WASHINGTON BRIDGE, OVER HARLEM RIVER, NEW YORK CITY. BUILT BY THE PASSAIC ROLLING MILL CO.

WM. A. KINSEY JULY 28 - 97.

A MANUAL

OF

USEFUL INFORMATION AND TABLES
APPERTAINING TO THE USE OF

STRUCTURAL STEEL,

AS MANUFACTURED BY

THE PASSAIC ROLLING MILL CO.,

PATERSON, NEW JERSEY.

(NEW YORK OFFICE, 45 BROADWAY.)

FOR ENGINEERS, ARCHITECTS AND BUILDERS.

BY

GEO. H. BLAKELEY, C. E.

M. AM. SOC. C. E.

1897.

THE DE VINNE PRESS, NEW-YORK.

PREFACE.

This manual is a new work throughout. It is intended to supply such special information and tables as, it was thought, would prove of value and service to those who are engaged in the design of structural steel work in general, and the patrons of the publishers, THE PASSAIC ROLLING MILL CO., in particular.

The tables, with a few exceptions, were computed expressly for this work, and many of them are original in both matter and form.

The author hopes that they will be found to possess the qualities of accuracy and reliability.

Such of the tables as were not calculated for this work were obtained from works of presumably independent origin, which were compared for the detection of errors.

The tables of the weights and ultimate strengths of materials have been compiled by comparison of all the available data on the subject.

No attempt has been made to encumber the work with abridgments of mathematical tables, as such tables, to be of value, must be very extended and complete. Only such matter is given as the author has found to be of service in his own practice.

TABLE OF CONTENTS.

EXPLANATORY NOTES.

All weights given are for steel, and are per lineal foot of the section.

The manner in which the weights of various sections are increased is illustrated on page 30. .

For channels and **I** beams, the enlargement of the section adds an equal amount to the thickness of the web and the width of the flanges. Lithograph sections are given for the principal weights of beams and channels. The dimensions of other weights of beams and channels can be obtained from the tables of minimum and maximum weights and dimensions of **I** beams and channels.

The effect of spreading the rolls, to increase the thickness of angles, slightly increases the length of the legs. Where the thickness is rolled in finishing grooves, the exact length of the legs is maintained. The finishing grooves for angles are given in the table on page 33. Intermediate and thicker sections have slightly increased length of legs.

Z bars are increased in thickness in the same manner as angles. The dimensions of the various thicknesses of **Z** bars are given in the tables of the weights and properties of **Z** bars.

T shapes do not admit of any variation, and can only be rolled to the weights given.

Beams, Channels, and **Z** bars are rolled only of steel. Universal Mill Plates and Angles are rolled of steel, but can be rolled of iron by special arrangement. **T** shapes can be rolled of steel or iron. Merchant Bars can be rolled either of steel or iron.

In ordering sections, the weight or thickness wanted must be designated, but not both.

Unless stated to the contrary, all tables are for steel sections, as steel is now almost exclusively used for all structural purposes.

Unless otherwise arranged, all structural material will be cut to lengths with an extreme variation not exceeding ¾ of an inch.

SHAPES

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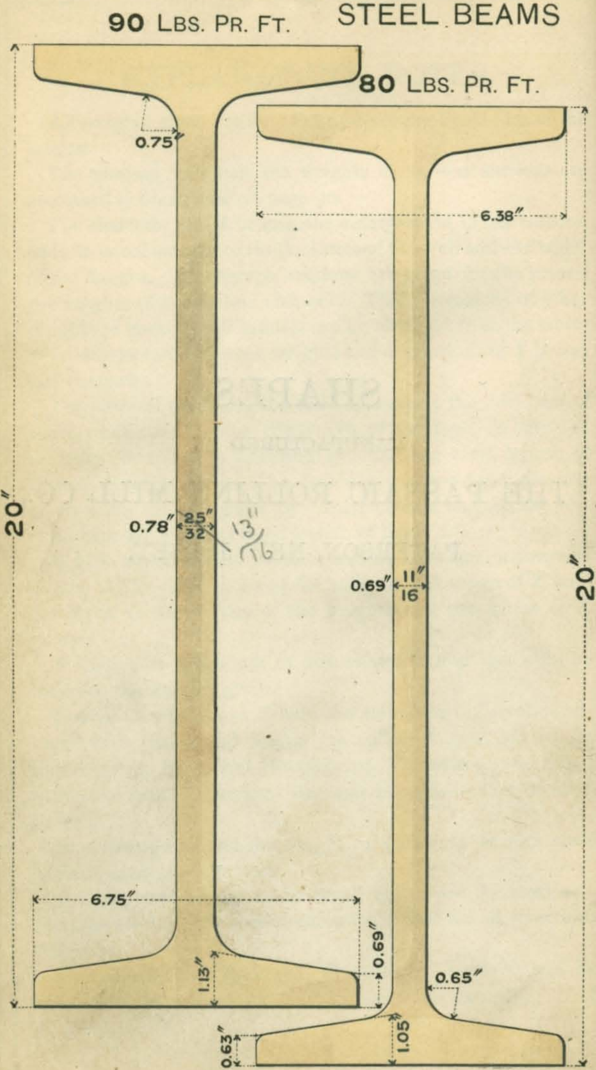
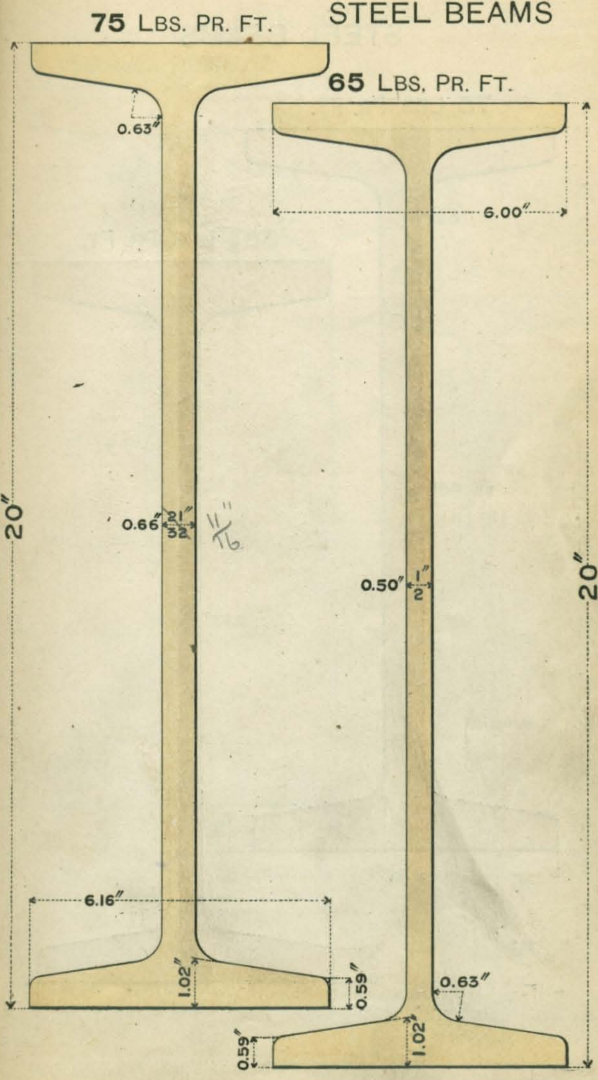
STEEL BEAMS

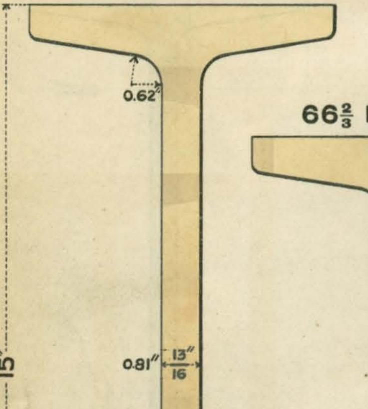

90 LBS. PR. FT.
80 LBS. PR. FT.
0.75″
6.38″
20″
0.78″
25″/32
13″/16
0.69″
11″/16
20″
6.75″
0.69″
1.13″
0.65″
0.63″
1.05″

STEEL BEAMS

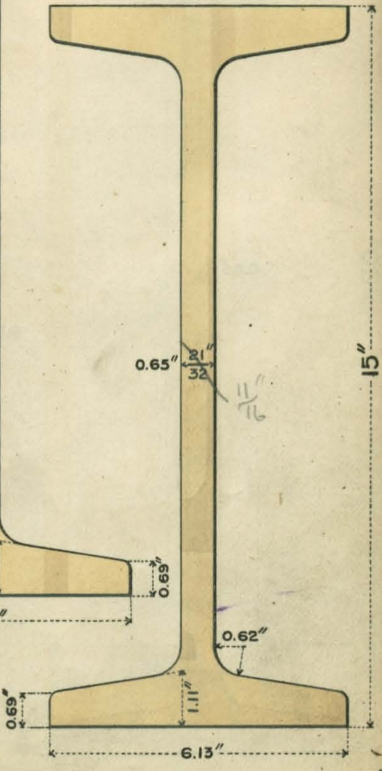

75 LBS. PR. FT.
65 LBS. PR. FT.
0.63″
6.00″
20″
0.66″ 21/32″
11/16″
0.50″ 1/2″
20″
6.16″
1.02″
0.59″
0.63″
1.02″
0.59″

STEEL BEAMS

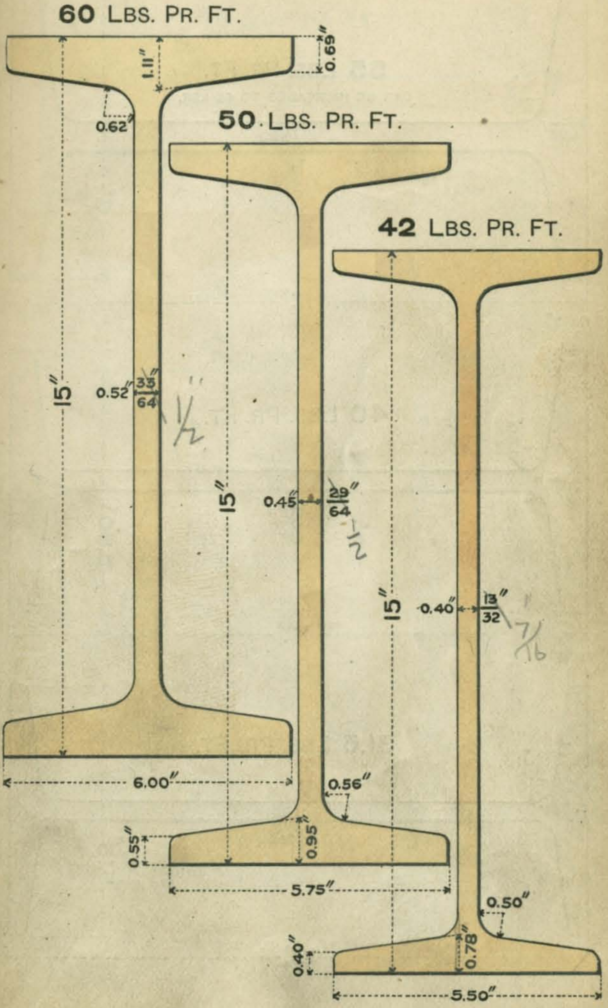
75 LBS. PR. FT.



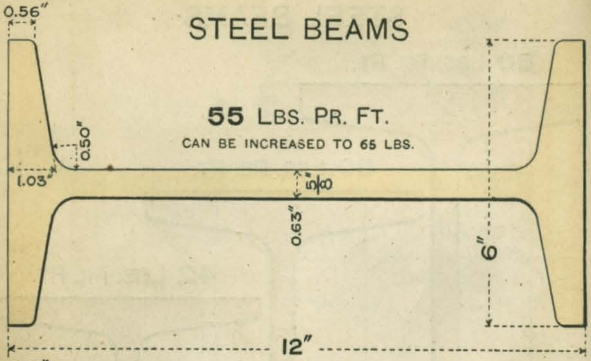
66⅔ LBS. PR. FT.

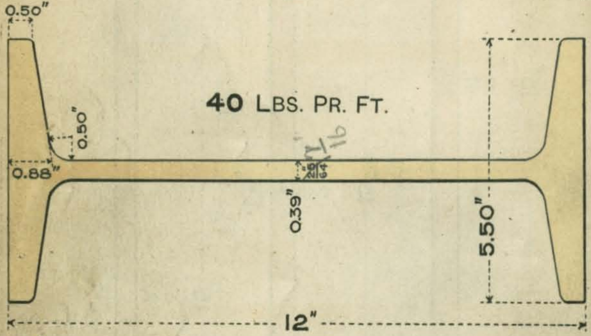


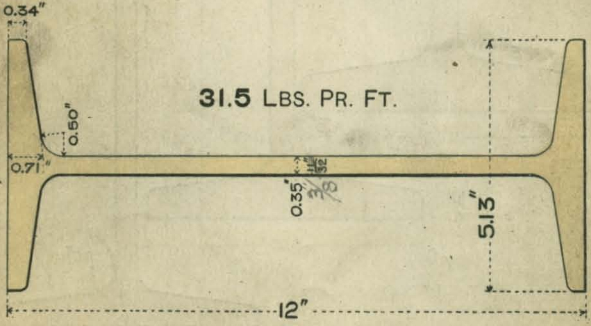
STEEL BEAMS

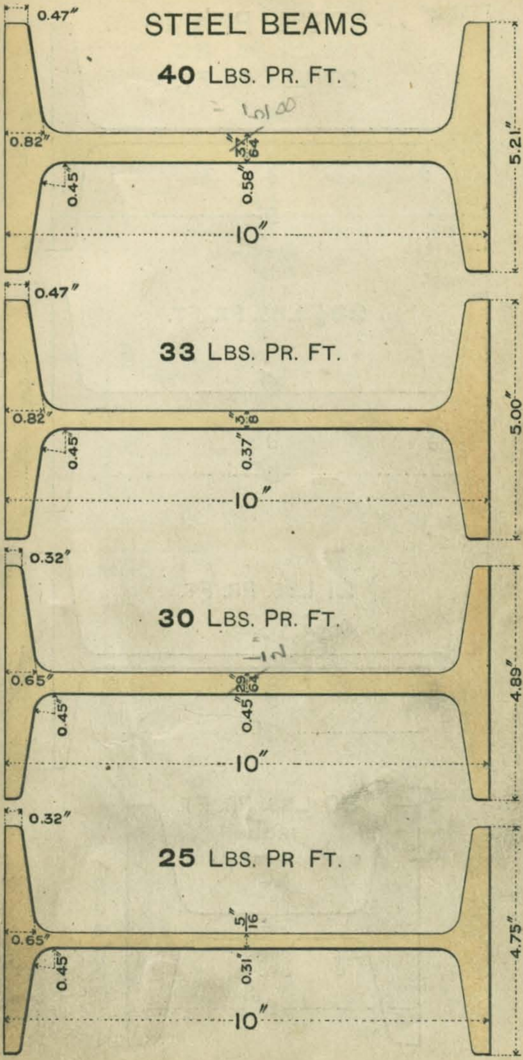

60 LBS. PR. FT.
0.69"
1.11"
0.62"
15"
0.52" 33/64"
1/2"
6.00"
50 LBS. PR. FT.
15"
0.45" 29/64"
1/2
0.56"
0.55"
0.95"
5.75"
42 LBS. PR. FT.
15"
0.40" 13/32"
7/16
0.50"
0.40"
0.78"
5.50"

STEEL BEAMS

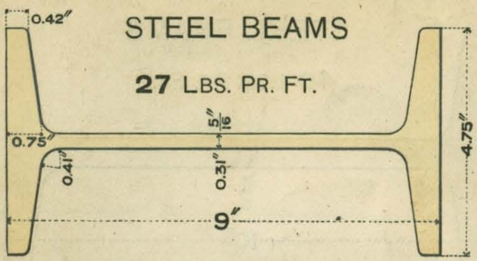

0.56"
55 LBS. PR. FT.
CAN BE INCREASED TO 65 LBS.
0.50"
1.03"
5/8"
0.63"
6"
12"

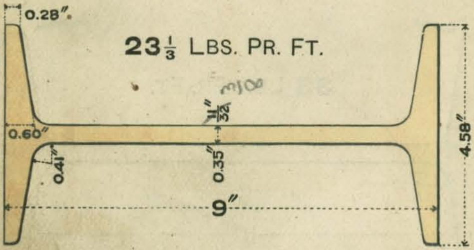

0.50"
40 LBS. PR. FT.
0.50"
0.88"
25/64"
1/16
0.39"
5.50"
12"

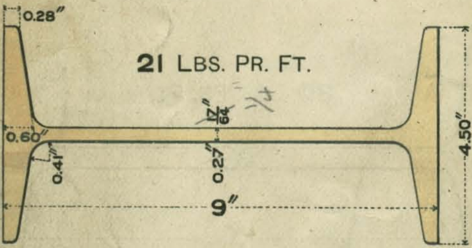

0.34"
31.5 LBS. PR. FT.
0.50"
0.71"
11/32"
3/8
0.35"
5.13"
12"

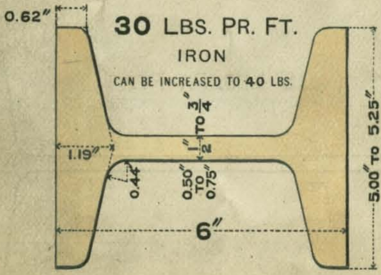

STEEL BEAMS
40 LBS. PR. FT.
0.47″
0.82″
0.45″
$\frac{37}{64}$″
0.58″
10″
5.21″
33 LBS. PR. FT.
0.47″
0.82″
0.45″
$\frac{3}{8}$″
0.37″
10″
5.00″
30 LBS. PR. FT.
0.32″
0.65″
0.45″
$\frac{29}{64}$″
0.45″
10″
4.89″
25 LBS. PR FT.
0.32″
0.65″
0.45″
$\frac{5}{16}$″
0.31″
10″
4.75″

STEEL BEAMS

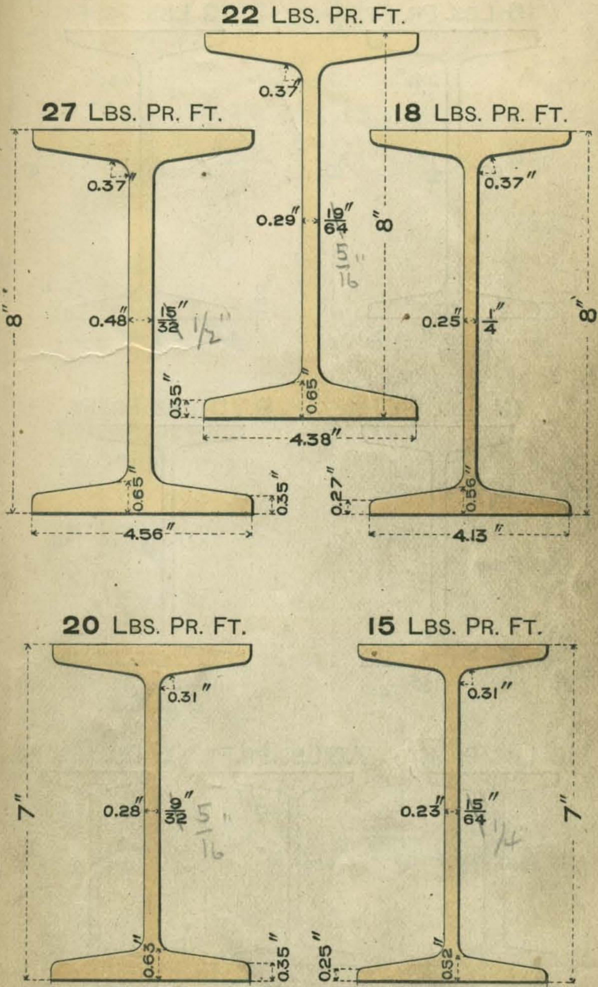

0.42″
27 LBS. PR. FT.
5″/16
0.75″
0.41″
0.31″
9″
4.75″


0.28″
23 1/3 LBS. PR. FT.
11″/32
0.60″
0.41″
0.35″
9″
4.58″


0.28″
21 LBS. PR. FT.
17″/64
0.60″
0.41″
0.27″
9″
4.50″

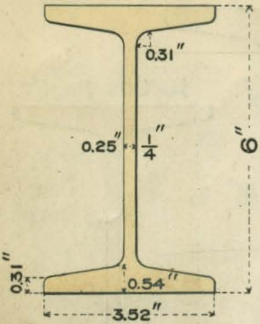

0.62″
30 LBS. PR. FT.
IRON
CAN BE INCREASED TO 40 LBS.
1″/2 TO 3″/4
1.19″
0.44
0.50″ TO 0.75″
6″
5.00″ TO 5.25″

STEEL BEAMS

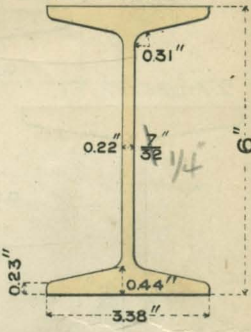

22 LBS. PR. FT.
0.37"
0.29"
19/64"
8"
5/16"
0.35"
0.65"
4.38"
27 LBS. PR. FT.
0.37"
8"
0.48"
15/32"
1/2"
0.65"
0.35"
4.56"
18 LBS. PR. FT.
0.37"
0.25"
1/4"
8"
0.27"
0.56"
4.13"
20 LBS. PR. FT.
0.31"
7"
0.28"
9/32"
5/16"
0.63"
0.35"
4.09"
15 LBS. PR. FT.
0.31"
0.23"
15/64"
1/4"
7"
0.25"
0.52"
3.88"

STEEL BEAMS

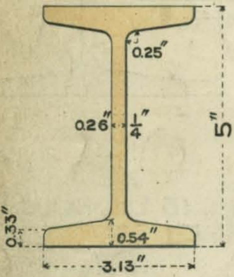
15 LBS. PR. FT.



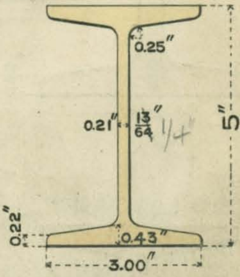
12 LBS. PR. FT.



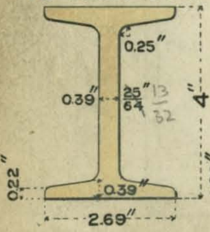
13 LBS. PR. FT.



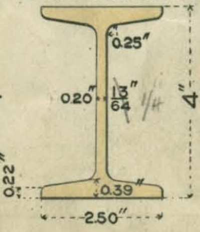
9.75 LBS. PR. FT.



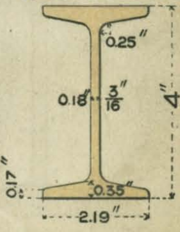
10 LBS. PR. FT.



7.5 LBS. PR. FT.

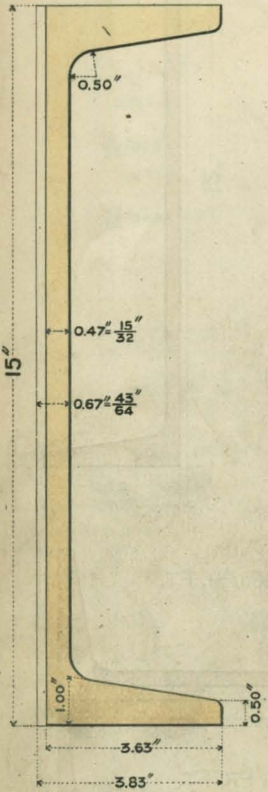


6 LBS. PR. FT.

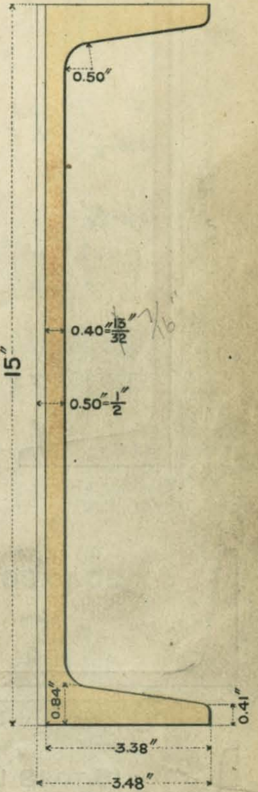


STEEL CHANNELS

40 TO **50** LBS. PR. FT.

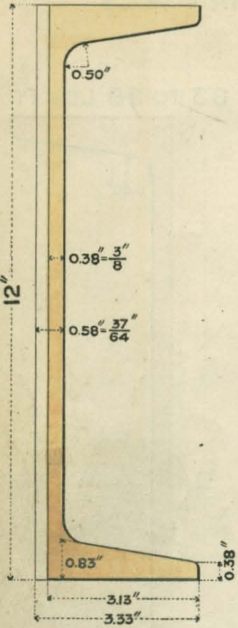


33 TO **38** LBS. PR. FT.

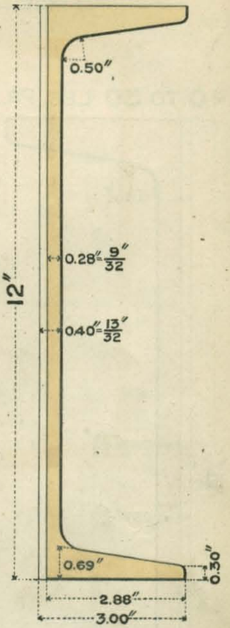


STEEL CHANNELS

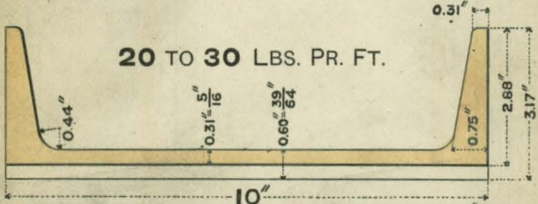
27 TO **35** LBS. PR. FT.



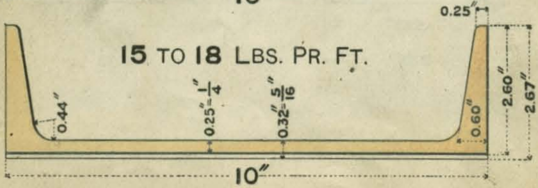
20 TO **25** LBS. PR. FT.



20 TO **30** LBS. PR. FT.

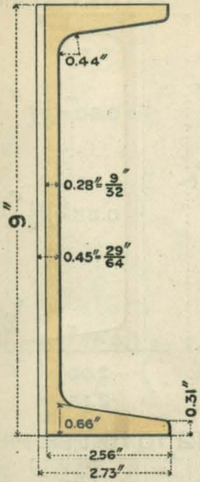


15 TO **18** LBS. PR. FT.

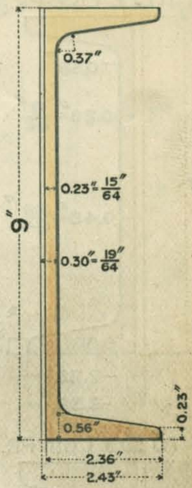


STEEL CHANNELS

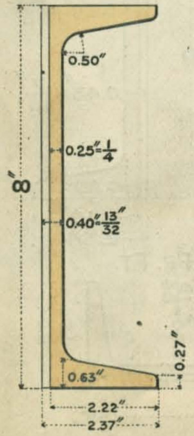
16 TO 21 LBS. PR. FT.



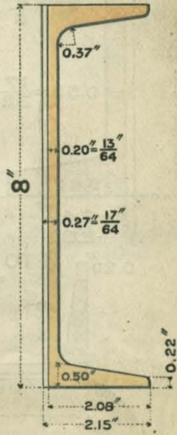
13 TO 15 LBS. PR. FT.



13 TO 17 LBS. PR. FT.

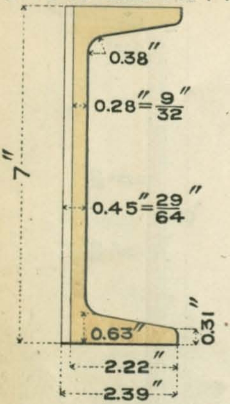


10 TO 12 LBS. PR. FT.

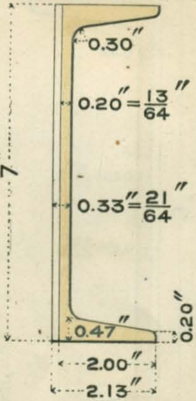


STEEL CHANNELS

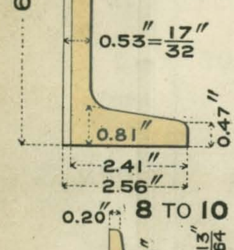
13 TO 17 LBS. PR. FT.



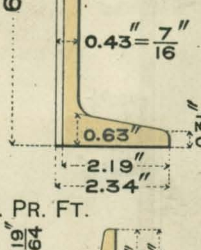
9 TO 12 LBS. PR. FT.



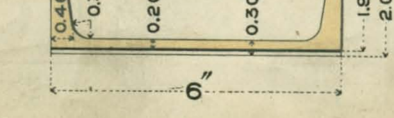
17 TO 20 LBS. PR. FT.



12 TO 15 LBS. PR. FT.

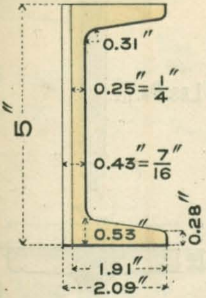


8 TO 10 LBS. PR. FT.

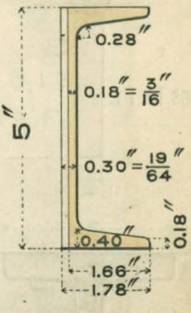


STEEL CHANNELS

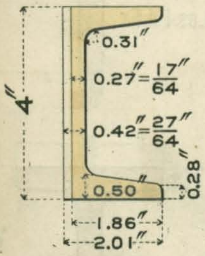
9 TO **12** LBS. PR. FT.



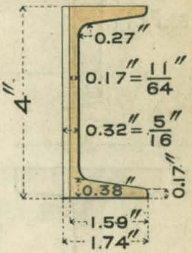
6 TO **8** LBS. PR. FT.



8 TO **10** LBS. PR. FT.

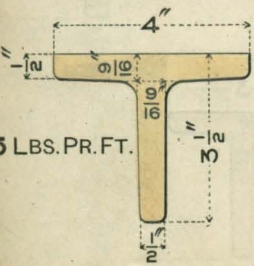


5 TO **7** LBS. PR. FT.

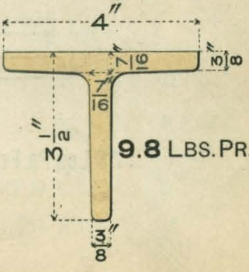


SPECIAL TEES

STEEL OR IRON

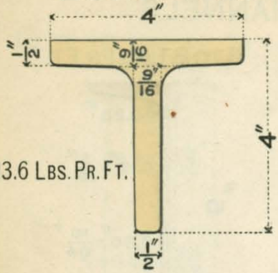


12.5 LBS. PR. FT.

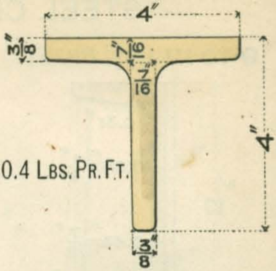


9.8 LBS. PR. FT.

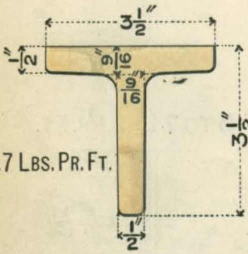
EQUAL TEES STEEL OR IRON.



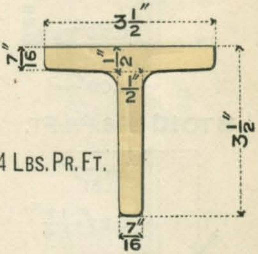
13.6 LBS. PR. FT.



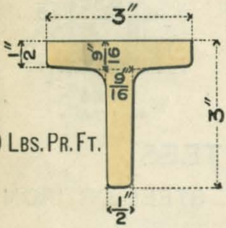
10.4 LBS. PR. FT.



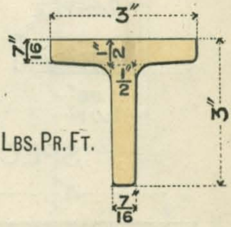
11.7 LBS. PR. FT.



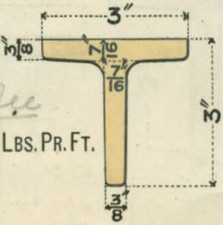
10.4 LBS. PR. FT.



10.0 LBS. PR. FT.



9.1 LBS. PR. FT.



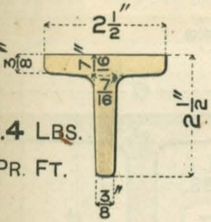
3x3x3/8 Tee

7.8 LBS. PR. FT.

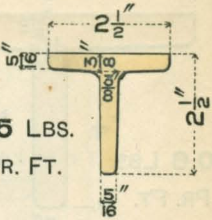
3x3x5/16
Weight = 6.6#

EQUAL TEES

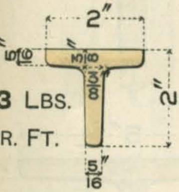
STEEL OR IRON



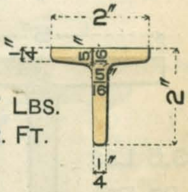
6.4 LBS.
PR. FT.



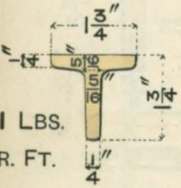
5.5 LBS.
PR. FT.



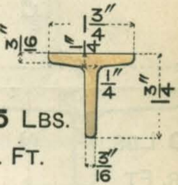
4.3 LBS.
PR. FT.



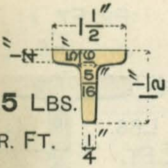
3.7 LBS.
PR. FT.



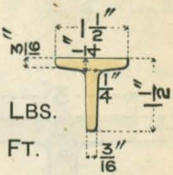
3.1 LBS.
PR. FT.



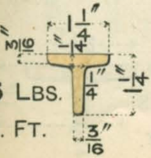
2.25 LBS.
PR. FT.



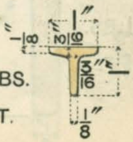
2.55 LBS.
PR. FT.



1.85 LBS.
PR. FT.

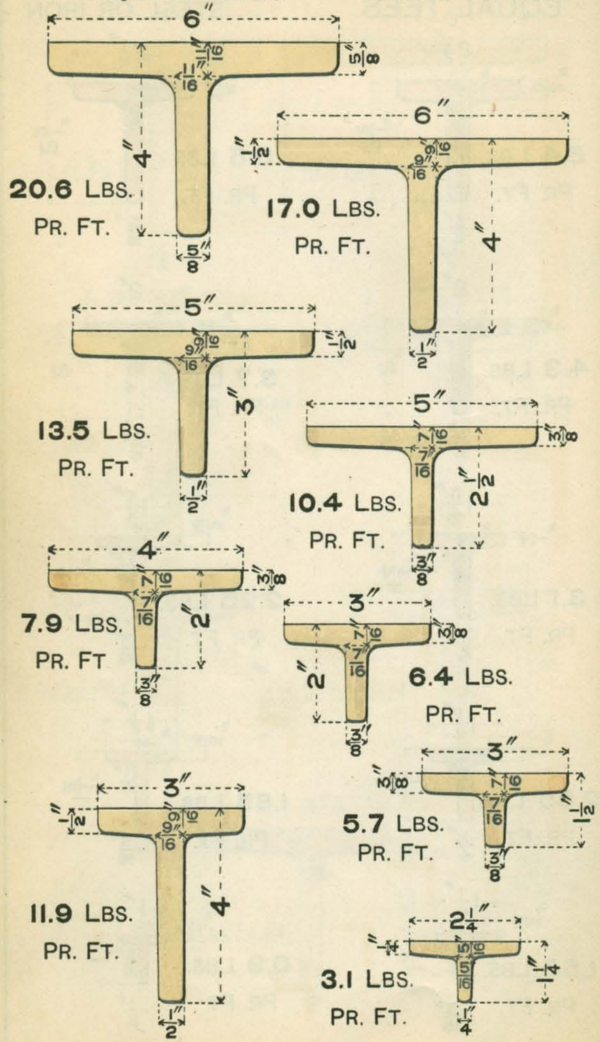


1.55 LBS.
PR. FT.

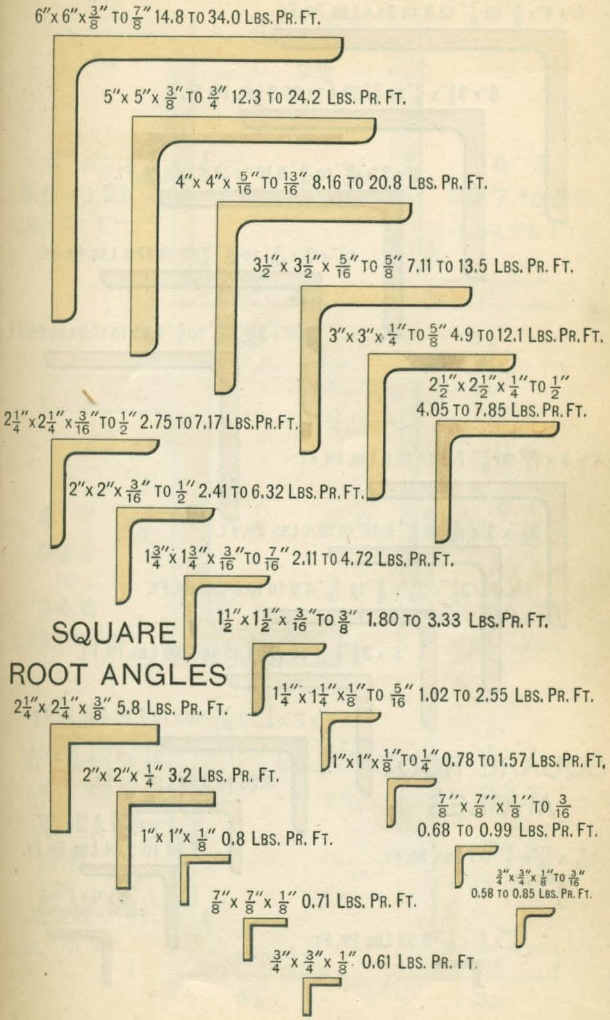


0.9 LBS.
PR. FT.

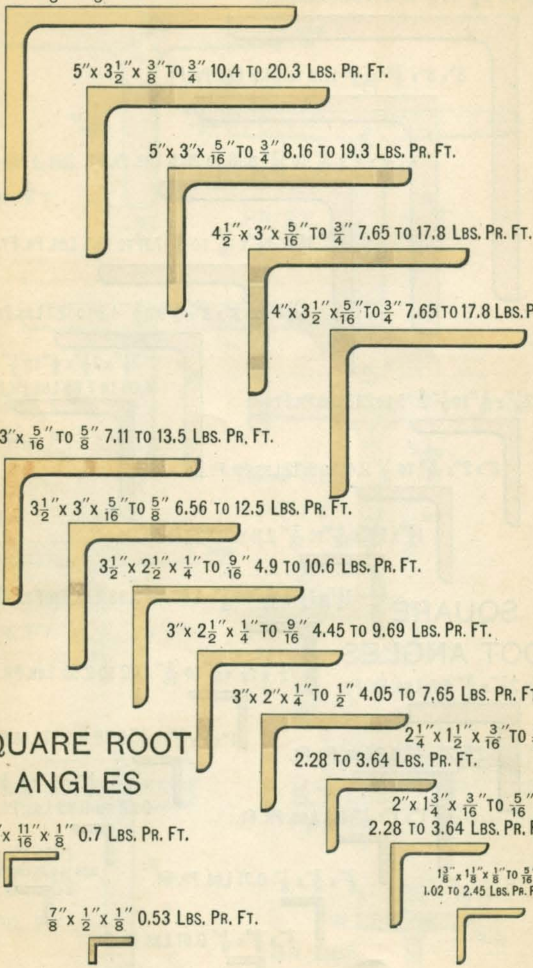
UNEQUAL TEES STEEL OR IRON



EQUAL ANGLES STEEL OR IRON



UNEQUAL ANGLES STEEL OR IRON



$6'' \times 4'' \times \frac{3}{8}''$ TO $\frac{7}{8}''$ 12.3 TO 28.4 LBS. PR. FT.

$5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ TO $\frac{3}{4}''$ 10.4 TO 20.3 LBS. PR. FT.

$5'' \times 3'' \times \frac{5}{16}''$ TO $\frac{3}{4}''$ 8.16 TO 19.3 LBS. PR. FT.

$4\frac{1}{2}'' \times 3'' \times \frac{5}{16}''$ TO $\frac{3}{4}''$ 7.65 TO 17.8 LBS. PR. FT.

$4'' \times 3\frac{1}{2}'' \times \frac{5}{16}''$ TO $\frac{3}{4}''$ 7.65 TO 17.8 LBS. PR. FT

$4'' \times 3'' \times \frac{5}{16}''$ TO $\frac{5}{8}''$ 7.11 TO 13.5 LBS. PR. FT.

$3\frac{1}{2}'' \times 3'' \times \frac{5}{16}''$ TO $\frac{5}{8}''$ 6.56 TO 12.5 LBS. PR. FT.

$3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ TO $\frac{9}{16}''$ 4.9 TO 10.6 LBS. PR. FT.

$3'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ TO $\frac{9}{16}''$ 4.45 TO 9.69 LBS. PR. FT.

$3'' \times 2'' \times \frac{1}{4}''$ TO $\frac{1}{2}''$ 4.05 TO 7.65 LBS. PR. FT.

$2\frac{1}{4}'' \times 1\frac{1}{2}'' \times \frac{3}{16}''$ TO $\frac{5}{16}''$ 2.28 TO 3.64 LBS. PR. FT.

$2'' \times 1\frac{3}{4}'' \times \frac{3}{16}''$ TO $\frac{5}{16}''$ 2.28 TO 3.64 LBS. PR. FT.

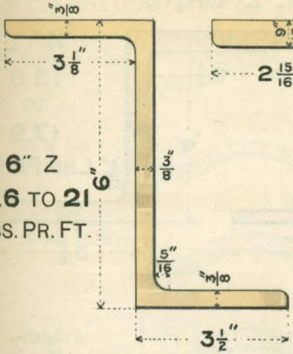
$1\frac{3}{8}'' \times 1\frac{1}{8}'' \times \frac{1}{8}''$ TO $\frac{5}{16}''$ 1.02 TO 2.45 LBS. PR. FT.

SQUARE ROOT ANGLES

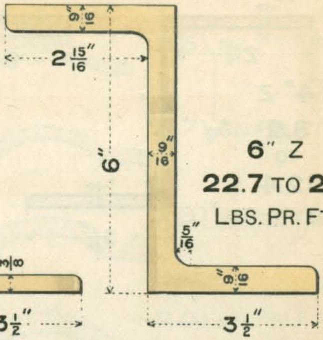
$1\frac{1}{16}'' \times \frac{11}{16}'' \times \frac{1}{8}''$ 0.7 LBS. PR. FT.

$\frac{7}{8}'' \times \frac{1}{2}'' \times \frac{1}{8}''$ 0.53 LBS. PR. FT.

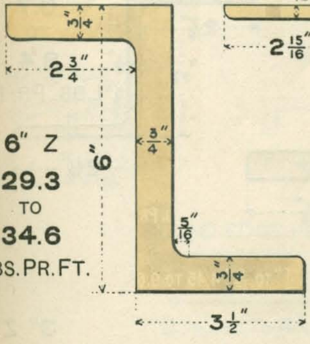
STEEL Z BARS



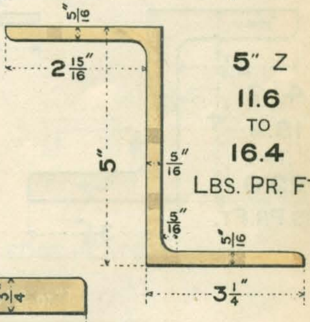
6" Z
15.6 TO 21
LBS. PR. FT.



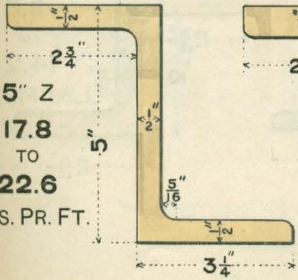
6" Z
22.7 TO 28
LBS. PR. FT.



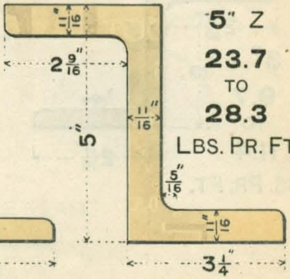
6" Z
29.3
TO
34.6
LBS. PR. FT.



5" Z
11.6
TO
16.4
LBS. PR. FT.

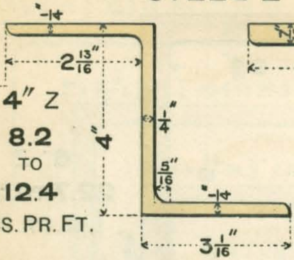


5" Z
17.8
TO
22.6
LBS. PR. FT.

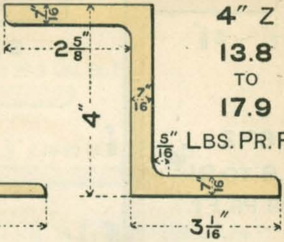


5" Z
23.7
TO
28.3
LBS. PR. FT.

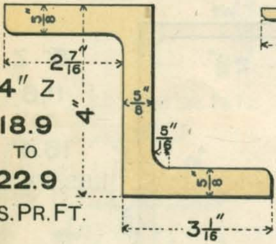
STEEL Z BARS



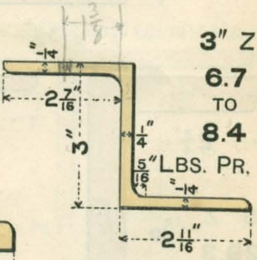
4" Z
8.2
TO
12.4
LBS. PR. FT.



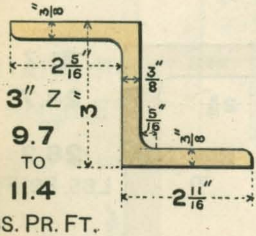
4" Z
13.8
TO
17.9
LBS. PR. FT.



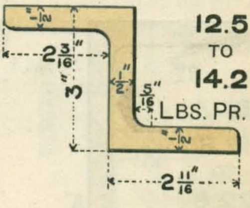
4" Z
18.9
TO
22.9
LBS. PR. FT.



3" Z
6.7
TO
8.4
LBS. PR. FT.



3" Z
9.7
TO
11.4
LBS. PR. FT.



3" Z
12.5
TO
14.2
LBS. PR. FT.

MISCELLANEOUS SHAPES

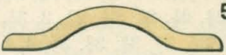
IRON ONLY

BEAD IRON.

3½" x 3/16" — 2.5 LBS. PR. FT.

4" x ¼" — 3.7 LBS. PR. FT.

4½" x 5/16" — 5.2 LBS. PR. FT.



5" x 3/8" — 7.0 LBS. PR. FT.

HAND RAIL.

2¼" x 1¼" x ¼"

GROOVES.

1½" x ¾" x ¼"

1½" x 1¼" x ¼"

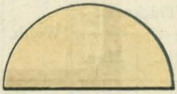
ROUND EDGE FLATS.

2½" x ¾" To 4" x 1"

HEXAGON.

7/16" To 1¼"

HALF ROUND.



3/8" To 3½"

PICTURE FRAME.

1½" x 1/8"

SIZES OF PASSAIC BARS,

STEEL OR IRON,

IN INCHES.

ROUNDS.

$\frac{3}{8}$, $\frac{7}{16}$, $\frac{1}{2}$, $\frac{9}{16}$, $\frac{5}{8}$, $\frac{11}{16}$, $\frac{3}{4}$, $\frac{13}{16}$, $\frac{7}{8}$, $\frac{15}{16}$, 1, $1\frac{1}{16}$, $1\frac{1}{8}$,
$1\frac{3}{16}$, $1\frac{1}{4}$, $1\frac{5}{16}$, $1\frac{3}{8}$, $1\frac{1}{2}$, $1\frac{5}{8}$, $1\frac{3}{4}$, $1\frac{7}{8}$, 2, $2\frac{1}{8}$,
$2\frac{1}{4}$, $2\frac{3}{8}$, $2\frac{1}{2}$, $2\frac{5}{8}$, $2\frac{3}{4}$, $2\frac{7}{8}$, 3, $3\frac{1}{8}$, $3\frac{1}{4}$,
$3\frac{3}{8}$, $3\frac{1}{2}$, $3\frac{5}{8}$, $3\frac{3}{4}$, $3\frac{7}{8}$, 4,
$4\frac{1}{4}$, $4\frac{1}{2}$, $4\frac{3}{4}$, 5.

SQUARES.

$\frac{3}{8}$, $\frac{7}{16}$, $\frac{1}{2}$, $\frac{9}{16}$, $\frac{5}{8}$, $\frac{11}{16}$, $\frac{3}{4}$, $\frac{7}{8}$, $\frac{15}{16}$, 1, $1\frac{1}{8}$, $1\frac{1}{4}$, $1\frac{3}{8}$, $1\frac{1}{2}$,
$1\frac{5}{8}$, $1\frac{3}{4}$, $1\frac{7}{8}$, 2, $2\frac{1}{4}$, $2\frac{1}{2}$, $2\frac{3}{4}$, 3, $3\frac{1}{4}$, $3\frac{1}{2}$, 4.

HALF-ROUNDS.

$\frac{3}{8}$, $\frac{7}{16}$, $\frac{1}{2}$, $\frac{9}{16}$, $\frac{5}{8}$, $\frac{11}{16}$, $\frac{3}{4}$, $\frac{13}{16}$, $\frac{7}{8}$, $\frac{15}{16}$, 1, $1\frac{1}{8}$,
$1\frac{1}{4}$, $1\frac{3}{8}$, $1\frac{1}{2}$, $1\frac{5}{8}$, $1\frac{3}{4}$, 2, $2\frac{1}{2}$, 3, $3\frac{1}{2}$.

HEXAGONS.

$\frac{7}{16}$, $\frac{1}{2}$, $\frac{5}{8}$, $\frac{11}{16}$, $\frac{3}{4}$, $\frac{7}{8}$, $\frac{15}{16}$, 1, $1\frac{1}{16}$, $1\frac{1}{8}$, $1\frac{1}{4}$.

ROUND EDGE FLATS.

$2\frac{1}{2}\times\frac{3}{4}$, $2\frac{1}{2}\times\frac{7}{8}$, $2\frac{3}{4}\times\frac{3}{4}$, $2\frac{3}{4}\times\frac{7}{8}$, $3\times\frac{7}{8}$, $4\times\frac{7}{8}$, 4×1.

FLATS.

Width.	Thickness.		Width.	Thickness.		Width.	Thickness.	
	Min.	Max.		Min.	Max.		Min.	Max.
$\frac{5}{8}$	$\frac{1}{8}$	$\frac{1}{2}$	$1\frac{3}{4}$	$\frac{1}{8}$	$1\frac{5}{8}$	$3\frac{3}{4}$	$\frac{7}{8}$	$3\frac{1}{2}$
$\frac{3}{4}$	$\frac{1}{8}$	$\frac{5}{8}$	2	$\frac{3}{16}$	$1\frac{7}{8}$	4	$\frac{1}{4}$	$3\frac{3}{4}$
$\frac{7}{8}$	$\frac{1}{8}$	$\frac{3}{4}$	$2\frac{1}{4}$	$\frac{1}{4}$	2	$4\frac{1}{4}$	$\frac{1}{4}$	$3\frac{3}{4}$
1	$\frac{1}{8}$	$\frac{7}{8}$	$2\frac{1}{2}$	$\frac{1}{4}$	$2\frac{1}{4}$	$4\frac{1}{2}$	$\frac{1}{4}$	$3\frac{3}{4}$
$1\frac{1}{8}$	$\frac{3}{8}$	1	$2\frac{3}{4}$	$\frac{1}{4}$	$2\frac{1}{2}$	5	$\frac{1}{4}$	$2\frac{1}{2}$
$1\frac{1}{4}$	$\frac{1}{8}$	1	3	$\frac{1}{4}$	$2\frac{3}{4}$	6	$\frac{1}{4}$	2
$1\frac{1}{2}$	$\frac{1}{8}$	1	$3\frac{1}{4}$	$\frac{1}{4}$	$1\frac{5}{8}$	7	$\frac{1}{4}$	$1\frac{9}{16}$
$1\frac{5}{8}$	$\frac{1}{4}$	$\frac{7}{8}$	$3\frac{1}{2}$	$\frac{1}{4}$	3	8	$\frac{1}{4}$	$1\frac{3}{4}$

PASSAIC UNIVERSAL MILL PLATES.

STEEL.

Universal mill plates can be rolled to any width between 6″ and 24″, varying in width by $\frac{1}{4}$″, and to any specified thickness from $\frac{1}{4}$″ upward, varying by $\frac{1}{16}$″, and to a maximum limit of length of 70 ft., provided the total weight of the plate does not exceed 3,000 lbs.

EXTREME LENGTHS OF UNIVERSAL PLATES, IN FEET.

Width of Plate, inches.	THICKNESS, IN INCHES.							
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1
6	40	45	60	70	70	70	70	70
7	″	″	″	″	″	″	″	″
8	″	″	″	″	″	″	″	″
9	″	″	″	″	″	″	″	″
10	″	″	″	″	″	″	″	″
11	″	″	″	″	″	″	″	″
12	″	″	″	″	″	″	″	″
13	″	″	″	″	″	″	″	68
14	″	″	″	″	″	″	″	63
15	″	″	″	″	″	″	67	59
16	″	″	″	″	″	″	63	55
17	″	″	″	″	″	69	59	52
18	″	″	″	″	″	64	56	48
19	″	″	″	″	″	62	53	46
20	″	″	″	″	″	59	50	44
21	″	″	″	″	67	56	48	42
22	″	″	″	″	64	52	45	40
23	″	″	″	″	60	50	44	38
24	″	″	″	″	58	48	42	36

METHOD OF INCREASING SECTIONAL AREAS

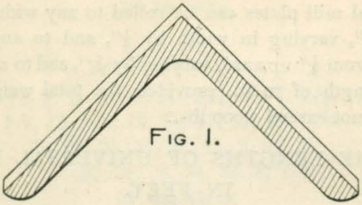


FIG. 1.

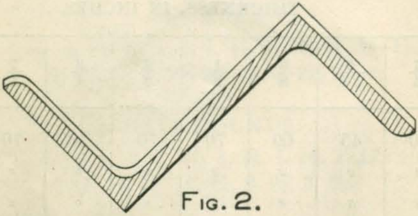


FIG. 2.

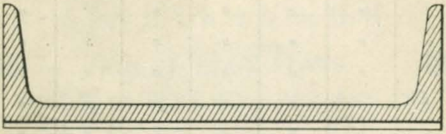


FIG. 3.

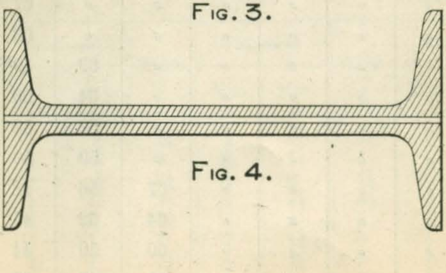


FIG. 4.

MINIMUM AND MAXIMUM

WEIGHTS AND DIMENSIONS OF PASSAIC

STEEL I BEAMS.

Depth of Beam, in inches.	Weight per foot, in lbs.		Width of Flanges, in inches.		Thickness of Web, in inches.		Increase of Web and Flanges for each lb. increase of weight.	Intermediate Weights, lbs. per foot.
	Min.	Max.	Min.	Max.	Min.	Max.		
20	**90**		6.75		0.78			
20	**80**	85	6.38	6.46	0.69	0.77	.015	
20	**65**	**75**	6.00	6.16	0.50	0.66	.015	66⅔ & 70
15	**60**	**75**	6.00	6.29	0.52	0.81	.020	**66⅔ & 70**
15	**50**	55	5.75	5.85	0.45	0.55	.020	
15	**42**	45	5.50	5.58	0.40	0.48	.020	
12	**55**	65	6.00	6.25	0.63	0.88	.025	60
12	**40**	**50**	5.50	5.75	0.39	0.64	.025	45
12	**31½**	35	5.13	5.21	0.35	0.43	.025	
10	**33**	**40**	5.00	5.21	0.37	0.58	.029	35
10	**25**	**30**	4.75	4.89	0.31	0.45	.029	27
9	**27**	33	4.75	4.95	0.31	0.51	.033	30
9	**21**	25	4.50	4.63	0.27	0.40	.033	**23⅓**
8	**22**	27	4.38	4.56	0.29	0.48	.037	25
8	**18**	20	4.13	4.20	0.25	0.32	.037	
7	**20**	22	4.09	4.17	0.28	0.36	.042	
7	**15**	17½	3.88	3.98	0.23	0.34	.042	
6	**15**	20	3.52	3.77	0.25	0.50	.049	17½
6	**12**	14	3.38	3.48	0.22	0.32	.049	13
5	**13**	15	3.13	3.25	0.26	0.38	.059	
5	**9¾**	12	3.00	3.12	0.21	0.33	.059	
4	**8**	10	2.54	2.69	0.24	0.39	.074	9
4	**6**	7½	2.19	2.50	0.18	0.20	.074	

WEIGHTS IN HEAVY-FACED TYPE ARE CONSTANTLY KEPT IN STOCK. OTHER WEIGHTS ARE ROLLED ONLY ON ORDER.

MINIMUM AND MAXIMUM WEIGHTS AND DIMENSIONS OF PASSAIC

STEEL CHANNELS.

Depth of Channel, in inches.	Weight per foot, in lbs.		Width of Flanges, in inches.		Thickness of Web, in inches.		Increase of Web and Flanges for each lb. increase of weight.	Intermediate Weights, lbs. per foot.
	Min.	Max.	Min.	Max.	Min.	Max.		
15	**40**	50	3.63	3.83	.47	.67	.020	45
15	**33**	38	3.38	3.48	.40	.50	.020	35
12	**27**	35	3.13	3.33	.38	.58	.025	**30**&**33**
12	**20**	**25**	2.88	3.00	.28	.40	.025	23
10	**20**	30	2.88	3.17	.31	.60	.029	**25**
10	**15**	**18**	2.60	2.67	.25	.32	.029	17
9	**16**	21	2.56	2.73	.28	.45	.033	18
9	**13**	15	2.36	2.43	.23	.30	.033	**14**
8	**13**	17	2.22	2.37	.25	.40	.037	**15**
8	**10**	12	2.08	2.15	.20	.27	.037	11
7	**13**	17	2.22	2.39	.28	.45	.042	15
7	**9**	12	2.00	2.13	.20	.33	.042	**10**
6	**17**	20	2.41	2.56	.38	.53	.049	18
6	**12**	15	2.19	2.34	.28	.43	.049	13
6	**8**	**10**	1.94	2.04	.20	.30	.049	9
5	**9**	12	1.91	2.09	.25	.43	.059	10
5	**6**	**8**	1.66	1.78	.18	.30	.059	7
4	**8**	10	1.86	2.01	.27	.42	.074	9
4	**5**	7	1.59	1.74	.17	.32	.074	**6**

WEIGHTS IN HEAVY-FACED TYPE ARE CONSTANTLY KEPT IN STOCK. OTHER WEIGHTS ARE ROLLED ONLY ON ORDER.

SIZES OF FINISHING GROOVES FOR PASSAIC STEEL ANGLES.

ALL DIMENSIONS ARE GIVEN IN INCHES.

EQUAL LEGS.		UNEQUAL LEGS.	
Size.	Thickness.	Size.	Thickness.
6 × 6	$\frac{3}{8}$ and $\frac{11}{16}$	6 × 4	$\frac{3}{8}$ and $\frac{5}{8}$
5 × 5	$\frac{3}{8}$ and $\frac{5}{8}$	5 × $3\frac{1}{2}$	$\frac{3}{8}$ and $\frac{5}{8}$
4 × 4	$\frac{5}{16}$, $\frac{7}{16}$ and $\frac{5}{8}$	5 × 3	$\frac{5}{16}$, $\frac{7}{16}$ and $\frac{9}{16}$
$3\frac{1}{2}$ × $3\frac{1}{2}$	$\frac{5}{16}$, $\frac{7}{16}$, $\frac{1}{2}$ and $\frac{5}{8}$	$4\frac{1}{2}$ × 3	$\frac{5}{16}$, $\frac{7}{16}$ and $\frac{5}{8}$
3 × 3	$\frac{1}{4}$, $\frac{5}{16}$ and $\frac{7}{16}$	4 × $3\frac{1}{2}$	$\frac{5}{16}$, $\frac{7}{16}$ and $\frac{5}{8}$
$2\frac{1}{2}$ × $2\frac{1}{2}$	$\frac{1}{4}$, $\frac{5}{16}$ and $\frac{7}{16}$	4 × 3	$\frac{5}{16}$, $\frac{7}{16}$ and $\frac{5}{8}$
$2\frac{1}{4}$ × $2\frac{1}{4}$	$\frac{3}{16}$, $\frac{1}{4}$ and $\frac{3}{8}$	$3\frac{1}{2}$ × 3	$\frac{5}{16}$, $\frac{3}{8}$, $\frac{1}{2}$ and $\frac{5}{8}$
2 × 2	$\frac{3}{16}$, $\frac{1}{4}$ and $\frac{3}{8}$	$3\frac{1}{2}$ × $2\frac{1}{2}$	$\frac{1}{4}$, $\frac{3}{8}$ and $\frac{1}{2}$
$1\frac{3}{4}$ × $1\frac{3}{4}$	$\frac{3}{16}$, $\frac{1}{4}$ and $\frac{3}{8}$	3 × $2\frac{1}{2}$	$\frac{1}{4}$, $\frac{3}{8}$ and $\frac{1}{2}$
$1\frac{1}{2}$ × $1\frac{1}{2}$	$\frac{3}{16}$, $\frac{1}{4}$ and $\frac{3}{8}$	3 × 2	$\frac{1}{4}$, $\frac{3}{8}$ and $\frac{1}{2}$
$1\frac{1}{4}$ × $1\frac{1}{4}$	$\frac{1}{8}$ and $\frac{3}{16}$	$2\frac{1}{4}$ × $1\frac{1}{2}$	$\frac{3}{16}$ and $\frac{5}{16}$
1 × 1	$\frac{1}{8}$ and $\frac{3}{16}$	2 × $1\frac{3}{4}$	$\frac{3}{16}$ and $\frac{5}{16}$
$\frac{7}{8}$ × $\frac{7}{8}$	$\frac{1}{8}$ and $\frac{3}{16}$	$1\frac{3}{8}$ × $1\frac{1}{8}$	$\frac{1}{8}$ and $\frac{1}{4}$
$\frac{3}{4}$ × $\frac{3}{4}$	$\frac{1}{8}$ and $\frac{3}{16}$		

When the angle is obtained from a finishing groove, the exact lengths of the legs are preserved; but for intermediate and greater thicknesses, the lengths of the legs are slightly increased. This increase of length amounts to about $\frac{1}{16}$ of an inch for each $\frac{1}{16}$ inch increase in thickness.

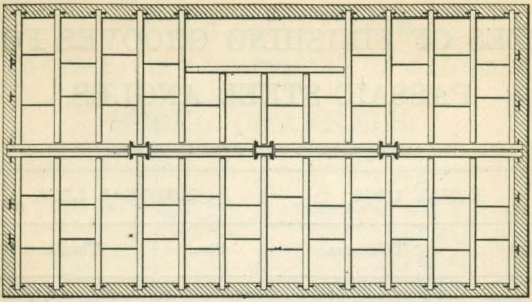


FIG.1

FIG.2

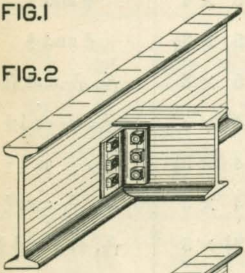


FIG.3

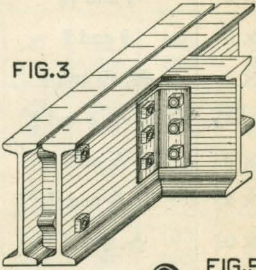


FIG.5

FIG.4

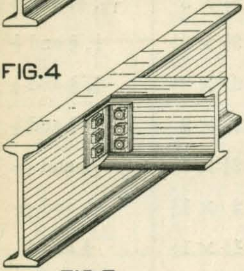


FIG.6

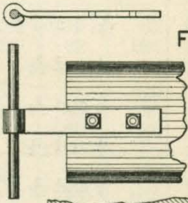


FIG.7

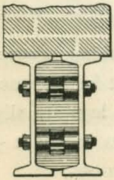
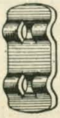


FIG.8



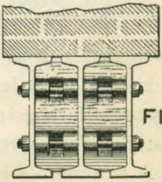
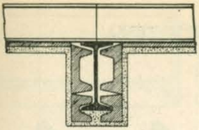
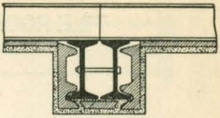


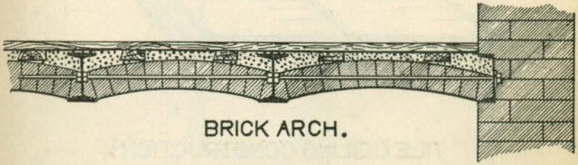
FIG.9



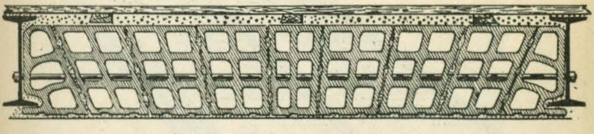
BEAM PROTECTION.



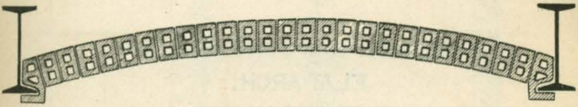
GIRDER PROTECTION.



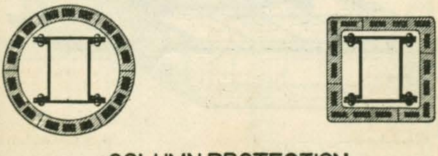
BRICK ARCH.



HOLLOW BRICK FLAT ARCH.

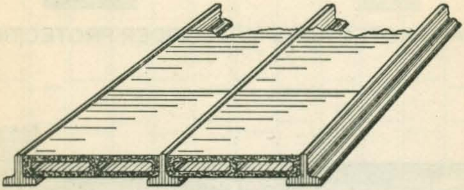


HOLLOW BRICK SEGMENTAL ARCH.

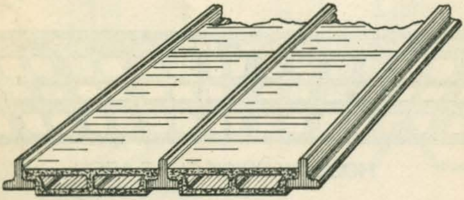


COLUMN PROTECTION.

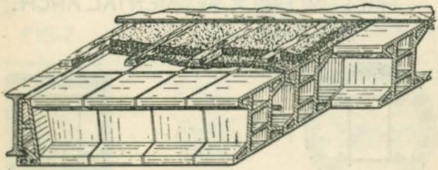
TILE ROOF CONSTRUCTION.

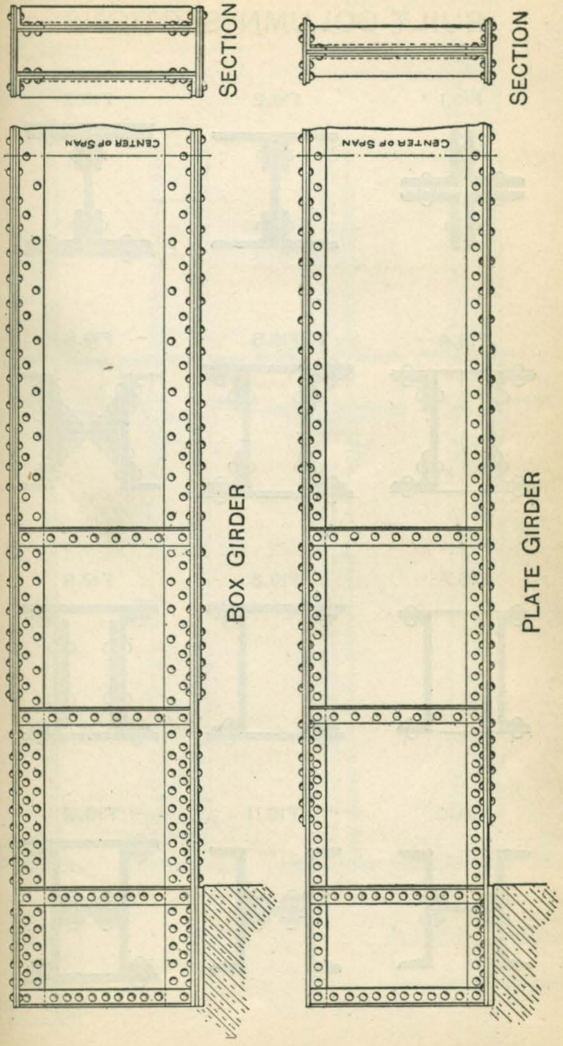


TILE CEILING CONSTRUCTION.



"EXCELSIOR" END CONSTRUCTION FLAT ARCH.




SECTION
CENTER OF SPAN
BOX GIRDER
SECTION
CENTER OF SPAN
PLATE GIRDER

BUILT-COLUMN SECTIONS

FIG.1

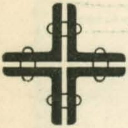


FIG.2

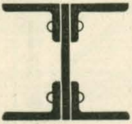


FIG.3

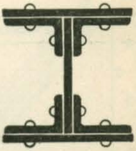


FIG.4

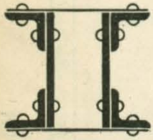


FIG.5

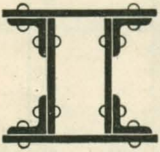


FIG.6

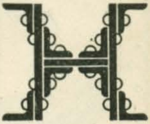


FIG.7

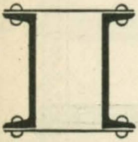


FIG.8

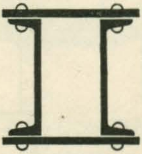


FIG.9

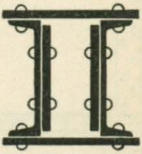


FIG.10

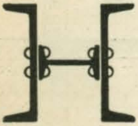


FIG.11

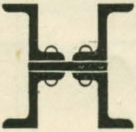
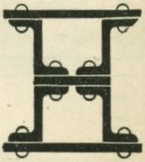
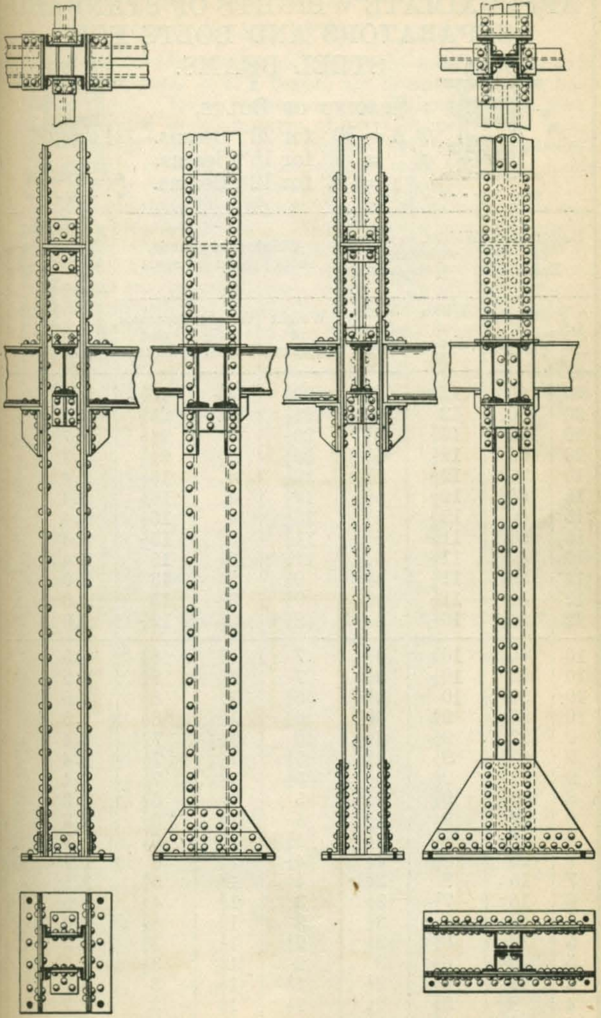


FIG.12

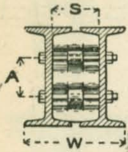


CHANNEL COLUMN Z BAR COLUMN

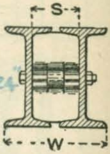


E. P. HEDDEN'S SEPARATORS
MADE LIKE PENCOYD'S.

APPROXIMATE WEIGHTS OF STANDARD SEPARATORS AND BOLTS FOR STEEL BEAMS.



SPACING OF BOLTS,
12" A = 10″ for 20″ Beams. & 24"
7½" = 7″ for 15″ Beams.
5" = 6″ for 12″ Beams.
9" = " 18" "



Designation of Beam.		Widths, in inches, with flanges ¼″ apart.		Weights, in pounds, with flanges ¼″ apart			Increase in weight of separator and bolts for one inch increase in width of girder.	Number of bolts in separator.
Depth in inches.	Weight in lbs. per foot.	Width of Girder, W	Width of Separator, S	Weight of Separator	Weight of Bolts.	Separator and Bolts.		
20	90	13¾	6¼	22¼	4½	26¾	3.7	Two ¾″ diameter bolts.
20	80	13	6	21¼	4	25¼	3.7	
20	75	12⅝	5¾	20½	4	24½	3.7	
20	65	12¼	5¾	20½	3¾	24¼	3.7	
15	75	12⅞	5¾	12¼	4	16¼	2.4	
15	66⅔	12½	5¾	12¼	4	16¼	2.4	
15	60	12¼	5¾	12¼	4	16¼	2.4	
15	50	11¾	5½	11¾	3¾	15½	2.4	
15	42	11¼	5⅜	11½	3½	15	2.4	
12	50	11¾	5⅜	9¼	3¾	13	2.0	
12	40	11¼	5⅜	9¼	3¾	13	2.0	
12	31½	10½	5	8¾	3½	12¼	2.0	
10	40	10⅝	4⅞	7	1¾	8¾	1.5	One ¾″ diameter bolt.
10	33	10¼	4⅞	7	1¾	8¾	1.5	
10	30	10	4⅝	6¾	1¾	8½	1.5	
10	25	9¾	4⅝	6¾	1¾	8½	1.5	
9	27	9¾	4⅝	6	1¾	7¾	1.4	
9	23⅓	9½	4½	5¾	1¾	7½	1.4	
9	21	9¼	4½	5¾	1¾	7½	1.4	
8	27	9⅜	4¼	5	1¾	6¾	1.3	
8	22	9	4¼	5	1¾	6¾	1.3	
8	18	8½	4⅛	4¾	1¾	6½	1.3	
7	20	8½	4⅛	4¼	1¾	6	1.1	
7	15	8	3⅞	4	1¾	5¾	1.1	
6	15	7¼	3½	3	1¾	4¾	1.0	
6	12	7	3⅜	3	1¾	4¾	1.0	
5	13	6½	3⅛	2¼	1½	3¾	0.9	
5	9¾	6¼	3	2	1½	3½	0.9	
4	10	5⅝	2½	1½	1½	3	0.7	
4	8	5¼	2½	1½	1½	3	0.7	
4	6	4⅝	2¼	1¼	1½	2¾	0.7	

STANDARD CONNECTION ANGLES

FOR PASSAIC STEEL I BEAMS.

The standard connection angles, for the principal sizes and weights of Passaic steel I beams, are illustrated on the following pages. These connections are designed on the basis of an allowable shearing strain of 9,000 lbs. per square inch, and a bearing strain of 18,000 lbs. per square inch on bolts. The number of bolts is dependent, in most instances, upon their bearing values on the webs of the beams.

The connections are proportioned to cover most cases occurring in ordinary practice. Where beams have short spans and are loaded to their full capacity, it may be found necessary to use connections having a greater number of bolts than is used in the standard connections. The minimum spans for which the standard connection angles may be used are given in the following table; and the approximate weights of the standard connections are also given.

Connection angles may be riveted to the beams, instead of being bolted, if so specified; but, unless ordered to the contrary, bolted connections are generally used.

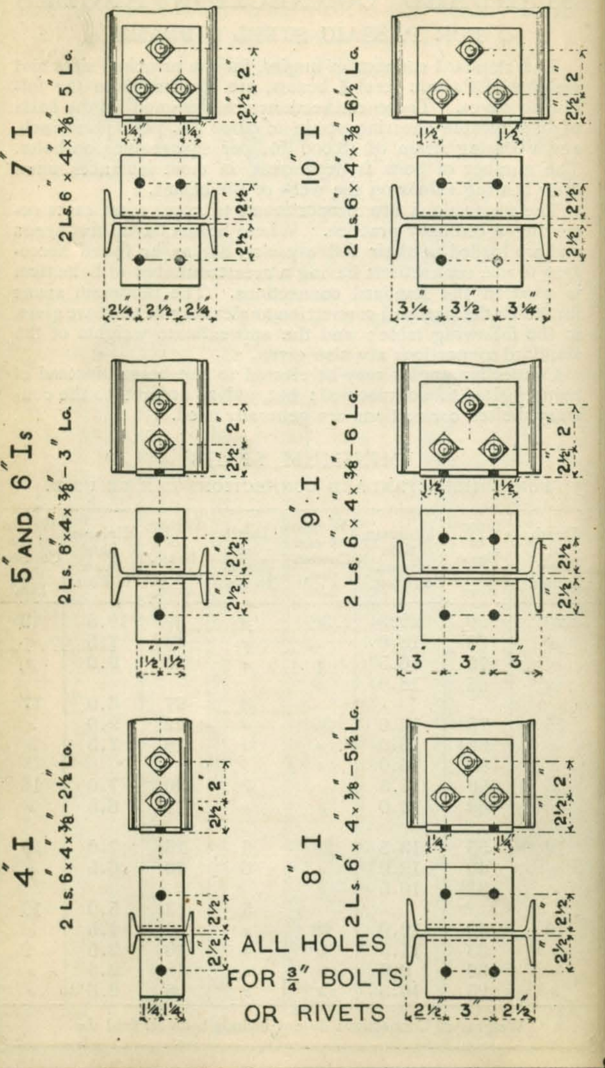
MINIMUM SPANS

FOR WHICH STANDARD CONNECTIONS CAN BE USED.

Depth of Beam, Inches.	Weight of Beam, Lbs. per Foot.	Minimum Safe Span, in Feet.	Weight of one Connection, Lbs.	Depth of Beam, Inches.	Weight of Beam, Lbs. per Foot.	Minimum Safe Span, in Feet.	Weight of one Connection, Lbs.
20	90	20.5	38	9	27	10.5	19
〃	80	18.0	〃	〃	23⅓	7.5	〃
〃	75	16.5	〃	〃	21	9.0	〃
〃	65	18.0	〃				
				8	27	6.0	17
15	75	16.0	30	〃	22	9.0	〃
〃	66⅔	15.0	〃	〃	18	7.5	〃
〃	60	16.0	〃				
〃	50	15.5	〃	7	20	7.0	16
〃	42	14.0	〃	〃	15	6.5	〃
12	55	13.5	28	6	16	7.0	10
〃	40	12.0	〃	6	13	6.5	〃
〃	31½	10.5	〃				
				5	13	5.0	10
10	40	12.0	20	〃	9¾	4.5	〃
〃	33	11.5	〃	4	10	2.5	9
〃	30	9.0	〃	〃	8	2.5	〃
〃	25	10.5	〃	〃	6	2.5	〃

Weights of Connections do not include bolts for field use.

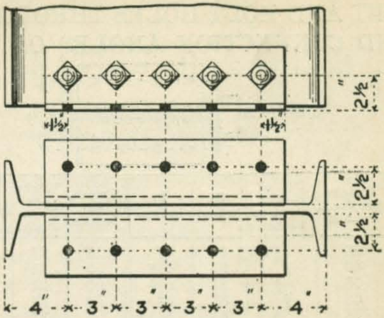
STANDARD BEAM CONNECTIONS



STANDARD BEAM CONNECTIONS

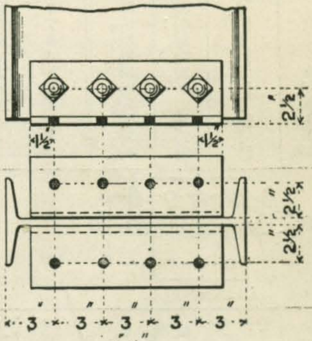
20" I

2 Ls. 4"×4"×3/8" – 1'-3" Lg.



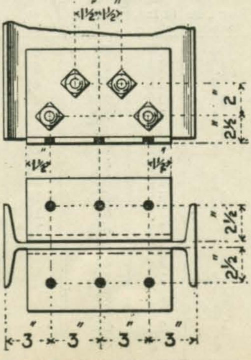
15" I

2 Ls. 4"×4"×3/8" – 1'-0" Lg.



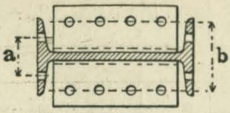
12" I

2 Ls. 6"×4"×3/8" – 9" Lg.



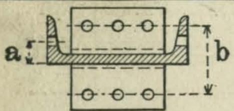
ALL HOLES FOR $\frac{3}{4}$" BOLTS OR RIVETS

STANDARD SPACING AND DIMENSIONS OF RIVET AND BOLT HOLES THROUGH FLANGES AND CONNECTION ANGLES OF I BEAMS.

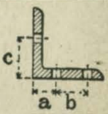


BOLT GRIP IN INCHES	Depth, in Inches.	Weight per Foot, Pounds.	Dia. of Bolt or Rivet, in Inches.	a, in Ins.	b, in Ins.
7/8	20	90	$\frac{3}{4}$	4	$5\frac{3}{4}$
3/4	20	80	$\frac{3}{4}$	$3\frac{1}{2}$	$5\frac{11}{16}$
3/4	20	75	$\frac{3}{4}$	$3\frac{1}{2}$	$5\frac{11}{16}$
3/4	20	65	$\frac{3}{4}$	$3\frac{1}{2}$	$5\frac{1}{2}$
7/8	15	75	$\frac{3}{4}$	$3\frac{1}{2}$	$5\frac{13}{16}$
7/8	15	$66\frac{2}{3}$	$\frac{3}{4}$	$3\frac{1}{2}$	$5\frac{5}{8}$
7/8	15	60	$\frac{3}{4}$	$3\frac{1}{2}$	$5\frac{1}{2}$
3/4	15	50	$\frac{3}{4}$	$3\frac{1}{4}$	$5\frac{7}{16}$
5/8	15	42	$\frac{3}{4}$	$3\frac{1}{4}$	$5\frac{3}{8}$
3/4	12	55	$\frac{3}{4}$	$3\frac{1}{4}$	$5\frac{5}{8}$
3/4	12	40	$\frac{3}{4}$	$3\frac{1}{4}$	$5\frac{3}{8}$
1/2	12	$31\frac{1}{2}$	$\frac{3}{4}$	3	$5\frac{3}{8}$
5/8	10	40	$\frac{3}{4}$	$2\frac{3}{4}$	$5\frac{9}{16}$
5/8	10	33	$\frac{3}{4}$	$2\frac{3}{4}$	$5\frac{3}{8}$
1/2	10	30	$\frac{3}{4}$	$2\frac{1}{2}$	$5\frac{7}{16}$
1/2	10	25	$\frac{3}{4}$	$2\frac{1}{2}$	$5\frac{5}{16}$

Depth, in Inches.	W'ght per Foot, P'nds.	Dia. of Bolt or Rivet, in Inches.	a, in Ins.	b, in Ins.	BOLT GRIP IN INCHES
9	27	$\frac{3}{4}$	$2\frac{1}{2}$	$5\frac{5}{16}$	5/8
9	$23\frac{1}{3}$	$\frac{3}{4}$	$2\frac{1}{2}$	$5\frac{3}{8}$	1/2
9	21	$\frac{3}{4}$	$2\frac{1}{2}$	$5\frac{1}{4}$	1/2
8	27	$\frac{3}{4}$	$2\frac{1}{4}$	$5\frac{7}{16}$	1/2
8	22	$\frac{3}{4}$	$2\frac{1}{4}$	$5\frac{1}{4}$	1/2
8	18	$\frac{3}{4}$	$2\frac{1}{4}$	$5\frac{1}{4}$	3/8
7	20	$\frac{5}{8}$	$2\frac{1}{4}$	$5\frac{1}{4}$	1/2
7	15	$\frac{5}{8}$	2	$5\frac{1}{4}$	3/8
6	15	$\frac{5}{8}$	2	$5\frac{1}{4}$	3/8
6	12	$\frac{5}{8}$	$1\frac{3}{4}$	$5\frac{1}{4}$	3/8
5	13	$\frac{1}{2}$	$1\frac{3}{4}$	$5\frac{1}{4}$	3/8
5	$9\frac{3}{4}$	$\frac{1}{2}$	$1\frac{1}{2}$	$5\frac{1}{4}$	3/8
4	10	$\frac{1}{2}$	$1\frac{1}{2}$	$5\frac{3}{8}$	1/4
4	$7\frac{1}{2}$	$\frac{1}{2}$	$1\frac{1}{2}$	$5\frac{1}{4}$	1/4
4	6	$\frac{1}{2}$	$1\frac{1}{8}$	$5\frac{3}{16}$	1/4



CHANNELS.



ANGLES.

BOLT GRIP IN INCHES	Depth, in Inches.	Weight per Foot, Pounds.	Dia. of Bolt or Rivet, in Inches.	a, in Ins.	b, in Ins.
3/4	15	40	$\frac{3}{4}$	$2\frac{1}{4}$	$5\frac{1}{2}$
5/8	15	33	$\frac{3}{4}$	$2\frac{1}{8}$	$5\frac{3}{8}$
1/2	12	27	$\frac{3}{4}$	$1\frac{7}{8}$	$5\frac{3}{8}$
1/2	12	20	$\frac{3}{4}$	$1\frac{5}{8}$	$5\frac{1}{4}$
1/2	10	20	$\frac{3}{4}$	$1\frac{3}{4}$	$5\frac{5}{16}$
3/8	10	15	$\frac{3}{4}$	$1\frac{1}{2}$	$5\frac{1}{4}$
1/2	9	16	$\frac{3}{4}$	$1\frac{3}{8}$	$5\frac{1}{4}$
3/8	9	13	$\frac{3}{4}$	$1\frac{1}{4}$	$5\frac{1}{4}$
1/2	8	13	$\frac{5}{8}$	$1\frac{1}{4}$	$5\frac{1}{4}$
3/8	8	10	$\frac{5}{8}$	$1\frac{1}{8}$	$5\frac{3}{16}$
1/2	7	13	$\frac{5}{8}$	$1\frac{1}{4}$	$5\frac{1}{4}$
3/8	7	9	$\frac{5}{8}$	$1\frac{1}{8}$	$5\frac{3}{16}$
5/8	6	17	$\frac{5}{8}$	$1\frac{3}{8}$	$5\frac{3}{8}$
1/2	6	12	$\frac{5}{8}$	$1\frac{1}{8}$	$5\frac{1}{4}$
3/8	6	8	$\frac{5}{8}$	1	$5\frac{3}{16}$
3/8	5	9	$\frac{1}{2}$	1	$5\frac{1}{4}$
1/4	5	6	$\frac{1}{2}$	$\frac{7}{8}$	$5\frac{3}{16}$
3/8	4	8	$\frac{1}{2}$	1	$5\frac{1}{4}$
1/4	4	5	$\frac{1}{2}$	$\frac{7}{8}$	$5\frac{3}{16}$

L'gth of Leg, in Inches.	Dia. of Bolt or Rivet, in Inches.	c, in Ins.	a, in Ins.	b, in Ins.
6	$\frac{7}{8}$	$4\frac{1}{2}$	$2\frac{1}{4}$	$2\frac{1}{4}$
5	$\frac{7}{8}$	$3\frac{1}{2}$	2	$1\frac{3}{4}$
$4\frac{1}{2}$	$\frac{7}{8}$	$2\frac{1}{2}$	2	$1\frac{1}{4}$
4	$\frac{7}{8}$	$2\frac{1}{4}$	$1\frac{3}{4}$	1
$3\frac{1}{2}$	$\frac{7}{8}$	2		
3	$\frac{3}{4}$	$1\frac{3}{4}$		
$2\frac{1}{2}$	$\frac{3}{4}$	$1\frac{3}{8}$		
$2\frac{1}{4}$	$\frac{5}{8}$	$1\frac{1}{4}$		
2	$\frac{5}{8}$	$1\frac{1}{8}$		
$1\frac{3}{4}$	$\frac{1}{2}$	1		
$1\frac{1}{2}$	$\frac{3}{8}$	$\frac{7}{8}$		
$1\frac{3}{8}$	$\frac{3}{8}$	$\frac{3}{4}$		

EXPLANATION OF TABLES

OF THE PROPERTIES OF PASSAIC STRUCTURAL SHAPES.

The properties of **I** beams are calculated for the principal weights of beams usually rolled. The increase of the coefficients of strength for 1 lb. increase in the weights of the beams is given, by means of which the coefficients of strength for intermediate or heavier weights of beams can be obtained, by multiplying the increase of the coefficient for 1 lb. by the number of lbs. the section is heavier than the section given in the table.

The properties of channels are given for the minimum weights of each section. The increase of the section modulus and of the coefficient of strength is given for 1 lb. increase in the weights of the channels. The coefficient of strength for the heavier weights of channels can be obtained by increasing the coefficient of strength given for the minimum weight; such increase being obtained by multiplying the increase of the coefficient for 1 lb. by the number of lbs. the section is heavier than the minimum section given. The section modulus for heavy sections may be obtained in the same way.

The properties of Tees are calculated for all weights rolled. The horizontal portion of the **T** is called the flange, and the vertical portion the stem. For the position of the neutral axis parallel to the flange, there are two values of the section modulus, and the smaller only is given, as the fiber strain calculated from it gives the greater strain in the extreme fibers.

The properties of angles are calculated for the minimum and maximum weights of each size of angle. The section modulus and the coefficient of strength for weights intermediate between the minimum and maximum are approximately proportional to the weights. There are two values of the section modulus for each position of the neutral axis, since the distance between the neutral axis and the extreme fiber is greater on one side of the axis than on the other side. The section modulus given in the table is the smaller of these two values.

The properties of **Z** bars are calculated for thicknesses varying by $\frac{1}{16}''$ for each size.

The coefficients of strength are calculated for a fiber strain of 16,000 lbs. per square inch, for all shapes. This corresponds to a strain of $\frac{1}{2}$ the elastic limit of the structural steel ordinarily used, and provides an ample margin of safety for building construction or other purposes where the loads are quiescent or nearly so. If moving loads are to be provided for, the fiber strain should not exceed 12,000 lbs. per square inch. The coefficients of strength for **I** beams and channels are also calculated for a fiber strain of 12,000 lbs. per square inch. If a load is suddenly applied, it produces an effect double that produced by the same load in a quiescent state, so that where structures are subjected to the sudden application of loads, as in railroad bridges, still smaller fiber strains than those given in the tables must be used. As the coefficients of strength are proportional to the fiber strains assumed, they can readily be determined for any assumed fiber strain by proportion. Thus, the coefficient of strength for a fiber strain of 8,000 lbs. per square inch, will be $\frac{1}{2}$ the coefficient for 16,000 lbs. fiber strain.

The coefficients of strength given in the tables furnish an easy means of determining the safe uniformly distributed load on any shape, by simply dividing the coefficient, given for the shape, by the length of the span, in feet; the quotient being the safe uniformly distributed load in lbs. Thus, if it is desired to find the safe uniformly distributed load on a 12″ × 40 lb. **I** beam on a span of 20 ft., allowing a maximum fiber strain of 16,000 lbs. per square inch, it is only necessary to divide the coefficient, 500,100, given in the table of properties, by 20; the quotient being 25,005, which is the safe load required, in lbs., including the weight of the beam itself. If a section is to be selected to sustain a certain load, for a given length of span, it will only be necessary to obtain the coefficient of strength required and refer to the tables for the section having a coefficient of that value. The coefficient required is obtained by multiplying the uniformly distributed load, in lbs., by the length of span in feet. Thus, if it is desired to find the size of an **I** beam required to carry a uniformly distributed load of 30,000 lbs., including its own weight, on a span 20 ft. between supports, allowing a fiber strain of 16,000 lbs. per square inch, the coefficient required is obtained by multiplying the load, in lbs., by the span, in feet, thus;

$$C = 30{,}000 \times 20 = 600{,}000 = \text{Coefficient required,}$$

and by reference to the table of properties of **I** beams, it will be found that a 15″ **I** beam, weighing 42 lbs. per foot, has a coefficient of strength of 611,000 and is sufficient for the purpose.

If the load is not uniformly distributed, but is concentrated at the center of the span, multiply the load by 2 and consider the result as a uniformly distributed load.

If the load is not uniformly distributed, or not concentrated at the center of the span, the bending-moment in foot-lbs. must be obtained; this bending-moment in foot-lbs. multiplied by 8 will give the coefficient required. Formulæ for the bending-moments for most cases occurring in ordinary practice are given on pages 88–92. The bending-moment will be in foot-lbs., if the lengths are taken in feet.

The section modulus is used to determine the fiber strain per square inch on a beam, or other shape, subjected to bending, by simply dividing the bending-moment expressed in inch-lbs. by the section modulus. The section modulus is also used to guide in the selection of a beam, or other shape, required to sustain a given load. The section modulus required is obtained by dividing the bending moment, in inch-lbs., by the allowable fiber strain per square inch.

The use of the radii of gyration, given in the tables of properties for all sections, is explained in connection with the tables of the strength of columns.

PROPERTIES OF PASSAIC STEEL I BEAMS.

Depth of Beam. Ins.	Weight per Foot. Lbs.	Area. Sq. Ins.	Thickness of Web. Ins.	Width of Flange. Inches.	Add to thickness of Web for each lb. increase in weight. Inches.	Moment of Inertia, neutral axis square to Web at center. I	Section Modulus, neutral axis as before. Q	Radius of Gyration, neutral axis as before, inches. r	in Foot Pounds Coefficient for fiber strain of 16,000 lbs. per sq. in. C	Add to coefficient for each lb. increase in weight of beam.	Coefficient for fiber strain of 12,000 lbs. per sq. in. C′	Add to coefficient for each lb. increase in weight of beam.	Moment of Inertia, neutral axis coincident with center line of web. I′	Radius of Gyration, neutral axis as before, inches. r′
20	90	26.4	.78	6.75	.015	1506.1	150.6	7.55	1,606,400		1,204,800		42.3	1.27
20	80	23.5	.69	6.38	.015	1345.1	134.5	7.55	1,434,700	10440	1,076,000	7830	33.2	1.19
20	75	22.1	.66	6.16	.015	1246.9	124.7	7.53	1,329,800		997,400		28.2	1.13
20	65	19.1	.50	6.00	.015	1148.6	114.9	7.76	1,225,400	10440	919,100	7830	25.5	1.16
15	75	22.1	.81	6.29	.020	720.4	96.0	5.72	1,024,400		768,300		34.6	1.25
15	66⅔	19.7	.65	6.13	.020	676.3	90.1	5.87	961,700		721,300		31.7	1.27
15	60	17.6	.52	6.00	.020	637.7	85.0	6.02	906,900	7830	680,200	5880	29.2	1.29
15	50	14.7	.45	5.75	.020	529.7	70.6	6.00	753,300	7830	565,000	5880	21.0	1.20
15	42	12.4	.40	5.50	.020	429.6	57.3	5.90	611,000	7830	458,300	5880	14.0	1.08
12	55	16.1	.63	6.00	.025	358.1	59.7	4.72	636,800	6400	477,600	4800	25.2	1.25
12	40	11.8	.39	5.50	.025	281.3	46.9	4.90	500,100	6270	375,100	4710	16.8	1.20
12	31½	9.3	.35	5.13	.025	220.5	36.7	4.88	392,100	6270	294,000	4710	10.3	1.04
10	40	11.8	.58	5.21	.029	178.5	35.7	3.89	380,800		285,600		13.5	1.07
10	33	9.7	.37	5.00	.029	161.3	32.3	4.08	344,000	5250	258,000	3930	11.8	1.10
10	30	8.8	.45	4.89	.029	134.5	26.9	3.90	287,500		215,600		8.1	0.96
10	25	7.3	.31	4.75	.029	122.5	24.5	4.06	261,200	5250	196,000	3930	7.3	0.99

PROPERTIES OF PASSAIC STEEL I BEAMS *(Continued).*

Depth of Beam.	Weight per Foot.	Area.	Thickness of Web.	Width of Flange.	Add to thickness of Web for each lb. increase in weight.	Moment of Inertia, neutral axis square to Web at center.	Section Modulus, neutral axis as before.	Radius of Gyration, neutral axis as before, inches.	Coefficient for fiber strain of 16,000 lbs. per sq. in.	Add to coefficient for each lb. increase in weight of beam.	Coefficient for fiber strain of 12,000 lbs. per sq. in.	Add to coefficient for each lb. increase in weight of beam.	Moment of Inertia, neutral axis coincident with center line of web.	Radius of Gyration, neutral axis as before, inches.
Ins.	Lbs.	Sq. Ins.	Ins.	Inches.	Inches.	I	Q	r	C		C′		I′	r′
9	27	7.9	.31	4.75	.033	110.6	24.6	3.72	262,200	4710	196,700	3540	9.10	1.07
9	23⅓	6.9	.35	4.58	.033	89.0	19.8	3.60	210,900		158,200		5.90	0.93
9	21	6.2	.27	4.50	.033	84.3	18.7	3.70	199,900	4710	149,900	3540	5.56	0.95
8	27	7.9	.48	4.56	.037	77.6	19.4	3.14	206,900		155,200		6.91	0.93
8	22	6.4	.29	4.38	.037	69.7	17.4	3.30	185,900	4170	139,400	3150	6.02	0.97
8	18	5.2	.25	4.13	.037	56.8	14.2	3.30	151,400	4170	113,500	3150	3.95	0.87
7	20	5.7	.28	4.09	.042	47.6	13.6	2.89	145,100	3660	108,800	2760	4.86	0.92
7	15	4.4	.23	3.88	.042	37.1	10.6	2.89	113,100	3660	84,800	2760	3.12	0.84
6	15	4.3	.25	3.52	.049	26.4	8.81	2.47	94,000	3150	70,500	2340	2.74	0.79
6	12	3.6	.22	3.38	.049	21.7	7.25	2.47	77,300	3150	58,000	2340	1.91	0.73
5	13	3.8	.26	3.13	.059	15.7	6.28	2.06	66,900	2610	50,200	1950	1.98	0.72
5	9¾	2.9	.21	3.00	.059	12.1	4.87	2.06	52,000	2610	38,900	1950	1.29	0.67
4	10	2.9	.39	2.69	.074	6.84	3.42	1.53	36,500		27,400		0.89	0.55
4	7½	2.2	.20	2.50	.074	5.86	2.93	1.63	31,200	2100	23,400	1560	0.70	0.56
4	6	1.8	.18	2.19	.074	4.59	2.30	1.61	24,500	2100	18,400	1560	0.38	0.47

PROPERTIES OF PASSAIC STEEL CHANNELS.

Depth of Channel.	Weight per Foot.	Area.	Thickness of Web.	Width of Flange.	Add to thickness of Web for each lb. increase in weight.	Neutral Axis perpendicular to Web at Center.								Neutral Axis parallel to back of Channel.		
						Moment of Inertia.	Section Modulus.	Add to Section Modulus for each lb. inc. in weight.	Radius of Gyration, inches.	Coefficient for fiber strain of 16,000 lbs. per square inch.	Add to Coefficient for each lb. increase in weight.	Coefficient for fiber strain of 12,000 lbs. per square inch.	Add to Coefficient for each lb. increase in weight.	Moment of Inertia.	Radius of Gyration, inches.	Center of Grav. from back of channel.
Ins.	Lbs.	Sq. In.	Ins.	Ins.	Inches.	I	Q		r	C		C′		I′	r′	Ins.
15	40	11.8	.47	3.63	.020	371.6	49.5	.735	5.62	528,400	7830	396,300	5880	11.50	.99	.89
15	33	9.7	.40	3.38	.020	304.2	40.5	.735	5.64	432,500	7830	324,400	5880	7.90	.92	.79
12	27	7.9	.38	3.13	.025	161.0	26.8	.588	4.54	286,200	6270	214,700	4710	5.73	.86	.78
12	20	5.9	.28	2.88	.025	124.7	20.8	.588	4.59	221,700	6270	166,300	4710	3.69	.79	.69
10	20	5.9	.31	2.88	.029	85.5	17.1	.492	3.81	182,400	5250	136,800	3930	3.75	.80	.74
10	15	4.4	.25	2.60	.029	66.82	13.4	.492	3.89	142,600	5250	106,900	3930	2.49	.74	.66
9	16	4.7	.28	2.56	.033	57.09	12.7	.441	3.48	135,300	4710	101,400	3540	2.59	.74	.66
9	13	3.8	.23	2.36	.033	45.48	10.1	.441	3.46	107,700	4710	80,800	3540	1.64	.64	.57
8	13	3.8	.25	2.22	.037	35.56	8.89	.393	3.07	94,800	4170	71,100	3150	1.47	.62	.58
8	10	3.0	.20	2.08	.037	28.20	7.05	.393	3.08	75,200	4170	56,400	3150	1.00	.58	.52
7	13	3.8	.28	2.22	.042	27.35	7.81	.343	2.69	83,300	3660	62,400	2750	1.89	.71	.62
7	9	2.61	.20	2.00	.042	19.05	5.43	.343	2.70	57,900	3660	43,500	2750	.814	.56	.51
6	17	4.85	.38	2.41	.049	25.43	8.48	.295	2.28	90,400	3140	67,800	2360	2.39	.70	.78
6	12	3.48	.28	2.19	.049	18.70	6.23	.295	2.32	66,400	3140	49,800	2360	1.38	.63	.65
6	8	2.35	.20	1.94	.049	12.75	4.25	.295	2.33	45,400	3140	34,000	2360	.710	.55	.52
5	9	2.59	.25	1.91	.059	9.67	3.87	.248	1.93	41,200	2650	31,000	1960	.810	.55	.57
5	6	1.76	.18	1.66	.059	6.53	2.61	.248	1.93	27,800	2650	20,900	1960	.390	.47	.45
4	8	2.31	.27	1.86	.074	5.47	2.73	.198	1.54	29,100	2110	21,800	1580	.685	.55	.59
4	5	1.46	.17	1.59	.074	3.59	1.80	.198	1.57	19,200	2110	14,400	1580	.293	.45	.46

PROPERTIES OF PASSAIC STEEL T SHAPES. EQUAL LEGS.

Size of T, in inches, flange by stem.	Thickness, Inches.	Weight per foot, Pounds.	Area of Section, Square inches.	Distance of Center of Gravity from top, Inches.	Neutral Axis parallel to Flange.				Neutral Axis square to Flange and coincident with Stem.			
					Moment of Inertia.	Section Modulus.	Coeff. of Strength.	Radius of Gyration.	Moment of Inertia.	Section Modulus.	Coeff. of Strength.	Radius of Gyration.
4 ×4	$\frac{1}{2}$	13.6	4.00	1.18	5.70	2.02	21,530	1.20	2.80	1.40	14,900	0.84
4 ×4	$\frac{3}{8}$	10.4	3.07	1.15	4.70	1.64	17,490	1.23	2.19	1.09	11,630	0.85
$3\frac{1}{2}\times3\frac{1}{2}$	$\frac{1}{2}$	11.7	3.45	1.06	3.72	1.52	16,360	1.04	1.89	1.08	11,500	0.74
$3\frac{1}{2}\times3\frac{1}{2}$	$\frac{7}{16}$	10.4	3.07	1.03	3.35	1.35	14,400	1.05	1.65	0.94	10,050	0.73
3 ×3	$\frac{1}{2}$	10.0	2.94	0.93	2.31	1.10	11,730	0.88	1.20	0.80	8,550	0.64
3 ×3	$\frac{7}{16}$	9.1	2.67	0.92	2.12	1.01	10,750	0.90	1.08	0.72	7,700	0.64
3 ×3	$\frac{3}{8}$	7.8	2.28	0.88	1.81	0.86	9,180	0.90	0.90	0.60	6,400	0.63
$2\frac{1}{2}\times2\frac{1}{2}$	$\frac{3}{8}$	6.4	1.89	0.76	1.00	0.59	6,270	0.74	0.52	0.42	4,480	0.53
$2\frac{1}{2}\times2\frac{1}{2}$	$\frac{5}{16}$	5.5	1.62	0.74	0.87	0.50	5,330	0.74	0.44	0.35	3,740	0.52
2 ×2	$\frac{5}{16}$	4.3	1.26	0.63	0.45	0.33	3,480	0.60	0.23	0.23	2,450	0.43
2 ×2	$\frac{1}{4}$	3.7	1.08	0.59	0.36	0.25	2,670	0.60	0.18	0.18	1,920	0.42
$1\frac{3}{4}\times1\frac{3}{4}$	$\frac{1}{4}$	3.1	0.90	0.54	0.23	0.19	2,070	0.51	0.12	0.14	1,490	0.37
$1\frac{3}{4}\times1\frac{3}{4}$	$\frac{3}{16}$	2.25	0.66	0.52	0.17	0.14	1,490	0.51	0.09	0.10	1,100	0.37
$1\frac{1}{2}\times1\frac{1}{2}$	$\frac{1}{4}$	2.55	0.75	0.42	0.15	0.14	1,540	0.49	0.08	0.10	1,070	0.34
$1\frac{1}{2}\times1\frac{1}{2}$	$\frac{3}{16}$	1.85	0.54	0.44	0.11	0.11	1,140	0.45	0.06	0.07	750	0.31
$1\frac{1}{4}\times1\frac{1}{4}$	$\frac{3}{16}$	1.55	0.45	0.38	0.064	0.07	780	0.37	0.031	0.05	530	0.26
1 × 1	$\frac{1}{8}$	0.9	0.26	0.29	0.022	0.031	360	0.29	0.011	0.022	230	0.21

Coefficients of Strength are calculated for a maximum fiber strain of 16,000 lbs. per square inch.

PROPERTIES OF PASSAIC STEEL T SHAPES. UNEQUAL LEGS.

Size of T, in inches, flange by stem.	Thickness, Inches.	Weight per foot, Pounds.	Area of Section, Square inches.	Distance of Center of Gravity from top, Inches.	Neutral Axis parallel to Flange.				Neutral Axis square to Flange and coincident with Stem.			
					Moment of Inertia.	Section Modulus.	Coeff. of Strength.	Radius of Gyration.	Moment of Inertia.	Section Modulus.	Coeff. of Strength.	Radius of Gyration.
6 ×4	5/8	20.6	6.06	1.04	7.66	2.58	27,550	1.13	11.50	3.83	40,800	1.38
6 ×4	1/2	17.0	5.00	0.99	6.37	2.11	22,500	1.13	9.22	3.07	32,700	1.34
5 ×3	1/2	13.5	3.97	0.75	2.60	1.15	12,300	0.83	5.23	2.09	22,400	1.18
5 ×2½	3/8	10.4	3.07	0.61	1.53	0.81	8,650	0.66	4.30	1.70	18,100	1.16
4 ×3½	1/2	12.5	3.70	1.01	3.97	1.60	17,070	1.04	2.88	1.44	15,360	0.88
4 ×3½	3/8	9.8	2.88	0.92	3.20	1.24	13,230	1.05	2.18	1.09	11,630	0.87
4 ×2	3/8	7.9	2.31	0.48	0.60	0.40	4,270	0.52	2.12	1.06	11,200	0.96
3 ×4	1/2	11.9	3.48	1.32	5.23	1.94	20,540	1.23	1.21	0.81	8,640	0.59
3 ×2	3/8	6.4	1.88	0.55	0.53	0.37	3,950	0.56	0.85	0.57	6,080	0.71
3 ×1½	3/8	5.7	1.68	0.40	0.22	0.20	2,130	0.34	0.85	0.57	6,080	0.75
2¼×1¼	1/4	3.1	0.90	0.32	0.09	0.10	1,070	0.34	0.24	0.21	2,240	0.54

Coefficients of Strength are calculated for a maximum fiber strain of 16,000 lbs. per square inch.

PROPERTIES OF PASSAIC STEEL
ANGLES
OF MAXIMUM AND MINIMUM THICKNESSES AND WEIGHTS.
EQUAL LEGS.

Size of Angle, in inches.	Thickness, inches.	Weight per Foot, pounds.	Area of Section, square inches.	Distance from Center of Gravity to Back of Flange, inches.	Moment of Inertia, axis parallel to flange. I	Section Modulus, axis as before. Q	Coefficient of Strength, axis as before. C	Radius of Gyration, axis as before. r	Least Radius of Gyration, axis diagonal. r′
6 ×6	$\frac{7}{8}$	34.0	10.03	1.87	35.3	8.17	87,100	1.87	1.20
6 ×6	$\frac{3}{8}$	14.8	4.36	1.64	15.4	3.52	37,500	1.88	1.20
5 ×5	$\frac{3}{4}$	24.2	7.11	1.56	17.0	4.78	51,000	1.55	1.00
5 ×5	$\frac{3}{8}$	12.3	3.61	1.39	8.74	2.42	25,800	1.56	1.00
4 ×4	$\frac{13}{16}$	20.8	6.11	1.35	9.45	3.32	35,400	1.24	.80
4 ×4	$\frac{5}{16}$	8.16	2.40	1.12	3.72	1.29	13,800	1.24	.80
$3\frac{1}{2}\times3\frac{1}{2}$	$\frac{5}{8}$	13.5	3.98	1.10	4.33	1.81	19,300	1.04	.70
$3\frac{1}{2}\times3\frac{1}{2}$	$\frac{5}{16}$	7.11	2.09	0.99	2.45	.98	10,400	1.08	.70
3 ×3	$\frac{5}{8}$	12.1	3.56	1.03	3.20	1.48	15,800	.94	.60
3 ×3	$\frac{1}{4}$	4.9	1.44	0.84	1.24	.58	6,190	.93	.60
$2\frac{1}{2}\times2\frac{1}{2}$	$\frac{1}{2}$	7.85	2.31	0.82	1.33	.76	8,160	.76	.50
$2\frac{1}{2}\times2\frac{1}{2}$	$\frac{1}{4}$	4.05	1.19	0.72	0.70	.40	4,270	.77	.50
$2\frac{1}{4}\times2\frac{1}{4}$	$\frac{1}{2}$	7.17	2.11	0.78	1.04	.65	6,940	.70	.45
$2\frac{1}{4}\times2\frac{1}{4}$	$\frac{3}{16}$	2.75	0.81	0.63	.39	.24	2,590	.69	.45
2 ×2	$\frac{1}{2}$	6.32	1.86	0.72	.72	.51	5,440	.62	.40
2 ×2	$\frac{3}{16}$	2.41	0.71	0.57	.28	.19	2,030	.62	.40
$1\frac{3}{4}\times1\frac{3}{4}$	$\frac{7}{16}$	4.72	1.39	0.61	.39	.32	3,450	.52	.35
$1\frac{3}{4}\times1\frac{3}{4}$	$\frac{3}{16}$	2.11	0.62	0.51	.18	.14	1,490	.54	.35
$1\frac{1}{2}\times1\frac{1}{2}$	$\frac{3}{8}$	3.33	0.98	0.51	.19	.19	2,000	.44	.30
$1\frac{1}{2}\times1\frac{1}{2}$	$\frac{3}{16}$	1.80	0.53	0.44	.110	.104	1,110	.46	.30
$1\frac{1}{4}\times1\frac{1}{4}$	$\frac{5}{16}$	2.55	0.75	0.46	.123	.134	1,370	.40	.25
$1\frac{1}{4}\times1\frac{1}{4}$	$\frac{1}{8}$	1.02	0.30	0.35	.044	.049	525	.38	.25
1 ×1	$\frac{1}{4}$	1.57	0.46	0.36	.045	.064	682	.31	.20
1 ×1	$\frac{1}{8}$	0.78	0.23	0.30	.022	.031	330	.31	.20
$\frac{7}{8}\times\frac{7}{8}$	$\frac{3}{16}$	0.99	0.29	0.29	.019	.033	352	.26	.175
$\frac{7}{8}\times\frac{7}{8}$	$\frac{1}{8}$	0.68	0.20	0.25	.014	.022	240	.27	.175
$\frac{3}{4}\times\frac{3}{4}$	$\frac{3}{16}$	0.85	0.25	0.26	.012	.024	256	.22	.15
$\frac{3}{4}\times\frac{3}{4}$	$\frac{1}{8}$	0.58	0.17	0.23	.009	.017	181	.23	.15

Coefficients of strength are calculated for a maximum fiber strain of 16,000 lbs. per square inch.

PROPERTIES OF PASSAIC STEEL ANGLES OF MAXIMUM AND MINIMUM THICKNESSES AND WEIGHTS.

UNEQUAL LEGS.

Size of Angle, in Inches.	Thickness, Ins.	Weight per foot, Lbs.	Area of Section, Square Inches.	Neutral Axis Parallel to Shorter Flange.					Neutral Axis Parallel to Longer Flange.					Least Radius of Gyration, Axis diagonal.
				Distance of Center of Gravity from back of Flange, Inches.	Moment of Inertia.	Section Modulus.	Coefficient of Strength.	Radius of Gyration.	Distance of Center of Gravity from back of Flange, Inches.	Moment of Inertia.	Section Modulus.	Coefficient of Strength.	Radius of Gyration.	
6 × 4	$\frac{7}{8}$	28.4	8.34	2.20	31.84	7.89	84,100	1.95	1.18	11.81	3.85	41,000	1.19	.98
6 × 4	$\frac{3}{8}$	12.3	3.61	1.94	13.51	3.32	35,400	1.93	.94	4.90	1.60	17,000	1.17	.92
5 × 3½	$\frac{3}{4}$	20.3	5.98	1.78	15.15	4.53	48,300	1.59	1.03	6.23	2.39	25,500	1.01	.82
5 × 3½	$\frac{3}{8}$	10.4	3.05	1.61	7.78	2.29	24,400	1.60	.86	3.18	1.21	12,900	1.02	.76
5 × 3	$\frac{3}{4}$	19.3	5.68	1.90	14.91	4.55	48,500	1.62	.88	4.27	1.85	19,700	.86	.73
5 × 3	$\frac{5}{16}$	8.16	2.40	1.68	6.26	1.89	20,100	1.61	.68	1.75	.75	7,990	.85	.71
4½ × 3	$\frac{3}{4}$	17.8	5.23	1.65	10.73	3.59	38,200	1.43	.91	3.89	1.75	18,650	.86	.72
4½ × 3	$\frac{5}{16}$	7.65	2.25	1.47	4.67	1.54	16,400	1.45	.72	1.70	.75	7,990	.87	.69
4 × 3½	$\frac{3}{4}$	17.8	5.23	1.37	8.12	2.95	31,400	1.24	1.12	5.83	2.33	24,800	1.05	.76
4 × 3½	$\frac{5}{16}$	7.65	2.25	1.18	3.57	1.24	13,200	1.26	.93	2.55	.99	10,600	1.07	.73
4 × 3	$\frac{5}{8}$	13.5	3.98	1.37	6.04	2.31	24,600	1.23	.87	2.73	1.28	13,600	.83	.66
4 × 3	$\frac{5}{16}$	7.11	2.09	1.26	3.38	1.23	13,100	1.27	.76	1.65	.74	7,890	.89	.65

Coefficients of strength are calculated for a maximum fiber strain of 16,000 lbs. per square inch.

PROPERTIES OF PASSAIC STEEL ANGLES OF MAXIMUM AND MINIMUM THICKNESSES AND WEIGHTS.

UNEQUAL LEGS.

Size of Angle, in Inches.	Thickness, Ins.	Weight per foot, Lbs.	Area of Section, Square Inches.	Neutral Axis Parallel to Shorter Flange. Distance of Center of Gravity from back of Flange, Inches.	Moment of Inertia.	Section Modulus.	Coefficient of Strength.	Radius of Gyration.	Neutral Axis Parallel to Longer Flange. Distance of Center of Gravity from back of Flange, Inches.	Moment of Inertia.	Section Modulus.	Coefficient of Strength.	Radius of Gyration.	Least Radius of Gyration, Axis diagonal.
$3\frac{1}{2} \times 3$	$\frac{5}{8}$	12.5	3.67	1.16	4.11	1.76	18,750	1.06	.92	2.75	1.32	14,080	.87	.63
$3\frac{1}{2} \times 3$	$\frac{5}{16}$	6.56	1.93	1.06	2.33	.96	10,220	1.10	.81	1.58	.72	7,680	.90	.63
$3\frac{1}{2} \times 2\frac{1}{2}$	$\frac{9}{16}$	10.6	3.13	1.24	3.76	1.62	17,280	1.10	.75	1.61	.89	9,490	.72	.58
$3\frac{1}{2} \times 2\frac{1}{2}$	$\frac{1}{4}$	4.90	1.44	1.11	1.80	.75	7,990	1.12	.61	.78	.41	4,370	.74	.55
$3 \times 2\frac{1}{2}$	$\frac{9}{16}$	9.69	2.84	1.04	2.44	1.21	12,900	.93	.79	1.53	.86	9,170	.73	.55
$3 \times 2\frac{1}{2}$	$\frac{1}{4}$	4.45	1.31	.91	1.17	.56	5,970	.95	.66	.74	.40	4,260	.75	.53
3×2	$\frac{1}{2}$	7.65	2.25	1.08	1.92	1.00	10,670	.92	.58	.67	.47	5,010	.55	.47
3×2	$\frac{1}{4}$	4.05	1.19	1.00	1.09	.54	5,760	.96	.49	.39	.26	2,770	.57	.46
$2\frac{1}{4} \times 1\frac{1}{2}$	$\frac{5}{16}$	3.64	1.07	.80	.53	.37	3,940	.70	.42	.19	.18	1,920	.42	.35
$2\frac{1}{4} \times 1\frac{1}{2}$	$\frac{3}{16}$	2.28	.67	.75	.34	.23	2,450	.72	.37	.12	.11	1,170	.43	.40
$2 \times 1\frac{3}{4}$	$\frac{5}{16}$	3.64	1.07	.65	.40	.30	3,200	.61	.52	.28	.23	2,450	.51	.36
$2 \times 1\frac{3}{4}$	$\frac{3}{16}$	2.28	.67	.60	.26	.19	2,020	.62	.47	.19	.15	1,600	.53	.36
$1\frac{3}{8} \times 1\frac{1}{8}$	$\frac{5}{16}$	2.45	.72	.51	.13	.15	1,546	.43	.38	.08	.10	1,067	.33	.25
$1\frac{3}{8} \times 1\frac{1}{8}$	$\frac{1}{8}$	1.02	.30	.42	.06	.06	619	.43	.29	:03	.04	426	.33	.24

Coefficients of strength are calculated for a maximum fiber strain of 16,000 lbs. per square inch.

AREAS
OF PASSAIC STEEL ANGLES.

Size of Angle, in Inches.	Areas, in Square Inches, for different Thicknesses.									
	$\frac{5}{16}''$	$\frac{3}{8}''$	$\frac{7}{16}''$	$\frac{1}{2}''$	$\frac{9}{16}''$	$\frac{5}{8}''$	$\frac{11}{16}''$	$\frac{3}{4}''$	$\frac{13}{16}''$	$\frac{7}{8}''$
6 × 6		4.36	5.11	5.86	6.61	7.36	7.78	8.52	9.28	10.03
6 × 4		3.61	4.23	4.86	5.48	5.86	6.48	7.11	7.73	8.34
5 × 5		3.61	4.23	4.86	5.48	5.86	6.48	7.11		
5 × 3½		3.05	3.58	4.11	4.64	4.92	5.45	5.98		
5 × 3	2.40	2.90	3.31	3.81	4.18	4.68	5.18	5.68		
4½ × 3	2.25	2.71	3.09	3.56	4.03	4.30	4.76	5.23		
4 × 4	2.40	2.90	3.31	3.81	4.31	4.61	5.11	5.61	6.11	
4 × 3½	2.25	2.71	3.09	3.56	4.03	4.30	4.76	5.23		
4 × 3	2.09	2.53	2.87	3.31	3.75	3.98				
3½ × 3½	2.09	2.53	2.87	3.25	3.69	3.98				
3½ × 3	1.93	2.30	2.71	3.00	3.41	3.67				

Size of Angle, in Inches.	Areas, in Square Inches, for different Thicknesses.									
	$\frac{1}{8}''$	$\frac{3}{16}''$	$\frac{1}{4}''$	$\frac{5}{16}''$	$\frac{3}{8}''$	$\frac{7}{16}''$	$\frac{1}{2}''$	$\frac{9}{16}''$	$\frac{5}{8}''$	$\frac{11}{16}''$
3½ × 2½			1.44	1.81	2.11	2.48	2.75	3.13		
3 × 3			1.44	1.78	2.15	2.43	2.81	3.18	3.56	
3 × 2½			1.31	1.66	1.92	2.27	2.50	2.84		
3 × 2			1.19	1.50	1.73	2.04	2.25			
2½ × 2½			1.19	1.46	1.78	2.00	2.31			
2½ × 2		.84	1.09	1.31	1.59	1.89	2.18			
2¼ × 2¼		.81	1.06	1.34	1.55	1.83	2.11			
2¼ × 1½		.67	.90	1.07						
2 × 2		.71	.94	1.19	1.36	1.61	1.86			
2 × 1¾		.67	.90	1.07						
1¾ × 1¾		.62	.81	1.03	1.17	1.39				
1½ × 1½		.53	.69	.87	.98					
1⅜ × 1⅛	.30	.45	.56	.72						
1¼ × 1¼	.30	.43	.59	.75						
1 × 1	.23	.34	.46							
⅞ × ⅞	.20	.29								
¾ × ¾	.17	.25								

WEIGHTS

OF PASSAIC STEEL ANGLES.

Size of Angle, in Inches.	Weights per foot for different thicknesses.									
	5/16″	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″	3/4″	13/16″	7/8″
6 × 6		14.8	17.4	19.9	22.5	25.0	26.4	29.0	31.5	34.0
6 × 4		12.3	14.4	16.6	18.6	19.9	22.0	24.2	26.2	28.4
5 × 5		12.3	14.4	16.5	18.6	19.9	21.8	24.2		
5 × 3½		10.4	12.2	14.0	15.8	16.7	18.5	20.3		
5 × 3	8.16	9.86	11.2	13.0	14.2	15.9	17.6	19.3		
4½ × 3	7.65	9.21	10.5	12.1	13.7	14.6	16.2	17.8		
4 × 4	8.16	9.86	11.2	12.9	14.7	15.7	17.4	19.1	20.8	
4 × 3½	7.65	9.21	10.5	12.1	13.7	14.6	16.2	17.8		
4 × 3	7.11	8.60	9.80	11.3	12.7	13.5				
3½ × 3½	7.11	8.60	9.76	11.0	12.5	13.5				
3½ × 3	6.56	7.82	9.21	10.2	11.6	12.5				

Size of Angle, in Inches.	Weights per foot for different thicknesses.									
	1/8″	3/16″	1/4″	5/16″	3/8″	7/16″	1/2″	9/16″	5/8″	11/16″
3½ × 2½			4.90	6.15	7.17	8.43	9.35	10.6		
3 × 3			4.90	6.05	7.30	8.26	9.56	10.8	12.1	
3 × 2½			4.45	5.64	6.53	7.72	8.50	9.69		
3 × 2			4.05	5.10	5.88	6.94	7.65			
2½ × 2½			4.05	4.96	6.05	6.80	7.85			
2½ x 2		2.75	3.70	4.45	5.40	6.42	7.45			
2¼ × 2¼		2.75	3.60	4.56	5.20	6.22	7.17			
2¼ × 1½		2.28	3.06	3.64						
2 × 2		2.41	3.19	4.05	4.62	5.47	6.32			
2 × 1¾		2.28	3.06	3.64						
1¾ × 1¾		2.11	2.75	3.50	3.98	4.72				
1½ × 1½		1.80	2.35	2.96	3.33					
1⅜ × 1⅛	1.02	1.53	1.90	2.45						
1¼ × 1¼	1.02	1.46	2.01	2.55						
1 × 1	.78	1.15	1.57							
⅞ × ⅞	.68	.99								
¾ × ¾	.58	.85								

PROPERTIES OF PASSAIC STEEL Z BARS.

Depth of Web, Ins.	Width of Flange, Ins.	Thickness of Metal, Ins.	Weight per Foot, Lbs.	Area of Section, Sq. Ins.	Neutral Axis perpendicular to Web.				Neutral Axis coincident with Web.				Least Radius of Gyration, neut. axis diagonal.
					Mom't of Inertia.	Section Modulus.	Rad. of Gyration.	Coeff. of Strength.	Mom't of Inertia.	Section Modulus.	Rad. of Gyration.	Coeff. of Strength.	
6	$3\frac{1}{2}$	$\frac{3}{8}$	15.6	4.59	25.32	8.44	2.35	90,000	9.11	2.75	1.41	67,500	0.83
$6\frac{1}{16}$	$3\frac{9}{16}$	$\frac{7}{16}$	18.3	5.39	29.80	9.83	2.35	104,800	10.95	3.27	1.43	78,600	0.84
$6\frac{1}{8}$	$3\frac{5}{8}$	$\frac{1}{2}$	21.0	6.19	34.36	11.22	2.36	119,700	12.87	3.81	1.44	89,800	0.84
6	$3\frac{1}{2}$	$\frac{9}{16}$	22.7	6.68	34.64	11.55	2.28	123,200	12.59	3.91	1.37	92,400	0.81
$6\frac{1}{16}$	$3\frac{9}{16}$	$\frac{5}{8}$	25.4	7.46	38.86	12.82	2.28	136,700	14.42	4.43	1.39	102,600	0.82
$6\frac{1}{8}$	$3\frac{5}{8}$	$\frac{11}{16}$	28.0	8.25	43.18	14.10	2.29	150,400	16.34	4.98	1.41	112,800	0.84
6	$3\frac{1}{2}$	$\frac{3}{4}$	29.3	8.63	42.12	14.04	2.21	149,800	15.44	4.94	1.34	112,300	0.81
$6\frac{1}{16}$	$3\frac{9}{16}$	$\frac{13}{16}$	32.0	9.40	46.13	15.22	2.22	162,300	17.27	5.47	1.36	121,800	0.82
$6\frac{1}{8}$	$3\frac{5}{8}$	$\frac{7}{8}$	34.6	10.17	50.22	16.40	2.22	174,900	19.18	6.02	1.37	131,200	0.83
5	$3\frac{1}{4}$	$\frac{5}{16}$	11.6	3.40	13.36	5.34	1.98	57,000	6.18	2.00	1.35	42,700	0.75
$5\frac{1}{16}$	$3\frac{5}{16}$	$\frac{3}{8}$	13.9	4.10	16.18	6.39	1.99	68,200	7.65	2.45	1.37	51,100	0.76
$5\frac{1}{8}$	$3\frac{3}{8}$	$\frac{7}{16}$	16.4	4.81	19.07	7.44	1.99	79,400	9.20	2.92	1.38	59,500	0.77
5	$3\frac{1}{4}$	$\frac{1}{2}$	17.8	5.25	19.19	7.68	1.91	81,900	9.05	3.02	1.31	61,400	0.74
$5\frac{1}{16}$	$3\frac{5}{16}$	$\frac{9}{16}$	20.2	5.94	21.83	8.62	1.91	91,900	10.51	3.47	1.33	69,000	0.75
$5\frac{1}{8}$	$3\frac{3}{8}$	$\frac{5}{8}$	22.6	6.64	24.53	9.57	1.92	102,100	12.06	3.94	1.35	76,600	0.76
5	$3\frac{1}{4}$	$\frac{11}{16}$	23.7	6.96	23.68	9.47	1.84	101,000	11.37	3.91	1.28	75,800	0.73
$5\frac{1}{16}$	$3\frac{5}{16}$	$\frac{3}{4}$	26.0	7.64	26.16	10.34	1.85	110,300	12.83	4.37	1.30	82,700	0.75
$5\frac{1}{8}$	$3\frac{3}{8}$	$\frac{13}{16}$	28.3	8.33	29.31	11.44	1.88	122,000	14.36	4.84	1.31	91,500	0.76

Coefficients are calculated for a maximum fiber strain of 16,000 lbs. per square inch.

PROPERTIES OF PASSAIC STEEL Z BARS *(Continued).*

Depth of Web, Ins.	Width of Flange, Ins.	Thickness of Metal, Ins.	Weight per Foot, Lbs.	Area of Section, Sq. Ins.	Neutral Axis perpendicular to Web.				Neutral Axis coincident with Web.				Least Radius of Gyration, neut. axis diagonal.
					Mom't of Inertia.	Section Modulus.	Rad. of Gyration.	Coeff. of Strength.	Mom't of Inertia.	Section Modulus.	Rad. of Gyration.	Coeff. of Strength.	
4	$3\frac{1}{16}$	$\frac{1}{4}$	8.2	2.41	6.28	3.14	1.62	33,500	4.23	1.44	1.33	25,100	0.67
$4\frac{1}{16}$	$3\frac{1}{8}$	$\frac{5}{16}$	10.3	3.03	7.94	3.91	1.62	41,700	5.46	1.84	1.34	31,300	0.68
$4\frac{1}{8}$	$3\frac{3}{16}$	$\frac{3}{8}$	12.4	3.66	9.63	4.67	1.62	49,800	6.77	2.26	1.36	37,400	0.69
4	$3\frac{1}{16}$	$\frac{7}{16}$	13.8	4.05	9.66	4.83	1.55	51,500	6.73	2.37	1.29	38,600	0.66
$4\frac{1}{16}$	$3\frac{1}{8}$	$\frac{1}{2}$	15.8	4.66	11.18	5.50	1.55	58,700	7.96	2.77	1.31	44,000	0.67
$4\frac{1}{8}$	$3\frac{3}{16}$	$\frac{9}{16}$	17.9	5.27	12.74	6.18	1.55	65,900	9.26	3.19	1.33	49,400	0.69
4	$3\frac{1}{16}$	$\frac{5}{8}$	18.9	5.55	12.11	6.05	1.48	64,500	8.73	3.18	1.25	48,400	0.66
$4\frac{1}{16}$	$3\frac{1}{8}$	$\frac{11}{16}$	20.9	6.14	13.52	6.65	1.48	70,900	9.95	3.58	1.27	53,200	0.67
$4\frac{1}{8}$	$3\frac{3}{16}$	$\frac{3}{4}$	22.9	6.75	14.97	7.26	1.49	77,400	11.24	4.00	1.29	58,100	0.69
3	$2\frac{11}{16}$	$\frac{1}{4}$	6.7	1.97	2.87	1.92	1.21	20,500	2.81	1.10	1.19	15,400	0.55
$3\frac{1}{16}$	$2\frac{3}{4}$	$\frac{5}{16}$	8.4	2.48	3.64	2.38	1.21	25,400	3.64	1.40	1.21	19,000	0.56
3	$2\frac{11}{16}$	$\frac{3}{8}$	9.7	2.86	3.85	2.57	1.16	27,400	3.92	1.57	1.17	20,600	0.55
$3\frac{1}{16}$	$2\frac{3}{4}$	$\frac{7}{16}$	11.4	3.36	4.57	2.98	1.17	31,800	4.75	1.88	1.19	23,800	0.56
3	$2\frac{11}{16}$	$\frac{1}{2}$	12.5	3.69	4.59	3.06	1.12	32,600	4.85	1.99	1.15	24,500	0.55
$3\frac{1}{16}$	$2\frac{3}{4}$	$\frac{9}{16}$	14.2	4.19	5.26	3.43	1.12	36,600	5.70	2.31	1.17	27,400	0.56

Coefficients are calculated for a maximum fiber strain of 16,000 lbs. per square inch.

EXPLANATION OF TABLES ON SAFE LOADS.

The following tables give the safe uniformly distributed loads, in tons of 2,000 lbs., on Passaic Structural Shapes calculated for a maximum fiber strain of 16,000 lbs. per square inch. The loads given in the tables include the weights of the shapes, which must be deducted from the tabular loads in order to obtain the net superimposed loads which the shapes will carry.

Safe loads are given for the principal weights of **I** beams usually rolled. The safe loads for intermediate or heavier weights of beams than those tabulated, can be obtained by the use of the separate column of corrections given for each size, which states the increase of safe load for each additional lb. increase in the weight per foot of the beam.

The safe loads of channels are tabulated only for the minimum weights. A separate column for each depth of channel gives the additional safe load for each lb. per foot increase in the weight of the channel, by the use of which the safe loads on the heavier weights of channels may be obtained.

The safe loads for Tees are given for all weights rolled.

The safe loads for Angles are given only for the minimum and maximum weights. The safe loads for intermediate weights may be obtained approximately by proportion.

The safe loads for **Z** Bars are given for all the weights rolled.

It is assumed in these tables that the compression flanges of the beams or shapes are secured against yielding sideways. They should be held in position at distances not exceeding 20 times the width of the flange, otherwise the allowable loads should be reduced according to the following table:

BEAMS UNSUPPORTED SIDEWAYS.

Unsupported Length of Beam.	Greatest Safe Load.	Unsupported Length of Beam.	Greatest Safe Load.
20 × flange width.	1.0 tabular load.	50 × flange width.	0.7 tabular load.
30 " " "	0.9 " "	60 " " "	0.6 " "
40 " " "	0.8 " "	70 " " "	0.5 " "

Deflection Coefficients are given for all the shapes, by the use of which the deflections, under the tabular loads, can be obtained by simply multiplying the Deflection Coefficient of the shape by the square of the span, in feet; the result being the deflection in inches. Thus, the deflection of a 15″ × 42 lb. **I** beam on a span of 20 feet, fully loaded, is obtained by multiplying the Deflection Coefficient (.001103) by $\overline{20}^2$; the result being 0.44, which is the deflection in inches, or about $\frac{7}{16}''$.

Beams used in floors should not only be strong enough to carry the superimposed loads, but also sufficiently rigid to prevent vibration. For beams carrying plastered ceilings, if the deflection exceeds $\frac{1}{360}$ of the distance between supports, or $\frac{1}{30}$ of an inch per foot of span, there is danger of cracking the plaster. This limit is indicated in the tables by heavy cross lines beyond which the beams should not be used if intended to carry plastered ceilings, unless the allowable loads given in the tables are reduced in the following manner:

Let $\triangle$ = deflection coefficient for the shape.

L = limiting span, in feet, at which the shape, fully loaded, has a deflection of $\frac{1}{360}$ of span.

L' = given span, in feet.

W' = tabular safe load for span L'.

W'' = load on span L' producing deflection of $\frac{1}{360}$ of span.

Then,

$$L = \frac{1}{30\,\triangle},\ (1);\quad W'' = \frac{W'}{30\,\triangle\,L'},\ (2);\quad W'' = \frac{L}{L'}\,W',\ (3).$$

Thus, if it is desired to find the load on a 10″ × 25 lb. **I** beam on a span of 30 ft., which will produce a deflection of only $\frac{1}{360}$ of the span; the safe load, 4.35 tons, given in the table for a span of 30 feet, must be reduced by formula (3) as follows:

$$W'' = \frac{20}{30} \times 4.35 = 2.90 \text{ tons.}$$

It may generally be assumed that the above limit of deflection is not exceeded, both for rolled and built beams, unless the depth of the beam is less than $\frac{1}{24}$ of the span. It should be noted, however, that some local building ordinances provide that no beam shall be of less depth than $\frac{1}{20}$ of the span.

Approximate Coef. of any Beam = 1000 x depth in inches x weight in lbs. per ft.

SAFE LOADS, UNIFORMLY DISTRIBUTED, FOR PASSAIC STEEL I BEAMS,

IN TONS OF 2000 LBS.,

BEAMS BEING SECURED AGAINST YIELDING SIDEWAYS.

Span, in Feet.	20″ I				Add for ea. Lb. Inc. in Weight.	15″ I					Add for ea. Lb. Inc. in Weight.
	90 Lbs. per Foot.	80 Lbs. per Foot.	75 Lbs. per Foot.	65 Lbs. per Foot.		75 Lbs. per Foot.	66⅔ Lbs. per Foot.	60 Lbs. per Foot.	50 Lbs. per Foot.	42 Lbs. per Foot.	
10	80.3	71.7	66.5	61.3	0.52	51.2	48.1	45.4	37.7	30.6	0.39
11	73.0	65.2	60.5	55.7	0.48	46.6	43.7	41.2	34.2	27.8	0.36
12	66.9	59.8	55.4	51.0	0.44	42.7	40.1	37.8	31.4	25.4	0.33
13	61.8	55.2	51.2	47.1	0.40	39.4	37.0	34.9	29.0	23.5	0.30
14	57.4	51.2	47.5	43.8	0.37	36.6	34.3	32.4	26.9	21.8	0.28
15	53.6	47.8	44.3	40.9	0.35	34.2	32.1	30.2	25.1	20.4	0.26
16	50.2	44.8	41.6	38.3	0.33	32.0	30.1	28.3	23.5	19.2	0.25
17	47.3	42.2	39.1	36.0	0.31	30.1	28.3	26.7	22.2	17.9	0.23
18	44.6	39.9	36.9	34.1	0.29	28.5	26.7	25.2	20.9	17.0	0.22
19	42.3	37.8	35.0	32.3	0.28	27.0	25.3	23.9	19.8	16.1	0.21
20	40.2	35.9	33.3	30.7	0.26	25.6	24.0	22.7	18.8	15.3	0.20
21	38.3	34.2	31.7	29.2	0.25	24.4	22.8	21.6	17.9	14.6	0.19
22	36.5	32.6	30.2	27.8	0.24	23.3	21.8	20.6	17.1	13.9	0.18
23	34.9	31.2	28.9	26.6	0.23	22.3	20.9	19.7	16.4	13.3	0.17
24	33.5	29.9	27.7	25.5	0.22	21.3	20.0	18.9	15.7	12.8	0.16
25	32.1	28.7	26.6	24.5	0.21	20.5	19.2	18.1	15.1	12.3	0.16
26	30.9	27.6	25.6	23.6	0.20	19.7	18.4	17.4	14.5	11.8	0.15
27	29.8	26.6	24.6	22.7	0.19	19.0	17.8	16.8	14.0	11.4	0.15
28	28.7	25.6	23.8	21.9	0.19	18.3	17.1	16.2	13.5	10.9	0.14
29	27.7	24.7	22.9	21.2	0.18	17.7	16.5	15.6	13.0	10.5	0.14
30	26.8	23.9	22.2	20.5	0.17	17.1	16.0	15.1	12.6	10.2	0.13
31	25.9	23.1	21.5	19.8	0.17	16.5	15.5	14.6	12.2	9.86	0.13
32	25.1	22.4	20.8	19.2	0.16	16.0	15.0	14.2	11.8	9.56	0.13
33	24.3	21.7	20.2	18.6	0.16	15.5	14.5	13.7	11.4	9.26	0.12
34	23.6	21.1	19.6	18.1	0.15	15.1	14.1	13.3	11.1	8.98	0.11
35	23.0	20.5	19.0	17.6	0.15	14.6	13.7	13.0	10.8	8.73	0.11
36	22.3	19.9	18.5	17.1	0.15	14.2	13.3	12.6	10.5	8.49	0.11
37	21.7	19.4	18.0	16.5	0.14	13.8	13.0	12.3	10.2	8.26	0.11
38	21.1	18.9	17.5	16.1	0.14	13.5	12.6	11.9	9.91	8.04	0.10
39	20.6	18.4	17.1	15.7	0.13	13.1	12.3	11.6	9.66	7.83	0.10
40	20.1	17.9	16.6	15.3	0.13	12.8	12.0	11.3	9.42	7.64	0.10
	Deflection Coefficient, .000828					Deflection Coefficient, .001103					

Safe loads given include weight of beam. Maximum fiber strain, 16,000 lbs. per square inch. Deflection of beam, in inches, under tabular load equals the product of the Deflection Coefficient by the square of the span, in feet.

FOR SPANS ABOVE RED ZIGZAG LINE

Above minimum sections the increased weight has but ½ the value of itself.

SAFE LOADS, UNIFORMLY DISTRIBUTED, FOR PASSAIC STEEL I BEAMS,

IN TONS OF 2000 LBS.,

BEAMS BEING SECURED AGAINST YIELDING SIDEWAYS.

Span, in Feet.	12″ I			Add for ea. Lb. Inc. in Weight.	10″ I				Add for ea. Lb. Inc. in Weight.
	55 Lbs. per Ft.	40 Lbs. per Ft.	31½ Lbs. per Ft.		40 Lbs. per Ft.	33 Lbs. per Ft.	30 Lbs. per Ft.	25 Lbs. per Ft.	
8	39.8	31.3	24.5	0.39	23.8	21.5	18.0	16.3	0.33
9	35.4	27.8	21.8	0.35	21.2	19.1	16.0	14.5	0.29
10	31.8	25.0	19.6	0.31	19.0	17.2	14.4	13.1	0.26
11	28.8	22.7	17.9	0.29	17.3	15.6	13.1	11.9	0.24
12	26.5	20.8	16.4	0.26	15.9	14.3	12.0	10.9	0.22
13	24.5	19.2	15.1	0.24	14.7	13.2	11.1	10.1	0.20
14	22.8	17.9	14.0	0.22	13.6	12.3	10.3	9.33	0.19
15	21.2	16.7	13.1	0.21	12.7	11.5	9.58	8.71	0.17
16	19.9	15.6	12.3	0.20	11.9	10.8	8.98	8.16	0.16
17	18.7	14.7	11.5	0.18	11.2	10.1	8.46	7.68	0.15
18	17.7	13.9	10.9	0.17	10.6	9.56	7.99	7.26	0.15
19	16.8	13.2	10.3	0.17	10.0	9.05	7.57	6.87	0.14
20	15.9	12.5	9.80	0.16	9.52	8.60	7.19	6.53	0.13
21	15.2	11.9	9.33	0.15	9.07	8.19	6.85	6.22	0.12
22	14.4	11.4	8.91	0.14	8.65	7.82	6.53	5.94	0.12
23	13.8	10.9	8.52	0.14	8.28	7.48	6.25	5.68	0.11
24	13.3	10.4	8.16	0.13	7.93	7.17	5.99	5.44	0.11
25	12.7	10.0	7.83	0.13	7.62	6.88	5.75	5.22	0.10
26	12.2	9.62	7.54	0.12	7.32	6.62	5.53	5.02	0.10
27	11.8	9.26	7.26	0.12	7.05	6.37	5.32	4.84	0.10
28	11.4	8.93	7.00	0.11	6.80	6.14	5.13	4.66	0.09
29	11.0	8.62	6.76	0.11	6.57	5.93	4.96	4.50	0.09
30	10.6	8.34	6.54	0.10	6.35	5.73	4.79	4.35	0.09
31	10.3	8.07	6.32	0.10	6.14	5.54	4.64	4.21	0.08
32	10.0	7.81	6.13	0.10	5.95	5.38	4.49	4.08	0.08
33	9.6	7.58	5.94	0.10	5.77	5.21	4.36	3.96	0.08
34	9.4	7.35	5.76	0.09	5.60	5.06	4.23	3.84	0.08
35	9.1	7.14	5.60	0.09	5.44	4.91	4.11	3.73	0.08
	Deflection Coefficient, .001379				Deflection Coefficient, .001655				

Safe loads given include weight of beam. Maximum fiber strain, 16,000 lbs. per square inch. Deflection of beam, in inches, under tabular load equals the product of the Deflection Coefficient by the square of the span, in feet.

SPECIAL CONNECTIONS MUST BE DESIGNED.

SAFE LOADS, UNIFORMLY DISTRIBUTED, FOR PASSAIC STEEL I BEAMS,

IN TONS OF 2000 LBS.,

BEAMS BEING SECURED AGAINST YIELDING SIDEWAYS.

Span, in Feet.	9″ I			Add for ea. Lb. Inc. in Weight.	8″ I			Add for ea. Lb. Inc. in Weight.	7″ I		Add for ea. Lb. Inc. in Weight.
	27 Lbs. per Foot.	23⅓ Lbs. per Foot.	21 Lbs. per Foot.		27 Lbs. per Foot.	22 Lbs. per Foot.	18 Lbs. per Foot.		20 Lbs. per Foot.	15 Lbs. per Foot.	
7	18.7	15.1	14.3	0.34	14.8	13.3	10.8	0.30	10.4	8.08	0.26
8	16.4	13.2	12.5	0.29	12.9	11.6	9.45	0.26	9.07	7.07	0.23
9	14.6	11.7	11.1	0.26	11.5	10.3	8.41	0.23	8.06	6.28	0.20
10	13.1	10.6	10.0	0.24	10.3	9.30	7.57	0.21	7.26	5.66	0.18
11	11.9	9.59	9.09	0.21	9.40	8.45	6.88	0.19	6.60	5.14	0.17
12	10.9	8.79	8.33	0.20	8.62	7.75	6.31	0.17	6.05	4.71	0.15
13	10.1	8.11	7.69	0.18	7.96	7.15	5.82	0.16	5.58	4.35	0.14
14	9.36	7.53	7.14	0.17	7.39	6.64	5.41	0.15	5.18	4.04	0.13
15	8.74	7.03	6.66	0.16	6.90	6.17	5.05	0.14	4.84	3.77	0.12
16	8.19	6.59	6.25	0.15	6.47	5.81	4.73	0.13	4.53	3.53	0.11
17	7.71	6.20	5.88	0.14	6.08	5.47	4.45	0.12	4.27	3.33	0.11
18	7.28	5.86	5.55	0.13	5.75	5.16	4.21	0.12	4.03	3.14	0.10
19	6.90	5.55	5.26	0.12	5.44	4.89	3.98	0.11	3.82	2.98	0.10
20	6.56	5.27	5.00	0.12	5.17	4.65	3.79	0.10	3.63	2.83	0.09
21	6.24	5.02	4.76	0.11	4.93	4.43	3.60	0.10	3.45	2.69	0.09
22	5.96	4.79	4.54	0.11	4.70	4.23	3.44	0.09	3.30	2.57	0.08
23	5.70	4.58	4.35	0.10	4.50	4.04	3.29	0.09	3.15	2.46	0.08
24	5.46	4.39	4.16	0.10	4.31	3.87	3.15	0.09	3.02	2.36	0.08
25	5.24	4.22	4.00	0.09	4.14	3.72	3.03	0.08	2.90	2.26	0.07
26	5.04	4.06	3.84	0.09	3.98	3.58	2.91	0.08	2.79	2.18	0.07
27	4.86	3.91	3.70	0.09	3.83	3.44	2.80	0.08	2.69	2.09	0.07
28	4.68	3.77	3.57	0.08	3.69	3.32	2.70	0.07	2.59	2.02	0.07
29	4.52	3.64	3.45	0.08	3.57	3.21	2.61	0.07	2.50	1.95	0.06
30	4.37	3.52	3.33	0.08	3.45	3.10	2.52	0.07	2.42	1.88	0.06
	Deflection Coefficient, .001839				Deflection Coefficient, .002069				Deflection Coeff., .002365		

Safe loads given include weight of beam. Maximum fiber strain, 16,000 lbs. per square inch. Deflection of beam, in inches, under tabular load equals the product of the Deflection Coefficient by the square of the span, in feet.

FOR SPANS ABOVE RED ZIGZAG LINE

SAFE LOADS, UNIFORMLY DISTRIBUTED, FOR PASSAIC STEEL I BEAMS,

IN TONS OF 2000 LBS.,

BEAMS BEING SECURED AGAINST YIELDING SIDEWAYS.

Span, in Feet.	6″ I		Add for ea. Lb. Inc. in Weight.	5″ I		Add for ea. Lb. Inc. in Weight.	4″ I			Add for ea. Lb. Inc. in Weight.
	15 Lbs. per Foot.	12 Lbs. per Foot.		13 Lbs. per Foot.	9¾ Lbs. per Foot.		10 Lbs. per Foot.	7½ Lbs. per Foot.	6 Lbs. per Foot.	
							3′-0″			
5	9.40	7.75	0.32	6.70	5.20	0.26	3.65	3.12	2.45	0.21
6	7.85	6.45	0.26	5.58	4.32	0.22	3.05	2.60	2.04	0.18
7	6.72	5.53	0.23	4.78	3.71	0.19	2.61	2.23	1.75	0.15
8	5.88	4.84	0.20	4.19	3.25	0.16	2.28	1.95	1.53	0.13
9	5.23	4.30	0.18	3.72	2.88	0.15	2.03	1.74	1.36	0.12
10	4.70	3.87	0.16	3.35	2.60	0.13	1.83	1.56	1.23	0.11
11	4.27	3.51	0.14	3.04	2.36	0.12	1.66	1.42	1.11	0.10
12	3.92	3.22	0.13	2.79	2.16	0.11	1.52	1.30	1.02	0.09
13	3.62	2.98	0.12	2.58	2.00	0.10	1.40	1.20	0.95	0.08
14	3.36	2.76	0.11	2.37	1.86	0.09	1.30	1.11	0.88	0.08
15	3.13	2.58	0.10	2.23	1.73	0.09	1.22	1.04	0.82	0.07
16	2.94	2.42	0.10	2.09	1.62	0.08	1.14	0.98	0.77	0.07
17	2.76	2.27	0.09	1.97	1.53	0.08	1.07	0.92	0.72	0.06
18	2.61	2.15	0.09	1.86	1.44	0.07	1.01	0.87	0.68	0.06
19	2.47	2.04	0.08	1.76	1.36	0.07	0.97	0.82	0.65	0.06
20	2.35	1.93	0.08	1.67	1.30	0.07	0.92	0.78	0.61	0.05
21	2.24	1.84	0.08	1.59	1.24	0.06	0.87	0.74	0.58	0.05
22	2.14	1.76	0.07	1.52	1.19	0.06	0.83	0.71	0.56	0.05
23	2.04	1.68	0.07	1.45	1.13	0.06	0.80	0.68	0.53	0.05
24	1.96	1.61	0.07	1.39	1.09	0.05	0.76	0.65	0.51	0.04
25	1.88	1.55	0.06	1.34	1.04	0.05	0.73	0.62	0.49	0.04
	Deflection Coeff., .002759			Deflection Coeff., .003310			Deflection Coefficient, .004138			

Safe loads given include weight of beam. Maximum fiber strain, 16,000 lbs. per square inch. Deflection of beam, in inches, under tabular load, equals the product of the Deflection Coefficient by the square of the span, in feet.

SPECIAL CONNECTIONS MUST BE DESIGNED.

SAFE LOADS, UNIFORMLY DISTRIBUTED, FOR
PASSAIC STEEL CHANNELS,
In tons of 2000 lbs.,

CHANNELS BEING SECURED AGAINST YIELDING SIDEWAYS.

Span, in Feet.	15″ 40 Lbs. per Ft.	15″ 33 Lbs. per Ft.	Add to Safe Load for each lb. per ft. increase in weight of Channel.	12″ 27 Lbs. per Ft.	12″ 20 Lbs. per Ft.	Add to Safe Load for each lb. per ft. increase in weight of Channel.
6	44.0	36.0	0.65	23.85	18.48	0.52
7	37.7	30.8	0.56	20.44	15.84	0.44
8	33.0	27.0	0.49	17.89	13.86	0.39
9	29.4	24.0	0.43	15.90	12.32	0.35
10	26.4	21.6	0.39	14.31	11.09	0.31
11	24.0	19.6	0.36	13.01	10.08	0.29
12	22.0	18.0	0.33	11.93	9.24	0.26
13	20.3	16.6	0.30	11.01	8.53	0.24
14	18.9	15.4	0.28	10.22	7.92	0.22
15	17.6	14.4	0.26	9.54	7.39	0.21
16	16.5	13.5	0.25	8.94	6.93	0.20
17	15.5	12.7	0.23	8.42	6.52	0.18
18	14.7	12.0	0.22	7.95	6.16	0.17
19	13.9	11.4	0.21	7.53	5.83	0.17
20	13.2	10.8	0.20	7.16	5.54	0.16
21	12.6	10.3	0.19	6.81	5.28	0.15
22	12.0	9.81	0.18	6.50	5.04	0.14
23	11.5	9.40	0.17	6.22	4.82	0.14
24	11.0	9.01	0.16	5.96	4.62	0.13
25	10.6	8.65	0.16	5.72	4.43	0.13
26	10.2	8.32	0.15	5.50	4.26	0.12
27	9.79	8.01	0.15	5.30	4.11	0.12
28	9.44	7.72	0.14	5.11	3.96	0.11
29	9.11	7.46	0.14	4.93	3.82	0.11
30	8.81	7.22	0.13	4.77	3.70	0.10
31	8.52	6.98	0.13	4.62	3.58	0.10
32	8.26	6.76	0.13	4.47	3.46	0.10
33	8.01	6.55	0.12	4.34	3.36	0.10
34	7.77	6.36	0.11	4.21	3.26	0.09
35	7.55	6.18	0.11	4.09	3.17	0.09
	Deflection Coefficient, .001103			Deflection Coefficient, .001379		

Safe loads given include weight of channel. Maximum fiber strain, 16,000 lbs. per square inch. Deflection of channel, in inches, under tabular load equals the product of the Deflection Coefficient by the square of the span, in feet.

SAFE LOADS, UNIFORMLY DISTRIBUTED, FOR

PASSAIC STEEL CHANNELS,

In tons of 2000 lbs.,

CHANNELS BEING SECURED AGAINST YIELDING SIDEWAYS.

Span, in Feet.	10″ 20 lbs. per Ft.	10″ 15 lbs. per Ft.	Add to Safe Load for each lb. per ft. inc. in weight of Channel.	9″ 16 lbs. per Ft.	9″ 13 lbs. per Ft.	Add to Safe Load for each lb. per ft. inc. in weight of Channel.	8″ 13 lbs. per Ft.	8″ 10 lbs. per Ft.	Add to Safe Load for each lb. per ft. inc. in weight of Channel.
5	18.2	14.3	0.52	13.5	10.8	0.48	9.48	7.52	0.42
6	15.2	11.9	0.44	11.3	8.98	0.40	7.90	6.27	0.34
7	13.0	10.2	0.38	9.67	7.69	0.34	6.77	5.37	0.30
8	11.4	8.91	0.33	8.46	6.73	0.29	5.92	4.70	0.26
9	10.1	7.92	0.29	7.52	5.98	0.26	5.27	4.18	0.23
10	9.12	7.13	0.26	6.76	5.38	0.24	4.74	3.76	0.21
11	8.29	6.48	0.24	6.15	4.90	0.21	4.31	3.42	0.19
12	7.60	5.94	0.22	5.64	4.49	0.20	3.95	3.14	0.17
13	7.02	5.48	0.20	5.20	4.14	0.18	3.65	2.89	0.16
14	6.52	5.09	0.19	4.83	3.85	0.17	3.39	2.69	0.15
15	6.08	4.75	0.17	4.51	3.59	0.16	3.16	2.51	0.14
16	5.70	4.46	0.16	4.23	3.37	0.15	2.96	2.35	0.13
17	5.37	4.19	0.15	3.98	3.17	0.14	2.79	2.21	0.12
18	5.07	3.95	0.15	3.76	2.99	0.13	2.64	2.09	0.12
19	4.80	3.75	0.14	3.56	2.83	0.12	2.50	1.98	0.11
20	4.56	3.56	0.13	3.38	2.69	0.12	2.37	1.88	0.10
21	4.34	3.40	0.12	3.22	2.56	0.11	2.26	1.79	0.10
22	4.14	3.24	0.12	3.08	2.45	0.11	2.16	1.71	0.09
23	3.96	3.10	0.11	2.94	2.34	0.10	2.06	1.63	0.09
24	3.80	2.96	0.11	2.82	2.24	0.10	1.97	1.56	0.09
25	3.65	2.85	0.10	2.71	2.15	0.09	1.90	1.50	0.08
26	3.51	2.74	0.10	2.60	2.07	0.09	1.83	1.44	0.08
27	3.38	2.64	0.10	2.50	2.00	0.09	1.76	1.39	0.08
28	3.26	2.54	0.09	2.41	1.93	0.08	1.69	1.34	0.07
29	3.15	2.45	0.09	2.33	1.86	0.08	1.63	1.30	0.07
30	3.04	2.38	0.09	2.26	1.80	0.08	1.58	1.26	0.07
	Deflection Coefficient, .001655			Deflection Coefficient, .001839			Deflection Coefficient, .002069		

Safe loads given include weight of channel. Maximum fiber strain, 16,000 lbs. per square inch. Deflection of channel, in inches, under tabular load, equals the product of the Deflection Coefficient by the square of the span, in feet.

SAFE LOADS, UNIFORMLY DISTRIBUTED, FOR

PASSAIC STEEL CHANNELS,

In tons of 2000 lbs.,

CHANNELS BEING SECURED AGAINST YIELDING SIDEWAYS.

Span, in Feet.	7″ 13 Lbs. per Ft.	7″ 9 Lbs. per Ft.	Add to Safe Load for each lb. per ft. increase in weight of Channel.	6″ 17 Lbs. per Ft.	6″ 12 Lbs. per Ft.	6″ 8 Lbs. per Ft.	Add to Safe Load for each lb. per ft. increase in weight of Channel.
5	8.33	5.79	0.37	9.05	6.64	4.54	0.31
6	6.94	4.83	0.32	7.53	5.53	3.78	0.26
7	5.95	4.13	0.26	6.46	4.74	3.24	0.22
8	5.21	3.62	0.23	5.64	4.15	2.84	0.20
9	4.63	3.22	0.20	5.02	3.69	2.52	0.17
10	4.17	2.90	0.18	4.52	3.32	2.27	0.16
11	3.79	2.62	0.17	4.10	3.02	2.06	0.14
12	3.47	2.41	0.15	3.76	2.77	1.89	0.13
13	3.20	2.22	0.14	3.48	2.55	1.74	0.12
14	2.98	2.06	0.13	3.23	2.35	1.62	0.11
15	2.78	1.93	0.12	3.01	2.21	1.51	0.10
16	2.60	1.81	0.11	2.82	2.07	1.42	0.10
17	2.45	1.70	0.11	2.66	1.95	1.33	0.09
18	2.32	1.61	0.10	2.51	1.84	1.26	0.09
19	2.19	1.52	0.10	2.38	1.75	1.19	0.08
20	2.08	1.45	0.09	2.26	1.66	1.13	0.08
21	1.97	1.38	0.09	2.15	1.58	1.08	0.07
22	1.89	1.32	0.08	2.05	1.51	1.03	0.07
23	1.82	1.26	0.08	1.96	1.44	.99	0.07
24	1.74	1.20	0.08	1.88	1.38	.95	0.07
25	1.67	1.16	0.07	1.81	1.33	.91	0.06
	Deflection Coefficient, .002364			Deflection Coefficient, .002760			

Safe loads given include weight of channel. Maximum fiber strain, 16,000 lbs. per square inch. Deflection of channel, in inches, under tabular load, equals the product of the Deflection Coefficient by the square of the span, in feet.

SAFE LOADS, UNIFORMLY DISTRIBUTED, FOR

PASSAIC STEEL CHANNELS,

In tons of 2000 lbs.,

CHANNELS BEING SECURED AGAINST YIELDING SIDEWAYS.

Span, in Feet.	5″ 9 Lbs. per Ft.	5″ 6 Lbs. per Ft.	Add to Safe Load for each lb. per ft. increase in weight of Channel.	4″ 8 Lbs. per Ft.	4″ 5 Lbs. per Ft.	Add to Safe Load for each lb. per ft. increase in weight of Channel.
5	4.12	2.78	0.26	2.91	1.92	0.21
6	3.43	2.32	0.22	2.42	1.60	0.18
7	2.94	1.99	0.19	2.08	1.37	0.15
8	2.58	1.74	0.17	1.82	1.20	0.13
9	2.29	1.54	0.15	1.62	1.07	0.12
10	2.06	1.39	0.13	1.46	.96	0.11
11	1.87	1.26	0.12	1.32	.87	0.10
12	1.71	1.16	0.11	1.21	.80	0.09
13	1.58	1.07	0.10	1.12	.74	0.08
14	1.47	.99	0.09	1.04	.69	0.08
15	1.37	.93	0.09	.97	.64	0.07
16	1.29	.87	0.08	.91	.60	0.07
17	1.21	.82	0.08	.86	.56	0.06
18	1.14	.77	0.07	.81	.53	0.06
19	1.08	.73	0.07	.77	.50	0.06
20	1.03	.70	0.07	.73	.48	0.05
21	.98	.66	0.06	.69	.45	0.05
22	.94	.63	0.06	.66	.44	0.05
23	.90	.60	0.06	.63	.42	0.05
24	.86	.58	0.06	.61	.40	0.04
25	.82	.56	0.05	.58	.38	0.04
	Deflection Coefficient, .00331			Deflection Coefficient, .00414		

Safe loads given include weight of channel. Maximum fiber strain, 16,000 lbs. per square inch. Deflection of channel, in inches, under tabular load equals the product of the Deflection Coefficient by the square of the span, in feet.

SAFE LOADS, UNIFORMLY DISTRIBUTED, FOR PASSAIC STEEL T SHAPES, EQUAL LEGS,

In tons of 2000 lbs., Tees having stem vertical and being secured against yielding sideways.

Size of T, Inches, Flange by Stem.	Thickness, Inches.	Weight per Foot, Pounds.	Distance between Supports, in Feet.															Deflection Coeff.
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	
4 × 4	1/2	13.6	10.76	5.38	3.59	2.69	2.15	1.79	1.54	1.35	1.20	1.08	0.98	0.90	0.83	0.77	0.72	.0029
4 × 4	3/8	10.4	8.75	4.38	2.92	2.19	1.75	1.46	1.25	1.09	0.97	0.88	0.79	0.73	0.68	0.63	0.58	.0029
3½ × 3½	1/2	11.7	8.19	4.09	2.73	2.05	1.64	1.36	1.17	1.02	0.91	0.82	0.74	0.68	0.63	0.58	0.55	.0034
3½ × 3½	7/16	10.4	7.20	3.60	2.40	1.80	1.44	1.20	1.03	0.90	0.80	0.72	0.65	0.60	0.55	0.51	0.48	.0034
3 × 3	1/2	10.0	5.87	2.93	1.96	1.47	1.17	0.98	0.84	0.73	0.65	0.59	0.53	0.49	0.45	0.42	0.39	.0040
3 × 3	7/16	9.1	5.38	2.69	1.79	1.34	1.08	0.90	0.77	0.67	0.60	0.54	0.49	0.45	0.41	0.38	0.36	.0040
3 × 3	3/8	7.8	4.59	2.29	1.53	1.15	0.92	0.76	0.66	0.57	0.51	0.46	0.42	0.38	0.35	0.33	0.31	.0039
3 × 3	5/16	6.6	3.97	1.99	1.32	0.99	0.79	0.66	0.57	0.50	0.44	0.40	0.36	0.33	0.31	0.28	0.27	.0039
2½ × 2½	3/8	6.4	3.14	1.57	1.05	0.78	0.63	0.52	0.45	0.39	0.35	0.31	0.29	0.26	0.24	0.22		.0048
2½ × 2½	5/16	5.5	2.66	1.33	0.89	0.67	0.53	0.44	0.38	0.33	0.30	0.27	0.24	0.22	0.20	0.19		.0047
2 × 2	5/16	4.3	1.74	0.87	0.58	0.44	0.35	0.29	0.25	0.22	0.19	0.17	0.16	0.15				.0060
2 × 2	1/4	3.7	1.34	0.67	0.45	0.33	0.27	0.22	0.19	0.17	0.15	0.13	0.12	0.11				.0059
1¾ × 1¾	1/4	3.1	1.03	0.52	0.34	0.26	0.21	0.17	0.15	0.13	0.12	0.10						.0068
1¾ × 1¾	3/16	2.25	0.74	0.37	0.25	0.19	0.15	0.12	0.11	0.09	0.08	0.07						.0067
1½ × 1½	1/4	2.55	0.77	0.39	0.26	0.19	0.15	0.13	0.11	0.10	0.09							.0077
1½ × 1½	3/16	1.85	0.57	0.29	0.19	0.14	0.11	0.10	0.08	0.07	0.06							.0078
1¼ × 1¼	3/16	1.55	0.39	0.19	0.13	0.10	0.08	0.06	0.06	0.05								.0095
1 × 1	1/8	0.90	0.18	0.09	0.06	0.04	0.04	0.03										.0116

Safe loads include weight of Tees. Maximum fiber strain of 16,000 lbs. per square inch. The loads given to the right of the zigzag line produce deflections exceeding 1/360 of the span.

Deflection of Tees, in inches, under tabular loads is equal to the product of the Deflection Coefficient by the square of the span, in feet.

SAFE LOADS, UNIFORMLY DISTRIBUTED, FOR PASSAIC STEEL T SHAPES, UNEQUAL LEGS,

In tons of 2000 lbs., Tees having stem vertical and being secured against yielding sideways.

Size of T, Inches, Flange by Stem.	Thickness, Inches.	Weight per Foot, Pounds.	Distance between Supports, in Feet.															Deflection Coeff.
			1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	
6 × 4	5/8	20.6	13.78	6.88	4.59	3.44	2.76	2.30	1.97	1.72	1.53	1.38	1.25	1.15	1.06	0.98	0.92	.0028
6 × 4	1/2	17.0	11.25	5.63	3.75	2.81	2.25	1.88	1.61	1.41	1.25	1.13	1.02	0.94	0.87	0.80	0.75	.0028
3 × 4	1/2	11.9	10.27	5.14	3.42	2.57	2.05	1.71	1.47	1.28	1.14	1.03	0.93	0.86	0.79	0.73	0.68	.0031
4 × 3½	1/2	12.5	6.72	3.36	2.24	1.68	1.35	1.12	0.96	0.84	0.75	0.67	0.61	0.56	0.52	0.48	0.45	.0033
4 × 3½	3/8	9.8	6.62	3.31	2.21	1.66	1.32	1.10	0.95	0.83	0.74	0.66	0.60	0.55	0.51	0.47	0.44	.0032
5 × 3	1/2	13.5	6.13	3.06	2.04	1.53	1.23	1.02	0.88	0.77	0.68	0.61	0.56	0.51	0.47	0.44	0.41	.0037
5 × 2½	3/8	10.4	4.32	2.16	1.44	1.08	0.86	0.72	0.62	0.54	0.48	0.43	0.39	0.36	0.33	0.31	0.29	.0044
4 × 2	3/8	7.9	2.13	1.07	0.71	0.53	0.43	0.36	0.31	0.27	0.24	0.21	0.19	0.18	0.16	0.15	0.14	.0054
3 × 2	3/8	6.4	1.97	0.99	0.66	0.49	0.40	0.33	0.28	0.25	0.22	0.20	0.18	0.16				.0057
3 × 1½	3/8	5.7	1.06	0.53	0.36	0.27	0.21	0.18	0.15	0.13	0.12							.0075
2¼ × 1¼	1/4	3.1	0.535	0.27	0.18	0.13	0.11	0.09	0.08	0.07								.0089

Safe loads include weight of Tees. Maximum fiber strain of 16,000 lbs. per square inch. The loads given to the right of the zigzag line produce deflections exceeding 1/360 of the span.

Deflection of Tees, in inches, under tabular loads, is equal to the product of the Deflection Coefficient by the square of the span, in feet.

Strength per square inch is the

SAFE LOADS, UNIFORMLY DISTRIBUTED, FOR PASSAIC STEEL ANGLES,

EQUAL LEGS, IN TONS OF 2,000 LBS.,

Angles being secured against yielding sideways.

Size of Angle, Inches.	Thickness, Ins.	Coefficient of strength, in tons.	Span in feet. 2	3	4	5	6	8	10	12	Deflection Coefficient.
6 × 6	7/8	43.6	21.8	14.5	10.9	8.71	7.26	5.44	4.36	3.63	.0020
6 × 6	3/8	18.8	9.38	6.25	4.69	3.75	3.13	2.34	1.88	1.56	.0019
5 × 5	3/4	25.5	12.8	8.50	6.38	5.10	4.25	3.19	2.55	2.13	.0024
5 × 5	3/8	12.9	6.45	4.30	3.23	2.58	2.15	1.61	1.29	1.08	.0023
4 × 4	13/16	17.7	8.85	5.90	4.43	3.54	2.95	2.21	1.77	1.48	.0031
4 × 4	5/16	6.90	3.45	2.30	1.73	1.38	1.15	.86	.69	.58	.0029
3½ × 3½	5/8	9.65	4.83	3.22	2.41	1.93	1.61	1.21	.97	.80	.0035
3½ × 3½	5/16	5.20	2.60	1.73	1.30	1.04	.87	.65	.52	.43	.0033
3 × 3	5/8	7.90	3.95	2.63	1.99	1.58	1.32	.99	.79	.66	.0042
3 × 3	1/4	3.10	1.55	1.03	.77	.62	.52	.39	.31	.26	.0038
2½ × 2½	1/2	4.08	2.04	1.36	1.02	.82	.68	.51	.41	.34	.0049
2½ × 2½	1/4	2.14	1.07	.71	.54	.43	.36	.27	.21	.18	.0047
2¼ × 2¼	1/2	3.47	1.74	1.16	.87	.69	.58	.43	.35	.29	.0056
2¼ × 2¼	3/16	1.30	.65	.43	.32	.26	.22	.16	.13	.11	.0051
2 × 2	1/2	2.72	1.36	.91	.68	.54	.45	.34	.27		.0065
2 × 2	3/16	1.02	.51	.34	.25	.20	.17	.13	.10		.0058
1¾ × 1¾	7/16	1.73	.86	.57	.43	.35	.29	.22	.17		.0073
1¾ × 1¾	3/16	.75	.37	.25	.19	.15	.12	.09	.07		.0067
1½ × 1½	3/8	1.00	.50	.33	.25	.20	.17	.13	.10		.0084
1½ × 1½	3/16	.56	.28	.19	.14	.11	.09	.07	.06		.0078
1¼ × 1¼	5/16	.69	.34	.23	.17	.14	.11	.09	.07		.0105
1¼ × 1¼	1/8	.26	.13	.09	.07	.05	.04	.03	.03		.0092
1 × 1	1/4	.34	.17	.11	.08	.07	.06	.04			.0129
1 × 1	1/8	.17	.08	.06	.04	.03	.03	.02			.0118
7/8 × 7/8	3/16	.18	.09	.06	.04	.04	.03				.0141
7/8 × 7/8	1/8	.12	.06	.04	.03	.024	.02				.0132
3/4 × 3/4	3/16	.13	.06	.04	.03	.03					.0169
3/4 × 3/4	1/8	.09	.05	.03	.022	.018					.0159

Safe loads given include weight of angle. Maximum fiber strain, 16,000 lbs. per sq. in.

Safe loads for intermediate spans can be obtained by dividing the coefficient of strength by the span, in feet.

Loads given to the right of the zigzag line produce deflections exceeding $\frac{1}{360}$ of the span. Deflections, in inches, under tabular loads, can be obtained by multiplying the Deflection Coefficient by the square of the span, in feet.

same for light and heavy sections

SAFE LOADS, UNIFORMLY DISTRIBUTED, FOR PASSAIC STEEL ANGLES,

UNEQUAL LEGS, IN TONS OF 2,000 LBS.

Long Leg Vertical. Angles being secured against yielding sideways.

Size of Angle, Inches.	Thickness, Ins.	Coefficient of strength, in tons.	Span in feet.								Deflection Coefficient.
			2	3	4	5	6	8	10	12	
6 ×4	7/8	42.1	21.0	14.0	10.5	8.41	7.01	5.26	4.21	3.50	.0022
6 ×4	3/8	17.7	8.85	5.90	4.43	3.54	2.95	2.21	1.77	1.48	.0020
5 ×3½	3/4	24.2	12.1	8.05	6.04	4.83	4.03	3.02	2.42	2.01	.0026
5 ×3½	3/8	12.2	6.10	4.07	3.05	2.44	2.03	1.53	1.22	1.02	.0024
5 ×3	3/4	24.3	12.1	8.08	6.06	4.85	4.04	3.03	2.43	2.02	.0027
5 ×3	5/16	10.1	5.03	3.35	2.51	2.01	1.68	1.26	1.01	.84	.0025
4½×3	3/4	19.1	9.55	6.37	4.78	3.82	3.18	2.39	1.91	1.59	.0029
4½×3	5/16	8.2	4.10	2.73	2.05	1.64	1.37	1.03	.82	.68	.0027
4 ×3½	3/4	15.7	7.85	5.23	3.93	3.14	2.62	1.96	1.57	1.31	.0031
4 ×3½	5/16	6.6	3.30	2.20	1.65	1.32	1.10	.83	.66	.55	.0029
4 ×3	5/8	12.3	6.15	4.10	3.08	2.46	2.05	1.54	1.23	1.03	.0032
4 ×3	5/16	6.55	3.28	2.18	1.64	1.31	1.09	.82	.66	.55	.0030
3½×3	5/8	9.38	4.69	3.12	2.34	1.87	1.56	1.17	.94	.78	.0036
3½×3	5/16	5.11	2.56	1.70	1.28	1.02	.85	.64	.51	.43	.0034
3½×2½	9/16	8.64	4.32	2.88	2.16	1.73	1.44	1.08	.86	.72	.0037
3½×2½	1/4	4.00	2.00	1.33	1.00	.80	.67	.50	.40	.33	.0035
3 ×2½	9/16	6.45	3.23	2.15	1.61	1.29	1.08	.81	.65	.54	.0042
3 ×2½	1/4	2.99	1.49	.99	.75	.60	.50	.37	.30	.25	.0040
3 ×2	1/2	5.34	2.67	1.78	1.44	1.07	.89	.67	.53	.44	.0043
3 ×2	1/4	2.88	1.44	.96	.72	.58	.48	.36	.29	.24	.0041
2¼×1½	5/16	1.97	.99	.66	.49	.39	.33	.25	.20	.16	.0057
2¼×1½	3/16	1.23	.61	.41	.31	.25	.20	.15	.12	.10	.0055
2 ×1¾	5/16	1.60	.80	.53	.40	.32	.27	.20	.16	.13	.0061
2 ×1¾	3/16	1.01	.50	.34	.25	.20	.17	.13	.10	.08	.0059
1⅜×1⅛	5/16	.77	.39	.26	.19	.15	.13	.10	.08	.06	.0096
1⅜×1⅛	1/8	.31	.15	.10	.08	.06	.05	.04	.03	.03	.0087

Safe loads given include weight of angle. Maximum fiber strain, 16,000 lbs. per sq. in.

Safe loads for intermediate spans can be obtained by dividing the coefficient of strength by the span, in feet.

Loads given to the right of the zigzag line produce deflections exceeding $\frac{1}{360}$ of the span. Deflections, in inches, under tabular loads, can be obtained by multiplying the Deflection Coefficient by the square of the span, in feet.

SAFE LOADS, UNIFORMLY DISTRIBUTED, FOR PASSAIC STEEL ANGLES,

UNEQUAL LEGS, IN TONS OF 2,000 LBS.

Short Leg Vertical. Angles being secured against yielding sideways.

Size of Angle, Inches.	Thickness, Ins.	Coefficient of strength in tons.	Span in feet.								Deflection Coefficient.
			2	3	4	5	6	8	10	12	
6 × 4	$\frac{7}{8}$	20.5	10.3	6.83	5.13	4.10	3.41	2.56	2.05	1.71	.0029
6 × 4	$\frac{3}{8}$	8.50	4.25	2.83	2.13	1.70	1.42	1.06	.85	.71	.0027
5 × 3½	$\frac{3}{4}$	12.75	6.38	4.25	3.19	2.55	2.13	1.59	1.28	1.06	.0034
5 × 3½	$\frac{3}{8}$	6.45	3.23	2.15	1.61	1.29	1.08	.81	.64	.54	.0031
5 × 3	$\frac{3}{4}$	9.85	4.93	3.28	2.46	1.97	1.64	1.23	.99	.82	.0039
5 × 3	$\frac{5}{16}$	3.99	2.00	1.33	1.00	.80	.67	.50	.40	.33	.0036
4½ × 3	$\frac{3}{4}$	9.33	4.66	3.11	2.33	1.87	1.55	1.17	.93	.78	.0040
4½ × 3	$\frac{5}{16}$	3.99	2.00	1.33	1.00	.80	.67	.50	.40	.33	.0036
4 × 3½	$\frac{3}{4}$	12.4	6.20	4.13	3.10	2.48	2.07	1.55	1.24	1.03	.0035
4 × 3½	$\frac{5}{16}$	5.3	2.65	1.77	1.33	1.06	.88	.66	.53	.44	.0032
4 × 3	$\frac{5}{8}$	6.8	3.40	2.27	1.70	1.36	1.13	.85	.68	.57	.0039
4 × 3	$\frac{5}{16}$	3.95	1.97	1.32	.99	.79	.66	.49	.40	.33	.0037
3½ × 3	$\frac{5}{8}$	7.04	3.52	2.35	1.76	1.41	1.17	.88	.70	.59	.0040
3½ × 3	$\frac{5}{16}$	3.84	1.92	1.28	.96	.77	.64	.48	.38	.32	.0038
3½ × 2½	$\frac{9}{16}$	4.75	2.37	1.58	1.19	.95	.79	.59	.48	.40	.0047
3½ × 2½	$\frac{1}{4}$	2.19	1.09	.73	.55	.44	.36	.27	.22	.18	.0044
3 × 2½	$\frac{9}{16}$	4.59	2.29	1.53	1.15	.93	.76	.57	.46	.38	.0048
3 × 2½	$\frac{1}{4}$	2.13	1.07	.71	.53	.43	.36	.27	.21	.18	.0045
3 × 2	$\frac{1}{2}$	2.51	1.25	.83	.63	.50	.42	.31	.25	.21	.0058
3 × 2	$\frac{1}{4}$	1.39	.69	.46	.35	.28	.23	.17	.14	.12	.0055
2¼ × 1½	$\frac{5}{16}$	.96	.48	.32	.24	.19	.16	.12	.10	....	.0077
2¼ × 1½	$\frac{3}{16}$	.59	.29	.19	.15	.12	.10	.07	.06	....	.0073
2 × 1¾	$\frac{5}{16}$	1.23	.61	.41	.31	.25	.20	.15	.12	....	.0067
2 × 1¾	$\frac{3}{16}$	.80	.40	.27	.20	.16	.13	.10	.08	....	.0065
1⅜ × 1⅛	$\frac{5}{16}$	.53	.27	.18	.13	.11	.09	.07	....	....	.0111
1⅜ × 1⅛	$\frac{1}{8}$	.21	.11	.07	.05	.04	.04	.03	....	....	.0099

Safe loads given include weight of angle. Maximum fiber strain, 16,000 lbs. per sq. in.

Safe loads for intermediate spans can be obtained by dividing the coefficient of strength by the span, in feet.

Loads given to the right of the zigzag line produce deflections exceeding $\frac{1}{360}$ of the span. Deflections, in inches, under tabular loads, can be obtained by multiplying the Deflection Coefficient by the square of the span, in feet.

SAFE LOADS, UNIFORMLY DISTRIBUTED, FOR PASSAIC STEEL Z BARS,

IN TONS OF 2000 LBS.

Web vertical. Z bars being secured against yielding sideways.

Size of Z bar, Ins.	Thickness, Ins.	Coefficient of strength, in tons.	Span in feet.								Deflection Coefficient.
			2	3	4	5	6	8	10	12	
6	3/8	45.0	22.5	15.0	11.2	9.00	7.50	5.62	4.50	3.75	.0028
6 1/16	7/16	52.4	26.2	17.5	13.1	10.5	8.73	6.55	5.24	4.37	.0027
6 1/8	1/2	59.9	29.9	19.9	14.9	11.9	9.98	7.49	5.99	4.99	.0027
6	9/16	61.6	30.8	20.5	15.4	12.3	10.3	7.70	6.16	5.13	.0028
6 1/16	5/8	68.4	34.2	22.8	17.1	13.7	11.4	8.55	6.84	5.70	.0027
6 1/8	11/16	75.2	37.6	25.1	18.8	15.0	12.5	9.40	7.52	6.27	.0027
6	3/4	75.0	37.5	25.0	18.8	15.0	12.6	9.38	7.50	6.25	.0028
6 1/16	13/16	81.2	40.6	27.1	20.3	16.2	13.5	10.2	8.12	6.77	.0027
6 1/8	7/8	87.5	43.8	29.2	21.9	17.5	14.6	10.9	8.75	7.29	.0027
5	5/16	28.5	14.3	9.5	7.12	5.7	4.75	3.56	2.85	2.38	.0033
5 1/16	3/8	34.1	17.1	11.4	8.52	6.82	5.67	4.26	3.41	2.84	.0033
5 1/8	7/16	39.7	19.9	13.2	9.92	7.94	6.62	4.96	3.97	3.31	.0032
5	1/2	41.0	20.5	13.7	10.2	8.20	6.83	5.13	4.10	3.42	.0033
5 1/16	9/16	46.0	23.0	15.3	11.5	9.20	7.67	5.75	4.60	3.83	.0033
5 1/8	5/8	51.1	25.6	17.0	12.8	10.2	8.52	6.39	5.11	4.26	.0032
5	11/16	50.5	25.3	16.8	12.6	10.1	8.42	6.31	5.05	4.21	.0033
5 1/16	3/4	55.2	27.6	18.4	13.8	11.0	9.20	6.90	5.52	4.60	.0033
5 1/8	13/16	61.0	30.5	20.3	15.2	12.2	10.2	7.63	6.10	5.10	.0032
4	1/4	16.8	8.4	5.6	4.2	3.36	2.80	2.10	1.68	1.40	.0041
4 1/16	5/16	20.9	10.5	6.97	5.22	4.18	3.48	2.61	2.09	1.74	.0041
4 1/8	3/8	24.9	12.5	8.30	6.22	4.98	4.15	3.11	2.49	2.08	.0040
4	7/16	25.8	12.9	8.60	6.45	5.16	4.30	3.23	2.58	2.15	.0041
4 1/16	1/2	29.4	14.7	9.80	7.35	5.88	4.90	3.68	2.94	2.45	.0041
4 1/8	9/16	33.0	16.5	11.0	8.25	6.60	5.50	4.13	3.30	2.75	.0040
4	5/8	32.3	16.2	10.8	8.04	6.46	5.38	4.04	3.23	2.69	.0041
4 1/16	11/16	35.5	17.8	11.8	8.88	7.10	5.92	4.88	3.55	2.96	.0041
4 1/8	3/4	38.7	19.4	12.9	9.68	7.76	6.45	4.84	3.87	3.23	.0040
3	1/4	10.3	5.15	3.43	2.58	2.06	1.72	1.29	1.03	.86	.0055
3 1/16	5/16	12.7	6.35	4.23	3.18	2.54	2.12	1.59	1.27	1.06	.0054
3	3/8	13.7	6.85	4.57	3.42	2.74	2.28	1.71	1.37	1.01	.0055
3 1/16	7/16	15.9	7.95	5.30	3.98	3.18	2.65	1.99	1.59	1.33	.0054
3	1/2	16.3	8.15	5.43	4.08	3.26	2.72	2.04	1.63	1.36	.0055
3 1/16	9/16	18.3	9.15	6.10	4.58	3.66	3.05	2.29	1.83	1.53	.0054

Safe loads given include weight of Z. Maximum fiber strain, 16,000 lbs. per sq. in. Safe loads for intermediate spans can be obtained by dividing the coefficient of strength by the span, in feet.

Loads given to the right of the zigzag line produce deflections exceeding 1/360 of the span. Deflection, in inches, under tabular loads, can be obtained by multiplying the Deflection Coefficient by the square of the span, in feet.

BEAM GIRDERS.

It frequently happens in building construction that a single **I** beam is insufficient to carry the imposed load. Where heavy loads, such as brick walls, vaults, etc., are to be supported, a single **I** beam is inadequate and two or more beams are used side by side, bolted together with cast iron or steel separators, as shown on page 34, Figs. 7, 8, and 9. These separators serve to hold the compression flanges of the beams in position to prevent deflection sideways, and also, in a measure, to cause the beams to act together and distribute the load uniformly on the component beams of the girder. Separators should be provided at the supports and at points where heavy loads are imposed and at intervals of not exceeding 6 feet. A table is given on page 40 by which the approximate weights of separators can be obtained for any size and width of beam girders.

In designing floors for buildings, it is desirable to have a minimum number of interior supporting columns consistent with economy, and a beam girder, consisting of a pair of **I** beams, is frequently advantageous for supporting the steel floor joists as in Figs. 1 and 3 on page 34.

Girders, composed of two or more **I** beams, are commonly used to span openings in brick walls. If the wall to be supported is thoroughly seasoned and without openings, the weight carried by the girder can safely be assumed to that of a rectangle of wall having a length equal to the opening and a height of $\frac{1}{3}$ of the opening; for, if the girder should fail, the line of rupture of the brickwork would be found within this rectangle. If the wall is newly built, or if it has openings for windows or other purposes, the girder must be designed to carry the entire wall above the girder and between the supports.

In obtaining the weight of brick walls, it is customary to assume a cubic foot of brickwork as weighing 120 lbs. The weights, per superficial square foot, for different walls, are,

8″ wall,..........	80 lbs.	20″ wall,.........	200 lbs.
12″ "	120 "	24″ "	240 "
16″ "	160 "	28″ "	280 "

When walls are faced with stone, the weight of the stonework, taken at 160 lbs. per cubic foot, must be added. If the walls are plastered, add 5 lbs. per square foot for the weight of the plastering.

NOTE

STEEL BEAM BOX GIRDERS.

A box girder consisting of a pair of steel **I** beams, with top and bottom flange plates, furnishes an economical girder for short spans. The flange plates are riveted to the beams with $\frac{3}{4}''$ diameter rivets spaced from $6''$ to $9''$ centers. In short girders, care must be taken to have a sufficient number of rivets in each plate, between the end of the girder and the center of span, to develop the full tensile or compressive strength of the plate.

The safe loads in the following tables have been computed from the moments of inertia of the sections, deducting the rivet holes in each flange. A maximum fiber strain of 15,000 lbs. per square inch is used, instead of the 16,000 lbs. fiber strain allowed on rolled beams, to allow for the injury to the strength of the material due to punching the holes for the rivets.

Suppose it is required to select a beam box girder to safely support a load of 45 tons, including the weight of the girder itself, over a span of 25 feet. By referring to the tables it will be found that a girder, composed of two $15'' \times 42$ lb. **I** beams with flange plates $14'' \times \frac{5}{8}''$, has a safe load of only 40.0 tons on this span; but each $\frac{1}{16}''$ increase in thickness of flange plates adds 2.16 tons to the safe load, so that the flange plates would require to be $\frac{3}{16}''$ thicker, or $\frac{13}{16}''$ for each plate.

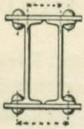
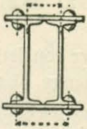
The deflection of the girder under this load, in inches, would be obtained by multiplying the Deflection Coefficient by the square of the span in feet; or,

$$.00102 \times \overline{25}^2 = 0.64''.$$

STEEL BEAM BOX GIRDERS.

Safe Loads, in Tons of 2000 Lbs., Uniformly Distributed.

2–12″ Steel I Beams and 2 Steel Plates 14″ × ½″

Span, center to center of Bearings, in Feet.	6″  2 plates 14″ × ½″ — 12″ I Beams 40.0 lbs. per foot. — 9¼″		6″  2 plates, 14″ × ½″ — 12″ I Beams 31.5 lbs. per foot. — 9″	
	Safe Loads, includ'g Wgt. of Girder, in Tons.	Inc. in Safe Load for $\frac{1}{16}$ In. Increase in Thickness of Flange Plates.	Safe Loads, includ'g Wgt. of Girder, in Tons.	Inc. in Safe Load for $\frac{1}{16}$ in. Increase in Thickness of Flange Plates.
12	61.8	3.61	55.3	3.65
13	57.0	3.33	51.0	3.37
14	53.0	3.09	47.4	3.13
15	49.5	2.89	44.2	2.92
16	46.4	2.71	41.5	2.74
17	43.6	2.55	39.0	2.58
18	41.2	2.41	36.8	2.43
19	39.0	2.28	34.9	2.31
20	37.1	2.17	33.2	2.19
21	35.3	2.06	31.6	2.09
22	33.7	1.97	30.2	1.99
23	32.3	1.88	28.8	1.90
24	30.9	1.80	27.6	1.83
25	29.7	1.73	26.5	1.75
26	28.5	1.67	25.5	1.68
27	27.5	1.60	24.6	1.62
28	26.5	1.55	23.7	1.56
29	25.6	1.49	22.9	1.51
30	24.7	1.44	22.1	1.46
31	23.9	1.40	21.4	1.41
32	23.2	1.35	20.7	1.37
33	22.5	1.31	20.1	1.33
34	21.8	1.27	19.5	1.29
35	21.2	1.24	19.0	1.25
36	20.6	1.20	18.4	1.22
37	20.1	1.17	17.9	1.18
38	19.5	1.14	17.5	1.15
39	19.0	1.11	17.0	1.12
	Wgt. per lineal ft. of girder, includ'g rivet heads=131 lbs.		Wgt. per lineal ft. of girder, includ'g rivet heads=115 lbs.	

Deflection Coefficient = .00127

Increase in Weight of Girders for each $\frac{1}{16}$ Inch increase in Thickness of Flange Plates = 6 Lbs. per Lineal Foot.

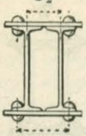
Maximum fiber strain of 15,000 lbs, per square inch; holes for ¾″ rivets in both flanges deducted.

Deflection, in inches, under tabular loads, equals the product of the Deflection Coefficient by the square of the span, in feet.

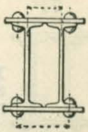
STEEL BEAM BOX GIRDERS.

SAFE LOADS, IN TONS OF 2000 LBS., UNIFORMLY DISTRIBUTED.

2–15″ Steel I Beams and 2 Steel Plates 14″ × ⅝″.



2 plates 14″ × ⅝″ — 15″ I Beams 60.0 lbs. per foot.



2 plates 14″ × ⅝″ — 15″ I Beams 42.0 lbs. per foot.

Span, center to center of Bearings, in Feet.	Safe Loads, includ'g Wgt. of Girder, in Tons. (15″ I Beams 60.0 lbs.)	Inc. in Safe Load for $\frac{1}{16}$ in. Increase in Thickness of Flange Plates.	Safe Loads, includ'g Wgt. of Girder, in Tons. (15″ I Beams 42.0 lbs.)	Inc. in Safe Load for $\frac{1}{16}$ in. Increase in Thickness of Flange Plates.
12	105.3	4.32	83.4	4.49
13	97.2	3.99	77.0	4.15
14	90.3	3.71	71.5	3.85
15	84.3	3.46	66.7	3.59
16	79.0	3.24	62.6	3.37
17	74.4	3.05	58.9	3.17
18	70.2	2.88	55.6	2.99
19	66.5	2.73	52.7	2.83
20	63.2	2.60	50.1	2.69
21	60.2	2.47	47.7	2.57
22	57.5	2.36	45.5	2.45
23	55.0	2.26	43.5	2.34
24	52.7	2.16	41.7	2.25
25	50.6	2.08	40.0	2.16
26	48.6	2.00	38.5	2.07
27	46.8	1.92	37.1	2.00
28	45.1	1.85	35.8	1.92
29	43.6	1.79	34.5	1.86
30	42.1	1.73	33.4	1.80
31	40.8	1.67	32.3	1.74
32	39.5	1.62	31.3	1.68
33	38.3	1.57	30.3	1.63
34	37.2	1.53	29.4	1.59
35	36.1	1.48	28.6	1.54
36	35.1	1.44	27.8	1.50
37	34.2	1.40	27.1	1.46
38	33.3	1.37	26.3	1.42
39	32.4	1.33	25.7	1.38
40	31.6	1.30	25.0	1.35
	Wgt. per lineal ft. of girder, includ'g rivet heads=183 lbs.		Wgt. per lineal ft. of girder, includ'g rivet heads=147 lbs.	

Deflection Coefficient = .00102

Increase in Weight of Girders for each $\frac{1}{16}$ Inch increase in Thickness of Flange Plates = 6 Lbs. per Lineal Foot.

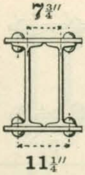
Maximum fiber strains of 15,000 lbs. per square inch; holes for ¾″ rivets in both flanges deducted.

Deflection, in inches, under tabular loads, equals the product of the Deflection Coefficient by the square of the span, in feet.

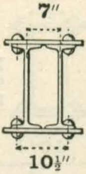
STEEL BEAM BOX GIRDERS.

SAFE LOADS, IN TONS OF 2000 LBS., UNIFORMLY DISTRIBUTED.

2–20″ Steel **I** Beams and 2 Steel Plates 16″ × ¾″



2 plates 16″ × ¾″ — 20″ **I** Beams 80.0 lbs. per foot.



2 plates, 16″ × ¾″ — 20″ **I** Beams 65.0 lbs. per foot.

Span, center to center of Bearings, in Feet.	20″ I Beams 80.0 lbs. per foot: Safe Loads, includ'g Wgt. of Girder, in Tons.	20″ I Beams 80.0 lbs. per foot: Inc. in Safe Load for $\frac{1}{16}$ in. Increase in Thickness of Flange Plates.	20″ I Beams 65.0 lbs. per foot: Safe Loads, includ'g Wgt. of Girder, in Tons.	20″ I Beams 65.0 lbs. per foot: Inc. in Safe Load for $\frac{1}{16}$ in. Increase in Thickness of Flange Plates.
14	154.3	6.01	144.1	6.06
15	144.1	5.61	134.5	5.66
16	135.1	5.26	126.1	5.30
17	127.1	4.95	118.7	4.99
18	120.1	4.68	112.1	4.72
19	113.7	4.43	106.2	4.47
20	108.1	4.21	100.8	4.24
21	102.9	4.01	96.0	4.04
22	98.2	3.83	91.7	3.86
23	93.9	3.66	87.7	3.69
24	90.0	3.51	84.0	3.54
25	86.4	3.37	80.7	3.40
26	83.1	3.24	77.6	3.26
27	80.0	3.12	74.7	3.14
28	77.2	3.01	72.0	3.03
29	74.5	2.90	69.6	2.93
30	72.0	2.81	67.2	2.83
31	69.7	2.72	65.0	2.74
32	67.5	2.63	63.0	2.65
33	65.5	2.55	61.1	2.57
34	63.6	2.48	59.3	2.50
35	61.7	2.41	57.6	2.43
36	60.0	2.34	56.0	2.36
37	58.4	2.27	54.5	2.29
38	56.9	2.22	53.1	2.23
39	55.4	2.16	51.7	2.18
40	54.0	2.10	50.4	2.12
	Wgt. per lineal ft. of girder, includ'g rivet heads=245 lbs.		Wgt. per lineal ft. of girder, includ'g rivet heads=215 lbs.	

Deflection Coefficient = .00077

Increase in Weight of Girders for each $\frac{1}{16}$ Inch increase in Thickness of Flange Plates = 7 Lbs. per Lineal Foot.

Maximum fiber strains of 15,000 lbs. per square inch; holes for ¾″ rivets in both flanges deducted.

Deflection, in inches, under tabular loads, equals the product of the Deflection Coefficient by the square of the span, in feet.

NOTES ON THE STRENGTH AND DEFLECTION OF BEAMS.

Let A = area of section, in square inches.
L = length of span, in feet.
l = length of span, in inches.
W = load, uniformly distributed, in lbs.
P = load, concentrated at any point, in lbs.
h = height of cross-section, in inches.
M = bending moment, in foot-lbs.
m = bending moment, in inch-lbs.
n = greatest distance of center of gravity of section from top or from bottom, in inches.
S = strain per square inch in extreme fibers of beam, either top or bottom, in lbs., according as n refers to distance from top or from bottom of section.
D = maximum deflection, in inches.
I = moment of inertia of section, neutral axis through center of gravity.
I′ = moment of inertia of section, neutral axis parallel to above, but not through center of gravity.
z = distance between these neutral axes.
Q = section modulus.
R = least moment of resistance of section, in inch-lbs.
r = radius of gyration, in inches.
C = coefficient of transverse strength, in lbs.
E = modulus of elasticity (27,000,000 for wrought iron and 29,000,000 for steel).

For a beam of any cross-section the following formulæ express the relation existing between the properties of the section.

$$I' = I + Az^2; \qquad r = \sqrt{\frac{I}{A}}; \qquad Q = \frac{I}{n};$$

$$R = \frac{I}{n} S = QS; \qquad C = \frac{2}{3} QS.$$

If a beam, supported at the ends, is loaded with a weight, this weight produces reactions at the two supports, the sum of which is equal to the weight. The weight and the reactions are the external forces acting on the beam. They produce a bending of the beam, by which the fibers of the upper portion of the beam are shortened and the fibers of the lower portion are elongated, the result of a compressive strain in the upper portion and a tensile strain in the lower portion of the cross-section of the beam. Between the top and the bottom of the cross-section is a place where no shortening or lengthening of the fibers occurs, and this is called the *neutral axis.* In steel, and in other homogeneous materials having equal resistances to compression and tension alike, the neutral axis is coincident with the center of gravity of the section, and in symmetrical sections, as in I-beams, this is at the middle of the depth of the beam.

At any point in the length of the beam, the tendency to produce bending is equal to the algebraic sum of the moments of the external forces at that point. This moment of the external forces is called the "bending moment." A beam resists bending at any point by the resistance of its particles to extension or compression, the sum of the moments of which about the neutral axis of the cross-section is called the "moment of resistance." The fundamental principle of the strength of beams is that the bending moment of the external forces is equal to the moment of resistance of the internal forces resisting flexure. As the moment of resistance of a section is generally expressed in inch-pounds, the bending-moment must also be expressed in inch-pounds. The following formulæ give the relations existing between bending-moment, moment of resistance, section modulus, and the strain per square inch.

$$m = \mathrm{R}; \qquad \mathrm{Q} = \frac{m}{\mathrm{S}};$$

$$m = \mathrm{QS}; \qquad \mathrm{S} = \frac{m}{\mathrm{Q}}.$$

If the bending-moment is in foot-pounds the following relations are convenient:

$$\mathrm{C} = 8\,\mathrm{M}; \qquad \mathrm{M} = \frac{\mathrm{C}}{8};$$

and for a uniformly distributed load, W, in lbs., the span, L, being taken in feet,

$$\mathrm{C} = \mathrm{WL}; \qquad \mathrm{W} = \frac{\mathrm{C}}{\mathrm{L}}.$$

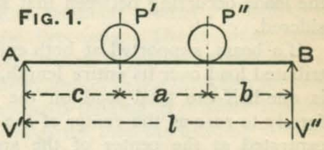
These last two formulæ are of great practical convenience for obtaining the safe uniformly distributed loads for the various sections, as it is only necessary to divide the coefficient of strength by the span, in feet, to obtain the safe uniformly distributed load, in lbs. If the uniformly distributed load, in lbs., is given, multiply it by the span in feet and the result is the required coefficient of strength, and the proper section required can be obtained by inspection of the tables.

The moment of inertia, section modulus, radius of gyration, and coefficient of strength are given in the tables of properties for all sections of structural shapes of steel rolled by the Passaic Rolling Mill Co.

REACTIONS.

If a beam resting at its extremities upon two supports is loaded with a weight, each support reacts with an upward pressure, which is called the *reaction* of the support. This reaction is equal to the weight carried by the support. The sum of the reactions of the two supports will equal the total load on the beam. If the load is either uniformly distributed, applied at the center of the span, or symmetrically placed on each side of the center of the span, the reaction of the two supports will be the same and each equal to one-half the load.

FIG. 1.



When the loads are not symmetrically placed, the reactions are determined in the following manner: — Let AB represent a beam supported at A and B and loaded with the weights P′ and P″. The reaction at one support due to a weight is equal to the weight multiplied by the distance of its center of gravity from the other support and divided by the length of the span. The total reaction at the support is equal to the sum of the reactions produced by all the loads. Then,

$$\frac{P'' \, b}{l} = \text{reaction at A due to weight } P'',$$

$$\frac{P' \, (a+b)}{l} = \text{reaction at A due to weight } P',$$

$$V' = \frac{P'' \, b}{l} + \frac{P' \, (a+b)}{l} = \text{total reaction at A.}$$

In the same way the total reaction V″, at B is obtained, and as a check on the calculations, $V' + V''$ must equal $P' + P''$.

SHEAR.

The loads and opposing reactions on a beam not only tend to bend the beam but also to shear it across vertically. The vertical force which tends to produce shearing is called the *shear*. The shear at an abutment or support is equal to the reaction of the support. At any point between the supports the shear is equal to the difference between the reaction at one support and the total load occurring between that support and the point considered. Thus, referring to Fig. 1, the shear at the support A is equal to the reaction V′. The shear at all points between A and the point of application of the load P′ is uniform and equal to the reaction V′, for the reason that no load occurs to be deducted from the reaction. The shear at any point between P′ and P″ is obtained by deducting the load P′ from the reaction V′, and the shear is therefore uniform between the points of application of these loads. Where a beam is loaded with concentrated weights, changes in the amount of shear occur only at the points where the loads are applied. If the load is distributed, the shear changes in amount at every point of the loaded length. In all cases the shear can be calculated by first finding the reaction at one support produced by the total load, and the shear at any point will be the difference between this reaction and the sum of all the loads occurring between that support and the point considered.

If a beam, supported at both ends, carries a uniformly distributed load over its entire length, the shear at each support is one-half the total load on the beam, and decreases uniformly to zero at the center of the span. If the load is concentrated at the center of the span, the shear is uniform throughout the entire length of the beam, and equal to one-half the load.

If the reaction, which acts upward, is considered as positive, and the loads, which act downward, are considered as negative, the shear at any point is the algebraic sum of the vertical forces acting on the beam between either support and the point considered.

BENDING-MOMENT.

The applied loads and their reactions constitute the external forces which tend to bend the beam. This bending is measured by the moment of the external forces, which is called the *bending-moment*. Let AB be a beam supported at its ends and loaded with the weights P_1, P_2, and P_3. These weights produce reactions at A and B, which are represented by V' and V'' respectively. If a section is taken at k, at a distance, x, from the left support, and the left-hand portion only of the beam is considered, the tendency to produce bending at k is measured by the moment of the reaction about that point. The moment of a force being equal to the product of the force by the lever arm of its action, the bending-moment at k is equal to the reaction V' multiplied by the distance x. Similarly the bending-moment at P_1 is equal to the product of the reaction V' by the distance a. At P_2 the reaction V' produces a moment equal to the product of the reaction by its distance from P_2, and the weight P_1 also produces a moment equal to the weight P_1 multiplied by its distance from P_2. The reaction acts upward and tends to produce rotation about P_2 in the direction of the motion of the hands of a watch. The weight P_1 acts downward and tends to produce rotation around P_2 in a direction opposite to the motion of the hands of a watch. The reaction V' and the weight P_1, therefore, produce moments around P_2 tending to produce rotation in opposite directions. The resulting bending-moment at P_2 is the difference of the two moments. If moments tending to produce rotation in one direction are considered as positive, and moments tending to produce rotation in the opposite direction as negative, then the bending moment at any point is obtained by taking the algebraic sum of the moments of all the forces, acting on the beam between either support and the point considered, around that point. From this it follows that the bending moment

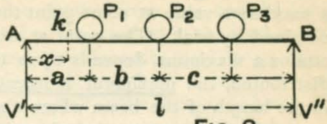


FIG. 2.

at $P_1 = V' a.$
at $P_2 = V' (a + b) - P_1 b.$
at $P_3 = V' (a + b + c) - P_1 (b + c) - P_2 c.$

In calculating the bending moment the weights are taken in pounds. If the distances are taken in feet the bending-moment will be expressed in foot-lbs. If the distances are taken in inches the bending-moment will be in inch lbs.

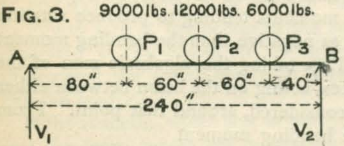
The bending-moment varies from point to point and attains a maximum value at some point the location of which can be obtained by trial. The point at which the bending-moment attains a maximum depends upon the shear. If the load is distributed, the maximum moment will occur at that point in the length of the beam where the shear becomes equal to zero; that is, at the point where the load on the beam between one support or abutment and the point considered becomes equal to the reaction of that support. If the loads are concentrated at several points, maximum bending will always occur at the point of application of one of the loads. The particular load at which maximum bending occurs, is the one at which the sum of all the loads on the beam between one support or abutment up to and including the load in question, first becomes equal to or greater than the reaction at the support.

In general, the bending-moment is a maximum at the point where the shear becomes equal to zero, or, due regard being paid to the algebraic sign of the shear, at the point where the shear changes from a positive value to a negative value, or the reverse.

EXAMPLE.

Let AB represent a beam, 20 feet long between centers of supports, loaded in the manner shown:

FIG. 3.



The portion of the load P_3 carried by the left-hand support is $\frac{40}{240}$ of P_3, or 1,000 lbs.; the portion of P_2 carried by the left-hand support is $\frac{100}{240}$ of P_2, or 5,000 lbs.; similarly the portion of P_1 carried by the same support is $\frac{160}{240}$ of P_1; or 6,000 lbs. The reaction, V_1, of the left support is the sum of these three, or 12,000 lbs. In the same manner the reaction V_2, at the right-hand support, can be obtained by taking the sum of the portions of the loads going to that support, and will be found to be 15,000 lbs. The sum of the two reactions must equal the sum of the loads on the beam.

If the bending-moment is taken at the point of application of the load P_2, and the left-hand portion of the beam only is considered, the reaction V_1 produces a moment equal to the product of the reaction by its distance from P_2; and the load P_1 produces a moment equal to the product of the load by its distance from P_2. As these two moments tend to produce rotation in opposite directions, the resultant moment of the external forces around P_2 is equal to the difference between these two moments, or the bending moment, in inch-lbs.,

$$m = V_1 \times 140 - P_1 \times 60 = 12{,}000 \times 140 - 9{,}000 \times 60$$
$$= 1{,}140{,}000 \text{ inch-lbs.}$$

In this case this is the maximum bending-moment on the beam, because at the load P_2 the sum of the loads on the beam between the support A up to and including P_2 first becomes equal to, or greater than, the reaction at A.

If it is required to find the proper size of steel beam necessary to safely carry the above loads, the section modulus is found from the foregoing formulæ, assuming a fiber strain of 16,000 lbs. per square inch, as follows:

$$Q = \frac{m}{S} = \frac{1{,}140{,}000}{16{,}000} = 71.25$$

A 15″ steel I-beam, weighing 50 lbs. to the foot, has a section modulus of 70.6, and is sufficient for the purpose.

If the bending-moment is wanted in foot-pounds, the lengths are taken in feet instead of in inches; and

$$M = V_1 \times 11\tfrac{2}{3} - P_1 \times 5 = 12{,}000 \times 11\tfrac{2}{3} - 9{,}000 \times 5$$
$$= 95{,}000 \text{ foot-lbs.}$$

and the coefficient of strength required for a steel beam to carry the loads is,

$$C = 8M = 8 \times 95{,}000 = 760{,}000$$

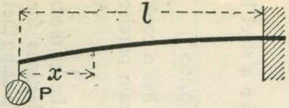
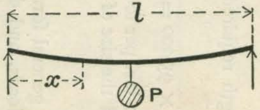
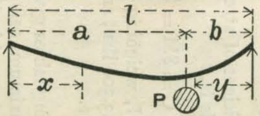
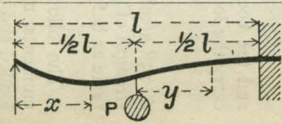
A 15″ steel I, weighing 50 lbs. per foot, has a coefficient of strength of 753,300 lbs., and the size of beam required is the same as before.

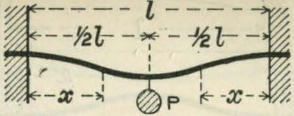
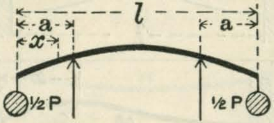
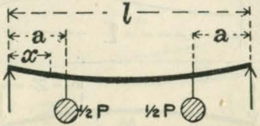
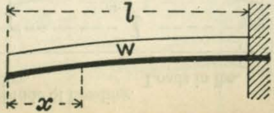
The following tables give general formulæ for the bending-moments, maximum safe loads, and deflections for beams loaded and supported in different ways. In using these tables to obtain loads, or deflections, all lengths must be expressed in inches.

8 TIMES BENDING MOMENT IN FOOT-POUNDS EQUALS COEFFICIENT OF STRENGTH.
BEND. MOM. IN INCH POUNDS ÷ ALLOWABLE FLANGE STRAIN EQUALS SECTION MODULUS.

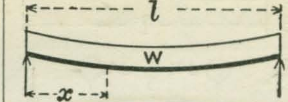
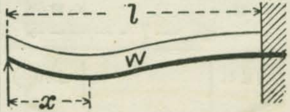
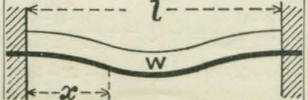
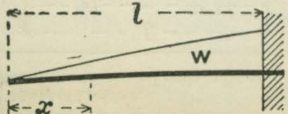
8M = Coef

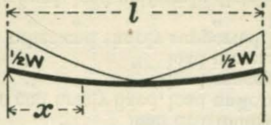
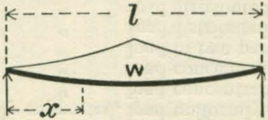
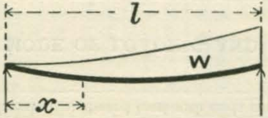
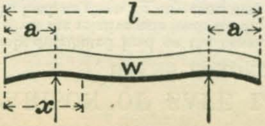
$\frac{M}{S} = K$ (section modulus)

Mode of Loading. Lengths in inches.	Loads in lbs.	Bending Moment, inch lbs.	Max. Load, Lbs.	Deflection, inches.	Remarks.
One end firmly fixed, other end loaded.		Px Max. when $x = l$	$\frac{SQ}{l}$	$\frac{Pl^3}{3EI}$	Weakest section at right support.
Supported at both ends, loaded at center.		$\frac{Px}{2}$ Max. $= \frac{Pl}{4}$	$\frac{4SQ}{l}$	$\frac{Pl^3}{48EI}$	Weakest section at center of beam.
Supported at both ends, loaded any place.		For the left side, $\frac{Pbx}{l}$ For the right side, $\frac{Pay}{l}$	$\frac{lSQ}{ab}$	$\frac{Pab(2l-a)\sqrt{3a(2l-a)}}{27lEI}$	Weakest section at point of application of load.
One end fixed, other end supported, loaded at center.		For the left side, $\frac{5Px}{16}$ For the right side, $Pl\left(\frac{5}{32}-\frac{11y}{16l}\right)$	$\frac{16SQ}{3l}$	$\frac{3Pl^3}{322EI}$	Weakest section at right support.

Mode of Loading. Lengths in inches.	Loads in lbs.	Bending Moment, inch lbs.	Max. Load, Lbs.	Deflection, inches.	Remarks.
Both ends fixed, load at center.		$\frac{Pl}{2}\left(\frac{x}{l}-\frac{1}{4}\right)$ Max. $= \frac{Pl}{8}$	$\frac{8SQ}{l}$	$\frac{Pl^3}{192EI}$	Weakest sections at either support, and at center.
Loaded at each end, two supports between ends.		$\frac{Px}{2}$ Max. $= \frac{Pa}{2}$	$\frac{2SQ}{a}$	For overhang: $\frac{Pa}{12EI}\left(3al-4a^2\right)$ For part between supports: $\frac{Pa}{16EI}\left(l-2a\right)^2$	Weakest sections at either support, and at all points between supports.
Both ends supported, two symmetrical loads.		$\frac{Px}{2}$ Max. $= \frac{Pa}{2}$	$\frac{2SQ}{a}$	$\frac{Pa}{48EI}\left(3l^2-4a^2\right)$	Weakest sections at points of application of loads, and at all points between loads.
One end fixed, load uniformly distributed.		$\frac{Wx^2}{2l}$ Max. $= \frac{Wl}{2}$	$\frac{2SQ}{l}$	$\frac{Wl^3}{8EI}$	Weakest section at right support.

$\cancel{M} = SK = S\frac{I}{c}$ M = Bending Moment in inch pounds
S = Greatest allowable stress in any fibre in pounds
K = Section Modulus. I = Moment of Inertia about neutral axis
c = distance of farthest edge of section from N. axis.

Mode of Loading. l = Lengths in inches. W = Loads in lbs.		Bending Moment, inch lbs.	Max. Load, Lbs.	Deflection, inches.	Remarks.
Both ends supported, load uniformly distributed.		$\frac{Wx}{2}\left(1-\frac{x}{l}\right)$ Max. $=\frac{Wl}{8}$	$\frac{8SQ}{l}$	$\frac{5Wl^3}{384EI}$	Weakest section at center.
One end supported, other end fixed, load uniformly distributed.		$\frac{Wx}{2}\left(\frac{3}{4}-\frac{x}{l}\right)$ Max. $=\frac{Wl}{8}$	$\frac{8SQ}{l}$	$\frac{5Wl^3}{926EI}$	Weakest section at right support.
Both ends fixed, load uniformly distributed.		$\frac{Wl}{2}\left(\frac{x}{l}-\frac{x^2}{l^2}-\frac{1}{6}\right)$ Max. $=\frac{Wl}{12}$	$\frac{12SQ}{l}$	$\frac{Wl^3}{384EI}$	Weakest section at either support.
One end fixed, load distributed increasing uniformly towards the fixed end.		$\frac{Wx^3}{3l^2}$ Max. $=\frac{Wl}{3}$	$\frac{3SQ}{l}$	$\frac{Wl^3}{15EI}$	Weakest section at right support.

Mode of Loading. Lengths, in inches.	Loads, in lbs.	Bending Moment, inch lbs.	Max. Load, lbs.	Deflection, inches.	Remarks.
Both ends supported, load distributed, decreasing uniformly toward the center.		$Wx\left(\frac{1}{2}-\frac{x}{l}+\frac{2x^2}{3l^2}\right)$ Max. $=\frac{Wl}{12}$	$\frac{12SQ}{l}$	$\frac{3Wl^3}{320EI}$	Weakest section at center of span.
Both ends supported, load distributed, increasing uniformly toward the center.		$Wx\left(\frac{1}{2}-\frac{2x^2}{3l^2}\right)$ Max. $=\frac{Wl}{6}$	$\frac{6SQ}{l}$	$\frac{Wl^3}{60EI}$	Weakest section at center of span.
Both ends supported, load distributed, increasing uniformly toward one end.		$\frac{Wx}{3}\left(1-\frac{x^2}{l^2}\right)$ Max. $=\frac{104\,Wl}{810}$	$\frac{810SQ}{104l}$	$\frac{47Wl^3}{3600EI}$	Weakest section at $x = 0.52l$
Two symmetrical supports, load uniformly distributed.		At either support: $\frac{Wa^2}{2l}$ At center of span: $\frac{W}{2}\left(a-\frac{l}{4}\right)$	The supporting power varies with the relation of a to l, and becomes a maximum when $a = 0.207l$, in which case, Max. Bend. Mom. $=\frac{3Wl}{140}$; Max. Load $=\frac{140SQ}{3l}$		

COMPARISON OF SAFE LOADS AND CORRESPONDING DEFLECTIONS,

BEAMS LOADED AND SUPPORTED IN VARIOUS WAYS.

The safe uniformly distributed load on the beam, having its ends simply supported, is taken as a unit of comparison, and the second column of the table gives the relative safe loads for the various ways of applying the load and supporting the beam. The third column gives a factor by which the load, as given for any case, may be multiplied and the result considered as a uniformly distributed load on the beam, having each end simply supported. The last column gives the relative deflections for the various cases under their safe loads, the deflection under the safe uniformly distributed load with ends simply supported, being taken as the unit of comparison.

MODE OF LOADING AND SUPPORTING BEAM.	Relative Safe Load.	Factor for Obtaining Equiv. Uniform Load, Ends Sup'd.	Relative Deflection under Safe Load.
Both ends simply supported, load uniformly distributed	1	1	1
" " " " load concentrated at center of beam	$\frac{1}{2}$	2	0.80
" " " " load concentrated anywhere between supports	$l^2 \div 8ab$	$8ab \div l^2$	Variable.
" " " " load in two parts symmetrically concentrated	$l \div 4a$	$4a \div l$	Variable.
" " " " load distributed, decreasing uniformly toward the center	$1\frac{1}{2}$	$\frac{2}{3}$	1.07
" " " " load distributed, increasing uniformly toward the center	$\frac{3}{4}$	$1\frac{1}{3}$	0.96
" " " " load distributed, increasing uniformly toward one end	0.97	1.03	0.97
Cantilever; one end firmly fixed, load uniformly distributed	$\frac{1}{4}$	4	2.40
" " " " " load concentrated at other end	$\frac{1}{8}$	8	3.20
" " " " " load increasing uniformly toward fixed end	$\frac{3}{8}$	$2\frac{2}{3}$	1.92
One end fixed; other end simply supported, load uniformly distributed	1	1	0.42
" " " " " " " load concentrated at center	$\frac{2}{3}$	$1\frac{1}{2}$	0.48
Both ends firmly fixed, load uniformly distributed	$1\frac{1}{2}$	$\frac{2}{3}$	0.30
" " " " load concentrated at center	1	1	0.40
Two symmetrical supports between ends, load in two parts, concentrated at each end	$l \div 4a$	$4a \div l$	Variable.
Two symmetrical supports between ends, load uniformly distributed, supports economically placed	5.83	0.17	

Bending Moment in Foot Pounds = 222.5 bd^2
b = width of Plate. d = Depth.

MOMENT OF INERTIA OF RECTANGLES.

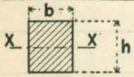
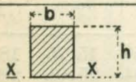
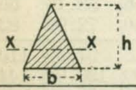
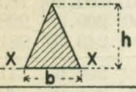
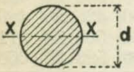
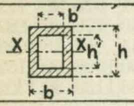
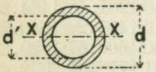
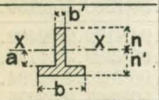
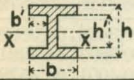
b

AX IS. $I = \frac{bd^3}{12}$

d

Depth, in inches.	Width of Rectangle, in inches.						
	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1
6	4.50	6.75	9.00	11.25	13.50	15.75	18.00
7	7.15	10.72	14.29	17.86	21.44	25.01	28.58
8	10.67	16.00	21.33	26.67	32.00	37.33	42.67
9	15.19	22.78	30.38	37.97	45.56	53.16	60.75
10	20.83	31.25	41.67	52.08	62.50	72.92	83.33
11	27.73	41.59	55.46	69.32	83.18	97.06	110.92
12	36.00	54.00	72.00	90.00	108.00	126.00	144.00
13	45.77	68.66	91.54	114.43	137.31	160.20	183.08
14	57.17	85.75	114.33	142.92	171.50	200.08	228.67
15	70.31	105.47	140.63	175.78	210.94	246.09	281.25
16	85.33	128.00	170.67	213.33	256.00	298.67	341.33
17	102.35	153.53	204.71	255.89	307.06	358.24	409.42
18	121.50	182.25	243.00	303.75	364.50	425.25	486.00
19	142.90	214.34	285.79	357.24	428.68	500.14	571.58
20	166.67	250.00	333.33	416.67	500.00	583.33	666.67
21	192.94	289.41	385.88	482.34	578.81	675.28	771.75
22	221.83	332.75	443.67	554.58	665.50	776.42	887.33
23	253.48	380.22	506.96	633.70	760.44	887.18	1013.92
24	288.00	432.00	576.00	720.00	864.00	1008.00	1152.00
25	325.52	488.28	651.04	813.80	976.56	1139.32	1302.08
26	366.17	549.25	732.33	915.42	1098.50	1281.58	1464.67
27	410.06	615.09	820.13	1025.16	1230.19	1435.22	1640.25
28	457.33	686.00	914.67	1143.33	1372.00	1600.67	1829.33
29	508.10	762.16	1016.21	1270.26	1524.31	1778.36	2032.42
30	562.50	843.75	1125.00	1406.25	1687.50	1968.75	2250.00
31	620.65	930.97	1241.30	1551.62	1861.94	2172.26	2482.60
32	682.67	1024.00	1365.33	1706.67	2048.00	2389.33	2730.67
33	748.69	1123.03	1497.38	1871.72	2246.06	2620.40	2994.76
34	818.83	1228.25	1637.67	2047.08	2456.50	2865.92	3275.33
35	893.23	1339.84	1786.46	2233.07	2679.68	3126.30	3572.92
36	972.00	1458.00	1944.00	2430.00	2916.00	3402.00	3888.00
37	1055.27	1582.90	2110.54	2638.17	3165.80	3693.44	4221.08
38	1143.17	1714.75	2286.33	2857.92	3429.50	4001.08	4572.67
39	1235.81	1853.72	2471.62	3089.53	3707.44	4325.34	4943.24
40	1333.33	2000.00	2666.67	3333.33	4000.00	4666.67	5333.33

MOMENT OF INERTIA AND SECTION MODULUS FOR USUAL SECTIONS.

Sections.	Moment of Inertia, I.	Section Modulus, Q.
	$I=\frac{bh^3}{12}$	$\frac{bh^2}{6}$
	$I'=\frac{bh^3}{3}$	
	$I=\frac{bh^3}{36}$	Min. $=\frac{bh^2}{24}$
	$I'=\frac{bh^3}{12}$	
	$I=\frac{\pi d^4}{64}$ $=0.0491\ d^4$	$\frac{\pi d^3}{32}$ $=0.0982\ d^3$
	$I=\frac{bh^3-b'h'^3}{12}$	$\frac{I}{0.5h}$
	$I=0.0491\ (d^4-d'^4)$	$0.0982\left(d^3-\frac{d'^4}{d}\right)$
	$I=\frac{b'n^3+bn'^3-(b-b')a^3}{3}$	Min. $=\frac{I}{n}$
	$I=\frac{bh^3-2b'h'^3}{12}$	$\frac{I}{0.5h}$

XX Denotes position of neutral axis.

FIREPROOF CONSTRUCTION.

A simple type of fireproof construction is illustrated in Fig. 1, page 34. Figs. 2, 3 and 4 show the manner of connecting the beams and girders with each other by means of connection angles, which are riveted or bolted to the beams and girders. The standard sizes of these connection angles and the number of bolts or rivets required are given on pages 41–43. The manner of connecting the beams and girders to the columns is illustrated by the drawings on page 39.

Brick arches were formerly largely used for the construction of fireproof floors in buildings. This type of construction consists usually of a 4″ course of brick, resting on the lower flanges of the **I** beams against brick skewbacks, the arch having a rise at the center of not less than 3″, and not less than $1\frac{1}{4}''$ rise for each foot of span; in case the floor is to carry heavy loads, two or more courses of brick should be used. The **I** beam joists should be spaced about 5 or 6 ft. centers. The space above the arches is filled with concrete in which wooden strips are imbedded, to which the floor is nailed. The plastered ceiling is applied directly to the under side of the brick arches. The horizontal thrust of the arches must be provided for by the use of tie-rods, generally $\frac{3}{4}''$ diameter, spaced at intervals of from 5 to 6 ft. The tie-rods should pass through the beams as near the center of the skewback as possible; generally, the tie-rods should pass through the beams at a distance from the bottom of the beam equal to $\frac{1}{3}$ the depth of the beam. The thrust of the arches, in lbs. per lineal foot, can be found by the formula, $T=\frac{3\,W\,L^2}{2R}$, in which W is the load per square foot, L the span of the arch in feet, and R the rise of the arch in inches. A channel or an angle should be used to support the arches abutting against the walls, and to properly distribute the loads upon the walls. The tie-rods in the arches abutting against the walls should be securely anchored to the wall channels or angles. The excessive weight and the lack of adequate protection of the lower flanges of the beams are serious objections to this type of construction; and where flat ceilings are required it is unavailable.

Hollow brick flat arches of the types shown on pages 35 and 36 are very generally used for the construction of fireproof floors. These arches are generally of porous terra-cotta material, which is made of a mixture of clay and sawdust subjected to an intense heat, which consumes the combustible material, leaving the brick porous and reducing the weight materially while preserving the fireproof qualities intact. For arches, partitions, furring, column covering, roof and ceiling tiles, etc., it is particularly adapted. It receives and holds plaster and readily admits driving of nails, which hold equally as well as if driven in wood. The underside of the arch being flat permits the construction of a level ceiling. The joints in the arches are made radial, and the blocks should be thoroughly cemented together. The beams should be spaced from 4 to 6 ft. apart and connected together with $\frac{3}{4}''$ diameter tie-rods at intervals not exceeding 6 ft. The arch should have a thickness of at least $1\frac{1}{4}''$ for each foot of span. The space above the arches is filled with a light concrete consisting of cinders and cement, into which wooden strips are imbedded, to which the flooring is nailed.

Fireproof partitions are constructed of porous terra-cotta hollow brick blocks set with broken joints and held in place at intervals with light angle iron or Tee iron studding.

Roofs and ceilings are constructed of hollow tiles set between Tee irons, as shown on page 36. Suspended ceilings may also be constructed of light Tee irons covered with wire lathing and plastered.

All ironwork should be protected by a covering of fireproof material. The arches should always have a protection flange covering the underside of the beams. Beams, girders and columns, not inclosed in the flooring or partitions, should have a covering of fireproof material similar to the types illustrated on page 35. Particular attention should always be given to the proper covering of all ironwork with fireproof material in order that it may be protected from heat and prevent warping and settlement in case of fire.

The following table gives approximate safe loads, in lbs. per square foot, for ordinary flat arches, with a factor of safety of from 6 to 8, deduced from recent experiments on arches of this type. The margin of safety should be large for the reason that, owing to the hasty and imperfect manner in which the arches are built in ordinary construction, they are liable to fail under much lighter loads than if carefully set.

APPROXIMATE SAFE LOADS ON FLAT ARCHES,

Pounds per Square Foot.

Depth of Arch, Inches.	Distance between Beams.				
	4 ft.	5 ft.	6 ft.	7 ft.	8 ft.
6	150	100			
7	200	150			
8	275	175	125		
9	300	200	140		
10	325	225	150	100	
12	400	250	200	125	100

The weight of the fireproof construction should be calculated for each case. The floor weight consists of the weight of the arches, filling, flooring, plaster ceiling, and steel construction. Where partitions are permanent the floor beams immediately under them should be calculated to carry the partitions in addition to the regular floor load; but where partitions are not permanent, as in office buildings, it is customary to add 20 lbs. per sq. ft. to the weight of the floor construction in order to cover the weight of the partitions, thus permitting them to be changed, from time to time, as circumstances may require. The approximate weights of different types of fireproof floor construction are given in the following table.

The weights of the arches are taken from catalogues of standard manufacturers. The weight of the cinder concrete filling is taken at 72 lbs. per cubic foot. The finished floor line is assumed to be 3″ above the top of the steel **I** beams, and the finished plaster line 2″ below the underside of the **I** beams, except for brick arches. Cinder concrete is sometimes assumed to weigh 48 lbs. per cubic foot, but samples of perfectly dry cinder concrete from filling in New York buildings will average 72 lbs. per cubic foot.

APPROXIMATE WEIGHTS OF FIREPROOF FLOORS,

Exclusive of Partitions.

Type of Arch.	Depth of I Beam, Ins.	Thickness of Arch, Ins.	Thickness of Floor, Ins.	Weight, in lbs., per Square Foot. Arches.	Filling.	Flooring.	Ceiling.	Steel.	Total.
Ordinary Brick Arch.	8	4	12	40	18	4	4	8	74
	9	4	12	40	18	4	4	8	74
	10	4	13	40	24	4	4	9	81
	12	4	15	40	36	4	4	10	94
	15	4	18	40	54	4	4	11	113
Hollow Brick Flat Arch, Ordinary Type.	8	6	13	29	30	4	4	7	74
	8	8	13	35	18	4	4	7	68
	9	6	14	29	36	4	4	7	80
	9	9	14	37	18	4	4	7	70
	10	8	15	35	30	4	4	8	81
	10	10	15	41	18	4	4	8	75
	12	8	17	35	42	4	4	8	93
	12	12	17	48	18	4	4	8	82
	15	8	20	35	60	4	4	10	113
	15	12	20	48	36	4	4	10	102
Hollow Brick Flat Arch, End Construction Type.	8	8	13	30	18	4	4	7	63
	9	8	14	30	24	4	4	7	69
	9	9	14	32	18	4	4	7	65
	10	8	15	30	30	4	4	8	76
	10	10	15	34	18	4	4	8	68
	12	8	17	30	42	4	4	8	88
	12	12	17	37	18	4	4	8	71
	15	8	20	30	60	4	4	10	108
	15	12	20	37	36	4	4	10	91

PLASTER 5# PER SQ. FT.

In addition to the weight of the floor construction, which is called the dead load, the floors must be designed to carry a live load of sufficient amount, which is usually determined by the purpose for which the building is to be used. The live load comprises the people in the building, furniture, movable stocks of goods, small safes, and varying loads of any character. Large safes require special provision usually embodied in the construction. The following live loads, per sq. ft., are recommended as good practice in building construction:

Plaster on wirelath. Wirelath secured to light angles – 26# per sq ft
3x16 Hemlock 16" c to c 6# " "
Heavy Horse 40# " "

3x10 SPRUCE 16" c to c 5 LBS
" 12 " 6 "
1 3/4 N.C. FLOORING 5 "

Dwellings	50 lbs.
Offices	70 "
Hotels and apartment houses	70 "
Theatres and churches	120 "
Ball-rooms and drill-halls	120 "
Lofts for light manufacturing purposes	150 "
Factories	from 150 " up.
LIBRARIES & Warehouses	" 250 " "
PLASTER	5 "
2x4 SPRUCE 12" C. to C	2 "

Dwellings (no crowd), hotels, Country houses, any light buildings = 40#

Heavy coaches

Retail Stores = 100# up
Wholesale " = 150# "
Schools and churches = 40# to 50#

The weight of a crowd of people is usually assumed at 80 lbs. per sq. ft., but the weight of a very densely packed crowd may be as much as 120 lbs. The latter load can scarcely occur under the conditions governing an office building. Large crowds seldom collect in offices except on the lower floors devoted to stores and banking purposes, for which floors proper allowance for live loads is usually made. The actual live loads on office floors are generally much less than given in the preceding table. Messrs. Blackall & Everett, Architects, of Boston, made a careful canvass of the live loads in 210 Boston offices, and found that the average live load for the entire number of offices was about 17 lbs. per sq. ft. The greatest live load in any one office was 40 lbs. per sq. ft., while the average live load for the heaviest 10 offices was 33 lbs. per sq. ft. These figures give some idea of the average actual live loads in such buildings; but the use of such light average loads is not to be recommended, as the actual live load is liable to be concentrated, thus producing an effect greater than represented by the average load. Provision should be made for all possibilities of extreme, either present or future. No single floor should be proportioned for a live load less than those previously given. In high office buildings, hotels, and apartment houses, the foundations and lower tiers of columns may safely be proportioned for a live load of 50 lbs. per sq. ft. on all the floors; but the floors themselves and the upper tiers of columns should be proportioned for the full live loads previously given. Factories, warehouses, and similar buildings should be proportioned throughout for the full live load on each floor.

Building ordinances regulate the design of buildings in several of the larger cities, and the designer must be governed accordingly. The salient features of the Building Laws of New York, Chicago, and Boston are embodied in the following table.

COMPARISON OF BUILDING LAWS.

	New York.	Chicago.	Boston.
Floor Loads, lbs. per sq. ft.			
Dwellings	~~70~~ 60	70	70
Hotels and Apartments	~~70~~ 60	70	70
Office Buildings	~~100~~ 70	70	100
Places of Public Assembly	~~120~~ 90	70	150
Stores, Warehouses, Factories, etc.	150 up	150 up	250 up
Allowable Strains, lbs. per sq. in.			
Rolled Steel Beams and Shapes	16000 ~~15,000~~	16,000	16,000
Tension, Steel Shapes	16000	16,000	15,000
Compression Flanges, built Steel Beams	15,000	13,500	12,000
Shearing, Steel Web Plates	7,000	10,000	10,000
Shearing, Shop Rivets, Steel	9,000	9,000	10,000
Shearing, Field Rivets, Steel	....	7,500	...
Bearing on Steel Pins and Rivets	15,000	....	18,000
Bending on Steel Pins	....	....	22,500
Steel Columns	$\dfrac{12{,}000}{1+\dfrac{l^2}{36{,}000r^2}}$	$17{,}000-60\dfrac{l}{r}$ and not to exceed 13,500	$\dfrac{12{,}000}{1+\dfrac{l^2}{36{,}000r^2}}$
Round Cast Iron Columns	$\dfrac{16{,}000}{1+\dfrac{l^2}{400d^2}}$	$\dfrac{10{,}000}{1+\dfrac{l^2}{600d^2}}$	$\dfrac{10{,}000}{1+\dfrac{l^2}{800d^2}}$
Square Cast Iron Columns	$\dfrac{16{,}000}{1+\dfrac{l^2}{500d^2}}$	$\dfrac{10{,}000}{1+\dfrac{l^2}{800d^2}}$	$\dfrac{10{,}000}{1+\dfrac{l^2}{1{,}066d^2}}$
Allowable Pressures, tons per sq. ft.			
Granite	....	38	60
Marble and Limestone	....	30	40
Sandstone	....	24	30
Brickwork in Portland Cement Mortar	....	15	....
Brickwork in ordinary Cement Mortar	15	12	15
Brickwork in Cement and Lime Mortar	11½	....	12
Brickwork in Lime Mortar	8	8	8
Clay, 15 ft. thick	....	2	....
Dry Sand, 15 ft. thick	....	1¾	....
Clay and Sand	....	1½	....
Good Solid Natural Earth	4	....	....
Loads on piles, tons each	20	25	....

EXPLANATION OF TABLES ON PASSAIC STEEL I BEAMS USED AS FLOOR JOISTS AND GIRDERS.

The tables on pages 102–109, inclusive, furnish a convenient means of selecting the proper number, size, and weight of steel **I** beams for floor joists and girders for total floor loads of 125 to 200 lbs. per sq. ft.

Thus, if it is desired to select a steel **I** beam used as a floor joist, when the beams are spaced 5 ft. centers on a span of 20 ft., to support a total uniformly distributed load of 175 lbs. per sq. ft., it is found by reference to the table of floor joists, page 106, opposite a span of 20 ft. and in the column 5 ft. centers, that the beam required is a 12″ **I** weighing 31½ lbs. per foot.

The girders for the same floor may be selected in a similar manner, knowing their span and spacing. Thus, if the girders are spaced 20 ft. centers (which is the span of the joists as before), and the columns supporting the girders are spaced 20 ft. apart, it is found by referring to the table of girders, page 107, opposite a span of 20 ft. and in the column 20 ft. centers, that the girder required must consist of two 15″ × 50 lb. **I** beams.

It must be observed that these tables are calculated for *uniformly distributed* loads, and if the loads are concentrated it is advisable to make a separate calculation in each case.

Where the loads on girders are concentrated at a few points, or irregularly spaced, the tables of girders are not exact in all cases; but for ordinary cases where the joists are regularly spaced, and the length of the girder is at least twice the spacing of the joists, the tables of girders are sufficiently exact for all practical purposes.

Strict accuracy in the design of girders can only be obtained by calculation, following the method outlined in the article on The Strength and Deflection of Beams, and using the actual concentrations of load.

PASSAIC STEEL I BEAMS

USED AS FLOOR JOISTS.

Total Uniformly distributed Load, 125 lbs. per square foot.

Span of Joist, in feet.	Size and Weight of Steel I Beams required for Joists, when Joists are Spaced,						
	4 ft. centers.	5 ft. centers.	6 ft. centers.	7 ft. centers.	8 ft. centers.	9 ft. centers.	10 ft. centers.
	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.
5	4×6	4×6	4×6	4×6	4×6	4×7½	4×7½
6	4×6	4×6	4×8	4×7½	5×9¾	5×9¾	5×9¾
7	4×6	4×7½	5×9¾	5×9¾	5×9¾	6×12	6×12
8	4×8	5×9¾	5×9¾	6×12	6×12	6×12	6×12
9	5×9¾	5×9¾	6×12	6×12	6×15	6×15	6×15
10	5×9¾	6×12	6×12	6×15	7×15	7×15	8×18
11	6×12	6×12	6×15	7×15	8×18	8×18	8×18
12	6×12	6×15	7×15	8×18	8×18	9×21	9×21
13	6×15	7×15	8×18	8×18	9×21	9×21	9×23⅓
14	7×15	8×18	8×18	9×21	9×21	10×25	10×25
15	8×18	8×18	9×21	9×21	10×25	10×25	10×30
16	8×18	9×21	9×21	10×25	10×25	10×30	12×31½
17	9×21	9×21	10×25	10×25	10×30	12×31½	12×31½
18	9×21	9×21	10×25	10×30	12×31½	12×31½	12×31½
19	9×21	10×25	10×30	12×31½	12×31½	12×31½	12×40
20	10×25	10×25	12×31½	12×31½	12×31½	12×40	12×40
21	10×25	12×31½	12×31½	12×31½	12×40	12×40	15×42
22	10×30	12×31½	12×31½	12×40	12×40	15×42	15×42
23	12×31½	12×31½	12×31½	12×40	15×42	15×42	15×50
24	12×31½	12×31½	12×40	12×40	15×42	15×50	15×50
25	12×31½	12×40	15×42	15×42	15×50	15×50	15×60
26	12×31½	12×40	15×42	15×42	15×50	15×50	15×60
27	12×40	15×42	15×42	15×50	15×50	15×60	15×60
28	15×42	15×42	15×42	15×50	15×60	15×60	15×66⅔
29	15×42	15×42	15×50	15×50	15×60	15×66⅔	15×75
30	15×42	15×42	15×50	15×60	15×60	15×75	20×65

Deflections not exceeding $\frac{1}{360}$ of the span.

PASSAIC STEEL I BEAMS USED AS FLOOR GIRDERS.

Total uniformly distributed Load, 125 lbs. per square ft.

Span of Girder, in ft.	Number, size and weight of beams required, for each girder, when girders are spaced,										
	10 ft. centers.	12 ft. centers.	14 ft. centers.	16 ft. centers.	18 ft. centers.	20 ft. centers.	22 ft. centers.	24 ft. centers.	26 ft. centers.	28 ft. centers.	30 ft. centers.
	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.
10	1– 8×18	1– 8×18	1– 9×21	1– 9×21	1–10×25	1–10×25	1–10×30	1–12×31½	1–12×31½	1–12×31½	1–12×31½
11	1– 8×18	1– 9×21	1– 9×23⅓	1–10×25	1–10×30	1–12×31½	1–12×31½	1–12×31½	1–12×31½	1–12×40	1–12×40
12	1– 9×21	1–10×25	1–10×25	1–10×30	1–12×31½	1–12×31½	1–12×31½	1–12×40	1–12×40	1–12×40	1–15×42
13	1– 9×23⅓	1–10×25	1–12×31½	1–12×31½	1–12×31½	1–12×40	1–12×40	1–12×40	1–15×42	1–15×42	1–15×50
14	1–10×25	1–10×30	1–12×31½	1–12×31½	1–12×40	1–12×40	1–15×42	1–15×42	1–15×50	1–15×50	1–15×50
15	1–10×30	1–12×31½	1–12×31½	1–12×40	1–12×40	1–15×42	1–15×50	1–15×50	1–15×50	1–15×60	1–15×60
16	1–12×31½	1–12×31½	1–12×40	1–15×42	1–15×42	1–15×50	1–15×50	1–15×50	1–15×60	1–15×60	1–15×66⅔
17	1–12×31½	1–12×40	1–12×40	1–15×42	1–15×50	1–15×50	1–15×60	1–15×60	1–15×66⅔	1–15×75	2–15×42
18	1–12×31½	1–12×40	1–15×42	1–15×50	1–15×50	1–15×60	1–15×60	1–15×66⅔	2–15×42	2–15×42	2–15×42
19	1–12×40	1–15×42	1–15×50	1–15×50	1–15×60	1–15×60	1–15×75	2–15×42	2–15×42	2–15×50	2–15×50
20	1–12×40	1–15×42	1–15×50	1–15×60	1–15×60	1–15×75	2–15×42	2–15×42	2–15×50	2–15×50	2–15×50
21	1–15×42	1–15×50	1–15×60	1–15×60	1–15×75	2–15×42	2–15×42	2–15×50	2–15×50	2–15×60	2–15×60
22	1–15×42	1–15×50	1–15×60	1–15×66⅔	2–15×42	2–15×42	2–15×50	2–15×50	2–15×60	2–15×60	2–15×60
23	1–15×50	1–15×60	1–15×60	2–15×42	2–15×42	2–15×50	2–15×50	2–15×60	2–15×60	2–15×66⅔	2–15×75
24	1–15×50	1–15×60	1–15×75	2–15×42	2–15×50	2–15×50	2–15×60	2–15×60	2–15×66⅔	2–15×75	2–20×65
25	1–15×60	1–15×66⅔	2–15×42	2–15×50	2–15×50	2–15×60	2–15×60	2–15×66⅔	2–15×75	2–20×65	2–20×65

Deflections not exceeding $\frac{1}{360}$ of the span.

PASSAIC STEEL I BEAMS

USED AS FLOOR JOISTS.

Total uniformly distributed Load, 150 lbs. per square foot.

Span of Joist, in feet.	Size and Weight of Steel I Beams required for Joists, when Joists are Spaced,						
	4 ft. centers.	5 ft. centers.	6 ft. centers.	7 ft. centers.	8 ft. centers.	9 ft. centers.	10 ft. centers.
	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.
5	4×6	4×6	4×6	4×7½	4×7½	5×9¾	5×9¾
6	4×6	4×7½	5×9¾	5×9¾	5×9¾	5×9¾	6×12
7	4×7½	5×9¾	5×9¾	5×9¾	6×12	6×12	6×12
8	5×9¾	5×9¾	6×12	6×12	6×12	6×15	7×15
9	5×9¾	6×12	6×12	6×15	7×15	7×15	8×18
10	6×12	6×12	6×15	7×15	8×18	8×18	8×18
11	6×12	6×15	7×15	8×18	8×18	9×21	9×21
12	6×15	7×15	8×18	8×18	9×21	9×21	10×25
13	7×15	8×18	8×18	9×21	9×21	10×25	10×25
14	8×18	8×18	9×21	9×23⅓	10×25	10×25	10×30
15	8×18	9×21	9×21	10×25	10×30	12×31½	12×31½
16	8×18	9×21	10×25	10×30	12×31½	12×31½	12×31½
17	9×21	10×25	10×25	12×31½	12×31½	12×31½	12×40
18	9×21	10×25	10×30	12×31½	12×31½	12×40	12×40
19	10×25	10×30	12×31½	12×31½	12×40	12×40	15×42
20	10×25	12×31½	12×31½	12×40	12×40	15×42	15×42
21	10×30	12×31½	12×31½	12×40	15×42	15×42	15×50
22	12×31½	12×31½	12×40	12×40	15×42	15×50	15×50
23	12×31½	12×31½	12×40	15×42	15×50	15×50	15×60
24	12×31½	12×40	15×42	15×42	15×50	15×60	15×60
25	12×31½	12×40	15×42	15×50	15×50	15×60	15×66⅔
26	12×40	15×42	15×42	15×50	15×60	15×60	15×75
27	15×42	15×42	15×50	15×50	15×60	15×66⅔	20×65
28	15×42	15×42	15×50	15×60	15×66⅔	20×65	20×65
29	15×42	15×50	15×50	15×60	15×75	20×65	20×75
30	15×42	15×50	15×60	15×66⅔	20×65	20×65	20×75

Deflections not exceeding $\frac{1}{360}$ of the span.

PASSAIC STEEL I BEAMS USED AS FLOOR GIRDERS.

Total uniformly distributed Load, 150 lbs. per square ft.

Span of Girder, in ft.	Number, size and weight of beams required, for each girder, when girders are spaced,										
	10 ft. centers.	12 ft. centers.	14 ft. centers.	16 ft. centers.	18 ft. centers.	20 ft. centers.	22 ft. centers.	24 ft. centers.	26 ft. centers.	28 ft. centers.	30 ft. centers.
	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.
10	1– 8×18	1– 9×21	1– 9×23⅓	1–10×25	1–10×30	1–12×31½	1–12×31½	1–12×31½	1–12×31½	1–12×40	1–12×40
11	1– 9×21	1–10×25	1–10×25	1–10×30	1–12×31½	1–12×31½	1–12×31½	1–12×40	1–12×40	1–12×40	1–15×42
12	1–10×25	1–10×25	1–12×31½	1–12×31½	1–12×31½	1–12×40	1–12×40	1–15×42	1–15×42	1–15×42	1–15×50
13	1–10×25	1–12×31½	1–12×31½	1–12×31½	1–12×40	1–12×40	1–15×42	1–15×42	1–15×50	1–15×50	1–15×50
14	1–10×30	1–12×31½	1–12×40	1–12×40	1–15×42	1–15×42	1–15×50	1–15×50	1–15×50	1–15×60	1–15×60
15	1–12×31½	1–12×31½	1–12×40	1–15×42	1–15×42	1–15×50	1–15×50	1–15×60	1–15×60	1–15×66⅔	1–15×75
16	1–12×31½	1–12×40	1–15×42	1–15×42	1–15×50	1–15×50	1–15×60	1–15×60	1–15×75	2–15×42	2–15×42
17	1–12×40	1–15×42	1–15×42	1–15×50	1–15×60	1–15×60	1–15×66⅔	1–15×75	2–15×42	2–15×42	2–15×50
18	1–12×40	1–15×42	1–15×50	1–15×60	1–15×60	1–15×66⅔	2–15×42	2–15×42	2–15×50	2–15×50	2–15×50
19	1–15×42	1–15×50	1–15×50	1–15×60	1–15×66⅔	2–15×42	2–15×42	2–15×50	2–15×50	2–15×50	2–15×60
20	1–15×42	1–15×50	1–15×60	1–15×66⅔	2–15×42	2–15×42	2–15×50	2–15×50	2–15×60	2–15×60	2–15×60
21	1–15×50	1–15×60	1–15×60	2–15×42	2–15×42	2–15×50	2–15×50	2–15×60	2–15×60	2–15×66⅔	2–15×75
22	1–15×50	1–15×60	1–15×75	2–15×42	2–15×50	2–15×50	2–15×60	2–15×60	2–15×66⅔	2–15×75	2–20×65
23	1–15×60	1–15×66⅔	2–15×42	2–15×50	2–15×50	2–15×60	2–15×60	2–15×66⅔	2–15×75	2–20×65	2–20×65
24	1–15×60	1–15×75	2–15×42	2–15×50	2–15×60	2–15×60	2–15×66⅔	2–15×75	2–20×65	2–20×65	2–20×75
25	1–15×66⅔	2–15×42	2–15×50	2–15×50	2–15×60	2–15×66⅔	2–15×75	2–20×65	2–20×65	2–20×75	2–20×80

Deflections not exceeding $\frac{1}{360}$ of the span.

PASSAIC STEEL I BEAMS

USED AS FLOOR JOISTS.

Total uniformly distributed Load, 175 lbs. per square foot.

Span of Joist, in feet.	Size and Weight of Steel I Beams required for Joists, when Joists are Spaced,						
	4 ft. centers.	5 ft. centers.	6 ft. centers.	7 ft. centers.	8 ft. centers.	9 ft. centers.	10 ft. centers.
	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.
5	4×6	4×6	4× 7½	4× 7½	4×10	5× 9¾	5× 9¾
6	4×7½	4×7½	5× 9¾	5× 9¾	5× 9¾	6×12	6×12
7	5×9¾	5×9¾	5× 9¾	6×12	6×12	6×12	6×15
8	5×9¾	6×12	6×12	6×12	6×15	7×15	7×15
9	6×12	6×12	6×15	7×15	7×15	8×18	8×18
10	6×12	6×15	7×15	8×18	8×18	9×21	9×21
11	6×15	7×15	8×18	8×18	9×21	9×21	9×23⅓
12	7×15	8×18	8×18	9×21	9×21	10×25	10×25
13	8×18	8×18	9×21	9×23⅓	10×25	10×25	10×30
14	8×18	9×21	9×23⅓	10×25	10×30	12×31½	12×31½
15	9×21	9×21	10×25	10×30	12×31½	12×31½	12×31½
16	9×21	10×25	10×30	12×31½	12×31½	12×31½	12×40
17	9×21	10×25	12×31½	12×31½	12×31½	12×40	12×40
18	10×25	10×30	12×31½	12×31½	12×40	12×40	15×42
19	10×25	12×31½	12×31½	12×40	12×40	15×42	15×50
20	10×30	12×31½	12×40	12×40	15×42	15×50	15×50
21	12×31½	12×31½	12×40	15×42	15×42	15×50	15×60
22	12×31½	12×40	12×40	15×42	15×50	15×50	15×60
23	12×31½	12×40	15×42	15×50	15×50	15×60	15×60
24	12×31½	12×40	15×42	15×50	15×60	15×60	15×75
25	12×40	15×42	15×50	15×50	15×60	15×75	20×65
26	15×42	15×42	15×50	15×60	15×66⅔	20×65	20×65
27	15×42	15×50	15×50	15×60	15×75	20×65	20×75
28	15×42	15×50	15×60	15×66⅔	20×65	20×65	20×80
29	15×42	15×50	15×60	15×75	20×65	20×75	20×90
30	15×50	15×60	15×66⅔	20×65	20×75	20×80	20×90

Deflections not exceeding $\frac{1}{360}$ of the span.

PASSAIC STEEL I BEAMS USED AS FLOOR GIRDERS.

Total uniformly distributed Load, 175 lbs. per square ft.

Span of Girder, in ft.	Number, size and weight of beams required, for each girder, when girders are spaced,										
	10 ft. centers.	12 ft. centers.	14 ft. centers.	16 ft. centers.	18 ft. centers.	20 ft. centers.	22 ft. centers.	24 ft. centers.	26 ft. centers.	28 ft. centers.	30 ft. centers.
	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.
10	1–9×21	1–9×23⅓	1–10×25	1–10×30	1–12×31½	1–12×31½	1–12×31½	1–12×40	1–12×40	1–12×40	1–15×42
11	1–9×23⅓	1–10×25	1–12×31½	1–12×31½	1–12×31½	1–12×40	1–12×40	1–12×40	1–15×42	1–15×42	1–15×50
12	1–10×25	1–12×31½	1–12×31½	1–12×31½	1–12×40	1–12×40	1–15×42	1–15×42	1–15×50	1–15×50	1–15×50
13	1–12×31½	1–12×31½	1–12×40	1–12×40	1–15×42	1–15×42	1–15×50	1–15×50	1–15×50	1–15×60	1–15×60
14	1–12×31½	1–12×40	1–12×40	1–15×42	1–15×42	1–15×50	1–15×50	1–15×60	1–15×60	1–15×66⅔	1–15×75
15	1–12×31½	1–12×40	1–15×42	1–15×50	1–15×50	1–15×60	1–15×60	1–15×66⅔	1–15×75	2–15×42	2–15×42
16	1–12×40	1–15×42	1–15×50	1–15×50	1–15×60	1–15×60	1–15×66⅔	2–15×42	2–15×42	2–15×50	2–15×50
17	1–12×40	1–15×42	1–15×50	1–15×60	1–15×60	1–15×75	2–15×42	2–15×42	2–15×50	2–15×50	2–15×50
18	1–15×42	1–15×50	1–15×60	1–15×60	1–15×75	2–15×42	2–15×42	2–15×50	2–15×50	2–15×60	2–15×60
19	1–15×50	1–15×50	1–15×60	1–15×75	2–15×42	2–15×50	2–15×50	2–15×50	2–15×60	2–15×60	2–15×66⅔
20	1–15×50	1–15×60	1–15×66⅔	2–15×42	2–15×50	2–15×50	2–15×50	2–15×60	2–15×60	2–15×66⅔	2–20×65
21	1–15×60	1–15×66⅔	2–15×42	2–15×42	2–15×50	2–15×60	2–15×60	2–15×60	2–15×75	2–20×65	2–20×65
22	1–15×60	1–15×75	2–15×42	2–15×50	2–15×50	2–15×60	2–15×66⅔	2–15×75	2–20×65	2–20×65	2–20×75
23	1–15×66⅔	2–15×42	2–15×50	2–15×50	2–15×60	2–15×66⅔	2–15×75	2–20×65	2–20×65	2–20×75	2–20×80
24	1–15×75	2–15×42	2–15×50	2–15×60	2–15×60	2–15×75	2–20×65	2–20×65	2–20×75	2–20×80	2–20×90
25	2–15×42	2–15×50	2–15×50	2–15×60	2–15×66⅔	2–20×65	2–20×65	2–20×75	2–20×80	2–20×90	2–20×90

Deflections not exceeding $\frac{1}{360}$ of the span.

PASSAIC STEEL I BEAMS

USED AS FLOOR JOISTS.

Total uniformly distributed Load, 200 lbs. per square foot.

Span of Joist, in feet.	Size and Weight of Steel I Beams required for Joists, when Joists are Spaced,						
	4 ft. centers.	**5** ft. centers.	**6** ft. centers.	**7** ft. centers.	**8** ft. centers.	**9** ft. centers.	**10** ft. centers.
	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.	ins. lbs.
5	4×6	4×6	4× 7½	5× 9¾	5× 9¾	5× 9¾	5× 9¾
6	4×7½	4×7½	5× 9¾	5× 9¾	6×12	6×12	6×12
7	5×9¾	5×9¾	6×12	6×12	6×12	6×15	7×15
8	5×9¾	6×12	6×12	6×15	7×15	7×15	8×18
9	6×12	6×15	7×15	7×15	8×18	8×18	9×21
10	6×15	7×15	8×18	8×18	9×21	9×21	9×21
11	7×15	8×18	8×18	9×21	9×21	10×25	10×25
12	7×15	8×18	9×21	9×21	10×25	10×25	10×30
13	8×18	9×21	9×21	10×25	10×30	12×31½	12×31½
14	9×21	9×21	10×25	10×30	12×31½	12×31½	12×31½
15	9×21	10×25	10×30	12×31½	12×31½	12×31½	12×40
16	9×23⅓	10×25	12×31½	12×31½	12×40	12×40	15×42
17	10×25	10×30	12×31½	12×31½	12×40	15×42	15×42
18	10×25	12×31½	12×31½	12×40	15×42	15×42	15×50
19	10×30	12×31½	12×40	12×40	15×42	15×50	15×50
20	12×31½	12×31½	12×40	15×42	15×50	15×50	15×60
21	12×31½	12×40	15×42	15×42	15×50	15×60	15×60
22	12×31½	12×40	15×42	15×50	15×60	15×60	15×66⅔
23	12×40	15×42	15×50	15×50	15×60	15×66⅔	20×65
24	12×40	15×42	15×50	15×60	15×60	15×75	20×65
25	15×42	15×50	15×50	15×60	15×75	20×65	20×75
26	15×42	15×50	15×60	15×66⅔	20×65	20×65	20×75
27	15×42	15×50	15×60	15×75	20×65	20×75	20×80
28	15×50	15×60	15×66⅔	20×65	20×75	20×80	20×90
29	15×50	15×60	15×75	20×65	20×75	20×90	
30	15×50	15×60	20×65	20×75	20×80	20×90	

Deflections not exceeding $\frac{1}{360}$ of the span.

PASSAIC STEEL I BEAMS USED AS FLOOR GIRDERS.

Total uniformly distributed Load, 200 lbs. per square ft.

Span of Girder, in ft.	Number, size and weight of beams required, for each girder, when girders are spaced,										
	10 ft. centers.	12 ft. centers.	14 ft. centers.	16 ft. centers.	18 ft. centers.	20 ft. centers.	22 ft. centers.	24 ft. centers.	26 ft. centers.	28 ft. centers.	30 ft. centers.
	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.	No. ins. lbs.
10	1– 9×21	1–10×25	1–10×30	1–12×31½	1–12×31½	1–12×31½	1–12×40	1–12×40	1–15×42	1–15×42	1–15×42
11	1–10×25	1–10×30	1–12×31½	1–12×31½	1–12×40	1–12×40	1–15×42	1–15×42	1–15×50	1–15×50	1–15×50
12	1–10×30	1–12×31½	1–12×31½	1–12×40	1–15×42	1–15×42	1–15×50	1–15×50	1–15×50	1–15×60	1–15×60
13	1–12×31½	1–12×31½	1–12×40	1–15×42	1–15×42	1–15×50	1–15×50	1–15×60	1–15×60	1–15×66⅔	1–15×75
14	1–12×31½	1–12×40	1–15×42	1–15×50	1–15×50	1–15×60	1–15×60	1–15×66⅔	1–15×75	2–15×42	2–15×42
15	1–12×40	1–15×42	1–15×50	1–15×50	1–15×60	1–15×60	1–15×75	2–15×42	2–15×42	2–15×50	2–15×50
16	1–15×42	1–15×42	1–15×50	1–15×60	1–15×60	1–15×75	2–15×42	2–15×42	2–15×50	2–15×50	2–15×50
17	1–15×42	1–15×50	1–15×60	1–15×60	1–15×75	2–15×42	2–15×50	2–15×50	2–15×50	2–15×60	2–15×60
18	1–15×50	1–15×60	1–15×60	1–15×75	2–15×42	2–15×50	2–15×50	2–15×60	2–15×60	2–15×60	2–15×66⅔
19	1–15×50	1–15×60	1–15×75	2–15×42	2–15×50	2–15×50	2–15×60	2–15×60	2–15×66⅔	2–15×75	2–20×65
20	1–15×60	1–15×66⅔	2–15×42	2–15×50	2–15×50	2–15×60	2–15×60	2–15×66⅔	2–15×75	2–20×65	2–20×65
21	1–15×60	2–15×42	2–15×42	2–15×50	2–15×60	2–15×60	2–15×66⅔	2–20×65	2–20×65	2–20×65	2–20×75
22	1–15×66⅔	2–15×42	2–15×50	2–15×60	2–15×60	2–15×66⅔	2–20×65	2–20×65	2–20×75	2–20×75	2–20×80
23	2–15×42	2–15×50	2–15×50	2–15×60	2–15×66⅔	2–20×65	2–20×65	2–20×75	2–20×80	2–20×90	2–20×90
24	2–15×42	2–15×50	2–15×60	2–15×60	2–15×75	2–20×65	2–20×75	2–20×80	2–20×90	2–20×90	
25	2–15×50	2–15×50	2–15×60	2–15×75	2–20×65	2–20×75	2–20×80	2–20×90	2–20×90		

Deflections not exceeding $\frac{1}{360}$ of the span.

RIVETED GIRDERS.

Riveted girders are used where rolled beams are not sufficiently strong for carrying the load. Riveted girders with single webs, known as plate girders, are more economical than those with double webs, known as box girders; but the latter are stiffer laterally, and should always be used where a great length of span requires a wide top flange for lateral stiffness. If the girder is not held in position laterally, the width of the top flange of the girder should be at least $\frac{1}{20}$ of the span, otherwise the section of the top flange should be increased as follows:

Let A $=$ the gross area required in the top flange, the girder being supported laterally.

A′ $=$ the gross area required in the top flange, the girder being unsupported laterally.

b $=$ length of span $\div$ width of flange, both in inches.

$$\text{Then } A' = A\left(1 + \frac{b^2}{5000}\right)$$

The web of the girder must be made of such a thickness that the vertical shearing strain shall not exceed 7500 lbs. per square inch on a vertical cross section of the web. This shearing strain is greatest at the supports; and, if the load is symmetrically applied, is obtained by dividing one-half the load upon the girder by the area of the vertical cross section of the web. In addition, the web of the girder must either be of sufficient thickness to resist any tendency to buckle, or else it must be stiffened by means of vertical angles riveted to it at intervals not exceeding the depth of the girder. Such stiffeners must be used when the shearing strain, per square inch, exceeds the strain allowed by the formula:

$$\text{Allowable shearing strain per square inch} = \frac{12000}{1 + \dfrac{h^2}{3000\,t^2}}$$

in which "h" represents depth of the web between flanges of girder, and "t" the thickness of one web plate, both in inches. The stiffeners should always reach over the vertical sides of the angles forming the chords of the girder, and there should be filling pieces between the stiffening angles and the web plate. In every case, whether intermediate stiffeners are used or not, the web at the ends of the girder, where it rests upon supports, should be reinforced by stiffeners so that the reaction of the support may be resisted by an increased section. These end stiffeners should be considered as columns taking the entire load upon the support and transferring it to the web of the girder; and should have sufficient rivets connecting them to the web of the girder to transmit the total reaction at the support. The strain upon the end stiffeners should not exceed 15,000 lbs. per square inch of cross section. Stiffeners should always be used at any point where there is concentration of heavy loads; the duty of the stiffeners in such cases is to prevent buckling of the web, and to transmit the load to the web by means of the abutting areas and the rivets, both of which must be sufficient for the purpose.

The rivets used should generally be $\frac{3}{4}''$ or $\frac{7}{8}''$ diameter, the latter size being preferable and often necessary where girders are to carry heavy loads. Rivets should never be spaced exceeding six inches centers; but in all cases the pitch of the rivets must be closer at the ends of the girder. At any point of the girder there must be sufficient rivets connecting the web to each flange, in a length of flange equal to the depth of the girder, to transmit the total shear at that point. At the end of the girder there must be sufficient rivets connecting the web to each flange, in a length equal to the depth of the girder, to transmit the end reaction of the girder. In the calculation of rivet spacing for girders used in buildings it is customary to allow 9,000 lbs. per square inch for shearing and 18,000 lbs. per square inch for bearing on the rivets. In plate girders the rivet pitch will usually be determined by the bearing value of the rivets, and in box girders by the shearing value of the rivets. The shearing and bearing values of rivets, for use in building construction, are given on pages 220–221.

Plate girders should never be made too shallow, on account of the deflection; they should have a depth of not less than one-twentieth of the clear span; if built shallower, more material must be put in the flanges so as to reduce the strain per square inch, and the deflection in proportion.

The flange of a riveted girder comprises all the metal at the top or the bottom of the girder. It is customary in building construction to consider $\frac{1}{6}$ of the area of the web plate as available for flange section, in which case care should be taken to avoid splicing the web plate at or near the center of the girder; if this is observed, it is proper to consider $\frac{1}{6}$ of the web as a part of each flange. If a pair of angle irons does not provide sufficient area for the flange, it is customary to use flange plates to make up the required area. Where flange plates are used, the angles should comprise one-half of the flange section, but in heavy flanges where this is impossible, the flange angles should be the heaviest sections rolled. The unsupported width of a flange plate, subjected to compression, should not exceed thirty-two times its thickness, nor should the flange plate extend beyond the outer line of rivets more than five inches, nor more than eight times its thickness.

It is customary in building construction to allow a strain of 15,000 lbs. per square inch on the net section of the bottom or tension flange. Care must be observed to deduct all the area lost by rivet holes, and the rivets should be arranged in the flanges of the girder to make this reduction of area as small as possible. In deducting area lost by rivet holes, the diameter of the holes should be taken $\frac{1}{8}$ inch greater than the rivets, to compensate for injury done the metal by punching. The top or compression flange of the girder is usually made of the same gross area as the bottom or tension flange.

DESIGN OF A RIVETED GIRDER.

Box girder, to carry a wall 20 inches wide.

Span, 30 feet between centers of supports = 360 inches.

Total weight to be carried, 200 tons = 400,000 lbs.

Depth available, 36 inches over all.

Load on each support, $\frac{1}{2} \times 400{,}000 = 200{,}000$.

Web section required, $200{,}000 \div 7{,}500 = 26.66$ sq. ins.

Two web plates, $33\frac{1}{2}'' \times \frac{7}{16}'' = 29.3$ sq. ins.

Bending moment at center of span,

$\frac{1}{8} \times 400{,}000 \times 360 = 18{,}000{,}000$ inch lbs.

Depth of girder, center of gravity of flanges, 33 inches.

Maximum flange strain, $18{,}000{,}000 \div 33 = 545{,}450$ lbs.

Net flange area required, $545{,}450 \div 15{,}000 = 36.4$ sq. ins.

This section is made up as follows:

	Gross.	Net.
$\frac{1}{6}$ of area of web	4.88 sq. ins.	4.88 sq. ins.
2 angles, 6″ × 4″ × $\frac{11}{16}$″	12.96 "	11.58 "
2 plates, 20″ × $\frac{9}{16}$″	22.50 "	20.25 "
		36.71 "

In obtaining the above net area of the flange, one rivet hole has been deducted from the area of each angle, and two rivet holes from the area of each cover plate. This deduction is made upon the assumption that the rivets connecting the angles to the web plates are arranged to stagger with the rivets connecting the angles to the flange plates. It is, generally, possible to effect such an arrangement of rivets for a considerable length at the center of the span. If such an arrangement of rivets is not possible, then two rivet holes should be deducted from the area of each angle, and $\frac{1}{6}$ the gross area of the web should be reduced by the area lost for a rivet hole at the extreme edge of the web connecting it to the flange. If a stiffener is used at or near the center of the span, the net area of the web plate available for flange section should be taken at $\frac{1}{8}$ the gross area of the web.

The end reaction of 200,000 lbs. on this girder requires 37 rivets, $\frac{7}{8}$″ diameter, in single shear to transmit it to either flange in a length equal to the depth of the girder. The depth of the girder for this purpose is taken as the depth, center to center of gravity of flanges; there being two lines of rivets, one line connecting each web to the flange, the rivets will require to be spaced $1\frac{3}{4}$″ pitch at the end of the girder. This requires an angle having a 6″ leg against the web.

The area required for the stiffeners over the supports is 200,000 lbs. ÷ 15,000 = 13.33 square inches. Four angles, $3\frac{1}{2} \times 3\frac{1}{2}$″ × $\frac{1}{2}$″, provide an area of 13 square inches, and are sufficient for the purpose at each end of the girder.

Applying the formula already given for the allowable shearing strain in the web, it will be found that 6,500 lbs. per square inch is the maximum allowable shearing strain, unless the webs are stiffened. Stiffeners of $3\frac{1}{2}$″ × $3\frac{1}{2}$″ × $\frac{3}{8}$″ angles will, therefore, be required for a short distance near each support where the shearing strain exceeds 6,500 lbs. per square inch.

As the bending moment is greatest at the center of the span and diminishes to zero at the supports, it is unnecessary to have the full flange section the whole length of the girder; and, in the present case, one of the two flange plates can be stopped off, short of the supports, without affecting the strength of the girder.

Let A = total net flange area of girder.

A'' = total net area of that portion of the flange which is to be stopped off.

L = length of girder, centers of supports, in feet.

L' = required length, symmetrically arranged about the center of span, of that portion of the flange which is to be stopped off, in feet.

$$\text{Then } L' = 2 + L\sqrt{\frac{A''}{A}}$$

In the present instance

$$L' = 2 + 30\sqrt{\frac{10.12}{36.71}} = 17.7$$

so that the outer flange plates need only be 17¾ feet long, placed symmetrically about the center of the span.

This girder is illustrated on page 37.

The following table furnishes a convenient means for finding the net area required in the flange of riveted girders when the load, span, and depth are given.

To obtain the net flange area required, multiply the coefficient given in the table for the given span and depth by the uniformly distributed load in tons of 2,000 lbs. The result will be the net area in square inches required for each flange allowing a maximum fiber strain of 15,000 lbs. per square inch of net area. To illustrate the application of this table, take the box girder already proportioned in detail. By reference to the table, the coefficient for a span of 30 feet and depth of 32 inches is 0.187, and the coefficient for the same span with a depth of 34 inches is 0.177. The coefficient for a depth of 33 inches will be the mean of these two values, or 0.182; and multiplying this by the load, 200 tons, gives 36.4 as the number of square inches of net area required in the flange. This is the same result as that obtained by the extended calculations already illustrated.

RIVETED GIRDERS.

Multiply the coefficient given in the table by the uniformly distributed load, in tons of 2000 lbs. The result will be the net area, in square inches, required for each flange, allowing a maximum fiber strain of 15,000 lbs. per square inch of net area.

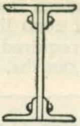
Span, in Feet.	Depth, Center to Center of Gravity of Flanges, in Inches.										
	22	24	26	28	30	32	34	36	38	40	42
10	.091	.083	.077	.071	.067	.063	.059	.055	.053	.050	.047
11	.100	.092	.085	.079	.073	.069	.065	.061	.058	.055	.053
12	.109	.100	.092	.086	.080	.075	.071	.067	.063	.060	.057
13	.118	.109	.100	.093	.087	.081	.077	.072	.068	.065	.062
14	.127	.117	.108	.100	.093	.087	.083	.078	.073	.070	.067
15	.137	.125	.115	.107	.100	.094	.088	.083	.079	.075	.071
16	.145	.133	.123	.114	.107	.100	.094	.089	.084	.080	.076
17	.155	.142	.131	.121	.113	.106	.100	.095	.089	.085	.081
18	.163	.150	.139	.129	.120	.113	.106	.100	.095	.090	.086
19	.173	.159	.146	.136	.127	.119	.112	.105	.100	.095	.091
20	.182	.167	.154	.143	.133	.125	.117	.111	.105	.100	.095
21	.191	.175	.161	.150	.140	.131	.123	.117	.110	.105	.100
22	.200	.183	.169	.157	.147	.137	.129	.122	.115	.110	.105
23	.209	.192	.177	.164	.153	.144	.135	.128	.121	.115	.109
24	.218	.200	.185	.171	.160	.150	.141	.133	.126	.120	.114
25	.227	.209	.192	.179	.167	.156	.147	.139	.131	.125	.119
26	.237	.217	.200	.186	.173	.163	.153	.145	.137	.130	.124
27	.245	.225	.208	.193	.180	.169	.159	.150	.142	.135	.129
28	.255	.233	.215	.200	.187	.175	.165	.155	.147	.140	.133
29	.263	.242	.223	.207	.193	.181	.171	.161	.153	.145	.138
30	.273	.250	.231	.214	.200	.187	.177	.167	.157	.150	.143
31	.282	.259	.239	.221	.207	.194	.183	.172	.163	.155	.147
32	.291	.267	.246	.229	.213	.200	.188	.178	.168	.160	.152
33	.300	.275	.254	.236	.220	.206	.194	.183	.173	.165	.157
34	.309	.283	.261	.243	.227	.213	.200	.189	.179	.170	.162
35	.318	.292	.269	.250	.233	.219	.206	.195	.184	.175	.167
36	.327	.300	.277	.257	.240	.225	.212	.200	.189	.180	.171
37	.337	.309	.285	.264	.247	.231	.217	.205	.195	.185	.176
38	.345	.317	.292	.271	.253	.237	.223	.211	.199	.190	.181
39	.355	.325	.300	.279	.260	.244	.229	.217	.205	.195	.185
40	.364	.333	.307	.286	.267	.250	.235	.222	.210	.200	.191

If the section of a girder is given, the safe uniformly distributed load (in tons of 2000 lbs.) can be obtained by dividing the net area of the flange by the coefficient given in the table.

STEEL PLATE GIRDERS.

SAFE LOADS, IN TONS OF 2000 LBS., UNIFORMLY DISTRIBUTED.

No stiffeners required except at ends, over supports only.



Girders equivalent to a 24″ I beam.

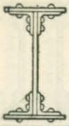
Web.	24″ × 3/8″		26″ × 3/8″		28″ × 3/8″		30″ × 3/8″	
Angles.	5″ × 3½″ × ½″		5″ × 3½″ × 7/16″		5″ × 3½″ × 3/8″		5″ × 3″ × 3/8″	
Span, Centers of Bearings, Feet.	Safe Load, Tons.	Increase for 1/16″ Increase in Thickness of Angles.	Safe Load, Tons.	Increase for 1/16″ Increase in Thickness of Angles.	Safe Load, Tons.	Increase for 1/16″ Increase in Thickness of Angles.	Safe Load, Tons.	Increase for 1/16″ Increase in Thickness of Angles.
20	47.2	5.3	46.5	5.8	45.1	6.2	47.7	6.4
21	44.9	5.0	44.3	5.5	42.9	5.9	45.5	6.1
22	42.9	4.8	42.3	5.2	41.0	5.7	43.4	5.8
23	41.0	4.6	40.4	5.0	39.2	5.4	41.5	5.5
24	39.3	4.4	38.8	4.8	37.6	5.2	39.8	5.3
25	37.7	4.2	37.2	4.6	36.1	5.0	38.2	5.1
26	36.3	4.1	35.8	4.4	34.7	4.8	36.7	4.9
27	34.9	3.9	34.4	4.3	33.4	4.6	35.4	4.7
28	33.7	3.8	33.2	4.1	32.2	4.5	34.1	4.5
29	32.5	3.6	32.1	4.0	31.1	4.3	32.9	4.4
30	31.4	3.5	31.0	3.8	30.0	4.2	31.8	4.2
31	30.4	3.4	30.0	3.7	29.1	4.0	30.8	4.1
32	29.4	3.3	29.1	3.6	28.2	3.9	29.8	4.0
33	28.6	3.2	28.2	3.5	27.3	3.8	28.9	3.9
34	27.7	3.1	27.4	3.4	26.5	3.7	28.1	3.7
35	26.9	3.0	26.6	3.3	25.8	3.6	27.3	3.6
36	26.2	2.9	25.8	3.2	25.0	3.5	26.5	3.5
37	25.5	2.8	25.1	3.1	24.4	3.4	25.8	3.4
38	24.8	2.8	24.5	3.0	23.7	3.3	25.1	3.3
39	24.2	2.7	23.8	2.9	23.1	3.2	24.5	3.3
40	23.6	2.6	23.3	2.9	22.5	3.1	23.9	3.2
Wgt. per ft., lbs.	88	7.2	84	7.2	79	7.2	79	6.8

Safe loads given include weight of girder.
Weights of girders given include weight of rivet heads, but not stiffeners.
Maximum fiber strain, 15,000 lbs. per square inch of net area, holes for 3/4″ rivets being deducted.

STEEL PLATE GIRDERS.

SAFE LOADS, IN TONS OF 2000 LBS., UNIFORMLY DISTRIBUTED.

No stiffeners required except at ends, over supports only.



Girders equivalent to two 24″ I Beams.

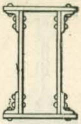
Web.	$24'' \times \frac{9}{16}''$		$26'' \times \frac{9}{16}''$		$28'' \times \frac{1}{2}''$		$30'' \times \frac{1}{2}''$	
Angles.	$5'' \times 5'' \times \frac{1}{2}''$		$5'' \times 5'' \times \frac{7}{16}''$		$5'' \times 5'' \times \frac{3}{8}''$		$5'' \times 5'' \times \frac{3}{8}''$	
Plates.	$12'' \times \frac{1}{2}''$		$12'' \times \frac{1}{2}''$		$12'' \times \frac{1}{2}''$		$12'' \times \frac{3}{8}''$	
Span, Centers of Bearings, Feet.	Safe Load, Tons.	Increase for $\frac{1}{16}''$ Increase in Thickness of Plates.	Safe Load, Tons.	Increase for $\frac{1}{16}''$ Increase in Thickness of Plates.	Safe Load, Tons.	Increase for $\frac{1}{16}''$ Increase in Thickness of Plates.	Safe Load, Tons.	Increase for $\frac{1}{16}''$ Increase in Thickness of Plates.
20	90.8	3.6	93.6	3.9	93.6	4.3	91.7	4.6
21	86.5	3.4	89.1	3.7	89.1	4.1	87.3	4.3
22	82.5	3.3	85.1	3.6	85.0	3.9	83.4	4.1
23	78.9	3.1	81.3	3.4	81.3	3.7	79.7	3.9
24	75.6	3.0	78.0	3.3	78.0	3.6	76.4	3.8
25	72.6	2.9	74.8	3.1	74.8	3.4	73.3	3.6
26	69.8	2.8	72.0	3.0	72.0	3.3	70.5	3.5
27	67.2	2.7	69.3	2.9	69.3	3.2	67.9	3.4
28	64.8	2.6	66.8	2.8	66.8	3.1	65.5	3.3
29	62.6	2.5	64.5	2.7	64.5	3.0	63.2	3.1
30	60.5	2.4	62.4	2.6	62.4	2.9	61.1	3.0
31	58.6	2.3	60.4	2.5	60.4	2.8	59.2	2.9
32	56.7	2.2	58.5	2.5	58.5	2.7	57.3	2.8
33	55.0	2.2	56.7	2.4	56.7	2.6	55.6	2.8
34	53.4	2.1	55.0	2.3	55.0	2.5	53.9	2.7
35	51.9	2.0	53.5	2.3	53.5	2.4	52.4	2.6
36	50.4	2.0	52.0	2.2	52.0	2.4	50.9	2.5
37	49.1	1.9	50.6	2.1	50.6	2.3	49.6	2.5
38	47.8	1.9	49.2	2.1	49.2	2.3	48.3	2.4
39	46.6	1.8	48.0	2.0	48.0	2.2	47.0	2.3
40	45.4	1.8	46.8	2.0	46.8	2.1	45.8	2.3
Wgt. per ft., lbs.	158	5.1	153	5.1	143	5.1	136	5.1

Safe loads given include weight of girder.
Weights of girders given include weight of rivet heads, but not stiffeners.
Maximum fiber strain, 15,000 lbs. per square inch of net area, holes for $\frac{3}{4}''$ rivets being deducted.

STEEL BOX GIRDERS.

SAFE LOADS, IN TONS OF 2000 LBS., UNIFORMLY DISTRIBUTED.

No stiffeners required except at ends, over supports only.



Girders equivalent to two 24″ I beams.

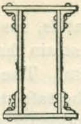
Webs.	$24'' \times \frac{3}{8}''$		$26'' \times \frac{3}{8}''$		$28'' \times \frac{3}{8}''$		$30'' \times \frac{3}{8}''$	
Angles.	$5'' \times 3'' \times \frac{1}{2}''$		$5'' \times 3'' \times \frac{7}{16}''$		$5'' \times 3'' \times \frac{3}{8}''$		$5'' \times 3'' \times \frac{3}{8}''$	
Plates.	$14'' \times \frac{9}{16}''$		$14'' \times \frac{1}{2}''$		$14'' \times \frac{7}{16}''$		$14'' \times \frac{3}{8}''$	
Span, Centers of Bearings, Feet.	Safe Load, Tons.	Increase for $\frac{1}{16}''$ Increase in Thickness of Plates.	Safe Load, Tons.	Increase for $\frac{1}{16}''$ Increase in Thickness of Plates.	Safe Load, Tons.	Increase for $\frac{1}{16}''$ Increase in Thickness of Plates.	Safe Load, Tons.	Increase for $\frac{1}{16}''$ Increase in Thickness of Plates.
20	93.8	4.3	93.5	4.7	92.9	5.1	95.6	5.4
21	89.3	4.1	89.0	4.5	88.5	4.8	91.1	5.2
22	85.3	3.9	85.0	4.3	84.5	4.6	86.9	4.9
23	81.6	3.8	81.3	4.1	80.8	4.4	83.2	4.7
24	78.2	3.6	77.9	3.9	77.4	4.2	79.7	4.5
25	75.0	3.5	74.8	3.8	74.3	4.1	76.5	4.3
26	72.2	3.3	71.9	3.6	71.5	3.9	73.6	4.2
27	69.5	3.2	69.2	3.5	68.8	3.8	70.8	4.0
28	67.1	3.1	66.8	3.4	66.3	3.6	68.3	3.9
29	64.7	3.0	64.4	3.2	64.0	3.5	66.0	3.7
30	62.5	2.9	62.3	3.1	61.9	3.4	63.8	3.6
31	60.5	2.8	60.3	3.0	60.0	3.3	61.7	3.5
32	58.6	2.7	58.4	2.9	58.1	3.2	59.8	3.4
33	56.9	2.6	56.6	2.8	56.3	3.1	58.0	3.3
34	55.2	2.5	55.0	2.7	54.6	3.0	56.3	3.2
35	53.6	2.5	53.4	2.7	53.1	2.9	54.7	3.1
36	52.1	2.4	51.9	2.6	51.6	2.8	53.1	3.0
37	50.7	2.3	50.5	2.5	50.2	2.7	51.7	2.9
38	49.4	2.3	49.2	2.5	48.9	2.7	50.3	2.9
39	48.1	2.2	47.9	2.4	47.6	2.6	49.0	2.8
40	46.9	2.2	46.7	2.4	46.4	2.6	48.0	2.8
Wgt. per ft., lbs.	174	6.0	166	6.0	159	6.0	158	6.0

Safe loads given include weight of girder.
Weights of girders given include weight of rivet heads, but not stiffeners.
Maximum fiber strain, 15,000 lbs. per square inch of net area, holes for $\frac{3}{4}''$ rivets being deducted.

STEEL BOX GIRDERS.

SAFE LOADS, IN TONS OF 2000 LBS., UNIFORMLY DISTRIBUTED.

No stiffeners required except at ends, over supports only.



Girders equivalent to a 24″ Beam Box Girder.

Webs.	$24'' \times \frac{3}{8}''$		$26'' \times \frac{3}{8}''$		$28'' \times \frac{3}{8}''$		$30'' \times \frac{3}{8}''$	
Angles.	$5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$		$5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$		$5'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$		$5'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$	
Plates.	$18'' \times \frac{3}{4}''$		$18'' \times \frac{5}{8}''$		$18'' \times \frac{9}{16}''$		$18'' \times \frac{1}{2}''$	
Span, Centers of Bearings, Feet.	Safe Load, Tons.	Increase for $\frac{1}{16}''$ Increase in Thickness of Plates.	Safe Load, Tons.	Increase for $\frac{1}{16}''$ Increase in Thickness of Plates.	Safe Load, Tons.	Increase for $\frac{1}{16}''$ Increase in Thickness of Plates.	Safe Load, Tons.	Increase for $\frac{1}{16}''$ Increase in Thickness of Plates.
20	130.7	5.9	129.5	6.3	128.4	6.8	131.7	7.3
21	124.5	5.6	123.3	6.0	122.3	6.5	125.4	7.0
22	118.8	5.4	117.7	5.8	116.8	6.2	119.7	6.7
23	113.6	5.1	112.6	5.5	111.7	6.0	114.5	6.4
24	108.9	4.9	107.9	5.3	107.0	5.7	109.7	6.1
25	104.5	4.7	103.6	5.1	102.8	5.5	105.3	5.9
26	100.5	4.5	99.6	4.9	98.8	5.3	101.3	5.6
27	96.8	4.4	95.9	4.7	95.1	5.1	97.5	5.4
28	93.3	4.2	92.5	4.5	91.7	4.9	94.1	5.2
29	90.1	4.1	89.3	4.4	88.6	4.7	90.8	5.1
30	87.1	3.9	86.3	4.2	85.6	4.6	87.8	4.9
31	84.3	3.8	83.5	4.1	82.9	4.4	85.0	4.7
32	81.7	3.7	80.9	4.0	80.3	4.3	82.4	4.6
33	79.2	3.6	78.5	3.8	77.8	4.1	79.8	4.4
34	76.9	3.5	76.2	3.7	75.6	4.0	77.5	4.3
35	74.7	3.4	74.0	3.6	73.4	3.9	75.2	4.2
36	72.6	3.3	71.9	3.5	71.4	3.8	73.2	4.1
37	70.6	3.2	70.0	3.4	69.4	3.7	71.2	4.0
38	68.8	3.1	68.1	3.3	67.6	3.6	69.3	3.9
39	67.0	3.0	66.4	3.3	65.9	3.5	67.5	3.8
40	65.3	2.9	64.7	3.2	64.2	3.4	65.8	3.7
Wgt. per ft., lbs.	216	7.7	206	7.7	196	7.7	193	7.7

Safe loads given include weight of girder.
Weights of girders given include weight of rivet heads, but not stiffeners.
Maximum fiber strain, 15,000 lbs. per square inch of net area, holes for $\frac{3}{4}''$ rivets being deducted.

SUDDENLY APPLIED LOADS.

If a load is suddenly, that is, instantaneously, applied to a beam, it produces twice the strain that the same load would produce if at rest upon the beam. The safe suddenly applied load is, therefore, only one-half the safe static load.

If the load is not only suddenly applied, but falls upon the beam from a height, it produces more than twice the strain that the same load statically applied would produce.

Let P = the weight that falls upon the beam.

h = height of fall, in inches.

P′ = equivalent static load producing the same strain as that produced by the falling weight.

d = deflection of beam, in inches, produced by the weight, P, if statically applied.

B = the weight of the beam together with its superimposed dead load, such as arches and flooring, whose combined mass tends to absorb the impact.

Then, if $m = \dfrac{1}{1 + \dfrac{17\,B}{35\,P}}$,

$$P' = P\left(1 + \sqrt{\frac{2\,m\,h}{d} + 1}\right)$$

From which the equivalent static load, P′, is obtained, and the strain can then be computed in the ordinary manner.

The uniformly distributed static load, equivalent to the falling weight, can be obtained in the following manner: —

Let W′ = equivalent uniformly distributed load.

W = safe uniformly distributed load on beam, from the tables.

D = deflection, in inches, under safe uniformly distributed load.

$$\text{Then, } W' = 2\,P\left(1 + \sqrt{\frac{5\,W\,h\,m}{4\,P\,D} + 1}\right)$$

In applying these formulæ P′ and W′ will be in tons or pounds according as the weights are taken in tons or pounds.

LINTELS.

Lintels of steel shapes or of cast iron are employed to span openings in walls over doors and windows. It is generally necessary that the lintels should have a flat soffit. Where the load to be carried is small, steel channels, laid flat, furnish a very satisfactory lintel on moderate spans. The table on page 122 gives the safe uniformly distributed loads, in tons of 2,000 lbs., for Passaic steel channels used as lintels, by which the channel required for any given span and load may be easily selected.

Sometimes the load to be carried by a lintel consists of a uniformly distributed load from the wall above and also the concentration from a floor joist which rests upon the wall at or near the center of the span. In such instances, the concentrated load must be multiplied by 2, the result being considered as an equivalent uniform load, which, added to the regular distributed load, may be taken as the equivalent total uniformly distributed load. Thus, if a lintel spanning an opening of 4 ft. is to carry a uniformly distributed load of 2 tons and a concentrated load of 2 tons at the center of the span, the concentrated load multiplied by 2 and added to the distributed load gives 6 tons as the equivalent distributed load. By referring to the table, it will be found that a 15″ × 50 lb. steel channel, which has a safe load of 6.28 tons, is required.

Where the loads are considerable and the use of beam girders is not advisable, cast iron lintels are used. The table on page 123 gives the coefficients of strength, in tons of 2,000 lbs., for cast iron lintels, by which the safe uniformly distributed loads, in tons, for any given span may be found by dividing the coefficient given by the span in ft. Thus, if it is required to find the safe uniformly distributed load on a cast iron lintel, 12″ wide, 10″ deep and 1″ metal, on a span of 6 ft., by referring to the table, the coefficient of strength given for this lintel is 72.2 tons, which divided by the span gives the safe load as 12.03 tons.

If a part of the load is concentrated, it must first be multiplied by 2, and the result considered as the equivalent uniform load. The proper lintel required for any given span and load may be found by multiplying the equivalent uniform load, in tons, by the span, in feet, the result being the coefficient required; then, by reference to the table, the lintel, having the required coefficient of strength, can be easily selected. Thus, if it is required to select a lintel carrying a 20″ wall on a span of 8 ft. to support a uniformly distributed load of 5 tons, and a concentrated load of 5 tons at the center, the method is as follows. The concentrated load must first be reduced to an equivalent uniform load by multiplying it by 2, and added to the regular uniform load, giving 15 tons as the equivalent uniform load on the span which, multiplied by the span in feet, gives the coefficient required as 120 tons. Then, referring to the table it will be found that a lintel, 20″ wide, 10″ deep and 1″ metal, which has a coefficient of 125.4 tons, will be required.

SAFE LOADS, UNIFORMLY DISTRIBUTED, FOR PASSAIC STEEL CHANNELS,

IN TONS OF 2000 LBS.,

WEB HORIZONTAL.

Safe loads given, include weight of channel.

Depth of Channel, ins.	Wgt. per ft., lbs.	Coefficient of strength, in tons.	Span in feet. 2	3	4	5	6	7	8	9	10	Deflection Coefficient.
15	50	25.1	12.6	8.37	6.28	5.02	4.18	3.59	3.14	2.79	2.51	.0028
15	40	22.4	11.2	7.47	5.60	4.48	3.73	3.20	2.80	2.49	2.24	.0030
15	33	16.3	8.20	5.43	4.08	3.26	2.71	2.33	2.04	1.81	1.63	.0032
12	35	15.0	7.50	5.00	3.75	3.00	2.50	2.14	1.88	1.67	1.50	.0032
12	27	12.9	6.45	4.30	3.23	2.58	2.15	1.84	1.61	1.43	1.29	.0035
12	20	8.97	4.49	2.99	2.24	1.79	1.50	1.28	1.12	1.00	.90	.0038
10	30	11.7	5.85	3.90	2.93	2.34	1.95	1.67	1.46	1.30	1.17	.0034
10	20	9.33	4.67	3.11	2.33	1.87	1.56	1.33	1.17	1.04	.93	.0039
10	15	6.66	3.33	2.22	1.67	1.33	1.11	.95	.83	.74	.67	.0041
9	21	8.21	4.11	2.74	2.05	1.64	1.37	1.17	1.03	.91	.82	.0040
9	16	7.25	3.63	2.42	1.81	1.45	1.21	1.04	.91	.81	.73	.0044
9	13	4.90	2.45	1.63	1.23	.98	.82	.70	.61	.54	.49	.0046
8	17	5.50	2.75	1.83	1.38	1.10	.93	.79	.69	.61	.55	.0046
8	13	4.80	2.40	1.60	1.20	.96	.80	.69	.60	.53	.48	.0051
8	10	3.41	1.71	1.14	.85	.68	.57	.49	.43	.38	.34	.0053
7	17	7.04	3.52	2.35	1.76	1.41	1.17	1.01	.88	.78	.70	.0047
7	13	6.30	3.15	2.10	1.56	1.26	1.05	.90	.79	.70	.63	.0052
7	9	2.94	1.47	.98	.74	.59	.49	.42	.37	.33	.29	.0056
6	20	8.91	4.46	2.97	2.23	1.78	1.49	1.27	1.11	.99	.89	.0047
6	17	7.84	3.92	2.61	1.96	1.57	1.31	1.12	.98	.87	.78	.0051
6	12	4.80	2.40	1.60	1.20	.96	.80	.69	.60	.53	.48	0054
6	8	2.67	1.34	.89	.67	.53	.45	.38	.33	.30	.27	.0058
5	12	3.89	1.95	1.30	.97	.78	.65	.56	.49	.43	.39	.0055
5	9	3.20	1.60	1.07	.80	.64	.53	.46	.40	.36	.32	.0062
5	6	1.71	.86	.57	.43	.34	.29	.24	.21	.19	.17	.0068
4	10	3.36	1.68	1.12	.84	.67	.56	.48	.42	.37	.34	.0059
4	8	2.88	1.44	.96	.72	.58	.48	.41	.36	.32	.29	.0065
4	5	1.39	.70	.46	.35	.28	.23	.20	.17	.15	.14	.0073

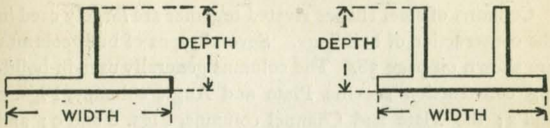
Safe loads, uniformly distributed, in tons of 2,000 lbs., for intermediate spans can be obtained by dividing the Coefficient of Strength by the span, in feet. Deflection, in inches, under tabular load, can be obtained by multiplying the Deflection Coefficient by the square of the span, in feet.

W = Safe Load in Tons of 2240 lbs. distributed
l = Span in inches. d = Extreme depth in inches
a = Area of bottom flange.

$W = \frac{52\,ad}{5l}$ (see Birkmire page 32)

COEFFICIENTS OF STRENGTH FOR CAST IRON LINTELS,

IN TONS OF 2000 LBS.



SINGLE WEB LINTEL. DOUBLE WEB LINTEL.

Width of flange, Ins.	Depth of lintel, Ins.	Thickness of metal, in inches. $\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	No. of Webs.
28	6	59.5	64.9	69.8	74.6	77.8	2
"	8	95.5	106.2	115.0	123.0	130.5	2
"	10	140.5	150.5	164.8	176.2	192.0	2
"	12	171.4	196.5	216.1	236.3	256.7	2
"	16	235.8	272.5	307.4	342.0	375.0	2
24	6	52.8	57.4	62.6	66.6	70.0	2
"	8	83.4	93.4	102.4	109.6	117.0	2
"	10	116.0	130.4	144.4	156.2	167.6	2
"	12	150.4	168.6	189.6	207.0	223.0	2
"	16	225.0	257.0	286.0	316.5	345.0	2
20	6	47.2	51.4	55.1	58.5	62.0	2
"	8	72.6	84.7	89.5	96.0	102.5	2
"	10	100.5	113.2	125.4	136.0	146.8	2
"	12	122.6	141.8	158.0	174.7	189.5	2
"	16	196.4	224.7	251.4	277.2	301.5	2
16	6	33.0	35.1	37.7	40.3	41.8	1
"	8	52.1	57.7	62.8	67.2	71.6	1
"	10	72.2	81.2	89.6	96.8	104.0	1
"	12	92.4	106.1	117.5	128.8	138.8	1
"	16	139.4	159.0	177.8	196.0	214.0	1
12	6	26.4	28.7	31.3	33.3	35.0	1
"	8	41.7	46.7	51.2	54.8	58.5	1
"	10	58.0	65.2	72.2	78.1	83.8	1
"	12	75.2	84.3	94.8	103.5	111.5	1
8	6	19.7	21.7	23.4	24.9	26.4	1
"	8	30.6	34.4	37.7	40.7	43.3	1
"	10	42.6	48.1	53.0	57.8	62.9	1
"	12	55.4	62.4	70.0	76.7	83.5	1

Coefficients are calculated for a maximum tensile strain of 3,000 lbs. per square inch. The safe uniformly distributed load, in tons, for any given span may be found by dividing the coefficient, as above, by the span in feet.

COLUMNS.

Columns of steel shapes riveted together are largely used in the construction of buildings. Several types of built columns are shown on page 38. The columns generally used in building construction are the Plate and Angle columns, Figs. 2 and 3; the Plate and Channel columns, Figs. 8 and 9; and the **Z**-Bar columns, Figs. 11 and 12. Where these do not furnish sufficient section for carrying the loads, the column shown in Fig. 5 can be advantageously used and made large enough for very heavy loads by increasing the thickness of the material. The manner of connecting the segments of the columns together, and the mode of attaching beams and girders is illustrated on page 39. Abutting segments of columns should be thoroughly connected in a manner to preserve the continuity of strength, thus adding to the stiffness of the steel frame work.

The strength of a column depends upon its shape and length. Long columns have less strength than shorter columns of the same size for the reason that they are liable to fail by lateral flexure, and of two columns having the same area and length, the one in which the material is placed at a greater distance from the center will develop greater strength. If all the material in the cross section were concentrated at a distance from the neutral axis equal to the radius of gyration, the resistance to flexure would be the same as for the material distributed over the cross section. Formulæ for the strength of columns therefore take into consideration the length of the column and the radius of gyration of the section. The manner of securing the ends of the columns also has an appreciable effect upon their strength. Columns fixed so firmly at the ends that they are liable to fail in the body of the column before rupturing their end connections develop greater strength than columns connected by means of pins through the ends. Columns with square ends develop less ultimate strength than if the ends are firmly fixed, but greater than if the ends are pin connected. Medium steel columns develop practically a uniform strength for all lengths up to 50 radii of gyration, and soft steel columns develop practically a uniform strength for all lengths up to 30 radii of gyration, the ultimate for both grades of steel being about 48,000 lbs. per sq. in., up to the lengths indicated.

The following straight-line formulæ represent very closely the ultimate strength, in lbs. per sq. in., of columns whose lengths are between 50 and 150 radii of gyration,

	Medium Steel.	Soft Steel.
Fixed Ends,	$60{,}000 - 210\frac{l}{r}$	$54{,}000 - 185\frac{l}{r}$
Square Ends,	$60{,}000 - 230\frac{l}{r}$	$54{,}000 - 200\frac{l}{r}$
Pin Ends,	$60{,}000 - 260\frac{l}{r}$	$54{,}000 - 225\frac{l}{r}$

where l = length of column, and r = least radius of gyration, both in inches. Columns used in building construction may be considered as having square ends, as pin connections are seldom used; and as it is usual to allow a factor of safety of 4 for such columns, the following formulæ may, therefore, be taken as giving the allowable strain, in lbs. per sq. in., on square ended columns for building construction.

$$\text{Medium Steel}\begin{cases}12{,}000 \text{ for lengths up to 50 radii of gyration.}\\ 15{,}000 - 57\frac{l}{r} \text{ for lengths over 50 radii.}\end{cases}$$

$$\text{Soft Steel}\begin{cases}12{,}000 \text{ for lengths up to 30 radii of gyration.}\\ 13{,}500 - .50\ \frac{l}{r} \text{ for lengths over 30 radii.}\end{cases}$$

No column should be used having a length greater than 150 radii of gyration, or whose length exceeds 45 times the least dimension of the column.

The following tables of safe loads on steel columns have been calculated from the foregoing formulæ. The tables for the safe loads on Angle and **I** Beam columns have been calculated for soft steel. The tables of safe loads for Plate and Angle columns, Channel and Plate columns and **Z** Bar columns have been calculated for medium steel, that being the grade of steel advisable to use for such columns.

The weights given for the various columns do not include rivets or connections of any kind. Rivets should be spaced not exceeding 3″ centers at the ends of a column for a distance equal to twice the width of the column. The distance between centers of rivets, in the line of strain, should not exceed 16 times the least thickness of metal of the parts joined; and the distance between rivets, at right angles to the line of strain, should not exceed 32 times the least thickness of metal.

The table on page 128 gives the ultimate strength of wrought iron columns calculated from Gordon's formulæ. This table may be of use in determining the safety of existing structures of wrought iron. Steel columns are now exclusively used instead of wrought iron, because of their superiority of strength without increased cost.

Cast iron columns are sometimes used in buildings of moderate height, but their use is not to be recommended for buildings where the iron framework must be rigid and afford sufficient lateral stability. The manner in which cast iron columns are connected together, and the mode of attaching beams and girders to them does not permit obtaining sufficient rigidity for such buildings. Cast iron columns have more or less internal strains due to the unequal cooling of the metal in the moulds, which makes it necessary to employ a large factor of safety. No cast iron column should be used in a building with a factor of safety less than 8. Particular attention should be paid to the designing of the cast iron brackets for supporting the beams and girders, in order that they may not be subjected to large internal strains making them liable to break off under a sudden shock. The tables on pages 170–172, inclusive, furnish an easy method of determining the safe loads on round and square cast iron columns. Where the loads are eccentrically applied, producing bending strains in the columns, cast iron columns are inadmissible because of their inability to resist such strains.

The safe loads given in the tables are calculated for concentric loading, *i. e.*, the center of gravity of the load being coincident with the center of gravity of the column. Where this is not the case, the load being greater on one side of the column than on the other, or the entire load being applied on one side only of the column, the effect of the eccentricity must be investigated. If the unbalanced load, in lbs., is multiplied by the distance of its point of application from the center of the column, in inches, the result is the bending moment in inch lbs., which, being divided by the section modulus of the column, gives the strain per sq. in. on the extreme fiber produced by the bending. The load on the column produces a uniform compressive strain on the entire cross section to which must be added the bending strain, the sum being the maximum strain on the extreme fiber. Where the loads are very eccentrically applied, the bending effect is very considerable and must never be neglected. If the maximum fiber strain, due to direct compression and bending, exceeds the allowable strains per sq. in. on the column by more than 25%, the section of the column should be increased. Thus if the allowable strain on a column from direct load is 10,000 lbs. per sq. in., the combined bending and compression should not exceed 12,500 lbs. per sq. in.

Tables are given of the properties of all columns, for which safe loads are calculated, by means of which the effects of eccentric loading are easily calculated.

EXAMPLE.

A 12″ channel column, 16 ft. long, consisting of two 12″ × 20 lb. channels and two 14″ × $\frac{3}{8}$″ plates sustains a total load of 100 tons of which 40 tons are unbalanced by opposing loads. Find the fiber strain, the point of application of the eccentric load being 6$\frac{3}{8}$″ from the center of the column, producing bending around the axis XX.

Referring to the table of Properties of Channel Columns, on page 137, the area of the column is found to be 22.3 sq. ins., and its Section Modulus around the axis XX is found to be 102. The calculation then is as follows:

Bending moment = 80,000 × 6$\frac{3}{8}$″ = 510,000 in. lbs.

	lbs. per sq. in.
Strain due to bending,	
510,000 ÷ Section modulus (=102) =	5,000
Strain due to direct compression,	
200,000 ÷ Area (= 22.3) =	8,960
Maximum Fiber Strain, =	13,960

ULTIMATE STRENGTHS OF WROUGHT IRON COLUMNS.

For Fixed Ends.	For Square Ends.	For Pin Ends.
$\dfrac{40{,}000}{1+\dfrac{l^2}{40{,}000r^2}}$	$\dfrac{40{,}000}{1+\dfrac{l^2}{30{,}000r^2}}$	$\dfrac{40{,}000}{1+\dfrac{l^2}{20{,}000r^2}}$

l = length in inches. r = least radius of gyration in inches.

Ratio of Length to Radius of Gyration. $\frac{l}{r}$	Ultimate Strength, lbs. per sq. in.			Ratio of Length to Diameter.			
	Fixed Ends.	Square Ends.	Pin Ends.	Z Bar Column.	Box Column.	Open Column.	Star Column.
30	39,100	38,800	38,300	9	10	12	7
35	38,800	38,400	37,700	10	12	13	8
40	38,500	38,000	37,000	12	13	15	9
45	38,100	37,500	36,300	13	15	17	10
50	37,700	36,900	35,600	15	17	19	11
55	37,200	36,300	34,800	16	18	21	12
60	36,700	35,700	33,900	18	20	23	13
65	36,200	35,100	33,000	19	22	25	14
70	35,600	34,400	32,100	21	23	27	15
75	35,100	33,700	31,200	22	25	29	17
80	34,500	33,000	30,300	24	27	31	18
85	34,000	32,200	29,400	25	28	33	19
90	33,300	31,500	28,500	26	30	35	20
95	32,600	30,800	27,600	28	32	36	21
100	32,000	30,000	26,700	29	33	38	22
105	31,400	29,300	25,800	31	35	40	23
110	30,700	28,500	24,900	32	37	42	24
115	30,100	27,800	24,100	34	38	44	25
120	29,300	27,000	23,300	35	40	46	27
125	28,800	26,300	22,500	37	42	48	28
130	28,100	25,600	21,700	38	43	50	29
135	27,500	24,900	20,900	40	45	52	30
140	26,800	24,200	20,200	41	47	54	31
145	26,200	23,500	19,500	43	48	56	32
150	25,600	22,900	18,800	44	50	58	33

For safe quiescent loads, as in buildings, divide above values by 4.

ULTIMATE STRENGTHS OF SOFT AND MEDIUM STEEL COLUMNS,

Calculated from the following Formulæ.

SOFT STEEL.	MEDIUM STEEL.
Fixed Ends $= \mathbf{54{,}000 - 185}\,\frac{l}{r}$	Fixed Ends $= \mathbf{60{,}000 - 210}\,\frac{l}{r}$
Square Ends $= \mathbf{54{,}000 - 200}\,\frac{l}{r}$	Square Ends $= \mathbf{60{,}000 - 230}\,\frac{l}{r}$
Pin Ends $= \mathbf{54{,}000 - 225}\,\frac{l}{r}$	Pin Ends $= \mathbf{60{,}000 - 260}\,\frac{l}{r}$

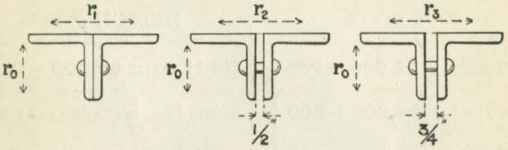
l = length in inches. r = least radius of gyration in inches.

Ratio of Length to Radius of Gyration, $\frac{l}{r}$	Ultimate Strength, lbs. per sq. in.					
	Soft Steel.			Medium Steel.		
	Fixed Ends.	Square Ends. ROOF TRUSSES	Pin Ends.	Fixed Ends.	Square Ends.	Pin Ends.
30	48,500	48,000	47,300			
35	47,500	47,000	46,100			
40	46,600	46,000	45,000			
45	45,700	45,000	43,900			
50	44,800	44,000	42,800	49,500	48,500	47,000
55	43,800	43,000	41,600	48,500	47,400	45,700
60	42,900	42,000	40,500	47,400	46,200	44,400
65	42,000	41,000	39,400	46,400	45,100	43,100
70	41,100	40,000	38,300	45,300	43,900	41,800
75	40,100	39,000	37,100	44,300	42,800	40,500
80	39,200	38,000	36,000	43,200	41,600	39,200
85	38,300	37,000	34,900	42,200	40,500	37,900
90	37,400	36,000	33,800	41,100	39,300	36,600
95	36,400	35,000	32,600	40,100	38,200	35,300
100	35,500	34,000	31,500	39,000	37,000	34,000
105	34,600	33,000	30,400	38,000	35,900	32,700
110	33,700	32,000	29,300	36,900	34,700	31,400
115	32,700	31,000	28,100	35,900	33,600	30,100
120	31,800	30,000	27,000	34,800	32,400	28,800
125	30,900	29,000	25,900	33,800	31,300	27,500
130	30,000	28,000	24,800	32,700	30,100	26,200
135	29,000	27,000	23,600	31,700	29,000	24,900
140	28,100	26,000	22,500	30,600	27,800	23,600
145	27,200	25,000	21,400	29,600	26,700	22,300
150	26,300	24,000	20,300	28,500	25,500	21,000

For safe quiescent loads, as in buildings, divide above values by 4.

RADII OF GYRATION FOR TWO ANGLES

PLACED BACK TO BACK.



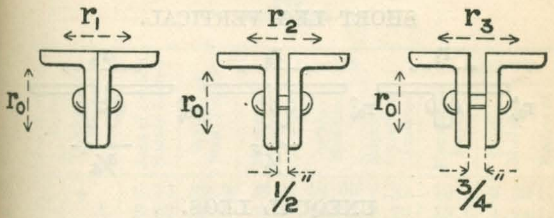
EQUAL LEGS.

Radii of Gyration given correspond to directions of the arrow-heads.

Size, inches.	Thickness, inches.	Radii of Gyration. r_0	r_1	r_2	r_3
6×6	$\frac{7}{8}$	1.87	2.64	2.83	2.92
6×6	$\frac{3}{8}$	1.88	2.49	2.66	2.75
5×5	$\frac{3}{4}$	1.55	2.20	2.38	2.48
5×5	$\frac{3}{8}$	1.56	2.09	2.27	2.36
4×4	$\frac{13}{16}$	1.24	1.83	2.03	2.12
4×4	$\frac{5}{16}$	1.24	1.67	1.85	1.94
$3\frac{1}{2}\times3\frac{1}{2}$	$\frac{5}{8}$	1.04	1.51	1.70	1.81
$3\frac{1}{2}\times3\frac{1}{2}$	$\frac{5}{16}$	1.08	1.46	1.65	1.74
3×3	$\frac{5}{8}$	.94	1.40	1.59	1.69
3×3	$\frac{1}{4}$	.93	1.25	1.43	1.53
$2\frac{1}{2}\times2\frac{1}{2}$	$\frac{1}{2}$	.76	1.12	1.31	1.42
$2\frac{1}{2}\times2\frac{1}{2}$	$\frac{1}{4}$	.77	1.05	1.25	1.34
$2\frac{1}{4}\times2\frac{1}{4}$	$\frac{1}{2}$	.70	1.05	1.25	1.35
$2\frac{1}{4}\times2\frac{1}{4}$	$\frac{3}{16}$	.69	.94	1.12	1.22
2×2	$\frac{1}{2}$	.62	.95	1.15	1.26
2×2	$\frac{3}{16}$	.62	.84	1.03	1.13

RADII OF GYRATION FOR TWO ANGLES

PLACED BACK TO BACK, LONG LEG VERTICAL.



UNEQUAL LEGS.

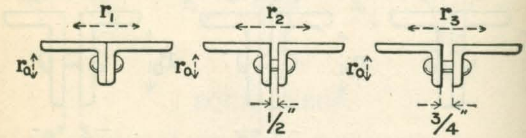
Radii of Gyration given correspond to directions of the arrow-heads.

Size, inches.	Thickness, inches.	Radii of Gyration.			
		r_0	r_1	r_2	r_3
6×4	$\frac{7}{8}$	1.95	1.68	1.87	1.97
6×4	$\frac{3}{8}$	1.93	1.50	1.67	1.76
5×3½	$\frac{3}{4}$	1.59	1.44	1.63	1.73
5×3½	$\frac{3}{8}$	1.60	1.34	1.51	1.61
5×3	$\frac{3}{4}$	1.62	1.23	1.42	1.52
5×3	$\frac{5}{16}$	1.61	1.09	1.26	1.36
4½×3	$\frac{3}{4}$	1.43	1.25	1.44	1.55
4½×3	$\frac{5}{16}$	1.45	1.13	1.31	1.40
4×3½	$\frac{3}{4}$	1.24	1.53	1.72	1.83
4×3½	$\frac{5}{16}$	1.26	1.41	1.58	1.69
4×3	$\frac{5}{8}$	1.23	1.20	1.39	1.50
4×3	$\frac{5}{16}$	1.27	1.17	1.35	1.45
3½×3	$\frac{5}{8}$	1.06	1.27	1.46	1.56
3½×3	$\frac{5}{16}$	1.10	1.21	1.39	1.49
3½×2½	$\frac{9}{16}$	1.10	1.04	1.23	1.34
3½×2½	$\frac{1}{4}$	1.12	.96	1.17	1.24
3×2½	$\frac{9}{16}$	.93	1.07	1.27	1.37
3×2½	$\frac{1}{4}$	.95	1.00	1.18	1.28
3×2	$\frac{1}{2}$	.92	.80	1.00	1.10
3×2	$\frac{1}{4}$	.96	.75	.93	1.04
2¼×1½	$\frac{5}{16}$	.70	.60	.79	.91
2¼×1½	$\frac{3}{16}$	.72	.57	.75	.86

RADII OF GYRATION FOR TWO ANGLES

PLACED BACK TO BACK,

SHORT LEG VERTICAL.

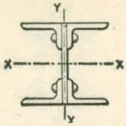


UNEQUAL LEGS.

Radii of Gyration given correspond to direction of the arrow-heads.

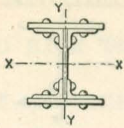
Size, inches.	Thickness, inches.	Radii of Gyration.			
		r_0	r_1	r_2	r_3
6×4	7/8	1.19	2.94	3.13	3.23
6×4	3/8	1.17	2.74	2.92	3.02
5×3½	3/4	1.01	2.39	2.58	2.68
5×3½	3/8	1.02	2.27	2.45	2.55
5×3	3/4	.86	2.50	2.69	2.79
5×3	5/16	.85	2.33	2.51	2.61
4½×3	3/4	.86	2.18	2.38	2.46
4½×3	5/16	.87	2.06	2.25	2.33
4×3½	3/4	1.05	1.85	2.04	2.14
4×3½	5/16	1.07	1.73	1.91	2.00
4×3	5/8	.83	1.84	2.03	2.13
4×3	5/16	.89	1.79	1.97	2.07
3½×3	5/8	.87	1.57	1.76	1.87
3½×3	5/16	.90	1.53	1.71	1.81
3½×2½	9/16	.72	1.66	1.85	1.95
3½×2½	1/4	.74	1.58	1.76	1.86
3×2½	9/16	.73	1.40	1.59	1.69
3×2½	1/4	.75	1.32	1.49	1.60
3×2	1/2	.55	1.42	1.62	1.72
3×2	1/4	.57	1.39	1.57	1.68

PROPERTIES OF PASSAIC STEEL PLATE AND ANGLE COLUMNS.



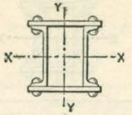
Width of Plate, Inches.	Size of Angles, Inches.	Thickness of Plate and Angles, Inches.	Area of Section, Square Inches.	Weight per Foot, Pounds.	Axis XX.			Axis YY.		
					Moment of Inertia.	Section Modulus.	Radius of Gyration, Inches.	Moment of Inertia.	Section Modulus.	Radius of Gyration, Inches.
6	3 × 2½	¼	6.74	22.9	36.3	12.09	2.32	10.4	3.32	1.24
″		5/16	8.52	29.0	44.6	14.87	2.29	13.6	4.24	1.26
″		7/16	11.71	39.8	59.0	19.68	2.25	21.1	6.42	1.34
″		½	13.00	44.2	64.6	21.53	2.23	24.7	7.60	1.38
7	3½ × 2½	¼	7.51	25.5	58.3	16.65	2.78	16.1	4.43	1.46
″		5/16	9.43	32.1	71.9	20.55	2.76	20.8	5.59	1.49
″		7/16	12.98	44.1	95.8	27.38	2.72	30.8	8.15	1.54
″		½	14.50	49.3	105.1	30.02	2.69	36.3	9.69	1.58
8	4 × 3	5/16	10.86	36.9	107.5	26.88	3.14	30.3	7.30	1.67
″		⅜	13.12	44.6	128.5	32.13	3.13	37.4	8.79	1.69
″		7/16	14.98	50.9	144.6	36.15	3.11	44.4	10.54	1.72
″		½	17.24	58.6	163.5	40.88	3.08	53.1	12.29	1.75
″		9/16	19.50	66.3	182.9	45.73	3.06	61.9	14.04	1.78
″		⅝	20.92	71.1	193.5	48.38	3.04	69.1	16.04	1.82
9	4½ × 3	5/16	11.81	40.1	154.2	34.26	3.62	42.6	9.15	1.90
″		⅜	14.22	48.3	183.5	40.78	3.59	52.9	11.13	1.93
″		7/16	16.30	55.5	207.5	46.12	3.57	63.1	13.37	1.97
″		½	18.74	63.7	235.9	52.44	3.55	75.3	15.64	2.01
″		9/16	21.18	72.0	263.0	58.44	3.52	87.9	17.90	2.04
″		⅝	22.83	77.6	279.1	62.24	3.50	99.0	20.57	2.08
10	5 × 3	5/16	12.73	43.3	211.8	42.36	4.08	57.6	11.16	2.13
″		⅜	15.35	52.2	252.7	50.54	4.06	71.9	13.68	2.17
″		7/16	17.62	59.9	286.4	57.28	4.03	85.9	16.46	2.21
″		½	20.24	68.8	326.0	65.20	4.01	102.2	19.22	2.25
″		9/16	22.35	76.0	355.7	71.14	4.00	118.1	22.36	2.29
″		⅝	24.97	84.9	392.3	78.46	3.97	136.6	25.43	2.34
12	6 × 4	⅜	18.94	64.4	443.6	73.37	4.85	119.6	19.34	2.51
″		7/16	22.17	75.4	513.6	85.60	4.81	144.5	23.03	2.55
″		½	25.44	86.5	584.5	97.42	4.80	171.8	26.96	2.60
″		9/16	28.67	97.5	651.0	108.5	4.77	199.7	30.91	2.64
″		⅝	30.94	104.9	693.4	115.6	4.75	223.4	35.39	2.69
″		11/16	34.17	116.2	760.1	126.8	4.72	255.7	39.88	2.73
″		¾	37.44	127.3	825.3	137.6	4.70	288.7	44.44	2.78

PROPERTIES OF PASSAIC STEEL PLATE AND ANGLE COLUMNS.



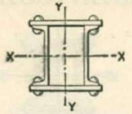
Section of Column.	Thickness of Cover Plates, Inches.	Area of Section, Square Inches.	Weight per Foot, Pounds.	Axis XX. Moment of Inertia.	Axis XX. Section Modulus.	Axis XX. Radius of Gyration, Inches.	Axis YY. Moment of Inertia.	Axis YY. Section Modulus.	Axis YY. Radius of Gyration, Inches.
4 Angles 6″ × 4″ × 1/2″ 1 Web Plate 12″ × 3/4″ 2 Cover Plates 13″ wide.	1/2	41.44	140.9	1129	174.0	5.22	366.1	56.32	2.98
	9/16	43.07	146.5	1199	182.6	5.27	389.0	59.84	3.00
	5/8	44.69	152.0	1269	192.0	5.33	411.8	63.36	3.04
	11/16	46.32	157.5	1340	200.3	5.38	434.7	66.88	3.07
	3/4	47.94	163.0	1415	209.8	5.44	457.6	70.40	3.10
	13/16	49.57	168.5	1492	219.3	5.49	480.5	73.92	3.12
	7/8	51.19	174.0	1563	227.2	5.52	503.4	77.44	3.14
	15/16	52.82	179.6	1642	237.0	5.59	526.2	81.00	3.16
	1	54.44	185.1	1723	246.0	5.64	549.1	84.48	3.18
	1 1/16	56.07	190.6	1803	256.1	5.68	572.0	88.00	3.20
	1 1/8	57.69	196.2	1884	264.9	5.72	594.9	91.52	3.22
	1 3/16	59.32	201.7	1965	274.3	5.75	617.8	95.04	3.23
	1 1/4	60.94	207.2	2050	283.2	5.80	640.6	98.56	3.25
	1 5/16	62.57	212.8	2143	292.7	5.85	663.5	102.1	3.26
	1 3/8	64.19	218.3	2224	301.8	5.88	686.4	105.6	3.27
	1 7/16	65.82	223.8	2311	311.2	5.93	709.3	109.1	3.29
	1 1/2	67.44	229.3	2406	321.3	5.98	732.2	112.6	3.30
4 Angles 6″ × 4″ × 3/4″ 1 Web Plate 14″ × 3/4″ 2 Cover Plates 15″ wide.	1/2	53.94	183.2	1981	264.1	6.05	569.5	75.93	3.25
	9/16	55.82	189.8	2088	276.2	6.12	604.6	80.61	3.30
	5/8	57.69	196.1	2195	288.3	6.17	639.8	85.30	3.33
	11/16	59.57	202.6	2304	299.8	6.22	674.9	89.98	3.37
	3/4	61.44	208.8	2417	312.8	6.28	710.1	94.66	3.40
	13/16	63.32	215.3	2533	325.3	6.32	745.2	99.36	3.44
	7/8	65.19	221.6	2645	336.2	6.36	780.4	104.0	3.46
	15/16	67.07	228.1	2765	349.4	6.41	815.5	108.7	3.49
	1	68.94	234.4	2885	361.4	6.48	850.7	113.4	3.52
	1 1/16	70.82	240.8	3004	373.2	6.51	885.9	118.1	3.54
	1 1/8	72.69	247.1	3131	386.1	6.57	921.0	122.8	3.56
	1 3/16	74.57	253.6	3251	398.4	6.61	956.2	127.5	3.58
	1 1/4	76.44	259.9	3383	409.7	6.66	991.3	132.2	3.60
	1 5/16	78.32	266.3	3510	421.6	6.70	1026.5	136.9	3.62
	1 3/8	80.19	272.6	3635	437.3	6.74	1061.6	141.5	3.64
	1 7/16	82.07	279.1	3770	448.2	6.79	1096.8	146.2	3.66
	1 1/2	83.94	285.4	3903	459.9	6.83	1132.0	150.9	3.68

PROPERTIES OF PASSAIC STEEL CHANNEL COLUMNS.



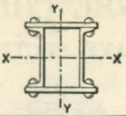
Designation.	Weight of each channel, Lbs. per Ft.	Thickness of Cover Plate, Inches.	Area of Section, Square Inches.	Axis XX.			Axis YY.		
				Moment of Inertia.	Section Modulus.	Radius of Gyration, Inches.	Moment of Inertia.	Section Modulus.	Radius of Gyration, Inches.
6″ Channel Column: 2 Channels 6″ deep and 2 cover plates 8″ wide.	8	1/4	8.70	64.0	19.9	2.72	46.7	11.7	2.31
	10	1/4	9.88	68.1	21.0	2.62	50.3	12.6	2.27
	″	5/16	10.88	78.2	23.6	2.68	56.0	14.0	2.27
	″	3/8	11.88	90.1	26.6	2.75	61.0	15.3	2.27
	12	3/8	12.96	98.9	29.2	2.75	71.8	18.0	2.35
	″	7/16	13.96	110.	32.0	2.81	77.2	19.3	2.35
	″	1/2	14.96	122.	34.9	2.86	82.5	20.6	2.35
	15	1/2	16.72	127.	36.4	2.76	86.9	21.7	2.28
	″	9/16	17.72	138.	39.3	2.81	92.2	23.1	2.28
	″	5/8	18.72	152.	42.1	2.86	97.6	24.4	2.28
	17	5/8	19.70	161.	44.4	2.86	111.	27.8	2.38
	″	11/16	20.70	174.	47.2	2.90	116.	29.1	2.37
	″	3/4	21.70	188.	50.2	2.94	122.	30.4	2.37
	″	13/16	22.70	203.	53.1	2.98	127.	31.8	2.37
	″	7/8	23.70	217.	56.0	3.02	132.	33.1	2.36
	″	15/16	24.70	233.	59.0	3.06	138.	34.4	2.36
	″	1	25.70	248.	62.1	3.10	143.	35.8	2.36
7″ Channel Column: 2 Channels 7″ deep and 2 cover plates 9″ wide.	9	1/4	9.72	97.1	25.9	3.16	71.4	15.8	2.71
	″	5/16	10.85	113.	29.7	3.23	79.0	17.6	2.70
	13	5/16	13.23	129.	34.1	3.13	100.	22.3	2.75
	″	3/8	14.35	146.	37.8	3.20	108.	24.0	2.74
	″	7/16	15.48	163.	41.6	3.26	115.	25.7	2.73
	″	1/2	16.60	181.	45.4	3.33	123.	27.4	2.72
	17	1/2	18.95	191.	47.8	3.17	133.	29.6	2.66
	″	9/16	20.08	209.	51.5	3.23	141.	31.4	2.66
	″	5/8	21.20	228.	55.3	3.28	149.	33.1	2.65
	″	11/16	22.33	247.	59.1	3.33	156.	34.7	2.65
	″	3/4	23.45	267.	63.0	3.38	163.	36.4	2.64
	″	13/16	24.58	288.	66.8	3.43	171.	38.1	2.64
	″	7/8	25.70	309.	70.7	3.47	179.	39.8	2.64
	″	15/16	26.83	331.	74.7	3.51	187.	41.5	2.64
	″	1	27.95	354.	78.6	3.56	194.	43.1	2.64

PROPERTIES OF PASSAIC STEEL CHANNEL COLUMNS.



Designation.	Weight of each Channel, lbs. per foot.	Thickness of Cover Plates, inches.	Area of Section, square inches.	Axis XX. Moment of Inertia.	Axis XX. Section Modulus.	Axis XX. Radius of Gyration, inches.	Axis YY. Moment of Inertia.	Axis YY. Section Modulus.	Axis YY. Radius of Gyration, inches.
8″ Channel Column: 2 channels 8″ deep and 2 cover plates 10″ wide.	10	$\frac{1}{4}$	11.0	141	33.3	3.58	107	21.5	3.12
	"	$\frac{5}{16}$	12.3	164	38.1	3.66	118	23.6	3.09
	13	$\frac{5}{16}$	13.9	179	41.6	3.59	136	27.3	3.14
	"	$\frac{3}{8}$	15.1	203	46.3	3.66	147	29.3	3.12
	"	$\frac{7}{16}$	16.4	227	51.2	3.73	157	31.4	3.10
	"	$\frac{1}{2}$	17.6	252	56.1	3.79	167	33.5	3.08
	17	$\frac{1}{2}$	20.0	265	58.7	3.64	184	36.8	3.04
	"	$\frac{9}{16}$	21.2	290	63.8	3.70	194	39.0	3.03
	"	$\frac{5}{8}$	22.5	317	68.2	3.76	205	40.9	3.02
	"	$\frac{11}{16}$	23.7	344	73.3	3.81	215	43.0	3.02
	"	$\frac{3}{4}$	25.0	372	78.2	3.86	225	45.2	3.02
	"	$\frac{13}{16}$	26.2	400	83.1	3.91	236	47.2	3.00
	"	$\frac{7}{8}$	27.5	430	88.3	3.96	246	49.3	2.99
	"	$\frac{15}{16}$	28.7	459	93.1	4.00	257	51.4	2.99
	"	1	30.0	490	98.2	4.04	267	53.4	2.99
9″ Channel Column: 2 channels 9″ deep and 2 cover plates 11″ wide.	13	$\frac{5}{16}$	14.5	240	49.8	4.07	167	30.4	3.40
	"	$\frac{3}{8}$	15.9	272	55.7	4.14	181	32.9	3.38
	16	$\frac{3}{8}$	17.7	295	60.5	4.09	208	37.8	3.43
	"	$\frac{7}{16}$	19.0	329	66.7	4.16	222	40.3	3.41
	"	$\frac{1}{2}$	20.4	364	72.8	4.23	236	42.9	3.41
	21	$\frac{1}{2}$	23.4	383	76.6	4.05	259	47.0	3.33
	"	$\frac{9}{16}$	24.8	417	82.5	4.11	273	49.5	3.32
	"	$\frac{5}{8}$	26.1	453	88.3	4.16	287	52.1	3.31
	"	$\frac{11}{16}$	27.5	489	94.0	4.21	300	54.6	3.30
	"	$\frac{3}{4}$	28.9	528	100	4.27	314	57.0	3.30
	"	$\frac{13}{16}$	30.3	566	106	4.33	328	59.6	3.29
	"	$\frac{7}{8}$	31.6	604	113	4.38	342	62.2	3.29
	"	$\frac{15}{16}$	33.0	648	119	4.43	356	64.8	3.28
	"	1	34.4	686	125	4.47	370	67.3	3.28
	"	$1\frac{1}{16}$	35.8	726	131	4.50	383	69.6	3.27
	"	$1\frac{1}{8}$	37.1	771	137	4.55	397	72.2	3.27
	"	$1\frac{3}{16}$	38.5	816	144	4.60	411	74.8	3.27
	"	$1\frac{1}{4}$	39.9	859	149	4.64	425	77.3	3.27

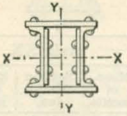
PROPERTIES OF PASSAIC STEEL CHANNEL COLUMNS.



Designation.	W't of each Channel. lbs. per ft.	Thickness of Cover Plates, Ins.	Area of Section, sq. inches.	Axis XX. Mom. of Inertia.	Axis XX. Section Modulus.	Axis XX. Rad. of Gyr., inches.	Axis YY. Moment of Inertia.	Axis YY. Section Modulus.	Axis YY. Rad. of Gyr., inches.
10″ Channel Column: 2 channels 10″ deep, 2 cover plates 12″ wide.	15	5/16	16.3	336	63.2	4.49	227	37.9	3.69
	〃	3/8	18.2	377	70.2	4.55	245	40.9	3.67
	20	3/8	20.8	412	77.0	4.46	286	47.7	3.71
	〃	7/16	22.3	457	84.0	4.53	304	50.7	3.69
	〃	1/2	23.8	502	91.5	4.60	322	53.7	3.68
	25	1/2	26.7	526	95.8	4.45	348	58.0	3.61
	〃	9/16	28.2	572	103	4.51	366	61.1	3.61
	〃	5/8	29.7	619	110	4.56	384	64.0	3.60
	30	5/8	32.6	643	114	4.44	408	68.0	3.54
	〃	11/16	34.1	691	122	4.50	426	71.0	3.53
	〃	3/4	35.6	740	129	4.56	444	74.0	3.53
	〃	13/16	37.1	790	136	4.62	462	77.0	3.53
	〃	7/8	38.6	841	144	4.68	480	80.0	3.53
	〃	15/16	40.1	893	150	4.73	498	83.0	3.52
	〃	1	41.6	949	158	4.78	516	86.0	3.52
	〃	1 1/8	44.6	1059	172	4.87	552	92.0	3.52
	〃	1 1/4	47.6	1173	188	4.97	588	98.0	3.51
	〃	1 3/8	50.6	1292	203	5.05	624	104	3.51
	〃	1 1/2	53.6	1416	217	5.14	660	110	3.51
12″ Channel Column: 2 channels 12″ deep, 2 cover plates 14″ wide.	20	3/8	22.3	650	102	5.40	429	61.3	4.39
	〃	7/16	24.1	724	112	5.48	457	65.3	4.36
	25	7/16	27.1	760	118	5.30	505	72.1	4.31
	〃	1/2	28.8	833	128	5.38	534	76.3	4.31
	30	1/2	31.6	891	137	5.32	600	85.7	4.36
	〃	9/16	33.4	964	147	5.37	628	89.7	4.34
	〃	5/8	35.1	1043	157	5.45	657	93.9	4.33
	〃	11/16	36.9	1118	168	5.51	686	98.0	4.31
	〃	3/4	38.6	1198	178	5.57	714	102	4.30
	35	3/4	41.6	1234	183	5.44	753	108	4.25
	〃	13/16	43.4	1316	193	5.50	782	112	4.25
	〃	7/8	45.1	1396	204	5.56	810	116	4.24
	〃	15/16	46.9	1482	214	5.63	840	120	4.24
	〃	1	48.6	1565	224	5.68	867	124	4.22
	〃	1 1/8	52.1	1742	245	5.79	925	132	4.21
	〃	1 1/4	55.6	1922	266	5.90	981	140	4.21
	〃	1 3/8	59.1	2105	287	5.98	1039	148	4.19
	〃	1 1/2	62.6	2302	308	6.08	1096	157	4.19

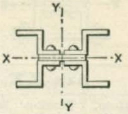
PROPERTIES OF PASSAIC STEEL CHANNEL COLUMNS,

HEAVY SECTION.



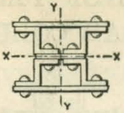
Designation.	Thickness of Cover Plates, Inches.	Weight of Column, Lbs. per Ft.	Area of Section, Square Inches.	Axis XX. Moment of Inertia.	Axis XX. Section Modulus.	Axis XX. Radius of Gyration, Inches.	Axis YY. Moment of Inertia.	Axis YY. Section Modulus.	Axis YY. Radius of Gyration, Inches.
10″ Extra Heavy Channel Column: 2 Channels 10″×30 lbs., 2 cover plates 12″ wide, and 2 web plates 9″×3/4″.	3/4	167.0	49.1	831	145	4.11	522	87	3.26
	13/16	172.1	50.6	881	152	4.18	540	90	3.27
	7/8	177.2	52.1	932	159	4.23	558	93	3.27
	15/16	182.3	53.6	985	166	4.30	576	96	3.28
	1	187.4	55.1	1039	173	4.35	594	99	3.28
	1 1/8	197.6	58.1	1149	188	4.45	630	105	3.29
	1 1/4	207.8	61.1	1264	203	4.55	666	111	3.30
	1 3/8	218.0	64.1	1384	218	4.65	702	117	3.31
	1 1/2	228.2	67.1	1507	233	4.75	738	123	3.31
	1 5/8	238.4	70.1	1637	247	4.84	774	129	3.32
	1 3/4	248.6	73.1	1766	263	4.92	810	135	3.33
	1 7/8	258.8	76.1	1910	279	5.00	846	141	3.33
	2	269.0	79.1	2057	293	5.10	882	147	3.34
	2 1/8	279.2	82.1	2208	311	5.19	918	153	3.34
	2 1/4	289.4	85.1	2361	326	5.27	954	159	3.34
12″ Extra Heavy Channel Column: 2 Channels 12″×35 lbs., 2 cover plates 14″ wide, and 2 web plates 11″×3/4″.	3/4	197.6	58.1	1402	208	4.91	927	132	3.99
	13/16	203.5	59.9	1478	217	4.97	956	136	3.99
	7/8	209.4	61.6	1563	228	5.05	985	141	4.00
	15/16	215.4	63.4	1646	237	5.10	1013	145	4.00
	1	221.3	65.1	1729	247	5.15	1041	149	4.00
	1 1/8	233.3	68.6	1907	268	5.28	1099	157	4.00
	1 1/4	245.1	72.1	2090	288	5.38	1156	165	4.01
	1 3/8	257.0	75.6	2272	309	5.49	1213	173	4.01
	1 1/2	269.0	79.1	2466	329	5.59	1271	181	4.01
	1 5/8	280.8	82.6	2665	349	5.69	1328	189	4.02
	1 3/4	292.7	86.1	2876	371	5.78	1385	198	4.02
	1 7/8	304.7	89.6	3081	391	5.86	1442	206	4.02
	2	316.5	93.1	3313	415	5.97	1499	214	4.02
	2 1/8	328.4	96.6	3538	435	6.05	1557	222	4.02
	2 1/4	340.4	100.1	3773	458	6.15	1614	231	4.02

PROPERTIES OF PASSAIC STEEL Z BAR COLUMNS.



Designation.	Section of Column.	Thickness of Z bars and web plate.	Area of Section, square inches.	Axis XX.			Axis YY.		
				Moment of Inertia.	Section Modulus.	Radius of Gyration, inches.	Moment of Inertia.	Section Modulus.	Radius of Gyration, inches.
12″ Z bar Column.	4 Z bars 6″ deep and 1 web plate 8″ wide.	3/8	21.4	287	46.5	3.67	337	46.5	3.97
		7/16	25.1	347	55.2	3.72	391	54.0	3.95
		1/2	28.8	409	64.1	3.77	445	61.3	3.92
		9/16	31.2	427	67.9	3.69	469	66.4	3.88
		5/8	34.8	489	76.8	3.74	518	73.4	3.86
		11/16	38.5	556	85.9	3.79	567	80.0	3.83
		3/4	40.5	562	88.2	3.72	579	84.2	3.78
		13/16	44.1	629	97.3	3.77	624	90.7	3.76
		7/8	47.7	700	106.6	3.82	664	96.5	3.73
10″ Z bar Column.	4 Z bars 5″ deep and 1 web plate 7″ wide.	5/16	15.8	149	29.0	3.08	197	30.1	3.54
		3/8	19.0	186	35.5	3.13	235	35.8	3.52
		7/16	22.3	225	42.0	3.17	272	42.1	3.50
		1/2	24.5	236	44.9	3.10	290	45.5	3.44
		9/16	27.7	275	51.5	3.16	324	50.8	3.42
		5/8	30.9	318	58.4	3.21	358	56.1	3.40
		11/16	32.7	320	59.9	3.13	365	59.0	3.34
		3/4	35.8	363	66.8	3.18	393	63.5	3.32
		13/16	39.0	411	74.3	3.25	428	69.2	3.30
8″ Z bar Column.	4 Z bars 4″ deep and 1 web plate 6½″ wide.	1/4	11.3	68.7	16.6	2.47	123	20.0	3.31
		5/16	14.2	89.8	21.3	2.52	152	24.6	3.28
		3/8	17.1	113	26.1	2.57	184	29.8	3.28
		7/16	19.0	118	28.1	2.49	198	33.1	3.23
		1/2	21.9	142	32.9	2.54	225	37.6	3.21
		9/16	24.7	167	37.8	2.59	252	41.9	3.19
		5/8	26.3	167	38.8	2.52	258	44.3	3.13
		11/16	29.0	193	43.8	2.58	281	48.4	3.11
		3/4	31.9	221	49.0	2.63	305	52.4	3.09
6″ Z bar Column.	4 Z bars 3″ deep and 1 web plate 6″ wide.	1/4	9.38	32.3	10.3	1.86	86.7	15.6	3.04
		5/16	11.8	42.8	13.3	1.91	108	19.3	3.02
		3/8	13.7	48.0	15.1	1.87	121	22.3	2.97
		7/16	16.1	59.5	18.1	1.92	140	25.8	2.95
		1/2	17.8	63.6	19.6	1.89	150	28.3	2.91
		9/16	20.1	76.0	22.7	1.94	168	31.7	2.89

PROPERTIES OF PASSAIC STEEL Z BAR COLUMNS.



Designation.	Section of Column.	Thickness of Cover Plates.	Area of Section, sq. inches.	Axis XX. Mom. of Inertia.	Axis XX. Section Modulus.	Axis XX. Rad. of Gyr., inches.	Axis YY. Moment of Inertia.	Axis YY. Section Modulus.	Axis YY. Rad. of Gyr., inches.
14″ Z bar column.	4 Z bars 6″ × 3/4″, 1 web plate 8″ × 3/4″, 2 cover plates 14″ wide.	3/8	51.0	1014	150.0	4.46	750.5	107.2	3.84
		7/16	52.8	1094	160.7	4.55	779.2	111.3	3.84
		1/2	54.5	1180	171.6	4.65	808.0	115.4	3.85
		9/16	56.3	1260	181.6	4.72	836.2	119.5	3.85
		5/8	58.0	1344	192.2	4.82	864.7	123.5	3.86
		11/16	59.8	1431	202.7	4.89	893.7	127.7	3.87
		3/4	61.5	1511	212.0	4.96	922.0	131.7	3.88
		13/16	63.3	1609	223.9	5.04	951.2	135.9	3.88
		7/8	65.0	1701	234.5	5.11	979.5	139.9	3.88
14″ Z bar column.	4 Z bars 6 1/8″ × 7/8″, 1 web plate 8″ × 7/8″, 2 cover plates 14″ wide.	11/16	66.9	1618	223.2	4.92	979.3	139.7	3.83
		3/4	68.7	1711	234.0	4.99	1007	143.8	3.84
		13/16	70.5	1805	244.8	5.06	1035	147.9	3.84
		7/8	72.2	1901	255.7	5.13	1064	152.0	3.84
		15/16	74.0	1999	266.5	5.20	1092	156.2	3.84
		1	75.7	2098	277.5	5.26	1121	160.2	3.85
		1 1/16	77.5	2198	288.3	5.32	1150	164.2	3.85
		1 1/8	79.2	2300	299.1	5.39	1178	168.2	3.85
		1 3/16	81.0	2405	310.4	5.45	1207	172.5	3.86
		1 1/4	82.7	2510	321.3	5.51	1236	176.5	3.86
16″ Z bar column.	4 Z bars 6 1/8″ × 7/8″, 1 web plate 10″ × 7/8″, 2 cover plates 16″ wide.	1	81.4	2298	303.8	5.31	1726	216.2	4.60
		1 1/16	83.4	2413	316.5	5.38	1769	221.6	4.60
		1 1/8	85.4	2531	329.5	5.44	1811	226.8	4.60
		1 3/16	87.4	2650	341.9	5.50	1854	232.2	4.60
		1 1/4	89.4	2771	354.4	5.56	1897	237.6	4.60
		1 5/16	91.4	2895	367.6	5.62	1939	242.9	4.60
		1 3/8	93.4	3019	380.4	5.69	1982	248.2	4.60
		1 7/16	95.4	3146	393.3	5.74	2025	253.6	4.60
		1 1/2	97.4	3275	406.3	5.80	2067	258.9	4.60
		1 9/16	99.4	3406	419.2	5.86	2110	264.1	4.60
		1 5/8	101.4	3539	432.3	5.91	2153	269.4	4.61
		1 11/16	103.4	3674	445.5	5.96	2195	274.8	4.61
		1 3/4	105.4	3811	458.5	6.01	2238	280.1	4.61
		1 13/16	107.4	3951	471.8	6.06	2280	285.4	4.61
		1 7/8	109.4	4092	485.0	6.12	2323	290.8	4.61
		1 15/16	111.4	4235	498.3	6.17	2366	296.2	4.61
		2	113.4	4381	511.7	6.21	2409	301.4	4.61
		2 1/16	115.4	4528	524.9	6.26	2451	306.8	4.61
		2 1/8	117.4	4679	538.6	6.31	2494	312.2	4.61
		2 3/16	119.4	4831	552.1	6.36	2537	317.4	4.61
		2 1/4	121.4	4985	565.3	6.41	2579	322.9	4.61

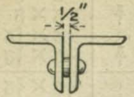
SAFE LOADS FOR PASSAIC STEEL ANGLES, EQUAL LEGS, USED AS STRUTS OR COLUMNS, IN TONS OF 2000 LBS. SQUARE ENDS.

Neutral Axis Diagonal.

Size of Angle, in inches.	Thickness of Angle, inches.	Least Radius of Gyr., Axis Diagonal, inches.	Area of Section, sq. inches.	Unsupported length of Column, in feet.													
				1	2	3	4	5	6	7	8	9	10	11	12	13	14
6 × 6	7/8	1.20	10.03			60.2	57.7	55.2	52.7	50.2	47.7	45.2	42.6	40.1	37.6	35.1	32.6
6 × 6	3/8	1.20	4.36			26.2	25.1	24.0	22.9	21.8	20.7	19.6	18.5	17.4	16.3	15.2	14.1
5 × 5	3/4	1.00	7.11			41.7	39.6	37.4	35.3	33.1	31.0	28.9	26.8	24.6	22.5	20.4	
5 × 5	3/8	1.00	3.61			21.2	20.2	19.1	18.0	16.9	15.8	14.7	13.6	12.5	11.4	10.3	
4 × 4	13/16	.80	6.11		36.7	34.4	32.1	29.8	27.5	25.2	22.9	20.6	18.3				
4 × 4	5/16	.80	2.40		14.4	13.5	12.6	11.7	10.8	9.9	9.0	8.1	7.2				
$3\frac{1}{2} \times 3\frac{1}{2}$	5/8	.70	3.98		23.4	21.7	20.0	18.3	16.6	14.9	13.2	11.5					
$3\frac{1}{2} \times 3\frac{1}{2}$	5/16	.70	2.09		12.4	11.5	10.6	9.7	8.8	7.9	7.0	6.1					
3 × 3	5/8	.60	3.56		20.5	18.7	16.9	15.2	13.4	11.6	9.8						
3 × 3	1/4	.60	1.44		8.3	7.6	6.9	6.1	5.4	4.7	4.0						
$2\frac{1}{2} \times 2\frac{1}{2}$	1/2	.50	2.31	13.8	12.8	11.4	10.1	8.7	7.2								
$2\frac{1}{2} \times 2\frac{1}{2}$	1/4	.50	1.19	7.2	6.5	5.8	5.1	4.4	3.7								
$2\frac{1}{4} \times 2\frac{1}{4}$	1/2	.45	2.11	12.6	11.2	9.9	8.5	7.1	5.7								
$2\frac{1}{4} \times 2\frac{1}{4}$	3/16	.45	.81	4.9	4.3	3.8	3.3	2.8	2.2								
2 × 2	1/2	.40	1.86	11.2	9.8	8.4	7.0	5.6									
2 × 2	3/16	.40	.71	4.3	3.7	3.2	2.7	2.2									

Strains per square inch;
12,000 lbs. for lengths of 30 radii and under.
$13{,}500-50\frac{l}{r}$ for lengths over 30 radii.

Add 25% to these values for Columns subject to both Bending and Compression.



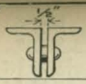
SAFE LOADS FOR PASSAIC STEEL ANGLES, USED AS STRUTS OR COLUMNS, IN TONS OF 2000 LBS.

EQUAL LEGS, SQUARE ENDS.

Two Angles, placed back to back, $\frac{1}{2}''$ apart.

Size of Angles, in inches.	Thickness of Angles, inches.	Least Radius of Gyration, inches.	Area of Section, sq. inches.	Unsupported length of Column, in feet. 2	3	4	5	6	8	10	12	14	16	18	20	22	24
6×6	$\frac{7}{8}$	1.87	20.06				119	116	109	103	96.6	90.2	83.8	77.4	71.0	64.6	58.2
6×6	$\frac{3}{8}$	1.88	8.72				51.9	50.5	47.7	45.0	42.2	39.4	36.6	33.8	31.0	28.2	25.4
5×5	$\frac{3}{4}$	1.55	14.22			85.0	82.2	79.5	74.0	68.5	63.0	57.4	51.9	46.4	40.9		
5×5	$\frac{3}{8}$	1.56	7.22			43.2	41.9	40.5	37.7	34.9	32.1	29.3	26.6	23.8	21.0		
4×4	$\frac{13}{16}$	1.24	12.22		73.3	70.7	67.8	64.9	59.0	53.0	47.1	41.1	35.2				
4×4	$\frac{5}{16}$	1.24	4.80		28.8	27.7	26.6	25.4	23.1	20.8	18.4	16.1	13.8				
$3\frac{1}{2} \times 3\frac{1}{2}$	$\frac{5}{8}$	1.04	7.96		46.9	44.4	42.2	39.9	35.3	30.8	26.2						
$3\frac{1}{2} \times 3\frac{1}{2}$	$\frac{5}{16}$	1.08	4.18		24.7	23.7	22.5	21.3	19.0	16.6	14.3						
3×3	$\frac{5}{8}$	.94	7.12	42.7	41.2	38.9	36.7	34.4	29.9	25.3	20.8						
3×3	$\frac{1}{4}$	.93	2.88	17.2	16.7	15.8	14.8	13.9	12.0	10.2	8.3						
$2\frac{1}{2} \times 2\frac{1}{2}$	$\frac{1}{2}$	.76	4.62	27.5	25.7	23.9	22.0	20.2	16.6	12.9							
$2\frac{1}{2} \times 2\frac{1}{2}$	$\frac{1}{4}$	.77	2.38	14.2	13.2	12.3	11.4	10.5	8.7	6.8							
$2\frac{1}{4} \times 2\frac{1}{4}$	$\frac{1}{2}$	.70	4.22	24.9	23.1	21.3	19.5	17.6	14.0								
$2\frac{1}{4} \times 2\frac{1}{4}$	$\frac{3}{16}$	.69	1.62	9.5	8.8	8.1	7.4	6.7	5.3								
2×2	$\frac{1}{2}$	.62	3.72	21.5	19.7	17.9	16.1	14.3	10.7								
2×2	$\frac{3}{16}$	.62	1.42	8.2	7.5	6.8	6.1	5.4	4.1								

Strains per square inch;
12,000 lbs. for lengths of 30 radii and under.
$13{,}500-50\frac{l}{r}$ for lengths over 30 radii.

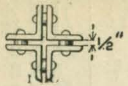


SAFE LOADS FOR PASSAIC STEEL ANGLES, UNEQUAL LEGS, USED AS STRUTS OR COLUMNS, IN TONS OF 2000 LBS. SQUARE ENDS.

Two angles, long legs placed back to back, ½" apart.

Size of Angles, in inches.	Thickness, inches.	Least Radius of Gyration, inches.	Area of Section, sq. inches.	Unsupported length of column, in feet. 2	3	4	5	6	8	10	12	14	16	18	20	22
6 × 4	⅞	1.87	16.68				99.2	96.5	91.2	85.8	80.5	75.2	69.8	64.5	59.1	53.8
6 × 4	⅜	1.67	7.22			43.4	42.2	40.9	38.3	35.7	33.1	30.6	28.0	25.4	22.8	
5 × 3½	¾	1.59	11.96			71.8	69.5	67.3	62.8	58.3	53.8	49.2	44.7	40.2	35.7	
5 × 3½	⅜	1.51	6.10			36.3	35.1	33.9	31.5	29.1	26.6	24.2	21.8	19.4		
5 × 3	¾	1.42	11.36			67.1	64.7	62.3	57.5	52.7	47.8	43.0	38.2	33.4		
5 × 3	5/16	1.26	4.80		28.8	27.8	26.7	25.5	23.2	21.0	18.7	16.4	14.1			
4½ × 3	¾	1.43	10.46		62.8	61.8	59.6	57.4	53.0	48.6	44.2	39.8	35.4			
4½ × 3	5/16	1.31	4.50		27.0	26.2	25.2	24.2	22.1	20.1	18.0	16.0	13.9			
4 × 3½	¾	1.24	10.46		62.8	60.5	58.0	55.4	50.4	45.3	40.2	35.2	30.1			
4 × 3½	5/16	1.26	4.50		27.0	26.1	25.0	24.0	21.8	19.7	17.5	15.4	13.2			
4 × 3	⅝	1.23	7.96		47.8	46.0	44.1	42.1	38.2	34.3	30.4	26.5	22.6			
4 × 3	5/16	1.27	4.18		25.1	24.3	23.3	22.3	20.3	18.4	16.4	14.4	12.4			
3½ × 3	⅝	1.06	7.34		43.3	41.2	39.1	37.1	32.9	28.7	24.6	20.4				
3½ × 3	5/16	1.10	3.86		22.9	21.9	20.8	19.7	17.6	15.5	13.4	11.3				
3½ × 2½	9/16	1.10	6.26		37.1	35.4	33.7	32.0	28.6	25.1	21.7	18.3				
3½ × 2½	¼	1.12	2.88		17.1	16.4	15.6	14.8	13.2	11.7	10.1	8.6				
3 × 2½	9/16	.93	5.68	34.2	32.9	31.1	29.2	27.4	23.7	20.1	16.4					
3 × 2½	¼	.95	2.62	15.7	15.2	14.4	13.6	12.7	11.1	9.4	7.8					
3 × 2	½	.92	4.50	27.0	25.9	24.4	23.0	21.5	18.6	15.7	12.8					
3 × 2	¼	.93	2.38	14.3	13.9	13.1	12.3	11.5	10.0	8.4	6.9					
2¼ × 1½	5/16	.70	2.14	12.6	11.6	10.6	9.7	8.8	7.1							
2¼ × 1½	3/16	.72	1.34	7.9	7.4	6.8	6.3	5.7	4.6							

Strains per square inch;
12,000 lbs. for lengths of 30 radii and under.
$13,500-50\frac{l}{r}$ for lengths over 30 radii.



SAFE LOADS FOR PASSAIC STEEL ANGLES,
USED AS STRUTS OR COLUMNS, IN TONS OF 2000 LBS.

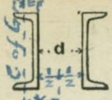
Four Angles, placed back to back, ½" apart.

EQUAL LEGS.
SQUARE ENDS.

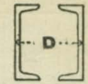
Size of Angles, in inches.	Thickness of Angles, inches.	Least Radius of Gyration, inches.	Area of Section, sq. inches.	Unsupported length of Column, in feet.													
				3	4	5	6	7	8	10	12	14	16	18	20	22	24
6 × 6	7/8	2.83	40.12					241	237	229	220	211	203	194	186	177	169
6 × 6	3/8	2.66	17.44					104	102	98.1	94.1	90.2	86.3	82.3	78.4	74.4	70.5
5 × 5	3/4	2.38	28.44				171	167	164	157	149	142	135	128	121	113	106
5 × 5	3/8	2.27	14.44				86.1	84.2	82.3	78.5	74.7	70.8	67.0	63.2	59.4	55.4	51.7
4 × 4	13/16	2.03	24.44			147	143	139	136	129	121	114	107	99.6	92.4	85.2	77.9
4 × 4	5/16	1.85	9.60			57.6	55.5	53.9	52.4	49.2	46.1	43.0	39.9	36.8	33.6	30.5	27.4
3½ × 3½	5/8	1.70	15.92		95.5	93.5	90.7	87.9	85.1	79.4	73.8	68.2	62.6	56.9	51.3	45.6	
3½ × 3½	5/16	1.65	8.36		50.2	48.8	47.3	45.8	44.2	41.2	38.2	35.1	32.1	29.0	26.0		
3 × 3	5/8	1.59	14.24		85.4	82.8	80.1	77.4	74.7	69.3	63.9	58.5	53.2	47.8	42.4		
3 × 3	1/4	1.43	5.76		34.0	32.8	31.6	30.4	29.2	26.7	24.3	21.9	19.5	17.1			
2½ × 2½	1/2	1.31	9.24	55.5	53.9	51.8	49.7	47.6	45.4	41.2	37.0	32.7	28.5				
2½ × 2½	1/4	1.25	4.76	28.6	27.6	26.5	25.3	24.1	23.0	20.7	18.3	16.0	13.7				
2¼ × 2¼	1/2	1.25	8.44	50.6	48.9	47.0	45.0	42.9	40.9	36.9	32.8	28.8	24.5				
2¼ × 2¼	3/16	1.12	3.24	19.2	18.4	17.5	16.6	15.8	14.9	13.2	11.5	9.8					
2 × 2	1/2	1.15	7.44	44.4	42.5	40.5	38.6	36.7	34.7	30.8	27.0	23.1					
2 × 2	3/16	1.03	2.84	16.7	15.9	15.0	14.2	13.4	12.6	10.9	9.3						

Strains per square inch;
12,000 lbs. for lengths of 30 radii and under.
$13{,}500-50\frac{l}{r}$ for lengths over 30 radii.

Rad. of Gyration = $\sqrt{x^2+r^2}$ $x = E + \frac{1}{2}d$ r = Rad. of Gyr. of [par. to bac



SAFE LOADS FOR PASSAIC STEEL LATTICED CHANNEL COLUMNS, SQUARE ENDS, IN TONS OF 2000 LBS.,

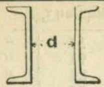


For the following unsupported lengths of columns.

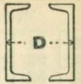
Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii and under. $15{,}000-57\frac{l}{r}$ for lengths over 50 radii.

Depth of Channels, inches.	Weight of each Channel, lbs. per ft.	**d**, inches.	**D**, inches.	Area of Section, sq. inches.	Least Radius of Gyration, inches.	8 ft. or less.	9 ft.	10 ft.	11 ft.	12 ft.	14 ft.	16 ft.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft	28 ft.
5	6	2.8	4.6	3.52	1.93	21	21	20	20	19	18	16	15	14				
"	7	2.7	4.5	4.11	1.86	25	24	23	23	22	20	19	17	16				
"	8	2.7	4.4	4.70	1.82	28	27	26	25	24	23	21	19	17				
"	9	2.6	4.8	5.18	1.93	31	31	30	29	28	26	24	23	21				
"	10	2.5	4.8	5.77	1.89	34	34	33	32	31	29	27	24	22				
"	12	2.3	4.6	6.95	1.82	42	41	39	38	35	32	30	28	27				
6	8	3.5	5.6	4.70	2.33	28	28	28	28	27	26	24	23	22	20	19		
"	9	3.4	5.4	5.29	2.28	32	32	32	31	30	29	27	25	24	22	20		
"	10	3.3	5.3	5.88	2.23	35	35	35	34	33	31	30	28	26	24	23		
"	12	3.2	5.8	6.96	2.32	42	42	42	41	40	38	36	34	31	29	27		
"	13	3.1	5.7	7.55	2.28	45	45	45	44	42	40	37	35	33	31	28		
"	15	2.9	5.5	8.72	2.21	52	52	52	51	49	47	44	41	39	36	33		
"	17	2.8	5.9	9.70	2.28	58	58	58	57	56	53	50	47	44	41	38		
"	18	2.7	5.9	10.3	2.26	62	62	62	60	59	56	53	50	46	43	40		
"	20	2.6	5.8	11.5	2.22	69	69	68	66	65	61	58	54	51	47	44		
7	9	4.3	6.3	5.22	2.70	31	31	31	31	31	30	28	27	26	24	23	22	20
"	10	4.1	6.1	5.81	2.64	35	35	35	35	35	33	32	30	29	27	26	24	23
"	12	4.0	6.0	6.98	2.55	42	42	42	42	41	39	37	35	33	32	30	28	26
"	13	4.0	6.4	7.60	2.69	46	46	46	46	45	43	41	39	37	36	34	32	30
"	15	3.7	6.2	8.78	2.60	53	53	53	53	52	50	47	45	43	40	38	36	33
"	17	3.6	6.1	9.95	2.53	60	60	60	60	58	55	53	50	48	45	42	40	37

The channels must be latticed together to ensure unity of action, and separated not less than the distances **d** or **D** respectively.



SAFE LOADS FOR PASSAIC STEEL LATTICED CHANNEL COLUMNS, SQUARE ENDS, IN TONS OF 2000 LBS.,

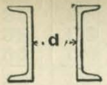


For the following unsupported lengths of columns.

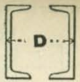
Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii and under. $15{,}000 - 57\frac{l}{r}$ for lengths over 50 radii.

Depth of Channels, inches.	Wgt. of each Channel, lbs. per ft.	**d**, inches.	**D**, inches.	Area of Section, sq. inches.	Least Radius of Gyration, inches.	12 ft. or less.	14 ft.	16 ft.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	32 ft.	36 ft.	40 ft.
8	10	5.0	7.1	6.00	3.08	36	36	35	33	32	31	29	28	26	25			
"	11	4.9	6.9	6.59	3.01	40	39	37	36	34	33	31	30	28	27			
"	12	4.8	6.8	7.18	2.96	43	42	40	39	37	35	34	32	31	29			
"	13	4.9	7.2	7.60	3.07	46	45	43	42	40	38	37	35	34	32			
"	15	4.7	7.0	8.78	2.97	54	52	50	48	46	44	42	39	37	35			
"	17	4.5	6.8	9.97	2.90	60	58	55	53	51	48	46	44	41	39			
9	13	5.7	7.9	7.60	3.46	46	46	45	44	42	41	39	38	36	35	33	30	
"	14	5.6	7.8	8.19	3.40	49	49	48	45	43	42	41	39	37	36	35	31	
"	15	5.5	7.7	8.79	3.36	53	53	51	49	47	45	43	42	41	39	37	34	
"	16	5.5	8.1	9.40	3.48	56	56	56	54	52	50	49	47	45	43	41	38	
"	18	5.3	8.0	10.6	3.40	64	64	62	60	58	56	54	52	50	47	45	41	
"	21	5.1	7.7	12.4	3.30	74	74	72	69	67	64	62	59	57	54	51	46	
10	15	6.3	8.9	8.80	3.89	53	53	53	53	51	49	48	46	45	43	42	38	35
"	17	6.1	8.7	10.1	3.77	60	60	60	59	57	55	54	52	50	48	46	43	39
"	18	6.0	8.6	10.7	3.73	64	64	64	63	61	59	57	55	53	51	49	45	41
"	20	6.0	8.9	11.8	3.81	71	71	71	69	67	65	63	61	59	57	55	50	46
"	25	5.7	8.6	14.7	3.65	88	88	88	85	82	80	77	74	71	69	66	60	55
"	30	5.4	8.4	17.6	3.53	106	106	105	101	98	94	91	87	84	81	77	70	64

The channels must be latticed together to ensure unity of action, and separated not less than the distances **d** or **D** respectively.



SAFE LOADS FOR PASSAIC STEEL LATTICED CHANNEL COLUMNS, SQUARE ENDS, IN TONS OF 2000 LBS.,



For the following unsupported lengths of columns.

Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii and under. $15{,}000 - 57\frac{l}{r}$ for lengths over 50 radii.

Depth of Channels, Inches.	Weight of each Channel, lbs. per foot.	**d**, Inches.	**D**, Inches.	Area of Section, sq. ins.	Least Radius of Gyration, Inches.	18 ft. or less.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	32 ft.	36 ft.	40 ft.	44 ft.	48 ft.	52 ft.
12	20	7.7	10.4	11.8	4.59	71	71	69	67	66	64	62	60	57	53	49	46	
"	23	7.4	10.2	13.6	4.47	82	81	79	77	75	73	71	69	64	60	56	52	
"	25	7.3	10.0	14.8	4.39	89	88	86	83	81	79	76	74	69	65	60	56	
"	27	7.4	10.5	15.8	4.54	95	95	93	90	88	85	83	81	76	71	66	61	
"	30	7.1	10.2	17.6	4.42	106	105	102	100	97	94	91	89	83	78	73	67	
"	33	7.0	10.1	19.4	4.34	116	115	112	109	106	103	100	97	91	85	79	72	
"	35	6.9	10.0	20.6	4.29	123	122	119	116	112	109	105	102	95	89	83	76	
15	33	9.5	12.7	19.4	5.64	116	116	116	116	114	111	109	107	102	98	93	88	84
"	35	9.3	12.5	20.6	5.53	124	124	124	124	121	118	116	113	108	103	98	93	88
"	38	9.2	12.3	22.4	5.45	135	135	135	135	134	131	128	125	119	113	108	102	96
"	40	9.3	12.8	23.6	5.62	142	142	142	142	140	137	134	131	126	120	114	108	103
"	45	9.0	12.6	26.6	5.48	160	160	160	160	156	153	149	146	139	133	126	120	113
"	50	8.8	12.4	29.6	5.38	178	178	178	177	173	169	166	162	154	147	139	131	124

The channels must be latticed together to ensure uniformity of action, and must be separated not less than the distances **d** and **D** respectively.

SAFE LOADS FOR PASSAIC STEEL I BEAMS USED AS STRUTS OR COLUMNS,

SQUARE ENDS.

Beams Supported against Yielding Sideways.

Depth of Beam.	20″	20″	15″	15″	15″	12″	12″	12″	10″	10″	9″	9″	8″	8″	7″	7″	6″	6″	5″	5″	4″	4″
Weight per Foot, in Lbs.	80	65	60	50	42	55	40	31½	33	25	27	21	22	18	20	15	15	12	13	9¾	7½	6
Unsupported L'gth, Ft.	Safe Load, in Tons of 2000 Lbs.																					
4																			23	17	13	11
6															34	26	26	22	22	17	12	10
8											47	37	38	31	34	26	25	21	21	16	12	9.4
10									58	44	47	37	38	31	33	25	24	20	20	15	11	8.8
12						97	71	56	57	43	46	36	36	30	31	24	23	19	19	14	10	8.1
14			106	88	74	94	69	55	56	42	45	35	35	29	30	23	22	18	18	14	9.2	7.4
16			105	88	74	92	68	54	54	41	43	34	34	28	29	22	21	17	17	13		
18	141	115	103	86	73	90	67	53	53	40	42	33	33	27	28	22	20	16	16	12		
20	140	114	101	85	72	88	65	52	51	38	41	32	32	26	27	21	19	16				
22	138	113	100	83	70	86	64	51	50	37	39	31	31	25	25	20	18	15				
24	136	111	98	82	69	84	62	50	48	36	38	30	29	24	24	19						
28	132	108	94	79	67	80	59	47	45	34	36	28	27	22								
32	128	105	91	76	64	76	57	45	43	32	33	26										
36	125	102	87	73	62	72	54	43	40	30												
40	121	99	84	70	59	68	51	40														

Strains per square inch:

12,000 lbs. for lengths of 30 radii and under.

$13{,}500-50\frac{l}{r}$ for lengths over 30 radii.

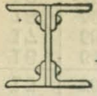
SAFE LOADS FOR PASSAIC STEEL I BEAMS USED AS STRUTS OR COLUMNS,

SQUARE ENDS, Beams not Supported against Yielding Sideways.

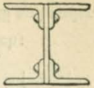
Depth of Beam.	20″	20″	15″	15″	15″	12″	12″	12″	10″	10″	9″	9″	8″	8″	7″	7″	6″	6″	5″	5″	4″	4″
W'g't per Foot, Lbs.	80	65	60	50	42	55	40	31½	33	25	27	21	22	18	20	15	15	12	13	9¾	7½	6
Unsup't'd L'gth, Ft.	Safe Load, in Tons of 2000 Lbs.																					
2	141	115	106	88	74	97	71	56	58	44	47	37	38	31	34	26	26	21	23	17	13	9.9
3	141	114	106	88	73	97	71	55	58	43	47	36	37	30	33	25	24	20	21	16	11	8.7
4	135	109	102	84	70	93	68	53	55	40	45	34	35	28	31	23	23	18	19	14	10	7.6
5	130	104	97	81	66	89	65	50	52	38	42	32	33	26	29	22	21	17	18	13	9.0	6.4
6	124	99	93	77	63	85	62	47	50	36	40	30	31	24	27	20	19	16	16	12	7.8	5.3
7	118	94	89	73	59	81	59	44	47	34	38	28	29	23	25	19	18	14	15	11	6.6	4.1
8	112	89	85	70	56	78	56	42	44	32	36	26	27	21	24	17	16	13	13	9.2	5.4	3.0
9	106	84	81	66	53	74	53	39	42	29	33	24	25	19	22	16	14	11	11	7.9	4.3	
10	100	79	77	62	49	70	50	36	39	27	31	22	23	17	20	14	13	9.6	9.8	6.6		
11	94	74	73	59	46	66	47	34	36	25	29	20	21	15	18	12	11	8.1	8.2	5.3		
12	88	70	69	55	42	62	44	31	34	23	27	18	19	14	16	11	9.5	6.6				
13	82	65	65	51	39	58	41	28	31	21	25	16	18	12	14	9.3	7.9					
14	76	60	61	48	35	55	38	25	28	18	22	15	16	10	13	7.7						
15	70	55	57	44	32	51	35	23	26	16	20	13	14	8.2	11							
16	64	50	53	40	29	47	32	20	23	14	18	11	12									
17	58	45	49	37	25	43	29	17	21	12	16											
18	52	40	45	33	22	39	27	15	18													
19	46	35	41	29	18	35	24	12														
20	40	30	37	26	15	31	21															

Strains per square inch:

$\begin{cases} \text{12,000 lbs. for lengths of 30 radii and under.} \\ 13{,}500-50\frac{l}{r} \text{ for lengths over 30 radii.} \end{cases}$



SAFE LOADS FOR PASSAIC STEEL PLATE AND ANGLE COLUMNS, SQUARE ENDS, IN TONS OF 2000 LBS.



For the following unsupported lengths of columns.

Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii or under; $15,000 - 57\,\frac{l}{r}$ for lengths over 50 radii.

Width of Plate, Inches.	Size of Angles, Inches.	Thickness of Plate and Angles, Ins.	Weight of Column, lbs. per foot.	Area of Section, sq. ins.	Least Radius of Gyration, Inches.	6 ft. or less.	7 ft.	8 ft.	9 ft.	10 ft.	11 ft.	12 ft.	14 ft.	16 ft.	18 ft.	20 ft.	22 ft.
6	3×2½	¼	22.9	6.74	1.24	39	37	35	34	32	30	28	24	21			
6		5/16	29.0	8.52	1.26	50	48	45	43	41	39	36	32	27			
6		7/16	39.8	11.71	1.34	70	67	64	61	58	55	52	46	40			
6		½	44.2	13.00	1.38	78	75	72	69	65	62	59	52	46			
7	3½×2½	¼	25.5	7.51	1.46	45	44	42	41	39	37	35	32	28	25		
7		5/16	32.1	9.43	1.49	56	56	54	52	49	47	45	41	36	32		
7		7/16	44.1	12.98	1.56	78	77	74	71	69	66	63	57	52	46		
7		½	49.3	14.50	1.60	87	87	84	81	77	74	71	65	58	52		
8	4 × 3	5/16	36.9	10.86	1.67	65	65	64	62	60	57	55	51	46	42	37	33
8		⅜	44.6	13.12	1.69	79	79	77	74	72	69	66	61	56	51	45	40
8		7/16	50.9	14.98	1.72	90	90	89	86	83	80	77	71	65	59	53	47
8		½	58.6	17.24	1.75	103	103	102	99	95	92	89	82	76	69	63	56
8		9/16	66.3	19.50	1.78	117	117	116	112	109	105	101	94	86	79	71	64
8		⅝	71.1	20.92	1.82	126	126	126	122	118	114	110	102	95	87	79	71

SAFE LOADS FOR PASSAIC STEEL PLATE AND ANGLE COLUMNS, SQUARE ENDS, IN TONS OF 2000 LBS.,

For the following unsupported lengths of columns.

Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii or under. $15{,}000 - 57\frac{l}{r}$ for lengths over 50 radii.

Width of Plate, ins.	Size of Angles, ins.	Thickness of Plate and Angles, ins.	Weight of Column, lbs. per ft.	Area of Section, Square ins.	Least Radius of Gyrat'n, ins.	9 ft. or less.	10 ft.	11 ft.	12 ft.	14 ft.	16 ft.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.
9	$4\frac{1}{2} \times 3$	$\frac{5}{16}$	40.2	11.81	1.90	70	67	65	63	59	55	50	46	42	38			
9		$\frac{3}{8}$	48.4	14.22	1.93	84	82	79	76	71	66	61	56	51	46			
9		$\frac{7}{16}$	55.4	16.30	1.97	97	94	91	88	83	77	71	66	60	54			
9		$\frac{1}{2}$	63.7	18.74	2.01	112	109	106	103	96	90	83	77	71	64			
9		$\frac{9}{16}$	72.0	21.18	2.04	127	123	120	116	109	102	95	88	81	74			
9		$\frac{5}{8}$	77.6	22.83	2.08	137	133	130	126	118	111	104	96	89	81			
10	5×3	$\frac{5}{16}$	43.3	12.73	2.13	76	75	73	71	67	63	59	55	51	46	42		
10		$\frac{3}{8}$	52.2	15.35	2.17	92	91	89	86	81	77	72	67	62	57	52		
10		$\frac{7}{16}$	59.9	17.62	2.21	106	105	102	100	94	89	83	78	72	67	61		
10		$\frac{1}{2}$	68.8	20.24	2.25	122	121	118	115	109	103	96	90	84	78	72		
10		$\frac{9}{16}$	76.0	22.35	2.29	134	134	131	128	121	114	108	101	94	88	81		
10		$\frac{5}{8}$	84.9	24.97	2.34	150	150	147	143	136	129	121	114	106	99	92		
12	6×4	$\frac{3}{8}$	64.4	18.94	2.51	114	114	114	111	106	101	96	90	85	80	75	70	65
12		$\frac{7}{16}$	75.4	22.17	2.55	133	133	133	130	124	118	112	106	100	94	89	83	77
12		$\frac{1}{2}$	86.5	25.44	2.60	152	152	152	151	144	138	131	125	118	112	105	98	92
12		$\frac{9}{16}$	97.5	28.67	2.64	172	172	172	171	164	156	149	142	134	127	120	112	105
12		$\frac{5}{8}$	104.9	30.94	2.69	186	186	186	186	178	170	163	155	147	139	131	123	116
12		$\frac{11}{16}$	116.2	34.17	2.73	205	205	205	205	197	188	180	171	163	154	146	138	129
12		$\frac{3}{4}$	127.3	37.44	2.78	224	224	224	224	216	208	199	190	181	172	163	154	145

SAFE LOADS FOR PASSAIC STEEL PLATE AND ANGLE COLUMNS, SQUARE ENDS, IN TONS OF 2000 LBS.,

For the following unsupported lengths of columns.

Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii or under. $15{,}000 - 57\frac{l}{r}$ for lengths over 50 radii.

Section of Column.	Thickness of Cover Plates, ins.	Weight of Column, lbs. per ft.	Area of Section, square ins.	Least Radius of Gyrat'n, ins.	12 ft. or less.	14 ft.	16 ft.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	32 ft.	36 ft.	40 ft.
1 Plate 12″ × 3/4″ 4 Angles 6″ × 4″ × 1/2″ 2 Cover Plates, 13″ wide.	1/2	140.9	41.44	2.98	248	244	235	225	216	206	197	187	178	168	159	140	
	9/16	146.5	43.07	3.00	259	255	245	235	225	215	205	196	186	176	166	146	
	5/8	152.0	44.69	3.04	268	265	255	245	235	225	215	205	194	184	174	154	
	11/16	157.5	46.32	3.07	277	276	266	255	245	234	224	213	203	192	182	161	
	3/4	163.0	47.94	3.10	288	286	275	265	254	243	233	222	212	201	190	169	
	13/16	168.5	49.57	3.12	298	296	285	274	263	252	241	231	220	209	198	176	
	7/8	174.0	51.19	3.14	307	306	295	284	273	261	250	239	228	217	205	183	
	15/16	179.6	52.82	3.16	317	316	305	293	282	270	259	247	236	224	213	190	
	1	185.1	54.44	3.18	328	327	315	303	292	280	268	256	244	232	221	197	173
	1 1/16	190.6	56.07	3.19	337	337	325	313	301	289	277	265	253	241	229	205	178
	1 1/8	196.2	57.69	3.22	345	345	334	322	310	297	285	273	261	249	236	212	188
	1 3/16	201.7	59.32	3.23	356	356	345	333	321	308	296	283	270	257	244	219	194
	1 1/4	207.2	60.94	3.25	365	365	354	341	328	316	303	290	277	264	252	226	200
	1 5/16	212.8	62.57	3.26	375	375	364	351	338	325	312	299	286	273	260	234	208
	1 3/8	218.3	64.19	3.27	385	385	375	361	348	334	321	307	294	280	267	240	213
	1 7/16	223.8	65.82	3.29	395	395	384	371	357	342	330	316	302	289	275	248	221
	1 1/2	229.3	67.44	3.30	405	405	394	380	366	353	338	324	310	296	282	254	226

SAFE LOADS FOR PASSAIC STEEL PLATE AND ANGLE COLUMNS, SQUARE ENDS, IN TONS OF 2000 LBS.,

For the following unsupported lengths of columns.

Section of Column.	Thickness of Cover Plates, ins.	Weight of Column, lbs. per ft.	Area of Section, square ins.	Least Radius of Gyrat'n, ins.	Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii or under. $15{,}000 - 57\frac{l}{r}$ for lengths over 50 radii.												
					14 ft. or less.	16 ft.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	32 ft.	36 ft.	40 ft.	44 ft.
1 Plate 14″ × ¾″ 4 Angles 6″ × 4″ × ¾″ 2 Cover Plates, 15″ wide.	½	183.2	53.94	3.25	324	313	302	290	279	268	256	245	234	222	200	177	
	9/16	189.8	55.82	3.30	335	326	314	300	291	280	268	256	245	233	210	187	
	⅝	196.1	57.69	3.33	346	338	326	314	302	291	279	267	255	243	220	196	
	11/16	202.6	59.57	3.37	357	350	338	326	314	302	290	277	265	253	229	205	
	¾	208.8	61.44	3.40	368	363	351	338	326	313	301	288	276	263	238	213	
	13/16	215.3	63.32	3.44	380	374	361	349	336	324	311	298	286	273	248	223	
	⅞	221.6	65.19	3.46	391	386	373	360	347	334	321	308	296	283	257	231	
	15/16	228.1	67.07	3.49	402	398	385	372	358	345	332	319	306	293	266	240	
	1	234.4	68.94	3.52	413	409	396	382	369	356	342	329	315	302	275	249	222
	1 1/16	240.8	70.82	3.54	425	421	407	394	380	366	353	339	325	312	285	257	230
	1⅛	247.1	72.69	3.56	436	433	419	405	391	377	363	349	335	322	294	266	238
	1 3/16	253.6	74.57	3.58	447	445	431	417	402	388	374	360	346	331	303	274	246
	1¼	259.9	76.44	3.60	458	457	443	428	414	399	385	370	356	341	312	283	254
	1 5/16	266.3	78.32	3.62	470	469	454	439	424	410	395	380	365	350	320	291	261
	1⅜	272.6	80.19	3.64	481	481	466	451	436	421	405	390	375	360	330	299	269
	1 7/16	279.1	82.07	3.66	492	492	478	463	447	432	416	401	385	370	339	308	277
	1½	285.4	83.94	3.68	503	503	489	473	458	442	426	411	395	380	348	317	286

SAFE LOADS FOR PASSAIC STEEL CHANNEL COLUMNS, SQUARE ENDS, IN TONS OF 2000 LBS.,

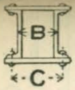
$B = 3\frac{7}{8}''$

$C = 5\frac{3}{4}''$

For the following unsupported lengths of columns.

Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii and under. $15{,}000 - 57\frac{l}{r}$ for lengths over 50 radii.

Designation.	Weight of each Channel, lbs. per ft.	Thickness of Cover Plates, ins.	Weight of Column, lbs. per ft.	Area of Section, square ins.	Least Radius of Gyrat'n, ins.	10 ft. or less.	11 ft.	12 ft.	13 ft.	14 ft.	16 ft.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.
6″ Channel Column; 2 channels 6″ deep and 2 cover plates 8″ wide.	8	$\frac{1}{4}$	29.6	8.70	2.31	52	51	49	48	47	44	42	39	37	34	32	29
	10	$\frac{1}{4}$	33.6	9.88	2.27	59	57	55	53	51	50	47	44	41	38	35	32
	"	$\frac{5}{16}$	37.0	10.88	2.27	65	63	62	61	59	55	52	49	45	42	39	36
	"	$\frac{3}{8}$	40.4	11.88	2.27	71	69	67	66	64	61	57	53	50	46	43	39
	12	$\frac{3}{8}$	44.1	12.96	2.35	78	76	74	72	71	67	63	59	55	51	47	43
	"	$\frac{7}{16}$	47.5	13.96	2.35	84	81	79	77	75	71	67	63	58	54	50	46
	"	$\frac{1}{2}$	50.9	14.96	2.35	90	87	84	82	80	76	71	67	62	58	53	49
	15	$\frac{1}{2}$	56.8	16.72	2.28	100	98	95	92	90	85	80	75	70	65	60	55
	"	$\frac{9}{16}$	60.3	17.72	2.28	106	104	101	98	95	89	84	79	73	68	63	58
	"	$\frac{5}{8}$	63.7	18.72	2.28	112	109	106	103	100	95	89	84	78	73	67	62
	17	$\frac{5}{8}$	67.0	19.70	2.38	118	116	113	110	107	102	96	91	85	80	74	69
	"	$\frac{11}{16}$	70.4	20.70	2.37	124	121	119	116	113	107	101	95	90	84	78	72
	"	$\frac{3}{4}$	73.8	21.70	2.37	130	127	124	121	118	112	106	100	94	88	82	76
	"	$\frac{13}{16}$	77.2	22.70	2.37	136	132	128	125	122	116	110	104	97	91	85	79
	"	$\frac{7}{8}$	80.6	23.70	2.36	142	138	134	131	128	122	115	108	102	95	89	82
	"	$\frac{15}{16}$	84.0	24.70	2.36	148	144	141	137	134	127	120	113	106	99	92	85
	"	1	87.4	25.70	2.36	154	150	147	144	140	132	124	117	110	102	95	88



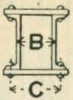
SAFE LOADS FOR PASSAIC STEEL CHANNEL COLUMNS, B=$4\frac{7}{8}''$

SQUARE ENDS, IN TONS OF 2000 LBS., C=$6\frac{3}{4}''$

For the following unsupported lengths of columns.

Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii and under. $15{,}000-57\frac{l}{r}$ for lengths over 50 radii.

Designation.	Weight of each Channel, lbs. per ft.	Thickness of Cover Plates, ins.	Weight of Column, lbs. per ft.	Area of Section, Square ins.	Least Radius of Gyrat'n, ins.	11 ft. or less.	12 ft.	13 ft.	14 ft.	16 ft.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	32 ft.
7″ Channel Column: 2 Channels 7″ deep, and 2 Cover Plates 9″ wide.	9	$\frac{1}{4}$	33.1	9.72	2.71	58	57	56	54	52	50	47	45	43	40	38	36	34
	9	$\frac{5}{16}$	36.9	10.84	2.71	65	63	61	60	57	55	53	50	47	45	42	39	37
	13	$\frac{5}{16}$	45.0	13.23	2.76	79	77	75	74	71	68	65	62	59	56	53	50	47
	13	$\frac{3}{8}$	48.8	14.35	2.75	86	84	82	80	77	74	71	67	64	60	57	53	50
	13	$\frac{7}{16}$	52.6	15.48	2.73	93	92	90	89	85	81	77	73	69	66	62	58	54
	13	$\frac{1}{2}$	56.4	16.60	2.73	100	98	96	94	90	86	82	78	74	70	66	62	58
	17	$\frac{1}{2}$	64.4	18.95	2.66	114	112	109	107	102	98	93	88	83	79	74	69	64
	17	$\frac{9}{16}$	68.3	20.08	2.65	122	119	116	113	108	103	98	93	88	83	78	73	68
	17	$\frac{5}{8}$	72.1	21.20	2.65	127	124	122	119	114	109	103	98	93	87	82	76	71
	17	$\frac{11}{16}$	75.9	22.33	2.65	134	131	129	126	120	114	109	103	97	92	86	81	75
	17	$\frac{3}{4}$	79.7	23.45	2.64	140	136	133	130	124	118	112	106	100	94	88	82	76
	17	$\frac{13}{16}$	83.6	24.58	2.64	148	145	142	139	132	125	119	113	107	100	94	88	82
	17	$\frac{7}{8}$	87.4	25.70	2.64	154	150	147	144	138	131	125	118	112	105	99	92	86
	17	$\frac{15}{16}$	91.2	26.83	2.64	161	157	154	150	143	137	130	123	116	110	103	96	89
	17	1	95.0	27.95	2.63	168	164	161	157	150	143	136	129	121	114	107	100	93



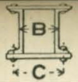
SAFE LOADS FOR PASSAIC STEEL CHANNEL COLUMNS, SQUARE ENDS, IN TONS OF 2000 LBS.,

B = $5\frac{7}{8}''$
C = $7\frac{3}{4}''$

For the following unsupported lengths of columns.

Allowable strains per square inch: $\begin{cases} 12{,}000 \text{ lbs. for lengths of 50 radii and under.} \\ 15{,}000-57\frac{l}{r} \text{ for lengths over 50 radii.} \end{cases}$

Designation.	Weight of each Channel, lbs. per ft.	Thickness of Cover Plates, ins.	Weight of Column, lbs. per ft.	Area of Section, Square ins.	Least Radius of Gyration, inches.	12 ft. or less.	13 ft.	14 ft.	16 ft.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	32 ft.	34 ft.	36 ft.
8″ Channel Column; 2 channels 8″ deep and 2 cover plates 10″ wide.	10	$\frac{1}{4}$	37.4	11.00	3.12	66	66	66	64	62	60	57	54	52	49	46	44	41	39
	″	$\frac{5}{16}$	41.7	12.25	3.09	74	74	73	70	67	65	62	59	57	54	51	49	46	43
	13	$\frac{5}{16}$	47.1	13.85	3.14	83	83	83	80	77	74	71	68	65	62	59	56	53	50
	″	$\frac{3}{8}$	51.3	15.10	3.12	91	91	90	87	84	80	77	74	70	67	64	60	57	54
	″	$\frac{7}{16}$	55.6	16.35	3.10	98	98	97	94	90	87	83	79	76	72	69	65	62	58
	″	$\frac{1}{2}$	59.8	17.60	3.08	106	106	105	102	98	94	90	86	82	78	74	70	66	62
	17	$\frac{1}{2}$	67.9	19.97	3.04	120	120	118	114	110	105	101	96	92	87	82	78	73	69
	″	$\frac{9}{16}$	72.1	21.22	3.03	127	127	126	120	115	110	106	101	96	91	87	83	78	73
	″	$\frac{5}{8}$	76.4	22.47	3.02	135	135	133	128	123	117	112	107	102	97	92	87	82	77
	″	$\frac{11}{16}$	80.7	23.72	3.02	142	142	140	135	129	123	118	113	108	102	97	92	86	81
	″	$\frac{3}{4}$	84.9	24.97	3.02	150	150	147	141	136	130	125	119	114	108	103	97	92	86
	″	$\frac{13}{16}$	89.1	26.22	3.00	157	157	155	149	143	137	131	125	119	113	107	101	95	89
	″	$\frac{7}{8}$	93.4	27.47	2.99	165	165	162	156	150	144	137	131	125	118	112	106	99	93
	″	$\frac{15}{16}$	97.6	28.72	2.99	172	172	169	163	156	150	143	136	130	123	117	110	104	97
	″	1	101.9	29.97	2.99	180	180	177	171	164	157	150	143	136	129	122	115	108	101



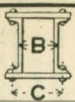
SAFE LOADS FOR PASSAIC STEEL CHANNEL COLUMNS, SQUARE ENDS, IN TONS OF 2000 LBS.,

B = $6\frac{3}{8}''$ C = $8\frac{1}{2}''$

For the following unsupported lengths of columns.

Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii and under. $15{,}000-57\frac{l}{r}$ for lengths over 50 radii.

Designation.	Weight of each Channel, lbs. per ft.	Thickness of Cover Plates, ins.	Weight of Column, lbs. per ft.	Area of Section, square ins.	Least Radius of Gyrat'n, ins.	14 ft. or less.	16 ft.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	32 ft.	36 ft.	40 ft.
9″ Channel Column; 2 channels 9″ deep and 2 cover plates 11″ wide.	13	$\frac{5}{16}$	49.2	14.48	3.40	87	85	82	79	76	73	70	67	65	62	56	50
	″	$\frac{3}{8}$	53.9	15.85	3.38	95	93	90	87	83	80	77	74	71	67	61	55
	16	$\frac{3}{8}$	60.0	17.65	3.43	106	104	100	97	93	90	86	83	79	76	69	62
	″	$\frac{7}{16}$	64.7	19.03	3.41	114	112	108	105	101	98	94	90	86	82	75	67
	″	$\frac{1}{2}$	69.4	20.40	3.41	122	120	115	111	107	103	99	95	91	87	79	71
	21	$\frac{1}{2}$	79.5	23.37	3.33	140	136	131	126	121	117	112	107	103	98	89	79
	″	$\frac{9}{16}$	84.2	24.75	3.32	149	146	140	135	130	125	120	115	110	104	94	84
	″	$\frac{5}{8}$	88.8	26.12	3.31	157	153	148	142	137	131	126	120	115	109	99	88
	″	$\frac{11}{16}$	93.5	27.50	3.30	165	161	156	150	145	139	133	127	121	116	104	92
	″	$\frac{3}{4}$	98.2	28.87	3.30	173	168	163	157	151	145	139	133	127	121	109	97
	″	$\frac{13}{16}$	102.9	30.25	3.29	182	177	171	164	158	151	145	139	132	126	113	101
	″	$\frac{7}{8}$	107.5	31.62	3.29	190	186	180	173	166	160	153	146	139	133	119	105
	″	$\frac{15}{16}$	112.2	33.00	3.28	198	194	187	180	173	166	159	152	145	138	124	110
	″	1	116.9	34.37	3.28	206	202	195	187	180	173	165	158	151	143	129	114
	″	$1\frac{1}{16}$	121.6	35.75	3.27	215	209	202	194	187	179	172	164	157	149	134	119
	″	$1\frac{1}{8}$	126.2	37.12	3.27	224	218	211	203	195	187	179	171	163	155	139	123
	″	$1\frac{3}{16}$	130.9	38.50	3.27	231	227	218	210	201	193	185	177	169	160	144	128
	″	$1\frac{1}{4}$	135.5	39.87	3.27	239	233	227	218	209	201	192	184	175	167	149	132



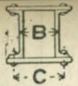
SAFE LOADS FOR PASSAIC STEEL CHANNEL COLUMNS, SQUARE ENDS, IN TONS OF 2000 LBS.,

$\mathbf{B} = 6\frac{3}{4}''$
$\mathbf{C} = 9\frac{3}{8}''$

For the following unsupported lengths of columns.

Allowable strains per square inch: $\begin{cases} 12{,}000 \text{ lbs. for lengths of 50 radii and under.} \\ 15{,}000 - 57\dfrac{l}{r} \text{ for lengths over 50 radii.} \end{cases}$

Designation.	Weight of each Channel, lbs. per ft.	Thickness of Cover Plates, ins.	Weight of Column, lbs. per ft.	Area of Section, Square ins.	Least Radius of Gyration, inches.	15 ft. or less.	16 ft.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	32 ft.	34 ft.	36 ft.	40 ft.
10″ Channel Column; 2 channels 10″ deep and 2 cover plates 12″ wide.	15	$\frac{5}{16}$	56.8	16.7	3.68	100	100	97	94	91	88	85	82	79	76	73	70	64
	〃	$\frac{3}{8}$	61.9	18.2	3.67	109	109	106	103	100	97	93	90	86	83	79	75	68
	20	$\frac{3}{8}$	70.7	20.8	3.70	125	125	121	117	113	110	106	102	98	94	90	87	79
	〃	$\frac{7}{16}$	75.8	22.3	3.69	135	135	130	126	122	118	114	109	105	101	97	93	85
	〃	$\frac{1}{2}$	80.9	23.8	3.68	143	143	139	135	130	126	121	117	113	108	104	99	90
	25	$\frac{1}{2}$	90.8	26.7	3.61	160	160	155	150	145	139	134	129	124	119	114	109	99
	〃	$\frac{9}{16}$	95.9	28.2	3.60	169	169	163	157	152	147	141	136	131	126	121	115	104
	〃	$\frac{5}{8}$	101.0	29.7	3.60	178	178	172	166	160	155	149	143	138	132	126	121	110
	30	$\frac{5}{8}$	110.9	32.6	3.54	196	193	187	180	174	168	162	156	149	143	137	131	119
	〃	$\frac{11}{16}$	116.0	34.1	3.54	205	202	196	189	183	176	169	163	156	150	144	137	124
	〃	$\frac{3}{4}$	121.1	35.6	3.53	214	212	206	199	192	185	178	171	164	157	150	143	129
	〃	$\frac{13}{16}$	126.2	37.1	3.53	223	221	214	207	200	192	185	178	171	163	156	149	135
	〃	$\frac{7}{8}$	131.3	38.6	3.53	232	230	222	215	208	200	192	185	178	170	163	155	140
	〃	$\frac{15}{16}$	136.4	40.1	3.53	240	238	230	223	215	207	199	192	184	176	168	161	145
	〃	1	141.5	41.6	3.52	250	247	239	231	223	215	207	198	190	182	174	166	150
	〃	$1\frac{1}{8}$	151.7	44.6	3.52	268	265	256	247	239	230	222	213	204	196	187	179	162
	〃	$1\frac{1}{4}$	161.9	47.6	3.52	286	283	273	264	255	245	236	227	218	208	199	190	171
	〃	$1\frac{3}{8}$	172.1	50.6	3.51	304	301	290	280	271	261	251	242	232	222	213	203	184
	〃	$1\frac{1}{2}$	182.3	53.6	3.51	322	318	308	297	287	277	266	256	245	235	225	214	193



SAFE LOADS FOR PASSAIC STEEL CHANNEL COLUMNS, SQUARE ENDS, IN TONS OF 2000 LBS.,

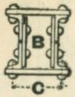
B = $8\frac{3}{8}''$

C = $11\frac{1}{8}''$

For the following unsupported lengths of columns.

Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii and under. $15{,}000 - 57\frac{l}{r}$ for lengths over 50 radii.

Designation.	Weight of each Channel, lbs. per ft.	Thickness of Cover Plates, ins.	Weight of Column, lbs. per ft.	Area of Section, square ins.	Least Radius of Gyrat'n, ins.	18 ft. or less.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	32 ft.	34 ft.	36 ft.	38 ft.	40 ft.	42 ft.
12″ Channel Column; 2 channels 12″ deep and 2 cover plates 14″ wide.	20	$\frac{3}{8}$	75.8	22.30	4.39	134	132	129	125	122	118	115	111	108	104	101	97	94
	"	$\frac{7}{16}$	81.8	24.05	4.36	144	143	140	136	133	129	125	121	117	114	110	106	102
	25	$\frac{7}{16}$	91.9	27.05	4.31	163	160	155	151	147	143	138	134	130	125	121	117	113
	"	$\frac{1}{2}$	97.9	28.80	4.31	173	171	165	160	156	151	147	142	138	133	129	124	120
	30	$\frac{1}{2}$	107.4	31.60	4.36	190	188	183	178	173	168	163	158	153	148	143	138	133
	"	$\frac{9}{16}$	113.4	33.35	4.34	200	197	191	186	181	175	170	165	160	154	149	144	139
	"	$\frac{5}{8}$	119.3	35.10	4.33	211	208	202	197	191	186	181	175	169	164	158	153	147
	"	$\frac{11}{16}$	125.3	36.85	4.31	221	218	212	206	200	194	189	183	177	171	165	159	154
	"	$\frac{3}{4}$	131.3	38.60	4.30	232	228	222	216	210	204	198	192	186	180	174	168	162
	35	$\frac{3}{4}$	141.5	41.60	4.25	250	248	242	235	228	221	214	207	200	193	186	179	172
	"	$\frac{13}{16}$	147.4	43.35	4.25	260	254	247	240	233	227	220	213	206	200	193	186	179
	"	$\frac{7}{8}$	153.3	45.10	4.24	271	265	258	251	243	236	229	222	214	207	200	193	186
	"	$\frac{15}{16}$	159.3	46.85	4.24	281	275	267	260	252	245	238	230	223	215	208	200	193
	"	1	165.2	48.60	4.22	292	286	278	270	262	254	246	238	230	222	214	206	199
	"	$1\frac{1}{8}$	177.2	52.10	4.21	313	306	298	290	281	273	264	256	247	239	231	222	214
	"	$1\frac{1}{4}$	189.0	55.60	4.21	333	326	317	308	299	290	281	272	263	254	245	236	227
	"	$1\frac{3}{8}$	200.9	59.10	4.19	354	346	337	327	318	308	299	289	280	270	261	251	242
	"	$1\frac{1}{2}$	212.7	62.60	4.19	375	366	356	346	336	326	316	306	296	286	276	266	256



SAFE LOADS FOR PASSAIC STEEL CHANNEL COLUMNS, SQUARE ENDS, HEAVY SECTION, IN TONS OF 2000 LBS.,

B = $6\frac{3}{4}''$
C = $9\frac{3}{8}''$

For the following unsupported lengths of columns.

Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii and under. $15{,}000-57\frac{l}{r}$ for lengths over 50 radii.

Designation.	Thickness of Cover Plates, inches.	Weight of Column, lbs. per ft.	Area of Section, square ins.	Least Radius of Gyration, inches.	14 ft. or less.	16 ft.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	32 ft.	34 ft.	36 ft.	40 ft.
10″ Extra Heavy Channel Column; 2 channels 10″ × 30 lbs., 2 cover plates 12″ wide, and 2 web plates 9″ × ¾″	$\frac{3}{4}$	167.0	49.1	3.26	295	286	275	265	255	245	234	224	214	203	193	183	162
	$\frac{13}{16}$	172.1	50.6	3.27	304	295	284	273	263	252	242	231	221	210	200	189	168
	$\frac{7}{8}$	177.2	52.1	3.27	313	304	293	282	271	260	249	238	227	216	205	194	172
	$\frac{15}{16}$	182.3	53.6	3.28	321	312	300	288	277	266	255	244	233	222	211	200	178
	1	187.4	55.1	3.28	331	322	310	299	287	276	264	253	241	230	218	207	184
	$1\frac{1}{8}$	197.6	58.1	3.29	349	340	329	317	305	291	279	267	255	243	231	218	194
	$1\frac{1}{4}$	207.8	61.1	3.30	367	358	345	333	320	307	295	282	269	257	244	231	206
	$1\frac{3}{8}$	218.0	64.1	3.31	385	375	361	348	335	322	309	296	283	270	257	243	217
	$1\frac{1}{2}$	228.2	67.1	3.31	403	393	381	367	353	339	325	311	297	283	269	255	227
	$1\frac{5}{8}$	238.4	70.1	3.32	421	410	396	381	367	353	338	324	310	295	281	267	238
	$1\frac{3}{4}$	248.6	73.1	3.33	439	427	412	397	382	367	352	337	322	308	293	278	248
	$1\frac{7}{8}$	258.8	76.1	3.33	457	445	429	414	398	383	367	352	336	320	305	289	258
	2	269.0	79.1	3.34	475	463	446	430	414	398	382	366	350	334	318	301	269
	$2\frac{1}{8}$	279.2	82.1	3.34	493	481	465	448	431	415	398	382	365	348	331	314	280
	$2\frac{1}{4}$	289.4	85.1	3.35	511	500	484	466	448	431	413	396	378	361	343	326	291

SAFE LOADS FOR PASSAIC STEEL CHANNEL COLUMNS, SQUARE ENDS, HEAVY SECTION, IN TONS OF 2000 LBS.,

B = $8\frac{3}{8}''$ **C** = $11\frac{1}{8}''$

For the following unsupported lengths of columns.

Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii and under. $15{,}000 - 57\frac{l}{r}$ for lengths over 50 radii.

Designation.	Thickness of Cover Plates, inches.	Weight of Column, lbs. per ft.	Area of Section, sq. ins.	Least Radius of Gyration, Inches.	16 ft. or less.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	34 ft.	38 ft.	42 ft.	46 ft.	50 ft.
12″ Extra Heavy Channel Column; 2 Channels 12″ × 35 lbs., 2 cover plates 14″ wide, and 2 web plates 11″ × $\frac{3}{4}$″	$\frac{3}{4}$	197.6	58.1	3.99	349	346	336	326	316	306	296	286	266	246	226	206	186
	$\frac{13}{16}$	203.5	59.9	3.99	359	357	346	336	326	316	306	296	275	254	233	212	191
	$\frac{7}{8}$	209.4	61.6	4.00	370	368	356	345	334	324	313	303	283	262	241	220	199
	$\frac{15}{16}$	215.4	63.4	4.00	381	378	367	357	346	335	324	313	292	270	248	226	204
	1	221.3	65.1	4.00	391	388	377	366	355	344	332	320	298	276	254	232	210
	$1\frac{1}{8}$	233.3	68.6	4.00	411	409	398	387	375	364	352	339	316	292	268	245	222
	$1\frac{1}{4}$	245.1	72.1	4.01	433	431	419	407	395	382	370	358	333	308	283	258	233
	$1\frac{3}{8}$	257.0	75.6	4.01	454	452	439	426	413	400	387	374	348	322	296	270	244
	$1\frac{1}{2}$	269.0	79.1	4.01	476	473	459	445	432	418	405	391	364	337	310	283	256
	$1\frac{5}{8}$	280.8	82.6	4.02	496	493	478	464	450	436	422	408	380	352	324	296	268
	$1\frac{3}{4}$	292.7	86.1	4.02	518	515	501	487	472	457	443	428	398	369	339	309	280
	$1\frac{7}{8}$	304.7	89.6	4.02	538	536	520	504	489	474	458	443	413	382	352	321	291
	2	316.5	93.1	4.02	560	556	541	526	510	494	478	462	430	398	366	334	302
	$2\frac{1}{8}$	328.4	96.6	4.02	580	577	561	545	529	512	496	479	446	413	380	347	314
	$2\frac{1}{4}$	340.4	100.1	4.02	602	598	581	565	547	530	513	495	461	427	393	359	325

SAFE LOADS FOR PASSAIC STEEL Z BAR COLUMNS,

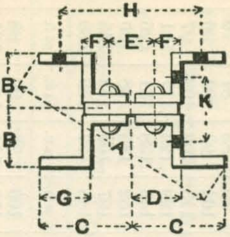
Square Ends, in tons of 2000 lbs., for the following unsupported lengths of columns.

Allowable strains per square inch: { 12,000 lbs. for lengths of 50 radii and under. 15,000 — 57 $\frac{l}{r}$ for lengths over 50 radii.

Designation.	Section of Column.	Thickness of Z Bars and Web Plate, Inches.	Weight of Column, lbs. per foot.	Area of Section, sq. ins.	Least Radius of Gyration, Inches.	8 ft. or less.	9 ft.	10 ft.	11 ft.	12 ft.	13 ft.	14 ft.	16 ft.	18 ft.	20 ft.	22 ft.	24 ft.
6″ Z barcolumn	4 Z bars 3″ deep and 1 Web Plate 6″ wide.	$\frac{1}{4}$	31.9	9.4	1.86	56	55	53	52	50	48	46	43	39	36	32	29
		$\frac{5}{16}$	40.1	11.8	1.91	71	70	68	66	64	61	59	55	51	47	42	38
		$\frac{3}{8}$	46.6	13.8	1.87	83	79	77	74	72	69	67	62	57	53	48	43
		$\frac{7}{16}$	54.7	16.1	1.93	97	94	91	88	85	83	80	74	69	63	57	52
		$\frac{1}{2}$	60.5	17.8	1.89	107	104	100	97	94	91	89	83	75	70	63	57
		$\frac{9}{16}$	68.3	20.1	1.94	121	118	115	110	107	103	100	93	86	80	73	66
						11 ft. or less.	12 ft.	13 ft.	14 ft.	16 ft.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.
8″ Z bar column.	4 Z bars 4″ deep and 1 Web Plate $6\frac{1}{2}$″ wide.	$\frac{1}{4}$	38.4	11.3	2.47	67	66	64	63	59	56	53	50	47	44	41	38
		$\frac{5}{16}$	48.2	14.2	2.52	85	83	81	79	76	72	68	64	60	57	53	49
		$\frac{3}{8}$	58.1	17.1	2.57	103	101	99	97	92	88	83	78	74	69	65	60
		$\frac{7}{16}$	64.5	19.0	2.49	114	111	108	106	100	95	90	85	80	74	69	64
		$\frac{1}{2}$	74.4	21.9	2.54	131	128	125	122	117	111	105	99	93	87	82	76
		$\frac{9}{16}$	83.9	24.8	2.59	149	146	142	139	133	126	120	113	107	100	94	87
		$\frac{5}{8}$	89.4	26.3	2.52	158	155	151	148	140	133	126	119	111	104	97	90
		$\frac{11}{16}$	98.6	29.0	2.57	174	171	166	162	155	147	140	132	125	117	110	102
		$\frac{3}{4}$	108.3	31.9	2.63	191	189	185	181	172	164	156	148	140	131	123	115

Z BAR COLUMN DIMENSIONS,

in inches.



6″ Columns;
4 **Z** bars, $3''-3\frac{1}{16}''$ deep,
1 Web plate $6'' \times$ thickness of **Z** bars.

Diameter of bolt or rivet, $\frac{3}{4}''$	Thickness of Metal.	A	B	C	D	E	F	G	H	K
	$\frac{1}{4}$	$12\frac{3}{4}$	$3\frac{1}{8}$	$5\frac{9}{16}$	$3\frac{1}{8}$	3	$1\frac{5}{8}$	$2\frac{11}{16}$	9	$3\frac{1}{4}$
	$\frac{5}{16}$	$12\frac{13}{16}$	$3\frac{7}{32}$	$5\frac{9}{16}$	$3\frac{1}{8}$	3	$1\frac{5}{8}$	$2\frac{3}{4}$	$8\frac{7}{8}$	$3\frac{3}{8}$
	$\frac{3}{8}$	$12\frac{5}{8}$	$3\frac{3}{16}$	$5\frac{7}{16}$	$3\frac{1}{8}$	3	$1\frac{5}{8}$	$2\frac{11}{16}$	$8\frac{3}{4}$	$3\frac{3}{8}$
	$\frac{7}{16}$	$12\frac{11}{16}$	$3\frac{9}{32}$	$5\frac{7}{16}$	$3\frac{1}{8}$	3	$1\frac{5}{8}$	$2\frac{3}{4}$	$8\frac{5}{8}$	$3\frac{1}{2}$
	$\frac{1}{2}$	$12\frac{7}{16}$	$3\frac{1}{4}$	$5\frac{5}{16}$	$3\frac{1}{8}$	3	$1\frac{5}{8}$	$2\frac{11}{16}$	$8\frac{1}{2}$	$3\frac{1}{2}$
	$\frac{9}{16}$	$12\frac{9}{16}$	$3\frac{11}{32}$	$5\frac{5}{16}$	$3\frac{1}{8}$	3	$1\frac{5}{8}$	$2\frac{3}{4}$	$8\frac{3}{8}$	$3\frac{5}{8}$

8″ Columns;
4 **Z** bars, $4''-4\frac{1}{8}''$ deep,
1 Web plate $6\frac{1}{2}'' \times$ thickness of **Z** bars.

Diameter of bolt or rivet, $\frac{3}{4}''$	Thickness of Metal.	A	B	C	D	E	F	G	H	K
	$\frac{1}{4}$	$14\frac{7}{8}$	$4\frac{1}{8}$	$6\frac{3}{16}$	$3\frac{3}{8}$	$3\frac{1}{4}$	$1\frac{3}{4}$	$3\frac{1}{16}$	$9\frac{3}{4}$	$4\frac{1}{4}$
	$\frac{5}{16}$	15	$4\frac{7}{32}$	$6\frac{3}{16}$	$3\frac{3}{8}$	$3\frac{1}{4}$	$1\frac{3}{4}$	$3\frac{1}{8}$	$9\frac{5}{8}$	$4\frac{3}{8}$
	$\frac{3}{8}$	$15\frac{1}{16}$	$4\frac{5}{16}$	$6\frac{3}{16}$	$3\frac{3}{8}$	$3\frac{1}{4}$	$1\frac{3}{4}$	$3\frac{3}{16}$	$9\frac{1}{2}$	$4\frac{1}{2}$
	$\frac{7}{16}$	$14\frac{11}{16}$	$4\frac{7}{32}$	6	$3\frac{3}{8}$	$3\frac{1}{4}$	$1\frac{3}{4}$	$3\frac{1}{16}$	$9\frac{3}{8}$	$4\frac{7}{16}$
	$\frac{1}{2}$	$14\frac{3}{4}$	$4\frac{5}{16}$	6	$3\frac{3}{8}$	$3\frac{1}{4}$	$1\frac{3}{4}$	$3\frac{1}{8}$	$9\frac{1}{4}$	$4\frac{9}{16}$
	$\frac{9}{16}$	$14\frac{7}{8}$	$4\frac{13}{32}$	6	$3\frac{3}{8}$	$3\frac{1}{4}$	$1\frac{3}{4}$	$3\frac{3}{16}$	$9\frac{1}{8}$	$4\frac{11}{16}$
	$\frac{5}{8}$	$14\frac{1}{2}$	$4\frac{5}{16}$	$5\frac{13}{16}$	$3\frac{3}{8}$	$3\frac{1}{4}$	$1\frac{3}{4}$	$3\frac{1}{16}$	9	$4\frac{5}{8}$
	$\frac{11}{16}$	$14\frac{9}{16}$	$4\frac{13}{32}$	$5\frac{13}{16}$	$3\frac{3}{8}$	$3\frac{1}{4}$	$1\frac{3}{4}$	$3\frac{1}{8}$	$8\frac{7}{8}$	$4\frac{3}{4}$
	$\frac{3}{4}$	$14\frac{11}{16}$	$4\frac{1}{2}$	$5\frac{13}{16}$	$3\frac{3}{8}$	$3\frac{1}{4}$	$1\frac{3}{4}$	$3\frac{3}{16}$	$8\frac{3}{4}$	$4\frac{7}{8}$

SAFE LOADS FOR PASSAIC STEEL Z BAR COLUMNS,

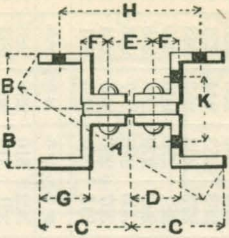
Square Ends, in tons of 2000 lbs., for the following unsupported lengths of columns.

Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii and under. $15{,}000 - 57\frac{l}{r}$ for lengths over 50 radii.

Designation.	Section of Column.	Thickness of Z bars and web plate, inches.	Weight of Column, lbs. per ft.	Area of Section, sq. inches.	Least Radius of Gyration, inches.	13 ft. or less.	14 ft.	16 ft.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	32 ft.	34 ft.	36 ft.
10″ Z bar Column.	4 Z bars 5″ deep and 1 web plate 7″ wide.	$\frac{5}{16}$	53.7	15.8	3.08	95	95	91	88	84	81	77	75	71	66	63	60	56
		$\frac{3}{8}$	64.7	19.0	3.13	114	114	110	106	101	97	93	89	85	81	76	72	68
		$\frac{7}{16}$	75.8	22.3	3.18	134	134	129	124	119	114	109	104	99	94	90	85	80
		$\frac{1}{2}$	83.3	24.5	3.10	147	146	141	135	130	124	119	113	108	102	97	91	86
		$\frac{9}{16}$	94.2	27.7	3.16	166	166	160	154	148	142	135	129	123	117	111	105	99
		$\frac{5}{8}$	105.2	30.9	3.21	185	185	179	172	164	158	151	145	138	131	126	119	113
		$\frac{11}{16}$	111.2	32.7	3.13	196	195	188	181	173	166	159	152	145	138	130	123	116
		$\frac{3}{4}$	121.8	35.8	3.18	215	213	204	198	190	183	174	167	160	152	145	137	130
		$\frac{13}{16}$	132.6	39.0	3.24	234	234	226	218	210	201	193	185	177	169	160	152	144
						16 ft. or less.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	32 ft.	34 ft.	36 ft.	38 ft.	40 ft.
12″ Z bar Column.	4 Z bars 6″ deep and 1 web plate 8″ wide.	$\frac{3}{8}$	72.7	21.4	3.66	128	125	120	116	112	108	104	100	96	92	88	84	80
		$\frac{7}{16}$	85.2	25.0	3.72	150	146	141	137	132	128	123	119	114	109	105	101	96
		$\frac{1}{2}$	97.8	28.8	3.77	173	170	165	160	154	149	144	138	133	128	123	117	112
		$\frac{9}{16}$	106.2	31.2	3.69	187	182	177	171	165	159	153	147	141	135	130	124	118
		$\frac{5}{8}$	118.5	34.8	3.74	209	204	198	191	185	178	172	165	159	152	146	139	133
		$\frac{11}{16}$	130.9	38.5	3.79	231	227	220	214	207	200	193	186	180	172	165	158	151
		$\frac{3}{4}$	137.8	40.5	3.73	243	237	230	222	215	207	200	192	185	177	170	162	155
		$\frac{13}{16}$	149.9	44.1	3.76	265	258	250	242	234	226	218	210	202	194	186	178	170
		$\frac{7}{8}$	162.1	47.7	3.73	286	280	271	262	253	245	236	227	218	209	201	192	183

Z BAR COLUMN DIMENSIONS,

in inches.



10″ Columns;
4 **Z** bars, 5″–5⅛″ deep,
1 Web plate 7″ × thickness of **Z** bars.

Diameter of bolt or rivet, ¾″

Thickness of Metal.	A	B	C	D	E	F	G	H	K
$\frac{5}{16}$	$16\frac{11}{16}$	$5\frac{5}{32}$	$6\frac{9}{16}$	$3\frac{5}{8}$	$3\frac{1}{2}$	$1\frac{7}{8}$	$3\frac{1}{4}$	$10\frac{3}{8}$	$5\frac{5}{16}$
$\frac{3}{8}$	$16\frac{13}{16}$	$5\frac{1}{4}$	$6\frac{9}{16}$	$3\frac{5}{8}$	$3\frac{1}{2}$	$1\frac{7}{8}$	$3\frac{5}{16}$	$10\frac{1}{4}$	$5\frac{7}{16}$
$\frac{7}{16}$	$16\frac{15}{16}$	$5\frac{11}{32}$	$6\frac{9}{16}$	$3\frac{5}{8}$	$3\frac{1}{2}$	$1\frac{7}{8}$	$3\frac{3}{8}$	$10\frac{1}{8}$	$5\frac{9}{16}$
$\frac{1}{2}$	$16\frac{1}{2}$	$5\frac{1}{4}$	$6\frac{3}{8}$	$3\frac{5}{8}$	$3\frac{1}{2}$	$1\frac{7}{8}$	$3\frac{1}{4}$	10	$5\frac{1}{2}$
$\frac{9}{16}$	$16\frac{5}{8}$	$5\frac{11}{32}$	$6\frac{3}{8}$	$3\frac{5}{8}$	$3\frac{1}{2}$	$1\frac{7}{8}$	$3\frac{5}{16}$	$9\frac{7}{8}$	$5\frac{5}{8}$
$\frac{5}{8}$	$16\frac{3}{4}$	$5\frac{7}{16}$	$6\frac{3}{8}$	$3\frac{5}{8}$	$3\frac{1}{2}$	$1\frac{7}{8}$	$3\frac{3}{8}$	$9\frac{3}{4}$	$5\frac{3}{4}$
$\frac{11}{16}$	$16\frac{3}{8}$	$5\frac{11}{32}$	$6\frac{3}{16}$	$3\frac{5}{8}$	$3\frac{1}{2}$	$1\frac{7}{8}$	$3\frac{1}{4}$	$9\frac{5}{8}$	$5\frac{11}{16}$
$\frac{3}{4}$	$16\frac{1}{2}$	$5\frac{7}{16}$	$6\frac{3}{16}$	$3\frac{5}{8}$	$3\frac{1}{2}$	$1\frac{7}{8}$	$3\frac{5}{16}$	$9\frac{1}{2}$	$5\frac{13}{16}$
$\frac{13}{16}$	$16\frac{5}{8}$	$5\frac{17}{32}$	$6\frac{3}{16}$	$3\frac{5}{8}$	$3\frac{1}{2}$	$1\frac{7}{8}$	$3\frac{3}{8}$	$9\frac{3}{8}$	$5\frac{15}{16}$

12″ Columns;
4 **Z** bars, 6″–6⅛″ deep,
1 Web plate 8″ × thickness of **Z** bars.

Diameter of bolt or rivet, ¾″

Thickness of Metal.	A	B	C	D	E	F	G	H	K
$\frac{3}{8}$	$19\frac{1}{16}$	$6\frac{3}{16}$	$7\frac{1}{4}$	$4\frac{1}{8}$	$4\frac{1}{4}$	2	$3\frac{1}{2}$	$11\frac{1}{2}$	$6\frac{3}{8}$
$\frac{7}{16}$	$19\frac{3}{16}$	$6\frac{9}{32}$	$7\frac{1}{4}$	$4\frac{1}{8}$	$4\frac{1}{4}$	2	$3\frac{9}{16}$	$11\frac{3}{8}$	$6\frac{5}{8}$
$\frac{1}{2}$	$19\frac{5}{16}$	$6\frac{3}{8}$	$7\frac{1}{4}$	$4\frac{1}{8}$	$4\frac{1}{4}$	2	$3\frac{5}{8}$	$11\frac{1}{4}$	$6\frac{3}{4}$
$\frac{9}{16}$	$18\frac{7}{8}$	$6\frac{9}{32}$	$7\frac{1}{16}$	$4\frac{1}{8}$	$4\frac{1}{4}$	2	$3\frac{1}{2}$	$11\frac{1}{8}$	$6\frac{9}{16}$
$\frac{5}{8}$	19	$6\frac{3}{8}$	$7\frac{1}{16}$	$4\frac{1}{8}$	$4\frac{1}{4}$	2	$3\frac{9}{16}$	11	$6\frac{11}{16}$
$\frac{11}{16}$	$19\frac{1}{8}$	$6\frac{15}{32}$	$7\frac{1}{16}$	$4\frac{1}{8}$	$4\frac{1}{4}$	2	$3\frac{5}{8}$	$10\frac{7}{8}$	$6\frac{13}{16}$
$\frac{3}{4}$	$18\frac{3}{4}$	$6\frac{3}{8}$	$6\frac{7}{8}$	$4\frac{1}{8}$	$4\frac{1}{4}$	2	$3\frac{1}{2}$	$10\frac{3}{4}$	$6\frac{3}{4}$
$\frac{13}{16}$	$18\frac{7}{8}$	$6\frac{15}{32}$	$6\frac{7}{8}$	$4\frac{1}{8}$	$4\frac{1}{4}$	2	$3\frac{9}{16}$	$10\frac{5}{8}$	$6\frac{7}{8}$
$\frac{7}{8}$	19	$6\frac{9}{16}$	$6\frac{7}{8}$	$4\frac{1}{8}$	$4\frac{1}{4}$	2	$3\frac{5}{8}$	$10\frac{1}{2}$	7

SAFE LOADS FOR PASSAIC STEEL Z BAR COLUMNS,

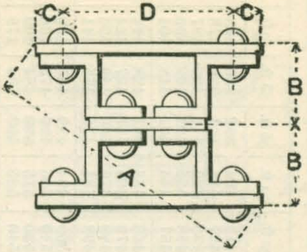
Square Ends, in tons of 2000 lbs., for the following unsupported lengths of columns.

Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii and under. $15{,}000 - 57\frac{l}{r}$ for lengths over 50 radii.

Designation.	Section of Column.	Thickness of Cover Plates.	Weight of Column, lbs. per ft.	Area of Section, sq. inches.	Least Radius of Gyration, inches.	16 ft. or less.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	32 ft.	34 ft.	36 ft.	38 ft.	40 ft.
14″ Z bar Column.	4 Z bars 6″ × 3/4″, 1 web plate 8″ × 3/4″, 2 cover plates 14″ wide.	3/8	173.5	51.0	3.84	306	300	291	282	273	264	255	246	237	228	219	210	201
		7/16	179.5	52.8	3.84	317	311	301	292	283	273	264	254	245	235	226	216	207
		1/2	185.3	54.5	3.85	327	321	311	302	292	282	273	263	253	244	234	224	215
		9/16	191.7	56.3	3.85	338	332	323	313	303	293	283	273	263	253	243	233	223
		5/8	197.3	58.0	3.86	348	342	332	322	311	301	291	281	270	260	250	240	229
		11/16	203.5	59.7	3.87	358	352	342	331	321	310	300	289	279	268	258	247	237
		3/4	209.1	61.5	3.88	368	362	352	342	331	321	310	300	288	277	266	255	244
		13/16	215.2	63.3	3.88	380	374	363	352	341	329	318	307	296	285	274	262	251
		7/8	220.6	64.9	3.89	389	384	372	361	350	338	327	315	304	293	281	270	258
						16 ft. or less.	18 ft.	20 ft.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	32 ft.	34 ft.	36 ft.	38 ft.	40 ft.
14″ Z bar Column.	4 Z bars 6 1/8″ × 7/8″, 1 web plate 8″ × 7/8″, 2 cover plates 14″ wide.	11/16	227.5	66.9	3.83	401	393	381	370	358	346	334	322	310	298	286	274	262
		3/4	233.5	68.7	3.84	412	404	392	380	368	355	343	331	318	306	294	282	270
		13/16	239.7	70.4	3.84	422	415	402	390	377	365	352	340	328	315	303	290	278
		7/8	245.2	72.2	3.84	433	426	413	400	388	375	362	349	336	324	311	298	285
		15/16	251.6	74.0	3.85	444	436	423	410	397	384	371	357	344	331	318	305	292
		1	257.3	75.7	3.86	454	446	433	419	407	393	380	366	353	339	326	312	299
		1 1/16	263.3	77.5	3.86	465	457	443	430	416	402	388	375	361	348	334	321	307
		1 1/8	269.4	79.2	3.86	475	467	454	440	426	412	398	384	370	356	342	328	314
		1 3/16	275.4	81.0	3.86	486	477	463	449	434	420	406	392	377	363	349	335	321
		1 1/4	281.2	82.7	3.86	496	487	472	457	443	428	414	399	385	370	356	341	327

Z BAR COLUMN DIMENSIONS,

in inches.



14″ Columns;
4 **Z** bars, 6″ × $\frac{3}{4}$″; 1 Web plate 8″ × $\frac{3}{4}$″;
2 cover plates 14″ wide.

	Thickness of Cover Plates.	A	B	C	D
Diameter of bolt or rivet, $\frac{7}{8}$″	$\frac{3}{8}$	$19\frac{7}{16}$	$6\frac{3}{4}$	$1\frac{5}{8}$	$10\frac{3}{4}$
	$\frac{7}{16}$	$19\frac{1}{2}$	$6\frac{13}{16}$	$1\frac{5}{8}$	$10\frac{3}{4}$
	$\frac{1}{2}$	$19\frac{5}{8}$	$6\frac{7}{8}$	$1\frac{5}{8}$	$10\frac{3}{4}$
	$\frac{9}{16}$	$19\frac{3}{4}$	$6\frac{15}{16}$	$1\frac{5}{8}$	$10\frac{3}{4}$
	$\frac{5}{8}$	$19\frac{13}{16}$	7	$1\frac{5}{8}$	$10\frac{3}{4}$
	$\frac{11}{16}$	$19\frac{7}{8}$	$7\frac{1}{16}$	$1\frac{5}{8}$	$10\frac{3}{4}$
	$\frac{3}{4}$	20	$7\frac{1}{8}$	$1\frac{5}{8}$	$10\frac{3}{4}$
	$\frac{13}{16}$	$20\frac{1}{16}$	$7\frac{3}{16}$	$1\frac{5}{8}$	$10\frac{3}{4}$
	$\frac{7}{8}$	$20\frac{1}{8}$	$7\frac{1}{4}$	$1\frac{5}{8}$	$10\frac{3}{4}$

14″ Columns;
4 **Z** bars, $6\frac{1}{8}$″ × $\frac{7}{8}$″; 1 Web plate 8″ × $\frac{7}{8}$″;
2 cover plates 14″ wide.

	Thickness of Cover Plates.	A	B	C	D
Diameter of bolt or rivet, $\frac{7}{8}$″	$\frac{11}{16}$	$20\frac{1}{8}$	$7\frac{1}{4}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	$\frac{3}{4}$	$20\frac{1}{4}$	$7\frac{5}{16}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	$\frac{13}{16}$	$20\frac{5}{16}$	$7\frac{3}{8}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	$\frac{7}{8}$	$20\frac{7}{16}$	$7\frac{7}{16}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	$\frac{15}{16}$	$20\frac{1}{2}$	$7\frac{1}{2}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	1	$20\frac{5}{8}$	$7\frac{9}{16}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	$1\frac{1}{16}$	$20\frac{11}{16}$	$7\frac{5}{8}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	$1\frac{1}{8}$	$20\frac{13}{16}$	$7\frac{11}{16}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	$1\frac{3}{16}$	$20\frac{7}{8}$	$7\frac{3}{4}$	$1\frac{3}{4}$	$10\frac{1}{2}$
	$1\frac{1}{4}$	21	$7\frac{13}{16}$	$1\frac{3}{4}$	$10\frac{1}{2}$

SAFE LOADS FOR PASSAIC STEEL Z BAR COLUMNS,

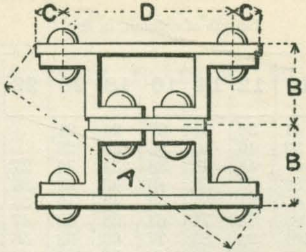
Square ends, in tons of 2000 lbs., for the following unsupported lengths of columns.

Allowable strains per square inch: 12,000 lbs. for lengths of 50 radii and under. $15{,}000 - 57\frac{l}{r}$ for lengths over 50 radii.

Designation.	Section of Column.	Thickness of Cover Plates, ins.	Weight of Column, lbs. per ft.	Area of Section, sq. inches.	Least Radius of Gyration, inches.	20 ft. or less.	22 ft.	24 ft.	26 ft.	28 ft.	30 ft.	32 ft.	34 ft.	36 ft.	38 ft.	40 ft.	42 ft.	44 ft.
16″ Z bar Column.	4 Z bars 6″ × $\frac{7}{8}$″, 1 web plate 10″ × $\frac{7}{8}$″, 2 cover plates 16″ wide.	1	276.8	81.43	4.60	488	476	464	452	440	428	416	404	392	380	368	356	344
		$1\frac{1}{16}$	283.7	83.43	4.60	501	488	475	463	450	439	427	415	402	390	378	365	353
		$1\frac{1}{8}$	290.4	85.43	4.60	513	501	488	475	463	450	438	425	412	400	387	374	362
		$1\frac{3}{16}$	297.2	87.43	4.60	525	513	500	487	474	461	448	435	422	409	396	383	370
		$1\frac{1}{4}$	304.1	89.43	4.60	537	523	510	497	484	470	457	444	431	418	404	391	378
		$1\frac{5}{16}$	310.8	91.43	4.60	548	536	522	509	495	482	468	455	441	428	414	401	387
		$1\frac{3}{8}$	317.6	93.43	4.60	561	548	534	520	506	493	479	465	451	437	424	410	396
		$1\frac{7}{16}$	324.5	95.43	4.60	573	559	545	531	517	503	489	475	460	446	432	418	404
		$1\frac{1}{2}$	331.2	97.43	4.60	585	571	556	542	527	513	498	484	470	455	441	426	412
		$1\frac{9}{16}$	338.0	99.43	4.60	597	583	568	553	539	524	509	494	480	465	450	436	421
		$1\frac{5}{8}$	344.9	101.43	4.61	609	594	579	564	549	534	519	505	490	475	460	445	430
		$1\frac{11}{16}$	351.7	103.43	4.61	621	607	592	577	561	546	530	515	500	484	469	453	438
		$1\frac{3}{4}$	358.4	105.43	4.61	633	618	602	587	571	556	540	525	509	494	478	463	447
		$1\frac{13}{16}$	365.2	107.43	4.61	645	631	616	600	584	568	552	536	520	504	488	472	456
		$1\frac{7}{8}$	372.0	109.43	4.61	657	643	626	610	593	577	561	544	528	512	496	480	464
		$1\frac{15}{16}$	378.8	111.43	4.61	669	654	637	621	603	587	570	554	538	521	505	488	472
		2	385.7	113.43	4.61	681	666	648	631	615	598	581	565	548	531	514	498	481
		$2\frac{1}{16}$	392.5	115.43	4.61	693	676	659	642	625	608	591	574	557	540	523	506	489
		$2\frac{1}{8}$	399.2	117.43	4.61	705	689	671	654	636	619	601	584	567	549	532	514	497
		$2\frac{3}{16}$	406.1	119.43	4.61	717	701	682	664	647	629	612	594	576	559	541	524	506
		$2\frac{1}{4}$	412.8	121.43	4.61	729	713	694	676	658	640	622	604	586	568	550	532	514

Z BAR COLUMN DIMENSIONS,

IN INCHES.



16″ Columns;
4 Z bars $6\frac{1}{8}'' \times \frac{7}{8}''$
1 web plate $10'' \times \frac{7}{8}''$
2 cover plates 16″ wide.

	Thickness of Cover Plates.	A	B	C	D
Diameter of Bolt or Rivet, $\frac{7}{8}''$	1	22	$7\frac{9}{16}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$1\frac{1}{16}$	$22\frac{1}{8}$	$7\frac{5}{8}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$1\frac{1}{8}$	$22\frac{3}{16}$	$7\frac{11}{16}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$1\frac{3}{16}$	$22\frac{1}{4}$	$7\frac{3}{4}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$1\frac{1}{4}$	$22\frac{3}{8}$	$7\frac{13}{16}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$1\frac{5}{16}$	$22\frac{7}{16}$	$7\frac{7}{8}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$1\frac{3}{8}$	$22\frac{9}{16}$	$7\frac{15}{16}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$1\frac{7}{16}$	$22\frac{5}{8}$	8	$1\frac{3}{4}$	$12\frac{1}{2}$
	$1\frac{1}{2}$	$22\frac{11}{16}$	$8\frac{1}{16}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$1\frac{9}{16}$	$22\frac{13}{16}$	$8\frac{1}{8}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$1\frac{5}{8}$	$22\frac{7}{8}$	$8\frac{3}{16}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$1\frac{11}{16}$	23	$8\frac{1}{4}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$1\frac{3}{4}$	$23\frac{1}{16}$	$8\frac{5}{16}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$1\frac{13}{16}$	$23\frac{1}{8}$	$8\frac{3}{8}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$1\frac{7}{8}$	$23\frac{1}{4}$	$8\frac{7}{16}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$1\frac{15}{16}$	$23\frac{5}{16}$	$8\frac{1}{2}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	2	$23\frac{7}{16}$	$8\frac{9}{16}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$2\frac{1}{16}$	$23\frac{1}{2}$	$8\frac{5}{8}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$2\frac{1}{8}$	$23\frac{5}{8}$	$8\frac{11}{16}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$2\frac{3}{16}$	$23\frac{11}{16}$	$8\frac{3}{4}$	$1\frac{3}{4}$	$12\frac{1}{2}$
	$2\frac{1}{4}$	$23\frac{13}{16}$	$8\frac{13}{16}$	$1\frac{3}{4}$	$12\frac{1}{2}$

Brackets on Columns carry a
of section. This includes both

SAFE LOADS, IN TONS OF 2000 LBS., FOR HOLLOW CYLINDRICAL CAST IRON COLUMNS.

Square ends. Factor of safety of 8.

Outside diam., inches.	Thickness of metal, inches.	Length of column, in feet.									Area of Section, sq. ins.	Wgt. per ft. of Cols., lbs.
		8	10	12	14	16	18	20	22	24		
6	¾	47	41	36	31	27	24	21			12.4	39
6	1	60	52	46	40	35	30	26			15.7	49
7	¾	60	54	48	43	38	34	30	27	24	14.7	46
7	1	76	69	62	55	49	43	38	34	30	18.9	60
8	¾	72	67	61	55	50	45	40	36	33	17.1	53
8	1	93	86	78	71	64	58	52	47	42	22.0	69
8	1¼	112	104	94	86	77	69	62	56	51	26.5	83
9	¾	85	80	74	68	62	57	52	47	43	19.4	61
9	1	110	103	95	88	80	73	67	61	55	25.1	78
9	1¼	133	125	115	106	97	89	81	73	67	30.4	95
9	1½	155	145	134	123	113	103	94	85	78	35.3	110
10	1	127	120	112	105	97	89	82	76	69	28.3	88
10	1¼	154	146	136	127	118	109	100	92	84	34.4	107
10	1½	180	170	159	148	137	127	117	107	98	40.1	125
10	1¾	203	192	180	168	155	143	132	121	111	45.4	142
11	1	144	137	129	122	114	106	100	91	85	31.4	98
11	1¼	175	167	158	148	139	129	122	112	103	38.3	119
11	1½	204	195	184	173	161	151	143	130	121	44.8	140
11	1¾	232	221	209	197	184	172	162	148	137	50.9	159
11	2	258	246	233	219	205	191	181	164	152	56.6	176
12	1	160	154	147	139	131	123	115	108	101	34.6	108
12	1¼	196	188	180	170	160	150	141	132	123	42.2	131
12	1½	229	220	210	199	187	176	165	154	144	49.5	154
12	1¾	261	251	239	226	213	201	188	176	164	56.4	176
12	2	291	279	266	252	238	224	210	196	183	62.8	196
13	1	177	170	163	156	148	140	132	124	117	37.7	118
13	1¼	216	209	200	191	181	172	162	152	143	46.1	144
13	1½	254	245	235	224	213	201	190	179	168	54.2	169
13	1¾	289	280	268	256	243	229	217	204	192	61.9	193
13	2	324	312	300	286	272	257	242	228	214	69.1	216
14	1	193	187	180	173	165	157	149	141	134	40.8	128
14	1¼	237	229	221	212	203	193	183	173	164	50.1	156
14	1½	278	270	260	250	239	227	215	204	193	58.9	184
14	1¾	318	308	297	285	273	260	246	233	220	67.4	210
14	2	356	345	333	320	305	291	276	261	247	75.4	235
15	1	209	204	197	190	183	175	167	159	151	44.0	137
15	1¼	257	250	242	233	224	214	205	195	185	54.0	168
15	1½	303	295	285	275	264	253	241	229	218	63.6	199
15	1¾	347	337	327	315	302	289	276	263	249	72.9	227
15	2	389	378	366	353	339	324	309	294	280	81.7	255
16	1¼	277	270	262	254	245	235	225	216	206	57.8	180
16	1½	327	319	311	300	290	278	267	255	244	68.4	214
16	1¾	375	366	356	344	332	319	306	292	279	78.4	245
16	2	421	411	400	387	373	358	343	328	313	88.0	275
16	2¼	465	454	441	427	412	396	379	363	346	97.2	304

safe load of 1000 [illegible] re indi-
horizontal and vertical parts.

See Eng. News Jan. 20/98

SAFE LOADS, IN TONS OF 2000 LBS., FOR HOLLOW SQUARE CAST IRON COLUMNS.

Square ends. Factor of safety of 8.

Side of Column, Ins.	Thickness of metal, inches.	Length of column, in feet. 8	10	12	14	16	18	20	22	24	Area of Section, sq. ins.	Wgt. per ft. of Cols., lbs.
6	¾	64	57	51	45	40	36	32			15.8	49
6	1	81	73	65	58	51	45	40			20.0	63
7	¾	80	73	67	61	55	50	45			18.8	59
7	1	102	94	86	78	70	63	57			24.0	75
8	¾	96	90	83	77	71	65	59	54	49	21.8	68
8	1	123	116	107	99	91	83	76	69	63	28.0	88
8	1¼	149	139	129	119	110	100	92	84	76	33.8	106
9	¾	112	106	100	93	87	80	74	69	63	24.8	77
9	1	144	137	129	121	112	104	96	89	82	32.0	100
9	1¼	175	166	156	146	136	126	116	107	99	38.8	121
9	1½	203	193	182	170	158	146	135	125	115	45.0	141
10	1	166	159	151	142	134	125	117	109	101	36.0	113
10	1¼	201	193	183	173	163	152	142	132	123	43.8	137
10	1½	235	225	214	202	189	177	166	154	143	51.0	159
10	1¾	266	254	242	228	215	201	188	175	162	57.8	181
11	1	187	180	172	164	156	147	138	130	122	40.0	125
11	1¼	227	219	210	200	190	179	169	158	148	48.8	152
11	1½	266	256	246	234	222	209	197	185	174	57.0	178
11	1¾	302	291	279	266	252	238	224	210	197	64.8	202
11	2	336	324	310	295	280	264	249	234	219	72.0	225
12	1	208	201	194	186	177	169	160	151	143	44.0	138
12	1¼	254	246	237	227	217	206	196	185	174	53.8	168
12	1½	297	288	278	266	254	242	229	217	205	63.0	197
12	1¾	338	328	316	303	289	275	261	247	233	71.8	224
12	2	377	366	352	338	323	307	291	275	260	80.0	250
13	1	228	222	215	208	199	191	182	173	164	48.0	150
13	1¼	279	272	263	254	244	233	223	212	201	58.8	184
13	1½	328	319	309	298	286	274	261	249	236	69.0	216
13	1¾	375	365	353	341	327	313	298	284	270	78.8	246
13	2	419	407	394	380	365	350	334	317	301	88.0	275
14	1	249	243	236	229	221	213	204	195	186	52.0	163
14	1¼	305	298	290	281	271	261	250	239	228	63.8	199
14	1½	359	351	341	330	319	307	294	281	268	75.0	234
14	1¾	411	401	390	378	365	351	336	322	307	85.8	268
14	2	460	449	437	423	408	393	376	360	344	96.0	300
15	1	270	264	258	250	243	235	226	217	208	56.0	175
15	1¼	331	324	316	308	298	288	277	266	255	68.8	215
15	1½	390	382	373	362	351	339	327	314	301	81.0	253
15	1¾	446	437	427	415	402	388	374	359	345	92.8	289
15	2	501	490	479	465	451	436	420	403	386	104.0	325
16	1¼	357	350	343	334	325	315	305	294	286	73.8	231
16	1½	421	413	404	394	383	372	359	347	334	87.0	272
16	1¾	482	474	463	452	440	426	412	397	383	99.8	312
16	2	541	532	520	507	493	478	463	446	429	112.0	350
16	2¼	598	588	575	561	545	529	511	493	475	123.8	387

ULTIMATE STRENGTH OF HOLLOW CYLINDRICAL AND RECTANGULAR CAST IRON COLUMNS.

Ultimate Strength in Pounds per Square Inch:

CYLINDRICAL COLUMNS.			RECTANGULAR COLUMNS.		
Square Bearing:	Pin and Square:	Pin Bearing:	Square Bearing:	Pin and Square:	Pin Bearing:
$\dfrac{80000}{1+\dfrac{(12L)^2}{800\,d^2}}$	$\dfrac{80000}{1+\dfrac{3(12L)^2}{1600\,d^2}}$	$\dfrac{80000}{1+\dfrac{(12L)^2}{400\,d^2}}$	$\dfrac{80000}{1+\dfrac{3(12L)^2}{3200\,d^2}}$	$\dfrac{80000}{1+\dfrac{9(12L)^2}{6400\,d^2}}$	$\dfrac{80000}{1+\dfrac{3(12L)^2}{1600\,d^2}}$

L = Length of Column, in feet.
d = External diameter or least side of rectangle, in inches.

$\frac{L}{d}$	CYLINDRICAL COLUMNS. Ultimate Strength in lbs. per sq. in.			RECTANGULAR COLUMNS. Ultimate Strength in lbs. per sq. in.		
	Square Bearing.	Pin and Square.	Pin Bearing.	Square Bearing.	Pin and Square.	Pin Bearing.
0.5	76560	74940	73390	77380	76150	74940
0.6	75130	72910	70820	76290	74560	72910
0.7	73520	70650	68000	75030	72780	70650
0.8	71740	68210	65020	73640	70820	68210
0.9	69820	65640	61940	72110	68730	65640
1.0	67800	62990	58820	70480	66520	62990
1.1	65690	60300	55730	68790	64260	60300
1.2	63530	57600	52690	67000	61940	57600
1.3	61340	54930	49740	65140	59600	54960
1.4	59140	52310	46900	63260	57270	52320
1.5	56940	49770	44200	61350	54960	49760
1.6	54760	47300	41630	59450	52680	47300
1.7	52620	44940	39210	57550	50460	44960
1.8	50530	42670	36930	55670	48300	42670
1.9	48490	40510	34790	53800	46230	40510
2.0	46510	38460	32790	51940	44200	38460
2.1	44600	36520	30920	50160	42260	36520
2.2	42750	34680	29180	48400	40400	34680
2.3	40980	32940	27540	46670	38630	32950
2.4	39280	31310	26030	44990	36930	31310
2.5	37650	29770	24620	43390	35310	29760
2.6	36090	28320	23300	41820	33770	28320
2.7	34600	26950	22070	40320	32310	26950
2.8	33180	25670	20930	38870	30920	25670
2.9	31820	24460	19860	37470	29600	24460

For safe quiescent loads, as in buildings, divide the above values by 8.

BEARINGS AND FOUNDATIONS.

If steel beams are supported on walls they should have a bearing on the wall, at each end, not less than the following:

6″ Beams and under	6″ bearing.
7″ and 8″ Beams	8″ "
9″, 10″ and 12″ Beams	10″ "
15″ and 20″ Beams	12″ "

Steel bearing plates should be used under the ends of steel beams where they rest upon a brick wall to distribute the pressure and prevent crushing the material of the wall. The bearing plate should be as long as the bearing of the beam on the wall, and of sufficient width so that the pressure, per square inch, on first-class brickwork of hard burned bricks shall not exceed,

On brickwork laid in cement mortar	200 lbs. per sq. inch
" " " " cement and lime mortar	150 "
" " " " lime mortar	100 "

For good brickwork laid in cement mortar, the following sizes of bearing plates will suffice for the ordinary spans on which the beams are used : —

Size of Beam.	Standard Bearing Plates.			Safe End Reaction, in Tons.			Weight of one Plate, Lbs.
	Length.	Width.	Thickness.	100 lbs. per Sq. In.	150 lbs. per Sq. In.	200 lbs. per Sq. In.	
20″	12″	16″	$\frac{3}{4}$″	9.6	14.4	19.2	40.8
15″	12″	12″	$\frac{5}{8}$″	7.2	10.8	14.4	25.5
12″	10″	10″	$\frac{5}{8}$″	5.0	7.5	10.0	17.7
10″ and 9″	10″	8″	$\frac{1}{2}$″	4.0	6.0	8.0	11.3
8″ and 7″	8″	8″	$\frac{1}{2}$″	3.2	4.8	6.4	9.1
6″	6″	8″	$\frac{1}{2}$″	2.4	3.6	4.8	6.8

A template of bluestone, or other hard quality of stone, is frequently used, instead of a steel bearing plate, at the wall ends of steel beams. Where the pressure is great, as at the ends of girders, both steel bearing plates and stone templates should be used, the size of the bearing plate being sufficient to limit the pressure between it and the bluestone template to 300 lbs. per square inch. The size of the stone template must be sufficient to limit its pressure on the brickwork to the proper pressure as given above. The stone template should not project beyond the bearing plate, in any direction, more than ¾ of the thickness of the stone.

FOUNDATIONS.

The proper design of foundations is of the utmost importance. The maximum load carried by the foundation must first be obtained. The loads to be considered in buildings are of two kinds: the dead load, which is the actual weight of the materials of construction; and the live load, which is the weight that the floors may be required to support. The live load is variable. In office buildings, parts of the floors may be loaded to their full capacity, but the probability of the entire structure being so loaded is remote; while in breweries, storage warehouses and buildings for similar purposes, all the floors may be fully loaded. The maximum of both dead and live loads must be considered, and the area of the footing of the foundation must be such that the greatest pressure on different soils does not exceed the following:

Kind of material.	Safe pressure in tons per sq. ft.
Compact bed rock, if of granite	30
" " " " " limestone	25
" " " " " sandstone	18
Soft friable rock	5 to 10
Clay, in thick beds, absolutely dry	4
" " " " moderately dry	2
Soft clay	1
Dry coarse gravel, well packed and confined	6
Compact dry sand, well cemented and confined	4
Clean dry sand, in natural beds and confined	2
Good solid dry natural earth	4

Except where foundations are upon rock, the possibility of the bearing material being loosened, by water or by adjacent building operations, must be considered and proper precautions must be taken to prevent it.

Foundations upon yielding material will always settle more or less. In order that this settlement shall be uniform, it is essential that the various foundations in a structure shall produce equal pressures per unit of area on their footings; that is, the areas of the foundations must be proportional to the loads carried. In office buildings, where the actual live load is variable and rarely approaches the load assumed, the best results in the way of equal settlement of the foundations are obtained by proportioning the areas of the footings so that the dead loads produce equal pressures. Thus, if in such a building the maximum foundation supports a dead load of 200 tons and a live load of 200 tons, and another foundation a dead load of 150 tons and a live load of 100 tons, the total load on the first foundation is 400 tons and, assuming the soil to carry a load of 4 tons per sq. ft., the area required is 100 sq. ft. This corresponds with a pressure of 2 tons per sq. ft. for the dead load alone. Using this same pressure for dead load requires an area of 75 sq. ft. for the second foundation, instead of an area of 62.5 sq. ft. which would have been obtained had the foundation been proportioned for the total live and dead load at 4 tons per sq. ft.

The foundation illustrated in Fig. 1 is frequently used when the soil is good dry natural earth capable of safely supporting from 3 to 4 tons per square foot. Such a foundation must be designed to distribute the concentrated load which it supports over the proper area of footing required. The capstone should be of granite or limestone having a minimum thickness of one foot, and not less than one-fifth its greatest dimension. The body of the pier should be of first quality brick laid in Portland cement mortar, and the footing of a layer of concrete not less than 18″ thick. When the load is great, a heavy cast iron pedestal should be used to distribute the load over the capstone. The height of this pedestal should be one-half the greatest dimension of its base. The requisite spread of footing is obtained by offsets in the successive courses, and the proper design of the foundation is based upon the following values: —

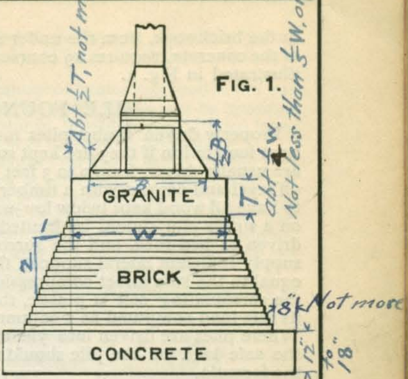


FIG. 1.

	Maximum pressure, lbs. per sq. in.	Maximum offset of course in terms of thickness.
Granite	350	⅝
Limestone	300	⅝
Sandstone	250	½
Brickwork in Portland cement	200	⅜
Concrete	200	½

To illustrate the application of these principles they will be applied to the design of a foundation for a load of 400 tons on a soil capable of supporting a load of 4 tons per square foot. The size of the cast iron base will be determined by limiting its pressure on the granite cap to 350 lbs. per square inch; then,

$$400 \text{ tons} = 800{,}000 \text{ lbs.} \div 350 = 2286 \text{ sq. ins. required.}$$

A base, 48″ square, having an area of 2304 sq. ins., will be required.

The size of the granite cap will be determined by limiting its pressure on the brickwork to 200 lbs. per sq. in.; then,

$$800{,}000 \text{ lbs.} \div 200 = 4{,}000 \text{ sq. ins. required.}$$

A capstone, 5′ 4″ square, has an area of 4096 sq. ins., and is the size required. Its thickness will be 15″, or about one-fourth its base.

The area of the footing required is,

$$400 \text{ tons} \div 4 = 100 \text{ sq. ft. required.}$$

The footing will be of concrete, 10 ft. square, and 18″ thick. The projection of this footing will be one-half its thickness, or 9″, all around; so that the brickwork must be 8′ 6″ square where it rests upon the concrete. The projection of a single course of brickwork is limited to 1″. Each course of brick thus adds 2″ to the spread of the foundation, and to obtain the necessary spread in the brickwork, from the under side of the capstone to the top of the concrete, requires 19 courses of brick. This foundation is illustrated in Fig. 1.

PILE FOUNDATIONS.

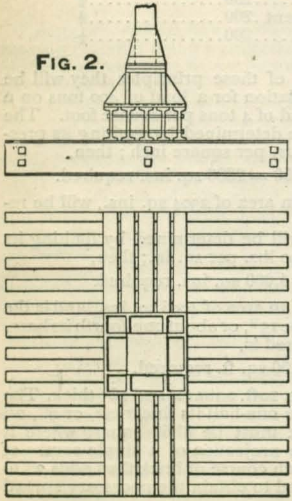
Properly driven timber piles make a satisfactory and permanent foundation if they are kept submerged under water. Piles are usually driven from 2 to 3 feet between centers, the tops cut off level and capped with a timber grillage, care being observed to have all wood kept below low-water line. The maximum load on a single pile should be limited to 20 tons. Where piles are driven to bed rock, and the surrounding soil is stiff enough to supply sufficient lateral support, the bearing power of the pile is equal to the safe direct compression on its least cross section; if the surrounding soil is plastic, the bearing power of the pile is its safe load computed as a column of the total length of the pile. Where piles are driven into yielding soil without reaching rock, the safe load on the pile should not exceed the value given by the formula,

$$L = \frac{2WH}{p + 1}$$

where L is the safe load in tons on the pile; W is the weight of the hammer in tons; H is the fall of the hammer in feet; and p is the penetration of the pile, under the last blow of the hammer, in inches. The broom and splinters should be removed from the head of the pile in obtaining the penetration under the last blow.

STEEL BEAM GRILLAGE.

FIG. 2.



Where foundations rest upon a yielding stratum, a grillage consisting of two or more layers of steel **I** beams furnishes an economical and satisfactory method of distributing the load. Fig. 2 illustrates such a foundation. A bed of concrete, not less than 12 inches thick, is laid, on which the steel **I** beams are placed side by side, a sufficient number of proper size being used to distribute the load over the desired area. This layer of beams is covered with concrete well rammed between the beams. The second layer of beams on which the foot of the column is to rest is laid across the first layer, reaching to the extreme outer edge of the first layer, and is also filled between and covered with concrete. The beams of each layer should be connected with separators and tie rods. The beams should have a clear space of at least 3 inches between flanges to permit ramming the concrete, and should not be spaced exceeding 18 inches on centers.

When the load is great, the number of beams required in the second layer may necessitate a greater spread than can be spanned by the shoe or the foot of the column, in which case a third layer of short beams or a box girder may be used to advantage.

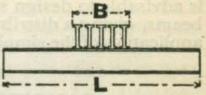
This type of foundation is adapted for heavy loads, as the requisite spread of foundation area is obtained in small depth. A useful application of the method is in situations where a thin and compact stratum overlies another of a more yielding nature, and where the available height of foundation is limited; as the requisite area of the footing may be obtained without penetrating the firmer stratum, and without undue vertical encroachment.

The method of calculating the strength of grillage beams is as follows: —

Let W = Superimposed load on beam.

B = Length over which superimposed load is applied.

L = Length of beam.



The superimposed load is considered as uniformly distributed over the length on which it is applied, and the pressure of the soil as uniformly distributed over the entire length of the beam. The maximum bending moment is at the center of the length of the beam and is equal to $\frac{1}{8} W(L-B)$. If the load is taken in pounds, the bending moment will be found either in foot lbs. or in inch lbs., according as the lengths are taken in feet or in inches; and the size of the steel beam required can be found in the manner explained under the Strength of Beams.

To facilitate calculation, the following table gives the greatest safe loads on Passaic steel **I** beams used in grillages for various values of $(L-B)$. In using this table, it is only necessary to assume the number of beams to be used in the layer. The superimposed load on each beam equals the total load on the layer divided by the number of beams in the layer, and by reference to the table, the proper beam capable of supporting this load is at once determined.

To illustrate the application of the table, take a foundation carrying a load of 400 tons on a soil capable of supporting a load of 2 tons per square foot. The required area of the footing will be 200 sq. ft. If a square footing is used, a square with 14-ft. sides has an area of 196 sq. ft. and will be assumed as ample. The upper layer of beams will be proportioned first.

The base of the column will be assumed as 4 ft. square; then, in this case, B is 4 ft., L is 14 ft., and $(L-B)$ is 10 ft. The upper layer will be assumed to consist of 5 beams, as this number is the greatest that will provide sufficient space between the flanges of the beams to permit satisfactory ramming of the concrete filling. Each beam will then take $\frac{1}{5}$ the total load, or 80 tons. By referring to the table, a 20″ × 90 lb. **I** has a safe load of 80.3 tons when $L-B$ is 10 ft. The upper layer will, therefore, consist of five 20″ × 90 lb. **I** beams,

In the under layer, in this instance, L and B have the same values as in the upper layer. If the beams are spaced about 12″

on centers, there will be 15 beams in the layer, each carrying $\frac{1}{15}$ the total load, or 26⅔ tons. By referring to the table, the lightest beam, whose safe load is nearest to this, is a 15″ × 42 lb. **I** which has a safe load of 30.6 tons. A less number of beams can therefore be used. Thirteen beams, 15″ × 42 lbs., will provide for the total load within a small amount, which considering the nature of the load, can be neglected. This foundation is illustrated in Fig. 2.

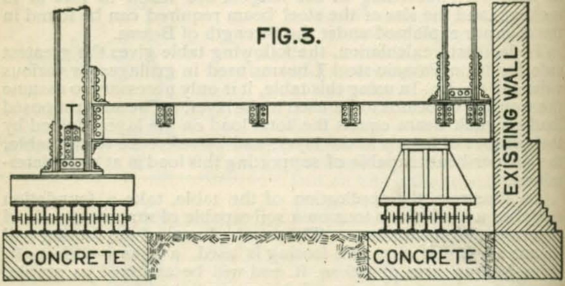
Where two columns, carrying unequal loads, rest upon the same grillage, care should be taken to have the center of gravity of the grillage coincide with the point of application of the resultant of the loads on the columns, in order to secure uniform pressure on the footing.

Frequently three columns are supported on the same grillage, the beams being continuous. The calculation of such a foundation is involved, and the distribution of pressure uncertain. It is advisable to design such a foundation with a system of simple beams, giving a distribution of weight readily determined by the application of the simple law of the lever.

CANTILEVER FOUNDATIONS.

Where it is not advisable to undermine existing walls on adjoining property, or where it is not possible to have the wall columns over the center of the foundations along an existing wall, cantilever girders are used to carry the wall columns adjacent to the building line. A simple type of such a foundation is illustrated in Fig. 3.

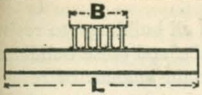
FIG. 3.



The foundation is placed as near the existing wall as possible, and the wall column rests upon a girder which overhangs the foundation and is anchored to one of the interior columns. The maximum bending moment is obtained by multiplying the load on the wall column by the distance between the center of the column and the center of the supporting foundation. The size of cantilever beams can then be determined in the manner already given in the article on Strength and Deflection of Beams. Care must be observed to have the minimum load on the interior column greater than the maximum lifting tendency produced by the cantilever.

PASSAIC STEEL I BEAMS,

USED AS GRILLAGE BEAMS IN FOUNDATIONS.



L = Length of Beam in Feet.

B = Length, in Feet, over which superimposed Load is distributed.

Total Safe Load on a single Beam, in Tons of 2000 Lbs., for the following values of **L-B.**

Beam.		Unloaded Length of Beam, **L-B**, in feet.										
Dep. Ins.	Wgt., lbs. per Ft.	5	6	7	8	9	10	11	12	13	14	15
20	90			115	100	89.2	80.3	73.0	66.9	61.8	57.4	53.6
"	80			102	89.6	79.8	71.7	65.2	59.8	55.2	51.2	47.8
"	75			95.0	83.2	73.8	66.5	60.5	55.4	51.2	47.5	44.3
"	65			87.5	76.8	68.1	61.3	55.7	51.1	47.1	43.8	40.9
15	75			73.2	64.0	57.0	51.2	46.6	42.7	39.4	36.6	34.2
"	66⅔			68.6	60.2	53.4	48.1	43.7	40.1	37.0	34.3	32.1
"	60			64.8	56.6	50.4	45.4	41.2	37.8	34.9	32.4	30.2
"	50			53.8	47.0	41.8	37.7	34.2	31.4	29.0	26.9	25.1
"	42			43.7	38.2	34.0	30.6	27.7	25.5	23.5	21.9	20.4
12	55		53.0	45.6	39.8	35.4	31.8	28.8	26.5	24.5	22.8	21.2
"	40		41.6	35.8	31.3	27.8	25.0	22.7	20.8	19.2	17.9	16.7
"	31½		32.6	28.0	24.5	21.8	19.6	17.8	16.3	15.1	14.0	13.1
10	40	38.0	31.8	27.2	23.8	21.2	19.0	17.3	15.9	14.7	13.6	12.7
"	33	34.4	28.6	24.6	21.5	19.1	17.2	15.6	14.3	13.2	12.3	11.5
"	30	28.8	24.0	20.6	18.0	16.0	14.4	13.1	12.0	11.1	10.3	9.6
"	25	26.2	21.8	18.7	16.3	14.5	13.1	11.9	10.9	10.1	9.3	8.7
9	27	26.2	21.8	18.7	16.4	14.6	13.1	11.9	10.9	10.1	9.4	8.7
"	23⅓	21.2	17.6	15.1	13.2	11.7	10.6	9.6	8.8	8.1	7.5	7.0
"	21	20.0	16.7	14.3	12.5	11.1	10.0	9.1	8.3	7.7	7.1	6.7
8	27	20.7	17.2	14.8	12.9	11.5	10.3	9.4	8.6	7.9		
"	22	18.6	15.5	13.3	11.6	10.3	9.3	8.4	7.7	7.1		
"	18	15.1	12.6	10.8	9.4	8.4	7.6	6.9	6.3	5.8		
		2	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
7	20			18.1	14.5	12.1	10.4	9.1	8.1	7.3	6.6	6.1
"	15			14.1	11.3	9.4	8.1	7.1	6.3	5.7	5.1	4.7
6	15		15.7	11.8	9.4	7.8	6.7	5.9	5.2	4.7		
"	12		12.9	9.7	7.8	6.5	5.5	4.8	4.3	3.9		
5	13		11.2	8.4	6.7	5.6	4.8	4.2				
"	9¾		8.6	6.6	5.3	4.4	3.8	3.3				
4	10	9.2	6.1	4.6	3.7	3.1	2.6					
"	7½	7.8	5.2	3.9	3.1	2.6	2.2					
"	6	6.1	4.1	3.1	2.5	2.0	1.8					

Maximum fiber strain, 16,000 lbs. per square inch.

WIND BRACING.

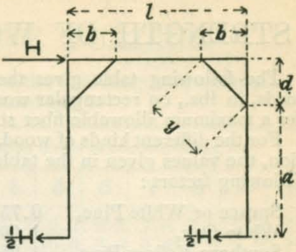
Adequate provision must be made in all buildings to resist horizontal wind pressure. In mercantile and office buildings the walls and partitions provide a certain amount of resistance, though in the skeleton construction, now extensively used for tall buildings, the thin curtain walls and the extremely light tile partitions provide a very uncertain means of resistance.

A building, whose height does not exceed twice its base, and which has a well-constructed steel frame, scarcely needs a special system of wind bracing to make it secure, if the exterior walls are well built and of sufficient thickness, or if it is provided with substantial interior brick partitions. The columns should be of steel of any of the usual types, and be in lengths of two or more stories and thoroughly spliced at the joints with plates and rivets sufficient to make the section nearly continuous as far as the transverse bending is concerned. The column splices should be arranged so that not more than one-half the total number of columns splice at any one floor level. All connections between columns, girders and beams should be riveted.

Buildings, whose height exceeds twice their base, should have wind-bracing, of some form, calculated to resist a horizontal wind pressure of 30 lbs. per sq. ft. on their greatest exposed surface. It is seldom possible to use diagonal rods between the columns, and either of the two following forms of bracing are generally used in buildings. The columns in massive buildings may be considered as fixed at the ends, but in sheds and low mill and shop buildings the columns are not fixed at the ends unless special provision is made to anchor them very securely to foundations of much larger size than is generally provided. The total strains, due to the combination of the maximum effects of live, dead and wind loads, should not exceed the following, in lbs. per sq. in.,

	Massive Buildings.	Shed Buildings.
Tension	20,000	18,000
Compression....	$20{,}000 - 75\,\frac{l}{r}$	$18{,}000 - 75\,\frac{l}{r}$

The wind increases the compression on the leeward columns and also produces a bending in the columns, both of which effects must be considered.



H = total horizontal force acting at top of frame.

Posts considered as fixed at both ends.

All members constructed to resist tension or compression.

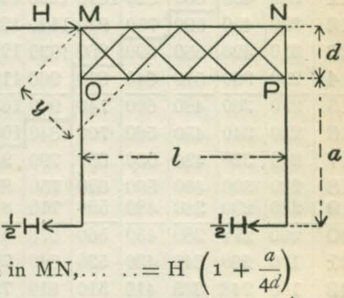
Tension or compression in brackets, $= H\left(\frac{1}{2} + \frac{a}{4d}\right)\frac{y}{b}$

" " " " posts, ... $= H\left(d + \frac{a}{2}\right)\frac{1}{l}$

" " " " girder, .. $= H\left(1 + \frac{a}{4d}\right)$

Bending moment on posts, ... $= H\frac{a}{4}$

" " " girder, ... $= H\left(\frac{1}{2} - \frac{b}{l}\right)\left(d + \frac{a}{2}\right)$



H = total horizontal force acting at top of frame.

Posts considered as fixed at both ends.

All members constructed to resist tension or compression.

Tension or compression in MN,... .. $= H\left(1 + \frac{a}{4d}\right)$

" " " " OP, ... $= H\left(\frac{1}{2} + \frac{a}{4d}\right)$

" " " " diagonals, $= H\left(\frac{d}{2} + \frac{a}{4}\right)\frac{y}{ld}$

" " " " posts, ... $= H\left(d + \frac{a}{2}\right)\frac{1}{l}$

Bending moment on posts, ... $= H\frac{a}{4}$

NOTE.—If the posts are not fixed at the ends, substitute $2a$ for a in the above formulæ.

d Section Modulus = $\frac{bd^2}{6}$

b

STRENGTH OF WOODEN BEAMS.

The following table gives the safe uniformly distributed loads, in lbs., on rectangular wooden beams one inch thick, for a maximum allowable fiber strain of 1,000 lbs. per sq. in.

For the different kinds of wood, ordinarily used in construction, the values given in the table are to be multiplied by the following factors:

	For ordinary purposes.	For purely static loads.
Spruce or White Pine,	0.75	1.00
White Oak,	1.00	1.25
Southern Yellow Pine,	1.25	1.50

Span, in feet.	DEPTH OF BEAM IN INCHES.										
	6	7	8	9	10	11	12	13	14	15	16
5	800	1090	1420	1800	2220	2690	3200	3980	4380	5000	5690
6	670	910	1190	1500	1850	2240	2670	3220	3650	4170	4740
7	570	780	1020	1290	1590	1920	2290	2840	3130	3570	4060
8	500	680	890	1130	1390	1680	2000	2490	2740	3130	3560
9	440	610	790	1000	1230	1490	1780	2210	2430	2780	3160
10	400	540	710	900	1110	1340	1600	1990	2190	2500	2840
11	360	495	650	820	1010	1220	1450	1810	1990	2270	2590
12	330	450	590	750	930	1120	1330	1660	1820	2080	2370
13	310	420	550	690	860	1030	1230	1530	1690	1930	2200
14	290	390	510	640	800	960	1150	1430	1570	1790	2040
15	270	360	480	600	740	900	1070	1330	1460	1670	1900
16	250	340	450	560	700	840	1000	1250	1370	1570	1780
17	240	320	420	530	650	790	940	1170	1290	1470	1680
18	220	300	400	500	620	750	890	1110	1220	1390	1590
19	210	290	380	480	590	710	840	1050	1150	1320	1500
20	200	272	360	450	560	670	800	990	1090	1250	1420
21	190	260	340	430	530	640	760	950	1040	1190	1360
22	180	248	325	410	510	610	730	910	1000	1140	1300
23	175	237	310	390	480	590	700	870	950	1090	1240
24	167	228	297	380	460	560	670	830	910	1040	1190
25	160	218	285	360	450	540	640	800	880	1000	1140
26	154	210	275	350	430	520	620	770	840	960	1100
27	149	202	265	330	410	500	590	740	810	930	1060
28	143	195	255	315	400	480	570	710	780	890	1020
29	138	188	246	307	380	465	550	690	750	860	980
30	134	182	237	297	370	450	530	660	730	830	950

Loads given below the zig-zag line produce deflections liable to crack plastered ceilings.
To obtain the safe load for any thickness, multiply the values given for one inch by the thickness of the beam.
To obtain the required thickness for any load, divide by safe load given for one inch.

WHITE PINE PURLINS.

Maximum Spans in feet, for the following total uniformly distributed loads.

Total Load.	Size of Joists, inches.	Distance from center to center of joists, feet.									
		1	2	3	4	5	6	7	8	9	10
40 lbs. per square foot of roof.	3 × 8	16.2	12.9	11.3	10.3	9.2	8.5	7.8	7.3	6.9	6.6
	4 × 8		14.1	12.4	11.2	10.6	9.8	9.0	8.5	8.0	7.6
	6 × 8		16.2	14.2	12.9	11.9	11.2	10.7	10.4	9.8	9.3
	3 × 10	20.3	16.1	14.1	12.5	11.1	10.2	9.4	8.8	8.3	7.9
	4 × 10	22.3	17.7	15.5	14.0	12.9	11.8	10.9	10.2	9.6	9.1
	6 × 10		20.3	17.8	16.1	15.0	14.0	13.4	12.5	11.8	11.2
	8 × 10			19.5	17.7	16.4	15.4	14.8	14.1	13.5	12.9
	3 × 12	24.4	19.4	16.9	15.0	13.4	12.3	11.3	10.6	10.0	9.5
	4 × 12	26.8	21.2	18.6	16.8	15.5	14.2	13.1	12.3	11.6	11.0
	6 × 12		24.4	21.3	19.3	18.0	16.9	16.0	15.0	14.2	13.4
	8 × 12		26.8	23.4	21.2	19.7	18.5	17.7	16.9	16.2	15.5
	10 × 12			25.2	22.8	21.2	20.0	19.0	18.2	17.4	16.9
	3 × 14	28.4	22.5	19.8	17.5	15.7	14.3	13.3	12.4	11.7	11.1
	4 × 14	31.2	24.7	21.6	19.6	18.1	16.5	15.3	14.3	13.5	12.8
	6 × 14		28.5	24.8	22.6	21.0	19.8	18.8	17.5	16.6	15.7
	8 × 14		31.2	27.2	24.7	23.0	21.6	20.6	19.6	18.9	18.1
	10 × 14			29.4	26.6	24.8	23.2	22.2	21.2	20.4	19.7
60 lbs. per square foot of roof.	3 × 8	14.1	11.3	9.8	8.4	7.5	6.9	6.4	6.0	5.6	5.4
	4 × 8	15.5	12.3	10.8	9.8	8.7	8.0	7.4	6.9	6.5	6.2
	6 × 8	17.9	14.1	12.4	11.3	10.5	9.8	9.1	8.5	8.0	7.6
	3 × 10	17.7	14.0	11.5	10.2	9.1	8.3	7.7	7.2	6.8	6.5
	4 × 10	19.4	15.4	13.5	11.4	10.5	9.6	8.9	8.3	7.8	7.4
	6 × 10	22.4	17.7	15.5	14.1	12.9	11.8	10.9	10.2	9.6	9.1
	8 × 10	24.5	19.4	17.0	15.4	14.3	13.4	12.6	11.7	11.0	10.5
	3 × 12	21.3	16.9	14.2	12.3	10.9	10.0	9.2	8.7	8.2	7.8
	4 × 12	23.4	18.5	16.2	14.1	12.7	11.6	10.7	10.0	9.5	9.0
	6 × 12	26.8	21.3	18.6	16.8	15.5	14.2	13.1	12.3	11.6	10.9
	8 × 12	29.4	23.4	20.4	18.5	17.2	16.1	15.1	14.1	13.2	12.7
	10 × 12		25.2	22.0	19.9	18.5	17.5	16.6	15.8	14.9	14.1
	3 × 14	24.8	19.6	16.6	14.3	12.8	11.7	10.8	10.1	9.6	9.1
	4 × 14	27.2	21.6	18.9	16.6	14.8	13.5	12.5	11.7	11.0	10.5
	6 × 14	31.4	24.8	21.8	19.7	18.1	16.6	15.4	14.3	13.6	12.8
	8 × 14	34.3	27.2	23.8	21.6	20.0	18.9	17.6	16.6	15.6	14.8
	10 × 14		29.3	25.6	23.2	21.6	20.2	19.4	18.5	17.4	16.5

The maximum spans given in the table for the above loads, are determined by limiting the deflection to $\frac{1}{400}$ of the span, and the maximum fiber strain to 750 lbs. per square inch, the lesser value given by either condition being used.

YELLOW PINE PURLINS.

Maximum Spans in feet, for the following total uniformly distributed loads.

Total Load.	Size of Joists, inches.	Distance from center to center of joists, feet.									
		1	2	3	4	5	6	7	8	9	10
40 lbs. per square foot of roof.	3 × 8	19.4	15.4	13.4	12.2	11.3	10.5	9.7	9.2	8.6	8.2
	4 × 8		16.9	14.8	13.4	12.5	11.7	11.2	10.5	10.0	9.4
	6 × 8			16.9	15.4	14.3	13.5	12.8	12.2	11.8	11.5
	3 × 10	24.2	19.2	16.8	15.3	14.2	13.3	12.2	11.4	10.7	10.2
	4 × 10		21.2	18.5	16.8	15.6	14.7	13.9	13.2	12.4	11.7
	6 × 10			21.2	19.2	17.9	16.8	15.9	15.2	14.7	14.2
	8 × 10				21.2	19.6	18.5	17.5	16.8	16.2	15.6
	3 × 12	29.1	23.1	20.2	18.3	17.0	15.9	14.6	13.8	12.9	12.2
	4 × 12		25.4	22.2	20.1	18.7	17.6	16.7	15.8	14.9	14.1
	6 × 12			25.4	23.1	21.4	20.1	19.2	18.3	17.6	17.0
	8 × 12				25.4	23.6	22.2	21.1	20.2	19.4	18.7
	10 × 12					25.4	23.9	22.7	21.7	20.9	20.2
	3 × 14	34.0	26.9	23.6	21.4	19.8	18.5	17.1	16.0	15.1	14.3
	4 × 14		29.6	25.9	23.5	21.8	20.5	19.5	18.5	17.4	16.6
	6 × 14			29.6	27.0	25.0	23.5	22.4	21.4	20.5	19.8
	8 × 14				29.6	27.5	25.9	24.6	23.5	22.6	21.8
	10 × 14					29.6	27.9	26.5	25.4	24.4	23.6
60 lbs. per square foot of roof.	3 × 8	16.9	13.4	11.7	10.5	9.4	8.6	7.9	7.5	7.0	6.7
	4 × 8	18.6	14.8	12.9	11.7	10.8	9.9	9.2	8.6	8.1	7.7
	6 × 8		16.9	14.8	13.4	12.5	11.8	11.2	10.5	9.9	9.4
	3 × 10	21.2	16.8	14.7	13.1	11.8	10.8	9.9	9.3	8.8	8.3
	4 × 10	23.3	18.5	16.1	14.7	13.6	12.4	11.5	10.8	10.1	9.6
	6 × 10		21.1	18.5	16.8	15.6	14.7	13.9	13.2	12.4	11.8
	8 × 10			20.3	18.5	17.1	16.1	15.3	14.7	14.1	13.5
	3 × 12	25.4	20.2	17.6	15.8	14.1	13.0	12.0	11.2	10.5	9.9
	4 × 12	28.0	22.2	19.4	17.6	16.3	14.9	13.8	12.9	12.1	11.5
	6 × 12		25.3	22.2	20.2	18.7	17.6	16.8	15.8	14.9	14.1
	8 × 12			24.5	22.2	20.6	19.4	18.4	17.6	16.9	16.3
	10 × 12				23.9	22.2	20.9	19.8	18.9	18.2	17.6
	3 × 14	29.6	23.5	20.6	18.5	16.5	15.1	14.0	13.1	12.3	11.7
	4 × 14	32.6	25.8	22.6	20.5	19.0	17.4	16.2	15.1	14.2	13.5
	6 × 14		29.7	25.8	23.6	21.8	20.5	19.6	18.5	17.4	16.5
	8 × 14			28.5	25.8	24.0	22.6	21.5	20.5	19.7	18.9
	10 × 14				27.9	25.8	24.4	23.1	22.2	21.3	20.6

The maximum spans given in the table for the above loads, are determined by limiting the deflection to $\frac{1}{400}$ of the span, and the maximum fiber strain to 1250 lbs. per square inch, the lesser value given by either condition being used.

YELLOW PINE JOISTS.

Maximum Spans in feet, for the following total uniformly distributed loads.

Total Load.	Size of Joists, inches.	Distance from center to center of joists, inches.						
		12	14	16	18	20	22	24
80 lbs. per square foot of floor.	2 × 8	13.4	12.8	12.2	11.7	11.0	10.5	10.1
	3 × 8	15.4	14.6	13.9	13.4	12.9	12.6	12.2
	2 × 10	16.8	15.9	15.3	14.7	14.2	13.7	13.2
	3 × 10	19.2	18.2	17.4	16.7	16.2	15.7	15.2
	2 × 12	20.2	19.1	18.3	17.6	17.0	16.5	15.8
	3 × 12	23.1	21.9	20.9	20.1	19.4	18.9	18.3
	3 × 14	26.9	25.5	24.4	23.4	22.7	22.0	21.3
	4 × 14	29.6	28.2	26.9	25.9	25.0	24.2	23.6
	3 × 16	30.8	29.2	27.9	26.8	25.9	25.1	24.4
	4 × 16	33.9	32.2	30.8	29.6	28.6	27.7	27.0
100 lbs. per square foot of floor.	2 × 8	12.6	11.8	11.3	10.9	10.3	9.8	9.4
	3 × 8	14.3	13.5	12.9	12.4	12.0	11.7	11.3
	2 × 10	15.6	14.8	14.2	13.6	12.9	12.3	11.8
	3 × 10	17.8	16.9	16.2	15.6	15.0	14.5	14.1
	2 × 12	18.7	17.7	17.0	16.3	15.5	14.8	14.1
	3 × 12	21.5	20.3	19.4	18.7	18.0	17.5	16.9
	3 × 14	25.0	23.7	22.6	21.9	21.0	20.4	19.8
	4 × 14	27.5	26.1	25.0	24.0	23.2	22.5	21.8
	3 × 16	28.5	27.0	25.9	25.0	24.0	23.2	22.6
	4 × 16	31.4	29.8	28.6	27.5	26.6	25.7	25.0
120 lbs. per square foot of floor.	2 × 8	11.7	11.1	10.6	10.0	9.4	8.9	8.6
	3 × 8	13.4	12.7	12.2	11.7	11.3	11.0	10.5
	2 × 10	14.7	13.9	13.2	12.4	11.8	11.2	10.8
	3 × 10	16.8	15.9	15.2	14.6	14.1	13.7	13.2
	2 × 12	17.6	16.7	15.8	14.9	14.2	13.5	12.9
	3 × 12	20.1	19.1	18.3	17.5	16.9	16.5	15.8
	3 × 14	23.5	22.3	21.3	20.4	19.8	19.2	18.6
	4 × 14	25.9	24.6	23.6	22.6	21.8	21.2	20.6
	3 × 16	26.8	25.5	24.4	23.4	22.6	21.9	21.0
	4 × 16	29.6	28.2	26.9	25.8	25.0	24.2	23.5

The maximum spans given in the table for the above loads, are determined by limiting the deflection to $\frac{1}{400}$ of the span, and the maximum fiber strain to 1250 lbs. per square inch, the lesser value given by either condition being used.

YELLOW PINE JOISTS.

Maximum Spans in feet, for the following total uniformly distributed loads.

Total Load.	Size of Joists, inches.	Distance from center to center of joists, feet.								
		2	3	4	5	6	7	8	9	10
125 lbs. per square foot of floor.	4 × 10	14.4	12.2	10.6	9.5	8.6	8.0	7.5	7.1	6.7
	6 × 10	16.5	14.5	12.9	11.5	10.5	9.8	9.2	8.6	8.2
	8 × 10	18.2	15.9	14.4	13.3	12.2	11.3	10.5	9.9	9.4
	10 × 10	19.6	17.1	15.6	14.4	13.6	12.6	11.8	11.1	10.6
	4 × 12	17.3	14.6	12.7	11.3	10.3	9.6	9.0	8.4	8.0
	6 × 12	19.9	17.4	15.5	13.8	12.7	11.7	11.0	10.3	9.8
	8 × 12	21.9	19.1	17.3	16.0	14.6	13.5	12.7	11.9	11.3
	10 × 12	23.6	20.6	18.6	17.3	16.3	15.1	14.1	13.4	12.7
	12 × 12	25.0	21.9	19.8	18.4	17.4	16.5	15.5	14.6	13.9
	4 × 14	20.2	17.1	14.8	13.2	12.1	11.2	10.5	9.9	9.4
	6 × 14	23.3	20.2	18.2	16.2	14.8	13.7	12.8	12.1	11.5
	8 × 14	25.6	22.2	20.2	18.7	17.1	15.8	14.8	14.0	13.2
	10 × 14	27.6	24.0	21.7	20.2	19.0	17.7	16.5	15.6	14.8
	12 × 14	29.2	25.5	23.1	21.5	20.3	19.3	18.1	17.1	16.2
	4 × 16	23.2	19.5	16.9	15.1	13.8	12.7	11.9	11.3	10.7
	6 × 16	26.6	23.2	20.6	18.4	16.8	15.6	14.6	13.8	13.0
	8 × 16	29.2	25.4	23.1	21.3	19.5	18.0	16.9	15.9	15.1
	10 × 16	31.4	27.4	24.8	23.1	21.8	20.1	18.8	17.8	16.9
	12 × 16	33.4	29.2	26.4	24.6	23.2	22.0	20.6	19.5	18.5
175 lbs. per square foot of floor.	4 × 10	12.9	10.3	8.9	8.0	7.3	6.7	6.3	5.9	5.6
	6 × 10	14.8	12.6	10.9	9.8	8.9	8.2	7.7	7.3	6.9
	8 × 10	16.3	14.2	12.6	11.3	10.3	9.5	8.9	8.4	8.0
	10 × 10	17.5	15.3	13.9	12.6	11.5	10.6	10.0	9.4	8.9
	4 × 12	15.1	12.3	10.7	9.6	8.7	8.1	7.6	7.1	6.8
	6 × 12	17.8	15.1	13.1	11.7	10.7	9.9	9.3	8.7	8.3
	8 × 12	19.6	17.1	15.1	13.5	12.3	11.4	10.7	10.1	9.6
	10 × 12	21.0	18.4	16.6	15.1	13.8	12.8	11.9	11.3	10.7
	12 × 12	22.4	19.5	17.7	16.5	15.1	14.0	13.1	12.3	11.7
	4 × 14	17.7	14.4	12.5	11.2	10.2	9.4	8.9	8.4	7.9
	6 × 14	20.8	17.7	15.3	13.7	12.5	11.5	10.8	10.2	9.7
	8 × 14	22.8	19.9	17.7	15.8	14.4	13.3	12.5	11.8	11.2
	10 × 14	24.5	21.4	19.4	17.6	16.1	14.9	13.9	13.2	12.4
	12 × 14	26.2	22.9	20.7	19.3	17.7	16.3	15.3	14.4	13.7
	4 × 16	20.1	16.4	14.2	12.7	11.6	10.7	10.1	9.5	9.0
	6 × 16	23.7	20.1	17.5	15.6	14.3	13.2	12.3	11.6	11.0
	8 × 16	26.0	22.8	20.1	18.0	16.4	15.2	14.2	13.4	12.7
	10 × 16	28.0	24.5	22.2	20.1	18.4	17.0	15.9	15.0	14.2
	12 × 16	29.9	26.1	23.7	22.0	20.2	18.6	17.4	16.5	15.6

The maximum spans given in the table for the above loads are determined by limiting the deflection to $\frac{1}{400}$ of the span, and the maximum fiber strain to 1250 lbs. per square inch, the lesser value given by either condition being used.

SAFE LOADS FOR SEASONED RECTANGULAR TIMBER POSTS,

Calculated from the following formulæ for the safe loads, in lbs. per square inch, on square-ended posts.

Southern Yellow Pine.	White Oak.	White Pine and Spruce.
$\dfrac{1125}{1+\dfrac{l^2}{1100d^2}}$	$\dfrac{925}{1+\dfrac{l^2}{1100d^2}}$	$\dfrac{800}{1+\dfrac{l^2}{1100d^2}}$

These formulæ are deduced from the latest tests of timber posts, and give safe loads of one-fourth the ultimate strength for short posts, decreasing to one-fifth the ultimate for long posts.

Ratio of Length to Least Side, $\frac{l}{d}$	Safe Loads, in lbs. per square inch of Section.		
	Southern Yellow Pine.	White Oak.	White Pine and Spruce.
12	1000	820	710
14	960	790	680
16	910	750	650
18	870	710	620
20	830	680	590
22	780	640	560
24	740	610	530
26	700	570	500
28	660	540	470
30	620	510	440
32	580	480	410
34	550	450	390
36	520	420	370
38	490	400	350
40	460	380	330

l = length of post, in inches.
d = width of smallest side, in inches.

SAFE LOADS FOR

SQUARE TIMBER COLUMNS,

In tons of 2000 lbs.

Kind of Timber.	Unsupported length of Col., in ft.	Size of Column, in inches. 6×6	8×8	9×9	10×10	12×12	14×14	16×16
White Pine or Spruce.	6	12.8						
	8	11.7	22.7	29.6				
	10	10.6	21.3	28.0	35.5			
	12	9.54	19.8	26.3	33.7	51.1		
	14	8.46	18.4	24.7	31.9	49.0	69.6	
	16	7.38	17.0	23.1	30.1	46.8	67.0	91.0
	18		15.5	21.5	28.3	44.7	64.5	88.0
	20		14.1	19.8	26.5	42.5	62.0	85.2
	22			18.2	24.7	40.3	59.5	82.3
	24				22.9	38.2	57.0	79.4
White Oak.	6	14.8						
	8	13.5	26.2	34.0				
	10	12.2	24.6	32.4	41.0			
	12	11.0	22.7	30.4	39.1	59.1		
	14	9.73	21.1	28.4	36.7	56.9	80.4	
	16	8.64	19.5	26.5	34.6	54.0	77.8	105
	18		17.8	24.7	32.4	51.1	74.5	102
	20		16.3	22.7	30.5	49.0	71.3	98.5
	22			21.1	28.2	46.1	68.3	94.7
	24				26.4	43.9	65.5	90.9
Southern Yellow Pine.	6	18.0						
	8	16.4	32.0	41.6				
	10	14.9	29.9	39.4	50.0			
	12	13.3	27.8	36.9	47.6	72.0		
	14	11.9	25.8	34.7	44.7	69.1	98.0	132
	16	10.4	23.7	32.3	42.3	65.5	94.6	128
	18		21.8	30.0	39.5	62.6	90.7	124
	20		19.8	27.8	37.0	59.8	86.9	120
	22			25.7	34.6	56.2	83.6	115
	24				32.2	53.3	80.0	111

$$\text{Safe load in pounds per square inch} = \frac{C}{1 + \frac{l^2}{1100d^2}}$$

Where l = length of column, in inches, and d = width of side, in inches.
For White Pine or Spruce, C = 800; for White Oak, C = 925;
for Southern Yellow Pine, C = 1125.

ROOFS.

The types of roof trusses generally used for spans from 30 ft. to 100 ft. are shown on pages 192 and 193. The King and Queen truss, Fig. 1, is the type usually employed when the construction is a combination of wood and iron; the rafters, diagonal struts and bottom chord being of wood and the verticals of iron or steel rods. This type is sometimes used when the entire construction is to be of steel, though it is not as economical of material as the Belgian or Fink type of trusses, Figs. 2, 3 and 4, which are the most commonly used for steel roofs over mills, shops, warehouses, etc., for spans up to 100 ft. The lower chord is usually horizontal, though for some special reason it may be raised at the center as shown in Figs. 5, 6 and 7 on page 193. This camber of the lower chord materially increases the strains in the truss members, and should therefore, if economy of material is a consideration, be made as small as possible.

Roof trusses are usually made with riveted connections as being the cheapest construction for the usual short spans. A pair of angles may be used for the rafters if the purlins are supported only at the joints, but if the purlins are carried by the rafter at points between the joints, the bending strains produced are usually too great to be sustained by a rafter of this cross section, in which case, the rafter may consist of a pair of angles and a vertical web plate, deeper than the angles, forming a built-up **T** section. The bottom chord, main struts and tension members are best constructed of a pair of angles, while the secondary struts and tension members may be single angles.

For long spans, or heavy loading, pin connections may be desirable, affording convenience in transportation and economy in erection. The compression members are conveniently made of a pair of channels, latticed, and the tension members of steel eyebars or square rods with loop eyes.

When the purlins rest on the rafter between the panel points, the rafter is subjected to a bending strain which must be considered. If the rafter is continuous over panel points it may be considered as a partially continuous beam, and at the center of span between joints the bending will produce compression in the upper fibers and tension in the lower fibers, while at the joints the bending produces reverse effects. The rafters must be proportioned so that the total compressive strain per square inch, due to direct compression and bending, shall not exceed ½ the elastic limit of the material. If the bending moment on the rafter between adjacent panel points be calculated as if for a beam with ends simply supported, the bending moments at the ends and at the

Spans up to 40 ft = 4 per sq. ft. of area covered
" 40 ft " 60 " = 5 " " " " "
" 60 " 80 " = 6 " " " " "
" 80 " 100 " = 7 " " " " "

center of the panel for the continuous rafter may be taken as ⅔ of the maximum bending moment for the simple beam.

The slope of the rafter is usually determined by the kind of roof covering used. Slate should not be used on a slope less than 1 to 3 and preferably 1 to 2. Gravel should not be used on a slope greater than 1 to 4. Corrugated iron if used on a slope less than 1 to 3 is apt to leak under a driving rain, and when possible the slope should not be less than 1 to 2.

Weight of Roof Trusses = Area covered, divided roof area, and multiplied by weight per square foot from above table.

ALLOWABLE STRAINS IN STEEL ROOF TRUSSES.

	lbs. per sq. in.
Tension (shapes)	15,000
Tension rods and eye-bars	18,000
Maximum fiber stress on I beams	16,000
Combined bending and direct strain	15,000
Compression	$13{,}500 - 50\,\frac{l}{r}$

where l = length of member and r = least radius of gyration of member, both in inches.

APPROXIMATE WEIGHT, PER SQUARE FOOT, OF ROOF COVERINGS, EXCLUSIVE OF STEEL CONSTRUCTION.

Corrugated iron, unboarded, No. 26 to No. 18	1 to 3 lbs.
Felt and asphalt, without sheathing	2 "
Felt and gravel, " "	8 to 10 "
Slate, without sheathing, $\frac{3}{16}''$ to $\frac{1}{4}''$,	7 to 9 "
Copper, " "	1 to 1½ "
Tin, " "	1 to 1½ "
Shingles, with lath	2¼ "
Skylight of glass, $\frac{3}{16}''$ to $\frac{1}{2}''$, including frame	4 to 10 "
White pine sheathing, 1″ thick	3 "
Yellow " " 1″ thick	4 "
Spruce sheathing, 1″ thick	2 "
Lath and plaster ceiling	8 to 10 "
Tile, flat	15 to 20 "
Tile, corrugated	8 to 10 "
Tile, on 3″ fireproof blocks	30 to 35 "
Cement 1″ thick	10 "
Spanish Tiling	9 "
Ceiling — Wire Lath, Plaster, Angles 20″ C.t.C.	20 "

Snow for Roof with slope less than $\frac{4}{12}$ = 12# per Sq. Ft.
" " " " " more " " = 8# " " "

The weight of the steel roof construction must be added to the above. For ordinary light roofs, without ceilings, the weight of the steel construction may be taken at 5 lbs. per square foot for spans up to 50 ft., and 1 lb. additional for each 10 ft. increase of span.

It is customary to add 30 lbs. per square foot to the above for wind and snow. No roof should be calculated for a total load less than 40 lbs. per sq. ft.

The total load found as above is to be considered as distributed over the entire truss. It is not necessary to consider the separate effects of wind and snow on spans of less than 100 ft., but for greater spans separate calculations should be made.

The relation between the velocity and pressure of wind against surfaces at right angles to the direction of the wind is given in the following table, based upon experiments conducted by the U. S. Signal Service, at Mt. Washington.

Vel. in miles per hour.	Pressure, lbs. per square foot.	
10	0.4	Fresh breeze.
20	1.6	
30	3.6	Strong wind.
40	6.4	High wind.
50	10.0	Storm.
60	14.4	Violent storm.
80	25.6	Hurricane.
100	40.0	Violent Hurricane.

The components of pressure caused by wind acting upon inclined surfaces are given in the following table:

A = Angle of surface of roof with direction of wind.
F = Force of wind, in lbs. per square foot.
N = Pressure normal to surface of roof.
V = Pressure perpendicular to direction of wind.
H = Pressure parallel to direction of wind.

Angle of Roof.	5°	10°	20°	30°	40°	50°	60°	70°	80°	90°
N = F ×	.125	.24	.45	.66	.83	.95	1.00	1.02	1.01	1.00
V = F ×	.122	.24	.42	.57	.64	.61	.50	.35	.17	.00
H = F ×	.01	.04	.15	.33	.53	.73	.85	.96	.99	1.00

ROOF TRUSSES

LIGHT LINES INDICATE TENSION MEMBERS
HEAVY LINES INDICATE COMPRESSION MEMBERS

FIG. 1.

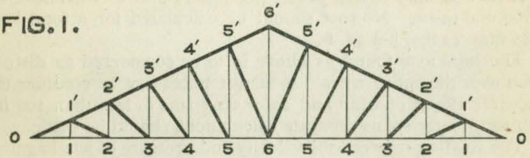


FIG. 2.

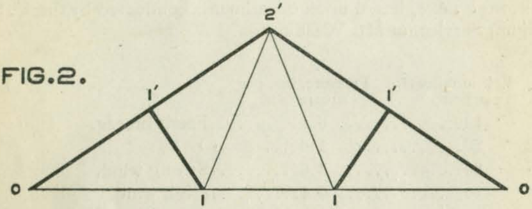


FIG. 3.

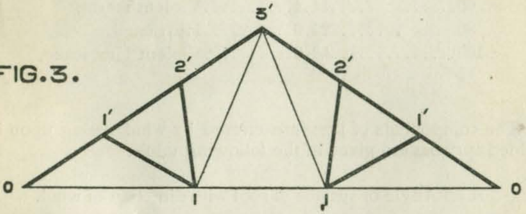
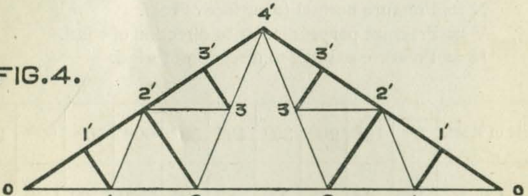


FIG. 4.



CAMBERED ROOF TRUSSES

LIGHT LINES INDICATE TENSION MEMBERS
HEAVY LINES INDICATE COMPRESSION MEMBERS

FIG. 1.

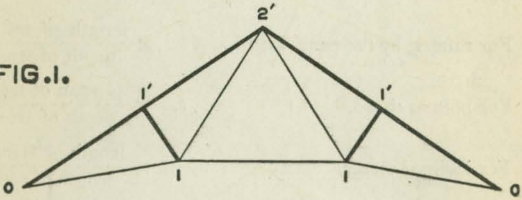


FIG. 2.

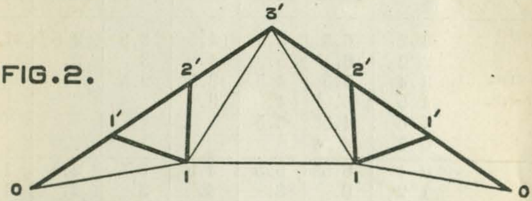
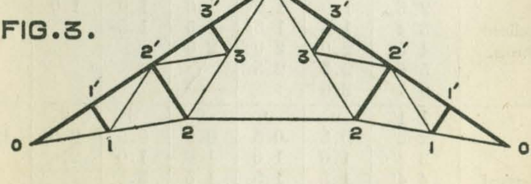


FIG. 3.



MAXIMUM STRAINS IN KING AND QUEEN ROOF TRUSSES.

Fig. 1, Page 192.

To find the maximum strains in any member of these trusses, multiply the co-efficients given here below.

1. For rafters, by the panel load .... $\times \frac{\text{length of rafter}}{\text{depth of truss}}$

2. For bottom chord, " .... $\times \frac{\frac{1}{2}\text{ span of truss}}{\text{depth of truss}}$

3. For inclined struts, " .... $\times \frac{\text{length of strut}}{\text{length of rod}}$

4. For vertical rod, " .... $\times$ 1

	Member.	14 Panel.	12 Panel.	10 Panel.	8 Panel.	6 Panel.	4 Panel.
Bottom Chords.	0 2	6.5	5.5	4.5	3.5	2.5	1.5
	2 3	6.	5.	4.	3.	2.	
	3 4	5.5	4.5	3.5	2.5		
	4 5	5.	4.	3.			
	5 6	4.5	3.5				
	6 7	4.					
Rafters.	0 1′	6.5	5.5	4.5	3.5	2.5	1.5
	1′ 2′	6.	5.	4.	3.	2.	1.
	2′ 3′	5.5	4.5	3.5	2.5	1.5	
	3′ 4′	5.	4.	3.	2.		
	4′ 5′	4.5	3.5	2.5			
	5′ 6′	4.	3.				
	6′ 7′	3.5					
Inclined Struts.	1′ 2	0.5	0.5	0.5	0.5	0.5	0.5
	2′ 3	1.0	1.0	1.0	1.0	1.0	
	3′ 4	1.5	1.5	1.5	1.5		
	4′ 5	2.0	2.0	2.0			
	5′ 6	2.5	2.5				
	6′ 7	3.0					
Vertical Rods.	1 1′	0	0	0	0	0	0
	2 2′	0.5	0.5	0.5	0.5	0.5	1.
	3 3′	1.0	1.0	1.0	1.0	2.	
	4 4′	1.5	1.5	1.5	3.		
	5 5′	2.0	2.0	4.			
	6 6′	2.5	5.				
	7 7′	6.					

MAXIMUM STRAINS IN BELGIAN OR FINK ROOF TRUSSES.

Figs. 2, 3 and 4, Page 192.

Ratio of depth to length of span.			0.333 $\frac{1}{3}$	0.289 $\frac{1}{3.464}$	0.250 $\frac{1}{4}$	0.200 $\frac{1}{5}$	0.167 $\frac{1}{6}$	0.125 $\frac{1}{8}$
Inclinat'n of rafters.			33° 41′	30°	26° 34′	21° 48′	18° 26′	14° 2′
8-panel truss, Fig. 4.	Bottom chord.	0 1	5.25	6.06	7.00	8.75	10.50	14.00
		1 2	4.50	5.19	6.00	7.50	9.00	12.00
		2 2	3.00	3.46	4.00	5.00	6.00	8.00
	Top chord.	0 1′	6.30	7.00	7.83	9.42	11.08	14 44
		1′2′	5.75	6.50	7.38	9.05	10.76	14.20
		2′3′	5.20	6.00	6.93	8.68	10.45	13.95
		3′4′	4.65	5.50	6.48	8.31	10.13	13.71
	Tension braces.	2 3	1.50	1.73	2.00	2.50	3.00	4.00
		3 4′	2.25	2.60	3.00	3.75	4.50	6.00
		12′ & 32′	0.75	0.87	1.00	1.25	1.50	2.00
	Struts.	11′ & 33′	0.83	0.87	0.89	0.93	0.95	0.97
		2 2′	1.66	1.73	1.78	1.86	1.90	1.94
6-panel truss, Fig. 3.	Bottom chord.	0 1	3.75	4.33	5.00	6.25	7.50	10.00
		1 1	2.25	2.60	3.00	3.75	4.50	6.00
	Top chord.	0 1′	4.51	5.00	5.59	6.74	7.91	10.31
		1′2′	3.53	4.00	4.55	5.59	6.65	8.77
		2′3′	3.40	4.00	4.70	6.00	7.29	9.83
	Tension brace.	1 3′	1.50	1.73	2.00	2.50	3.00	4.00
	Struts.	11′ & 12′	.93	1.00	1.07	1.22	1.34	1.62
4-panel truss, Fig. 2.	Bottom chord.	0 1	2.25	2.60	3.00	3.75	4.50	6.00
		1 1	1.50	1.73	2.00	2.50	3.00	4.00
	Top chord.	0 1′	2.70	3.00	3.35	4.04	4.75	6.19
		1′2′	2.15	2.50	2.90	3.67	4.44	5.95
	Rod.	1 2′	0.75	0.87	1.00	1.25	1.50	2.00
	Strut.	1 1′	0.83	0.87	0.89	0.93	0.95	0.97

To find the maximum strain in any member of these trusses, multiply the coefficients given in the table above by the panel load.

MAXIMUM STRAINS IN CAMBERED BELGIAN OR FINK ROOF TRUSSES.

CAMBER = $\frac{1}{6}$ TOTAL HEIGHT.

Figs. 1, 2 and 3, Page 193.

To find the maximum strain in any member of these trusses, multiply the coefficients given in the table below, by the panel load.

Ratio of depth to length of span.			0.333 $\frac{1}{3}$	0.289 $\frac{1}{3.464}$	0.250 $\frac{1}{4}$	0.200 $\frac{1}{5}$	0.167 $\frac{1}{6}$	0.125 $\frac{1}{8}$
Inclinat'n of rafters.			33° 40′	30°	26° 34′	21° 48′	18° 26′	14° 2′
8-panel truss, Fig. 3.	Bottom chord.	0 1	7.17	8.44	9.90	12.61	15.31	20.66
		1 2	6.15	7.23	8.48	10.81	13.12	17.71
		2 2	3.60	4.16	4.80	6.00	7.20	9.60
	Top chord.	0 1′	8.49	9.63	10.96	13.49	16.05	21.21
		1′2′	7.94	9.13	10.51	13.11	15.73	20.98
		2′3′	7.39	8.63	10.06	12.74	15.41	20.74
		3′4′	6.83	8.13	9.61	12.37	15.10	20.49
	Tension braces.	2 3	2.87	3.37	3.96	5.04	6.12	8.26
		3 4′	3.89	4.58	5.37	6.85	8.31	11.21
		12′ & 32′	1.02	1.21	1.41	1.80	2.19	2.95
	Struts.	11′ & 33′	0.83	0.87	0.89	0.93	0.95	0.97
		2 2′	1.66	1.73	1.79	1.86	1.89	1.94
6-panel truss, Fig. 2.	Bottom chord.	0 1	5.12	6.03	7.07	9.01	10.94	14.76
		1 1	2.70	3.12	3.60	4.50	5.40	7.20
	Top chord.	0 1′	6.09	6.88	7.83	9.64	11.47	15.15
		1′2′	4.89	5.63	6.48	8.10	9.72	12.98
		2′3′	4.96	5.88	6.93	8.89	10.83	14.67
	Tie.	1 3′	2.66	3.13	3.67	4.69	5.69	7.67
	Struts.	11′ & 12′	1.04	1.15	1.26	1.49	1.71	2.17
4-panel truss, Fig. 1.	Bottom chord.	0 1	3.07	3.62	4.24	5.40	6.56	8.85
		1 1	1.80	2.08	2.40	3.00	3.60	4.80
	Top chord.	0 1′	3.64	4.13	4.70	5.78	6.88	9.09
		1′2′	3.09	3.63	4.25	5.41	6.56	8.85
	Tie.	1 2′	1.43	1.69	1.98	2.52	3.06	4.11
	Strut.	1 1′	0.83	0.87	0.89	0.93	0.95	0.97

MAXIMUM STRAINS

IN TRUSSES WITH PARALLEL CHORDS.

The maximum strains in the different members of ordinary trusses with parallel chords can be determined by the use of the following tables, if the dead and moving loads are given. In many cases it will be sufficient to consider only a uniform dead load and a uniform live load. The third column gives the influence of a heavier load in front of a uniform load; such as a locomotive at the head of a train of cars.

The panel points are numbered, beginning with 0 at the abutment, those of the bottom chord with plain numbers and those of the top chord with a prime (′) so as to indicate the position of the different members without it being necessary to refer to the diagram.

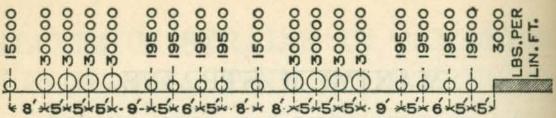
In calculating these tables, the loads were supposed to be concentrated at the lower chord joints for through-bridges, and at the upper chord joints for deck-bridges. In through-bridges the strain, obtained in this manner, for the web members under compression should be increased by the weight of a panel of top chord and top lateral bracing.

Highway bridges are calculated for a live load of 100 lbs. per sq. ft. of floor for all spans up to 100 ft., and 80 lbs. for spans over 200 ft., due provision being made for concentrated loads, such as heavy steam road rollers or electric cars. The dead weight of ordinary highway bridges, exclusive of timber flooring, is given, approximately, by the following formula:

Weight of metal, lbs. per lineal foot of span $= \frac{1}{5} b\, l + 150$

where $l =$ length of bridge, and $b =$ width of floor, both in feet.

Railroad bridges are calculated for concentrated loads typical of the actual load of two locomotives at the head of a train of cars on each track. The following diagram of such a loading is from Theodore Cooper's 1896 Specification for Railroad Bridges, and represents two 106.5 ton locomotives followed by a uniform load of 3,000 lbs. per lineal ft. on one track. For short spans an alternate loading of 100,000 lbs., equally distributed on two driving wheel axles spaced $7\frac{1}{2}$ ft. centers, is also specified.



This loading may be represented by an equivalent uniform load; or, it may also be represented by a uniform load combined with an engine excess. The representation by an equivalent load is not applicable to the calculation of trusses with more than one system of web bracing. Such trusses must be calculated by a uniform load combined with an engine excess. Either method is only an approximation and may give results materially in error. The following table gives the equivalent loads by either method for the above loading for a single track.

Span in feet.	Equivalent Uniform Load, lbs. per foot of Track.		Uniform Load, with Engine Excess.	
	Moments.	Shears.	Uniform Load, lbs. per foot of Track.	Engine Excess, lbs.
10	10,000	12,500	3,400	33,000
15	7,500	10,000	"	32,000
20	6,600	8,100	"	32,000
25	5,900	6,800	"	31,000
30	5,500	6,300	"	30,000
40	4,900	5,600	"	30,000
50	4,600	5,200	"	30,000
75	4,100	4,700	"	30,000
100	4,000	4,500	"	30,000
150	3,800	4,200	"	30,000
200	3,700	3,900	"	30,000
300	3,500	3,700	"	30,000

The weight of track material (ties, rails and guard-rails) is about 400 lbs. per ft. of single track. The weights of railroad bridges, per lineal ft. of span, exclusive of track material, are given, approximately, by the following formulæ, where l = length of span in ft.

Single track, deck plate girder,	$9\,l + 100$
" " " lattice "	$8\,l + 100$
" " through pin truss,	$6\,l + 400$
" " deck " "	$6\,l + 300$
Double track, through pin truss,	$12\,l + 1000$
" " deck " "	$12\,l + 800$

EXAMPLE OF APPLICATION OF TABLE.

WARREN TRUSS, DECK BRIDGE WITH INTERMEDIATE POSTS, FOR SINGLE TRACK RAILROAD.

Span, 150′; Depth, 20′.
Number of panels 10, of 15′ each.
Dead load, 1,600 lbs. per lin. ft. of bridge.
Live load, 3,400 lbs. per lin. ft. of bridge.
D = dead load = 12,000 lbs. per panel for 1 truss.
L = live load = 25,500 " " " 1 "
E = excess of locomotive weight = 15,000 lbs. for 1 truss.

$$l = \frac{25,500}{10} = 2,550$$

$$e = \frac{15,000}{10} = 1,500$$

Length of diagonal members, 25 ft.

$$\text{Sec.} = \frac{25}{20} = 1.25 \qquad \text{Tang.} = \frac{15}{20} = 0.75$$

Strain in middle piece of bottom chord 4—6,

$$\begin{aligned} 12.5\ (D + L) &= 468,750 \\ 25\ e &= \underline{\ 37,500} \\ &\ 506,250 \times \text{tang.} = 379,687. \end{aligned}$$

Compressive strain in brace, 45′.

$$\begin{aligned} 0.5\ D &= 6,000 \\ 15.\ l &= 38,250 \\ 5.\ e &= \underline{\ 7,500} \\ &\ 51,750 \times \text{sec.} = 64,687. \end{aligned}$$

Tensile strain in brace, 5′ 6.

$$\begin{aligned} -0.5\ D &= -6,000 \\ 10.\ l &= 25,500 \\ 4.\ e &= \underline{\ 6,000} \\ &\ 25,500 \times \text{sec.} = 31,875. \end{aligned}$$

It will be observed that, by beginning with 0 at the left-hand abutment, the compression member 45′ becomes the tension member 5′ 6, and the maximum strains change from 64,687 compression to 31,875 tension. The strains in the other members are found in a similar manner.

The load on any of the intermediate posts is found as follows:

$$\begin{aligned} 15 \text{ ft.} \times 1,700 &= 25,500 \\ E &= \underline{16,000} \\ &\ 41,500 \end{aligned}$$

TRUSSES WITH PARALLEL CHORDS

FIG. 1.

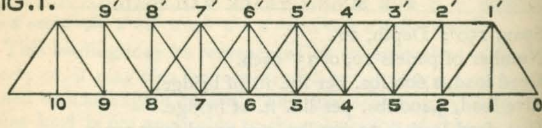


FIG. 2.

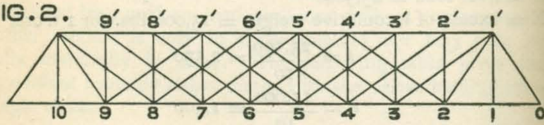


FIG. 3.

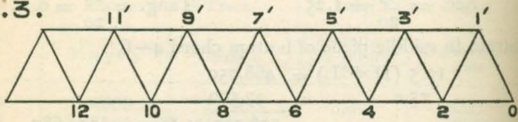


FIG. 4.

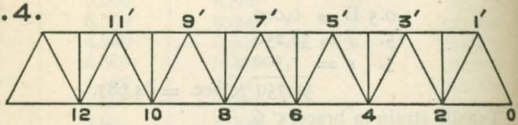


FIG. 5.

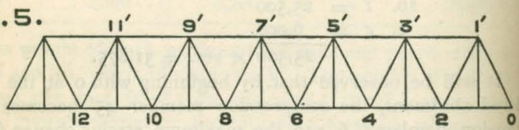
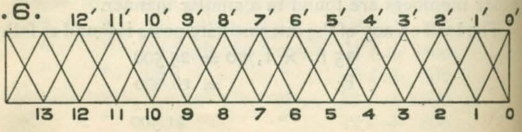


FIG. 6.



MAXIMUM STRAINS PRODUCED BY DEAD AND LIVE LOADS IN SINGLE INTERSECTION RECTANGULAR TRUSSES.

(Fig. 1, page 200.) END-POSTS INCLINED, EQUAL PANELS, THROUGH AND DECK BRIDGES.

Member.			12 Panel Truss.			11 Panel Truss.			10 Panel Truss.			9 Panel Truss.			8 Panel Truss.			7 Panel Truss.			6 Panel Truss.			5 Panel Truss.			4 Panel Truss.		
Thro'gh Post.	Deck Post.	Diag.	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*
		0 1′	5.5	66	11	5	55	10	4.5	45	9	4	36	8	3.5	28	7	3	21	6	2.5	15	5	2	10	4	1.5	6	3
	2 2′	2 1′	4.5	55	10	4	45	9	3.5	36	8	3	28	7	2.5	21	6	2	15	5	1.5	10	4	1	6	3	0.5	3	2
2 2′	3 3′	3 2′	3.5	45	9	3	36	8	2.5	28	7	2	21	6	1.5	15	5	1	10	4	0.5	6	3	0	3	2	-0.5	1	1
3 3′	4 4′	4 3′	2.5	36	8	2	28	7	1.5	21	6	1	15	5	0.5	10	4	0	6	3	-0.5	3	2	-1	1	1			
4 4′	5 5′	5 4′	1.5	28	7	1	21	6	0.5	15	5	0	10	4	-0.5	6	3	-1	3	2	-1.5	1	1						
5 5′	6 6′	6 5′	0.5	21	6	0	15	5	-0.5	10	4	-1	6	3	-1.5	3	2												
6 6′		7 6′	-0.5	15	5	-1	10	4	-1.5	6	3																		
		8 7′	-1.5	10	4	-2	6	3																					
		9 8′	-2.5	6	3																								

$l = \frac{L}{n}$ $e = \frac{E}{n}$

Diagonals: multiply by Sec.
Posts: multiply by 1.
Chords: multiply by Tang.

Top.	Bott.	12 D	12 L	12 *e*	11 D	11 L	11 *e*	10 D	10 L	10 *e*	9 D	9 L	9 *e*	8 D	8 L	8 *e*	7 D	7 L	7 *e*	6 D	6 L	6 *e*	5 D	5 L	5 *e*	4 D	4 L	4 *e*
	0 2	5.5	5.5	11	5	5	10	4.5	4.5	9	4	4	8	3.5	3.5	7	3	3	6	2.5	2.5	5	2	2	4	1.5	1.5	3
1′ 2′	2 3	10.	10.	20	9	9	18	8.	8.	16	7	7	14	6.	6.	12	5	5	10	4.	4.	8	3	3	6	2.	2.	4
2′ 3′	3 4	13.5	13.5	27	12	12	24	10.5	10.5	21	9	9	18	7.5	7.5	15	6	6	12	4.5	4.5	9						
3′ 4′	4 5	16.	16.	32	14	14	28	12.	12.	24	10	10	20	8.	8.	16												
4′ 5′	5 6	17.5	17.5	35	15	15	30	12.5	12.5	25																		
5′ 6′		18.	18.	36																								

$\text{Sec.} = \frac{\text{Length of inclined member}}{\text{Depth of truss}}$ $\text{Tang.} = \frac{\text{Length of panel}}{\text{Depth of truss}}$

n = No. of panels.
D = Dead load per panel.
L = Live load per panel.
E = Excess of engine load over general live load on 1 panel.

MAXIMUM STRAINS PRODUCED BY DEAD AND LIVE LOADS IN DOUBLE INTERSECTION RECTANGULAR TRUSSES.

(Fig. 2, Page 200.)

WITH INCLINED END-POSTS, EQUAL PANELS, FOR THROUGH AND DECK BRIDGES.

Vertical Members.		Inclined Memb'rs.	18 Panels.			17 Panels.			16 Panels.			15 Panels.			14 Panels.			13 Panels.			12 Panels.			11 Panels.			10 Panels.		
Through.	Deck.		*d*	*l*	*e*	*d*	*l*	*e*	*d*	*l*	*e*	*d*	*l*	*e*	*d*	*l*	*e*	*d*	*l*	*e*	*d*	*l*	*e*	*d*	*l*	*e*	*d*	*l*	*e*
		0 1′	153	153	17	136	136	16	120	120	15	105	105	14	91	91	13	78	78	12	66	66	11	55	55	10	45	45	9
	2 2′	2 1′	72	72	16	63	64	15	56	56	14	48	49	13	42	42	12	35	36	11	30	30	10	24	25	9	20	20	8
	3 3′	3 1′	63	64	15	56	56	14	48	49	13	42	42	12	35	36	11	30	30	10	24	25	9	20	20	8	15	16	7
2 2′	4 4′	4 2′	54	56	14	46	49	13	40	42	12	33	36	11	28	30	10	22	25	9	18	20	8	13	16	7	10	12	6
3 3′	5 5′	5 3′	45	49	13	39	42	12	32	36	11	27	30	10	21	25	9	17	20	8	12	16	7	9	12	6	5	9	5
4 4′	6 6′	6 4′	36	42	12	29	36	11	24	30	10	18	25	9	14	20	8	9	16	7	6	12	6	2	9	5	0	6	4
5 5′	7 7′	7 5′	27	36	11	22	30	10	16	25	9	12	20	8	7	16	7	4	12	6	0	9	5	−2	6	4	−5	4	3
6 6′	8 8′	8 6′	18	30	10	12	25	9	8	20	8	3	16	7	0	12	6	−4	9	5	−6	6	4	−9	4	3	−10	2	2
7 7′	9 9′	9 7′	9	25	9	5	20	8	0	16	7	−3	12	6	−7	9	5	−9	6	4	−12	4	3	−13	2	2			
8 8′		10 8′	0	20	8	−5	16	7	−8	12	6	−12	9	5	−14	6	4	−17	4	3									
9 9′		11 9′	−9	16	7	−12	12	6	−16	9	5	−18	6	4															
		12 10′	−18	12	6	−22	9	5	−24	6	4																		

n = Number of panels.
D = Dead load per panel.
L = Live load per panel.
E = Excess of engine load over general live load on first panel loaded.

Multiply for inclined members by Sec.
" " vertical " " one.

$d=\frac{D}{n}$ $l=\frac{L}{n}$ $e=\frac{E}{n}$

MAXIMUM STRAINS PRODUCED BY DEAD AND LIVE LOADS IN DOUBLE INTERSECTION RECTANGULAR TRUSSES—*Continued.*

Top Chord	Bottom Chord.	18 Panels.		17 Panels.		16 Panels.		15 Panels.		14 Panels.		13 Panels.		12 Panels.		11 Panels.		10 Panels.	
		D+L	*e*	*d+l*	*e*	D+L	*e*	*d+l*	*e*	D+L	*e*	*d+l*	*e*	D+L	*e*	*d+l*	*e*	D+L	*e*
	0 2	8.5	17	136	16	7.5	15	105	14	6.5	13	78	12	5.5	11	55	10	4.5	9
	2 3	12.5	32	199	30	11	28	153	26	9.5	24	113	22	8	20	79	18	6.5	16
1′ 2′	3 4	19.5	45	311	42	17	39	237	36	14.5	33	173	30	12	27	119	24	9.5	21
2′ 3′	4 5	25.5	56	403	52	22	48	303	44	18.5	40	217	36	15	32	145	28	11.5	24
3′ 4′	5 6	30.5	65	481	60	26	55	357	50	21.5	45	251	40	17	35	163*	30	12.5	25
4′ 5′	6 7	34.5	72	539	66	29	60	393	54	23.5	48	269*	42	18	36	167	30		
5′ 6′	7 8	37.5	77	583	70	31	63	417*	56	24.5	49	277	42					4′ 5′=	3′ 4′
6′ 7′	8 9	39.5	80	607*	72	32	64	423	56					5′ 6′=	4′ 5′	*56=			
7′ 8′		40.5	81	617	72			*78=		6′ 7′=	5′ 6′	*67=				159	30		
		8′ 9′=	7′ 8′	8′ 9′=	7′ 8′	7′ 8′=	6′ 7′	411	56			261	42						
				*89=															
				597	72														

$$\text{Sec.} = \frac{\text{Length of inclined member}}{\text{Depth of truss}}$$

Chords: multiply by Tang.

$$\text{Tang.} = \frac{\text{Length of panel}}{\text{Depth of truss}}$$

Maximum Strains Produced by Dead and Live Loads in Single Intersection Triangular or Warren Through Trusses.

(Fig. 3, Page 200.)

Multiply by Secant.

Members. Comp.	Members. Tens.	20 Panels. D	l	e	18 Panels. D	l	e	16 Panels. D	l	e	14 Panels. D	l	e	12 Panels. D	l	e	10 Panels. D	l	e	8 Panels. D	l	e	6 Panels. D	l	e	4 Panels. D	l	e
0 1′	1′ 2	4.5	90	18	4	72	16	3.5	56	14	3	42	12	2.5	30	10	2	20	8	1.5	12	6	1	6	4	0.5	2	2
2 3′	3′ 4	3.5	72	16	3	56	14	2.5	42	12	2	30	10	1.5	20	8	1	12	6	0.5	6	4	0	2	2			
4 5′	5′ 6	2.5	56	14	2	42	12	1.5	30	10	1	20	8	0.5	12	6	0	6	4	-0.5	2	2						
6 7′	7′ 8	1.5	42	12	1	30	10	0.5	20	8	0	12	6	-0.5	6	4	-1	2	2									
8 9′	9′ 10	0.5	30	10	0	20	8	-0.5	12	6	-1	6	4	-1.5	2	2												
10 11′	11′ 12	-0.5	20	8	-1	12	6	-1.5	6	4	-2	2	2															
12 13′	13′ 14	-1.5	12	6	-2	6	4	-2.5	2	2																		
14 15′	15′ 16	-2.5	6	4	-3	2	2																					

$$\text{Sec.} = \frac{\text{Length of braces}}{\text{Depth of truss}}$$

n = Number of panels.
D = Dead load per double panel.
L = Live load per double panel.
E = Excess of engine load over general live load.

Multiply by Tang.

Chords.		20 Panels. D+L	e	18 Panels. D+L	e	16 Panels. D+L	e	14 Panels. D+L	e	12 Panels. D+L	e	10 Panels. D+L	e	8 Panels. D+L	e	6 Panels. D+L	e	4 Panels. D+L	e
0	2	4.5	18	4	16	3.5	14	3	12	2.5	10	2	8	1.5	6	1	4	0.5	2
1′	3′	9	36	8	32	7	28	6	24	5.	20	4	16	3.	12	2	8	1.0	4
2	4	12.5	48	11	42	9.5	36	8	30	6.5	24	5	18	3.5	12	2	6		
3′	5′	16	64	14	56	12	48	10	40	8.	32	6	24	4.	16				
4	6	18.5	70	16	60	13.5	50	11	40	8.5	30	6	20						
5′	7′	21	84	18	72	15	60	12	48	9.	36								
6	8	22.5	84	19	70	15.5	56	12	42										
7′	9′	24	96	20	80	16	64												
8	10	24.5	90	20	72														
9′	11′	25	100																

$$\text{Tang.} = \frac{\text{Length of panel}}{\text{Depth of truss}} \qquad l = \frac{L}{n} \qquad e = \frac{E}{n}$$

Maximum Strains Produced by Dead and Live Loads in Single Intersection Triangular or Warren Deck Trusses. (Fig. 3, Page 200.)

	Members.		20 Panels.			18 Panels.			16 Panels.			14 Panels.			12 Panels.			10 Panels.			8 Panels.			6 Panels.			4 Panels.		
	Tens.	Compr.	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*
Multiply by Sec.		0 1′	5	100	19	4.5	81	17	4	64	15	3.5	49	13	3	36	11	2.5	25	9	2	16	7	1.5	9	5	1	4	3
	1′ 2	2 3′	4	81	17	3.5	64	15	3	49	13	2.5	36	11	2	25	9	1.5	16	7	1	9	5	0.5	4	3	0	1	1
	3′ 4	4 5′	3	64	15	2.5	49	13	2	36	11	1.5	25	9	1	16	7	0.5	9	5	0	4	3	-0.5	1	1			
	5′ 6	6 7′	2	49	13	1.5	36	11	1	25	9	0.5	16	7	0	9	5	-0.5	4	3	-1	1	1						
	7′ 8	8 9′	1	36	11	0.5	25	9	0	16	7	-0.5	9	5	-1	4	3	-1.5	1	1									
	9′ 10	10 11′	0	25	9	-0.5	16	7	-1	9	5	-1.5	4	3	-2	1	1												
	11′ 12	12 13′	-1	16	7	-1.5	9	5	-2	4	3	-2.5	1	1															
	13′ 14	14 15′	-2	9	5	-2.5	4	3	-3	1	1																		
	15′ 16	16 17′	-3	4	3																								
	Chords.		D+L		*e*	D+L		*e*	D+L		*e*	D+L		*e*	D+L		*e*	D+L		*e*	D+L		*e*	D+L		*e*	D+L		*e*
Multiply by Tang.	0	2	5		19	4.5		17	4		15	3.5		13	3		11	2.5		9	2		7	1.5		5	1		3
	1′	3′	9		36	8.		32	7		28	6.		24	5		20	4.5		16	3		12	2		8	1		4
	2	4	13		51	11.5		45	10		39	8.5		33	7		27	5.5		21	4		15	2.5		9			
	3′	5′	16		64	14.		56	12		48	10.		40	8		32	6.		24	4		16						
	4	6	19		75	16.5		65	14		55	11.5		45	9		35	6.5		25									
	5′	7′	21		84	18.		72	15		60	12.		48	9		36												
	6	8	23		91	19.5		77	16		63	12.5		49															
	7′	9′	24		96	20.		80	16		64																		
	8	10	25		99	20.5		81																					
	9′	11′	25		100																								

MAXIMUM STRAINS PRODUCED BY DEAD AND LIVE LOADS IN SINGLE INTERSECTION TRIANGULAR OR WARREN GIRDERS,

THROUGH OR DECK BRIDGES, WITH INTERMEDIATE SUSPENDERS OR POSTS (Figs. 4 and 5, Page 200), WITH INCLINED END-POSTS AND EQUAL PANELS.

Inclined Members, multiply by Sec.

Members.		20 Panels.			18 Panels.			16 Panels.			14 Panels.			12 Panels.			10 Panels.			8 Panels.			6 Panels.			4 Panels.		
Compr.	Tens.	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*	D	*l*	*e*
0 1′		9.5	190	19	8.5	153	17	7.5	120	15	6.5	91	13	5.5	66	11	4.5	45	9	3.5	28	7	2.5	15	5	1.5	6	3
	1′ 2	8.5	171	18	7.5	136	16	6.5	105	14	5.5	78	12	4.5	55	10	3.5	36	8	2.5	21	6	1.5	10	4	0.5	3	2
2 3′		7.5	153	17	6.5	120	15	5.5	91	13	4.5	66	11	3.5	45	9	2.5	28	7	1.5	15	5	0.5	6	3	-0.5	1	1
	3′ 4	6.5	136	16	5.5	105	14	4.5	78	12	3.5	55	10	2.5	36	8	1.5	21	6	0.5	10	4	-0.5	3	2			
4 5′		5.5	120	15	4.5	91	13	3.5	66	11	2.5	45	9	1.5	28	7	0.5	15	5	-0.5	6	3	-1.5	1	1			
	5′ 6	4.5	105	14	3.5	78	12	2.5	55	10	1.5	36	8	0.5	21	6	-0.5	10	4	-1.5	3	2						
6 7′		3.5	91	13	2.5	66	11	1.5	45	9	0.5	28	7	-0.5	15	5	-1.5	6	3	-2.5	1	1						
	7′ 8	2.5	78	12	1.5	55	10	0.5	36	8	-0.5	21	6	-1.5	10	4	-2.5	3	2									
8 9′		1.5	66	11	0.5	45	9	-0.5	28	7	-1.5	15	5	-2.5	6	3	-3.5	1	1									
	9′ 10	0.5	55	10	-0.5	36	8	-1.5	21	6	-2.5	10	4	-3.5	3	2												
10 11′		-0.5	45	9	-1.5	28	7	-2.5	15	5	-3.5	6	3	-4.5	1	1												
	11′ 12	-1.5	36	8	-2.5	21	6	-3.5	10	4	-4.5	3	2															
12 13′		-2.5	28	7	-3.5	15	5	-4.5	6	3	-5.5	1	1															
	13′ 14	-3.5	21	6	-4.5	10	4	-5.5	3	2																		
14 15′		-4.5	15	5	-5.5	6	3	-6.5	1	1																		
	15′ 16	-5.5	10	4	-6.5	3	2																					
16 17′		-6.5	6	3	-7.5	1	1																					

SINGLE INTERSECTION TRIANGULAR OR WARREN GIRDERS— *Continued.*

Chords, multiply by Tang.

Memb's	20 Panels.		18 Panels		16 Panels.		14 Panels.		12 Panels.		10 Panels.		8 Panels.		6 Panels.		4 Panels.	
Chords.	D+L	*e*	D+L	*e*	D+L	*e*	D+L	*e*	D+L	*e*	D+L	*e*	D+L	*e*	D+L	*e*	D+L	*e*
0 2	9.5	19	8.5	17	7.5	15	6.5	13	5.5	11	4.5	9	3.5	7	2.5	5	1.5	3
1′ 3′	18.	36	16.	32	14.	28	12.	24	10.	20	8.	16	6.	12	4.	8	2.	4
2 4	25.5	51	22.5	45	19.5	39	16.5	33	13.5	27	10.5	21	7.5	15	4.5	9		
3′ 5′	32.	64	28.	56	24.	48	20.	40	16.	32	12.	24	8.	16				
4 6	37.5	75	32.5	65	27.5	55	22.5	45	17.5	35	12.5	25						
5′ 7′	42.	84	36.	72	30.	60	24.	48	18.	36								
6 8	45.5	91	38.5	77	31.5	63	24.5	49										
7′ 9′	48.	96	40.	80	32.	64												
8 10	49.5	99	40.5	81														
9′ 11′	50.	100																

n = No. of panels
D = Dead load per panel.
L = Live load per panel.
E = Excess of engine load over general live load, for 1 panel.

$$l = \frac{L}{n} \qquad e = \frac{E}{n}$$

$$\text{Sec.} = \frac{\text{Length of inclined member}}{\text{Depth of truss}}$$

$$\text{Tang.} = \frac{\text{Length of panel}}{\text{Depth of truss}}$$

MAXIMUM STRAINS PRODUCED BY DEAD AND LIVE LOADS IN DOUBLE INTERSECTION TRIANGULAR OR WARREN GIRDERS,

THROUGH OR DECK BRIDGES. (Fig. 6, Page 200.)

Members. Deck.		Through.		20 Panels.			18 Panels.			16 Panels.			14 Panels.			12 Panels.			10 Panels.			8 Panels.			6 Panels.			4 P'ls.		
Compr.	Tens.	Compr.	Tens.	D	l	e	D	l	e	D	l	e	D	l	e	D	l	e	D	l	e	D	l	e	D	l	e	D	l	e
1' 0			0' 1	5	100	19	4.5	81	17	4	64	15	3.5	49	13	3	36	11	2.5	25	9	2.	16	7	1.5	9	5	1	4	3
2' 1	1 0'	0 1'	1' 2	4.5	90	18	4	72	16	3.5	56	14	3	42	12	2.5	30	10	2	20	8	1.5	12	6	1	6	4	0.5	2	2
3' 2	2 1'	1 2'	2' 3	4	81	17	3.5	64	15	3	49	13	2.5	36	11	2	25	9	1.5	16	7	1	9	5	0.5	4	3	0	1	1
4' 3	3 2'	2 3'	3' 4	3.5	72	16	3	56	14	2.5	42	12	2	30	10	1.5	20	8	1	12	6	0.5	6	4	0	2	2			
5' 4	4 3'	3 4'	4' 5	3	64	15	2.5	49	13	2	36	11	1.5	25	9	1	16	7	0.5	9	5	0	4	3	-0.5	1	1			
6' 5	5 4'	4 5'	5' 6	2.5	56	14	2	42	12	1.5	30	10	1	20	8	0.5	12	6	0	6	4	-0.5	2	2						
7' 6	6 5'	5 6'	6' 7	2	49	13	1.5	36	11	1	25	9	0.5	16	7	0	9	5	-0.5	4	3	-1	1	1						
8' 7	7 6'	6 7'	7' 8	1.5	42	12	1	30	10	0.5	20	8	0	12	6	-0.5	6	4	-1	2	2									
9' 8	8 7'	7 8'	8' 9	1	36	11	0.5	25	9	0	16	7	-0.5	9	5	-1	4	3	-1.5	1	1									
10' 9	9 8'	8 9'	9' 10	0.5	30	10	0	20	8	-0.5	12	6	-1	6	4	-1.5	2	2												
11' 10	10 9'	9 10'	10' 11	0	25	9	-0.5	16	7	-1	9	5	-1.5	4	3	-2	1	1												
12' 11	11 10'	10 11'	11' 12	-0.5	20	8	-1.0	12	6	-1.5	6	4	-2	2	2															
13' 12	12 11'	11 12'	12' 13	-1	16	7	-1.5	9	5	-2	4	3	-2.5	1	1															
14' 13	13 12'	12 13'	13' 14	-1.5	12	6	-2	6	4	-2.5	2	2																		
15' 14	14 13'	13 14'	14' 15	-2	9	5	-2.5	4	3	-3.5	1	1																		
16' 15	15 14'	14 15'	15' 16	-2.5	6	4	-3	2	2																					
17' 16	16 15'	15 16'	16' 17	-3	4	3	-3.5	1	1																					
For Inclined End-posts 1' 1 = 0' 1, multiplied by 1. The Strains in End-posts are:																														
0 1'		0 1'		9.5	190	19	8.5	153	17	7.5	120	15	6.5	91	13	5.5	66	11	4.5	45	9	3.5	28	7	2.5	15	5	1.5	6	3

n = Number of panels.
D = Dead load per panel.
L = Live load per panel.
E = Excess of engine load over general live load for 1 panel.

$$l = \frac{L}{n} \qquad e = \frac{E}{n}$$

DOUBLE INTERSECTION WARREN GIRDERS — *Continued.*

Chords. Deck. Top.	Deck. Bottom.	Through. Top.	Through. Bottom.	20 Panels. D+L	20 Panels. *e*	18 Panels. D+L	18 Panels. *e*	16 Panels. D+L	16 Panels. *e*	14 Panels. D+L	14 Panels. *e*	12 Panels. D+L	12 Panels. *e*	10 Panels. D+L	10 Panels. *e*	8 Panels. D+L	8 Panels. *e*	6 Panels. D+L	6 Panels. *e*	4 Panels. D+L	4 Panels. *e*
0′ 1′			0 1	4.5	18	4	16	3.5	14	3	12	2.5	10	2	8	1.5	6	1	4	0.5	2
	0 1	0′ 1′		5	19	4.5	17	4	15	3.5	13	3	11	2.5	9	2	7	1.5	5	1	3
1′ 2′			1 2	13.5	34	12	30	10.5	26	9	22	7.5	18	6	14	4.5	10	3	6	1.5	2
	1 2	1′ 2′		14	36	12.5	32	11	28	9.5	24	8	20	6.5	16	5	12	3.5	8	2	4
2′ 3′			2 3	21.5	48	19	42	16.5	36	14	30	11.5	24	9	18	6.5	12	4	6		
	2 3	2′ 3′		22	51	19.5	45	17	39	14.5	33	12	27	9.5	21	7	15	4.5	9		
3′ 4′			3 4	28.5	60	25	52	21.5	44	18	36	14.5	28	11	20	7.5	12				
	3 4	3′ 4′		29	64	25.5	56	22	48	18.5	40	15	32	11.5	24	8	16				
4′ 5′			4 5	34.5	70	30	60	25.5	50	21	40	16.5	30	12	20						
	4 5	4′ 5′		35	75	30.5	65	26	55	21.5	45	17	35	12.5	25						
5′ 6′			5 6	39.5	78	34	66	28.5	54	23	42	17.5	30								
	5 6	5′ 6′		40	84	34.5	72	29	60	23.5	48	18	36								
6′ 7′			6 7	43.5	84	37	70	30.5	56	24	42										
	6 7	6′ 7′		44	91	37.5	77	31	63	24.5	49										
7′ 8′			7 8	46.5	88	39	72	31.5	56												
	7 8	7′ 8′		47	96	39.5	80	32	64												
8′ 9′			8 9	48.5	90	40	72														
	8 9	8′ 9′		49	99	40.5	81														
9′ 10′			9 10	49.5	90																
	9 10	9′ 10′		50	100																
For Inclined End-posts:																					
	0 1		0 1	9.5	19	8.5	17	7.5	15	6.5	13	5.5	11	4.5	9	3.5	7	2.5	5	1.5	3

Multiply diagonals by Sec.
" chords by Tang.
" vertical members by one.

$$\text{Sec.} = \frac{\text{Length of inclined member}}{\text{Depth of truss}} \qquad \text{Tang.} = \frac{\text{Length of panel}}{\text{Depth of truss}}$$

THE PASSAIC ROLLING MILL COMPANY'S STANDARD TURNTABLES.

The table is entirely center bearing, and rests on hardened steel discs, which offer very little resistance to turning, and at the same time are of sufficiently large diameter to give ample bearing surface to maintain them in good working order, and prevent abrasion by excessive pressure. The discs are six inches in diameter for the smaller tables, and seven inches for the larger sizes. The tables are suspended from the saddle and center pin by two bolts made of rolled iron. Two bolts are used, in preference to four, to avoid the uneven distribution of the load produced by the tightening of the bolts, which is liable to occur when more than two are used. A wrought iron band is shrunk around the top of the pivot casting, and is effective, in case of the binding of the discs, in resisting the strains produced by the tipping of the table. The vertical adjustment of the table is easily made with the suspending bolts, and without the use of packing plates or other devices. The flanges are made of six inch angle irons, extending the full length of the table without splices, and re-enforced at the center with cover plates. The sections of the flanges are proportioned with due regard to the effect of the reversal of strains at any point of either flange due to the shifting position of the locomotive, and the stresses are kept low to avoid excessive deflection at the ends of the table when loaded. The girders are connected to each other with rigid angle iron bracing effectively secured to the flanges, and with six transverse frames, also of angle iron. The center and saddle castings and the end bearing wheels are open hearth steel castings.

The 55 ft. and the 60 ft. turntables are made in three weights,

(1). A heavy pattern for turning 103 ton locomotives.
(2). A medium pattern " " 90 " "
(3). A light pattern " " 75 " "

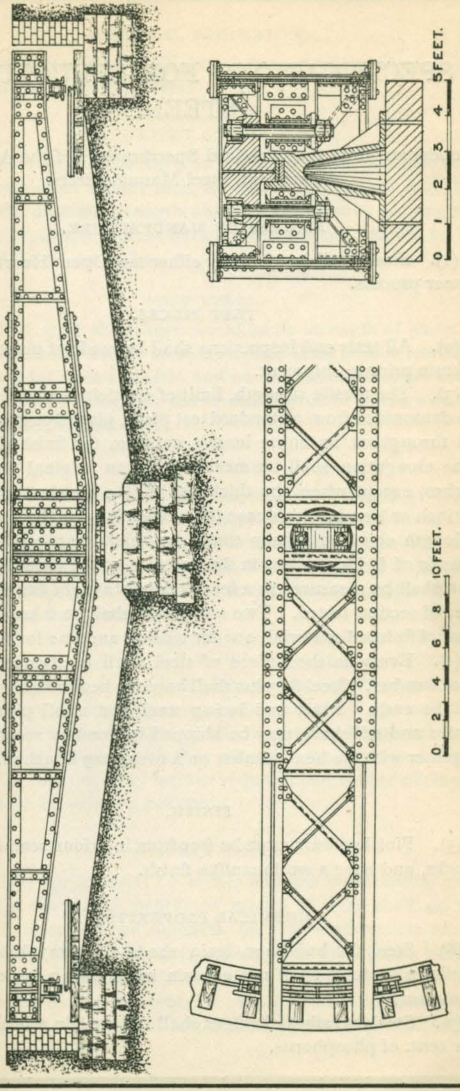
Where shipment can be made by rail, the tables are loaded on cars, complete, ready to set in the pit. Dimensions for building the pit, and instructions for setting the table accompany each contract.

When the pits are already built the tables can be made to fit them at a slight additional cost.

DIMENSIONS OF PASSAIC STANDARD TURNTABLES.

Diameter of Pit	40′ 0″	45′ 0″	50′ 0″	55′ 0″	60′ 0″
Length of Girder, out to out....	39′ 4″	44′ 4″	49′ 6″	54′ 6″	59′ 6″
Diameter of Circular Tracks, center to center of Rail	36′ 0″	41′ 0′	46′ 0″	51′ 0″	56′ 0″
Depth from top of Rail on Table to top of Center Stone........	5′ 0″	5′ 0″	5′ 6″	5′ 6″	5′ 6″
Depth from top of Rail on Table to top of Rail of Circular Track.	3′ 4″	3′ 4″	3′ 10″	3′ 10″	3′ 10″
Depth from top of Rail on Table to top of Rail of Circular Track, for Special Turn Table with shallow Pit	2′ 0″	2′ 0″	2′ 6″	2′ 6″	2′ 6″

PASSAIC STANDARD TURNTABLES



SPECIFICATIONS FOR STRUCTURAL STEEL.

Condensed from the Standard Specifications of the Association of American Steel Manufacturers.

PROCESS OF MANUFACTURE.

(1). Steel shall be made by either the Open Hearth or Bessemer process.

TEST PIECES.

(2). All tests and inspections shall be made at place of manufacture prior to shipment.

(3). The tensile strength, limit of elasticity and ductility shall be determined from a standard test piece, planed or turned parallel throughout its entire length, cut from the finished material. The elongation shall be measured on an original length of 8 inches, except when the thickness of the finished material is $\frac{5}{16}$ inch or less, in which case the elongation shall be measured in a length equal to sixteen times the thickness; and except in rounds of $\frac{5}{8}$ inch or less in diameter, in which case the elongation shall be measured in a length equal to eight times the diameter of section tested. Two test pieces shall be taken from each heat of finished material, one for tension and one for bending.

(4). Every finished piece of steel shall be stamped with the heat number. Steel for pins shall have the heat numbers stamped on the ends. Rivet and lacing steel, and small pieces for tie plates and stiffeners, may be shipped in bundles securely wired together with the heat number on a metal tag attached.

FINISH.

(5). Finished bars must be free from injurious seams, flaws or cracks, and have a workmanlike finish.

CHEMICAL PROPERTIES.

(6). Steel for buildings, train sheds, highway bridges and similar structures shall not contain more than 0.10 per cent. of phosphorus.

(7). Steel for railway bridges shall not contain more than 0.08 per cent. of phosphorus.

PHYSICAL PROPERTIES.

(8). Structural steel shall be of three grades: Rivet Steel, Soft Steel, and Medium Steel.

RIVET STEEL.

(9). Rivet steel shall have an ultimate strength of 48,000 to 58,000 pounds per square inch, an elastic limit of not less than one-half the ultimate strength, and an elongation of 26 per cent., and shall bend, 180 degrees flat on itself, without fracture on the outside of the bent portion.

SOFT STEEL.

(10). Soft steel shall have an ultimate strength of 52,000 to 62,000 pounds per square inch, an elastic limit of not less than one-half the ultimate strength, and an elongation of 25 per cent., and shall bend 180 degrees, flat on itself, without fracture on the outside of the bent portion.

MEDIUM STEEL.

(11). Medium steel shall have an ultimate strength of 60,000 to 70,000 pounds per square inch, an elastic limit of not less than one-half the ultimate strength, and an elongation of 22 per cent., and shall bend 180 degrees, around a curve having a diameter equal to the thickness of the piece tested, without fracture on the outside of the bent portion.

PIN STEEL.

(12). Pins made from either of the above mentioned grades of steel shall, on specimen test pieces cut at a depth of one inch from the surface of finished material, fill the physical requirements of the grade of steel from which they are rolled for ultimate strength, elastic limit and bending, but the required percentage of elongation shall be decreased 5 per cent.

EYE-BAR STEEL.

(13). Eye-bar material $1\frac{1}{2}$ inches and less in thickness, made of either of the above mentioned grades of steel, shall, on test pieces cut from finished material, fill the requirements of the grade of steel from which it is rolled. For thicknesses greater than $1\frac{1}{2}$ inches, there will be allowed a reduction in percentage of elongation of one per cent. for each $\frac{1}{8}$ of an inch increase in thickness, to a minimum of 20 per cent. for medium steel and 22 per cent. for soft steel.

FULL SIZE TEST OF STEEL EYE-BARS.

(14). Full size tests of steel eye-bars shall be required to show not less than 10 per cent. elongation in the body of the bar, and a tensile strength not more than 5,000 pounds below the minimum tensile strength required in specimen tests of the grade of steel from which the bars are rolled. The bars will be required to break in the body; should a bar break in the head, but develop 10 per cent. elongation and the ultimate strength specified, it shall not be cause for rejection, provided not more than one-third of the total number of bars tested break in the head.

VARIATION IN WEIGHT.

(15). A variation in cross-section or weight of more than 2½ per cent. from that specified will be sufficient cause for rejection, except in the case of sheared plates.

When sheared plates are ordered by weight, the permissible variation shall not be more than 2½ per cent. from that specified, except for plates ¼″ to $\frac{5}{16}$″ thick (10.2 to 12.75 lbs. per square foot), which, when ordered to weight, shall not average a variation greater than 5 per cent. above or below the theoretical weight for plates over 75″ wide.

When sheared plates are ordered to gauge, the overweight shall not exceed the percentages given in the following table: —

PERCENTAGES OF ALLOWABLE OVERWEIGHTS FOR SHEARED PLATES WHEN ORDERED TO GAUGE.

Thickness of Plate.	Width of Plate.		
	Up to 75 inches.	75 to 100 inches.	Over 100 inches.
¼ inch.	10	14	18
$\frac{5}{16}$ "	8	12	16
⅜ "	7	10	13
$\frac{7}{16}$ "	6	8	10
½ "	5	7	9
$\frac{9}{16}$ "	4½	6½	8½
⅝ "	4	6	8
Over ⅝ inch.	3½	5	6½

CORRUGATED IRON.

Corrugated iron is largely used for roofing and siding of buildings and can be applied directly upon steel purlins or studding by means of clips of hoop iron, placed not more than 12″ apart, which encircle the purlin or stud. The projecting edges at the gables and eaves must be secured to prevent the sheets being loosened or folded up by the wind.

The usual dimensions of corrugated iron are given in the subjoined table. The $2\frac{1}{2}$ inch corrugation is the one generally employed for roofing and siding, and the regular lengths of sheets are 6, 7, 8, 9 and 10 ft.

DIMENSIONS OF SHEETS AND CORRUGATIONS.

Width of Corrugation.	Depth of Corrugation.	No. of Corrugations to the Sheet.	Cov. width after lapping one Corrugation.	Width of Sheet after Corrugation.	Length of longest Sheets.
$2\frac{1}{2}$ inch.	$\frac{5}{8}$ inch.	10	24 inch.	26 inch.	10 ft.
$1\frac{1}{4}$ "	$\frac{1}{2}$ "	$19\frac{1}{2}$	24 "	26 "	8 ft.
$\frac{3}{4}$ "	$\frac{1}{4}$ "	$34\frac{1}{2}$	25 "	26 "	8 ft.

Roofing is measured by the square, equal to 100 sq. ft. of finished roofing in place. The corrugated sheets are usually laid with one corrugation lap on the sides and an end lap of 6″ for roofing and 2″ for siding.

NUMBER OF SQUARE FEET OF $2\frac{1}{2}''$ CORRUGATED IRON REQUIRED TO LAY ONE SQUARE.

Side Lap, One Corrugation.

Length of Sheet, Feet.	Length of End Lap.					
	1 inch.	2 inch.	3 inch.	4 inch.	5 inch.	6 inch.
5	110	112	114	116	118	120
6	110	111	113	115	117	118
7	110	110	112	114	115	117
8	109	110	112	113	114	115
9	109	110	112	113	114	115
10	108	109	110	111	112	113

CORRUGATED IRON *(Continued).*

The maximum spans for roofing and siding are as follows:

	No. 16.	No. 18.	No. 20.	No. 22.	No. 24.	No. 26.	No. 28.
Roofing,	5′ 9″	5′ 0″	4′ 3″	4′ 0″	3′ 6″	3′ 0″	2′ 9″
Siding,	7′ 0″	6′ 3″	5′ 3″	4′ 9″	4′ 3″	3′ 9″	3′ 3″

and if used on greater spans the excessive deflection is liable to impair the tightness of the joints.

Numbers 20 and 22 are the gauges most in use for roofs, and number 24 for siding. The sheets may be either painted or galvanized.

The United States standard gauge, adopted by Act of Congress in 1893, is in general use by manufacturers of sheet iron. The following table gives the thickness and weight of corrugated iron in accordance with United States standard gauge.

No. by United States Gauge.	Thickness, inches.	Weight per Square Foot, Flat, lbs.	Weight per Sq. Ft., Corrugated, lbs.	Weight per Square of 100 Square Feet, when laid, allowing 6″ lap in length, and 2½″ or one Corrugation in width of sheet, for sheet lengths of:						Galvanized. Wgt. per Sq. Ft., Corrugated.	
				5′	6′	7′	8′	9′	10′		Cole
16	.0625	2.50	2.75	331	325	320	318	315	311	2.91	
18	.05	2.00	2.20	264	260	256	254	252	249	2.36	
20	.0375	1.50	1.65	198	195	193	190	189	187	1.82	
22	.0313	1.25	1.38	166	163	161	159	158	156	1.54	1.51
24	.025	1.00	1.11	134	131	130	128	127	126	1.27	1.25
26	.0188	.75	.84	101	100	99	98	96	95	.99	
28	.0156	.63	.69	83	82	81	80	79	78	.86	

TRANSVERSE STRENGTH OF CORRUGATED IRON.

The transverse strength of corrugated iron may be calculated in the following manner:

l = unsupported length of sheet, in inches.

t = thickness of sheet, in inches.

b = width of sheet, in inches.

d = depth of corrugation, in inches.

w = safe uniformly distributed load, in pounds.

Then, $w = \frac{25{,}000\ b\ t\ d}{l}$

RIVETS AND PINS.

In proportioning riveted work the friction is neglected between the parts connected as it is an uncertain element. The rivets must resist the whole strain which is to be transmitted from one part to the other, and they must be of sufficient size and number to present ample resistance to shearing, and afford sufficient bearing area so as not to cause a crushing of the metal at the rivet holes. It is, therefore, always necessary to calculate rivet connections for shear as well as for bearing. The usual strains, lbs. per square inch, allowable on riveted work are as follows: —

Rivets.	Shearing.	Bearing.
Iron rivets, railroad bridges,	6,000	12,000
Iron rivets, highway bridges and buildings,	7,500	15,000
Steel rivets, railroad bridges,	7,500	15,000
Steel rivets, highway bridges and buildings,	9,000	18,000

The following tables give the shearing and bearing values of rivets, of different diameters, for the above strains. Single shear occurs when a single shearing across the body of the rivet suffices to produce separation of the parts connected; as, for instance, when a thick plate is connected with another single thick plate by means of a rivet, the connection can fail only by a single shearing of the body of the rivet. If, however, the plates are thin they may not offer sufficient bearing against the rivet to prevent rupture by the rivet bodily crushing the plates; the latter condition is determined by the bearing value of the rivet upon the plates. If a $\frac{3}{4}''$ diameter rivet is used, and the plates are only $\frac{1}{4}''$ thick, by reference to the tables, it will be found that the bearing value of the rivet on a $\frac{1}{4}''$ plate is less than its value in single shear, and the bearing value of the rivet determines the strength of the connection.

Pins are subject to strains by shearing, bearing and bending, but their resistance to the latter two, in almost every case, determines the size of the pin to be used. The usual allowable strains, lbs. per square inch, on pins are as follows:

Pins.	Shearing.	Bearing.	Bending.
Iron pins, railroad bridges,	7,500	12,000	15,000
Iron pins, highway bridges and buildings,	9,000	15,000	18,000
Steel pins, railroad bridges,	9,000	15,000	18,000
Steel pins, highway bridges and buildings,	11,250	18,000	22,500

The following tables give the shearing, bearing and bending values of pins, of different diameters, for the above strains.

SHEARING AND BEARING VALUES AND MAXIMUM BENDING MOMENTS OF PINS.

Diameter of Pin, Inches.	Area of Pin, Sq. Ins.	Maximum Bending Moments. S=15,000 lbs. per □ in.	S=18,000 lbs. per □ in.	S=22,500 lbs. per □ in.	Bearing Values for 1″ Thickness of Plate. At 12,000 lbs. per □ in.	At 15,000 lbs. per □ in.	At 18,000 lbs. per □ in.	Shearing Values. At 7,500 lbs. per □ in.	At 9,000 lbs. per □ in.	At 11,250 lbs. per □ in.
$1\frac{7}{16}$	1.62	4,370	5,240	6,550	17,200	21,600	25,900	12,150	14,600	18,200
$1\frac{11}{16}$	2.24	7,070	8,480	10,610	20,200	25,300	30,400	16,800	20,200	25,200
$1\frac{15}{16}$	2.95	10,700	12,840	16,050	23,200	29,100	34,900	22,100	26,550	33,200
$2\frac{3}{16}$	3.76	15,400	18,480	23,110	26,200	32,800	39,400	28,200	33,800	42,300
$2\frac{7}{16}$	4.67	21,310	25,580	31,970	29,200	36,600	43,900	35,000	42,000	52,500
$2\frac{11}{16}$	5.67	28,570	34,280	42,850	32,200	40,300	48,400	42,500	51,000	63,800
$2\frac{15}{16}$	6.78	37,310	44,770	55,960	35,200	44,100	52,900	50,850	61,000	76,300
$3\frac{3}{16}$	7.98	47,670	57,200	71,500	38,200	47,800	57,400	59,850	71,800	89,800
$3\frac{7}{16}$	9.28	59,790	71,750	89,680	41,200	51,600	61,900	69,600	83,500	104,400
$3\frac{11}{16}$	10.68	73,810	88,570	110,710	44,200	55,300	66,400	80,100	96,100	120,150
$3\frac{15}{16}$	12.18	89,860	107,830	134,790	47,200	59,100	70,900	91,350	109,600	137,000
$4\frac{3}{8}$	15.03	123,300	147,960	185,000	52,500	65,600	78,750	112,700	135,300	169,100
$4\frac{5}{8}$	16.80	145,700	174,800	218,500	55,500	69,400	83,300	126,000	151,200	189,000
$4\frac{7}{8}$	18.66	170,600	204,700	255,900	58,500	73,100	87,750	140,000	167,900	209,900
$5\frac{3}{8}$	22.69	228,700	274,400	343,000	64,500	80,600	96,750	170,150	204,200	255,200

SHEARING AND BEARING VALUES AND MAXIMUM BENDING MOMENTS OF PINS *(Continued).*

Diameter of Pin, Inches.	Area of Pin, Sq. Ins.	Maximum Bending Moments.			Bearing Values for 1″ Thickness of Plate.			Shearing Values.		
		S = 15,000 lbs. per □ in.	S = 18,000 lbs. per □ in.	S = 22,500 lbs. per □ in.	At 12,000 lbs. per □ in.	At 15,000 lbs. per □ in.	At 18,000 lbs. per □ in.	At 7,500 lbs. per □ in.	At 9,000 lbs. per □ in.	At 11,250 lbs. per □ in.
5⅝	24.85	262,100	314,500	393,100	67,500	84,400	101,250	187,000	224,000	280,000
5⅞	27.11	298,600	358,300	447,900	70,500	88,100	105,750	203,000	244,000	305,000
6	28.27	318,100	381,700	477,100	72,000	90,000	108,000	212,000	254,000	318,000
6¼	30.68	359,500	431,400	539,300	75,000	93,800	112,500	230,000	276,000	345,000
6½	33.18	404,400	485,300	606,600	78,000	97,500	117,000	249,000	299,000	373,000
6¾	35.79	452,900	543,500	679,400	81,000	101,300	121,500	268,000	322,000	403,000
7	38.48	505,100	606,100	757,700	84,000	105,000	126,000	289,000	346,000	433,000
7½	44.18	621,300	745,500	931,900	90,000	112,500	135,000	331,000	398,000	497,000
8	50.27	754,000	904,800	1,131,000	96,000	120,000	144,000	377,000	452,000	568,000
8½	56.75	904,400	1,085,200	1,356,600	102,000	127,500	153,000	426,000	511,000	638,000
9	63.62	1,073,500	1,288,200	1,610,300	108,000	135,000	162,000	477,000	573,000	716,000
9½	70.88	1,262,600	1,515,100	1,893,900	114,000	142,500	171,000	532,000	638,000	797,000
10	78.54	1,472,600	1,767,100	2,208,900	120,000	150,000	180,000	589,000	707,000	884,000
11	95.03	1,960,100	2,352,100	2,940,100	132,000	165,000	198,000	712,000	855,000	1,069,000
12	113.10	2,544,700	3,053,600	3,817,000	144,000	180,000	216,000	848,000	1,018,000	1,272,000

SHEARING AND BEARING VALUE OF RIVETS.

Diam. of Bolt at Root of Thread — Single Shear at 6000 lbs. per Sq. In.	Diameter of Rivet, Inches.	Area of Rivet, Sq. In.	Single Shear at 6,000 lbs. per Sq. In.	Bearing Value at 12,000 lbs. per Sq. In. for Different Thicknesses of Plate, in Inches.								
				1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4
408	3/8	.110	660	1120								
756	1/2	.196	1180	1500	1880	2250						
1212	5/8	.307	1840	1860	2320	2790	3250	3720				
1812	3/4	.442	2650	2250	2810	3370	3940	4500	5060			
2520	7/8	.601	3610	2630	3280	3940	4590	5250	5910	6560	7220	
3300	1	.785	4710	3000	3750	4500	5250	6000	6750	7500	8250	9000

Single Shear at 7500 lbs per Sq. In.	Diameter of Rivet, Inches.	Area of Rivet, Sq. In.	Single Shear at 7,500 lbs. per Sq. In.	Bearing Value at 15,000 lbs. per Sq. In. for Different Thicknesses of Plate, in Inches.								
				1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4
340	5/16	.077	580	1170								
510	3/8	.110	830	1410								
945	1/2	.196	1470	1880	2340	2810						
1515	5/8	.307	2300	2340	2930	3520	4100					
2265	3/4	.442	3310	2810	3520	4220	4920	5630	6330			
3150	7/8	.601	4510	3280	4100	4920	5740	6560	7380	8200	9020	
4125	1	.785	5890	3750	4690	5620	6560	7500	8440	9380	10310	11250

SHEARING AND BEARING VALUE OF RIVETS *(Continued)*.

Single Shear at 9000 lbs per Sq. In.	Diameter of Rivet, Inches.	Area of Rivet, Sq. In.	Single Shear at 9,000 lbs. per Sq. In.	Bearing Value at 18,000 lbs. per Sq. In. for Different Thicknesses of Plate, in Inches.								
				$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$
612	$\frac{3}{8}$	.110	990	1680								
1134	$\frac{1}{2}$	.196	1770	2250	2820	3370						
1818	$\frac{5}{8}$	.307	2760	2790	3480	4180	4870	5580				
2718	$\frac{3}{4}$	.442	3970	3370	4210	5050	5910	6750	7590			
3780	$\frac{7}{8}$	.601	5410	3940	4920	5910	6880	7870	8860	9840	10830	
4950	1	.785	7060	4500	5620	6750	7870	9000	10120	11250	12370	13500

Diameter of Rivet, Inches.	Area of Rivet, Sq. In.	Single Shear at 10,000 lbs. per Sq. In.	Bearing Value at 20,000 lbs. per Sq. In. for Different Thicknesses of Plate, in Inches.								
			$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{11}{16}$	$\frac{3}{4}$
$\frac{3}{8}$	.110	1100	1880								
$\frac{1}{2}$	.196	1960	2500	3130	3750						
$\frac{5}{8}$	.307	3070	3130	3910	4690	5470					
$\frac{3}{4}$	.442	4420	3750	4690	5630	6560	7500	8440			
$\frac{7}{8}$	.601	6010	4380	5470	6570	7660	8750	9840	10940	12030	
1	.785	7850	5000	6250	7500	8750	10000	11250	12500	13750	15000

WEIGHT OF RIVETS, AND ROUND-HEADED BOLTS WITHOUT NUTS, PER 100.

Lengths from under head.

Length, Inches.	$\frac{3}{8}''$ Dia.	$\frac{1}{2}''$ Dia.	$\frac{5}{8}''$ Dia.	$\frac{3}{4}''$ Dia.	$\frac{7}{8}''$ Dia.	$1''$ Dia.	$1\frac{1}{4}''$ Dia.
$1\frac{1}{4}$	5.4	12.6	21.5	28.7	43.1	65.3	123.
$1\frac{1}{2}$	6.2	13.9	23.7	31.8	47.3	70.7	133.
$1\frac{3}{4}$	6.9	15.3	25.8	34.9	51.4	76.2	142.
2	7.7	16.6	27.9	37.9	55.6	81.6	150.
$2\frac{1}{4}$	8.5	18.0	30.0	41.0	59.8	87.1	159.
$2\frac{1}{2}$	9.2	19.4	32.2	44.1	63.0	92.5	167.
$2\frac{3}{4}$	10.0	20.7	34.3	47.1	68.1	98.0	176.
3	10.8	22.1	36.4	50.2	72.3	103.	184.
$3\frac{1}{4}$	11.5	23.5	38.6	53.3	76.5	109.	193.
$3\frac{1}{2}$	12.3	24.8	40.7	56.4	80.7	114.	201.
$3\frac{3}{4}$	13.1	26.2	42.8	59.4	84.8	120.	210.
4	13.8	27.5	45.0	62.5	89.0	125.	218.
$4\frac{1}{4}$	....	28.9	47.1	65.6	93.2	131.	227.
$4\frac{1}{2}$	....	30.3	49.2	68.6	97.4	136.	236.
$4\frac{3}{4}$	....	31.6	51.4	71.7	102.	142.	244.
5	....	33.0	53.5	74.8	106.	147.	253.
$5\frac{1}{4}$	....	....	55.6	77.8	110.	153.	261.
$5\frac{1}{2}$	....	....	57.7	80.9	114.	158.	270.
$5\frac{3}{4}$	....	....	59.9	84.0	118.	163.	278.
6	....	....	62.0	87.0	122.	169.	287.
$6\frac{1}{2}$	....	...	....	93.2	131.	180.	304.
7	....	....	....	99.3	139.	191.	321.
$7\frac{1}{2}$	....	....	....	106.	147.	202.	338.
8	....	....	....	112.	156.	213.	355.
100 Heads.	1.8	5.7	10.9	13.4	22.2	38.0	82.0

LENGTH OF RIVET SHANK REQUIRED TO FORM ONE RIVET HEAD.

All dimensions in inches.

Grip.	Button Head.					Countersunk Head.				
	Diameter of Rivet.					Diameter of Rivet.				
	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1
$\frac{1}{2}$ to $1\frac{3}{8}$	1	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$	$1\frac{5}{8}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{3}{4}$	$\frac{7}{8}$	$\frac{7}{8}$
$1\frac{1}{2}$ to $2\frac{7}{8}$	$1\frac{1}{8}$	$1\frac{3}{8}$	$1\frac{1}{2}$	$1\frac{5}{8}$	$1\frac{3}{4}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	$\frac{7}{8}$	1
3 to $4\frac{3}{8}$	$1\frac{1}{4}$	$1\frac{1}{2}$	$1\frac{5}{8}$	$1\frac{3}{4}$	$1\frac{7}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	1	$1\frac{1}{8}$
$4\frac{1}{2}$ to $5\frac{1}{2}$	$1\frac{3}{8}$	$1\frac{5}{8}$	$1\frac{3}{4}$	$1\frac{7}{8}$	2				1	$1\frac{1}{8}$

WEIGHT OF 100 BOLTS WITH SQUARE HEADS AND NUTS.

(Hoopes and Townsend's List.)

Length under head to point.	DIAMETER OF BOLTS.								
	$\frac{1}{4}$ in.	$\frac{5}{16}$ in.	$\frac{3}{8}$ in.	$\frac{7}{16}$ in.	$\frac{1}{2}$ in.	$\frac{5}{8}$ in.	$\frac{3}{4}$ in.	$\frac{7}{8}$ in.	1 in.
	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.	lbs.
1½	4.0	7.0	10.5	15.2	22.5	39.5	63.0	.....	...
1¾	4.4	7.5	11.3	16.3	23.8	41.6	66.0	.....	...
2	4.8	8.0	12.0	17.4	25.2	43.8	69.0	109.0	163
2¼	5.2	8.5	12.8	18.5	26.5	45.8	72.0	113.3	169
2½	5.5	9.0	13.5	19.6	27.8	48.0	75.0	117.5	174
2¾	5.8	9.5	14.3	20.7	29.1	50.1	78.0	121.8	180
3	6.3	10.0	15.0	21.8	30.5	52.3	81.0	126.0	185
3½	7.0	11.0	16.5	24.0	33.1	56.5	87.0	134.3	196
4	7.8	12.0	18.0	26.2	35.8	60.8	93.1	142.5	207
4½	8.5	13.0	19.5	28.4	38.4	65.0	99.1	151.0	218
5	9.3	14.0	21.0	30.6	41.1	69.3	105.2	159.6	229
5½	10.0	15.0	22.5	32.8	43.7	73.5	111.3	168.0	240
6	10.8	16.0	24.0	35.0	46.4	77.8	117.3	176.6	251
6½	....	....	25.5	37.2	49.0	82.0	123.4	185.0	262
7	....	....	27.0	39.4	51.7	86.3	129.4	193.7	273
7½	....	....	28.5	41.6	54.3	90.5	135.0	202.0	284
8	....	....	30.0	43.8	59.6	94.8	141.5	210.7	295
9	....	....	....	46.0	64.9	103.3	153.6	227.8	317
10	....	....	....	48.2	70.2	111.8	165.7	224.8	339
11	....	....	....	50.4	75.5	120.3	177.8	261.9	360
12	....	....	....	52.6	80.8	128.8	189.9	278.9	382
Per in. additional.	1.4	2.1	3.1	4.2	5.5	8.5	12.3	16.7	21.8

WEIGHTS OF NUTS AND BOLT-HEADS, IN POUNDS.

For Calculating the Weight of Longer Bolts.

Diameter of Bolt in Inches.		$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$
Weight of Hexagon Nut and Head..................	....	.017	.057	.128	.267	.43	.73
Weight of Square Nut and Head..................	....	.021	.069	.164	.320	.55	.88
Diameter of Bolt in Inches.	1	1¼	1½	1¾	2	2½	3
Weight of Hexagon Nut and Head..................	1.10	2.14	3.78	5.6	8.75	17	28.8
Weight of Square Nut and Head..................	1.31	2.56	4.42	7.0	10.5	21	36.4

12500 [illegible]

BOLTS AND NUTS.

BOLTS. U. S. Standard Screw Threads.					NUTS. Manufacturers Standard.			
					Hexagon.		Square.	
Diam. of Bolt, Ins.	No. of Threads per Inch.	Diam. at Root of Thread, Inches.	Area of Body of Bolt, Sq. Ins.	Area at Root of Thread, Sq. Ins.	Short Diam., Ins.	Long Diam., Ins.	Side of Square, Ins.	Diagonal, Ins.
$\frac{1}{4}$	20	.185	.049	.027	$\frac{1}{2}$	0.58	$\frac{1}{2}$	0.71
$\frac{5}{16}$	18	.240	.077	.045	$\frac{5}{8}$	0.72	$\frac{5}{8}$	0.88
$\frac{3}{8}$	16	.294	.110	.068	$\frac{3}{4}$	0.87	$\frac{3}{4}$	1.06
$\frac{7}{16}$	14	.344	.150	.093	$\frac{7}{8}$	1.01	$\frac{7}{8}$	1.24
$\frac{1}{2}$	13	.400	.196	.126	1	1.15	1	1.41
$\frac{9}{16}$	12	.454	.249	.162	$1\frac{1}{8}$	1.30	$1\frac{1}{8}$	1.59
$\frac{5}{8}$	11	.507	.307	.201	$1\frac{1}{4}$	1.44	$1\frac{1}{4}$	1.77
$\frac{3}{4}$	10	.620	.442	.302	$1\frac{3}{8}$	1.59	$1\frac{1}{2}$	2.12
$\frac{7}{8}$	9	.731	.601	.419	$1\frac{5}{8}$	1.88	$1\frac{3}{4}$	2.47
1	8	.837	.785	.550	$1\frac{3}{4}$	2.02	2	2.83
$1\frac{1}{8}$	7	.940	.994	.694	2	2.31	$2\frac{1}{4}$	3.18
$1\frac{1}{4}$	7	1.06	1.23	.890	$2\frac{1}{4}$	2.60	$2\frac{1}{2}$	3.54
$1\frac{3}{8}$	6	1.16	1.48	1.06	$2\frac{1}{2}$	2.89	$2\frac{3}{4}$	3.89
$1\frac{1}{2}$	6	1.28	1.77	1.29	$2\frac{3}{4}$	3.18	3	4.24
$1\frac{5}{8}$	$5\frac{1}{2}$	1.39	2.07	1.51	3	3.46	$3\frac{1}{4}$	4.60
$1\frac{3}{4}$	5	1.49	2.40	1.74	$3\frac{1}{4}$	3.75	$3\frac{1}{2}$	4.95
$1\frac{7}{8}$	5	1.61	2.76	2.05	$3\frac{1}{2}$	4.04	$3\frac{3}{4}$	5.30
2	$4\frac{1}{2}$	1.71	3.14	2.30	$3\frac{1}{2}$	4.04	4	5.66
$2\frac{1}{4}$	$4\frac{1}{2}$	1.96	3.98	3.02	$3\frac{3}{4}$	4.33	$4\frac{1}{4}$	6.01
$2\frac{1}{2}$	4	2.17	4.91	3.71	$4\frac{1}{4}$	4.91	$4\frac{1}{2}$	6.36
$2\frac{3}{4}$	4	2.42	5.94	4.62	$4\frac{1}{2}$	5.20	$4\frac{3}{4}$	6.72
3	$3\frac{1}{2}$	2.63	7.07	5.43	$4\frac{3}{4}$	5.48	5	7.07
$3\frac{1}{4}$	$3\frac{1}{2}$	2.88	8.30	6.51	5	5.77	$5\frac{1}{2}$	7.78
$3\frac{1}{2}$	$3\frac{1}{4}$	3.10	9.62	7.55	$5\frac{1}{4}$	6.06	$5\frac{3}{4}$	8.13
$3\frac{3}{4}$	3	3.32	11.04	8.64	6	6.93	$6\frac{1}{2}$	9.19
4	3	3.57	12.57	10.00	$6\frac{1}{2}$	7.51	7	9.90
$4\frac{1}{4}$	$2\frac{7}{8}$	3.80	14.19	11.33	7	8.09	$7\frac{1}{2}$	10.61
$4\frac{1}{2}$	$2\frac{3}{4}$	4.03	15.90	12.74	$7\frac{1}{2}$	8.58	8	11.31
$4\frac{3}{4}$	$2\frac{5}{8}$	4.25	17.72	14.23	$7\frac{3}{4}$	8.95	$8\frac{1}{4}$	11.67
5	$2\frac{1}{2}$	4.48	19.63	15.76	8	9.24	$8\frac{1}{2}$	12.02

Diam. of Upset.
$\frac{7}{8}$, 1, $1\frac{1}{8}$, $1\frac{1}{4}$, $1\frac{3}{8}$, $1\frac{1}{2}$, $1\frac{3}{4}$, $1\frac{7}{8}$, 2, $2\frac{1}{8}$, $2\frac{1}{4}$, $2\frac{3}{8}$, $2\frac{5}{8}$, 3, $3\frac{1}{4}$, $3\frac{1}{2}$, $3\frac{3}{4}$, 4

Weight of 1 Nut (Square)

Length of Threads cut on H & T Bolts.
Length of Bolt = 1 to $1\frac{1}{2}$, $1\frac{5}{8}$ to 2, $2\frac{1}{8}$ to $2\frac{1}{2}$, $2\frac{5}{8}$ to 3, $3\frac{1}{8}$ to 4
Size " " $\frac{5}{8}$ = $1\frac{1}{4}$ " " " "
" " " $\frac{3}{4}$ = $1\frac{1}{2}$ " " " "

MANUFACTURERS STANDARD,

SQUARE AND HEXAGON HOT-PRESSED NUTS.

NUMBER OF EACH SIZE IN 100 LBS.

Size of Bolt, Inches.	Number of Square.	Number of Hexagon.	Size of Bolt, Inches.	Number of Square.	Number of Hexagon.
$\frac{1}{4}$	6,800	8,000	$1\frac{3}{8}$	41.0	56.0
$\frac{5}{16}$	3,480	4,170	$1\frac{1}{2}$	31.3	42.0
$\frac{3}{8}$	2,050	2,410	$1\frac{5}{8}$	24.8	33.4
$\frac{7}{16}$	1,290	1,460	$1\frac{3}{4}$	19.9	26.7
$\frac{1}{2}$	850	1,020	$1\frac{7}{8}$	16.2	21.5
$\frac{9}{16}$	600	710	2	13.4	22.4
$\frac{5}{8}$	440	520	$2\frac{1}{4}$	10.7	17.7
$\frac{3}{4}$	251	370	$2\frac{1}{2}$	8.9	12.3
$\frac{7}{8}$	159	226	$2\frac{3}{4}$	7.3	10.2
1	106	176	3	6.2	8.7
$1\frac{1}{8}$	73	104	$3\frac{1}{4}$	4.7	7.5
$1\frac{1}{4}$	54	75	$3\frac{1}{2}$	4.0	6.3

STANDARD SIZES OF WASHERS.

NUMBER IN 100 LBS.

Size of Bolt, Inches.	Diameter of Washer, Inches.	Size of Hole, Inches.	Thickness, Wire Gauge.	Average Number in 100 lbs.	Per lb.
$\frac{1}{4}$	$\frac{3}{4}$	$\frac{5}{16}$	16	13,845	
$\frac{5}{16}$	$\frac{7}{8}$	$\frac{3}{8}$	16	11,220	
$\frac{3}{8}$	1	$\frac{7}{16}$	14	6,573	66
$\frac{7}{16}$	$1\frac{1}{4}$	$\frac{1}{2}$	14	4,261	
$\frac{1}{2}$	$1\frac{3}{8}$	$\frac{9}{16}$	12	2,683	27
$\frac{9}{16}$	$1\frac{1}{2}$	$\frac{5}{8}$	12	2,249	
$\frac{5}{8}$	$1\frac{3}{4}$	$\frac{11}{16}$	10	1,315	13
$\frac{3}{4}$	2	$\frac{13}{16}$	10	1,013	10
$\frac{7}{8}$	$2\frac{1}{4}$	$\frac{15}{16}$	9	858	
1	$2\frac{1}{2}$	$1\frac{1}{16}$	9	617	
$1\frac{1}{8}$	$2\frac{3}{4}$	$1\frac{1}{4}$	9	516	
$1\frac{1}{4}$	3	$1\frac{3}{8}$	9	403	
$1\frac{3}{8}$	$3\frac{1}{4}$	$1\frac{1}{2}$	8	320	
$1\frac{1}{2}$	$3\frac{1}{2}$	$1\frac{5}{8}$	8	278	
$1\frac{5}{8}$	$3\frac{3}{4}$	$1\frac{3}{4}$	8	247	
$1\frac{3}{4}$	4	$1\frac{7}{8}$	8	224	
$1\frac{7}{8}$	$4\frac{1}{4}$	2	8	200	
2	$4\frac{1}{2}$	$2\frac{1}{8}$	8	180	
$2\frac{1}{4}$	$4\frac{3}{4}$	$2\frac{3}{8}$	6	110	
$2\frac{1}{2}$	5	$2\frac{5}{8}$	6	91	

BUCKLE PLATES.

Buckle plates are used for concrete, asphaltor stone paved floors of buildings and highway bridges. The width of the plates varies from 3 ft. to 5 ft., and the thickness from $\frac{1}{4}''$ to $\frac{3}{8}''$. The thickness should never be less than $\frac{1}{4}''$, while $\frac{5}{16}''$ is the usual thickness for bridge floors.

Buckle plates are made in long lengths having several buckles or domes in each plate. They are usually supported along the two longitudinal edges and at the extreme ends, and should be bolted or riveted to the supports, with $\frac{5}{8}''$ or $\frac{3}{4}''$ bolts or rivets spaced not over 6'' centers. If the ends of the buckle plates do not rest on supports, they should be spliced with **T** iron or a pair of angles riveted together.

The approximate total safe uniformly distributed loads are given in the following table for different thicknesses and sizes of buckle plates, well bolted down, calculated from the formula,

$$W = 4\ Sdt$$

where W = total safe uniform load, in lbs., on a single square.
S = allowable unit strain, in lbs., per square inch.
d = depth of buckle, inches.
t = thickness of plate, inches.

TOTAL SAFE UNIFORMLY DISTRIBUTED LOADS, IN LBS., ON BUCKLE PLATES.

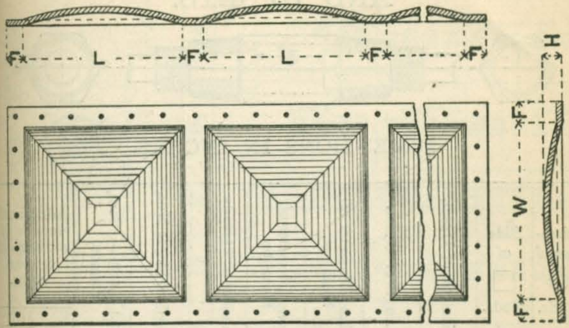
Size of Plate.	30'' Square.	36'' Square.	42'' Square.	48'' Square.	54'' Square.	60'' Square.
Thickness, in Inches.	2 Inches, Depth of Buckle.					
$\frac{1}{4}$	11,000	9,100	7,300	6,000	5,000	4,200
$\frac{5}{16}$	16,400	13,800	11,800	10,000	8,600	7,300
$\frac{3}{8}$	22,200	19,400	17,000	14,700	12,700	11,200
	$2\frac{1}{2}$ Inches, Depth of Buckle.					
$\frac{1}{4}$	13,800	11,300	9,100	7,500	6,300	5,300
$\frac{5}{16}$	20,500	17,300	14,800	12,500	10,700	9,200
$\frac{3}{8}$	27,600	24,300	21,300	18,400	15,900	13,900
	3 Inches, Depth of Buckle.					
$\frac{1}{4}$	16,600	13,600	10,900	9,000	7,500	6,300
$\frac{5}{16}$	24,600	20,700	17,700	15,000	12,900	11,000
$\frac{3}{8}$	33,200	29,000	25,400	22,100	19,100	16,700

Note

If the buckles are inverted, *i. e.*, suspended, the safe loads will be increased from 2 to 4 times that given in the above table, depending upon the size of the plate.

Buckle plates are preferably made of soft steel.

PASSAIC BUCKLE PLATES.

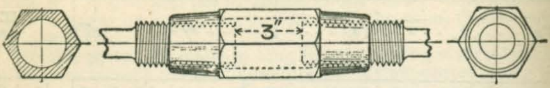


DIMENSIONS OF BUCKLE PLATES.

No. of Plate.	Buckle. L.	Buckle. W.	Depth of Buckle. H.	Number of Buckles in One Plate.	Fillets. F.
1	2′—2½″	2′—3½″	2½″	1 to 8	Maximum, 6″
2	2′—5″	3′—2″	2½″	1 to 6	
3	2′—7″	2′—7″	3″	1 to 6	
4	2′—7″	2′—7″	2″	1 to 6	
5	3′—2″	3′—4″	3″	1 to 6	
6	3′—4″	3′—9″	2½″	1 to 6	Minimum, 2½″

Buckles of other dimensions than those given in table may be made by special arrangement.

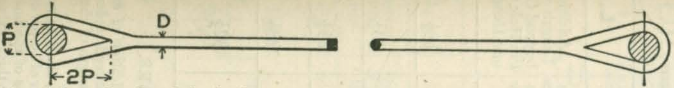
STANDARD SLEEVE NUTS AND UPSETS.



DIMENSIONS IN INCHES.

Diameter of ○ Rods.	Side of □ Rods.	Diameter of Upset.	Length of Upset.	Short Diameter of Hexagon.	Long Diameter of Hexagon.	Number of Threads per inch.	Length of Sleeve Nut.	Weight of Sleeve Nut, Pounds.	Additional length of rod req'red for one upset. ○	□
3/4	5/8	1	4	2 1/4	2 5/8	8	8 1/4	4	3 3/4	4 3/4
7/8	3/4	1 1/8	4	2 1/4	2 5/8	7	8 1/2	5	3 1/4	3 3/4
1	7/8	1 3/8	4 1/2	2 3/8	2 3/4	6	9 1/4	7	4 3/4	5
1 1/8	1	1 1/2	4 1/2	2 7/8	3 5/16	6	9 1/4	8	4 1/4	4 1/4
1 1/4	1 1/8	1 5/8	4 1/2	2 7/8	3 5/16	5 1/2	9 1/2	9	3 3/4	3 1/2
1 3/8	1 1/4	1 7/8	5	3 1/4	3 3/4	5	10 1/4	13	5 1/4	4 1/2
1 1/2	1 3/8	2	5	3 1/4	3 3/4	4 1/2	10 1/4	13	4 3/4	4
1 5/8	1 1/2	2 1/8	5	3 5/8	4 3/16	4 1/2	10 1/2	16	4 1/4	3 1/2
1 3/4		2 1/4	5 1/2	3 3/4	4 5/16	4 1/2	11	18	4 1/4	
1 7/8	1 5/8	2 3/8	5 1/2	4	4 5/8	4	11 1/4	21	4	4 1/2
2	1 3/4	2 1/2	5 1/2	4	4 5/8	4	11 1/4	22	3 3/4	4
2 1/8	1 7/8	2 5/8	6	4 5/8	5 3/8	4	12	29	3 3/4	4
2 1/4	2	2 7/8	6	4 3/4	5 1/2	3 1/2	12 1/4	33	4 1/2	4 1/2
2 1/2	2 1/4	3 1/4	6	5 1/8	5 15/16	3 1/2	12 1/2	40	5	4 3/4
2 3/4	2 1/2	3 1/2	6	5 1/2	6 3/8	3 1/4	12 3/4	47	4 1/2	4
3		3 3/4	6	5 7/8	6 3/4	3	13	58	4	

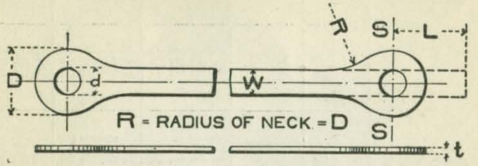
SQUARE OR ROUND IRON RODS WITH LOOP EYES.



Additional length of rod, in inches, required beyond center of pin, to make one eye.

Diameter of Pin, inches.	Diameter or side of Rod, in inches.													
	¾	⅞	1	1⅛	1¼	1⅜	1½	1⅝	1¾	1⅞	2	2⅛	2¼	2½
$1\frac{7}{16}$	9¼	9¾	10¼	10¾	11¼	11¾	12							
$1\frac{11}{16}$	10¼	10¾	11¼	11¾	12	12½	13	13½	14					
$1\frac{15}{16}$	11¼	11¾	12	12½	13	13½	14	14½	15	15½	15¾			
$2\frac{3}{16}$	12	12½	13	13½	14	14½	15	15½	15¾	16¼	16¾	17¼	17¾	
$2\frac{7}{16}$	13	13½	14	14½	15	15½	15¾	16¼	16¾	17¼	17¾	18¼	18½	19½
$2\frac{11}{16}$	14	14½	15	15½	15¾	16¼	16¾	17¼	17¾	18¼	18½	19	19½	20½
$2\frac{15}{16}$	15	15½	15¾	16¼	16¾	17¼	17¾	18¼	18½	19	19½	20	20½	21½
$3\frac{3}{16}$	15¾	16¼	16¾	17¼	17¾	18¼	18½	19	19½	20	20½	21	21½	22¼
$3\frac{7}{16}$	16¾	17¼	17¾	18¼	18½	19	19½	20	20½	21	21½	21¾	22¼	23¼
$3\frac{11}{16}$	17¾	18¼	18½	19	19½	20	20½	21	21½	21¾	22¼	22¾	23¼	24¼
$3\frac{15}{16}$	18½	19	19½	20	20½	21	21½	21¾	22¼	22¾	23¼	23¾	24¼	25
4⅜	20	20½	21	21½	21¾	22¼	22¾	23¼	23¾	24¼	24½	25	25½	26½
4⅝	21	21½	21¾	22¼	22¾	23¼	23¾	24¼	24½	25	25½	26	26½	27½
4⅞	21¾	22¼	22¾	23¼	23¾	24¼	24½	25	25½	26	26½	27	27½	28¼
5⅜	23¾	24¼	24½	25	25½	26	26½	27	27½	27¾	28¼	28¾	29¼	30
5⅞	25½	26	26½	27	27½	27¾	28¼	28¾	29¼	29¾	30	30½	31	32

STANDARD STEEL EYE BARS.



W.	t.	D.	d.	S—S.	L.
Width of Bar, Inches.	Minimum Thickness of Bar, Inches.	Diameter of Head, Inches.	Diameter of Largest Pin Hole, Inches.	Sectional Area of Head on Lines S — S in excess of that in Body of Bar.	Additional Length of Bar beyond Cen. of Pin Hole to form one Head, Ins.
3	$\frac{3}{4}$	7	$2\frac{11}{16}$	42%	$14\frac{1}{2}$
3	$\frac{3}{4}$	8	$3\frac{11}{16}$	42	$18\frac{1}{2}$
4	$\frac{3}{4}$	$9\frac{1}{2}$	$3\frac{15}{16}$	$37\frac{1}{2}$	$18\frac{1}{2}$
4	$\frac{3}{4}$	$10\frac{1}{2}$	$4\frac{7}{8}$	39	$23\frac{1}{2}$
5	$\frac{3}{4}$	$11\frac{1}{2}$	$4\frac{3}{8}$	41	21
5	$\frac{3}{4}$	$12\frac{1}{2}$	$5\frac{3}{8}$	41	$25\frac{1}{2}$
6	$\frac{7}{8}$	$13\frac{1}{2}$	$4\frac{7}{8}$	42	22
6	$\frac{7}{8}$	$14\frac{1}{2}$	$5\frac{7}{8}$	42	$26\frac{1}{2}$
7	1	16	$5\frac{7}{8}$	43	28
8	$1\frac{1}{8}$	18	7	$37\frac{1}{2}$	$32\frac{1}{2}$
10	$1\frac{1}{4}$	23	9	40	40

NOTES ON PASSAIC STEEL EYE BARS.

Passaic standard steel eye bars are forged without the addition of extraneous metal and without welds of any kind, and are guaranteed under the conditions given in the above table to develop the full strength of the bar when tested to destruction.

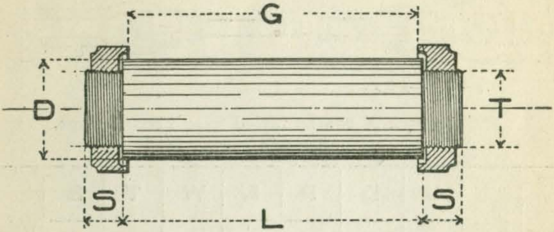
The maximum sizes of pin holes, given in the above table, allow an excess in the net section of the head over that of the body of the bar of 40 per cent., when the thickness of the head is the same as the thickness of the body of the bar. The thickness of the head is usually 1-16 of an inch thicker than the body of the bar; and where a number of eye bars are to be placed closely together, as at a joint, the thicknesses of the heads should be considered 1-8 of an inch greater than the bodies of the bars in order to allow for the increased thickness of the heads and for the usual roughness of forged work.

Unless otherwise specified, the steel manufactured by us for the use of eye bars is open hearth medium steel conforming with the standard specifications of the Association of American Steel Manufacturers.

All eye bars are finished to length, and the eyes bored at the specified distances, center to center, according to U. S. standard measurements.

Eye bars having larger or smaller heads than the above standards can be furnished by special arrangement.

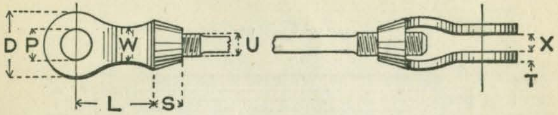
STANDARD PINS AND NUTS.



G=GRIP. $L=G+\frac{3}{8}''$.

D. Diameter of Pin, Inches.	T. Diameter of Thread, Inches.	S. Length of Thread, Inches.	Short Dia. of Nut, Inches.	Long Dia. of Nut, Inches.	Weight of One Nut, Lbs.
$1\frac{3}{16}$	1	$1\frac{1}{2}$	$1\frac{3}{4}$	2	
$1\frac{7}{16}$	1	″	$1\frac{3}{4}$	2	
$1\frac{11}{16}$	$1\frac{1}{2}$	″	$3\frac{1}{4}$	$3\frac{3}{4}$	1.5
$1\frac{15}{16}$	$1\frac{1}{2}$	″	$3\frac{1}{4}$	$3\frac{3}{4}$	1.5
$2\frac{3}{16}$	$1\frac{1}{2}$	$1\frac{1}{2}$	$3\frac{1}{4}$	$3\frac{3}{4}$	1.5
$2\frac{7}{16}$	$1\frac{3}{4}$	″	$3\frac{1}{4}$	$3\frac{3}{4}$	1.5
$2\frac{11}{16}$	2	″	$3\frac{3}{4}$	$4\frac{1}{4}$	2.5
$2\frac{15}{16}$	$2\frac{1}{4}$	″	$4\frac{1}{2}$	$5\frac{1}{4}$	3.0
$3\frac{3}{16}$	$2\frac{1}{2}$	$1\frac{1}{2}$	$4\frac{1}{2}$	$5\frac{1}{4}$	2.8
$3\frac{7}{16}$	$2\frac{1}{2}$	″	$4\frac{1}{2}$	$5\frac{1}{4}$	2.8
$3\frac{11}{16}$	$2\frac{3}{4}$	″	$4\frac{3}{4}$	$5\frac{1}{2}$	3.0
$3\frac{15}{16}$	3	″	$4\frac{3}{4}$	$5\frac{1}{2}$	3.0
$4\frac{3}{8}$	$3\frac{1}{2}$	$1\frac{1}{2}$	$5\frac{1}{2}$	$6\frac{1}{4}$	3.8
$4\frac{5}{8}$	$3\frac{1}{2}$	″	$5\frac{1}{2}$	$6\frac{1}{4}$	3.8
$4\frac{7}{8}$	4	″	6	7	6.7
$5\frac{3}{8}$	4	2	6	7	6.7
$5\frac{7}{8}$	4	2	7	8	9.1
7	5	$2\frac{1}{4}$	8	$9\frac{1}{4}$	12.0
8	6	$2\frac{1}{4}$	$10\frac{1}{2}$	12	22.8
9	7	$2\frac{1}{4}$	$10\frac{1}{2}$	12	18.8

PASSAIC STANDARD CLEVISES.



The distance **X** can be varied to suit connections.

Number of Clevis.	Side of Square Bar, inches.	U Upset for Square Bar.	D Diameter of Eye, inches.	P Diameter of Pin, inches.	L Length of Fork, inches.	W Width of Fork, inches.	T Thickness of Fork, inches.	S Length of Thread inches.	Weight of one Clevis, lbs.
1	1	$1\frac{1}{2}$	$4\frac{1}{2}$	$2\frac{3}{16}$	$6\frac{1}{2}$	2	$\frac{5}{8}$	$2\frac{1}{4}$	12
	$1\frac{1}{8}$	$1\frac{5}{8}$							
	$1\frac{1}{4}$	$1\frac{7}{8}$							
2	$1\frac{3}{8}$	2	$5\frac{1}{2}$	$2\frac{11}{16}$	7	$2\frac{1}{2}$	$\frac{3}{4}$	$2\frac{1}{2}$	20
	$1\frac{1}{2}$	$2\frac{1}{8}$							
3	$1\frac{5}{8}$	$2\frac{3}{8}$	$6\frac{1}{2}$	$2\frac{15}{16}$	8	3	$\frac{7}{8}$	3	28
	$1\frac{3}{4}$	$2\frac{1}{2}$							
	$1\frac{7}{8}$	$2\frac{3}{4}$							
4	2	$2\frac{7}{8}$	8	$3\frac{7}{16}$	9	$3\frac{1}{2}$	1	$3\frac{1}{2}$	45
	$2\frac{1}{4}$	$3\frac{1}{4}$							

Passaic clevises are proportioned to develop the full strength of iron or steel bars of the sizes given.

The size of pin given is the maximum for each size of clevis when the largest bar is used.

LINEAL EXPANSION OF SUBSTANCES BY HEAT.

To find the increase in the length of a bar of any material due to an increase of temperature, multiply the number of degrees of increase of temperature by the coefficient for 100° and by the length of the bar, and divide by one hundred.

NAME OF SUBSTANCE.	Coefficient for 100° Fahrenheit.	Coefficient for 180° Fahrenheit, or 100° Centigrade.
Aluminum (cast)	.001234	.00222
Brass (cast)	.000957	.00172
Brick	.000306	.00055
Bronze	.000986	.00177
Cement, Portland	.000594	.00107
Concrete	.000795	.00143
Copper	.000887	.00160
Glass, flint	.000451	.00081
Granite	.000438	.00079
Gold, pure	.000786	.00142
Iron, wrought	.000648	.00117
" cast	.000556	.00100
Lead	.001571	.00283
Marble { from	.000308	.00055
Marble { to	.000786	.00142
Masonry, brick { from	.000256	.00046
Masonry, brick { to	.000494	.00089
Mercury (cubic expansion)	.009984	.01797
Sandstone	.000652	.00117
Silver, pure	.001079	.00194
Slate	.000577	.00104
Steel, cast	.000636	.00114
" structural	.000663	.00119
" tempered	.000689	.00124
Tin	.001163	.00210
Wood, pine	.000276	.00050
Zinc	.001407	.00253

AREAS AND WEIGHTS OF SQUARE AND ROUND STEEL BARS.

Thickness, Inches.	□ Area.	□ Weight per ft.	○ Area.	○ Weight per ft.	Thickness, Inches.	□ Area.	□ Weight per ft.	○ Area.	○ Weight per ft.
0					2	4.000	13.60	3.142	10.68
1/16	0.004	0.013	0.003	0.010	1/16	4.254	14.46	3.341	11.36
1/8	.016	.053	.012	.042	1/8	4.516	15.35	3.547	12.06
3/16	.035	.119	.028	.094	3/16	4.785	16.27	3.758	12.78
1/4	.062	.212	.049	.167	1/4	5.063	17.22	3.976	13.52
5/16	.098	.333	.077	.261	5/16	5.348	18.19	4.200	14.28
3/8	.141	.478	.110	.375	3/8	5.641	19.18	4.430	15.07
7/16	.191	.651	.150	.511	7/16	5.941	20.20	4.666	15.86
1/2	.250	.850	.196	.667	1/2	6.250	21.25	4.909	16.69
9/16	.316	1.076	.248	.845	9/16	6.566	22.33	5.157	17.53
5/8	.391	1.328	.307	1.043	5/8	6.891	23.43	5.412	18.40
11/16	.473	1.608	.371	1.262	11/16	7.223	24.56	5.673	19.29
3/4	.562	1.913	.442	1.502	3/4	7.563	25.71	5.940	20.20
13/16	.660	2.245	.518	1.763	13/16	7.910	26.90	6.213	21.12
7/8	.766	2.603	.601	2.044	7/8	8.266	28.10	6.492	22.07
15/16	.879	2.989	.690	2.347	15/16	8.629	29.34	6.777	23.04
1	1.000	3.400	.785	2.670	3	9.000	30.60	7.069	24.03
1/16	1.129	3.838	.887	3.014	1/16	9.379	31.89	7.366	25.04
1/8	1.266	4.303	.994	3.379	1/8	9.766	33.20	7.670	26.08
3/16	1.410	4.795	1.108	3.766	3/16	10.16	34.55	7.980	27.13
1/4	1.563	5.312	1.227	4.173	1/4	10.56	35.92	8.296	28.20
5/16	1.723	5.857	1.353	4.600	5/16	10.97	37.31	8.618	29.30
3/8	1.891	6.428	1.485	5.049	3/8	11.39	38.73	8.946	30.42
7/16	2.066	7.026	1.623	5.518	7/16	11.82	40.18	9.281	31.56
1/2	2.250	7.650	1.767	6.008	1/2	12.25	41.65	9.621	32.71
9/16	2.441	8.301	1.918	6.520	9/16	12.69	43.14	9.968	33.90
5/8	2.641	8.978	2.074	7.051	5/8	13.14	44.68	10.32	35.09
11/16	2.848	9.682	2.237	7.604	11/16	13.60	46.24	10.68	36.31
3/4	3.063	10.41	2.405	8.178	3/4	14.06	47.82	11.05	37.56
13/16	3.285	11.17	2.580	8.773	13/16	14.54	49.42	11.42	38.81
7/8	3.516	11.95	2.761	9.388	7/8	15.02	51.05	11.79	40.10
15/16	3.754	12.76	2.948	10.02	15/16	15.50	52.71	12.18	41.40

AREAS AND WEIGHTS OF SQUARE AND ROUND STEEL BARS

(Continued).

Thickness, Inches.	□ Area.	□ Weight per ft.	○ Area.	○ Weight per ft.	Thickness, Inches.	□ Area.	□ Weight per ft.	○ Area.	○ Weight per ft.
4	16.00	54.40	12.57	42.73	6	36.00	122.4	28.27	96.14
1/16	16.50	56.11	12.96	44.07	1/8	37.52	127.6	29.47	100.2
1/8	17.02	57.85	13.36	45.44	1/4	39.06	132.8	30.68	104.3
3/16	17.54	59.62	13.77	46.83	3/8	40.64	138.2	31.92	108.5
1/4	18.06	61.41	14.19	48.24	1/2	42.25	143.6	33.18	112.8
5/16	18.60	63.23	14.61	49.66	5/8	43.89	149.2	34.47	117.2
3/8	19.14	65.08	15.03	51.11	3/4	45.56	154.9	35.79	121.7
7/16	19.69	66.95	15.47	52.58	7/8	47.27	160.8	37.12	126.2
1/2	20.25	68.85	15.90	54.07	7	49.00	166.6	38.49	130.9
9/16	20.82	70.78	16.35	55.59	1/4	52.56	178.7	41.28	140.4
5/8	21.39	72.73	16.80	57.12	1/2	56.25	191.3	44.18	150.2
11/16	21.97	74.70	17.26	58.67	3/4	60.06	204.2	47.17	160.3
3/4	22.56	76.71	17.72	60.25	8	64.00	217.6	50.27	171.0
13/16	23.16	78.74	18.19	61.84	1/4	68.06	231.4	53.46	181.8
7/8	23.77	80.81	18.67	63.46	1/2	72.25	245.6	56.75	193.0
15/16	24.38	82.89	19.15	65.10	3/4	76.56	260.3	60.13	204.4
5	25.00	85.00	19.64	66.76	9	81.00	275.4	63.62	216.3
1/16	25.63	87.14	20.13	68.44	1/4	85.56	290.9	67.20	228.5
1/8	26.27	89.30	20.63	70.14	1/2	90.25	306.8	70.88	241.0
3/16	26.91	91.49	21.14	71.86	3/4	95.06	323.2	74.66	253.9
1/4	27.56	93.72	21.65	73.60	10	100.0	340.0	78.54	267.0
5/16	28.22	95.96	22.17	75.37	1/4	105.1	357.2	82.52	280.6
3/8	28.89	98.23	22.69	77.15	1/2	110.3	374.9	86.59	294.4
7/16	29.57	100.5	23.22	78.95	3/4	115.6	392.9	90.76	308.6
1/2	30.25	102.8	23.76	80.77	11	121.0	411.4	95.03	323.1
9/16	30.94	105.2	24.30	82.62	1/4	126.6	430.3	99.40	337.9
5/8	31.64	107.6	24.85	84.49	1/2	132.3	449.6	103.9	353.1
11/16	32.35	110.0	25.41	86.38	3/4	138.1	469.4	108.4	368.6
3/4	33.06	112.4	25.97	88.29	12	144.0	489.6	113.1	384.5
13/16	33.79	114.9	26.54	90.22					
7/8	34.52	117.4	27.11	92.17					
15/16	35.25	119.9	27.69	94.14					

WEIGHTS OF STEEL FLATS,

PER LINEAL FOOT.

Thickness, in Inches.	1″	$1\frac{1}{4}$″	$1\frac{1}{2}$″	$1\frac{3}{4}$″	2″	$2\frac{1}{4}$″	$2\frac{1}{2}$″	$2\frac{3}{4}$″	3″
$\frac{1}{16}$	.21	.26	.32	.37	.43	.48	.53	.58	.63
$\frac{1}{8}$	.42	.53	.64	.75	.85	.96	1.06	1.17	1.28
$\frac{3}{16}$	.63	.79	.96	1.11	1.28	1.44	1.59	1.75	1.91
$\frac{1}{4}$	.85	1.06	1.28	1.49	1.70	1.91	2.12	2.34	2.55
$\frac{5}{16}$	1.06	1.33	1.59	1.86	2.12	2.39	2.65	2.92	3.19
$\frac{3}{8}$	1.28	1.59	1.92	2.23	2.55	2.87	3.19	3.51	3.83
$\frac{7}{16}$	1.49	1.86	2.23	2.60	2.98	3.35	3.72	4.09	4.46
$\frac{1}{2}$	1.70	2.12	2.55	2.98	3.40	3.83	4.25	4.67	5.10
$\frac{9}{16}$	1.92	2.39	2.87	3.35	3.83	4.30	4.78	5.26	5.74
$\frac{5}{8}$	2.12	2.65	3.19	3.72	4.25	4.78	5.31	5.84	6.38
$\frac{11}{16}$	2.34	2.92	3.51	4.09	4.67	5.26	5.84	6.43	7.02
$\frac{3}{4}$	2.55	3.19	3.83	4.47	5.10	5.75	6.38	7.02	7.65
$\frac{13}{16}$	2.76	3.45	4.14	4.84	5.53	6.21	6.90	7.60	8.29
$\frac{7}{8}$	2.98	3.72	4.47	5.20	5.95	6.69	7.44	8.18	8.93
$\frac{15}{16}$	3.19	3.99	4.78	5.58	6.38	7.18	7.97	8.77	9.57
1	3.40	4.25	5.10	5.95	6.80	7.65	8.50	9.35	10.20
$1\frac{1}{16}$	3.61	4.52	5.42	6.32	7.22	8.13	9.03	9.93	10.84
$1\frac{1}{8}$	3.83	4.78	5.74	6.70	7.65	8.61	9.57	10.52	11.48
$1\frac{3}{16}$	4.04	5.05	6.06	7.07	8.08	9.09	10.10	11.11	12.12
$1\frac{1}{4}$	4.25	5.31	6.38	7.44	8.50	9.57	10.63	11.69	12.75
$1\frac{5}{16}$	4.46	5.58	6.69	7.81	8.93	10.04	11.16	12.27	13.39
$1\frac{3}{8}$	4.67	5.84	7.02	8.18	9.35	10.52	11.69	12.85	14.03
$1\frac{7}{16}$	4.89	6.11	7.34	8.56	9.78	11.00	12.22	13.44	14.66
$1\frac{1}{2}$	5.10	6.38	7.65	8.93	10.20	11.48	12.75	14.03	15.30
$1\frac{9}{16}$	5.32	6.64	7.97	9.30	10.63	11.95	13.28	14.61	15.94
$1\frac{5}{8}$	5.52	6.90	8.29	9.67	11.05	12.43	13.81	15.19	16.58
$1\frac{11}{16}$	5.74	7.17	8.61	10.04	11.47	12.91	14.34	15.78	17.22
$1\frac{3}{4}$	5.95	7.44	8.93	10.42	11.90	13.40	14.88	16.37	17.85
$1\frac{13}{16}$	6.16	7.70	9.24	10.79	12.33	13.86	15.40	16.95	18.49
$1\frac{7}{8}$	6.38	7.97	9.57	11.15	12.75	14.34	15.94	17.53	19.13
$1\frac{15}{16}$	6.59	8.24	9.88	11.53	13.18	14.83	16.47	18.12	19.77
2	6.80	8.50	10.20	11.90	13.60	15.30	17.00	18.70	20.40

WEIGHTS OF STEEL FLATS,

PER LINEAL FOOT

(Continued).

Thickness, in Inches.	$3\frac{1}{2}''$	$4''$	$4\frac{1}{2}''$	$5''$	$6''$	$7''$	$8''$	$9''$	$10''$
$\frac{1}{16}$	.75	.85	.96	1.06	1.28	1.49	1.70	1.91	2.13
$\frac{1}{8}$	1.49	1.70	1.92	2.13	2.55	2.98	3.40	3.82	4.25
$\frac{3}{16}$	2.23	2.55	2.87	3.19	3.83	4.46	5.10	5.74	6.38
$\frac{1}{4}$	2.98	3.40	3.83	4.25	5.10	5.95	6.80	7.65	8.50
$\frac{5}{16}$	3.72	4.25	4.78	5.31	6.38	7.44	8.50	9.56	10.62
$\frac{3}{8}$	4.47	5.10	5.74	6.38	7.65	8.93	10.20	11.48	12.75
$\frac{7}{16}$	5.20	5.95	6.70	7.44	8.93	10.41	11.90	13.40	14.88
$\frac{1}{2}$	5.95	6.80	7.65	8.50	10.20	11.90	13.60	15.30	17.00
$\frac{9}{16}$	6.70	7.65	8.61	9.57	11.48	13.39	15.30	17.22	19.14
$\frac{5}{8}$	7.44	8.50	9.57	10.63	12.75	14.87	17.00	19.13	21.25
$\frac{11}{16}$	8.18	9.35	10.52	11.69	14.03	16.36	18.70	21.04	23.38
$\frac{3}{4}$	8.93	10.20	11.48	12.75	15.30	17.85	20.40	22.96	25.50
$\frac{13}{16}$	9.67	11.05	12.43	13.81	16.58	19.34	22.10	24.86	27.62
$\frac{7}{8}$	10.41	11.90	13.39	14.87	17.85	20.83	23.80	26.78	29.75
$\frac{15}{16}$	11.16	12.75	14.34	15.94	19.13	22.32	25.50	28.69	31.88
1	11.90	13.60	15.30	17.00	20.40	23.80	27.20	30.60	34.00
$1\frac{1}{16}$	12.65	14.45	16.26	18.06	21.68	25.29	28.90	32.52	36.12
$1\frac{1}{8}$	13.39	15.30	17.22	19.13	22.95	26.78	30.60	34.43	38.25
$1\frac{3}{16}$	14.13	16.15	18.17	20.19	24.23	28.26	32.30	36.34	40.38
$1\frac{1}{4}$	14.87	17.00	19.13	21.25	25.50	29.75	34.00	38.26	42.50
$1\frac{5}{16}$	15.62	17.85	20.08	22.32	26.78	31.23	35.70	40.16	44.64
$1\frac{3}{8}$	16.36	18.70	21.04	23.38	28.05	32.72	37.40	42.08	46.75
$1\frac{7}{16}$	17.10	19.85	21.99	24.44	29.33	34.21	39.10	44.00	48.88
$1\frac{1}{2}$	17.85	20.40	22.95	25.50	30.60	35.70	40.80	45.90	51.00
$1\frac{9}{16}$	18.60	21.25	23.91	26.57	31.88	37.19	42.50	47.82	53.14
$1\frac{5}{8}$	19.34	22.10	24.87	27.63	33.15	38.67	44.20	49.73	55.25
$1\frac{11}{16}$	20.08	22.95	25.82	28.69	34.43	40.16	45.90	51.64	57.38
$1\frac{3}{4}$	20.83	23.80	26.78	29.75	35.70	41.65	47.60	53.56	59.50
$1\frac{13}{16}$	21.57	24.65	27.73	30.81	36.98	43.14	49.30	55.46	61.62
$1\frac{7}{8}$	22.31	25.50	28.69	31.87	38.25	44.63	51.00	57.38	63.75
$1\frac{15}{16}$	23.06	26.35	29.64	32.94	39.53	46.12	52.70	59.29	65.88
2	23.80	27.20	30.60	34.00	40.80	47.60	54.40	61.20	68.00

WEIGHTS OF STEEL PLATES PER LINEAL FOOT.

Thickness, in Inches.	12″	13″	14″	15″	16″	17″	18″	19″	20″	21″	22″	23″	24″	25″	¼″	½″	¾″
1/16	2.55	2.76	2.97	3.19	3.40	3.61	3.83	4.04	4.25	4.46	4.68	4.89	5.10	5.32	.05	.11	.16
1/8	5.10	5.53	5.95	6.38	6.80	7.22	7.65	8.08	8.50	8.92	9.35	9.78	10.20	10.63	.11	.22	.32
3/16	7.66	8.28	8.92	9.56	10.20	10.84	11.48	12.10	12.76	13.40	14.04	14.64	15.32	15.96	.16	.33	.48
1/4	10.20	11.06	11.90	12.75	13.60	14.44	15.30	16.16	17.00	17.84	18.69	19.56	20.40	21.26	.21	.43	.64
5/16	12.76	13.81	14.88	15.94	17.00	18.06	19.12	20.20	21.24	22.32	23.36	24.44	25.52	26.56	.27	.53	.80
3/8	15.30	16.58	17.86	19.14	20.40	21.68	22.96	24.24	25.50	26.78	28.06	29.36	30.60	31.88	.32	.64	.96
7/16	17.86	19.34	20.82	22.32	23.80	25.28	26.79	28.28	29.75	31.24	32.72	34.24	35.72	37.20	.37	.76	1.12
1/2	20.40	22.10	23.80	25.50	27.20	28.89	30.60	32.31	34.00	35.70	37.40	39.10	40.80	42.50	.43	.85	1.28
9/16	22.96	24.86	26.78	28.70	30.60	32.52	34.44	36.34	38.27	40.16	42.04	44.00	45.92	47.80	.48	.96	1.44
5/8	25.50	27.62	29.74	31.88	34.00	36.12	38.25	40.37	42.50	44.64	46.76	48.88	51.00	53.12	.53	1.06	1.60
11/16	28.06	30.39	32.72	35.06	37.40	39.72	42.08	44.42	46.74	49.08	51.40	53.76	56.12	58.44	.59	1.17	1.76
3/4	30.60	33.16	35.71	38.26	40.80	43.36	45.92	48.46	51.00	53.56	56.10	58.66	61.20	63.76	.64	1.28	1.92
13/16	33.15	35.91	38.67	41.43	44.20	46.96	49.72	52.48	55.25	58.01	60.79	63.53	66.29	69.06	.69	1.38	2.07
7/8	35.70	38.68	41.65	44.62	47.60	50.60	53.56	56.52	59.50	62.49	65.44	68.43	71.40	74.38	.75	1.49	2.23
15/16	38.25	41.44	44.63	47.82	51.00	54.20	57.38	60.57	63.76	66.96	70.13	73.32	76.50	79.68	.80	1.60	2.39
1	40.80	44.20	47.60	51.00	54.40	57.80	61.20	64.60	68.00	71.40	74.80	78.20	81.60	85.00	.85	1.70	2.55

WEIGHTS OF STEEL PLATES PER LINEAL FOOT *(Continued).*

Thickness, in Ins.	26″	27″	28″	29″	30″	32″	34″	36″	38″	40″	42″	44″	46″	1″	$1\frac{1}{4}$″	$1\frac{1}{2}$″	$1\frac{3}{4}$″
$\frac{1}{16}$	5.52	5.74	5.95	6.16	6.37	6.80	7.23	7.65	8.08	8.50	8.93	9.39	9.77	.21	.27	.32	.37
$\frac{1}{8}$	11.06	11.48	11.90	12.32	12.75	13.60	14.44	15.29	16.16	17.00	17.85	18.69	19.56	.43	.53	.64	.75
$\frac{3}{16}$	16.56	17.20	17.84	18.48	19.12	20.40	21.68	22.96	24.20	25.52	26.80	28.08	29.29	.64	.80	.96	1.11
$\frac{1}{4}$	22.12	22.96	23.80	24.64	25.50	27.20	28.88	30.59	32.32	34.00	35.68	37.38	39.11	.85	1.06	1.28	1.49
$\frac{5}{16}$	27.62	28.68	29.76	30.80	31.88	34.00	36.12	38.24	40.39	42.48	44.64	46.72	48.88	1.06	1.33	1.59	1.86
$\frac{3}{8}$	33.16	34.44	35.72	37.00	38.28	40.80	43.36	45.92	48.48	51.00	53.56	56.12	58.71	1.28	1.59	1.92	2.23
$\frac{7}{16}$	38.68	40.17	41.65	43.14	44.64	47.60	50.57	53.58	56.56	59.50	62.48	65.44	68.47	1.49	1.86	2.23	2.60
$\frac{1}{2}$	44.20	45.92	47.60	49.28	51.00	54.40	57.78	61.20	64.62	68.00	71.40	74.80	78.20	1.70	2.12	2.55	2.98
$\frac{9}{16}$	49.73	51.64	53.56	55.48	57.40	61.22	65.04	68.88	72.68	76.54	80.32	84.09	88.00	1.92	2.39	2.87	3.35
$\frac{5}{8}$	55.24	57.37	59.49	61.60	63.76	68.00	72.24	76.50	80.74	85.00	89.28	93.52	97.76	2.12	2.65	3.19	3.72
$\frac{11}{16}$	60.78	63.11	65.44	67.77	70.13	74.80	79.44	84.15	88.84	93.48	98.16	102.81	107.53	2.34	2.92	3.51	4.09
$\frac{3}{4}$	66.32	68.88	71.42	73.97	76.53	81.61	86.72	91.84	96.92	102.00	107.12	112.20	117.31	2.55	3.19	3.83	4.47
$\frac{13}{16}$	71.82	74.58	77.34	80.10	82.86	88.39	93.91	99.44	104.96	110.50	116.02	121.56	127.06	2.76	3.45	4.14	4.84
$\frac{7}{8}$	77.36	80.33	83.30	86.29	89.24	95.20	101.20	107.12	113.04	119.00	124.98	130.89	136.86	2.98	3.72	4.47	5.20
$\frac{15}{16}$	82.88	86.07	89.26	92.44	95.64	102.00	108.40	114.76	121.14	127.52	133.92	140.27	146.64	3.19	3.99	4.78	5.58
1	88.40	91.80	95.20	98.60	102.00	108.80	115.60	122.40	129.20	136.00	142.80	149.60	156.40	3.40	4.25	5.10	5.95

WEIGHTS OF STEEL PLATES PER LINEAL FOOT *(Continued).*

Thickness, in Ins.	48″	50″	52″	54″	56″	58″	60″	62″	64″	66″	68″	70″	72″	74″	¼″	½″	¾″
1/16	10.20	10.63	11.05	11.47	11.90	12.32	12.75	13.18	13.60	14.02	14.44	14.86	15.29	15.73	.05	.11	.16
1/8	20.40	21.26	22.12	22.96	23.80	24.64	25.50	26.36	27.20	28.04	28.88	29.72	30.59	31.46	.11	.22	.32
3/16	30.64	31.92	33.12	34.40	35.68	36.96	38.24	39.50	40.80	42.08	43.36	44.64	45.92	47.20	.16	.33	.48
1/4	40.80	42.52	44.24	45.92	47.60	49.28	51.00	52.72	54.40	56.08	57.76	59.44	61.18	62.92	.21	.43	.64
5/16	51.04	53.12	55.24	57.36	59.51	61.60	63.76	65.88	68.00	70.08	72.24	74.32	76.48	78.62	.27	.53	.80
3/8	61.20	63.76	66.32	68.88	71.44	74.00	76.56	79.08	81.60	84.16	86.72	89.28	91.84	94.40	.32	.64	.96
7/16	71.44	74.40	77.37	80.34	83.30	86.28	89.28	92.24	95.20	98.16	101.1	104.1	107.2	110.1	.37	.74	1.12
1/2	81.60	85.03	88.40	91.84	95.20	98.56	102.0	105.4	108.8	112.2	115.6	119.0	122.4	125.8	.43	.85	1.28
9/16	91.84	95.62	99.46	103.3	107.1	111.0	114.8	118.6	122.4	126.2	130.1	133.9	137.8	141.5	.48	.96	1.44
5/8	102.0	106.2	110.5	114.7	119.0	123.2	127.5	131.8	136.0	140.2	144.5	148.7	153.0	157.2	.53	1.06	1.60
11/16	112.2	116.9	121.6	126.2	130.9	135.5	140.3	145.0	149.6	154.2	158.9	163.6	168.3	173.0	.59	1.17	1.76
3/4	122.4	127.5	132.6	137.8	142.8	147.9	153.1	158.2	163.2	168.3	173.4	178.6	183.7	188.7	.64	1.28	1.92
13/16	132.6	138.1	143.6	149.2	154.7	160.2	165.7	171.2	176.8	182.3	187.8	193.4	198.9	204.4	.69	1.38	2.07
7/8	142.8	148.8	154.7	160.7	166.6	172.6	178.5	184.4	190.4	196.4	202.4	208.3	214.2	220.2	.75	1.49	2.23
15/16	153.0	159.4	165.8	172.2	178.5	184.9	191.3	197.6	204.0	210.4	216.8	223.2	229.5	235.9	.80	1.60	2.39
1	163.2	170.0	176.8	183.6	190.4	197.2	204.0	210.8	217.6	224.4	231.2	238.0	244.8	251.6	.85	1.70	2.55

WEIGHTS OF STEEL PLATES PER LINEAL FOOT *(Continued)*.

Thickness, in Ins.	76″	78″	80″	82″	84″	86″	88″	90″	92″	94″	96″	98″	100″	1″	1¼″	1½″	1¾″
1/16	16.16	16.58	17.00	17.42	17.84	18.28	18.69	19.13	19.55	20.00	20.40	20.84	21.26	.21	.27	.32	.37
⅛	32.32	33.16	34.00	34.84	35.68	36.56	37.38	38.26	39.11	39.99	40.80	41.68	42.52	.43	.53	.64	.75
3/16	48.40	49.72	51.04	52.32	53.60	54.88	56.16	57.36	58.58	59.92	61.28	62.56	63.84	.64	.80	.96	1.11
¼	64.64	66.32	68.00	69.68	71.36	73.12	74.76	76.52	78.22	79.98	81.60	83.36	85.04	.85	1.06	1.28	1.49
5/16	80.78	82.88	84.96	87.10	89.28	91.36	93.44	95.60	97.76	99.92	102.1	104.1	106.2	1.06	1.33	1.59	1.86
⅜	96.90	99.48	102.0	104.6	107.1	109.7	112.2	114.8	117.4	119.9	122.4	125.0	127.5	1.28	1.59	1.92	2.23
7/16	113.1	116.0	119.0	122.0	125.0	127.9	130.9	133.9	136.9	139.9	142.9	145.8	148.8	1.49	1.86	2.23	2.60
½	129.2	132.6	136.0	139.4	142.8	146.2	149.6	153.0	156.4	159.8	163.2	166.6	170.0	1.70	2.12	2.55	2.98
9/16	145.4	149.2	153.1	156.9	160.6	164.5	168.2	172.1	176.0	179.8	183.7	187.4	191.2	1.92	2.39	2.87	3.35
⅝	161.5	165.7	170.0	174.2	178.6	182.8	187.0	191.3	195.5	199.8	204.0	208.2	212.5	2.12	2.65	3.19	3.72
11/16	177.7	182.3	187.0	191.7	196.3	201.0	205.6	210.4	215.1	219.8	224.5	229.1	233.8	2.34	2.92	3.51	4.09
¾	193.8	198.9	204.0	209.1	214.2	219.4	224.4	229.5	234.6	239.8	244.8	250.0	255.0	2.55	3.19	3.83	4.47
13/16	209.9	215.5	221.0	226.6	232.0	237.5	243.1	248.7	254.1	259.6	265.2	270.6	276.2	2.76	3.45	4.14	4.84
⅞	226.1	232.0	238.0	244.0	250.0	255.6	261.8	267.8	273.7	279.6	285.6	291.5	297.5	2.98	3.72	4.47	5.20
15/16	242.3	248.6	255.0	261.4	267.8	274.2	280.5	286.9	293.3	299.7	306.0	312.4	318.7	3.19	3.99	4.78	5.58
1	258.4	265.2	272.0	278.8	285.6	292.4	299.2	306.0	312.8	319.6	326.4	333.2	340.0	3.40	4.25	5.10	5.95

AREAS OF FLATS.

Thickness in Inches.	1″	1¼″	1½″	1¾″	2″	2¼″	2½″	2¾″	3″
1/16	.063	.078	.094	.109	.125	.141	.156	.172	.188
1/8	.125	.156	.188	.219	.250	.281	.313	.344	.375
3/16	.188	.234	.281	.328	.375	.422	.469	.516	.563
1/4	.250	.313	.375	.438	.500	.563	.625	.688	.750
5/16	.313	.391	.469	.547	.625	.703	.781	.859	.938
3/8	.375	.469	.563	.656	.750	.844	.938	1.03	1.13
7/16	.438	.547	.656	.766	.875	.984	1.09	1.20	1.31
1/2	.500	.625	.750	.875	1.00	1.13	1.25	1.38	1.50
9/16	.563	.703	.844	.984	1.13	1.27	1.41	1.55	1.69
5/8	.625	.781	.938	1.09	1.25	1.41	1.56	1.72	1.88
11/16	.688	.859	1.03	1.20	1.38	1.55	1.72	1.89	2.06
3/4	.750	.938	1.13	1.31	1.50	1.69	1.88	2.06	2.25
13/16	.813	1.02	1.22	1.42	1.63	1.83	2.03	2.23	2.44
7/8	.875	1.09	1.31	1.53	1.75	1.97	2.19	2.41	2.63
15/16	.938	1.17	1.41	1.64	1.88	2.11	2.34	2.58	2.81
1	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00
1 1/16	1.06	1.33	1.59	1.86	2.13	2.39	2.66	2.92	3.19
1⅛	1.13	1.41	1.69	1.97	2.25	2.53	2.81	3.09	3.38
1 3/16	1.19	1.48	1.78	2.08	2.38	2.67	2.97	3.27	3.56
1¼	1.25	1.56	1.88	2.19	2.50	2.81	3.13	3.44	3.75
1 5/16	1.31	1.64	1.97	2.30	2.63	2.95	3.28	3.61	3.94
1⅜	1.38	1.72	2.06	2.41	2.75	3.09	3.44	3.78	4.13
1 7/16	1.44	1.80	2.16	2.52	2.88	3.25	3.59	3.95	4.31
1½	1.50	1.88	2.25	2.63	3.00	3.38	3.75	4.13	4.50
1 9/16	1.56	1.95	2.34	2.73	3.13	3.52	3.91	4.30	4.69
1⅝	1.63	2.03	2.44	2.84	3.25	3.66	4.06	4.47	4.88
1 11/16	1.69	2.11	2.53	2.95	3.38	3.80	4.22	4.64	5.06
1¾	1.75	2.19	2.63	3.06	3.50	3.94	4.38	4.81	5.25
1 13/16	1.81	2.27	2.72	3.17	3.63	4.08	4.53	4.98	5.44
1⅞	1.88	2.34	2.81	3.28	3.75	4.22	4.69	5.16	5.63
1 15/16	1.94	2.42	2.91	3.39	3.88	4.36	4.84	5.33	5.81
2	2.00	2.50	3.00	3.50	4.00	4.50	5.00	5.50	6.00

AREAS OF FLATS.

(Continued.)

Thickness in Inches.	$3\frac{1}{2}''$	$4''$	$4\frac{1}{2}''$	$5''$	$6''$	$7''$	$8''$	$9''$	$10''$
$\frac{1}{16}$	.219	.250	.281	.313	.375	.438	.500	.563	.625
$\frac{1}{8}$	.438	.500	.563	.625	.750	.875	1.00	1.13	1.25
$\frac{3}{16}$	.656	.750	.844	.938	1.13	1.31	1.50	1.69	1.88
$\frac{1}{4}$	.875	1.00	1.13	1.25	1.50	1.75	2.00	2.25	2.50
$\frac{5}{16}$	1.09	1.25	1.41	1.56	1.88	2.19	2.50	2.81	3.13
$\frac{3}{8}$	1.31	1.50	1.69	1.88	2.25	2.63	3.00	3.38	3.75
$\frac{7}{16}$	1.53	1.75	1.97	2.19	2.63	3.06	3.50	3.94	4.38
$\frac{1}{2}$	1.75	2.00	2.25	2.50	3.00	3.50	4.00	4.50	5.00
$\frac{9}{16}$	1.97	2.25	2.53	2.81	3.38	3.94	4.50	5.06	5.63
$\frac{5}{8}$	2.19	2.50	2.81	3.13	3.75	4.38	5.00	5.63	6.25
$\frac{11}{16}$	2.41	2.75	3.09	3.44	4.13	4.81	5.50	6.19	6.88
$\frac{3}{4}$	2.63	3.00	3.38	3.75	4.50	5.25	6.00	6.75	7.50
$\frac{13}{16}$	2.84	3.25	3.66	4.06	4.88	5.69	6.50	7.31	8.13
$\frac{7}{8}$	3.06	3.50	3.94	4.38	5.25	6.13	7.00	7.88	8.75
$\frac{15}{16}$	3.28	3.75	4.22	4.69	5.63	6.56	7.50	8.44	9.38
1	3.50	4.00	4.50	5.00	6.00	7.00	8.00	9.00	10.00
$1\frac{1}{16}$	3.72	4.25	4.78	5.31	6.38	7.44	8.50	9.56	10.63
$1\frac{1}{8}$	3.94	4.50	5.06	5.63	6.75	7.88	9.00	10.13	11.25
$1\frac{3}{16}$	4.16	4.75	5.34	5.94	7.13	8.31	9.50	10.69	11.88
$1\frac{1}{4}$	4.38	5.00	5.63	6.25	7.50	8.75	10.00	11.25	12.50
$1\frac{5}{16}$	4.59	5.25	5.91	6.56	7.88	9.19	10.50	11.81	13.13
$1\frac{3}{8}$	4.81	5.50	6.19	6.88	8.25	9.63	11.00	12.38	13.75
$1\frac{7}{16}$	5.03	5.75	6.47	7.19	8.63	10.06	11.50	12.94	14.38
$1\frac{1}{2}$	5.25	6.00	6.75	7.50	9.00	10.50	12.00	13.50	15.00
$1\frac{9}{16}$	5.47	6.25	7.03	7.81	9.38	10.94	12.50	14.06	15.63
$1\frac{5}{8}$	5.69	6.50	7.31	8.13	9.75	11.38	13.00	14.63	16.25
$1\frac{11}{16}$	5.91	6.75	7.59	8.44	10.13	11.81	13.50	15.19	16.88
$1\frac{3}{4}$	6.13	7.00	7.88	8.75	10.50	12.25	14.00	15.75	17.50
$1\frac{13}{16}$	6.34	7.25	8.16	9.06	10.88	12.69	14.50	16.31	18.13
$1\frac{7}{8}$	6.56	7.50	8.44	9.38	11.25	13.13	15.00	16.88	18.75
$1\frac{15}{16}$	6.78	7.75	8.72	9.69	11.63	13.56	15.50	17.44	19.38
2	7.00	8.00	9.00	10.00	12.00	14.00	16.00	18.00	20.00

Weight per Square Foot of Sheets of Wrought Iron, Steel, Copper, and Brass.

THICKNESS BY BIRMINGHAM GAUGE.

No. of Gauge.	Thickness in Inches.	Iron.	Steel.	Copper.	Brass.
0000	.454	18.22	18.46	20.57	19.43
000	.425	17.05	17.28	19.25	18.19
00	.38	15.25	15.45	17.21	16.26
0	.34	13.64	13.82	15.40	14.55
1	.3	12.04	12.20	13.59	12.84
2	.284	11.40	11.55	12.87	12.16
3	.259	10.39	10.53	11.73	11.09
4	.238	9.55	9.68	10.78	10.19
5	.22	8.83	8.95	9.97	9.42
6	.203	8.15	8.25	9.20	8.69
7	.18	7.22	7.32	8.15	7.70
8	.165	6.62	6.71	7.47	7.06
9	.148	5.94	6.02	6.70	6.33
10	.134	5.38	5.45	6.07	5.74
11	.12	4.82	4.88	5.44	5.14
12	.109	4.37	4.43	4.94	4.67
13	.095	3.81	3.86	4.30	4.07
14	.083	3.33	3.37	3.76	3.55
15	.072	2.89	2.93	3.26	3.08
16	.065	2.61	2.64	2.94	2.78
17	.058	2.33	2.36	2.63	2.48
18	.049	1.97	1.99	2.22	2.10
19	.042	1.69	1.71	1.90	1.80
20	.035	1.40	1.42	1.59	1.50
21	.032	1.28	1.30	1.45	1.37
22	.028	1.12	1.14	1.27	1.20
23	.025	1.00	1.02	1.13	1.07
24	.022	.883	.895	1.00	.942
25	.02	.803	.813	.906	.856
26	.018	.722	.732	.815	.770
27	.016	.642	.651	.725	.685
28	.014	.562	.569	.634	.599
29	.013	.522	.529	.589	.556
30	.012	.482	.488	.544	.514
31	.01	.401	.407	.453	.428
32	.009	.361	.366	.408	.385
33	.008	.321	.325	.362	.342
34	.007	.281	.285	.317	.300
35	.005	.201	.203	.227	.214
Specific Gravity..		7.704	7.806	8.698	8.218
Weight Cubic ft..		481.25	487.75	543.6	513.6
Weight Cubic in.		.2787	.2823	.3146	.2972

Weight per Square Foot of Sheets of Wrought Iron, Steel, Copper, and Brass.

THICKNESS BY AMERICAN GAUGE.

No. of Gauge.	Thickness in Inches.	Iron.	Steel.	Copper.	Brass.
0000	.46	18.46	18.70	20.84	19.69
000	.4096	16.44	16.66	18.56	17.53
00	.3648	14.64	14.83	16.53	15.61
0	.3249	13.04	13.21	14.72	13.90
1	.2893	11.61	11.76	13.11	12.38
2	.2576	10.34	10.48	11.67	11.03
3	.2294	9.21	9.33	10.39	9.82
4	.2043	8.20	8.31	9.26	8.74
5	.1819	7.30	7.40	8.24	7.79
6	.1620	6.50	6.59	7.34	6.93
7	.1443	5.79	5.87	6.54	6.18
8	.1285	5.16	5.22	5.82	5.50
9	.1144	4.59	4.65	5.18	4.90
10	.1019	4.09	4.14	4.62	4.36
11	.0907	3.64	3.69	4.11	3.88
12	.0808	3.24	3.29	3.66	3.46
13	.0720	2.89	2.93	3.26	3.08
14	.0641	2.57	2.61	2.90	2.74
15	.0571	2.29	2.32	2.59	2.44
16	.0508	2.04	2.07	2.30	2.18
17	.0453	1.82	1.84	2.05	1.94
18	.0403	1.62	1.64	1.83	1.73
19	.0359	1.44	1.46	1.63	1.54
20	.0320	1.28	1.30	1.45	1.37
21	.0285	1.14	1.16	1.29	1.22
22	.0253	1.02	1.03	1.15	1.08
23	.0226	.906	.918	1.02	.966
24	.0201	.807	.817	.911	.860
25	.0179	.718	.728	.811	.766
26	.0159	.640	.648	.722	.682
27	.0142	.570	.577	.643	.608
28	.0126	.507	.514	.573	.541
29	.0113	.452	.458	.510	.482
30	.0100	.402	.408	.454	.429
31	.0089	.358	.363	.404	.382
32	.0080	.319	.323	.360	.340
33	.0071	.284	.288	.321	.303
34	.0063	.253	.256	.286	.270
35	.0056	.225	.228	.254	.240

As there are many gauges in use differing from each other, and even the thicknesses of a certain specified gauge, as the Birmingham, are not assumed the same by all manufacturers, orders for sheets and wire should always state the weight per □ foot or the thickness in thousandths of an inch.

DIFFERENT STANDARDS FOR WIRE GAUGE IN USE IN THE U. S.

DIMENSIONS IN DECIMAL PARTS OF AN INCH.

Number of Wire Gauge.	American, or Brown & Sharpe.	Birmingham, or Stubs'.	Washburn & Moen Mnfg. Co., Worcester, Mass.	Trenton Iron Co., Trenton, N. J.	United States Standard.	Old English, from Brass Mfrs. List.
000000			.46		.46875	
00000			.43	.45	.4375	
0000	.46	.454	.393	.4	.40625	
000	.40964	.425	.362	.36	.375	
00	.3648	.38	.331	.33	.34375	
0	.32495	.34	.307	.305	.3125	
1	.2893	.3	.283	.285	.28125	
2	.25763	.284	.263	.265	.26563	
3	.22942	.259	.244	.245	.25	
4	.20431	.238	.225	.225	.23438	
5	.18194	.22	.207	.205	.21875	
6	.16202	.203	.192	.19	.20313	
7	.14428	.18	.177	.175	.1875	
8	.12849	.165	.162	.16	.17188	
9	.11443	.148	.148	.145	.15625	
10	.10189	.134	.135	.13	.14063	
11	.090742	.12	.12	.1175	.125	
12	.080808	.109	.105	.105	.10938	
13	.071961	.095	.092	.0925	.09375	
14	.064084	.083	.08	.08	.07813	.083
15	.057068	.072	.072	.07	.07031	.072
16	.05082	.065	.063	.061	.0625	.065
17	.045257	.058	.054	.0525	.05625	.058
18	.040303	.049	.047	.045	.05	.049
19	.03539	.042	.041	.039	.04375	.04
20	.031961	.035	.035	.034	.0375	.035
21	.028462	.032	.032	.03	.03438	.0315
22	.025347	.028	.028	.027	.03125	.0295
23	.022571	.025	.025	.024	.02813	.027
24	.0201	.022	.023	.0215	.025	.025
25	.0179	.02	.02	.019	.02188	.023
26	.01594	.018	.018	.018	.01875	.0205
27	.014195	.016	.017	.017	.01719	.01875
28	.012641	.014	.016	.016	.01563	.0165
29	.011257	.013	.015	.015	.01406	.0155
30	.010025	.012	.014	.014	.0125	.01375
31	.008928	.01	.0135	.013	.01094	.01225
32	.00795	.009	.013	.012	.01016	.01125
33	.00708	.008	.011	.011	.00938	.01025
34	.006304	.007	.01	.01	.00859	.0095
35	.005614	.005	.0095	.009	.00781	.009

WIRE—Iron, Steel, Copper, Brass.

Weight of 100 Feet in Pounds.

BIRMINGHAM WIRE GAUGE.

No. of Gauge.	PER 100 LINEAL FEET.			
	Iron.	Steel.	Copper.	Brass.
0000	54.62	55.13	62.39	58.93
000	47.86	48.32	54.67	51.64
00	38.27	38.63	43.71	41.28
0	30.63	30.92	34.99	33.05
1	23.85	24.07	27.24	25.73
2	21.37	21.57	24.41	23.06
3	17.78	17.94	20.3	19.18
4	15.01	15.15	17.15	16.19
5	12.82	12.95	14.65	13.84
6	10.92	11.02	12.47	11.78
7	8.586	8.667	9.807	9.263
8	7.214	7.283	8.241	7.783
9	5.805	5.859	6.63	6.262
10	4.758	4.803	5.435	5.133
11	3.816	3.852	4.359	4.117
12	3.148	3.178	3.596	3.397
13	2.392	2.414	2.732	2.58
14	1.826	1.843	2.085	1.969
15	1.374	1.387	1.569	1.482
16	1.119	1.13	1.279	1.208
17	.8915	.9	1.018	.9618
18	.6363	.6423	.7268	.6864
19	.4675	.472	.534	.5043
20	.3246	.3277	.3709	.3502
21	.2714	.274	.31	.2929
22	.2079	.2098	.2373	.2241
23	.1656	.1672	.1892	.1788
24	.1283	.1295	.1465	.1384
25	.106	.107	.1211	.1144
26	.0859	.0867	.0981	.0926
27	.0678	.0685	.0775	.0732
28	.0519	.0524	.0593	.056
29	.0448	.0452	.0511	.0483
30	.0382	.0385	.0436	.0412
31	.0265	.0267	.0303	.0286
32	.0215	.0217	.0245	.0231
33	.017	.0171	.0194	.0183
34	.013	.0131	.0148	.014
35	.0066	.0067	.0076	.0071
36	.0042	.0043	.0048	.0046

WROUGHT-IRON WELDED TUBES, FOR STEAM, GAS, OR WATER.

1¼ inch and below, Butt Welded; proved to 300 lbs. per square inch, Hydraulic Pressure.
1½ inch and above, Lap Welded; proved to 500 lbs. per square inch, Hydraulic Pressure.

TABLE OF STANDARD SIZES.—MORRIS, TASKER & CO.

Inside Diameter.	Actual Inside Diameter.	Actual Outside Diameter.	Thickness.	Internal Circumference.	External Circumference.	Length of Pipe per □ foot inside Surface.	Length of Pipe per □ ft. outside Surface.	Internal Area.	External Area.	Length of Pipe containing 1 cubic ft.	Weight per foot of Length.	No. of Threads per inch of Screw
Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Feet.	Feet.	Inches.	Inches.	Feet.	Lbs.	
⅛	.270	.405	.068	.848	1.272	14.15	9.44	.0572	.129	2500.	.243	27
¼	.364	.54	.088	1.144	1.699	10.50	7.075	.1041	.229	1385.	.422	18
⅜	.494	.675	.091	1.552	2.121	7.67	5.657	.1916	.358	751.5	.561	18
½	.623	.84	.109	1.957	2.652	6.13	4.502	.3048	.554	472.4	.845	14
¾	.824	1.05	.113	2.589	3.299	4.635	3.637	.5333	.866	270.	1.126	14
1	1.048	1.315	.134	3.292	4.134	3.679	2.903	.8627	1.357	156.9	1.670	11½
1¼	1.380	1.66	.140	4.335	5.215	2.768	2.301	1.496	3.164	96.25	2.258	11½
1½	1.611	1.9	.145	5.061	5.969	2.371	2.01	2.038	2.835	70.65	2.694	11½
2	2.067	2.375	.154	6.494	7.461	1.848	1.611	3.355	4.430	42.36	3.667	11½
2½	2.468	2.875	.204	7.754	9.032	1.547	1.328	4.783	6.491	30.11	5.773	8
3	3.067	3.5	.217	9.636	11.996	1.245	1.091	7.388	9.621	19.49	7.547	8
3½	3.548	4.	.226	11.146	12.566	1.077	.955	9.887	12.566	14.56	9.055	8
4	4.026	4.5	.237	12.648	14.137	.949	.849	12.730	15.904	11.31	10.728	8
4½	4.508	5.	.247	14.153	15.708	.848	.765	15.939	19.635	9.03	12.492	8
5	5.045	5.563	.259	15.849	17.475	.757	.629	19.990	24.299	7.20	14.564	8
6	6.065	6.625	.280	19.054	20.813	.63	.577	28.889	34.471	4.98	18.767	8
7	7.023	7.625	.301	22.063	23.954.	.544	.505	38.737	45.663	3.72	23.410	8
8	7.982	8.625	.322	25.076	27.096	.478	.444	50.039	58.426	2.88	28.348	8
9	9.001	9.688	.344	28.277	30.433	.425	.394	63.633	73.715	2.26	34.077	8
10	10.019	10.75	.366	31.475	33.772	.381	.355	78.838	90.762	1.80	40.641	8

LAP-WELDED AMERICAN CHARCOAL IRON BOILER TUBES.

TABLES OF STANDARD SIZES.

MORRIS, TASKER & CO.

External Diameter.	Internal Diameter.	Thickness.	External Circumference.	Internal Circumference.	Length of Pipe per □ foot inside surface.	Length of Pipe per □ foot outside surface.	Internal Area.	External Area.	Weight per foot.
Inch.	Inch.	Inch.	Inch.	Inch.	Feet.	Feet.	Inch.	Inch.	Lbs.
1	0.856	0.072	3.142	2.689	4.460	3.819	0.575	0.785	0.708
1¼	1.106	0.072	3.927	3.474	3.455	3.056	0.960	1.227	0.9
1½	1.334	0.083	4.712	4.191	2.863	2.547	1.396	1.767	1.250
1¾	1.560	0.095	5.498	4.901	2.448	2.183	1.911	2.405	1.665
2	1.804	0.098	6.283	5.667	2.118	1.909	2.556	3.142	1.981
2¼	2.054	0.098	7.069	6.484	1.850	1.698	3.314	3.976	2.238
2½	2.283	0.109	7.854	7.172	1.673	1.528	4.094	4.909	2.755
2¾	2.533	0.109	8.639	7.957	1.508	1.390	5.039	5.940	3.045
3	2.783	0.109	9.425	8.743	1.373	1.273	6.083	7.069	3.333
3¼	3.012	0.119	10.210	9.462	1.268	1.175	7.125	8.296	3.958
3½	3.262	0.119	10.995	10.248	1.171	1.091	8.357	9.621	4.272
3¾	3.512	0.119	11.781	11.033	1.088	1.018	9.687	11.045	4.590
4	3.741	0.130	12.566	11.753	1.023	0.955	10.992	12.566	5.320
4½	4.241	0.130	14.137	13.323	0.901	0.849	14.126	15.904	6.010
5	4.72	0.140	15.708	14.818	0.809	0.764	17.497	19.635	7.226
6	5.699	0.151	18.849	17.904	0.670	0.637	25.509	28.274	9.346
7	6.657	0.172	21.991	20.914	0.574	0.545	34.805	38.484	12.435
8	7.636	0.182	25.132	23.989	0.500	0.478	45.795	50.265	15.109
9	8.615	0.193	28.274	27.055	0.444	0.424	58.291	63.617	18.002
10	9.573	0.214	31.416	30.074	0.399	0.382	71.975	78.540	22.19

WROUGHT-IRON WELDED TUBES.

EXTRA STRONG.

Nominal Diameter.	Actual Outside Diameter.	Thickness. Extra Strong.	Thickness. Double Extra Strong.	Actual Inside Diameter. Extra Strong	Actual Inside Diam. Double Extra Strong
⅛	.405	.100		.205	
¼	.54	.123		.294	
⅜	.675	.127		.421	
½	.84	.149	.298	.542	.244
¾	1.05	.157	.314	.736	.422
1	1.315	.182	.364	.951	.587
1¼	1.66	.194	.388	1.272	.884
1½	1.9	.203	.406	1.494	1.088
2	2.375	.221	.442	1.933	1.491
2½	2.875	.280	.560	2.315	1.755
3	3.5	.304	.608	2.892	2.284
3½	4.	.321	.642	3.358	2.716
4	4.5	.341	.682	3.818	3.136

SPIKES, NAILS AND TACKS.

STANDARD STEEL WIRE NAILS.						STEEL WIRE SPIKES.		
		Common.		Finishing.				
Sizes.	Length.	Diam., inches.	No. per pound.	Diam., inches.	No. per pound.	Length.	Diam., inches.	No. per pound.
2d	1″	.0524	1060	.0453	1558	3″	.1620	41
3d	1¼″	.0588	640	.0508	913	3½″	.1819	30
4d	1½″	.0720	380	.0508	761	4″	.2043	23
5d	1¾″	.0764	275	.0571	500	4½″	.2294	17
6d	2″	.0808	210	.0641	350	5″	.2576	13
7d	2¼″	.0858	160	.0641	315	5½″	.2893	11
8d	2½″	.0935	115	.0720	214	6″	.2893	10
9d	2¾″	.0963	93	.0720	195	6½″	.2249	7½
10d	3″	.1082	77	.0808	137	7″	.2249	7
12d	3¼″	.1144	60	.0808	127	8″	.3648	5
16d	3½″	.1285	48	.0907	90	9″	.3648	4½
20d	4″	.1620	31	.1019	62			
30d	4½″	.1819	22					
40d	5″	.2043	17					
50d	5½″	.2294	13					
60d	6″	.2576	11					

WOOD SCREWS.

No.	Diam.	No.	Diam.	No.	Diam.	No.	Diam.	No.	Diam.
0	.056	6	.135	12	.215	18	.293	24	3/8 .374
1	.069	7	.149	13	.228	19	.308	25	.387
2	.082	8	.162	14	1/4 .241	20	5/16 .321	26	.401
3	.096	9	.175	15	.255	21	.334	27	.414
4	.109	10	3/16 .188	16	.268	22	.347	28	.427
5	1/8 .122	11	.201	17	.281	23	.361	29	.440
								30	.453

WROUGHT SPIKES.

Number to a keg of 150 lbs.

L'gth, inch.	¼ inch. No.	5/16 inch. No.	⅜ inch. No.	L'gth, inch.	¼ inch. No.	5/16 in. No.	⅜ inch. No.	7/16 in. No.	½ inch. No.
3	2250			7	1161	662	482	445	306
3½	1890	1208		8		635	455	384	256
4	1650	1135		9		573	424	300	240
4½	1464	1064		10			391	270	222
5	1380	930	742	11				249	203
6	1292	868	570	12				236	180

NAILS AND SPIKES.

Size, Length, and Number to the Pound.

ORDINARY.

Size.	Length.	No. to Lb.
2^d	$1''$	800
3	$1\frac{1}{4}''$	400
4	$1\frac{1}{2}''$	300
5	$1\frac{3}{4}''$	200
6	$2''$	150
7	$2\frac{1}{4}''$	120
8	$2\frac{1}{2}''$	85
9	$2\frac{3}{4}''$	75
10	$3''$	60
12	$3\frac{1}{4}''$	50
16	$3\frac{1}{2}''$	40
20	$4''$	20
30	$4\frac{1}{2}''$	16
40	$5''$	14
50	$5\frac{1}{2}''$	11
60	$6''$	8

LIGHT.

Size.	Length.	No. to Lb.
4^d	$1\frac{3}{8}''$	373
5	$1\frac{3}{4}$	272
6	2	196

BRADS.

Size.	Length.	No. to Lb.
6^d	$2''$	163
8	$2\frac{1}{2}$	96
10	$2\frac{3}{4}$	74
12	$3\frac{1}{8}$	50

CLINCH.

Length.	No. to Lb.
$2''$	152
$2\frac{1}{4}$	133
$2\frac{1}{2}$	92
$2\frac{3}{4}$	72
3	60
$3\frac{1}{4}$	43

FENCE.

Length.	No. to Lb.
$2''$	96
$2\frac{1}{4}$	66
$2\frac{1}{2}$	56
$2\frac{3}{4}$	50
3	40

SPIKES.

Length.	No. to Lb.
$3\frac{1}{2}''$	19
4	15
$4\frac{1}{2}$	13
5	10
$5\frac{1}{2}$	9
6	7

BOAT.

Length.	No. to Lb.
$1\frac{1}{2}''$	206

FINISHING.

Size.	Length.	No. to Lb.
4^d	$1\frac{3}{8}''$	384
5	$1\frac{3}{4}$	256
6	2	204
8	$2\frac{1}{2}$	102
10	3	80
12	$3\frac{5}{8}$	65
20	$3\frac{7}{8}$	46

CORE.

Size.	Length.	No. to Lb.
6^d	$2''$	143
8	$2\frac{1}{2}$	68
10	$2\frac{1}{3}$	60
12	$3\frac{1}{8}$	42
20	$3\frac{3}{4}$	25
30	$4\frac{1}{4}$	18
40	$4\frac{3}{4}$	14
W H	$2\frac{1}{2}$	69
W H L	$2\frac{1}{4}$	72

SLATE.

Size.	Length.	No. to Lb.
3^d	$1\frac{5}{16}''$	288
4	$1\frac{7}{16}$	244
5	$1\frac{3}{4}$	187
6	2	146

TACKS.

Size.	Length.	No. to Lb.	Size.	Length.	No. to Lb.	Size.	Length.	No. to Lb.
1 oz.	$\frac{1}{8}$	16000	4 oz.	$\frac{7}{16}$	4000	14 oz.	$\frac{13}{16}$	1143
$1\frac{1}{2}$	$\frac{3}{16}$	10666	6	$\frac{9}{16}$	2666	16	$\frac{7}{8}$	1000
2	$\frac{1}{4}$	8000	8	$\frac{5}{8}$	2000	18	$\frac{15}{16}$	888
$2\frac{1}{2}$	$\frac{5}{16}$	6400	10	$\frac{11}{16}$	1600	20	1	800
3	$\frac{3}{8}$	5333	12	$\frac{3}{4}$	1333	22	$1\frac{1}{16}$	727

WINDOW GLASS.

Number of Lights per Box of 50 Feet.

Inches.	No.	Inches.	No.	Inches.	No.	Inches.	No.
6× 8	150	12×18	33	16×44	10	26×32	9
7 9	115	12 20	30	18 20	20	26 34	8
8 10	90	12 22	27	18 22	18	26 36	8
8 11	82	12 24	25	18 24	17	26 40	7
8 12	75	12 26	23	18 26	15	26 42	7
8 13	70	12 28	21	18 28	14	26 44	6
8 14	64	12 30	20	18 30	13	26 48	6
8 15	60	12 32	18	18 32	13	26 50	6
8 16	55	12 34	17	18 34	12	26 54	5
9 11	72	13 14	40	18 36	11	26 58	5
9 12	67	13 16	35	18 38	11	28 30	9
9 13	62	13 18	31	18 40	10	28 32	8
9 14	57	13 20	28	18 44	9	28 34	8
9 15	53	13 22	25	20 22	16	28 36	7
9 16	50	13 24	23	20 24	15	28 38	7
9 17	47	13 26	21	20 26	14	28 40	6
9 18	44	13 28	19	20 28	13	28 44	6
9 20	40	13 30	18	20 30	12	28 46	6
10 12	60	14 16	32	20 32	11	28 50	5
10 13	55	14 18	29	20 34	11	28 52	5
10 14	52	14 20	26	20 36	10	28 56	4
10 15	48	14 22	23	20 38	9	30 36	7
10 16	45	14 24	22	20 40	9	30 40	6
10 17	42	14 26	20	20 44	8	30 42	6
10 18	40	14 28	18	20 46	8	30 44	5
10 20	36	14 30	17	20 48	8	30 46	5
10 22	33	14 32	16	20 50	7	30 48	5
10 24	30	14 34	15	20 60	6	30 50	5
10 26	28	14 36	14	22 24	14	30 54	4
10 28	26	14 40	13	22 26	13	30 56	4
10 30	24	14 44	11	22 28	12	30 60	4
10 32	22	15 18	27	22 30	11	32 42	5
10 34	21	15 20	24	22 32	10	32 44	5
11 13	50	15 22	22	22 34	10	32 46	5
11 14	47	15 24	20	22 36	9	32 48	5
11 15	44	15 26	18	22 38	9	32 50	4
11 16	41	15 28	17	22 40	8	32 54	4
11 17	39	15 30	16	22 44	8	32 56	4
11 18	36	15 32	15	22 46	7	32 60	4
11 20	33	16 18	25	22 50	7	34 40	5
11 22	30	16 20	23	24 28	11	34 44	5
11 24	27	16 22	20	24 30	10	34 46	5
11 26	25	16 24	19	24 32	9	34 50	4
11 28	23	16 26	17	24 36	8	34 52	4
11 30	21	16 28	16	24 40	8	34 56	4
11 32	20	16 30	15	24 44	7	36 44	5
11 34	19	16 32	14	24 46	7	36 50	4
12 14	43	16 34	13	24 48	6	36 56	4
12 15	40	16 36	12	24 50	6	36 60	3
12 16	38	16 38	12	24 54	5	36 64	3
12 17	35	16 40	11	24 56	5	40 60	3

ROOFING SLATE.

General Rule for the Computation of Slate.

A square of slating is 100 sq. ft. of finished roofing. Slating is usually laid so that the third slate laps the first slate by three inches. To compute the number of slates, of a given size, required to cover a square of roof; subtract three inches from the length of the slate, multiply the remainder by the width of the slate and divide by 2; the result is the number of sq. ins. of roof covered per slate; divide 14,400 (the number of sq. ins. in a square) by the number so found, and the result will be the number of slates required for a square.

Weight per Cubic Foot, - 174 Pounds.

Weight per Square Foot.

Thickness......	$\frac{1}{8}$	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	1 inch.
Weight........	1.81	2.71	3.62	5.43	7.25	9.06	10.87	14.5 lbs.

TABLE OF SIZES AND NUMBER OF SLATE IN ONE SQUARE.

Size in Inches.	No. of Slate in Square.	Size in Inches.	No. of Slate in Square.	Size in Inches.	No. of Slate in Square.
6×12	533	8×16	277	12×20	141
7 12	457	9 16	246	14 20	121
8 12	400	10 16	221	11 22	137
9 12	355	12 16	184	12 22	126
10 12	320	9 18	213	14 22	108
12 12	266	10 18	192	12 24	114
7 14	374	11 18	174	14 24	98
8 14	327	12 18	160	16 24	86
9 14	291	14 18	137	14 26	89
10 14	261	10 20	169	16 26	78
12 14	218	11 20	154		

CAPACITY OF CISTERNS OR TANKS,

In Gallons, for Each Foot in Depth.

Diameter in Feet.	Gallons.	Diameter in Feet.	Gallons.
2.	23.5	9.	475.87
2.5	36.7	9.5	553.67
3.	52.9	10.	587.5
3.5	71.96	11.	710.9
4.	94.02	12.	846.4
4.5	119.	13.	992.9
5.	146.8	14.	1,151.5
5.5	177.7	15.	1,321.9
6.	211.6	20.	2,350.0
6.5	248.22	25.	3,570.7
7.	287.84	30.	5,287.7
7.5	330.48	35.	7,189.
8.	376.	40.	9,367.2
8.5	424.44	45.	11,893.2

The American standard gallon contains 231 cubic inches, or 8⅓ pounds of pure water. A cubic foot contains 62.3 pounds of water, or 7.48 gallons. Pressure per square inch is equal to the depth or head in feet multiplied by .433. Each 27.72 inches of depth gives a pressure of one pound to the square inch.

SKYLIGHT AND FLOOR GLASS.

Weight per Cubic Foot, - 156 Pounds.

Weight per Square Foot.

Thickness.	1/8	3/16	1/4	3/8	1/2	5/8	3/4	1 inch.
Weight	1.62	2.43	3.25	4.88	6.50	8.13	9.75	13 lbs.

FLAGGING.

Weight per Cubic Foot, - 168 Pounds.

Weight per Square Foot.

Thickness	1	2	3	4	5	6	7	8 inch.
Weight	14	28	42	56	70	84	98	112 lbs.

NOTES ON BRICKWORK.

IN ordinary brickwork, one cubic foot of wall will require 21 bricks of 8 in. × 2½ in. × 3½ in.

For 1000 ordinary bricks is required 1 barrel of good lime, 2 cartloads of ordinary sharp sand.

One brick as above weighs 4 lbs., dry; if perfectly soaked in water, 5 lbs. It will absorb 1 lb. or one pint of water.

Edgewise arches will require about 7 bricks per square foot of floor, and endwise arches will require about 14 bricks of the size given above.

For 1 cubic yard of concrete is required 1 barrel of cement, 2 barrels of good sharp sand, 1 cubic yard of broken stone.

TRANSVERSE STRENGTH OF BUILDING STONES.

b = width of stone, in inches.

d = thickness of stone, in inches.

l = length of span, in inches.

The safe uniformly distributed loads, in tons of 2000 lbs., for a factor of safety of 10, can be obtained by multiplying the coefficients, given in the table, by $\frac{b\,d^2}{l}$

	Coefficients.
Bluestone	0.18
Granite	0.12
Limestone	0.10
Sandstone	0.08
Slate	0.36

Thus, a granite lintel, 24 inches wide and 12 inches thick, spanning an opening of 48 inches would sustain a safe load of

$$\frac{24 \times 144}{48} \times 0.12 = 8.64 \text{ tons.}$$

If the loads are concentrated at the center of the span, the safe load will be one-half the safe uniform load given by the table.

NOTES ON STEEL AND IRON.

Wrought iron weighs 480 lbs. per cubic foot. A bar, 1 in. square and 3 ft. long, weighs, therefore, exactly 10 lbs. Hence:

The sectional area, in sq. ins. = the weight per foot $\times \frac{3}{10}$

The weight per foot, in lbs. = sectional area $\times \frac{10}{3}$

Steel weighs 490 lbs. per cubic foot, or 2 per cent. greater than wrought iron. Hence for steel:

The sectional area, in sq. ins. = weight per foot ÷ 3.4

The weight per foot in lbs. = sectional area × 3.4

The melting-points of iron and steel are about as follows:

Wrought Iron	3,000°	Fahrenheit
Cast Iron	2,000°	"
Steel	2,400°	"

The welding heat of wrought iron is 2,700° Fahrenheit.

The contraction of a wrought-iron rod in cooling is about equivalent to $\frac{1}{10000}$ of its length for a decrease of 15° Fahr., and the strain thus induced is about *one ton* (2240 lbs.) for every square inch of sectional area in the bar.

For a rod of the lengths given below, the contraction will be as follows:

Length of rod in feet	10	20	30	40	50	100	150
Contrac'n in inches for **15°**	.012	.024	.036	.048	.060	.120	.180
" " **150°**	.120	.240	.360	.480	.600	1.200	1.800
" " **100°**	.080	.160	.240	.320	.400	.800	1.200

Contraction and expansion being equal the pressure per square inch induced by heating or cooling is as follows:

For temperatures varying by 15° Fahr.:

Variation	15	30	45	60	75	105	120	150 degrees.
Pressure	1	2	3	4	5	7	8	10 tons.

AVERAGE ULTIMATE STRENGTHS OF MATERIALS.

Lbs. per square inch.

TIMBERS.	Tension.		Compression.			Transverse.		Shearing.	
			With Grain.						
	With Grain.	Across Grain.	End bearing.	Cols. under 15 Diams.	Across Grain.	Extreme fiber stress.	Modulus of Elasticity.	With Grain.	Across Grain.
White oak	10,000	2,000	7,000	4,500	2,000	6,000	1,100,000	800	4,000
White pine	7,000	500	5,500	3,500	800	4,000	1,000,000	400	2,000
Southern, Long-Leaf, or Georgia yellow pine	12,000	600	8,000	5,000	1,400	7,000	1,700,000	600	5,000
Douglass, Oregon and Washington fir or pine: yellow fir	12,000	.....	8,000	6,000	1,200	6,500	1,400,000	600	.....
Douglass, Oregon and Washington fir or pine: red fir	10,000	.....	.....	.....	.....	5,000	....	...	.....
Northern or Short-leaf yellow pine	9,000	500	6,000	4,000	1,000	6,000	1,200,000	400	4,000
Red pine	9,000	500	6,000	4,000	800	5,000	1,200,000	...	.....
Norway pine	8,000	.....	6,000	4,000	800	4,000	1,200,000	...	.....
Canadian (Ottawa) white pine	10,000	.....	.....	5,000	.....	.....	....	350	.....
Canadian (Ontario) red pine	10,000	.....	.....	5,000	.....	5,000	1,400,000	400	.....
Spruce and Eastern fir	8,000	500	6,000	4,000	700	4,000	1,200,000	400	3,000
Hemlock	6,000	.....	.....	4,000	600	3,500	900,000	350	2,500
Cypress	6,000	.....	6,000	4,000	700	5,000	900,000	...	.....
Cedar	8,000	.....	6,000	4,000	700	5,000	700,000	...	1,500
Chestnut	9,000	.....	.....	5,000	900	5,000	1,000,000	600	1,500
California redwood	7,000	.....	.....	4,000	800	4,500	700,000	400	.....
California spruce	....	.....	.....	4,000	.....	5,000	1,200,000	...	.....

For quiescent loads, as in buildings, divide above values by the following factors; Tension, 10; Compression, 5; Transverse, 6; Shearing, 5.

AVERAGE ULTIMATE STRENGTHS OF MATERIALS *(Continued).*

Lbs. per square inch.

METALS.	Compression.	Tension.	Elastic Limit.	Shearing.	Modulus of Rupture.	Modulus of Elasticity.
Aluminum, commercial	12,000	15,000	6,500	12,000	......	11,000,000
" nickel	......	40,000	22,000	......	......	....
Brass, cast	(30,000)	24,000	6,000	36,000	20,000	9,000,000
" wire, annealed	......	50,000	......	......	......	....
" " unannealed	......	80,000	16,000	......	......	14,000,000
Bronze, aluminum	120,000	75,000	......	......	......	....
" gun metal	(20,000)	32,000	10,000	......	53,000	10,000,000
" manganese	120,000	60,000	30,000	......	......	....
" phosphor	......	50,000	24,000	......	......	14,000,000
" Tobin	......	66,000	40,000	......	......	4,500,000
Copper, bolts	30,000	30,000	......	......	......	....
" cast	(40,000)	24,000	6,000	30,000	22,000	10,000,000
" wire, annealed	......	36,000	......	......	......	15,000,000
" " unannealed	......	60,000	10,000	......	......	18,000,000
Gold, cast	......	20,000	4,000	......	......	8,000,000
Iron, cast	80,000	15,000	6,000	18,000	30,000	12,000,000
" chains	......	35,000	......	......	......	....
" corrugated	......	......	......	......	40,000	....
" wire, annealed	......	60,000	......	......	......	15,000,000
" " unannealed	......	80,000	27,000	......	......	25,000,000
" wrought, shapes	46,000	48,000	26,000	40,000	44,000	27,000,000
" " rerolled bars	48,000	50,000	27,000	40,000	48,000	26,000,000

Compression values enclosed in parentheses indicate loads producing 10% reduction in original lengths.

AVERAGE ULTIMATE STRENGTHS OF MATERIALS *(Continued).*

Lbs. per square inch.

METALS.	Compression.	Tension.	Elastic Limit.	Shearing.	Modulus of Rupture.	Modulus of Elasticity.
Lead, cast	......	2,000	1,000	......	......	1,000,000
" pipe	......	1,600	......	......	......	....
Silver, cast	......	40,000	4,000	......	......	10,000,000
Steel, castings	70,000	70,000	40,000	60,000	70,000	30,000,000
" structural, 0.10% carbon	56,000	56,000	30,000	48,000	54,000	29,000,000
" " 0.15% "	64,000	64,000	33,000	50,000	60,000	29,000,000
" wire, annealed	......	80,000	40,000	......	......	29,000,000
" " unannealed	......	120,000	60,000	......	......	30,000,000
" " crucible	......	180,000	80,000	......	......	30,000,000
" " for suspension bridges	......	200,000	90,000	......	......	30,000,000
" " special tempered	......	300,000	......	......	......	....
Tin, cast	(6,000)	3,500	1,800	......	4,000	4,000,000
Zinc, cast	(20,000)	5,000	4,000	......	7,000	13,000,000
MISCELLANEOUS:						
Flax yarn	......	25,000	......	......	......	....
Glass, common green	20,000	3,000	3,000	......	4,000	8,000,000
" flooring	10,000	3,000	......	......	3,000	....
" wire, for skylights	......	......	5,000	......	5,000	....
Leather, ox	......	4,000	......	......	......	240,000
Rope, hemp	......	8,000	......	......	......	....
" manila	......	9,000	......	......	......	....
Silk, fiber	......	5,000	......	......	......	1,300,000

Compression values enclosed in parentheses indicate loads producing 10% reduction in original lengths.

AVERAGE ULTIMATE STRENGTHS OF MATERIALS

Lbs. per Square Inch. (*Continued*).

MATERIAL.	Compression.	Tension.	Modulus of Rupture.
BUILDING STONES:			
Bluestone	13,500	1,400	2,700
Granite, average	15,000	600	1,800
" Connecticut	12,000	....	....
" New Hampshire	15,000	....	1,500
" Massachusetts	16,000	....	1,800
" New York	15,000	....	....
Limestone, average	7,000	1,000	1,500
" Hudson River, N. Y.	17,000	....	....
" Ohio	12,000	....	1,500
Marble, average	8,000	700	....
" Vermont	8,000	700	1,200
Sandstone, average	5,000	150	1,200
" New Jersey	12,000	....	650
" New York	10,000	....	1,700
" Ohio	9,000	100	700
Slate	10,000	10,000	5,000
Stonework	($\frac{4}{10}$ Strength	of	Stone.)
BRICKS:			
Bricks, light red	1,000	40	....
" good common	10,000	200	600
" best hard	12,000	400	800
" Phila. pressed	6,000	200	600
Brickwork, common (lime mortar)	1,000	50	....
Brickwork, good (cement and lime mortar)	1,500	100	....
Brickwork, best (cement mortar).	2,000	300	....
Terra Cotta	5,000	....	....
" " work	2,000	....	....
CEMENTS, ETC.:			
Cement, Rosendale, 1 month old.	1,200	200	200
" Portland, 1 " "	2,000	400	400
" Rosendale, 1 year old.	2,000	300	400
" Portland, 1 " "	3,000	500	800
Mortar, lime, 1 year old	400	50	100
" lime & Rosendale, 1 y. old	600	75	200
Mortar, Rosendale cement, 1 year old	1,000	125	300
Mortar, Portland cement, 1 y. old.	2,000	250	600
Concrete, Portland, 1 month old	1,000	200	100
" Rosendale, 1 " "	500	100	50
" Portland, 1 year old.	2,000	400	150
" Rosendale, 1 " ".	1,000	200	75

Safe strengths of Stone, Brick and Cement, $\frac{1}{10}$ to $\frac{1}{30}$ of ultimate.

WEIGHTS OF VARIOUS SUBSTANCES.

NAME OF SUBSTANCE.	Average Weight per cubic foot, lbs.
Alcohol, commercial	52
Aluminum	166
Antimony, cast	418
Apple	47
Ash, American, perfectly dry	38
" Canadian, " "	38
Asphalt, pavement composition	130
" refined	93
" Trinidad, natural state	80
Basalt	181
Beech	48
Birch	43
Bismuth, cast	614
Bluestone	160
Boxwood, perfectly dry	62
Brass	523
Brick, best pressed	135 to 150
" common hard	110 " 125
" fire	140 " 150
" soft, inferior	100
Brickwork, pressed brick	112 to 140
" ordinary	110 " 112
Bronze	552
Calcite, transparent	170
Cedar	39 to 41
Cement, Louisville	50
" Portland	80 to 100
" Rosendale	56 " 60
Chalk	156
Charcoal	15 to 30
Cherry, perfectly dry	42
Chestnut, " "	41
Clay, potters', dry	119
" dry, loose	63
Coal, anthracite, broken	52 to 56
" " moderately shaken	56 " 60
" " solid	93
" " heaped bushel, loose	(77 to 83)
" bituminous, solid	84
" " broken, loose	54
" " heaped bushel, loose	(74)
Coke, of good coal, loose	30 to 50
Concrete	120 " 140
Copper, cast	552

WEIGHTS OF VARIOUS SUBSTANCES *(Continued).*

NAME OF SUBSTANCE.	Average Weight per cubic foot, lbs.
Cork	15
Earth, dry, loose	72 to 80
" " moderately rammed	90 " 100
" moist, moderately packed	90 " 100
" as a soft flowing mud	104 " 112
" firm, solid	115
Elm, Canadian, dry	47
Emery	250
Fat	58
Feldspar	166
Fir, New England	40
Flint	162
Glass, common window	163
" flint	186
" Millville, N. J., flooring glass	158
Gneiss, common	168
" in loose piles	96
" Hornblendic	175
Gold, cast, pure	1204
Granite	170
Gravel	117 to 125
Greenstone, trap	187
" " quarried, loose	107
Gunpowder	56
Gutta Percha	61
Hemlock, perfectly dry	26
Hickory, " "	48 to 53
Hornblende, black	200 " 220
Ice	57
India rubber	58
Iron, cast	450
" rolled wrought	480
" sheet	485
Isinglass	70
Ivory	114
Lard	59
Lead, commercial cast	712
Lignum Vitæ, perfectly dry	83
Lime, quick	95
" " loose	53 to 59
" " thoroughly shaken	75
Limestone	170
" quarried, loose	96
Loam, soft	110

WEIGHTS OF VARIOUS SUBSTANCES *(Continued).*

NAME OF SUBSTANCE.	Average Weight per cubic foot, lbs.
Locust	46
Magnesia, carbonate	150
Mahogany, Spanish, perfectly dry	53
" Honduras, " "	35
Manganese	499
Maple, perfectly dry	42 to 49
Marble	164
Masonry, granite or limestone	165
" " " " rubble	154
" " " " dry rubble	138
" " " " rough mortar rubble	150
" " " " " dry rubble	125
" of sandstone	145
Mercury at 32° Fah.	849
Mica	183
Mortar, hardened	90 to 100
Mud, wet, moderately pressed	110 " 130
" " fluid	104 " 120
Naphtha	53
Nickel	488 to 549
Oak, live, perfectly dry	59 " 69
" Canadian	54
" white, perfectly dry	48 to 52
" red, black, etc.	32 " 45
" red	52
Oils, whale, olive	57
" of turpentine	54
Peat, dry, unpressed	20 to 30
Petroleum	55
Pewter	453
Pine, Canadian	33
" Northern	34
" pitch	65
" Southern	45 to 48
" white	25 " 28
Pitch	75
Plaster of Paris	142
" " " in irregular lumps	82
" " " ground, loose	56
" " " well shaken	64
Platinum	1342
Plumbago	142
Poplar (white wood)	27
Porphyry	170

PLASTER ON CEILINGS — 5 PER SQ FT.

WEIGHTS OF VARIOUS SUBSTANCES *(Continued).*

NAME OF SUBSTANCE.	Average Weight per cubic foot, lbs.
Pumice Stone	56
Quartz, common, pure	165
" quarried, loose	94
Redwood, California	23
Rosin	68
Salt, solid	134
" coarse	65
" fine table	80
Saltpetre	130
Sand, pure quartz, dry, loose	90 to 106
" perfectly wet	118 " 129
" sharp, of pure quartz, dry	117
Sandstone, building, dry	144 to 151
" quarried and piled	86
Shale, red or black	162
" quarried and piled	92
Silver	655
Slate	160 to 180
Snow, fresh fallen	5 " 12
" solid, saturated with moisture	15 " 50
Soapstone, or Steolite	170
Spruce, perfectly dry	25 to 28
Steel, structural	490
Sulphur	125
Sycamore, perfectly dry	37 to 40
Tallow	59
Tar	63
Terra-cotta	110
" " masonry work	112
Tile	110 to 120
Tin, cast	462
Traprock, quarried and piled	107
" compact	187
Turf, or peat, unpressed	20 to 30
Walnut, black, dry	39
Water, pure or distilled, 32° Fah	62.5
" sea	64.08
Wax, bees'	60.5
Whalebone	81
Willow	34
Wines	62.3
Zinc, or Spelter	438

Green timbers $\frac{1}{5}$ to $\frac{1}{2}$ more than dry.

WEIGHTS OF MERCHANDISE.

Measurements and weights given are for one case, box, cask, crate, barrel, bale, or bag, etc.

MATERIAL.	Measurements. Floor Space Occupied. Sq. Ft.	Measurements. Floor Space Occupied. Cu. Ft.	Weights. Lbs. per Cu. Ft.	Weights. Lbs. per Sq. Ft.
Cassimeres, woolen, in cases	10.5	28.0	20	52
Cement, American, in barrels	3.8	5.5	59	86
" English, in barrels	3.8	5.5	73	105
Cheese	....	....	30	....
Corn, in bags	3.6	3.6	31	31
Cotton, in bales	8.1	44.2	12	64
" extra compressed, in bales	1.25	3.13	40	100
Crockery, in casks	13.4	42.5	14	52
" in crates	9.9	36.6	40	162
Dress goods, woolen, in cases	5.5	22.0	21	84
Flannels, heavy woolen, in cases	7.1	15.2	22	46
Flour, in barrels	4.1	5.4	40	53
Glass, in boxes	....	....	60	....
Hay, in bales	5.0	20.0	14	57
" extra compressed, in bales	1.75	5.25	24	72
Hides, raw, in bales	6.0	30.0	23	117
Leather, sole, in bales	12.6	8.9	16	22
" " in piles	....	....	17	....
Lime, in barrels	3.6	4.5	50	63
Oats, in bags	3.3	3.6	27	29
Oil, lard, in barrels	4.3	12.3	34	98
Paper, manila	...	....	37	....
" newspaper	....	....	38	....
" super-calendered book	....	....	69	....
" wrapping	....	....	10	....
" writing	....	....	64	....
Prints, cotton, in cases	4.5	13.4	31	93
Rags, jute butts, in bales	2.8	11.0	36	143
" woolen, in bales	7.5	30.0	20	80
" white cotton, in bales	9.2	40.0	18	78
" " linen, in bales	8.5	39.5	23	107
Sheetings, bleached cotton, in cases	4.8	11.4	30	69
Starch, in barrels	3.0	10.5	23	83
Straw, extra compressed, in bales	1.75	5.25	19	57
Sugar, brown, in barrels	3.0	7.5	45	113
Tickings, cotton, in bales	3.3	8.8	37	99
Tin, in boxes	2.7	0.5	278	99
Wheat, in bags	4.2	4.2	39	39
" in bulk	....	....	41	....
Wool, Australian, in bales	5.8	26.0	15	66
" Californian, " "	7.5	33.0	17	73
" South American, in bales	7.0	34.0	29	143

WEIGHTS OF FIREPROOFING MATERIALS.

POROUS TERRA COTTA FLOOR ARCHES.

Kind of Arch.	Max. Span between Beams, Feet.	Depth of Arch, Inches.	Weight, lbs. per Sq. Ft.
"Excelsior" End Construction..	5 to 6	8	30
" " " ..	6 to 7	9	32
" " " ...	7 to 8	10	34
" " " ..	8 to 9	12	37
Ordinary Flat Arch	3½ to 4	6	29
" " "	4 to 4½	7	33
" " "	4½ to 5	8	37
" " "	5½ to 6	9	40
" " "	6 to 6½	10	43
" " "	6½ to 7	12	48
Segmental Arch (Hollow Brick).	3 to 8	4	20
" " " " .	5 to 10	6	30
" " " " .	6 to 12	8	37

PARTITIONS, FURRING, CEILING, ROOFING.

	Thickness, Inches.	Weight, lbs. per Sq. Ft.
Hollow Brick Partitions	3	15
" " "	4	20
" " "	5	24
" " "	6	28
Porous Terra Cotta Partitions ..	3	14
" " " " ..	4	18
" " " " ..	5	23
" " " " ..	6	27
Hollow Brick Furring.........	2	12
Porous Terra Cotta Furring....	2	8
" " " Ceiling	2	12
" " " "	3	15
" " " "	4	20
Porous Terra Cotta Roofing....	2	12
" " " "	3	16
" " " "	4	20

NOTES ON MENSURATION.

Triangle ..... Area = $\frac{1}{2}$ base × altitude.
= $\frac{1}{2}$ product of two adjacent sides × sine of the included angle.

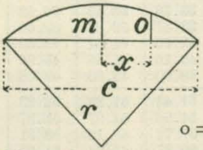
Parallelogram. Area = base × altitude.
= product of two adjacent sides × sine of the included angle.

Trapezoid ... Area = $\frac{1}{2}$ sum of parallel sides × altitude.

Trapezium ... Area = product of diagonals × sine included angle.
= sum of areas of composing triangles.

Circle .... Circumference = 3.14159 × diameter.
Diameter = 0.31831 × circumference.
Area = 3.14159 × square of radius.
= 0.78540 × square of diameter.
Length of an arc = No. of degrees × diameter × 0.0087267.
Area of sector = length of arc × half radius.

Circular Arc



$$m = r - \sqrt{r^2 - \frac{c^2}{4}}$$

$$r = \frac{4m^2 + c^2}{8m}$$

$$o = \sqrt{r^2 - x^2} - (r - m)$$

Ellipse .... Circumference (approximately) = 1.82 × long diameter + 1.32 × short diameter.
Area = 3.14159 × product of the semi-axes.

Parabola ..... Area = $\frac{2}{3}$ base × altitude.

Prism, right or oblique. Convex surface = perimeter of right section × length of lateral edge.
Contents = area of base × perpendicular height.

Cylinder, right or oblique. Convex surface = perimeter of right section × length.
Contents = area of base × perpendicular height.

Pyramid and Cone. Convex surface (right pyramid or cone) = $\frac{1}{2}$ perimeter of base × slant height.
Contents (right or oblique pyramid or cone) = $\frac{1}{3}$ area of base × perpendicular height.

Frustum of Pyramid and Cone. Convex surface (right frustum) = sum of perimeters of bases × $\frac{1}{2}$ slant height.
Contents (right or oblique frustum) = $\frac{1}{3}$ altitude × sum of upper base, lower base and a mean proportional,

$$= \tfrac{1}{3}\text{ alt.}\left(B + B' + \sqrt{BB'}\right)$$

Sphere .... Surface = 3.14159 × square of diameter.
Contents = 0.52360 × cube of diameter.

Prismoid ..... A prismoid is a solid bounded by six plane surfaces, only two of which are parallel. To find the contents; add the areas of the two parallel surfaces and four times the area of a section midway between and parallel to them and multiply the sum by one sixth the altitude.

CIRCUMFERENCES OF CIRCLES.

Advancing by Eighths.

Diameter.	0	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$
0	.0	.3927	.7854	1.178	1.571	1.963	2.356	2.749
1	3.142	3.534	3.927	4.320	4.712	5.105	5.498	5.890
2	6.283	6.676	7.069	7.461	7.854	8.246	8.639	9.032
3	9.425	9.817	10.21	10.60	10.99	11.39	11.78	12.17
4	12.56	12.96	13.35	13.74	14.13	14.53	14.92	15.31
5	15.71	16.10	16.49	16.88	17.28	17.67	18.06	18.45
6	18.85	19.24	19.63	20.02	20.42	20.81	21.20	21.60
7	21.99	22.38	22.77	23.17	23.56	23.95	24.34	24.74
8	25.13	25.52	25.92	26.31	26.70	27.09	27.49	27.88
9	28.27	28.66	29.06	29.45	29.84	30.23	30.63	31.02
10	31.41	31.81	32.20	32.59	32.98	33.38	33.77	34.16
11	34.55	34.95	35.34	35.73	36.13	36.52	36.91	37.30
12	37.70	38.09	38.48	38.87	39.27	39.66	40.05	40.45
13	40.84	41.23	41.62	42.02	42.41	42.80	43.19	43.59
14	43.98	44.37	44.76	45.16	45.55	45.94	46.34	46.73
15	47.12	47.51	47.91	48.30	48.69	49.08	49.48	49.87
16	50.26	50.66	51.05	51.44	51.83	52.23	52.62	53.01
17	53.40	53.80	54.19	54.58	54.97	55.37	55.76	56.15
18	56.55	56.94	57.33	57.72	58.12	58.51	58.90	59.29
19	59.69	60.08	60.47	60.87	61.26	61.65	62.04	62.43
20	62.83	63.22	63.61	64.01	64.40	64.79	65.19	65.58
21	65.97	66.36	66.76	67.15	67.54	67.93	68.33	68.72
22	69.11	69.50	69.90	70.29	70.68	71.08	71.47	71.86
23	72.25	72.65	73.04	73.43	73.82	74.22	74.61	75.00
24	75.40	75.79	76.18	76.57	76.97	77.36	77.75	78.14
25	78.54	78.93	79.32	79.71	80.11	80.50	80.89	81.29
26	81.68	82.07	82.46	82.86	83.25	83.64	84.03	84.43
27	84.82	85.21	85.60	86.00	86.39	86.78	87.18	87.57
28	87.96	88.35	88.75	89.14	89.53	89.93	90.32	90.71
29	91.10	91.50	91.89	92.28	92.67	93.07	93.46	93.85
30	94.24	94.64	95.03	95.42	95.82	96.21	96.60	96.99
31	97.39	97.78	98.17	98.57	98.96	99.35	99.75	100.14
32	100.53	100.92	101.32	101.71	102.10	102.49	102.89	103.28
33	103.67	104.07	104.46	104.85	105.24	105.64	106.03	106 42
34	106.81	107.21	107.60	107.99	108.39	108.78	109.17	109.56
35	109.96	110.35	110.74	111.13	111.53	111.92	112.31	112.71
36	113.10	113.49	113.88	114.28	114.67	115.06	115.45	115.85
37	116.24	116.63	117.02	117.42	117.81	118.20	118.60	118.99
38	119.38	119.77	120.17	120.56	120.95	121.34	121.74	122.13
39	122.52	122.92	123.31	123.70	124.09	124.49	124.88	125.27
40	125.66	126.06	126.45	126.84	127.24	127.63	128.02	128.41
41	128.81	129.20	129.59	129.98	130.38	130.77	131.16	131.55
42	131.95	132.34	132.73	133.13	133.52	133.91	134.30	134.70
43	135.09	135.48	135.87	136.27	136.66	137.05	137.45	137.84
44	138.23	138.62	139.02	139.41	139.80	140.19	140.59	140.98
45	141.37	141.76	142.16	142.55	142.94	143.34	143.73	144.12
46	144.51	144.91	145.30	145.69	146.08	146.48	146.87	147.26
47	147.66	148.05	148.44	148.83	149.23	149.62	150.01	150.40
48	150.80	151.19	151.58	151.97	152.37	152.76	153.15	153.55
49	153.94	154.33	154.72	155.12	155.51	155.90	156.29	156.69

CIRCUMFERENCES OF CIRCLES (Continued).

Advancing by Eighths.

Diameter.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
50	157.08	157.47	157.87	158.26	158.65	159.04	159.44	159.83
51	160.22	160.61	161.01	161.40	161.79	162.19	162.58	162.97
52	163.36	163.76	164.15	164.54	164.93	165.33	165.72	166.11
53	166.50	166.90	167.29	167.68	168.08	168.47	168.86	169.25
54	169.65	170.04	170.43	170.82	171.22	171.61	172.00	172.40
55	172.79	173.18	173.57	173.97	174.36	174.75	175.14	175.54
56	175.93	176.32	176.72	177.11	177.50	177.89	178.29	178.68
57	179.07	179.46	179.86	180.25	180.64	181.03	181.43	181.82
58	182.21	182.61	183.00	183.39	183.78	184.18	184.57	184.96
59	185.35	185.75	186.14	186.53	186.93	187.32	187.71	188.10
60	188.50	188.89	189.28	189.67	190.07	190.46	190.85	191.24
61	191.64	192.03	192.42	192.82	193.21	193.60	193.99	194.39
62	194.78	195.17	195.56	195.96	196.35	196.74	197.14	197.53
63	197.92	198.31	198.71	199.10	199.49	199.88	200.28	200.67
64	201.06	201.46	201.85	202.24	202.63	203.03	203.42	203.81
65	204.20	204.60	204.99	205.38	205.77	206.17	206.56	206.95
66	207.35	207.74	208.13	208.52	208.92	209.31	209.70	210.09
67	210.49	210.88	211.27	211.67	212.06	212.45	212.84	213.24
68	213.63	214.02	214.41	214.81	215.20	215.59	215.98	216.38
69	216.77	217.16	217.56	217.95	218.34	218.73	219.13	219.52
70	219.91	220.30	220.70	221.09	221.48	221.88	222.27	222.66
71	223.05	223.45	223.84	224.23	224.62	225.02	225.41	225.80
72	226.20	226.59	226.98	227.37	227.77	228.16	228.55	228.94
73	229.34	229.73	230.12	230.51	230.91	231.30	231.69	232.09
74	232.48	232.87	233.26	233.66	234.05	234.44	234.83	235.23
75	235.62	236.01	236.41	236.80	237.19	237.58	237.98	238.37
76	238.76	239.15	239.55	239.94	240.33	240.73	241.12	241.51
77	241.90	242.30	242.69	243.08	243.47	243.87	244.26	244.65
78	245.04	245.44	245.83	246.22	246.62	247.01	247.40	247.79
79	248.19	248.58	248.97	249.36	249.76	250.15	250.54	250.94
80	251.33	251.72	252.11	252.51	252.90	253.29	253.68	254.08
81	254.47	254.86	255.25	255.65	256.04	256.43	256.83	257.22
82	257.61	258.00	258.40	258.79	259.18	259.57	259.97	260.36
83	260.75	261.15	261.54	261.93	262.32	262.72	263.11	263.50
84	263.89	264.29	264.68	265.07	265.47	265.86	266.25	266.64
85	267.04	267.43	267.82	268.22	268.61	269.00	269.39	269.78
86	270.18	270.57	270.96	271.36	271.75	272.14	272.53	272.93
87	273.32	273.71	274.10	274.50	274.89	275.28	275.68	276.07
88	276.46	276.85	277.25	277.64	278.03	278.42	278.82	279.21
89	279.60	279.99	280.39	280.78	281.17	281.57	281.96	282.35
90	282.74	283.14	283.53	283.92	284.31	284.71	285.10	285.49
91	285.89	286.28	286.67	287.06	287.46	287.85	288.24	288.63
92	289.03	289.42	289.81	290.21	290.60	290.99	291.38	291.78
93	292.17	292.56	292.95	293.35	293.74	294.13	294.52	294.92
94	295.31	295.70	296.10	296.49	296.88	297.27	297.67	298.06
95	298.45	298.84	299.24	299.63	300.02	300.42	300.81	301.20
96	301.59	301.99	302.38	302.77	303.16	303.56	303.95	304.34
97	304.73	305.13	305.52	305.91	306.31	306.70	307.09	307.48
98	307.88	308.27	308.66	309.05	309.45	309.84	310.23	310.63
99	311.02	311.41	311.80	312.20	312.59	312.98	313.37	313.77

AREAS OF CIRCLES.

Advancing by Eighths.

Diameter.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
0	.0	.0122	.0491	.1104	.1963	.3068	.4418	.6013
1	.7854	.9940	1.227	1.485	1.767	2.074	2.405	2.761
2	3.1416	3.546	3.976	4.430	4.908	5.411	5.939	6.492
3	7.068	7.670	8.296	8.946	9.621	10.32	11.04	11.79
4	12.56	13.36	14.18	15.03	15.90	16.80	17.72	18.66
5	19.63	20.63	21.65	22.69	23.76	24.85	25.96	27.10
6	28.27	29.46	30.68	31.92	33.18	34.47	35.78	37.12
7	38.48	39.87	41.28	42.72	44.18	45.66	47.17	48.70
8	50.26	51.85	53.45	55.09	56.74	58.42	60.13	61.86
9	63.61	65.39	67.20	69.03	70.88	72.76	74.66	76.59
10	78.54	80.51	82.51	84.54	86.59	88.66	90.76	92.88
11	95.03	97.20	99.40	101.6	103.9	106.1	108.4	110.7
12	113.1	115.5	117.9	120.3	122.7	125.2	127.7	130.2
13	132.7	135.3	137.9	140.5	143.1	145.8	148.5	151.2
14	153.9	156.7	159.5	162.3	165.1	168.0	170.9	173.8
15	176.7	179.7	182.7	185.7	188.7	191.7	194.8	197.9
16	201.1	204.2	207.4	210.6	213.8	217.1	220.3	223.6
17	227.0	230.3	233.7	237.1	240.5	244.0	247.4	250.9
18	254.5	258.0	261.6	265.2	268.8	272.4	276.1	279.8
19	283.5	287.3	291.0	294.8	298.6	302.5	306.3	310.2
20	314.2	318.1	322.1	326.0	330.1	334.1	338.2	342.2
21	346.4	350.5	354.7	358.8	363.0	367.3	371.5	375.8
22	380.1	384.5	388.8	393.2	397.6	402.0	406.5	411.0
23	415.5	420.0	424.6	429.1	433.7	438.4	443.0	447.7
24	452.4	457.1	461.9	466.6	471.4	476.3	481.1	486.0
25	490.9	495.8	500.7	505.7	510.7	515.7	520.8	525.8
26	530.9	536.0	541.2	546.3	551.6	556.8	562.0	567.3
27	572.6	577.9	583.2	588.6	594.0	599.4	604.8	610.3
28	615.7	621.3	626.8	632.4	637.9	643.5	649.2	654.8
29	660.5	666.2	672.0	677.7	683.5	689.3	695.1	701.0
30	706.9	712.8	718.7	724.6	730.6	736.6	742.6	748.7
31	754.8	760.9	767.0	773.1	779.3	785.5	791.7	798.0
32	804.3	810.5	816.9	823.2	829.6	836.0	842.4	848.8
33	855.3	861.8	868.3	874.9	881.4	888.0	894.6	901.3
34	907.9	914.6	921.3	928.1	934.8	941.6	948.4	955.2
35	962.1	969.0	975.9	982.8	989.8	996.8	1003.8	1010.8
36	1017.9	1025.0	1032.1	1039.2	1046.3	1053.5	1060.7	1068.0
37	1075.2	1082.5	1089.8	1097.1	1104.5	1111.8	1119.2	1126.7
38	1134.1	1141.6	1149.1	1156.6	1164.2	1171.7	1179.3	1186.9
39	1194.6	1202.3	1210.0	1217.7	1225.4	1233.2	1241.0	1248.8
40	1256.6	1264.5	1272.4	1280.3	1288.2	1296.2	1304.2	1312.2
41	1320.3	1328.3	1336.4	1344.5	1352.7	1360.8	1369.0	1377.2
42	1385.4	1393.7	1402.0	1410.3	1418.6	1427.0	1435.4	1443.8
43	1452.2	1460.7	1469.1	1477.6	1486.2	1494.7	1503.3	1511.9
44	1520.5	1529.2	1537.9	1546.6	1555.3	1564.0	1572.8	1581.6
45	1590.4	1599.3	1608.2	1617.0	1626.0	1634.9	1643.9	1652.9
46	1661.9	1670.9	1680.0	1689.1	1698.2	1707.4	1716.5	1725.7
47	1734.9	1744.2	1753.5	1762.7	1772.1	1781.4	1790.8	1800.1
48	1809.6	1819.0	1828.5	1837.9	1847.5	1857.0	1866.5	1876.1
49	1885.7	1895.4	1905.0	1914.7	1924.4	1934.2	1943.9	1953.7

AREAS OF CIRCLES *(Continued).*

Advancing by Eighths.

Diameter.	0	1/8	1/4	3/8	1/2	5/8	3/4	7/8
50	1963.5	1973.3	1983.2	1993.1	2003.0	2012.9	2022.8	2032.8
51	2042.8	2052.8	2062.9	2073.0	2083.1	2093.2	2103.3	2113.5
52	2123.7	2133.9	2144.2	2154.5	2164.8	2175.1	2185.4	2195.8
53	2206.2	2216.6	2227.0	2237.5	2248.0	2258.5	2269.1	2279.6
54	2290.2	2300.8	2311.5	2322.1	2332.8	2343.5	2354.3	2365.0
55	2375.8	2386.6	2397.5	2408.3	2419.2	2430.1	2441.1	2452.0
56	2463.0	2474.0	2485.0	2496.1	2507.2	2518.3	2529.4	2540.6
57	2551.8	2563.0	2574.2	2585.4	2596.7	2608.0	2619.4	2630.7
58	2642.1	2653.5	2664.9	2676.4	2687.8	2699.3	2710.9	2722.4
59	2734.0	2745.6	2757.2	2768.8	2780.5	2792.2	2803.9	2815.7
60	2827.4	2839.2	2851.0	2862.9	2874.8	2886.6	2898.6	2910.5
61	2922.5	2934.5	2946.5	2958.5	2970.6	2982.7	2994.8	3006.9
62	3019.1	3031.3	3043.5	3055.7	3068.0	3080.3	3092.6	3104.9
63	3117.2	3129.6	3142.0	3154.5	3166.9	3179.4	3191.9	3204.4
64	3217.0	3229.6	3242.2	3254.8	3267.5	3280.1	3292.8	3305.6
65	3318.3	3331.1	3343.9	3356.7	3369.6	3382.4	3395.3	3408.2
66	3421.2	3434.3	3447.2	3460.2	3473.2	3486.3	3499.4	3512.5
67	3525.7	3538.8	3552.0	3565.2	3578.5	3591.7	3605.0	3618.3
68	3631.7	3645.0	3658.4	3671.8	3685.3	3698.7	3712.2	3725.7
69	3739.3	3752.8	3766.4	3780.0	3793.7	3807.3	3821.0	3834.7
70	3848.5	3862.2	3876.0	3889.8	3903.6	3917.5	3931.4	3945.3
71	3959.2	3973.1	3987.1	4001.1	4015.2	4029.2	4043.3	4057.4
72	4071.5	4085.7	4099.8	4114.0	4128.2	4142.5	4156.8	4171.1
73	4185.4	4199.7	4214.1	4228.5	4242.9	4257.4	4271.8	4286.3
74	4300.8	4315.4	4329.9	4344.5	4359.2	4373.8	4388.5	4403.1
75	4417.9	4432.6	4447.4	4462.2	4477.0	4491.8	4506.7	4521.5
76	4536.5	4551.4	4566.4	4581.3	4596.3	4611.4	4626.4	4641.5
77	4656.6	4671.8	4686.9	4702.1	4717.3	4732.5	4747.8	4763.1
78	4778.4	4793.7	4809.0	4824.4	4839.8	4855.2	4870.7	4886.2
79	4901.7	4917.2	4932.7	4948.3	4963.9	4979.5	4995.2	5010.9
80	5026.5	5042.3	5058.0	5073.8	5089.6	5105.4	5121.2	5137.1
81	5153.0	5168.9	5184.9	5200.8	5216.8	5232.8	5248.9	5264.9
82	5281.0	5297.1	5313.3	5329.4	5345.6	5361.8	5378.1	5394.3
83	5410.6	5426.9	5443.3	5459.6	5476.0	5492.4	5508.8	5525.3
84	5541.8	5558.3	5574.8	5591.4	5607.9	5624.5	5641.2	5657.8
85	5674.5	5691.2	5707.9	5724.7	5741.5	5758.3	5775.1	5791.9
86	5808.8	5825.7	5842.6	5859.6	5876.5	5893.5	5910.6	5927.6
87	5944.7	5961.8	5978.9	5996.0	6013.2	6030.4	6047.6	6064.9
88	6082.1	6099.4	6116.7	6134.1	6151.4	6168.8	6186.2	6203.7
89	6221.1	6238.6	6256.1	6273.7	6291.2	6308.8	6326.4	6344.1
90	6361.7	6379.4	6397 1	6414.9	6432.6	6450.4	6468.2	6486.0
91	6503.9	6521.8	6539.7	6557.6	6575.5	6593.5	6611.5	6629.6
92	6647.6	6665.7	6683.8	6701.9	6720.1	6738.2	6756.4	6774.7
93	6792.9	6811.2	6829.5	6847.8	6866.1	6884.5	6902.9	6921.3
94	6939.8	6958.2	6976.7	6995.3	7013.8	7032.4	7051.0	7069.6
95	7088.2	7106.9	7125.6	7144.3	7163.0	7181.8	7200.6	7219.4
96	7238.2	7257.1	7276.0	7294.9	7313.8	7332.8	7351.8	7370.8
97	7389.8	7408.9	7428.0	7447.1	7466.2	7485.3	7504.5	7523.7
98	7543.0	7562.2	7581.5	7600.8	7620.1	7639.5	7658.9	7678.3
99	7697.7	7717.1	7736.6	7756.1	7775.6	7795.2	7814.8	7834.4

LONG MEASURE.

Inches.	Feet.	Yards.	Fath.	Poles.	Furl.	Mile.	Metres.
1.	= .083 =	.02778 =	.0139 =	.005 =	.000126 =	.0000158 =	.0254
12.	1.	.333	.1667	.0606	.00151	.0001894	.3048
36.	3.	1.	.5	.182	.00454	.000568	.9144
72.	6.	2.	1.	.364	.0091	.001136	1.8288
198.	16½.	5½.	2¾.	1.	.025	.003125	5.0292
7920.	660.	220.	110.	40.	1.	.125	201.168
63360.	5280.	1760.	880.	320.	8.	1.	1609.344

A palm = 3 inches.
A span = 9 inches.
A hand = 4 inches.
A cable's length = 120 fathoms.

SQUARE MEASURE.

Inches.	Feet.	Yards.	Perches.	Roods.	Acre.	Metres.
1.	= .00694 =	.000772 =	.0000255 =	.00000064 =	.000000159 =	.000645
144.	1.	.111	.00367	.0000918	.000023	.0929
1296.	9.	1.	.0331	.000826	.0002066	.8362
39204.	272¼.	30¼.	1.	.025	.00625	25.294
1568160.	10890.	1210.	40.	1.	.25	1011.78
6272640.	43560.	4840.	160.	4.	1.	4047.11

100 square feet = 1 square.
10 square chains = 1 acre.
1 chain wide = 8 acres per mile.
1 hectare = 2.471044 acres.

$$1 \text{ square mile} \begin{cases} = 27878400 \text{ square feet.} \\ = 3097600 \text{ square yards.} \\ = 640 \text{ acres.} \end{cases}$$

Acres × .0015625 = square miles.
Square yards × .000000323 = square miles.
Acres × 4840 = square yards.
Square yards × .0002066 = acres.
A section of land is 1 mile square, and contains 640 acres.
A square acre is 208.71 ft. at each side; or 220 × 198 ft.
A square ½-acre is 147.58 ft. at each side; or 110 × 198 ft.
A square ¼-acre is 104.355 ft. at each side; or 55 × 198 ft.
A circular acre is 235.504 feet in diameter.
A circular ½-acre is 166.527 feet in diameter.
A circular ¼-acre is 117.752 feet in diameter.

CUBIC MEASURE.

Inches.		Feet.		Yard.		Metres.
1.	=	.0005788	=	.000002144	=	.000016387
1728.		1.		.03704		.028317
46656.		27.		1.		.764552

A cord of wood = 128 cubic feet, being four feet high, four feet wide, and eight feet long.

Forty-two cubic feet = a ton of shipping, British.

Forty cubic feet = a ton of shipping, U. S.

A perch of masonry contains $24\frac{3}{4}$ cubic feet.

A CUBIC FOOT IS EQUAL TO

1728 cubic inches.
.037037 cubic yard.
.803564 U. S. struck bushel of 2150.42 cubic inches.
3.21426 U. S. pecks.
7.48052 U. S. liquid galls. of 231 cubic inches.
6.42851 U. S. dry galls.
29.92208 U. S. liquid quarts.
25.71405 U. S. dry quarts.
59.84416 U. S. liquid pints.
51.42809 U. S. dry pints.
239.37662 U. S. gills.
.26667 flour barrel of 3 struck bushels.
.23748 U. S. liquid barrel of $31\frac{1}{2}$ galls.

MEASURES OF CAPACITY.

LIQUID MEASURE.

Gill.	Pint.	Quart.	Gallon.	Cubic Inches.	Cubic Metres.
1	.25	.125	.03125	7.21875	.000118
4	1.	.5	.125	28.875	.000473
8	2.	1.	.25	57.75	.000947
32	8.	4.	1.	231.	.003786

DRY MEASURE.

Pint.	Quart.	Peck.	Bushel.	Cubic Inches.	Cubic Metres.
1	.50	.0625	.015625	33.6003	.000551
2	1.	.125	.03125	67.2006	.001101
16	8.	1.	.25	537.605	.008811
64	32.	4.	1.00	2150.42	.035245

AVOIRDUPOIS WEIGHT.

The standard avoirdupois pound is the weight of 27.7015 cubic inches of distilled water, weighed in the air, at 39.83 degrees Fahr., barometer at thirty inches.

27.343 grains = 1 drachm.

Drachms.	Ounces.	Lbs.	Qrs.	Cwts.	Ton.	Grammes.
1. =	.0625 =	.0039 =	.000139 =	.000035 =	.00000174 =	1.77189
16.	1.	.0625	.00223	.000558	.000028	28.3502
256.	16.	1.	.0357	.00893	.000447	453.603
7168.	448.	28.	1.	.25	.0125	12700.884
28672.	1792.	112.	4.	1.	.05	50803.536
573440.	35840.	2240.	80.	20.	1.	1016070.72

A stone = 14 pounds.

A quintal = 100 pounds.

7000 grains = one avoirdupois pound = 1.21528 troy pounds.

5760 grains = one troy pound = .82285 avoirdupois pounds.

SURVEYING MEASURE (LINEAL).

Inches.	Links.	Feet.	Yards.	Chains.	Mile.	Metres.
1. =	.126 =	.0833 =	.0278 =	.00126 =	.0000158 =	.0254
7.92	1.	.66	.22	.01	.000125	.2012
12.	1.515	1.	.333	.01515	.000189	.3048
36.	4.545	3.	1.	.04545	.000568	.9144
792.	100.	66.	22.	1.	.0125	20.1168
63360.	8000.	5280.	1760.	80.	1.	1609.344

One knot or geographical mile = 6086.07 feet = 1855.11 metres = 1.1526 statute miles.

One admiralty knot = 1.1515 statute miles = 6080 feet.

CONVERSION TABLE.

METRIC SYSTEM TO U. S. WEIGHTS AND MEASURES.

Millimetres × 0.03937 = inches.
Centimetres × 0.3937 = "
Metres × 39.37 = " (Act Congress.)
Metres × 3.2809 = feet.
Metres × 1.0936 = yards.
Kilometres × 0.6214 = miles.
Kilometres × 3280.9 = feet.
Square Millimetres × 0.00155 = sq. ins.
Square Centimetres × 0.155 = "
Square Metres × 10.7641 = sq. ft.
Square Kilometres × 247.10 = acres.
Hectare × 2.47104 = "
Cubic Centimetres × 0.0610 = cu. ins.
Cubic " × 0.2704 = fl. drachms. (U. S. P.)
Cubic " × 0.0338 = fl. ounces. (U. S. P.)
Cubic Metres × 35.3155 = cu. ft.
Cubic " × 1.3080 = cu. yards.
Cubic " × 264.1785 = gallons. (231 cu. ins.)
Litres × 61.025 = cu. in. (Act Congress.)
Litres × 33.8006 = fl. ounces. (U. S. P.)
Litres × 0.2642 = gallons. (231 cu. ins.)
Litres × 0.0353 = cu. ft.
Hectolitres × 3.5315 = "
Hectolitres × 2.8378 = bushels. (2150.42 cu. ins.)
Hectolitres × 0.1308 = cu. yards.
Hectolitres × 26.42 = gallons. (231 cu. ins.)
Grammes × 15.432 = grains. (Act Cong.)
Grammes (water) × 0.03381 = fl. ounces.
Grammes × 0.03527 = ozs. avoirdupois.
Grammes per cu. cent. × 0.0361 = lbs. per cu. in.
Kilogrammes × 2.2046 = pounds.
Kilogrammes × 35.2736 = ozs. avoirdupois.
Kilogrammes × 0.0011023 = tons. (2000 lbs.)
Kilogrammes per sq. cent. × 14.223 = lbs. per sq. in.
Kilogram-metres × 7.2331 = ft. lbs.
Kilogram per metre × 0.6720 = lbs. per ft.
Kilogram per cu. metre × 0.0624 = lbs. per cu. ft.
Kilo per cheval × 2.235 = lbs. per H. P.
Kilowatts × 1.34 = H. P.
Calorie × 3.968 = B. T. U.
Cheval vapeur × .9863 = H. P.
1° Centigrade = 1°.8 Fahrenheit.
(Degrees, Cent. Therm. × 1.8) + 32 = degrees, Fahr. Therm.

NATURAL SINES, ETC.

Deg.	Sine.	Cover.	Cosecnt.	Tangt.	Cotang.	Secant.	Versin.	Cosine.	Deg.
0	.00	1.00000	Infinite.	.0	Infinite.	1.00000	.0	1.00000	90
1	.01745	.98254	57.2986	.01745	57.2899	1.00015	.0001	.99984	89
2	.03489	.96510	28.6537	.03492	28.6362	1.00060	.0006	.99939	88
3	.05233	.94766	19.1073	.05240	19.0811	1.00137	.0013	.99862	87
4	.06975	.93024	14.3355	.06992	14.3006	1.00244	.0024	.99756	86
5	.08715	.91284	11.4737	.08748	11.4300	1.00381	.0038	.99619	85
6	.10452	.89547	9.5667	.10510	9.5143	1.00550	.0054	.99452	84
7	.12186	.87813	8.2055	.12278	8.1443	1.00750	.0074	.99254	83
8	.13917	.86082	7.1852	.14054	7.1153	1.00982	.0097	.99026	82
9	.15643	.84356	6.3924	.15838	6.3137	1.01246	.0123	.98768	81
10	.17364	.82635	5.7587	.17632	5.6712	1.01542	.0151	.98480	80
11	.19080	.80919	5.2408	.19438	5.1445	1.01871	.0183	.98162	79
12	.20791	.79208	4.8097	.21255	4.7046	1.02234	.0218	.97814	78
13	.22495	.77504	4.4454	.23086	4.3314	1.02630	.0256	.97437	77
14	.24192	.75807	4.1335	.24932	4.0107	1.03061	.0297	.97029	76
15	.25881	.74118	3.8637	.26794	3.7320	1.03527	.0340	.96592	75
16	.27563	.72436	3.6279	.28674	3.4874	1.04029	.0387	.96126	74
17	.29237	.70762	3.4203	.30573	3.2708	1.04569	.0436	.95630	73
18	.30901	.69098	3.2360	.32491	3.0776	1.05146	.0489	.95105	72
19	.32556	.67443	3.0715	.34432	2.9042	1.05762	.0544	.94551	71
20	.34202	.65797	2.9238	.36397	2.7474	1.06417	.0603	.93969	70
21	.35836	.64163	2.7904	.38386	2.6050	1.07114	.0664	.93358	69
22	.37460	.62539	2.6694	.40402	2.4750	1.07853	.0728	.92718	68
23	.39073	.60926	2.5593	.42447	2.3558	1.08636	.0794	.92050	67
24	.40673	.59326	2.4585	.44522	2.2460	1.09463	.0864	.91354	66
25	.42261	.57738	2.3662	.46630	2.1445	1.10337	.0936	.90630	65
26	.43837	.56162	2.2811	.48773	2.0503	1.11260	.1012	.89879	64
27	.45399	.54600	2.2026	.50952	1.9626	1.12232	.1089	.89100	63
28	.46947	.53052	2.1300	.53170	1.8807	1.13257	.1170	.88294	62
29	.48480	.51519	2.0626	.55430	1.8040	1.14335	.1253	.87461	61
30	.50000	.50000	2.0000	.57735	1.7320	1.15470	.1339	.86602	60
31	.51503	.48496	1.9416	.60086	1.6642	1.16663	.1428	.85716	59
32	.52991	.47008	1.8870	.62486	1.6003	1.17917	.1519	.84804	58
33	.54463	.45536	1.8360	.64940	1.5398	1.19236	.1613	.83867	57
34	.55919	.44080	1.7882	.67450	1.4825	1.20621	.1709	.82903	56
35	.57357	.42642	1.7434	.70020	1.4281	1.22077	.1808	.81915	55
36	.58778	.41221	1.7013	.72654	1.3763	1.23606	.1909	.80901	54
37	.60181	.39818	1.6616	.75355	1.3270	1.25213	.2013	.79863	53
38	.61566	.38433	1.6242	.78128	1.2799	1.26901	.2119	.78801	52
39	.62932	.37067	1.5890	.80978	1.2348	1.28675	.2228	.77714	51
40	.64278	.35721	1.5557	.83909	1.1917	1.30540	.2339	.76604	50
41	.65605	.34394	1.5242	.86928	1.1503	1.32501	.2452	.75470	49
42	.66913	.33086	1.4944	.90040	1.1106	1.34563	.2568	.74314	48
43	.68199	.31800	1.4662	.93251	1.0723	1.36732	.2686	.73135	47
44	.69465	.30534	1.4395	.96568	1.0355	1.39016	.2806	.71933	46
45	.70710	.29289	1.4142	1.00000	1.0000	1.41421	.2928	.70710	45
	Cosine.	Versin.	Secant.	Cotang.	Tangt.	Cosecant.	Cover.	Sine.	

DECIMALS OF AN INCH FOR EACH $\frac{1}{64}$TH.

$\frac{1}{32}$ds.	$\frac{1}{64}$ths.	Decimal.	Fraction.	$\frac{1}{32}$ds.	$\frac{1}{64}$ths.	Decimal.	Fraction.
	1	.015625			33	.515625	
1	2	.03125		17	34	.53125	
	3	.046875			35	.546875	
2	4	.0625	1–16	18	36	.5625	9–16
	5	.078125			37	.578125	
3	6	.09375		19	38	.59375	
	7	.109375			39	.609375	
4	8	.125	1–8	20	40	.625	5–8
	9	.140625			41	.640625	
5	10	.15625		21	42	.65625	
	11	.171875			43	.671875	
6	12	.1875	3–16	22	44	.6875	11–16
	13	.203125			45	.703125	
7	14	.21875		23	46	.71875	
	15	.234375			47	.734375	
8	16	.25	1–4	24	48	.75	3–4
	17	.265625			49	.765625	
9	18	.28125		25	50	.78125	
	19	.296875			51	.796875	
10	20	.3125	5–16	26	52	.8125	13–16
	21	.328125			53	.828125	
11	22	.34375		27	54	.84375	
	23	.359375			55	.859375	
12	24	.375	3–8	28	56	.875	7–8
	25	.390625			57	.890625	
13	26	.40625		29	58	.90625	
	27	.421875			59	.921875	
14	28	.4375	7–16	30	60	.9375	15–16
	29	.453125			61	.953125	
15	30	.46875		31	62	.96875	
	31	.484375			63	.984375	
16	32	.5	1–2	32	64	1.	1

DECIMALS OF A FOOT FOR EACH $\frac{1}{32}$ OF AN INCH.

Inches.	16ths.	Decimal.	Inches.	16ths.	Decimal.	Inches.	16ths.	Decimal.	Inches.	16ths.	Decimal.
0	0		1	8	.1250	3	0	.2500	4	8	.3750
		.0026			.1276			.2526			.3776
	1	.0052		9	.1302		1	.2552		9	.3802
		.0078			.1328			.2578			.3828
	2	.0104		10	.1354		2	.2604		10	.3854
		.0130			.1380			.2630			.3880
	3	.0156		11	.1406		3	.2656		11	.3906
		.0182			.1432			.2682			.3932
	4	.0208		12	.1458		4	.2708		12	.3958
		.0234			.1484			.2734			.3984
	5	.0260		13	.1510		5	.2760		13	.4010
		.0286			.1536			.2786			.4036
	6	.0313		14	.1563		6	.2813		14	.4063
		.0339			.1589			.2839			.4089
	7	.0365		15	.1615		7	.2865		15	.4115
		.0391			.1641			.2891			.4141
	8	.0417	2	0	.1667		8	.2917	5	0	.4167
		.0443			.1693			.2943			.4193
	9	.0469		1	.1719		9	.2969		1	.4219
		.0495			.1745			.2995			.4245
	10	.0521		2	.1771		10	.3021		2	.4271
		.0547			.1797			.3047			.4297
	11	.0573		3	.1823		11	.3073		3	.4323
		.0599			.1849			.3099			.4349
	12	.0625		4	.1875		12	.3125		4	.4375
		.0651			.1901			.3151			.4401
	13	.0677		5	.1927		13	.3177		5	.4427
		.0703			.1953			.3203			.4453
	14	.0729		6	.1979		14	.3229		6	.4479
		.0755			.2005			.3255			.4505
	15	.0781		7	2031		15	.3281		7	.4531
		.0807			.2057			.3307			.4557
1	0	.0833		8	.2083	4	0	.3333		8	.4583
		.0859			.2109			.3359			.4609
	1	.0885		9	.2135		1	.3385		9	.4635
		.0911			.2161			.3411			.4661
	2	.0938		10	.2188		2	.3438		10	.4688
		.0964			.2214			.3464			.4714
	3	.0990		11	.2240		3	.3490		11	.4740
		.1016			.2266			.3516			.4766
	4	.1042		12	.2292		4	.3542		12	.4792
		.1068			.2318			.3568			.4818
	5	.1094		13	.2344		5	.3594		13	.4844
		.1120			.2370			.3620			.4870
	6	.1146		14	.2396		6	.3646		14	.4896
		.1172			.2422			.3672			.4922
	7	.1198		15	.2448		7	.3698		15	.4948
		.1224			.2474			.3724			.4974

DECIMALS OF A FOOT FOR EACH $\frac{1}{32}$ OF AN INCH *(Continued).*

Inches.	16ths.	Decimal.	Inches.	16ths.	Decimal.	Inches.	16ths.	Decimal.	Inches.	16ths.	Decimal.
6	0	.5000	**7**	8	.6250	**9**	0	.7500	**10**	8	.8750
		.5026			.6276			.7526			.8776
	1	.5052		9	.6302		1	.7552		9	.8802
		.5078			.6328			.7578			.8828
	2	.5104		10	.6354		2	.7604		10	.8854
		.5130			.6380			.7630			.8880
	3	.5156		11	.6406		3	.7656		11	.8906
		.5182			.6432			.7682			.8932
	4	.5208		12	.6458		4	.7708		12	.8958
		.5234			.6484			.7734			.8984
	5	.5260		13	.6510		5	.7760		13	.9010
		.5286			.6536			.7786			.9036
	6	.5313		14	.6563		6	.7813		14	.9063
		.5339			.6589			.7839			.9089
	7	.5365		15	.6615		7	.7865		15	.9115
		.5391			.6641			.7891			.9141
	8	.5417	**8**	0	.6667		8	.7917	**11**	0	.9167
		.5443			.6693			.7943			.9193
	9	.5469		1	.6719		9	.7969		1	.9219
		.5495			.6745			.7995			.9245
	10	.5521		2	.6771		10	.8021		2	.9271
		.5547			.6797			.8047			.9297
	11	.5573		3	.6823		11	.8073		3	.9323
		.5599			.6849			.8099			.9349
	12	.5625		4	.6875		12	.8125		4	.9375
		.5651			.6901			.8151			.9401
	13	.5677		5	.6927		13	.8177		5	.9427
		.5703			.6953			.8203			.9453
	14	.5729		6	.6979		14	.8229		6	.9479
		.5755			.7005			.8255			.9505
	15	.5781		7	7031		15	.8281		7	.9531
		.5807			.7057			.8307			.9557
7	0	.5833		8	.7083	**10**	0	.8333		8	.9583
		.5859			.7109			.8359			.9609
	1	.5885		9	.7135		1	.8385		9	.9635
		.5911			.7161			.8411			.9661
	2	.5938		10	.7188		2	.8438		10	.9688
		.5964			.7214			.8464			.9714
	3	.5990		11	.7240		3	.8490		11	.9740
		.6016			.7266			.8516			.9766
	4	.6042		12	.7292		4	.8542		12	.9792
		.6068			.7318			.8568			.9818
	5	.6094		13	.7344		5	.8594		13	.9844
		.6120			.7370			.8620			.9870
	6	.6146		14	.7396		6	.8646		14	.9896
		.6172			.7422			.8672			.9922
	7	.6198		15	.7448		7	.8698		15	.9948
		.6224			.7474			.8724			.9974

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