DESIGN OF MURAYAMA-BASHI

HIGHWAY & RAILWAY COMBINED

FOR

NAGANO-KEN & NAGANO-ELECTRIC RY. CO.

DESIGNED BY
J. MASUDA

ASSISTED BY
MESSRS T.M. KISHIDA, K. KITAMURA, T. OGAWA
AND K. OGUNI.

APRIL 1924

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Final Design of Murayama-bashi for Nagano-Ken and Nagano-Electric Ry Co.

This bridge will be located on the main highway between Nagano and Suzaka 140 Ken down stream of the present wooden bridge which is old enough to be renewed in the immediate near future, And also the river channel at this site has been improved and widened which necessitate to built bridge between harbor lines of river channel. The approximate distance between the said harbor lines is about 450 Ken and the total length of bridge is about 2670'. Nagano-Ken decided to build new bridge 18' wide and also the Nagano-Electric Ry Company newly organized to build electric railway line between the Cities of Nagano and Suzaka seven and one-half miles long which must cross the Chikuma-gawa at the same site; on the side of Electric Railway Company, the building cost of the bridge amounts to considerable sum compared to the construction cost of the electric railway route and company proposed to build the new bridge side by side with highway bridge to reduce the construction cost of bridge. We estimated the cost of bridges separate and also combined and found the cost will be reduced about 20% when combined the structure on the river portion. And the members of the company and officials of the Nagano-Ken concerned in this case agreed to build bridge side by side and to share the profit of construction cost of the said bridge. We estimated the economic span length of the bridge and also the type of the bridge and the following is the general layout

Approach spans on East (Railway side) Steel girder span 33 @ 44'-6" between C piers
do (Highway side) Reinforced Concrete girder spans 33 @ 44'-6" between C Bents or 11 sets of 3-44'-6" continuous spans.
River span (Railway and Highway Combined) 7 spans @ 165'-0" c to c of end bearings and Electric railway tracks outside of 18' roadway.
Approach span on West (Railway side) one steel girder span @ 23'-8"
do (highway side) one span of reinforced Concrete girder 23'-8".

Requirements of the bridge -
The electric railway tracks will be located down stream and highway on upstream due to the local condition. Highway floor will be built reinforced concrete paved with 3" asphaltic concrete with reinforced concrete handrails with cast iron panels on both sides. Electric railway will be open deck.

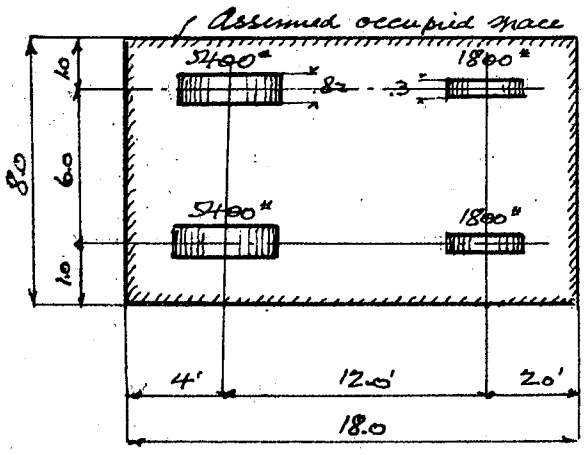
Loadings of the bridge -
Electric railway loading Cooper's E25

$$\text{Impact allowance} = \frac{300}{300+l} + \frac{30}{60}$$

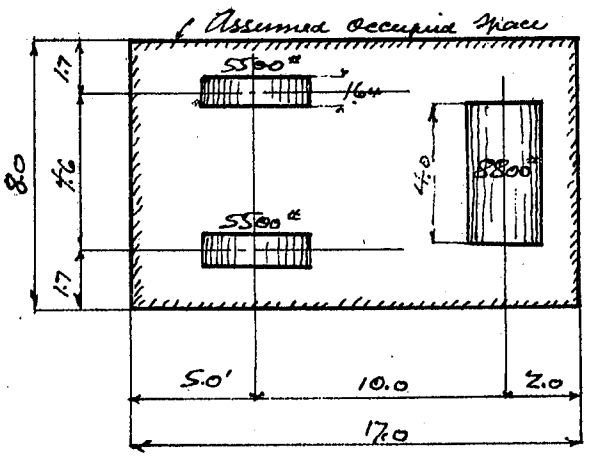
where l = span length or loaded length in ft and 30 assumed max velocity or speed in miles per hour

Highway loading
Uniform Live Load $q^{kg/m^2} = \frac{100.000}{170+l}$ but not over $500^{kg/m^2}$ where l = span length in meter
Motor truck loading 6.4 tons distribution of wheel concentrations assumed as in sketch
Road roller concentration 9.0 tons distribution of load as shown on sketch below.

Motor trucks Concentration



Road roller concentration



Impact allowance : No impact for uniform live load and road roller concentration
For motor trucks: 25% impact for trusses and ⅓ impact for others assumed
2 motor trucks side by side on span with impact specified above; Unoccupied space to be filled in with uniform live loads.
For Road roller: 1 road roller without impact on span; unoccupied space to be filled in with uniform live loads.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Assumed working strengths

Concrete 1:2:4 mixture

Fibre stress in concrete	600 #/□"	for positive moment
Fibre stress in concrete	700 #/□"	for negative moment
Direct compression	500 #/□"	
Shear (reinforced)	120 #/□"	
Shear (plain concrete)	57 #/□"	
Punching shear	120 #/□"	
Bond stress	85 #/□"	
Reinforcing Bars Tension	16000 #/□"	

Structural steel

Tension		16000 #/□"
Compression		16000 − 70 l/r (Railway side.)
Fibre stress		16000
Shear	(shop rivets)	12000
	(field rivets)	10.000
Bearing	(shop rivets)	24.000
	(field rivets)	20.000

Expansion Rollers per inch 600d d = diameter of roller in inches

Compression on plate girder flange $16000\left(1-0.012\frac{l}{b}\right)$

where l = unsupported length of plate girder flange

b = width of flange.

Compression member highway only gross section $16000\left(1-0.0055\frac{l}{r}\right)$

where l = unsupported length of member

r = least radius of gyration

with max. of 14000 #/□" and l/r limited to 100 for main member and 140 assumed for wind bracings and secondary members. For tension members 200.

Miscellaneous data:-

Ratio between moduli of elasticities of steel and concrete = 15

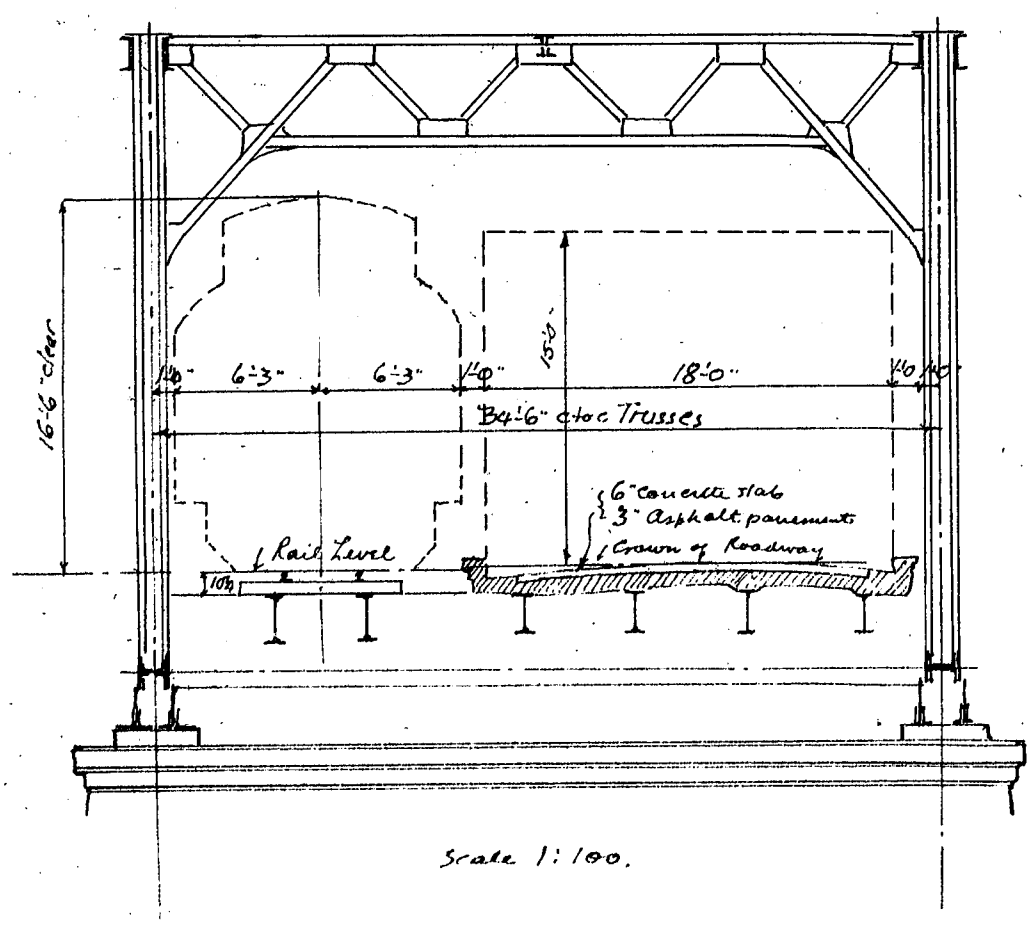
Expansion coefficient of Concrete = 0.0000055 per 1° of Fahrenheit

" " " Steel = 0.0000060 " " " "

Temperature change for concrete structure = ±40°F Standard temperature

Temperature change for steel structure = ±60°F assumed as 6/2°.

Cross section of structure as shown on sketch below.



Scale 1:100.

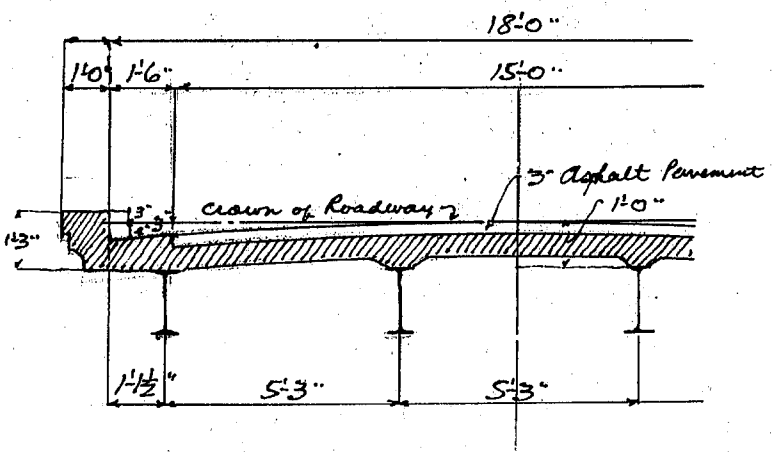
Note:- Cross section of approach span similar to this where Electric Ry side will be plate girder and highway will be reinforced Concrete girder design.

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Design of River span

span length 165'-0" c to c end bearings which is divided into 8 panels @ 20'-7½"

Highway Floor ;- Concrete Floor span length of slab = 5'-3"



Dead Load

Asphalt Pavement	30.0
Slab 6" assumed	75.0
	105.0 #/0'

call this 110 #/0' including fillet over stringer as shown on sketch

Dead Load moment = ⅛ · 110 · 5.25² = 303 '#

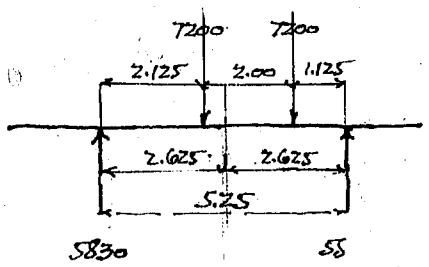
Dead Load shear = ½ · 110 · 5.25 = 289 #

Live Load ;-

Motor truck Loading Rear wheel concentration 5400 # Front wheel conc. with impact = 2400 #

⅓ impact 1800

7200 #



Reaction = 7200 · 4.25/5.25 = 5830 # Moment = 5830 · 2.125 = 12.380 '#

For continuity of slabs m = 12,380 · 0.8 = 9904 '#

Distribution of wheel concentration on slab assumed as follows

Effective width = 0.6 · 5.25 + 1.0 = 4.15'

moment per ft strip = 9904 : 4.15 = 2384 '#

Live Load End shear neglecting continuity of slab and figured as simple beam ; 1st concentrated load on support

Reaction = 7200 · 3.25/5.25 = 4460 #

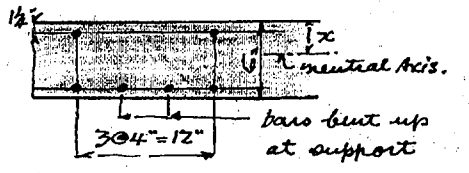
7200

11660 #

For one foot strip = 11660 ÷ 4.15 = 2810 #

Summary for moments and shears	moments	shears
Dead Load	303	289
Live Load	2384	2810
	2687 '#	3099 #

Slab assumed 6" ; Amount of reinforcement = 0.590 · Insuration say 1.25" (This has been changed to 1" on the drawing) Effective depth = 4.75"



½" dia bars in slabs.

For one ft strip

Compression side	1-½"φ bar =	0.1963
Tension side	3-½"φ bars =	0.5889
	Total area	0.7852 □"

Position of neutral Axis = 1.95" from top

Moment of Inertia J = 12 · 1.95³/3 = 29.66

= 15 · 0.1963 · (1.95-1.25)² = 1.44

= 15 · 0.5889 · (4.75-1.95)² = 6940.

100.50

Section modulus S = 15 · 0.5889 · (4.75-1.95) = 24.75 (inch)³

Fibre Stress in concrete $G_b = \frac{2687 \times 12 \cdot 1.95}{100.5} = 627$ #/□"

Stress in compressive steel $G_c' = 15 \times 627 \cdot \frac{1.95-1.25}{1.95} = 3375$ #/□"

Stress in tensile steel $G_c = 15 \cdot 627 \cdot \frac{4.75-1.95}{1.95} = 13500$ #/□"

Horizontal Limit shear $T_{max} = \frac{3099 \times 24.75}{12 \cdot 100.5} = 63.6$ #/□"

2 bars as shown in sketch bent up at support to take care of shearing force.

Bond Stress $T_z = \frac{63.6 \cdot 12}{4 \cdot 1.57} = 121.0$ #/□"

Overhanging slab carrying handrail

Total weight P = 360.8 # including handrail, coping and gutter where handrail assumed 100.0 # per lin ft.

Moment at End stringer = 57145 inch-lbs

Average Arm o= M/P = 15.8"

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

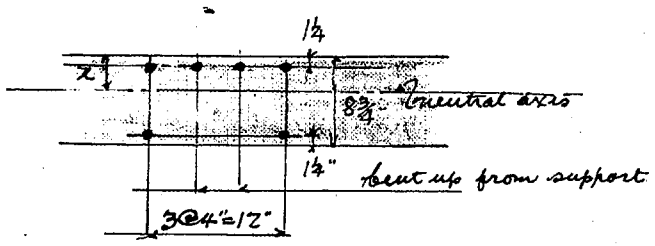
Live Load Moment one wheel concentration near curb line

moment = 7200 × 7.5" = 54.000 "#

Distribution assumed 2' wide For one ft strip = 54000 ÷ 2 = 27000 "#

Dead Load moment = 5710

Total moment 32.710 per ft strip



½"ϕ bars spaced as shown

Shear = Dead load 360

Live load 3600

3960" per ft strip

Steel Area compressive side 1-½"ϕ bar = 0.1963

Tension side 3-½"ϕ bars = 0.5889

Total Area = 0.7852

Position of Neutral Axis = 2.568"

Moment of Inertia $J = \frac{12 \times 2.568^3}{3} = 67.732$

$= 15 \times 0.1963 \times (2.568 - 1.25)^2 = 5.113$

$= 15 \times 0.5889 \times (7.5 - 2.568)^2 = 214.860$

287.705 (inch)⁴

Section modulus $S = 15 \times 0.5889 \times (7.5 - 2.568) = 43.56$ (inch)³

Fibre stress in concrete $f_b = \frac{32.710 \times 2.568}{287.705} = 292$ #/o" ok

Stress in compressive steel $f_s' = 15 \times 292 \times \frac{2.568 - 1.25}{2.568} = 2247$ #/o"

Stress in tensile steel $f_s = 15 \times 292 \times \frac{7.5 - 1.25}{2.568} = 10.661$ #/o"

Horizontal Unit Shear $J_{max} = \frac{3960 \times 43.56}{12 \times 287.705} = 50.0$ #/o"

Bond stress $J_h = \frac{50 \times 12}{4 \times 1.5708} = 96$ #/o" ok

STRINGERS span length 20'-7½" spaced 5'-3"

Neglect continuity of slab and take reaction as simple beam

Dead Load Floor 110 × 5.25 = 578.0 moment = ⅛ × 630 × 20.625² = 33500 "#

Stringer assumed 52.0 Shear =

630.0 # assumed

If count negative reaction due to overhanging slab, the load will be $630 - \frac{5710}{12 \times 5.25} = 540$ # per ft

moment = ⅛ × 540 × 20.625² = 28700 "#

Shear = 540 × $\frac{20.625}{2}$ = 5580 #

End Stringer Dead Load ½ panel = $110 \times \frac{5.25}{2}$ = 389

overhanging arm direct load = 360

Extra reaction $\frac{5710}{12 \times 5.25}$ = 90

740 #

Stringer assumed 50

790 # per lin ft.

moment = ⅛ × 790 × 20.625² = 42.000 #

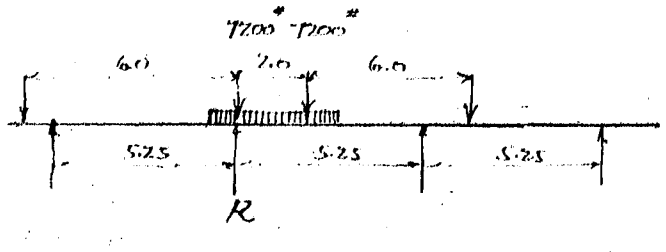
Shear = 790 × $\frac{20.625}{2}$ = 8160 #

Live Load :— Motor trucks loading Rear wheel with ⅓ impact = 7200 #

Front wheel with " " = 2400 #

Spacing of wheel concentration as shown on diagram on pp 1 and unoccupied space of roadway to be filled by uniform load of 100 #/o'

Intermediate Steel Stringer



Rear wheel $7200 \times \frac{3.25}{5.25}$ = 4450

7200

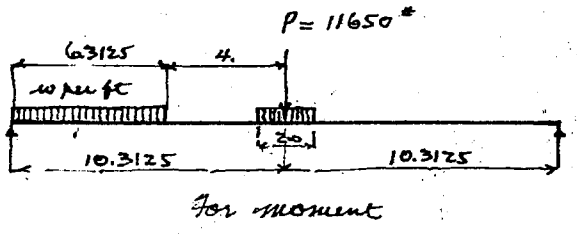
11650 #

Front wheel ⅓ of Rear wheel = 3880

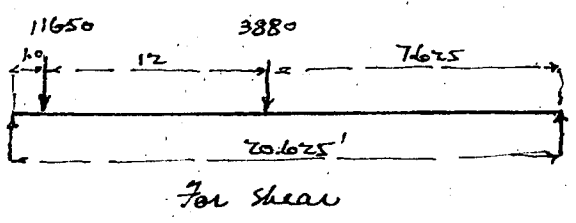
Total 15530 #

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Live Load Moment due to motor truck loading



For moment



For shear

For End Stringer

wheel concentration Rear = 7200#
Front = 2400#

Moment = 35300'# Equivalent uniform load = 660# per ft max Reaction = 7700#

concentration = 11650# at center of span
Uniform load at rear 525# × 6.3125 = 3310#
Reaction 3310 × 3.156 ÷ 20.625 = 560
11650 ÷ 2 = 5825
6385

For Intermediate Stringer

moment 6385 × 10.3125 = 66000
Less $\frac{11650}{4}$ = 2910
63090 say 63100'#

Equivalent uniform load = $\frac{63090 \times 8}{20.625^2}$ = 1185#

Reaction max shear

11650 × 19.625 ÷ 20.625 = 11.100
3880 × 7.625 ÷ 20.625 = 1400
12.500#

Summary for moments and shears

	For Intermediate Stringer		For End Stringer	
	Moment	Shear	Moment	Shear
Dead Load	28700'#	5580	42000	8160
Live Load	63100	12500	35300	7100
Total	91800'#	18080#	77300'#	15860#

Equivalent uniform load = $\frac{91800 \times 8}{20.625^2}$ = 1730# per lin ft

Section modulus required for Intermediate Stringer = $\frac{91800 \times 12}{16000}$ = 69.0

Section modulus required for End Stringer = $\frac{77300 \times 12}{16000}$ = 58.0

Use 18". 54.7# I beam for both intermediate and end stringers S.M. = 88.4 (inch)^3

Deflection of beam

$$\frac{\delta}{l} = \frac{5}{24}\left(\frac{S}{E}\right) \times \left(\frac{P}{q}\right) \times \left(\frac{l}{L}\right) = \frac{5}{24} \times \frac{16000}{30.000.000} \times \frac{1185}{1730} \times \frac{20.625 \times 12}{18} = \frac{1}{955}$$

Weight of Stringers 4 Stringers @ 55 = 220#
details say 10
230# per lin ft of span

Electric Railway Stringers

Dead Load

main rail 60# per yard 2 @ 60 = 120
Guard rail do 2 @ 60 = 120
240 ÷ 3 = 80.0# per ft

Ties $\frac{6" \times 8"}{144} \times 7.0 = 2.33$ 2' spacing say $2.33 \times \frac{50}{2}$ = 60.0
Timber guard rails 2 @ 0.5 × 0.5 @ 50# = 25.0
Foot walk 2 ft at center of tracks 2.0 × 0.1 @ 50 = 10.0
Spikes, splices, bolts, hook bolts etc say 10.0
185.0 ÷ 2 = 92.5#
Dead Load beam and lateral bracings assumed 102.5
195.0# per ft of Stringer

Dead Load moment = $\frac{1}{8} \times 195 \times 20.625^2$ = 10370'#
Dead Load shear = $\frac{1}{2} \times 195 \times 20.625$ = 2010#

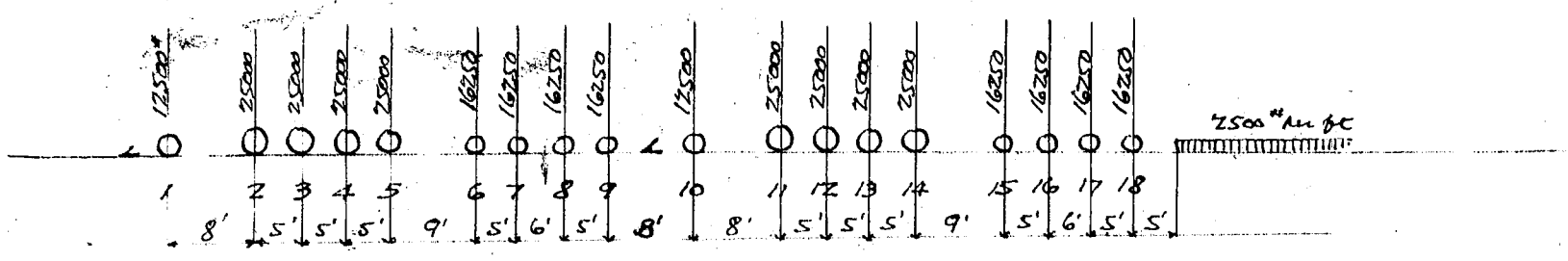
Live Load Cooper's E 25

Impact coefficient = $\frac{300}{300+l} \times \frac{V}{60}$ where V = assumed 30. miles per hour
l = loaded length

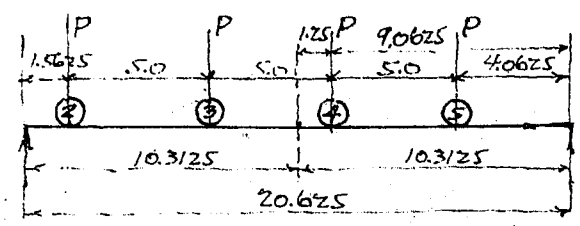
$$I = \frac{300}{320.625} \times \frac{1}{2} = 0.468$$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Diagram of Cooper's E25 loading



Live load Moment



Reaction R.

Concentrated load = P

Reaction $R = P \cdot \frac{36.25}{20.625} = P \cdot 1.757$

Moment = $P \cdot 1.757 \times 9.0625 = 15.92\ P$

Less $5.00\ P$

$10.92\ P$

where $P = 12500^{\#}$ per rail

Impact 46.8% 5850

$18350^{\#}$

Max moment = $18350 \cdot 10.92 = 200.300^{\#}$ Equivalent Uniform load = $\frac{200.300 \times 8}{20.625^2} = 3760^{\#}$ per ft

Max reaction = $P' \cdot \frac{31.875}{20.625} = 1.545\ P$

$1.000\ P$

$2.545\ P$

Reaction or max End shear = $2.545 \times 18350 = 46750^{\#}$ including impact

Summary for moments and shears

	Moments	Shears
Dead Load	10.370	2010
Live Load	200.300	46750
	$210.670^{\#}$	$48760^{\#}$

Equivalent Uniform load for full load $= \frac{210.670 \cdot 8}{20.625^2} = 3960^{\#}$ per ft

Section Modulus = $\frac{210.670 \times 12}{16000} = 158.0$

Use 24" × 80# I beam Section modulus = 173.9

Deflection $\frac{\delta}{\ell} = \frac{5}{24} \cdot \frac{16000}{30.000.000} \cdot \frac{3760}{3960} \cdot \frac{20.625 \times 12}{24} = \frac{1}{920} = 0.00109$

Weight of Electric Railway Stringers

2 @ 80# = 160#

details say 2 @ 5.0 = 10

Lateral bracing assumed = 25

195# per lin ft call this 200# per ft

Floor Beam span length = 34'-6" ctrs of trusses spacing = 20'-7½"

Dead Load

Highway loading pavement and floor slabs 110# × 15 = 1650

Ct. beyond paved roadway including Handrail 2 @ 360 = 720

Steel stringers 230

$2600^{\#}$

Uniformly distributed load say 135.5# per square ft

or 135.5 × 20.625 = 2.800# per ft

Railway loading Weight of Deck 190

stringers and lateral bracings 200

390# per lin. ft of tracks

For Panel concentration = 390 × 20.625 = 8050# per tracks

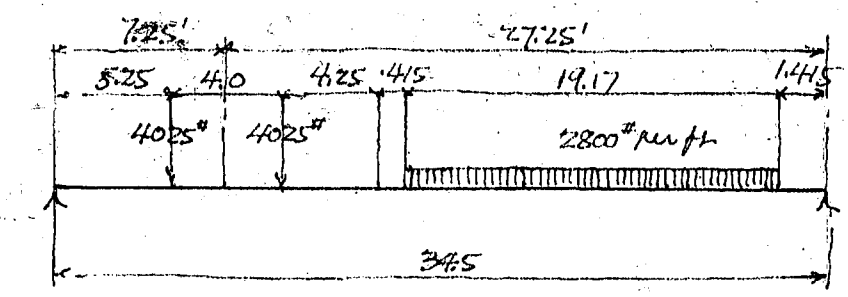
or 4025# per stringer

Dead load of Floor Beam assumed 220# per lin ft

or Reaction of Floor beam = $220 \cdot \frac{34.5}{2} = 3.800^{\#}$

Dead load moment of Floor Beam

増田淳氏関係資料
（独立行政法人 土木研究所蔵）



23400
3800
27200

FB

38350
3800
42150

Reaction :

2800 × 19.17 = 53700 × 11.0 = 590.000
8050 × 27.25 = 219.000
61750# 809.000 1#

809000 ÷ 34.5 = 23400
38350

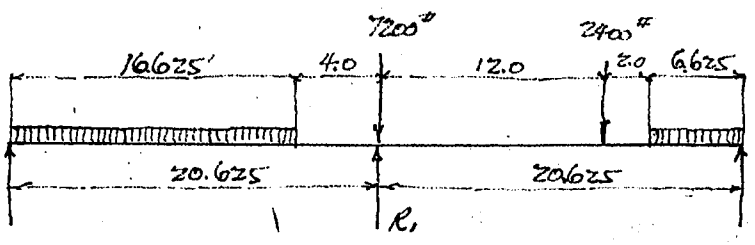
Point of Zero shear

38350 ÷ 2800 = 13.70'
1.415
15.115

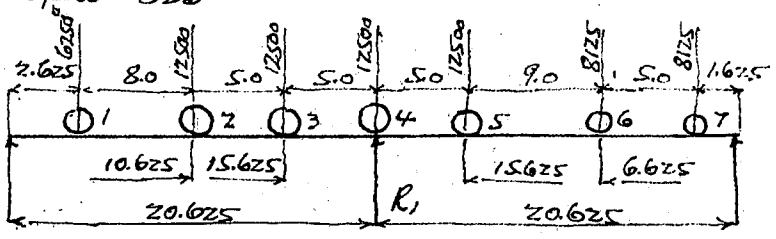
max moment = 38350 × 15.115 = 580,000
Less 2.800 × $\frac{13.7^2}{2}$ = 263.000
317.000 1#

Dead Load moment of Floor Beam 32.700
349.700 1#

Live load
Highway loading



Floor Beam concentration for Electric Ry
Cooper's E25



Reaction R_1 2400 × $\frac{8.625}{20.625}$ = 1.000
7200
8200 #

Total load 2 motor trucks 2 @ 8200 = 16400#
Uniform load on sides of motor trucks neglected

Uniform load 100 × $\frac{6.625^2}{2 \times 20.625}$ = 106

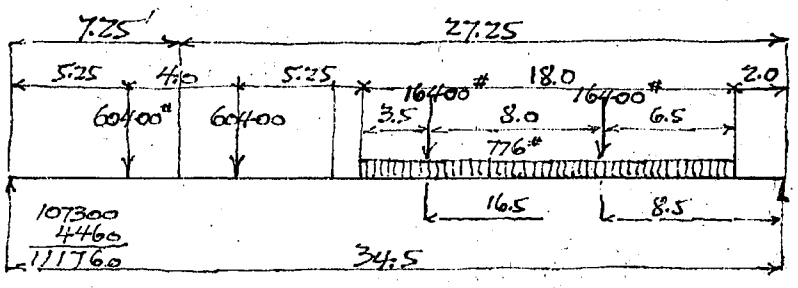
do 100 × $\frac{16.625^2}{2 \times 20.625}$ = 670
776# per ft.

Reaction R_1

12500 × (15.625 × 2 + 10.625) = 523.437
8125 × (6.625 + 1.625) = 67031
6250 × 2.625 = 16.406
606.874

606.874 ÷ 20.625 = 29424
12500
41924
Impact = 44% 18470
60.394
call this 60400#

Live Load Moment :-



Uniform load :-
776 × 18 = 14.000#
R = 14000 × $\frac{11}{34.5}$ = 4460#
14000 − 4460 = 9540#

Reaction :-

motor truck	16400	× 8.5	=	139400
do	16400	× 16.5	=	270600
Electric Ry	120800	× 27.25	=	3295000
	153600			3705000 ÷ 34.5 = 107.300#
	107300			
	46300#			

Live load moment 111760 × 9.25 = 1032000
Less 60400 × 4.00 = 241600
790400 1#

Summary for Moments and Shears

	Moment	shear (Ry side)	shear (-Ry side)
Dead Load	349.700 1#	27200	42150
Live Load	790.400	111760	55840
	1140.100 1#	138.960 #	97990 #

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Final Design of Murayama-bashi for Nagano-Ken and Nagano-Electric Ry Co

Try 48"·⅜" web = 18.00" & web = 2.250" Effective Depth say = 3.876' Stress = 296.000#

Section required = 296.000 ÷ 16000 = 18.50 □" or 18.50 - 2.25 = 16.25" net

Use 2L 6.6.⅝" = 12.86□" gross 10.61□" nett

1PL 12½"·½" = 6.25□" 5.25

19.11□" 15.86□" nett Unit stress = 296000 ÷ 1891 = 16350#/□" say 0.4.

Weight of one intermediate Floor Beam

web plate	1Pl.	48"·⅜	@ 61.20#	·	33'-4"	= 2040
flanges	4Ls	6".6".⅝"	@ 21.90	·	33'-4"	= 2920
Cover Pls	2Pls.	12½"·½"	@ 21.25	·	19'-1"	= 811
Ry Stiffs	4Ls	5.3½.⅜	@ 10.40	·	3'-11⅜	= 164
Fillo	4Pls	3½·9/16	@ 6.69	·	3'-0½"	= 81
Fillo	4Pls	3½"·⅜"	@ 6.69	·	1'-1¼"	= 36
Hy Stiffs	8Ls	4.3.9/16	@ 7.20	·	4'-0½"	= 233
	8Ls	3½.3½.9/16	@ 7.20	·	0'-9"	= 43
End Stiffs Ry	2Ls	6.4.½"	@ 16.20	·	3'-11⅜"	= 128
Fillo	2Pls	6.⅜	@ 11.48	·	3'-0¼"	= 69
End Stiffs Hy	2Ls	4".4.½"	@ 12.80	·	3'-11⅜	= 101
Fillo	2Pls	4.⅜	@ 7.65	·	3'-0¼"	= 46

6672

Rivet heads and variation 3½% 234

6906#

6906# ÷ 33.33 = 208.0# per lin ft

End Floor Beams

End Floor beams as shown on drawing # 6 with each stringer beared on pin; this will serve strut between trusses; flange angles 2Ls 3½.3½.9/16 with 24"·9/16 web for railway side and 30"·9/16" web on highway side which is safe enough to carry loads above.

Weight of one End Floor Beam complete

web Pl. Ry	1Pl.	24.9/16	@ 25.50	·	13'-7 5/16"	= 347
web Pl. Hy	1Pl	30.9/16	@ 31.88	·	19'-6 9/16	= 622
Flanges	2Ls	3½.3½.9/16	@ 7.2	·	13'-10 5/16	= 200
"	2Ls	3½.3½.9/16	@ 7.2	·	19'-8 13/16	= 284
"	2Ls	3½.3½.9/16	@ 7.2	·	33'-1⅝	= 477
Splice	2Plo	7.9/16	@ 7.44	·	1'-4¾"	= 21
Stiffs Ry	8Ls	3.3.⅜	@ 7.2	·	1'-1⅜	= 112
fillo	4Pls	6.9/16	@ 6.38	·	1'-4¾"	= 36
Stiffs Hy	16Ls	3.3.9/16	@ 6.1	·	2'-5⅜	= 239
fillo	8Pl	6.9/16	@ 6.38	·	1'-10¾	= 97
End Stiffs	4Ls	4.3.⅜	@ 8.5	·	1'-1⅜	= 66
fills	4Pls	3.9/16	@ 3.19	·	1'-4¾"	= 18
Base Pl.	4Pls	12½.½	@ 21.25	·	1'-1"	= 92
"	2Pls	14.½	@ 23.80	·	2'-0"	= 95
Sole Pl.	4Pls	7.½	@ 11.90	·	1'-0"	= 48
Base Pl.	4Plo	13¼.½	@ 22.53	·	1'-2"	= 105
"	8Pls	3.⅜	@ 3.83	·	1'-2"	= 36
"	8Plo	3.3.9/16	@ 6.10	·	1'-2"	= 57
	1Pl	10.9/16	@ 10.63	·	1'-2	= 12

29614

Rivet heads & variations 3½% 100

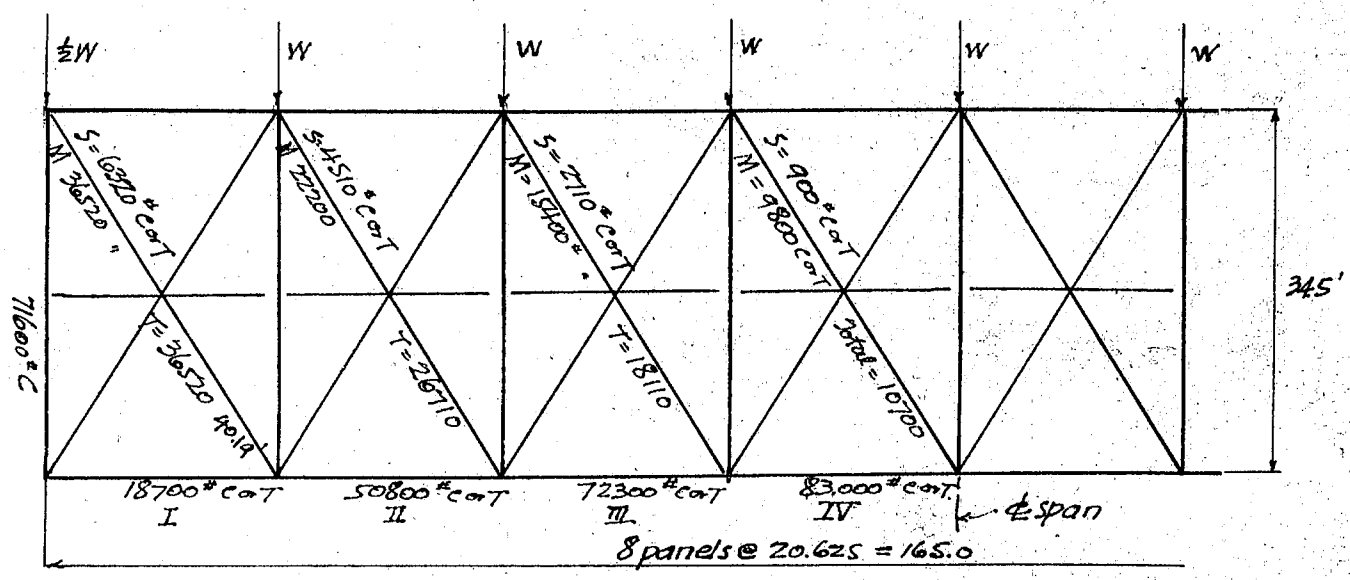
3064#

Summary for Floor system

Highway Stringers		230
Electric Ry Stringers		200
Int. Floor Beam	6910 ÷ 20.625 =	335
		765# per ft

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Bottom Laterals.



note :- when diagonals resist to tension only the stresses should be doubled

$\sec\theta = \frac{40.19}{345} = 1.165$ $\quad \tan\theta = \frac{20.625}{345} = 0.598$

Static wind load 30# per square ft of exposed surface or assume $5 \cdot 30 = 150$# per lin ft of truss
Moving wind load $12 \cdot 30 = 360$# per lin ft of truss; same moving load assumed for railway truss but neglecting static load.

Panel concentration static load $W_1 = 150 \cdot 20.625 = 3100$#
Hy moving load $W_2 = 360 \cdot 20.625 = 7400$#
Ry moving load W_2 = do = 7400
$W = 17900$#

In details assumed the diagonals take tension only. To prevent sag of members the strut provided at center line of bridge between floor beams
Use 2L's 5 × 3½ × ⅜" for first panel and 2L's 5 × 3½ × 5/16" for 2nd, 3rd and 4th panels.

Weight of Bottom Laterals

12L's	5 × 3½ × ⅜"	@ 10.4	× 18'-4½"	=	2293
4L's	do	@ 10.4	× 18'-1½"	=	754
24L's	5 × 3½ × 5/16	@ 8.7	× 18'-4½	=	3837
24L's	5 × 3½ × 5/16	@ 8.7	× 18'-1½	=	3785
					10669
	Rivet heads & variations 3½%				370
					11039#

Strut main section

4L's	3 × 3 × 5/16	@ 6.1	× 20'-5½"	=	499
1 Pl.	9 × 5/16	@ 9.56	× 20'-5½"	=	196
	Details say				161
		8 @ 856		=	7048
					18087#

$18087 \div 165 = 110$# per lin ft of span
Panel cone say $110 \cdot 20.625 =$ say 2280#

Upper Lateral Bracing

Static wind load 50# per square ft of exposed surface
Assumed $30 \cdot 50 = 150$# per lin ft or Panel concentration $= 150 \cdot 20.625 = 3100$#
Use 2L's 5 × 3½ × 5/16 riveted together with strut on center line of bridge as shown for lower Lateral Bracing

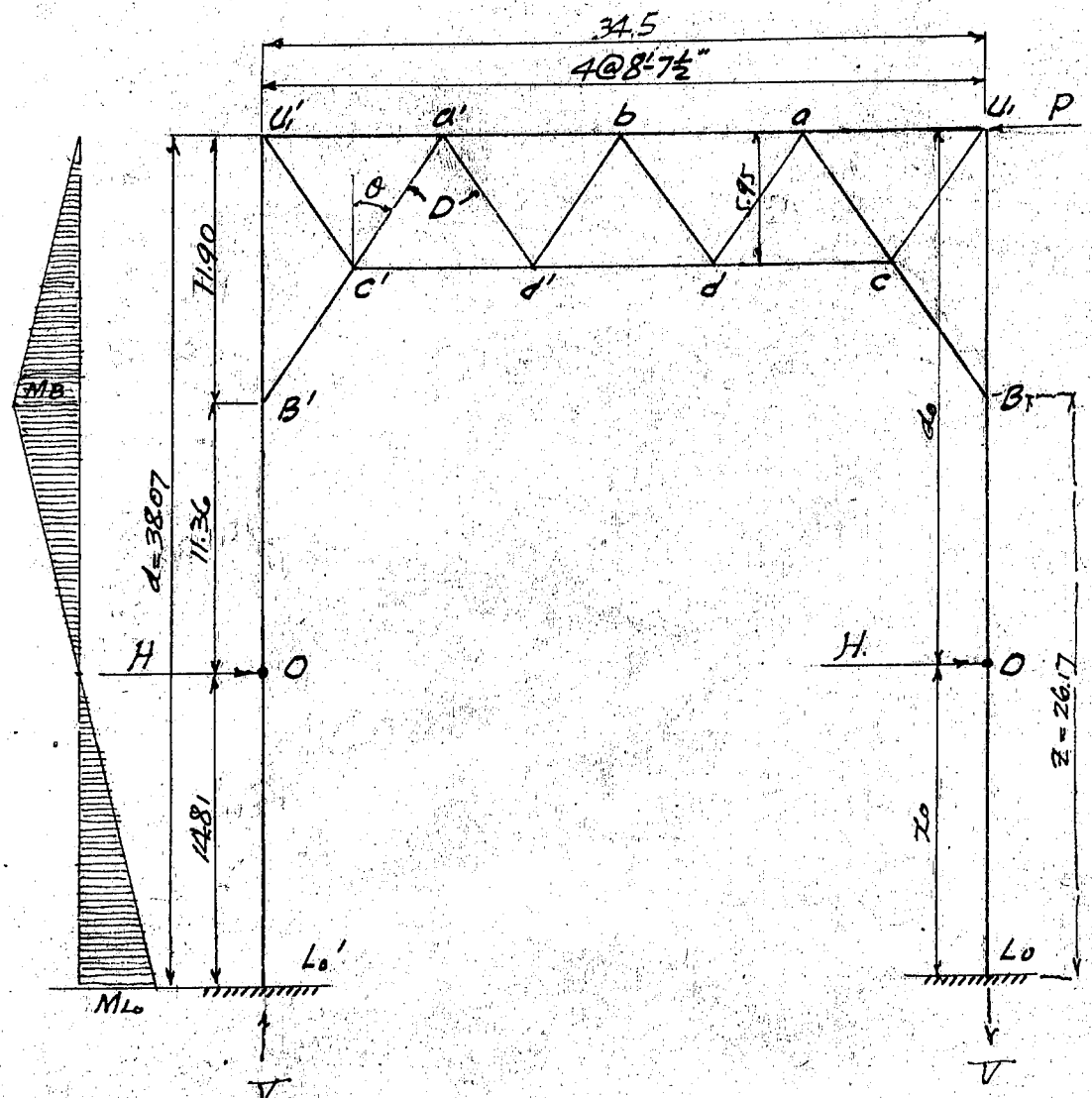
weight of the lateral bracing

diagonals	8 L's	5 × 3½ × 5/16	@ 8.7#	× 18'-3"	= 1270
strut	4 L's	3 × 3 × 5/16	@ 6.1	× 20'-5½	= 500
	1 Pl.	8½ × 5/16	@ 9.03	× 20'-5½	= 185
	Details say				225
					2180# per panel

8 @ 2180 = 13080#

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

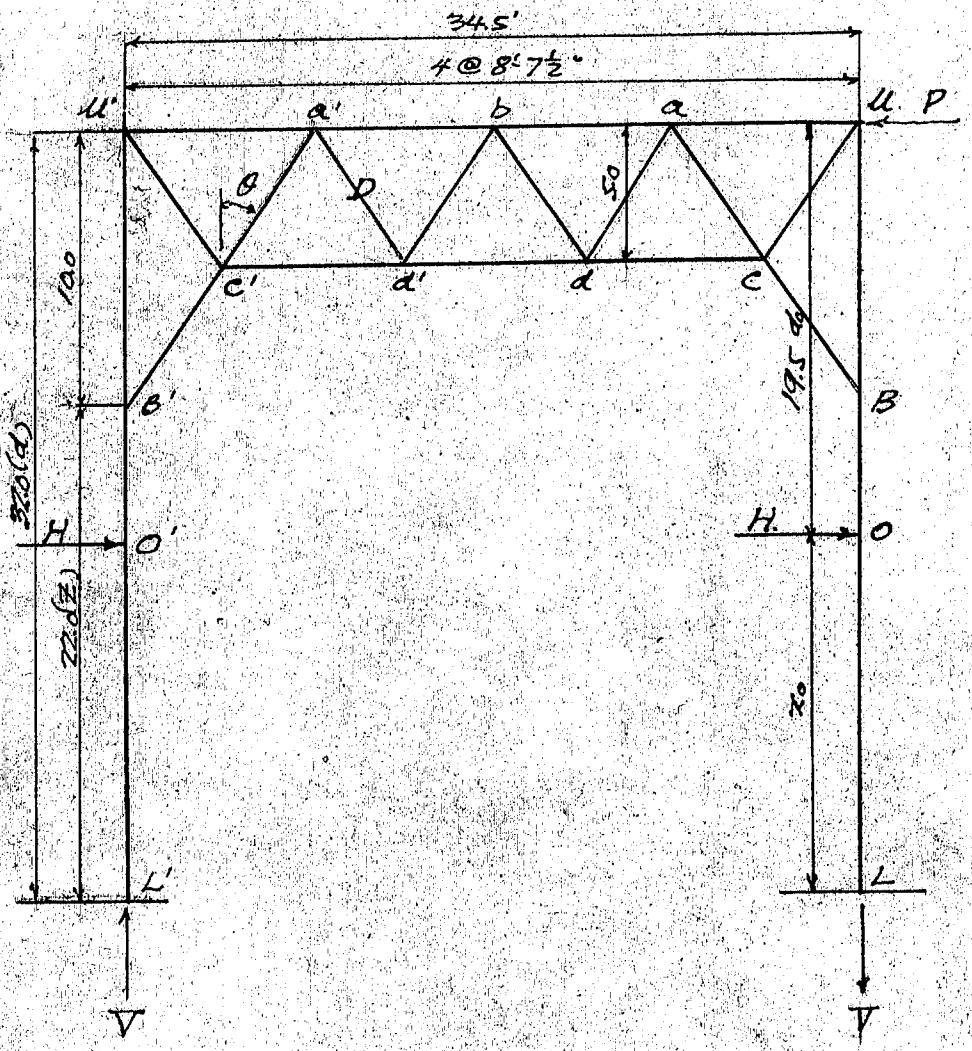
Portal Bracing



Length of End Post = $\sqrt{32^2+20.625^2}$ = 38.07'
x = 11.9' Length of diagonals = 7.6'
sinθ = 0.568
secθ = 1.277
Panel load = 3.5 × 3100 = 10850 #
$x_0 = \frac{z}{2} \cdot \frac{2d+z}{d+z}$ = 14.81'
$d_0 = d - x_0$ = 23.26'
$V = P \cdot \frac{d_0}{b}$ = 7315 #
D = ± V secθ = ± 9340 #
BC = B'C' = ± 13.550 #
U.C = U'C' = ± 9340 #
U.a = U'a' = − 21320 # or +10480 #
ab = a'b = − 10730 # or − 120 #
cd = c'd' = ± 10600 #
dd' = 0
H = $\frac{P}{2}$ = 5425 #
Moment at B = 5425 × 11.36 = 61630 #

Moment at L_0 = 5425 × 14.81 = 80340 #

Sway Bracing



Length of diagonals = 6.6'
sinθ = 0.653
secθ = 1.32
P = 3100 #
H = P÷2 = 1550 #
$x_0 = \frac{z}{2} \cdot \frac{2d+z}{d+2z}$ = 12.5'

$d_0 = d - x_0$ = 195'
$V = 3100 \cdot \frac{19.5}{34.5}$ = 1750 #
D = ± V secθ = ± 2300 #
BC = B'C' = ± 4630 #
UC = U'C' = ± 2300 #
Ua = Ua' = − 6060 # or +2990 #
ab = a'b = − 3040 # or − 32 #
cd = c'd' = ± 3020 #
dd' = 0

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Weight of Portal Bracing

Top section	4 L's 4"×3"×5/16	@ 7.2	× 32'-6½	=	937#
Diagonals	8 L's 4"×3"×5/16	@ 7.2	× 14'-6"	=	835
Middle section	4 L's 4"×3"×5/16	@ 7.2	× 23'-5"	=	675
Diagonal laces	6 L's 3"×3"×5/16	@ 6.1	× 6'-7½	=	243
			Details say		1160
					3850#

2 Portals @ 3850 = 7700#

Weight of Sway Bracings

Top section	2 L's 4"×3"×5/16	@ 7.2	× 32'-6½	=	468
Bottom section	2 L's 4"×3"×5/16	@ 7.2	× 24'-0"	=	345
Diagonal Braces	4 L's 4"×3"×5/16	@ 7.2	× 13'-4"	=	385
Diagonals	6 L's 3"×3"×5/16	@ 6.1	× 6'-4"	=	231
		Details say 3			331
					1760#

5 Sways @ 1760 = 8800#

Summary for top laterals, Portal Bracings and Sways.

Top Laterals	6 @ 2180	= 13080
Portal Bracings	2 @ 3850	= 7700
Sway Bracings	5 @ 1760	= 8800
		29580#

29580 ÷ 165 = 179# per lin. ft. Call this 180# per lin ft

Main trusses

On account of simplicity of fabrication and also considering truss heights and sections of members we decided to adopt parallel chord type instead of curved chords.

Truss height 32'-0" Span length 165'-0" center to center of bearings, The spans is divided into 8 panels @ 20'-7½" each.

Approximate Deflection of truss

$$\text{Deflection at center of span} = \frac{6c + 6t}{2E}\left[\frac{\lambda^2 n(n+2)}{4h} + (n-2)h\right]$$

where
- $6c$ = Unit stress of Compression members 14.000#/□"
- $6t$ = Unit stress of tension members 14.000#/□"
- E = Modulus of Elasticity of Steel = 30.000.000#/□"
- λ = Panel length of truss = 20.625'
- h = depth of truss = 32.0'
- n = number of panels of truss = 8.0

$$\text{Deflection} = \frac{28000}{60.000.000} \times \left[\frac{20625^2 \times 80}{128} + 192\right] = 0.214' \text{ or } 2.57"$$

Use camber of truss lengthening the top chord by 3/8"

Dead load on truss

Lower Panel concentration

	For Ry truss	For Highway truss
Floor Beam Reaction	27200	42150
Lower Lateral Bracing	1140	1140
Truss assumed	7200	5950
	35540#	49240#

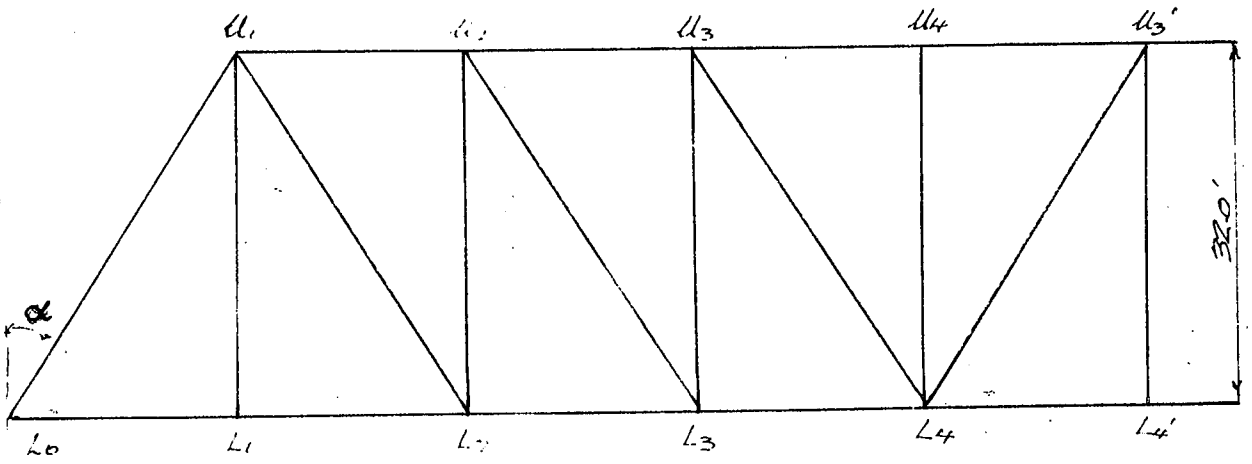
増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Dead Load on truss

Upper Panel Concentration

	For Railway truss	For Highway truss
Upper Lateals & Sway	1860	1860
Truss assumed	7000	6000
	8860#	7860#

Dead Load Stresses



8 panels @ 20'-7½" = 165'-0" c to c end bearings

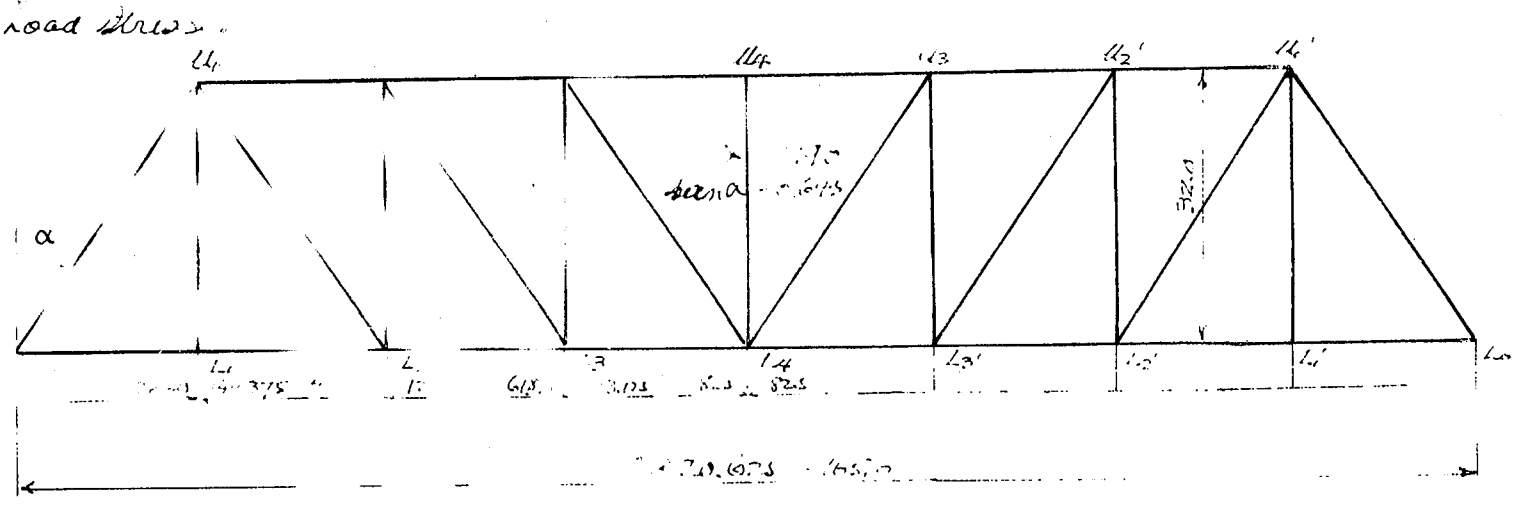
Length of diagonal = $\sqrt{20.625^2 + 32^2}$ = 38.07' Sec α = 1.170 tan α = 0.645

	For Railway truss	For Highway truss
Lower Panel Load	35540#	49240
Upper Panel Load	8860#	7860
Total	44400# 43.7%	57100# 56.3%

Stresses in members

Member		For Railway truss	For Highway truss
L_0U_1	End Post	184940#C	237830#C
U_1-U_2	Top chord	171840 C	220980 C
U_2-U_3	"	214800 C	276230 C
U_3-U_4	"	229120 C	294640 C
L_0-L_2	Bott. chord	100240 T	128910 T
L_2-L_3	"	171840 T	220980 T
L_3-L_4	"	214800 T	276230 T
U_1-L_1	Hanger	35540 T	49240 T
U_2-L_2	Vert Post	75460 C	93510 C
U_3-L_3	"	31060 C	36410 C
U_4-L_4		8860 C	7860 C
U_1-L_2	Diagonal	3[illegible]00 T	169880 T
U_2-L_3	"	1[illegible]2[illegible] "	101[illegible]30 T
U_3-L_4	"	[illegible]84420 T	33980 T

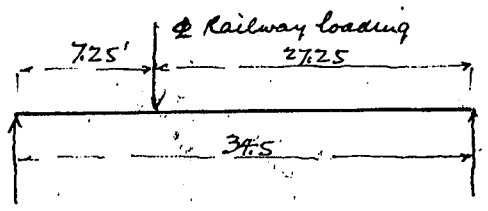
Live load Stresses



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Live Load stresses due to Electric Railway loading

For convenience's sake railway stresses figured as Cooper's E40 loading and reduced to E25 loading. The proportion of stresses on railway and highway trusses determined proportional to reactions on trusses



For Railway truss = $\frac{27.25}{34.5}$ = 79.0 %

For Highway truss = $\frac{7.25}{34.5}$ = 21.0 %

End Post L_0L_1. No 3 wheel on panel point L_1

E40 loading per rail total Reaction 185000# Stress = 185 000 × 1.190 = 220.000#

For railway truss 217.200 220.000 × $\frac{50}{40}$ = 275 000#

For Highway truss 57800

275000#

Bottom chord L_0-L_2 No 3 wheel on panel point L_1

Reaction E40 loading per rail = 185000# stress = 185000 × 0.645 = 119.000#

For E25 loading = 119000 × $\frac{25}{20}$ = 149.000#

For railway truss 117700

For Highway truss 31300

149 000#

Top chord U_1-U_2 and bottom chord L_2-L_3 No 5 wheel on L_2

stress E 40 loading per rail 195.400# For E25 loading 195400 × $\frac{25}{20}$ = 244200#

railway truss 192900

Highway truss 51300

244200#

Top chord U_2U_3 and bottom chord L_3-L_4 no 9 wheel on L_3

stress E40 loading per rail 236000# stress E25 loading 236000 × $\frac{25}{20}$ = 295000#

railway truss 233000

highway truss 62000

295 000#

Top chord U_3-U_4 No 12 wheel on L_4

stress E40 loading per rail 253500#

stress E25 loading 253500 × $\frac{25}{20}$ = 316900#

Railway truss = 250.400

Highway truss = 66 500

316 900#

Hanger U_1L_1 loading as shown on pp 7

Total Concentration on floor beam = 83850#

Railway truss = 66180

Highway truss = 17670

83850

U_1L_2 - diagonal No 3 load on L_2

stress E25 loading 209100#

Railway truss = 165200#

Highway truss = 43900#

209 100

U_2-L_2 No 3 wheel on L_3

stress E25 loading 125700#

Railway truss = 99300

Highway truss = 26400

125700

U_2-L_3 No 3 wheel on L_3

stress E25 loading 149600#

Railway truss = 118150

Highway truss = 31450

149600

U_3-L_3 No 3 wheel on L_4

stress E25 loading 82200#

Railway truss = 64900

Highway truss = 17300

82200#

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Diagonal U_3L_4 3rd wheel on L_4
Stress E25 loading 97800#

Railway truss = 77200
Highway truss = 20600
97800#

Diagonal U_3L_4 No2 wheel on L_3'
Stress E25 loading 56150

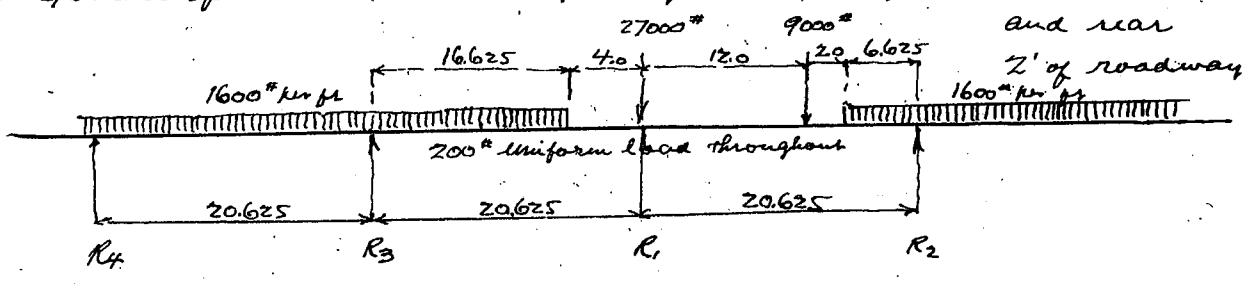
Railway truss = 44350
Highway truss = 11800
56150#

Live load Stresses due to Highway loadings.
Uniform live load = 100.0#/0'
2 motor trucks side by side on span. Impact 25%. throughout
motor trucks

Rear wheel	52400	
Impact 25%	1350	
4@	67.50# =	27000# for 2 trucks
Front wheel 4@	2250# =	9000# " " "

motor trucks assumed to occupy 16' of roadway toward center of bridge and uniform loads at front and rear
2' of roadway near highway truss to be filled by uniform load of 100# per square ft

Arrangement of live loads on highway



Panel concentration due to 200# per ft = 200 × 20.625 = 4125#
Uniform load 16.625' = 1600 × 16.625 = 26600#
Uniform load 6.625' = 1600 × 6.625 = 10600#
Uniform load 20.625 = 1600 × 20.625 = 33000#

Reaction R_1

Rear wheel motor trucks		27000
Front wheel motor trucks	9000 × 8.625/20.625 =	3770
Uniform load	26600 × 8.3125 ÷ 20.625 =	10700
Uniform load	10600 × 8.3125 ÷ 20.625 =	1700
		43170

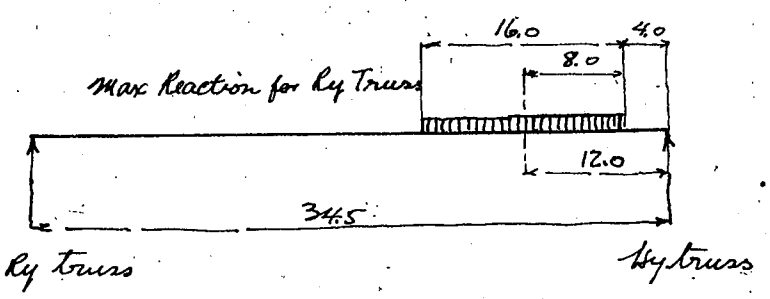
Reaction R_2

9000 × 12 ÷ 20.625	=	5230
10600 × 17.31 ÷ 20.625	=	8900
33000 ÷ 2	=	16500
		30630#

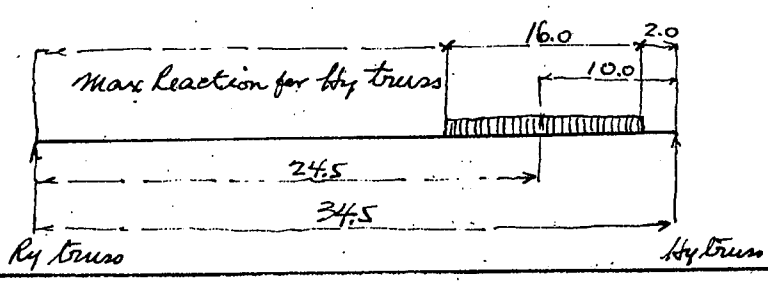
Reaction R_3

Uniform load	26600 × 12.3125 ÷ 20625 =	15900
Uniform load	33000 ÷ 2 =	16500
		32400#

Distribution of loading for 16' of roadway as figured in the following manner.

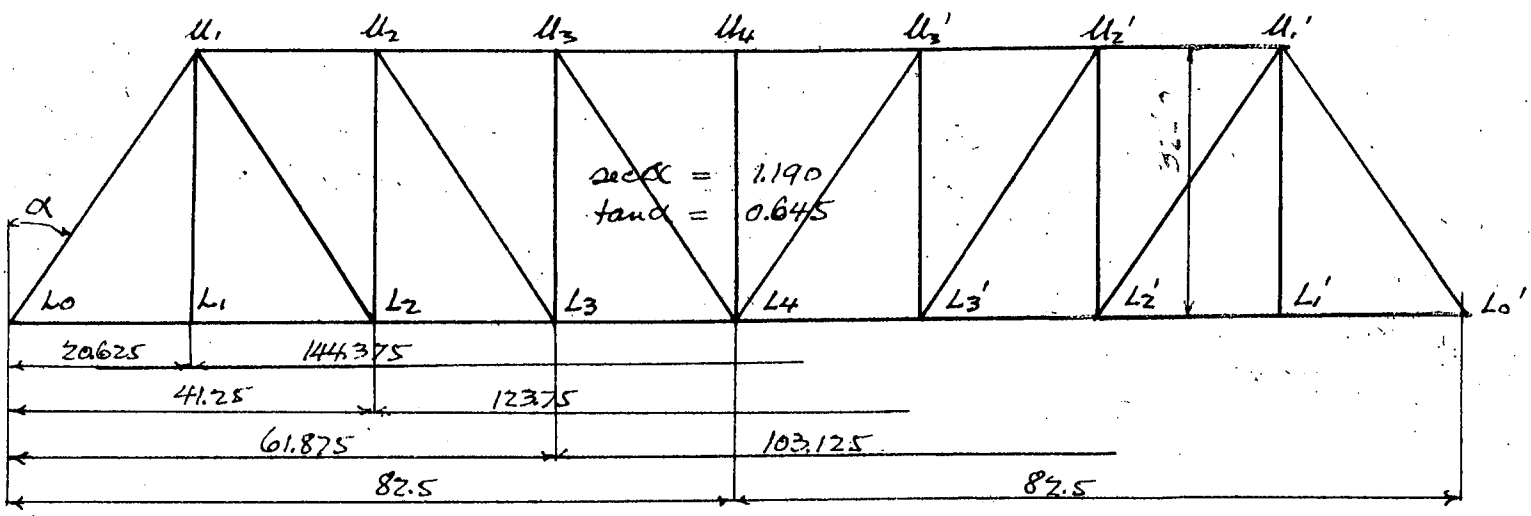


Max percentage of Reaction for railway truss = 12.0/34.5 = 34.8 %



Max. percentage of Reaction for highway truss = 24.5/34.5 = 71.0 %

増田淳氏関係資料
（独立行政法人 土木研究所蔵）



L_0-U_1

Panel point	loading	load		arm		
L_1	R_1	43170	×	144.375	=	6230.000
L_2	R_3	32400	×	123.75	=	4010.
L_3 to L_1'	R_4	33000	×	315.00	=	10400.

20640 ÷ 165 = 125 000

Stress = 125 000 × 1.190 = 148750 ----- Ry truss 34.8 % 51750 #
Hy truss 71.0 % 105600 #

L_0-L_1-L_2 Stress = 125 000 × 0.645 = 80625 ----- Ry truss 34.8 % 28000 #
Hy truss 71.0 % 57300 #

U_1U_2 and L_2L_3

Panel Point	loading	load		arm		
L_1	R_2	30630	×	144.375	=	4425 000
L_2	R_1	43170	×	123.75	=	5340 000
L_3	R_3	32400	×	103.125	=	3345 000
	R_4	33000	×	206.25	=	6820 000

19930 000 ÷ 165 = 120800 #

moment

120800 × 41.25 = 4.990.000
30630 × 20.625 = − 632.000
4.358.000 ÷ 32 = 131.000 # Ry truss 47400 #
Hy truss 96900 #

U_2-U_3 and L_3-L_4

Panel Pt	loading	load		arm		
L_2	R_2	30.630	×	123.75	=	3793 000
L_3	R_1	43170	×	103.125	=	4455 000
L_4	R_3	32400	×	82.50	=	2670 000
	R_4	33000	×	268.125	=	8860 000

19778.000 ÷ 165 = 119.900 #

moment

119900 # × 61.875 = 7.420.000
33000 × 41.25 = 1360.000
30630 × 20625 = 632.000
less 1.992.000
5.428.000 ÷ 32 = 169.600 #
Ry truss 59000 #
Hy truss 120400 #

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

U_3-U_4

Panel Point	loading	Load	Arm	
L_3	R_2	30630	103.125	= 3.165 000
L_4	R_1	43170	82.50	= 3560.000
L_3'	R_3	32400	61.875	= 2000.000
	R_4	33000	330.0	= 10.890.000

19615.000 ÷ 165 = 118900 #

moment

118900 . 82.5 = 9820.000

R_4	33000 . 103.125	=	3.403.000
R_2	30630 . 20.625	=	632.000
			4035.000

5.785.000 ÷ 32 = 180.400 #

Ry truss 62900 #

Hy truss 128100 #

U_1-L_1

max Reaction at L_1 = 43170 #

Ry truss = 15.000 #

Hy truss = 30600 #

U_1-L_2

Panel Point	loading	Load	Arm	
L_1	R_2	5.230	144.375	= 756.000
L_2	R_1	43.170	123.75	= 5340.000
L_3	R_3	32.400	103.125	= 3345.000
	R_4	33.000	206.25	= 6.820.000

16.261.000 ÷ 165 = 98.550. #

Less 5230

Panel shear 93320 #

stress U_1L_2 = 93320 . 1.190 = 111.000 #

Ry truss 38600 #

Hy truss 78800 #

U_2-L_2 and U_2L_3

Panel Pt	loading	Load	Arm	
L_2	R_2	5.230	123.75	= 648 000
L_3	R_1	43.170	103.125	= 4455 000
L_4	R_3	32.400	82.50	= 2.670.000
	R_4	33.000	123.75	= 4084.000

11.857.000 ÷ 165 = 71.880

5.230

stress U_2-L_2 66.650 #

Ry truss 23200 #

Hy truss 47300 #

stress U_2-L_3 = 66650 . 1.190 = 79.300 # ------ Ry truss 27600 #

Hy truss 56300 #

U_3-L_3 and U_3L_4

Panel Pt	loading	Load	Arm	
L_3	R_2	5.230	103.125	= 540.000
L_4	R_1	43170	82.50	= 3560.000
L_3'	R_3	32400	61.875	= 2000.000
	R_4	33000	61.875	= 2040.000

8140.000 ÷ 165 = 49.330 #

less 5230

44100 #

Ry truss = 15340 #

Hy truss = 31300 #

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Stress U_4-L_4 = 44100 · 1.190 = 52500# ---- Ry truss 18.260#
Hy truss 37300#

U_3'-L_4

Panel Pt	loading	load	Arm	
L_4	R_2	5230 .	82.5 =	432.000
L_3'	R_1	43170 .	61.875 =	2670.000
L_2	R_3	32400 .	41.25 =	1.337.000
L_1	R_4	33000 .	20.625 =	681.000
				5120.000 ÷ 165 = 31,030
				5.230
				25.800#

Stress U_3'-L_4 = 25.800 · 1.190 = 30.700# ..... Ry truss 10680#
Hy truss 21800#

Stresses due to highway loading
Panel concentration 4125# = W

2' strip on Roadway
W tan α = 4125 · 0.645 = 2.660
W sec α = 4125 · 1.190 = 4.910

For Ry truss = $\frac{3}{34.5}$ = 8.7%

For Hy truss = $\frac{19}{34.5}$ = 55.2%

	member	Stress	Ry truss	Hy truss
End Post	L_0U_1	17.185 C	1490 C	7720 C
Top chord	U_1-U_2	15.960 C	1390 C	7180 C
" "	U_2-U_3	19950 C	1740 C	8970 C
" "	U_3-U_4	21280 C	1850 C	9570 C
Bott chord	L_0-L_2	9310 T	810 T	4190 T
" "	L_2-L_3	15960 T	1390 T	7180 T
" "	L_3-L_4	19950 T	1740 T	8970 T
Hanger	U_1-L_1	4125 T	358 T	1850 T
Post	U_2-L_2	7734 C	672 C	3480 C
Post	U_3-L_3	5156 C	448 C	2320 C
diagonals	U_1-L_2	12889 T	1120 T	5790 T
"	U_2-L_3	9206 T	800 T	4140 T
"	U_3-L_4	6138 T	533 T	2760 T
"	U_3'-L_4	3683 C	320 C	1.660 C

Summary stresses due to highway loadings

		For Railway truss			For highway truss		
		Load on 16' strip	Load on 2' strip	Summary	Load on 16' strip	Load on 2' strip	Summary
End Post	L_0-U_1	51800 C	1500 C	53300 C	105600 C	7700 C	113300 C
Top chord	U_1-U_2	47400 C	1400 C	48800 C	96900 C	7200 C	104100 C
" "	U_2-U_3	59000 C	1800 C	60800 C	120400 C	9000 C	129400 C
" "	U_3-U_4	62900 C	1900 C	64800 C	128100 C	9600 C	137700 C
Bott. Chord	L_0-L_2	28000 T	800 T	28800 T	57300 T	4200 T	61500 T
" "	L_2-L_3	47400 T	1400 T	48800 T	96900 T	7200 T	104100 T
" "	L_3-L_4	59000 T	1800 T	60800 T	120400 T	9000 T	129400 T
Hanger	U_1-L_1	15000 T	400 T	15400 T	30600 T	1900 T	32500 T
Post	U_2-L_2	23200 C	700 C	23900 C	47300 C	3500 C	50800 C
"	U_3-L_3	15400 C	500 C	15900 C	31300 C	2300 C	33600 C
diagonal	U_1-L_2	38600 T	1100 T	39700 T	78800 T	5800 T	84600 T
"	U_2-L_3	27600 T	800 T	28400 T	56300 T	4200 T	60500 T
"	U_3-L_4	18300 T	500 T	18800 T	37300 T	2800 T	40100 T
"	U_3'-L_4	10700 C	300 C	11000 C	21800 C	1700 C	23500 C

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Live load stresses due to E25 loading including impact.

		Coefficient of impact	Stresses in Ry truss without impact	Stresses in Ry truss with impact	Stresses in Sy truss without impact	Stresses in Sy truss with impact
End Post	L_0-U_1	0.3225	217200 C	287000 C	57800 C	76600 C
Topchord	U_1-U_2	do	192900 C	255000 C	51300 C	68000 C
" "	U_2-U_3	do	233000 C	308000 C	62000 C	82000 C
" "	U_3-U_4	do	250400 C	331500 C	66500 C	88000 C
Bott chord	L_0-L_2	do	117700 T	155800 T	31300 T	41400 T
" "	L_2-L_3	do	192900 T	255000 T	51300 T	68000 T
" "	L_3-L_4	do	233000 T	308000 T	62000 T	82000 T
Hanger	U_1L_1	0.440	66180 T	95500 T	17670 T	25300 T
Post	U_2L_2	0.361	99300 C	135100 C	26400 C	36000 C
"	U_3L_3	0.380	64900 C	89600 C	17300 C	23900 C
diag.	U_1-L_2	0.343	165200 T	222100 T	43900 T	59100 T
"	U_2-L_3	0.361	118150 T	161000 T	31450 T	42900 T
"	U_3-L_4	0.380	77200 T	106600 T	20600 T	28400 T
"	U_3'-L_4	0.406	44350 C	62400 C	11800 C	16600 C

For summary stresses of Dead and live loads combined see pages 19 and 20

Load on bearing shoe

End stringers rest on cross beam of pier ; let us figure load on bearing from the stress in End Post

Stresses in End Post

	For Ry truss	For Sy truss
Dead Load	184940	237830
Railway Live Load	287000	76600
Highway live load	53300	113300
	525240 #	427730 #
vertical component	442000 #	360.000 #

Use same details of shoes for both railway and highway trusses.

Size of base pl = 44" × 34" = 1496 □"

Unit bearing pressure = $\frac{442.000}{1496}$ = 296 #/□" ok

For roller End Use 6" roller 6×600 = 3600# per inch

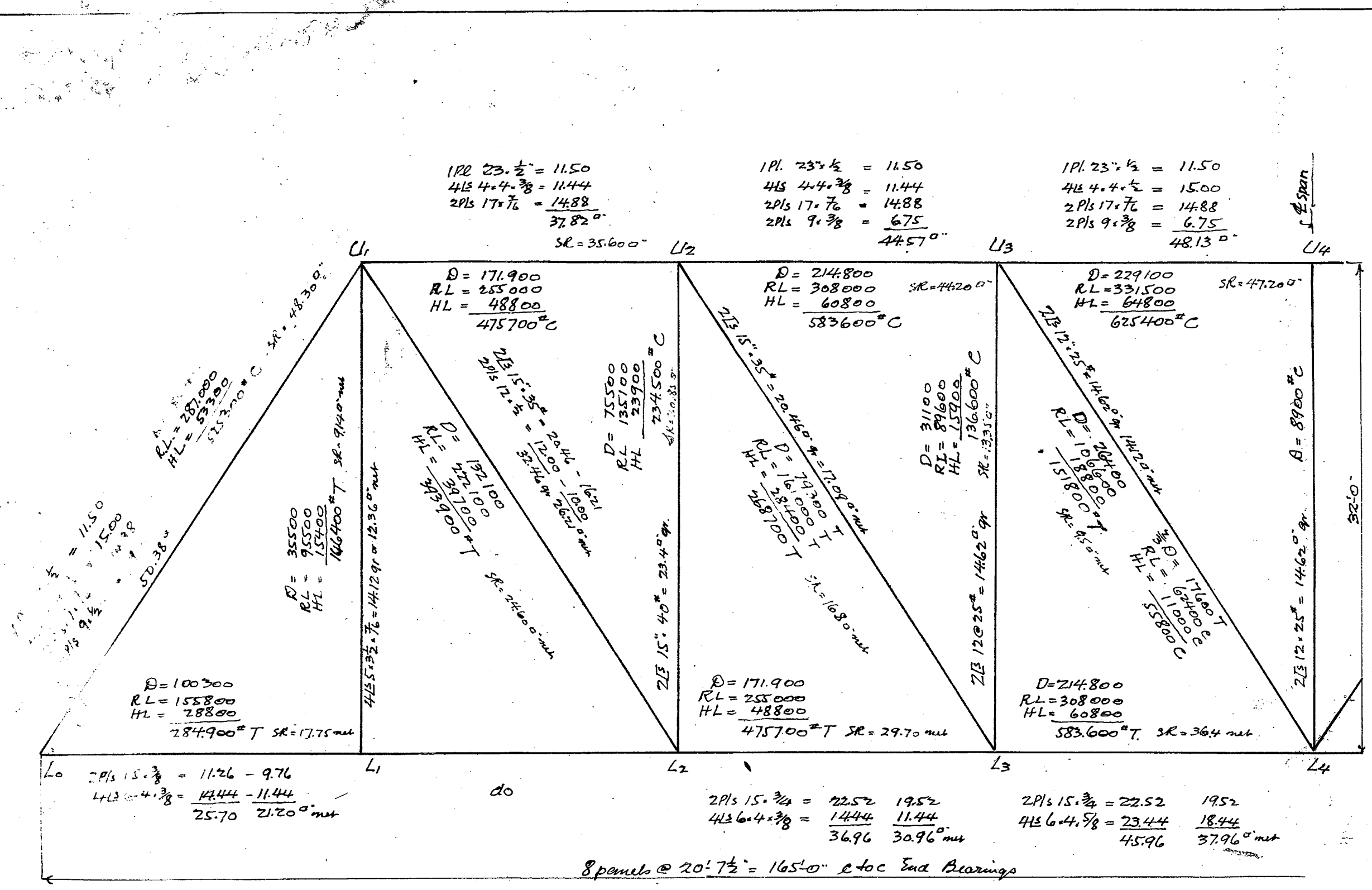
Required length of roller = 123 inch or 5 @ 24.6" use at least

Assume 7" pin thickness of pin plate $1\frac{1}{16}$" Unit bearing = $\frac{442.000}{7 \cdot 1.\frac{1}{16} \times 2}$ = 18700 #

moment on pin = 221.000 · 1.937 = 427000 "# 7" pin good for of bending moment of 1.051000 "# ok

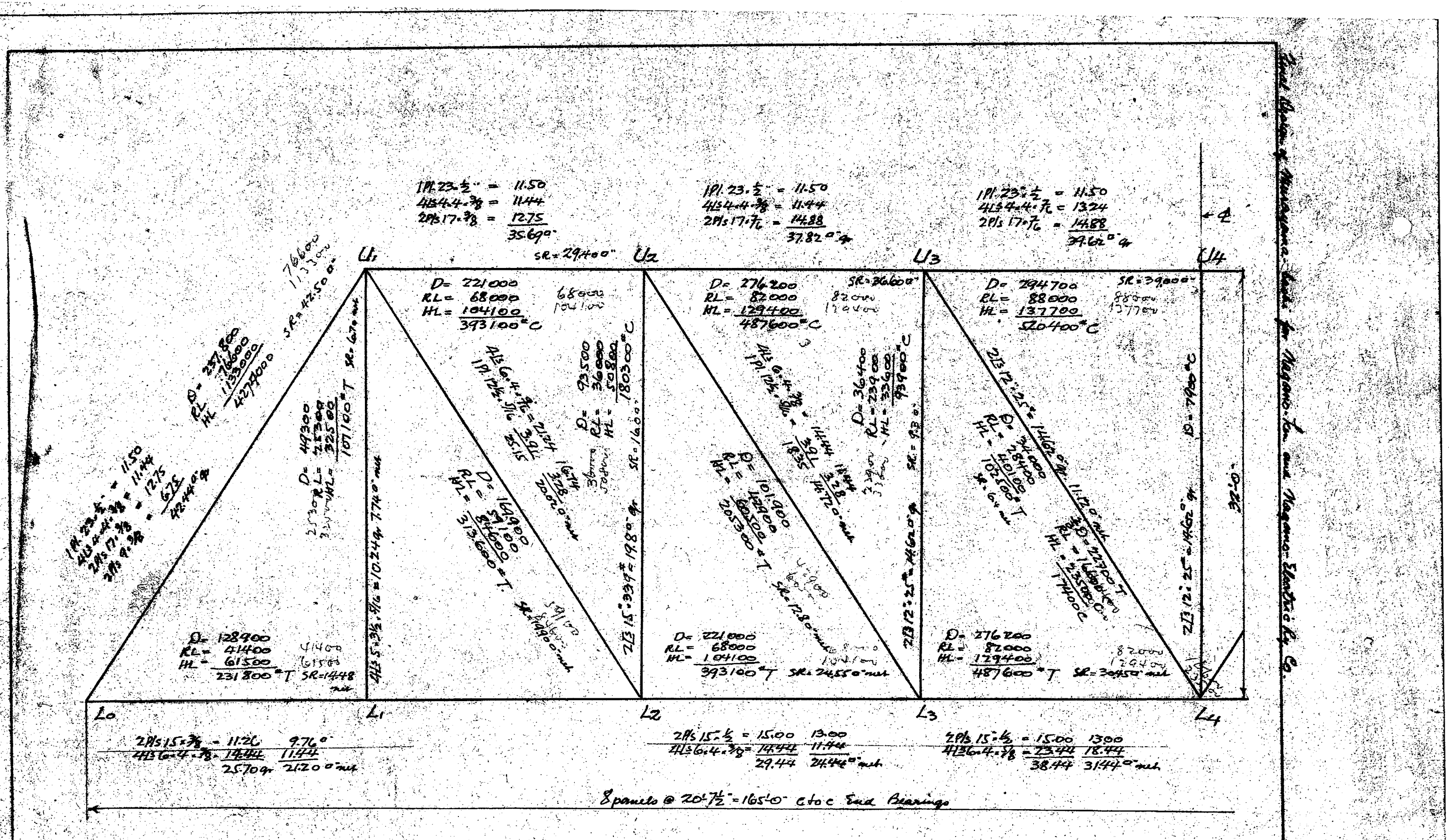
増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Final Design of Niagara-Lockport and Ontario Power Co



19

増田淳氏関係資料
(独立行政法人土木研究所蔵)
20



Stresses and Sections of Highway Truss.

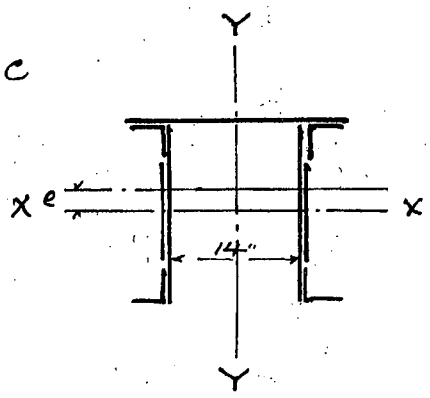
増田淳 民間採鉱論 (独立行政法人 土木研究所蔵)

Sections of members.

End Post L0 U_1 Total Direct Stress = 525300 #C

section of member Railway truss

1 cover pl	23 × ½"	=	11.50
4 L's	4×4×½"	=	15.00
2 P/s	17 × 7/16"	=	14.38
2 P/s	9 × ½"	=	9.00
			50.38 □"



eccentricity = 2.03"

moment of inertia XX axis = 1.953

Radius of gyration XX axis = 6.22"

moment of inertia YY axis = 2929

Radius of gyration YY axis = 7.62

allowable unit stress = 10850 #/□"

Section Required = 525300 ÷ 10850 = 48.300 □"

Unit stress YY axis = 16000 − 4200 = 11800 #/□"

For wind stress increasing 25% = 15950 #/□"

Bending Stress = $\frac{80.340 \times 12 \times 11.5}{2929}$ = 3780

15950

11170 #/□"

Section Required = 525300 ÷ 11170 = 47.20 □"

End Post L0 U_1 Highway Truss S = 427700 #C

1 pl	23 × ½	=	11.50
4 L's	4×4×⅜	=	11.44
2 P/s	17 × ⅜"	=	12.75
2 P/s	9 × ⅜"	=	6.75
			42.44

Unit stress = 16000 − 5120 = 10880 #/□"

section req'd = 427700 ÷ 10880 = 39.40 □"

Eccentricity = 2.4"

Moment of Inertia XX axis = 1658

Radius of gyration do = 6.25

moment of Inertia YY axis = 2385

Radius of gyration YY axis = 7.5"

Unit stress YY axis = 16000 − 4260 = 11740 #/□"

Increasing 25% for wind stress = 14700 #/□"

Bending stress = $\frac{80.340 \times 12 \times 11.5}{2.385}$ = 4650

14700

10050#

section required = 427700 ÷ 10050 = 42.50 □"

Railway truss U_1-U_2 S = 475700 #C

1 Pl	23 × ½	=	11.50
4 L's	4×4×⅜	=	11.44
2 P/s	17 × 7/16	=	14.88
			37.82 □"

Eccentricity = 2.7" moment of inertia = 1632

r = 6.56"

Unit stress = 16000 − 2640 = 13360 #/□"

section required = 475700 ÷ 13360 = 35.60 □"

Highway truss U_1-U_2 S = 393100 #C

1 Pl	23 × ½	=	11.50
4 L's	4×4×⅜	=	11.44
2 P/s	17 × ⅜"	=	12.75
			35.69

Eccentricity = 2.86" moment of inertia = 1565

r = 6.6

Unit stress = 16000 − 2620 = 13380 #/□"

section Required = 393100 ÷ 13380 = 29.40 □"

Railway truss U_2-U_3 S = 583600 #C

1 Pl	23 × ½	=	11.50
4 L's	4×4×⅜	=	11.44
2 P/s	17 × 7/16	=	14.88
2 P/s	9 × ⅜	=	6.75
			44.57

Eccentricity = 2.29" I = 1720 r = 6.2"

Unit stress = 16000 − 2.800 = 13200

section required = 583600 ÷ 13200 = 44.20 □"

Highway truss U_2-U_3 S = 487600 #C

1 Pl	23 × ½	=	11.50
4 L's	4×4×⅜	=	11.44
2 P/s	17 × 7/16	=	14.88
			37.82 □"

Eccentricity =

Unit stress = 13360 #/□"

section required = 487600 ÷ 13360 = 36.60 □"

Railway truss U_3-$U_{3'}$ 625400 # C

1 Pl	23 × ½	=	11.50
4 L's	4×4×½"	=	15.00
2 Pl's	17 × 7/16	=	14.88
2 Pl's	9 × ⅜	=	6.75
			48.13

Eccentricity = 2.12" I = 1928 r = 6.32

Unit stress = 16000 − 2740 = 13260 #/□"

section required = 625400 ÷ 13260 = 47.20 □"

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Highway truss [illegible] S = [illegible]

1 Pl [illegible] 11.50 [illegible]centricity = 174. r = 6.63

4 L's 4x4 [illegible] 3.24 Stress = 16000 13380 #

[illegible] 17.7 lb 38 [illegible]uired = 13380 = [illegible]

Railway truss [illegible] L2 S

2 [15" [illegible] 73.40 #

r = 5.44

[illegible] Stress = 16000 [illegible] = 11050 [illegible]uired = 230.4

Highway truss [illegible] S = 11[illegible]

2 [15" x 33# [illegible] 180 0"

r = 5.62

Unit Stress = 16000 [illegible] = 11210 #. [illegible] uired = 1712 [illegible] 16.0

Railway truss U_3-L_3 S = 132900 #

2 [12" 25# = 146.4 0"

r = 4.43

Unit stress = 16000 - 6070 = 9930 #/0. Section required = 132900 ÷ [illegible] 13350

Highway truss U_3-L_3 S = 92000 #

2 [12" 25# = 14.64

r = 4.43

Unit stress = 16000 - 6070 = 9930 #/0. Section required = 92000 ÷ 9930 = 9.300

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Final Design of Murayama-bashi for Nagano-ken and Nagano Electric Ry Co

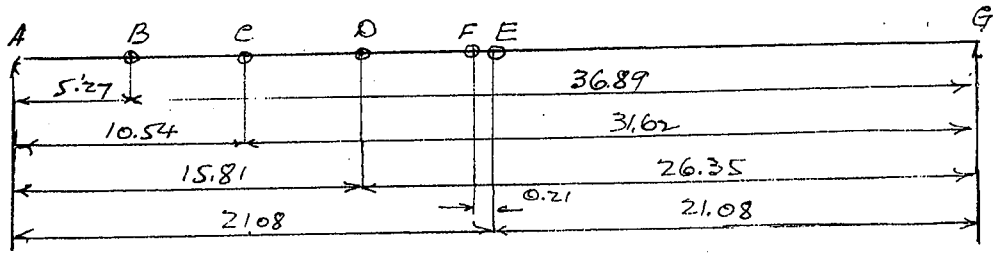
Design of plate girder span for West approach

Loading Cooper's E25 impact coefficient = $\frac{300}{300+l} \times \frac{30}{60}$ where l = loaded length in ft.

Clear span 40'-0" Center to Center of bearings = 42'-2" ; Girder spacing = 4'-0"

Dead Load

Deck assumed	300# per lin ft
Own weight	440
	740 ÷ 2 = 370# per lin ft for one girder.



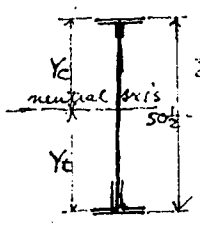
Impact = 0.437

Moments	Point B	Point C	Point D	Point E	Point F
Dead Load moment	431.600 ft#	739900 ft#	924800 ft#	986500 ft#	986400 ft#
Live Load Moment	2480.500	4150800	5102600	5386100	5387000
Impact.	1084.000	1813900	2229800	2353700	2354000
Total moment	3996100	6704600	8257200	8726300	8727400

Shears	Point A	Point B	Point C	Point D	Point E
Dead Load	7800	5900	3900	2000	0
Live Load	48900	39200	30000	21600	14300
Impact	21400	17100	13100	9400	6300
Summary	78100#	62200#	47000#	33000#	20600#

Plate Girder section -

1 Pl.	12 × 7/16	= 5.25 qr	5.25	
2 L 3	5 × 3½ × 7/16	= 7.06	7.06	
1 Pl	50 × 3/8	= 18.75	13.83	
2 L 3	5 × 3½ × 7/16	= 7.06	5.53	
1 Pl	12 × 7/16	= 5.25	4.48	
			36.15 □" net	



back to back of L's 50½"
Eccentricity due to reduction of net holes 1.56"
Yt = 27.25" Ye = 24.13
moment of inertia = 16.460 in⁴

actual stress $f_t = \frac{8727400 \times 27.25}{16.460} = 14400$ #/□" ok allowable stress = 16000 #/□"

$f_c = \frac{8727400 \times 24.13}{16.460} = 12800$ #/□" ok allowable stress = $16000 - 200 \frac{112}{12} = 14100$ #/□" ok

Deflection at center of span

$\max \delta = \frac{5}{24} \times \frac{f}{E} \times \frac{l^2}{h}$ $f = 14400$ #/□" $l = 506$" $E = 30.000.000$ #/□" $h = 50.5$"

$\max \delta = \frac{5}{24} \times \frac{14400 \times 506^2}{30.000.000 \times 50.5} = 0.507"$ or $\frac{0.507}{506} = \frac{1}{1000}$

Shoes Bearing Area = 16×16 = 256 □" Unit bearing = $\frac{78100}{256} = 305$ #/□"

Use standard details as for 鉄道省 bridges.

Use 2 End frames at ends and 1 Intermediate frame at center of span.
Provide lateral bracings of 1 L 3.3. 5/16

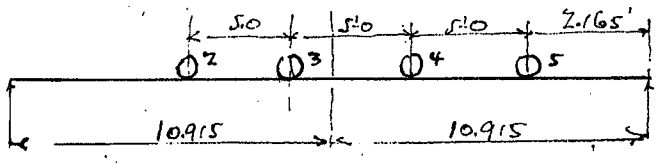
増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Deck plate girder span on west approach
Loading Cooper's E25 impact coefficient = $\frac{300}{300+l} \times \frac{30}{60}$ where l = loaded length in ft
Span length 21'-10" c to c of bearings girder spacing 4'-0"
Dead Load moment

Deck assumed	200
girder + bracings	340
	540 ÷ 2 = 270# per ft per girder

Dead load moment at centre = $\frac{1}{8} \cdot 270 \cdot 21.83^2 \times 12 = 193.000$ in#

Live Load Moment



max moment
$= \left[\frac{600 + 80 \times 2.165}{21.83} \times 9.665 - 100\right] \cdot \frac{25}{40} \times 12000 =$
$= 1.808.000$ in# reduced from E40 loading.

Impact = $1\,808\,000 \cdot 0.466 = 842.000$ in#.

Summary for moment

Dead Load m	193 000
Live Load m	1808 000
Impact	842 000
	2843.000 in#

Max End shear

Dead Load	$\frac{1}{2} \cdot 270 \times 21.83 =$	2950
Live Load	$20 + \frac{300 + 60 \cdot 683}{21.83} \times 0.625 \times 1000 =$	32.800
Impact	$32800 \cdot 0.466 =$	15300
	Tot.	51050#

Web assumed $30 \cdot \frac{3}{8} = 11.250$" $\frac{1}{8}$ web = 1.41 Effective depth of girder = 28.8"
Stress in flange = $2843.000 \div 28.8 = 99000$# Section required = $99000 \div 16000 = 6.2$ sq in
For girder flange = $6.2 - 1.41 = 4.79$ " sq in
Use 2 L 5 × 3½ · ⅜ = 6.10" gross 5.44" net OK

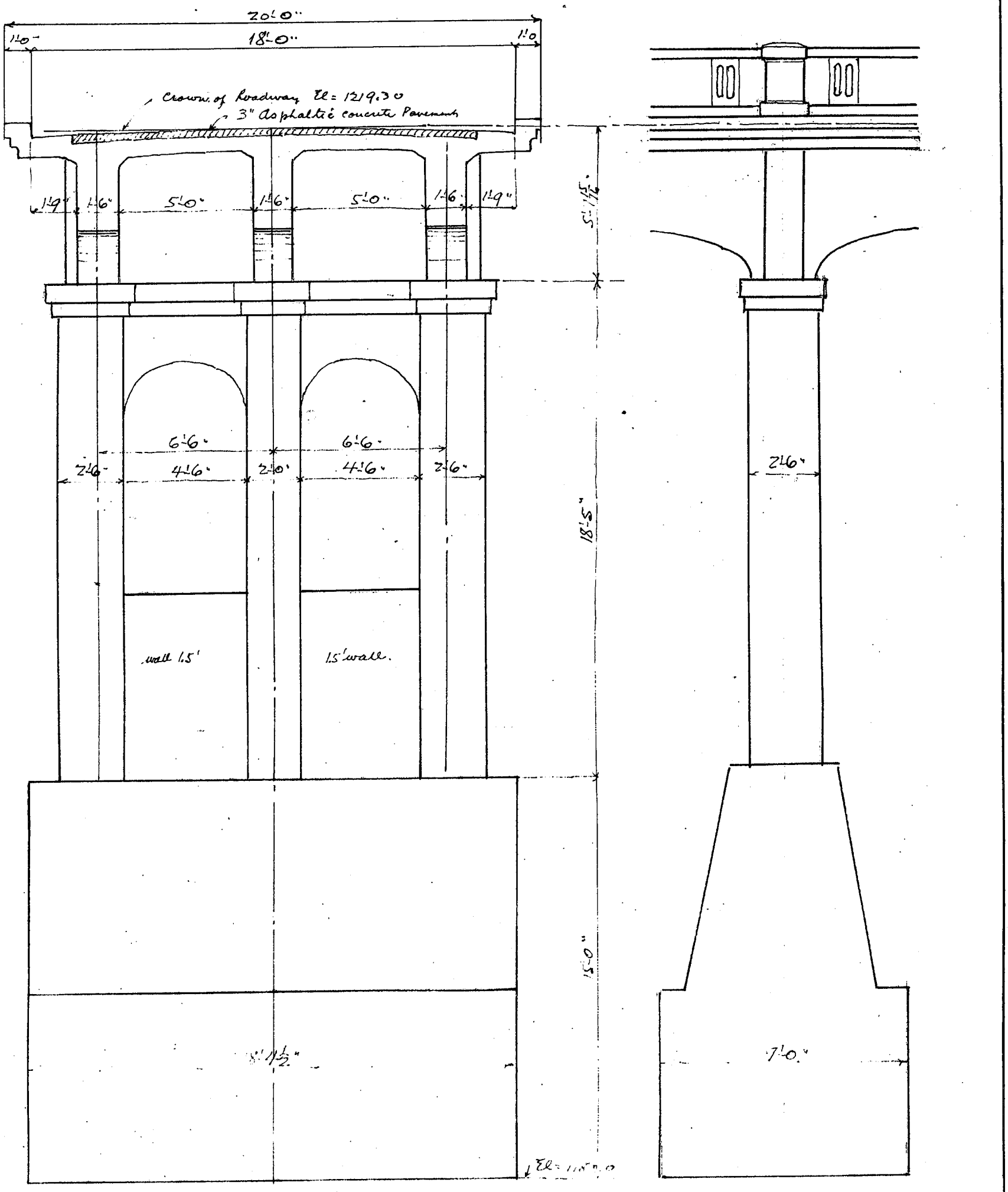
Max deflection = $\frac{5}{24} \cdot \frac{5 \text{ sq } 16000}{30.000.000} \cdot \frac{262^2}{30.5} = 0.25$" $\frac{0.25}{262} = \frac{1}{1050}$

Use same bearing shoe as for 40' span which is ample for bearing
Use lateral bracing as for 40' span with end frames and 1 intermediate frame.

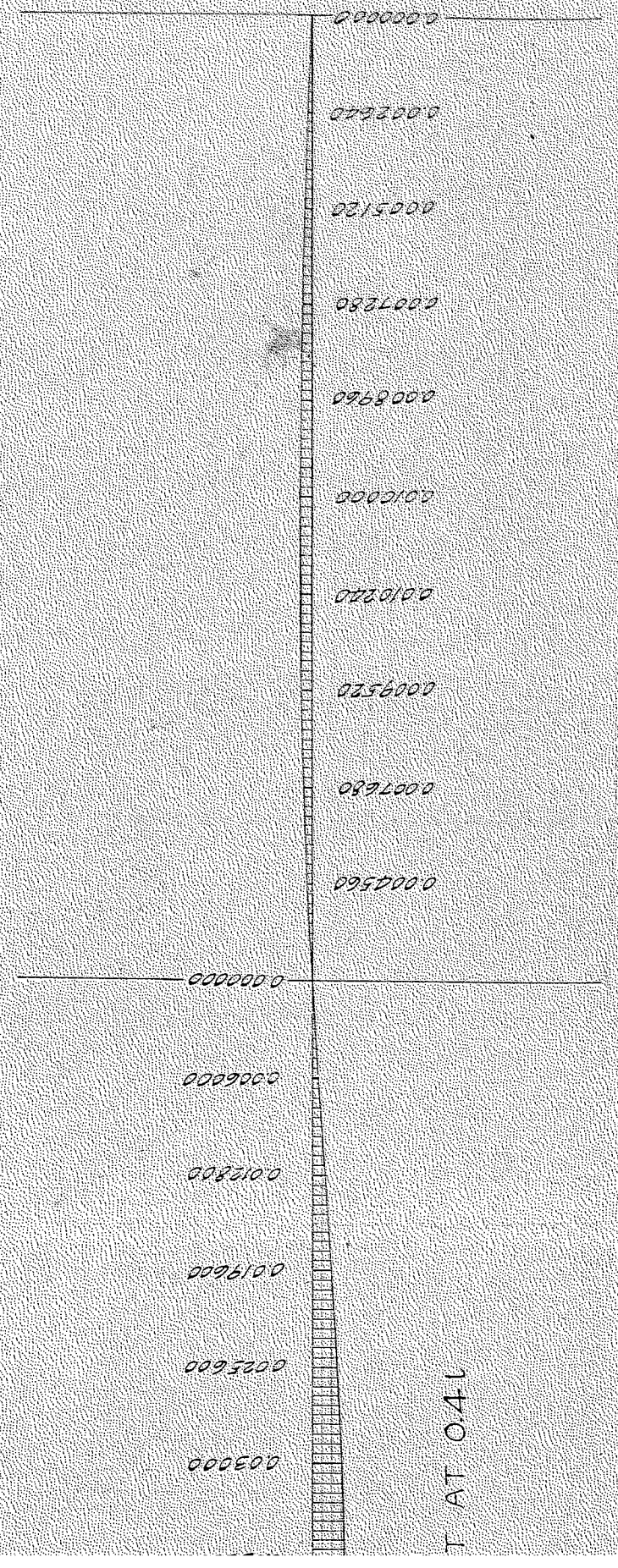
増田淳氏関係資料
（独立行政法人 土木研究所蔵）

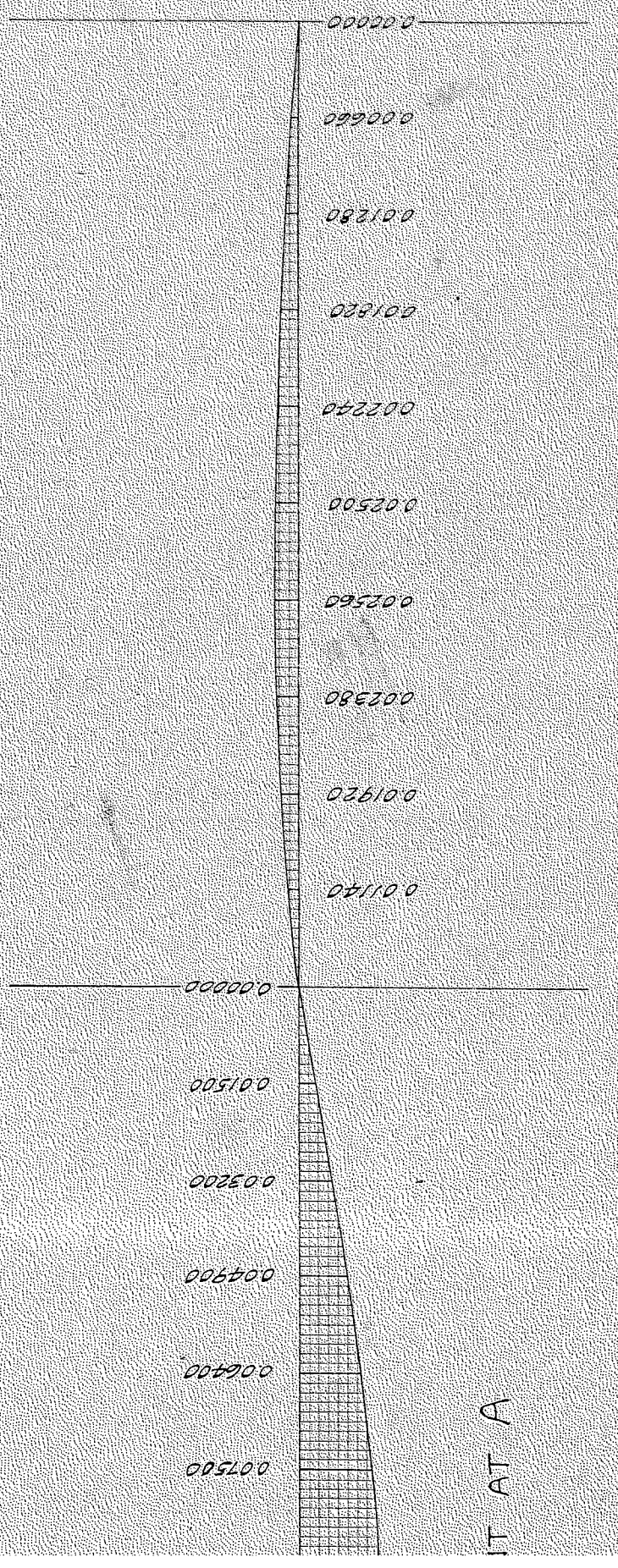
Final Design of "Murayama-Bashi" for Nagano-Ken and Nagano-Electric Ry Co.

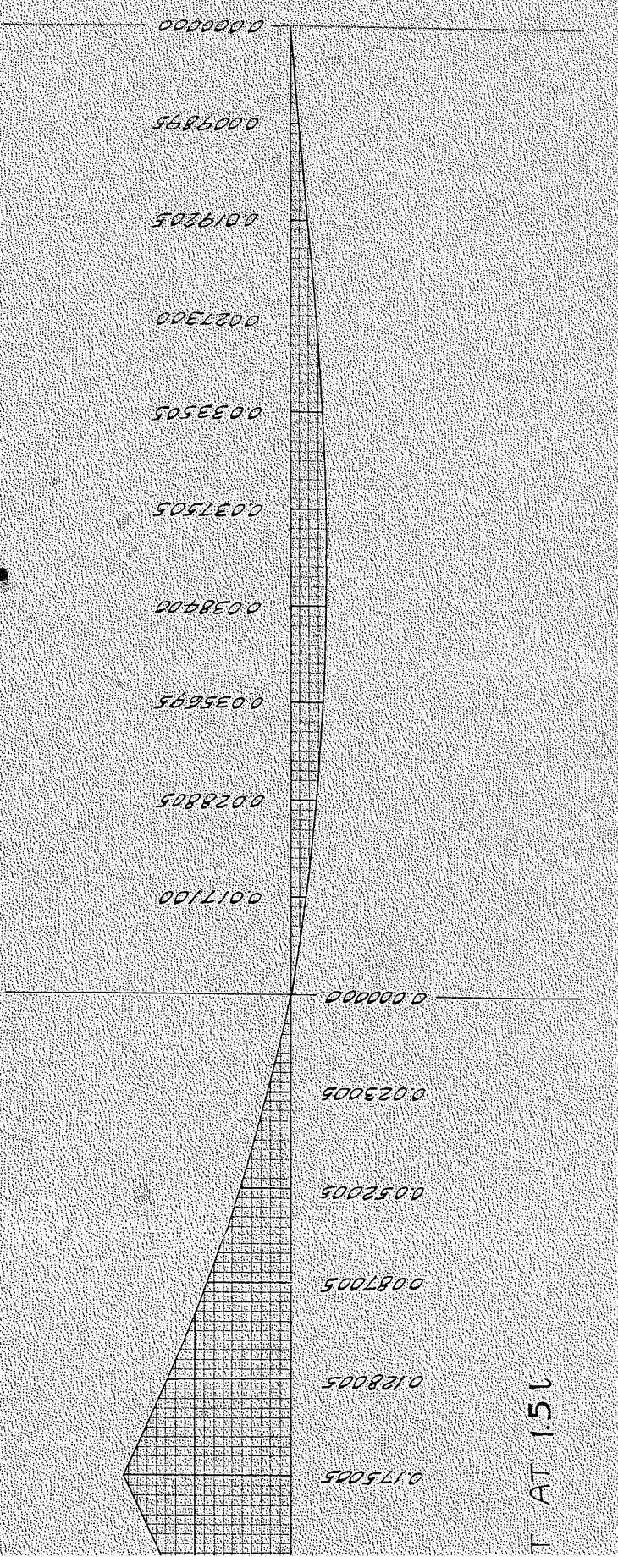
Design of highway Approach span 3-44'-6" continuous span of reinforced concrete
Cross section of structure as shown on sketch below.

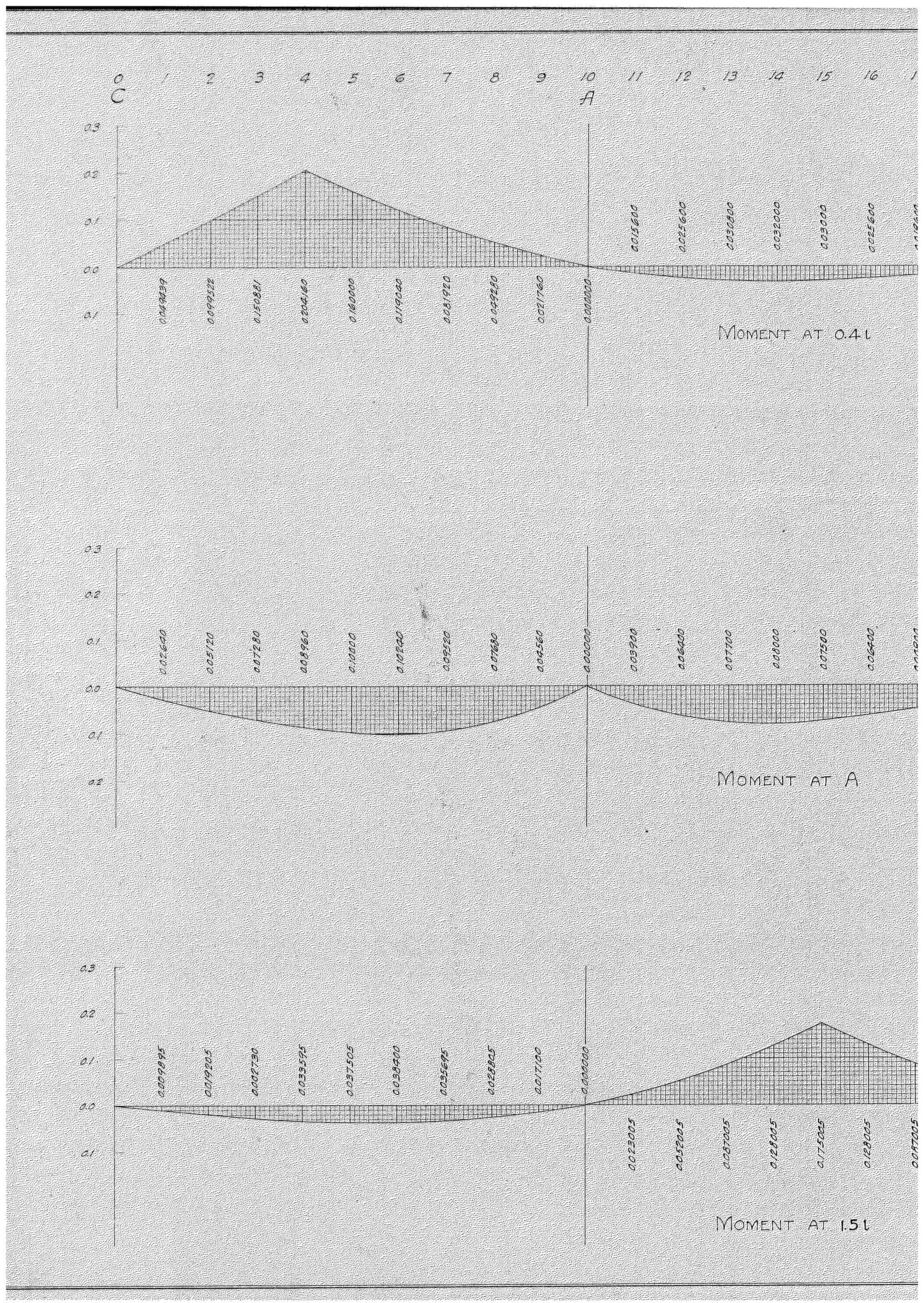


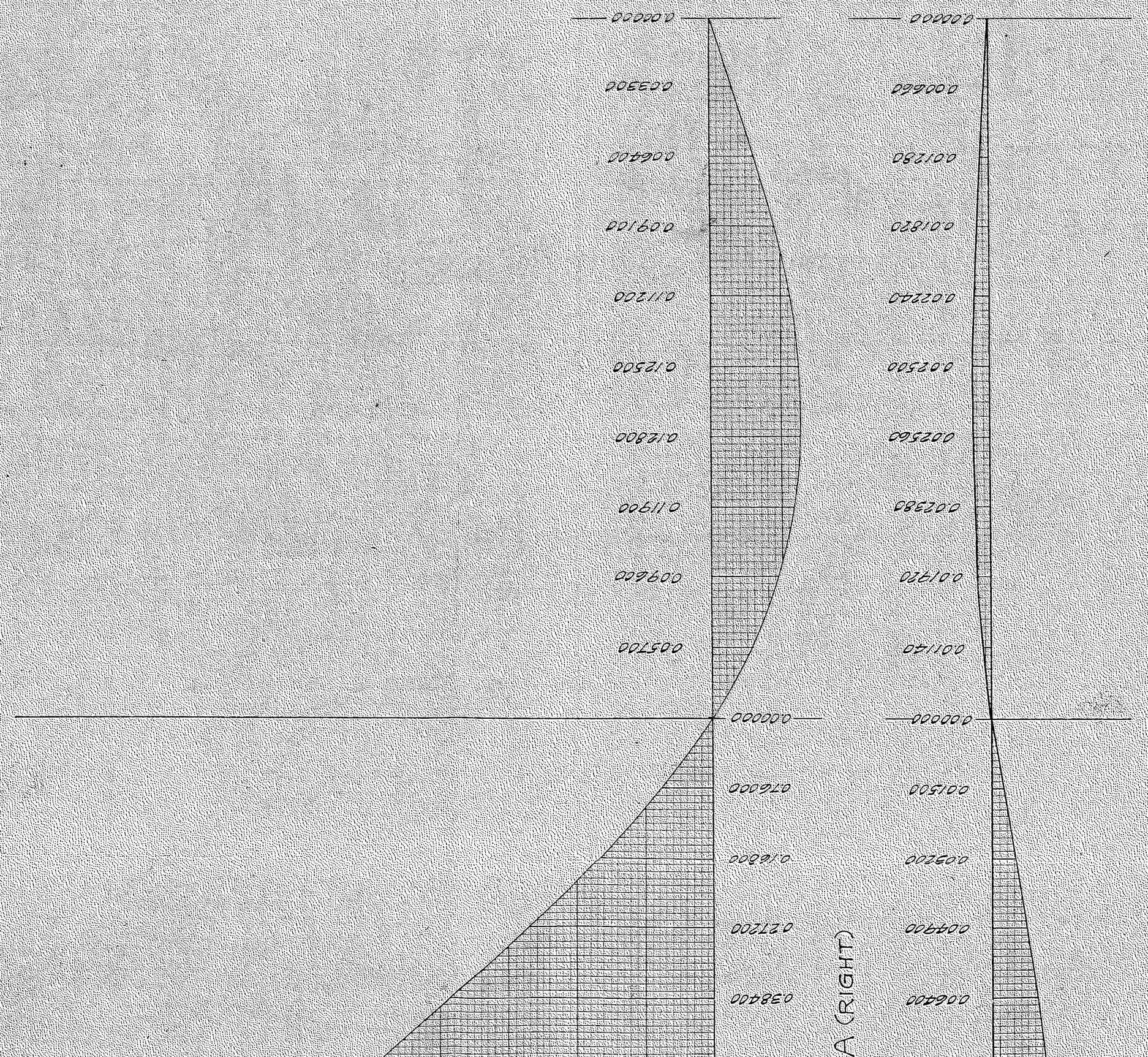
増田淳氏関係資料
（独立行政法人 土木研究所蔵）

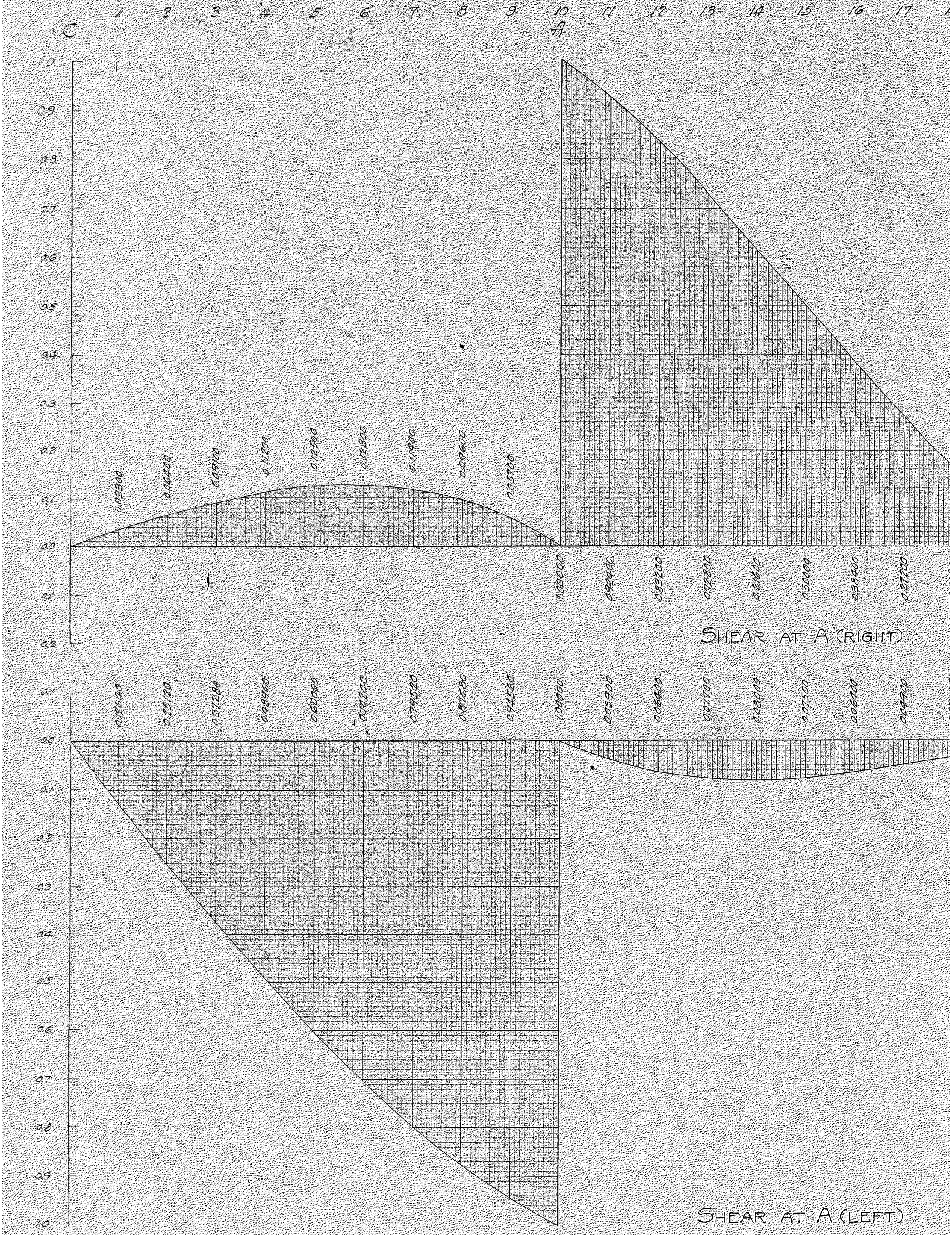

0.000000
0.000000
0.002640
0.005120
0.007280
0.008960
0.010000
0.010220
0.009520
0.007680
0.004560
0.000000
0.006000
0.012800
0.019600
0.025600
0.030000
30
29
28
27
26
25
24
23
22
21
20
19
18
17
16
15
D
B
PLOT AT 0.4L


0.000000
0.000000
0.006680
0.012680
0.018620
0.022740
0.025500
0.025680
0.023780
0.019120
0.011420
0.000000
0.001500
0.003200
0.004900
0.006400
0.007500
PLOT AT A


0.000000
0.000000
0.009895
0.019205
0.027300
0.033505
0.037505
0.038400
0.035695
0.028805
0.017100
0.000000
0.002300
0.005200
0.008700
0.012800
0.017500
PLOT AT 1.5L


0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16
C
A
0.3
0.2
0.1
0.0
0.1
0.026400 0.005120 0.012800 0.008960 0.010000 0.010240 0.009520 0.007680 0.005760 0.000000 0.003600 0.004400 0.007200 0.009000 0.007500 0.004800
Moment at 0.4 l
0.3
0.2
0.1
0.0
0.1
0.2
0.026400 0.005120 0.012800 0.008960 0.010000 0.010240 0.009520 0.007680 0.005760 0.000000 0.003600 0.004400 0.007200 0.009000 0.007500 0.004800
Moment at A
0.3
0.2
0.1
0.0
0.1
0.000625 0.001205 0.002250 0.003595 0.003750 0.003200 0.003595 0.002805 0.001100 0.000000 0.002205 0.005625 0.009000 0.012805 0.017505 0.012805 0.008000
Moment at 1.5 l


A (LEFT)
A (RIGHT)
B
C
D
30
29
28
27
26
25
24
23
22
21
20
19
18
17
16
15
0.00000
0.00660
0.01280
0.01820
0.02240
0.02500
0.02560
0.02380
0.01920
0.01120
0.00000
0.01500
0.03200
0.04900
0.06400
0.07500
0.08120
0.08400
0.08300
0.07600
0.06400
0.04700
0.02500
0.00000


C
A
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17
1.0 0.9 0.8 0.7 0.6 0.5 0.4 0.3 0.2 0.1 0.0 0.1 0.2
0.03600 0.07200 0.10800 0.14400 0.18000 0.21600 0.25200
1.00000 0.96400 0.92800 0.89200 0.85600 0.82000 0.78400 0.74800
SHEAR AT A (RIGHT)
0.11800 0.15400 0.19000 0.22600 0.26200 0.29800 0.33400 0.37000 0.40600 0.44200 0.47800
1.00000 0.96400 0.92800 0.89200 0.85600 0.82000 0.78400 0.74800
SHEAR AT A (LEFT)

E

0.5750
0.4490
0.3210
0.2000
0.0910
0.0000
0.0684
0.1152
0.1428
0.1536
0.1500
0.1311
0.1092
0.0768
0.0396
0.0000

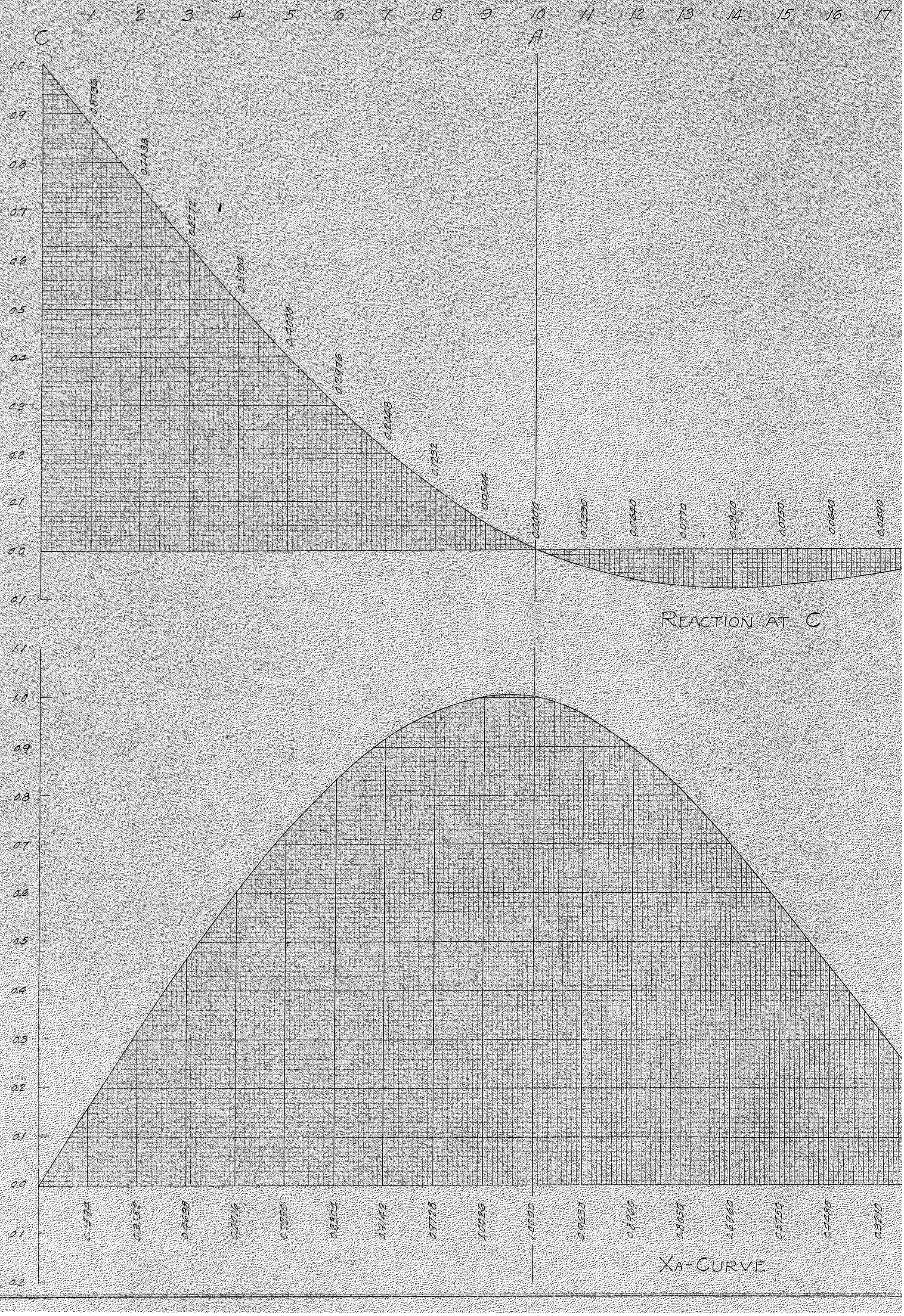
AT C

0.0750
0.0640
0.0490
0.0320
0.0150
0.0000
0.0114
0.0192
0.0238
0.0256
0.0250
0.0224
0.0182
0.0128
0.0066
0.0000

B

5 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30

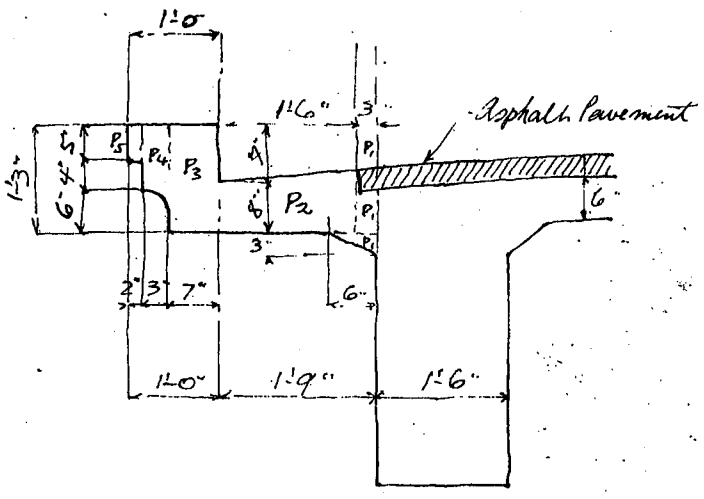
D


1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17
C
A
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0.1
0.9736
0.8948
0.7212
0.5476
0.3740
0.2004
0.0268
0.0268
0.0536
0.0804
0.1072
0.1340
0.1608
0.1876
0.2144
1
REACTION AT C
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0.1
0.2
0.0268
0.0536
0.0804
0.1072
0.1340
0.1608
0.1876
0.2144
0.2412
0.2680
0.2948
0.3216
0.3484
0.3752
0.4020
0.4288
0.4556
0.4824
0.5092
0.5360
0.5628
0.5896
0.6164
0.6432
0.6700
0.6968
0.7236
0.7504
0.7772
0.8040
0.8308
0.8576
0.8844
0.9112
0.9380
0.9648
0.9916
XA-CURVE

Floor slabs.

As shown on sketch concrete slabs rigidly fixed into main girder. Let us take clear span as Effective span which is 5'-0" Reinforcement in the slabs same as for floor slabs on River span see page no 3 for detailed calculation

Overhanging Arm of slabs.



Dead Load

Pavement	30# × ¼	=	7.5#	
	$\frac{3\times6}{144}$ @ 150	=	18.75	
	$\frac{3\times6}{2\times144}$ @ 150	=	9.38	
			35.63	
P_1	Wall this	360#	× 1.7"	= 61
P_2	$\frac{9"+8"}{24}$ × 1.5 @ 150	= 160.0	× 11.0	= 176
P_3	$\frac{7"\times6"}{144}$ @ 150	= 44	× 24.5	= 1077
P_4	$\frac{10\times4}{144}$ @ 150	= 42	× 26.0	= 1090
P_5	$\frac{12.5}{144}$ @ 150	= 62	× 27	= 1672
Handrail assumed		100	× 28.5 say	= 2850
		408#		6926 in#

Average arm 6926 ÷ 408 = 16.9"

Negative reaction for intermediate or center beam

$= \frac{6926}{12\times6.5} = 89.0$# per ft assumed

Depth of slab = 9"

Effective depth required $= \sqrt{\frac{61000}{12\times94.5}} = 7.3"$

Steel area required $= \frac{61000}{7.75\times\frac{7}{8}\times16000} = 0.5630"$

Use 3-½" bars = 0.5889" @ 4" center for one ft strips.

Live Load Rear wheel cone with impact = 7200

moment = 7200 × 15" = 108000 in#

For one foot strip = 54000 in#

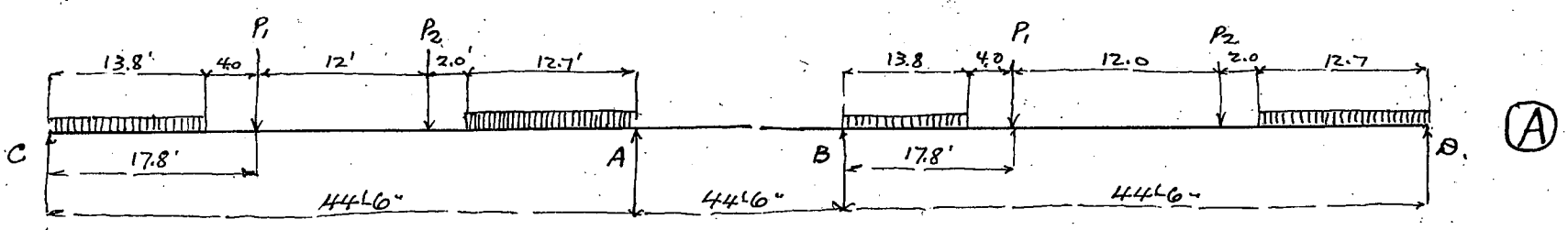
Dead Load moment say 7000

61000 in#

Design of main Beam

Center Beam

Live Load Positive moment at 0.4 l (End span)



By using influence line diagram for 3 continuous span

$$M = \Big[650\times\Big\{\Big(\frac{0+0.0494}{2}+\frac{0.0494+0.0995}{2}+\frac{0.0995+0.151}{2}\Big)\times4.45+\Big(\frac{0.151+0.156}{2}\times0.45\Big)$$

$$+\frac{11.430}{2}\times\Big(\frac{0.192+0.204\times2+0.195}{2}\times1.0\Big)+\frac{3.810}{2}\times\Big(\frac{0.100+0.083}{2}\times2\Big)$$

$$+650\times\Big\{\Big(\frac{0.076+0.0493}{2}\times3.8\Big)+\Big(\frac{0.0493+0.0218}{2}+\frac{0.0218+0}{2}\Big)\times4.45\Big\}$$

$$+650\times\Big\{\Big(\frac{0+0.0046}{2}+\frac{0.0046+0.0077}{2}+\frac{0.0077+0.0095}{2}\Big)\times4.45+\Big(\frac{0.0095+0.0096}{2}\times0.45\Big)$$

$$+\frac{11430}{2}\times\Big(\frac{0.0100+0.0102\times2+0.010}{2}\times1.0\Big)+\frac{3.810}{2}\times\Big(\frac{0.0082+0.0074}{2}\times2\Big)$$

$$+650\times\Big\{\Big(\frac{0.0072+0.0051}{2}\times3.8\Big)+\Big(\frac{0.0051+0.0026}{2}+\frac{0.0026+0}{2}\Big)\times4.45\Big\}\Big]\times44.5$$

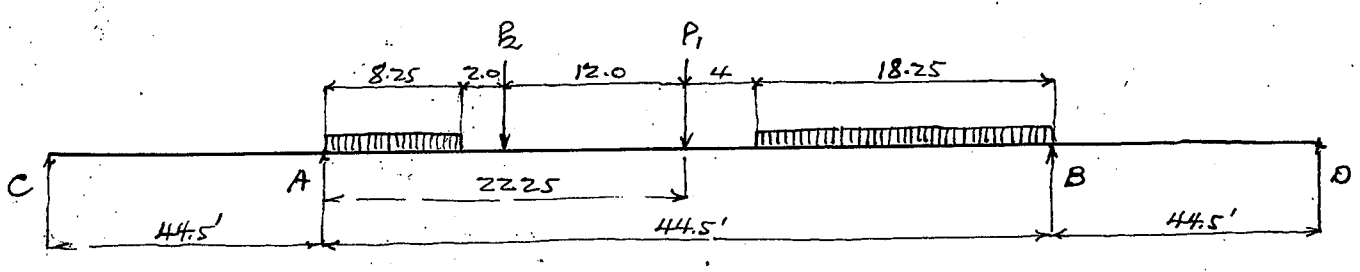
$= 170479.$ in# For Equivalent Uniform load $= 0.1pl^2$

$P = \frac{170479}{44.5^2\times0.1} = 860.9$# per lin ft

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Final Design of Murayama-bashi for Nagano-Ken and Nagano-Electric Ry Co

Positive Live Load moment at 1.5 l (Center span).



(B)

$$M = \Big[650' \times \Big(\frac{0+0.023}{2} \times 4.45 + \frac{0.023+0.048}{2} \times 3.8\Big) + \frac{3.810}{2} \cdot \Big(\frac{0.053+0.070}{2} \times 2\Big)$$

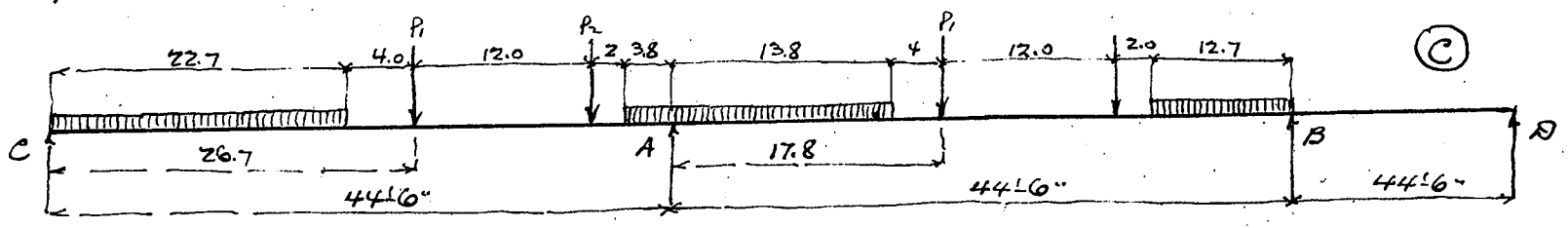
$$+ \frac{11430}{2} \cdot \Big(\frac{0.1562 + 2 \cdot 0.175 + 0.1562}{2} \times 1\Big) \cdot$$

$$+ 650 \cdot \Big\{\Big(\frac{0.132+0.128}{2} \cdot 0.45\Big) + \Big(\frac{0.128+0.087}{2} + \frac{0.087+0.052}{2} + \frac{0.052+0.023}{2} + \frac{0.023+0}{2}\Big) \cdot 4.45\Big\} \cdot 445$$

$= 130.821.^{1\#}$ For Equivalent uniform load $= 0.075 \times pl^2$

$$p = \frac{130821}{0.075 \cdot 445^2} = 880.8 \text{ lbs per lin ft of span}$$

Negative Live Load Moment at 1.0 l



(C)

$$M = \Big[650 \cdot \Big\{\Big(\frac{0+0.0264}{2} + \frac{0.0264+0.0512}{2} + \frac{0.0512+0.0728}{2} + \frac{0.0728+0.0896}{2} + \frac{0.0896+0.100}{2}\Big) \times 4.45$$

$$+ \Big(\frac{0.100+0.102}{2} \cdot 0.45\Big)\Big\} + \frac{11430}{2} \times \Big(\frac{0.102 + 2 \cdot 0.1024 + 0.102}{2} \times 1\Big) + \frac{3810}{2} \Big(\frac{0.062+0.047}{2} \times 2\Big)$$

$$+ 650\Big(\frac{0.038+0}{2} \cdot 3.8\Big) + 650 \cdot \Big\{\Big(\frac{0+0.039}{2} + \frac{0.039+0.064}{2} + \frac{0.064+0.077}{2}\Big) \times 4.45 + \Big(\frac{0.077+0.078}{2} \times 0.45\Big)\Big\}$$

$$+ \frac{11.430}{2} \cdot \Big(\frac{0.078 + 2 \cdot 0.08 + 0.078}{2} \times 1\Big) + \frac{3810}{2} \times \Big(\frac{0.056 + 0.050}{2} \times 2\Big)$$

$$+ 650 \cdot \Big\{\Big(\frac{0.047+0.032}{2} \cdot 3.8\Big) + \Big(\frac{0.032+0.015}{2} + \frac{0.015+0}{2}\Big) \cdot 4.45\Big\}\Big] \cdot 445$$

$= 178.663^{1\#}$ For Equivalent uniform load $= 0.11667\, pl^2$

$$p = \frac{178663}{0.11667 \cdot 445^2} = 773.4 \text{ lbs per lin ft of span}$$

Summary Uniform load for max. moments (Center Girder).

For positive moment	End span	860.9 #	per lin. ft
	Center span	880.8 #	" " "
For negative moment	at support	773.4 #	" " "

Outside Beam

Positive moment at 0.4 l (End span).

Position of loading same as for (A)

Uniform load 575# per lin ft Concentration $P_1 = 9350$#

Concentration $P_2 = 3120$#

Total moment $= 142\,853.0^{1\#}$ For uniform load $= 0.10\, pl^2$

Equivalent uniform load $= \frac{142853}{810 \cdot 445^2} = 771.9$ lbs per lin ft of span

Positive moment at 1.5 l (Center span).

Position of loading same as per (B)

Uniform load 575# per lin ft Concentration $P_1 = 9350$#

Concentration $P_2 = 3120$#

Total moment $= 109434^{1\#}$ For uniform load $= 0.075\, pl^2$

Equivalent uniform load $= \frac{109434}{0.075 \cdot 445^2} = 1369$ # per lin ft of span.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Neg. Max. moment at [illegible]

Position of loading same as for (c)

Uniform load 575# per lin. ft wheel conc. P_1 = 9350#

" " P_2 = 3120#

Total moment = 150.721 [1]# For uniform load $m = 0.11667\ pl^2$

Equivalent uniform load = $\frac{150.721}{0.11667 \times 44.5^2}$ = 652.4# per lin. ft of span.

Dead Load for Outside Beam

Slab and pavement	110# × 2.5	= 275
over-hanging arm including kerb		408
Extra reaction	$\frac{M}{6.5}$	= 89
main beam	$1.5 \times \frac{39.4}{12} @ 150$	= 750
misc concrete	0.545 @ 150	= 82
Pavement over beam		= 45
	g =	1649# lin ft

gl^2 = 3.268.000 [1]#

Dead Load of Center beam

Slab and pavement	110 × 6.5	= 715
beam	1.5 × 3.0 @ 150	= 675
misc concrete		= 82
		1472#
Negative Reaction ½ effective say		89
	g' =	1383# per lin ft.

$g'l^2$ = 2740.000 [1]#

x/l	Dead Load moment: Coef of gl^2	g=1649	g'=1383	Live Load negative: Coef of pl^2	P=6524 P'=693.0	P=7734 P'=821.0	Live Load Positive: Coef Pl^2	P=721.9 P'=736.9	P=860.9 P'=880.8
0.10	+0.0359gl^2	114.400	95900	0.005	6500	7700	0.040	57200	68200
0.20	+0.060	196.100	164400	0.010	12900	15300	0.070	100.100	119300	
0.30	+0.075	245100	205500	0.015	19400	23000	0.090	128700	153400
0.40	+0.080	261500	219200	0.020	25800	30600	0.100	143000	170500
0.50	+0.075	245100	205500	0.025	32300	38300	"	"	"
0.60	+0.060	196100	164400	0.030	38800	46000	0.090	128700	153400
0.70	+0.035	114400	95900	0.035	45200	53600	0.070	100100	119300
0.80	0.000	0	0	0.04022	52000	61600	0.04022	57500	68600
0.85	−0.02125	−69500	−58300	0.04898	63300	75000	0.02773	39600	47300
0.90	−0.0450	−147100	−123300	0.06542	84500	100200	0.02042	29200	34800
0.95	−0.07125	−232900	−195200	0.08831	114100	135300	0.01706	24400	29100
1.00	−0.1000	−326800	−274000	0.11667	150700	178700	0.01667	23800	28400
1.05	−0.07625	−249200	−208900	0.09033	116700	138400	0.01408 $P'l^2$	20600	24600
1.10	−0.05500	−179800	−150700	0.07014	90600	107400	0.01514	22100	26400
1.15	−0.03625	−118500	−99300	0.05678	73400	87000	0.02053	30000	35800
1.20	−0.0200	−65400	−54700	0.0500 $P'l^2$	68600	81300	0.0300	43800	52300
1.2764	0.000	0	0	"	"	"	0.0500	73000	87200
1.30	+0.005	16400	13700	"	"	"	0.0550	80300	96000
1.40	+0.020	65400	54800	"	"	"	0.0700	102200	122100
1.50	+0.025	81700	68500	"	"	"	0.0750	109500	130800
		gl^2=3268000	$g'l^2$=2740,000		Pl^2= 1.291.900	Pl^2= 1531500		Pl^2=1429500	Pl^2=1.704800
					$P'l^2$=1.372300	$P'l^2$=1.625800		$P'l^2$=1459200	$P'l^2$=174.4200

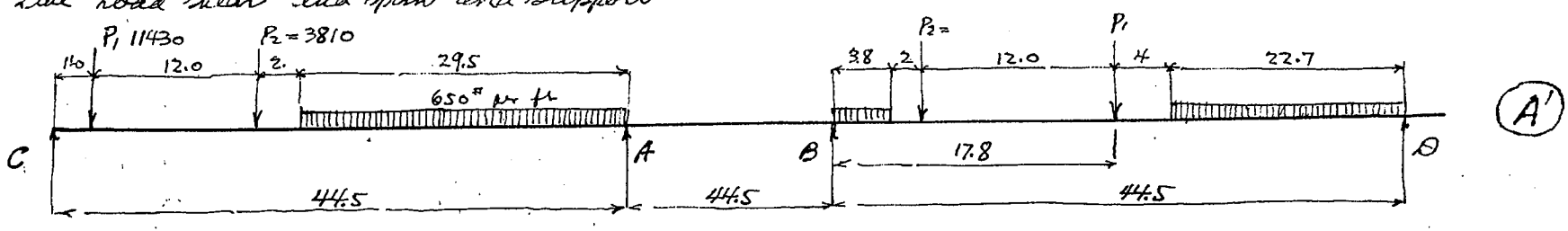
増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Summary of dead and live load moments

	Outside Beam		Center Beam	
x/ℓ	negative	Positive	negative	Positive
.10	107.900	171.600	88200	164100
.20	183200	296200	149100	283700
.30	225700	373800	182500	358900
.40	235700	404500	188600	389700
.50	212800	388100	167200	376000
.60	157300	324800	118400	317800
.70	69200	214500	42300	215200
.80	-52000	57500	-61600	68600
.85	-132800	-29900	-133300	-11.000
.90	-231600	-117900	-223500	-88500
.95	-347000	-208500	-330500	-166100
1.00	-477500	-303000	-452700	-245600
1.05	-365900	-228600	-347300	-184300
1.10	-270400	-157700	-258100	-124300
1.15	-191900	-88500	-186300	-63500
1.20	-134000	-21600	-136100	-2500
1.2764	-68600	73000	-81300	87.200
1.30	-52200	96700	-67600	109.700
1.40	-3200	167600	-26500	176900
1.50	+13100	191200	-12.800	199.300

Center Beam

Live Load shear end span end support



Live Load shear by influence diagram

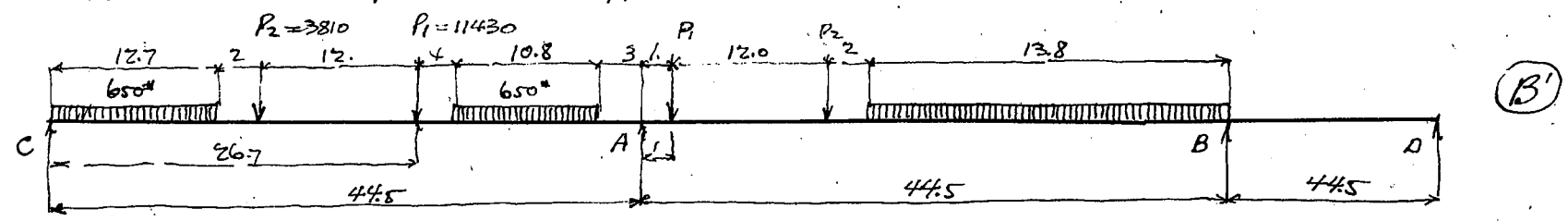
$$Q_L = \frac{11430}{2}\times\left(\frac{1.0+0.946}{2}\times 2\right)+\frac{3810}{2}\times\left(\frac{0.666+0.612}{2}\times 2\right)+650\times\left\{\left(\frac{0.586+0.5104}{2}\right)2.8+\left(\frac{0.5104+0.400}{2}+\frac{0.400+0.2976}{2}\right.\right.$$
$$\left.+\frac{0.2976+0.2048}{2}+\frac{0.2048+0.1232}{2}+\frac{0.1232+0.0544}{2}+\frac{0.0544+0}{2}\right)\cdot 4.45\Big\}+650\times\left(\frac{0+0.010}{2}\times 3.8\right)$$
$$+\frac{3810}{2}\times\left(\frac{0.0116+0.0160}{2}\times 2\right)+\frac{11.430}{2}\times\left(\frac{0.0240+2\times 0.0254+0.024}{2}\times 1\right)+650\times\left\{\left(\frac{0.026+0.025}{2}\times 0.45\right)\right.$$
$$\left.+\frac{0.025+0.0224}{2}+\frac{0.0224+0.0182}{2}+\frac{0.0182+0.0128}{2}+\frac{0.0128+0.0060}{2}+\frac{0.0060+0}{2}\right)\times 4.45\Big\}$$

$= 18977^{\#}$. 18977

Dead Load shear $Q_D = 0.4\times 1.383\times 445 = 24617$

Total shear = 43594 #

Live Load shear Center span 2nd support



$$Q_L = 650\cdot\left\{\left(\frac{0+0.03}{2}+\frac{0.03+0.064}{2}\right)\cdot 4.45+\left(\frac{0.064+0.082}{2}\right)\cdot 3.8\right\}+\frac{3810}{2}\times\left(\frac{0.092+0.12}{2}\times 2\right)+\frac{11.430}{2}\times$$
$$\left(\frac{0.126+2\times 0.128+0.126}{2}\times 1\right)+650\times\left\{\left(\frac{0.120+0.119}{2}\right)\cdot 0.45+\left(\frac{0.119+0.096}{2}+\frac{0.096+0.057}{2}+\frac{0.057+0}{2}\right)\times 4.45\right.$$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

$$\frac{11430}{2}\cdot\left(\frac{1+0.97}{2}\times 2\right)+\frac{3810}{2}\cdot\left(\frac{0.76+0.71}{2}\times 2\right)+650\cdot\left\{\left(\frac{0.632+0.616}{2}\right)\cdot 0.8+\left(\frac{0.616+0.500}{2}\right.\right.$$

$$\left.\left.+\frac{0.500+0.384}{2}+\frac{0.384+0.272}{2}+\frac{0.272+0.168}{2}+\frac{0.168+0.076}{2}+\frac{0.076+0}{2}\right)\cdot 4.45\right\}$$

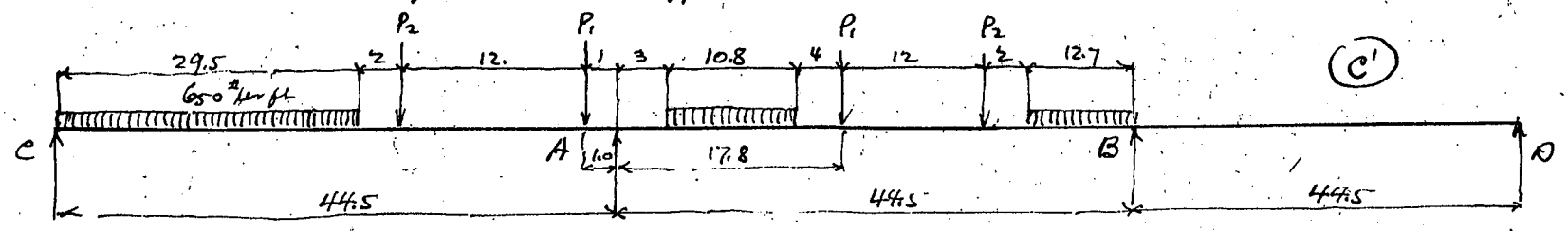
$$= 23.291$$

Dead load shear $Q_D = 0.5 \cdot 1383 \cdot 44.5 = 30772$

23.291

54063 #

Live Load shear End span 2nd support



$$Q_L = 650\cdot\left\{\left(\frac{0+0.1264}{2}+\frac{0.1264+0.2512}{2}+\frac{0.2512+0.3728}{2}+\frac{0.3728+0.4896}{2}+\frac{0.4896+0.600}{2}+\frac{0.600+0.7024}{2}\right)\cdot 4.45\right.$$

$$\left.+\left(\frac{0.7024+.762}{2}\right)\times 2.8\right\}+\frac{3810}{2}\times\left(\frac{0.782+0.820}{2}\times 2\right)+\frac{11430}{2}\cdot\left(\frac{0.976+1.0}{2}\times 2\right)+650\left\{\left(\frac{0.023+0.039}{2}\right)\cdot 1.45\right.$$

$$+\left(\frac{0.039+0.064}{2}+\frac{0.064+0.077}{2}\right)\cdot 4.45+\left(\frac{0.077+0.078}{2}\right)\cdot 0.45\Big\}+\frac{11430}{2}\cdot\left(\frac{0.078+2\times 0.08+0.078\times 1}{2}\right)$$

$$+\frac{3810}{2}\cdot\left(\frac{0.056+0.056}{2}\times 2\right)+650\cdot\left\{\left(\frac{0.045+0.032}{2}\right)\cdot 3.8+\left(\frac{0.032+0.015}{2}+\frac{0.015+0}{2}\right)\cdot 4.45\right\}$$

$$= 23710\ \#$$

23710

Dead load shear $Q_D = 0.6 \cdot 1.383 \times 44.5 = 36926$

Tot 60636 #

Outside Beam

Live Load shear End span End support

Position of loading same as for (A') Uniform load 575 # per ft $P_1 = 9350$# $P_2 = 3120$#

Live Load shear = 11797

Dead load shear = $0.4 \times 1649 \times 445 = 29352$

41149

Live Load shear Center span 2nd support

Position of loading same as for (B') Uniform load 575 # per ft $P_1 = 9350$# $P_2 = 3120$#

Live load shear = 19911

Dead Load shear = $0.5 \times 1649 \times 445 = 36690$

56601

Live Load shear End span 2nd support

Position of loading same as for (C') uniform load 575# per ft $P_1 = 9350$# $P_2 = 3120$#

Live Load shear 19948

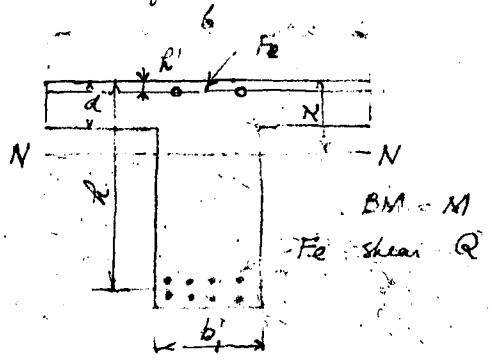
Dead load shear = $0.6 \cdot 1649 \cdot 44.5 = 44028$

63976 #

Summary for shears

	Center Beam	Outside Beam
End span End support	43594 #	41149 #
End span 2nd support	60636	63976
Center span 2nd support	54063	56601

Formulae for Reinforced Concrete T Beam



Double Reinforcements

Fe' steel area on compression side

Fe steel area on tension side

N-N neutral axis

Position of Neutral Axis $\equiv x = \dfrac{bd^2+2n(Fe'h'+Feh)}{2bd+2n(Fe'+Fe)}$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

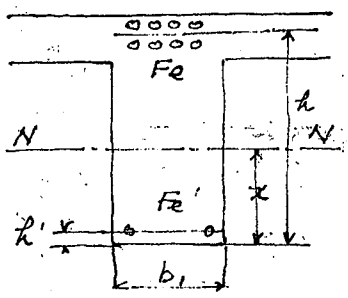
Moment of initia of the section $J = \frac{bx^3}{3} - \frac{b(x-d)^3}{3} + nFe'(x-h')^2 + nFe(h-x)^2$

Fibre stress of Concrete $\sigma_b = \frac{Mx}{J}$

Tensile stress of steel at Fe $\cdot \sigma_e = n\frac{M(h-x)}{J}$ do at Fe' $\sigma_e = n\frac{M(x-h')}{J}$

Geometrical Moment $S = bd(x-\frac{d}{2}) + nFe'(x-h') = nFe(h-x)$.
Geometrical moment for upper strip $S' = bh'(x-\frac{h'}{2}) + nFe'(x-h')$
U or U' = circumference of steel at Fe or at Fe'
Max Unit shear at neutral axis $\tau_o = \frac{QS}{b_1 J}$ = unit shear at Fe
Bond stress at Fe - $\tau_h = \frac{QS}{UJ}$ Bond stress at Fe' $\tau_h' = \frac{QS'}{U'J}$
Unit shear at Fe' - $\tau_o' = \frac{QS'}{b_1 J}$

Formulae for Rectangular section



$$x = -\frac{n(Fe+Fe')}{b_1} + \sqrt{\left\{\frac{n(Fe+Fe')}{b_1}\right\}^2 + \frac{2n}{b_1}(hFe + h'Fe')}$$

$$J = \frac{b_1 x^3}{3} + nFe'(x-h')^2 + nFe(h-x)^2$$

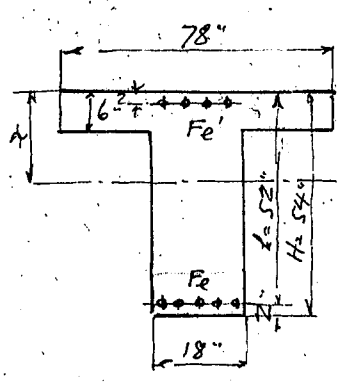
$$\sigma_b = \frac{M.x}{J} \qquad \sigma_e' = n\sigma_b\frac{x-h'}{x} \qquad \sigma_e = n\sigma_b\frac{h-x}{x}$$

$$S = \frac{b_1 x^2}{2} + nFe'(x-h') = nFe(h-x).$$

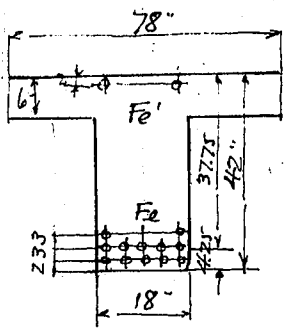
$$\tau_o = \frac{QS}{b_1 J} \qquad \tau_h = \frac{QS}{UJ}$$

$$S' = b_1 h'(x-\frac{h'}{2}) + nFe'(x-h')$$

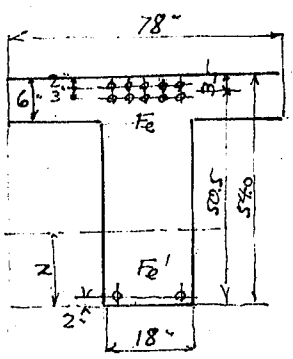
$$\tau_o = \frac{QS'}{b_1 J} \qquad \tau_h' = \frac{QS'}{U'J}$$



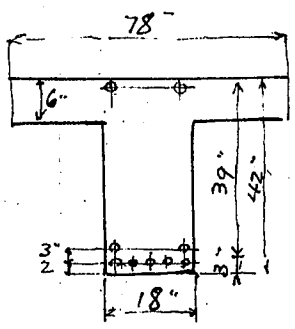
at 0.0l
Center Beam



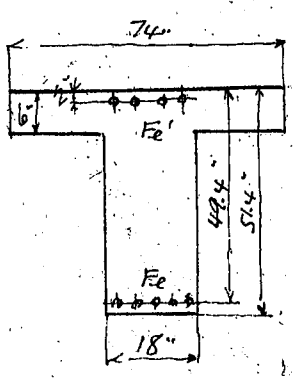
at 0.4l
Center beam



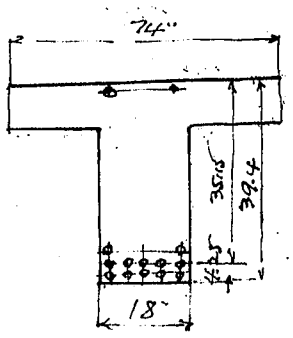
at 1.0l
Center Beam



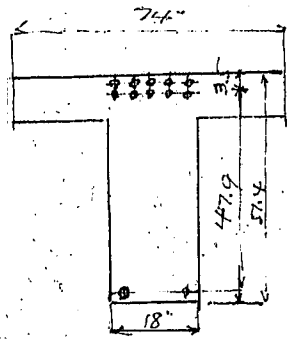
at 1.5l
Center Beam



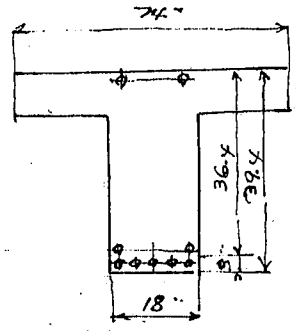
at 0.0l
Outside beam



at 0.4l
Outside Beam



at 1.0l
Outside Beam



at 1.5l
Outside Beam

Summary for Stresses in Beams

	Point	Fe'	Fe	x	J	M	σ_b	σ_o'	σ_e
Center Beam	0.4l	2@1"φ 1.57080"	12+1"φ 9.42480"	10.73"	134.378	+389700 lbs	373 #/o"	4557 #/o"	14.105 #/o"
	1.0l	2@1"φ 1.5708	10-1"φ 7.8540	19.13	164.853	-452.700	641	8557	15.769
	1.5l	2@1"φ 1.5708	7-1" 5.4978	8.13	93.193	+199.300	209	2360	11.883
Outside Beam	0.4l	do	12-1"φ 9.4248	10.43	113906	+404.500	444	5389	15801
	1.0l	do	10-1"φ 7.8540	18.49	143.485	- 477500	739	9866	17.293
	1.5l	do	7-1"φ 5.4978	7.96	79.794	+191.200	229	2571	8.178

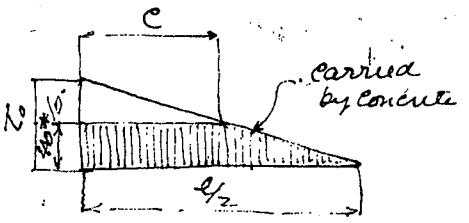
増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Summary for Stresses in Beams.

		Fe'	Fe	x	J	S	S'	U	U'	Q	τ_0	τ_h	τ_0'	τ_h
Center Beam	0.0 l	4-1" 3.1416	5@1" 3.927	7.95"	138767 (in)⁴	2.672³	1313	15.708"	12.5664"	43.594#	46.6#/□"	53.3#/□"	22.8#/□"	39.7#/□"
	1.0 l	2@1" 1.5708	10@1" 7.854	19.13"	164853	3698	1056	31.416"	62.832"	60636	75.6	43.3	21.7	61.8
Outside Beam	0.0 l	4@1" 3.1416	5@1" 3.927	7.88"	115.090	2.444	1.295	15.708"	12.566"	41149#	48.5	55.6	25.7	36.4
	1.0 l	2@1" 1.5708	10-1" 7.854	18.49"	143.482	4.465	1018	31.416"	62.832"	63976	110.6	63.3	25.2	22.2

Stirrups in Beams

	Center Beam				Outside Beam			
	C	area of stirrup	size	no	C	area	size	no
0.0 l	34"	0.1678	½"φ	1	51"	0.3521"	½"φ	1
1.0 l	126"	1.6825	½"φ	5	170"	9.000"	½"φ	23



Seismic stability of Column at bottom for sliding support

Compressive Stress due to Dead Load

Beam Slab and Pavement	0.8 × (1649 × 2 + 1383) × 44.5	=	166.700#
Bent and Strut	(37+29+2)@150	=	14300
Column	17.2 × 2.5 × 2 @150	=	12900
do	2 - 17.2 × 2.5 × 2.5 @150	=	32300
		W₁ =	226200#
web	7 × 9 × 1.5 @150		14150
Pedestal	$\frac{4+6}{2}$ × 18.4 × 8 @150		110.400
			350.750#

Unit Compressive Stress for Column = $\frac{226200}{30 \cdot 30 \times 2 + 30 \times 24}$ = 90#/□"

Bending moment due to earthquake

Intensity of Earthquake assumed 2000 mm/sec². 2000 ÷ 9800 = .204

moments	166700 × .204 × 23.2'	=	790.000#
	14300 × .204 × 18.0'	=	52.500
	45200 × .204 × 8.6'	=	79.300
			921.800#

Depth 30" and total width 84" 25-1"φ bars on both faces.

neutral axis x = 9.1"

3h - x = 3 × 28 - 9.1" = 74.9"

$6x^2(3h-x)$ = 84 × 83 . 74.9 = 522000

$6nFe'(x-h')(h-h')$ = 90 × 19.635 × 7.1 × 26 = 326.000

848,000

Stress in Concrete $\sigma_b = \frac{6 \times 921.800 \times 12 \times 9.1}{848000}$ = 712#/□" Ok

Stress in Steel. $\sigma_e = n\sigma_b \frac{h-x}{x}$ = 15 × 712 × $\frac{18.9}{9.1}$ = 22.200#/□" Ok

Seismic Stability of Column for fixed support

Compressive stress due to dead load

Beam slab and pavement	1.1 × (1649 × 2 + 1383) × 44.5	=	229.100
Bent and strut			14300
Column			12900
do			32300
		W₁ —	288600
web			14150
Pedestal			110.400
		W₂ —	443.150#

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Unit compressive stress in Col. $= \frac{288.600}{2520} = 114.6\ ^{\#}/_{\square''}$ Ok

Bending Moment due to Earthquake = 229100 × .204 × 23.2 = 1.081.000
52500
79300
1.212.800 ft

Depth 30" and total width 84" 21-1"ϕ bars on both faces
neutral axis $x = 8.60"$

$3h - x = 3 \cdot 28 - 8.60 = 75.40$
$6x^2(3h-x) = 84 \times 73.96 \times 75.40 = 468\,000$
$6nFe'(x-h')(h-h') = 90 \times 15.708 \times 66.26 = 242.800$
710.800

concrete stress $6b = \frac{6 \cdot 606400 \times 12 \times 8.6}{710800} = 528\ ^{\#}/_{\square''}$ Ok

Steel Stress $6e = n6b \cdot \frac{h-x}{x} = 15 \times 528 \cdot \frac{28-8.60}{8.6} = 17.820\ ^{\#}/_{\square''}$ ok.

Bearing on soil
Superimposed load to Pedestal 413 150
weight of base $9 \times 7 \times 18.2 \ @150^{\#} =$ 171 990
585 140 #

Uniform live load $100^{\#}/_{\square'}$ or $1800^{\#}$ per lin ft
max load on column $= 1.1 \times 1800 \cdot 445 =$ 88110 #
Dead load 585140
673250

Unit Bearing $= \frac{673250}{9 \times 18.2} = 4120\ ^{\#}/_{\square'}$ or $1.835\ ^{ton}/_{\square'}$ ok.

" " $\frac{585140}{9 \times 18.2} = 3570\ ^{\#}/_{\square'}$ or $1.595\ ^{ton}/_{\square'}$ ok

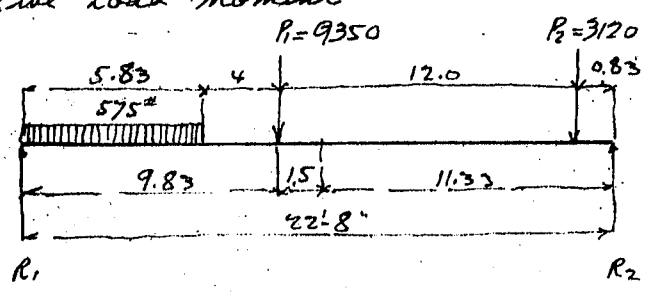
Bearing pressure at the bottom of base due to Earth quake
Earthquake effect assumed 10' below ground surface where is about bottom of column, Take this bending moment effected to toe pressure of bottom of base.

Total earthquake moment = 921.800 ft
Fibre stress due to this moment $= \frac{6 \cdot 921.800}{18.2 \cdot 9^2} = 3760\ ^{\#}/_{\square'}$ or $1.68\ ^{tons}/_{\square'}$

Uniform bearing pressure 1.595
3.275 tons/□'

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Reinforced Concrete span for west approach
span length 22'-8" c-to-c end bearings assumed 0 to c 23'-8"
Roadway 18'-0" wide
Cross section of structure similar to Concrete span on East approach except depth of main girder, see page no 25 for general cross section.
Slab and Pavement same as for 44'-6" span.
Design of Outside beam
Live Load Moment



$R_1 = (3120 \cdot 0.83 + 9.350 \cdot 12.83 + 5.83 \cdot 575 \times 19.75) \times \frac{1}{22.66}$
$= 8303^{\#}$
$R_2 = 15823 - 8303 = 7520^{\#}$

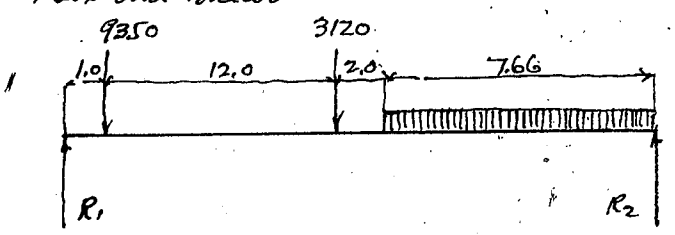
$M_L = 7520 \cdot 11.33 - 3.120 \cdot 10.5' = 52.434^{1\#}$

Dead Load Moment:

Slab and Pavement	110 · 2.5 =	275
Overhanging arm		408
Extra Reaction		89
Beam	1.5 · 2.5 · 1.0 · 150 =	563
fillet for Beam at support	0.3 @150 =	45
Pavement over beam	30* · 1.50 =	45
		1425#/per ft

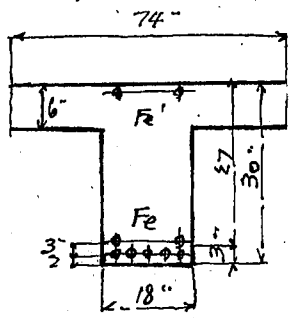
$M = \frac{1}{8} \cdot 1425 \cdot 22.66^2 = \frac{91454}{143888^{1\#}}$

Max End shear

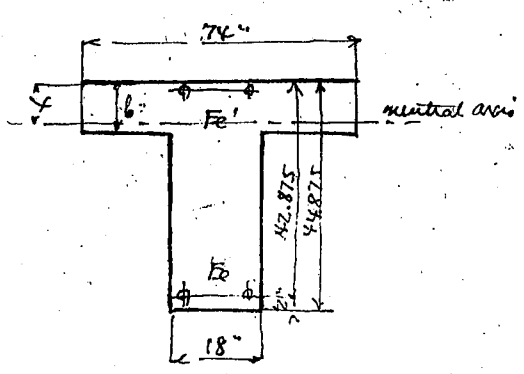


Live Load shear
$R_1 = (9350 \cdot 21.66 + 3120 \cdot 9.66 + 575 \cdot 7.66 \cdot 3.88) \times \frac{1}{22.66}$
$= 11012$
Dead Load shear $= \frac{1425 \cdot 22.66}{2} = 16145$
Live-load shear $\frac{11012}{27157^{\#}}$

Section of beam



$Fe' = 2@1"\phi = 1.5708^{0"}$ $Fe = 7@1"\phi = 5.4970"$
Position of neutral axis $= x = 6.55"$
$J = 41.917^{"4}$ $\delta b = \frac{Mx}{J} = 270^{\#/0"}$
$\delta e' = \frac{nM(x-h')}{J} = 2812$
$\delta e = \frac{nM.(h-x)}{J} = 12.638^{\#/0"}$



Fe $2@1"\phi = 1.5708^{0"}$ Fe $2@1"\phi = 1.5708'$
$x = 4.75"$ $J = 36900^{(in)4}$

$S = \frac{bx^2}{2} + nFe'(x-h').. = 900^{(in)3}$
$U = 6.2832"$
$S' = bn(x - \frac{h'}{2}) + nFe'(x-h').. = 636^{(in)3}$ $U' = 6.2832"$
$\tau_o = \frac{QS}{bJ} = 9^{\#/0"}$ $\tau_i = 106^{\#/0"}$ $\tau_o' = \frac{Q_1 S'}{bJ} = 6^{\#/0"}$ $\tau_i' = \frac{QS'}{JU} = 75^{\#/0"}$

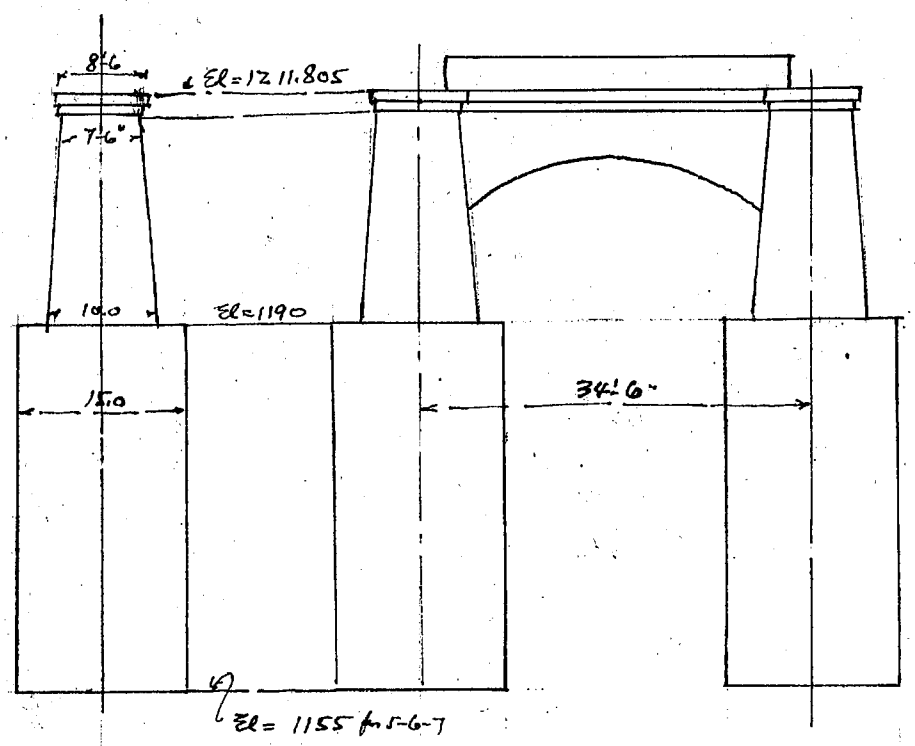
Use same reinforcing bars for Center beam.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Design of substructure

River Piers 2-3-4-5-6 and 7

Intermediate Piers for 165.' spans.



coping of shaft

8'-6" dia = 56.75 × .75 = 42.5

8'-0" dia 50.27 × .75 = 37.6

80.1

Shaft

top 7.5$^{\phi}$ = 44.18

bott 10.0. 78.54

122.72 ÷ 2 = 61.36 · 20.178 = 1235.0

1315.1

weight = 184000 #

strut between shafts

Pedestal 30 × 2.416 × 4.5 = 326

strut 7.5 × 28 × 4.5 = 945

1271 cuft

weight = 180.000 #

for one half = 90.000 #

Total weight = 274 000 #

Load at bottom of shaft

For Highway superimposed load 455.000

shaft and strut 274.000

729.000 #

Bottom area of shaft 10' dia = 78.54 □'

Unit pressure = $\frac{729.000}{78.54 \times 144} = 64.4$ #/□"

well 15.0' dia area = 176.70'

height 32' volume 5.660 cub ft wt = 794000

" 35' " 6190 " = 867000

Bottom area of well 15'6" dia = 188.7 □'

Unit Pressure (32') = $\frac{1523.000}{188.7} = 8070$ #/□' = 56.1 #/□"

Unit Pressure (35') = $\frac{1596.000}{188.7} = 8460$ #/□' = 58.8 #/□"

Superstructure and shaft	729.000	729.000
well	794.000	867.000
	(32') 1523.000 #	(35') 1.596000 #
Considering live load	h = 32'	h = 35'
Dead load	1523.000	1596000
Superimposed load 4/7	455.000	455000
	1068000	1141000
superimpose D+L Load 3/7	1074000	1074000
	2142000 #	2215000 #

Unit Pressure (32') $\frac{2142.000}{188.7} = 11.360$ #/□' = 5.08 tons/□'

Unit Pressure (35') $\frac{2215000}{188.7} = 11.740$ #/□' = 5.25 tons/□'

Earthquake effect for highway side 2000 mm/sec²

moment	superstructure	455 000	× 31 × $\frac{2000}{9800}$ =	2.880.000
	strut	90.000	× 20 × .204 =	367.000
	shaft	184000	× 10.5 × .204 =	394.000
				3.641.000 lbs

2. Rows of Reinforcements

Dia. of outer row = 9'6" Dia of inner row = 9'0"

Circumference = 29.845' Circumference = 28.274'

Steel Area 64-1" ϕ = 50.30 □" Steel Area 32-1" ϕ = 25.10 □"

Equivalent concrete area = 755 □" Equivalent concrete area = 377 □"

Area of half circle = 35.5 □' Area of ½ circle = 31.8

Average height = $\frac{35.5}{9.5} = 3.74 = 44.9$" Average height = $\frac{31.8}{9} = 3.53' = 42.4$"

sq 12015 cm² sq 1.800 cm²

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Moment of inertia = 755 × 2.015 = 1.520.000 (in)⁴ Moment of Inertia = 377 × 1.800 = 680.000 (in)⁴

680.000

7 200.000

Summary

Moment of Inertia of Concrete = 0.049 × 120⁴ = 10.200.000 (in)⁴

" " " " Steel 2.200.000

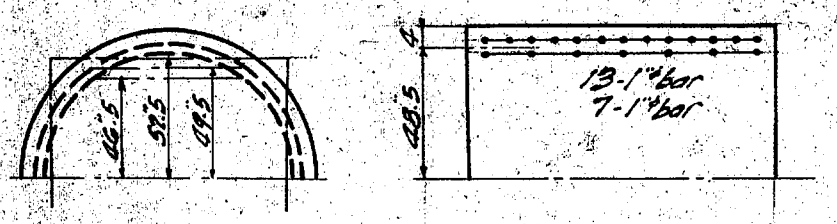
Total 12.400.000

Fiber stress = $\frac{364.000 \times 12 \times 60}{12.400.000}$ = 212 #/□" Tension or Compression

for comp. 212 + 64.4 = 276.4 #/□"

for tension 212 − 64.4 = 147.6 #/□"

Equivalent square section $d = \sqrt[4]{10.200.000 \times 12}$ = 105"



Fe = 0.7854 × 20 = 15.708 □"

Fe' 15.708

Fe + Fe' 31.416 □"

Position of Neutral axis $\frac{n(Fe+Fe')}{b} = \frac{15 \times 31.416}{105} = 4.49 = 4.5$

sq = 20.2

hFe = 101 × 15.708

h'Fe' = 4 × 15.708

hFe + h'Fe' = 105 × 15.708 = 1.650

$\frac{2n}{b}(hFe + h'Fe') = \frac{30}{105} \times 1650 = 471$

$\left\{\frac{n(Fe+Fe')}{b}\right\}^2$ = 20.2 / 491.2 sq. root = 22.2

less 4.5

x (position of neutral axis) = 17.7 x^2 = 314

Unit stress of Concrete :-

3h − x = 303 − 17.7 = 285.3

$bx^2(3h-x) = 105 \times 314 \times 285.3 = 9.425.000$

$6nFe'(x-h')(h-h') = 90 \times 15.708 \times 13.7 \times 97 = 1.880.000$

11.305.000

Unit stress of Concrete $6_b = \frac{6 \times 3.641.280 \times 12 \times 17.7}{11.305.000} \times \frac{147.6}{212}$ = 786 #/□"

$6_e = n6_b \frac{h-x}{x} = 15 \times 786 \times \frac{83.3}{17.7}$ = 20.200 #/□" o.k.

At the section of Well

Section considered at 13 ft. below the top of Well

ΣM - superstructure 7.880.000 '# × $\frac{24}{31}$ = 4.090.000 '#

Strut 367.000 × $\frac{33}{20}$ = 605.000

Shaft 394.000 × $\frac{23.5}{10.5}$ = 882.000

Well 176°.7 × 13'.140 # × $\frac{2.000}{9.300}$ × 6.5 = 427.000

6.004.000

Sec. mod. of well sect. = 0.098 × 180³ = 571.000 (in)³

Max. fiber stress = $\frac{6004.000 \times 12}{571.000}$ = 126 #/□"

At the base of Well

Sec. mod. of well base = 0.098 × 15.5³ = 365 (in)³

Max. fiber stress = $\frac{6.004.000}{365}$ = 16.200 #/□" = 7.33 tons/□'

Uniform pressure 4.68

12.01 tons/□'

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

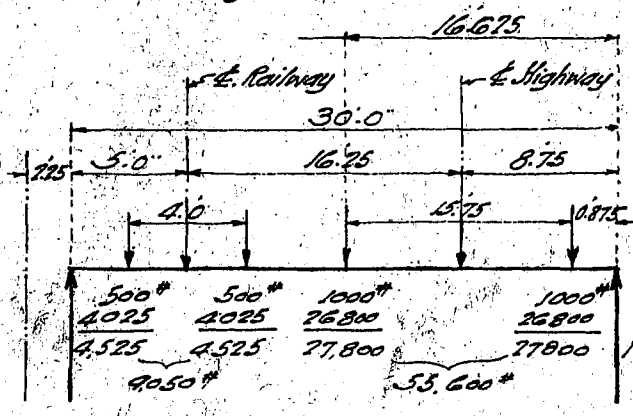
DESIGN OF SUBSTRUCTURE
INTERMEDIATE PIERS Nos 7-3-4-5-6-7

Reinforcements in between shaft

Span length assumed as 30'-0"

Dead load

Own weight = 6000 #/lin.ft. assumed



Floor Beam

For Railway bearing = 500#

For Highway bearing = 1000#

Stringer and Flooring

For Railway bearing = 4.025#

For Highway bearing = $2.600 \times \frac{20.625}{7} = 76.800$#

Dead load Moment

due to own weight $M = \frac{1}{8} \times 6000 \times 30^2 = 675.000$'# $R = \frac{1}{2} \times 6000 \times 30 = 90.000$#

due to floor system R: 55.600 × 21.25 = 1.180.000

9.050 × 5.0 = 45.250

64.650 1.225.250 ÷ 30 = 40.900#

− 40.900

23.750

M: 40.900 × 16.625 = 680.000 '#

less 27.800 × 15.750 = 438.000

242.000

675.000

Summay 917.000 '#

Live load

R: 32.800# × 10.25 = 336.000

14.000 × 8.75 = 122.200

120.800 × 25.00 = 3.020.000

3.478.700 ÷ 30 = 116.000#

M: 116.000 × 7 = 812.000 '#

less 60.400 × 4 = 241.600

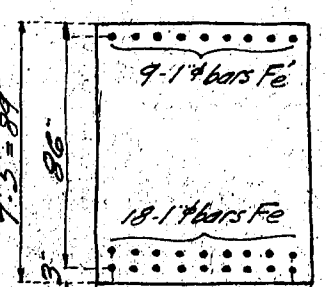
570.400 '#

Summary of dead and live load moments

M = 917.000 + 570.400 = 1.487.400 '# × 0.8 = 1.189.920 '# negative and Positive

Max. Shear Q = 90,000 + 23.750 + 116.000 = 229.750#

Section



Fe = 0.7854 × 18 = 14.1372"

Fe' = 7.0686

21.2058

$\frac{n(Fe+Fe')}{b} = \frac{15.21.2058}{54} = 5.89$ sq. = 34.69

hFe = 86 × 14.1372 = 1215.8

h'Fe' = 3 × 7.0686 = 23.1 ?

hFe + h'Fe' = 1238.9

$\frac{2n}{b}(hFe+Fe'h') = \frac{30}{54} \times 1238.9 = 688.3$

$\left\{\frac{n(Fe+Fe')}{b}\right\}^2$ = 34.7

723.0

Sq. root = 26.9

less 5.9

x = 21.0

x^2 = 441.0 x^3 = 9261

x − h' = 18" $(x-h')^2$ = 324

h − x = 65" $(h-x)^2$ = 4225

Fe' $(x-h')^2$ = 2.290

Fe $(h-x)^2$ = 59.700

61.990 × 15 = 929,850

$\frac{bx^3}{3}$ = 18 × 9261 = 166.700

J = 1.096.550

$\sigma_b = \frac{1.189.920 \times 12.21}{1.096.550} = 273$ #/□"

$\sigma_e = 15 \times 273 \times \frac{65}{21} = 12.670$ #/□"

$\frac{bx^2}{2}$ = 27 × 441 = 11.900

n.Fe' (x − h') = 15 × 7.0686 × 18 = 1.910

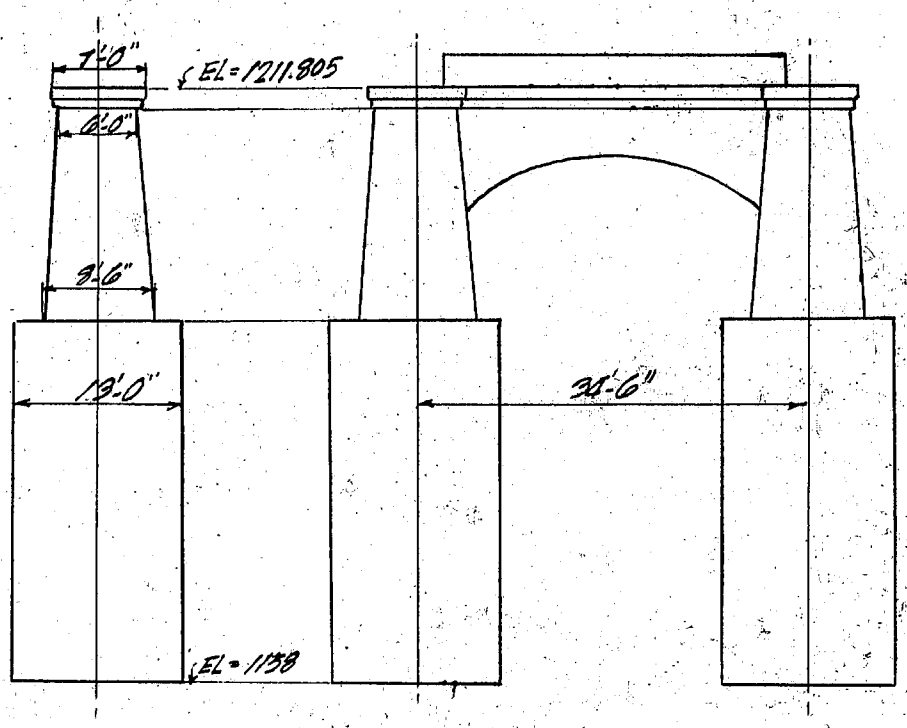
S = 13.810

$\tau_o = \frac{229.750 \times 13.810}{54 \times 1.096.550} = 53.7$ #/□"

$\tau_h = \frac{229.750 \times 13.810}{56.6 \times 1.096.550} = 51.2$ #/□"

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

River Piers No.1 & No.8
End Piers for 165' spans



Coping of Shaft
7'-0" dia = 38.5×7.5
6'-6" dia = 33.2×7.5
71.7×.75 — cub.ft. 53.7

Shaft
top. 6'-0"ϕ = 28.3
bott. 8'-6"ϕ = 56.7
85.0 ÷ 2 = 42.5×20'7" = 858.0
911.7

Weight = 127,700#

Strut between shaft.
pedestal 30×7.416×4.25 = 302 cub.ft.
Strut 7.5×28×2.5 = 525
833

Weight = 116,500#
for one half = 58,300#
Total Weight = 186,000#

Load at bottom of Shaft.
For highway superinposed load { 224,000# for truss span / 82,750# for girder span } 307,350#
Shaft and strut 186,000#
493,350#

Bottom area of shaft 8'-6" dia = 56.75 □' Unit pressure = 493,350 / 56.75×144 = 60.4 #/□"

Well. 13'-0" dia area = 132.7 □'
height 32' Vol. = 4,250 cub.ft. Weight = 595,000#
Super structure & shaft 493,350#
Total = 1088,350#

Bottom area of well 13'-6" dia = 143.1 □'
Unit pressure = 1088,350 / 132.7 = 8.210 #/□' = 56.9 #/□"

Considering live load
Dead load 1,088,350#
Superinposed load Hy 307,350#
781,000#
Superimposed D&L load Hy 534,000# for truss span
48,100#
124,900# for girder span
1498,000#

Unit pressure = 1,498,000 / 143.1 = 10,390 #/□' = 4.64 tons/□'

Earthquake effect for highway side 2,000 mm/sec² assumed
Moment Superstructure 307,350# × 31 × 2000/9800 = 1960,000 × 0.204
Strut 59300# × 20 × 0.204 = 1166,000 × "
Shaft 127,700# × 105 × 0.204 = 1340,000 × "
12046,000 × 0.204 = 2460,800

2 Rows of reinforcements
Dia of outer row = 8'-0"
Circumference = 25.133
Steel area = 64 - 1"ϕ = 39.4 □"
Equiv. Conc. area = 576 □"
area of half Circle = 35.13 □"
Average height = 35.13/9 = 3.90 = 37.7
Sg = 1420 □"

Dia. of inner row = 7'-6"
Circumference = 23.562
Steel area 32-1"ϕ = 19.2 □"
Equiv. Conc. area = 288 □"
area of half circle = 22.09 □"
Average height = 22.09/7.5 = 2.94 = 35.3
Sg = 1,250 □"

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Moment of inertia $576 \cdot 1420 = 820.000$

$288 \cdot 1250 = 360.000$

1180.000

Moment of inertia of concrete $= 0.049 \cdot 102^4 = 5.310.000^{(in)4}$

" " " Steel $= 1180.000$

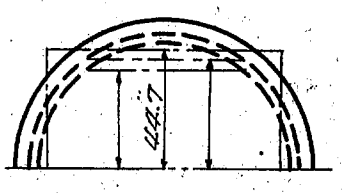
Total 6490.000

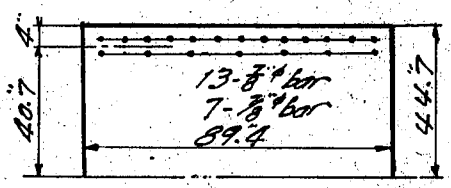
Fibre stress $= \frac{2460.000 \cdot 12.51}{6490.000} = 232$ #/o" Tension or compression

For compression $232 + 60.4 = 292.4$ #/o"

For tension $232 - 60.4 = 171.6$ #/o"

Equivalent square section $d = \sqrt[4]{5310.000 \cdot 12} = 89.4"$





$Fe = 0.6013 \times 20 = 12.026$ o"

$Fe' = 12.026$

Total 24.052 o"

Position of neutral axis $\frac{n(Fe+Fe')}{b} = \frac{15 \cdot 24.052}{89.4} = 4.04"$ $\quad sq = 16.3$

$hFe = 85.4 \cdot 12.026$

$h'Fe' = 4 \cdot 12.025$

$hFe + h'Fe' = 89.4 \cdot 12.025 = 1073$

$\frac{2n}{b}(hFe + h'Fe') = \frac{30}{89.4} \cdot 1073 = 360$

$\{\frac{n(Fe+Fe')}{b}\}^2 = 16.3$

376.3 sq root $= 19.4"$

less 4.04

(x) position of N axis 15.36 say $15.4"$ $x^2 = 236$

Unit stress of concrete

$3h - x = 256.2 - 15.4 = 240.8$

$bx^2(3h-x) = 89.4 \times 236 \times 240.8 = 5.080.000$

$6nFe'(x-h')(h-h') = 90 \cdot 12.026 \cdot 11.4 \cdot 81.4 = 1004.000$

6084.000

Unit stress of concrete $G_b = \frac{6 \cdot 2460.000 \times 12 \cdot 15.4}{6084000} = 447.5$ #/o.

$G_e = n G_b \frac{h-x}{x} = 15 \cdot 447.5 \cdot \frac{70}{15.4} = 30.500$ #/o.

At the section of well

section modulus of well section $= 0.098 \cdot 156^3 = 373.000^{(in)3}$

Max Fibre Stress $= \frac{2460.000 \times 12}{373.000} = 79.2$ #/o. Tension or compression

At the bottom of well

section modulus of bottom of well $= 0.098 \cdot 13.5^3 = 242^{(in)3}$

Max Fibre Stress $= \frac{2460.000}{242} = 10150$ #/o'

$= 4.53$ tons/o'

Uniform Pressure 8210 #/o' or $= 3.67$

Dead load only 8.20 tons/o'

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

END PIERS Nos 1 & 8

Reinforcements in between shaft

Span length assumed as 30'-0"

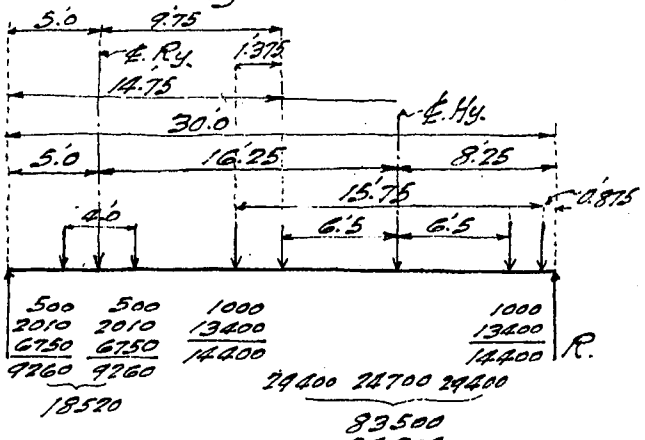
Dead load

Pedestal 30 × 2.416 × 4.25 = 308 cub ft.

Strut 7.5 × 28 × 2.5 = 525

833 × 140 = 116,500#

Its own weight assumed as 4,000 #/lin. ft.



	for Railway	for Highway	
Floor Beam	500#	1000#	
Stringer & flooring	2010	13400	
	6750	14400#	truss side
	9260#	29400 24700 29400	girder side

Dead load Moments

due to its own weight $M = \frac{1}{8} \times 4000 \times 30^2 = 450{,}000$ 1# R = 60000#

R: 112300 × 21.25 = 2,390,000

18.520 × 5.0 = 92,600

130.820 2482,600 ÷ 30 = 82,750#

− 82.750

48.070

M: 48,070 × 14.75 = 710.000 1#

less 18570 × 9.75 = 180.200

14400 × 1.375 = 19.800

700,000

510.000

450.000

Summary 960,000 1#

Live load

due to E 75 12500 × (15.625 + 10.625) = 328,000

6250 × 2.625 = 16.400

344,400 ÷ 20.625 = 16,700# ..... from Truss

6250 × 4.5 = 28.100#

8125 × (12.5 + 17.5 + 23.5 + 28.5) = 666000

12.500 × 37.5 = 469000

1163100 ÷ 42.5 = 27.400 ..... from Girder

44.100#

impact included	62.400
for one track	124.800

due to Highway loading

Motor trucks = 32.800#

Uniform load

$100 \times \frac{6.625^2}{2 \times 20.625} = 106$ # from Truss span

$100 \times \frac{38.5^2}{2 \times 42.5} = 1740$ # from Girder Span

1846 × 18 = 33200#

R: 32,800# × 10.25 = 336,000

33.200 × 8.75 = 290,000

124,800 × 25.00 = 3,120,000

3,746,000 ÷ 30 = 124,900#

M. 124.900 × 7 = 874.300 1#

less 62.400 × 4 = 249.600

624.700 Say 625,000 1#

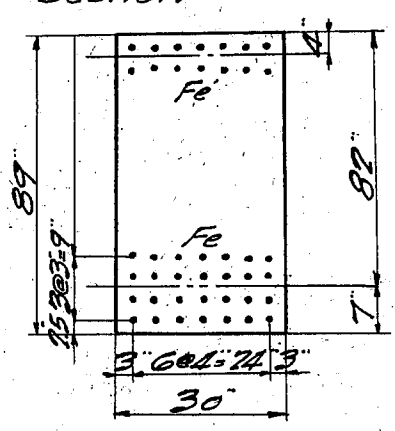
Summary of dead and live loads moments

M = 960.000 + 625000 = 1,585000 1# × 0.8 = 1.267,000 1# negative and positive

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Max. Shear $Q = 60,000 + 48,070 + 124,900 = 232,970\#$

Section



$Fe = 0.7854 \cdot 28 = 22.0$

$Fe' = \quad , \quad \cdot 14 = 11.0$

33.0

$\frac{n(Fe+Fe')}{b} = \frac{15 \cdot 33}{30} = 16.5$ Sq. = 272.25

$hFe = 82 \cdot 22 = 1.800$

$hFe' = 4 \cdot 11 = 44$

$1.844 = \frac{2n}{b}(hFe + h'Fe')$

$\frac{272}{2.116} = \left\{\frac{n(Fe+Fe')}{b}\right\}^2$

Sq. root 46.0

less 16.5

$x = 29.5 \quad x^2 = 870 \quad x^3 = 25670$

$x - h' = 25.5 \quad$ Sq. = 650

$h - x = 52.5 \quad$ Sq. = 2.756

$Fe'(x-h')^2 = 7.150$

$Fe(h-x)^2 = 60600$

$67.750 \times 15 = 1.015.000$

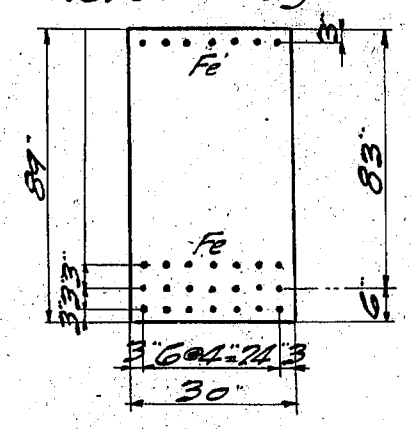
$\frac{bx^3}{3} \qquad 256.700$

$J = 1.271.700$

$\sigma_b = \frac{1.767.000 \times 12 \times 29.5}{1.271.700} = 353\ \#/\square''$

$\sigma_e = 15 \cdot 353 \cdot \frac{57.5}{29.5} = 9.420\ \#/\square''$

Revised Design continned



$Fe = 0.7854 \cdot 21 = 16.5$

$Fe' = \quad , \quad \times 7 = 5.5$

22.0

$\frac{n(Fe+Fe')}{b} = \frac{22}{2} = 11.0 \quad Sq = 121$

$hFe = 83 \times 16.5 = 1368.5$

$h'Fe' = 3 \cdot 5.5 = 16.5$

$1385.0 = \frac{2n}{b}(hFe + hFe')$

$\frac{121.0}{1506.0}$ Sq. root = 38.8

less 11.0

$x = 27.8$

$x^2 = 773$

$x^3 = 21,500$

$(x-h') = 24.8 \quad Sq. = 615$

$(h-x) = 55.2 \quad Sq. = 3047$

$Fe'(x-h')^2 = 3,380$

$Fe(h-x)^2 = 50.300$

$53.680 \times 15 = 805,000$

$\frac{bx^3}{3} \qquad 215.000$

$J = 1020.000$

$\sigma_b = \frac{1.767,000 \times 12 \cdot 27.8}{1,020,000} = 414\ \#/\square''$

$\sigma_e = 15 \cdot 414 \cdot \frac{55.2}{27.8} = 17,340\ \#/\square''$

$\frac{bx^2}{2} =$

$= 11,600$

$n.Fe'(x-h') = 15 \cdot 5.5 \cdot 24.8 = 2.040$

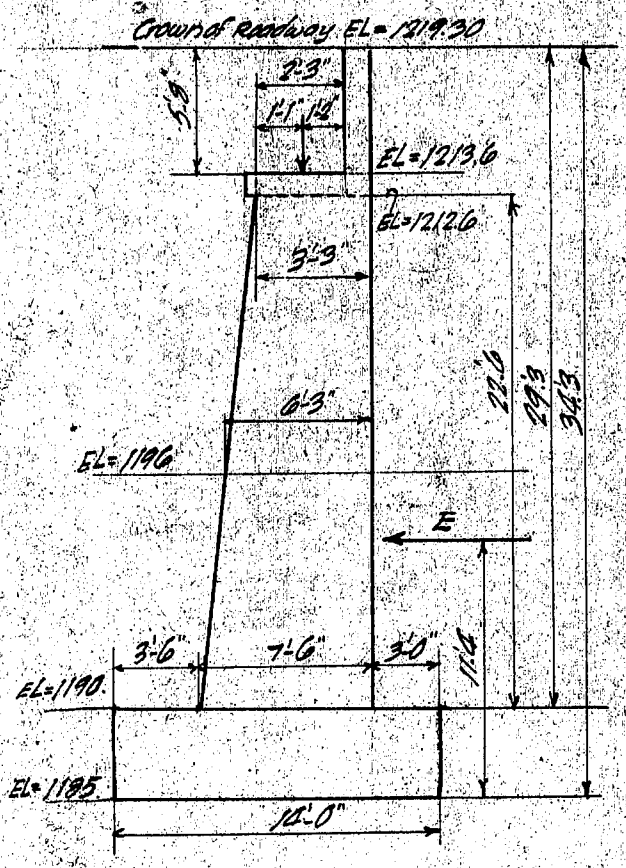
$S = 13,640$

$\tau_o = \frac{232,970 \times 13,640}{30 \times 1,020,000} = 104\ \#/\square''$

$\tau_h = \frac{232,970 \times 13,640}{66 \times 1,020,000} = 47.3\ \#/\square''$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Abutment at Suzaka Side



Reaction from girder section

Dead load reaction
$= 0.4 \times 5{,}630 \times 44.5 = 101{,}700^{\#} \div 20 = 5{,}060^{\#}/\text{ft.}$

Live load reaction
$= 0.4 \times 1{,}800 \times 44.5 = 32{,}040^{\#} \div 20 = 1{,}600^{\#}/\text{ft.}$

Dead load.

earth	$3 \times 29.3 \times 100 =$	$87.90^{\#}$	arm 1.5	$M = 13.190$
parapet	$1 \times 5.7 \times 150 =$	$855^{\#}$	" 3.5	$M = 2.990$
coping	$1' \times 3.5 \times 150 =$	$525^{\#}$	" 4.75	$M = 2.490$
Shaft	$\frac{3.25+7.5}{2} \times 22.6 \times 150 =$	$18,222^{\#}$	" 5.8	$M = 105,680$
base	$5' \times 14 \times 150 =$	$10,500^{\#}$	" 7.0	$M = 73,500$
dead load from girder	$= 5,060^{\#}$		" 5.2	$M = 26,300$
	$\Sigma W = 43,950^{\#}$			$\Sigma M = 224,150$

Position of center gravity $= \frac{224,050}{43,950} = 5.1'$

live load	$= 1,600^{\#}$	arm 5.2	$M = 8,320$
	$\Sigma W = 43,950^{\#}$		$\Sigma M = 232,470$

Position of center of gravity $= \frac{232,470}{45,550} = 5.1'$

Earth pressure $E = \frac{1}{2} \times 100 \times \overline{34.3}^2 \times \frac{1}{3}$
$= 19,600^{\#}/\text{ft.}$

For Sliding

Dead load only
$\tan\theta = \frac{19,600}{43,950} = 0.446$
$\theta = 24° - 3' < 30°$

Live load included
$\tan\theta = \frac{19,600}{45,550} = 0.4305$
$\theta = 23° - 17' < 30°$

For Overturning

Dead load only
$ac = 5.1'$
$kc = 11.4 \times 0.446 = 5.1'$
$ak = ac + kc = 10.2'$
$bk = 14 - 10.2 = 3.8'$

Live load included
$ac = 5.1'$
$kc = 11.4 \times 0.4305 = 4.9'$
$ak = 10.0'$
$bk = 4.0'$

For Bearing

Dead load only
$b = \frac{2}{3} \times \frac{43,950}{3.8}$
$= 7,710^{\#}/0'$
$= 3.42^{\text{tons}}/0'$

Live load included
$b = \frac{2}{3} \times \frac{45,550}{4}$
$= 7,590^{\#}/0'$
$= 3.39^{\text{tons}}/0'$

Effect of earthquake motion at the base of Shaft.

Earth	$500^{\#} \times 29.3 \times \frac{1}{2} = 7,330^{\#}$	arm 19.5'	$M = 142,940$
Dead load from girder	$= 5,060^{\#}$	" 28.3	$M = 143,200$
Parapet	$= 855^{\#}$	" 26.45	$M = 22,620$
Coping	$= 525^{\#}$	" 23.10	$M = 12,130$
Shaft	$= 18,220^{\#}$	" 11.0	$M = 200,420$
			$\Sigma M = 521,310 \times 0.204 = 106,350^{\#}$

Sec. mod. of the sect. at the base of shaft $= \frac{1 \times \overline{7.5}^2}{6} = 9.375$
Max. fibre stress $= \frac{106,350}{9.375} = 11,380^{\#}/0' = 80^{\#}/0"$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Base $14 \times 1' \times 5' \times 150 = 10.500^{\#}$ arm 2.5 $M = 26.250$

Earth $590^{\#} \times 34.3 \times \frac{1}{2} = 10.120^{\#}$ arm 22.9 $M = 231,750$

$779,310$

Less $142,940$

$636,370 \times 0.204 = 129,820'^{\#}$

Sec. mod. of the base $= \frac{1 \times 14^2}{6} = 32.67'^3$

Max fibre stress $= \frac{129,820}{32.67} = 3,973^{\#}/0' = 1.77^{tons}/0'$

Combined fibre stress $= 3.44 + 1.77 = 5.21^{tons}/0'$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Abutments

Abutment at Nagano Side

Reaction from girder Span

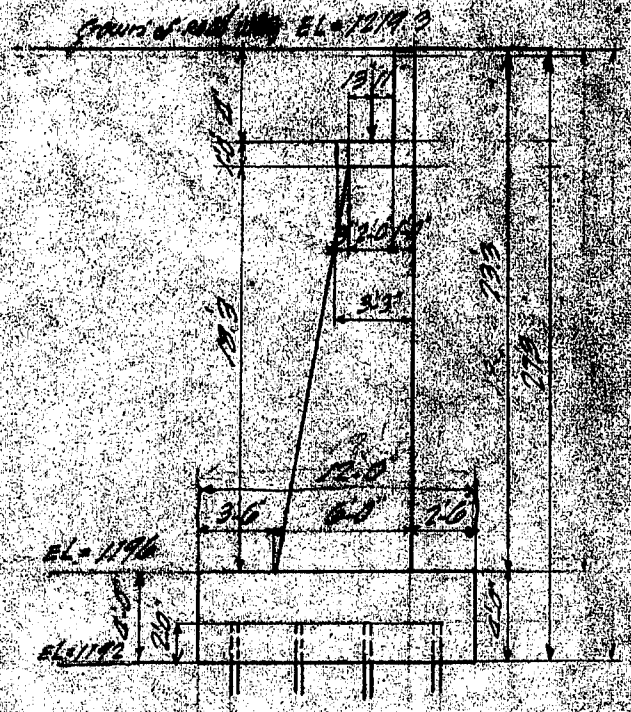
Dead Load assuming as 5,000 #/lin.ft.

$R = 4,400 \times \frac{22.66}{2} = 25,300\ \# \div 20 = 2,265\ \#/ft.$ of abutment

Live load assuming as 1,800 #/lin.ft.

$R = 1,800 \times \frac{22.66}{2} = \underline{20,400\ \#} \div 20 = 1,020\ \#/ft.$ of abutment

$65,700\ \#$



1. Parapet wall

$M = \frac{5}{3}$

Assumed surcharge for live load = 6 ft. of earth

Pressure intensity at a = 5/3.6.100 = 200 #/lf

" " " b = 5/3.10.100 = 333 #/lf

533 #/lf

one half = 267 #/lf

Resultant of earth pressure = E = 267 x 4 = 1068 #

Moment at b = 1068 # x 1.8 = 1924 '#

Wall thickness 12"

Steel area. double reinforcements of $\frac{5}{8}$"ϕ 12" c/c C

area of $\frac{5}{8}$"ϕ bar = 0.1963 (for one bar)

2. Dead load of abutment

Earth	2.5 x 233 x 100 # = 5825 #	arm 1.25	M =	7,280 #
parapet wall	1 x 4 x 150 # = 600	arm 3.0	M =	1,800
Coping	1 x 325 x 150 # = 490	arm 4.125	M =	2,000
Slab	$\frac{3'6}{2}$ x 183 x 150 # = 12,350	arm 4.85	M =	59,900
base	4 x 12 x 150 # = 7,200	arm 6.0	M =	43,200
Dead load from girder	2265	arm 4.42	M =	10,000
	ΣW = 28730		ΣM =	124,180

Position of center of Gravity = $\frac{\Sigma M}{\Sigma W} = \frac{124,180}{28,730} = 4.32$

Live load	= 1,020	arm 4.42	M = 5300
	ΣW = 29,750		ΣM = 129,480

Position of center of Gravity = $\frac{\Sigma M}{\Sigma W} = \frac{129,480}{29,750} = 4.35$

Earth pressure $E = \frac{1}{2} \times 100 \times \overline{17.3}^2 \times \frac{1}{3}$

$= 12,400\ \#/lin.ft.$

For sliding. Dead load only

$\tan\theta = \frac{12,400}{28,730} = 0.43$

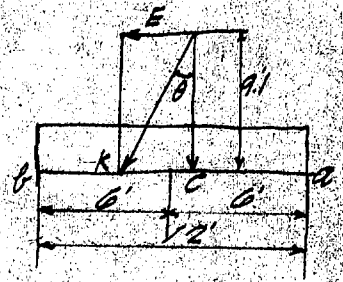
$\theta = 23° - 20' < 23°$

Live load included

$\tan\theta = \frac{12,400}{29,750} = 0.42$

$\theta = 22° - 50' < 30°$

For Overturning



dead load only,

ac = 4.32

kc = 9.1 x 0.43 = 3.92

ak = ac + kc = 8.24

bk = 12 - 8.24 = 3.76

live load included

ac = 4.35

kc = 9.1 x 0.42 = 3.82

ak = ac + kc = 8.17

bk = 12 - 8.17 = 3.83

included

For Bearing

dead load only

$6 = \frac{2}{3} \times \frac{28,730}{3.76}$

$= 5,090\ \#/lf = 2.27\ tons/lf$

live load included

$8 = \frac{2}{3} \times \frac{29,750}{3.83}$

$= 5,177\ \#/lf = 2.31\ tons/lf$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Effect of earthquake motion at the base of Shaft.

Earth	$400^{\#} \times 23.3 \times \frac{1}{2} = 4,660^{\#}$	arm 15.5	M = 72,230
Dead load fom Girder	2,265	arm 22.7	M = 51.420
Parapet	600	arm 21.3	M = 12,780
Coping	490	arm 18.8	M = 9,210
Shaft	12,350	arm 9.0	M = 111,150

$\Sigma M = 256,790 \times 0.204 = 52,390'^{\#}$

Sec. mod. of the sectional the bottom of Shaft $= \frac{1 \times 6^2}{6} = 6$

Max fibre Stress $= \frac{52,390}{6} = 8,730^{\#}/\square' = 61^{\#}/\square''$ tension or Compression

Base	$12' \times 1' \times 4' \times 150 = 7,200^{\#}$	arm 2'	M = 14,400
Earth	$470^{\#} \times 27.3 \times \frac{1}{2} = 6,420^{\#}$	arm 18.2	M = 116,800
			387,990
Less			72,230
Less			$315,760 \times 0.204 = 64,420'^{\#}$

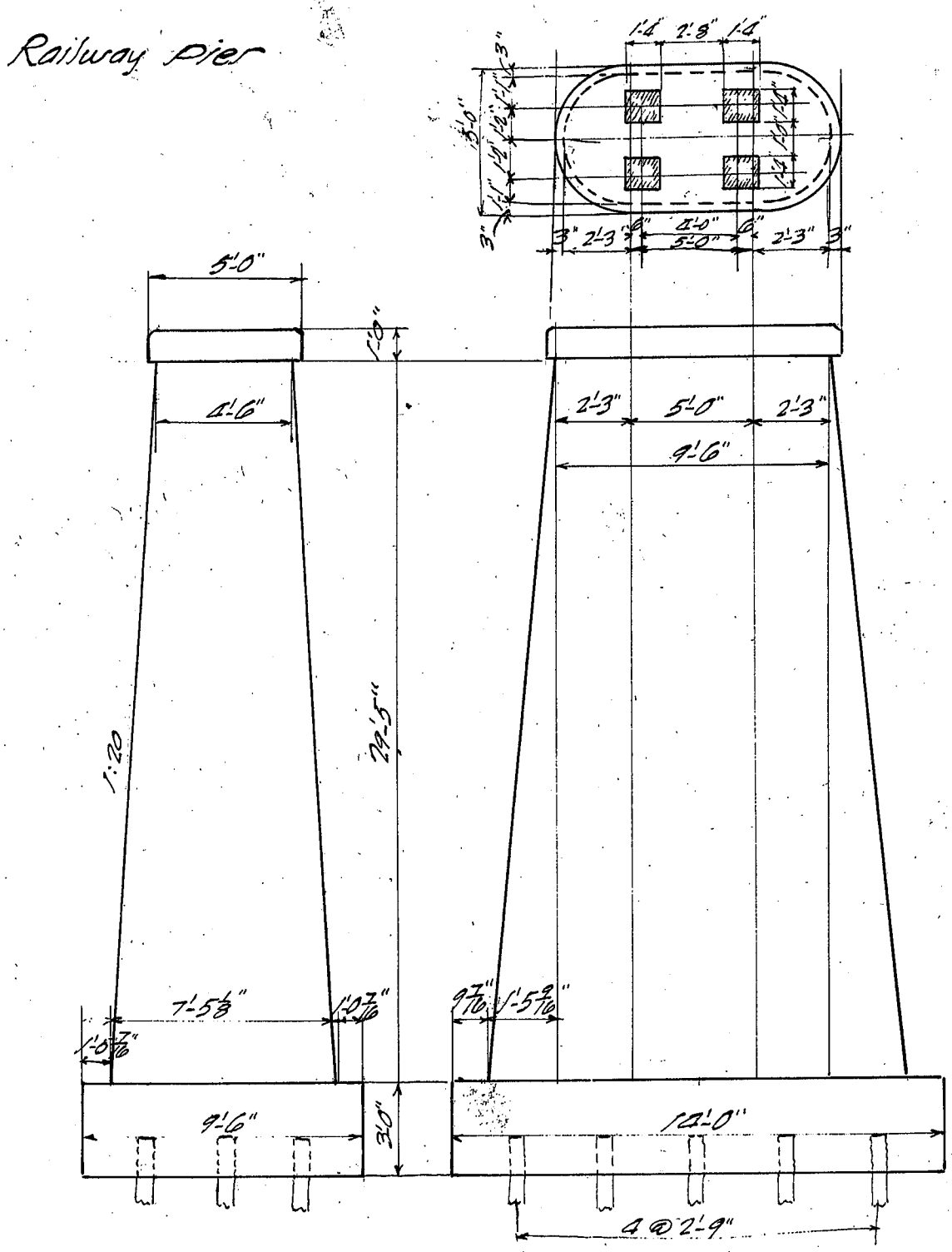
Sec. mod. of the base $= \frac{1 \times 12^2}{6} = 24$

Max fibre stress $= \frac{64,420}{24} = 2,680^{\#}/\square' = 1.19^{\text{tons}}/\square'$

Combined fibre stress $= 2.27 + 1.19 = 3.46^{\text{tons}}/\square'$

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Railway pier



Concrete in coping of shaft.

5.0×5 = 25.0

dia 5'0 = 19.6

44.6×1×150 = 6,700 # arm 29.92

Shaft top area 4.5×5 = 22.5

dia 4.5 = 15.9

38.4

bottom area 7.43×5 = 37.1

dia 7.43 = 43.3

80.4

118.8 ÷2 = 59.4×29.42×150 = 262,500 # arm 14.0

Dead load of girder

(200+440)÷2×21.08 = 6,750 # per one girder

4 arm 33.0

27,000

Effect of earthquake at the bottom of shaft

Superstructure	27,000 #	arm 33.0	M = 891,000
Coping	6,700 #	" 29.92	M = 200,000
Shaft	262,500 #	" 14.0	y = 3680,000
	296,200 #		4,771,000×0.204 = 974,000 '#

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Sec. mod. of section at the bottom of shaft.

$$\frac{7.43 \times 5^2}{6} = 30.9$$

$$0.098 \times 7.43^2 = \frac{40.3}{71.2}\ (1)^3$$

Max fibre stress $= \frac{974.000}{71.2} = 13,650\ ^{\#}/_{0'} = 95\ ^{\#}/_{0''}$

For bearing

above shaft 296.200 #

base $3 \times 9.5 \times 14 \times 150 = 59,900$ * arm 1.5'

356.100 #

Sec. mod. of base $= \frac{9.5 \times 14^2}{6} = 210\ (1)^3$

$M = 4771.000$

$M = 90,000$

$4861.000 \times 0.204 = 992,000$ '#

Edge Stress $= \frac{992.000}{210} = 4,720\ ^{\#}/_{0'}$

$= 2.1\ tons/_{0'}$

Uniform pressure $= \frac{356.100}{14 \times 9.5} = 2,630\ ^{\#}/_{0'}$

$= 1.2\ tons/_{0'}$

Combined pressure $= 2.1 + 1.2 = 3.3\ tons/_{0'}$

When live load considered

Uniform pressure $= \frac{356.100 + 70.300}{14 \times 9.5} = 3,200\ ^{\#}/_{0'}$

$= 1.43\ tons/_{0'}$

Design of railway Abutment Omitted.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

LIST OF DROWINGS FOR MURAYAMA-BASHI

Sheef No. 1.	Map showing general location of bridge
" " 2.	General plan and profile and cross section
" " 3.	Cross sections of structure and details of Railway Girder for West approach
" " 4.	Details of Railway Truss
" " 5.	Details of Highway Truss
" " 6.	Details of Floor Beam stringers for both Railway & Highway
" " 7.	Details of Portal, Sway bracings and Top Laterals
" " 8.	Details of Bottom Laterals & Thrust Frame
" " 9.	Details of Shoes and Stress Diagrams of Trusses.
" " 10.	Details of Railway Girder for East Approach
" " 11.	Details of River Piers Nos. 1 & 8
" " 12.	Details of River Piers Nos. 2-3-4-5-6-7
" " 13.	Details of Reinforced Concrete Slabs, Handrail etc.
" " 14.	Details of Reinforced Concrete Highway Girder for East Approach.
" " 15.	Details of Highway Bents for East Approach
" " 16.	Details of Reinforced Concrete Highway Span for West Approach & West Highway Abutment
" " 17.	Details of East Abutment for Highway
" " 18.	Details of Railway Piers & Abutment

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

村山橋材料調書

長野縣

長野電氣鐵道株式會社

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

(1)

Number	Description	Dimension		Weight (#)		Remarks
		Size (inch)	Length	Unit	Total	
		Railway Truss				
		End Post Lo~U1				
2	p/s.	32 x 1/2	37'-6"	39.10	2835	Cover p/s.
4	"	17 x 7/16	33'-8 1/2"	25.29	3410	Web p/s.
4	Ls	4 x 4 x 1/2	37'-0"	12.8	1894	
4	"	"	36'-6"	"	1869	
4	p/s.	9 x 1/2	36'-11"	1530	2259	Side p/s.
4	Ls.	4 x 4 x 3/8	2'-3"	9.8	88	
4	"	"	2'-1 1/2"	"	83	
4	"	"	1'-3"	"	49	
4	Fills	4 x 1/2	0'-6"	6.80	14	
4	Ls	4 x 4 x 3/8	1'-0"	9.8	39	
4	p/s.	23 x 7/16	2'-0"	3421	274	Tie p/s.
2	"	24 x 7/16	2'-9"	35.70	196	"
2	"	13 x 5/16	2'-0"	13.91	55	Gusset p/s.
2	"	17 x 5/16	4'-3"	18.06	154	Beat p/s.
2	"	"	2'-2"	"	78	"
2	"	20 x 1/2	1'-9"	34.00	114	Splice p/s.
2	"	4 x 1/2	2'-3"	6.80	31	"
4	"	16 x 3/8	1'-5"	20.40	116	"
68	Flats.	2 1/2 x 7/16	2'-6"	3.72	632	Locing bars.
					14,195	
		U1~JU1				
2	p/s.	23 x 1/2	18'-0"	39.10	1408	Cover p/s.
4	"	17 x 7/16	16'-0 1/2"	25.29	1622	
4	Ls.	4 x 4 x 3/8	17'-11 1/2"	9.8	704	
4	"	"	17'-4"	"	679	
4	p/s.	49 x 1/2	4'-6"	81.60	1142	Gusset p/s.
4	"	31 x 7/16	3'-0"	46.11	461	Splice "
4	"	9 x 1/2	2'-6"	15.30	138	" "
8	"	15 1/2 x 3/8	2'-6"	19.76	395	" "
2	"	23 x 1/2	2'-3"	39.10	176	" "
2	"	23 x 7/16	2'-3"	34.21	154	" "
4	"	9 x 3/8	1'-3"	11.48	57	Fills
2	p/s.	23 x 7/16	2'-0"	34.21	137	Tie p/s.
32	Flats	2 1/2 x 7/16	2'-6"	3.72	298	Locing bars
4	Ls	4 x 4 x 3/8	1'-2"	9.8	45	
4	p/s	3 x 3/8	0'-10"	1.275	4	Fills
4	"	"	0'-3"	"	5	"
					7,421	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Number	Description	Dimension		Weight (#)		Remarks
		Size (inch)	Length	Unit	Total	
		$JU_1 \sim U_2 \sim JU_2$				
2	p/s.	23 × 1/2	20'-7 3/8"	39.10	1612	Cover p/s.
4	"	17 × 7/16	"	25.29	2085	Web p/s.
8	Ls.	4 × 4 × 3/8	"	9.8	1616	
4	p/s.	9 × 3/8	"	11.48	947	Side p/s.
4	"	42 × 1/2	3'-9"	71.40	1001	
2	"	20 × 5/16	3'-0"	21.25	128	
8	"	15 1/2 × 3/8	2'-6"	19.76	395	Splice p/s.
2	"	23 × 1/2	2'-3"	39.10	176	" "
2	"	23 × 7/16	"	34.21	154	" "
4	"	9 × 3/8	2'-6"	3.825	38	Fills
8	"	3 × 3/8	1'-3"	1.275	13	"
2	"	9 1/2 × 1/2	0'-9"	14.05	22	"
2	"	23 × 7/16	2'-0"	34.21	137	Tie p/s.
32	Flats.	2 1/2 × 7/16	2'-6"	3.72	298	Lacing bars
					9622	
		$JU_2 \sim U_3 \sim U_4$				
2	p/s.	23 × 1/2	23'-7 3/8"	39.10	1850	Cover p/s.
4	"	17 × 7/16	"	25.29	2393	Web p/s.
8	Ls.	4 × 4 × 1/2	"	15.80	2422	
4	p/s.	9 × 3/8	"	11.48	1086	Side p/s.
4	"	36 × 1/2	3'-0"	61.20	673	
2	"	30 × 1/2	2'-6"	51.00	204	
2	"	15 1/2 × 3/8	"	19.76	99	Splice p/s.
2	"	9 × 3/8	"	3.825	19	Filler
2	"	9 1/2 × 1/2	0'-9"	14.05	22	"
3	"	17 × 5/16	2'-10"	18.06	158	Gusset p/s.
4	"	23 × 7/16	2'-0"	34.21	274	Tie p/s.
2	"	4 × 1/2	2'-3"	6.80	31	Splice plats
1	"	20 × 1/2	"	34.00	77	Splice plate
32	Flats	2 1/2 × 7/16	2'-6"	3.72	298	Lacing bars
					9606	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Number	Description	Dimension		Weight (#)		Remarks
		Size (inch)	Length	Unit	Total	
		$L_0 \sim L_1 \sim JL_1$				
4	pls.	15 x 3/8	36-4"	19.13	2780	
8	Ls.	6 x 4 x 3/8	36-1"	12.3	3551	
4	pls.	15 x 3/8	3'-10½"	19.13	296	splice pls.
8	Ls.	6 x 4 x 3/8	3'-10½"	12.3	381	" Ls
8	pls.	5½ x 3/8	1'-11"	7.01	108	F/Fs
2	"	11½ x 3/8	3'-10½"	14.34	111	
2	"	"	2'-3"	"	65	Tie pls.
20	"	"	0'-9"	"	215	" "
4	"	47 x ½	4'-11"	79.90	1225	Gusset pls.
4	"	41 x 7/16	4'-7"	60.99	954	Splice pls.
4	"	20 x ¾	3'-1"	51.00	510	pin pls.
8	Ls.	4 x 3 x 3/8	1'-6"	8.50	102	Diaphragm
2	pls.	12½ x 3/8	1'-6"	15.94	48	"
4	Ls.	4 x 3 x 5/16	0'-9"	7.2	22	
2	pls.	12 x 5/16	1'-3"	12.75	32	
2	Ls	6 x 3½ x 3/8	1'-0"	11.7	31	Connection Ls
4	"	4 x 3 x 5/16	3'-0"	7.20	86	" "
2	pls.	22 x 3/8	2'-9½"	28.05	131	Gusset pls
2	"	36 x 3/8	3'-1"	45.90	275	" "
2	"	24 x ½	4'-9"	35.70	286	" "
2	"	"	2'-0"	"	155	" " "
					11.264	
		$JL_1 \sim L_2 \sim L_3 \sim JL_2$				
4	pls.	15 x 3/4	29'-0½"	38.30	4.009	
8	Ls	6 x 4 x 3/8	21'-8½"	12.30	2148	
8	"	6 x 4 x 5/8	7'-4"	20.00	1173	
8	"	6 x 4 x 3/8	2'-7½"	12.3	256	Splice Ls.
8	pls.	5½ x ½	1'-5½"	4.675	48	F/Fs
8	"	3½ x ½	1'-3½"	2.763	29	"
4	"	15 x 3/8	6'-11½"	19.13	532	Splice pls.
4	"	"	3'-5½"	"	265	" "
8	Ls	6 x 4 x 3/8	3'-5½"	12.00	553	" Ls
2	pls.	14 x 3/8	3'-5½"	14.34	99	" pls.
2	"	"	2'-7½"	"	75	" "
14	"	"	0'-9"	"	151	Tie pls
4	Ls	4 x 3 x 5/16	3'-0"	7.2	86	Connection Ls.
4	"	"	2'-3"	"	67	" "
4	pls	37 x ½	4'-0"	62.90	1090	Gusset pls
4	"	39 x ½	4'-0"	66.30	995	" "
2	"	35 x 5/16	2'-1"	37.19	217	" "
2	"	32 x 5/16	2'-3"	34.00	181	" "
					12.414	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Number	Description	Dimension		Weight (#)		Remarks
		Size (inch)	Length	Unit	Total	
		JL2 ~ L4 ~ JL2				
2	p/s.	15 × 3/4	32'-7"	38.30	2496	
4	Ls.	6 × 4 × 5/8	32'-7"	20.00	2607	
1	"	4 × 3 × 5/16	2'-8"	7.2	38	Connection Ls.
9	p/s.	11 1/4 × 3/8	0'-9"	14.34	97	Tie p/s.
2	"	38 × 1/2	3'-7"	64.60	252	Gusset p/s.
1	"	32 × 5/16	2'-8"	34.00	91	
					5,781	
		Hip Vertical L1 – U1				
8	ls.	5 × 3 1/2 × 7/16	29'-1 1/2"	12.00	2796	
2	p/s.	12 × 3/8	3'-6"	15.30	137	Tie p/s.
2	"	"	1'-4"	"	41	"
2	"	3 1/2 × 3/8	1'-4"	4.46	12	Fills
2	"	"	1'-0"	"	9	"
2	"	"	3'-3"	"	29	"
2	"	"	3'-6"	"	31	"
74	"	2 1/2 × 3/8	1'-3 1/2"	3.19	305	Lacing bars.
					3,330	
		L2 ~ U2				
4	Es.	15 @ 40#	31'-10"		5093	
8	Ls.	4 × 3 × 3/8	3'-4"	8.50	127	Diaphragm Ls
8	"	4 × 3 × 5/16	1'-6"	7.20	96	" "
2	p/s.	12 × 3/8	3'-4"	15.30	120	" p/s.
2	"	12 × 5/16	1'-6"	12.75	38	" "
8	"	12 × 3/8	1'-3"	15.30	153	Tie p/s.
4	"	"	1'-6"	"	92	" "
176	"	2 1/2 × 3/8	1'-2"	3.19	655	Lacing bars
					6464	
		L3 – U3				
4	Es	12 @ 25#	31'-10"		3183	
8	Ls.	4 × 3 × 5/16	1'-6"	7.20	86	Diaphragm Ls.
8	"	4 × 3 × 3/8	3'-0"	9.50	204	" "
2	p/s.	12 × 5/16	1'-6"	12.75	38	" p/s.
2	"	12 × 3/8	3'-0"	15.30	92	" "
4	"	"	1'-0"	"	61	Tie p/s.
8	"	"	1'-3"	"	153	" "
188	"	2 1/2 × 3/8	1'-2"	3.19	700	Lacing bars
					4517	
		L4 ~ U4				
2	Es.	12 @ 25#	31'-10"		1592	
4	Ls.	4 × 3 × 5/16	1'-6"	7.2	43	Diaphragm Ls.
4	"	4 × 3 × 3/8	3'-0"	8.5	102	" "
1	pl.	12 × 5/16	1'-6"	12.75	19	" Pl.
1	"	12 × 3/8	3'-0"	15.30	46	" "
2	"	"	1'-0"	"	32	Tie p/s.
4	"	"	1'-3"	"	77	" "
1	"	9 × 1/2	1'-1/2"	15.30	17	Fills.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Number	Description	Dimension		Weight (#)		Remarks
		Size (inch)	Length	Unit	Total	
94	p/s.	2½ × ⅜	1'-2"	3.19	350	Lacing bars
					2.278 ✓	
		Diagonal $U_1 \sim L_2$				
4	Cs.	15 ⌀ 35#	35'-6½"		4976	
4	p/s.	12 × ½	33'-9½"	20.40	2757	Side p/s.
8	"	12 × ⅜	2'-0"	15.3	245	Tie p/s.
228	"	2½ × ⅜	1'-2"	3.19	949	Lacing bars.
					8.827 ✓	
		$U_2 \sim L_3$				
4	Cs.	15 ⌀ 35#	35'-0½"		4906	
8	p/s.	12 × ⅜	1'-6"	15.3	184	Tie p/s.
240	"	2½ × ⅜	1'-2"	3.19	893	Lacing bars.
					5.983 ✓	
		$U_3 \sim L_4$				
4	Cs	12 ⌀ 25#	35'-4"		3533	
8	p/s.	12 × ⅜	1'-3"	15.3	153	Tie p/s.
252	"	2½ × ⅜	1'-2"	3.19	938	Lacing bars
					4624 ✓	

Total sum. 105326# = 47.021 Ton

Rivet Heads Variation 3½ % = 1.646 Ton.

Total Weight of Railway Truss = 48.667 Ton.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

(6)

Number	Description	Dimension		Weight (#)		Remarks
		Size (inch)	Length	Unit	Total	
		Highway Truss				
		End post Lo ~ U1				
2	pls	23 × 1/2	37'-6"	39.10	2835	Cover pls.
4	"	17 × 3/8	33'-8 1/2"	21.68	2923	Web pls.
4	Ls.	4 × 4 × 3/8	37'-0"	9.8	1450	
4	"	"	36'-6"	"	1431	
4	pls.	9 × 3/8	36'-11"	11.48	1695	Side pls.
4	Ls.	4 × 4 × 3/8	2'-3"	9.8	88	
4	"	"	2'-1 1/2"	"	83.	
4	"	"	1'-3"	"	49	
4	pls.	4 × 1/2	0'-6"	6.8	14	Fills.
4	Ls.	4 × 4 × 3/8	1'-0"	9.8	39	
4	pls.	23 × 7/16	2'-0"	34.21	274	Tie pls.
2	"	24 × 7/16	2'-9"	35.70	196	"
2	"	13 × 5/16	2'-0"	13.81	55	Gusset pls.
2	"	17 × 5/16	4'-3"	18.06	154	Bent pls.
2	"	"	2'-2"	"	78	12 × 5/16 × 2'-2 "
2	"	20 × 1/2	1'-9"	34.00	119	Splice pls.
2	"	4 × 1/2	2'-3"	6.8	31	" "
4	"	16 × 3/8	1'-5"	20.4	116	" "
68	"	2 1/2 × 7/16	2'-6"	3.72	632	Lacing bars.
					12,262	
		U1 ~ JU1				
2	pls.	23 × 1/2	19'-0"	39.10	1409	Cover pls.
4	"	17 × 3/8	16'-0 1/2"	21.68	1391	Web pls.
4	Ls.	4 × 4 × 3/8	17'-11 1/2"	9.8	704	
4	"	"	17'-4"	"	679	
4	pls.	48 × 1/2	4'-6"	81.60	1142	Gusset pls.
4	"	29 × 3/8	3'-0"	36.98	370	Splice pls.
4	"	9 × 3/8	2'-6"	11.48	115	" "
4	"	15 1/2 × 3/8	"	19.76	198	" "
4	"	"	1'-0"	"	79	" "
2	"	23 × 1/2	2'-3"	39.10	176	" "
2	"	23 × 7/16	"	34.21	154	" "
4	"	9 × 1/16	1'-3"	1.913	10	Fillers
8	"	3 × 1/16	0'-6"	0.638	3	"
2	"	23 × 7/16	2'-0"	34.21	137	Tie pls.
32	"	2 1/2 × 7/16	2'-6"	3.72	298	Lacing bars
4	Ls	4 × 4 × 3/8	1'-2"	9.8	45	
					6,909	
		JU1 ~ U2 ~ JU2				
2	pls	23 × 1/2	20'-7 3/8"	39.10	1612	Cover pls.
4	"	17 × 7/16	"	25.29	2085	Web pls.
8	Ls.	4 × 4 × 3/8	"	9.8	1616	
4	pls.	42 × 1/2	3'-9"	71.40	1001	Gusset pls
2	"	20 × 5/16	3'-0"	21.25	128	"
4	"	15 1/2 × 3/8	2'-6"	19.76	198	Splice pls.
4	"	"	1'-0"	"	79	" "
2	"	23 × 1/2	2'-3"	39.10	176	" "

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Number	Description	Dimension		Weight (#)		Remarks
		Size (inch)	Length	Unit	Total	
2	pls.	23 × 7/16	2'-3"	34.21	154	splice pls.
8	"	3 × 7/16	0'-6"	0.638	3	Fillers
2	"	8½ × 7/8	0'-9"	25.29	38	"
2	"	23 × 7/16	2'-0"	34.21	137	Tie pls.
32	"	2½ × 7/16	2'-6"	3.72	298	Lacing bars.
					7.525	
		JU2 ~ U3 ~ U4				
2	pls.	23 × ½	23'-7⅜"	39.10	1850	Cover pls.
4	"	17 × 7/16	"	25.29	2393	Web pls.
8	Ls.	4 × 4 × 7/16	"	11.3	2139	
4	pls.	36 × ½	3'-0"	61.20	673	Gusset pls
2	"	30 × ½	2'-6"	51.00	204	"
2	"	15½ × ⅜	"	19.76	99	Splice pls.
2	"	9 × 7/16	"	13.39	67	" "
2	"	8½ × 7/8	0'-9"	25.29	38	Fillers
1	"	8½ × 7/16	"	1.806	1	"
3	"	17 × 5/16	2'-10"	19.06	158	Gusset pls.
4	"	23 × 7/16	2'-0"	34.21	274	Tie pls.
2	"	4 × ½	2'-3"	6.8	31	Splice pls.
1	plate	20 × ½	2'-3"	34.00	77	" pls.
32	pls	2½ × 7/16	2'-6"	3.72	298	Lacing bars
					8,302	
		Lo ~ L1 ~ JL1				
4	pls.	15 × ⅜	36'-4"	19.13	2780	
8	Ls.	6 × 4 × ⅜	36'-1"	12.3	3551	
4	pls.	15 × ⅜	3'-10½"	19.13	296.	Splice pls.
8	Ls	6 × 4 × ⅜	"	12.3	391	" Ls.
8	pls.	5½ × ⅜	1'-11"	2.338	37	Fillers
2	"	11¼ × ⅜	3'-10½"	14.34	111	Splice pls.
2	"	"	2'-3"		65	Tie pls
20	"	"	0'-9"	"	215	" "
4	"	47 × ½	4'-11"	79.90	1225	Gusset pls.
4	"	41 × ⅜	4'-7"	52.28	732	Splice pls.
4	"	20 × ¾	3'-1"	51.00	510	pin pls.
8	Ls.	4 × 3 × 5/16	1'-6"	7.2	86	Diaphragm Ls.
2	pls.	12½ × 5/16	"	13.28	40	" pls.
4	Ls.	4 × 3 × 5/16	0'-9"	7.2	22	
2	pls.	12 × 5/16	1'-3"	12.75	32	
2	Ls	6 × 3½ × ⅜	1'-4"	11.7	31	Connection Ls.
4	"	4 × 3 × 5/16	3'-0"	7.2	86	"
2	pls.	22 × ⅜	2'-9½"	28.05	131	Gusset pls
2	"	36 × ⅜	3'-1"	45.90	275	
2	"	21 × ½	4'-9"	35.70	286	
2	"	"	2'-4"	"	155	
					11.047	
		JL1 ~ L2 ~ L3 ~ JL2				
4	pls.	15 × ½	29'-0½"	25.5	2962	
8	Ls.	6 × 4 × ⅜	21'-8½"	12.30	2148	
8	"	6 × 4 × ⅜	7'-4"	20.00	1173	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

(8)

Number	Description	Dimension		Weight (#)		Remarks
		Size (inch)	Length	Unit	Tops	
8	Ls	6×4×3/8	2'-7½"	12.3	256	Splice Ls.
8	p/s.	5½×¼	1'-3½"	4.675	48	Fillers
8	〃	3¼×¼	〃	2.763	29	〃
4	〃	15×½	6'-11½"	25.5	710	Splice p/s.
8	Ls.	6×4×5/16	3'-5½"	20.00	553	〃 Ls.
2	p/s.	11¼×3/8	〃	14.34	99	〃 p/s.
2	〃	〃	2'-7½"	〃	75	〃 〃
14	〃	〃	0'-9"	〃	151	Tie p/s.
4	Ls.	4×3×5/16	3'-0"	7.2	96	Connection Ls
4	〃	〃	2'-8"	〃	67	〃 〃
4	p/s.	37×½	4'-4"	62.9	1090	Gusset p/s
4	〃	39×½	4'-0"	66.3	995	〃
2	〃	35×5/16	2'-11"	38.19	217	〃
2	〃	32×5/16	2'-8"	34.00	181	〃
					10.840	

JL2 ~ L4 ~ JL2

2	p/s.	15×½	32'-7"	25.5	1660	
4	Ls	6×4×5/8	〃	20.0	2607	
2	〃	4×3×5/16	2'-8"	7.2	38	Connection Ls.
9	p/s.	11¼×3/8	0'-9"	14.34	97	Tie p/s.
2	〃	38×½	3'-7"	64.60	452	Gusset p/s
1	〃	32×5/16	2'-8"	34.00	91	〃 〃
					4.945	

Hip Vertical U1 ~ L1

8	Ls.	5×3½×5/16	29'-1½"	8.7	2027	
2	p/s.	12×5/16	3'-6"	12.75	89	Tie p/s.
2	〃	〃	1'-4"	〃	34	〃
2	〃	3½×5/16	3'-6"	3.72	26	Fillers
2	〃	〃	3'-3"	〃	24	〃
2	〃	〃	1'-4"	〃	10	〃
2	〃	〃	1'-0"	〃	7	〃
74	〃	2½×5/16	1'-3½"	2.66	254	Lacing bars
					2.471	

Vertical L2 ~ U2

4	Ls.	15@33.9#	31'-10"	33.9	4317	
8	Ls.	4×3×5/16	3'-4"	7.2	192	Diaphragm
8	〃	〃	1'-6"	〃	86	〃
2	p/s.	12×5/16	3'-4"	12.75	85	〃
1	〃	〃	1'-6"	〃	38	〃
8	〃	〃	1'-3"	〃	128	Tie p/s.
4	〃	〃	1'-6"	〃	77	〃
176	〃	2½×5/16	1'-2"	2.66	546	Lacing Bars
					5469	

Vertical L3 ~ U3

4	Ls.	12@25#	31'-10"		3183	
8	Ls.	4×3×5/16	3'-0"	7.2	173	Diaphragm
8	〃	〃	1'-6"	〃	96	〃

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Number	Description	Dimension		Weight (*)		Remarks
		Size (inch)	Length	Unit	Total	
2	pls.	12 × 5/16	3'-0"	12.75	77	Diaphragm
2	"	"	1'-6"	"	38	"
4	"	"	1'-0"	"	51	Tie pls.
8	"	"	1'-3"	"	128	"
198	"	2½ × 5/16	1'-2"	2.66	583	Lacing bars
					4319	
		Vertical L_4~U_4				
2	Ls.	M@25#	31'-10"		1592	
4	Ls.	4 × 3 × 5/16	3'-0"	7.2	86	Diaphragm
4	"	"	1'-6"	"	43	"
1	pls.	12 × 5/16	3'-0"	12.75	38	"
1	"	"	1'-6"	"	19	"
2	"	"	1'-0"	"	26	Tie pls.
4	"	"	1'-3"	"	64	"
94	"	2½ × 5/16	1'-2"	2.66	297	Lacing bars
					2160	
		Diagonal U_1~L_2				
4	Ls.	6 × 4 × 9/16	35'-0"	18.1	2534	
4	"	"	35'-6"	"	2570	
2	pls.	12½ × 5/16	34'-10"	13.28	925	
					6029	
		Diagonal U_2~L_3				
8	Ls.	6 × 4 × 3/8	34'-10"	12.3	3428	
2	pls.	12½ × 5/16	34'-6"	13.28	916	
					4344	
		Diagonal U_3~L_4				
4	Ls.	12@25#	35'-4"		3533	
8	pls.	12 × 5/16	1'-3"	12.75	128	Tie pls.
252	"	2½ × 5/16	1'-2"	2.66	782	Lacing bars
					4443	

Total sum. 91065# = 40.654 ton

Rivet Head Variation 3½% = 1.423 ton

Total Weight of Highway Truss 42.077 ton

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Number	Description	Dimension		Weight (#)		Remarks
		Size (inch)	Length	Unit	Total	
		Top Lateral				
48	Ls	5 x 3½ x 5/16	18'-3"	8.7	7621	
4	"	3 x 3 x 5/16	21'-3"	6.1	519	Strut
4	"	"	20'-5"	"	498	"
16	"	"	19'-10¾"	"	1939	"
4	pls.	9½ x 5/16	20'-5½"	9.03	739	"
2	"	"	20'-2½"	"	365	"
4	"	26 x 5/16	3'-0"	27.63	332	Gusset pls
2	"	29 x 5/16	3'-6"	30.81	216	"
2	"	21 x 5/16	1'-9"	22.31	78	Bent pls.
5	"	12½ x 5/16	1'-8"	13.28	111	Gusset pls.
4	Ls	4 x 3 x 5/16	0'-11½"	7.2	28	Connection Ls.
20	"	"	0'-8½"	"	102	"
2	pls.	9½ x 5/16	0'-9"	9.03	14	Fillers
10	"	"	0'-6"	"	45	"
					12607	
		Rivet Heads Variations 3½ %			443	
					13050#	
		Portal Bracing				
8	Ls.	4 x 3 x 5/16	32'-6½"	7.2	1874	
2	pls.	17½ x 5/16	1'-8"	18.59	62	Tie pls.
4	"	"	1'-5"	"	105	"
4	"	"	2'-0"	"	149	"
6	"	12 x 5/16	1'-6"	12.75	115	Gusset pls.
8	"	18 x 5/16	2'-0"	19.13	306	"
8	"	"	2'-4"	"	357	"
12	"	17½ x 5/16	1'-3"	18.59	279	Tie pls.
4	"	"	1'-6"	"	112	"
4	"	"	1'-9"	"	130	"
8	Ls.	4 x 3 x 5/16	14'-6"	7.2	835	
8	"	"	14'-1½"	"	814	
8	"	"	23'-5"	"	1349	
12	"	3 x 3 x 5/16	6'-7½"	6.1	485	
114	pls.	2½ x 5/16	1'-6½"	2.66	468	Lacing Bars
					7440	
		Rivet Heads Variations 3½ %			260	
					7700 #	
		Sway Bracing				
10	Ls.	4 x 3 x 5/16	32'-6½"	7.2	2342	
10	"	"	24'-0"	"	1728	
20	"	"	13'-4"	"	1920	
20	"	3 x 3 x 5/16	6'-4"	6.1	773	
10	"	"	5'-10"	"	356	
20	"	4 x 3 x 5/16	1'-3"	7.2	180	
20	"	"	0'-9"	"	108	
5	pls.	16 x 5/16	1'-10"	17.00	156	Gusset pls.
20	"	11 x 5/16	1'-4"	11.69	312	"
10	"	13 x 5/16	1'-7"	13.81	219	"
10	"	10 x 5/16	1'-3"	10.63	133	"
10	"	12 x 5/16	1'-0"	12.75	128	"

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Number	Description	Dimension		Weight (#)		Remarks
		Size (inch)	Length	Unit	Total	
300	P/s.	22½ × 5/16		1669	130	Ring fillers ◎

8455
Rivet Heads Variations 3½ % 297
8782

Summary for Top Lateral, Sway bracing, & Portal.

Top Lateral 13050
Portal Bracing 7700
Sway Bracing 8782
29532 = 13.184 ton

Floor Beam (FB, & FB₂)

1	p/s.	48 × 3/8	33'-4"	61.20	2040	Web p/s.
4	L's.	6 × 6 × 9/16	33'-4"	21.90	2920	Flange L's.
2	p/s.	12½ × ½	19'-1"	21.25	811	Cover p/s.
4	L's.	5 × 3½ × 3/8	3'-11⅜"	10.40	164	Int. stiffeners
4	p/s	3½ × 9/16	3'-0½"	6.69	91	Fillers
4	"	"	1'-4¼"	"	36	"
8	L's.	4 × 3 × 5/16	4'-0½"	7.7	233	Int. stiffs.
8	"	3½ × 3½ × 5/16	0'-9"	"	43	
2	"	6 × 4 × ½	3'-11⅜"	16.20	128	End stiffs. (R)
2	p/s.	6 × 9/16	3'-0¼"	11.48	69	Fillers
2	L's.	4 × 4 × ½	3'-11⅜"	12.8	101	End stiffs (H)
2	p/s.	4 × 9/16	3'-0¼"	7.65	46	Fillers

6672
Rivet Heads Variation 3½ % 234
6906
7- Required 7 @ 6906 = 48342

End Floor Beam

1	pl.	24 × 5/16	13'-7 5/16"	25.5	347	Web (R)
1	"	30 × 5/16	19'-6 5/16"	31.98	622	" (H)
2	L's.	3½ × 3½ × 5/16	13'-10 5/16"	7.7	200	Flanges
2	"	"	19'-8 13/16"	"	294	"
2	"	"	33'-1 5/8"	"	477	"
2	p/s.	7 × 5/16	1'-4 3/4"	7.44	21	Web splice.
8	L's.	3 × 3 × 3/8	1'-11⅜"	7.7	112	Stiffs. (Ry)
4	p/s	6 × 5/16	1'-4 3/4"	6.38	36	Fillers
16	L's.	3 × 3 × 5/16	2'-5⅜"	6.1	239	Stiffs. (Hy)
8	p/s.	6 × 5/16	1'-10¾"	6.38	97	Fillers
4	L's.	4 × 3 × 3/8	1'-11⅜"	9.5	66	End stiffs.
4	p/s.	3 × 5/16	1'-4¾"	3.19	19	Fillers
4	"	12½ × ½	5'-1"	21.25	92	Base pl. for stringer
2	"	14 × ½	2'-0"	23.80	95	(1'-4") cut " (Ry)
4	"	7 × ½	1'-0"	11.90	49	Sole p/s.
4	"	13½ × ½	1'-2"	22.53	105	Base p/s.
4	"	3 × ⅜	1'-2"	3.33	36	"
8	L's	3 × 3 × 5/16	1'-2"	6.1	57	"

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Number	Description	Dimension Size (inch)	Length ft	Weight (#) Unit	Total	Remarks
1	PL	10 x ⅜	1-2"	30.63	12	
					2203	
		Rivets Heads Variations		3½ %	100	
					3264#	
		2 Required			6528#	
			Summary			
					23300	
					6123	
					54470= 28.317 ton.	
		Stringer				
		Highway Stringer				
24	I S	1240 50#	20'-5½"		26953	HSt
48	L3	6 x 3½ x ⅜	0'-10½"	11.7	959	Connection
8	I S	1240 50#	21'-11½"		9609	HSt
16	L3	6 x 3½ x ⅜	0'-10½"	11.7	760	Connection
32	" "	3 x 3 x 5/16	1'-0¾"	6.1	272	Stiffs.
					37955#	
		Railway Stringer (RS 1 & 2)				
4	I S	24 Ø 50#	21'-11½"		7027	
4	L3	5 x 3½ x ⅜	1'-7½"	10.4	69	Connection
16	"	4 x 3 x ⅜	1'-11¼"	8.5	264	Stiffs.
8	PS.	11 x 5/16	2'-0"	11.69	187	Lateral connection
4	"	10 x 5/16	1'-6"	10.63	64	"
10	L3	3 x 3 x 5/16	0'-3½"	6.1	262	Lateral
4	"	3½ x 3½ x ⅜	3'-10"	9.5	130	Struf
2	PS.	15½ x 5/16	3'-10"	16.47	128	"
					8130#	
		RS3				
12	I S	24 Ø 50#	20'-5½"		19640	
24	L3	5 x 3½ x ⅜	1'-7½"	10.4	400	Connection
24	PS.	11 x 5/16	2'-0"	11.69	561	
12	"	10 x 5/16	1'-6"	10.63	191	
30	L3	3 x 3 x 5/16	0'-3½"	6.1	785	Laterals.
					21583#	
			Summary			
					37955#	
					8130	
					21583	
		Rivet Heads Variations		2%	1350	
					69921# = 30.708 ton	
		Bottom Lateral Bracing				
12	L S	5 x 3½ x ⅜	18'-4½"	10.4	2249	diagonals
4	"	"	19'-1½"	"	754	"
24	"	5 x 3½ x 5/16	18'-4½"	9.7	3837	"
24	"	"	19'-1½"	"	3785	"
					10669	
		Rivet Heads Variation		3½ %	370	
					11039#	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

(13)

Number	Description	Dimension		Weight (#)		Remarks
		Size (inch.)	Length	Unit	Total	
		LS_1				
2	Ls.	$3\times3\times\frac{5}{16}$	20'-5 3/4"	6.1	250	Strut LS_1
2	"	"	20'-2 3/4"	"	247	
1	pl.	$9\times\frac{5}{16}$	20'-5 3/4"	9.56	196	
2	Ls.	$4\times3\times\frac{5}{16}$	0'-8 7/8"	7.2	10	End Connection
2	pls.	$3\times\frac{5}{16}$	0'-3 1/4"	3.19	2	Fillers
1	"	$3\frac{1}{2}\times\frac{9}{16}$	0'-8 5/16"	6.69	5	"
2	Ls.	$4\times3\times\frac{5}{16}$	0'-9 3/4"	7.2	11	Connection
2	pls.	$3\times\frac{5}{16}$	0'-6"	3.19	3	Fillers
1	"	$5\frac{7}{8}\times\frac{5}{16}$	0'-8 5/16"	6.25	5	"
1	"	$33\times\frac{3}{8}$	4'-0"	42.08	168	Diagonal connection
					897	
				Rivet Heads Variation 3½ %	32	
					929#	
				2-Required 2@929 =	1858#	
		LS_2				
4	Ls.	$3\times3\times\frac{5}{16}$	20'-5½"	6.1	499	Flange
1	pl.	$9\times\frac{5}{16}$	"	9.56	196	Web
4	Ls.	$4\times3\times\frac{5}{16}$	0'-8 7/8"	7.2	21	End Connection
4	pls.	$3\times\frac{5}{16}$	0'-3 1/4"	3.19	3	Fillers
2	"	$3\frac{1}{2}\times\frac{9}{16}$	0'-8 5/16"	6.69	9	"
1	pl.	$28\times\frac{5}{16}$	3'-4"	29.75	99	
					827	
				Rivet Heads Variation 3½ %	29	
					856#	
				2-Required 2@856 =	1712#	
		LS_3				
		main Section same as LS_2			728	
1	pl.	$24\times\frac{5}{16}$	3'-0"	25.50	77	
					805#	
				Rivet Heads Variation 3½ %	29	
					834#	
				4-Required 4@834 =	3336#	
				Diagonal Bracing	11039	
				Struts LS_1	1858	
				" LS_2	1712	
				" LS_3	3336	
				Summary	17945# = 8.011	ton
		Shoes.				
2	cast steel				1772	Std end
2	"				1718	"
4	"				528	"
2	"				3840	Fixed end
10	steel	6" dia.			2375	Rollers
4	"	$3\times\frac{1}{2}$	2'-3"	5.1	46	Roller Spacer
4	"	$2\frac{3}{4}\times\frac{1}{2}$	0'-9"	4.68	14	Tooth bar
4	"	7" dia.	2'-1 7/8"		1215	pin.

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Number	Description	Dimension		Weight (#)		Remarks.
		Size (inch)	Length	Unit	Total.	
32	steels	1½" dia	1'-10½"	4.172	292	(2'-2½") anchor bolts.
				cast	7858	
				steel	3942	
					11800# = 5.268 Ton	
		Thrust Frame				
1	Ls	3½ x 3½ x ⅜	5'-9½"	8.50	50	
1	"	"	3'-4½"	"	29	
1	"	"	5'-0"	"	43	
2	"	"	0'-1½"	"	70	
2	pls.	14 x ½	2'-1½"	23.80	101	
1	"	15 x ½	1'-4"	25.50	34	
1	"	12 x ½	1'-9"	20.40	36	
1	L	3½ x 3½ x ⅜	0'-9"	8.50	7	
1	pl.	3"φ x ½			1	Ring Filler.
					371	
				Rivet Heads Variation 3½ %	10	
					381# Ton	
			2-Required	2@381# =	762# = 0.340 Ton	

	Railway	Highway	Total weight (Ton)
Truss body	48.667	42.077	90.744
Floor Beam	(40%) 9.727	(60%) 14.590	24.317
Stringer	13.530	17.238	30.768
Top Lateral & Sway	(40%) 5.274	(60%) 7.910	13.184
Bottom Lateral	(40%) 3.204	(60%) 4.807	8.011
Thrust Frame	0.340		0.340
Shoe	(50%) 2.634	(50%) 2.634	5.268
Summary	83.376	89.256	172.632
Percentage	48.3 %	51.7 %	

7-Required 7@172.632 Ton ---- 1208.424 Ton.

Plate Girders

For East Approach

Total Weight 7.863 Ton

33-Required 33@7.863 Ton = 259.479 Ton

For West Approach

Total Weight 3.179 Ton

1-Required 1@3.179 Ton = 3.179 Ton

For East Approach 259.479 Ton

For West Approach 3.179 Ton

Total Summary 262.658 Ton

Note; See Sheet No 10. For East Approach

See Sheet No. 3. For West Approach

増田淳氏関係資料
(独立行政法人 土木研究所蔵)

Number	Description	Dimension		Weight (#)		Remarks
		Size (inch)	Length	Unit	Total	
		Curb Shoes				
		Piers Nos. 1 & 8				
4	pls.	20 × ¼	10'-7⅛"	17.00	720	Out Side pls.
4	Ls.	4 × 4 × ½	10'-7⅛"	12.8	542	Curb Ls.
4	pls.	4 × ¼	1'-8"	3.4	23	splice A
16	"	"	1'-9"	3.4	95	" B
16	"	24 × ¼	2'-7¾"	20.4	864	Bottom pls. cut to shape
16	"	6 × ¼	1'-8"	5.1	136	Diaphragm A
16	Ls.	2½ × 2½ × ¼	1'-0"	4.1	66	"
					2446	
			Rivet Heads Variation 3%		74	
					2520# = 1.125 ton	
			4- Required ----		4.500 ton	
		Piers Nos. 2-3-4-5-6-7				
4	pls.	20 × ¼	12'-2"	17.0	827	Out Side pls.
4	Ls.	4 × 4 × ½	12'-2"	12.8	623	Curb Ls.
4	pls.	4 × ¼	1'-8"	3.4	23	Splice A
16	"	"	1'-9"	"	95	" B
16	"	24 × ¼	3'-0"	20.4	979	Bottom pls. cut to shape
16	"	6 × ¼	1'-8"	5.1	136	Diaphragm A
16	Ls.	2½ × 2½ × ¼	1'-0"	4.1	66	"
					2749	
			Rivet Heads Variation 3%		83 Ton	
					2832# = 1.264	
			12- Required ----		15.168 Ton.	
			(Piers Nos 1-8)		4.500 Ton	
			(Piers No 2-3-4-5-6-7)		15.168 Ton	
			Total Weight ----		= 19.668 Ton.	
		Exp. Shoes of Concrete Girder				
		Top Shoes				
1	pl.	17 × ½	1'-3"	28.90	36	Plane from 9/16"
2	"	1¼ × ½	1'-3"	2.13	5	
2	Ls.	4 × 4 × ⅜	1'-1"	9.8	21	
16	Rivet Head	⅞" ϕ		0.1928	3	
					65 #	
			6- Required 6@65 = 390#			
1	pl.	17 × ½	2'-6"	28.90	72	
2	pls.	1¼ × ½	2'-6"	2.13	10	
4	Ls.	4 × 4 × ⅜	1'-1"	9.8	42	
32	Rivet Heads	⅞" ϕ		0.1928	6	
					130	
			30- Required 30@130 = 3900 #			

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Number	Description	Dimension		Weight (#)		Remarks
		Size (inch)	Length	Unit	Total	
		Bottom Shoes				
2	Ls.	4×4×½	1'-1"	12.8	28	
1	Pl.	14⅜×½	1'-3"	24.44	31	plane from 9/16"
4	Rods.	1"ϕ	5'-0"	2.67	60	with 2 Nuts
8	Rivet Heads.	⅞"ϕ		0.1928	2	Shop Rivets.
					121	

66 - Required 66@121 = 7986

Total Weight ----- 122776# = 5.480 Ton.

Expansion Joints

For River span

1	Pl.	9½×⅜	18'-0"	12.11	219	
1	"	9⅜×¼	"	8.298	149	Bent Pl.
2	"	2×⅜	"	2.55	92	Flat Bars.
1	L.	2½×2½×5/16	"	5.00	90	
36	Bolts	½ϕ	0'-5"	0.40	14	
36	"	"	0'-1½"	0.20	7	
72	Rivet heads	"			3	Shop Rivets.
					573	

7 - Required 7@573# = 4011# = 1.791 Ton.

For Slab & Approach Span.

1	Sheetlead	9×½	13'-0"		99	

For Slab of River span

14 - Required 14@99# = 1396#

For Approach Concrete Span

11 - Required 11@99# = 1099#

Cast iron Drain Hole.

one drain. 239.69 cub inch. ----- 62.4#

220 - Required

220@62.4# = 13728# = 6.129 Ton.

Cast iron panel in Handrail.

Cast iron panel A

one panel 201.59 cub inch ----- 52.5#

2954 - Required 2954@52.5# = 155085#

Cast iron panel B

one panel 131.90 cub inch ----- 34.3#

29 - Required 29@34.3# = 961#

Summary

Cast iron panel A ----- 155085#

Cast iron panel B ----- 961#

Total Weight 156046# = 69.663 Ton

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Summary of Reinforcements

	Required	Weight (Ton)	Total Weight (Ton)	Remark's
Pier's NOS 1-8	2@	7.815	15.630	
" Special	1@	0.075	0.075	
" NOS 2.3.4	3@	8.674	26.022	
" NOS 5-6-7	3@	8.928	26.784	
Roadway Slab for River span	Total		68.511	
Roadway Slab for River span	7@	5.467	38.269	
	Total		38.269	
Approach Span				
Inside Girder	11@	2.799	30.789	
Outside Girder	22@	2.799	61.578	
Roadway Slab	11@	4.166	45.826	
West Approach span				
Inside Girder	1@	0.320	0.320	
Outside Girder	1@	0.637	0.637	
Cross Beam CB1	1@	0.058	0.058	
Slab	1@	0.788	0.788	
	Total		139.996	
Handrail on Approach Span	11@	0.745	8.195	
Handrail on West Approach span	1@	0.145	0.145	
Handrail on Truss span	7@	0.870	6.090	
	Total		14.430	
Bents NOS 1.2.4.5.7.8.10.11.13.14.16 17.19.20.22.23.25.26.28.29.31.32	22@	1.883	41.426	
Bents NOS 3.6.9.12.15.18.21.24.27.30	10@	1.842	18.420	
	Total		59.846	
Abutment for Highway				
East	1@	0.414	0414	
West	1@	0.132	0.132	
	Total		0.546	
Abutment for Railway				
East	1@	0.117	0.117	
West	1@	0.032	0.032	
	Total		0.149	
Total Summary			Ton. 321.767	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

	Marks	Numbers	size	Length	Total length	Unit W't(#)	Total Weight#	Remarks
	Piers NOS 1.8							
	PP1	2.32=64	⅜"φ	10-0	640-0"	2,044	1,308	
	PP2	2.32-64	"	15-0	960-0"	"	1,962	
	PP3	2x32-64	"	25-0"	1,600-0	"	3,270	
	HH1	2x1 = 2	½φ	19-0	38-0	0.668	25	Hoops
	HH2	2x1 = 2	"	20-0	40-0	"	27	"
	HH3	2x1 =2	"	20 6"	41-0	"	27	"
	HH4	2x1 = 2	"	21 6	43-0"	"	29	"
	HHs	2x1 = 2	"	22-6	45-0	"	30	"
	HH6	2x1 =2	"	23-0"	46-0	"	31	"
	HH7	2x1 =2	"	24-0	48-0"	"	32	"
	HH8	2x1 = 2	"	25-0	50-0"	"	33	"
	HH9	2x1 = 2	"	25-6	[illegible]	"	34	"
	HH10	2x1 = 2	"	24-0"	48-0	"	32	"
	HH11	2x1 =2	"	26-6"	53-0"	"	[illegible]	
	HH12	2x1 =2	"	[illegible]-0"	50-0"	"	33	"
	HH13	2x1 = 2	"	[illegible]	[illegible]-0"	"	37	"
	HH14	2x1 = 2	"	25-6	[illegible]		34	"
	HH15	2x1 =2	"	28-0"	56-0	"	[illegible]	
	PP4	8	1"φ	[illegible]6	276-0	2,670	[illegible]	
	PP5	2.16-32	"	[illegible]	368-0"	"	983	
	PP6	2x7 =14	"	13-0	140-0	"	1,308	
	PP7	7	"	23-0	161-0	"	430	
	PP8	4	"	32-0	128-0	"	[illegible]	
	HH16	6	½"φ	32-0"	192-0"	0.668	[illegible]	
	HH17	2x2 = 4	"	8-0"	32-0"	"	21	
	ST1	13	⅝"φ	14-0"	182-0"	1.043	190	Stirup
	ST2	3	"	17-6	52-6"	"	55	"
	ST3	2	"	18-6"	37-0	"	39	"
	ST4	2	"	19-6"	39-0"	"	41	"
	ST5	2	"	21-0"	42-0"	"	44	"
	ST6	2	"	23-0"	46-0"	"	48	"
	ST7	2	"	25-6"	51-0"	"	53	"
					Summary		11.435#	
WELL	VP1	2x16=32	⅝"φ	14-6"	464-0"	1.043	484	Vert
	VP2	2x16=32	"	14-0"	448-0"	"	467	"
	VP3	2x32=64	"	12-0"	768-0"	"	801	"
	VP4	2x32=64	"	9-0"	576-0"	"	601	"
	HHP1	2x18=36	½"φ	22-6"	810-0"	0.668	541	Hoop's
	HHP2	2x18=36	"	18-0"	648-0"	"	443	"
	HHP3	2x52=104	"	21-6"	2,236-6"	"	1.494	"
	HHP4	2x52=104	"	18-0"	1,872-0"	"	1,250	"
					Summary		6.071#	Ton.
					Total Weight		17.506# = 7.815	
		2 Reqd					= 15.630	
Special		3x3 = 9	1"φ	7-0"	63-0"	2.670	168	U Shape
					Total		168# = 0.075 Ton	
		1 Reqd					= 0.075	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

	Marks	Numbers	size	Length	Total length	Unit wt(#)	Total Weight(#)	Remark's
	Piers Nos 2.3.4.							
	P1	2x32=64	1"ø	10'-0"	640-0"	2.670	1,709	
	P2	2x32=64	"	15'-0"	960-0"	"	2,563	
	P3	2x32=64	"	25'-0"	1,600-0"	"	4,272	
	H1	2x1 = 2	½"ø	22'-6"	45'-0"	0.668	30	Hoop's
	H2	2x1 = 2	"	23'-6"	47'-0"	"	31	"
	H3	2x 1=2	"	24'-6"	49'-0"	"	33	"
	H4	2x 1=2	"	25'-0"	50'-0"	"	33	"
	H5	2x 1=2	"	26'-0"	52'-0"	"	35	"
	H6	2x 1=2	"	27'-0"	54'-0"	"	36	"
	H7	2x 1=2	"	27'-6"	55'-0"	"	37	"
	H8	2x 1=2	"	28'-6"	57'-0"	"	38	"
	H9	2x 1=2	"	29'-0"	58'-0"	"	39	"
	H10	2x1=2	"	28'-0"	56'-0"	"	37	"
	H11	2x1=2	"	30'-0"	60'-0"	"	40	"
	H12	2x1=2	"	28'-6"	57'-0"	"	38	"
	H13	2x1=2	"	31'-0"	62'-0"	"	41	"
	H14	2x1=2	"	29'-6"	59'-0"	"	39	"
	H15	2x1=2	"	32'-0"	64'-0"	"	43	"
	P4	9	1"ø	34'-6"	310'-6"	2,670	829	
	P5	9	"	35'-0"	315'-0"	"	841	
	P6	9	"	35'-0"	315'-0"	"	841	
	H H1	2x3=6	½"ø	30'-0"	180'-0"	0.668	120	
	H H2	2x1=2	"	9'-0"	18'-0"	"	12	
	S T1	1	⅝"ø	15'-8"	15'-8"	1.043	16	
	S T2	2	"	16'-0"	32'-0"	"	33	
	S.T3	4	"	16'-6"	66'-0"	"	69	
	S T4	2	"	17'-0"	34'-0"	"	35	
	S T5	2	"	18'-6"	37'-0"	"	39	
	S T6	2	"	19'-10"	39'-8"	"	41.	
	P7	2x9=18	1"ø	16'-6"	297'-0"	2.670	793.	
					Summary		12,763#	
WELL.	V P1	2x16=32	⅝"ø	14'-6"	464'-0"	1.043	484	Vert
	V P2	2x16=32	"	14'-0"	448'-0"	"	467	"
	V P3	2x32=64	"	12'-0"	768'-0"	"	801	"
	V P4	2x32=64	"	9'-0"	576'-0"	"	601	"
	H P1	2x18=36	½"ø	25'-6"	918'-0"	0.668	613	Hoop's
	H P2	2x18=36	"	21'-0"	756'-0"	"	505	"
	H P3	2x52=104	"	25'-0"	2,600'-0"	"	1,737	"
	H P4	2x52=104	"	21'-0"	2,184'-0"	"	1,459	"
					Summary		6,667#	Ton
					Total Weight		19,430# = 8.674	
								=26.022.
	3 Req'd							
	Piers NOS 5.6-7							
	P1	2x32=64	1"ø	10'-0"	640-0"	2.670	1,709	
	P2	2x32=64	"	15'-0"	960-0"	"	2,563	
	P3	2x32=64	"	25'-0"	1,600-0"	"	4,272	
	H1	2x1 =2	½"ø	22'-6"	45'-0"	0.668	30	Hoop's
	H2	2x1 =2	"	23'-6"	47'-0"	"	31	"
	H3	2x1 =2	"	24'-6"	49'-0"	"	33	"
	H4	2x1 =2	"	25'-0"	50'-0"	"	33	"
	H5	2x1 =2	"	26'-0"	52'-0"	"	35	"
	H6	2x1 =2	"	27'-0"	54'-0"	"	36	"

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

(20)

	Marks	Numbers	Size	Length	Total length	Unit W't (#)	Total Weight (#)	Remarks
	H7	2x1=2	½"ϕ	27'-6"	55'-0"	0.668	37	Hoop's
	H8	2x1=2	"	28'-6"	57'-0"	"	38	"
	H9	2x1=2	"	29'-0"	58'-0"	"	39	"
	H10	2x1=2	"	28'-0"	56'-0"	"	37	"
	H11	2x1=2	"	30'-0"	60'-0"	"	40	"
	H12	2x1=2	"	28'-6"	57'-0"	"	38	"
	H13	2x1=2	"	31'-0"	62'-0"	"	41	"
	H14	2x1=2	"	29'-6"	59'-0"	"	39	"
	H15	2x1=2	"	32'-0"	64'-0"	"	43	"
	P4	9	1"ϕ	34'-6"	310'-6"	2.670	829	
	P5	9	"	35'-0"	315'-0"	"	841	
	P6	9	"	35'-0"	315'-0"	"	841	
	P7	2x9=18	"	16'-6"	297'-0"	"	793	
	ST1	1	⅝"ϕ	15'-8"	15'-8"	1.043	16	
	ST2	2	"	16'-0"	32'-0"	"	33	
	ST3	4	"	16'-6"	66'-0"	"	69	
	ST4	2	"	17'-0"	34'-0"	"	35	
	ST5	2	"	18'-6"	37'-0"	"	39	
	ST6	2	"	19'-10"	39'-8"	"	41	
	HH1	2x3=6	½"ϕ	30'-0"	180'-0"	0.668	120	
	HH2	2x1=2	"	9'-0"	18'-0"	"	12	
					Summary		12,763#	
WELL	VP1	2x16=32	⅝"ϕ	14'-6"	464'-0"	1.043	484	Vert
	VP2	2x16=32	"	14'-0"	448'-0"	"	467	"
	VP3	2x64=128	"	12'-0"	1,536'-0"	"	1,602	"
	HP1	2x18=36	½"ϕ	25'-0"	918'-0"	0.668	613	Hoop's
	HP2	2x18=36	"	21'-0"	756'-0"	"	505	
	HP3	2x58=116	"	25'-0"	2,900'-0"	"	1,937	
	HP4	2x58=116	"	21'-0"	2,436'-0"	"	1,627	
					Summary		7,235#	Ton
					Total Weight		19,998# = 8.928	
		3 Req'd					=26".784	

Roadway Slab for River Span

S1	2x59=118	½"ϕ	20'-8"	2,438'-8"	0.668	1,629	Slab Panel A
S2	2x59=118	"	22'-4"	2,635'-4"	"	1,760	"
S3	2x116=232	"	21'-4"	4,949'-4"	"	3,306	"
LB1	66	"	18'-0"	1,188'-0"	"	794	Slab Panel B
S1	53	"	20'-8"	1,095'-4"	"	732	"
S2	53	"	22'-4"	1,183'-8"	"	791	"
S3	102	"	21'-4"	2,176'-0"	"	1,454	"
LB1	2x44=88	"	18'-0"	1,584'-0"	"	1,058	Panel A
LB2	2x22=44	"	24'-7"	1,081'-8"	"	723	Ton"
				Total Weight		12,247# = 5.467	
	7 Req'd					=38".269	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

	Marks	Numbers	Size	Length	Total length	Unit Wt.(#)	Total Weight(#)	Remarks
Inside Girder. 2-End span								
	B_1	2×2=4	1"φ	43'-2"	172'-8"	2.670	461	
	B_2	2×3=6	〃	43'-0"	258'-0"	〃	689	
	B_3	2×3=6	〃	30'-9"	184'-6"	〃	493	
	B_4	2×2=4	〃	41'-4"	165'-4"	〃	441	
	B_5	2×2=4	〃	32'-2"	128'-8"	〃	344	
	B_6	2×2=4	〃	33'-0"	132'-0"	〃	352	
	B_{11}	2×2=4	〃	13'-0"	52'-0"	〃	139	
	B_{12}	2×3=6	〃	10'-8"	64'-0"	〃	171	
	B_{13}	2×3=6	〃	31'-3"	187'-6"	〃	501	
	B_{14}	2×2=4	〃	24'-11"	99'-8"	〃	266	
	B_{15}	2×3=6	〃	20'-0"	120'-0"	〃	320	
	B_{16}	2×2=4	〃	20'-0"	80'-0"	〃	214	
	B_{17}	15	〃	7'-5"	111'-3"	〃	297	
	B_{18}	2×3=6	〃	9'-4"	56'-0"	〃	150	
	H_1	2×6=12	½"φ	5'-0"	60'-0"	0.668	40	Hoops
	ST_1	2×11=22	〃	8'-10"	194'-4"	〃	130	Near Center
	ST_1	2×6=12	〃	10'-0"	120'-0"	〃	80	Near Column
	ST_2	2×12=24	〃	11'-6"	276'-0"	〃	184	
Center span								
	B_7	2	1"φ	38'-10"	77'-8"	2.670	207	
	B_8	3	〃	42'-6"	127'-6"	〃	340	
	B_9	2	〃	36'-10"	73'-8"	〃	197	
	B_{10}	2	〃	18'-8"	37'-4"	〃	100	
	ST_1	13	½"φ	9'-10"	114'-10"	0.668	77	
	ST_2	10	〃	11'-6"	115'-0"	〃	77	
						Total Weight	6270#= 2.799 ton	
	11 Req'd						=30.789	
Outside Girder								
2-End span.								
	BB_1	2×2=4	1"φ	43'-2"	172'-8"	2.670	461	
	BB_2	2×3=6	〃	43'-0"	258'-0"	〃	689	
	BB_3	2×3=6	〃	30'-9"	184'-6"	〃	493	
	BB_4	2×2=4	〃	41'-4"	165'-4"	〃	441	
	BB_5	2×2=4	〃	32'-2"	128'-8"	〃	344	
	BB_6	2×2=4	〃	33'-0"	132'-0"	〃	352	
	BB_{11}	2×2=4	〃	13'-0"	52'-0"	〃	139	
	BB_{12}	2×3=6	〃	10'-9"	64'-0"	〃	171	
	BB_{13}	2×3=6	〃	31'-3"	187'-6"	〃	501	
	BB_{14}	2×2=4	〃	24'-11"	99'-8"	〃	266	
	BB_{15}	2×3=6	〃	20'-0"	120'-0"	〃	320	
	BB_{16}	2×2=4	〃	20'-0"	80'-0"	〃	214	
	BB_{17}	15	〃	7'-5"	111'-3"	〃	297	
	BB_{18}	2×3=6	〃	9'-4"	56'-0"	〃	150	
	H_2	2×6=12	½"φ	5'-0"	60'-0"	0.668	40	Hoops
	ST_3	2×11=22	〃	8'-6"	187'-0"	〃	125	Near Center
	ST_3	2×6=12	〃	9'-8"	116'-0"	〃	77	Near Column
	ST_4	2×12=24	〃	11'-2"	268'-0"	〃	179	
	BB_{19}	1×4=4	〃	6'-0"	24'-0"	〃	16	
	ST	1×7=7	⅜"φ	4'-0"	28'-0"	0.376	11	Hoops.
1-Center span								
	BB_7	2	1"φ	38'-10"	77'-8"	2.670	207	
	BB_8	3	〃	42'-6"	127'-6"	〃	340	
	BB_9	2	〃	36'-10"	73'-8"	〃	197	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

	Marks	Number	Size	Length	Total Length	Unit Wt(#)	Total Weight(#)	Remarks
	BB_{10}	2	1"φ	19'-8"	37'-4"	2.670	100	
	ST_3	13	½"φ	8'-6"	110'-6"	0.668	74	
	ST_4	10	"	9'-8"	96'-8"	"	65	
						Total Weight --- 6269# = 2.799 Ton		
		22 Req'd					=61.578	
	West Approach Concrete Girder							
Center Girder								
	AS_1	15	⅜"φ	7'-0"	105'-0"	0.376	39	
	AS_2	4	¾"φ	2'-6"	10'-0"	1.502	15	
	AS_3	6	"	4'-6"	27'-0"	"	41	
	AS_4	8	"	2'-0"	16'-0"	"	24	
	A_1	2	1"φ	23'-3"	46'-6"	2.670	124	
	A_2	2	"	25'-0"	50'-0"	"	134	
	A_3	1	"	20'-0"	20'-0"	"	53	
	A_4	2	"	26'-0"	52'-0"	"	139	
	A_5	2	"	24'-0"	48'-0"	"	128	
	AH_1	5	⅜"φ	3'-0"	15'-0"	0.376	6	
	AH_2	7	"	5'-3"	36'-9"	"	14	
							717# = 0.320 Ton	
		1 Req'd					=0.320	
Outside Girder								
	AAS_1	30	⅜"φ	6'-6"	195'-0"	0.376	73	
	AAS_2	8	¾"φ	2'-6"	20'-0"	1.502	30	
	AAS_3	12	"	4'-6"	54'-0"	"	81	
	AAS_4	16	"	2'-0"	32'-0"	"	48	
	AA_1	4	1"φ	23'-3"	93'-0"	2.670	248	
	AA_2	4	"	25'-0"	100'-0"	"	267	
	AA_3	2	"	20'-0"	40'-0"	"	107	
	AA_4	4	"	26'-0"	104'-0"	"	278	
	AA_5	4	"	24'-0"	96'-0"	"	256	
	AAH_1	10	⅜"φ	3'-0"	30'-0"	0.376	11	
	AAH_2	14	"	5'-3"	73'-6"	"	28	
							1427# = 0.637 Ton	
		1 Req'd					=0.637	
Cross Beam CB1								
	AC	4	¾"φ	14'-6"	58'-0"	1.502	87	
	AS_5	10	⅜"φ	7'-9"	77'-6"	0.376	29	
	AS	4	½"φ	5'-0"	20'-0"	0.668	13	
							129# = 0.058 Ton	
		1 Req'd					=0.058	
Slab								
	S_1a	5	½"φ	19'-0"	95'-0"	0.668	63	
	S_2a	5	"	19'-6"	97'-6"	"	65	
	S_4a	8	"	19'-6"	156'-0"	"	104	
	S_1b	3	"	20'-0"	60'-0"	"	40	
	S_2b	3	"	20'-6"	61'-6"	"	41	
	S_4b	6	"	20'-6"	123'-0"	"	82	
	S_1c	1	"	17'-3"	17'-3"	"	12	
	S_2c	1	"	18'-9"	18'-9"	"	13	
	S_4c	3	"	17'-9"	53'-3"	"	36	
	S_1	15	"	20'-8"	310'-0"	"	207	
	S_2	15	"	22'-4"	335'-0"	"	224	
	S_4	30	"	21'-4"	640'-0"	"	428	
	S_{C1}	30	"	4'-0"	120'-0"	"	80	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

	Marks	Numbers	Size	Length	Total Length	Unit Wt(#)	Total Weight(#)	Remarks
	SSc2	4	½"ϕ	3'-0"	12'-0"	0.668	8	
	SL1	20	"	23'-7½"	472'-6"	"	316	
	SL2	5	"	5'-0"	25'-0"	"	17	
	SL3	6	"	5'-0"	30'-0"	"	20	
	SL4	3	"	4'-0"	12'-0"	"	8	Ton
							1.764# = 0.788	
								= 0.788
	1 Reqd							
	Roadway Slab for Approach Span							
	S1	3×44=132	½"ϕ	20'-8"	2,728'-0"	0.668	1,822	
	S2	3,44=132	"	22'-4"	2,948'-0"	"	1,969	
	S4	3.86-258	"	21'-4"	5,504'-0"	"	3,677	
	LBA	6×20-120	"	23'-3"	2,790'-0"	"	1,864	Ton
							9,332# = 4.166	
	11 Reqd							= 45,826
	Handrail on Concrete Span							
	H1	3×12=36	½"ϕ	23'-3"	837'-0"	0.668	559	
	H2	3×8=24	"	23'-3"	558'-0"	"	373	
	H3	3×36=108	"	7'-0"	756'-0"	"	505	U shape
	H4	3×36=108	"	2'-6"	270'-0"	"	180	Hoop's
	H5	3×88=264	¼"ϕ	1'-2"	308'-0"	0.167	51	Ton
							1,668# = 0.745	
	1 Reqd							= 8".195
	Handrail on West Approach span							
	A5	5	½"ϕ	24'-0"	120'-0"	0.668	80	
	H3	10	"	7'-0"	70'-0"	"	47	U shape
	H4	10	"	2'-6"	25'-0"	"	17	Hoop's
	H5	23	¼"ϕ	1'-2"	26'-10"	0.167	4	"
	SL1	5	½"ϕ	23'-7½"	118'-1½"	0.668	79	
	SL2	5	"	5'-0"	25'-0"	"	17	
	H3	12	"	7'-0"	84'-0"	"	56	U shape
	H4	12	"	2'-6"	30'-0"	"	20	Hoop's
	H5	28	¼"ϕ	1'-2"	32'-8"	0.167	5	Ton
							325# = 0.145	
	1 Reqd							= 0.145
	Handrail on Truss Span							
	H1	70	½"ϕ	18'-0"	1,260'-0"	0.668	842	
	H2	20	"	24'-7"	491'-8"	"	328	
	H3	124	"	7'-0"	868'-0"	"	580	Past U Shapes
	H4	16	"	7'-0"	112'-0"	"	75	Exp. Past "
	H5	334	¼"ϕ	1'-2"	389'-8"	0.167	65	Hoop's
	H6	124	"	2'-6"	310'-0"	"	52	"
	H7	24	"	1'-6"	36'-0"	"	6	Ton
							1.948 = 0.870	
	7 Reqd							= 6.090

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

	Marks	Numbers	Size	Length	Total length	Unit Wt (#)	Total Weight (#)	Remarks
Bents NOS 1.2.4.5.7.8.10.11.13.14.16.17.19.20.22.23.25.26.28.29.31.32								
	C_4	2x12=24	1"ø	27'-0"	648'-0"	2.670	1730	
	C_3	2x6=12	"	21'-9"	261'-0"	"	697	
	C_4	1x10=10	"	27'-0"	270'-0"	"	721	
	C_5	1x4=4	"	21'-9"	87'-0"	"	232	
	H_1	2x21=42	3/8"ø	10'-0"	420'-0"	0.376	158	
	H_2	1x21=21	"	9'-0"	189'-0"	"	71	
	H_3	2x4=8	"	8'-0"	64'-0"	"	24	
	H_4	2x4=8	"	8'-0"	64'-0"	"	24	
	H_5	1x4=4	"	8'-0"	32'-0"	"	12	
	H_6	1x4=4	"	7'-0"	28'-0"	"	11	
		2x7=14	1/2"ø	15'-0"	210'-0"	0.668	140	Web Hor
		2x4=8	3/8"ø	11'-6"	92'-0"	1.043	96	" Ver
		2x2=4	"	9'-0"	36'-0"	"	38	
	ST	2x2=4	1/2"ø	10'-0"	40'-0"	0.668	27	Stirup
	"	2x2=4	"	9'-0"	36'-0"	"	24	"
		2x6=12	"	15'-0"	180'-0"	"	120	Wall B
		2x8=16	"	4'-6"	72'-0"	"	48	"
		2x4=8	"	5'-0"	40'-0"	"	27	
		2	1/2"ø	14'-6"	29'-0"	"	19	Ton
							4219#	1.883
22 Reqd								=41.426
Bents NOS 3.6.9.12.15.18.21.24.27.30								
	C_1	2x4=8	1"ø	21'-9"	174'-0"	2.670	465	
	C_2	2x10=20	"	22'-9"	455'-0"	"	1215	
	C_2	1x12=12	"	22'-9"	273'-0"	"	729	
	C_3	2x8=16	"	13'-6"	216'-0"	"	577	
	C_3	1x6=6	"	13'-6"	81'-0"	"	216	
	H_1	2x21=42	3/8"ø	10'-0"	420'-0"	0.376	158	"
	H_2	1x21=21	"	9'-0"	189'-0"	"	71	
		2x7=14	1/2"ø	15'-0"	210'-0"	0.668	140	Web Hor
		2x4=8	3/8"ø	11'-6"	92'-0"	1.043	96	" Ver
		2x2=4	"	9'-0"	36'-0"	"	38	"
	ST	2x2=4	1/2"ø	10'-0"	40'-0"	0.668	27	Stirups
	"	2x2=4	"	9'-0"	36'-0"	"	24	"
		2x8=16	"	4'-6"	72'-0"	"	48	Wall A Ver
		2x6=12	"	15'-0"	180'-0"	"	120	" Hor
		2	"	14'-6"	29'-0"	"	19	
		2x2=4	3/4"ø	14'-6	58'-0"	1.502	87	Beam A Hor
		4	1/2"ø	14'-6	58'-0"	0.668	39	" "
	ST	10	1/2"ø	7'-9"	77'-6	0.376	29	" Stirups
		2x4=8	1/2"ø	5'-0"	40'-0	0.668	27	Ton
							4125#	1.842
10 Reqd								=18.420
East Abutment for Highway								
	E_1	2x6=12	1/2"ø	5'-3"	63'-0"	0.668	42	
	E_2	2x3=6	"	11'-0"	66'-0"	"	44	
	E_3	20	"	13'-0"	260'-0"	"	174	
	E_4	1x5=5	"	7'-6"	37'-6"	"	25	
	E_5	2x15=30	"	6'-9"	202'-6"	"	135	
	E_6	1x5=5	"	9'-6"	47'-6"	"	32	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

	Marks	Numbers	Size	Length	Total length	Unit Wt.(#)	Total Weight(#)	Remark's
	E_7	1×6=6	½"ϕ	9'-6"	57'-0"	0.668	38	
	E_8	1×14=14	⅝"ϕ	9'-6"	133'-0"	1.043	139	
	E_9	2×3=6	½"ϕ	12'-3"	73'-6"	0.668	49	
	E_{10}	1×2=2	"	7'-6"	15'-0"	"	10	
	E_{11}	1×2=2	"	13'-0"	26'-0"	"	17	
	E_{12}	1	"	3'-3"	3'-3"	"	2	
	E_{13}	1	"	3'-0"	3'-0"	"	2	
	E_{14}	1	"	2'-6"	2'-6"	"	2	
	E_{15}	1	"	2'-0"	2'-0"	"	1	
		3×3=9	1"ϕ	9'-0"	81'-0"	2.670	216	U Shape
							928# = 0.Ton 414	
		1 Req'd					= 0."414	
	West Abutment for Highway							
	W_1	26	½"ϕ	5'-0"	130'-0"	0.668	87	
	W_2	6	"	18'-0"	108'-0"	"	72	
	W_3	6	"	7'-6"	45'-0"	"	30	
	W_4	30	"	5'-3"	157'-6"	"	105	
	W_5	6	"	11'-0"	66'-0"	"	44	
	W_6	2	"	2'-3"	4'-6"	"	3	
							341# = 0.Ton 152	
		1 Req'd					= 0."152	
	East Abutment for Railway							
	EE_1	5	½"ϕ	9'-6"	47'-6"	0.668	32	
	EE_2	9	"	6'-6"	58'-6"	"	39	
	EE_4	6	"	7'-3"	43'-6"	"	29	
	EE_5	2	"	7'-0"	14'-0"	"	9	
	EE_6	2	"	6'-6"	13'-0"	"	9	
	EE_7	2	"	7'-0"	14'-0"	"	9	
	EE_8	6	"	6'-6"	39'-0"	"	26	
	EE_9	6	"	3'-0"	18'-0"	"	12	
	EE_{10}	2	"	16'-0"	32'-0"	"	21	
	EE_{11}	2	"	10'-6"	21'-0"	"	14	
	EE_{12}	2	"	8'-0"	16'-0"	"	11	
	EE_{13}	2	"	5'-6"	11'-0"	"	7	
	EE_3	2	¾"ϕ	15'-0"	30'-0"	1.502	45	Ton.
							263# = 0.117	
		1 Req'd					= 0."117	
	West Abutment for Railway							
	WW_1	4	½"ϕ	9'-6"	38'-0"	0.668	25	
	WW_2	11	"	5'-0"	55'-0"	"	37	
	$W.W_3$	5	"	3'-0"	15'-0"	"	10	Ton.
							72# = 0.032	
		1 Req'd					= 0.032	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Summary of Concrete Volume

	Number	Volume 立坪	Total Volume 立坪	Mixtur
Piers				
Nos. 1-8 Strut and Pedestal	2@	3.92	7.84	1:2:4
Nos. 2-3-4-5-6-7 "	6@	5.04	30.24	"
Total			37.88	"
NOS. 1-8 Coping & Shaft	4@	4.22	16.88	"
NOS. 2-3-4-5-6-7 "	12@	6.10	73.20	"
Total			90.08	
NOS. 1-8 Filling	4@	3.27	13.08	1:2:4
Nos. 2-3-4-5-6-7 Filling	12@	4.71	56.52	"
Total			69.60	
NOS 1-8 Filling	4@	8.29	33.16	Rubble Concrete
NOS 2-3-4 "	6@	11.94	71.64	"
NOS 5-6-7 "	6@	13.51	81.06	"
Total			185.86	"
NOS 1-8 Well	4@	7.83	31.32	1:2:4
NOS 2-3-4 "	6@	9.28	55.68	"
NOS 5-6-7 "	6@	10.06	60.36	"
Total			147.36	
Slab at River Span	7@	9.28	64.96	1:2:4
Total			64.96	
Slab at Approach Span	11@	16.37	180.07	1:2:4
West Approach Span (Center Girder)	1@	0.36	0.36	"
" " " (Outside Girder)	2@	0.32	0.64	"
" Slab	1@	1.38	1.34	"
" Cross Beam CB.	1@	0.07	0.07	"
Total			182.52	"
Hand rail on Truss Span	7@	0.78	5.46	1:2:4
Handrail on Approach Span	11@	0.63	6.94	"
Handrail on West "	1@	0.13	0.13	"
Pedestal	2@	0.07	0.14	"
Pedestal Support for East Abutment	1@	0.10	0.10	"
Total			12.76	
Bents Nos. 1-2-4-5-7-8-10-11-13-14-16-17-19 20-22-23-25-26-28-29-31-32	22@	5.90	129.80	1:2:4
" Nos. 3-6-9-12-15-18-21-24-27-30	10@	5.95	59.50	"
Total			188.30	"
Bents Basement Nos. 1-2-4-5-7-8-10-11-13-14-16-17-19-20 22-23-25-26-28-29-31-32	22@	5.39	118.58	1:3:6
" " Nos 3-6-9-12-15-18-21-24-27-30	10@	5.39	53.90	"
Total.			172.48	"
West Abutment for Highway	1@	11.54	11.54	1:2:4
East " " "	1@	10.97	10.97	"
Total			22.51	"
West Abutment Basement for Highway	1@	6.67	6.67	1:3:6
East " " " "	1@	5.83	5.83	"
Total.			12.50	"
Approach Pier for Railway				
Coping	32@	0.21	6.72	1:2:4
Shaft	32@	3.63	116.16	"
Total.			122.88	"
Shaft	32@	4.47	143.04	Rubble Concrete
Total			143.04	"
Basement	32@	1.85	59.20	1:3:6
Total.			59.20	"

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

West Abutment for Railway	Number	Volume	Total Volume	Mixtur
Parapet Wall, Coping & Shaft	1@	4.44 立坪	4.44 立坪	1:2:4
East Abutment for Railway				
Parapet Wall, Coping & Shaft	1@	7.36	7.36	1:2:4
Total			11.80	〃
West Abutment for Railway				
Basements	1@	3.22	3.22	1:3:6
East Abutment for Railway				
Basements	1@	4.70	4.70	1:3:6
Total.			7.92	〃
Total Sum.				
				1:2:4
				1:3:6
				Rubble Concrete
			1531.63	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

End pier

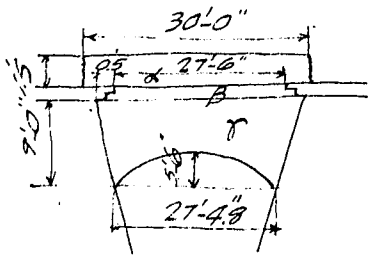
(1) Well — Ex Pa

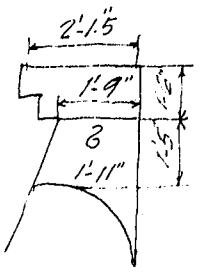
		Area	Volume
Lower Edge			
	$\frac{\pi}{4} \times 1.75 \times 1.75$	= 1.53125	
Circumference	$\pi \times 12.33 = 38.74$		59.32 cub ft
	$\frac{1}{2} \times (1.75 + 1.5) \times 3.92$	= 6.370	
Circumference	$\pi \times 11.625 = 36.52$		232.63
Summary			291.95
Upper Part			
	$(31.31 - 6') \times 1.5$	= 38.72	
Circumference	$\pi \times 11.5 = 36.13$		1398.95
Total			1690.90 (1:2:4)
			= 7.83 立坪
Filling			
	$\pi \times \overline{10.0}^2 \div 4$	= 78.54	
hight	$3' + 6' = 9'$		706.86 (1:2:4)
			= 3.27 立坪
Hight	$31.31 - 9' = 22.31$		1791.50 (Rubble Concrete)
			= 8.29 立坪

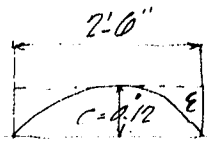
(2) Coping & Shaft :—

Shaft			
Top area	$\pi \times \overline{6.0}^2 \div 4$	= 28.27	
Bottom area	$\pi \times \overline{8.5}^2 \div 4$	= 56.75	
Mean area		85.02 ÷ 2 = 42.51	
Height	20.178		857.77
Coping	$\pi \times \overline{7.0}^2 \div 4$	= 38.48	
	$\pi \times \overline{6.5}^2 \div 4$	= 33.18	
		71.66	
Height	0.75		53.75
			911.52 (1:2:4)
			= 4.22 立坪

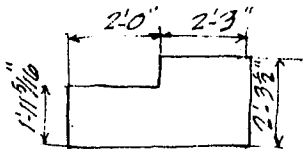
(3) Strut & Pedestal







	α : 2.25×2.29	= 5.15	
	2.0×1.925	= 3.89	
		9.04	
	30.0		271.20 (1:2:4)
	β : 2.5×1.5	=	
	27.75		
	γ : $27.95 \times 9'-0"$	= 251.55	
	Circular Segment	= 103.62	
		147.93	
	2.5'		369.83 (1:2:4)
	δ : 1.835×1.417	= 2.60	
	$1.913 \times 1.913 \times (1 - \frac{\pi}{4})$	= 0.79	
	2.125×1.5	= 3.19	
		6.58	
	27.5		190.45 (1:2:4)
	ε : $2.5 \times 0.12 \times 0.3321 \times 2$	= 0.20	
	$9 + 1.5 = 10.5$		2.10 (1:2:4)
Total			322.03
			= 3.82 立坪



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Intermediate Pier

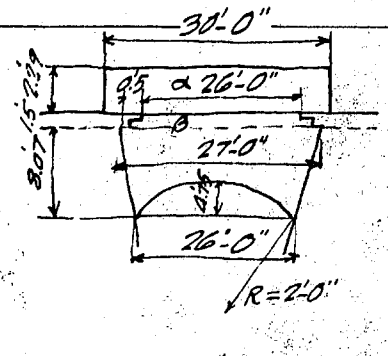
(1) Well:— P2-3-4-5-6-7

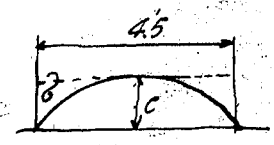
			Area (□')	Volume (cub ft)
Lower Edge :—				
	$\frac{1}{2} \times 1.75 \times 1.75$	=	1.53125	
Circumference	$\pi \times 14'33 = 45'02$			68.94
	$\frac{1}{2} \times (1.75+1.5) \times 3.92$	=	6.370	272.64
Circumference	$\pi \times 13'625 = 42'80$			341.58
Upper part :—	P5-6-7			
	$(34'80-6') \times 1.5$	=	43.2	
Circumference	$\pi \times 13'5 = 42.41$			1832.11
Total				2173.69 (1:2:4)
				= 10.06 立坪
Filling :—	P2-3-4-5-6-7			
	$\pi \times \overline{12.0}^2 \div 4$	=	113.10	
Height	$3'+6' = 9'$			1017.90 (1:2:4)
				= 4.71 立碼
	P5-6-7			
Height	$34'8 - 9' = 25'8$		113.10	2917.98 (Rubble Concrete)
				= 13.51 立碼
	P2-3-4			
Upper part of well :—	$(31'81-6) \times 1.5$	=	38.72	
Circumference	42'41			1642.11
Lower part				341.58
Total				1983.69 (1:2:4)
				= 9.28 立碼
Filling	P2-3-4			
height	$31'81 - 9' = 22'81$		113.10	2579.81 (Rubble Concrete)
				= 11.94 立碼

(2) Coping & Shaft:—

Shaft :—				
Top area	$\pi \times \overline{7.5}^2 \div 4$	=	44.18	
bottom area	$\pi \times \overline{10.0}^2 \div 4$	=	78.54	
Mean area		=	122.72 ÷ 2 = 61'36 □'	
Height	20'178			1238.12
Coping :—				
	$\pi \times \overline{8.5}^2 \div 4$	=	56.75	
	$\pi \times \overline{8.0}^2 \div 4$	=	50.27	
			107.02	
Height	0.75			80.26
				1318.38 (1:2:4)
				= 6.10 立坪

(3) Strut & Pedestal





for d = 8'25 c = 0.7
d = 9'0 C = 0.7

α :	$4'5 \times 2'29$	=	10'31 □'	
	30'0			309.30 cub ft
β :	$4'5 \times 1'5$	=	6'75 □'	
	26'25			177.19 cub ft
γ :	$26'5 \times 8'07$	=	213'86 □'	
	Circular segment	=	84.50 □'	
			129.36	
	4'5			582.12 cub ft.
δ :	$4'5 \times 0'7 \times 0'3206 \times 2$	=	2'0 □'	
	$8'07+1'5 = 9'57$			19.14 cub ft.
Total				1087.75 (1:2:4)
				= 5.04 五坪

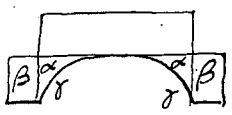
増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Slab at River Span

		Area (□')	Volume (cub ft)
2 lines of Curb			
	12"×5" = 60□"		
	10"×4" = 40□"		
	7"×6" = 42□"		
	142□"×2 = 284□"		
2 Overhanging arms			
	9.5×18"×2 = 306□"		
Slab.			
	6"×180" = 1080□"		
Fillet			
	(15+6")×2 = 48□"		
	1718□" ÷ 144 = 11.93	□'	
Length 168'			2004.24 cub ft (1:2:4)
			= 9.28 立坪

Concrete Girder & Slab

		Area (□')	Volume (Cub. ft)
2 lines of Curb			
	12"×5" = 60□"		
	10"×4" = 40□"		
	7"×6" = 42□"		
	142×2 = 284□"		
2 Overhanging arms			
	9.5×18"×2 = 306□"		
Slab			
	6"×180" = 1.080□"		
Fillet			
	3"×6"×3 = 54□"		
	1724□" ÷ 144 =	11.98 □'	
Summary Length	44.5×3 = 133.5		1599.33 cub ft (1:2:4)
Beam			
2- Out Beam	18"×33 3/4"×2 = 1215□"		
1- Inside Beam	18"×36" = 648□"		
	1,863□"		
3- Beam 2α	3×504"×12"×18"×3 = 108864□"		
2β	30"×18"×22 15/16"×3 = 37.159		
2γ	(91□" - 64□")×15"×2×3 = 1.836		
	147859 ÷ 534"		
	= 227□"	□'	
Summary	2090 ÷ 144 =	14.51	
Length	44.5×3 = 133.5		1937.08 cub ft (1:2:4)
			3536.41 (1:2:4)
		Summary	= 16.37 立坪



増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Center Girder (West Approach Span) No 1

	2.0 × 1.5	= 3.00	---- area (□')	Volume (cub ft)
2'6 / 1'2	Length 23.67			71.01 cub ft (1:2:4)
	2 × (3.40 - .21)	= 4.38		
	thickness 1.5			6.57 (1:2:4)
				77.58 cub ft
				= 0.36 立碼 (1:2:4)

Outside Girder

	1.4' × 1.5	= 2.72		
2'6 / 1'2	Length 23.67			64.38 cub ft (1:2:4)
	2 × (2.62 - .095)	= 3.59		
	thickness 1.5			5.37 cub ft (1:2:4)
				69.75 cub ft
				= 2.32 立碼 (1:2:4)

Slab

2 lines of Curb

12" × 5' = 60"
10" × 4" = 40"
7" × 6' = 42"
142 × 2 = 284"

2 Overhanging arms

9.5 × 18" × 2 = 306"

Slab

6" × 180" = 1090"

Fillet

3" × 6" × 3 = 54"

Summary 1724" ÷ 144 = 11.98'

Length	15.25		182.70 cub ft (1:2:4)
Slab			
1 line of Curb			
"	12" × 5" = 60"		
"	10" × 4" = 40"		
"	7" × 6" = 42"		
1 Overhanging arm			
	9.5 × 18" = 153"		
Slab			
	6" × 180" = 1080"		
Fillet			
	3" × 6" × 3 = 54"		
	1429" ÷ 144"	= 9.92'	84.32 cub ft (1:2:3)
Length	9.5		
	24.75' - 14.70'	= 15.05"	10.08 cub ft
thickness	0.67		
	11.0 × 1.0	= 11.0'	5.50 cub ft (1:2:3)
thickness	0.5		
Curb			
	12" × 5" = 60"		
	10" × 4" = 40"		
	7" × 6" = 42"		
1 Overhanging arm			
	9.5 × 18" = 153"		

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

West Approach Span No 2

Area (㎡) Volume

295 ㎝ / 100 = 2.95 ㎥/m

Length 7.30 m

16.11 ㎥ (1:2:4)

Summary

238.7 (cub ft)

= 1.33 立坪 (1:2:4)

Cross Mean CB,

2.175 × 0.71 = 2.29 ㎡

Length 5.0 m

11.45 ㎥ (1:2:4)

2.025 × 0.71 = 0.355 ㎡

Length 4.5 m

1.60 ㎥ (1:2:4)

12.05

= 0.07 立坪 (1:2:4)

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Handrail on Truss span

Top rail		Area	Volume	
	0.5 × 0.25	= 0.125		
	½ × 0.5 × 0.062	= 0.016		
Botom rail	0.5 × 0.36	= 0.180		
		0.321		
Length	305.0		97.91 cub.ft.	(1.2.4)
Post A				
	4 × 0.916 × 0.79	= 0.181	cub.ft	
Thickness	0.063		0.011.	
	0.916 × 0.79	= 0.724		
Thickness	0.94		0.681	
	0.75 × 0.63	= 0.473		
Thickness	1.60		0.757	
		Summary	1.449 cub.ft.	
	34 Required	34@	49.27 cub.ft.	(1.2.4)
Post B				
	0.5 × 0.75	= 0.375		
Thickness	1.87		0.633 cub.ft.	
	28 Required	28@	19.12 cub.ft.	(1.2.4)
Post				
	0.38 × 0.5	= 0.190		
Thickness	1.87		0.346 cub.ft.	
	9 Req'd	9@	7.77 cub.ft.	(1.2.4)
		Total	164.07 cub.ft.	
			= 9.78 ton	(1.2.4)
	7 Required	7@ 0.78	= 5.46 ton	(1.2.4)

Handrail on East Approach Concrete Slab

Top rail	0.5 × 0.25	= 0.125		
	½ × 0.5 × 0.062	= 0.016		
Botom rail	0.5 × 0.36	= 0.180		
		0.321		
Length	7 × 10.28 = 240.6		77.70 cub.ft.	(1.2.4)
Post A	2 × 0.916 × 0.79	= 0.181	cub.ft	
Thickness	0.063		0.011	
	0.916 × 0.79	= 0.724		
Thickness	0.94		0.681	
	0.75 × 0.63	= 0.473		
Thickness	1.60		0.757	
			1.449 cub.ft.	
	18 Required	18@	26.08 cub.ft.	(1.2.4)
Post B	0.5 × 0.75	= 0.375		
Thickness	1.87		0.633 cub.ft.	
	24 Required	24@	16.37 cub.ft.	(1.2.4)
Post D	½ × 0.333 × 0.79	= 0.131		
	½ × 0.222 × 0.79	= 0.171		
		0.333	0.016 cub.ft.	
Thickness	0.063			
	0.79 × 0.79	= 0.624		
Thickness	0.94		0.587	
	0.63 × 0.71	= 0.447		
Thickness	1.60		0.715	
			1.302	
	4 Required	4@	5.21 cub.ft.	(1.2.4)

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

			Area	Volume
Post C				
	½ × 0.75 × 0.79	=	0.296	
	½ × 0.46 × 0.79	=	0.182	
			0.478	
thickness	0.063			0.030 cub ft.
	1.666 × 0.97	=	1.32	
thickness	0.94			1.241
	0.63 × 1.5	=	0.95	
thickness	1.60			1.520
				2.791
	4 Required		4 @	11.16 cub ft. (1:2:4)
			Total	136.10 " "
				= 0.63 立方碼 (1:2:4)
	11 Required		11 @ 0.63	= 6.93 立方碼
Handrail on West Approach concrete Span				
Top rail				
"	0.5 × 0.25	=	0.125	
"	½ × 0.5 × 0.062	=	0.016	
Bottom rail	0.5 × 0.36	=	0.180	
			0.321	
	length 45.27			14.53 cub ft. (1:2:4)
Post A	½ × 0.916 × 0.79	=	0.181	
	thickness 0.063			0.011 cub ft.
	0.916 × 0.79	=	0.724	
	thickness 0.96			0.695
	0.75 × 0.63	=	0.473	
	thickness 1.60			0.757
				1.463
	6 Required		6 @	8.74 cub ft. (1:2:4)
Post B				
	0.5 × 0.75	=	0.375	
	thickness 1.82			0.683 cub ft.
	4 Required		4 @	2.73 cub ft. (1:2:4)
Post D				
	½ × 0.333 × 0.79	=	0.132	
	⅓ × 0.460 × 0.79	=	0.121	
	thickness 0.063		0.253	0.016 cub ft.
	0.79 × 0.79	=	0.624	
	thickness 0.94			0.597
	0.63 × 0.71	=	0.447	
	thickness 1.81			0.809
				1.412
	1 Required		1 @	1.41 cub ft. (1:2:4)
			Total	27.45 cub ft. (,)
				= 0.13 立方碼
	1 Required			

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Pedestal

			Area	Volume
	7.0 . 2.0	=	14.0	
Height	3.63			14.57 cub ft.
	1.54 . 1.54	=	2.37	
Height	0.5			1.19
	7.38 × 1.38	=	1.90	
Thickness	½ × 0.125			0.06
	12 . 0.25 . 0.063	=	0.19	
Thickness	0.38			0.07
	4 × 0.25 × 0.063	=	0.06	
Thickness	0.38			0.02
				15.86 cub ft.
				= 0.07 立方ヤード (1:2:4)
	2 Required	2 @ 0.07	=	0.14 立方ヤード

Pedestal Support for East Abutment

	½ . (0.75 + 1.16) . 2.5	=	2.39	
Height	6.0			14.34 cub ft.
	0.25 × 2.5	=	0.63	
Height	11.5			7.25
				21.59
				= 0.10 立方ヤード (1:2:4)
	1 Required			

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Bents Nos 1-2-4-5-7-8-10-11-13-14-16-17-19-20-22-23
25-26-28-29-31-32

			Area	Volume	
Wall B		2-60"×55⅞" = 6,693.6 □"			
		2×54"×3" = 324			
		7,017.6÷144 =	48.73 □'	cub.ft.	
	thickness	1-6"		73.10	(1:2:4)
Struts		2×26"×9" = 468 □" ÷144 =	3.25 □'		
	length	3-10"		12.46	(")
		2×22"×6" = 264 □" ÷144 =	1.83 □'		
	length	4-2"		7.63	(")
		2×626 □" = 1252 ÷144 =	8.69 □'		
	thickness	1-6"		13.04	(")
Webs		2×4-6"×1-6" =	13.50 □'		
	length	7-0"		94.50	(")
Columns	Outside	2×38×38" = 2888 □"			
	Inside	38×32" = 1216			
		4104 □" ÷144 =	28.50 □'		
	thickness	0-9"		21.38	(")
	Outside	2×34"×34" = 2312			
	Inside	34×28" = 952			
		3264 □" ÷144 =	22.66 □'		
	thickness	0-6"		11.33	(")
	Outside	2×2-6"×2-6"	12.50 □'		
	Inside	2-6"×2-0"	5.00		
			17.50		
	length	17-2"		300.42	(")
Pedestal		5-0"×8-0"×18-6" =		740.00	(")
			Sum.	1773.80 cub.ft.	(")
				= 5.90 立坪 (1:2:4)	
Basement		9-0"×7-0"×18-6" =		1165.50 cub.ft. (1:3:6)	
				= 5.39 立坪 (1:3:6)	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Bents 3-6-9-12-15-18-21-24-27-30

Cross Beam A	2-8½"×44½" = 756.50	Area	Volume	
Wall A	2-8½"×55 15/16" = 948.76			
	1704.76÷144 = 11.84	sq.ft.	cub ft.	
length	5'-0"		59.20	(1:2:4)
Cross Beam A	2-8½"×3" = 51.0"			
Wall A	2-8½"×3" = 51			
	102.0"÷144 = 0.71	sq.ft.	cub ft.	
length	4'-6"		3.20	(")
Struts	2×26"×9" = 468.0"÷144 = 3.25	sq.ft.	cub ft.	
length	3'-10"		12.46	(")
"	2×22"×6" = 264.0"÷144 = 1.83	sq.ft.	cub ft.	
length	4'-2"		7.63	
"	2×626.0" = 1252.0"÷144 = 8.69	sq.ft.	cub ft.	
thickness	1'-6"		13.04	(")
Webs	2×18"×54" = 1944.0"÷144 = 13.50	sq.ft.	cub ft.	
length	7'-0"		94.50	(")
Columns outside	2×38"×38" = 2888.0"÷144 = 20.06	sq.ft.	cub ft.	
thickness	0'-9"		15.05	(")
"	2×34"×34" = 2312.0"÷144 = 16.05	sq.ft.	cub ft.	
thickness	0'-6"		8.03	(")
"	2×30"×30" = 1800.0"÷144 = 12.50	sq.ft.	cub ft.	
Length	17'-2"		214.58	(")
side	38"×32" = 1216.0"÷144 = 8.44	sq.ft.	cub ft.	
thickness	0'-9"		6.33	(")
	34"×28" = 952.0"÷144 = 6.61	sq.ft.	cub ft.	
thickness	0'-6"		3.31	(")
	30"×24" = 720.0"÷144 = 5.00	sq.ft.	cub ft.	
length	17'-2"		85.83	(")
Pedestal	5'-0"×8'-0"×18'-6" =		cub ft. 740.00	(")
		Sum.	1763.16 cub ft. (1:2:4)	
		=	5.85立坪 (1:2:4)	
Basement	9'-0"×7'-0"×18'-6" =		1165.50	(1:3:6)
		=	5.39立坪 (1:3:6)	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

West Abutment for Highway

			Area	Volume	
Wing wall					
	1.25 × 11.40	=	14.25		
	2.0 × 11.16	=	22.32		
			36.57	cub.ft. 36.57	(1:7:4)
thickness	1.0				
Parapet Wall					
	4.198 × 1.0	=	4.198		
length	15.25			cub.ft. 64.02	(1:2:4)
Coping					
	3.25 × 1.0	=	3.25		
length	16.5			cub.ft. 53.63	(1:2:4)
Shaft	4.5 × 18.16	=	81.72		
length	16.0			cub.ft. 1,307.52	(1:2:4)
Wall					
	2.25 × 3.0	=	6.75		
length	7.5			cub.ft. 50.63	(1:2:4)
	1.875 × 3.0	=	5.63		
length	3.0			cub.ft. 16.89	(1:2:4)
	2.5 × 2.5	=	6.25		
Height	6.5			cub.ft. 40.63	(1:2:4)
Coping	2.5 × 1.0	=	2.50		
length	9.5			cub.ft. 23.75	(1:2:4)
Shaft	4.125 × 18.16	=	74.91		
length	12.0			cub.ft. 898.92	(1:2:4)
				Sum. 2,492.56 cub.ft	
				= 11.54 立坪	(1:2:4)
Basement	12.0 × 4.0	=	48.0		
length	30.0			1440.0 cub.ft.	
				= 6.67 立坪	(1:3:6)

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

East obutment for Railway

Parapet wall.			Area	Volume
	1.25 × 5.92	=	7.40	
	1.5 × 6.48	=	9.72	
			17.12	
length	3.25			55.64 cub ft.
	2.40 × 1.5	=	3.60	
Length	3.0			10.80 "
	4.08 × 1.5	=	6.12	
Length	2.38			14.57 "
			7.40	
Length	0.875			6.48 "
	2.25 × 7.25	=	16.31	
thickness				16.31 "
	1.25 × 5.34	=	6.68	
	1.5 × 6.48	=	9.72	
			16.40	
Length	1.0			16.40 "
	1.25 × 5.92	=	7.40	
	1.5 × 5.22	=	7.83	
			15.23	
Length	2.25			34.27 "
	1.25 × 5.34	=	6.68	
	1.5 × 5.22	=	7.83	
			14.51	
Length	1.0			14.51 "
	1.25 × 5.75	=	7.19	
Length	16.0			115.04 "
Coping				
	4.0 × 1.0	=	4.00	
Length	16.5			6.60 "
Shaft	(3.75+7.5) ÷ 2 × 22.45	=	126.28	
Length	16.0			2020.48 "
			Summary	2370.50 cub ft.
				= 10.97 立坪 (1:2:4)
Basement				
	14.0 × 5.0	=	70.0	
Length	18.0			1260.00 cub. ft.
				= 5.83 立坪 (1:3:6)

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Approach Pier for Railway

			Area	Volume
Coping				
	5'.5'	=	25.0	
	5' dia.	=	19.63	
			44.63	
	Thickness 1.0			44.63 cub ft.
				= 0.21 立坪 (1:2:4)
Shaft.	Top area 4.5 × 5	= 22.50		
	4.5 dia	= 15.90		
			38.40	
	Bottom area 5' × 7.43	= 37.15		
	7.43 dia.	= 43.35		
			80.50	
	Mean area		118.90 ÷ 2 = 59.45	
	Height 29.42			1749.07 cub. ft.
				= 8.10 立坪 − 4.47 = 3.63 立坪 (1:2:4)
Basement	9.5 × 14.0	=	133.0	
	Thickness 3.0			399.00 cub ft.
				= 1.85 立坪 (1:3:6)
Shaft.	5.0 × 2.5 = 12.5			
	2.5 dia. = 4.91			
		=	17.41	
	5.6 × 5.28 = 26.40			
	5.28 dia. = 21.81			
			48.21	
			65.62 ÷ 2 = 32.81	965.77 cub ft.
	Height 29.42			= 4.47 立坪 (Rubble Concrete)

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

West Abutment for Railway

			Area (□')	Volume (cub ft)
Parapet wall				
	1'×5'	=	5.0	
Length	2'-0"			10.0 cub ft
	1'×2.99	=	2.99	
Length	10.0			29.9 cub ft
				39.9 cub ft
Coping				
	3.25×1.0	=	3.25	
Length	10.5			34.13 cub ft
Shaft				
	(3.0+6.0)÷2×18.16	=	81.72	
Length	10.0			817.20 cub ft
	(4.25+6.0)÷2×10.66	=	54.63	
Length	1.25			68.29 cub ft
				959.52 (1:2:4)
				= 4.44 立±平
Basement				
	12×14.5	=	174.0	
thickness	4.0			696.0 cub ft (1:3:6)
				= 3.22 立±平
Piling	20本 15.0×0.5			

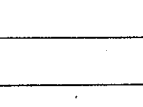
East Abutment for Railway

			Area (□')	Volume (cub ft)
Wing wall				
	(1'+1.33)÷2×2.5	=	2.91	
Length	(6.5+6.25)÷2=6.375			18.55 cub ft
	(1.33+1.58)÷2×2.16	=	3.14	
Length	(6.25+4.58)÷2=5.42			17.02 cub ft
	(1.58+2.33)÷2×6.92	=	13.53	
Length	2.29			30.98 cub ft
				66.55
Parapet wall				
	1.25×4.66	=	5.83	
Length	10.0			58.30 cub ft
Coping				
	4.25×1.0	=	4.25	
Length	10.5			44.63 cub ft
Shaft	(4.0+7.5)÷2×22.44	=	129.03	
Length	10.0			1290.30 cub ft
	(4.75+7.5)÷2×16.94	=	103.76	
Length	1.25			129.70 cub ft
				1589.48 (1:2:4)
				= 7.36 立±平
Basement	14.5×14.0	=	203.0	
Thickness	5.0			1015.0 cub ft (1:3:6)
				4.70 立±平

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

	Required	Area (面積)	Total (面積)	Remarks
Piers NOS 1-8		Summary of.		
Sturt. Pedestal Shaft.	2@	45.2	90.4	
Piers NOS 2.3.4.5.6.7 "	6@	53.0	318.0	
	Total		408.4	
Piers NOS 1-8				
WELL	2@	121.0	242.0	
" NOS 2.3.4 "	3@	142.1	426.3	
" NOS 5-6-7 "	3@	157.0	471.0	
	Total		1.139.3	
Slab of Riverspan	7@	105.0	735.0	
	Total		735.0	
Approach span.				
Inside Girder	11@	28.3	311.3	
Outside Girder	22@	27.1	596.2	
West Approach span Girder	3@	3.6	10.8	
" Cross Beam	1@	1.3	1.3	
" Slab	1@	14.3	14.3	
Approach span Slab	11@	73.0	803.0	
	Total		1.736.9	
Handrail on Truss span	7@	27.1	189.7	
Handrail on concrete span	11@	21.4	235.4	
Handrail on West "	1@	4.4	4.4	
Pedestal	2@	0.9	1.8	
Pedestal supports	1@	1.0	1.0	
	Total		432.3	
Bents				
NOS 1.2.4.5.7.8.10.11.13.14.16 17.19.20.22.23.25.26.28.29.31.32	22@	34.6	761.2	
NOS 3.6.9.12.15.18.21.24.27.30	10@	36.9	369.0	
	Total		1.130.2	
Abutment for Highway				
West	1@	44.2	44.2	
East	1@	40.7	40.7	
	Total		84.9	
Approach Piers Railway	32@	24.5	784.0	
	Total		784.0	
Abutment for Railway				
West	1@	16.0	16.0	
East	1@	29.4	29.4	
	Total		45.4	
	Total Summary		6.496.4	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

	Required	Area (sq ft)	Area (㎡)	Remarks
Piers NOS 1-8				
Coping	2 x 20.42 x 0.58 =	23.7		
"	2 x 20.42 x 0.25 =	10.2		
"	2 x 22.0 x 0.92 =	40.5		
"	2 x 22.0 x 0.25 =	11.0		
Shaft	2 x 22.77 x 20.18 =	919.0		
Strut	2 x 27.33 x 1.16 =	63.4		
"	2 x 27.83 x 0.83 =	46.2		
"	1 x 28.33 x 1.50 =	42.5		
"	3.0 x 28.33 =	85.0		
" 	2 x (137°-93°) =	88.0		
	2.5 x 29.8 =	74.5		
	7.82 x 28.5 =	222.9		
	Total	1.626.9	= 45.2	
	2 Req'd		= 90.4	
WELL	2 x 40.84 x 30.15 =	2.462.7		
"	2 x 31.42 x 30.15 =	1.894.6		
	Total	4.357.3	= 121.0	
	2 Req'd		= 242.0	
Piers NOS 2-3-4-5-6-7				
Shaft	2 x 27.58 x 20.18 =	1.113.1		
Coping	2 x 29.84 x 1.18 =	70.4		
"	2 x 26.70 x 0.83 =	44.3		
Strut	2 x 30.0 x 2.30 =	138.0		
"	2 x 4.5 x 2.30 =	20.7		
"	2 x 27.0 x 5.50 =	297.0		
"	4.5 x 29.8 =	134.1		
"	2 x (137°-93°) =	88.0		
	Total	1.905.6	= 53.0	
	6 Req'd		= 318.0	
Piers NOS 2-3-4				
WELL	2 x 47.12 x 30.16 =	2.842.3		
"	2 x 37.70 x 30.16 =	2.274.1		
	Total	5.116.4	= 142.1	
	3 Req'd		= 426.1	
Piers NOS 5-6-7				
WELL	2 x 47.12 x 33.33 =	3.141.0		
"	2 x 37.70 x 33.33 =	2.513.1		
	Total	5.654.1	= 157.0	
	3 Req'd		= 471.0	
Slab at River span				
	22.5 x 168.0 =	3.780.0		
	Total	3.780.0	= 105.0	
	7 Req'd		= 735.0	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

		Area (squ. ft)	Area (面ヲ)	Remarks
Approach span.				
Inside Girder.				
One Req'd Thus	2×2.75 = 5.5			
" B □ a 1.92a □ B 2aB	$\frac{1}{3}$ × 1.92 = 0.64			
"	1 × 1.50 = 1.50			
Summary	= 7.64			
Length	3×44.5 = 133.5	1,019.9	= 28.3	
	11 Req'd		= 311.3	
Out side Girder.				
One Req'd Thus	2×2.58 = 5.16			
"	$\frac{1}{3}$ × 1.92 = 0.64			
"	1 × 1.50 = 1.50			
Summary	= 7.30			
Length	3×44.5 = 133.5	974.6	= 27.1	
	22 Req'd		= 596.2	
Roadway Slab.				
	19.67 × 133.5 =	2,625.9		
	Total	2,625.9	= 73.0	
	11 Req'd		= 803.0	
West Approach Span				
Girder.	2×1.75×23.67 =	82.8		
"	1×1.50×23.67 =	35.5		
	2×3.5×1.5 =	10.5		
	Total	128.8	= 3.6	
	3 Req'd		= 10.8	
Slab	19.66×23.66 =	465.2		
"	(81° 64.6 =	16.4		
"	14.1×2.33 =	32.9		
	Total	514.5	= 14.3	
	1 Req'd		= 14.3	
Cross Beam.	2×5.0×2.0 =	20.0		
	5.0×0.7 =	3.5		
	Total	23.5	= 0.65	
	2 Req'd		= 1.3	
Handrail.				
Handrail on Truss Span				
top rail	2×0.31 = 0.62			
bottom rail	2×0.50 = 1.00			
Summary	= 1.62			
Length	304.0	492.5		
Post A				
	2×0.916×2.85 = 5.22		1.	
	2×0.79×2.85 = 4.50			
Summary	= 9.72			
	34 Req'd	330.5		
Post B				
	2×0.75×1.82 = 2.73			
	2×0.50×1.82 = 1.82			
Summary	= 4.55			
	28 Req'd	127.4		

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

		Area (squ ft)	Area (㎡)	Remark's
Post				
	2×0.4×1.82 = 1.46			
	2×0.5×1.82 = 1.82			
Summary	= 3.28			
	8 Reqd	26.2		
	Total	976.6	= 27.1	
	7 Reqd		= 189.7	
Handrail on Concrete Span.				
top rail	2×0.31 = 0.62			
bottom rail	2×0.50 = 1.00			
Summary	= 1.62			
Length	240.5			
		389.6		
Post A				
	2×0.916×2.85 = 5.22			
	2×0.79×2.85 = 4.50			
Summary	= 9.72			
	18 Reqd	175.0		
Post B				
	2×0.75×1.82 = 2.73			
	2×0.50×1.82 = 1.82			
Summary	= 4.55			
	24 Reqd	109.2		
Post D				
	2×0.916×2.85 = 5.22			
	2×0.79×2.85 = 4.50			
Summary	= 9.72			
	4 Reqd	38.9		
Post C				
	2×1.67×2.85 = 9.52			
	2×0.79×2.85 = 4.50			
Summary	= 14.02			
	4 Reqd	56.1		
	Total	768.8	= 21.4	
	11 Reqd		= 235.4	
Handrail on West Approach Span.				
top rail	2×0.31 = 0.62			
bottom rail	2×0.50 = 1.00			
Summary	= 1.62			
Length	45.27	73.3		
Post A				
	2×0.916×2.85 = 5.22			
	2×0.79×2.85 = 4.50			
Summary	= 9.72			
	6 Reqd	58.3		
Post B				
	2×0.75×1.82 = 2.73			
	2×0.50×1.82 = 1.82			
Summary	= 4.55			
	4 Reqd	18.2		
Post D	2×0.916×2.85 = 5.22			
	2×0.79×2.85 = 4.50			
Summary	= 9.72			
	1 Reqd	9.72		

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

		Area (sq.ft)	Area(面積)	Remark's
	Total	159.5 =	4.4	
	1 Req'd	=	4.4	
Pedestal				
	4×2.0×4.25 =	34.0		
	Total	34.0 =	0.9	
	2 Req'd	=	1.8	
Pedestal Support on East Abutment for Highway				
	2.5×6.0 =	15.0		
	2×½(0.75+1.16)×6.0=	11.5		
	2×0.25×11.5 =	5.8		
	0.75×2.5 =	1.9		
	Summary =	34.2 =	1.0	
	1 Req'd	=	1.0	
Bents				
NOS 1-2-4-5-7-8-10-11-13-14-16-17 19-20-22-23-25-26-28-29-31-32				
WALL B	2×5.0×5.0 = 50.0			
	2 Req'd	100.0		
Columnn	2×3.17×4×0.75=	19.0		Outside
	2×2.83×4×0.50 =	11.3		"
	2×2.50×4×17.17 =	343.4		"
	2×3.17×4×0.17 =	4.3		"
	2×2.83×4×0.17 =	3.8		"
	2×3.17×0.75 =	4.8		Inside
	2×2.83×0.50 =	2.8		"
	2×2.67×0.75 =	4.0		"
	2×2.33×0.50 =	2.3		"
	2×2.5×17.17 =	85.9		"
	2×2.0×17.17 =	68.7		"
Sturt	2×3.83×0.75=5.70			
	2×4.17×0.50 =4.20			
	2×3.83×0.17 = 1.30			
	2×4.17×0.17 = 1.40			
	2×4.5×1.67 =15.0			
45	20.25-15.9 = 4.4			
	7.1×1.5 =10.7			
Web	2×4.5×7.0 =63.0			
	Summary =105.7			
	2 Req'd	211.4		
Pedestal	2×8.17×18.5 =	302.3		
	2×5.0×8.0 =	80.0		
	Total	1.244.0 =	34.6	
	22 Req'd	=	761.2	
Bents				
NOS 3.6-9.12.15.18.21.24.27.30				
Cross Beam A	2×5.0×3.75 = 37.5			
	1×0.7×5.0 = 3.5			
WALL A	2×5.0×5.0 = 50.0			
	Summary = 91.0			
	2 Req'd	182.0		
Columnn	2×3.17×4×0.75 =	19.0		Outside
	2×2.83×4×0.5 =	11.3		"

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

		Area(sqm A)	Area(面積)	Remark's
Columnmn	2×2.5×4×17.17 =	343.4		Outside
	2×3.17×4×0.17 =	4.3		"
	2×2.83×4×0.17 =	3.8		"
	2×3.17×0.75 =	4.8		Inside.
	2×2.83×0.50 =	2.8		"
	2×2.67×0.75 =	4.0		"
	2×2.33×0.50 =	2.3		"
	2×2.5×17.17 =	85.9		"
	2×2.0×17.17 =	68.7		"
Sturt	2×3.83×0.75=~~5.7~~			
	2×4.17×0.50 = 4.2			
	2×3.83×0.17 = 1.3			
	2×4.17×0.17 = 1.4			
	2×4.5×1.67 =15.0			
(45)	20.25-15.9 = 4.4			
	7.1×1.5 =10.7			
Web	2×4.5×7.0 =63.0			
Summary	=105.7			
	2 Reqd	211.4		
Pedestal	2×8.17×18.5 =	302.3		
	2×5.0×8.0 =	80.0		
	Total	1.326.0	= 36.9	
	10 Reqd		= 369.0	
Abutment for Highway.				
West. parapet Wall	2×11.4×3.25 =	74.1		
"	2×4.25×28.0 =	238.0		
"	1×1.0×11.4 =	11.4		
"	1×9.5×2.5 =	23.8		
Coping	2×1.0×33.16 =	66.3		
Shaft	18.25×28.0=	511.0		
"	18.16×28.0 =	508.5		
"	4.5×18.16 =	81.7		
"	4.13×18.16 =	75.0		
	Total	1.589.8	= 44.2	
	1 Reqd		= 44.2	
East. Parapet Wall	2×5.83×16.0 =	186.6		
Wing Wall	2×4.25×12.42 =	105.6		
"	2×7.42×3.0 =	44.5		
"	1.5×17.42 =	26.1		
"	1.6×7.5 =	7.5		
"	2×11.16×3.25 =	72.5		
"	1.5×11.16 =	16.7		
Coping	1.0×16.33 =	16.3		
"	0.33×16.33 =	5.4		
"	2×4.0×1.0 =	8.0		
Shaft	22.75×16.0 =	364.0		
"	22.50×16.0 =	360.0		
"	2×5.58×22.5 =	251.1		
	Total	1.464.3	= 40.7	
	1 Reqd		= 40.7	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

		Area(SQU ft)	Area (面坪)	Remarks
Approach Piers for Railwy.				
NOS. 1 to 32.				
Coping	2x5.0x1.5 =	15.0		
"	15.7x1.5 =	23.6		
Shaft	2x5.0x29.44 =	294.4		
"	½(14.14+23.24)x29.44=	550.2		
	Total	883.2	= 24.5	
	32 Req'd		= 784.0	
Abutment for Railway.				
West Wing Wall	2x5.0x2.0 =	20.0		
	1.0x5.0 =	5.0		
Parapet Wall	2x3.0x10.0 =	60.0		
"	2x3.0x1.0 =	6.0		
Coping	2.0x15.0 =	30.0		
Shaft	2x7.5x10.0 =	150.0		
"	2x3.6x7.5 =	54.0		
"	2x11.25x10.6=	238.5		
"	2x5.2x10.9 =	113.4		
	Total	576.9	= 16.0	
	1 Req'd		= 16.0	
East.				
Wing Wall	2x7.0x6.5 =	91.0		
Parapet Wall	2x4.7x10.0 =	94.0		
"	2x4.7x1.25 =	11.8		
Coping	2x4.0x14.5 =	116.0		
Shaft	2x4.25x5.5 =	46.8		
"	2x10.0x5.5 =	110.0		
"	2x11.25x17.0 =	382.5		
"	2x6.1x17.0 =	207.4		
	Total	1059.5	= 29.4	
	1 Req'd		= 29.4	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

	Number	[illegible]	[illegible]	Remarks
		下部工合計		
Pedestal	2 Regd	1.06	2.12	
East Pedestal for Supports				
	1 Regd	1.76	1.76	
West "	1 Regd	1.55	1.55	
Handrail on Concrete Span				
	11 Regd	27.50	302.50	
Handrail on West Approach span				
	1 Regd	5.50	5.50	
Handrail on Truss span				
	7 Regd	35.00	245.00	
	Total		= 558.43	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

		Area (sq ft)	Area (sq ft)	Remarks
Pedestal	4.25 × 1.125 =	24.0		
"	3.0 × 2.0 =	4.0		
	Summary	28.0	= 1.06	
	2 Req'd		= 2.12	
Support for pedestal				
East	2.5 × 6.0 =	15.0		
	2 × ½(0.75+1.16) × 6.0 =	11.5		
	2 × 0.25 × 11.5 =	5.8		
	2.5 × 11.5 =	28.8		
	0.25 × 9.0 =	2.3		
	Total	63.4	= 1.76	
	1 Req'd		= 1.76	
West	3 × 2.5 × 6.25 =	46.9		
	1 × 1.0 × 6.5 =	6.5		
	0.25 × 9.0 =	2.3		
	Total	55.7	= 1.55	
	1 Req'd		= 1.55	
Handrail on Concrete Stem.				
top rail	1.666			
bottom rail	0.25			
Summary	= 2.791			
Length	240.5	671.2		
Post A				
	2 × 0.92 × 0.94 = 1.73			
	2 × 0.79 × 0.94 = 1.49			
	0.92 × 0.79 = 0.73			
	2 × 0.75 × 1.57 = 2.36			
	2 × 0.63 × 1.57 = 1.98			
Summary	= 8.29			
less	= 0.60			
	= 7.69			
	18 Req'd	138.4		
Post B				
	2 × 0.75 × 1.81 = 2.72			
	2 × 0.50 × 1.81 = 1.81			
Summary	= 4.53			
	24 Req'd	108.7		
Post C				
	2 × 1.67 × 0.94 = 3.14			
	2 × 0.94 × 0.79 = 1.49			
	1.67 × 0.94 = 1.57			
	2 × 1.5 × 1.57 = 4.71			
	2 × 0.63 × 1.57 = 1.98			
Summary	= 12.89			
less	= 0.60			
	= 12.29			
	4 Req'd	49.2		
Post D				
	2 × 0.833 × 0.94 = 1.57			
	1 × 0.79 × 0.94 = 0.74			
	0.833 × 0.79 = 0.66			
	2 × 0.75 × 1.57 = 2.36			
	1 × 0.63 × 1.57 = 0.99			

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

		Area(SQ.FT.)	Area(面積)	Remarks
Summary	= 6.32			
less	= 0.30			
	= 6.02			
	4 Req'd	24.1		
	Total	991.6	= 27.50	
	11 Req'd		= 302.50	
Handrail on West	Approach Span.			
top rail	1.666			
bottom rail	1.125			
Summary	2.791			
Length	45.27	126.3		
Post A				
	2x0.92x0.94 = 1.73			
	2x0.79x0.94 = 1.49			
	0.92x0.79 = 0.73			
	2x0.75x1.57 = 2.36			
	2x0.63x1.57 = 1.98			
Summary	= 8.29			
less	= 0.60			
	= 7.69			
	6 Req'd	46.1		
Post B.				
	2x0.75x1.81 = 2.72			
	2x0.50x1.81 = 1.81			
Summary	= 4.53			
	4 Req'd	18.1		
Post D				
	2x0.833x0.94 = 1.57			
	1x0.79x0.94 = 0.74			
	0.833x0.79 = 0.66			
	2x0.75x1.57 = 2.36			
	1x0.63x1.57 = 0.99			
Summary	= 6.32			
less	= 0.30			
	= 6.02			
	1 Req'd	6.0		
	Total	196.5	= 5.50	
	1 Req'd		= 5.50	
Handrail on Truss Span				
top rail	1.666			
bottom rail	1.125			
Summary	2.791			
Length	305.0	851.3		
Post A.				
	2x0.92x0.94 = 1.73			
	2x0.79x0.94 = 1.49			
	0.92x0.79 = 0.73			
	2x0.75x1.57 = 2.36			
	2x0.63x1.57 = 1.98			
Summary	= 8.29			
less	= 0.60			
	= 7.69			

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

		Area (squ ft).	Area (面坪).	
	34 Reg'd	261.5		
Post B.				
	2x0'75x1'81 = 2.72			
	2x0'50x1'81 = 1.81			
Summary	= 4.53			
	28 Reg'd	126.8		
Post.				
	2x0'71x1'81 = 2.57			
	2x0'63x1'81 = 2.28			
Summary	= 4.85			
	4 Reg'd	19.4		
	Total	1.259.0	= 35.00	
	7 Reg'd		= 245.00	
Mortar finish				
Approach Concrete Span.				
	2x2'68 = 5'36			
Length	3x44'5 = 133'5	715.6	= 19.88	
	11 Reg'd		= 218.68	
West Approach Span				
	1x2'68 = 2'68			
Length	51'63	138.4	= 3.84	
	1 Reg'd		= 3.84	
	Total Summary		= 22.52	
River Span.				
	2x2'68 = 5'36			
Length	168'0			
	Total	900.5	= 25.01	
	7 Reg'd		= 175.07	

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Excavation.

Pier's NOS 1.2.3.4.5.6.8

NOS 1&8 143°1'4 × 2 × (44+44) = 25,192.64

NOS 2.3.4.5.6. 188°.69 × 2 × (38+38+39+44+38) = 74,343.86

Summary = 99,536.50

÷ 216 = 460.82 立坪

Pier NO 7

188°.69 × 2 × 26' ÷ 216 = 45.43 立坪

Railway Piers AP1 ~ AP32.

15' × 11.5 × 627' ÷ 216 = 500.73 立坪

Railway Abutments

East Abutment 15.5 × 16 × 27' = 6,696.00

West Abutment 15.5 × 14 × 25' = 5,425.00

Summary = 12,121.00

÷ 216 = 56.12 立坪

Highway Bents NOS 1 ~ 32

19.5 × 11.5 × 627' ÷ 216 = 650.95 立坪

Highway Abutments

East Abutment 19.0 × 16.0 × 27' = 8,208.00

West Abutment 31' × 14' × 25 = 10,850.00

Summary = 19,058.00

÷ 216 = 88.23 立坪

Asphalt 補裝工面坪

A = 2,668.2" × 15.0" ÷ 36 = 1,111.73 面坪

石積

Highway

East ½(2.2+1.6) × ½(2.3+1.3) = 3.4 面坪

" ½ × 1.3 × 15.0 = 9.8 "

West ½ × 1.8 × 15.0 = 13.5 "

Total = 26.7 "

Railway

East ½ × 1.0 × 13.0 = 6.5 "

West ½ × 1.8 × 15.0 = 13.5 "

Total = 20.0 "

Numbers of Pile.

Highway

West Abutment 40

Bents 6@18 = 108

Total = 148

Railway

West Abutment = 20

Piers 6@15 = 90

Total = 110

Total Sum = 258

増田淳氏関係資料
（独立行政法人 土木研究所蔵）

Copyright © (2004) by P.W.R.I.

All rights reserved. No part of this book may be reproduced by any means, nor transmitted, nor translated into a machine language without the written permission of the Chief Executive of P.W.R.I.

この資料は、独立行政法人土木研究所理事長の承認を得て刊行したものである。したがって、本資料の全部又は一部の転載、複製は、独立行政法人土木研究所理事長の文書による承認を得ずしてこれを行ってはならない。