58 + 11,300 \times 9.16 + 12,100 \times 13.74)$

$= 812,625 - (92,516 + 103,508 + 166,254) = 450,347$ ft#

at A_2 $49,250 \times 11.92 - (11,300 \times 4.58 + 12,100 \times 9.16)$

$= 587,050 - (51,754 + 110,836) = 424,460$

at A_3 $49,250 \times 7.34 - 12,100 \times 4.58 = 361,495 - 55,418 = 306,077$ ft#

at A_4 $49,250 \times 2.76 = 135,930$ ft#

Summary for moment due to dead load and live load

	A_1	A_2	A_3	A_4
dead load	133,300	120,100	57,100	−41,400
live load	450,300	424,500	306,100	135,900
	583,600	544,600	360,200	94,000 ft#

Section req'd $\frac{583,600 \times 12}{17,000 \times 40.3} = 42 \times \frac{1}{16} \times \frac{1}{2} = 10.2 - 2.0 = 8.16$ in

2L 5 × 3½ × 7/8 @ 30" − 6.10 in − 1.5 = 4.6 in

1 cov pl 12 × 5/8 @ 6.00 − 1.0 = 5.0 in

12.60 in = 9.6 in

6.0

18.6

weight = $\frac{1870 \times 3.4 \times 1.3}{2} = 80$ #/ft say

mt = $\frac{1}{8} \times 80 \times 33^2 = 10,890$ ft#

weight = $24,200 \times 16.2 = 40.4 \times 3.4 \times 1.3 = 180$ #/in ft say

mt = $\frac{1}{8} \times 180 \times 33^2 = 24,700$ ft#

Total mt $583,600 + 24,700 = 608,100$

S.R. = $\frac{608,100 \times 12}{17.0 \times 41.2} - 2.0 = 10.4 - 2.0 = 8.4$ in

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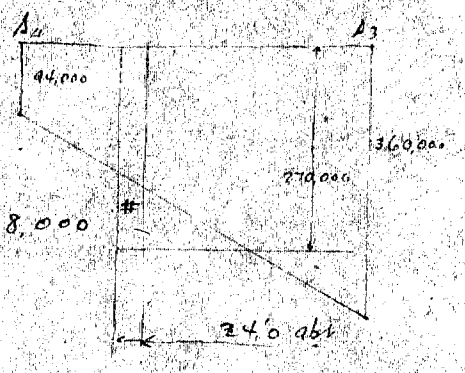
CALCULATIONS FOR 42.B Bridge for city of Tokyo.

Resisting mt of 4Ls 5×3½×⅜, 1 rivet 39"×15/16" 2920,000 #" = 243,000 1#

R.M = 270,000 1#

Length of cover plate 240 abt taken



End max shear due to dead load and live load

49,250 + 28,590 = 77,840 take 78,000

☐ Approximate weight of steel of floor systems.

stringer	6×12				
	3 @ 14"×6" I @ 46.0 × 15.3	=	72 @ 704	=	50,688 90 #
	2×12 @ 12"×5" I @ 32.0×15.3	=	48/24 @ 490	=	23,520
Bracing	2"@ 50 × 15.3	=	24 @ 765	=	18,360 including details
Strut (S,)	2 12 @ 110 × 15.3	=	12 @ 1680	=	20,160 "
					27,950
Floor Beam	Side floor beam 2×13	@	1075	=	25,800 "
	main floor beam 13	@	180 × 33.0	=	77,220 "
					217,900 #
			Rivet heads etc		2,100
					220,000 = 98.0 ton

46+19+6=

220,000 ÷ 1836 = 120.0 #/ sq ft of Bridge.

☐ Panel load

Dead load

floor slab	106 . 14.76 × 15.3	=	24,000
	60 × 12.54 × 15.3	=	11,800
coping, curbstone & Hand Rail	400 × 15.3	=	6,100
water main conduit	300 400 × 15.3	=	4,600 6,400
			48,000 #
			46,500
steel weight of flooring	600 × 15.3	=	9,200 #
truss weight assumed	1200 × 15.3	=	18,400 16,800
Laterce weight assumed	240 × 15.3	=	3,800 6
			31,200 29,600 # 29,600
			2

Total panel dead load = 48,000 + 31,200 = 79,200 # 77,700 #
28,100 74,600 take 78,000
29,600 76,100

Live load

$$\omega = \frac{120,000}{170+\ell} = \frac{120,000}{170+56} = 530 \text{ Kg/m}^2 \text{ or } 106^{\#}$$

$\ell = 183.5' = 56.0^{m}$

$$\omega' = \frac{100,000}{170+56} = 90.6^{\#}$$

Total panel live load = 109 × 14.76 + 91 × 9.84 = 1608.5 + 895.4 = 2,504.2 / sq ft = 2,500 # / sq ft

2,500 × 15.3 = 38,300 # take 38,000 #

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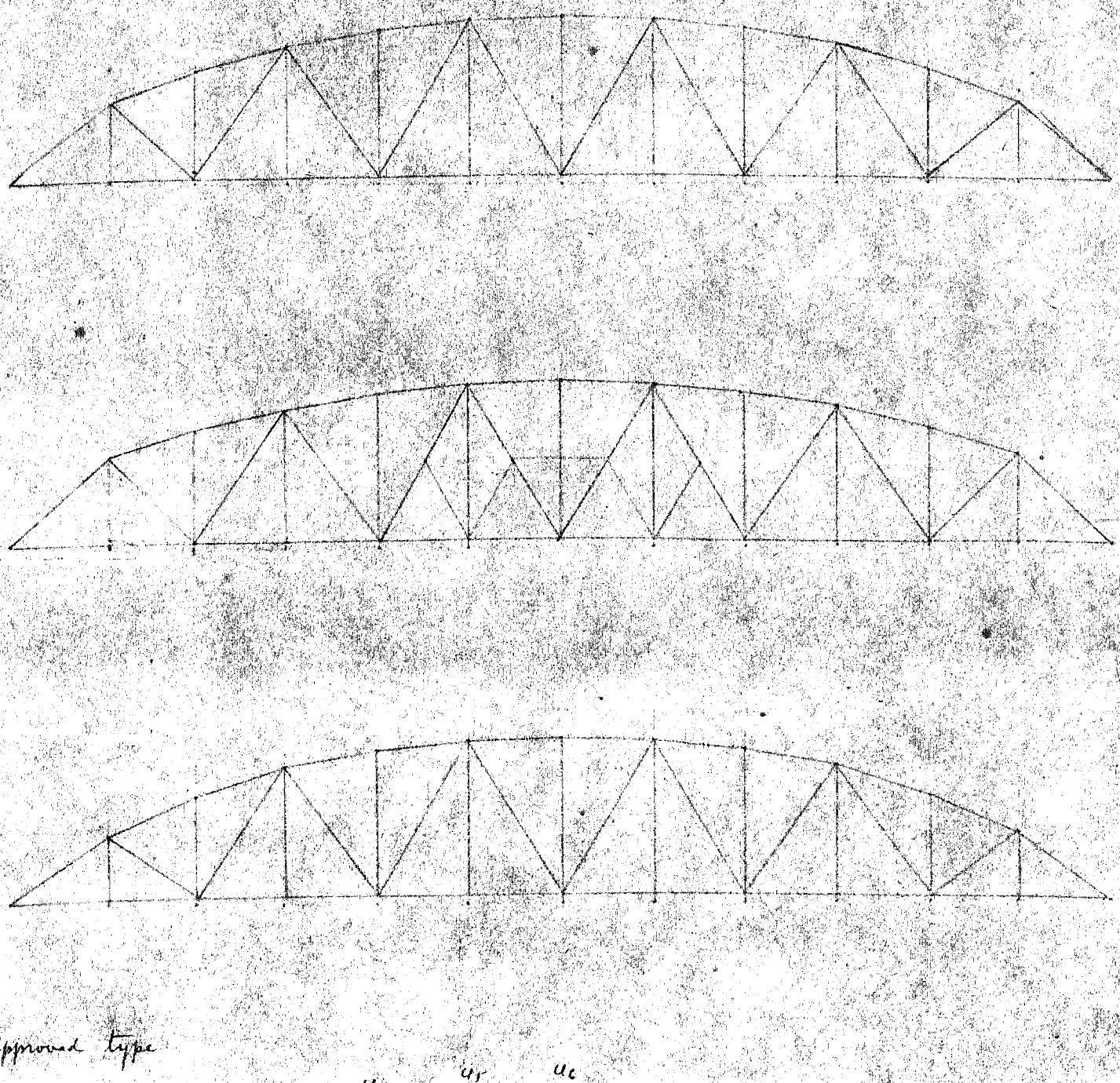
CALCULATIONS FOR 42.B Bridge for city of Tokyo.

☐ Skelton diagram of truss.



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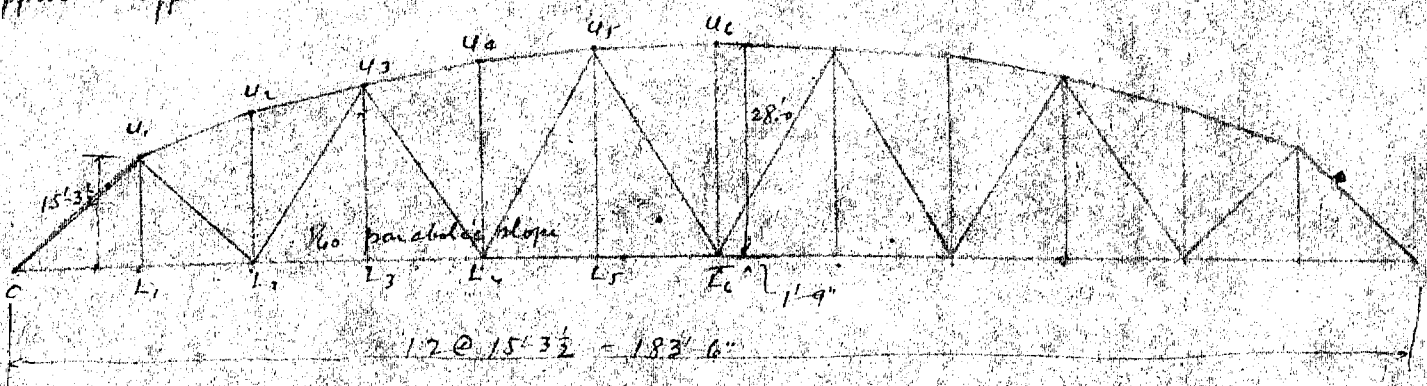

$\frac{1}{60} \times 91.75 = 1.529166 = 1'-5\frac{11}{32}"$

$2\frac{21}{32}"$ deflection assumed

$1'-9"$ take at mean center of bridge as a camber

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CALCULATIONS FOR

Max top chord stress

panel load D.L 76,000 #

L.L 38,000 "

Reaction 11×76,000×½ = 418,000

D.L. stress (418,000×91.75 − 5×76,000×3×15.3) ÷ 27 =

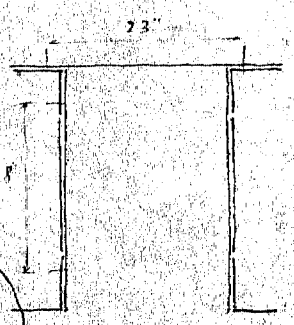
= (38400,000 − 17500,000) ÷ 27 = 20,900,000 ÷ 27 = 774,000 #

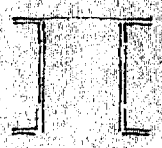
L.L. stress 387,000

Total stress 387,000+774,000 = 1,161,000 #

R.R 1161,000 ÷ 14,000 = 83 D"

1 cov pl.	30×⅞	18.75	1 cov pl	30×⅞	18.75
4 Ls	6×6×½	23.00	4 Ls	6×6×½	23.00
2 webs	24×½	24.00	2 webs	27×½	27.00
2 pls	12×½	12.00	2 pls	15×½	15.00
2 pl.	6×½	6.00			
		88.75			83.75 D"





Weight of truss assumed 28

~~D.R~~ 1161,000 × 20900,000 ÷ 28 = 747,000

374000

1121,000

D.R 1121,000 ÷ 14,000 = 80 D"

End upper chord ~~1121,000 ÷ 14.14~~ = 1"

stress 418,000×1.5×1.414 = 886,000 #

R.R 886,000 ÷ 14,000 = 63.3 D"

1 cov pl	30×⅞	18.75	1 cov pl	30×⅞	18.75
4 Ls	6×6×½	23.00	4 Ls	6×6×½	23.00
2 webs	24×½	24.00	2 webs	27×½	27.00
2 pls	6×½	6.00			
		71.75			68.75 D"

65.75

Max Low chord stress

(418,000×76.45 − 5×76,000×30.6) ÷ 27

= (32,000,000 − 11,600,000) ÷ 27 = 20400,000 ÷ 27 = 75,6000 # 74.2 m

37.8000 37.1 m

113,4000 # 111,3000

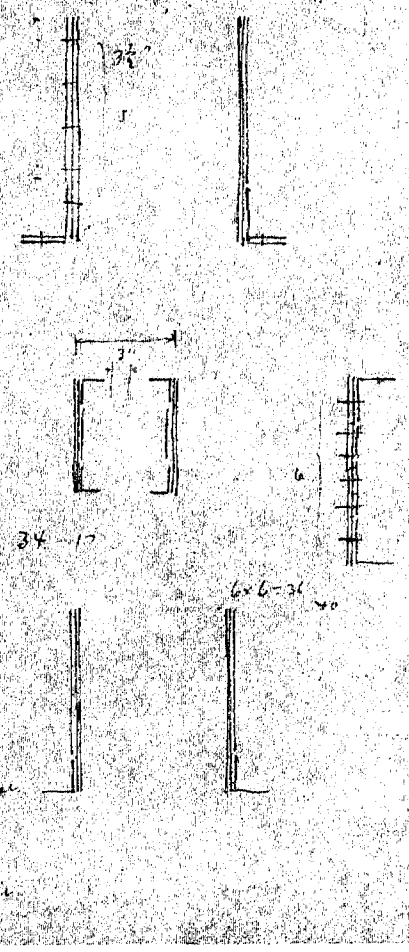
113,400 ÷ 17,000 = 67 D"

111.3000 65.5

4 pls	24×½	48.00	
2 Ls	6×6×½	11.50	
2 pls	18×½	18.00	
2 pls	6×½	77.50 D"	
		83.50 − 17 = 66.5 D"	

6 pls	21×½	63.00	
2 Ls	6×6×½	11.50	
2 pl	15×½	15.00	
		89.50 D"	

4 pls	24×½	48.00	
4 Ls	6×6×½	23.00	
2 pls	12×½	12.00	
		83.00 D" − 17 = 66.0 D"	34 − 17



End Bottom chords

stress 418,000×1.5 = 627,000 #

R.R 627,000 ÷ 17,000 = 37.0 D" net

4 pls	24×½	48.00	
2 Ls	6×6×½	11.50	
		59.50 − 12.00 = 47.5 D" net	

4 pls	24×½	48.00	
2 Ls	6×6×½	11.50	
2 pls	15×½	15.00	
		59.50 − 11.0 = 48.5 D" net	

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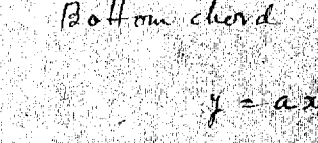
CALCULATIONS FOR 42-B Bridge for city of Tokyo

☐ Length of each members.

Top chords as a parabolic curve assumed.

$(28'-0" + 1'4") - 15'-3\frac{1}{2}" = 29'-9" - 15'-3\frac{1}{2}" = 14'-5\frac{1}{2}"$



$f = 14'-5\frac{1}{2}"$

$l = 91'-9" - 15'-3\frac{1}{2}" = 76'-8\frac{1}{2}"$

$y = ax^2$

$14'-5\frac{1}{2}" = a\,(76'-5\frac{1}{2}")^2$

$a = \frac{14'-5\frac{1}{2}"}{(76'-8\frac{1}{2}")^2}$

$\therefore y = \frac{14'-5\frac{1}{2}"}{(76'-5\frac{1}{2}")^2} x^2 = \frac{14'-5\frac{1}{2}"}{76'-5\frac{1}{2}"} \times \frac{(15'-3\frac{1}{2}")^2}{(15'-3\frac{1}{2}")^2} x^2 = 0.57834\,x^2$

$14'-5\frac{1}{2}"$	→ 1.16012	$76'-5\frac{1}{2}"$	→ 1.88342
$15'-3\frac{1}{2}"$	→ 1.18445		1.88342 +
	1.18445 +		3.76684
	3.52902		
	3.76684 -		
	1.76218	→ 0.57834	= $6\frac{11}{16}"$

$x/15'-3\frac{1}{2}"$	$(\frac{x}{15'-3\frac{1}{2}"})^2$		Y	$y = 14'-5\frac{1}{2}" - Y$	Take
1	1	0.57834	$6\frac{11}{16}"$	$29'-2\frac{1}{16}"$	$29'-2\frac{1}{16}"$
2	4	2.31336	$2'-3\frac{3}{4}"$	$27'-5\frac{1}{4}"$	$27'-5\frac{1}{4}"$
3	9	5.20506	$5'-2\frac{47}{32}"$	$24'-6\frac{17}{32}"$	$24'-6\frac{1}{2}"$
4	16	9.25344	$9'-3\frac{1}{32}"$	$20'-5\frac{31}{32}"$	$20'-6"$
5	25	14.45850	$14'-5\frac{1}{2}"$	$15'-3\frac{1}{2}"$	$15'-3\frac{1}{2}"$

Bottom chord as a parabolic curve 1/60 about.

$y = ax^2$ $\quad y = 1'-9"$ $\quad x = 91'-9"$ $\quad a = \frac{y}{x^2} = \frac{1'-9"}{(91'-9")^2}$

$y = \frac{1'-9"}{(91'-9")^2} x^2 = \frac{1'-9"}{91'-9"} \cdot \frac{15'-5}{15'-3\frac{1}{2}"^2} \cdot \frac{x^2}{(15'-3\frac{1}{2}")^2} =$

$1'-9"$	→ 0.24304	$91'-9"$	→ 1.96261
$15'-3\frac{1}{2}"$	→ 1.18445		1.96261 +
	1.18445 +		3.92522
	2.61894		
	3.92522 -		
	2.68672	→ 0.04861	= $\frac{19}{32}"$

$x/15'-3\frac{1}{2}"$	$(\frac{x}{15'-3\frac{1}{2}"})^2$		Y	$y = 9-\frac{19}{32}" - Y$	Take
0	0	0.00000	0	$1'-9"$	$1'-9"$
1	1	0.04861	$\frac{19}{32}"$	$1'-8\frac{13}{32}"$	$1'-8\frac{3}{8}"$
2	4	0.19444	$2\frac{11}{32}"$	$1'-6\frac{21}{32}"$	$1'-6\frac{5}{8}"$
3	9	0.43749	$5\frac{1}{2}"$	$1'-3\frac{3}{4}"$	$1'-3\frac{3}{4}"$
4	16	0.77776	$9\frac{11}{32}"$	$0'-11\frac{21}{32}"$	$0'-11\frac{5}{8}"$
5	21	1.21525	$1'-2\frac{19}{32}"$	$0'-6\frac{13}{32}"$	$0'-6\frac{5}{8}"$
6	36	1.74996	$1'-9"$	$0'-0"$	$0'-0"$

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CALCULATIONS FOR 47' B. Bridge for City of Tokyo.

Length of Verticals

number			
$U_4 - L_4$	$15'-3\frac{1}{2} - 6\frac{3}{8} = 12'-9\frac{1}{8}"$		14.7604
$U_2 - L_2$	$20'-6" - 11\frac{3}{8} = 19'-6\frac{3}{8}"$		19.5313
$U_3 - L_3$	$24'-6\frac{1}{2} - 1'-3\frac{3}{4} = 23'-2\frac{3}{4}"$		23.2292
$U_4 - L_4$	$27'-5\frac{1}{4} - 1'-6\frac{3}{8} = 25'-10\frac{7}{8}"$		25.8854
$U_5 - L_5$	$29'-2\frac{1}{16} - 1'-8\frac{3}{8} = 27'-5\frac{11}{16}"$		27.4740
$U_6 - L_6$	$29'-9" - 1'-9" = 28'-0"$		28.0000

Length of upper chords

+233.8351 L

number					
$U_6 - U_5$	$29'-9" - 29'-2\frac{1}{16}" = 6\frac{15}{16}"$	→ 0.3342	234.1693	→ $15'-3\frac{5}{8}"$	15.3021
$U_5 - U_4$	$29'-2\frac{1}{16}" - 27'-5\frac{1}{4}" = 1'-8\frac{13}{16}"$	→ 3.0081	236.5432	→ $15'-4\frac{9}{16}"$	15.3802
$U_4 - U_3$	$27'-5\frac{1}{4} - 24'-6\frac{1}{2} = 2'-10\frac{3}{4}"$	→ 8.3859	242.2202	→ $15'-6\frac{3}{4}"$	15.5625
$U_3 - U_2$	$24'-6\frac{1}{2} - 20'-6" = 4'-0\frac{1}{2}"$	→ 16.3351	250.1702	→ $15'-9\frac{13}{16}"$	15.8177
$U_2 - U_1$	$20'-6" - 15'-3\frac{1}{2} = 5'-2\frac{1}{2}"$	→ 27.1267	260.9618	→ $16'-1\frac{27}{32}"$	16.1536
$U_1 - U_0$	$15'-3\frac{1}{2} - 0 = 15'-3\frac{1}{2}"$	→ 233.8351	467.6702	→ $21'-7\frac{1}{2}"$	21.6250

Length of Bottom chords

15'3½ → 233.8351

$L_6 - L_5$	$1'-9" - 1'-8\frac{3}{8} = 0\frac{5}{8}"$	→ 0.00271	233.8378	→ $15'-3\frac{1}{2}"$	2917 15.3021
$L_5 - L_4$	$1'-8\frac{3}{8} - 1'-6\frac{7}{8} = 1\frac{3}{4}"$	→ 0.02127	233.8564	→ $15'-3\frac{1}{2}"$	2917 15.3021
$L_4 - L_3$	$1'-6\frac{7}{8} - 1'-3\frac{3}{4} = 2\frac{1}{8}"$	→ 0.05740	233.8925	→ $15'-3\frac{17}{32}"$	15.2943
$L_3 - L_2$	$1'-3\frac{3}{4} - 0'-11\frac{3}{8} = 4\frac{7}{8}"$	→ 0.11816	233.9533	→ $15'-3\frac{17}{32}"$	15.2943
$L_2 - L_1$	$0'-11\frac{3}{8} - 0'-6\frac{7}{8} = 5\frac{1}{2}"$	→ 0.19141	234.0265	→ $15'-3\frac{9}{16}"$	15.2969
$L_1 - L_0$	$0'-6\frac{3}{4} - 0'-0" = 6\frac{3}{8}"$	→ 0.28223	234.1173	→ $15'-3\frac{5}{8}"$	15.3021

Length of Diagonals

+233.8351 L

$U_1 - L_2$	$14'-9\frac{1}{8}" - 5\frac{1}{4}" = 14'-3\frac{7}{8}"$	→ 205.1459	438.9810	→ $20'-11\frac{7}{16}"$	20.9531
$L_2 - U_3$	$23'-2\frac{3}{4} + 4\frac{1}{8}" = 23'-6\frac{7}{8}"$	→ 555.6824	789.5175	→ $28'-1\frac{3}{16}"$	28.0990
$U_3 - L_4$	$23'-2\frac{3}{4} - 2\frac{7}{8}" = 22'-11\frac{7}{8}"$	→ 528.5209	762.3560	→ $27'-7\frac{11}{32}"$	27.6120
$L_4 - U_5$	$27'-5\frac{11}{16}" + 1\frac{3}{4}" = 27'-7\frac{7}{16}"$	→ 762.8529	996.6880	→ $31'-6\frac{27}{32}"$	31.5499
$U_5 - L_6$	$27'-5\frac{11}{16}" - 0\frac{5}{8}" = 27'-5\frac{1}{16}"$	→ 751.9592	985.7943	→ $31'-4\frac{25}{32}"$	31.3984

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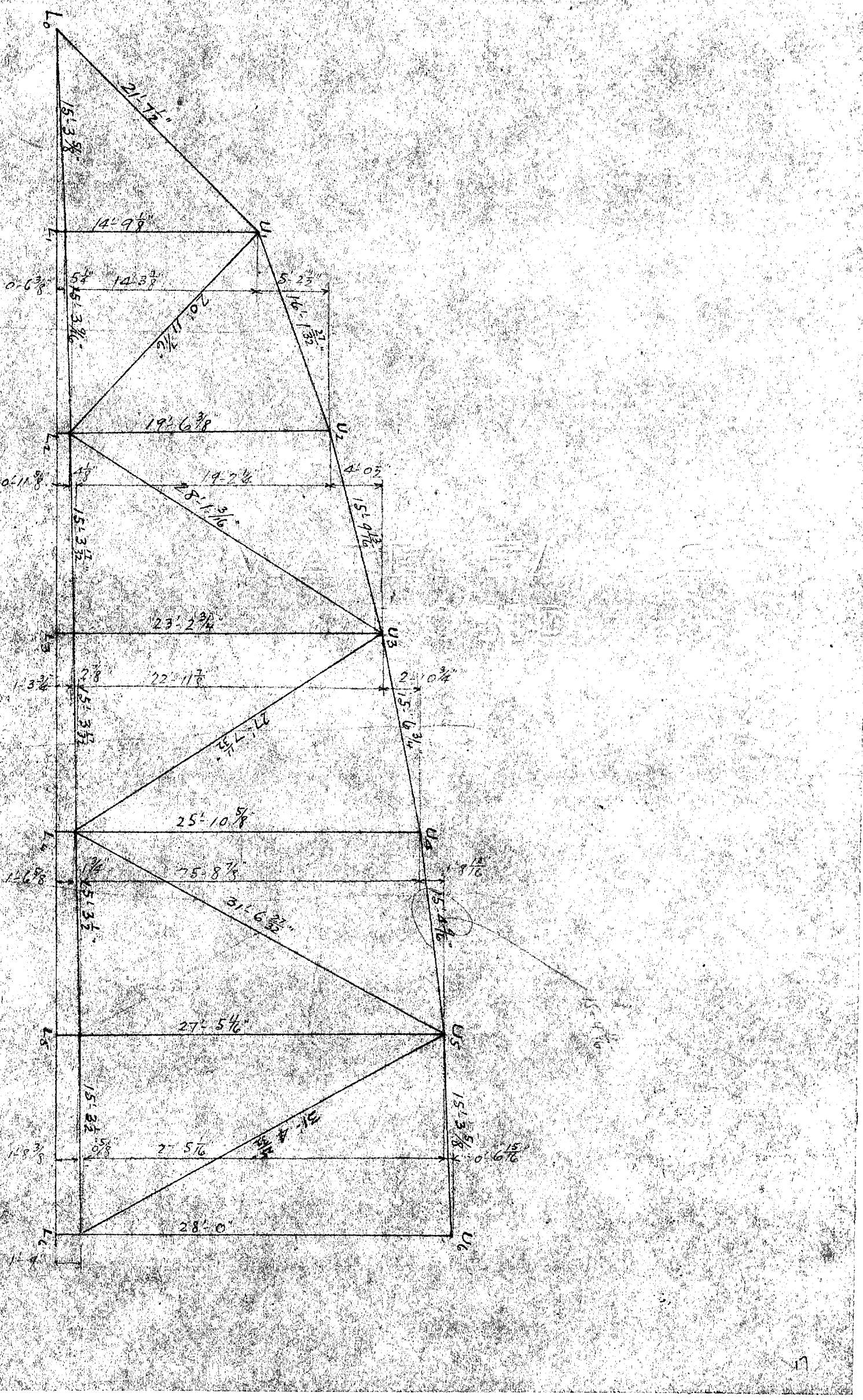
CALCULATIONS FOR 42 ft. Bridge for city of Tokyo.



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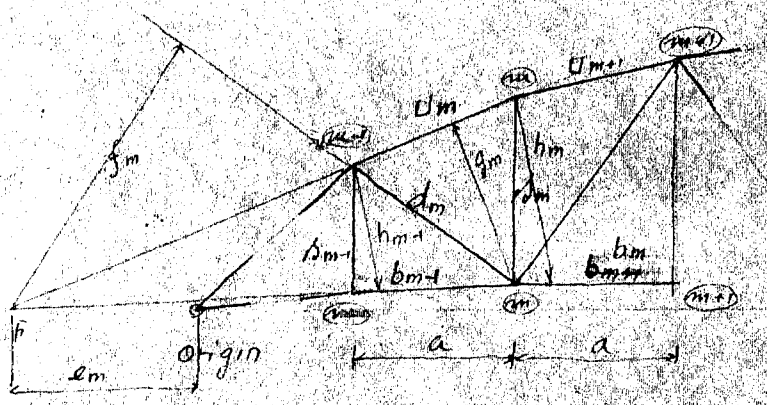
CALCULATIONS FOR



g_m = Perpendicular distance from lower panel pt m to upper chord U_m

h_m = Perpendicular distance from upper panel pt m to lower chord b_m

f_m = perpendicular distance from (F) to diagonal d_m, where F is a intersection pt of upper chord U_m and lower chord b_m.

$$g_m = \frac{a}{U_m}\delta_m \qquad h_m = \frac{a}{b_m}\delta_m \qquad f_m = \frac{\delta_{m-1}}{\delta_m - \delta_{m-1}} \cdot \frac{a}{d_m} \cdot \delta_m$$

$$l_m = \frac{\delta_{m-1}}{\delta_m - \delta_{m-1}} a + (m-1)a$$

Find g_m.

panel pt. (m)	a	U_m	~~δ_m~~	δ_m	g_m	Remark
1	15.29	21.68	~~14.76~~	—	—	
2	"	16.15	19.53	19.53	18.50	for U_1-U_2
2'	"	15.82	23.23	19.53	18.88	U_2-U_3
4	"	15.56	25.89	25.89	25.44	U_3-U_4
4'	"	15.38	27.47	25.89	25.75	U_4-U_5
6	"	15.30	28.00	28.00	28.00	U_5-U_6

Find h_m

panel pt (m)	a	b_m	δ_m	h_m	Remarks
1	15.29	15.30	14.76	14.76	for L_0-L_1
1'	"	15.30	14.76	14.76	L_1-L_2
3	"	15.29	23.23	23.23	L_2-L_3
3'	"	15.29	23.23	23.23	L_3-L_4
5	"	15.29	27.47	27.47	L_4-L_5
5'	"	15.29	27.47	27.47	L_5-L_6

Find f_m

panel pt (m)	a	d_m	δ_m	δ_{m-1}	$\delta_m - \delta_{m-1}$	f_m
2	15.29	20.95	19.53	14.76	4.77	44.23
3	"	28.10	23.23	19.53	3.70	66.70
4	"	27.61	25.89	23.23	2.66	124.70
5	"	31.56	27.47	25.89	1.58	218.00
6	"	31.40	28.00	27.47	0.53	706.00

Find l_m

panel pt (m)	δ_{m-1}	$\delta_m - \delta_{m-1}$	a	$\frac{\delta_{m-1}}{\delta_m - \delta_{m-1}} a$	$(m-1)a$	l_m
2	14.76	4.77	15.29	47.20	15.29	31.9
3	19.53	3.70	"	80.80	30.6	50.2
4	23.23	2.66	"	133.70	45.9	87.8
5	25.89	1.58	"	250.80	61.2	189.6
6	27.47	0.53	"	793.00	67. 76.5	716.5

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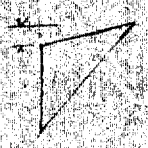
CALCULATIONS FOR 42-B. Bridge for City of Tokyo

☐ Influence Surfaces for upper chord

	L_0-U_1	U_1-U_2	U_2-U_3	U_3-U_4	U_4-U_5	U_5-U_6
0	0.000	0.000	0.000	0.000	0.000	0.000
1	~~-0.953~~ -1.300	-0.689	-0.675	-0.401	-0.396	-0.273
2	~~-1.300~~ -0.866 -1.300	-1.378	-1.350	-0.802	-0.792	-0.546
3	0.788 -1.065	-1.240	-1.215	-1.203	-1.188	-0.819
4	-0.693 -0.946	-1.102	-1.080	-1.603	-1.583	-1.092
5	-0.606 -0.828	-0.965	-0.945	-1.402	-1.385	-1.365
6	-0.520 -0.710	-0.827	-0.810	-1.202	-1.187	-1.638
5'	-0.433 -0.591	-0.689	-0.675	-1.002	-0.989	-1.365
4'	-0.347 -0.473	-0.551	-0.540	-0.801	-0.791	-1.092
3'	-0.260 -0.355	-0.413	-0.405	-0.601	-0.593	-0.819
2'	-0.173 -0.237	-0.276	-0.270	-0.401	-0.396	-0.546
1'	-0.087 -0.118	-0.138	-0.135	-0.200	-0.198	-0.273
0	0.000 -0.000	0.000	0.000	0.000	0.000	0.000
	-7.806					
Sum	~~-5.708~~	-8.268	-8.100	-9.618	-9.498	-9.828

L_0-U_1 unit load on 1

Reaction = 11/12



$6\frac{3}{8}$
$14\frac{1}{2}\ 9\frac{1}{8}$

$6\frac{3}{8}$ → 1.72530
$14'9\frac{1}{8}$ → 1.16910
2.55620 → 0.04

$\delta_{1\sim0} = \frac{11}{12} \times 1.04 \times 1.44^{m} = 0.953 = 1.300$

U_1-U_2 unit load on 2

$\delta = \frac{10}{12} \times 2 \times 15.29 \div 18.50 = 1.378$

U_2-U_3 load on 2, $\delta = \frac{10}{12} \times 2 \times 15.29 \div 18.88 = 1.350$

U_3-U_4 load on 4, $\delta = \frac{8}{12} \times 4 \times 15.29 \div 25.44 = 1.603$

U_4-U_5 " " $\delta =$ " $\div 25.79 = 1.583$

U_5-U_6 load on 6, $\delta = \frac{1}{2} \times 6 \times 15.29 \div 28.00 =$

☐ Stresses of upper chords

	L_0-U_1	U_1-U_2	U_2-U_3	U_3-U_4	U_4-U_5	U_5-U_6
Influence Surfa	-7806 -5708	-8,268	-8,100	-9,618	-9,498	-9,828
D.L. stress (76,000)	-593300 -433800	-628,400	-616600	-731,000	-721800	-746,900
L.L. stress (38,000)	-296700 -216,900	-314,200	-308300	-365,500	-360,900	-373,500
Total stress	-890,000 -650.700	-942,600	-924,900	-1096,500	-1082,700	-1120,400
A.R	46.5 63.6°	67.4	66.1	78.3	77.4	80.0 R"

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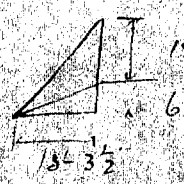
CALCULATIONS FOR 43-B Bridge for city of Tokyo

☐ Influence Surfaces for Lower chords

L_0-L_1



$\frac{15'\,3\frac{5}{8}}{14'\,9\frac{7}{8}}$

$15'\,3\frac{5}{8} \rightarrow 1.18475$
$14'\,9\frac{7}{8} \rightarrow 1.16910$
$0.01565 \rightarrow 1.004$ say

load on 1 stress $\frac{11}{12} \times 1.004 = 0.953$

L_0-L_2

load on 1 stress $\frac{11}{12} \times 15.29 \div 14.76 = 0.950$

L_2-L_3, L_3-L_4 } load on 3 $\delta = \frac{9}{12} \times 3 \times 15.29 \div 23.23 = 1.482$

L_4-L_5, L_5-L_6 } load on 5 $\delta = \frac{9}{12} \times 5 \times 15.29 \div 27.07 = 1.623$

	L_0-L_1	L_1-L_2	L_2-L_3. L_3-L_4	L_4-L_5. L_5-L_6
0	0.000	0.000	0.000	0.000
1	0.953	0.950	0.741	0.406
2	0.866	0.864	1.482	0.812
3	0.780	0.778	1.332	1.217
4	0.693	0.691	1.184	1.623
5	0.606	0.605	1.036	1.420
6	0.520	0.518	0.888	1.217
5'	0.433	0.432	0.741	1.015
4'	0.347	0.346	0.592	0.812
3'	0.260	0.259	0.444	0.609
2'	0.173	0.173	0.296	0.406
1'	0.087	0.087	0.148	0.203
0	0.000	0.000	0.000	0.000
Area	5.708	5.703	8.884	9.738

☐ Stresses of Lower chords and req'd section

	L_0-L_1	L_1-L_2	L_2-L_3', L_3-L_4	L_4-L_5, L_5 L_6
Influence Surface	5.708	5.703	8.884	9.738
D.L. stress	433,800	433,400	675,200	740,000
L.L. stress	216,900	216,700	337,600	370,000
Total stress	650,700	650,100	1012,800	1110,000
S.R	38.3"	38.3	59.6	65.3"

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CALCULATIONS FOR

☐ Influence Surfaces for Diagonals

$U_1 - L_2$ load on 2 $\beta = \frac{10}{12} \cdot \frac{31.9}{44.33} = 0.600$

load on 1 $\beta' = \frac{11}{12} \cdot \frac{31.9}{44.33} - \frac{4}{10} \frac{15.29+3.19}{44.33} = 0.661 - 1.067 = -0.406$

$L_2 - U_3$ load on 3 $\beta = \frac{9}{12} \cdot \frac{50.2}{66.7} =$ [illegible]

load on 2 $\beta' = -\frac{10}{12} \times \frac{50.2}{66.7} + \frac{50.2 + 2 \times 15.29}{66.7} = -0.628 + 1.212 = 0.584$

$U_3 - L_4$ load on 4 $\beta = \frac{8}{12} \times \frac{87.8}{124.7} = 0.470$

load on 3 $\beta' = \frac{9}{12} \times \frac{87.8}{124.7} - \frac{87.8 + 3 \times 15.29}{124.7} = 0.528 - 1.055 = -0.527$

$L_4 - U_5$ load on 5 $\beta = -\frac{7}{12} \times \frac{189.6}{218.0} = -0.507$

load on 4 $\beta' = -\frac{8}{12} \times \frac{189.6}{218.0} + \frac{189.6 + 4 \times 15.29}{218.0} = -0.580 + 1.150 = 0.570$

$U_5 - L_6$ load on 6 $\beta = \frac{1}{2} \times \frac{716.5}{706.0} = +0.507$

load on 5 $\beta' = \frac{7}{12} \times \frac{716.5}{706.0} - \frac{716.5 + 5 \times 15.29}{706.0} = 0.592 - 1.123 = -0.531$

	$U_1 - L_2$	$L_2 - U_3$	$U_3 - L_4$	$L_4 - U_5$	$U_5 - L_6$
0	0.000	0.000	0.000	0.000	0.000
1	−0.406	0.292	−0.176	0.143	−0.106
2	0.600	0.184	−0.352	0.285	−0.213
3	0.540	−0.565	−0.527	0.428	−0.319
4	0.480	−0.502	0.470	0.570	−0.425
5	0.420	−0.440	0.411	−0.507	−0.531
6	0.360	−0.377	0.353	−0.435	0.507
5′	0.300	−0.314	0.294	−0.362	0.423
4′	0.240	−0.251	0.235	−0.290	0.338
3′	0.180	−0.188	0.176	−0.217	0.254
2′	0.120	−0.126	0.118	−0.145	0.169
1′	0.060	−0.063	0.059	−0.072	0.085
0	0.000	0.000	0.000	0.000	0.000
Area +A	3.300	0.876	2.116	1.426	1.776
−A	0.406	−2.826	−1.055	−2.028	−1.594
Total	+2.894	−1.950	1.061	−0.602	0.182

☐ Stresses and Req'd Section by Diagonals

	$U_1 - L_2$	$L_2 - U_3$	$U_3 - L_4$	$L_4 - U_5$	$U_5 - L_6$
D.L. stress	219,900	−148,200	80,600	−45,800	13,800
L.L. stress	125,400	−107,400	80,400	−77,100	67,500
				−54,200	−60,600
Total stress	345,300	−255,600	161,000	−127,100	104,700
					−90,900
Section req'd	203 sq in net	18.3 sq in gross (16,000)	9.5 net	9.4 sq in gross (σ_a = 13,500)	6.2 sq in net

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CALCULATIONS FOR 42-B Bridge for city of Tokyo

☐ Stresses and Req'd Section of Vertical members.

member U_1-L_1 } Dead load stress - 76.000
U_3-L_3 } L.L stress - 38.000
U_5-L_5 } 114,000 # T. S.R - 6.7" rel.

member U_2-L_2

$\overline{U_1-U_2}\cos\alpha - \overline{U_2-U_3}\cos\beta$

$\cos\alpha = \frac{5'\,2\frac{1}{2}"}{16'\,1\frac{27}{32}"}$

$5'\,2\frac{1}{2}" \rightarrow 0.71670$
$16'\,1\frac{27}{32}" \rightarrow 1.20827$
$\overline{-}\,0.50843 \rightarrow 0.322$

$\cos\beta = \frac{4'\,0\frac{1}{2}"}{15'\,9\frac{13}{16}"}$

$4'\,0\frac{1}{2}" \rightarrow 0.60656$
$15'\,9\frac{13}{16}" \rightarrow 1.19914$
$\overline{-}\,1.40742 \rightarrow 0.256$

D.L. Stress

$628,400 \times 0.322 - 616,600 \times 0.256 = 202,300 - 157,800 = 44,500$ #

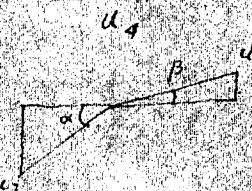
L.L. Stress

panel load on U_2 assumed 10,000 # — 22,300 — 66,800 #

Resultant U.L Stress $44,500 - 10,000 = 35,500$ #
L.L Stress 22,300
57,800 # tension

S.R = $57,800 \div 17,000 = 3.4$" rel

member U_4-L_4



$\cos\alpha = \frac{2'-10\frac{3}{4}"}{15'\,6\frac{3}{4}"}$

$2'\,10\frac{3}{4}" \rightarrow 0.46177$
$15'\,6\frac{3}{4}" \rightarrow 1.19208$
$\overline{-}\,1.26969 \rightarrow 0.186$

$\cos\beta = \frac{1'-8\frac{13}{16}"}{15'\,4\frac{9}{16}"}$

$1'\,8\frac{13}{16}" \rightarrow 0.23914$
$15'\,4\frac{9}{16}" \rightarrow 1.18696$
$\overline{-}\,1.05218 \rightarrow 0.113$

D.L Stress $\frac{731,000}{776,600}$ $\frac{721,800}{736,000}$
$366,500 \times 0.186 - 360,900 \times 0.113$
$= 68,200 - 40,800 =$ 27,400 #
136,400 - 81,600 54,800
panel dead load assumed 10,000 10,400
44,400
27,000
27,400
13,700
Total 71,800 # T
40,700

L.L. Stress

S.R = $\frac{71,800}{40,700} \div 17,000 = 2.4$" rel
422

member U_6-L_6

$\cos\alpha = \frac{6\frac{15}{16}"}{15'\,3\frac{5}{8}"}$

$6\frac{15}{16}" \rightarrow 1.76202$
$15'\,3\frac{5}{8}" \rightarrow 1.18473$
$\overline{-}\,2.57727 \rightarrow 0.038$

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CALCULATIONS FOR 42-B Bridge for city of Tokyo.

W.L. stress 746,900 × 0.038 × 2 = 56,800
Panel Wear load assumed 10,800
46,800
L.L. stress 28,400
Total stress 75,200
S.R = 75,200 ÷ 17,000 = 4.43" net

☐ Assumed Section of Each members.

upper chords.

$U_5 - U_6$ S.R = 80.0" gross

1 cov pl.	30 × 5/8	18.75
2 web pls	24 × 1/2	24.00
4 Ls	6×6× 1/2	23.00
2 side pls	12 × 1/2	12.00
2 pls	6 × 1/2	6.00
		83.75 □"

$U_4 - U_5$ S.R = 77.4 Same section used as $U_5 - U_6$
$U_3 - U_4$ S.R = 78.3 "
$U_2 - U_3$ S.R = 66.1

1 cov pl	30 × 7/8	18.75
2 web pls	24 × 1/2	24.00
4 Ls	6×6× 1/2	23.00
2 pls	6 × 1/2	6.00
		71.75 □"

$U_1 - U_2$ S.R = 67.4 Same section used as $U_2 - U_3$.
$U_0 - U_1$ S.R = 63.6 "

Lower chords.

$L_5 - L_6$, $L_4 - L_5$ S.R = 65.3" net

4 pls	24 × 1/2	48.00	38.00
2 Ls	6×6× 1/2	11.50	9.50
2 side pls	18 × 1/2	18.00	14.00
2 pls	6 × 1/2	6.00	5.00
		83.50	66.50 □" net

$L_3 - L_4$, $L_2 - L_3$ S.R = 59.6 □" net

4 pls	24 × 1/2	48.00	38.00
2 Ls	6×6× 1/2	11.50	9.50
2 side pls	18 × 1/2	18.00	14.00
		77.50	61.50 □"

$L_1 - L_2$, $L_0 - L_1$ S.R = 38.3 □" net

4 pls	24 × 1/2	48.00	38.00
2 Ls	6×6× 1/2	11.50	9.50
		59.50	47.50 □"

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CALCULATIONS FOR -

Diagonals

$L_5 - L_6$: $S.R = 6.2$" net

2Ls 15×4"[@ 41.4 area = 20.12.3.4 = 24.668"

used 4Ls 3½×3½×⅜ @ 24.8 = 99.2"
2 Pls PL×5/16 @ 46.9 = 8.76 / 4.38
6.14 — 143.0 / 18.68 "

$L_5 - L_4$: $S.R = 9.4$" 9.70"

4Ls 3½×3½×⅜ } used
2 pls 15×5/16

Rad of gyration 0.36×14 = 5.04"

$\frac{L}{r} = \frac{31.5 \times 12}{5.04} = 75$

$\sigma = 21,300 (1 - 0.0013 \times 75) = 21300 \times 0.58.75 = 12,500$ #/□"

$S.R = 10.2$"

$L_3 - L_4$: $S.R = 9.5$" net

4Ls 3½×3½×⅜" / 2 pls 14×5/16 — 18.68 / 45.88" gross

$L_2 - L_3$: $S.R = 25,600$ # comp.

$\sigma = 21300 (1 - 0.0013 \times \frac{28 \times 2}{30 \times})$ = 21,300 × 0.633 = 13,500 #

$S.R = 19.0$

4Ls 5×3½×⅜ @ 30.5 = 12.20"
2 pls 14×⅜ — 10.50
22.70"

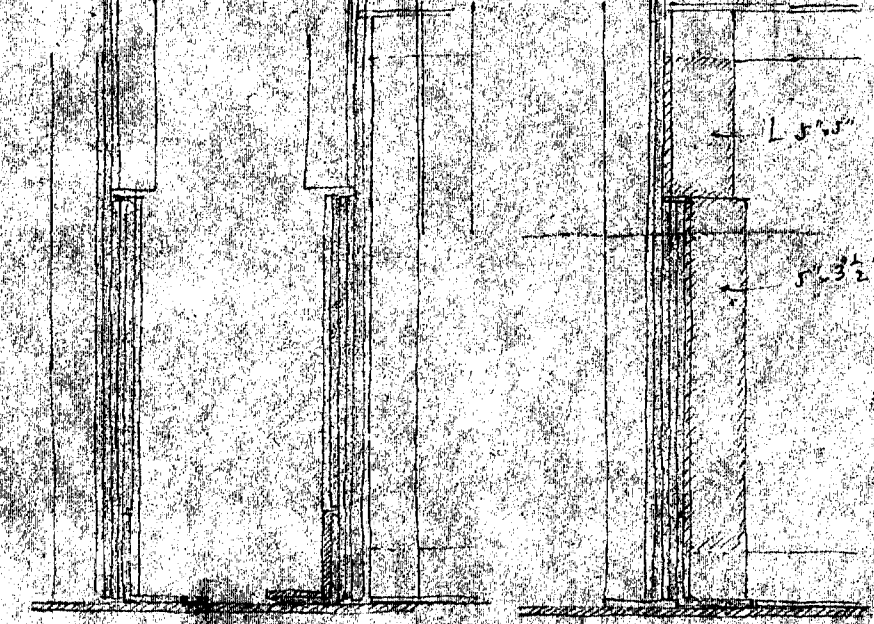
$L_1 - L_2$ $S.R$ 20.3" net

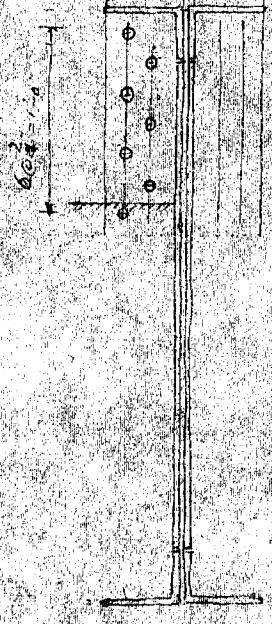
4Ls 6×6×½ @ 23.0 = 19.0
2 pls 16×½ ⅜ — 12.0 = 9.75
35.0 — 28.75

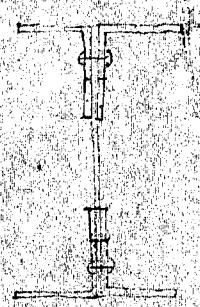
Verticals all members

4Ls 6×4×½ @ 4.75 = 3.61 / 14.44 / 6.14 / 20.63"
1 web 16½×⅜×5/16

19.60







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CALCULATIONS FOR 42. B Bridge for City of Tokyo.

Diagonals

U_5-L_6 Stress = 104,700 #T
90,900 #C

2 [12" × 3½ @ 26.10# area = 2 × 7.67 = 15.34 □"

r = 4.117

21,300 (1 − 0.00 SJ × $\frac{31.5 \times 12}{4.117}$) = 10700 #

R.R = 90,900 ÷ 10,700 = 8.5 □"

L_4-U_5 Stress 127,100 #C R.R = $\frac{127,100}{10,700}$ = 11.9 □"

Used 2 [12 × 3½ @ 26.10# Ded. area 15.35 □"

U_3-L_4 161,000 # R.R = 9.5

Section used 2 [12 × 3½ @ 26.10 #

L_2-U_3 Stress = 255,600 # C

21,300 (1 − 0.00 SJ × $\frac{28 \times 12}{4.47}$) = 21300 (1.0 − 0.46) = 12,~~498~~ 498 #
441

R.R = 255,600 ÷ 12,498 = 20.5 □"

21,300 (1 − 0.00 SJ × $\frac{28 \times 12}{5.529}$) = 14,000

R.R = 255,600 ÷ 14,000 = 18.3 □"

2 [15 × 4 [@ 41.94 # area = 2 × 12.334 = 24.668

U_1-L_2

R.R = 20.3 □" used

used 2 [15 × 4 @ 41.94 # 24.668 − 4.4 = 20.3 □"

Verticals all members

4L 5 × 3½ × 7/16 → @ 3.05 = 12,200
1 pl. 15½ × 7/16 4,840
17,040 #

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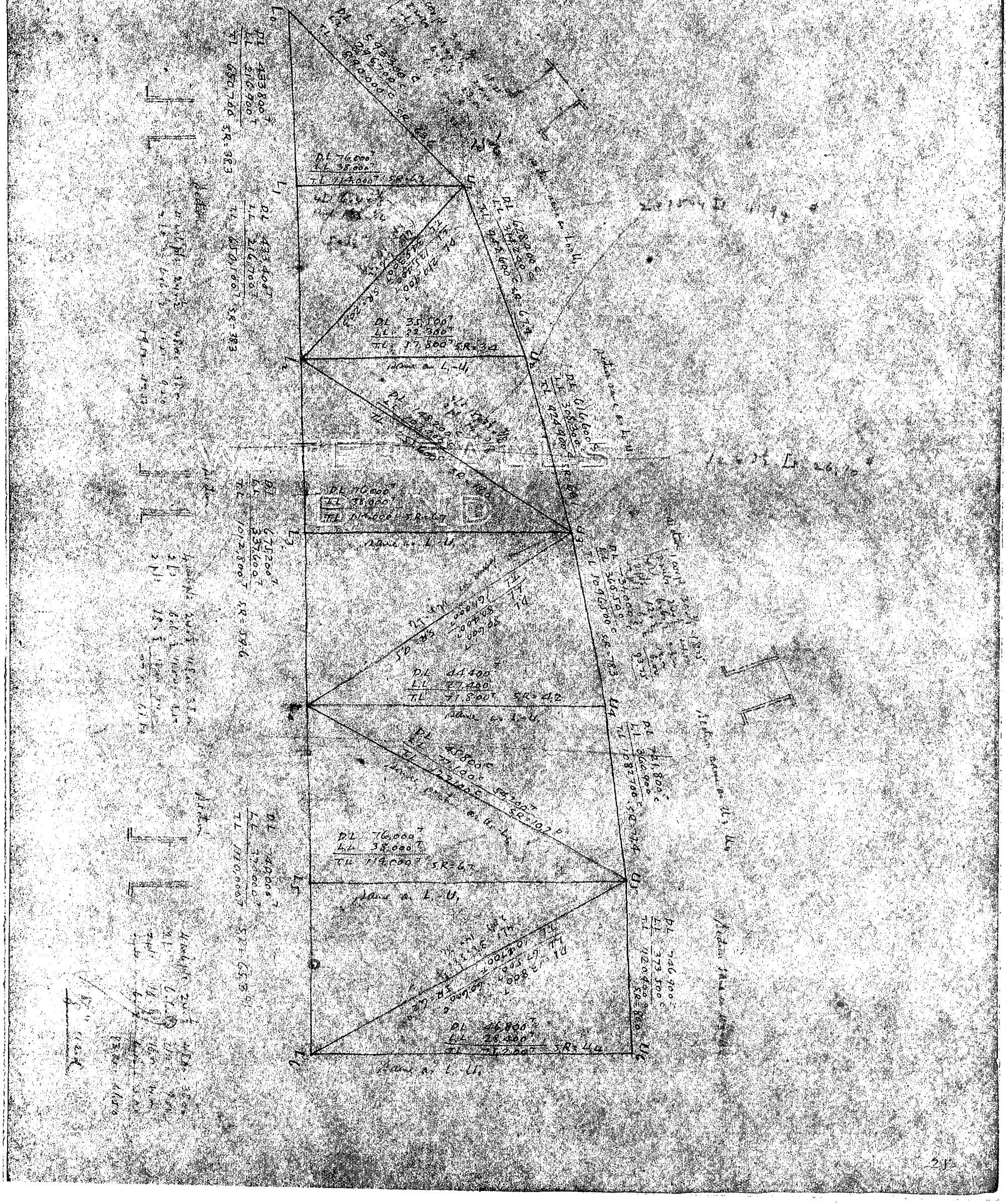
CALCULATIONS FOR



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CALCULATIONS FOR 41 B Bridge for city of Tokyo

1. Estimate of Weight of Truss members

				detail	Total wt say
$L_0 - U_1$	71.75 × 3.4 × 21.6	= 5,260	~~4,300~~ 2000		7,300
$U_1 - U_2$	" × 16.1	= 3,930	6,300		5,200
$U_2 - U_3$	" × 19.8	= 3,850	1,270		5,800
$U_3 - U_4$	83.75 × 3.4 × 15.6	= 4,440	1,450		5,900
$U_4 - U_5$	" × 15.7	= 4,380	"		5,800
$U_5 - U_6$	" × 14.6	= 4,380	-		5,800
		20,240			31,100 × 2 = 70,200
$L_0 - L_1$	19.50 × 3.4 × 14.3	= 3,100	1000		4,100
$L_1 - L_2$	" × 15.3	= 3,100	"		4,100
$L_2 - L_3$	77.50 × 3.4 × 15.3	= 4030	1300		5,200
$L_3 - L_4$	"	= 4,030	"		5,200
$L_4 - L_5$	83.50 × 3.4 × 15.3	= 4,340	1400		5,700
$L_5 - L_6$	"	= 4,340	1,400		5,700
		32,940			30,000 × 2 = 60,000
					2,000
$U_1 - L_2$	2 × 42.0 × 21.6	= 1,770	350		2,800
$L_2 - U_3$	" × 28.8	= 2,360	470		1,700
$U_3 - L_4$	2 × 26.1 × 27.6	= 1,440	290		2,000
$L_4 - U_5$	" × 31.5	= 1,650	330		2,000
$U_5 - L_6$	" × 31.5	= 1,650	330		10,300 × 2 = 21,000
		8,870			
$U_1 - L_1$	17.0 × 3.4 × 14.8	= 860			900
$U_2 - L_2$	" × 19.5	= 1,130			1100
$U_3 - L_3$	" × 23.2	= 1,340			1300
$U_4 - L_4$	" × 25.9	= 1,500			1500
$U_5 - L_5$	" 27.5	= 1,590			1600
$U_6 - L_6$	" 28.0	= 1,620			1600
					Σ = 14,400

Total 130.5 ton

Total = 165,600 #
Allow etc 14,400
180,000 #

165,600 = 783.5 =

180,000 ÷ 130.5 ton = 1,38
Adds 38 %

180,000 - 1835 = 980 #/ in ft ca. 1000 #
Say 1000 lbs

2 @ 180,000 - 2240 = 162.0 ton

2. Upper panel load

Truss → Top chord 40,100
Bracing 8,500
Wind load 5,200
54,500 ÷ 7 = 7,800 #
Lateral 1,200
9,000 #

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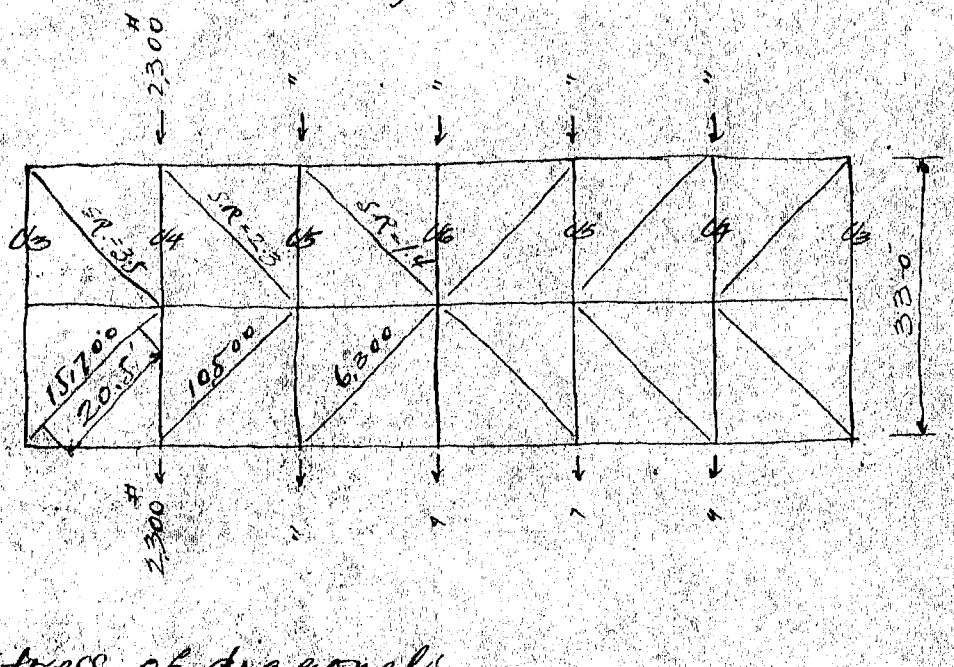
CALCULATIONS FOR 4/13 Bridge for city of Tokyo

Upper Lateral Bracings



Stress of diagonals

Reaction = 5 @ 2300 = 11,500 #

11,500 × 1.363 = 15,700 C

$\frac{2,300 \times 20}{6} \times 1.363 = 10500$ C

$\frac{2,300 \times 12}{6} \times 1.363 = 6,300$ C

$\frac{l}{r} = \frac{246}{1.76} = 140$

2 Ls 6 × 4 × 3/8" r = 1.76 area of 2 Ls = 7.22 □"

21,300 × (1 − 0.00055 × 140) = 4,540

Reaction = 5 @ 2300 = 11500 #

Stress of diagonals 11500 × 1.363 = 15,700 # Tension or comp.

S.R. = 1.2 □"

$\frac{l}{r}$ = 200 for tension. req'd r = $\frac{220 \times 12}{200}$ = 1.35"

⌐ 5" × 3½" × 3/8" r = 1.60"

Main stringer chord 11500 # S.R. = 1.0 □"

$\frac{l}{r}$ = 140 for comp. mem. req'd r = $\frac{33 \times 12}{140}$ = 2.83"

3/8" equal 3/8

⊤ 5" × 3½" × 3/8" r = 2.45" 4 × 10.4 = 41.6 #

r = $\frac{16.5 \times 12}{140}$ = 1.42"

□ Estimating stress in for upper lateral angle

chord 7 @ 80 × 33 = 2640 # × 7 = 18,480 # for 2 Ls 5" × 3½" × 3/8" Tube

Lateral 6 × 4 × 25 # × 22.5 = 15,225 × 2 = 30,450 #

31,980 # = 2,740 = 143 lbs

31,980 ÷ (2 × 183.5) = 87 #/ lin. ft

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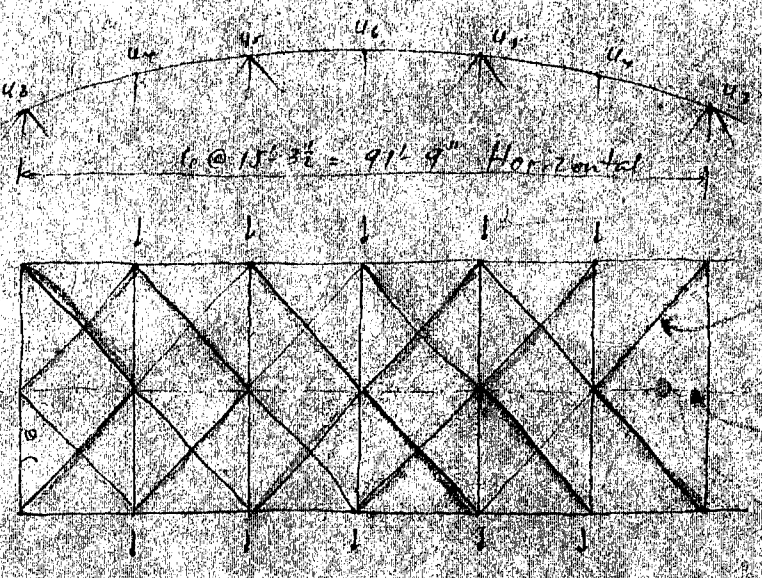
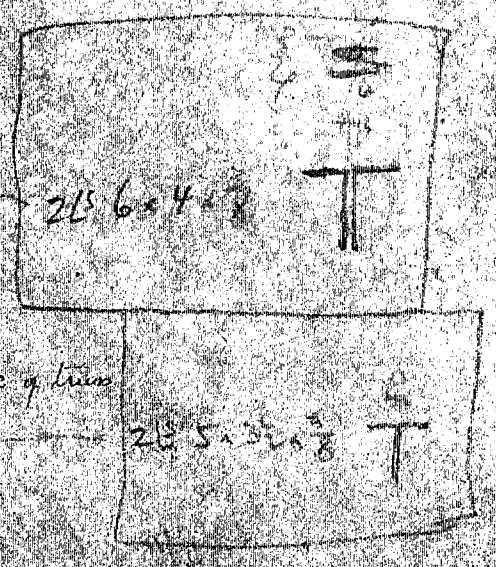
Skeleton diagram.





Wind load.

Span length = 183'-6" = 56 m.

w = intensity of wind load = 200 kg/m or 135 #/ft for 50 m or less

40' = 15 kg/m or 10 #/ft for 10 m over 50 m

panel wind load = (135 + 10) x 15 x 2/3 = 2300 # say

panel load due to earth quake

Coef of Earth quake = ⅓ K = ⅓

panel load 9,000 x ⅓ x 1/8 = 1700 #

wind load effect is larger than earth quake effect

Max stress of diagonals

sec θ = $\frac{\sqrt{33^2 + 15^2}}{33.0} = \frac{36.4}{33.0} = 1.110$

sec θ = $\frac{\sqrt{16.5^2 + 15^2}}{33 \times 16.5} = \frac{22.3}{16.1} = 1.363$

Reaction = 5 P = 2300 = 11,500 #

stress of diagonals 11,500 x 1.363 = 15,700 # Tension or comp.

R.R = 1.2 "

$\frac{l}{r}$ = 200 for tension req'd r = $\frac{2.2 \times 12}{200} = 1.35$"

⌐ 5" x 3½ x ⅞" r = 1.60"

Max stress of strut 11,500 # R.R = 1.0 "

$\frac{l}{r}$ = 140 for comp. member req'd r = $\frac{33 \times 12}{140} = 2.83$"

⅞" thick equal ⅞ r = $\frac{16.5 \times 12}{140} = 1.42$"

⌐ 5 x 3½ x 9/16 r = 1.45" 4 x 10.4 = 41.6 #

Estimating steel wt for upper lateral system

Strut	7 @ 80" x 33 = 2640 x 7 =	18,480 #	for 2L 5 x 3½ x ⅞ Lace
Lateral	6 x 4 x 25# x 22.5 = 552.5 x 24 =	13,260 #	
		31,980 # ÷ 2,240 =	14.3 ton
		31,980 ÷ (2 x 183') =	87 #/l.c. ft

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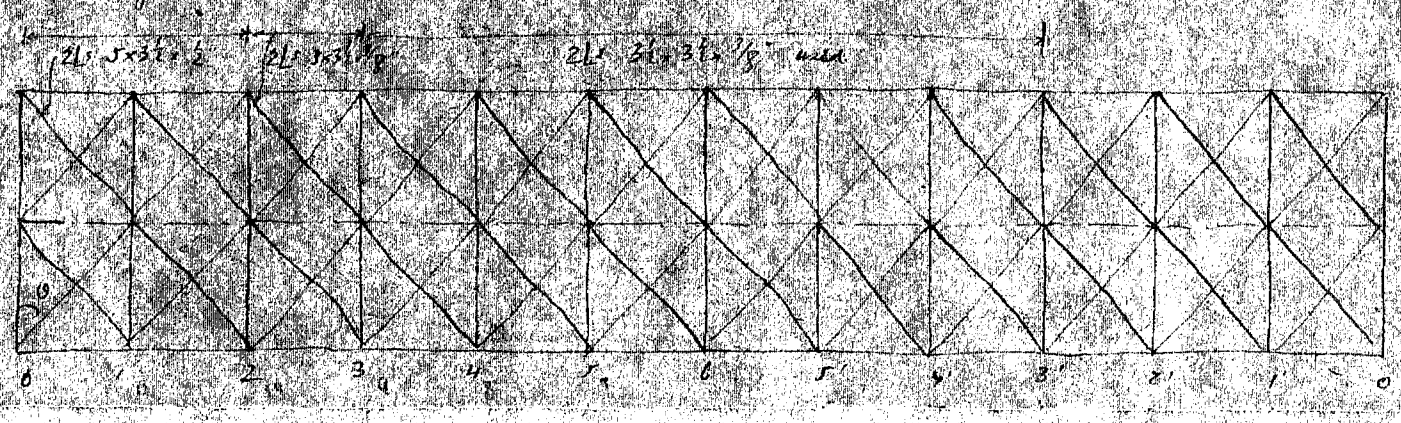
CALCULATIONS FOR 4/13 Bridge for city of Tokyo

☐ Bottom Lateral system

Skeleton diagram



Panel dead load 76.000 × 0.4 = 30,400
Reaction = 30,400 × 11 = 334,400
sec θ = 1.363 17,000 × 1.8 = 30,600
6,010 × 1.8 = 10,800

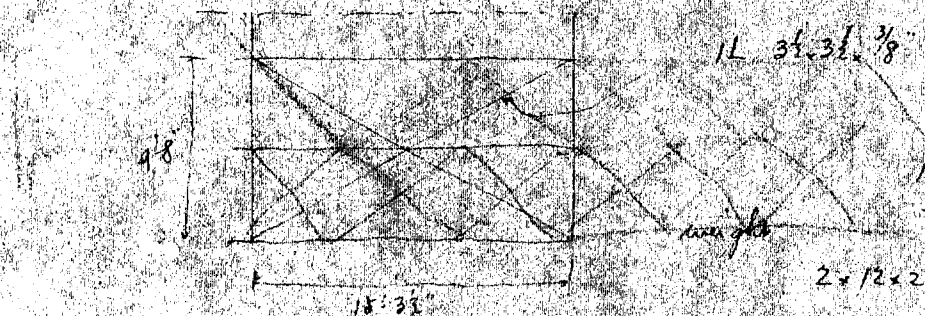
Panel shear

Panel	0	1	2	3	4	5	6
shear	335,000	274,000	213,000	154,000	91,200	30,400	
diagonal stress	457,000	374,000	290,000	210,000	124,000	41,500	
for one member	227,000	186,000	145,000	103,200	62,000	20,700	
Section Req'd.	7.45	6.07	4.74	3.38	2.02	0.68	
Section used	2L 6×6×½	2L 6×6×⅞	2L 5×5×⅞	2L 5×3½×⅝	2L 5×3½×⅜	2L 5×3½×⅜	do.
Gross area	11.5	8.72	7.22	6.1	6.1	6.1	do
No of rivets (7/8")	21	18	14	10	6	6	6

Stringer and floor system

Truss (including shoe) 162.0
upper Lateral 14.3
Bottom Lateral 18.4
~~384.7~~
292.5 ton Say 300. tons

Bottom Lateral under foot-way.



1L 3½×3½×⅜" used

9'-8" → 93.4444
15'3½ → 233.8351
327.2795 → 18'-1" ac.

2 × 12 × 2 @ 8.5 × 18 = 7,344#
details 31,600
105,00 #

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1st and dead load 76,000 × 3 × 48 = 14,000

Reaction = 14,000 × 11 = 154,000

Per θ = 1.363

Panel Shear

Panel	0	1	2	3	4	5	6
Shear	154,000	126,000	98,000	70,000	42,000	14,000	
diagonal stress	210,000	172,000	134,000	95,400	57,200	19,100	
for one member	105,000	86,000	67,000	47,700	28,600	9,600	
Sect'l Req'd	6.25	5.16	4.0	2.8	1.7	0.6	
Section used	2L 4×3½×½	Same	Same	2L 3½×3½×⅜			
Gross area	8.0	8.0	8.0	4.96			
No of rivets (⅞"ø)	18	18	12	8	6		

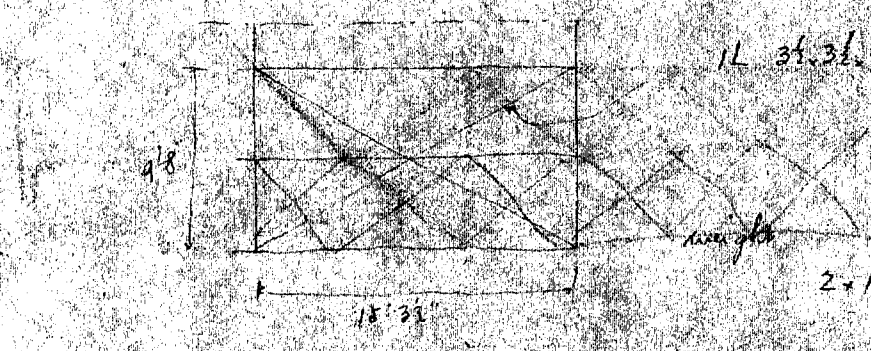
☐ Approx. weight of Bottom Laterals.

2×16 @	8"×3½×½×22.5	13.6×22.5×3.2 =	9800
2×8	8"×3½×⅜×22.5	10.4×22.5×16 =	2800
2×24	3½×3½×⅜×22.1	8.5×22.5×48 =	9200
			21800 #
		Gusset etc	8200
			30000 # ÷ 183.5×2 = 82.0 #/Lin ft
		Laterals for Bracket	10500
weight	41,500 / 30,000 ÷ 2240 =	43.4	40500 ÷ 183.5×2 = 110.0 #/Lin ft
		18.2 ton	

☐ Total weight of metal

Stringer and floor Beams	98.0 ton
Truss (including shoe)	162.0
upper Lateral	14.3
Botton Lateral	18.2
	292.5 ton
	Say 300 tons

Botton Lateral under foot-way.



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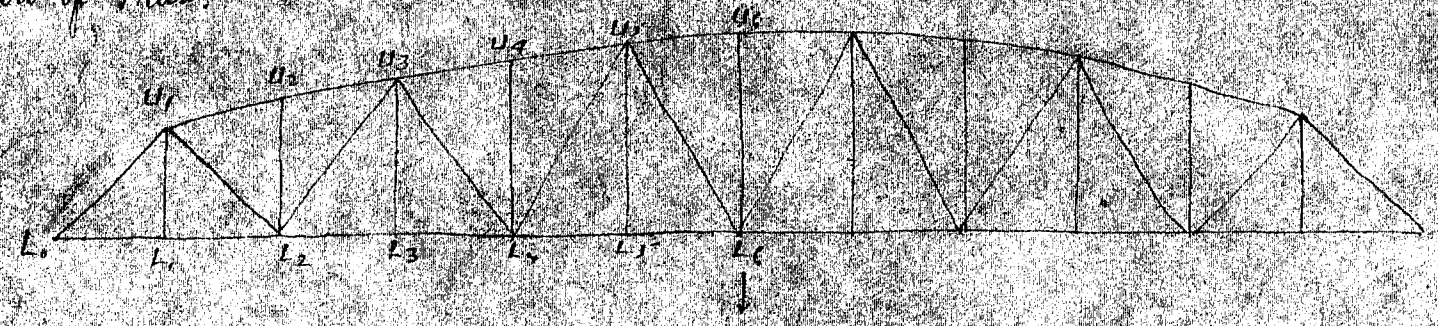
CALCULATIONS FOR 42-B Bridge for city of Tokyo

☐ Deflection of Truss.



Members	Length L (in)	Gr. Area A (sq. in)	Stresses S 1000(lbs)	Deformation $E\Delta L = \frac{SL}{A} 1000$ (lbs/in)	$\bar{S}_6$	$\bar{S}_6 \frac{SL}{A}$
L_0-U_1	21.63	71.75	− 7.806	−2.920	− 0.710	2.07
U_1-U_2	16.15	"	− 8.268	− 2.240	−0.827	1.85
U_2-U_3	25.82	"	− 8.100	− 2.090	−0.810	1.73
U_3-U_4	15.56	83.75	−9.618	− 2.200	−1.202	2.57
U_4-U_5	15.38	"	−9.998	− 2.090	−1.187	2.48
U_5-U_6	15.30	"	− 9.828	− 2.150	−1.638	3.52
						Σ = 28.44
L_0-L_1	15.30	58.50	5.708	1.770	0.570	0.92
L_1-L_2	15.30	59.50	5.703	1.770	0.518	0.92
L_2-L_3	15.29	71.50	8.884	2.090	0.888	1.86
L_3-L_4	15.29	71.50	8.884	2.090	0.888	1.86
L_4-L_5	15.29	83.50	10.318	2.140	1.217	2.60
L_5-L_6	15.29	83.50	10.318	2.140	1.217	2.60
						Σ = 21.52
U_1-L_2	20.93	24.67	2.894	2.950	0.360	1.06
L_2-U_3	28.16	24.67	−1.950	−2.670	−0.377	1.01
U_3-L_4	27.61	13.35	1.061	2.290	0.353	0.81
L_4-U_5	31.56	13.81	−0.602	−1.490	−0.435	0.65
U_5-L_6	31.40	14.35	0.182	0.370	0.507	0.19
						Σ = 3.72
U_1-L_1	14.76	17.05	1.000	1.040	0	0
U_2-L_2	19.53	"	0.586	0.810	0.039	0.005
U_3-L_3	23.23	"	1.000	1.640	0	0
U_4-L_4	25.89	"	0.538	1.070	0.090	0.096
U_5-L_5	27.47	"	1.000	1.940	0	0
* U_6-L_6	28.00	"	0.747	1.470	0.124	0.0382
						Σ = 0.38
					$\Sigma \bar{S}_6 \frac{SL}{A}$ =	37.78

Deflection for unit load applied $\frac{37.78}{130.000.000} = 0.000001926$

Deflection for dead load 76,000# applied = $0.000001926 \times 76.000 = 1.463$" say $1\frac{1}{2}$"

for live load 38,000 " $= 0.732$ $\frac{3}{4}$"

2.195" = $2\frac{1}{4}$"

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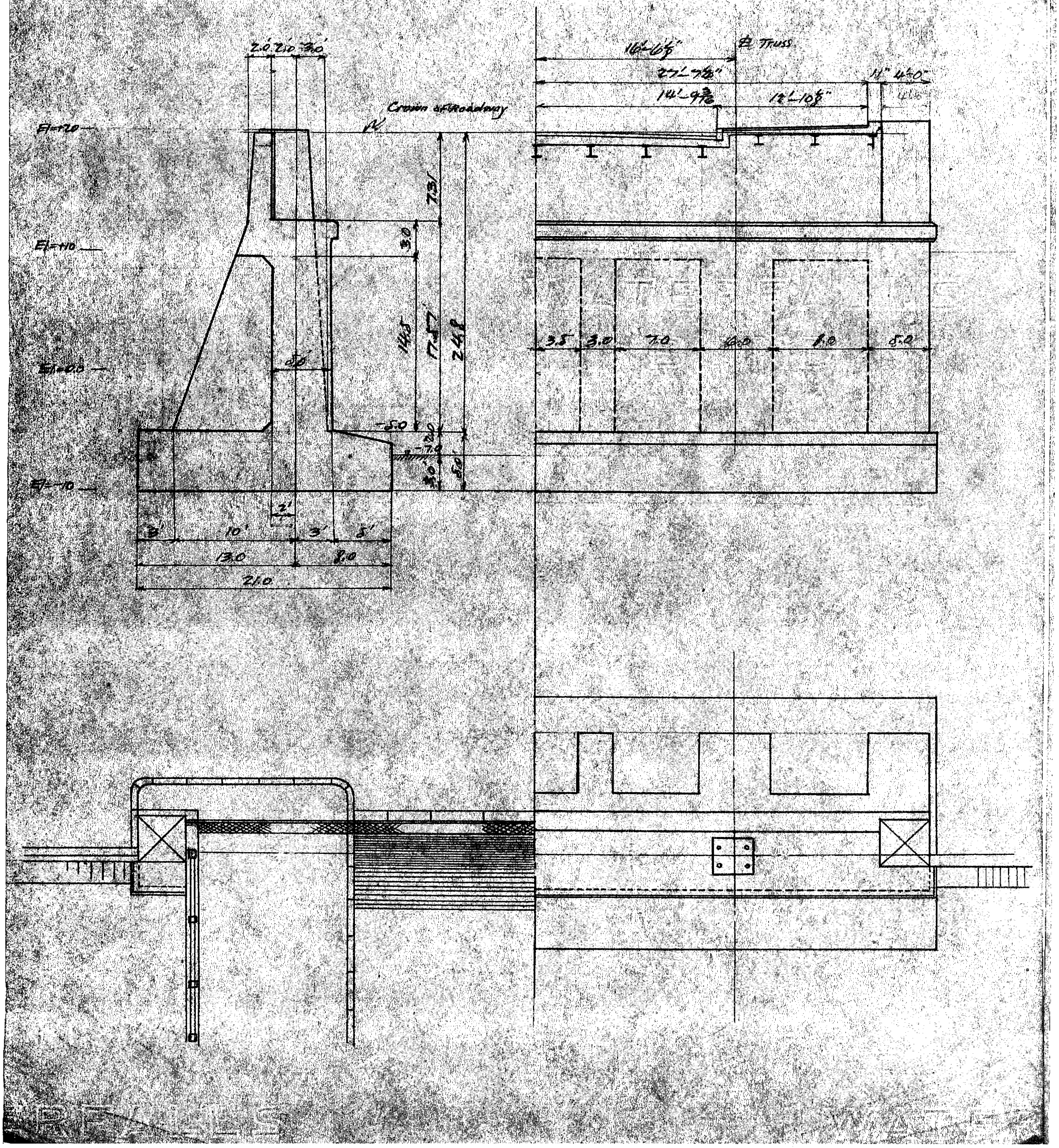
CALCULATIONS FOR

42 B Bridge for City of Tokyo.
Design of Abutment.
Max. Load on Shoe.

Dead Panel Load 6 @ 76,000 = 456,000#
Live " " 6½ @ 38,000 = 247,000
703,000#
Weight of shoe etc say 7,000
710,000#

Total dead load = 463,000#
Earthquake thrust on shoe
= 463,000 × 0.4 × ¾ = 138,900 say 139,000#



増田淳氏関係資料
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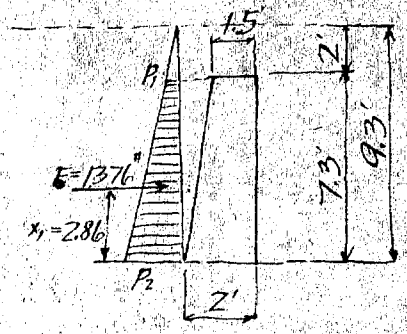
CALCULATIONS FOR

42 B Bridge for City of Tokyo.
Parapet wall.

Case 1. Stability at normal case. 2 ft. Surcharge due to live load assumed.

Unit earth pressure $p_1 = 100 \times \frac{2}{3} = 67$ #/□'
do $p_2 = 100 \times \frac{9.3}{3} = 310$ "

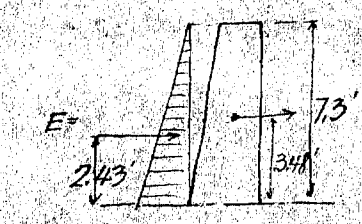


Earth pressure $E = \frac{67+310}{2} \times 7.3 = 1{,}376$ # per ft strip of wall.
Moment at bottom of wall = 1376 × 2.86 = 3935 '#
Effective depth of wall say 22"
Steel area reqd = $\frac{3935 \times 12}{17{,}000 \times \frac{7}{8} \times 22} = 0.1443$" per ft strip
use 1/2"φ bar 1' spacing = 0.1963" per ft strip.
Steel ratio $p = \frac{0.1963}{12 \times 22} = .00075$

C.g. of Pressure diagram
$x_1 = \frac{d}{3} \frac{b+2b_1}{b+b_1} = \frac{7.3}{3} \frac{310+2\cdot67}{310+67}$
$= 2.86'$

$K = .145$ $j = .95$
$f_s = \frac{3935 \times 12}{.1963 \times .95 \times 22} = 11{,}510$ #/□" OK
$f_c = \frac{11510 \times .145}{15(1-.145)} = 130$ #/□" OK
unit shear $v = \frac{1376}{12 \times .95 \times 22} = 5.5$ #/□" OK
unit bond $u = \frac{1376}{1.571 \times .95 \times 22} = 42$ #/□" OK

Case 2. Stability during Earthquake. Earthquake coeff. $0.4 \times \frac{3}{4} = 0.3$.
Earth pressure during earthquake.
$0.662 \frac{wh^2}{2}$ Inclination of resultant to horiz. line $\theta = 27°-5'$



$E = 0.662 \times \frac{100 \times 7.3^2}{2} = 1{,}765$ #
weight of wall = $\frac{1.5+2.0}{2} \times 7.3 \times 150 = 1915$ #

Moment		Shear
due to earth pressure	= 1,765 × 2.43 = 4290	1,765
" wt of wall	= 1915 × .3 × 3.48 = 2000	575
	6290 '#	2,340 #

$f_s = \frac{6290 \times 12}{.1963 \times .95 \times 22} = 18{,}400$ #/□" OK (allowable f_s = 30600 #/□")
$f_c = \frac{18400 \times .145}{15(1-.145)} = 207$ #/□" OK (" f_c = 1,182)
Shear $v = \frac{2340}{12 \times .95 \times 22} = 9.3$ " " (" v = 103)
bond $u = \frac{2340}{1.571 \times .95 \times 22} = 71.3$ " " (" u = 153)

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CALCULATIONS FOR

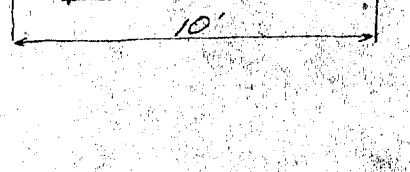
42B Bridge for City of Tokyo.

Curtain wall. Span length = 10.0' say thickness of wall 5.0' effective depth say 57"

Case 1. Stability at normal state.

Height of wall Earth fill at bottom of wall = 26.8'

Earth pressure at bottom of wall = $\frac{100 \times 26.8}{3} = 893$ #/ft' (including 2' Surcharge.)



Bending moment per ft strip = $\frac{893 \times 4 \times 10}{2} - \frac{893 \times 4^2}{2} = 10{,}150$ '#

for continuity of slab $10150 \times .8 = 8{,}120$ '#

Max. End shear = $893 \times 4 = 3{,}570$ #

Steel area req'd = $\frac{8120 \times 12}{17000 \times \frac{7}{8} \times 57} = 0.115$ □" per ft strip.

Steel ratio $p = \frac{.295}{12 \times 57} = .00043$ Use 3/4" ⌀ bar 1'-6" spacing = 0.295 □"

$K = \sqrt{2pn + (pn)^2} - pn$

$= \sqrt{30 \times .00043 + .0065^2} - .0065$

$= .109$

$f_s = \frac{8120 \times 12}{.295 \times .964 \times 57} = 6010$ #/□" OK.

$f_c = \frac{6010 \times .109}{15(1 - .109)} = 49$ #/□" OK.

$j = 1 - \frac{K}{3} = 1 - \frac{.109}{3} = .964$ Unit Shear $v = \frac{3570}{12 \times .964 \times 57} = 5$ #/□" OK

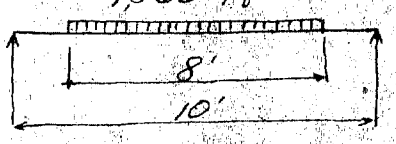
Unit Bond $u = \frac{3570}{2.356 \div 1.5 \times .964 \times 57} = 41.4$ #/□" OK.

Case 2. Stability during Earthquake $K = 0.4 \times \frac{3}{4} = 0.3$ assumed.

Height of earth fill = 24.8'

Earth pressure at bottom of wall = $.662 \times 100 \times 24.8 = 1641$ #/□'

Seismic force due to weight of wall = $5 \times 150 \times .3 = \frac{225}{1{,}866}$ #/ft of span.



Bending moment per ft strip $\frac{1866 \times 4 \times 10}{2} - \frac{1866 \times 4^2}{2} = 22{,}380$ '# $= 17{,}900$ '#

for continuity of slab $22380 \times .8$

Max end shear $= 1{,}866 \times 4 = 7{,}470$ #

$f_s = \frac{17900 \times 12}{0.295 \times .964 \times 57} = 13{,}250$ #/□" OK.

$f_c = \frac{13250 \times .109}{15(1 - .109)} = 108$ #/□" OK.

Unit Shear $v = \frac{7470}{12 \times .964 \times 57} = 11.3$ #/□" OK.

Unit bond $u = \frac{7470}{2.356 \div 1.5 \times .964 \times 57} = 86.4$ #/□" OK.

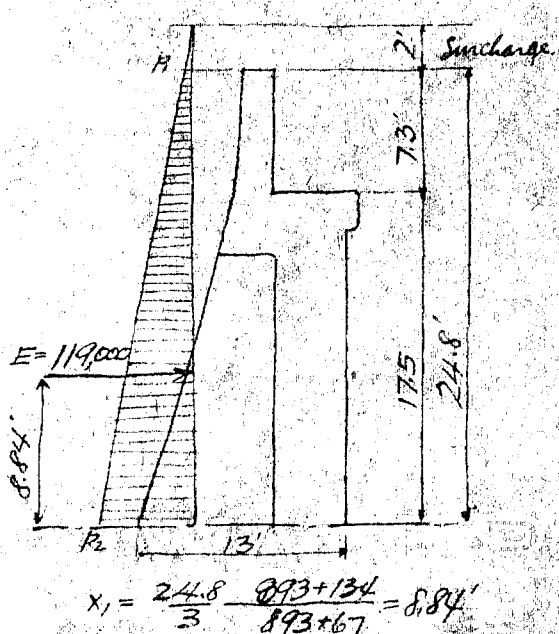
Note.

Use reinforcements on both sides of wall.

Use same details for all panels.

Counterfort walls at center.

Case 1. Stability at normal state: Spacing of counterfort wall = 10'



Earth pressure = $p_1 = \frac{100 \times 2}{3} = 67$ #/□' $p_2 = \frac{100 \times 26.8}{3} = 893$ #/□'

Earth pressure on counterfort = $\frac{67 + 893}{2} \times 24.8 \times 10 = 119{,}000$ #

Moment at bottom = $119000 \times 8.84 = 1{,}051{,}000$ '#

Shear = 119,000 #

Effective depth = $13.0 \times 12 - \frac{4}{8} = 152$"

Steel req'd = $\frac{1{,}051{,}000 \times 12}{17000 \times \frac{7}{8} \times 152} = 5.58$ □"

Use 8-1"⌀ = 6.28 □"

10-1"⌀ = 7.854 □" Revised 2-8-25.

$x_1 = \frac{24.8}{3} \frac{893 + 134}{893 \times 67} = 8.84'$

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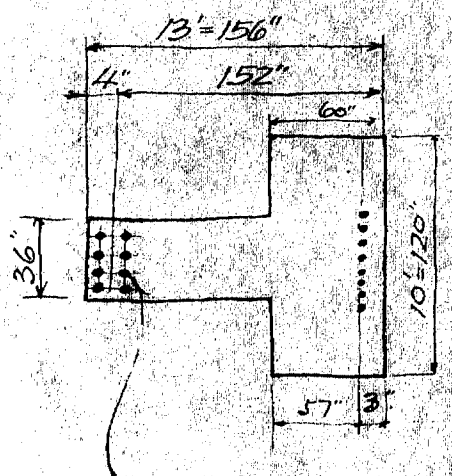
CALCULATIONS FOR

42 B. Bridge for Tokyo City



10-1"ϕ revised 2-8-25

Design as a double reinforced T-Beam.

Steel ratio $p=p'=\frac{6.28}{120\times152}=0.000344$

$\Delta=\frac{t}{d}=\frac{60}{152}=.395$. $\frac{d'}{d}=\frac{3}{152}=.02$

$$K=\frac{p+p'(\frac{d'}{d})+\frac{\Delta^2}{2n}}{p+p'+\frac{\Delta}{n}}=\frac{p(1+\frac{d'}{d}+\frac{\Delta^2}{2n}}{2p+\frac{\Delta}{n}}=\frac{.000344\times1.02+\frac{.395^2}{30}}{.000688+\frac{.395}{15}}=0.205$$

$Kd=152\times.205=31.1"$

Neutral axis in the flange

Design as a double reinforced rectangular beam.

$$K=\sqrt{2n(p+p'\tfrac{d'}{d})+n^2(p+p')^2}-n(p+p')=\sqrt{2np(1+\tfrac{d'}{d})+4n^2p^2}-2np.$$

$$=\sqrt{30\times.000344\times1.02+4\times225\times.000344^2}-30\times.000344=.098$$

$$j=\frac{K^2(1-\frac{K}{3})+2p'n(K-\frac{d'}{d})(1-\frac{d'}{d})}{K^2+2p'n(K-\frac{d'}{d})}=\frac{.098^2(1-\frac{.098}{3})+2\times.000344\times15(.098-.02)(1-.02}{.098^2+2\times.000344\times15(.098-.02)}$$

$=.97$

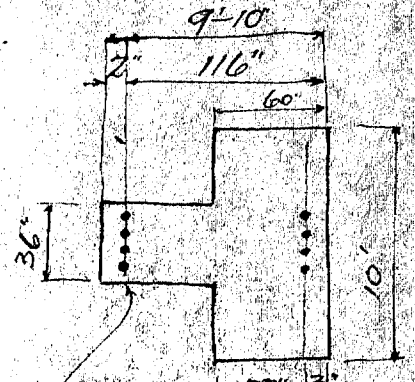
$f_s=\frac{1051.000\times12}{6.28\times.97\times152}=13630$ #/□" OK.

$f_c=\frac{13630\times.098}{15(1-.098)}=99$ #/□" OK

$v=\frac{119,000}{36\times.97\times152}=22.4$ #/□" OK

$u=\frac{119.000}{3.142\times8\times.97\times152}=32.1$ #/□" OK.

Section at 10 ft above bottom of wall.



Earth pressure $p_1=67$#, $p_2=\frac{100\times16.8}{3}=560$#

Earth pressure on counterfort $=\frac{67+560}{2}\times14.8\times10=46,500$ #

$x'=\frac{14.8}{3}\cdot\frac{560+134}{560+67}=5.45'$

Moment $=46,500\times5.45=253,500$ '#

Steel area req'd $=\frac{253500\times12}{17000\times\frac{7}{8}\times116}=1.77$ □"

use 4-1"ϕ bars $=3.14$ □"

5-1"ϕ " $=3.93$ □" Revised 2-8-25

Case 2. Stability during earthquake. at bottom section. K=0.3, toward river side

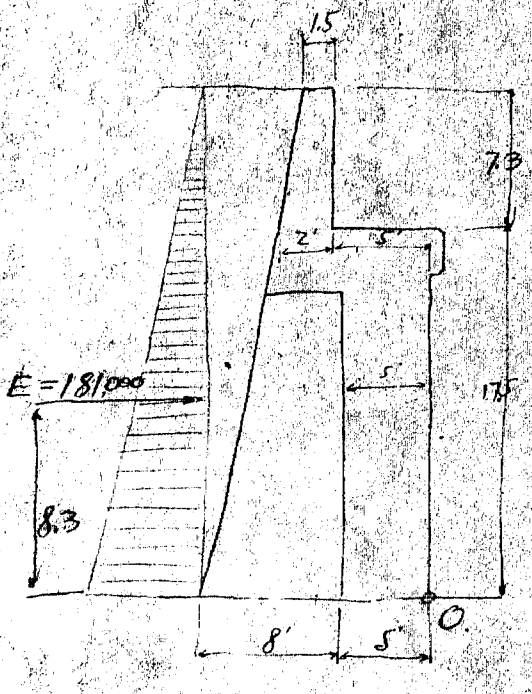
Height of earth fill $=24.8'$

Earth pressure $=\frac{.662\times100\times24.8^2}{2}\times10'=203500$ #

Hor. component $=.89\times203,500=181,000$#

Vert. " $=.455\times203,500=92,600$

taking moment about O.



	Weight	Coeff. K	Arm		Moment abt O.
Parapet wall	$\frac{1.5+2.0}{2}\times7.3\times10@150$ = 19150#	× .3	× 21.00'	=	120,500 '#
Hor. slab	$3\times7.6\times10@150$ = 34200#	× .3	× 16.00	=	164000
Curtain wall	$5\times14.5\times10@150$ = 108,700#	× "	× 7.25	=	236,200
Web of counterfort	$\frac{3+8}{2}\times3\times14.5@150$ = 35,900#	× "	× 6.15	=	66,200
Earth between counterforts	$\frac{3+8}{2}\times14.5\times7@100$ = 55,800#	× "	× 6.15	=	103,000
Earth behind (approx)	$\frac{8\times24.8}{2}\times10@100$ = 99,200#	× "	× 16.5	=	491,000
Hor. earth pressure	181,000	× 8.3			1503,000
	ΣH = 160,185#		ΣM_b		2,683,900 '#

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CALCULATIONS FOR

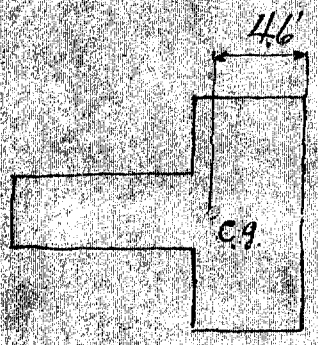
4Z-B Bridge for Tokyo City

Moment due to vertical forces

parapet wall	19150 #	×	−5.85	= −112,000
Hor. Slab	34,200	×	−3.8	= −130,000
Curtain wall	108,700	×	−2.5	= −270,000
Web of counterfort	35,900	×	−8.0	= −287,000
Earth on counterfort	$55800\times\frac{2}{10}=29800$	×	−10.3	= −307,000
Friction of earth on curtain wall	$55800\times.4=22300$	×	−5.0	= −111,500
" " " parapet	$\frac{5+5}{2}\times7.3\times100\times.4=19000$	×	−6.75	= −128,000
	$\Sigma V = 269,050$			$\Sigma M_V = -1,345,500$ '#

Summary of moment about O

Moment due to Hor. forces = 2,683,900
" " Vert. " = −1,345,500
1,338,400 '#



Distance of Resultant force from O = $\frac{1338400}{269050}=4.97'$

Center of gravity of bottom section.

10×5 = 250	× 2.5	=	125.0
3×8 = 24	× 9.0	=	216.0
area 74	moment	341.0	÷74 = 4.6'

Eccentricity = 4.6+4.97 = 9.57'

Bending moment for bottom section = 269050×9.57 = 2,575,000 '#

$f_s=\frac{2575000\times12}{6.28\times.97\times152}=33350$ #/□" Over Stress.

Use 10−1" ϕ bars = 7.854 □"

$f_s=\frac{2575000\times12}{7854\times.97\times152}=26,700$ #/□" OK

$f_c=\frac{26700\times.098}{15(1-.098)}=193.5$ #/□" OK

$v=\frac{160185}{36\times.97\times152}=34.5$ #/□" OK

$u=\frac{160185}{3.142\times10\times.97\times152}=30.2$ #/□" OK

Case 3 Stability during earthquake at bottom section. earthquake toward bankside.

	moment due to Hor forces		moment due to vert forces		
Parapet wall	19,150×.3×21.0	= 120,500	19150 × 5.85	=	112,000
Hor. Slab	34,200×.3×16.0	= 164,000	34,200 × 3.8	=	130,000
Curtain wall	108,700×.3×7.25	= 236,200	108,700 × 2.5	=	270,000
Web of counterfort	35,900×.3×6.15	= 66,200	35900 × 8.0	=	287,000
	$\Sigma H = 59385$ #	$\Sigma M_h = 586900$	$\Sigma V = 197,950$	$\Sigma M_v = 799,000$ '#	

$\Sigma M_h = 586900$

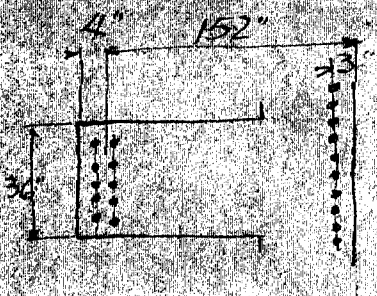
$\Sigma M = 1,385,900$

Distance of resultant force from front face of wall = $\frac{1,385,900}{197,950}=7.00'$

Eccentricity = 7.0−4.6 = 2.4'

Bending moment = 197,950×2.4 = 477,000 '#

Steel ratio $p=p'=\frac{7.854}{36\times152}=.00144$ $\quad d'/d=\frac{3}{152}=.02$



$K=\sqrt{2np(1+\frac{d'}{d})+4n^2p^2}-2pn=\sqrt{30\times.00144\times1.02+4\times225\times.00144^2}-30\times.00144$

$=.163$

$j=\frac{K^2(1-\frac{K}{3})+2p'n(K-\frac{d'}{d})(1-\frac{d'}{d})}{K^2+2p'n(K-\frac{d'}{d})}=\frac{.163^2\times.946+30\times.00144\times.163\times.98}{.163^2+30\times.00144\times.163}=.954$

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CALCULATIONS FOR

42B Bridge for City of Tokyo.

$$f_s = \frac{477000 \times 12}{7.854 \times .954 \times 152^2} = 5020\ \#/\square'' \quad OK$$

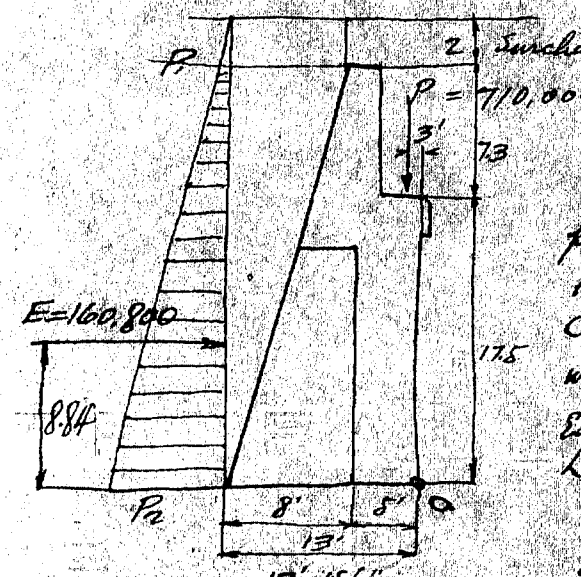
$$f_c = \frac{5020 \times .168}{15(1-.163)} = 65.2\ \#/\square'' \quad OK$$

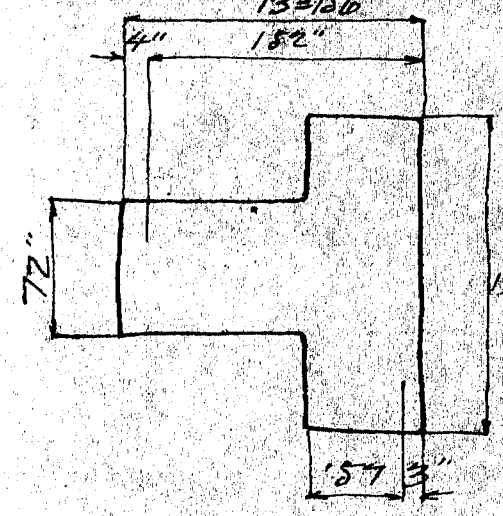
$$v = \frac{59385}{36 \times .954 \times 152} = 11.4\ \#/\square'' \quad OK$$

$$u = \frac{59385}{3.142 \times 10 \times .954 \times 15^2} = 13\ \#/\square'' \quad OK$$

Counterfort under shoe

Case 1 Stability at normal state





Earth pres. $p_1 = 67\ \#/\square'$ $p_2 = 893\ \#/\square'$

Earth pres on counterfort $= \frac{67+893}{2} \times 24.8 \times 13.5 = 160,800\#$

moment about O

		weight	arm	moment
parapet wall	$\frac{1.5+2.0}{2} \times 7.3 \times 13.5 @ 150$	= 25,850	× 5.85	= 151,300
Hor slab	$3 \times 7.6 \times 13.5 @ 150$	= 46,700	× 3.8	= 178,600
Curtain wall	$5 \times 14.5 \times 13.5 @ 150$	= 146,800	× 2.5	= 367,000
web of counterfort	$\frac{3+8}{2} \times 6 \times 14.5 @ 150$	= 71,800	× 8.0	= 574,000
Earth on counterfort	$\frac{8 \times 24.8}{2} \times 6 @ 100$	= 59,400	× 10.3	= 612,000
Load on shoe		710,000	× 3.0	= 2,130,000
	$\Sigma V =$	1,060,050		4,009,900
Earth pressure	$(\Sigma H) =$	160,800	× 8.84	= −1,422,000
	$\Sigma M =$			2,587,900 #'

Distance of resultant force from point O

$$= \frac{2,587,900}{1,060,050} = 2.44'$$

Eccentricity $e = 6.5 - 2.44 = 4.06'$

Bending moment $= 1,060,050 \times 4.06 = 4,305,000\ '\#$

Steel req'd for simple bending $\frac{4,305,000 \times 12}{17,000 \times \frac{7}{8} \times 152} = 22.86\ \square''$

Use 20-1"ϕ $= 15.71\ \square''$

Steel ratio $p = \frac{15.71}{72 \times 156} = .0014$ $p_0 = 2p = .0028$

$$\frac{e}{h} = \frac{4.06}{13} = .312$$

$k = .665$ $L = .107$

$$f_c = \frac{M}{Lbh^2} = \frac{4,305,000 \times 12}{.107 \times 72 \times 156^2} = 275.5\ \#/\square'' \quad OK$$

$$f_s = nf_c\left(\frac{d}{kh} - 1\right) = 15 \times 275.5\left(\frac{152}{.665 \times 156} - 1\right) = 1,920\ \#/\square'' \quad OK$$

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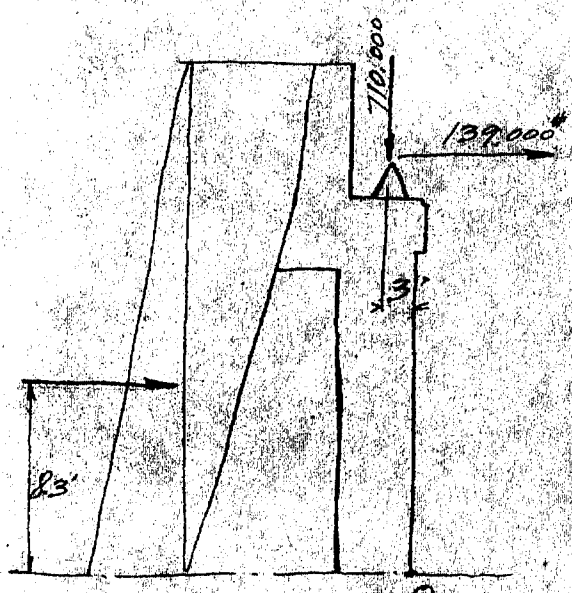
CALCULATIONS FOR

42 B Bridge for City of Tokyo
Case 2. Stability during earthquake toward river side



Taking moment about O

Moment due to Vert. forces

parapet wall	25850	× 5.85	=	151,100'#
Hor slab	46.200	× 3.80	=	175,500
Curtain wall	146,800	× 2.50	=	367,000
Web of counterfort	71.800	× 8.00	=	574,000
Earth on counterfort	59.400	× 10.30	=	612,000
Friction of earth on curtain wall	23,900	× 5.0	=	119,500
" parapet wall	25,600	× 6.75	=	172,700
Load on shoe	710.000	× 3.0	=	2,130,000
	ΣV = 1,109,550#.			ΣMV = 4,301,800'#

Earth pressure during earthquake
Height of earth fill = 24.8'

$$\text{Earth press.} = \frac{.642 \times 100 \times 24.8^2}{2} \times 13.5 = 275{,}000\#$$

Hor. component .89 × 275,000 = 244,700#
Vert " .455 × 275,000 = 125,100#

Moment due to Hor. forces

parapet wall	25,850 × .3 = 7,760	× 21.0	=	16,300
Hor slab	46,200 × .3 = 13,850	× 16.0	=	221,700
Curtain wall	146,800 × .3 = 44,100	× 7.25	=	319,000
web of Counterfort	71,800 × .3 = 21,550	× 6.15	=	132,500
Earth between counterfort	89,800 × .3 = 17,950	× 6.15	=	110,300
" behind	134,000 × .3 = 40,200	× 16.50	=	663,000
Earth pressure	244,700	× 8.3	=	2,025,000
Load on shoe	139,000	× 19.7	=	2740000
	ΣH = 529,110			ΣMH = 6,374,500'#

Summary of Moment about O
Moment due to Hor forces = + 6,374,500
" " " Vert " = − 4,301,800
2,072,700'#

Distance of resultant face from face of wall
$= \frac{2072700}{1109550} = 1.87'$

Eccentricity = 6.5 + 1.87 = 8.37'
Bending moment at bottom section = 1,109,550 × 8.37 = 9,280,000'#

$p_0 = 0.0028$ $e/k = \frac{8.37}{13} = .644$
$k = .335$ $L = .093$

$$f_c = \frac{9280{,}000 \times 12}{.093 \times 72 \times 156^2} = 683\ \#/\text{in}^2 \quad \text{OK}$$

$$f_s = 15 \times 683 \left(\frac{156}{.335 \times 156} - 1\right) = 19{,}570\ \#/\text{in}^2 \quad \text{OK}$$

Case 3 Earthquake toward bank side
In this case moment is clearly smaller than for Case 2, use the same details as for Case 2.

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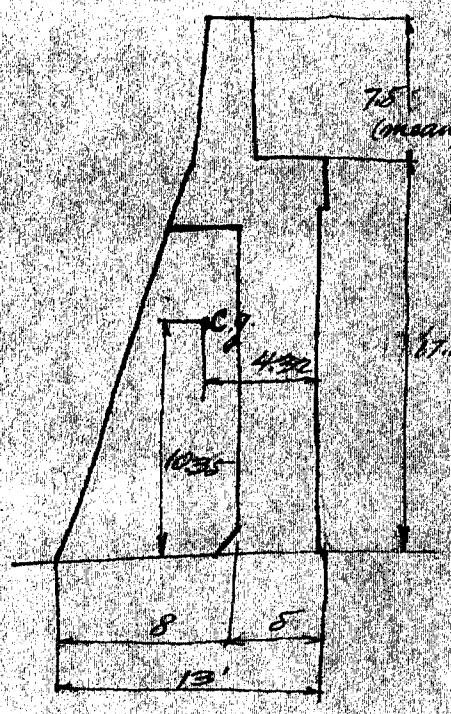
CALCULATIONS FOR

42 B Bridge for City of Tokyo.
Design of foundation



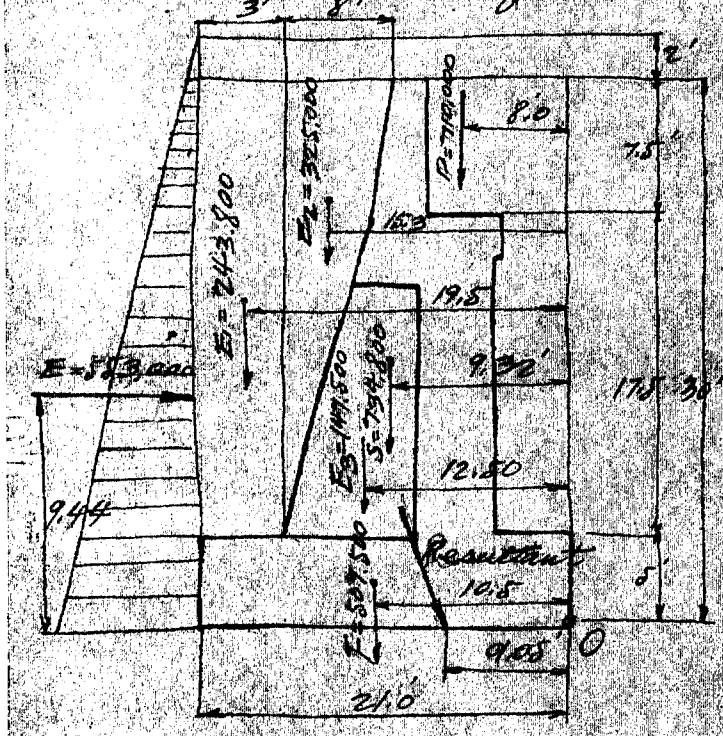
parapet wall	$\frac{1.5+2.5}{2}$ × 2.5 × 28.5 @ 150 =	56,100 × 22.1	= 1238,000
Hor. slab	3 × 7.6 × 28.5 @ 150 =	97,400 × 17.0	= 1,650,000
Col. under lamp post	4 × 4 × 7.5 @ 150 =	18,000 × 22.1	= 397,800
Light pedestal	3 × 3 × 5 @ 150 =	6,800 × 28.5	= 194,000
Curtain wall	5 × 18.5 × 32.5 @ 150 =	355,000 × 7.8	= 2,770,000
web of Counterfort	$\frac{2.5+1.5}{2}$ × 14.5 × 14 @ 150 =	168,000 × 6.6	= 1,110,000
		$\Sigma H = 701,300$	ΣM_H = 7,359,500 ft#

Vert. distance of c.g. above top of base
$= \frac{7,359,500}{701,300} = 10.35'$

parapet wall		56,100 × 8.85	= 328,000
Hor Slab		97,400 × 3.80	= 370,000
Col. under lamp post		18,000 × 4.0	= 72,000
Light pedestal		6,800 × 4.0	= 27,000
Curtain wall		355,000 × 2.50	= 888,000
Web of counterfort		168,000 × 8.00	= 1,344,000
		ΣV = 701,300	ΣM_V = 3,029,000 ft#

Hor distance of c.g. from face of curtain wall
$= \frac{3,029,000}{701,300} = 4.32'$

Case I. Stability at normal state Calculation for one half of abutment



Earth press. p_1 = 67 $p_2 = \frac{100 \times 32}{3}$ = 1,067
Earth press on abutment = $\frac{67+1067}{2} \times 30 \times 32.5$ = 553,000 ft#

Wt. of earth $E_1 = 3 \times 2.5 \times 32.5 @ 100$ = 243,800 ft#
" " $E_2 = \frac{8 \times 25}{2} \times 32.5 @ 100$ = 325,000 ft#
" " $E_3 = \frac{3+8}{2} \times 14.5 \times 185 @ 100$ = 147,500
weight of foundation 21 × 33 × 5 @ 150 = 509,800 ft#

Taking moment abt 0 (toe)

Loads

P = 718,000	×	8.0	= 5,640,000
S = 734,800	×	9.32	= 6,850,000
E_3 = 147,500	×	12.56	= 1,855,000
E_2 = 325,000	×	15.3	= 4,970,000
E_1 = 243,800	×	19.5	= 4,754,000
F = 509,800	×	10.5	= 5,345,000
2,670,600			29,414,000

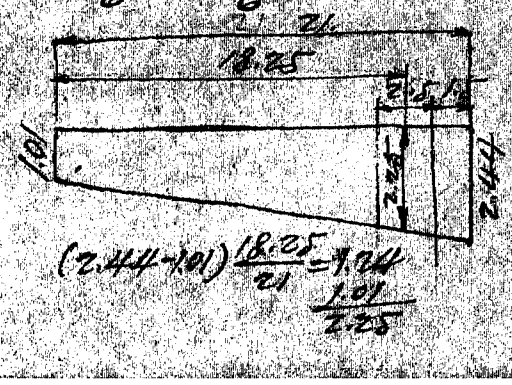
$X = \frac{30 \times (1067+67)}{3\ (1067+134)} = 9.44$

E 553,000 × 9.44 = −5,220,000
24,194,000 ft#

Dist. of resultant force from 0 = $\frac{24,194,000}{2,670,600} = 9.05'$

Eccentricity = 10.5 − 9.05 = 1.45
Moment = 2,670,600 × 1.45 = 3,870,000 ft#
Max toe pressure = $\frac{2670600}{21 \times 33} \pm \frac{3870000}{2,400} = 3860 \pm 1600$
= 5460 #/□' = 2.44 tons/□'

Section modulus of base
$= \frac{bh^2}{6} = \frac{33 \times 21^2}{6} = 2400$ (ft³)



Spacing of piles 2.5' on both directions at toe = 2260 = 1.01 "
Load on one pile = $\frac{2.44+2.45}{2} \times 2.5 \times 2.5$ = 14.65 tons
Use Piles 0.7' dia at tip 36' long Surface area of pile = 93.5 □'
Friction on pile surface = $\frac{14.65}{93.5} \times 2240$ = 351 #/□'

(2.44+1.01) $\frac{18.25}{21}$ = 1.44
$\frac{1.01}{2.25}$

Note: Base revised to 22' wide, for revised figure, See page 41

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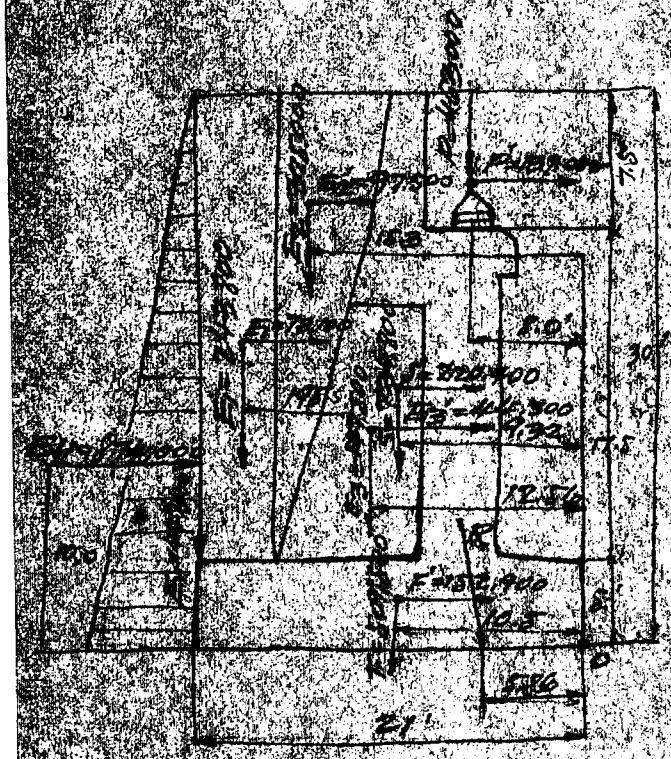
CALCULATIONS FOR

K2B Bridge for City of Tokyo

Case 2 Stability during earthquake toward river side



Earthpressure during earthquake

$E = \frac{0.166c \times 100 \times 30}{2} \times 33 = 982,000\#$

$E_V = 982,000 \times .455 = 447,000\#$

$E_H = 982,000 \times .89 = 874,000$

Taking moment about O (toe)

Moment due to the forces

$P' = 187,000$	× 25.7	=	3,570,000
$S' = 224,400$	× 15.35	=	3,440,000
$E_3' = 44,300$	× 12.15	=	538,000
$E_2' = 97,500$	× 21.7	=	2,075,000
$E_1' = 73,100$	× 17.5	=	1,279,000
$F' = 152,900$	× 2.5	=	382,000
$E_h = 874,000$	× 10.0	=	8,740,000
$\Sigma H = 1,661,200$	ΣM_0		20,024,000 '#

Moment due to Vert. Loads

$P = 463000$	× − 8.0	=	− 3,703,000
$S = 734,800$	× − 9.32	=	− 6,849,000
$E_3 = 149,500$	× − 12.56	=	− 1,854,000
$E_2 = 325,000$	× − 13.3	=	− 4,970,000
$E_1 = 249,800$	× − 19.5	=	− 4,750,000
$F = 509,500$	× − 10.5	=	− 5,342,000
$E_V = 447,000$	× − 21.0	=	− 9,390,000
$\Sigma V = 2,870,600$	ΣM_V		− 36858,000 '#

Total moment = −16834,000

Distance of resultant force from O

$= \frac{-16,834,000}{2,870,600} = -5.86'$

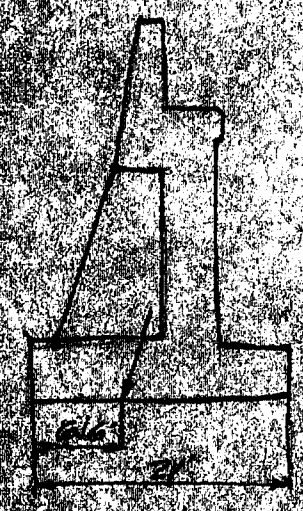
Eccentricity = 10.5 − 5.86 = 4.64' > $\frac{21}{6} = 3.5$

Neglecting tension

Max. toe press. $= \frac{2 \times 2,870,600}{3 \times 5.86 \times 33} = 9900\#/0' = 4.42^{tons}/0'$

Load on one pile during earthquake = 4.42 × 6.25 = 27.6 tons

Case 3 Stability during earthquake toward back side



Taking moment about O

Moment due to hor. forces

due to P'	3,570,000	See Case 1
" S'	3,440,000	
" F'	382,000	
ΣM_H	−7,392,000 '#	

Moment due to Vert. forces

due to P	−3703,000
S	− 6849,000
E_3	− 1,854,000
E_2	− 4,970,000
E_1	− 4,750,000
F	− 5,342,000
$\Sigma V = 2423600$ ΣM_V	−27,468,000

Total moment = −34860,000 '#

Distance of resultant force from O $= \frac{34,860,000}{2423600} = 14.4'$

Eccentricity = 14.4 − 10.5 = 3.9'

Neglecting tension

Max. load press. $= \frac{2 \times 2,423,600}{3 \times 6.6 \times 33} = 7420\#/0' = 3.32^{tons}/0'$

Load on one pile during earthquake = 3.32 × 6.25 = 24.9 tons

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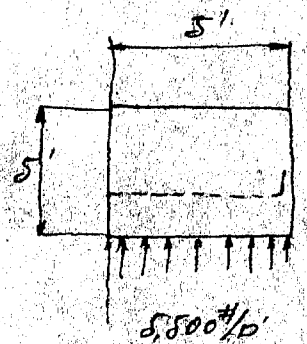
CALCULATIONS FOR

42 B Bridge for City of Tokyo

Design of base

Cantilever footing



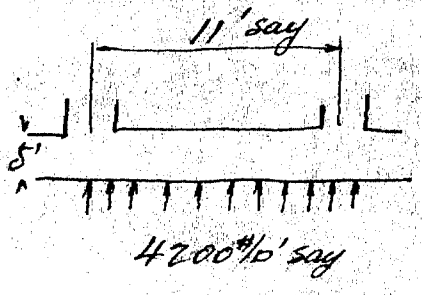
Upward pressure 5,500#/□'
weight of footing = 5×150 = 750 "
4,750#/□'

Moment of fixture = $\frac{4750\times5^2}{2}$ = 59,400'#/ft strip

Steel reqd = $\frac{59,400\times12}{17000\times\frac{7}{8}\times48"}$ = 1.0□"/ft strip

Use 2 - 7/8"ϕ bars = 1.20□"
Revised to 2 - 1"ϕ bars = 1.57□" ——2-8-29

Base between counterfort

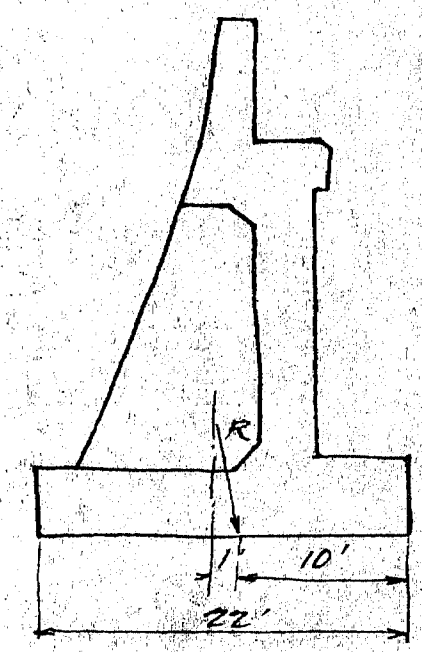


upward pressure 4200#/□' —— (max upward press during earthquake = 7,420#/□')
wt of base 750
3450#/□'

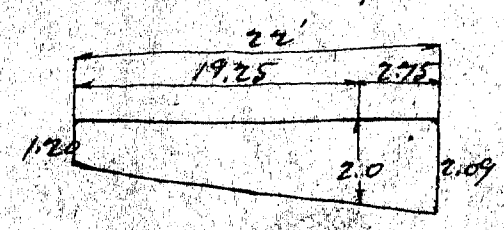
Moment = $\frac{3450\times11^2}{8}$ = 52,200'#

for continuity of base
moment = 52,200 × .8 = 41,800'#
Steel reqd = $\frac{41800\times12}{17000\times\frac{7}{8}\times48}$ = .70□"/ft strip
Use 1"ϕ bar 18 spacing = .785□"

Revised calcuration for the base Use 22' base.
Case 1. Stability at normal state



Overturning moment = 2,670,600'# Say Section modulus $\frac{33\times22^2}{6}$ = 2660
Max. toe pressure = $\frac{2,670,600}{22\times33} \pm \frac{2670600}{2660}$
= 3680 ± 1005 = 4685#/□' = 2.09 tons/□'
or = 2675 " = 1.20 "

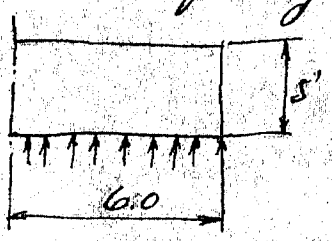


(2.09-1.2)$\frac{19.25}{22}$ = .80
1.20
2.00 say tons/□'

Pile Pile load on one pile = $\frac{2.09+2.0}{2}\times2.5\times2.5$ = 12.8 tons OK

friction on pile surface = $\frac{12.8\times2240}{93.5}$ = 307#/□'

Cantilever footing



Upward pressure = 2.09 tons/□' = 4680#/□'
Weight of footing = 5×150 = 750
3930#/□'
Moment of fixture = $\frac{3930\times6^2}{2}$ = 70,700'#
Steel req'd = $\frac{70700\times12}{17000\times\frac{7}{8}\times48}$ = 1.19□"/ft strip
Use 2 - 1"ϕ bars = 1.57□"/ft strip
Max. end shear = 3,930 × 6 = 23600#
Unit shear = $\frac{23600}{12\times.918\times48}$ = 44.7#/□' OK
Unit bond = $\frac{23600}{2\times3.142\times.918\times48}$ = 85.6#/□' OK

$p = \frac{1.57}{12\times48}$ = .00273
K = .244
j = .918

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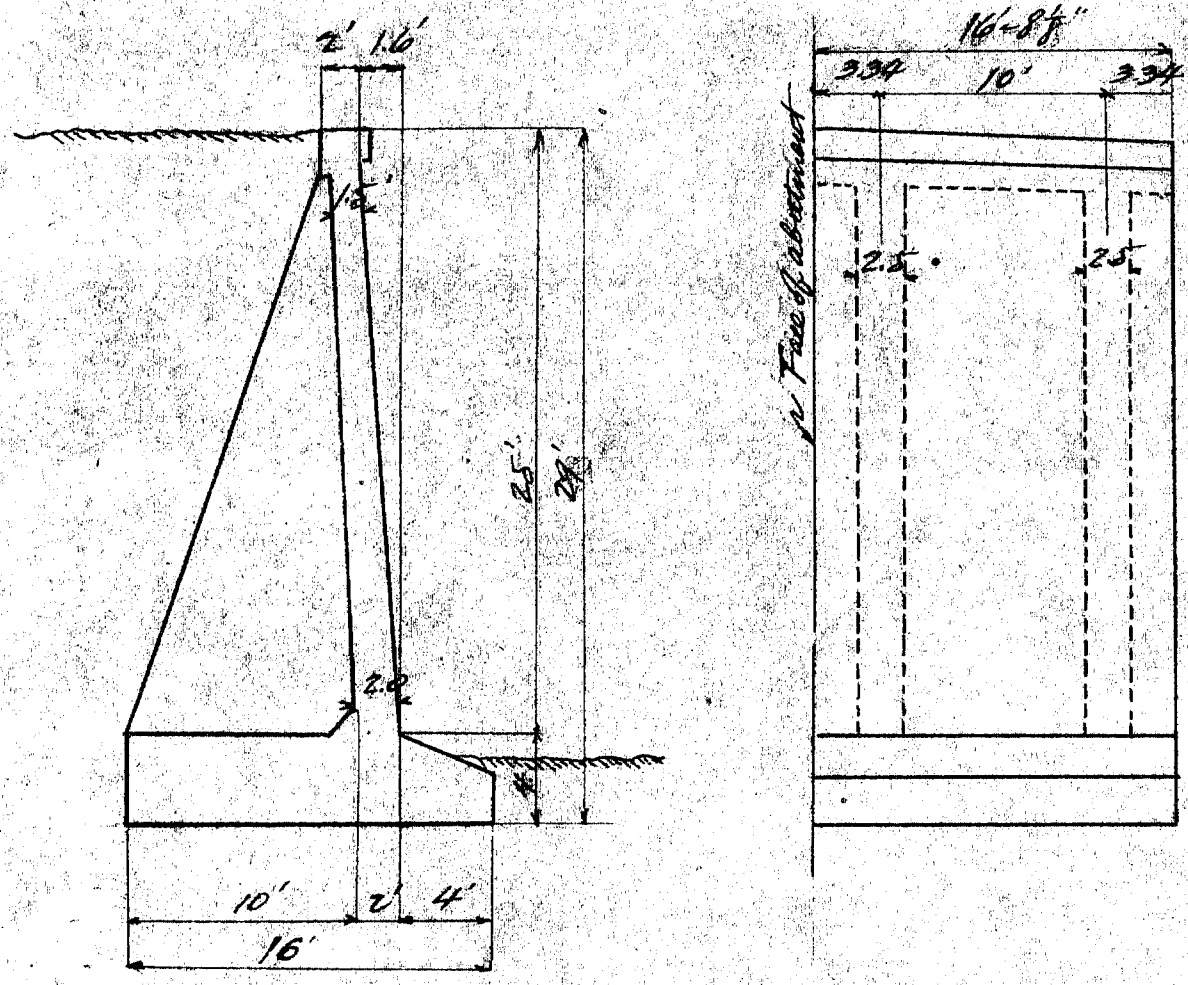
CALCULATIONS FOR

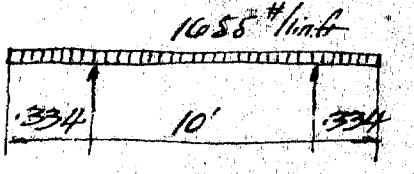
42B Bridge for City of Tokyo
Design of Wing Wall Reinforced concrete counterforted retaining wall 30' hight and 16'-8¼" long



Curtain wall Span 10' Bottom section

Earth pressure = $\frac{100\times25}{3}$ = 833 #/ft' At normal state

= .662×100×25 = 1655 #/ft' during earthquake



latter case control

moment $\frac{1655\times10^2}{8}$ = 20,700

less neg. moment $\frac{1655\times.334^2}{2}$ = −9,230

Total moment = 11,470 '#

Steel req'd = $\frac{11,470\times12}{30,600\times\frac{7}{8}\times22}$ = .233 □"/ft

Use 5/8"φ 12" spacing = .3068 □"/ft

At 10' above top of base thickness of wall = 21.5" abt.

Earth pressure during earthquake

= .662×10d×15 = 993 #/ft'

moment $\frac{993\times10^2}{8}$ = 12,400 '#

less neg. moment $\frac{993\times3.34^2}{2}$ = 5540

Total moment = 6860 '#/ft.

Steel req'd = $\frac{6860\times12}{30600\times\frac{7}{8}\times19.5}$ = .1575

Use 5/8"φ 18" spacing = 0.204 □"/ft

Counterfort Wall Earth pressure at normal state = $\frac{100\times25^2}{6}$ = 10,420 #

" " during earthquake = $\frac{.662\times100\times25^2}{2}$ = 20,680 # $\frac{20,680}{1.8}$ = 11,500 #

Case of earthquake controls

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CALCULATIONS FOR

42-B Bridge for City of Tokyo

Total earth pressure on wall = 20680×16.68 = 345,000#

Moment = 345,000 × 8.33 = 2,870,000 in#

Steel req'd = $\frac{2,870,000 \times 12}{30,600 \times \frac{7}{8} \times 212}$ = 6.07 sq in

Use 12 - 7/8" ϕ = 7.23 sq in on bottom 8 - 7/8" ϕ = 4.81 sq in on Top

Steel ratio $p = \frac{7.23}{200 \times 212}$ = 0.000171 $p' = .000114$

$\Delta = x/d = 24/212 = .113$ $\frac{d'}{d} = \frac{3}{212} = .014$

$k = \frac{p + p'(\frac{d'}{d}) + \frac{\Delta^2}{2n}}{p + p' + \frac{\Delta}{n}} = \frac{.000171 + .000114 \times .014 + \frac{.113^2}{30}}{.000171 + .000114 + \frac{.113}{15}} = .0764$

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

$= \frac{.113(2 \times .0764 - .113) - \frac{.113^2}{3}(2 \times .0764 - 2 \times .113) + 2 \times .000114 \times 15(.0764 - .014)(1 - 0.014)}{.113(2 \times .0764 - .113) + 2 \times .000114 \times 15(.0764 - .014)}$

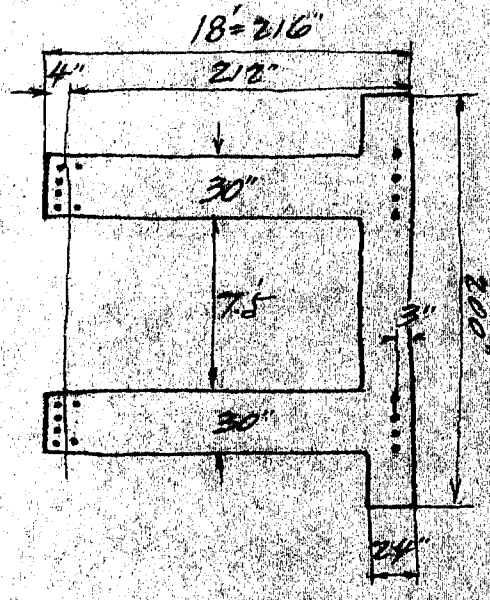
$= .98$

$f_s = \frac{2,870,000 \times 12}{7.23 \times .98 \times 212} = 22,900$ #/□" ok

$f_c = \frac{f_s k}{n(1-k)} = \frac{22900 \times .0764}{15(1 - .0764)} = 127$ #/□" ok

$v = \frac{345000}{60 \times .98 \times 212} = 27.7$ #/□" ok

$u = \frac{345000}{2.75 \times 12 \times .98 \times 212} = 50.2$ #/□" ok



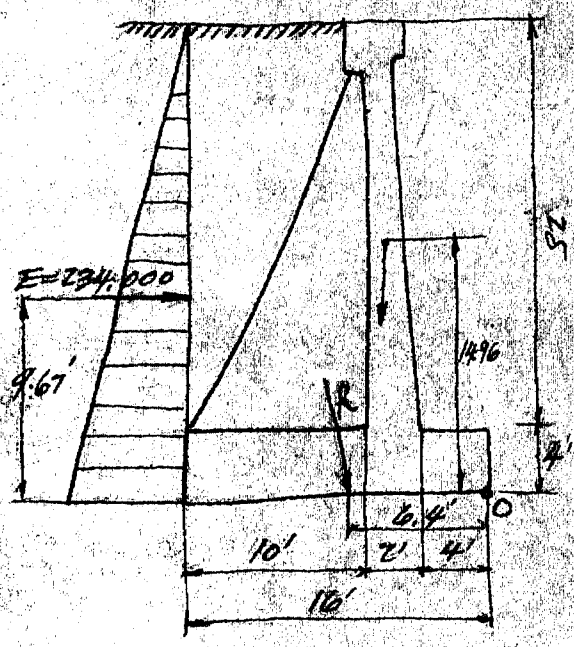
Foundation

Case I Stability at normal case.

taking moment about O (Toe)

Coping	2.3×2 × 16.68 @150 =	11500	× 6.45 =	74.200	#
Wall	$\frac{1.5+2}{2}$ × 23×16.68 @150 =	100,600	× 5.9 =	594.200	
Counterfort	$\frac{10 \times 23}{2} \times 5$ @ 150 =	12,380	× 9.5 =	117.500	
base	4×16×16.68 @ 150 =	160,000	× 8.0 =	1280.000	
Earth	9.5×25×16.68 @ 100 =	395,500	× 11.5 =	4,550,000	
Earthpress		234.000	× 9.67 =	-2,262.000	
	ΣV =	679,980		4,353,900	#
	ΣH =	234,000			

6,615,900



distance of resultant force from O

$= \frac{4,353,900}{679,980} = 6.4$

Eccentricity = 8 − 6.4 = 1.60

Moment at bottom = 679,980 × 1.6 = 1,088,000 in#

max. toe pressure = $\frac{679,980}{16 \times 16.68} \pm \frac{1,088,000}{712}$

$= 2,550 \pm 1530 = 4080$ #/□' = 1.82 tons/□'

or = 1020 " = 0.45 "

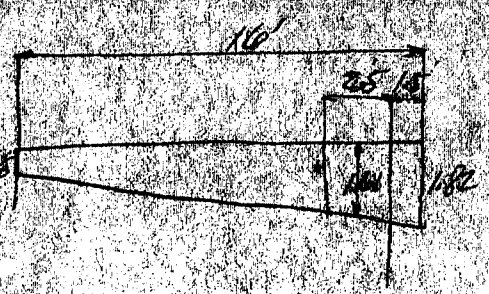
Earth press = $\frac{100 \times 23^2}{6} \times 16.68$

= 234,000#

Section modulus of base

$= \frac{16.68 \times 16^2}{6} = 712$

Spacing of piles 3' longitudinally and 2.5' transversly near toe



$\frac{(1.82 - 0.45)13.5}{16} + 0.45 = 1.56$ tons/□'

Load on the extream pile

$= \frac{1.16 + 1.82}{2} \times 2.5 \times 3 = 12.86$ tons

Friction on pile surface = $\frac{12.86 \times 2240}{935} = 308$ #/□' OK

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CALCULATIONS FOR

42-B Bridge for City of Tokyo

Case 2. Stability during earthquake toward river side

Earth pressure = $\frac{.662 \times 100 \times 29^2}{2} \times 16.68 = 463,500\#$

$E_H = .89 \times 463,500 = 442,000\#$

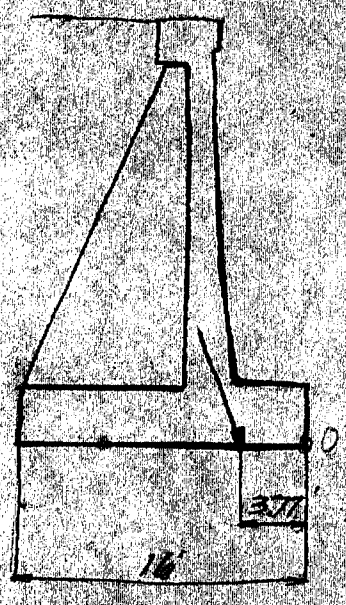
$E_V = .458 \times 463,500 = 211,000\#$

Moment about 0 due to vert. forces

Moment due to wt. of structure & earthfill = 4,353,900 + 2,262,000 = 6,615,900 '# (See case 1.)

Due to Earth pres. EV $211,000 \times 16 = 3,375,000$

$\Sigma V = 890,980\#$ $\Sigma M_V = -9,990,900$ '#



Moment about 0 due to Hor. forces

Coping	11,500 × .3 =	3,450 ×	28.0	= 96,600
Wall	100,600 × .3 =	30,200 ×	15.0	= 453,000
Counterfort	12,380 × .3 =	3,720 ×	12.3	= 45,800
Base	160,000 × .3 =	48,000 ×	2.0	= 96,000
Earth on base	395,500 × .3 =	118,600 ×	16.5	= 1,958,000
Earth pressure E_H		412,000 ×	9.67	= 3,985,000

$\Sigma M_H = +6,634,400$ '#

Moment due to Hor. forces = +6634,400

" " " Vert " = −9,990,900

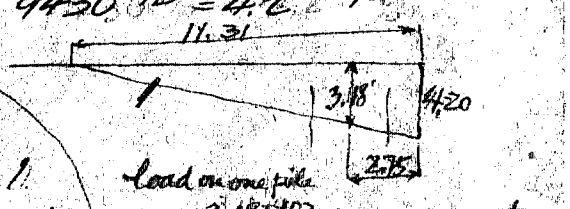
Total moment about 0 = −3,356,500

Distance of resultant force from 0 = $\frac{3,356,500}{890,980} = 3.77'$ OK

Eccentricity = 8 − 3.77 = 4.23

neglecting tension

max. toe pres. = $\frac{2 \times 890.980}{3 \times 3.77 \times 16.68} = 9430\#/0' = 4.2^{kg/cm^2}/0'$



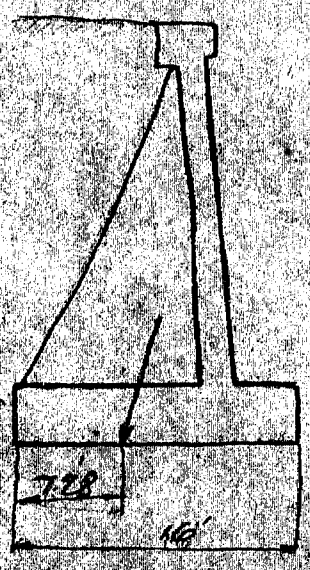
Case 3 Stability during earthquake toward bank side

Taking moment about 0 (toe)

Moment due to Vert. forces See case 1.

Coping	11,500 × 6.45	=	74,200 '#
Wall	100,600 × 5.9	=	594,200
Counterfort	12,380 × 9.5	=	117,500
Base	160,000 × 8.0	=	1,280,000
Earth on base	396,500 × 11.5	=	4,550,000

$\Sigma V = 679,980\#$ $\Sigma M_V = +6,615,900$ '#



Moment due to Hor forces.

Coping	3.450 ×	28.0	= 96.600
Wall	30.200 ×	15.0	= 453.000
Counterfort	3.720 ×	12.3	= 45,800
Base	48.000 ×	2.0	= 96.000

$\Sigma M_H = -691.400$ '#

$M_V = 6.615.900$

$M_H = -691.400$

Total M about 0 = 5,924,500 '#

Distance of resultant force from 0

= $\frac{5,924,500}{679.980} = 8.72'$

Eccentricity = 8.78 − 8 = 0.78' OK

Overturning moment = 679,980 × 0.78 = 530,000 '#

Max. soil pres. = $\frac{679.980}{16 \times 16.68} \pm \frac{530000}{712}$ or = 3290 #/0' = 1.47 kg/cm²

1,810 = 0.81 "

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MADE BY K. Inada DATE 2-8-30 FILE NO.
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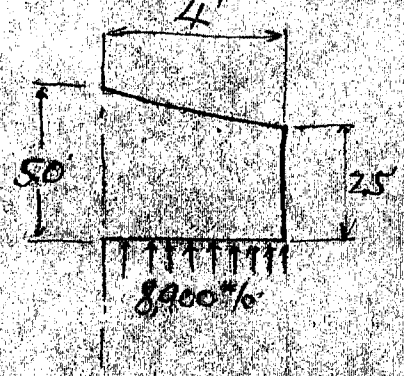
CALCULATIONS FOR

42 B Bridge for City of Tokyo.
Design of Foundation
Cantilever footing at Toe. Case 2 Controls (Case of earthquake)



upward pressure = 4.2 tons/□' = 9,430 #/0'
less Weight of footing = 3.5 @ 150 (conc.) = − 525
8,905 call this 8,900 #/0'

Moment of fixture = $\frac{8900 \times 4^2}{2}$ = 71,200 '#

Steel required = $\frac{71{,}200 \times 12}{30600 \times \frac{7}{8} \times 46}$ = 0.80 □"/ft strip.

use 2 − 7/8"□ bars = 1.20 □"/ft Strip
max. end shear = 8900 × 4 = 35,600 #
Steel ratio = $\frac{1.20}{12 \times 46}$ = .0022

K = 0.222 j = 926

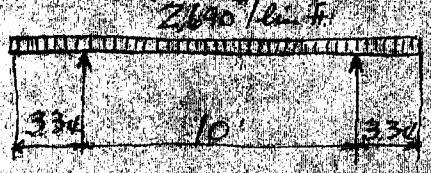
Unit shear = $\frac{35600}{12 \times .926 \times 46}$ = 69.8 #/0" OK (allowable shear = 57.8 + 103 #/0")

Unit bond = $\frac{35600}{2 \times 2.75 \times 926 \times 46}$ = 152 #/0" OK (allowable bond = 85 × 1.8 = 153 #/0")

Base between Counterforts

Case 2 controls (Earthquake)

Upward pressure = 3,290 #/0'
Base weight of base = 4 × 150 = − 600
2,690 #/0'



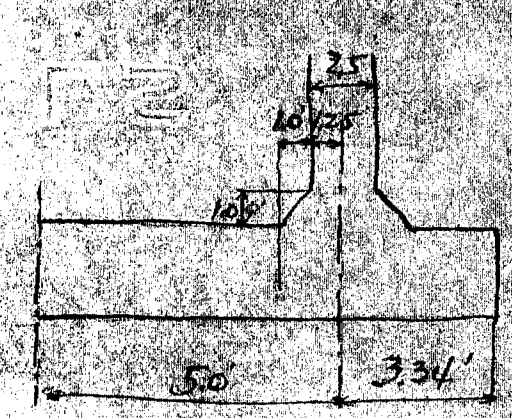
Neg. moment at support = $\frac{1}{2} \times 2{,}690 \times 3.34^2$ = −15,000 '#
Moment at center as a Simple beam = $\frac{1}{8} \times 2{,}690 \times 10^2$ = +34600
Max pos moment at center = +19,600 '#

Steel req'd = $\frac{19600 \times 12}{30600 \times \frac{7}{8} \times 34}$ = .258 □"/ft strip.

use 3/4"□ bars 1' spacing = 0.442 □"/ft strip.

Max end shear = 2,690 × 5 = 13450 #

Unit shear = $\frac{13450}{12 \times \frac{7}{8} \times 46}$ = 27.9 #/0" OK



Unit bond = $\frac{13450}{2.36 \times \frac{7}{8} \times 46}$ = 141 #/0" OK

Shear at 2.25' from support
= 2690 × (5 − 2.25) = 7400 #

Unit shear = $\frac{7400}{12 \times \frac{7}{8} \times 34}$ = 15.9 #/0" OK

Unit bond = $\frac{7400}{2.36 \times \frac{7}{8} \times 34}$ = 105 #/0" OK

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MADE BY _____ DATE _____ FILE NO _____

CHECKED BY _____ DATE _____ PAGE NO 0

CALCULATIONS FOR

東京市 昭和二年十月 一〇八A 新設橋梁 設計々算書

設計者
増田橋梁事務所

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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MADE BY J. Masuda DATE 2-9-27 FILE NO
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CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo.

After surveying of bridge site, the center line of bridge has been changed. Exact dimension and angles shall be figured later. Let us assume the length of bridge 114.8' out to out as 11 panels @ 10.3' = 113.3' center to center of end bearings.

1.5

114.8' out to out of structure

with skew of 39° to center line of canal.

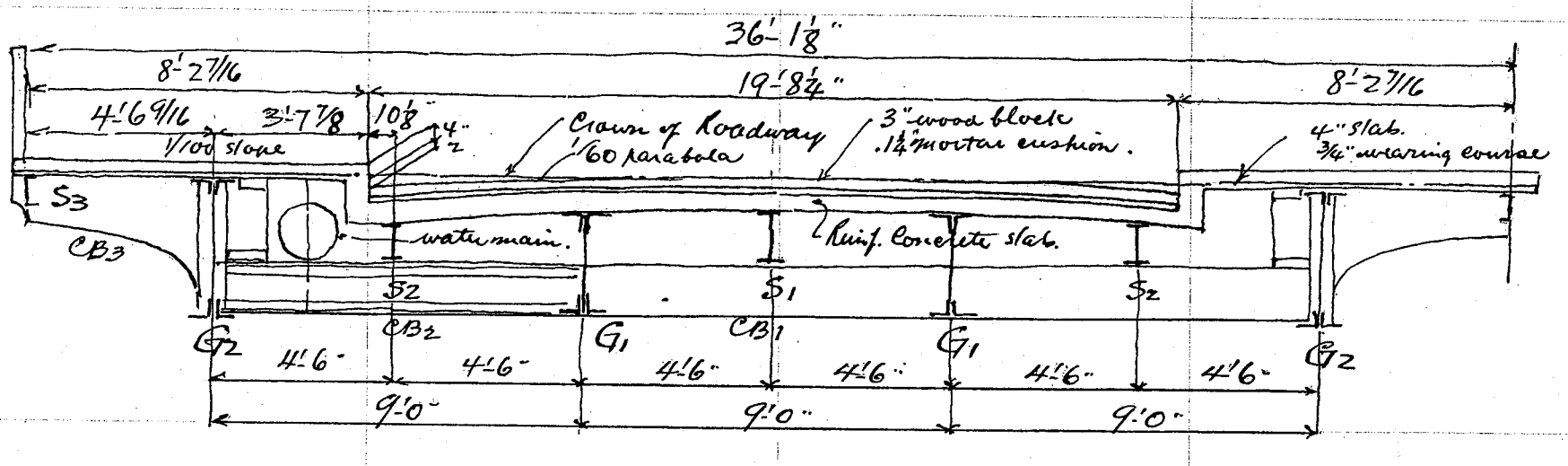
The bridge is divided into the following span (This type has been approved by City Bridge Engineer Mr. Koike)

Anchor span 3@10.3' = 30.9' on both sides
Cantilever arm 1 panel 10.3 - 1.12 = 9.18' to ℄ hinge.
Suspended span 3@10.3 = 30.9
panel point to pin 2@1.12 = 2.25
33.15'

Roadway 6 metres = 19.685' = 19'-8¼"
Sidewalks 2 - 2.5 metres = 8.202 = 8'-2 7/16"

19'-8¼"
2@8'-2 7/16" = 16'-4 7/8"
Total effective width = 36'-1⅛" c/s to c/s of Handrails.

Loading as per specification
Cross section of bridge assumed as Sketch below.



Inside girder Outside girder.

Design of floor slab. span length 4'-6"

Dead Load
3" wood block pavement @ 60#/12 = 15.0
1¼" mortar cushion @ 110#/12 = 11.5
floor slab assumed 75.0
allowance 3.5
105.0# per sq ft.

Dead load moment = 1/10 × 105 × 4.5² = 212.0 ft
" " shear = ½ × 105 × 4.5 = 237 #

Live Load motor truck loading rear wheel concentration = 9920
30% impact 2980
12900 #

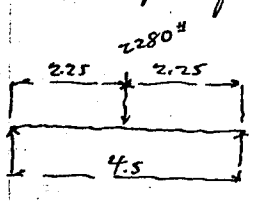
Front wheel concentration say 12900 ÷ 3 = 4300 #

Distribution of wheel concentration on floor slab.
Longitudinal distribution = 2@ 4¼" = .720
20 cm = .660
a = 1.36'

Transverse distribution = b = 1.28 + .70 = 1.98'

Effective width = ⅔ (4.50 + 1.98) + 1.36 = 5.67' less than 2 metres

Load per ft strip = 12900 ÷ 5.67 = 2280 #



moment = 1140 × 2.25 = 2560 ft
for continuity of slab reduce the moment to
0.8 × 2560 = 2050 ft

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MADE BY J. Masuda DATE 2-9-28 FILE NO
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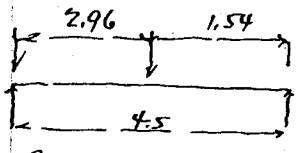
CALCULATIONS FOR

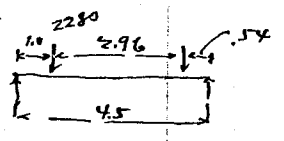
Design of 108 A Bridge for City of Tokyo.

End shear — Effective width assumed same as for moment

NBO NBO Take ½ for shear



1140 × 1.54/4.50 = 390
1140
1530# × 2 = 3060#
1140
1920



shear = 2280 × 4.04/4.50 = 2080#

Summary for moment and shear

	moment	shear
Dead Load	212	237
		2080
Live Load	2050	3060
	2262'#	3297#
		2317#

Effective depth of slabs for 17000# steel stress and 640#/□" Concrete stress n = 15

$d = \sqrt{\frac{2262}{102}} = 4.72"$

Use 6" slab with 1" insulation at bottom

Steel area required = $\frac{2262 \times 12}{7/8 \times 5 \times 17000} = 0.365$ □" per ft strip

½"ϕ bars 6" centers = 0.39 □" per ft strip.

End shear $\frac{2317}{7/8 \times 12} = 44\#/□"$

perimeter = 2-½"ϕ = 3.14
2-⅜"ϕ = 1.41
4.55"

Bond stress = $\frac{2317}{7/8 \times 5 \times 4.55} = 117\#/□" < 130\#/□"$ ok

Use 2-⅜" bars at support in addition to 2-½"ϕ bars per ft. strip for bond stress.

Sidewalk slab. span length 4.54'

Dead Load

¾" wearing course (mortar)	7"
6" Concrete slab.	50
misc. assumed	3
	60# per sq ft.

Dead Load moment = 1/10 × 60 × 4.54² = 123'#
" " shear = ½ × 60 × 4.54 = 136#

Live Load 100# per sq ft.

Live Load moment = 1/10 × 100 × 4.54² = 206'#
shear = ½ × 100 × 4.54 = 227#

Summary for moment and shear

	moment	shear
Dead Load	123	136
Live Load	206	227
	329'#	363#

Effective depth = $\sqrt{\frac{329}{102}} = 1.8"$

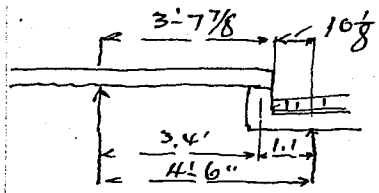
use 4" slab with 1" insulation at bottom

Steel area = $\frac{329 \times 12}{7/8 \times 3 \times 17000} = 0.088$ □" per ft strip

use ⅜"ϕ bars 6" centers = 0.22□"

This should be bent up at support to take care of negative moment

Slab at curb line.



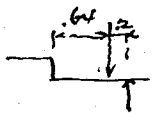
Construction assumed as sketch
sidewalk slab same as for outside of main girder
1.1' cantilever portion shall be designed.

Dead Load sidewalk slab. 60 × 3.4/2 = 102#
Curb assumed 75
177#

Moment = ½ × 105 × 1.1² = 63
177 × 1.1 = 195
258'#

Live Load 100 × 3.4/2 = 170
at 3' edge 25
195# per lin ft

Motor truck loading rear wheel concentration with impact = 12900#



longitudinal distribution assumed arbitrarily 2' wide 12900 ÷ 2 = 6450#

Moment = 6450 × 0.2 = 1290
195 × 1.1 = 214
1504'#

slab 6" reinf. 2-½"ϕ } bars.
2-⅜"ϕ }

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CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

Longitudinal stringer for roadway S1 Span length 10.3'

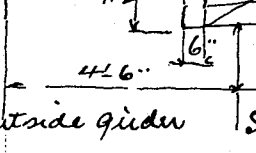
Dead Load

Slab and pavement 105 × 4.5 = 472#
stringer assumed 48
520# per lin ft.

Dead load moment = $\frac{1}{8} \times 520 \times 10.3^2$ = 6900 ft#
shear = $\frac{1}{2} \times 520 \times 10.3$ = 2680 #

Live load motor truck rear wheel with impact = 12900#

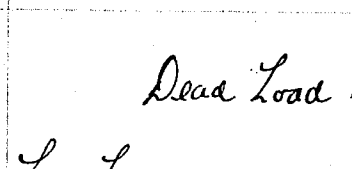
Load on stringer



12900 × $\frac{1.54}{4.50}$ = 4410
12900
Load on stringer 17310#

moment at center = $\frac{17310}{2} \times 5.15$ = 44700 ft#

End shear uniform live load 120#/0' × 4.5 = 540# per lin ft of span



Reaction = 540 × 3.74 × $\frac{1.87}{10.3}$ = 367
17310
17677 #

Summary for moment and shear

	moment	shear
Dead Load	6900	2680
Live Load	44700	17677
	51600 ft#	20357 #

Section modulus reqd = $\frac{51600 \times 12}{15600}$ = 38.00 Use 14" × 6" I @ 46.01# S_x = 62.946

Unit stress = $\frac{51600 \times 12}{62.946}$ = 9860 #/0"

Longitudinal stringer S2 Span length 10.3'

Dead Load

Sidewalk slab 60 × 3.65 = 219#
05 × .96 @ 150 = 72
105 × .84 = 88#
379 #

Load on S2 = 219 × 1.82 = 399
72 × 3.40 = 245
88 × 4.08 = 359
379 1003 ÷ 4.5 = 223#

Load on outside girder = 379 − 223 = 156#

Load on S2 105 × 2.25 = 237
460 #.

Weight of stringer 35
495 #

Dead Load moment = $\frac{1}{8} \times 495 \times 10.3^2$ = 6550 ft#
shear = $\frac{1}{2} \times 495 \times 10.3$ = 2550 #

Live load.

Uniform live load 100 × 3.65 = 365# per ft.
load on S2 = 365 × $\frac{1.82}{4.5}$ = 147# per ft.

motor truck loading rear wheel with impact 12900#
Load on S2 assumed 12900#

moment = $\frac{1}{8} \times 147 \times 10.3^2$ = 1950 ft# uniform load
moment = 6450 × 5.15 = 33200 motor truck
35150

uniform load on roadway .84 × 120 × $\frac{.42}{4.5}$ = 94#
120 × 2.25 = 270
364# per lin. ft.

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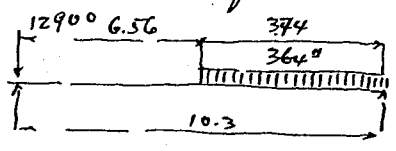
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

End shear for live load.

Uniform load on sidewalk = ½ · 147 · 10.3 = 758#



364 × 3.74 × 1.87/10.3 = 247
motor truck 12900
13147
13905 #

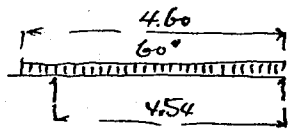
Summary for moment and shear

	moment	shear
Dead Load	6550	2550
Live Load	35150	13905
	41700 ft	16455 #

Section modulus reqd = 41700 × 12 / 15000 = 32.0
use 12" × 5" I @ 31.99 # Sm = 36.69
fibre stress = 41700 × 12 / 36.69 = 13600 #/□" ok

Sidewalk Stringer S3

Dead Load



60" × 4.60 × 2.30/4.54 = 140 #
Handrail assumed 75
Stringer assumed 20
235 # per lin ft.

DL moment = 1/8 · 235 · 10.3² = 3120 ft
shear = ½ · 235 · 10.3 = 1210 #

Live Load 100 × 4.54/2 = 227 # per lin ft.

LL moment = 1/8 · 227 · 10.3² = 3010
shear = ½ · 227 · 10.3 = 1170

Summary for moment and shear

	moment	shear
Dead Load	3120	1210
Live Load	3010	1170
	6130 ft	2380 #

Section modulus reqd = 6130 × 12 / 15600 = 4.72
use 8" × 2½" I @ 15.12 # per ft. Sm = 10.273
Unit stress = 6130 × 12 / 10273 = 7150 #/□"

Cantilever Bracket on sidewalk spacing 10.3'

Dead Load 235 × 10.3 = 2420 # Concentration at S3.

moment = 2420 × 4.54 = 11.000 ft#
Cantilever ½ · 100 · 4.54² = 1030
12030 ft#

Shear = ½ · 100 · 4.54 = 227 #
2420
2647 #

Live Load 227 × 10.3 = 2340 # at S3

moment = 2340 × 4.54 = 10600
Shear = 2340

Summary for moment and shear

	moment	shear
Dead Load	12.030	2647
Live Load	10600	2340
	22630 ft#	4987 #

Depth of beam at connection assumed 24" about.
Flange section 2 ls 3.3. 5/16 = 3.56 web 5/16 thick.
Effective depth for tension plate 24"
stress = 22630 / 2.0 = 11315 #

SR = 11315 ÷ 17000 = .66" ner.
Assumed section ok

CB1

Cross beam between main inside girders span length 9'-0" 3° skew neglected

Dead Load

Concentration at center from S1 520 # × 10.3 = 5350 #

moment = 5350/2 × 4.5 = 12050 ~~24100~~ ft#
DL beam = 8 × 80 · 9.0² = 810
~~24910~~ 12860 ft#

End shear = 5350 ÷ 2 = 2675
Cross beam 360
3035 # at end

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MADE BY J. Masuda DATE 2-9-28 FILE NO
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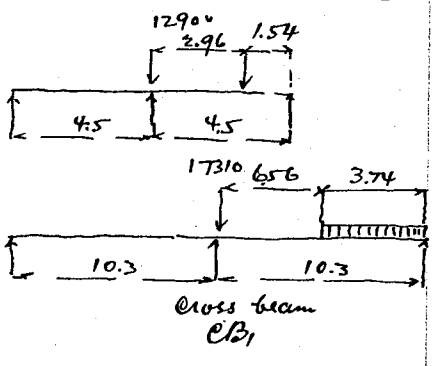
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo

Live Load. motor truck loading rear wheel with impact = 12900#



$12900 \times \frac{1.54}{4.50} = 4410$

12900

17310#

Unif. load $120 \cdot 4.50 = 540$

Load on cross beam $= 540 \times 3.74 \times \frac{1.87}{10.3} = 367$#

17310

17677#.

Moment $= \frac{17677}{2} \cdot 4.5 = 44300$ '#

Shear $= 17677 \div 2 = 8839$#

Summary for moment and shear

	moment	shear
Dead Load	12860	3035
Live Load	44300	8839
	57160 '#	11874#

Depth 12 5/8" web = 12¼" · 3/8" = 4.60"

1/8 web = 0.570"

Effective depth 12.62
2.02
10.60"

Flange stress $= \frac{57160 \times 12}{10.60} = 64600$# Section Rq'd $= \frac{64600}{17069} = 3.790$" net

.57

3.220" net

Unit flange stress on compression side $= 17067 (1 - 0.012 \frac{l}{b}) \leqq 15600$ #/□"

$l = 54"$ $b = 7.37$ $17067 (1 - 0.012 \cdot \frac{54}{7.37}) = 15600$ #/□"

Dry 2L 3½ · 3½ · 3/8 = 4.96 − 0.66 = 4.30□" net.

.57

4.93

87

Unit flange stress $= \frac{64600}{4.93\frac{87}{87}} = 13300$ #/□"

weight of Cross beam 2@4.96 = 9.92

web 4.60

14.52 @ 3.4 = 50#

Detail say 25% 13

63# per lin ft.

Cross beam CB2. Span length 9.0'

Dead Load. Stringer Concentration S2 495# × 10.3 = 5100#

Dead load water main.

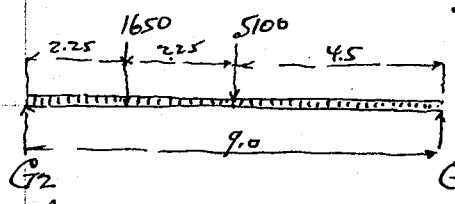
12" main. water Contents = .78 @ 62 = 48#

⅜" pipe with details say 60"

protection assumed 52

160# per lin ft.

Concentration on CB = 160 × 10.3 = 1650#



Dead Load moment = 3790 × 4.5 = 17050

1650 × 2.25 = 3710

13340 '#

beam = ⅛ · 80 · 9.0² = 810

14150 '#

R = 1650 × 6.75 ÷ 9.0 = 1240

2550

3790#

Beam 360

4150# End shear G2

Dead Load cantilever moment should be reduced from the above

Cantilever moment 12030 ÷ 2 = 6015 '#
at center of span

On safe side this moment neglected.

1650 − 1240 = 410

Conc. 2550

beam 360

3320# End shear G1

Live Load. see page 4. 2@758 = 1516 on sidewalk.

13147

14663# motor truck and unif. on Roadway

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CALCULATIONS FOR

Design of 108 A for City of Tokyo.

moment = $\frac{1466}{2} \times 4.5$ = 33.000 ft

Shear = 7330 #

Summary for moment and shear

	moment	shear G_2	shear G_1
Dead Load	14150	4150	3320
Live Load	33.000	7330	7330
	47150 ft	11480 #	10650 #

Depth 12 5/8" web 12 1/4 × 3/8 = 4.60
1/8 web = 0.570
Effective depth = 10.60"

flange stress = $47150 \times \frac{12}{10.60}$ = 53300 #

Section req'd = $\frac{53300 \times \text{F.S.}}{17067}$ = 3.12
.57
2.550" net

use 2 L 3 1/2 × 3 1/2 × 3/8 = 4.96 − 0.66 = 4.30 3/4" Rivet.
.57
4.870" net.

Unit flange stress = $\frac{53300}{4.87}$ = 10950 #/0" ok

Approximate weight of cross beams and Cantilever Brackets.

Cross beam CB1

4 L 3 1/2 × 3 1/2 × 3/8	@ 8.50 #	× 9.0	=	306.
1 web. 12 1/4" × 3/8	@ 15.62	× 9.0	=	141
2 Stiffs L 3 × 3 × 5/16	@ 6.1	× 1.05	=	13
2 End stiffs L 5 × 3 1/2 × 3/8	@ 10.4	× 1.0	=	21
2 fills 5 × 3/8	@ 6.38	× .48	=	6
Rivet heads + misc say				15
				502 # ÷ 9.0 = 56 # per lin ft.

Cantilever Bracket CB3 and bracket at rear.

4 L 3 × 3 × 5/16	@ 6.1	× 4.5	=	110
1 web. 24 × 5/16	@ 25.50	× 4.5	=	115
2 L 3 × 3 × 5/16	@ 6.1	× 1.0	=	12
2 L 3 × 3 × 5/16	@ 6.1	× 1.5	=	19
1 web 24 × 5/16	@ 25.50	× 1.5	=	38
2 L 5 × 3 1/2 × 3/8	@ 10.4	× 2.0	=	42
2 L 3 × 3 × 5/16	@ 6.1	× 1.0	=	12
2 fills 3 × 5/16	@ 3.19	× 0.5	=	3
				351
Misc details say				49
				400 #

Total weight of cross beams and cantilever

3 cross beams	@ 502	=	1506
2 Cantilever	@ 400	=	800
			2306 # ÷ 36.1 = 64 # per lin ft.

Suspended span span length 33.15'

Design of main girder for G1 (Inside girder).

Dead load

Uniform	105 × 4.5 =	473 #
girder assumed		167
		640 # per lin ft.

Panel concentration from floor beam for intermediate point.

End reaction CB1	3035
" " CB2	3320
	6355 #

Panel Concentration from End floor beam length 5.15 + 1.12 = 6.27

From CB1 520 × 6.27 = 3260 3260 ÷ 2 = 1630
End floor beam 360
1990 #

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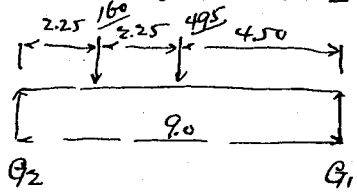
CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo.

From CB_2



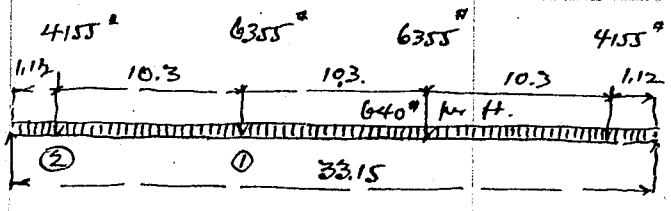
160 · 2.25/9.00 = 40.0
495 ÷ 2 = 248.
288 on G_1

288 × 6.27 = 1805
Cross beam 360
From cross beam CB_2 2165 #
" " " CB_1 1990
4155 #

Reaction = 6355
4155
10510 #.

4.95
1.60
6.55 − 288 = 367 # on G_2

Dead load moment



moment due to concentration.
Moment ① 10510 × 11.42 = 120.000
4155 × 10.30 = −42800
77200 ##

Moment ② 10510 × 1.12 = 11800 ##

Moment due to uniform load
at center $\frac{1}{8} \times 640 \times 33.15^2$ = 88.000 ##
at ① $\frac{1}{2} \times 640 \times 11.42 \times 21.73$ = 79500
at ② $\frac{1}{2} \times 640 \times 1.12 \times 32.03$ = 11500

End shear
Concentration = 10510
Uniform $640 \cdot \frac{33.15}{2}$ = 10600
21110 #

Summary for moments

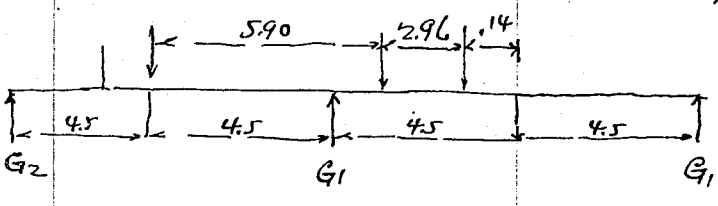
	1	at 1	2	Center
Concentration		11800	77200	77200
Uniform		11500	79500	88000
		23300 ##	156700 ##	165200 ##

Girder section will be determined at center of span and the same section will be used throughout. Let us figure the live load moment at center of span only.

Live load motor truck loading impact = $\frac{20}{60 + \frac{33.15}{3.28}}$ = 28.5 %.

Rear wheel Concentration 9920 #
Impact 28.5% 2820
12740 #

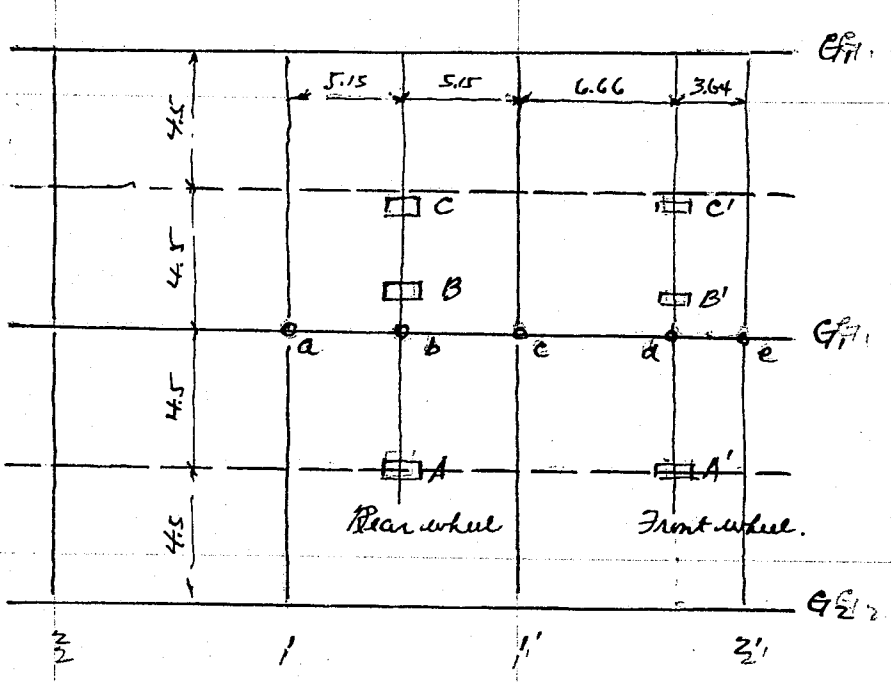
Front wheel with impact
12740 ÷ 3 = 4250 #
Rear wheels 12740 #



A. 12740 ÷ 4 = 3185 a and c

B+C 12740 × .14/4.5 = 396 12740−396 = 12344
12740 × 3.1/4.5 = 8780 12740−8780 = 3960
9176 16304
Divide on girder b. on stringer

9176 −
16304 ÷ 4 = 4076 # on a + c



Front wheels. 4250 #

A' 4250 × 3.64/10.30 = 1500 750 # on e
4250 − 1500 = 2750 1375 on d

B'+C' 4250 × .14/4.5 = 132
4250 × 3.1/4.5 = 2927
3059 # on girder (d).
8500 − 3059 = 5440 # on stringer
5440 × 3.64/10.3 = 1920 960 on e
5440 − 1920 = 3520 1760 on e

a	b	c	d	e
3185		3185		
4076	9176	4076		
		750	3059	1375
		960		1760
7261	9176	8971	3059	3135

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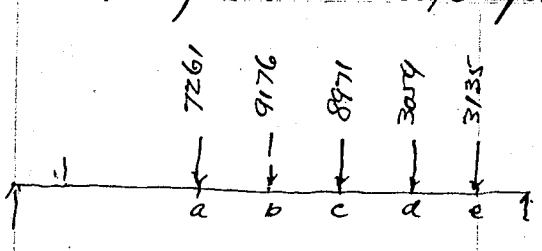
CALCULATIONS FOR

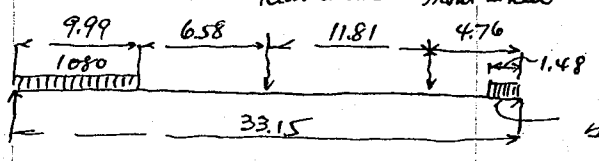
Design of 108A Bridge for City of Tokyo



Reaction
3135 × 1.12 ÷ 33.15 = 106
3059 × 4.76 ÷ 33.15 = 439
8971 × 11.42 ÷ 33.15 = 3100
9176 ÷ 2 = 4588
7261 × 21.72 ÷ 33.15 = 4750
12983 #

moment = 12983 × 16.57 = 215.000
7261 × 5.15 = − 37400
177.600 '#

Uniform live load at rear of motor truck



On safe side 120 × 9.0 = 1080 # assumed.
Uniform load on sides of motor truck neglected.
Uniform loading assumed as sketch
neglected

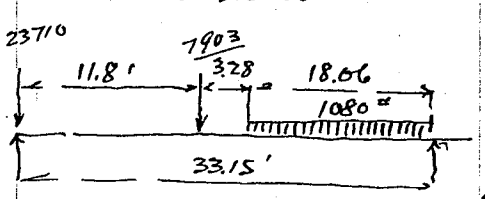
1080 × 9.99 = 10800 #
1080 × 1.48 = 1600 #

Reaction = 10800 × 5.0 ÷ 33.15 = 16300
1600 × 32.41 ÷ 33.15 = 1570.
3200.0 #

moment = 3200 × 16.57 = 53000
1600 × 15.83 = − 25400
27600 '#

Due to Uniform load 27600 '#
" " motor truck 177600
205200 '#

Max End shear motor truck rear wheel.



1080 × 18.06 = 19500 #

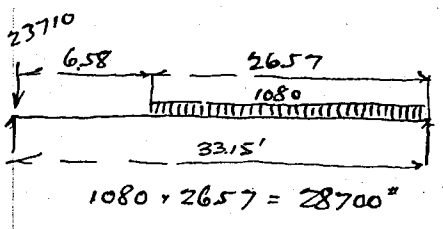
A. 12740 ÷ 2 = 6370
B 12740 × 7.60 ÷ 9.0 = 10780
C 12740 × 4.64 ÷ 9.0 = 6560
23710 #

For front wheel 23710 ÷ 3 = 7903 #
Uniform load at rear or front of motor truck assumed 1080 # per ft.

motor truck 7903 × 21.34 ÷ 33.15 = 5090
23710

28800 #

Unif. load 19500 × 9.03 ÷ 33.15 = 5320
34120 #



1080 × 26.57 = 28700 #

Unif. 28700 × 13.29/33.15 = 11500
motor truck 23710
35210 #

mom

Summary for moment and shear

	moment	shear
Dead Load	165200	21110
Live load	205200	35210
	370400 '#	56320 #

b to b of ls 27½" web 27 × 3/8" = 10.12
⅛ web = 1.270"
½ effective depth = 2.01
flange stress = 370400 ÷ 2.01 = 184.000 #
SR = 184.000 ÷ 17067 = 10.80
1.27
9.530" net

11.50
1.27.
12.77

9.63
1.27.
10.90

Duy 2ls 6×6.½ = 11.50 − 1.87 = 9.630" net.
Reduction of hole 2-⅞ = .87
2-1" = 1.00
1.87

Unit flange stress = 184000/10.90 = 16900 #/□"
Compression side = 184000/12.77 = 14450 #/□"
Unit shear = 56320/10.12 = 5550 #/□"

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CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

Outside girder G2.

Dead Load.

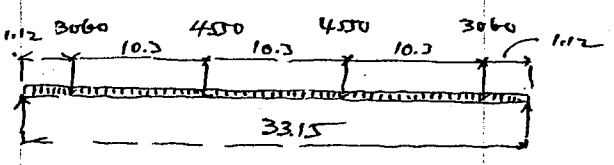
				arm		
Uniform load. see page 4	60 × 4.60 =	276.	×	2.30	=	635.
Handrail		75	×	4.54	=	341
Stringer assumed.		20	×	4.54	=	91
		371				1067
Extra reaction 1067 ÷ 9.0 =		120				
		491#	per lin ft. from cantilever side.			
Direct load on Outside girder page 3.		156				
		647#				
Girder assumed		123				
		770#	per lin ft. of span.			

Load on stringer S2. see page 3 495# per lin ft. including weight of stringer.
water main 160# per lin ft.
Reaction on Outside girder through cross beam
495 ÷ 2 = 248
160 × 6.75/9.0 = 120
368# per lin ft. see page 7.

Panel Concentration 368 × 10.30 = 3800#
Cross beam + Cantilever ~~368 × 6.27~~ say. 750
4550

Panel Concentration 368 × 6.27 = 2310
Cross beam + Cantilever say 750
3060#

Moment



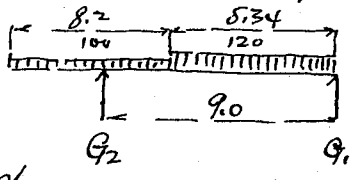
4550
3060
7610#

m = 7610 × 13.42 = 87000
3060 × 10.3 = − 31500
55500

Uniform load 1/8 × 770 × 33.15² = 105500
161000 1#

End shear due to concentration 7610
Uniform. 770 × 33.15/2 = 12750
20360#

Live Load. Uniform live load on sidewalk. 8.2 × 100 = 820# per lin ft.



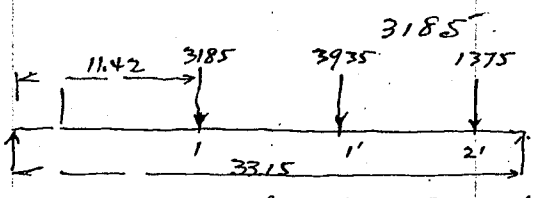
max load on G2 820 × 9.44/9.00 = 860# per lin ft.
Uniform live load on roadway 120 × 5.34 = 640#
load on G2 = 640 × 2.67/9.0 = 190# per lin ft.

These uniform load assumed act direct on girder.

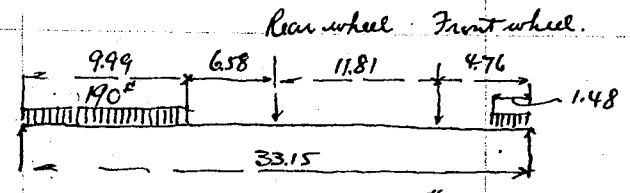
motor Truck loading rear wheel Concentration with impact 12740# see page 7
Front wheel 4250# " " "

Loading as shown on page 7. panel Concentration from wheel load A + A'

1	1'	2'
3185	3185	
	750	1375
3185	3935	1375



Rear wheel Front wheel.



190 × 9.99 = 1900#
190 × 1.48 = 280

Reaction 3185 × 21.72 ÷ 33.15 = 2100
3935 ÷ 2 = 1968
1375 × 1.12 ÷ 33.15 = 46
4114#

Moment = 4114 × 11.42 = 47000 1# at 1
4114 × 21.72 = 89300 1#
3185 × 10.3 = −32800
56500 1# at 1'

Reaction. 1900 × 5 ÷ 33.15 = 286.0
280 × 32.41 ÷ 33.15 = 274.0
560#

m = 560 × 16.57 = 9300
280 × 15.83 = − 4440
4860 1#
Live load on Rwy — 56500
61360 1#

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CALCULATIONS FOR

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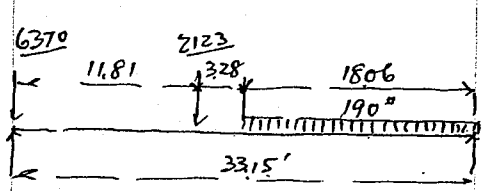
Uniform load on sidewalk = ½ × 860 × 33.15² = 118.000 #
due to load on roadway 61360
179360 #

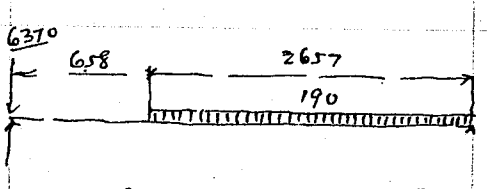
Live Load End shear



190 × 18.06 = 3430#

Motor truck loading
Rear wheel 12740 ÷ 2 = 6370 # direct
Front wheel 6370 ÷ 3 = 2123
motor truck 2123 · 21.34/33.15 = 1365
6370
7735 #

Uniform load 3430 · 9.03/33.15 = 935
8670 #



190 · 26.57 = 5050 #

Unif. load 5050 · 13.29/33.15 = 2020
6370
8390 #

Uniform load on sidewalk. ½ × 860 × 33.15 = 14250
load on roadway max 8670
22920 #

Summary for moment and shear G2

	moment	shear
Dead Load	161000	20360
Live Load	179360	22920
	340360 #	43280 #

web assumed 38" × 3/8" = 14.25 38½ b to b L's.
⅛ web = 1.780" Effective depth = 3.21
.16
3.05'

Flange stress = 340360 ÷ 3.05 = 111400 #
SR = 111400 ÷ 17067 = 6.54
1.78
4.760" net

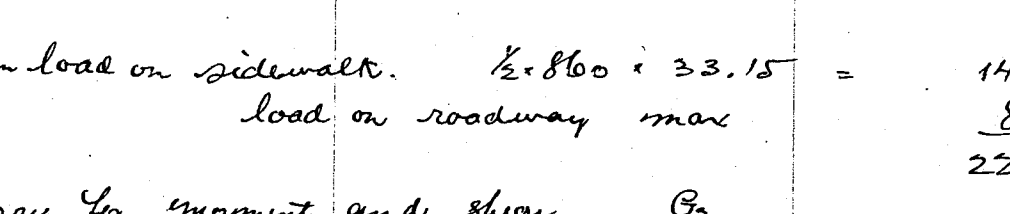
net section 2 - 1" = .75
2 - ⅞ = .66
1.41

2 L's 6 × 4 × 3/8" = 7.22 - 1.41 = 5.81 0" net.
1.78 1.78
9.00 7.59

Unit flange stress (tension) = 111400/7.59 = 14700 #/□" net section.

Compression = 111400/9.0 = 12400 #/□"

Approximate weight of main girder
Inside girder.



flange L's	4 L's 6×6×½	@	19.6	× 33.15	=	2600
	1 web. 27 × ⅜	@	34.43	× 34.15	=	1180
Stiffs Int.	18 L's 5×3½×⅜	@	10.40	× 2.3	=	430
Stiff Panel	8 L's 5×3½×⅜	@	10.40	× 2.3	=	192
fillers	4 Pl's 7½×½	@	12.75	× 1.3	=	66
Stiffs	8 L's 5×5×½	@	16.20	× 1.1	=	143
fillers	4 Pl's 15×½	@	25.50	× 2.2	=	224
bed plate	2 Pl's 12×¾	@	30.60	× 1.0	=	61
splice	2 Pl's 12½×½	@	21.25	× 1.25	=	53
"	4 Pl's 5½×⅜	@	7.01	× 2.00	=	56
	4 L's 6×6×½	@	19.6	× 1.60	=	440 125
						5115
Rivet heads + misc say 5%						250
						5365 ÷ 33.15 = 162# per lin ft.
On account of curveture for bottom add						500
						5865 #

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CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

Outside girder.

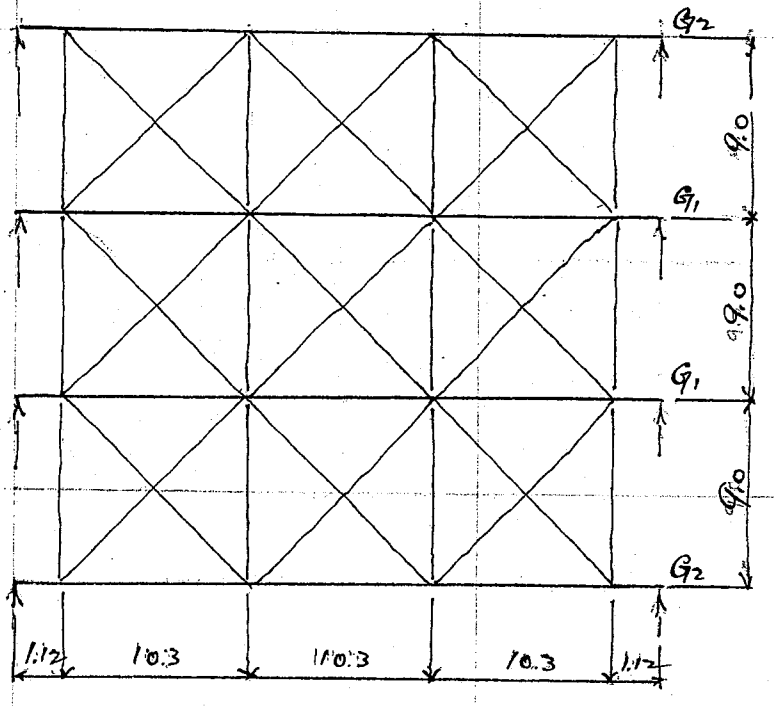
flange Ls	4Ls 6×4×3/8	@	12.3	×	33.15	= 1630
1 web	38×3/8	@	48.45	×	34.15	= 1650
Stiffs	18Ls 5×3½×5/16	@	8.7	×	3.2	= 500
stiffs	8Ls 5×3½×3/8	@	10.4	×	3.2	= 266
filler	4Pls. 7½×3/8	@	9.56	×	2.53	= 97
Stiffs	8Ls 5×5×½	@	16.20	×	1.60	= 207
filler	4Pls. 26×3/8	@	33.15	×	2.2	= 292
end plate	2Pls. 12×3/4	@	30.60	×	1.0	= 61
splice	2Pls. 12½×3/8	@	15.94	×	1.25	= 40
"	4Pls. 6×3/8	@	7.65	×	2.6	= 77
at bearing	4Ls 6×6×½	@	19.6	×	1.6	= 125
						4945
	Rivet heads & c		5%.			245
						5190 # ÷ 33.15 = 156# per lin ft.
For Curvature of bottom flange					add	500
						5690 #

Bottom lateral Bracing.

30° skew. For design say ~~the structure~~ as square.

diagonal length = $\sqrt{10.3^2 + 9^2}$ = 13.7'

$\sec\theta = \frac{13.7}{9.0} = 1.52$



Inside girder.

640 × 10.3 =	6600	
	6355	
	12955 #	for intermediate panel
640 × 6.27 =	4020	
	4155	
	8175 #	for end panel.

Outside girder

770 × 10.3 =	7950	
	4550	
	12500	for intermediate point
770 × 6.27 =	4830	
	3060	
	7890	for end panel point.

For intermediate panel point 2@ 12955 = 25910
2@ 12500 = 25000
Hor. force due to Earthquake - 50910 × 0.4 = 20300 #

For End panel point 2@ 8175 = 16350
2@ 7890 = 15780
Hor. force for Earthquake 32130 × 0.4 = 12800 #
End reaction 33100 #

End shear for diagonal 20300 #
For one diagonal 20300 ÷ 3 = 6770 #
Stress in diagonal 6770 × 1.52 = 10300 # T.
Section required = $\frac{10300\#}{17067 \times 1.8}$ = 0.340 □" net.

Use 1L 3×3×5/16 = 1.780 □" − .28 = 1.500 □" net.
$\ell/r = \frac{165}{0.92}$ = 180 times. Use 3-3/4" Risolto ✓
Approximate weight.

2Ls 3×3×5/16	@ 6.1	× 13.0	=	158
Center connection.	1		=	12
end connection	4@ 20		=	80
			250	add 15 = 265 #

9@ 265 = 2385 #

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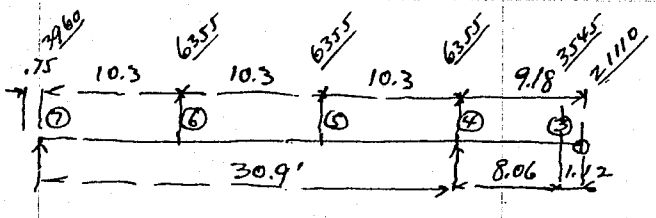
CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo.

Approximate weight of Suspended span

Stringers	153 @ 33.15	=	5070
Floor beams	4 @ 2306	=	9224
Inside girders	2 @ 5365	=	10730
Outside girders	2 @ 5190	=	10380
Lateral Bracing			2385
			37789 ÷ 2240 = 16.90 tons

Design of Anchor span
Inside girder G1 span length 30.9' Cantilever arm 9.18'
Dead Load.



Concentration ⑤-⑥ 6355# see page 6.
Concentration at ③ 4.03 + 1.12 = 5.15
From CB1 520 × 5.15 = 2680 2680 ÷ 2 = 1340
Cross beam 360
1700#

520 ÷ 2 = 260
288
548 × 5.15 = 2830
Cross beams 720
3550# →

From CB2 288 × 5.15 = 1485
cross beam 360
1845
1700
OK. ← 3545#

Concentration at ④ length 9.18'
for extra strut across ~~the~~ shoe and deeper girder assume the panel load same as for ⑤
Concentration at ⑦ length 5.9
panel load 548 × 5.9 = 3240
cross beams 720
3960#

Concentration at hinge. Dead load reaction from suspended span 21110# see p8
moments at various points

3. 21110 × 1.12 = - 23,600 ft# neg
4. 21110 × 9.18 = 193500
3545 × 8.06 = 28600
222,100 ft# neg.

moment at 5 6355 × 10.3 = + 65,500
222,100 × 2/3 = - 148.000
- 82500 neg.

moment at 6 6355 × 10.3 = + 65.500
222,100 × 1/3 = - 74000
- 8,500 ft# neg.

Uniform dead load on girder 640# per lin ft.
at 3. 640 × 1.12 = 720# m = 720 × 0.56 = 404 ft# neg
at 4 640 × 9.18 = 5880 m = 5880 × 4.59 = 27000 ft# neg.
at 5 m = 1/2 × 640 × 10.3 × 20.6 = + 67,800 ft#
27000 × 2/3 = - 18000
+ 49,800 ft#
at 6. m = + 67800
27000 × 1/3 - 9000
+ 58800 ft#

Summary for moment

	3	4	5	6	7
Due to Concentration	- 23600	- 222.100	- 82500	- 8500	0
Due to uniform	- 404	- 27.000	+ 49800	+ 58800	0
	- 24004	- 249100	- 32700	+ 50300	0.

End shears and reactions.: —
Cantilever arm at hinge 21110
panel concentration 3545
uniform load 5880
30535# /

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CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

Shear between 4 and 5
Shear as simple beam 6355
From cantilever effect 249100 ÷ 30.9 = 8050
222100 14405 #
13555

Shear between 6 and 7
Load on ⑦ as simple beam 6355
3960
10315 #
7200
From cantilever ~~249100~~ ÷ 30.9 = 8050
222100 + ~~3265~~ #
3115

As shear 3960
480
4440 #

12595
14745 - 4440 = 8155
7305 #

Load on pier ⑭
due to concentration. Cantilever arm 30535
Cantilever effect 8050
As simple beam 6355
direct load. 6355
9900
due to uniform. ~~with cantilever effect~~ 16450
640 × 30.90/2 = ~~68245~~ #
61195 #

Uniform 640 × 40.08 × 20.04/30.9 = 16950
- 640 × 9.18 = - 5880
11070 #
14405 13555
25475 #
24625

Total.
Uniform 640 × 40.08 × 10.86/30.9 = say 9000
projection 640 × .75 = 480
+ 9480
3115
2265
+ 14745 12595

Loo. load on shoe.

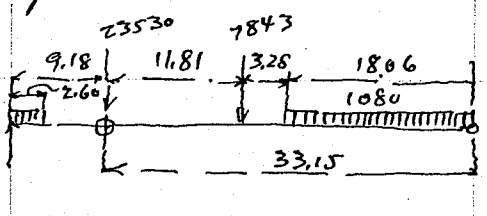
Live Load.

Cantilever moment and max negative at panel point of simple span
motor truck loading
Span length 38.90
Cantilever 9.18
Suspended 33.15
73.23 for full loading.
For cantilever arm = 42.33.

impact for cantilever arm = 20 / (60 + 42.33/328) = 27.5%.

impact for simple span = 20 / (60 + 30.9/32.8) = 28.8%.

for full loading = 20 / (60 + 73.23/328) = 24.4%.



motor truck rear wheel 9920
27.5% 2720
12640 #

1080 × 18.06 = 19500 #
1080 × 2.60 = 2810 #

See page 8

Load on girder loading same as for suspended span see page 7

A	12640 ÷ 2	= 6320
B	12640 × 7.60 ÷ 9.0	= 10700
C	12640 × 4.64 ÷ 9.0	= 6510
		23530 #

Front wheel 23530 ÷ 3 = 7843 #

Reaction at hinge. 7843 × 21.34 ÷ 33.15 = 5050
23530
28580

Unif. load 19500 × 9.03 ÷ 33.15 = 5320
33900 #

moment at pier = 33900 × 9.18 = 311,000
2810 × 1.30 = 3660
36710 #
314660 #

neg moment at ③ = 33,900 × 1.12 = -38,000 #

max shear = 36710 #

Rear wheel at hinge, uniform load at rear see p8
Uniform load 28700 × 13.29/33.15 = 11500
motor truck 23530
35030 #

moment at pier = 35030 × 9.18 = 321000 # max.

End shear = 35030 #

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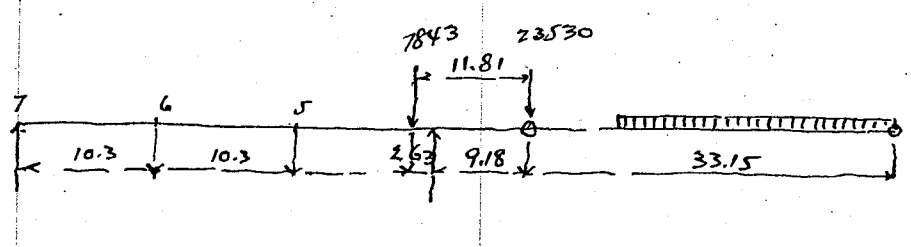
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

moment at panel points 5 and 6.



Reaction = 7843 × 2.63/30.9 = 668 #

moment at 6 668 × 10.3 = 6880
321000 × 1/3 = − 107000
− 100120 ft

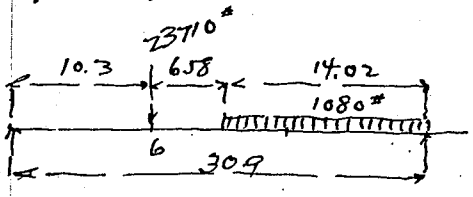
moment at 5 13760
321000 × 2/3 = 214000
− 200240 ft

Rear wheel at hinge and uniform load at rear and front.
moment at 6 314660 × 1/3 = − 104700 ft
at 5 = − 209400 ft } max negative.
max negative reaction at 7 314660 ÷ 30.9 = − 10200 # max.

321000 ÷ 30.9 = − 10400 #
+ 668
9732 # −

Loaden shoe (abutment).
~~Max~~ Dead load - + 12,595
− 10200
2,395 # positive reaction.

Live Load on simple span for positive moments.
motor truck loading impact taken same as for suspended span
panel point 6.



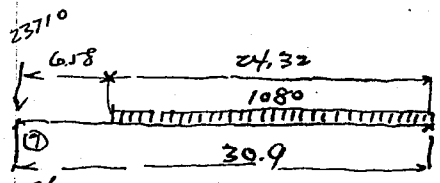
1080 × 14.02 = 15150 #

Reaction 23710 ÷ 3 = 7900 #
15150 × 7.01/30.9 = 3430
11330 #

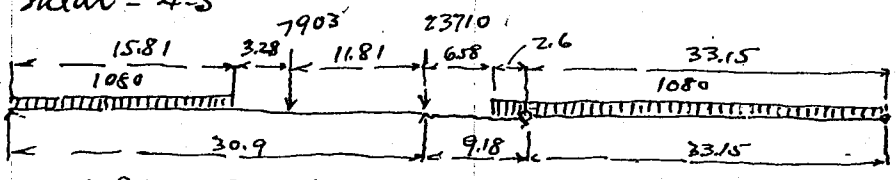
moment = 11330 × 10.3 = 117,000 ft

panel point 5. same bending moment assumed

Max Excd shear at 7 uniform load 1080 × 24.32 = 26200
Reaction = 26200 × 12.16/30.9 = 10300
23710
34010 #



Shear - 4-5



positive shear for simple span
7903 × 19.09/30.90 = 4880
23710
28590

1080 × 15.81 = 17100

Unif. 17100 × 7.90/30.9 = 4370
32960 #

Extra shear due to cantilever moment.
1080 × 2.60 = 2810 #
End reaction at hinge 1080 × 33.15/2 = 17900 #
moment = 17900 × 9.18 = 164,000
2810 × 7.88 = 22100
186,100

Extra reaction 186,100 ÷ 30.9 = 6020
38980 #

Load on 4 (pier.) positive shear 38980 shear 4-5
Uniform. 17900
" 2810
59690 #

motor truck on suspended span shear 36710 #
Extra load on pier 314660 ÷ 30.9 = 10200
46,910 #
Uniform load on simple span 1080 × 30.9/2 = 16700
63610 # max load on pier

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CALCULATIONS FOR

Design of 108 ft Bridge for City of Tokyo

Summary for moments Dead Load and negative LL moment

	7	6	5	4	3
Dead Load	0	+ 50300	- 32700	- 249100	- 24004
Live Load -	0	- 104700	- 209400	- 321.000	- 38000
		- 54400	- 242100	- 570100 ft	- 62004

Dead Load and pos. LL. moment

	7	6	5	4
Dead Load	0	+ 50300	- 32700	- 249100
Live Load	0	+ 117000	+ 117000	0
		+ 167300 ft	+ 84300 ft	- 249100

Summary for shears and loads.

	7-6	5-4	4-3	Load on 7 max	Load on 7 min.	Load on pier.
Dead Load	~~8155~~ + 7305	~~24625~~ + 25475	+ 30535	~~12595~~ + ~~4745~~	12595 + 4745	61195 + ~~68245~~
Live Load	+ 34010	+ 38980	+ 36710	+ 34010	- 10200	+ 63610
	~~41315~~ 42165	~~64455~~ * 63605	67245 ft	+ ~~45755~~ * 46605	+ ~~1545~~ * 2395	+ ~~131.855~~ 124805

Section of Inside girder. G1

Cantilever portion max. moment = 570.100 ft*

Depth = 4'6 1/2" b to b of angles. web 54 × 3/8 = 20.25 1/8 web = 2.530"

Effective depth of girder. 4.27' Flange stress = 570.100 ÷ 4.27 = 133.500 #

Section req'd = 133500 ÷ 17067 = 7.83 o" net

2.53

5.30 o" net.

Use 2 ls 6×6×3/8 = 8.72 - 1.41 = 7.31 o" net.

2.53 2.53

11.25 o. 9.84 o"

Unit flange stress for tension = 133500 / 9.84 = 13600 #/o. tension

Unit flange stress for compression = 133500 / 11.25 = 11900 #/o. comp.

Shear 33 ton

At panel point 5. Tension in top 242100

1/2 Reversal 42150

284250 ft* tension on top.

Depth of girder at 5 assumed 27 1/2" b to b of angles. web 27 × 3/8 = 10.12 o"

1/8 web = 1.27 o" Effective depth 2.02'

Flange stress = 284250 ÷ 2.02 = 141.000 #

Section req'd = 141.000 ÷ 17067 = 8.27 o"

1.27

7.00 o" net

Use 2 ls 6×6×3/8 = 8.72 - 1.41 = 7.31 o" net.

1.27 1.27

9.990" gr. 8.58 o" net.

Unit flange stress for tension = 141.000 / 8.58 = 16450 #/o.

Unit flange stress for compression = 141.000 / 9.99 = 14100 #/o.

Allowable 15000

Shear 62.90 ton

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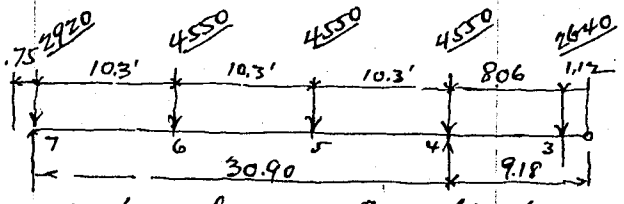
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

Design of Anchor span

Outside girder G2. span length 30.9' Cantilever arm 9.18'

Dead Load.



Uniform load 770# per lin. ft.

Concentration at hinge = 20360# p.10.

Concentration at 5 and 6 4550#
Concentration at 4 assumed same
Concentration at 3.
368 × 5.15 = 1890
Cross beam & Cantilever 750
2640#

Concentration at 7 368 × 5.90 = 2170
Cross beam and cantilever 750
2920#

Moments at various points
moment due to panel concentration.

panel pt 3 m = 20360 × 1.12 = 22800#

panel pt 4 m = 20360 × 9.18 = 187000
2640 × 8.06 = 21300
208300#

panel pt 5 m = 4550 × 10.3 = +47000
208300 × 2/3 = −138500
−91500#

Panel pt 6 m = 4550 × 10.3 = 47000
208300 × 1/3 = −69500
−22500#

moment due to uniform load

③ m = 770 × 1.12 = 860#
860 × 0.56 = 480#
load 770 × 9.18 = 7070#
④ m = 7070 × 4.59 = 32400#

⑤ m = 1/2 × 770 × 10.3 × 20.6 = 82,000#
32400 × 2/3 = −21600
+60400

⑥ m = 82.000
32400 × 1/3 = −10800
+71200#

Summary for moments

	3	4	5	6	7
Due to Concentration	−22800	−208300	−91500	−22500	0
Due to uniform load	−480	−32400	+60400	+71200	0
	−23280	−240700	−31100	+48700	0

End shears and reactions.

Shear for Cantilever arm

Conc at hinge	20360
Concentration	2640
Uniform load	7070
	30070#

Shear between 4 and 5

Shear as simple beam 4550 Unif. 770 × 40.08 × 20.04/30.90 = 20.000
Cantilever effect 208300 ÷ 30.9 = 6750 770 × 9.18 = −7070
11300# 12930#
Due to Concentration 11300
24230#

Shear between 6 and 7.

Shear as simple beam 4550 Unif. 770 × 40.08 × 10.86/30.90 = +10850
Cantilever effect. −6750
−2200 Due to Concentration −2200
+8650

Load on shoe 8650
Concentration 2920
Unif. 770 × .75 = 580
+12150#

Load on pier due to Concentration Cantilever arm 30070
Cantilever effect. 240700 ÷ 30.90 = 7780
Concentration 2@4550 = 9100
Uniform load 770 × 30.9/2 = 11900
58850#

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CALCULATIONS FOR

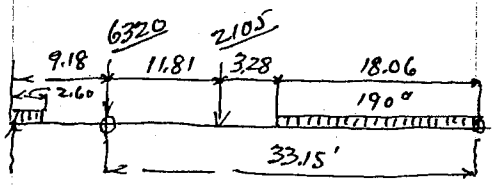
Design of 108A Bridge for City of Tokyo
Live Load
Cantilever moment and max negative moments at panel pts of simple span.

Motor truck loading rear wheel 9920
27.5% impact 2720
12640#

Front wheel 12640 ÷ 3 = 4210#
load on girder Rear wheel 12640 ÷ 2 = 6320#
Front wheel 4210 ÷ 2 = 2105#

Uniform live load at rear or front of motor truck 190# per lin. ft.
" " " on side walk 860# per lin ft. (see page 9)

Cantilever portion

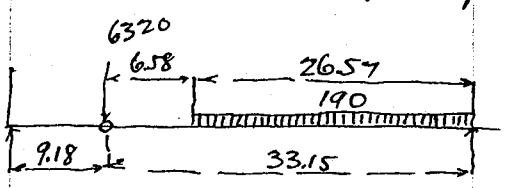


190 × 18.06 = 3430.
190 × 2.60 = 493
½ span 860 × 33.15/2 = 14250
860 × 9.18 = 7900

Reaction at hinge = 2105 × 21.34/33.15 = 1355
6320
7675
Unif. 3430 × 9.03/33.15 = 935
"
14250
22860# neg moment at ③
moment at pier. 22860 × 9.18 = 210000 22860×1.12=25600#
493 × 1.30 = 640
7900 × 4.59 = 36200
31253#
246840 11# max.

Shear = 31253# max

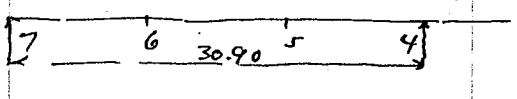
Rear wheel at hinge, followed by uniform load.



190 × 26.57 = 4850

load at hinge = 4850 × 13.28/33.15 = 1945
14250
motor truck 6320
22515#
Moment at pier 22515 × 9.18 = 206500
7900 × 4.59 = 36200
30415# 242700 1#

negative moments at panel points 5 and 6

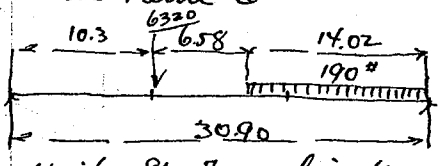


neg. m at 5 = 246840 × 2/3 = − 166000 1#
neg m at 6 = " × 1/3 = − 83.000 1#

negative reaction at 7. = 246840 ÷ 30.90 = − 7980#

Load on shoe (min) Dead load + 12150
Live load − 7980
+ 4170 on abutment

Live Load on Simple Span for positive moments.
Panel Point 6

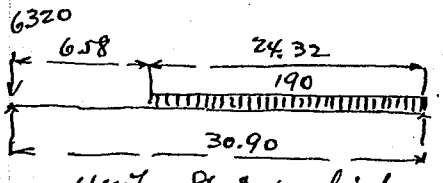


Unif. 860# per lin. ft.
190 × 14.02 = 2660.

Reaction 6320 ÷ 3 = 2110
2660 × 7.01/30.90 = 603
2713
moment = 2713 × 10.30 = 27900 1#
½ × 860 × 10.3 × 20.6 = 91200
119100 1#

Panel Point 5 Same bending moment assumed.

Max End Shear at 7



Unif. 860# per lin ft.

Unif. load = 190 × 24.32 = 4610.0#
Reaction 4610 × 12.16/30.90 = 1815.
Unif. load 860 × 15.45 = 13300
motor truck 6320
21435#

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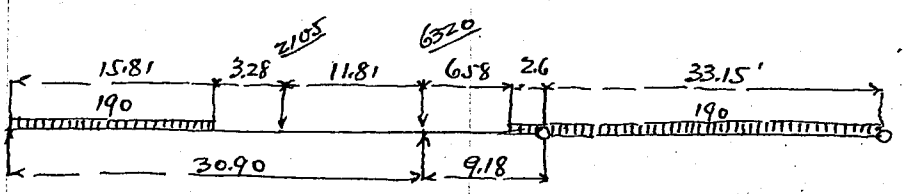
CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo.
Shear 4-5



860# per lin ft.

190 × 15.81 = 3000# 190 × 2.60 = 493

Extra shear due to cantilever moment.
Reaction at hinge = 1050 × 33.15/2 = 17400
860 × 9.18 = 7900
moment at pier.
17400 × 9.18 = 159500
7900 × 4.59 = 36300
493 × 7.88 = 3900
25793 Extra shear. 199700 ÷ 30.90 = 6470#
Direct shear. 21687.
max shear 28157#

Load on Pier.
shear 4-5 28157
load on cant. arm 25793
53950#

Motor truck on suspended span.
shear 31253 see page 17
Extra reaction 246840 ÷ 30.9 = 7980

39233
Unif. load on simple span 1050 × 30.9/2 = 16250
55483 max. load on pier.

Reaction = 3000 × 7.91/30.90 = 767
2105 × 19.09/30.90 = 1300
6320
8387#
Uniform load 860 × 15.45 = 13300
Direct shear 21687#

860
190
1050#

Summary for moments

	7	6	5	4	3
Dead Load	0	+48700	-31100	-240700	-23280
Live Load (neg)	0	-83000	-166000	-246840	-25600
		-34300	-197100	-487540	-48880 #
Dead Load	0	+48700	-31100	-240700	
Live Load (pos)	0	+119100	+119100	0	
		+167800	+88000	-240700 #	

Summary for shears and loads.

	7-6	5-4	4-3	On abutment: Load on 7 max.	Load on 7 min.	Load on pier
Dead Load	+8650	+24230	+30070	+12150	+12150	+58850
Live Load.	+21435	+28157	+31253	+21435	-7980	+55483
	30085	+52387	+61323	+33585	+4170	114333

Section of outside Girder G2
Particulars Portion max moment = 487540 #
Depth of girder 5'-5½" about web 65" × 3/8" = 2440" ⅛ web = 3.05 0"
Effective depth = 5.30 flange stress = 487540 ÷ 5.30 = 92000 #
Section req'd = 92.000 ÷ 17067 = 5.40
3.05
2.35 0"

Use 2 LS 6 × 4 × 3/8 = 7.22 - 1.41 = 5.81 ins
3.05 3.05
10.27 8.86

Not Counting ⅛ web for flange section Unit stress tension = 92000/5.81 = 15850 #/□"
" " comp. = 92000/7.22 = 12750 #/□"

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CALCULATIONS FOR

Design of 108A Budge for City of Tokyo.

Panel point nos. moment = 197100 Tension on top flange
½ 44000 Reversal
241100 #

web assumed 38" x 3/8" = 14.25 □" 1/8 web = 1.78 Effective depth = 3.05

Flange stress = 241100 ÷ 3.05' = 79,100 #
Section req'd = 79100 ÷ 17067 = 4.63
2.30
2.33 □" net

use 2L 6.4 x 3/8" = 7.22 - 1.41 = 5.81.

Not counting 1/8 web for flange section

Unit stress (tension) = 79100/5.81 = 13610 #/□"
" " (comp) = 79100/7.22 = 10950 #/□"

Approximate weight of main girder (anchor span).

Inside Girder.

flange	2 L's	6 x 6 x 3/8	@ 14.9	x 40.8	=	1216
"	2 L's	6 x 6 x 3/8	@ 14.9	x 42.0	=	1250
web	1 Pl.	42" x 3/8	@ 53.55	x 4.25	=	228
"	1 Pl.	72 x 3/8	@ 91.80	x 4.25	=	390
"	1 Pl.	36 x 3/8	@ 40.80	x 24.40	=	1000
"	1 Pl.	36 x 3/8	@ 40.80	x 6.70	=	274
Int stiff.	22 L's	5 x 3½ x 5/16	@ 8.7	x 7.75 ave	=	527
	6 L's	5 x 3½ x 3/8	@ 10.4	x 2.25	=	141
fillers	6 Pls.	7½ x 3/8	@ 9.56	x 1.25	=	71
End stiffs	8 L's	5 x 3½ x 3/8	@ 10.4	x 4.25	=	354
fillers	4 Pls.	18" x 3/8	@ 22.95	x 3.25	=	297
web splice	6 Pls.	12½ x 3/8	@ 15.94	x 2.50	=	192
"	12 Pls.	5½ x 3/8	@ 7.01	x 2.0	=	168
flange sp.	2 Pls.	12½ x 3/8	@ 15.94	x 3.0	=	96
"	4 Pls.	5½ x 3/8	@ 7.01	x 3.0	=	84
fillers	2 Pls.	15 x 3/8	@ 19.13	x 2.2	=	84
	2 L's	6 x 6 x 3/8	@ 14.9	x 1.6	=	48
Bed plate	1 Pl.	12 x 3/4	@ 30.60	x 1.0	=	31
"	2 Pls.	12 x 3/4	@ 30.60	x 1.5	=	92
						6543
		3½% variation				230
						6773 # ÷ 40.08 = 169 # per lin ft.

Outside girder.

flange.	2 L's	6 x 4 x 3/8	@ 12.3	x 40.8	=	1005
"	2 L's	6 x 4 x 3/8	@ 12.3	x 42.0	=	1032
web.	1 Pl.	42 53" x 3/8	@ 53.55	x 4.25	=	281
"	1 Pl.	72 83" x 3/8	@ 91.80	x 4.25	=	482
"	1 Pl.	47" x 3/8	@ 59.93	x 24.40	=	1460
"	1 Pl.	47 x 3/8	@ "	x 6.7	=	401
Int. Stiffs	22 L's	5 x 3½ x 5/16	@ 8.7	x 3.65	=	698
"	6 L's	5 x 3½ x 3/8	@ 10.4	x 3.15	=	196
fillers	6 Pls.	7½ x 3/8	@ 9.56	x 2.1	=	120
End stiffs	8 L's	5 x 3½ x 3/8	@ 10.4	x 5.15	=	428
fillers	4 Pls.	18 x 3/8	@ 22.95	x 4.45	=	408
web splice	6 Pls.	12½ x 3/8	@ 15.94	x 3.25	=	311
"	12 Pls.	5½ x 3/8	@ 7.01	x 2.0	=	168

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CALCULATIONS FOR

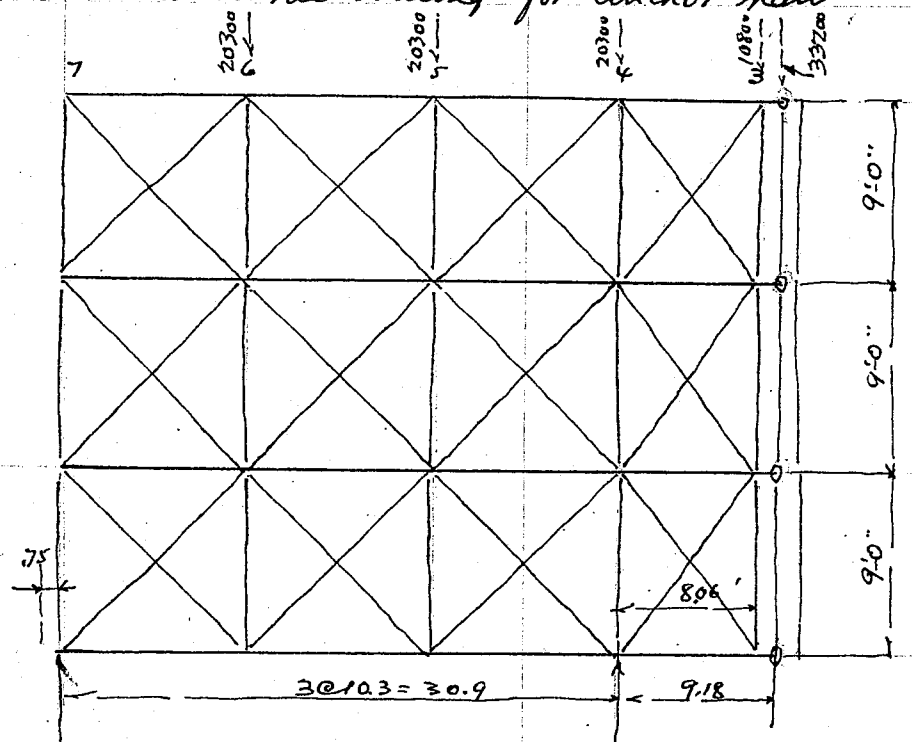
Design of 108 A Bridge for City of Tokyo.

flange pl.	2 Pls.	12½ × 3/8	@ 15.94	× 3.0	=	96
"	4 Pls.	3½ × 3/8	@ 4.46	× 2.0	=	36
fillers	2 Pls.	19 × 3/8	@ 24.23	× 2.2	=	107
	2 Ls	6 × 4 × 3/8	@ 12.3	× 1.6	=	40
Bed Pl.	1 Pl	12 × 3/4	@ 30.60	× 1.0	=	31
"	2 Pls.	12 × 3/4	@ 30.60	× 1.5	=	92
						7392
			3% variation			222
						7614 ÷ 40.08 = 190# per lin ft.

Design the lateral Bracing as square.
diagonal length = 13.7' } for 10.3' panel.
secθ = 1.52

for 8.06' panel.
Diagonal length = $\sqrt{9^2 + 8.06^2}$ = 12.1'
secθ' = 12.10/9.0 = 1.345

Bottom Lateral Bracing for Anchor span



Load at hinge see p11
Inside girder 12955 + 8175 = 21130
Outside girder 12500 + 7890 = 20390
41520

Horizontal force 2@41520 × 0.4 = 33200#

Panel load at 3. inside girder.
Unif. 640 × 5.15 = 3300
Conc 3545
6845#

Panel load at 3 outside girder
Unif. 770 × 5.15 = 3970
Concentration 2640
6610#

Summary for Panel load at 3.
Inside girders 2@6845 = 13690
Outside girders 2@6610 = 13220
Hor. force due to Earthquake 26910 × 0.4 = 10800#
Panel load at 4-5-6 = 20300#, see page 11.

Shear for panels 3-4
33200
10800
44000# ÷ 3 = 14700#

Diagonal stress = 14700 × 1.345 = 19800# T

Shear panel 4-5 20300 from simple span
Extra shear due to cantilever moment
m 33200 × 9.18 = 305.000
10800 × 8.06 = 87.000
392.000 ÷ 30.9 = 12700
20300
33000# ÷ 3 = 11000

Diagonal stress = 11000 × 1.52 = 16700# T

Shear panel 5-6. 12700 ÷ 3 = 4230 Stress = 4230 × 1.52 = 6430# T
Shear panel 6-7 20300 ÷ 3 = 6770 Stress = 6770 × 1.52 = 10300# T

max Reaction at panel point 4. (Horizontal)
Shear at 4 44000 from cantilever
" 20300 " simple beam
Panel load 20300
12700 Extra load due to cantilever.
97300#

max horizontal load at 7.
Inside girder 640 × 5.9 = 3770
3960
7730
Outside girder 770 × 5.9 = 4550
2920
7470

Load Inside girder 2@7730 = 15460
Outside girder 2@7470 = 14940
30400# × 0.4 = 12200

20300
12200
32500# max Hor. load.

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CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo.

Section of lateral 3-4 $\frac{19800}{17067 \times 1.8} = 0.645^{\square}$

Use L 3 × 3 × 9/16 = $1.78^{\square} - 0.28 = 1.50^{\square}$ net Use 3-3/4" rivets for connection.

panels 4-7 use same section.

Approximate weight of lateral Bracing (see page 11).

12 @ 265	=	3180 #
strut at 4 and 7 2@2200	=	4400
extra details at bridge say		1500
		9080 #

Summary weights of Anchor span

Stringers	153 @ 40.	=	6120
floor beams	5 @ 2306	=	11530
Inside girders	2 @ 6773	=	13546
Outside girders	2 @ 7614	=	15228
Lateral Bracings		=	9080
			55504 # ÷ 2240 = 24.8 tons

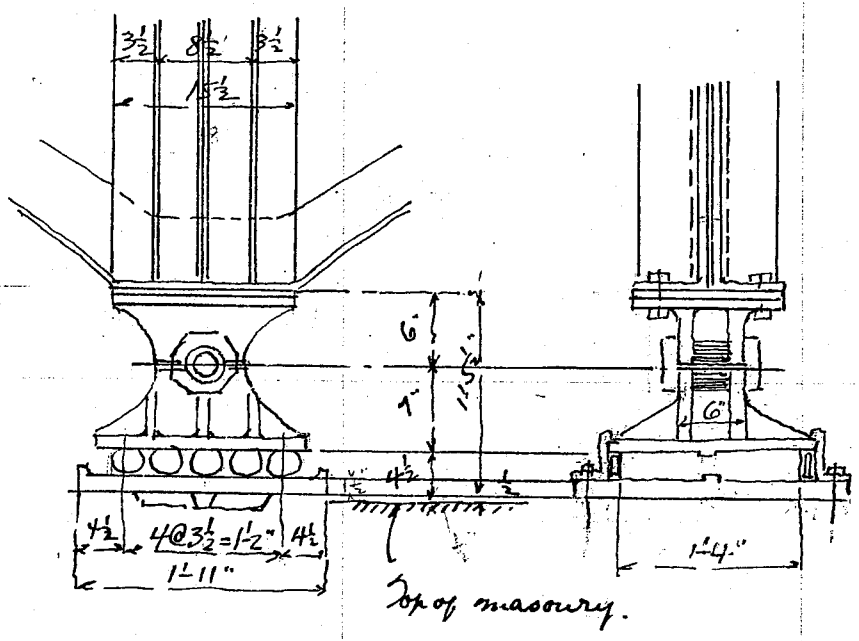
Anchor spans for 2 spans	2 @ 24.8 =	49.60
Suspended span		16.90
shoes and misc details		3.50
		70.00 tons

70.00 ÷ 114.6 = 0.61 ton per tsubo.

Shoes on pin.

Inside girder		Outside girder	
DL.	61195	DL.	58850
LL	63610		55483
	124805 #		114333 #
for shoe	1000	for shoe	1000
	125805		115333
Call this	126000	Call this	116000 #

Design shoes for 126000 # load.



diameter of pin assume 3"
bearing area 3" × 6" = 18"
126000 ÷ 18 = 7000 #/o-

Roller 3" 3" × 610 = 1830 # per lin. inch
Length of Roller 126000 ÷ 1830 = 69"
for 5 rollers 69 ÷ 5 = 13.8" each

Roller out to out = 1'-4"
2-5/8" = 1¼"
Guard 2-4" = 8
Transverse —— 2'-1¼"
Longitudinal 4 @ 3½ = 1'-2"
3"
allowance 2 @ 3" 6
1'-11"

Base area = 25¼" × 23 = 580 □" 126000 ÷ 580 = 217 #/□" on masonry
Fixed shoe use similar base
Shoes on abutment

Inside girder		Outside girder	
DL.	+ 12595	DL.	+ 12150
LL	+ 34010	LL	+ 21435
	46605 #		33585
wt of shoe	700	for shoe	700
	47305		34285

Design shoes for 48000 # load.

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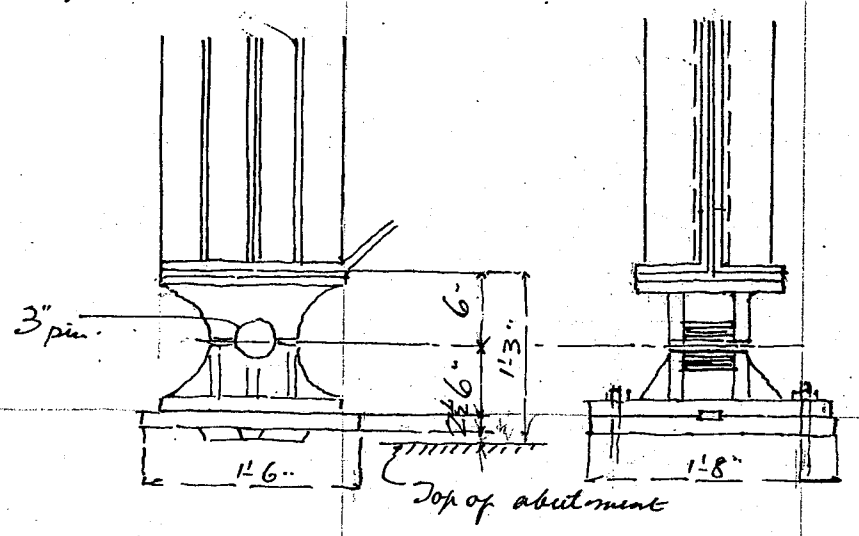
Design of 108A Bridge for City of Tokyo.

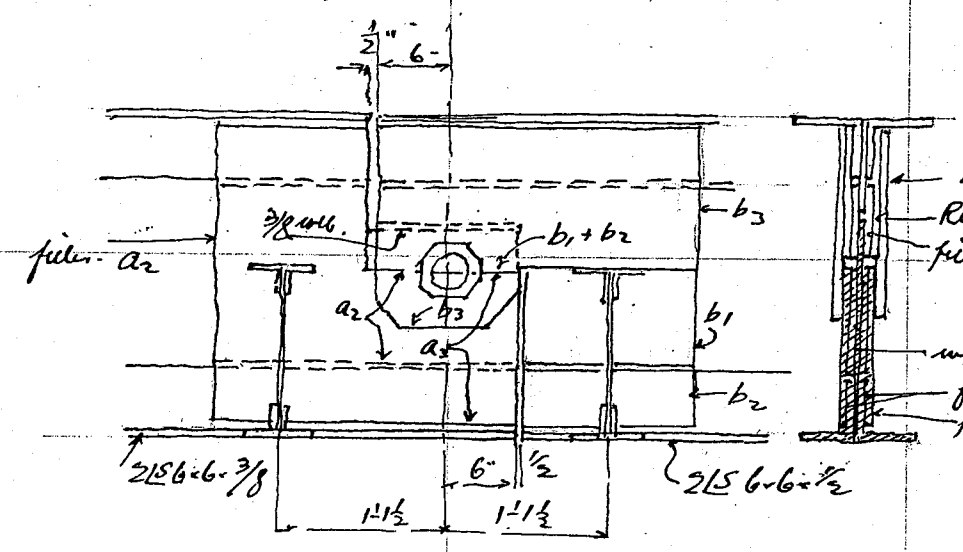
Sketch of shoe on abutment.



Pin 3" dia same as for on pier.
sliding shoe.
Bearing area = 18 × 20 = 360□"

Unit bearing = 48000 ÷ 360 = 134 #/□" on masonry.

Details of hinge for suspended span



Inside girder DL. 21110 ✓
LL 35210 ✓
56320 # ✓

Outside girder DL. 20360 ✓
22920. ✓
43280 # ✓

Bearing area
for anchor span
3/8" web = 3/8 pin plate
fillers 2 @ 3/8 = 3/4
" 2 @ 1/2 = 1"
2 1/8

Suspended span
fillers 2 - 1/2 = 1"
Reinf. Pls. 2 - 3/8 = 3/4
pin Pls. 2 - 3/4 = 3/4
2 1/2"

Pin assumed 2 1/2"
bearing area = 2 1/2 × 2 1/8 = 5.31 □"
unit bearing = 56320 ÷ 5.31 = 10600 #/□"
Bending = 56320 × 3/8" ✓ = 21100 "#
fibre stress = 14000 #/□" about ok

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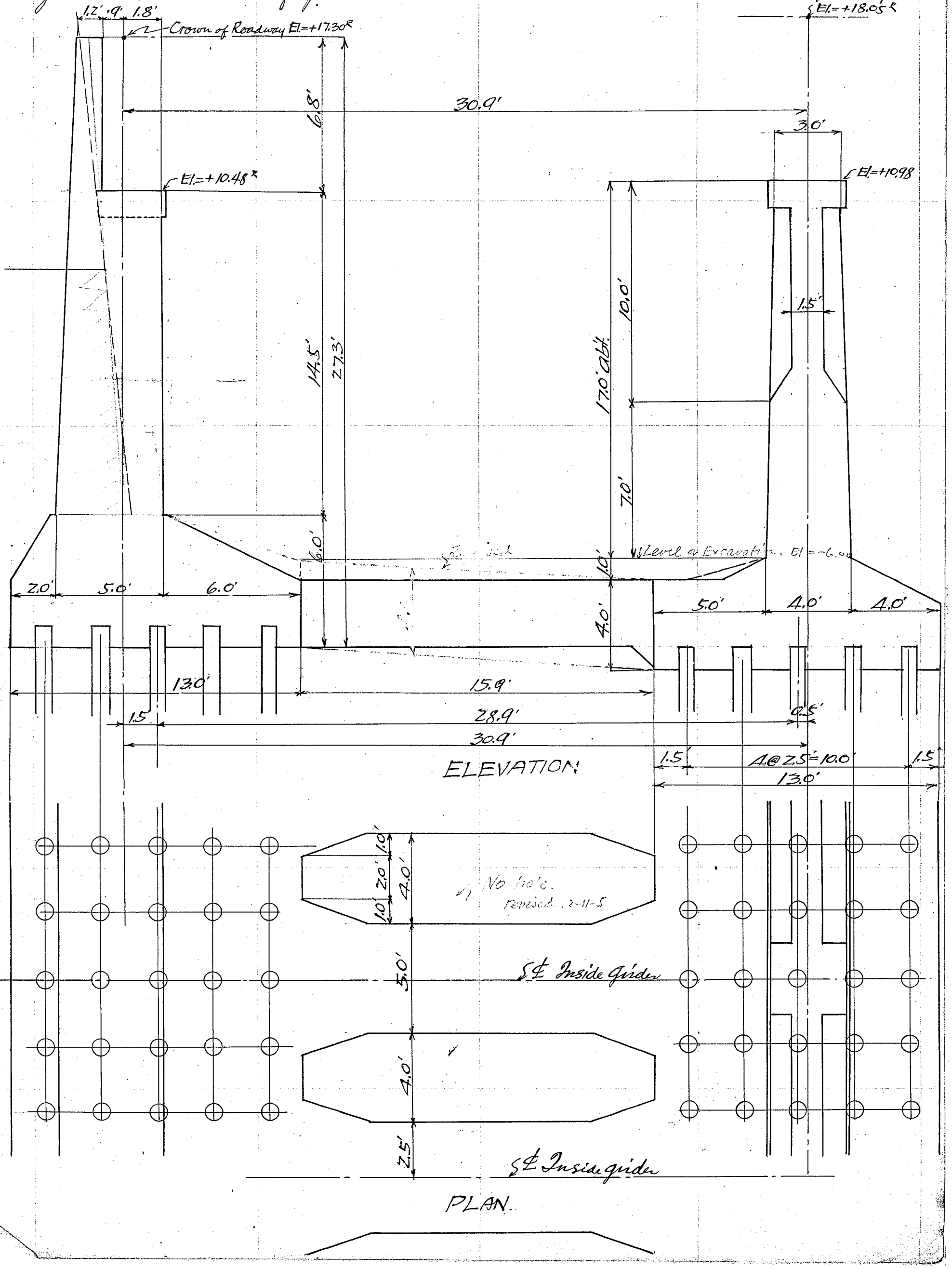
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

General dimensions of Pier and Abutment.



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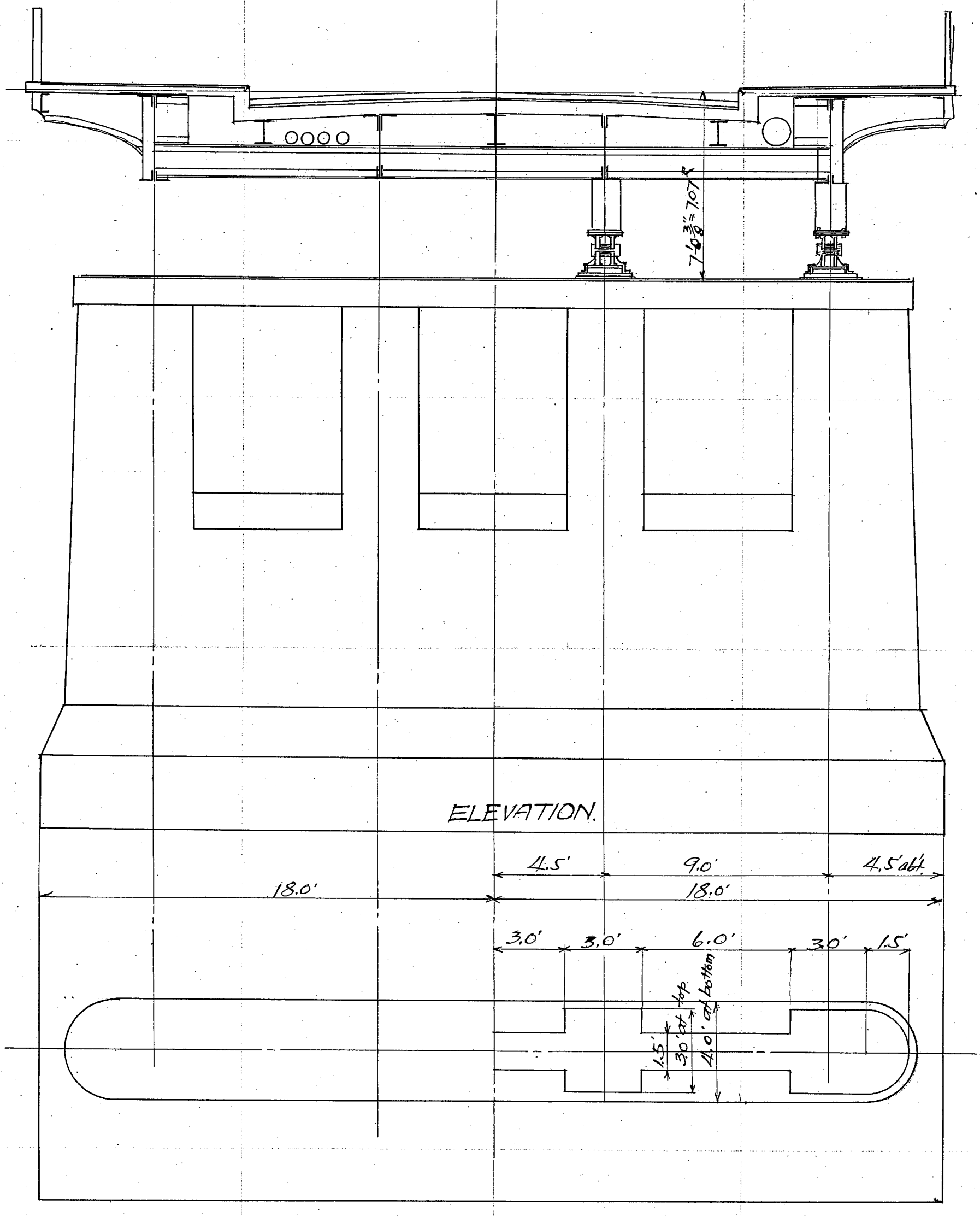
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.
General Sketch of Pier.



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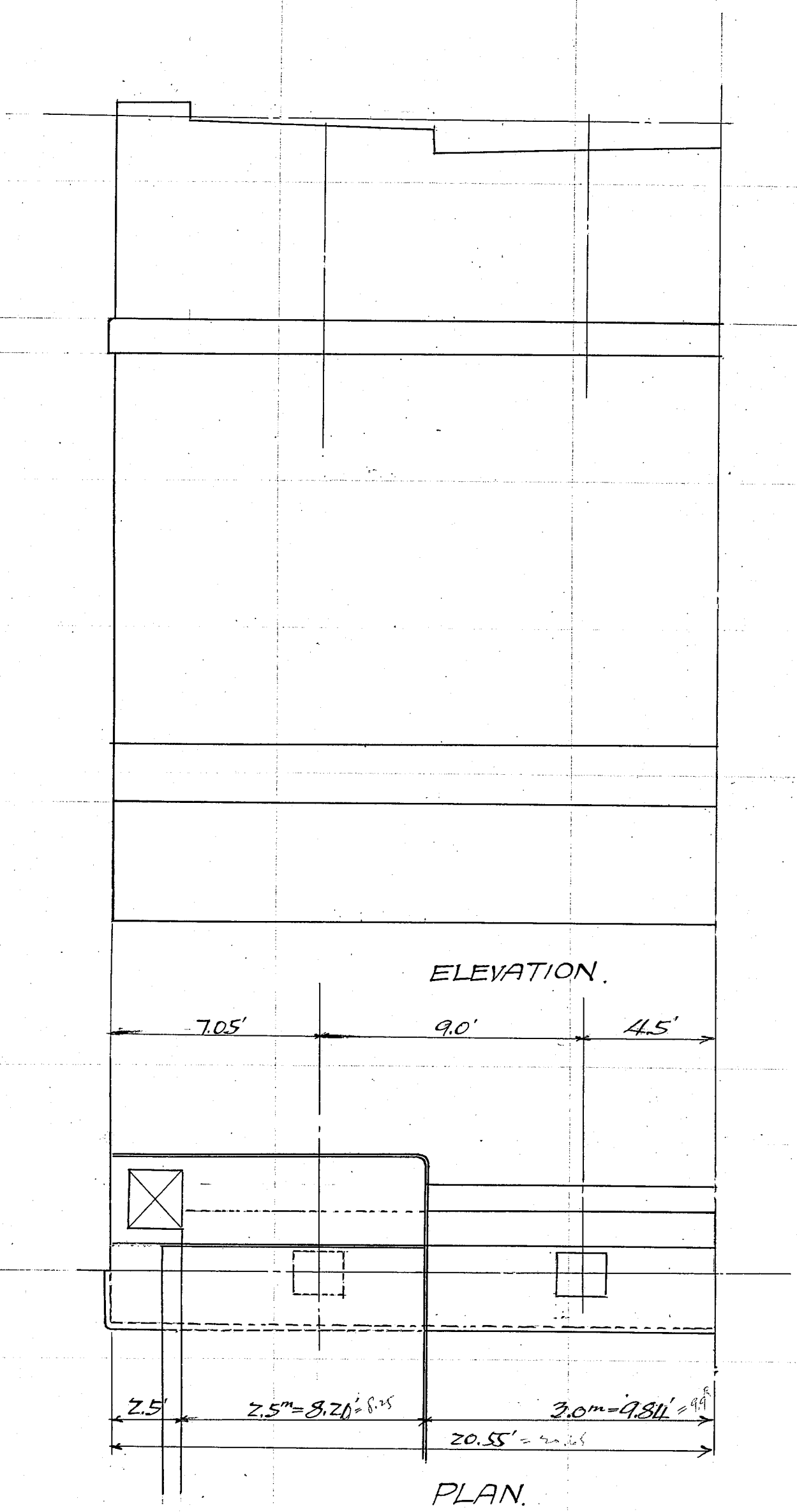
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.
General Sketch of Abutment.



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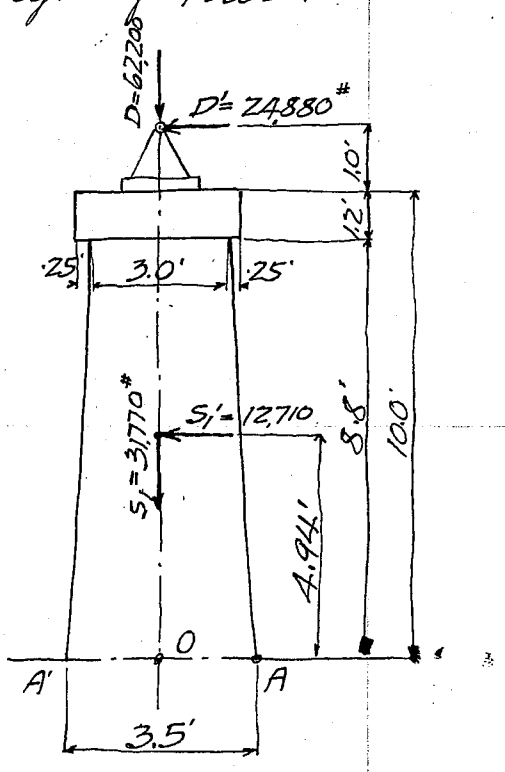
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.
Design of Pier.



Super imposed load.

Dead Load	= 61,195#	Total Dead Load = 62,200# say
Weight of Shoe	= 1,000#	seismic force = 62200×.4 = 24,880#
Live Load	= 63,610#	
	125,805	Call this 126,000# for one girder

Case A.

Stability of Pier shaft during earthquake. K=0.4.

Center of gravity of shaft ~~above~~ above section AA'

Names	Dimensions		Weight		leverarm		moment
Coping	3.5×1.2×9.0	@ 150 =	5,670	×	9.4	=	53,300
Shaft	3.0×3.25×8.8	@ " =	12,870	×	3.93	=	50,500
Curtainwall	1.5×6.0×8.8	@ " =	11,880	×	4.4	=	52,300
Chamfer	1.0×1.5×6.0	@ " =	1350	×	0.5	=	680
			31,770#				156,780'#

Vertical distance of C.g. of shaft from O.

$$= \frac{156,780}{31,770} = 4.94'$$

Seismic force = 31,770 × 0.4 = 12,710#

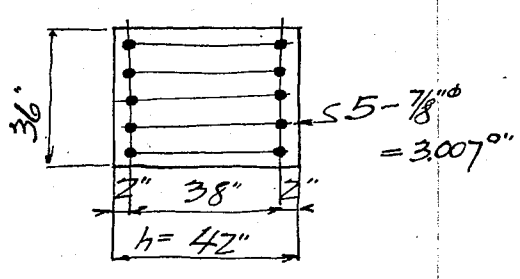
Seismic moment about O.

	Vert load.	Seismic forces		lever arm		moment
D	62,200#	24,880	×	11.0	=	273,700
S_1	31,770	12,710	×	4.94	=	62,800
	93,970#	37,590#				336,500'#

Eccentricity $\varepsilon = \frac{336500}{93,970} = 3.58'$

$\varepsilon/h = 3.58/3.5 = 1.025$, $d'/h = 2/42 = 0.048$

$p_0 = 2p = \frac{3.007 \times 2}{36 \times 42} = 0.00398$



K = 0.283, L = 0.1005, j = 0.906

$f_c = \frac{M}{Lbh^2} = \frac{336500 \times 12}{0.1005 \times 36 \times 42^2} = 633$ #/□" OK

$f_s = nf_c\left(\frac{d}{kh} - 1\right) = 15 \times 633\left(\frac{40}{.283 \times 42} - 1\right) = 22,450$ #/□" OK (<30,000)

Unit Shear $v = \frac{37590}{36 \times .906 \times 40} = 28.8$ #/□" OK

unit bond $u = \frac{37590}{2.75 \times 5 \times .906 \times 40} = 75.4$ #/□" OK.

Case B.

Stability of Pier shaft for full load at normal state.

Super imposed load = 126000#

No bending moment occurs in the shaft.

unit compression on concrete $= \frac{157,770}{36 \times 42 + 3.007 \times 14 \times 2} = 99$ #/□" OK

" " " Steel = 99 × 15 = 1,481 #/□" OK.

Load on shaft

Super imposed load = 126,000#	
Shaft	31,770
	157,770

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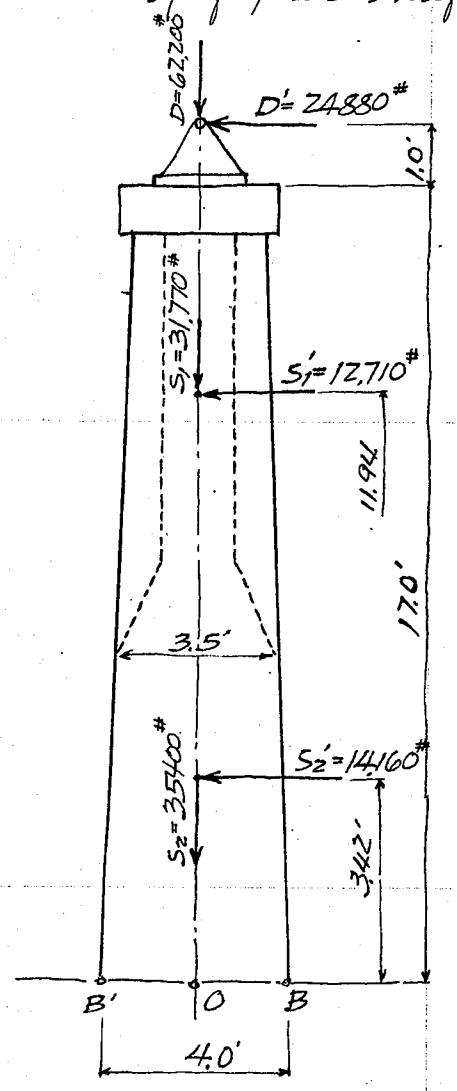
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.
Stability of Pier Shaft at Bottom.



Case A. Stability during earthquake.
Center of gravity of shaft of lower 7 ft.

$$x_1 = \frac{7(4.0+2\times3.5)}{3(4+3.5)} = \frac{77}{3\times7.5} = 3.42' \text{ above O.}$$

Weight = 3.75 × 9 × 7 @ 150 = 35,400#
Seismic force = 35400 × .4 = 14,160#
Seismic moment about O.

Loads		Seismic forces		arm		moment
D 62,200	D'	24,880#	×	18.0	=	447,500'#
S_1 31,770	S_1'	12,710	×	11.94	=	151,700
S_2 35,400	S_2'	14,160	×	7.00	=	99,100
129,370#		51,750#				698,300'#

Eccentricity $\varepsilon = \frac{698.300}{129,370} = 5.40'$

$\varepsilon/h = \frac{5.4}{4.0} = 1.35$, $d'/h = \frac{2}{48} = 0.0042$

$p_0 = 2p = \frac{2\times6.013}{48\times108} = 0.00232$

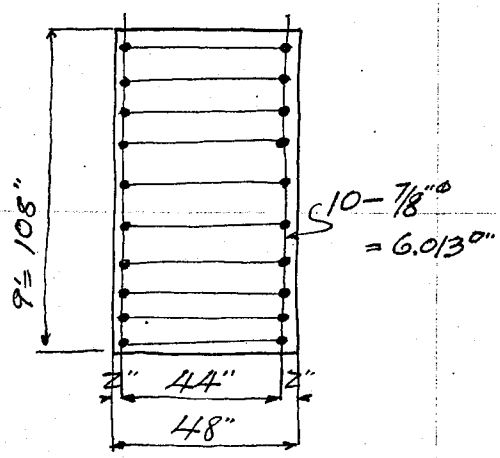
K = 0.217, j = 0.928, L = 0.08

$f_c = \frac{698.300\times12}{0.08\times108\times48^2} = 421$ #/□" OK.

$f_s = 15\times421\left(\frac{46}{.217\times48}-1\right) = 21{,}600$ #/□" OK

unit shear $v = \frac{51750}{108\times.928\times46} = 11.2$ #/□" OK

unit bond $u = \frac{51750}{2.75\times10\times.928\times46} = 43.9$ #/□" OK.

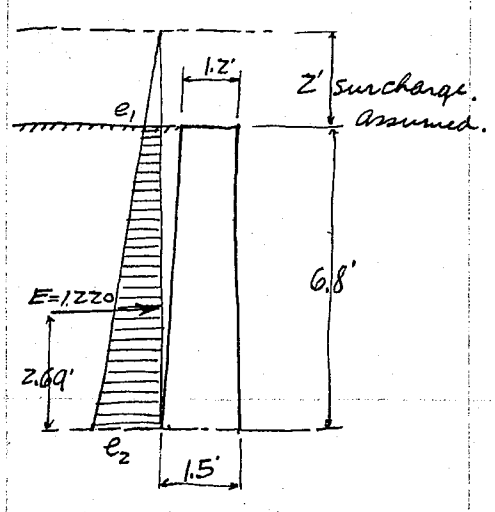


Case B.
Stability for full load at normal state.
Super imposed load = 120,000#
Shaft 31,770
35,400
193,170#

No bending moment occurs in the shaft
Direct compression on Concrete = $\frac{193,170}{108\times48+14\times6.013\times2} = 36$ #/□" OK

$f_s = 36\times15 = 540$ #/□" Comp. OK.

Design of Abutment.
parapet wall.
Case A. Under full load at normal state.



2 ft. surcharge for live load assumed.
earth pressure $e_1 = \frac{100\times2}{3} = 67$#/□', $e_2 = \frac{100\times8.8}{3} = 293$#/□'
Total earth pressure on parapet wall = $\frac{67+293}{2}\times6.8 = 1220$ #/ft strip.
moment at bottom = 1220 × 2.69 = 3280'#/ft. strip
steel required = $\frac{3280\times12}{17067\times\frac{7}{8}\times16} = 0.165$□" per ft. strip.

use 1-½"φ bar = 0.1963□" per ft. strip
$p = \frac{0.1963}{16\times12} = 0.00102$, j = 0.947, K = 0.158
$f_s = \frac{3280\times12}{0.1963\times0.947\times16} = 13{,}230$ #/□" OK
$f_c = \frac{13230\times.158}{15(1-.158)} = 166$ #/□" OK
$v = \frac{1220}{12\times.947\times16} = 6.7$ #/□" OK
$u = \frac{1220}{1.571\times.947\times16} = 51.2$ #/□" OK.

point of application of E
$x = \frac{6.8(293+134)}{3(293+67)} = 2.69'$

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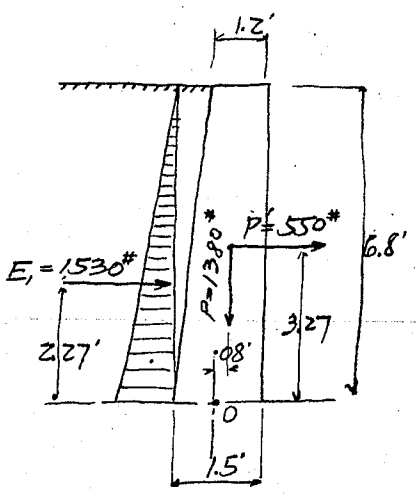
CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo.
Design of Abutment continued.
Parapet wall Case B. Stability during earthquake.



Coefficient of earth pressure for 3/4 K = .44 × 3/4 = .3.
= 0.662

Earth pressure = $0.662 \times \frac{100 \times 6.8^2}{2}$ = 1,530#
Weight of parapet wall = 1.35 × 6.8 @ 150 = 1,380
moment due to

earth pressure	1530 × 2.27 =	3,470 '#
Seismic force	550 × 3.27 =	1,800
wt. of wall	1380 × 0.08 =	110
		5,380 '#

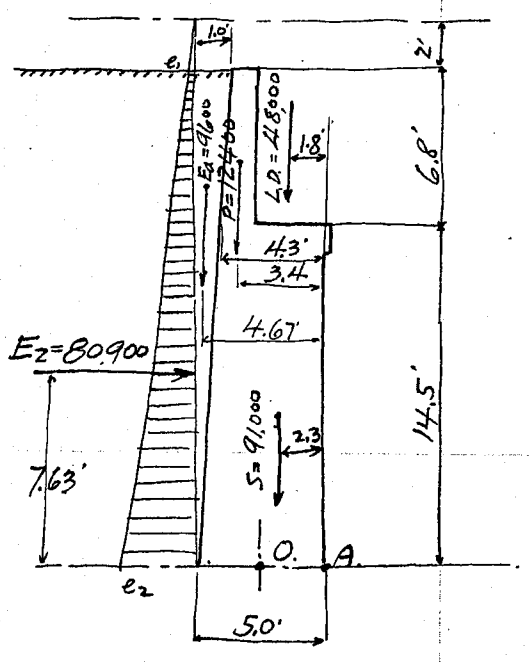
Steel req'd. = $\frac{5380 \times 12}{30700 \times \frac{7}{8} \times 16}$ = 0.151 0" per ft. strip.

Previous assumed section OK.

Stability of Abutment shaft.
Shaft under inside girder.
Case A Stability under full load at normal state.

Super imposed loads.

Dead load	12,595#		
wt. of shoe	700		
	13295#	Call this	13,300#
Live load	34,010	"	34,000
			47,300 Call this 48,000#



Let us figure for shaft 9' wide under inside girder.
Weight of shaft = $\frac{4.3+5.0}{2} \times 14.5 \times 9$ @ 150 = 91,000#
Weight of parapet wall = 1380 × 9 = 12,400

Earth pressure $e_1 = \frac{2 \times 100}{3}$ = 67 #/'
" $e_2 = \frac{23.3 \times 100}{3}$ = 777 '

$E_2 = \frac{67+777}{2} \times 21.3 \times 9$ = 80,900#
wt. of earth behind =
$E_a = \frac{1.0 \times 21.3 \times 9 \times 100}{2}$ = 9,600#

$x = \frac{21.3(777+134)}{3(777+67)} = 7.63'$

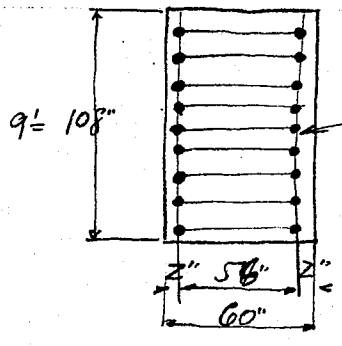
Taking moment about A.

Loads	Hor forces	vert forces	leverarm	moment
Super imposed load		48,000 ×	−1.8 =	−86,400
parapet wall		12,400 ×	−3.4 =	−42,200
Shaft		91,000 ×	−2.3 =	−209,100
Earth behind		9,600 ×	−46.7 =	−448,00
Earth pressure	80,900 ×		+7.63 =	+617,000

ΣH = 80,900# ΣV = 161,000# Σm = +234,500'#

Eccentricity $\varepsilon = \frac{234,500}{161,000} + 2.5 = 1.46 + 2.5 = 3.96'$

Moment about O = 161,000 × 3.96 = 637,500'#
assumed section.



steel ratio $p_0 = 2p = \frac{5.412 \times 2}{60 \times 108} = 0.00167$

$\varepsilon/h = \frac{3.96}{5.0} = 0.792$

$d'/h = \frac{2}{60} = 0.0333$

K = 0.255, j = 0.915, L = 0.0762

$f_c = \frac{637500 \times 12}{0.0762 \times 108 \times 60^2}$ = 259 #/'' OK

$f_s = 15 \times 259 \left(\frac{58}{.255 \times 60} - 1\right)$ = 10,850 #/'' OK

unit shear = $\frac{80900}{108 \times .915 \times 58}$ = 14.1 #/'' OK

" bond = $\frac{80900}{2.749 \times 9 \times 0.915 \times 58}$ = 61.6 #/'' OK

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CALCULATIONS FOR

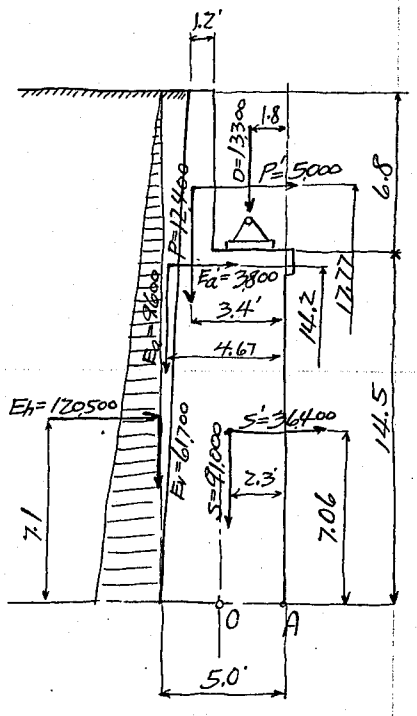
Design of 108 A Bridge for City of Tokyo.

Case B. Stability of shaft during Earthquake for seismic forces acting toward river side.

Earthpressure during earthquake.

Hor. component of earth pressure = $0.295 \times 100 \times 21.3^2 \times 9 = 120{,}500^{\#}$

Vert. " " " " " = $0.151 \times 100 \times 21.3^2 \times 9 = 61{,}700^{\#}$



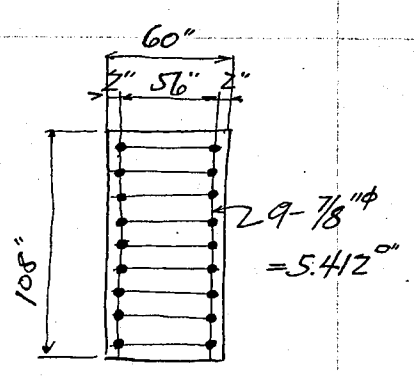
Taking moment about A.

Loads	Hor. forces	Vert. forces	lever arm	moment
D		13,300	−1.8	− 23,930 '#
P		12,400	−3.4	− 42,150
P'	5,000		+17.77	+ 88,850
S		91,000	−2.3	− 209,200
S'	36,400		+7.06	+ 257,000
Ea		9,600	−4.67	− 44,800
Ea'	3,800		+14.2	+ 54,000
Ev		61,700	−5.0	− 308,500
Eh	120,500		+7.1	+ 855,000
	ΣH=165,700#	EV=188,000#		ΣM = +626,270 '#

Eccentricity $\Sigma = \frac{626270}{188{,}000} + \frac{5.0}{2} = 3.33 + 2.5 = 5.83'$

Moment about O. = $188{,}000 \times 5.83 = 1{,}096{,}000$ '#

$e/h = \frac{5.83}{5.00} = 1.17$, $d'/h = \frac{2}{60} = .0333$



$p_o = 2p = \frac{5.412 \times 2}{60 \times 108} = 0.00167$

From prepared diagrams, we have

$K = .203$, $j = .933$, $L = .073$

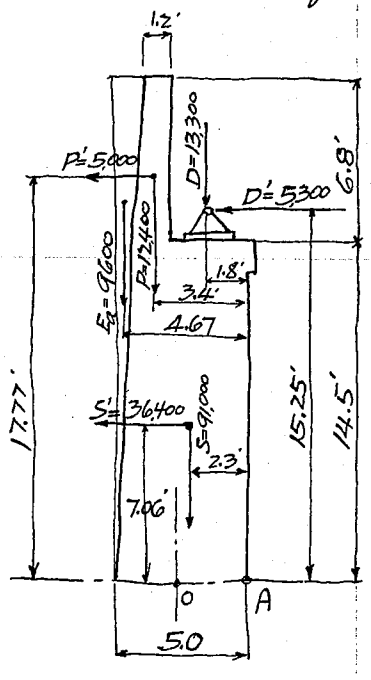
$f_c = \frac{M}{Lbh^2} = \frac{1096000 \times 12}{0.073 \times 108 \times 60^2} = 464$ #/□" OK

$f_s = n f_c \left(\frac{d}{Kh} - 1\right) = 15 \times 464 \left(\frac{58}{0.203 \times 60} - 1\right) = 26200$ #/□" < 30700 OK.

Shear stress $v = \frac{165.700}{108 \times .933 \times 58} = 28.4$ #/□" OK

bond stress $u = \frac{165{,}700}{2.749 \times 9 \times .933 \times 58} = 123.5$ #/□" < 153 OK.

Case C. Seismic forces reversal to case B.



Taking moment about A.

Loads	Hor. forces	Vert. forces	lever arm	moment
D		13,300	−1.80	− 23,900
D'	5,300		−15.25	− 80,800
P		12,400	− 3.40	− 42,150
P'	5,000		−17.77	− 88,850
S		91,000	−2.30	−209,200
S'	36,400		−7.06	−257,000
Ea		9,600	−4.67	− 44800
	ΣH=46,700	ΣV=126,300		ΣM = −746,730 '#

Eccentricity $\Sigma = \frac{746{,}730}{126{,}300} - 2.5 = 5.9 - 2.5 = 3.4'$

Moment about O = $126{,}300 \times 3.4 = 429{,}500$ '#

Since all loads & moments smaller than those for case B, it is unnessary to investigate stresses.

Shaft under outside girder. Superimposed loads.

Dead Load =	12,150#	
Weight of shoe =	700	
	12,850#	Call this 12,900#
Live Load	21,435	, 21,500
		Total = 34,400 '#

. Use similar details as for shaft under inside girder.

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MADE BY K. Inaba DATE 2-10-13 FILE NO
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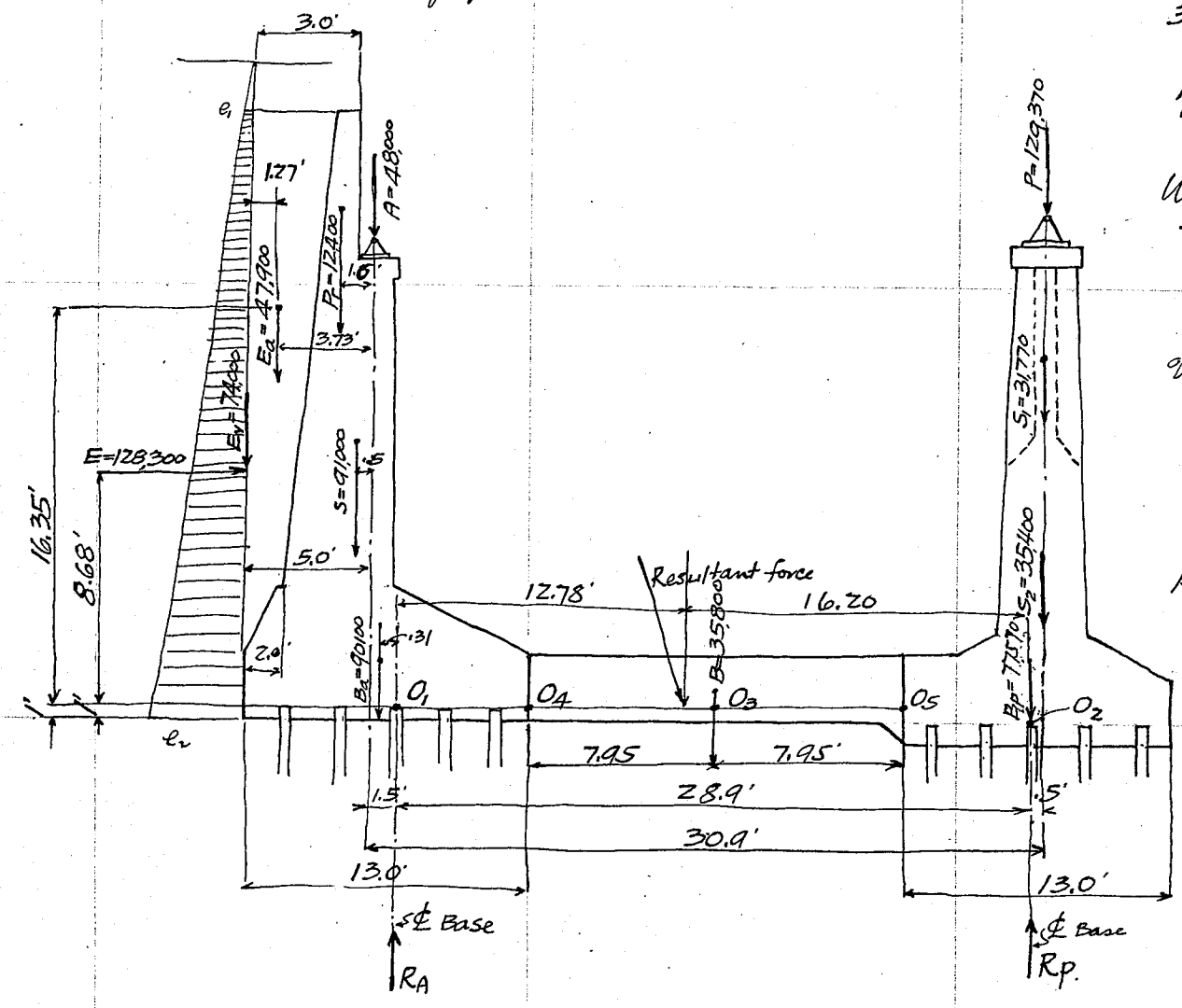
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.
Design of combined base for Abutment and Piers.
Base under inside girder 9' wide.
Case A Case of full loads at normal state.



Taking moments of several forces about O_1.

Loads	Hor. forces	Vert. forces.	Lever arm	moments
A		48,000	−1.5	−72,000
P_r		12,400	−3.1	−38,400
S		91,000	−2.0	−182,000
B_a		90,100	−0.31	− 27,900
E_a		47,900	−5.28	−250,500
E_v		74,000	−6.50	−481,000
E	128,300		+8.68	+1,115,000
	128,300#	363,400#		+ 63,200'#
P		129,370	+29.4	+ 3,801,000
S_1		31,770	+29.4	+ 934,000
S_2		35,400	+29.4	+ 1,041,000
B_p		77,570	+28.9	+ 2,240,000
		274,110#		+8,016,000'#
B		35,800	+14.45	+517,000
		35,800#		+517,000'#

Total. $\Sigma H = 128,900^{\#}$ $\Sigma V = 673,310^{\#}$ $\Sigma M = +8,596,200^{'\#}$

Reaction for Pier $R_p = \frac{8,596,200}{28.9} = 297,000^{\#}$
$= 132.7^{tons}$

Reaction for abutment $R_A = 673,310 - 297,000 = 376,310^{\#}$
$= 168.0^{tons}$

Weight of Pier base.

	arm	moment
$\frac{8.0+5}{2} \times 9 @ 150 =$	8,770 × 4.35 =	38,100'#
12.0 × 9 @ 150 =	16,200 × 3.5 =	56,700
3 × 13 × 9 @ 150 =	52,600 × 1.5 =	78,900
	77,570#	173,700'#

Vert. dist. of c.g. $= \frac{173,700}{77,570} = 2.24'$ from bott.

Weight of Abutment base.

$\frac{5.5+13}{2} \times 9 @ 150 =$	37400 × 4.3 =	160,700'#
3 × 13 × 9 @ 150 =	52,700 × 1.5 =	79,000
	90,100#	249,700'#

Vert. dist. of c.g. $= \frac{249,700}{90100} = 2.77'$ from bottom.

37400 × 5.74 = 214,800
52,700 × 6.50 = 342,500
90,100# 557,300'#

Hor. dist. of c.g. $= \frac{557300}{90,100} = 6.19'$ from heel.

$x = 6.5 - 6.19 = 0.31$ rear side of ℄ base.

Weight of Beam between pier and abutment
Assume the section be 3' × 5'
3 × 5 × 15.9 @ 150 = 35,800#
Weight of earth behind of abutment
2.5 × 21.3 × 9 @ 100 = 47,900#
Earth pressure
$e_1 = \frac{2 \times 100}{3} = 67\#/'$
$e_2 = \frac{29.3 \times 100}{3} = 977$

Earth pressure on abutment
$E_h = \frac{67 + 977}{2} \times 27.3 \times 9 = 128,300^{\#}$

$E_V = 128,300 \times \tan 30° = 128300 \times .577 = 74,000^{\#}$

Point of application of Resultant forces.
Distance from O_1.
$= \frac{8,596,200}{673,310} = 12.78'$

Load on one pile
for Pier 132.7 ÷ 15 = 8.85 tons on one pile.
for abutment 168.0 ÷ 15 = 11.20 tons on one pile.

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CALCULATIONS FOR

108 A Bridge for city of Tokyo.

Moment at center of beam O_3.

Load	lever arm	moment
R_p = +297,000	14.45	+ 4,290,000 '#
P = −129,370	14.95	− 1,850,000
S_1 = − 31,770	14.95	− 474,500
S_2 = − 35,400	14.95	− 529,000
B_p = − 77,570	14.45	− 1,120,000
$\frac{1}{2}B$ = − 17,900	4.23	− 75,700
+ 5,000 #		+ 2,40,800 '#

Assume 3' earth fill for settlement of earth after completion.
Assume weight of earth in water = 80 #/cu. ft.
Moment at center of span Span being assumed 20'

$$= \frac{5 \times 80 \times 3 \times 20^2}{8} = 60{,}000\ '\#$$

For continuity of Beam $M = 60{,}000 \times .8 = 48{,}000\ '\#$

Total moment at O_3 =

	240,800
	48,000
	+ 288,800 '#

Moment at toe of abutment O_4

moment at O_3	+ 240,800 '#
+5000 × 7.95	= + 39,800
−17,900 × 3.98	= − 71,200
	+ 209,400 '#

Shear at end of abutment base on beam (O_4)
= −5,000 + 17,900 = +12,900

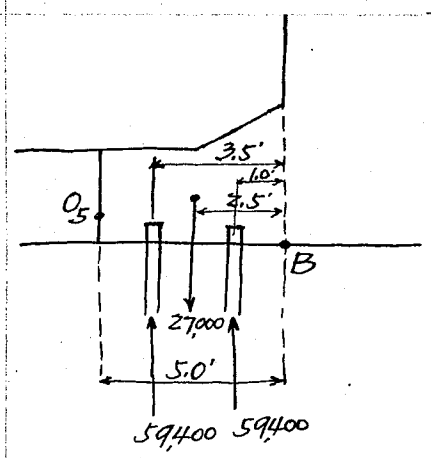
Shear at end of Pier base on beam. (O_5)
= −5,000 − 17,900 = −22,900 #

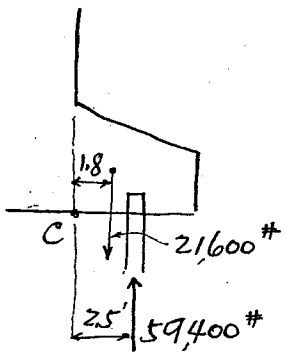
Moment at toe of Pier O_5

moment at O_3	= + 240,800
5,000 × 7.95	= − 39,800
17,900 × 3.98	= + 71,200
	+ 272,200 '#

Moment at fixture of abutment base for case A.

load on one pile = 8.85 tons.
load for 3 piles = 59,400 #
Shear at O_5 = − 22,900 #
Moment at fixture B.
piles 59,400 × 4.5 = +267,500
Shear − 22,900 × 5 = −114,500
footing − 27,000 × 2.5 = − 67,500
moment at O_5 = +272,200
+ 357,700 '#



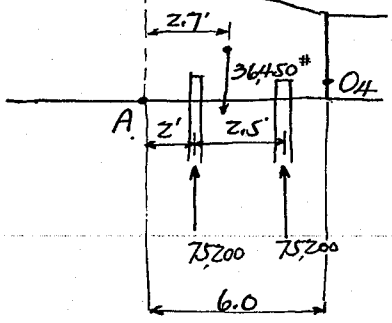


Moment at C
59,400 × 2.5 = +148,500
21,600 × 1.8 = − 38,900
+ 109,600 '#

Shear at C
−59,400 + 21,600
= −37,800 #

Moment of fixture for abutment footing in case A.

load on one pile = 11.2 tons.
load for 3 piles = 33.6 tons = 75,200 #
Moment at A.
moment at O_4 = +209,400 '#
Shear at O_4 12,900 × 6 = − 77,800
pile 75,200 × 6.5 = +488,000
footing 36,450 × 2.7 = − 98,400
+ 521,200 '#



Shear at A

Shear at O_4 =	+ 12,900
footing	+ 36,450
piles	−150,400
	−101,050 #

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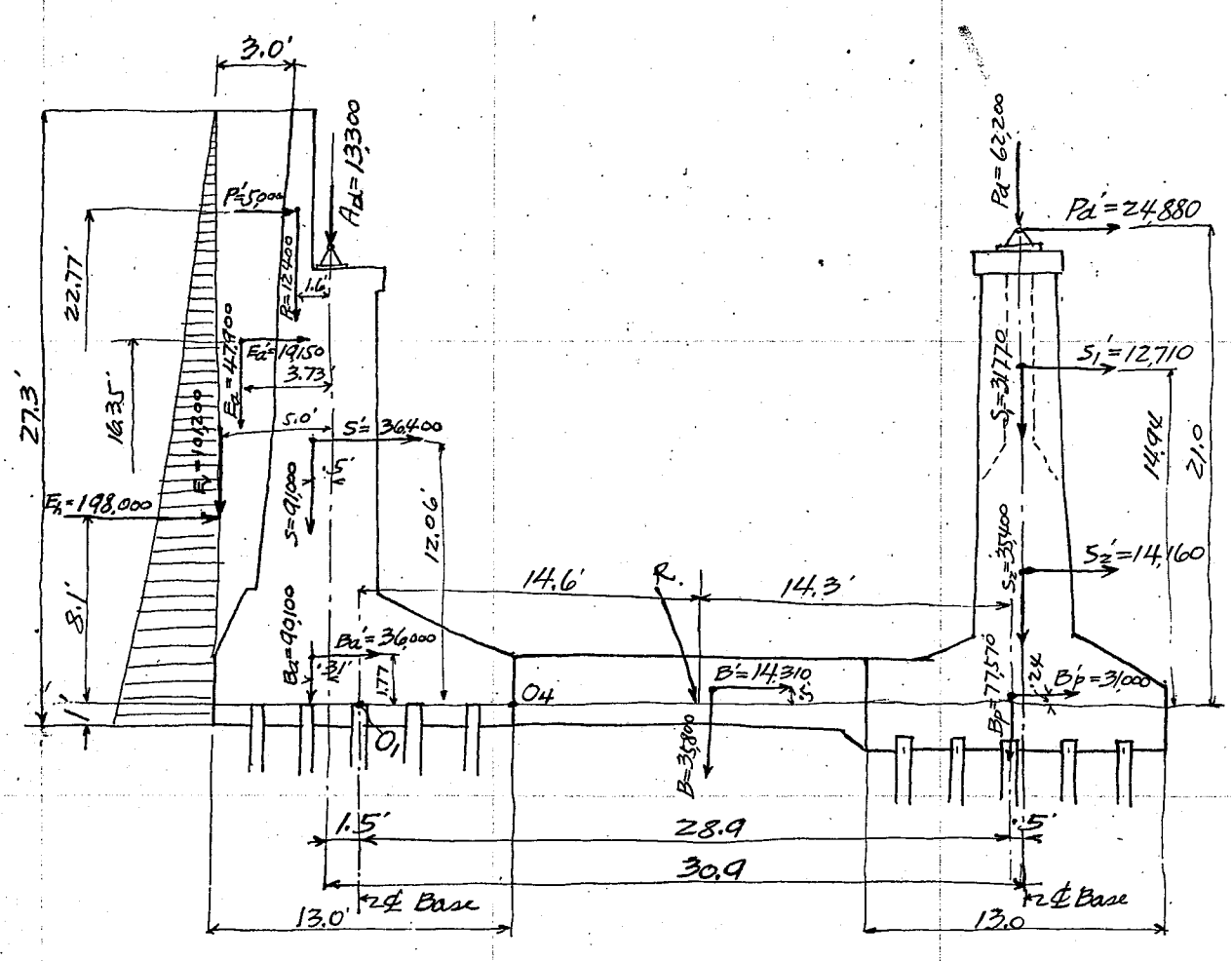
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo

Case B. Stability during earthquake for seismic forces acting toward river side

Earthpressure during earthquake.
Hor. component.
$E_h = .295 \times 100 \times 27.3^2 \times 9 = 198{,}000^{\#}$
Vert. component
$E_v = .151 \times 100 \times 27.3^2 \times 9 = 101{,}200^{\#}$



Taking moment about O_1.

Loads	Hor. forces	Vert. forces	lever arm	moment
Ad		13,300	−1.5	− 19,900
Pr		12,400	−3.1	− 38,400
Pr'	5,000		+22.77	+ 113,900
S		91,000	− 2.0	− 182,000
S'	36,400		+12.06	+ 439,000
Ba		90,100	− 0.31	− 27,900
Ba'	36,000		+1.77	+ 63,700
Ea		47,900	−5.23	− 250,500
Ea'	19,150		+16.35	+ 313,000
Ev		101,200	− 6.5	− 657,500
Eh	198000		+ 8.1	+ 1,604,000
	294,550	355,900		+ 1,357,400 '#
Pd		62,200	+ 29.4	+ 1,828,000
Pd'	24,880		+ 21.0	+ 522,000
S1		31,770	+ 29.4	+ 934,000
S1'	12,710		+ 14.94	+ 189,900
S2		35,400	+ 29.4	+ 1,041,000
S2'	14,160		+ 6.42	+ 90,900
Bp		77,570	+ 28.9	+ 2,240,000
Bp'	31,000		+ 0.24	+ 7400
	82,750	206,940		+ 6,853,200 '#
B		35,800	+ 14.45	+ 517,000
B'	14,310		+ 0.5	+ 7,200
	14,310	35,800		+ 524,200 '#

Total $\Sigma H = 391{,}610^{\#}$ $\Sigma V = 598{,}640^{\#}$ $\Sigma M = + 8{,}734{,}800^{\#}$

Reaction for Pier
$R_p = \frac{8{,}734{,}800}{28.9} = 302{,}300^{\#} = 135.0^{\text{tons.}}$

Reaction for abutment
$R_A = 598{,}640 - 302{,}300 = 296{,}340^{\#} = 132.3^{\text{tons}}$

Load on one pile for pier $= \frac{135}{15} = \underline{9.0^{\text{tons.}}}$

" " " " abutment $= \frac{132.3}{15} = \underline{8.83^{\text{tons}}}$

Position of Resultant force.
Hor dist. from $O_1 = \frac{8{,}734{,}800}{598{,}640} = 14.6'$

Moment at O_3 on beam

Load	Hor. forces	Vert. forces	lever arm	moment
Rp		+302,300	14.45	+4,368,000
Pd		− 62,200	14.95	− 930,000
Pd'	24,880		21.00	− 522,000
S1		−31,770	14.95	− 475,000
S1'	12,710		14.94	− 190,000
S2		−35,400	14.95	− 529,000
S2'	14,160		6.42	− 90,900
Bp		−77,570	14.45	−1,120,000
Bp'	31,000		0.24	− 7,500
½B		−17,900	3.98	− 71,200
½B'	8,950		.50	− 4,500
	91,700#	+77,460#		+ 427,900 '#

moment due to 3' settlement of earth = + 48,000
+475,900 '#

Shear at O_3 = − 77,460#

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CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo.

Moment at O4 on beam.

moment at O3	=	+427,900'#
77460 × 7.95	=	+616,000
17900 × 3.98	=	− 71,300
		+972,600'#

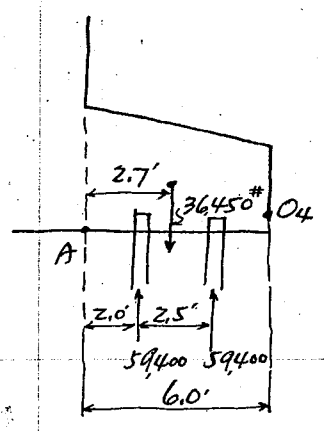
Shear at O4 = −77460 + 17900 = −59,560#

Moment at O5 on beam

moment at O3		+427,900'#
Shear 95360 × 7.95	=	−758,000
beam 17900 × 3.98	=	+ 71,300
		−258,800'#

Shear at O5 (the end of pier footing)
= −77460 − 17900 = −95,360#

Moment of fixture of footing for abutment (point A)



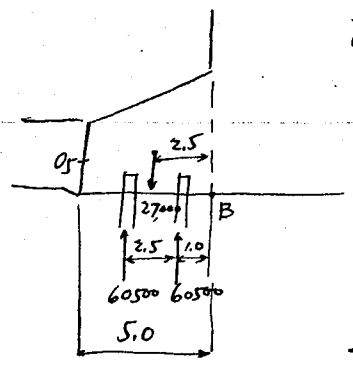
Weight of footing 4.5 × 6 × 9 @ 150 = 36,450#
upward pressure due to piles
8.83 × 3 piles × 2240 = 59,400# per row.

moment at O4	=	+972,600'#
Shear 59560 × 6	=	+357,000
pile 59,400 × 6.5	=	+386,000
Wt. of footing 36450 × 2.7	=	− 95,700
		+1,619,900'#

Shear at A

Shear at O4	−59,560#
pile	−118,800
wt. of footing	+36450
	−141,910#

Moment at Pier footing at B for case B.
load on pile = 9 tons per pile
for 3 piles = 27 tons = 60,500#
moment at B.



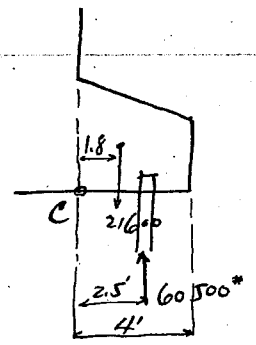
moment at O5 =		−258800'#
Shear	9536 × 5 =	−476,500
piles	60500 × 4.5 =	+272,500
footing	27,000 × 2.5 =	− 67,500
		−530,300'#

Shear at B

Shear at O5	−95,360
footing	−27,000
piles	+118,800
	+3,560#

Moment at pier footing at C for case B.

pile	60500 × 2.5 =	+151,200
footing	21,600 × 1.8 =	− 38,900
		+112,300'#



Shear at C

pile	−60,500
footing	+21,600
	−38,900#

Case C. Case of seismic forces reversed to case B.
Reffering to the figure for Case B.
Taking moments about O1.

Loads	Hor. forces	Vert. forces	leverarm	moment
Ad		13,300	− 1.5	− 19,900
Ad'	5,300		−20.25	− 107,400
Pr		12,400	− 3.1	− 38,400
Pr'	5,000		−22.77	− 113,900
S		91,000	− 2.0	−182,600
S'	36,400		−12.06	−439,000
Ba		90,100	− 0.31	− 27,900
Ba'	36,000		− 1.77	− 63,700
Ea		47,900	− 5.23	− 250,500
	− 82,700#	254,700#		−1,242,700
Pd		62,200	+29.4	+1,828,000
Pd'	24,880		−21.0	− 522,000
S1		31,770	+29.4	+ 934,000
S1'	12,710		−14.94	− 189,900
S2		35,400	+29.40	+1,041,000
S2'	14,160		−6.42	− 90,900
Bp		77,570	+28.9	+2,240,000
Bp'	31,000		− 0.24	− 7,400
	82,750	206,940		+5,232,800
B		35,800	+14.45	+517,000
B'	14,310		− 0.5	− 7,200
	14,310	35,800		+509,800

ΣH = 179,760# ΣV = 497,440# ΣM = +4,499,900'#

Reaction for Pier
$R_p = \frac{4,499,900}{28.9} = 155,800\# = 69.5$ tons

Reaction for abutment
$R_a = 497,440 - 155,800 = 341,640 = 152.6$ tons

Loads on piles (15 piles for reaction)
For piers = $\frac{69.5}{15} = \underline{4.64}$ tons

For abutment = $\frac{152.6}{15} = \underline{10.18}$ tons.

Point of application of Resultant force
$= \frac{4,499,900}{497,440} = 9.05'$ from O1.

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CALCULATIONS FOR

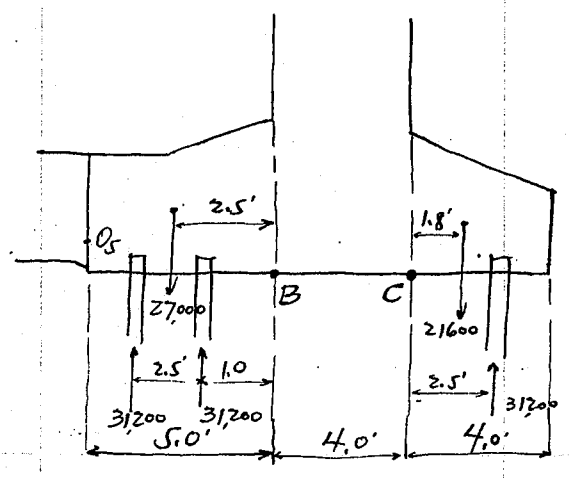
Design of 108A Bridge for City of Tokyo.

Moment at O_3 on beam.

Loads	Hor. forces	Vert. forces	leverarm	moment
Rp		+155,800	14.45 +	2,251,000
Pd		- 62,200	14.95 -	930,000
Pd'	+ 24,880		21.00 +	522,000
S_1		- 31,770	14.95 -	475,000
S_1'	+ 12,710		14.94 +	190,000
S_2		- 35,400	14.95 -	529,000
S_2'	+14,160		6.42 +	90,900
Bp		- 77,570	14.45 -	1,120,000
Bp'	+31,000		0.24 +	7,500
½B		-17,900	3.98 -	71,200
½B'	+ 8,950		0.50 +	4,500
	ΣH = 91,700	ΣV = -69040	ΣM = -59,300	

Shear at O_3 = +69,040#

Moment of fixture for Pier footings.



moment at B.
Wt. of footing = 5×4×9@150 = 27,000

moment at O_5		= +417,900'#
Shear	51140×5	- +255,700
piles	31,200×4.5	= +140,500
footing	27,000×2.5	= - 67,500
		+746,600'#

Shear at B.

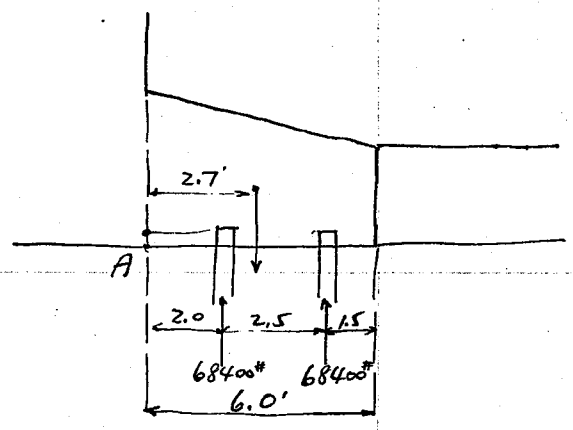
Shear at O_5	= +51,140#
piles	= +62,400
footing	= -27,000
	+86,540#

moment at C
Wt. of footing = 4×4×9@150 = 21,600#

moment at C		
piles	31200×2.5	= +78,000'#
footing	21600×1.8	= - 38,900
		+39,100'#

Shear = -31,200 + 21,600
= -9600#

Moment of fixture for Abutment footing (at point A).



Load on one pile = 10.18 tons.
" " 3 " = 30.54 tons @ 2240 = 68,400#

moment at A.

moment at O_4	=	-679,600
Shear " "	= 86940×6	= -521,500
piles	68400×6.5	= +444,500
footing	36450×2.7	= - 98,400
		- 855,000'#

Shear at A

Shear at O_4	+86,940#
footing	+36,450
piles	-136,800
	-13,410#

Moment at O_4 on beam.

moment at O_3 =		-59,300'#
Shear	-69,040×7.95 =	-549,000
beam	-17,900×3.98 =	- 71,300
		-679,600'#

Shear at O_4
= +69,040+17,900 = +86,940#

Shear at O_5 = +69040 -17900 = +51,140#

moment at O_5

moment at O_3		= -59,300
Shear	51140×7.95 =	+406,000
beam	17,900×3.98 =	+ 71,200
		+417,900'#

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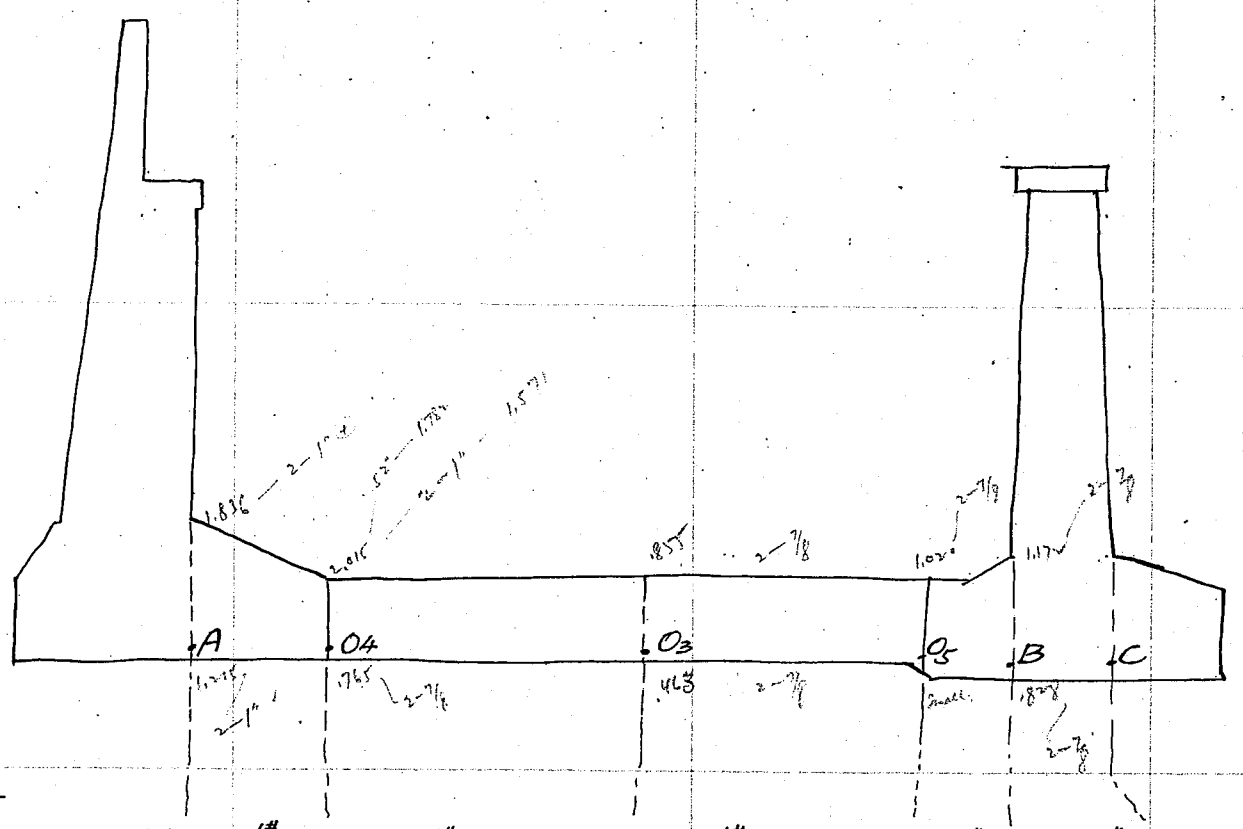
CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo.
Moments and Shears for several cases at several points in the combined footing.



Moments at						
Normal, case	A.	+521,200'#	+ 209,400'#	+ 288800'#	+272,200'#	+357,700'#	+109600'#
Earthquake, "	B	+1,619,900	+ 972600	+475,900	-258,800	-530,300	+112,300
" "	C	-855,000	-679,600	- 59,000	+417,900	+746600	+ 39,100
Shears at							
Normal, case	A	-101,050 #	- 12,900 #	- 5,000 #	- 22,900#	+68900*	-37800*
Earthquake "	B	-141,910	-59,560	- 77,460	-95 360	+ 3,560	-38900
" "	C	- 13,400	+86,940	+69,040	+51140	+86,540	- 9,600

Load on Piles at		Abutment tons	Pier tons
Normal	case A	11.20	8.85
Earthquake	" B	8.83	9.00
"	" C	10.18	4.64

Design of Footing and Beams.
Footing for Abutment. (all figures are for footing 9' wide)
max. moment { + 1,619,900'# during earthquake case B
— 855,000 " " " C

max shear — 101,050# for full loads at normal case.

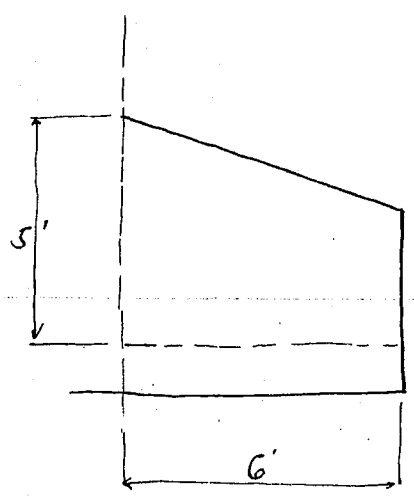
Reinforcement req'd = $\frac{1,619,900 \times 12}{30,700 \times \frac{7}{8} \times 58"} = 12.46^{\square"}$

use 18 – 1"ϕ = 14.14 $^{\square"}$ $p = \frac{14.14}{58 \times 108"} = 0.00226$

$K = .225$, $j = .925$

$f_s = \frac{1,619,900 \times 12}{14.14 \times .925 \times 58} = 25,600$ #/□" < 30700 #/□" OK.

$F_c = \frac{f_s K}{n(1-K)} = \frac{25600 \times .225}{15(1-.225)} = 496$ #/□" OK



Steel for req. moment
$= \frac{855,000 \times 12}{30700 \times \frac{7}{8} \times 70} = 5.46^{\square"}$

use 9 – 1"ϕ = 7.07 $^{\square"}$

Shear stress = $\frac{101050}{108 \times \frac{7}{8} \times 58} = 18.45$ #/□" OK

bond stress = $\frac{101050}{3.142 \times 18 \times \frac{7}{8} \times 58} = 35.2$ #/□" OK

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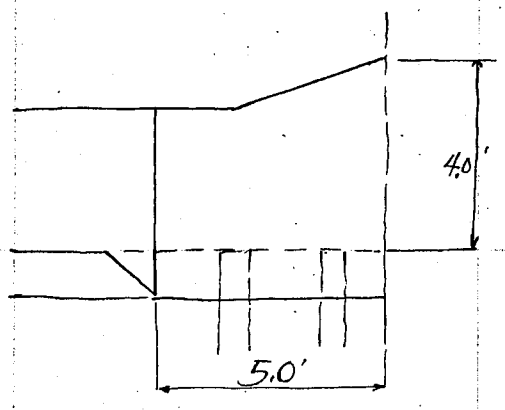
CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo.
Footing for Piers. (Side Span side)

Max. moment = + 746,600'# during earthquake, Case C.
= - 530,300'# " " " B
Max Shear = + 68,900# for full load. Case A.



Steel required $= \frac{746600\times12}{30700\times\frac{7}{8}\times46} = 7.25^{\square"}$

Use $10-1"\phi = 7.845^{\square"}$

$p=\frac{7.854}{108\times46} = 0.00158$, $k=0.191$, $j=0.935$

$f_s = \frac{746600\times12}{7.854\times.935\times46} = 26,500\#/\square"$ OK

$f_c = \frac{26500\times.191}{15(1-.191)} = 417\#/\square"$ OK

Shear stress $= \frac{68,900}{108\times.935\times46} = 14.8\#/\square"$ OK

bond stress $= \frac{68900}{3.142\times10\times.935\times46} = 50.9\#/\square"$ OK

Steel for Neg. moment
$= \frac{530300\times12}{30700\times\frac{7}{8}\times58} = 4.09^{\square"}$

Use $10-\frac{7}{8}"\phi = 6.01^{\square"}$

Footing for Pier (Center Span side)

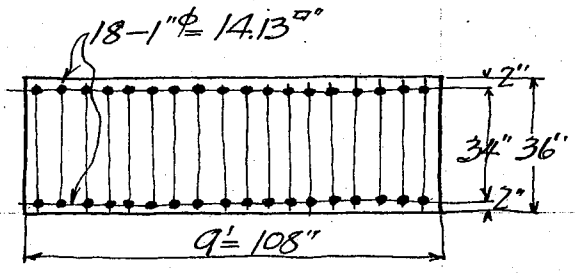
Max moment = + 109,600'# for full load at normal case A
max Shear = - 37,800# " " "
Steel required $= \frac{109,600\times12}{17067\times\frac{7}{8}\times46} = 1.915^{\square"}$

Use $5-1"\phi = 3.93^{\square"}$

Shear $= \frac{37800}{108\times\frac{7}{8}\times46} = 8.7\#/\square"$ OK.

Design of Beam between Abutment and Pier
Section at O4 (near abutment)

Max moment { + 972,600'# during earthquake Case B
- 679,600'# " " " C
Max. Shear + 86,940# " " C



Assumed Section 9' wide 3' thick (assumption of width at start 7' being revised to 9')

Steel required $= \frac{972,600\times12}{30700\times\frac{7}{8}\times34} = 12.8^{\square"}$ for single reinforcement

Use $18-1"\phi = 14.13^{\square"}$ on Top & Bottom.

$d'/d = 2/34 = 0.059$, $P=P'=\frac{14.13^{\square"}}{34\times108} = .00385$

From the prepared diagram, we have,
$K = 0.250$, $j = .922$

$f_s = \frac{972600\times12}{14.13\times.922\times34} = 26,350\#/\square"$ OK

$f_c = \frac{26,350\times.25}{15(1-.25)} = 585\#/\square"$ OK

$v = \frac{86,940}{108\times.922\times34} = 25.7\#/\square"$ OK

$u = \frac{86940}{18\times3.142\times.922\times34} = 49\#/\square"$ OK.

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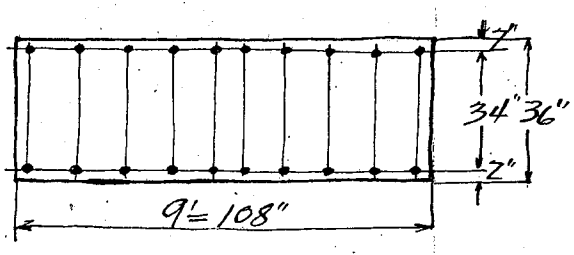
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.
Section at O_3 (Center of span of Beam)



Max. moment { $+288800'^{\#}$ for full load at normal case A.
$-59,000'^{\#}$ during earthquake case C
Max. Shear $+77000^{\#}$ " " " " " B
Assumed section 9' wide 3' thick (Revised from 6' to 9')
Steel required $= \frac{288,800 \times 12}{17067 \times \frac{7}{8} \times 34} = 6.82^{\square"}$ for single reinforcement
Use $10-1"\phi$ bars $= 7.85^{\square"}$ on Top and Bott.
$d'/d = 2/34 = 0.059$, $p = p' = \frac{7.854}{34 \times 108} = 0.00214$
$K = 0.206$, $j = .94$

$$f_s = \frac{288800 \times 12}{7.854 \times .94 \times 34} = 13800^{\#}/\square" \text{ OK}$$

$$f_c = \frac{13800 \times .206}{15(1-.206)} = 239^{\#}/\square" \text{ OK}$$

$$v = \frac{77,000}{108 \times .94 \times 34} = 22.3^{\#}/\square" \text{ OK}$$

$$u = \frac{77000}{10 \times 3.142 \times .94 \times 34} = 76.7^{\#}/\square" \text{ OK}$$

Section at O_5 (near pier).

Max. moment { $+272,200^{\#}$ for full load at normal case A
$-258,800$ during earthquake Case B
Max. Shear $-95,360$ " " " B
Use the same reinforcements as for section at O_3

$$\text{Shear } v = \frac{95360}{108 \times .94 \times 34} = 27.6^{\#}/\square" \text{ OK}$$

$$\text{bond } u = \frac{95360}{10 \times 3.142 \times .94 \times 34} = 94.8^{\#}/\square" < 153 \text{ OK}.$$

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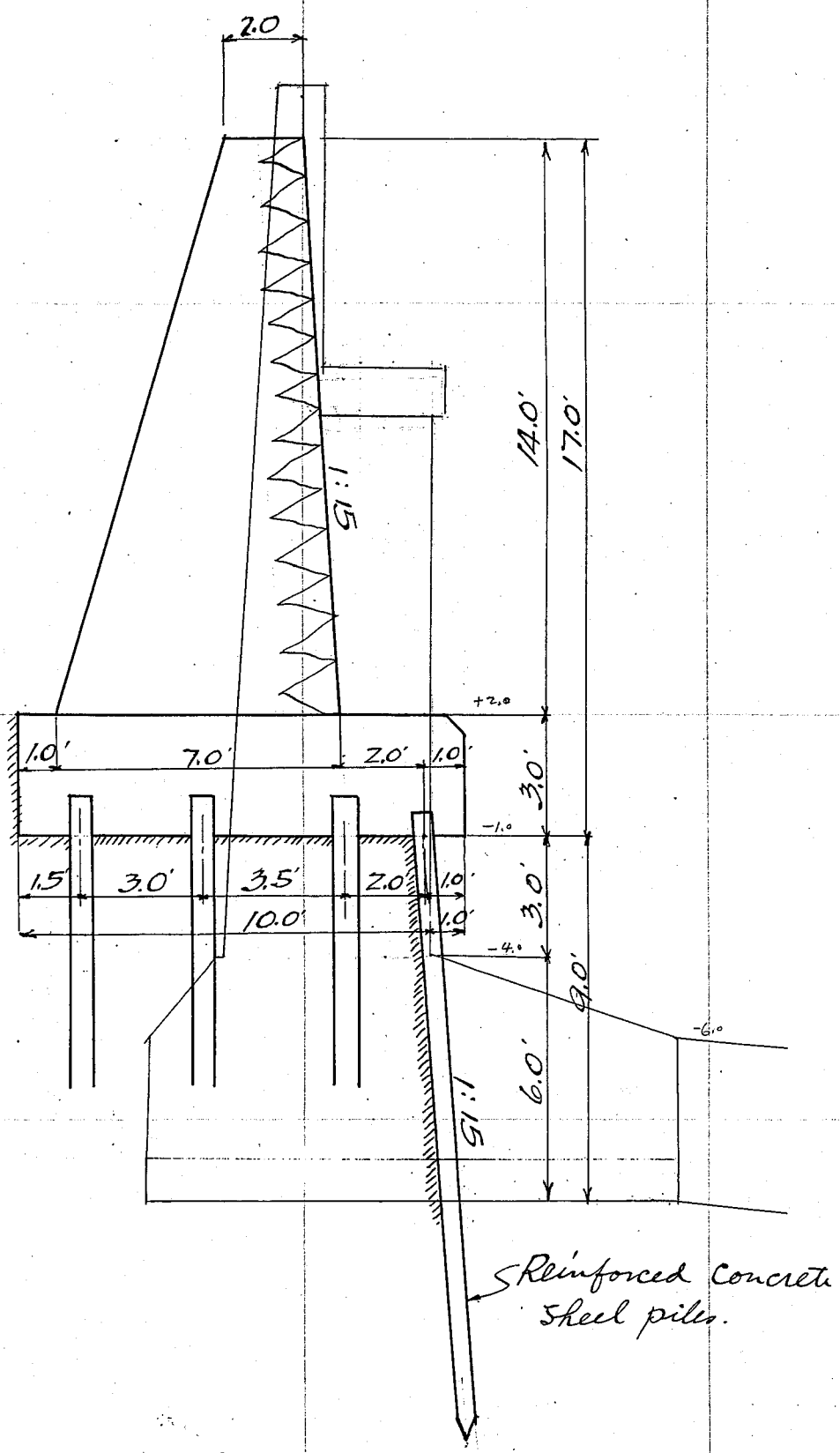
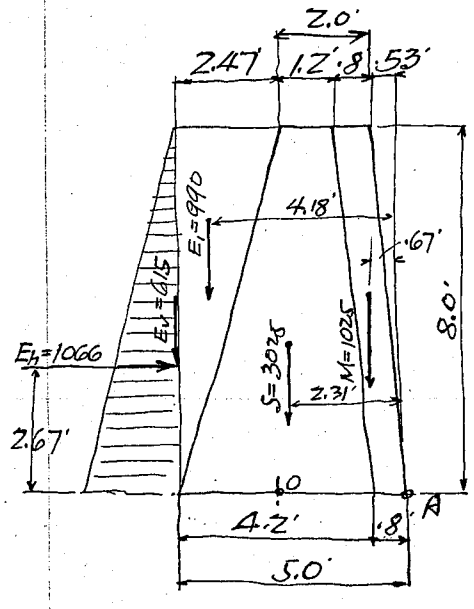
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.
Design of Wingwall.



Stability of Shaft.
Section at 8 ft below top of wall.
Case A At normal case.



Weight of masonry $M = 0.8 \times 8 @ 160 = 1{,}025$ #
" " concrete $S = \frac{1.2+4.2}{2} \times 8 @ 140 = 3{,}025$
" " earth behind $E_1 = \frac{2.47 \times 8}{2} @ 100 = 990$
Earth pressure $E = \frac{100 \times 8^2}{6} = 1{,}066$
Friction of earth on wall $= 1066 \tan 30° = 615$

Taking moments about A.

Loads.	Hor. forces	vert. forces	lever arm	moments
M		1025#	-.67'	-687'#
S		3025	-2.31	-6980
E_1		990	-4.18	-4,135
E_v		615	-5.00	-3,075
E_h	1066		+2.67	+2845
	$\Sigma H = 1{,}066$#	$\Sigma V = 5{,}655$#		$\Sigma M = -12{,}032$'#

Eccentricity $\varepsilon = \frac{-12032}{5{,}655} + 2.5 = +0.37'$

Resultant force cut base between middle third points. Ok.

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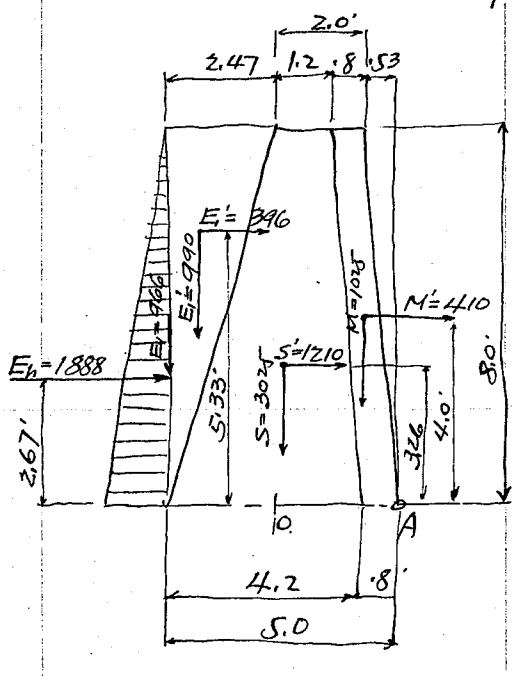
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

Case B. Stability during earthquake for seismic forces acting toward river side.



Earth pressure during earthquake.

Hor. component of earth pressure = $E_h = .295 \times 100 \times 8^2 = 1{,}888^{\#}$

Vert. " " " " " $= E_v = .151 \times 100 \times 8^2 = .966$

Taking moments about ℄ A

Loads	Hor. forces.	vert. forces.	lever arm	moments.
M		1025	−.67'	− 687'#
M'	410		+4.00	+ 1640
S		3025	−2.31	− 6980
S'	1,210		+3.26	+ 3945
E_1		990	−4.18	− 4135
E_1'	396		+5.33	+ 2110
E_V		966	−5.00	− 4830
E_h	1.888		+2.67	+ 5040
	ΣH = 3,904#	ΣV = 6006#		ΣM = −3,897'#

Eccentricity $\varepsilon = \frac{-3897}{6006} + 2.5 = +1.86'$

moment abt. O = $6006 \times 1.86 = 10{,}810^{'\#}$

Section modulus of bed area $= \frac{1 \times 5^2}{6} = 4.17$

Extream fibre stress $= \pm \frac{10810}{4.17} + \frac{6006}{5} = \pm 2590 + 1{,}200$

$= 3790^{\#}/0' = 26^{\#}/0''$ Compression OK

or $= 1390^{\#} = 9.6^{\#}/0''$ Tension OK.

Case C Stability during Earthquake for seismic forces reversed to case B.

Reffering to figure for case B. Moments about A.

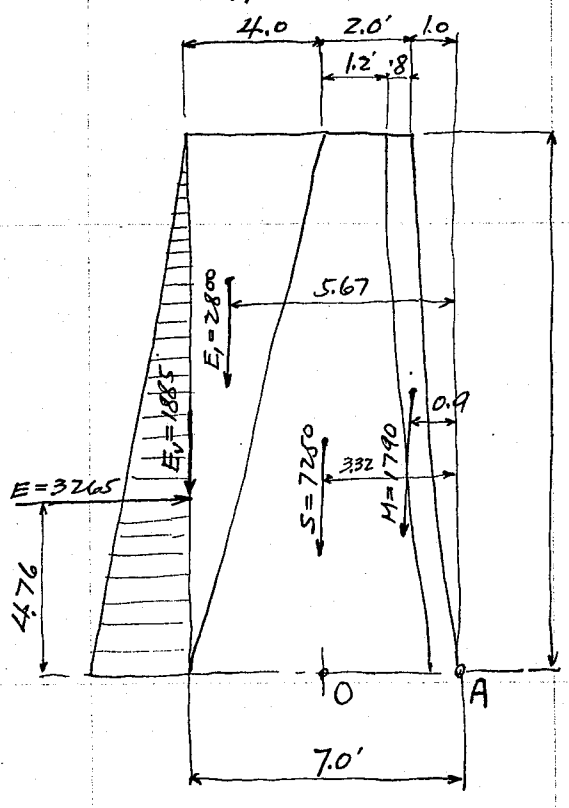
Loads	Hor forces	vert forces	lever arm	moments.
M		1025	−.67	−687
M'	410		−4.00	−1,640
S		3025	−2.31	−6,980
S'	1,210		−3.26	−3945
E_1		990	−4.18	−4,135
	ΣH = 1,620	ΣV = 5,040		ΣM = −17,387'#

Eccentricity $\varepsilon = \frac{-17387}{5040} + 2.5 = -0.95'$

Resultant forces cut base between middle 3rd points. OK.

Stability at Bottom of shaft.

Case A at normal state.



Weight of masonry $14 \times 1.8 @ 160 = 1{,}790^{\#} = M$

" " concrete $14 \times \frac{1.2+6.2}{2} @ 140 = 7250 = S$

Earth behind $\frac{4}{2} \times 14 @ 100 = 2{,}800 = E_1$

Earth pressure $\frac{100 \times 14^2}{6} = 3265 = E$

friction on wall $3265 \times 5.77 = 1885 = E_V$

Taking moment about A.

Loads	Hor. forces	vert forces	lever arm	moments.
M		1,790	−0.9	− 1,610
S		7,250	−3.32	− 24,050
E_1		2,800	−5.67	− 15,870
E_V		1,885	−7.00	− 13,200
E	3265		+4.67	+ 15,250
	ΣH = 3265#	ΣV = 13,725#		ΣM = −39,480'#

Eccentricity $\varepsilon = \frac{-39480}{13725} + 3.5' = +0.625'$

Resultant force cut base between middle 3rd points.

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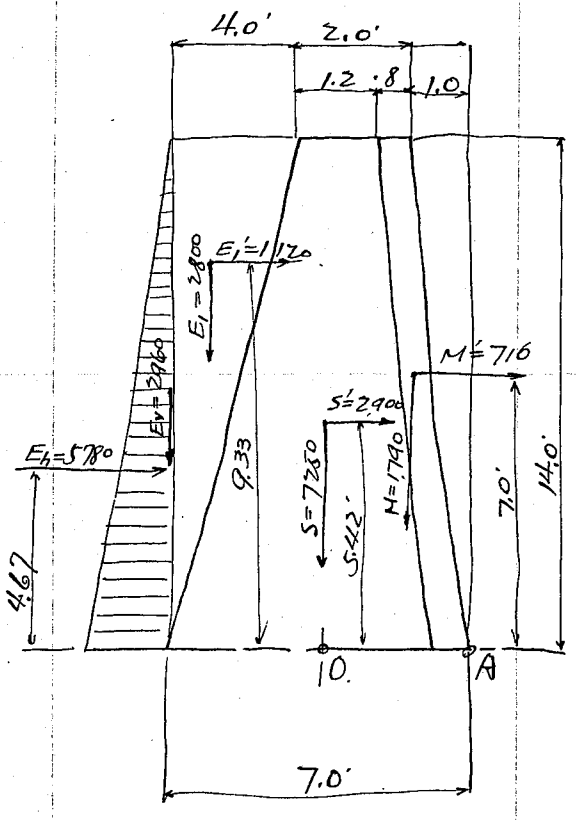
CALCULATIONS FOR

Design of 108 A Bridge for city of Tokyo.
Case B. Stability during earthquake for Seismic forces acting toward river.



Earthpressure
Hor. component $E_h = .295 \times 100 \times 14^2 = 5{,}780^{\#}$
Vert. " $E_v = .151 \times 100 \times 14^2 = 2{,}960$

Loads	Hor. forces	vert. forces	lever arm	moments
M		1,790	− 0.5	− 1,610
M'	716		+ 7.0	+ 5,010
S		7,250	− 3.32	− 24,050
S'	2,900		+ 5.42	+ 15,710
E_1		2,800	− 5.67	− 15,870
E_1'	1,120		+ 9.33	+ 10450
E_V		2,960	− 7.00	− 20,710
E_h	5,780		+ 4.67	+ 27,000
	$\Sigma H = 10{,}516^{\#}$	$\Sigma V = 14{,}800^{\#}$		$\Sigma M = -4{,}070^{'\#}$

Eccentricity $\varepsilon = \frac{-4{,}070}{14{,}800} + 3.5 = +3.23'$

Moment about O. $= 14{,}800 \times 3.27' = 48{,}400^{'\#}$
Use reinforcement on back side.
Steel area req'd $= \frac{48{,}400 \times 12}{30700 \times \frac{7}{8} \times 80^{\circ}} = 0.27^{\square''}$ per ft strip of wall.

Use 3/4"ϕ bars at 1'-6" spacing $= 0.295^{\square''}$ per ft. strip.
Revised to 7/8"ϕ at 24" c to c $= 0.30$ " 2-11-11

Case C. Stability during Earthquake for Seismic forces reversed to case B.
Reffer to the figure for case B.

Sectional area of bottom area
$= 1 \times 7 = 7.0^{\square'}$
Section modulus of bottom area
$= \frac{1 \times 7^2}{6} = 8.17$

Loads	Hor forces	Vert. forces	lever arm	moments abt A.
M		1,790	− 0.5	− 1,610'#
M'	716		− 7.0	− 5,010
S		7,250	− 3.32	− 24,050
S'	2,900		− 5.42	− 15,710
E_1		2,800	− 5.67	− 15,870
	$\Sigma H = 3616^{\#}$	$\Sigma V = 11{,}840^{\#}$		$\Sigma M = -62{,}250^{'\#}$

Eccentricity $\varepsilon = \frac{-62250}{11840} + 3.5 = -1.75'$

Moment about O $= 11840 \times 1.75 = 20{,}700^{'\#}$
Extream fibre stresses $= \pm \frac{20700}{8.17} + \frac{11840}{7.0} = \pm 2540 + 1690$
$= 4{,}230^{\#}/0' = 29.3^{\#}/0''$ Comp. OK
or $= 850^{\#}/0' = 5.9^{\#}/0''$ Tens. OK.

Stability at bottom of base.
Case A Under full loads at normal state.
Weight of earth behind $E_2 = 14 \times 1 @ 100 = 1{,}400^{\#}$
earth pressure $E = \frac{100 \times 17^2}{6} = 4820^{\#}$

Friction of earth on wall $E_V = 4820 \times .577 = 2{,}780^{\#}$
Weight of base $B = 10 \times 3 @ 150 = 4{,}500^{\#}$
Taking moment about A.

Loads	Hor. forces	Vert. forces	lever arm	moment.
M		1,790	− 2.9	− 5,190'#
S		7250	− 5.32	− 38,600
E_1		2,800	− 7.67	− 21,480
E_2		1,400	− 9.50	− 13,300
B		4,500	− 5.00	− 22,500
E_V		2,780	− 10.00	− 27,800
E	4820		+ 5.67	+ 27300
	$\Sigma H = 4820^{\#}$	$\Sigma V = 20{,}520^{\#}$		$\Sigma M = -101{,}570'^{\#}$

Eccentricity $\varepsilon = \frac{-101{,}570}{20{,}520} + 5.0 = 0.05'$

Moment about O $= 20520 \times 0.05 = +1{,}030'^{\#}$

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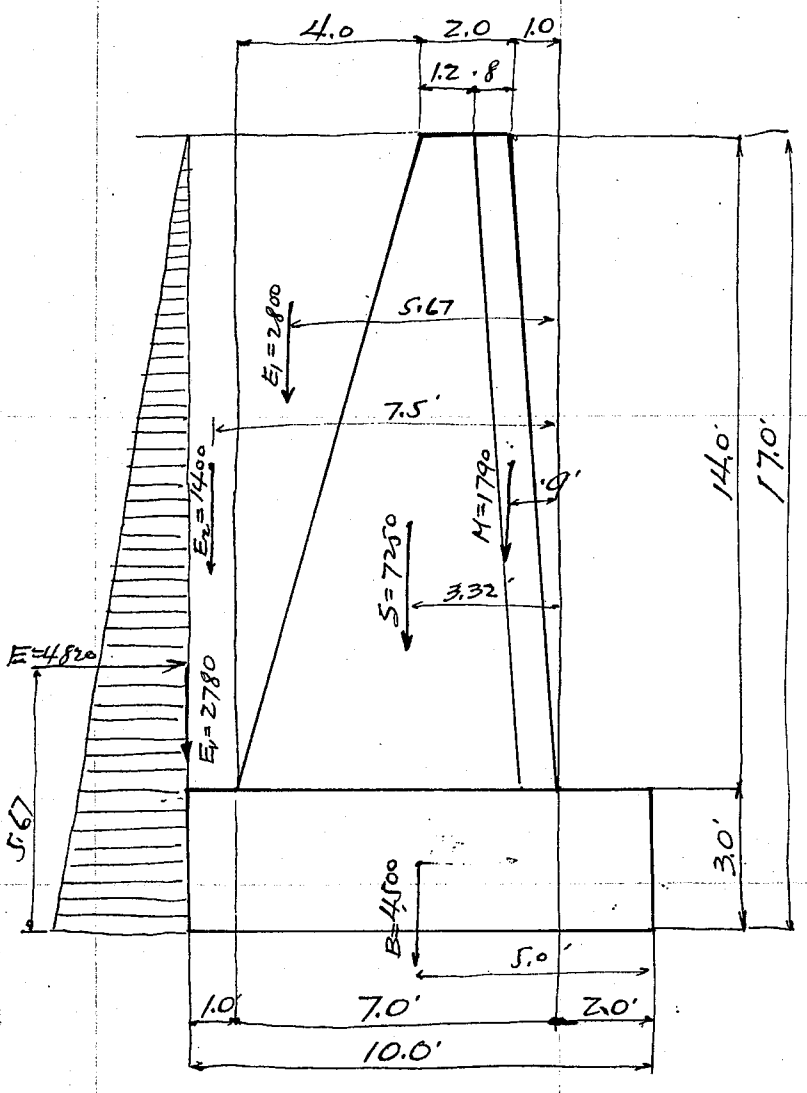
CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo.



Section modulus of bed area = $\frac{10^2}{6} = 16.7^{(1.3)}$ per ft. of wall

Max toe pressure = $\frac{20520}{10} \pm \frac{1030}{16.7} = 2052 \pm 62$

= 2114 #/' = 0.945 tons/' Comp.

or = 1990 " = 0.89 " "

Max. load on one pile.

On sheet piles 0.945 × 1.0' = 0.95

projection of footing 3@150=450# = 0.20

1.15 tons/per ft.

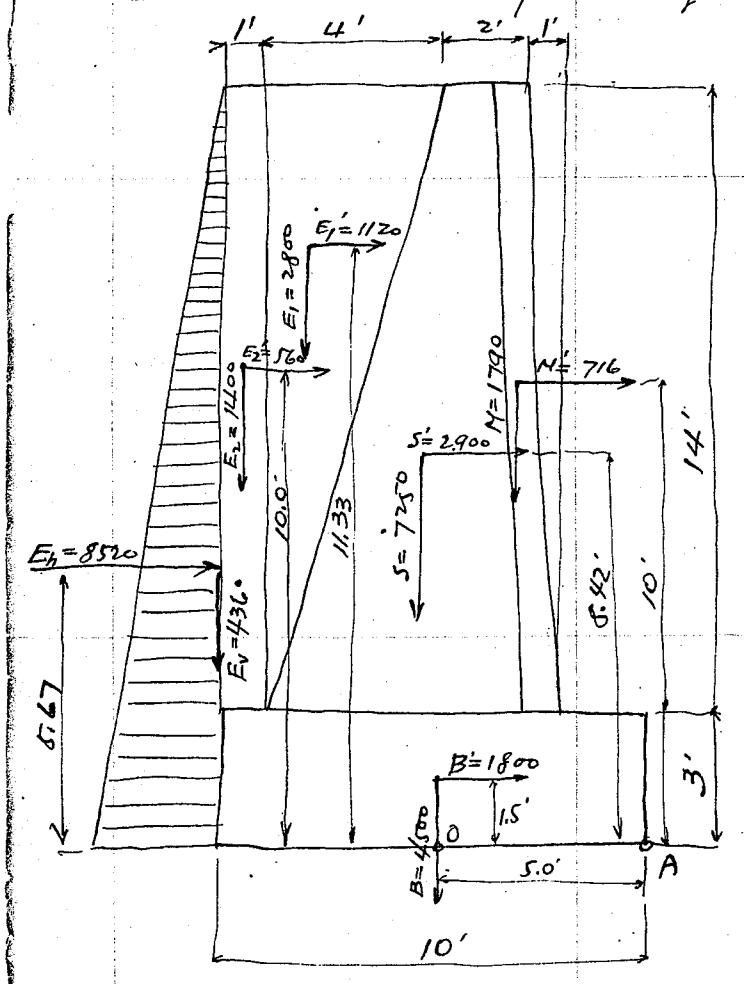
On a pile nearest to the toe.

0.931 × 3.0' × 2.5' = 7.00 tons.

On a pile at heel.

$0.9 \times \frac{3.0}{2.75} \times 3.0$ = 8.10 tons.

Case B. Stability during earthquake for seismic forces acting toward river side.



Earth pressure

$E_h = .295 \times 100 \times 17^2 = 8520^{\#}$

$E_v = .152 \times 100 \times 17^2 = 4360^{\#}$

Loads	Hor. forces	Vert. forces	lever arm	moments
M		1,790#	− 2.9	(− 5,190
M'	716		+ 10.0	+ 7,160
S	2900	7250	− 5.32	−38,600
S'			+ 8.42	+ 24,400
E_1	1120	2800	− 7.67	− 21,480
E_1'			+ 11.33	+ 12,700
E_2	560	1400	− 9.50	− 13,300
E_2'			+ 10.00	+ 5,600
E_V		4360	− 10.00	− 43,600
E_h	8520		+ 5.67	+ 48,300
B		4,500	− 5.00	− 22,500
B'	1800		+ 1.50	+ 2,700
	ΣH = 15,616#	ΣV = 22,100#		ΣM = −43,790

Eccentricity $\varepsilon = \frac{-43,790}{22,100} + 5.0 = +3.02'$

Moment at O.

22100 × 3.02 = 66,700 '#

Max. toe pressure = $\frac{22100}{10} \pm \frac{66700}{16.7}$ =

= 2210 ± 4,000 = 6210#/' C

= 1,790 ' T

Neglecting tension

max toe pressure = $\frac{2 \times 22100}{3 \times 1.98 \times 1}$ = 7450 #/' = 3.3 tons/'

Load on sheet pile = 3.3 × 1.00 + 0.20 = 3.50 tons per lin. ft.

" " a pile nearest to toe = $3.3 \times \frac{3.19}{5.94} \times 3 \times 2.75$ = 14.60 tons.

増田淳氏関係資料
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CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo.

Case C Stability during earthquake for seismic forces reversed to case B.

Reffer to the figure of case B.

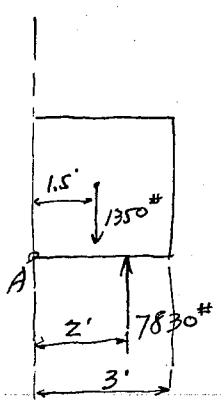
Loads	Hor forces	Vert forces	lever arm	moment
M		1790	− 2.9	− 5190
M'	716		− 10.0	− 7100
S		7250	− 5.32	− 38,600
S'	2900		− 8.42	− 24,420
E_1		2800	− 7.67	− 21,480
E_2		1400	− 9.50	− 13,300
B		4500	− 5.00	− 22,500
B'	1,800		− 1.50	− 2700
	ΣH = 5416	ΣV = 17740		ΣM = −135,290 '#

Eccentricity $\varepsilon = \frac{-135290}{17740} + 5.0 = -2.63'$

neglecting tension, max toe pressure $= \frac{2 \times 17740}{3 \times 2.37 \times 1} = 4980$ #/□' = 2.23 tons/□'.

Load on one pile at heel. $= 2.23 \times \frac{5.61}{3 \times 2.37} \times 3 \times 3 = 16.8$ tons.

Footing at toe.



Weight of footing = 3×3@150 = 1350#

upward pressure of sheet pile = 3.5×2240 = 7,830#

moment at A.

due to sheet pile 7830 × 2 = +15,660'#

" " wt. of footing 1350 × 1.5 = 2030

ΣM = 13,630'#

End shear = 7830 − 1350 = 6480#

Steel req'd for moment $= \frac{13630 \times 12}{30700 \times \frac{7}{8} \times 24} = 0.25$ □" per ft. strip

Use 7/8" ϕ bars at 1'-6" c to c = 0.44 □" per ft. strip OK.

Shearing stress $= \frac{6480}{12 \times \frac{7}{8} \times 24} = 26$ #/□". OK

bond stress $= \frac{6480}{\frac{2.75}{1.5} \times \frac{7}{8} \times 24} = 167$ #/□". OK.

Use 26" effective depth if possible bond stress = 153 #/□".

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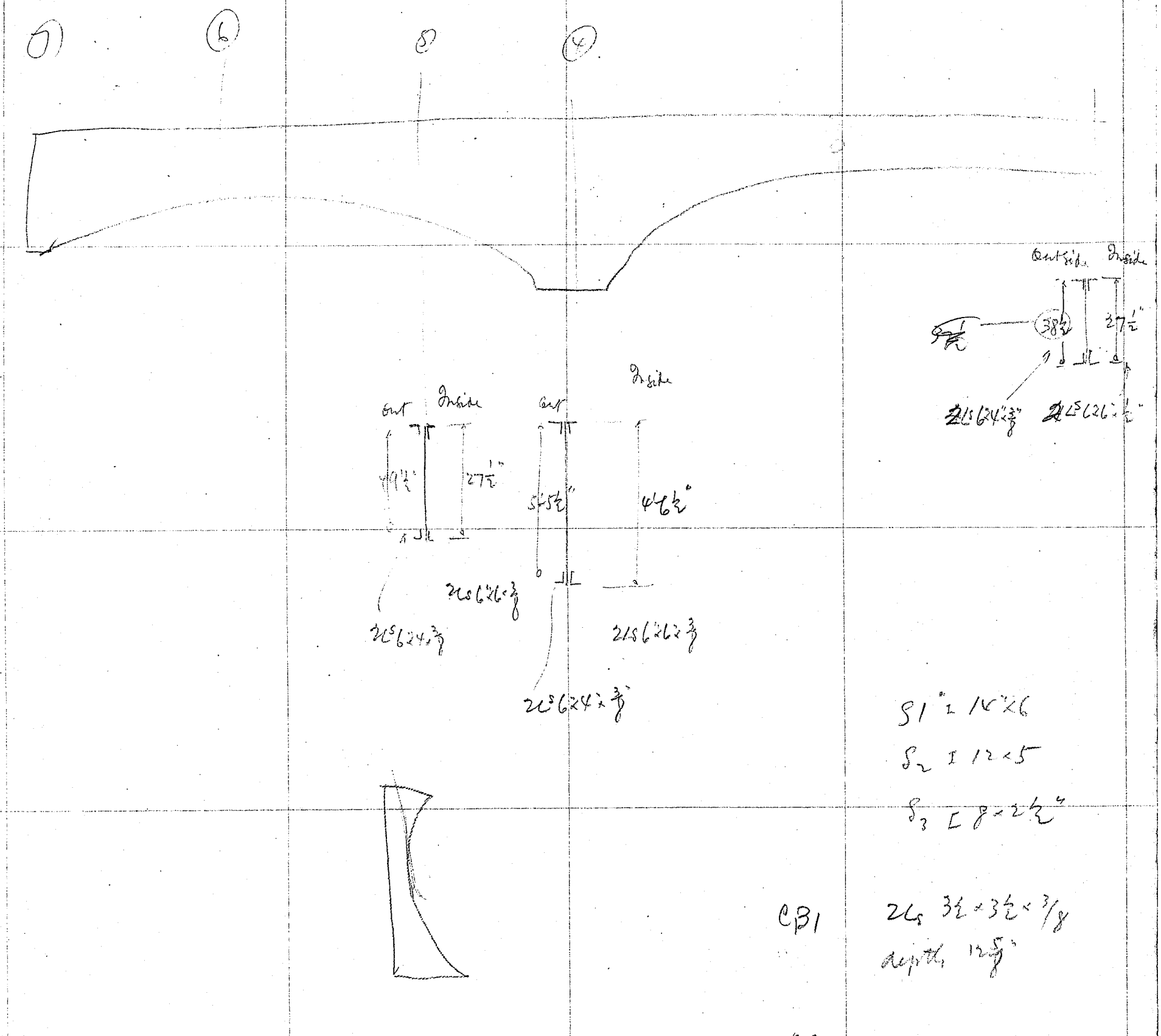
CALCULATIONS FOR



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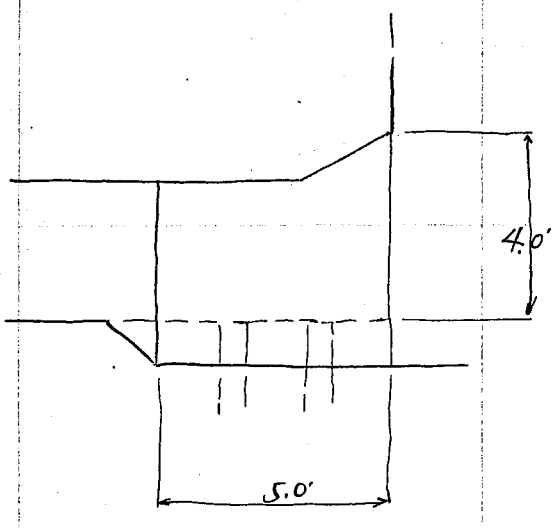
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.
Footing for Pier (Side Span).

max. moment = + 746,600'# during earthquake case C
= − 530,300 " " " B
max. shear = + 68,900 # for full load case A.



Steel required = $\frac{746600\times12}{30700\times\frac{7}{8}\times46} = 7.25$ □"

Use 10−1"φ = 7.854 □"

$p = \frac{7.854}{108\times46} = 0.00158$, $k = 0.191$, $j = 0.935$

$f_s = \frac{746600\times12}{7.854\times.935\times46} = 26{,}500$ #/□" < 30700 #/□" Ok.

$f_c = \frac{26500\times.191}{15(1-.191)} = 417$ #/□" Ok

Shear stress = $\frac{68900}{108\times.935\times46} = 14.8$ #/□" Ok

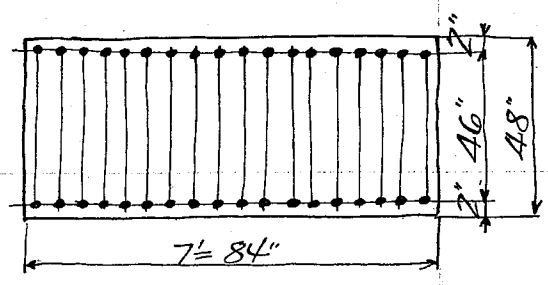
bond stress = $\frac{68900}{3.142\times10\times.935\times46} = 50.9$ #/□" Ok

Steel for neg. moment
= $\frac{530300\times12}{30700\times\frac{7}{8}\times58} = 4.09$ □"
use 10− 7/8"φ = 6.01 □"

Footing for Pier (~~Side~~ Center span)
Max. moment = +109,600'# for full load at normal state.
max shear = − 37,800 # " " " " " "
Steel req'd = $\frac{109.600\times12}{17{,}067\times\frac{7}{8}\times46} = 1.915$ □"
use 5−1"φ = 3.93 □"
Shear = $\frac{37800}{108\times\frac{7}{8}\times46} = 8.7$ #/□" Ok.

Design of Beam between abutment and pier.
Section at O₄ (near abutment)

Max. moment { +972,600'# During earthquake case B.
{ −679,600 " " " " C
max. shear + 86,940# " " " " C



Assumed section 7ft wide 4' thick (Assumption at start of thickness 3' being revised to 4')
Steel required = $\frac{972.600\times12}{30700\times\frac{7}{8}\times46} = 9.49$ □" for single reinforcement.
Use 14−1"φ bars on top & Bottom @ 11.0 □"
d'/d = 2/46 = 0.044, $p = p' = \frac{11}{46\times84} = 0.00285$

From the prepared diagram, we have
K = 0.23, j = 0.93
$f_s = \frac{972600\times12}{11\times.93\times46} = 24{,}800$ #/□" < 307,000 Ok.

$f_c = \frac{24800\times.23}{15(1-.23)} = 494$ #/□" Ok

$v = \frac{86940}{84\times.93\times46} = 23.9$ #/□" Ok

$u = \frac{86940}{14\times3.142\times.93\times46} = 46.2$ #/□" Ok.

増田淳氏関係資料
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CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

Section at O3 (center of span of beam)

Max. moment { + 288,800'# for Full loads at normal case. A
— 59,000 during earthquake case C

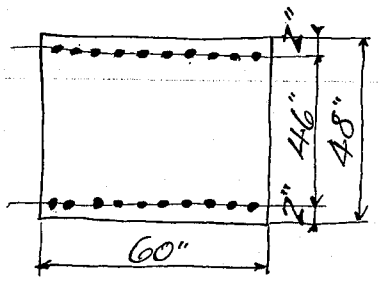
Max. Shear + 77,000 " " " B.

Assumed section 5 ft. wide 4 ft thick

Steel required $= \frac{288,800 \times 12}{17067 \times \frac{7}{8} \times 46} = 5.05$ □" for single reinforcement.

Use 10–1"ϕ = 7.85 □" on top + bottom.

$d'/d = 0.044$, $p = p' = \frac{7.85}{46 \times 60} = 0.00285$



$K = 0.23$, $j = 0.93$

$f_s = \frac{288,800 \times 12}{7.85 \times .93 \times 46} = 10,300$ #/□" < 17067 OK

$f_c = \frac{10300 \times .23}{15 \times (1-.23)} = 205$ #/□" OK

$v = \frac{77000}{60 \times .93 \times 46} = 30$ #/□" OK

$u = \frac{77,000}{10 \times 3.142 \times .93 \times 46} = 57.2$ #/□" OK.

Section at O5 (near pier)

max. moment { + 272,200'# for full load at normal case A)
— 258,800 during earthquake case B

max. shear — 95,360 " " " B

Use the same reinforcements as for section at O3.

shear stress $= \frac{95360}{84 \times \frac{7}{8} \times 46} = 28.2$ #/□" OK

bond stress $= \frac{95,360}{10 \times 3.142 \times \frac{7}{8} \times 46} = 75.3$ #/□" OK

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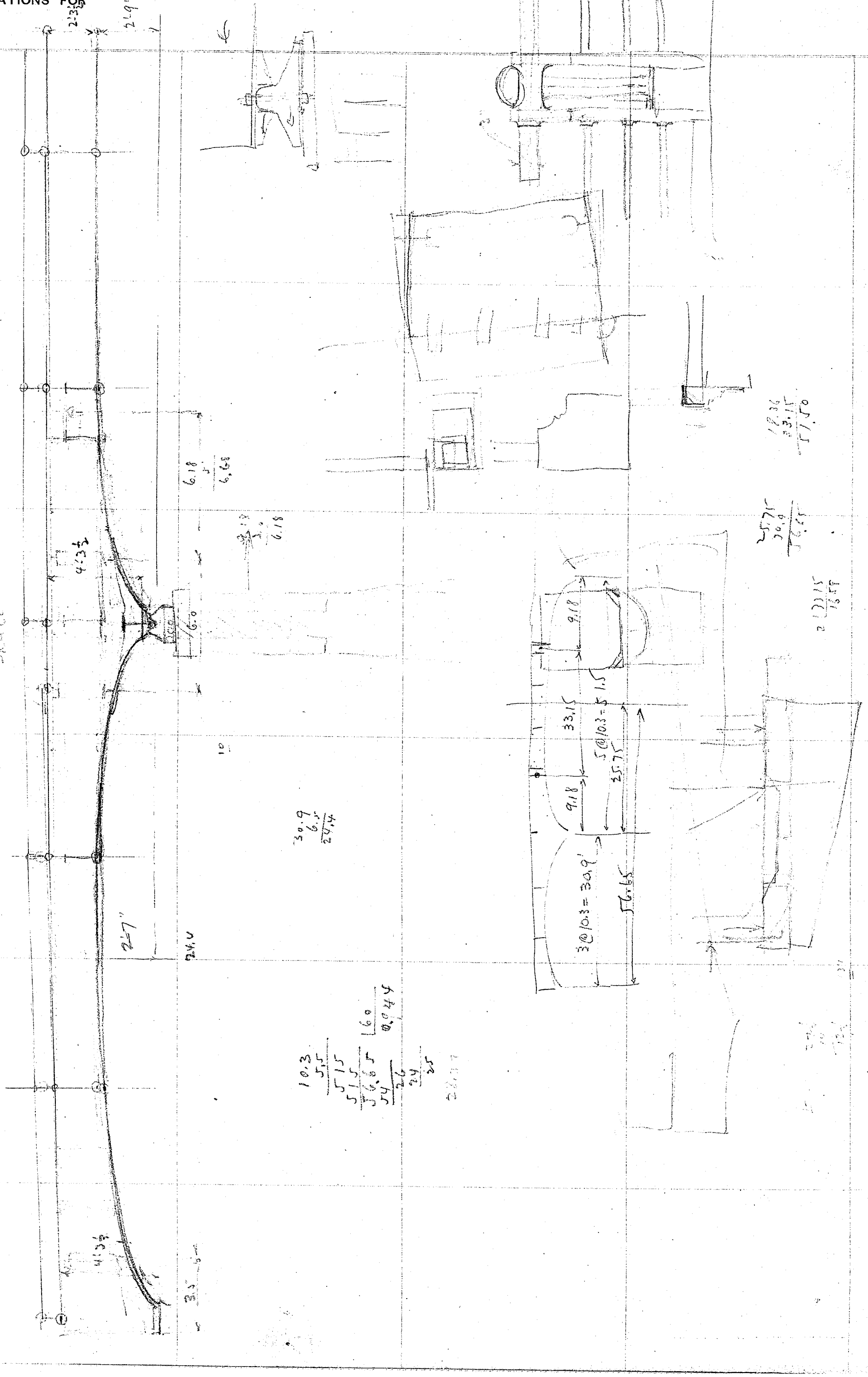
CALCULATIONS FOR



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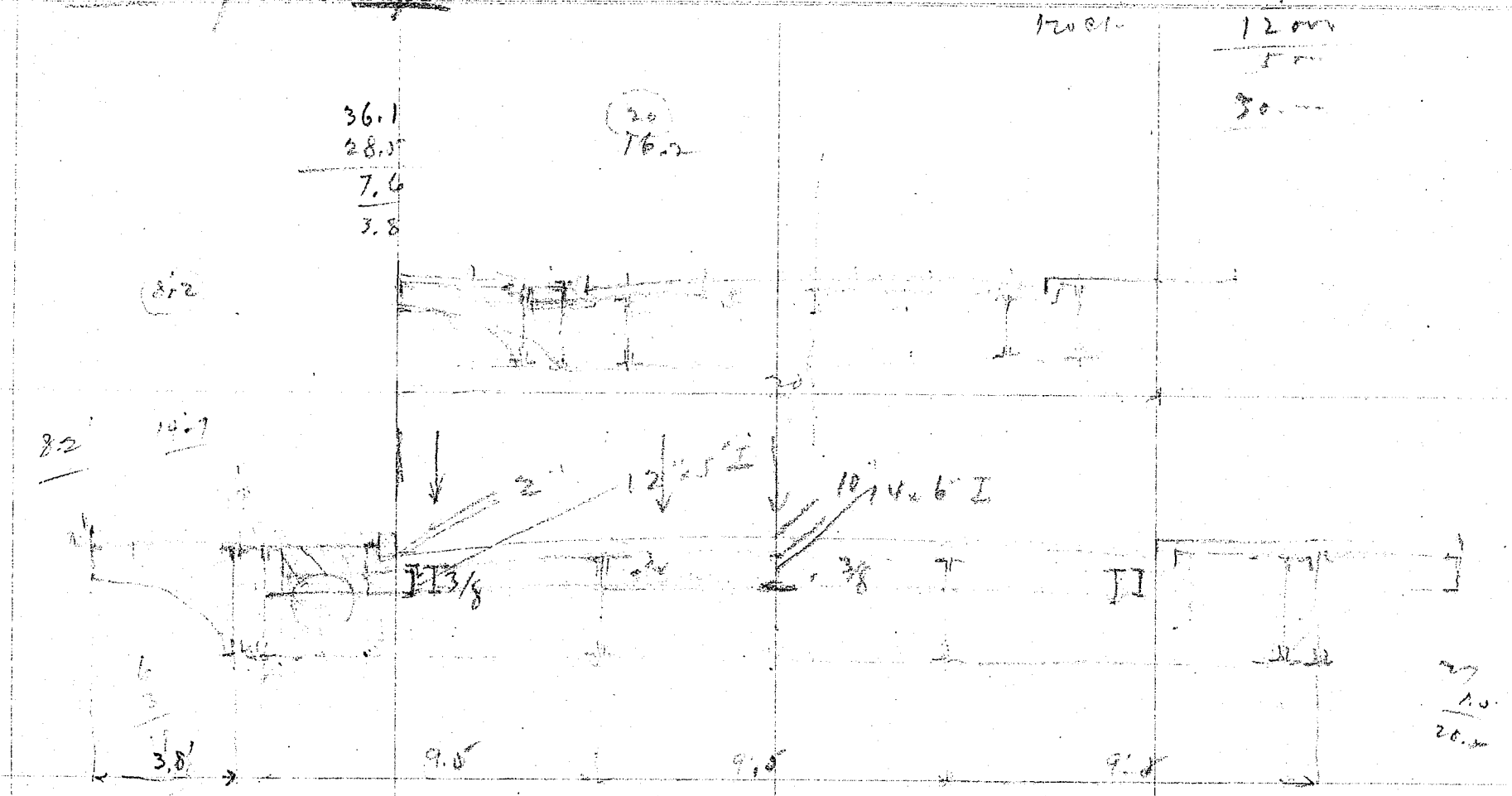
CALCULATIONS FOR

108 A. for Tokyo

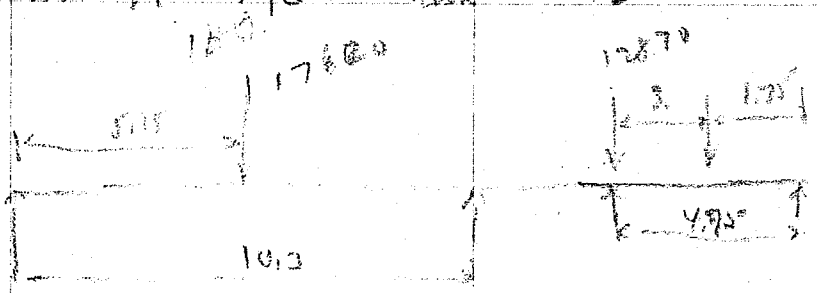


Design of slab 6" slab.

wood blocks 3"
cushion 1"
slab 6"

Sidewalk slab. 4" slab
pavement 1"
5" @ 150 = 62" per sq ft.

Typical slab used 10.3'



Dead load = 1/8 × 557 × 10.3² = 7380 ft

110 × 4.75 = 522
35
557

Shearing at end slab
live load 12870 / 2 × 5.15' = 33200
26400
live load 1/8 × 200 × 10.3² = 7780
DL. 47240 × 12 / 12840 = 22.6

15"
12
75"
102 min. say 110" per sq ft.

30#. 9900
2970
12870 × 1.25 / 4.75 = 4750
12870
17620

R_A = 17620 / 2 × 5.15 = 45400
292. = 7380
52780

D_A = 52780 × 12 / 1540 = 41.0

Need 12" × 6" @ 41.0 = 2.57
or 14" × 6" @ 46.0

4700

Need 12" × 5" @ 2199 = 5/2 = 36.6#

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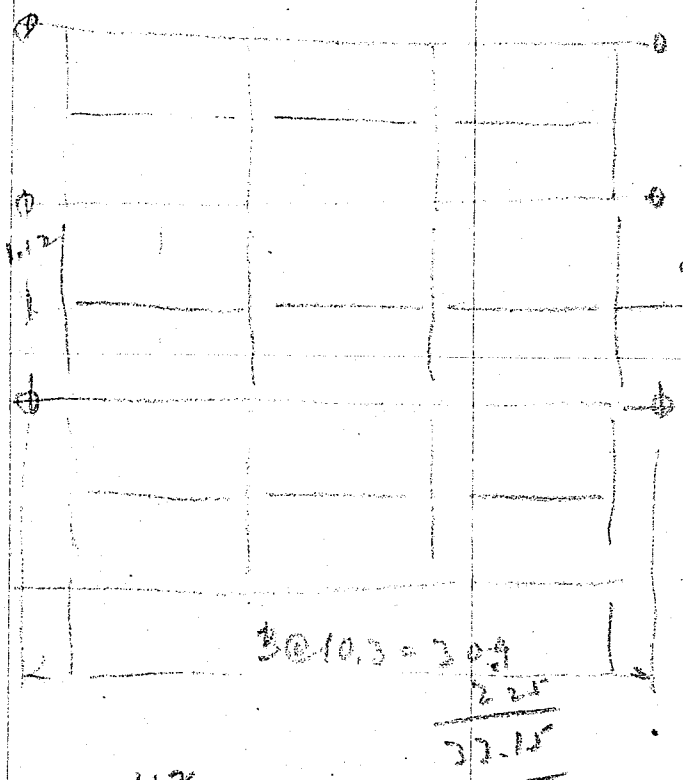
CALCULATIONS FOR

108A for Tokyo.



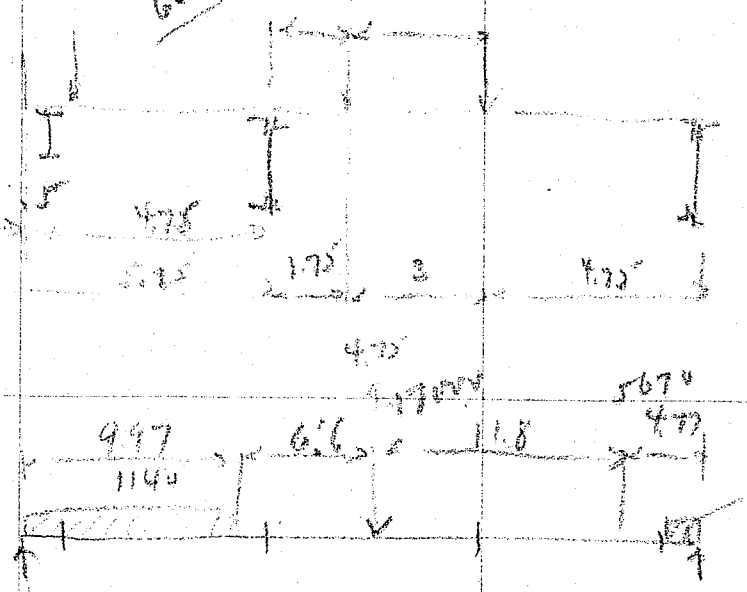
Dead load. Strain concentration.

Q2. 357 × 10.3 = 3680#
57 × 6.27 = 360#
4040

Q2. 57m
1257
6570 = to the ft.

Q2 m = ½ · 6570 · 32.15² = 89200'#

m = 9357 : 11.42 = 107.0m
3257 × 11.20 = 39.2m
67.5m
89.2m
156.7m



12890 × 7.75 / 9.5 = 105m
6493
16930 − 12570

Form wheel = 5670

K = 5670 × 4.72 / 33.15 = 815
6930
9315

m = 9315 × 16.57 = 154m

m = 1720 × 16.57 = 284m
182.4m
156.7m
339.1m

16.57 / 6.6 / 9.91
16.57 / 11.80 / 4.77

Uniform load say 120 × 9.5 = 1140#
1140 × 10 = 11400
1140m × 33.15 / 33.15 = 1720

2ry (26") 31 rods × 3/8 = 9.75 / 1.92 = 1.22 sq in
339.1m / 1.92 = 177m
339.1m / 177m = 1.92 sq in

19.0m / 1.22 / 8.82 = m

33.15 / 11.42 / 21.73
33.15 / 1.14 / 32.01
5.90 / 2.96 / 8.86

2.28 / .28 / 2.56
2.28 / .28 / 2.56

use 25 bars, ½ : 11.50 − 9.50 in sq in

30 ½ / 31 / 14 / 16 sq in

339.1m / 162.00 = 2.09 / 16 / 2.25

@ 154m = 162.00"

27½"

28 / 14 / 13 sq in

(27") 14 / 13

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CALCULATIONS FOR

1081

Cross beam
span 9.5'
Dz. concentrated 537# × 10.7 = 5750#

Dz. m = 2875 × 4.75 = 13700 in#
½ × 57 × 9.5² = 1426
14260

LL. 12870

12870 × 3.00 / 4.75 = 8800
8800
17600

9.5
4.75
1.5
3.25

4.75 = 48/10"
57
8.37'

3½"

17mm (1 − 0.012 e/6)
.988
.918

56860 / 1554 = 36.6#

93
907
web 4pl 3½ × 3½ × 3/8 @ 8.5 = 340
1700
1 web. 16¼ × 5/16
5950
unit distrib. dry
60# in kips 6.2

weight 5950 × 9.5 = 4857.5

2L 8×3×5/16 @ 6.1 × 1.35 = 17.0
2 pl 6×3/8 @ 7.65 × .77 = 44.0
2L 6×3½×3/8 @ 11.7 @ 1.35 = 21.0

4857.5
60
5457.5
200
5710 ÷ 9.5 = 601# in kips

Cross beam 3@570 = 1710
2@270 = 540
2250 ÷ 10.3 = 218# in kips

Quarter Bracket
(2L 7×3½×5/16 @ 6.1 × 1.35 = 122.
90
212
60
272

Secondary stringer 9" channel × 3' @ 19.3# = in kips

stringer
1 - 14"×6" - 46.
2 - 12"×5" - 64
2 - 9"×2½ - 40
5 Others. 10
160# in kips

3
9.
1'
10"

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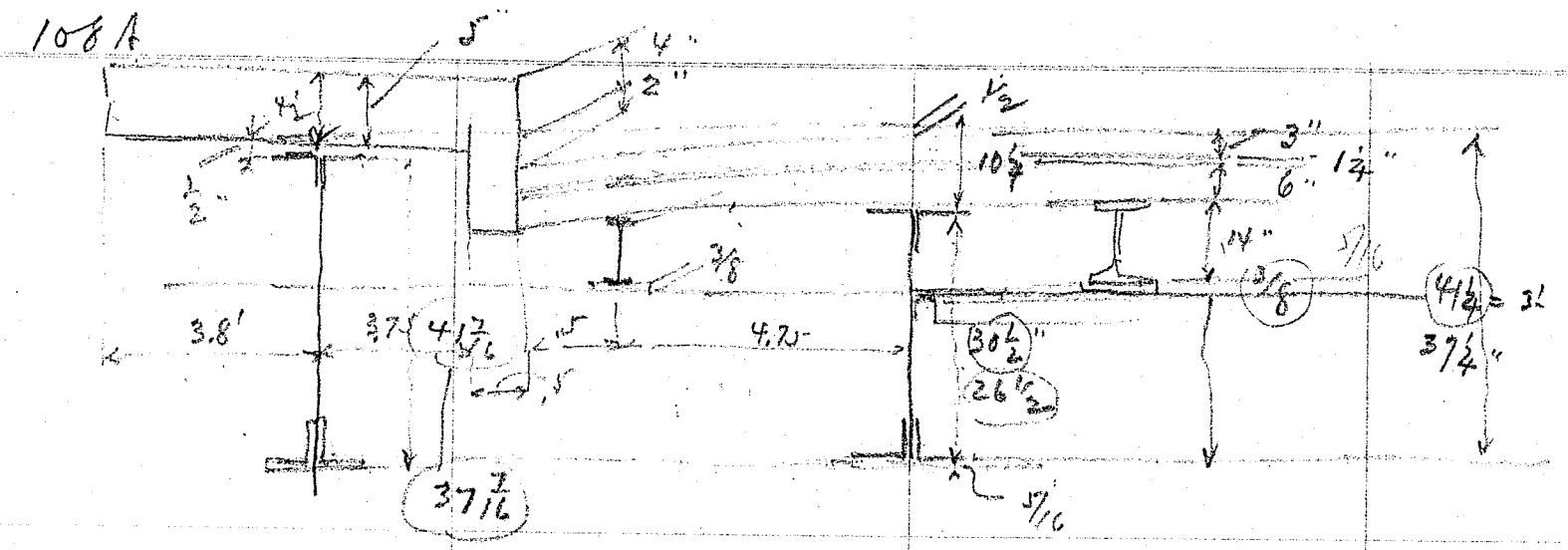
CALCULATIONS FOR



Outside girder.

Approximate load. 76. @ 62 = 470.
Curb &c. 120
Handrail. 80.
extra reaction. 50
weight of wat. mi. 150
870# per lin ft.
weight of girder 150
1020.

$m = \frac{1}{8} \times 1120 \times 33.15^2 = 140,000$

Live Load 8.2 × 100 = 820

$m = \frac{1}{8} \times 820 \times 33.15^2 = 113,000$

Motor truck loading = 12870. = 6435# Rear wheel.
2 2140 front wheel

$R = 2140 \times \frac{477}{33.15} = 308$
3218
3526

$m = 3526 \times 16.57 = 58,400$
113,000
171,400
add. —
LL Total 181,400
DL. 140,000
321,400

curb. 37 41 × 3/8 = 13.85
Effective depth 3.17
16
2.96

$\frac{1}{8}$ curb = 1.73

$H_{tm} = 321{,}400 \div 2.96 = 108{,}500$
$108{,}500 \div 12{,}000 = 6.40$
1.73
4.67 — 0" out

Use 2L 6 - (3½) × 3/8 — 7.22 6.84 × 1.5 = (5.34) — out

weight of main girder.
Main Girder. — 4L 6×6×½ = 23.0
curb. 9.75
32.75 @ 2.4 = 112"
30
142# per lin ft

26%.

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CALCULATIONS FOR

108/4.

Weight of Outside girder.

4 ℄ @ 3½ . ⅜ - 13.68
web. 13.85
27.5 ℄ @ 3.4 = 93.5
20%. 27.0
120.5 call kips 120.

2 @ 120. = 240
2 @ 142 = 284.
525 # per lin ft.

Lateral Bracing.

4 L 3 × 3 - 7/16 @ 6.1 × 14.0 = 342.
5 @ 25 = 125
467

3 @ 467 = 1401 # ÷ 10.3 = 136 # per lin ft.

2 ℄ — — = 171
100
271 × 3 = 813
813 ÷ 10.3 = 79 # per lin ft.

12 L 3 × 3 - 7/16

Approximate weight

Stringers -	160 ×	33.15	=	5304 -
Main girders.	525 ×	33.15	=	1740 -
floor beams	4 @ 2250		=	9000
Lateral.	3 @ 810		=	4430
				34130 ÷ 33.15 = 1030 # per ft

103

36 6 × 5.5 = 33 (46) kips per ft

82.5 = 140. cm
157. cm
297 cm

LL. 820
120
½ . 2020 × 33.15² = 272 cm
520. cm
300. cm
620 cm

630. cm + 256 = 216 cm

2 ℄ 6 × 6 × 7/16 = 8.72 × 1.5 = 7.22
13 × 3/8 = 4.87
13.59
27.15
13.8 ℄
103 @ 3.4 = 140.
20%. 42
182 × 2 = 365

15
20
64
23
126
10
136 × 2 = 270 # per ft

27 × 33.15 = 895.
36 cm × 4 = 144 cm
24 cm
121 cm
378.5

36 cm

13.2
13.2
33.6 @ 3.4 =

99

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CALCULATIONS FOR

1084

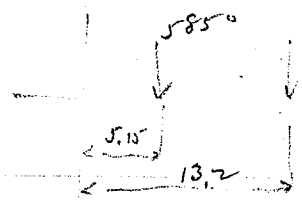
Cantilever arm.

Inside girder

Dead Load - unif. 650# $\frac{1}{2} \times 650 \times 13.2^2 = 56,700^{\#}$

Concentrated. 557 × 10.3 = 5850#

557 × $\frac{9.18}{2}$ = 2560.



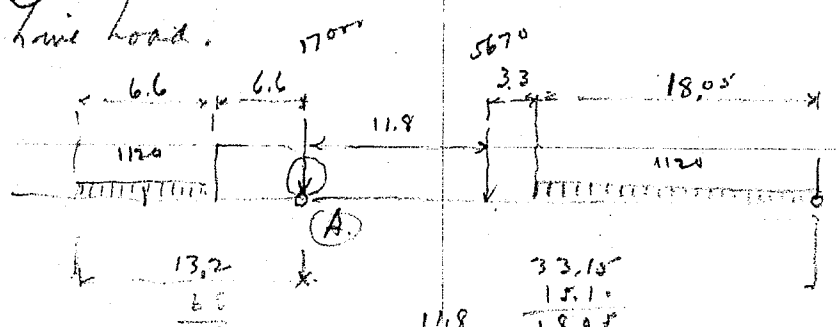
m = 2560 × 13.2 = 33,800
5850 × 5.5 = 30200
8410 64,000

8410
860
17,010 — 17010
935 2015
10900 37160
2015

Due to concentration at end. 650 × $\frac{33.15}{2}$ = 10800

m = 10800 × 13.2 = 142,000.
9350 × 13.2 = 123,000
265,000
5670
640
385,700 #

Live Load.



Live Load. 1120 × 6.6 = 26160 #
1740
33560 #

18.05 × 1120 = 20200 × $\frac{9.0}{33.15}$ = 5500 #

5670 × $\frac{21.35}{33.15}$ = 3660.
17000
2616 0 #

10.3
1.18
9.18

m = 2616 × 13.2 = 346,000
1120 × $\frac{1}{2}$ × 13.2² = 98,000
444,000 #
385,700
829,700 #

Q2

54 27 (27)

54 . 3/8 = 20.25

4.0 #
.28
3.76
.5
4.26

1/8 wt = 2.53

829,000 ÷ 426 = 195,000 ÷ 17000 = 11.50
2.53
8.970 #

wt. 2L 6.6 × 1/2 = 11.50 - 9.50 0

Outside girder.

D.L. $\frac{1}{2} \times 1020 \times 13.2^2$ = 89,000 #

$\frac{1020 \times 33.15}{2}$ = 16,900 × 13.2 = 223,000
312,000 #

Live load.

unif. 1/2 × 820 × 13.2² = 715,000
179,000
250,000
272,000
472,500
312,000
784,500 #

1020 × 13.2 = 13500
1690
3040

37.
27
64". 3/8 = 24.0 1/8 wt = 3.0

29.6
3.25
5.21

784500 ÷ 5.21 = 150,000.
150,000 ÷ 17000 = 8.84
3.00
5.84

motor trucks + dunf. (1/2)

2L 6.4 × 3/8 = 7.22 - 1.5 = 5.720 #

approximate weight —

Inside girder. 4L 6.6 × 5/2 = 23.0
20.25
43.25 @ 3.4 = 147. #
27% 40
187 #

187.
17.0
357
2
714 # per lin ft.

Outside girder. 4L 6.4 × 3/8 = 14.22
24.00
38.22 @ 3.4 = 130
40
170.

30%

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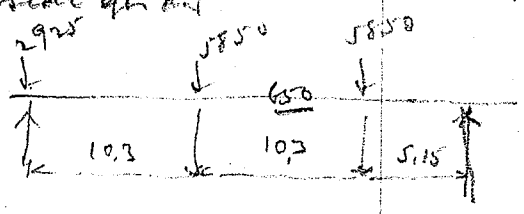
CALCULATIONS FOR

108 Bridge

Anchor 2 cars span length 25.75

Distributed 9tn Rd



D2. 5,850 × 5.15 = 30,100
5850 × 15.45 = 90,300
1206 00 ÷ 25.75 = 4680

650 × 25.75/2 = 8350

2925
7605 #
8350
15955 #
15 cars
+ 955 # on abutment

D2. Cantilever moment. 385700 ÷ 25.75 = 15000 (a)
no uplift at end for Dead load.

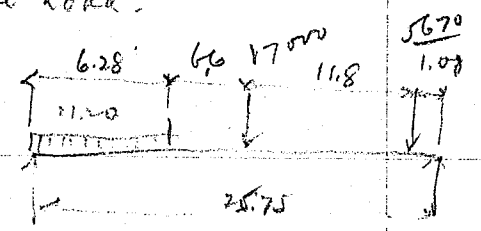
D2 Moment. + 4680 × 10.30 = 48200 "
1/8 × 650 × 25.75² = 53800
102,000
1928 00
− 9080 "

neg m at abut.

On pier.
17000
− 4680
7020 #
8350
15370
37160
15000
67530 # on pier

From cant. Due to moment. D2.

Live Load.



12.88
6.6
6.28

1288
1180
1.08

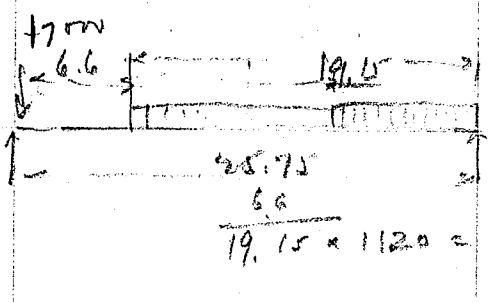
5670 · 1.08/25.75 = 238#
85 cars
8738. × 12.88 = 112,500 "

1120 × 6.28 = 7000 × 3.74/25.75 = 850 × 1.08/12.88 = 52300

Live load on 1 car 112,500 "

max. L.L. neg. moment. 444 00 ÷ 25.75 = 17200 #

max. load on abutment.



19.15 × 1120 = 21400

214 00 × 9.57/25.75 = 8000
17 cars
25,000 #

full uniform load.
1120 × 12.88 = 14.400 #
33560 -

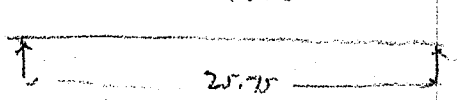
from cant.
Due to moment 444000 ÷ 25.75 = 172 00
65160 on pier.

max on abutment.
D2. + 955
LL. + 25 00
+ 25 955

+ 955
− 17 200
− 16245

on pier
D2. 67530
LL 65160
132,690 # -

Outside girder.
Dead load



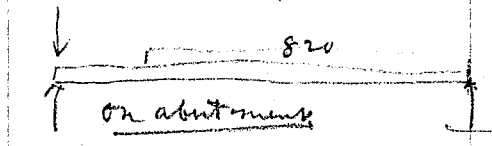
1020 × 25.75/2 = + 1310
312000 ÷ 25.75 = − 1210
+ 100 # on abutments.

+ 131 00
304 00
121 00
55600 on pier.

Live load.

1/2 — 820 × 25.75/2 = 10600
12500
23100

820 × 13.2 = 10800
27160 × 1/2 = 18500
29300



820
500
1330

1330 × 25.75/2 = 17100
4725 00 ÷ 25.75 = 29300
18300
64700

D2. + 100 00
+ 2310 00
+ 2410 00.

+ 100 00
− 18300
− 17200

On Pier.
D2. 55600
LL 64700
120300 #

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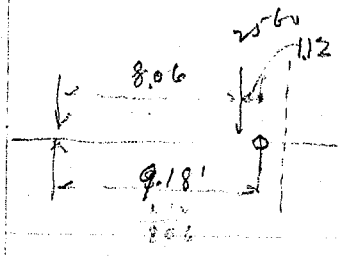
CALCULATIONS FOR

108 ft

Revised - sham

Partition' area 10.3 − 1.12 = 9.18'

Inside girder -



$\frac{650}{2} \cdot 9.18 = 2980$

Due to Concentration - 557 × $\frac{9.18}{2}$ = 2560

$m = 2560 \times 8.06 = 20600$ "

$m = \frac{1}{2} \times 650 \times 9.18^2 = 27400$

48000

conc. uniform = 650 × $\frac{22.15}{2}$ = 10800

Due to Concentration at end $m = 10800 \times 9.18 = 99200$

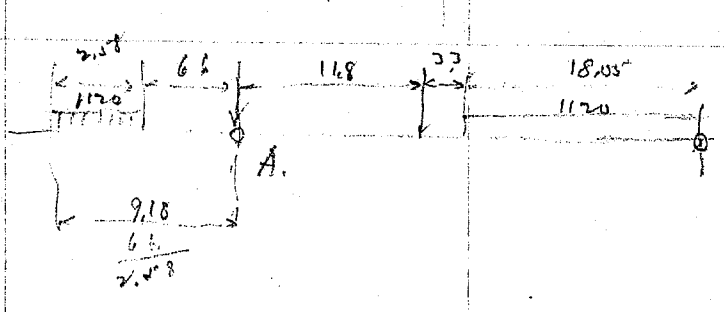
$= 9350 \times 9.18 = 86000$

185200

D.L. 48000

Total B.M. 233200

Live Load.



Reaction at A. = 26160

uniform load. 11.20 × 2.58 = 29 m

29060

$m_0 = 26160 \times 9.18 = 240000$

$= 29 m \times 1.29 = 37600$

277600

25.60
2980
5540
10800
9350
1450
21150
25690
5850
31540

Dead shear

Live moment

on page 6

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MADE BY J. O. DATE 2-10-22 FILE NO 1

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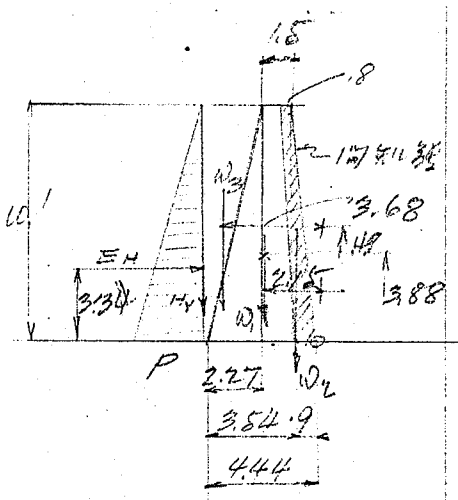
CALCULATIONS FOR

Wing 第5号 P 1-4

Design of Wingwall (mass concrete)
Normal case for 10' height from top.



weight $W_1 = \frac{.7+3.54}{2} \times 10' @ 140^{\#} = 2970^{\#}$

$W_2 = \frac{.8+.9}{2} \times 10 @ 160^{\#} = 1360^{\#}$

$W_3 = \frac{1}{2} \times 2.27 \times 10 @ 100^{\#} = 1135^{\#}$

$E = \frac{10}{2} \times 100 \times \frac{10}{3} = 1667^{\#}$

taking moment about O

	Load		arm		moments
W_1	2970#	×	2.34	=	6950'#
W_2	1360	×	.77	=	1050#
W_3	1135	×	3.68	=	4180#
ΣV	= 5465				12180
E	1667#	×	−3.34	=	−5570'#
			total moment	=	6610'#

pt of application 6610÷5465 = 1.212

eccentricity 2.22 − 1.212 = 1.008

1.48
1.212
.27

~~1350~~

$W_3 = \frac{1}{2} \times 2.7 \times 10 \times 100 = 1350$

$W_1 = \frac{.7+3.97}{2} \times 10 \times 100 = 3270$

W_1	~~3270~~ × 2.6	=	8550
W_2	1350 × 4.0	=	5400
W_2	1360	=	1050
	6000		15000
			5577
			9430

4.43 − 1.57 = .86

Vertical component of earth pressure
1667# × tan 30 = 1667 × 0.577 = 962

taking moment about O

W_1	2970	×	2.34	= 6950'#
W_2	1360	×	.77	= 1050
W_3	1135	×	3.68	= 4180
E_V	962	×	4.44	= 4270
	6427#			16450
				− 5570
E				total moment 10880'#

$\frac{10880}{6427} = 1.69$ eccentricity 2.22 − 1.69 = 0.53

Stability for earthquake Case 1 toward river side.

$K = 0.4 \times \frac{4}{3} = 0.3$ assumed

earth pressure = $.464 \times 100 \times 10 \times \frac{10}{2} = 3310^{\#}$

Vertical friction of earth press

3310 × 0.485 = 1410

3310 × 0.89 = 2950

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CALCULATIONS FOR

taking moment about O due to hor. forces.

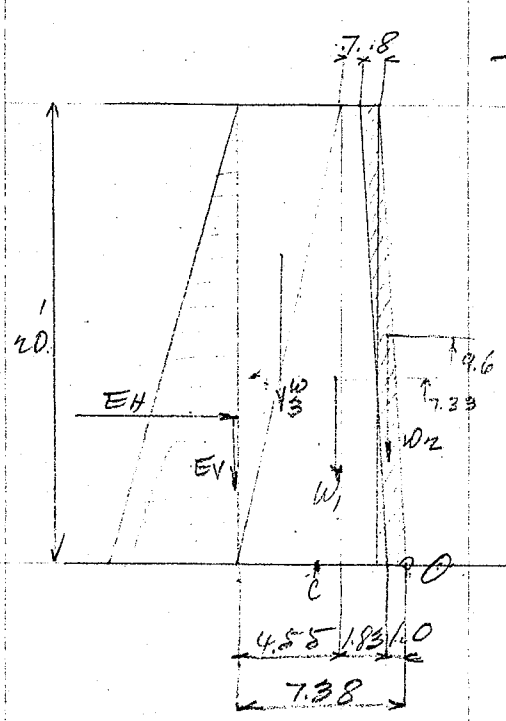
Load		arm	moment.
W_1'	$2970 \times 0.3 = 890$ #	$\times$ 3.88	= 3460 '#
W_2'	$1360 \times 0.3 = 410$	$\times$ 4.90	= 2010
W_3'	$1135 \times 0.3 = 440$	$\times$ 6.67	= 2930
ΣH	2950	$\times$ 3.33	= 9820
	4690		−18220

due to Ver. load

W_1	2970	$\times$ 2.34	= 6950
W_2	1360	$\times$ .77	= 1050
W_3	1135	$\times$ 3.68	= 4180
ΣV	1410	$\times$ 4.44	= 6260
	6875		18440 '#

∴ totalmoment = 220 '#

$\frac{220'\#}{6875} = 0.032$



Stability in normal case per 20' height from top

Weight

$W_1 = \frac{1.7+6.38}{2} \times 20$ @ 140 #/12' = 9930

$W_2 = \frac{1.8+1.0}{2} \times 20$ @ 160 #/12" = 2880

$W_3 = \frac{4.55 \times 20}{2}$ @ 100 " = 4550

earth pressure $= \frac{20}{2} \times 100 \times \frac{20}{3} = 6667$ #

Vertical friction of earth pressure

6667 × tan 30° = 6667 × 0.577 = 3840 #

taking moment about O

Loads		arms.	moments.
W_1	9930 # $\times$	3.86	= 35400 '#
W_2	2880 $\times$	1.05	= 3020
W_3	4550 $\times$	5.86	= 26650
ΣV	3840 $\times$	7.38	= 28400
Σ	21200 #		93470
ΣH	6667 $\times$	−6.67	= −44400 '#
			49070

pt of applica $\frac{49070}{21200} = 2.31$ $\frac{7.38}{3} = 2.46$

eccentricity $\frac{7.38}{2} - 2.31 = 1.38$

$\frac{21200}{7.38}\left(1 \pm \frac{6 \times 1.38}{7.38}\right) = 2850(1 \pm 1.123)$

$= 2850 \times 2.123 = 6060$ #/12' comp.

$= -2850 \times 1.123 = 350$ #/12' ten. (2 #5 @ 12")

Stability during earthquake k = 0.3

toward riverside

earth pres. $= 0.662 \times 100 \times 20 \times \frac{20}{2} = 13240$ #

EV = 13240 × 0.455 = 6030 #

EH = 13240 × 0.89 = 11800 #

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CALCULATIONS FOR

Taking moment about O

			due to hor. loads
W_1	$9930 \times .3$	$= 2980$	$\times 7.33 = 21810$ #'
W_2'	$2880 \times .3$	$= 865$	$\times 9.6 = 8300$
W_3'	$4550 \times .3$	$= 1368$	$\times 13.34 = 18230$
E_H		11800	$\times 6.67 = 78700$
		17010 #	-127040 #'

due to Vertical loads

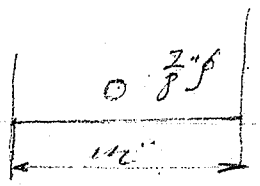
W_1	9930	$\times$	$3.56 = 35400$
W_2	2880	$\times$	$1.05 = 3020$
W_3	4550	$\times$	$5.86 = 26650$
E_V	6030	$\times$	$7.38 = 44500$
	23390		109570 #'

Total moment $= -17470$ #'

pt of appln. $\frac{-17470}{23390} = -0.7475$

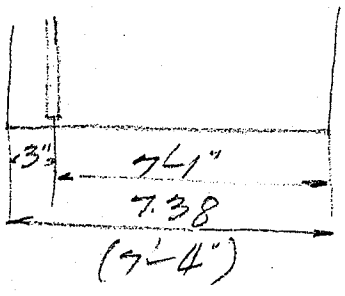
eccentricity $\frac{7.38}{2} + 0.747 = 4.437$ ft.

Moment about center $23390 \times 4.437 = 103700$ #'



Steel req'd $= \frac{103700 \times 12}{30000 \times \frac{7}{8} \times 7.14 \times 12} = .853$ in²

Use 1-7/8"ϕ per ft. 0.6013 in²



$p = \frac{0.6013}{12 \times 85} = 0.00059$

$k = \sqrt{2pn + (pn)^2} - pn$

$= \sqrt{2 \times .00059 \times 150 + (0.00059 \times 150)^2} - .00059 \times 150$

$= \sqrt{.177 + .00783} - 0.0885$

$= \sqrt{.17483} - 0.0885 = 0.418 - 0.0885 = 0.3295$ (≒0.33)

$\therefore j = 1 - \frac{.3295}{3} = 1 - 0.1098 = .89$

$f_s = \frac{103700 \times 12}{0.6013 \times .89 \times 85} = 27400$ #/in²

$f_c = \frac{2pf_s}{k} = \frac{2 \times 0.00059 \times 27400}{0.3295} = 98$ #/in²

Unit shear $= \frac{23390}{12 \times .89 \times 85} = 26$ #/in²

" bond $= \frac{23390}{2.75 \times .89 \times 85} = 112.5$ #/in²

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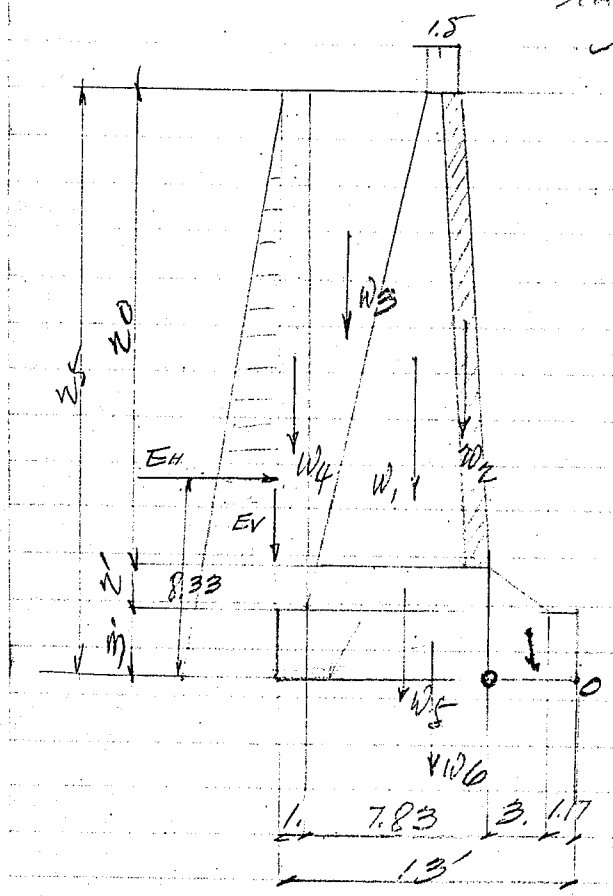
CALCULATIONS FOR

Stability for normal state at bottom depth 25'

Earth pressure $= \frac{25}{2} \times 100 \times \frac{25}{3} = 10400^{\#}$

Vertical friction of Earth press. $= 10400 \times 1.577 = 6000^{\#}$

Taking moment about toe and due to Vertical Load



Loads		arm	moments
W_1	9730	× 7.73	= 76800 '#
W_2	2880	× 8.27	= 15190
$W_3 = \frac{5'}{2} \times 20 @ 100 = 5000$		× 10.33	= 51650
$W_4 = 1 \times 22 @ 100 = 2200$		× 12.50	= 27500
$W_5 = \frac{7.38+11}{2} \times 2 @ 140 = 2570$		× 7.19	= 18500
$W_6 = 13 \times 3 @ 140 = 5460$		× 6.50	= 35500
	~~28040~~ #		~~225140~~ '#
EV	6000# × 13		= 78000
	34040#		303140 '#
EH	10400	× -8.33	= -86630
		total moment	216510 '#

point of application $216510 \div 34040 = 6.36$

eccentricity $6.5 - 6.36 = 0.14$'

Max. toe pressure

$$\frac{34040}{13}\left(1+\frac{6\times 0.14}{13}\right) = \frac{34040}{13} \times 1.065$$

$= 2790^{\#}/12' = 19.4^{\#}/12"$

Stability during earthquake at bottom

earth pressure during earthquake

$E = 0.662 \times 100 \times 25 \times \frac{25}{2} = 20690$

$E_V = 20690 \times 0.455 = 9410$

$E_H = 20690 \times 0.89 = 18410$

Taking moment due to hor. load

Loads		arm	moments
W_1'	9730 × .3	× 12.33	= 36800 '#
W_2'	2880 × .3	× 14.60	= 12640
W_3'	5000 × .3	× 17.67	= 26500
W_4'	2200 × .3	× 14.00	= 9240
W_5'	2570 × .3	× 3.94	= 3450
W_6'	5,460 × .3	× 1.5	= 2460
EH	18,410	× 8.33	= 153800
			= 244540 '#

Moment due to Vertical loads

Loads		arm	moments
$W_1 - W_6$	28040		225140 '#
EV	9410	× 13.00	= 122330
	37450		347470 '#

total moment $= 102880$ '#

pt of application $102880 \div 37450 = 2.75$

eccentricity $6.5 - 2.75 = 3.75$'

(max. toe pressure $= \frac{37450}{13} \times \left(1+\frac{6\times 3.75}{13}\right) = 2880 \times 2.733$

$= 7860^{\#}/12'$

$= 55^{\#}/12"$)

増田淳氏関係資料
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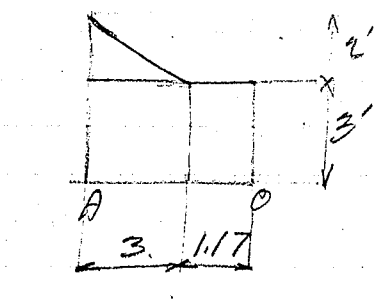
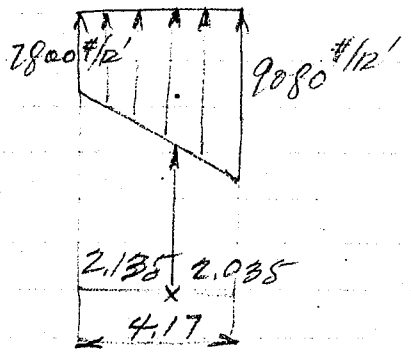
CALCULATIONS FOR

tension neglect max toe pressure $= \frac{2 \times 37450}{3 \times 2.75} = 9080$ #/12"
$= 63$ #/12"

total upward force $= \frac{2800+9080}{2} \times 4.17 = 35100$ #
~~down~~ $= (2 \times 3 \times \frac{1}{2} + 3 \times 4.17) \times 140 = 2170$ #



moment about pt O
35100 × 2.135 = 74800 1#
less 3×140 × 1 = − 420 1#
3×4.17×140×4.17/2 = − 3640 1#
70740 1#



steel req'd $= \frac{70740 \times 12}{30000 \times \frac{7}{8} \times 54} = 0.867$ D"

$\frac{4.17}{3} \times \frac{9080+2\times7800}{9080+7800}$
$= \frac{4.17 \times 24680}{3 \times 16880} = 2.035$

during earthquake toward bankside

due to hor loads moment
W1' 36800
W2' 12640
W3' 26500
W4' 9240
W5' 3450
W6' 2460
91090 1#
due to vertical load 225140
total moment 316230 1#
pt of applic. $316230^{\#} + 28040^{\#} = 11.3'$

Tension neglect max heel press.
$= \frac{2 \times 28040}{3 \times (13-11.3)} = \frac{2 \times 28040}{3 \times 1.7} = 11000$ #

増田淳氏関係資料
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CALCULATIONS FOR

東京市一〇八A橋
新
應力計算書

増田淳氏関係資料
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CALCULATIONS FOR

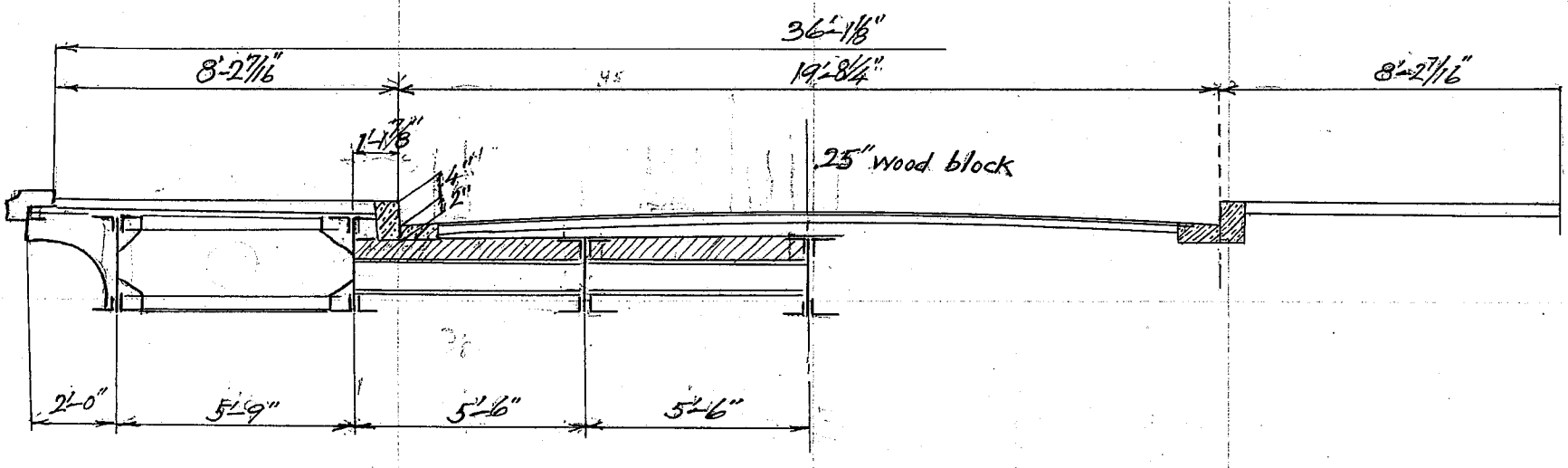
Design of 108A Bridge for City of Tokyo

width of canal 35.0 metres about
Centre span between piers 50'-0" suspended span ~~29'-4½"~~ 29'-4½" with cantilever arm of 10'-3¾" each
side span will be 32'-6" ~~to~~ ℄ of bearings on abutment
The Bridge will be of 20° skew

Roadway 6 metres = 19.685' = 19'-8¼"
2 sidewalks 2.5 metres each = 8.202 = 8'-2 7/16" 2@8'-2 7/16" = 16'-4 7/8"
19'-8¼"
36'-1⅛" effective.

Loading as per specification

Cross section of bridge assumed as sketch below.



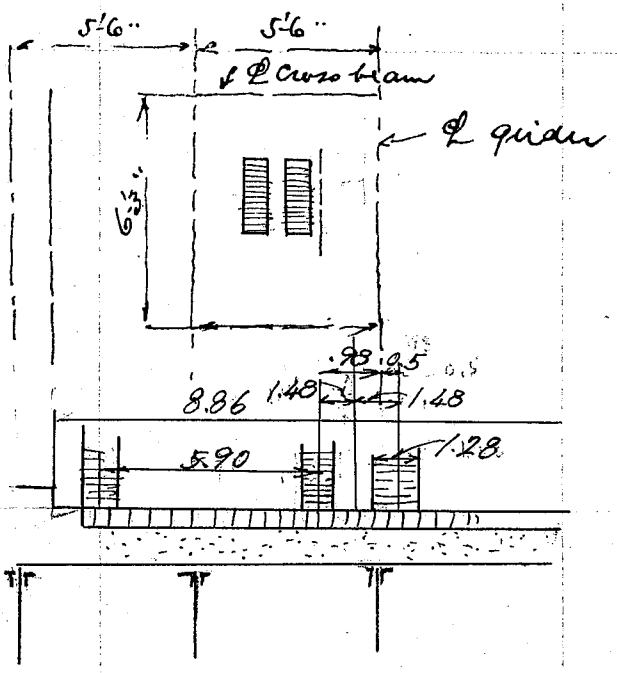
Design of floor slab. span length 6'-3" longitudinally.
Dead Load 3" wood block pavement @ 60#/12 = 15.0
4½" mortar cushion + Concrete fill @ 140 = 56.0
floor slabs assumed .625 @ 150 = 94.0
165.0# per lin ft.

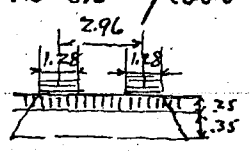
Dead Load moment and 1/10 · 165 · 6.25² = 645 ft#
" " shear = ½ · 165 · 6.25 = 515#

Live Load motor truck loading rear wheel concentration = 9920
30% impact 2980
12900#

Front wheel concentration 12900 ÷ 3 = 4300#

Distribution of wheel concentration on floor slabs.





Distribution for 2 wheels
transv. distribution 2.96
t = 1.28
2d = 1.20
5.44'

2 wheels = 2 · 12900 = 25800#
25800 ÷ 5.44 = 4750# per ft in average.

distribution for single wheel t = 1.28
2d = 1.20
2.48

12900 ÷ 2.48 = 5200# per ft.

wheel B direct on top flange of main girder. flange assumed 1'-0"
distribution on floor slab 1.24 − 0.5 = .74'
Load on slab = 5200 · .74 = 3850#
One wheel 12900
16750#

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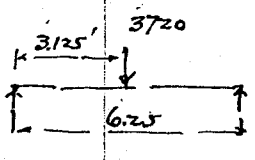
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo

distribution at support 4'-6" assumed

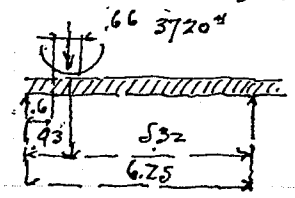
16750 ÷ 4.5 = 3720 # per ft strip



Live Load moment = $\frac{3720}{2} \times 3.125 = 5820$ '#

for Continuity of slab = 5820 × 0.8 = 4650 '#

Live Load shear 3720 # × $\frac{5.32}{6.25}$ = 3170 #



Summary for moment and shear

	moment	shear
Dead Load	645	515
Live Load	4650	3170
	5295 '#	3685 #

Effective depth of slabs for 17000 # steel stress and 640 # Concrete stress n=15

$d = \sqrt{\frac{5295}{102}} = 7.2$" make slab 8¼" over all 1" insulation at bottom of slab.

Steel area required = $\frac{5295 \times 12}{7/8 \times 7.25 \times 17067} = .588$ □" per ft.

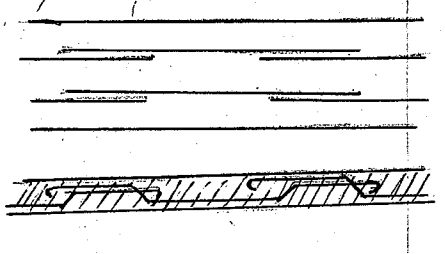
Use ½" bar 4" centers = .589 □"

Unit shear = $\frac{3685}{7/8 \times 7.25 \times 12} = 48.4$ #/□"

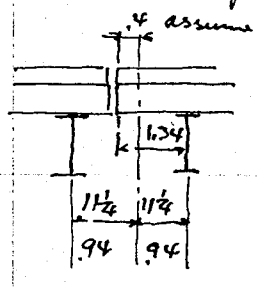
Bond stress = $\frac{3685}{7/8 \times 7.25 \times 7.85} = 240$ #/□" OK (14.0)

3-½" ϕ per ft strip
perimeter = 5 @ 1.57 = 7.85

Use all Reinf - plain round.



Cantilever portion of floor slab. projection 1.34



Dead Load moment = $165 \times \frac{1.34^2}{2} = 148$ '#

Dead Load shear = 165 × 1.34 = 221 #

motor truck concentration at end assumed 3720 # at 1.0' out.

Live Load moment = 3720 × 1.0 = 3720 '#

Live Load shear = 3720 #

Summary for moment and shear

	moment	shear
Dead Load	148	221
Live Load	3720	3720
	3868	3941

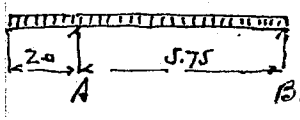
Use ½" ϕ bars 4" centers slab 8¼"
same as for intermediate floor slab.

Sidewalk slabs. span length 5'-9" overhang 2'-0"

Neglecting the stiffness in this slab assume 2.0' cantilever effect of slab. Handrail + C carried by fascia girder directly.

Dead Load slab. assumed 0.5 @ 150 = 75 # per sq ft. including wearing course.

75 #



Cantilever moment of slab. = ½ × 75 × 2² = 150 '#

span AB as simple beam = $\frac{1}{8} \times 75 \times 5.75^2 = 310$ '#
$\frac{1}{2} \times 150$ '# − 75
23.5 '#

Live Load uniform live load 100 # per sq ft.

Live Load moment = ⅘ × 100 × 5.75² = 413
648 '#

End shear say ½ × 175 × 5.75 = 503 #

Effective depth of slab. = $\frac{503}{\text{insulation}} \sqrt{\frac{648}{102}} = 2.52$"
1"
3.52"

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CALCULATIONS FOR

Design of 108ft Bridge for City of Tokyo

Sidewalk Slab. 0.5 slab. at coping 1/100 slope at curb line.
wearing course 3/4"
Insulation at bottom 3/4" effective depth at center of span = 5 3/4 − 1 1/2 = 4 1/4" about.

Steel area = $\frac{648 \times 12}{7/8 \times 4.25 \times 17067} = 0.123$ Use 3/8"ϕ bars 6" centers = 0.22" per ft.

As temperature bar use 3/8"ϕ bars 1-0" about.

Cantilever arm of sidewalk slab. Overhang 2.0'
Effect of floor fascia girder neglected.
approximate Dead Load. Handrail 80# per lin ft.
Coping Stone say 70
fascia girder say 30
180#

Cantilever moment of slab = $75 \times \frac{2.0^2}{2} = 150$
Handrail &c say $180 \times 1.80 = 324$
474 in#

Live Load m = $\frac{1}{2} \times 100 \times 2.0^2 = 200$
674 in#

Section of sidewalk slab ample for shear.
Reinforcing bar for Cantilever = $\frac{674 \times 12}{7/8 \times 4.25 \times 17067} = 0.128$" per ft.

Bend up bottom reinforcement at support 1.0' centers.
used area = 0.110"
part of load carried by fascia girder 0.110" negative reinforcement will be OK over support.

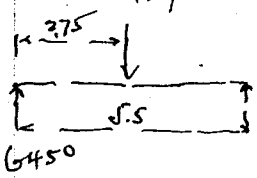
Design of Cross beam span length 5'-6" spacing 6'-3"
Revised dead load.

3" woodblock pavement	@ 60/12	= 15.0
1 1/2 mortar cushion	@ 110#/10	= 11.0
asphalt cloth.		1.0
Concrete fill	.3 @ 140	42.0
Concrete slab.	.69 @ 150	= 104.0
misc Concrete say.		2.0
		175.0 # per sq ft.

Load on cross beam 175. × 6.25 = 1095# per lin ft of span
weight of cross beam say 25
1120#

DL moment = 1/8 × 1120 × 5.5² = 4250 in#
DL shear = 1/2 × 1120 × 5.5 = 3080#

Live Load motor truck loading assumed rear wheel at center of span
rear wheel with impact = 12900#



moment = 6450 × 2.75 = 17700 in#

shear assumed 12900#

Summary for moment and shear

	moment	shear
Dead Load	4250	3080
Live Load	17700	12900
	21950 in#	15980#

section modulus req'd = $\frac{21950 \times 12}{15600} = 16.9$

Use I 8" × 5" @ 28.0# Sm = 22.339

Unit stress = $\frac{21950 \times 12}{22.339} = 11,780$#/in²

増田淳氏関係資料
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CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

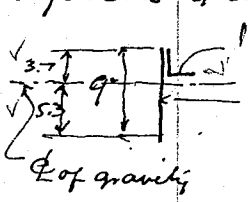
Cantilever Bracket at side walk projection 2'-0" spacing 6'-3"
Handrail, coping stone and fascia girder assumed 180
Dead Load slabs. 75 × 1.0 75
255 #
Live Load 100 # per lin ft.
355 #

Fascia girder. span length 6'-3".
DL moment = 1/8 × 255 × 6.25² = 1245 ft#
DL. shear = 1/2 × 255 × 6.25 = 797 #

Live Load moment = 1/8 × 100 × 6.25² = 488 ft#
Live Load shear = 1/2 × 100 × 6.25 = 312 #

Summary for moment and shear (fascia girder).

	moment	shear
Dead Load	1245	797
Live Load	488	312
	1733 ft#	1109 #



1L 3½ × 3½ × 3/8
9 × 3/8"

		area			Moment of inertia			
1L 3½ × 3½ × 3/8	=	2.480	× 2.49	=	6.18	2.480 × 1.21²	+ 2.9	= 6.54
1Pl. 9 × 3/8	=	3.375	× 4.5	=	15.19	3.375 × 0.8²	+ 2.27	= 4.43
		5.855	3.7		21.37			10.97

5.855
5.855 × .4 = 19.9 ft# 19.9

Fibre stress comp. flange = 1733 × 12 × 3.7 / 10.97 = 7010 #/o- Comp.

Fibre stress tension flange = 1733 × 12 × 5.3 / 10.97 = 10.000 #/o- tension.

The above stresses are for simple beam if make the beam continuous at panel point the stresses will be reduced by (1/8 - 1/10)

weight of fascia girder including details say 26# per lin ft.

Load on Cantilever Bracket
assumed the concentration at end of bracket

Dead Load Concentration		255 × 6.25	= 1590 #
Live Load "		100 × 6.25	= 625
			2215 #
Dead Load moment	=	1590 × 2.0	= 3180 ft#
Live Load "	=	625 × 2.0	= 1250
			4430 ft#

depth of Cantilever at support assumed 1.5'
Flange stress = 4430 / 1.5 = 2960 # SR = 2960 / 17067 = 0.17 0" net.

Use 2-¾"ϕ Rivets for connection of tension flange.
Use structural bracket of ornamental design.
2 L's 3 × 3 × 3/8" for top and bottom flanges.

Strut between main girders under sidewalk. length 5-9"
Try 2 L's 3½ × 3½ × 3/8 radius of gyration 1.07
unsupported length 69" l/r = 69 / 1.07 = 64.5

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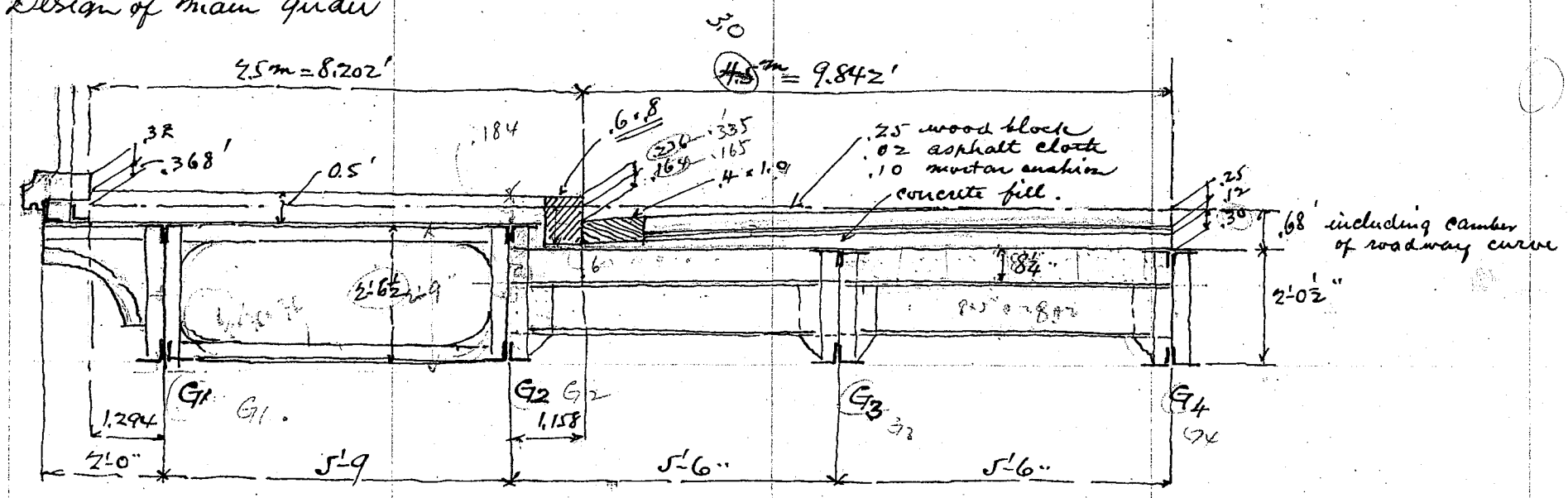
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.
Design of main girder



Dead Load on main girders.

G_4 pavement Concrete fill and slabs.		175		
Cross beam assumed		6		
	181 × 5.5	=	995#	per lin ft.

G_2 and G_3

Load between G_2 and G_3.

				G_3	G_2
Concrete slabs.	104 × 5.5	=	570	× 3.83/5.5	285	285
pavement and fill	70 × 3.342	=	234	× 1.66/5.5	163	71
Gutter	.4 @ 160#	=	64	× .86/5.5	19	45
Curb stone	.6 × .8 × 1.0 @ 160	=	77	×	12	65
Sidewalk	.5 × .56 @ 150	=	42	× .28/5.5	2	40
			987		481#	506#

Load on G_3

		481
Load from G_1-G_2	175 × 5.5/2 =	482
		963
Cross beam assumed		32
		995# per lin. ft.

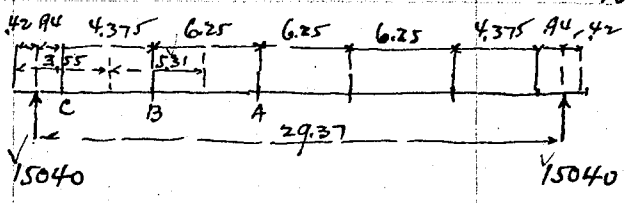
Load on G_2

		506
Sidewalk say	75# × 5.75/2 =	216
Cross beam assumed		33
		755# per lin ft

Load on Girder G1

Sidewalk slab.	75 × 7.75²/(2 × 5.75)	=	392
Handrail, coping + beam	180 × 7.50/5.75	=	235
Bracket and struts	say		28
			655# per lin ft.

Suspended Span.
Girder G_4



Panel Concentration

A	995	× 6.25	=	6220
B	995	× 5.31	=	5280
C	995	× 3.55	=	3540
				15040#

Dead load moment at A.

	15040	× 11.56	=	174.000	
	5280	× 6.25	=	− 33.000	
	3540	× 10.625	=	− 37.600	
				103.400 ft#	103.400 ft#
B	15040	× 5.31	=	79800	
	3540	× 4.375	=	− 15500	
				64300 ft#	
C	15040	× .94	=	14100 ft#	

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CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo

Dead Load girder assumed 200# per lin. Ft.

at	entire	=	1/8 × 200 × 29.37²	=	21600	lbs
"	A	=	1/2 × 200 × 11.56 × 17.81	=	20600	"
"	B	=	" × 5.31 × 24.06	=	12800	"
"	C	=	" × .94 × 28.43	=	2680	"

Summary for Dead Load moment SG4

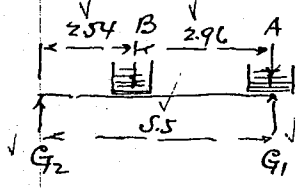
	C	B	A	max entire
Due to ~~motor truck~~ Deck	14100	64300	103400	103400
" " ~~unif. load~~ own weight.	2680	12800	20600	21600
	16780	77100	124000	125000 lbs

Dead Load shear SG4 Due to ~~motor truck~~ deck 15040 (P5)
" unif. load (cost of girder) 200/2 × 29.37 = 2940
17980 call this 18000 #

Live Load motor truck loading impact = 20/(60+8.95) = .29

motor truck rear wheel concentration 9920
impact 29.0% 2880
12800 #

front wheel conc. 12800 ÷ 3 = 4270
17070 #

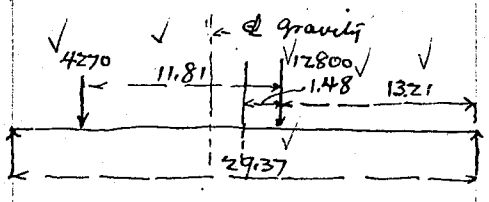


wheel A assumed direct on girder SG4

Reaction from wheel B 2.54/5.5 = .462

4270 × 11.81 = 50400
12800
17070

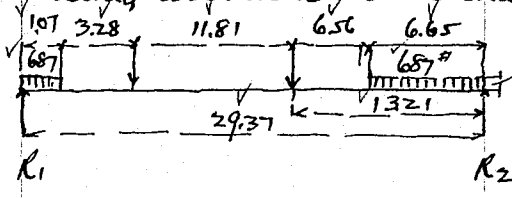
50400 ÷ 17070 = 2.96



absolute max moment = 17070 × 13.21² / 29.37 = 101500 #
moment due to wheel B assumed 46.2% = 46900
148400

Uniform Live Load 125# × 5.5 = 687# per lin. Ft

Loading assumed as in diagram load assumed direct on girder on safe side instead of load reduced to panel load.



This load beyond support neglected

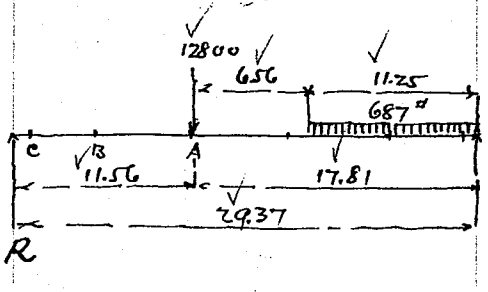
687	× 6.65	= 4560	× 3.33 ÷ 29.37	=	518
687	× 1.07	= 735	× 28.84 ÷ 29.37	=	720
		5295			1238

R1

R2 = 5295 - 1238 = 4057

moment = 4057 × 13.21 = 53700
4560 × 9.89 = -45100
8600 lbs

Live Load moment at Panel Point A

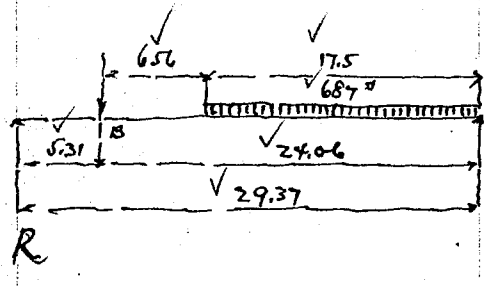


motor truck wheel A R = 12800 × 17.81/29.37 = 7780#
moment 7780 × 11.56 = 90.000
wheel B 46.2% = 41500
131500 lbs

Uniform load 11.25 × 687 = 7730# R = 7730 × 5.62/29.37 = 1480#

moment = 1480 × 11.56 = 17.100 lbs

Live load moment at panel point B.



motor truck wheel A R = 12800 × 24.06/29.37 = 10500#
moment 10500 × 5.31 = 55800 lbs
wheel B 46.2% = 25800
81600 lbs

Unif. load = 17.5 × 687 = 12000 R = 12000 × 8.75/29.37 = 3580#

moment = 3580 × 5.31 = 19000 lbs

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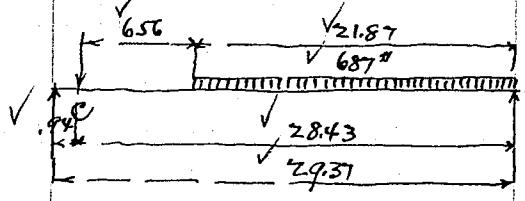
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

Live load moment at C



motor truck wheel A $R = 12800 \times \frac{28.43}{29.37} = 12400$

Moment = 12400 × .94 = 11700 ft#
wheel B 46.2 % 5400
17100 ft#

Uniform load = 21.87 × 687 = 15000 $R = 15000 \times \frac{10.93}{29.37} = 5600$ #
Moment = 5600 × .94 = 5270 ft#

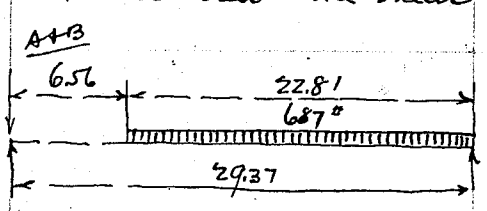
Summary for Live load moment

	C	B	A	Center
Due to motor truck loading	17100	81600	131500	148400
" " Uniform load (687#)	5270	19000	17100	8600
	22370	100600	148600	157000 ft#

Summary for dead and Live Load moments

	C	B	A	absolute max
Dead Load	16780	77100	124000	125000
Live Load	22370	100600	148600	157000
	39150	177700	272600	282000 ft#

Max Live Load End shear or reaction.



motor truck loading wheel A 12800
wheel B 46.2% 5910
18710 #

Uniform load = 22.81 × 687 = 15700 #
Reaction = 15700 × $\frac{11.41}{29.37}$ = 6120

motor truck	18710
Uniform load	6120
	24830

Summary for End Reactions

Dead Load	18000 #
Live Load	24830 #
	42830 #

Section of Girder SG_4 max moment 282 000 ft# max Reaction = 43000 #
web assumed 24 × 3/8 = 9.00" 1/8 web = 1.12 o"
Dry 2 L 6 × 3½ × ½ = 9.00" − .88 = 8.12 o" net
Effective depth = 20.4 − 1.4 = 19.0' 2-¾" rivet out
flange stress = 282.000 ÷ 1.90 = 14850 #
Section required = 148500 ÷ 17067 = 8.70
⅛ web 1.12
7.58 o"

Fibre stress = $\frac{282.000 \times 12 \times 12.25}{2799}$ = 14800 #/o"

4 L 6×3½×½ = 18.0 × 11.42² + 17.2 = 2367
Pl. 24 × ⅜ = 9.0 / 27.0 432
2799
2 L 5×3½×½
Pl. 10½×⅜

Revised 3-2-20

Main Girder SG_3.
Dead Load on SG_3 same as for Girder SG_4
Summary for Dead Load moment (deck slab pavement and own weight). see page 6

Panel point	C	B	A	max Center
	16780 ft#	77100	124000	125000

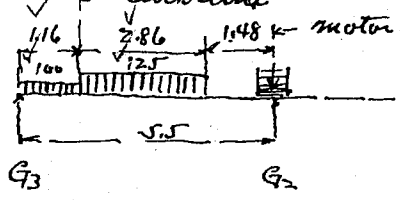
End shear 18000 # (see page 6).

Live Load motor truck loading impact 29.0 % rear wheel one with impact 12800 #
front wheel " " " 4270 #

max motor truck loading one wheel direct on girder.
(see design of main girder SG_4)

panel point	C	B	A	Absolute max
	11700 ft#	55800 ft#	90.000 ft#	101500 ft#

Uniform Live Load



Reaction on G_2

On carriageway	125 × 2.86 = 358	× 2.59/5.5	= 169
On sidewalk	100 × 1.16 = 116	× .58/5.5	= 11
			180 # per lin ft.

at Centre of span	= ⅛ × 180 × 29.37²	= 19300 ft#
at A	= ½ × 180 × 11.56 × 17.81	= 18500 ft#
" B	= " × 5.31 × 24.06	= 11500 "
" C	= " × 0.94 × 28.43	= 2400 "

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CALCULATIONS FOR

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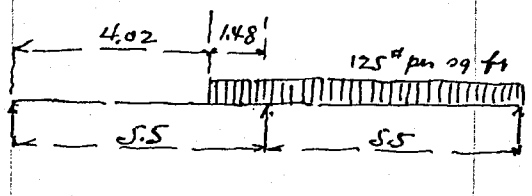
Uniform live load on rear or front of motor truck



$175 \cdot 1.48 = 185 \cdot \frac{1.76}{5.5} = 160$
$125 \cdot \frac{5.5}{2} = 344$
504 # per lin ft of span

Proportion 504 : 687 = .735

Summary for live load (uniform) moment

	C	B	A	Center
Unif. load on side of truck	2400	11500	18500	19300
" " on rear or front.	3900	14000	12600	6300
	6300	25500	31100	25600

73.5% of G1.

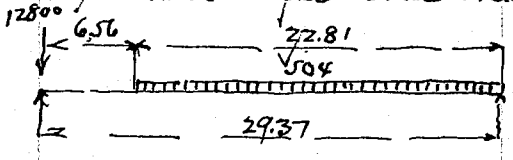
Summary for live load moments

	C	B	A	max
Due to motor truck	11700	55800	90000	101500
" " unif. load	6300	25500	31100	25600
	18000	81300	121100	127100

Summary for Dead and Live Load moments

	C	B	A	max
Dead Load	16780	77100	124000	125000
Live Load	18000	81300	121100	127100
	34780	158400	245100	252100 #

Max Live Load End shear or Reaction. refer to page 7



motor truck loading wheel A 12800
Unif. load $504 \cdot 22.81 \cdot \frac{11.41}{29.37}$ = 4470
Unif. load $\frac{180}{2} \cdot 29.37$ = 2640 } 7110
19910 call this 20.000 #

Summary for End Reaction Dead Load 18000
Live Load 20.000
38.000 #

Section of girder SG2. max moment = 252100 # max shear = 38.000 #
web assumed 24 × 3/8 = 9.0 1/8 web = 1.12 □" b to b of ls 24½" = 2.04
Effective depth = 1.90 flange stress = 252100 ÷ 1.90 = 133000 #
section required = 133.000 ÷ 17067 = 7.79
1/8 web 1.12
6.67
2 ls 5×3½×½ = 8.00 − .88 = 7.12 □" net

Main girder SG2.

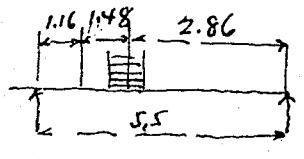
Dead Load on SG2. 755 # per lin ft. $\frac{755}{995} = .76$ of SG_4

Dead Load moment

	C	B	A	Center
Load on girder 76% of G1	10700	48800	78500	78500
weight of girder 200# per lin ft	2680	12800	20600	21600
	13380	61600	99100	100100 #

End shear 15040 × .76 + 2940 = 14340 #

Live Load moment



motor truck loading rear wheel with impact 12800 #
front wheel " " 4270 #

$12800 \times \frac{2.86}{5.5} \cdot (.52) = 6650$ # 52% of single load.

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CALCULATIONS FOR

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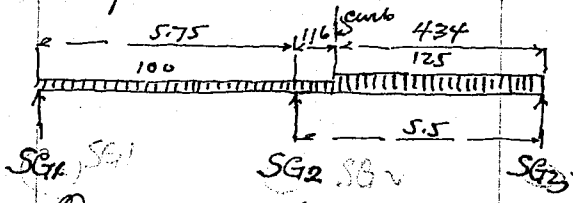
Uniform live load



Load on SG2 SG2

Load on sidewalk. $100 \times 1.16 = 116$ load $\times \frac{4.92}{5.5} = 104$ on SG2

$100 \times \frac{5.75}{2}$ $= 288$

Throughout on girder $392^{\#}$ per lin ft.

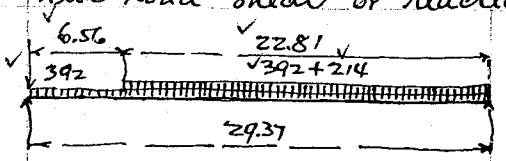
On rear or front of motortruck loading $125 \times 4.34 = 542 \times \frac{2.17}{5.5} = 214^{\#}$ per lin ft.

Live Load moment		C	B	A	Center
motortruck loading	52% wheel A.	6100	29000	46800	52800
Unif. load sidewalk	392/180 = 2.18	5230	25100	40300	42100
" " Roadway	214/687 = .312	1640	5920	5330	2690
		12970	60020	92430	97590

Summary for Dead and Live Load moments

	C	B	A	Center
Dead Load	13380	61600	99100	100100
Live Load	12970	60020	92430	97590
	26350	121620	191530	197690

Live Load shear or reaction



motortruck loading $214 \times 22.81 \times \frac{11.41}{29.37} = 1900$

Uniform load $392 \times \frac{29.37}{2} = 5760$

$14310^{\#}$ } $7660^{\#}$

6650

Summary for dead and live load reactions Dead Load 14340
Live Load 14310
$28650^{\#}$

Section of girder SG2. b to b of angles 30½"

4 L's 5×3½×⅜	= 12.20	× 14.39² + 12.8	= 2533
1 Pl. 30×⅜	= 11.25		844
			3377

⅛ web = 1.41
Effective d. = 2.4

Fibre stress $= \frac{197690 \times 12 \times 16.25}{3377} = 10{,}700^{\#}/\square"$

Flange stress $= \frac{197690}{2.4} = 82{,}300$ SR $= 82300 \div 17067 = 4.82$ □"

2 L's 5×3½×⅜	= 6.18	− 0.66	= 5.44 □" net
⅛ web			1.41
			3.41

Main Girder SG1

Dead Load on SG1 = $655^{\#}$ per lin ft. SG1/SG4 = 655 ÷ 995 = .6575

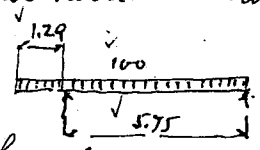
Dead Load moment

		C	B	A	Center
Load on girder	65.75% of SG4	9300	42300	68000	68000
weight of girder	200# per ft	2680	12800	20600	21600
		11980	55100	88600	89600

Dead Load End shear 9900 15040 × .6575 + 2940 = $12840^{\#}$

Live Load. Uniform live load on sidewalk.

$100 \times \frac{7.04 \times 7.04}{2 \times 5.75} = 430^{\#}$ per lin ft.



		C	B	A	Center
Live Load moment	430/180 = 2.39	5730	27500	44200	46100

End shear $\frac{430}{2} \times 29.37 = 6300^{\#}$

Summary for Dead and Live Load moments

	C	B	A	Center
Dead Load	11980	55100	88600	89600
Live Load	5730	27500	44200	46100
	17710	82600	132800	135700 #

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CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo.

End Reaction Dead Load 12840 ✓
Live Load 6300 ✓
19140 #

Section of SG1 b to b of 15 30½" web 30×⅜" = 11.25 ⅛ web = 1.41
Effective depth = 240 flange stress = 135700 / 24 = 5650 SR = 5650 / 17000 = 3.31
⅛ web 1.41
2½×5×3½×⅜ = 6.10 − 0.66 = 5.44 □" net 1.90 □" net
moment of inertia of girder = 3377
flange stress = 135700 × 12 × 15.25 / 3377 = 7330 #/□"

Design of pin.		Pin plate.	Pin assumed	unit bearing.	Bending stress 3" pin good for
SG4	✓ 43000	2−½	3" dia.	✓ 14340 #/□"	63600 "#
SG3	✓ 38000	2−½	〃	✓ 12660 〃	〃
SG2	✓ 28650	2−⅜	〃	✓ 12730 〃	〃
SG1	✓ 19140	2−⅜.	〃	✓ 8500 〃	〃

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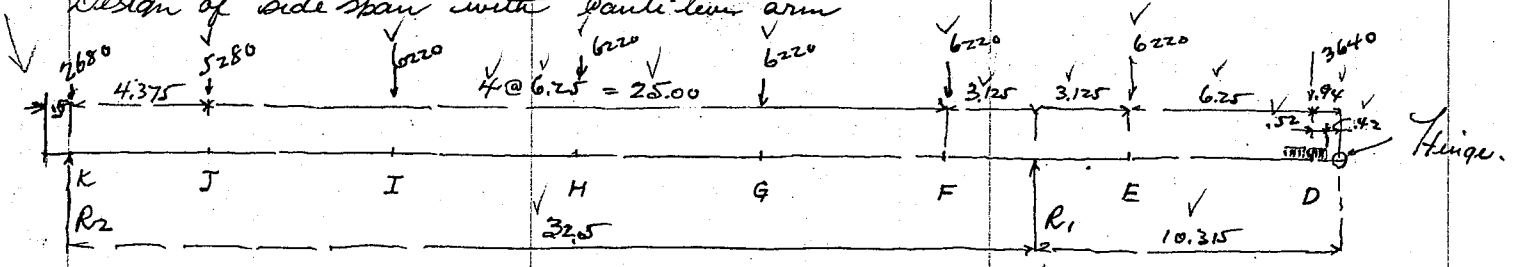
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

Design of side span with cantilever arm



main girder AG4

Cantilever arm

Dead load slab and ✓ 995# per lin. ft
weight of girder assumed ✓ 200# " " "

D.	✓995	× ✓3.66	=	✓3640 #
ΣFGH + I	✓995	× 6.25	=	✓6220 #
J	✓995	× 5.31	=	✓5280 "
K	✓995	× ✓2.69	=	✓2680 "

D1. Concentration at hinge from Girder G1 = ✓18000# (P.6) including weight of girder.

moment due to concentration 18000#

D.	✓18000	× ✓.94	=	16900 ✓
E	✓18000	× ✓7.19	=	129400 ✓
Support	✓18000	× ✓10.32	=	186.000 ✓

moment due to panel concentration of slab &c

E	✓3640	× ✓6.25	=	22750 ft
Support	✓3640	× ✓9.38	=	35200
	✓6220	× ✓3.13	=	✓19500
				54700 ft

moment due to weight of girder 200# per lin ft.

D.	200 ×	✓ $\frac{.94^2}{2}$	=	✓ 90 ft
E	200 ×	✓ $\frac{7.19^2}{2}$	=	✓ 5160
Support	200 ×	✓ $\frac{10.32^2}{2}$	=	✓ 10680 ft

Summary for dead load moment (Cantilever arm AG4)

	D	E	Support
Concentration at hinge 18000	✓ 16900	✓ 129400	✓ 186.000
floor load (concentration)	✓ 0	✓ 22750	54700
weight of girder 200# per ft	✓ 90	✓ 5160	✓ 10680
	✓ 16990 ft	✓ 157310 ft	250380 ft

Shear at support

concentration at hinge		✓ 18000	(P.6)
floor load	6220 + 3640	= ✓ 9860	
main girder	200 × 10.32	= ✓ 2060	
		✓ 29920 #	

Side span AG4

Dead Load stresses.

Dead Load reaction of floor load

✓5280 × ✓4.375 = ✓23100
✓6220 × ✓80.0 = ✓497500
R_1 = ✓520600 ÷ 32.5 = ✓16020 #
R_2 = 30160 − ✓16020 = ✓14140 #

moment due to floor load

F	✓16020	× ✓3.125	=	50.100 ft ✓
G	✓16020	× ✓9.375	=	150.000 ✓
	✓6220	× 6.25	=	− 38900 ✓
				111.100 " ✓
H	✓16020	× ✓15.625	=	250500 ✓
	✓6220	× ✓18.75	=	− 116800 ✓
				133700 " ✓
I	✓16020	× ✓21.875	=	350.000 ✓
	✓6220	× ✓37.5	=	− 233500 ✓
				116500 " ✓
J	✓~~16020~~ 14140	× ✓4.375	=	61900 ft ✓

Negative moment due to concentration at hinge

✓ $\frac{186000}{32.5}$ × ✓29.375 (.904) = 168.000

✓ $\frac{186.000}{32.5}$ × ✓23.125 (.711) = ✓132.200

✓ $\frac{186.000}{32.5}$ × ✓16.875 (.519) = ✓96500 ft

✓ $\frac{186.000}{32.5}$ × ✓10.625 (.327) = ✓60800

✓ $\frac{186.000}{32.5}$ × ✓4.375 (.1345) = ✓25000

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CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

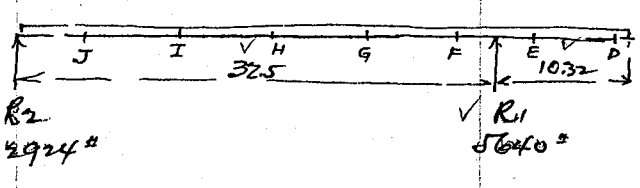
negative moment due to floor load

F	54700 × .904	=	49400 ft
G	54700 × .711	=	38900 ft
H	54700 × .519	=	28400 ft
I	54700 × .327	=	17900 ft
J	54700 × .1345	=	7350

Summary for moment due to floor load. with cantilever arm

	J	I	H	G	F
	61900	116500	133700	111100	50100
	-7350	-17900	-28400	-38900	-49400
	+54550	+98600	+105300	+72200	+0700
Concentration at hinge.	-25000	-60800	-96500	-132200	-168000
	+29550	37800	+8800	-60000	-167300

Dead Load Moment due to weight of main girder 200# per lin ft.



Total load 200 × 42.82 = 8564#

$R_1 = \frac{200 \times 42.82^2}{2 \times 32.5} = 5640$ 5640 − 2924# = R_2

Moment

J	2924 × 4.375	= 12800	G.	2924 × 23.125	= 67700
	$\frac{200}{2} \times 4.375^2$	= -1920		$\frac{200}{2} \times 23.125^2$	= -53400
		+10880 ft			14300 ft
I	2924 × 10.625	= 31100	F	2924 × 29.375	= 86000
	$\frac{200}{2} \times 10.625^2$	= -11300		$\frac{200}{2} \times 29.375^2$	= -86100
		+19800			-100
H.	2924 × 16.875	= 49400			
	$\frac{200}{2} \times 16.875^2$	= -28450			
		+2095o			

Summary for moment due to floor load and weight of girder (combined)

	J	I	H	G	F
floor load	54550	98600	105300	72200	+700
weight of girder	10880	19800	20950	14300	-100
	+65430	+118400	+126250	+86500	+600
due to Concentration at hinge 18000	-25000	-60800	-96500	-132200	-168000
	+40430	+57600	+29750	-45700	-167400

End shear at R2

Due to floor load	141140	
-54700 ÷ 32.5 =	-1680	
		12460#
Due to weight of girder		2924
~~End shear :~~		15384#
~~Floor load at end~~		~~2680~~
~~load on shoe :~~		~~18064~~
Due to concentration at hinge 186.000 ÷ 32.5		-5720
End shear		96644
floor load at end		2680
load on shoe.		12344#

Load on R1

Due to floor load	16020	
+ 54700 ÷ 32.5 =	1680	neg. moment due to slab end
		17700#
Cantilever arm floorload		9860
due to weight of girder		5640
		33200#
		18.000
Due to concentration at hinge 186.000 ÷ 32.5		5.720
Total load on shoe.		56920
floorload.	30160	between supports
girder 200×32.5 =	6500	
	36660	total downward load
- 9664	-9664	" upward "
shear :-	26996	
	29920	
	56916	

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CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo

Live load motor truck loading impact taken same as for suspended span on safe side.

Rear wheel A with impact ✓ 12800 #
Front wheel ✓ 4270 #

Load on girder same as for suspended span SG4 (SG4) motor truck ✓ 1.462 wheels.
uniform load ✓ 687 # per lin ft.
rear or front of motor truck

wheel A (D)	12800	×	0.94 =	12040 ✓	D	6120	× 0.94 = 5750 # ✓
wheel B	46.2%			5560 ✓			
				17600 # ✓	E	6120	× 7.17 = 43900 # ✓
(E) wheel A	12800	×	7.17 =	91800 ✓			
wheel B	46.2%			42500 ✓	Support	6120	× 10.32 = 63200 # ✓
				134300 # ✓			
Support R_1 wheel A	12800	×	10.32 =	132100 ✓	Concentration at hinge due to uniform live load = 6120 # (P.7)		
wheel B	46.2%			61000 ✓			
				193100 # ✓			

Summary for Live Load (Cantilever arm).

	D	E	Support
Due to motor truck 12800 × 1.462	17600 ✓	134300 ✓	193100 ✓
Due to uniform load	5750 ✓	43900 ✓	63200 ✓
	-23350 ✓	-178200 ✓	-256300 # ✓

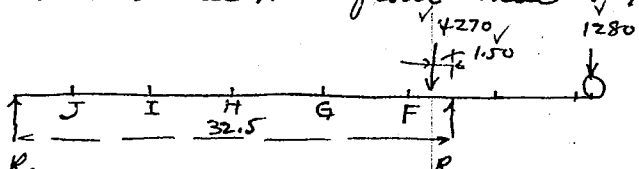
Live load on side span negative moment

Due to wheel A. Rear wheel 12800 #

Due to reaction at hinge (unif. load).

F	132100 / 32.5	×	29.375 ✓	= -119.500 # ✓	63200 / 32.5	×	29.375 ✓	= -57000 ✓
G	"	×	23.125 ✓	= -94200 ✓	"	×	23.125 ✓	= -44900 ✓
H	"	×	16.875 ✓	= -68700 ✓	"	×	16.875 ✓	= -32800 ✓
I	"	×	10.625 ✓	= -43300 ✓	"	×	10.625 ✓	= -20600 ✓
J	"	×	4.375 ✓	= -17800 ✓	"	×	4.375 ✓	= -8500 ✓
K				=				

Due to wheel A front wheel 4270 # on side span



$R_2 = \frac{4270 \times 1.5}{32.5} = 197$ # ✓

		Front wheel A 4270	wheel B 46.2%	Summary #		Rear wheel A 12800	wheel B 46.2%	Summary
J	197 × 4.375 =	+ 860 ✓	400 ✓	= +1260 ✓	F	-119.500 ✓	-55100 ✓	-174600 ✓
I	" × 10.625 =	+ 2090 ✓	965 ✓	= +3055 ✓	G	-94200 ✓	-43500 ✓	-137700 ✓
H	" × 16.875 =	+ 3320 ✓	1530 ✓	= +4850 ✓	H	-68700 ✓	-31700 ✓	-100400 ✓
G	" × 23.125 =	+ 4550 ✓	2100 ✓	= +6650 ✓	I	-43300 ✓	-20000 ✓	-63300 ✓
F	" × 29.375 =	+ 5780 ✓	2670 ✓	= +8450 ✓	J	-17800 ✓	-8200 ✓	-26000 ✓
Support				0				

Summary for negative Live Load Moment

	J	I	H	G	F
Rear wheel	-26000 ✓	-63300 ✓	-100400 ✓	-137700 ✓	-174600 ✓
Front wheel	1260 ✓	3055 ✓	4850 ✓	6650 ✓	8450 ✓
	-24740 ✓	-60245 ✓	-95550 ✓	-131050 ✓	-166150 ✓
Unif. load reaction at hinge	-8500 ✓	-20600 ✓	-32800 ✓	-44900 ✓	-57000 ✓
Grand total.	-33240 ✓	-80845 ✓	-128350 ✓	-175950 ✓	-223150 # ✓

増田淳氏関係資料
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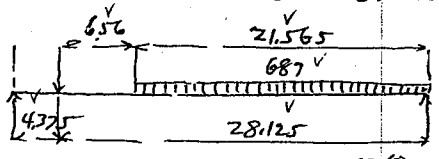
CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

Live load Positive moment

J



wheel A Rear wheel $12800 \cdot \frac{28.125}{32.5} = 11070^{\#}$

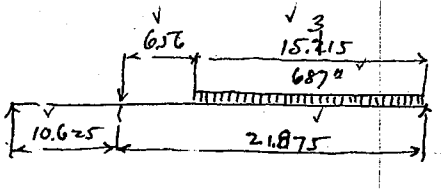
$m = 11070 \cdot 4.375 = 48400$

wheel B 46.2% 22300

Unif load $\frac{687 \cdot 21.565^2}{2 \cdot 32.5} = 4900$ (P6) $m = 4900 \cdot 4.375 =$ 21450

70700 / 92150

I



wheel A Rear wheel $72800 \cdot \frac{21.875}{32.5} = 8610$ / 8570

moment $= 8570 \cdot 10.625 = 91500$

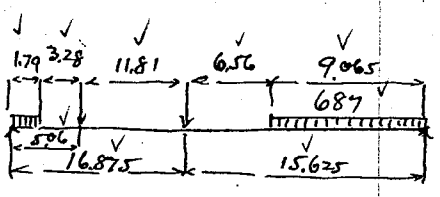
wheel B 46.2% 42200

Unif. load R $= \frac{687 \cdot 15.315^2}{2 \cdot 32.5} = 2480$ / 2450

moment $= 2480 \cdot 10.625 =$ 26400 / 25000

133200 / 158200 / 160100

H.



wheel A

Rear wheel $12800 \cdot \frac{16.875}{32.5} = 6640$

front wheel $4270 \cdot \frac{5.06}{32.5} = 665$

7305

moment $= 7305 \cdot 15.625 = 114200$

wheel B 46.2% 52800

Unif load

$687 \cdot 9.065 = 6230 \cdot \frac{27.97}{32.5} = 5350$

$687 \cdot 1.79 = 1230 \cdot \frac{.90}{32.5} = 34$

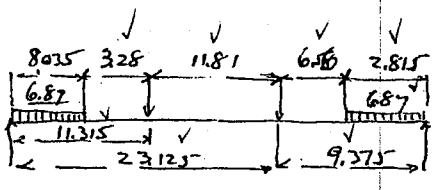
5384

$m = 5384 \cdot 15.625 = 84200$

$6230 \cdot 11.09 = -69200$

167000 / 15.000 / 182.000

G.



wheel A

Rear wheel $12800 \cdot \frac{23.125}{32.5} = 9100$

front wheel $4270 \cdot \frac{11.315}{32.5} = 1490$

10590

moment $= 10590 \cdot 9.375 = 99200$

wheel B. 46.2% 45750

Unif. load

$687 \cdot 2.815 = 1935 \cdot \frac{3109}{32.5} = 1852$

$687 \cdot 8.035 = 5525 \cdot \frac{4.02}{32.5} = 693$

7460 / 2545

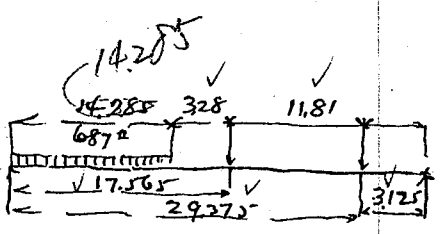
$m = 2545 \cdot 9.375 = 23850$

$1935 \cdot 7.97 = -15400$

8450

153400 / 144950

F



wheel A

rear wheel $12800 \cdot \frac{29.375}{32.5} = 11570$

front wheel $4270 \cdot \frac{17.565}{32.5} = 2305$

13875

moment $= 13875 \cdot 3.125 = 43400$

wheel B 46.2% $= 20050$

Unif. load $687 \cdot 14.285 \cdot \frac{7.14}{32.5} = 2160$ / 2160

$m = 2160 \cdot 3.125 =$ 6750 / 6750

63460 / 70200 / 70200

Summary for Live load Positive moment

	J	I	H	G	F
due to motor truck	48400	91500	114200	99200	43400
46.2%	22300	42200 / 26400	52800	45750	20050
Unif. load 687#	21450	25000	15000	8450	6750
Total	+ 92150	+ 158200 / 160100	+ 182.000	+ 153400	+ 70200 / + 70200

増田淳氏関係資料
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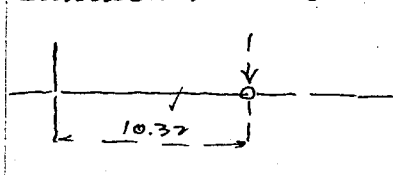
CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo.

Live load shear and reaction.

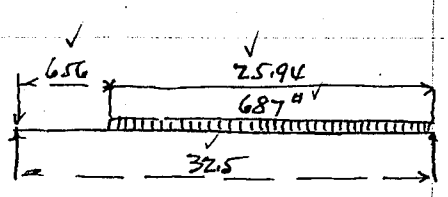
Cantilever arm



motor truck loading wheel A 12800 ✓
wheel B 46.2%. 5910 ✓
18710 ✓
Uniform load on suspended span 6120 ✓
24830 # ✓

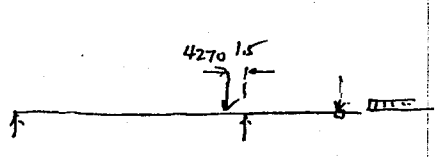
moment at support = 256300 # ✓

Side span On abutment End shear or reaction.



max positive wheel A ✓ 12800 ✓
wheel B 46.2% = 5910 ✓
18710 ✓

Uniform load 687 × 25.94² / (2 × 32.5) = 7110 ✓
+ 25820 # ✓

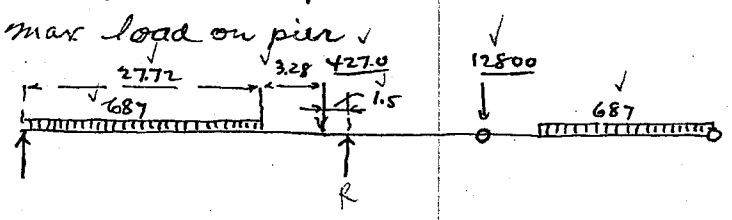


max negative
front wheel 197 ✓
46.2% 91 ✓
288 ✓ say 300 ✓

256300 ✓ ÷ 32.5 ✓ = − 7880 ✓
+ 300 ✓
− 7580 # ✓

Side span On pier.

max load on pier ✓

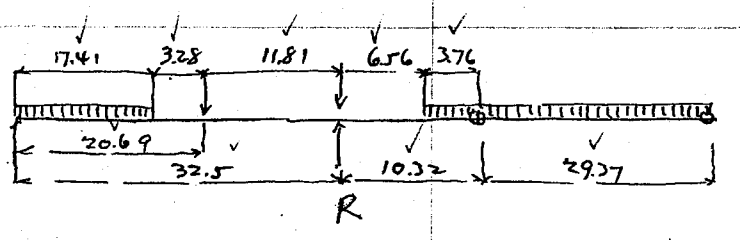


front wheel 4270 × 31.0 / 32.5 = 4070 ✓
wheel B. 46.2%. 1880 ✓
5950 ✓

Uniform load 687 × 27.72² / (2 × 32.5) = 8120 ✓
14070 # ✓

direct load from cantilever arm 24830 ✓
Extra reaction due to cantilever 7880 ✓
46780 # ✓

Side span End shear on pier.



wheel A Rear ✓ 12800 ✓
front 4270 × 20.69 / 32.5 = 2710 ✓
15510 ✓

wheel B 46.2% ✓ 7160 ✓
22670 ✓

Uniform load 687 × 17.41² / (2 × 32.5) = 3210 ✓
25880 # ✓

3880 ✓
29760 # ✓

Extra reaction due to cantilever moment
Reaction from suspended span = 29.37 × 687/2 = 10100 ✓
moment = 10100 × 10.32 = 104400 ✓
Unif. 687 × 3.76 × 8.44 = 21800 ✓
126200 ✓ ÷ 32.5 = 3880 ✓

Summary for moment (dead and negative live load)

	J	I	H	G	F	On pier	E	D
Dead Load moment	+40430	+57600	+29750	−45700	−167400	−250380	−157310	−16990
Live load neg.	−33240	−89845	−128350 ✓	−175950 ✓	−223150 ✓	−256300 ✓	−178200 ✓	−23350 ✓
	+ 7190 ✓	− 22245	−98600 ✓	−221650 ✓	−390550 ✓	−506680 ✓	−335510 ✓	−40340 ✓

	J	I	H	G	F	On pier
Dead Load	+ 40430 ✓	+ 57600 ✓	+ 29750 ✓	− 45700 ✓	− 167400 ✓	−250380 ✓
Live Load Pos.	+ 92150 ✓	+ ~~158200~~ 169100 ✓	+ 182000 ✓	+153400 ✓	+ 70200 ✓	0
	+ 132580 ✓	+ ~~215800~~ 217700 ✓	+ 211750 ✓	+109700 ✓	− 97200 ✓	−250380 ✓

増田淳氏関係資料
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CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo.

Summary for End shears and reactions

Cantilever arm
Dead Load 29920 ✓ (P11)
Live Load 24830 ✓
54750 # ✓

Side span on abutment.
Shear. max
Dead Load 9664
Live Load 25820
35484 #

Min load on shoe. Dead Load · 9664 + 2680 ✓ = 12344
Live Load - 7580 ✓
4764 #

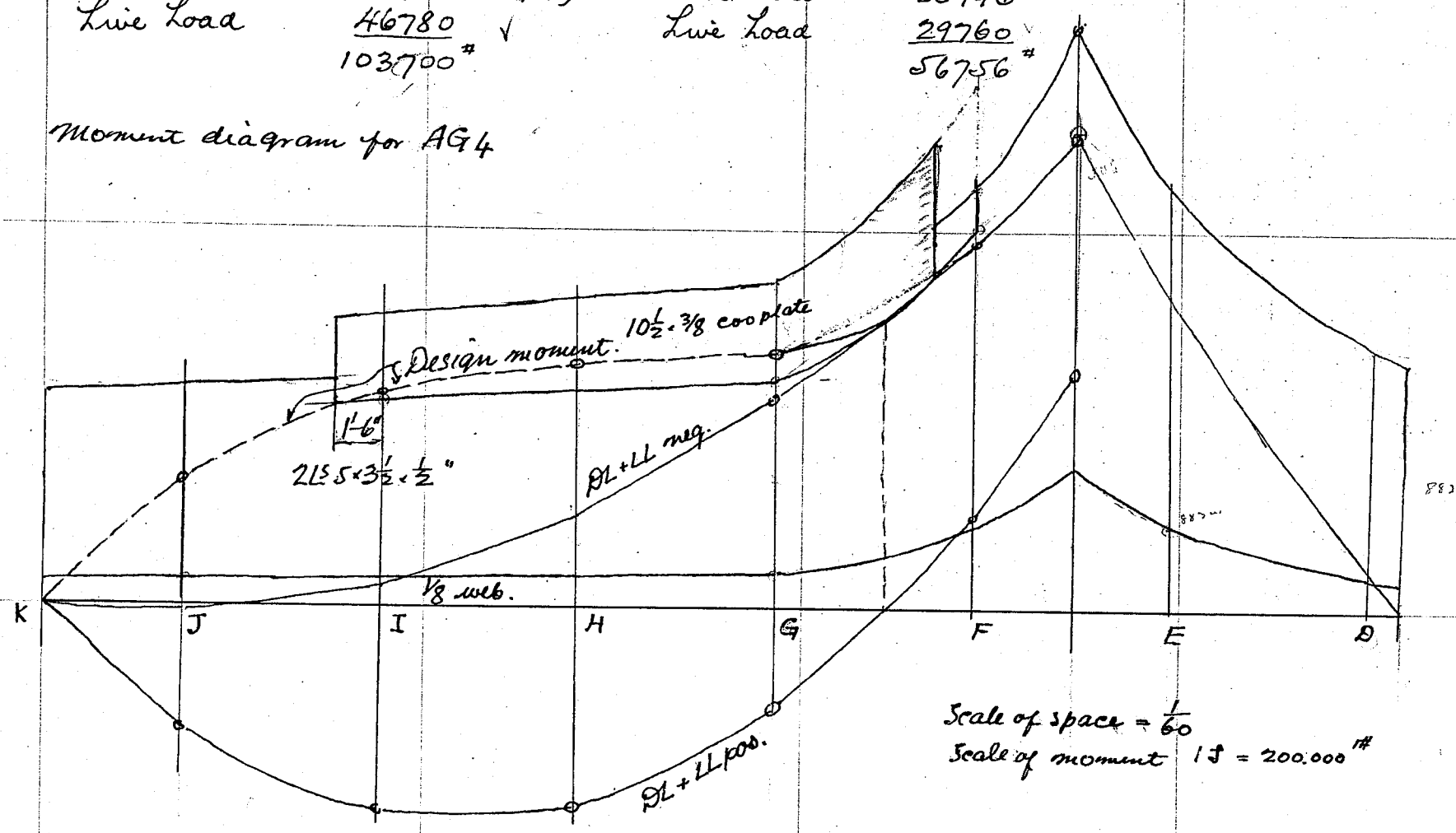
Max Load on shoe · Dead Load 12344
Live Load 25820
38164 #

Side span shear and load on pier.

Load on pier.
Dead Load 56920 (P.12)
Live Load 46780 ✓
103700 #

End shear
Dead Load 26996
Live Load 29760 ✓
56756 #

Moment diagram for AG4



Scale of space = 1/60
Scale of moment 1" = 200.000 #

Section of AG4 ✓ ✓ ✓ ✓ ✓ ✓ ✓
Support:- 2 L 5 × 3½ × ½ @ 4.00 = 8.80 - .88 = 7.12 " net
web 48 × 3/8 = 18.00 " 1/8 web = 2.25 "
Effective depth = 4.04 - .14 = 3.9'
flange stress = 506680 / 3.9 = 129.600 #
SR = 129.600 ÷ 17067 = 7.6 / 2.25 / 5.35 " ✓

F. Depth = 3'-1½" web = 37 × 3/8" = 13.88 "
1/8 web = 1.74 " Effective depth = 2.98
flange stress = 390550 / 2.98 = 131.000 #
SR = 131.000 ÷ 17067 = 7.67 / 1.74
5.93 " net

G. Depth 2'-0 3/4" 24½ × 3/8 = 9.08 " 1/8 web = 1.13
depth effective = 1.97
2 L 5 × 3½ × ½ = 8.00 — 3.28
1 Pl. 10½ × 3/8 = 3.94 — 6.24
9.52 "
Stress (m) 221650
½ × 107700 53850
275500 #
flange stress = 275500 / 1.97 = 139800 #
SR = 139800 ÷ 17067 = 8.19 / 1.13
7.06 " net

H. 1'-11½" Section same as G
web = 23 × 3/8 = 8.630 " 1/8 web = 1.070 "
Effective depth = 1.87'
Stress (m) 98600 / 222585 = 217700 / 11623 / 229323 / 229323 / 261050
flange stress = 261050 / 1.87 = 139600 #
139600 / 17067 = 8.19 / 1.07
7.12 " net

I. depth = 1'-11" 22½ × 3/8" = 8.45 " 1/8 web = 1.05 "
depth = 1.92 - 0.09 = 1.83
stress = 217700 / 11623 / 229323 / 229323
23245 = 23245 / 2
flange stress = 229323 / 1.83 = 125000
SR = 125000 / 17067 = 7.33 / 1.05
6.28 " net

J depth = 1'-10¼" 21¾ × 3/8 = 8.2 1/8 web = 1.02 "
stress = 132580 # Effective depth = 1.71
flange SR stress = 132580 ÷ 1.71 = 77500 #
SR = 77500 ÷ 17067 = 4.58 - 4.58 / 1.02 - 1.02
3.56 " net
3.56

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CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

Main girder AG3
floor load 995# per lin ft } same as for AG4.
Girder 200# per lin ft }
Dead load stresses of Cantilever arm and side span same as for AG4

Live load motor truck wheel A only with impact Rear wheel 12800#
Front wheel 4270#
Uniform live load on rear and front of motor truck 504# per lin. ft. p8
This loading to the loading of AG4 504 : 687 = .735
or stresses will be of 735% of AG4
Uniform live load side of motor truck 180# per lin ft. p7.

Reaction at hinge motor truck loading 12800#
Uniform live load 7110# (p8.)

	Due to motor truck				Due to uniform load				
D.	wheel load								
	wheel A.	12800	× 0.94	= 12040	7110	× 0.94	= 6690		hinge reaction
					180	× $\frac{0.94^2}{2}$	= 80		U_2.
E	"	12800	× 7.17	= 91800	7110	× 7.17	= 51000		hinge reaction
					180	× $\frac{7.17^2}{2}$	= 4630		U_2
Support		12800	× 10.32	= 132100	7110	× 10.32	= 73500		hinge reaction
					180	× $\frac{10.32^2}{2}$	= 9600		U_2

Summary for live load (Cantilever arm).

	D	E	Support
Due to motor truck	12040	91800	132100
" " Uniform load	6690	51000	73500
	18730	142800	205600
" " Unif. U_2 180	80	4630	9600
	18810	147430	215200

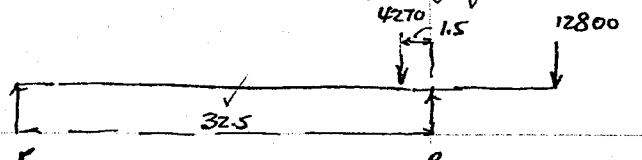
Live Load on side span negative moment

Due to wheel A Rear wheel at hinge = 12800

Due to unif. load at hinge U_2 = 9600

F	$\frac{132100}{32.5}$	× 29.375	= − 119500	$\frac{73500}{32.5}$	× 29.375	= − 66400		− 8700
G	"	× 23.125	= − 94200	"	× 23.125	= − 52300		− 6800
H	"	× 16.875	= − 68700	"	× 16.875	= − 38200		− 5000
I	"	× 10.625	= − 43300	"	× 10.625	= − 24000		− 3100
J	"	× 4.375	= − 17800	"	× 4.375	= − 9900		− 1300

Front wheel on side span



$R_2 = 4270 \cdot \frac{1.5}{32.5} = 197.$

J	F	197 × 4.375	=	+ 860
I	G	" × 10.625	=	+ 2090
H	H	" × 16.875	=	+ 3320
G	I	" × 23.125	=	+ 4550
F	J	" × 29.375	=	+ 5780

Summary for negative Live Load Moment

	J	I	H	G	F
Rear wheel 12800	− 17800	− 43300	− 68700	− 94200	− 119500
Front wheel 4270	+ 860	+ 2090	+ 3320	+ 4550	+ 5780
	− 16940	− 41210	− 65380	− 89650	− 113720
Unif. load at hinge 79360 + 73500	− 9900	− 24000	− 38200	− 52300	− 66400
	− 26840	− 65210	− 103580	− 141950	− 180120
Unif. load U_2	− 1300	− 3100	− 5000	− 6800	− 8700
Grand summary →	− 28140	− 68310	− 108580	− 148750	− 188820

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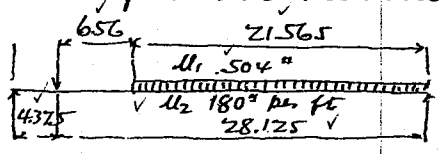
CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo

Live Load, positive moment of side span.

J



504 ÷ 687 = .735

Rear wheel A $12800 \times \frac{28.125}{32.5} = 11070$

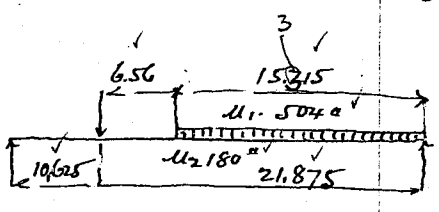
$M = 11070 \times 4.375 = 48400$ ft#

Uniform load M_1 $21450 \times .735 = 15750$

M_2 Unif. load $\frac{180}{2} \times 4.375 \times 28.125 = 11100$

75250 ft#

I



wheel A Rear wheel $12800 \times \frac{21.875}{32.5} = 8610$ #

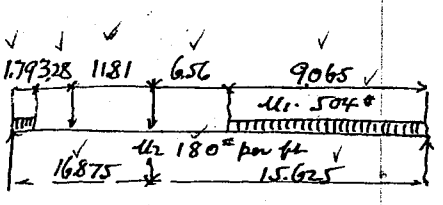
moment = $8610 \times 10.625 = 91500$ ft#

Unif load M_1 $26400 \times .735 = 19400$

Unif. load M_2 $\frac{1}{2} \times 180 \times 10.625 \times 21.875 = 20800$

131700 ft#

H



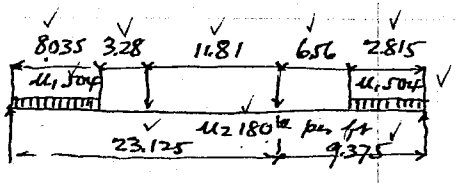
Rear and front wheel ~~114200 × .735~~ = 114200

M_1 $15000 \times .735 = 11000$

M_2 $\frac{1}{2} \times 180 \times 16.875 \times 15.625 = 23800$

~~118800~~ 148000 ft#

G.



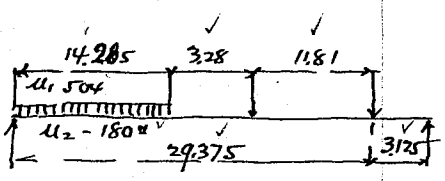
Rear and Front wheel 99200×1.0 = 99200

M_1 $8450 \times .735$ = 6210

M_2 $\frac{1}{2} \times 180 \times 23.125 \times 9.375$ = 19500

124910 ft#

F



Rear and front wheel 6,750

M_1 $6750 \times .735$ = ~~4960~~ 4960

M_2 $\frac{1}{2} \times 180 \times 29.375 \times 3.125$ = 8250

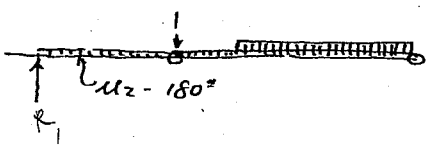
~~56610~~ 56610

Summary for Positive Live Load Moment

	J	I	H	G	F
Due to motor truck	48400	91500	114200	99200	~~63450~~ 43400
Unif. Rear and front of truck 504#	15750	19400	11000	6210	~~5010~~ 4960
Unif. side of motor truck 180#	11100	20800	23800	19500	8250
	75250 ft#	131700	148000	124910	~~76710~~ 56610 ft#

Live Load shear and reactions

Cantilever arm



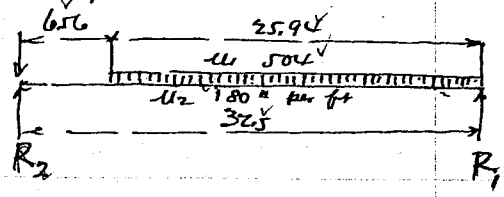
motor truck loading 12800

Unif. load on suspended span 7110

M_2 180×10.32 = 19910 #

moment at support = ~~205600~~ 215200 ft# ~~1860~~ 21770 # 1860. 21,770.

Side span End shear or load on abutment



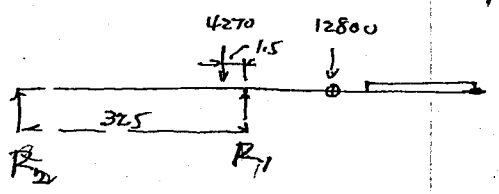
max positive shear or reaction

motor truck loading 12800

Unif. M_1 $7110 \times .735$ = 5220

" M_2 $\frac{1}{2} \times 180 \times 32.5$ = 2920

20940 #



max negative shear or reaction

motor truck and uniform load on suspended span ~~205600~~ 215200 ÷ 32.5 = 6620 − 6320

Front wheel positive reaction say + 200

− 6420 #

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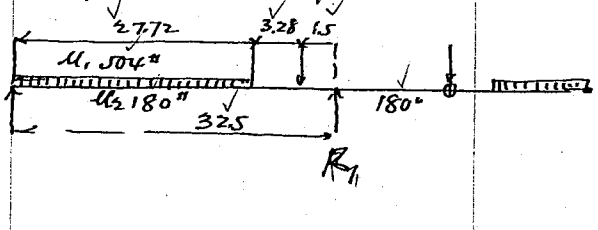
CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo

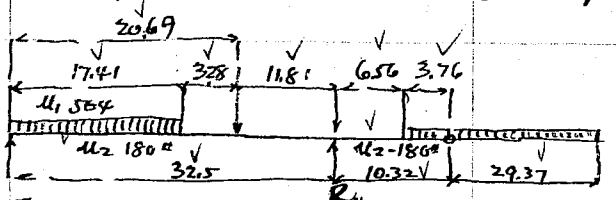
Side span shear end load on pier.

Max load on pier



front wheel	4270 · 31.0/32.5	=	4070
Uniform. load u_1	8120 · .735	=	5970
Uniform. load u_2	½ · 180 · 32.5	=	2920
			12960
Direct from cantilever			20940
Extra due to cantilever	215200 ÷ 32.5	=	6620
			40520 #

Side span max end shear at pier.



Rear wheel			12800
Front wheel	4270 × 20.69/32.5	=	2710
			15510
u_1 -	3210 · .735	=	2360
u_2 -	½ · 180 · 32.5	=	2920
			20790
Extra due to cant. moment			3990
			24780 #

End reaction of suspended span

504 + 180 = 684#

29.37 · 684/2 = 10050 #

Moment	10050 · 10.32	=	104.000
u_1	504 · 3.76 × 8.44	=	16.000
u_2	180 · 10.32²/2	=	9600
			129600 ÷ 32.5 = 3990

Summary for moment (dead and live loads)

	J	I	H	G	F	on pier	E	D
Dead Load.	+ 40430	+57600	+29750	-45700	-167400	-250380	-157310	-16990
Live load neg.	- 28140	-68310	-108580	-148750	-188820	-215200	-147430	-18810
	+12290	-10710	-78830	-194450	-356220	-465580	-304740	-35800

					+56,610.		
Dead Load	+40430	+57600	+29750	-45700	-167400	-250380	
Live Load pos.	+75250	+131700	+148000	+124910	+56610	0	
	+115680	+189300	+178750	+79210	-110790	-250380	

-110,790.

Summary for end shear and reaction

Cantilever arm

Dead Load	29920
Live Load	21770 21,770.
	51690#
	51,690.

Side span shear max

on abutment

Dead Load			9664
Live Load			20940
			30604
Max load on shoe			
Dead Load	9664 + 2680	=	12344
Live Load			20940
			33284 #
Min load on shoe			
Dead Load			12344
Live Load neg.		-	6420
			5924 #

Side span on pier.

Load on pier	
Dead Load	56920
Live load	40520
	97440 #
	4

End shear

Dead Load	26996
Live Load	24780
	51776 #

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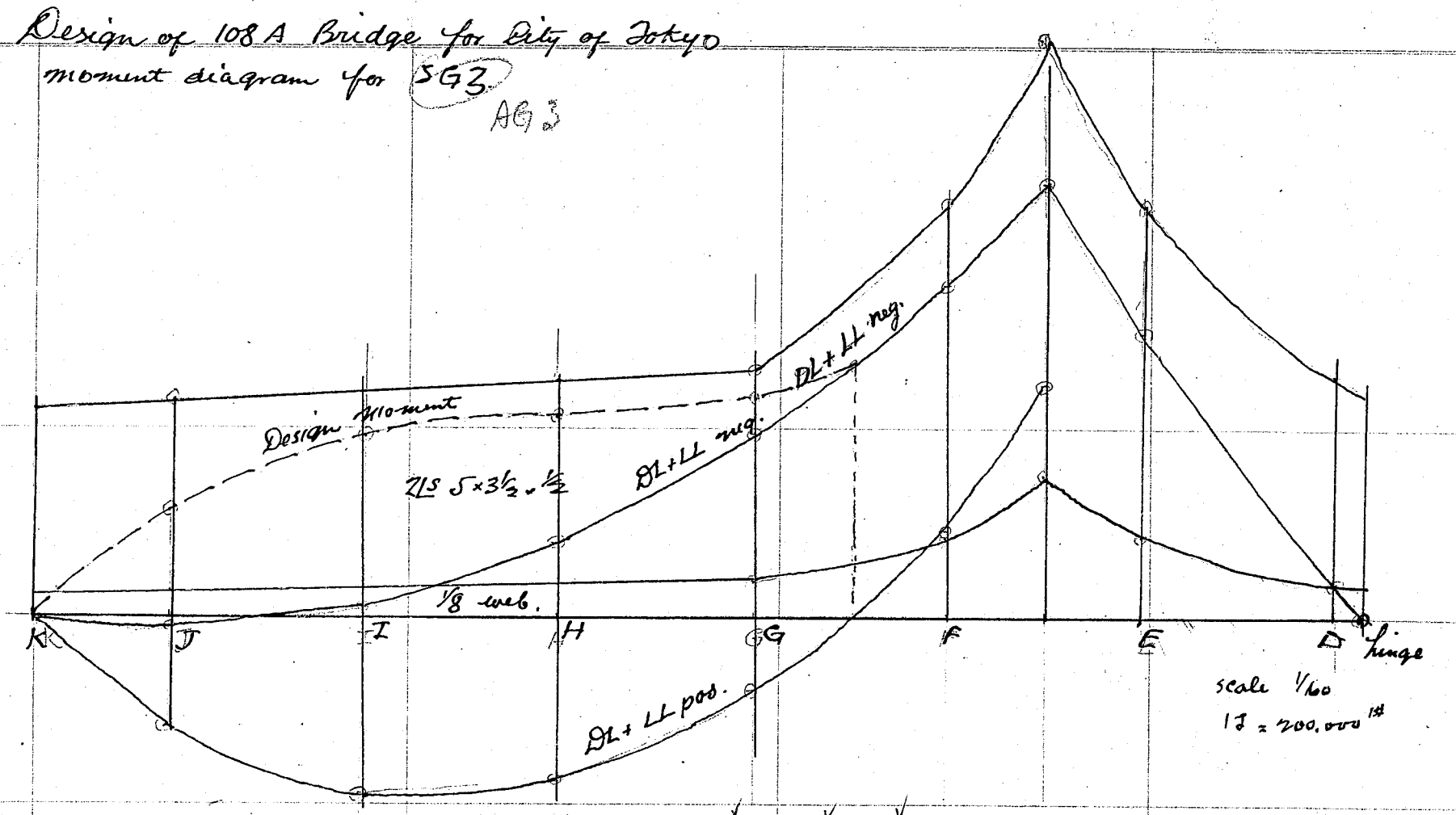
CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo
moment diagram for ~~SG3~~ AG3



scale 1/60
1 ♯ = 200.000 ♯

Section of AG3 flange = 2 LS 5×3½×½ = 8.00 - .88 = 7.120" net.
Support depth of girder same as for SG1

flange stress = 465580 / 3.9 = 119500 #
SR = 119500 ÷ 17067 = 7.00 / 2.35 / 4.75 □" net
⅛ web

F flange stress = 356220 / 2.98 = 119500 #
SR = 119500 ÷ 17067 = 7.00 / 1.74 / 5.26 □" net
⅛ web

G. Design moment 194450 Effective depth 1.92'
79210 ÷ 2 = 39605
234055
flange stress = 234055 / 1.92 = 122000 #
SR = 122000 ÷ 17067 = 7.15 / 1.13 / 6.020" net
⅛ web

H. Design moment = 178750 Eff. depth 1.82
78830 ÷ 2 = 39415
218165 #
flange stress = 218165 / 1.82 = 119900
SR = 119900 ÷ 17067 = 7.08 / 1.07 / 5.960" net
⅛ web

I. moment = 189300
10710 ÷ 2 = 5355
194655 #
flange stress = 194655 / 1.78 = 109200 #
SR = 109200 ÷ 17067 = 6.40 / 1.05 / 5.35 □" net
⅛ web

J. flange stress = 115680 / 1.71 = 67600 #
SR = 67600 ÷ 17067 = 3.97 / 1.02 / 2.950" net
⅛ web

Main side girder AG2.
floor load 755 # per lin ft. Stress proportional to AG4 755 ÷ 995 = .76
weight of girder assumed 200 # per lin ft. Load at hinge = 14340
Dead Load stresses
Cantilever arm
Dead Load moment (refer to p11).

	D	E	Support
Concentration at hinge 14340/18000 = .797	13480	103.000	148.000
floor load 76.9%	0	17300	41600
weight of girder 200 # per ft.	90	5160	10680
	13570 #	- 125460 #	- 200280 #

Shear at support

Concentration at hinge	14340
floor load 76% × 9860 =	7500
main girder 200 × 10.32 =	2060
	23900 #

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CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

Side span AG2 between supports

Dead load reactions of floor load (P.11)

R_1 at pier	16020 ✓	×	.76 ✓	= 12200 #✓
R_2 at abutment	14140 ✓	×	.76 ✓	= 10770 #✓

Summary for moment due to floor load and weight of girder see p12.

	J	I	H	G	F
floor load 76.0%	41500 ✓	75000 ✓	80.000	54900 ✓	530 ✓
weight of girder 200#	10880 ✓	19800 ✓	20950 ✓	14300 ✓	− 100 ✓
	52380 ✓	94800 ✓	100950 ✓	69200 ✓	430 ✓
Concentration at hinge .797	−19900 ✓	−48500 ✓	−76800 ✓	−105300 ✓	−134000 ✓
	+32480 ✓	+46300 ✓	+24150 ✓	−36100 ✓	−133570 ✓

End shear at abutment R_2

Due to floor load ✓	10770 ✓	
Cantilever effect 44600 ÷ 32.5 = ✓ ✓	− 1280 ✓	
	9490 #✓	
weight of girder	2924 ✓	
	12414 #✓	
Due to concentration at hinge		
148.000 ✓ ÷ 32.5 ✓	− 4550 ✓	
End shear →	+ 7864 ✓	
floor load at end panel 2680 ✓ × .76 ✓ =	2040 ✓	
Load on shoe	9904 #✓	

Load on pier R_1

Due to floor load	12200 ✓	
Cantilever effect	1280 ✓	
	✓ ✓ 13480 ✓	
Cantilever arm floor load 9860 × .76 =	7500 ✓	
due to weight of girder	5640 ✓	
	26620 ✓	
Concentration at hinge	14340 ✓	
Cantilever effect	4550 ✓	
Total load on shoe	45510 #✓	

Shear: — ✓ ✓

floor load 30160 × .76 =	22900 ✓
weight of girder 200 × 32.5 =	6500 ✓
Total downward load	29400 ✓
upward reaction	− 7864 ✓
End shear: —	21536 #✓

Live Load motor truck loading (P8) 12800 ✓ × 2.86 ✓ / 5.5 ✓ = 6650 ✓ # 52% of single concentration

4270 ✓ × 2.86 / 5.5 = 2220 ✓ #

Uniform Live Load U_1 rear or front of motor truck 214# per lin ft ✓ 214 / 687 ✓ = 0.312 (P9)

Uniform Live Load U_2 side of motor truck throughout on girder 392# ✓ per lin. ft (P9)

Load at hinge due to uniform load 7660# ✓ see p9

Summary for Live Load moment (cantilever arm). see page 17.

	D	E	Support
Due to motor truck ✓ 6650 / 12800 ✓ = .52	6260 ✓	47800 ✓	68800 ✓
Concentration at hinge ✓ 7660 ÷ 7110 ✓ = 1.08 ✓	7210 ✓	55100 ✓	79300 ✓
Uniform load U_2. 392 : 180 = 2.180 ✓	170 ✓	10100 ✓	20900 ✓
	−13640 #✓	−113000 #✓	−169000 #✓

Summary for negative Live Load moments see p17.

	J	I	H	G	F
Rear wheel 52% ✓	− 9250 ✓	− 22500 ✓	− 35700 ✓	− 49000 ✓	− 62100 ✓
front wheel 52%. ✓	447 ✓	1090 ✓	1730 ✓	2360 ✓	3000 ✓
	− 8803 ✓	− 21410 ✓	− 33970 ✓	− 46640 ✓	− 59100 ✓
Uniform load at hinge 1.08 ✓ (79300 ÷ 73500)	− 10700 ✓	− 25900 ✓	− 41250 ✓	− 56500 ✓	− 71700 ✓
Uniform load U_2. 2.180 ✓	− 2840 ✓	− 6750 ✓	− 10900 ✓	− 14800 ✓	− 19000 ✓
	− 22343 ✓	− 54060 ✓	− 86120 ✓	− 117940 ✓	− 149800 ✓

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CALCULATIONS FOR

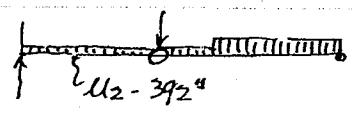
Design of 108 A Bridge for City of Tokyo

Summary for positive live load moment

	J	I	H	G	F
motor truck, 52%	25200	47500	59300	51600	~~22600~~ 22,600
Unif. u_1 214 : 504 = 42.5%	6700	8250	4680	2640	~~2110~~ 2,110
Unif. u_2 392 ÷ 180 = 2.180	24200	45300	51800	42500	18000
	56100 #	101050	115780	96740	~~42710~~ 42,710. #

Live load shears and reactions.

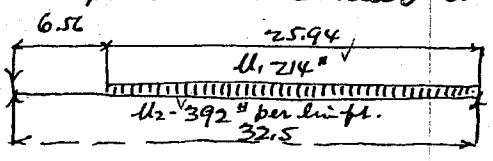
Cantilever arm



motor truck loading 6650
Unif. load on suspended span 7660
Unif. 392 ÷ 180 = 2.180 @ 1860 = ~~4060~~ 4,060.
~~18370~~ 18,370. #

Moment at support = -169000

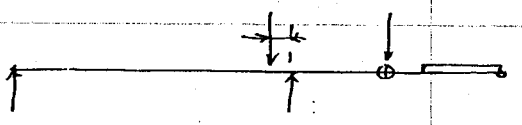
Side span End shear or load on abutment



See page 18

Max positive shear or reaction

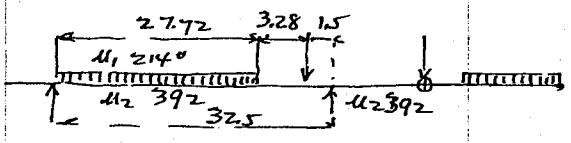
motor truck loading		6650
u_1	425 × 5220 =	2220
u_2	2.18 × 2920 =	6360
		15230 #



Max negative shear or reaction

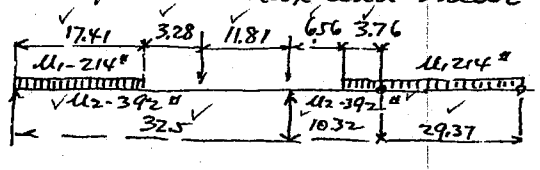
motor truck and uniform load on suspended span	169000 ÷ 32.5 =	- 5200
front wheel on side span	.52 × 200 say	+ 100
		- 5100 #

Side span ~~Shear and~~ load on pier.



front wheel	4070 × .52 =	2120
Unif. load u_1 $\frac{214}{504}$ = .425	@ 5970 =	2540
Unif. load u_2	2.18 × 2920 =	6370
		11030 #
Direct from cantilever		~~18370~~ 18,370.
Extra due to cantilever	169,000 ÷ 32.5 =	5200
		~~34690~~ 34,690. #

Side span max end shear at pier.



End reaction of suspended span

214 + 392 = 606 #

29.37 × $\frac{606}{2}$ = 8900

moment	8900 × 10.32 =	92000
u_1	214 × 376 × 8.44 =	6800
u_2	392 × $\frac{10.32^2}{2}$ =	20900
		119700 ÷ 32.5 = 3680

Rear wheel 6650
Front wheel 2920 × $\frac{20.69}{32.5}$ = ~~1350~~ 1,350.
~~8000~~ 8,000.
u_1 2360 × .425 = 1000
u_2 ½ × 392 × 32.5 = 6360
~~15360~~ 15,360.

Extra due to cant. moment 3680
19040 #
19040

Summary for dead and live load moments

	J	I	H	G	F	on pier	E	D
Dead Load	+32480	+46300	+24150	-36100	-133570	-200280	-125460	-13570
Live Load neg.	-22343	-54060	-86120	-117900	-149800	-169000	-113000	-13640
	+10137	-7760	-61970	-154000	-283370	-369280	-238460	-27210
Dead Load	+32480	+46300	+24150	-36100	-133570	-200280		
Live Load pos.	+56100	+101050	+115780	+96740	+42760	0		
	+88580	+147350	+139930	+60640	-90860	-200280		

42,710.

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CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo.

Summary for end shear and reactions

Cantilever arm
Dead Load 23900 ✓
Live Load. 18370 — 18,370
42270# ✓
42,270.

Side span on abutment
shear max
Dead Load 7864 ✓
Live Load. 15230 ✓
23094 # ✓

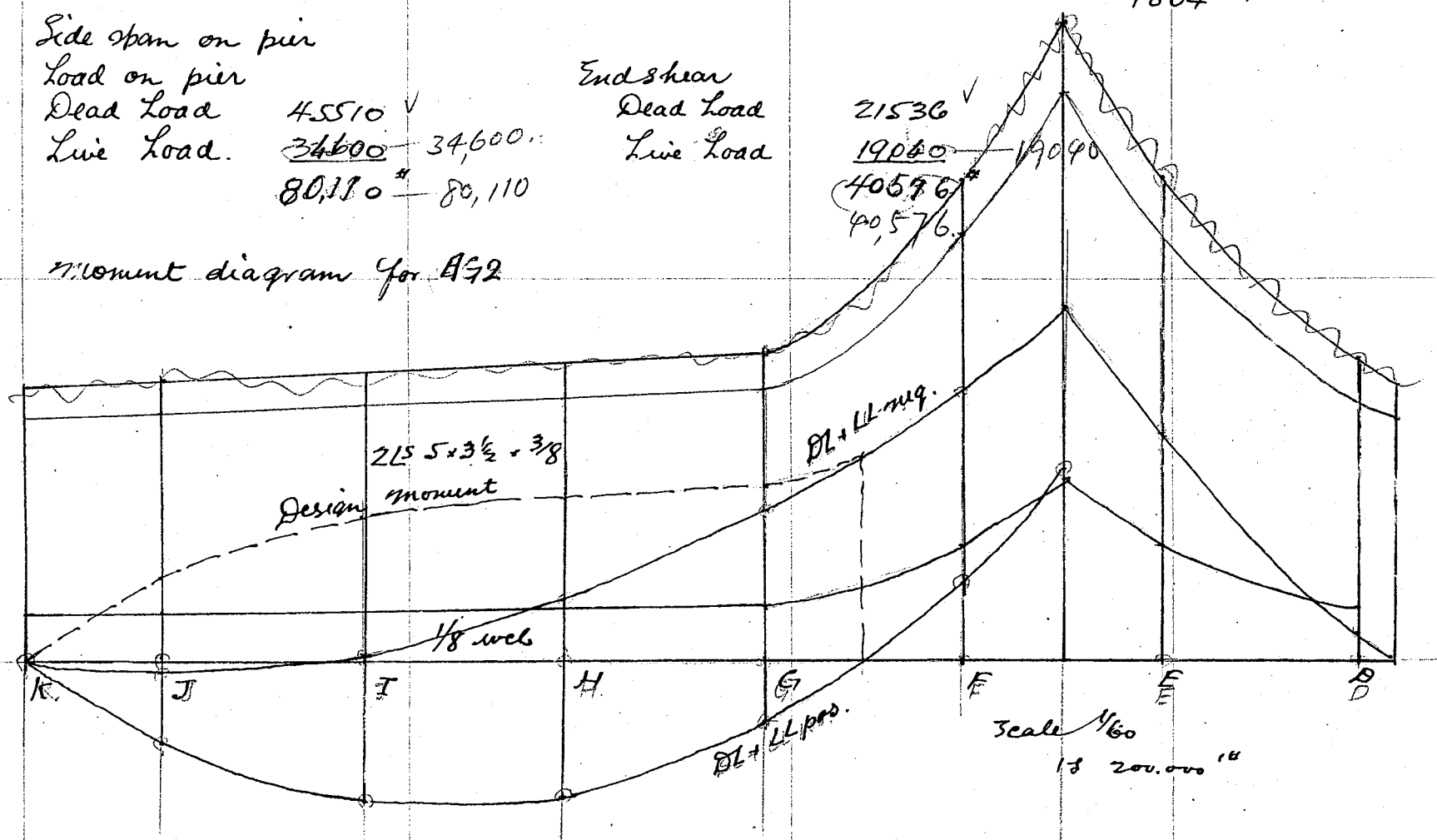
max load on shoe
Dead Load 9904 ✓
Live Load 15230 ✓
25134 # ✓

Min Load on shoe
Dead Load 9904 ✓
Live Load — 5100 ✓
4804 # ✓

Side span on pier
Load on pier
Dead Load 45510 ✓
Live Load. 34600 — 34,600.
80,110 # — 80,110

End shear
Dead Load 21536 ✓
Live Load 19060 — 19,060
40596 # ✓
40,596.

Moment diagram for AG2



Section of AG2: 2L 5×3½×3/8 - 6/10 - 66 = 5.44 □" net

Support 4-6½" b to b. web .54×3/8" = 20.25 1/8 web = 2.53□"
Effective depth = 4.4 flange stress = 369280/4.4 = 83800 ✓
SR = 83800 ÷ 17067 = 4.90
1/8 web = 2.53
2.37□" net

A. 2-6" b to b. 29½×3/8 = 11.05□" 1/8 web = 1.38□"
moment 139930
61970÷2 30985
170915#
stress = 170915/2.36 = 72400 #
SR = 72400 ÷ 17067 = 4.25
1/8 web 1.38
2.87□" net

F 3-8" b to b. 43½×3/8 = 16.3□" 1/8 web = 2.04□"
Effective depth = 3.52 stress = 283370/3.52 = 80500 #
SR = 80500 ÷ 17067 = 4.72
1/8 web 2.04
2.68□" net

I. 2-5½" b to b 29×3/8 = 10.88□" 1/8 web = 1.36□"
moment 147380
7760÷2 = 3880
151230
stress = 151230/2.32 = 65200 #
SR = 65200 ÷ 17067 = 3.82
1.36
2.46□" net

G 2-6¾" b to b 30¼×3/8 = 11.32□" 1/8 web = 1.41□"
Effective depth 2.42
moment 154000
66640÷2 = 30320
184320
stress = 184320/2.42 = 76000
SR = 76000 ÷ 17067 = 4.46
1/8 web 1.41
3.05□" net

J 2-4½" b to b. 28×3/8 = 10.50 1/8 web = 1.31□"
depth 2.23
flange stress = 88580/2.23 = 39700 #
SR = 39700 ÷ 17067 = 2.33□"
1/8 web 1.31
1.02□" net

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CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo

Main Side girder AG4-
floor load 655# per lin ft Stresses proportional to AG4 655 ÷ 995 = .6575
weight of girder assumed 200# per lin. ft load at hinge = 12840
Dead Load Stresses
Cantilever arm
Dead Load Moment refer to P11.

		D	E	Support
Concentration at hinge	12840 ÷ 18000 = .713	12050	92400	132900
floor load 65.75%.			45000	36000
weight of girder 200# per lin. ft.		90	5160	10680
		-12140	-112560	-179580

Shear at support

Concentration at hinge		12840
floor load	9860 · 6575 =	6480
girder	200 · 10.32 =	2060
		21380#

Side span AG4 between supports
Dead Load Reactions of floor load

R_1 at	pier	16020	.6575 =	10550#
R_2 at	abutment	14140	.6575 =	9300

Summary for Dead Load Moment See page 12

	J	I	H	G.	F
floor load 65.75%	35900	64800	69300	47500	460
weight of girder 200#	10880	19800	20950	14300	- 100
	46780	84600	90250	61800	360
Concentration at hinge 12840/18000 = .713	-17800	-43300	-68800	-94500	-119800
	+28980	+41300	+21450	-32700	-119440

End Shear at abutment R_2

Due to floor load		9300
Cantilever effect 36000 ÷ 32.5 =		- 1110
		8190
weight of girder		2924
		11114
Concentration at hinge 132900 ÷ 32.5 =		- 4090
End shear		7024#
floor load at end panel 2680 · .6575 =		1760
load on shoe		8784#

Load on pier R_1

Due to floor load		10550
Cantilever effect		1110
		11660
Cantilever arm floor load 9860 · .6575 =		6480
due to weight of girder		5640
		23780
Concentration at hinge		12840
Cantilever effect		4090
Total load on shoe		40710
Shear		
floor load 30160 · .6575 =		19800
weight of girder 200 · 32.5 =		6500
Total downward load		26300
upward reaction		-7024
		19276#

Live Load Uniform Live Load on sidewalk 430# per lin ft see p9
Load at hinge due to uniform load 6300# see p10.

Live Load Moment Cantilever arm (see page 17)
Unif. load 430# per lin ft.

	D	E	Support
Concentration at hinge 6300 ÷ 7110 = .885	5920	45100	65000
Unif. M2 430# per ft ÷ 180 = 2.39	190.	10800	22900
	-6110	-55900	-87900

増田淳氏関係資料
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CALCULATIONS FOR

Design of 108A Bridge for City of Tokyo

Side span between supports

Summary for negative live Load moments — See page 17

	J	I	H	G	F
Liuge reaction $\frac{65000}{73500} = .88$	-8750	-21200	-33800	-46250	-58600
Unif. U_2 $430 \div 180 = 2.39$	-3100	-7400	-11950	-16250	-20800
Summary →	-11850	-28600	-45750	-62500	-79400

Summary for positive live loads moments

	J	I	H	G	F
Unif. U_2. $430 \div 180 = 2.39$	26500	49700	56800	46600	19700

Live load shears and reactions

Cantilever arm

Uniform load on suspended span 6300
Unif. U_2 $430 \div 180 = 2.39$ @ 930 = 2220 — 4,450.
8520 # — 10,750.

Moment at support = 87900 # — 1860.

Side span End shear or load on abutment

End shear Uniform load $430 \times \frac{32.5}{2} = 7000$ #

Max negative shear or reaction $87900 \div 32.5 = -2700$ #

Side span ~~shear and~~ load on pier.

Uniform load 430 # $\times \frac{32.5}{2} = 7000$
direct from cantilever 8520 — 10,750.
Extra due to cantilever effect $87900 \div 325 = 2760$
18280 # — 20,510.

Side span max end shear on pier

End reaction of suspended span 6300 #

$m = 6300 \times 10.32 = 65000$
$430 \times \frac{10.32^2}{2} = 22900$
$87900 \div 32.5 = -2700$

Unif. load $= 430 \times \frac{32.5}{2} = 7000$
2700
9700 #

Summary for dead and Live Load moments

	J	I	H	G	F	on pier	E	D
Dead Load	+28980	+41300	+21450	-32700	-119440	-179580	-112560	-12140
Live Load neg.	-11850	-28600	-45750	-62500	-79400	-87900	-55900	-6110
	+17130	+12700	-24300	-95200	-198840	-267480	-168460	-18250
Dead Load	+28980	+41300	+21450	-32700	-119440	-179580		
Live Load pos.	+26500	+49700	+56800	+46600	+19700	0		
	+55480	+91000	+78250	+13900	-99740	-179580		

Summary for end shears and reactions

Cantilever arm
Dead Load 21380
Live Load 10750 — 10,750
32130 #
32,130.

Side span on pier. (Load on pier).
Dead Load 40710
Live Load 20580 — 20,510.
61290 # — 61,220.

Shear at pier
Dead Load 19276
Live Load 9700
28976 #

Side span on abutment
shear max
Dead Load 7024
Live Load 7000
14024

Max load on shoe
Dead Load 8784
Live Load 7000
15784 #

Min load on shoe
Dead Load 8784
Live Load -2700
6084 #

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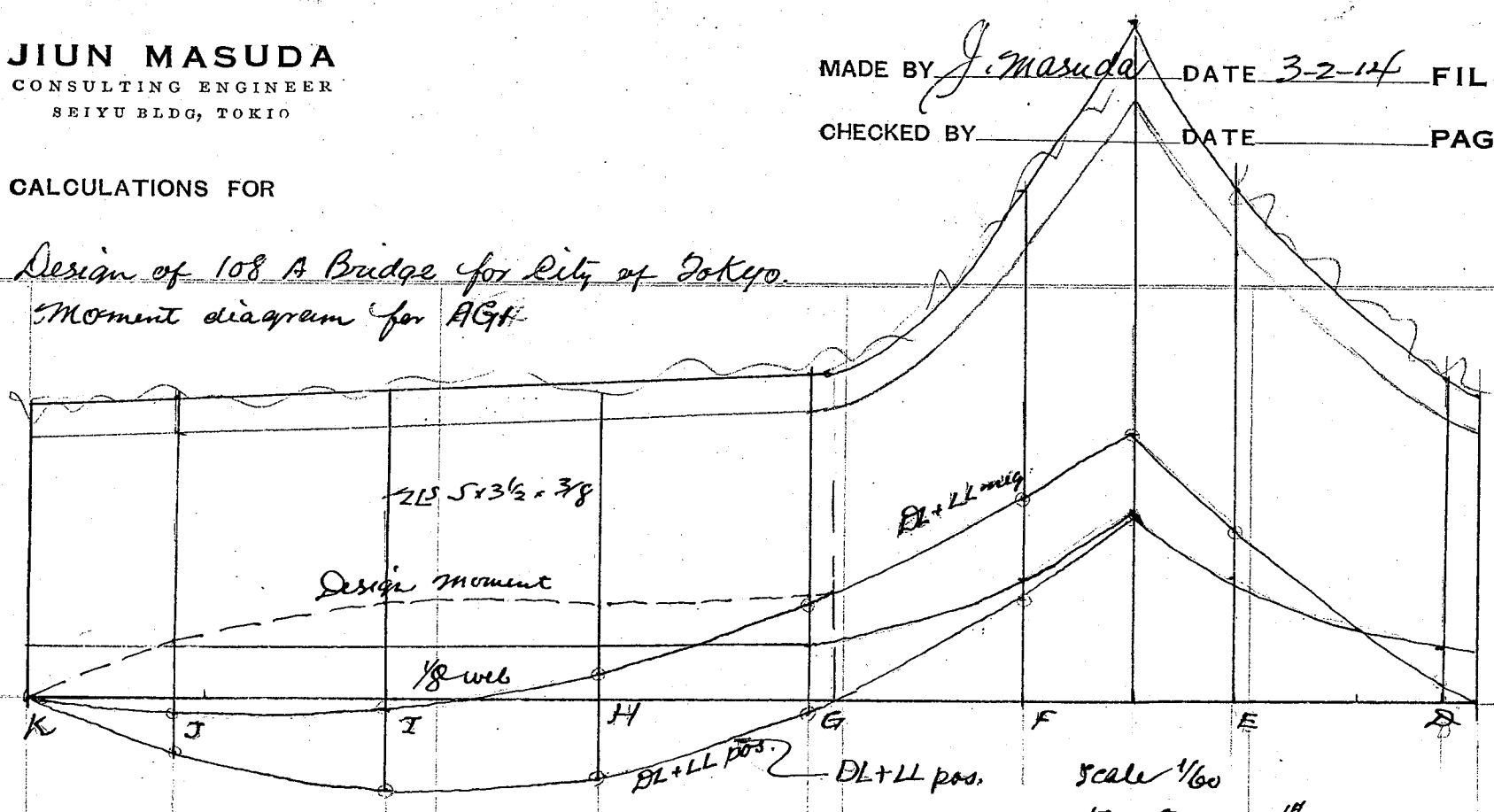
CALCULATIONS FOR

Design of 108 A Bridge for City of Tokyo.
Moment diagram for AG1



Scale 1/60
1" = 200.000 lb

Section of AG1. 2LS 5-3½-⅜ = 6.10 - .66 = 5.44 o" net
depth of girder same as for AG2.

Support
flange stress = 267480 / 4.4 = 60750
SR = 60750 ÷ 17067 = 3.57
⅛ web 2.53
1.04 o" net

F flange stress = 198840 / 3.52 = 56400
SR = 56400 ÷ 17067 = 3.310
⅛ web 2.04
1.27 o" net

G moment, 95200
13900 ÷ 2 = 6950
102150
flange stress = 102150 / 2.42 = 42200
SR = 42200 ÷ 17067 = 2.48
⅛ web 1.41
1.07

H. moment 78250
24300 ÷ 2 = 12150
90400
flange stress = 90400 / 2.36 = 38400
SR = 38400 / 17067 = 2.25
⅛ web 1.38
.87 o" net

I.
flange stress = 91000 / 2.32 = 39200 #
SR = 39200 ÷ 17067 = 2.30
⅛ web 1.36
.940" net

J. flange stress = 55480 / 2.23 = 24900 #
SR = 24900 ÷ 17067 = 1.46
⅛ web = 1.31
.15 o" net.

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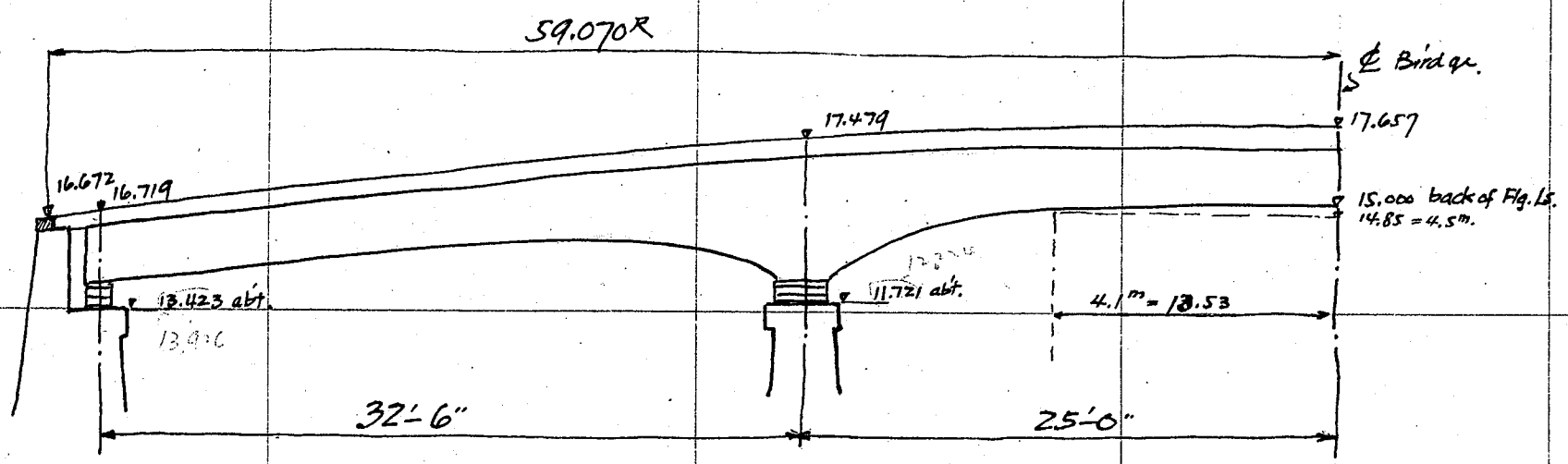
CALCULATIONS FOR

108A Bridge for City of Tokyo.
Design of Substructures.
Super Imposed Load on Pier and Abutment.



Abutment
Load on Shols.

Girder	D.L.	L.L.	Total load
SG1	12,344#	25,820#	38,164
SG2	12,344	20,940	33,284
SG3	9,904	15,230	25,134
SG4	8,784	7,000	15,784

Total load on abutment 1-SG1, 2-SG2, SG3, SG4.

74,408	112,160	186,568
wt. of Shoes. say 2,592		
77,000#	112,160	189,160

Call this 190,000#

Pier
Load on Shols.

D.L.	L.L.	Total Load.
56,920	46,780	103,700
56,920	40,420	97,340
45,510	32,570	78,080
40,710	18,280	58,990

Total load on Pier 1-SG1, 2-SG2, SG3, SG4.

343,200	229,320	572,520
Shoes. say 4800		
348,000#	229,320	577,320

Call this 578,000#

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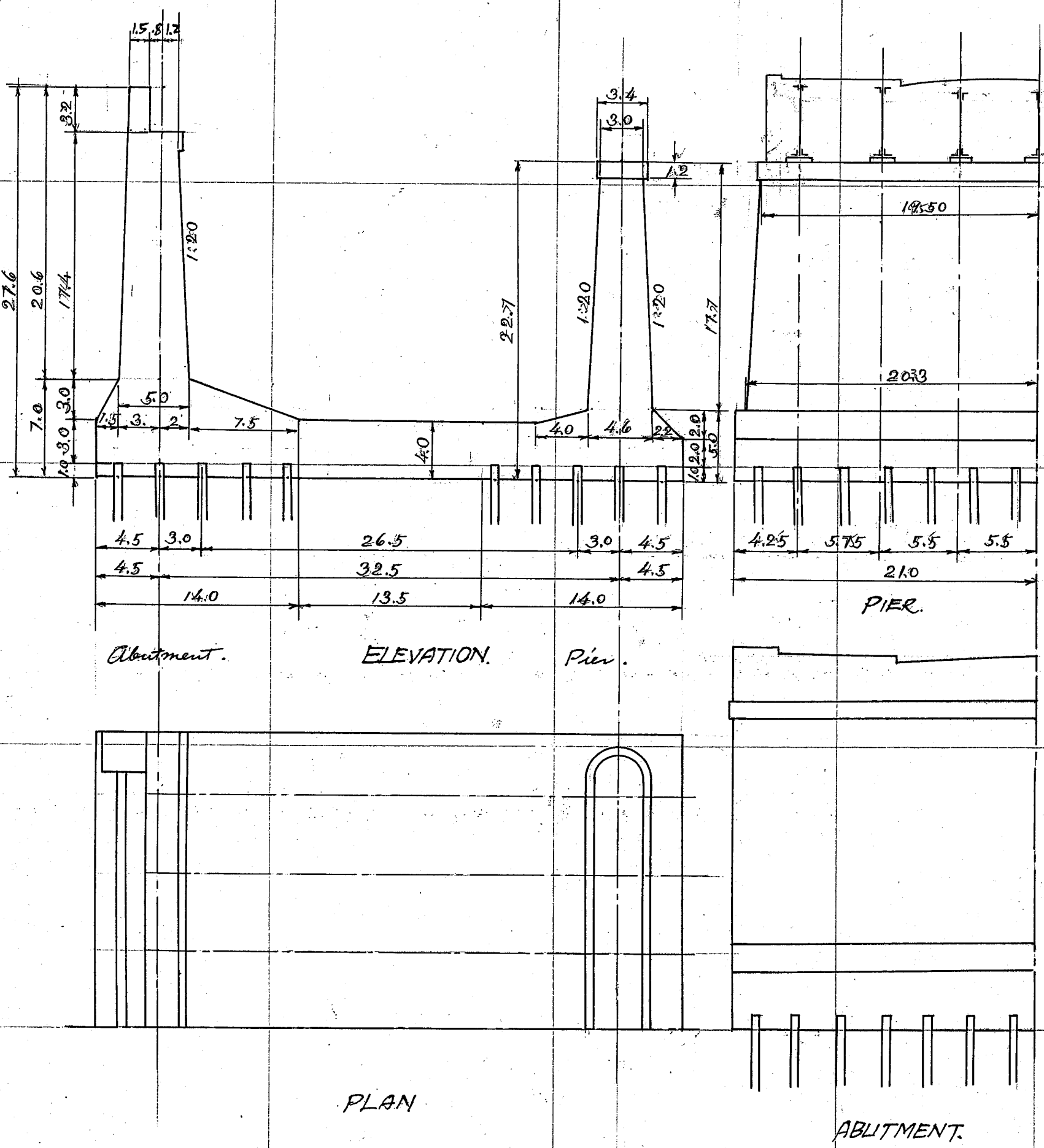
CALCULATIONS FOR

108 A Bridge for City of Tokyo.
Sketch for Abutment and Pier.



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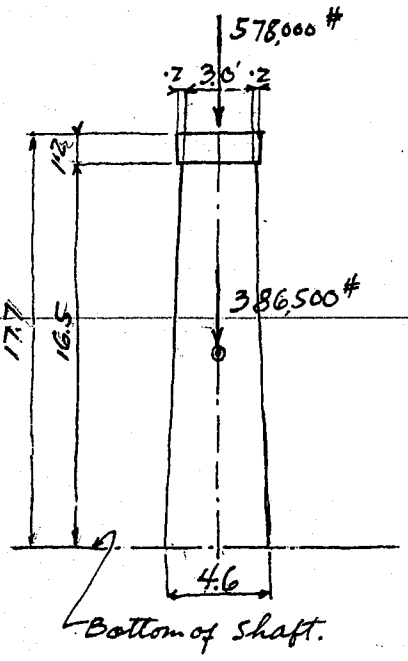
CALCULATIONS FOR

108 A Bridge for City of Tokyo.

Design of Pier
Stability of Shaft.
Case 1. Normal state (full load).



Center of gravity of Shaft.

	section	mean length	volume			lev.arm	moment
Coping	3.4 × 1.2	× 38.4	= 156.6 cub.ft.	@ 150#	= 23,500	× 17.1	= 402,000 #'
Shaft.	3.8 × 16.5	× 38.6	= 2420.0	@ "	= 363,000	7.67	= 2,783,000
					386,500 #		3,185,000 #'

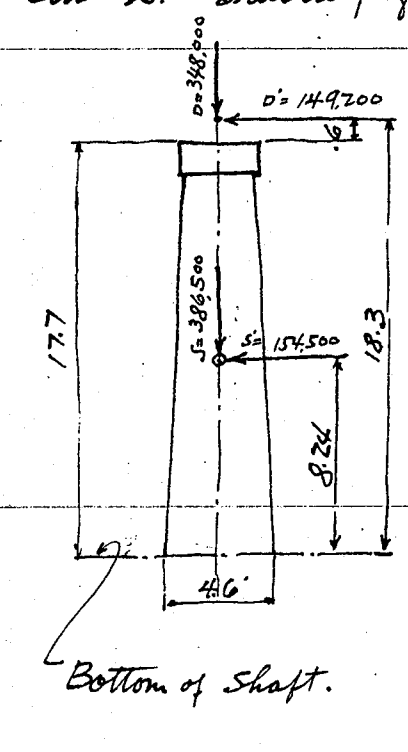
Center of gravity $\frac{3,185,000}{386,500} = 8.24'$ above bottom

Compression on concrete 578,000 + 386500 = 964,500 #

unit compression $f_c = \frac{964,500\#}{4.6 \times 39.} = 5,375\#/' = 37.4\#/"$ Compression ok.

$f_s = 37.4 \times 15 = 560\#/'$ ok.

Case 2. Stability of Shaft during Earthquake: K = 0.4.



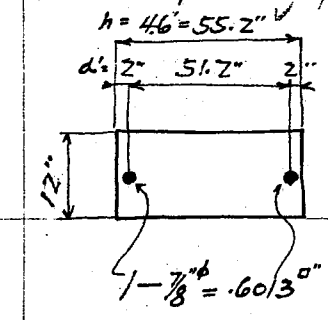
Loads.

		K.	Seismic force	lev. arm	moment.
Superimposed dead load	348000#	× 0.4 =	139,200	× 18.3 =	2,523,000 #'
Weight of Shaft	386,500	× 0.4 =	154,500	× 8.24 =	1,273,000
	ΣV = 734,500		ΣH = 293,700	ΣM =	3,796,000 #'

Mean length of shaft at bottom = 39'

Average stresses per ft strip ΣV = 18,800 # ΣH = 7580# ΣM = 197,300 #'

Eccentricity $\varepsilon = \frac{197,300}{18,800} = 5.18'$



Assumed section is as shown in the sketch.

Steel ratio $p = p' = \frac{.6013}{12 \times 55.2} = .000908$

$P_0 = 2p = 2 \times .000908 = 0.00182$

$\varepsilon/h = \frac{5.18}{4.6} = 1.13$, $\frac{d'}{h} = \frac{2}{55.2} = 0.0364$

From the prepared diagrams, we have,

K = .21, L = .073, j = .878

Compression in Concrete $f_c = \frac{M}{Lbh^2} = \frac{197,300 \times 12}{.073 \times 12 \times 55.2^2} = 461\#/"$ ok.

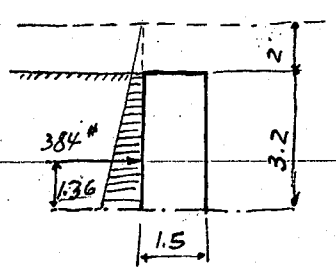
Tension in steel $f_s = nf_c\left(\frac{d}{kh} - 1\right) = 15 \times 461\left(\frac{53.2}{.21 \times 55.2} - 1\right) = 23,820\#/'$ ok

Shear $v = \frac{7580}{12 \times 7/8 \times 53.2} = 13.5\#/'$

bond $u = \frac{7500}{2.749 \times 7/8 \times 53.2} = 58.8\#/'$ ok.

Design of Abutment.
Parapet wall.
Case 1. Under full load or normal state.

2 ft Surcharge due to live load assumed.



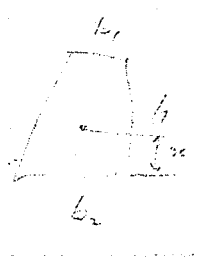
Earth pressure $e_1 = \frac{100 \times 2}{3} = 67\#/'$, $e_2 = \frac{100 \times 5.2}{3} = 173\#/'$

Total earth pressure on parapet wall = $\frac{67+173}{2} \times 3.2 = 384$ # per ft strip of wall.

Moment on wall = 384 × 1.36 = 522 '#

Steel area req'd. for moment = $\frac{522 \times 12}{17067 \times \frac{7}{8} \times 17} = .020"$ per ft strip.

use 1—1/2"ϕ = 0.196" per ft. strip.



$x = \frac{h}{3}\left(\frac{2b_1 + b_2}{b_1 + b_2}\right)$

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CALCULATIONS FOR

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108A Bridge for City of Tokyo.

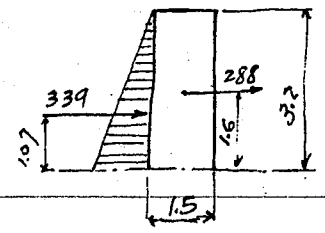
Design of Abutment continued

Parapet wall. Case 2. Stability during Earthquake. $K = 0.4$. Coeff. $\frac{3}{4}K = \frac{3}{4} \cdot 0.4 = 0.3$ ✓

Earthpressure for $K=0.3$ $= 0.662 \cdot \frac{wh^2}{2} = .331\,wh^2$ ✓

Earth pressure $= 0.331 \times 100 \times 3.2^2 = 339^{\#}$ ✓

wt. of wall $= 1.5 \times 3.2 \times 150 = 720^{\#} \times .4 = 288^{\#}$ ✓



Moment on wall = $288 \times 1.6 = 461$ ✓

$339 \times 1.07 = 363$ ✓

$824^{1\#}$ ✓

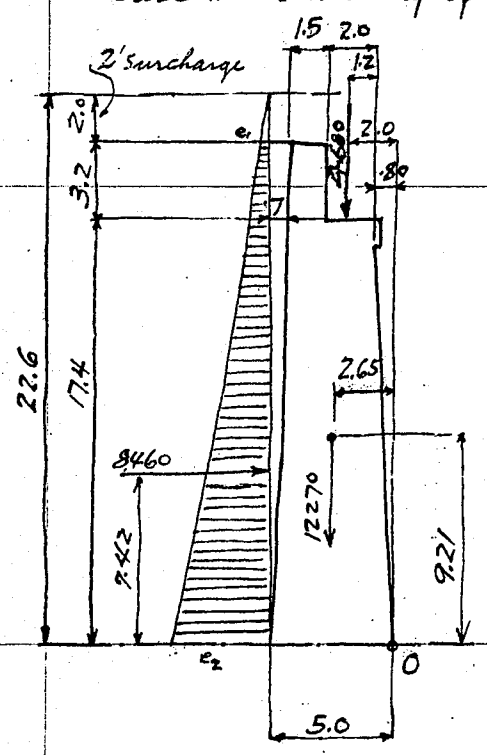
Steel area req'd. for moment $= \frac{824 \times 12}{17{,}067 \times 1.8 \times \frac{7}{8} \times 17} = 0.022^{\square"}$ per ft. strip of wall. ✓

Assumed section is ample.

$v = \frac{627^{\#}}{12 \times \frac{7}{8} \times 17} = 3.5^{\#/\square"}$ ✓ ok.

Shaft :—

Case 1. Stability of Shaft at normal state in full load.



Center of gravity of shaft. (All figures, for one-half of abutment)

Names	Volumes	unit wt.	Weight	Vert. lev. arm	moment	hor. lev. arm	moment abt. O
Parapet wall	$3.1 \times 1.5 \times 9.9$	@ 150 =	6900 ✓	× 18.95 =	130,800 ✓	3.55	24,500 ✓
"	$3.6 \times 2.5 \times 11.25$	@ " =	15,180 ✓	× 19.20 =	291,500	4.05	61,500 ✓
Column under lamp	$3.0 \times 0.5 \times 7.5$	@ " =	1,690 ✓	× 16.85 =	28,500 ✓	5.55	9,400 ✓
Coping	$1.2 \times 0.2 \times 23.0$	@ " =	830 ✓	× 16.8 =	13,900 ✓	0.9	750 ✓
Shaft, wall	$17.4 \times 4.25 \times 21.0$	@ " =	233,000 ✓ / 257,600 ✓	× 8.18 =	1,906,000 ✓ / 2,370,700 $^{1\#}$ ✓	2.52	587,000 ✓ / 683,150 $^{1\#}$ ✓

Center of gravity of Shaft. Vertical lev. arm $= \frac{2{,}370{,}700}{257{,}600} = 9.21$ ✓ from O.

Horizontal " " $= \frac{683{,}150}{257{,}600} = 2.65$ ✓ from O.

Average wt. of shaft $= \frac{257{,}600}{21.} = 12{,}270^{\#}$ ✓ per lin. ft. of wall.

Super imposed load. (Dead + live). $= \frac{190{,}000}{578{,}000}^{\#}$ ✓

average load $= \frac{190000}{40.6} = 4{,}680^{\#}$ per lin. ft. of wall.

Earth pressure $e_1 = \frac{2 \times 100}{3} = 67^{\#/\square'}$, $e_2 = \frac{22.6 \times 100}{3} = 754^{\#/\square'}$ ✓

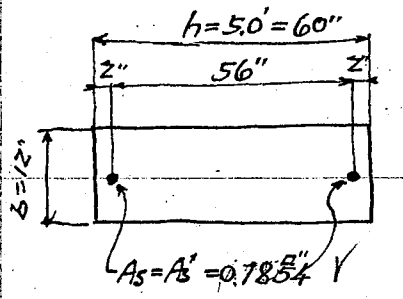
Total earth pressure on wall $= \frac{67 + 754}{2} \times 20.6 = 8460^{\#}$ ✓ per lin ft. of wall.

Overturning moment (about O).		lever arm		moment
Super imp. load	$14{,}680^{\#}$	− 2.0	=	$-29{,}360^{'\#}$ ✓
Wt. of shaft	12,270	− 2.65	=	−32,500 ✓
Earth pressure	8460 ✓	7.42	=	62,800 ✓
$\Sigma V = 16{,}950^{\#}$ ✓	$\Sigma H = 8{,}460^{\#}$ ✓			$+20{,}940^{'\#}$ ✓

Distance of resultant force from point O $= \frac{20{,}940}{16{,}950} = 1.24$ ✓

Eccentricity $= 2.5 + 1.24 = 3.74'$ ✓

moment at bottom section $= 16950 \times 3.74 = 63{,}400^{'\#}$ ✓



Reinforcements

1"ϕ bars 14" spacing 42 bars

~~2 1"ϕ bars under each girder 14.~~

~~Total 56 bars~~

~~Total steel area = 56 @ .7854 = 44.0"~~

~~Steel area per ft. strip of wall~~

~~$= \frac{44"}{42'} = 1.05"$~~

~~Perimeter $= 3.142 \times 1.05 = 3.3"$ / ft strip~~

Steel ratio $P_c = 2P = \frac{.7854 \times 2}{12 \times 60} = 0.00218$ ✓

$\frac{c}{h} = \frac{3.74}{5.00} = 0.748$ ✓, $\frac{d'}{h} = \frac{2}{60} = 0.033$ ✓

From the prepared diagrams

$K = .267$, $L = .094$ ✓

$f_c = \frac{634{,}00 \times 12}{0.074 \times 12 \times 60^2} = 237^{\#/\square"}$ ✓ ok

$f_s = 15 \times 237 \left(\frac{58}{267 \times 60} - 1\right) = 9300^{\#/\square"}$ ✓ ok

Shear $v = \frac{8460}{12 \times \frac{7}{8} \times 58} = 13.9^{\#/\square"}$ ✓ ok

~~Bond~~ $u = \frac{8460}{3.142 \times \frac{7}{8} \times 58} = 48.^{\#/\square"}$ ✓ ok.

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CALCULATIONS FOR

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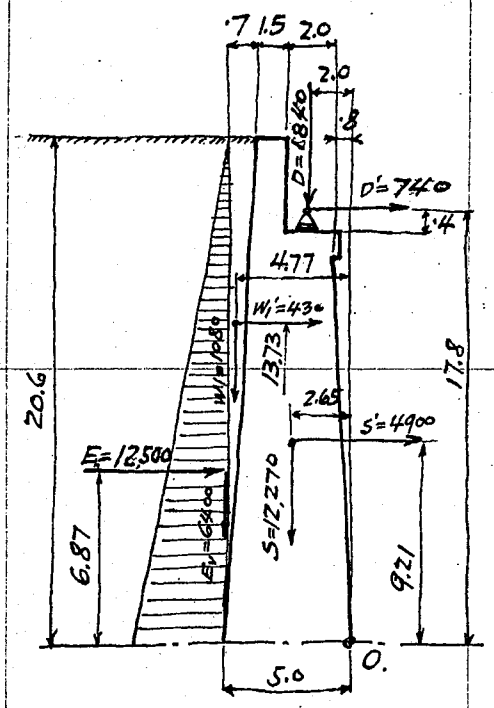
108 A Bridge for City of Tokyo.

Case 2. Stability of abeutment Shaft during Earthquake $K = 0.4$ $\frac{3}{4}K = 0.3$.



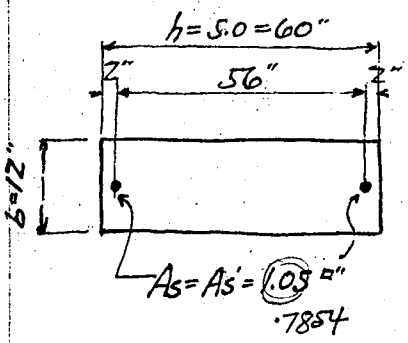
Super imposed Dead Load = 7+1,000 #
average 71,000 ÷ 42 = D = 1,840 # per lin. ft. of wall, Seismic force D' = 1,840 × .4 = 740 #/ft.
Weight of Shaft = 12,270 # " " " S' = 12,270 × .4 = 4,900 "
Weight of earth behind = $W_1 = \frac{.7 \times 20.6}{2}$ @ 150# = 1,080 # " W_1' = 1080 × .4 = 430 "
Earth pressure = 0.331 × 100 × 20.6² = 14,050 #
vertical component Ev = .151 × 100 × 20.6² = 6,400
horizontal " Eh = .295 × " " " = 12,500

Taking moment about O.

Loads		Vert. forces	hor. forces	lever arm	moment
Super imp. D.L.	D	1,840#		- 2.00	- 3,680
"	D'		740	17.80	+ 13,170
Shaft	S	12,270		- 2.65	- 32,500
"	S'		4,900	9.21	45,100
Earth behind	W_1	1,080		- 4.77	- 5,150
"	W_1'		430	13.73	5,910
Earth pressure	Ev	6,400		- 5.00	- 32,000
"	Eh		12500	6.87	85,800
		ΣV = 21,590	ΣH = 18,570	ΣM =	176,550 '#

Distance of resultant force from O = $\frac{176,550}{21,590} = 8.55'$

Eccentricity $\varepsilon = 3.55 + 2.5 = 6.05$
Bending moment at the section = 21,590 × 6.05 = 130,500 '#
Assumed Section is as shown in the sketch below.



Steel ratio $P_0 = ZP = \frac{.7854 \times 2}{12 \times 60} = 0.00218$

$\varepsilon/h = \frac{6.05}{5.00} = 1.21$, $d'/h = \frac{2}{60} = .033$

From the prepared diagrams.
$K = .223$, $L = .070$, $j = .92$

$f_c = \frac{130,500 \times 12}{.070 \times 12 \times 60^2} = 518$ #/□" OK.

$f_s = 15 \times 518 \left(\frac{58}{1223 \times 60} - 1\right) = 25,900$ #/□" OK (allowable unit tension 17067 × 1.8 = 30,700 #/□")

$v = \frac{18,570}{12 \times .92 \times 58} = 29$ #/□" OK

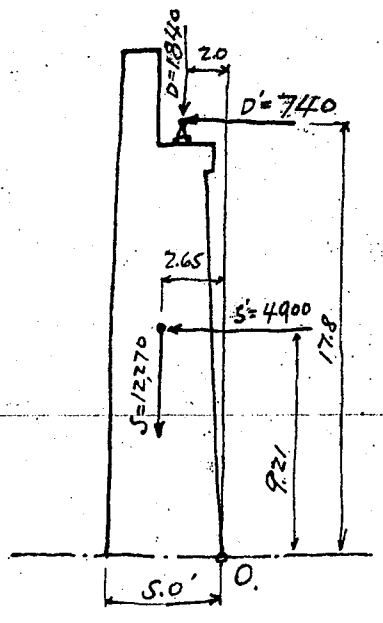
$u = \frac{18,570}{3.14 \times .92 \times 58} = 111$ #/□" OK. (allowable unit bond stres = 85 × 1.8 = 153 #/□")

Reinforcement.
1" □ bars 1'-0" spacing 42
~~2-1" □ bars under each girder = 44~~
~~56~~ bars.

~~Total Steel area = 56 @ .7854 = 44.0 □"~~
~~Steel area per ft. strip of wall~~
$= \frac{44.0}{42'} = 1.05$ □"

~~Perimeter = 3.142 × 1.05 = 3.3"~~/ft.

Case 3. Seismic forces acting in reversed direction to case 2.



Taking moment at O.

Loads		Vert. forces	hor. forces	lever arm	moment.
Super imp. D.L.	D	1840		- 2.0	- 3,680
"	D'		740	17.8	13,170
Shaft	S	12,270		- 2.65	- 32,500
"	S'		4900	9.21	45,000
		ΣV = 14,110 #	ΣH = 5640	ΣM =	27,990 '#

Distance of resultant force from O. = $\frac{24990}{14,110} = 1.56'$

Eccentricity $\varepsilon = 2.5 - 1.56 = -0.84'$

Bending moment at the bottom section = 14,110 × 0.84 = 11,860 '#

In this case moment and direction load are smaller than for Case 2. Assumed section is ample.

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CALCULATIONS FOR

108 A Bridge for City of Tokyo.

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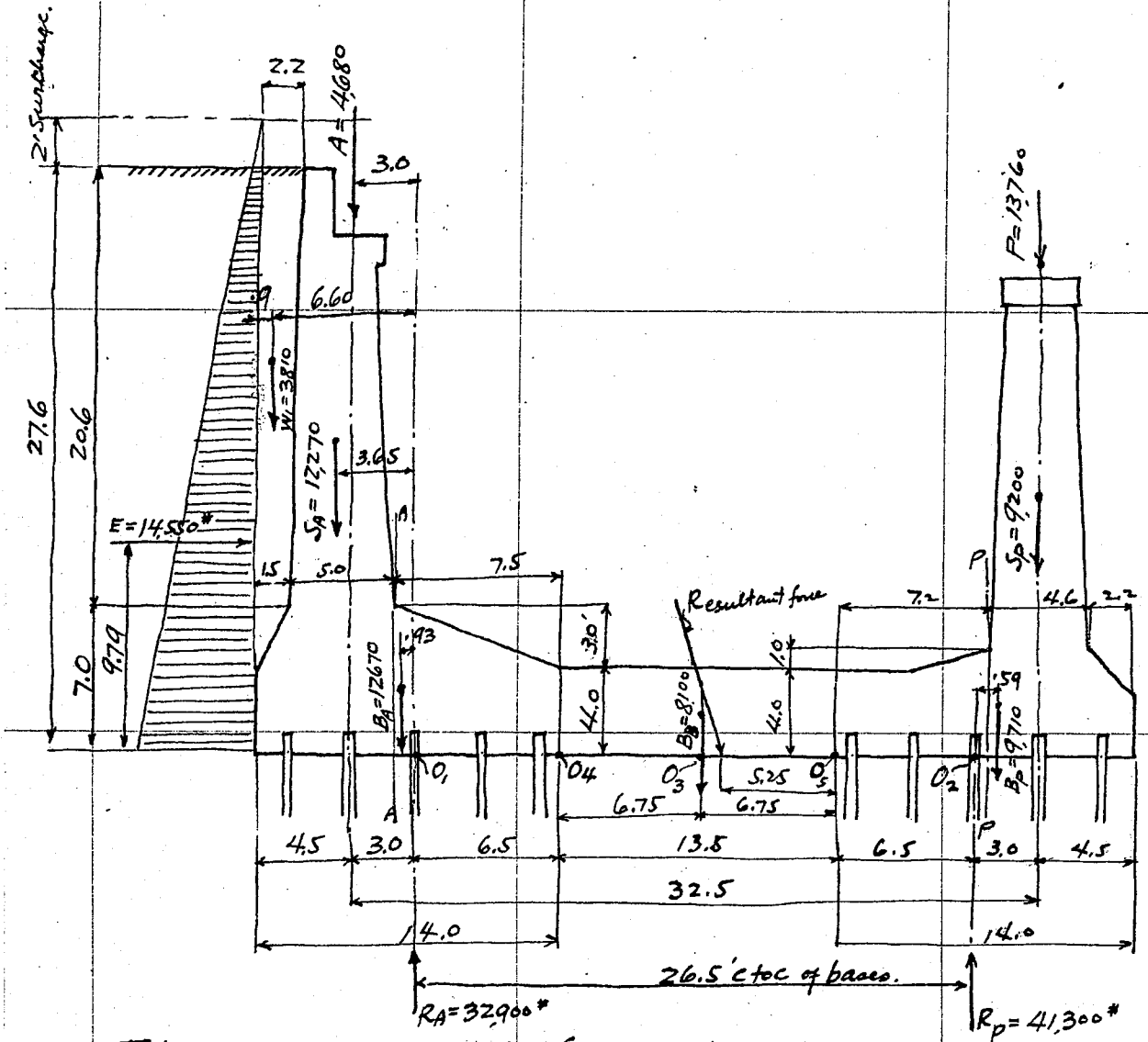
Design of Combined footing for Pier and Abutment.
Case 1. Stability at normal State (in full load).



Loads on pier per lin ft. strip.
Superimposed load.
D.L. 348,000 ÷ 42 = 8,300 #/ft strip
L.L. 229,320 ÷ 42 = 5,460 "
total = 13,760 "

Weight of Pier Shaft per ft Strip
386,500 ÷ 42 = 9,200 #/ft. strip

Weights of Pier base
2.2 × 4.25 @ 150 = 1,400 × 1.19 = 1,670 #
4.6 × 5.0 @ " = 3,450 × 4.5 = 15,530
7.2 × 4.5 @ " = 4,860 × 10.27 = 49,850
9,710 # 67,050 #

Hor. dist. of C.g. from the end
= 67050 / 9710 = 6.91 '

Vert. dist. of C.g. from bottom.
1400 × 1.95 = 2,730
3450 × 2.50 = 8620
4,860 × 2.25 = 10930
22,280 + 9710 = 2.29

Wt. of abutment base	Hor arm	m	vert arm	m
1.5 × 5.5 @ 150 = 1,240	× .82	1,010	2.8	3,470
5.0 × 7.0 @ " = 5,250	× 4.0	21,000	3.5	18,380
7.5 × 5.5 @ " = 6,180	× 9.91	61,200	2.8	17,300
12,670		83,210		39,150

C.g. 83210 / 12670 = 6.57' from the end
39150 / 12670 = 3.09' from bottom.

Wt. of Beam 13.5 × 4 @ 150 = 8,100 #

Wt. of Earth behind of abutment Shaft
1.85 × 20.6 @ 100 = 3,810 #

Earth pressure on abutment
e1 = 100 × 2 / 3 = 67 #/', e2 = 100 × 29.6 / 3 = 987 #/'
E = (67 + 987) / 2 × 27.6 = 14,550 #
x = 27.6 / 3 × (134 + 987) / (67 + 987) = 9.79'

Taking moment about O1 (Center of base of abutment).

Loads	Hor. forces		Vert. forces		lever arm		Moment.
A			4,680	×	−3.00	=	−14,050
S_A			12,270	×	−3.65		−44,800
W_1			3,810	×	−6.60		−25,100
B_A			12,670	×	−0.93		−11,800
E	14,550			×	−9.79		+142,400
	ΣH = 14,550		ΣV = 33,430 #				ΣM = +46,650
P			13,760	×	+29.5	=	+406,000
Sp			9,200	×	+29.5		+271,500
B_P			9,710	×	+27.09		+263,000
	ΣH = 0		ΣV = 32,670				ΣM = +940,500
B_B			8,100	×	+13.25	=	+107,300
	ΣH = 0		ΣV = 8,100				ΣM = +107,300
Total	ΣH = 14,550 #		ΣV = 74,200 #				ΣM = +1,094,450

Reaction on Pier foundation $R_p = \frac{1,094,450}{26.5} = 41,300$ #

Load on one pile = 41,300 × 3 / 5 = 24,800 # = 11.06 tons.

Reaction on abutment foundation R_A = 74,200 − 41,300 = 32,900 #

Load on one pile = 32,900 × 3 / 5 = 19,800 # = 8.84 tons.

Point of application of Resultant force = 1,094,450 / 74,200 = 14.75' from O1 Right

Surface frictional load on pile.
Use Pile 0.8' dia at tip 36' long
mean diam say 0.95'
Surface area of pile
= 3.142 × .95 × 35 = 104.5 □'

Surface friction
Piles for pier = 11.06 × 2240 / 104.5 = 237 #/□'
Piles for abutment 8.84 × 2240 / 104.5 = 190 #/□'
OK.

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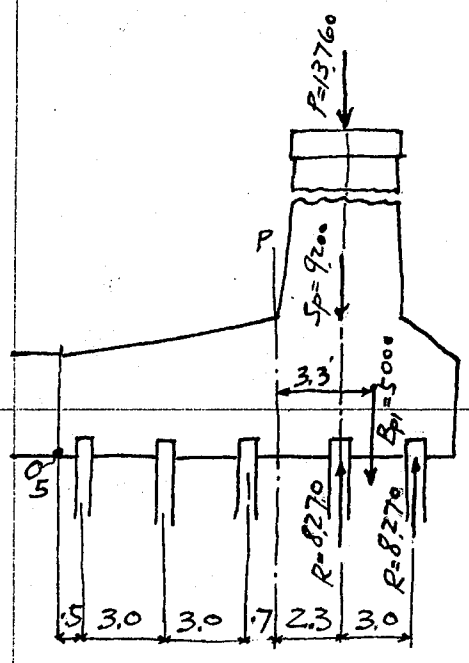
CALCULATIONS FOR

7

108 A Bridge for City of Tokyo.

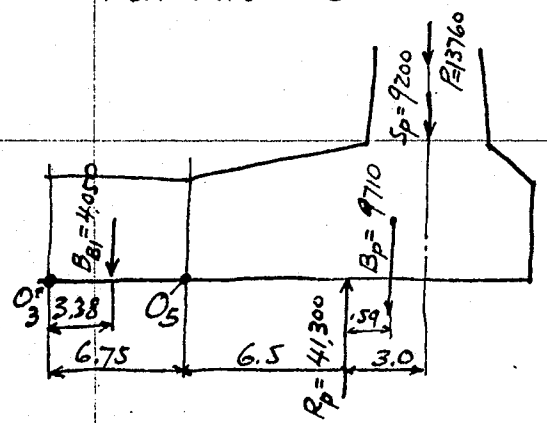
Moment and Shear at Section P-P (Effect of horizontal reaction due to frictional resistance on bed neglected.



Loads		lever arm	Moment.
P =	13,760	× −2.3	= − 31,650 ✓
Sp =	9,200	× −2.3	= − 21 150 ✓
Bp, =	5,000	× −3.3	= − 16 500 ✓
R =	− 8,270	× −2.3	= + 19,030 ✓
R =	− 8,270	× −5.3	= + 43850 ✓
ΣV =	+ 11,420 # ✓		ΣM = − 6,420 '# ✓

Pile reaction
$\frac{24800}{3}$ = 8270# per ft. strip ✓

Moment and Shear at O_5 (Toe of Pier base).



Loads		lever arm	moments
P =	13,760 ×	−9.5	= − 130,700 '# ✓
Sp =	9200 ×	−9.5	= − 87,400 ✓
Bp =	9,710 ×	−7.09	= − 68,800 ✓
Rp =	−41,300 ×	−6.50	= +268,500 ✓
ΣV =	−8,630 # ✓		ΣM = − 18,400 '# ✓

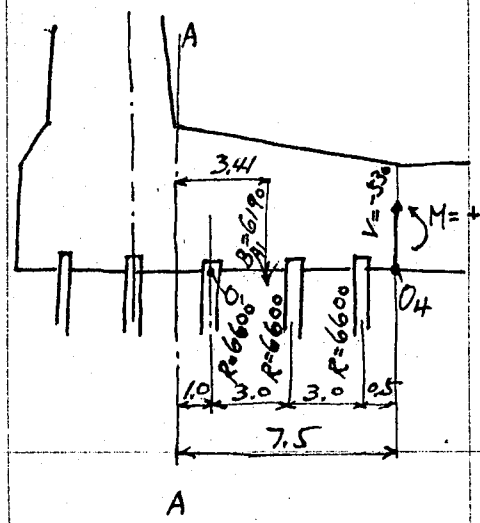
Moment and Shear at O_3 (Center of Beam)
See figure above.

Loads		lever arm	moment.
P =	13,760 ×	−16.25	= −223,500 ✓
Sp =	9200 ×	−16.25	= −149,500 ✓
Bp =	9710 ×	−13.84	= −134,300 ✓
BBI =	4050 ×	−3.38	= − 13,700 ✓
Rp =	−41,300 ×	−13.25	= +547,500 ✓
ΣV =	−4,580 ✓		ΣM = + 26,500 '# ✓

Moment & Shear at O_4 (Toe of abutment base)

M at O_3				+ 26,500 ✓
Shear at O_3	−4,580	× −6.75	=	+ 30,900 ✓
BBI	+4,050	× −3.38	=	− 13,700 ✓
ΣV =	− 530 # ✓		ΣM =	+ 43,700 '# ✓

Moment at section A-A (fixed point of base to abutment)



Loads				
M at O_4				+43,700 ✓
V at "	− 530	× −7.5		+ 3,970 ✓
B_{A1}	6,190	× −3.41		− 21,100 ✓
R	− 6,600	× −1.0		+ 6,600 ✓
R	− 6,600	× −4.0		+ 26,400 ✓
R	− 6,600	× −7.0		+ 46,200 ✓
ΣV =	−14,140 # ✓		ΣM =	+105,770 '# ✓

Pile reaction
$\frac{19800}{3}$ = 6600# per ft. strip ✓

Wt. of footing
7.5 × 5.5 @ 150 = 6,190# = B_{A1} ✓

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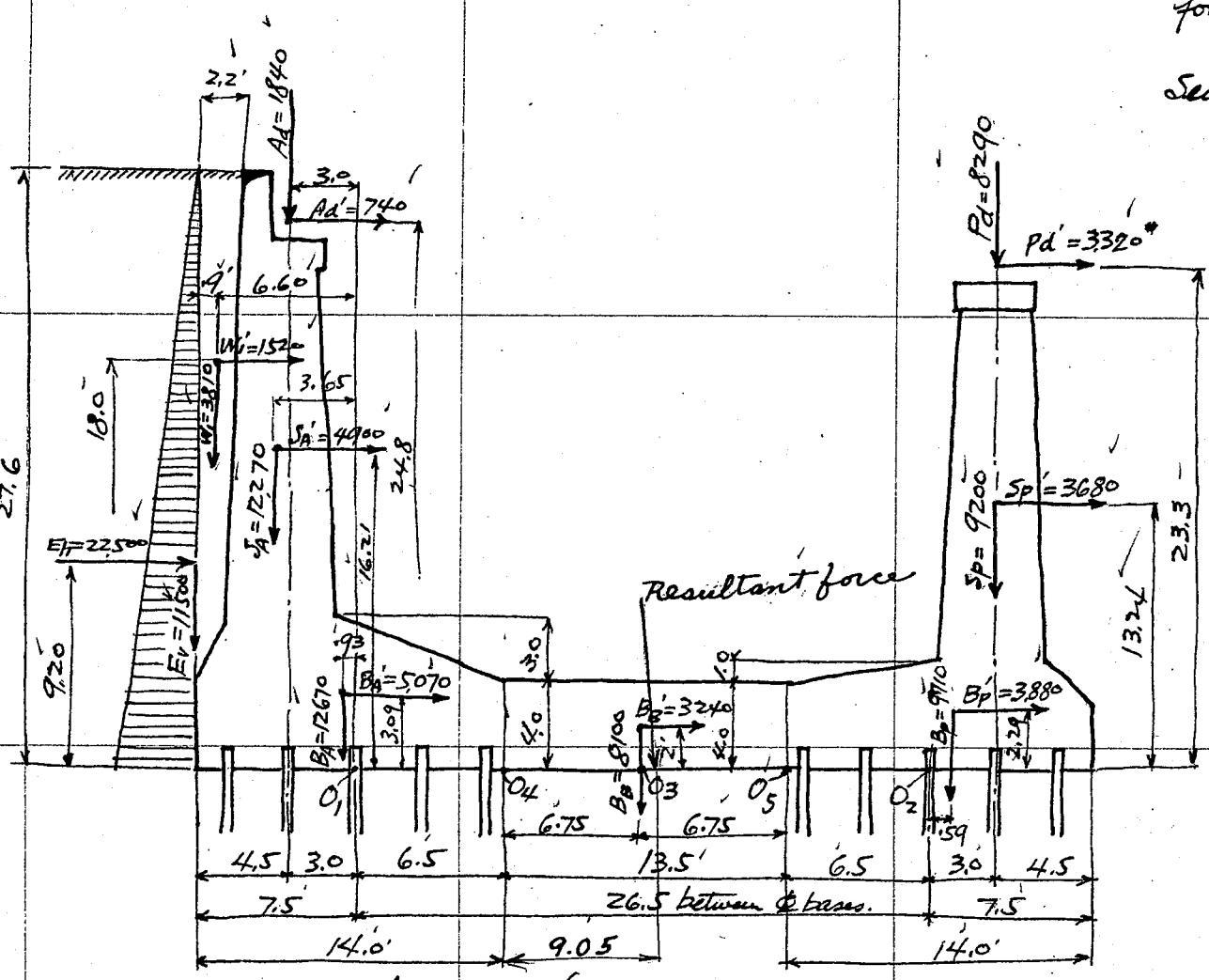
CALCULATIONS FOR

Case 2. Stability of Combined footing during Earthquake.
(Seismic forces acting forward (pier))

Coeff. $\frac{3}{4}K = 0.30$

Super imposed D.L. on pier = 348,000#
for one ft. strip $\frac{348,000}{42} = 8,290$#

Seismic force $Pd' = 8290 \times .4 = 3320$# per ft strip
" $Sp' = 9200 \times .4 = 3,680$ "
" $Bp' = 9710 \times .4 = 3,880$ "
" $Ad' = 1840 \times .4 = 740$ "
" $S_A' = 12270 \times .4 = 4,900$ "
" $W_1' = 3,810 \times .4 = 1520$ "
" $B_A' = 1267 \times .4 = 5,070$ "
" $B_B' = 8,100 \times .4 = 3,240$ "



Earth pressure during earthquake.
$0.331 \times 100 \times 27.6^2 = 25,200$#
Vert. Component $E_V = .151 \times 100 \times 27.6^2 = 11,500$#
Hor. " $E_H = .1295 \times 100 \times 27.6^2 = 22,500$

Taking moment about O_1 (Center of pile foundation of abutment).

Loads	Hor. forces	Vert. forces	Lever arm	Moment
Ad =		1,840	−3.0	−5,520
Ad' =	740		+24.8	+18,350
S_A =		12270	−3.65	−44790
S_A' =	4900		+16.21	+79430
B_A =		12,670	−0.93	−11,780
B_A' =	5,070		+3.09	+15,670
W_1 =		3,840	−6.60	−25340
W_1' =	1,520		+18.0	+27,360
E_V =		11,500	−7.50	−86,250
E_H =	22500		+9.20	+207000
	ΣH=34,730	ΣV=42,120		ΣM=+174,130
Pd =		8290	+29.5	+244,560
Pd' =	3,320		+23.3	+77,360
Sp =		9200	+29.5	+271,400
Sp' =	3,680		+13.24	+48,720
B_P =		9710	+27.09	+263,000
B_P' =	3880		+2.29	+8,880
	ΣH=10,880	ΣV=27,200		ΣM=+913,920
B_B =		8,100	+13.25	+107,300
B_B' =	3240		+2.00	+6,480
	ΣH=3,240	ΣV=8,100		ΣM=+113,780
Total	ΣH=48,850	ΣV=77,420		ΣM=+1,201,830

Reaction on pier foundation
$R_P = \frac{+1201830}{26.5} = 45,400$# per lin ft strip
Load on one pile $= \frac{45400 \times 3}{5} = 27,200$# $= 12.0$ tons

Reaction on abutment foundation
$R_A = 77,420 - 45400 = 32,020$# per lin. ft. strip
Load on one pile $= \frac{32020 \times 3}{5} = 19,210$# $= 8.5$ tons

Position of Resultant force
Hor. distance from $O_1 = \frac{1,201,830}{77420} = 15.55$

Horizontal reaction due to frictional distance on foundation bed
$= \frac{48850}{41.5} = 1177$ #/□' of bed area

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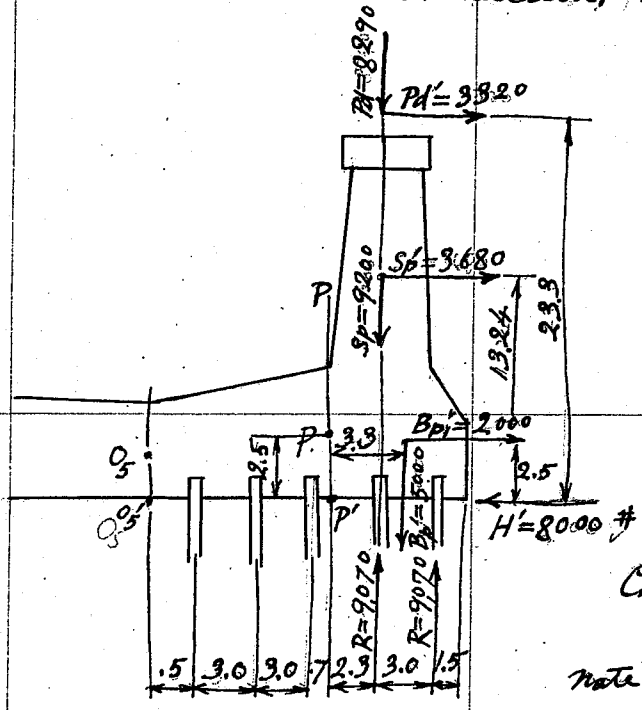
CALCULATIONS FOR

Moment & Shear on section P-P (fixed point of base to pier)



Taking moment about p'

Loads	hor. forces	vert forces	lever arm	moment
Pd		8290	- 2.3	- 19,070 ✓
Pd'	3320		- 23.3	- 77,400 ✓
Sp		9200	- 2.3	- 21,150 ✓
Sp'	3680		- 13.24	- 48,700 ✓
Bp1		5000	- 3.3	- 16,500 ✓
Bp1'	2000		- 2.5	- 5,000 ✓
R		- 9,070	- 2.3	+ 20,850 ✓
R		- 9,070	- 5.3	+ 48,100 ✓
H'	- 8,000		0	- 0

ΣH = + 1,000 # ΣV = +~~4,350~~ 4,350 ΣM = - 118,870 #

Changing center of moment from P' + P ---- 1,000 × 2.5 = + 2,500 ✓

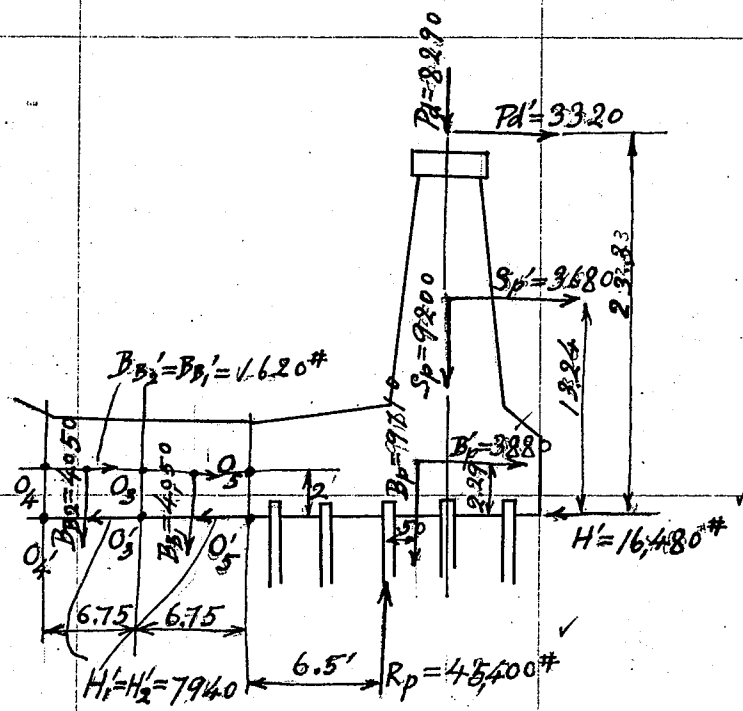
Moment at P+P = - 116,370 # ✓

note :- frictional resistance of foundation bed
= 1,177 × 6.8 = 8,000 # ✓

Pile reaction = 27,200 ÷ 3 = 9,070 # ✓

Moment & Shear at section O5

Taking moment about O5



Pd		8290	- 9.5	- 78800 ✓
Pd'	3,320		- 23.3	- 77,400 ✓
Sp		9200	- 9.5	- 87,400 ✓
Sp'	3680		- 13.24	- 48,700 ✓
Bp		9710	- 7.09	- 68,800 ✓
Bp'	3880		- 2.29	- 8,900 ✓
Rp		- 45400	- 6.50	+ 295000 ✓
H'	- 16480 ✓		0	0

ΣH = - 5,600 # ΣV = - 18,200 # ✓ ΣM05 = - 75,000 '# ✓

- 5600 × 2.0 = - 11,200 ✓

ΣM05 = - 86,200 '# ✓

Moment & Shear at O3

moment at O5				- 86,200 '# ✓
Shear " "		- 18,200	- 6.75	+ 123,000 ✓
BB1		+ 4,050	- 3.38	- 13,800 ✓
BB1'	1,620		0	0
H1'	- 7,940		2.00	- 15,880 ✓
ΣH for O5	- 5,600 ✓			

ΣH = - 11,920 ΣV = + 14,150 ✓ ΣM03 = + 7,120 '# ✓

Moment & Shear at O4

Moment at O3				± 7,120 ✓
Shear " "		- 14,150	- 6.75	+ 95,500 ✓
		+ 4,050	- 3.38	- 13,800 ✓
BB2				
BB2'	1620		0	0
H2'	= 7940		2.0	- 15,880 ✓
ΣH for O3 = - 11920				

ΣH = - 18,240 ✓ ΣV = - 10,100 ✓ ΣM04 = + 72,940 '# ✓

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CALCULATIONS FOR

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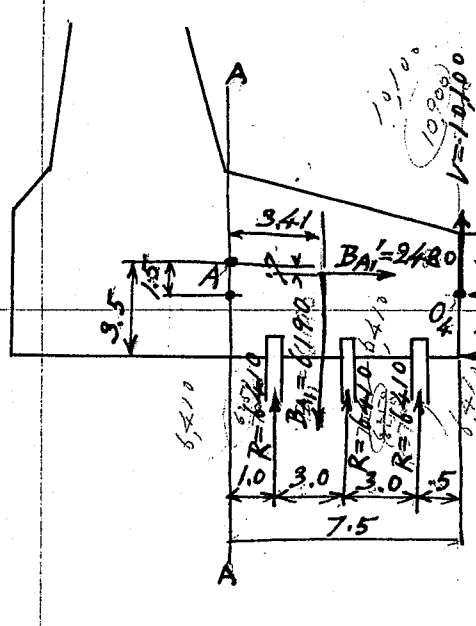
Moment & Shear at Section A–A (fixed point of base to abutment)

Taking moment about A.

Loads	hor. forces	vert. forces	lever arm	moment
Moment at O_4				+ 72,940
Shear " "		−40,100	− 7.50	+ 75,800
Thrust " H	−18240		+ 1.50	− 27,360
friction H_3'	−8820		+ 3.50	− 30,850
B_{A1}		+6190	− 3.41	− 21,100
B_{A1}'	+2480		+ 0.70	+ 1,740
R		−6410	− 1.0	+ 6,410
R		−6410	− 4.0	+ 25,600
R		−6410	− 7.0	+ 44,800
	$\Sigma H = -24,580^{\#}$	$\Sigma V = -23,140^{\#}$		$\Sigma M_A = +147,990^{\prime\#}$



Case 3. Stability of Combined footing during Earth quake.

(Seismic forces acting toward abutment)

Reffer to the sketch for Case 2.

Taking moments about O_1.

Loads	Hor. forces	vert. forces	lever arm	moments
Ad		1,840	− 3.0	− 5,520
Ad'	740		−24.8	= 18,350
S_A		12,270	−3.65	− 44,790
S_A'	4900		−16.21	− 79,430
B_A		12670	− 0.93	− 11,780
B_A'	5070		− 3.09	− 15,670
W,		3840	− 6.60	− 25340
	$\Sigma H = 10710^{\#}$	$\Sigma V = 30,620^{\#}$		$\Sigma M = -200,880$
P_d		8,290	+ 29.5	+ 244,560
P_d'	3,320		− 23.3	− 77,360
S_p		9,200	+ 29.5	+ 271,400
S_p'	3,680		− 13.24	− 48,700
B_p		9,710	+ 27.09	+ 263,000
B_p'	3,880		− 2.29	− 8,880
	$\Sigma H = 10,880$	$\Sigma V = 27,200$		$\Sigma M = +644,020$
B_B		8,100	+13.25	+ 107,300
B_B'	3240		− 2.00	− 6,480
	$\Sigma H = 3,240$	$\Sigma V = 8100$		$\Sigma M = +100,820$
Total	$\Sigma H = 24,830^{\#}$	$\Sigma V = 65,920^{\#}$		$\Sigma M = +543,960^{\prime\#}$

Reaction on Pier foundation

$R_p = \frac{543960}{26.5} = 20,500^{\#}$ per lin. ft. strip

Load on one pile $= \frac{20500 \times 3}{5} = 12,300^{\#} = 5.49$ tons

Reaction on abutment foundation

$R_A = 65920 - 20500 = 45420^{\#}$/lin. ft strip

Load on one pile $= \frac{45420 \times 3}{5} = 27200^{\#} = 12.15$ tons

Position of resultant force from O_1

$= \frac{543960}{65920} = 8.2'$

Hor. reaction due to frictional resistance on foundation bed

$= \frac{24830}{41.5} = 600^{\#}/\square'$ of bed area

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CALCULATIONS FOR

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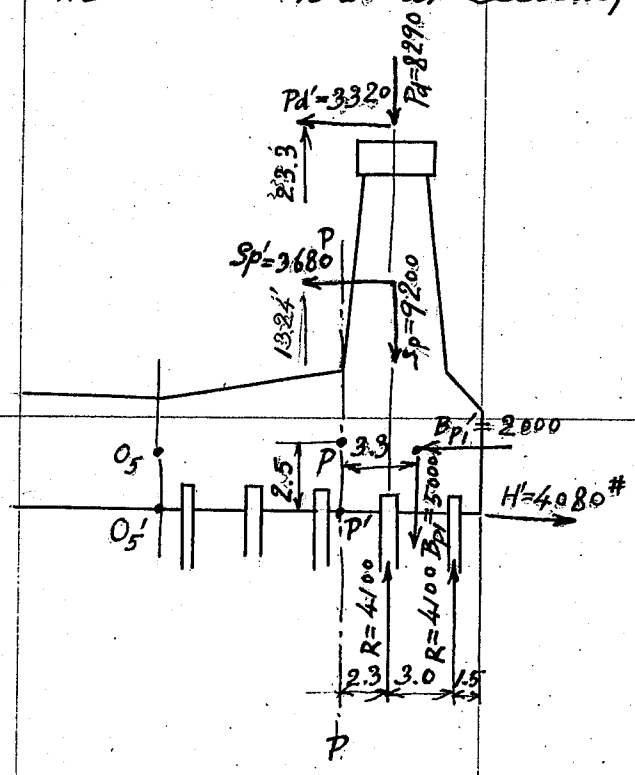
Moment & Shear at section P–P (fixed point of base to pier)

Pile reaction per lin. ft strip = $\frac{12,300}{3} = 4,100^{\#} = R$ ✓

Frictional reaction on bed = $6.8 \times 600 = 4,080^{\#} = H'$ ✓

Taking moment about P'

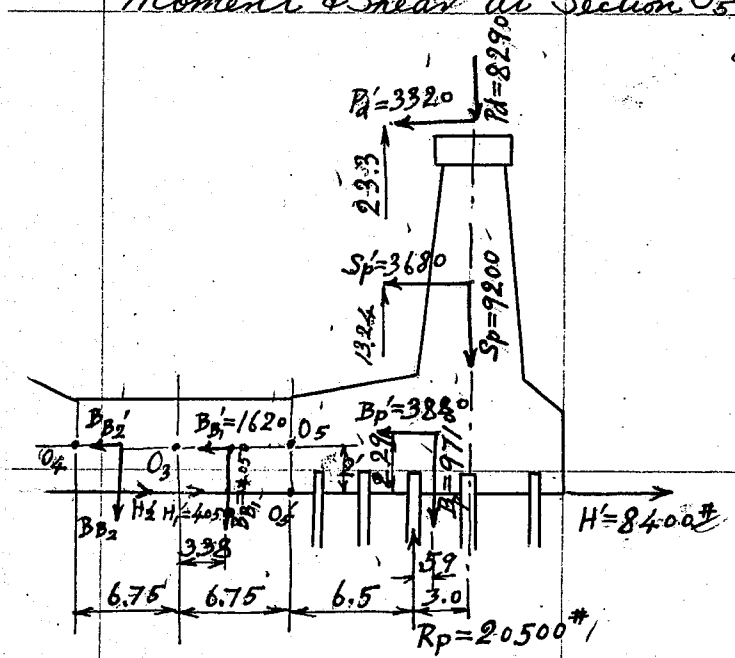


Loads	hor. forces	vert. forces	lever arm		moment	
P_d		8,290	− 2.3	−	19,070	✓
P_d'	− 3320		− 23.3	+	77,400	✓
S_p		9,200	− 2.3	−	21,150	✓
S_p'	− 3 680		− 13.24	+	48,700	✓
B_{p_1}		5,000	− 3.3	+	16,500	✓
B_{p_1}'	− 2,000		− 2.5	+	5,000	✓
R		− 4100	− 2.3	+	9,440	✓
R		− 4100	− 5.3	+	21,700	✓
H'	+ 4,080		0	−	0	
	$\Sigma H = -4,920$ ✓	$\Sigma V = +14,290$ ✓		$\Sigma M_{p'}$ = +	103,850	✓

Changing center of moment from p' to p $-4,920 \times 2.5$ = − 12,300 ✓

ΣM_p = + 91550 '# ✓

Moment & Shear at Section O_5 Taking moment at O_5'



Loads	hor. forces	vert. forces	lever arm		moment	
P_d		8,290	− 9.5	=	78,700	✓
P_d'	−3,320		− 23.3	+	77,400	✓
S_p		9,200	− 9.5	−	87,400	✓
S_p'	−3,680		− 13.24	+	48,700	✓
B_p		9,710	− 7.09	−	68,800	✓
B_p'	− 3,880		− 2.29	+	8,880	✓
R_p		−20,500	− 6.5	+	133,300	✓
H'	+ 8,400		0		0	
	$\Sigma H = -2,480$ ✓	$\Sigma V = +6,700$ ✓		$\Sigma M_{O_5'}$ =	+ 33,380	'# ✓

Changing center of moment from O_5' to O_5 $-2,480 \times 2.0$ = − 4,960 ✓

ΣM_{O_5} = + 28,420 '# ✓

Moment & Shear at O_3 (Center of beam)

Moment at O_5				+ 28,420	✓
Shear " "		+ 6700 ✓	− 6.75 ✓	− 45,200	✓
B_{B_1}		+ 4,050 ✓	− 3.38 ✓	− 13,800	✓
B_{B_1}'	− 1,620		0	0	
H_1'	+ 4,050 ✓		+ 2.0 ✓	+ 8,100	✓
ΣH for O_5 =	− 2,480 ✓				
ΣH =	− 50 # ✓	$\Sigma V = +10,750$ # ✓		ΣM_{O_3} =	− 22,480 '# ✓

Moment & Shear at O_4

Moment at O_3				− 22,480	✓
Shear " "		+ 10,750 ✓	− 6.75 ✓	− 72,600	✓
B_{B_2}		+ 4,050 ✓	− 3.38 ✓	− 13,800	✓
B_{B_2}'	− 1,620 ✓		0	0	
H_2'	+ 4050 ✓		+ 2.0 ✓	+ 8,100	✓
ΣH for O_3	− 50 ✓				
ΣH =	+ 2,380 # ✓	$\Sigma V = +14,800$ # ✓		ΣM_{O_4} =	− 100,780 '# ✓

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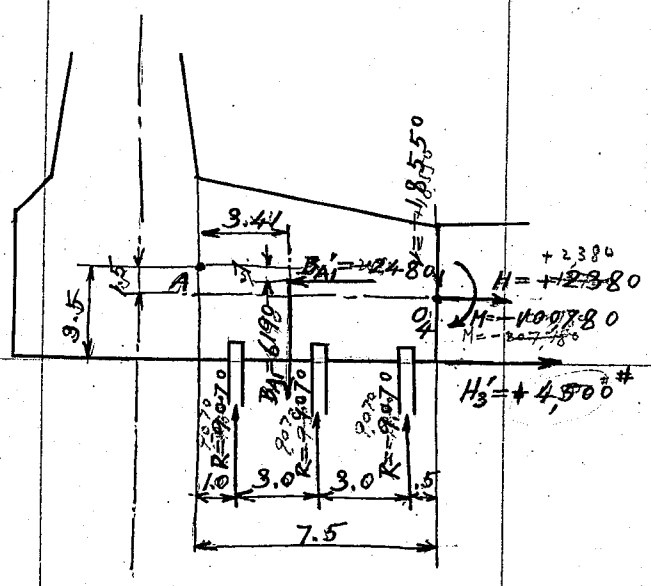
CALCULATIONS FOR

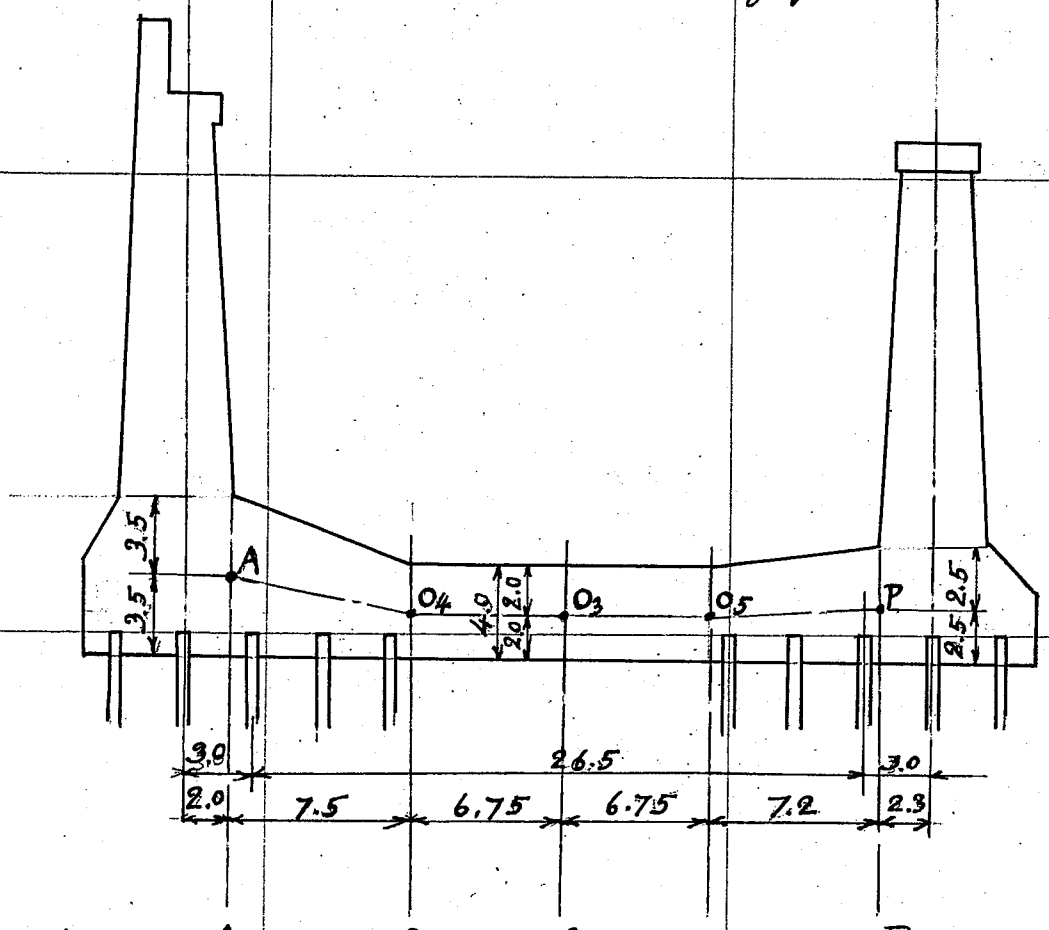
Moment and Shear at Section A-A (fixed point of base to abutment)



Taking moment about A

Loads	hor. forces	vert. forces	leverarm	moment
Moment at O_4				-100,780 ✓
Shear " "		+14,800 ✓	- 7.5	-111,000 ✓
Thrust H	-+2,380 ✓		+ 1.5	+ 3,570 ✓
friction H_3'	+ 4,500 ✓		+ 3.5	+15,750 ✓
B_{A1}		+ 6,190 ✓	- 3.41	-21,100 ✓
B_{A1}'	- 2,480 ✓		+ 0.7	- 1,740 ✓
R		- 9070 ✓	- 1.0	+ 9,070 ✓
R		- 9070 ✓	- 4.0	+36.220 ✓
R		- 9070 ✓	- 7.0	+63,500 ✓
	ΣH = -2,400#	ΣV = -6220# ✓		ΣM_A = -106,510 1# ✓

Table of Max. Moment & Shears at various Sections of foundation base



Bending Moments at		A	O_4	O_3	O_5	P	
Normal state	Case 1	+105,770 1#	+43700 1# ✓	+26500 1# ✓	-18,400 1# ✓	-6420 1# ✓	
Earthquake	Case 2	+147,990 ✓	+72940 ✓	+ 7,120 ✓	-86,200 ✓	-116,370 (÷1.8)	Seismic forces acting toward Pier →
"	Case 3	-106,510 ✓	-100,780 ✓	-22,480 ✓	+28,420 ✓	+91,550 (÷1.8)	" " " " abutment ←

Shears							
Normal State	Case 1	-14,140# ✓	- 530# ✓	- 4,580# ✓	-8,630# ✓	+11420# ✓	
Earthquake	Case 2	-23,140 ✓	-10,100 ✓	-14,150 ✓	-18200 ✓	+ 4,350 ✓ (÷1.8)	→
"	Case 3	- 6,220 ✓	+14,800 ✓	+10,750 ✓	+6700 ✓	+14,290 ✓ (÷1.8)	←

Load on one pile for		Abutment ✓	Pier ✓	
Normal State	Case 1	8.84 tons ✓ (friction 1/20 #/□)	11.06 tons (friction 2/37 #/□)	Piles 0.8′ dia at tip 36.0′ long
Earthquake	Case 2	8.38 ✓	12.35 ✓	
"	Case 3	12.15 ✓	5.49 ✓	

Note:— Figures underlined show stresses which control the section
Full line positive, dotted line negative

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CALCULATIONS FOR

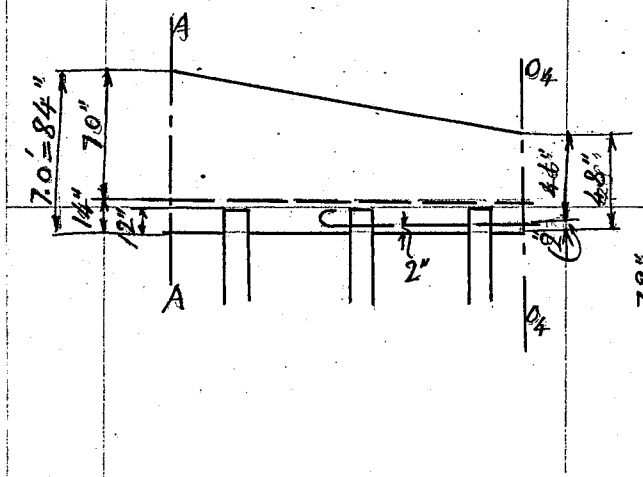
13

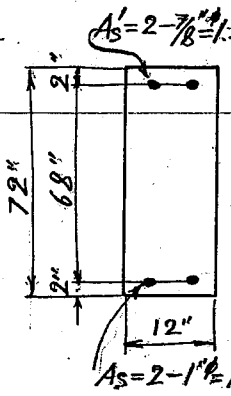
Sections of Footing & Beam.

Section at A–A.

Max. moment { +105,770 #/ft strip at normal state
−106,510 " " during earth quake

Max. Shear −14,140 # " at normal state





Design as a double reinfaced beam

use 2−1"ϕ bars = 1.571 □" on bottom + 2−7/8"ϕ = 1.203 on top

Effective depth = 70" insulation 2" on top & bottom

Steel area req'd for + moment = $\frac{105,770\times12}{17067\times\frac{7}{8}\times70}$ = 1.215 □" on bottom ✓

do. for − moment = $\frac{106,510\times12}{30720\times\frac{7}{8}\times70}$ = 0.679 □" on top ✓

Steel ratio $p=\frac{1.571}{12\times70}=.00187$ ✓ $p'=\frac{1.203}{12\times70}=.00143$ ✓

$p'=\frac{.00143}{.00187}p=0.765p$ ✓

$\frac{d'}{d}=\frac{2}{70}=0.0286$ ✓

$K=0.208$ ✓ $j=0.931$ ✓

$f_s=\frac{105,770\times12}{1.571\times.931\times70}=$ 12400 #/□" for case 1 OK

$f_c=\frac{12400\times0.208}{15(1-.208)}=$ 217 #/□" " OK

$v=\frac{14,140}{12\times.931\times70}=$ 18.2 #/□" ✓ " OK

$u=\frac{14,140}{2\times3.142\times.931\times70}=$ 35 #/□" ✓ " OK

Section at O4–O4

Max. moment { +43,700 #/ at normal state case 1
−100,780 during earth quake case 3

Max. Shear ∓14,800 # " "

use 2−7/8"ϕ bars = 1.203 □" on top & bottom

$p=p'=\frac{1.203}{12\times46}=.00218$ ✓ $\frac{d'}{d}=\frac{2}{46}=0.0435$ ✓

$K=0.221$ ✓ $j=0.926$ ✓

$f_s=\frac{100,780\times12}{1.203\times.926\times46}=$ 23,630 #/□" for case 3 OK ✓

$f_c=\frac{23,630\times0.221}{15(1-.221)}=$ 447 #/□" " OK ✓

$v=\frac{14,800}{12\times.926\times46}=$ 29 #/□" OK

$u=\frac{14,800}{2\times2.749\times.926\times46}=$ 63.2 #/□" OK ✓

Section at O3–O3

Max. moment { +26,500 #/ at normal state case 1
−22,480 during earth quake case 3

Max. Shear −14,150 # / " " case 2

Use Same Section as for O4–O4.

Section at O5–O5

Max. moment { −86200 #/ during earth quake case 2
+28420 #/ " " case 3

Max. Shear −18,200 # / " " case 2

Use Same section as for O4–O4

f_s & f_c < allowable stress

$v=\frac{18200}{12\times.926\times46}=$ 35.6 #/□" ✓ OK

$u=\frac{18200}{2\times2.749\times.926\times46}=$ 78.3 #/□" ✓ OK

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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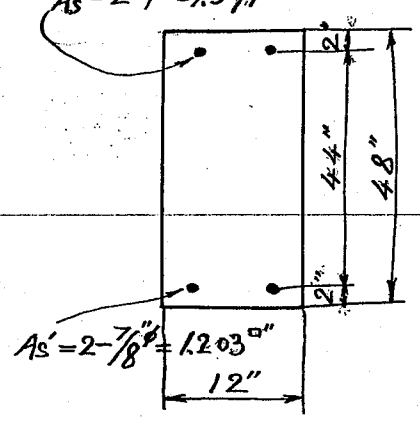
CALCULATIONS FOR

Section at P-P

Max. moment { + 91,350 during earthquake case 3
− 116370 " " Case 2

Max. Shear + 11,420 at normal state case 1.
+ 14,290 during earthquake case 3.



Use 2-1"φ bars = 1.571□" on top
2-7/8"φ " = 1.203□" on bottom
Effective depth = 46" Insulation 2"

$p = \frac{1.571}{12\times46} = 0.00285$ $p' = \frac{1.203}{12\times46} = 0.00218$

for neg. moment $K = 0.251$ $j = .916$

$f_s = \frac{116378\times12}{1.571\times.916\times46} = 21040\ \#/\square" < 30720$ OK

$f_c = \frac{21040\times.251}{15(1-.251)} = 476\ \#/\square" < 1152$ OK

$v = \frac{11,420}{12\times.916\times46} = 22.6\ \#/\square" < 57$ OK

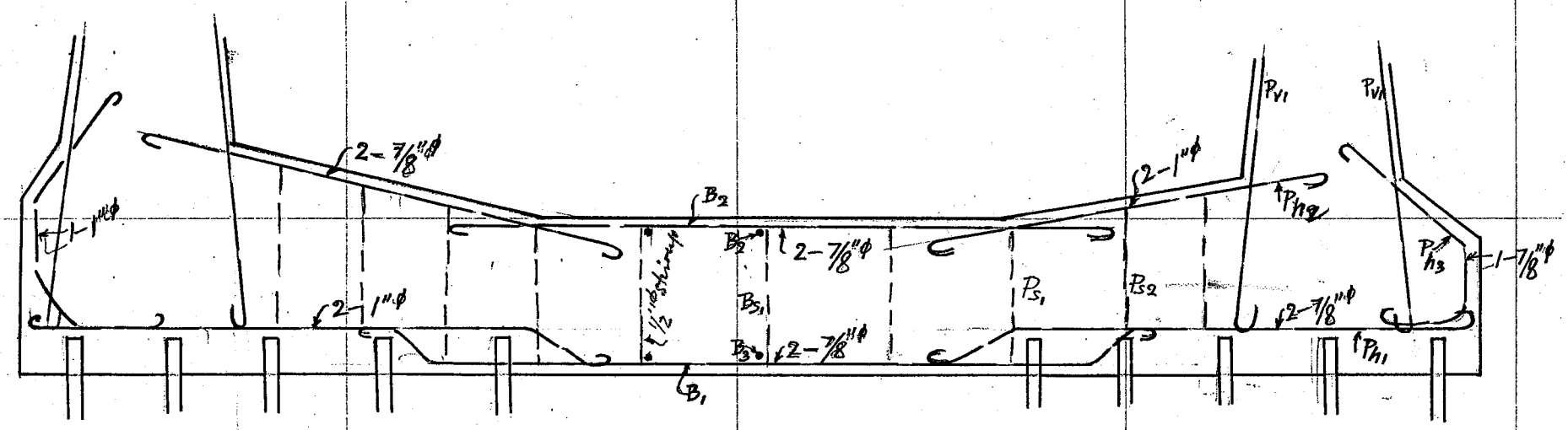
$u = \frac{11420}{2\times3.142\times.916\times46} = 43.1\ \#/\square" < 85$ OK

for pos. moment $K = .222$ $j = .926$

$f_s' = \frac{91550\times12}{1.203\times.926\times46} = 21420\ \#/\square" < 30,720$ OK

$f_c = \frac{21420\times.222}{15(1-.222)} = 408\ \#/\square" < 1152$ OK

$u = \frac{14,290}{2\times2.749\times.926\times46} = 61\ \#/\square" < 153$ OK



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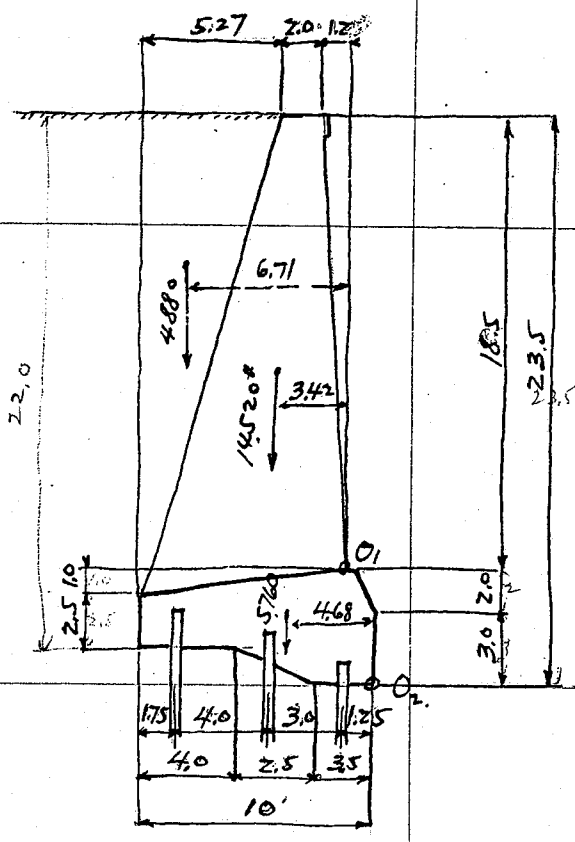
CALCULATIONS FOR

15

Design of 108 A Bridge for City of Tokyo.

Design of wing wall.



Materials of wall shall be as follows.
Shaft. Rubble concrete with stone facing. (大正門石) 150#/cu.ft. assume 140 "
Base 1:3:6 concrete
Weight & Center of gravity of wall.

Shaft. $2.6 \times 18.5 = 48.0 \text{ @ } 150 = 7200 - 550 = 7320^{\#}$ × 1.85 = 13320
$\frac{5.27}{2} \times 18.5 = 48.8 \text{ @ } " = 7320^{\#}$ × 4.96 = 36300
14520# 49620

Center of gravity $\frac{49620}{14520} = 3.42'$ from toe.

Base

1.0 × 4.0 =	4.0 @ 140 =	560#	× .6 =	340
2.5 × 5.0 =	12.5 " =	1750	× 2.25 =	3940
2.5 × 4.25 =	10.63 " =	1490	× 4.68 =	6970
4.0 × 3.5 =	14.0 " =	1960	× 8.00 =	15680
		5760	4.68	26,930

Earth $\frac{5.27}{2} \times 18.5 = 48.8 \text{ @ } 100 = 4880^{\#}$

Stability of shaft.

Shaft.	14520#	× 3.42 =	-49620
Earth rear.	4,880	× 6.71 =	-32,730
Earth pressure	5700	× 6.17 =	35,170
	ΣH = 5.700	ΣV = 19,400	ΣM = -47,180

Earth pressure on shaft.
$\frac{100 \times 18.5^2}{6} = 5,700^{\#}$

Max toe pressure: pressure area = 2.43 × 3 = 7.29 sq.ft.
$= \frac{2 \times 19400}{7.29} = 5330^{\#}/\text{sq.ft.} = 37^{\#}/\text{sq.in.}$ C ok.

Dist. of resultant force from O1 = $\frac{-47,180}{19,400} = -2.43'$

Eccentricity = 5.0 − 2.43 = 2.57'

Stability of wall at bottom of Base.
Taking moment about O2.

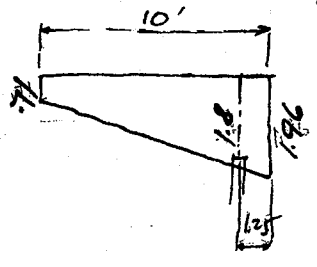
Shaft	14,520	× -4.92 =	-71,400
Earth rear	4,880	× -8.21 =	-40050
Earth pressure	8,060	× 7.33 =	59,050
Vert. comp. of earth press.	4650	× 10.0 =	-46500
Base.	5760	× -4.68 =	-26950
	ΣH = 8,060	ΣV = 29,810	ΣM = -125,850

Earth pressure.
$\frac{100 \times 22.0^2}{6} = 8,060^{\#}$ Vert comp. 8060 × .577 = 4650

Distance of resultant force from O2 = $\frac{-125,850}{29,810} = -4.22'$
Eccentricity = 5.0 − 4.22 = 0.78'

Resultant force within middle third.
Max. toe pressure = $\frac{29810}{10}\left(1 \pm \frac{6 \times 0.78}{10}\right) = 4380^{\#}/\text{sq.ft.} = 1.96$ tons/sq.ft.
or 1,586 = 0.71 " "

Use piles 0.7' ϕ × 30.' long at the intervals of 3.0 × 2.5
Load on one pile at extream fibre = 1.8 × 3.0 × 2.5 = 13.5 tons.



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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CALCULATIONS FOR

東京市四一〇新架橋設計々算書

昭和三年一月

増田橋梁事務所

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

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MADE BY J. Masuda DATE 2-12-30 FILE NO ____
CHECKED BY ____ DATE ____ PAGE NO 1

CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.

General Datas.

The widet of Canal 40.0 meters between faces of walls at Elevation + 10.0 A.P.
The span length of bridge between end bearings 39.0 meters = 127.953'
Making the panel length 14'-2 5/8" (horizontal dimension) for 9 panels = 127'-11 5/8" or 127.969'
Skew of bridge assumed 1½°
Pratt Truss Type has been approved by the City Bridge Engineer Mr. Koike on account of appearance suitable to the site instead of Warren type which will be most economical.
Roadway between curb lines 9.0 meters = 29.528'
Sidewalk from clearance of truss to edge of coping stone 3 meters clear or 9.843'

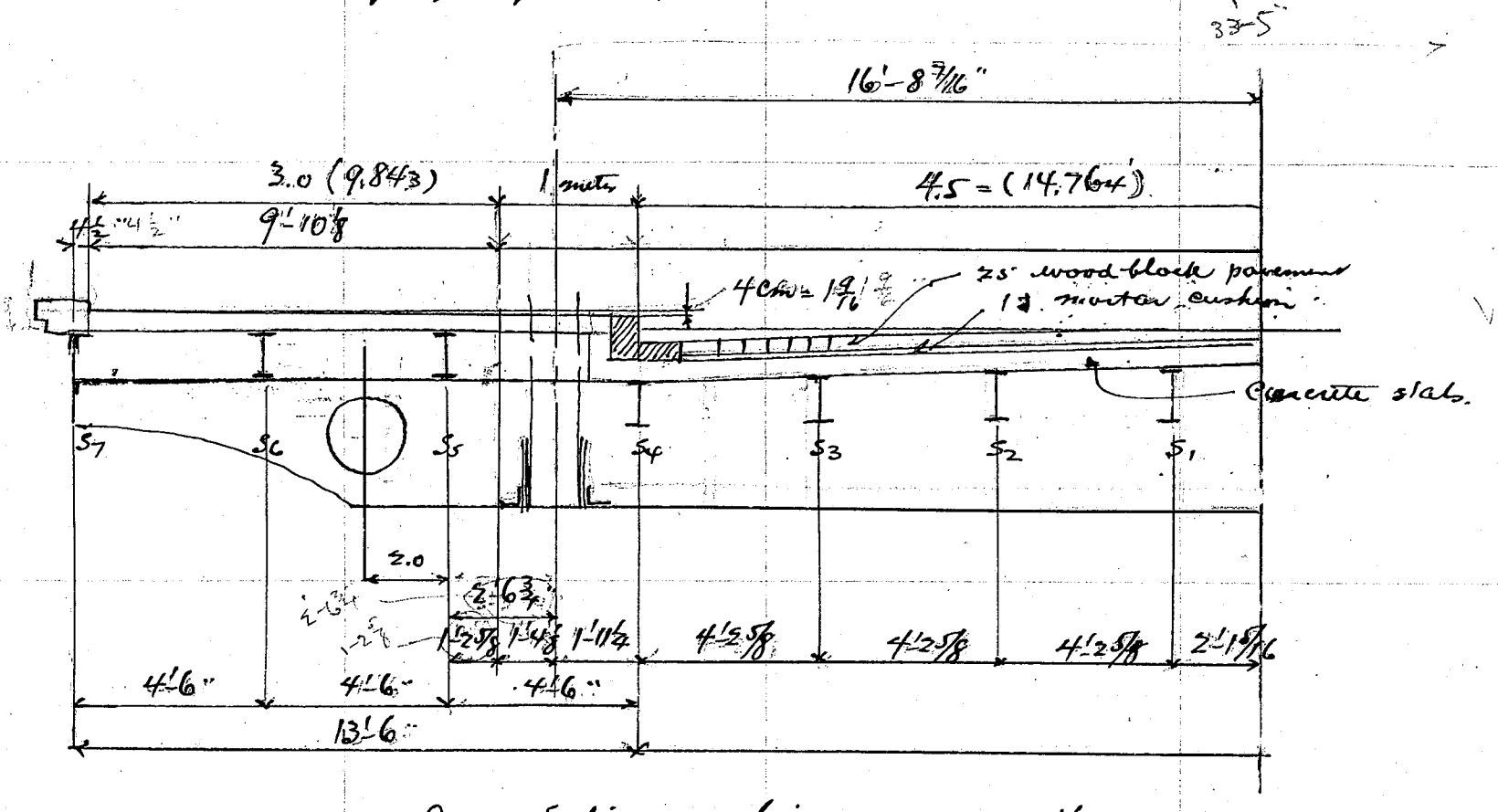
From curb line to edge of protection of truss chord 1.00 meter, which makes 15 meters clear for roadway and sidewalks and 17 meters between inside lines of coping stone at edge of sidewalks.

Longitudinal slope of bridge toward center of span 1/60 parabolic slope and
Transverse slope of roadway also 1/100 parabolic slope.
Slope of sidewalk 1/100 toward curb line of roadway.

Loading on bridge as per specification (Standard City Bridge specification of City of Tokyo).

Other miscellaneous datas given by Mr. Koike (City Bridge Engineer) in contract paper.

General Cross Section of Bridge assumed as Sketch below.



Cross Section of Bridge Scale 1/50

Design of Floor slab. span length 4'-2 5/8 = 4.22'

Dead load	3" wood block pavement @ 60#/12	=	15.0 #
	1/4" mortar cushion @ 110/12	=	11.5
	floor slab assumed		75.0
	allowance		3.5
			105.0 # per sq. ft.

Note slab changed to 6 1/4" thick

Dead Load moment = 1/10 × 105 × 4.22² = 187 ft#
Dead Load shear = ½ × 105 × 4.22 = 222 # per ft strip

増田淳氏関係資料
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CALCULATIONS FOR

Design of HID Bridge for City of Tokyo.

Design of floor slab.

Live Load motor truck loading rear wheel Concentration = 9920 #
30% impact 2980
12900 #

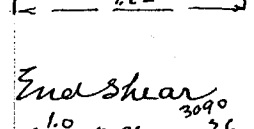
Front wheel Concentration say 12900 ÷ 3 = 4300 #

Distribution of wheel Concentration on floor slab.

Longitudinal distribution a = 2 × 4¼" = .70
20 cm = .66
a = 1.36

Effective width = 2/3 l + a = 2/3 × 4.22 + 1.36 = 4.17

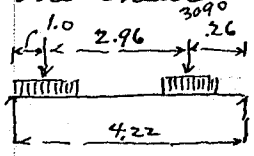
Load per ft strip 12900 ÷ 4.17 = 3090 #



Moment = 1545 × 2.11 = 3260 ''#

for Continuity of slab. reduce the moment to
0.8 × 3260 = 2600 ''#

End Shear



Transverse distribution b = 1.28 + .70 = 1.98 call this 2.0'
From distribution shown on diagram assume the end shear 3090 #
when loaded symmetrically.
Considering concentration only 3090 × 3.48/4.22 = 2550 #

Summary for moment and shear

	moment	shear
Dead Load	187	222
Live Load	2600	2550
	2787 ''	2772 #

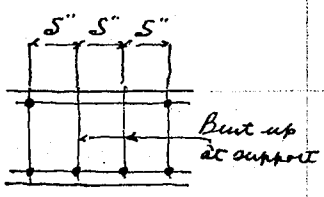
Effective depth of slabs for 17000 #/□" Steel stress and 640 #/□" Concrete stress n = 15

$d = \sqrt{\frac{2787}{102}} = 5.22''$

Make depth of Concrete slab 6¼" over all with 1" insulation at bottom

Steel Area required = $\frac{2787 \times 12}{7/8 \times 5.25 \times 17067} = 0.4270''$ per ft strip

½"ϕ bars .455' spacing required
Use ½"ϕ bars 5" spacing = .467 0" per ft



Unit shear at End = $\frac{2772}{5.25 \times 12} = 44$ #/□" ok

Bondstress for 15" strip. = $\frac{2772 \times 15/12}{7/8 \times 5.25 \times 6.28} = 120$ #/□"

Use 4 - ½"ϕ bars in 15" strip
perimeter = 4 - ½"ϕ = 6.28

Use deformed bars in slab

Sidewalk slab. span length 4'-6"

Dead Load

¾" wearing course (Concrete)	9.5
4" Concrete slab.	50.0 Note changed to 3½" slab
Miscellaneous concrete say	0.5
	60.0 # per sq. ft.

Dead Load moment = 1/10 × 60 × 4.5² = 122 ''#
Dead Load shear = ½ × 60 × 4.5 = 135 #

Live Load 100 #/□'

Live Load moment = 1/10 × 100 × 4.5² = 202 ''#
Live Load shear = ½ × 100 × 4.5 = 225 #

Summary for moment and shear

	moment	shear
Dead Load	122	135
Live Load	202	225
	324 ''	360 #

Effective depth = $\sqrt{\frac{324}{102}} = 1.79''$

Use 3½" Concrete slab with 1" insulation at bottom and ¾" wearing course at top.
Over all depth 4¼" including wearing course

増田淳氏関係資料
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CALCULATIONS FOR

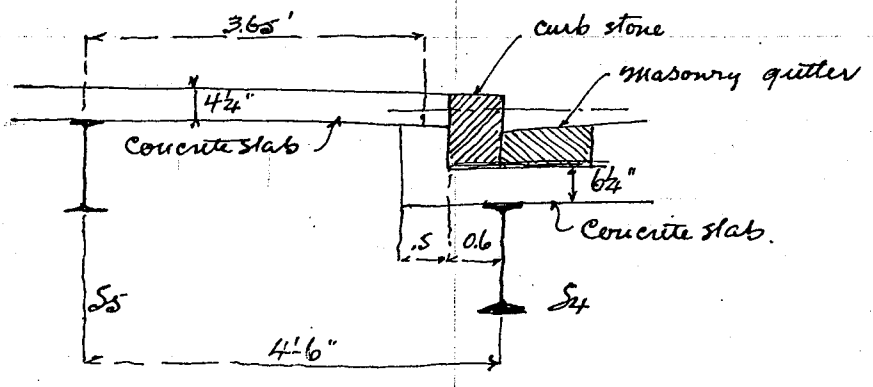
Design of 41D Bridge for City of Tokyo.

Steel area required = $\frac{324 \times 12}{\frac{7}{8}" \times 2.5 \times 17067}$ = 0.1040" per ft strip

Use 3/8" ϕ bars 6" centres = 0.220"

Every other bar should be bent up at support to take care of negative moment

Slab at curb line. Construction assumed as shown on sketch



Sidewalk slab span length 3.65'

Dead Load

4½" slab	150/12 #	=	53 #
misc say			2
			55 # per sq ft.

End reaction = 55 × ½ × 3.65 = say 100 # per lin. ft.

moment = $\frac{1}{10}$ · 55 × 3.65² = 733 '#

Overhang beyond S4

Dead Load	Curb stone 0.6 × 0.9 @ 160	=	86.5
	Slab and cushion say 6¾" × 0.6	=	50.5
			137.0 # arm 0.3'
	Side Concrete beam 0.5 × .85 @150	=	64.0
	Reaction from sidewalk slab		100.
			164.0 # arm 0.85

Dead Load moment	137.0 × 0.3	=	41.0
	164.0 × 0.85	=	140.0
	301.0		181.0 '#

Live Load say 100 #/o'.

	Reaction = ½ · 100 · 3.65'	=	182.5
	direct. 100 · .85	=	85.0
Live Load moment	182.5 × 0.85	=	155
	85.0 · .425	=	36
	267.5		191 '#

Summary for moment and shear

	moment	shear
Dead Load	181.0	301.0
Live Load	191.0	267.5
	372.0 '#	568.5 #

Depth of slab. 6¼"
Effective depth 5.25

Steel area = $\frac{372 \times 12}{\frac{7}{8} \times 5.25 \times 17067}$ = 0.0570"

Longitudinal stringer for Roadway. 51. Span length 14'-2⅞" = 14.22'

Dead Load.

3" woodblock pavement	@ 60/12 #	=	15.0
¼ mortar cushion			11.5
floor slab 6¼"			78.0
allowance for fillet etc			3.5
			108.0 # per sq ft.
Load on stringer	108.0 · 4.22	=	455.
stringer assumed			50
			505 # per lin ft.

増田淳氏関係資料
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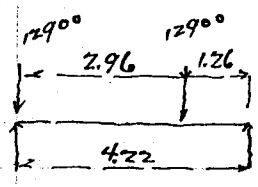
CALCULATIONS FOR

Design of H.D Bridge for City of Tokyo.

Dead Load moment = $\frac{1}{8} \cdot 505 \cdot 14.22^2$ = 12780 ft
Dead Load shear = $\frac{1}{2} \cdot 505 \cdot 14.22$ = 3600 #

Live Load motor truck rear wheel with impact = 12900 #
front wheel with impact = 4300 #

Load on stringer

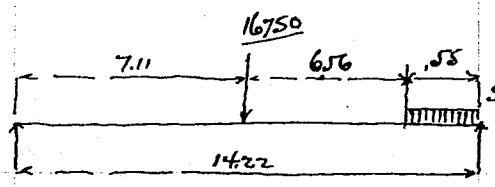


$12900 \cdot \frac{1.26}{4.22}$ = 3850
12900
16750 # for rear wheel
16750 ÷ 3 = 5580 # for front wheel

Uniform Live Load. $w = \frac{120.000}{170 + l} = \frac{120.000}{170 + 4.33} = 687$ kg/m

For 600 kg per sq. meter = 125 # per sq ft.

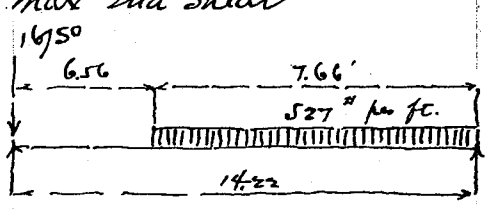
Unif. load = 125 × 4.22 = 527 # per lin ft.



0.55 × 527 = 290 #
Reaction = $290 \cdot \frac{.275}{14.22}$ = 5.6 #

moment due to motor truck $\frac{16750}{2} \cdot 7.11$ = 59500
moment due to unif. load 5.6 · 7.11 = 40
59540 ft

Max End Shear



Uniform load $\frac{527 \cdot 7.66^2}{2 \cdot 14.22}$ = 1090 #
Motor truck loading 16750
17840 #

Summary for	moment	shear
Dead Load	12780	3600
Live Load	59540	17840
	72320 ft	21440 #

Section modulus reqd = $\frac{72320 \times 12}{15600}$ = 55.5
Use 14" × 6" @ 46.01 # Sm = 62.946
Unit stress = $\frac{72320 \times 12}{62.946}$ = 13800 #/0"

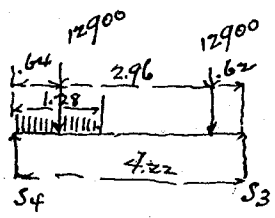
Stringers S2 and S3 same as Stringer S1

Design of stringer S4 at curb line.
Approximate Dead Load
from sidewalk side Reaction p3. 301.0 #
pavement and slab. 108.0
Extra load say 15
123 # . $\frac{4.22}{2}$ = 260.0
561 #
stringer assumed 50
611 # per lin ft.

Dead Load moment = $\frac{1}{8} \cdot 611 \cdot 14.22^2$ = 15500 ft
Dead Load shear = $\frac{1}{2} \cdot 611 \cdot 14.22$ = 4350 #

Live Load

Load on stringer



$12900 \cdot \frac{4.20}{4.22}$ = say 12900 # motor truck rear wheel. with 30% impact

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MADE BY J. Masuda DATE 2-12-31 FILE NO ____
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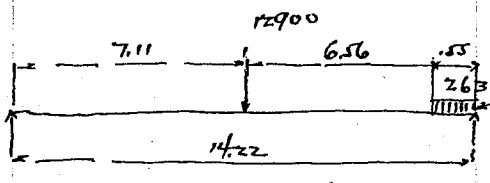
CALCULATIONS FOR

Design of HD Bridge for City of Tokyo.

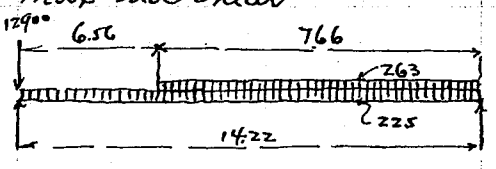
Uniform Live Load on roadway. at rear of motor truck = $\frac{527}{2}$ = 263# per lin. ft.
" " " on sidewalk
100 × $\frac{4.5}{2}$ = 225# per lin ft. throughout ~~the~~ span.



This uniform load neglected.

Moment due to motor truck $\frac{12900}{2}$ × 7.11 = 45900
" " " uniform load $\frac{1}{8}$ × 225 × 14.22² = 5700
51600#

Max End Shear



Uniform load $\frac{263 \times 7.66^2}{2 \times 14.22}$ = 543
" $\frac{1}{2}$ × 225 × 14.22 = 1600
2143
motor truck loading 12900
15043#

Summary for moment and shear

	Moment	Shear
Dead Load	15500	4350
Live Load	51600	15043
	67100#	19393#

Mise 14".6"@46.01# Sm = 62.946

Unit fibre stress = $\frac{67100 \times 12}{62.946}$ = 12800 #/□" OK.

Sidewalk Stringer S5-S6 Span length 14.22 Spacing 4'-6".

Dead Load

Slab and wearing Course 4¼" 53#
Misc fillet &c say 2
55# × 4.5 = 248#
Stringer and bracing say 45
293

Water main assumed 1.5' dia
water content $\frac{1.77}{2.27} \times 63$# = 112 ~~143~~
⅜" metal 15# × 4.7 = 70
Details say 22
Protection 25
117
~~260~~ 229
Hangers & Transverse beam say ~~45~~ 16
~~275~~ 245
Reaction on stringer ~~275~~ 245 × $\frac{2.5}{4.5}$ = 136 ~~153~~ 136
From floor and stringer 293
~~446~~ 429# per lin ft.
call this 430#

Dead Load moment = ⅛ × ~~446~~ 430 × 14.22² = ~~43300~~ 10900#
Dead Load shear = ½ × ~~446~~ 430 × 14.22 = ~~3170~~ 3060#

Live Load 100#/□' Load on stringer 450# per lin. ft.

Live Load moment = ⅛ × 450 × 14.22² = 11400#
Live Load shear = ½ × 450 × 14.22 = 3200#

Summary for moment and shear

	Moment	Shear
Dead Load	~~14300~~ 10900	~~3170~~ 3060
Live Load	11400	3200
	~~22700~~ 22300#	~~6370~~ 6260#

Section modulus req'd = $\frac{22300 \times 12}{15600}$ = ~~17.45~~ 17.15

Mse 12" × 5" @ 31.99# Sm = 36.686

Unit flange stress = $\frac{22300 \times 12}{36.686}$ = ~~7420~~ 7300 #/□" OK

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CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.

Design of Fascia Girder S7 span length 14.22'

Dead Load

Slab	55 × 4.5/2 = 2×62.0 # =	124 #	
Coping ~~stone~~ assumed		100	
Handrail assumed		100	
Stringer assumed		60	
		384 #	per lin. ft.

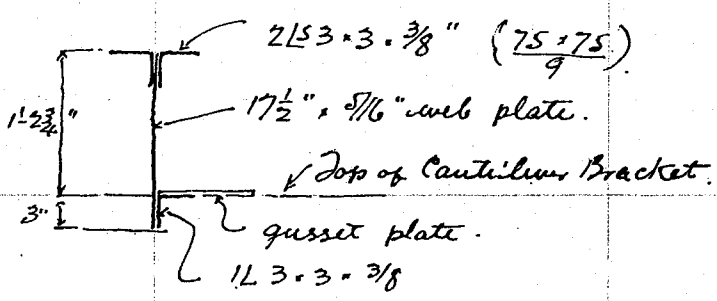
Dead Load moment = 1/8 × 384 × 14.22² = 9680 '#
Dead Load Shear = 1/2 × 384 × 14.22 = 2730 #

Live Load 100 # or 225 # per lin. ft of span assumed.

Live load moment = 1/8 × 225 × 14.22² = 5680 '#
Live load shear = 1/2 × 225 × 14.22 = 1600 #

Summary for moment and shear

	Moment	Shear
Dead Load	9680	2730
Live Load	5680	1600
	15360 '#	4330 #



℄ of Gravity of Section.

		Area				Moment of inertia.
2LS 3×3×3/8	=	4.22	× 0.89 =	3.76	4.22 × 6.4² + 3.6 =	177
1Pl. 17 1/2 × 5/16	=	5.47	× 9.00 =	49.25	5.47 × 1.82² + 139.0 =	157
1L 3×3×3/8	=	2.11	× 15.64 =	33.00	2.11 × 8.35² + 1.8 =	149
		11.80 □"		7.29	86.01	483

Fibre stress compressive side = 15360 × 12 × 7.29 / 483 = 2780 #/□"

Fibre stress tension side = 15360 × 12 × 10.46 / 483 = 4000 #/□"

Approximate weight. 11.80 □" × 3.4 # = 40.0 #
Details & Fillet 200
600 # per lin. ft of span

Dead Load on Cantilever Bracket and Floor Beam between trusses.

Load on Stringer

S1 - S2 - and S3. page 3.

floor and pavement	108 × 4.22 =	458	
Stringer assumed	=	50	
		505 #	

Stringer S4

floor and pavement	561.0
Stringer assumed	50.0
	611.0 # per ft.

Stringer S5 page 3.

pavement & slab. between S4-S5	1/2 × 55 × 3.65 =	say	100
between S5-S6.	55 × 2.25 =		124
water main.	275 × 2.5/4.5 =		153
			377
Stringer and bracing			45
			422
Including conduit pipes &c		say	450 # per lin ft.

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CALCULATIONS FOR

Design of 4D Bridge for City of Tokyo.

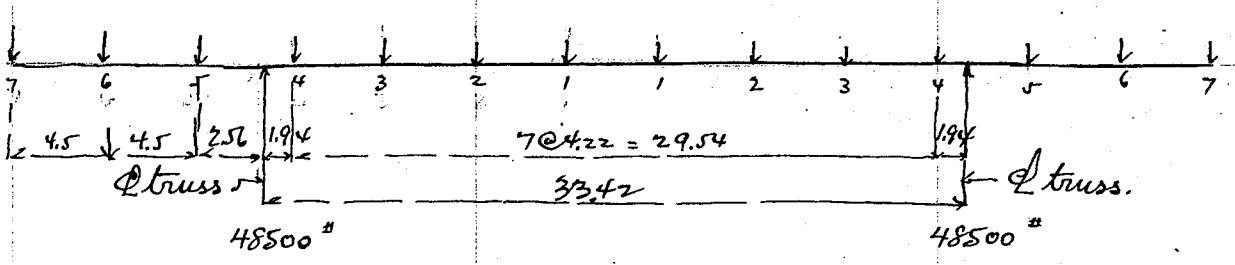
Stringer S6.

Slab.	55 × 4.5 =	248	
stringer and bracing	=	45	
		293	
Water main	275 × 2.0/4.5 =	122	
Conduit pipes etc		35	
		450# per lin ft.	

Stringer S7. page 6 384# per lin ft.

Concentration on floor beam including stringer

S_1	505#	× 14.22 =	7180#	7180
S_2	505	" " =	7180	7180
S_3	505	" " =	7180	7180
S_4	611	" " =	8700	8700
S_5	450	" " =	6400	30240# reaction for cross beam between trusses.
S_6	450	" " =	6400	
S_7	384	" " =	5460	
			48500#	



moment at 6	54600	× 4.5 =	-24600 1#		shear	54600#
moment at 5	54600	× 9.0 =	49200		shear	11860#
	6400	× 4.5 =	28800			
			-78000 1#			
moment at ℄ truss	54600	× 11.56 =	63000		shear	18260#
	6400	× 7.06 =	45200			
	6400	× 2.56 =	16400			
			-124600 1#			
Moment at 4	30240	× 1.94 =	58700		Shear at end	30240
			-124600		Shear 4-3	21540
				-65900 1#		
Moment at 3	30240	× 6.16 =	186200		shear 3-2	14360
	8700	× 4.22 =	-36700			
			-124600			
				+~~34900~~ 24900 1#		
Moment at 2	30240	× 10.38 =	314000		shear 2-1	7180#
	8700	× 8.44 =	-73400			
	7180	× 4.22 =	-30300			
			-124600			
				+85700 1#		
Moment at 1	30240	× 14.60 =	442.000		shear 1-1	0
	8700	× 12.66 =	-110.000			
	7180	× 8.44 =	-60600			
	7180	× 4.22 =	-30300			
			-124600			
				+1165500 1#		

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CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.

Floor beam and Cantilever bracket.
Cantilever Bracket assumed 130# per lin ft.
Floor beam assumed 230# per lin ft including bottom lateral bracings.

Moment at 6. $130 \times \frac{4.5^2}{2} = -1320$ ft#

Shear = $130 \times 4.5 = 585$ #

Moment at 5 $130 \times \frac{9.0^2}{2} = -5270$

Shear = $130 \times 9.0 = 1170$ #

Moment at L truss $130 \times \frac{11.56^2}{2} = -8680$

Shear = $130 \times 11.56 = 1500$ #

Moment at 4 $\frac{230}{2} \times 1.94 \times 31.48 = +70440$
-8680
-1640 ft#

End shear = $230 \times 16.71 = 3850$ #
Shear = $3850 - 230 \times 1.94 = 3404$ #
446

Moment at 3 $\frac{230}{2} \times 6.16 \times 27.26 = 19300$
-8680
$+10620$ ft#

Shear = 3850
$230 \times 6.16 = 1420$
2430 #

Moment at 2 $\frac{230}{2} \times 10.38 \times 23.04 = 27500$
-8680
$+18820$

Shear 3850
$230 \times 10.38 = 2390$
1460 #

Moment at 1 $\frac{230}{2} \times 14.60 \times 18.82 = 31600$
-8680
$+22920$

Shear 3850
$230 \times 14.60 = 3360$
490 #

Summary for Dead Load Moment and shear

Moment at various panel points

	7	6	5	L truss	4	3	2	1
From floor	0	−24600	−78000	−124600	−65900	+34900	+85700	+116500
Own wt	0	−1320	−5270	−8680	−1640	+10620	+18820	+22920
		−25920	−83270	−133280	−67540	+45520	+104520	+139420

24900

Shear at various panel Point

	7-6	6-5	5 to L truss	End Reaction (floor beam)	4-3	3-2	2-1
From floor	5460	11860	18260	30240	21540	14360	7180
Own wt	585	1170	1500	3850	3404	2430	1460
	6045	13030	19760	34090	24944	16790	8640

Live Load moment and shear for Cantilever Bracket.
Uniform live load on sidewalk. $w = \frac{100.000}{170 + l}$ where $l = 8.67$
$= 560$ kg/m² use 100#/0'

Concentration at stringer connection.

at 7	100 × 2.25	= 225	@ 14.22	=	3200 #
" 6	100 × 4.50	= 450	@ "	=	6400 #
" 5		= 450	@ "	=	6400 #

Moment at 6 $3200 \times 4.5 = 14400$ ft#

Moment at 5 $3200 \times 9.0 = 28800$
$6400 \times 4.5 = 28800$
57600 ft#

Moment at L truss $3200 \times 11.56 = 37000$
$6400 \times 7.06 = 45200$
$6400 \times 2.56 = 16400$
16000 98600 ft#

Shear = 16000 #

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CALCULATIONS FOR

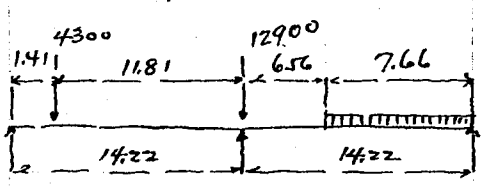
Design of 41D Bridge for City of Tokyo.

Live Load Moment and Shear for Cross beam span length 33.42' spacing 14.22'

Motor truck loading rear wheel with 30% impact = 12900 #
front wheel " " " = 4300 #

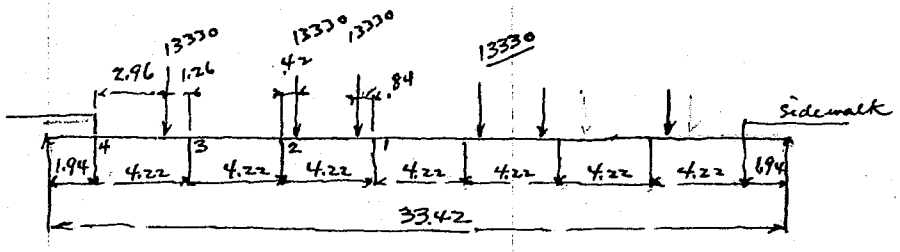
Uniform live load 125 # per sq ft. on Roadway
100 # " " " sidewalk.

Load on floor beam



$\frac{125 \times 7.66^2}{2 \times 14.22} = 257$ # per ft strip

Motor truck $4300 \cdot \frac{1.41}{14.22} = 430$
12900
13330 #



Reaction 3 · 13330 = 39990 #

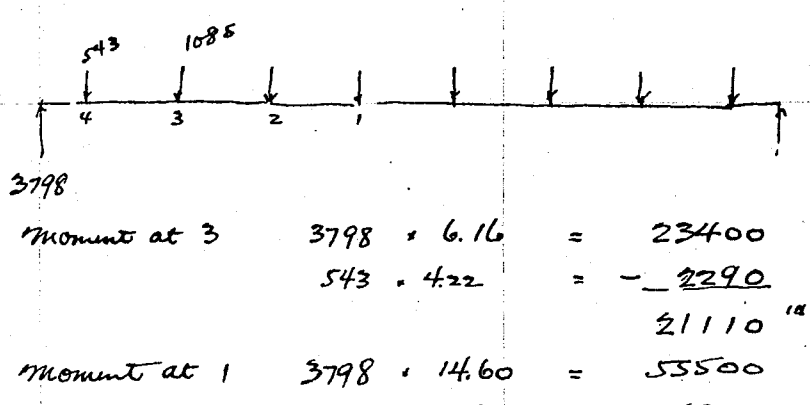
Moment at 4 39990 · 1.94 = 77600 '#

Moment at 3 39990 · 6.16 = 246000
13330 · 1.26 = − 16800
229200 '#

Moment at 2 39990 · 10.38 = 415000
13330 · 5.48 = − 73100
341900 '#

Moment at 1 39990 · 14.60 = 584000
13330 · 14.34 = − 191500
392500 '#

Uniform Live Load on Roadway



257 · 4.22 = 1085 #

Reaction = 3 · 1085 = 3255
543
3798 #

Moment at 4 − 3798 · 1.94 = 7350 '#

Moment at 3 3798 · 6.16 = 23400
543 · 4.22 = − 2290
21110 '#

Moment at 2 3798 · 10.38 = 39400
543 · 8.44 = − 4580
1085 · 4.22 = − 4580
30240 '#

Moment at 1 3798 · 14.60 = 55500
543 · 3 · 4.22 = − 6875
1085 · 3 · 4.22 = − 13750
34875 '#

19.76 ⁰

Uniform live load on sidewalk.

Concentration at 4 225 · 14.22 = 3200 #

moment at 1-2-3-4 3200 · 1.94 = 6200 '#

Summary for Live Load Moment (Cross Beam).

	4	3	2	1	End shear
					43500
motor truck loading	77600	229200	341900	392500	~~39990~~
Uniform load on Roadway	7350	21110	30240	34875	3798
" " " Sidewalk	6200	6200	6200	6200	3200
	91150	256510	378340	433575 '#	~~46988~~ #
					50498 #

Motor truck $6 \times 13330 \cdot \frac{18.17}{33.42} = 43500$ #

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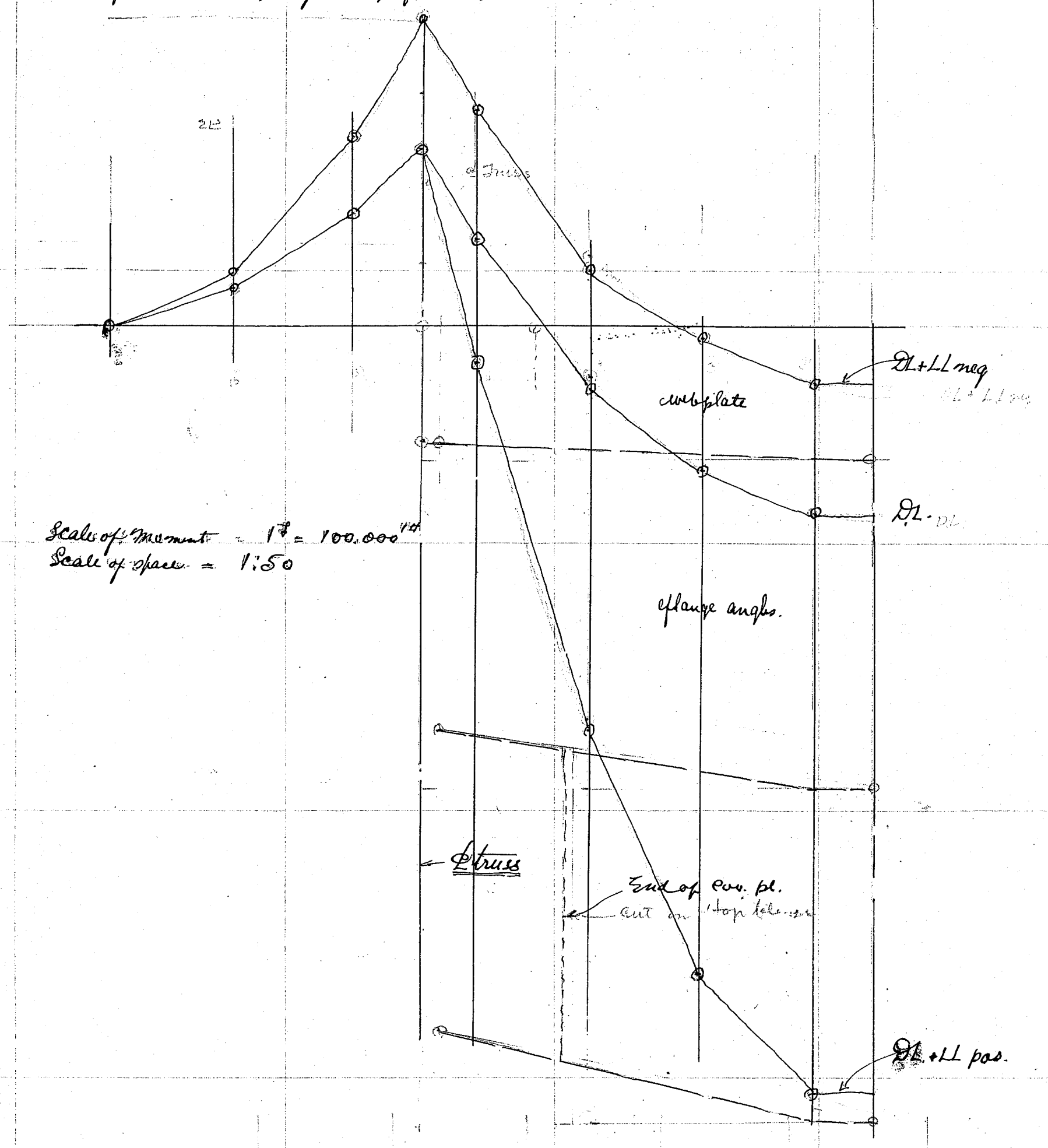
CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.



Scale of moment = 1" = 100.000 "
Scale of space = 1:50

	6	5	L Truss	4	3	2	1
Dead Load	- 25920	- 83270	-133280	- 67540	+ 45520	+ 104520	+ 139420
neg. LL m	- 14400	+ 57600	- 98600	- 98600	- 98600	- 98600	- 98600
	- 40320	- 140870	- 231880	- 166140	- 53080	+ 5920	+ 40820
Dead Load m			- 133280	- 67540	+ 45520	+ 104520	+ 139420
pos. LL m			0	+ 91150	+ 256510	+ 378340	+ 433575
			- 133280	+ 23610	+ 302030	+ 482860	+ 572995

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CALCULATIONS FOR

Design of HD Bridge for City of Tokyo.

Section of floor beam between trusses

web assumed 38 × 3/8 = 14.25 1/8 web = 1.78 0" b to b of Ls 38 1/2"

Section assumed

2 Ls 5 × 3 1/2 · 3/8	= 6.10	− 1.5	=	4.6
1 Cov 11 × 1/2	= 5.50	− 1.0	=	4.5
				9.1 0" net

Effective depth 3.21 − 0.03 = 3.18' Flange stress = 572995 / 3.18 = 180.000 #

Section required = 180.000 / 17068 = 10.55
− 1.78
8.77 0" net

Section of Cantilever Bracket.

Depth 34 1/2" b to b of Ls web assumed 34" × 3/8"

Section assumed 2 Ls 5 × 3 1/2 · 3/8 = 6.10 Effective depth for comp flange = 2.80' abt

Flange stress = 231880 / 2.80 = 82700 # Flange stress on flange Ls only / unit = 82700 / 6.10 = 13550 #/0"

For tension plate = 82700 / 17067 = 4.85 0" net

1/2" tension plate 9 3/4" wide net.

no of rivet 7/8" ϕ 82700 ÷ 6100 = 14 Rivets

3/4" ϕ 82700 ÷ 4420 = 19 Rivets

For Compressive side all butt end ~~to be~~ fit exact ~~to~~ get full bearing.

Approximate weight of Floor Beam between trusses.

	4 Ls 5 × 3 1/2 · 3/8	@	10.4	×	32.0	=	1330
	2 Cov. Pl. 11 × 1/2	@	19.55	×	23.0	=	900
	1 web. 38 × 3/8	@	48.45	×	32.0	=	1550
Stiffs	16 Ls 5 × 3 1/2 · 3/8	@	10.4	×	3.1	=	515
fills	16 Pls. 3 1/2 · 3/8	@	4.46	×	2.5	=	179
	4 Ls 5 × 5 × 3/8	@	12.30	×	2.8	=	138
	4 Pls. 5 × 3/8	@	6.38	×	2.2	=	56
web splice	2 Pls. 12 1/2 · 3/8	@	15.94	×	2.1	=	67
" "	4 Pls. 6 · 3/8	@	7.65	×	3.0	=	92
	misc. fillers &c						100
	Rivet heads &c						153
							5080 #

5080 ÷ 32.0 = 159# per lin ft of girder.

Approximate weight of Cantilever Bracket

	4 Ls 5 × 3 1/2 · 3/8	@	10.4	×	10.4	=	431
	1 Pl. 34 · 3/8"	@	43.35	×	10.4	=	440
Stiffs aver.	6 Ls 3 1/2 × 3 1/2 · 3/8	@	8.5	×	2.5	=	128
	6 Pls. 3 1/2 × 3/8	@	4.46	×	1.9	=	51
End stiffs	2 Ls 5 × 3 1/2 · 3/8	@	10.4	×	2.8	=	58
fills	2 Pls. 3 1/2 · 3/8	@	4.46	×	2.2	=	20
	Bracket at fascia strings				say		40
	Around water main						60
	tension plate for end connection						135
	Rivet heads &c						47
							1410 #

1410 ÷ 10.4 = 136# per lin ft.

2 Cantilever brackets	2@	1410	=	2820
1 floor beam				5080
				7900 #

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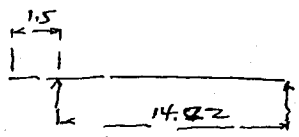
CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.
End floor Beam and Cantilever Bracket.
Stresses will be figured by proportion to intermediate floor beam and cantilever brackets.

Dead Load stress.



max reaction = $\frac{15.72^2}{2 \times 14.22}$ = 8.70 ℓ

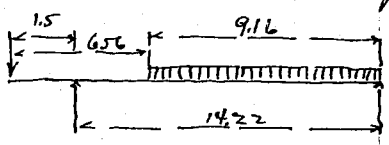
Proportion 8.70 ÷ 14.22 = .611 ℓ

	7	6	5	℄ truss	4	3	2	1
From floor 61.1%		-15020	-47700	-76100	-40300	+~~21300~~ 15200	+52400	+71200
Own wt		-1320	-5270	-8680	-1640	+10620	+18820	+22920
		-16340	-52970	-84780	-41940	+~~31920~~ 25820	+71220	+94120

Cantilever Br. End shear 18260 × .611 = 11150 #
Cant. bracket 1500
12650 #

Floor Beam End shear 30240 × .611 = 18550
Own wt 3850
22400 #

Live Load moment and shear
motor truck loading 12900 × $\frac{15.72}{14.22}$ = 14300 #



14300 ÷ 13330 = 1.070

Uniform load on Roadway. $\frac{125 \times 9.16^2}{2 \times 14.22}$ = 369 369 ÷ 257 = 1.435

Live Load moment Cantilever Bracket | 6 | 5 | ℄ truss
61.1% of interm. Br. -8800 -35200 -60300
shear 16000 × .611 = 9800 #

Live Load Moment and Shear for Cross beam between trusses.
Summary

		4	3	2	1	Shear
Motor truck loading	1.070	83000	245000	366.000	420.000 #	~~46600~~ 42900 #
Uniform load on Roadway	1.435	10550	30300	43400	49900	5450
" " on sidewalk	0.611	3790	3790	3790	3790	1950
		97340 #	279090 #	413190 #	473690 #	~~49300~~ 54,000 #

Summary for Dead and Live Load Moments.

	6	5	℄ truss	4	3	2	1
Dead Load	-16340	-52970	-84780	-41940	+~~31920~~ 25820	+71220	+94120
neg. LL	-8800	-35200	-60300	-60300	-60300	-60300	-60300
	-25140	-88170	-145080	-102240	-~~28380~~ 34480	+10920	+33820
Dead Load			-84780	-41940	+~~31920~~ 25820	+71220	+94120
pos. LL.			0	+97340	+279090	+413190	+473690
			-84780	~~-139280~~ +55400	+~~311010~~ 304910	+484410	+567810

Use same section as for intermediate floor beam and cantilever bracket.

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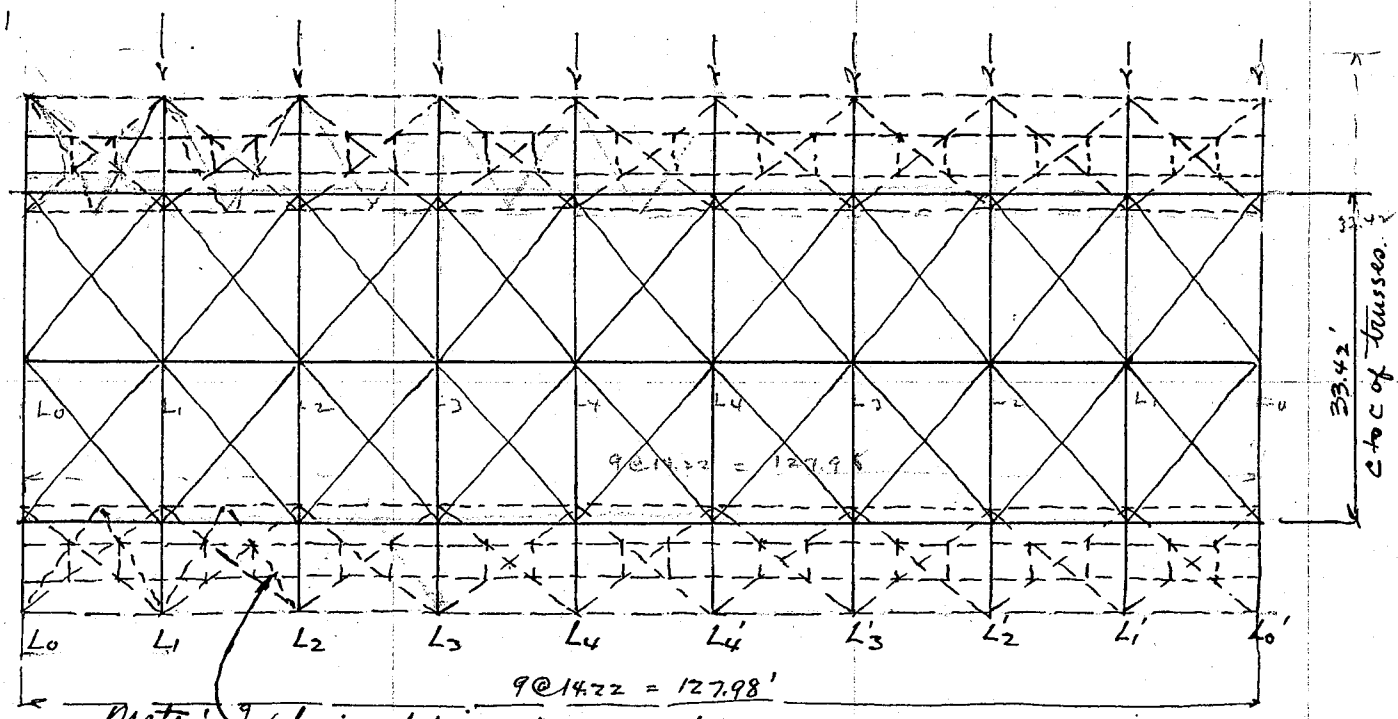
CALCULATIONS FOR

Design of 41.D Bridge for City of Tokyo.
Bottom Lateral Bracing between trusses.



Bridge will be 1½° skew.

Note! In final detail changed these

diagonal length. $\sqrt{14.22^2 + 16.71^2} = 22.0$ $\frac{22.0}{16.71} = 1.315$

Dead load panel concentration on truss

floor load	48500	truss	400 × 14.22 = 5700#
truss lower chord	5700		
	54200		

For two trusses 2 @ 54200 = 108400#

Horizontal thrust due to Earthquake 108400 × 0.4 = 43300#

	Shear	For 1 diagonal		Stress	Unit stress	SR. net	Rivet	
L0-L1	173200	86600	× 1.315 =	114000	30700	3.71	10 7/8"	2L's 3½×3½×⅜ = 4.96 - 4.21
L1-L2	129900	64950	"	85500		2.78	8	" = "
L2-L3	86600	43300	"	57000		1.86	6	" = "
L3-L4	43300	21650	"	28500		.93	3	1L 3½×3½×⅜ = 2.48 - 2.10
L4-L4'	0						3	" = "

strut at center use 2L's 5×3½×⅜ = $r = 1.42$ $\frac{l}{r} = \frac{14.22 \times 12}{1.42} = 120$

Approximate weight of Bottom Lateral Bracings between trusses.

End panel	8L's 3½×3½×⅜	@ 8.5 . 20.0	=	1360
strut	2L's 5×3½×⅜	@ 10.4 . 13.2	=	275
	Center connection	2@45#	=	90
	Both ends of strut	2@70	=	140
	Miscellaneous detail & Rivet Heads			35
				1900#
Center panel				
	4L's 3½×3½×⅜	@ 8.5 . 20.0	=	680
	2L's 5×3½×⅜	@ 10.4 . 13.2	=	275
	Center Connection	2@20		40
	ends of struts	2@50		100
	detail &c	say		25
				1120#

6 panels	@	1900	= 11400
3 "	@	1120	= 3360
			14760# ÷ 128 = 115# per lin ft of span

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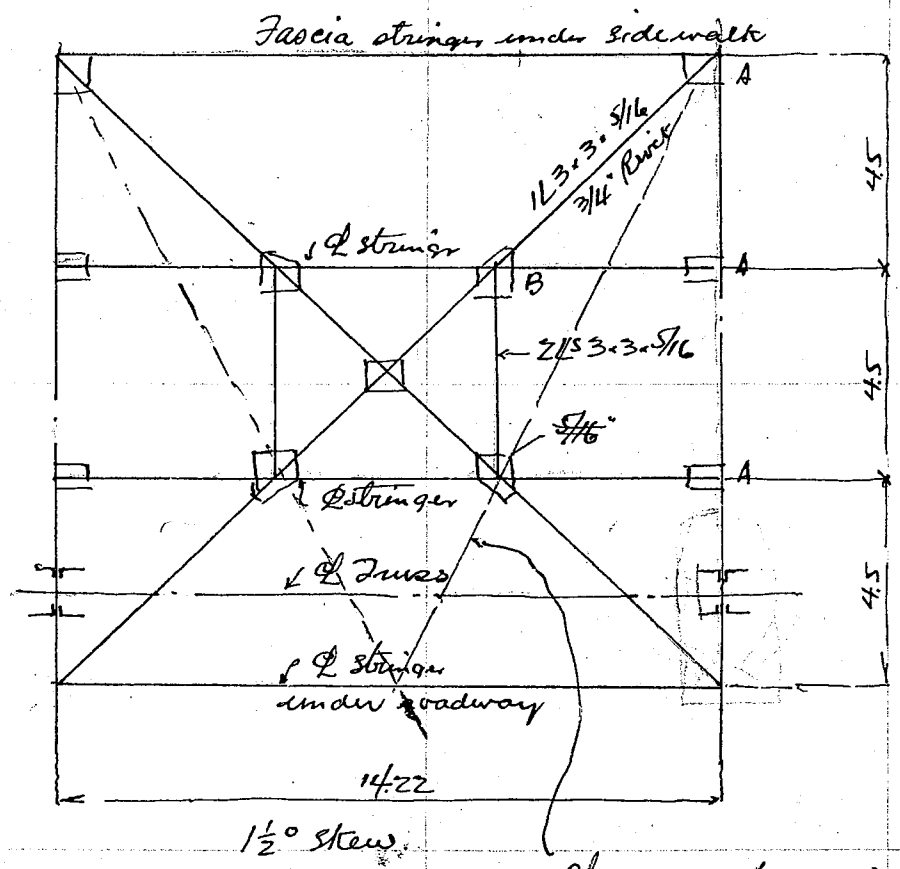
CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.
Lateral Bracings under sidewalk slabs.



Changed thus in final details.

Approximate weight of sidewalk bracings

2LS 3.3.5/16 @6.1 × 19.0	=	232
Center connection		25
Connection A 6@20	=	120
Struts 4LS 3×3×5/16 @6.1 × 4.0	=	98
Connection 4@25	=	100
		575 # per panel

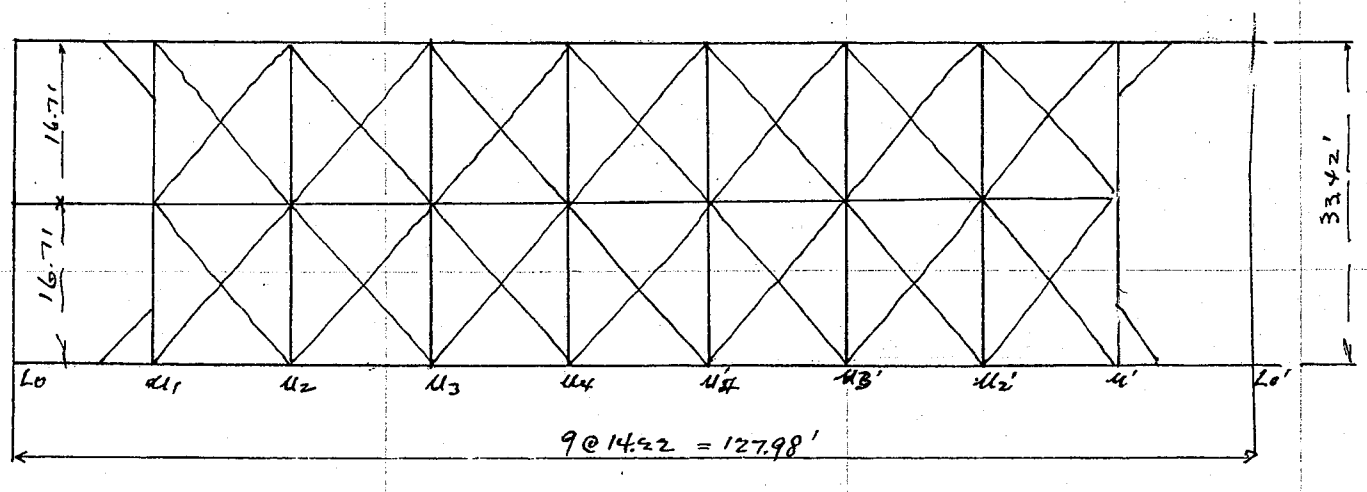
575 × 9 = 5175 # on one side

Both sides of trusses 2@5175 = 10350 #

10350 ÷ 128 = 81 # per lin ft of span

Under sidewalks	10350
Under roadway	14760
	25110 #

Top Lateral Bracings.



Bridge will be 1½° skew.

Upper half of truss 400 × 14.22 = 5700 panel concentration
For both trusses 2@ 5700 = 11400 #
Top lateral sway & portal sway 400 × 14.22 = 5700
17100 #

Horizontal force due to Earthquake = 17100 × 0.4 = 6840 #

	Shear	For 1 diagonal	stress		SR	Rivet No. 3/4 (7950)
U_1-U_2	20420	10210 #	× 1.315 = 13400 #	÷ 30700. =	.436	2
U_2-U_3	13680	6840	9000		.293	
U_3-U_4	6840	3420	4500 #		.146	
U_4-U'_4		0				

Length of member 22.0' Try 2LS 5×3½×3/8 ll riveted back to back r = 1.33

l/r = 22.0×12 / 1.33 = 199. Use 6 Rivets for connection.

Longitudinal strut. 4LS 3½.3½.3/8 depth 18½" about with double lacing

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CALCULATIONS FOR

Design of 4D Bridge for City of Tokyo

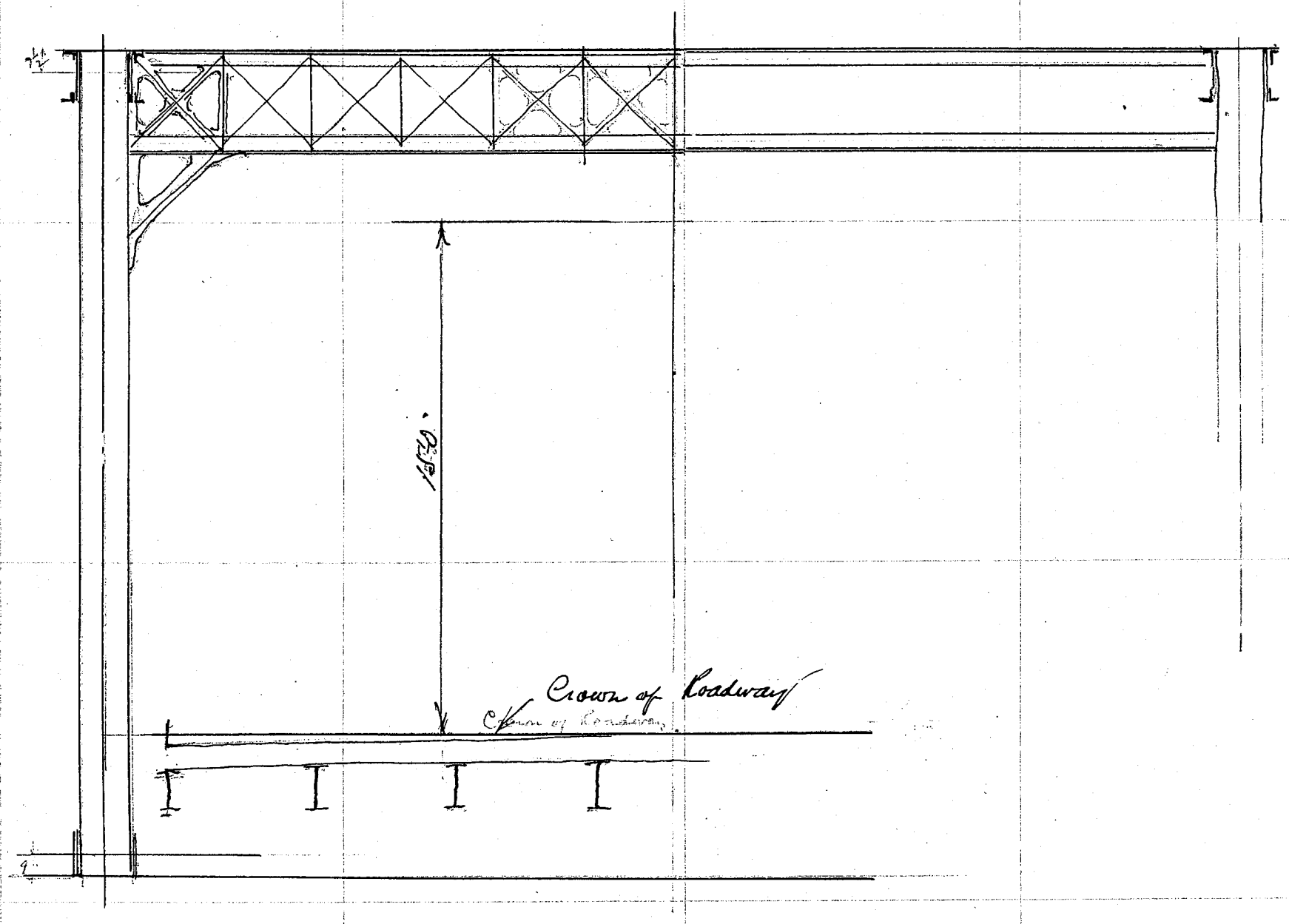
Approximate weight of top latival bracing and longitudinal strut.

	8 Ls 5 × 3½ × ⅜	@	10.4	× 20.0	=	1667
	Center connection		2 @ 50		=	100
Strut	4 Ls 3½ × 3½ × ⅜	@	8.5	× 13.2	=	449
	lacing bars		60 × 10		=	60
	tie Pls.		2 @ 75		=	150
	Connection plo		2 @ 50		=	100
	misc details	say				50
						2576 call this 2580

for 7 panels @ 2580 = 18060 ~~23220~~ #

Details of Sway Bracing and approximate weight.

Top and Bottom flange Ls 2 Ls 6×4×⅜ r = 1.60 $l/r = \frac{16.7 \times 12}{1.60} = 125.5$



Approximate weight of Sway bracing

	4 Ls 6×4×⅜	@	12.3	× 32.0	=	1580
vertical	28 Ls 3×3×7/16	@	6.1	× 3.0	=	514
diagonal	24 Ls 3×3×7/16	@	6.1	× 3.0	=	439
Cente connection	12 @ 15				=	180
	26 @ 25				=	650
Bracket say	2 @ 175					350
misc details say						150
						3863 call this 3900#

6 @ 3900 = 23400#

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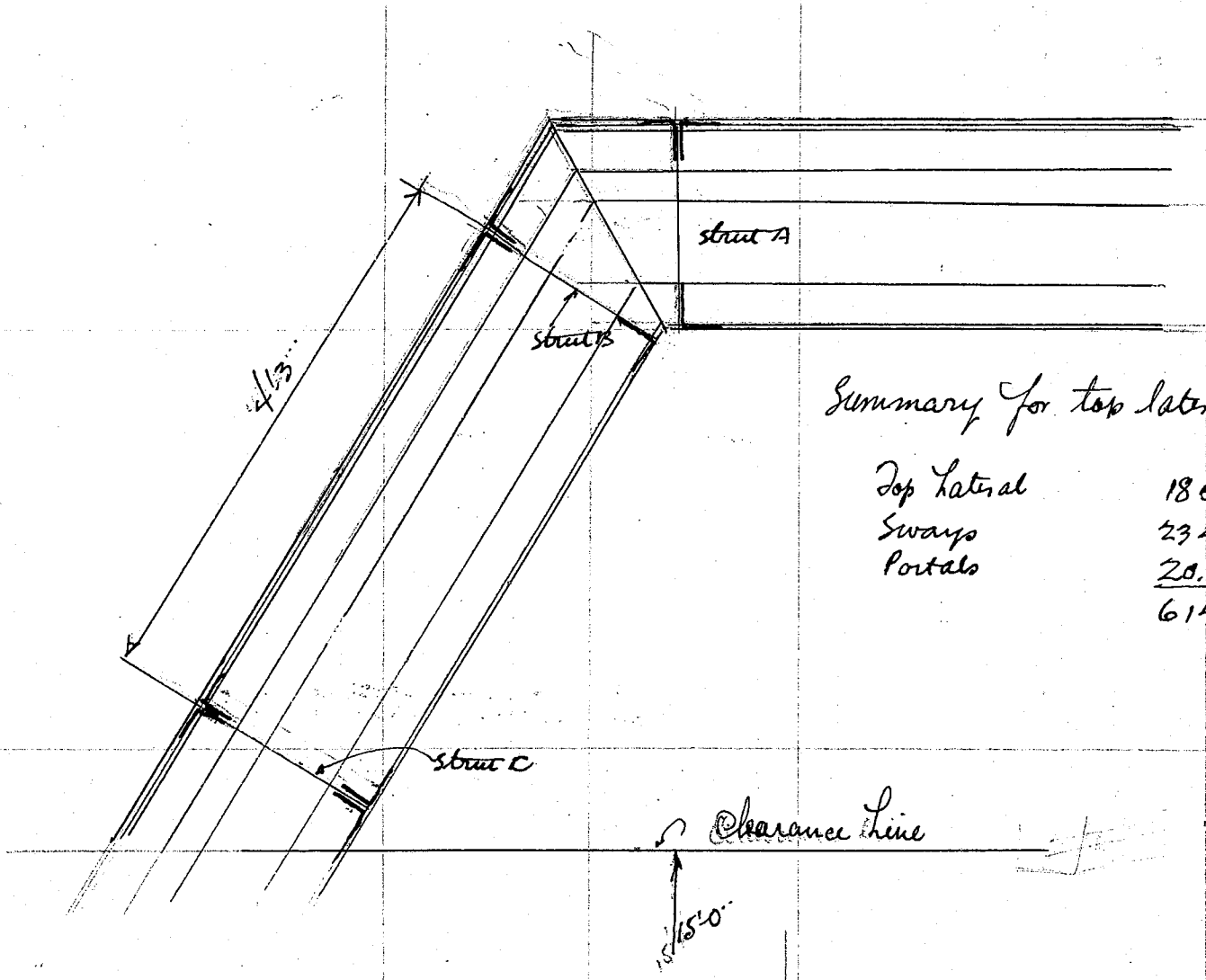
CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo
Details of Portal Bracing and approximate weight.



Summary for top lateral Bracing Sways & Portals

Top Lateral	18060
Sways	23400
Portals	20.000
	61460 # ÷ 128 = 480 # per lin ft.

Strut A.	3 Ls 4 × 4 × 3/8 @ 9.8 × 32.0 =	940
	3 Pls. 3@50	150
	lacing bars 6 × 24	144
		1234
Strut B	do.	
Bent Plate at top	45 × 35.4 =	1595
Strut C	4 Ls 4 × 4 × 3/8 @ 9.8 × 32.0 =	1255
	3 pls and lacings	290
		1445
Verticals	4 Ls 2½ × 2½ × 5/16 @ 5.0 × 4.25 =	85
	1 Pl. 18 × 5/16 @ 19.13 × 4.25 =	82
	Connection Ls	36
		203 × 7 = 1421
Diagonals	2 Ls 4 × 4 × 3/8 @ 9.8 × 4.5 =	88
	2 tie Pls. 2@25 =	50
	lacing bars	10
		148 × 10 = 1480
Gusset pls.	36 @ 30 =	1080 #

Summary for Portal Bracings

Strut A	1234
Strut B	1234
Bent Plate	1595
Strut C	1545
verticals	1421
Diagonals	1480
Gusset pls.	1080
misc details	411
	10,000 # per piece

2 @ 10.000 = 20.000 #

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CALCULATIONS FOR

Design of 41 D Bridge for City of Tokyo.

Dead Load on truss.

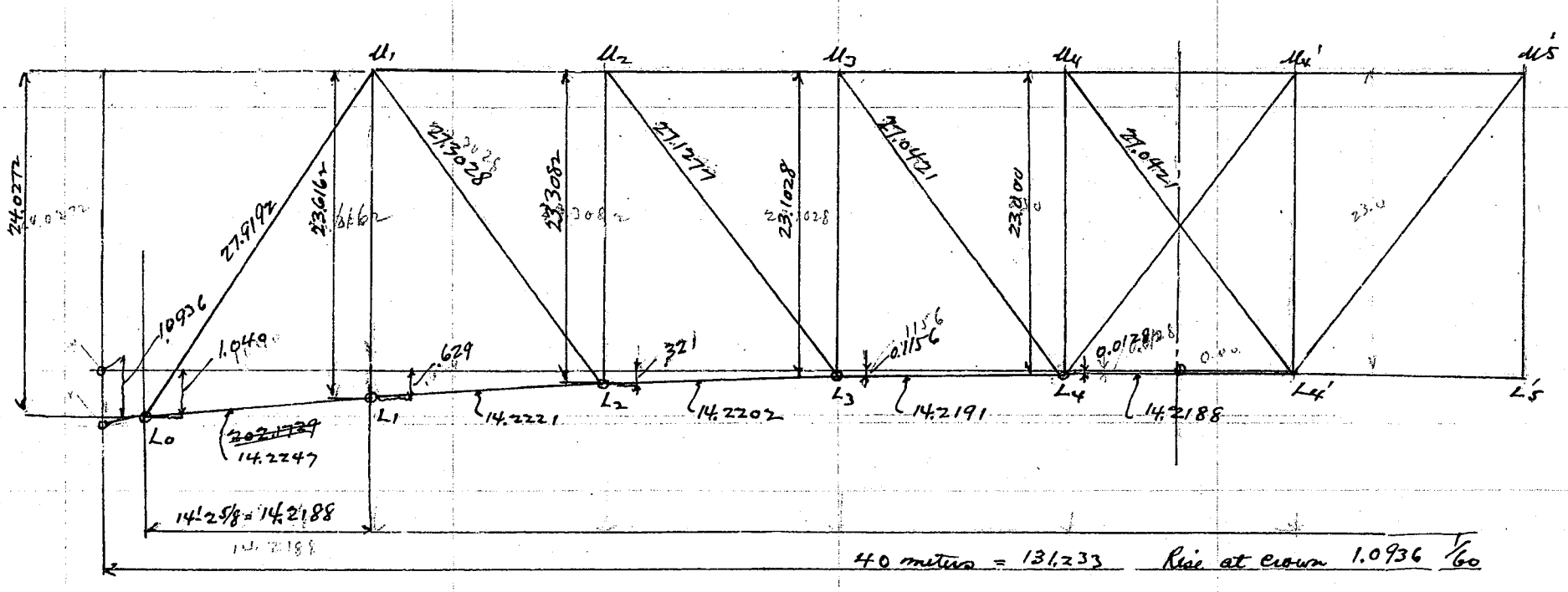
Roadway Floor and pavement ½	108 × 13.764 =	1485	
Gutter and slab 1' wide		148	
Curb stone and Concrete beam		201	
Sidewalk slabs. 55" × 12.90'		710	
Coping and Handrail assumed		200	
Water main and accessories assumed		275	
Gas and Conduit pipes assumed		100	
		3119 #	for ½ width

Structural steel in Bridge

Stringers	8 @ 50 =	400		
Sidewalk st.	4 @ 35 =	140		
Fascia stringer	2 @ 60 =	120		
		660		
Floor beams & Cantilevers	7900 ÷ 14.22 =	555		
Bottom Laterals sidewalks & Rwy		196	For lower chord	
trusses (Lower Half).		800	floor -	3119
		2211	steel -	1106
For one truss.	2211 ÷ 2 =	1106 #		4225 #
trusses (upper half)		800		
Top Lateral Bracings		480		
		1280		
For one truss	1280 ÷ 2 =	640 #		

Lower chord Panel Concentration 4225 × 14.22 = Say 60,000 #
Upper chord " " 640 × 14.22 = 9100 #

Truss: —
Top chord of truss - horizontal Bottom chord 1/60 parabolic curve Span - 40 meters
Crown 40.0 meters = 131.233 1/60 = 1.0936' = 1'-1⅛" ≒ 1.16 ft
Form of truss and dimensions as shown on sketch



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CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.

Dead Load stresses of truss.

Reaction = lower Panel 60.000
upper panel 9100
69100 × 4 = 276400 #

End Post. L_0-U_1 $276400 \times \frac{14.2188}{12.027} = 327.000^{\#}$ C

Top chord U_4-U_4', U_3-U_4
Bott. chord L_4-L_4' $69100 \times 10 \times \frac{14.2188}{23} = 427500^{\#}$

Note ;- For moment arms see page 17 and 19

Top chord U_2-U_3
Bott chord L_3-L_4 $69100 \times 9 \times \frac{14.2188}{23.1028} = 383500^{\#}$

Top chord U_1-U_2
Bott. chord L_2-L_3 $69100 \times 7 \times \frac{14.2188}{23.308} = 296.000^{\#}$

Bott. chord L_1-L_2 $69100 \times 4 \times \frac{14.2188}{23.611} = 165800^{\#}$ T

Bott. chord L_0-L_1 $69100 \times 4 \times \frac{14.2188}{23.567} = 166400^{\#}$ T

Diagonal U_4-L_2

$276400 \cdot 1104.82 = 305.000.000$
$69100 \cdot 1090.6 = 75500.000$
$229500.000 \div 931.03 = 246.500^{\#}$ T

Diagonal U_2-L_3

$276400 \cdot 1641.16 = 454.000.000$
$-\ 69100 \cdot 3239.66 = -224.000000$
$230.000.000 \div 1373.45 = 167500^{\#}$ T

Diagonal U_3-L_4

$276400 \cdot 3238.13 = 895.000.000$
$69100 \cdot 9629.07 = 665.000.000$
$230.000.000 \div 2717.83 = 84.750^{\#}$ T

Diagonal U_4-L_4' stress = 0.

Vertical U_2-L_2

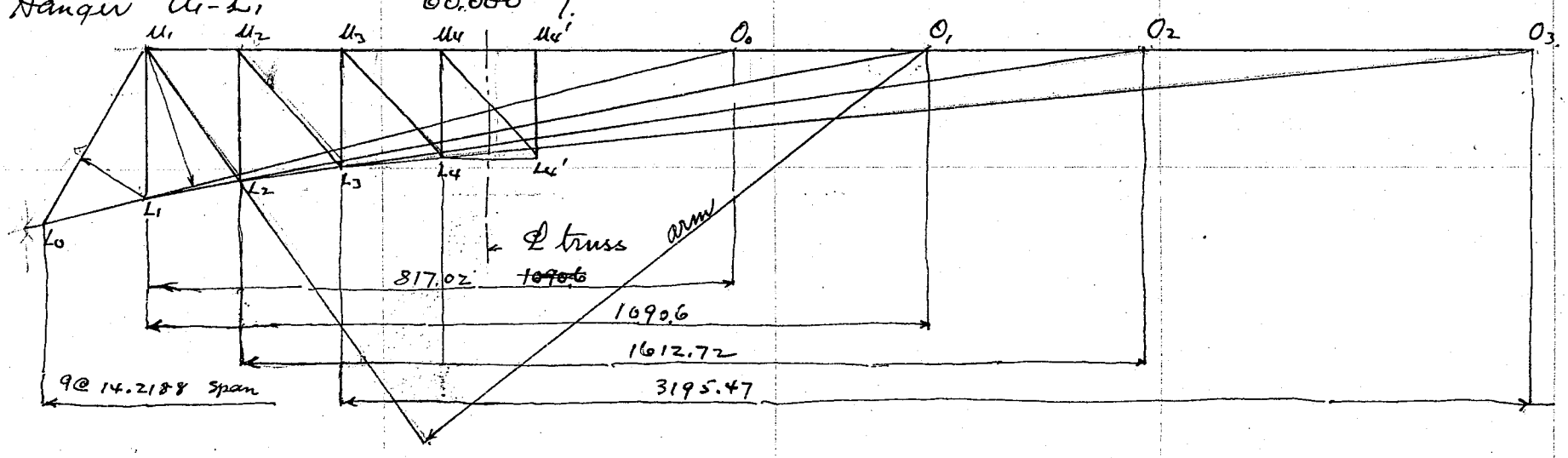
$276400 \cdot 1641.16 = 454.000.000$
$69100 \cdot 1626.94 = 112.500.000$
$69000 \cdot 1612.72 = 96800.000$ } $-\ 209300.000$
$244700.000 \div 1612.72 = 151800^{\#}$ C

Vertical U_3-L_3

$276400 \cdot 3238.13 = 895.000.000$
$69100 \cdot 6433.60 = 444.000.000$
$60000 \cdot 3195.47 = 191800.000$ } $-\ 635800.000$
$159200.000 \div 3195.47 = 49900^{\#}$ C

Vertical U_4-L_4 $9100^{\#}$ C

Hanger U_1-L_1 $60.000^{\#}$ T.



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CALCULATIONS FOR

Design of 4D Bridge for City of Tokyo.

		Lever Arm	Center of moment
Top chord members	U_1-U_2	23.308	L_2
	U_2-U_3	23.103	L_3
	U_3-U_4	23.000	L_4
	U_4-U_4'	23.000	L_4'
End post	L_0-U_1	12.027	L_1
Bottom chord members	L_0-L_1	23.567	U_1
	L_1-L_2	23.611	U_1
	L_2-L_3	23.306	U_2
	L_3-L_4	23.102	U_3
	L_4-L_4'	23.000	U_4
Verticals	U_1-L_1	14.157	O_4
	U_2-L_2	1612.72	O_2
	U_3-L_4	3195.47	O_3
	U_4-L_4		
Diagonals	U_1-L_2	931.03	O_1
	U_2-L_3	1373.45	O_2
	U_3-L_4	2717.83	O_3
	U_4-L_4'		

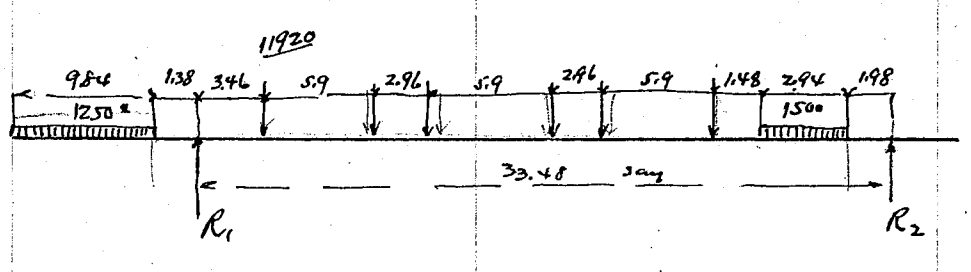
Live load stresses of truss members.

assumed loadings motor truck rear wheel concentration 9920
Impact = $\frac{20}{60+L} = \frac{20}{60+39} = 20.2\%$ 2000
11920#

Front wheel concentration 11920 ÷ 3 = 3975#

Uniform live load on roadway $w = \frac{120.000}{170+l} = \frac{120.000}{209} = 575\ kg/m^2 = 118\#/0'$ say $120\#/0'$

Uniform live load on sidewalk $w = \frac{100.000}{170+l} = \frac{100.000}{209} = 480\ kg/m^2 = 98\#/0'$ say $100\#/0'$



Uniform load on side of truck
for rear wheel 12.48' assumed
" front wheel 9.19' "
On Roadway 120 × 12.48 = " 1500#
On sidewalk 100 × 12.48 = " say 1850#

For rear wheel

9.19 ÷ 12.48 = .735

Uniform load	4400	×	3.45	= 15.200	@ .735	= 11.200#
Rear wheel	11920	×	6.40	76300		
"	"	×	12.30	146700		
"	"	×	15.26	182000	1302500 × 1/3	= 434200
"	"	×	21.16	252000		
"	"	×	24.12	287500		
"	"	×	30.02	358000		
Unif. load	12300	×	39.78	489.000	@ .735	= 360.000
				1806700#		805400

For rear wheel R_1 = 1806700 ÷ 33.48 = 54000# including uniform load on sides of motor truck.

For front wheel R_1 = 805400 ÷ 33.48 = 24.000#

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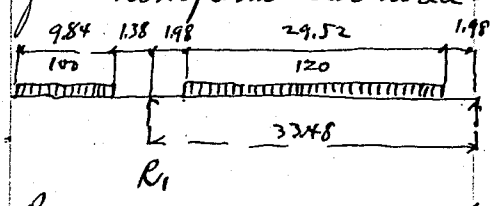
CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.

Full Uniform live load

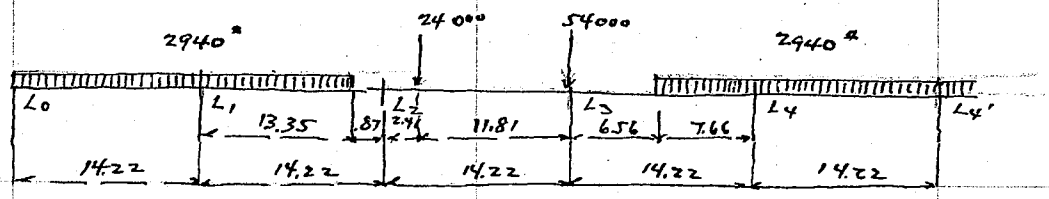


Reaction R1

984 × 3978/3348 = 1170
29.52 × 120 ÷ 2 = 1770
2940

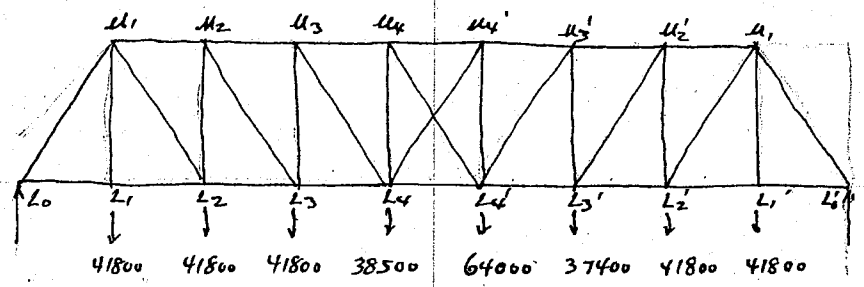
Panel Concentration 2940 × 14.22 = 41800#

Panel Concentration due to live load assumed as below.



	18500	54000		
		4000		
20900	20.000	6100	20900	20900
20800			16500	20900
41700				
say 41800	38500	64.100	37400#	41800#

Chord members.



U3 U4 - U4'
L4 - L4'

Reaction

41800 × 24/9 = 111.600
37400 × 3/9 = 12500
64000 × 4/9 = 28.450
38500 × 5/9 = 21.450
174.000#

Moment at L4'

174.000 × 14.22 × 5 = 12,390.000
38500 × 14.22 = - 547.000
41800 × 14.22 × 9 = - 5355.000
6488 000 ÷ 23.00 = 282.000#

Top chord U2-U3
Bott. chord L3-L4

Rear wheel at L3

Reaction

41800 × 18 = 753.000
37400 × 5 = 187.000
64000 × 6 = 384.000
38500 × 7 = 269.000
1593.000 ÷ 9 = 177000#

Moment about panel point 3.

177.000 × 14.22 × 3 = 7,550.000
38500 × 14.22 = - 547.000
41800 × 14.22 × 2 = - 1188.000
5815000 ÷ 23.102 = 251800#

Top chord U1-U2 and Bottom chord L2-L3.

Rear wheel at L2

Reaction

41800 × 15 = 628.000
37400 × 6 = 224500
64000 × 7 = 448000
38500 × 8 = 308 000
1608500 ÷ 9 = 178500#

moment about panel point 2.

178500 × 14.22 × 2 = 5,080.000
- 38500 × 14.22 = - 547.000
4533.000 ÷ 23.306 = 194500#

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CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.

Lower chord L_0-L_1 and L_1-L_2 Rear wheel at L_1

Reaction R_1

41800 × 21 = 877.000
37400 × 7 = 261800
64000 × 8 = 512.000
1650800 ÷ 9 = 184.000 #

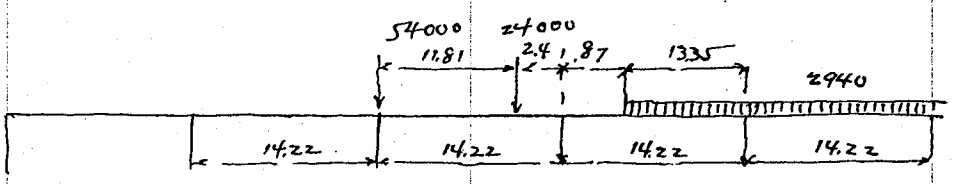
Moment at U_1 184000 × 14.22 = 2.618.000 ##

Stress in L_1-L_2 = 2618000 ÷ 23.611 = 110600 # T
Stress in L_0-L_1 = 2618000 ÷ 23.567 = 111200 # T

End Post L_0-U_1 loading same as above.

Stress in L_0-U_1 = 2618000 ÷ 12.027 = 217.500 # C

Live load for max shear



54000			
4000	20.000	20900	
	18500	20800	
58000	38500	Sy 41800	41800

Diagonal Members U_1-L_2 Rear wheel at L_2

Reaction 41800 × 15 = 627.000
38500 × 6 = 231.000
58000 × 7 = 406.000
1.264.000
1.264.000 ÷ 9 = 140.500 #

Centre of moment at O_1 arm 931.03'
140.500 × 1104.82 = 155.500.000 ##

U_1-U_2 = 155,500.000 ÷ 931.03 = 167.000 # T

Diagonal U_2-L_3 Rear wheel at L_3

Reaction 41800 × 10 = 418.000
38500 × 5 = 192500
58000 × 6 = 348000
958.500 ÷ 9 = 106500

Centre of moment at O_2 arm 137345
106500 × 1641.16 = 175.000.000 ##

U_2-L_3 = 175.000.000 ÷ 1373.45 = 127500 # T

Diagonal U_3-L_4 Rear wheel at L_4

Reaction 41800 × 6 = 255000
38500 × 4 = 154.000
58000 × 5 = 290.000
699.000 ÷ 9 = 77800

Centre of moment at O_3 arm = 2717.83
77800 × 3238.13 = 251.500.000 ##

U_3-L_4 = 251.500.000 ÷ 2717.83 = 92600 # T

Diagonal U_4-L_4' Rear wheel at L_4'

Reaction 41800 × 3 = 125.500
38500 × 3 = 115500
58000 × 4 = 232.000
473.000 ÷ 9 = 52.600

U_4-L_4 = 52600 × 27.042 / 23.0 = 61.800 # T

For one diagonal 61800 ÷ 2 = 30900 # T or C

Diagonal ~~U_4-L_4'~~ U_3-L_4 Counter stress

Reaction 41800 × 1 = 41800
38500 × 2 = 77000
58000 × 3 = 174000
292800 ÷ 9 = 32580 #

Moment = 32580 × 3110.15 = 101.300.000

U_3-L_4 = 101.300.000 / 2717.83 = 37300 # C

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CALCULATIONS FOR

Design of HID Bridge for City of Tokyo.

Verticals U_2-L_2 Rear wheel at L_3 1612.72 + 14.22 × 2 = 1641.16

Reaction 106500#

$$\text{stress} = 106500 \times \frac{1641.16}{1612.72} = 108500\#\ C$$

Vertical U_3-L_3 Rear wheel at L_4 3195.47 + 14.22 × 3 = 3238.13

Reaction = 77800

$$\text{stress} = 77800 \times \frac{3238.13}{3195.47} = 78900\#\ C$$

Vertical U_4-L_4 Rear wheel at L_4'

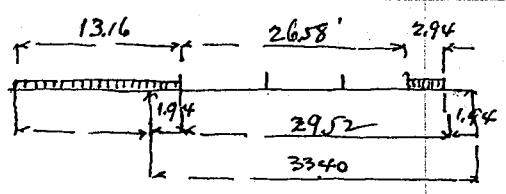
Reaction = 52600 stress = 52600# C

Vertical U_3-L_3 tension in member

Reaction	41800	× 1 =	41.800	
	38500	× 2 =	77.000	
	58000	× 3 =	174.000	
			292.800 ÷ 9 =	32500

Hanger U_3-L_1

Live load floor beam reaction.



motor truck loading 6 × 13330 × 18.17/33.42 = 43500# per

Unif. load 125 × 2.94 = 368.0 per ft

100 × 13.16 = 1316 " "

368.0 × 3.41 ÷ 33.40 = 38.7 × 14.22 = 550

1316 × 38.04 ÷ 33.40 = 1500 × 14.22 = 21400

21950

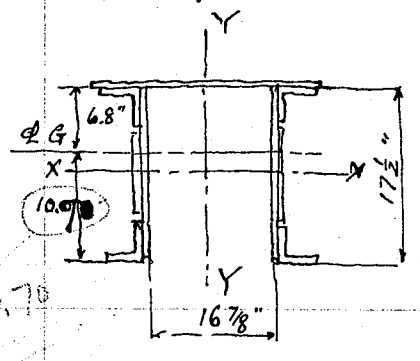
motor truck loading = 43500

Unif. live load 21950

65450 #

Panting component of bottom chord member assume LL. stress 70.000# T

Section of top chord.



10.70

Center of gravity of section X-X									M of inertia. XX.	
2 L's 4×4×½	=	7.50	×	1.18	=	8.85	×	9.52^2	+ 11.2	= 691.0
2 Pls 17×½	=	17.00	×	8.75	=	148.80	×	1.95^2	+ 409.	= 474.0
2 Pls. 9×½	=	9.00	×	8.75	=	78.70	×	1.95^2	+ 60.7	= 95.0
2 L's 4×4×½	=	7.50	×	16.32	=	122.50	×	5.62^2	+ 11.2	= 247.0
1 Cov Pl. 26.7/16	=	11.38	×	17.72	=	202.00	×	7.02^2	+	= 558.0
		52.38		10.7"		560.85				2065.0

$$r = \sqrt{\frac{2065}{52.38}} = 6.28$$

Moment of inertia Y Y axis

					Moment of inertia
1 Cov. Pl.	26×7/16	= 11.38	×	=	641
2 webs.	17×½	= 17.00	× 8.64^2	=	1270
4 L's	4×4×½	= 15.00	× 10.05^2 + 22.4	=	1540
2 Pls.	9×½	= 9.00	× 9.12^2	=	747
		52.38			4198

$$r = \sqrt{\frac{4198}{52.38}} = 8.95$$

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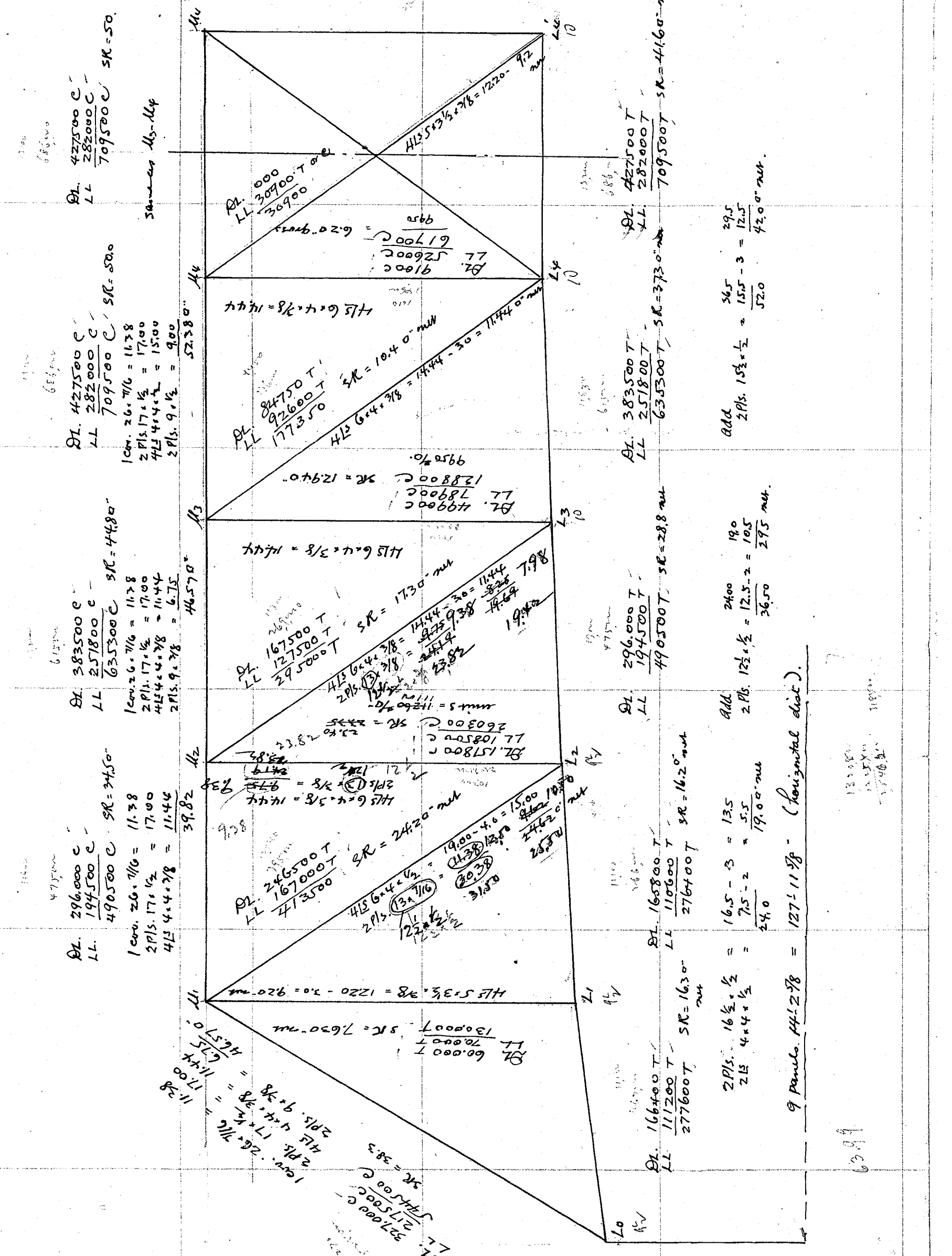
CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.
Stresses and sections of truss.


DL. 427500 C
LL 282000 C
709500 C SR=50.
Same as U5-U4
DL. 427500 T
LL 282000 T
709500 T SR=41.60 mm.
DL. 383500 C
LL 251800 C
635300 C SR=44.80
DL. 383500 T
LL 251800 T
635300 T
DL. 296000 C
LL 194500 C
490500 C SR=34.50
DL. 296000 T
LL 194500 T
490500 T SR=28.8 mm
DL. 167500 T
LL 127500 T
295000 T SR=17.30 mm
DL. 84750 T
LL 92600 T
177350 T SR=10.40 mm
DL. 60.000 T
LL 70.000 T
130000 T SR=7.630 mm
DL. 246500 T
LL 167000 T
413500 SR=24.20 mm
DL. 166800 T
LL 111200 T
277600 T SR=16.30 mm
DL. 166400 T
LL 111200 T
277600 T SR=16.30 mm
DL. 127800 C
LL 108500 C
236300 C SR=23.80
DL. 44900 C
LL 78900 C
128000 C SR=12.90
DL. 27000 C
LL 19000 C
46000 C SR=4.70
DL. 27000 T
LL 19000 T
46000 T SR=4.70

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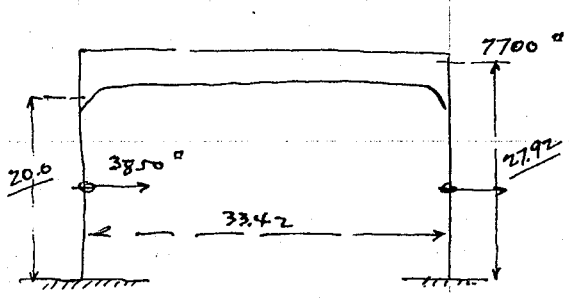
CALCULATIONS FOR

Design of #1 D Bridge for City of Tokyo.

End post
assumed section

1 Cov Pl.	26 × 7/16	=	11.38		=	641	
2 webs	17 × 1/2	=	17.00	× 8.64²	=	1270	
4 L³	4×4×3/8	=	11.44	× 10.01²	+ 17.6 =	1162	$r = \sqrt{\frac{3629}{46.57}} = 8.8$
2 Pls.	9 × 3/8	=	6.75	× 9.08²	=	556	
			46.570			3629	



wind load 4 × 135# × 14.22 = 7700#
Reaction = $7700 \times \frac{27.92}{33.42} = 6430^{\#}$

Moment = 3850 × 10 = 38500 in#
fibre stress = $\frac{38500 \times 12 \times 13}{3629} = 1665$ #/□"

allowable unit stress = 14200 × 1.25 = 17750 #
1665
16085 #/□"

Direct stress = D+L. 544500
W 6430
550930 ÷ 16085 = 34.30□" req'd.

For direct stress only D+L 544500 ÷ 14200 = 38.30□" gross.
Assumed section is ample.

Vertical U_2-L_2.
section

					M of I		
4 L³	6×4×3/8	=	14.44	× 2.13² + 54 =	119.3	119.3	$r = \sqrt{\frac{2413}{27.82}} = 3.21$
2 Pls.	13 × 3/8	=	4.88		137.2	1330	
			19.32		256.5	241.3	$r = \sqrt{\frac{2565}{24.19}} = 3.26$ 3.21
					241.3	2413	
						27.82	

allowable stress = $21300\left(1 - 0.0055 \times \frac{23.3 \times 12}{3.26\ 3.21}\right) = 11260$ #/□"
11100

SR = 260300# ÷ 11260 = 23.10□" OK.
23.40

Vertical U_3-L_3.
4 L² 6×4×3/8 = 14.44 × r = 2.87 $P = 21300\left(1 - 0.0055 \cdot \frac{23.1 \times 12}{2.87}\right) = 9950$ #/□"

SR = 128800# ÷ 9950 = 12.940□" gross.

Verticals U_4-L_4 SR = 61700 ÷ 9950 = 6.200□"
use 4 L 6×4×3/8 = 14.44 0□" gross.

Diagonal U_4-L_4' unsupported length = 13.5 × 12 = 162".
section assumed 4 L 5×3½×3/8 = 12.20 r = 2.40
allowable P = $21300\left(1 - 0.0055 \times \frac{162}{2.40}\right) = 13400$ #/□"

stress 30900 C
15450
46350 ÷ 13400 = 3.450□" gross. OK

Load on pin.
Dead Load Floor 3119 × 62 + 45 = 209,000
1746 × 65.5 psf = 114500
323,500# call this 330.000#

Structural steel 1106 +
640
1746

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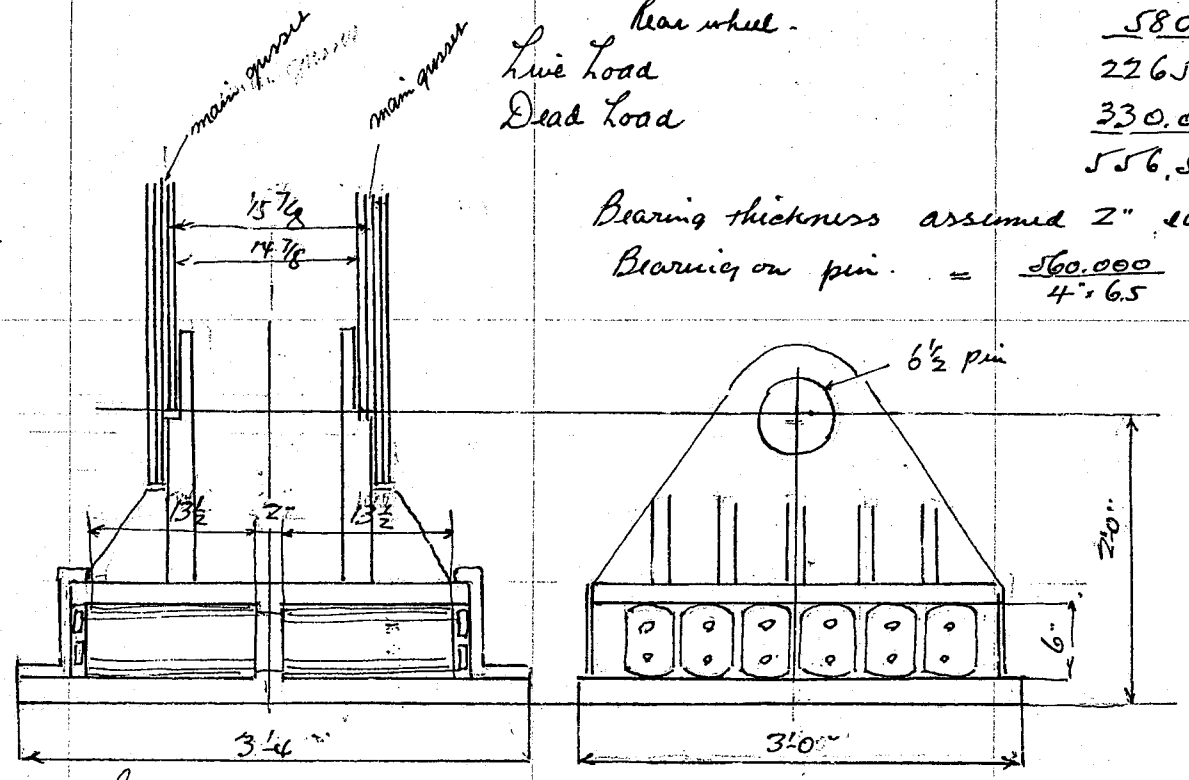
CALCULATIONS FOR

Design of 41 D Bridge for City of Tokyo.

Live Load. 38500 × 9 = 346000
41800 × 28 = 1.170.000
1516.000 ÷ 9 = 168500
58000
226500 #

Rear wheel.
Live Load 226500 #
Dead Load 330.000
556.500 # say 560.000 #

Bearing thickness assumed 2" each rib. 6½" pin assumed.
Bearing on pin. = 560.000 / 4" × 6.5 = 21600 #/0"



Bending moment of pin.
say for max 280.000 × 1.5 = 420.000 "#

fibre stress = 420.000 × 3.25 / 0.049 × 6.25⁴ = 18200 #/0"

Load on Bearing. 560.000 # assumed.
Roller assumed 6" high. 6 × 610 = 3660 " per lin inch
length of roller required = 560.000 ÷ 3660 = 153" net.
Using two rollers @ 75.5" net use 27" net 2" recess at centre. 29" over all.

Size of base 40" × 36" = 1440 □"
Unit bearing = 560.000 ÷ 1440 = 389 #/0" on masonry.

Approximate weight of truss.

End post.	46.57.	@ 3.4	×	27.93	= 4430
U_1-U_2	39.82	@ "	×	14.25	= 1930
U_2-U_3	46.57	"	×	14.25	= 2260
U_3-C	52.38	"	×	21.37	= 3800
L_0-L_2	24.00	"	×	28.44	= 2320
L_2-L_3	36.50	"	×	14.22	= 1770
L_3-C	52.00	"	×	21.33	= 3760
U_1-L_2	30.38	"	×	27.30	= 2820
U_2-L_3	24.19	"	×	27.13	= 2230
U_3-L_4	14.44	"	×	27.05	= 1330
U_3-L_4'	12.20	"	×	27.00	= 1130
U_1-L_1	12.20	"	×	23.60	= 980
U_2-L_2	24.19	"	×	23.30	= 1920
U_3-L_3	14.44	"	×	23.10	= 1140
U_4-L_4	14.44	"	×	23.00	= 1130
				32950 × 2 =	65900"
		Details 38%.			25000
					90900

For one span = 2 × 90900 = 181800 #

Summary for structural steel and casting in one span

Stringers	660 × 131.0	= 86400
Floor beams	10 @ 7900	= 79000
Bottom Bracings		25110
Top Bracings sways & Portals.		61460
trusses		181800
Shoes and Expansion Joints say		15000
		448770 ÷ 2240 = 200.0 tons.

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CALCULATIONS FOR

Design of 4 H D Bridge for City of Tokyo.

Deflection of truss.
For dead Load.

$E = 28.000.000$ #/□"

Member	L	L	Dl. stress S	Area A	$\frac{SL}{EA}$	u.	Deflection
L_0-U_1	335"		327000#	46.57 □"	0.0840	0.590	0.0495"
U_1-U_2	171		296000	39.82	0.0455	0.610	0.0278
U_2-U_3	171		383500	46.57	0.0504	0.924	0.0465
U_3-U_4	171		427500	52.38	0.0500	1.238	0.0618
U_4 - Center	85.5		427500	52.38	0.0250	1.238	0.0310
L_0-L_1	171		166400	24.00	0.0423	0.302	0.0302 (128)
L_1-L_2	171		165800	24.00	0.0423	0.302	0.0446 (128)
L_2-L_3	171		296000	36.50	0.0495	0.610	0.0302
L_3-L_4	171		383500	52.00	0.0450	0.924	0.0416
L_4 - Center	85.5		427500	52.00	0.0252	1.238	0.0312
U_1-L_2	328"		246500	31.50	0.0916	0.594	0.0545
U_2-L_3	325		167500	23.82	0.0817	0.598	0.0488
U_3-L_4	324		84750	14.44	0.0678	0.595	0.0404
U_4-L_4'							
U_2-L_2	280		151800	23.82	0.0638	0.510	0.0326
U_3-L_3	277		49900	14.44	0.0342	0.506	0.0173
U_4-L_4							0.5388

for one span 0.5388 × 2 = 1.0776"

Structural Steel = 1748 # per lin ft.
Slab & pavement 3119
4867 " " "

Full live load on span (without motor trucks)
Unif. load on roadway ½ width 14.76 × 120 = 1770
" " " sidewalk 9.84 × 100 = 984
2754 # per lin ft.

Deflection at center
Structural Steel only 1.0776 × 1748/4867 = .388"
Slab + pavement " × 3119/4867 = .690
1.078"
Live load 1.0776 × 2754/4867 = .613
1.691" = 1 11/16"

Taking allowance for motor truck loading Use Camber 2" at center.

Lengthening upper chord member 1/16" allowance at each joint is taken in addition to 2" Camber.

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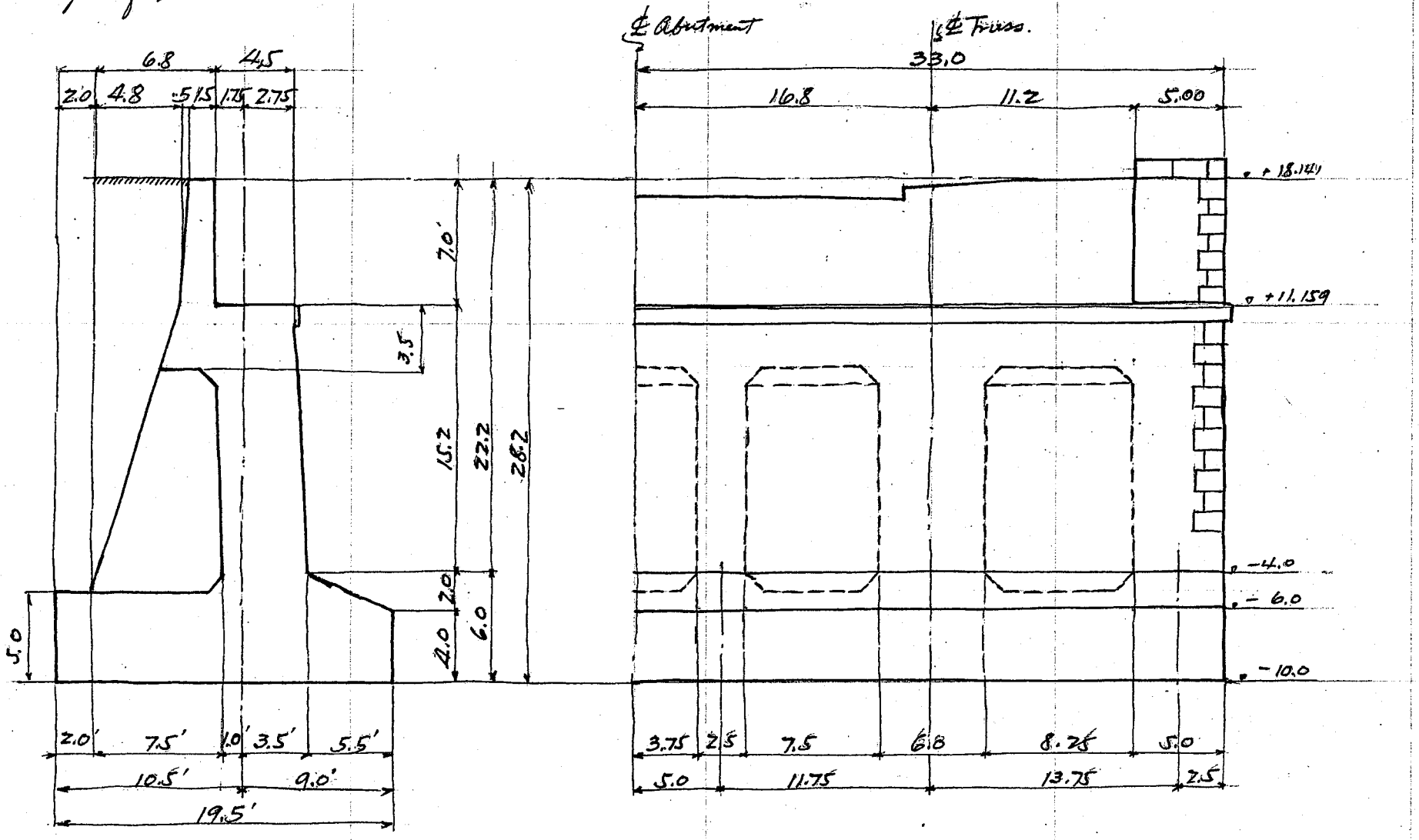
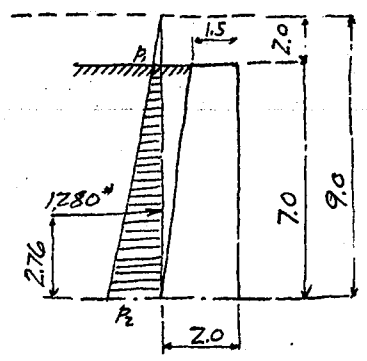
CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.
Design of Abutment.



Parapet wall. under Carriage way.
Case 1. Stability as Normal Case. 2 ft Surcharge due to live load assumed.

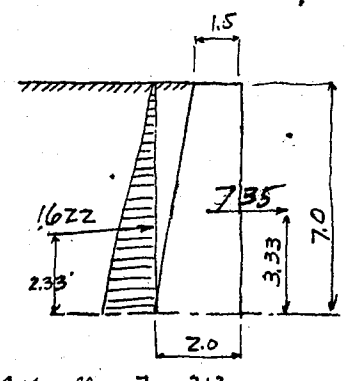


C.g. of pressure diagram.
$x = \frac{d}{3} \times \frac{b+2b_1}{b+b_1} = \frac{7}{3} \times \frac{300+134}{367}$
= 2.76' from bottom.

Unit earth pressure $p_1 = 100 \times \frac{2}{3} = 67$ #/'
$p_2 = 100 \times \frac{9}{3} = 300$ "
367 ÷ 2 = 183 #/'

Total earth pressure on wall = 183 × 7.0 = 1,280 #/lin ft of wall.
Moment at bottom of wall = 1,280 × 2.76 = 3,530 '#
Effective depth say 22"
Steel required for moment = $\frac{3530 \times 12}{17067 \times \frac{7}{8} \times 22} = 0.129$ "per ft. of wall.
Use ½"ϕ — 12" spacing = 0.196" per ft. of wall.
Steel ratio $p = \frac{0.196}{12 \times 22} = .00075$
$k = .145$, $j = .95$
$f_s = \frac{3530 \times 12}{.196 \times .95 \times 22} = 10350$ #/" Ok.
$f_c = \frac{10350 \times .145}{15(1-.145)} = 117$ #/" Ok
Unit shear $v = \frac{1280}{12 \times .95 \times 22} = 5.1$ #/" Ok
Unit bond $u = \frac{1280}{1.571 \times .95 \times 22} = 39$ #/" Ok.

Case 2. Stability during earthquake. Earthquake coefficient $0.4 \times \frac{3}{4} = 0.3$.
Earth pressure during earth $.662 \times \frac{100 \times 7^2}{2} = 1,622$ # per ft. of wall.



C.g. of wall = $\frac{7}{3} \times \frac{3+2}{3.5} = 3.33'$

Weight of wall = $\frac{1.5+2.0}{2} \times 7.0 \times 150 = 1837$ #
Seismic force = 1,837 × .4 = 735 #
Moment. Earth press. 1622 × 2.33 = 3,780 '#
wall 735 × 3.33 = 2,450
2,357 # 6,230 '#

Allowable stress = $f_s = 17067 \times 1.8 = 30,700$ #/", $f_c = 640 \times 1.8 = 1,150$ #/"
$v = 57 \times 1.8 = 103$ #/" , $u = 85 \times 1.8 = 128$ ".

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CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.

$f_s = \frac{6230 \times 12}{.196 \times .95 \times 22} = 18{,}250$ #/□" Ok.

$f_c = \frac{18250 \times .145}{15(1-.145)} = 206$ #/□" OK

$v = \frac{2357}{12 \times .95 \times 22} = 9.4$ " OK

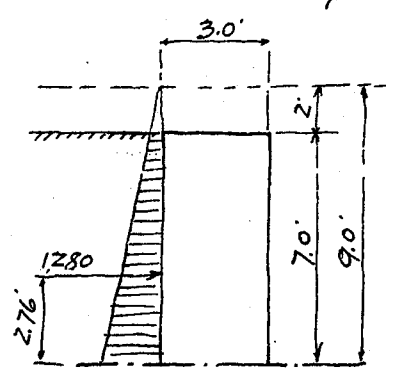
$u = \frac{2357}{1.571 \times .95 \times 22} = 72$ " OK.

Use the same reinforcement on both sides to take care of earthquake stress on both directions.

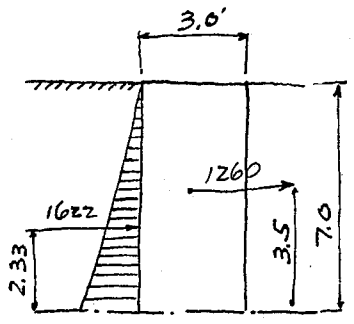
Parapet wall under Sidewalk.

Case 1. Stability at Normal State. 2 ft. surcharge assumed for live load.

Earth pressure and its moment on wall same as for parapet wall under carriage way. Use the same reinforcements as before. see ☞ last page.



Case 2. Stability during Earthquake.



Earth pressure and its moment on wall same as before.

Earth pressure 1622 #.

weight of wall = 3 × 7 × 150 = 3150 # Seismic force 3150 × .4 = 1,260 #

moment at bottom of wall.

One to earth pressure $1622 \times 2.33 = 3{,}780$

" " Seismic force $\frac{1260 \times 3.5}{2{,}882\ \#} = \frac{4{,}410}{8{,}190'\#}$

Steel ratio $p = \frac{0.196}{12 \times 34} = .00048$

$k = .13$, $j = .96$

$f_s = \frac{8190 \times 12}{.196 \times .96 \times 34} = 15{,}380$ #/□" OK

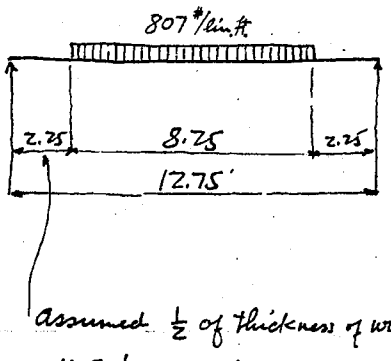
$f_c = \frac{15380 \times .13}{15(1-.13)} = 153$ #/□" OK

$v = \frac{2882}{12 \times .96 \times 34} = 7.4$ " OK

$u = \frac{2882}{1.571 \times .96 \times 34} = 56.$ " OK.

Curtain wall. under sidewalk. clear span 8.25' Effective span say 12.75' thickness 4.5' eff. depth = 52"

Case 1. Stresses are at normal state. 2 ft surcharge due to live load assumed.



Height of earthfill at bottom of wall. = 22.2 + 2.0 = 24.2'

Earth pressure at bottom of wall = $\frac{100 \times 24.2}{3} = 807$ #/□'

Bending moment per ft strip of wall = $\frac{807 \times 4.125 \times 12.75}{2} - \frac{807 \times 4.125^2}{2} = 14{,}330'\#$

for continuity of slab $M_L = 14330 \times .8 = 11{,}470'\#$

max end shear = $807 \times 4.125 = 3{,}330$ #

Steel area required = $\frac{11{,}470 \times 12}{17067 \times \frac{7}{8} \times 52} = 0.177$ □" per hor. ft strip.

use 3/4"ϕ bars at 1'-6" spacing = 0.295 □"

Shearing stress $v = \frac{3330}{12 \times \frac{7}{8} \times 52} = 6.1$ #/□" OK

bond " $u = \frac{3330}{2.356 \div 1.5 \times \frac{7}{8} \times 52} = 47$ #/□" OK.

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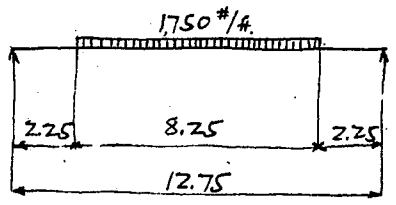
CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.
Case 2. Stresses during Earthquake



steel ratio $p = \frac{.295}{12\times52} = .00047$

$K = .13$, $j = .96$

Height of earthfill = 22.2'
Earthpressure at bottom of curtain wall = .662 × 100 × 22.2 = 1,480 #/ft
weight of curtain wall per hor. ft strip = 4.5 × 150 = 675 #
Seismic force 675 × .4 = 270
1,750 #/lin ft of span.

Bending moment = $1{,}750 \times 4.125 \times 6.375 = 46{,}000$
$1{,}750 \times \frac{4.125^2}{2} = -\ 14{,}900$
$31{,}100$ '#

For continuity of wall $m = 31{,}100 \times .8 = 24{,}900$ '#
max end shear = 1750 × 4.125 = 7,220 #

$f_s = \frac{24900\times12}{.295\times.96\times52} = 20{,}300$ #/□" OK.

$f_c = \frac{20300\times.13}{15(1-.13)} = 153$ #/□" OK

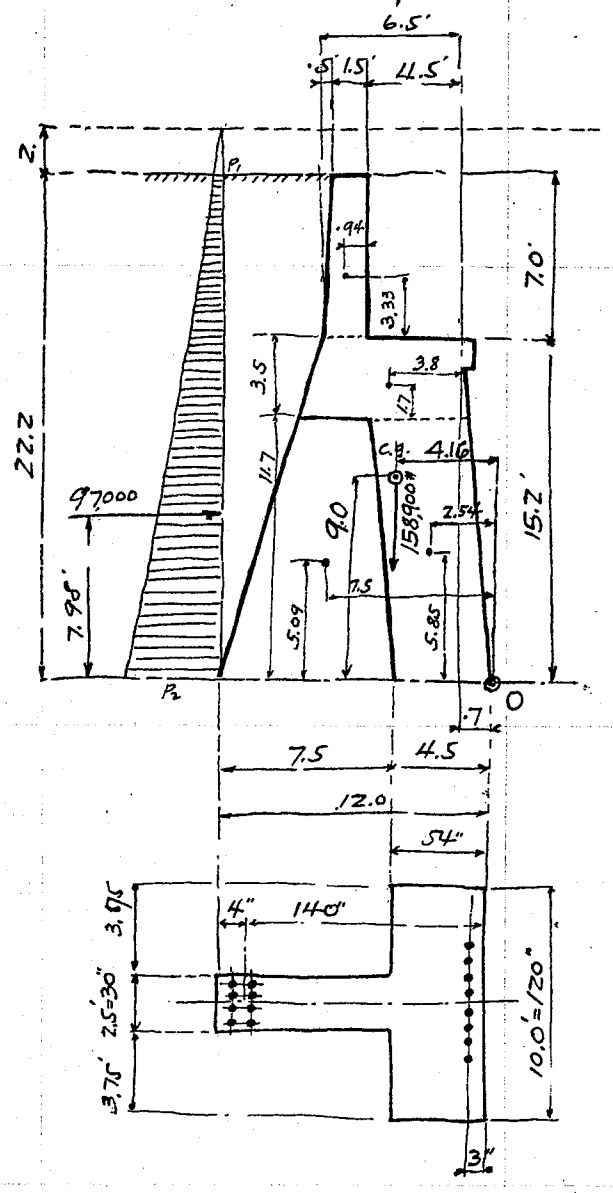
$v = \frac{7220}{12\times.96\times52} = 12$ #/□" OK

$u = \frac{7220}{\frac{2.356}{1.5}\times.96\times52} = 92$ #/□" OK

Assumed section is ample.

Note: - Use same details for all panels and same reinforcements on both sides of wall.

Counterfort walls at Center. Thickness of counterfort 2.5' assumed.
Case 1. Stability at normal state: spacing of counterfort wall = 10.0'



Center of gravity of shaft. Taking moment about O.

Name	Volume	Weight	vert. lev. arm	moment	hor. lev. arm	moment
Parapet wall	1.75 × 7.0 × 10 @ 150 =	18,400 ×	18.53	341,000	6.14	113,000
Hor. Beam	3.5 × 7.2 × 10 @ 150 =	37,800 ×	13.40	506,000	4.50	170,200
Curtain wall	4.5 × 11.7 × 10 @ 150 =	79,000 ×	5.85	462,000	2.54	200,700
Counterfort wall	5.4 × 11.7 × 2.5 @ 150 =	23,700 ×	5.09	121,000	7.50	177,700
		158,900#		1,430,000'#		661,600'#

Center of gravity of shaft vert. lever arm = 1,430,000 ÷ 158,900 = 9.00' from O.
hor. " " = 661,600 ÷ 158,900 = 4.16 " O

Earthpressure $p_1 = 100 \times 2 \div 3 = 67$ #/□', $p_2 = 100 \times 24.2 \div 3 = 807$ #/□'.
Total earth pressure = $\frac{67+807}{2} \times 22.2 \times 10 = 97{,}000$ #
Moment at bottom = 97,000 × 7.98 = 774,000'#
neglecting the effect of dead load of the structure.
Steel area req'd. = $\frac{774{,}000\times12}{17067\times\frac{7}{8}\times140} = 4.45$ □"
Use 8-1"ϕ bars = 6.283 □" on both sides.
Design as a double reinforced T-Beam.
steel ratio = $p = p' = \frac{6.283}{120\times140} = 0.000374$

$\Delta = \frac{t}{d} = \frac{54}{140} = 0.386$, $\frac{d'}{d} = \frac{3}{140} = .0215$

$K = \frac{p+p'(\frac{d'}{d})+\frac{\Delta^2}{2n}}{p+p'+\frac{\Delta}{n}} = \frac{p(1+\frac{d'}{d})+\frac{\Delta^2}{2n}}{2p+\frac{\Delta}{n}} = \frac{0.000374\times1.0215+\frac{.386^2}{30}}{0.000748+\frac{.386}{15}} = 0.202$

$Kd = 140\times.202 = 28.3$" Neutral axis in the flange.
Design as a double reinforced rectangular beam.

$K = \sqrt{2n(p+p'\frac{d'}{d})+n^2(p+p')^2} - n(p+p') = \sqrt{2pn(1+\frac{d'}{d})+4n^2p^2} - 2pn$
$= \sqrt{30\times.000374\times1.0215+4\times225\times.000374^2} - 30\times.000374 = .097$

$j = \frac{K^2(1-\frac{K}{3})+2p'n(K-\frac{d'}{d})(1-\frac{d'}{d})}{K^2+2p'n(K-\frac{d'}{d})} = \frac{.097^2(1-\frac{.097}{3})+2\times.000374\times15(.097-.0215)(1-.0215)}{.097^2+2\times.000374\times15(.097-.0215)}$
$= 0.97$

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CALCULATIONS FOR

Design of 410 Bridge for City of Tokyo.

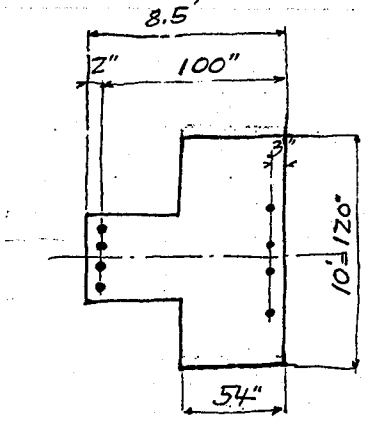
$f_s = \frac{774000 \times 12}{6.283 \times .97 \times 140} = 10{,}900$ #/□" OK

$f_c = \frac{10900 \times .097}{15(1-.097)} = 71$ #/□" OK

$v = \frac{97.000}{30 \times .97 \times 140} = 24$ #/□" OK

$u = \frac{97.000}{3.142 \times 8 \times .97 \times 140} = 28$ #/□" OK.

Section at 9 ft above bottom of wall.



Earth pressure $p_1 = 67$ #/□" , $p_2 = \frac{100 \times 15.2}{3} = 507$ #/□"

Total earth pressure = $\frac{67+507}{2} \times 13.2 = 3{,}790$ #/□" $\times 10 = 37{,}900$ #

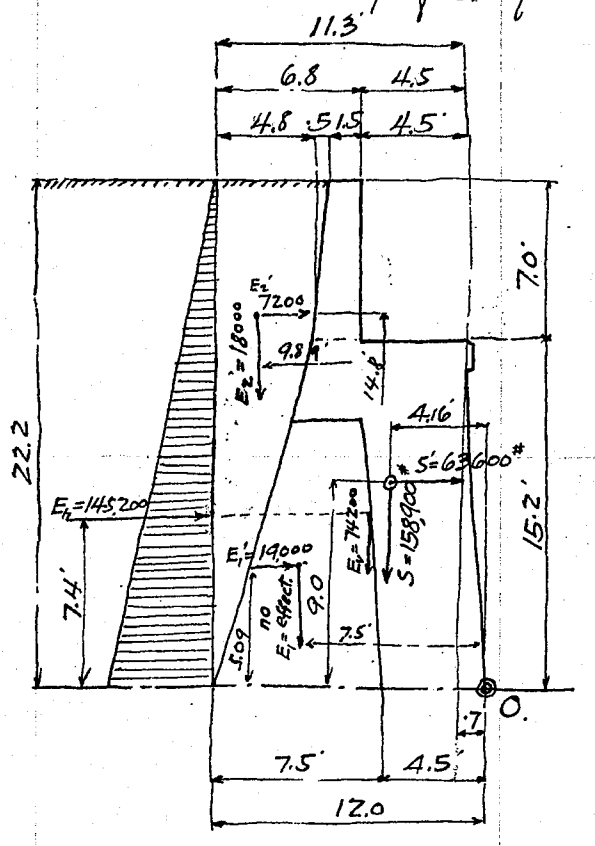
point of application of earth pressure $x = \frac{13.2}{3} \times \frac{134+507}{67+507} = 4.91'$ from bottom.

Moment at the section = $37{,}900 \times 4.91 = 186{,}000$'#

Steel area req'd. = $\frac{186000 \times 12}{17067 \times \frac{7}{8} \times 100} = 1.50$ □"

Use 4 — 1"ϕ bars = 3.14 □".

Case 2. Stability of shaft during Earthquake at bottom section. (Reffer to the last page.)



Height of earth fill = 22.2'

Earth pressure $E = \frac{.662 \times 100 \times 22.2^2}{2} \times 10 = 163{,}200$ # Slope 27° 05' to hor. line.

Hor. component $E_h = .89 \times 163{,}200 = 145{,}200$ #

Vert. " $E_v = .455 \times 163{,}200 = 74{,}200$

Seismic force due to weight of Shaft = $158{,}900 \times .4 = 63{,}600$ #

Weight of earth between Counterfort

~~5.#~~ × 5.4 × 11.7 × 7.5 @ 100 = 47,400# Seismic force × .4 = 19,000#

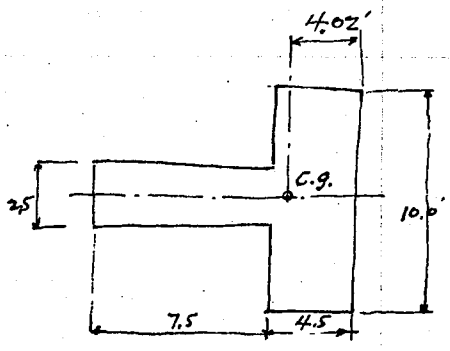
Earth on Counterfort (approx)

6.5 × $\frac{22.2}{2}$ × 2.5 @ 100 = 18,000# × .4 = 57,200#

Taking moment about O.

loads		Vert forces	hor. forces	lev. arm	moment
Shaft	S	158,900		− 4.16	− 657,000
"	S'		63,600	9.00	572,000
Earth on counterfort	E_2	18,000		− 9.89	− 178,000
"	E_2'		7,200	14.80	107,000
Earth between Counterfort	E_1	——		——	——
"	E_1'		19,000	5.09	96,800
Earth pressure	E_h		145,200	7.40	1,075,000
"	E_V	74,200		− 4.80	−356,000
		ΣV=250,200	ΣH=235,000		ΣM= 659,800'#

Distance of resultant from O. = $\frac{659800}{250{,}200} = 2.64$



Center of gravity of the section

10 × 4.5 = 45.0 × 2.25 = 101.3

2.5 × 7.5 = 18.75 × 8.25 = 154.7

63.75 256.0

256 ÷ 63.75 = 4.02'

Eccentricity = 2.64 + 4.02 = 6.66'

Bending moment at bottom section = 250,200 × 6.66 = 1,666,000'#

$f_s = \frac{1666000 \times 12}{6.283 \times .97 \times 140} = 23{,}420$ #/□" OK.

$f_c = \frac{23420 \times .097}{15(1-.097)} = 168$ #/□" OK

Direct comp. $f_c = \frac{250.200}{63.75 \times 144} = \frac{27}{195}$ #/□" OK.

$v = \frac{235.000}{30 \times .97 \times 140} = 58$ #/□" ok

$u = \frac{235.000}{3.142 \times 8 \times .97 \times 140} = 69$ #/□" OK.

Assumed section is ample.

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CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.

Case 3. Stability of shaft during Earthquake. Seismic forces toward bank side.

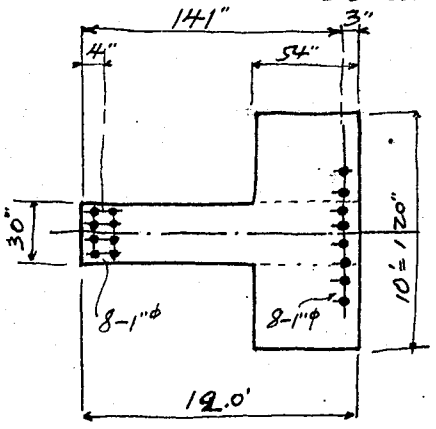
Refer to the sketch shown in case 2. Taking moment about O.

loads		vert. forces	hor. forces.	lev. arm.	moment.
Shaft	5	158,900		4.16	657,000
"	5'		63,600	9.00	572,000
Earth on counterfort	E_2	18,000		9.89	178,000
		176,900	63,600		1,407,000

Distance of resultant forces from O. = 1,407,000 ÷ 176,900 = 7.95'

Eccentricity = 7.95 − 4.02 = 3.93'

Bending moment at bottom section = 176,900 × 3.93 = 695,000'#



Beam 30" wide × 12.0" deep. with double reinforcements.

steel ratio = $p = p' = \frac{6.283}{141 \times 30} = .00149$, $\frac{d'}{d} = \frac{4}{141} = .028$

$k = \sqrt{2np(1+\frac{d'}{d}) + 4n^2p^2} - 2pn = \sqrt{30 \times .00149(1+.028) + 4 \times 225 \times .00149^2} - 30 \times .00149$

$= .174$

$j = \frac{k^2(1-\frac{k}{3}) + 2p'n(k-\frac{d'}{d})(1-\frac{d'}{d})}{k^2 + 2p'n(k-\frac{d'}{d})} = \frac{.174^2(1-.058) + 30 \times .00149(.174-.028)(1-.058)}{.174^2 + 30 \times .00149(.174-.028)} = .941$

$f_s = \frac{695,000 \times 12}{6.283 \times .941 \times 141} = 10,000$ #/□" OK

$f_c = \frac{10000 \times .174}{15(1-.174)} = 141$ #/□" OK

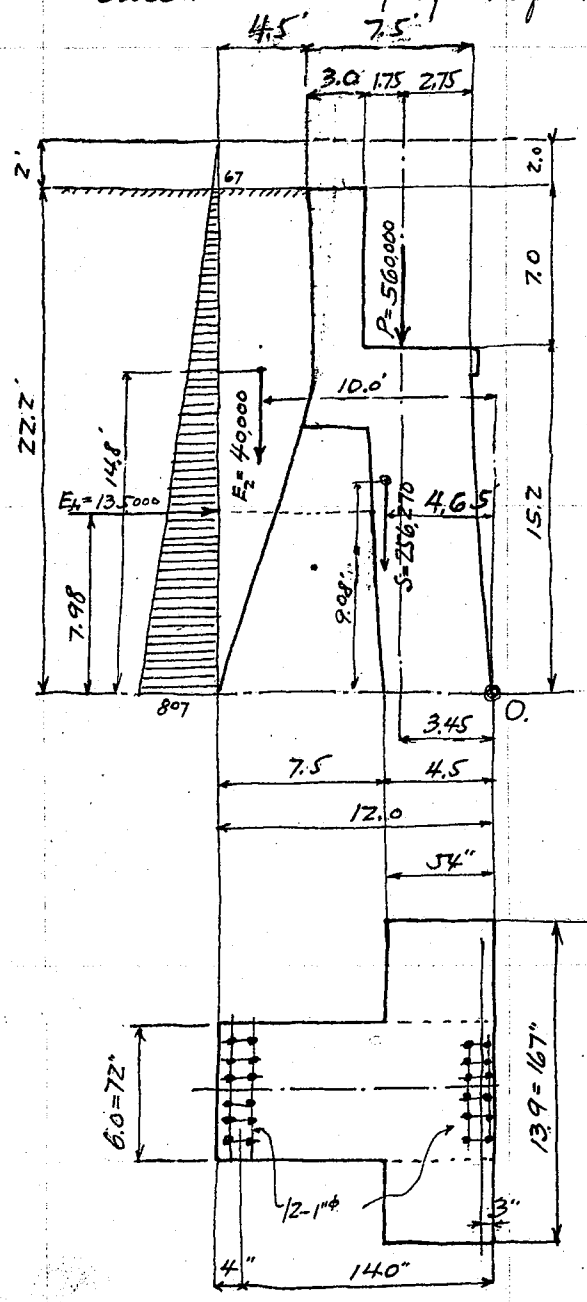
$v = \frac{63600}{30 \times .941 \times 141} = 16$ #/□" OK

$u = \frac{63600}{3.142 \times 8 \times .941 \times 141} = 19$ #/□" ok.

Assumed section is ample.

Counterfort walls under Truss shoes.

Case 1. Stability of shaft under normal state. Thickness of counterfort wall 6', width of flange 13.9'



Superimposed loads on shoe. See page 25.

Live load 230,000 #

Dead Load 330,000 Seismic force 330,000 × .4 = 132,000 #.

560,000 # on one shoe.

Design as a column subjected to combined stress of Direct Compression & bending.

Center of gravity of shaft. Taking moments about O.

name	volume	weight	vert. lev. arm	moment	hor. lev. arm	moment
Parapet wall	1.75×7.0×4.8 @150 =	8,820#	18.53'	= 163,500'#	6.14	61,400
"	3.0×7.0×9.1 @150 =	28,650	18.70	= 532,500	6.70	192,000
Hor. beam	3.5×7.2×13.9 @150 =	52300	13.40	= 701,000	4.50	235,500
Curtain wall	4.5×11.7×13.9 @150 =	109,700	5.85	= 641,000	2.54	278,400
Counterfort wall	5.4×11.7×6.0 @150 =	56800	5.09	= 289,000	7.50	426000
		256,270#		2,327,000'#		1,193,300'#

Center of gravity of shaft vert. lev. arm = 2,327,000 ÷ 256,270 = 9.08' from O.

hor. " " = 1,193,300 ÷ 256,270 = 4.65' " O

Earth on counterfort wall = $\frac{6. \times 22.2}{2} \times 6.0 @ 100 =$ 40,000#

Earth pressure on wall = $\frac{67+80}{2} \times 22.2 \times 13.9 =$ 135,000#

Moment about O.

Super-imposed load	560,000#	×	3.45'	= −1,933,000'#
wt. of Shaft	256,270	×	4.65	= −1,191,000
" Earth on counterfort	40,000	×	10.00	= − 400,000
Earth pressure	135,000#	×	7.98	= 1,078,000
	856,270#			− 2,446,000'#

Distance of resultant force from O = −2,446,000 ÷ 856,270 = 2.86' from O.

Eccentricity = 6.0 − 2.86 = 3.14'

Bending moment at bottom section = 856,270 × 3.14 = 2,690,000'#

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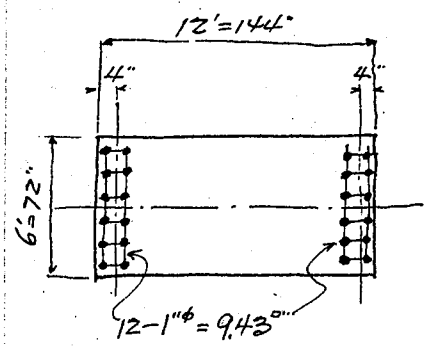
CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.



Try 12- 1"ϕ bars = 9.43" on both sides

Steel ratio $p=p'=\frac{.7854\times12}{144\times72}=.00091$ $P_0=p+p'=0.00182$

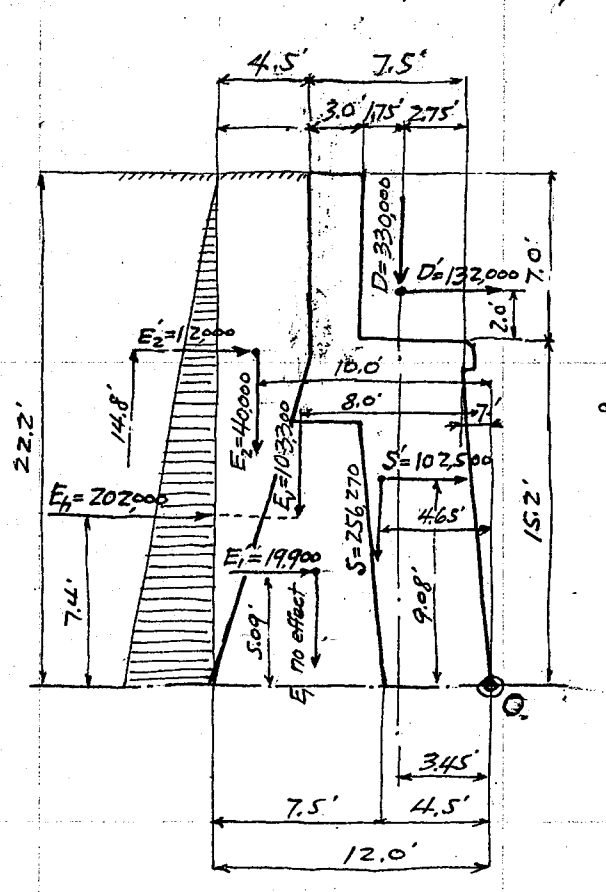
ratio $\varepsilon/h=\frac{3.14'}{12}=.262$, $d'/h=4/144=.028$

$K=.761$ $L=.0988$

$f_c=\frac{M}{Lbh^2}=\frac{2,690,000\times12}{.0988\times72\times144^2}=219$ #/□" ok

$f_s=nf_c\left(\frac{d}{Kh}-1\right)=15\times219\left(\frac{140}{.761\times144}-1\right)=920$ #/□" ok

Case 2. Stability during Earthquake (Seismic forces acting toward river side)



Weight of earth between counterfort $=5.4\times11.7\times7.88@100=49,800$ #

Seismic force $=49800\times.4=19,900$ #

Earth pressure $=\frac{.662\times100\times22.2^2}{2}\times13.9=227,000$ # Slope 27°05' to hor. line.

Hor. Component $E_h=227,000\times.89=202,000$ #

Vert. " $E_v=227,000\times.455=103,300$ #

Taking moment about O.

Loads		Vert. forces	hor. forces	lever arm	moment
Superimposed D.L.	D	330,000		−3.45	− 1,138,000
" "	D'		132,000	17.20	2,270,000
Shaft	S	256,270		−4.65	− 1,192,000
"	S'		102,500	9.08	930,000
Earth on Counterfort	E_2	40,000		− 10.00	− 400,000
" "	E_2'		12,000	14.80	178,000
Earth between	E_1	no effect			
" "	E_1'		19,900	5.09	101,000
Earth pressure	E_h		202,000	7.40	1,495,000
"	E_v	103,300		−8.00 say	− 827,000
		$E_v=729,570$ #	$\Sigma H=468,400$		$\Sigma M=1,417,000$ '#

Distance of Resultant force from O $=\frac{1,417,000}{729,570}=1.94'$ outside of O.

Eccentricity $=6.0+1.94=7.94'$

Bending moment at the bottom section $=729,570\times7.94=5,795,000$ '#

$P_0=0.00182$, $\varepsilon/h=7.94\div12=.662$, $d'/h=.028$

$K=.30$, $L=.081$

$f_c=\frac{5,795,000\times12}{.081\times72\times144^2}=576$ #/□" ok

$f_s=15\times576\left(\frac{140}{.30\times144}-1\right)=19,400$ #/□" ok

$v=\frac{468,400}{72\times\frac{7}{8}\times140}=53$ #/□" ok

$u=\frac{468,400}{3.142\times12\times\frac{7}{8}\times140}=101$ #/□" < 153 ok.

Assumed section is ample.

Case 3. Stability of Shaft during Earthquake, Seismic forces acting toward bankside.
In this case, Overturning moment at the bottom section is clearly smaller than for Case 2, it is not necessary to investigate.

Counterfort wall at end.
use 10—1"ϕ bars on both sides each.

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CALCULATIONS FOR

Design of H10 Bridge for City of Tokyo.
Design of Foundation of Abutment.
Center of gravity of shaft for abutment as a whole.
Center of gravity for End counterfort.

Name	Volume	Weight	Vert. lev. arm	moment	hor. lev. arm	moment
Parapet wall	3.0×7.0×4.13@150 =	13,000	18.70	243000	6.70	87,200
Horizontal beam	3.5×7.2×9.13@150 =	34500	13.40	462,000	4.50	155,300
Curtain wall.	4.5×11.7×9.13@150 =	72,100	5.85	421,500	2.54	183,000
Counterfort wall	5.4×11.7×5.00@150 =	47,400	5.09	241,500	7.50	355500
Column under light pedestal.	5.0×5.0×8.00@150 =	30,000	19.20	576,000	7.20	216,000
do.	2.5×5.0×8÷2@150 =	7.300	12.53	94000	8.17	61,300
		204,500#		2,038,000'#		1,058,300'#

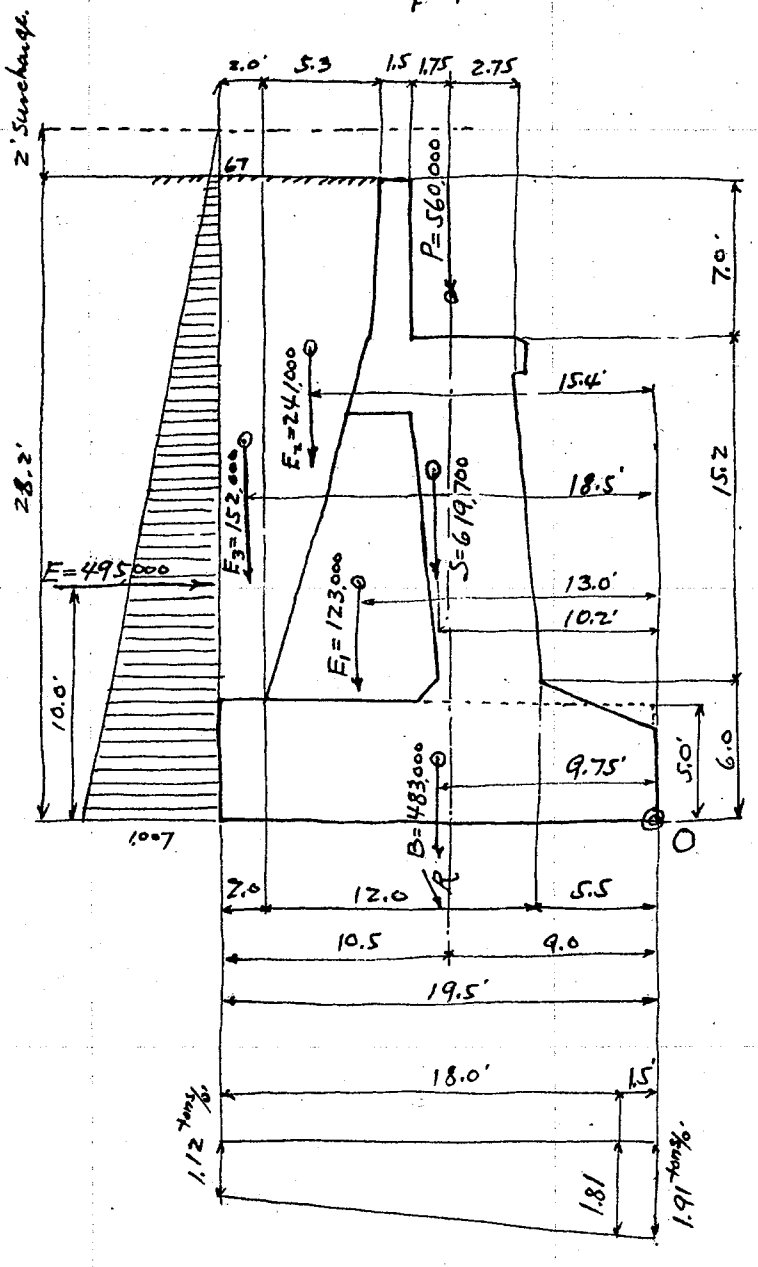
Center of gravity of End counterfort wall.
Vert. lever Arm = 2,038,000 ÷ 204,500 = 9.98' from O
Hor. " " = 1,058,300 ÷ 204,500 = 5.18 " "

Center of gravity of shaft for one half of abutment

name	weight	Vert. lev. arm	moment	hor. lev. arm	moment
Shaft at center	158,900# ×	9.00' =	1,430,000	4.16	661,600'#
" under shoe	256,300 ×	9.08 =	2,327,000	4.65	1,193,300
" at end	204500 ×	9.98 =	2038000	5.18	1,058,300
Total wt. =	619,700#		5,795,000		2,913,200

Center of gravity of shaft as a whole.
Vert. lev. arm = 5,795000 ÷ 619,700 = 9.36' above O
Hor. lev. arm = 2,913,200 ÷ 619,700 = 4.70' from O

Stability of Abutment.
Case 1. Stability of abutment at Normal stale. Let us figure for one half of Abutment.



Weight of earth between counterfortwall E_1 = 5.4×11.7×19.5@100 = 123,000#
" " behind of shaft E_2 = 6.3×23.2/2×33@100 = 241,000
" " " " E_3 2×23.2×33@100 = 152,000

Earth pressure P_1 = 67#/□' P_2 = 100×30.2÷3 = 1,007#/□'
" $E = \frac{67+1,007}{2} \times 28.2 \times 33 = 495,000$ #
point of appl. of $E = \frac{28.2}{3} \times \frac{134+1007}{1074} = 10.0'$ above bottom.

Weight of base B = 5×19.5×33@150 = 483,000#
Taking moment about O.

Load		vert. forces	hor. forces	lever arm	moment
Super imp. load	P	560,000		- 9.0	- 5,040,000
Shaft	S	619,700		- 10.2	- 6,310,000
Earth	E_1	123,000		- 13.0	- 1,600,000
"	E_2	241,000		- 15.4	- 3,710,000
"	E_3	152,000		- 18.5	- 2,810,000
Earth pressure	E		495,000	+ 10.0	+ 4950,000
Base	B	483,000		- 9.75	- 4,700,000
		ΣV = 2,178,700	ΣH = 495000	ΣM =	- 19, 220,000'#

Distance of Resultant force from O = $\frac{-19220000}{2,178,700}$ = 8.90' from O.

Eccentricity = 9.75 - 8.90 = 0.85'
Resultant force within middle third.
max. toe pressure = $\frac{2,178,700}{19.5 \times 33}\left(1 \pm \frac{6 \times 0.85}{19.5}\right)$ = 4,280 #/□' = 1.91 tons/□' C
or = 2,500 " = 1.12 " C.

Use foundation piles 0.8'' × 36.0' at the intervals of 3.0' in both directions
max. load on one pile 1.81 × 3 × 2.75 = 14.9 tons.
Friction on pile surface = $\frac{14.9 \times 2240}{1.0 \times 3.142 \times 34}$ = 310 #/□'

Base Revised to 20.5' See next page.
max load on one pile = 12.65 tons

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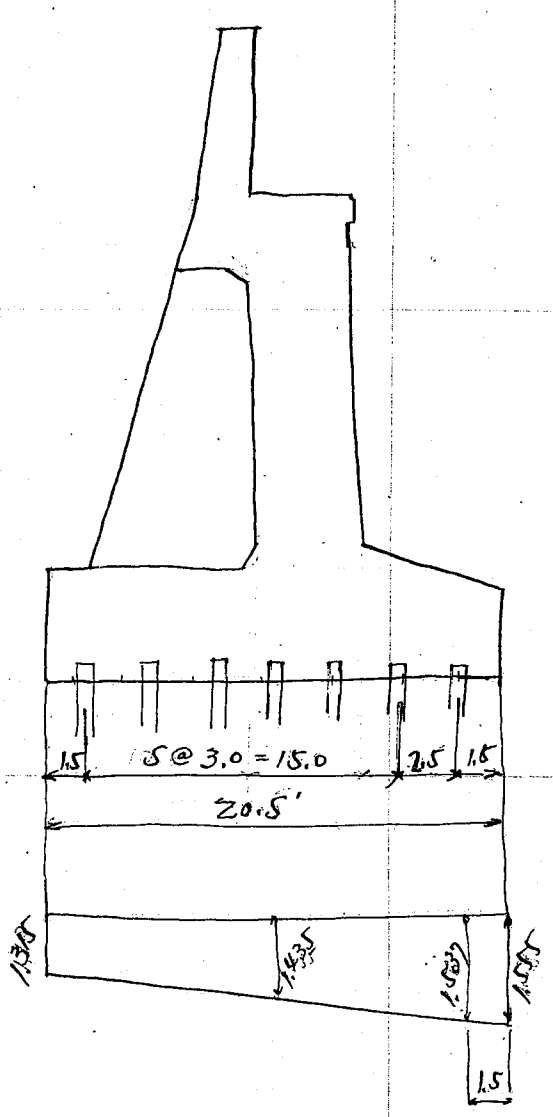
CALCULATIONS FOR

H1-D Bridge for City of Tokyo
Revised Calculation for Abutment



Case 1.

Eccentricity = 0.3'

max. toe pressure $= \frac{217870}{20.5 \times 33}\left(1 \pm \frac{6 \times 1.3}{20.5}\right) = 3490\ \#/\text{ft}^2 = 1.555\ \text{tons/ft}^2$
or = 2940 = 1.315 "

Use foundation piles 0.8 dia at tip & 36.0 long

max load on pile = $1.537 \times 3 \times 2.75 = 12.65$ tons

Surface friction on pile = $\frac{12.65 \times 2240}{1 \times 3.142 \times 35} = 258\ \#/\text{ft}^2$

Pile at center of base

load on one pile = $1.435 \times 3 \times 3 = 12.90$ tons.

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CALCULATIONS FOR

Design of HID Bridge for City of Tokyo.

Case 2. Stability during Earthquake, Seismic forces toward river side.

Earth pressure during Earthquake $K = .4 \times \frac{3}{4} = .3$

$E = \frac{.662 \times 100 \times 28.2^2}{2} \times .33 = 867{,}000$ # Slope 27°05' to hor. line.

Hor. component $E_h = 867{,}000 \times .89 = 782{,}000$ #

Vert. " $E_v = 867{,}000 \times .455 = 394{,}000$

Taking moment about O.

Loads		Vert. forces	Hor. forces	Lever arm	Moment.
Super imposed D. L.	D	330,000		− 9.0	−2,970,000
" " "	D'		132,000	23.2	3,060,000
Weight of Shaft	S	619,700		−10.2	−6,310,000
" "	S'		247,900	15.98	3,960,000
" Earth	E_1	123,000		−13.0	−1,600,000
" "	E_1'		49,200	11.09	545,000
" "	E_2	241,000		−15.4	−3,715,000
" "	E_2'		96,400	20.47	1,975,000
" "	E_3	152,000		−18.5	−2,810,000
" "	E_3'		60,800	16.6	1,010,000
" Base	B	483,000		− 9.75	−4710,000
" "	B'		193,200	2.5	483,000
Earth pressure	E_h		782,000	9.4	7,350,000
"	E_v	394,000		− 19.5	−7,680,000
		$\Sigma V = 2{,}342{,}700$	$\Sigma H = 1{,}561{,}500$		−11,412,000

Distance of Resultant force from O $= \frac{11{,}412{,}000}{2{,}342{,}700} = 4.88'$ inside of O.

Eccentricity $= 9.75 - 4.88 = 4.87'$

Resultant force out of middle third, Pressure area $= 4.88 \times 3 = 14.64$

max. toe pressure $= \frac{2 \times 2{,}342{,}700}{14.64 \times 33} = 9{,}710$ #/o' $= 4.33$ tons/o'

max. load on one pile $= 3.89 \times 3 \times 3 = 35.0$ tons.

Case 3. Stability during Earthquake, Seismic forces acting toward bankside.

Reffer to the figure for Case 2.

Loads		Vert. forces	Hor. forces	Lever arm	Moment.
Superimposed D.L.	D	330,000		9.0	2,970,000
" "	D'		132,000	23.2	3,060,000
Weight of Shaft	S	619,700		10.2	6,310,000
"	S'		247,900	15.98	3,960,000
weight of earth	E_1	123,000		13.00	1,600,000
"	E_2	241,000		15.40	3,715,000
"	E_3	152,000		18.50	2,810,000
weight of Base.	B	483,000		9.75	4,710,000
"	B'		193,200	2.5	483,000
	$\Sigma V =$	1,948,700	573,100		29,618,000

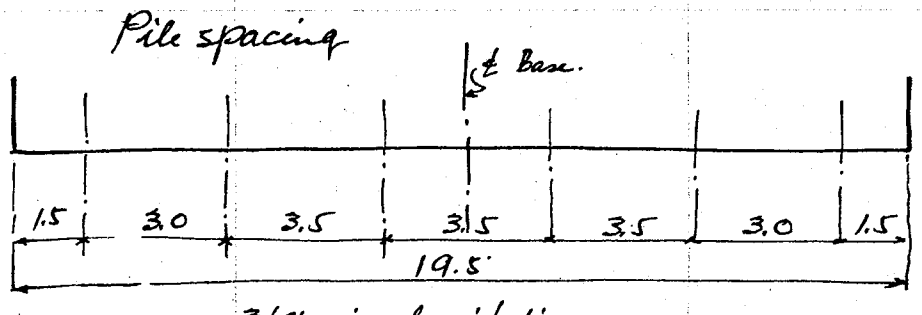
Distance of Resultant force from O $= \frac{29{,}618{,}000}{1{,}948{,}700} = 15.2$ from O.

Eccentricity $= 15.2 - 9.75 = 5.45'$ resultant out of middle third.

Pressure area $= (19.50 - 15.2) \times 3 = 12.90'$

Max. toe pressure $= \frac{2 \times 1{,}948{,}700}{12.9 \times 33} = 9{,}150$ #/o' $= 4.09$ tons/o'

Pile spacing



3' spacing longitudinary.

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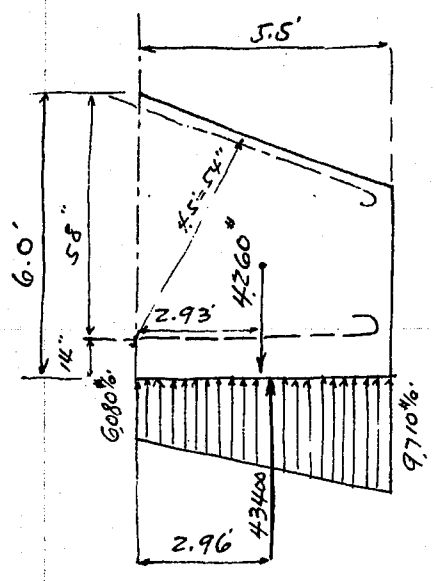
CHECKED BY DATE PAGE NO 36

CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo.

Cantilever footing. Case 2. controls the section.



upward pressure 9710#/' at toe.

do. at fixture $9710 \times \frac{14.64-5.5}{14.64} = 6080$#/'

Total upward pressure = $\frac{9710+6080}{2} \times 5.5 = 43400$# per ft strip of footing.

Pt. of appl. = $\frac{5.5}{3} \times \frac{9710\times2+6080}{9710+6080} = 2.96$

Weight of footing = $\frac{4+6}{2} \times 5.5 @ 150 = 4{,}260$# per ft. strip of footing.

pt. of application = $\frac{5.5}{3} \times \frac{6\times2+4}{6+4} = 2.93'$

Place the reinforcement 2" above top of pile

Effective depth = 58." abt.

moment of fixture = $43{,}400 \times 2.96 - 4{,}260 \times 2.93 = 116{,}000'$#

Steel area required = $\frac{116{,}000\times12}{17067\times1.8\times\frac{7}{8}\times58} = .89$" per ft strip

Use 7/8"ϕ bars 0.65' spacing = .93" per ft strip.

or 1"ϕ " 0.85' " = 0.93" " " .

neg. moment = $4{,}260 \times 2.93 = 12{,}500'$#

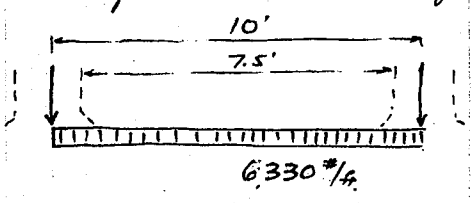
steel req'd = $\frac{12500\times12}{17067\times1.8\times\frac{7}{8}\times54} = .103$" per ft strip.

use 3/4"ϕ bars 2.0 spacing = .22" per ft strip.

max endshear = $43{,}400 - 4260 = 39140$#

unit shear = $\frac{39140}{12\times\frac{7}{8}\times58} = 64$#/" OK.

Longitudinal Reinforcement on rear side of Base.



Case 3 controls the section for this case.

Upward pressure = 9,150 #/'

downward " = 28.2 × 100 = −2820

6330#/' upward.

moment = $\frac{6330\times10^2}{8} = 79{,}100'$#

For continuity of base.

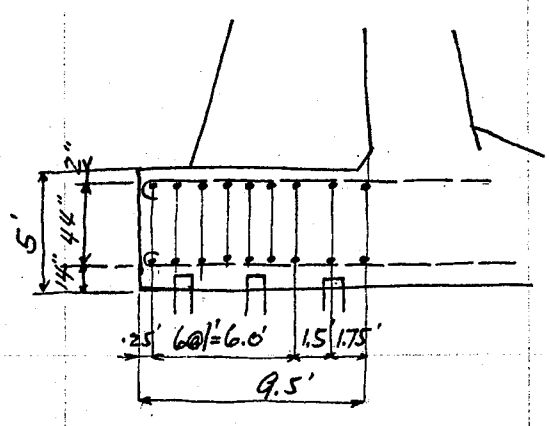
moment = $79{,}100 \times .8 = 63{,}300'$#

max end shear = $6330 \times 3.8 = 24{,}100$#

Steel req'd. = $\frac{63300\times12}{17067\times1.8\times\frac{7}{8}\times46} = .61$" per ft. strip. on top + bottom.

Use 7/8"ϕ 1' spacing = .60" per ft.

Shear = $\frac{24100}{12\times\frac{7}{8}\times46} = 50$#/" OK.



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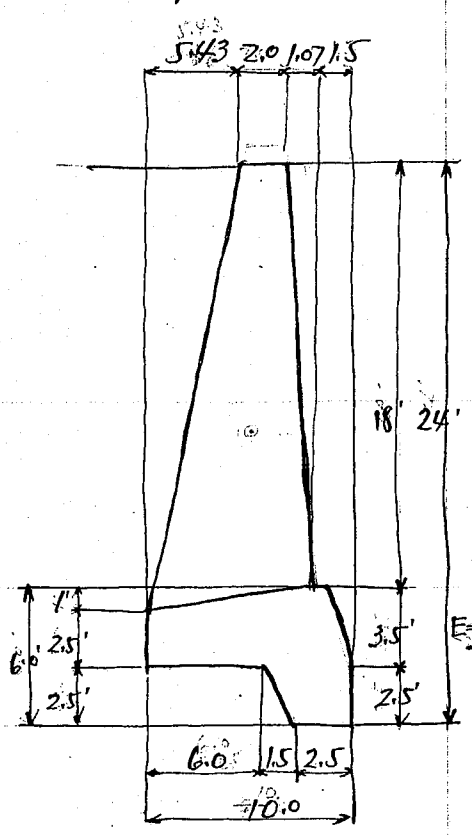
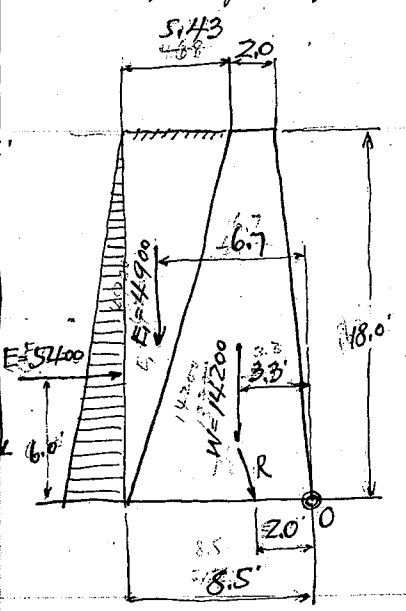
CALCULATIONS FOR

Design of 41D Bridge for City of Tokyo

Design of Wingwall.





Materials of wall shall be as follows.
Shaft Rubble Concrete with stone facing
Unit weight assumed to be 150 #/cub ft as a whole.
Foundation base 1:3:6 Concrete whose weight assumed to be 140 #/cub ft.

Shaft.

Stability of shaft at the bottom section
Height of earth filling 18', Earth pressure = $\frac{100\times18^2}{6}$ = 5,400 # per ft strip

Weight of Shaft $\frac{2+8.5}{2}\times18$ @ 150 = 14,200 #
point of application = 3.3' from O horizontally
weight of earth on the back of shaft
= $\frac{5.43\times18}{2}$ @100 = 4,900 #

Taking moment about O

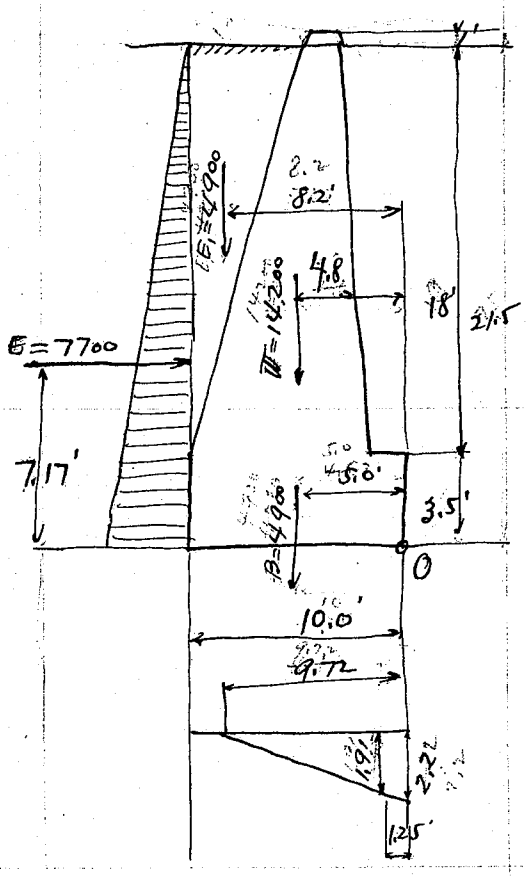
Loads	Vert. forces	hor. forces	lev. arm	Moment
W	14,200		3.3	− 46,900
E1	4,900		6.7	− 32,800
E		5,400	6.00	32,400
	ΣV = 19,100	ΣH = 5,400		− 47,300

Distance of resultant force from O.
= $\frac{-47,300}{19,100}$ = 2.48'

Eccentricity = 4.25 − 2.48 = 1.77

Resultant force out of middle third, neglecting tension.
Pressure area = 2.48 × 3 = 7.44
max toe pressure = $\frac{2\times19,100}{7.44}$ = 5,130 #/' = 36 #/" comp. ok.

Stability at bottom of base.



Earth pressure = $\frac{100\times21.5^2}{6}$ = 7,700

wt. of base = 3.5 × 10 @ 140 = 4,900 #

Taking moment about O.

loads	vert load	hor. load	lev arm	moment
W	14,200		− 4.8	− 68,200
E1	4,900		− 8.2	− 40,200
B	4,900		− 5.0	− 24,500
E		7,700	7.17	55,200
	ΣV = 24,000	ΣH = 7,700		− 77,700

Dist of resultant force from O = $\frac{-77,700}{24,000}$ = 3.24' from O.

Eccentricity = 5.00 − 3.24 = 1.76'

Resultant force out side of middle third, neglecting tension,
pressure area = 3.24 × 3 = 9.72
max. toe pressure = $\frac{2\times24,000}{9.72}$ = 4940 #/' = 2.2 tons/'

Use foundation piles 0.7'ϕ × 30' at the intervals of 3' × 2.5'
max. load on one pile = 1.91 × 3 × 2.5 = 14.3 tons.

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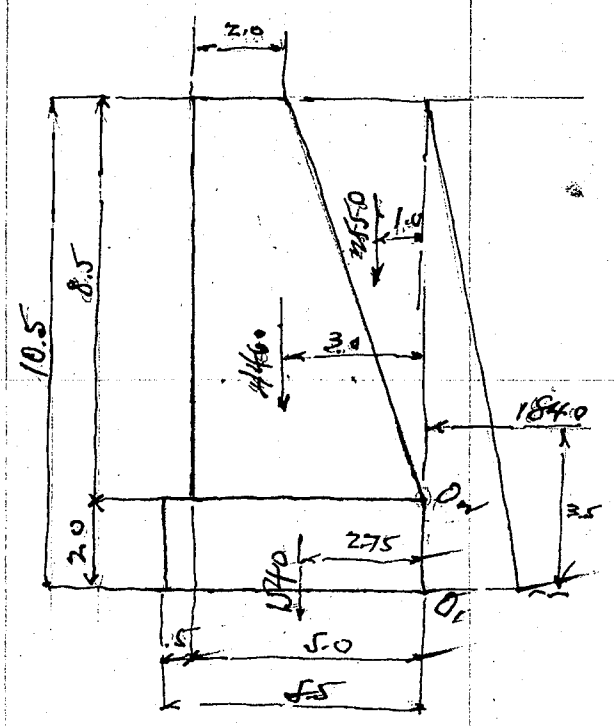
CALCULATIONS FOR

H.D. Bridge for City of Tokyo

Design of Retaining wall.



Weight of shaft $\frac{2.0+5.0}{2} \times 8.5 @ 150 = 4460$ #

" " Earth fill $3 \times 8.5 @ 100 = 2550$ #

Weight of base $2 \times 5.5 @ 140 = 1540$ #

Earth pressure $= \frac{100 \times 10.5^2}{6} = 1840$ #

Taking moment about O_1

4.460	× 3.0	= 13.380
2550	× 1.0	= 2550
1540	× 2.75	= 4230
1840	× 3.5	= 6440
10.390		26600

Distance of resultant force from O_1

$= 26600 \div 10390 = 2.56'$

Eccentricity $= 2.75 - 2.56 = 0.19$

Resultant within middle third

max toe pressure $= \frac{10390}{5.5}\left(1 \pm \frac{6 \times 0.19}{5.5}\right) = 2280$ #/□' $= 11.02$ ton/□'

or $1.5000 = 1.67$ "

Load on one pile $= 1.62 \times 3 \times 3 = 9.2$ tons

Stability at top of Base

Earth pressure $= \frac{100 \times 8.5^2}{6} = 1110$ #

taking moment about O_2

4460	× 3.0	= 13.380
1110	× 2.83	= 3.140
2550	× 1.0	= 2.550
8120		19070

Dist of resultant from O_2

$= 19070 \div 8120 = 2.35'$

Eccentricity $= 2.75 - 2.35 = 0.4'$

Resultant within middle third Ok

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