AN ANALYSIS OF THE PRINCIPAL PARTS OF THE STATE HIGHWAY BRIDGE AT GRANDVILLE, MICHIGAN

THESIS FOR THE DEGREE OF B. S.

Henry B. Wildschut

1932

THESIS

Bridges

Civil engineering Bridge work

An Analysis of the Principal Parts of the State Highway Bridge at Grandville, Michigan

A Thesis

submitted to the faculty of the

Michigan State College

of

Agriculture and Applied Science

by

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Bachelor of Science

Index to Thesis

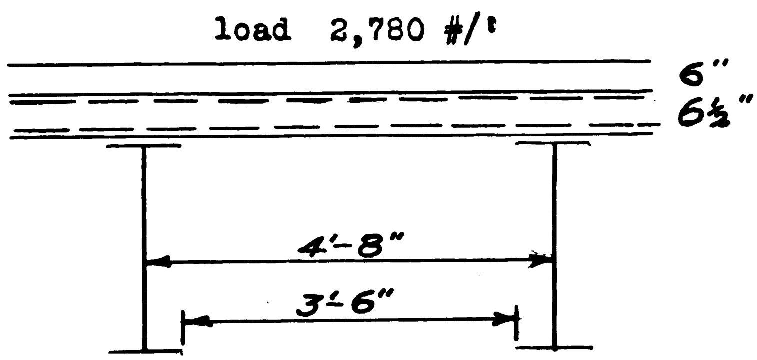
Introduction

The following thesis, which is an analysis of the principal parts of a Michigan State Highway Bridge, is written with the specific purpose in mind of making a general application of the knowledge and information which I have acquired during the last two years of study in the Civil Engineering Department to an actual life project. It is an attempt to more or less crystallize whatever theoretical knowledge I may have assimilated into practical everyday knowledge which I shall be forced to use in following the career I have chosen; namely, Civil Engineering.

The parts which I have analyzed in this thesis are: (1) The Roadway or floorslab, (2) The four types of girders , (3) An Abutment, and (4) A pier.

The general technique which I used is that which is employed by the State Highway Department and which one will find affixed to this thesis under cover of the Michigan State Highway Specifications. Throughout the paper I have used formulas and theories that are found in Sutherland and Clifford's text, "Reinforced Concrete Design".

Analysis of Floor Slab



Floor Beams	---	4'-8"	(center to center)
Flange edge to Flange edge	---	3'-6"	
Depth of Steel from surface of Concrete	------	5"	
Spacing of Bars	-----	4"	(center to center)
Bars ---- ½ inch square	Area ---	.25 square inches.	
Loading-Uniform Dead Load	------	156	# per foot
Uniform Live Load	------	450	" "
Concrete Load (21000/9)	----	2330	" "

Moments

D.L. $(1/12\ w\ l^2)$ --- $1/12 \times 156 \times (3.5)^2 \times 12$

Ans. 1,910 inch lbs.

L.L. $(1/12\ w\ l^2)$ --- $1/12 \times 2780 \times (3.5) \times 12$

Ans. 34,1000 inch lbs

I. (.3 x L.L.M.) --- .3 x 34,100

Answer 10,230 inch lbs

Total Moment ---- 46,240 Inch Lbs.

Shear

D.L.	3.5	x	156	x	$\frac{1}{2}$	=	270	inch lbs.
L.L.	3.5	x	450	x	$\frac{1}{2}$	=	790	" "
I.	.3	x	790			=	230	" "
							1290	" "
			Plus 50%				645	" "
			Total Shear				1935	" "

$$f_s = \frac{46,240}{.75 \times 7/8 \times 5} = 14,100 \text{ Lbs. per sq.inch.}$$

O.K.

$$f_c = \frac{6\ M}{b\ d^2} = \frac{6 \times 46,240}{12 \times 25} = 925 \text{ Lbs. per sq.in.}$$

O.K.

Allowable

.43 x 2500 = 1070 lbs. per sq. inch (Sp.)

$$v = \frac{1935}{12 \times 7/8 \times 5} = 37 \text{ Lbs. per sq. inch}$$

O.K.

Allowable

.03 x 2500 = 75 lbs per sq. inch (Sp.)

$$A_s = \frac{M}{14000\ d} = \frac{46,240}{70,000} = .66 \text{ Sq. in.}$$

Allowed ---------- .75 sq. inch (Plans)

O.K.

Analysis of 50' Girder — G_3

Length ------------	49' -5½"
Web Plate ----------	44' - 9/16"
Flange Angles ------	(4 angles) 6" x 6" x 7/16"
Depth of Girder ----	3' - 8½"

Web Plate Thickness

t should be not less than $1/20 \sqrt{D}$

D = distance between flanges in inches

$$t = 1/20 \sqrt{44} = 1/20 \times 6.64 = .332 \text{ or } 11/32 \text{ in.}$$

11/32" is less than 9/16"

O.K.

Flange Angles

Distance between centers of gravity of top and bottom flanges.

$$44\tfrac{1}{2} - 2(1.66) = 41.18"$$

Flange Stress --- $\dfrac{577,800 \times 12}{41.18} = 166,000$ Lbs.

Required Area $\dfrac{166,000}{16,000} = 10.4$ sq. inches

Web equivalent 1/8" x 9/16" x 44" = 3.1 sq. in.

Total Required area ------ 7.3 Sq. in.

Gross Area of Angles --------------- 10.12 sq. in.

Rivet Hole area 2(3/4" x 7/16") 0.66 sq. in.

Total Provided Area = 9.46 sq. in.

O.K.

Section Modulus

Required $\quad I/C = M/S = \dfrac{577{,}800 \times 12}{16000} = 433 \text{ in.}^3$

Provided

$$I = \frac{b\,d^3}{12} + (I + Ad^2) = 7/16 \times 44^3 + (17.7 + 5.06 \times 20.59^2)$$

$$I = 3100 + 8672 = 11{,}772$$

$$I/C = \frac{11{,}772}{22.25} = 528''^3$$

O.K.

Rivet Spacing

For compression the rivets at end should be at the minimum spacing ($2\frac{1}{2}$ in.) for a distance of approximately $1\frac{1}{2}$ width of member.

$1.5 \times 40 = 60$ or say 21 rivets @ $2\frac{1}{2}$ in. $= 44\frac{1}{2}$ in.

Spacing for main body of member.

Pitch $\quad \dfrac{7900 \times 41.18}{47{,}200} = \dfrac{326000}{47{,}200} = 6.9$ in.

But since the maximum spacing for $\frac{3}{4}''$ rivets is 5 inches, a 5 in. spacing is used.

Analysis of 50' Girder ------ G_4

Length ---------------------------- 49' - $5\frac{1}{2}$"

Rolled I - Beam --------- 33" @ 125 #/'

Section modulus

Required: $I/C = M/S = \dfrac{499{,}000 \times 12}{16000} = 375 \text{ in.}^3$

Provided: I/C as given by Carnegie Hdbk. for rolled I - Beam 33" @ 125 #/' $= 384.7 \text{ in.}^3$

O.K.

Analysis of 75' Girder --- G_2

Length ---------------------- 74' - 5"

Web Plate ------------------------- 44" x 9/16"

Flange Angles ------------(four angles) 8" x 6" x 11/16"

Depth of Girder ---------------- 3' - $8\frac{1}{2}$"

Web Plate Thickness:

$t = 1/20 \sqrt{D} = 1/20 \sqrt{44} = .332$ or 11/32"

11/32" is less than 9/16"

O.K.

Flange Angles:

Distance between centers of gravity of top and bottom flanges. $44\frac{1}{2}" - 2(1.54) = 41.42$

Flange Stress $= \dfrac{1{,}087{,}000 \times 12}{41.42} = 316{,}000$ lbs.

Required Area $\frac{316,000}{16000}$ ———————— 19.75 sq. in.

Web Equivalent 1/8 x 9/16 x 44 ——— 3.10 sq. in

Total Required 16.65 sq. in.

Gross Area of Angles ———————— 18.30 sq. in.

Rivet Hole area 2 (3/4 x 11/16) ———————— 1.03 sq. in.

Total Provided 17.27 sq. in.

O.K.

Section Modulus

Required: $I/C = M/S = \frac{1,087,000 \times 12}{16000} = 815 \text{ in.}^3$

Provided: Web $I = \frac{b\,d^3}{12} = \frac{9/16 \times 44^3}{12} = 3996 \text{ in.}^4$

Angles $4(58.8 + 9.15 \times \overline{20.71}^2) = 15,916 \text{ in.}^4$

$I/C = \frac{19,912}{22.25} = 880 \text{ in.}^3$

O.K.

Rivet Spacing for this beam the same as for G_3

Analysis of 75' Girder ------ G_1

Length -------- 74' - 5"

Web Plate --- 44 x 9/16

Flange Angles ------ 4 angles - 8" x 6" x 9/16"

Cover Plates ------ 18" x ½" x 72' -5/2"

Depth of Girder ----- 3' - 8½" b to b of angles.

Web Plate thickness

t = 1/20 $\sqrt{44}$ = 1/20 x 6.64 = .332 = 11/32"

11/32" is less than 9/16"

O.K.

Flange angles and Cover Plates

Actual Effective Depth

Part		Area	D from Axis to center of grav.	AxD
2 Angles	6"x6"x9/16"	15.12 sq. in.	0	0
1 Angle	18" x ½"	9.0 sq. in.	1.54 + 25 = 1.79	16.1
Totals		24.12 sq. in.		16.1

Ad/A = $\frac{16.1}{24.12}$ = .67 1.54 - .67 = .87"

Effective Depth 45.5 - .87 = 44.63

Stress in Angles and Plate

$\frac{1,366,300 \times 12}{44.63}$ = 367,000 lbs.

Required Area: $\frac{367,000}{16000}$ -------------- 22.90 sq. in.

Web Equivalent 1/8" x 9/16" x 44 ------ 3.10 sq. in.

Total Required 19.80 sq. in.

Gross Area of Angles ----------------- 15.12 sq. in.

Area of cover plate --------------------- 9.00 sq. in.

Rivet hole area 2(3/4" x 9/16") -2($\frac{3}{4}$" x $\frac{1}{2}$") 1.60 sq. in.

Total area provided --- 22.52 sq. in.

O.K.

Section Modulus:

Required $I/C = M/C = \frac{1,366,300 \times 12}{16000} = 1025$ in.^{3}

Provided Web $I = 3996$ in.^{4}

Angles -- $4(49.3 + 7.56 \times \overline{20.71}^2) = 13200$ in.^{4}

Cover plates $2(.25 + 9 \times \overline{22.5}^2) = 9000$ in.^{4}

Total 26,196 in.^{4}

$I/C = \frac{26,200}{22.75} = 1150$ in.^{4}

Rivet spacing on flanges and cover plate same as on beam G_2

Abutment Analysis

Uniform L.L. --15 ton truck

Uniform D.L. ------ 10" pavement

Surcharge from L.L. ------------------	2.0'
Surcharge from D.L. ------------------	1.3'

Angle of Repose of fill ---------- $\phi = 30°$

Weight of fill ------------------------	100 lbs/cu.ft.
Height of retaining -------------------	18.8 ft.
Total height of fill ------------------	23.6 ft.

For most unsatisfactory conditions, assume in the analysis of the abutment, that the total fill is placed in position before the beams are placed on the abutment, The placing of the beams on the abutment will make it more stable as to sliding friction and overturning. Assune the height of the fill placed behind the abutment as 23.6 ft. and neglect surcharge as it is very probable that it will be added before placing the beams.

Earth pressure $D = C\ wh^2/2$

$$C = \text{cosine}\ \theta \left\{ \frac{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta + \sqrt{\cos^2\theta - \cos^2\phi}} \right\} \qquad \begin{matrix}\theta = 0° \\ \phi = 30°\end{matrix}$$

$$C = 1 \left\{ \frac{1 - \sqrt{.25}}{1 + \sqrt{.25}} \right\} = 1\ (.5/1.5) = 1/3$$

$P = 1/3 \times 100 \times (\overline{23.6}^{\,2} / 2) = \frac{55700}{6} = 9{,}285$ lbs.

Weight of earth	=	100	x	23.6	x	6.25	=	14,800~~0~~ lbs.
Weight of base	=	150	x	2	x	11.6	=	3,500 lbs.
Weight of Wall	=	150	x	16.8	x	1.5	=	3,800 lbs.
						Total	=	22,100 lbs.

Overturning Moment

9,285 x 7.83 = 72,700 Ft. lbs.

Resisting Moment

4.5 x 3,800 + 5.75 x 3,500 + 8.4 x 14,800 =

17,100 + 20,100 + 124,000 = 161,200 ft. lbs.

The abutment is safe for overturning.

For friction between masonry and earth use the coefficient of friction as 0.35

Sliding force = 9,285 lbs.

Resisting force (22,100 x .35) = 7,750 lbs.

The abutment is not safe from sliding without the abutment anchor that is provided.

Analysis of Abutment Anchor

Resisting area of Anchor (5' x 46') = 230 sq. ft.

Total force acting on anchor

9,285 - 7,750 = 1,535 ft. lbs.

Force acting on anchor in lbs. persq. ft.

1,535 ÷ 5 = 307 lbs. / sq. ft.

Passive resistance of earth in front of Anchor

$$P = C\ w\ h^2/2$$

$$C = 1 \left(\frac{1 + .5}{1 - .5} \right) = 3$$

$$P = 3 \times 100 \times \overline{5/2}^{\,2} = 3750 \text{ lbs./ ft.}$$

Safety factor for sliding should be about 2 --- or larger.

3750 is larger than 2 x 1,535

O.K.

Bars connecting anchor to the abutment:

14 bars @ 3' ¢ to ¢ ; Diameter = $1\frac{1}{2}$" ϕ

Area = 1.77 sq. inches

Length of anchor held by each bar is 3 ft.

Stress on each bar 3 x 1,535 = 4,605 lbs.

Each bar can hold 1.77 x 16,000 = 28,300 lbs.

Bars are not encased, therefore such a large safety factor has been introduced.

Pressure on footing:

$$y = \frac{161,200 - 72,700}{22,100} = \frac{88,500}{22,100} = 4$$

$$e = 5.75 - 4 = 1.75'$$

$$S = P/A\ (1 \pm 6e/b) = \frac{22,100}{11.5}\left(1 \pm \frac{10.5}{11.5}\right) = 3,680\ \#/\text{sq.}''$$

or 165 # / sq.'

Thickness of stem —— consider stem as a series of simple beams between counterforts.

$$P = C\,w\,h = 1/3 \times 100 \times 23.6 = 790\ \#/\text{sq.}'$$

$$B.M. = \frac{790 \times \overline{10.25}^{\,2}}{10} = 8300\ '\#$$

$$V = 790 \times 5.12 = 4050\ \#$$

Depth:

Moment - $d = \sqrt{\frac{8300 \times 12}{1108 \times 12}} = \sqrt{77} = 9''$

D = 12"

Shear $d = \frac{4050}{75 \times .875 \times 12} = 5.2''$ or 6"

D = 9"

D as provided is 18"

O.K.

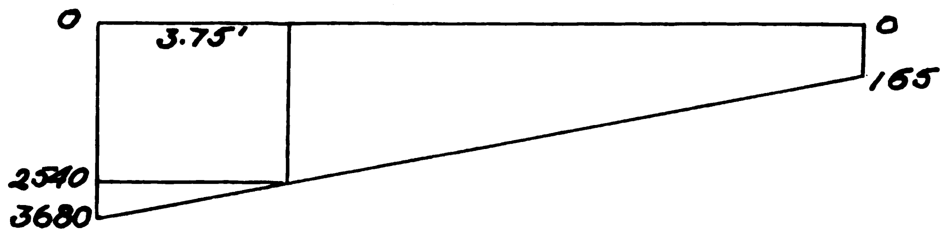
Bond

Allowable .06 x 2500 = 150 #

½" square ; 8" ¢ to ¢ : bars used in stem

$$u = \frac{V}{d\,j\,\Sigma o} = \frac{4350}{15 \times .875 \times 4} = 82\ \#$$

Analysis of Toe



B.M. $= 2540 \times 3.75 \times 1.81 + 1140 \times 3.75 \times 2.5 = 28,600$ '#

V $= 3110 \times 3.75 - 300 \times 3.75 = 10,525$ #

Depth of Toe:

Moment d $= \sqrt{\dfrac{28,600}{108}} = \sqrt{265} = 16.3"$ or $17"$

D = 20"

Shear d $= \dfrac{10,525}{12 \times .875 \times 75} = \dfrac{10,525}{790} = 13.3"$ or $14"$

D = 17"

Heel - Average pressure

$\dfrac{3680 - 165}{11.5} = 306 / 2 = 153; \quad 165 + 153 = 318$#

w $= 23.6 \times 100 + 300 - 318 = 2340$#

B.M. $= wl^2/10 = \dfrac{2340 \times \overline{10.25}^2}{10} = 24,600$ '#

v $= 2340 \times 5.12 = 12,000$ #

Depth of heel

Moment d $= \sqrt{\dfrac{24,600}{108}} = \sqrt{228} = 15.1"$ or $16"$

D = 19"

Shear d $= \dfrac{12,000}{790} = 15.2"$ or $16"$; D = 19"

Allowable Stress of Steel in base:

$f_s = \dfrac{M \times 12}{A_s\ j\ d} = \dfrac{24,600 \times 12}{2 \times .875 \times 21} = 8000$ # / sq."

Bond $u = \frac{rb}{0} = \frac{75 \times 5.5}{3} = 137\#$ O.K.

Analysis of Counterfort

Height of counterfort = 16.8'

$$P = 100 \times \overline{16.8}^2 / 2 \times .333 \times 11.75 = 55{,}350\ \#$$

$$B.M. = 55{,}350 \times 6.25 = 346{,}000\ '\#$$

Depth of Counterfort = d' = 70"

$$R = \frac{346{,}000 \times 12}{18 \times 70^2} = \frac{4{,}150{,}000}{88{,}200} = 47$$

From tables (Sutherland and Clifford) $p = .0036$

Required A_s = .0036 x 18 x 70 = 4.54 sq. in.

Provided: 7 - 1" sq. bars = 7 sq. in.

These rods have a large hook in the bottom to anchor counterfort to footing.

Horizomtal rods to tie counterfort to vertical wall.

Shear at top of footing:

$$100 \times 1/3 \times 16.8 \times 10.25 = 5750\ \#$$

Bars used are 5/8" ∅ ; area .306 sq. in.

Required number = $\frac{5750}{.306 \times 16000}$ = 1.17 bars per foot.

Spacing should be 11" in lower part of counterfort; actual spacing used is 1' - 4"/

Shear at 3' - 0" above footing:

33.3 x 13.8 x 10.25 = 4700#

Required number = $\frac{4700}{4900}$ = .96 bars per ft.

Spacing should be 1' - 2" ; Spacing used is 1' - 8"

Shear at 8' above footing:

33.3 x 8.8 x 10.25 = 2900#

Required number = $\frac{2900}{4900}$ = .59 bars per ft.

Spacing should be 2' - 0"; spacing used is 2' - 0"

Shear at 10' - 0" above footing:

33.3 x 6.8 x 10.25 = 2300 #

Required number = $\frac{23000}{49000}$ =0 .47 bars per ft.

Spacing should be 2' - 2" ; spacing used is 2' - 4'

Analysis of Pier #5

Girders resting on Pier are G-1 and G-2

Weight on Pier:

G-1	weight	73,800 x 4	=	295,200 #
G-2	" "	68,000 x 20	=	1,376,000#
			Total weight	1,671,200 #

Size of bolster block 1.3' x 3.0' x 56.0' = 218 cu. ft.

Weight of bolster block 218 x 150 = 32,700#

Assume the top part of the pier as a continuous beam supported by columns:

Total height of beam = 5' - 0"

Effective width of beam = 1' - 6"

Depth of tension steel from surface = 4' - 9"

Tention steel 4 1" square bars; area 4 sq.in.

The girders are laid so that we can assume that only one girder is supported by the beam and that the others are supported directly by the columns. Take the free span as having a length of 8 ft.

Weight of beam is equal to 1500 #/'

Shear	$2 \times 68,800 \times \frac{1}{2}$	=	68,800 #
	$1500 \times 8 \times \frac{1}{2}$	=	6,000 #
		Total	74,800 #

B.M. 4 x 74,800 x 12 = 3,590,000 "#

$$f_s = \frac{3,590,000}{4 \times .875 \times 57} \doteq 17,900 \text{ \#/ sq.in.}$$

This is a little too large.

$$f_c = \frac{6 \times 3,590,000}{18 \times \overline{57}^2} = 370 \text{ \# / sq. in.}$$

O.K.

Allowable Shear (v) .12 x 2500 = 300# / sq. in.

$$v = \frac{74,800}{18 \times .875 \times 57} = 84 \text{ \# / sq. in.}$$

O.K.

$$A_s = \frac{3,590,000}{14000 \times 57} = 4.5 \text{ sq. in.}$$

Only 4.0 sq. in. is provided.

This beam is difficult to analyze because the contruction is such that various assumptions can be made.

Analysis of Columns

Short Column:

$$h / D = 12 \text{ or less}$$

h = unsupported length of column

D = least outside diameter.

$$h/D = 15.75 / 1.5 = 10.5$$ which is less than twelve.

Class A concrete; f_c = 2500 #/ sq. in.

Total load on one half of pier.

Total load = 2 x 73,800 + 10 x 68,800 = 836,400#

Outside Column:

$$\text{Load} = \frac{11.2}{15.22} \times 147,600 - \frac{6.5}{15.22} \times 137,600 + \frac{1;85}{15.22}$$

$$\text{(con't)} \times 137,600 = 108,500 + 59,000 + 16,750 = 184,250\#$$

Weight of Pier	3' x 3' x 14.5 x 150	=	15,500
Weight of beam	1500 x 8.5	=	12,700
	Total Load	=	212,450 #

$$f_c = P / A = \frac{212,450}{2.5 \times 2.5 \times 144} = 240 \text{ \# / sq. in.}$$

This is rather low, but since the pier was designed with some consideration for its beauty, some area is added to improve the graceful lines of the bridge.

Inside Columns:

$$\text{Load} = \frac{13.4}{15.22} \times 137.6 + \frac{8.7}{15.22} \times 147.6 + \frac{11.65}{15.22}$$

$$\text{(con't)} \quad \times 137.6 + \frac{7.0}{15.22} \times 137.6 + \frac{2.33}{15.22} \times 137.6 = 426{,}600\#$$

Weight of Pier	8.75 x 11.5 x 150	=	15,100#
Weight of beam	15 x 1500	=	22,500#
	Total load	=	464,200#

$$f_c = \frac{464{,}200}{5 \times 144} = \frac{464{,}200}{720} = 645 \ \# / \text{sq. in.}$$

This is safe as allowable f_c = 1050 # / sq. in. Piers 2 - 3 - 4 - 5 - 6- all carry about the same loads. Piers 1 and 7 carry lighter loads but are the same size, so they are also safe for unit stresses.

Design of Base

The footing is desired to be three feet or more below stream bed and the top of the wall of the footing should be above average high water level. For these reasons the footings plus the wall were made 11' - 4¾". The footing and wall are made of grade "B" concrete.

Height of wall and footing ---- = 11' - 4¾"

Assume bearing power of soil as 3 ton/ sq. in.

Thickness for punching shear:

Soil reaction under column (inside column) -

(Approximately) 6000 x 6 = 36,000#

Allowable punching shear - .12 x 2000 = 240 #/sq. in.

Total punching Shear:

$$464{,}200 - 36{,}000 = 428{,}200\#$$

Thickness $\dfrac{428{,}200}{240 \times 16 \times 12} = 10''$ say $1' - 2''$

Footing way safe for punching shear.

Total weight above footing:

$$3 \times 464{,}200 + 2 \times 212{,}450 = 1{,}814{,}900\ \#$$

Weight of wall plus footing:

(approx) $11.5 \times 70 \times 5.67 \times 150 = \underline{675{,}000\ \#}$

Total $= 2{,}489{,}900\ \#$

Area of bottom of footing $7 \times 74 = 518$ sq. ft.

Pressure on footing $\dfrac{2{,}489{,}900}{518} = 4{,}800$ #/ sq. in.

The footing has a large enough area to keep pressure of footing on the soil less than the bearing power of the soil. The wall and footing are made of plain concrete without any reinforcements, except for some steel in the wall which runs vertically into the footing. This anchors the wall to the footing, whic h otherwise would not be since they are poured separately.

Discussion of Analysis

The bridge analyzed crosses the Grand River at Grandville, Michigan. It is on the proposed belt-line which is to go around Grand Rapids. It is of re-inforced concrete construction, with structural steel girders running from pier to pier. The total length of the bridge from abutment to abutment is 550 feet. The span on each end is 50 feet long, with all the other spans being 75 feet in length. It has a roadway 40 feet wide with a 6 foot side-walk on each side.

The test-holes showed that the soil ranges from sand to gravel. Piles were not to be used unless so ordered by the engineer on the job. The loadings of the bridge are the standard h -15 loading of the Michigan State Highway Department. The moments and shears used in the analysis of the girders are the ones used by the designer.

The floor slab was analyzed first and was found to check very closely with the plans. The re-inforcement runs transverse to the girders with bars running perpendicular to hold the re-inforcement in place. The floor slab was considered as a continuous beam. Expansion joints were provided at every pier except #4. The wearing surface was placed directly on the floor slab.

The girders were checked next. There were four types to consider, due to the different lengths and total loads. On each span are twelve(12) girders laid parallel with a spacing of 4'-8". The outside girders on the 50' span are built-up girders, and the inside ones are rolled I-beams. On the 75' span, the girders are all built-up with the outside ones provided with coverplates.

In the analysis of the abutment, the wall and the footing were considered as continuous beams with the counterforts as supports. The wing walls are identical to the abutment but have smaller stresses on them. The anchor provided for the abutment is much too large for the actual stresson it, but it was made that way to introduce the required safety factor.

The pier was the last part to be checked. It was analyzed as a continuous beam supported by columns which stand on a combined wall and foundation.

Conclusion of Thesis

In the checking and analysis involved in this thesis, various phases of civil engineering were used. A working knowledge of steel-girder design, re-inforced concrete beams and columns, counterfort retaining walls, and foundations was obtained.

Many assumptions were made as is customary in reiforced concrete design. Some of these may not agree with those of the reader, but the assumptions made in this thesis were always done with the object in mind of being on the safe side.

This thesis provided many problems which called for a re-consideration of many of the theories I have studied during the last two years, and so has served its purpose of uniting and clarifying my knowledge concerning civil engineering.

- Finis -

500—11-3-31

MICHIGAN
STATE HIGHWAY DEPARTMENT

1926

Standard Road and Bridge Specifications

Reprinted November, 1931

DIVISION 8—BRIDGE DESIGN

Grover C. Dillman, State Highway Commissioner

Martin DeGlopper, Deputy Commissioner

Charles M. Ziegler, Deputy Commissioner

C. A. Melick, Bridge Engineer

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DIVISION 8—BRIDGE DESIGN SPECIFICATIONS

DIVISION 8—BRIDGE DESIGN

SECTION 1—LOCATION

1.1 Alignment.

1.1.1. Curved Approaches.—In locating highway bridges, alignment is a most important consideration. Abrupt turns and sharp curves on bridge approaches are not permissible for modern traffic. In establishing locations for new bridges the following curvature limitations should apply for approaches: For main highways, maximum curvature should not exceed 5° 30′ and tangent distances should be not less than 300 feet. For secondary highways, 10° and 300 feet respectively are desirable. The point of curve at ends of the bridge should be back of the ends of wing walls, and far enough to permit a spiral run-off for superelevation of usually not less than 50 feet. Under some conditions bridges have been built on curves but topographic conditions in Michigan do not justify this practice.

1.1.2 Sight Distance.—Adequate sight distance is essential for safety of modern high speed traffic and approaching vehicles should be visible to a driver for a distance of not less than 500 feet. In alignment studies, the effect of deep cuts, buildings, trees and thru bridge superstructures must be considered.

1.1.3 Angle of Crossing.—In locating bridges right angled crossings are desirable from the standpoints of cost, appearance, and future widening, but skewed crossings are often necessary for highway bridges. If possible, the angle of crossing shall not be less than 45 degrees. Skewed structures require wider bridge seats than right angled crossings and the length of substructure units increases rapidly as the crossing angle is reduced, varying with the cosecant of the angle. Effective waterway for a given span is reduced with reduction of the crossing angle and varies with the sine of the angle. For grade separation structures, a crossing angle of less than 45 degrees is occasionally justified, but the increased cost of the structure should be estimated for comparison of the cost of the project on a better structural location, if such a location is otherwise feasible.

1.2 Approach Grades.

1.2.1 Vertical Curves.—The bridge approach grades are usually controlled to a considerable extent by adjacent topography and the height of superstructure necessary to clear high water or navigation. Maximum sight distance and avoidance of abrupt change in grade are requisites. Vertical curves should be not less than 200 feet long for main highways, and preferably not less than 400 feet long, and should provide a minimum sight distance of 500 feet. For secondary highways, vertical curves should be not less than 100 feet long, and preferably not less than 300 feet long. Short tangents in the profile shall be avoided as far as possible, and in no case should approaches dip lower than 2 feet above the high water elevation. Ends of vertical curves shall generally be not closer than 10 feet from the ends of a bridge.

1.2.2 Cambered Grades.—On long bridges and on bridges with multiple spans of deck type, the roadway may have slight opposing grades intersecting at the middle of the bridge with the crown of roadway on a long vertical curve from end to end of bridge, giving a cambered appearance to the structure as a whole. This arrangement is effective for appearance and also reduces the quantity of approach fill if the approaches are on ascending grades towards the bridge. For other cases a sag in the grade line on the approaches is required.

1.2.3 Maximum Gradient.—Maximum gradient of approaches shall be consistent with prevailing grades on the road improvement and ordinarily will not exceed 4%. For long grades on secondary highways, grades up to 6 or 7 percent are often used.

1.2.4 Maximum Gradient for Grade Separations.—Grade Separation approach grades shall generally not exceed 4% even if ruling grades for the road exceed this. This limitation is justified from an economic standpoint as reducing total rise and fall in the grade and reducing motor vehicle operating costs, and producing greatly increased sight distances and safety.

1.2.5 Economical Height.—Where the elevation of the bridge is not controlled by high water clearance, a study shall be made to determine the economical height, taking into consideration the rapid increase in cost of substructure with increase in height, the possibility of approximate balance of cut and fill quantities for earth work in the approaches, and the availability of borrow or disposition for spoil, and the possible increased length of structure required for increased heights. In the northern part of the state, heavy cuts should be avoided on account of snow drifts. Expensive rock cuts should be avoided, if possible.

1.3 Channel Changes.—Where the bridge location and alignment has been determined by desired road alignment, conditions may often be improved by relocating the bridge and channel or by retaining a present bridge location and relocating the channel. This is particularly desirable when the crossing occurs at a sharp bend in the stream with the effect that flood waters are directed to one side and may undermine the substructure or damage the approach. Channel change may reduce the cost of the project by increasing the angle of crossing and diminishing the disadvantages of a sharply skewed structure. The cost of the structure is also reduced when the substructure can be built away from the present stream location and with little or no cofferdam. Where there is an existing bridge, there is also the possibility of maintenance of traffic over the old bridge and around the proposed site, leaving the channel excavation and filling the old bridge opening as the last operation. With suitable material, channel pilot cuts may sometimes be made and the old channel blocked off, leaving the widening of the cut to the action of flood waters.

1.4 Stream Conditions.—The choice of location will depend in some cases upon local conditions in the stream. Other conditions being equal, the crossing should be made at a narrow part of the stream, on a stretch fairly straight and of uniform section for 100 feet each way. Location subject to scour or bank erosion are to be avoided if possible. Erosion will occur at bottom velocities of ½ foot to 4 feet per second depending upon the material, the former applying to silt and the latter to gravel. For small streams, protection from scour is feasible by rip-rapping or leaving cofferdam sheeting in place. Substructure units shall be so located that they will not unduly deflect the thread of stream and any obstruction which will cause serious eddies shall be avoided or removed.

1.5 Foundation Conditions.

1.5.1 Shallow Foundations.—In considering the relative merits of different bridge locations, the foundation conditions greatly influence the cost. In a given locality these conditions generally will not vary greatly unless rock occurs near foundation depths, but borings will frequently disclose small pockets or deposits of sedimentary origin. The most desirable foundation condition occurs when rock, shale, hard clay, or gravel in thick beds is found near the elevation of the stream bed in shallow water. In this case, shallow, single wall cofferdams suffice.

1.5.2 Intermediate Depths.—When a firm foundation bed occurs at somewhat greater depths, piling may be used, but the minimum length below cut-off shall be 10 feet. For intermediate conditions, where firm foundation beds are at too high an elevation to permit the satisfactory use of piling, but at sufficient depth to entail considerable expense if footings are carried down, open caissons may be sunk at intervals and filled with concrete to support the footings.

1.5.3 Deep Foundations.—In deep water, steel sheet piling cofferdams are used to a depth of about 30 feet. Substructure depths exceeding this are unusual in Michigan, and involve expensive open or pneumatic caissons. In general, cofferdams of the same area will vary in cost approximately with the square of the depth. Where soft clay, silt or sand extends to a considerable depth, foundation piling is used, the maximum length being about 60 feet for timber piles. Long piles require lateral stability, particulary when subject to abutment thrusts. For small structures, under these conditions, concrete struts may often be used between abutments. Long piles without lateral stability must be considered as columns. Foundations on long, unsupported piles, in muck or other soft material may be stiffened by filling the foundation area with rock.

1.6 Dams and Reservoirs.—In the vicinity of dams, bridges should be located downstream, if possible, as involving shorter spans with less depth of water for construction of substructure, inspection, maintenance and future widening. Location downstream is more economical in first cost and for future widening. With inferior types of dams, the possibility of failure must be considered and the bridge provided with sufficient vertical clearance to avoid damage to the superstructure, and the substructure protected from undermining. Artificial alteration of the water stage for any water course, except drains established by public authority, is illegal without approval of the highway authorities having jurisdiction. See Act 354, Public Acts of 1925.

1.7 Navigable Streams.

1.7.1 Waterways Subject to Authority of Boards of County Supervisors.—All streams within the state or on the borders thereof which are navigable for small or large boats or even those capable of carrying only logs are considered navigable under Act 354, Public Acts of 1925, and bridges to be constructed over such streams are subject to approval of the Board of County Supervisors, who have full authority to order reconstruction and specify clearances. The usual requirement in such cases is vertical clearance to permit passage of small boats or launches under ordinary conditions. Where not otherwise specified, a clear height of 4'6" above ordinary water stage for row boats and 8'6" for launches and 16'0" for tugs is desirable. In any case a clear height not less than 3'0" is desirable for access for inspection, maintenance or painting.

1.7.2 Waterways Subject to Authority of the United States War Dept.—Bridges proposed over any waterway which is considered by the Federal Government as navigable, including intrastate waterways which are suitable for interstate traffic, are subject to the approval of the War Department. In all cases, the navigation interests are very closely guarded. Application must be made for approval of plans on a prescribed form, in quadruplicate, to the Secretary of War, through the United States District Engineer in charge of the waterway. The District Engineer at Milwaukee, has jurisdiction in Michigan over waterways tributary to Lake Michigan from St. Joseph to the Straits of Mackinac. South of St. Joseph is territory of the District Engineer at Chicago. The District Engineer at Duluth has jurisdiction over waterways tributary to Lake Superior. The St. Marys River, St. Marys Canal, the Straits of Mackinac and all waterways in the eastern part of the lower peninsula are under the jurisdiction of the District Engineer at Detroit. The application must cite the legal authority for the structure, the identification of the applicant, and must be accompanied by four certified copies of the proceedings of the Board of County Supervisors, under which the project is approved in accordance with Act 354 of Public Acts of 1925. The application must also be accompanied by plans, in quadruplicate, in such form and detail as prescribed by the Secretary of War. All plans must provide for channel project widths and depths, as established by the War Department and Acts of Congress.

1.7.3 Waterways Subject to Congressional Authority.—Bridges over navigable waterways which cross state or international boundaries, and bridges over waterways which form interstate or international boundaries, must be authorized by Act of Congress prior to application to the Secretary of War for approval of plans. For interstate bridges, approval must be secured from all authorities having jurisdiction, in each state.

1.7.4 Navigable Stream Crossings.—In crossing a navigable stream, either a movable bridge or a high-level fixed-span bridge is required. The latter is to be preferred as avoiding delays to highway and water-borne traffic. A further advantage is the saving of substantial operating charges. Where navigation is at present negligible, it may be permissible to construct semi-permanent structures, subject to replacement, or to build a fixed span bridge with substructure provision for a future movable span. Movable bridges must be built on crossing angles as nearly rectangular as possible, so that the span length can be kept to a minimum consistent with navigation and safety requirements. It is also desirable that movable bridges have sufficient vertical clearance to pass small craft without opening and delay to traffic. Frequent opening may even nullify the purpose for which the bridge is built, namely, to carry highway traffic.

1.8 Grade Separations.

1.8.1 Location.—The location of railroad grade separation structures is frequently fixed by the requirement for maintaining a straight highway alignment. Where the angle of intersection between the highway and railroad center lines is less than 45 degrees, or where topographic conditions are favorable, the possibility of relocation or minor alteration of the highway should be considered, if such a relocation will result in material reduction of the total cost of the project, including approaches, and at the same time provide a satisfactory alignment and grade.

Where there is considerable latitude in possible location for a grade separation structure, a careful study shall be made of all conditions which will affect the design and location, with the object of establishing a grade separation which will have the desired safety, convenience, utility and appearance at a minimum cost consistent with these requirements. The limitations specified for highway alignment and approach gradients for bridges, likewise in general apply for grade separation structures. Requirements for roadway and clearances are, specified in Section 3, Division 8.

1.8.2 Methods of Separation.—Grade separation may be achieved by elevation or depression of the highway or railroad, or by elevation of one and depression of the other, or by local relocation of one or both in combination with grade changes. Alteration of the railroad grade or alignment is usually not justified unless the project would otherwise be impracticable or expensive. Maximum gradients for Class A railroads are limited to approximately 0.4% and are subject to the ruling gradient on the line. In a study for location and design the above possibilities shall be given due consideration. The length of railroad grade as compared to highway grade, to achieve a foot change in elevation will ordinarily be in the ratio of about 10 to 1.

1.8.3 Drainage.—If the highway or railroad is depressed, drainage outlet must be provided, if necessary, by automatic pumping equipment, where power is available. In the latter case, operating costs shall be capitalized for comparative estimates.

1.8.4 Choice of Type.—Where there is a possible choice of a subway or a highway overhead crossing, the relative merits of the two shall be considered, as referred to in Section 4, Division 8, but, in general, from the standpoint of safety and community welfare, a greater expenditure is warranted for the subway, as high embankments are often exceedingly objectionable.

The cost of right of way, property damages, maintenance of highway and railroad traffic and interference with existing utilities shall be considered in comparative estimates of cost.

SECTION 2—WATERWAY

2.1 Determination of Waterway Area.—For the determination of the waterway area to be provided by any drainage structure, a careful study shall be made of local conditions, including flood height and flow, size and performance of other openings in the vicinity carrying the same stream, characteristics of the channel and of the watershed area, climatic conditions, available rainfall records and any other information pertinent to the problem and likely to affect the safety or economy of the structure.

For culverts and small bridges, waterway formulas or drainage tables may be used to assist in fixing the proper size of opening. The use of such formulas or tables is justified only to the extent that they are known to fit local conditions. However, their use shall serve merely as a guide and shall not obviate the need for careful field observation and the exercise of intelligent judgment.

In general, the waterway provided shall be sufficient to insure the discharge of flood waters without undue backwater head and at a velocity which will not increase the erosive action of the stream to such an extent as to endanger the structure, or cause damages to upstream property.

2.2 Overflows.—Short overflow spans, for the purpose of supplementing waterway area under flood conditions, shall generally be avoided as uneconomic and their utility limited by hydraulic conditions. For existing overflow channels the wet areas and hydraulic radius at high water shall be considered as a measure of the proportionate run off thru overflow and main channels, due consideration being given to rank growths of vegetation and brush usually encountered in same, and to the channel conditions at each approach.

2.3 Backwater.—If high water results from a back water condition from restriction of run off down stream, the bridge waterway shall provide for the estimated maximum discharge under the low velocity conditions resulting. Under these conditions the vertical clearance between reported extreme high water and superstructure may be reduced if not subject to accumulation of ice or drift, but a minimum clearance of 6 inches shall be provided. Back water from a lake will be considerably affected by winds, depending upon the size and configuration of the lake. Rise of back water from a proposed dam is difficult of accurate computation but allowance should be made for friction head on the basis of velocity and hydraulic radius. In the replacement of bridges subject to back water, care shall be taken to avoid property damage from further backing up the water by restricting the bridge waterway.

2.4 Divided Streams.—Bridges crossing branches of streams with divided outlets shall provide waterways based on the hydraulic radii of the branches at high water. The possibility of future dredging or improvement of one or other branch shall be considered in determining required waterway.

2.5 Restricted Waterways.—When it is necessary to restrict the waterway to such an extent that the stream will be discharged at erosive velocities, protection against damage due to scour shall be afforded by deep foundations, curtain or cut-off walls, rip-rap, stream bed paving, or other suitable means. Likewise, embankment slopes adjacent to all structures subject to erosion shall be adequately protected by rip-rap, brush mattresses, tree retards, wing dams, or other suitable construction.

2.6 Dredged Channels.—In the upper reaches of dredged channels, the hydraulic gradient may be safely assumed as being lowered by the depth of the dredge cut, in the absence of reliable data. Near outlet other conditions will govern. Bridge waterways in channels subject to periodic dredging and silting shall make a reasonable allowance for the effect of silting so that overflow will occur from back water and not from restriction at the bridge. The bridge waterway shall conform as closely as possible to the channel conditions as to width and depth, avoiding change in velocity at the structure with resulting scour or silting.

2.7 Drains.—Bridges crossing drains are subject to the following regulations: Plans are subject to approval of the township, county or state highway commissioner having jurisdiction. Cost of construction is considered a part of the drain, but maintenance as a part of the highway. Refer to Act 367, Public Acts of 1921.

Bridges proposed over streams where future drain projects are a probability shall have footings at a sufficient depth estimated to prevent instability or undermining when the drain project is carried out, or else a temporary structure shall be provided pending the development of the drain project.

2.8 Channel Obstructions.—Natural obstructions, such as islands, rocks, trees and brush which retard or deflect the stream in the vicinity of a bridge shall be removed and such portions of the channel shall be cleaned out as are necessary to straighten out the stream at the bridge and prevent eddies or scour. Booms, rafts, logs or other obstructions which are a menace to a bridge or which interfere with construction or maintenance of a bridge, may be legally removed under authority of Act 354, Public Acts of 1925, but when the entire stream is similarly obstructed, no great expense is warranted.

2.9 Stream Relocations.—In many locations a road crosses a meandering stream more than once and the economics of a relocation of the stream should be studied when bridge construction is proposed. Bridges may be entirely eliminated in some cases, while other cases will require the substitution of small culverts so that property owners may not be deprived of fresh water for stock or other purposes.

2.10 Channel Openings.—Channel openings in navigable streams shall conform in width, height, and location to the requirements of the U. S. War Department. In accordance with these requirements, application for a permit shall be submitted on a designated form accompanied by plans conforming with the regulations of the War Department. Plans shall provide for the project channel width and depth, if any, established by the War Department.

Prior to starting detailed plans for any bridge, a preliminary plan and estimate shall be prepared and issued to interested parties having responsibility for construction, or knowledge of local conditions. A field examination shall be arranged at the bridge site for consideration of the proposed design. If there is a possibility of clearances being desired for small boats or for other purposes, preliminary plans shall be submitted for the approval of the County Board of Supervisors.

The clear width of all openings and the clear vertical distance between the superstructure and flood water elevation shall be sufficient for the passage, without damage to the structure, of ice floes and of the largest drift or debris which may be expected. Ordinarily a minimum 1 foot vertical clearance shall be provided above extreme high water.

2.11 Pier Spacing and Location.—Piers shall be located in such manner as to meet the above specified requirements for channel openings. They shall be located so as to afford the minimum restriction of the waterway, especially in the main stream channel. In general, piers shall be placed as nearly parallel with the direction of the stream current as is practicable, due consideration being given to the velocity and direction of current at both ordinary and highwater stages, so as to avoid such deflections of the current as might prove destructive to the foundations of the structure or to the adjacent stream banks. In case of bad ice conditions, a minimum clear opening of 35 feet should be provided in each span affected. The spacing of piers shall be such as to reduce the cost of the whole structure to a minimum consistent with the above conditions.

2.12 Combined Dams and Bridges.—Structures involving combination of a bridge and a dam shall generally be avoided. The investment for a bridge in such a structure shall not be considered unless the proposed dam will be founded on material of unquestionable stability and unless the investment in the dam will be relatively large as compared to the cost for the bridge. The life of the bridge will be dependent upon the stability and maintenance of the dam and upon the responsibility of the owners of the dam. Plans shall be subject to approval of the Engineer, and if there is any element of doubt as to the permanence of the structure or the responsibility of the owners of the dam, the bridge should be located elsewhere.

2.13 Size of Culvert Openings.—In general, culverts shall be proportioned to carry the maximum flood discharge without head. When necessary, culverts are protected against undermining by means of adequate pavements and apron or cut-off walls and adjacent embankments are protected from erosion by rip-rap or other suitable means.

SECTION 3—ROADWAY

3.1 Width of Roadway and Sidewalk.—The width of roadway shall be the clear width measured at right angles to the longitudinal center line of the bridge between the tops of curbs or wheel guard timbers, if these exist; otherwise it shall be considered as one foot less then the clear width inside to inside of the handrails, girders or trusses which protect traffic.

Upon structures having sidewalks adjacent to curbs, the clear width of sidewalk shall be measured at right angles to the curb or wheel guard timbers and from the face thereof to the extreme inside portion of the handrail. For structures having trusses, girders or parapet walls adjacent to the curbs, the width of sidewalk shall be measured from their extreme outside portions to the inside of the handrail.

Widths of bridge roadways and sidewalks, if any, shall generally conform with the following minimum requirements:

	Roadway	Sidewalks
On Secondary County or Township Roads	22′	3′
On Trunk Line Roads or Main County Roads	30′	5′
On Main Trunk Line Roads or in Towns and Cities	40′	6′
On Superhighways	Special for individual cases	
Grade Separation Subways—All Trunk Lines	40′	6′
Grade Separations—Overhead	The same as for bridges	

Other roadway widths than given above will be used frequently to meet local conditions. The minimum legal width of roadway for new bridges is 18 feet.

3.2 Bridge Clearances.—The clearance width and height shall be the clear dimensions available for the passage of vehicular traffic as shown on the clearance diagrams, figures 1 and 2. Roadway widths for vehicular traffic only shall generally be 22 feet for two way traffic and 10 feet for each lane of traffic, for three or more lanes. In no case shall a bridge be constructed with a roadway less than 18 feet wide for two lane traffic or less than 9 feet per lane for three or more lanes.

3.3 Railroad Clearances.—Clearances required for a grade separation structure over a railroad shall be obtained from the Chief Engineer of the railroad. For purposes of preliminary design and preliminary estimate, and in the absence of definite information, limiting clearances and dimensions for steam railroads shall be assumed as shown in Figure 3. Minimum construction clearances may be assumed as 19 feet vertically above top of high rail, 7½ feet horizontally from center line of track for forms or falsework, and 6 feet 9 inches horizontally from center line of track, for footing sheeting. If necessary, the footing sheeting may be used as a footing form instead of forming inside the sheeting. In all cases involving curved tracks, the lateral clearances shall be increased in amount corresponding to that required to maintain the standard clearances with allowance for superelevation of the outer rail.

3.4 Highway Grade Separation Clearances.—Minimum highway clearances and roadway dimensions for a grade separation subway shall generally conform with figure 4. In exceptional cases these clearances and dimensions will be modified to suit various conditions. In no case shall roadways of 40 feet or less be divided by a pier or other obstruction which will be a menace to traffic.

3.5 Light Posts and Pilasters.—Light posts shall not be located inside the roadway clearance lines at top of curbs. Preferably they shall be placed on pilasters or pylons. Short span structures are usually not adapted to ornamentation by lighting and if illumination is desired under such a condition, it shall, in general, be obtained by placing street lights in the vicinity at each end of the bridge, as suggested in Figure 21. The minimum distance from the projection of a pilaster nearest the curb to the face of the curb on a bridge shall be 1′6″. Similarly light posts on a bridge or on bridge approaches shall not be located closer to the face of the curb than 1′6″ in the clear.

3.6 Heights of Lights.—For effective bridge roadway illumination from boulevard lights on bridges, globes shall be located approximately 11 feet above the roadway, when supported on standards carried on pilasters, but greater heights may be used when supported on independent standards. Lights which are intended primarily for ornamentation may be otherwise located, but lighting shall generally provide adequate illumination.

3.7 Poles on Bridges.—Poles for electric power, telephone or other utilities will not be permitted on bridge roadways excepting provision for electric trolley lines on bridges carrying electric railway traffic. In that case, poles will not be permitted within the clearance lines shown on the clearance diagrams. Where provision for other utilities is necessary, suitable conduits shall be provided in a concealed location.

3.8 Overhead Wires and Overhanging Obstructions.—No overhead wires, light brackets or other overhanging obstructions shall be located within the clearance lines shown on the clearance diagrams. Electric supply lines crossing over streets and highways are required to have a minimum clearance of 22 feet under Act 288, Public Acts of 1921. Order No. 1679 of the Michigan Public Utilities Commission, effective July 1, 1926 stipulates minimum clearances crossing streets and highways, of 18 feet for communication lines and 16 feet for trolley lines.

3.9 Curbs.—The width of the curb measured from the face of the curb to the closest vertical projection of the railing pilaster, or of the super-structure, shall preferably be not less than 1′6″ as a safety zone for occasional pedestrians. In no case shall this width be less than 9″. The curb height shall preferably be not less than 10 inches and in no case less than 9 inches. Curbs shall be of substantial construction capable of resisting a transverse force of not less than 500 pounds per lineal foot of curb, applied at the top of the curb.

3.10 Railings.

3.10.1 Railing Height.—Substantial railings shall be provided along each side of the bridge for the protection of traffic. The top of railing shall be not less than 3′0″ above the top of the curb, and when on a sidewalk, not less than 3′0″ above the top of the sidewalk.

3.10.2 Classes of Railing.—In general, railings shall be of three classes, as follows:

1. Metal
2. Masonry
3. Timber

3.10.3 Design Forces.—Railings shall be designed to resist a horizontal force of not less than 150 pounds per lineal foot, applied at the top of the rail, and a vertical force of not less than 100 pounds per lineal foot.

3.10.4 Metal Railings.—Metal railings shall consist of an upper and lower horizontal rail connected by a suitable web. The clear distance between the top of curb or sidewalk and the lower rail shall not exceed 6 inches. The railing shall contain no opening of greater width than 6 inches.

All connections to posts, truss members, etc., shall contain not less than two rivets or bolts each. Ample provision shall be made for inequality in the rate of movement of the railing and the supporting super-structure, due to temperature or erection conditions.

3.10.5 Concrete Railings.—Concrete railings shall be designed with either a solid web or with a solid plinth supporting precast concrete spindles and a top railing cast in place. Openings in concrete railings shall be proportioned with due regard to the safety of persons using the structure and shall not be wider than 6 inches.

Provision for inequality in the rate of movement due to temperature of spindle railings and supporting superstructures shall consist of a heavy coat of white lead paint or equivalent at construction joints in the railings at the pilasters. Additional provision shall be made for expansion and contraction at all expansion joints in the structure and at abutting railings at ends of bridges. This provision shall preferably consist of a small clear opening between abutting railings.

3.10.6 Timber Railings.—Timber railings shall consist of three or more lines of railing timbers supported on posts not more than 8 feet center to center. The maximum clear spacing between railing timbers shall not exceed 8 inches.

3.11 Drainage.—Transverse drainage of roadways shall be secured by means of a suitable crown in the roadway surface. Longitudinal drainage shall be secured by means of scuppers or drains of ample size constructed in the gutters or curbs at suitable intervals, in no case exceeding 30 feet. The details of floor drains shall be such as to prevent the discharge of drainage water against any portion of the structure. Overhanging details in concrete and timber floors shall be provided with drip beads. Drainage from structures over railroads shall not be permitted to fall on the railroad roadbed or slopes to cause erosion, but shall be carried to the ends of the bridge and there disposed of.

3.12 Wearing Surfaces.

3.12.1 Concrete Wearing Surfaces.—Concrete wearing surfaces shall preferably consist of a separately placed concrete slab with metal fabric reinforcement at mid-depth anchored to the floor slab with wire anchors. The cross sectional area of the metal fabric shall be not less than 0.025 square inches per foot of width both longitudinally and transversely. The minimum thickness at any point in a separately placed concrete wearing surface shall be 3 inches. The wearing surface shall consist of Grade A Concrete. 1 inch class C coarse aggregate shall be used, which will conform with the requirements of Division 12, Section 4.

Monolithic concrete wearing surfaces may be used for structures on secondary roads, or for steel truss bridges on which a minimum dead load is desirable. In the latter case, the floor shall be divided by transverse joints over the floor beams, and the contact surfaces painted with red lead, and further protected from corrosion by covering with a continuous sheet of metal to be specified by the Engineer, this construction permitting repair or replacement of slabs, by panels, and to provide for opening of the joints due to deflection of the slab and stringers. For monolithic construction, the additional slab thickness shall be not less than ¾ inch and preferably not less than 1 inch. The floor and wearing surface shall be of Grade A concrete.

3.12.2 Timber Wearing Surfaces.—Timber wearing surfaces shall be used only on temporary bridges, or for maintenance of existing bridges. Plank or laminated timber floors shall conform with the requirements of Division 8, Section 11.

3.12.3 Block or Brick Wearing Surfaces.—Wearing surfaces of blocks or bricks of any kind shall be constructed on a cement mortar bed of ½ inch minimum thickness, of stiff consistency, in accordance with Division 5, Section 18, Article 4.

3.12.4 Bituminous Wearing Surfaces.—Wearing surfaces of bituminous concrete or sheet asphalt may be used on bridge floors where pavement of the approaches with similar surface courses is contemplated. In such cases, the wearing surface shall be constructed in accordance with the requirements of Division 4, Sections 9 and 10, and the depth of wearing surface shall conform as closely as possible to a minimum depth of 3 inches regardless of the thickness used in the adjacent pavement.

3.13 Camber.—Concrete bridges shall be so constructed that all longitudinal belt or shadow lines, under full dead load on the structure will not appear as having deflected. For this purpose, parabolic camber diagrams shall be shown on the plans.

Steel girder bridges with concrete fascias extending to the bottom flanges, shall have the fascias project a short distance below the flanges to conceal the girder deflection.

Steel trusses shall be cambered in accordance with the requirements of Division 5, Section 10, Article 7.

Concrete floor slabs and curbs shall be constructed so that camber will be practically eliminated from the roadway by the dead load deflection of the structure.

3.14 Crown.—Bridge roadway surfaces shall be constructed with transverse parabolic crown for drainage. The height of crown shall be not less than given by the formula $C = \frac{R^2}{533}$, where C = Crown in inches and R = Roadway width in feet. The minimum crown shall be 1 inch. Timber floors of 22 feet roadway or less, need not be crowned. Standard crowns of 1 inch, 1¾ inches and 3 inches shall, in general, be provided for roadways of 22 feet, 30 feet and 40 feet respectively.

3.15 Length of Culverts.—The length of culverts shall be sufficient to provide the full required width of roadway or width at the top of embankment. The assumed slope of the embankment shall be suitable for the particular filling material involved, and shall be such as to eliminate any tendency for the embankment slopes to slip or slide. For average conditions, the slope shall be taken as one and one half horizontal to one vertical.

SECTION 4—TYPES OF STRUCTURES AND GOVERNING SPAN LENGTHS

4.1 Choice of Type.—Before deciding upon the type of bridge to be used the conditions to be considered shall include the following:

4.1.1 Traffic Conditions and Future Widening.—Under light traffic with little possibility of much development, traffic conditions will not be a determining factor. Where traffic is heavy or has possibility of becoming heavy, either from natural development, or from probable rerouting of important highways, provision shall, in general, be made for widening bridges with minimum loss of original investment. In such cases adoption of a steel deck type of structure is dictated, and substructures shall either be constructed of probable ultimate width, or shall be adaptable to future extension. The wing walls shall preferably consist of extended abutment walls with a sloping upper part, removable for widening of the superstructure. Such provision shall be preferred even though an excess in the overall length of the superstructure for the narrow roadway is required, so that the backfill will be properly retained when the future ultimate roadway is built. Superstructures shall be adaptable to widening by removal of curbs and railings and installation of additional steel girders, new railings and addition of widened floors and wearing surfaces. Deck bridges of 30 foot roadway, or less, with sidewalks shall be constructed so the roadway can be widened by removal of the sidewalk slabs.

4.1.2 Foundation Conditions.—Where foundations are of very unstable material, a light type of superstructure is preferable and substructures must be designed to give minimum foundation pressures and great resistance to thrust. For economy, when foundations are expensive, long spans with a minimum number of substructure units shall be used. Movement from thrust can often be prevented by the use of skeleton or spill-thru abutments, and for short spans abutment struts will often prove effective. Unyielding foundations, such as rock, gravel or hard pan in thick beds, are favorable for the construction of arches or multiple spans designed as continuous girders. Arches shall not be founded on piles or on any compressible material which might permit vertical or horizontal movement of abutments. Unfavorable foundation conditions do not favor the construction of bridges of heavy types requiring considerable falsework. Steel deck girder structures require no falsework.

4.1.3 Stream Conditions.—Streams which are subject to great fluctuation of water level and flooding, will usually require a minimum number of spans for substructure economy and least stream obstruction, but if high water approaches the desired roadway elevation, spans are preferred of a type with minimum depth from roadway to underclearance line. Within their roadway limitations, through concrete girders or half through plate girders are suitable for the latter condition. Short spans of concrete slabs, concrete T beams or steel deck girders also have quite shallow decks. Deck concrete arches cause considerable obstruction to stream flow under flood conditions as the water rises toward the crown and shall not be used where flood waterway would thus be restricted; but where appearance is an important consideration, cantilevers can often be used for such cases. Cantilever construction shall, in general, be used in preference to multiple arch spans with high springing elevations. Short multiple spans shall be avoided in streams which carry logs, drift or heavy ice. The minimum clear span for such conditions shall preferably be 35 feet.

4.1.4 Topographic Conditions.—Shallow stream channels in flat country require structures with minimum depth of deck or underclearance; otherwise the elevation of roadway will be raised, adding to the cost of approaches and interfering with the continuity of the road profile. Conversely, where considerable clearance is available above high water, deck types of structure are preferable. Single spans with shallow abutments are generally suitable for deep channels with steep banks.

4.1.5 Economy.

4.1.5.1 Economy of Short Spans.—Economic considerations generally favor short multiple spans, but the type of structure and span lengths shall be chosen so that the cost of substructure is not proportionately excessive.

4.1.5.2 Economy of Material.—Superstructure economy depends upon efficient design of the structural parts to minimize total dead load. Various types of structures have rather well defined limits of span for economy, which vary slightly with different conditions. Floor systems shall be light as possible consistent with permanence and cost. For efficiency, members and parts shall be designed so that little material is unstressed or slightly stressed. Curved girders and curved chord trusses are efficient forms which are economical if the increased cost of fabrication or construction is less than the saving in materials.

4.1.5.3 Concrete versus Steel.—Reinforced concrete superstructures are economical for maintenance, but in girder spans, rolled or fabricated steel eliminates expensive construction falsework.

4.1.5.4 Foundation Requirements.—The greatest possibility for economy in bridge design is in the foundations. Reliable borings are necessary to estimate safe bearing capacity. On rock or highly stable material, advantage shall be taken of the higher allowable footing loads. Mass abutments shall be avoided, except for low heights, and the proportion of the total footing load due to the weight of the abutment and backfill shall not be excessive. High abutments with unsatisfactory foundations can be avoided economically in some cases by lengthening the superstructure to extend to higher banks. Skeleton substructure types are effective in reducing costs. For abutments, stability is assured by sloping earth fills in front of the abutments. For high abutments, increase in span length will be necessary to maintain waterway and the saving in cost of substructure may be absorbed in the superstructure.

4.1.5.5 Local Materials.—The availability of local construction materials and accessibility of the site are factors to be considered for economy. In remote locations with light traffic, local materials such as timber and rock can be used to advantage. Local supply of suitable gravel is favorable to concrete construction. For long haul over poor roads, structural steel will give the least tonnage, but maximum weight and total length per piece must be restricted.

4.1.6 Aesthetics.—Due consideration shall be given to ornamentation and appearance of all structures having regard to the environs and the suitability of a pleasing type of design. Properly proportioned arches are very effective, where they can be used. Other curved forms, such as curved girders or cantilevers, can also be used with pleasing results. The appearance of all deck type structures can be greatly enhanced by ornamental concrete spindle railings. Exposed steel girders can be concealed by encasement in concrete with suitable outside fascia forms.

4.2 Limitations of Types for Span and Roadway.—Preferably, span lengths and roadway widths for various types of bridges shall be within the following limitations.

4.2.1 Steel Structures.	Span	Roadway
Rolled Beams	30′ to 60′	No limit
Plate girders—Half through	65′ to 110′	Up to 30′ plus sidewalks
Plate girders—Deck	65′ to 110′	No limit
Low riveted trusses	75′ to 120′	Up to 30′ plus sidewalks
Through riveted trusses	120′ to 300′	Up to 40′ plus sidewalks
Riveted or pin-connected trusses	Above 300′	Up to 60′ plus sidewalks
Steel Arches, cantilever trusses or suspension bridges	Long Spans	Up to 60′ plus sidewalks

4.2.2 Concrete and Masonry Structures.

	Span	Roadway
Slab spans	Up to 24′	No limit
T beam or deck girder spans	22′ to 50′	No limit
Simple girder spans—Half through	35′ to 90′	Up to 22′ plus sidewalks
Continuous girder spans—Half through	40′ to 75′	Up to 22′ plus sidewalks
Concrete cantilevers—Deck	40′ to 75′	Up to 60′ plus sidewalks
Through concrete arches	100′ to 150′	Up to 22′ plus sidewalks
Deck concrete arches—Spandrel filled	25′ to 100′	Up to 60′ plus sidewalks
Deck concrete arches—Open spandrel	75′ to 200′	No limit

4.3 Grade Separations.

4.3.1 Choice of Type.—Grade separations ordinarily involve a choice of type between the subway and overhead highway crossing. The choice shall be dependent primarily upon relative economy and utility, which are subject to influence by topographic conditions, prospective property damages, drainage conditions and structural conditions.

4.3.2 The Subway.—The following conditions favor the subway for grade separation:—The subway involves least earth work, where the topography is flat, the vertical clearance for highway being less than two-thirds the vertical clearance for railroad. It is generally more sightly and does not greatly alter the landscape. Resulting property damages are usually least, particularly in built-up urban territory, where highway overhead structures are particularly objectionable. The subway involves least rise and fall of highway grade and saving to the public in cost of motor vehicle operation. Subway construction usually involves heavy expense for maintenance of railroad traffic. In some instances it involves heavier and more expensive structures, but the heavier loading of railroad traffic compared to highway traffic may be offset by greater width required for an overhead highway structure. In general, the subway shall be given preference over the overhead type, even at somewhat greater expense.

4.3.3 The Overhead Highway.—The overhead highway crossing is favored where gravity roadway drainage is impossible. Installation, maintenance and operation of automatic pumping equipment is costly even where electric power is available. Highway overhead structures shall preferably consist of concrete deck girders or encased steel deck girders. Under ordinary conditions, approach spans are required to reduce the cost of the abutments and the total cost of the structure. Provision is also frequently required for possible future railroad tracks. Spans of more than about 45 feet will usually require more head room for deck concrete girders than for encased steel girders. If the clearance over tracks is less than 20 feet, or if otherwise required, on account of heavy trains on adverse grades, blast plates shall be provided as specified in Division 8, Section 3.

4.3.4 Railroad Structures.—Railroad structures, which span subways, will usually consist of deck plate girders with a concrete slab or steel plate ballasted deck. The girders may be encased or unencased. Half through plate girders are occasionally used, but are objectionable for railroad side clearance for multiple tracks.

SECTION 5—LOADS

5.1 Proportioning of Structural Parts.—Unless otherwise provided, the component parts of a structure shall be proportioned for the stresses produced by the following loads, with load distribution as herein specified, and with maximum unit stresses not exceeding the allowable unit stresses specified in Section 6.

5.2 Stress Sheets.—For all superstructures and structural parts, the stresses due to dead load, live load, impact and other loads, if any, shall be shown upon the drawings as stress sheets or stress tabulations. Loading diagrams or tabulations for all loadings shall also be shown.

5.3 Dead Load.—The dead load shall consist of the weight of the structure complete, including the weight of the roadway floor, car tracks, conduits, cables, or other utility services supported thereby. For analysis of existing structures, the actual weights of materials shall be computed as closely as possible. The following weights are to be used in computing the dead load:

SUBSTANCE	Weight Per Cu. Ft. Lbs.
Steel	490
Iron, cast	450
Bronze	524
Timber, dry, seasoned	
Oak	60
Long leaf yellow pine	44
Douglas fir	32
Concrete, Portland Cement	
Plain	144
Reinforced	150
Loose sand and earth	100
Compact sand or gravel	120
Macadam or gravel, rolled	140
Cinder filling	60
Asphalt wearing surface	125
Granite block paving	160
Vitrified brick paving	150

5.4 Allowance for Future Wearing Surface.—In the case of structures having concrete slab floors without wearing surfaces, an adequate allowance shall be made in the design dead load to provide for the weight of a suitable future wearing surface. This allowance will depend upon the type of wearing surface contemplated; it shall be in addition to the weight of any monolithically placed concrete wearing surface; and it shall be not less than 30 pounds per square foot of roadway.

5.5 Live Load.

5.5.1 Highway Loadings.—All structures and component parts shall be designed for maximum stresses based on the passage of a train of trucks on each traffic lane. The truck dimensions and concentrations assumed shall be those of the typical or standard truck shown in Figure 5.

5.5.1.1 For Floor Systems.—The integral parts of bridge floor systems and their direct connections, and integral members or parts of trusses and girders subjected to the direct action of floor loads shall be designed for the standard truck concentrations, or their equivalent, as hereinafter defined. For this purpose, the standard trucks shall be one of the following:

Standard	20 ton trucks
Standard	15 ton trucks
Standard	12½ ton trucks
Standard	10 ton trucks

Unless otherwise specified, all bridges shall be designed for standard 15 ton trucks.

For the purpose of design, the application and distribution of specification loads shall be assumed as hereinafter specified in this section.

5.5.1.2 For Trusses, Girders and Arches.—Deck trusses, girders and arches, and through trusses or girders, including bearings and substructures, shall be designed for the stresses produced by a load on each traffic lane composed of a uniform load per linear foot of lane with a concentrated load which shall be considered as the approximate practical equivalent of the truck trains referred to in Section 5, and which shall be considered as so located longitudinally as to produce maximum stresses. These loads shall be subject to reduction as hereinafter specified for the improbability of full load on multiple lane bridges. The concentrated load shall be considered as uniformly distributed transversely on a line having a length equal to the width of the lane. The standard truck clearance width of 9 feet shall be assumed as constituting the width of one traffic lane. These assumed equivalent loadings shall be classified as follows:

Equivalent Loading H-20. A total load on each traffic lane composed of a uniform load of 600 lbs. per linear foot and a single concentrated load of 28,000 lbs.

Equivalent Loading H-15. A total load on each traffic lane composed of a uniform load of 450 lbs. per linear foot and a single concentrated load of 21,000 lbs.

Equivalent Loading H-12½. A total load on each traffic lane composed of a uniform load of 375 lbs. per linear foot and a single concentrated load of 17,500 lbs.

Equivalent Loading H-10. A total load on each traffic lane composed of a uniform load of 300 lbs. per linear foot and a single concentrated load of 14,000 lbs.

Unless otherwise specified all bridges shall be designed for class H-15 loading which is approximately equivalent to the typical truck loadings shown in Figure 6, which is considered as the approximate practical equivalent of one 15 ton truck followed by, or both followed and preceded by a line of 11¼ ton trucks of indefinite length spaced as shown in Figure 6 and assumed to occupy a clearance or lane width of 9 feet. Loading H-20 is 8/6, loading H-12½ is 5/6 and loading H-10 is 4/6 of loading H-15, or the loadings of H-20, H-15, H-12½ and H-10 have relative values of 8, 6, 5 and 4 respectively.

5.5.1.3 Sidewalk Loads.—All sidewalk stringers, brackets and slabs shall be designed to support a live load of not less than 100 pounds per square foot of sidewalk area.

Girders, trusses or arches supporting sidewalks shall be designed to support a sidewalk live load as determined by the following formula, provided that in no case shall the live load be less than 20 pounds per square foot of sidewalk area:

$$P=(80-\frac{L}{8})\ (1-\frac{W}{40})$$

where P = live load in pounds per square foot of sidewalk area
L = loaded length of sidewalk in feet
W = clear width of sidewalk in feet

No impact increment shall be added to sidewalk load.

In general, provision shall be made by suitable height of curb, or otherwise, to prevent encroachment of roadway loads upon the sidewalk area. If the details of the structure permit such encroachment, the sidewalks shall be designed for the roadway loads and impacts so involved.

5.5.1.4 Railings.—Railings shall be designed to resist a horizontal force of not less than 150 pounds per linear foot, applied at the top of the rail, and a vertical force of not less than 100 pounds per linear foot.

5.5.1.5 Curbs.—Curbs shall be designed to resist a lateral force of not less than 500 pounds per linear foot of curb, applied at the top of curb.

5.5.2 Electric Railway Loads.—When highway bridges support electric railway traffic in addition to highway traffic, the railway loading shall be selected with due regard to the class of traffic which may be expected to operate over the railway lines. Special consideration shall be given to the possibility that the freight rolling stock of steam railroads may be operated. The loading used shall not be less than the requirements of the railway company operating over the bridge, unless the railway company's loadings are limited by franchise or agreement. When not otherwise specified, and in the absence of more exact data, the electric railway loading on each track shall be a train of two electric cars followed by, or preceded by, or both followed and preceded by, a uniform load, as shown on Figure 7. The railway loading used shall be shown on the sheets. Highway bridges supporting electric railway traffic shall be designed for the following loading conditions:

1. The highway loads upon any portion of the roadway area including that portion occupied by the railway.

2. The electric railway loads on the car tracks and the highway loads on the remaining traffic lanes, the street railway loading being assumed to occupy a lane width of 10 feet, or in case of double track, a width of 10 feet plus the track spacing for the two railway lanes.

5.6 Analysis of Existing Structures.—For the purpose of analysis of existing structures, the legal loading specified by current statutes, in general, corresponds very closely with, and may be considered as the equivalent of, H-12½ Loading, and the net live load stress capacity of any member or part of such a structure shall be expressed as a percentage of the stress in such members due to H-12½ loading, and shall be further listed as the proportional H loading, to the second decimal point. For example, if 80%, it would be considered 0.80x12.5 = H 10.00.

When it is desired to post an existing structure for its limiting capacity, the posting shall specify the maximum axle loading corresponding to the proportional H loading. For example, if H 9.62 post for $\frac{9.62}{15}\times 24{,}000\# = 15{,}392\#$ but not to exceed the legal maximum axle load, or in general multiply the H loading by 1600.

5.7 Comparative and Legal Loadings.—In general, for the purpose of comparing loadings from various types of vehicles with varying axle spacings, number of axles, and variation in axle concentrations, the maximum axle concentration which will produce stresses for any span equal to those resulting from H 15 loading may be secured from Figure 12 and the equivalent H loading for any given piece of equipment may be considered as H 15 times the ratio of the actual maximum axle load to the axle concentration secured from Figure 12, and the ratio of this H loading to H 12½ will indicate approximately the percentage of overload or margin of safety over the legal loading. The ratio of the maximum axle secured from Figure 12 to the maximum wheel loads permitted by the statutes will indicate exactly the amount of overload or margin of safety.

5.8 Impact.—All live load stresses, except those due to sidewalk loads and centrifugal, tractive, wind and lateral forces shall be increased by an allowance for dynamic, vibratory and impact effects.

For end floor beams, floor beam hangers, column supporting floor beam concentrations and all floor beam and stringer connections, the impact allowance shall be 60% of the live load stress.

For all other portions of structures, the impact allowance or increment is expressed as a coefficient of the live load stress, varying with the loaded length of the structures and the width of the roadway area. Its intensity is determined by the following formulas in which

I = impact coefficient

L = loaded length of span in feet producing the maximum static stress in the member considered.

For Electric Railway Loads:

$$I = \frac{L + 900}{12L + 1200}$$

For Highway Loads:

$$I = \frac{L + 250}{10L + 500}$$

For highway loads, the maximum value of "I", as given by the above formulas, shall not exceed 0.30. Curves showing values of I are shown in Figure 15.

5.9 Longitudinal Force.—Upon bridges carrying electric railway traffic, the forces due to traction and momentum shall be considered as longitudinal forces having a magnitude equal in amount to 20 per cent of the moving railway load on one track only. Provision shall be made in the design of floor members, girders, trusses, lateral systems, trestle towers, etc., for these forces applied at the elevation of the tops of rails. Bridges constructed on a grade shall be designed to avoid longitudinal forces to the greatest extent possible and when unavoidable, the structure shall be designed to withstand these forces.

5.10 Lateral Force.—The force due to wind and lateral vibrations shall consist of a horizontal moving load equal to 30 pounds per square foot on the side area of any exposed floor construction, the side area of all railings, and 1½ times the side area of each truss, girder or arch. In addition to the foregoing, a moving load of 150 pounds per linear foot shall be considered as acting in the plane of the loaded chord on highway bridges and 300 pounds per linear feet upon bridges for combined highway and electric railway service. However, in the case of structures having a reinforced concrete floor slab effectively anchored to the supporting structure, this additional loaded chord load need not be considered in designing the lateral system, but must be considered in designing shoes.

The transverse bracing and anchorage of trestle towers and bents shall be proportioned to resist the above specified lateral forces acting on the superimposed spans, plus a pressure of 30 pounds per square foot on 1½ times the side areas of all columns, struts, and bracing, forming the viaduct.

The longitudinal bracing of trestle towers shall be proportioned to resist not less than 0.7 of the transverse lateral forces above described, in addition to the tractive force due to electric railway traffic, when the latter exists.

5.11 Centrifugal Forces.—Bridges carrying electric railway traffic on a curve shall be designed to resist a lateral force equal to 10 per cent of the moving railway load, without impact allowance. This load shall be assumed to act 4 feet above the rail.

5.12 Forces of Stream.—All piers and other portions of structures which are subjected to the force of flowing water together with floating ice, or drift shall be designed to resist an assumed force of 10,000 pounds per foot of width subject to such action. This force shall be assumed to act at high water elevation, and the direction of the line of action shall be assumed as parallel to the channel. A force of 500 pounds per foot length of piers shall also be assumed to act at highwater elevation at right angles to the axis of a pier.

5.13 Pressure from Retained Material.—Structures designed to retain fills shall be proportioned to withstand pressures as given by the following formulas. All designs shall provide for the thorough drainage of back filling materials wherever possible, and when impossible to do so, the backfill material shall consist of a wall of granular material capable of standing safely at 1½ to 1 slope when thoroughly wetted. In general, in the absence of definite data regarding the filling material, the angle of repose shall be assumed as 30 degrees. The line of action and location of the resultant earth pressure shall be assumed as shown in Figures 8 to 11 inclusive, for various cases. In the following formulas:

ϕ = Angle of repose of the filling.
δ = Angle of surcharge between a horizontal line and the surface of the filling.
h = Vertical height of the wall in feet.
h_1 = Height of surcharge in feet, surcharge assumed to cover at least entire width of supported prism.
h_2 = Height of equivalent live load surcharge in feet.
k = Ratio of the height of a uniform dead load surcharge, in feet, equivalent to a sloping surcharge, to the height of the wall.
P = Resultant earth pressure per foot length of wall.
w = Weight of the filling per cubic foot.
x = Encroachment of live load surcharge on supported earth prism.—See Figure 9.
y = Height from the base of the wall to the point of application of the resultant earth pressure.

5.13.1 Level fill to top of wall.—See Figure 8.

$$P = \tfrac{1}{2}wh^2 \frac{1-\sin\phi}{1+\sin\phi}$$
$$= 16.7\ h^2 \text{ for } \phi = 30^\circ \text{ and } w = 100$$
$$y = \frac{h}{3}$$

5.13.2 Level fill with a uniform surcharge loading, or a uniform live load surcharge, or both.—See Figure 9.

5.13.2.1 For a uniform surcharge loading.—$h_2 = 0$ and

$$P = \tfrac{1}{2}\ wh\ (h+2h_1)\ \frac{1-\sin\phi}{1+\sin\phi}$$
$$= 16.7\ h\ (h+2h_1) \text{ for } \phi = 30^\circ \text{ and } w = 100$$
$$y = \frac{h}{3}\ \frac{h+3h_1}{h+2h_1}$$

5.13.2.2 For a uniform live load surcharge, $h_1 = 0$

and

$$P = \tfrac{1}{2}\ wh\ (h+\frac{2xh_2}{h})\ \frac{1-\sin\phi}{1+\sin\phi}$$
$$= 16.7\ h\ (h+\frac{2xh_2}{h}) \text{ for } \phi = 30^\circ \text{ and } w = 100$$
$$y = \frac{h}{3}\ \frac{(h^2+3xh_2)}{h^2+2xh_2}$$

5.13.2.3 For a uniform surcharge loading with a superimposed uniform live load surcharge.

$$P = \tfrac{1}{2}\ wh\ (h+2h_1+\frac{2xh_2}{h})\ \frac{1-\sin\phi}{1+\sin\phi}$$
$$= 16.7\ h\ (h+2h_1+\frac{2xh_2}{h}) \text{ for } \phi = 30^\circ \text{ and } w = 100$$
$$y = \frac{h}{3}\ \frac{(h^2+3hh_1+3xh_2)}{(h^2+2hh_1+2xh_2)}$$

5.13.3 Sloping surcharge.—See Figure 10.

$$P \doteq \tfrac{1}{2}\ wh\ (h+2kh)\ \frac{1-\sin\phi}{1+\sin\phi}$$
$$k = [\cos\tfrac{3}{4}\ \delta \frac{\cos\tfrac{3}{4}\delta - \sqrt{\cos^2\tfrac{3}{4}\delta - \cos^2\phi}}{\cos\tfrac{3}{4}\delta + \sqrt{\cos^2\tfrac{3}{4}\delta - \cos^2\phi}} + \frac{2(1-\sin\phi)}{1+\sin\phi}] - \tfrac{1}{2}$$

For $\delta = \phi = 30^\circ$ and $w = 100$
$k = .175$
$P = 22.5h^2$
$y = .376h$

5.13.4 Sloping surcharge with superimposed live load.—See Figure 11.

$$P = \tfrac{1}{2}\ wh\ (h+2kh+2x\frac{h_2}{h})\ \frac{1-\sin\phi}{1+\sin\phi}$$

For $\delta = \phi = 30^\circ$ and $w = 100$

$$P = 16.7h\ (1.35h+2x\frac{h_2}{h})$$
$$y = \frac{h}{3}\ \frac{(1.525h^2+3xh_2)}{(1.35h^2\ +2xh_2)}$$

5.14 Thermal Forces.—In all statically determinate structures, provision shall be made for expansion and contraction, or for the forces required to restrain such action, due to variation of temperature, as follows:

5.14.1 For Metal or Partially Encased Structures.—An extreme range of temperature of 150 degrees Fahrenheit shall be assumed, with a corresponding movement of approximately 1/8 inch in 10 feet.

5.14.2 For Reinforced Concrete or Completely Encased Structures.—The range of temperature in the structure shall be assumed 75 degrees Fahrenheit, with a corresponding movement of approximately 1/16 inch in 10 feet.

5.14.3 In all Statically Indeterminate Structures, and for Erection Conditions of all Structures.—Provision shall be made for stresses resulting from the following variations in temperature:

For Metal Structures.—From -20 degrees Fahr. to $+120$ degrees Fahr. The rise and fall in temperature shall be figured from an assumed mean temperature at the time field connections are completed or operations performed.

For Concrete Structures.—A temperature rise of 30 degrees and a temperature drop of 45 degrees from an assumed mean temperature of the structure at the time of final cast, shall be considered.

5.14.4 Rigidly Restrained Structures.—Structures, or structural parts, which are rigidly restrained from any expansion, shall be designed for the following combination temperature and shrinkage stresses, in pounds per square inch:

For Steel, 180R
For Concrete 18R
Where R = assumed change in temperature in the material in degrees Fahrenheit.

Concrete temperature stress may be assumed as 9R if precautions are taken to minimize shrinkage by keeping the concrete thoroughly dampened during seasoning, for a period of two weeks, or by suitable construction joints.

SECTION 6—APPLICATION AND DISTRIBUTION OF LOADS

6.1 Application of Loads to Girders, Trusses and Arches.—Trusses, girders and arches shall be designed to support as many traffic lane loads as the width of roadway will permit, assuming them to be so placed as to produce maximum concentrations of loading on the member. The assumed lane loading shall be considered as applied over a width of 9 feet. A series of such lane loads contiguous to each other, depending in number upon the width of the bridge, shall be so arranged as to produce the maximum concentration on any line of supporting longitudinal members, considering the floor system as transmitting the load to the member by simple beam action, except in those cases where stringer concentrations, as hereafter specified, may govern. For this purpose, the lane loading near the curb shall be considered as extending 1′6″ outside of the face of the curb.

6.1.1 Cantilevered Decks.—In case of cantilevered loads, the transverse floor system shall be designed and considered as a continuous cantilever extending over the outside line of longitudinal members to the first intermediate line, which shall be considered as receiving the negative reaction from the cantilevered load, and the outside line of longitudinal members under the cantilevered load shall be designed for the maximum loading which can be brought upon it, whether the cantilevered loading be roadway or sidewalk or both, and whether the loading be live or dead In the design of the first interior line of longitudinal members in addition to designing for the maximum positive flexure, the effect of the dead and live loading from the cantilevered members on the first interior line of longitudinal girders shall be considered, and the floor system shall be designed to transmit the negative reaction from the cantilevered loading to the first interior line of longitudinal girders.

6.1.2 Probability Factor.—Whenever a single longitudinal line of members receives a live loading which exceeds a single lane loading, the maximum live load on the member shall be multiplied by a factor C, as follows, for improbability of maximum loading on all contiguous lanes:

$$C = 6\frac{-N}{5}$$

Where N is the live load on the member in terms of single lane loading.

6.1.3 Application of Loads to Filled Spandrel Arches.—For Filled Spandrel arches, the live load per unit width of arch shall be computed on the assumption that all traffic lanes are loaded, and shall be considered as equal to the specified load per lane multiplied by the number of lanes and divided by the width of the arch ring, outside to outside. When eccentric loading will produce greater intensity of arch stresses, the distribution over the arch ring as above described shall be corrected to provide that the resultant distribution shall fall vertically under the resultant eccentric load on the assumption of lineal variation in distribution across the arch ring. These loads shall, however, be increased in designing the arch face to sustain the resultant maximum toe pressures from the spandrel walls.

6.2 Application of Loads to Floor Beams.—Floor beams, with or without stringers, and their connections or supports, shall be designed for a load per traffic lane, as given for the single equivalent concentrated load in Figure 12, and as many adjacent traffic lanes of 9 feet each as the width of roadway will permit, shall be considered as loaded. The equivalent single concentrated load per traffic lane, from Figure 12, shall be taken for a span "L" equal to the sums of the adjacent panel lengths. For end floor beams, "L" shall be taken as the length from the roadway expansion break, or from the end of the floor, to the next intermediate floor beam. For floor beams without stringers, the load per traffic lane shall be assumed as distributed uniformly over the width of the traffic lane. For floor beams with stringers, the load per traffic lane shall be assumed to be distributed on the assumption that the floor slab distributes the load to the stringers as a simple or cantilever slab.

6.2.1 Floor Beams without Stringers.—When longitudinal stringers are omitted and the floor is supported directly on the floorbeams the equivalent concentrated load per traffic lane, distributed transversely over the lane width, as specified in the preceding paragraph, shall also be considered as distributed longitudinally for bending moments in the floor beam. No longitudinal distribution shall be made for shear. The longitudinal distribution for bending moment shall be determined as follows:

Let M = total maximum bending moment due to the equivalent concentrated load for a span equal to twice the beam spacing.
S = longitudinal spacing of floor beams in feet.
M_1 = bending moment in one floor beam.

$M_1 = \frac{MS}{4}$ for longitudinal plank floors.

$M_1 = \frac{MS}{4.5}$ for longitudinal laminated timber floors 4 inches in thickness, or for wood or asphalt blocks on a 4 inch plank subfloor.

$M_1 = \frac{MS}{5.5}$ for longitudinal laminated timber floor 6 inches or more in thickness.

$M_1 = \frac{MS}{6}$ for reinforced concrete floors.

The value of "M_1", as given by the above equations, shall be considered as "M" when the calculated values exceed the maximum moment "M".

6.2.2 Reduction of Floor Beam Live Load Stresses for Multiple Lanes.—In the design of floor beams and their supports the following percentages of the resultant live load stresses shall be used for various loading conditions.

One or two traffic lanes.......... 100 per cent
Three traffic lanes.......... 90 per cent
Four traffic lanes.......... 80 per cent

6.3 Classification of Bridge Floors.—Bridge floors shall, in general, be classified in accordance with the material, surfacing and structural arrangement as follows:

CLASS
A. Single transverse plank.
B. Longitudinal plank, or strips, on transverse plank.
C. Laminated plank without wearing surface.
D. Laminated plank with non-rigid wearing surface.
E. Laminated plank with rigid wearing surface.
F. Single diagonal plank.
G. Double diagonal plank.
H. Concrete slab, transverse, without wearing surface.
I. Concrete slab, transverse, with non-rigid wearing surface.
J. Concrete slab, transverse, with rigid wearing surface.
K. Concrete jack arch, with tie-rods.
L. Concrete jack arch, without tie-rods.
M. Concrete slab, longitudinal, without wearing surface.
N. Concrete slab, longitudinal, with non-rigid wearing surface.
O. Concrete slab, longitudinal, with rigid wearing surface.

6.4 Distribution of Loads to Stringers or Longitudinal Deck Girders.—In determining bending moments in deck girders or stringers, the effect of the stiffness and strength of the floor on the transverse distribution of the load shall be considered. For this purpose, the equivalent single concentrated load per traffic lane, as given in Figure 12, shall be assumed without longitudinal distribution. The lateral distribution may be determined by the following method:—

Let M = the bending moment produced by one lane traffic.
S = the spacing of stringers or girders, in feet.
M_1 = the bending moment in one interior beam when the floor system is designed for a single lane of traffic.
M_2 = the bending moment in one interior beam when the floor system is designed for two or more lanes of traffic.
$M_2 = 1.2\ M_1$ for the type of floor involved.

When the stringer spacing is such that "M_1" or "M_2", as the case may be, exceeds "M", the stringer loads shall be determined on the assumption that the flooring between stringers acts as simple beams, as under Article 6.1 "Application of Loads to Girders, Trusses and Arches."

Distribution to Stringers:—

Floor Class	M_1	M_2
A	$\frac{MS}{8}$	$\frac{MS}{6.67}$
B	$\frac{MS}{8.5}$	$\frac{MS}{7.08}$
C	$\frac{MS}{9}$	$\frac{MS}{7.5}$
D	$\frac{MS}{9.3}$	$\frac{MS}{7.75}$
E	$\frac{MS}{9.6}$	$\frac{MS}{8}$
F	$\frac{MS}{7.5}$	$\frac{MS}{6.25}$
G	$\frac{MS}{8}$	$\frac{MS}{6.67}$
H	$\frac{MS}{12}$	$\frac{MS}{10}$
I	$\frac{MS}{12.5}$	$\frac{MS}{10.4}$
J	$\frac{MS}{13}$	$\frac{MS}{10.83}$

Distribution to Stringers.—(*Con.*)

K	$\frac{MS}{12}$	$\frac{MS}{10.0}$
L	$\frac{MS}{8}$	$\frac{MS}{6.67}$

6.4.1 Total Capacity of Stringers.—The combined load capacity of the stringers in a panel shall not be less than the total live and dead load in the panel.

6.4.2 Outside Stringers.—The live load supported by the outside stringers shall be determined on the assumptions specified under Article 6.1.

6.4.3 Shear in Stringers.—In calculating end shears and end reactions in longitudinal girders or stringers, the shears, as determined from the preceding distribution of loading to stringers, shall be increased by 50% to provide for rigidity and restriction of distribution at the supports.

6.5 Distribution of Loads to Floors.

6.5.1 Nomenclature:—

P = Rear wheel load in pounds = 12,000 for Standard 15-ton truck.
b = Width of rear wheel tread in inches = 15″ for Standard 15-ton truck.
f = Allowable fibre stress in pounds per square inch.
t = Net thickness in inches.
a = Width of base plank in inches.
L = Net span in inches at right angles to stringers.
c = Type coefficient.
W = Load capacity in pounds per square foot on concrete jack arch floors.
W_0 = Permissible load in pounds per square foot on concrete slabs from Figures 13 and 14.
R = Rise of jack arch.

6.5.2 Distribution of Loads in Timber Floors.—Classes A, B, C, D, E, F, G.—Bending moments from concentrated wheel loads shall be considered as distributed as follows, where M = Maximum bending moment in floor in inch pounds per foot width of section:

Class of Floor	M
A	$\frac{3P\,(L-b)}{a}$
B	$\frac{P\,(L-b)}{a}$
C	$\frac{3P\,(L-b)}{4T}$
D	$\frac{3P\,(L-b)}{6T}$
E	$\frac{3P\,(L-b)}{8T}$
F	$\frac{3P\,(1.4L-0.7b)}{a}$
G	$\frac{P\,(1.4L-0.7b)}{a}$

6.5.3 Distribution of Loads in Reinforced Concrete Slab Floors.—Moments and shears shall be computed for uniformly distributed loads as given in Figures 13 and 14 for various spans. These loads are assumed as equivalent to the standard 15-ton truck loading. For other truck loadings, proportionate values shall be used for the given uniformly distributed loads. In computation for vertical shear, the maximum shear shall be assumed as equal to the end reaction from the uniformly distributed load but shall be assumed as increased by 50% to allow for less favorable distribution than for moments.

6.5.3.1 Concrete Floor Slabs with Main Reinforcement Parallel to the Direction of Traffic.—Fo uniformly distributed loads equivalent to standard 15-ton truck loading, refer to Figure 13.

6.5.3.2 Concrete Floor Slabs with Main Reinforcement Transverse to the Direction of Traffic.—For uniformly distributed loads equivalent to standard 15-ton truck loading, refer to Figure 14. This shall also apply to floors of types K and L.

6.5.3.3 Wearing Surface Coefficients.—The following coefficients "C" shall be applied to the moments and shears, determined as above, for the assumed effect of various wearing surfaces upon distribution of loading and intensity of impact, but the resulting live load stresses shall be increased by an impact allowance, as specified in Article 5.8. In each case, the values as obtained above shall be divided by the following coefficients:—

Type	C
H. K. L. & M.	1.0
I. & N.	1.05
J. & O.	1.15

6.5.4 **Distribution of Loads in Jack-Arch Concrete Floors.—Types K and L.**—Jack arch floors shall not be constructed on new bridges. For analysis or review of existing jack arch floors, wheel load capacity shall be assumed as follows:

Type K $\quad P = \dfrac{1152 \text{ R. f. t.}}{W_0 L^2} \times 12000$, or $W = \dfrac{1152 \text{ R. f. t.}}{L^2}$

Type L $\quad P = \dfrac{384 \text{ R. f.t.}}{W_0 L^2} \times 12000$, or $W = \dfrac{384 \text{ R. f.t.}}{L^2}$

6.6 **Distribution of Loads in Flat Slabs Supported on Four Sides.**—In the case of flat slabs supported along four edges and reinforced in both directions, the proportion of the load carried by the short span of the slab shall be assumed as given by the following equations:

Load Uniformly Distributed. $\quad p = \dfrac{b^4}{a^4 + b^4}$

Load Concentrated at Center. $\quad p = \dfrac{b^3}{a^3 + b^3}$

where p = proportion of load carried by short span.
a = length of short span of slab.
b = length of long span of slab.

When the length of the slab exceeds 1½ times its width, the entire load shall be assumed to be carried by the transverse reinforcement. The uniform load per square foot for design shall be selected for the shorter span. In placing the reinforcement in such slabs, consideration shall be given to the fact that the bending moment is greater near the center of the slab than near the edges. Also, in the design of the supporting beams consideration shall be given the fact that the loads delivered to the supporting beams are not uniformly distributed along the beams. For such cases the minimum moment per unit of slab width shall be assumed as 50% of the maximum and the beam distribution shall be assumed as triangular and symmetrical about the center line of beam.

6.7 **Distribution of Electric Railway Wheel Loads.**—Electric railway wheel loads may be assumed to be uniformly distributed longitudinally over 3 ties or a minimum of 3 feet. In the case of ballasted floors, or concrete encased ties and rails, a lateral distribution of 10 feet for an axle load may be assumed, provided ties are not less than 8 feet length.

6.8 **Distribution of Load Through Earth Fills.**—When live load is transmitted through a fill, the concentrated loads per traffic lane, as described in Section 5, shall be considered as uniformly distributed laterally over a width equal to the number of 9 foot lanes plus 1¾ times the depth of the fill, but not over a greater width than the width of the supporting structure. Longitudinally, distribution shall be assumed over a length equal to 1¾ times the depth of the fill.

6.9 **Distribution of Dead Load of Fills to Culverts, Short Span Slabs and Footings.**—All culverts and short span slab structures carrying a superimposed earth fill shall be proportioned to carry the entire weight of all the filling material directly above the structures. Footings of substructures shall, in addition to masonry and other loadings, be designed to carry the entire weight of earth fill vertically over the footing. For column footings, this shall be increased by 25%, when such increase will produce maximum critical stresses, in order to provide for the increased loading resulting from soil settlement causing cohesion failures outside of the vertical planes bounding the footings.

SECTION 7—UNIT STRESSES

7.1 **General Application.**—Except as otherwise provided herein, the several parts of a structure shall be so proportioned that the unit stresses will not exceed the following. Unless otherwise noted, all unit stresses are given in pounds per square inch.

7.2 **Steel Structures.**

7.2.1 **Structural Grade and Rivet Steel.**

7.2.1.1 **Tension.**

Axial tension, structural members, net section 15,000
Rivets in tension, where permitted,—50% of single shear values.
Bolts, area at root of thread 10,000

7.2.1.2 **Axial Compression.**

Axial compression, gross section 15,000-50L/r
but not to exceed 13,500.
L = Length of member, in inches.
r = Least radius of gyration, in inches.

7.2.1.3 **Bending on Extreme Fiber.**

Rolled shapes, built sections and girders, net section 16,000
Pins 24,000

7.2.1.4 **Compression Flanges of Beams and Girders.**

For unsupported length of flanges exceeding 12 times the flange width but not more than 40 times the flange width .... $19{,}000 - 250\dfrac{L}{b}$
(maximum 16,000), where
L = Length in inches of unsupported flange between lateral connections, diaphragms or knee braces.
b = Flange width in inches.

7.2.1.5 **Shear.**

Girder webs, gross section 10,000
Pins and shop driven rivets 12,000
Power driven field rivets and turned bolts 10,000
Hand driven rivets and unfinished bolts 7,500

7.2.1.6 Bearing.

Pins, steel parts in contact and shop driven rivets	24,000
Power driven field rivets and turned bolts	20,000
Hand driven rivets and unfinished bolts	15,000
Expansion rollers, pounds per linear inch	600d

where
d = diameter of roller in inches.

7.2.1.7 Countersunk Rivets.—In metal 3/8 inch thick and over, half the depth of countersink shall be omitted in calculating bearing area. In metal less than 3/8 inch thick, countersunk rivets shall not be assumed to carry calculated stress.

7.2.1.8 Diagonal Tension.

In webs of girders and rolled beams, at sections where maximum shear and bending occur simultaneously	16,000

7.2.2 Other Metals.

7.2.2.1 Axial Tension.

Wrought iron		12,000
Carbon steel forgings	.40% carbon. Untreated	20,000
Structural nickel steel	3.5 % nickel .35% carbon	28,000
Forged nickel steel	3.5 % nickel. Heat treated .35% carbon	32,000
Plow steel cable		50,000
Structural silicon steel	0.4 % carbon 0.15% silicon	25,000

7.2.2.2 Bending on Extreme Fiber.

Cast steel, annealed		12,000
Cast iron		3,000
Nickel steel pins	3.5 % nickel .35% carbon	36,000

7.2.2.3 Shear.

Cast steel, annealed		10,000
Cast iron		3,000
Nickel steel rivets, Power driven	3.5 % nickel	
Field rivets		14,000
Shop rivets		16,000

7.2.2.4 Bearing.

Cast steel, annealed		14,000
Cast iron		10,000
Bronze sliding expansion bearings		3,000
Carbon steel forgings	.40% carbon. Untreated	25,000
Nickel steel rivets, Power driven	3.5 % nickel.	
	Field rivets	28,000
	Shop rivets	32,000
Structural nickel steel	3.5 % nickel .35% carbon	32,000
Nickel steel forgings	3.5 % nickel. Heat treated .35% carbon	40,000

Use the lowest of the above values for the materials in contact.

7.3 Bearing on Bridge Seats.

Bearing on stone masonry	400
Bearing on concrete masonry, Grade A concrete	600

7.4 Concrete Structures.

7.4.1 Assumed Compressive Strengths.—The following allowable unit stresses for concrete are based on assumed compressive strengths at the date when the structure is exposed to the full loading for which it is designed and for ordinary conditions, assumed as the strength at 28 days, as given below, subject to proportioning and field control as specified under Section 8, Division 5, and when tested in accordance with the Tentative Standard Methods of Sampling and Testing of the American Association of State Highway Officials. The unit stresses are also subject to design details and conditions as specified in Sections 8 and 9.

For Structural Grades Portland Cement Concrete

Grade	Approx. Proportions	28 Day Strength
A.	1 : 2 : 3½	2500
B.	1 : 2½ : 4	2000
C.	1 : 3 : 5	1500

7.4.2 Concrete Grades.—The concrete grades for various uses shall preferably be as follows:

Grade A All superstructure concrete.
Concrete railings.
Concrete wearing surfaces.
Tremie concrete.
Subaqueous waterproof pits.
Abutment wings above bridge seat copings.

Grade B Reinforced concrete structures except bridge superstructures calculated as structural beams and slab units, and all exposed concrete of 18 inch thickness, or less.

Concrete Grades.—(*Con.*)

Grade C Mass concrete.

Semi-gravity retaining walls and abutments.
In general, all concrete not otherwise classified.

In the following formulae for working stresses, f'_c is the 28 day strength as given above.

7.4.3 Direct Compression.—Piers and Pedestals of plain concrete, depth not greater than three times least width 0.25 f'_c

7.4.4 Columns with Axial Loading.

7.4.4.1 Short Columns. $\frac{h}{D} = 12$ or less, where

h = unsupported length of column.
D = least outside diameter for columns without spiral reinforcement, or the diameter center to center of hooping for columns with spiral reinforcement.

Columns with Longitudinal Reinforcement and Ties.
Allowable unit stress on total concrete area, .20 f'_c

Columns with Spiral and Longitudinal Reinforcement.
Allowable unit stress on concrete core area, $300 + (.10 + 4p)\ f'_c$, where
p = ratio of effective area of longitudinal reinforcement to area of concrete core.

7.4.4.2 Long columns. $\frac{h}{D} =$ more than 12.

Columns with Longitudinal Reinforcement and Ties.

Allowable unit stress on total concrete area, $0.20\ f'_c \left\{1.33 - \frac{h}{36D}\right\}$

Columns with Spiral and Longitudinal Reinforcement.

Allowable unit stress on concrete core area, $300 + (.10 + 4p)\ f'_c \left\{1.33 - \frac{h}{36D}\right\}$

Composite Columns of Structural Steel and Concrete.

Allowable concrete unit stress on concrete core area, 0.25 f'_c

7.4.5 Columns subject to Bending Stresses—Short Columns. $\frac{h}{D} = 12$ or less

Columns with Longitudinal Reinforcement and Ties.
Allowable maximum unit stress, .30 f'_c

Columns with Spiral and Longitudinal Reinforcement.
Allowable maximum stress on concrete core area, $360 + (.12 + 4.8p)\ f'_c$

7.4.6 Compression in Extreme Fiber Due to Bending.

Beams and Slabs .... 0.375 f'_c
Continuous Beams and Slabs, Adjacent to Supports .... 0.43 f'_c
Arch Rings and Ribs, including temperature and rib shortening .... 0.40 f'_c
Arch Rings, excluding temperature and shortening, Short Spans .... 0.25 f'_c

7.4.7 Tension in Extreme Fiber Due to Bending.

Reinforced concrete .... 0.
Plain concrete massive sections of secondary importance .... 0.03 f'_c

7.4.8 Shear (Diagonal Tension).

Beams without web reinforcement:
Longitudinal bars without special anchorage .... 0.02 f'_c
Beams without web reinforcement:
Longitudinal bars with special anchorage as specified in Section 8 .... 0.03 f'_c
Beams with web reinforcement:
Longitudinal bars without special anchorage .... 0.06 f'_c
Beams with web reinforcement:
Longitudinal bars with special anchorage as specified in Section 8 .... 0.12 f'_c

7.4.9 Punching Shear .... 0.12 f'_c

7.5 Reinforcement.

7.5.1 Tension in Beams and Slabs 16,000
Arch rings and ribs including temperature and rib shortening 20,000

7.5.2 Compression.—Stress in surrounding concrete multiplied by the ratio of the moduli of elasticity of steel to concrete as specified in Section 8.

7.5.3 Bond.
Bars without special anchorage .... 0.04 f'_c
Bars with special anchorage as specified in Section 8 .... 0.06 f'_c

7.5.4 Compression on Structural Steel in Composite Steel and Concrete Columns, $18,000 - 70\frac{L}{r}$ but not more than 16,000.

L = Unsupported length in inches.
r = Least radius of gyration of the steel section.

7.6 Bearing Power of Soils.—In the absence of bearing tests, the following unit bearing values may be assumed. This tabulation covers only the broad basic groups of materials, for which approximate maximum ranges in bearing values are given. In the design of foundations, where boring information is inconclusive or unfavorable, bearing pressures not in excess of 2½ tons per square foot shall be used, and, if possible, not in excess of 2 tons per square foot. In the event that the foundations are found, on excavation, to be incapable of supporting the design load, piling, caissons or other means will be provided for the stability of the structure, and footings redesigned to correspond, if necessary.

Material	Safe Bearing Power Tons per Sq. Ft. Min.	Max.
Alluvial soils, soft clay or unconfined wet sand	½	1
Sand, confined	1	4
Clay and dry sand	2	3
Clay, moderately dry, or clean dry sand	2	4
Gravel	2	4
Cemented sand and gravel	5	10
Rock	5	—

7.7 Timber Structures.—The following unit stresses for structual grades of timber are for use with computed stresses which contain no allowance for live load impact. When impact is included in the computed stresses, these unit stresses may be increased by 25%.

Species of Wood	Axial Tension and Bending in Extreme Fiber		Compression Parallel to Grain Short Columns C_s		Compression Perpendicular to Grain. All Grades C_p	Horizontal Shear in Beams		Ultimate Modulus of Elasticity All Grades
	Select	Common	Select	Common		Select	Common	
Cedar, Northern White	650	550	500	400	140	70	60	800,000
Cedar, Port Orford	1000	800	825	700	200	100	80	1,200,000
Cedar, Western Red	800	650	700	600	150	80	65	1,000,000
Chestnut	850	700	700	600	200	90	75	1,000,000
Cypress, Southern	1200	900	1000	800	250	100	80	1,400,000
Fir, Douglas (Coast)	1400	1100	1000	800	225	90	75	1,600,000
" Dense Structural	1600	....	1150	....	275	90	....	1,600,000
Fir, Red (Inter-mountain)	900	750	800	650	225	85	70	1,200,000
Gum, Black	900	750	750	625	200	100	80	1,200,000
Hemlock, West Coast	1100	900	900	750	225	75	60	1,400,000
Larch, Western	1100	900	1000	800	225	100	80	1,300,000
Oak, Red and White	1200	1000	900	750	375	125	105	1,500,000
Pine, Idaho White, Northern White, Pondosa and Sugar	800	650	750	625	150	85	70	1,000,000
Pine, Norway	1000	800	800	650	175	85	70	1,200,000
Pine, Southern Yellow	1400	1100	1000	800	225	110	90	1,600,000
Pine, Southern Yellow, Dense Select	1600	....	1150	....	275	110	....	1,600,000
Redwood	1000	800	900	750	150	70	60	1,200,000
Spruce, Eastern and Sitka	900	750	750	625	150	85	70	1,200,000
Spruce, Engelmann	650	550	550	450	140	70	60	800,000
Tamarack	1100	900	900	750	225	100	80	1,300,000

Values for Direct Shear Parallel to Grain in details of joints may be taken 50% greater than the values for Horizontal Shear in Beams.

7.7.1 Axial Compression in Timber Columns.

$p = 4/3\ C_s(1 - \frac{L}{40D})$

The value of "p" shall not exceed the value of "C_s"

p = unit compressive stress in column.

C_s = unit stress for compression parallel to grain in short columns.

L = unsupported length of column.

D = least diameter of column.

7.2.2 Bearing on Inclined Surfaces.

$p = C_p + (C_s - C_p)\ \frac{\phi^2}{8100}$

p = unit bearing stress on inclined surface.

C_p = unit stress for compression perpendicular to grain.

C_s = unit stress for compression parallel to grain in short columns.

ϕ = angle in degrees between bearing surface and direction of fibers (or axis of piece).

7.7.3 Horizontal Shear in Beams.—Horizontal shear in beams shall be computed from the maximum shear occurring at a distance from the support equal to three times the depth of the beam.

7.8 Analyses of Existing Structures.—For analyses of existing structures, the allowable unit stresses previously specified may be increased by the following percentages:—

7.8.1 For structural steel..........50%

7.8.2 For Concrete..........30%

7.8.3 For Timber..........No Increase

SECTION 8—CONCRETE DESIGN

8.1 General Assumptions.—The design of reinforced concrete members under these specifications shall be based on the following assumptions:

(a) Calculations are made with reference to unit working stresses and safe loads, as elsewhere specified herein, rather than with reference to ultimate strength and ultimate loads.

(b) A plane section before bending remains plane after bending.

(c) The modulus of elasticity of concrete in compression is constant within the limits of working stresses; the distribution of compressive stress in flexure is therefore rectilinear.

(d) The value of the modulus of elasticity of concrete in computations of strength shall be assumed as one-twelfth (1/12) that of steel for Grade "A" concrete, or one-fifteenth (1/15) that of steel for all other grades and shall be assumed as one-eighth (1/8) that of steel in computing the deflection of reinforced concrete beams.

(e) Concrete shall be assumed as offering no tensile resistance except as noted in Section 7, Article 7.4.7 and in Section 8, Article 8.6.2.

(f) The bond between concrete and metal reinforcement is assumed to remain unbroken throughout the range of working stresses. Under compression the two materials are therefore stressed in proportion to their moduli of elasticity.

(g) Initial stress in the reinforcement due to contraction or expansion of the concrete is neglected, except in the design of reinforced concrete columns and except for the arbitrary addition of reinforcement to provide for such stresses as hereinafter specified.

8.2 Standard Notation.

8.2.1 For Rectangular Beams.

f_s = tensile unit stress in longitudinal reinforcement.

f_c = compressive unit stress in extreme fiber of concrete.

E_s = modulus of elasticity of steel.

E_c = modulus of elasticity of concrete.

$n = \frac{E_s}{E_c}$

M = bending moment to be resisted.

M_r = Moment of resistance of the section to provide for M.

A_s = effective cross-sectional area of tension reinforcement.

b = width of beam.

d = effective depth, or depth from compression surface of beam to effective center of tension reinforcement

k = ratio of depth of neutral axis to effective depth, d.

j = ratio of lever arm of resisting couple M. to depth, d.

$jd = d - z$ = arm of resisting couple M_r.

p = ratio of effective area of tension reinforcement to the concrete area $bd = \frac{A_s}{bd}$

8.2.2 Additional for T-Beams.

b = width of flange.
b' = width of stem.
t = thickness of flange.

8.2.3 Additional for Beams Reinforced for Compression.

A'_s = area of compressive steel.
p' = ratio of effective area of compression reinforcement to the concrete area bd $= \frac{A'_s}{bd}$
f'_s = compressive unit stress in longitudinal reinforcement.
C = total compressive stress in concrete.
C' = total compressive stress in steel.
d' = depth from compression surface of beam to effective center of compression reinforcement.
z' = depth from compression surface of beam to resultant of compressive stresses.

8.2.3 Additional for Shear, Bond and Web Reinforcement.

V = total shear.
V' = Shear to be carried by the steel and equals external shear on any section after deducting that carried by the concrete.
V' = 4/5V
v = shearing unit stress.
u = bond stress per unit of area of surface of bar.
0 = perimeter of bar.
$\Sigma 0$ = sum of perimeters of bars.
s = spacing of web reinforcement bars, measured at the neutral axis and in the direction of the longitudinal axis of the beam.
A_v = total area of web reinforcement in tension within a distance, a.
α = angle between web bars and longitudinal bars.
f_v = tensile unit stress in web reinforcement.

8.2.4 Additional for Columns.

A = Net area of concrete (total effective column area minus area of reinforcement).
A'_c = area of enclosed concrete core measured from center of spiral reinforcement.
a_s = effective cross-sectional area of longitudinal reinforcement.
a'_s = area of structural steel core in a composite column.
P = total safe load.
f'_c = assumed strength of concrete at time of loading as given in Section 7.
h = unsupported length of column.
D = least diameter of column for columns without spiral reinforcement, or the core diameter for columns with spiral reinforcement, measured to center of spiral steel.
L = unsupported length of structural steel core in a composite column.
r = least radius of gyration of structural steel core.

8.2.5 Columns Subject to Compression and Flexure.—See Figures 19 and 20.

8.3 Design Formulas.—Flexure of Rectangular Reinforced Concrete Beams and Slabs.
Computation of flexure in rectangular reinforced concrete beams and slabs shall be based on the following formulas:

8.3.1 Reinforced for Tension Only.—See Figure 16.

Position of neutral axis,

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

Arm of resisting couple,

$$j = 1 - \frac{k}{3}$$

Compressive unit stress in extreme fiber of concrete,

$$f_c = \frac{2M}{jkbd^2} = \frac{2pfs}{k}$$

Tensile unit stress in longitudinal reinforcement,

$$f_s = \frac{M}{A_s jd} = \frac{M}{pjbd^2}$$

Steel ratio for balanced reinforcement,

$$p = \frac{1}{2} \frac{1}{\frac{f_s}{f_c}\left(\frac{f_s}{nf_c} + 1\right)}$$

Note:—For approximate computations, the following assumptions may be made:

j = 7/8
k = 3/8

Reinforced for Tension Only.—(*Con.*)

$$A_s = \frac{M}{14000d}$$

$$f_c = \frac{6M}{bd^2}$$

8.3.2 Reinforced for Both Tension and Compression.—See Figure 17.

Position of neutral axis,

$$k = \sqrt{2n(p+p'\frac{d'}{d})+n^2(p+p')^2} - n(p+p')$$

Position of resultant compression,

$$z = \frac{1/3k^3d+2p'nd'\ (k-\frac{d'}{d})}{k^2+2p'n\ (k-\frac{d'}{d})}$$

Arm of resisting couple,

$$jd = d - z$$

Compressive unit stress in extreme fiber of concrete

$$f_c = \frac{6M}{bd^2[3k-k^2+\frac{6p'n}{k}(k-\frac{d'}{d})\ (1-\frac{d'}{d})]}$$

Tensile stress in longitudinal reinforcement

$$f_s = \frac{M}{pjbd^2} = \frac{nf_c(1-k)}{k}$$

Compressive stress in longitudinal reinforcement,

$$f'_s = nf_c \frac{(k-\frac{d'}{d})}{k}$$

8.4 Flexure of Reinforced Concrete T-Beams.—See Figure 18.

Computations of flexture in reinforced concrete T-Beams shall be based on the following formulas:

8.4.1 Neutral Axis in the Flange.—Use the formulas for rectangular beams and slabs.

8.4.2 Neutral Axis Below the Flange.—The following formulas neglect the compression in the stem.

Postition of neutral axis,

$$kd = \frac{2ndA_s+bt^2}{2nA_s+2bt}$$

Position of resultant compression,

$$z = \frac{(3kd-2t)}{2kd-\ t}\frac{t}{3}$$

Arm of resisting couple,

$$jd = d - z$$

Compressive unit stress in extreme fiber of concrete,

$$f_c = \frac{Mkd}{bt(kd-\frac{1}{2}t)jd} = \frac{f_s}{n}\frac{k}{1-k}$$

Tensile unit stress in longitudinal reinforcement,

$$f_s = \frac{M}{A_sjd}$$

(For approximate results, the formulas for rectangular beams may be used.)

The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem:

Position of neutral axis,

$$kd = \sqrt{\frac{2ndA_s+(b-b')t^2}{b'}+\left\{\frac{(nA_s+(b-b')t}{b'}\right\}^2} - \frac{nA_s+(b-b')t}{b'}$$

Position of resultant compression,

$$z = \frac{b(kdt^2-2/3t^3)+b'\left\{(kd-t)^2\ [t+1/3\ (kd-t)]\right\}}{bt(2kd-t)+b'\ (kd-t)^2}$$

Arm of resisting couple,

$$jd = d - z$$

Neutral Axis Below the Flange.—(*Con.*)

Compressive unit stress in extreme fiber of concrete,

$$f_c = \frac{2Mkd}{[(2kd - t)bt + b'(kd - t)^2]jd}$$

Tensile unit stress in longitudinal reinforcement,

$$f_s = \frac{M}{A_s jd}$$

8.5 Shear, Bond and Web Reinforcement.

8.5.1 Shear and Diagonal Tension.—Diagonal tension and shear in reinforced concrete beams shall be calculated by the following formulas:

Shearing unit stress in concrete,

$$v = \frac{V}{bjd}$$

8.5.2 Web Reinforcement.—In a distance "s," measured at the neutral axis and in the direction of the longitudinal axis of the beam, the required area of web reinforcement, "A_v" measured at right angles to its axis, shall be computed as follows:

8.5.2.1 Vertical Web Reinforcement.

$$A_v = \frac{V's,}{f_v jd} \quad \text{or} \quad s = \frac{A_v f_v jd}{V'}$$

8.5.2.2 Inclined Web Reinforcement.

$$A_v = \frac{V's \sin\alpha,}{f_v jd} \quad \text{or} \quad s = \frac{A_v f_v jd}{V' \sin\alpha}$$

8.5.2.3 Web Reinforcement Consisting of Bars Bent up at a Single Point.

$$A_v = \frac{V'}{f_v \sin\alpha}$$

8.5.2.4 Two or More Types in Combination.—The area of web reinforcement A_v, for one type shall be proportioned to carry an assumed portion of the shear V′, and the remaining type or types of reinforcement shall be designed to carry the balance of the shear V′. In any space tributary to bent up bars, stirrups shall be designed for the shear V′.

8.5.3 Bond.—The bond between concrete and reinforcement bars in reinforced concrete beams and slabs shall be computed by the formula:

$$u = \frac{V}{jd\Sigma o}$$

(For approximate results "j", in the above formulas may be taken as 7/8.)

As regards shear and bond stress for tensile steel, the above formulae apply also to beams reinforced for compression.

8.6 Columns.

8.6.1 Columns with Axial Loadings.

8.6.1.1 Short Columns.

$\frac{h}{D} = 12$, or less

(a) Columns with Longitudinal Reinforcement and Ties.

$P = f_c (a + a_s n)$, where f_c is the permissible concrete stress as given in Section 7, Article 7.4.4.1 for this case, or

$P = 0.20 f'_c (a + a_s n)$.

(b) Columns with Spiral and Longitudinal Reinforcement

$P = f_c a'_c (1 - p + np)$, where f_c is the permissible concrete stress as given in Section 7, Article 7.4.4.1 for this case, or

$P = [300 + (0.10 + 4p) f'_c] A'_c (1 - p + np)$.

8.6.1.2 Long Columns.

$\frac{h}{D} =$ more than 12

(a) Columns With Longitudinal Reinforcement and Ties.

$P = f_c (a + a_s n)$ where f_c is the permissible concrete stress as given in Section 7, Article 7.4.4.2 for this case, or

$P = 0.20 f'_c (a + a_s n) (1.33 - \frac{h}{36D})$

(b) Columns with Spiral and Longitudinal Reinforcement.

$P = f_c A'_c (1 - p + np)$ where f_c is the permissible concrete stress as given in Section 7, Article 7.4.4.2 for this case, or

$P = [300 + (0.10 + 4p) f'_c] A'_c (1 - p + np) (1.33 - \frac{h}{36D})$.

8.6.2 Structure Units subject to Combination of Compression and Flexure.—When a section of width "b" inches is subject to an axial loading N in pounds and a flexural moment $M_s = N\,x_o$ as indicated in Figures 19 and 20 respectively, the distribution of stress will be as indicated in Figure 19 if there is no tension in the concrete or if the resulting tension is not great enough to be serious. For this purpose a maximum negative or tension value of f'_c shall be taken as not to exceed 10% of the maximum allowable value of f_c. In case such maximum tension values are exceeded the design or analysis shall be computed on the assumptions of Figure 20, where all tension in the concrete is neglected. The following formulae will apply:

(a) $f'_s = (1-\frac{d'}{k})$ n.f_c. Compression. Either Case 1 or Case 2.

(b) $f_s = (1-\frac{d}{k})$ n.f_c. Compression, if positive. Tension, if negative. Either Case 1 or Case 2.

(c) $f'_c = (1-\frac{1}{k})\, f_c$. Compression, if positive. Negative or Tension, if k is less than 1. Either Case 1 or Case 2.

(d) $f_c = \frac{N}{b.t.}\,\frac{X_o}{Q}$ Compression. Either Case 1 or Case 2.

(e) $f_c = \frac{N}{b.t.}\,\frac{1}{R}$ Compression. Either Case 1 or Case 2.

(f) $x_o = \frac{Q}{R}$ Positive. Either Case 1 or Case 2.

(g) $Q = (n-1)\,p'\,(1-\frac{d'}{k})\,(½-d') - (n-1)\,p\,(1-\frac{d}{k})\,(d-½) + \frac{1}{12k}$ General for Case 1.

(h) $R = (n-1)\,(p+p) - \frac{(n-1)}{k}\,(p'd'+pd) + \frac{(1-1\ )}{2k}$ General for Case 1.

(i) $Q = (n-1)\,p'\,\frac{(1-d)}{k}\,(½-d') - (n-1)\,p\frac{(1-d)}{k}\,(d-½) + \frac{k(3-2k)}{12}$ General for Case 2.

(j) $R = (n-1)\,(p+p') - \frac{(n-1)}{k}\,(p'd'+p.d) + \frac{k}{2}$ General for Case 2. Generally, $p = p'$ and $d = 1-d'$ in which case

(k) $Q = \frac{1+24\,(n-1)\,p'\,(½-d')^2}{12.k}$ For Case 1.

(l) $R = [1+2\,p'.\,(n-1)]\frac{(1-1)}{2k}$ For Case 1.

(m) $R = \frac{x_o\,[1+2(n-1)\,p']\ \ [1+24(n+1)\,p'\ \ (½-d')^2]}{6.x_o\,[1+2(n-1)\,p'] + [1+24(n-1)\,p'\ \ (½-d')^2]}$ For Case 1.

(n) $Q = \frac{2\,(n-1)\,p'\ \ (½-d')^2}{k} + \frac{k\,(3-2k)}{12}$ For Case 2.

(o) $R = 2(n-1)\,p'\,(1-\frac{1}{2k}) + \frac{k}{2}$ For Case 2.

The application of these formulae will be greatly facilitated by using diagrams such as those published in various text books. For Case 1, values of R in equation m will be the reciprocal of the values of K given in Hool and Johnson Diagrams 1 to 6 inclusive, or Diagram 6 in Kalman Steel Company's "Useful Data" in which their value x_o/t is simply the x_o of the above equations, but their value p_o is twice the above.

8.7 Effective Span Lengths.—The effective span length of freely supported beams and slabs shall in general be the distance between the effective centers of the bearing areas, but shall not exceed the clear span plus the gross depth of beam or slab. The span length for continuous or restrained beams built monolithically with supports shall be considered as the clear distance between faces of supports. Where brackets having a width not less than the width of the beam, and making an angle of 45° or more with the axis of a restrained beam, are built monolithic with the beam and support, the span shall be measured from the section where the combined depth of the beam and bracket is at least one-third more than the depth of the beam. Maximum negative moments are to be considered as existing at the end of the span, as above defined. No portion of a bracket shall be considered as adding to the effective depth of the beam.

8.8 Moments in Continuous Slabs and Beams.

8.8.1 Live Load Moments.—Concrete floor slabs continuous over supports shall be designed for positive live load moments near the centers of spans, and negative live load moments at the supports equal to eight tenths of the maximum live load bending moments in simply supported slabs of the same span. Reinforcement shall be provided for the positive and negative moments, as above, and for the net negative moment produced in any span by live loading on an adjacent span.

8.8.2 Dead Load Moments.—Continuous or restrained beams and slabs of equal span, with uniformly distributed dead loads shall be designed for the following maximum dead load bending moments:

	Conditions at End Supports of End Spans	
(a) Beams and Slabs of One Span.	**Free**	**Restrained**
At Supports	0	$\frac{-wl^2}{12}$
At Center	$\frac{+wl^2}{8}$	$\frac{+wl^2}{10}$
(b) Beams and Slabs Continuous for Two Spans Only.		
At End Supports	0	$\frac{-wl^2}{12}$
At Interior Supports	$\frac{-wl^2}{8}$	$\frac{-wl^2}{10}$
Near Centers of Spans	$\frac{+wl^2}{12}$	$\frac{+wl^2}{12}$
(c) Beams and Slabs Continuous Over More than Two Supports.	**Free**	**Restrained**
At End Supports	0	$\frac{-wl^2}{12}$
At First Interior Support	$\frac{-wl^2}{10}$	$\frac{-wl^2}{12}$
At Other Interior Supports	$\frac{-wl^2}{12}$	$\frac{-wl^2}{12}$
Near Center of End Span	$\frac{+wl^2}{10}$	$\frac{+wl^2}{12}$
Near Centers of Interior Spans	$\frac{+wl^2}{12}$	$\frac{+wl^2}{16}$

8.9 Joints.

8.9.1. Construction Joints.—Construction joints shall be shown on the plans and shall be so designed and located as to provide the least chance of impairing the strength and appearance of the structure. Vertical construction joints in slabs and girders shall, in general, be located at or near the center of the span, where the shear is a minimum. In all construction joints which are subject to shearing forces keys shall be provided of sufficient width to take the maximum shear, but not exceeding one third of the depth or thickness of the concrete. If necessary, keyways shall be supplemented by steel dowels designed to take vertical shear. In columns, joints shall, in general, be made flush with the bottom of the lowest intersecting member. All exposed construction joints shall be carefully finished to avoid defacement due to overlapping thin wedges and irregular edges of concrete, as may be specified on the plans. If possible, construction joints shall be located symmetrically, and the location shall be governed by the necessity of limiting a day's run of the concrete mixer to approximately 60 cubic yards for small work, or 90 cubic yards for work involving a large amount of concrete. Horizontal construction joints in columns or walls shall be placed at intervals not exceeding 12 feet, to facilitate puddling in thin walls containing reinforcement. Construction joints which are subject to hydraulic pressure shall be made water tight by embedding thin sheets of copper, lead or zinc in the concrete, transversely through the joint, and the addition of membrane waterproofing of surfaces exposed to the water. Metal sheets for the above purpose shall be soldered or otherwise designed to form a continuous watertight sheet at all joints.

8.9.2 Expansion Joints.—Provision shall be made in long concrete structures for movement induced by temperature changes or shrinkage in the concrete due to seasoning.

8.9.2.1 Expansion on Bridge Seats.—Provision for expansion shall be made in all concrete slab or girder bridges having a span length in excess of 30 feet. When multiple span construction is used, provision shall be made for movement on each bridge seat at the abutments, and at roadway breaks between alternate spans for cantilever construction. For continuous slab or girder construction, provision shall be made for movement on each pier bridge seat and at the abutments; otherwise, the entire structure shall be designed as an elastic frame. Unless otherwise provided expansion on bridge seats between concrete surfaces shall be facilitated by placing a layer of quarter inch thickness of hard asphaltic felt, of approved quality, on the entire bearing surface of bridge seats. On bridge seats which are subject to very heavy loading or considerable movement, special provision shall be made using lead or copper plates, or otherwise by special design.

8.9.2.3 At Abutting Surfaces.—A complete break shall be made between abutting surfaces at expansion joints, and reinforcement shall not extend through joints. Exposed edges, shall be carefully finished as noted for construction joints. A space shall be provided between abutting surfaces at expansion joints, and this space shall preferably consist of a clear opening, if the opening can be kept free from obstruction and collection of water, ice and dirt. Such opening shall be not less than the anticipated movement as specified in Section 5, under "Thermal Forces". If a filler is necessary, asphaltic felt, of approved quality with a thickness of twice the anticipated movement, shall be provided.

8.9.2.4 Through Bridge Decks and Floors.—Expansion joints, on bridge decks shall be carried through the floor and the railings and provision made for ample movement. Refer to "Railings" in Section 3. Special provision shall be made for the protection of expansion joints through floor wearing surfaces where the expansion is large. Metal plates spanning the openings, securely anchored on one side and free to move on the opposite side, shall be set flush with the top of the wearing surface, or other special provision shall be made. At expansion joints which are subject to slight vertical displacement, a double joint shall be made through the wearing surface.

8.9.2.5 Through Spandrel Walls of Arches.—Vertical expansion joints shall be placed through the spandrel walls of arches to provide for movement due to temperature change and arch deflection. These joints shall in general be placed over the springing lines and at intermediate points generally not more than 40 feet apart, symmetrically about center of span, and with not less than four such joints in any individual span. On open spandrel arches expansion joints shall be provided through the floor system and railings at intervals as above specified for filled spandrels.

8.9.2.6 Through Retaining Walls.—In retaining walls keyed expansion joints shall be provided at intervals not exceeding 30 feet for gravity walls and 50 feet for reinforced concrete walls.

8.9.3 Shrinkage and Wet Seasoning.—For the purpose of segregating cracks due to seasoning of concrete in air, a vertical triangular moulding ¾ inch wide shall be formed in longitudinal members at intervals of approximately 15 feet. In exposed superstructure members, irregular shrinkage cracks shall be avoided by thoroughly wetting the forms preliminary to casting the concrete, by wetting the concrete as soon as possible after casting without damage to the surface or finish, and by keeping the concrete thoroughly wet for a period of 7 days thereafter.

8.10 T Beams.

8.10.1 Effective Flange Width.—In beam and slab construction, effective and adequate bond and shear resistance shall be provided at the junction of the beam and slab. The slab may then be considered an integral part of the beam but its assumed effective width as a T-beam flange shall not exceed the following:

(a) One-fourth of the span length of the beam
(b) The distance center to center of beams
(c) Eight times the least thickness of the slab plus the width of the girder stem.

For beams having a flange on one side only, the effective flange width to be used in design shall not exceed one tenth of the span length of the beam, and its overhanging width from the face of the web shall not exceed six times the thickness of the slab nor one-half the clear distance to the next beam.

8.10.2 Shear.—The flange of the slab shall not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

8.10.3 Isolated Beams.—Isolated Beams in which the T-form is used only for the purpose of providing additional compression area, shall have a flange thickness of not less than one-half the width of the web, and a total flange width of not more than 4 times the web thickness.

8.10.4 Diaphragms.—For T-beam spans over 40 feet in length diaphragms or spreaders shall be placed between the beams at third points.

8.10.5 Continuous T-Beams.—In continuous T-beam construction, due consideration shall be given to the tensile and compressive stresses at the supports.

8.10.6 Transverse Reinforcement.—Where the principal slab reinforcement is parallel to the beams, transverse reinforcement for negative bending moments, shall be provided in the top of the slab in the amount of not less than 0.3% of the sectional area of the slab, and shall extend across the beam into the slab not less than two thirds of the width of the effective overhang.

8.11 Reinforcement.

8.11.1 Spacing in Beams.—The lateral or vertical spacing of parallel bars in beams shall not be less than 3 diameters center to center. The distance from the side or bottom of the beam to the center of the nearest bar shall not be less than 2 diameters, nor less than 2 inches. Not more than 3 layers of bars shall be used, unless alternate layers are separated by spacer bars of the same diameter and securely wired together.

8.11.2 Spacing in Slabs.—The maximum spacing center to center of main reinforcing bars in slabs shall be 1½ times the effective depth of the slab. The minimum spacing, center to center shall be 3 inches. In slabs, the minimum covering measured from the surface of the concrete to the center of bars shall not be less than 1½ diameters, nor less than 1½ inches.

8.11.3 Minimum Covering in Footings.—In the footings of abutments and retaining walls, and in piers, the minimum covering shall be 3 inches, measured to the center of the bars.

8.11.4 Maximum Spacing of Secondary Steel.—The maximum spacing of secondary reinforcing bars in slabs or walls, for temperature stresses, for supporting the main reinforcement, or for other purposes shall be 2 feet.

8.11.5 Crossing Layers.—Where two layers of bars cross each other in contact, the bars forming the main reinforcement shall be placed nearest the surface of the concrete.

8.11.6 Splicing.—Splices in tensile reinforcement, shall, if possible, be avoided at points of maximum stress. Spliced bars shall be lapped sufficiently to develop the full strength in bond, and shall be wired together for the full length of the splice with No. 12 wire, or equivalent.

In columns, bars more than ¾ inch diameter, not subject to tension, shall be properly squared and butted into a suitable sleeve; smaller bars may be treated as indicated for tensile reinforcement.

8.11.7 Anchorage.—In simple or continuous beams or slabs, at least one fourth of the area of the maximum positive tensile reinforcement shall be embedded not less than 20 bar diameters beyond the face of the support, or shall be provided with standard hooks. Standard hooks shall be formed by bending 180 degrees on a radius of not less than 3 bar diameters, with a final tangent length of 4 bar diameters. Dowels or other short bars subject to tensile stress shall be embedded not less than 48 diameters. For this purpose, standard hooks shall be assumed as equivalent to 24 diameters embedment.

In continuous or restrained beams or slabs, the negative reinforcement shall be carried to or beyond the point of inflection, to develop the negative bending moment. In continuous, restrained or cantilever beams or slabs, the negative reinforcement shall be embedded back of the face of the support not less than 48 bar diameters, nor less than the depth of the beam.

8.11.8 Special Anchorage.—If special anchorage of longitudinal reinforcing steel is provided, increased shear and bond stresses may be used as specified in Section 7.

In simple beams or slabs, or at freely supported ends of continuous beams or slabs, an embedment of 24 bar diameters back of the face

Special Anchorage.—(*Con.*)

of the support of all tensile reinforcement, or standard hooks beyond the fact of the support on all tensile reinforcement shall be considered as special anchorage. If one-half or more of the area of the tensile reinforcement is provided with special anchorage in this manner, the same proportion of the allowable increase in shear and bond stresses for special anchorage may be used.

In cantilevered beams or slabs, special anchorage shall not be considered, as permitting increased unit stresses except for footing slabs as herein specified.

In cantilevered footing slabs, special anchorage, as above, at the toe, shall be considered as extending forward from a point one foot, back of the toe. Where a wall bears on a cantilevered footing, special anchorage of the toe reinforcement at the heel shall be considered as extending backward from a point 40 bar diameters back of the face of the wall.

In continuous and restrained beams or slabs, special anchorage shall consist of an embedment of 16 bar diameters, or of standard hooks beyond the point of inflection, on at least one third of the area of the negative reinforcement; and the same embedment, or hooks, on at least one third of the area of the positive reinforcement back of the face of the support.

8.11.9 Maximum Sizes.—The maximum size of bar reinforcement shall be 1¼ inches square or equivalent, unless the particular conditions warrant the adoption of special reinforcement design. When structural steel shapes are used for reinforcement, no section having a surface area per foot of length of more than 150 square inches shall be used as a reinforcing member unless mechanical bond is provided by means of lugs, bars or other details which will effectively bond the member to the surrounding concrete.

8.11.10 Bar Chairs and Spacers.—Bar chairs and spacers shall be provided as specified in Division 5, Section 9, to ensure correct location and spacing of the reinforcing steel. Bar chairs in contact with the forms are required to be of precast concrete, of approved type.

8.12 Design of Web Reinforcement.

8.12.1 Methods.—When the allowable unit shearing stress for concrete is exceeded, web reinforcement shall be provided by one of the following methods:

(a) Longitudinal bars bent up in series or in a single plane.
(b) Vertical stirrups.
(c) Combination of bent-up bars and vertical stirrups.

When any of the above methods of reinforcement are used, the concrete may be assumed to carry one fifth of the external vertical shear, the remainder of the shear being carried by the web reinforcement.

The webs of T-beams shall be reinforced with vertical stirrups in all cases.

8.12.2 Bent-Up Bars.—Bent-up bars used as web reinforcement may be bent at any angle between 20 and 45 degrees with the longitudinal reinforcement. The radius of bend shall not be less than 4 diameters of the bar.

8.12.3 Spacing of Web Reinforcement.—The distance between two successive stirrups, or between two successive points of bending up of bars, or from the point of bending up of a bar to the edge of the supports, measured in the direction of the axis of the beam, shall not exceed ¾ of the effective depth of the beam, when the shearing stress is not greater than 0.06 f'_c; when the shearing stress is greater than 0.06 f'_c, the distance shall not exceed ½ of the effective depth of the beam. The distance from the edge of the support to the first stirrup, measured along the axis of the beam and in the plane of the lower reinforcement, shall not exceed "s" in the formulae in Section 8, Articles 8.5.2.1, 8.5.2.2 and 8.5.2.3, when V' is computed at the face of the support. When the web reinforcement consists of bars bent up at a single point, the distance from the edge of the support to the point of bending up, shall not exceed ¾ of the effective depth of the beam.

8.13 Columns.

8.13.1 Slenderness Ratio.—The ratio of the unsupported length to the least diameter or dimension shall not exceed 3 for unreinforced piers and pedestals, 24 for reinforced concrete sections with axial loading, and 12 for reinforced concrete sections subject to bending stresses. The least diameter or dimension shall in no case be less than 15 inches. For reinforced concrete viaduct construction or for "pedestal" abutments, the least dimension shall be not less than 24 inches.

8.13.2 Columns with Longitudinal Reinforcement and Ties.—The longitudinal reinforcement shall consist of not less than 4 bars of 1 inch minimum diameter, and shall have a total cross-sectional area of not less than 0.5 per cent of the total area of the columns. Reinforcement in excess of 2.0 per cent of the cross-sectional area of the column for axially loaded columns, or 4 per cent for columns subject to bending, shall not be considered in computing stresses.

Laterial ties shall be not less than ¼ inch in diameter, spaced not more than 8 inches apart.

8.13.3 Columns with Spiral and Longitudinal Reinforcement.—The longitudinal reinforcement shall consist of at least 8 bars of 1 inch minimum diameter. and its effective cross-sectional area shall be not less than 1 per cent nor more than 6 per cent of that of the core, measured to the center of the spiral reinforcing.

The spiral reinforcement shall consist of evenly spaced continuous spirals held firmly in place and true to line by vertical spacer bars. The spacing of the spirals shall be not greater than 3 inches, nor greater than 1/6 of the diameter of the core. The spiral reinforcement in any unit of height shall be not less than ¼ the volume of the longitudinal reinforcement.

8.13.4 Composite Columns with Spiral Reinforcement.—In composite columns of structural steel and concrete, the structural steel shall be thoroughly encased in a circumferentially reinforced concrete core, with spiral reinforcement of not less than 0.5 per cent of the volume of the core within the spiral. The spiral shall conform in all other respects with the requirements for spirally reinforced concrete columns.

In the design of composite columns, special brackets shall be arranged on the structural steel core to receive directly the loads from adjoining concrete members. Ample section of concrete and continuity of reinforcement shall be provided at the junction with beams and girders.

8.14 Concrete Arches.

8.14.1 Limitations.—Concrete arches shall be avoided where there is any doubt in regard to stability of the foundations, or where foundation piling is necessary. Concrete arches on piles are liable to movement of abutments, and more or less serious failure, due to the difficulty of driving and bracing piling at a batter parallel to the resultant thrust on the foundations. Flat arches with a small rise to span ratio require unusually stable foundation, preferably rock.

Arches shall be avoided in locations where high water clearance is limited and where flood waterway would be restricted by arches and spandrels.

8.14.2 Design and Shape of Arch Ring.—Hingeless arches of not more than 40 feet clear span with a ratio of rise to span greater than 0.25 may be designed statically, assuming the resistance line for external loading to pass through the upper limit of the middle third of the crown section, and through the lower limit of the middle third at the skewbacks. Such arches shall be designed for dead load with live load over the whole span and over one-half the span. The effects of temperature and arch shortening are neglected except for reduction of allowable working stress, as given in Section 7.

The analysis of stresses in hingeless arches with spans in excess of 40 feet, or in shorter spans with a rise ratio in excess of 0.25, shall be based on the elastic theory, and the effect of temperature changes and rib-shortening shall be computed. Preference shall be given to a combination of analytical and graphical treatments which will give the resultant line of thrust for a combination of dead and live loads together with rise and fall of temperature. The following live load conditions shall be considered: Live Load over the whole span, over one-half the span, over the middle third of the span and over the end thirds of the span, all in combination with the extreme rise and fall respectively of temperature as provided in Section 5, Article 5.14.3.

Arch rings shall be selected as to shape in such manner that the axis of the ring shall conform, as nearly as practicable, to the equilibrium polygon for full dead load plus one half live load over the whole span.

8.14.3 Reinforcement.—Arch ribs in reinforced concrete construction shall be reinforced with a complete double line of longitudinal reinforcement consisting of an introdosal system and an extrodosal system connected by a series of stirrups or tie-rods. Suitable transverse tie bars shall be used to maintain the alignment and spacing of the reinforcement and distribute the loading laterally over the arch ring. In narrow arch ribs the tie bars shall be designed to act as hoops as in the case of reinforced concrete columns.

In earth-filled spandrel arches, transverse tie bars shall similarly be used to maintain alignment and spacing of the longitudinal reinforcement and to resist bending stresses due to overturning action of the spandrel walls.

For earth-filled arches, the arch rings are to have a total longitudinal reinforcement of not less than one per cent of the crown section. At the springing points, this steel shall be increased, if necessary, in order to provide not less than one-fourth per cent of reinforcement in each face.

In large arches, where the arch rib or arch ring is constructed in alternate sections for the purpose of minimizing shrinkage stresses, the splices in the longitudinal reinforcement shall be so arranged, adjacent to the first section to be concreted, that slight movement may take place at the laps.

8.14.4 Minimum Fill at Crown.—Earth filled arches with pavement shall have not less than 1½ feet of earth fill at the crown. When there is no pavement, there shall be not less than 2 feet of fill at the crown, including the non-rigid surfacing on earth-filled arches.

For electric or steam railway loading on earth-filled arches there shall be not less than 3 feet of fill below the base of rail at the crown.

8.14.5 Drainage and Waterproofing.— The top of the arch ring and the interior faces of spandrel walls shall be waterproofed with two ply membrane waterproofing and a system of blind drains or tile drains shall be laid along the intersections of spandrel walls and arch rings as specified in Section 16, Division 5.

8.15 Viaduct Bents and Towers.—When concrete columns are used in viaduct construction, bents and towers shall be effectively braced by means of longitudinal and transverse struts. For heights greater than 40 feet, both longitudinal and transverse cross bracing preferably shall be used and the footings for the columns forming a single bent shall be thoroughly tied together.

8.16 Rigid Frames.—Rigid frames shall be designed by the slope-deflection method, or otherwise so as to determine the maximum stresses at all critical points in a frame. Provision shall be made for temperature stresses in accordance with Section 5, Article 5.14.3, but the allowable unit stresses, including temperature stresses, may be increased by 25%.

8.17 Cantilever Superstructures.—Superstructures consisting of cantilevered deck girders, shall be continuous over two supports and statically determinate, with breaks, in general, in alternate spans. All supports shall have a factor of safety of two against the negative reaction produced by live load on the adjacent span. The design shall provide for maximum stresses from dead load moments in combination with maximum live load moments, whether positive or negative.

8.18 Simplications of Forms.—Concrete details shall be designed so that commercial sizes of lumber can be used advantageously without unnecessary cutting. Such details as panels, copings or other offsets shall preferably be dimensioned to utilize lumber with commercial finished thickness. The possibility of duplication of details shall be considered to permit re-use of forms.

Acute or re-entrant angles in concrete sections shall be avoided.

8.19 Architectural Treatment and Surface Finish.—Bridge structures shall receive suitable aesthetic treatment, considering anticipated long life, the amount of the investment, and the advisibility of producing a favorable impression on the public. In locations where the natural beauty of the surroundings is a public attraction, special consideration shall be given to design and details which will produce an attractive and pleasing appearance.

The usual treatment of ordinary concrete substructures shall consist of copings for retaining walls and bridge seats. Superstructures shall terminate with suitable pilasters. Wing walls shall neatly retain the grade with the specified slopes. Large flat surfaces within the view of the traveling public, as in subway construction, shall preferably receive special treatment. The surfaces may be broken up by the use of mouldings in the forms, to simulate masonry joints, or with pilasters, copings and panelling. External superstructure surfaces may be broken up by copings, offsets, pilasters and panelling. Steel deck girders shall preferably be concealed by a concrete fascia, cambered to conceal the deflection. Concrete railings shall preferably consist of precast concrete spindles, with plinth and top railing cast in place and with pilasters at approximately 10 foot centers.

A variety of surface finishes are required on various concrete surfaces as specified in Division 5, Section 8. Scrubbed finishes, or other special finishes, if required, shall be specifically noted on the plans.

SECTION 9—SUBSTRUCTURE AND RETAINING WALLS

9.1 Piles.

9.1.1 Use of Piling.—In general, piling shall be used when footings can not, at a reasonable expense, be founded on rock or other solid foundation material. Piling shall not be used in lengths less than 10 feet below cut-off. Except for temporary trestle work untreated timber piles shall be used only below permanent ground water level. The use of treated piles partially exposed above permanent ground water level will be dependent on the usage proposed and the approval of the Engineer.

Long piles in muck or other soft material without lateral stability, if subject to horizontal thrust, shall be restrained by a suitable rock or gravel fill. In abutment foundations, on piles in soft material, designed with struts between units, or otherwise to prevent horizontal movement, the effect of bending on the piles due to flowage of retained materials behind the abutments shall be investigated.

9.1.2 Design Loads.—Preferably, structures shall be proportioned to limit the maximum design load on timber piles to 15 tons per pile. In no case shall they be designed to support more than 20 tons per pile. The maximum design load on concrete piles may be assumed as from 25 to 35 tons per pile, depending on conditions. Piles shall be designed to carry the entire superimposed load, no allowance being made for the supporting value of the material between the piles. The supporting power of piles shall be determined by the application of test loads or by the use of formulas as specified in Division 7.

9.1.3 Spacing.—Footing areas shall be so proportioned that pile spacing shall not be less than 2′6″, center to center. The distance from the center of any pile to the nearest edge of the footing shall be not less than 15 inches. In the design of pile foundations, the effect of eccentric loadings due to earth thrust or other causes shall be considered and the pile spacings shall generally be designed so as to require a minimum number of piles, consistent with the permissible pile loading. In such cases, the distribution of the foundation loading to the piles shall be computed by the following method.

Nomenclature: See Figure 22.

P = Load, in tons, per lineal foot of footing
s = Spacing, in feet, for any row of piles
$n = \frac{1}{s}$ = Number of piles per foot in any row
e = Eccentricity of loading
L = Width of footing, in feet
a = Distance, in feet, from heel of footing to any row of piles
d_o = Settlement in inches of heel of footing
k = Change in amount of settlement per foot width of footing
d = Settlement, in inches, of any pile under load
B = Load, in tons per pile, to cause a settlement of one inch
p = Load, in tons, on any single pile
r = Distance, in feet, from heel of footing to locus of the center of gravity of the piles

Then $d = d_o + ka$

$$\Sigma n(d_o + ka) = \frac{P}{B}$$

$$\Sigma na(d_o + ka) = \frac{P}{B} \times L\ (e + \tfrac{1}{2})$$

From which $$Bd_o = \frac{p\Sigma na^2 - L(e+\frac{1}{2})\Sigma na}{\Sigma n \Sigma na^2 - (\Sigma na)^2}$$

$$Bk = \frac{P\ L\ (e+\frac{1}{2})\Sigma n - \Sigma na}{\Sigma n \Sigma na^2 - (\Sigma na)^2}$$

$$p = Bd_o + aBk \text{ and } x = \frac{\Sigma na}{\Sigma n}$$

9.1.4 Concrete Piles.—Precast concrete piles shall be of approved size and shape. If a square section is employed, the corners shall be chamfered at least one inch. Piles preferably shall be cast with a driving point and for hard driving preferably shall be shod with a metal shoe of approved pattern. Piling may be either of uniform section or tapered. In general, tapered piling shall not be used for trestle construction except for that portion of the pile which lies below the ground line; nor shall tapered piles be used in any location where the piles are to act as columns. In general, concrete piles shall have a cross sectional area, measured above the taper, of not less than 140 square inches.

Reinforcement for precast concrete piling shall consist of longitudinal bars in combination with lateral reinforcement in the form of hoops or spirals. The longitudinal reinforcement shall be not less than one per cent and preferably not less than one and one-half per cent of the total cross-section of the pile, and shall be designed to resist handling stresses. The reinforcement shall be placed at a clear distance from the face of the pile of not less than 2 inches. The driving point and also the top of the pile shall be protected against impact by means of special spiral winding or bands designed for this purpose. The reinforcing system preferably shall be of the "unit" type rigidly wired or fastened at all intersections. When piles exceed fifty-five feet in length, additional longitudinal reinforcement shall be added throughout the central one-third of the length. Piling under retaining walls, abutments, etc., shall be designed to withstand the lateral stresses induced.

9.2 Footings.

9.2.1 Depth.—The depths of footings shall be determined with respect to the character of the foundation materials and the possibility of undermining. Except where solid rock is encountered or in other special cases, the footings of all structures, other than culverts, which are exposed to the erosive action of stream currents preferably shall be founded at a depth of not less than 3 feet below the permanent bed of the stream. Stream piers, in locations where the stream bed is of such a material, or is subject to such conditions that scour is probable, shall either be extended to a safe distance below the bed of the stream, or other adequate provision shall be made for resisting scour. The above preferred minimum depths shall be increased as conditions may require, and protection shall be afforded against rapid, erosive currents by rip-rap, timber sheeting, or otherwise. Footings not exposed to the action of stream currents shall be founded on a firm foundation and at a depth below frost.

Footings for culverts shall be carried to an elevation sufficient to secure a firm foundation, or a heavy reinforced floor shall be used to distribute the pressure over the entire horizontal area of the structure. In the latter case, proper provision shall be made for negative bending moments at the center of the slab, for positive bending moment at the ends of the slab, and for shear and bend stresses. In any location liable to erosion, apron or cut-off walls shall be used at both ends of the culvert and, where necessary, the entire floor area between the wing walls shall be paved. Baffle walls or struts across the unpaved bottom of a culvert barrel shall not be used where the stream bed is subject to erosion. When conditions require, culvert footings shall be reinforced longitudinally.

9.2.2 Anchorage.—Footings on solid rock, shall be effectively anchored by means of anchor bolts, dowels, keys or other suitable means.

9.2.3 **Sub-footing.**—A 4 inch concrete slab of the same grade as the footing concrete and extending beyond the footing outline to the cofferdam shall generally be provided under all footings. When piling is used, the pile heads shall extend through the sub-footing 3 inches into the footing. In that case, the portion of the footing below the cut-off shall not be computed as effective concrete. Sub-footings provide a light seal coat and a clean base for placing footing steel.

9.2.4 **Concrete Seal.**—Tremied concrete seals used to make water-tight cofferdams shall be designed to resist hydrostatic buoyancy. The seal shall be effectively anchored to piling, or shall have a depth of 4/10 of the hydrostatic head.

9.2.5 **Distribution of Pressure.**—All footings shall be designed to keep the maximum soil pressures within safe bearing values. In order to prevent unequal settlement, footings, shall be designed to keep the pressure as nearly uniform as practicable. In footings having unequal pressures and requiring piling, the spacing of the piles shall be such as to secure as nearly equal loads on each pile as may be practicable.

9.2.6 **Spread Footings.**—Spread footings which act as cantilevers may be decreased in thickness from the junction of the footing slab with column or wall toward the edge of the footing, provided sufficient section is maintained at all points to provide the necessary resistance to diagonal tension and bending stresses. This decrease in section may be accomplished by sloping the upper surface of the footing or by means of vertical steps. Stepped footings shall be cast monolithically.

Except in small structures, and for footings properly reinforced, no footing shall have a thickness at the edge of less than 2 feet.

9.2.7 **Internal Stresses in Spread Footings.**—Spread footings shall be considered as under the action of downward forces, due to the super-imposed loads, resisted by an upward pressure exerted by the foundation material, or foundation piling, or other supports, and distributed over the area of the footings as determined by the eccentricity of the resultant of the downward forces. Where piles are used under footings, the upward reaction of the foundation shall be considered as a series of concentrated loads applied at the pile centers, each pile being asumed to carry its computed proportion of the total footing load, in accord with Article 9.1.3.

Footings shall be designed for bending stresses, for diagonal tension and bond stresses, and for punching shear around the periphery of the column or pier shaft. The critical section for bending and bond stresses shall be taken at the face of the column, wall or pier shaft. Bending need not be considered unless the projection of the footing is more than one-half the depth.

The critical section for diagonal tension in a rectangular footing supporting a wall, shall be computed on a vertical section parallel to the face of the wall and at a distance from the face equal to the effective depth of the footing slab.

In a square or rectangular footing which is reinforced in two directions, supporting a column, diagonal tension shall be computed on a vertical section around the perimeter of the column and at a distance from the column equal to the effective depth of the footing slab.

When a single spread footing supports a column, pier or wall, this footing shall be assumed to act as a cantilever. When two or more piers or columns are placed upon a common footing, the footing slab shall be designed for the actual conditions of continuity and restraint.

9.2.8 **Reinforcement.**—Footing slabs shall be reinforced for bending stresses and, where necessary, for diagonal tension. All bars shall be effectively anchored to develop in bond the computed stress in the bar.

The reinforcement for square footings shall consist of two or more bands of bars. The reinforcement necessary to resist the bending moment in each direction in the footing shall be determined as for a reinforced concrete beam; the effective depth of the footing shall be the depth from the top to the plane of the reinforcement. The required reinforcement shall be spaced uniformly across the footing unless the footing width is greater than the side of the column or pedestal plus twice the effective depth of the footing, in which case the width over which the reinforcement is spread may equal the width of the column or pedestal plus twice the effective depth of the footing plus one-half the remaining width of the footing. In order that no considerable area of the footing shall remain unreinforced, additional bars shall be placed outside of the width specified, but such bars shall not be considered as effective in resisting the calculated bending moment. For the extra bars a spacing double that used for the reinforcement within the effective belt may be used.

Where bars are used in diagonal bands, the sectional area of a bar multiplied by the cosine of the angle between the direction of the axis of the bar and any other direction may be considered effective as reinforcement in that direction. Corners of cantilevered footing toe projections shall be provided with a diagonal band of reinforcement, with an area per foot of width equal to the area per foot of width of the straight reinforcement in the toe multiplied by the secant of the angle between the diagonal bars and the straight bars.

When reinforcement is used in more than one direction, the allowable unit bond stresses shall be reduced as follows:

For two-way reinforcement..25%

For each additional direction..10%

9.2.9 **Transfer of Stress from Vertical Reinforcement.**—The stresses in the vertical reinforcement of columns or walls shall be transferred to the footings by extending the reinforcement into them a sufficient distance to develop the strength of the bars in bond, or by means of dowels anchored in the footings and overlapping or fastened to the vertical bars in such manner as to develop their strength. If the dimensions of the footings are not sufficient to permit the use of straight bars, the bars may be hooked or otherwise mechanically anchored in the footings.

9.3 Abutments.

9.3.1 **Design Conditions.**—Abutments shall be designed to withstand earth pressure, the weight of abutment and superstructure, live load over any portion of the superstructure or approach fill, wind forces, and tractive force when the latter exists. The design shall be investigated for any combination of these forces which may produce the most severe condition of loading. Wind forces may ordinarily be neglected, except when long span superstructures are supported by high, narrow abutments.

9.3.2 **Live Load Surcharge.**—The effect of live load on the approach fills shall be assumed as a uniform live load surcharge equivalent to a 2 feet increase in the height of the fill, and the computations for resultant thrust and its point of application shall be in accordance with Section 5.

For railroad loading, the equivalent surcharge for abutments shall be taken as the axle load divided by an area equal to the axle spacing multiplied by 14 feet, or multiplied by the center to center track spacing. The surcharge shall be increased by 50% for impact. For skewed abutments the equivalent surcharge as above shall be multiplied by the sine of the angle of intersection between the face of the abutment and the center-line of track.

9.3.3 **Loading.**—In general, three conditions of abutment loading, as follows, will require investigation for stability, assuming the abutment to be backfilled to grade:

1. No superstructure loading or live load surcharge.
2. Superstructure dead load with live load surcharge behind the abutment.
3. Superstructure dead and live loads without live load surcharge behind the abutment.

9.3.4 Stability.—Abutments and retaining walls shall be designed to be safe against overturning about the toe of the footing, against sliding on the footing base and against crushing of foundation material or overloading of piles at the point of maximum pressure.

Under ordinary conditions, the maximum load applied on the abutments will result in maximum foundation pressures, but will result in maximum stability against overturning about the toe, if allowable unit foundation pressures are not exceeded.

The factor of safety against overturning, or the ratio about the toe, of the moments of the vertical loads to the moment of the horizontal thrust, shall preferably be not less than 2, and in no case shall be less than 1¾. The resultant base pressure shall intersect the foundation within the middle third of the base of wall for gravity walls, and within the middle third of the bottom of footing for semi-gravity or reinforced concrete walls. Where the abutment rests on solid rock the resultant base pressure may strike slightly outside the middle third, provided the abutment is safe against overturning, as above specified.

In computing stresses in abutments, the weight of filling material directly over an inclined or stepped rear face, or over a reinforced concrete spread footing extending back from the face wall, may be considered as part of the effective weight of the abutment. In the case of a spread footing, the rear projection shall be designed as a cantilever supported at the abutment stem.

9.3.5 Caissons.—Where foundations are designed with a number of solid concrete caissons constructed down to a firm strata, with clear spacings laterally between caissons, consideration shall be given to the effect of concentration of the resultant load at the top of the caissons. The substructure shall be adequately anchored to the caissons by reinforcement against sliding and rotation, and the substructure above the caissons shall be reinforced as a beam between caissons. Footing toe projections, between caissons, shall be designed on the assumption that there is no vertical reaction at the bottom of the toe, with exception of the portion of the footing bearing on the caissons. Investigation shall also be made for stability against overturning and excess unit pressure at the front edge of the bottom of the caissons.

9.3.6 Semi-gravity Abutments.—Mass concrete abutments or retaining walls of semi-gravity section are designed to take advantage of the additional stability afforded by anchorage of the body of the wall to the footing. Anchorage shall consist of dowel bars spaced along the rear of the walls and extending into the wall and the footing 24 diameters or more, with hooked ends. The dowels shall be designed for a stress equivalent to the overturning moment at the top of the footing, divided by 9/10 of the base width of the wall, and the minimum reinforcement shall consist of ¾ inch round bars spaced at 18 inch centers. The base width of the wall at the top of the footing shall be not less than 30% of the depth of fill above the footing. The resulting tensile stress in the concrete shall not exceed .03 f'_c.

9.3.7 Spill-through Abutments.—Abutments which are open at the bottom to permit the retained material to spill through, shall be designed with the openings not less than 12 feet below the crown of roadway, in fill sections and not less than to the natural ground elevation where the latter is less than 12 feet below the crown of roadway. Backfill shall be carried up in front of the abutments to a line 3 feet above the top of the openings, with a slope of not more than 1 vertical to 1½ horizontal. Where the openings extend several feet above the natural ground level and the stability of the fill behind the abutments would be questionable, the fill shall be of coarse or bank run gravel, and shall be made both in front and at the back of the abutment, sloping away from the abutment in each case. When the gravel fill is used, the lower part of foundation excavation shall be backfilled with earth up to one foot below the ground line.

9.3.8 Cantilever Abutments.—The unsupported toe and heel of the base slab shall be considered as cantilever beams fixed at the edge of the support. The rear projection of the heel shall be designed to support the weight of the superimposed material and the weight of the heel, less the foundation reaction on the heel. The front projection of the toes shall be designed for the foundation reaction less the weight of the toe. The vertical stem of the cantilever wall shall be designed as a cantilever fixed at the top of the base. At least one fourth of the vertical reinforcement in the stem shall extend to the top of the wall. The main reinforcement in the stem and the base shall be anchored each way from points of maximum stress by suitable embedment or hooks as specified in Section 8. Excessive bond stress in the toe reinforcement may be provided for by special anchorage as specified in Section 8.

9.3.9 Counterfort Abutments.—Counterforts shall preferably be located under or near points of concentrated load. The face walls and the back of the base slabs shall be designed as continuous slabs. For equally spaced counterforts, positive and negative moments of $1/12\ wl^2$ shall be assumed, provided that the end supports for the series of continuous slabs offers suitable restraint; otherwise $1/10\ wl^2$ shall be assumed for positive and negative moments.

The back of the base slab shall be designed for the weight of the superimposed material, plus the weight of the slab, less the reaction from the foundation.

Counterforts shall be designed as T-beams, and the reinforcement in the back of the counterfort shall be designed on the assumption that the moment arm of the resisting couple is equal to the horizontal distance from the center of the front wall to the center of the counterfort reinforcement, multiplied by the sine of the angle between the back of the counterfort and a horizontal plane. Vertical and horizontal stirrups, connecting the counterforts to the face wall and base slab, shall be designed to take respectively the vertical and horizontal earth pressures on one-half the clear space each side of the counterforts. Stirrups shall be anchored as near the exposed faces of the face wall, and as close to the bottom of the base as the requirements for protective covering permit.

The rear of the base slab shall be investigated for diagonal tension and bond stresses at the edge of the counterforts.

The toe projection shall be designed in accordance with the specification for cantilever abutments, above.

9.3.10 Abutments in Deep, Soft Soils.—When abutments occur in deep, soft soils without lateral resistance to horizontal thrust, provision shall be made to secure lateral stability by means of a rock or gravel fill, or otherwise; and backfill shall be made simultaneously behind and in front of the abutments. The backfill in front of the abutments shall be carried up to sufficient height to insure stability. In soils with no appreciable lateral stability for a depth of 6 feet or more below the proposed elevation of bottom of footings, a special design shall be made, which shall neglect the lateral resistance of such soil.

9.3.11 Reinforcement for Temperature and Shrinkage and Construction Convenience.—Except in gravity abutments, not less than one-eighth (0.125) square inch of horizontal reinforcement per foot of height shall be provided near exposed surfaces not otherwise reinforced, to resist the formation of temperature and shrinkage cracks.

Where bars are not otherwise provided in any plane of reinforcing sufficient to properly wire and hold to place the main reinforcement, additional bars of nominal section and spacing shall be provided for this purpose.

In mass concrete abutment and wing walls, reinforcing bars shall be used at the top and bottom of walls. The reinforcement shall consist of not less than 5 one inch square bars, arranged with 2 bars at the top of the wall and 3 bars at the bottom of the wall, in horizontal layers, symmetrically spaced.

9.3.12 Wing Walls.—Wing walls shall be of sufficient length to retain the roadway embankment to the required slope, grade and shoulder width, and to furnish protection against erosion. For ordinary materials, in the absence of accurate data, the slope of the fill shall be assumed as 1½ horizontal to 1 vertical and wing lengths computed on this basis. The earth spill around the wing walls shall not constrict the stream. Generally, it may intersect ordinary water elevation at the face of the abutments, except in the case of skeleton abutments, or abutments placed back in the banks for the purpose of reducing abutment costs at the expense of superstructure.

9.3.13 Drainage.—The filling material behind abutments and retaining walls shall be effectively drained by drain holes 6 inches square placed at suitable intervals not more than 12 feet apart, with tile outlets. At least one drain shall be provided for each pocket formed by counterforts. At the backs of the walls, provision shall be made for collection of the drainage by placing a layer of porous backfill above the openings, or by constructing box drains of rough lumber, filled with gravel. The latter system shall consist of vertical boxes above each drain hole, with horizontal lateral connections at the drain holes and above the drain holes, if necessary.

9.3.14 Edge Distance to Steel Bearings.—Steel bearing plates or castings, which distribute heavy concentrated loads on abutment or pier bridge seats, shall be located so that the edge of the bearing is not less than 6 inches from the edge of the masonry at the bottom of the coping. A light bar grillage shall, in general, be placed under all such bearings at an embedment depth of about 4 inches.

9.3.15 Anchor-Bolt Wells.—Anchor bolts set in concrete shall have provision for slight lateral adjustment at the time the superstructure is erected. The anchor bolts shall be set with the upper ends enclosed in creosoted wood fibre conduit of inside diameter about 2 inches larger than the bolt, from which concrete is temporarily excluded, by a wooden cap swabbed on the outside with a heavy coat of asphalt and one layer of fabric, properly lapped. The pipe shall project above the surface of the bridge seat about 1 inch, but shall be carefully tooled back to ¼ inch recess just before placing the shoes and grouting the openings around the bolts.

9.3.16 Expansion Keyways.—Keyways shall in general be provided in the concrete between bridge seats and abutment backwalls which are integral with the superstructure. The keyways shall be lined at the sides and ends with ¾ inch soft asphaltic felt. In multiple span bridges similar keyways shall be provided at the piers.

9.4 Retaining Walls.

9.4.1 Design Condition.—Retaining walls shall be designed to withstand earth pressure, including any live load surcharge, and the weight of the wall, in accordance with the general principles specified above for abutments.

Stone masonry walls shall be of the gravity type and plain concrete walls shall be of the semi-gravity type. Reinforced concrete walls may be of either the cantilever, counterforted, buttressed, or cellular types.

9.4.2 Expansion Joints.—Expansion joints shall be provided at intervals not exceeding 50 feet for either gravity or reinforced walls.

9.4.3 Drainage.—Adequate drainage of the filling material shall be provided as specified for abutment drainage.

9.4.4 Reinforcement for Temperature and Shrinkage.—Temperature reinforcement in retaining walls shall conform with the requirements for temperature reinforcement in abutments.

9.5 Piers.

9.5.1 Design Conditions.—Piers shall be designed to withstand the dead and live loads superimposed thereon; the forces due to stream current, floating ice and drift, and tractive forces, if any, at the fixed ends of spans. The effect on stability and foundation loading of wind forces, eccentric superstructure loading, and the buoyant force on the masonry in porous foundations shall be investigated, excepting those cases where it is evident that these effects are negligible. The minimum sectional width of a pier exposed to ice action shall, in general, be 3 feet.

9.5.2 Scour Protection.—Piers which are subject to scouring action or possible undermining shall be protected by placing the footing at a greater depth than the usual requirement, by rip-rap around the pier, by driving cofferdam sheeting below any probable scouring action, or by a combination of any of these methods.

9.5.3 Pier Noses and Ice Breakers.—In streams carrying heavy ice or drift, the pier nose shall be designed as an ice-breaker, or an independent ice breaker unit shall be constructed in front of the piers. Ice breakers shall be designed with a triangular nose having an angle of 90 degrees between faces, with a suitable batter for the purpose of raising and breaking cakes or sheets of ice. A 6"x6"x¾" steel angle, or other metal nosing shall be secured to the masonry by means of suitable anchorage. Other corners or edges exposed to ice action shall either be rounded, bevelled, or protected by metal edges.

9.5.4 Pier Footings.—The previous specifications for footings, as given in this Section, shall apply to pier footings. The dimensions for pier footings shall in general, be subject to one of the following conditions. The area of a footing shall be sufficient to keep the foundation pressure within the allowable limits, or to provide for piling without exceeding the allowable pile loads, with not less than the minimum spacing nor less than the minimum edge distance. The latter conditions shall apply where there is a possibility of piling being required after excavation and examination of the foundations. In some cases, the area and shape of the footings will be determined by the size of and shape of the pier, subject to necessary bridge seat dimensions, and pier batter, and the necessity for footing projections for stability.

9.5.5 Tubular Steel Piers.—Preferably tubular steel piers shall not be used and they shall never be used in locations where they will be subject to lateral earth pressure, nor in unstable foundation material. Steel tubes resting on gravel foundation shall be carried down to a depth not less than 8 feet below the permanent bed of the stream. Piling used in connection with tubular steel piers shall extend into the concrete filling not less than 6 to 8 feet above the bottom of the concrete, provided that satisfactory bearing thickness of concrete is provided above cut off. The concrete filling shall consist of Grade A concrete. The minimum diameter of steel cylinders used for piers shall be 4 feet. The minimum thickness of metal in the shells shall be 3/8 inch. Tubular steel piers shall be adequately braced to resist ice and drift.

9.5.6 Mass Concrete Piers.—Mass concrete piers shall have side batters of ¼ inch to ½ inch per foot, with the least batter for high piers. The body of the pier shall be adequately dowelled to the footing with not less than ¾ inch round bars at 18 inch centers. To prevent cracking from temperature and shrinkage stresses or from unequal settlement, three 1 inch square bars shall be placed in a horizontal layer near the top and similarly near the bottom of the body section. Large surfaces shall be further reinforced against shrinkage and temperature stresses by placing not less than 0.25 square inches of horizontal reinforcing steel, per foot of height, near each surface, with sufficient vertical bars to serve as spacers and support for the horizontal steel.

9.5.7 Pedestal Piers.—Pedestal piers shall consist of two or more concrete pedestals, or large concrete columns, which may or may not be connected by a transverse girder at the top. The pedestals may rest on separate pedestal footings, on a continuous reinforced concrete footing, or on a mass concrete pier, when transverse top girders are used. Analysis of the stresses in the pedestals and connecting girders shall be made on the assumption that the members form a part of a rigid frame, fixed at the base. Adequate reinforcement shall be provided for both positive and negative bending moments.

9.5.8 Rock-filled Timber Piers.—For secondary structures of limited life, timber crib piers may be constructed of hewn or sawed timbers, drift-bolted together and filled with rock. The minimum width of pier shall be 6 feet or not less than one fourth of the height from the stream bed to the bridge seat. The bridge seat shall preferably consist of a reinforced concrete cap not less than 18 inches thick. The crib shall be so designed that the loading will be carried directly by the timbers and not by the rock filling, and adequate bearing area on this intersecting crib timbers and also on the foundation soil shall be provided by the grillage system. If the cribs are

Rock-filled Timber Piers:—(*Con.*)

capped well below low water, with a concrete pier, this construction may be used for permanent construction. Such cribs shall be set firmly on suitable material excavated at least 3 feet below the permanent stream bed and back-filled around the cribs with coarse rock fill to adequately protect from scour and sliding.

9.5.9 Concrete Cribbing.—When the use of precast concrete cribbing units is considered for retaining wall purposes, the selection of type shall be subject to the approval of the engineer. In all cases, cribbing shall be placed on a properly prepared concrete base, to distribute pressure and to provide for leveling up and alignment.

SECTION 10—STRUCTURAL STEEL DESIGN

10.1 General Proportions and Clearances.

10.1.1 Spacing of Trusses and Girders.—Main trusses and girders shall be spaced a sufficient distance apart center to center, to be secure against overturning by the assumed lateral and other forces. For long span through trusses, the spacing shall be not less than one-twentieth of the span length. The spacing of through trusses and girders is subject to roadway clearances and curb widths, as referred to in Section 3 and shown in Figures 1 and 2.

10.1.2 Clearances.—No parts of a structure shall encroach on the space shown in the clearance diagrams, Figures 1 to 4 inclusive.

10.1.3 Effective Span.—For the calculation of stresses, span lengths shall be assumed as follows:

Beams and girders, distance between centers of bearings.
Trusses, distance between centers of end pins or of bearings.
Floorbeams, distance between centers of trusses or girders.
Stringers, distance between centers of floor beams.

10.1.4 Effective Depth.—For the calculation of stresses, effective depths shall be assumed as follows:

Riveted trusses, distance between centers of gravity of the chords.
Pin-connected trusses, distance between centers of chord pins.
Plate girders, distance between centers of gravity of the flanges but not to exceed the distance back to back of flange angles.

10.1.5 Depth Ratios.—Trusses preferably shall have a depth not less than 1/10 the span, plate girders a depth not less than 1/15 the span and rolled beams a depth of not less than 1/20 the span. Deck plate girders with compression flanges continuously stayed in a concrete slab, and with provision for concealment of deflection by means of a cambered concrete fascia, may have a depth not less than 1/20 span. If less depths than these are used, the sections shall be increased so that the maximum deflection will not be greater than if these limiting ratios had not been exceeded.

10.2 Proportion of Parts.

10.2.1 Unit stresses.—All structures shall be so proportioned that the sum of the maximum stresses, excepting as specifically noted for alternate and combined stresses, shall not exceed the allowable stresses given in Section 7.

10.2.2 Limiting Lengths of Members.

(a) Compression Members.—The ratio of unsupported length to the least radius of gyration shall not exceed 100 from main compression and stiffening members nor 120 for laterals and sway bracing. In proportioning the top chords of low trusses the unsupported length shall be assumed as the length between the rigid verticals.

(b) Tension Members.—For main riveted tension members the ratio of length to least radius of gyration shall not exceed 200.

10.2.3 Effective Area of Angles in Tension.—The effective area of single angles in tension shall be assumed as the net area of the connected leg plus 50% of the area of the unconnected leg. The effective area of a double angle tension member shall be assumed as 80% of the net area of the member unless the end details and connections are such that the individual angles are held against bending in both directions, in which case the full net area may be used. When the angles connect to separate gusset plates, as in the case of a double-webbed truss, the gusset plates shall be stiffened by diaphragms in the line of the connected angles or by tie plates extending to the ends of the angles if they are to be considered as offering such resistance to bending that the full net area can be used. When the angles are connected back to back on opposite sides of a single gusset plate, the support may be assumed to be sufficient to allow the use of the full net section.

Lug angles shall not be considered as effective in transmitting stress.

10.2.4 Net Section of Riveted Tension Members.—In calculating the required area of riveted tension members, net sections shall be used in all cases and, in deducting rivet holes, they shall be taken as 1/8 inch larger than the nominal diameter of the rivet.

The net section shall be the least area which can be obtained by deducting from the gross sectional area, the area of holes cut by any plane normal to the axis of the member and in addition deducting fractional parts successive holes if the stagger "s" is equal to or less than one-half of the gage "g". No additional deduction shall be made if "s" is equal to or greater than $\frac{g}{2}$. The fractional parts for intermediate values shall be that part of the hole represented by $\frac{g-2s}{g}$.

10.2.5 Proportioning Rolled Beams.—Rolled beams shall be proportioned by the moments of inertia of their sections. Proper allowance shall be made for any reduction in strength due to rivet holes in the tension flange or for any reduction in allowable stress due to the length of unsupported compression flange.

10.2.6 Compression Flanges of Beams and Girders.—The gross area of the compression flanges of beams and plate girders shall be not less than the gross area of the tension flanges.

The minimum thickness of outstanding flanges shall be not less than 1/12 of the outstanding width for girders and main compression members. For laterals and other secondary members this minimum thickness may be 1/14 of the outstanding width.

Compression Flanges of Beams and Girders.—(*Con.*)

The laterally unsupported length of the compression flanges of beams and girders shall not exceed 40 times the flange width. When the unsupported length of flange exceeds 12 times the flange width, the compressive stress in pounds per square inch shall not exceed

$$19000-250\frac{L}{b} \quad \text{(Maximum value, 16000 lbs.)}$$

where

L = length, in inches, of unsupported flange, between lateral connections or knee braces.

b = flange width in inches

10.2.7 Reversal of Stress and Allowance for Overload.—Members subject to reversal of stress during the passage of live load shall be proportioned as follows:

Determine the tensile and the compressive stresses and increase each by 50% of the smaller; then proportion the member so that it will be capable of resisting either increased stress. The connections shall be proportioned for the sum of the actual stresses. When the live load and dead load stresses are of opposite sign, only 70% of the dead load stress shall be considered as effective in counteracting the live load stress.

No counters will be permitted except in the middle panel of a truss having an odd number of panels. Counters shall consist of compression members, properly connected at the intersections, with a ratio of 1/r not more than 120.

No pin-connected member shall be subjected to reversal of stress.

10.2.8 Combined Stresses.

10.2.8.1 Axial and Bending. —Members subject to both axial and bending stresses shall be proportioned so that the combined fiber stresses, excepting wind and lateral stresses, will not exceed the allowable axial stress.

10.2.8.2 Stresses due to Lateral and Longitudinal Forces and Temperature.—In proportioning the various parts of the structure, provision shall be made for the following stress combination:

Dead Load
Live Load
Impact
Centrifugal Force
Lateral Force
Longitudinal Force
Temperature

The unit stresses given in Section 7 may be increased 25%, for the maximum combination of the loadings but the sections shall not be less than required for dead load, live load, impact and centrifugal force at the unit stresses specified.

10.2.9 Secondary Stresses.—Members and their details shall be proportioned to reduce secondary stresses to a minimum. In simple trusses without subdivided panels the secondary stresses due to deformation in any member whose width measured in the plane of flexure is less than one-tenth of its length need not be considered. When this ratio is exceeded, or where subdivided panels are used, the secondary stresses shall be computed.

In members designed for secondary stresses in combination with other stresses the specified allowable unit stresses may be increased 30% but the sections shall be not less than required for primary stresses.

10.3 Details of Design.

10.3.1 Accessibility of Parts.—The accessibility of all parts of a structure for inspection, cleaning and painting shall be insured by the proper proportioning of members and the design of their details.

10.3.2 Open Sections and Pockets.—Closed sections shall in general be avoided. Pockets or depressions which will retain water shall be avoided as far as possible and those which are unavoidable shall be provided with effective drain holes or preferably shall be effectively filled with water-proof material, with a sloped surface for drainage.

Details shall be arranged so that the retention of dirt, leaves or other foreign matter will be reduced to a minimum. Wherever angles are used, either singly or in pairs, they preferably shall be placed with the vertical legs extending downward.

10.3.3 Symmetrical Sections.—Main members shall be proportioned so that their neutral axes shall be as nearly as practicable in the center of the section. In general, the gravity axes of main truss and other important members, meeting to form a joint, shall intersect in a common point so as to avoid eccentricity of stress. In cases of unavoidable eccentricity the members affected thereby shall be proportioned and the connection details designed to resist the stresses produced.

10.3.4 Minimum Thickness of Metal.—The minimum thickness of structural steel shall preferably be 3/8 inch except for fillers, lattice bars and railings. Gusset plates shall in no case be less than 3/8 inch in thickness. Bars of latticed railings shall be not less than 1/4 inch nor less than 1/60 of the distance center to center of connecting rivets.

Metal subjected to marked corrosive influence shall be increased in thickness.

Cast Steel shall not be less than one inch and cast iron not less than 1¼ inch thick, except for filler blocks.

10.3.5 Compression Members.—In built compression members the center of gravity of the section shall be as near the center line of the member as practicable.

10.3.5.1 For Low Trusses.—For low trusses it shall be preferable to use two-rivet double lacing instead of a cover plate.

10.3.5.2 Cover Plates.—Cover plates of built compression members and cover plates on the compression flanges of plate girders shall have a minimum thickness of 1/40, and the web plates of compression members a minimum thickness of 1/30, of the transverse distance between the lines of rivets connecting them to the flanges. However, failing to meet this requirement, the width of plate between the connecting lines of rivets in excess of 40 times the thickness for cover plates and 30 times the thickness for web plates, shall not be considered as effective in resisting stress.

10.3.5.3 Forked Ends.—Forked ends on compression members will be permitted only when unavoidable. When used, a sufficient number of pin plates shall be provided to give each jaw the full strength of the compression member. At least one pair of these plates shall extend to the far edge of the tie plates, and another not less than 6 inches beyond the near edge of the tie plates.

10.3.6 Tie Plates.

10.3.6.1 On Compression Members.—The open sides of compression members shall be provided with lacing bars and shall have tie plates as near each end as practicable and at intermediate points where the lacing is interrupted. Compression members composed of two angles and a cover plate shall have, on their open sides, ties composed of short lengths of channel sections with the flanges riveted to the vertical legs of the angles.

10.3.6.2 On Tension Members.—Tension members composed of shapes shall have their separate segments connected by tie plates or by tie plates and lacing bars.

10.3.6.3 Thickness.—The thickness of the tie plates shall be not less than 1/50 of the distance between the connecting lines of rivets. Tie plates shall be connected by not less than 3 rivets on each side and in members having lacing bars the last rivet in the tie plate shall preferably also pass through the end of the adjacent bar.

10.3.6.4 Length.—For main compression members, the end tie plates shall have a length not less than 1½ times the perpendicular distance between the lines of rivets connecting them to the member, and the intermediate tie plates a length not less than that perpendicular distance. For main tension members the end tie plates shall have the length above specified for end tie plates on main compression members and the length of the intermediate tie plates shall be not less than ¾ the length specified for intermediate tie plates on compression members. In tension members whose elements are connected by tie plates only, the distance center to center of plates shall not exceed 3 feet.

For lateral struts and other secondary members the length of end and intermediate tie plates shall be not less than ¾ the perpendicular distance between the lines of rivets connecting them to the member.

10.3.7 Lacing Bars.—The lacing of compression members shall be proportioned to resist a transverse shear not less than that calculated by the formula:

$$S = 300\ A$$

where S = transverse shear in pounds

A = gross area of member in square inches

This shear shall be considered as divided equally among all stiffening parts in parallel planes, whether made up of continuous plates or of lattice. The stress in the individual lacing bar shall be taken as the component of the shear, in the direction of the bar, in case single lacing is used, and half that amount if double lacing is used. The size of the bar shall be determined by the column formula in which "L" shall be taken as the distance between the connections to the main sections.

The inclination of single lacing shall generally be about sixty degrees and for double lacing it shall be about forty-five degrees to the axis of the member. Furthermore, the maximum spacing of lacing bars shall be such that the ratio of length to radius of gyration (L/r) for the portion of a single flange between consecutive connections will be smaller than this ratio for the member as a whole.

Shapes of equivalent strength may be used instead of flats, subject to approval for appearance.

10.3.7.1 Minimum Width.—The minimum width of lacing bars shall be:

For 7/8 inch diameter rivets—2½ inches
For 3/4 inch diameter rivets—2¼ inches
For 5/8 inch diameter rivets—2 inches

Lacing bars having two rivets in each end shall be used for flanges 5 inches or more in width.

10.3.7.2 Minimum Thickness.—The minimum thickness of bars shall be 1/40 of the distance between end rivets in the case of single lacing and 1/60 of this distance for double lacing.

10.3.7.3 Double Lacing.—Double lacing, riveted at the intersections, shall be used when the perpendicular distance between rivet lines exceeds 15 inches.

10.3.8 Strength of Connections.—All connections shall be proportioned to develop not less than the full strength of the members connected provided that the full strength does not exceed the maximum computed stress by more than 50%, in which case the latter shall govern.

No connection, except for lacing bars and handrails, shall contain less than three rivets.

19.3.9 Splices.

10.3.9.1 In Compression Members.—Continuous compression members in riveted structures, such as chords and trestle posts, shall have milled ends and full contact bearing at the splices.

10.3.9.2 Proportioning.—All splices, whether in tension or compression, shall be proportioned to develop the full strength of the members spliced and no allowance shall be made for milled ends of compression members.

10.3.9.3 Location.—Splices shall be located as close to panel points as possible and, in general, shall be on that side of the panel point which is subjected to the smaller stress.

10.3.9.4 Arrangement.—The arrangement of the plates, angles or other splice elements shall be such as to make proper provision for the stresses in the component parts of the members spliced, not more than one pair of elements being spliced in any single plane, and field splices shall be so arranged as to permit the adjacent members to be readily and easily assembled and adjusted in the field.

In all splice plates not in direct contact with the parts they connect, the number of rivets on each side of the joint shall be in excess of the number which would otherwise be required for a contact splice to the extent of two extra transverse lines for each intervening plate.

10.3.10 Fillers.—Where indirect splices involve rivets carrying stress and passing through fillers, the fillers shall be extended beyond the splicing material and extension secured by additional rivets sufficient in numbers to develop the section of the filler. When the filler is less than ¼ inch thick the splicing material shall also be extended.

10.3.11 Gusset Plates.—Gusset or connecting plates shall be used for connecting all main members, except in pin-connected structures. In proportioning and detailing these plates the rivets connecting each member shall be located, as nearly as practicable, symmetrically with the axis of the member. However, the full development or the elements of the member shall be given due consideration. The gusset plates shall be of ample thickness to resist shear, direct stress and flexure acting on the weakest or critical section of maximum stress.

Re-entrant cuts shall be avoided as far as possible, but curvature of the gussets will be required for exposed work, as may be specified by the engineer, in which case the curved edges shall be neatly finished.

10.3.12 Effective Bearing Area.—The effective bearing area of a pin, bolt, or rivet shall be its nominal diameter multiplied by the thickness of the metal on which it bears.

10.3.13 Pins and Pin Plates.

10.3.13.1 Location.—Pins shall be located, with respect to the neutral axis of the members, so as to reduce to a minimum secondary stresses due to bending.

10.3.13.2 Pin Plates.—Pin plates shall be of sufficient thickness to provide the required bearing area upon the pin; they shall be as wide as the dimensions of the member will allow; and their length, measured from pin center to end, shall be at least equal to their width. Pin plates shall contain sufficient rivets to distribute their due proportion of the pin pressure to the full cross section of the member; only the rivets located within two lines drawn from the pin center toward the end of the member for tension members or away from the end for compression members and inclined at 45 degrees to the axis of the member shall be considered effective for this purpose. In the case of members composed of web plates and flange angles (with or without a cover plate) there shall be at least one outside pin plate covering the vertical legs of the flange angles.

At the ends of compression members at least one pair of pin plates shall extend not less than 6 inches beyond the near edge of the tie plate.

All pin-connected compression members shall be provided with hinge plates having a minimum thickness of 3/8 inch.

10.3.13.3 Net Section at Pins.—Pin-connected riveted tension members shall have a net section, both through the pin hole and back of the pin hole, at least 25% in excess of the net section of the body of the member.

10.3.13.4 Pins.—Pins shall be proportioned for the maximum shears and bending moments produced by the stresses in the members connected. If there are eye-bars among the parts connected, the diameter of the pin shall be not less than 2/3 of the width of the widest bar attached.

Pins shall be of sufficient length to secure a full bearing of all parts connected upon the turned body of the pin. They shall be secured in position by hexagonal chambered nuts. Where the pins are bored, through rods with chambered cap washers may be used. In general, malleable castings conforming to the requirements of Section 11, Division 12, shall be used for pin nuts. Pin nuts shall be secured by cotters in the screw ends. Connecting members shall be so designed and arranged that the maximum moments on pins shall be a minimum.

10.3.14 Rivets.

10.3.14.1 Effective Diameter.—In proportioning rivets, the nominal diameter of the rivet shall be used.

10.3.14.2 Size.—Rivets shall be of the size specified but generally shall be 3/4 inch or 7/8 inch in diameter. 5/8 inch rivets shall not be used in members carrying calculated stress except in 2½ inch legs of angles and in flanges of 6 inch and 7 inch beams and channels.

The diameter of rivets in angles carrying calculated stress shall not exceed one-fourth of the width of the leg in which they are driven. In angles whose size is not so determined 5/8 inch rivets may be used in 2 inch legs, 3/4 inch rivets in 2½ inch legs and 7/8 inch rivets in 3 inch legs.

In no case, except in handrails, shall structural shapes be used which do not admit the use of 5/8 inch diameter rivets.

In general, all rivets and bolts in any single plane of a member shall be of the same size, to avoid a second routing through the shop.

10.3.14.3 Minimum Rivet Size in Thick Metal.—Metal more than ¾ inch in thickness is required to be subpunched and reamed, or drilled from the solid, as specified in Division 5, Section 20, and the minimum size to avoid damage to punches, which may be used in metal more than ¾ inch thick shall be as follows:

Metal 1-1/8 inch thick	1 inch diameter rivets
Metal 1 inch thick	¾ inch diameter rivets

The maximum thickness of metal to be punched shall not exceed the nominal diameter of the rivet.

10.3.14.4 Pitch.—The minimum allowable distance between centers of rivets shall be three times the diameter of the rivet but preferably shall be not less than the following:

For 1 inch diameter rivets—3½ inches
For 7/8 inch diameter rivets—3 inches
For 3/4 inch diameter rivets—2½ inches
For 5/8 inch diameter rivets—2¼ inches

The maximum allowable pitch in the line of stress shall not exceed twice the minimum spacing as tabulated above, or 16 times the thickness of the thinnest outside plate or angle connected, except in angles having two gage lines with rivets staggered where the pitch in each line may be twice the above with a maximum of 10 inches.

In webs of members composed of two or more plates in contact, the rivets shall be spaced not more than three times the minimum spacing as tabulated above between centers in gage and pitch, provided such rivets serve no other purpose than to hold the plates in close contact. Tension members composed of two angles in contact shall be stitch riveted using a pitch not greater than four times the minimum spacing as tabulated above.

10.3.14.5 Pitch in Ends of Compression Members.—In the ends of built compression members the pitch of rivets connecting the component parts of the member shall not exceed the minimum spacing as tabulated above for a length equal to 1½ times the maximum width of the member until the maximum spacing is reached. In angles having two lines of staggered rivets in one leg, the pitch on each line may be twice the minimum spacing specified above but not greater than that allowed for the body of the member.

10.3.14.6 Edge Distance of Rivets.—The minimum distance from the center of any rivet to a sheared edge shall be:

For 1 inch diameter rivets—1¾ inches
For 7/8 inch diameter rivets—1½ inches
For 3/4 inch diameter rivets—1¼ inches
For 5/8 inch diameter rivets—1-1/8 inches

The minimum distance from rolled or planed edges, except flanges of beams and channels, shall be:

For 1 inch diameter rivets—1½ inches
For 7/8 inch diameter rivets—1¼ inches
For 3/4 inch diameter rivets—1-1/8 inches
For 5/8 inch diameter rivets—1 inch

Edge Distance of Rivets.—(*Con.*)

The maximum distance from any edge shall be eight times the thickness of thinnest outside plate, but shall not exceed 5 inches.

10.3.14.7 Long Rivets.—Long rivets subjected to calculated stress and having a grip in excess of 4½ diameters shall be increased in number at least one per cent for each additional 1/16 inch of grip. If the grip exceeds 6 times the diameter of the rivet, specially designed rivets shall be used.

10.3.14.8 Rivets in Tension.—Rivets in direct tension shall, in general, not be used. However, where so used their value shall be one half that permitted for rivets in shear. Countersunk rivets shall not be used in tension.

10.3.15 Bolts.—Unless specifically authorized, bolted connections will not be permitted. Bolts, when used, shall be unfinished or turned as specified. Bolts in tension shall have double nuts.

10.3.15.1 Unfinished Bolts.—Unfinished bolts shall be standard bolts with hexagonal heads and nuts. Bolts transmitting shear shall be threaded to such a length that not more than one thread will be within the grip of the metal. The bolts shall be of lengths which will extend entirely through their nuts, but not more than ¼ inch beyond them. The diameter of the bolt holes shall be 1/16 inch greater than the diameter of the bolts used.

10.3.15.2 Turned Bolts.—Holes for turned bolts, shall be carefully reamed or drilled and the bolts turned to a driving fit by being given a finishing cut. The threads shall be entirely outside of the holes and the heads and nuts shall be hexagonal. Approved nut-locks shall preferably be used on all bolts. When nut locks are not required, round washers having a thickness of ¼ inch shall be placed under the nuts.

10.3.16 Bars.—Bars and rods with screw ends shall be upset to provide a cross sectional area at the root of the thread which shall exceed the net section of the body of the member by at least 15 per cent.

Sleeve nuts shall not be used

10.3.17 Expansion.—Provision for expansion and contraction, to the extent of 1/8 inch for each 10 feet of span, shall be made for all bridges. Expansion ends shall be firmly secured against lifting or lateral movement.

10.3.18 Bearings and Shoes.

10.3.18.1 Expansion Bearings.—Spans of less than 80 feet may be arranged to slide upon metal plates with smooth surfaces. Spans of 80 feet and over shall be provided with rollers or rockers, or with the special sliding bearings described below. Neither rollers nor rockers shall be used for expansion bearings at the top of trestle posts.

10.3.18.2 Fixed Bearings.—Fixed bearings shall be firmly anchored.

10.3.18.3 Hinged or Pin Bearings.—Spans of 80 feet and over shall have hinged or pin bearings at both ends. The pedestals or shoes shall be so designed that all loads will act through the end pins, which will be located directly over the geometrical center of the bearing.

10.3.18.4 Rollers.—Expansion rollers shall be not less than 4 inches in diameter for span lengths of 100 feet or less and this minimum shall be increased not less than 1 inch for each additional 100 feet of span, and proportionally for intermediate lengths, and such rollers shall usually be segmental in type and of as large effective diameter as permissible, and designed to avoid the possibility of interlocking. They shall be connected together by substantial side bars and shall be effectually guided so as to prevent lateral movement, skewing or creeping. The rollers and bearing plates shall be protected from dirt and water as far as possible and construction shall be such that water will not be retained and that the roller nests may be inspected and cleaned with the minimum difficulty.

10.3.18.5 Rockers.—Pin bearing expansion rockers shall, in general, be of cast steel.

10.3.18.6 Sliding Expansion Bearings.—Sliding plates for the expansion bearings of spans of 80 feet and over shall be of Class A Bronze conforming to the requirements of Section 11, Division 12. These plates shall be chamfered at the ends and shall be held securely in position, usually being inset into the metal of the pedestals and sole plates. Provision shall be made against any accumulation of dirt which will obstruct their free movement.

10.3.18.7 Pedestals and Shoes.—Pedestals and shoes shall be designed to secure rigidity and stability and to distribute the reaction uniformly over the entire bearing area. They shall be made of cast steel or structural steel. The bottom bearing widths shall not exceed the top bearing widths by more than twice the depth of pedestal and when involving pin bearings, this depth shall be measured from the center of pin.

Where built pedestals and shoes are used, the web-plates and the angles connecting them to the base plates shall be not less than ½ inch thick. If the size of the pedestal permits, the webs shall be rigidly connected transversely.

10.3.18.8 Inclined Bearings.—For spans on an inclined grade without pin or hinged bearings, the sole plates shall be beveled so that the substructure bridge seats will be level.

10.3.18.9 Anchor Bolts.—Anchor bolts shall be not less than 1¼ inch diameter and shall extend not less than 12 inches into the masonry. Plate washers and nuts shall be provided at the lower end of anchor bolts to engage the masonry, and washers shall be provided under the nuts at the upper end. Anchor bolt holes in masonry and sole plates shall be 3/8 inch larger in diameter than the anchor bolts, except that in addition at expansion ends the holes in the sole plates shall be slotted sufficient to permit full expansion either way.

Anchor bolts subject to tension, as in the column bases of trestle bents and towers, shall be designed to engage a mass of masonry which will secure a resistance equal to 1½ times the calculated uplift.

10.3.19 Blast Protection Plates.—Members of bridges spanning steam railroad tracks, and which have a clearance over the tracks of less than 20 feet, shall be protected from the blast of locomotives by cast iron blast plates at least 4 feet wide located over the center line of each track and so supported that they may be replaced conveniently.

10.3.20 Concrete Encasement.—Concrete encasement of structural steel shall be reinforced with wire mesh or rods, with provision for mechanical anchorage to the structural steel. Encasement shall preferably be cast in forms. When bottom flanges or webs of steel girders are encased in gunite, the design shall permit gunite to be applied at right angles to the surface. Structural steel, to be encased, shall be painted with one light shop coat of painting mixture No. 1.

10.4 Floor System.

10.4.1 Floorbeams.—Floorbeams preferably shall be at right angles to the trusses or main girders and shall be rigidly connected thereto. In general, floorbeam connections shall be located above the bottom chord and, in riveted work, the bottom chord lateral system shall engage both the bottom chord and the floorbeam. Floorbeam connections to pin connected trusses preferably shall be above the bottom chord pins but, if located below, the vertical posts shall be extended below the pins to secure rigid connections to the floor beams.

10.4.2 End Floorbeams.—End floorbeams shall be provided in all truss and girder spans with intermediate floorbeams. End floorbeams preferably shall be designed to permit the use of jacks for the future lifting of the superstructure, under which condition the specified unit stresses shall not be exceeded by more than 50 per cent.

End floor beams and bridge seats shall be arranged to permit future painting of the sides of the beams adjacent to the abutment backwalls.

10.4.3 Stringers.—Steel stringers preferably shall be riveted between the floorbeams, with end connections to the floorbeam webs.

10.4.4 End Connections for Floorbeams and Stringers.—The end connection angles of floorbeams and stringers shall be not less than 7/16 inch in thickness. When milled ends are required, the thickness of connection angles shall be 1/16 inch greater than for connection angles not required to be milled. Except in cases of special end floorbeam details, end connections for floorbeams and stringers shall be made with two angles at each end. End connection angles shall develop the full depth of the webs by having a length as great as the flanges will permit. Bracket or shelf angles which may be used to furnish support during erection shall be located with ½ inch clear space below the lower flanges of the stringers. In the preparation of end connection details, special care shall be exercised to provide ample clearance for the driving of field connection rivets.

The use of any type of floorbeam hanger which does not prevent all rotation or longitudinal motion of the floorbeam, will not be permitted.

Stringer connection angles shall be shop riveted to the stringer webs at one end. At the opposite end, holes shall be drilled through the webs and field connections riveted after the trusses have been swung from the falsework.

10.4.5 Expansion Joints.—To provide for expansion and contraction movement, suitable floor expansion joints shall be provided at the expansion end of all spans and at other points where they may be required. Apron plates when used, shall be designed to properly bridge the joint and to prevent, as far as possible, the deposit of roadway debris upon the bridge seats.

10.5 Bracing.

10.5.1 Design of Bracing.—Lateral, longitudinal and tranverse bracing shall be composed of angles or other shapes offering resistance to deformation when subjected to compressive stress, and shall have riveted connections. In general, bracing shall consist of either a double system or a single system of diagonal members with transverse members. The diagonals in each system shall be proportioned to carry the total lateral stress in tension or compression, the transverse floorbeams acting as transverse members. When a double system of bracing is used, both systems may be considered simultaneously effective.

All intersections of lateral and sway bracing shall be riveted to add rigidity and prevent deformations, and, in general, shall be rigidly connected at intersections with stringers.

10.5.2 Lateral Bracing.—Bottom lateral bracing shall be provided in all bridges except I-beam spans, from which it may be omitted. Top lateral bracing shall, in general, be provided in deck truss spans and in through spans.

The minimum sized angle to be used in lateral bracing shall be 3½ x 3 x 3/8 inches. Not less than three rivets through each end of the angles shall be used at the connections. Laterals shall have sufficient rivets to develop the strength of the member, but not to exceed the calculated maximum stress by more than 50%. Bottom laterals for deck trusses when the underclearance depth to high water is 2 feet or less or when there is danger of either log or ice jams shall be of the double plane or box section and the slenderness ratio for any member shall not exceed 120. Lateral bracing for compression chords shall preferably consist of either two or four angle latticed sections; and so designed as to effectively engage both flanges of the chords. Lateral bracing shall have concentric connections to chords at end joints, and preferably throughout. The connections between the lateral bracing and the chords shall be designed to avoid, as far as possible any bending stress in the truss members.

10.5.3 Portal and Sway Bracing.—Through truss spans shall have portal bracing, of the two plane or box type, rigidly connected to the end post and top chord flanges, and constructed as deep as the minimum clearance will allow, with batters at the knee connections to the end post. The portal bracing shall be designed to take the full end reaction of the top chord lateral system and the end posts shall be designed to transfer this reaction to the truss bearings. Portals shall preferably be of the A type or modification of same.

Deck truss spans shall have adequate sway bracing at the ends and at all intermediate panel points. This bracing shall occupy the full depth of the trusses below the floor system. The bracing shall be proportioned to transfer the end reaction of the top lateral system to the substructure.

Through truss spans shall have sway bracing at each intermediate panel point. Sway bracing shall be of ample strength to transfer one-half of the wind pressure to the leeward truss.

10.5.4 Cross Frames.—Deck plate girder spans shall be provided with cross frames at each end proportioned to resist all lateral forces, and shall have intermediate cross frames or reinforced concrete diaphragms at intervals not exceeding 15 feet. The frames shall be connected to the outstanding legs of the stiffener angles and to the girder flanges. Concrete diaphragms shall be connected to the girder web by rods extending through the diaphragms at the webs, and bolted to the latter.

10.5.5 Low Truss Spans.—The vertical truss members of low truss spans shall be proportioned to resist a lateral force, applied at the top chord panel points of the truss, determined by the following equation:

$$R = 150(A+P)$$

Where R = lateral force in pounds

A = area of cross section of chord in square inches

P = panel length in feet.

Connections shall be capable of developing the lateral flexure of the posts, considering the latter as fixed at the top of the floor beams, and assuming a unit stress increase of 50% for the connections.

This rigidity may be secured by inside knee braces and gussets connecting the posts and the floor beams. Outrigger brackets attached to the vertical posts on the outside of the trusses shall not be used.

10.5.6 Through Girder Spans.—Through plate girder spans shall be stiffened against lateral deformation by means of gusset plates, or knee braces with solid webs, attached to the stiffener angles and floorbeams. If the unsupported length of the inclined edge of the gusset plate exceeds 60 times its thickness, the gusset plate shall have stiffener angles riveted along its edge. These braces generally shall extend to the clearance line and preferably shall be spaced not farther apart than 15 feet.

10.6 Plate Girders.

10.6.1 Proportioning.—Plate girders shall be proportioned either by assuming the flanges to be concentrated at their centers of gravity or by the moment of inertia of the net section. In the former case 1/8 of the gross area of the web is available as net flange area but the effective depth shall not be assumed to be greater than the distance back to back of flange angles. For girders having unusual cross sections the moment of inertia method shall be used.

10.6.2 Flange Sections.—The gross section of the compression flange shall be not less than the gross section of the tension flange. The compression flange shall be stayed against lateral deflection at intervals not exceeding 12 times its width. The flange angles shall form as large a portion of the gross area of the flange as practicable. Side plates shall not be used except when flange angles exceeding 1 inch in thickness would otherwise be required. When flange cover plates are used, at least one plate on the top flange shall extend the full length of the girder. Any additional flange plates shall extend not less than 18 inches beyond its theoretical end, and there shall be a sufficient number of rivets at the ends of each plate to develop its full stress value before the theoretical end of the next outside plate is reached. Flange cover plates shall be equal in thickness, or shall diminish in thickness from the flange angles outward. No plate shall have a thickness greater than that of the flange angles.

10.6.3 Web Plates.—Web plates shall be proportioned for both the vertical and horizontal shearing stresses. Splices in web plates shall be avoided as far as possible, but, when used, they shall be designed to develop the full value of the web plate for both bending and shearing stresses. The thickness of web plates, except when encased in concrete, shall not be less the $1/20\sqrt{D}$, where D represents the distance between the flanges in inches.

10.6.4 Flange Rivets.—The number of rivets connecting the flange angles to the web plates shall be sufficient to develop the increment of flange stress transmitted to the flange angles, combined with any load that is applied directly to the flange. For electric railways one wheel load, when applied directly to the flange, by means of cross ties, shall be assumed to be distributed uniformly over a length of 3 feet.

10.6.5 Flange Splices.—Splices in flange members shall not be used except by special permission of the Engineer. Two members shall not be spliced at the same cross section and, if practicable, splices shall be located at points where there is an excess of section. The net section of the splice shall exceed by 10 per cent, the net section of the member spliced. Flange angle splices shall consist of two angles, one on each side. Splice angles shall be fitted to secure close contact with the material spliced. Sheared edges of splice angles shall be given a smooth finish.

10.6.6 Web Splices.—Web plates shall be symmetrically spliced by plates on each side. The splice shall be equal in strength to the web in both shear and moment. There shall be at least two rows of rivets on each side of the joint.

10.6.7 End Stiffeners.—Plate girders shall have stiffener angles over end bearings, the outstanding legs of which shall be as wide as the flange angles will allow and shall fit tightly against them. These end stiffeners shall be proportioned for bearing on the outstanding legs of the flange angles, no allowance being made for the legs fitted to the fillets of the flange angles. End stiffeners shall be arranged to transmit the total end reaction and to distribute it over the bearings. They shall not be crimped and the connection to the web shall contain a sufficient number of rivets to transmit the entire reaction.

10.6.8 Intermediate Stiffeners.—Intermediate stiffener angles shall be riveted in pairs to the web of the girder. The outstanding leg of each angle shall have a width of not more than 16 times its thickness and not less than 2 inches plus 1/30 of the depth of the girder. Intermediate stiffeners shall be spaced at intervals not exceeding:

(a) 6 feet;
(b) The depth of the web;
(c) The distance given by the formula,

$$d = \frac{t}{40}(12000 - s)$$

where d = distance between rivet lines of stiffeners, in inches.
t = thickness of web, in inches.
s = web, shear, in pounds per square inch, at the point considered.

When the depth of the web between the flange angles or side plates is less than 60 times the web thickness, intermediate stiffeners may be omitted. Intermediate stiffener angles shall be placed at points of concentrated loading and shall be designed to transmit the reactions to the girder web. Such stiffeners shall not be crimped. Other intermediate stiffeners may be crimped unless otherwise specified.

10.6.9 Ends of Through Girders.—The upper corners of through plate girders, where exposed, shall be neatly rounded to a radius consistent with the size of the flange angles and the vertical height of the girder above the roadway. The first flange plate or a plate of the same width will be bent around the curve and continued to the bottom of the girder. In a bridge consisting of two or more spans the top flanges shall be curved down with a taper at the ends and the corners neatly rounded.

10.6.10 End Bearings.—End bearings of girders on masonry shall be raised above the bridge seat by metal pedestals or plates a height of at least 2 inches.

10.6.11 Sole and Masonry Plates.—Sole and masonry plates shall each be not less than ¾ inch thick.

10.6.12 Camber.—In general, camber will not be required in plate girders except for long spans or special conditions. When used, it shall be specified on the plans. In girders which are web-spiced at two or more symmetrical points, and do not have a full depth concrete fascia, camber shall always be provided to the extend specified, but not, in general, less than 1/600 of the span.

10.7 Trusses.

10.7.1 Main Features.—Preference will be given to trusses with single intersecting web members or other forms of trusses possessing the least ambiguity in computed stresses and the greatest elements of serviceability. Adjustable members in any part of the structure shall be avoided. Members shall be symmetrical about the central planes of trusses and all parts shall be so designed that they can be inspected, cleaned and painted. Through riveted and pin-connected spans will generally have inclined end posts. Low truss spans shall be of the riveted type. In low truss spans, laterally unsupported hip joint or "flying hips" shall be avoided.

10.7.2 Top Chords and End Posts.—Top chords and end posts of low and through truss spans shall be made usually of two side segments with tie plates and lacing on the top and bottom. Where the shape of the truss permits, compression chords shall be built continuous, with splices located as near the panel points as possible and preferably on the side subjected to the smaller stress.

Top Chords and End Posts.—(*Con.*)

Top chords of deck trusses subjected to direct loading shall be designed for the cross bending occasioned by the dead, live and impact loads of the floor system, in addition to the direct chord stresses, and all top chord splices shall be proportioned for these stresses and any shearing stresses they may receive.

The top chord sections of low truss spans shall be so proportioned that the radius of gyration about the vertical axis of the member shall be at least 1½ times the radius of gyration about the horizontal axis.

10.7.3 Bottom Chords.—Bottom chords of riveted trusses shall be made of symmetrical section and splices shall generally be located near panel points and on the side farthest away from the center of the span.

10.7.4 Gusset Plates.—The thickness of gusset plates connecting the chords and web members of the truss shall be proportionate to the stress to be transferred, but shall be not less than 3/8 inch.

10.7.5 Working Lines and Gravity Axes.—For compression members formed of side segments and a cover plate, the working line shall coincide as nearly as practicable with the gravity axis of the section. For symmetrical sections, the working line shall coincide with the gravity axis. For two-angle bottom chord or diagonal members the working line may be taken as the gage line nearest the back of the angle.

10.7.6 Camber.—The length of members of truss spans shall be such that the camber will be equal to the deflection produced by the combined dead and full live loads without impact.

10.7.7 Rigid Members in Pin-Connected Trusses.—Pin-connected trusses shall have stiff riveted members in the first two main panels of the bottom chords at each end of the span, and all web members performing the function of suspenders shall be of stiff riveted construction, with a ratio of 1/r not more than 140.

10.7.8 Web Members.—Tension web members not subject to reversal shall be designed as struts with an 1/r ratio not exceeding 140.

All other web members shall have an 1/r ratio not less than 100, and such a ratio as will permit of an economical use of the metal.

10.7.9 Eye-Bars.—Eye-bar heads shall have a cross sectional area through the center of the pin hole exceeding that of the body of the bar by at least 40 per cent. The net section adjacent to the head shall be not less than that of the main body of the bar. The thickness of the bar shall be not less than 1/8 of the width and not greater than 2 inches. The form of the head shall be submitted to the Engineer for approval before the bars are made. The diameter of the pin shall be not less than 2/3 of the width of the widest bar connected.

10.7.10 Packing Eye-Bars.—The eye-bars of a set shall be packed symmetrically about the central plane of the truss and as nearly parallel as practicable, but in no case shall the inclination of any bar to the plane of the truss exceed 1/16 inch per foot. Bars shall be packed as closely as practicable and held against lateral movement, but they shall be arranged so that adjacent bars in the same panel will be separated by at least ¾ inch to permit painting. All intersecting diagonal bars not far enough apart to clear each other at all times shall be well clamped together at intersections. Removable steel filling rings shall be provided, when required, to prevent lateral movements of eye-bars or other members connected upon pins.

10.7.11 Diaphragms.—Diaphragms shall be provided in the trusses at the end connections of all floorbeams and at points of transfer of dead concentrations. In general, such diaphragms shall extend down to the bottom flange of the floor beam and for at least two rivet spaces above the top flange. The gusset plates engaging the pedestal pin at the end of a truss shall be rigidly connected by a diaphragm. Similarly, the pedestal webs shall, where practicable, be connected by a diaphragm. A diaphragm shall be provided between gusset plates engaging main members whenever the end tie plate is located at a distance of 4 feet or more from the point of intersection of the members. In general, the web of this diaphragm shall be located in the plane of the latticed flange.

10.7.12 Sole and Masonry Plates.—Sole and masonry plates supporting trusses and column shall each have a thickness of not less than ¾ inch. The bottom chords of trusses shall be raised above the bridge seat at least 2 inches by the use of metal plates or pedestals.

10.8 Viaducts.

10.8.1 Type.—Viaducts shall consist usually of alternate tower spans and free spans of plate girders or riveted trusses supported on trestle towers. However, in viaducts having a column height less than 35 feet, trestle bents may alternate with the towers. The bents and towers shall generally be of reinforced concrete. In viaducts requiring freedom of waterway and in structures having a less total column height than 20 feet, the number of intermediate trestle bents may be increased to two. Ample rigidity shall be secured in the attachment of the superimposed spans to the column caps of the bents.

10.8.2 Bents and Towers.—Each trestle bent shall be composed preferably of two main supporting columns. Towers shall be composed of two bents rigidly braced and strutted both longitudinally and transversely.

10.8.3 Single Bents.—For viaduct spans supported on single bents, the bents shall generally be pin-connected to their base sections, or they shall be designed to resist bending.

10.8.4 Batter.—Columns generally shall have a transverse batter of not less than 1 horizontal to 12 vertical.

10.8.5 Depth of Girders.—The depths of plate girders in viaducts preferably shall be uniform.

10.8.6 Girder Connections.—Girders of tower spans shall be fastened at each end to the tops of the columns or to the cross girders. Girders between towers shall have one end riveted, and shall be provided with an effective expansion joint at the other end. No bracing or sway frame shall be common to abutting spans.

10.8.7 Bracing.—Towers shall be thoroughly braced both transversely and longitudinally, with a double system of stiff tension diagonals having riveted connections. Longitudinal and transverse struts shall be placed at caps and bases and at all intermediate panel points. All bracing connections shall be made by gusset plates, and bracing in adjacent planes shall connect at or near a common point in the intersection of such planes. Column splices generally shall be located close to and above the panel points of the bracing. Horizontal diagonal bracing shall be provided at the tops and bases of towers and at least at all intermediate panel points of the lateral bracing where the tower columns are spliced.

Provision shall be made in column bearings for expansion of the tower bracing. The struts at the base of towers shall be strong enough to slide the movable shoes with the structure unloaded. The co-efficient of friction shall be taken at 0.3. The slotting of holes shall be in such directions and locations as will permit freedom of motion both longitudinally and laterally.

10.8.8 Sole and Masonry Plates.—Sole and masonry plates shall each be not less than ¾ inch thick.

10.8.9 Anchorage.—Viaduct bents preferably shall have a sufficient spread at the base to prevent tension in any windward leg. When this is impracticable, the column anchorages shall be designed to safely resist not less than 1½ times the calculated uplift.

10.9 Cantilever Superstructures.—Cantilever deck plate girder superstructures shall be designed as specified under section 8, Article 8.17.

10.10 Approval of Plans.—The construction plans shall consist of shop detail, erection and other working plans showing details, dimensions, sizes of material and other information and data necessary for the complete fabrication and erection of the metal work. Approval of the construction plans shall be secured before fabrication of steel work is commenced.

The Contractor shall furnish the Engineer with such blue print copies of the plans as may be required for approval and construction purposes and upon completion of the work, approved tracings or negatives of the original plans shall be supplied to the Engineer. No deviation from the approved plans will be permitted without the written order of the Engineer.

SECTION 11—DESIGN OF TIMBER STRUCTURES

11.1 Bolts.—Bolts of diameters not exceeding 1 inch preferably shall be spaced not closer than 6 inches center to center, not less than 6 inches from the center of the bolt to the end of any timber, and not less than 2½ inches from the center of the bolt to the side of any timber. These distances preferably shall be increased for bolts larger than 1 inch in diameter. Inclined bolts through timber preferably shall be provided with beveled cast washers to eliminate the cutting of inclined daps in the timber.

11.2 Washers.—A washer shall be used under all bolt heads and nuts which would otherwise come in contact with wood. Washers may be cast or plate and shall be designed to prevent excessive crushing of the wood or flexture of the washer when the bolts are tightened. For bolts in important locations, such as joints and splices, and for rods, the washers shall be designed to develop the bolt or rod in tension at the unit bearing stresses specified for compression perpendicular to the grain of timbers.

A standard circular washer shall be used under the heads of all lag screws.

11.3 Columns and Posts.—No column shall have an unsupported length greater than 30 times its least dimension. The strength of built-up columns composed of two or more sticks bolted together, either with or without packing blocks, shall be considered as equal to the combined strength of the single sticks, each considered as an independent column.

11.4 Pile and Framed Bents.

11.4.1 Pile Bents.—Pile bents generally shall not exceed 40 feet in height. Pile bents over 10 feet high shall be sway braced transversely with diagonal braces on each side of the bent, and shall be adequately braced longitudinally. In general, pile bents shall contain not less than 4 piles each and the outside piles preferably shall be battered. The piles shall be designed for safe bearing and for column action.

11.4.2 Framed Bents.—Framed bents may be supported on piles, concrete pedestals, mud sills, or timber cribs. All bents shall be sway braced transversely and adequate provision shall be made for longitudinal bracing. In general, framed bents shall contain not less than 4 posts each and the outside posts of the bent shall generally be battered. The posts shall be designed as columns.

11.4.3 Sills and Mud Sills.—Mud sills, and all sills which are to be located in close proximity to the ground surface shall be given a thorough preservative treatment. When possible, the design shall be such as to insure that sills will be located clear of all earth so that there may be a free circulation of air around them, or else located entirely below low water level. Sills shall be fastened to mud sills or piles with drift bolts of not less than ¾ inch diameter and extending into the mud sills or piles at least 6 inches. Sills shall be fastened to pedestals with dowels of not less than ¾ inch diameter, set in the pedestals and extending into the sills at least 6 inches. Sills shall be not less in size than 10 by 12 inches, with the 12 inch dimension vertical.

Posts shall be fastened to sills by dowels of not less than ¾ inch diameter extending at least 6 inches into posts and sills, or by drift bolts of not less than ¾ inch diameter driven diagonally through the base of the posts and extending at least 9 inches into the sill, or preferably with ½ inch plates on each side of the sills. Posts shall never be fastened direct to pedestals but shall always rest on sills.

11.4.4 Caps.—Timber caps shall be not less in size than 10 by 12 inches with the 12 inch dimension vertical. They shall be fastened with drift bolts of not less than ¾ inch diameter extending at least 9 inches into the piles or posts, or preferably with ½ inch steel plates on each side of the post and cap.

11.4.5 Bracing.—Single story bracing shall not exceed 20 feet in height. The minimum size of transverse sway braces shall be 3 by 8 inches. All bracing shall be bolted through the piles, posts or caps at the ends; at intermediate intersections it shall be bolted with one bolt and spiked in addition. In all cases, spikes shall be provided in addition to bolts. The bolts used shall be of not less than ¾ inch diameter.

11.4.6 Pile Bent Abutments.—Pile bent abutments shall be adequately braced or anchored to resist earth pressure. Bulkhead plank shall be not less than 4 inches thick and shall always be treated. It shall be fastened to the piles with spikes, the length of which shall be at least 3 inches greater than the thickness of the plank. When pile bents are used as abutments, the distance from the grade to bottom of backing plank preferably shall be not more than 7 feet.

11.5 Trusses.

11.5.1 Joints and Splices.—Joints shall be detailed to shed water to the maximum degree practicable. Joints and splices shall be designed to develop the computed stresses in the members connected and generally to develop the full strength of those members. Posts or struts bearing against the sides of timber members preferably shall be provided with metal end bearings. Joints involving end bearing on inclined surfaces shall be avoided, preference being given to square-cut ends of timbers bearing against blocks. No daps in chords for butt blocks shall be less than ¾ inch deep.

In end-shoe plates and tension spliced plates the bearing faces of lugs or tables shall have a smooth even surface. If rolled plates or bars are used for tables, they shall be milled or cold sawed on the bearing edges. The bolts holding the lugs or tables in the notches in the timber shall be placed as near to the lugs or tables as possible. No metal lug or table shall have a bearing face less than ¾ inch thick. In details of end-shoes employing lugs or tables set in the lower chord, the spacing of such lugs or tables shall be arranged so that no lug or table occurs directly under the end of the end post. The end joint between lower chord and end post shall provide definite lines of ac-

Joints and Splices.—(*Con.*)

tion and shall be a simple joint depending for its strength upon one type of detail. When inclined bolts are used to connect the main members of an end joint, such bolts shall be at an angle of not more than 60 degrees with the center line of lower chord. Holes in timbers for inclined bolts in details employing end shoe plates shall be 1/16 inch larger than the nominal diameter of the bolt.

Tension splices shall be of such type that the effects of cross shrinkage of timber will be a minimum. Neither steel table fish plates nor shear pin splices shall be used on timbers over 8 inches thick, since the cross shrinkage of the timber will allow the plates or pads to separate. The shear-pin joint shall be used only with fully seasoned timber and gas-pipe shall not be used for shear pins.

11.5.2 Floorbeams.—Floorbeams shall be sized at bearing points. In floorbeams composed of two or more timbers, the timbers shall be separated by at least 2 inches for air circulation. Floorbeams shall be connected to the main truss members by means of rods or structural shapes.

11.5.3 Hangers.—Hangers generally shall be rods having upset ends with a suitably designed washer or bearing plate at each end. Upset ends shall conform to the requirements specified for Structural Steel Design, Section 10.

11.5.4 Eyebars and Counters.—The requirements specified for Structural Steel Design, for counters, eyebars and eyebar packing shall apply to such members when used in timber trusses.

11.5.5 Bracing.—Timber trusses shall be provided with a rigid system of laterals in the plane of the loaded chord. When the details will permit, this lateral bracing shall be securely fastened to all longitudinal stringers. Lateral bracing, preferably rigid, in the plane of the unloaded chord, and rigid portal and sway bracing shall be provided in all trusses having sufficient head room. Outrigger brackets connected to extensions of the floorbeams shall be used for bracing through trusses having headroom insufficient for a top lateral system.

11.5.6 Camber.—Camber, in addition to that required to provide for dead load and shrinkage, shall be provided in timber trusses in sufficient amount to give the structure a good appearance.

11.6 Floors and Railing.

11.6.1 Stringers.—Timber stringers shall be of sufficient length to take bearing over the full width of caps or floor beams, except outside stringers which may have butt joints. Preferably they shall be of two panel lengths placed with staggered joints. The lapped ends of untreated stringers shall be separated at least ½ inch for air circulation. Stringers shall be adequately secured to caps or floorbeams.

11.6.2 Bridging.—Stringers shall be adequately braced by cross bridging in each panel. The bridging shall be not less in size than 2"x4".

11.6.3 Nailing Strips.—When timber floors are supported by steel joists, the joists shall be provided with nailing strips which shall be bolted on the top flanges or preferably the timber floors shall be fastened to the flanges of the steel beams at adequate intervals by notched plates which slip over the flange and are spiked to the face of the timber.

When nailing strips are bolted to the flanges they shall be used on all joists. They shall be not less than 4 inches deep and shall be wider than the supporting flange. They shall be secured with 5/8 inch U bolts around the flanges, spaced not more than 4 feet apart and not more than 18 inches from the ends of the strips.

11.6.14 Flooring.—Roadway floor plank shall have a nominal thickness of not less than 3 inches, for a single plank floor, nor less than 1/6 of the center to center spacing of stringers or supports. Plank shall be of not less than 10 inch width laid heart side down, with a clearance of ¼ inch between planks.

A transverse plank floor with longitudinal plank wearing surface shall consist of transverse plank not less than 3 inches nominal thickness and longitudinal plank not less than 2 inches nominal thickness. The sub-flooring shall be creosoted by full-cell or brush treatment. Timber floors shall be of long leaf yellow pine, douglas fir or oak, of dense, select grade. Sidewalk floor plank shall have a nominal thickness of not less than 2 inches, and shall consist of 2"x4" laid flat with 3/8 inch open joints.

The minimum size of material used for laminated or strip floors should be 2"x4". The maximum beam spacing for 4 inch laminated floor without wearing surface shall be 2 feet 6 inches and for 6 inch laminated floor shall be 5 feet.

11.6.5 Retaining Pieces.—Retaining pieces, where required, shall be not less than 6 inches in width. In general they shall be secured in place by ¾ inch bolts at 3 foot intervals and spiked at 1 foot intervals.

11.6.6 Wheel Guards.—Wheel guards having a cross section of not less than 6"x6" shall be provided on each side of the roadway. The guard timbers shall be in lengths of not less than 12 feet. They shall be secured with ¾ inch bolts at the ends and at intermediate points not more than 4 feet apart, and shall generally be of white oak. The wheel guards shall be supported on the flooring by scupper blocks not less than 4 inches thick and 1 foot 6 inches long, held in place by spikes and a bolt through the wheel guard and flooring, and spaced not more than 5 feet apart.

11.6.7 Drainage.—Adequate provision shall be made for the proper drainage of timber floors.

11.6.8 Railings.—Wood railings shall consist of not less than 3 horizontal lines of rails. Rails shall have a cross section not less than 2"x6" with a horizontal cap of not less than 2"x6" section. No space between railings shall be greater than 8 inches and preferably not greater than 6 inches. Rail posts shall have a cross section not less than 6"x6" and shall be spaced not more than 8 feet apart, and shall be rigidly fastened so as to develop the flexural strength of the post.

Preferably, rails shall be surfaced four sides (S4S) and painted white. Railings shall have a height of not less than 3'-0" above the top of curb or walk and shall clear the curb face horizontally not less than 9 inches, and preferably not less than 18 inches.

11.7 Fire Stops.—To check the spread of fire lengthwise of the structure, timber floors or trestles of any considerable length preferably shall be provided with fire stops. In timber floors these fire stops should be provided at intervals not over 75 feet apart. They may consist of diaphragms of wood or fire resistant material at least as thick as the floorings, located over caps or floorbeams and completely filling the openings between the joists. In timber trestle bridges, in addition to the fire stops in the floor, fire curtains should be provided at intervals of 100 feet or more. These curtains may consist of plank or asbestos-covered metal spiked to the bents. They should extend downward from the bottom of the joists at least five feet and horizontally at least to the ends of the caps. A fire stop between the joists should be located over each curtain.

11.8 Preservation of Timber.

11.8.1 Material to be Creosoted.—Unless otherwise specified, all timber with exception of that permanently below low water, and excepting wearing surface railing and curb timbers, shall be creosoted by either the full cell or empty-cell process, as specified in Division 12, Section 13.

11.8.2 Holes and Cuts.—Any wood which is to be treated shall have framing and boring done as far as practicable before treating. Treated timber and piling which is cut or bored after treatment, shall have the surfaces so exposed treated with three applications of hot creosote oil. Where holes are bored, they shall be poured full of creosote if possible.

11.8.3 Bearing Surfaces.—All bearing surfaces which are subject to loading, shall be covered with a thick coat of asphalt.

11.8.4 Floor Plank.—Wearing surface plank shall be untreated, as its life is limited by wear. The top surfaces of creosoted base plank, stringers and other sub floor members upon which the surface plank bears, shall be painted with a heavy coat of asphalt.

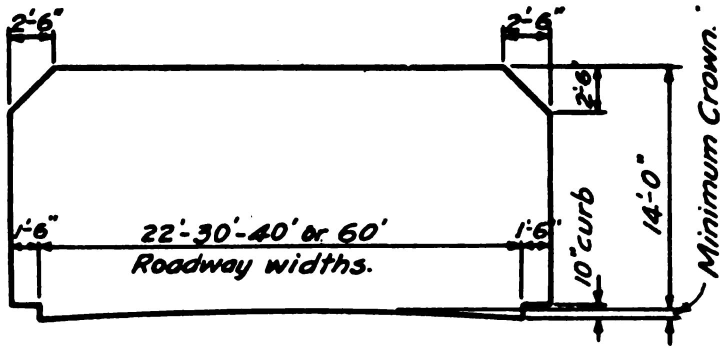


FIGURE 1.
Showing Clearances for Highway Bridges.

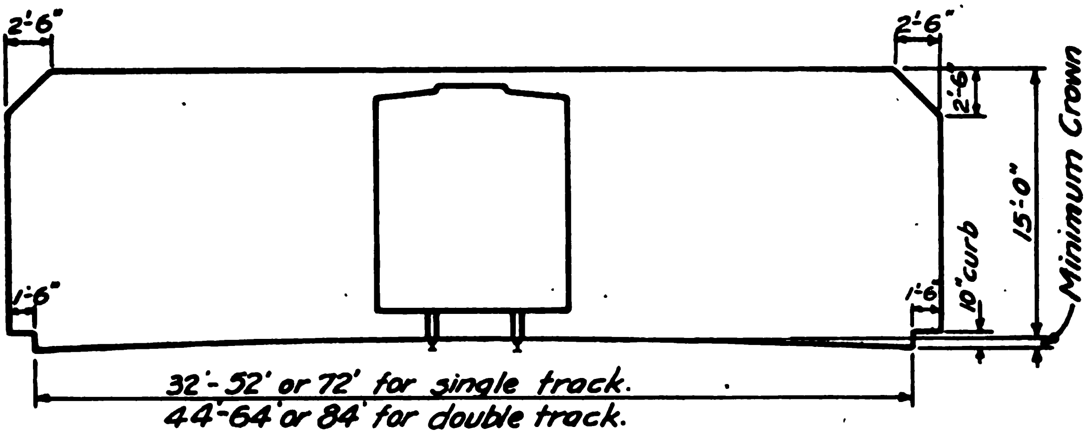


FIGURE 2.
Showing Clearances for Highway and Electric Railroad Bridges.

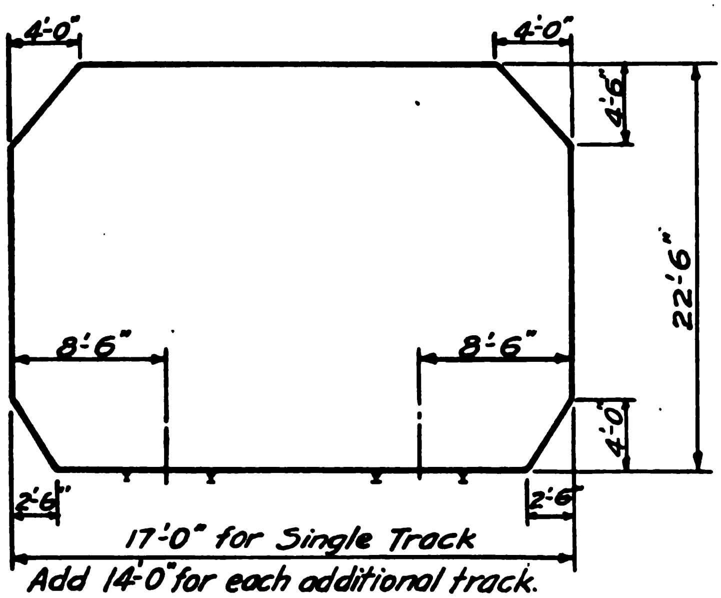


FIGURE 3
Showing Clearances for Through RailRoad Bridges and Grade Separation Structures over Rail Roads.

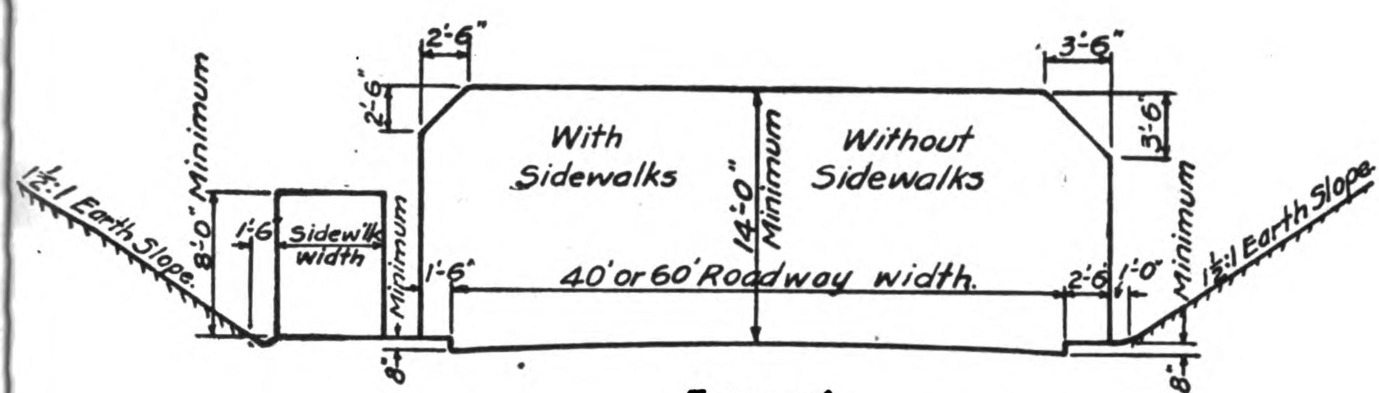


FIGURE 4
Showing Clearances for Grade Separation Subways

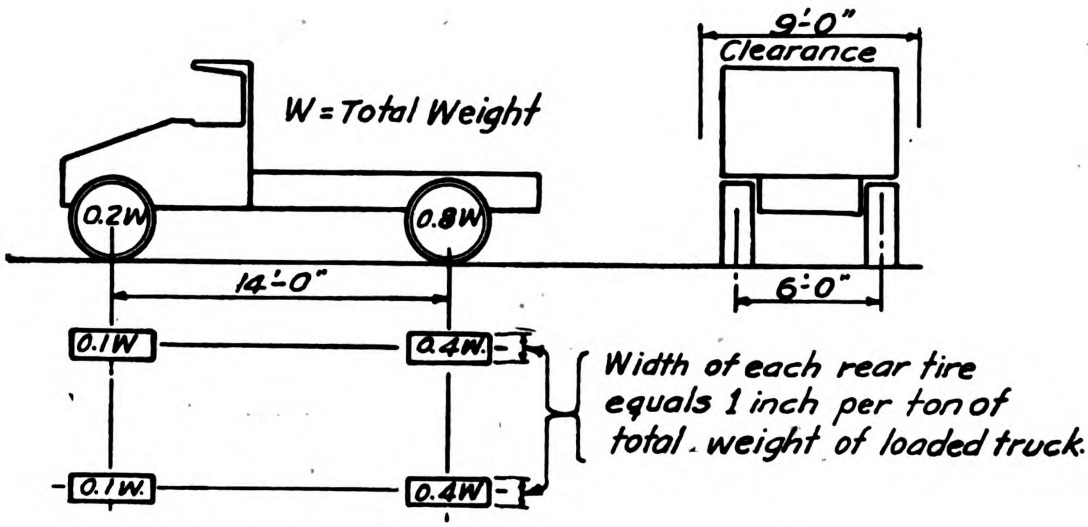


FIGURE 5
Standard Truck Loading.

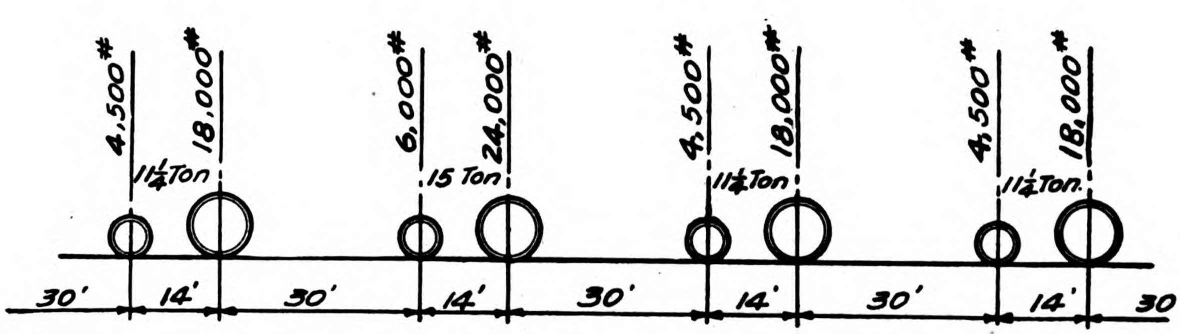


FIGURE 6.

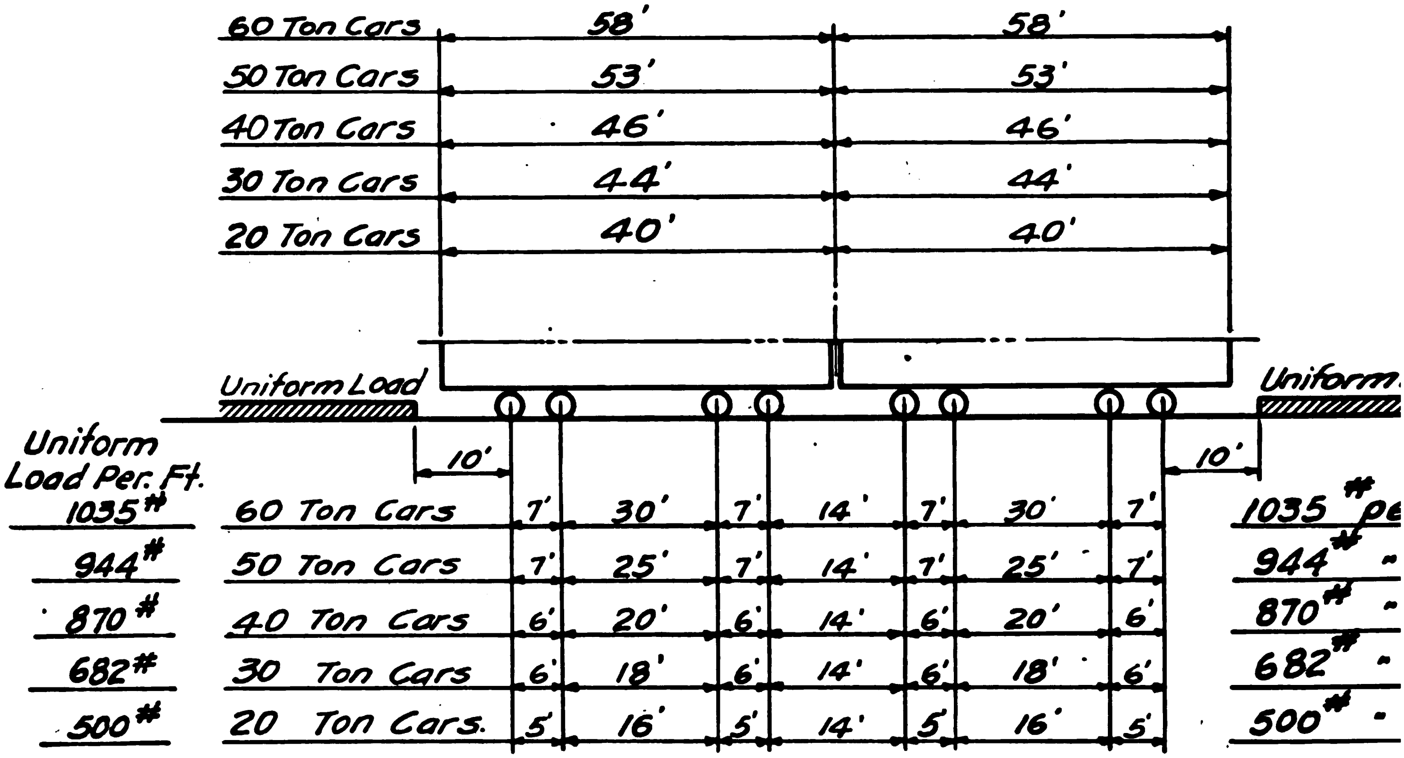
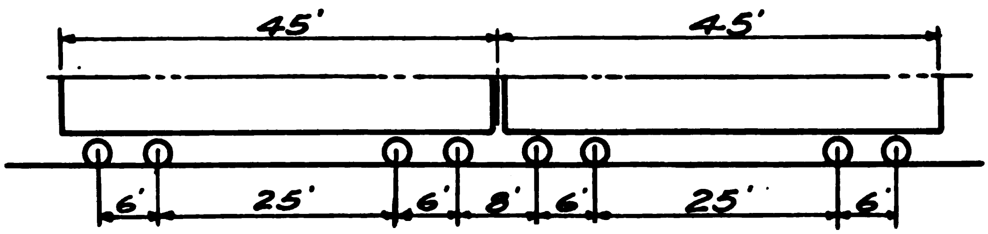


FIGURE 7.
Electric Railway Loading.



Total Loaded Weight per Car, including 10% Overload.

40	Ton	Capacity	-	128,000#
50	"	"	-	152,000#
60	"	"	-	176,000#
70	"	"	-	200,000#
80	"	"	-	224,000#

FIGURE 7a.

Typical Freight Cars

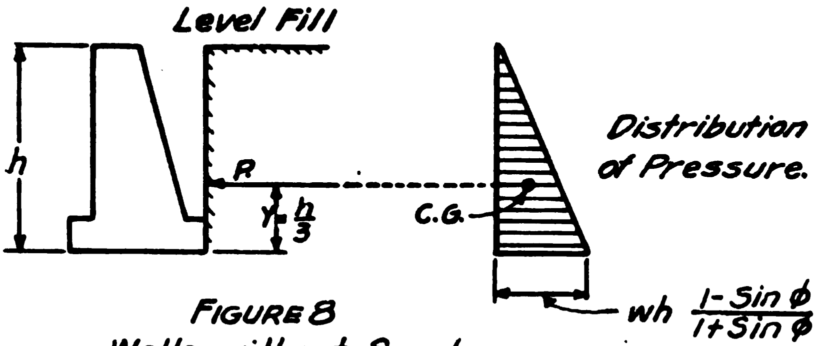


FIGURE 8
Walls without Surcharge

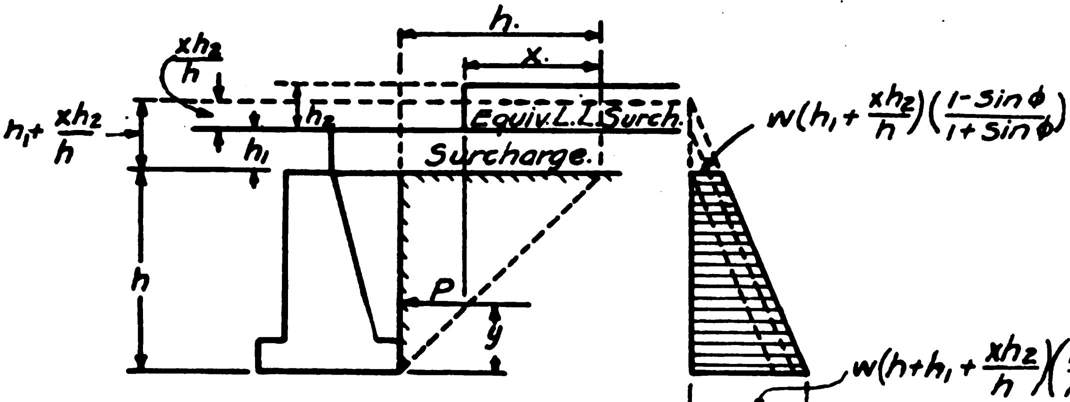


FIGURE 9
Walls with Level Surcharge - Dead or Live or Both.

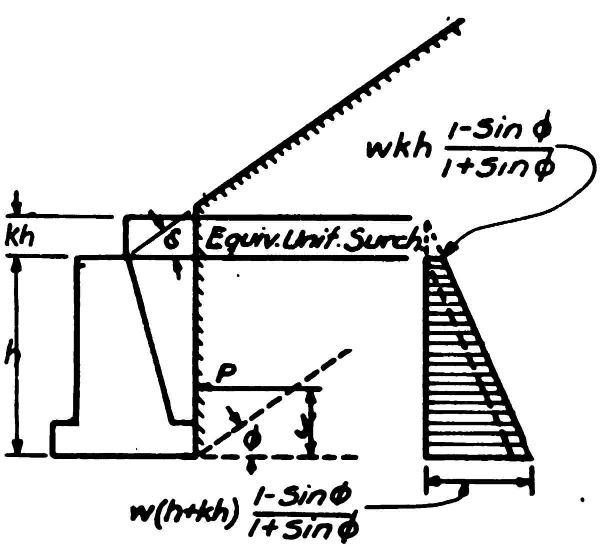


FIGURE 10
Walls with Sloping Surcharge
Dead Only.

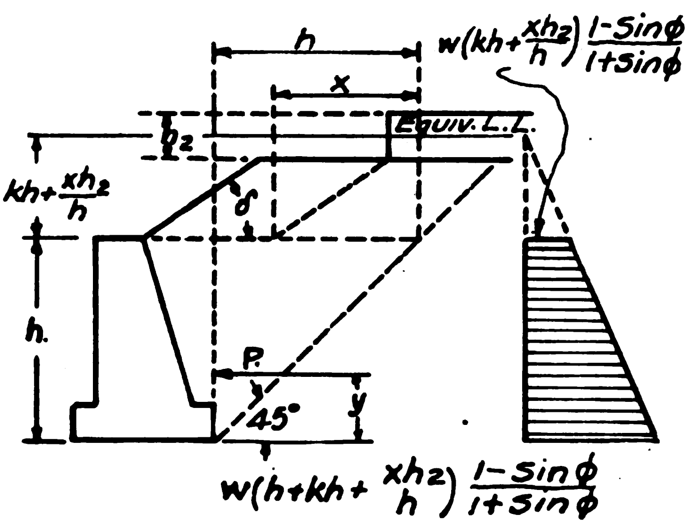


FIGURE 11
Walls with Sloping Surcharge
and Superimposed Live Surcharge.

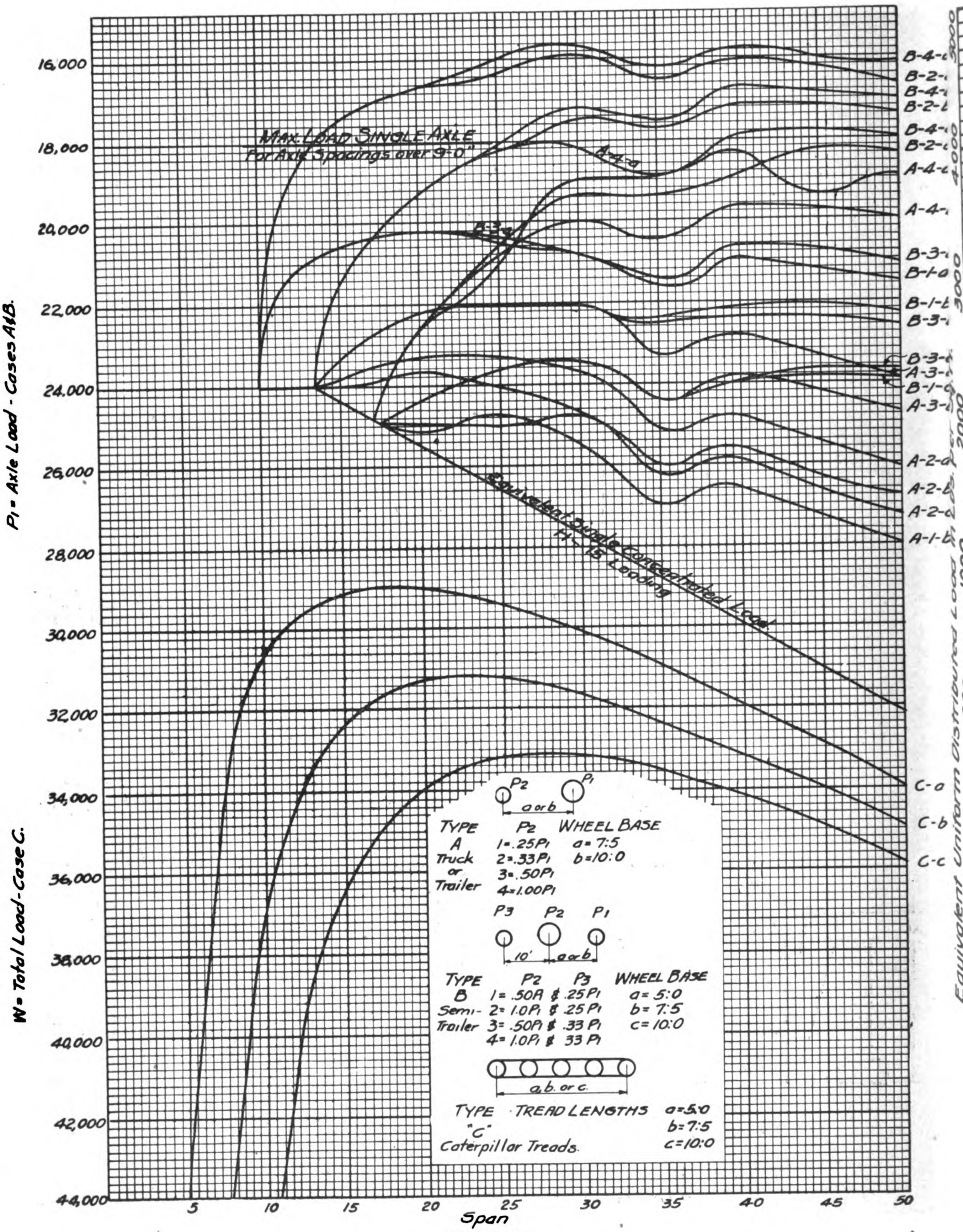


FIGURE 12
Loadings Equivalent to H15.

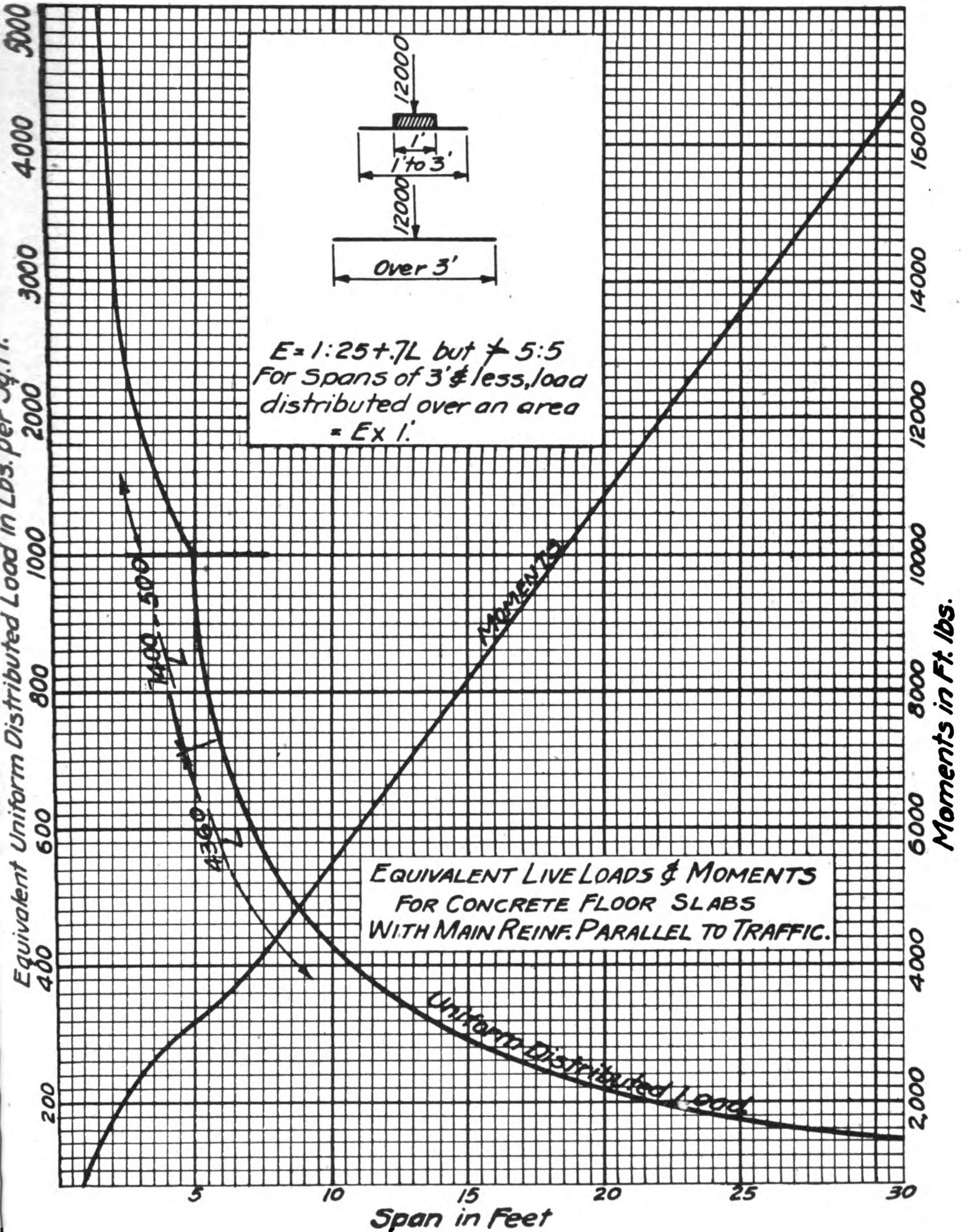

12000
1'
1'to 3'
12000
Over 3'
E= 1:25+.7L but ≯ 5:5
For Spans of 3' & less, load
distributed over an area
= E x 1'.
MOMENTS
EQUIVALENT LIVE LOADS & MOMENTS
FOR CONCRETE FLOOR SLABS
WITH MAIN REINF. PARALLEL TO TRAFFIC.
Uniform Distributed Load
1400/L - 300
4360/L
Equivalent Uniform Distributed Load in Lbs. per
Moments in Ft. lbs.
Span in Feet

FIGURE 13

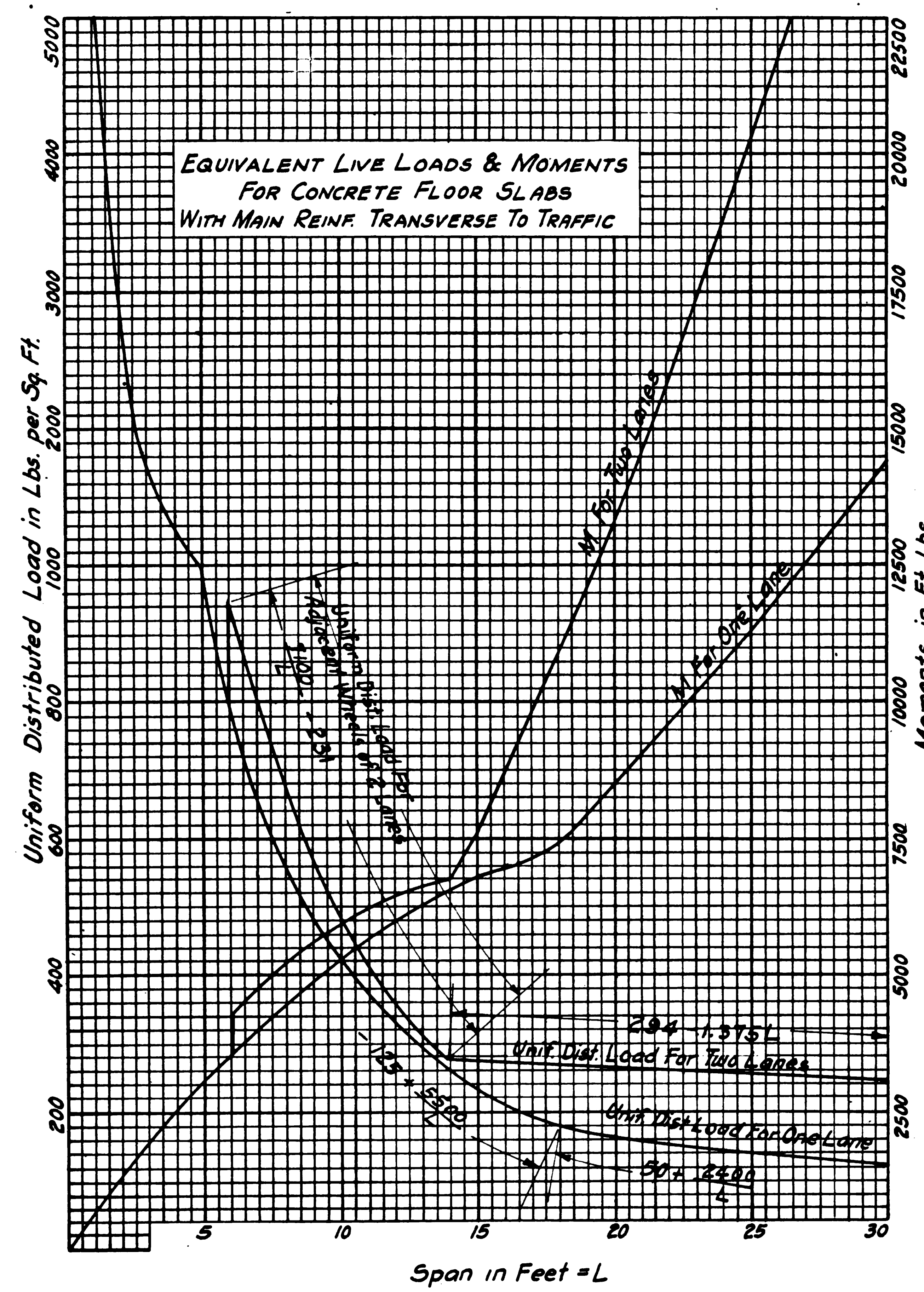

EQUIVALENT LIVE LOADS & MOMENTS
FOR CONCRETE FLOOR SLABS
WITH MAIN REINF. TRANSVERSE TO TRAFFIC
Uniform Distributed Load in Lbs. per Sq. Ft.
200
400
600
800
1000
2000
3000
4000
5000
Moments in Ft. Lbs.
2500
5000
7500
10000
12500
15000
17500
20000
22500
M For Two Lanes
M For One Lane
Uniform Dist. Load For Adjacent Wheels of 2 Lanes
4100/L − 231
294 − 1.375L
Unif. Dist. Load For Two Lanes
25 + 5500/L
Unif. Dist. Load For One Lane
50 + 2400/L
5
10
15
20
25
30
Span in Feet = L

FIGURE 14

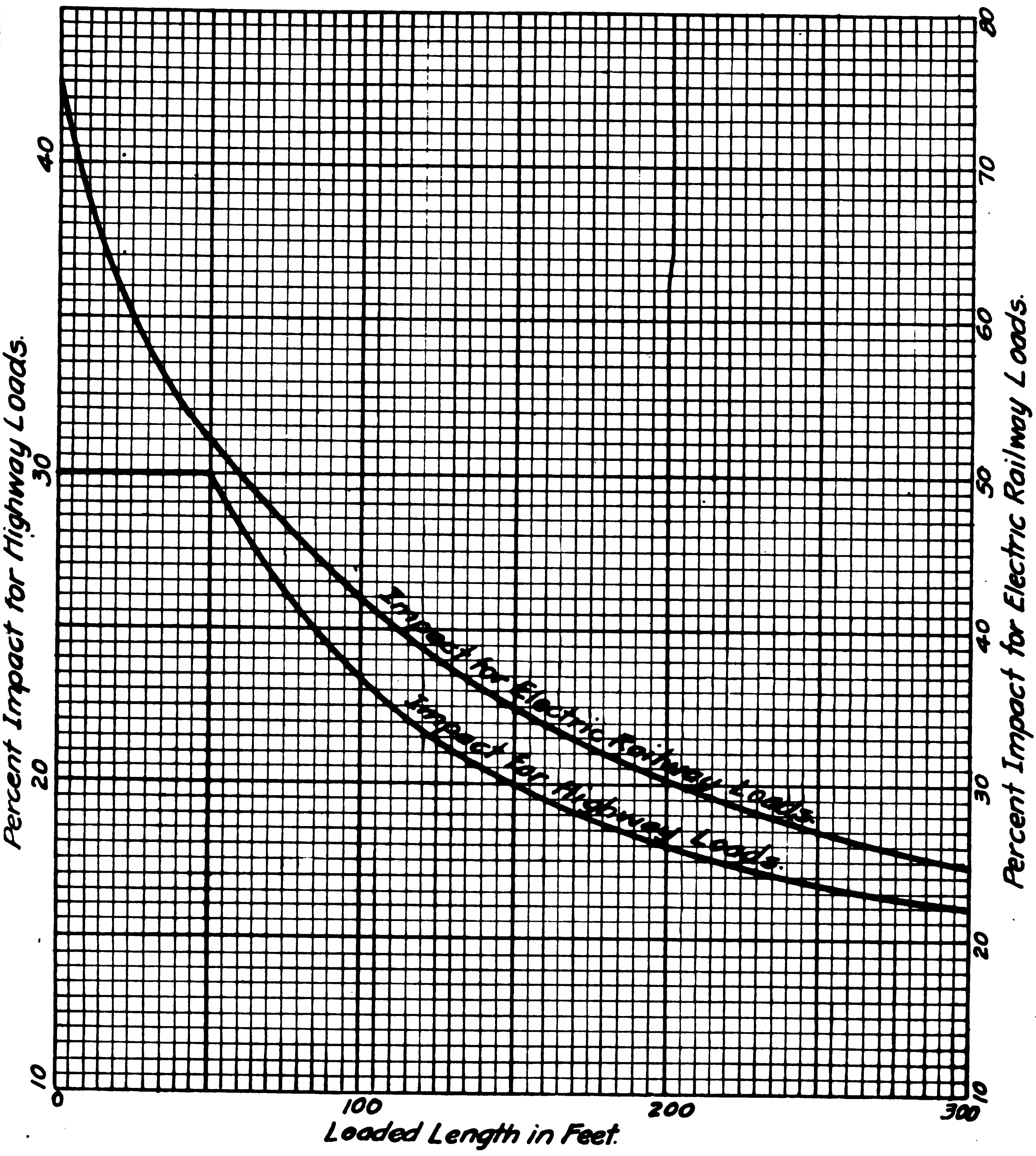


FIGURE 15

Showing Impact.

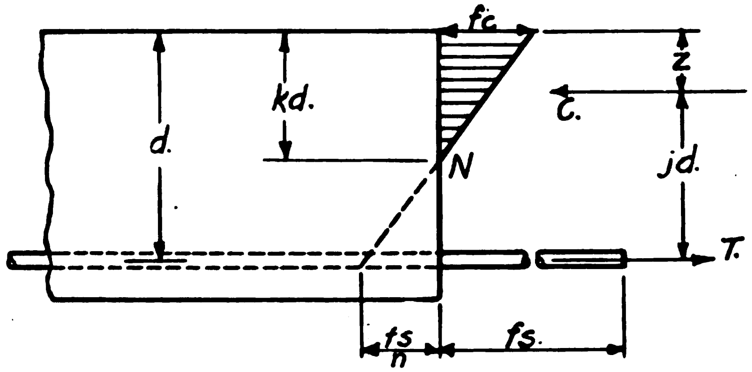


FIGURE 16

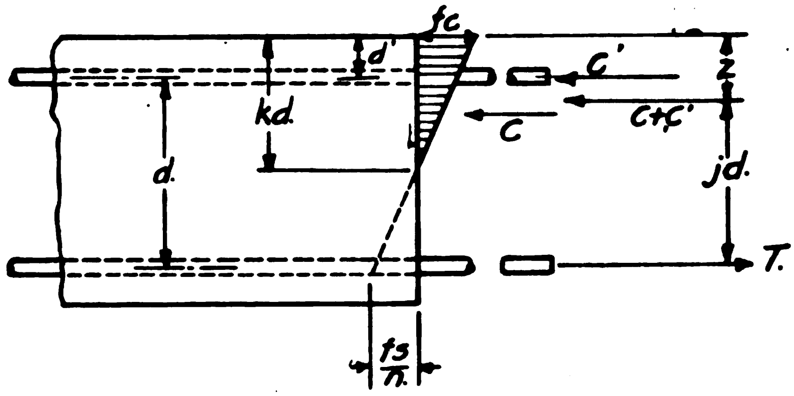


FIGURE 17

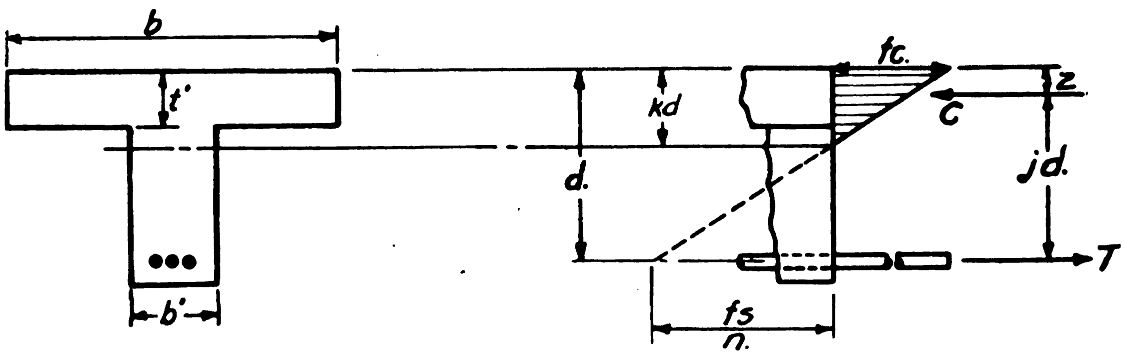
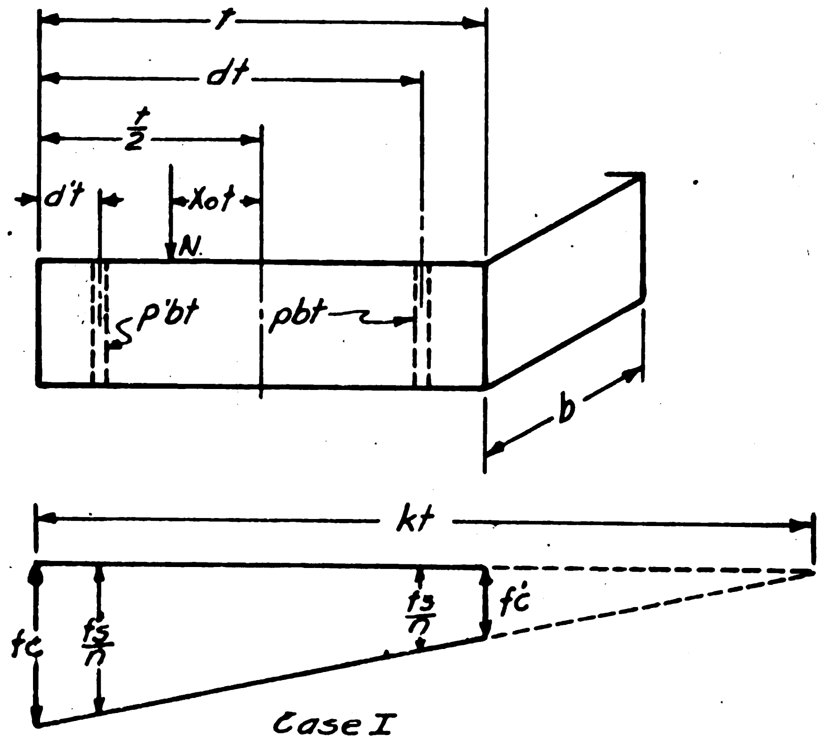


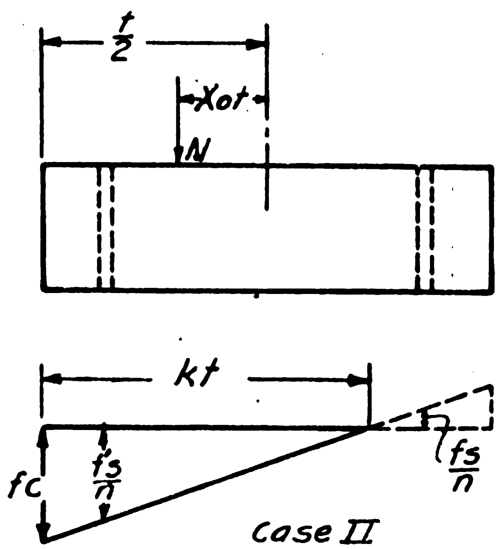
FIGURE 18



Case I

FIGURE 19

For either Compression over entire section or tension over Part of section.



Case II

FIGURE 20

Tension in Concrete neglected

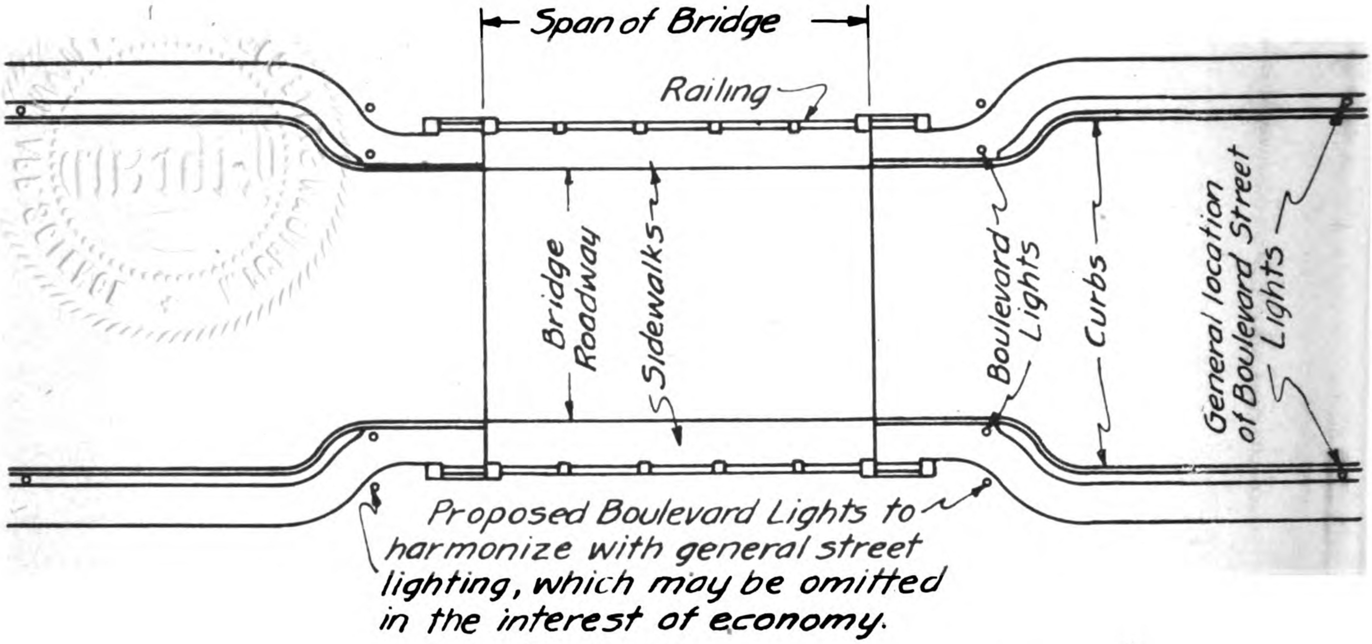


FIGURE 21
Showing Boulevard Lights at ends of Bridge.

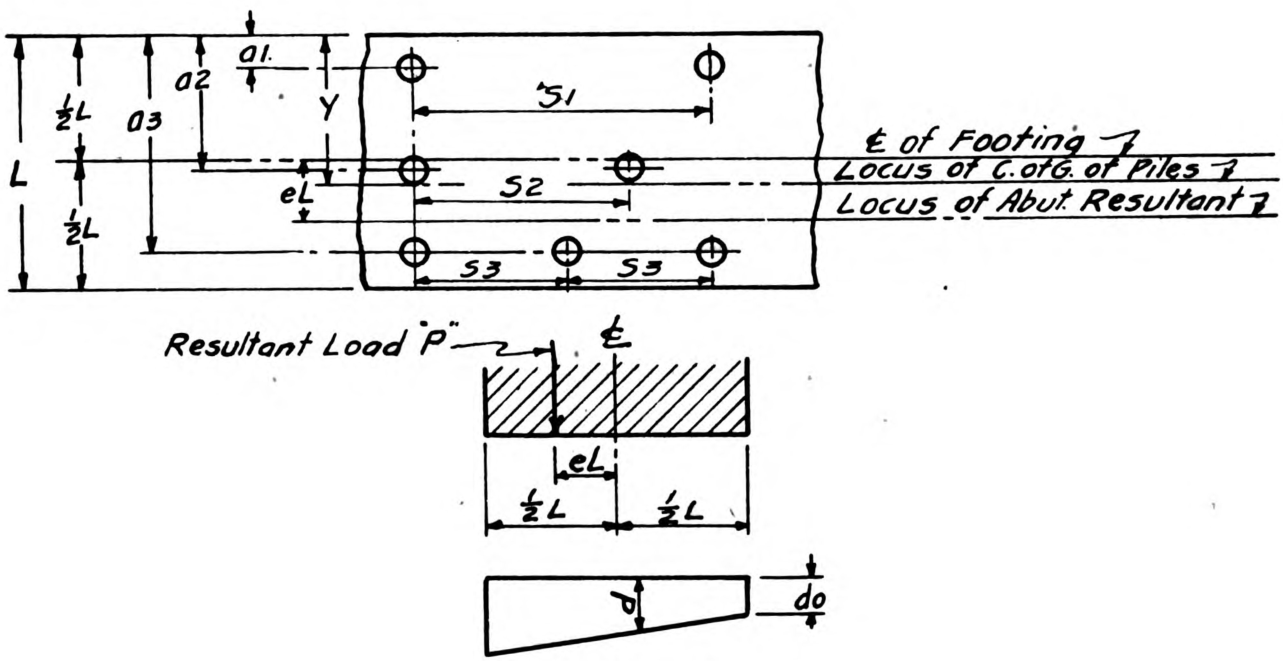


FIGURE 22
Diagram for Computation of Pile Loads.