PREFACE

THE extraordinary demand for the 6th Edition of our Section Book, issued only five years ago, has encouraged us to proceed with yet another edition. In doing so, we have taken the opportunity of carefully revising the existing matter, thereby bringing this, the 7th Edition right up to date.

The special features of the 6th Edition are retained, and, in addition, we have included various other useful and very essential data, particularly in respect of Portal Frames. Details and bending moments of these are given, fully developed, for the benefit of those who may not have the time or the Staff to work out the details themselves.

The whole of the calculations have been worked out in our Design Department.

As in our previous book, space has again been a difficulty, and there are many features which would have been included, but unfortunately for this reason had to be omitted.

Welding is now making such rapid strides that we may find it necessary at an early date to revise some of our matter. In such an event, we will not hesitate to issue a supplement to this book.

W. A. Fleming

Member, Institute of Struct. Engineers

December, 1944

CONTENTS

GENERAL NOTES

BEAMS, CHANNELS AND COMPOUND GIRDERS

As an emergency measure pending more detailed consideration of the question, permission has been granted for the revision of the British Standard Specification No. 449, 1937, increasing the working stress (on extreme fibres) of beams under bending from **8 to 10 tons per square inch** (axial stress to remain at 8 tons per square inch).

This revision is applicable during the period of the War, and 6 months thereafter, but it is anticipated that before the expiry of these provisions, the amendments will be confirmed as a permanent alteration to the Standard.

In addition to our Tables of Beams, Channels and Compound Girders calculated for extreme fibre stress of 8 tons per square inch, we have included Tables of Tabulated Loads for Beams, Channels and Single Joist Compound Girders calculated for extreme fibre stress of 10 tons per square inch in conformity with the above revision. Space does not permit us to extend this to double and treble joist compound girders. If, however, the tabulated loads for these Girders be increased by $\frac{1}{4}$, the result would be similar to the girders being stressed to 10 tons per square inch. Care should be taken that if the loads are in excess of the safe web buckling values of the joist, web stiffeners must be provided. It should also be noted that the span of the beam or girder stressed to 10 tons per square inch must not exceed 19·15 times its depth, unless the load is decreased to keep the deflection within 1/325th of the span.

Weights per foot

Weights per foot in lbs. for compound girders include an allowance for rivet heads at 6" pitch. The weights of stiffeners, end angles, etc., require to be added. All weights are calculated per foot run; weight of steel being 489·6 lbs. per cubic ft.

Sections

Beams and channels used are British Standard Sections, as listed in the B.S.I. Publication No. 4, 1932, with the exception of those marked with an asterisk. These are the old British Standard and special sections, which are still rolled by the various mills.

Tabulated Loads

The loads given in the tables are in tons, and are the safe loads which the Simply Supported Beam, Channel or Girder will carry evenly distributed over the entire span, calculated as centres of Bearings. Each load includes the weight of the Beam or Girder. For Point Loads or Loads not evenly distributed over the entire span or for Beams which have Fixed Ends, special calculations have to be made.

In order to stress the Beam or Girder to its full extent, i.e., to 8 or 10 tons/sq. in., it is necessary to ensure that the top or Compression Flange is stayed laterally at a distance not exceeding twenty times the width of this Flange (if the Beam is encased then twenty times the width of the encasement), and the loads given for all the sections are based on this assumption. It is not possible, however, to always get this convenient tying arrangement, and when that is the case then the Stresses in the Beam have to be reduced in certain ratio to the span divided by the flange breadth. These reduced Stresses naturally lower the safe carrying loads, and in the tables of Single Beams and Channels we have shown in italics immediately under the ordinary 8 and 10 Tons Stress Loads, the safe distributed loads which the respective beam or channel would carry if there were no Ties up to fifty times the Flange breadth. (Beyond this limit, ties are essential, and must be put in or else increase the Flange breadth.) These requirements are in accordance with the BSS.449, 1940 revised, and on pages 338-341 we have given Tables of the reduced Stresses from which these additional Safe Loads are calculated.

Rivet Pitch

Compound girders require rivets at 6" pitch on line, staggered, unless for loads printed in Bold and Italic Type. These require a closer pitch and need further calculation. The diameter of rivets is given for each girder.

Moments of Inertia

The tabulated safe loads of each Channel, Joist and Compound Girder are calculated relative to the maximum moment of inertia about axis XX, passing through the centre of gravity of the section and parallel to the Flanges. These moments of inertia are calculated on the gross sectional area of the section for the Single Beam or Channel and on the nett sectional area of the section for Compound Girders, a deduction for one Rivet Hole being made in each Flange of Joist of Channel.

Moduli of Section

For each Joist, Channel and Symmetrical Compound Girder, the maximum modulus of section is tabulated about axis XX, corresponding to the maximum moment of inertia and derived from the Gross Area of each section for Single Beams and Channels and the nett area of the section for Compound Girders. The Section Moduli have been printed in bold type as these are more frequently referred to.

Modulus for 1" Plate Breadth

An additional column is given in the table of Single Joist Compound Girders showing the modulus of section of one inch of Flange plate width. By adding or subtracting these values to or from the modulus of section of the respective Girders, the modulus of a Compound Girder with a broader or narrower Flange Plate than the section listed can be obtained.

Flange Plates

In compound girder tables, the column headed 'Flange Plates' means the total thickness of plates for *each* flange. This thickness can, if necessary, be made up by using separate plates $\frac{3}{8}$" or $\frac{1}{2}$" thick.

Bold Type

Loads printed in Bold Type indicate that the joist or girder must have web stiffeners to prevent web buckling, and *also* that a closer pitch of rivets than 6" is required.

Italic Type

Loads printed in Italic Type indicate only that a closer pitch of rivets than 6" is required.

Zig-Zag Line

The span of a beam (except filler beams in concrete and asymmetrical sections) must not exceed 24 times its depth stressed to 8 tons per square inch and 19·15 times its depth stressed to 10 tons per square inch, unless the calculated deflection of the beam is less than 1/325th of the span (B.S. Requirements). The tabulated loads to the left of the zig-zag lines are for beams with spans not exceeding 24 and 19·15 times their depths. Loads to right of zig-zag lines are for beams with spans exceeding these ratios, but these loads have been decreased so that the stress of the beams is less than 8 and 10 tons per square inch respectively and the deflection kept within 1/325th of the span.

Asymmetrical Sections

Tables are given for beams having one plate on top flange only. This type of girder is often required to support walls over openings when a good width of top flange is necessary to keep the overhang of brickwork to a minimum. A wide flange breadth is also required to comply with the B.S. Requirements for a beam which is not stayed laterally, and which must have a flange width of not less than 1/20th of the span. If the girder is not encased in concrete, a very heavy beam results unless a top flange plate is adopted. In the case of deflection the ratio of span to depth varies, the value being 48 times the maximum distance in inches from the neutral axis to the extreme fibres of the section, when the girder is stressed to 8 tons per square inch. This ratio has been indicated by a zig-zag line, and loads to right of this line have been decreased to keep the deflection within 1/325th of the span.

The neutral axis of this section not being centrally placed between the Flanges, two values of section moduli can be obtained. The one given in our tables is calculated from the bottom flange of the section and, being less than the modulus calculated from the top flange, is termed Minimum.

In order to calculate the strength of the section on the YY Axis due to transverse loading from cranes, the modulus of section is given for the Top Flange only.

Filler Joists

Tables are given for filler joists at various centres embedded in concrete, designed in conformity with B.S.S. No. 449, 1937 and 1940 revision, on the basis of the combined moment of inertia of steel and surrounding concrete, taking the modular ratio of steel to concrete as 15. The maximum bending stress of concrete is based on 750 lbs. per square inch, and the steel beams being stressed to not more than 9 and 11 tons per square inch respectively. Where the above is not applicable the loads are based on the strength of the steel beams alone, the stress being $9+t$ and $11+t$ tons per square inch respectively; t being the thickness of the top cover of concrete in inches and is tabulated as 2 in. and 1 in. for each section of beam; three varying centres of beams, 24 in., 30 in. and 36 in. being given for both cover thicknesses. If the filler joists are spaced further apart than 6 times the concrete floor slab thickness, suitable transverse reinforcement must be provided.

Calculations are based on Beams being simply supported. Tabular loads include the weight of beam and concrete slab.

The maximum span of a filler joist must not exceed $32d$; d being distance from top of concrete to underside of filler beam (B.S. Requirements). This limit has been indicated by a zig-zag line. If, however, a filler beam is continuous over three or more spans, this ratio, we consider, may be increased, providing the Local Authority responsible approves. Tabular loads have been given beyond the zig-zag line for this purpose.

STANCHIONS AND STRUTS

Tabulated Loads

The safe tabulated loads for stanchions, struts, solid round columns and tubular columns are in accordance with the formula given in the B.S.S. No. 449, 1937. A table of working stresses based on this formula will be found on pages 144 to 147. For tubular Cols. the Yield Stress $(P_y) = 16$ Tons/sq. in.

All tabulated loads are calculated on the least radius of gyration of the section in every case, and on the effective height of the stanchion or strut, which is equivalent to a stanchion having pin ends. The loads are assumed to be concentric, i.e. applied directly in line with the centre of the stanchion.

Rivet Pitch

The rivets in all compound stanchions can be taken at 6" pitch, but this should be not less than 16 times the thickness of the thinnest plate (B.S. Requirements). The diameter of rivets is given in the tables for each stanchion.

Rivet Holes

No deduction is made for rivet holes in the radii of gyration or moduli of section of the compound stanchions.

Weights per foot

Each weight per foot in lbs. for compound stanchions includes an allowance for rivet heads at standard pitch. Also weights per foot are for plain or riveted shaft only. Weights of base, cap, connections, lattice bracing, batten plates and extra rivets require to be added in estimating the total weight of a complete stanchion.

Batten Plates

Tables are given for double joists, double channels and double angles battened together at intervals forming stanchions or struts. The spacing of these batten plates should be as follows :

Clear distance between plates should be such, that the slenderness ratio of the battened flange will be less than that of the complete stanchion.

Clear distance between plates shall also be not more than 4 times the length of the batten plate measured along the compression members (i.e. the built-up stanchion). Alternatively, the slenderness ratio of the individual members of the built-up stanchion shall not exceed 40.

Moduli of Section

In the tables of Stanchions, the moduli of section axes XX and YY are given. These moduli are based on the gross moments of inertia of the section.

In the tables of Struts, the maximum and minimum moduli of section axes XX and YY are given. These moduli are based on the gross moments of inertia of the section.

The maximum moduli has been calculated on the least distance of the neutral axis from the extreme fibres. This modulus has only to be used for Angles as Struts and when the load is applied on the outer face of the angle, such as a Roof Truss Rafter.

Ratio of Slenderness

If the height of a stanchion is divided by its *least* radius of gyration (in same unit dimensions) the quotient is called the ratio of slenderness. This ratio (generally called the l/r) is the controlling factor governing the height of a stanchion. In the tables of stanchions the nearest even height to which the ratio of slenderness equals 150 is the maximum height for which a safe load is given. The B.S.S. however permit this ratio to be increased to 240 for subsidiary members in compression and in our Tables of Angles, etc., used as struts, we have given Tabular loads for ratio of slenderness values up to 240 ,but have inserted a zig-zag line in the tables indicating the slenderness ratio of 150. This latter ratio has not to be exceeded for ordinary stanchions and struts, i.e. main compression members. It should be noted that the L.C.C. Code of Practice permits the slenderness ratio for subsidiary members up to 200 only. Loads beyond this ratio are in *italics*.

Effective Heights

The effective length or height (*l*) of a stanchion for the purpose of determining axial stress, shall be computed as follows :

Where both ends are held in position and restrained in direction ·7 of actual length.

Where both ends are held in position and one end restrained in direction ·85 of the actual length.

Where both ends are held in position but unrestrained in direction, the actual length.

Where one end is held in position but unrestrained in direction and the other end is restrained in direction but not held in position 1·0 to 1·5 times the actual length depending upon the degree of restraint.

Radii of Gyration

The radii of gyration Axes 'XX' and 'YY' are given for all stanchions and struts. These values correspond to the gross moments of inertia. The *least* radius of gyration for each section is printed in Bold Type, as these are more frequently referred to.

Areas

Areas are in square inches, and are based on the gross area of the section. No deduction has been made for rivet holes either in the case of plain sections or compound sections.

Flange Plates

In compound stanchion tables, the column headed 'Flange plates' means the total thickness of plates for each flange. This thickness can, if necessary, be made up by using separate plates $\frac{3}{8}''$ or $\frac{1}{2}''$ thick.

Tubular Columns

The Tubular Section used as a Strut or Stanchion is the most economical, provided the load is axial. The sections given are those which can be got from stock or rolled quickly. Base and Cap Plates should be welded to the Tubes. If screwed Caps or Base Plates are used, an allowance should be made for the reduction of area.

MISCELLANEOUS NOTES

Ties

A table is given with tabulated loads for angles used as tension members. In calculating the safe loads, we have allowed for one and two $\frac{13}{16}''$ holes to be deducted, in addition to the area of half the outstanding leg of each angle. This latter deduction is in accordance with B.S. Requirements to allow for the eccentricity which is set up if the tension bar is connected on one flange only.

Trusses

Scantlings and total all-in weights have been tabulated for estimating purposes for roof trusses, Ridge and North light type of various spans, spaced at 15 ft. centres. This we consider is the most economical spacing for trusses carrying sheeting fixed to steel purlins. If timber purlins are used, it might be advisable to reduce this to 12 ft. 6 in., but the weight of trusses could remain, as the difference is almost negligible. Should the roof covering be slates on boarding instead of sheeting, special calculations should be made.

In the design of trusses, the maintie is assumed as straight. Trusses with raised mainties will require to be treated separately.

Trusses have been designed for the usual B.S.S. Loadings, i.e. 15 lbs. Wind with or without 10 lbs. Suction acting normal to the Rafter Slopes, plus the usual vertical dead loads.

The Struts have been calculated from the B.S.S. Strut Formula (F1) plus the permissible increase of $33\frac{1}{3}$% for Wind.

Timber Beams

Tabulated safe loads given are per 1 inch of beam breadth, i.e. if a beam is 2·5" wide the safe loads can be multiplied by 2·5. Likewise also the modulus values shewn can be multiplied by the required beam width.

For Bending Stresses other than 1000 lbs. per square inch, the safe loads can be obtained by proportioning the tabular loads in the ratio of the required stress to 1000.

The given safe loads are limited to $1\frac{1}{3}$ × Area of the Section × 100 lbs. per sq. in. (Shearing Stress).

London County Council Requirements for Ceiling timbers state that these beams should not deflect more than $\frac{1}{360}$ of the span. Loads to left of the dotted zig-zag line are approximately within these limits if considered applied temporarily (if loads are considered permanent then the deflections will be approximately doubled or the tabular values halved). Loads to the left of the full zig-zag line are for normal requirements.

Loads to the right of the full zig-zag line should only be used where deflections are allowed to exceed normal limits.

The Deflections for each beam carrying the given loads can be calculated by dividing the coefficient given at the foot of the table by the depth of the beam in inches.

The tabular safe loads include the weight of the beam.

For Loads to L.C.C. Stresses. *See page* M.

JOISTS—Safe Distributed Loads in

8 tons/sq. in.

8 tons/sq. in. SPANS

Section	12	14	16	18	20	22	24	26
*24 × 7½ × 100	98·2 *98·2*	84·2 *81·0*	73·8 *66·4*	65·5 *54·9*	59·0 *45·7*	53·5 *38·1*	49·0 *31·9*	45·4 *27·2*
24 × 7½ × 95	93·7 *93·7*	80·4 *77·4*	70·3 *63·3*	62·4 *52·3*	56·3 *43·6*	51·2 *36·5*	46·9 *30·5*	43·3 *26·0*
22 × 7 × 75	67·8 *67·0*	58·0 *53·7*	50·9 *43·9*	45·2 *36·2*	40·7 *30·0*	36·9 *24·4*	33·7 *20·2*	31·2 *16·8*
20 × 7½ × 89	74·3 *74·3*	63·8 *61·4*	55·6 *50·0*	49·5 *41·5*	44·6 *34·6*	40·6 *28·9*	37·2 *24·2*	34·4 *20·6*
20 × 6½ × 65	54·4 *52·4*	46·8 *41·5*	40·8 *33·7*	36·4 *27·3*	32·6 *22·4*	29·7 *18·2*	27·2 *15·0*	25·2 *12·0*
18 × 8 × 80	63·8 *63·8*	54·7 *53·3*	47·8 *44·2*	42·6 *37·3*	38·2 *31·0*	34·8 *26·5*	31·8 *22·3*	29·4 *19·1*
18 × 7 × 75	56·8 *56·1*	48·7 *45·0*	42·6 *36·7*	37·9 *30·3*	34·1 *25·1*	31·0 *20·5*	28·5 *17·1*	26·2 *14·1*
18 × 6 × 55	41·5 *38·4*	35·7 *30·3*	31·1 *24·1*	27·7 *19·4*	24·9 *15·6*	22·6 *12·4*	20·8 *9·9*	19·1
16 × 8 × 75	54·1 *54·1*	46·4 *45·2*	40·6 *37·6*	36·0 *31·5*	32·4 *26·3*	29·5 *22·5*	27·0 *18·9*	24·8 *16·1*
16 × 6 × 62	40·2 *37·2*	34·5 *29·3*	30·2 *23·4*	26·8 *18·8*	24·2 *15·1*	21·9 *12·0*	20·1 *9·6*	18·5
16 × 6 × 50	34·3 *31·7*	29·4 *25·0*	25·7 *19·9*	22·9 *16·0*	20·6 *12·9*	18·7 *10·3*	17·2 *8·2*	15·8
*15 × 6 × 59	37·3 *34·5*	31·9 *27·1*	28·0 *21·7*	24·7 *17·3*	22·4 *14·0*	20·3 *11·2*	18·7 *8·9*	17·2
15 × 6 × 45	29·1 *26·9*	24·8 *21·1*	21·8 *16·9*	19·4 *13·6*	17·4 *10·9*	15·9 *8·8*	14·6 *6·9*	13·4
15 × 5 × 42	25·3 *21·2*	21·8 *16·4*	19·0 *12·4*	16·9 *9·5*	15·2 *7·2*	13·9	12·6	11·7

The lower figures in italics are safe loads for beams not tied laterally, i.e. where l/b exceeds 20. *For general notes see page vi.*

Tons and Dimensions and Properties

IN FEET						Standard Thicknesses		Moment of Inertia	Modulus of Section
28	32	36	40	44	48	Web	Flange	x—x	x—x
42·1 *22·6*	36·9	32·8	29·6	26·8	24·6	·60	1·070	2655	221·0
40·1 *21·6*	35·1	31·1	28·2	25·6	23·5	·57	1·011	2533	211·1
29·0 *13·8*	25·5	22·6	20·4	18·5	15·5	·50	·834	1677	152·4
31·8 *17·1*	27·8	24·8	22·3	18·4	15·4	·60	1·010	1673	167·3
23·3	20·4	18·2	16·3	13·5	11·3	·45	·820	1226	122·6
27·3 *16·0*	23·9 *11·4*	21·3	17·2	14·2	11·9	·50	·950	1292	143·6
24·3 *11·5*	21·3	18·9	15·3	12·6	10·7	·55	·928	1151	127·9
17·8	15·5	13·8	11·2	9·3	7·8	·42	·757	842	93·5
23·2 *13·6*	20·2 *9·6*	16·0	12·8	10·7	9·0	·48	·938	974	121·7
17·3	15·1	11·9	9·6	8·0	6·7	·55	·847	725	90·6
14·7	12·8	10·1	8·2	6·8	5·7	·40	·726	618	77·3
16·0	13·2	10·4	8·4	6·9	5·8	·50	·880	629	83·8
12·5	10·2	8·0	6·6	5·4	4·6	·38	·655	492	65·6
10·8	8·9	7·0	5·7	4·7	4·0	·42	·647	428	57·1

All above loads based on Extreme Fibre Stress 8 Tons/sq. in.

JOISTS—Safe Distributed Loads in

X—X 8 tons/sq. in.	8 tons/sq. in.							SPANS
Section	8	10	12	14	16	18	20	22
14 × 8 × 70	67·2	53·7	44·7	38·4	33·5	29·8	26·8	24·4
	58·0	*53·7*	*44·7*	*37·4*	*31·0*	*26·1*	*21·8*	*18·6*
14 × 6 × 57	50·8	40·6	33·8	29·0	25·3	22·6	20·3	18·4
	50·8	*40·6*	*31·3*	*24·7*	*19·6*	*15·8*	*12·7*	*10·1*
14 × 6 × 46	41·8	33·7	28·1	24·0	21·0	18·7	16·8	15·4
	41·8	*33·7*	*26·0*	*20·4*	*16·3*	*13·1*	*10·5*	*8·5*
*14 × 5½ × 40	36·0	28·7	23·9	20·5	18·0	16·0	14·5	13·1
	36·0	*27·6*	*21·2*	*16·4*	*13·1*	*10·2*	*8·2*	*6·2*
13 × 5 × 35	29·2	23·2	19·3	16·7	14·5	12·9	11·7	10·5
	29·2	*21·5*	*16·2*	*12·5*	*9·4*	*7·3*	*5·6*	
12 × 8 × 65	54·1	43·3	36·2	30·9	27·1	24·1	21·6	19·7
	54·1	*43·3*	*36·2*	*30·1*	*25·1*	*21·1*	*17·6*	*15·0*
12 × 6 × 54	41·8	33·4	27·8	23·8	20·8	18·6	16·7	15·1
	41·8	*33·4*	*25·7*	*20·2*	*16·1*	*13·0*	*10·4*	*8·3*
12 × 6 × 44	35·0	28·1	23·4	20·2	17·5	15·6	14·0	12·7
	35·0	*28·1*	*21·6*	*17·2*	*13·6*	*10·9*	*8·7*	*7·0*
*12 × 5 × 39	29·0	23·2	19·3	16·5	14·5	12·9	11·6	10·5
	29·0	*21·5*	*16·2*	*12·4*	*9·4*	*7·3*	*5·5*	
12 × 5 × 32	24·4	19·6	16·4	14·0	12·2	10·9	9·8	8·9
	24·4	*18·1*	*13·7*	*10·5*	*7·9*	*6·1*	*4·7*	
*12 × 5 × 30	23·0	18·4	15·3	13·1	11·5	10·2	9·2	8·4
	23·0	*17·0*	*12·8*	*9·8*	*7·5*	*5·7*	*4·4*	
*10 × 8 × 70	46·0	36·8	30·8	26·3	23·0	20·4	18·3	15·4
	46·0	*36·8*	*30·8*	*25·6*	*21·3*	*17·9*	*14·9*	*12·7*
10 × 8 × 55	38·3	30·7	25·6	21·9	19·2	17·2	15·3	12·8
	38·3	*30·7*	*25·6*	*21·4*	*17·8*	*15·1*	*12·4*	*10·7*
*10 × 6 × 42	28·2	22·6	18·8	16·0	14·1	12·5	11·2	9·4
	28·2	*22·6*	*17·4*	*13·6*	*10·9*	*8·8*	*7·0*	*5·6*

The lower figures in italics are safe loads for beams not tied laterally, i.e. where l/b exceeds 20. For general notes see page vi.

Tons and Dimensions and Properties

IN FEET						Standard Thick-nesses		Moment of Inertia	Modulus of Section
24	26	28	30	32	36	Web	Flange	x—x	x—x
22·3 *15·6*	20·6 *13·4*	19·1 *11·2*	16·7 *9·6*	14·7 *8·0*	11·6	·46	·920	705·6	**100·8**
16·9 *8·0*	15·6	14·5	12·6	11·1	8·8	·50	·873	533·3	**76·2**
14·0 *6·6*	12·9	12·0	10·4	9·2	7·3	·40	·698	442·6	**63·2**
12·0	11·0	10·3	8·9	7·8	6·2	·37	·627	377·1	**53·9**
9·6	8·9	7·7	6·8	5·9	4·7	·35	·604	283·5	**43·6**
18·0 *12·6*	15·4 *10·8*	13·2 *9·1*	11·6 *7·8*	10·2 *6·4*	8·0	·43	·904	487·8	**81·3**
13·9 *6·6*	11·8	10·2	8·9	7·8	6·2	·50	·883	375·8	**62·6**
11·7 *5·6*	9·9	8·6	7·5	6·6	5·2	·40	·717	316·8	**52·8**
9·7	8·5	7·3	6·2	5·4	4·3	·44	·664	260·9	**43·5**
8·2	6·9	6·0	5·2	4·6	3·6	·35	·550	221·1	**36·8**
7·7	6·5	5·5	4·9	4·3	3·4	·33	·507	206·9	**34·5**
12·8 *10·7*	10·9 *9·2*	9·4 *7·7*	8·2 *6·6*	7·2 *5·5*	5·7	·60	·970	344·9	**69·0**
10·6 *9·0*	9·1 *7·7*	7·8 *6·5*	6·8 *5·5*	6·0 *4·6*	4·7	·40	·783	288·7	**57·7**
7·9 *4·5*	6·8	5·8	5·0	4·4	3·5	·40	·736	211·5	**42·3**

All above loads based on Extreme Fibre Stress 8 Tons/sq. in.

JOISTS—Safe Distributed Loads in

X–X

8 tons/sq. in.

8 tons/sq. in. SPANS

Section	6	7	8	9	10	12	14	16
10 × 6 × 40	36·4	31·2	27·4	24·2	21·8	18·3	15·6	13·7
	36·4	*31·2*	*27·4*	*24·2*	*21·8*	*16·9*	*13·3*	*10·6*
10 × 5 × 30	26·0	22·2	19·5	17·4	15·6	12·9	11·2	9·7
	26·0	*22·2*	*19·5*	*17·0*	*14·4*	*10·8*	*8·4*	*6·3*
10 × 4½ × 25	21·8	18·6	16·3	14·4	13·0	10·8	9·4	8·1
	21·8	*18·6*	*15·9*	*13·3*	*11·4*	*8·4*	*6·4*	*4·7*
*9 × 7 × 58	44·7	38·8	34·0	30·2	27·3	22·7	19·4	17·0
	44·7	*38·8*	*34·0*	*30·2*	*27·3*	*22·4*	*17·9*	*14·7*
9 × 7 × 50	41·1	35·4	30·8	27·4	24·6	20·6	17·6	15·4
	41·1	*35·4*	*30·8*	*27·4*	*24·6*	*20·3*	*16·3*	*13·3*
9 × 4 × 21	16·0	13·7	12·0	10·7	9·6	8·0	6·8	6·0
	16·0	*13·4*	*11·1*	*9·4*	*7·8*	*5·6*	*4·0*	*2·9*
*8 × 8 × 38	29·2	25·0	21·9	19·5	17·5	14·6	12·5	10·8
	29·2	*25·0*	*21·9*	*19·5*	*17·5*	*14·6*	*12·2*	*10·0*
8 × 6 × 35	25·5	21·9	19·2	17·0	15·3	12·7	10·9	9·6
	25·5	*21·9*	*19·2*	*17·0*	*15·3*	*11·7*	*9·3*	*7·4*
8 × 5 × 28	19·8	17·0	14·9	13·3	11·9	9·9	8·6	7·4
	19·8	*17·0*	*14·9*	*13·0*	*11·0*	*8·3*	*6·5*	*4·8*
8 × 4 × 18	12·4	10·5	9·2	8·2	7·5	6·1	5·2	4·6
	12·4	*10·2*	*8·5*	*7·2*	*6·1*	*4·3*	*3·1*	*2·2*
7 × 4 × 16	10·0	8·6	7·5	6·7	6·0	5·0	4·3	3·2
	10·0	*8·4*	*6·9*	*5·9*	*4·9*	*3·5*	*2·5*	*1·8*
*7 × 3½ × 15	9·2	7·9	6·8	6·1	5·5	4·6	3·9	3·0
	9·1	*7·3*	*5·9*	*4·9*	*4·1*	*2·8*	*2·1*	*1·6*
6 × 5 × 25	12·9	11·0	9·8	8·6	7·7	6·4	4·7	3·6
	12·9	*11·0*	*9·8*	*8·4*	*7·1*	*5·4*	*4·2*	*3·2*
6 × 4½ × 20	10·2	8·8	7·7	6·8	6·1	5·1	3·7	2·8
	10·2	*8·8*	*7·5*	*6·3*	*5·3*	*4·0*	*3·0*	*2·2*

The lower figures in italics are safe loads for beams not tied laterally, i.e. where l/b exceeds 20. For general notes see page vi.

Tons and Dimensions and Properties

IN FEET					Standard Thicknesses		Moment of Inertia	Modulus of Section
18	20	22	24	26	Web	Flange	x—x	x—x
12·1 *8·5*	10·9 *6·8*	9·1 *5·5*	7·8 *4·3*	6·5	·36	·709	204·8	41·0
8·6 *4·8*	7·8 *3·7*	6·5	5·4	4·6	·36	·552	146·2	29·2
7·2 *3·4*	6·5	5·4	4·5	3·9	·30	·505	122·3	24·5
15·1 *12·1*	12·3 *10·0*	10·0 *8·3*	8·5 *6·8*	7·2 *5·6*	·55	·924	229·5	51·0
13·7 *11·0*	11·1 *9·1*	9·3 *7·3*	7·7 *6·2*	6·6 *5·1*	·40	·825	208·1	46·3
5·3	4·3	3·6	3·0	2·6	·30	·457	81·1	18·0
8·8 *8·5*	7·0 *7·1*	5·8 *6·1*	4·9 *5·1*	4·1 *4·3*	·33	·550	131·3	32·8
7·5 *6·0*	6·1 *4·8*	5·1 *3·9*	4·3 *3·1*	3·6	·35	·648	115·1	28·8
5·9 *3·7*	4·7 *2·8*	4·0	3·3	2·8	·35	·575	89·7	22·4
3·6	2·9	2·5	2·1	1·8	·28	·398	55·6	13·9
2·6	2·1	1·7	1·5	1·2	·25	·387	39·5	11·3
2·4	1·9	1·6	1·3	1·1	·25	·398	35·9	10·3
2·9 *2·4*	2·3 *1·8*	1·9	1·6	1·4	·41	·520	43·7	14·6
2·3 *1·6*	1·9	1·5	1·3	1·1	·37	·431	34·7	11·6

All above loads based on Extreme Fibre Stress 8 Tons/sq. in.

JOISTS—Safe Distributed Loads in

8 tons/sq. in.	8 tons/sq. in.							SPANS
Section	3	4	5	6	7	8	9	10
6 × 3 × 12	12·4 *12·4*	9·3 *9·3*	7·4 *7·4*	6·2 *5·7*	5·4 *4·6*	4·7 *3·6*	4·1 *2·9*	3·7 *2·3*
5 × 4½ × 20	17·8 *17·8*	13·3 *13·3*	10·6 *10·6*	8·8 *8·8*	7·7 *7·7*	6·6 *6·4*	5·9 *5·5*	5·3 *4·6*
*5 × 4½ × 18	16·2 *16·2*	12·1 *12·1*	9·7 *9·7*	8·1 *8·1*	6·9 *6·9*	6·1 *6·0*	5·4 *5·0*	4·8 *4·2*
5 × 3 × 11	9·7 *9·7*	7·2 *7·2*	5·8 *5·8*	4·8 *4·4*	4·1 *3·5*	3·6 *2·8*	3·2 *2·2*	2·9 *1·8*
*5 × 2½ × 9	7·8 *7·8*	5·8 *5·8*	4·7 *4·4*	3·9 *3·3*	3·4 *2·6*	2·9 *1·9*	2·6 *1·5*	2·3 *1·1*
4¾ × 1¾ × 6·5	5·0 *4·9*	3·7 *3·2*	3·0 *2·2*	2·5 *1·5*	2·1 *1·0*	1·8	1·6	1·5
*4½ × 2 × 7	5·3 *5·3*	3·9 *3·6*	3·2 *2·6*	2·6 *1·8*	2·3 *1·4*	2·1 *1·0*	1·7	1·4
4 × 3 × 10	6·9 *6·9*	5·1 *5·1*	4·2 *4·2*	3·4 *3·2*	2·9 *2·5*	2·5 *1·9*	2·0 *1·6*	1·6 *1·3*
*4 × 3 × 9·5	6·7 *6·7*	5·0 *5·0*	4·1 *4·1*	3·3 *3·1*	2·9 *2·5*	2·5 *1·9*	1·9 *1·6*	1·5 *1·3*
4 × 1¾ × 5	3·2 *3·2*	2·4 *2·1*	1·9 *1·4*	1·7 *1·0*	1·3 *·62*	1·2	·96	·78
3 × 3 × 8·5	4·5 *4·5*	3·3 *3·3*	2·7 *2·7*	2·2 *2·0*	1·6 *1·6*	1·2 *1·3*	1·0 *1·1*	·81 *·84*
3 × 1½ × 4	1·9 *1·8*	1·4 *1·1*	1·1 *·69*	·98 *·47*	·72	·55	·45	·36

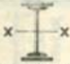
The lower figures in italics are safe loads for beams not tied laterally, i.e. where l/b exceeds 20. For general notes see page vi.

Tons and Dimensions and Properties

IN FEET					Standard Thicknesses		Moment of Inertia	Modulus of Section
11	12	14	16	18	Web	Flange	x—x	x—x
3·3 *1·8*	3·1 *1·5*	2·2	1·7	1·4	·23	·377	21·0	7·0
4·4 *4·0*	3·7 *3·4*	2·7 *2·6*	2·1 *1·9*	1·6 *1·4*	·29	·513	25·0	10·0
4·1 *3·6*	3·4 *3·1*	2·5 *2·4*	1·9 *1·8*	1·5 *1·3*	·29	·448	22·7	9·1
2·4 *1·5*	2·0 *1·2*	1·5	1·1	·90	·22	·376	13·7	5·5
1·9	1·6	1·2	·91	·72	·20	·347	10·9	4·4
1·1	·99	·73	·56	·44	·18	·325	6·7	2·8
1·2	·98	·72	·55	·44	·19	·322	6·6	2·9
1·4 *1·1*	1·2 *·85*	·85	·65	·51	·24	·347	7·8	3·9
1·3 *1·0*	1·1 *·79*	·82	·63	·49	·22	·336	7·5	3·8
·65	·55	·40	·31	·24	·17	·239	3·7	1·8
·67 *·68*	·56 *·53*	·41	·32	·25	·20	·332	3·8	2·5
·30	·25	·19	·14	·11	·16	·249	1·7	1·1

** Sections marked thus are Old B.S. and Special Sections*
All above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS— Safe Distributed Loads in Tons



8 tons/sq. in.

Joist	Flange Plates	Weight per foot in lbs.	SPANS 24	26	28	30	32
24 × 7½ × 100	12 × 2	267·0	Riv	ets ⅞	dia.		118
24 × 7½ × 100	12 × 1¾	246·5				114	107
24 × 7½ × 100	12 × 1½	226·0		118	110	102	96·3
24 × 7½ × 100	12 × 1¼	206·0	114	105	97·8	91·3	85·7
24 × 7½ × 100	12 × 1	185·5	100	92·3	85·8	80·1	75·1
24 × 7½ × 100	12 × ⅞	175·0	92·5	85·9	79·7	74·4	69·8
24 × 7½ × 100	12 × ¾	165·0	86·1	79·4	73·8	68·6	64·6
24 × 7½ × 100	12 × ⅝	155·0	79·1	73·0	67·9	63·3	59·3
24 × 7½ × 100	12 × ½	144·5	72·1	66·6	61·6	57·7	54·1
24 × 7½ × 100	12 × ⅜	134·3	65·4	60·4	56·0	52·4	49·0
24 × 7½ × 95	12 × 2	261·8	Riv	ets ⅞	dia.		116
24 × 7½ × 95	12 × 1¾	241·4					105
24 × 7½ × 95	12 × 1½	221·0			108	101	94·9
24 × 7½ × 95	12 × 1¼	200·7	112	103	96·4	90·0	84·3
24 × 7½ × 95	12 × 1	180·2	98·2	90·7	84·3	78·6	73·8

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia $x—x$	Moduli of Section $x—x$	
36	40	44	48	52		Girder	1" Pl. breadth
104	94·2	85·6	78·6	72·5	9891	706·5	48·4
95·3	85·6	78·0	71·3	66·0	8826	641·9	42·3
85·6	77·0	70·2	64·5	59·5	7799	577·7	36·2
76·1	68·5	62·3	57·1	52·7	6810	513·9	30·1
66·7	60·0	54·6	50·5	46·7	5857	450·5	24·1
62·0	55·8	50·7	46·5	42·6	5394	418·9	21·0
57·4	51·6	46·9	43·0	39·0	4940	387·4	18·0
52·7	47·4	43·1	39·6	35·4	4495	356·0	15·0
48·1	43·3	39·3	36·1	32·2	4058	324·6	12·0
43·6	39·2	35·6	32·8	28·8	3646	294·6	9·0
103	93·1	84·6	77·6	71·6	9783	698·8	48·4
94·0	84·4	76·9	70·4	65·0	8718	633·9	42·3
84·4	75·9	69·1	63·3	58·4	7691	569·7	36·2
74·8	67·4	61·3	56·2	51·9	6702	505·9	30·1
65·5	58·9	53·6	49·1	45·3	5749	442·2	24·1

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Joist	Flange Plates		24	26	28	30	32
24 × 7½ × 95	12 × ⅞	170·0	91·2	84·2	78·1	72·9	68·3
24 × 7½ × 95	12 × ¾	159·8	84·3	77·7	72·1	67·4	63·1
24 × 7½ × 95	12 × ⅝	149·6	77·2	71·2	66·1	61·7	57·8
24 × 7½ × 95	12 × ½	139·5	70·2	64·8	60·1	56·1	52·7
24 × 7½ × 95	12 × ⅜	129·2	63·2	58·3	54·1	50·5	47·3
22 × 7 × 75	12 × 2	241·8	Rivets ⅞ dia.				
22 × 7 × 75	12 × 1¾	221·4					
22 × 7 × 75	12 × 1½	201·0				86·7	81·2
22 × 7 × 75	12 × 1¼	180·6		87·9	81·6	76·2	71·4
22 × 7 × 75	12 × 1	160·3	82·0	75·6	70·4	65·6	61·5
22 × 7 × 75	12 × ⅞	150·0	75·5	69·6	64·8	60·3	56·7
22 × 7 × 75	12 × ¾	139·8	69·0	63·7	59·3	55·2	51·8
22 × 7 × 75	12 × ⅝	129·6	62·6	57·7	53·7	50·1	47·0
22 × 7 × 75	12 × ½	119·5	56·1	51·8	48·1	44·8	42·2
22 × 7 × 75	12 × ⅜	109·2	49·6	45·8	42·6	39·7	37·3

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia x—x	Moduli of Section x—x	
34	36	40	44	48		Girder	1″ Pl. breadth
64·3	60·8	54·7	49·7	45·6	5286	410·6	21·0
59·4	56·1	50·5	45·9	42·1	4832	379·0	18·0
54·4	51·5	46·2	42·1	38·6	4387	347·5	15·0
49·5	46·8	42·1	38·4	35·1	3950	316·0	12·0
44·6	42·2	38·0	34·6	31·6	3520	284·5	9·0
	89·9	80·9	73·6	67·4	7892	607·1	44·4
85·7	81·0	72·8	66·3	60·7	6974	546·9	38·8
76·4	72·2	64·9	59·1	54·1	6093	487·5	33·2
67·2	63·5	57·1	51·9	47·6	5245	428·2	27·6
57·9	54·7	49·3	44·7	41·0	4433	369·4	22·1
53·3	50·3	45·3	41·1	37·3	4038	340·1	19·3
48·7	46·0	41·4	37·6	33·8	3653	310·9	16·5
44·2	41·7	37·5	34·1	30·2	3274	281·8	13·8
39·6	37·4	33·7	30·6	26·9	2905	252·6	11·0
35·0	33·1	29·7	27·1	23·3	2541	223·4	8·2

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in. Joist	Flange Plates	Weight per foot in lbs.	SPANS 22	24	26	28	30
20 × 7½ × 89	12 × 2	256·0	Riv	ets ⅞	dia.		101
20 × 7½ × 89	12 × 1¾	235·5				98·7	92·1
20 × 7½ × 89	12 × 1½	215·0		103	95·4	88·5	82·6
20 × 7½ × 89	12 × 1¼	195·0	100	91·6	84·6	78·5	73·3
20 × 7½ × 89	12 × 1	174·5	87·0	79 8	73·6	68·4	63·8
20 × 7½ × 89	12 × ⅞	164·0	80·5	73·8	68·1	63·3	59·0
20 × 7½ × 89	12 × ¾	153·8	74·2	68·0	62·9	58·3	54·4
20 × 7½ × 89	12 × ⅝	144·0	67·9	62·2	57·4	53·4	49·7
20 × 7½ × 89	12 × ½	133·4	61·6	56·4	52·1	48·4	45·1
20 × 7½ × 89	12 × ⅜	123·2	55·3	50·6	46·8	43·5	40·5
20 × 6½ × 65	10 × 2	204·6	Riv	ets ⅞	dia.		
20 × 6½ × 65	10 × 1¾	187·6					
20 × 6½ × 65	10 × 1½	170·6				70·1	65·4
20 × 6½ × 65	10 × 1¼	153·6		72·1	66·5	61·8	57·8
20 × 6½ × 65	10 × 1	136·6	68·1	62·3	57·6	53·4	49·8

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia x—x	Moduli of Section x—x	
32	34	36	40	44		Girder	1" Pl. breadth
95·4	*89·8*	*84·8*	76·4	69·4	6876	**573·0**	40·4
86·4	81·3	76·7	69·1	62·8	6096	**518·7**	35·3
77·5	72·9	68·8	62·0	56·3	5348	**465·1**	30·2
68·7	64·6	61·0	54·9	50·0	4632	**412·0**	25·2
59·8	56·2	53·1	47·8	43·5	3948	**358·8**	20·1
55·3	52·1	49·2	44·3	40·0	3617	**332·5**	17·6
51·0	48·0	45·3	40·8	36·2	3294	**306·4**	15·0
46·6	43·9	41·5	37·2	32·8	2978	**280·3**	12·5
42·4	39·8	37·6	33·8	29·4	2669	**254·2**	10·0
38·0	35·7	33·7	30·3	27·0	2365	**228·1**	7·6
	71·7	**67·7**	60·9	55·4	5486	**457·2**	40·4
68·6	64·6	61·0	54·9	49·9	4847	**412·4**	35·3
61·3	57·7	54·5	49·0	44·6	4235	**368·2**	30·2
54·0	50·9	48·0	43·3	39·3	3648	**324·2**	25·2
46·7	44·0	41·5	37·4	34·0	3088	**280·8**	20·1

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

x—x 8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Joist	Flange Plates		22	24	26	28	30
20 × 6½ × 65	10 × $\frac{7}{8}$	128·4	62·7	57·5	53·0	49·3	45·9
20 × 6½ × 65	10 × $\frac{3}{4}$	119·8	57·6	52·7	48·6	45·2	42·1
20 × 6½ × 65	10 × $\frac{5}{8}$	111·5	52·3	47·9	44·2	41·1	38·3
20 × 6½ × 65	10 × $\frac{1}{2}$	102·8	47·1	43·1	39·8	37·0	34·6
20 × 6½ × 65	10 × $\frac{3}{8}$	94·4	42·0	38·4	35·4	33·0	30·7
18 × 8 × 80	12 × 2	246·8	Rivets $\frac{7}{8}$ dia.				
18 × 8 × 80	12 × 1¾	226·4					
18 × 8 × 80	12 × 1½	206·0					73·2
18 × 8 × 80	12 × 1¼	185·6			74·8	69·4	64·8
18 × 8 × 80	12 × 1	165·2	76·7	70·3	64·9	60·2	56·2
18 × 8 × 80	12 × $\frac{7}{8}$	155·0	70·9	65·0	60·0	55·7	51·9
18 × 8 × 80	12 × $\frac{3}{4}$	144·8	65·2	59·8	55·2	51·3	47·8
18 × 8 × 80	12 × $\frac{5}{8}$	134·6	59·6	54·6	50·4	46·7	43·7
18 × 8 × 80	12 × $\frac{1}{2}$	124·4	53·9	49·4	45·6	42·3	39·5
18 × 8 × 80	12 × $\frac{3}{8}$	114·2	48·2	44·2	40·8	38·0	35·3

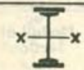
For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia x—x	Moduli of Section x—x	
32	34	36	40	44		Girder	1″ Pl. breadth
43·1	40·6	38·3	34·5	31·0	2816	259·0	17·6
39·5	37·3	35·1	31·6	28·1	2552	237·4	15·0
35·9	33·8	31·9	28·8	25·2	2292	215·8	12·5
32·3	30·4	28·7	25·9	22·4	2040	194·3	10·0
28·7	27·0	25·5	23·0	19·8	1793	172·7	7·6
		75·5	67·9	61·7	5607	509·7	36·5
	72·2	68·1	61·3	54·6	4951	460·6	31·8
68·6	64·6	61·0	54·9	47·6	4328	412·2	27·2
60·7	57·1	53·9	48·6	41·4	3732	364·1	22·7
52·7	49·6	46·8	42·2	34·8	3165	316·5	18·1
48·7	45·9	43·4	38·6	32·1	2893	292·9	15·8
44·8	42·2	39·9	35·0	28·9	2626	269·3	13·5
40·9	38·5	36·4	31·7	26·3	2367	245·9	11·3
37·0	34·8	32·9	28·1	23·2	2114	222·5	9·0
33·0	31·1	29·4	24·8	20·6	1866	199·1	6·7

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons



8 tons/sq. in.

Joist	Flange Plates	Weight per foot in lbs.	SPANS 22	24	26	28	30
18 × 7 × 75	12 × 2	241·9	Riv	ets ⅞	dia.		
18 × 7 × 75	12 × 1¾	221·5					**79·4**
18 × 7 × 75	12 × 1½	201·1			**81·8**	**75·9**	70·8
18 × 7 × 75	12 × 1¼	180·7	**85·1**	**78·0**	71·9	66·8	62·3
18 × 7 × 75	12 × 1	160·3	73·3	67·2	62·0	57·6	53·7
18 × 7 × 75	12 × ⅞	150·1	67·4	61·9	57·0	53·0	49·5
18 × 7 × 75	12 × ¾	139·9	61·7	56·6	52·2	48·5	45·3
18 × 7 × 75	12 × ⅝	129·7	56·0	51·4	47·4	44·0	41·1
18 × 7 × 75	12 × ½	119·5	50·3	46·1	42·6	39·5	36·9
18 × 7 × 75	12 × ⅜	109·3	44·6	40·8	37·8	35·0	32·7
18 × 6 × 55	10 × 2	193·5	Riv	ets ¾	dia.		
18 × 6 × 55	10 × 1¾	176·5					
18 × 6 × 55	10 × 1½	159·5					**57·1**
18 × 6 × 55	10 × 1¼	142·5			**57·6**	**53·6**	49·9
18 × 6 × 55	10 × 1	125·5	**58·5**	**53·6**	49·4	45·9	42·6

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia x—x	Moduli of Section x—x	
32	34	36	40	44		Girder	1" Pl. breadth
82·8	77·9	73·6	66·2	60·2	5466	496·9	36·5
74·5	70·1	66·2	59·6	53·5	4811	447·4	31·8
66·4	62·5	59·0	53·1	46·1	4187	398·8	27·2
58·5	55·0	52·0	46·8	40·1	3592	350·4	22·7
50·4	47·4	44·8	40·3	33·3	3025	302·5	18·1
46·3	43·6	41·2	36·8	30·4	2750	278·5	15·8
42·4	39·9	37·7	33·1	27·3	2485	254·9	13·5
38·5	36·2	34·2	29·8	24·7	2255	231·3	11·3
34·6	32·5	30·7	26·3	21·7	1973	207·7	9·0
30·7	28·8	27·2	23·0	19·0	1728	184·3	6·7
		59·4	53·6	48·7	4439	403·5	36·5
	56·7	53·5	48·2	43·0	3896	362·4	31·8
53·7	50·4	47·6	42·8	37·2	3378	321·8	27·2
46·9	44·1	41·7	37·5	31·7	2883	281·4	22·7
40·3	37·8	35·7	32·1	26·5	2412	241·2	18·1

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Joist	Flange Plates		20	22	24	26	28
18 × 6 × 55	10 × ⅞	116·8	59·0	53·6	49·1	45·4	42·1
18 × 6 × 55	10 × ¾	108·3	53·7	48·8	44·7	41·3	38·4
18 × 6 × 55	10 × ⅝	99·5	48·4	44·0	40·3	37·3	34·6
18 × 6 × 55	10 × ½	91·5	43·1	39·2	35·9	33·2	30·8
18 × 6 × 55	10 × ⅜	82·9	38·0	34·4	31·5	29·1	27·0
16 × 8 × 75	12 × 2	241·8	Riv	ets ⅞	dia.		
16 × 8 × 75	12 × 1¾	221·4					
16 × 8 × 75	12 × 1½	201·0					
16 × 8 × 75	12 × 1¼	180·6				65·1	60·5
16 × 8 × 75	12 × 1	160·2		66·7	61·1	56·4	52·4
16 × 8 × 75	12 × ⅞	150·0	67·8	61·5	56·4	52·1	48·4
16 × 8 × 75	12 × ¾	139·8	62·2	56·5	51·8	47·8	44·4
16 × 8 × 75	12 × ⅝	129·6	56·7	51·5	47·2	43·6	40·5
16 × 8 × 75	12 × ½	119·4	51·1	46·5	42·6	39·3	36·5
16 × 8 × 75	12 × ⅜	109·2	45·6	41·5	38·0	35·0	32·5

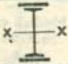
For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia $x—x$	Moduli of Section $x—x$	
30	32	34	36	40		Girder	1" Pl. breadth
39·3	36·8	34·7	32·7	29·2	2185	**221·3**	15·8
35·8	33·5	31·6	29·8	26·1	1964	**201·5**	13·5
32·2	30·3	28·5	26·8	23·3	1748	**181·8**	11·3
28·7	26·9	25·4	23·9	20·5	1539	**162·0**	9·0
25·2	23·6	22·3	21·0	17·8	1334	**142·2**	6·7
		69·9	66·0	59·6	4476	**447·6**	32·4
		63·2	59·7	52·4	3938	**403·9**	28·3
64·0	60·0	56·5	53·4	45·6	3425	**360·5**	24·2
56·5	52·9	49·8	47·0	39·2	2938	**317·8**	20·2
48·9	45·8	43·1	40·7	33·0	2478	**275·3**	16·1
45·1	42·3	39·9	37·1	30·2	2258	**254·2**	14·0
41·4	38·8	36·6	33·6	27·6	2042	**233·4**	12·0
37·8	35·4	33·3	30·4	24·6	1834	**212·6**	10·0
34·1	31·9	30·0	26·8	21·8	1631	**191·8**	8·0
30·4	28·4	26·2	23·6		1432	**171·0**	6·0

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons



8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Joist	Flange Plates		20	22	24	26	28
16 × 6 × 62	10 × 2	200·5	Rivets ¾ dia.			**74·2**	**69·2**
16 × 6 × 62	10 × 1¾	183·2		**79·0**	**72·6**	*66·9*	*62·1*
16 × 6 × 62	10 × 1½	166·5	**77·5**	**70·4**	*64·6*	*59·6*	*55·3*
16 × 6 × 62	10 × 1¼	149·5	*68·0*	*61·8*	*56·7*	52·3	48·6
16 × 6 × 62	10 × 1	132·5	58·5	53·2	48·9	45·0	41·8
16 × 6 × 62	10 × ⅞	123·8	53·8	48·9	44·9	41·4	38·4
16 × 6 × 62	10 × ¾	115·4	49·2	44·7	41·0	37·9	35·1
16 × 6 × 62	10 × ⅝	107·0	44·5	40·5	37·1	34·3	31·8
16 × 6 × 62	10 × ½	98·5	39·9	36·3	33·3	30·7	28·5
16 × 6 × 62	10 × ⅜	89·7	35·3	32·1	29·4	27·2	25·2
16 × 6 × 50	10 × 2	188·3	Rivets ¾ dia.				
16 × 6 × 50	10 × 1¾	171·5					
16 × 6 × 50	10 × 1½	154·6					
16 × 6 × 50	10 × 1¼	137·5				**50·1**	**46·5**
16 × 6 × 50	10 × 1	120·6		**50·6**	**46·4**	42·8	39·7

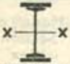
For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia x—x	Moduli of Section x—x	
30	32	34	36	40		Girder	1" Pl. breadth
64·6	60·6	56·8	53·5	48·2	3636	363·6	32·4
57·9	54·3	51·2	48·4	42·7	3188	327·0	28·3
51·6	48·4	45·6	43·1	36·8	2763	290·8	24·2
45·3	42·5	40·0	37·8	31·5	2359	254·9	20·2
39·0	36·7	34·4	32·5	26·3	1976	219·5	16·1
35·9	33·6	31·6	29·5	23·8	1793	201·9	14·0
32·8	30·7	28·9	26·5	21·6	1615	184·5	12·0
29·7	27·8	26·1	23·6	19·2	1441	167·1	10·0
26·7	24·9	23·4	20·9	17·0	1273	149·7	8·0
23·5	22·0	20·3	18·2		1108	132·3	6·0
			52·2	47·2	3539	354·0	32·4
		49·6	46·9	41·2	3092	317·1	28·3
49·8	46·7	44·0	41·6	35·5	2666	280·7	24·2
43·5	40·7	38·3	36·2	30·1	2262	244·6	20·2
37·1	34·8	32·8	30·9	25·0	1880	208·9	16·1

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

 8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Joist	Flange Plates		20	22	24	26	28
16 × 6 × 50	10 × $\frac{7}{8}$	111·8	**50·8**	**46·2**	42·4	39·1	36·3
16 × 6 × 50	10 × $\frac{3}{4}$	103·3	**46·2**	42·0	38·6	35·6	33·0
16 × 6 × 50	10 × $\frac{5}{8}$	94·8	41·6	37·8	34·6	31·9	29·6
16 × 6 × 50	10 × $\frac{1}{2}$	86·3	36·9	33·5	30·6	28·2	26·3
16 × 6 × 50	10 × $\frac{3}{8}$	77·8	32·2	29·2	26·8	24·7	23·0
15 × 6 × 45	10 × 2	183·3	Rivets $\frac{3}{4}$ dia.				
15 × 6 × 45	10 × 1$\frac{3}{4}$	166·3					
15 × 6 × 45	10 × 1$\frac{1}{2}$	149·3					
15 × 6 × 45	10 × 1$\frac{1}{4}$	132·3				**45·9**	**42·6**
15 × 6 × 45	10 × 1	115·3		**46·1**	**42·2**	39·1	36·2
15 × 6 × 45	10 × $\frac{7}{8}$	106·8	**46·2**	**42·0**	38·5	35·5	32·9
15 × 6 × 45	10 × $\frac{3}{4}$	98·3	41·8	38·1	34·8	32·1	29·8
15 × 6 × 45	10 × $\frac{5}{8}$	89·8	37·4	34·0	31·2	28·8	26·6
15 × 6 × 45	10 × $\frac{1}{2}$	81·3	33·0	30·0	27·5	25·4	23·5
15 × 6 × 45	10 × $\frac{3}{8}$	72·8	28·6	26·0	23·9	22·0	20·4

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia $x—x$	Moduli of Section $x—x$	
30	32	34	36	40		Girder	1" Pl. breadth
33·9	31·8	30·0	27·8	22·7	1696	191·1	14·0
30·8	28·9	27·2	24·9	20·3	1518	173·5	12·0
27·7	25·9	24·5	22·1	17·9	1344	156·0	10·0
24·6	23·1	21·7	19·3	15·6	1177	138·4	8·0
21·5	20·1	18·6	16·6		1013	121·0	6·0
			48·2	41·4	3106	326·9	30·6
		45·7	43·4	36·0	2702	292·1	26·6
45·8	42·9	40·4	38·1	30·9	2320	257·8	22·8
39·8	37·2	35·1	32·2	26·0	1957	224·0	19·0
33·8	31·6	29·8	26·6	21·6	1616	190·2	15·2
30·7	28·8	26·8	24·0		1452	173·5	13·2
27·8	26·2	23·8	21·3		1294	156·9	11·3
24·9	23·4	21·2	18·9		1140	140·4	9·4
22·1	20·6	18·2	16·3		991	123·8	7·5
19·1	17·5	15·6			846	107·2	5·6

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in. Joist	Flange Plates	Weight per foot in lbs.	SPANS 20	22	24	26	28
15 × 5 × 42	9 × 1½	136·5	Rivets 3/4 dia.	55·2	50·7	46·7	43·4
15 × 5 × 42	9 × 1¼	121·0	52·6	47·9	43·9	40·5	37·6
15 × 5 × 42	9 × 1	105·5	44·7	40·6	37·2	34·3	31·9
15 × 5 × 42	9 × ⅞	97·9	40·6	37·0	33·9	31·2	29·0
15 × 5 × 42	9 × ¾	90·5	36·7	33·5	30·6	28·2	26·3
15 × 5 × 42	9 × ⅝	83·0	32·7	29·8	27·3	25·2	23·4
15 × 5 × 42	9 × ½	75·0	28·8	26·3	24·0	22·2	20·7
15 × 5 × 42	9 × ⅜	67·5	25·0	22·6	20·7	19·2	17·8
14 × 8 × 70	10 × 2	210·0	Rivets ⅞ dia.				
14 × 8 × 70	10 × 1¾	193·0					
14 × 8 × 70	10 × 1½	176·0				54·8	50·9
14 × 8 × 70	10 × 1¼	159·0			52·6	48·5	45·1
14 × 8 × 70	10 × 1	142·0	55·1	50·1	45·9	42·4	39·4
14 × 8 × 70	10 × ⅞	133·5	51·2	46·5	42·6	39·3	36·5
14 × 8 × 70	10 × ¾	125·0	47·2	42·9	39·4	36·3	33·7

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia x—x	Moduli of Section x—x	
30	32	34	36	40		Girder	1" Pl. breadth
40·5	37·9	35·7	33·7	27·3	2053	228·0	22·8
35·1	32·9	30·9	28·4	23·0	1728	197·7	19·0
29·7	27·9	26·2	23·4	19·0	1424	167·5	15·2
27·0	25·4	23·6	21·0		1278	152·6	13·2
24·4	22·9	20·9	18·7		1137	137·8	11·3
21·8	20·5	18·4	16·5		1000	123·1	9·4
19·2	18·0	15·9	14·3		866	108·3	7·5
16·6	15·3	13·5			737	93·5	5·6
		51·7	48·8	39·6	2966	329·6	28·5
53·0	49·7	46·7	42·9	34·7	2609	298·2	24·9
47·5	44·5	41·9	37·4	30·2	2272	267·3	21·3
42·1	39·4	36·0	32·1		1954	236·9	17·7
36·7	34·4	30·5	27·2		1655	206·9	14·1
34·1	31·5	27·9			1513	192·1	12·3
31·5	28·6	25·3			1374	177·3	10·5

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Joist	Flange Plates		18	20	22	24	26
14 × 8 × 70	10 × $\frac{5}{8}$	116·5	48·1	43·3	39·4	36·1	33·3
14 × 8 × 70	10 × $\frac{1}{2}$	108·0	43·8	39·4	35·9	32·9	30·3
14 × 8 × 70	10 × $\frac{3}{8}$	99·5	39·6	35·6	32·4	29·6	27·4
14 × 6 × 57	10 × $1\frac{1}{2}$	161·5	Riv. $\frac{5}{8}$	dia.	60·8	55·7	*51·5*
14 × 6 × 57	10 × $1\frac{1}{4}$	144·5	65·0	58·5	*53·3*	*48·7*	45·0
14 × 6 × 57	10 × 1	127·5	55·8	50·2	45·8	41·8	38·6
14 × 6 × 57	10 × $\frac{7}{8}$	119·0	51·2	46·1	41·9	38·4	35·4
14 × 6 × 57	10 × $\frac{3}{4}$	110·3	46·7	42·0	38·3	35·0	32·3
14 × 6 × 57	10 × $\frac{5}{8}$	102·0	42·2	38·1	34·6	31·7	29·2
14 × 6 × 57	10 × $\frac{1}{2}$	93·5	37·7	34·0	30·9	28·4	26·1
14 × 6 × 57	10 × $\frac{3}{8}$	85·0	33·2	30·0	27·2	25·0	23·0
14 × 6 × 46	10 × $1\frac{1}{2}$	150·5	Riv	ets $\frac{3}{4}$	dia.		
14 × 6 × 46	10 × $1\frac{1}{4}$	133·5				46·6	43·1
14 × 6 × 46	10 × 1	116·5		47·6	43·3	39·6	36·6
14 × 6 × 46	10 × $\frac{7}{8}$	108·0	48·2	43·4	39·4	36·2	33·3

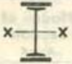
For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia $x—x$	Moduli of Section $x—x$	
28	30	32	34	36		Girder	1" Pl. breadth
30·9	28·9	25·8	22·8		1240	162·7	8·8
28·2	26·3	23·1	20·4		1111	148·1	7·0
25·4	23·2	20·5			985	133·5	5·2
47·8	*44·6*	41·8	39·3	35·2	2134	251·0	21·3
41·8	39·0	36·6	33·4	30·0	1812	219·7	17·7
35·9	33·6	31·4	27·8	24·8	1508	188·5	14·1
32·9	30·7	28·4	24·9		1364	173·1	12·3
30·0	28·0	25·4	22·6		1223	157·8	10·5
27·1	25·3	22·6	20·0		1088	142·7	8·8
24·2	22·7	19·9	17·6		956	127·6	7·0
21·3	19·5	17·1	15·3		828	112·3	5·2
46·0	42·8	*40·2*	37·9	33·8	2054	241·7	21·3
40·0	37·3	34·9	31·9	28·6	1732	210·0	17·7
34·0	31·8	29·7	26·3	23·5	1429	178·7	14·1
31·0	28·9	26·6	23·5		1284	163·0	12·3

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons



8 tons/sq. in.

Joist	Flange Plates	Weight per foot in lbs.	SPANS 16	18	20	22	24
14 × 6 × 46	10 × $\frac{3}{4}$	99·5	49·2	43·7	39·3	35·8	32·8
14 × 6 × 46	10 × $\frac{5}{8}$	91·0	44·0	39·1	35·2	32·0	29·2
14 × 6 × 46	10 × $\frac{1}{2}$	82·6	38·9	34·6	31·1	28·3	25·9
14 × 6 × 46	10 × $\frac{3}{8}$	74·0	33·8	30·1	27·0	24·6	22·5
14 × 5½ × 40	9 × 1½	134·5	Riv	ets $\frac{3}{4}$	dia.		
14 × 5½ × 40	9 × 1¼	119·0				44·8	39·8
14 × 5½ × 40	9 × 1	103·0		46·4	39·8	38·2	34·9
14 × 5½ × 40	9 × $\frac{7}{8}$	95·0	47·3	39·8	38·2	34·7	31·8
14 × 5½ × 40	9 × $\frac{3}{4}$	87·5	39·8	38·4	34·6	31·3	28·7
14 × 5½ × 40	9 × $\frac{5}{8}$	80·0	38·6	34·2	30·8	28·1	25·7
14 × 5½ × 40	9 × $\frac{1}{2}$	71·6	33·9	30·1	27·1	24·7	22·6
14 × 5½ × 40	9 × $\frac{3}{8}$	64·0	29·5	26·2	23·5	21·5	19·6
13 × 5 × 35	9 × 1½	129·0	Riv	ets $\frac{3}{4}$	dia.		
13 × 5 × 35	9 × 1¼	113·9					36·9
13 × 5 × 35	9 × 1	98·5			37·3	33·9	31·1

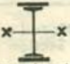
For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia $x-x$	Moduli of Section $x-x$	
26	28	30	32	34		Girder	1" Pl. breadth
30·2	28·1	26·2	23·9	21·1	1144	147·6	10·5
27·0	25·1	23·4	21·0	18·6	1008	132·3	8·8
23·9	22·3	20·7	18·2	16·1	877	116·9	7·0
20·8	19·3	17·7	15·5	13·7	750	101·5	5·2
43·7	40·6	37·9	35·5	33·4	1812	213·2	21·3
38·0	35·2	32·9	30·8	28·2	1526	185·0	17·7
32·2	29·9	27·9	26·2	23·3	1256	157·0	14·1
29·5	27·3	25·4	23·4	20·9	1126	143·0	12·3
26·5	24·6	23·0	20·9	18·4	1002	129·2	10·5
23·7	22·0	20·6	18·4	16·4	881	115·6	8·8
20·9	19·5	18·1	15·8	14·1	763	101·8	7·0
18·1	16·8	15·3	13·5	12·1	650	88·3	5·2
	36·7	34·3	32·1	28·4	1542	192·8	19·8
34·1	31·6	29·5	27·0	24·0	1287	166·1	16·5
28·7	26·7	24·8	21·8	19·4	1050	140·1	13·1

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons



8 tons/sq. in. Joist	Flange Plates	Weight per foot in lbs	SPANS 14	16	18	20	22
13 × 5 × 35	9 × $\frac{7}{8}$	90·9	Riv.	$\frac{3}{4}$ dia.	37·6	33·8	30·7
13 × 5 × 35	9 × $\frac{3}{4}$	83·5		38·0	33·8	30·4	27·6
13 × 5 × 35	9 × $\frac{5}{8}$	76·0	38·6	33·7	30·0	26·9	24·5
13 × 5 × 35	9 × $\frac{1}{2}$	67·9	33·7	29·4	26·2	23·6	21·5
13 × 5 × 35	9 × $\frac{3}{8}$	60·5	28·8	25·1	22·4	20·1	18·3
12 × 8 × 65	12 × 1$\frac{1}{2}$	191·0	Riv	ets $\frac{7}{8}$	dia.		
12 × 8 × 65	12 × 1$\frac{1}{4}$	170·6					
12 × 8 × 65	12 × 1	150·2					
12 × 8 × 65	12 × $\frac{7}{8}$	140·0					43·7
12 × 8 × 65	12 × $\frac{3}{4}$	129·8				43·9	39·9
12 × 8 × 65	12 × $\frac{5}{8}$	119·6			44·2	39·7	36·2
12 × 8 × 65	12 × $\frac{1}{2}$	109·4		44·5	39·6	35·6	32·4
12 × 8 × 65	12 × $\frac{3}{8}$	99·2	45·3	39·3	35·0	31·5	28·6
12 × 6 × 54	10 × 1$\frac{1}{2}$	158·4	Riv	ets $\frac{3}{4}$	dia.		51·4
12 × 6 × 54	10 × 1$\frac{1}{4}$	141·3			54·8	49·4	44·9

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia $x-x$	Moduli of Section $x-x$	
24	26	28	30	32		Girder	1" Pl. breadth
28·1	26·0	24·1	22·2	19·5	936	126·9	11·5
25·3	23·4	21·7	19·6	17·2	827	114·1	9·8
22·4	20·7	19·2	17·0	15·0	721	101·3	8·2
19·6	18·1	16·8	14·7	12·9	619	88·6	6·5
16·8	15·5	14·1	12·2		520	75·7	4·9
		49·6	46·3	40·7	1956	260·8	18·3
	46·8	43·4	39·7	34·8	1656	228·5	15·2
43·6	40·2	37·3	32·5	28·6	1374	196·3	12·1
40·0	37·0	33·5	29·4		1239	180·3	10·6
36·6	33·8	30·2	26·3		1112	164·8	9·0
33·1	30·6	26·8	23·5		989	149·3	7·5
29·7	27·4	23·6	20·6		870	133·8	6·0
26·3	23·8	20·4	17·8		754	118·3	4·5
47·2	43·6	40·5	37·7	33·1	1593	212·4	18·3
41·2	38·0	35·3	31·8	27·9	1343	185·6	15·2

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Joist	Flange Plates		12	14	16	18	20
12×6×54	10×1	124·3			52·8	46·9	42·2
12×6×54	10×⅞	115·8		55·3	48·3	42·9	38·7
12×6×54	10×¾	107·2	58·7	50·3	44·0	39·1	35·2
12×6×54	10×⅝	98·9	53·0	45·4	39·7	35·3	31·8
12×6×54	10×½	90·4	47·2	40·5	35·4	31·5	28·3
12×6×54	10×⅜	81·8	41·4	35·6	31·1	27·7	24·8
12×6×44	10×1½	148·5	Riv	ets ¾	dia.		
12×6×44	10×1¼	131·6					
12×6×44	10×1	114·5					40·3
12×6×44	10×⅞	106·0				40·8	36·7
12×6×44	10×¾	97·5			41·5	36·9	33·2
12×6×44	10×⅝	89·0		42·4	37·1	33·0	29·7
12×6×44	10×½	80·6	43·7	37·4	32·8	29·2	26·2
12×6×44	10×⅜	72·0	38·0	32·4	28·4	25·2	22·7
12×6×44	9×⅜	69·5	36·0	30·8	27·0	24·0	21·6

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia $x-x$	Moduli of Section $x-x$	
22	24	26	28	30		Girder	1" Pl. breadth
38·4	35·2	32·5	30·2	26·3	1110	158·7	12·1
35·1	32·2	29·8	27·1	23·6	999	145·3	10·6
32·1	29·3	27·1	24·2	21·1	893	132·4	9·0
28·8	26·5	24·5	21·6	18·9	790	119·3	7·5
25·7	23·6	21·8	18·9	16·3	691	106·3	6·0
22·6	20·7	18·8	16·0		595	93·3	4·5
		42·1	39·1	36·5	1542	205·6	18·3
	39·6	36·6	34·0	30·5	1293	178·4	15·2
36·7	33·6	31·0	28·8	25·0	1058	151·3	12·1
33·4	30·5	28·2	25·6	22·5	948	137·9	10·6
30·3	27·6	25·5	22·8	19·9	841	124·7	9·0
27·0	24·7	22·8	20·0	17·6	739	111·5	7·5
23·8	21·8	20·1	17·5	15·1	640	98·5	6·0
20·6	19·0	17·1	14·7		544	85·3	4·5
19·6	18·0	16·2	14·0		515	80·8	4·5

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Joist	Flange Plates		14	16	18	20	22
12×5×32	9×1½	126·3	Riv	ets ¾	dia.		
12×5×32	9×1¼	111·0					36·5
12×5×32	9×1	95·6			37·4	33·7	30·6
12×5×32	9×⅞	88·0		38·1	33·8	30·5	27·7
12×5×32	9×¾	80·3	39·0	34·1	30·3	27·4	24·8
12×5×32	9×⅝	72·7	34·5	30·2	26·8	24·2	21·9
12×5×32	9×½	65·0	30·0	26·2	23·4	21·0	19·0
12×5×32	9×⅜	57·4	25·5	22·3	19·8	17·9	16·1
10×8×55	12×1½	181·0	Riv	ets ⅞	dia.		
10×8×55	12×1¼	160·6					
10×8×55	12×1	140·2					37·5
10×8×55	12×⅞	130·0				37·7	34·2
10×8×55	12×¾	119·8				34·2	31·1
10×8×55	12×⅝	109·6			34·1	30·8	27·9
10×8×55	12×½	99·4		34·1	30·3	27·3	24·8

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia $x—x$	Moduli of Section $x—x$	
24	26	28	30	32		Girder	1" Pl. breadth
	35·9	33·4	31·2	27·4	1316	175·3	18·3
33·5	30·9	28·7	25·8	22·8	1094	150·9	15·2
28·1	25·9	24·0	20·9	18·5	885	126·6	12·1
25·3	23·4	21·4	18·7		787	114·4	10·6
22·7	21·0	18·9	16·3		692	102·6	9·0
20·1	18·5	16·5	14·2		600	90·7	7·5
17·5	16·2	13·9	12·1		512	78·8	6·0
14·9	13·5	11·6			427	67·0	4·5
	43·0	37·0	32·0	28·3	1362	209·6	15·3
40·5	36·3	31·2			1140	182·4	12·7
34·4	29·3	25·3	22·0		930	155·0	10·1
30·8	26·2				831	141·5	8·8
27·3	23·3	20·1			739	128·5	7·5
24·3	20·6				650	115·5	6·3
20·8	17·7				564	102·5	5·0

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq.in.		Weight per foot in lbs.	SPANS				
Joist	Flange Plates		10	12	14	16	18
10 × 8 × 55	12 × ⅜	89·2			34·2	29·8	26·5
10 × 6 × 40	10 × 1½	144·3	Riv	ets ¾	dia.		
10 × 6 × 40	10 × 1¼	127·3					
10 × 6 × 40	10 × 1	110·4					
10 × 6 × 40	10 × ⅞	101·8					
10 × 6 × 40	10 × ¾	93·3				33·5	29·8
10 × 6 × 40	10 × ⅝	84·9			34·1	29·9	26·5
10 × 6 × 40	10 × ½	76·3		35·0	30·0	26·3	23·4
10 × 6 × 40	10 × ⅜	67·8	36·4	30·2	25·9	22·7	20·1
10 × 5 × 30	9 × 1½	124·1	Riv	ets ¾	dia.		
10 × 5 × 30	9 × 1¼	108·9					
10 × 5 × 30	9 × 1	93·6				34·7	30·7
10 × 5 × 30	9 × ⅞	85·8			35·7	31·2	27·7
10 × 5 × 30	9 × ¾	78·2		37·2	31·9	27·9	24·8
10 × 5 × 30	9 × ⅝	71·0		32·8	28·1	24·6	21·8

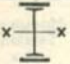
For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia $x-x$	Moduli of Section $x-x$	
20	22	24	26	28		Girder	1" Pl. breadth
23·8	21·2	17·7			481	89·5	3·7
		37·5	34·6	29·8	1097	168·7	15·3
	35·3	32·4	28·8	24·8	913	146·0	12·7
	29·8	27·4	23·2	20·0	738	123·0	10·1
29·7	27·0	24·3	20·7		656	111·8	8·8
26·8	24·5	21·4	18·2		579	100·7	7·5
23·9	21·7	18·9	16·0		504	89·9	6·3
21·0	19·1	16·0	13·7		434	78·9	5·0
18·1	16·1	13·4			366	67·9	3·7
		32·2	29·8	25·6	943	145·1	15·3
33·1	30·1	27·6	24·8	21·2	778	124·4	12·7
27·6	25·1	23·0	19·6	16·9	623	103·8	10·1
24·9	22·7	20·4	17·3		551	93·7	8·8
22·3	20·3	17·7	15·2		482	83·8	7·5
19·6	17·9	15·5	13·2		416	73·9	6·3

Above loads based on Extreme Fibre Stress 8 Tons/sq.in.

COMPOUND GIRDERS— Safe Distributed Loads in Tons



8 tons/sq. in. Joist	Flange Plates	Weight per foot in lbs.	SPANS 10	12	14	16	18
10 × 5 × 30	9 × ½	62·9	34·1	28·4	24·3	21·4	18·9
10 × 5 × 30	9 × ⅜	55·5	28·8	24·0	20·5	18·0	16·0
10 × 4½ × 25	9 × 1	88·4	Riv	ets ¾	dia.		
10 × 4½ × 25	9 × ⅞	80·8					26·6
10 × 4½ × 25	9 × ¾	73·5				26·6	23·6
10 × 4½ × 25	9 × ⅝	66·0			26·6	23·2	20·7
10 × 4½ × 25	9 × ½	57·8		26·6	22·8	19·9	17·7
10 × 4½ × 25	9 × ⅜	50·5	26·6	22·1	19·0	16·6	14·7
9 × 7 × 50	10 × 1½	155·6	Riv	ets ⅞	dia.		
9 × 7 × 50	10 × 1¼	138·6					
9 × 7 × 50	10 × 1	122·0					
9 × 7 × 50	10 × ⅞	113·5					31·3
9 × 7 × 50	10 × ¾	105·0				32·1	28·5
9 × 7 × 50	10 × ⅝	96·5			33·1	29·0	25·7
9 × 7 × 50	10 × ½	88·0		34·4	29·5	25·8	22·9

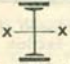
For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia $x—x$	Moduli of Section $x—x$	
20	22	24	26	28		Girder	1" Pl. breadth
17·1	15·5	13·0	11·2		352	64·0	5·0
14·4	12·9	10·7			292	54·1	3·7
26·7	24·2	22·2	18·9	16·3	601	100·1	10·1
24·0	21·7	19·6	16·6		528	89·9	8·8
21·3	19·3	17·0	14·4		459	79·9	7·5
18·6	16·9	14·3	12·3		393	70·0	6·3
15·9	14·5	12·2	10·4		330	60·0	5·0
13·2	11·8	9·8			269	50·0	3·7
	37·8	34·7	29·5	25·5	937	156·2	13·9
36·3	32·9	29·3	24·9		783	136·1	11·5
30·9	28·0	23·6	20·1	17·3	637	115·9	9·1
28·2	25·1	21·2			570	106·0	8·0
25·7	22·3	18·7			506	96·5	6·8
23·1	19·7	16·6			445	87·0	5·7
20·6	17·0	14·3			387	77·5	4·5

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons



8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Joist	Flange Plates		6	8	10	12	14
9 × 7 × 50	10 × $\frac{3}{8}$	79·1				30·2	26·0
8 × 6 × 35	10 × 1	105·3	Riv	ets $\frac{3}{4}$	dia.		
8 × 6 × 35	10 × $\frac{7}{8}$	96·8					
8 × 6 × 35	10 × $\frac{3}{4}$	88·3					
8 × 6 × 35	10 × $\frac{5}{8}$	79·8					26·0
8 × 6 × 35	10 × $\frac{1}{2}$	71·3				26·4	22·6
8 × 6 × 35	10 × $\frac{3}{8}$	62·8			26·9	22·5	19·2
8 × 5 × 28	9 × 1	91·5	Riv	ets $\frac{3}{4}$	dia.		
8 × 5 × 28	9 × $\frac{7}{8}$	84·0					
8 × 5 × 28	9 × $\frac{3}{4}$	76·3					25·0
8 × 5 × 28	9 × $\frac{5}{8}$	69·0				25·7	22·0
8 × 5 × 28	9 × $\frac{1}{2}$	61·0			26·6	22·2	19·0
8 × 5 × 28	9 × $\frac{3}{8}$	53·5		28·1	22·5	18·7	16·0
6 × 5 × 25	9 × $\frac{1}{2}$	58·1		23·6	18·9	15·8	13·5
6 × 5 × 25	9 × $\frac{3}{8}$	50·5	26·4	19·8	15·8	13·2	10·9

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET					Moment of Inertia x—x	Moduli of Section x—x	
16	18	20	22	24		Girder	1" Pl. breadth
22·6	20·1	17·7	14·5	12·2	332	68·0	3·3
	28·0	25·3	20·9	17·5	475	95·0	8·1
	25·4	22·3	18·5	15·3	418	85·9	7·1
25·6	22·8	19·5	16·1	13·5	366	77·0	6·1
22·8	20·2	16·7	13·8		315	68·3	5·0
19·8	17·6	14·2	11·7		267	59·4	4·0
16·8	14·6	11·8			222	50·5	3·0
	24·0	21·8	18·0	15·1	410	82·0	8·1
24·5	21·7	19·1	15·8	13·0	360	73·8	7·1
21·9	19·4	16·6	13·7		313	65·8	6·1
19·2	17·1	14·2	11·7		268	57·8	5·0
16·6	14·8	11·9	9·9		225	50·0	4·0
14·0	12·1	9·8			185	42·2	3·0
10·3	8·2				124	35·5	3·0
8·3					100	29·7	2·3

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

 x—x 8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Two Joists	Flange Plates		24	26	28	30	32
24 × 7½ × 100	18 × 2½	510·0	Riv	ets ⅞	dia.	**234**	**220**
24 × 7½ × 100	18 × 2	448·0		**232**	**216**	*200*	*189*
24 × 7½ × 100	18 × 1½	387·0	**209**	*194*	*180*	*168*	*158*
24 × 7½ × 100	18 × 1¼	357·0	*189*	*175*	*162*	152	142
24 × 7½ × 100	18 × 1	326·0	169	156	145	135	127
24 × 7½ × 100	18 × ¾	295·5	149	138	128	119	112
24 × 7½ × 100	18 × ⅝	280·0	139	128	119	111	104
24 × 7½ × 100	18 × ½	265·0	129	119	111	103	97·0
24 × 7½ × 95	18 × 2½	500·0	Riv	ets ⅞	dia.		**214**
24 × 7½ × 95	18 × 2	438·0			**210**	**196**	*184*
24 × 7½ × 95	18 × 1½	377·0	**204**	*189*	*175*	*163*	*153*
24 × 7½ × 95	18 × 1¼	347·0	*184*	*170*	*158*	147	138
24 × 7½ × 95	18 × 1	316·0	164	151	140	131	123
24 × 7½ × 95	18 × ¾	285·5	144	133	123	115	108
24 × 7½ × 95	18 × ⅝	270·0	134	123	115	108	101

For notes relating to above see page vi.

and Properties and Dimensions

IN FEET						Moment of Inertia $x—x$	Modulus of Section $x—x$
36	40	44	48	52	56		
196	*176*	*160*	*146*	*135*	*125*	19110	1318
168	*151*	*137*	*126*	*116*	*108*	15834	1131
140	126	114	105	97·2	86·8	12758	945
126	114	103	94·8	87·6	77·0	11316	854
113	102	92·4	84·6	78·2	67·4	9906	762
99·8	89·5	81·5	74·6	67·5	58·2	8568	672
93·0	83·5	76·0	69·7	62·3	53·8	7916	627
86·4	77·5	70·5	64·5	57·3	49·6	7275	582
191	*171*	*156*	*144*	*133*	*123*	18776	1295
164	147	*134*	*123*	*113*	105	15502	1107
136	122	111	102	94·6	84·7	12453	922
123	111	101	92·4	85·3	75·1	11011	831
109	98·6	89·7	82·2	75·9	65·4	9621	740
96·2	86·6	78·7	72·1	65·3	56·3	8285	650
89·6	80·6	73·2	67·2	60·5	51·7	7636	605

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Two Joists	Flange Plates		22	24	26	28	30
24 × 7½ × 95	18 × ½	255·0	135	124	114	106	99·6
22 × 7 × 75	18 × 2½	460·0	Riv	ets ⅞	dia.		
22 × 7 × 75	18 × 2	398·4					**168**
22 × 7 × 75	18 × 1½	337·2		**172**	**159**	*147*	*137*
22 × 7 × 75	18 × 1¼	307·0	**168**	*153*	*142*	*131*	*122*
22 × 7 × 75	18 × 1	276·0	*147*	*134*	124	115	107
22 × 7 × 75	18 × ¾	245·4	126	116	107	99·6	92·8
22 × 7 × 75	18 × ⅝	230·2	116	107	98·5	91·6	85·4
22 × 7 × 75	18 × ½	214·9	106	97·6	90·0	83·7	78·0
20 × 7½ × 89	18 × 2½	487·7	Riv	ets ⅞	dia.	**201**	**187**
20 × 7½ × 89	18 × 2	426·5		**200**	**185**	*172*	*160*
20 × 7½ × 89	18 × 1½	365·4	**181**	*166*	*153*	*142*	*133*
20 × 7½ × 89	18 × 1¼	334·7	*163*	*149*	*138*	*128*	120
20 × 7½ × 89	18 × 1	304·1	*144*	132	122	113	106
0 × 7½ × 89	18 × ¾	273·6	126	116	107	99·5	92·9

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET						Moment of Inertia x—x	Modulus of Section x—x
32	36	40	44	48	52		
93·3	82·9	74·6	67·8	62·2	55·2	7001	560
	165	*149*	*135*	*125*	*115*	15151	**1122**
157	*140*	*126*	*114*	*105*	*97·1*	12319	**948**
129	*114*	103	94·1	86·2	76·5	9698	**776**
115	102	92·2	83·8	76·7	66·6	8465	**691**
101	89·8	80·8	73·5	67·3	57·4	7278	**607**
87·1	77·4	69·6	63·3	56·8	48·4	6141	**523**
80·1	71·2	64·2	58·3	51·6	44·2	5590	**481**
73·2	65·0	58·6	53·2	46·7	39·8	5051	**439**
176	*156*	*140*	*127*	*117*	*104*	13260	**1061**
150	*133*	*120*	*109*	*100*	85·5	10840	**903**
124	*110*	99·8	90·8	79·7	67·9	8613	**749**
112	99·8	89·8	81·7	70·0	60·0	7571	**673**
99·6	88·5	79·6	72·4	60·8	51·8	6572	**597**
87·2	77·4	69·7	61·9	52·0		5618	**523**

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Two Joists	Flange Plates		22	24	26	28	30
20 × 7½ × 89	18 × ⅝	258·2	117	108	99·5	92·5	86·4
20 × 7½ × 89	18 × ½	242·9	108	99·6	91·9	85·4	79·8
20 × 6½ × 65	16 × 2	351·3	Rivets ⅞ dia.			143	133
20 × 6½ × 65	16 × 1½	296·8		136	126	*117*	*109*
20 × 6½ × 65	16 × 1¼	269·6	132	*121*	*112*	105	97·6
20 × 6½ × 65	16 × 1	242·4	116	106	98·7	91·6	85·5
20 × 6½ × 65	16 × ¾	215·2	100	92·1	85·0	78·9	73·8
20 × 6½ × 65	16 × ⅝	201·6	92·4	84·7	78·2	72·6	67·8
20 × 6½ × 65	16 × ½	188·0	84·4	77·4	71·4	66·3	61·9
18 × 8 × 80	18 × 2½	469·6	Rivets ⅞ dia.				
18 × 8 × 80	18 × 2	408·4					142
18 × 8 × 80	18 × 1½	347·2		147	136	*126*	*117*
18 × 8 × 80	18 × 1¼	316·6	144	132	*122*	*113*	*105*
18 × 8 × 80	18 × 1	286·0	*127*	117	108	100	93·0
18 × 8 × 80	18 × ¾	255·4	111	102	94·0	87·0	81·0

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET						Moment of Inertia $x—x$	Modulus of Section $x—x$
32	36	40	44	48	52		
80·9	71·9	64·7	56·9	47·6		5158	485·5
74·7	66·4	59·7	51·8	43·5		4708	448·4
125	*111*	*100*	*91·3*	83·6	71·2	9031	752·6
102	91·2	82·0	74·6	65·5	55·8	7080	615·8
91·4	81·2	73·1	66·5	57·0	48·9	6167	548·1
80·1	71·2	64·1	58·3	49·0	41·7	5292	481·1
69·1	61·4	55·3	49·1	41·2		4457	414·6
63·5	56·5	50·8	44·8	37·6		4055	381·6
58·0	51·6	46·4	40·3	33·8		3660	348·6
158	140	*126*	*115*	*100*	*86·0*	10868	945·0
134	*119*	*107*	*97·0*	*81·5*	69·6	8805	800·5
110	*98·0*	88·0	76·2	64·2		6942	661·1
99·0	88·0	79·0	67·0	56·2		6074	592·6
87·0	78·0	70·0	57·8	48·5		5247	524·7
76·0	68·0	59·8	49·2			4460	457·5

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Two Joists	Flange Plates		**22**	**24**	**26**	**28**	**30**
18 × 8 × 80	18 × $\frac{5}{8}$	240·1	103	94·0	87·0	80·8	75·4
18 × 8 × 80	18 × $\frac{1}{2}$	224·8	95·0	87·0	80·0	74·0	70·0
18 × 7 × 75	18 × $2\frac{1}{2}$	459·8	Riv	ets $\frac{7}{8}$	dia.		**162**
18 × 7 × 75	18 × 2	398·6		**172**	**158**	*147*	*137*
18 × 7 × 75	18 × $1\frac{1}{2}$	337·4	**153**	*140*	*130*	*120*	*112*
18 × 7 × 75	18 × $1\frac{1}{4}$	306·8	*137*	*125*	*115*	*108*	*100*
18 × 7 × 75	18 × 1	276·2	*120*	110	101	94·5	88·2
18 × 7 × 75	18 × $\frac{3}{4}$	245·6	103	95·2	87·9	81·6	76·2
18 × 7 × 75	18 × $\frac{5}{8}$	230·3	95·7	87·7	81·0	75·2	70·2
18 × 7 × 75	18 × $\frac{1}{2}$	215·0	87·5	80·2	74·1	68·8	64·2
18 × 6 × 55	16 × 2	329·9	Riv	ets $\frac{3}{4}$	dia.		**117**
18 × 6 × 55	16 × $1\frac{1}{2}$	275·5		**118**	**109**	*101*	*95·0*
18 × 6 × 55	16 × $1\frac{1}{4}$	248·3	**115**	*105*	*97·0*	*90·0*	*83·0*
18 × 6 × 55	16 × 1	221·1	*99·5*	*91·1*	*84·1*	*78·1*	*72·9*
18 × 6 × 55	16 × $\frac{3}{4}$	193·9	84·6	77·5	71·6	66·4	62·0

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET						Moment of Inertia $x—x$	Modulus of Section $x—x$
32	36	40	44	48	52		
70·6	62·8	54·0	44·6			4081	424·0
65·0	58·0	49·3	40·6			3714	390·9
152	*134*	*121*	*111*	*97·7*	*83·6*	10565	918·7
129	*114*	*103*	*93·9*	*78·9*	*67·2*	8524	774·9
105	93·9	84·5	73·3	61·6		6660	634·3
94·3	83·8	75·5	63·9	53·7		5801	566·0
82·7	73·5	66·2	54·7	45·9		4966	496·6
71·4	63·5	55·7	46·0			4179	428·7
65·8	58·5	50·7	41·9			3801	395·0
60·2	53·5	45·7	37·8			3432	361·3
110	*97·9*	*88·2*	*80·1*	*67·3*	*57·3*	7274	661·4
89·1	*79·2*	*71·2*	*61·8*	*51·9*		5613	534·5
78·8	*70·0*	63·0	53·7	44·8		4848	473·0
68·3	60·7	54·7	45·2	37·9		4102	410·2
58·1	51·6	45·3	37·4			3401	348·8

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Two Joists	Flange Plates		18	20	22	24	26
18×6×55	16×⅝	180·3	94·3	84·8	77·2	70·7	65·3
18×6×55	16×½	166·7	85·3	76·7	69·8	63·9	59·1
16×6×62	14×2	316·5	**157**	**141**	*128*	*118*	*109*
16×6×62	14×1½	268·9	*129*	*116*	*105*	*96·9*	*89·4*
16×6×62	14×1¼	245·1	*115*	*103*	*94·5*	*86·6*	*80·0*
16×6×62	14×1	221·3	*101*	*91·5*	*83·0*	*76·1*	70·3
16×6×62	14×¾	197·5	87·9	79·1	71·9	65·9	60·8
16×6×62	14×⅝	185·6	81·1	73·1	66·4	60·8	56·3
16×6×62	14×½	173·7	74·4	67·0	60·9	55·8	51·6
16×6×50	14×2	292·7	All	Riv	ets ¾	dia.	**105**
16×6×50	14×1½	245·1			**100**	**92·4**	*85·3*
16×6×50	14×1¼	221·5	**109**	**98·3**	*89·4*	*81·9*	*75·6*
16×6×50	14×1	197·5	**95·2**	*85·7*	*77·9*	*71·4*	65·9
16×6×50	14×¾	173·8	81·4	73·2	66·7	61·0	56·3
16×6×50	14×⅝	161·8	74·5	67·2	61·1	55·9	51·6

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET						Moment of Inertia $x-x$	Modulus of Section $x-x$
28	30	32	36	40	44		
60·6	56·6	53·0	47·1	41·0	34·0	3064	318·4
54·8	51·1	47·9	42·6	36·4	30·1	2735	287·9
101	*94·6*	*88·6*	*78·8*	*70·9*	*58·6*	5321	532·1
83·0	*77·5*	*72·7*	*64·6*	55·3	45·6	4144	436·2
74·3	*69·3*	65·0	57·7	48·0	39·6	3600	389·2
65·3	60·9	57·1	50·7	41·1	34·1	3085	342·8
56·6	52·7	49·4	42·8	34·6		2597	296·8
52·1	48·6	45·7	38·9	31·6		2365	274·2
47·8	44·6	41·9	35·2	28·5		2137	251·5
97·7	*91·1*	*85·4*	*75·9*	*68·3*	56·4	5128	512·8
79·2	*73·9*	*69·4*	*61·7*	52·6	43·4	3951	415·8
70·2	*65·5*	*61·5*	54·7	45·5	37·4	3407	368·4
61·2	57·1	53·6	47·6	38·5	31·8	2892	321·4
52·3	48·8	45·8	39·5	32·0		2405	274·9
47·9	44·7	41·9	35·8	29·0		2175	251·9

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.

Two Joists	Flange Plates	Weight per foot in lbs.	SPANS 16	18	20	22	24
16×6×50	14×½	149·9	76·2	67·7	61·0	55·5	50·8
15×6×45	14×2	282·7	Riv	ets ¾	dia.		
15×6×45	14×1½	235·1				**91·8**	**84·2**
15×6×45	14×1¼	211·4		**99·2**	**89·3**	*81·1*	*74·4*
15×6×45	14×1	187·5	**96·6**	**85·8**	*77·3*	*70·2*	*64·4*
15×6×45	14×¾	163·8	*82·0*	72·9	65·6	59·6	54·6
15×6×45	14×⅝	151·8	74·7	66·4	59·8	54·4	49·8
15×6×45	14×½	139·9	67·5	60·0	54·0	49·1	45·0
15×5×42	12×2	249·5	Riv. ¾	dia.	**105**	**95·4**	*87·5*
15×5×42	12×1½	208·8	**106**	*94·6*	*85·1*	*77·3*	*70·9*
15×5×42	12×1¼	188·3	*94·0*	*83·5*	*75·3*	*68·3*	*62·6*
15×5×42	12×1	167·9	*81·5*	72·4	65·3	59·2	54·3
15×5×42	12×¾	147·5	69·2	61·5	55·4	50·3	46·1
15×5×42	12×⅝	137·3	63·2	56·2	50·6	45·9	42·1
15×5×42	12×½	127·1	57·1	50·8	45·7	41·5	38·1

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET						Moment of Inertia x—x	Modulus of Section x—x
26	28	30	32	34	36		
46·9	43·5	40·6	38·1	35·8	32·0	1945	228·8
	89·6	83·6	78·4	73·8	69·8	4470	470·7
77·7	72·1	67·4	63·1	59·4	56·1	3411	379·0
68·7	63·8	59·5	55·8	52·5	48·1	2925	334·2
59·4	55·2	51·5	48·3	45·4	40·5	2464	289·8
50·4	46·8	43·7	41·0	37·4	33·5	2030	246·0
45·9	42·6	39·9	37·3	33·8	29·9	1822	224·3
41·6	38·5	36·0	33·7	29·9	26·7	1621	202·6
80·7	75·0	70·0	65·6	61·7	58·3	3744	394·1
65·4	60·8	56·7	53·3	50·0	47·2	2873	319·3
57·8	53·6	50·0	46·9	44·2	40·7	2465	281·7
50·1	46·5	43·4	40·7	38·3	34·2	2079	244·6
42·7	39·5	36·9	34·7	31·6	28·2	1715	207·8
38·8	36·1	33·6	31·5	28·4	25·3	1541	189·7
35·1	32·6	30·4	28·6	25·3	22·5	1372	171·6

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.

Two Joists	Flange Plates	Weight per foot in lbs.	SPANS 16	18	20	22	24
15 × 5 × 42	12 × ⅜	117·0	51·1	45·4	40·9	37·2	34·1
14 × 8 × 70	18 × 2½	449·6	Riv	ets ⅞	dia.		
14 × 8 × 70	18 × 2	388·4					
14 × 8 × 70	18 × 1½	327·2					
14 × 8 × 70	18 × 1¼	296·6				**106**	**97·5**
14 × 8 × 70	18 × 1	266·0		**114**	**103**	*93·0*	86·0
14 × 8 × 70	18 × ¾	235·4	**111**	**98·8**	89·0	81·0	74·0
14 × 8 × 70	18 × ⅝	220·1	**103**	91·0	82·0	74·5	68·3
14 × 8 × 70	18 × ½	204·8	**94·0**	84·0	75·0	68·0	63·0
14 × 6 × 57	14 × 2	306·8	Riv. ¾ dia.		**122**	**111**	*101*
14 × 6 × 57	14 × 1½	259·3	**124**	**110**	*99·8*	*90·7*	*83·2*
14 × 6 × 57	14 × 1¼	235·4	**111**	*98·8*	*89·0*	*80·8*	*74·1*
14 × 6 × 57	14 × 1	211·6	*97·5*	*86·7*	*78·0*	*70·9*	*65·0*
14 × 6 × 57	14 × ¾	187·8	84·1	74·8	67·4	61·2	56·1
14 × 6 × 57	14 × ⅝	175·9	77·5	69·0	62·0	56·4	51·7

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET						Moment of Inertia $x-x$	Modulus of Section $x-x$
26	28	30	32	34	36		
31·4	29·2	27·2	25·1	22·3		1209	153·5
				111	105	6726	708
			100	94·4	*89·2*	5400	600·0
101	94·0	*88·0*	*82·0*	*77·0*	*68·3*	4183	492·1
90·0	*83·0*	*78·0*	73·0	66·5	59·0	3617	438·4
79·0	73·0	69·0	64·0	56·6	50·4	3085	385·6
68·0	64·0	59·0	54·0	47·5		2585	333·6
63·0	58·6	54·7	49·0	43·1		2345	307·5
58·0	54·0	50·0	44·1	38·8		2116	282·1
94·1	*87·5*	*81·5*	*76·4*	*71·9*	*67·9*	4131	459·0
76·8	*71·3*	*66·5*	*62·4*	*58·7*	52·3	3183	374·6
68·4	*63·5*	*59·3*	55·6	50·7	45·2	2749	333·2
60·1	55·7	52·0	48·7	43·1	38·5	2341	292·6
51·7	48·1	44·8	40·7	36·1		1957	252·5
47·7	44·3	41·3	37·0	32·8		1775	232·8

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.

Two Joists	Flange Plates	Weight per foot in lbs.	SPANS 16	18	20	22	24
14 × 6 × 57	14 × ½	164·0	70·9	63·1	56·7	51·6	47·3
14 × 6 × 46	14 × 2	285·1	Riv	ets ¾	dia.		
14 × 6 × 46	14 × 1½	237·5			94·8	86·2	79·1
14 × 6 × 46	14 × 1¼	213·7	105	93·1	83·8	76·1	69·6
14 × 6 × 46	14 × 1	189·9	90·9	80·8	72·7	66·2	60·6
14 × 6 × 46	14 × ¾	166·1	77·3	68·7	61·8	56·2	51·6
14 × 6 × 46	14 × ⅝	154·2	70·6	62·7	56·6	51·3	47·2
14 × 6 × 46	14 × ½	142·3	63·9	56·8	51·2	46·4	42·7
14 × 5½ × 40	12 × 1½	204·7	Rivets ¾" dia.	89·0	80·0	72·7	66·5
14 × 5½ × 40	12 × 1¼	184·3	87·0	78·3	70·5	63·0	58·6
14 × 5½ × 40	12 × 1	163·9	76·0	68·0	61·0	56·0	51·0
14 × 5½ × 40	12 × ¾	143·5	65·0	58·0	52·0	47·0	43·0
14 × 5½ × 40	12 × ⅝	133·3	59·0	53·0	48·0	43·0	40·0
14 × 5½ × 40	12 × ½	123·1	54·0	48·0	43·0	39·0	36·0
14 × 5½ × 40	12 × ⅜	113·0	48·2	42·9	38·6	35·1	32·1

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET						Moment of Inertia $x—x$	Modulus of Section $x—x$
26	28	30	32	34	36		
43·6	40·5	37·8	33·2	29·4		1597	213·0
90·5	84·1	*78·4*	*73·5*	*69·2*	*65·4*	3972	441·4
72·9	*67·8*	*63·2*	*59·2*	*55·8*	49·7	3024	355·8
64·4	*59·9*	*55·8*	*52·3*	47·8	42·6	2590	314·0
55·9	51·9	48·5	45·4	40·2	35·9	2182	272·7
47·5	44·1	41·2	37·5	33·1		1798	232·0
43·4	40·3	37·6	33·8	30·0		1615	211·8
39·3	36·5	34·0	29·9	26·5		1438	191·8
61·5	*57·0*	*53·3*	*50·0*	*47·0*	41·6	2541	298·9
54·0	50·4	47·0	44·0	40·1	35·6	2177	263·9
47·0	44·0	41·0	38·0	33·9	29·9	1834	229·3
40·0	37·0	35·0	31·4	28·0		1512	195·2
37·0	34·0	32·0	28·3	25·2		1359	178·3
33·0	31·0	29·0	25·2	22·4		1211	161·5
29·7	27·5	25·7	24·1			1067	144·7

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Two Joists	Flange Plates		14	16	18	20	22
13×5×35	12×1½	194·7	Riv. ¾	dia.	79·0	71·2	64·6
13×5×35	12×1¼	174·3		78·0	69·2	62·3	56·7
13×5×35	12×1	153·9	76·7	67·1	59·7	53·7	48·8
13×5×35	12×¾	133·6	64·5	56·4	50·2	45·1	41·0
13×5×35	12×⅝	123·3	58·5	51·2	45·5	40·9	37·2
13×5×35	12×½	113·4	52·5	45·9	40·8	36·7	33·4
13×5×35	12×⅜	103·0	46·5	40·7	36·2	32·5	29·6
12×6×54	14×2	300·7	Riv	ets ¾	dia.	104	94·3
12×6×54	14×1½	253·4		105	93·4	84·0	76·3
12×6×54	14×1¼	229·4	107	93·2	82·8	74·6	67·7
12×6×54	14×1	205·6	93·1	81·5	72·5	65·2	59·2
12×6×54	14×¾	181·7	80·1	70·0	62·4	56·0	50·9
12×6×54	14×⅝	169·8	73·6	64·4	57·3	51·5	46·9
12×6×54	14×½	157·9	67·2	58·8	52·3	47·0	42·8
12×6×44	14×2	281·1					

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET						Moment of Inertia $x—x$	Modulus of Section $x—x$
24	26	28	30	32	34		
59·2	*54·7*	*50·8*	*47·4*	*44·5*	39·3	2134	266·7
52·0	*48·0*	*44·5*	41·6	37·6	33·4	1812	233·8
44·7	41·3	38·3	35·8	31·4	27·8	1511	201·5
37·6	34·8	32·2	29·1	25·5		1228	169·4
34·1	31·5	29·2	25·9	22·7		1094	153·6
30·6	28·2	26·2	22·8	20·1		965	137·9
27·1	25·0	22·8	19·9			840	122·2
86·1	*79·9*	*73·9*	*69·0*	*64·7*	*57·3*	3106	388·3
70·0	*64·6*	*60·0*	*56·0*	49·2	*43·6*	2363	315·0
62·1	*57·3*	*53·2*	*48·1*	42·2		2026	279·5
54·3	50·1	46·5	40·5	35·7		1712	244·6
46·7	43·1	38·7	33·6			1419	210·3
43·0	39·8	34·8	30·4			1281	193·4
39·2	36·3	31·2	27·2			1148	176·6
83·9	77·9	72·2	67·5	63·2	55·7	3028	378·5

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Two Joists	Flange Plates		10	12	14	16	18
12×6×44	14×1½	233·5			Rivets ¾		dia.
12×6×44	14×1¼	209·7				88·6	78·7
12×6×44	14×1	185·8			87·5	76·7	*68·1*
12×6×44	14×¾	162·1		86·7	74·2	*65·0*	57·7
12×6×44	14×⅝	150·2	94·8	79·0	*67·7*	59·2	52·7
12×6×44	14×½	138·3	85·7	71·4	61·2	53·5	47·6
12×6×44	14×⅜	126·0	76·6	63·9	54·7	47·9	42·6
12×5×32	12×1½	188·9	Rivets ¾		dia.		71·4
12×5×32	12×1¼	168·5			80·3	70·1	*62·3*
12×5×32	12×1	148·2		80·1	68·7	*60·2*	*53·4*
12×5×32	12×¾	127·7	80·3	66·9	*57·4*	50·2	44·7
12×5×32	12×⅝	117·5	72·5	*60·4*	51·8	45·3	40·2
12×5×32	12×½	107·3	64·7	53·9	46·2	40·4	35·9
12×5×32	12×⅜	97·0	57·0	47·5	40·7	35·6	31·6
10×6×40	14×1½	225·4	Rivets ¾		dia.		

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET						Moment of Inertia $x-x$	Modulus of Section $x-x$
20	22	24	26	28	30		
80·3	73·0	66·9	61·8	57·3	53·6	2260	301·3
70·6	64·3	58·9	54·4	50·6	45·5	1923	265·2
61·2	55·7	51·0	47·1	43·8	38·1	1609	229·8
52·0	47·2	43·3	40·0	35·8	31·3	1316	195·0
47·4	43·1	39·5	36·4	32·0	28·0	1178	177·8
42·8	38·9	35·7	32·9	28·4	24·7	1045	160·7
38·3	34·8	31·9	28·9	24·9		917	143·8
64·3	58·4	53·5	49·4	45·9	42·8	1808	241·1
56·1	51·1	46·7	43·1	40·1	36·1	1526	210·5
48·0	43·7	40·0	36·9	34·4	29·9	1262	180·3
40·1	36·5	33·4	30·9	27·6	24·1	1017	150·7
36·2	32·9	30·2	27·9	24·5	21·4	901	136·0
32·3	29·4	26·9	24·9	21·4	18·7	790	121·5
28·5	25·9	23·7	21·5	18·5		682	107·0
	59·5	54·6	50·3	43·4	37·9	1596	245·5

Above loads based on Extreme Fibre Stress 8Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Two Joists	Flange Plates		8	10	12	14	16
10×6×40	14×1½	201·3					
10×6×40	14×1	177·5					61·7
10×6×40	14×¾	153·7			69·4	59·5	*52·0*
10×6×40	14×⅝	141·8		75·7	63·0	54·0	47·3
10×6×40	14×½	129·9		68·1	56·7	48·6	42·5
10×6×40	14×⅜	118·0	75·7	60·6	50·5	43·2	37·8
10×5×30	12×1½	184·8	Riv	ets ¾	dia.		66·0
10×5×30	12×1	144·0			65·3	56·0	*49·1*
10×5×30	12×¾	123·7	81·3	65·2	*54·3*	*46·5*	40·7
10×5×30	12×⅝	113·4	73·4	58·7	*48·9*	41·9	36·6
10×5×30	12×½	103·2	65·3	52·2	43·5	37·3	32·6
10×5×30	12×⅜	93·0	57·3	45·8	38·2	32·7	28·6
8×6×35	14×1½	215·1	Riv	ets ¾	dia.		
8×6×35	14×1	167·5				53·7	47·0
8×6×35	14×¾	143·7			52·2	44·7	*39·2*

For notes relating to above see page vi.

and Dimensions and Properties

IN FEET						Moment of Inertia x—x	Modulus of Section x—x
18	20	22	24	26	28		
63·8	57·4	*52·2*	*47·8*	*42·5*	*36·6*	1344	215·0
54·9	*49·4*	*44·9*	*41·1*	35·0	30·3	1112	185·3
46·2	41·7	37·8	33·2	28·3		898	156·2
42·0	37·8	34·4	29·7	25·2		799	142·0
37·8	34·0	30·9	26·0	22·1		702	127·8
33·6	30·3	26·9	22·6			611	113·7
58·6	*52·8*	*48·0*	*44·0*	*40·6*	*35·1*	1288	198·1
43·5	*39·2*	*35·6*	32·7	27·8	24·0	882	147·0
36·2	32·6	29·6	26·0	22·1		703	122·3
32·6	29·3	26·7	23·0	19·6		620	110·2
29·0	26·1	23·8	19·9	17·0		539	98·0
25·4	22·9	20·3	17·1			462	86·0
	50·7	46·1	*38·8*	*33·0*		1047	190·4
41·8	*37·6*	*31·2*	*26·1*			706	141·2
34·8	29·7	24·6				559	117·6

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

8 tons/sq. in.		Weight per foot in lbs.	SPANS				
Two Joists	Flange Plates		8	10	12	14	16
8 × 6 × 35	14 × ⅝	131·8		56·6	47·1	40·4	35·3
8 × 6 × 35	14 × ½	119·9		50·5	42·0	36·0	31·5
8 × 6 × 35	14 × ⅜	108·0	55·6	44·5	37·1	31·8	27·8
8 × 5 × 28	12 × 1¼	160·6	Riv	ets ¾	dia.	51·3	44·9
8 × 5 × 28	12 × 1	140·2			51·1	43·8	*38·4*
8 × 5 × 28	12 × ¾	119·8	63·4	50·7	*42·3*	*36·3*	31·7
8 × 5 × 28	12 × ⅝	109·6	57·0	45·6	*38·1*	32·6	28·6
8 × 5 × 28	12 × ½	99·4	50·5	40·4	33·7	28·9	25·3
8 × 5 × 28	12 × ⅜	89·0	44·2	35·3	29·4	25·2	22·1

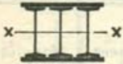
For notes relating to above see page vi.

and Dimensions and Properties

IN FEET						Moment of Inertia $x—x$	Modulus of Section $x—x$
18	20	22	24	26	28		
31·4	26·3	21·6				491	106·2
28·0	22·8	18·7				426	94·7
24·0	19·4					365	83·5
39·9	*35·9*	*31·0*	*26·3*			707	134·7
34·1	*30·7*	25·3	21·3			576	115·1
28·2	24·1	19·9				452	95·3
25·2	21·0	17·4				395	85·5
22·4	18·2	15·0				341	75·9
19·1	15·4					290	66·4

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons



8 tons/sq. in.

Three Joists	Flange Plates	Weight per foot in lbs.	SPANS 24	26	28	30	32
24 × 7½ × 95	24 × 2	618·5	Rivets 7/8dia.	**310**	**288**	*269*	*252*
24 × 7½ × 95	24 × 1½	535·0	**283**	*261*	*242*	*226*	212
24 × 7½ × 95	24 × 1	454·5	230	212	197	184	173
24 × 7½ × 95	24 × ½	372·0	178	164	153	142	133
22 × 7 × 75	24 × 2	559·5	Rivets 7/8dia.	**264**	**246**	*229*	*215*
22 × 7 × 75	24 × 1½	475·0	**236**	*216*	*202*	*189*	*177*
22 × 7 × 75	24 × 1	393·5	187	173	161	150	141
22 × 7 × 75	24 × ½	312·0	139	128	119	111	104
20 × 6½ × 65	22 × 2	502·5	Rivets 7/8 dia.		**200**	**186**	*174*
20 × 6½ × 65	22 × 1½	425·0	**192**	*177*	*164*	*153*	144
20 × 6½ × 65	22 × 1	350·0	151	140	130	121	114
20 × 6½ × 65	22 × ½	275·0	112	103	96·0	89·4	84·0
18 × 8 × 80	26 × 2	601·5	Riv	ets ⅞	dia.	**207**	**194**
18 × 8 × 80	26 × 1½	513·5	**214**	**198**	*184*	*171*	*161*
18 × 8 × 80	26 × 1	422·0	171	158	146	137	128

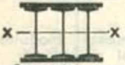
For notes relating to above see page vi.

and Dimensions and Properties

IN FEET						Moment of Inertia $x—x$	Modulus of Section $x—x$
34	36	38	40	42	44		
237	*224*	*213*	*202*	*192*	*183*	21221	**1516**
200	189	179	170	162	155	17214	**1275**
162	153	145	138	131	125	13494	**1038**
126	119	113	107	102	97·6	10049	**804**
132	*191*	*181*	*172*	*164*	*156*	16747	**1288**
167	158	149	142	135	129	13302	**1064**
132	125	118	112	107	102	10123	**843**
98·3	92·7	87·8	83·4	79·5	75·9	7197	**626**
164	*155*	*147*	*140*	133	127	12576	**1048**
135	128	121	115	110	105	9926	**863**
107	101	95·7	90·9	86·6	82·6	7497	**682**
78·9	74·5	70·6	67·0	63·8	58·2	5280	**503**
183	*172*	*163*	*155*	*148*	*141*	12806	**1164**
151	143	135	129	122	112	10126	**964**
121	114	108	103	93·0	84·6	7689	**769**

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons



8 tons/sq. in. Three Joists	Flange Plates	Weight per foot in lbs.	SPANS 24	26	28	30	32
18 × 6 × 55	20 × 1½	374·5	*151*	*139*	129	121	113
18 × 6 × 55	20 × 1	306·5	118	109	101	94·8	88·7
18 × 6 × 55	20 × ½	238·5	86·2	79·6	73·9	69·0	64·7
16 × 8 × 75	26 × 2	586·5					**170**
16 × 8 × 75	26 × 1½	498·5		**172**	**160**	*149*	*140*
16 × 8 × 75	26 × 1	407·0	148	136	127	118	111
16 × 6 × 50	20 × 1½	359·5	*131*	*120*	112	104	97·9
16 × 6 × 50	20 × 1	291·5	101	93·6	87·0	81·2	76·1
16 × 6 × 50	20 × ½	223·5	73·0	67·4	62·6	58·4	54·8
15 × 6 × 45	20 × 1½	344·5	**119**	*110*	102	95·0	89·1
15 × 6 × 45	20 × 1	276·5	91·4	84·3	78·3	73·1	68·5
15 × 6 × 45	20 × ½	208·5	64·6	59·6	55·4	51·7	48·4
14 × 8 × 70	26 × 1½	483·5		**147**	**137**	*127*	*119*
14 × 8 × 70	26 × 1	395·0	125	116	108	100	94·1
						Ri	vets

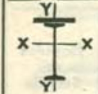
For notes relating to above see page vi.

and Dimensions and Properties

IN FEET						Moment of Inertia $x—x$	Modulus of Section $x—x$
34	36	38	40	42	44		
106	101	95·3	90·5	86·2	78·4	7128	679
83·5	78·8	74·7	71·0	64·6	58·5	5322	532
60·9	57·5	54·5	49·0	44·8		3687	388
160	*151*	*143*	*136*	*123*	*112*	10176	1018
132	124	118	106	96·8		7971	839
104	98·5	88·1	79·4			5987	665
92·1	87·0	82·4	74·0	67·8		5580	587
71·6	67·6	60·6	54·5			4109	456
51·5	45·6	41·2				2792	328
83·8	79·2	71·0	63·9			4811	534
64·5	57·1	51·5				3495	411
42·9	38·0					2325	291
112	99·5	89·8				6094	717
83·3	73·6					4515	564
all $\frac{7}{8}$ dia.							

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

		Weight per foot in lbs.	8 tons/sq. in.			SPANS	
Joist	Top Flange Plate		14	16	18	20	24
24 × 7½ × 95	14 × ⅝	126·0	89·4	78·3	69·6	62·6	52·2
24 × 7½ × 95	12 × ⅝	121·9	88·4	77·4	68·8	61·9	51·6
24 × 7½ × 95	12 × ½	116·8	86·9	76·1	67·6	60·9	50·7
24 × 7½ × 95	12 × ⅜	111·7	85·2	74·6	66·2	59·7	49·8
24 × 7½ × 95	10 × ⅜	109·0	84·4	73·8	65·6	59·1	49·2
22 × 7 × 75	12 × ⅝	101·9	65·0	56·9	50·6	45·5	37·9
22 × 7 × 75	12 × ½	96·8	63·9	55·9	49·7	44·7	37·3
22 × 7 × 75	12 × ⅜	91·7	62·6	54·8	48·6	43·8	36·5
22 × 7 × 75	10 × ⅜	89·0	61·7	54·1	48·0	43·2	36·0
20 × 6½ × 65	12 × ⅝	91·9	52·5	46·0	40·9	36·8	30·6
20 × 6½ × 65	12 × ½	86·8	51·6	45·2	40·2	36·1	30·1
20 × 6½ × 65	12 × ⅜	81·7	50·5	44·3	39·4	35·3	29·4
20 × 6½ × 65	10 × ⅜	79·0	49·9	43·6	38·8	34·9	29·1
18 × 6 × 55	12 × ⅜	71·0	39·0	34·0	30·3	27·3	22·7
18 × 6 × 55	10 × ½	73·4	39·3	34·4	30·6	27·5	22·9

For notes relating to above see page x.

Plate top flange only and Dimensions and Properties

IN FEET						Nett Moment of Inertia $x-x$	Moduli of Section	
28	32	36	40	44	48		Axis $x-x$ Min.	Axis $y-y$ Top Flange
44·7	39·1	34·8	31·4	28·5	26·2	3419	**234·9**	22·8
44·2	38·7	34·4	31·0	28·1	25·8	3294	**232·1**	17·7
43·5	38·0	33·8	30·4	27·7	25·4	3140	**228·2**	14·9
42·7	37·3	33·1	29·8	27·2	24·8	2976	**224·1**	12·1
42·3	36·9	32·8	29·6	26·9	24·6	2885	**221·7**	9·9
32·5	28·5	25·3	22·8	20·7	19·0	2300	**170·7**	16·5
31·9	28·0	24·8	22·4	20·3	18·6	2181	**167·7**	13·7
31·3	27·4	24·4	22·0	20·0	18·2	2052	**164·1**	10·8
30·8	27·0	24·0	21·6	19·7	18·0	1980	**162·1**	8·4
26·3	23·0	20·5	18·4	16·7	15·4	1729	**137·9**	16·1
25·8	22·6	20·1	18·1	16·4	15·0	1636	**135·5**	13·2
25·2	22·1	19·7	17·7	16·1	14·2	1533	**132·6**	10·4
24·9	21·8	19·4	17·5	15·9	13·7	1475	**131·0**	7·9
19·5	17·0	15·0	13·7	12·0	10·1	1094	**102·4**	10·0
19·6	17·2	15·3	13·7	12·3	10·3	1113	**103·1**	9·4

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

Joist	Top Flange Plate	Weight per foot in lbs.	8 tons/sq. in. SPANS 14	16	18	20	22
18 × 6 × 55	10 × $\frac{3}{8}$	68·8	38·4	33·6	30·0	26·9	24·5
18 × 6 × 55	9 × $\frac{3}{8}$	67·5	38·2	33·4	29·7	26·7	24·3
16 × 6 × 50	12 × $\frac{3}{8}$	66·0	32·2	28·2	25·1	22·6	20·4
16 × 6 × 50	10 × $\frac{1}{2}$	68·4	32·5	28·5	25·3	22·8	20·7
16 × 6 × 50	10 × $\frac{3}{8}$	63·5	31·9	27·9	24·8	22·3	20·2
16 × 6 × 50	9 × $\frac{3}{8}$	62·5	31·6	27·7	24·6	22·1	20·1
15 × 6 × 45	12 × $\frac{3}{8}$	61·0	27·5	24·1	21·4	19·3	17·5
15 × 6 × 45	10 × $\frac{1}{2}$	63·4	27·8	24·3	21·6	19·4	17·6
15 × 6 × 45	10 × $\frac{3}{8}$	58·5	27·3	23·8	21·2	19·1	17·3
15 × 6 × 45	9 × $\frac{3}{8}$	57·5	27·0	23·6	21·0	18·9	17·2
15 × 5 × 42	9 × $\frac{5}{8}$	62·1	25·2	22·0	19·6	17·6	16·0
15 × 5 × 42	9 × $\frac{1}{2}$	58·3	24·6	21·5	19·1	17·2	15·7
15 × 5 × 42	9 × $\frac{3}{8}$	54·5	24·0	21·0	18·7	16·8	15·3
14 × 5½ × 40	10 × $\frac{5}{8}$	62·3	23·6	20·6	18·3	16·5	15·0
14 × 5½ × 40	10 × $\frac{1}{2}$	58·0	23·1	20·2	18·0	16·2	14·7

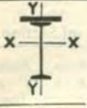
For notes relating to above see page x.

Plate top flange only and Dimensions and Properties

IN FEET						Nett Moment of Inertia $x—x$	Moduli of Section	
							Axis $x—x$ Min.	Axis $y—y$ Top Flange
24	26	28	30	32	36			
22·4	20·7	19·2	17·9	16·8	15·0	1051	101·1	7·5
22·3	20·5	19·1	17·8	16·7	14·8	1028	100·4	6·4
18·6	17·3	16·1	15·0	14·1	12·5	817	84·8	10·0
19·0	17·5	16·3	15·2	14·2	12·7	831	85·4	9·3
18·6	17·1	15·9	14·8	13·9	12·5	783	83·7	7·4
18·4	17·0	15·8	14·7	13·8	12·3	765	83·1	6·3
16·0	14·8	13·7	12·8	12·0	10·7	665	72·4	9·8
16·2	15·0	13·9	13·0	12·1	10·8	678	72·9	9·1
15·9	14·7	13·6	12·7	12·0	10·5	638	71·6	7·2
15·7	14·5	13·5	12·6	11·8	10·2	622	71·0	6·1
14·7	13·6	12·6	11·8	11·0	9·8	637	66·1	8·8
14·4	13·3	12·3	11·5	10·8	9·7	598	64·6	7·2
14·0	12·9	12·0	11·2	10·5	9·2	557	63·0	5·6
13·8	12·7	11·8	11·0	10·3	9·2	574	61·9	10·6
13·5	12·5	11·6	10·8	10·1	8·8	539	60·7	8·7

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons



Joist	Top Flange Plate	Weight per foot in lbs.	8 tons/sq. in. SPANS 6	8	10	12	14
$14 \times 5\frac{1}{2} \times 40$	$10 \times \frac{3}{8}$	53·7	52·7	39·6	31·7	26·4	22·6
$13 \times 5 \times 35$	$12 \times \frac{3}{8}$	51·0	43·7	32·7	26·2	21·8	18·7
$13 \times 5 \times 35$	$9 \times \frac{1}{2}$	51·3	44·0	33·0	26·3	21·9	18·8
$13 \times 5 \times 35$	$9 \times \frac{3}{8}$	47·5	42·7	32·0	25·7	21·4	18·4
$12 \times 5 \times 32$	$12 \times \frac{3}{8}$	48·0	37·0	27·9	22·3	18·6	15·9
$12 \times 5 \times 32$	$10 \times \frac{3}{8}$	45·4	36·7	27·3	21·9	18·1	15·5
$12 \times 5 \times 32$	$9 \times \frac{3}{8}$	44·5	36·4	27·3	21·9	18·2	15·6
$10 \times 4\frac{1}{2} \times 25$	$10 \times \frac{3}{8}$	38·5	24·6	18·5	14·8	12·3	10·5
$10 \times 4\frac{1}{2} \times 25$	$9 \times \frac{3}{8}$	37·5	24·4	18·3	14·6	12·2	10·4
$9 \times 4 \times 21$	$10 \times \frac{3}{8}$	34·5	18·6	13·9	11·1	9·3	8·0
$9 \times 4 \times 21$	$9 \times \frac{3}{8}$	33·5	18·4	13·8	11·0	9·2	7·8
$8 \times 5 \times 28$	$10 \times \frac{3}{8}$	41·5	22·4	16·8	13·4	11·2	9·6
$8 \times 5 \times 28$	$9 \times \frac{3}{8}$	40·5	22·2	16·6	13·3	11·1	9·5

All Rivets $\frac{3}{4}$ dia. for 4″ pitch on pages 72 to 73

For notes relating to above see page x.

Plate top flange only and Dimensions and Properties

IN FEET						Nett Moment of Inertia $x—x$	Moduli of Section	
16	18	20	22	24	28		Axis $x—x$ Min.	Axis $y—y$ Top Flange
19·8	17·6	15·8	14·4	13·2	11·3	503	59·3	6·7
16·2	14·5	13·0	11·9	10·9	9·3	408	49·2	9·4
16·4	14·6	13·1	12·0	11·0	9·4	408	49·3	7·2
16·1	14·3	12·9	11·7	10·7	9·2	378	48·2	5·6
13·9	12·4	11·0	10·0	9·0	7·9	325	41·9	9·3
13·7	12·2	11·1	10·1	9·2	7·9	310	41·3	6·6
13·7	12·2	10·9	9·9	9·1	7·8	302	41·0	5·5
9·2	8·2	7·4	6·7	6·2	4·9	181	27·8	6·4
9·1	8·1	7·3	6·6	6·1	4·7	176	27·6	5·3
6·9	6·2	5·5	5·0	4·6	3·5	128	21·0	6·4
6·9	6·0	5·5	4·9	4·6	3·4	124	20·8	5·2
8·4	7·4	6·7	5·7	4·8		129	25·2	6·7
8·3	7·4	6·6	5·5	4·6		126	25·0	5·5

Girders, pages 72 to 77
and 6″ pitch pages 74 to 77

Above loads based on Extreme Fibre Stress 8 Tons/sq. in.

CHANNELS—Safe Distributed Loads

X—[—X 8 tons /sq.in. Section	Weight per foot in lbs.	8 tons/sq. in. SPANS					
		12	14	16	18	20	22
17×4	51·28	29·8 *20·9*	25·6 *15·0*	22·3 *10·6*	19·8	17·8	16·2
17×4	44·34	27·2 *19·0*	23·3 *13·7*	20·4 *9·7*	18·2	16·3	14·8
15×4	42·49	22·6 *15·8*	19·4 *11·4*	17·0 *8·1*	15·1	13·6	12·3
15×4	36·37	20·6 *14·4*	17·8 *10·5*	15·6 *7·4*	13·7	12·4	11·2
13×4	38·92	18·5 *13·0*	15·8 *9·3*	13·8 *6·6*	12·3	11·1	10·0
13×4	33·18	16·8 *11·8*	14·4 *8·5*	12·6 *6·0*	11·2	10·1	9·3
12×4	36·63	16·2 *11·3*	13·8 *8·1*	12·1 *5·8*	10·8	9·7	8·8
12×4	31·33	14·8 *10·4*	12·7 *7·5*	11·1 *5·3*	9·8	8·8	8·0
12×3½	30·45	12·8 *7·7*	11·0 *5·2*	9·6	8·5	7·7	7·0
12×3½	26·37	11·8 *7·1*	10·1 *4·8*	8·8	7·8	7·0	6·4
11×3½	30·52	12·3 *7·4*	10·5 *5·0*	9·2	8·2	7·4	6·7
*11×3½	29·82	12·0 *7·2*	10·3 *4·9*	9·0	8·0	7·2	6·6
11×3½	26·78	11·4 *6·8*	9·8 *4·7*	8·6	7·6	6·8	6·2
*10×4	18·86	7·3 *5·1*	6·3 *3·7*	5·5 *2·6*	4·9	4·4	3·6
10×3½	28·54	10·6 *6·4*	9·1 *4·3*	7·9	7·0	6·3	5·2

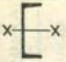
The lower figures in italics are safe loads for beams not tied laterally, i.e. where l/b exceeds 20. For general notes see page vi.

in Tons and Dimensions and Properties

IN FEET					Standard Thicknesses		Moment of Inertia	Modulus of Section
24	26	28	32	34	Web	Flange	x—x	x—x
14·8	13·7	12·7	11·2	10·5	·60	·68	569·3	66·98
13·6	12·6	11·6	10·2	9·6	·48	·68	520·2	61·20
11·3	10·4	9·7	7·9	7·0	·53	·62	382·8	51·05
10·3	9·5	8·8	7·2	6·4	·41	·62	349·2	46·55
9·2	8·5	7·3	5·6	5·0	·53	·62	270·7	41·64
8·4	7·7	6·7	5·1	4·6	·40	·62	246·8	37·98
8·1	6·9	5·9	4·6	4·0	·53	·60	218·8	36·47
7·4	6·3	5·4	4·2	3·7	·40	·60	200·1	33·35
6·4	5·4	4·7	3·6	3·2	·48	·50	174·1	29·02
5·9	5·0	4·3	3·3	2·9	·38	·50	159·7	26·62
5·6	4·8	4·2	3·2	2·8	·48	·58	152·9	27·81
5·5	4·7	4·0	3·1	2·7	·48	·58	148·6	27·02
5·2	4·4	3·9	3·0	2·6	·38	·58	141·9	25·80
3·1	2·6	2·2	1·7	1·5	·31	·31	82·6	16·52
4·4	3·8	3·3	2·5	2·2	·48	·56	119·5	23·90

All above loads based on Extreme Fibre Stress 8 Tons/sq. in.

CHANNELS—Safe Distributed Loads

 8 tons/sq. in. Section	Weight per foot in lbs.	8 tons/sq. in. SPANS 6	7	8	9	10	12
10 × 3½	24·46	19·5 *19·3*	16·7 *15·4*	14·6 *12·6*	13·0 *10·4*	11·6 *8·6*	9·7 *5·8*
10 × 3	21·33	15·6 *14·4*	13·3 *11·3*	11·7 *9·1*	10·4 *7·3*	9·3 *5·8*	7·7 *3·7*
10 × 3	19·28	14·7 *13·6*	12·6 *10·7*	11·0 *8·5*	9·8 *6·9*	8·8 *5·5*	7·3 *3·5*
9 × 3½	25·63	17·6 *17·4*	15·1 *14·0*	13·2 *11·4*	11·7 *9·4*	10·5 *7·7*	8·8 *5·3*
9 × 3½	23·49	16·8 *16·6*	14·4 *13·3*	12·6 *10·9*	11·2 *9·0*	10·0 *7·4*	8·4 *5·0*
9 × 3½	22·27	16·3 *16·1*	14·0 *13·0*	12·2 *10·5*	10·9 *8·7*	9·7 *7·2*	8·1 *4·9*
9 × 3	19·91	13·3 *12·3*	11·4 *9·7*	10·0 *7·8*	8·9 *6·2*	7·9 *4·9*	6·6 *3·1*
9 × 3	17·46	12·4 *11·5*	10·6 *9·0*	9·2 *7·1*	8·2 *5·7*	7·4 *4·6*	6·1 *2·9*
8 × 3½	23·20	14·5 *14·3*	12·4 *11·5*	10·8 *9·3*	9·6 *7·7*	8·7 *6·4*	7·2 *4·3*
8 × 3½	20·21	13·4 *13·2*	11·6 *10·7*	10·0 *8·6*	8·9 *7·1*	8·0 *5·9*	6·7 *4·0*
8 × 3	18·68	11·3 *10·5*	9·7 *8·3*	8·5 *6·6*	7·5 *5·3*	6·8 *4·3*	5·6 *2·7*
8 × 3	15·96	10·3 *9·5*	8·8 *7·5*	7·7 *6·0*	6·9 *4·8*	6·2 *3·9*	5·1 *2·4*
7 × 3½	20·18	11·4 *11·3*	9·8 *9·1*	8·5 *7·3*	7·6 *6·1*	6·8 *5·0*	5·7 *3·4*
7 × 3½	18·28	10·8 *10·7*	9·3 *8·6*	8·1 *7·0*	7·2 *5·8*	6·5 *4·8*	5·4 *3·2*
7 × 3	17·07	9·1 *8·4*	7·8 *6·6*	6·8 *5·3*	6·1 *4·3*	5·5 *3·4*	4·5 *2·1*

The lower figures in italics are safe loads for beams not tied laterally, i.e. where l/b exceeds 20. For general notes see page vi.

in Tons and Dimensions and Properties

IN FEET					Standard Thicknesses		Moment of Inertia	Modulus of Section
14	16	18	20	22	Web	Flange	x—x	x—x
8·3 3·9	7·3	6·4	5·8	4·8	·36	·56	109·5	21·90
6·6	5·8	5·1	4·6	3·8	·38	·45	87·7	17·53
6·2	5·5	4·8	4·4	3·6	·32	·45	82·7	16·53
7·5 3·6	6·6	5·8	4·7	3·9	·45	·54	89·3	19·84
7·2 3·4	6·3	5·6	4·5	3·7	·38	·54	85·1	18·90
6·9 3·3	6·1	5·4	4·4	3·6	·34	·54	82·6	18·36
5·7	4·9	4·4	3·5	2·9	·38	·44	67·4	14·97
5·2	4·6	4·2	3·3	2·9	·30	·44	62·5	13·89
6·2 3·0	5·4	4·2	3·4	2·9	·43	·52	65·3	16·32
5·7 2·7	5·0	3·9	3·2	2·7	·32	·52	60·6	15·14
4·8	4·2	3·3	2·7	2·2	·38	·44	51·0	12·75
4·4	3·8	3·0	2·4	2·1	·28	·44	46·7	11·68
4·9 2·3	3·7	2·9	2·4	2·0	·38	·50	45·2	12·89
4·6 2·2	3·5	2·8	2·3	1·9	·30	·50	42·8	12·24
3·9	3·0	2·1	1·9	1·6	·38	·42	36·2	10·34

All above loads based on Extreme Fibre Stress 8 Tons/sq. in.

CHANNELS—Safe Distributed Loads

Section (8 tons/sq. in.)	Weight per foot in lbs.	8 tons/sq. in. SPANS 4	5	6	7	8	9
7 × 3	14·22	12·5 *12·5*	9·9 *9·9*	8·3 *7·7*	7·1 *6·0*	6·2 *4·8*	5·5 *3·9*
*7 × 2⅛	9·75	7·8 *7·4*	6·2 *5·2*	5·2 *3·8*	4·5 *2·9*	3·9 *2·1*	3·5
6 × 3½	16·48	12·8 *12·8*	10·2 *10·2*	8·5 *8·4*	7·3 *6·8*	6·4 *5·5*	5·7 *4·6*
6 × 3	17·53	12·0 *12·0*	9·6 *9·6*	8·0 *7·4*	6·9 *5·9*	6·0 *4·7*	5·3 *3·7*
6 × 3	16·51	11·6 *11·6*	9·3 *9·3*	7·7 *7·1*	6·6 *5·6*	5·8 *4·5*	5·1 *3·6*
6 × 3	13·64	9·9 *9·9*	7·9 *7·9*	6·6 *6·1*	5·6 *4·8*	4·9 *3·8*	4·4 *3·1*
6 × 3	12·41	9·4 *9·4*	7·5 *7·5*	6·3 *5·8*	5·4 *4·6*	4·7 *3·6*	4·2 *2·9*
5 × 2½	11·24	6·6 *6·6*	5·3 *4·9*	4·4 *3·7*	3·8 *2·9*	3·3 *2·2*	2·9 *1·6*
5 × 2½	10·22	6·3 *6·3*	5·0 *4·6*	4·3 *3·6*	3·6 *2·7*	3·1 *2·0*	2·8 *1·6*
4 × 2	7·91	3·5 *3·2*	2·8 *2·3*	2·3 *1·6*	2·0 *1·2*	1·7 *·81*	1·4
4 × 2	7·09	3·3 *3·1*	2·6 *2·1*	2·2 *1·5*	1·9 *1·1*	1·6 *·76*	1·3
*3½ × 2	6·75	2·8 *2·6*	2·3 *1·9*	1·9 *1·3*	1·6 *·94*	1·2 *·67*	·97
*3 × 1½	5·27	1·8 *1·4*	1·4 *·88*	1·2 *·57*	·87	·66	·53
3 × 1½	5·11	1·7 *1·3*	1·3 *·81*	1·1 *·52*	·84	·64	·50
3 × 1½	4·60	1·6 *1·2*	1·3 *·81*	1·0 *·48*	·79	·61	·47

The lower figures in italics are safe loads for beams not tied laterally, i.e. where l/b exceeds 20. For general notes see page vi.

in Tons and Dimensions and Properties

IN FEET					Standard Thicknesses		Moment of Inertia	Modulus of Section
10	11	12	14	16	Web	Flange	x—x	x—x
4·9 *3·1*	4·6 *2·5*	4·1 *2·0*	3·5	2·7	·26	·42	32·8	9·36
3·1	2·8	2·6	2·2	1·7	·23	·33	20·5	5·85
5·1 *3·8*	4·6 *3·1*	4·2 *2·5*	3·2 *1·7*	2·4	·28	·48	28·9	9·63
4·8 *3·0*	4·3 *2·4*	4·0 *1·9*	2·9	2·2	·43	·48	27·2	9·06
4·6 *2·9*	4·2 *2·3*	3·8 *1·8*	2·8	2·1	·38	·48	26·3	8·76
3·9 *2·4*	3·6 *2·0*	3·3 *1·6*	2·4	1·8	·31	·38	22·4	7·45
3·7 *2·3*	3·4 *1·9*	3·1 *1·5*	2·3	1·7	·25	·38	21·3	7·09
2·6 *1·2*	2·2	1·8	1·4	1·0	·31	·38	12·5	5·00
2·5 *1·2*	2·0	1·7	1·3	·99	·25	·38	11·9	4·76
1·1	·95	·80	·59	·45	·30	·31	5·4	2·69
1·0	·90	·76	·56	·43	·24	·31	5·1	2·53
·79	·65	·55	·40	·31	·25	·31	3·7	2·12
·43	·35	·30	·22	·17	·25	·31	2·0	1·33
·41	·34	·28	·21	·16	·25	·28	1·9	1·29
·38	·32	·27	·20	·15	·20	·28	1·8	1·22

All above loads based on Extreme Fibre Stress 8 Tons/sq. in.

JOISTS—Safe Distributed Loads in

x — x 10 tons/sq. in. Section	Area in sq. ins.	10 tons/sq. in. SPANS 12	14	16	18	20	22
*24 × 7½ × 100	29·40	123 *123*	105 *99·8*	92·2 *81·1*	81·9 *65·5*	73·7 *54·5*	66·9 *44·2*
24 × 7½ × 95	27·94	117 *117*	100 *95·0*	87·9 *77·4*	78·0 *62·4*	70·4 *52·1*	64·0 *42·2*
22 × 7 × 75	22·06	84·7 *83·9*	72·5 *66·0*	63·6 *53·4*	56·5 *42·9*	50·9 *34·6*	46·1 *28·1*
20 × 7½ × 89	26·19	92·9 *92·9*	79·7 *75·7*	69·5 *61·2*	61·9 *49·5*	55·7 *41·2*	50·7 *33·5*
20 × 6¼ × 65	19·12	68·0 *64·6*	58·5 *50·9*	51·0 *40·3*	45·5 *31·9*	40·7 *25·2*	37·1 *20·0*
18 × 8 × 80	23·53	79·7 *79·7*	68·4 *67·0*	59·7 *54·3*	53·2 *44·7*	47·7 *37·2*	43·5 *30·9*
18 × 7 × 75	22·09	71·0 *70·3*	60·9 *55·4*	53·2 *44·7*	47·4 *36·0*	42·6 *29·0*	38·7 *23·6*
18 × 6 × 55	16·18	51·9 *47·2*	44·6 *36·6*	38·9 *28·4*	34·6 *22·1*	31·1 *17·4*	28·2 *13·3*
16 × 8 × 75	22·06	67·6 *67·6*	57·9 *56·7*	50·6 *46·0*	45·0 *37·8*	40·6 *31·7*	36·8 *26·1*
16 × 6 × 62	18·21	50·3 *45·8*	43·1 *35·3*	37·8 *27·6*	33·6 *21·5*	30·2 *16·9*	27·4 *12·9*
16 × 6 × 50	14·71	42·9 *39·0*	36·7 *30·1*	32·1 *23·4*	28·6 *18·3*	25·8 *14·4*	23·4 *11·0*
*15 × 6 × 59	17·35	46·6 *42·4*	39·9 *32·7*	35·0 *25·6*	30·9 *19·8*	28·0 *15·7*	25·4 *11·9*
15 × 6 × 45	13·24	36·4 *33·1*	31·3 *25·7*	27·3 *19·9*	24·2 *15·5*	21·7 *12·2*	19·8 *9·3*
15 × 5 × 42	12·36	31·7 *25·7*	27·3 *19·1*	23·8 *14·0*	21·1 *10·3*	19·0 *7·2*	17·4

The lower figures in italics are safe loads for beams not tied laterally, i.e. where l/b exceeds 20. The zig-zag line indicates the

Tons and Dimensions and Properties

IN FEET								Moment of Inertia	Modulus of Section
24	26	28	32	36	40	44	48	x—x	x—x
61·2 *36·1*	56·7 *29·5*	52·6 *23·7*	46·1	41·0	35·4	29·1	24·5	2655	221·0
58·6 *34·6*	54·1 *28·1*	50·1 *22·5*	43·9	38·9	33·7	27·8	23·3	2533	211·1
42·2 *22·4*	39·0 *17·9*	36·2 *13·8*	31·9	27·6	22·3	18·5	15·5	1677	152·4
46·5 *27·4*	43·0 *22·4*	39·7 *17·9*	34·7	27·5	22·3	18·4	15·5	1673	167·3
34·0 *15·6*	31·5 *12·0*	29·1	25·5	20·1	16·3	13·5	11·4	1226	122·6
39·7 *25·8*	36·7 *21·3*	34·1 *17·4*	26·9 *11·4*	21·4	17·2	14·2	12·0	1292	143·6
35·6 *18·9*	32·7 *15·0*	30·4 *11·6*	23·9	19·0	15·3	12·7	10·7	1151	127·9
26·0 *9·9*	23·9	22·2	17·5	13·8	11·2	9·3	7·8	842	93·5
33·8 *22·0*	30·7 *18·2*	26·5 *14·9*	20·3 *9·6*	16·0	13·0	10·7	9·0	974	121·7
25·1 *9·5*	22·8	19·7	15·1	11·9	9·7	8·0	6·7	725	90·6
21·4 *8·1*	19·5	16·8	12·9	10·2	8·2	6·8	5·7	618	77·3
23·3 *8·9*	19·9	17·0	13·1	10·4	8·4	6·9	5·8	629	83·8
18·2 *6·9*	15·5	13·3	10·3	8·1	6·6	5·4	4·6	492	65·6
15·8	13·5	11·6	8·9	7·0	5·7	4·7	4·0	428	57·1

ratio of span to depth of 19·15. The loads to right of this line are reduced to keep the deflection within 1/325th of the span.

For general notes see page vi.

JOISTS—Safe Distributed Loads in

10 tons/sq. in.	Area in sq. ins.	10 tons/sq. in.					SPANS
Section		5	6	7	8	10	12
14×8 ×70	20·59				84·0 *84·0*	67·1 *67·1*	55·9 *55·9*
14×6 ×57	16·78	102 *102*	84·6 *84·6*	72·6 *72·6*	63·5 *63·5*	50·7 *50·7*	42·3 *38·5*
14×6 ×46	13·59		70·2 *70·2*	60·2 *60·2*	52·2 *52·2*	42·0 *42·0*	35·0 *31·9*
*14×5½×40	11·77	71·9 *71·9*	59·9 *59·9*	51·3 *51·3*	45·0 *45·0*	35·9 *34·5*	30·0 *25·8*
13×5 ×35	10·30	58·1 *58·1*	48·4 *48·4*	41·5 *41·5*	36·5 *36·5*	29·0 *26·4*	24·2 *19·6*
12×8 ×65	19·12				67·6 *67·6*	54·2 *54·2*	45·1 *45·1*
12×6 ×54	15·89	83·4 *83·4*	69·6 *69·6*	59·6 *59·6*	52·2 *52·2*	41·7 *41·7*	34·7 *31·6*
12×6 ×44	13·00	70·4 *70·4*	58·7 *58·7*	50·2 *50·2*	43·7 *43·7*	35·1 *35·1*	29·3 *26·7*
*12×5 ×39	11·47	58·0 *58·0*	48·3 *48·3*	41·4 *41·4*	36·2 *36·2*	29·0 *26·4*	24·1 *19·5*
12×5 ×32	9·45	49·0 *49·0*	40·9 *40·9*	35·0 *35·0*	30·5 *30·5*	24·5 *22·3*	20·4 *16·5*
*12×5 ×30	8·83	46·0 *46·0*	38·3 *38·3*	32·8 *32·8*	28·7 *28·7*	23·0 *20·9*	19·1 *15·5*
*10×8 ×70	20·60		76·7 *76·7*	65·7 *65·7*	57·5 *57·5*	46·0 *46·0*	38·5 *38·5*
10×8 ×55	16·18			55·0 *55·0*	47·9 *47·9*	38·4 *38·4*	32·0 *32·0*
*10×6 ×42	12·35	56·4 *56·4*	47·0 *47·0*	40·3 *40·3*	35·2 *35·2*	28·2 *28·2*	23·5 *21·4*

The lower figures in italics are safe loads for beams not tied laterally, i.e. where l/b exceeds 20. The zig-zag line indicates the

Tons and Dimensions and Properties

IN FEET								Moment of Inertia	Modulus of Section
14	16	18	20	22	24	26	28	x—x	x—x
48·0	41·9	37·3	33·6	30·5	26·0	22·0	19·1	705·6	100·8
47·0	*38·1*	*31·3*	*26·2*	*21·7*	*18·2*	*15·0*	*12·2*		
36·1	31·6	28·2	25·3	23·0	19·5	16·6	14·4	533·3	76·2
29·6	*23·1*	*18·0*	*14·2*	*10·8*	*8·1*				
30·0	26·3	23·4	21·0	19·0	16·1	13·7	12·0	442·6	63·2
24·6	*19·2*	*15·0*	*11·8*	*8·9*	*6·7*				
25·5	22·5	20·0	18·0	16·4	14·0	11·9	10·3	377·1	53·9
19·6	*15·1*	*11·4*	*8·6*	*6·2*					
20·7	18·1	16·1	14·5	12·3	10·4	8·9	7·7	283·5	43·6
14·5	*10·7*	*7·9*	*5·5*						
38·7	33·8	30·1	25·9	21·4	18·1	15·4	13·3	487·8	81·3
37·9	*30·8*	*25·3*	*21·0*	*17·5*	*14·7*	*12·1*	*9·8*		
29·8	26·0	23·1	19·9	16·4	13·9	11·9	10·2	375·8	62·6
24·4	*19·0*	*14·8*	*11·7*	*9·0*	*6·6*				
25·1	21·9	19·5	16·7	13·8	11·7	10·0	6·6	316·8	52·8
20·6	*16·0*	*12·5*	*9·8*	*7·5*	*5·6*				
20·6	18·1	16·1	14·0	11·5	9·7	8·2	7·1	260·9	43·5
14·4	*10·7*	*7·9*	*5·5*						
17·5	15·3	13·6	11·6	9·6	8·2	7·0	6·0	221·1	36·8
12·3	*9·0*	*6·7*	*4·7*						
16·4	14·3	12·7	11·0	9·1	7·7	6·5	5·6	206·9	34·5
11·5	*8·4*	*6·2*	*4·4*						
33·0	28·7	22·7	18·5	15·2	12·8	10·9	9·4	344·9	69·0
32·3	*26·1*	*21·4*	*17·9*	*14·8*	*12·4*	*10·2*	*8·4*		
27·4	24·0	18·9	15·2	12·7	10·7	9·1	7·9	288·7	57·7
26·9	*21·8*	*18·0*	*15·0*	*12·4*	*10·4*	*8·5*	*7·0*		
20·0	17·6	13·9	11·3	9·3	7·8	6·7	5·8	211·5	42·3
16·4	*12·8*	*10·0*	*7·9*	*6·0*	*4·5*				

ratio of span to depth of 19·15. The loads to right of this line are reduced to keep the deflection within 1/325th of the span.

For general notes see page vi.

JOISTS—Safe Distributed Loads in

10 tons/sq. in.	Area in sq. ins.	10 tons/sq. in.				SPANS	
Section		4	5	6	7	8	9
10 × 6 × 40	11·77				39·1 *39·1*	34·2 *34·2*	30·3 *30·3*
10 × 5 × 30	8·85		39·0 *39·0*	32·6 *32·6*	27·9 *27·9*	24·3 *24·3*	21·5 *20·6*
10 × 4½ × 25	7·35		32·6 *32·6*	27·2 *27·2*	23·3 *23·3*	20·3 *19·7*	18·0 *16·4*
*9 × 7 × 58	17·06			56·0 *56·0*	48·6 *48·6*	42·6 *42·6*	37·8 *37·8*
9 × 7 × 50	14·71					38·5 *38·5*	34·3 *34·3*
9 × 4 × 21	6·18	29·9 *29·9*	24·0 *24·0*	20·0 *20·0*	17·2 *16·9*	15·0 *13·7*	13·3 *11·3*
*8 × 8 × 38	11·13				31·3 *31·3*	27·4 *27·4*	24·4 *24·4*
8 × 6 × 35	10·30			32·0 *32·0*	27·4 *27·4*	23·9 *23·9*	21·3 *21·3*
8 × 5 × 28	8·28		29·8 *29·8*	24·9 *24·9*	21·3 *21·3*	18·7 *18·7*	16·6 *15·9*
8 × 4 × 18	5·30	23·2 *23·2*	18·5 *18·5*	15·3 *15·3*	13·2 *12·9*	11·6 *10·6*	10·3 *8·8*
7 × 4 × 16	4·75	18·9 *18·9*	15·0 *15·0*	12·5 *12·5*	10·8 *10·6*	9·3 *8·5*	8·4 *7·1*
*7 × 3½ × 15	4·42	17·2 *17·2*	13·8 *13·8*	11·5 *11·4*	9·9 *9·0*	8·5 *7·1*	7·6 *5·8*
*6 × 5 × 25	7·37	24·3 *24·3*	19·4 *19·4*	16·2 *16·2*	13·8 *13·8*	12·0 *12·0*	10·8 *10·4*
6 × 4½ × 20	5·89	19·3 *19·3*	15·4 *15·4*	12·7 *12·7*	10·9 *10·9*	9·6 *9·3*	8·7 *7·9*

The lower figures in italics are safe loads for beams not tied laterally, i.e. where l/b exceeds 20. The zig-zag line indicates the

Tons and Dimensions and Properties

IN FEET							Moment of Inertia	Modulus of Section
10	12	14	16	18	20	22	x—x	x—x
27·4	22·8	19·6	17·1	13·5	10·9	9·0	204·8	**41·0**
27·4	*20·7*	*16·1*	*12·5*	*9·7*	*7·6*	*5·8*		
19·6	16·2	13·9	12·2	9·6	7·8	6·5	146·2	**29·2**
17·8	*13·1*	*9·7*	*7·2*	*5·3*	*3·7*			
16·3	13·6	11·6	10·2	8·0	6·5	5·4	122·3	**24·5**
13·9	*10·1*	*7·2*	*5·1*	*3·5*				
34·1	28·4	24·3	19·2	15·1	12·3	10·1	229·5	**51·0**
34·1	*28·1*	*22·1*	*17·9*	*14·3*	*11·6*	*9·5*		
30·8	25·6	22·1	17·2	13·8	11·1	9·2	208·1	**46·3**
30·8	*25·3*	*20·1*	*16·2*	*12·9*	*10·5*	*8·5*		
12·1	10·0	8·5	6·8	5·3	4·3	3·6	81·1	**18·0**
9·4	*6·5*	*4·4*	*2·8*					
21·9	18·3	14·3	11·0	8·6	7·0	5·8	131·3	**32·8**
21·9	*18·3*	*14·3*	*11·0*	*8·6*	*7·0*	*5·8*		
19·2	15·8	12·6	9·5	7·6	6·1	5·1	115·1	**28·8**
19·2	*14·4*	*11·2*	*8·8*	*6·9*	*5·3*	*4·1*		
14·8	12·4	9·8	7·4	5·9	4·8	4·0	89·7	**22·4**
13·5	*10·0*	*7·5*	*5·5*	*4·1*	*2·9*			
9·2	7·8	5·9	4·6	3·7	3·0	2·5	55·6	**13·9**
7·2	*5·1*	*3·5*	*2·2*					
7·6	5·8	4·3	3·3	2·6	2·1	1·7	39·5	**11·3**
5·9	*4·1*	*2·8*	*1·8*					
6·9	5·3	3·9	3·0	2·4	1·9	1·6	35·9	**10·3**
4·7	*3·0*	*1·9*						
9·3	6·5	4·8	3·6	2·9	2·3	1·9	43·7	**14·6**
8·8	*6·5*	*4·8*	*3·6*	*2·6*	*1·9*			
7·4	5·2	3·8	2·9	2·3	1·9	1·5	34·7	**11·6**
6·6	*4·8*	*3·4*	*2·4*	*1·6*				

ratio of span to depth of 19·15. The loads to right of this line are reduced to keep the deflection within 1/325th of the span.

For general notes see page vi.

JOISTS—Safe Distributed Loads in

X—I—X 10 tons/sq. in. Section	Area in sq. ins.	10 tons/sq. in. SPANS 3	4	5	6	7	8
6 × 3 × 12	3·53	15·6 *15·6*	11·6 *11·6*	9·4 *9·4*	7·7 *7·0*	6·7 *5·6*	5·8 *4·3*
5 × 4½ × 20	5·88	22·2 *22·2*	16·6 *16·6*	13·4 *13·4*	11·2 *11·2*	9·5 *9·5*	8·3 *8·1*
*5 × 4½ × 18	5·29	20·2 *20·2*	15·2 *15·2*	12·1 *12·1*	10·1 *10·1*	8·6 *8·6*	7·6 *7·4*
5 × 3 × 11	3·26	12·1 *12·1*	9·2 *9·2*	7·3 *7·3*	6·0 *5·5*	5·3 *4·4*	4·5 *3·3*
*5 × 2½ × 9	2·65	9·7 *9·7*	7·2 *7·2*	5·9 *5·4*	4·9 *4·0*	4·2 *2·9*	3·6 *2·1*
4¾ × 1¾ × 6·5	1·91	6·2 *6·1*	4·8 *4·0*	3·8 *2·6*	3·1 *1·7*	2·6 *·99*	2·2
*4½ × 2 × 7	2·06	6·6 *6·6*	4·9 *4·5*	4·0 *3·1*	3·2 *2·1*	2·9 *1·5*	2·2 *·93*
4 × 3 × 10	2·94	8·6 *8·6*	6·5 *6·5*	5·2 *5·2*	4·3 *3·9*	3·3 *3·1*	2·6 *2·4*
*4 × 3 × 9·5	2·79	8·4 *8·4*	6·2 *6·2*	5·1 *5·1*	4·1 *3·7*	3·2 *3·0*	2·5 *2·3*
4 × 1¾ × 5	1·47	4·0 *4·0*	3·2 *2·7*	2·4 *1·7*	2·0 *1·1*	1·6 *·67*	1·2
3 × 3 × 8·5	2·52	5·6 *5·6*	4·2 *4·2*	3·3 *3·3*	2·2 *2·2*	1·7 *1·7*	1·3 *1·3*
3 × 1½ × 4	1·18	2·4 *2·2*	1·8 *1·3*	1·4 *·84*	·99 *·48*	·74	·57

** Sections marked thus are old*

The lower figures in italics are safe loads for beams not tied laterally, i.e. where l/b exceeds 20. The zig-zag line indicates the

Tons and Dimensions and Properties

IN FEET							Moment of Inertia	Modulus of Section
9	10	11	12	13	14	15	x—x	x—x
5·2 *3·4*	4·4 *2·6*	3·8 *2·0*	3·1 *1·5*	2·7	2·3	2·0	21·0	7·0
6·6 *6·6*	5·3 *5·3*	4·4 *4·4*	3·7 *3·7*	3·2 *3·2*	2·7 *2·7*	2·4 *2·4*	25·0	10·0
6·0 *6·0*	4·9 *4·9*	4·0 *4·0*	3·4 *3·4*	2·9 *2·9*	2·5 *2·5*	2·2 *2·2*	22·7	9·1
3·6 *2·6*	2·9 *2·1*	2·4 *1·6*	2·0 *1·2*	1·7	1·5	1·3	13·7	5·5
2·9 *1·6*	2·3 *1·1*	1·9	1·6	1·4	1·2	1·0	10·9	4·4
1·8	1·4	1·2	·99	·85	·73	·64	6·7	2·8
1·7	1·4	1·2	·98	·83	·72	·63	6·6	2·9
2·1 *1·9*	1·7 *1·5*	1·4 *1·1*	1·2 *·83*	·99	·85	·74	7·8	3·9
2·0 *1·8*	1·6 *1·4*	1·3 *1·1*	1·1 *·79*	·95	·82	·71	7·5	3·8
·97	·79	·65	·55	·47	·40	·35	3·7	1·8
1·0 *1·0*	·81 *·80*	·67 *·70*	·56 *·53*	·48	·41	·36	3·8	2·5
·45	·36	·30	·25	·22	·19	·16	1·7	1·1

British Standard and Special Sections.

ratio of span to depth of 19·15. The loads to right of this line are reduced to keep the deflection within 1/325th of the span.

For general notes see page vi.

CHANNELS—Safe Distributed Loads

Section (10 tons/sq. in.)	Weight per foot in lbs.	Area in Square inches	10 tons/sq. in. SPANS 10	12	14	16
17 × 4	51·28	15·08	44·7 *34·9*	37·1 *24·1*	31·8 *16·5*	28·0 *10·6*
17 × 4	44·34	13·04	40·9 *31·9*	34·0 *22·1*	29·2 *15·2*	25·5 *9·7*
15 × 4	42·49	12·50	34·0 *26·5*	28·4 *18·5*	24·4 *12·7*	21·2 *8·1*
15 × 4	36·37	10·70	31·0 *24·2*	25·9 *16·8*	22·1 *11·5*	19·4 *7·4*
13 × 4	38·92	11·45	27·7 *21·6*	23·2 *15·1*	19·9 *10·3*	17·4 *6·6*
13 × 4	33·18	9·76	25·4 *19·8*	21·2 *13·8*	17·9 *9·3*	15·8 *6·0*
12 × 4	36·63	10·77	24·4 *19·0*	20·3 *13·2*	17·4 *9·1*	15·1 *5·7*
12 × 4	31·33	9·21	22·2 *17·3*	18·5 *12·0*	15·9 *8·3*	13·8 *5·2*
12 × 3½	30·45	8·96	19·3 *13·1*	16·2 *8·6*	13·8 *5·2*	12·0
12 × 3½	26·37	7·76	17·8 *12·1*	14·7 *7·8*	12·6 *4·8*	11·0
11 × 3½	30·52	8·98	18·6 *12·6*	15·5 *8·2*	13·2 *5·0*	11·5
*11 × 3½	29·82	8·77	18·0 *12·2*	15·0 *8·0*	12·9 *4·9*	11·2
11 × 3½	26·78	7·88	17·3 *11·8*	14·3 *7·6*	12·2 *4·6*	10·7
*10 × 4	18·86	5·55	11·0 *8·6*	9·1 *5·9*	7·9 *4·1*	6·9 *2·6*
10 × 3½	28·54	8·39	16·0 *10·9*	13·3 *7·1*	11·4 *4·3*	9·9

The lower figures in italics are safe loads for beams not tied laterally, i.e. where l/b exceeds 20. Loads to right of zig-zag line

in Tons and Dimensions and Properties

IN FEET							Moment of Inertia	Modulus of Section
18	20	22	24	26	28	30	x—x	x—x
24·9	22·3	20·2	18·7	17·2	15·5	13·3	569·3	66·98
22·5	20·5	18·6	17·0	15·6	14·1	12·4	520·2	61·20
19·0	17·0	15·3	14·1	12·0	10·5	9·1	382·8	51·05
17·3	15·5	14·1	12·9	11·0	9·6	8·3	349·1	46·55
15·4	13·9	12·0	10·0	8·5	7·4	6·4	270·7	41·64
14·0	12·7	10·8	9·2	7·8	6·7	5·9	246·8	37·98
13·6	11·6	9·7	8·1	6·9	6·0	5·2	218·8	36·47
12·4	10·6	8·8	7·4	6·3	5·4	4·7	200·1	33·35
10·7	9·2	7·6	6·4	5·5	4·7	4·1	174·1	29·02
9·8	8·6	7·0	5·9	5·0	4·3	3·8	159·7	26·62
10·0	8·1	6·7	5·7	4·8	4·2	3·6	152·9	27·81
9·8	7·9	6·6	5·5	4·7	4·0	3·5	148·6	27·02
9·3	7·5	6·3	5·3	4·5	3·9	3·4	141·9	25·80
5·4	4·4	3·6	3·1	2·6	2·2	2·0	82·6	16·52
7·9	6·3	5·3	4·4	3·8	3·3	2·8	119·5	23·90

are reduced to keep the deflection within 1/325th of the span.
For general notes see page vi.

CHANNELS—Safe Distributed Loads

X—[—X 10 tons/sq. in. Section	Weight per foot in lbs.	Area in Square inches	10 tons/sq. in. SPANS 5	6	7	8
10 × 3½	24·46	7·19	29·2 *29·2*	24·3 *24·1*	20·9 *19·0*	18·2 *15·3*
10 × 3	21·33	6·27	23·4 *23·4*	19·4 *17·7*	16·6 *13·8*	14·6 *10·8*
10 × 3	19·28	5·67	22·0 *22·0*	18·3 *16·7*	15·7 *13·0*	13·7 *10·1*
9 × 3½	25·63	7·54	26·4 *26·4*	22·0 *21·8*	18·9 *17·2*	16·5 *13·9*
9 × 3½	23·49	6·91	25·2 *25·2*	21·1 *20·9*	18·0 *16·4*	15·7 *13·2*
9 × 3½	22·27	6·55	24·5 *24·5*	20·4 *20·2*	17·5 *15·9*	15·3 *12·9*
9 × 3	19·91	5·86	20·0 *20·0*	16·6 *15·1*	14·2 *11·8*	12·4 *9·2*
9 × 3	17·46	5·14	18·5 *18·5*	15·5 *14·1*	13·2 *11·0*	11·5 *8·5*
8 × 3½	23·20	6·82	21·8 *21·8*	18·2 *18·0*	15·6 *14·2*	13·6 *11·4*
8 × 3½	20·21	5·94	20·1 *20·1*	16·9 *16·7*	14·4 *13·1*	12·6 *10·6*
8 × 3	18·68	5·49	17·0 *17·0*	14·2 *12·9*	12·2 *10·1*	10·6 *7·8*
8 × 3	15·96	4·69	15·5 *15·5*	13·0 *11·8*	11·2 *9·3*	9·8 *7·3*
7 × 3½	20·18	5·94	17·1 *17·1*	14·3 *14·2*	12·3 *11·2*	10·7 *9·0*
7 × 3½	18·28	5·38	16·3 *16·3*	13·6 *13·5*	11·7 *10·6*	10·2 *8·6*
7 × 3	17·07	5·02	13·8 *13·8*	11·5 *10·5*	9·8 *8·1*	8·7 *6·4*

The lower figures in italics are safe loads for beams not tied laterally, i.e. where l/b exceeds 20. Loads to right of zig-zag line

in Tons and Dimensions and Properties

IN FEET							Moment of Inertia	Modulus of Section
9	10	12	14	16	18	20	x—x	x—x
16·2 *12·3*	14·6 *9·9*	12·2 *6·5*	10·5 *4·0*	9·1	7·2	5·8	109·5	21·90
13·0 *8·5*	11·6 *6·5*	9·8 *3·7*	8·3	7·3	5·7	4·6	87·7	17·53
12·2 *7·9*	11·1 *6·2*	9·1 *3·5*	7·8	6·9	5·4	4·4	82·7	16·53
14·6 *11·1*	13·3 *9·0*	11·0 *5·8*	9·4 *3·6*	7·4	5·8	4·8	89·3	19·84
14·0 *10·6*	12·6 *8·6*	10·5 *5·6*	9·0 *3·4*	7·0	5·6	4·5	85·1	18·90
13·6 *10·3*	12·3 *8·4*	10·2 *5·4*	8·7 *3·3*	6·8	5·4	4·4	82·6	18·36
11·1 *7·2*	9·9 *5·5*	8·3 *3·2*	7·1	5·6	4·4	3·6	67·4	14·97
10·3 *6·7*	9·2 *5·2*	7·7 *2·9*	6·6	5·2	4·1	3·3	62·5	13·89
12·1 *9·2*	10·8 *7·3*	9·0 *4·8*	7·1 *3·0*	5·4	4·3	3·5	65·3	16·32
11·2 *8·5*	10·1 *6·9*	8·4 *4·5*	6·5 *2·7*	5·0	4·0	3·2	60·6	15·14
9·4 *6·1*	8·6 *4·8*	7·0 *2·7*	5·5	4·2	3·4	2·7	51·0	12·75
8·6 *5·6*	7·8 *4·4*	6·4 *2·4*	5·0	3·8	3·1	2·5	46·7	11·68
9·6 *7·3*	8·5 *5·8*	6·6 *3·8*	4·9 *2·3*	3·8	3·0	2·4	45·2	12·98
9·1 *6·9*	8·2 *5·6*	6·3 *3·6*	4·6 *2·2*	3·6	2·8	2·3	42·8	12·24
7·6 *4·9*	6·9 *3·9*	5·3 *2·2*	3·9	3·0	2·4	1·9	36·2	10·34

are reduced to keep the deflection within 1/325th of the span.
For general notes see page vi.

CHANNELS—Safe Distributed Loads

x—[—x 10 tons/sq. in. Section	Weight per foot in lbs.	Area in Square inches	10 Tons/sq. in. SPANS 3	4	5	6
7 × 3	14·22	4·18	20·9 *20·9*	15·6 *15·6*	12·4 *12·4*	10·5 *9·6*
*7 × 2½	9·75	2·86	13·0 *13·0*	9·8 *9·2*	7·8 *6·4*	6·5 *4·5*
6 × 3½	16·48	4·85	21·4 *21·4*	16·1 *16·1*	12·8 *12·8*	10·7 *10·6*
6 × 3	17·53	5·16	20·2 *20·2*	15·1 *15·1*	12·0 *12·0*	10·1 *9·2*
6 × 3	16·51	4·86	19·5 *19·5*	14·6 *14·6*	11·7 *11·7*	9·6 *8·7*
6 × 3	13·64	4·01	16·5 *16·5*	12·4 *12·4*	10·0 *10·0*	8·2 *7·5*
6 × 3	12·41	3·65	15·8 *15·8*	11·8 *11·8*	9·5 *9·5*	7·8 *7·1*
5 × 2½	11·24	3·31	11·2 *11·2*	8·3 *8·3*	6·7 *6·1*	5·5 *4·5*
5 × 2½	10·22	3·01	10·6 *10·6*	7·8 *7·8*	6·3 *5·7*	5·2 *4·2*
4 × 2	7·91	2·33	5·9 *5·9*	4·4 *4·0*	3·5 *2·7*	3·0 *2·0*
4 × 2	7·09	2·09	5·6 *5·6*	4·2 *3·8*	3·2 *2·5*	2·8 *1·8*
*3½ × 2	6·75	1·99	4·7 *4·7*	3·5 *3·2*	2·9 *2·3*	2·2 *1·5*
*3 × 1½	5·27	1·55	3·0 *2·7*	2·3 *1·7*	1·7 *1·0*	1·2 *·56*
3 × 1½	5·11	1·50	2·8 *2·6*	2·1 *1·6*	1·6 *·95*	1·1 *·54*
3 × 1½	4·60	1·35	2·7 *2·5*	2·0 *1·5*	1·5 *·90*	1·0 *·51*

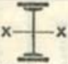
The lower figures in italics are safe loads for beams not tied laterally, i.e. where l/b exceeds 20. Loads to right of zig-zag line

in Tons and Dimensions and Properties

IN FEET							Moment of Inertia	Modulus of Section
7	8	9	10	11	12	14	x—x	x—x
9·0	7·8	6·9	6·3	5·6	4·8	3·5	32·8	9·36
7·5	*5·8*	*4·5*	*3·5*	*2·6*	*2·0*			
5·6	4·9	4·4	3·9	3·5	3·0	2·6	20·5	5·85
3·2	*2·2*							
9·2	8·0	7·2	6·1	5·0	4·3	3·1	28·9	9·63
8·4	*6·7*	*5·5*	*4·4*	*3·6*	*2·8*	*1·7*		
8·6	7·6	6·7	5·8	4·7	4·0	3·0	27·2	9·06
7·1	*5·6*	*4·4*	*3·4*	*2·6*	*1·9*			
8·4	7·3	6·4	5·6	4·6	3·9	2·9	26·3	8·76
7·0	*5·4*	*4·2*	*3·3*	*2·5*	*1·9*			
7·0	6·2	5·5	4·7	3·9	3·3	2·4	22·4	7·45
5·8	*4·6*	*3·6*	*2·8*	*2·1*	*1·6*			
6·7	5·9	5·2	4·5	3·7	3·2	2·3	21·3	7·09
5·6	*4·4*	*3·4*	*2·7*	*2·0*	*1·5*			
4·8	4·1	3·2	2·6	2·2	1·9	1·4	12·5	5·00
3·4	*2·4*	*1·8*	*1·3*					
4·4	3·9	3·1	2·5	2·1	1·8	1·3	11·9	4·75
3·1	*2·3*	*1·7*	*1·2*					
2·3	1·7	1·4	1·2	·95	·80	·59	5·4	2·69
1·3	*·85*							
2·2	1·6	1·3	1·1	·90	·76	·56	5·1	2·53
1·3	*·81*							
1·6	1·2	·97	·79	·65	·55	·40	3·7	2·12
1·1	*·67*							
·87	·67	·53	·43	·35	·30	·22	2·0	1·33
·84	·63	·50	·41	·34	·28	·21	1·9	1·29
·79	·60	·47	·38	·32	·27	·20	1·8	1·22

are reduced to keep the deflection within 1/325th of the span.
For general notes see page vi.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons



10 tons/sq. in. Joist	Flange Plates	Weight per foot in lbs.	10 tons/sq.in. SPANS 24	26	28	30
24 × 7½ × 100	12 × 2	267·0	Rivets ⅞" dia.			157
24 × 7½ × 100	12 × 1½	226·0		148	138	128
24 × 7½ × 100	12 × 1	185·5	125	116	108	100
24 × 7½ × 100	12 × ¾	165·0	108	99·2	92·2	86·8
24 × 7½ × 100	12 × ⅝	155·0	98·9	91·3	84·8	79·2
24 × 7½ × 100	12 × ½	144·5	90·2	83·3	77·0	72·2
24 × 7½ × 100	12 × ⅜	134·3	81·8	75·6	70·0	65·6
24 × 7½ × 95	12 × 2	261·8	Rivets ⅞ dia.			155
24 × 7½ × 95	12 × 1½	221·0			135	126
24 × 7½ × 95	12 × 1	180·2	122	113	105	98·2
24 × 7½ × 95	12 × ⅞	170·0	114	105	97·6	91·2
24 × 7½ × 95	12 × ¾	159·8	105	97·2	90·2	84·2
24 × 7½ × 95	12 × ⅝	149·6	96·5	89·0	82·6	77·2
24 × 7½ × 95	12 × ½	139·5	87·8	81·0	75·2	70·2
24 × 7½ × 95	12 × ⅜	129·2	79·0	73·0	67·6	63·2

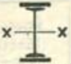
The above loads are based on a safe working stress of 10 tons per square inch. Loads to right of zig-zag line

and Dimensions and Properties

IN FEET						Moment of Inertia	Moduli of Section x—x	
32	36	40	44	48	52	x—x	Girder	I in Plate Breadth
148	131	118	107	91·4	78·0	9891	706·5	48·4
121	107	96·3	86·0	72·0	61·6	7799	577·7	36·2
93·9	83·4	75·1	64·7	54·3		5857	450·5	24·1
80·7	71·7	64·5	54·6	45·6		4940	387·4	18·0
74·2	65·9	59·3	49·6	41·7		4495	356·0	15·0
67·7	60·2	54·2	44·8	37·6		4058	324·6	12·0
61·4	54·6	48·7	40·2	33·8		3646	294·6	9·0
145	129	116	105	90·5	77·3	9783	698·8	48·4
118	105	94·9	84·8	71·2		7691	569·7	36·2
92·2	81·8	73·8	63·3	53·2		5749	442·2	24·1
85·5	76·0	68·4	58·3	49·0		5286	410·6	21·0
78·8	70·1	63·2	53·2	44·8		4832	379·0	18·0
72·3	64·3	57·8	48·5	40·6		4387	347·5	15·0
65·8	58·5	52·6	43·5	36·6		3950	316·0	12·0
59·2	52·8	46·9	38·8	32·6		3520	284·5	9·0

are reduced to keep the deflection within 1/325th of the span. For General Notes see page vi.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

 10 tons/sq. in.		Weight per foot in lbs.	10 tons/sq. in.		SPANS	
Joist	Flange Plates		24	26	28	30
22×7 ×75	12×2	241·8	Riv	ets ⅞"	dia.	
22×7 ×75	12×1¾	221·4				122
22×7 ×75	12×1½	201·0				108
22×7 ×75	12×1	160·3	102	94·7	87·9	82·0
22×7 ×75	12×¾	139·8	86·3	79·7	74·0	69·0
22×7 ×75	12×⅝	129·6	78·2	72·0	67·0	62·5
22×7 ×75	12×½	119·5	70·1	64·7	60·1	56·1
22×7 ×75	12×⅜	109·2	61·9	57·1	53·2	49·5
20×7½×89	12×2	256·0	Riv	ets ⅞"	dia.	128
20×7½×89	12×1½	215·0		119	110	103
20×7½×89	12×1	174·5	99·6	92·0	85·4	79·7
20×7½×89	12×¾	153·8	85·1	78·5	72·9	68·0
20×7½×89	12×⅝	144·0	77·8	71·8	66·7	62·2
20×7½×89	12×½	133·4	70·6	65·1	60·5	56·4
20×7½×89	12×⅜	123·2	63·3	58·5	54·3	50·7

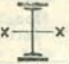
The above loads are based on a safe working stress of 10 tons per square inch. Loads to right of zig-zag line

and Dimensions and Properties

IN FEET						Moment of Inertia	Moduli of Section x—x	
32	34	36	40	44	48	x—x	Girder	1 in. Plate Breadth
127	119	112	101	86·9	*73·0*	7892	607·1	44·4
114	107	102	91·0	77·0	64·6	6974	546·9	38·8
101	95·5	90·2	81·2	67·1	56·4	6093	487·5	33·2
76·9	72·4	68·4	59·1	48·8		4433	369·4	22·1
64·7	60·9	57·5	48·7	40·2		3653	310·9	16·5
58·6	55·1	52·0	43·6	36·1		3274	281·8	13·8
52·6	49·5	46·7	38·7	32·0		2905	252·6	11·0
46·5	43·8	41·3	33·9	28·0		2541	223·4	8·2
119	112	106	91·6	75·7		6876	573·0	40·4
96·8	91·1	86·1	71·3	58·9		5348	465·1	30·2
74·7	70·3	64·9	52·6			3948	358·8	20·1
63·8	60·0	54·2	43·9			3294	306·4	15·0
58·3	54·9	49·1	39·7			2978	280·3	12·5
52·9	49·2	43·9				2669	254·2	10·0
47·5	43·6	39·0				2365	228·1	7·6

are reduced to keep the deflection within 1/325th of the span. *For General Notes see page vi.*

COMPOUND GIRDERS—
Safe Distributed Loads in Tons



10 tons/sq. in. Joist	Flange Plates	Weight per foot in lbs.	10 tons/sq. in. 22	24	26	SPANS 28
$20 \times 6\frac{1}{2} \times 65$	10×2	204·6	Riv	ets $\frac{7}{8}$"	dia.	
$20 \times 6\frac{1}{2} \times 65$	$10 \times 1\frac{3}{4}$	187·6				
$20 \times 6\frac{1}{2} \times 65$	$10 \times 1\frac{1}{2}$	170·6				**87·6**
$20 \times 6\frac{1}{2} \times 65$	10×1	136·6	**85·0**	**77·9**	**71·9**	**66·8**
$20 \times 6\frac{1}{2} \times 65$	$10 \times \frac{3}{4}$	119·8	**71·9**	**65·9**	60·8	56·5
$20 \times 6\frac{1}{2} \times 65$	$10 \times \frac{5}{8}$	111·5	**65·3**	**59·9**	55·2	51·3
$20 \times 6\frac{1}{2} \times 65$	$10 \times \frac{1}{2}$	102·8	58·8	53·9	49·8	46·2
$20 \times 6\frac{1}{2} \times 65$	$10 \times \frac{3}{8}$	94·4	52·5	48·0	44·3	41·2
$18 \times 8 \times 80$	12×2	246·8	Riv	ets $\frac{7}{8}$"	dia.	
$18 \times 8 \times 80$	$12 \times 1\frac{1}{2}$	206·0				**98·0**
$18 \times 8 \times 80$	12×1	165·2	**95·9**	**87·9**	**81·1**	**75·3**
$18 \times 8 \times 80$	$12 \times \frac{3}{4}$	144·8	**81·6**	**74·8**	**69·0**	64·1
$18 \times 8 \times 80$	$12 \times \frac{5}{8}$	134·6	**74·6**	**68·3**	63·0	58·5
$18 \times 8 \times 80$	$12 \times \frac{1}{2}$	124·4	**67·4**	61·7	57·0	52·9
$18 \times 8 \times 80$	$12 \times \frac{3}{8}$	114·2	60·2	55·2	51·0	47·5

The above loads are based on a safe working stress of 10 tons per square inch. Loads to right of zig-zag line

and Dimensions and Properties

IN FEET						Moment of Inertia	Moduli of Section x—x	
30	32	34	36	40	44	x—x	Girder	1 in. Plate Breadth
102	95·2	89·6	84·6	73·1	60·4	5486	457·2	40·4
91·8	85·8	80·8	76·3	64·5	53·2	4847	412·4	35·3
81·8	76·7	72·2	68·1	56·4	46·6	4235	368·2	30·2
62·3	58·4	55·0	50·8	41·1		3088	280·8	20·1
52·7	49·4	46·5	42·0	34·0		2552	237·4	15·0
47·9	44·9	42·3	37·8			2292	215·8	12·5
43·1	40·4	37·6	33·5			2040	194·3	10·0
38·4	36·0	33·1	29·6			1793	172·7	7·6
113	106	99·9	92·2	74·7	61·7	5607	509·7	36·5
91·5	85·8	79·8	71·2	57·7		4328	412·2	27·2
70·3	65·9	58·4	52·1			3165	316·5	18·1
59·8	54·7	48·4				2626	269·3	13·5
54·7	49·3	43·7				2367	245·9	11·3
49·4	44·0	39·0				2114	222·5	9·0
44·0	38·9	34·5				1866	199·1	6·7

are reduced to keep the deflection within 1/325th of the span. *For General Notes see page vi.*

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

x—x 10 tons/sq. in. Joist	Flange Plates	Weight per foot in lbs.	10 tons/sq. in. SPANS 22	24	26	28
18×7×75	12×2	241·9	Riv	ets $\frac{7}{8}$"	dia.	
18×7×75	12×1½	201·1			102	94·9
18×7×75	12×1	160·3	91·6	84·0	77·5	72·0
18×7×75	12×¾	139·9	77·2	70·8	65·3	60·6
18×7×75	12×⅝	129·7	70·0	64·2	59·2	55·0
18×7×75	12×½	119·5	62·9	57·6	53·2	49·4
18×7×75	12×⅜	109·3	55·8	51·1	47·2	43·8
18×6×55	10×2	193·5	Riv	ets ¾"	dia.	
18×6×55	10×1½	159·5				76·5
18×6×55	10×1	125·5	73·1	67·0	61·8	57·4
18×6×55	10×⅞	116·8	67·1	61·5	56·8	52·7
18×6×55	10×¾	108·3	61·0	55·9	51·6	47·9
18×6×55	10×⅝	99·5	55·0	50·4	46·6	43·3
18×6×55	10×½	91·5	49·0	44·9	41·5	38·5
18×6×55	10×⅜	82·9	43·0	39·4	36·4	33·8

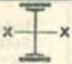
The above loads are based on a safe working stress of 10 tons per square inch. Loads to right of zig-zag line

and Dimensions and Properties

IN FEET						Moment of Inertia	Moduli of Section $x—x$	
30	32	34	36	40	44	$x—x$	Girder	1 in. Plate Breadth
	103	97·4	89·9	*72·8*	*60·2*	5466	**496·9**	36·5
88·6	83·0	77·2	68·9	56·0		4187	**398·8**	27·2
67·2	63·0	55·8	49·7			3025	**302·5**	18·1
56·6	51·7	45·8				2485	**254·9**	13·5
51·3	46·7	41·6				2255	**231·3**	11·3
46·1	41·1	36·4				1973	**207·7**	9·0
40·9	36·0	32·0				1728	**184·3**	6·7
89·6	84·0	79·1	73·2	59·3	*48·9*	4439	**403·5**	36·5
71·4	67·0	62·3	55·6	*44·9*		3378	**321·8**	27·2
53·6	50·2	44·5	39·7			2412	**241·2**	18·1
49·2	45·5	40·3	36·0			2185	**221·3**	15·8
44·7	40·9	36·2				1964	**201·5**	13·5
40·4	36·4	32·3				1748	**181·8**	11·3
35·9	32·0	28·3				1539	**162·0**	9·0
31·6	27·8	24·7				1334	**142·2**	6·7

are reduced to keep the deflection within 1/325th of the span. For General Notes see page vi.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

 10 tons/sq. in.		Weight per foot in lbs.	10 tons/sq. in.		SPANS	
Joist	Flange Plates		18	20	22	24
16×8×75	12×2	241·8	Riv	ets $\frac{7}{8}''$	dia.	
16×8×75	12×$1\frac{1}{2}$	201·0				
16×8×75	12×1	160·2			83·4	76·4
16×8×75	12×$\frac{3}{4}$	139·8		77·8	70·8	64·8
16×8×75	12×$\frac{5}{8}$	129·6	78·7	70·9	64·5	59·0
16×8×75	12×$\frac{1}{2}$	119·4	71·1	64·0	58·2	53·2
16×8×75	12×$\frac{3}{8}$	109·2	63·3	57·0	51·8	47·5
16×6×62	10×2	200·5	Riv	ets $\frac{3}{4}''$	dia.	101
16×6×62	10×$1\frac{1}{2}$	166·5		97·0	88·0	80·7
16×6×62	10×1	132·5	81·4	73·3	66·5	61·1
16×6×62	10×$\frac{7}{8}$	123·8	74·7	67·2	61·2	56·0
16×6×62	10×$\frac{3}{4}$	115·4	68·4	61·5	56·0	51·3
16×6×62	10×$\frac{5}{8}$	107·0	61·8	55·7	50·6	46·3
16×6×62	10×$\frac{1}{2}$	98·5	55·4	49·9	45·2	41·4
16×6×62	10×$\frac{3}{8}$	89·7	49·0	44·1	40·2	36·6

The above loads are based on a safe working stress of 10 tons per square inch. Loads to right of zig-zag line

and Dimensions and Properties

IN FEET						Moment of Inertia	Moduli of Section x—x	
26	28	30	32	34	36	x—x	Girder	1 in. Plate Breadth
	107	99·3	93·1	82·7	73·7	4476	447·6	32·4
92·5	85·8	80·2	71·3	63·2	56·3	3425	360·5	24·2
70·6	65·5	58·7	51·6			2478	275·3	16·1
59·8	55·6	48·4	42·6			2042	233·4	12·0
54·5	49·9	43·5				1834	212·6	10·0
49·1	44·3	38·7				1631	191·8	8·0
43·8	39·1	34·0				1432	171·0	6·0
92·8	86·6	80·8	75·8	*67·2*	*60·0*	3636	363·6	32·4
74·6	69·2	64·6	*57·6*	*51·0*		2763	290·8	24·2
56·3	52·3	46·9	41·1			1976	219·5	16·1
51·7	48·0	42·6	37·4			1793	201·9	14·0
47·3	43·9	38·3	33·6			1615	185·5	12·0
42·8	39·2	34·3				1441	167·1	10·0
38·2	34·6	30·2				1273	149·7	8·0
33·9	30·0	26·3				1108	132·3	6·0

are reduced to keep the deflection within 1/325th of the span. *For General Notes see page vi.*

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

10 tons/sq. in. Joist	Flange Plates	Weight per foot in lbs.	10 tons/sq. in. SPANS 18	20	22	24
16 × 6 × 50	10 × 2	188·3	Riv	ets ¾"	dia.	
16 × 6 × 50	10 × 1½	154·6				
16 × 6 × 50	10 × 1	120·6		69·7	63·4	58·1
16 × 6 × 50	10 × ⅞	111·8	70·6	63·6	57·8	53·0
16 × 6 × 50	10 × ¾	103·3	64·3	57·8	52·6	48·2
16 × 6 × 50	10 × ⅝	94·8	57·8	51·9	47·2	43·3
16 × 6 × 50	10 × ½	86·3	51·3	46·2	42·0	38·4
16 × 6 × 50	10 × ⅜	77·8	44·8	40·3	36·6	33·6
15 × 6 × 45	10 × 2	183·3	Riv	ets ¾"	dia.	
15 × 6 × 45	10 × 1½	149·3				
15 × 6 × 45	10 × 1	115·3		63·4	57·6	52·8
15 × 6 × 45	10 × ¾	98·3	58·0	52·3	47·5	43·6
15 × 6 × 45	10 × ⅝	89·8	52·0	46·7	42·6	38·9
15 × 6 × 45	10 × ½	81·3	45·8	41·3	37·6	34·4
15 × 6 × 45	10 × ⅜	72·8	39·7	35·8	32·5	29·8

The above loads are based on a safe working stress of 10 tons per square inch. Loads to right of zig-zag line

and Dimensions and Properties

IN FEET						Moment of Inertia	Moduli of Section x—x	
26	28	30	32	34	36	x—x	Girder	1 in. Plate Breadth
	84·3	78·6	73·8	65·3	58·2	3539	354·0	32·4
71·8	66·8	62·3	55·5	49·2	*43·9*	2666	280·7	24·2
53·6	49·7	44·5	39·1			1880	208·9	16·1
48·9	45·5	40·2	35·3			1696	191·1	14·0
44·4	41·4	35·9	31·6			1518	173·5	12·0
40·0	36·6	31·9				1344	156·0	10·0
35·4	32·0	27·8				1177	138·4	8·0
31·0	27·7	24·1				1013	121·0	6·0
	78·0	72·6	64·7	57·2	51·1	3106	326·9	30·6
66·0	61·3	54·9	48·3	42·8	*38·3*	2320	257·8	22·8
48·7	43·9	38·3	33·8			1616	190·2	15·2
40·2	35·2	30·6				1294	156·9	11·3
36·0	31·1	27·0				1140	140·4	9·4
31·2	26·9					991	123·8	7·5
26·7	23·0					846	107·2	5·6

are reduced to keep the deflection within 1/325th of the span. — For General Notes see page vi.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

10 tons/sq. in. Joist	Flange Plates	Weight per foot in lbs.	10 tons/sq. in. SPANS 18	20	22	24
15 × 5 × 42	9 × 1½	136·5	Rivets ¾″ d.		69·0	63·3
15 × 5 × 42	9 × 1¼	121·0		65·8	60·0	55·0
15 × 5 × 42	9 × 1	105·5	62·0	55·8	50·7	46·5
15 × 5 × 42	9 × ⅞	97·9	56·3	50·7	46·3	42·3
15 × 5 × 42	9 × ¾	90·5	51·0	45·9	41·8	38·3
15 × 5 × 42	9 × ⅝	83·0	45·6	41·0	37·2	34·2
15 × 5 × 42	9 × ½	75·0	40·1	36·1	32·8	30·0
15 × 5 × 42	9 × ⅜	67·5	34·6	31·2	28·4	25·9
14 × 8 × 70	12 × 2	236·8	Rivets ⅞″ dia.			
14 × 8 × 70	12 × 1½	196·0				86·0
14 × 8 × 70	12 × 1	155·2		78·3	71·2	65·2
14 × 8 × 70	12 × ¾	134·8	73·5	66·1	60·1	55·1
14 × 8 × 70	12 × ⅝	124·6	66·8	60·0	54·6	50·0
14 × 8 × 70	12 × ½	114·4	60·0	54·0	49·1	45·1
14 × 8 × 70	12 × ⅜	104·2	53·2	48·0	43·6	39·4

The above loads are based on a safe working stress of 10 tons per square inch. Loads to right of zig-zag line

and Dimensions and Properties

IN FEET						Moment of Inertia	Moduli of Section x—x	
26	28	30	32	34	36	x—x	Girder	1 in. Plate Breadth
58·5	54·3	48·6	*42·7*	*38·0*	*33·9*	2053	227·9	22·8
50·8	*47·0*	*41·2*	36·0	32·0		1728	197·7	19·0
43·0	38·7	33·7	29·7			1424	167·5	15·2
39·1	34·8	30·4	26·6			1278	152·6	13·2
35·4	30·9	26·9				1137	137·8	11·3
31·5	27·2	23·8				1000	123·1	9·4
27·3	23·5					866	108·3	7·5
23·3	20·1					737	93·5	5·6
99·1	92·0	82·5	72·5	64·1		3479	386·6	28·5
79·4	71·6	62·4	54·7			2634	309·9	21·3
59·4	51·2	44·5				1881	235·1	14·1
48·5	41·8					1538	198·4	10·5
43·4	37·4					1377	180·3	8·8
38·2	33·0					1216	162·1	7·0
33·5	29·0					1061	143·9	5·2

are reduced to keep the deflection within 1/325th of the span. *For General Notes see page vi.*

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

10 tons/sq. in. Joist	Flange Plates	Weight per foot in lbs.	10 tons/sq. in. SPANS 16	18	20	22
14 × 6 × 57	10 × 1½	161·5	Riv	ets ¾″	dia.	76·1
14 × 6 × 57	10 × 1	127·5	78·6	69·8	62·9	57·2
14 × 6 × 57	10 × ⅞	119·0	72·0	64·1	57·7	52·5
14 × 6 × 57	10 × ¾	110·3	65·8	58·4	52·7	47·8
14 × 6 × 57	10 × ⅝	102·0	59·3	52·8	47·4	43·2
14 × 6 × 57	10 × ½	93·5	53·2	47·3	42·6	38·6
14 × 6 × 57	10 × ⅜	85·0	46·8	41·6	37·5	34·0
14 × 6 × 46	10 × 1½	150·5	Riv	ets ¾″	dia.	
14 × 6 × 46	10 × 1¼	133·5				63·6
14 × 6 × 46	10 × 1	116·5			59·6	54·2
14 × 6 × 46	10 × ⅞	108·0		60·5	54·5	49·4
14 × 6 × 46	10 × ¾	99·5	61·6	54·5	49·2	44·7
14 × 6 × 46	10 × ⅝	91·0	55·0	48·9	44·1	40·0
14 × 6 × 46	10 × ½	82·6	48·8	43·4	38·9	35·5
14 × 6 × 46	10 × ⅜	74·0	42·2	37·6	33·8	30·7

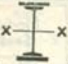
The above loads are based on a safe working stress of 10 tons per square inch. Loads to right of zig-zag line

and Dimensions and Properties

IN FEET						Moment of Inertia	Moduli of Section x—x	
24	26	28	30	32	34	x—x	Girder	1 in. Plate Breadth
69·8	64·3	58·1	*50·6*	*44·4*	*39·5*	2134	251·0	21·3
52·3	47·6	41·1	35·8	31·4		1508	188·5	14·1
48·0	43·1	37·2	32·4			1364	173·1	12·3
43·8	38·7	33·2				1223	157·8	10·5
39·6	34·4	29·6				1088	142·7	8·8
35·5	30·1	26·1				956	127·6	7·0
30·7	26·1	22·6				828	112·3	5·2
67·2	62·0	55·8	48·6	42·8	*37·8*	2054	241·7	21·3
58·2	53·8	47·0	*41·1*	36·0		1732	210·0	17·7
49·7	45·0	38·8	33·8			1429	178·7	14·1
45·4	40·6	35·0				1284	163·0	12·3
41·1	36·1	31·2				1144	147·6	10·5
36·7	31·8	27·4				1008	132·3	8·8
32·4	27·5	23·8				877	116·9	7·0
27·8	23·7	20·4				750	101·5	5·2

are reduced to keep the deflection within 1/325th of the span. *For General Notes see page vi.*

COMPOUND GIRDERS—
Safe Distributed Loads in Tons



10 tons/sq. in.		Weight per foot in lbs.	10 tons/sq. in.		SPANS	
Joist	Flange Plates		14	16	18	20
$14 \times 5\frac{1}{2} \times 40$	$9 \times 1\frac{1}{2}$	134·5	Riv	ets $\frac{3}{4}$"	dia.	
$14 \times 5\frac{1}{2} \times 40$	9×1	103·0				52·4
$14 \times 5\frac{1}{2} \times 40$	$9 \times \frac{7}{8}$	95·0			53·0	47·7
$14 \times 5\frac{1}{2} \times 40$	$9 \times \frac{3}{4}$	87·5		53·9	47·9	43·1
$14 \times 5\frac{1}{2} \times 40$	$9 \times \frac{5}{8}$	80·0	55·0	48·2	42·7	38·5
$14 \times 5\frac{1}{2} \times 40$	$9 \times \frac{1}{2}$	71·6	48·5	42·4	37·7	33·9
$14 \times 5\frac{1}{2} \times 40$	$9 \times \frac{3}{8}$	64·0	42·2	36·9	32·8	29·5
$13 \times 5 \times 35$	$9 \times 1\frac{1}{2}$	129·0	Riv	ets $\frac{3}{4}$"	dia.	
$13 \times 5 \times 35$	$9 \times 1\frac{1}{4}$	113·9				55·3
$13 \times 5 \times 35$	9×1	98·5			52·0	46·6
$13 \times 5 \times 35$	$9 \times \frac{7}{8}$	90·9		52·9	47·0	42·3
$13 \times 5 \times 35$	$9 \times \frac{3}{4}$	83·5	54·4	47·5	42·2	38·1
$13 \times 5 \times 35$	$9 \times \frac{5}{8}$	76·0	48·2	42·2	37·6	33·8
$13 \times 5 \times 35$	$9 \times \frac{1}{2}$	67·9	42·2	36·9	32·6	29·5
$13 \times 5 \times 35$	$9 \times \frac{3}{8}$	60·5	36·0	31·6	28·1	25·2

The above loads are based on a safe working stress of 10 tons per square inch. Loads to right of zig-zag line

and Dimensions and Properties

IN FEET						Moment of Inertia	Moduli of Section $x-x$	
22	24	26	28	30	32	$x-x$	Girder	1 in. Plate Breadth
64·7	59·3	54·7	49·3	43·0	37·7	1812	213·2	21·3
47·6	43·7	39·7	34·2	29·8		1256	157·0	14·1
43·3	39·7	35·4	30·5			1126	143·0	12·3
39·2	35·8	31·6	27·3			1002	129·2	10·5
34·9	32·1	27·8	24·0			881	115·6	8·8
30·9	28·2	24·1	20·8			763	101·8	7·0
26·8	24·0	20·5				650	88·3	5·2
58·4	53·6	48·6	42·0	36·5	32·2	1542	192·8	19·8
50·3	46·1	40·6	35·0	30·5		1287	166·1	16·5
42·5	38·9	33·1	28·6			1050	140·1	13·1
38·5	34·7	29·6	25·6			936	126·9	11·5
34·6	30·7	26·2				827	114·1	9·8
30·7	26·7	22·8				721	101·3	8·2
26·8	22·8	19·5				619	88·6	6·5
22·9	19·3	16·4				520	75·7	4·9

are reduced to keep the deflection within 1/325th of the span. *For General Notes see page vi.*

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

10 tons/sq. in.

Joist	Flange Plates	Weight per foot in lbs.	10 tons/sq. in. SPANS 14	16	18	20
12 × 8 × 65	12 × $1\frac{1}{2}$	191·0	Riv	ets $\frac{7}{8}$"	dia.	
12 × 8 × 65	12 × 1	150·2				65·2
12 × 8 × 65	12 × $\frac{7}{8}$	140·0			66·8	60·1
12 × 8 × 65	12 × $\frac{3}{4}$	129·8		68·8	61·0	54·9
12 × 8 × 65	12 × $\frac{5}{8}$	119·6	71·2	62·2	55·3	49·8
12 × 8 × 65	12 × $\frac{1}{2}$	109·4	63·7	55·7	49·5	44·5
12 × 8 × 65	12 × $\frac{3}{8}$	99·2	56·4	49·3	43·8	39·6
12 × 6 × 54	10 × $1\frac{1}{2}$	158·4	Riv	ets $\frac{3}{4}$"	dia.	70·8
12 × 6 × 54	10 × $1\frac{1}{4}$	141·3			68·6	61·8
12 × 6 × 54	10 × 1	124·3		66·0	58·7	52·8
12 × 6 × 54	10 × $\frac{7}{8}$	115·8	69·2	60·5	53·8	48·3
12 × 6 × 54	10 × $\frac{3}{4}$	107·2	62·9	55·1	48·9	44·0
12 × 6 × 54	10 × $\frac{5}{8}$	98·9	56·8	49·8	44·2	39·8
12 × 6 × 54	10 × $\frac{1}{2}$	90·4	50·6	44·3	39·3	35·4
12 × 6 × 54	10 × $\frac{3}{8}$	81·8	44·3	38·8	34·6	31·0

The above loads are based on a safe working stress of 10 tons per square inch. Loads to right of zig-zag line

and Dimensions and Properties

IN FEET						Moment of Inertia	Moduli of Section x—x	
22	24	26	28	30	32	x—x	Girder	1 in. Plate Breadth
79·1	72·5	61·7	53·2	46·4	40·8	1956	260·8	18·3
59·4	50·8	43·3	37·3	32·5		1374	196·3	12·1
54·5	45·8	39·1	33·6			1239	180·3	10·6
49·0	41·2	35·1				1112	164·8	9·0
43·5	36·6					989	149·3	7·5
38·3	32·2					870	133·8	6·0
33·2	27·9					754	118·3	4·5
64·3	59·0	50·2	*43·4*	*37·8*	*33·2*	1593	212·4	18·3
56·2	49·6	*42·5*	*36·6*	*31·9*		1343	185·6	15·2
48·0	*41·1*	35·0	30·2			1110	158·7	12·1
44·0	37·0	31·6				999	145·3	10·6
39·3	33·0					893	132·4	9·0
34·8	29·2					790	119·3	7·5
30·4	25·5					691	106·3	6·0
26·2	22·0					595	93·3	4·5

are reduced to keep the deflection within 1/325th of the span. For General Notes see page vi.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

10 tons/sq. in. Joist	Flange Plates	Weight per foot in lbs.	10 tons/sq. in. SPANS 14	16	18	20
12 × 6 × 44	10 × 1½	148·5	Riv	ets ¾″	dia.	
12 × 6 × 44	10 × 1	114·5				50·4
12 × 6 × 44	10 × ⅞	106·0			51·0	46·0
12 × 6 × 44	10 × ¾	97·5		51·9	46·1	41·5
12 × 6 × 44	10 × ⅝	89·0	53·1	46·5	41·3	37·1
12 × 6 × 44	10 × ½	80·6	46·8	41·1	36·5	32·9
12 × 6 × 44	10 × ⅜	72·0	40·6	35·5	31·6	28·5
12 × 5 × 32	9 × 1½	126·3	Riv	ets ¾″	dia.	
12 × 5 × 32	9 × 1¼	111·0				50·2
12 × 5 × 32	9 × 1	95·6			46·8	42·1
12 × 5 × 32	9 × ⅞	88·0	54·5	47·7	42·3	38·1
12 × 5 × 32	9 × ¾	80·3	48·7	42·7	37·8	34·2
12 × 5 × 32	9 × ⅝	72·7	43·2	37·8	33·5	30·2
12 × 5 × 32	9 × ½	65·0	37·6	32·7	29·1	26·3
12 × 5 × 32	9 × ⅜	57·4	32·0	28·0	24·9	22·4

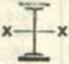
The above loads are based on a safe working stress of 10 tons per square inch. Loads to right of zig-zag line

and Dimensions and Properties

IN FEET						Moment of Inertia x—x	Moduli of Section x—x	
22	24	26	28	30	32		Girder	1 in. Plate Breadth
62·7	57·1	48·6	42·0	36·5	*32·2*	1542	205·6	18·3
45·8	39·3	33·3	28·8	25·0		1058	151·3	12·1
41·6	35·1	29·8	25·7			948	137·9	10·6
37·1	31·2	26·5				841	124·7	9·0
32·5	27·4					739	111·5	7·5
28·1	23·7					640	98·5	6·0
24·0	20·2					544	85·3	4·5
53·2	48·7	41·5	35·8	*31·1*	*27·5*	1316	175·3	18·3
45·7	40·3	34·6	*29·8*	*26·0*		1094	150·9	15·2
38·3	32·8	28·0	24·0			885	126·6	12·1
34·7	29·2	24·8				787	114·4	10·6
30·3	25·5					692	102·6	9·0
26·5	22·2					600	90·7	7·5
22·4	18·9					512	78·8	6·0
18·8	15·8					427	67·0	4·5

are reduced to keep the deflection within 1/325th of the span. For General Notes see page vi.

COMPOUND GIRDERS—
Safe Distributed Loads in Tons



10 tons/sq. in.		Weight per foot in lbs.	10 tons/sq. in. SPANS			
Joist	Flange Plates		12	14	16	18
10 × 8 × 55	12 × 1½	181·0	Riv	ets ⅞"	dia.	
10 × 8 × 55	12 × 1	140·2			64·7	57·3
10 × 8 × 55	12 × ⅞	130·0		67·3	59·0	52·5
10 × 8 × 55	12 × ¾	119·8	71·5	61·2	53·5	47·5
10 × 8 × 55	12 × ⅝	109·6	64·0	55·0	48·0	42·8
10 × 8 × 55	12 × ½	99·4	56·9	48·8	42·7	37·1
10 × 8 × 55	12 × ⅜	89·2	49·7	42·6	37·4	31·6
10 × 6 × 40	10 × 1½	144·3	Riv	ets ¾"	dia.	
10 × 6 × 40	10 × 1¼	127·3			60·7	54·0
10 × 6 × 40	10 × 1	110·4		58·7	51·4	45·7
10 × 6 × 40	10 × ⅞	101·8	62·0	53·2	46·4	41·3
10 × 6 × 40	10 × ¾	93·3	55·9	47·9	42·0	37·4
10 × 6 × 40	10 × ⅝	84·9	50·0	42·8	37·4	33·2
10 × 6 × 40	10 × ½	76·3	43·8	37·4	32·9	28·5
10 × 6 × 40	10 × ⅜	67·8	37·8	32·3	28·3	24·1

The above loads are based on a safe working stress of 10 tons per square inch. Loads to right of zig-zag line

and Dimensions and Properties

IN FEET						Moment of Inertia	Moduli of Section x—x	
20	22	24	26	28	30	x—x	Girder	1 in. Plate Breadth
70·0	60·0	50·1	42·9	37·0	32·1	1362	209·6	15·3
49·5	40·9	34·5	29·3	25·3	22·0	930	155·0	10·1
44·4	36·6	30·9	26·2	22·6		831	141·5	8·8
39·5	32·6	27·4	23·4			739	128·5	7·5
34·6	28·6	24·0				650	115·5	6·3
30·1	24·8					564	102·5	5·0
25·7	21·2					481	89·5	3·7
56·2	45·1	40·6	34·6	29·9	*26·0*	1097	168·7	15·3
48·6	40·2	33·9	28·9	24·9		913	146·0	12·7
39·4	32·5	27·3	23·1	20·0		738	123·0	10·1
35·0	28·9	24·3	20·7			656	111·8	8·8
30·8	25·4	21·4	18·2			579	100·7	7·5
26·9	22·2	18·8				504	89·9	6·3
23·1	19·1					434	78·9	5·0
19·5	16·1					366	67·9	3·7

are reduced to keep the deflection within 1/325th of the span. *For General Notes see page vi.*

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

10 tons/sq. in. Joist	Flange Plates	Weight per foot in lbs.	10 tons/sq. in. SPANS 10	12	14	16
10 × 5 × 30	9 × 1½	124·1	Riv	ets ¾"	dia.	
10 × 5 × 30	9 × 1¼	108·9				52·0
10 × 5 × 30	9 × 1	93·6			49·5	43·2
10 × 5 × 30	9 × ⅞	85·8		52·2	44·6	39·0
10 × 5 × 30	9 × ¾	78·2	55·7	46·5	39·8	34·9
10 × 5 × 30	9 × ⅝	71·0	49·3	41·1	35·2	30·8
10 × 5 × 30	9 × ½	62·9	42·6	35·5	30·5	26·7
10 × 5 × 30	9 × ⅜	55·5	36·1	30·0	25·8	22·5
10 × 4½ × 25	9 × 1	88·4	Riv	ets ¾"	dia.	41·8
10 × 4½ × 25	9 × ⅞	80·8			42·7	37·4
10 × 4½ × 25	9 × ¾	73·5		44·4	38·1	33·3
10 × 4½ × 25	9 × ⅝	66·0	46·6	38·9	33·3	29·1
10 × 4½ × 25	9 × ½	57·8	40·0	33·4	28·6	25·0
10 × 4½ × 25	9 × ⅜	50·5	33·2	27·7	23·7	20·8

The above loads are based on a safe working stress of 10 tons per square inch. Loads to right of zig-zag line

and Dimensions and Properties

IN FEET						Moment of Inertia	Moduli of Section x—x	
18	20	22	24	26	28	x—x	Girder	1 in. Plate Breadth
53·7	48·2	41·5	34·9	29·7	25·6	943	145·1	15·3
46·2	41·4	34·4	28·9	24·7	21·2	778	124·4	12·7
38·4	33·3	27·4	22·9	19·6		623	103·8	10·1
34·8	29·4	24·3	20·4			551	93·7	8·8
31·0	25·7	21·3	17·8			482	83·8	7·5
27·3	22·2	18·3				416	73·9	6·3
23·2	18·7					352	64·0	5·0
19·2	15·6					292	54·1	3·7
37·0	32·1	26·4	22·2	19·0	16·3	601	100·1	10·1
33·3	28·2	23·4	19·6	16·7		528	89·9	8·8
29·6	24·5	20·3	17·0			459	79·9	7·5
25·9	21·0	17·4				393	70·0	6·3
21·8	17·6					330	60·0	5·0
17·7	14·4					269	50·0	3·7

are reduced to keep the deflection within 1/325th of the span. *For General Notes see page vi.*

COMPOUND GIRDERS—
Safe Distributed Loads in Tons

10 tons/sq. in. Joist	Flange Plates	Weight per foot in lbs.	10 tons/sq. in. SPANS 6	8	10	12
9 × 7 × 50	10 × 1½	155·6	Riv	ets ⅞″	dia.	
9 × 7 × 50	10 × 1	122·0				64·2
9 × 7 × 50	10 × ¾	105·0			64·2	53·6
9 × 7 × 50	10 × ½	88·0		64·6	51·6	43·0
9 × 7 × 50	10 × ⅜	79·1	75·4	56·6	45·3	37·8
8 × 6 × 35	10 × 1	105·3	Riv	ets ¾″	dia.	
8 × 6 × 35	10 × ¾	88·3				42·6
8 × 6 × 35	10 × ⅝	79·8			45·6	38·0
8 × 6 × 35	10 × ½	71·3		49·4	39·6	33·1
8 × 6 × 35	10 × ⅜	62·8	56·0	42·0	33·6	28·0
8 × 5 × 28	9 × 1	91·5	Riv	ets ¾″	dia.	
8 × 5 × 28	9 × ¾	76·3			43·8	36·6
8 × 5 × 28	9 × ⅝	69·0		48·2	38·5	32·1
8 × 5 × 28	9 × ½	61·0	55·6	41·7	33·4	27·7
8 × 5 × 28	9 × ⅜	53·5	46·7	35·1	28·1	23·4

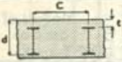
The above loads are based on a safe working stress of 10 tons per square inch. Loads to right of zig-zag line

and Dimensions and Properties

IN FEET						Moment of Inertia	Moduli of Section x—x	
14	16	18	20	22	24	x—x	Girder	1 in. Plate Breadth
	65·0	57·8	45·0	41·2	34·7	937	**156·2**	13·9
55·2	48·3	41·9	33·8	28·1	23·6	637	**115·9**	9·1
46·0	40·1	33·4	27·1	22·4	18·8	506	**96·5**	6·8
36·8	32·2	25·5	20·7	17·1		387	**77·5**	4·5
32·4	27·6	21·8	17·7			332	**68·0**	3·3
45·2	39·6	31·2	25·4	21·0	*17·5*	475	**95·0**	8·1
36·7	30·4	24·0	19·5	16·1	13·5	366	**77·0**	6·1
32·6	26·3	20·7	16·8	13·9		315	**68·3**	5·0
28·2	22·1	17·6	14·3			267	**59·4**	4·0
24·0	18·5	14·6				222	**50·5**	3·0
39·0	34·1	26·9	21·9	*18·1*	15·2	410	**82·0**	8·1
31·3	26·1	20·6	16·6	13·7		313	**65·8**	6·1
27·6	22·4	17·7	14·3			268	**57·8**	5·0
23·9	18·7	14·9				225	**50·0**	4·0
20·0	15·4	12·2				185	**42·2**	3·0

are reduced to keep the deflection within 1/325th of the span. *For General Notes see page vi.*

FILLER JOISTS—In accordance
Safe Distributed Loads in lbs./sq. ft.



Filler Joist	Weight of Joist lbs. sq. ft.	Concrete Cover t ins.	Centres of Joists c ins.	SPANS 10	11	12
7 × 4 × 16	8·00	2	24			
7 × 4 × 16	6·40	2	30			554
7 × 4 × 16	5·33	2	36		560	472
7 × 4 × 16	8·00	1	24			
7 × 4 × 16	6·40	1	30			485
7 × 4 × 16	5·33	1	36		487	410
7 × 3½ × 15	7·50	2	24			
7 × 3½ × 15	6·00	2	30			515
7 × 3½ × 15	5·00	2	36		522	439
7 × 3½ × 15	7·50	1	24			535
7 × 3½ × 15	6·00	1	30		532	446
7 × 3½ × 15	5·00	1	36	544	450	378
6 × 3 × 12	6·00	2	24		515	432
6 × 3 × 12	4·80	2	30	537	444	373
6 × 3 × 12	4·00	2	36	458	379	319

For notes relating to above see page x
See pages 134 to 141 for Neutral axis depths

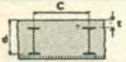
with B.S.S. 449, 1937
for Filler Joists embedded in concrete

IN FEET

13	14	15	16	17	18	19	20	22	24
550	475	413	363	321	286	258	232	191	160
472	407	355	312	276	246	221	200	165	138
402	346	302	265	235	209	188	170	140	118
499	432	374	329	291	260	234	211	174	146
413	356	310	273	242	216	193	175	145	121
349	300	262	230	204	182	163	148	122	102
500	431	375	329	292	260	234	212	174	146
438	378	330	289	256	229	205	185	153	128
374	322	282	246	218	195	175	158	130	109
455	393	342	300	266	238	213	193	159	134
380	328	286	251	222	200	178	161	133	112
322	278	242	213	188	168	151	136	113	95
368	318	277	244	215	192	173	156	129	108
318	274	239	210	186	166	149	134	111	93
271	234	204	179	159	142	127	115	95	80

Loads to right of zig-zag line are for spans greater than 32d.

FILLER JOISTS—In accordance
Safe Distributed Loads in lbs./sq. ft.



Filler Joist	Weight of Joist lbs. sq. ft.	Concrete Cover t ins.	Centres of Joists c ins.	SPANS		
				8	9	10
6 × 3 × 12	6·00	1	24			550
6 × 3 × 12	4·80	1	30		556	450
6 × 3 × 12	4·00	1	36		473	384
5 × 3 × 11	5·50	2	24			483
5 × 3 × 11	4·40	2	30		541	438
5 × 3 × 11	3·67	2	36		474	384
5 × 3 × 11	5·50	1	24		511	414
5 × 3 × 11	4·40	1	30	556	440	356
5 × 3 × 11	3·67	1	36	478	377	305
5 × 2½ × 9	4·50	2	24		542	439
5 × 2½ × 9	3·60	2	30		455	368
5 × 2½ × 9	3·00	2	36	490	387	313
5 × 2½ × 9	4·50	1	24		448	363
5 × 2½ × 9	3·60	1	30	466	368	298
5 × 2½ × 9	3·00	1	36	388	306	248

For notes relating to above see page x
See pages 134 to 141 for Neutral axis depths

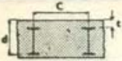
with B.S.S. 449, 1937
for Filler Joists embedded in concrete

IN FEET

11	12	13	14	15	16	17	18	19	20
455	383	326	282	245	215	191	170	153	138
372	313	266	230	200	176	156	139	125	113
316	266	227	196	170	150	133	118	106	96
399	335	286	246	215	188	167	149	134	121
362	304	259	224	195	171	152	135	121	110
318	267	228	196	171	150	133	119	107	97
342	288	245	211	184	162	143	128	115	104
294	247	211	182	158	139	123	110	99	89
252	212	181	156	136	119	106	94	85	
363	305	260	224	195	172	152	136	122	110
304	256	218	188	164	144	128	114	102	92
259	218	185	160	139	122	108	97	87	
300	252	215	185	161	142	125	112	101	91
247	207	177	152	133	117	103	92	83	
205	172	147	126	110	97	86			

Loads to right of zig-zag line are for spans greater than 32d.

FILLER JOISTS—In accordance
Safe Distributed Loads in lbs./sq. ft.



Filler Joist	Weight of Joist lbs. sq. ft.	Concrete Cover t ins.	Centres of Joists c ins.	SPANS 6	SPANS 7	SPANS 8
4¾×1¾×6½	3·25	2	24			500
4¾×1¾×6½	2·60	2	30		535	410
4¾×1¾×6½	2·17	2	36		449	344
4¾×1¾×6½	3·25	1	24		515	394
4¾×1¾×6½	2·60	1	30		417	319
4¾×1¾×6½	2·17	1	36	480	353	270
4×3 ×10	5·00	2	24			550
4×3 ×10	4·00	2	30			505
4×3 ×10	3·33	2	36			464
4×3 ×10	5·00	1	24			454
4×3 ×10	4·00	1	30		528	405
4×3 ×10	3·33	1	36		465	356
4×1¾×5	2·50	2	24		465	356
4×1¾×5	2·00	2	30	518	380	292
4×1¾×5	1·67	2	36	437	322	246

For notes relating to above see page x
See pages 134 to 141 for Neutral axis depths

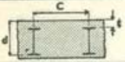
with B.S.S. 449, 1937
for Filler Joists embedded in concrete

IN FEET

9	10	11	12	13	14	15	16	17	18
395	319	264	222	189	163	142	125	110	99
324	262	216	182	155	134	116	102	91	81
272	220	182	153	130	112	98	86		
311	252	208	175	149	129	112	98	87	
252	204	169	142	121	104	91	80		
214	173	143	120	102	88				
435	352	291	245	208	180	157	138	122	109
400	323	267	224	191	165	144	126	112	100
366	297	246	206	176	152	132	116	103	92
358	290	240	202	172	148	129	114	100	
320	259	214	180	153	132	116	101	90	
282	228	188	158	135	116	102	89		
281	228	188	158	135	116	102	89		
230	186	154	130	110	95	84			
195	157	130	110	93	81				

Loads to right of zig-zag line are for spans greater than 32d.

FILLER JOISTS—In accordance
Safe Distributed Loads in lbs./sq. ft.



Filler Joist	Weight of Joist lbs. sq. ft.	Concrete Cover t ins.	Centres of Joists c ins.	SPANS 4	5	6
$4 \times 1\frac{3}{4} \times 5$	2·50	1	24			478
$4 \times 1\frac{3}{4} \times 5$	2·00	1	30		565	392
$4 \times 1\frac{3}{4} \times 5$	1·67	1	36		480	334
$3 \times 3 \times 8\frac{1}{2}$	4·25	2	24			
$3 \times 3 \times 8\frac{1}{2}$	3·40	2	30			
$3 \times 3 \times 8\frac{1}{2}$	2·83	2	36			
$3 \times 3 \times 8\frac{1}{2}$	4·25	1	24			527
$3 \times 3 \times 8\frac{1}{2}$	3·40	1	30			461
$3 \times 3 \times 8\frac{1}{2}$	2·83	1	36			426
$3 \times 1\frac{1}{2} \times 4$	2·00	2	24			447
$3 \times 1\frac{1}{2} \times 4$	1·60	2	30		530	368
$3 \times 1\frac{1}{2} \times 4$	1·33	2	36		453	314
$3 \times 1\frac{1}{2} \times 4$	2·67	1	18		581	404
$3 \times 1\frac{1}{2} \times 4$	2·00	1	24		440	306
$3 \times 1\frac{1}{2} \times 4$	1·60	1	30	578	371	257

For notes relating to above see page x
See pages 134 to 141 for Neutral axis depths

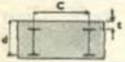
with B.S.S. 449, 1937
for Filler Joists embedded in concrete

IN FEET

7	8	9	10	11	12	13	14	15	16
352	269	212	172	142	119	102	88		
288	221	174	141	117	98	84			
245	188	148	120	100	84				
497	380	300	244	201	169	144	124	108	95
462	353	279	226	187	157	134	115	100	88
433	332	262	212	175	147	125	109	94	83
387	297	234	190	157	132				
339	259	205	166	137	115				
313	240	189	153	127	107				
328	251	199	161	133	112				
270	207	163	132	109	92				
231	177	140	113	94					
296	227	179	145	120	101				
224	172	136	110	91					
189	145	114	93						

Loads to right of zig-zag line are for spans greater than 32d.

FILLER JOISTS — In accordance
Safe Distributed Loads in lbs./sq. ft.



Filler Joist	Wt. of Joist lbs. sq. ft.	Concrete Cover, t inches	Centres of Joists, c inches	SPANS 11	12	13
7 × 4 × 16	8·00	2	24			
7 × 4 × 16	6·40	2	30			519
7 × 4 × 16	5·33	2	36		510	435
7 × 4 × 16	8·00	1	24			
7 × 4 × 16	6·40	1	30		563	480
7 × 4 × 16	5·33	1	36	558	470	401
7 × 3½ × 15	7·50	2	24			
7 × 3½ × 15	6·00	2	30		555	473
7 × 3½ × 15	5·00	2	36	582	490	417
7 × 3½ × 15	7·50	1	24			545
7 × 3½ × 15	6·00	1	30		511	436
7 × 3½ × 15	5·00	1	36	519	436	371
6 × 3 × 12	6·00	2	24	562	472	402
6 × 3 × 12	4·80	2	30	467	393	335
6 × 3 × 12	4·00	2	36	433	365	310

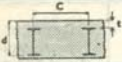
Concrete floor may be haunched between Filler Joists; concrete depth **not** *less than neutral axis distance given + 1″.*

with B.S.S. 449—1940 (revision). for Filler Joists embedded in Concrete

IN FEET									Depth of Neutral Axis from top of Concrete in inches
14	15	16	17	18	19	20	22	24	
561	488	428	380	340	305	275	227	191	3·46
448	391	344	305	272	244	220	182	152	3·26
375	327	287	254	226	204	184	152	127	3·08
517	451	396	351	313	281	254	209	176	2·97
415	361	317	281	251	225	203	168	141	2·82
345	301	265	235	209	188	169	140	117	2·68
509	443	390	346	308	277	249	206	173	3·39
408	356	313	277	247	222	200	165	138	3·18
360	314	275	244	218	196	177	146	122	3·02
470	410	361	319	285	256	230	191	160	2·93
376	327	288	255	227	204	184	152	128	2·76
320	279	246	217	193	174	157	130	110	2·62
347	303	266	236	210	188	170	141	118	2·98
288	252	221	196	175	157	141	117	98	2·79
268	234	205	182	162	145	132	108	92	2·64

For Notes relating to above see page x. Loads to right of zig-zag line are for spans greater than 32d.

FILLER JOISTS — In accordance
Safe Distributed Loads in lbs./sq. ft.



Filler Joist	Wt. of Joist lbs. sq. ft.	Concrete Cover, t inches	Centres of Joists, c inches	SPANS 9	10	11
6 × 3 × 12	6·00	1	24			518
6 × 3 × 12	4·80	1	30		502	415
6 × 3 × 12	4·00	1	36	556	451	373
5 × 3 × 11	5·50	2	24		532	440
5 × 3 × 11	4·40	2	30	541	438	362
5 × 3 × 11	3·67	2	36	505	409	338
5 × 3 × 11	5·50	1	24		490	406
5 × 3 × 11	4·40	1	30	486	394	325
5 × 3 × 11	3·67	1	36	420	340	282
5 × 2½ × 9	4·50	2	24	542	438	363
5 × 2½ × 9	3·60	2	30	497	401	333
5 × 2½ × 9	3·00	2	36	464	376	311
5 × 2½ × 9	4·50	1	24	483	392	324
5 × 2½ × 9	3·60	1	30	413	335	277
5 × 2½ × 9	3·00	1	36	377	305	253

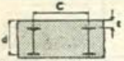
Concrete floor may be haunched between Filler Joists; concrete depth **not** *less than neutral axis distance given + 1″.*

with B.S.S. 449--1940 (revision).
for Filler Joists embedded in Concrete

IN FEET									Depth of Neutral Axis from top of Concrete in inches
12	13	14	15	16	17	18	19	20	
437	372	321	279	245	218	194	174	157	2·54
347	298	257	224	197	174	155	139	126	2·38
314	267	230	201	177	157	140	125	113	2·26
370	315	272	236	208	184	164	147	133	2·74
304	259	224	195	171	152	135	121	110	2·53
284	242	208	182	160	141	126	113	102	2·42
340	291	250	218	192	170	152	136	123	2·24
273	233	201	175	154	136	121	109	98	2·10
237	202	174	152	133	118	106	95	85	2·02
305	260	224	195	172	152	136	122	110	2·54
280	239	205	179	157	140	124	112	101	2·38
262	223	193	168	147	131	117	105	95	2·24
273	232	200	175	153	136	121	109	98	2·13
233	198	171	149	131	116	104	93	83	2·00
213	181	156	136	120	106	95	85		1·87

For Notes relating to above see page x. Loads to right of zig-zag line are for spans greater than 32d.

FILLER JOISTS—In accordance
Safe Distributed Loads in lbs./sq. ft.



Filler Joist	Wt. of Joist lbs. sq. ft.	Concrete Cover, t inches	Centres of Joists, c inches	SPANS 7	8	9
$4\frac{3}{4} \times 1\frac{3}{4} \times 6\frac{1}{2}$	3·25	2	24		558	442
$4\frac{3}{4} \times 1\frac{3}{4} \times 6\frac{1}{2}$	2·60	2	30		504	398
$4\frac{3}{4} \times 1\frac{3}{4} \times 6\frac{1}{2}$	2·17	2	36	552	423	335
$4\frac{3}{4} \times 1\frac{3}{4} \times 6\frac{1}{2}$	3·25	1	24		452	357
$4\frac{3}{4} \times 1\frac{3}{4} \times 6\frac{1}{2}$	2·60	1	30	513	394	311
$4\frac{3}{4} \times 1\frac{3}{4} \times 6\frac{1}{2}$	2·17	1	36	437	335	265
$4 \times 3 \times 10$	5·00	2	24			467
$4 \times 3 \times 10$	4·00	2	30		505	400
$4 \times 3 \times 10$	3·33	2	36		470	371
$4 \times 3 \times 10$	5·00	1	24		545	430
$4 \times 3 \times 10$	4·00	1	30	570	436	345
$4 \times 3 \times 10$	3·33	1	36	485	372	294
$4 \times 1\frac{3}{4} \times 5$	2·50	2	24	544	416	329
$4 \times 1\frac{3}{4} \times 5$	2·00	2	30	470	360	284
$4 \times 1\frac{3}{4} \times 5$	1·67	2	36	395	303	239

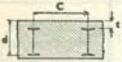
Concrete floor may be haunched between Filler Joist s; concrete depth **not** *less than neutral axis distance given + 1".*

with B.S.S. 449—1940 (revision).
for Filler Joists embedded in Concrete

IN FEET									Depth of Neutral Axis from top of Concrete in inches
10	11	12	13	14	15	16	17	18	
358	296	249	212	183	159	140	124	109	2·26
322	267	224	191	165	144	126	112	100	2·09
271	224	188	160	138	120	106	94	84	1·95
289	238	201	171	148	129	113	100	90	1·89
252	208	175	149	129	112	98	87		1·75
214	177	149	127	110	96	84			1·65
378	312	262	224	193	168	148	131	117	2·42
323	267	224	191	165	144	126	112	100	2·26
300	249	209	178	154	134	118	104	93	2·14
349	289	243	207	178	155	137	121		1·96
279	231	194	165	143	124	109	97		1·84
238	197	165	141	122	105	93	83		1·75
266	221	185	158	136	119	104	93	82	1·95
230	191	160	136	118	102	90			1·80
194	160	135	115	99	86				1·68

For Notes relating to above see page x. Loads to right of zig-zag line are for spans greater than 32d.

FILLER JOISTS—In accordance
Safe Distributed Loads in lbs./sq. ft.



Filler Joist	Wt. of Joist lbs. sq. ft.	Concrete Cover, t inches	Centres of Joists, c inches	SPANS 5	6	7
$4 \times 1\frac{3}{4} \times 5$	2·50	1	24		566	416
$4 \times 1\frac{3}{4} \times 5$	2·00	1	30		482	354
$4 \times 1\frac{3}{4} \times 5$	1·67	1	36		411	302
$3 \times 3 \times 8\frac{1}{2}$	4·25	2	24			503
$3 \times 3 \times 8\frac{1}{2}$	3·40	2	30			462
$3 \times 3 \times 8\frac{1}{2}$	2·83	2	36			433
$3 \times 3 \times 8\frac{1}{2}$	4·25	1	24			466
$3 \times 3 \times 8\frac{1}{2}$	3·40	1	30		506	372
$3 \times 3 \times 8\frac{1}{2}$	2·83	1	36		426	313
$3 \times 1\frac{1}{2} \times 4$	2·00	2	24		513	377
$3 \times 1\frac{1}{2} \times 4$	1·60	2	30		450	331
$3 \times 1\frac{1}{2} \times 4$	1·33	2	36	557	387	285
$3 \times 1\frac{1}{2} \times 4$	2·67	1	18		404	296
$3 \times 1\frac{1}{2} \times 4$	2·00	1	24	515	358	263
$3 \times 1\frac{1}{2} \times 4$	1·60	1	30	456	317	233

Concrete floor may be haunched between Filler Joists; concrete depth **not** *less than neutral axis distance given + 1″.*

with B.S.S. 449—1940 (revision).
for Filler Joists embedded in Concrete

IN FEET									Depth of Neutral Axis from top of Concrete in inches
8	9	10	11	12	13	14	15	16	
319	252	204	169	142	121	104	91		1·61
271	214	174	143	121	103	89			1·49
231	183	148	122	103	87				1·41
386	305	247	204	172	146	126	110	94	2·10
353	279	226	187	157	134	115	100	88	1·96
332	262	212	175	147	125	108	94	83	1·85
357	282	228	188	158	135	117			1·65
285	225	182	151	126	108	93			1·55
240	189	153	127	107	91				1·47
289	229	185	153	128	110	95			1·65
254	200	162	134	113	96				1·52
218	173	139	115	97	83				1·43
227	179	145	120	101	86				1·44
201	159	129	106	90					1·30
179	141	114	95						1·23

For Notes relating to above see page x. Loads to right of zig-zag line are for spans greater than 32d.

CAST-IN-SITU FLOORS

Approx. THICKNESSES in inches of Concrete Floors (with suitable continuous reinforcement) for various Super Loads

Span in Feet	Superimposed Loads in lbs. per sq. ft.							
	40	84	112	168	224	280	336	448
6′	3	3	3	3½	4	4½	4½	5
7′	3	3	3½	4	4½	5	5½	6
8′	3	3½	4	4½	5	5½	6	7
9′	3½	4	4½	5	5½	6½	7	8
10′	4	4½	5	5½	6½	7	7½	8½
11′	4½	5	5½	6½	7	8	8½	—
12′	5	5½	6	7	8	8½	—	—
13′	5½	6	6½	7½	8½	—	—	—
14′	6	6½	7	8	8½	—	—	—

Note.—Thickness of Concrete Floors should not be less than 1/30th Span.

The weight of above floors can be calculated from Concrete weighing 144 lbs. per c. ft.

The above tables are purely to act as a guide to estimate the thickness, spans and weights for

PRE-CAST FLOORS

Approx. SPANS in Feet of pre-cast Concrete Slabs (suitably reinforced) for various Super Loads

Depth of Floor	Wt. in lbs./sq. ft.	Superimposed Loads in lbs. per Sq. Ft.						Condition of Fixity*
		50	80	100	168	224	336	
5"	36	11·0	10·0	9·5	8·0	7·0	6·0	F.
		—	11·0	10·5	9·0	8·0	7·0	O.E.F.
6"	40	13·0	12·0	11·0	9·5	8·5	7·0	F.
		—	13·0	12·0	10·5	9·5	8·0	O.E.F.
7"	45	15·0	14·0	13·0	11·0	10·0	8·5	F.
		—	15·0	14·5	12·5	11·5	9·5	O.E.F.
8"	50	—	16·0	15·5	13·5	12·0	10·0	F.
		—	—	16·5	15·0	13·5	11·5	O.E.F.
10"	70	20·0	18·5	18·0	16·0	15·0	12·5	F.
		—	20·0	19·5	17·5	16·5	14·0	O.E.F.

* F indicates Free; O.E.F. indicates One End Free.

design purposes. The weights and thicknesses do not include Floor finish, or Ceiling.

Table of SAFE WORKING STRESSES in Tons per Sq. In. for Fl values

l = length of stan. in inches

$\frac{l}{r}$	Tons per sq. in. Gross Section	$\frac{l}{r}$	Tons per sq. in. Gross Section	$\frac{l}{r}$	Tons per sq. in. Gross Section	$\frac{l}{r}$	Tons per sq. in. Gross Section
10	7·40	25	7·05	40	6·64	55	6·10
11	7·38	26	7·03	41	6·61	56	6·06
12	7·36	27	7·00	42	6·57	57	6·02
13	7·33	28	6·98	43	6·54	58	5·98
14	7·31	29	6·95	44	6·51	59	5·93
15	7·29	30	6·92	45	6·48	60	5·89
16	7·27	31	6·90	46	6·44	61	5·84
17	7·24	32	6·87	47	6·41	62	5·80
18	7·22	33	6·84	48	6·37	63	5·75
19	7·20	34	6·81	49	6·33	64	5·71
20	7·17	35	6·79	50	6·30	65	5·66
21	7·15	36	6·76	51	6·26	66	5·61
22	7·13	37	6·73	52	6·22	67	5·56
23	7·10	38	6·70	53	6·18	68	5·51
24	7·08	39	6·67	54	6·14	69	5·46

See page xiv to determine

for STANCHIONS and STRUTS (BSS Par. 15) of $\frac{l}{r}$ from 10 to 129.

r = least radius of gyration in inches

$\frac{l}{r}$	Tons per sq. in. Gross Section	$\frac{l}{r}$	Tons per sq. in. Gross Section	$\frac{l}{r}$	Tons per sq. in. Gross Section	$\frac{l}{r}$	Tons per sq. in. Gross Section
70	5·41	85	4·60	100	3·81	115	3·13
71	5·36	86	4·55	101	3·76	116	3·09
72	5·31	87	4·49	102	3·71	117	3·05
73	5·25	88	4·44	103	3·66	118	3·01
74	5·20	89	4·39	104	3·61	119	2·97
75	5·15	90	4·33	105	3·57	120	2·93
76	5·09	91	4·28	106	3·52	121	2·89
77	5·04	92	4·22	107	3·47	122	2·85
78	4·99	93	4·17	108	3·43	123	2·82
79	4·93	94	4·12	109	3·38	124	2·78
80	4·88	95	4·06	110	3·34	125	2·75
81	4·82	96	4·01	111	3·29	126	2·71
82	4·77	97	3·96	112	3·25	127	2·68
83	4·71	98	3·91	113	3·21	128	2·64
84	4·66	99	3·86	114	3·17	129	2·61

length l for various conditions of End Fixity.

Table of SAFE WORKING STRESSES in Tons per Sq. In. for Fl values

l = length of stan. in inches

$\frac{l}{r}$	Tons per sq. in. Gross Section	$\frac{l}{r}$	Tons per sq. in. Gross Section	$\frac{l}{r}$	Tons per sq. in. Gross Section	$\frac{l}{r}$	Tons per sq. in. Gross Section
130	2·58	145	2·15	160	1·81	175	1·54
131	2·54	146	2·12	161	1·79	176	1·52
132	2·51	147	2·10	162	1·77	177	1·51
133	2·48	148	2·07	163	1·75	178	1·49
134	2·45	149	2·05	164	1·73	179	1·48
135	2·42	150	2·02	165	1·71	180	1·46
136	2·39	151	2·00	166	1·69	181	1·45
137	2·36	152	1·98	167	1·67	182	1·43
138	2·33	153	1·96	168	1·66	183	1·42
139	2·31	154	1·93	169	1·64	184	1·41
140	2·28	155	1·91	170	1·62	185	1·39
141	2·25	156	1·89	171	1·61	186	1·38
142	2·22	157	1·87	172	1·59	187	1·36
143	2·20	158	1·85	173	1·57	188	1·35
144	2·17	159	1·83	174	1·56	189	1·34

See page xiv to determine

for STANCHIONS and STRUTS (BSS Par. 15) of $\frac{l}{r}$ from 130 to 240.

r = least radius of gyration in inches

$\frac{l}{r}$	Tons per sq. in. Gross Section	$\frac{l}{r}$	Tons per sq. in. Gross Section	$\frac{l}{r}$	Tons per sq. in. Gross Section	$\frac{l}{r}$	Tons per sq. in. Gross Section
190	1·33	205	1·15	220	1·01	235	·89
191	1·31	206	1·14	221	1·00	236	·89
192	1·30	207	1·13	222	·99	237	·88
193	1·29	208	1·12	223	·98	238	·87
194	1·28	209	1·11	224	·98	239	·86
195	1·26	210	1·10	225	·97	240	·86
196	1·25	211	1·09	226	·96		
197	1·24	212	1·08	227	·95		
198	1·23	213	1·07	228	·94		
199	1·22	214	1·06	229	·94		
200	1·21	215	1·05	230	·93		
201	1·19	216	1·05	231	·92		
202	1·18	217	1·04	232	·91		
203	1·17	218	1·03	233	·91		
204	1·16	219	1·02	234	·90		

length l for various conditions of End Fixity.

TABLE OF SAFE WORKING (BSS 449 Pars. 15 and 17) for concentrically and

$\frac{l}{r}$	Fl	Actual Direct Compressive							
		·75	1·00	1·25	1·50	1·75	2·00	2·25	2·50
		Increased Working Stresses							
35	6·79	6·95	6·94	6·93	6·93	6·92	6·91	6·90	6·89
40	6·64	6·87	6·86	6·84	6·83	6·82	6·81	6·80	6·79
45	6·48	6·78	6·77	6·75	6·74	6·72	6·71	6·69	6·68
50	6·30	6·70	6·68	6·66	6·64	6·63	6·61	6·59	6·57
55	6·10	6·60	6·58	6·56	6·53	6·51	6·48	6·46	6·44
60	5·89	6·51	6·48	6·45	6·42	6·39	6·36	6·33	6·30
65	5·66	6·41	6·37	6·33	6·29	6·25	6·22	6·18	6·14
70	5·41	6·30	6·26	6·21	6·16	6·11	6·06	6·01	5·96
75	5·15	6·20	6·14	6·08	6·02	5·96	5·90	5·84	5·78
80	4·88	6·09	6·01	5·94	5·87	5·80	5·73	5·66	5·59
85	4·60	5·96	5·87	5·78	5·70	5·61	5·52	5·43	5·34
90	4·33	5·83	5·72	5·61	5·52	5·41	5·29	5·18	5·08
95	4·06	5·71	5·58	5·45	5·33	5·22	5·08	4·95	4·83
100	3·81	5·57	5·42	5·28	5·13	4·99	4·84	4·71	4·55
105	3·57	5·42	5·26	5·09	4·92	4·75	4·59	4·42	4·25
110	3·34	5·27	5·08	4·88	4·69	4·50	4·31	4·11	3·92
115	3·13	5·13	4·91	4·71	4·48	4·27	4·05	3·83	3·62

STRESSES for F1 and F2 Values STANCHIONS and STRUTS, eccentrically loaded

Stress (fc) Tons per Square Inch									
2·75	3·00	3·25	3·50	4·00	4·50	5·00	5·50	6·00	6·50
(F2) Tons per Square Inch									
6·88	6·88	6·87	6·86	6·84	6·83	6·81	6·79	6·78	6·76
6·78	6·76	6·75	6·74	6·72	6·70	6·67	6·65	6·63	6·61
6·67	6·65	6·64	6·62	6·59	6·56	6·53	6·51	6·48	
6·55	6·54	6·52	6·50	6·46	6·43	6·39	6·36	6·32	
6·41	6·39	6·37	6·34	6·30	6·25	6·20	6·16	6·11	
6·27	6·24	6·21	6·18	6·12	6·07	6·01	5·95		
6·10	6·06	6·02	5·98	5·91	5·83	5·75	5·67		
5·92	5·87	5·82	5·77	5·67	5·58	5·48			
5·72	5·66	5·60	5·54	5·42	5·30	5·19			
5·51	5·44	5·37	5·30	5·16	5·01				
5·25	5·17	5·08	4·99	4·81	4·64				
4·97	4·86	4·75	4·64	4·43					
4·71	4·58	4·45	4·33	4·08					
4·41	4·26	4·12	3·97						
4·09	3·92	3·75	3·58						
3·73	3·53	3·34							
3·42	3·19								

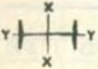
TABLE OF SAFE WORKING (BSS 449 Pars. 15 and 17) for concentrically and

$\frac{l}{r}$	FI	Actual Direct Compressive				
		·75	1·00	1·25	1·50	1·75
		Increased Working Stresses				
120	2·93	4·98	4·73	4·49	4·25	4·01
125	2·75	4·84	4·58	4·32	4·06	3·80
130	2·58	4·70	4·42	4·13	3·85	3·56
135	2·42	4·55	4·24	3·93	3·62	3·32
140	2·28	4·39	4·05	3·71	3·38	3·04
145	2·15	4·22	3·85	3·48	3·11	2·74
150	2·02	4·03	3·63	3·22	2·81	2·41
155	1·91	3·88	3·45	3·02	2·59	2·16
160	1·81	3·73	3·27	2·81	2·35	1·89
165	1·71	3·56	3·07	2·58	2·09	
170	1·62	3·38	2·86	2·33	1·81	
175	1·54	3·27	2·73	2·19	1·66	
180	1·46	3·15	2·60	2·05		
185	1·39	2·94	2·35	1·76		
190	1·33	2·72	2·07	1·43		
195	1·26	2·58	1·95			
200	1·21	2·44	1·75			

STRESSES for F1 and F2 Values STANCHIONS and STRUTS, eccentrically loaded

Stress (fc) Tons per Square Inch						
2·00	2·25	2·50	2·75	3·00	3·25	3·50
(F2) Tons per Square Inch						
3·77	3·53	3·29	3·05			
3·53	3·27	3·01				
3·28	3·01	2·71				
3·01	2·70					
2·72	2·37					
2·37						

JOIST STANCHIONS—
Safe Concentric Loads in Tons



Section	Area in sq. ins.	EFFECTIVE					
		7	8	9	10	11	12
*24 × 7½ × 100	29·40	178	168	156	143	130	118
24 × 7½ × 95	27·94	169	159	148	136	124	112
22 × 7 × 75	22·06	128	118	108	97·6	87·2	77·7
20 × 7½ × 89	26·19	160	151	142	131	120	109
20 × 6½ × 65	19·12	109	100	90·7	81·1	72·0	63·9
18 × 8 × 80	23·53	149	142	135	127	118	109
18 × 7 × 75	22·09	132	123	114	104	94·4	84·9
18 × 6 × 55	16·18	88·0	79·5	70·6	62·2	54·6	48·0
16 × 8 × 75	22·06	140	135	128	121	113	105
16 × 6 × 62	18·21	99·5	90·1	80·3	70·8	62·2	54·7
16 × 6 × 50	14·71	81·2	73·8	66·0	58·4	51·4	45·3
*15 × 6 × 59	17·35	97·2	88·9	79·8	71·0	62·8	55·4
15 × 6 × 45	13·24	72·7	66·0	58·9	52·0	45·8	40·3
15 × 5 × 42	12·36	56·4	48·3	41·1	35·0	30·0	25·9
14 × 8 × 70	20·59	132	127	121	114	107	100

For notes relating to above see page xi.

and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
13	14	15	16	18	20	Axis $y-y$	Axis $x-x$	Axis $y-y$	Axis $x-x$
106	95·5	86·1	77·6	63·8		17·8	221·1	1·50	9·50
100	90·8	81·8	73·8	60·6		16·7	211·1	1·50	9·52
69·2	61·9	55·1	49·5			11·7	152·4	1·36	8·72
98·9	89·3	80·6	71·9	60·0		16·7	167·3	1·55	7·99
56·6	50·4	44·9	40·2			10·0	122·6	1·31	8·01
101	92·3	84·2	76·9	64·1	53·9	17·4	143·6	1·72	7·41
76·2	68·2	61·3	55·2	45·2		13·3	127·9	1·45	7·22
42·2	37·3	33·2				7·9	93·5	1·21	7·21
97·2	89·0	81·5	74·5	62·3	52·5	17·1	121·7	1·76	6·64
48·1	42·6	37·9				9·1	90·6	1·22	6·32
39·9	35·3	31·4				7·5	77·3	1·24	6·48
49·0	43·4	38·7				9·4	83·8	1·27	6·02
35·5	31·4	27·9				6·6	65·6	1·23	6·10
						4·7	57·1	·98	5·89
92·8	85·5	78·4	71·8	60·2	50·9	16·7	100·8	1·80	5·85

See page xiv regarding various conditions of End Fixity

JOIST STANCHIONS—
Safe Concentric Loads in Tons

Section	Area in sq. ins.	EFFECTIVE					
		5	6	7	8	9	10
14 × 6 × 57	16·78	108	101	94·8	86·9	78·4	69·9
14 × 6 × 46	13·59	86·7	81·7	75·7	69·0	62·0	55·1
*14 × 5½ × 40	11·77	72·5	67·0	60·6	53·7	47·0	40·8
13 × 5 × 35	10·30	61·5	55·7	49·3	42·8	36·7	31·5
12 × 8 × 65	19·12	131	127	123	119	113	108
12 × 6 × 54	15·89	103	97·5	91·2	84·1	76·4	68·6
12 × 6 × 44	13·00	83·6	79·1	73·8	67·7	61·2	54·6
*12 × 5 × 39	11·47	68·5	62·1	55·0	47·7	41·0	35·2
12 × 5 × 32	9·45	55·7	50·4	44·4	38·3	32·8	28·1
*12 × 5 × 30	8·83	52·0	46·7	41·0	35·3	30·1	25·7
*10 × 8 × 70	20·60	141	138	133	128	123	117
10 × 8 × 55	16·18	110	107	104	100	96·2	91·3
*10 × 6 × 42	12·35	80·3	76·4	71·8	66·4	60·6	54·7
10 × 6 × 40	11·77	76·5	72·8	68·3	63·3	57·7	52·2
10 × 5 × 30	8·85	53·2	48·5	43·1	37·6	32·4	27·9
10 × 10 × 62·7 Brd. flge. Area 18·42 sq. ins.					124	121	118

and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
11	12	14	16	18	20	Axis y—y	Axis x—x	Axis y—y	Axis x—x
62·0	54·8	43·0	34·4			9·3	76·2	1·29	5·64
48·6	42·8	33·6				7·2	63·2	1·26	5·71
35·5	30·9	23·8				5·4	53·9	1·12	5·66
27·1	23·5					4·3	43·6	1·03	5·25
102	95·5	81·9	69·2	58·3	49·4	16·3	81·3	1·85	5·05
61·1	54·2	42·9	34·3			9·4	62·6	1·33	4·86
48·5	42·9	33·8	27·0			7·4	52·8	1·30	4·94
30·2	26·2					4·9	43·5	1·03	4·77
24·1	20·8					3·9	36·8	1·01	4·84
22·1	19·2					3·5	34·5	1·00	4·84
110	103	88·9	75·2	63·4	53·7	17·9	69·0	1·86	4·09
86·0	80·4	68·9	58·2	49·0	41·4	13·7	57·7	1·84	4·22
48·9	43·5	34·6	27·8			7·6	42·3	1·36	4·13
46·6	41·4	32·9	26·4			7·2	40·9	1·36	4·17
24·1	20·8					3·9	29·3	1·05	4·06
114	111	103	94·3	84·9	75·7	23·98	67·79	2·55	4·29

JOIST STANCHIONS—
Safe Concentric Loads in Tons

Section	Area in sq. ins.	EFFECTIVE					
		4	5	6	7	8	9
10 × 4½ × 25	7·35	45·9	42·0	37·2	32·0	27·2	23·0
*9 × 7 × 58	17·06	118	115	111	107	102	95·8
9 × 7 × 50	14·71	102	99·2	95·9	92·1	87·7	82·8
9 × 4 × 21	6·18	36·8	32·4	27·5	22·8	18·7	15·6
*8 × 8 × 38	11·13	78·4	76·6	74·7	72·4	69·7	67·1
8 × 6 × 35	10·30	69·9	67·2	64·0	60·2	55·9	51·1
8 × 5 × 28	8·28	54·1	50·8	46·8	42·3	37·4	32·6
8 × 4 × 18	5·30	31·3	27·5	23·2	19·2	15·8	13·1
7 × 4 × 16	4·75	28·5	25·3	21·6	18·0	14·9	12·4
*7 × 3½ × 15	4·42	25·0	21·2	17·4	14·0	11·4	9·3
6 × 5 × 25	7·37	48·1	45·2	41·7	37·6	33·3	29·0
6 × 4½ × 20	5·89	37·0	34·0	30·3	26·3	22·4	19·0
6 × 3 × 12	3·53	18·1	14·5	11·4	8·9	7·1	
5 × 4½ × 20	5·88	38·0	35·4	32·4	28·9	25·2	21·8
*5 × 4½ × 18	5·29	34·0	31·6	28·6	25·3	22·0	18·8

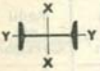
For notes relating to above see page xi.

and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
10	12	14	16	18	20	Axis $y—y$	Axis $x—x$	Axis $y—y$	Axis $x—x$
19·4						2·9	24·5	·94	4·08
89·5	75·9	62·9	51·9	43·0	36·0	13·2	51·0	1·64	3·66
77·5	65·8	54·7	45·1	37·4	31·3	11·5	46·3	1·65	3·76
13·0						2·1	18·0	·82	3·62
63·8	56·5	49·0	41·6	35·3	29·8	9·9	32·8	1·89	3·44
46·2	37·0	29·5	23·7			6·5	28·8	1·38	3·34
28·3	21·4					4·1	22·4	1·11	3·29
10·9						1·7	13·9	·81	3·24
10·4						1·7	11·3	·84	2·89
						1·4	10·3	·74	2·85
25·2	19·0					3·6	14·6	1·11	2·44
16·2	11·9					2·4	11·6	·96	2·43
						·97	7·0	·64	2·44
18·8	14·0					2·9	10·0	1·06	2·06
16·2	12·1					2·5	9·1	1·03	2·07

See page xiv regarding various conditions of End Fixity

JOIST STANCHIONS—
Safe Concentric Loads in Tons

 Section	Area in sq. ins.	EFFECTIVE					
		3	4	5	6	7	8
5 × 3 × 11	3·26	20·0	17·3	14·1	11·2	8·9	7·1
*5 × 2½ × 9	2·65	14·9	11·8	8·8	6·8		
4¾ × 1¾ × 6·5	1·91	7·5	4·9				
*4½ × 2 × 7	2·06	9·6	6·7	4·7			
4 × 3 × 10	2·94	18·0	15·6	12·7	10·1	8·0	6·4
*4 × 3 × 9·5	2·79	17·3	15·0	12·3	9·8	7·8	6·2
4 × 1¾ × 5	1·47	5·6	3·6				
3 × 3 × 8·5	2·52	15·7	13·8	11·5	9·2	7·3	5·9
3 × 1½ × 4	1·18	3·9	2·5				

** Sections marked thus are Old British Standard and Special Sections*

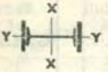
For notes relating to above see page xi.

and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
9	10	12	14	16	18	Axis y—y	Axis x—x	Axis y—y	Axis x—x
						·97	5·5	·67	2·05
						·63	4·36	·55	2·03
						·30	2·83	·37	1·88
						·38	2·96	·43	1·80
						·88	3·90	·67	1·63
						·85	3·76	·68	1·64
						·21	1·83	·36	1·58
						·83	2·54	·70	1·23
						·17	1·11	·33	1·19

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons

		Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
Joist	Flange Plates			12	14	16	18	20
18 × 8 × 80	14 × 2	274·0	79·53	525	507	486	463	437
18 × 8 × 80	14 × 1½	226·4	65·53	430	413	395	375	352
18 × 8 × 80	14 × 1	178·8	51·53	333	319	303	285	265
18 × 8 × 80	14 × ⅞	166·9	48·03	309	295	279	262	242
18 × 8 × 80	14 × ¾	155·0	44·53	284	271	255	238	219
18 × 8 × 80	14 × ⅝	143·1	41·03	260	246	231	214	196
18 × 8 × 80	14 × ½	131·2	37·53	234	221	206	189	172
				Ri	vet	s ⅞	dia.	for
18 × 6 × 55	12 × 2	220·5	64·18	410	391	369	344	318
18 × 6 × 55	12 × 1½	179·7	52·18	331	314	295	273	251
18 × 6 × 55	12 × 1	138·9	40·18	250	236	219	201	182
18 × 6 × 55	12 × ⅞	128·7	37·18	230	216	200	182	164
18 × 6 × 55	12 × ¾	118·5	34·18	209	195	180	163	146
18 × 6 × 55	12 × ⅝	108·3	31·18	188	175	160	144	128
18 × 6 × 55	12 × ½	98·1	28·18	167	154	139	124	109

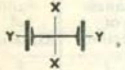
For notes relating to above see page xi.

Single Joist with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
22	24	26	30	34	38	Axis y—y	Axis x—x	Axis y—y	Axis x—x
409	379	350	293	245	205	141	628	3·52	9·32
327	302	277	230	191	160	108	504	3·40	8·99
244	222	202	166	136	113	75	382	3·20	8·62
222	202	183	149	122	102	67	352	3·13	8·51
200	181	164	133	109	90·1	59	322	3·04	8·40
178	160	144	116	95·0		51	292	2·94	8·28
155	138	124	99·5	80·8		43	262	2·82	8·15
18 ×	8 an	d ¾	dia.	for 1	8 × 6				
290	263	237	193	158	131	100	514	3·06	9·39
227	205	184	149	121		76	407	2·96	9·06
163	146	130	104	84·6		52	301	2·78	8·66
147	131	116	93·2	75·6		46	275	2·72	8·54
130	115	103	81·7			40	249	2·65	8·42
113	100	88·8	70·2			34	223	2·56	8·29
96·3	84·5	74·5	58·6			28	197	2·44	8·14

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons

		Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
Joist	Flange Plates			12	14	16	18	20
16 × 8 × 75	14 × 2	269·0	78·05	516	499	479	456	431
16 × 8 × 75	14 × 1½	221·4	64·05	422	405	388	368	346
16 × 8 × 75	14 × 1	173·8	50·05	325	311	296	279	260
16 × 8 × 75	14 × ⅞	161·9	46·55	301	287	273	256	237
16 × 8 × 75	14 × ¾	150·0	43·05	276	263	249	233	215
16 × 8 × 75	14 × ⅝	138·1	39·55	251	239	225	209	192
16 × 8 × 75	14 × ½	126·5	36·05	226	214	200	185	168
				Ri	vet	s ⅞	dia.	for
16 × 6 × 62	12 × 2	227·4	66·20	422	402	379	352	324
16 × 6 × 62	12 × 1½	186·6	54·20	342	324	304	281	256
16 × 6 × 62	12 × 1	145·8	42·20	261	245	227	207	187
16 × 6 × 62	12 × ⅞	135·6	39·20	240	225	208	189	169
16 × 6 × 62	12 × ¾	125·4	36·20	220	205	188	169	151
16 × 6 × 62	12 × ⅝	115·2	33·20	198	184	167	150	133
16 × 6 × 62	12 × ½	105·0	30·20	177	162	146	129	113

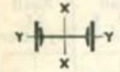
For notes relating to above see page xi.

Single Joist with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
22	24	26	30	34	38	Axis y—y	Axis x—x	Axis y—y	Axis x—x
404	375	346	291	244	204	140	553	3·55	8·42
323	298	274	228	190	159	108	442	3·43	8·09
240	219	200	164	135	113	75	333	3·24	7·74
218	199	181	148	122	101	67	306	3·17	7·64
197	179	161	131	108	89·7	59	280	3·09	7·54
175	158	142	115	94·1		51	253	3·00	7·43
152	136	122	98·5	79·9		42	227	2·87	7·31
16 ×	8 an	d $\frac{3}{4}$	dia.	for 1	6 × 6				
295	267	241	195	159		101	463	3·02	8·36
232	209	187	151	123		76	367	2·91	8·02
168	149	133	106	86·1		52	274	2·73	7·64
151	134	119	95·0			46	250	2·67	7·53
134	119	105	83·5			40	227	2·59	7·41
117	102	91·2	72·1			34	204	2·50	7·28
99·6	87·3	76·7				28	181	2·38	7·14

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS— Safe Concentric Loads in Tons



Joist	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
				10	12	14	16	18
16 × 6 × 50	12 × 2	215·5	62·70	418	402	384	363	339
16 × 6 × 50	12 × 1½	174·7	50·70	336	322	307	289	268
16 × 6 × 50	12 × 1	133·9	38·70	253	242	229	213	196
16 × 6 × 50	12 × ⅞	123·7	35·70	233	222	209	194	178
16 × 6 × 50	12 × ¾	113·5	32·70	212	201	189	175	159
16 × 6 × 50	12 × ⅝	103·3	29·70	191	181	168	155	140
16 × 6 × 50	12 × ½	93·1	26·70	170	159	148	134	120
						Ri	vet	s
15 × 6 × 45	12 × 2	210·5	61·24	409	394	376	356	333
15 × 6 × 45	12 × 1½	169·7	49·24	327	314	299	282	263
15 × 6 × 45	12 × 1	128·9	37·24	245	234	221	207	191
15 × 6 × 45	12 × ⅞	118·7	34·24	224	214	202	188	173
15 × 6 × 45	12 × ¾	108·5	31·24	203	193	182	169	154
15 × 6 × 45	12 × ⅝	98·3	28·24	182	173	162	149	135
15 × 6 × 45	12 × ½	88·1	25·24	161	152	141	129	116

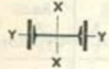
For notes relating to above see page xi.

Single Joist with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
20	22	24	28	32	36	Axis y—y	Axis x—x	Axis y—y	Axis x—x
313	286	260	212	173	143	100	452	3·09	8·49
246	224	202	164	133	109	76	356	2·99	8·17
178	161	144	115	93·0		52	262	2·83	7·80
161	144	129	102	82·8		46	238	2·77	7·69
143	128	114	90·5	72·5		40	215	2·70	7·58
125	111	98·8	77·8	62·2		34	192	2·61	7·46
107	94·5	83·0	64·9			28	169	2·50	7·33
⅞	dia.								
309	283	257	210	172	142	99	418	3·12	8·06
242	221	199	162	132	108	75	328	3·03	7·74
174	157	141	113	91·7		51	239	2·88	7·38
157	141	126	101	81·6		45	217	2·82	7·28
139	125	111	88·8	71·3		39	195	2·75	7·18
122	108	96·4	76·2	61·0		33	173	2·66	7·06
103	91·7	81·0	63 6			27	151	2 55	6·93

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons



Joist	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
				12	14	16	18	20
15×5×42	10 × 1	112·3	32·36	188	172	154	136	119
15×5×42	10 × 7/8	103·8	29·86	172	156	139	122	107
15×5×42	10 × 3/4	95·3	27·36	155	140	124	108	94·6
15×5×42	10 × 5/8	86·8	24·86	138	124	109	94·6	81·8
15×5×42	10 × 1/2	78·3	22·36	121	107	93·1	80·0	68·7
15×5×42	10 × 3/8	69·8	19·86	102	89·3	76·3	64·8	55·1
15×5×42	9 × 3/8	67·3	19·08	90·0	75·5	63·1	52·7	44·4
				Ri	vets	3/4	dia.	for
14×8×70	14 × 2	264·0	76·59	508	490	471	450	425
14×8×70	14 × 1½	216·4	62·59	412	397	380	362	341
14×8×70	14 × 1	168·8	48·59	316	303	289	273	255
14×8×70	14 × 7/8	156·9	45·09	292	280	266	250	233
14×8×70	14 × 3/4	145·0	41·59	268	256	242	227	210
14×8×70	14 × 5/8	133·1	38·09	243	232	218	204	188
14×8×70	14 × ½	121·2	34·59	218	207	194	180	164

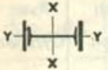
For notes relating to above see page xi.

Single Joist with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
22	24	26	30	34	38	Axis y—y	Axis x—x	Axis y—y	Axis x—x
104	91·6	80·4				36	201	2·35	7·27
93·4	81·6	71·5				31	183	2·30	7·16
82·1	71·5	62·6				27	165	2·24	7·05
70·7	61·4	53·6				23	147	2·16	6·92
59·0	51·1					19	129	2·06	6·78
47·2	40·8					15	111	1·93	6·63
						13	105	1·73	6·58
15 ×	5 an	d ⅞ d	ia. fo	r 14	× 8				
399	371	343	289	242	203	140	479	3·58	7·50
318	294	271	226	188	158	107	381	3·47	7·19
235	216	197	163	134	112	75	285	3·28	6·85
214	196	178	146	120	99·9	67	262	3·22	6·76
193	176	159	130	107	88·9	58	239	3·14	6·67
171	155	140	114	93·2	77·1	50	215	3·04	6·57
149	134	120	97·1	79·1		42	192	2·92	6·46

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads, in Tons

 Joist	Flange Plates	Weight per foot in lbs.	Area in sq.ins.	EFFECTIVE 12	14	16	18	20
14×6×57	12×2	222·5	64·78	414	395	372	348	320
14×6×57	$12 \times 1\frac{1}{2}$	181·7	52·78	334	317	298	276	253
14×6×57	12×1	140·9	40·78	254	239	222	204	185
14×6×57	$12 \times \frac{7}{8}$	130·7	37·78	233	219	203	185	167
14×6×57	$12 \times \frac{3}{4}$	120·5	34·78	213	199	183	166	149
14×6×57	$12 \times \frac{5}{8}$	110·3	31·78	192	178	163	147	131
14×6×57	$12 \times \frac{1}{2}$	100·1	28·78	170	157	142	127	112
							Riv	ets
14×6×46	12×2	211·7	61·60	396	378	358	335	310
14×6×46	$12 \times 1\frac{1}{2}$	170·9	49·60	316	301	284	264	243
14×6×46	12×1	130·1	37·60	236	223	209	192	175
14×6×46	$12 \times \frac{7}{8}$	120·0	34·60	216	204	190	174	158
14×6×46	$12 \times \frac{3}{4}$	109·9	31·60	195	184	170	156	141
14×6×46	$12 \times \frac{5}{8}$	99·5	28·60	175	164	151	137	123
14×6×46	$12 \times \frac{1}{2}$	89·4	25·60	154	143	131	117	104

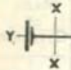
For notes relating to above see page xi.

Single Joist with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
22	24	26	28	32	36	Axis y—y	Axis x—x	Axis y—y	Axis x—x
292	265	239	216	176	144	101	402	3·05	7·48
230	207	186	167	135	111	77	318	2·95	7·16
166	148	132	118	95·1		53	236	2·78	6·80
149	133	118	105	84·9		47	215	2·72	6·70
133	118	104	93·2	74·6		41	195	2·65	6·60
116	102	90·6	80·4			35	175	2·56	6·48
98·6	86·7	76·3	67·5			29	155	2·44	6·36
¾	dia.								
284	258	234	211	172	142	99	392	3·11	7·57
222	201	181	163	132	109	76	307	3·02	7·26
158	142	127	114	92·3		52	224	2·87	6·91
142	127	113	101	82·1		46	204	2·81	6·81
126	112	100	89·4	71·8		40	183	2·74	6·71
109	97·3	86·4	76·9	61·6		34	163	2·65	6·60
92·7	81·8	72·3	64·2			28	143	2·54	6·48

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons

		Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
Joist	Flange Plates			12	14	16	18	20
13×5×35	10 × 1½	139·3	40·30	243	226	206	185	165
13×5×35	10 × 1	105·3	30·30	179	164	148	132	116
13×5×35	10 × ⅞	96·8	27·80	162	149	134	118	104
13×5×35	10 × ¾	88·3	25·30	146	133	119	105	91·9
13×5×35	10 × ⅝	79·8	22·80	129	117	104	91·2	79·3
13×5×35	10 × ½	71·3	20·30	112	101	88·7	77·0	66·5
13×5×35	10 × ⅜	62·8	17·80	95·3	84·0	72·8	62·3	53·2
				Ri	vets	¾	dia.	for
12×8×65	14 × 2	259·0	75·12	499	482	463	443	419
12×8×65	14 × 1½	211·4	61·12	403	389	373	355	335
12×8×65	14 × 1	163·8	47·12	307	295	282	266	249
12×8×65	14 × ⅞	151·9	43·62	283	272	259	244	228
12×8×65	14 × ¾	140·0	40·12	259	248	235	221	206
12×8×65	14 × ⅝	128·1	36·62	235	224	212	198	183
12×8×65	14 × ½	116·2	33·12	210	200	188	174	160

For notes relating to above see page xi.

Single Joist with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
22	24	26	30	34	38	Axis y—y	Axis x—x	Axis y—y	Axis x—x
146	128	114	90·1			52	233	2·54	6·80
102	89·8	79·1	62·1			35	169	2·42	6·46
91·3	80·0	70·3				31	153	2·37	6·37
80·2	70·0	61·4				27	137	2·32	6·27
68·8	60·1	52·6				23	121	2·25	6·16
57·4	49·9	43·6				19	106	2·15	6·04
45·8	39·6					15	90	2·03	5·90
13 × 5 and $\frac{7}{8}$ dia. for 12 × 8									
394	367	340	287	241	202	140	406	3·61	6·58
313	290	268	224	187	157	107	321	3·51	6·28
231	213	194	161	133	111	75	239	3·33	5·96
210	193	176	145	119	99·8	66	219	3·27	5·88
189	173	157	128	106	88·1	58	199	3·19	5·78
167	152	138	112	92·2	76·4	50	179	3·10	5·69
145	131	118	95·8	78·2		42	159	2·98	5·59

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons

Joist	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
				10	12	14	16	18
12×6×54	12×2	219·5	63·89	426	409	390	369	344
12×6×54	12×1½	178·7	51·89	343	329	313	294	274
12×6×54	12×1	137·9	39·89	261	249	235	219	201
12×6×54	12×⅞	127·7	36·89	240	229	215	200	183
12×6×54	12×¾	117·5	33·89	219	208	195	180	164
12×6×54	12×⅝	107·3	30·89	198	187	175	160	145
12×6×54	12×½	97·1	27·89	177	166	154	140	125
							Riv	ets
12×6×44	12×2	209·7	61·00	409	392	375	355	333
12×6×44	12×1½	168·9	49·00	326	313	298	281	262
12×6×44	12×1	128·1	37·00	243	233	221	207	191
12×6×44	12×⅞	117·9	34·00	223	213	201	188	173
12×6×44	12×¾	107·7	31·00	202	192	181	168	154
12×6×44	12×⅝	97·5	28·00	181	172	161	149	135
12×6×44	12×½	87·3	25·00	160	151	141	129	116

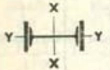
For notes relating to above see page xi.

Single Joist with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
20	22	24	28	32	36	Axis $y-y$	Axis $x-x$	Axis $y-y$	Axis $x-x$
318	291	264	215	175	144	101	343	3·08	6·55
251	228	206	166	135	111	77	270	2·98	6·24
183	164	147	117	94·9		53	199	2·82	5·91
165	148	132	105	84·7		47	181	2·76	5·81
147	131	117	92·8	74·4		41	164	2·68	5·72
129	115	101	80·2	64·0		35	147	2·60	5·62
111	97·9	86·1	67·3			29	130	2·49	5·50
¾	dia.								
308	283	257	211	172	142	100	336	3·13	6·63
241	221	200	162	132	109	76	262	3·04	6·33
174	157	141	114	92·2	75·5	52	190	2·90	6·00
157	141	127	101	82·1		46	173	2·84	5·91
139	125	112	89·3	71·9		40	155	2·77	5·82
122	109	97·0	76·7	61·5		34	138	2·69	5·72
104	92·2	81·6	64·1	51·2		28	121	2·58	5·61

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS— Safe Concentric Loads in Tons

		Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
Joist	Flange Plates			10	12	14	16	18
12×5×32	10 × 1½	136·4	39·44	252	238	222	203	183
12×5×32	10 × 1	102·4	29·44	186	174	161	146	130
12×5×32	10 × ⅞	93·9	26·94	169	158	145	131	116
12×5×32	10 × ¾	85·4	24·44	152	142	130	116	103
12×5×32	10 × ⅝	76·9	21·94	135	126	114	101	89·4
12×5×32	10 × ½	68·4	19·44	118	109	98·2	86·5	75·2
12×5×32	10 × ⅜	59·9	16·94	101	91·8	81·3	70·6	60·6
				Ri	vets	¾	dia.	for
10×8×55	14 × 2	249·0	72·18	494	480	465	447	428
10×8×55	14 × 1½	201·4	58·18	396	385	372	357	341
10×8×55	14 × 1	153·8	44·18	300	290	279	267	253
10×8×55	14 × ⅞	141·9	40·68	275	265	255	244	230
10×8×55	14 × ¾	130·0	37·18	250	241	232	221	208
10×8×55	14 × ⅝	118·1	33·68	226	217	208	197	185
10×8×55	14 × ½	106·2	30·18	201	193	184	174	162

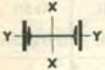
For notes relating to above see page xi.

Single Joist with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
20	22	24	28	32	36	Axis $y-y$	Axis $x-x$	Axis $y-y$	Axis $x-x$
163	144	127	100	80·1		52	212	2·57	6·36
115	101	88·8	69·3			35	152	2·45	6·02
102	90·0	79·0	61·4			31	138	2·40	5·93
90·3	78·9	69·1	53·6			27	123	2·35	5·83
77·8	67·8	59·1	45·8			23	109	2·28	5·73
65·1	56·4	49·0				19	94	2·19	5·61
52·0	44·8	38·8				14	80	2·06	5·48
12 ×	5 an	d ⅞	dia.	for 1	0 × 8				
406	383	357	306	258	217	138	332	3·66	5·67
322	302	281	238	200	167	106	259	3·57	5·38
237	221	204	171	142	118	73	190	3·40	5·08
216	200	184	153	127	105	65	173	3·34	4·99
194	179	164	136	112	93·3	57	156	3·27	4·91
172	158	144	119	97·6	80·6	49	139	3·18	4·82
150	136	124	101	82·5	67·9	40	123	3·06	4·73

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons



Joist	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE 12	14	16	18	20
10×6×40	12×2	205·5	59·77	385	369	350	328	304
10×6×40	$12\times1\frac{1}{2}$	164·7	47·77	306	292	276	258	238
10×6×40	12×1	123·9	35·77	226	215	202	187	171
10×6×40	$12\times\frac{7}{8}$	113·7	32·77	206	195	183	169	154
10×6×40	$12\times\frac{3}{4}$	103·5	29·77	186	176	164	151	137
10×6×40	$12\times\frac{5}{8}$	93·3	26·77	166	156	144	132	119
10×6×40	$12\times\frac{1}{2}$	83·1	23·77	145	136	125	113	101
							Riv	ets
10×5×30	$10\times1\frac{1}{2}$	134·3	38·85	235	219	201	182	162
10×5×30	10×1	100·3	28·85	172	159	144	129	114
10×5×30	$10\times\frac{7}{8}$	91·8	26·35	155	143	129	115	101
10×5×30	$10\times\frac{3}{4}$	83·3	23·85	139	128	115	102	89·6
10×5×30	$10\times\frac{5}{8}$	74·8	21·35	123	112	100	88·4	77·2
10×5×30	$10\times\frac{1}{2}$	66·3	18·85	106	96·4	85·3	74·4	64·6
10×5×30	$10\times\frac{3}{8}$	57·8	16·35	89·6	79·8	69·6	60·0	51·6

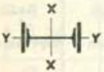
For notes relating to above see page xi.

Single Joist with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
22	24	26	30	32	36	Axis y—y	Axis x—x	Axis y—y	Axis x—x
280	255	231	189	171	141	100	278	3·16	5·72
218	198	179	145	131	108	76	216	3·08	5·42
155	140	125	101	91·6	75·1	52	156	2·94	5·11
139	125	112	90·4	81·4	66·6	46	141	2·89	5·02
123	110	98·8	79·2	71·2		40	126	2·83	4·94
107	95·6	85·2	68·0	61·0		34	112	2·75	4·85
90·5	80·3	71·2	56·6	50·7		28	97	2·64	4·75
¾	dia.								
143	127	112	89·3	80·0		52	176	2·59	5·43
100	88·4	78·0	61·3			35	125	2·47	5·11
89·5	78·6	69·2	54·3			31	113	2·43	5·02
78·4	68·7	60·4				27	101	2·38	4·93
67·5	58·8	51·6				23	89	2·31	4·84
56·0	48·8	42·7				19	77	2·22	4·73
44·5	38·6	33·7				14	65	2·00	4·61

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS— Safe Concentric Loads in Tons



Joist	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
				10	12	14	16	18
10×4½×25	9×1	88·4	25·35	156	144	130	116	101
10×4½×25	9×⅞	80·8	23·10	141	130	117	103	90·6
10×4½×25	9×¾	73·1	20·85	126	116	104	91·7	79·6
10×4½×25	9×⅝	65·5	18·60	111	102	90·9	79·3	68·4
10×4½×25	9×½	57·8	16·35	96·8	87·6	77·2	66·7	57·1
10×4½×25	9×⅜	50·2	14·10	81·4	72·6	63·0	53·7	45·6
10×4½×25	8×⅜	47·8	13·35	71·7	61·6	51·5	42·7	35·6
					Ri	vets	¾ dia	. for
9×7 ×50	12×2	216·8	62·71	419	404	386	365	342
9×7 ×50	12×1½	176·0	50·71	337	324	309	291	272
9×7 ×50	12×1	135·2	38·71	255	244	231	217	200
9×7 ×50	12×⅞	125·0	35·71	234	224	212	198	182
9×7 ×50	12×¾	114·8	32·71	214	204	192	179	164
9×7 ×50	12×⅝	104·6	29·71	193	183	172	159	146
9×7 ×50	12×½	94·4	26·71	172	163	152	140	126

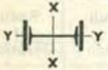
For notes relating to above see page xi.

Single Joist with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
20	22	24	28	32	36	Axis $y-y$	Axis $x-x$	Axis $y-y$	Axis $x-x$
88·2	76·6	66·8	51·5			28	111	2·25	5·13
78·5	68·0	59·3				25	100	2·21	5·06
68·7	59·4	51·6				22	89	2·16	4·96
58·9	50·8	44·0				18	78	2·11	4·86
48·9	42·0	36·3				15	67	2·03	4·76
38·8	33·1	28·5				12	57	1·92	4·64
29·9						10	53	1·70	4·60
10 × 4½ and ⅞ dia. for 9 × 7									
317	291	265	217	177	146	103	258	3·13	5·18
250	229	207	169	137	113	79	201	3·05	4·88
183	166	149	120	97·5	79·8	55	147	2·91	4·57
166	150	134	108	87·3		49	134	2·86	4·49
149	134	119	95·6	77·0		43	121	2·80	4·41
131	117	104	83·2	66·7		37	108	2·72	4·33
113	101	89·5	70·6	56·6		31	96	2·63	4·24

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons



Joist	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
				10	12	14	16	18
8 × 8 × 38	14 × 2	232·0	67·13	462	450	435	420	403
8 × 8 × 38	14 × 1½	184·4	53·13	364	355	344	330	316
8 × 8 × 38	14 × 1	136·8	39·13	267	259	250	240	229
8 × 8 × 38	14 × ⅞	124·9	35·63	242	235	228	218	208
8 × 8 × 38	14 × ¾	113·0	32·13	218	212	204	195	186
8 × 8 × 38	14 × ⅝	101·1	28·63	194	187	180	172	163
8 × 8 × 38	14 × ½	89·2	25·13	170	164	157	149	141
8 × 8 × 38	14 × ⅜	77·3	21·63	145	139	133	126	118
					Riv	ets	⅞ dia	. for
8 × 6 × 35	12 × 1½	159·7	46·30	309	298	284	269	252
8 × 6 × 35	12 × 1	118·9	34·30	227	218	207	195	181
8 × 6 × 35	12 × ⅞	108·7	31·30	207	198	188	176	163
8 × 6 × 35	12 × ¾	98·5	28·30	186	178	168	157	145
8 × 6 × 35	12 × ⅝	88·3	25·30	165	158	149	138	127
8 × 6 × 35	12 × ½	78·1	22·30	144	137	129	119	109

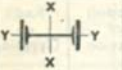
For notes relating to above see page xi.

Single Joist with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
20	22	24	28	32	36	Axis y—y	Axis x—x	Axis y—y	Axis x—x
383	362	340	292	248	211	136	259	3·76	4·80
302	283	264	228	192	165	104	198	3·70	4·52
216	203	188	160	134	114	71	140	3·56	4·23
196	182	169	143	119	101	63	126	3·51	4·15
175	163	149	127	105	88·4	55	112	3·46	4·07
153	142	131	108	89·0	75·8	47	99	3·36	4·00
132	122	111	92·0	76·5	63·0	38	85	3·28	3·91
109	100	91·1	74·4	61·0	50·2	30	72	3·12	3·82
8×8	and	¾ dia	. for	8×6					
233	214	195	159	130	107	75	170	3·12	4·49
166	151	137	111	90·4	74·2	51	121	2·99	4·19
150	136	122	99·0	80·2	65·8	45	109	2·95	4·11
133	120	108	86·7	70·1	57·4	39	97	2·88	4·03
115	104	93·2	74·4	59·9		33	85	2·81	3·95
98·2	87·7	78·1	61·9	49·7		27	74	2·71	3·86

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons



Joist	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
				8	10	12	14	16
8 × 5 × 28	$10 \times 1\frac{1}{2}$	132·4	38·28	258	246	233	217	199
8 × 5 × 28	10×1	98·4	28·28	189	180	169	156	142
8 × 5 × 28	$10 \times \frac{7}{8}$	89·9	25·78	172	163	153	141	128
8 × 5 × 28	$10 \times \frac{3}{4}$	81·4	23·28	154	146	137	126	113
8 × 5 × 28	$10 \times \frac{5}{8}$	72·9	20·78	137	129	121	110	99·3
8 × 5 × 28	$10 \times \frac{1}{2}$	64·4	18·28	120	112	104	94·7	84·2
8 × 5 × 28	$10 \times \frac{3}{8}$	55·9	15·78	103	95·8	87·6	78·4	68·7
							Riv	ets
6 × 5 × 25	$10 \times 1\frac{1}{2}$	129·3	37·37	252	241	228	213	197
6 × 5 × 25	10×1	95·3	27·37	183	175	165	153	140
6 × 5 × 25	$10 \times \frac{7}{8}$	86·8	24·87	166	159	149	138	125
6 × 5 × 25	$10 \times \frac{3}{4}$	78·3	22·37	149	141	133	122	110
6 × 5 × 25	$10 \times \frac{5}{8}$	69·8	19·87	132	125	117	107	96·9
6 × 5 × 25	$10 \times \frac{1}{2}$	61·3	17·37	114	108	100	91·2	81·4
6 × 5 × 25	$10 \times \frac{3}{8}$	52·8	14·87	97·0	90·7	83·5	75·3	66·4

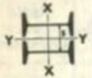
For notes relating to above see page xi.

Single Joist with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
18	20	22	24	28	30	Axis y—y	Axis x—x	Axis y—y	Axis x—x
180	161	143	126	100	89·2	52	140	2·61	4·49
128	113	100	88·1	68·9	61·2	35	99	2·50	4·19
114	101	89·1	78·4	61·2	54·4	31	89	2·46	4·11
101	89·1	78·1	68·6	53·4	47·4	27	79	2·41	4·03
87·6	76·8	67·1	58·6	45·5		23	70	2·35	3·94
73·8	64·2	55·8	48·7	37·6		19	60	2·26	3·86
59·6	51·4	44·4	38·6			14	51	2·15	3·74
¾	dia.								
178	159	142	126	98·6	88·1	52	105	2·63	3·55
125	112	98·5	86·6	68·0	60·4	35	73	2·53	3·25
112	99·7	88·4	77·9	60·5	53·9	31	65	2·50	3·18
98·6	87·0	76·8	67·3	52·0	46·6	27	57	2·44	3·10
86·1	75·8	66·4	58·1	45·2	40·1	23	50	2·40	3·02
71·5	62·4	54·4	47·4	36·9		18	43	2·30	2·93
57·5	50·0	43·2	37·6			14	36	2·19	2·84

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons

 Two Joists	Web Centres S in ins.	Area in sq. ins.	EFFECTIVE				
			12	14	16	18	20
24 × 7½ × 95	19·0	55·88	407	403	400	397	393
22 × 7 × 75	17·0	44·12	319	316	314	311	307
20 × 7½ × 89	15·5	52·38	377	374	370	366	362
20 × 6½ × 65	16·0	38·24	276	273	270	267	264
18 × 8 × 80	14·5	47·06	338	334	330	326	322
18 × 7 × 75	14·0	44·18	316	313	309	305	301
18 × 6 × 55	14·0	32·36	231	229	226	223	220
16 × 8 × 75	13·0	44·13	314	310	306	302	298
16 × 6 × 62	12·0	36·42	258	254	250	246	242
16 × 6 × 50	12·5	29·42	209	206	203	200	197
15 × 6 × 45	12·0	26·48	187	185	182	179	176
15 × 5 × 42	11·0	24·72	173	171	168	165	161
14 × 8 × 70	11·0	41·18	290	286	281	276	271
14 × 6 × 57	11·0	33·58	236	232	228	224	219
14 × 6 × 46	11·0	27·18	191	188	185	181	178

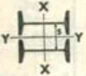
For notes relating to above see page xi.

Two Joists Battened Together and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
24	28	32	36	40	44	Axis $y—y$	Axis $x—x$	Axis $y—y$	Axis $x—x$
386	378	370	361	351	340	390	**422**	9·62	**9·52**
301	294	286	277	267	257	**272**	305	**8·61**	8·72
353	343	332	320	306	291	**284**	335	**7·90**	7·99
258	251	243	235	225	214	223	**245**	8·11	**8·01**
313	304	293	280	266	252	232	**287**	7·45	**7·41**
292	283	271	259	245	230	**215**	256	**7·15**	7·22
214	206	198	189	178	168	**163**	187	**7·10**	7·21
288	276	264	249	233	216	190	**243**	6·73	**6·64**
233	222	210	195	180	164	**152**	181	**6·12**	6·31
190	182	172	162	150	139	**129**	154	**6·37**	6·48
169	161	152	142	131	120	110	**131**	6·12	**6·10**
154	145	135	124	112	101	**96**	114	**5·59**	5·89
259	245	230	212	194	176	**145**	202	**5·79**	5·85
209	198	184	169	154	139	126	**152**	5·65	**5·64**
170	160	149	137	124	113	**102**	126	**5·64**	5·71

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons

 Two Joists	Web Centres S in ins.	Area in sq. ins.	EFFECTIVE				
			12	14	16	18	20
13 × 5 × 35	10·5	20·60	144	141	138	136	132
12 × 8 × 65	9·5	38·24	266	261	256	250	244
12 × 6 × 54	9·0	31·78	219	214	210	204	198
12 × 6 × 44	9·0	26·00	179	175	171	167	162
12 × 5 × 32	9·0	18·90	130	127	124	121	117
10 × 8 × 55	8·5	32·36	220	214	208	202	194
10 × 6 × 40	8·0	23·54	160	156	151	146	141
10 × 5 × 30	7·0	17·70	117	114	109	104	99·5
10 × 4½ × 25	8·0	14·70	99·6	97·0	94·1	90·9	87·3
9 × 7 × 50	7·5	29·42	196	190	184	176	168
9 × 4 × 21	7·0	12·36	81·9	79·3	76·2	72·7	68·8
8 × 8 × 38	8·5	22·25	146	141	135	128	120
8 × 6 × 35	6·5	20·60	134	129	123	116	109
8 × 5 × 28	6·0	16·56	107	102	97·5	91·6	85·2
8 × 4 × 18	6·5	10·60	68·8	65·9	62·7	59·0	55·0

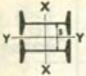
For notes relating to above see page xi.

Two Joists Battened Together and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
22	24	28	32	36	40	Axis $y—y$	Axis $x—x$	Axis $y—y$	Axis $x—x$
129	125	117	108	97·8	87·6	76	87	5·35	5·25
237	230	213	194	174	155	113	163	5·10	5·05
192	185	169	151	134	117	93	125	4·69	4·86
157	151	138	123	109	95·9	76	106	4·69	4·94
113	109	99·5	88·7	78·1	68·2	57	74	4·61	4·84
186	178	158	138	119	102	84	115	4·63	4·22
135	128	114	99·2	85·5	73·4	60	82	4·22	4·17
93·6	87·4	74·7	63·0	53·0	44·8	39	58	3·65	4·06
83·3	79·0	69·7	60·4	51·8	44·4	40	49	4·11	4·08
158	148	128	109	92·0	78·0	68	92	4·10	3·76
64·6	60·2	51·2	43·0	36·2	30·4	29	36	3·59	3·62
112	104	86·7	72·3	60·5	50·6	58	66	4·65	3·43
101	93·4	77·8	64·3	53·5	44·8	41	57	3·53	3·34
78·5	71·7	59·0	48·4	40·0		31	45	3·20	3 29
50·8	46·5	38·4	31·6	26·2	21·9	23	28	3·35	3·24

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons



Two Joists	Web Centres S in ins.	Area in sq. ins.	EFFECTIVE				
			10	12	14	16	18
7 × 4 × 16	5·5	9·50	62·6	59·7	56·6	52·9	48·8
7 × 3½ × 15	5·5	8·84	58·0	55·6	52·4	48·7	45·0
6 × 5 × 25	5·5	14·74	93·3	87·4	80·5	72·9	65·0
6 × 4½ × 20	4·5	11·78	74·3	69·7	64·2	58·1	51·7
6 × 3 × 12	4·5	7·06	44·1	41·1	37·6	33·7	29·6
5 × 4½ × 20	5·0	11·76	70·1	63·6	56·5	48·7	41·9
5 × 4½ × 18	5·5	10·58	63·3	57·4	50·9	44·0	37·9
5 × 3 × 11	4·0	6·52	38·8	35·2	31·1	27·0	23·1
5 × 2½ × 9	3·5	5·30	29·9	26·2	22·4	18·9	15·9
4¾ × 1¾ × 6·5	3·5	3·82	21·2	18·5	15·8	13·3	11·0
4 × 3 × 10	3·0	5·88	30·7	25·9	21·5	17·7	14·7
4 × 3 × 9·5	3·5	5·58	29·2	24·9	20·6	17·0	14·1
4 × 1¾ × 5	3·0	2·94	14·7	12·2	9·9	8·1	6·7
3 × 3 × 8·5	3·0	5·04	19·8	15·4	12·0		
3 × 1½ × 4	2·5	2·35	8·8	6·8	5·3		

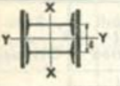
For notes relating to above see page xi.

Two Joists Battened Together and Dirnensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
20	22	24	26	30	34	Axis y—y	Axis x—x	Axis y—y	Axis x—x
44·5	40·2	36·1	32·3	26·0	21·2	16·5	22·6	2·88	2·89
40·8	36·9	33·0	29·5	23·7	19·2	15·9	20·5	2·84	2·85
57·2	50·3	44·3	39·0	30·9		24·7	29·1	2·97	2·44
45·5	40·0	35·1	30·9	24·3		15·7	23·1	2·45	2·43
26·1	22·7	19·8	17·5			10·3	14·0	2·34	2·44
36·1	31·0	26·8				18·3	20·0	2·71	2·06
32·6	28·2	24·4				18·3	18·1	2·93	2·07
19·9	17·0	14·9				8·3	10·9	2·11	2·05
13·5	11·5					5·9	8·7	1·83	2·03
9·4	8·0					4·7	5·7	1·79	1·88
12·3						5·3	7·8	1·64	1·63
11·8						6·0	7·5	1·87	1·64
						2·9	3·7	1·54	1·58
						4·6	5·1	1·66	1·23
						2·0	2·2	1·29	1·19

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons



Two Joists	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	Web Centres S in ins.	EFFECTIVE			
					16	18	20	22
16 × 6 × 62	16 × 2	343·7	100·42	8·0	656	638	618	595
16 × 6 × 62	$16 \times 1\frac{1}{2}$	289·3	84·42	8·0	551	535	518	499
16 × 6 × 62	16 × 1	234·9	68·42	8·0	446	432	418	402
16 × 6 × 62	$16 \times \frac{7}{8}$	221·3	64·42	8·0	419	407	393	378
16 × 6 × 62	$16 \times \frac{3}{4}$	207·7	60·42	8·0	393	380	368	354
16 × 6 × 62	$16 \times \frac{5}{8}$	194·1	56·42	8·0	366	355	343	330
16 × 6 × 62	$16 \times \frac{1}{2}$	180·5	52·42	8·0	340	330	318	306
							Riv	ets
16 × 6 × 50	16 × 2	319·9	93·42	8·0	610	594	575	555
16 × 6 × 50	$16 \times 1\frac{1}{2}$	265·5	77·42	8·0	506	492	476	458
16 × 6 × 50	16 × 1	211·1	61·42	8·0	400	389	376	362
16 × 6 × 50	$16 \times \frac{7}{8}$	197·5	57·42	8·0	374	363	351	338
16 × 6 × 50	$16 \times \frac{3}{4}$	183·9	53·42	8·0	348	337	326	314
16 × 6 × 50	$16 \times \frac{5}{8}$	170·3	49·42	8·0	320	312	300	289
16 × 6 × 50	$16 \times \frac{1}{2}$	156·7	45·42	8·0	295	286	276	265

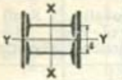
For notes relating to above see page xi.

Double Joists with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
24	26	28	32	36	40	Axis y—y	Axis x—x	Axis y—y	Axis x—x
570	543	515	456	399	346	250	665	4·47	8·14
478	455	431	381	332	288	208	540	4·44	7·80
385	366	346	305	266	230	165	418	4·39	7·42
362	344	325	287	249	215	154	388	4·38	7·31
338	322	304	267	232	201	144	358	4·36	7·20
315	299	283	249	216	187	133	328	4·34	7·09
292	277	261	230	199	172	122	299	4·32	6·96
¾	dia.								
532	507	481	426	373	324	235	644	4·49	8·30
439	418	397	351	307	266	192	518	4·46	7·97
346	330	312	276	240	208	150	394	4·42	7·60
323	307	291	257	224	194	139	364	4·40	7·50
300	285	270	238	207	179	128	334	4·39	7·38
277	263	249	219	190	165	118	304	4·37	7·28
254	241	227	200	174	150	107	274	4·34	7·16

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons



Two Joists	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	Web Centres S in ins.	EFFECTIVE 14	16	18	20
15 × 6 × 45	16 × 2	309·9	90·48	8·0	607	592	576	557
15 × 6 × 45	16 × 1½	255·5	74·48	8·0	499	487	473	458
15 × 6 × 45	16 × 1	201·1	58·48	8·0	391	381	370	358
15 × 6 × 45	16 × ⅞	187·5	54·48	8·0	364	355	345	332
15 × 6 × 45	16 × ¾	173·9	50·48	8·0	337	329	319	308
15 × 6 × 45	16 × ⅝	160·3	46·48	8·0	310	302	293	284
15 × 6 × 45	16 × ½	146·7	42·48	8·0	280	276	268	259
							Riv	ets
15 × 5 × 42	14 × 1½	229·1	66·72	7·0	436	421	405	388
15 × 5 × 42	14 × 1	181·5	52·72	7·0	343	332	319	305
15 × 5 × 42	14 × ⅞	169·6	49·22	7·0	320	309	297	284
15 × 5 × 42	14 × ¾	157·7	45·72	7·0	297	287	276	263
15 × 5 × 42	14 × ⅝	145·8	42·22	7·0	274	265	254	242
15 × 5 × 42	14 × ½	133·9	38·72	7·0	250	242	233	222
15 × 5 × 42	14 × ⅜	122·0	35·22	7·0	228	220	211	201

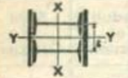
For notes relating to above see page xi.

Double Joists with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
22	24	28	32	36	40	Axis $y-y$	Axis $x-x$	Axis $y-y$	Axis $x-x$
538	515	467	414	362	315	229	592	4·50	7·89
441	423	382	339	296	257	186	473	4·47	7·56
345	330	298	263	229	199	143	357	4·43	7·20
320	307	276	244	213	184	133	328	4·41	7·10
297	284	256	225	196	170	122	300	4·40	7·00
273	261	234	206	180	155	111	271	4·38	6·89
248	237	213	188	163	141	101	243	4·35	6·77
¾	dia.								
368	347	302	258	220	187	145	414	3·90	7·47
289	272	236	201	171	145	112	312	3·86	7·09
269	253	220	187	159	135	104	287	3·84	6·99
249	234	203	173	146	124	96	262	3·83	6·87
229	216	186	159	134	114	87	237	3·81	6·75
210	197	170	144	121	103	79	212	3·79	6·62
190	178	153	130	110	93·4	71	188	3·76	6·48

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons

		Weight per foot in lbs.	Area in sq. ins.	Web Centres S in ins.	EFFECTIVE			
Two Joists	Flange Plates				14	16	18	20
14 × 8 × 70	20 × 2	415·6	121·18	10·0	839	825	810	793
14 × 8 × 70	20 × 1½	347·6	101·18	10·0	700	688	675	662
14 × 8 × 70	20 × 1	279·6	81·18	10·0	561	551	541	530
14 × 8 × 70	20 × ⅞	262·6	76·18	10·0	526	517	507	497
14 × 8 × 70	20 × ¾	245·6	71·18	10·0	490	483	474	464
14 × 8 × 70	20 × ⅝	228·6	66·18	10·0	457	449	440	431
14 × 8 × 70	20 × ½	211·6	61·18	10·0	422	414	407	398
					Ri	vet	s ⅞	dia.
14 × 6 × 57	16 × 2	334·0	97·56	8·0	654	638	620	600
14 × 6 × 57	16 × 1½	279·6	81·56	8·0	547	533	518	501
14 × 6 × 57	16 × 1	225·2	65·56	8·0	438	427	415	401
14 × 6 × 57	16 × ⅞	211·6	61·56	8·0	411	401	389	376
14 × 6 × 57	16 × ¾	198·0	57·56	8·0	385	375	363	351
14 × 6 × 57	16 × ⅝	184·4	53·56	8·0	357	348	338	326
14 × 6 × 57	16 × ½	170·8	49·56	8·0	330	322	312	302

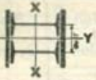
For notes relating to above see page xi

Double Joists with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
22	24	28	32	36	40	Axis $y-y$	Axis $x-x$	Axis $y-y$	Axis $x-x$
776	757	714	665	612	555	383	729	5·62	7·36
647	631	595	554	508	460	316	591	5·59	7·06
518	505	475	442	405	366	250	458	5·55	6·72
486	473	445	414	379	343	233	425	5·53	6·63
452	442	416	386	352	319	216	393	5·51	6·54
421	410	386	358	327	295	200	360	5·49	6·44
389	378	356	330	301	272	183	328	5·47	6·34
for	14×	8 an	d ¾ d	ia. f	or 1	4×6			
579	555	502	445	389	338	245	576	4·48	7·29
482	462	417	370	323	280	202	466	4·45	6·97
386	369	333	294	256	222	159	359	4·41	6·62
362	346	312	275	240	207	149	332	4·40	6·52
338	323	291	256	223	193	138	306	4·38	6·42
314	300	269	237	206	178	127	280	4·36	6·32
290	277	248	218	190	164	117	254	4·34	6·20

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons



Two Joists	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	Web Centres S in ins.	EFFECTIVE			
					14	16	18	20
14 × 6 × 46	16 × 2	312·3	91·18	8·0	612	597	580	562
14 × 6 × 46	16 × 1½	257·9	75·18	8·0	504	491	478	462
14 × 6 × 46	16 × 1	203·5	59·18	8·0	396	386	375	363
14 × 6 × 46	16 × ⅞	189·9	55·18	8·0	369	360	349	338
14 × 6 × 46	16 × ¾	176·3	51·18	8·0	342	333	324	313
14 × 6 × 46	16 × ⅝	162·7	47·18	8·0	315	307	298	288
14 × 6 × 46	16 × ½	149·1	43·18	8·0	288	280	271	263
							Riv	ets
13 × 5 × 35	14 × 1½	215·1	62·60	7·0	409	396	381	365
13 × 5 × 35	14 × 1	167·5	48·60	7·0	317	306	295	282
13 × 5 × 35	14 × ⅞	155·6	45·10	7·0	294	284	273	261
13 × 5 × 35	14 × ¾	143·7	41·60	7·0	271	261	250	240
13 × 5 × 35	14 × ⅝	131·8	38·10	7·0	248	239	230	219
13 × 5 × 35	14 × ½	119·9	34·60	7·0	225	217	208	199
13 × 5 × 35	14 × ⅜	108·0	31·10	7·0	202	195	187	178

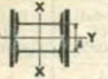
For notes relating to above see page xi

Double Joists with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
22	24	28	32	36	40	Axis $y-y$	Axis $x-x$	Axis $y-y$	Axis $x-x$
542	519	470	417	365	317	230	556	4·50	7·41
445	427	386	342	298	259	188	444	4·47	7·09
349	334	301	266	232	201	145	336	4·43	6·74
325	311	280	247	216	187	134	309	4·41	6·64
301	288	259	229	199	172	124	283	4·40	6·54
277	265	238	210	182	158	113	256	4·38	6·44
253	242	216	191	166	142	102	230	4·36	6·32
¾	dia.								
346	327	285	244	208	177	137	348	3·92	6·67
267	252	219	187	159	135	104	259	3·88	6·32
247	233	202	173	147	125	96	237	3·87	6·23
228	214	186	159	133	114	88	215	3·85	6·12
208	195	169	144	122	104	80	194	3·83	6·02
188	177	153	130	110	93·8	72	172	3·81	5·90
168	158	136	116	98·3	83·4	64	151	3·78	5·77

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons



Two Joists	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	Web Centres S in ins.	EFFECTIVE 14	16	18	20
12 × 8 × 65	20 × 2	405·6	118·24	10·0	819	805	790	775
12 × 8 × 65	20 × $1\frac{1}{2}$	337·6	98·24	10·0	680	668	656	643
12 × 8 × 65	20 × 1	269·6	78·24	10·0	541	532	522	511
12 × 8 × 65	20 × $\frac{7}{8}$	252·6	73·24	10·0	506	497	488	478
12 × 8 × 65	20 × $\frac{3}{4}$	235·6	68·24	10·0	471	463	454	445
12 × 8 × 65	20 × $\frac{5}{8}$	218·6	63·24	10·0	436	429	420	412
12 × 8 × 65	20 × $\frac{1}{2}$	201·6	58·24	10·0	402	395	387	379
					Rivets $\frac{7}{8}$ dia.			
12 × 6 × 54	14 × 2	300·7	87·77	7·0	575	556	536	513
12 × 6 × 54	14 × $1\frac{1}{2}$	253·1	73·77	7·0	482	467	449	430
12 × 6 × 54	14 × 1	205·5	59·77	7·0	390	377	363	347
12 × 6 × 54	14 × $\frac{7}{8}$	193·6	56·27	7·0	367	355	340	326
12 × 6 × 54	14 × $\frac{3}{4}$	181·7	52·77	7·0	344	332	320	305
12 × 6 × 54	14 × $\frac{5}{8}$	169·8	49·27	7·0	321	310	298	285
12 × 6 × 54	14 × $\frac{1}{2}$	157·9	45·77	7·0	298	288	276	264

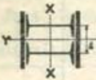
For notes relating to above see page xi

Double Joists with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
22	24	28	32	36	40	Axis y—y	Axis x—x	Axis y—y	Axis x—x
757	739	698	650	597	543	375	615	5·63	6·45
628	613	578	538	494	449	309	496	5·61	6·15
499	487	459	426	391	354	242	381	5·56	5·84
467	455	429	399	365	331	225	353	5·55	5·76
435	424	399	371	339	307	209	325	5·53	5·67
403	392	369	342	313	283	192	298	5·51	5·58
370	361	339	315	288	260	175	270	5·49	5·49
for	12 ×	8 an	d ¾ d	ia. f	or 1	2 × 6			
487	460	402	345	294	250	194	439	3·94	6·33
408	385	336	288	245	209	162	356	3·92	6·02
329	310	270	231	196	167	129	277	3·89	5·69
308	291	253	216	184	156	121	257	3·88	5·61
290	273	237	202	172	146	113	238	3·87	5·52
270	254	220	188	160	136	104	219	3·85	5·42
250	235	204	174	147	125	96	200	3·84	5·33

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons



Two Joists	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	Web Centres S in ins.	EFFECTIVE			
					14	16	18	20
12 × 6 × 44	14 × 1½	233·5	68·00	7·0	445	430	414	397
12 × 6 × 44	14 × 1	185·9	54·00	7·0	353	341	328	314
12 × 6 × 44	14 × ⅞	174·9	50·50	7·0	330	319	306	293
12 × 6 × 44	14 × ¾	162·1	47·00	7·0	306	296	285	272
12 × 6 × 44	14 × ⅝	150·2	43·50	7·0	283	274	263	251
12 × 6 × 44	14 × ½	138·3	40·00	7·0	260	252	242	231
12 × 6 × 44	14 × ⅜	126·0	36·50	7·0	237	229	220	210
						Ri	vet	s
12 × 5 × 32	12 × 1½	188·9	54·91	6·0	345	330	312	293
12 × 5 × 32	12 × 1	148·1	42·91	6·0	269	257	243	227
12 × 5 × 32	12 × ⅞	137·9	39·91	6·0	250	238	225	211
12 × 5 × 32	12 × ¾	127·7	36·91	6·0	231	220	208	195
12 × 5 × 32	12 × ⅝	117·5	33·91	6·0	212	202	191	178
12 × 5 × 32	12 × ½	107·3	30·91	6·0	193	184	173	162
12 × 5 × 32	12 × ⅜	97·1	27·91	6·0	174	165	156	145

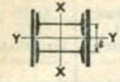
For notes relating to above see page xi

Double Joists with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
22	24	28	32	36	40	Axis y—y	Axis x—x	Axis y—y	Axis x—x
377	356	310	266	227	193	**150**	341	**3·93**	6·13
298	281	245	209	178	151	**117**	260	**3·90**	5·80
278	262	228	195	166	141	**109**	240	**3·89**	5·72
258	243	211	181	153	129	**101**	220	**3·88**	5·63
238	224	195	167	141	120	**93**	201	**3·86**	5·53
219	206	178	152	129	110	**84**	182	**3·85**	5·43
199	187	162	138	117	100	**76**	162	**3·83**	5·33
¾	dia.								
272	250	209	173	144	121	**104**	279	**3·36**	6·17
211	194	161	133	111	93·2	**80**	208	**3·34**	5·83
195	180	150	124	102	86·3	**74**	191	**3·33**	5·74
180	166	138	114	94·7	79·3	**68**	174	**3·31**	5·64
165	151	126	104	86·3	72·3	**62**	157	**3·30**	5·54
150	137	114	94·2	78·1	65·4	**56**	140	**3·28**	5·43
134	123	102	84·2	69·8	58·4	**50**	123	**3·27**	5·32

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons



Two Joists	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	Web Centres S in ins.	EFFECTIVE 14	16	18	20
10 × 8 × 55	18 × 2	358·4	104·36	9·0	714	699	684	668
10 × 8 × 55	18 × 1½	297·2	86·36	9·0	590	578	566	552
10 × 8 × 55	18 × 1	236·0	68·36	9·0	465	456	445	434
10 × 8 × 55	18 × ⅞	220·7	63·86	9·0	434	424	414	403
10 × 8 × 55	18 × ¾	205·4	59·36	9·0	402	393	384	373
10 × 8 × 55	18 × ⅝	190·1	54·86	9·0	371	362	353	343
10 × 8 × 55	18 × ½	174·8	50·36	9·0	339	331	322	313
					Riv	ets	⅞ d	ia.
10 × 6 × 40	14 × 1½	225·1	65·54	7·0	429	415	400	383
10 × 6 × 40	14 × 1	177·5	51·54	7·0	337	326	314	300
10 × 6 × 40	14 × ⅞	165·6	48·04	7·0	314	303	292	279
10 × 6 × 40	14 × ¾	153·7	44·54	7·0	291	281	270	258
10 × 6 × 40	14 × ⅝	141·8	41·04	7·0	268	259	249	238
10 × 6 × 40	14 × ½	129·9	37·54	7·0	244	236	227	217
10 × 6 × 40	14 × ⅜	118·0	34·00	7·0	221	214	206	196

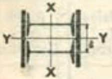
For notes relating to above see page xi

Double Joists with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
22	24	28	32	36	40	Axis y—y	Axis x—x	Axis y—y	Axis x—x
650	630	585	535	481	428	301	456	5·09	5·53
537	520	483	441	396	352	247	365	5·07	5·24
421	407	376	342	305	270	193	278	5·04	4·95
391	378	348	314	280	247	179	257	5·03	4·86
361	349	320	288	256	225	166	236	5·02	4·78
332	320	292	262	231	203	152	216	5·00	4·70
302	291	265	236	208	182	139	195	4·98	4·62
for	10 ×	8 an	d ¾ d	ia. f	or 1	0 × 6			
364	344	300	258	219	187	145	278	3·94	5·25
285	269	234	201	171	145	113	210	3·91	4·94
265	250	218	186	158	135	105	193	3·90	4·86
245	231	201	172	146	124	96	177	3·89	4·78
226	213	185	158	134	114	88	161	3·88	4·68
206	194	168	144	122	104	80	145	3·86	4·60
186	175	152	129	110	94	72	129	3·85	4·51

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons



Two Joists	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	Web Centres S in ins.	EFFECTIVE 12	14	16	18
10 × 5 × 30	12 × 1½	184·8	53·70	6·0	351	338	323	306
10 × 5 × 30	12 × 1	144·0	41·70	6·0	272	262	250	236
10 × 5 × 30	12 × ⅞	133·8	38·70	6·0	252	243	232	219
10 × 5 × 30	12 × ¾	123·6	35·70	6·0	233	224	213	201
10 × 5 × 30	12 × ⅝	113·4	32·70	6·0	213	205	195	184
10 × 5 × 30	12 × ½	103·2	29·70	6·0	193	185	177	167
10 × 5 × 30	12 × ⅜	93·0	26·70	6·0	172	166	158	149
					Ri	vet	s ¾	dia.
9 × 7 × 50	16 × 2	321·2	93·42	8·0	642	628	612	596
9 × 7 × 50	16 × 1½	266·8	77·42	8·0	532	520	507	493
9 × 7 × 50	16 × 1	212·4	61·42	8·0	421	410	400	390
9 × 7 × 50	16 × ⅞	198·8	57·42	8·0	393	383	373	363
9 × 7 × 50	16 × ¾	185·2	53·42	8·0	364	356	346	336
9 × 7 × 50	16 × ⅝	171·6	49·42	8·0	336	328	319	308
9 × 7 × 50	16 × ½	158·0	45·42	8·0	308	300	291	282

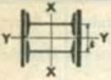
For notes relating to above see page xi

Double Joists with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
20	24	28	32	36	40	Axis y—y	Axis x—x	Axis y—y	Axis x—x
287	246	205	170	141	118	102	229	3·37	5·27
221	189	158	130	108	91·0	78	170	3·35	4·96
205	175	146	120	100	84·1	72	156	3·34	4·86
189	161	134	110	92·1	77·2	66	141	3·33	4·77
172	147	122	101	83·8	70·2	60	127	3·31	4·68
156	132	110	91·0	75·5	63·2	54	113	3·30	4·58
139	118	98·4	80·5	67·2	56·2	48	99	3·28	4·48
for	10 ×	5 an	d $\frac{7}{8}$ d	ia. f	or 9	× 7			
577	534	484	431	377	328	239	365	4·53	5·04
478	442	400	355	311	271	197	291	4·51	4·76
377	348	314	278	243	210	154	222	4·48	4·46
350	322	290	255	222	192	143	205	4·47	4·38
324	297	266	233	202	174	133	188	4·46	4·30
297	271	242	211	182	157	122	172	4·45	4·22
271	246	218	189	163	140	111	155	4·43	4·14

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS— Safe Concentric Loads in Tons



Two Joists	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	Web Centres S in ins.	EFFECTIVE 12	14	16	18
8 × 8 × 38	18 × 1½	263·2	76·26	9·0	523	510	497	482
8 × 8 × 38	18 × 1	202·0	58·26	9·0	395	386	374	362
8 × 8 × 38	18 × ⅞	186·7	53·76	9·0	364	354	343	332
8 × 8 × 38	18 × ¾	171·4	49·26	9·0	333	322	313	302
8 × 8 × 38	18 × ⅝	156·1	44·76	9·0	301	293	283	272
8 × 8 × 38	18 × ½	140·8	40·26	9·0	271	262	253	243
8 × 8 × 38	18 × ⅜	125·5	35·76	9·0	239	231	223	214
					Ri	vet	s ⅞	dia.
8 × 6 × 35	14 × 1½	215·1	62·60	7·0	422	410	397	382
8 × 6 × 35	14 × 1	167·5	48·60	7·0	327	318	308	296
8 × 6 × 35	14 × ⅞	155·6	45·10	7·0	303	295	285	274
8 × 6 × 35	14 × ¾	143·7	41·60	7·0	280	272	263	253
8 × 6 × 35	14 × ⅝	131·8	38·10	7·0	255	247	239	230
8 × 6 × 35	14 × ½	119·9	34·60	7·0	231	224	216	207
8 × 6 × 35	14 × ⅜	108·0	31·10	7·0	207	200	193	184

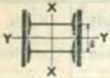
For notes relating to above see page xi

Double Joists with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
20	24	28	32	36	40	Axis y—y	Axis x—x	Axis y—y	Axis x—x
468	432	388	342	298	260	221	271	5·10	4·43
348	316	279	243	209	180	167	199	5·08	4·13
318	288	253	218	188	161	153	182	5·06	4·05
289	260	228	196	169	143	140	164	5·05	3·98
260	233	204	174	147	126	126	148	5·04	3·90
232	206	178	152	129	109	113	131	5·02	3·82
203	180	155	131	110	93·6	100	114	5·00	3·73
for	8 × 8	and	¾ di	a. fo	r 8 ×	6			
366	329	288	247	210	179	140	216	3·95	4·36
283	254	222	190	162	138	107	160	3·93	4·06
262	235	205	176	149	127	99	146	3·92	3·98
242	216	188	161	137	117	91	133	3·91	3·90
219	195	169	144	122	103	82	120	3·89	3·82
197	174	150	127	107	90·9	74	107	3·88	3·74
175	153	131	111	93·2	78·5	66	95	3·86	3·65

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons



Two Joists	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	Web Centres S in ins.	EFFECTIVE 12	14	16	18
8 × 5 × 28	12 × 1½	181·0	52·56	6·0	344	331	316	300
8 × 5 × 28	12 × 1	140·2	40·56	6·0	265	255	243	230
8 × 5 × 28	12 × ⅞	130·0	37·56	6·0	245	236	225	213
8 × 5 × 28	12 × ¾	119·8	34·56	6·0	225	217	207	196
8 × 5 × 28	12 × ⅝	109·6	31·56	6·0	206	198	189	178
8 × 5 × 28	12 × ½	99·4	28·56	6·0	186	179	170	161
8 × 5 × 28	12 × ⅜	89·2	25·56	6·0	166	160	152	143
						Ri	vet	s
6 × 5 × 25	12 × 1½	174·9	50·74	6·0	334	321	308	290
6 × 5 × 25	12 × 1	134·1	38·74	6·0	251	240	227	214
6 × 5 × 25	12 × ⅞	123·9	35·74	6·0	229	218	206	192
6 × 5 × 25	12 × ¾	113·7	32·74	6·0	208	198	186	173
6 × 5 × 25	12 × ⅝	103·5	29·74	6·0	188	178	167	155
6 × 5 × 25	12 × ½	93·3	26·74	6·0	167	158	147	135
6 × 5 × 25	12 × ⅜	83·1	23·74	6·0	147	138	129	118

For notes relating to above see page xi

Double Joists with Plates and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
20	24	28	32	36	40	Axis y—y	Axis x—x	Axis y—y	Axis x—x
281	241	202	167	139	117	**100**	181	**3·38**	4·36
216	184	154	127	106	89·2	**76**	133	**3·36**	4·06
199	170	142	118	98·0	82·2	**70**	122	**3·35**	3·98
183	156	130	108	89·7	75·3	**64**	110	**3·34**	3·90
167	142	118	98·1	81·5	68·2	**58**	99	**3·33**	3·81
150	128	106	88·2	73·2	61·3	**52**	88	**3·31**	3·73
134	114	94·8	78·2	64·9	54·3	**46**	77	**3·29**	3·63
¾	dia.								
272	234	194	164	135	113	97	**133**	3·38	**3·44**
198	165	136	111	91·2		73	**96**	3·36	**3·15**
178	147	120	98·4	80·4		67	**87**	3·35	**3·07**
159	131	106	86·0	71·0		61	**78**	3·34	**2·99**
141	115	92·0	74·6	61·1		55	**70**	3·33	**2·91**
123	99·0	79·0	64·0			49	**61**	3·32	**2·83**
106	84·8	67·2	54·1			43	**53**	3·30	**2·74**

See page xiv regarding various conditions of End Fixity

CHANNEL STANCHIONS—
Safe Concentric Loads in Tons

Section	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
			4	5	6	7	8
17 × 4	51·28	15·08	97·4	91·0	83·2	74·2	64·8
17 × 4	44·34	13·04	84·6	79·2	72·7	65·1	57·2
15 × 4	42·49	12·50	81·3	76·2	70·0	62·9	55·4
15 × 4	36·37	10·70	70·0	65·9	60·8	55·0	48·8
13 × 4	38·92	11·45	75·0	70·7	65·4	59·3	52·7
13 × 4	33·18	9·76	64·1	60·4	56·0	50·9	45·3
12 × 4	36·63	10·77	70·6	66·5	61·6	55·8	49·5
12 × 4	31·33	9·21	60·6	57·2	53·1	48·3	43·1
12 × 3½	30·45	8·96	56·0	51·2	45·3	39·0	33·2
12 × 3½	26·37	7·76	48·8	44·8	39·9	34·6	29·5
11 × 3½	30·52	8·98	57·0	52·6	47·4	41·4	35·5
•11 × 3½	29·82	8·77	55·6	51·2	45·9	40·0	34·3
11 × 3½	26·78	7·88	50·2	46·4	41·8	36·8	31·6
•10 × 4	18·86	5·55	36·4	34·3	31·7	28·8	25·6
10 × 3½	28·54	8·39	53·5	49·6	44·3	39·4	34·0

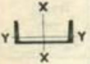
For notes relating to above see page xi

and Dimensions and Properties

HEIGHTS IN FEET					Moduli of Section		Radii of Gyration	
9	10	11	12	13	Axis y—y	Axis x—x	Axis y—y	Axis x—x
56·0	48·2	41·6	36·1	31·6	5·3	66·9	1·06	6·14
49·6	42·9	37·1	32·2	28·1	5·0	61·2	1·08	6·32
48·1	41·6	36·0	31·3	27·4	4·7	51·1	1·09	5·54
42·8	37·1	32·2	28·0	24·5	4·4	46·5	1·12	5·71
46·1	40·1	34·9	30·4	26·6	4·6	41·6	1·13	4·86
39·8	34·6	30·1	26·3	23·1	4·3	37·9	1·14	5·03
43·4	37·8	32·8	28·6	25·1	4·4	36·5	1·13	4·51
37·9	33·1	28·8	25·2	22·1	4·1	33·4	1·15	4·66
28·0	23·8	20·3			2·9	29·0	·94	4·41
25·0	21·3	18·2	15·7		2·7	26·6	·96	4·54
30·4	25·8	22·2	19·1		3·3	27·8	·99	4·13
29·2	24·9	21·3	18·4		3·2	27·0	·98	4·12
27·0	23·0	19·8	17·1		3·1	25·8	1·00	4 24
22·4	19·5	16·9	14·8	12·9	2·3	16·5	1·13	3·86
29·1	24·9	21·4	18·5		3·2	23·9	1·01	3·77

See page xiv regarding various conditions of End Fixity

CHANNEL STANCHIONS—
Safe Concentric Loads in Tons

 Section	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
			4	5	6	7	8
10 × 3½	24·46	7·19	46·0	42·7	38·6	34·1	29·5
10 × 3	21·33	6·27	37·5	33·1	28·2	23·5	19·4
10 × 3	19·28	5·67	34·1	30·2	25·8	21·6	17·8
*10 × 2·6	15·06	4·43	24·7	20·8	16·9	13·6	10·9
9 × 3½	23·49	6·91	44·3	41·2	37·4	33·1	28·7
9 × 3½	22·27	6·55	42·0	39·0	35·4	31·3	27·2
9 × 3	19·91	5·86	35·3	31·5	27·0	22·6	18·8
9 × 3	17·46	5·14	31·1	27·8	24·0	20·1	16·8
8 × 3½	23·20	6·82	43·7	40·6	36·9	32·6	28·3
8 × 3½	20·21	5·94	38·2	35·5	32·3	28·7	24·9
8 × 3	18·68	5·49	33·4	29·9	25·9	21·8	18·2
8 × 3	15·96	4·69	28·5	25·6	22·1	18·6	15·5
*8 × 2·26	11·22	3·30	16·8	13·4	10·4	8·2	
7 × 3½	18·28	5·38	34·6	35·7	29·3	26·0	22·6
7 × 3	17·07	5·02	30·7	27·6	23·9	20·2	16·9

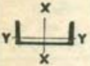
For notes relating to above see page xi

and Dimensions and Properties

HEIGHTS IN FEET					Moduli of Section		Radii of Gyration	
9	10	11	12	13	Axis y—y	Axis x—x	Axis y—y	Axis x—x
25·3	21·7	18·6	16·1		2·9	21·9	1·02	3·90
16·2	13·5				1·8	17·5	·83	3·74
14·8	12·4				1·8	16·5	·84	3·82
8·9					1·2	13·4	·72	3·87
24·6	21·1	18·2	15·7		2·8	18·9	1·03	3·51
23·3	20·0	17·2	14·9		2·8	18·4	1·03	3·56
15·6	13·1				1·8	14·9	·85	3·39
14·0	11·8				1·7	13·9	·86	3·49
24·3	20·9	17·9	15·5		2·8	16·3	1·03	3·09
21·6	18·4	15·9	13·7	12·0	2·6	15·1	1·04	3·19
15·2	12·8				1·8	12·7	·87	3·05
13·0	10·9				1·6	11·7	·87	3·16
					·77	8·0	·63	3·10
19·4	16·8	14·4	12·4	10·8	2·4	12·2	1·04	2·82
14·1	11·9	10·1			1·7	10·3	·88	2·68

See page xiv regarding various conditions of End Fixity

CHANNEL STANCHIONS— Safe Concentric Loads in Tons

 Section	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
			3	4	5	6	7
7 × 3	14·22	4·18	27·6	25·5	22·9	19·9	16·8
*7 × $2\frac{1}{8}$	9·75	2·86	17·0	14·2	11·2	8·6	6·7
6 × $3\frac{1}{2}$	16·48	4·85	33·0	31·2	29·1	26·5	23·6
6 × 3	16·51	4·86	32·0	29·6	26·5	22·9	19·3
6 × 3	13·64	4·01	26·5	24·5	22·0	19·1	16·1
6 × 3	12·41	3·65	24·1	22·3	20·0	17·4	14·8
*6 × 2	10·75	3·15	17·5	13·8	10·4	7·8	
*6 × 1·92	8·75	2·57	14·5	11·5	8·7	6·6	
5 × $2\frac{1}{2}$	10·22	3·01	19·1	17·0	14·5	11·8	9·5
4 × 2	7·91	2·33	13·5	11·0	8·4	6·4	5·0
4 × 2	7·09	2·09	12·1	9·8	7·6	5·8	4·4
*$3\frac{1}{2}$ × 2	6·75	1·99	11·7	9·7	7·6	5·8	4·5
*3 × $1\frac{1}{2}$	5·27	1·55	7·4	5·2	3·8		
3 × $1\frac{1}{2}$	5·11	1·50	7·1	5·0	3·6		
3 × $1\frac{1}{2}$	4·60	1·35	6·4	4·6	3·2		

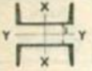
For notes relating to above see page xi

and Dimensions and Properties

HEIGHTS IN FEET					Moduli of Section		Radii of Gyration	
8	9	10	11	12	Axis y—y	Axis x—x	Axis y—y	Axis x—x
14·1	11·8	9·9	8·4		1·5	9·4	·88	2·80
					·68	5·9	·61	2·67
20·6	17·8	15·3	13·1	11·4	2·3	9·6	1·05	2·44
16·1	13·4	11·3			1·8	8·8	·87	2·33
13·5	11·3	9·5	8·1		1·4	7·4	·88	2·36
12·3	10·3	8·6	7·3		1·3	7·1	·88	2·41
					·62	5·2	·54	2·23
					·56	4·6	·55	2·32
7·7	6·3				·95	4·7	·74	1·99
					·54	2·7	·58	1·52
					·50	2·5	·58	1·56
					·53	2·1	·60	1·37
					·29	1·3	·44	1·13
					·28	1·3	·44	1·14
					·26	1·2	·44	1·16

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons

 Two Channels	Weight per foot in lbs.	Area in sq. ins.	Space between webs S in ins.	EFFECTIVE			
				8	10	12	16
17×4 ×44·34	88·68	26·08	10·5	189	187	185	180
15×4 ×36·37	72·74	21·40	9·5	155	153	150	146
13×4 ×33·18	66·36	19·52	7·5	140	137	135	130
12×4 ×31·33	62·66	18·42	7·0	131	129	127	121
12×3½×26·37	52·74	15·52	7·0	110	108	106	101
11×3½×26·78	53·56	15·76	6·5	112	109	107	101
10×3½×24·46	48·92	14·38	5·5	101	99·1	96·6	90·6
10×3 ×21·33	42·66	12·54	6·0	88·3	86·1	83·8	78·4
10×3 ×19·28	38·56	11·34	6·0	79·9	78·1	76·0	71·2
10×2·6×15·06	30·12	8·86	6·5	62·5	61·1	59·5	55·9
9×3½×22·27	44·54	13·10	5·0	91·9	89·6	86·7	80·4
9×3 ×19·91	39·82	11·72	5·0	81·7	79·4	76·7	70·6
9×3 ×17·46	34·92	10·28	5·0	71·6	69·6	67·4	62·0
8×3½×23·20	46·40	13·64	4·0	94·2	91·0	87·7	79·1
8×3½×20·21	40·42	11·88	4·0	82·3	79·8	76·8	69·8

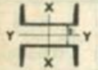
For notes relating to above see page xi

Two Channels Battened Together and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
20	24	28	32	36	40	Axis y—y	Axis x—x	Axis y—y	Axis x—x
174	167	160	152	142	132	111	122	6·26	6·32
140	134	127	118	109	99·5	83	93	5·83	5·71
123	116	107	97·3	86·9	76·8	61	76	4·92	5·03
114	106	97·6	87·4	77·1	67·4	54	67	4·70	4·66
95·2	87·8	79·2	70·0	61·1	53·0	44	53	4·44	4·54
95·1	86·9	77·5	67·7	58·5	50·4	43	52	4·30	4·24
83·2	74·2	64·5	55·0	46·7	39·7	34	44	3·86	3·90
71·5	63·3	54·4	46·1	39·0	32·9	30	35	3·82	3·74
65·3	58·1	50·3	42·9	36·3	30·8	28	33	3·83	3·82
51·4	45·8	39·8	34·0	28·9	24·6	24	27	3·95	3·87
72·4	63·1	53·5	44·7	37·5	31·6	29	37	3·65	3·55
62·5	53·5	44·6	37·0	30·7	25·8	24	30	3·36	3·39
55·1	47·3	39·6	32·8	27·3	22·9	21	28	3·39	3·49
68·2	56·6	46·1	37·8	31·0		25	33	3·18	3·09
61·0	51·2	42·2	34·6	28·6		22	30	3·22	3·19

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons



Two Channels	Weight per foot in lbs.	Area in sq. ins.	Space between webs S in ins.	EFFECTIVE 4	6	8	12
8 × 3 × 18·68	37·36	10·98	4·5	80·0	77·8	75·6	70·2
8 × 3 × 15·96	31·92	9·38	4·5	68·4	66·7	64·9	60·5
8 × 2·26 × 11·22	22·44	6·60	5·0	48·0	46·9	45·5	42·4
7 × 3½ × 18·28	36·56	10·76	3·0	77·8	75·6	73·2	67·1
7 × 3 × 17·07	34·14	10·04	3·5	72·5	70·3	68·0	61·7
7 × 3 × 14·22	28·44	8·36	3·5	60·5	58·8	56·8	52·0
7 × 2⅛ × 9·75	19·50	5·72	4·0	41·2	40·0	38·6	34·9
6 × 3 × 16·51	33·02	9·72	2·5	69·6	67·0	64·1	56·4
6 × 3 × 12·41	24·82	7·30	3·0	52·3	50·6	48·5	43·0
6 × 2 × 10·75	21·50	6·30	3·5	45·0	43·2	41·2	35·8
6 × 1·92 × 8·75	17·50	5·14	3·5	36·8	35·4	33·8	29·7
5 × 2½ × 10·22	20·44	6·01	2·0	42·4	40·4	37·9	31·0
4 × 2 × 7·96	15·92	4·68	2·5	32·4	30·1	27·2	19·7
4 × 2 × 7·09	14·18	4·17	2·0	28·7	26·8	24·3	17·5
3 × 1½ × 4·60	9·20	2·70	2·0	17·8	15·7	12·8	7·5

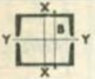
For notes relating to above see page xi

Two Channels Battened Together and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
16	20	24	28	32	36	Axis y—y	Axis x—x	Axis y—y	Axis x—x
63·1	54·2	44·8	36·6	29·8	24·5	21·1	**25·5**	3·18	**3·05**
54·9	47·8	40·0	32·8	26·9	22·2	18·3	**23·4**	3·20	**3·16**
38·3	33·2	27·5	22·6	18·3	15·2	13·7	**16·0**	3·14	**3·10**
58·8	48·9	39·3	31·3	25·2		**16·8**	24·5	**2·79**	2·82
53·4	43·7	34·6	27·6	22·2		15·8	**20·7**	2·73	**2·68**
45·5	37·7	30·2	24·1	19·4		**13·5**	18·7	**2·77**	2·80
30·0	24·3	19·1	15·1	12·1		**9·5**	11·7	**2·62**	2·67
46·0	35·5	27·1	21·0			**12·4**	17·5	**2·33**	**2·33**
35·8	28·0	21·4	16·6			10·5	**14·2**	2·55	**2·41**
28·6	21·7	16·4				9·2	**10·5**	2·33	**2·23**
24·2	18·7	14·3	11·0			7·7	**9·2**	2·34	**2·32**
22·9	16·5	12·1				**6·3**	9·5	**1·92**	1·99
13·2						5·8	**5·7**	2·00	**1·56**
11·8						4·0	**5·1**	1·70	**1·56**
						2·6	**2·4**	1·54	**1·16**

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons

 Two Channels	Weight per foot in lbs.	Area in sq. ins.	Breadth B in ins.	EFFECTIVE			
				4	8	12	16
12×3½×26·37	52·74	15·52	12	115	110	107	102
10×3½×24·46	48·92	14·38	10	106	102	96·8	90·9
10×3 ×19·28	38·56	11·34	10	83·2	79·9	76·0	71·2
10×2·6×15·06	30·12	8·86	10	65·0	62·5	59·5	55·9
9×3 ×17·46	34·92	10·28	10	75·2	71·8	67·9	62·5
8×3 ×15·96	31·92	9·38	10	68·4	64·9	60·5	54·9
8×2·26×11·22	22·44	6·60	8	48·0	45·5	42·3	38·3
7×3 ×14·22	28·44	8·36	10	60·5	56·9	52·2	45·8
7×2½× 9·75	19·50	5·72	10	41·3	38·7	35·1	30·4
6×3 ×12·41	24·82	7·30	8	52·4	48·5	43·0	35·8
6×2 ×10·75	21·50	6·30	8	45·0	41·2	35·8	28·6
6×1·92×8·75	17·50	5·14	8	36·7	33·8	29·8	24·2
5×2½×10·22	20·44	6·01	8	42·5	38·2	31·8	23·9
4×2 × 7·09	14·18	4·17	6	28·8	24·3	17·5	11·7
3×1½× 4·60	9·20	2·70	5	17·8	12·8	7·5	

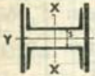
For notes relating to above see page xi

Two Channels Battened Together and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
20	24	28	32	36	40	Axis y—y	Axis x—x	Axis y—y	Axis x—x
96·0	88·9	80·7	71·8	63·0	54·8	71·5	**53·2**	5·26	**4·54**
83·7	74·7	65·4	55·8	47·5	40·6	49·7	**43·8**	4·16	**3·90**
65·3	58·1	50·3	42·9	36·3	30·8	42·8	**33·1**	4·34	**3·82**
51·4	45·7	39·8	34·0	28·9	24·6	34·7	**26·8**	4·42	**3·87**
56·3	48·4	40·9	34·3	28·7	24·0	38·1	**27·8**	4·31	**3·49**
47·8	40·0	32·8	26·9	22·2		34·1	**23·4**	4·26	**3·16**
33·2	27·5	22·6	18·3	15·2		20·0	**16·0**	3·48	**3·10**
38·1	30·6	24·5	19·7			29·8	**18·7**	4·22	**2·80**
24·8	19·6	15·5	12·4			23·1	**11·7**	4·49	**2·67**
27·9	21·5	16·7				25·8	**14·2**	4·20	**2·41**
21·8	16·4					19·6	**10·46**	3·52	**2.23**
18·7	14·3	11·0				15·8	**9·2**	3·50	**2·32**
17·6	13·0					16·5	**9·5**	3·31	**1·99**
						8·5	**5·1**	2·47	**1·56**
						4·6	**2·4**	2·07	**1·16**

See page xiv regarding various conditions of End Fixity

COMPOUND STANCHIONS— Safe Concentric Loads in Tons



Two Channels	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	Space between webs S in ins.	EFFECT 12	EFFECT 16
10×3½×24·46	12 × 1½	174·9	50·38	4·0	330	303
10×3½×24·46	12 × 1	134·1	38·38	4·0	251	230
10×3½×24·46	12 × ⅞	123·9	35·38	4·0	231	212
10×3½×24·46	12 × ¾	113·7	32·38	4·0	211	193
10×3½×24·46	12 × ½	92·0	26·38	4·0	171	157
10×3 ×19·28	12 × 1½	163·2	47·34	5·0	311	287
10×3 ×19·28	12 × 1	122·4	35·34	5·0	232	214
10×3 ×19·28	12 × ⅞	112·2	32·34	5·0	212	195
10×3 ×19·28	12 × ¾	102·0	29·34	5·0	192	177
10×3 ×19·28	12 × ½	82·0	23·34	5·0	153	141
9×3½×22·27	12 × 1½	170·5	49·10	4·0	321	296
9×3½×22·27	12 × 1	129·7	37·10	4·0	242	223
9×3½×22·27	12 × ⅞	119·5	34·10	4·0	223	204
9×3½×22·27	12 × ¾	109·3	31·10	4·0	203	186
9×3½×22·27	12 × ½	88·0	25·10	4·0	163	150

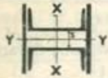
For notes relating to above see page xi

Double Channels with Plates and Dimensions and Properties

IVE HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
20	24	28	32	36	40	Axis y—y	Axis x—x	Axis y—y	Axis x—x
269	231	192	159	133	111	96	218	3·37	5·30
204	174	145	120	100	83·8	72	158	3·35	4·98
187	160	133	110	91·7	76·8	66	143	3·34	4·88
171	146	121	100	83·4	69·9	60	129	3·32	4·78
138	117	97·7	80·6	66·8	55·9	48	100	3·29	4·57
256	221	185	154	128	108	93	210	3·44	5·36
191	164	138	114	95·6	80·3	69	149	3·43	5·02
174	150	126	104	87·4	73·5	63	134	3·42	4·93
158	136	114	95·0	79·2	66·5	57	119	3·42	4·84
125	108	90·5	75·2	62·6	52·7	45	90	3·41	4·61
263	226	189	156	130	109	94	194	3·39	4·87
198	169	141	117	97·5	81·8	70	139	3·36	4·56
181	155	129	107	89·2	74·8	64	126	3·35	4·46
165	141	117	97·4	80·9	67·8	58	113	3·34	4·37
132	112	93·8	77·5	64·3	54·0	46	87	3·31	4·17

Rivets $\frac{7}{8}$ dia. for $10 \times 3\frac{1}{2}$ and $9 \times 3\frac{1}{2}$ and $\frac{3}{4}$ dia. for 10×3

COMPOUND STANCHIONS— Safe Concentric Loads in Tons

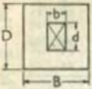
 Two Channels	Flange Plates	Weight per foot in lbs.	Area in sq. ins.	Space between webs S in ins.	EFFECT 8	12
9 × 3 × 17·46	12 × 1½	159·6	46·28	5·0	323	304
9 × 3 × 17·46	12 × 1	118·8	34·28	5·0	239	225
9 × 3 × 17·46	12 × ⅞	108·6	31·28	5·0	218	205
9 × 3 × 17·46	12 × ¾	98·4	28·28	5·0	197	186
9 × 3 × 17·46	12 × ½	78·0	22·28	5·0	155	146
8 × 3½ × 20·21	10 × 1	112·0	31·88	2·0	215	196
8 × 3½ × 20·20	10 × ⅞	103·5	29·38	2·0	198	180
8 × 3½ × 20·20	10 × ¾	95·0	26·88	2·0	181	164
8 × 3½ × 20·20	10 × ⅝	86·5	24·38	2·0	164	148
8 × 3½ × 20·20	10 × ½	78·0	21·88	2·0	147	132
8 × 3 × 15·96	10 × 1	102·1	29·38	3·0	199	182
8 × 3 × 15·96	10 × ⅞	93·6	26·88	3·0	182	166
8 × 3 × 15·96	10 × ¾	85·1	24·38	3·0	165	151
8 × 3 × 15·96	10 × ⅝	76·6	21·88	3·0	148	135
8 × 3 × 15·96	10 × ½	68·1	19·38	3·0	131	119

For notes relating to above see page xi

Double Channels with Plates and Dimensions and Properties

IVE HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
16	20	24	28	32	36	Axis y—y	Axis x—x	Axis y—y	Axis x—x
281	251	216	182	151	126	92	187	3·45	4·93
208	185	160	134	112	93·5	68	132	3·44	4·61
189	169	146	122	102	85·2	62	119	3·44	4·53
171	153	132	110	92·3	76·9	56	105	3·44	4·42
135	120	103	87·1	72·4	60·3	44	79	3·43	4·22
169	138	110	87·1	69·8		46	106	2·68	4·07
155	127	100	79·6	63·7		42	96	2·66	3·99
141	115	90·9	71·7	57·4		37	86	2·64	3·91
127	103	81·1	64·1	51·2		33	76	2·62	3·81
113	91·4	71·6	56·4	44·9		29	67	2·58	3·73
159	132	106	84·3	67·8		45	100	2·77	4·14
145	120	96·3	76·6	61·6		41	90	2·75	4·04
131	108	86·8	69·1	55·4		37	80	2·74	3·95
117	97·1	77·2	61·3	49·3		32	71	2·72	3·86
103	85·1	67·6	53·6	42·9		28	61	2·70	3·77

Rivets ¾ dia. for 9 × 3 and 8 × 3 and ⅞ dia. for 8 × 3½



SLAB BASES FOR R.S. STANCHIONS

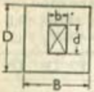
Safe Uniformly Distributed Loads in Tons

Allowing for a Concrete Bearing Pressure up to 40 tons per sq. ft.

Size of Slab in inches	Area of Slab in sq. ft.	d = 12″				d = 13″		d = 14″				d = 15″	
		b				b		b				b	
D × B × T		6″	8″	10″	12″	10″	12″	8″	10″	12″	14″	12″	14″
21 × 21 × 1½	3·063	58	80	118		113		76	109				
24 × 24 × 1½	4·000	54	71	97	144	94	137	67	91	131		126	
24 × 24 × 2	4·000	96	126			167		120	162				
27 × 27 × 1½	5·063	51	64	85	117	82	112	62	80	108	155	104	148
27 × 27 × 2	5·063	91	115	150	207	145	199	110	142	191		185	
30 × 30 × 2	6·250	87	107	135	178	132	172	103	129	166	225	161	216
30 × 30 × 2½	6·250	136	167	212		206		161	201	260		252	
33 × 33 × 2½	7·563	132	159	195	247	190	240	153	186	233	302	227	292

Size of Slab in inches	Area of Slab in sq. ft.	d = 12"				d = 13"		d = 14"				d = 15"	
		b				b		b				b	
D × B × T		6"	8"	10"	12"	10"	12"	8"	10"	12"	14"	12"	14"
33 × 33 × 3	7·563	190	228	281		274		220	268				
36 × 36 × 2½	9·000	128	152	183	225	179	219	147	175	214	268	209	260
36 × 36 × 3	9·000	185	219	263	324	257	315	211	252	308		301	
39 × 39 × 2½	10·560	126	146	173	209	170	204	142	167	199	243	195	237
39 × 39 × 3	10·560	181	211	249	300	244	293	204	240	287	350	281	342
39 × 39 × 3½	10·560			339	409	333	399	278	327	390		382	
42 × 42 × 3	12·250	178	205	238	282	234	276	198	230	271	324	265	317
42 × 42 × 3½	12·250			324	384	319	376	270	313	368	441	361	431
42 × 42 × 4	12·250						491	353	409	481		472	

The above safe loads are calculated in accordance with the B.S.S. 449—1937. Thickness 'T' given is the finished Machined Thickness. Slabs bearing on Concrete do not require to be machined on under surface. See Page 232 for Round Col. Bases.



SLAB BASES FOR R.S. STANCHIONS

Safe Uniformly Distributed Loads in Tons

Allowing for a Concrete Bearing Pressure up to 40 tons per sq. ft.

Size of Slab in inches	Area of Slab in sq. ft.	$d = 8''$		$d = 9''$			$d = 10''$				$d = 11''$		
		b		b			b				b		
$D \times B \times T$		6″	8″	6″	7″	9″	6″	8″	9″	10″	8″	9″	10″
$21 \times 21 \times 1\frac{1}{2}$	3·063	65	94	63	74	110	61	86	105		83	100	124
$24 \times 24 \times 1\frac{1}{2}$	4·000	60	81	58	67	92	56	75	88	106	73	85	101
$24 \times 24 \times 2$	4·000	106	144	103	119	164	101	134	157		129	151	
$27 \times 27 \times 2$	5·063	100	129	97	110	144	95	121	139	161	118	135	156
$27 \times 27 \times 2\frac{1}{2}$	5·063	156	202	152	171		148	189			184	210	
$30 \times 30 \times 2$	6·250	95	119	93	103	131	91	113	127	144	110	123	139
$30 \times 30 \times 2\frac{1}{2}$	6·250	148	186	145	161	204	142	176	198	225	171	192	218
$33 \times 33 \times 2\frac{1}{2}$	7·563	143	174	140	154	189	137	166	184	206	162	180	200

Size of Slab in inches	Area of Slab in sq. ft.	d – 8″ b		d – 9″ b			d – 10″ b				d – 11″ b		
D × B × T		6″	8″	6″	7″	9″	6″	8″	9″	10″	8″	9″	10″
33 × 33 × 3	7·563	205	251	201	221	272	197	239	265	296	233	258	288
36 × 36 × 2½	9·000	138	165	135	148	178	133	158	174	192	155	170	187
36 × 36 × 3	9·000	199	238	195	212	256	191	228	250	276	223	244	269
39 × 39 × 2½	10·560	134	158	132	143	169	130	152	165	181	149	162	177
39 × 39 × 3	10·560						187	219	238	260	215	233	255
42 × 42 × 3	12·250						183	212	229	248	208	224	243
42 × 42 × 3½	12·250									338	283	306	331
45 × 45 × 3	14·060						180	206	221	238	203	217	234
45 × 45 × 3½	14·060									324	276	296	318

The above safe loads are calculated in accordance with the B.S.S. 449—1937. Thickness 'T' given is the finished Machined Thickness. Slabs bearing on Concrete do not require to be machined on under surface.

SOLID ROUND COLUMNS—
Safe Concentric Loads in Tons

Dia. ins.	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
			6	8	10	12	14
12	384·5	113·10	800	776	750	720	685
10	267·0	78·54	545	525	500	470	435
9	216·3	63·63	437	416	392	362	328
8	170·9	50·26	340	320	295	266	234
7½	150·2	44·19	295	276	252	223	192
7	130·8	38·48	254	235	210	182	154
6½	112·8	33·18	215	196	172	146	120
6	96·1	28·27	180	161	137	113	91·9
5½	80·8	23·76	147	128	106	85·1	67·6
5	66·8	19·64	117	99·2	78·7	61·2	47·9
4½	54·1	15·9	90·7	72·9	55·4	42·0	32·4
4	42·7	12·57	66·6	50·5	36·8	27·2	
3½	32·7	9·62	45·7	32·2	22·7		
3	24·0	7·08	28·3	18·7			
2½	16·7	4·91	15·2				

For notes relating to above see page xi

and Dimensions and Properties

HEIGHTS IN FEET 16	18	20	22	24	26	28	30	Modulus of Section	Radius of Gyration
645	600	550	502	453	408	367	331	170·0	3·00
396	355	315	277	244	216	191	170	98·2	2·50
291	255	221	192	168	147	129		71·6	2·25
201	172	147	126	108				50·3	2·00
163	137	116	99·7					41·4	1·87
129	107	90·8						33·7	1·75
99·4	82·4	68·9						27·0	1·62
74·8	61·3							21·2	1·50
54·4								16·3	1·37
								12·3	1·25
								9·0	1·12
								6·3	1·00
								4·2	·87
								2·7	·75
								1·5	·62

Modulus of Section
for Solid Round Columns
$= 0{\cdot}0982\ D^3$
(D = diameter in ins.)

See pages 232, 233 for thickness of Slab Caps and Bases

THICKNESSES OF SLAB CAPS & For Given Uniformly Distributed Pressure of 30 tons

 Size of Slab in ins.	Maximum Load on Slab in tons	DIAMETER OF SOLID					
		$2\frac{1}{2}$	3	$3\frac{1}{2}$	4	$4\frac{1}{2}$	5
9 × 9	17·0	1·25	1·25				
12 × 12	30·0	1·50	1·50	1·75	1·75	1·75	
15 × 15	47·0			2·00	2·00	2·00	2·25
18 × 18	68·0				2·50	2·50	2·50
21 × 21	92·0					2·75	2·75
24 × 24	120						3·00
27 × 27	152						
30 × 30	188						
33 × 33	227						
36 × 36	270						
39 × 39	317						
42 × 42	368						
45 × 45	422						
48 × 48	480						
51 × 51	542						
54 × 54	608						

Dimensions given are the Finished Thickness of Slabs after machining.

Where the Cap or Base is not square special Calculations have to be made.

BASES for Solid Round Columns Loads based on a Concrete Bearing per square foot.

ROUND COLUMNS IN INCHES									Area of Slab in sq. ft.
5½	6	6½	7	7½	8	9	10	12	
									·563
									1·000
									1·563
2·50	2·50	2·50							2·250
2·75	3·00	3·00	3·00						3·063
3·25	3·25	3·25	3·25	3·25	3·50	3·50			4·000
3·50	3·50	3·50	3·50	3·75	3·75	3·75	4·00		5·063
	3·75	4·00	4·00	4·00	4·00	4·00	4·25		6·250
		4·25	4·25	4·25	4·25	4·50	4·50		7·563
			4·50	4·75	4·75	4·75	5·00		9·000
				5·00	5·00	5·00	5·25	5·50	10·560
					5·50	5·50	5·50	5·75	12·250
						5·75	5·75	6·00	14·060
						6·00	6·25	6·25	16·000
							6·50	6·75	18·060
							7·00	7·00	20·250

HOLLOW ROUND COLUMNS
Safe Concentric Loads in Tons

Outside Dia. in ins.	Thickness in ins.	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE			
				5	6	7	8
$5\frac{1}{2}$	$\frac{3}{8}$	20·528	6·04	36·8	35·8	34·8	33·5
	$\frac{5}{16}$	17·315	5·09	31·1	30·3	29·3	28·3
	$\frac{1}{4}$	14·019	4·12	25·2	24·5	23·8	23·0
	6 i.s.w.g.	10·885	3·20	19·6	19·1	18·5	17·9
	8 i.s.w.g.	9·126	2·68	16·4	16·0	15·5	15·0
$4\frac{1}{2}$	$\frac{3}{8}$	16·523	4·86	28·7	27·6	26·3	24·8
	$\frac{5}{16}$	13·978	4·11	24·3	23·4	22·3	21·1
	$\frac{1}{4}$	11·348	3·34	19·8	19·1	18·2	17·3
	6 i.s.w.g.	8·835	2·60	15·4	14·9	14·2	13·5
	8 i.s.w.g.	7·417	2·17	12·9	12·4	11·9	11·3
4	$\frac{5}{16}$	12·308	3·62	20·9	19·8	18·7	17·3
	$\frac{9}{32}$	11·172	3·29	19·0	18·1	17·0	15·8
	$\frac{1}{4}$	10·013	2·95	17·1	16·3	15·3	14·2
	6 i.s.w.g.	7·809	2·29	13·3	12·6	11·9	11·1
	8 i.s.w.g.	6·562	1·92	11·1	10·6	10·0	9·4

For Notes relating to these Tables see Page xi

and Dimensions and Properties

HEIGHTS IN FEET								Modulus of Section	Radius of Gyration
9	10	11	12	14	16		20		
32·2	30·7	29·1	27·4	23·8	20·3	17·3	14·7	7·25	1·82
27·3	26·0	24·7	23·3	20·3	17·4	14·8	12·6	6·25	1·84
22·2	21·2	20·1	19·0	16·6	14·2	12·2	10·4	5·18	1·86
17·2	16·5	15·7	14·8	13·0	11·1	9·5	8·1	4·10	1·87
14·5	13·9	13·2	12·4	10·9	9·4	8·0	6·9	3·47	1·88
23·2	21·5	19·7	17·9	14·7	12·0	9·9		4·63	1·47
19·8	18·4	16·9	15·4	12·7	10·4	8·6		4·03	1·49
16·2	15·1	13·9	12·7	10·5	8·6	7·1		3·36	1·51
12·7	11·8	10·9	10·0	8·2	6·8	5·6		2·68	1·52
10·6	9·9	9·1	8·4	6·9	5·7	4·7		2·27	1·53
15·8	14·4	12·9	11·5	9·2	7·4			3·10	1·31
14·5	13·2	11·8	10·6	8·5	6·8			2·86	1·32
13·1	11·9	10·7	9·6	7·7	6·2			2·60	1·33
10·2	9·3	8·4	7·5	6·0	4·9			2·08	1·34
8·6	7·9	7·1	6·4	5·1	4·1			1·76	1·35

These values do not exceed $l/r = 150$

i.s.w.g. = Imperial Standard Wire Gauge

HOLLOW ROUND COLUMNS—
Safe Concentric Loads in Tons

Outside Dia. in ins.	Thickness in ins.	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE			
				3	4	5	6
$3\frac{1}{2}$	$\frac{5}{16}$	10·639	3·13	19·1	18·3	17·4	16·2
	$\frac{9}{32}$	9·669	2·84	17·4	16·7	15·8	14·8
	$\frac{1}{4}$	8·678	2·55	15·6	15·0	14·2	13·3
	6 i.s.w.g.	6·784	1·99	12·2	11·7	11·1	10·4
	8 i.s.w.g.	5·708	1·67	10·3	9·9	9·4	8·8
3	$\frac{5}{16}$	8·971	2·64	15·8	14·9	13·8	12·4
	$\frac{9}{32}$	8·167	2·40	14·4	13·6	12·6	11·4
	$\frac{1}{4}$	7·343	2·16	13·0	12·2	11·4	10·3
	6 i.s.w.g.	5·759	1·69	10·2	9·6	8·9	8·1
	8 i.s.w.g.	4·853	1·42	8·6	8·1	7·5	6·9
$2\frac{3}{8}$	$\frac{1}{4}$	5·674	1·68	9·6	8·7	7·6	6·4
	4 i.s.w.g.	5·310	1·56	8·9	8·1	7·1	6·0
	6 i.s.w.g.	4·476	1·32	7·6	6·9	6·1	5·1
	7 i.s.w.g.	4·134	1·22	7·0	6·4	5·6	4·8
	9 i.s.w.g.	3·431	1·01	5·8	5·3	4·7	4·0

For Notes relating to these Tables see Page xi

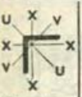
and Dimensions and Properties

HEIGHTS IN FEET								Modulus of Section	Radius of Gyration
7	8	9	10	11	12	13	14		
14·8	13·4	11·9	10·4	9·2	8·0	7·1	6·3	2·29	1·13
13·5	12·2	10·9	9·6	8·4	7·4	6·5	5·8	2·12	1·14
12·2	11·1	9·9	8·7	7·7	6·7	5·9	5·3	1·94	1·15
9·7	8·8	7·8	6·9	6·1	5·4	4·8	4·2	1·56	1·17
8·1	7·4	6·6	5·9	5·2	4·6	4·1	3·6	1·33	1·18
11·0	9·5	8·2	7·0	6·0	5·2			1·61	0·96
10·1	8·7	7·5	6·5	5·6	4·8			1·50	0·97
9·2	8·0	6·9	5·9	5·1	4·4			1·37	0·98
7·2	6·3	5·4	4·7	4·0	3·5			1·12	0·99
6·1	5·4	4·6	4·0	3·5	3·0			0·95	1·00
5·3	4·4	3·6						0·80	0·76
4·9	4·1	3·4						0·76	0·76
4·3	3·5	2·9						0·67	0·77
4·0	3·3	2·8						0·63	0·78
3·4	2·8	2·3						0·53	0·79

These values do not exceed $l/r = 150$.

i.s.w.g. = Imperial Standard Wire Gauge.

STRUTS—Equal Angles
Safe Concentric Loads in Tons

 Section	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE					
			3	4	5	6	7	8
$5 \times 5 \times \frac{1}{2}$	16·16	4·75	31·9	30·0	27·6	24·7	21·5	18·4
$5 \times 5 \times \frac{3}{8}$	12·28	3·62	24·3	22·9	21·1	18·9	16·5	14·0
$4\frac{1}{2} \times 4\frac{1}{2} \times \frac{5}{8}$	17·80	5·25	34·6	31·8	28·6	24·7	20·8	17·4
$4\frac{1}{2} \times 4\frac{1}{2} \times \frac{1}{2}$	14·46	4·25	28·1	25·9	23·4	20·2	17·0	14·2
$4\frac{1}{2} \times 4\frac{1}{2} \times \frac{3}{8}$	11·00	3·24	21·3	19·8	17·8	15·4	13·1	10·9
$4\frac{1}{2} \times 4\frac{1}{2} \times \frac{5}{16}$	9·24	2·72	18·0	16·7	15·1	13·1	11·1	9·3
$4 \times 4 \times \frac{5}{8}$	15·68	4·61	29·5	26·8	23·0	19·2	15·7	12·7
$4 \times 4 \times \frac{1}{2}$	12·76	3·75	24·1	21·7	18·8	15·7	12·8	10·4
$4 \times 4 \times \frac{3}{8}$	9·73	2·86	18·4	16·7	14·6	12·0	9·8	8·0
$4 \times 4 \times \frac{5}{16}$	8·17	2·40	15·4	14·0	12·1	10·1	8·2	6·8
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{8}$	13·55	3·99	24·6	21·4	17·4	13·8	11·1	8·8
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$	11·05	3·25	20·0	17·4	14·3	11·3	9·1	7·2
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$	8·45	2·49	15·3	13·4	11·0	8·7	6·9	5·5
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16}$	7·11	2·09	12·9	11·2	9·3	7·4	5·8	4·7
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{4}$	5·74	1·69	10·5	9·2	7·6	6·1	4·8	3·9

For notes relating to above see page xi

and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section Axes x—x		Radii of Gyration	
9	10	11	12	13	14	Max.	Min.	Axis v—v	Axis u—u
15·7	13·3	11·4	9·9	8·7	7·5	7·78	3·08	·98	1·92
12·0	10·3	8·8	7·7	6·6	5·8	6·23	2·35	·98	1·94
14·3	12·2	10·4	8·9	7·7	6·7	7·13	3·03	·87	1·70
11·9	10·2	8·5	7·4	6·3	5·6	6·14	2·47	·88	1·72
9·1	7·7	6·5	5·6	4·8	4·2	4·96	1·89	·88	1·74
7·8	6·6	5·6	4·8	4·2	3·6	4·27	1·59	·89	1·75
10·7	8·7	7·4	6·3	*5·4*	*4·7*	5·38	2·36	·77	1·50
8·6	7·2	6·1	5·1	*4·5*	*3·9*	4·67	1·93	·78	1·52
6·6	5·5	4·6	4·0	3·4	*3·0*	3·81	1·48	·78	1·54
5·6	4·6	3·9	3·4	2·9	*2·5*	3·28	1·24	·78	1·55
7·2	5·8	5·1	4·2	*3·7*		3·92	1·77	·68	1·30
5·9	4·8	4·1	*3·4*	*3·0*		3·40	1·46	·68	1·32
4·6	3·8	3·1	2·7	2·3		2·80	1·12	·68	1·34
3·8	3·1	2·5	2·2	*1·9*		2·45	·94	·68	1·35
3·2	2·6	2·2	*1·9*	*1·6*		2·04	·76	·69	1·35

See specially note on p. xiii regarding **Maximum** *Moduli*

STRUTS—Equal Angles
Safe Concentric Loads in Tons

Section	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE			
			2	3	4	5
3 × 3 × ½	9·35	2·75	18·2	15·9	12·9	9·9
3 × 3 × ⅜	7·17	2·11	13·9	12·2	9·9	7·7
3 × 3 × 5/16	6·04	1·78	11·8	10·4	8·5	6·5
3 × 3 × ¼	4·89	1·45	9·4	8·3	6·8	5·3
2½ × 2½ × ⅜	5·90	1·73	10·9	8·9	6·7	4·6
2½ × 2½ × 5/16	4·98	1·45	9·2	7·6	5·7	4·0
2½ × 2½ × ¼	4·05	1·19	7·5	6·1	4·6	3·3
2¼ × 2¼ × ⅜	5·26	1·56	9·5	7·1	5·1	3·5
2¼ × 2¼ × 5/16	4·45	1·31	7·9	6·1	4·3	3·0
2¼ × 2¼ × ¼	3·61	1·06	6·4	5·0	3·4	2·4
2¼ × 2¼ × 3/16	2·75	·81	4·9	3·8	2·7	1·9
2 × 2 × 5/16	3·92	1·15	6·6	4·7	3·1	2·1
2 × 2 × ¼	3·19	·94	5·4	3·8	2·5	1·7
2 × 2 × 3/16	2·43	·71	4·1	2·8	1·9	1·2

For notes relating to above see page xi

and Dimensions and Properties

HEIGHTS IN FEET				Moduli of Section		Radii of Gyration	
				Axes $x—x$			
6	7	8	9	Max.	Min.	Axis $v—v$	Axis $u—u$
7·6	5·8	4·7	3·6	2·37	1·05	·58	1·12
5·8	4·6	3·6	2·9	1·95	·81	·58	1·14
4·9	3·8	3·1	2·4	1·73	·68	·58	1·15
4·1	3·1	2·4	2·0	1·45	·55	·59	1·15
3·5	2·6	2·1	*1·6*	1·28	·55	·48	·94
2·9	2·2	1·7	*1·4*	1·14	·47	·48	·95
2·4	1·9	1·5	*1·1*	·97	·38	·49	·95
2·5	1·9	*1·5*		1·00	·44	·43	·84
2·2	1·6	*1·3*		·89	·37	·43	·85
1·8	1·4	*1·0*		·77	·30	·44	·85
1·3	1·0	*·83*		·61	·23	·44	·86
1·4	*1·1*			·66	·29	·38	·75
1·2	*·99*			·59	·24	·39	·75
·98	*·75*			·46	·18	·39	·76

See specially note on p. xiii regarding **Maximum** *Moduli*

STRUTS—Unequal Angles
Safe Concentric Loads in Tons

Section	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE					
			4	5	6	7	8	9
7 × 4 × ⅝	22·05	6·48	39·2	35·0	30·1	25·2	21·0	17·5
7 × 4 × ½	17·85	5·25	31·8	28·4	24·5	20·6	17·1	14·3
7 × 3½ × ⅝	20·99	6·18	34·7	29·6	24·3	19·5	15·8	13·0
7 × 3½ × ½	17·00	5·00	28·3	24·1	19·8	16·0	12·9	10·6
7 × 3½ × ⅜	12·91	3·80	21·6	18·4	15·1	12·2	9·9	8·2
6 × 4 × ⅝	19·93	5·86	35·5	31·6	27·2	22·8	18·9	15·8
6 × 4 × ½	16·16	4·75	28·8	25·7	22·2	18·7	15·5	12·9
6 × 3½ × ½	15·30	4·50	25·6	21·9	18·0	14·6	11·8	9·7
6 × 3½ × ⅜	11·63	3·42	19·6	16·8	13·8	11·2	9·1	7·5
6 × 3 × ½	14·45	4·25	21·7	17·3	13·5	10·5	8·5	6·8
6 × 3 × ⅜	11·00	3·24	16·6	13·3	10·3	8·1	6·6	5·2
6 × 3 × 5/16	9·24	2·72	13·8	11·2	8·8	6·9	5·5	4·4
5 × 4 × ⅝	17·80	5·24	31·4	27·9	23·7	19·8	16·4	13·7
5 × 4 × ½	14·45	4·25	25·5	22·7	19·4	16·2	13·4	11·1
5 × 4 × ⅜	11·00	3·24	19·5	17·3	14·9	12·5	10·3	8·6

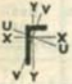
For notes relating to above see page xi

and Dimensions and Properties

HEIGHTS IN FEET				Maximum Moduli of Section		Minimum Moduli of Section		Radii of Gyration	
10	11	12	13	Axis x—x	Axis y—y	Axis x—x	Axis y—y	Axis v—v	Axis u—u
14·7	12·6	10·6	9·3	13·1	8·04	7·02	2·51	·86	2·32
12·0	10·2	8·7	7·6	11·0	7·03	5·70	2·04	·86	2·34
10·8	9·2	7·7	6·7	12·0	6·35	6·86	1·91	·74	2·29
8·9	7·4	6·3	*5·5*	10·0	5·62	5·57	1·56	·74	2·31
6·8	5·8	4·9	*4·3*	7·90	4·66	4·23	1·19	·75	2·32
13·2	11·4	9·6	8·4	10·3	7·23	5·23	2·48	·86	2·02
10·9	9·2	7·9	6·8	8·70	6·30	4·25	2·02	·86	2·04
8·1	6·8	5·8	*5·0*	7·94	5·04	4·15	1·54	·75	2·00
6·2	5·2	4·5	*3·8*	6·28	4·17	3·16	1·18	·76	2·01
5·7	4·7	*4·0*		7·15	3·85	4·05	1·13	·63	1·97
4·3	*3·6*	*3·1*		5·66	3·25	3·09	·87	·64	1·98
3·7	*3·1*	*2·6*		4·85	2·85	2·59	·73	·64	1·99
11·4	9·7	8·2	7·1	7·73	6·32	3·67	2·43	·84	1·74
9·3	7·9	6·8	5·8	6·60	5·50	2·99	1·98	·84	1·76
7·2	6·1	5·2	4·5	5·28	4·49	2·28	1·52	·85	1·77

See specially note on p. xiii regarding **Maximum** *Moduli*

STRUTS—Unequal Angles
Safe Concentric Loads in Tons

 Section	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
			3	4	5	6	7
5 × 4 × 5/16	9·23	2·72	17·8	16·4	14·6	12·5	10·5
5 × 3½ × ½	13·61	4·00	25·5	22·7	19·4	15·9	12·9
5 × 3½ × 3/8	10·37	3·05	19·4	17·4	14·9	12·2	9·9
5 × 3½ × 5/16	8·71	2·56	16·3	14·7	12·6	10·4	8·5
5 × 3 × ½	12·75	3·76	22·6	19·4	15·4	12·1	9·5
5 × 3 × 3/8	9·73	2·86	17·3	14·8	11·9	9·3	7·3
5 × 3 × 5/16	8·17	2·40	14·6	12·5	10·1	7·9	6·1
5 × 3 × ¼	6·59	1·94	11·8	10·1	8·2	6·4	5·0
4½ × 3 × ½	11·92	3·50	21·1	18·1	14·4	11·2	8·8
4½ × 3 × 3/8	9·09	2·66	16·2	13·8	11·1	8·7	6·8
4½ × 3 × 5/16	7·64	2·25	13·6	11·6	9·4	7·3	5·8
4½ × 3 × ¼	6·16	1·81	11·0	9·4	7·6	6·0	4·7
4 × 3½ × 5/8	14·61	4·30	26·9	23·8	19·9	16·1	12·9
4 × 3½ × ½	11·92	3·50	22·0	19·4	16·4	13·3	10·6
4 × 3½ × 3/8	9·09	2·67	16·8	14·9	12·5	10·1	8·1

For notes relating to above see page xi

and Dimensions and Properties

HEIGHTS IN FEET				Maximum Moduli of Section		Minimum Moduli of Section		Radii of Gyration	
8	9	10	11	Axis x—x	Axis y—y	Axis x—x	Axis y—y	Axis v—v	Axis u—u
8·7	7·3	6·1	5·2	4·55	3·87	1·92	1·28	·85	1·77
10·5	8·6	7·3	6·0	6·00	4·40	2·93	1·52	·75	1·70
8·1	6·6	5·5	4·7	4·80	3·64	2·24	1·16	·75	1·72
6·9	5·7	4·7	4·0	4·15	3·20	1·88	·98	·76	1·72
7·5	6·1	5·0	4·2	5·40	3·39	2·86	1·11	·64	1·66
5·8	4·7	3·9	3·3	4·32	2·86	2·18	·85	·65	1·68
4·9	4·0	3·3	2·8	3·70	2·50	1·84	·72	·65	1·68
4·0	3·3	2·7	2·3	3·06	2·10	1·48	·58	·65	1·69
7·0	5·7	4·7	3·9	4·57	3·14	2·33	1·10	·64	1·51
5·3	4·4	3·7	3·0	3·68	2·63	1·79	·85	·64	1·53
4·6	3·7	3·1	2·6	3·19	2·34	1·50	·71	·65	1·53
3·7	3·0	2·5	2·1	2·63	1·96	1·21	·57	·65	1·54
10·4	8·5	7·0	5·9	4·87	4·28	2·31	1·81	·71	1·41
8·6	7·0	5·8	4·8	4·23	3·76	1·90	1·48	·72	1·43
6·6	5·4	4·5	3·7	3·44	3·10	1·45	1·14	·72	1·45

See specially note on p. xiii regarding **Maximum** *Moduli*

STRUTS—Unequal Angles
Safe Concentric Loads in Tons

Section	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE			
			3	4	5	6
4 × 3½ × 5/16	7·64	2·26	14·1	12·5	10·6	8·6
4 × 3½ × ¼	6·16	1·81	11·4	10·2	8·6	7·0
4 × 3 × ½	11·06	3·25	19·5	16·5	13·3	10·3
4 × 3 × ⅜	8·45	2·49	15·1	12·7	10·1	7·9
4 × 3 × 5/16	7·11	2·09	12·6	10·7	8·6	6·7
4 × 3 × ¼	5·74	1·69	10·2	8·7	7·0	5·5
4 × 2½ × ⅜	7·81	2·30	12·7	9·9	7·5	5·5
4 × 2½ × 5/16	6·58	1·93	10·7	8·4	6·3	4·7
4 × 2½ × ¼	5·33	1·56	8·7	6·8	5·1	3·8
3½ × 3 × ½	10·20	3·00	17·8	14·9	11·7	9·1
3½ × 3 × ⅜	7·82	2·30	13·7	11·5	9·0	7·0
3½ × 3 × 5/16	6·58	1·94	11·5	9·7	7·6	5·9
3½ × 3 × ¼	5·32	1·56	9·3	7·8	6·2	4·8
3½ × 2½ × ⅜	7·17	2·11	11·6	9·0	6·7	5·0
3½ × 2½ × 5/16	6·05	1·78	9·8	7·6	5·7	4·2

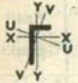
For notes relating to above see page xi

and Dimensions and Properties

HEIGHTS IN FEET				Maximum Moduli of Section		Minimum Moduli of Section		Radii of Gyration	
7	8	9	10	Axis x—x	Axis y—y	Axis x—x	Axis y—y	Axis v—v	Axis u—u
6·9	5·5	4·5	3·8	2·99	2·69	1·22	·96	·72	1·46
5·7	4·6	3·7	3·1	2·47	2·26	·99	·77	·73	1·46
8·0	6·4	5·2	4·3	3·77	2·89	1·85	1·09	·63	1·36
6·2	4·8	4·0	3·3	3·06	2·43	1·42	·84	·64	1·38
5·2	4·2	3·4	2·8	2·66	2·12	1·20	·71	·64	1·39
4·3	3·5	2·8	2·3	2·22	1·81	·96	·57	·64	1·40
4·2	3·4	2·7	2·2	2·69	1·80	1·38	·58	·53	1·34
3·6	2·8	2·3	*1·8*	2·34	1·59	1·17	·49	·54	1·34
2·9	2·3	*1·8*	*1·5*	1·95	1·38	·94	·40	·54	1·35
7·2	5·6	4·6	3·7	3·04	2·62	1·43	1·07	·61	1·23
5·5	4·4	3·5	2·9	2·50	2·20	1·10	·83	·62	1·25
4·6	3·8	3·0	2·4	2·18	1·95	·92	·70	·62	1·26
3·8	3·0	2·4	2·0	1·84	1·64	·75	·56	·62	1·27
3·8	3·0	*2·4*	*2·0*	2·18	1·63	1·07	·57	·53	1·19
3·2	2·5	*2·0*	*1·7*	1·91	1·45	·90	·48	·53	1·20

See specially note on p. xiii regarding **Maximum** *Moduli*

STRUTS—Unequal Angles
Safe Concentric Loads in Tons

 Section	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE HEIG 2	3	4	5
$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	4·89	1·44	9·3	7·9	6·2	4·6
$3 \times 2\frac{1}{2} \times \frac{3}{8}$	6·54	1·92	12·3	10·4	8·0	5·9
$3 \times 2\frac{1}{2} \times \frac{5}{16}$	5·51	1·62	10·5	8·8	6·7	5·0
$3 \times 2\frac{1}{2} \times \frac{1}{4}$	4·47	1·31	8·4	7·1	5·5	4·0
$3 \times 2 \times \frac{3}{8}$	5·90	1·73	10·4	7·9	5·5	3·8
$3 \times 2 \times \frac{5}{16}$	4·98	1·46	8·8	6·7	4·6	3·2
$3 \times 2 \times \frac{1}{4}$	4·04	1·19	7·1	5·5	3·8	2·6
$2\frac{1}{2} \times 2 \times \frac{3}{8}$	5·26	1·55	9·2	6·9	4·7	3·3
$2\frac{1}{2} \times 2 \times \frac{5}{16}$	4·45	1·31	7·8	5·9	4·1	2·8
$2\frac{1}{2} \times 2 \times \frac{1}{4}$	3·60	1·06	6·2	4·8	3·3	2·3
$2\frac{1}{2} \times 2 \times \frac{3}{16}$	2·76	·81	4·8	3·6	2·5	1·7
$2\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$	3·19	·94	4·7	2·9	1·8	1·2
$2\frac{1}{2} \times 1\frac{1}{2} \times \frac{3}{16}$	2·43	·71	3·6	2·1	1·5	·96
$2 \times 1\frac{1}{2} \times \frac{1}{4}$	2·76	·81	4·1	2·5	1·6	1·0
$2 \times 1\frac{1}{2} \times \frac{3}{16}$	2·12	·63	3·1	1·9	1·2	·82

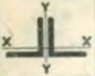
For notes relating to above see page xi

and Dimensions and Properties

HTS IN FEET			Maximum Moduli of Section		Minimum Moduli of Section		Radii of Gyration	
6	7	8	Axis x—x	Axis y—y	Axis x—x	Axis y—y	Axis v—v	Axis u—u
3·4	2·6	2·0	1·60	1·23	·73	·39	·54	1·20
4·4	3·3	2·6	1·72	1·46	·79	·56	·52	1·05
3·7	2·8	2·2	1·51	1·30	·67	·48	·52	1·06
3·0	2·3	1·8	1·28	1·10	·54	·39	·52	1·07
2·8	2·1	*1·6*	1·46	1·00	·76	·36	·42	1·00
2·3	1·8	*1·4*	1·29	·88	·65	·30	·43	1·00
1·9	1·4	*1·1*	1·08	·79	·52	·25	·43	1·01
2·5	*1·8*	*1·4*	1·09	·86	·53	·35	·41	·85
2·0	*1·5*	*1·2*	·96	·78	·45	·30	·42	·86
1·6	*1·2*	*1·0*	·82	·68	·37	·24	·42	·87
1·2	·97	*·76*	·65	·56	·28	·18	·42	·88
·91			·68	·41	·35	·14	·32	·82
·69			·54	·35	·27	·10	·32	·83
·74			·48	·37	·23	·13	·31	·68
·61			·38	·29	·17	·10	·32	·68

See specially note on p. xiii regarding **Maximum** *Moduli*

COMPOUND STRUTS—
Safe Concentric Loads in Tons



Two equal angles	Weight per foot in lbs.	Area in sq. ins.	Space between angles ins.	EFFECTIVE			
				8	10	12	14
8 × 8 × 1	102·0	30·00	$\frac{3}{4}$	200	189	177	163
8 × 8 × $\frac{7}{8}$	90·0	26·48	$\frac{3}{4}$	176	167	156	144
8 × 8 × $\frac{3}{4}$	77·8	22·88	$\frac{3}{4}$	153	144	135	125
8 × 8 × $\frac{5}{8}$	65·4	19·22	$\frac{3}{4}$	128	121	114	105
7 × 7 × $\frac{7}{8}$	78·2	22·97	$\frac{5}{8}$	149	138	126	112
7 × 7 × $\frac{3}{4}$	67·6	19·89	$\frac{5}{8}$	129	120	109	98·2
7 × 7 × $\frac{5}{8}$	56·8	16·72	$\frac{5}{8}$	108	101	92·9	83·1
7 × 7 × $\frac{1}{2}$	45·9	13·50	$\frac{5}{8}$	87·7	82·0	75·3	67·4
6 × 6 × $\frac{7}{8}$	66·2	19·48	$\frac{5}{8}$	120	108	94·9	80·7
6 × 6 × $\frac{3}{4}$	57·4	16·87	$\frac{5}{8}$	104	94·4	82·6	70·6
6 × 6 × $\frac{5}{8}$	48·3	14·23	$\frac{5}{8}$	88·1	79·9	70·0	59·8
6 × 6 × $\frac{1}{2}$	39·1	11·50	$\frac{5}{8}$	71·4	64·9	57·2	48·9
6 × 6 × $\frac{3}{8}$	29·7	8·73	$\frac{5}{8}$	54·3	49·3	43·5	37·3

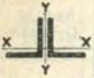
For notes relating to above see page xi

Two Equal Angles and Dimensions and Properties

HEIGHTS IN FEET								Radii of Gyration	
16	18	20	22	24	28	32	36	Axis y—y	Axis x—x
147	131	115	101	89·0	69·3	55·1	44·5	3·64	2·42
130	116	102	90·2	79·2	61·7	49·0	39·7	3·62	2·43
113	101	89·4	78·6	69·1	53·8	42·7	34·7	3·59	2·45
95·9	85·6	75·7	66·6	58·5	45·6	36·2	29·3	3·57	2·46
98·7	85·1	73·3	63·3	54·9	42·2	33·2	26·7	3·18	2·12
85·9	74·3	64·1	54·7	48·0	36·8	29·1	*23·3*	3·15	2·13
72·8	63·1	54·4	47·0	40·9	31·3	24·7	*19·7*	3·12	2·14
59·2	51·3	44·3	38·3	33·3	25·6	20·1	*16·3*	3·10	2·16
67·8	56·9	48·2	40·9	35·1	26·6	*20·8*	*16·8*	2·78	1·80
59·4	49·8	42·0	35·8	30·9	23·3	*18·1*	*14·6*	2·75	1·81
50·3	42·3	35·7	30·4	26·2	19·9	*15·6*	*12·7*	2·73	1·83
41·4	34·8	29·4	25·1	21·7	16·4	*12·8*	*10·3*	2·70	1·84
31·5	26·6	22·6	19·1	16·5	12·6	*9·8*	*7·9*	2·67	1·85

See note on page xiii regarding ratio of slenderness

COMPOUND STRUTS—
Safe Concentric Loads in Tons

 Two equal angles	Weight per foot in lbs.	Area in sq. ins.	Space between angles ins.	EFFECTIVE			
				4	5	6	7
5 × 5 × ¾	47·3	13·88	½	95·2	91·9	88·2	83·9
5 × 5 × ⅝	39·9	11·72	½	80·6	77·9	74·8	71·2
5 × 5 × ½	32·3	9·50	½	65·3	63·2	60·7	57·8
5 × 5 × ⅜	24·7	7·22	½	49·7	48·1	46·2	44·1
4½ × 4½ × ¾	42·1	12·39	½	83·6	80·2	76·1	71·3
4½ × 4½ × ⅝	35·7	10·47	½	70·9	68·1	64·6	60·6
4½ × 4½ × ½	28·9	8·50	½	57·6	55·3	52·6	49·5
4½ × 4½ × ⅜	22·0	6·46	½	43·9	42·1	40·2	37·8
4 × 4 × ⅝	31·5	9·22	½	61·1	57·9	54·0	49·6
4 × 4 × ½	25·5	7·50	½	49·7	47·2	44·2	40·7
4 × 4 × ⅜	19·5	5·72	½	38·0	36·1	33·8	31·1
3½ × 3½ × ⅝	27·1	7·97	⅜	51·1	47·6	43·1	38·1
3½ × 3½ × ½	22·2	6·50	⅜	41·8	38·9	35·4	31·4
3½ × 3½ × ⅜	16·9	4·97	⅜	32·1	30·1	27·4	24·4
3½ × 3½ × $\frac{5}{16}$	14·2	4·18	⅜	27·0	25·2	23·0	20·5

For notes relating to above see page xi

Two Equal Angles and Dimensions and Properties

HEIGHTS IN FEET								Radii of Gyration	
8	10	12	14	16	18	20	22	Axis y—y	Axis x—x
78·9	67·3	55·3	44·6	36·2	29·7	24·8	20·8	2·31	**1·50**
67·1	57·4	47·4	38·4	31·2	25·7	21·4	18·1	2·29	**1·51**
54·5	46·9	38·8	31·5	25·6	21·1	17·6	14·8	2·26	**1·52**
41·7	35·8	29·6	24·2	19·8	16·2	13·6	11·4	2·23	**1·54**
65·8	53·8	42·6	33·8	27·0	22·1	18·2	15·4	2·12	**1·34**
56·1	46·0	36·6	29·0	23·2	18·9	15·7	13·1	2·09	**1·35**
45·8	37·7	30·1	23·9	19·2	15·6	12·9	10·9	2·06	**1·37**
35·2	29·0	23·2	18·5	14·8	12·1	10·1	8·4	2·03	**1·38**
44·6	34·7	26·7	20·6	16·4	13·2	*10·9*	*9·1*	1·89	**1·19**
36·5	28·5	21·9	17·0	13·5	10·9	9·0	7·7	1·86	**1·21**
28·1	22·0	16·8	13·2	10·4	8·4	7·0	*6·0*	1·84	**1·22**
33·1	24·4	18·1	13·9	10·9	*8·8*	7·2		1·65	**1·03**
27·3	20·2	15·0	11·6	9·0	*7·4*	*6·0*		1·62	**1·05**
21·3	15·9	11·9	9·1	7·1	*5·8*	*4·7*		1·59	**1·06**
17·9	13·3	10·0	7·6	6·0	*4·9*	*4·0*		1·58	**1·07**

See note on page xiii regarding ratio of slenderness

COMPOUND STRUTS—
Safe Concentric Loads in Tons

Two equal angles	Weight per foot in lbs.	Area in sq. ins.	Space between angles ins.	EFFECTIVE			
				2	3	4	5
$3 \times 3 \times \frac{1}{2}$	18·8	5·50	$\frac{3}{8}$	38·4	36·3	33·6	30·2
$3 \times 3 \times \frac{3}{8}$	14·3	4·22	$\frac{3}{8}$	29·5	28·0	26·0	23·5
$3 \times 3 \times \frac{5}{16}$	12·1	3·56	$\frac{3}{8}$	24·8	23·5	21·9	19·8
$3 \times 3 \times \frac{1}{4}$	9·9	2·88	$\frac{3}{8}$	20·1	19·1	17·8	16·1
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$	11·8	3·47	$\frac{3}{8}$	23·7	22·1	19·6	16·8
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$	10·0	2·93	$\frac{3}{8}$	20·1	18·6	16·7	14·3
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	8·2	2·38	$\frac{3}{8}$	16·3	15·1	13·6	11·6
$2\frac{1}{4} \times 2\frac{1}{4} \times \frac{3}{8}$	10·5	3·09	$\frac{3}{8}$	20·9	19·0	16·4	13·3
$2\frac{1}{4} \times 2\frac{1}{4} \times \frac{5}{16}$	8·9	2·62	$\frac{3}{8}$	17·7	16·2	13·9	11·4
$2\frac{1}{4} \times 2\frac{1}{4} \times \frac{1}{4}$	7·2	2·13	$\frac{3}{8}$	14·4	13·1	11·3	9·2
$2\frac{1}{4} \times 2\frac{1}{4} \times \frac{3}{16}$	5·6	1·62	$\frac{3}{8}$	10·9	10·0	8·7	7·1
$2 \times 2 \times \frac{5}{16}$	7·8	2·30	$\frac{3}{8}$	15·2	13·4	11·0	8·6
$2 \times 2 \times \frac{1}{4}$	6·4	1·87	$\frac{3}{8}$	12·3	10·9	8·9	6·8
$2 \times 2 \times \frac{3}{16}$	4·9	1·43	$\frac{3}{8}$	9·5	8·5	7·0	5·5

For notes relating to above see page xi

Two Equal Angles and Dimensions and Properties

HEIGHTS IN FEET								Radii of Gyration	
6	7	8	9	10	11	12	14	Axis y—y	Axis x—x
26·2	22·2	18·6	15·5	13·0	11·1	9·6	7·2	1·42	·89
20·5	17·5	14·7	12·3	10·4	8·8	7·6	5·7	1·40	·90
17·4	14·7	12·3	10·4	8·7	7·4	6·4	4·8	1·38	·91
14·1	12·0	10·1	8·6	7·2	6·1	5·4	4·0	1·37	·91
13·7	11·0	8·9	7·3	6·1	5·1	4·4	3·3	1·20	·74
11·8	9·5	7·7	6·3	5·3	4·4	3·8	2·9	1·18	·75
9·5	7·7	6·2	5·2	4·2	3·6	3·1	2·4	1·17	·76
10·5	8·3	6·7	5·4	4·5	3·7	3·2		1·10	·67
9·0	7·2	5·7	4·6	3·8	3·3	2·8		1·09	·67
7·3	5·8	4·7	3·7	3·1	2·6	2·3		1·07	·68
5·7	4·5	3·6	2·8	2·4	2·1	1·8		1·05	·68
6·5	5·0	4·0	3·2	2·7	2·3			·99	·59
5·3	4·1	3·2	2·6	2·3	1·9	1·6		·97	·60
4·2	3·3	2·7	2·1	1·7	1·4	1·2		·96	·60

See note on page xiii regarding ratio of slenderness

COMPOUND STRUTS — Two
Safe Concentric Loads in Tons

 Two unequal angles	Weight per foot in lbs.	Area in sq. ins.	Space between angles ins.	EFFECTIVE			
				4	5	6	8
7 × 3½ × ⅝	42·0	12·35	½	84·0	80·8	77·0	67·8
7 × 3½ × ½	34·0	10·00	½	67·9	65·1	62·0	54·1
7 × 3½ × ⅜	25·8	7·59	½	51·3	49·2	46·7	40·6
6 × 4 × ¾	47·2	13·88	⅝	97·1	94·6	91·9	85·2
6 × 4 × ⅝	39·9	11·72	⅝	81·8	79·7	77·3	71·6
6 × 4 × ½	32·4	9·51	⅝	66·3	64·5	62·5	57·6
6 × 3½ × ⅝	37·7	11·09	½	76·0	73·3	70·2	62·5
6 × 3½ × ½	30·7	9·00	½	61·5	59·2	56·7	50·2
6 × 3½ × ⅜	23·3	6·84	½	46·6	44·9	42·8	37·8
6 × 3 × ⅝	35·6	10·47	½	70·0	66·7	62·8	52·9
6 × 3 × ½	28·9	8·50	½	56·7	53·8	50·4	42·1
6 × 3 × ⅜	22·1	6·47	½	42·9	40·6	37·9	31·4
5 × 4 × ⅝	35·6	10·48	½	72·1	69·8	67·1	60·6
5 × 4 × ½	28·9	8·50	½	58·6	56·7	54·6	49·4
5 × 4 × ⅜	22·0	6·47	½	44·7	43·4	41·7	37·8

For notes relating to above see page xi

Unequal Angles—long legs connected and Dimensions and Properties

HEIGHTS IN FEET								Radii of Gyration	
10	12	14	16	18	20	22	24	Axis y—y	Axis x—x
56·4	45·4	36·2	29·2	23·8	19·8	16·6	*14·1*	1·40	2·22
44·7	35·7	28·5	22·9	18·6	15·5	12·9	*11·0*	1·37	2·24
33·1	26·4	20·8	16·7	13·6	11·2	9·4	*8·0*	1·34	2·25
76·7	66·7	56·6	47·5	39·7	33·4	28·4	24·5	1·77	1·87
64·2	55·6	46·8	39·1	32·7	27·5	23·3	20·0	1·74	1·88
51·4	44·3	37·2	30·9	25·8	21·8	18·4	15·8	1·72	1·90
53·0	43·3	34·9	28·3	23·2	19·3	16·2	13·8	1·47	1·89
42·2	34·3	27·5	22·2	18·3	15·1	12·7	*10·8*	1·44	1·91
31·4	25·4	20·2	16·4	13·3	11·1	9·3	*7·9*	1·41	1·92
42·0	32·6	25·5	20·3	16·6	13·7	*11·4*	*9·6*	1·25	1·89
33·3	25·5	19·9	15·8	12·8	10·6	*8·8*	*7·5*	1·22	1·91
24·4	18·7	14·6	11·6	9·3	*7·6*	*6·4*		1·19	1·93
52·2	43·4	35·4	28·8	23·7	19·8	16·7	14·3	1·79	1·54
42·8	35·6	29·1	23·8	19·6	16·4	13·9	11·8	1·76	1·56
32·9	27·4	22·5	18·4	15·2	12·7	10·7	9·1	1·73	1·57

See note on page xiii regarding ratio of slenderness

COMPOUND STRUTS — Two Safe Concentric Loads in Tons

 Two unequal angles	Weight per foot in lbs.	Area in sq. ins.	Space between angles in ins.	EFFECTIVE			
				4	5	6	7
5 × 3½ × ⅝	33·5	9·85	½	67·9	65·7	63·2	60·4
5 × 3½ × ½	27·3	8·01	½	55·1	53·2	51·1	48·7
5 × 3½ × ⅜	20·7	6·10	½	41·8	40·4	38·8	36·9
5 × 3½ × 5/16	17·4	5·12	½	35·1	33·8	32·4	30·8
5 × 3 × ½	25·6	7·50	⅜	50·1	47·7	44·8	41·5
5 × 3 × ⅜	19·5	5·73	⅜	38·0	36·1	33·9	31·1
5 × 3 × 5/16	16·3	4·81	⅜	31·9	30·2	28·3	25·9
5 × 3 × ¼	13·2	3·88	⅜	25·8	24·3	22·6	20·8
4½ × 3 × ½	23·8	7·00	⅜	47·0	44·9	42·3	39·4
4½ × 3 × ⅜	18·2	5·35	⅜	35·7	34·0	32·0	29·7
4½ × 3 × 5/16	15·3	4·50	⅜	30·0	28·6	26·8	24·8
4½ × 3 × ¼	12·3	3·62	⅜	24·1	22·0	21·4	19·7
4 × 3½ × ½	23·8	7·00	⅜	46·6	44·3	41·5	38·3
4 × 3½ × ⅜	18·2	5·35	⅜	35·6	33·9	31·8	29·5
4 × 3½ × 5/16	15·3	4·50	⅜	30·1	28·6	26·8	24·8

For notes relating to above see page xi

Unequal Angles—long legs connected and Dimensions and Properties

HEIGHTS IN FEET								Radii of Gyration	
8	10	12	14	16	18	20	22	Axis y—y	Axis x—x
57·1	49·3	41·0	33·6	27·3	22·5	18·7	15·8	1·55	1·55
45·9	39·4	32·6	26·5	21·6	17·7	14·8	12·4	1·52	1·57
34·7	29·5	24·2	19·6	15·9	13·1	10·9	9·2	1·49	1·58
28·8	24·6	20·0	16·1	13·1	10·8	8·9	7·5	1·47	1·59
37·6	29·8	23·1	18·0	14·4	11·6	9·6	8·0	1·24	1·58
28·1	22·0	17·0	13·3	10·5	8·5	7·1	5·8	1·21	1·59
23·4	18·3	14·0	10·9	8·6	7·0	5·7	4·8	1·20	1·60
18·6	14·4	11·1	8·6	6·8	5·5	4·5	3·8	1·18	1·60
36·0	28·8	22·5	17·7	14·1	11·5	9·5	7·9	1·28	1·41
27·0	21·4	16·6	13·0	10·4	8·4	6·9	5·8	1·25	1·42
22·5	17·8	13·7	10·8	8·6	6·9	5·8	4·7	1·23	1·43
17·8	14·0	10·8	8·4	6·6	5·4	4·5	3·7	1·21	1·43
34·7	27·3	21·1	16·4	13·1	10·6	8·7	7·3	1·56	1·22
26·7	21·1	16·4	12·8	10·1	8·2	6·9	5·7	1·54	1·24
22·5	17·8	13·8	10·8	8·6	7·1	5·8	4·8	1·52	1·24

See note on page xiii regarding ratio of slenderness

COMPOUND STRUTS — Two Safe Concentric Loads in Tons

Two unequal angles	Weight per foot in lbs.	Area in sq. ins.	Space between angles in ins.	EFFECTIVE			
				3	4	5	6
4 × 3½ × ¼	12·3	3·62	⅜	25·2	24·2	23·0	21·6
4 × 3 × ½	22·1	6·50	⅜	45·2	43·4	41·3	38·9
4 × 3 × ⅜	16·9	4·97	⅜	34·6	33·2	31·8	29·8
4 × 3 × 5/16	14·2	4·18	⅜	29·1	28·0	26·7	25·1
4 × 3 × ¼	11·5	3·38	⅜	23·6	22·6	21·6	20·3
4 × 2½ × ⅜	15·6	4·60	⅜	31·4	29·7	27·7	25·4
4 × 2½ × 5/16	13·2	3·87	⅜	26·3	24·8	23·2	21·1
4 × 2½ × ¼	10·7	3·13	⅜	21·2	20·1	18·6	16·9
3½ × 3 × ½	20·5	6·00	⅜	40·9	38·8	36·2	33·1
3½ × 3 × ⅜	15·6	4·60	⅜	31·4	29·8	27·9	25·6
3½ × 3 × 5/16	13·3	3·87	⅜	26·4	25·1	23·5	21·6
3½ × 3 × ¼	11·0	3·13	⅜	21·4	20·3	19·0	17·5
3½ × 2½ × ⅜	14·4	4·22	⅜	28·9	27·5	25·7	23·7
3½ × 2½ × 5/16	12·1	3·56	⅜	24·4	23·1	21·6	19·9
3½ × 2½ × ¼	9·8	2·88	⅜	19·6	18·6	17·4	15·8

For notes relating to above see page xi

Unequal Angles—long legs connected and Dimensions and Properties

HEIGHTS IN FEET								Radii of Gyration	
7	8	9	10	12	14	16	18	Axis y—y	Axis x—x
20·0	18·2	16·2	14·3	11·2	8·7	6·9	5·7	1·50	**1·24**
35·9	32·6	29·2	25·8	20·0	15·6	12·5	10·1	1·32	**1·24**
27·6	25·1	22·5	20·1	15·6	12·1	9·7	7·8	1·29	**1·25**
23·4	21·2	19·0	16·9	13·1	10·3	8·2	6·7	1·28	**1·26**
18·9	17·2	15·4	13·7	10·7	8·3	6·7	5·4	**1·26**	**1·26**
22·5	19·7	17·0	14·6	11·0	8·6	6·6	*5·3*	**1·06**	1·26
18·7	16·3	14·1	12·1	9·1	6·9	5·4	*4·3*	**1·04**	1·27
14·9	12·9	11·0	9·6	7·1	5·4	4·3	*3·4*	**1·03**	1·27
29·7	25·9	22·4	19·4	14·4	11·1	8·7	*7·0*	1·37	**1·06**
22·9	20·1	17·5	15·1	11·3	8·8	6·8	*5·5*	1·34	**1·08**
19·3	17·1	14·9	12·7	9·6	7·3	5·8	4·7	1·33	**1·08**
15·7	13·8	12·0	10·4	7·8	6·0	4·7	3·8	1·31	**1·09**
21·3	18·7	16·3	14·1	10·7	8·3	6·5	5·3	1·10	**1·09**
17·8	15·7	13·6	11·7	8·8	6·8	5·3	*4·3*	**1·08**	1·10
14·2	12·6	10·9	9·3	7·0	5·4	4·2	*3·4*	**1·07**	1·10

See note on page xiii regarding ratio of slenderness

COMPOUND STRUTS — Two Safe Concentric Loads in Tons

 Two unequal angles	Weight per foot in lbs.	Area in sq. ins.	Space between angles ins.	EFFECTIVE 2	3	4
3 × 2½ × ⅜	13·1	3·85	⅜	27·2	25·6	23·8
3 × 2½ × 5/16	11·0	3·24	⅜	22·7	21·6	20·1
3 × 2½ × ¼	8·8	2·63	⅜	18·4	17·6	16·4
3 × 2 × ⅜	11·8	3·48	⅜	24·3	23·1	21·6
3 × 2 × 5/16	10·0	2·93	⅜	20·6	19·4	18·0
3 × 2 × ¼	8·1	2·38	⅜	16·6	15·7	14·5
2½ × 2 × ⅜	10·5	3·09	5/16	21·2	19·8	17·6
2½ × 2 × 5/16	8·9	2·62	5/16	18·1	16·7	15·0
2½ × 2 × ¼	7·2	2·13	5/16	14·6	13·6	12·3
2½ × 2 × 3/16	5·5	1·62	5/16	11·3	10·4	9·4
2½ × 1½ × ¼	6·4	1·88	5/16	12·7	11·5	9·9
2½ × 1½ × 3/16	4·9	1·43	5/16	9·6	8·7	7·4
2 × 1½ × ¼	5·6	1·63	5/16	10·9	9·7	8·1
2 × 1½ × 3/16	4·2	1·24	5/16	8·3	7·5	6·2

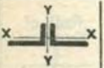
For notes relating to above see page xi

Unequal Angles—long legs connected and Dimensions and Properties

HEIGHTS IN FEET								Radii of Gyration	
5	6	7	8	9	10	12	14	Axis $y—y$	Axis $x—x$
21·6	19·1	16·3	13·8	11·6	9·8	7·2	5·4	1·14	·92
18·3	16·2	13·9	11·7	9·8	8·3	6·1	4·6	1·13	·93
14·9	13·1	11·4	9·6	8·1	6·8	5·0	3·9	1·11	·93
19·4	17·1	14·6	12·2	10·3	8·7	6·5	4·9	·91	·93
16·2	14·2	12·0	10·1	8·4	7·2	5·2	3·9	·89	·94
13·0	11·3	9·6	8·0	6·7	5·6	4·1	3·1	·87	·94
15·2	12·6	10·2	8·3	6·8	5·7	4·1	*3·1*	·93	·76
13·0	10·7	8·8	7·1	5·9	4·8	3·6	2·6	·91	·77
10·6	8·9	7·2	5·8	4·8	4·0	2·9	*2·1*	·90	·77
8·3	6·8	5·6	4·6	3·8	3·2	2·3	*1·7*	·88	·78
8·1	6·4	5·1	4·0	3·3	2·7	*1·9*		·66	·78
6·0	4·7	3·7	2·9	2·4	2·0	*1·4*		·64	·79
6·4	4·9	3·8	3·1	2·5	2·1	*1·4*		·70	·61
4·9	3·8	3·1	2·4	1·8	1·6	*1·1*		·69	·62

See note on page xiii regarding ratio of slenderness

COMPOUND STRUTS — Two Safe Concentric Loads in Tons



Two unequal angles	Weight per foot in lbs.	Area in sq. ins.	Space be-tween angles ins.	EFFECTIVE		
				4	5	6
7 × 3½ × ⅝	42·0	12·34	½	76·4	69·3	60·9
7 × 3½ × ½	34·0	10·00	½	62·2	56·5	49·8
7 × 3½ × ⅜	25·8	7·59	½	47·4	43·2	38·3
6 × 4 × ¾	47·2	13·88	⅝	90·6	85·2	78·6
6 × 4 × ⅝	39·8	11·72	⅝	76·7	72·3	66·7
6 × 4 × ½	32·3	9·50	⅝	62·3	58·7	54·4
6 × 3½ × ⅝	37·7	11·09	½	69·5	63·5	56·3
6 × 3½ × ½	30·6	9·00	½	56·6	51·9	46·2
6 × 3½ × ⅜	23·3	6·84	½	43·2	39·7	35·5
6 × 3 × ⅝	35·6	10·47	½	60·6	52·4	43·7
6 × 3 × ½	28·9	8·50	½	49·6	43·1	36·0
6 × 3 × ⅜	22·1	6·48	½	38·1	33·1	27·8
5 × 4 × ⅝	35·6	10·47	½	69·0	65·1	60·6
5 × 4 × ½	28·9	8·50	½	56·1	53·1	49·4
5 × 4 × ⅜	22·0	6·47	½	42·8	40·6	37·8

For notes relating to above see page xi

Unequal Angles—short legs connected and Dimensions and Properties

HEIGHTS IN FEET								Radii of Gyration	
7	8	9	10	12	14	16	18	Axis y—y	Axis x—x
51·9	43·8	36·9	31·2	22·8	17·4	*13·5*	*10·9*	3·58	·91
42·6	36·1	30·4	25·7	18·9	14·3	*11·1*	8·9	3·55	·92
32·8	27·8	23·4	19·9	14·6	11·1	8·6	6·9	3·51	·93
70·8	62·7	54·6	47·4	35·8	27·7	21·7	17·5	3·02	1·11
60·4	53·5	46·9	40·7	30·7	23·7	18·8	15·1	3·00	1·12
49·3	43·9	38·5	33·6	25·4	19·6	15·5	12·6	2·97	1·13
48·7	41·3	34·8	29·7	21·7	16·6	*13·0*	*10·4*	3·03	·95
40·1	34·1	28·9	24·6	18·1	13·7	*10·8*	*8·6*	3·00	·96
30·8	26·3	22·4	19·0	14·0	10·7	8·4	6·7	2·96	·97
35·6	29·0	23·9	19·8	14·3	*10·8*			3·11	·77
29·5	24·1	19·9	16·6	12·1	*9·0*			3·08	·78
22·9	18·8	15·5	12·9	9·3	*7·0*			3·05	·80
55·2	49·3	43·5	37·9	28·9	22·4	17·7	14·4	2·41	1·16
45·2	40·6	35·8	31·3	23·9	18·5	14·7	11·8	2·39	1·17
34·6	31·1	27 6	24·2	18·5	14·3	11·4	9·2	2·36	1·18

See note on page xiii regarding ratio of slenderness

COMPOUND STRUTS — Two Safe Concentric Loads in Tons

 Two unequal angles	Weight per foot in lbs.	Area in sq. ins.	Space between angles ins.	EFFECTIVE			
				3	4	5	6
5 × 3½ × ⅝	33·5	9·85	½	66·4	62·4	57·4	51·5
5 × 3½ × ½	27·2	8·00	½	54·0	50·8	46·9	42·3
5 × 3½ × ⅜	20·7	6·10	½	41·3	38·9	36·0	32·4
5 × 3½ × 5/16	17·4	5·12	½	34·7	32·8	30·3	27·3
5 × 3 × ½	25·6	7·50	⅜	48·8	44·6	39·3	33·3
5 × 3 × ⅜	19·5	5·72	⅜	37·2	34·2	30·2	25·7
5 × 3 × 5/16	16·3	4·81	⅜	31·4	28·8	25·6	21·8
4½ × 3 × ½	23·8	7·00	⅜	45·7	42·0	37·2	31·8
4½ × 3 × ⅜	18·2	5·35	⅜	35·0	32·2	28·7	24·6
4½ × 3 × 5/16	15·3	4·50	⅜	29·6	27·3	24·3	20·9
4½ × 3 × ¼	12·3	3·62	⅜	23·8	21·8	19·5	16·7
4 × 3½ × ⅝	29·2	8·60	½	58·2	55·0	51·1	46·1
4 × 3½ × ½	23·8	7·00	½	47·5	44·9	41·8	37·9
4 × 3½ × ⅜	18·2	5·35	½	36·3	34·4	32·0	29·1
4 × 3½ × 5/16	15·3	4·50	½	30·5	29·0	27·1	24·6

For notes relating to above see page xi

Unequal angles—short legs connected and Dimensions and Properties

HEIGHTS IN FEET								Radii of Gyration	
7	8	9	10	11	12	14	16	Axis $y—y$	Axis $x—x$
45·1	38·5	32·8	27·9	23·9	20·6	15·8	12·3	2·48	·98
37·0	31·9	27·1	23·2	19·9	17·1	13·1	10·2	2·46	·99
28·6	24·6	21·0	18·1	15·4	13·3	10·2	8·1	2·43	1·01
24·1	20·8	17·8	15·2	13·0	11·3	8·6	6·8	2·41	1·01
27·5	22·7	18·8	15·7	13·3	11·4	8·6	6·7	2·49	·82
21·4	17·6	14·7	12·3	10·4	8·9	6·7	5·2	2·46	·83
18·1	15·0	12·4	10·5	8·8	7·7	5·7	4·4	2·44	·84
26·5	21·9	18·2	15·2	12·9	11·1	8·3	6·5	2·21	·84
20·6	17·1	14·2	11·9	10·1	8·6	6·5	5·1	2·19	·85
17·4	14·6	12·1	10·1	8·6	7·3	5·5	4·3	2·17	·85
14·0	11·6	9·7	8·1	6·9	5·9	4·5	3·5	2·15	·85
40·7	35·2	30·1	25·8	22·3	19·2	14·6	11·5	1·95	1·02
33·5	29·1	25·0	21·4	18·4	15·9	12·2	9·6	1·93	1·03
25·9	22·5	19·4	16·6	14·3	12·5	9·5	7·5	1·90	1·04
21·9	19·1	16·5	14·1	12·2	10·5	8·1	6·3	1·88	1·05

See note on page xiii regarding ratio of slenderness

COMPOUND STRUTS — Two Safe Concentric Loads in Tons

Two unequal angles	Weight per foot in lbs.	Area in sq. ins.	Space between angles in ins.	EFFECTIVE 2	3	4	5
4 × 3½ × ¼	12·3	3·62	½	25·7	24·6	23·4	21·8
4 × 3 × ½	22·1	6·50	⅜	45·3	42·6	39·3	35·0
4 × 3 × ⅜	16·9	4·97	⅜	34·7	32·7	30·2	27·1
4 × 3 × 5/16	14·1	4·18	⅜	29·2	27·5	25·6	22·8
4 × 3 × ¼	11·48	3·38	⅜	23·6	22·3	20·6	18·7
4 × 2½ × ⅜	15·6	4·60	⅜	31·2	28·6	24·9	20·6
4 × 2½ × 5/16	13·2	3·87	⅜	26·3	24·1	21·1	17·5
4 × 2½ × ¼	10·6	3·13	⅜	21·2	19·6	17·1	14·2
3½ × 3 × ½	20·4	6·00	⅜	41·9	39·5	36·6	32·8
3½ × 3 × ⅜	15·6	4·60	⅜	32·1	30·4	28·2	25·3
3½ × 3 × 5/16	13·2	3·87	⅜	27·1	25·6	23·7	21·4
3½ × 3 × ¼	10·6	3·13	⅜	22·0	20·8	19·3	17·4
3½ × 2½ × ⅜	14·3	4·22	5/16	28·7	26·4	23·2	19·4
3½ × 2½ × 5/16	12·1	3·56	5/16	24·2	22·3	19·6	16·6
3½ × 2½ × ¼	9·9	2·88	5/16	19·6	18·1	16·0	13·4

For notes relating to above see page xi

Unequal angles—short legs connected and Dimensions and Properties

HEIGHTS IN FEET								Radii of Gyration	
6	7	8	9	10	11	12	14	Axis y—y	Axis x—x
19·8	17·7	15·4	13·2	11·5	9·8	8·5	6·5	1·87	1·05
30·1	25·2	21·0	17·5	4·7	12·4	10·7	8·0	1·95	·85
23·3	19·6	16·3	13·6	11·5	9·7	8·3	6·3	1·92	·87
19·7	16·7	13·9	11·6	9·8	8·3	7·1	5·4	1·90	·87
16·1	13·6	11·4	9·5	8·0	6·8	5·9	4·4	1·88	·88
16·6	13·2	10·5	8·6	7·1	6·1	5·1		1·99	·69
14·0	11·2	9·0	7·3	6·1	5·2	4·3		1·98	·70
11·4	9·1	7·3	6·0	5·0	4·2	3·5	2·5	1·96	·70
28·4	23·9	20·1	16·7	14·0	11·8	10·2	7·7	1·68	·87
22·0	18·7	15·6	13·1	11·0	9·3	8·0	6·1	1·65	·88
18·7	15·8	13·3	11·2	9·5	8·0	6·8	5·2	1·64	·89
15·3	13·0	10·9	9·1	7·7	6·6	5·7	4·3	1·62	·90
15·7	12·5	10·1	8·2	6·8	5·7	4·9	3·7	1·70	·71
13·4	10·7	8·6	7·0	5·8	4·9	4·2	3·1	1·68	·71
10·9	8·7	7·1	5·8	4·8	4·1	3·4	2·5	1·67	·72

See note on page xiii regarding ratio of slenderness

COMPOUND STRUTS — Two Safe Concentric Loads in Tons

Two unequal angles	Weight per foot in lbs.	Area in sq. ins.	Space between angles ins.	EFFECTIVE		
				2	3	4
$3 \times 2\frac{1}{2} \times \frac{3}{8}$	13·1	3·85	$\frac{3}{8}$	26·3	24·2	21·5
$3 \times 2\frac{1}{2} \times \frac{5}{16}$	11·0	3·24	$\frac{3}{8}$	22·2	20·5	18·2
$3 \times 2\frac{1}{2} \times \frac{1}{4}$	8·9	2·63	$\frac{3}{8}$	18·0	16·6	14·7
$3 \times 2 \times \frac{3}{8}$	11·8	3·48	$\frac{3}{8}$	22·6	19·6	15·5
$3 \times 2 \times \frac{5}{16}$	10·0	2·93	$\frac{3}{8}$	19·1	16·7	13·2
$3 \times 2 \times \frac{1}{4}$	8·1	2·38	$\frac{3}{8}$	15·5	13·6	10·8
$2\frac{1}{2} \times 2 \times \frac{3}{8}$	10·5	3·09	$\frac{5}{16}$	20·3	17·7	14·3
$2\frac{1}{2} \times 2 \times \frac{5}{16}$	8·9	2·62	$\frac{5}{16}$	17·2	15·0	12·2
$2\frac{1}{2} \times 2 \times \frac{1}{4}$	7·3	2·13	$\frac{5}{16}$	14·1	12·3	10·0
$2\frac{1}{2} \times 2 \times \frac{3}{16}$	5·5	1·63	$\frac{5}{16}$	10·6	9·4	7·7
$2\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$	6·4	1·87	$\frac{5}{16}$	11·1	8·2	5·5
$2\frac{1}{2} \times 1\frac{1}{2} \times \frac{3}{16}$	4·8	1·43	$\frac{5}{16}$	8·6	6·3	4·3
$2 \times 1\frac{1}{2} \times \frac{1}{4}$	5·5	1·63	$\frac{5}{16}$	9·8	7·4	5·1
$2 \times 1\frac{1}{2} \times \frac{3}{16}$	4·2	1·24	$\frac{5}{16}$	7·5	5·8	4·0

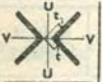
For notes relating to above see page xi

Unequal Angles—short legs connected and Dimensions and Properties

HEIGHTS IN FEET								Radii of Gyration	
5	6	7	8	9	10	11	12	Axis y—y	Axis x—x
18·2	14·8	11·8	9·6	7·8	6·6	5·5	4·7	1·46	·73
15·4	12·6	10·1	8·2	6·7	5·6	4·7	4·0	1·44	·73
12·6	10·3	8·3	6·7	5·5	4·6	3·8	3·4	1·43	·74
11·7	8·8	6·8	5·4	4·3	3·6	*3·0*		1·53	·55
10·0	7·5	5·8	4·6	3·6	*3·1*	*2·6*		1·52	·56
8·2	6·2	4·8	3·8	3·0	2·5	*2·1*		1·50	·56
10·8	8·3	6·4	5·0	4·1	*3·3*	*2·8*		1·24	·57
9·4	7·1	5·5	4·4	3·5	2·9	*2·4*		1·22	·57
7·7	5·9	4·6	3·6	2·9	2·4	*2·0*		1·21	·58
5·9	4·5	3·5	2·7	2·2	*1·8*	*1·5*		1·19	·58
3·8	2·8	*2·1*	*1·6*					1·28	·40
3·1	2·1	*1·6*	*1·3*					1·27	·41
3·6	2·6	1·8	*1·5*					1·02	·42
2·8	2·0	1·5	*1·2*					1·00	·43

See note on page xiii regarding ratio of slenderness

COMPOUND STRUTS — Two Safe Concentric Loads in Tons



Two equal angles	Weight per foot in lbs.	Area in sq. ins.	Thickness t of Batten Pls. ins	EFFECTIVE			
				8	10	12	14
$8 \times 8 \times 1$	102·0	30·00	1	206	199	191	182
$8 \times 8 \times \frac{7}{8}$	90·0	26·47	$\frac{7}{8}$	182	176	169	161
$8 \times 8 \times \frac{3}{4}$	77·8	22·89	$\frac{3}{4}$	158	152	146	140
$8 \times 8 \times \frac{5}{8}$	65·4	19·22	$\frac{5}{8}$	133	128	123	118
$7 \times 7 \times \frac{7}{8}$	78·1	22·98	$\frac{7}{8}$	155	148	141	132
$7 \times 7 \times \frac{3}{4}$	67·6	19·88	$\frac{3}{4}$	135	128	122	114
$7 \times 7 \times \frac{5}{8}$	56·8	16·72	$\frac{5}{8}$	113	108	103	96·8
$7 \times 7 \times \frac{1}{2}$	45·9	13·50	$\frac{1}{2}$	91·6	87·9	83·5	78·4
$6 \times 6 \times \frac{7}{8}$	66·3	19·47	$\frac{7}{8}$	128	120	111	101
$6 \times 6 \times \frac{3}{4}$	57·4	16·87	$\frac{3}{4}$	110	104	96·9	88·1
$6 \times 6 \times \frac{5}{8}$	48·3	14·22	$\frac{5}{8}$	93·6	88·5	82·1	74·9
$6 \times 6 \times \frac{1}{2}$	39·1	11·50	$\frac{1}{2}$	75·8	71·6	66·6	60·7
$6 \times 6 \times \frac{3}{8}$	29·7	8·72	$\frac{3}{8}$	57·5	54·4	50·7	46·3
$5 \times 5 \times \frac{3}{4}$	47·2	13·88	$\frac{3}{4}$	86·8	79·3	70·3	60·6
$5 \times 5 \times \frac{5}{8}$	39·9	11·72	$\frac{5}{8}$	73·6	67·4	59·9	51·8

For notes relating to above see page xi

Equal Angles—Battened at intervals and Dimensions and Properties

HEIGHTS IN FEET								Radii of Gyration	
16	18	20	24	28	32	36	40	Axis v—v	Axis u—u
172	160	148	122	99·9	81·3	66·9	55·8	**3·05**	4·32
152	142	131	109	88·9	72·5	59·7	49·7	**3·07**	4·18
132	123	114	95·1	77·7	63·3	52·1	43·6	**3·09**	4·03
111	104	96·6	80·4	65·7	53·8	44·2	36·9	**3·11**	3·89
121	110	99·4	78·7	62·2	49·8	40·7	33·6	**2·67**	3·78
105	96·5	86·8	68·8	54·6	43·7	35·6	29·4	**2·69**	3·64
89·6	81·7	73·5	58·4	46·3	37·2	30·3	25·1	**2·71**	3·49
72·6	66·3	59·8	47·6	37·8	30·3	24·7	20·6	**2·72**	3·35
89·8	78·7	68·5	51·8	40·1	31·7	25·5	*20·9*	**2·26**	3·38
78·5	68·9	60·1	45·6	35·2	27·8	22·5	*18·5*	**2·28**	3·24
66·7	58·6	51·2	39·0	30·2	23·8	19·3	*15·9*	**2·30**	3·09
54·3	47·8	41·8	31·9	24·7	19·5	15·8	*13·0*	**2·32**	2·95
41·4	36·5	32·0	24·4	18·9	15·1	12·3	*10·0*	**2·34**	2·80
51·4	43·5	36·8	27·1	20·6	*16·1*	*12·9*		**1·88**	2·84
44·1	37·3	31·7	23·3	17·7	*13·8*	*11·1*		**1·90**	2·70

See note on page xiii regarding ratio of slenderness

COMPOUND STRUTS — Two
Safe Concentric Loads in Tons

 Two equal angles	Weight per foot in lbs.	Area in sq. ins.	Thickness t of Batten Pls. ins.	EFFECTIVE			
				4	6	8	10
5 × 5 × ½	32·3	9·50	½	67·0	63·7	59·8	54·9
5 × 5 × ⅜	24·6	7·22	⅜	50·9	48·6	45·7	41·9
4½ × 4½ × ¾	42·1	12·39	¾	86·1	81·0	74·4	66·1
4½ × 4½ × ⅝	35·7	10·47	⅝	72·9	68·8	63·3	56·3
4½ × 4½ × ½	28·9	8·51	½	59·3	55·9	51·7	46·1
4½ × 4½ × ⅜	22·0	6·47	⅜	45·2	42·6	39·4	35·3
4½ × 4½ × 5/16	18·5	5·44	5/16	38·0	35·9	33·1	29·8
4 × 4 × ⅝	31·4	9·22	⅝	63·3	58·7	52·6	44·9
4 × 4 × ½	25·6	7·50	½	51·6	47·9	43·0	37·0
4 × 4 × ⅜	19·5	5·72	⅜	39·4	36·6	33·1	28·5
4 × 4 × 5/16	16·3	4·80	5/16	33·2	30·8	27·8	24·0
3½ × 3½ × ⅝	27·2	7·97	⅝	53·6	48·4	41·5	33·4
3½ × 3½ × ½	22·1	6·51	½	43·8	39·8	34·2	27·8
3½ × 3½ × ⅜	16·8	4·97	⅜	33·5	30·6	26·4	21·6
3½ × 3½ × 5/16	14·2	4·17	5/16	28·2	25·7	22·2	18·3

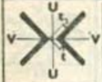
For notes relating to above see page xi

Equal angles—Battened at intervals and Dimensions and Properties

HEIGHTS IN FEET								Radii of Gyration	
12	14	16	18	20	22	24	28	Axis v—v	Axis u—u
48·9	42·4	36·2	30·8	26·1	22·3	19·2	14·6	1·92	2·55
37·4	32·5	27·8	23·5	20·2	17·2	14·8	11·3	1·94	2·41
56·5	47·2	39·0	32·4	27·2	23·1	19·7	14·9	1·68	2·64
48·3	40·5	33·7	28·0	23·6	19·9	17·1	12·9	1·70	2·50
39·7	33·4	27·8	23·1	19·4	16·6	14·2	10·7	1·72	2·35
30·6	25·7	21·6	17·8	15·0	12·8	10·9	8·4	1·74	2·21
25·8	21·8	18·2	15·4	12·8	10·9	9·3	7·1	1·75	2·14
36·9	29·9	24·4	19·9	16·7	14·1	11·8	9·0	1·50	2·30
30·6	24·8	20·2	16·7	13·8	11·7	9·9	7·5	1·52	2·16
23·6	19·3	15·7	12·9	10·8	9·2	7·7	5·9	1·54	2·01
19·9	16·2	13·2	11·0	9·1	7·7	6·5	4·9	1·54	1·95
26·3	20·7	16·5	13·5	11·1	9·3	7·9		1·30	2·10
21·9	17·3	13·9	11·2	9·3	7·9	6·7		1·32	1·96
17·1	13·5	10·8	8·8	7·4	6·1	5·2		1·34	1·81
14·5	11·5	9·2	7·6	6·2	5·3	4·5		1·35	1·74

See note on page xiii regarding ratio of slenderness

COMPOUND STRUTS — Two
Safe Concentric Loads in Tons

 Two equal angles	Weight per foot in lbs.	Area in sq. ins.	Thickness t of Batten Pls. ins.	EFFECTIVE			
				2	3	4	5
3½ × 3½ × ¼	11·5	3·38	¼	24·4	23·7	23·0	21·9
3 × 3 × ½	18·7	5·50	½	39·3	37·7	35·9	33·8
3 × 3 × ⅜	14·3	4·22	⅜	30·1	29·0	27·8	26·1
3 × 3 × 5/16	12·1	3·56	5/16	25·4	24·4	23·3	22·0
3 × 3 × ¼	9·8	2·88	¼	20·6	19·9	18·9	17·8
2½ × 2½ × ⅜	11·8	3·47	⅜	24·4	23·1	21·6	19·7
2½ × 2½ × 5/16	10·0	2·93	5/16	20·6	19·7	18·4	16·8
2½ × 2½ × ¼	8·1	2·38	¼	16·8	15·9	14·9	13·6
2¼ × 2¼ × ⅜	10·5	3·09	⅜	21·6	20·3	18·6	16·4
2¼ × 2¼ × 5/16	8·9	2·62	5/16	18·2	17·1	15·8	14·1
2¼ × 2¼ × ¼	7·2	2·13	¼	14·8	13·9	12·8	11·5
2¼ × 2¼ × 3/16	5·6	1·62	3/16	11·2	10·7	9·8	8·7
2 × 2 × 5/16	7·8	2·30	5/16	15·9	14·6	13·2	11·1
2 × 2 × ¼	6·4	1·87	¼	12·8	11·9	10·7	9·1
2 × 2 × 3/16	4·9	1·43	3/16	9·8	9·1	8·2	7·0

For notes relating to above see page xi

Equal Angles—Battened at intervals and Dimensions and Properties

HEIGHTS IN FEET								Radii of Gyration	
6	7	8	10	12	14	16	18	Axis v—v	Axis u—u
20·8	19·6	18·1	14·8	11·8	9·3	7·5	6·1	1·35	1·68
31·3	28·2	25·1	19·0	14·3	11·0	8·7	7·1	1·12	1·76
24·2	21·9	19·6	14·9	11·3	8·7	6·9	5·5	1·14	1·61
20·4	18·6	16·7	12·7	9·7	7·4	5·9	4·7	1·15	1·54
16·7	15·2	13·5	10·4	7·9	6·2	4·8	3·9	1·15	1·47
17·5	15·0	12·7	9·1	6·7	5·2	*4·0*	*3·2*	·94	1·41
14·8	12·8	10·9	7·9	5·7	4·4	*3·4*	*2·7*	·95	1·34
12·1	10·6	8·9	6·3	4·7	3·6	*2·8*	*2·3*	·95	1·27
14·0	11·7	9·7	6·7	4·9	*3·6*	*2·9*		·84	1·32
12·0	10·1	8·3	5·8	4·2	3·1	*2·4*		·85	1·24
9·8	8·3	6·7	4·8	3·4	2·6	*2·0*		·85	1·17
7·6	6·3	5·2	3·7	2·6	2·1	*1·6*		·86	1·09
9·1	7·4	6·0	4·1	2·9	2·2			·75	1·14
7·6	6·2	4·9	3·5	2·4	*1·8*			·75	1·07
5·8	4·7	3·8	2·6	1·9	*1·4*			·76	1·00

See note on page xiii regarding ratio of slenderness

STRUTS—Tees
Safe Concentric Loads in Tons

 Section	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE				
			6	7	8	9	10
6 × 6 × ½	19·62	5·78	34·2	31·6	28·5	25·4	22·4
6 × 4 × ½	16·22	4·77	27·2	24·6	21·9	19·2	16·8
6 × 3 × ½	14·51	4·28	18·1	14·8	12·1	10·1	8·3
6 × 3 × ⅜	11·08	3·26	14·0	11·4	9·4	7·8	6·6
5 × 4 × ½	14·50	4·27	23·9	21·4	18·8	16·3	14·1
5 × 4 × ⅜	11·06	3·25	18·0	16·0	14·0	12·1	10·4
5 × 3 × ½	12·80	3·77	16·7	13·8	11·4	9·4	7·9
5 × 3 × ⅜	9·79	2·88	12·9	10·7	8·8	7·2	6·3
4 × 4 × ½	12·78	3·76	16·9	14·1	11·6	9·6	8·1
4 × 4 × ⅜	9·77	2·88	12·6	10·4	8·6	7·2	5·9
4 × 3 × ½	11·09	3·26	15·1	12·7	10·5	8·7	7·4
4 × 3 × ⅜	8·49	2·50	11·7	9·8	8·2	6·8	5·7
3 × 3 × ⅜	7·20	2·12	6·5	5·0	4·1	3·2	2·7
2½ × 2½ × ⅜	5·92	1·74	4·0	3·1	2·4	*1·9*	*1·6*
2½ × 2½ × ¼	4·07	1·20	2·6	2·0	1·6	*1·3*	*1·0*

For notes relating to above see page xi

and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
11	12	13	14	16	18	Axis y—y	Axis x—x	Axis y—y	Axis x—x
19·6	17·3	15·2	13·4	10·7	8·6	2·85	4·36	1·22	1·82
14·5	12·6	11·0	9·7	7·7	6·2	2·88	2·00	1·35	1·13
7·0	6·0	5·3	4·6			2·89	1·14	1·42	·78
5·6	4·7	4·0	3·6			2·13	·87	1·40	·80
12·2	10·6	9·2	8·1	6·4	5·1	2·01	1·96	1·09	1·16
9·0	7·9	6·9	6·0	4·7	*3·8*	1·48	1·49	1·07	1·17
6·7	5·7	4·9	*4·3*	*3·3*		2·01	1·11	1·16	·82
5·2	4·5	3·8	*3·3*	2·6		1·49	·85	1·14	·83
6·8	5·8	5·0	*4·4*	*3·4*		1·30	1·90	·83	1·20
5·0	4·4	3·7	*3·2*	*2·5*		·95	1·45	·81	1·21
6·3	5·3	4·6	3·9	*3·1*		1·30	1·08	·89	·85
4·8	4·1	3·7	3·0	2·4		·96	·83	·87	·86
2·2	*1·9*					·54	·80	·62	·90
						·38	·55	·52	·74
						·24	·38	·50	·75

See note on page xiii regarding ratio of slenderness

ANGLES AS BEAMS

(Long Leg Vertical)

Safe Distributed Loads in Tons

X X Section	Weight per foot in lbs.	Grs. Moment of Inertia x—x	8 tons/sq. in. SPANS IN FEET 6	8	10	12	14	16
6 × 4 × ½	16·16	17·14	3·77	2·83	2·27	1·89	1·62	1·41
6 × 4 × ⅜	12·28	13·21	2·87	2·15	1·72	1·43	1·23	1·08
6 × 3½ × ½	15·30	16·36	3·68	2·76	2·21	1·84	1·58	1·36
6 × 3½ × ⅜	11·63	12·62	2·81	2·11	1·69	1·40	1·20	1·05
6 × 3 × ½	14·45	15·51	3·60	2·70	2·16	1·80	1·54	1·29
6 × 3 × ⅜	11·00	11·99	2·75	2·06	1·65	1·38	1·18	1·00
6 × 3 × 5/16	9·24	10·13	2·30	1·73	1·38	1·15	0·99	0·85
5 × 4 × ½	14·45	10·29	2·66	1·99	1·60	1·33	1·12	0·86
5 × 4 × ⅜	11·00	7·97	2·03	1·52	1·22	1·01	0·87	0·66
5 × 3½ × ½	13·61	9·84	2·61	1·95	1·56	1·30	1·07	0·82
5 × 3½ × ⅜	10·37	7·63	1·99	1·49	1·19	0·99	0·83	0·64
5 × 3½ × 5/16	8·71	6·46	1·67	1·25	1·00	0·84	0·70	0·54
5 × 3 × ⅜	9·73	7·25	1·94	1·45	1·16	0·97	0·79	0·60
5 × 3 × 5/16	8·17	6·14	1·63	1·23	0·98	0·81	0·67	0·51

The above Loads are based on Angles being simply supported over Single Spans.

For Angle Purlins see page No. 416.

ANGLES AS BEAMS

(Long Leg Vertical)

Safe Distributed Loads in Tons

Section	Weight per foot in lbs.	Grs. Moment of Inertia x—x	8 tons/sq. in. SPANS IN FEET 4	6	8	10	12	14
$4\frac{1}{2} \times 3 \times \frac{3}{8}$	9·09	5·41	2·38	1·59	1·19	0·95	0·79	0·60
$4\frac{1}{2} \times 3 \times \frac{5}{16}$	7·64	4·59	2·00	1·33	1·00	0·80	0·66	0·50
$4\frac{1}{2} \times 3 \times \frac{1}{4}$	6·16	3·73	1·61	1·07	0·81	0·65	0·54	0·40
$4 \times 3\frac{1}{2} \times \frac{3}{8}$	9·09	4·09	1·94	1·29	0·97	0·77	0·60	0·44
$4 \times 3\frac{1}{2} \times \frac{5}{16}$	7·64	3·47	1·62	1·08	0·81	0·65	0·51	0·37
$4 \times 3\frac{1}{2} \times \frac{1}{4}$	6·16	2·82	1·32	0·88	0·66	0·53	0·41	0·30
$4 \times 3 \times \frac{3}{8}$	8·45	3·89	1·89	1·26	0·94	0·76	0·57	0·42
$4 \times 3 \times \frac{5}{16}$	7·11	3·30	1·60	1·06	0·80	0·64	0·49	0·36
$4 \times 3 \times \frac{1}{4}$	5·74	2·69	1·28	0·85	0·64	0·51	0·40	0·29
$4 \times 2\frac{1}{2} \times \frac{3}{8}$	7·81	3·66	1·84	1·22	0·92	0·74	0·54	0·40
$4 \times 2\frac{1}{2} \times \frac{5}{16}$	6·58	3·11	1·56	1·04	0·78	0·62	0·46	0·34
$4 \times 2\frac{1}{2} \times \frac{1}{4}$	5·33	2·54	1·25	0·83	0·62	0·50	0·37	0·27
$3\frac{1}{2} \times 3 \times \frac{3}{8}$	7·82	2·67	1·47	0·98	0·73	0·57	0·40	0·29
$3\frac{1}{2} \times 3 \times \frac{5}{16}$	6·58	2·27	1·22	0·82	0·61	0·48	0·33	0·25

The above Loads are based on Angles being simply supported over Single Spans.

For Angle Purlins see page No. 416.

ANGLES AS BEAMS
(Long Leg Vertical)
Safe Distributed Loads in Tons

Section	Weight per foot in lbs.	Grs. Moment of Inertia x—x	8 tons/sq. in.	SPANS IN FEET				
			3	4	6	8	10	12
$3\frac{1}{2} \times 3 \times \frac{1}{4}$	5·32	1·86	1·33	1·00	0·66	0·50	0·39	0·27
$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$	7·17	2·51	1·90	1·42	0·95	0·71	0·53	0·37
$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$	6·05	2·14	1·60	1·20	0·80	0·60	0·45	0·31
$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	4·89	1·75	1·29	0·97	0·65	0·48	0·37	0·26
$3 \times 2\frac{1}{2} \times \frac{3}{8}$	6·54	1·62	1·40	1·05	0·70	0·52	0·34	0·24
$3 \times 2\frac{1}{2} \times \frac{5}{16}$	5·51	1·39	1·18	0·89	0·59	0·44	0·29	0·20
$3 \times 2\frac{1}{2} \times \frac{1}{4}$	4·47	1·14	0·95	0·72	0·48	0·35	0·24	0·17
$3 \times 2 \times \frac{3}{8}$	5·90	1·50	1·35	1·01	0·67	0·50	0·32	0·22
$3 \times 2 \times \frac{5}{16}$	4·98	1·29	1·15	0·87	0·58	0·43	0·27	0·19
$3 \times 2 \times \frac{1}{4}$	4·04	1·06	0·92	0·69	0·46	0·34	0·22	0·15
$2\frac{1}{2} \times 2 \times \frac{3}{8}$	5·26	0·89	0·94	0·70	0·47	0·29	0·19	0·13
$2\frac{1}{2} \times 2 \times \frac{5}{16}$	4·45	0·77	0·80	0·60	0·40	0·25	0·16	0·11
$2\frac{1}{2} \times 2 \times \frac{1}{4}$	3·60	0·63	0·66	0·49	0·33	0·21	0·13	0·09
$2\frac{1}{2} \times 2 \times \frac{3}{16}$	2·76	0·49	0·49	0·37	0·25	0·16	0·10	0·07

The above Loads are based on Angles being simply supported over Single Spans.

For Angle Purlins see page No. 416.

ANGLES AS BEAMS

(Equal Angles)

Safe Distributed Loads in Tons

X X Section	Weight per foot in lbs.	Grs. Moment of Inertia x—x	8 tons/sq. in. SPANS IN FEET 4	6	8	10	12	14
6 × 6 × $\frac{3}{8}$	14·82	14·95	4·53	3·02	2·27	1·81	1·51	1·30
5 × 5 × $\frac{1}{2}$	16·16	11·04	4·10	2·73	2·05	1·64	1·36	1·17
5 × 5 × $\frac{3}{8}$	12·28	8·53	3·13	2·09	1·57	1·25	1·04	0·90
4 × 4 × $\frac{1}{2}$	12·76	5·46	2·58	1·72	1·29	1·03	0·81	0·59
4 × 4 × $\frac{3}{8}$	9·73	4·26	1·97	1·31	0·98	0·79	0·63	0·46
$3\frac{1}{2}$ × $3\frac{1}{2}$ × $\frac{3}{8}$	8·45	2·80	1·49	0·99	0·74	0·60	0·41	0·30
$3\frac{1}{2}$ × $3\frac{1}{2}$ × $\frac{5}{16}$	7·11	2·38	1·25	0·84	0·63	0·50	0·35	0·26
$3\frac{1}{2}$ × $3\frac{1}{2}$ × $\frac{1}{4}$	5·74	1·94	1·01	0·68	0·50	0·40	0·28	0·21
3 × 3 × $\frac{3}{8}$	7·17	1·72	1·08	0·72	0·54	0·37	0·25	0·18
3 × 3 × $\frac{5}{16}$	6·04	1·47	0·90	0·60	0·45	0·31	0·22	0·16
3 × 3 × $\frac{1}{4}$	4·89	1·20	0·73	0·49	0·36	0·26	0·18	0·13
$2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{3}{8}$	5·90	0·96	0·73	0·49	0·32	0·20	0·14	0·10
$2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{5}{16}$	4·98	0·83	0·62	0·41	0·27	0·18	0·12	0·09
$2\frac{1}{2}$ × $2\frac{1}{2}$ × $\frac{1}{4}$	4·05	0·68	0·50	0·33	0·22	0·15	0·10	0·07

The above Loads are based on Angles being simply supported over Single Spans.

For Angle Purlins see page No. 416.

TIES—Safe Loads in tons for single Angles in Tension. One $\frac{13}{16}$ hole and half outstanding leg deducted.

	Safe Loads in tons						
	Thickness of Angle in inches						
Section	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{5}{8}$
$2 \times 1\frac{1}{2}$	2·94	3·85		$\frac{11}{16}''$	hole	dedu	cted
2×2	3·30	4·41	5·42	6·36	,,	,,	,,
$2\frac{1}{4} \times 2\frac{1}{4}$	3·91	5·12	6·38	7·51	,,	,,	,,
$2\frac{1}{2} \times 1\frac{1}{2}$	3·48	4·65					
$2\frac{1}{2} \times 2$	3·91	5·13	6·27	7·57			
$2\frac{1}{2} \times 2\frac{1}{2}$		5·67	6·95	8·26	9·60	10·8	
3×2		6·17	7·57	9·01	10·5		
$3 \times 2\frac{1}{2}$		6·63	8·22	9·78	11·4		
3×3		7·15	8·88	10·5	12·2	13·8	
$3\frac{1}{2} \times 2\frac{1}{2}$		7·67	9·50	11·3	13·1		
$3\frac{1}{2} \times 3$		8·11	10·1	12·1	13·9	15·8	
$3\frac{1}{2} \times 3\frac{1}{2}$		8·67	10·7	12·8	14·8	16·8	20·7
$4 \times 2\frac{1}{2}$		8·63	10·7	12·8	14·8		
4×3		9·15	11·4	13·6	15·7	17·8	
$4 \times 3\frac{1}{2}$		9·63	12·0	14·3	16·6	18·8	23·1
4×4			12·6	15·1	17·5	19·8	24·3

TIES—Safe Loads in tons for single Angles in Tension. Two $\frac{13}{16}$ holes and half outstanding leg deducted.

	Safe Loads in tons						
	Thickness of Angle in inches						
Section	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{3}{4}$
$4\frac{1}{2} \times 4\frac{1}{2}$	12·5	14·9	17·3	19·6	21·8	24·1	28·5
5×3	11·8	14·1	16·4	18·6	20·7		
$5 \times 3\frac{1}{2}$	12·5	14·9	17·3	19·6	21·9	24·0	
5×4		15·7	18·2	20·6	23·0	25·3	
5×5		17·2	19·9	22·6	25·2	27·8	33·0
6×3	14·4	17·2	19·9	22·6	25·2	27·8	
$6 \times 3\frac{1}{2}$	15·0	17·9	20·7	23·6	26·4	29·0	
6×4		18·7	21·7	24·6	27·5	30·3	36·0
6×6		21·7	25·1	28·6	31·9	35·3	42·0
$7 \times 3\frac{1}{2}$		20·9	24·3	27·6	30·9	34·0	
7×4				28·6	31·9	35·3	42·0
8×4				32·6	36·4	40·3	48·0

For notes relating to these two tables see page xv

The above loads are based on 8 Tons per square inch stress

TIES—Safe Concentric Loads in One $\frac{13}{16}$ Hole Deducted.

Sum of Flanges ins.	Safe loads in tons						
	Thickness of angle in inches						
	$\frac{3}{16}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{5}{8}$
3½	3·92	5·12		$\frac{13}{16}$″	hole	dedu	cted
4	4·64	6·16		″	″	″	″
4½	5·28	6·9	8·4	9·9			
5		7·9	9·7	11·4	13·1	14·8	
5½		8·9	10·9	12·9	14·9	16·8	
6		9·9	12·2	14·4	16·6	18·8	22·8
6½		10·9	13·4	15·9	18·4	20·8	25·3
7			14·7	17·4	20·0	22·8	27·8
7½			16·0	19·0	21·9	24·8	30·3
8			17·2	20·5	23·6	26·8	32·8
8½			18·5	22·0	25·4	28·8	35·3
9			19·7	23·5	27·0	30·8	37·8
9½			20·9	24·9	28·9	32·8	40·3
10			22·2	26·5	30·6	34·8	42·8

The Above Loads are concentric and are based on one hole only being deducted from angle. For deduction of Half outstanding leg see page 284.

Tons for Angles in Tension. Two $\frac{13}{16}$ Holes Deducted.

Sum of Flanges ins.	Safe loads in tons						
	Thickness of angle in inches						
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$
6	8·3	10·2	12·0	13·8	15·5	18·8	
6½	9·3	11·4	13·5	15·5	17·5	21·3	
7		12·7	15·0	17·3	19·5	23·8	
7½		13·9	16·5	19·0	21·5	26·3	
8		15·2	18·0	20·8	23·5	28·8	
8½		16·4	19·5	22·5	25·5	31·3	
9		17·7	21·0	24·3	27·5	33·8	39·8
9½		18·9	22·5	26·0	29·5	36·3	42·8
10		20·2	24·0	27·8	31·5	38·8	45·8
10½			25·5	29·5	33·5	41·3	48·8
11			27·0	31·3	35·5	43·7	51·7
12			30·0	34·8	39·5	48·8	57·8

The Above Loads are concentric and are based on two holes only being deducted from angle. For deduction of Half outstanding leg see page 285.

The above Loads are based on 8 Tons per Square inch Stress.

Tabulated Areas in square inches of from Tension Members

Thickness of Member in ins.	9/16″ diameter				11/16″ diameter			
	No. of Holes				No. of Holes			
	1	2	3	4	1	2	3	4
3/16	·11	·22	·33	·44	·13	·26	·39	·52
1/4	·14	·28	·42	·56	·17	·34	·51	·68
5/16	·18	·36	·54	·72	·21	·42	·63	·84
3/8	·21	·42	·63	·84	·26	·52	·78	1·04
7/16	·25	·50	·75	1·00	·30	·60	·90	1·20
1/2	·28	·56	·84	1·12	·34	·68	1·02	1·36
9/16	·32	·64	.96	1·28	·39	·78	1·17	1·56
5/8	·35	·70	1·05	1·40	·43	·86	1·29	1.72
11/16	·39	·78	1·17	1·56	·47	·94	1·41	1·88
3/4	·42	·84	1·26	1·68	·52	1·04	1·56	2·08
7/8	·49	·98	1·47	1·96	·60	1·20	1·80	2·40
1	·56	1·12	1·68	2·24	·69	1·38	2·07	2·76
1¼	·70	1·40	2·10	2·80	·86	1·72	2·58	3·44
1½	·84	1·68	2·52	3·36	1·03	2·06	3·09	4·12
2	1·12	2·24	3·36	4·48	1·37	2·74	4·11	5·48

Nos. 1 to 4 Rivet Holes to be deducted of various thicknesses.

$\frac{13}{16}''$ diameter				$\frac{15}{16}''$ diameter				Thickness of Member in ins.
No. of Holes				No. of holes				
1	2	3	4	1	2	3	4	
·15	·30	·45	·60	·18	·36	·54	·72	$\frac{3}{16}$
·20	·40	·60	·80	·23	·46	·69	·92	$\frac{1}{4}$
·25	·50	·75	1·00	·29	·58	·87	1·16	$\frac{5}{16}$
·30	·60	·90	1·20	·35	·70	1·05	1·40	$\frac{3}{8}$
·36	·72	1·08	1·44	·41	·82	1·23	1·64	$\frac{7}{16}$
·41	·82	1·23	1·64	·47	·94	1·41	1·88	$\frac{1}{2}$
·46	·92	1·38	1·84	·53	1·06	1·59	2·12	$\frac{9}{16}$
·51	1·02	1·53	2·04	·59	1·18	1·77	2·35	$\frac{5}{8}$
·56	1·12	1·68	2·24	·64	1·28	1·92	2·56	$\frac{11}{16}$
·61	1·22	1·83	2·44	·70	1·40	2·10	2·80	$\frac{3}{4}$
·71	1·42	2·13	2·84	·82	1·64	2·46	3·28	$\frac{7}{8}$
·81	1·62	2·43	3·24	·94	1·88	2·82	3·76	1
1·02	2·04	3·06	4·08	1·17	2·34	3·51	4·68	$1\frac{1}{4}$
1·22	2·44	3·66	4·88	1·41	2·82	4·23	5·64	$1\frac{1}{2}$
1·62	3·24	4·86	6·48	1·87	3·74	5·61	7·48	2

STEEL CHEQUERED PLATES

Nett Thickness on Plain	Wt. in lbs. per sq. foot Ordinary Diamond	Wt. in lbs. per sq. foot Admiralty Diamond	Normal Thickness over chequer
$\frac{3}{16}$"	9·25	9·00	$\frac{1}{4}$"
$\frac{1}{4}$"	11·75	11·50	$\frac{5}{16}$"
$\frac{5}{16}$"	14·25	14·00	$\frac{3}{8}$"
$\frac{3}{8}$"	16·75	16·55	$\frac{7}{16}$"
$\frac{7}{16}$"	19·50	19·10	$\frac{1}{2}$"
$\frac{1}{2}$"	22·00	21·65	$\frac{9}{16}$"

Safe Uniformly Distributed Loads in lbs./sq. ft. on Plates simply supported 2 sides — 8^{T}/sq. in.

Nett Thickness on Plain	SPANS IN FEET 2	2·5	3	3·5	4	4·5	5	5·5	6
$\frac{3}{16}$"	210	134	93·3	68·6	52·5	41·5	33·6	27·8	23·3
$\frac{1}{4}$"	373	239	166	122	93·3	73·7	59·7	49·4	41·5
$\frac{5}{16}$"	583	373	259	190	146	115	93·3	77·1	64·8
$\frac{3}{8}$"	840	538	373	274	210	166	134	111	93·3
$\frac{7}{16}$"	1143	732	508	373	286	226	183	151	127
$\frac{1}{2}$"	1493	956	664	488	373	295	239	197	166

These Safe Loads include the weight of the plate itself, which must be deducted to obtain the nett superimposed load. Safe loads for fibre stress of 10 tons/sq. in. can be obtained from these values by multiplying them by $1\frac{1}{4}$. Care should be taken, however, to avoid excessive Deflection when using this higher Stress.

STEEL CHEQUERED PLATES

Safe Uniformly Distributed Loads in lbs./sq. ft. Simply supported 4 sides.
For Fixed Plates Multiply Loads by $1\frac{1}{2}$

8^T/ sq. in.

Nett Thickness on Plain	SPANS IN FEET								Breadth in Feet
	2·5	3	3·5	4	4·5	5	5·5	6	
3/16"	228	181	161	150	144	141	139	138	2·5
		158	130	116	108	103	100	98	3
			116	99	87	81	77	75	3·5
1/4"	405	322	285	267	257	250	248	244	2·5
		281	232	206	191	183	178	174	3
			206	175	155	144	138	133	3·5
				158	136	122	113	108	4
					125	110	98	92	4·5
						101	90	81	5
5/16"	632	504	446	417	401	391	387	382	2·5
		439	362	322	299	286	277	272	3
			322	274	243	225	215	208	3·5
				247	212	191	176	168	4
					195	171	153	143	4·5
						158	141	126	5
3/8"	910	725	642	601	578	563	557	550	2·5
		632	521	464	431	412	399	391	3
			464	395	350	324	310	300	3·5
				356	306	274	254	242	4
					281	247	221	206	4·5
						228	202	181	5
7/16"	1239	987	874	818	786	766	758	749	2·5
		860	710	632	586	560	544	532	3
			632	537	476	442	422	408	3·5
				484	416	373	345	330	4
					382	336	301	281	4·5
						310	275	247	5
1/2"	1618	1289	1141	1068	1027	1001	990	978	2·5
		1124	927	826	766	732	710	695	3
			826	702	622	577	551	533	3·5
				632	544	488	451	431	4

STEEL CHEQUERED PLATES 8T/sq. in.

Approx. DEFLECTIONS (in inches) of Plates carrying loads, on page 291.

Nett Thickness on Plain	SPANS IN FEET 2·5	3	3·5	4	4·5	5	5·5	6	Bth. in feet
3/16"	·560	·553	·551	·548	·546	·548	·548	·548	2·5
		·802	·800	·798	·795	·792	·791	·789	3·0
			1·09	1·10	1·08	1·08	1·07	1·08	3·5
1/4"	·419	·415	·412	·412	·411	·410	·412	·409	2·5
		·602	·602	·598	·593	·594	·594	·591	3·0
			·818	·823	·812	·808	·811	·807	3·5
				1·07	1·07	1·07	1·06	1·06	4·0
					1·36	1·37	1·34	1·35	4·5
						1·67	1·69	1·67	5·0
5/16"	·335	·333	·330	·329	·328	·328	·329	·328	2·5
		·482	·481	·478	·476	·475	·473	·473	3·0
			·654	·660	·652	·647	·647	·646	3·5
				·857	·857	·854	·843	·845	4·0
					1·08	1·09	1·07	1·08	4·5
						1·34	1·36	1·33	5·0
3/8"	·279	·277	·275	·275	·274	·274	·274	·273	2·5
		·401	·401	·399	·397	·396	·395	·393	3·0
			·546	·550	·543	·539	·540	·539	3·5
				·714	·716	·709	·704	·704	4·0
					·903	·913	·898	·897	4·5
						1·12	1·12	1·10	5·0
7/16"	·239	·237	·236	·235	·235	·234	·235	·234	2·5
		·344	·344	·342	·340	·339	·339	·337	3·0
			·468	·471	·465	·463	·463	·462	3·5
				·612	·613	·608	·603	·605	4·0
					·773	·782	·771	·771	4·5
						·956	·964	·949	5·0
1/2"	·209	·208	·206	·206	·205	·205	·206	·205	2·5
		·301	·301	·300	·297	·297	·296	·295	3·0
			·410	·413	·407	·405	·405	·404	3·5
				·535	·537	·533	·528	·529	4·0

BOLTS

BLACK—TURNED—TURNED BARREL—BRIGHT

Black. Owing to the difficulties in rolling ordinary round Black Bars exactly to the required diameter, it is customary to allow the shank diameter $\frac{1}{16}''$ less than the hole. The thread diameter is usually $\frac{1}{64}''$ less than the Bolt Shank.

Turned. These Bolts being specially made, the Black Bars are rolled a size larger and turned down to the required diameter. It is customary to allow the shank diameter $\frac{1}{32}''$ less than the hole. The thread diameter is usually about $\frac{1}{64}''$ less than the diameter of the shank.

Turned Barrel. These are turned Bolts but with the Shank diameter slightly larger than the thread, usually made to suit special circumstances. They are generally required for machinery purposes and seldom used by Structural Engineers. Inside faces of Heads and Nuts of both Turned and Turned Barrel Bolts are machined.

Bright. These are also specially made and turned down or forged to the exact diameter and are made with even more exact tolerance limits than Turned Bolts. They are machined all over, including all faces of Head, Nut and Washer.

Washers. For ordinary Black Bolts a round washer for each Bolt is supplied, and where the Bolt goes through the Flange of the Beam or Channel a square bevel washer is also supplied sometimes as well as the round washer.

Washers for Turned Bolts are $\frac{1}{4}''$ thick and are generally machined on all faces.

For Turned and Bright Bolts it is customary to specify the length of the threaded portion so that this part does not encroach on the Steelwork connected.

SHEARING AND BEARING VALUES for Turned Fitted Bolts and Rivets

Dia. of Bolt or Rivet in ins.	Shearing Values @ 6 tons/inch^2		Bearing Values @ 12 tons/inch^2						
			Thickness of Plate in ins.						
	Single Shear	Double Shear	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$
$\frac{3}{8}$	·66	1·33	1·13	1·41					
$\frac{1}{2}$	1·18	2·36	1·50	1·88	2·25				
$\frac{5}{8}$	1·84	3·68	1·88	2·34	2·81	3·75			
$\frac{3}{4}$	2·65	5·30	2·25	2·81	3·38	4·50	5·63		
$\frac{7}{8}$	3·61	7·22	2·63	3·28	3·94	5·25	6·56	7·88	
1	4·71	9·42	3·00	3·75	4·50	6·00	7·50	9·00	10·5

Dia. of Rivet in ins.	Shearing Values @ 5 tons/inch^2		Bearing Values @ 10 tons/inch^2						
			Thickness of Plate in ins.						
	Single Shear	Double Shear	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$
$\frac{3}{8}$	·55	1·10	·94	1·17	1·41				
$\frac{1}{2}$	·98	1·96	1·25	1·56	1·88	2·50			
$\frac{5}{8}$	1·53	3·07	1·56	1·95	2·34	3·13			
$\frac{3}{4}$	2·21	4·42	1·88	2·34	2·81	3·75	4·69		
$\frac{7}{8}$	3·01	6·01	2·19	2·73	3·28	4·38	5·47	6·56	
1	3·93	7·85	2·50	3·13	3·75	5·00	6·25	7·50	8·75

SHEARING AND BEARING VALUES for Black Bolts

Dia. of Bolt in ins.	Shearing Values @ 4 tons/inch²		Bearing Values @ 8 tons/inch²						
	Single Shear	Double Shear	Thickness of Plate in ins.						
			$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$
$\frac{3}{8}$	·44	·88	·75	·94	1·13				
$\frac{1}{2}$	·79	1·57	1·00	1·25	1·50	2·00			
$\frac{5}{8}$	1·23	2·45	1·25	1·56	1·88	2·50			
$\frac{3}{4}$	1·77	3·53	1·50	1·88	2·25	3·00	3·75		
$\frac{7}{8}$	2·41	4·81	1·75	2·19	2·63	3·50	4·38	5·25	
1	3·14	6·28	2·00	2·50	3·00	4·00	5·00	6·00	7·00

Extract from British Standard Specification (No. 449—1937) in connection with Shear and Bearing Stresses—

	Tons/inch²
For Parts in Shear	
On shop rivets and tight fitting turned bolts - - - -	6
On Field Rivets - - - -	5
On Black Bolts - - - -	4
For Parts in Bearing	
On shop rivets and tight fitting turned bolts - - - -	12
On Field Rivets - - - -	10
On Black Bolts - - - -	8

Note.—The Strength of rivets and bolts in double shear may be taken as twice that for single shear.

SHEARING AND BEARING VALUES for Turned Fitted Bolts and Rivets $+\frac{1}{3}$

Dia. of Bolt or Rivet in ins.	Shearing Values @ 6 tons/inch² + 33⅓%		Bearing Values @ 12 tons/inch² + 33⅓%						
			Thickness of Plate in ins.						
	Single Shear	Double Shear	¼	5/16	⅜	½	⅝	¾	⅞
⅜	0·88	1·77	1·51	1·88					
½	1·57	3·15	2·00	2·51	3·00				
⅝	2·45	4·91	2·51	3·12	3·75	5·00			
¾	3·53	7·07	3·00	3·75	4·51	6·00	7·51		
⅞	4·81	9·63	3·51	4·37	5·25	7·00	8·75	10·5	
1	6·28	12·56	4·00	5·00	6·00	8·00	10·0	12·0	14·0

Dia. of Rivet in ins.	Shearing Values @ 5 tons/inch² + 33⅓%		Bearing Values @ 10 tons/inch² + 33⅓%						
			Thickness of Plate in ins.						
	Single Shear	Double Shear	¼	5/16	⅜	½	⅝	¾	⅞
⅜	0·73	1·47	1·25	1·56	1·88				
½	1·31	2·61	1·67	2·08	2·51	3·33			
⅝	2·04	4·09	2·08	2·60	3·12	4·17			
¾	2·95	5·89	2·51	3·12	3·75	5·00	6·25		
⅞	4·01	8·01	2·92	3·64	4·37	5·84	7·29	8·75	
1	5·24	10·47	3·33	4·17	5·00	6·67	8·33	10·0	11·7

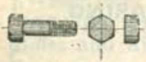
SHEARING AND BEARING VALUES for Black Bolts $+\frac{1}{3}$

Dia. of Bolt in ins.	Area in square ins.	Shearing Values @ 4 tons/inch² + 33⅓%		Bearing Values @ 8 tons/inch² + 33⅓%					
				Thickness of Plate in ins.					
		Single Shear	Double Shear	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$
$\frac{3}{8}$	0·1104	0·59	1·17	1·00	1·25	1·51			
				·109	*·137*	*·164*			
$\frac{1}{2}$	0·1963	1·05	2·09	1·33	1·67	2·00	2·67		
				·141	*·176*	*·211*	*·281*		
$\frac{5}{8}$	0·3068	1·64	3·27	1·67	2·08	2·51	3·33		
				·172	*·215*	*·258*	*·344*		
$\frac{3}{4}$	0·4418	2·36	4·71	2·00	2·51	3·00	4·00	5·00	
				·203	*·254*	*·305*	*·406*	*·508*	
$\frac{7}{8}$	0·6013	3·21	6·41	2·33	2·92	3·51	4·67	5·84	7·00
				·234	*·293*	*·352*	*·469*	*·586*	*·703*
1	0·7854	4·19	8·37	2·67	3·33	4·00	5·33	6·67	8·00
				·266	*·332*	*·398*	*·531*	*·664*	*·797*

These Tables are in accordance with the requirements of B.S.S. 449, clause 18.

Note.—Before adding the ⅓rd, it may be necessary if heavy Dead and Live loads are carried, to check the Stresses due from Loads other than wind, to see that they do not exceed the permissible Stresses given in the B.S.S. 449.

The figures in italics immediately under the Bearing Values above represent the Areas in square inches to be deducted from the given plate Thickness for respective Bolt or Rivet dia. The Size of hole being taken as $\frac{1}{16}$ more than the dia. of Bolt or Rivet.



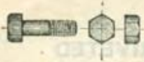
Hex. Round Hex.

BLACK BOLTS AND NUTS

To B.S.S. No. 28-1932

Dia. of Bolt in ins.	Nuts: Width across Flats in ins. Max.	Nuts: Width across Corners in ins. Max.	Nuts: Thickness in ins. Max.	Bolt Head Thickness in ins. Max.	Area of Shank in sq. ins.	Area at btm. of thread in sq. ins.
$\frac{1}{4}$	0·525	0·61	0·27	0·24	0·049	0·0272
$\frac{5}{16}$	0·600	0·69	0·33	0·29	0·076	0·0458
$\frac{3}{8}$	0·710	0·82	0·40	0·35	0·110	0·0683
$\frac{7}{16}$	0·820	0·95	0·46	0·40	0·150	0·0940
$\frac{1}{2}$	0·920	1·06	0·52	0·46	0·196	0·1215
$\frac{9}{16}$	1·010	1·17	0·58	0·51	0·248	0·1632
$\frac{5}{8}$	1·100	1·27	0·65	0·57	0·306	0·2032
$\frac{3}{4}$	1·300	1·50	0·77	0·68	0·442	0·3038
$\frac{7}{8}$	1·480	1·71	0·90	0·79	0·601	0·4216
1	1·670	1·93	1·02	0·90	0·785	0·5540
$1\frac{1}{8}$	1·860	2·15	1·16	1·01	0·994	0·6969
$1\frac{1}{4}$	2·050	2·37	1·28	1·12	1·227	0·8942
$1\frac{1}{2}$	2·410	2·78	1·53	1·34	1·767	1·3001
$1\frac{3}{4}$	2·760	3·19	1·78	1·56	2·405	1·7528
2	3·150	3·64	2·03	1·78	3·141	2·3111
$2\frac{1}{4}$	3·550	4·10	2·28	2·00	3·976	2·9249
$2\frac{1}{2}$	3·890	4·49	2·53	2·22	4·908	3·7318
$2\frac{3}{4}$	4·180	4·83	2·78	2·44	5·939	4·4641
3	4·530	5·23	3·03	2·66	7·068	5·4496
$3\frac{1}{4}$	4·850	5·60	3·28	2·87	8·296	6·4063
$3\frac{1}{2}$	5·180	5·98	3·53	3·09	9·621	7·5769

During present emergency, Black Bolts and Nuts should



Hex. Round Hex.

BLACK BOLTS AND NUTS

To B.S.S. No. 916-1940

Dia. of Bolt in ins.	Nuts: Width across Flats in ins.		Nuts: Width across Corners in ins.	Nuts: Thickness in ins.		Bolt Head Thickness in ins.	
	Max.	Min.	Max.	Max.	Min.	Max.	Min.
$\frac{1}{4}$	0·445	0·435	0·51	0·22	0·20	0·20	0·18
$\frac{5}{16}$	0·525	0·515	0·61	0·27	0·25	0·23	0·21
$\frac{3}{8}$	0·600	0·585	0·69	0·33	0·31	0·28	0·26
$\frac{7}{16}$	0·710	0·695	0·82	0·39	0·37	0·34	0·32
$\frac{1}{2}$	0·820	0·800	0·95	0·46	0·43	0·40	0·37
$\frac{9}{16}$	0·920	0·900	1·06	0·53	0·50	0·46	0·43
$\frac{5}{8}$	1·010	0·985	1·17	0·60	0·56	0·51	0·48
$\frac{3}{4}$	1·200	1·175	1·39	0·72	0·68	0·62	0·59
$\frac{7}{8}$	1·300	1·270	1·50	0·81	0·75	0·69	0·65
1	1·480	1·450	1·71	0·93	0·87	0·80	0·76
$1\frac{1}{8}$	1·670	1·640	1·93	1·06	1·00	0·91	0·87
$1\frac{1}{4}$	1·860	1·815	2·15	1·20	1·12	1·02	0·96
$1\frac{1}{2}$	2·220	2·175	2·56	1·45	1·37	1·24	1·18
$1\frac{3}{4}$	2·580	2·520	2·98	1·72	1·62	1·50	1·40
2	2·760	2·700	3·19	1·85	1·75	1·61	1·51
$2\frac{1}{4}$	3·150	3·090	3·64	1·97	1·87	1·77	1·67
$2\frac{1}{2}$	3·550	3·490	4·10	2·22	2·12	1·99	1·89
$2\frac{3}{4}$	3·890	3·830	4·49	2·47	2·37	2·16	2·06
3	4·180	4·080	4·83	2·77	2·62	2·43	2·28
$3\frac{1}{4}$	4·530	4·430	5·23	3·12	2·87	2·75	2·50
$3\frac{1}{2}$	4·850	4·750	5·60	3·27	3·12	2·84	2·71

be manufactured to B.S.S. 916 and not to B.S.S. 28.

BOLTED AND RIVETED BRACKETS

The following pages give tables of actual safe loads of various Rivet and Bolt groups fully worked out.

On pages 314 and 315 are given safe loads on Bolted Brackets for 2 vertical rows of bolts at any cross centres and varying vertical centres 3", 3½" and 4" and for 2 horizontal rows (i.e. 4 bolts) to 10 rows (i.e. 20 bolts). By simply increasing the number of vertical rows the safe load can be proportioned accordingly, i.e. if 2 vertical rows (16 bolts at 3" vertical centres and any C/C) can carry 4·25 tons at an eccentric arm (e) of 12", then 4 vertical rows (32 bolts at 3" vertical centres and any C/C) can carry 8·5 tons still on the 12" eccentric arm.

These safe loads have been calculated on the Theory that the Centre of Gravity of the Bolt Group is also the Centre of rotation of the bracket. As this assumption is not strictly correct (although often used) we have incorporated an additional Table in the Supplement (Pages K and L) based on the Theory that the rotation tendency occurs somewhere between the base of the bracket and the Centre of Gravity of the Bolt Group. This is in accordance with the reinforced Concrete Beam Theory and we believe is the more correct method of treating the problem.

MODULI VALUES

On pages 316 to 319 are tables giving moduli values $\left(\frac{\Sigma d^2}{y}\right)$ based on two vertical lines of Rivets or Bolts up to 9 horizontal rows deep at 3" vertical centres and of varying cross centres (C/C) from $2\frac{1}{2}$" to $23\frac{1}{2}$".

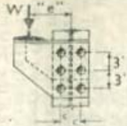
When a bracket carries a load on the same plane as the Rivet Group but which does not coincide with the centre of gravity, then this load is termed "eccentric", and the rivets in the bracket in addition to carrying the actual load have to be of sufficient size and number to withstand the moment which is set up. These moduli values can be used in determining the force due to this moment. To this force it will be necessary to add the force from the actual load itself, and as the two forces are not acting in the same direetion (although on the same plane), it will be necessary to resolve them into one resultant force to which the outermost rivet is subjected. This can be determined graphically (see examples on page 322) by drawing a vector diagram, the force from the moment being at right angles to a line between the centre of gravity of the Rivet Group and the outermost rivet. If the outermost rivet cannot be made of a practical size large enough to withstand this combined force, then an increase in the number of rivets has to be made until the safe stress per rivet is reached.

ECCENTRIC LOADS ON RIVETED BRACKETS

Load W on same plane as Rivet Group.

Rivet Centres (Vertical) 3"

Shearing Value of Rivets 6 tons/sq. in.



1½" c/c suitable for 3" Flange
2" c/c " 3½" "
2¼" c/c " 4" "
2½" c/c " 4½" "
2¾" c/c " 5" "

½" diam. Rivets	No. of Horz. Rows	SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)														
		0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"
1½" c/c	2	4·72	3·43	2·53	1·96	1·59	1·33	1·14	1·00	0·89	0·80	0·73	0·67	0·62	0·57	0·53
	3	7·07	5·87	4·62	3·69	3·03	2·55	2·20	1·93	1·71	1·54	1·40	1·28	1·18	1·10	1·02
	4	9·43	8·35	6·94	5·72	4·78	4·07	3·53	3·11	2·78	2·50	2·28	2·09	1·93	1·79	1·67
	5	11·8	10·8	9·36	7·95	6·78	5·85	5·12	4·53	4·06	3·67	3·35	3·07	2·84	2·64	2·46
2" c/c	2	4·72	3·40	2·53	1·99	1·63	1·37	1·19	1·05	0·93	0·84	0·77	0·70	0·65	0·60	0·56
	3	7·07	5·80	4·58	3·68	3·04	2·58	2·23	1·96	1·75	1·58	1·44	1·32	1·22	1·13	1·06
	4	9·43	8·26	6·86	5·67	4·76	4·08	3·55	3·13	2·80	2·53	2·30	2·11	1·95	1·81	1·69
	5	11·8	10·7	9·26	7·88	6·74	5·83	5·11	4·54	4·07	3·68	3·36	3·09	2·86	2·66	2·48
	6	14·2	13·2	11·7	10·2	8·90	7·80	6·89	6·15	5·54	5·03	4·61	4·24	3·93	3·66	3·42

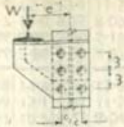
	No. of Horz. Rows	0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"
		SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)														
⅝" dia. Rivets at 2¼" c/c	1	3·68	1·95	1·33	1·00	0·81	0·68	0·58	0·51	0·45	0·41	0·37	0·34	0·32	0·29	0·27
	2	7·36	5·31	3·98	3·15	2·59	2·19	1·90	1·67	1·49	1·35	1·23	1·13	1·05	0·97	0·91
	3	11·0	9·00	7·13	5·76	4·77	4·06	3·52	3·10	2·72	2·50	2·28	2·09	1·93	1·79	1·68
	4	14·7	12·8	10·7	8·84	7·44	6·38	5·55	4·91	4·39	3·97	3·62	3·33	3·07	2·86	2·67
	5	18·4	16·7	14·4	12·3	10·5	9·10	7·98	7·09	6·37	5·77	5·27	4·84	4·49	4·17	3·90
	6	22·1	20·5	18·2	15·9	13·9	12·1	10·7	9·60	8·65	7·86	7·20	6·63	6·14	5·72	5·35
¾" dia. Rivs., at 2½" c/c	1	5·30	2·94	2·04	1·56	1·26	1·06	0·91	0·80	0·72	0·65	0·59	0·54	0·50	0·47	0·43
	2	10·6	7·66	5·78	4·59	3·79	3·22	2·80	2·47	2·21	2·00	1·82	1·68	1·55	1·45	1·36
	3	15·9	12·9	10·3	8·32	6·92	5·90	5·12	4·52	4·04	3·65	3·33	3·06	2·83	2·63	2·46
	4	21·2	18·4	15·3	12·7	10·7	9·21	8·04	7·12	6·37	5·77	5·26	4·84	4·47	4·16	3·89
	5	26·5	23·9	20·6	17·6	15·1	13·1	11·5	10·2	9·20	8·34	7·62	7·01	6·49	6·04	5·65
	6	31·8	29·5	26·2	22·8	19·9	17·5	15·5	13·8	12·5	11·3	10·4	9·58	8·88	8·27	7·74
⅞" dia. Rivs. at 2¾" c/c	2	10·6	7·68	5·84	4·66	3·86	3·29	2·87	2·53	2·27	2·06	1·88	1·73	1·60	1·49	1·39
	3	15·9	12·9	10·3	8·36	6·98	5·96	5·19	4·59	4·11	3·72	3·39	3·12	2·89	2·68	2·51
	4	21·2	18·3	15·3	12·7	10·8	9·25	8·09	7·17	6·43	5·82	5·31	4·89	4·52	4·21	3·93
	5	26·5	23·9	20·6	17·6	15·1	13·1	11·5	10·3	9·23	8·38	7·66	7·05	6·53	6·08	5·69
	6	31·8	29·4	26·1	22·8	19·9	17·5	15·5	13·8	12·5	11·4	10·4	9·61	8·91	8·30	7·77
	7	37·1	34·9	31·6	28·2	24·9	22·2	19·8	17·8	16·2	14·8	13·6	12·5	11·6	10·8	10·2
	8	42·4	40·4	37·2	33·7	30·2	27·1	24·5	22·1	20·2	18·5	17·0	15·8	14·7	13·7	12·9

ECCENTRIC LOADS ON RIVETED BRACKETS

Load W on same plane as Rivet Group.

Rivet Centres (Vertical) 3"

Shearing Value of Rivets 6 tons/sq. in.



3¼" c/c suitable for 5½" Flange
3½" c/c " 6" "
3¾" c/c " 6½" "

¾" dia. Rivets at 3¼" c/c

No. of Horz. Rows	SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)														
	0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"
1	5·30	3·28	2·38	1·86	1·53	1·30	1·13	1·00	0·90	0·81	0·74	0·68	0·63	0·59	0·55
2	10·6	7·75	5·97	4·82	4·03	3·45	3·02	2·68	2·41	2·18	2·00	1·84	1·71	1·59	1·49
3	15·9	12·9	10·3	8·46	7·11	6·11	5·34	4·74	4·25	3·85	3·52	3·24	3·01	2·80	2·62
4	21·2	18·2	15·2	12·7	10·8	9·36	8·21	7·29	6·55	5·95	5·44	5·01	4·64	4·32	4·04
5	26·5	23·7	20·5	17·5	15·1	13·2	11·6	10·4	9·33	8·48	7·77	7·16	6·64	6·19	5·79
6	31·8	29·2	25·9	22·6	19·8	17·4	15·5	13·9	12·6	11·4	10·5	9·69	8·99	8·38	7·85
7	37·1	34·7	31·4	28·0	24·8	22·1	19·8	17·8	16·2	14·8	13·6	12·6	11·7	10·9	10·2
8	42·3	40·2	37·0	33·5	30·1	27·0	24·4	22·1	20·2	18·5	17·1	15·8	14·7	13·8	12·9
9	47·7	45·7	42·6	39·1	35·5	32·2	29·3	26·7	24·5	22·5	20·8	19·3	18·0	16·9	15·9

SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)

Rivets	No. of Horz. Rows	0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"
$\frac{3}{4}$" dia. Rivets at $3\frac{1}{4}$" c/c	1	5·30	3·37	2·47	1·95	1·61	1·37	1·19	1·06	0·95	0·86	0·79	0·73	0·67	0·63	0·59
	2	10·6	7·82	6·07	4·92	4·13	3·55	3·10	2·75	2·49	2·25	2·07	1·89	1·77	1·66	1·55
	3	15·9	12·8	10·4	8·52	7·18	6·19	5·43	4·81	4·33	3·93	3·60	3·31	3·07	2·86	2·68
	4	21·2	18·2	15·2	12·8	10·9	9·41	8·28	7·37	6·62	6·00	5·50	5·07	4·70	4·39	4·10
	5	26·5	23·7	20·4	17·5	15·1	13·2	11·7	10·4	9·39	8·54	7·83	7·22	6·69	6·24	5·84
	6	31·8	29·2	25·8	22·6	19·7	17·4	15·5	13·9	12·6	11·5	10·5	9·74	9·04	8·44	7·90
	7	37·1	34·7	31·4	27·8	24·8	22·1	19·8	17·8	16·2	14·8	13·6	12·6	11·7	11·0	10·3
	8	42·4	40·2	36·9	33·4	30·0	27·0	24·4	22·1	20·2	18·5	17·1	15·8	14·7	13·8	12·9
	9	47·7	45·6	42·5	39·0	35·4	32·1	29·2	26·7	24·4	22·5	20·8	19·3	18·0	16·9	15·9
$\frac{7}{8}$" dia. Rivs. at $3\frac{3}{4}$ c/c	1	7·22	4·71	3·49	2·78	2·30	1·97	1·72	1·53	1·37	1·24	1·14	1·05	0·98	0·91	0·85
	2	14·4	10·7	8·35	6·80	5·72	4·93	4·33	3·85	3·47	3·16	2·90	2·68	2·49	2·32	2·18
	3	21·7	17·5	14·2	11·7	9·90	8·55	7·50	6·68	6·01	5·46	5·00	4·61	4·28	3·99	3·74
	4	28·9	24·7	20·7	17·4	14·9	12·9	11·4	10·1	9·13	8·30	7·60	7·01	6·50	6·06	5·67
	5	36·1	32·2	27·8	23·9	20·6	18·0	16·0	14·3	12·9	11·7	10·8	9·92	9·21	8·59	8·04
	6	43·3	39·7	35·1	30·7	27·0	23·8	21·2	19·0	17·2	15·7	14·4	13·3	12·4	11·6	10·8
	7	50·5	47·2	42·6	38·0	33·7	30·1	27·0	24·3	22·1	20·2	18·6	17·2	16·0	15·0	14·1
	8	57·8	54·6	50·2	45·4	40·8	36·7	33·2	30·1	27·5	25·2	23·3	21·6	20·1	18·8	17·7
	9	65·0	62·1	57·8	53·0	48·2	43·7	39·8	36·3	33·3	30·7	28·4	26·4	24·6	23·1	21·7

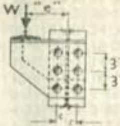
N.B.—*All Bracket Plates to be $\frac{3}{8}$" minimum thickness. It may be necessary to stiffen inclined edge of Plate to prevent Buckling.*

ECCENTRIC LOADS ON RIVETED BRACKETS

Load W on same plane as Rivet Group.

Rivet Centres (Vertical) 3"

Shearing Value of Rivets 6 tons/sq. in.



4" c/c suitable for 7" Flange
$4\frac{1}{2}$" c/c „ $7\frac{1}{2}$" „
$4\frac{3}{4}$" c/c „ 8" „

$\frac{7}{8}$" dia. Rivets at 4" c/c

No. of Horz. Rows	SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches) 0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"
1	7·22	4·81	3·61	2·89	2·41	2·06	1·81	1·60	1·44	1·31	1·20	1·11	1·03	0·96	0·90
2	14·4	10·8	8·45	6·92	5·84	5·04	4·43	3·96	3·57	3·25	2·99	2·76	2·56	2·39	2·25
3	21·7	17·5	14·2	11·8	10·0	8·66	7·62	6·79	6·12	5·57	5·10	4·71	4·37	4·08	3·82
4	28·9	24·7	20·7	17·5	15·0	13·0	11·5	10·2	9·24	8·40	7·70	7·11	6·60	6·15	5·76
5	36·1	32·1	27·7	23·9	20·7	18·1	16·0	14·4	13·0	11·8	10·8	10·0	9·29	8·67	8·12
6	43·3	39·6	35·1	30·7	27·0	23·8	21·2	19·1	17·3	15·8	14·5	13·4	12·5	11·6	10·9
7	50·5	47·1	42·5	37·9	33·7	30·1	27·0	23·3	22·2	20·3	18·7	17·3	16·1	15·1	14·1
8	57·8	54·5	50·1	45·3	40·8	36·7	33·2	30·1	27·5	25·3	23·3	21·6	20·2	18·9	17·7
9	65·0	62·0	57·7	52·8	48·1	43·7	39·7	36·3	33·3	30·7	28·4	26·4	24·6	23·1	21·7

Rivets	No. of Horz. Rows	0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"
		SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)														
⅞" dia. Rivets at 4½" c/c	1	7·22	5·00	3·82	3·10	2·60	2·24	1·97	1·76	1·59	1·44	1·33	1·23	1·14	1·07	1·00
	2	14·4	10·9	8·66	7·15	6·07	5·27	4·65	4·17	3·77	3·44	3·16	2·93	2·72	2·55	2·39
	3	21·7	17·6	14·4	12·0	10·3	8·91	7·87	7·04	6·36	5·80	5·32	4·92	4·57	4·27	4·01
	4	28·9	24·7	20·8	17·6	15·2	13·2	11·7	10·5	9·47	8·64	7·93	7·33	6·81	6·35	5·96
	5	36·1	32·0	27·7	23·9	20·8	18·3	16·2	14·6	13·2	12·0	11·1	10·2	9·50	8·86	8·31
	6	43·3	39·5	34·9	30·7	27·0	23·9	21·4	19·2	17·5	16·0	14·7	13·6	12·7	11·8	11·1
	7	50·5	46·9	42·4	37·8	33·7	30·1	27·0	24·5	22·3	20·4	18·8	17·5	16·3	15·2	14·3
	8	57·8	54·4	49·9	45·1	40·7	36·6	33·2	30·2	27·6	25·4	23·4	21·8	20·3	19·0	17·9
	9	65·0	61·8	57·5	52·6	47·9	43·6	39·7	36·3	33·3	30·7	28·5	26·5	24·7	23·2	21·8
⅞" dia. Rvs. at 4⅝" c/c	1	7·22	5·08	3·92	3·19	2·69	2·32	2·05	1·83	1·65	1·51	1·39	1·28	1·19	1·12	1·05
	2	14·4	11·0	8·77	7·27	6·20	5·39	4·77	4·27	3·87	3·53	3·25	3·01	2·80	2·62	2·46
	3	21·7	17·6	14·5	12·1	10·4	9·04	8·00	7·16	6·48	5·91	5·44	5·03	4·68	4·36	4·10
	4	28·9	24·6	20·8	17·6	15·2	13·3	11·8	10·6	9·54	8·70	7·99	7·39	6·87	6·45	6·02
	5	36·1	32·0	27·7	23·9	20·9	18·4	16·3	14·7	13·3	12·1	11·2	10·3	9·60	8·97	8·41
	6	43·3	39·4	34·9	30·7	27·0	24·0	21·4	19·3	17·6	16·1	14·8	13·7	12·7	11·9	11·2
	7	50·5	46·8	42·3	37·8	33·7	30·2	27·1	24·5	22·4	20·5	18·9	17·5	16·4	15·3	14·4
	8	57·8	54·3	49·8	45·1	40·6	36·6	33·2	30·2	27·6	25·4	23·5	21·8	20·4	19·1	17·9
	9	65·0	61·8	57·4	52·4	47·8	43·5	39·7	36·3	33·3	30·8	28·5	26·5	24·8	23·3	21·9

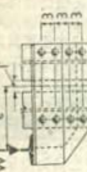
N.B.—All Bracket Plates to be ⅜" minimum thickness. It may be necessary to stiffen inclined edge of Plate to prevent Buckling.

ECCENTRIC LOADS ON RIVETED BRACKETS

Load W on same plane as Rivet Group.

Rivet Centres (Vertical) 3"

Shearing Value of Rivets 6 tons/sq. in.



7½" c/c for 4" Flanges

8" c/c " 4½" "

8¾" c/c " 5" "

¾" dia. Rivets at 7½" c/c

No. of Horz. Rows	SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)														
	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"	15"
1	2·88	2·37	2·01	1·75	1·55	1·39	1·26	1·15	1·06	0·98	0·91	0·85	0·80	0·76	0·72
2	5·94	4·96	4·25	3·72	3·30	2·97	2·69	2·46	2·27	2·11	1·96	1·84	1·73	1·63	1·55
3	9·20	7·81	6·74	5·92	5·26	4·73	4·29	3·93	3·62	3·35	3·12	2·92	2·75	2·59	2·45
4	12·7	10·9	9·50	8·37	7·46	6·71	6·09	5·57	5·13	4·75	4·42	4·13	3·88	3·66	3·46
5	16·2	14·2	12·5	11·1	9·93	8·95	8·14	7·44	6·86	6·35	5·91	5·52	5·18	4·88	4·61
6	19·9	17·8	15·8	14·1	12·7	11·5	10·4	9·56	8·81	8·16	7·60	7·10	6·67	6·28	5·94
7	23·7	21·4	19·2	17·3	15·6	14·2	13·0	11·9	11·0	10·2	9·51	8·89	8·35	7·87	7·44
8	27·4	25·1	22·8	20·7	18·8	17·2	15·7	14·5	13·4	12·5	11·6	10·9	10·2	9·65	9·13
9	31·2	28·9	26·5	24·2	22·2	20·3	18·7	17·3	16·0	14·9	13·5	13·1	12·3	11·6	11·0

	No. of Horz. Rows	SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)														
		1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"	15"
¾" dia. Rivets at 8" c/c	1	4·24	3·53	3·03	2·65	2·36	2·12	1·93	1·77	1·63	1·51	1·41	1·33	1·25	1·18	1·12
	2	8·68	7·32	6·33	5·57	4·97	4·48	4·08	3·75	3·46	3·22	3·01	2·82	2·66	2·51	2·38
	3	13·4	11·4	9·95	8·78	7·84	7·08	6·45	5·92	5·47	5·08	4·74	4·44	4·18	3·95	3·74
	4	18·3	15·9	13·9	12·3	11·0	9·97	9·08	8·33	7·69	7·14	6·66	6·24	5·86	5·53	5·24
	5	23·4	20·6	18·3	16·3	14·6	13·2	12·0	11·1	10·2	9·46	8·82	8·26	7·77	7·32	6·93
	6	28·7	25·6	22·9	20·5	18·5	16·8	15·3	14·1	13·0	12·1	11·3	10·5	9·90	9·34	8·83
	7	34·0	30·8	27·8	25·1	22·7	20·7	18·9	17·4	16·1	15·0	14·0	13·1	12·3	11·6	11·0
	8	39·4	36·1	32·9	29·9	27·3	24·9	22·9	21·1	19·6	18·2	17·0	16·0	15·0	14·2	13·4
	9	44·8	41·5	38·1	35·0	32·0	29·4	27·1	25·1	23·3	21·7	20·3	19·1	18·0	17·0	16·1
¾" dia. Rvs. at 8¾" c/c	1	4·31	3·64	3·14	2·77	7·47	2·23	2·04	1·87	1·73	1·61	1·51	1·42	1·33	1·26	1·20
	2	8·79	7·49	6·51	5·76	5·16	4·68	4·27	3·93	3·64	3·39	3·17	2·98	2·81	2·66	2·52
	3	13·5	11·6	10·2	9·02	8·09	7·34	6·70	6·17	5·71	5·32	4·97	4·67	4·40	4·16	3·94
	4	18·4	16·0	14·1	12·6	11·3	10·3	9·38	8·63	7·98	7·42	6·94	6·51	6·13	5·79	5·49
	5	23·5	20·8	18·4	16·5	14·9	13·5	12·3	11·4	10·5	9·77	9·13	8·56	8·06	7·61	7·20
	6	28·7	25·7	23·0	20·7	18·7	17·1	15·6	14·4	13·3	12·4	11·6	10·9	10·2	9·64	9·13
	7	34·0	30·8	27·9	25·2	22·9	20·9	19·2	17·7	16·4	15·3	14·3	13·4	12·6	11·9	11·3
	8	39·4	36·1	32·9	30·0	27·4	25·1	23·1	21·4	19·9	18·5	17·3	16·3	15·3	14·5	13·7
	9	44·7	41·4	38·1	34·9	32·1	29·5	27·2	25·2	23·5	21·9	20·5	19·3	18·2	17·2	16·3

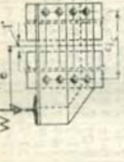
N.B.—All Bracket Plates to be ⅜" minimum thickness. It may be necessary to stiffen inclined edge of Plate to prevent Buckling.

ECCENTRIC LOADS ON RIVETED BRACKETS

Load W on same plane as Rivet Group.

Rivet Centres (Vertical) 3"

Shearing Value of Rivets 6 tons/sq. in.



10½" c/c for 6" Flanges
12" c/c „ 7" „
13¾" c/c „ 8" „

⅞" dia. Rivets at 10½" c/c

SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)

No. of Horz. Rows	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"	15"
1	4·45	3·84	3·37	3·01	2·71	2·47	2·27	2·10	1·95	1·82	1·71	1·61	1·52	1·45	1·37
2	9·00	7·81	6·90	6·17	5·58	5·09	4·68	4·34	4·03	3·77	3·54	3·34	3·16	2·99	2·85
3	13·7	12·0	10·7	9·55	8·65	7·91	7·27	6·73	6·26	5·86	5·50	5·18	4·89	4·64	4·41
4	18·6	16·4	14·6	13·2	11·9	10·9	10·0	9·29	8·64	8·07	7·57	7·13	6·73	6·38	6·06
5	23·6	21·1	18·9	17·1	15·5	14·2	13·1	12·1	11·2	10·5	9·84	9·26	8·74	8·28	7·86
6	28·7	25·9	23·4	21·2	19·4	17·7	16·4	15·1	14·1	13·2	12·3	11·6	11·0	10·4	9·84
7	34·0	31·0	28·2	25·7	23·5	21·6	19·9	18·5	17·2	16·1	15·1	14·2	13·4	12·7	12·0
8	39·3	36·1	33·1	30·3	27·9	25·7	23·8	22·1	20·6	19·2	18·1	17·0	16·1	15·2	14·4
9	44·6	41·4	38·2	35·2	32·4	30·0	27·8	25·9	24·1	22·6	21·2	20·0	18·9	17·9	17·0

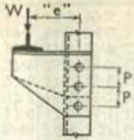
	No. of Horz. Rows.	SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches) 1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"	15"
$\frac{7}{8}$" dia. Rivets at 12" c/c	1	6·19	5·42	4·81	4·33	3·94	3·61	3·33	3·09	2·89	2·71	2·55	2·41	2·28	2·17	2·06
	2	12·5	11·0	9·78	8·83	8·04	7·38	6·82	6·34	5·92	5·55	5·23	4·94	4·68	4·45	4·24
	3	18·9	16·8	15·0	13·6	12·4	11·4	10·5	9·77	9·13	8·56	8·06	7·61	7·21	6·85	6·53
	4	25·5	22·8	20·5	18·6	17·0	15·6	14·4	13·4	12·5	11·7	11·0	10·4	9·87	9·37	8·92
	5	32·3	29·1	26·3	23·9	21·9	20·1	18·6	17·3	16·1	15·1	14·2	13·4	12·7	12·1	11·5
	6	39·3	35·6	32·4	29·5	27·1	25·0	23·1	21·5	20·1	18·8	17·7	16·7	15·8	15·0	14·3
	7	46·3	42·4	38·7	35·5	32·6	30·1	27·9	26·0	24·3	22·8	21·4	20·2	19·1	18·2	17·3
	8	53·5	49·3	45·3	41·7	38·5	35·6	33·1	30·8	28·8	27·1	25·5	24·0	22·7	21·6	20·5
	9	60·8	56·4	52·2	48·3	44·7	41·5	38·7	36·1	33·8	31·7	29·9	28·2	26·7	25·4	24·1
$\frac{7}{8}$" dia. Rvs. at 13$\frac{3}{4}$" c/c	1	6·30	5·59	5·02	4·56	4·17	3·85	3·57	3·33	3·12	2·94	2·77	2·62	2·49	2·37	2·26
	2	12·7	11·3	10·2	9·25	8·48	7·83	7·27	6·79	6·37	5·99	5·66	5·36	5·09	4·85	4·63
	3	19·2	17·2	15·5	14·1	13·0	12·0	11·1	10·4	9·75	9·18	8·67	8·21	7·80	7·43	7·09
	4	25·8	23·2	21·1	19·2	17·7	16·4	15·2	14·2	13·3	12·5	11·8	11·2	10·6	10·1	9·65
	5	32·6	29·5	26·9	24·6	22·6	21·0	19·5	18·2	17·1	16·0	15·1	14·3	13·6	12·9	12·3
	6	39·4	36·0	32·9	30·2	27·9	25·8	24·0	22·5	21·1	19·8	18·7	17·7	16·8	16·0	15·2
	7	46·5	42·7	39·2	36·1	33·4	31·0	28·9	27·0	25·3	23·8	22·5	21·3	20·2	19·2	18·3
	8	53·5	49·5	45·7	42·3	39·2	36·4	34·0	31·8	29·8	28·1	26·5	25·1	23·8	22·6	21·5
	9	60·7	56·5	52·5	48·8	45·4	42·3	39·5	37·0	34·8	32·8	31·0	29·3	27·8	26·5	25·2

N.B.—All Bracket Plates to be $\frac{3}{8}$" minimum thickness. It may be necessary to stiffen inclined edge of Plate to prevent Buckling.

ECCENTRIC LOADS ON RIVETED BRACKETS

Load W on same plane as Rivet Group.

Shearing Value of Rivets 6 tons/sq. in.



P – Vertical Centres of Rivets in inches.

⅝" diam. Rivets	No. of Rivets	SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)														
		0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"
P = 2½"	3	5·52	4·73	3·53	2·68	2·12	1·75	1·48	1·28	1·13	1·01	0·91	0·83	0·76	0·70	0·65
	4	7·36	6·63	5·31	4·20	3·40	2·83	2·41	2·10	1·85	1·66	1·50	1·37	1·26	1·16	1·08
	5	9·20	8·54	7·18	5·89	4·88	4·11	3·54	3·09	2·74	2·46	2·23	2·04	1·88	1·74	1·62
	6	11·0	10·4	9·11	7·70	6·50	5·56	4·83	4·25	3·78	3·40	3·09	2·83	2·61	2·42	2·25
P = 3"	3	5·52	4·94	3·90	3·06	2·47	2·05	1·75	1·52	1·34	1·20	1·08	0·99	0·91	0·84	0·78
	4	7·36	6·83	5·75	4·71	3·90	3·29	2·83	2·48	2·22	1·97	1·78	1·63	1·50	1·39	1·29
	5	9·20	8·73	7·66	6·51	5·52	4·73	4·11	3·62	3·23	2·91	2·64	2·42	2·23	2·07	1·93
	6	11·0	10·6	9·59	8·38	7·27	6·33	5·56	4·94	4·43	4·00	3·65	3·35	3·09	2·87	2·68
	7	12·9	12·5	11·5	10·3	9·11	8·05	7·14	6·39	5·76	5·23	4·78	4·40	4·07	3·79	3·54

¾" diam. Rivets	No. of Rivets	0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"
		SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)														
P = 3"	3	7·96	7·11	5·62	4·41	3·56	2·95	2·51	2·18	1·93	1·72	1·56	1·42	1·31	1·21	1·12
	4	10·7	9·83	8·28	6·79	5·62	4·74	4·08	3·57	3·16	2·84	2·58	2·35	2·16	2·00	1·86
	5	13·2	12·6	11·0	9·37	7·95	6·82	5·93	5·22	4·65	4·19	3·81	3·49	3·21	2·98	2·78
	6	16·0	15·3	13·8	12·1	10·4	9·12	8·03	7·11	6·37	5·76	5·25	4·82	4·45	4·13	3·86
	7	18·6	18·0	16·6	14·9	13·1	11·6	10·3	9·20	8·30	7·53	6·89	6·34	5·86	5·46	5·10
	8	21·2	20·8	19·4	17·6	15·9	14·2	12·7	11·5	10·4	9·48	8·70	8·03	7·44	6·94	6·49
P = 3½"	3	7·96	7·31	6·04	4·88	4·01	3·36	2·88	2·51	2·23	2·00	1·81	1·65	1·52	1·40	1·31
	4	10·7	10·0	8·75	7·40	6·26	5·35	4·65	4·09	3·64	3·28	2·98	2·73	2·51	2·32	2·17
	5	13·2	12·7	11·5	10·0	8·73	7·60	6·68	5·93	5·31	4·80	4·38	4·02	3·71	3·44	3·21
	6	16·0	15·4	14·2	12·9	11·3	10·0	9·00	8·01	7·23	6·57	6·01	5·53	5·12	4·76	4·45
	7	18·5	18·1	17·0	15·7	14·1	12·7	11·4	10·3	9·35	8·54	7·85	7·25	6·73	6·27	5·87
	8	21·2	20·9	19·9	18·4	16·9	15·3	14·0	12·8	11·6	10·7	9·85	9·13	8·50	7·94	7·44
	9	23·8	23·5	22·5	21·2	19·7	18·1	16·6	15·2	14·0	13·0	12·0	11·1	10·4	9·76	9·17
⅞" dia. Rivets P = 4"	3	10·8	10·1	8·66	7·20	6·01	5·10	4·40	3·86	3·42	3·08	2·79	2·55	2·35	2·18	2·03
	4	14·4	13·8	12·3	10·7	9·23	8·00	7·00	6·20	5·55	5·00	4·56	4·18	3·86	3·58	3·34
	5	18·0	17·5	16·1	14·4	12·8	11·2	10·0	8·96	8·07	7·33	6·70	6·17	5·71	5·31	4·96
	6	21·7	21·1	20·0	18·2	16·4	14·8	13·3	12·0	11·0	9·97	9·16	8·46	7·85	7·32	6·85
	7	25·2	24·9	23·7	22·0	20·2	18·4	16·8	15·3	14·0	12·9	11·9	11·0	10·2	9·58	8·99
	8	28·9	28·4	27·4	25·9	24·0	22·2	20·4	18·9	17·3	16·0	14·9	13·8	13·0	12·1	11·3
	9	32·5	32·1	31·1	29·6	27·9	26·0	24·1	22·4	20·8	19·3	18·0	16·8	15·8	14·8	14·0

N.B.—All bracket Plates to be ⅜" Minimum Thickness. It may be necessary to stiffen inclined edge of Plate to prevent Buckling.

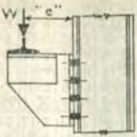
ECCENTRIC LOADS ON BOLTED BRACKETS

TWO VERTICAL ROWS

Load W not on same plane as Bolt Group.

P – Vertical Centres of Bolts in ins.

c/c of Bolts can vary.



See note on page 300

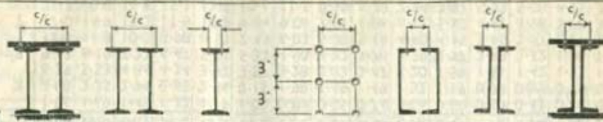
[illegible]" dia. Bolts P – 3"

No. of Horz. Rows		SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)														
		6"	7"	8"	9"	10"	11"	12"	14"	16"	18"	20"	24"	28"	30"	36"
2		0·71	0·61	0·54	0·48	0·43	0·39	0·36	0·31	0·27						
	3	1·42	1·22	1·07	0·95	0·86	0·78	0·72	0·61	0·54	0·48	0·43	0·36	0·31	0·29	
4		2·35	2·02	1·78	1·58	1·43	1·30	1·20	1·02	0·90	0·80	0·72	0·60	0·51	0·48	0·40
	5	3·50	3·02	2·65	2·36	2·13	1·94	1·78	1·53	1·34	1·19	1·08	0·90	0·77	0·72	0·60
6		4·85	4·20	3·70	3·30	2·98	2·71	2·49	2·14	1·88	1·67	1·50	1·25	1·08	1·00	0·84
	7	6·41	5·55	4·90	4·38	3·95	3·61	3·31	2·85	2·49	2·22	2·00	1·67	1·43	1·34	1·12
8		8·13	7·08	6·25	5·59	5·06	4·62	4·25	3·65	3·21	2·85	2·57	2·15	1·84	1·72	1·43
	9	10·0	8·76	7·76	6·95	6·30	5·75	5·29	4·56	4·00	3·56	3·21	2·68	2·30	2·15	1·79
10		12·1	10·6	9·40	8·44	7·65	6·99	6·44	5·55	4·87	4·35	3·92	3·27	2·81	2·63	2·19

Bolts	No. of Horz. Rows	6"	7"	8"	9"	10"	11"	12"	14"	16"	18"	20"	24"	28"	30"	36"
$\frac{3}{4}$" dia. Bolts P = $3\frac{1}{2}$"	2	1·24	1·07	0·93	0·83	0·75	0·68	0·63	0·54	0·47	0·42	0·38	0·31	0·27	0·25	
	3	2·46	2·12	1·86	1·66	1·49	1·36	1·25	1·07	0·94	0·83	0·75	0·63	0·54	0·50	0·42
	4	4·08	3·52	3·10	2·76	2·49	2·27	2·08	1·79	1·56	1·39	1·26	1·05	0·90	0·84	0·70
	5	6·03	5·22	4·60	4·10	3·71	3·38	3·10	2·67	2·14	2·08	1·87	1·56	1·34	1·25	1·04
	6	8·32	7·23	6·38	5·71	5·16	4·71	4·33	3·72	3·27	2·91	2·62	2·19	1·88	1·75	1·46
	7	11·0	9·55	8·45	7·57	6·85	6·26	5·76	4·95	4·35	3·87	3·49	2·91	2·50	2·34	1·95
	8	13·8	12·1	10·8	9·65	8·74	7·99	7·35	6·34	5·57	4·96	4·48	3·74	3·21	2·99	2·50
	9	17·0	14·9	13·3	12·0	10·9	9·93	9·14	7·89	6·94	6·19	5·58	4·67	4·01	3·74	3·12
	10	20·4	18·0	16·1	14·5	13·2	12·1	11·1	9·61	8·46	7·55	6·81	5·70	4·90	4·58	3·82
$\frac{7}{8}$" dia. Bolts P = 4"	2	1·96	1·69	1·48	1·32	1·19	1·08	0·99	0·85	0·74	0·66	0·60	0·50	0·43	0·40	0·33
	3	3·88	3·35	2·94	2·62	2·36	2·15	1·98	1·69	1·49	1·32	1·19	0·99	0·85	0·79	0·66
	4	6·39	5·53	4·86	4·34	3·92	3·57	3·28	2·82	2·47	2·20	1·98	1·65	1·42	1·32	1·10
	5	9·43	8·19	7·23	6·47	5·85	5·33	4·90	4·22	3·70	3·29	2·97	2·48	2·12	1·98	1·65
	6	13·0	11·3	10·0	8·98	8·13	7·43	6·83	5·88	5·16	4·60	4·15	3·46	2·97	2·77	2·31
	7	17·0	14·9	13·2	11·9	10·8	9·84	9·06	7·81	6·86	6·12	5·52	4·61	3·96	3·70	3·08
	8	21·3	18·8	16·8	15·1	13·7	12·6	11·6	9·99	8·79	7·84	7·07	5·92	5·08	4·75	3·96
	9	26·0	23·0	20·6	18·6	16·9	15·5	14·3	12·4	10·9	9·76	8·82	7·38	6·34	5·92	4·95
	10	31·1	27·7	24·9	22·5	20·5	18·9	17·4	15·1	13·3	11·9	10·8	9·00	7·74	7·23	6·04

Loads are calculated for Black Bolts having a Permissible Tensile Value of 5 Tons/sq. in. and Shearing Value of 4 tons/sq. in. Note.—All Bracket Plates to be $\frac{3}{8}$" minimum thickness; It may be necessary to stiffen inclined edge of Plate to prevent Buckling.

Tables of MODULI VALUES $\left(\frac{\Sigma d^2}{y}\right)$ Suitable for 2 lines of Rivets or Bolts at 3" Vertical centres and varying c/c, and for loads acting on same plane as rivet group

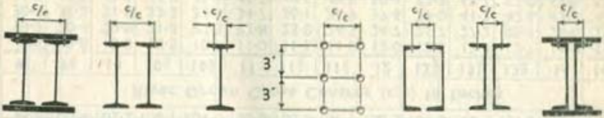


No. of Horiz. Rows (3" Crs.)	Rivet Group Cross Centres (c/c) in Inches													
	2½	2¾	3¼	3½	3¾	3⅞	4	4¼	4⅜	4½	4¾	4⅞	5	5¼
1	2·50	2·75	3·25	3·50	3·75	3·87	4·0	4·25	4·38	4·5	4·75	4·88	5·0	5·25
2	7·81	8·14	8·84	9·22	9·61	9·80	10·0	10·4	10·6	10·8	11·2	11·5	11·7	12·1
3	14·0	14·3	15·2	15·6	16·2	16·4	16·6	17·2	17·4	17·7	18·2	18·5	18·8	19·4
4	22·0	22·3	23·2	23·7	24·2	24·5	24·8	25·3	25·6	25·9	26·3	26·9	27·2	27·9
5	31·9	32·3	33·2	33·7	34·2	34·5	34·8	35·4	35·7	36·0	36·6	37·0	37·3	38·0
6	43·9	44·3	45·2	45·7	46·2	46·5	46·8	47·4	47·7	48·0	48·6	49·0	49·3	50·0
7	57·9	58·3	59·2	59·6	60·2	60·5	60·7	61·3	61·6	62·0	62·6	63·0	63·3	64·0
8	73·9	74·2	75·1	75·6	76·2	76·4	76·7	77·3	77·6	78·0	78·6	79·0	79·3	80·0
9	91·9	92·2	93·1	93·6	94·1	94·4	94·7	95·3	95·6	96·0	96·6	97·0	97·3	98·0

No. of Horiz. Rows (3" Crs.)	Rivet Group Cross Centres (c/c) in Inches													
	5½	6	6½	6¾	7	7¼	7½	7¾	8	8¼	8½	8¾	9	9¼
1	5·5	6·0	6·5	6·8	7·0	7·3	7·5	7·8	8·0	8·3	8·5	8·8	9·0	9·3
2	12·5	13·4	14·3	14·8	15·2	15·7	16·2	16·6	17·1	17·6	18·0	18·5	19·0	19·4
3	20·0	21·2	22·5	23·1	23·8	24·4	25·1	25·7	26·4	27·1	27·8	28·4	29·1	29·8
4	28·5	30·0	31·4	32·2	33·0	33·7	34·6	35·4	36·2	37·0	37·9	38·7	39·6	40·5
5	38·7	40·3	41·9	42·7	43·6	44·4	45·3	46·2	47·2	48·1	49·0	50·0	51·0	52·0
6	50·8	52·3	54·0	54·9	55·8	56·7	57·7	58·7	59·6	60·7	61·7	62·7	63·8	64·9
7	64·8	66·4	68·1	69·0	70·0	70·9	71·9	72·9	73·9	75·0	76·0	77·1	78·3	79·4
8	80·8	82·4	84·2	85·1	86·0	87·0	88·0	89·0	90·1	91·2	92·3	93·4	94·5	95·7
9	98·8	100·4	102·2	103·1	104·0	105·0	106·0	107·1	108·2	109·3	110·4	111·5	112·7	113·9

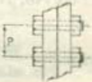
No. of Horiz. Rows (3" Crs.)	Rivet Group Cross Centres (c/c) in Inches													
	9½	9¾	10	10½	10¾	11	11½	11¾	12	12½	13¼	13¾	14	14¼
1	9·5	9·8	10·0	10·5	10·8	11·0	11·5	11·8	12·0	12·5	13·3	13·7	14·0	14·3
2	19·9	20·4	20·9	21·8	22·3	22·8	23·8	24·3	24·7	25·7	27·2	28·1	28·6	29·1
3	30·5	31·2	31·9	33·3	34·0	34·7	36·1	36·9	37·6	39·0	41·2	42·6	43·3	44·1
4	41·3	42·2	43·1	44·8	45·8	46·7	48·6	49·5	50·4	52·3	55·1	57·0	57·9	58·9
5	53·0	54·0	55·1	57·1	58·2	59·3	61·4	62·5	63·6	65·9	69·2	71·5	72·7	73·8
6	66·0	67·1	68·2	70·5	71·7	72·9	75·3	76·5	77·8	80·3	84·1	86·7	88·0	89·3
7	80·6	81·7	82·9	85·4	86·7	87·9	90·5	91·9	93·2	95·9	100·1	103·0	104·4	105·8
8	96·9	102·0	99·4	102·0	103·3	104·6	107·3	108·7	110·0	113·0	117·5	120·5	122·0	123·6
9	115·1	116·4	117·7	120·3	121·7	123·1	125·9	127·3	129·0	131·8	136·4	140·0	141·2	142·9

Tables of MODULI VALUES $\left(\frac{\Sigma d^2}{y}\right)$ Suitable for 2 lines of Rivets or Bolts at 3" Vertical centres and varying c/c, and for loads acting on same plane as rivet group



No. of Horiz. Rows (3" Crs.)	14½	14¾	15	15½	15¾	16	17½	17¾	18	19¼	19¾	20	21	23½
	Rivet Group Cross Centres (c/c) in Inches													
1	14·5	14·8	15·0	15·5	15·8	16·0	17·5	17·8	18·0	19·3	19·8	20·0	21·0	23·5
2	29·6	30·1	30·6	31·6	32·1	32·6	35·5	36·0	36·5	39·0	40·0	40·5	42·4	47·4
3	44·8	45·5	46·2	47·7	48·4	49·2	53·6	54·3	55·0	58·7	60·2	60·9	63·9	71·3
4	59·8	60·8	61·7	63·7	64·6	65·6	71·4	72·4	73·3	78·2	80·2	81·2	85·1	94·9
5	75·0	76·1	77·3	79·6	80·8	82·0	89·1	90·3	91·5	97·5	100·0	101·2	106·0	118·3
6	90·7	92·0	93·3	96·0	97·4	98·8	107·1	108·5	109·9	116·9	119·8	121·2	126·9	141·4
7	107·3	108·8	110·2	113·2	114·7	116·3	125·5	127·1	128·7	136·7	139·9	141·5	148·1	164·6
8	123·8	126·7	128·3	131·6	133·2	134·8	144·9	146·7	148·4	157·1	160·7	162·5	169·7	188·2
9	144·5	146·2	147·9	151·3	153·0	154·8	165·5	167·4	169·2	178·6	182·4	184·4	192·2	212·3

Table of MODULI VALUES $\left(\frac{\Sigma d^2}{y}\right)$
Suitable for one Single line of Bolts of Varying Vertical centres from 2 to 4 inches
Load NOT acting on same plane as bolts



Number of Bolts	Vertical Spacing ' P ' inches				
	2"	2½"	3"	3½"	4"
2	2·0	2·5	3·0	3·5	4·0
3	4·0	5·0	6·0	7·0	8·0
4	6·7	8·3	10·0	11·7	13·3
5	10·0	12·5	15·0	17·5	20·0
6	14·0	17·5	21·0	24·5	28·0
7	18·7	23·3	28·0	32·7	37·3
8	24·0	30·0	36·0	42·0	48·0
9	30·0	37·5	45·0	52·5	60·0
10	36·7	45·8	55·0	64·2	73·3

N.B.—Moduli values for one single line of rivets on the same plane as the load are similar to above and can also be used as such. *See note page 301*

For example using these values see page 321

EXAMPLES OF ECCENTRIC LOADS ON RIVETED BRACKETS

See Sketch No. 1. Page 322.

Example 1 **Eccentric Floor Beam Connections**

Bending Moment $= We = 7T \times 2\frac{1}{2}''$ $= 17{\cdot}5$ in. tons

Force due to Moment $= \dfrac{BM}{\left(\frac{\Sigma d^2}{y}\right)} = \dfrac{17{\cdot}5}{11{\cdot}2}$ $= 1{\cdot}59$ tons

(See page No. 316)

Direct Force $= \dfrac{W}{\text{No. of Rivs.}} = \dfrac{7}{4} = 1{\cdot}75$ tons

The Resultant Force found graphically $= 3{\cdot}18$ tons

From Tables on page No. 294 Single Shear Value of $\frac{7}{8}''$ dia. Rivet (6 tons/sq. in.) is 3·61 tons. Hence assumed Rivet Group is suitable for combined stresses.

See Sketch No. 2. Page 322.

Example 2 **Gantry Girder Connections**

Bending Moment $= We = 8T \times 14''$

" " $= 112$ in. tons

Force due to Moment $= \dfrac{BM}{\left(\frac{\Sigma d^2}{y}\right)} = \dfrac{112}{61{\cdot}4}$ $= 1{\cdot}8$ tons

(See page No. 317)

Direct Force $= \dfrac{W}{\text{No. of Rivs.}} = \dfrac{8}{10} = 0{\cdot}8$ tons

The Resultant Force found graphically $= 2{\cdot}46$ tons

From Tables on page 294 Single Shear Value of $\frac{3}{4}''$ dia. Rivet (6 tons/sq. in.) is 2·65 tons. Hence Rivet Group is suitable for combined stresses.

Example 3. Load not in same plane as Bolt Group
The usual practice of designing this type of Bracket is by assuming that the Rotation tendency or moment of the Bolts connecting the Bracket to Stanchion is about the centre of gravity of the group. This assumption is not strictly correct, as the actual Centre of Rotation will lie somewhere between the foot of the Bracket and the centre of the group. However, it errs on the safe side and is generally accepted. For Moduli Values see page 319.
Bending Moment = We = 5T × 9" = 45 in. tons
Assume a group of 10 Bolts $3\frac{1}{2}$" Vertical Centres.
Modulus of Bolt Group

$$= 2 \times \left(\frac{\Sigma d^2}{y}\right) \text{ (See page 319)} = 2 \times 17{\cdot}5 = 35.$$

Referring to page 362, $M = \dfrac{fI}{y} = fZ.$

$$\therefore ft \text{ (tensile)} = \frac{M}{Z} = \frac{M}{2 \times \left(\frac{\Sigma d^2}{y}\right) \times \text{nett Area of 1 Bolt}}$$

$$\therefore ft = \frac{45}{35 \times 0{\cdot}3038} = 4{\cdot}23 \text{ tons/sq. in.}$$

$$fs \text{ (Shear)} = \frac{W}{NA} = \frac{5}{10 \times 0{\cdot}442} = 1{\cdot}13 \text{ tons/sq. in.}$$

Taking the area of bolt for tensile stress as the cross-sectional area at bottom of thread and "A" as gross area of bolt for the shearing stress, "N" being the number of bolts in the group. Then from the theory of the "Ellipse of Stress" the maximum combined tensile and shearing stresses is given by the formula :

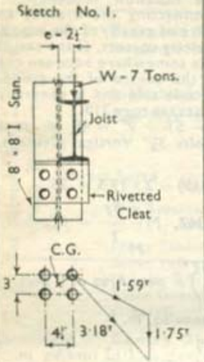
$$\text{Maximum stress} = \frac{ft}{2}\left(1 + \sqrt{1 + \frac{4fs^2}{ft^2}}\right)$$

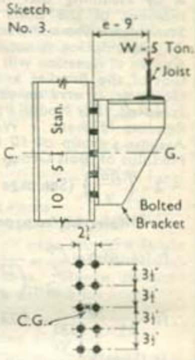
$$\text{,,} \quad = \frac{4{\cdot}23}{2}\left(1 + \sqrt{1 + \frac{4 \times 1{\cdot}13^2}{4{\cdot}23^2}}\right)$$

$$\text{,,} \quad = 4{\cdot}51 \text{ tons/sq. in. (Tension).}$$

Permissible stress = 5·0 tons/sq. in. Hence bolt Group is suitable. **See Sketch No. 3, Page 322.**

SKETCHES FOR EXAMPLES OF BOLTED AND RIVETED BRACKETS on Pages 320-321





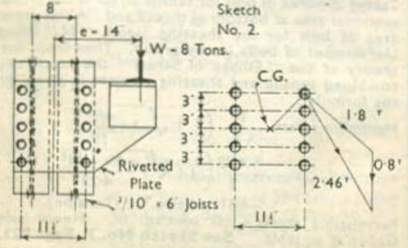


PLATE GIRDERS

Example. To find the Flange Area of Girder having a Bending Moment of 824 foot-tons and a depth of 48 inches and stressed to 10 tons/sq. in. (assuming that the Top Flange is held laterally $L/B < 20$). Max. Reaction 66 tons.

Flange Area

$A = \frac{M \times 12}{D \times Fb}$ A = Nett Flange Area in Square Ins.

$A = \frac{824 \times 12}{48 \times 10}$ M = Bending Moment in Foot-Tons.

$A = 20{\cdot}6$ sq. ins. D = Depth of Girder in ins. over Flange angles.

Fb = Permissible Safe Fibre Stress.

See page 324 for make-up of Flange Area.

Web Thickness

T = Thickness of Web in Ins.

$T = \frac{R}{D \times Fs}$ R = Max. Reaction in Tons.

Fs = Permissible Safe Shear Stress.

$T = \frac{66}{48 \times 5}$

$T = 0{\cdot}275$ ins., say $\frac{3}{8}$ ins.

Alternative Method

Find total nett Inertia of Girder.

$M = \frac{FI}{y}$ (see page 362).

$\therefore$ Nett $I = \frac{My}{Fb} = \frac{12 \times 824 \times 24}{10}$

$\therefore$ Nett $I \doteq 23731$ ins.^{4}

From tables in pages 326 to 337 the Inertia of Flange and Web Sections can be obtained to make up this total.

Note that Inertia of holes must be deducted to give nett value. (See sketch above.)

Table shewing suggested areas of Plate

Girder Depths are measured over

Plt. Girder 2′ 6″ deep (¾″ dia. Rivs.)		Plt. Girder 4′ 0″ deep (¾″ dia. Rivs.)	
⅛th Web Plt.	1·41	⅛th Web Plt.	2·25
2 Ls. 4″ × 3″ × ⅜″ (less 2 holes)	4·36	2 Ls. 4″ × 4″ × ½″ (less 2 holes)	6·69
1 Plt. 10″ × ⅜″ (less 2 holes)	3·14	1 Plt. 12″ × ½″ (less 2 holes)	5·19
1 Plt. 10″ × ½″ (less 2 holes)	4·19	1 Plt. 12″ × ⅝″ (less 2 holes)	6·48
Nett Area, sq. in.	13·10	Nett Area, sq. in.	20·61

Plt. Girder 3′ 0″ deep (¾″ dia. Rivs.)		Plt. Girder 4′ 6″ deep (¾″ dia. Rivs.)	
⅛th Web Plt.	1·69	⅛th Web Plt.	2·53
2 Ls. 4″ × 3″ × ½″ (less 2 holes)	5·69	2 Ls. 5″ × 5″ × ⅜″ (less 2 holes)	6·61
1 Plt. 10″ × ½″ (less 2 holes)	4·19	1 Plt. 14″ × ⅜″ (less 2 holes)	4·64
1 Plt. 10″ × ⅝″ (less 2 holes)	5·23	1 Plt. 14″ × ½″ (less 2 holes)	6·19
Nett Area, sq. in.	16·80	Nett Area, sq. in.	19·97

Plt. Girder 3′ 6″ deep (¾″ dia. Rivs.)		Plt. Girder 5′ 0″ deep (¾″ dia. Rivs.)	
⅛th Web Plt.	1·97	⅛th Web Plt.	2·81
2 Ls. 4″ × 4″ × ⅜″ (less 2 holes)	5·11	2 Ls. 5″ × 5″ × ½″ (less 4 holes)	7·88
1 Plt. 12″ × ⅜″ (less 2 holes)	3·89	1 Plt. 14″ × ½″ (less 2 holes)	6·19
1 Plt 12″ × ½″ (less 2 holes)	5·19	1 Plt. 14″ × ⅝″ (less 2 holes)	7·73
Nett Area, sq. in.	16·16	Nett Area, sq. in.	24·61

These combinations are not necessarily the most
Areas for each of the most commonly

Girder Flanges in square inches.

flange angles. Web plates $\frac{3}{8}$" thick.

Plt. Girder 5′ 6″ deep ($\frac{3}{4}$" dia. Rivs.)

$\frac{1}{8}$th Web Plt.	3·10
2 Ls. 5″ × 5″ × $\frac{5}{8}$″ (less 4 holes)	9·69
1 Plt. 15″ × $\frac{3}{8}$″ (less 2 holes)	5·02
1 Plt. 15″ × $\frac{1}{2}$″ (less 2 holes)	6·69
Nett Area, sq. in.	24·50

Plt. Girder 7′ 0″ deep ($\frac{7}{8}$" dia. Rivs.)

$\frac{1}{8}$th Web Plt.	3·95
2 Ls. 6″ × 6″ × $\frac{5}{8}$″ (less 4 holes)	11·88
1 Plt. 16″ × $\frac{1}{2}$″ (less 2 holes)	7·06
1 Plt. 16″ × $\frac{5}{8}$″ (less 2 holes)	8·82
Nett Area, sq. in.	31·71

Plt. Girder 6′ 0″ deep ($\frac{3}{4}$" dia. Rivs.)

$\frac{1}{8}$th Web Plt.	3·38
2 Ls. 6″ × 6″ × $\frac{3}{8}$″ (less 4 holes)	7·50
1 Plt. 15″ × $\frac{1}{2}$″ (less 2 holes)	6·69
1 Plt. 15″ × $\frac{5}{8}$″ (less 2 holes)	8·36
Nett Area, sq. in.	25·93

Plt. Girder 7′ 6″ deep ($\frac{7}{8}$" dia. Rivs.)

$\frac{1}{8}$th Web Plt.	4·22
2 Ls. 6″ × 6″ × $\frac{3}{4}$″ (less 4 holes)	14·07
1 Plt. 18″ × $\frac{3}{8}$″ (less 2 holes)	6·05
1 Plt. 18″ × $\frac{1}{2}$″ (less 2 holes)	8·06
Nett Area, sq. in.	32·40

Plt. Girder 6′ 6″ deep ($\frac{3}{4}$" dia. Rivs.)

$\frac{1}{8}$th Web Plt.	3·66
2 Ls. 6″ × 6″ × $\frac{1}{2}$″ (less 4 holes)	9·88
1 Plt. 16″ × $\frac{3}{8}$″ (less 2 holes)	5·39
1 Plt. 16″ × $\frac{1}{2}$″ (less 2 holes)	7·19
Nett Area, sq. in.	26·12

Plt. Girder 8′ 0″ deep ($\frac{7}{8}$" dia. Rivs.)

$\frac{1}{8}$th Web Plt.	4·5
2 Ls. 6″ × 6″ × $\frac{3}{4}$″ (less 4 holes)	14·07
1 Plt. 20″ × $\frac{3}{8}$″ (less 2 holes)	6·80
1 Plt. 20″ × $\frac{1}{2}$″ (less 2 holes)	9·06
1 Plt. 20″ × $\frac{5}{8}$″ (less 2 holes)	11·33
Nett Area, sq. in.	45·76

economical but are intended to give a variety of used Angles and Plates.

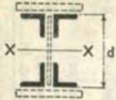
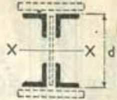


PLATE GIRDERS

Gross Moments of Inertia of 4 Flange Angles (axis x—x)



Angle Section	Thickness	Depth over Flange Angles (d) in inches								
		$36\frac{1}{2}$	$39\frac{1}{2}$	$42\frac{1}{2}$	$45\frac{1}{2}$	$48\frac{1}{2}$	$51\frac{1}{2}$	$54\frac{1}{2}$	$57\frac{1}{2}$	$60\frac{1}{2}$
$3\frac{1}{2} \times 3\frac{1}{2}$	$\frac{5}{16}$	2505	2957	3447	3976	4540	5144	5783	6461	7178
	$\frac{3}{8}$	2975	3513	4095	4723	5395	6113	6875	7681	8533
	$\frac{1}{2}$	3860	4560	5318	6136	7010	7946	8938	9988	11098
4 × 3	$\frac{5}{16}$	2431	2877	3359	3881	4439	5037	5669	6339	7049
	$\frac{3}{8}$	2888	3416	3992	4610	5274	5984	6740	7538	8380
	$\frac{1}{2}$	3746	4436	5186	5990	6856	7778	8760	9802	10900
4 × 3	$\frac{5}{16}$	2566	3026	3520	4052	4624	5230	5876	6562	7282
	$\frac{3}{8}$	3051	3597	4183	4817	5499	6223	6991	7805	8661
	$\frac{1}{2}$	3959	4669	5433	6259	7147	8087	9089	10151	11269

4 × 4	$\frac{5}{16}$ $\frac{3}{8}$ $\frac{1}{2}$	2836 3373 4396	3352 3989 5200	3912 4651 6070	4514 5371 7006	5156 6139 8012	5846 6955 9086	6576 7825 10224	7352 8751 11434	8172 9725 12708
5 × 3½ ┐	$\frac{5}{16}$ $\frac{3}{8}$ $\frac{1}{2}$	2880 3416 4455	3412 4052 5285	3998 4746 6193	4624 5492 7171	5298 6294 8219	6016 7153 9339	6782 8060 10533	7596 9029 11799	8458 10050 13135
5 × 3½ ¬	$\frac{5}{16}$ $\frac{3}{8}$ $\frac{1}{2}$	3120 3704 4830	3680 4370 5702	4282 5088 6642	4934 5862 7656	5632 6690 8740	6374 7576 9896	7164 8514 11126	8000 9506 12426	8880 10558 13800
5 × 4 ┐	$\frac{3}{8}$ $\frac{1}{2}$ $\frac{5}{8}$	3664 4777 5854	4344 5665 6948	5082 6633 8134	5876 7673 9418	6736 8791 10794	7646 9987 12264	8618 11261 13828	9648 12609 15486	10738 14033 17242
5 × 4 ¬	$\frac{3}{8}$ $\frac{1}{2}$ $\frac{5}{8}$	3870 5049 6184	4568 5961 7310	5328 6951 8530	6144 8021 9844	7016 9165 11252	7952 10385 12752	8944 11683 14350	9990 13057 16040	11098 14505 17824

┐ Denotes Flange Angles with small leg outstanding.

¬ Denotes Flange Angles with large leg outstanding.

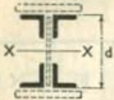
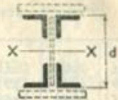


PLATE GIRDERS
Gross
Moments of Inertia
of
4 Flange Angles (axis x—x)



Angle Section	Thickness	Depth over Flange Angles (d) in inches								
		$42\frac{1}{2}$	$45\frac{1}{2}$	$48\frac{1}{2}$	$51\frac{1}{2}$	$54\frac{1}{2}$	$57\frac{1}{2}$	$60\frac{1}{2}$	$72\frac{1}{2}$	$84\frac{1}{2}$
5×5	$\frac{3}{8}$	5740	6634	7592	8614	9708	10862	12078	17602	24166
	$\frac{1}{2}$	7518	8688	9946	11292	12718	14236	15836	23094	31718
	$\frac{5}{8}$	9223	10667	12217	13869	15631	17497	19471	28407	39033
	$\frac{3}{4}$	10880	12586	14416	16374	18454	20662	22992	33562	46136
$6 \times 3\frac{1}{2}$	$\frac{3}{8}$	5751	6623	7555	8551	9605	10723	11901	17233	23553
	$\frac{1}{2}$	7527	8671	9899	11205	12591	14059	15607	22613	30913
	$\frac{5}{8}$	9240	10650	12156	13762	15470	17276	19182	27808	38034

6 × 4 ┐	$\frac{3}{8}$ $\frac{1}{2}$ $\frac{5}{8}$ $\frac{3}{4}$	5455 7131 8753 10319	6323 8275 10157 11981	7259 9499 11667 13765	8259 10811 13285 15677	9323 12211 15005 17713	10455 13695 16833 19871	11649 15265 18763 22157	17079 22397 27547 32547	23551 30897 38019 44935
6 × 4 ┐	$\frac{3}{8}$ $\frac{1}{2}$ $\frac{5}{8}$ $\frac{3}{4}$	5987 7838 9623 11340	6903 9038 11097 13082	7881 10322 12677 14952	8921 11690 14365 16944	10031 13144 16159 19060	11201 14686 18053 21304	12439 16314 20055 23670	18043 23672 29121 34390	24685 32400 39875 47110
6 × 6	$\frac{3}{8}$ $\frac{1}{2}$ $\frac{5}{8}$ $\frac{3}{4}$	6788 8904 10953 12935	7852 10308 12683 14985	9000 11814 14543 17187	10222 13424 16529 19541	11524 15140 18647 22047	12904 16956 20889 24703	14366 18876 23261 27513	20986 27598 34025 40271	28864 37972 46835 55459
8 × 4 ┐	$\frac{1}{2}$ $\frac{5}{8}$ $\frac{3}{4}$	9598 11809 13963	11058 13609 16095	12620 15537 18381	14286 17593 20817	16056 19823 23407	17930 22089 26147	19906 24529 29039	28848 35571 42129	39448 48659 57649

┐ Denotes Flange Angles with small leg outstanding.

┐ Denotes Flange Angles with large leg outstanding.

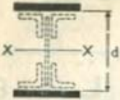
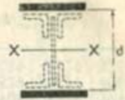


PLATE GIRDERS
Gross Moments of Inertia of Two Flange Plates (axis x—x) per inch of Plate width



Depth d inches	Thickness of each plate in inches									
	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{4}$	$1\frac{1}{2}$	$1\frac{3}{4}$	2
$36\frac{1}{2}$	255	342	431	520	611	703	891	1084	1281	1484
$39\frac{1}{2}$	298	400	503	608	713	820	1038	1261	1490	1724
$42\frac{1}{2}$	345	462	581	702	823	946	1197	1453	1714	1982
$45\frac{1}{2}$	395	529	665	802	941	1081	1366	1657	1954	2257
$48\frac{1}{2}$	448	600	754	910	1067	1225	1547	1876	2210	2552
$51\frac{1}{2}$	504	676	849	1024	1200	1378	1739	2107	2482	2863
$54\frac{1}{2}$	565	756	950	1145	1342	1540	1943	2353	2769	3194
$57\frac{1}{2}$	628	841	1056	1272	1491	1711	2157	2611	3073	3542
$60\frac{1}{2}$	695	926	1168	1407	1648	1891	2383	2884	3392	3908
$72\frac{1}{2}$	996	1328	1660	2012	2356	2701	3400	4108	4825	5552
$84\frac{1}{2}$	1351	1800	2251	2725	3189	3655	4596	4801	6510	7484

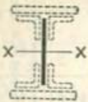


PLATE GIRDERS

Gross Moments of Inertia of Web Plates (axis x—x)

Thickness	Depth of Web Plate in inches										
	36	39	42	45	48	51	54	57	60	72	84
$\frac{1}{4}$	972	1236	1544	1898	2304	2764	3281	3858	4500	7776	12348
$\frac{5}{16}$	1215	1545	1929	2373	2880	3454	4101	4823	5625	9720	15435
$\frac{3}{8}$	1458	1854	2315	2848	3456	4145	4921	5787	6750	11664	18522
$\frac{1}{2}$	1944	2472	3087	3797	4608	5527	6561	7716	9000	15552	24696

From all these tables the Gross Moment of Inertia of a built-up Plate Girder can be obtained by simply adding together the Inertia of each component part; and by deducting the Total Inertia of the Rivet Holes (see pages 334-337), the Nett Moment of Inertia of the complete sum is thus obtained. From this the Section Modulus is obtained by the formula: $Z = I/y$, y being the distance from the Neutral Axis to the outermost Fibre, i.e. the outer edge of the Flange Plates or Angles, I being the Nett moment of Inertia.

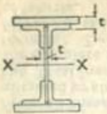
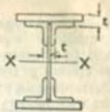


PLATE GIRDERS
Gross
Areas in square ins.
of
Component Parts



AREAS OF FOUR ANGLES

Angle Section	Thickness of each angle $\frac{5}{16}''$	$\frac{3}{8}''$	$\frac{1}{2}''$	$\frac{5}{8}''$	$\frac{3}{4}''$
$3\frac{1}{2} \times 3\frac{1}{2}$	8·36	9·94	13·0		
4×3	8·36	9·94	13·0		
4×4	9·60	11·44	15·0		
$5 \times 3\frac{1}{2}$	10·24	12·20	16·0		
5×4		12·96	17·0	20·94	
5×5		14·44	19·0	23·44	27·76
$6 \times 3\frac{1}{2}$		13·68	18·0	22·18	
6×4		14·44	19·0	23·44	27·76
6×6		17·44	23·0	28·44	33·74
8×4			23·0	28·44	33·76

AREAS OF WEB PLATES

Depth	Thickness $\frac{1}{4}''$	$\frac{5}{16}''$	$\frac{3}{8}''$	$\frac{1}{2}''$
36	9·0	11·25	13·5	18·0
39	9·75	21·19	14·62	19·5
42	10·5	13·12	15·8	21·0
45	11·25	14·06	16·87	22·5
48	12·0	15·00	18·0	24·0
51	12·75	15·94	19·12	25·5
54	13·5	16·88	20·3	27·0
57	14·25	17·81	21·37	28·5
60	15·0	18·75	22·5	30·0
72	18·0	22·50	27·0	36·0
84	21·0	26·25	31·5	42·0

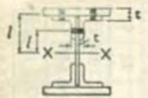
AREAS OF TWO FLANGE PLATES

See Supplement

Width	Thickness of each Plate $\frac{3}{8}''$	$\frac{1}{2}''$	$\frac{5}{8}''$	$\frac{3}{4}''$	$\frac{7}{8}''$	1″	$1\frac{1}{4}''$	$1\frac{1}{2}''$	$1\frac{3}{4}''$	2″
10	7·5	10·0	12·5	15·0	17·5	20·0	25·0	30·0	35·0	40·0
12	9·0	14·0	15·0	18·0	21·0	24·0	30·0	36·0	42·0	48·0
14	10·5	16·0	17·5	21·0	24·5	28·0	35·0	42·0	49·0	56·0
16	12·0	18·0	20·0	24·0	28·0	32·0	40·0	48·0	56·0	64·0
18	13·5	20·0	22·5	27·0	31·5	36·0	45·0	54·0	63·0	72·0
20	15·0	10·0	25·0	30·0	35·0	40·0	50·0	60·0	70·0	80·0

AREAS OF HOLES

Diameter of Hole	Number of Holes	Thickness t of hole in inches $\frac{5}{16}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{4}$	$1\frac{1}{2}$	2	$2\frac{1}{4}$	$2\frac{1}{2}$	$2\frac{3}{4}$
$\frac{13}{16}''$	1	0·25	0·30	0·41	0·51	0·61	0·71	0·81	1·02	1·22	1·62	1·83	2·03	2·23
	2	0·50	0·60	0·82	1·02	1·22	1·42	1·62	2·04	2·44	3·24	3·66	4·06	4·46
$\frac{15}{16}''$	1	0·29	0·35	0·47	0·59	0·70	0·82	0·94	1·17	1·41	1·87	2·11	2·34	2·58
	2	0·58	0·70	0·94	1·18	1·40	1·64	1·88	2·34	2·82	3·74	4·22	4·68	5·16



PLATE

Moments of

of $\frac{15}{16}''$ Dia.

At Varying Distances

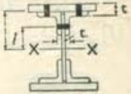
Distance l in ins.	Thickness (t)						
	$\frac{11}{16}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$
15	145	158	185	211	237	264	290
16	165	180	210	240	270	300	330
18	209	228	266	304	342	380	418
$18\frac{1}{2}$	221	241	281	321	361	401	441
19	233	254	296	339	381	423	465
21	284	310	362	414	465	517	568
$21\frac{1}{2}$	298	325	379	434	488	542	596
22	312	340	397	454	511	567	624
24	372	405	472	540	608	675	742
$24\frac{1}{2}$	387	422	492	563	633	703	774
25	403	439	513	586	659	733	806
27	470	512	598	684	769	854	940
$27\frac{1}{2}$	488	532	620	709	798	886	975
28	506	551	643	735	827	919	1011
30	581	633	738	844	950	1055	1160
$30\frac{1}{2}$	600	654	763	873	981	1090	1199
31	620	676	788	901	1014	1126	1239
34	746	813	948	1084	1220	1355	1490
36	836	911	1063	1216	1367	1519	1671
$36\frac{1}{2}$	859	937	1092	1250	1406	1561	1717
37	883	962	1123	1284	1444	1604	1765
40	1032	1125	1312	1501	1688	1875	2062
42	1138	1240	1446	1655	1861	2067	2274
$42\frac{1}{2}$	1165	1270	1481	1694	1906	2117	2328
43	1193	1300	1516	1734	1951	2167	2383

GIRDERS

Inertia (x—x)

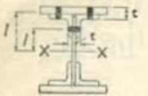
Holes

from Neutral Axis



of Hole in inches

$1\frac{1}{2}$	$1\frac{5}{8}$	$1\frac{3}{4}$	$1\frac{7}{8}$	2	$2\frac{1}{4}$	$2\frac{1}{2}$	$2\frac{3}{4}$
316	343	369	396	422	475	527	580
360	390	420	450	480	540	600	660
456	493	532	570	608	683	759	835
481	521	562	602	642	722	802	882
508	550	592	635	677	761	846	931
620	672	724	775	827	930	1034	1137
650	704	759	813	867	975	1084	1192
681	737	794	851	908	1021	1134	1248
810	877	945	1013	1080	1215	1350	1485
844	914	985	1055	1125	1266	1407	1547
879	952	1026	1099	1172	1318	1465	1611
1025	1110	1196	1282	1367	1537	1709	1879
1063	1152	1241	1329	1418	1595	1773	1950
1102	1194	1287	1378	1470	1653	1838	2021
1265	1371	1477	1582	1688	1898	2110	2320
1308	1417	1527	1635	1744	1962	2181	2398
1351	1464	1577	1689	1802	2027	2253	2477
1625	1761	1897	2032	2168	2438	2710	2980
1822	1974	2127	2278	2430	2733	3038	3341
1873	2029	2186	2342	2498	2810	3123	3435
1925	2085	2247	2407	2567	2887	3209	3529
2250	2437	2626	2813	3000	3374	3750	4125
2480	2687	2895	3101	3308	3720	4135	4548
2540	2751	2964	3175	3387	3809	4234	4657
2600	2816	3034	3251	3467	3900	4334	4767



PLATE

Moments of

of $\frac{13''}{16}$ Dia.

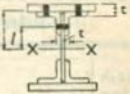
At Varying Distances

Distance l in ins.	Thickness (t) $\frac{11}{16}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$
15	126	137	160	183	206	229	251
16	143	156	182	208	234	260	286
18	181	197	230	263	296	329	362
$18\frac{1}{2}$	191	208	243	278	313	348	382
19	202	220	257	293	330	367	403
21	247	269	314	358	403	448	493
$21\frac{1}{2}$	258	282	329	376	422	470	516
22	271	295	344	393	442	492	541
24	322	351	410	468	526	585	643
$24\frac{1}{2}$	336	366	427	488	549	610	670
25	349	381	444	508	571	635	698
27	408	444	518	592	666	741	814
$27\frac{1}{2}$	423	461	538	614	691	768	845
28	438	477	557	637	717	797	876
30	503	548	640	731	823	914	1005
$30\frac{1}{2}$	520	567	661	756	850	945	1039
31	537	585	683	781	878	976	1073
34	646	704	822	939	1057	1174	1291
36	724	789	921	1053	1185	1317	1448
$36\frac{1}{2}$	745	811	947	1082	1218	1354	1488
37	765	834	973	1112	1251	1391	1529
40	894	974	1138	1300	1462	1626	1787
42	986	1074	1254	1433	1612	1792	1970
$42\frac{1}{2}$	1010	1100	1284	1468	1651	1835	2018
43	1034	1126	1315	1502	1690	1879	2065

GIRDERS

Inertia (x—x)

Holes



from Neutral Axis

of Hole in inches

$1\frac{1}{2}$	$1\frac{5}{8}$	$1\frac{3}{4}$	$1\frac{7}{8}$	2	$2\frac{1}{4}$	$2\frac{1}{2}$	$2\frac{3}{4}$
274	297	320	343	366	411	457	503
312	338	364	390	416	468	520	572
395	428	461	493	527	592	658	724
417	452	487	521	556	626	695	765
440	477	513	550	587	660	733	806
538	582	627	673	717	806	896	985
563	610	657	704	751	845	939	1033
590	639	688	737	787	885	983	1081
702	760	819	877	936	1053	1170	1287
732	792	854	914	975	1097	1219	1341
762	825	889	952	1016	1143	1269	1396
889	962	1037	1110	1185	1333	1481	1629
922	998	1075	1152	1229	1382	1536	1689
956	1035	1115	1194	1274	1433	1592	1751
1097	1188	1280	1371	1463	1645	1828	2011
1134	1228	1323	1417	1512	1700	1889	2078
1171	1269	1367	1464	1561	1757	1952	2147
1409	1526	1644	1761	1879	2113	2348	2583
1580	1711	1843	1974	2106	2369	2632	2895
1624	1759	1894	2029	2165	2435	2706	2976
1669	1807	1947	2085	2225	2503	2780	3058
1950	2112	2275	2437	2600	2925	3250	3574
2150	2328	2508	2687	2867	3225	3583	3941
2202	2384	2568	2751	2935	3302	3668	4035
2254	2441	2629	2816	3005	3380	3755	4131

Maximum Permissible Compressive Beams or Girders when the ratio of

In accordance with B.S.S.

Width of top Flange in ins. B	SPANS									
	6	7	8	9	10	11	12	13	14	15
2½	6·7	6·0	5·2	4·5	3·8					
3	7·4	6·8	6·2	5·6	5·0	4·4	3·8			
3½	7·9	7·4	6·9	6·4	5·9	5·4	4·8	4·3	3·8	
4		7·8	7·4	7·0	6·5	6·1	5·6	5·2	4·7	4·3
4½			7·8	7·4	7·0	6·6	6·2	5·8	5·4	5·0
5				7·8	7·4	7·0	6·7	6·3	6·0	5·6
5½					7·7	7·4	7·1	6·7	6·4	6·1
6						7·7	7·4	7·1	6·8	6·5
6½							7·7	7·4	7·1	6·9
7							7·9	7·7	7·4	7·1
7½								7·9	7·7	7·4
8									7·8	7·7
10										
12										

In no case may the ratio $\frac{L}{B}$ exceed 50.

Stresses in tons per sq. in. in uncased Top Flange Breadth to Span exceeds 20

Formula $(11 - {\cdot}15\,\frac{L}{B})$ tons/sq. in.

IN FEET

16	17	18	19	20	22	24	26	28	30	32
3·8										
4·6	4·2	3·8								
5·2	4·9	4·5	4·2	3·8						
5·8	5·4	5·1	4·8	4·5	3·8					
6·2	5·9	5·6	5·3	5·0	4·4	3·8				
6·6	6·3	6·0	5·7	5·5	4·9	4·4	3·8			
6·9	6·6	6·4	6·1	5·9	5·3	4·8	4·3	3·8		
7·2	6·9	6·7	6·4	6·2	5·7	5·2	4·8	4·3	3·8	
7·4	7·2	7·0	6·7	6·5	6·1	5·6	5·2	4·7	4·3	3·8
	7·9	7·8	7·6	7·4	7·0	6·7	6·3	6·0	5·6	5·2
					7·7	7·4	7·1	6·8	6·5	6·2

This Table is in accordance with B.S.S. 449—1937
L=Span in Inches

Maximum Permissible Compressive Beams or Girders when the ratio of

In accordance with B.S.S.

Width of Top Flange in ins. B	SPANS								
	6	7	8	9	10	11	12	13	14
2½	8·1	7·0	5·9	4·9	3·8				
3	9·1	8·3	7·4	6·5	5·6	4·7	3·8		
3½	9·9	9·1	8·4	7·6	6·8	6·1	5·3	4·6	3·8
4		9·8	9·1	8·5	7·8	7·1	6·5	5·8	5·2
4½			9·7	9·1	8·5	7·9	7·4	6·8	6·2
5				9·6	9·1	8·6	8·1	7·6	7·0
5½					9·6	9·1	8·6	8·2	7·7
6						9·6	9·1	8·7	8·2
6½							9·5	9·1	8·7
7							9·9	9·5	9·1
7½								9·8	9·5
8									9·8
10									
12									

In no case may the ratio $\frac{L}{B}$ exceed 50.

Stresses in tons per sq. inch in uncased Top Flange Breadth to Span exceeds 20.

Formula $\left(14{\cdot}4 - {\cdot}22\frac{L}{B}\right)$ **tons/sq. in.**

IN FEET

15	16	17	18	19	20	22	24	26	28	30	32
4·5	3·8										
5·6	5·0	4·4	3·8								
6·5	5·9	5·4	4·9	4·4	3·8						
7·2	6·7	6·2	5·7	5·3	4·8	3·8					
7·8	7·3	6·9	6·4	6·0	5·6	4·7	3·8				
8·3	7·9	7·5	7·0	6·7	6·2	5·4	4·6	3·8			
8·8	8·4	8·0	7·6	7·2	6·8	6·1	5·3	4·6	3·8		
9·1	8·8	8·4	8·0	7·7	7·4	6·6	5·9	5·2	4·5	3·8	
9·4	9·1	8·8	8·4	8·1	7·8	7·1	6·5	5·8	5·1	4·5	3·8
		9·9	9·6	9·4	9·1	8·6	8·1	7·5	7·0	6·5	5·9
						9·5	9·1	8·7	8·2	7·8	7·3

This table is in accordance with B.S.S. 449—1940
L=Span in Inches

TIMBER JOIST SIZES

Calculated on Super Load of 40 lbs./sq. ft.

and dead load of 22 lbs./sq. ft.

Timber to be good quality Redwood or equal

Centres of Joists	SPANS IN FEET												
	4	6	8	10	12	13	14	15	16	17	18	19	20
12"	3 × 2	4 × 2	5 × 2	7 × 2	8 × 2	8 × 2	9 × 2	11 × 2	11 × 2	11 × 2	11 × 2½	11 × 2½	11 × 3
13"	3 × 2	4 × 2	5 × 2	7 × 2	8 × 2	9 × 2	9 × 2	11 × 2	11 × 2	11 × 2	11 × 2½	11 × 3	11 × 3
14"	3 × 2	4 × 2	6 × 2	7 × 2	8 × 2	9 × 2	9 × 2½	11 × 2	11 × 2	11 × 2	11 × 2½	11 × 3	11 × 4
15"	3 × 2	4 × 2	6 × 2	7 × 2	8 × 2	9 × 2	9 × 2½	11 × 2	11 × 2	11 × 2½	11 × 3	11 × 3	11 × 4
16"	3 × 2	4½ × 2	6 × 2	7 × 2	9 × 2	9 × 2	9 × 2½	11 × 2	11 × 2	11 × 2½	11 × 3	11 × 4	11 × 4

Joist Sizes given above are in inches. The Joists shown require to be tied on top with Floor Boards.

RECTANGULAR TIMBER BEAMS

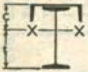
Safe Uniformly Distributed Loads in Pounds **per 1 Inch of Beam Width**
Maximum bending Stress 1,000 lbs. per Sq. Inch, Horz. Shearing Stress 100 lbs. per Sq. Inch

Depth of Beam in Inches	SPANS IN FEET														Section Modulus of 1″ wide Beams
	5	6	7	8	9	10	12	14	16	18	20	22	24	26	
3	200	167	143	125											1·500
4	356	296	254	222	198										2·667
5	556	463	397	347	309	278	231								4·166
6	800	666	571	500	444	400	333	286							6·000
7	1089	907	778	681	605	544	454	389	340						8·166
8	1422	1185	1016	889	790	711	593	508	444	395					10·666
9	1800	1499	1285	1125	1000	900	750	643	562	500	450				13·500
10		1851	1589	1389	1235	1111	926	794	694	617	556	505			16·666
11			1920	1680	1494	1344	1120	960	840	747	672	611			20·166
12			2285	2000	1778	1600	1333	1143	1003	889	800	727	667		24·000
14				2725	2421	2178	1815	1556	1361	1210	1089	990	907	838	32·666
Defl'n.	0·63	0·90	1·23	1·60	2·03	2·50	3·60	4·90	6·40	8·10	10·0	12·1	14·4	16·9	Co-eff't.

Tabular Safe Loads include the Weight of the Beam and are applicable only when beams are sufficiently stiffened against Lateral Deflection

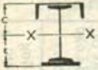
(See page xvi for notes)

CRANE GANTRY GIRDERS—

		Wt. per Foot in lbs.	Area in sq. inches	Gross Section Moduli Axis x—x		Dimensions in Ins.	
Joist	Top Channel			c	t	c	t
24×7½L	17×4 ×44·34	140·8	40·98	424·5	**238·0**	8·8	15·7
24×7½L	15×4 ×36·37	132·8	38·64	386·8	**234·1**	9·2	15·2
24×7½L	12×3½×26·37	122·8	35·70	338·4	**231·0**	9·9	14·5
22×7	17×4 ×44·34	120·8	35·10	342·8	**174·8**	7·6	14·9
22×7	15×4 ×36·37	112·8	32·76	309·4	**171·9**	8·0	14·4
22×7	12×3½×26·37	102·8	29·82	270·1	**168·3**	8·6	13·8
20×7½	17×4 ×44·34	134·8	39·23	340·3	**188·2**	7·3	13·2
20×7½	15×4 ×36·37	126·8	36·89	306·9	**186·1**	7·7	12·7
20×7½	12×3½×26·37	116·8	33·95	271·5	**182·5**	8·2	12·2
20×6½	17×4 ×44·34	110·8	32·16	295·5	**140·3**	6·6	13·9
20×6½	15×4 ×36·37	102·8	29·82	264·4	**138·1**	7·0	13·4
20×6½	12×3½×26·37	92·8	26·88	228·6	**135·7**	7·6	12·8
18×7	15×4 ×36·37	112·8	32·79	250·4	**143·4**	6·7	11·7
18×7	12×3½×26·37	102·8	29·85	219·0	**140·8**	7·2	11·2
		Rive	ts $\frac{7}{8}''$	dia.			
18×6	15×4 ×36·37	92·3	26·88	219·0	**106·0**	6·0	12·4
18×6	12×3½×26·37	82·3	23·94	186·7	**104·4**	6·6	11·8
16×6 L	15×4 ×36·37	87·3	25·41	183·8	**87·7**	5·3	11·1
16×6 L	12×3½×26·37	77·3	22·47	157·8	**86·3**	5·8	10·6
15×6 L	12×3½×26·37	72·3	21·00	140·0	**73·5**	5·3	10·1
14×6 L	12×3½×26·37	73·3	21·35	132·4	**70·4**	5·0	9·4
		Rive	ts $\frac{3}{4}''$	dia.			

Minimum Zxx = Ixx divided by "t" or "c" whichever is the greater.

Dimensions and Properties



Joist	Top Channel	Btm. Plate	Wt. per Foot in lbs.	Nett Section Moduli Axis x—x		Dimensions in Ins.	
				c	t	c	t
24×7½L	15×4 ×36·37	12 × 1	175·0	417·8	424·4	12·8	12·6
24×7½L	12×3½×26·37	12 × ⅝	149·4	348·2	**342·7**	12·4	12·6
24×7½L	12×3½×26·37	10 × ½	140·9	339·3	**295·9**	11·6	13·3
22×7	15×4 ×36·37	12 × ⅞	149·9	339·5	**330·9**	11·5	11·8
22×7	12×3½×26·37	12 × ⅝	129·7	281·8	**276·9**	11·4	11·6
22×7	12×3½×26·37	10 × ⅜	116·9	267·9	**213·1**	10·1	12·7
20×7½	15×4 ×36·37	12 × ⅞	163·9	328·8	**325·7**	10·6	10·7
20×7½	12×3½×26·37	12 × ½	138·6	273·0	**255·3**	10·1	10·8
20×6½	15×4 ×36·37	12 × ⅞	139·9	289·2	**286·5**	10·6	10·7
20×6½	12×3½×26·37	12 × ½	114·6	233·8	**214·5**	10·0	10·9
20×6½	12×3½×26·37	9 × ⅞	120·9	241·9	**235·2**	10·5	10·8
18×7	15×4 ×36·37	12 × ⅞	149·9	274·0	**271·1**	9·6	9·7
18×7	12×3½×26·37	12 × ½	124·6	224·0	**208·0**	9·1	9·8
18×7	12×3½×26·37	9 × ⅞	130·9	232·1	**225·0**	9·5	9·8
		Rivets ⅞" dia.					
18×6	15×4 ×36·37	12 × ⅞	129·4	**241·4**	244·0	9·7	9·6
18×6	12×3½×26·37	12 × ½	104·1	194·3	**176·7**	9·0	9·9
18×6	12×3½×26·37	10 × ½	100·7	195·1	**162·9**	8·6	10·3
16×6 L	15×4 ×36·37	12 × ⅞	124·4	**207·9**	210·3	8·7	8·6
16×6 L	12×3½×26·37	12 × ½	99·1	164·9	**151·8**	8·1	8·8
16×6 L	12×3½×26·37	10 × ½	95·7	166·2	**139·1**	7·7	9·2
15×6 L	12×3½×26·37	12 × ½	94·1	148·7	**136·1**	7·6	8·3
15×6 L	12×3½×26·37	10 × ½	90·7	149·7	**123·9**	7·2	8·7
15×6 L	12×3½×26·37	10 × ⅜	86·4	145·3	**109·8**	6·8	9·0
		Rivets ¾" dia.					

JOISTS—
Maximum Reaction, Vertical Shear and Web Buckling Values in Tons

Section	Web Thickness in inches	Nett Web Depth in inches	Nett Web Area in sq. ins.	Safe Reactions or Concentrated Load in Tons	Web Buckling per inch Run	Vertical Shear in Tons on Web
24×7½×100	0·60	21·30	12·78	52·1	2·73	72·0
24×7½×95	0·57	21·41	12·20	48·79	2·43	68·4
22×7 ×75	0·50	19·81	9·90	38·77	2·00	55·0
20×7½×89	0·60	17·41	10·45	45·33	3·23	60·0
20×6½×65	0·45	17·87	8·04	31·46	1·79	45·0
18×8 ×80	0·50	15·50	7·75	33·02	2·56	45·0
18×7 ×75	0·55	15·61	8·59	37·48	3·00	49·5
18×6 ×55	0·42	16·04	6·73	26·76	1·76	37·8
16×8 ×75	0·48	13·53	6·49	28·39	2·63	38·4
16×6 ×62	0·55	13·85	7·62	34·22	3·21	44·0
16×6 ×50	0·40	14·10	5·64	23·07	1·83	32·0
15×6 ×59	0·50	12·80	6·40	28·6	2·90	37·5
15×6 ×45	0·38	13·24	5·03	20·66	1·75	28·5
15×5 ×42	0·42	13·33	5·60	23·68	2·11	31·5
14×8 ×70	0·46	11·57	5·32	23·91	2·68	32·2
14×6 ×57	0·50	11·80	5·90	26·88	3·00	35·0
14×6 ×46	0·40	12·15	4·86	20·83	2·08	28·0
14×5½×40	0·37	12·33	4·56	19·00	1·80	25·9
13×5 ×35	0·35	11·42	4·00	16·76	1·72	22·7
12×8 ×65	0·43	9·60	4·13	19·02	2·63	25·8
12×6 ×54	0·50	9·78	4·89	23·06	3·19	30·0
12×6 ×44	0·40	10·12	4·05	18·16	2·33	24·0
12×5 ×39	0·44	10·34	4·55	20·75	2·65	26·4
12×5 ×32	0·35	10·52	3·68	15·83	1·84	21·0
12×5 ×30	0·33	10·61	3·50	14·75	1·65	19·8
10×8 ×55	0·40	7·84	3·14	14·80	2·55	20·0
10×6 ×42	0·40	8·12	3·24	15·20	2·53	20·0

JOISTS—

Maximum Reaction, Vertical Shear and Web Buckling Values in Tons

Section	Web Thickness in inches	Nett Web Depth in inches	Nett Web Area in sq. ins.	Safe Reaction or Concentrated Load in Tons	Web Buckling per inch Run	Vertical Shear in Tons on Web
10 ×6 ×40	0·36	8·14	2·93	13·46	2·19	18·0
10 ×5 ×30	0·36	8·52	3·07	13·97	2·16	18·0
10 ×4½×25	0·30	8·65	2·60	11·28	1·62	15·0
9 ×7 ×58	0·55	6·68	3·68	18·5	3·81	24·75
9 ×7 ×50	0·40	6·83	2·73	13·16	2·63	18·0
9 ×4 ×21	0·30	7·78	2·34	10·42	1·72	13·5
8 ×8 ×38	0·33	6·62	2·18	10·24	2·09	13·2
8 ×6 ×35	0·35	6·26	2·19	10·48	2·28	14·0
8 ×5 ×28	0·35	6·47	2·27	10·79	2·26	14·0
8 ×4 ×18	0·28	6·90	1·93	8·72	1·65	11·2
7 ×4 ×16	0·25	5·93	1·48	6·74	1·49	8·7
7 ×3½×15	0·25	5·94	1·48	6·73	1·50	8·75
6 ×5 ×25	0·41	4·58	1·88	9·49	2·86	12·3
6 ×4½×20	0·37	4·80	1·77	8·84	2·54	11·1
6 ×3 ×12	0·23	5·02	1·15	5·34	1·42	6·9
5 ×4½×20	0·29	3·64	1·05	5·27	2·00	7·2
5 ×4½×18	0·29	3·76	1·09	5·40	2·00	7·25
5 ×3 ×11	0·22	4·02	0·88	4·22	1·42	5·5
5 ×2½×9	0·20	4·11	0·82	3·84	1·26	5·0
4¾×1¾×6·5	0·18	3·96	0·71	3·30	1·11	4·2
4½×2 ×7	0·19	3·70	0·70	3·30	1·22	4·28
4 ×3 ×10	0·24	3·08	0·74	3·68	1·65	4·8
4 ×3 ×9·5	0·22	3·09	0·68	3·36	1·50	4·4
4 ×1¾×5	0·17	3·39	0·58	2·71	1·08	3·4
3 ×3 ×8·5	0·20	2·11	0·42	2·14	1·40	3·0
3 ×1½×4	0·16	2·39	0·38	1·87	1·08	2·4

JOISTS
Properties and Dimensions

Section	*Nett Area in Square Inches	Notch to clear Fillets	Clear Depth between Fillets	Cross Centres of Holes	Dia. of Holes	Gross Moment of Inertia about y—y
24 × 7½ × 100	27·39	1·92	20·16	4½	15/16	66·92
24 × 7½ × 95	26·04	1·89	20·22	4½	15/16	62·54
22 × 7 × 75	20·50	1·66	18·68	4	15/16	41·07
20 × 7½ × 89	24·30	1·89	16·22	4½	15/16	62·54
20 × 6½ × 65	17·58	1·60	16·81	3¾	15/16	32·56
18 × 8 × 80	21·75	1·88	14·23	4¾	15/16	69·43
18 × 7 × 75	20·35	1·75	14·50	4	15/16	46·56
18 × 6 × 55	14·95	1·48	15·04	3½	13/16	23·64
16 × 8 × 75	20·30	1·87	12·26	4¾	15/16	68·30
16 × 6 × 62	16·83	1·57	12·86	3½	13/16	27·14
16 × 6 × 50	13·53	1·45	13·10	3½	13/16	22·47
15 × 6 × 59	15·92	1·59	11·82	3½	13/16	28·22
15 × 6 × 45	12·18	1·38	12·24	3½	13/16	19·87
15 × 5 × 42	11·31	1·27	12·46	2¾	13/16	11·81

JOISTS

Properties and Dimensions

Section	*Nett Area in Square Inches	Notch to clear Fillets	Clear Depth between Fillets	Cross Centres of Holes	Dia. of Holes	Gross Moment of Inertia about y—y
14 × 8 × 70	18·86	1·85	10·30	$4\frac{3}{4}$	$\frac{13}{16}$	66·67
14 × 6 × 57	15·36	1·60	10·80	$3\frac{1}{2}$	$\frac{13}{16}$	27·94
14 × 6 × 46	12·46	1·43	11·14	$3\frac{1}{2}$	$\frac{13}{16}$	21·45
14 × $5\frac{1}{2}$ × 40	10·75	1·30	11·40	$3\frac{1}{4}$	$\frac{13}{16}$	14·79
13 × 5 × 35	9·32	1·23	10·54	$2\frac{3}{4}$	$\frac{13}{16}$	10·82
12 × 8 × 65	17·42	1·84	8·32	$4\frac{3}{4}$	$\frac{13}{16}$	65·18
12 × 6 × 54	14·45	1·61	8·78	$3\frac{1}{2}$	$\frac{13}{16}$	28·28
12 × 6 × 44	11·83	1·44	9·12	$3\frac{1}{2}$	$\frac{13}{16}$	22·12
12 × 5 × 39	10·39	1·30	9·41	$2\frac{3}{4}$	$\frac{13}{16}$	12·16
12 × 5 × 32	8·56	1·17	9·66	$2\frac{3}{4}$	$\frac{13}{16}$	9·69
12 × 5 × 30	8·00	1·13	9·74	$2\frac{3}{4}$	$\frac{13}{16}$	8·77
10 × 8 × 70	18·78	1·84	6·32	$4\frac{3}{4}$	$\frac{13}{16}$	71·67
10 × 8 × 55	14·71	1·72	6·56	$4\frac{3}{4}$	$\frac{13}{16}$	54·74
10 × 6 × 42	11·15	1·37	7·27	$3\frac{1}{2}$	$\frac{13}{16}$	22·95

JOISTS

Properties and Dimensions

Section	*Nett Area in Square Inches	Notch to clear Fillets	Clear Depth between Fillets	Cross Centres of Holes	Dia. of Holes	Gross Moment of Inertia about y—y
10 × 6 × 40	10·62	1·44	7·12	$3\frac{1}{2}$	$\frac{13}{16}$	21·76
10 × 5 × 30	7·95	1·18	7·64	$2\frac{3}{4}$	$\frac{13}{16}$	9·73
10 × $4\frac{1}{2}$ × 25	6·53	1·08	7·84	$2\frac{1}{2}$	$\frac{13}{16}$	6·49
9 × 7 × 58	15·33	1·71	5·58	4	$\frac{15}{16}$	46·30
9 × 7 × 50	13·16	1·66	5·68	4	$\frac{15}{16}$	40·17
9 × 4 × 21	5·55	0·98	7·04	$2\frac{1}{4}$	$\frac{11}{16}$	4·15
8 × 8 × 38	10·10	1·03	5·94	$4\frac{3}{4}$	$\frac{15}{16}$	39·70
8 × 6 × 35	9·25	1·38	5·24	$3\frac{1}{2}$	$\frac{13}{16}$	19·54
8 × 5 × 28	7·35	1·20	5·60	$2\frac{3}{4}$	$\frac{13}{16}$	10·19
8 × 4 × 18	4·75	0·92	6·16	$2\frac{1}{4}$	$\frac{11}{16}$	3·51
7 × 4 × 16	4·22	0·91	5·18	$2\frac{1}{4}$	$\frac{11}{16}$	3·37
7 × $3\frac{1}{2}$ × 15	3·98	0·87	5·26	2	$\frac{9}{16}$	2·41
6 × 5 × 25	6·52	1·14	3·72	$2\frac{3}{4}$	$\frac{13}{16}$	9·10
6 × $4\frac{1}{2}$ × 20	5·19	1·00	4·00	$2\frac{1}{2}$	$\frac{13}{16}$	5·40

JOISTS

Properties and Dimensions

 Section	*Nett Area in Square Inches	Notch to clear Fillets	Clear Depth between Fillets	Cross Centres of Holes	Dia. of Holes	Gross Moment of Inertia about y—y
6 × 3 × 12	3·11	0·80	4·40	1½	$\frac{9}{16}$	1·46
5 × 4½ × 20	5·05	1·09	2·83	2½	$\frac{13}{16}$	6·59
5 × 4½ × 18	4·56	0·94	3·12	2½	$\frac{13}{16}$	5·66
5 × 3 × 11	2·84	0·80	3·40	1½	$\frac{9}{16}$	1·45
5 × 2½ × 9	2·34	0·72	3·57	1⅜	$\frac{7}{16}$	0·79
4¾ × 1¾ × 6·5	1·71	0·62	3·52	⅞	$\frac{5}{16}$	0·26
4½ × 2 × 7	1·86	0·64	3·22	1⅛	$\frac{5}{16}$	0·38
4 × 3 × 10	2·55	0·77	2·47	1½	$\frac{9}{16}$	1·33
4 × 3 × 9·5	2·42	0·71	2·58	1½	$\frac{9}{16}$	1·28
4 × 1¾ × 5	1·32	0·53	2·94	⅞	$\frac{5}{16}$	0·19
3 × 3 × 8·5	2·15	0·75	1·50	1½	$\frac{9}{16}$	1·25
3 × 1½ × 4	1·02	0·52	1·97	¾	$\frac{5}{16}$	0·13

*Nett Area allows for deduction of I hole in each Flange

CHANNELS
Properties and Dimensions

 Section	*Nett Area in Square Inches	Notch to clear Fillets	Clear Depth between Fillets	Back Mark of Flange Hole	Dia. of Holes	Gross Moment of Inertia about y—y
17 × 4 × 51·28	13·81	1·38	14·24	2¼	$\frac{15}{16}$	16·96
17 × 4 × 44·34	11·77	1·38	14·23	2¼	$\frac{15}{16}$	15·26
15 × 4 × 42·49	11·34	1·32	12·36	2¼	$\frac{15}{16}$	14·97
15 × 4 × 36·37	9·54	1·32	12·35	2¼	$\frac{15}{16}$	13·34
13 × 4 × 38·92	10·29	1·32	10·36	2¼	$\frac{15}{16}$	14·51
13 × 4 × 33·18	8·60	1·32	10·35	2¼	$\frac{15}{16}$	12·76
12 × 4 × 36·63	9·65	1·30	9·40	2¼	$\frac{15}{16}$	13·80
12 × 4 × 31·33	8·09	1·30	9·39	2¼	$\frac{15}{16}$	12·12
12 × 3½ × 30·45	8·02	1·13	9·75	2	$\frac{15}{16}$	7·96
12 × 3½ × 26·37	6·82	1·13	9·74	2	$\frac{15}{16}$	7·15
11 × 3½ × 30·52	7·89	1·21	8·59	2	$\frac{15}{16}$	8·86
11 × 3½ × 29·82	7·68	1·20	8·61	2	$\frac{15}{16}$	8·42
11 × 3½ × 26·78	6·79	1·21	8·58	2	$\frac{15}{16}$	7·93
10 × 4 × 18·86	4·97	0·96	8·09	2¼	$\frac{15}{16}$	7·14
10 × 3½ × 28·54	7·34	1·19	7·63	2	$\frac{15}{16}$	8·50

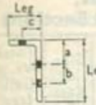
CHANNELS
Properties and Dimensions

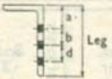
Section	*Nett Area in Square Inches	Notch to clear Fillets	Clear Depth between Fillets	Back Mark of Flange Hole	Dia. of Holes	Gross Moment of Inertia about y—y
10 × 3½ × 24·46	6·14	1·19	7·62	2	$\frac{15}{16}$	7·42
10 × 3 × 21·33	5·54	1·01	7·99	1¾	$\frac{13}{16}$	4·31
10 × 3 × 19·28	4·94	1·01	7·99	1¾	$\frac{13}{16}$	3·98
10 × 2·6 × 15·0	3·78	1·00	8·00	1⅜	$\frac{13}{16}$	2·30
9 × 3½ × 23·49	5·90	1·17	6·66	2	$\frac{15}{16}$	7·26
9 × 3½ × 22·27	5·54	1·18	6·65	2	$\frac{15}{16}$	6·90
9 × 3 × 19·91	5·15	1·00	7·01	1¾	$\frac{13}{16}$	4·18
9 × 3 × 17·46	4·43	1·00	7·00	1¾	$\frac{13}{16}$	3·75
8 × 3½ × 23·20	5·85	1·15	5·70	2	$\frac{15}{16}$	7·30
8 × 3½ × 20·21	4·97	1·16	5·69	2	$\frac{15}{16}$	6·37
8 × 3 × 18·68	4·78	1·00	6·01	1¾	$\frac{13}{16}$	4·11
8 × 3 × 15·96	3·98	1·00	6·00	1¾	$\frac{13}{16}$	3·58
8 × 2·26 × 11·22	2·69	0·88	6·24	1¼	$\frac{13}{16}$	1·30
7 × 3½ × 18·28	4·44	1·14	4·73	2	$\frac{15}{16}$	5·83
7 × 3 × 17·07	4·34	0·98	5·05	1¾	$\frac{13}{16}$	3·87

CHANNELS
Properties and Dimensions

 Section	*Nett Area in Square Inches	Notch to clear Fillets	Clear Depth between Fillets	‖ Back Mark of Flange Hole	Dia. of Holes	Gross Moment of Inertia about y—y
7 × 3 × 14·22	3·50	0·98	5·04	1¾	13/16	3·26
7 × 2⅛ × 9·75	2·32	0·68	5·64	1⅛	13/16	1·07
6 × 3½ × 16·48	3·95	1·12	3·77	2	15/16	5·29
6 × 3 × 17·53	4·38	1·03	3·94	1¾	13/16	3·95
6 × 3 × 16·51	4·08	1·04	3·93	1¾	13/16	3·70
6 × 3 × 12·41	3·03	0·94	4·12	1¾	13/16	2·83
6 × 2 × 10·75	2·64	0·75	4·50	1⅛	11/16	0·92
6 × 1·92 × 8·75	2·08	0·81	4·38	1⅛	11/16	0·78
5 × 2½ × 10·22	2·39	0·87	3·27	1⅜	13/16	1·64
4 × 2 × 7·91	1·90	0·72	2·57	1⅛	11/16	0·79
4 × 2 × 7·09	1·66	0·72	2·57	1⅛	11/16	0·70
3½ × 2 × 6·75	1·56	0·64	2·22	1⅛	11/16	0·71
3 × 1½ × 5·27	1·20	0·63	1·74	⅞	9/16	0·30
3 × 1½ × 5·11	1·19	0·61	1·78	⅞	9/16	0·30
3 × 1½ × 4·60	1·04	0·61	1·78	⅞	9/16	0·26

Spacing of Holes in
ANGLES AND TEES





Dimensions in Inches									Max. Dia. of Bolt
Leg	a	b	c	a	b	d	A	C	
10	3	5	—	3	$2\frac{1}{2}$	$2\frac{1}{2}$	8	$4\frac{3}{4}$	$\frac{7}{8}''$
9	3	4	—	3	2	2	$7\frac{1}{2}$	$4\frac{1}{2}$	$\frac{7}{8}''$
8	3	3	$4\frac{1}{2}$	$2\frac{1}{2}$	2	2	7	4	$\frac{7}{8}''$
7	$2\frac{1}{2}$	3	4				$6\frac{1}{2}$	$3\frac{3}{4}$	$\frac{7}{8}''$
$6\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{2}$	$3\frac{3}{4}$				6.	$3\frac{1}{2}$	$\frac{7}{8}''$
6	$2\frac{1}{4}$	$2\frac{1}{4}$	$3\frac{1}{2}$				$5\frac{1}{2}$	$3\frac{1}{4}$	$\frac{3}{4}''$
$5\frac{1}{2}$	$2\frac{1}{4}$	2	$3\frac{1}{4}$				5	$2\frac{3}{4}$	$\frac{3}{4}''$
5	2	$1\frac{3}{4}$	3				$4\frac{1}{2}$	$2\frac{1}{2}$	$\frac{3}{4}''$
$4\frac{1}{2}$			$2\frac{1}{2}$				4	$2\frac{1}{4}$	$\frac{5}{8}''$
4			$2\frac{1}{4}$				$3\frac{1}{2}$	2	$\frac{1}{2}''$
$3\frac{1}{2}$			2				3	$1\frac{1}{2}$	$\frac{1}{2}''$
3			$1\frac{3}{4}$				$2\frac{1}{2}$	$1\frac{3}{8}$	$\frac{3}{8}''$
$2\frac{3}{4}$			$1\frac{5}{8}$				$2\frac{1}{4}$	$1\frac{1}{4}$	$\frac{3}{8}''$
$2\frac{1}{2}$			$1\frac{3}{8}$				2	$1\frac{1}{8}$	$\frac{1}{4}''$
$2\frac{1}{4}$			$1\frac{1}{4}$				$1\frac{3}{4}$	$\frac{7}{8}$	$\frac{1}{4}''$
2			$1\frac{1}{8}$				$1\frac{1}{2}$	$\frac{3}{4}$	$\frac{1}{4}''$
$1\frac{3}{4}$			1						
$1\frac{1}{2}$			$\frac{7}{8}$						
$1\frac{1}{4}$			$\frac{3}{4}$						

TWO TIER GRILLAGE FOUNDATIONS

Safe Loads, Dimensions and Sections

Safe Loads in Tons	Size of Grillage L & B	No. of Joists	Top Tier Joists	No. of Joists	Bottom Tier Joists	Size of Base l & b in ins.	Total Wt. of Grillage Beams in lbs.
1 Ton/Foot² Bearing Pressure							
300	17′ 6″	3	24 × 7½ × 95	13	13 × 5 × 35	28·5	12951
250	16′ 0″	3	22 × 7 × 75	12	12 × 5 × 32	27·0	9744
200	14′ 3″	3	20 × 6½ × 65	11	10 × 5 × 30	27·0	7482
150	12′ 3″	3	15 × 6 × 45	11	9 × 4 × 21	24·0	6504
100	10′ 0″	3	12 × 5 × 30	9	7 × 4 × 16	21·0	2340
75	8′ 9″	3	10 × 4½ × 25	7	7 × 3½ × 15	21·0	1575
1½ Tons/Foot² Bearing Pressure							
350	15′ 3″	3	24 × 7½ × 95	13	13 × 5 × 35	28·5	11285
300	14′ 3″	3	22 × 7 × 75	13	12 × 5 × 30	27·0	8764
250	13′ 0″	3	20 × 6½ × 65	12	10 × 5 × 30	27·0	7215
200	11′ 9″	3	18 × 6 × 55	11	10 × 4½ × 25	24·0	5170
150	10′ 0″	3	14 × 5½ × 40	11	8 × 4 × 18	24·0	3180
100	8′ 3″	3	10 × 5 × 30	8	7 × 3½ × 15	21·0	1733
2 Tons/Foot² Bearing Pressure							
400	14′ 3″	3	24 × 7½ × 95	11	14 × 5½ × 40	28·5	10331
350	13′ 3″	3	20 × 7½ × 89	11	13 × 5 × 35	28·5	8639
300	12′ 3″	3	20 × 6½ × 65	9	13 × 5 × 35	27·0	6248
250	11′ 3″	3	18 × 6 × 55	8	13 × 5 × 35	24·0	5006
200	10′ 0″	3	15 × 6 × 45	8	10 × 4½ × 25	24·0	3350
150	8′ 9″	3	13 × 5 × 35	8	9 × 4 × 21	21·0	2389
100	7′ 3″	3	10 × 4½ × 25	6	7 × 4 × 16	21·0	1240

The Tabulated Safe Loads are for Grillages enveloped in fine Concrete 1-2-4 Mix. Safe Loads allow for Steel Stresses in accordance with B.S.S. No. 449—1940. Stiffeners are not required for the Tabulated Loads.

TWO TIER GRILLAGE FOUNDATIONS

Safe Loads, Dimensions and Sections

Safe Loads in Tons	Size of Grillage L & B	No. of Joists	Top Tier Joists	No. of Joists	Bottom Tier Joists	Size of Base l & b in ins.	Total Wt. of Grillage Beams in lbs.
	3 Tons/Foot² Bearing Pressure						
500	13′ 0″	3	24 × 7½ × 100	12	15 × 5 × 42	28·5	10452
450	12′ 3″	3	24 × 7½ × 95	11	14 × 5½ × 40	28·5	8881
400	11′ 6″	3	22 × 7 × 75	11	13 × 5 × 35	27·0	7016
350	10′ 9″	3	20 × 6½ × 65	10	12 × 5 × 32	27·0	5536
300	10′ 0″	3	20 × 6½ × 65	9	12 × 5 × 30	27·0	4650
250	9′ 3″	3	16 × 6 × 50	8	10 × 5 × 30	24·0	3608
200	8′ 3″	3	14 × 5½ × 40	7	10 × 4½ × 25	21·0	2434
150	7′ 0″	3	12 × 5 × 30	7	8 × 4 × 18	21·0	1512
100	5′ 9″	3	9 × 4 × 21	5	7 × 3½ × 15	18·0	793
	4 Tons/Foot² Bearing Pressure						
550	11′ 9″	3	24 × 7½ × 95	10	15 × 6 × 45	28·5	8637
500	11′ 3″	3	24 × 7½ × 95	9	15 × 6 × 45	28·5	7762
450	10′ 9″	3	20 × 7½ × 89	9	14 × 5½ × 40	28·5	6740
400	10′ 0″	3	22 × 7 × 75	8	14 × 5½ × 40	27·0	5450
350	9′ 6″	3	20 × 6½ × 65	7	14 × 5½ × 40	27·0	4513
300	8′ 9″	3	18 × 6 × 55	7	12 × 5 × 32	24·0	3404
250	8′ 0″	3	15 × 6 × 45	8	10 × 4½ × 25	24·0	2680
200	7′ 0″	3	13 × 5 × 35	7	9 × 4 × 21	21·0	1764
150	6′ 3″	3	10 × 5 × 30	6	8 × 4 × 18	21·0	1238
100	5′ 0″	3	9 × 4 × 21	5	7 × 3½ × 15	18·0	690

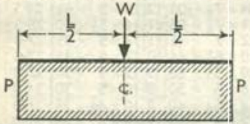
Grillage Beams to be completely embedded in solid Tamped Concrete having at least 4″ Cover on top, bottom and all sides (from Flange toes). Beams to be spaced not less than 3″ between Flange toes. The Safe Loads are in tons and include the weight of the beams and concrete.

PRESSURES ON FOUNDATIONS

P = Toe Pressure in Tons per Sq. Foot.
e = Eccentricity of Resultant Load W from Centre line of Foundation in Feet.
W = Applied Load in Tons.
L = Length of Foundation in Feet.
B = Breadth of Foundation in Feet.
K = Constant dependent on Eccentricity of Applied Load.

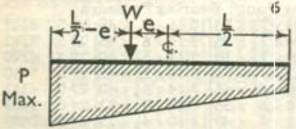
$$P = \frac{K\,W}{L \times B}$$

Where Eccentricity $e = 0$



$K = 1$

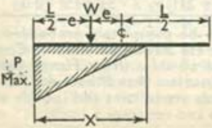
Where Eccentricity e is less than $\frac{L}{6}$



+ *for Max.*
− *for Min.*

$$K = \left(1 \pm \frac{6e}{L}\right)$$

Where Eccentricity e is greater than $\frac{L}{6}$

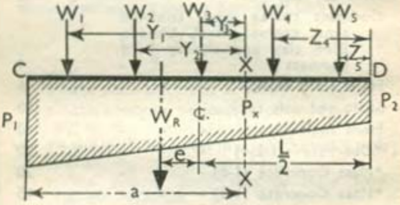


$$X = \frac{2W}{PB}$$

$$K = \frac{4}{3\left(1 - \frac{2e}{L}\right)}$$

PRESSURES ON FOUNDATIONS

Combined System of Loads.
P_1, P_2 and Px – Pressure in Tons per sq. ft.
e – Eccentricity of Resultant Load WR from Centre line of Foundation in Feet.
W_1, W_2, etc. – Applied Loads in Tons.
WR – Resultant applied Load in Tons (i.e. summation of all Loads).
Z_1, Z_2, etc. – Distance of Loads to Point " D ".
y_1 y_2, etc. – Distance of Loads to Section " x—x ".
a – Distance from point " C " to Section " x—x ".
B – Breadth of Foundation in Feet.
L – Length of Foundation in Feet.



$$W_1+W_2+W_3+W_4+W_5=WR.$$

To find eccentricity e of Resultant Load Wr take moments about point D

$$=W_1Z_1+W_2Z_2+W_3Z_3+W_4Z_4+W_5Z_5=\Sigma WZ$$

$$\therefore \quad e=\frac{\Sigma WZ}{WR}-\frac{L}{2}.$$

Calculate P_1 and P_2 as shown on page 358.

$$Px=\left(\frac{L-a}{L}\right)(P_1-P_2)+P_2.$$

$W_1y_1+W_2y_2+W_3y_3=\Sigma Wy$ = All Loads to left of Section " x—x."
$\therefore$ B.M. at Section " x—x "
$=\Sigma Wy-a^2/6\ (2P_1+Px)B$ ft./tons.

The above Formulae are applicable to any number of Loads WR. When e equals nil, $P_1=P_2=Px$.

SAFE BEARING PRESSURES

In tons per sq. ft.

Alluvial soil, made ground, very wet sand - - - -	Up to ½
Earth, ordinary, firm - - -	„ 1
Soft clay, wet or loose sand -	„ 1
Ordinary fairly dry clay, fine sand, loam - - - - -	„ 2
Firm dry clay - - - -	„ 3
Compact coarse sand, confined sand, coarse gravel, London Blue clay and similar hard compact - - - -	„ 4
Hard solid chalk - - -	„ 6
Shale and soft rock - - -	„ 10
Hard rock - - - - -	„ 40
*Concrete—(1-2-4) - - -	„ 30
*Mass Concrete (1-6) - -	„ 20
*Mass Concrete (1-8) - - -	„ 15
*Brickwork, ordinary quality (in cement mortar) - - -	„ 5-6
*Brickwork, good quality (in cement mortar) - - -	„ 10-12
*Limestone - - - - -	„ 12
*Sandstone - - - - -	„ 23
*Granite - - - - -	„ 30-40

*The above pressures may be exceeded by an amount up to 20% in all cases where such increased pressure is only of a local nature, as at Girder Bearings.

SLAB BASES

For solid round Steel Columns, when it can be assumed that the loading on the cap or under base is uniformly distributed, the minimum thickness in inches of a square cap or base shall be

$$t=\sqrt{\frac{9W}{16f}\;\frac{D}{D-d}}.$$

The cap or base plate shall not be less than 1·5 $(d+3)$ inches in length or diameter.

In the case of steel columns other than solid round columns the minimum thickness of a rectangular or square cap or base shall be :

$$t=\sqrt{\frac{3p}{f}\left(y^2-\frac{y_1^2}{4}\right)}.$$

Where

t = the plate thickness in inches.

p = the pressure or loading on base in tons per sq. in.

f = the working stress in steel taken as 9 tons per sq. in.

y = the greater projection of plate over column in inches.

y_1 = the lesser projection of plate over column in inches.

W = the total axial loading in tons.

D = the length of the side of cap or base in inches.

d = the diameter of the reduced end of the column in inches (usually taken as diameter of column less ¼").

When it cannot be assumed that the slab distributes the load uniformly or where the cap or base is not square in the case of Solid Round Columns, or where the cap or base is not rectangular or square in the case of other columns, special calculations have to be made.

FORMULAE

For simply supported beams uniformly loaded.

$$\text{Load } W = \frac{8 \times f \times Z}{l} = \frac{8 \times f \times I}{l \times Y}.$$

$$\text{Bending moment (inch tons)} = \frac{Wl}{8}.$$

$$\text{Deflection } D = \frac{5Wl^3}{384E.I}.$$

For simply supported beams with single point load P on centre of span.

$$\text{Load } P = \frac{4 \times f \times Z}{l} = \frac{4 \times f \times I}{l \times Y}.$$

$$\text{Bending moment (inch tons)} = \frac{Pl}{4}.$$

$$\text{Deflection } D = \frac{Pl^3}{48E.I}.$$

$$Z = \frac{I}{Y} \qquad r = \sqrt{\frac{I}{A}} \qquad I_0 = I + Ah^2.$$

$$M = \frac{fI}{Y} = fZ. \qquad f = \frac{MY}{I} = \frac{M}{Z}.$$

$$I = ZY. \qquad I = Ar^2.$$

Mr = Moment of Resistance = Zf.

NOTATION

M = Bending moment in inch tons.

Z = Modulus of section.

A = Area of cross section in square inches.

r = Radius of gyration in inches.

f = Safe stress in tons per square inch (extreme fibres).

I = Maximum moment of inertia about the neutral axis passing through the centre of gravity of the section.

I_0 = Moment of inertia parallel to the neutral axis, but not passing through the centre of gravity of the section.

Y = Distance in inches of outermost fibre from neutral axis (in a symmetrical section $Y = \frac{d}{2}$).

h = Distance in inches between neutral axis of built-up section and centre of gravity of area considered.
d = Total depth of cross section in inches.
W = Distributed load in tons.
P = Point load in tons.
l = Span of beam in inches (usually centres of bearings).
D = Deflection in inches.
E = Modulus of elasticity (taken at 13,000 tons per square inch for steel).

Z for solid round columns $= 0{\cdot}0982d^3$.
(d = diameter of column in inches).

Moment of inertia of rectangular beam $(I) = \dfrac{BD^3}{12}$.

Modulus of section of rectangular beam $(Z) = \dfrac{BD^2}{6}$.

B = Breadth (inches). D = Depth (inches).

Example using above Formulae

To determine the distributed load per square foot which a timber floor will carry, assuming the timber beams to be $11'' \times 3\frac{1}{2}''$ at $18''$ centres, $18'\ 0''$ span. Timber stressed at say 1,000 lbs. per square inch.

W = Safe distributed load in lbs. carried by timber beam.
L = Span in feet of timber beam.

$$\text{Bending moment } (M) = \frac{WL}{8} \quad \therefore \; M = \frac{W \times 18 \times 12}{8}.$$

$$M = fZ \qquad \therefore \; M = 1{,}000 \text{ lbs.} \times \frac{3{\cdot}5 \times 11^2}{6}.$$

$$\therefore \; \frac{W \times 18 \times 12}{8} = 1{,}000 \times \frac{3{\cdot}5 \times 11^2}{6}.$$

$$\therefore \; W = \frac{70600}{27} = 2615 \text{ lbs.}$$ (total distributed load carried by each timber beam).

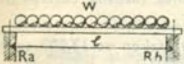
Area which is supported by $11'' \times 3\frac{1}{2}''$ beam $= 18'\ 0'' \times 1'\ 6'' = 27$ square feet.

$$\therefore \text{ Safe load per square foot} = \frac{2615}{27} = 97 \text{ lbs./sq. ft.}$$

Bending Moment, Shear, and Deflection of Simply Supported Beams of Uniform Section under Various Conditions of Loading.

UNIFORMLY DISTRIBUTED LOAD

Safe Load = Tabular Load



Shear

At any point $= \frac{W}{2l}(l-2x)$ Tons.

At Supports Ra, Rb (max.) $= \frac{W}{2}$ Tons.

Bending Moment

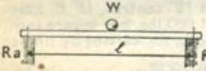
At any point $= \frac{Wx}{2l}(l-x)$ ins. Tons.

At centre (max.) $= \frac{Wl}{8}$ ins. Tons.

Deflection at Centre (max.) $= \frac{5}{384} \frac{Wl^3}{EI}$ ins.

CONCENTRATED LOAD AT MID SPAN

Safe Load = $\frac{1}{2}$ Tabular Load



Shear

At any point $= \frac{W}{2}$ Tons.

At Supports Ra, Rb (max.) $= \frac{W}{2}$ Tons.

Bending Moment

At any point $= \frac{Wx}{2}$ ins. Tons.

At centre (max.) $= \frac{Wl}{4}$ ins. Tons.

Deflection at Centre (max.) $= \frac{Wl^3}{48EI}$ ins.

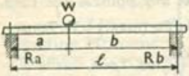
NOTATION

W = Applied Load in Tons.
l = Span in Inches.
I = Moment of Inertia in Inches^4.
E = Modulus of Elasticity = 13,000 tons per sq. in.
x = Dimension at any point along the beam from support in inches.

Bending Moment, Shear, and Deflection of Simply Supported Beams of Uniform Section under Various Conditions of Loading.

CONCENTRATED LOAD AT ANY POINT

Safe Load $(b > a)$ = Tabular Load $\times \frac{l^2}{8ab}$



Shear

Between Ra & $W = \frac{Wb}{l}$ Tons

Between Rb & $W = \frac{Wa}{l}$ Tons

Bending Moment

Between Ra and $W = \frac{Wb}{l}(l - x)$ ins. Tons.

Between Rb and $W = \frac{Wax}{l}$ ins. Tons.

x = distance from Rb in ins.

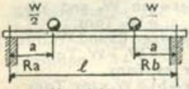
At W (max.) $= \frac{Wab}{l}$ ins. Tons.

Deflection (max.) $= \frac{Wab(l+a)}{9EI} \times \frac{x}{l}$ ins.

When $x = \sqrt{\frac{b(l+a)}{3}}$ from Rb ins.

TWO EQUAL CONCENTRATED LOADS SYMMETRICALLY PLACED

Safe Load = Tabular Load $\times \frac{l}{4a}$



Shear

Between Load and Support $= \frac{W}{2} = Ra = Rb$ Tons.

Between both Loads = 0.

Bending Moment

Between Load and Support $= Rax$ or Rbx ins. Tons.

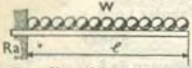
Between Loads (max.) $= \frac{Wa}{2}$ ins. Tons.

Deflection at Centre (max.) $= \frac{Wa}{48EI}(3l^2 - 4a^2)$ ins.

Bending Moment, Shear, and Deflection of Cantilever beams of Uniform Section under various Conditions of Loading.

UNIFORMLY DISTRIBUTED LOAD

Safe Load = $\frac{1}{4}$ Tabular Load



Shear

At any point = $\frac{Wx}{l}$ Tons.

At Ra (max.) = W Tons.

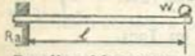
Bending Moment

At Support Ra (max.) = $\frac{Wl}{2}$ ins. Tons.

Deflection (max.) = $\frac{Wl^3}{8EI}$ ins.

CONCENTRATED LOAD AT FREE END

Safe Load = $\frac{1}{8}$ Tabular Load.



Shear

At any point = W Tons.

At Support Ra (max.) = W Tons.

Bending Moment

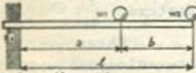
At any point = Wx ins. Tons.

At Support Ra (max.) = Wl ins. Tons.

Deflection (max.) = $\frac{Wl^3}{3EI}$ ins.

2 CONCENTRATED LOADS, AS SHOWN

Safe Load = Tabular Load $\times \frac{(W_1+W_2)l}{8(W_1a+W_2l)}$



Shear

Between W_1 and W_2 = W_2 Tons.

Between Ra and W_1 (max.) = W_1+W_2 Tons.

Bending Moment

Between W_1 and W_2 = W_2x ins. Tons.

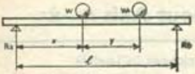
Between Ra and W_1 = $W_1(x-b)+W_2x$ ins. Tons.

At Support Ra (max.) = W_1a+W_2l ins. Tons.

Deflection (max.) = $\frac{W_1a^2(3l-a)+2W_2l^3}{6EI}$ ins.

Distance X referred to above is the dimension from the free end of each cantilever, not from the support.

Bending Moment and Shear of Simply Supported Beams of Uniform Section having Two Equal Concentrated Moving Loads.



Shear

Maximum occurs at Supports *Ra* or *Rb* when $x = 0$

$$= \frac{2W}{l}\left(l - \frac{Y}{2}\right) \text{ Tons.}$$

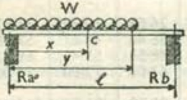
Bending Moment

Maximum occurs under load when distance

$$x = \frac{1}{2}\left(l - \frac{Y}{2}\right) \text{ ins. from Support } Ra$$

$$= \frac{W}{2l}\left(l - \frac{Y}{2}\right)^2 \text{ ins. Tons.}$$

Simply Supported Beam of Uniform Section having a Load uniformly Distributed over a Portion of its Length extending from one Support.

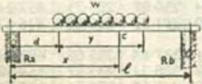


Formula for finding the position of point C where max. B.M. occurs.

Distance $x = Y\left(1 - \frac{Y}{2l}\right)$

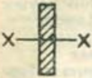
where x, Y, and l are in inches.

Simply Supported Beam of Uniform Section having a Load uniformly Distributed over a Portion of its Length, not extending to either Support.



Formula for finding position of point C where max. B.M. occurs. Distance $x = d + Y\left(1 - \frac{2d + Y}{2l}\right)$

where d, x, Y and l are in inches.



MOMENTS OF INERTIA OF SINGLE PLATES

about Axis x—x

Pl. Depth in Inches	Plate Thickness in Inches					
	$\frac{1}{4}''$	$\frac{5}{16}''$	$\frac{3}{8}''$	$\frac{7}{16}''$	$\frac{1}{2}''$	$\frac{5}{8}''$
1	0·021	0·026	0·031	0·036	0·042	0·052
2	0·17	0·21	0·25	0·29	0·33	0·42
3	0·56	0·70	0·84	0·98	1·13	1·41
4	1·33	1·67	2·00	2·33	2·67	3·33
5	2·60	3·26	3·91	4·56	5·21	6·51
6	4·50	5·63	6·75	7·88	9·00	11·25
7	7·15	8·93	10·72	12·51	14·29	17·86
8	10·67	13·33	16·00	18·67	21·33	26·67
9	15·19	18·98	22·78	26·58	30·38	37·97
10	20·83	26·04	31·25	36·46	41·67	52·08
11	27·73	34·66	41·59	48·53	55·46	69·32
12	36·00	45·00	54·00	63·00	72·00	90·00
13	45·77	57·21	68·66	80·10	91·54	114·43
14	57·17	71·46	85·75	100·04	114·33	142·92
15	70·31	87·89	105·47	123·05	140·63	175·78
16	85·33	106·67	128·00	149·33	170·67	213·33
17	102·35	127·94	153·53	179·12	204·71	255·89
18	121·50	151·88	182·25	212·63	243·00	303·75
19	142·90	178·62	214·34	250·07	285·79	357·24
20	166·67	208·33	250·00	291·67	333·33	416·67
21	192·94	241·17	289·41	337·64	385·88	482·34
22	221·83	277·29	332·75	388·21	443·67	554·58
23	253·48	316·85	380·22	443·59	506·96	633·70

MOMENTS OF INERTIA OF SINGLE PLATES

about Axis x—x

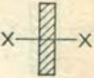
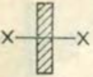


Plate Thickness in Inches						
$\frac{11}{16}''$	$\frac{3}{4}''$	$\frac{13}{16}''$	$\frac{7}{8}''$	$\frac{15}{16}''$	1″	Pl. Depth in Inches
0·057	0·062	0·068	0·073	0·078	0·083	1
0·46	0·50	0·54	0·58	0·63	0·67	2
1·55	1·69	1·83	1·97	2·11	2·25	3
3·67	4·00	4·33	4·67	5·00	5·33	4
7·16	7·81	8·46	9·11	9·77	10·42	5
12·38	13·50	14·63	15·75	16·88	18·00	6
19·65	21·44	23·22	25·01	26·80	28·58	7
29·33	32·00	34·67	37·33	40·00	42·67	8
41·77	45·56	49·36	53·16	56·95	60·75	9
57·29	62·50	67·71	72·92	78·13	83·33	10
76·26	83·19	90·12	97·05	103·98	110·92	11
99·00	108·00	117·00	126·00	135·00	144·00	12
125·87	137·31	148·75	160·20	171·64	183·08	13
157·21	171·50	185·79	200·08	214·38	228·67	14
193·36	210·94	228·52	246·09	263·67	281·25	15
234·67	256·00	277·33	298·67	320·00	341·33	16
281·47	307·06	332·65	358·24	383·83	409·42	17
334·13	364·50	394·88	425·25	455·63	486·00	18
392·96	428·69	464·41	500·14	535·86	571·58	19
458·33	500·00	541·67	583·33	625·00	666·67	20
530·58	578·81	627·05	675·28	723·52	771·75	21
610·04	665·50	720·96	776·42	831·87	887·33	22
697·07	760·44	823·81	887·18	950·55	1013·9	23



MOMENTS OF INERTIA OF SINGLE PLATES

about Axis x—x

Pl. Depth in Inches	Plate Thickness in Inches					
	$\frac{1}{4}''$	$\frac{5}{16}''$	$\frac{3}{8}''$	$\frac{7}{16}''$	$\frac{1}{2}''$	$\frac{5}{8}''$
24	288·00	360·00	432·00	504·00	576·00	720·00
25	325·52	406·90	488·28	569·66	651·04	813·80
26	366·17	457·71	549·25	640·79	732·33	915·42
27	410·06	512·58	615·09	717·61	820·13	1025·2
28	457·33	571·67	686·00	800·33	914·67	1143·3
29	508·10	635·13	762·16	889·19	1016·2	1270·3
30	562·50	703·13	843·75	984·38	1125·0	1406·3
32	682·67	853·33	1024·0	1194·7	1365·3	1706·7
34	818·83	1023·5	1228·2	1433·0	1637·7	2047·1
36	972·00	1215·0	1458·0	1701·0	1944·0	2430·0
38	1143·2	1429·0	1714·7	2000·5	2286·3	2857·9
40	1333·3	1666·7	2000·0	2333·3	2666·7	3333·3
42	1543·5	1929·4	2315·3	2701·1	3087·0	3858·8
44	1774·7	2218·3	2662·0	3105·7	3549·3	4436·7
46	2027·8	2534·8	3041·8	3548·7	4055·7	5069·6
48	2304·0	2880·0	3456·0	4032·0	4608·0	5760·0
50	2604·2	3255·2	3906·3	4557·3	5208·3	6510·4
52	2929·3	3661·7	4394·0	5126·3	5858·7	7323·3
54	3280·5	4100·6	4920·8	5740·9	6561·0	8201·3
56	3658·7	4573·3	5488·0	6402·7	7317·3	9146·7
58	4064·8	5081·0	6097·3	7113·5	8129·7	10162
60	4500·0	5625·0	6750·0	7875·0	9000·0	11250

MOMENTS OF INERTIA OF SINGLE PLATES about Axis x—x

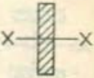
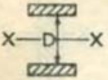


Plate Thickness In Inches						Pl. Depth in Inches
$\frac{11}{16}''$	$\frac{3}{4}''$	$\frac{13}{16}''$	$\frac{7}{8}''$	$\frac{15}{16}''$	1″	
792·00	864·00	936·00	1008·0	1080·0	1152·0	24
895·18	976·56	1057·9	1139·3	1220·7	1302·1	25
1007·0	1098·5	1190·0	1281·6	1373·1	1464·7	26
1127·7	1230·2	1332·7	1435·2	1537·7	1640·3	27
1257·7	1372·0	1486·3	1600·7	1715·0	1829·3	28
1397·3	1524·3	1651·3	1778·4	1905·4	2032·4	29
1546·9	1687·5	1828·1	1968·8	2109·4	2250·0	30
1877·3	2048·0	2218·7	2389·3	2560·0	2730·7	32
2251·8	2456·5	2661·2	2865·9	3070·6	3275·3	34
2673·0	2916·0	3159·0	3402·0	3645·0	3888·0	36
3143·7	3429·5	3715·3	4001·1	4286·9	4572·7	38
3666·7	4000·0	4333·3	4666·7	5000·0	5333·3	40
4244·6	4630·5	5016·4	5402·3	5788·1	6174·0	42
4880·3	5324·0	5767·7	6211·3	6655·0	7098·7	44
5576·5	6083·5	6590·5	7097·4	7604·4	8111·3	46
6336·0	6912·0	7488·0	8064·0	8640·0	9216·0	48
7161·5	7812·5	8463·5	9114·6	9765·6	10417	50
8055·7	8788·0	9520·3	10253	10985	11717	52
9021·4	9841·5	10662	11482	12302	13122	54
10061	10976	11891	12805	13720	14635	56
11178	12194	13211	14227	15243	16259	58
12375	13500	14625	15750	16875	18000	60



MOMENTS OF INERTIA OF TWO PLATES

per Inch of Pl. Width

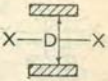
about Axis x—x

Distance 'D' Ins.	Thickness of Each Plate in Inches							
	$\frac{3}{8}''$	$\frac{1}{2}''$	$\frac{5}{8}''$	$\frac{3}{4}''$	$\frac{7}{8}''$	$1''$	$1\frac{1}{8}''$	$1\frac{1}{4}''$
4	3·60	5·08	6·73	8·53	10·5	12·7	15·0	17·6
5	5·43	7·58	9·93	12·5	15·2	18·2	21·3	24·7
6	7·63	10·6	13·8	17·2	20·8	24·7	28·8	33·2
7	10·2	14·1	18·2	22·6	27·2	32·2	37·4	42·9
8	13·2	18·1	23·3	28·8	34·6	40·7	47·1	53·8
9	16·5	22·6	29·0	35·7	42·8	50·2	57·9	66·0
10	20·2	27·6	35·3	43·4	51·9	60·7	69·9	79·4
11	24·3	33·1	42·3	51·8	61·8	72·2	82·9	94·1
12	28·7	39·1	49·9	61·0	72·6	84·7	97·1	110
13	33·6	45·6	58·1	71·0	84·3	98·2	112	127
14	38·8	52·6	66·9	81·7	96·9	113	129	146
15	44·3	60·1	76·3	93·1	110	128	146	165
16	50·3	68·1	86·4	105	125	145	165	186
17	56·6	76·6	97·1	118	140	162	185	208
18	63·3	85·6	108	132	156	181	206	232
19	70·4	95·1	120	146	173	200	228	257
20	77·8	105	133	162	191	221	251	283
21	85·7	116	146	177	209	242	276	310
22	93·9	127	160	194	229	265	301	338
23	102	138	174	212	249	288	328	368
24	111	150	190	230	271	313	355	399
25	121	163	205	249	293	338	384	431
26	130	176	222	268	316	365	414	464
27	141	189	239	289	340	392	445	499
28	151	203	256	310	365	421	477	535
29	162	218	274	332	391	450	511	572
30	173	233	293	355	417	481	545	611

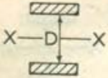
MOMENTS OF INERTIA OF TWO PLATES

per Inch of Pl. Width

about Axis x—x



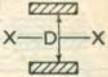
Thickness of Each Plate in Inches								
$1\frac{3}{8}''$	$1\frac{1}{2}''$	$1\frac{5}{8}''$	$1\frac{3}{4}''$	$1\frac{7}{8}''$	$2''$	$2\frac{1}{2}''$	$3''$	Distance 'D' Ins.
20·3	23·2							4
28·4	32·2	36·4						5
37·8	42·8	48·0	53·5	59·2	65·3	92·9	126	6
48·7	54·8	61·2	67·9	74·9	82·3	115	155	7
60·9	68·3	76·0	84·1	92·5	101	140	186	8
74·4	83·3	92·4	102	112	122	168	221	9
89·4	99·8	111	122	133	145	198	258	10
106	118	130	143	157	170	230	299	11
123	137	152	166	182	197	265	342	12
142	158	175	191	209	226	303	389	13
163	181	199	218	237	257	343	438	14
185	205	225	246	268	290	385	491	15
208	230	253	277	301	325	430	546	16
233	257	283	309	335	362	478	605	17
259	286	314	342	371	401	528	666	18
286	316	346	378	410	442	580	731	19
315	347	381	415	450	485	635	798	20
345	380	417	454	492	530	693	869	21
376	415	454	494	535	577	753	942	22
409	451	493	537	581	626	815	1019	23
443	488	534	581	629	677	880	1098	24
479	527	577	627	678	730	948	1181	25
516	568	621	675	730	785	1018	1266	26
554	610	666	724	783	842	1090	1355	27
594	653	714	775	838	901	1165	1446	28
635	698	763	828	895	962	1243	1541	29
677	745	813	883	954	1025	1323	1638	30



MOMENTS OF INERTIA OF TWO PLATES per Inch of Pl. Width about Axis x—x

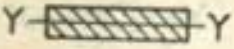
Distance 'D' Ins.	Thickness of Each Plate in Inches						
	$\frac{3}{8}''$	$\frac{1}{2}''$	$\frac{5}{8}''$	$\frac{3}{4}''$	$\frac{7}{8}''$	1″	$1\frac{1}{4}''$
31	185	248	313	378	445	512	650
32	197	264	333	402	473	545	691
33	209	281	353	427	502	578	733
34	222	298	375	453	532	613	777
35	235	315	397	479	563	648	822
36	248	333	419	507	595	685	868
37	262	352	442	534	628	722	915
38	276	371	466	563	661	761	963
39	291	390	491	593	696	800	1013
40	306	410	516	623	731	841	1064
41	321	431	541	654	767	882	1116
42	337	452	568	685	804	925	1169
44	369	495	622	751	881	1013	1280
46	403	541	679	820	961	1105	1396
48	439	588	739	891	1045	1201	1516
50	476	638	801	966	1132	1301	1642
52	514	689	865	1044	1223	1405	1773
54	554	743	933	1124	1318	1513	1908
60	683	915	1149	1384	1621	1861	2345
66	826	1106	1387	1671	1957	2245	2827
72	982	1314	1648	1985	2324	2665	3354
78	1152	1541	1932	2326	2722	3121	3926
84	1335	1785	2238	2694	3152	3613	4543
90	1531	2048	2567	3088	3613	4141	5204
96	1742	2328	2918	3510	4106	4705	5911

MOMENTS OF INERTIA OF TWO PLATES per Inch of Pl. Width about Axis x—x



Thickness of Each Plate in Inches							
$1\frac{3}{8}''$	$1\frac{1}{2}''$	$1\frac{3}{4}''$	$1\frac{7}{8}''$	2	$2\frac{1}{2}''$	$3''$	Distance 'D' Ins.
721	793	939	1014	1090	1405	1739	31
766	842	998	1077	1157	1490	1842	32
813	893	1058	1141	1226	1578	1949	33
861	946	1119	1208	1297	1668	2058	34
910	1000	1183	1276	1370	1760	2171	35
961	1055	1248	1346	1445	1855	2286	36
1013	1112	1315	1418	1522	1953	2405	37
1066	1171	1383	1492	1601	2053	2526	38
1121	1231	1454	1567	1682	2155	2651	39
1177	1292	1526	1645	1765	2260	2778	40
1235	1355	1600	1725	1850	2368	2909	41
1294	1420	1676	1806	1937	2478	3042	42
1416	1553	1832	1974	2117	2705	3318	44
1543	1693	1996	2150	2305	2943	3606	46
1676	1838	2167	2333	2501	3190	3906	48
1815	1990	2344	2524	2705	3448	4218	50
1959	2147	2529	2722	2917	3715	4542	52
2109	2311	2720	2928	3137	3993	4878	54
2590	2837	3337	3590	3845	4885	5958	60
3121	3418	4017	4320	4625	5868	7146	66
3702	4052	4760	5118	5477	6940	8442	72
4332	4741	5566	5982	6401	8103	9846	78
5012	5483	6435	6915	7397	9355	11358	84
5741	6280	7367	7915	8465	10698	12978	90
6519	7130	8362	8982	9605	12130	14706	96

MOMENTS OF INERTIA OF SINGLE PLATES



About Axis y—y

Pl. Thickness in Inches	Plate Width in Inches 6″	7″	8″	9″	10″	12″
1/4	0·008	0·009	0·010	0·012	0·013	0·016
5/16	0·015	0·018	0·020	0·023	0·025	0·030
3/8	0·026	0·031	0·035	0·040	0·044	0·053
7/16	0·042	0·049	0·056	0·063	0·070	0·084
1/2	0·062	0·073	0·083	0·094	0·104	0·125
9/16	0·089	0·104	0·119	0·133	0·148	0·178
5/8	0·122	0·142	0·163	0·183	0·203	0·244
11/16	0·162	0·190	0·217	0·244	0·271	0·325
3/4	0·211	0·246	0·281	0·316	0·352	0·422
13/16	0·268	0·313	0·358	0·402	0·447	0·536
7/8	0·335	0·391	0·447	0·502	0·558	0·670
15/16	0·412	0·481	0·549	0·618	0·687	0·824
1	0·500	0·583	0·667	0·750	0·833	1·000
1 1/16	0·600	0·700	0·800	0·900	1·000	1·199
1 1/8	0·712	0·831	0·949	1·068	1·187	1·424
1 3/16	0·837	0·977	1·116	1·256	1·395	1·675
1 1/4	0·976	1·139	1·302	1·465	1·628	1·953
1 5/16	1·130	1·319	1·507	1·696	1·884	2·261
1 3/8	1·300	1·517	1·733	1·950	2·166	2·600
1 7/16	1·485	1·733	1·980	2·228	2·475	2·970
1 1/2	1·687	1·969	2·250	2·531	2·812	3·375
1 9/16	1·907	2·225	2·543	2·861	3·179	3·815
1 5/8	2·145	2·503	2·861	3·218	3·576	4·291
1 11/16	2·403	2·803	3·204	3·604	4·005	4·805

MOMENTS OF INERTIA OF SINGLE PLATES

About Axis y—y

Y—Y

14"	15"	16"	18"	20"	24"	Pl. Thickness in Inches
		Plate Width in Inches				
0·018	0·020	0·021	0·023	0·026	0·031	$\frac{1}{4}$
0·035	0·038	0·040	0·046	0·051	0·061	$\frac{5}{16}$
0·062	0·066	0·070	0·079	0·088	0·105	$\frac{3}{8}$
0·098	0·105	0·112	0·126	0·140	0·167	$\frac{7}{16}$
0·146	0·156	0·167	0·187	0·208	0·250	$\frac{1}{2}$
0·208	0·222	0·237	0·267	0·297	0·356	$\frac{9}{16}$
0·285	0·305	0·326	0·366	0·407	0·488	$\frac{5}{8}$
0·379	0·406	0·433	0·487	0·542	0·650	$\frac{11}{16}$
0·492	0·527	0·562	0·633	0·703	0·844	$\frac{3}{4}$
0·626	0·670	0·715	0·805	0·894	1·073	$\frac{13}{16}$
0·782	0·837	0·893	1·005	1·117	1·340	$\frac{7}{8}$
0·961	1·030	1·099	1·236	1·373	1·648	$\frac{15}{16}$
1·167	1·250	1·333	1·500	1·667	2·000	1
1·399	1·499	1·599	1·799	1·999	2·399	$1\frac{1}{16}$
1·661	1·780	1·898	2·136	2·373	2·848	$1\frac{1}{8}$
1·954	2·093	2·233	2·512	2·791	3·349	$1\frac{3}{16}$
2·279	2·441	2·604	2·930	3·255	3·906	$1\frac{1}{4}$
2·638	2·826	3·015	3·391	3·768	4·522	$1\frac{5}{16}$
3·033	3·249	3·466	3·899	4·333	5·199	$1\frac{3}{8}$
3·466	3·713	3·961	4·456	4·951	5·941	$1\frac{7}{16}$
3·937	4·219	4·500	5·062	5·625	6·750	$1\frac{1}{2}$
4·450	4·768	5·086	5·722	6·358	7·629	$1\frac{9}{16}$
5·006	5·364	5·721	6·437	7·152	8·582	$1\frac{5}{8}$
5·606	6·006	6·407	7·208	8·009	9·611	$1\frac{11}{16}$

MOMENTS OF INERTIA OF SINGLE PLATES

Y—Y

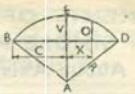
About Axis y—y

Pl. Thickness in Inches	Plate Width in Inches					
	6″	7″	8″	9″	10″	12″
$1\frac{3}{4}$	2·679	3·126	3·573	4·020	4·466	5·359
$1\frac{13}{16}$	2·977	3·473	3·970	4·466	4·962	5·954
$1\frac{7}{8}$	3·296	3·845	4·395	4·944	5·493	6·592
$1\frac{15}{16}$	3·637	4·243	4·849	5·455	6·061	7·273
2	4·000	4·667	5·333	6·000	6·667	8·000
$2\frac{1}{16}$	4·387	5·118	5·849	6·580	7·311	8·774
$2\frac{1}{8}$	4·798	5·598	6·397	7·197	7·996	9·596
$2\frac{3}{16}$	5·234	6·106	6·978	7·851	8·723	10·47
$2\frac{1}{4}$	5·695	6·645	7·594	8·543	9·492	11·39
$2\frac{5}{16}$	6·183	7·214	8·244	9·275	10·30	12·37
$2\frac{3}{8}$	6·698	7·814	8·931	10·05	11·16	13·40
$2\frac{7}{16}$	7·241	8·448	9·655	10·86	12·07	14·48
$2\frac{1}{2}$	7·812	9·115	10·42	11·72	13·02	15·62
$2\frac{9}{16}$	8·413	9·815	11·22	12·62	14·02	16·83
$2\frac{5}{8}$	9·044	10·55	12·06	13·57	15·07	18·09
$2\frac{11}{16}$	9·705	11·32	12·94	14·56	16·18	19·41
$2\frac{3}{4}$	10·40	12·13	13·87	15·60	17·33	20·80
$2\frac{13}{16}$	11·12	12·98	14·83	16·69	18·54	22·25
$2\frac{7}{8}$	11·88	13·86	15·84	17·82	19·80	23·76
$2\frac{15}{16}$	12·67	14·79	16·90	19·01	21·12	25·35
3	13·50	15·75	18·00	20·25	22·50	27·00
$3\frac{1}{4}$	17·16	20·02	22·88	25·75	28·61	34·33
$3\frac{1}{2}$	21·44	25·01	28·58	32·16	35·73	42·87
$3\frac{3}{4}$	26·37	30·76	35·16	39·55	43·94	52·73

MOMENTS OF INERTIA OF SINGLE PLATES
About Axis y—y

Plate Width in Inches						Pl. Thickness in Inches
14″	15″	16″	18″	20″	24″	
6·253	6·699	7·146	8·039	8·932	10·72	1¾
6·947	7·443	7·939	8·932	9·924	11·91	1 13/16
7·690	8·240	8·789	9·888	10·99	13·18	1⅞
8·485	9·091	9·698	10·91	12·12	14·55	1 15/16
9·333	10·00	10·67	12·00	13·33	16·00	2
10·24	10·97	11·70	13·16	14·62	17·55	2 1/16
11·20	11·99	12·79	14·39	15·99	19·19	2⅛
12·21	13·08	13·96	15·70	17·45	20·94	2 3/16
13·29	14·24	15·19	17·09	18·98	22·78	2¼
14·43	15·46	16·49	18·55	20·61	24·73	2 5/16
15·63	16·74	17·86	20·10	22·33	26·79	2⅜
16·90	18·10	19·31	21·72	24·14	28·96	2 7/16
18·23	19·53	20·83	23·44	26·04	31·25	2½
19·63	21·03	22·44	25·24	28·04	33·65	2 9/16
21·10	22·61	24·12	27·13	30·15	36·18	2⅝
22·65	24·26	25·88	29·12	32·35	38·82	2 11/16
24·26	25·99	27·73	31·20	34·66	41·59	2¾
25·96	27·81	29·66	33·37	37·08	44·50	2 13/16
27·72	29·70	31·69	35·65	39·61	47·53	2⅞
29·57	31·68	33·80	38·02	42·25	50·70	2 15/16
31·50	33·75	36·00	40·50	45·00	54·00	3
40·05	42·91	45·77	51·49	57·21	68·66	3¼
50·02	53·59	57·17	64·31	71·46	85·75	3½
61·52	65·92	70·31	79·10	87·89	105·47	3¾

MENSURATION



Circumference of Circle = Diameter × π.
Diameter of Circle = Circumference × ·3183.
Length of Arc = No. of Degrees × Radius × ·017453.
Degrees in an arc of length equal to Radius = 57·2957795°. π = 3·14159265+.
V = Versed sine. C = Half the chord. R = radius.
O = any ordinate. X = distance of ordinate from centre.

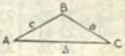
$$O = \sqrt{R^2 - X^2} - (R - V). \qquad R = \frac{V^2 + C^2}{2V}.$$

$$V = R - \sqrt{R^2 - C^2}, \qquad X = \sqrt{R^2 - (O + R - V)^2}.$$

Area of Triangle = Base × Half Perp. Height.
Area of Circle = πR^2, where R = radius.
Area of Parabola = Base × $\frac{2}{3}$ height.

TRIGONOMETRICAL FORMULAE

Oblique Angle Triangles

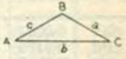


Given	Requd.	Formulae	
A, C, a	c	$c = \frac{a \sin C}{\sin A}$	
A, a, b	B	$\sin B = \frac{b \sin A}{a}$	
a, c, B	A	$\tan A = \frac{a \sin C}{b - a \cos C}$	*see page 381 for area
a, b, c	A, B, C	$\sin A = \frac{2\text{Area}^*}{bc}$; $\sin B = \frac{2\text{Area}^*}{ac}$; $\sin C = \frac{2\text{Area}^*}{ab}$	

TRIGONOMETRICAL FORMULAE

Oblique Angle Triangles—Contd.

Given	Areas Formulae
A, B, C, a	$\dfrac{a^2 \sin B \sin C}{2 \sin A}$
A, b, c	$\frac{1}{2}(b\,c \sin A)$
a, b, c	* $\sqrt{s(s-a)(s-b)(s-c)}$ where $s=\dfrac{a+b+c}{2}$



Right Angle Triangles

$\text{Sine}=\dfrac{\text{Perp.}}{\text{Hyp.}}$; $\text{Cosine}=\dfrac{\text{Base}}{\text{Hyp.}}$; $\text{Tangent}=\dfrac{\text{Perp.}}{\text{Base}}$;

$\text{Cotan}=\dfrac{\text{Base}}{\text{Perp.}}$; $\text{Sec}=\dfrac{\text{Hyp.}}{\text{Base}}$; $\text{Cosec}=\dfrac{\text{Hyp.}}{\text{Perp.}}$;

$\text{Versine}=\dfrac{\text{Hyp.}-\text{Base}}{\text{Hyp.}}$; $\text{Coversine}=\dfrac{\text{Hyp.}-\text{Perp.}}{\text{Hyp.}}$;

$\text{Hyp}=\sqrt{\text{Base}^2+\text{Perp.}^2}$; $\text{Base}=\sqrt{\text{Hyp.}^2-\text{Perp.}^2}$;

$\text{Perp.}=\sqrt{\text{Hyp.}^2-\text{Base}^2}$.

Given	Requd.	Formulae
A, a	C, c, b	$c=a\cot A$; $b=\dfrac{a}{\sin A}$; $C=90^\circ-A$
A, b	C, a, c	$a=b\sin A$; $c=b\cos A$; $C=90^\circ-A$
A, c	C, a, b	$a=c\tan A$; $b=\dfrac{c}{\cos A}$; $C=90^\circ-A$
a, b	A, C, c	$\sin A=\dfrac{a}{b}$; $\cos C=\dfrac{a}{b}$; $c=\sqrt{[(b+a)(b-a)]}$
a, c	A, C, b	$\tan A=\dfrac{a}{c}$; $\cot C=\dfrac{a}{c}$; $b=\sqrt{(a^2+c^2)}$.

TRIGONOMETRICAL RATIOS

Angle	Sine	Tangent	Co-tangent	Cosine	
0·5	0·0087	0·0087	114·589	1·0000	89·5
1·0	0·0175	0·0175	57·290	0·9998	89·0
1·5	0·0262	0·0262	38·188	0·9997	88·5
2·0	0·0349	0·0349	28·636	0·9994	88·0
2·5	0·0436	0·0437	22·904	0·9990	87·5
3·0	0·0523	0·0524	19·081	0·9986	87·0
3·5	0·0610	0·0612	16·350	0·9981	86·5
4·0	0·0698	0·0699	14·301	0·9976	86·0
4·5	0·0785	0·0787	12·706	0·9969	85·5
5·0	0·0872	0·0875	11·430	0·9962	85·0
5·5	0·0958	0·0963	10·385	0·9954	84·5
6·0	0·1045	0·1051	9·5144	0·9945	84·0
6·5	0·1132	0·1139	8·7769	0·9936	83·5
7·0	0·1219	0·1228	8·1443	0·9925	83·0
7·5	0·1305	0·1317	7·5958	0·9914	82·5
8·0	0·1392	0·1405	7·1154	0·9903	82·0
8·5	0·1478	0·1495	6·6912	0·9890	81·5
9·0	0·1564	0·1584	6·3138	0·9877	81·0
9·5	0·1650	0·1673	5·9758	0·9863	80·5
10·0	0·1736	0·1763	5·6713	0·9848	80·0
10·5	0·1822	0·1853	5·3955	0·9833	79·5
11·0	0·1908	0·1944	5·1446	0·9816	79·0
11·5	0·1994	0·2035	4·9152	0·9799	78·5
12·0	0·2079	0·2126	4·7046	0·9781	78·0
12·5	0·2164	0·2217	4·5107	0·9763	77·5
13·0	0·2250	0·2309	4·3315	0·9744	77·0
13·5	0·2334	0·2401	4·1653	0·9724	76·5
14·0	0·2419	0·2493	4·0108	0·9703	76·0
14·5	0·2504	0·2586	3·8667	0·9681	75·5
15·0	0·2588	0·2679	3·7321	0·9659	75·0
	Cosine	Co-tangent	Tangent	Sine	Angle

TRIGONOMETRICAL RATIOS

Angle	Sine	Tangent	Co-tangent	Cosine	
15·5	0·2672	0·2773	3·6059	0·9636	74·5
16·0	0·2756	0·2867	3·4874	0·9613	74·0
16·5	0·2840	0·2962	3·3759	0·9588	73·5
17·0	0·2924	0·3057	3·2709	0·9563	73·0
17·5	0·3007	0·3153	3·1716	0·9537	72·5
18·0	0·3090	0·3249	3·0777	0·9511	72·0
18·5	0·3173	0·3346	2·9887	0·9483	71·5
19·0	0·3256	0·3443	2·9042	0·9455	71·0
19·5	0·3338	0·3541	2·8239	0·9426	70·5
20·0	0·3420	0·3640	2·7475	0·9397	70·0
20·5	0·3502	0·3739	2·6746	0·9367	69·5
21·0	0·3584	0·3839	2·6051	0·9336	69·0
21·5	0·3665	0·3939	2·5386	0·9304	68·5
22·0	0·3746	0·4040	2·4751	0·9272	68·0
22·5	0·3827	0·4142	2·4142	0·9239	67·5
23·0	0·3907	0·4245	2·3559	0·9205	67·0
23·5	0·3987	0·4348	2·2998	0·9171	66·5
24·0	0·4067	0·4452	2·2460	0·9135	66·0
24·5	0·4147	0·4557	2·1943	0·9100	65·5
25·0	0·4226	0·4663	2·1445	0·9063	65·0
25·5	0·4305	0·4770	2·0965	0·9026	64·5
26·0	0·4384	0·4877	2·0503	0·8988	64·0
26·5	0·4462	0·4986	2·0057	0·8949	63·5
27·0	0·4540	0·5095	1·9626	0·8910	63·0
27·5	0·4617	0·5206	1·9210	0·8870	62·5
28·0	0·4695	0·5317	1·8807	0·8829	62·0
28·5	0·4772	0·5430	1·8418	0·8788	61·5
29·0	0·4848	0·5543	1·8040	0·8746	61·0
29·5	0·4924	0·5658	1·7675	0·8704	60·5
30·0	0·5000	0·5774	1·7321	0·8660	60·0
	Cosine	Co-tangent	Tangent	Sine	Angle

TRIGONOMETRICAL RATIOS

Angle	Sine	Tangent	Co-tangent	Cosine	
30·5	0·5075	0·5890	1·6977	0·8616	59·5
31·0	0·5150	0·6009	1·6643	0·8572	59·0
31·5	0·5225	0·6128	1·6319	0·8526	58·5
32·0	0·5299	0·6249	1·6003	0·8480	58·0
32·5	0·5373	0·6371	1·5697	0·8434	57·5
33·0	0·5446	0·6494	1·5399	0·8387	57·0
33·5	0·5519	0·6619	1·5108	0·8339	56·5
34·0	0·5592	0·6745	1·4826	0·8290	56·0
34·5	0·5664	0·6873	1·4550	0·8241	55·5
35·0	0·5736	0·7002	1·4281	0·8192	55·0
35·5	0·5807	0·7133	1·4019	0·8141	54·5
36·0	0·5878	0·7265	1·3764	0·8090	54·0
36·5	0·5948	0·7400	1·3514	0·8039	53·5
37·0	0·6018	0·7536	1·3270	0·7986	53·0
37·5	0·6088	0·7673	1·3032	0·7934	52·5
38·0	0·6157	0·7813	1·2799	0·7880	52·0
38·5	0·6225	0·7954	1·2572	0·7826	51·5
39·0	0·6293	0·8098	1·2349	0·7771	51·0
39·5	0·6361	0·8243	1·2131	0·7716	50·5
40·0	0·6428	0·8391	1·1918	0·7660	50·0
40·5	0·6494	0·8541	1·1708	0·7604	49·5
41·0	0·6561	0·8693	1·1504	0·7547	49·0
41·5	0·6626	0·8847	1·1303	0·7490	48·5
42·0	0·6691	0·9004	1·1106	0·7431	48·0
42·5	0·6756	0·9163	1·0913	0·7373	47·5
43·0	0·6820	0·9325	1·0724	0·7314	47·0
43·5	0·6884	0·9490	1·0538	0·7254	46·5
44·0	0·6947	0·9657	1·0355	0·7193	46·0
44·5	0·7009	0·9827	1·0176	0·7133	45·5
45·0	0·7071	1·0000	1·0000	0·7071	45·0
	Cosine	Co-tangent	Tangent	Sine	Angle

WEIGHTS OF FLAT STEEL SHEETS with Decimal and Fractional Equivalents of Gauge Thickness.

Gauge (B.G.)	Decimal Thickness in Inches	Nearest Fractional Equivalent	Weights in lbs. per sq. ft.	Gauge (B.G.)	Decimal Thickness in Inches	Nearest Fractional Equivalent	Weights in lbs. per sq. ft.
1	0·353	$\frac{23}{64}$	14·41	16	0·0625	$\frac{1}{16}$	2·55
2	0·315	$\frac{5}{16}$	12·84	17	0·0556	$\frac{1}{16}$	2·27
3	0·280	$\frac{9}{32}$	11·44	18	0·0495	$\frac{3}{64}$	2·02
4	0·2500	$\frac{1}{4}$	10·20	19	0·0440	$\frac{3}{64}$	1·80
5	0·2225	$\frac{7}{32}$	9·08	20	0·0392	$\frac{3}{64}$	1·60
6	0·1981	$\frac{13}{64}$	8·08	21	0·0349	$\frac{1}{32}$	1·42
7	0·1764	$\frac{11}{64}$	7·20	22	0·03125	$\frac{1}{32}$	1·28
8	0·1570	$\frac{5}{32}$	6·41	23	0·02782	$\frac{1}{32}$	1·13
9	0·1398	$\frac{9}{64}$	5·70	24	0·02476	$\frac{1}{32}$	1·01
10	0·1250	$\frac{1}{8}$	5·10	25	0·02204	$\frac{1}{64}$	0·90
11	0·1113	$\frac{7}{64}$	4·54	26	0·01961	$\frac{1}{64}$	0·80
12	0·0991	$\frac{3}{32}$	4·04	27	0·01745	$\frac{1}{64}$	0·71
13	0·0882	$\frac{3}{32}$	3·60	28	0·01563	$\frac{1}{64}$	0·64
14	0·0785	$\frac{5}{64}$	3·20	29	0·01390	$\frac{1}{64}$	0·57
15	0·0699	$\frac{1}{16}$	2·85	30	0·01230	$\frac{1}{64}$	0·50

B.G. denotes Birmingham Sheet and Hoop Gauge which is the customary commercial gauge used in Britain for Steel Sheets, Black or Galvanised, and which was legalised July 10th 1914.

AREAS OF CIRCLES

Diameter	0	1/8	1/4	3/8
0		0·0123	0·0491	0·1104
1	0·7854	0·9940	1·227	1·485
2	3·142	3·547	3·976	4·430
3	7·069	7·670	8·296	8·946
4	12·566	13·364	14·186	15·033
5	19·635	20·629	21·648	22·691
6	28·274	29·465	30·680	31·919
7	38·485	39·871	41·282	42·718
8	50·265	51·849	53·456	55·088
9	63·617	65·397	67·201	69·029
10	78·540	80·516	82·516	84·541
11	95·033	97·205	99·402	101·62
12	113·10	115·47	117·86	120·28
13	132·73	135·30	137·89	140·50
14	153·94	156·70	159·48	162·30
15	176·71	179·67	182·65	185·66
16	201·06	204·22	207·39	210·60
17	226·98	230·33	233·71	237·10
18	254·47	258·02	261·59	265·18
19	283·53	287·27	291·04	294·83
20	314·16	318·10	322·06	326·05
21	346·36	350·50	354·66	358·84
22	380·13	384·46	388·82	393·20
23	415·48	420·00	424·56	429·13
24	452·39	457·11	461·86	466·64
25	490·87	495·79	500·74	505·71
26	530·93	536·05	541·19	546·35
27	572·56	577·87	583·21	588·57
28	615·75	621·26	626·80	632·36
29	660·52	666·23	671·96	677·71
30	706·86	712·76	718·69	724·64
31	754·77	760·87	766·99	773·14
32	804·25	810·54	816·86	823·21
33	855·30	861·79	868·31	874·85
34	907·92	914·61	921·32	928·06
35	962·11	969·00	975·91	982·84
36	1017·9	1025·0	1032·1	1039·2

AREAS OF CIRCLES

$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	Diameter
0·1963	0·3068	0·4418	0·6013	0
1·767	2·074	2·405	2·761	1
4·909	5·412	5·940	6·492	2
9·621	10·321	11·045	11·793	3
15·904	16·800	17·721	18·665	4
23·758	24·850	25·967	27·109	5
33·183	34·472	35·785	37·122	6
44·179	45·664	47·173	48·707	7
56·745	58·426	60·132	61·862	8
70·882	72·760	74·662	76·589	9
86·590	88·664	90·763	92·886	10
103·87	106·14	108·43	110·75	11
122·72	125·19	127·68	130·19	12
143·14	145·80	148·49	151·20	13
165·13	167·99	170·87	173·78	14
188·69	191·75	194·83	197·93	15
213·82	217·08	220·35	223·65	16
240·53	243·98	247·45	250·95	17
268·80	272·45	276·12	279·81	18
298·65	302·49	306·35	310·24	19
330·06	334·10	338·16	342·25	20
363·05	367·28	371·54	375·83	21
397·61	402·04	406·49	410·97	22
433·74	438·36	443·01	447·69	23
471·44	476·26	481·11	485·98	24
510·71	515·72	520·77	525·84	25
551·55	556·76	562·00	567·27	26
593·96	599·37	604·81	610·27	27
637·94	643·55	649·18	654·84	28
683·49	689·30	695·13	700·98	29
730·62	736·62	742·64	748·69	30
779·31	785·51	791·73	797·98	31
829·58	835·97	842·39	848·83	32
881·41	888·00	894·62	901·26	33
934·82	941·61	948·42	955·25	34
989·80	996·78	1003·8	1010·8	35
1046·3	1053·5	1060·7	1068·0	36

AREAS OF CIRCLES

Diameter	0	1/8	1/4	3/8
37	1075·2	1082·5	1089·8	1097·1
38	1134·1	1141·6	1149·1	1156·6
39	1194·6	1202·3	1210·0	1217·7
40	1256·6	1264·5	1272·4	1280·3
41	1320·3	1328·3	1336·4	1344·5
42	1385·4	1393·7	1402·0	1410·3
43	1452·2	1460·7	1469·1	1477·6
44	1520·5	1529·2	1537·9	1546·6
45	1590·4	1599·3	1608·2	1617·0
46	1661·9	1670·9	1680·0	1689·1
47	1734·9	1744·2	1753·5	1762·7
48	1809·6	1819·0	1828·5	1837·9
49	1885·7	1895·4	1905·0	1914·7
50	1963·5	1973·3	1983·2	1993·1
51	2042·8	2052·8	2062·9	2073·0
52	2123·7	2133·9	2144·2	2154·5
53	2206·2	2216·6	2227·0	2237·5
54	2290·2	2300·8	2311·5	2322·1
55	2375·8	2386·6	2397·5	2408·3
56	2463·0	2474·0	2485·0	2496·1
57	2551·8	2563·0	2574·2	2585·4
58	2642·1	2653·5	2664·9	2676·4
59	2734·0	2745·6	2757·2	2768·8
60	2827·4	2839·2	2851·0	2862·9
61	2922·5	2934·5	2946·5	2958·5
62	3019·1	3031·3	3043·5	3055·7
63	3117·2	3129·6	3142·0	3154·5
64	3217·0	3229·6	3242·2	3254·8
65	3318·3	3331·1	3343·9	3356·7
66	3421·2	3434·2	3447·2	3460·2
67	3525·7	3538·8	3552·0	3565·2
68	3631·7	3645·0	3658·4	3671·8
69	3739·3	3752·8	3766·4	3780·0
70	3848·5	3862·2	3876·0	3889·8
71	3959·2	3973·1	3987·1	4001·1
72	4071·5	4085·7	4099·8	4114·0

AREAS OF CIRCLES

1/2	5/8	3/4	7/8	Diameter
1104·5	1111·8	1119·2	1126·7	37
1164·2	1171·7	1179·3	1186·9	38
1225·4	1233·2	1241·0	1248·8	39
1288·2	1296·2	1304·2	1312·2	40
1352·7	1360·8	1369·0	1377·2	41
1418·6	1427·0	1435·4	1443·8	42
1486·2	1494·7	1503·3	1511·9	43
1555·3	1564·0	1572·8	1581·6	44
1626·0	1634·9	1643·9	1652·9	45
1698·2	1707·4	1716·5	1725·7	46
1772·1	1781·4	1790·8	1800·1	47
1847·5	1857·0	1866·5	1876·1	48
1924·4	1934·2	1943·9	1953·7	49
2003·0	2012·9	2022·8	2032·8	50
2083·1	2093·2	2103·3	2113·5	51
2164·8	2175·1	2185·4	2195·8	52
2248·0	2258·5	2269·1	2279·6	53
2332·8	2343·5	2354·3	2365·0	54
2419·2	2430·1	2441·1	2452·0	55
2507·2	2518·3	2529·4	2540·6	56
2596·7	2608·0	2619·4	2630·7	57
2687·8	2699·3	2710·9	2722·4	58
2780·5	2792·2	2803·9	2815·7	59
2874·8	2886·6	2898·6	2910·5	60
2970·6	2982·7	2994·8	3006·9	61
3068·0	3080·3	3092·6	3104·9	62
3166·9	3179·4	3191·9	3204·4	63
3267·5	3280·1	3292·8	3305·6	64
3369·6	3382·4	3395·3	3408·2	65
3473·2	3486·3	3499·4	3512·5	66
3578·5	3591·7	3605·0	3618·3	67
3685·3	3698·7	3712·2	3725·7	68
3793·7	3807·3	3821·0	3834·7	69
3903·6	3917·5	3931·4	3945·3	70
4015·2	4029·2	4043·3	4057·4	71
4128·2	4142·5	4156·8	4171·1	72

FRACTIONS OF A FOOT

Vulgar and Decimal Fractions of a Foot		Equivalents in Inches	Vulgar and Decimal Fractions of a Foot		Equivalents in Inches	Vulgar and Decimal Fractions of a Foot		Equivalents in Inches
	·0052	$\frac{1}{16}$	$\frac{11}{64}$	·17187	$2\frac{1}{16}$		·3385	$4\frac{1}{16}$
	·0104	$\frac{1}{8}$		·1771	$2\frac{1}{8}$	$\frac{11}{32}$	·34375	$4\frac{1}{8}$
$\frac{1}{64}$	·01562	$\frac{3}{16}$		·1823	$2\frac{3}{16}$		·3490	$4\frac{3}{16}$
	·0208	$\frac{1}{4}$	$\frac{3}{16}$	·1875	$2\frac{1}{4}$		·3542	$4\frac{1}{4}$
	·0260	$\frac{5}{16}$		·1927	$2\frac{5}{16}$	$\frac{23}{64}$	·35937	$4\frac{5}{16}$
$\frac{1}{32}$	·03125	$\frac{3}{8}$		·1979	$2\frac{3}{8}$		·3646	$4\frac{3}{8}$
	·0365	$\frac{7}{16}$	$\frac{13}{64}$	·20312	$2\frac{7}{16}$		·3698	$4\frac{7}{16}$
	·0417	$\frac{1}{2}$		·2083	$2\frac{1}{2}$	$\frac{3}{8}$	·3750	$4\frac{1}{2}$
$\frac{3}{64}$	·04687	$\frac{9}{16}$		·2135	$2\frac{9}{16}$		·3802	$4\frac{9}{16}$
	·0521	$\frac{5}{8}$	$\frac{7}{32}$	·21875	$2\frac{5}{8}$		·3854	$4\frac{5}{8}$
	·0573	$\frac{11}{16}$		·2240	$2\frac{11}{16}$	$\frac{25}{64}$	·39062	$4\frac{11}{16}$
$\frac{1}{16}$	·0625	$\frac{3}{4}$		·2292	$2\frac{3}{4}$		·3958	$4\frac{3}{4}$
	·0677	$\frac{13}{16}$	$\frac{15}{64}$	·23437	$2\frac{13}{16}$		·4010	$4\frac{13}{16}$
	·0729	$\frac{7}{8}$		·2396	$2\frac{7}{8}$	$\frac{13}{32}$	·40625	$4\frac{7}{8}$
$\frac{5}{64}$	·07812	$\frac{15}{16}$		·2448	$2\frac{15}{16}$		·4115	$4\frac{15}{16}$
	·0833	1	$\frac{1}{4}$	·2500	3		·4167	5
	·0885	$1\frac{1}{16}$		·2552	$3\frac{1}{16}$	$\frac{27}{64}$	·42187	$5\frac{1}{16}$
$\frac{3}{32}$	·09375	$1\frac{1}{8}$		·2604	$3\frac{1}{8}$		·4271	$5\frac{1}{8}$
	·0990	$1\frac{3}{16}$	$\frac{17}{64}$	·26562	$3\frac{3}{16}$		·4323	$5\frac{3}{16}$
	·1042	$1\frac{1}{4}$		·2708	$3\frac{1}{4}$	$\frac{7}{16}$	·4375	$5\frac{1}{4}$
$\frac{7}{64}$	·10937	$1\frac{5}{16}$		·2760	$3\frac{5}{16}$		·4427	$5\frac{5}{16}$
	·1146	$1\frac{3}{8}$	$\frac{9}{32}$	·28125	$3\frac{3}{8}$		·4479	$5\frac{3}{8}$
	·1198	$1\frac{7}{16}$		·2865	$3\frac{7}{16}$	$\frac{29}{64}$	·45312	$5\frac{7}{16}$
$\frac{1}{8}$	·1250	$1\frac{1}{2}$		·2917	$3\frac{1}{2}$		·4583	$5\frac{1}{2}$
	·1302	$1\frac{9}{16}$	$\frac{19}{64}$	·29687	$3\frac{9}{16}$		·4635	$5\frac{9}{16}$
	·1354	$1\frac{5}{8}$		·3021	$3\frac{5}{8}$	$\frac{15}{32}$	·46875	$5\frac{5}{8}$
$\frac{9}{64}$	·14062	$1\frac{11}{16}$		·3073	$3\frac{11}{16}$		·4740	$5\frac{11}{16}$
	·1458	$1\frac{3}{4}$	$\frac{5}{16}$	·3125	$3\frac{3}{4}$		·4792	$5\frac{3}{4}$
	·1510	$1\frac{13}{16}$		·3177	$3\frac{13}{16}$	$\frac{31}{64}$	·48437	$5\frac{13}{16}$
$\frac{5}{32}$	·15625	$1\frac{7}{8}$		·3229	$3\frac{7}{8}$		·4896	$5\frac{7}{8}$
	·1615	$1\frac{15}{16}$	$\frac{21}{64}$	·32812	$3\frac{15}{16}$		·4948	$5\frac{15}{16}$
	·1667	2		·3333	4	$\frac{1}{2}$	·5000	6

FRACTIONS OF A FOOT

Vulgar and Decimal Fractions of a Foot		Equivalents in Inches	Vulgar and Decimal Fractions of a Foot		Equivalents in Inches	Vulgar and Decimal Fractions of a Foot		Equivalents in Inches
	·5052	$6\frac{1}{16}$	$\frac{43}{64}$	·67187	$8\frac{1}{16}$		·8385	$10\frac{1}{16}$
	·5104	$6\frac{1}{8}$		·6771	$8\frac{1}{8}$	$\frac{27}{32}$	·84375	$10\frac{1}{8}$
$\frac{33}{64}$	·51562	$6\frac{3}{16}$		·6823	$8\frac{3}{16}$		·8490	$10\frac{3}{16}$
	·5208	$6\frac{1}{4}$	$\frac{11}{16}$	·6875	$8\frac{1}{4}$		·8542	$10\frac{1}{4}$
	·5260	$6\frac{5}{16}$		·6927	$8\frac{5}{16}$	$\frac{55}{64}$	·85937	$10\frac{5}{16}$
$\frac{17}{32}$	·53125	$6\frac{3}{8}$		·6979	$8\frac{3}{8}$		·8646	$10\frac{3}{8}$
	·5365	$6\frac{7}{16}$	$\frac{45}{64}$	·70312	$8\frac{7}{16}$		·8698	$10\frac{7}{16}$
	·5417	$6\frac{1}{2}$		·7083	$8\frac{1}{2}$	$\frac{7}{8}$	·8750	$10\frac{1}{2}$
$\frac{35}{64}$	·54687	$6\frac{9}{16}$		·7135	$8\frac{9}{16}$		·8802	$10\frac{9}{16}$
	·5521	$6\frac{5}{8}$	$\frac{23}{32}$	·71875	$8\frac{5}{8}$		·8854	$10\frac{5}{8}$
	·5573	$6\frac{11}{16}$		·7240	$8\frac{11}{16}$	$\frac{57}{64}$	·89062	$10\frac{11}{16}$
$\frac{9}{16}$	·5625	$6\frac{3}{4}$		·7292	$8\frac{3}{4}$		·8958	$10\frac{3}{4}$
	·5677	$6\frac{13}{16}$	$\frac{47}{64}$	·73437	$8\frac{13}{16}$		·9010	$10\frac{13}{16}$
	·5729	$6\frac{7}{8}$		·7396	$8\frac{7}{8}$	$\frac{29}{32}$	·90625	$10\frac{7}{8}$
$\frac{37}{64}$	·57812	$6\frac{15}{16}$		·7448	$8\frac{15}{16}$		·9115	$10\frac{15}{16}$
	·5833	7	$\frac{3}{4}$	·7500	9		·9167	11
	·5885	$7\frac{1}{16}$		·7552	$9\frac{1}{16}$	$\frac{59}{64}$	·92187	$11\frac{1}{16}$
$\frac{19}{32}$	·59375	$7\frac{1}{8}$		·7604	$9\frac{1}{8}$		·9271	$11\frac{1}{8}$
	·5990	$7\frac{3}{16}$	$\frac{49}{64}$	·76562	$9\frac{3}{16}$		·9323	$11\frac{3}{16}$
	·6042	$7\frac{1}{4}$		·7708	$9\frac{1}{4}$	$\frac{15}{16}$	·9375	$11\frac{1}{4}$
$\frac{39}{64}$	·60937	$7\frac{5}{16}$		·7760	$9\frac{5}{16}$		·9427	$11\frac{5}{16}$
	·6146	$7\frac{3}{8}$	$\frac{25}{32}$	·78125	$9\frac{3}{8}$		·9479	$11\frac{3}{8}$
	·6198	$7\frac{7}{16}$		·7865	$9\frac{7}{16}$	$\frac{61}{64}$	·95312	$11\frac{7}{16}$
$\frac{5}{8}$	·6250	$7\frac{1}{2}$		·7917	$9\frac{1}{2}$		·9583	$11\frac{1}{2}$
	·6302	$7\frac{9}{16}$	$\frac{51}{64}$	·79687	$9\frac{9}{16}$		·9635	$11\frac{9}{16}$
	·6354	$7\frac{5}{8}$		·8021	$9\frac{5}{8}$	$\frac{31}{32}$	·96875	$11\frac{5}{8}$
$\frac{41}{64}$	·64062	$7\frac{11}{16}$		·8073	$9\frac{11}{16}$		·9740	$11\frac{11}{16}$
	·6458	$7\frac{3}{4}$	$\frac{13}{16}$	·8125	$9\frac{3}{4}$		·9792	$11\frac{3}{4}$
	·6510	$7\frac{13}{16}$		·8177	$9\frac{13}{16}$	$\frac{63}{64}$	·98437	$11\frac{13}{16}$
$\frac{21}{32}$	·65625	$7\frac{7}{8}$		·8229	$9\frac{7}{8}$		·9896	$11\frac{7}{8}$
	·6615	$7\frac{15}{16}$	$\frac{53}{64}$	·82812	$9\frac{15}{16}$		·9948	$11\frac{15}{16}$
	·6667	8		·8333	10	1	1·0000	12

CONVERSION TABLES
Equivalents of Feet in Metres

Ft.	·0	·1	·2	·3	·4
0		·03048	·06096	·09144	·12192
1	·30480	·33528	·36576	·39624	·42672
2	·60960	·64008	·67056	·70104	·73152
3	·91440	·94488	·97536	1·00584	1·03632
4	1·21920	1·24968	1·28016	1·31064	1·34112
5	1·52400	1·55448	1·58496	1·61544	1·64592
6	1·82880	1·85928	1·88976	1·92024	1·95072
7	2·13360	2·16408	2·19456	2·22504	2·25552
8	2·43840	2·46888	2·49936	2·52984	2·56033
9	2·74321	2·77369	2·80417	2·83465	2·86513
10	3·04801	3·07849	3·10897	3·13945	3·16993

Ft.	·5	·6	·7	·8	·9
0	·15240	·18288	·21336	·24384	·27432
1	·45720	·48768	·51816	·54864	·57912
2	·76200	·79248	·82296	·85344	·88392
3	1·06680	1·09728	1·12776	1·15824	1·18872
4	1·37160	1·40208	1·43256	1·46304	1·49352
5	1·67640	1·70688	1·73736	1·76784	1·79832
6	1·98120	2·01168	2·04216	2·07264	2·10312
7	2·28600	2·31648	2·34696	2·37744	2·40792
8	2·59081	2·62129	2·65177	2·68225	2·71273
9	2·89561	2·92609	2·95657	2·98705	3·01753
10	3·20041	3·23089	3·26137	3·29185	3·32233

British Units		Metrical Equivalents
1 inch	=	25·40 millimetres
1 inch	=	2·540 centimetres
1 inch	=	0.0254 metre
1 foot	=	0·3048 metre
1 yard	=	0·9144 metre
1 mile	=	1609·344 metres
1 mile	=	1·6093 kilometres

CONVERSION TABLES
Equivalents of Metres in Feet

Mtrs.	·0	·1	·2	·3	·4
0		0·3281	0·6562	0·9843	1·3123
1	3·2808	3·6089	3·9370	4·2651	4·5932
2	6·5617	6·8898	7·2178	7·5459	7·8740
3	9·8425	10·1706	10·4987	10·8268	11·1548
4	13·1233	13·4514	13·7795	14·1076	14·4357
5	16·4042	16·7323	17·0603	17·3884	17·7165
6	19·6850	20·0131	20·3412	20·6693	20·9973
7	22·9658	23·2939	23·6220	23·9501	24·2782
8	26·2467	26·5748	26·9028	27·2309	27·5590
9	29·5275	29·8556	30·1837	30·5118	30·8398
10	32·8083	33·1364	33·4645	33·7925	34·1206

Mtrs.	·5	·6	·7	·8	·9
0	1·6404	1·9685	2·2966	2·6247	2·9528
1	4·9213	5·2493	5·5774	5·9055	6·2336
2	8·2021	8·5302	8·8583	9·1863	9·5144
3	11·4829	11·8110	12·1391	12·4672	12·7953
4	14·7638	15·0918	15·4199	15·7480	16·0761
5	18·0446	18·3727	18·7008	19·0288	19·3569
6	21·3254	21·6535	21·9816	22·3097	22·6378
7	24·6063	24·9343	25·2624	25·5905	25·9186
8	27·8871	28·2152	28·5433	28·8713	29·1994
9	31·1679	31·4960	31·8241	32·1522	32·4803
10	34·4487	34·7768	35·1049	35·4330	35·7612

Metrical Units		British Equivalents
1 millimetre	–	0·03937 inch
1 centimetre	–	0·3937 ,,
1 metre	–	39·37008 inches
1 ,,	–	3·28084 feet
1 ,,	–	1·09361 yards
1 kilometre	–	3280·84 feet
1 ,,	–	1093·61 yards
1 ,,	–	0·621369 mile

CONVERSION TABLES
Equivalents of Pounds in Kilogrammes

Lbs.	·0	·1	·2	·3	·4
0		·04536	·09072	·13608	·18144
1	·45359	·49895	·54431	·58967	·63503
2	·90719	·95254	·99790	1·04326	1·08862
3	1·36078	1·40614	1·45150	1·49686	1·54222
4	1·81437	1·85973	1·90509	1·95045	1·99581
5	2·26796	2·31332	2·35868	2·40404	2·44940
6	2·72156	2·76692	2·81227	2·85763	2·90299
7	3·17515	3·22051	3·26587	3·31123	3·35659
8	3·62874	3·67410	3·71946	3·76482	3·81018
9	4·08233	4·12769	4·17305	4·21841	4·26377
10	4·53593	4·58129	4·62664	4·67200	4·71736

Lbs.	·5	·6	·7	·8	·9
0	·22680	·27216	·31751	·36287	·40823
1	·68039	·72575	·77111	·81647	·86183
2	1·13398	1·17934	1·22470	1·27006	1·31542
3	1·58757	1·63293	1·67829	1·72365	1·76901
4	2·04117	2·08653	2·13189	2·17724	2·22260
5	2·49476	2·54012	2·58548	2·63084	2·67620
6	2·94835	2·99371	3·03907	3·08443	3·12979
7	3·40194	3·44730	3·49266	3·53802	3·58338
8	3·85554	3·90090	3·94626	3·99162	4·03697
9	4·30913	4·35449	4·39985	4·44521	4·49057
10	4·76272	4·80808	4·8534	4·89880	4·94416

British Units	Metrical Equivalents
1 grain	= 64·79895 milligrammes
1 ,,	= 6·479895 centigrammes
1 ,,	= 0·06479895 gramme
1 ounce	= 28·34954 grammes
1 ,	= 0·02834954 kilogramme
1 pound	= 0·45359265 kilogramme
1 cwt.	= 0·50802377 quintal or 50·80 kilos
1 ton	= 1·01604754 tonnes or 1016 kilos

CONVERSION TABLES
Equivalents of Kilogrammes in Pounds

Kgs.	·0	·1	·2	·3	·4
0		·2205	·4409	·6614	·8818
1	2·2046	2·4251	2·6455	2·8660	3·0865
2	4·4092	4·6297	4·8502	5·0706	5·2911
3	6·6139	6·8343	7·0548	7·2752	7·4957
4	8·8185	9·0389	9·2594	9·4799	9·7003
5	11·0231	11·2436	11·4640	11·6845	11·9050
6	13·2277	13·4482	13·6687	13·8891	14·1096
7	15·4323	15·6528	15·8733	16·0937	16·3142
8	17·6370	17·8574	18·0779	18·2984	18·5188
9	19·8416	20·0621	20·2825	20·5030	20·7234
10	22·0462	22·2667	22·4871	22·7076	22·9281

Kgs.	·5	·6	·7	·8	·9
0	1·1023	1·3228	1·5432	1·7637	1·9842
1	3·3069	3·5274	3·7479	3·9683	4·1888
2	5·5116	5·7320	5·9525	6·1729	6·3934
3	7·7162	7·9366	8·1571	8·3776	8·5980
4	9·9208	10·1413	10·3617	10·5822	10·8026
5	12·1254	12·3459	12·5663	12·7868	13·0073
6	14·3300	14·5505	14·7710	14·9914	15·2119
7	16·5347	16·7551	16·9756	17·1960	17·4165
8	18·7393	18·9597	19·1802	19·4007	19·6211
9	20·9439	21·1644	21·3848	21·6053	21·8258
10	23·1485	23·3690	23·5894	23·8099	24·0304

Metrical Units	British Equivalents
1 milligramme	= 0·01543235 grain
1 centigramme	= 0·1543235 grain
1 gramme	= 15·43235 grains
1 gramme	= 0·0352739 ounce
1 kilogramme	= 35·27394 ounces
1 kilogramme	= 2·20462125 pounds
1 quintal	= 1·96841 cwts.
1 tonne	= 0·98420591 ton or 2204·6 lbs.

TABLE SHEWING LBS.
expressed in Decimals of a Ton

Lbs.	Ton	Lbs.	Ton	Lbs.	Ton
7	·003125	133	·059375	259	·115625
14	·00625	140	·0625	266	·11875
21	·009375	147	·065625	273	·121875
28	·0125	154	·06875	280	·125
35	·015625	161	·071875	287	·128125
42	·01875	168	·075	294	·13125
49	·021875	175	·078125	301	·134375
56	·025	182	·08125	308	·1375
63	·028125	189	·084375	315	·140625
70	·03125	196	·0875	322	·14375
77	·034375	203	·090625	329	·146875
84	·0375	210	·09375	336	·15
91	·040625	217	·096875	343	·153125
98	·04375	224	·1	350	·15625
105	·046875	231	·103125	357	·159375
112	·05	238	·10625	364	·1625
119	·053125	245	·109375	371	·165625
126	·05625	252	·1125	378	·16875

TABLE SHEWING LBS.
expressed in Decimals of a Ton

Lbs.	Ton	Lbs.	Ton	Lbs.	Ton
385	·171875	511	·228125	637	·284375
392	·175	518	·23125	644	·2875
399	·178125	525	·234375	651	·290625
406	·18125	532	·2375	658	·29375
413	·184375	539	·240625	665	·296875
420	·1875	546	·24375	672	·3
427	·190625	553	·246875	679	·303125
434	·19375	560	·25	686	·30625
441	·196875	567	·253125	693	·309375
448	·2	574	·25625	700	·3125
455	·203125	581	·259375	707	·315625
462	·20625	588	·2625	714	·31875
469	·209375	595	·265625	721	·321875
476	·2125	602	·26875	728	·325
483	·215625	609	·271875	735	·328125
490	·21875	616	·275	742	·33125
497	·221875	623	·278125	749	·334375
504	·225	630	·28125	756	·3375

TABLE SHEWING LBS.
expressed in Decimals of a Ton.

Lbs.	Ton	Lbs.	Ton	Lbs.	Ton
763	·340625	889	·396875	1015	·453125
770	·34375	896	·4	1022	·45625
777	·346875	903	·403125	1029	·459375
784	·35	910	·40625	1036	·4625
791	·353125	917	·409375	1043	·465625
798	·35625	924	·4125	1050	·46875
805	·359375	931	·415625	1057	·471875
812	·3625	938	·41875	1064	·475
819	·365625	945	·421875	1071	·478125
826	·36875	952	·425	1078	·48125
833	·371875	959	·428125	1085	·484375
840	·375	966	·43125	1092	·4875
847	·378125	973	·434375	1099	·490625
854	·38125	980	·4375	1106	·49375
861	·384375	987	·440625	1113	·496875
868	·3875	994	·44375	1120	·5
875	·390625	1001	·446875	1127	·503125
882	·39375	1008	·45	1134	·50625

TABLE SHEWING LBS.
expressed in Decimals of a Ton

Lbs.	Ton	Lbs.	Ton	Lbs.	Ton
1141	·509375	1267	·565625	1393	·621875
1148	·5125	1274	·56875	1400	·625
1155	·515625	1281	·571875	1407	·628125
1162	·51875	1288	·575	1414	·63125
1169	·521875	1295	·578125	1421	·634375
1176	·525	1302	·58125	1428	·6375
1183	·528125	1309	·584375	1435	·640625
1190	·53125	1316	·5875	1442	·64375
1197	·534375	1323	·590625	1449	·646875
1204	·5375	1330	·59375	1456	·65
1211	·540625	1337	·596875	1463	·653125
1218	·54375	1344	·6	1470	·65625
1225	·546875	1351	·603125	1477	·659375
1232	·55	1358	·60625	1484	·6625
1239	·553125	1365	·609375	1491	·665625
1246	·55625	1372	·6125	1498	·66875
1253	·559375	1379	·615625	1505	·671875
1260	·5625	1386	·61875	1512	·675

TABLE SHEWING LBS.
expressed in Decimals of a Ton

Lbs.	Ton	Lbs.	Ton	Lbs.	Ton
1519	·678125	1645	·734375	1771	·790625
1526	·68125	1652	·7375	1778	·79375
1533	·684375	1659	·740625	1785	·796875
1540	·6875	1666	·74375	1792	·8
1547	·690625	1673	·746875	1799	·803125
1554	·69375	1680	·75	1806	·80625
1561	·696875	1687	·753125	1813	·809375
1568	·7	1694	·75625	1820	·8125
1575	·703125	1701	·759375	1827	·815625
1582	·70625	1708	·7625	1834	·81875
1589	·709375	1715	·765625	1841	·821875
1596	·7125	1722	·76875	1848	·825
1603	·715625	1729	·771875	1855	·828125
1610	·71875	1736	·775	1862	·83125
1617	·721875	1743	·778125	1869	·834375
1624	·725	1750	·78125	1876	·8375
1631	·728125	1757	·784375	1883	·840625
1638	·73125	1764	·7875	1890	·84375

TABLE SHEWING LBS.
expressed in Decimals of a Ton

Lbs.	Ton	Lbs.	Ton	Lbs.	Ton
1897	·846875	2023	·903125	2149	·959375
1904	·85	2030	·90625	2156	·9625
1911	·853125	2037	·909375	2163	·965625
1918	·85625	2044	·9125	2170	·96875
1925	·859375	2051	·915625	2177	·971875
1932	·8625	2058	·91875	2184	·975
1939	·865625	2065	·921875	2191	·978125
1946	·86875	2072	·925	2198	·98125
1953	·871875	2079	·928125	2205	·984375
1960	·875	2086	·93125	2212	·9875
1967	·878125	2093	·934375	2219	·990625
1974	·88125	2100	·9375	2226	·99375
1981	·884375	2107	·940625	2233	·996875
1988	·8875	2114	·94375	2240	1·
1995	·890625	2121	·946875		
2002	·89375	2128	·95		
2009	·896875	2135	·953125		
2016	·9	2142	·95625		

CONVERSION TABLE

Tons into Pounds

Tons	Lbs.	Tons	Lbs.	Tons	Lbs.	Tons	Lbs.
1	2,240	26	58,240	51	114,240	76	170,240
2	4,480	27	60,480	52	116,480	77	172,480
3	6,720	28	62,720	53	118,720	78	174,720
4	8,960	29	64,960	54	120,960	79	176,960
5	11,200	30	67,200	55	123,200	80	179,200
6	13,440	31	69,440	56	125,440	81	181,440
7	15,680	32	71,680	57	127,680	82	183,680
8	17,920	33	73,920	58	129,920	83	185,920
9	20,160	34	76,160	59	132,160	84	188,160
10	22,400	35	78,400	60	134,400	85	190,400
11	24,640	36	80,640	61	136,640	86	192,640
12	26,880	37	82,880	62	138,880	87	194,880
13	29,120	38	85,120	63	141,120	88	197,120
14	31,360	39	87,360	64	143,360	89	199,360
15	33,600	40	89,600	65	145,600	90	201,600
16	35,840	41	91,840	66	147,840	91	203,840
17	38,080	42	94,080	67	150,080	92	206,080
18	40,320	43	96,320	68	152,320	93	208,320
19	42,560	44	98,560	69	154,560	94	210,560
20	44,800	45	100,800	70	156,800	95	212,800
21	47,040	46	103,040	71	159,040	96	215,040
22	49,280	47	105,280	72	161,280	97	217,280
23	51,520	48	107,520	73	163,520	98	219,520
24	53,760	49	109,760	74	165,760	99	221,760
25	56,000	50	112,000	75	168,000	100	224,000

RAFTER LENGTHS for various Purlin Centres : Pitch 1 in $2\frac{1}{2}$

Span of Truss in feet	Length of Rafter in feet
15′ 0″	8·078
20′ 0″	10·770
25′ 0″	13·463
30′ 0″	16·155
35′ 0″	18·848
40′ 0″	21·540
45′ 0″	24·232
50′ 0″	26·925
55′ 0″	29·617
60′ 0″	32·310
65′ 0″	35·002
70′ 0″	37·695
75′ 0″	40·388
80′ 0″	43·080

Purlins each slope	Centres of Purlins										
	3′ 0″	3′ 6″	4′ 0″	4′ 6″	5′ 0″	5′ 6″	6′ 0″	6′ 6″	7′ 0″	7′ 6″	8′ 0″
3	6′ 0″	7′ 0″	8′ 0″	9′ 0″	10′ 0″	11′ 0″	12′ 0″	13′ 0″	14′ 0″	15′ 0″	16′ 0″
4	9′ 0″	10′ 6″	12′ 0″	13′ 6″	15′ 0″	16′ 6″	18′ 0″	19′ 6″	21′ 0″	22′ 6″	24′ 0″
5	12′ 0″	14′ 0″	16′ 0″	18′ 0″	20′ 0″	22′ 0″	24′ 0″	26′ 0″	28′ 0″	30′ 0″	32′ 0″
6	15′ 0″	17′ 6″	20′ 0″	22′ 6″	25′ 0″	27′ 6″	30′ 0″	32′ 6″	35′ 0″	37′ 6″	40′ 0″
7	18′ 0″	21′ 0″	24′ 0″	27′ 0″	30′ 0″	33′ 0″	36′ 0″	39′ 0″	42′ 0″	45′ 0″	
8	21′ 0″	24′ 6″	28′ 0″	31′ 6″	35′ 0″	38′ 6″	42′ 0″	45′ 6″			
9	24′ 0″	28′ 0″	32′ 0″	36′ 0″	40′ 0″	44′ 0″					
10	27′ 0″	31′ 6″	36′ 0″	40′ 6″	45′ 0″						
11	30′ 0″	35′ 0″	40′ 0″	45′ 0″							
12	33′ 0″	38′ 6″	44′ 0″								

E.g.—40′ 0″ Span Truss, Asbestos "Big Six" Sheets. Find No. of Purlins. Rafter Length = 21·54 feet. Centres of Purlins = 4′ 6″ (page 425). Nearest equivalent rafter length from Table under 4′ 6″ col. = 22′ 6″, which indicates **6 purlins per slope or 12 per Truss.**

Extracts from B.S.S. No. 449, 1937
Floors and Wind

Type of Floor	lbs. per sq. ft. of floor area, excluding allowance for partitions. See Note*	
	Beams, Pillars, Piers, Walls & Foundations	Floor Slabs
Rooms used for domestic purposes, hotel bedrooms, hospital rooms and wards - -	40	50
Offices, floors above entrance floor - - - - -	50	80
Offices, entrance floor and floors below entrance floor -	80	80
Churches, schools, reading rooms, art galleries and similar uses - - - -	70	80
Retail-shops and garages for cars of not more than 2 tons deadweight - - - -	80	80
Assembly halls, drill halls, dance halls, gymnasia, light workshops, public spaces in hotels and hospitals, staircases and landings, theatres, cinemas, restaurants and grandstands - - - - -	100	100
Warehouses, book and stationery stores and similar premises, together with garages for motor vehicles exceeding 2 tons deadweight - -	Actual load to be calculated but not less than 200	

* For all floors in which partitions are intended but not located on the plans, a uniformly distributed load of not less than 20 lbs. per sq. ft. of floor area shall be provided for as an allowance to cover partitions.

showing superimposed Loads for Loads for Roofs

Roofs	lbs. per sq. ft. of covered area	
	Beams, Pillars, Piers, Walls & Foundations	Roof Slabs
Flat roofs and roofs inclined at an angle with the horizontal of not more than 20°	30	50

On roofs inclined at an angle with the horizontal of more than 20° a minimum superimposed load (deemed to include the wind load) of 15 lbs. per sq. ft. of surface shall be assumed acting normal to the surface, inwards on the windward side, and 10 lbs. per sq. ft. of surface acting outwards on the leeward side, provided that this requirement shall apply only in the design of the roof structure. The stresses resulting from wind pressure shall be considered separately with or without suction.

The design shall allow for a wind pressure in any horizontal direction of not less than 15 lb. per sq.ft. of the upper two thirds of the vertical projection of the surface of such buildings, with an additional pressure of 10 lb. per sq. ft. upon all projections above the general roof level or ridge. On the sea coast and in similarly exposed situations a further provision shall be made.

If the vertical projection of a building is less than twice its width, wind pressure may be neglected, provided that the building is adequately stiffened by floors and walls.

Extract from Codes of Practice
Loads for Floors

	Type of Floor	Load. in lbs. per sq. ft.	Min. Load* lbs. per ft. width on slabs or floorboards
I	Private dwellings of not more than two storeys -	30	240
II	Rooms in private dwellings of more than two storeys, including flats, hospital rooms and wards, bedrooms and private sitting-rooms in hotels and tenement houses and similar occupancies - - -	40	320
III	Rooms used as offices -	50	400
IV	Classrooms in schools and colleges, minimum for light workshops - - -	60	480
V	Banking halls and offices where the public may congregate - - - -	70	560
VI	Retail shops: places of assembly with fixed seating, i.e. churches and chapels: restaurants: garages for vehicles not exceeding 2½ tons gross weight (private cars, light vans, etc.), circulation space in machinery halls, power stations, pumping stations, etc., where not occupied by plant or equipment - - - -	80	640

Fixed Seating implies that their removal and the use of space for other purposes are improbable.

Committee showing superimposed uniformly distributed

	Type of Floor	Load in lbs. per sq. ft.	Min. Load* lbs. per ft. width on slabs or floorboards
VII	Places of assembly without fixed seating (public rooms in hotels, dance halls, etc.), minimum for filing or record rooms in offices: light workshops generally, including light machinery -	100	800
VIII	Garages to take all types of vehicles	100	†
	†Worst combination of actual wheel loads, or if the slab is capable of effective lateral distribution of load, 1·5 × maximum wheel load, but not less than 2,000 lbs. considered to be distributed over a floor area 2′ 6″ square.		
IX	Light storage space in commercial and industrial buildings, medium workshops -	150	—
X	Minimum for warehouse and general storage space in commercial and industrial buildings, heavy workshops. The loads imposed by heavy plant and machinery should be determined and allowed for	200	—

*Minimum load for slabs becomes operative at spans of less than 8 ft. Minimum load for beams becomes operative on areas less than 64 sq. ft.

RIDGE ROOF

ROOF TRUSSES, spaced 15′ 0″ apart, carry Corrugated Steel, Asbestos, B.S.S.

Rafter Sections in Bold Type in Brackets have from Purlins. Alternative weights including

Note.—Heavy lines indicate Struts, Light Lines, Ties

Weights given include Bolts, Gussets and Purlin Cleats

Up to 15′ 0″ span

(Weight 2 cwts.)
Weight 1¾ cwts.

(Rafter	**3 × 2½ × ¼L)**
Rafter	2½ × 2½ × ¼L
Main Tie	2 × 2 × ¼L
King Tie	2 × 2 × ¼L

Up to 20′ 0″ span

(Weight 2¾ cwts.)
Weight 2½ cwts.

(Rafter	**3 × 2½ × ¼L)**
Rafter	2½ × 2 × ¼L
Main Tie	2 × 2 × ¼L
Struts	2 × 2 × ¼L
King Tie	2 × 2 × ¼L

Up to 25′ 0″ span

(Weight 3½ cwts.)
Weight 3¼ cwts.

(Rafter	**3½ × 2½ × ¼L)**
Rafter	2½ × 2½ × ¼L
Main Tie	2 × 2 × ¼L
Struts	2 × 2 × ¼L
Queen Tie	2 × 2 × ¼L

TRUSSES : Scantlings and Weights

having rise of 1/5th span, designed to or A.P.M. Sheeting on Steel Purlins. Loading.

been designed for Flexural Stresses due to B.M. these Sections are also given in Bold Type.

Up to 30′ 0″ span

(Rafter	**$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$L)**
Rafter	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$L
Main Tie	$2 \times 2 \times \frac{1}{4}$L
Struts	$2 \times 2 \times \frac{1}{4}$L
Queen Tie	$2 \times 2 \times \frac{1}{4}$L

(Weight $4\frac{3}{4}$ cwts.)
Weight $4\frac{1}{2}$ cwts.

Up to 35′ 0″ span

(Rafter	**$3\frac{1}{2} \times 3 \times \frac{5}{16}$L)**
Rafter	$3 \times 3 \times \frac{1}{4}$L
Main Tie	$2 \times 2 \times \frac{1}{4}$L
Struts	$2 \times 2 \times \frac{1}{4}$L
Queen Tie	$2 \times 2 \times \frac{1}{4}$L

For trusses of 40′ 0″ span in this design weight $-6\frac{3}{4}$ cwts. using these scantlings (heavy rafter).

(Weight $5\frac{3}{4}$ cwts.)
Weight $5\frac{1}{4}$ cwts.

Up to 40′ 0″ span

(Rafter	**$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$L)**
Rafter	$3 \times 3 \times \frac{1}{4}$L
Side Main Tie	$2\frac{1}{2} \times 2 \times \frac{1}{4}$L
Centre „ „	$2 \times 2 \times \frac{1}{4}$L
Struts	$2 \times 2 \times \frac{1}{4}$L
Ties	$2 \times 2 \times \frac{1}{4}$L

(Weight 8 cwts.)
Weight $7\frac{1}{2}$ cwts.

RIDGE ROOF TRUSSES:

Note.—Heavy lines indicate Struts, Light Lines, Ties

Weights given include Bolts, Gussets and Purlin Cleats

Up to 45′ 0″ span

(Weight 9 cwts.)
Weight $8\frac{1}{2}$ cwts.

(Rafter	**$3\frac{1}{2} \times 3 \times \frac{5}{16}$L)**
Rafter	$3 \times 3 \times \frac{1}{4}$L
Side Main Tie	$3 \times 2 \times \frac{1}{4}$L
Centre ,, ,,	$2 \times 2 \times \frac{1}{4}$L
Struts	$2 \times 2 \times \frac{1}{4}$L
Ties	$2 \times 2 \times \frac{1}{4}$L

Up to 50′ 0″ span

(Weight $10\frac{1}{4}$ cwts.)
Weight $9\frac{1}{2}$ cwts.

(Rafter	**$4 \times 3 \times \frac{5}{16}$L)**
Rafter	$3\frac{1}{2} \times 3 \times \frac{1}{4}$L
Side Main Tie	$3 \times 2 \times \frac{1}{4}$L
Centre ,, ,,	$2 \times 2 \times \frac{1}{4}$L
Main Strut	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$L
Short Struts	$2 \times 2 \times \frac{1}{4}$L
Queen Tie	$2\frac{1}{2} \times 2 \times \frac{1}{4}$L
Ties	$2 \times 2 \times \frac{1}{4}$L

Up to 55′0″ span

(Weight $12\frac{1}{2}$ cwts.)
Weight $11\frac{1}{2}$ cwts.

(Rafter	**$4 \times 4 \times \frac{5}{16}$L)**
Rafter	$3 \times 3 \times \frac{5}{16}$L
Side Main Tie	$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$L
Centre ,, ,,	$2\frac{1}{2} \times 2 \times \frac{1}{4}$L
Main Struts	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$L
Short Struts	$2 \times 2 \times \frac{1}{4}$L
Queen Tie	$2\frac{1}{2} \times 2 \times \frac{1}{4}$L
Ties	$2 \times 2 \times \frac{1}{4}$L

Up to 60′ 0″ span

(Weight 14 cwts.)
Weight 13 cwts.

(Rafter	**$2/3 \times 2 \times \frac{1}{4}$Ls)**
Rafter	$3 \times 3 \times \frac{5}{16}$L
Side Main Tie	$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$L
Centre ,, ,,	$3 \times 2 \times \frac{1}{4}$L
Main Struts	$3 \times 3 \times \frac{1}{4}$L
Short Struts	$2 \times 2 \times \frac{1}{4}$L
Queen Tie	$2\frac{1}{2} \times 2 \times \frac{1}{4}$L
Ties	$2 \times 2 \times \frac{1}{4}$L

Scantlings and Weights *(contd.)*

Up to 65′ 0″ span

(Rafter	**2/3 × 2 × ¼Ls)**
Rafter	3½ × 3 × 5/16L
Side Main Tie	3½ × 2½ × 5/16L
Centre ,, ,,	3½ × 2½ × ¼L
Main Strut	3 × 3 × ¼L
Short Struts	2 × 2 × ¼L
Queen Tie	2½ × 2 × ¼L
Ties	2 × 2 × ¼L

(Weight 16¼ cwts.)
Weight 15¼ cwts.

Note.—Scantlings and arrangement for 70′ 0″ span similar to above, but weight increased 1¼ cwts.

Up to 75′ 0″ span

(Rafter	**2/3½ × 2½ × ¼Ls)**
Rafter	2/2½ × 2 × ¼Ls
Side Main Tie	2/2½ × 2 × ¼Ls
Centre ,, ,,	2/2 × 2 × ¼Ls
Main Strut	3 × 3 × ¼L
Short Struts	2½ × 2 × ¼L
Queen Tie	2½ × 2 × ¼L
Ties	2 × 2 × ¼L

(Weight 21 cwts.)
Weight 19 cwts.

Up to 80′ 0″ span

(Rafter	**2/3 × 3 × ¼Ls)**
Rafter	2/3 × 2 × ¼Ls
Side Main Tie	2/2½ × 2 × ¼Ls
Centre ,, ,,	2/2 × 2 × ¼Ls
Main Struts	3 × 3 × ¼L
Short Struts	2½ × 2 × ¼L
Queen Tie	3 × 2½ × ¼L
Ties	2 × 2 × ¼L

(Weight 23¾ cwts.)
Weight 22¼ cwts.

Note.—Above 80′ 0″ span to be specially designed.

NORTH LIGHT ROOF

TRUSSES spaced 15′ 0″ apart, having Sheeted Slope at 21° 48′ and Glazed Slope at 55° designed to carry Corrugated Sheets (Asbestos, A.P.M. or Steel) and ¼″ thick glass with Tee Astragals. B.S.S. Loading.

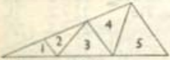
Weights given include Bolts, Gussets and Purlin Cleats

*12′ 7 7/16″ 5′ 8 11/16″

Up to 15′ 0″ span
(Weight 2 cwts.)
Weight 1¾ cwts.

(Left Rafter	**3 × 2½ × ¼L)**
Left Rafter	2 × 2 × ¼L
Right Rafter	2 × 2 × ¼L
Main Tie	2 × 2 × ¼L
Bar 1–2	2 × 2 × ¼L
,, 2–3	2½ × 2 × ¼L

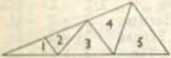
*16′ 9 13/16″ 7′ 7 9/16″



Up to 20′ 0″ span
(Weight 3 cwts.)
Weight 2¾ cwts.

(Left Rafter	**3 × 3 × ¼L)**
Left Rafter	2 × 2 × ¼L
Right Rafter	2½ × 2½ × ¼L
Main Tie	2 × 2 × ¼L
Bar 1–2	2 × 2 × ¼L
,, 2–3	2 × 2 × ¼L
,, 3–4	2 × 2 × ¼L
,, 4–5	2 × 2 × ¼L

*21′ 0 7/16″ 9′ 6 7/16″



Up to 25′ 0″ span

(Weight 3¾ cwts.)
Weight 3½ cwts.

(Left Rafter	**3½ × 3 × ¼L)**
Left Rafter	2½ × 2½ × ¼L
Right Rafter	3 × 3 × ¼L
Main Tie	2 × 2 × ¼L
Bar 1–2	2 × 2 × ¼L
,, 2–3	2¼ × 2¼ × ¼L
,, 3–4	2 × 2 × ¼L
,, 4–5	3 × 2½ × ¼L

*Dimensions given are rafter lengths.

TRUSSES : Scantlings and Weights

*25′ 2⅞″ 11′ 5$\frac{5}{16}$″

Up to 30′ 0″ span

(Weight 5¼ cwts.)
Weight 5 cwts.

(Left Rafter	**3½ × 3 × ¼L)**
Left Rafter	2½ × 2½ × ¼L
Right Rafter	2½ × 2½ × ¼L
Main Tie	2 × 2 × ¼L
Bar 1–2	2 × 2 × ¼L
,, 2–3	2 × 2 × ¼L
,, 3–4	2 × 2 × ¼L
,, 4–5	2½ × 2½ × ¼L
,, 5–6	3 × 2½ × ¼L
,, 6–7	2 × 2 × ¼L
,, 7–8	2 × 2 × ¼L

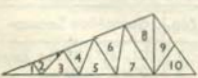
*29′ 5⅜″ 13′ 4$\frac{3}{16}$″

Up to 35′ 0″ Span

(Weight 7¾ cwts.)
Weight 7 cwts.

(Left Rafter	**4 × 3 × $\frac{5}{16}$L)**
Left Rafter	3 × 2½ × ¼L
Right Rafter	2½ × 2½ × ¼L
Main Tie	2½ × 2 × ¼L
Bar 1–2	2 × 2 × ¼L
,, 2–3	2½ × 2½ × ¼L
,, 3–4	2½ × 2½ × ¼L
,, 4–5	3 × 2½ × ¼L
,, 5–6	2/2½ × 2 × ¼LS
,, 6–7	2/2½ × 2 × ¼LS
,, 7–8	2½ × 2 × ¼L

*33′ 7⅞″ 15′ 3⅛″



Up to 40′ 0″ span

(Weight 9¼ cwts.)
Weight 8½ cwts.

(Left Rafter	**4 × 3 × $\frac{5}{16}$L)**
Left Rafter	3 × 3 × ¼L
Right Rafter	3 × 3 × ¼L
Main Tie	2½ × 2 × ¼L
Bar 1–2	2 × 2 × ¼L
,, 2–3	2½ × 2½ × ¼L
,, 3–4	2 × 2 × ¼L
,, 4–5	3 × 2½ × ¼L
,, 5–6	3 × 2½ × ¼L
,, 6–7	3½ × 3 × ¼L
,, 7–8	2/2½ × 2 × ¼LS
,, 8–9	2 × 2 × ¼L
,, 9–10	2½ × 2½ × ¼L

SIDE STANCHIONS—Sections of Stanchions for Single Span Buildings covered with Corrugated Asbestos or Corrugated Steel Sheeting

SIDE STANCHIONS (15'·0" Crs.)

Height in feet	Span of Building in feet										
	10	15	20	25	30	35	40	45	50	55	60
10	2	3	3	4	4	4	4	5	5	6	6
12	4	4	4	4	4	5	5	6	6	6	7
14	4	4	4	5	5	6	6	6	7	7	7
16	5	5	5	6	6	6	7	7	7	7	8
18	5	6	6	7	7	7	7	8	8	8	9
20	6	6	7	7	7	8	8	8	9	9	9
22	7	7	7	7	8	8	9	9	9	9	10
24	7	7	8	8	9	9	9	10	10	10	11
26	8	8	9	9	9	10	10	10	11	11	11
28	9	9	9	9	10	10	10	11	11	11	11
30	9	9	10	10	11	11	11	11	11	12	12

No.	Stanchion Section	No.	Stanchion Section
1	4"×3"×9·5 lbs. Joist	7	10"×4½"×25 lbs. Joist
2	5"×3"×11 „ „	8	10"×5" ×30 „ „
3	6"×3"×12 „ „	9	12"×5" ×32 „ „
4	7"×4"×16 „ „	10	13"×5" ×35 „ „
5	8"×4"×18 „ „	11	14"×5½"×40 „ „
6	9"×4"×21 „ „	12	14"×6" ×46 „ „

END STANCHIONS—Sections of Stanchions for Single Span Buildings covered with Corrugated Asbestos or Corrugated Steel Sheeting

END STANCHIONS

Height to Eaves in feet	Span of Building in feet								
	20	25	30	35	40	45	50	55	60
10	1	1	1	1	1	1	1	1	1
12	1	1	1	1	1	2	1	1	2
14	2	2	2	2	2	2	2	2	2
16	3	3	4	3	3	4	3	3	4
18	4	4	4	4	4	4	4	4	4
20	4	4	5	4	4	5	4	4	5
22	5	5	5	5	5	5	5	5	5
24	6	6	6	6	6	6	6	6	6
26	7	7	7	7	7	7	7	7	7
28	7	7	7	7	7	7	7	7	7
30	7	7	9	7	7	9	7	7	9

Span of Building	Number of End Stans. required
Up to 15′ 0″	None
15′ 0″ to 30′ 0″	1
30′ 0″ to 45′ 0″	2
45′ 0″ to 60′ 0″	3

Note.—Bracing at Main Tie Level should be provided for End Stanchions

PURLINS

Sections of Angle Purlins carrying Corrugated Steel, Asbestos or A.P.M. Sheeting

Section of Angle Purlins *f=8+33⅓% T/sq.in.	SPANS IN FEET	
	4′ 6″ Apart (Asbestos)	6′ 0″ Apart (Steel & A.P.M.)
2″ × 2″ × ¼″	8′ 0″	7′ 6″
2½″ × 2″ × ¼″	9′ 6″	9′ 0″
3″ × 2″ × ¼″	11′ 0″	10′ 9″
3″ × 2½″ × ¼″	11′ 3″	11′ 0″
3½″ × 2½″ × ¼″	13′ 0″	12′ 9″
3½″ × 3″ × ¼″	13′ 3″	13′ 0″
4″ × 2½″ × ¼″	14′ 9″	14′ 6″
4″ × 3″ × ¼″	15′ 0″	14′ 9″
4″ × 3½″ × ¼″	15′ 3″	15′ 0″

N.B.—Eaves Purlins increased to next section

Section of Angle Purlins *f=10+25% T/sq.in.	SPANS IN FEET	
	4′ 6″ Apart (Asbestos)	6′ 0″ Apart (Steel & A.P.M.)
2″ × 2″ × ¼″	8′ 0″	7′ 6″
2½″ × 2″ × ¼″	9′ 9″	9′ 6″
3″ × 2″ × ¼″	11′ 9″	11′ 3″
3″ × 2½″ × ¼″	12′ 0″	11′ 6″
3½″ × 2½″ × ¼″	14′ 6″	14′ 0″
3½″ × 3″ × ¼″	15′ 0″	14′ 6″
4″ × 2½″ × ¼″	15′ 9″	15′ 0″
4″ × 3″ × ¼″	16′ 0″	15′ 3″
4″ × 3½″ × ¼″	16′ 3″	15′ 6″

Purlin Sections are based on
BM = WL ÷ 10 for Corr. Asbestos Sheeting.
BM = WL ÷ 12 for Corr. Steel and A.P.M. Sheeting.

*f (stress) @ 8+33⅓% = BSS 449, 1937
f (stress) @ 10+25% = BSS 449—1939 (revision)

HORIZONTALS

Sections of Angle Side Rails carrying Corrugated Steel, Asbestos or A.P.M. Sheeting

Section of Angle Side Rails *f - 8+33⅓% T/sq.in.	SPANS IN FEET	
	6′ 0″ Apart (Asbestos)	8′ 0″ Apart (Steel & A.P.M.)
2″ × 2″ × ¼″	9′ 0″	8′ 6″
2½″ × 2″ × ¼″	10′ 0″	9′ 0″
2½″ × 2½″ × ¼″	11′ 6″	11′ 0″
3″ × 3″ × ¼″	13′ 6″	12′ 6″
3½″ × 3″ × ¼″	15′ 6″	14′ 6″
4″ × 3″ × ¼″	16′ 0″	15′ 0″

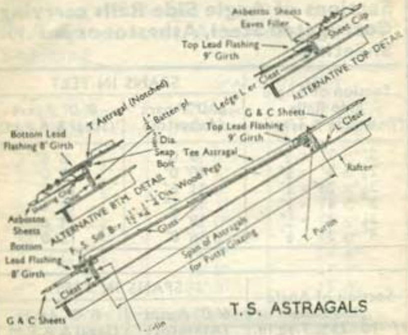
Section of Angle Side Rails *f - 10+25% T/sq.in.	SPANS IN FEET	
	6′ 0″ Apart (Asbestos)	8′ 0″ Apart (Steel & A.P.M.)
2″ × 2″ × ¼″	9′ 6″	9′ 3″
2½″ × 2″ × ¼″	11′ 0″	10′ 9″
2½″ × 2½″ × ¼″	12′ 0″	11′ 6″
3″ × 3″ × ¼″	14′ 0″	13′ 6″
3½″ × 3″ × ¼″	16′ 0″	16′ 0″
4″ × 3″ × ¼″	16′ 9″	16′ 6″

*f (stress) @ 8+33⅓% – BSS 449, 1937
f (stress) @ 10+25% – BSS 449—1939 (revision)

HANDRAIL TUBING

Nominal Bore in ins.	¾	1	1¼	1½	2
External Diam. in ins.	1 1/16	1 11/32	1 11/16	1 29/32	2⅜
Weight per ft. in lbs.	1·4	2·0	2·8	3·5	4·5

ROOF GLAZING



T.S. ASTRAGALS

SPANS OF ASTRAGALS (spaced at 2′ 0″ ctrs.)

$1\frac{1}{4} \times 1\frac{1}{4} \times \frac{1}{4}$ Tee 5′ 6″ Max.	$1\frac{3}{4} \times 1\frac{3}{4} \times \frac{1}{4}$ Tee 9′ 0″ Max.
$1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$ Tee 7′ 0″ Max.	$2 \times 2 \times \frac{1}{4}$ Tee 11′ 0″ Max.

LEAD FLASHING AT ENDS OF GLAZING

For G. & C. and R.P.M. Sheets - -	9″ Girth
For Standard Asbestos Sheets - -	10″ Girth
For Trafford Tiles and Big-Six - -	12″ Girth

SHEET GLASS—Thicknesses and Weights

21 oz. per sq. ft. - - - -	$\frac{1}{10}$ inch
26 oz. per sq. ft. - - - -	$\frac{1}{8}$ inch
32 oz. per sq. ft. - - - -	$\frac{5}{32}$ inch
44 oz. per sq. ft. - - - -	$\frac{3}{16}$ inch
56 oz. per sq. ft. - - - -	$\frac{1}{4}$ inch

GUTTER AND DOWNPIPE DATA

Suggested Sizes of Gutters and Downpipes

Roof Span	Half Round Gutter	D/Pipe Dia.	Roof Span	Half Round Gutter	D/Pipe Dia.
Up to 25′ 0″	5″	$2\frac{1}{2}$″ every 40′	50′-70′	8″	4″ every 40′
25′-50′	6″	$3\frac{1}{2}$″ every 40′	70′-100′	9″	5″ every 40′

This Table is based on taking 125 sq. ft. of Roof Slope for every 1 sq. inch of Downpipe and 1 sq. inch of Gutter to every 75 sq. ft. of Roof Slope. The cross-sectional areas of the Gutters should be approx. twice that of the Downpipes.

AREAS in sq. ins. and WEIGHTS in lbs. per foot run of Cast Iron Gutters and Downpipes.

H.R. Gutters			Downpipes					
Dia.	Area	Wt.	Dia.	Area	Wt.	Dia.	Area	Wt.
4	6·28	2·08	2	3·14	2·58	5	19·64	7·50
5	9·82	2·58	$2\frac{1}{2}$	4·91	3·17	6	28·29	9·33
6	14·14	4·00	3	7·07	3·83	7	38·49	15·67
7	19·24	6·66	$3\frac{1}{2}$	9·62	4·66	8	50·27	20·33
8	25·13	7·50	4	12·57	5·50	9	63·62	26·67
9	31·81	10·00	$4\frac{1}{2}$	15·91	6·25	10	78·55	30·00

Cast Iron weighs 450 lbs. per cubic foot: i.e. a C.I. plate $12 \times 12 \times \frac{1}{8}$ weighs 4·687 lbs. Gutters and Downpipes are supplied in Standard Lengths of 6′ 0″. Gutters should be supported every 3′ 0″ by F.S. Straps usually $1'' \times \frac{3}{16}''$. Downpipes are usually held in position by Holdfasts at 6′ 0″ crs. Gutters and Pipe Joints should be embedded in Red Lead Putty estimated at 3 Joints of 6″ Gutter to every 1 lb. and 1 joint of 3″ Downpipe to every 1 lb. of Putty.

STEEL FRAMED

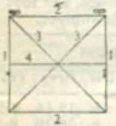
SCANTLINGS and WEIGHTS of hung on Coburn Track and covered Weights include Door Complete

7′ 0″ Square

Member 1 - $2\frac{1}{2} \times 2 \times \frac{1}{4}$L
,, 2 - $2\frac{1}{2} \times 2 \times \frac{1}{4}$L
,, 3 - $1\frac{1}{2} \times \frac{1}{4}$ Fl.

Steelwork Weight
= 1 cwt. 1 qur. 9 lbs.

Sheeting Weight 24 gauge.
= 2 qurs., 14 lbs.



8′ 0″ Square

Member 1 - $2\frac{1}{2} \times 2 \times \frac{1}{4}$L
,, 2 - $2\frac{1}{2} \times 2 \times \frac{1}{4}$L
,, 3 - $1\frac{1}{2} \times \frac{1}{4}$ Fl.
,, 4 - $3 \times 3 \times \frac{1}{4}$T

Steelwork Weight
= 1 cwt. 3 qurs. 20 lbs.

Sheeting Weight 24 gauge.
= 3 qurs. 6 lbs.

9′ 0″ Square

Member 1 - $2\frac{1}{2} \times 2 \times \frac{1}{4}$L
,, 2 - $2\frac{1}{2} \times 2 \times \frac{1}{4}$L
,, 3 - $1\frac{1}{2} \times \frac{1}{4}$ Fl.
,, 4 - $3 \times 3 \times \frac{1}{4}$T

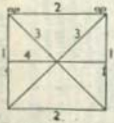
Steelwork Weight
= 2 cwts. 0 qurs. 15 lbs.

Sheeting Weight 24 gauge.
= 1 cwt. 0 qurs. 5 lbs.

For Sizes of Doors other than above
See TYPICAL

DOORS

SLIDING DOORS, single leaf type, with Corrugated Steel Sheeting. but are Exclusive of Track.

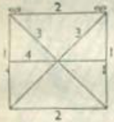


10′ 0″ Square

Member 1 - $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$L
" 2 - $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$L
" 3 - $2 \times \frac{1}{4}$ Fl.
" 4 - $3 \times 3 \times \frac{1}{4}$T

Steelwork Weight
–2 cwts. 2 qurs. 10 lbs.

Sheeting Weight 24 gauge.
– 1 cwt. 1 qur. 5 lbs.

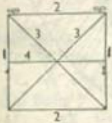


12′ 0″ Square

Member 1 - $3 \times 2\frac{1}{2} \times \frac{1}{4}$L
" 2 - $3 \times 2\frac{1}{2} \times \frac{1}{4}$L
" 3 - $2 \times \frac{1}{4}$ Fl.
" 4 - $3 \times 3 \times \frac{1}{4}$T

Steelwork Weight
–3 cwts. 2 qurs. 0 lbs.

Sheeting Weight 24 gauge.
– 1 cwt. 3 qurs. 14 lbs.



15′ 0″ Square

Member 1 - $3\frac{1}{2} \times 3 \times \frac{1}{4}$L
" 2 - $3\frac{1}{2} \times 3 \times \frac{1}{4}$L
" 3 - $2\frac{1}{2} \times \frac{1}{4}$Fl.
" 4 - $3 \times 3 \times \frac{5}{16}$T

Steelwork Weight
– 5 cwts. 0 qurs. 9 lbs.

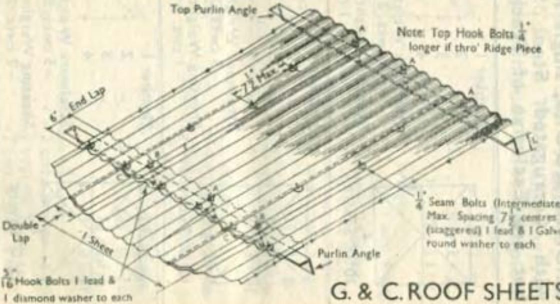
Sheeting Weight 24 gauge.
– 2 cwts. 3 qurs. 16 lbs.

Approx. Weights can be obtained pro rata
DETAILS on Page 464.

SHEET FIXING DATA

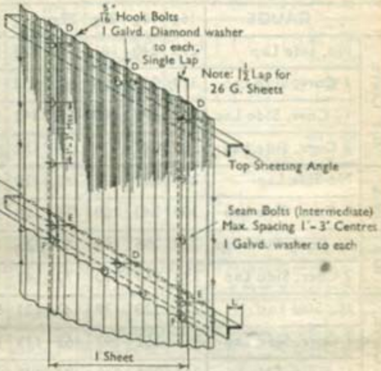
Hook Bolts : { A. $\frac{5}{16}''$ Dia., Lead and Diamond Washer, Length – L + $1\frac{1}{4}''$
{ B. $\frac{5}{16}''$ Dia., Lead and Diamond Washer, Length – L + $1\frac{1}{2}''$

Seam Bolts : { C. $\frac{1}{4}''$ Dia., Lead and Round Washer, Length – $1''$
{ (Inter.) $\frac{1}{4}''$ Dia., Lead and Round Washer, Length – $\frac{3}{4}''$



G. & C. ROOF SHEETS

SHEET FIXING DATA



G. & C. SIDE SHEETS

Hook Bolts :

D $\frac{5}{16}''$ Dia., Diamond Washer, Length – L + $1\frac{1}{4}''$

E. $\frac{5}{16}''$ Dia., Diamond Washer, Length – L + $1\frac{1}{2}''$

Seam Bolts :

F. $\frac{1}{4}''$ Dia., Round Washer, Length – 1"

(Inter.) $\frac{1}{4}''$ Dia., Round Washer, Length – $\frac{3}{4}''$

Note.:

Length of Hook Bolts $\frac{1}{4}''$ less for 26 Gauge Sheets

WEIGHTS PER SQUARE (100 sq. ft.) OF GALVANISED CORRUGATED STEEL SHEETS in lbs. (Ordinary Commercial Quality)

	GAUGE	16g	18g	20g	22g	24g	26g
8/3" Corrs.	No. Side Lap	295	240	181	149	123	92·7
	1 Corr. Side Lap	320	260	196	161	133	101
	1½ Corr. Side Lap	341	277	209	172	142	107
	2 Corr. Side Lap	365	297	224	184	152	115
10/3" Corrs.	No Side Lap	285	227	177	145	120	90·3
	1 Corr. Side Lap	304	242	189	155	128	96·3
	1½ Corr. Side Lap	320	255	199	163	135	102
	2 Corr. Side Lap	338	269	210	172	142	107
6/5" Corrs.	No Side Lap	286	229	179	147	121	90·9
	1 Corr. Side Lap	315	252	197	162	133	100
	1½ Corr. Side Lap	343	275	215	176	145	109
	2 Corr. Side Lap	378	302	236	194	160	120

"Ordinary Commercial Quality" is primarily for use in the Home Market and, based on the B.S.S. No. 798 (1938), provides only for galvanised Corrugated Sheets with a minimum of Coating Spelter of 1¾ oz. per square foot (including both sides).

The Tabulated weights are inclusive of Side Laps indicated, but the weight of the *end* Laps must be added.

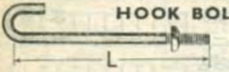
Additional allowance up to 5% is required to allow for Rolling Margin and variation in Weight of Sheets.

DATA FOR VARIOUS TYPES OF CORRUGATED SHEETING

TYPE OF CORRUGATED SHEETS	Thickness	Depth over Corrs	Centres of Purlins	Centres of Side Rails	Overall width of Sheet	Nett Width covered by One Sheet — Side Lap Corrs. ½	1	1½	2	Wt. lbs. sq. ft. laid	Approx. Price per sqr. fixed
Steel 10/3" 24 gauge	1/32"	¾"	6' 0"	6' 6"	2' 8"	—	2' 6"	2' 4½"	2' 3"	2	75/-
,, 10/3" 22 gauge	1/32"	¾"	7' 0"	7' 6"	2' 8"	—	2' 6"	2' 4½"	2' 3"	2¼	82/-
,, 10/3" 20 gauge	3/64"	¾"	7' 6"	8' 0"	2' 8"	—	2' 6"	2' 4½"	2' 3"	2½	90/-
,, 10/3" 18 gauge	3/64"	¾"	8' 0"	8' 6"	2' 8"	—	2' 6"	2' 4½"	2' 3"	2¾	106/-
,, 8/3" 24 gauge	1/32"	¾"	6' 0"	6' 6"	2' 2"	—	2' 0"	1' 10½"	1' 9"	2	78/6
,, 8/3" 22 gauge	1/32"	¾"	7' 0"	7' 6"	2' 2"	—	2' 0"	1' 10½"	1' 9"	2¼	86/-
Asbestos Big Six -	9/32"	2⅛"	4' 6"	6' 0"	3' 5½"	3' 3½"	—	—	—	3¼	59/6
,, Standard -	¼"	1"	3' 0"	5' 0"	2' 6"	—	—	2' 1½"	—	3	59/6
,, Super Seven	¼"	2¼"	4' 9"	6' 0"	3' 1½"	2' 10½"	—	—	—	3¼	—
,, Twin Twelve	9/32"	2¼"	4' 6"	6' 0"	4' 2½"	4' 0"	—	—	—	3¼	59/6
,, Trafford Tile	¼"	2"	4' 6"	6' 0"	3' 8"	—	3' 4"	—	—	3	59/6
,, Reinf. Troughing	⅜"	3½"	6' 3"	—	4' 0"	3' 9"	—	—	—	4¾	129/-
,, Reinf. Trofsec.	⅜"	3⅛"	6' 6"	—	4' 0"	3' 9"	—	—	—	4½	111/-
R.P.M. 10½/2⅝" 24g	3/32"	⅝"	5' 6"	6' 6"	2' 3½"	—	—	1' 11¾"	—	2½	130/-
,, 8/3" 24g	3/32"	1"	6' 3"	7' 3"	2' 2¼"	—	1' 11¾"	—	1' 8½"	2½	146/-
,, 10½/2⅝" 22g	3/32"	⅝"	6' 6"	7' 6"	2' 3½"	—	—	1' 11¾"	—	3	137/3
,, 8/3" 22g	3/32"	1"	7' 3"	8' 3"	2' 2¼"	—	1' 11¾"	—	1' 8½"	3	154/-
,, V Beam 24g	1/16"	1⅞"	8' 0"	—	1' 11½"	1' 9"	—	—	—	2½	144/-
,, V Beam 22g	1/16"	1⅞"	9' 0"	—	1' 11½"	1' 9"	—	—	—	3	152/-

GALVANISED SHEET FIXINGS
Sizes and Approx. Weights in Lbs. per gross and square

HOOK BOLTS



Each with Square Nut

Length, L	3½"		4"		4½"		5"	
Diameter	5/16"	3/8"	5/16"	3/8"	5/16"	3/8"	5/16"	3/8"
Wt. per Grs.	18·7	24·9	20·4	28·0	22·4	32·0	24·9	37·3
Wt. per Sq.	5·8	7·7	6·3	8·6	6·9	9·9	7·7	11·5

SHEETING BOLTS ¼" diameter

Length in Inches	¾	1¼	1½
Weight per Gross	3·6	4·7	5·1
Weight per Square	1·5	2·0	2·2

CUP HEADED RIVETS ¼" diameter

Length in Inches	3/8	½	5/8
Weight per Gross	2·0	2·15	2·3
Weight per Square	0·81	0·87	0·93

ROOFING NAILS ¼" diameter

Length in Inches	2½	3
Weight per Gross	5·1	5·9
Weight per Square	2·1	2·4

SCREWS ¼" diameter

Length in Inches	2½	3
Weight per Gross	5·3	7·0
Weight per Square	2·2	2·9

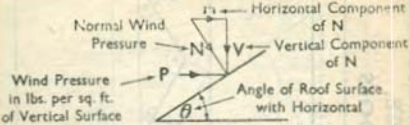
ROUND WASHERS ½" diameter

DIAMOND WASHERS ½" diameter

	Round Washers	Diamond Washers
Weights per Gross	1·96 pounds	5·22 pounds
Weights per Square	0·81 pounds	3·0 pounds

1 square – 100 square feet

WIND PRESSURES



Duchemin's Formula $N = P \times \frac{2 \sin \theta}{1 + \sin^2 \theta}$

Values of V.H. (to be measured on the roof slope length) taking N as 15 lbs. per sq. ft.

Roof Angle θ	(a) 20°	(b) ⅕ Span 21° 48′	(b) ¼ Span 26° 34′	(c) 30°	(c) ⅓ Span 33° 41′	(d) 45°	(e) 60°
N	15·0	15·0	15·0	15·0	15·0	15·0	15·0
V	14·1	13·9	13·4	13·0	12·5	10·6	7·5
H	5·13	5·57	6·71	7·50	8·32	10·6	13·0

(a) for Cinema roofs of large spans. (b) For ordinary corrugated sheeted roofs. (c) For slated or tiled roofs or north-light sheeted roof slope. (d) For church roofs or special structures. (e) For north-light glazed roof slope.

Wind Velocity (W)

High Wind	20 m.p.h.	Storm	60 m.p.h.
Gale	50 m.p.h.	Hurricane	80 m.p.h.

Pressure in lbs. sq. ft. Vert. Surface (P) $= 0{\cdot}003 \times W^2$

Wind Pressure

In lbs. per sq. ft. Vert. Surface on Chimneys and Towers

Circular Sections $= 0{\cdot}5 \; P \times A$

Hexagonal Sections $= 0{\cdot}65 \; P \times A$

Octagonal Sections $= 0{\cdot}75 \; P \times A$

A = Area of cross section in sq. ft.

Reduction of Wind Pressure on Multiple Span Roofs

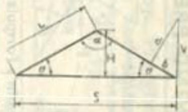
Windward Span $= P$ 1st Succeeding Span $= 0{\cdot}5P$

2nd Succeeding Span $= 0{\cdot}25P$. Remaining Spans $= 0{\cdot}125P$

RAFTER LENGTHS, Etc., OF RIDGE, NORTH LIGHT AND CURVED ROOFS

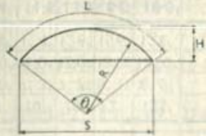
Ridge Roof

Rise	θ	α	L	v	b	H
1 in $1\frac{1}{2}$	33° 41′	112° 38′	·6009s	1·202d	$\frac{2}{3}d$	$\frac{1}{3}s$
1 in 1·732	30°	120°	·5773s	1·1548d	·5774d	·2887s
1 in 2	26° 34′	126° 52′	·5590s	1·1178d	$\frac{1}{2}d$	$\frac{1}{4}s$
1 in $2\frac{1}{2}$	21° 48′	136° 24′	·5385s	1·076d	$\frac{2}{5}d$	$\frac{1}{5}s$
1 in 3	18° 26′	143° 8′	·5270s	1·0542d	$\frac{1}{3}d$	$\frac{1}{6}s$
1 in 4	14° 2′	151° 56′	·5153s	1·0308d	$\frac{1}{4}d$	$\frac{1}{8}s$

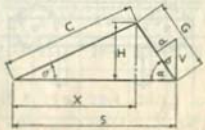


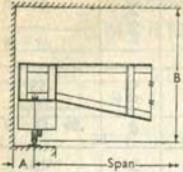
Curved and North Light Roofs

H	R	θ	L
$\frac{1}{6}S$	$\frac{5}{6}S$	73° 44′	1·0726s
$\frac{1}{5}S$	$\frac{29}{40}S$	87° 12′	1·1033s
$\frac{9}{40}S$	·668s	96° 54′	1·1278s
$\frac{1}{4}S$	$\frac{5}{8}S$	106° 16′	1·1596s



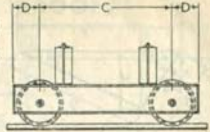
Rise	C	G	x	H	v	b
$\theta = 21° 48'$ $\alpha = 55°$	0·8414s	0·3815s	0·7812s	0·3125s	1·743d	1·4281d
$\theta = 30°$ $\alpha = 60°$	0·866s	0·5s	0·75s	0·433s	2d	1·7321d





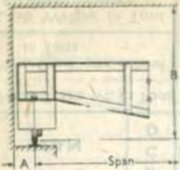
OVERHEAD ELECTRIC TRAVELLING CRANES

Double Girder Type



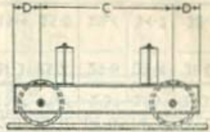
Working Load in Tons		1	2	3	5	10	15	20	25	30	40	50	60
20′ 0″ SPAN	A	8½″	8½″	8½″	8¾″	9¼″	10″	10″	12″	12¾″			
	B	4′ 8″	5′ 3″	5′ 4″	6′ 2″	6′ 10″	7′ 2″	7′ 7″	8′ 1″	8′ 5″			
	C	8′ 0″	8′ 0″	8′ 0″	9′ 0″	10′ 0″	11′ 0″	12′ 0″	12′ 0″	12′ 0″			
	D	10″	10″	10″	1′ 1″	1′ 2″	1′ 4″	1′ 6″	1′ 6″	1′ 6″			
Total Crane wt. in Tons		5·4	5·4	5·5	6·1	8·4	11·0	13·0	16·0	18·8			
Max. Load per wheel in Tons - -		2·9	3·3	3·9	5·0	8·0	11·0	14·0	17·0	23·9			

30′ 0″ SPAN	A	8½″	8½″	8½″	8¾″	9¼″	10″	10″	12″	12¾″	13½″	14½″	15″
	B	4′ 8″	5′ 3″	5′ 4″	6′ 2″	6′ 10″	7′ 2″	7′ 7″	8′ 1″	8′ 5″	9′ 10″	10′ 6″	11′ 3″
	C	8′ 0″	8′ 0″	8′ 0″	9′ 0″	10′ 0″	11′0″	12′0″	12′0″	12′ 0″	14′ 0″	14′ 9″	15′ 0″
	D	10″	10″	10″	1′ 1″	1′ 2″	1′ 4″	1′ 6″	1′ 6″	1′ 6″	2′ 3″	2′ 3″	2′ 3″
Total Crane wt. in Tons		6·6	6·6	6·7	8·0	10·5	12·5	15·0	18·7	22·6	26·6	29·7	33·9
Max. Load per wheel in Tons - -		3·2	3·8	4·3	5·5	8·7	11·6	14·3	18·3	24·9	27·8	33·9	38·0
40′ 0″ SPAN	A	8½″	8½″	8½″	8¾″	9¼″	10″	10″	12″	12¾″	13½″	14½″	15″
	B	4′10″	5′ 3″	5′ 4″	6′ 2″	7′ 3″	7′ 4″	7′ 7″	8′ 1″	8′ 5″	9′ 10″	10′ 6″	11′ 3″
	C	8′ 0″	8′ 0″	8′ 0″	9′ 0″	10′ 0″	11′0″	12′0″	12′0″	12′ 0″	14′ 0″	14′ 9″	15′ 0″
	D	10″	10″	10″	1′ 1″	1′ 2″	1′ 4″	1′ 6″	1′ 6″	1′ 6″	2′3″	2′ 3″	2′ 3″
Total Crane wt. in Tons		8·9	8·9	9·0	10·0	12·0	15·0	18·0	21·3	25·7	29·6	33·4	38·0
Max. Load per wheel in Tons - -		3·9	4·6	5·1	6·3	9·2	13·0	16·0	19·4	25·8	29·4	34·7	38·0
Crab Weight in Tons (All Spans similar)		1·5	1·8	1·8	2·0	3·5	4·5	5·5	7	8	11	13	15



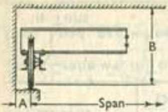
OVERHEAD ELECTRIC TRAVELLING CRANES

Double Girder Type



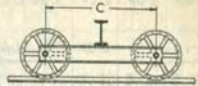
Working Load in Tons		1	2	3	5	10	15	20	25	30	40	50	60
50′ 0″ SPAN	A	8½″	8½″	8½″	8¾″	9¼″	10″	10″	12″	12¾″	13½″	14½″	15″
	B	4′10″	5′ 3″	5′ 4″	6′ 2″	7′ 3″	7′ 4″	7′ 7″	8′ 1″	8′ 5″	9′ 10″	10′ 6″	11′ 3″
	C	8′ 0″	8′ 0″	8′ 0″	9′ 0″	10′0″	11′0″	12′0″	12′0″	12′ 0″	14′ 0″	14′ 9″	15′ 0″
	D	10″	10″	10″	1′ 1″	1′ 2″	1′ 4″	1′ 6″	1′ 6″	1′ 6″	2′ 3″	2′ 3″	2′ 3″
Total Crane wt. in Tons		10·9	11·5	11·7	11·8	13·3	16·1	18·5	23·5	28·6	33·3	37·5	42·5
Max. Load per wheel in Tons - -		4·8	5·3	5·6	6·7	9·7	12·9	16·0	20·5	26·6	31·0	36·5	42·4

60′ 0″ SPAN	A			8¾″	9¼″	10″	10″	12″	12″	12¾″	13½″	14½″	15″
	B			5′ 10″	6′4′	7′ 3″	7′ 4″	7′ 9″	8′ 1″	8′ 5″	9′ 10″	10′ 6″	11′ 3″
	C			9′ 0″	10′0″	11′0″	12′0″	12′0″	12′0″	12′ 0″	14′ 0″	14′ 9″	15′ 0″
	D			10″	1′ 1″	1′ 2″	1′ 4″	1′ 6″	1′ 6″	1′ 6″	2′ 3″	2′ 3″	2′ 3″
Total Crane wt. in Tons				13·0	14·0	17·0	20·0	23·0	27·4	32·7	37·2	41·5	47·2
Max. Load per wheel in Tons - -				5·8	7·0	10·3	14·0	18·0	22·1	28·1	32·6	38·5	44·7
70′ 0″ SPAN	A				9¼″	10″	10″	12″	12″	12¾″	13½″	14½″	15″
	B				6′ 4″	7′ 3″	7′ 4″	7′ 9″	8′ 1″	8′ 5″	9′ 10″	10′ 6″	11′ 3″
	C				10′0″	11′0″	12′0″	12′0″	12′0″	12′ 0″	14′ 0″	14′ 9″	15′ 0″
	D				1′ 1″	1′ 2″	1′ 4″	1′ 6″	1′ 6″	1′ 6″	2′ 3″	2′ 3″	2′ 3″
Total Crane wt. in Tons					14·0	16·6	20·4	24·5	29·7	36·2	40·9	45·9	51·7
Max. Load per wheel in Tons - -					7·5	11·0	14·6	18·9	23·3	28·5	43·3	40·5	46·9
Crab Weight in Tons (All Spans similar)		1·5	1·8	1·8	2·0	3·5	4·5	5·5	7	8	11	13	15



OVERHEAD HAND TRAVELLING CRANES

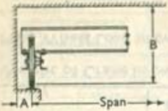
Single Girder Type



Working Load in Tons		½	1	1½	2	3	4	5	6	7½	10
10′ 0″ SPAN	A	7″	7″	7″	7″	7″	7½″	7½″	7½″	7½″	8½″
	B	1′ 10″	1′ 10″	1′ 10″	2′ 3″	2′ 3″	2′ 5″	2′ 7″	2′ 10″	2′ 11″	3′ 2″
	C	4′ 6″	4′ 6″	4′ 6″	5′ 3″	5′ 3″	6′ 0″	6′ 0″	7′ 0″	7′ 0″	7′ 3″
Weight of Crane in cwts.		9	9·5	10	13·5	14	20	21	26	29	40
Max. Wheel Load in cwts.		8	13	18	24	35	47	57	69	85	113

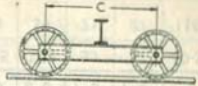
15′ 0″ SPAN	A	7″	7″	7″	7″	7″	7½″	7½″	7½″	7½″	8½″
	B	1′ 10″	1′ 10″	1′ 11″	2′ 3″	2′ 3″	2′ 7″	2′ 8″	3′ 0″	3′ 0″	3′ 4″
	C	4′ 6″	4′ 6″	4′ 6″	5′ 3″	5′ 3″	6′ 0″	6′ 0″	7′ 0″	7′ 0″	7′ 3″
Weight of Crane in cwts.		10·5	11	11·5	15·5	16	22·5	24	29·5	32·5	44·5
Max. Wheel Loadin cwts.		9	14	19	25	35	47	58	70	86	114
20′ 0″ SPAN	A	7″	7″	7″	7″	7″	7½″	7½″	7½″	7½″	8½″
	B	1′ 10″	2′ 1″	2′ 1″	2′ 5″	2′ 5″	2′ 9″	2′ 9″	3′ 1″	3′ 2″	3′ 8″
	C	4′ 6″	4′ 6″	4′ 6″	5′ 3″	5′ 3″	6′ 0″	6′ 0″	7′ 0″	7′ 0″	7′ 3″
Weight of Crane in cwts.		12	14	14·5	18	19	26·5	27·5	33	37	50·5
Max. Wheel Load in cwts.		9	14	19	25	36	48	59	71	87	116

These figures for both the hand and electric travelling cranes are approximate and should be used for design purposes only.



OVERHEAD HAND TRAVELLING CRANES

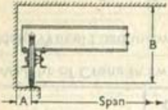
Single Girder Type



Working Load in Tons		½	1	1½	2	3	4	5	6	7½	10
25′ 0″ SPAN	A	7″	7″	7″	7″	7″	7½″	7½″	7½″	7½″	8½″
	B	2′ 1″	2′ 1″	2′ 3″	2′ 6″	2′ 7″	2′ 10″	2′ 11″	3′ 4″	3′ 6″	3′ 10″
	C	4′ 6″	4′ 6″	4′ 6″	5′ 3″	5′ 3″	6′ 0″	6′ 0″	7′ 0″	7′ 0″	7′ 3″
Weight of Crane in cwts.		15	15·5	17	21	23	30	31·5	38	43	55·5
Max. Wheel Load in cwts.		10	15	20	26	37	49	60	72	89	117

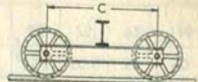
30′ 0″ SPAN	A	7″	7″	7″	7″	7″	7½″	7½″	7½″	7½″	8½″
	B	2′ 1″	2′ 3″	2′ 4″	2′ 7″	2′ 8″	3′ 1″	3′ 3″	3′ 6″	3′ 8″	4′ 0″
	C	4′ 6″	4′ 6″	4′ 6″	5′ 3″	5′ 3″	6′ 0″	6′ 0″	7′ 0″	7′ 0″	7′ 3″
Weight of Crane in cwts.		17	18·5	20·5	25	27	35·5	38·5	43·5	49·5	65
Max. Wheel Load in cwts.		10	16	21	27	38	51	61	73	90	119
35′ 0″ SPAN	A	7″	7″	7″	7″	7½″	7½″	7½″	7½″	8½″	9″
	B	2′ 3″	2′ 4″	2′ 5″	2′ 8″	2′ 11″	3′ 3″	3′ 5″	3′ 8″	3′ 10″	3′ 10″
	C	4′ 6″	4′ 6″	4′ 6″	5′ 3″	5′ 6″	6′ 0″	6′ 0″	7′ 0″	7′ 0″	7′ 8″
Weight of Crane in cwts.		19·5	22	24	28·5	32	42	45·5	50·5	58	90
Max. Wheel Load in cwts.		11	16	22	28	40	52	63	75	92	116

These figures for both the hand and electric travelling cranes are approximate and should be used for design purposes only.



OVERHEAD HAND TRAVELLING CRANES

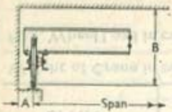
Single Girder Type



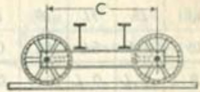
Working Load in Tons		½	1	1½	2	3	4	5	6	7½	10
40′ 0″ SPAN	A	7″	7″	7″	7″	7½″	7½″	7½″	7½″	8½″	9″
	B	2′ 5″	2′ 6″	2′ 7″	2′ 11″	3′ 1″	3′ 5″	3′ 7″	3′ 10″	3′ 10″	3′ 10″
	C	4′ 6″	4′ 6″	4′ 6″	5′ 3″	5′ 6″	6′ 0″	6′ 0″	7′ 0″	7′ 0″	7′ 8″
Weight of Crane in cwts.		25·5	28	30	34·5	39·5	49	55	60	85	100
Max. Wheel Load in cwts.		12	18	24	29	41	54	66	78	92	121

45′ 0″ SPAN	A	7″	7″	7″	7″	7½″	7½″	7½″	7½″	8½″	9″
	B	2′ 7″	2′ 9″	2′ 11″	3′ 1″	3′ 3″	3′ 7″	3′ 7″	3′ 8″	3′ 8″	3′ 8″
	C	4′ 6″	4′ 6″	4′ 6″	5′ 3″	5′ 6″	6′ 0″	6′ 0″	7′ 0″	7′ 0″	7′ 8″
Weight of Crane in cwts.		30·5	32·5	35	42·5	47·5	59·5	60	65	90	110
Max. Wheel Load in cwts.		13	19	25	31	43	57	67	75	94	123
50′ 0″ SPAN	A	7″	7″	7″	7½″	7½″	7½″	7½″	7½″	8½″	9″
	B	3′ 0″	3′ 0″	3′ 0″	3′ 3″	3′ 5″	3′ 5″	3′ 5″	3′ 6″	3′ 8″	3′ 8″
	C	5′ 0″	5′ 0″	5′ 0″	5′ 6″	5′ 6″	6′ 0″	6′ 0″	7′ 0″	7′ 0″	7′ 8″
Weight of Crane in cwts.		35	40	43	50	58	70	75	75	110	140
Max. Wheel Load in cwts.		15	21	27	33	46	57	67	76	97	129

These figures for both the hand and electric travelling cranes are approximate and should be used for design purposes only.



OVERHEAD HAND TRAVELLING CRANES

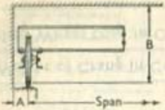


Double Girder Type

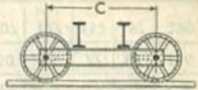
Working Load in Tons		2	3	4	5	7½	10	12½	15	20	30
10′ 0″ SPAN	A	7″	7″	7½″	7½″	7½″	8½″	8½″	9″	9″	
	B	2′ 8″	2′ 9″	3′ 2″	3′ 4″	3′ 10″	4′ 3″	4′ 10″	5′ 3″	5′ 3″	
	C	4′ 3″	4′ 3″	5′ 0″	5′ 0″	5′ 0″	6′ 5″	6′ 5″	9′ 4″	9′ 4″	
Weight of Crane in Cwts.	-	14	15	23	24	28	38	43	61	67	
Max. Wheel Load in Cwts.	-	22	30	41	49	72	95	107	125	162	

15′ 0″ SPAN	A	7″	7″	7½″	7½″	7½″	8½″	8½″	9″	9″	
	B	2′ 8″	2′ 10″	3′ 3″	3′ 6″	4′ 1″	4′ 6″	5′ 1″	5′ 6″	5′ 6″	
	C	4′ 3″	4′ 3″	5′ 0″	5′ 0″	5′ 0″	6′ 5″	6′ 5″	9′ 4″	9′ 4″	
Weight of Crane in Cwts.	-	17	19	25	26	31	42	49	70	77	
Max. Wheel Load in Cwts.	-	23	33	43	52	77	102	119	142	185	
20′ 0″ SPAN	A	7″	7″	7½″	7½″	7½″	8½″	8½″	9″	9″	9″
	B	2′ 9″	3′ 1″	3′ 5″	3′ 9″	4′ 4″	4′ 8″	5′ 3″	5′ 7″	5′ 7″	6′ 3″
	C	4′ 3″	4′ 3″	5′ 0″	5′ 0″	5′ 0″	6′ 5″	6′ 5″	9′ 4″	9′ 4″	9′ 4″
Weight of Crane in Cwts.	-	21	24	30	31	39	50	56	77	83	116
Max. Wheel Load in Cwts.	-	25	35	46	55	81	107	127	152	197	290

These figures are approximate and should be used for design purposes only.
Wheel Loads do not include impact.



OVERHEAD HAND
TRAVELLING CRANES



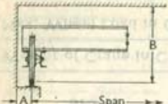
Double Girder Type

Working Load in Tons		2	3	4	5	7½	10	12½	15	20	30
25′ 0″ SPAN	A	7″	7″	7½″	7½″	7½″	8½″	8½″	9″	9″	9″
	B	2′ 11″	3′ 3″	3′ 7″	3′ 10″	4′ 4″	4′ 9″	5′ 6″	6′ 1″	6′ 1″	6′ 7″
	C	4′ 3″	4′ 3″	5′ 0″	5′ 0″	5′ 0″	6′ 5″	6′ 5″	9′ 4″	9′ 4″	9′ 4″
Weight of Crane in Cwts.	-	24	26	35	37	45	56	67	90	93	132
Max. Wheel Load in Cwts.	-	26	36	48	58	84	110	133	160	205	304

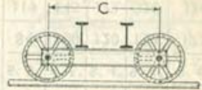
30′ 0″ SPAN											
	A	7″	7″	7½″	7½″	7½″	8½″	8½″	9″	9″	9″
	B	3′ 1″	3′ 5″	3′ 9″	4′ 0″	4′ 7″	5′ 2″	5′ 8″	6′ 3″	6′ 3″	6′ 7″
	C	4′ 3″	4′ 3″	5′ 0″	5′ 0″	5′ 0″	6′ 5″	6′ 5″	9′ 4″	9′ 4″	9′ 4″
Weight of Crane in Cwts.	-	31	36	43	47	54	72	78	100	113	148
Max. Wheel Load in Cwts.	-	28	39	50	60	87	115	138	165	214	315
35′ 0″ SPAN											
	A	7″	7½″	7½″	7½″	8½″	8½″	8½″	9″	9″	9″
	B	3′ 3″	3′ 9″	3′ 11″	4′ 3″	5′ 2″	5′ 2″	6′ 0″	6′ 5″	6′ 5″	6′ 10″
	C	4′ 3″	5′ 0″	5′ 0″	5′ 0″	6′ 5″	6′ 5″	6′ 5″	9′ 4″	9′ 4″	9′ 4″
Weight of Crane in Cwts.	-	39	48	53	57	76	84	100	120	123	172
Max. Wheel Load in Cwts.	-	30	42	53	63	93	119	145	172	220	324

These figures are approximate and should be used for design purposes only.
Wheel Loads do not include impact.

OVERHEAD HAND TRAVELLING CRANES



Double Girder Type

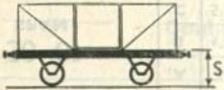
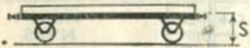


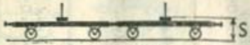
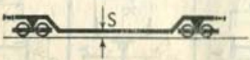
Working Load in Tons		2	3	4	5	7½	10	12½	15	20	30
40′ 0″ SPAN	A	7″	7½″	7½″	7½″	8½″	8½″	8½″	9″	9″	9″
	B	3′ 5″	4′ 2″	4′ 3″	4′ 5″	5′ 2″	5′ 6″	6′ 0″	6′ 5″	6′ 5″	6′ 10″
	C	4′ 3″	5′ 0″	5′ 0″	5′ 0″	6′ 5″	6′ 5″	6′ 5″	9′ 4″	9′ 4″	9′ 4″
Weight of Crane in Cwts.	-	53	68	70	71	90	105	111	130	149	194
Max. Wheel Load in Cwts.	-	33	47	57	67	97	125	149	177	228	333

45′ 0″ SPAN	A	7″	7½″	7½″	7½″	8½″	8½″	8½″	9″	9″	9″
	B	3′ 11″	4′ 4″	4′ 4″	4′ 5″	5′ 6″	5′ 6″	6′ 0″	6′ 8″	6′ 8″	6′ 10″
	C	4′ 3″	5′ 0″	5′ 0″	5′ 0″	6′ 5″	6′ 5″	6′ 5″	9′ 4″	9′ 4″	9′ 4″
Weight of Crane in Cwts.	-	78	84	95	96	110	120	125	147	167	212
Max. Wheel Load in Cwts.	-	40	51	64	74	102	129	152	183	234	341
50′ 0″ SPAN	A	7½″	7½″	7½″	7½″	8½″	8½″	9″	9″	9″	9″
	B	4′ 5″	4′ 6″	4′ 6″	4′ 9″	5′ 6″	5′ 6″	6′ 3″	6′ 8″	6′ 8″	6′ 10″
	C	5′ 0″	5′ 0″	5′ 0″	5′ 0″	6′ 5″	6′ 5″	9′ 4″	9′ 4″	9′ 4″	9′ 4″
Weight of Crane in Cwts.	-	102	103	105	115	118	130	157	181	185	236
Max. Wheel Load in Cwts.	-	46	56	66	79	104	132	162	192	240	350

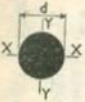
These figures are approximate and should be used for design purposes only.
Wheel Loads do not include impact.

RAILWAY WAGON DATA

Diagram	Capacity in Tons	Loading Length Inside	Width Inside	Dim. "S" from rail	Height of Sides
 GOODS WAGON	12	Up to 17′ 0″	7′ 7″	4′ 1¾″ to Floor	3′ 1¾″ from Floor
 4 WHEEL PLATE WAGON	20	24′ 0″ 27′ 0″ 30′ 6″	8′ 3″	4′ 3¼″ to Bearers	10⅜″ from Floor

 2 SINGLE BOLSTERS	16 (sections)	24′ 0″ to 28′ 0″	—	4′ 7″ to Bolster	—
 BOGIE BOLSTER	25 30 35 40	Up to 52′ 0″	8′ 0″ to 8′ 6″	5′ 0″ to 5′ 10″ to Bolster	—
 WELL WAGON	12 to 40	20′ 0″ to 34′ 0″	—	2′ 6″ to Floor	—

APPROXIMATE RADII OF GYRATION (R)
of Various Structural Sections



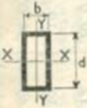
Rx–x = 0·25d

Ry–y = 0·25d

Rx–x = 0·35d

Ry–y = 0·35d

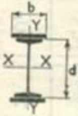
d = Mean dia.



Rx–x = 0·41d

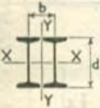
Ry–y = 0·41b

Mean d & b



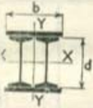
Rx–x = 0·49d

Ry–y = 0·23b



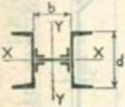
Rx–x = 0·41d

Ry–y = 0·52b



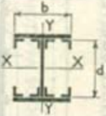
Rx–x = 0·48d

Ry–y = 0·28b



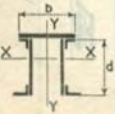
Rx–x = 0·29d

Ry–y = 0·50b



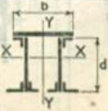
Rx–x = 0·45d

Ry–y = 0·30b



Rx–x = 0·40d

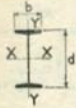
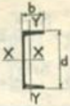
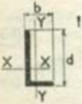
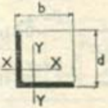
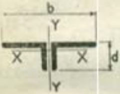
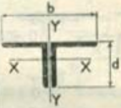
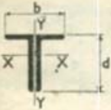
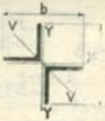
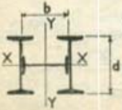
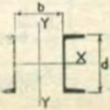
Ry–y = 0·34b



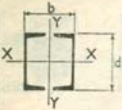
Rx–x = 0·40d

Ry–y = 0·33b

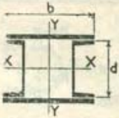
APPROXIMATE RADII OF GYRATION (R)
of Various Structural Sections

	Rx–x = 0·40d Ry–y = 0·20b		Rx–x = 0·38d Ry–y = 0·28b
	Rx–x = 0·31d Ry–y = 0·29b		Rx–x = 0·30d Ry–y = 0·30b b = d
	Rx–x = 0·29d Ry–y = 0·23b		Rx–x = 0·30d Ry–y = 0·22b
	Rx–x = 0·31d Ry–y = 0·21b		Rx–x = 0·30d Ry–y = 0·30b Rv–v = 0·19d b = d
	Rx–x = 0·32d Ry–y = 0·49b		Rx–x = 0·38d Ry–y = 0·60b

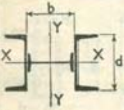
APPROXIMATE RADII OF GYRATION (R)
of Various Structural Sections



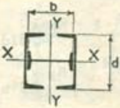
$R_{x-x} = 0{\cdot}38d$
$R_{y-y} = 0{\cdot}45b$



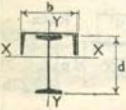
$R_{x-x} = 0{\cdot}45d$
$R_{y-y} = 0{\cdot}31b$



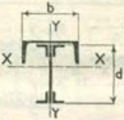
$R_{x-x} = 0{\cdot}31d$
$R_{y-y} = 0{\cdot}54b$



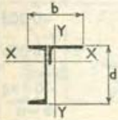
$R_{x-x} = 0{\cdot}31d$
$R_{y-y} = 0{\cdot}45b$



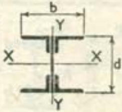
$R_{x-x} = 0{\cdot}40d$
$R_{y-y} = 0{\cdot}24b$



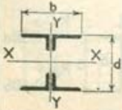
$R\ x-x = 0{\cdot}41d$
$R_{y-y} = 0{\cdot}22b$



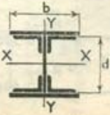
$R_{x-x} = 0{\cdot}38d$
$R_{y-y} = 0{\cdot}21b$



$R_{x-x} = 0{\cdot}40d$
$R_{y-y} = 0{\cdot}23b$

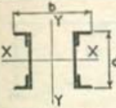


$R_{x-x} = 0{\cdot}39d$
$R_{y-y} = 0{\cdot}21b$

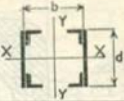


$R_{x-x} = 0{\cdot}45d$
$R_{y-y} = 0{\cdot}24b$

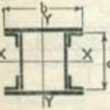
APPROXIMATE RADII OF GYRATION (R)
of Various Structural Sections



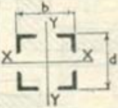
Rx–x = 0·36d
Ry–y = 0·32b



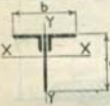
Rx–x = 0·36d
Ry–y = 0·45b



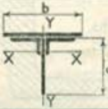
Rx–x = 0·45d
Ry–y = 0·30b



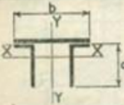
Rx–x = 0·42d
Ry–y = 0·42b



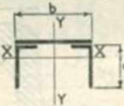
Rx–x = 0·31d
Ry–y = 0·18b



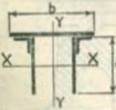
Rx–x = 0·25d
Ry–y = 0·21b



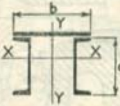
Rx–x = 0·32d
Ry–y = 0·32b



Rx–x = 0·25d
Ry–y = 0·37b



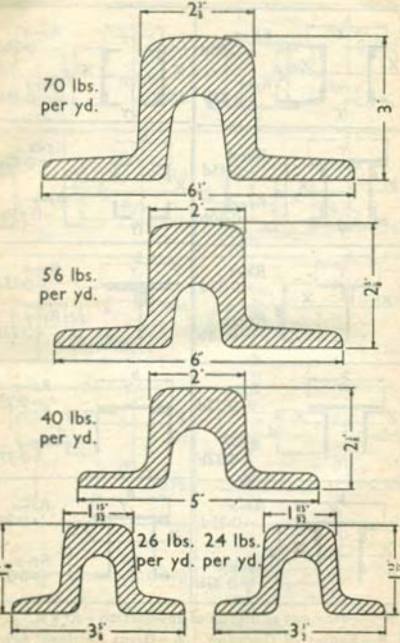
Rx–x = 0·25d
Ry–y = 0·32b



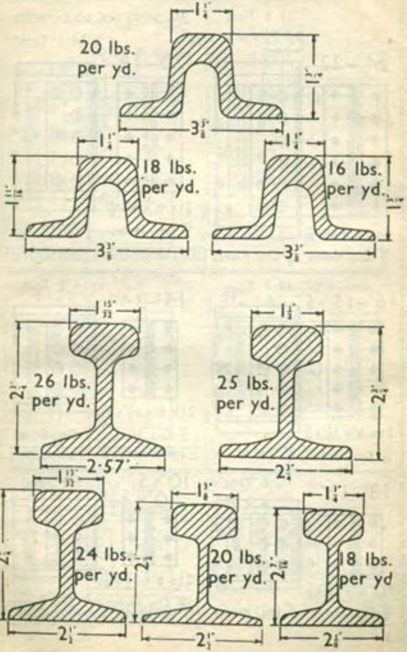
Rx–x = 0·39d
Ry–y = 0·32b

Moment of Inertia of any Section = $R^2 \times A$.
R = Radius of Gyration. A = Cross Sectional Area

BRIDGE RAILS



BRIDGE AND FLAT BOTTOM RAILS



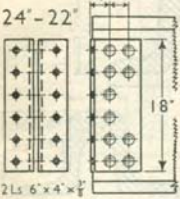
BEAM CONNECTIONS (HEAVY TYPE)

ALLOWING FOR ECCENTRICITY

24"–22"

28·9 Tons.

$2\frac{1}{2}''$ $2\frac{1}{2}''$ For all Angles

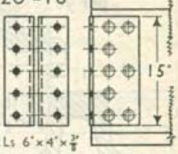


2 Ls 6" × 4" × $\frac{3}{8}$"

$\frac{15}{16}$" Dia. Holes $\frac{7}{8}$" Dia. Rivets

20"–18"

24·1 Tons.



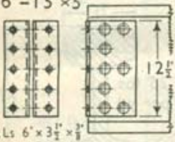
2 Ls 6" × 4" × $\frac{3}{8}$"

$\frac{15}{16}$" Dia. Holes $\frac{7}{8}$" Dia. Rivets

2 Ls 4" × 4" WITHOUT ECCENTRICITY

16"–15" × 5"

17·7 Tons.

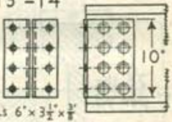


2 Ls 6" × $3\frac{1}{2}$" × $\frac{3}{8}$"

$\frac{13}{16}$" Dia. Holes $\frac{3}{4}$" Dia. Rivets

15"–14"

14·2 Tons.



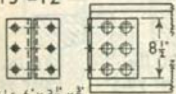
2 Ls 6" × $3\frac{1}{2}$" × $\frac{3}{8}$"

$\frac{13}{16}$" Dia. Holes $\frac{3}{4}$" Dia. Rivets

2 Ls $3\frac{1}{2}$" × $3\frac{1}{2}$" WITHOUT ECCENTRICITY

13"–12"

10·6 Tons.

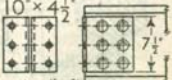


2 Ls 6" × $3\frac{1}{2}$" × $\frac{3}{8}$"

$\frac{13}{16}$" Dia. Holes $\frac{3}{4}$" Dia. Rivets

10" × 5" / 10" × $4\frac{1}{2}$"

9·9 Tons.

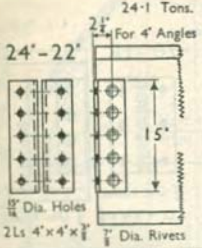


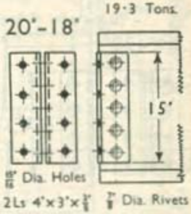
2 Ls 6" × $3\frac{1}{2}$" × $\frac{3}{8}$"

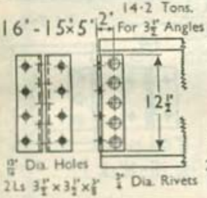
$\frac{13}{16}$" Dia. Holes $\frac{3}{4}$" Dia. Rivets

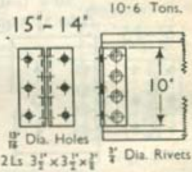
2 Ls $3\frac{1}{2}$" × $3\frac{1}{2}$" WITHOUT ECCENTRICITY

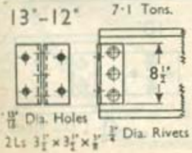
BEAM CONNECTIONS (MEDIUM TYPE)

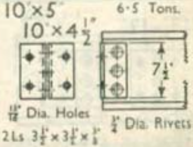








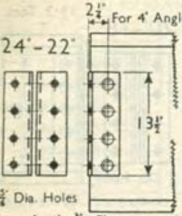




BEAM CONNECTIONS (LIGHT TYPE)

24"–22"

19·3 Tons.

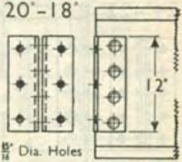


$\frac{15}{16}$" Dia. Holes

2Ls 4"×4"×$\frac{3}{8}$" $\frac{7}{8}$" Dia. Rivets

20"–18"

14·5 Tons.

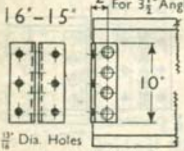


$\frac{15}{16}$" Dia. Holes

2Ls 4"×4"×$\frac{3}{8}$" $\frac{7}{8}$" Dia. Rivets

16"–15"

10·6 Tons.

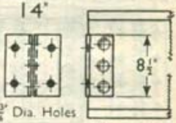


$\frac{13}{16}$" Dia. Holes

2Ls 3½"×3½"×$\frac{3}{8}$" ¾" Dia. Rivets

14"

7·1 Tons.

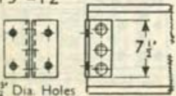


$\frac{13}{16}$" Dia. Holes

2Ls 3½"×3½"×$\frac{3}{8}$" ¾" Dia. Rivets

13"–12"

7·1 Tons.

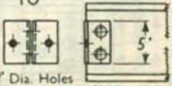


$\frac{13}{16}$" Dia. Holes

2Ls 3½"×3½"×$\frac{5}{16}$" ¾" Dia. Rivets

10"

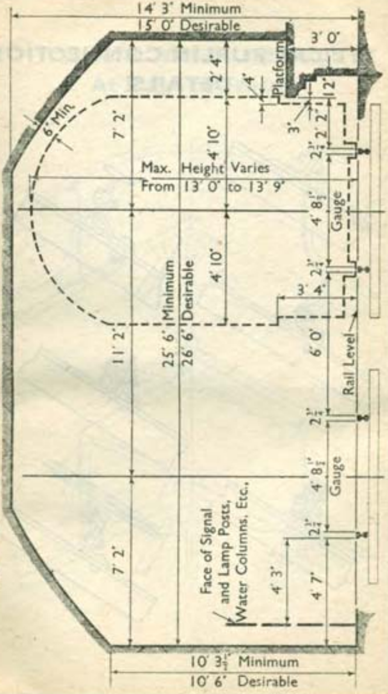
3·5 Tons.



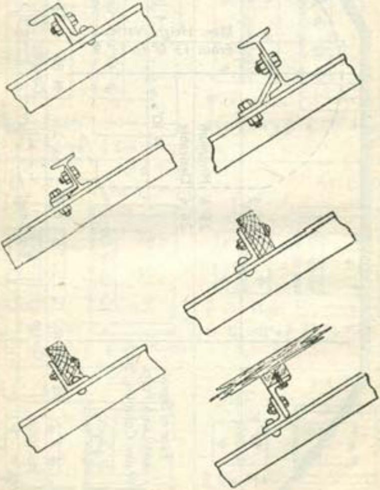
$\frac{13}{16}$" Dia. Holes

2Ls 3"×3"×$\frac{5}{16}$" ¾" Dia. Rivets

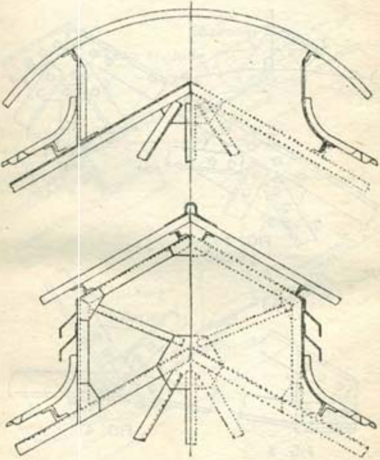
RAILWAY CLEARANCE LINES (See note Page 608)



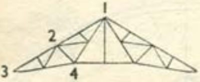
TYPICAL PURLIN CONNECTION DETAILS



TYPICAL VENT. DETAILS
At Ridge of Trusses



TYPICAL ROOF TRUSS DETAILS



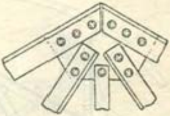


FIG. 1

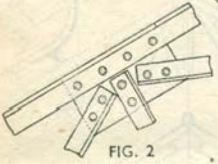


FIG. 2

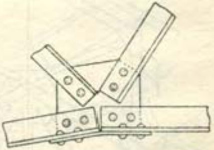


FIG. 4

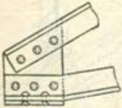
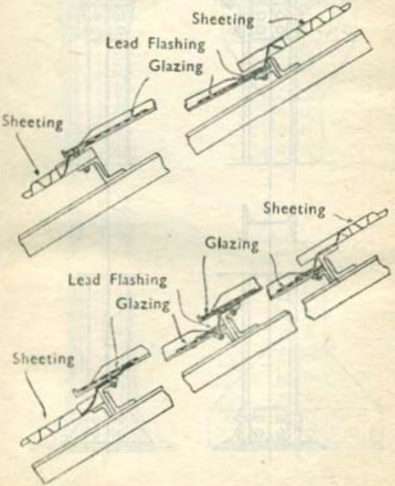


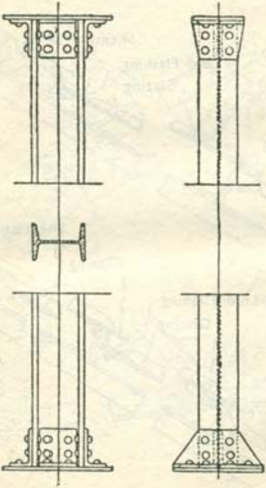
FIG. 3

TYPICAL ROOF GLAZING DETAILS
For T.S. Astragals



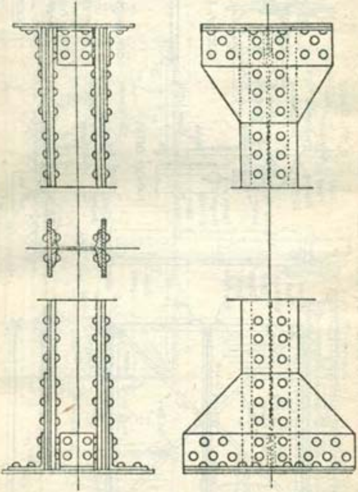
TYPICAL STANCHION DETAILS

For Light Loads

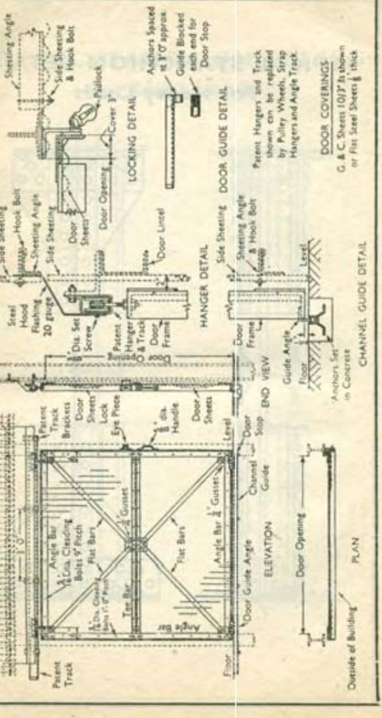


TYPICAL STANCHION DETAILS

For Heavy Loads



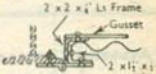
TYPICAL SINGLE LEAF SLIDING DOOR



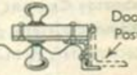
SINGLE LEAF HINGED DOOR

Standard Details

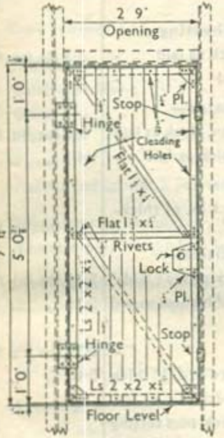
For 7' 1¼" high × 2' 9" wide opening



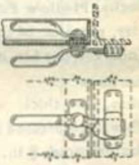
Section at Hinge



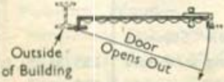
Section at Rimlock and Door Stop Cleat



ELEVATION



Interior Door Locking Handle



Door Cover: G & C Sheets or Flat Steel Sheets

SECTIONAL PLAN

Wt. of Steel 94 lbs. Approx. (Without Covering)

Unit weights of Building Materials Extracted by permission from BSS 648, 1935

Asbestos Cement Sheeting—	
Corrugated, ¼ in. thick as laid -	3·3 lbs./sq. ft.
Flat, ¼ in. thick as laid - - -	2·3 ,, ,,
Asbestos Cement Slates—	
Diamond pattern, 3/16 in. thick as laid	2·9 lbs./sq. ft.
Rectangular, 3/16 in. thick as laid -	4·1 ,, ,,
Asphalt—	
Rock and Mastic, per 1 in. of thickness, as laid - - - -	11 lbs./sq. ft.
Blocks, Hollow Partition—	
Clay, per 1 in. of thickness, as laid -	5·25 lbs./sq. ft.
Concrete, per 1 in. of thickness as laid	5·25 ,, ,,
Boards—	
Fibre, ½ in. thick - - - -	0·75 lbs./sq. ft.
Fibre, Compressed (hrd brd) ¼" thick	0·65 ,, ,,
Plaster cored, ⅜ in. thick - -	2 ,, ,,
Plaster cored, ⅜ in. thick plus setting coat - - - - - -	3 ,, ,,
Brickwork—	
"Common"—"London" Stock or "Flettons", including sand-lime	125 lbs./cu. ft.
Glazed, including sand-lime - -	130 ,, ,,
Heavy Pressed Brick includ. sand-lime	140 ,, ,,

Unit weights of Building Materials Extracted by permission from BSS 648, 1935

Cast Iron—	450 lbs./cu. ft.
Concrete—	
Ballast or Stone	140 lbs./cu. ft.
Brick	115 „ „
Clinker	90 „ „
Pumice	70 „ „
Reinforced (about 2 per cent. steel)	150 „ „
Corrugated Asbestos Cement Sheeting—¼ in. thick as laid	3·3 lbs./sq. ft.
Corrugated Steel Sheeting—	
Galvanized 18 B.G. as laid	2·72 lbs./sq. ft.
Felt Roofing—	
Bituminous (in layers)	1·5 lbs./sq. ft.
Flooring, Rubber—	
¼ in. thick	2·3 „ „
Flooring, Wood—	
Hardwood—Oak or Maple, ⅞ in.	3·3 lbs./sq. ft.
Hardwood—Oak or Maple, 1⅛ in.	4·3 „ „
Pitchpine (Longleaf Dense) ⅞ in.	3·0 „ „
Pitchpine (Longleaf Dense), 1⅛ in.	3·8 „ „

Unit weights of Building Materials Extracted by permission from BSS 648, 1935

Glass—	
Per $\frac{1}{4}$ in. of thickness - -	3·5 lbs./sq. ft.
Patent Glazing—	
$\frac{1}{4}$ in. glass including lead-covered steel bars at 24 in. centres - -	6 lbs./sq. ft.
Lead - - - - - -	707 lbs./cu. ft.
Lead, Sheet—	
Per $\frac{1}{16}$ in. of thickness, as laid -	8 lbs./sq. ft.
Plaster—	
Fibrous, $\frac{5}{8}$ in. thick - - -	3 lbs./sq. ft.
Gypsum or Lime $\frac{1}{2}$ in. thick - -	5 ,, ,,
Hydraulic Lime or Portland Cement, $\frac{1}{2}$ in. - - - - - -	6 ,, ,,
Add $1\frac{1}{4}$ lb. for wood or metal lathing	
Roof Boarding—	
Softwood, rough sawn, $\frac{3}{4}$ in. - -	2 lbs./sq. ft.
Softwood, rough sawn, 1 in. - -	2·5 ,, ,,
Softwood, rough sawn, $1\frac{1}{4}$ in. -	3 ,, ,,

Unit weights of Building Materials Extracted by permission from BSS 648, 1935

Slabs, Partition, Precast Concrete	
HOLLOW	
Clinker, per 1 in. of thickness -	5·8 lbs./sq. ft.
Coke breeze, per 1 in. of thickness	4·6 ,, ,,
Pumice, per 1 in. of thickness -	4·6 ,, ,,
Slag, granulated, per 1 in. of thickness	5·8 ,, ,,
SOLID	
Clinker, per 1 in. of thickness -	7·5 lbs./sq. ft.
Coke breeze, per 1 in. of thickness	5·8 ,, ,,
Pumice, per 1 in. of thickness -	5·8 ,, ,,
Slag, per 1 in. of thickness - -	7·5 ,, ,,
Slating—	
laid with 3 in. lap including nails but not battens.	
CORNISH. Medium grading - -	6 lbs./sq. ft.
Second grading - -	7·5 ,, ,,
Random sizes - -	9 ,, ,,
WELSH. First - - - -	5 ,, ,,
Medium - - - -	6 ,, ,,
Thick - - - -	8 ,, ,,
WESTMORLAND. First - -	9 ,, ,,
Second - -	11·3 ,, ,,
Third - -	15·5 ,, ,,

Unit weights of Building Materials Extracted by permission from BSS 648, 1935

Stone—	
Bath Stone - - - -	130 lbs./cu. ft.
Portland Stone - - -	140 ,, ,,
Marble - - - - -	170 ,, ,,
Sandstone - - - -	140 ,, ,,
Granite - - - - -	165 ,, ,,
Slate - - - - - -	177 ,, ,,
Terra Cotta. SOLID - -	132 lbs./cu. ft.
Tiling, Clay or concrete roof, plain, laid to 4 in. gauge - -	13-14·5 lbs./sq. ft.
Interlocking—Roman, Marseilles and Pan Type - - -	7·5 ,, ,,
Timber SEASONED	
Hardwoods—Teak, oak - -	45 lbs./cu. ft.
Pitchpine (Longleaf Dense) -	41 ,, ,,
Softwoods—Pine, spruce, Douglas fir - - - - - -	30 ,, ,,
Weather Boarding, ¾ in. -	1·5 lbs./sq. ft.
1 in. -	1·75 ,, ,,
Zinc Sheet, No. 14 Zinc Gauge, as laid - - - - - -	1·59 lbs./sq. ft.

Weights of Various Materials

Ashes and Coke, loose	46 lbs./cu. ft.
Barley, Wheat, in bags	40 ,, ,,
in bulk	45 ,, ,,
Cement, in bags	85 ,, ,,
Coal, loose	56 ,, ,,
Flour, in bags	44 ,, ,,
Potatoes, in bags	42 ,, ,,
in bulk	45 ,, ,,
Sand, loose, dry	100 ,, ,,
wet	125 ,, ,,
in bags	96 ,, ,,
Snow, newly fallen	6 ,, ,,
Wood, block paving	56 ,, ,,
Water, fresh	62·5 ,, ,,
sea	64 ,, ,,
1 gallon of fresh water	10 lbs.
(There are 6·25 galls. to 1 cu. ft.)	

PAINTING

Approximate Requirements for Red Lead or Oxide (ordinary quality)

Exposed Surfaces

Truss and Lattice Girder Work	0·9 gals. per ton of Steel
Plain Beam Work	0·5 ,, ,, ,,
Plated Beam Work	0·42 ,, ,, ,,
Single Storey Buildings	0·75 ,, ,, ,,

Contact Surfaces Only

Truss and Lattice Girder Work	0·06 gals. per ton of Steel
Plated Beam Work	0·42 ,, ,, ,,
Single Storey Buildings	0·07 ,, ,, ,,

Red Lead (ordinary quality) weighs 28 lbs. to 1 gal.
Red Oxide (ordinary quality) ,, 22½ ,, ,,

Corrugated Steel Sheeting

Black Oil - 1 gal. covers 68 sq. yds. of Corr. Surface
Red Oxide - 1 gal. covers 65 sq. yds. of Corr. Surface
Red Lead - 1 gal. covers 60 sq. yds. of Corr. Surface

WEIGHTS OF STEEL ANGLES IN LBS. PER LINEAL FOOT

Sum of Flanges in ins.	Thickness of Angle in Inches						
	3/16	1/4	5/16	3/8	1/2	5/8	3/4
2	1·15	1·49	1·79	2·07			
2½	1·47	1·91	2·32	2·71			
3	1·79	2·34	2·85	3·35	4·25		
3½	2·12	2·76	3·39	3·99	5·10		
4	2·43	3·19	3·92	4·62	5·95	7·17	
4½	2·75	3·61	4·45	5·26	6·80	8·23	
5	3·07	4·04	4·98	5·90	7·65	9·30	10·84
5½	3·39	4·47	5·51	6·54	8·50	10·36	12·11
6	3·71	4·89	6·04	7·17	9·35	11·42	13·39
6½	4·03	5·32	6·58	7·81	10·20	12·49	14·67
7	4·35	5·74	7·11	8·45	11·05	13·55	15·94
7½	4·67	6·16	7·64	9·09	11·92	14·61	17·22
8	4·98	6·59	8·17	9·73	12·75	15·68	18·49
8½	5·30	7·02	8·71	10·37	13·61	16·74	19·77
9	5·62	7·44	9·24	11·00	14·45	17·80	21·04
9½	5·94	7·86	9·76	11·63	15·30	18·86	22·31
10	6·26	8·29	10·29	12·28	16·16	19·93	23·59
10½	6·57	8·71	10·82	12·91	17·00	20·99	24·86
11	6·89	9·14	11·36	13·55	17·85	22·05	26·14
11½	7·21	9·56	11·89	14·19	18·70	23·11	27·41
12	7·53	9·99	12·42	14·82	19·55	24·17	28·69
12½		10·41	12·95	15·46	20·40	25·24	29·97
13		10·84	13·48	16·10	21·25	26·30	31·24
14			14·54	17·37	22·95	28·42	33·79
15			15·61	18·65	24·65	30·55	36·34
16				19·93	26·36	32·68	38·89

WEIGHT OF STEEL ROUND AND SQUARE BARS

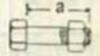
In Lbs. per Lineal Foot

Diameter or Side in Inches	Round ●	Square ■	Diameter or Side in Inches	Round ●	Square ■	Diameter or Side in Inches	Round ●	Square ■
$\frac{1}{4}$	·17	·21	$1\frac{3}{8}$	5·05	6·43	$3\frac{1}{2}$	32·71	41·65
$\frac{5}{16}$	·26	·33	$1\frac{1}{2}$	6·01	7·65	$3\frac{3}{4}$	37·55	47·81
$\frac{3}{8}$	·38	·48	$1\frac{5}{8}$	7·05	8·98	4	42·73	54·40
$\frac{7}{16}$	·51	·65	$1\frac{3}{4}$	8·18	10·41	$4\frac{1}{4}$	48·23	61·41
$\frac{1}{2}$	·67	·85	$1\frac{7}{8}$	9·39	11·95	$4\frac{1}{2}$	54·07	68·85
$\frac{5}{8}$	1·04	1·33	2	10·68	13·60	$4\frac{3}{4}$	60·25	76·71
$\frac{3}{4}$	1·50	1·91	$2\frac{1}{4}$	13·52	17·21	5	66·76	85·00
$\frac{7}{8}$	2·04	2·60	$2\frac{1}{2}$	16·69	21·25	$5\frac{1}{4}$	73·60	93·71
1	2·67	3·40	$2\frac{3}{4}$	20·19	25·71	$5\frac{1}{2}$	80·78	102·8
$1\frac{1}{8}$	3·38	4·30	3	24·03	30·60	$5\frac{3}{4}$	88·29	112·4
$1\frac{1}{4}$	4·17	5·31	$3\frac{1}{4}$	28·21	35·91	6	96·13	122·4

CALCULATED WEIGHTS IN STANDARD HEXAGONAL

To B.S.S. No. 28—1932

Length (a) in inches	Diameter of Bolt in Inches									
	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
1	·106	·222	·376	·612						
$1\frac{1}{8}$	·110	·229	·387	·628						
$1\frac{1}{4}$	·114	·236	·398	·643	·944					
$1\frac{3}{8}$	·118	·243	·408	·659	·965					
$1\frac{1}{2}$	·122	·250	·419	·675	·986	1·39				
$1\frac{5}{8}$	·126	·257	·430	·690	1·01	1·42				
$1\frac{3}{4}$	·130	·264	·441	·706	1·03	1·45	1·97			
$1\frac{7}{8}$	·134	·271	·452	·722	1·05	1·48	2·00			
2	·138	·278	·463	·737	1·07	1·50	2·04	2·67		
$2\frac{1}{4}$	·145	·292	·484	·769	1·11	1·56	2·11	2·76		
$2\frac{1}{2}$	·153	·305	·506	·800	1·16	1·62	2·18	2·84	3·57	
$2\frac{3}{4}$	·161	·319	·528	·831	1·20	1·67	2·25	2·93	3·68	
3	·169	·333	·549	·862	1·24	1·73	2·32	3·02	3·78	4·77
$3\frac{1}{4}$	·177	·347	·571	·894	1·28	1·78	2·39	3·11	3·89	4·89
$3\frac{1}{2}$	·185	·361	·593	·925	1·33	1·84	2·46	3·19	3·99	5·02
$3\frac{3}{4}$	·192	·375	·615	·956	1·37	1·89	2·53	3·28	4·10	5·14
4	·200	·389	·637	·988	1·41	1·95	2·60	3·37	4·20	5·27
$4\frac{1}{4}$	·208	·403	·658	1·02	1·45	2·00	2·67	3·45	4·31	5·39
$4\frac{1}{2}$	·216	·417	·680	1·05	1·50	2·06	2·74	3·54	4·41	5·52
$4\frac{3}{4}$	·224	·431	·702	1·08	1·54	2·12	2·81	3·63	4·52	5·64
5	·232	·445	·724	1·11	1·58	2·17	2·88	3·71	4·62	5·77
$5\frac{1}{4}$	·240	·459	·745	1·14	1·62	2·23	2·95	3·80	4·73	5·89
$5\frac{1}{2}$	·247	·472	·767	1·17	1·67	2·28	3·02	3·89	4·83	6·02
$5\frac{3}{4}$	·255	·486	·789	1·21	1·71	2·34	3·09	3·97	4·94	6·14
6	·263	·500	·810	1·24	1·75	2·39	3·16	4·06	5·04	6·27
Weight of Shank in lbs. per 1 inch of Length.										
	·031	·056	·087	·125	·170	·222	·282	·348	·421	·501
Weight in lbs. of one nut.										
	·034	·076	·139	·216	·320	·461	·638	·851	1·07	1·39



POUNDS OF WHITWORTH'S BOLTS AND NUTS

To B.S.S. No. 916—1940

Length (a) in inches	Diameter of Bolt in inches								
	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{1}{2}$
1	·069	·156	·285	·452					
$1\frac{1}{8}$	·073	·163	·296	·478					
$1\frac{1}{4}$	·076	·170	·307	·494	·618				
$1\frac{3}{8}$	·080	·176	·318	·510	·639				
$1\frac{1}{2}$	·084	·183	·329	·526	·660	0·96			
$1\frac{5}{8}$	·088	·190	·340	·542	·681	0·99			
$1\frac{3}{4}$	·091	·196	·351	·558	·702	1·02	1·48		
$1\frac{7}{8}$	·095	·203	·362	·574	·723	1·05	1·52		
2	·099	·210	·373	·590	·743	1·08	1·55	2·07	3·38
$2\frac{1}{4}$	·106	·223	·395	·623	·785	1·13	1·62	2·16	3·50
$2\frac{1}{2}$	·113	·236	·417	·655	·827	1·18	1·69	2·25	3·62
$2\frac{3}{4}$	·121	·249	·439	·687	·868	1·24	1·76	2·34	3·74
3	·128	·263	·461	·719	·910	1·29	1·83	2·42	3·85
$3\frac{1}{4}$	·136	·276	·483	·752	·952	1·35	1·90	2·51	3·97
$3\frac{1}{2}$	·143	·289	·505	·784	·993	1·40	1·97	2·59	4·09
$3\frac{3}{4}$	·150	·303	·527	·816	1·04	1·46	2·04	2·68	4·21
4	·158	·316	·548	·849	1·08	1·51	2·11	2·77	4·32
$4\frac{1}{4}$	·165	·330	·570	·877	1·12	1·56	2·18	2·85	4·44
$4\frac{1}{2}$	·172	·343	·592	·906	1 16	1·62	2·25	2·94	4·56
$4\frac{3}{4}$	·180	·356	·614	·937	1·20	1·68	2·32	3·03	4·68
5	·187	·369	·636	·969	1·24	1·73	2·39	3·11	4·79
$5\frac{1}{4}$	·194	·382	·657	·999	1·29	1·78	2·45	3·19	4·91
$5\frac{1}{2}$	·202	·396	·679	1·03	1·33	1·84	2·52	3·28	5·03
$5\frac{3}{4}$	·209	·409	·701	1·06	1·37	1·89	2·59	3·37	5·15
6	·216	·422	·723	1·10	1·41	1·95	2·66	3·45	5·26
Weight of Shank in lbs. per 1 inch of Length.									
	·031	·056	·087	·125	·170	·222	·282	·348	·501
Weight in lbs. of one nut.									
	·018	·049	·094	·158	·188	·290	·406	·575	·982

WEIGHT OF FLAT STEEL IN LBS. PER LINEAL FOOT

Width in inches	Thickness in Inches								
	1/4	5/16	3/8	7/16	1/2	5/8	3/4	7/8	1
1	·85	1·06	1·28	1·49	1·70	2·13	2·55	2·98	3·40
1¼	1·06	1·33	1·59	1·86	2·13	2·66	3·19	3·72	4·25
1½	1·28	1·59	1·91	2·23	2·55	3·19	3·83	4·46	5·10
1¾	1·49	1·86	2·23	2·60	2·98	3·72	4·46	5·21	5·95
2	1·70	2·13	2·55	2·98	3·40	4·25	5·10	5·95	6·80
2¼	1·91	2·39	2·87	3·35	3·83	4·78	5·74	6·69	7·65
2½	2·13	2·66	3·19	3·72	4·25	5·31	6·38	7·44	8·50
2¾	2·34	2·92	3·51	4·09	4·68	5·84	7·01	8·18	9·35
3	2·55	3·19	3·83	4·46	5·10	6·38	7·65	8·93	10·2
3¼	2·76	3·45	4·14	4·83	5·53	6·91	8·29	9·67	11·0
3½	2·98	3·72	4·46	5·21	5·95	7·44	8·93	10·4	11·9
3¾	3·19	3·98	4·78	5·58	6·38	7·97	9·56	11·2	12·7
4	3·40	4·25	5·10	5·95	6·80	8·50	10·2	11·9	13·6
4¼	3·61	4·52	5·42	6·32	7·23	9·03	10·8	12·6	14·4
4½	3·83	4·78	5·74	6·69	7·65	9·56	11·5	13·4	15·3
4¾	4·04	5·05	6·06	7·07	8·08	10·1	12·1	14·1	16·1

WEIGHT OF FLAT STEEL IN LBS. PER LINEAL FOOT

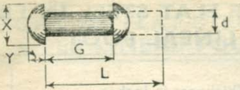
Width in Inches	Thickness in Inches								
	1/4	5/16	3/8	7/16	1/2	5/8	3/4	7/8	1
5	4·25	5·31	6·38	7·44	8·50	10·6	12·7	14·9	17·0
5¼	4·46	5·58	6·69	7·81	8·93	11·2	13·4	15·6	17·8
5½	4·68	5·84	7·01	8·18	9·35	11·7	14·0	16·4	18·7
6	5·10	6·38	7·65	8·93	10·2	12·7	15·3	17·8	20·4
6½	5·53	6·91	8·29	9·67	11·0	13·8	16·6	19·3	22·1
7	5·95	7·44	8·93	10·4	11·9	14·9	17·8	20·8	23·8
7½	6·38	7·97	9·56	11·2	12·7	15·9	19·1	22·3	25·5
8	6·80	8·50	10·2	11·9	13·6	17·0	20·4	23·8	27·2
8½	7·23	9·03	10·8	12·6	14·4	18·1	21·7	25·3	28·9
9	7·65	9·56	11·5	13·4	15·3	19·1	22·9	26·8	30·6
9½	8·08	10·1	12·1	14·1	16·1	20·2	24·2	28·3	32·3
10	8·50	10·6	12·7	14·9	17·0	21·2	25·5	29·7	34·0
10½	8·93	11·2	13·4	15·6	17·8	22·3	26·8	31·2	35·7
11	9·35	11·7	14·0	16·4	18·7	23·4	28·0	32·7	37·4
11½	9·78	12·2	14·7	17·1	19·5	24·4	29·3	34·2	39·1
12	10·2	12·7	15·3	17·8	20·4	25·5	30·6	35·7	40·8

WEIGHT OF FLAT STEEL IN LBS. PER LINEAL FOOT

Width in inches	Thickness in Inches								
	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{3}{8}$	$\frac{7}{16}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1
12½	10·6	13·3	15·9	18·6	21·2	26·6	31·9	37·2	42·5
13	11·0	13·8	16·6	19·3	22·1	27·6	33·1	38·7	44·2
13½	11·5	14·3	17·2	20·1	22·9	28·7	34·4	40·2	45·9
14	11·9	14·9	17·8	20·8	23·8	29·7	35·7	41·6	47·6
14½	12·3	15·4	18·5	21·6	24·6	30·8	37·0	43·1	49·3
15	12·7	15·9	19·1	22·3	25·5	31·9	38·2	44·6	51·0
15½	13·2	16·5	19·8	23·0	26·3	32·9	39·5	46·1	52·7
16	13·6	17·0	20·4	23·8	27·2	34·0	40·8	47·6	54·4
16½	14·0	17·5	21·0	24·5	28·0	35·1	42·1	49·1	56·1
17	14·4	18·1	21·7	25·3	28·9	36·1	43·3	50·6	57·8
17½	14·9	18·6	22·3	26·0	29·7	37·2	44·6	52·1	59·5
18	15·3	19·1	22·9	26·8	30·6	38·2	45·9	53·5	61·2
18½	15·7	19·7	23·6	27·5	31·4	39·3	47·2	55·0	62·9
19	16·1	20·2	24·2	28·3	32·3	40·4	48·4	56·5	64·6
19½	16·6	20·7	24·9	29·0	33·1	41·4	49·7	58·0	66·3
20	17·0	21·2	25·5	29·7	34·0	42·5	51·0	59·5	68·0

WEIGHT OF FLAT STEEL IN LBS. PER LINEAL FOOT

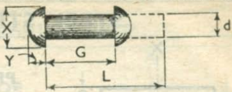
Width in Inches	Thickness in Inches								
	1/4	5/16	3/8	7/16	1/2	5/8	3/4	7/8	1
20½	17·4	21·8	26·1	30·5	34·8	43·6	52·3	60·1	69·7
21	17·8	22·3	26·8	31·2	35·7	44·6	53·5	62·5	71·4
21½	18·3	22·8	27·4	32·0	36·5	45·7	54·8	64·0	73·1
22	18·7	23·4	28·0	32·7	37·4	46·7	56·1	65·4	74·8
22½	19·1	23·9	28·7	33·5	38·2	47·8	57·4	66·9	76·5
23	19·5	24·4	29·3	34·2	39·1	48·9	58·6	68·4	78·2
23½	20·0	25·0	30·0	34·9	39·9	49·9	59·9	69·9	79·9
24	20·4	25·5	30·6	35·7	40·8	51·0	61·2	71·4	81·6
24½	20·8	26·0	31·2	36·4	41·6	52·1	62·5	72·9	83·3
25	21·2	26·6	31·9	37·2	42·5	53·1	63·7	74·4	85·0
25½	21·7	27·1	32·5	37·9	43·3	54·2	65·0	75·9	86·7
26	22·1	27·6	33·1	38·7	44·2	55·2	66·3	77·3	88·4
26½	22·5	28·2	33·8	39·4	45·0	56·3	67·6	78·8	90·1
27	22·9	28·7	34·4	40·2	45·9	57·4	68·8	80·3	91·8
27½	23·4	29·2	35·1	40·9	46·7	58·4	70·1	81·8	93·5
28	23·8	29·7	35·7	41·6	47·6	59·5	71·4	83·3	95·2



SNAP HEAD RIVETS

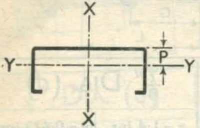
$\frac{1}{2}''$ DIA. (d)			$\frac{5}{8}''$ DIA (d)		
$x = 0{\cdot}8$ ins.	$y = 0{\cdot}35$ ins.		$x = 1{\cdot}0$ ins.	$y = 0{\cdot}438$ ins.	
Approx. Wt. in Lbs. of 100 Heads $= 3{\cdot}4$			Approx. Wt. in Lbs. of 100 Heads $= 6{\cdot}7$		
Length L ins.	Grip G ins.	Wt. per 100 Rivs.	Length L ins.	Grip G ins.	Wt. per 100 Rivs.
1	$\frac{1}{8}$	9·0	$1\frac{1}{2}$	$\frac{3}{8}$	19·7
$1\frac{1}{8}$	$\frac{3}{16}, \frac{1}{4}$	9·7	$1\frac{5}{8}$	$\frac{1}{2}, \frac{9}{16}$	20·8
$1\frac{1}{4}$	$\frac{5}{16}$	10·4	$1\frac{3}{4}$	$\frac{5}{8}, \frac{11}{16}$	21·9
$1\frac{3}{8}$	$\frac{3}{8}, \frac{7}{16}$	11·1	$1\frac{7}{8}$	$\frac{3}{4}$	23·0
$1\frac{1}{2}$	$\frac{1}{2}$	11·8	2	$\frac{13}{16}, \frac{7}{8}$	24·1
$1\frac{5}{8}$	$\frac{9}{16}$	12·4	$2\frac{1}{8}$	$\frac{15}{16}$	25·2
$1\frac{3}{4}$	$\frac{5}{8}, \frac{11}{16}$	13·1	$2\frac{1}{4}$	$1, 1\frac{1}{16}$	26·3
$1\frac{7}{8}$	$\frac{3}{4}, \frac{13}{16}$	13·8	$2\frac{3}{8}$	$1\frac{1}{8}$	27·3
2	$\frac{7}{8}, \frac{15}{16}$	14·5	$2\frac{1}{2}$	$1\frac{3}{16}, 1\frac{1}{4}$	28·4
$2\frac{1}{8}$	$1, 1\frac{1}{16}$	15·2	$2\frac{5}{8}$	$1\frac{5}{16}, 1\frac{3}{8}$	29·5
$2\frac{1}{4}$	$1\frac{1}{8}, 1\frac{3}{16}$	15·9	$2\frac{3}{4}$	$1\frac{7}{16}$	30·6
$2\frac{3}{8}$	$1\frac{1}{4}$	16·6	$2\frac{7}{8}$	$1\frac{1}{2}, 1\frac{9}{16}$	31·7
$2\frac{1}{2}$	$1\frac{5}{16}, 1\frac{3}{8}$	17·3	3	$1\frac{5}{8}, 1\frac{11}{16}$	32·8
$2\frac{5}{8}$	$1\frac{7}{16}, 1\frac{1}{2}$	18·0	$3\frac{1}{8}$	$1\frac{3}{4}$	33·9
$2\frac{3}{4}$	$1\frac{9}{16}$	18·7	$3\frac{1}{4}$	$1\frac{13}{16}, 1\frac{7}{8}$	35·0
$2\frac{7}{8}$	$1\frac{5}{8}, 1\frac{11}{16}$	19·4	$3\frac{3}{8}$	$1\frac{15}{16}$	36·0
3	$1\frac{3}{4}$	20·1	$3\frac{1}{2}$	$2, 2\frac{1}{16}, 2\frac{1}{8}$	37·1
$3\frac{1}{8}$	$1\frac{13}{16}, 1\frac{7}{8}$	20·8	$3\frac{5}{8}$	$2\frac{1}{4}, 2\frac{3}{16}$	38·2
$3\frac{1}{4}$	$1\frac{15}{16}, 2$	21·5	$3\frac{3}{4}$	$2\frac{5}{16}$	39·3
$3\frac{3}{8}$	$2\frac{1}{16}$	22·2	$3\frac{7}{8}$	$2\frac{3}{8}, 2\frac{7}{16}$	40·4
$3\frac{1}{2}$	$2\frac{1}{8}, 2\frac{3}{16}$	22·8	4	$2\frac{1}{2}, 2\frac{9}{16}$	41·5
$3\frac{5}{8}$	$2\frac{1}{4}$	23·6	$4\frac{1}{4}$	$2\frac{11}{16}, 2\frac{3}{4}$	43·6
$3\frac{3}{4}$	$2\frac{5}{16}, 2\frac{3}{8}$	24·3	$4\frac{1}{2}$	$2\frac{7}{8}, 2\frac{15}{16}$	45·8
$3\frac{7}{8}$	$2\frac{7}{16}, 2\frac{1}{2}$	25·0	$4\frac{3}{4}$	$3\frac{1}{8}, 3\frac{3}{16}$	48·0

SNAP HEAD RIVETS



$\frac{3}{4}''$ DIA. (d)			$\frac{7}{8}''$ DIA. (d)		
$x = 1{\cdot}2$ ins.	$y = 0{\cdot}525$ ins.		$x = 1{\cdot}4$ ins.	$y = 0{\cdot}613$ ins.	
Approx. Wt. in Lbs. of 100 Heads $= 11{\cdot}6$			Approx. Wt. in Lbs. of 100 Heads $= 18{\cdot}4$		
Length L ins.	Grip G ins.	Wt. per 100 Rivs.	Length L ins.	Grip G ins.	Wt. per 100 Rivs.
$1\frac{1}{2}$	$\frac{1}{4}$	30·4	2	$\frac{9}{16}, \frac{5}{8}$	52·5
$1\frac{5}{8}$	$\frac{5}{16}, \frac{3}{8}$	31·9	$2\frac{1}{8}$	$\frac{11}{16}$	54·6
$1\frac{3}{4}$	$\frac{1}{2}$	33·5	$2\frac{1}{4}$	$\frac{3}{4}, \frac{13}{16}$	56·7
$1\frac{7}{8}$	$\frac{9}{16}, \frac{5}{8}$	35·1	$2\frac{3}{8}$	$\frac{7}{8}, \frac{15}{16}$	58·9
2	$\frac{11}{16}$	36·6	$2\frac{1}{2}$	$1, 1\frac{1}{16}$	61·0
$2\frac{1}{8}$	$\frac{3}{4}, \frac{13}{16}$	38·2	$2\frac{5}{8}$	$1\frac{1}{8}$	63·1
$2\frac{1}{4}$	$\frac{7}{8}, \frac{15}{16}$	39·8	$2\frac{3}{4}$	$1\frac{3}{16}, 1\frac{1}{4}$	65·2
$2\frac{3}{8}$	$1, 1\frac{1}{16}$	41·3	$2\frac{7}{8}$	$1\frac{5}{16}, 1\frac{3}{8}$	67·4
$2\frac{1}{2}$	$1\frac{1}{8}, 1\frac{3}{16}$	42·9	3	$1\frac{7}{16}, 1\frac{1}{2}$	69·5
$2\frac{5}{8}$	$1\frac{1}{4}$	44·5	$3\frac{1}{8}$	$1\frac{9}{16}, 1\frac{5}{8}$	71·6
$2\frac{3}{4}$	$1\frac{5}{16}, 1\frac{3}{8}$	46·0	$3\frac{1}{4}$	$1\frac{11}{16}$	73·8
$2\frac{7}{8}$	$1\frac{7}{16}, 1\frac{1}{2}$	47·6	$3\frac{3}{8}$	$1\frac{3}{4}, 1\frac{13}{16}$	75·9
3	$1\frac{9}{16}$	49·2	$3\frac{1}{2}$	$1\frac{7}{8}, 1\frac{15}{16}$	78·0
$3\frac{1}{8}$	$1\frac{5}{8}, 1\frac{11}{16}$	50·7	$3\frac{5}{8}$	$2, 2\frac{1}{16}$	80·1
$3\frac{1}{4}$	$1\frac{3}{4}, 1\frac{13}{16}$	52·3	$3\frac{3}{4}$	$2\frac{1}{8}, 2\frac{3}{16}$	82·3
$3\frac{3}{8}$	$1\frac{7}{8}, 1\frac{15}{16}$	53·8	$3\frac{7}{8}$	$2\frac{1}{4}$	84·4
$3\frac{1}{2}$	$2, 2\frac{1}{16}$	55·4	4	$2\frac{5}{16}, 2\frac{3}{8}$	86·5
$3\frac{5}{8}$	$2\frac{1}{8}$	57·0	$4\frac{1}{4}$	$2\frac{9}{16}$	90·8
$3\frac{3}{4}$	$2\frac{3}{16}, 2\frac{1}{4}$	58·5	$4\frac{1}{2}$	$2\frac{3}{4}, 2\frac{13}{16}$	95·1
$3\frac{7}{8}$	$2\frac{5}{16}, 2\frac{3}{8}$	60·1	$4\frac{3}{4}$	3	99·0
4	$2\frac{7}{16}$	61·7	5	$3\frac{3}{16}, 3\frac{1}{4}$	104
$4\frac{1}{4}$	$2\frac{5}{8}, 2\frac{11}{16}$	64·8	$5\frac{1}{4}$	$3\frac{3}{8}, 3\frac{7}{16}$	108
$4\frac{1}{2}$	$2\frac{13}{16}, 2\frac{7}{8}$	67·9	$5\frac{1}{2}$	$3\frac{5}{8}, 3\frac{11}{16}$	112
$4\frac{3}{4}$	$3\frac{1}{16}$	71·1	$5\frac{3}{4}$	$3\frac{7}{8}$	116

STEEL STRIP

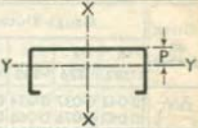


PROPERTIES OF COLD ROLLED CHANNEL SECTION

Gauge	Section in Inches	Wt/ft.	Area	I x–x
10 S.W.G.	$3 \times 1\frac{1}{2} \times \frac{1}{2}$ heel	2·35	0·75	0·996
	$3\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{2}$,,	2·56	0·812	1·444
	$4 \times 1\frac{1}{2} \times \frac{1}{2}$,,	3·00	0·937	2·227
	$4\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{2}$,,	3·20	0·999	2·952
	$5 \times 1\frac{1}{2} \times \frac{1}{2}$,,	3·41	1·064	3·787
	$5\frac{1}{2} \times 2 \times \frac{1}{2}$,,	4·10	1·249	5·685
	$6 \times 2 \times \frac{1}{2}$,,	4·45	1·310	6·973
	$7 \times 2 \times \frac{1}{2}$,,	4·89	1·438	10·058
	$8 \times 2 \times \frac{1}{2}$,,	5·31	1·563	13·869
12 S.W.G.	$2\frac{1}{2} \times 1 \times \frac{3}{8}$ heel	1·58	0·503	0·46
	$3 \times 1\frac{1}{4} \times \frac{3}{8}$,,	1·93	0·607	0·823
	$3\frac{1}{2} \times 1\frac{1}{4} \times \frac{3}{8}$,,	2·11	0·659	1·186
	$4 \times 1\frac{1}{2} \times \frac{3}{8}$,,	2·46	0·763	1·830
	$4\frac{1}{2} \times 1\frac{1}{2} \times \frac{3}{8}$,,	2·64	0·815	2·419
	$5\frac{1}{2} \times 2 \times \frac{3}{8}$,,	3·36	1·023	4·667
	$6 \times 2 \times \frac{3}{8}$,,	3·51	1·075	5·727
14 S.W.G.	$2\frac{1}{2} \times 1 \times \frac{1}{4}$ heel	1·19	0·374	0·352
	$3 \times 1\frac{1}{4} \times \frac{1}{4}$,,	1·47	0·454	0·628
	$3\frac{1}{2} \times 1\frac{1}{4} \times \frac{1}{4}$,,	1·61	0·494	0·904
	$4 \times 1\frac{1}{2} \times \frac{1}{4}$,,	1·88	0·574	1·393
	$4\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$,,	2·02	0·614	1·838
	$5\frac{1}{2} \times 2 \times \frac{1}{4}$,,	2·56	0·774	3·551
	$6 \times 2 \times \frac{1}{4}$,,	2·68	0·814	4·354
16 S.W.G.	$2 \times 1 \times \frac{1}{4}$ heel	0·88	0·271	0·173
	$2\frac{1}{2} \times 1 \times \frac{1}{4}$,,	0·99	0·301	0·294
	$3 \times 1\frac{1}{4} \times \frac{1}{4}$,,	1·21	0·368	0·513
	$3\frac{1}{2} \times 1\frac{1}{4} \times \frac{1}{4}$,,	1·31	0·400	0·736

STEEL STRIP

PROPERTIES OF COLD ROLLED CHANNEL SECTION



I y-y	Z x-x	Z y-y	R x-x	R y-y	P	
0·154	0·664	0·189	1·15	0·453	0·438	10 S.W.G.
0·161	0·825	0·191	1·33	0·446	0·408	
0·265	1·114	0·258	1·54	0·532	0·475	
0·275	1·312	0·262	1·72	0·525	0·449	
0·289	1·683	0·270	1·89	0·521	0·429	
0·598	2·067	0·420	2·133	0·692	0·576	
0·6194	2·324	0·428	2·304	0·687	0·554	
0·647	2·874	0·434	2·644	0·671	0·511	
0·6736	3·467	0·441	2·98	0·656	0·474	
0·064	0·368	0·097	0·96	0·357	0·338	12 S.W.G.
0·118	0·549	0·139	1·17	0·432	0·402	
0·121	0·678	0·138	1·34	0·429	0·373	
0·206	0·915	0·195	1·55	0·52	0·441	
0·214	1·075	0·198	1·72	0·514	0·417	
0·471	1·697	0·323	2·136	0·678	0·603	
0·483	1·909	0·326	2·3081	0·670	0·519	
0·045	0·282	0·064	0·972	0·347	0·302	14 S.W.G.
0·082	0·419	0·093	1·18	0·426	0·365	
0·084	0·517	0·092	1·35	0·414	0·337	
0·142	0·697	0·129	1·56	0·498	0·401	
0·148	0·817	0·132	1·73	0·492	0·378	
0·334	1·291	0·223	2·142	0·657	0·503	
0·341	1·451	0·224	2·313	0·647	0·48	
0·034	0·173	0·051	0·79	0·354	0·332	16 S.W.G.
0·036	0·234	0·051	0·99	0·346	0·296	
0·069	0·342	0·078	1·18	0·434	0·367	
0·073	0·42	0·08	1·36	0·426	0·34	

WEIGHT OF STEEL STRIP

Girth in inches	Gauge Thickness of Strip in inches							
	6	7	8	9	10	11	12	13
	·192	·176	·160	·144	·128	·116	·104	·092
$\frac{1}{16}$	0·041	0·037	0·034	0·031	0·027	0·025	0·022	0·020
$\frac{1}{8}$	0·082	0·075	0·068	0·061	0·054	0·049	0·044	0·039
$\frac{3}{16}$	0·122	0·112	0·102	0·092	0·082	0·074	0·066	0·059
$\frac{1}{4}$	0·163	0·150	0·136	0·122	0·109	0·098	0·088	0·078
$\frac{5}{16}$	0·204	0·187	0·170	0·153	0·136	0·123	0·111	0·098
$\frac{3}{8}$	0·245	0·224	0·204	0·184	0·163	0·148	0·133	0·117
$\frac{7}{16}$	0·286	0·262	0·238	0·214	0·190	0·172	0·155	0·137
$\frac{1}{2}$	0·326	0·299	0·272	0·245	0·218	0·197	0·177	0·156
$\frac{9}{16}$	0·367	0·337	0·306	0·275	0·245	0·221	0·199	0·176
$\frac{5}{8}$	0·408	0·374	0·340	0·306	0·272	0·246	0·221	0·195
$\frac{11}{16}$	0·449	0·411	0·374	0·337	0·299	0·271	0·243	0·214
$\frac{3}{4}$	0·490	0·449	0·408	0·367	0·326	0·295	0·265	0·234
$\frac{13}{16}$	0·530	0·486	0·442	0·398	0·354	0·320	0·287	0·254
$\frac{7}{8}$	0·571	0·524	0·476	0·428	0·381	0·344	0·309	0·273
$\frac{15}{16}$	0·612	0·561	0·510	0·459	0·408	0·370	0·331	0·293
1	0·653	0·598	0·544	0·490	0·435	0·394	0·354	0·313
2	1·306	1·197	1·088	0·979	0·870	0·789	0·707	0·626
3	1·958	1·795	1·632	1·469	1·306	1·183	1·061	0·938
4	2·611	2·394	2·176	1·958	1·741	1·578	1·414	1·251
5	3·264	2·992	2·720	2·448	2·176	1·972	1·768	1·564
6	3·917	3·590	3·264	2·938	2·611	2·366	2·122	1·877
7	4·570	4·189	3·808	3·427	3·046	2·761	2·475	2·190
8	5·222	4·787	4·352	3·917	3·482	3·155	2·829	2·502
9	5·875	5·386	4·896	4·406	3·917	3·550	3·182	2·815
10	6·528	5·984	5·440	4·896	4·352	3·944	3·536	3·128
11	7·181	6·582	5·984	5·386	4·787	4·338	3·890	3·441
12	7·834	7·181	6·528	5·875	5·222	4·733	4·243	3·754
13	8·487	7·779	7·072	6·365	5·658	5·127	4·597	4·066
14	9·139	8·378	7·616	6·854	6·093	5·522	4·950	4·379
15	9·792	8·976	8·160	7·344	6·528	5·916	5·304	4·692

IN LBS. PER LINEAL FOOT

Gauge Thickness of Strip in inches								Girth in inches
14	15	16	17	18	19	20	21	
·080	·072	·064	·056	·048	·040	·036	·032	
0·017	0·015	0·014	0·012	0·010	0·008	0·007	0·007	$\frac{1}{16}$
0·034	0·031	0·027	0·024	0·020	0·017	0·015	0·014	$\frac{1}{8}$
0·051	0·046	0·041	0·036	0·031	0·026	0·023	0·020	$\frac{3}{16}$
0·068	0·061	0·054	0·048	0·041	0·034	0·030	0·027	$\frac{1}{4}$
0·085	0·077	0·068	0·060	0·051	0·043	0·038	0·034	$\frac{5}{16}$
0·102	0·092	0·082	0·071	0·061	0·051	0·046	0·041	$\frac{3}{8}$
0·119	0·107	0·095	0·083	0·071	0·060	0·053	0·048	$\frac{7}{16}$
0·136	0·122	0·109	0·095	0·081	0·068	0·061	0·054	$\frac{1}{2}$
0·153	0·138	0·122	0·107	0·092	0·077	0·068	0·061	$\frac{9}{16}$
0·170	0·153	0·136	0·119	0·102	0·085	0·076	0·068	$\frac{5}{8}$
0·187	0·168	0·150	0·131	0·112	0·094	0·084	0·075	$\frac{11}{16}$
0·204	0·184	0·163	0·143	0·122	0·102	0·091	0·082	$\frac{3}{4}$
0·221	0·199	0·177	0·155	0·133	0·111	0·099	0·088	$\frac{13}{16}$
0·238	0·214	0·190	0·167	0·143	0·119	0·106	0·095	$\frac{7}{8}$
0·255	0·230	0·204	0·179	0·153	0·128	0·114	0·102	$\frac{15}{16}$
0·272	0·245	0·218	0·190	0·163	0·136	0·122	0·109	1
0·544	0·490	0·435	0·381	0·326	0·272	0·245	0·218	2
0·816	0·734	0·653	0·571	0·490	0·408	0·367	0·326	3
1·088	0·979	0·870	0·762	0·653	0·544	0·490	0·435	4
1·360	1·224	1·088	0·952	0·816	0·680	0·612	0·544	5
1·632	1·469	1·306	1·142	0·979	0·816	0·734	0·653	6
1·904	1·714	1·523	1·333	1·142	0·952	0·857	0·762	7
2·176	1·958	1·741	1·523	1·306	1·088	0·979	0·870	8
2·448	2·203	1·958	1·714	1·469	1·224	1·102	0·979	9
2·720	2·448	2·176	1·904	1·632	1·360	1·224	1·088	10
2·992	2·693	2·394	2·094	1·795	1·496	1·346	1·197	11
3·264	2·938	2·611	2·285	1·958	1·632	1·469	1·306	12
3·536	3·182	2·829	2·475	2·122	1·768	1·591	1·414	13
3·808	3·428	3·046	2·666	2·285	1·904	1·714	1·523	14
4·080	3·672	3·264	2·856	2·448	2·040	1·836	1·632	15

FOAM SLAG

Foam Slag is a light-weight, cellular, inert material usable as a concrete aggregate for Cast ' in situ ' work or in Precast form, or loose for insulating purposes.

The raw material, which is a specific molten blast furnace slag, is treated with applications of water by a patent apparatus. During this process the molten slag is "foamed" or inflated to approximately seven to ten times its original volume. The material after cooling is crushed and graded for aggregate.

PRECAST BLOCKS

Standard Size of Block = 18" × 9"

Weight per cubic foot = 70 lbs.

Thickness - -	2	2½	3	4
Weight per Block -	12	15	18	24

FOAM SLAG CONCRETE

Typical Weights and corresponding Crushing Strength (at 3 Months)

Lbs./cu. ft.	Mixture Cement :	Foam Slag: ⅛" to dust :	Foam Slag aggregate ⅛" to ⅜"	Crushing Strength Lbs./sq. in.
65	1 :	6 :	12	200
70	1 :	4 :	8	300
75	1 :	4 :	7	400
85	1 :	3 :	6	600
90	1 :	2 :	4	1,540
110	1 :	1½ :	3	3,770
115	1 :	1 :	2	4,900

WELDING DATA

FILLET AND BUTT WELDS

There are two principal types of weld, i.e. *Fillet* and *Butt*, and few occasions arise when one or other of the established forms of these welds cannot be used. The standard requirements are as follows:

The size of a Fillet Weld is specified by the **minimum leg length.**

The **throat** thickness from which the weld strength is calculated and which is a most important figure in design is, for right-angled fillet welds, in all cases taken as 0·7 × **minimum leg length.**

Figs. 1 and 2, page 491, explain these definitions and it will be noticed that the all-important triangle, which gives the leg length and throat thickness of a fillet weld, is the maximum isosceles right-angled triangle, which may be inscribed within the weld cross-section. Where fillet welds of unequal leg lengths are used, the same principles apply, and the strength of such welds is controlled by the shorter leg length.

The effective length of a fillet weld for the purpose of strength calculations is equal to the overall length of the weld minus twice the weld size. This allows for the craters which usually occur at the beginning and end of a weld run where the full throat value of the rest of the weld might not be obtained. The effective length should not be less than 2" nor less than six times the specified weld size.

The table on page 498 shows the strengths of standard fillet welds.

The majority of fillet welds are made between fusion faces forming an angle of 90° as referred to before, but where this is not so, and if the fillet weld is required to transmit load, the angles shown in Figures 3 and 4, page 491, should be regarded as maximum and minimum.

In such welds the inscribed triangles, determining leg length and throat thickness, are not right-angled, and the ratio of throat thickness to leg length is not the standard 0·7. The ratio varies from 0·57 in the case of the maximum obtuse angle (110°) to 0·87 for the minimum acute angle (60°). The actual strengths of the welds differ therefore from those given in the table on pages 498/499, which incorporates only the 0·7 throat value of the right-angled fillet weld.

In practice, however, it is usual to ignore the increase in throat thickness and strength obtained in *acute-angled* welds and to use the strength as in the table. This compensates, in part, for the difficulties in ensuring sound fusion at the root of the weld as the angle between the plates is decreased.

For *obtuse-angled* welds it is desirable that the values in the table should be reduced in the ratio of the lower throat thickness to the standard 0·7, i.e. for $110^\circ - \frac{0{\cdot}57}{0{\cdot}7} \times$ strength in table.

Regarding **Butt Welds,** the size here is the throat thickness. The throat thickness is taken as the thickness of the thinner part joined. When both plates are equal, the throat thickness is therefore equal to the plate thickness.

Whilst the strengthening effect of the reinforcement metal, as shown in Figure 5, page 492, is ignored, this additional metal should be provided and the amount should not be less than 10% of the weld size, or $\frac{1}{8}$ in., whichever is the lower value.

On page 493 are the more important forms of Butt Welds. Butt welds are normally made throughout the full length of the joint, and intermittent butt welds are not permitted where the joint is carrying load.

The permissible stresses in butt welds are varied from the standard figures given in pages 500/501 according to the type of weld. The principal points to observe are these :

Single V, U, J or bevel butt welds should be sealed wherever practicable by depositing a run of weld metal on the back of the Joint, as in Figure 5, page 492. Where this is not done the maximum permissible stress is reduced to 50% of the corresponding figure given in the table. If, however, another steel part is in contact with the back of the joints and a suitable gap is provided at the root so that deposited metal may penetrate into such backing part, ensuring sound fusion, the full working stress as in the table may be used.

In all J or bevel butt welds either, single or double, 75% only of the stresses given in the table are permitted.

All stresses are calculated on the throat thickness and reinforcement metal is ignored. If the reinforcement is ground flush, these stresses still apply.

Extracted from Welding Memorandum II.
Advisory Service on Welding,
Ministry of Supply,
London, N.W.I.

FILLET WELDS

Fig. 1

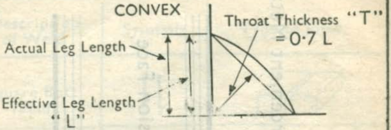


Fig. 2

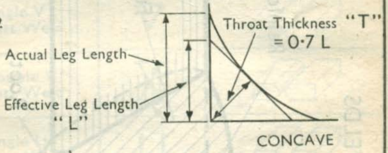


Fig. 3

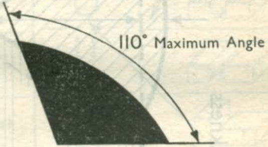
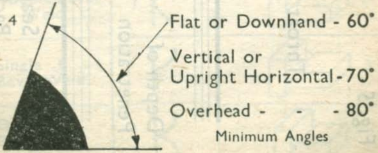


Fig. 4



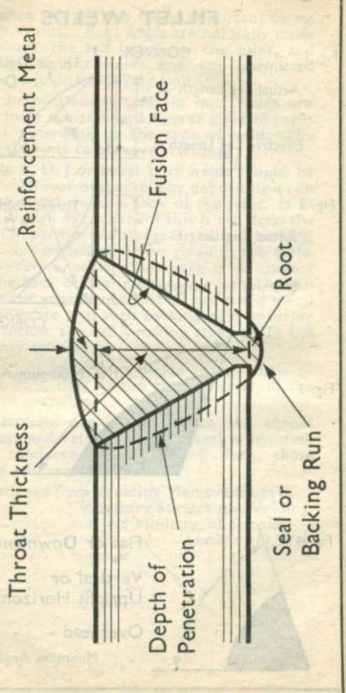


Fig. 5 BUTT WELDS

BUTT WELD SYMBOLS

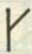
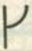
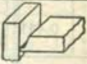
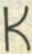
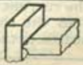
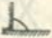
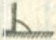
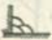
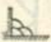
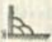
Description of Weld	Symbols	Type of Joint
Square Butt Weld		
Single V Butt Weld		
Double V Butt Weld		
Single U Butt Weld		
Double U Butt Weld		
Single Bevel Weld		
Double Bevel Weld		
Single J Bevel Weld		
Double J Bevel Weld		

CHART FOR
(conforming to

Welding Procedure for Downhand

Leg Length in Ins.	Throat Thickness in Ins.	POSITIONS OF RUNS FOR ALTERNATIVE PROCEDURES			
		10 S.W.G.	8 S.W.G.	6 S.W.G.	4 S.W.G.
$\frac{3}{16}$	0·133				
$\frac{1}{4}$	0·176				
$\frac{5}{16}$	0·221				
$\frac{3}{8}$	0·265				
$\frac{1}{2}$	0·354				
$\frac{5}{8}$	0·441				

* The current values may need to be varied according to the thickness of plate, but such variation is not to exceed the limits specified in Part IV.

QUASI-ARC ELECTRODES
B.S.S. 639—class A)

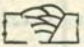
Vertical FILLET Welds

10 S.W.G.		8 S.W.G.		6 S.W.G.		4 S.W.G.	
No. of Runs. Length of Rod Deposited	Current* in Amps.	No. of Runs. Length of Rod Deposited	Current* in Amps.	No. of Runs. Length of Rod Deposited	Current* in Amps.	No. of Runs. Length of Rod Deposited	Current* in Amps.
1—9″	115	1—14″	160				
1—6″	115	1—9″	160	1—12″	200	1—17″	250
3—10″	115	3—15″	160	1—10″	200	1—12″	250
3—8″	115	3—10″	160	3—16″	200	1—10″	250
4—7″	115	3—8″	160	3—12″	200	3—14″	250
6—7″	115	6—10″	160	4—10″	200	3—11″	250

The procedures to the right of heavy line above should be used wherever possible, to increase the speed of welding and decrease welding costs.

CHART FOR
(Conforming to

Welding Procedure for Downhand

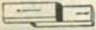
Thickness of Plate in ins.	10 S.W.G.	8 S.W.G.	6 S.W.G.	4 S.W.G.
	POSITIONS OF RUNS FOR ALTERNATIVE PROCEDURES Note—Sealing run should always be allowed as well as welds shewn			
$\frac{1}{4}''$				
$\frac{5}{16}''$				
$\frac{3}{8}''$				
$\frac{1}{2}''$				
$\frac{5}{8}''$				
$\frac{3}{4}''$				
$\frac{7}{8}''$				

QUASI-ARC ELECTRODES
B.S.S. 639—Class A)

Single V BUTT Welds

10 S.W.G.		8 S.W.G.		6 S.W.G.		4 S.W.G.	
No. of Runs. Length of Rod Deposited	Current * in Amps.	No. of Runs. Length of Rod Deposited	Current * in Amps.	No. of Runs. Length of Rod Deposited	Current * in Amps.	No. of Runs. Length of Rod Deposited	Current * in Amps.
2—6″	115	2—10″	155	1—15″	200	1—18″	245
3—9″	115	2—9″	155	2—14″	200	1—15″	245
3—6″	115	3—8″	155	2—12″	200	1—12″	245
5—9″	115	4—8″	155	3—11″	200	3—13″	245
		6—10″	155	4—10″	200	3—9″	245
		9—8″	155	5—8″	200	5—10″	245
				9—8″	200	6—9″	245

Strength of FILLET Welds For Downhand Welding

 End Weld  Side Weld		Working Loads in Tons per lineal inch					
		Ordinary Requirements		B.S.S. 538 1940		L.C.C. Requirements	
Size of Leg "L" in ins.	Throat Thickness in ins.	E	S	E	S	E	S
		8·0 tons/sq. in.	6 tons/sq.in.	7 tons/sq.in.	5 tons/sq.in.	6 tons/sq.in.	5 tons/sq.in.
$\frac{1}{8}$	·088	0·71	0·53	0·62	0·44	0·53	0·44
$\frac{3}{16}$	·131	1·05	0·79	0·92	0·66	0·79	0·66
$\frac{1}{4}$	·175	1·40	1·05	1·23	0·88	1·05	0·88
$\frac{5}{16}$	·219	1·75	1·31	1·53	1·10	1·31	1·10
$\frac{3}{8}$	·263	2·10	1·58	1·84	1·32	1·58	1·32
$\frac{7}{16}$	·306	2·45	1·84	2·14	1·53	1·84	1·53
$\frac{1}{2}$	·350	2·80	2·10	2·45	1·75	2·10	1·75
$\frac{9}{16}$	·394	3·15	2·36	2·76	1·97	2·36	1·97
$\frac{5}{8}$	·438	3·50	2·63	3·07	2·19	2·63	2·20
$\frac{11}{16}$	·481	3·85	2·89	3·37	2·40	2·89	2·40

Note the minimum effective length of a fillet weld should not be less than two inches nor less than six times the size of the weld.

Strength of FILLET Welds For Downhand Welding

End Weld

Side Weld

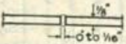
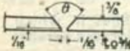
Size of Leg "L" in ins.	Throat thickness in ins.	Working Loads in Tons per lineal inch: Ordinary Requirements		B.S.S. 538 1940		L.C.C. Requirements	
		E	S	E	S	E	S
		8·0 tons/sq.in.	6 tons/sq.in.	7 tons/sq.in.	5 tons/sq.in.	6 tons/sq.in.	5 tons/sq.in.
$\frac{3}{4}$	·525	4·20	3·15	3·68	2·63	3·15	2·63
$\frac{13}{16}$	·569	4·55	3·42	3·98	2·85	3.42	2·85
$\frac{7}{8}$	·613	4·91	3·68	4·29	3·07	3·68	3·07
$\frac{15}{16}$	·656	5·25	3·94	4·59	3·28	3·94	3·28
1	·700	5·60	4·20	4·90	3·50	4.20	3.50

E = End or tension fillet in which the line of weld is located across the lines of stress.

S = Side or shear fillet in which the line of weld is located parallel to the lines of stress.

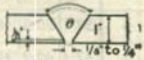
The effective length of a fillet weld shall be the overall length minus twice the weld size, or alternatively 1 in. return must be provided.

Strength of BUTT WELDS

Plate Thickness in inches	ANGLE θ: 60° Flat or Downhand; 70° Vertical and Upright; 80° Overhead — Type of Weld	Working Load in Tons per lineal inch		
		Compression and Tension 8 tons/sq. in.	Shear 6 tons/sq. in.	Shear 5 tons/sq. in.
$\frac{1}{8}''$		1·00	0·75	0·63
$\frac{3}{16}''$		1·50	1·13	0·94
$\frac{1}{4}''$		2·00	1·50	1·25
$\frac{5}{16}''$		2·50	1·88	1·56
$\frac{3}{8}''$		3·00	2·25	1·88

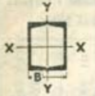
Compression and tension stresses at 8 tons per sq. inch accordance with the B.S. Specification 538, 1940, and per sq. inch refer to the L.C.C. requirements for must have a sealing run on the back of the V; where to not more than one half the corresponding stresses

Strength of BUTT WELDS

Plate Thickness in inches	ANGLE θ: 60° Flat or Downhand; 70° Vertical and Upright; 80° Overhead — Type of Weld	Working Loads in Tons per lineal inch		
		Compression and Tension 8 tons/sq. in.	Shear 6 tons/sq. in.	Shear 5 tons/sq in.
$\frac{1}{2}$"		4·00	3·00	2·50
$\frac{5}{8}$"		5·00	3·75	3·13
$\frac{3}{4}$"		6·00	4·50	3·75
$\frac{7}{8}$"		7·00	5·25	4·38
1"		8·00	6·00	5·00

and shear stresses at 5 tons per sq. inch are in the L.C.C. requirements. Shear stresses at 6 tons Webs of Plate Girders and Joists. All Butt welds this is not practicable the weld should be stressed indicated above.

COMPOUND STANCHIONS—
Safe Concentric Loads in Tons

 Two Channels	Weight per foot in lbs.	Area in sq. ins.	Breadth B in ins.	EFFECTIVE			
				4	6	8	10
12×3½×26·37	52·74	15·52	7	113	109	106	102
10×3½×24·46	48·92	14·38	7	104	101	97·5	93·6
10×3 ×19·28	38·56	11·34	6	81·4	78·5	75·3	71·5
9×3½×22·27	44·54	13·10	7	94·6	92·0	88·7	85·0
9×3 ×17·46	34·92	10·28	6	73·8	71·1	68·0	64·5
8×3½×20·21	40·42	11·88	7	85·9	83·1	80·3	76·8
8×3 ×15·96	31·92	9·38	6	67·1	64·8	62·0	58·5
7×3½×18·28	36·56	10·76	7	77·6	75·0	72·5	69·4
7×3 ×14·22	28·44	8·36	6	59·7	57·6	55·0	52·0
6×3½×16·48	32·96	9·70	7	69·6	67·3	64·6	61·4
6×3 ×16·51	33·02	9·71	6	69·4	67·0	63·7	60·0
6×3 ×12·41	24·82	7·30	6	52·1	50·3	48·0	45·2
5×2½×10·22	20·44	6·01	5	42·4	40·2	37·6	34·3
4×2 × 7·09	14·18	4·17	4	28·7	26·6	24·0	20·6
3×1½× 4·6	9·20	2·70	3	17·6	15·3	12·2	9·3

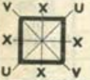
For general notes relating to above see page xi

Double Channels Welded Together and Dimensions and Properties

HEIGHTS IN FEET						Moduli of Section		Radii of Gyration	
14	18	22	26	30	34	Axis y—y	Axis x—x	Axis y—y	Axis x—x
91·9	79·0	64·9	51·8	41·6	33·8	35·7	53·2	2·84	4·54
83·6	70·9	56·9	45·3	36·0	29·2	30·5	43·8	2·73	3·90
61·5	49·5	38·1	29·5	23·1		22·0	33·1	2·41	3·82
75·8	63·0	51·3	40·7	30·2		27·3	36·7	2·70	3·55
55·4	44·0	33·8	26·1			19·4	27·8	2·38	3·49
68·1	57·1	45·6	36·2	28·6		24·0	30·3	2·66	3·19
50·0	39·4	30·1	23·2			17·1	23·4	2·34	3·16
61·5	51·1	40·8	32·0	25·4		21·2	24·5	2·63	2·82
43·9	34·4	26·2	20·0			14·8	18·7	2·30	2·80
53·0	42·8	33·2	25·6	20·2		18·4	19·3	2·58	2·44
50·5	39·1	30·0	22·8			16·7	17·5	2·27	2·33
38·3	30·0	22·8	17·3			12·7	14·2	2·29	2·41
26·2	18·8	13·7				8·5	9·5	1·88	1·99
13·8	9·3					4·8	5·1	1·52	1·56
						2·2	2·4	1·11	1·16

See pages 498 to 501 for Strength of Welds

COMPOUND STRUTS—
Safe Concentric Loads in Tons

 Two equal angles	Weight per foot in lbs.	Area in sq. ins.	EFFECTIVE			
			3	4	6	8
$5 \times 5 \times \frac{3}{8}$	24·57	7·22	52·0	51·0	48·6	45·6
$4\frac{1}{2} \times 4\frac{1}{2} \times \frac{3}{8}$	22·00	6·47	46·3	45·2	42·6	39·5
$4 \times 4 \times \frac{3}{8}$	19·46	5·73	40·6	39·5	36·8	33·0
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$	16·90	4·97	34·8	33·6	30·5	26·5
$3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{4}$	11·48	3·38	23·7	22·9	20·8	18·1
$3 \times 3 \times \frac{3}{8}$	14·34	4·22	29·0	27·8	24·2	19·7
$3 \times 3 \times \frac{1}{4}$	9·78	2·88	19·8	19·0	16·6	13·5
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{3}{8}$	11·80	3·47	23·2	21·8	17·6	12·9
$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	8·08	2·38	15·9	15·0	12·1	8·9
$2\frac{1}{4} \times 2\frac{1}{4} \times \frac{5}{16}$	8·90	2·62	17·2	15·8	12·1	8·4
$2\frac{1}{4} \times 2\frac{1}{4} \times \frac{1}{4}$	7·22	2·13	14·0	12·9	9·8	6·8
$2 \times 2 \times \frac{5}{16}$	7·84	2·30	14·6	13·1	9·2	6·1
$2 \times 2 \times \frac{1}{4}$	6·38	1·87	11·9	10·7	7·5	4·9
$1\frac{3}{4} \times 1\frac{3}{4} \times \frac{1}{4}$	5·52	1·63	9·9	8·5	5·4	3·4
$1\frac{1}{2} \times 1\frac{1}{2} \times \frac{1}{4}$	4·68	1·37	7·7	6·1	3·5	

For general notes relating to above see page xi

Double Angles Welded Together and Dimensions and Properties

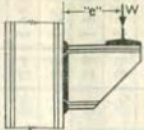
HEIGHTS IN FEET							Moduli of Section	Radii of Gyration	
10	12	14	16	18	20	22	Axes x—x	Axis v—v	Axis u—u
41·8	37·5	32·6	27·8	23·6	20·1	17·2	10·8	2·00	1·94
35·4	30·6	25·8	21·6	18·0	15·1	12·9	8·6	1·81	1·74
28·6	23·8	19·3	15·9	13·1			6·7	1·61	1·54
21·7	17·2	13·6	10·9				5·0	1·41	1·34
14·8	11·7	9·4	7·5				3·5	1·40	1·35
15·1	11·4	8·8					3·6	1·21	1·14
10·3	7·9	6·1					2·5	1·20	1·15
9·2	6·8						2·4	1·01	·94
6·5	4·8						1·7	1·00	·95
5·8							1·6	·90	·85
4·7							1·4	·90	·85
							1·2	·81	·75
							1·1	·80	·75
							·8	·70	·65
							·6	·60	·55

See pages 498 to 501 for Strength of Welds

ECCENTRIC LOADS ON WELDED BRACKETS

Welds not in same Plane as Load W

Safe Value of Welds taken as 5 Tons/sq.in. on Throat Thickness.



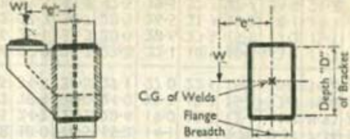
Fillet Weld Leg Length	R.S. Joist Bracket Section	Safe Loads in Tons (W) for Eccentricity "e" (inches)												
		4″	5″	6″	7″	8″	9″	10″	11″	12″	15″	18″	21″	24″
$\frac{3}{16}$″	8 × 4 × 18	9·48	7·97	6·84	5·97	5·28	4·74	4·29	3·92	3·61	2·91	2·43	2·09	1·83
$\frac{1}{4}$″	8 × 5 × 28	14·2	11·9	10·2	8·95	7·93	7·12	6·45	5·89	5·42	4·37	3·65	3·14	2·75
$\frac{1}{4}$″	8 × 6 × 35	16·0	13·5	11·6	10·2	9·02	8·10	7·40	6·70	6·17	4·97	4·16	3·58	3·14
$\frac{3}{16}$″	9 × 4 × 21	10·7	9·04	7·79	6·82	6·05	5·43	4·92	4·50	4·14	3·39	2·80	2·40	2·11
$\frac{5}{16}$″	9 × 7 × 50	24·6	21·0	18·1	15·9	14·2	12·7	11·6	10·6	9·75	7·87	6·59	5·67	4·97
$\frac{3}{16}$″	10 × $4\frac{1}{2}$ × 25	12·9	11·0	9·55	8·39	7·47	6·72	6·11	5·59	5·15	4·16	3·49	3·00	2·63
$\frac{1}{4}$″	10 × 5 × 30	18·3	15·7	13·6	12·0	10·7	9·60	8·72	7·98	7·36	5·95	4·98	4·29	3·76
$\frac{1}{4}$″	10 × 6 × 40	20·1	17·2	15·0	13·2	11·8	10·6	9·64	8·83	8·14	6·58	5·52	4·74	4·16
$\frac{5}{16}$″	10 × 8 × 55	30·9	26·6	23·2	20·5	18·3	16·5	15·1	13·8	12·7	10·3	8·65	7·44	6·53

Fillet Weld Leg Length	R.S. Joist Bracket Section	Safe Loads in Tons (W) for Eccentricity "e" (inches) 6"	7"	8"	9"	10"	11"	12"	15"	18"	21"	24"	27"	30"
$\frac{1}{4}$"	12 × 5 × 32	17·0	15·1	13·5	12·2	11·1	10·2	9·40	7·62	6·40	5·51	4·84	4·31	3·88
$\frac{5}{16}$"	12 × 6 × 44	23·3	20·7	18·5	16·8	15·3	14·0	12·9	10·5	8·83	7·60	6·67	5·94	5·36
$\frac{5}{16}$"	12 × 8 × 65	27·9	24·8	22·3	20·2	18·4	16·9	15·6	12·7	10·7	9·20	8·08	7·20	6·49
$\frac{1}{4}$"	13 × 5 × 35	18·5	16·4	14·8	13·3	12·2	11·2	10·3	8·39	7·05	6·08	5·33	4·75	4·28
$\frac{1}{4}$"	14 × 5$\frac{1}{2}$ × 40	21·4	19·1	17·2	15·6	14·3	13·1	12·2	9·90	8·33	7·19	6·31	5·63	5·07
$\frac{5}{16}$"	14 × 6 × 46	28·0	25·0	22·6	20·5	18·7	17·2	15·9	13·0	10·9	9·43	8·29	7·39	6·67
$\frac{5}{16}$"	14 × 8 × 70	33·0	29·6	26·7	24·3	22·3	20·5	19·0	15·5	13·1	11·3	9·91	8·84	7·97
$\frac{5}{16}$"	15 × 5 × 42	27·3	24·4	22·0	20·0	18·3	16·8	15·6	13·1	10·7	9·22	8·10	7·22	6·51
$\frac{1}{4}$"	15 × 6 × 45	24·4	21·9	19·8	18·0	16·5	15·2	14·1	11·5	9·70	8·37	7·36	6·56	5·92
$\frac{5}{16}$"	16 × 6 × 50	32·5	29·3	26·5	24·2	22·2	20·4	19·0	15·5	13·1	11·3	9·93	8·86	7·99
$\frac{3}{8}$"	16 × 6 × 62	38·3	34·5	31·2	28·4	26·0	24·0	22·2	18·2	15·3	13·4	11·6	10·4	9·37
$\frac{3}{8}$"	16 × 8 × 75	45·7	41·3	37·5	34·3	31·5	29·1	27·0	22·1	18·7	16·2	14·2	12·7	11·4
$\frac{5}{16}$"	18 × 6 × 55	37·1	33·6	30·6	27·9	25·7	23·8	22·1	18·1	15·3	13·2	11·7	10·4	9·40
$\frac{3}{8}$"	18 × 7 × 75	47·7	43·3	39·5	36·1	33·3	30·8	28·6	23·5	19·9	17·2	15·1	13·5	12·2
$\frac{5}{16}$"	20 × 6$\frac{1}{2}$ × 65	43·3	39·5	36·2	33·3	30·7	28·5	26·5	21·9	18·5	16·1	14·1	12·6	11·4
$\frac{7}{16}$"	20 × 7$\frac{1}{2}$ × 89	64·2	58·7	53·8	49·5	45·8	42·5	39·5	32·7	27·7	24·0	21·2	18·9	17·1
$\frac{3}{8}$"	22 × 7 × 75	60·1	55·2	50·8	46·9	43·5	40·4	37·7	31·3	26·6	23·1	20·4	18·2	16·5
$\frac{7}{16}$"	24 × 7$\frac{1}{2}$ × 95	78·1	72·2	66·8	61·9	57·5	53·7	50·2	41·8	35·7	31·0	27·4	24·5	22·2

ECCENTRIC LOADS ON WELDED BRACKETS

Welds in same Plane as Load W

Safe Value of Welds taken as 5 Tons/sq. in. on Throat Thickness.



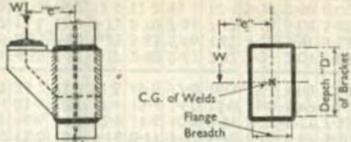
$\frac{3}{16}$" Fillet Welds	Depth "D" in Ins.	SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)														
		0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"
3" Flge.	3	7·92	5·01	3·54	2·72	2·20	1·84	1·58	1·39	1·24	1·11	1·01	0·93	0·86	0·80	0·74
	6	11·9	9·12	6·99	5·55	4·57	3·86	3·35	2·94	2·63	2·37	2·16	1·98	1·83	1·70	1·67
	9	15·8	13·4	10·9	8·92	7·47	6·39	5·56	4·91	4·43	3·98	3·62	3·33	3·08	2·87	2·67
	12	19·8	17·6	15·0	12·6	10·8	9·31	8·16	7·25	6·51	5·90	5·39	4·96	4·59	4·27	3·99
$3\frac{1}{2}$" Flge.	3	8·58	5·52	4·03	2·88	2·50	2·11	1·82	1·60	1·43	1·29	1·18	1·08	1·00	0·93	0·87
	6	12·6	9·69	7·52	6·24	5·01	4·26	3·71	3·27	2·93	2·65	2·42	2·22	2·05	1·91	1·79
	9	16·5	13·9	11·4	9·47	7·99	6·86	6·01	5·33	4·78	4·33	3·95	3·64	3·37	3·20	3·03
	12	20·5	18·2	15·5	13·2	11·3	9·86	8·68	7·74	6·97	6·33	5·60	5·34	5·10	4·61	4·31
	15	24·4	22·3	19·7	17·2	15·0	13·2	11·7	10·5	9·47	8·63	7·91	7·30	6·78	6·32	5·92

$\frac{1}{4}''$ Fillet Welds	Depth "D" in ins.	SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)														
		0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"
4" Flge.	3	12·3	8·02	5·83	4·55	3·72	3·15	2·72	2·45	2·15	1·94	1·77	1·63	1·50	1·40	1·31
	6	17·6	13·6	10·7	8·68	7·27	6·20	5·41	4·79	4·30	3·90	3·56	3·28	3·04	2·83	2·65
	9	22·9	19·3	15·6	13·3	11·3	9·77	8·58	7·63	6·87	6·23	5·71	5·26	4·87	4·54	4·25
	12	28·2	24·9	21·4	18·4	15·9	13·9	12·3	11·0	9·88	9·00	8·25	7·61	7·06	6·59	6·17
	15	33·4	30·5	27·0	23·7	20·8	18·3	16·3	14·7	13·3	12·1	11·2	10·3	9·58	8·94	8·38
	18	38·7	36·1	32·6	29·1	25·9	23·1	20·8	18·8	17·1	15·7	14·4	13·4	12·4	11·6	10·9
$4\frac{1}{2}''$ Flge.	3	13·2	8·70	6·39	5·02	4·13	3·50	3·04	2·68	2·40	2·17	1·99	1·83	1·69	1·57	1·47
	6	18·5	14·4	11·4	9·28	7·79	6·70	5·86	5·21	4·68	4·25	3·89	3·59	3·33	3·10	2·90
	9	23·8	20·0	16·7	14·0	12·0	10·4	9·16	8·17	7·37	6·70	6·15	5·67	5·26	4·93	4·60
	12	29·0	25·7	22·2	19·1	16·6	14·6	12·9	11·6	10·5	9·55	8·77	8·11	7·53	7·03	6·59
	15	34·3	31·3	27·8	24·4	21·5	19·1	17·1	15·4	14·0	12·8	11·8	10·9	10·1	9·46	8·88
	18	39·6	36·9	33·4	29·9	26·7	23·9	21·5	19·5	17·8	16·3	15·1	14·0	13·0	12·2	11·4
5" Flge.	3	14·1	9·40	6·96	5·51	4·55	3·87	3·37	2·98	2·67	2·42	2·21	2·04	1·89	1·76	1·64
	6	19·2	15·1	12·0	9·89	8·35	7·20	6·33	5·63	5·08	4·62	4·23	3·91	3·63	3·38	3·17
	9	24·6	20·8	17·4	14·7	12·6	11·0	9·73	8·70	7·86	7·17	6·58	6·08	5·65	5·28	4·94
	12	29·9	26·4	22·9	19·8	17·3	15·2	13·6	12·2	11·1	10·1	9·30	8·60	8·00	7·48	7·02
	15	35·0	32·1	28·5	25·2	22·3	19·8	17·8	16·1	14·6	13·4	12·4	11·4	10·7	9·97	9·36
	18	40·5	37·6	34·1	30·7	27·5	24·7	22·3	20·3	18·5	17·0	15·7	14·6	13·6	12·8	12·0
	21	45·8	43·1	39·8	36·2	32·8	29·8	27·1	24·7	22·7	20·9	19·4	18·1	16·9	15·8	14·9

ECCENTRIC LOADS ON WELDED BRACKETS

Welds in same Plane as Load W

Safe Value of Welds taken as 5 Tons/sq. In. on Throat Thickness.



$\frac{1}{4}''$ Fillet Welds	Depth "D" in Ins.	SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)														
		0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"
$5\frac{1}{2}''$ Flange	3	15·0	10·1	7·55	6·01	4·98	4·25	3·70	3·28	2·94	2·67	2·44	2·25	2·09	2·01	1·95
	6	20·2	15·8	12·7	10·5	8·88	7·69	6·78	6·05	5·45	4·79	4·56	4·21	3·92	3·66	3·43
	9	25·5	21·5	18·1	15·4	13·3	11·6	10·2	8·87	8·36	7·64	7·02	6·50	6·04	5·75	5·24
	12	30·8	26·5	23·6	20·5	18·0	15·9	14·2	12·8	11·6	10·7	9·81	9·10	8·47	7·92	7·44
	15	36·1	32·8	29·2	26·0	23·0	20·5	18·5	16·7	15·3	13·9	12·9	12·0	11·2	10·5	9·86
	18	41·4	38·4	34·9	31·4	28·2	25·4	23·0	21·0	19·2	17·7	16·4	15·2	14·2	13·3	12·5
	21	46·6	43·9	40·5	37·0	33·6	30·6	27·9	25·5	23·5	21·7	20·1	18·4	17·5	16·5	15·5
	24	51·7	49·4	46·1	42·6	39·1	35·8	32·9	30·3	27·9	25·9	24·1	22·5	21·1	19·8	18·7
	27	57·2	54·9	51·7	48·2	44·6	41·2	38·1	35·2	32·7	30·4	28·3	26·5	24·9	23·4	22·1

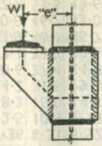
$\frac{5}{16}$" Fillet Welds	Depth "D" in Ins.	SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches) 0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"
6" Flange	3	19·8	13·5	10·2	8·14	6·77	5·79	5·05	4·49	4·03	3·66	3·35	3·09	2·87	2·67	2·50
	6	26·4	20·7	16·7	13·9	11·8	10·3	9·05	8·09	7·32	6·68	6·14	5·68	5·28	4·80	4·63
	9	33·0	27·8	23·5	20·0	17·4	15·2	13·6	12·2	11·1	10·1	9·32	8·68	8·04	7·59	7·06
	12	39·6	34·9	30·4	26·6	23·3	20·7	18·5	16·7	15·3	14·0	12·9	12·0	11·2	10·5	9·83
	15	46·2	42·0	37·5	33·3	29·7	26·6	23·9	21·7	19·9	18·3	16·9	15·7	14·7	13·7	12·9
	18	52·8	48·9	44·5	40·2	36·2	32·7	29·7	27·1	24·9	23·0	21·3	19·8	18·5	17·4	16·4
	21	59·4	55·8	51·6	47·1	43·0	39·1	35·8	32·8	30·2	28·0	26·0	24·3	22·7	21·3	20·1
	24	66·0	62·6	58·6	54·2	49·9	45·8	42·1	38·8	35·9	33·3	31·0	29·0	27·2	25·6	24·2
	27	72·6	69·6	65·6	61·2	56·8	52·5	48·6	45·0	41·8	38·9	36·3	34·1	32·0	30·2	28·5
6½" Flange	3	20·9	14·5	10·9	8·81	7·36	6·31	5·52	4·91	4·42	4·02	3·68	3·40	3·15	2·94	2·76
	6	27·5	21·7	17·5	14·6	12·5	10·9	9·78	8·64	7·83	7·15	6·58	6·09	5·67	5·30	4·98
	9	34·1	28·8	24·4	20·9	18·2	16·0	14·3	12·9	11·7	10·7	9·88	9·16	8·54	7·99	7·51
	12	40·7	35·9	31·4	27·4	24·2	21·5	19·3	17·5	16·0	14·7	13·6	12·6	11·8	11·0	10·4
	15	47·3	42·9	38·4	34·2	30·6	27·4	24·8	22·6	20·7	19·0	17·6	16·4	15·3	14·4	13·5
	18	53·9	49·9	45·4	41·1	37·1	33·7	30·6	28·0	25·7	23·8	22·1	20·6	19·3	18·1	17·0
	21	60·5	56·8	52·5	48·1	43·9	40·1	36·7	33·7	31·1	28·9	26·9	25·1	23·5	22·1	20·9
	24	67·1	63·7	59·5	55·1	50·8	46·7	43·0	39·8	37·3	34·2	31·9	29·9	28·1	26·4	25·0
	27	73·7	70·5	66·5	62·1	57·7	53·5	49·6	46·0	42·8	39·9	37·3	35·0	32·9	31·1	29·4

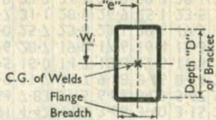
N.B.—All Bracket Plates to be ⅜" minimum thickness. It may be necessary to stiffen inclined edge of Plate to prevent Buckling.

ECCENTRIC LOADS ON WELDED BRACKETS

Welds in same Plane as Load W

Safe Value of Welds taken as 5 Tons/sq. in. on Throat Thickness.





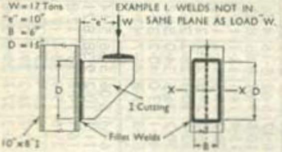
$\frac{5}{16}$" Fillet Welds	Depth "D" in Ins.	SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)														
		0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"
7" Flange	3	22·0	15·4	11·8	9·48	7·94	6·83	5·98	5·33	4·80	4·45	4·00	3·70	3·44	3·21	3·01
	6	28·6	22·6	18·4	15·4	13·2	11·5	10·2	9·17	8·31	7·60	7·00	6·49	6·04	5·65	5·30
	9	35·2	29·8	25·3	21·7	18·9	16·7	15·0	13·5	12·3	11·3	10·4	9·67	8·99	8·46	7·96
	12	41·8	36·9	32·3	28·3	25·1	22·4	20·1	18·3	16·7	15·4	14·2	13·2	12·3	11·6	10·9
	15	48·4	43·9	39·3	35·1	31·5	28·2	25·7	23·4	21·5	19·8	18·4	17·1	16·0	15·0	14·2
	18	55·0	50·9	45·8	41·4	37·4	34·0	31·0	28·4	26·2	24·2	22·5	21·0	19·7	18·6	17·5
	21	61·6	57·8	53·4	49·0	44·8	41·0	37·7	34·7	32·1	29·8	27·7	25·9	24·3	22·9	21·6
	24	68·2	64·7	60·5	56·0	51·7	47·7	44·0	40·7	37·8	35·2	32·9	30·8	28·9	27·3	25·8
	27	74·8	71·5	67·5	63·1	58·7	54·5	50·5	47·0	43·8	40·9	38·3	35·9	33·8	31·9	30·2

$\frac{3}{8}''$ Fillet Welds	Depth "D" in Ins.	SAFE LOADS IN TONS (W) FOR ECCENTRICITY "e" (inches)														
		0"	1"	2"	3"	4"	5"	6"	7"	8"	9"	10"	11"	12"	13"	14"
7½" Flange	3	27·5	19·4	14·9	12·1	10·1	8·74	7·67	6·84	6·17	5·62	5·16	4·76	4·43	4·14	3·88
	6	35·4	28·0	22·9	19·2	16·5	14·5	12·9	11·6	10·5	9·61	8·86	8·22	7·66	7·17	6·74
	9	43·2	36·6	31·1	26·9	23·5	20·8	18·6	16·9	15·4	14·1	13·1	12·1	11·3	10·6	10·0
	12	51·1	45·0	39·5	34·8	30·8	27·6	24·9	22·6	20·7	19·1	17·7	16·5	15·4	14·5	13·6
	15	59·0	53·4	47·9	42·9	38·5	34·8	31·6	28·8	26·5	24·5	22·7	21·2	19·8	18·7	17·6
	18	66·8	61·8	56·3	51·1	46·4	42·3	38·6	35·4	32·7	30·3	28·2	26·3	24·7	23·2	21·9
	21	74·7	70·0	64·7	59·5	54·5	50·0	45·9	42·4	39·2	36·5	34·0	31·8	29·9	28·2	26·6
	24	82·5	78·2	73·1	67·8	62·7	57·9	53·5	49·6	46·1	43·0	40·2	37·7	35·5	33·5	31·7
	27	90·4	86·4	81·5	76·2	71·0	66·0	61·3	57·1	53·3	49·8	46·7	43·9	41·3	39·1	37·0
8" Flange	3	28·8	20·5	15·9	12·9	10·9	9·38	8·25	7·36	6·64	6·06	5·56	5·14	4·85	4·47	4·19
	6	36·7	29·1	23·9	20·2	17·4	15·2	13·6	12·2	11·1	10·2	9·38	8·71	8·12	7·59	7·16
	9	44·5	37·7	32·2	27·9	24·4	21·7	19·5	17·6	16·1	14·8	13·7	12·8	11·9	11·2	10·5
	12	52·4	46·2	40·6	35·8	31·8	28·5	25·8	23·5	21·6	19·9	18·4	17·2	15·9	15·1	14·2
	15	60·3	54·6	49·0	44·0	39·6	35·8	32·6	29·8	27·4	25·4	23·6	22·0	20·6	19·4	18·3
	18	68·1	62·9	57·4	52·2	47·5	43·3	39·6	36·5	33·7	31·2	29·1	27·2	25·6	24·1	22·7
	21	76·0	71·2	65·9	60·6	55·6	51·1	47·0	43·5	40·3	37·5	35·0	32·8	30·8	29·1	27·5
	24	83·8	79·4	74·2	68·9	63·8	59·0	54·7	50·7	47·2	44·0	41·2	38·7	36·5	34·4	32·6
	27	91·7	87·5	82·6	77·3	72·1	67·1	62·5	58·2	54·4	50·9	47·8	44·9	42·4	40·1	38·0

N.B.—All Bracket Plates to be $\frac{3}{8}''$ minimum thickness. It may be necessary to stiffen inclined edge of Plate to prevent Buckling.

EXAMPLES OF ECCENTRIC LOADS ON WELDED BRACKETS

Example I.



$$I_{xx} \text{ (Top and Btm. Welds)} = 2\left\{B \times \left(\frac{D}{2}\right)^2\right\}$$
$$= 2(6 \times 7{\cdot}5^2) = 675{\cdot}0.$$

$$I_{xx} \text{ (Side welds)} = 2\left(\frac{D^3}{12}\right) = \frac{2 \times 15^3}{12} = 562{\cdot}5.$$

$$\text{Total } I_{xx} = 675{\cdot}0 + 562{\cdot}5 = 1237{\cdot}5 \text{ in.}^4.$$

$$\text{Shear } (F_s) = \frac{W}{A} = \frac{17}{42} = 0{\cdot}405 \text{ tons per lin. inch.}$$

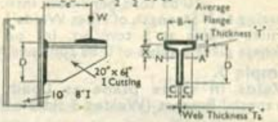
$$\text{Bending } (F_B) = \frac{M}{Z} = \frac{WeD}{2I} = \frac{17 \times 10 \times 15}{2 \times 1237{\cdot}5}$$
$$= 1{\cdot}03 \text{ tons per lin. inch.}$$

$$\text{Resultant } (F_R) = \sqrt{F_s^2 + F_B^2} = \sqrt{0{\cdot}405^2 + 1{\cdot}03^2}$$
$$= 1{\cdot}10 \text{ tons per lin. inch.}$$

Since the throat thickness was assumed unity in calculating this stress Fr, ∴ 1·10 is also the load in tons per lin. inch of fillet weld. Referring to the strength of fillet welds, table on p. 498, $\frac{5}{16}$" fillet weld at 5 tons per sq. inch on throat thickness gives a value of 1·10 tons per lin. inch.

Example 2. Welds not in same plane as Load "W"

W = 6 Tons "e" = 12" D = 10" B = $6\frac{1}{2}$"
T = 0·82" T_1 = 0·44" $C = \frac{B}{2} - \frac{T_1}{2} = 3{\cdot}03''$



If the weld group is unsymmetrical, then Moment of Inertia and Section Modulus must be calculated about neutral axis *N—A*.

To find distance $\overline{X}$, take moments about "*G-H*"

$$\begin{array}{rcrcl} & & 6{\cdot}5 \times 0 & = & 0 \\ 2 \times 3{\cdot}03 & = & 6{\cdot}06 \times 0{\cdot}82 & = & 4{\cdot}97 \\ 2(10 - 0{\cdot}82) & = & 18{\cdot}36 \times 5{\cdot}41 & = & 99{\cdot}33 \\ & & \underline{30{\cdot}92} & & \underline{104{\cdot}30} \end{array} \quad \therefore \overline{X} = \frac{104{\cdot}3}{30{\cdot}92} = 3{\cdot}37''$$

Assume throat thickness of Welds as unity.

$\therefore$ Total effective length of Fillet Weld
$= 6{\cdot}5 + 6{\cdot}06 + 18{\cdot}36 = 30{\cdot}92$ inches.

(Note.—Return ends of Fillet Welds have been neglected to allow for end craters).

$$\text{INA (Top Welds)} = (6{\cdot}5 \times 3{\cdot}37^2) + 2(3{\cdot}03 \times 2{\cdot}55^2) = 93{\cdot}52.$$

$$\text{INA (Side Welds)} = 2\left(\frac{9{\cdot}18^3}{12} + 9{\cdot}18 \times 2{\cdot}04^2\right) = 205{\cdot}34.$$

$$\text{Total INA} = 93{\cdot}52 + 205{\cdot}34 = 298{\cdot}86 \text{ in.}^4.$$

$$\text{Shear } (F_S) = \frac{W}{A} = \frac{6}{30{\cdot}92} = 0{\cdot}194 \text{ tons per linear inch}$$

$$\text{Bending } (F_B) = \frac{We\overline{X}}{\text{INA}} = \frac{6 \times 12 \times 3{\cdot}37}{298{\cdot}86}$$

(Tensile)

$= 0{\cdot}812$ tons per linear inch.

Note.—Maximum Compressive Stress has been ignored on the assumption that the bracket has a perfect bearing to Stanchion face. Therefore Tensile Value will only be considered as shown above.

Example 2.—Contd.

Welds not in same plane as Load "W"

Resultant $F_R = \sqrt{F_S^2+F_B^2} = \sqrt{0{\cdot}194^2+0{\cdot}812^2}$.

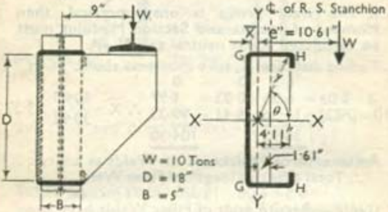
$= 0{\cdot}83$ tons per linear inch.

Referring to "Strength of Fillet Welds", p. 498, ¼" Fillet Welds at 5 tons sq. in. on throat thickness gives a value of 0·88 tons per linear in.

Example 3.

Welds in same plane as Load "W"

special Bracket (Welded 3 sides only)



Assume throat thickness of Welds as unity.

∴ Total effective length of weld = 18 + 5 + 5 = 28".

(Note.—Return ends of Fillet Welds have been neglected to allow for end craters.) It is essential to return ends at points "H", as maximum stress occurs at these points.

To find distance $\overline{X}$ take Moments about "GG".

$$\begin{array}{rcl} 18\times 0 & = & 0 \\ 2\times 5 = 10\times 2{\cdot}5 & = & 25 \\ \underline{28} & & \underline{25} \end{array} \qquad \therefore \overline{X} = \frac{25}{28} = 0{\cdot}89''.$$

$$\therefore e = 9{\cdot}0 \times 1{\cdot}61 = 10{\cdot}61''.$$

I_{xx} (Top & Btm. Welds) $= 2 \times 5 \times 9^2 = 810$ in.^{4}

I_{xx} (Side Weld) $= \dfrac{18^3}{12} = 486$ in.^{4}

Total $I_{xx} = \underline{1296}$ in.^{4}

Example 3—Contd.
Welds in same plane as Load "W"

Iyy (Top and Btm. Welds)

$$= 2\left(\frac{5^3}{12} + 5 \times 1{\cdot}61^2\right) = 46{\cdot}76 \text{ in.}^4$$

Iyy (Side Welds) $= 18 \times 0{\cdot}89^2 = 14{\cdot}26$ in.^{4}

Total Iyy $= \overline{61{\cdot}02}$ in.^{4}

Inertia for Welds on same plane as Load "W" is calculated from the Polar Moment of Inertia about its centre of gravity ; i.e. $Ip = Ixx + Iyy$

$\therefore Ip = 1296 + 61{\cdot}02 = 1357{\cdot}02$ in.^{4}.

$y = \sqrt{9^2 + 4{\cdot}11^2} = 9{\cdot}9''$ $\quad \text{Cos}\,\theta = \frac{4{\cdot}11}{9{\cdot}9} = 0{\cdot}42.$

Shear $Fs = \frac{W}{A} = \frac{10}{28} = 0{\cdot}357$ tons per lin. inch.

Bending $F_B = \frac{Wey}{Ip} = \frac{10 \times 10{\cdot}61 \times 9{\cdot}9}{1357{\cdot}02}$

$= 0{\cdot}775$ tons per lin. inch.

Resultant $F_R = \sqrt{F_S^2 + F_B^2 + 2F_BF_S \cos\theta}$

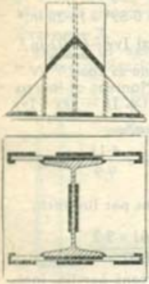
" $F_R = \sqrt{{\cdot}357^2 + {\cdot}775^2 + 2 \times {\cdot}775 \times {\cdot}357 \times {\cdot}42}$

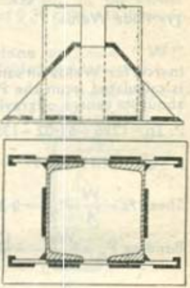
" $F_R = \sqrt{0{\cdot}96} = 0{\cdot}98$ tons per lin. inch.

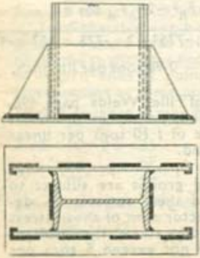
Referring to strength of Fillet Welds page 498, $\frac{5}{16}''$ Fillet Welds at 5 tons per sq. in. on throat thickness gives a value of 1·10 tons per linear inch and will be adopted.

Note.—When weld groups are subject to combined bending and shear stresses i.e. designed to carry the vector sum of shear stress and the maximum bending stress, the resultant working stress should not exceed 5 tons per sq. in. on the throat thickness.

TYPICAL STANCHION BASE DETAILS (Shop Welded)

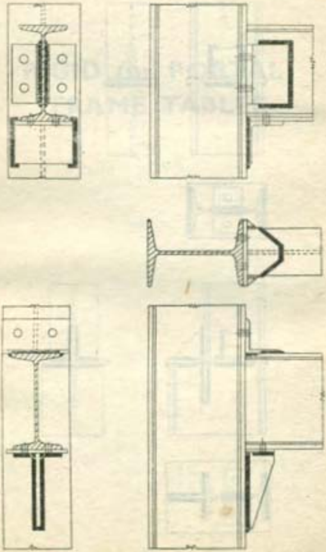




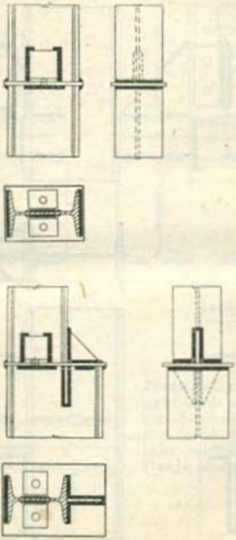


Note.—On Bases for light loads the welds can be made round the profile of the Joist without using Flange plates.

TYPICAL BEAM CONNECTION DETAILS (Shop Welded)



TYPICAL STANCHION SPLICE DETAILS
(Shop and Site Welded)



RIGID (or PORTAL) FRAME TABLES

SINGLE STOREY BUILDINGS

On the following page, No. 524, are shown diagrams of four types of Single Storey Buildings in common use today, and indicated in dotted lines are the shapes these Structures tend to take up when subjected to wind in the direction of the arrow.

In Figure 1 the Stanchions are fixed at the base and act as Cantilevers. This type of construction is very common for small spans but not very economical, nor to be recommended for spans over 30′, as the Stanchion section increases rapidly above this span and the size of concrete foundations must be large enough to withstand the bending moment or over-turning moment set up at the base.

The Structure in Figure 2 gets over the difficulty of the base moment by the insertion of a knee-brace. The bending moment is thereby transferred and is maximum at the junction of knee-brace and the stanchion, and the truss itself is designed to withstand it. The moment at the base is nil and the foundations can be designed for the vertical load only without any over-turning moment. This type of Structure has the advantage over that of Figure 1, in that the connection of column and truss is strong and ensures a Rigid Structure.

Figure 3 indicates a Structure which has a base detail similar to Figure 2 and cap connection similar to Figure 1, which means there is no bending moment at Base or Eaves. The Wind in this case is transmitted through a lattice girder at the Eaves level to the ends of the building, and thence via the end framing to the ground. The side Stanchions can therefore be designed to resist the maximum bending moment at their centre heights. This type of structure is economical, especially for a high building, provided that the building is not too long, as the Stresses in the Eaves Girder would then become very high. As an alternative to the

Eaves Girder, a Wind Girder may be placed on the Rafter Slope.

Figure 4 shows what is now commonly called a "Portal Frame". This type of construction embodies most of the good features of the other three schemes, i.e. has no bending moment at the base, has a rigid connection at the Eaves and does not need an Eaves Wind Girder. It is for this reason we have given the sections and weights of these frames for a large number of spans; and as the calculations are somewhat laborious, we have given tables of bending moments at various points in the frame to save working out.

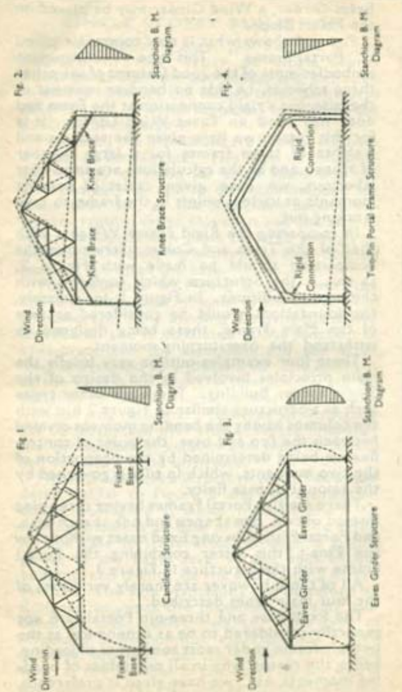
In comparing the Rigid Frame Weights with that of the Truss and Column Structures, the comparison should be made with Figure 2, as this is the Structure which conforms with the same conditions. In Figure 1, for example, the foundations would be considered as part of the Main Frame, these being designed to withstand the over-turning moment.

These four examples outline very briefly the main principles involved in the design of the Single Storey Building. There are other types such as a Structure similar to Figure 2 but with the columns having the bending moment divided between the top and base, the point of contra-flexure being determined by the proportion of the two moments, which in turn is governed by the amount of base fixity.

There are also Portal Frames having three pins instead of two, one at apex and one at each base, and Portal Frames having fixed bases without any any Pins ; this latter combining the Portal Frame with the Structure in Figure 1.

All of these however are merely variations of the four main types described.

The fixed base and three-pin Portals are not generally considered to be as economical as the two-pin frame under most conditions of loading, hence the reason why in all our tables of bending moments, etc., we have given it preference.



Cantilever Structure

Knee Brace Structure

Eaves Girder Structure

Two-Pin Portal Frame Structure

NOTES ON TWO-PIN RIGID FRAME TABLES

RIDGE TYPE

The tables on the following pages show the bending moments, reactions, scantlings and weights of Portal Frames from 20′ 0″ to 80′ 0″ span, each with three separate heights to eaves. The frames have been assumed as 15′ 0″ apart with roof slope of 21° 48′ (i.e. rise of 1/5th span) and covered with Corrugated Steel, Asbestos or Protected Metal Sheeting on Steel Purlins and Side Rails spaced at 5′ 0″ to 6′ 0″ centres respectively, or less.

The frames have been calculated to withstand the following loads :

W.1. Dead

Acting vertically on the Roof Slope, horizontally projected.
6 lbs./square foot up to 30′ 0″ span.
6·5 lbs./square foot up to 40′ 0″ span.
7 lbs./square foot up to 50′ 0″ span.
7·5 lbs./square foot up to 60′ 0″ span.
8 lbs./square foot up to 70′ 0″ span.
8·5 lbs./square foot up to 80′ 0″ span.

W.2. Wind on windward slope

15 lbs./square foot acting normally towards the roof slope.
Vertical Component (V.C.) 13·92 lbs. per square foot, on sloping surface.
Horizontal Component (H.C.) 5·57 lbs. per square foot, on sloping surface.

W.3. Wind on leeward slope (suction)
10 lbs./square foot acting normally away from the roof slope.
Vertical Component (V.C.) 9·3 lbs. per square foot, on sloping surface.
Horizontal Component (H.C.) 3·72 lbs. per square foot, on sloping surface.

W.4. Wind on windward side
15 lbs./square foot acting horizontally on a portion of the vertical side (i.e. upper two-thirds of height from ridge to ground less portion from ridge to eaves already taken in W.2).

The tabulated bending moments in foot tons are calculated at points b, c, d, e, and f, on the frame, i.e., eaves, ridge, and mid-way on roof slopes and these are noted as Mb, Mc, etc.

Where the bending moments are negative the value is prefixed by the minus sign thus indicating tension stresses on the outer face of the frame, and likewise the positive moments are prefixed by the plus sign denoting compression on the outer face of the frame.

The horizontal thrusts and vertical reactions acting on both pins are tabulated in tons. These are respectively Ha, Hg ; Va and Vg. The horizontal thrusts are negative when acting towards the frame and positive when acting away from the frame. Likewise the vertical reactions are positive when acting towards the frame and negative when acting away from the frame.

The section of Joist is given for each frame calculated from the maximum bending moment, the loads W.1 and W.2 and W.4 to be taken with or without W.3 (suction) and is worked in accordance with the British Standard Specification 449—1937 Strut Formula (F1) and the Eccentric load Formula (F2), plus the permissible increase of 33⅓% allowance for Wind, and assuming that the web of the Frame will be stiffened by the Purlins and horizontals. The sections of the vertical portions of the frame have been kept similar to the sloping portions, the inertia being constant throughout.

Where we have a member such as a portal frame, subject to both axial and bending stresses, the general practice has been to design the member as an ordinary column. It is agreed however among our foremost structural engineers that this practice is unreasonable where the major stresses are bending and only a small amount due to axial loads.

As well therefore as giving the section calculated from the B.S.S. Strut formula we have given an alternative section (marked with an asterisk and shewn immediately underneath the B.S.S. Section) based on a formula which combines both the bending and axial stresses in their true proportion.

Although not generally used today we believe this latter method will be accepted by the authorities in the near future. Until such is the case, however, we suggest that the B.S.S. method of calculating the section be worked to meantime.

Throughout, we have assumed the frames to be spaced 15′ apart. This is not always the most economical or desirable spacing for Portal Frames, but as the bending moments are tabulated, the section can be arrived at simply by proportioning the bending moment in the ratio of the desired centres and the section calculated accordingly.

The total weight of each frame is given in cwts. This weight includes, base, apex, eaves and Splice Connections, Purlin Cleats, Web

Stiffeners and bolts. The frames would be shop-welded in 3 portions up to 50′ span and 4 portions thereafter, the joints being turn bolted at site and positioned on the Roof Slope at points where the bending moment is least.

Owing to the limitations of space, we could only give one set of details for each set of 3 frames. These details have been designed for the largest height to eaves and on the basis of the combined Stress Formula, as this gives the maximum size of eaves, splice, knee and apex connection.

In each case the span dimensions are centre to centre of the section.

The overall length of the rafter has been given for each set of 3 frames and is calculated for the largest section in each case.

NORTH LIGHT TYPE

Likewise also tables show the bending moment, reactions, scantlings and weights of North Light Portal Frames and North Light Portal Roof Frames (without vertical legs), from 20′ 0″ to 40′ 0″ span also assumed as 15′ 0″ apart and covered either with Corrugated Steel, Asbestos or Protected Metal Sheeting on Steel Purlins spaced at 5′ 0″ centres or less on the sheeted slope (21° 48′) and ¼″ glass on Steel Purlins at 4′ to 5′ centres on the glazed slope (55°) and Corrugated Steel, Asbestos or Protected Metal Sheeting on Steel Horizontals spaced at 6′ 0″ centres or less on the sides.

The frames have been calculated to withstand the following loads:

W. Dead

Acting vertically on the sheeted slope, horizontally projected.

6 lbs./square foot up to 30′ 0″ span.
6·5 lbs./square foot up to 40′ 0″ span.

W.1. Dead
Acting vertically on the glazed slope, horizontally projected.
14·5 lbs./square foot up to 30′ 0″ span.
15·0 lbs./square foot up to 40′ 0″ span.

W.2. Wind on Windward sheeted slope
15 lbs./square foot acting normally towards the roof.
Vertical component (V.C.) 13·92 lbs.
Horizontal component (H.C.) 5·57 lbs.

W.3. Wind on Leeward sheeted slope (Suction)
10 lbs./square foot acting normally away from the roof.
Vertical component (V.C.) 9·28 lbs.
Horizontal component (H.C.) 3·71 lbs.

W.4. Wind on Windward glazed slope
15 lbs./square foot acting normally towards the roof.
Vertical component (V.C.) 8·60 lbs.
Horizontal Component (H.C.) 12·29 lbs.

W.5. Wind on Leeward glazed slope (Suction)
10 lbs./square foot acting normally away from the roof.
Vertical component (V.C.) 5·74 lbs.
Horizontal component (H.C.) 8·19 lbs.

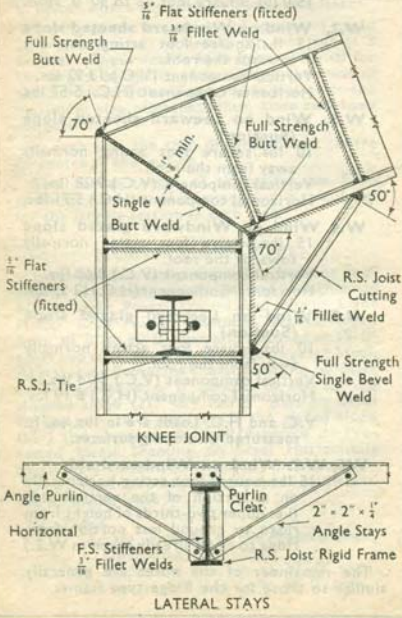
V.C. and H.C. Loads are in lbs./sq. ft. measured on sloping surfaces.

W6, W.7. Wind on Windward side
15 lbs./square foot acting horizontally on a portion of the vertical side (i.e. upper two-thirds of height from ridge to ground less portion from ridge to eaves already taken in W.2.)

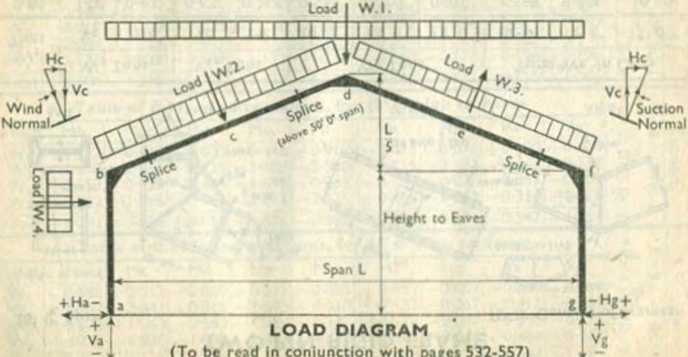
The remainder of the notes are generally similar to those for the Ridge type frames.

TYPICAL DETAIL OF KNEE JOINT AND PURLIN STAYS

For Portal Frames pages 532 to 557



RIGID FRAMES — Ordinary Ridged Type



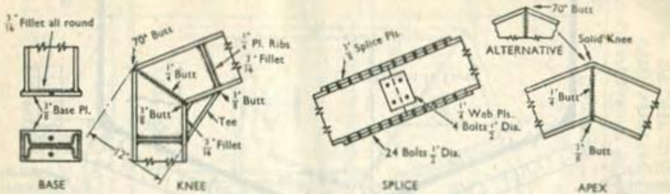
LOAD DIAGRAM

(To be read in conjunction with pages 532-557)

TWO-PIN RIGID FRAME

20′ 0″ Span

15′ 0″ Centres of Frames



W_1 Tons	W_2 Tons		W_3 Tons		W_4 Tons			Total Wt. in Cwts.		
	Vc	Hc	Vc	Hc	10′ 0″	12′ 6″	15′ 0″	10′ 0″	12′ 6″	15′ 0″
0·80	1·00	0·402	0·67	0·268	0·536	0·703	0·870	6·589	8·696	10·884
Length of Rafter Slope on Largest Section – 11′ $2\frac{1}{8}$″								*5·661	*7·393	*9·652

Section $7 \times 3\frac{1}{2} \times 15$ lb. I or *$6 \times 3 \times 12$ lb. I

Table of B.M.'s and reactions, for 10' 0" high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−0·940	+0·368	+0·672	+0·368	−0·940	−0·094	−0·094	+0·400	+0·400
W_2	+0·919	+2·209	+0·608	−1·236	−3·066	+0·092	−0·306	+0·511	+0·490
W_3	+2·044	+0·824	−0·405	−1·473	−0·613	+0·204	−0·061	−0·327	−0·341
W_4	+2·171	+0·839	−0·504	−1·132	−1·760	+0·360	−0·176	−0·196	+0·196
Total	+4·194	+4·240	+0·776	−3·473	−6·379	+0·562	−0·637	+0·715	+1·086

Section $8 \times 4 \times 18$ lb. I or *$7 \times 3\frac{1}{2} \times 15$ lb. I

Table of B.M.'s and reactions, for 12' 6" high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−0·912	+0·448	+0·800	+0·448	−0·912	−0·073	−0·073	+0·400	+0·400
W_2	+1·448	+2·543	+0·725	−1·408	−3·530	+0·115	−0·283	+0·463	+0·538
W_3	+2·353	+0·939	−0·483	−1·695	−0·965	+0·189	−0·077	−0·359	−0·309
W_4	+3·352	+1·296	−0·767	−1·871	−2·975	+0·465	−0·238	−0·316	+0·316
Total	+6·241	+5·226	+0·758	−4·526	−8·382	+0·696	−0·671	+0·547	+1·254

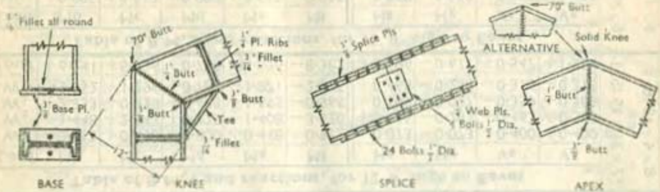
Section $9 \times 4 \times 21$ lb. I or *$8 \times 4 \times 18$ lb. I

Table of B.M.'s and reactions, for 15' 0" high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−0·885	+0·512	+0·880	+0·512	−0·885	−0·059	−0·059	+0·400	+0·400
W_2	+1·967	+2·841	+0·818	−1·595	−4·010	+0·132	−0·267	+0·412	+0·589
W_3	+2·673	+1·063	−0·545	−1·894	−1·311	+0·178	−0·088	−0·393	−0·275
W_4	+4·750	+1·826	−1·098	−2·814	−4·530	+0·568	−0·302	−0·464	+0·464
Total	+8·505	+6·242	+0·600	−5·791	−10·736	+0·819	−0·716	+0·348	+1·453

TWO-PIN RIGID FRAME

25′ 0″ Span | 15′ 0″ Centres of Frames



W_1 Tons	W_2 Tons		W_3 Tons		W_4 Tons			Total Wt. in Cwts.		
	Vc	Hc	Vc	Hc	10′ 0″	12′ 6″	15′ 0″	10′ 0″	12′ 6″	15′ 0″
1·00	1·250	0·502	0·833	0·334	0·502	0·670	0·837	8·000	9·750	12·125
Length of Rafter Slope on Largest Section – 13′ 10$\frac{3}{8}$″								*7·545	*8·482	*10·848

Table of B.M.'s and Reactions, for 10' 0" high to Eaves

Section: 7×4×16 lb. I or *7×3½×15 lb. I

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−1·490	+0·500	+0·900	+0·500	−1·490	−0·149	−0·149	+0·500	+0·500
W_2	+0·801	+3·053	+0·738	−1·715	−4·181	+0·081	−0·418	+0·691	+0·561
W_3	+2·787	+1·143	−0·492	−2·035	−0·534	+0·279	−0·054	−0·374	−0·461
W_4	+2·165	+0·821	−0·513	−1·056	−1·600	+0·342	−0·160	−0·151	+0·151
Total	+4·263	+5·517	+1·125	−4·306	−7·805	+0·553	−0·781	+1·040	+1·212

Table of B.M.'s and Reactions, for 12' 6" high to Eaves

Section: 8×4×18 lb. I or *7×3½×15 lb. I

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−1·475	+0·575	+1·050	+0·575	−1·475	−0·118	−0·118	+0·500	+0·500
W_2	+1·419	+3·435	+0·918	−1·940	−4·809	+0·113	−0·385	+0·640	+0·612
W_3	+3·206	+1·293	−0·612	−2·290	−0·946	+0·257	−0·075	−0·408	−0·427
W_4	+3·392	+1·310	−0·788	−1·769	−2·750	+0·450	−0·220	−0·245	+0·245
Total	+6·542	+6·613	+1·180	−5·424	−9·980	+0·702	−0·798	+0·895	+1·357

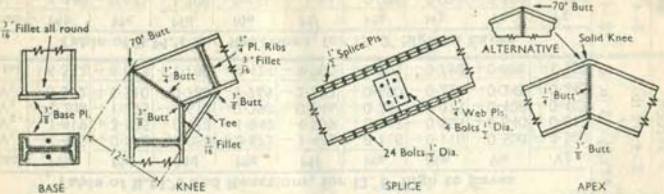
Table of B.M.'s and Reactions, for 15' 0" high to Eaves

Section: 9×4×21 lb. I or *8×4×18 lb. I

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−1·440	+0·675	+1·200	+0·675	− 1·44	−0·096	−0·096	+0·500	+0·500
W_2	+2·071	+3·847	+1·052	−2·174	− 5·40	+0·137	−0·361	+0·592	+0·660
W_3	+3·600	+1·449	−0·701	−2·565	− 1·38	+0·241	−0·091	−0·440	−0·395
W_4	+4·838	+1·865	−1·103	−2·666	− 4·23	+0·555	−0·282	−0·363	+0·363
Total	+9·069	+7·836	+1·149	−6·730	−12·45	+0·837	−0·830	+0·729	+1·523

TWO-PIN RIGID FRAME

30′ 0″ Span — 15′ 0″ Centres of Frames



W_1 Tons	W_2 Tons		W_3 Tons		W_4 Tons			Total Wt. in Cwts.		
	Vc	Hc	Vc	Hc	10′ 0″	12′ 6″	15′ 0″	10′ 0″	12′ 6″	15′ 0″
1·200	1·506	0·603	1·004	0·402	0·469	0·636	0·804	9·79	12·21	15·54
Length of Rafter Slope on Largest Section = 16′ 7¼″								*8·55	*11·23	*13·46

Section: 8×4×18 lb. I or *7×3½×15 lb. I

Table of B.M.'s and Reactions, for 10′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−2·140	+0·612	+1·080	+0·612	−2·140	−0·214	−0·214	+0·600	+0·600
W_2	+0·527	+3·931	+0·793	−2·335	−5·450	+0·052	−0·545	+0·869	+0·634
W_3	+3·633	+1·557	−0·529	−2·621	−0·351	+0·363	−0·035	−0·423	−0·579
W_4	+2·156	+0·825	−0·504	−0·972	−1·440	+0·325	−0·144	−0·120	+0·120
Total	+4·176	+6·925	+1·369	−5·316	−9·381	+0·526	−0·938	+1·349	+1·354

Section: 9×4×21 lb. I or *8×4×18 lb. I

Table of B.M.'s and Reactions, for 12′ 6″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−2·138	+0·720	+1·332	+0·720	− 2·14	−0·171	−0·171	+0·600	+0·600
W_2	+1·285	+4·462	+1·131	−2·536	− 6·19	+0·103	−0·495	+0·817	+0·686
W_3	+4·124	+1·691	−0·754	−2·975	− 0·86	+0·330	−0·069	−0·457	−0·545
W_4	+3·386	+1·289	−0·804	−1·677	− 2·55	+0·432	−0·204	−0·198	+0·198
Total	+6·657	+8·162	+1·659	−6·468	−11·74	+0·694	−0·939	+1·219	+1·484

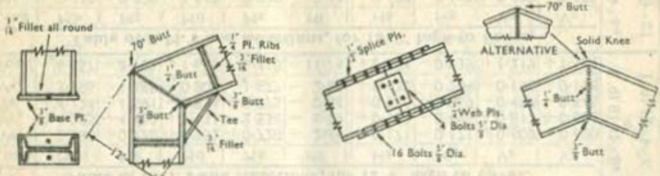
Section: 10×4½×25 lb. I or *9×4×21 lb. I

Table of B.M.'s and Reactions, for 15′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−2·130	+0·828	+1·512	+0·828	− 2·13	−0·142	−0·142	+0·600	+0·600
W_2	+2·066	+4·979	+1·328	−2·781	− 6·90	+0·137	−0·461	+0·769	+0·734
W_3	+4·600	+1·854	−0·885	−3·319	− 1·38	+0·307	−0·091	−0·489	−0·513
W_4	+4·884	+1·879	−1·119	−2·539	− 3·96	+0·540	−0·264	−0·295	+0·295
Total	+9·420	+9·540	+1·721	−7·811	−14·37	+0·842	−0·958	+1·074	+1·629

TWO-PIN RIGID FRAME

35′ 0″ Span | 15′ 0″ Centres of Frames



BASE | KNEE | SPLICE | APEX

W_1 Tons	W_2 Tons		W_3 Tons		W_4 Tons			Total Wt. in Cwts.		
	Vc	Hc	Vc	Hc	10′ 0″	12′ 6″	15′ 0″	10′ 0″	12′ 6″	15′ 0″
1·522	1·757	0·703	1·171	0·469	0·435	0·603	0·770	12·27	15·59	16·86
Length of Rafter Slope on Largest Section = 19′ $3\frac{9}{16}$″								*9·87	*11·88	*14·41

Section: 9×4×21 lb. I or *7×4×16 lb. I

Table of B.M.'s and Reactions, for 10′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−3·120	+0·794	+1·353	+0·794	− 3·12	−0·312	−0·312	+0·761	+0·761
W_2	+0·201	+4·979	+0·869	−2·954	− 6·77	+0·020	−0·678	+1·045	+0·707
W_3	+4·515	+1·969	−0·579	−3·319	− 0·13	+0·452	−0·013	−0·471	−0·697
W_4	+2·107	+0·804	−0·513	−0·906	− 1·30	+0·305	−0·130	−0·097	+0·097
Total	+3·703	+8·546	+1·709	−6·385	−11·32	+0·465	−1·133	+1·709	+1·565

Section: 10×4½×25 lb. I or *8×4×18 lb. I

Table of B.M.'s and Reactions, for 12′ 6″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−3·175	+0·943	+1·705	+0·943	− 3·18	−0·254	−0·254	+0·761	+0·761
W_2	+1·021	+5·587	+1·223	−3·223	− 7·70	+0·082	−0·615	+0·997	+0·756
W_3	+5·130	+2·149	−0·815	−3·725	− 0·68	+0·410	−0·055	−0·504	−0·665
W_4	+3·378	+1·286	−0·796	−1·573	− 2·35	+0·415	−0·188	−0·164	+0·164
Total	+6·354	+9·965	+2·132	−7·578	−13·91	+0·653	−1·112	+1·594	+1·681

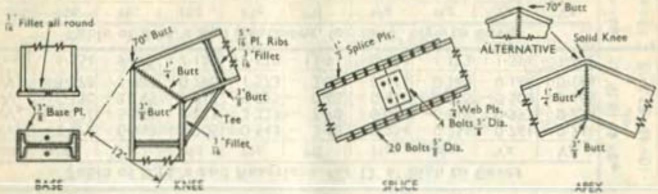
Section: 10×4½×25 lb. I or *9×4×21 lb. I

Table of B.M.'s and Reactions, for 15′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−3·165	+ 1·10	+2·009	+1·098	− 3·17	−0·211	−0·211	+0·761	+0·761
W_2	+1·883	+ 6·16	+1·544	−3·517	− 8·58	+0·126	−0·571	+0·946	+0·807
W_3	+5·718	+ 2·35	−1·029	−4·107	− 1·26	+0·381	−0·084	−0·538	−0·631
W_4	+4·893	+ 1·88	−1·129	−2·417	− 3·71	+0·523	−0·247	−0·246	+0·246
Total	+9·329	+11·49	+2·424	−8·943	−16·72	+0·819	−1·113	+1·461	+1·814

TWO-PIN RIGID FRAME

40′ 0″ Span | 15′ 0″ Centres of Frames



W_1 Tons	W_2 Tons		W_3 Tons		W_4 Tons			Total Wt. in Cwts.		
	Vc	Hc	Vc	Hc	10′ 0″	12′ 6″	15′ 0″	10′ 0″	12′ 6″	15′ 0″
1·740	2·008	0·803	1·339	0·535	0·402	0·569	0·737	15·99	17·10	21·56
Length of Rafter Slope on Largest Section = 21′ 11⅞″								*12·14	*14·67	*18·58

Section: 10×4½×25 lb. I or *8×4×18 lb. I

Table of B.M.'s and Reactions, for 10′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−4·010	+ 0·91	+1·462	+0·905	− 4·01	−0·401	−0·401	+0·870	+0·870
W_2	−0·266	+ 6·10	+0·870	−3·658	− 8·20	−0·022	−0·819	+1·222	+0·781
W_3	+5·464	+ 2·44	−0·580	−4·069	+ 0·18	+0·546	+0·015	−0·521	−0·815
W_4	+2·043	+ 0·78	−0·506	−0·838	− 1·17	+0·285	−0·117	−0·080	+0·080
Total	−4·010	+10·23	+1·826	−7·660	−13·38	+0·408	−1·337	+2·012	+1·731

Section: 10×4½×25 lb. I or *9×4×21 lb. I

Table of B.M.'s and Reactions , for 12′ 6″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−4·112	+ 1·11	+1·949	+1·114	− 4·11	−0·329	−0·329	+0·870	+0·870
W_2	+0·649	+ 6·81	+1·306	−4·004	− 9·31	+0·052	−0·745	+1·174	+0·829
W_3	+6·209	+ 2·67	−0·871	−4·539	− 0·43	+0·497	−0·035	−0·553	−0·783
W_4	+3·339	+ 1·27	−0·786	−1·474	− 2·16	+0·396	−0·173	−0·138	+0·138
Total	+6·085	+11·86	+2·469	−8·903	−16·01	+0·616	−1·282	+1·906	+1·837

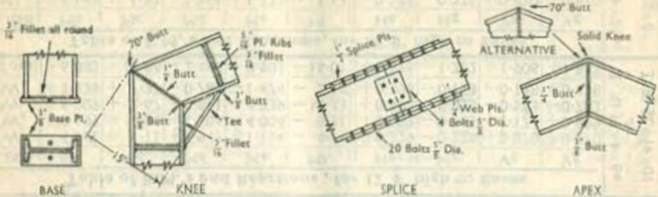
Section: 10×5×30 lb. I or *10×4½×25 lb. I

Table of B.M.'s and Reactions, for 15′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−4·140	+ 1·32	+2·366	+1·322	− 4·14	−0·276	−0·276	+0·870	+0·870
W_2	+1·606	+ 7·46	+1·743	−4·292	−10·35	+0·107	−0·690	+1·122	+0·881
W_3	+6·899	+ 2·86	−1·162	−4·971	− 1·07	+0·460	−0·071	−0·587	−0·748
W_4	+4·873	+ 1·86	−1·156	−2·318	− 3·48	+0·505	−0·232	−0·209	+0·209
Total	+9·238	+13·50	+2·953	−10·259	−19·04	+0·796	−1·269	+1·783	+1·960

TWO-PIN RIGID FRAME

45′ 0″ Span | 15′ 0″ Centres of Frames



W_1 Tons	W_2 Tons		W_3 Tons		W_4 Tons			Total Wt. in Cwts.		
	Vc	Hc	Vc	Hc	10′ 0″	12′ 6″	15′ 0″	10′ 0″	12′ 6″	15′ 0″
2·110	2·255	0·904	1·503	0·603	0·368	0·536	0·703	16·96	21·57	22·95
Length of Rafter Slope on Largest Section = 24′ 9¼″								*14·95	*16·03	*19·78

Section: 10×4½×25 lb. I or *9×4×21 lb. I

Table of B.M.'s and Reactions, for 10′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−5·350	+ 1·14	+1·709	+1·140	− 5·35	−0·535	−0·535	+1·055	+1·055
W_2	−0·718	+ 7·39	+0·738	−4·476	− 9·68	−0·073	−0·969	+1·402	+0·851
W_3	+6·456	+ 2·98	−0·492	−4·929	+ 0·48	+0·646	+0·049	−0·567	−0·935
W_4	+1·955	+ 0·73	−0·488	−0·768	− 1·05	+0·263	−0·105	−0·067	+0·067
Total	−5·350	+12·24	+1·959	−9·033	−16·08	−0·345	−1·609	+2·390	+1·973

Section: 10×5×30 lb. I or *9×4×21 lb. I

Table of B.M.'s and Reaction, for 12′ 6″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−5·500	+ 1·42	+2·412	+ 1·42	− 5·50	−0·440	−0·440	+1·055	+1·055
W_2	+0·263	+ 8·21	+1·489	− 4·72	−10·94	+0·022	−0·875	+1·351	+0·902
W_3	+7·295	+ 3·15	−0·993	− 5·47	− 0·18	+0·583	−0·015	−0·601	−0·901
W_4	+3·283	+ 1·25	−0·786	− 1·39	− 1·99	+0·377	−0·159	−0·117	+0·117
Total	−5·500	+14·03	+3·115	−10·16	−18·61	+0·542	−1·489	+2·289	+2·074

Section: 12×5×30 lb. I or *10×4½×25 lb. I

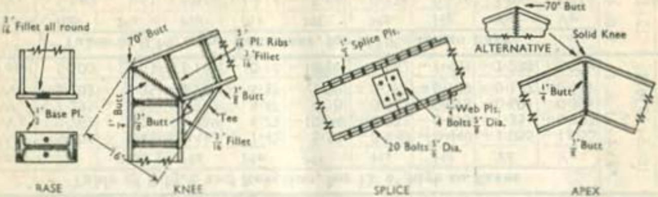
Table of B.M.'s and Reactions, for 15′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−5·640	+ 1·61	+2·849	+1·614	− 5·64	−0·376	−0·376	+1·055	+1·055
W_2	+1·205	+ 8·87	+1·815	−5·220	−12·24	+0·080	−0·817	+1·302	+0·951
W_3	+8·163	+ 3·48	−1·210	−5·915	− 0·80	+0·545	−0·053	−0·634	−0·868
W_4	+4·845	+ 1·85	−1·134	−2·187	− 3·24	+0·487	−0·216	−0·180	+0·180
Total	+8·573	+15·81	+3·530	−11·708	−21·92	+0·736	−1·462	+2·177	+2·186

TWO-PIN RIGID FRAME

50′ 0″ Span

15′ 0″ Centres of Frames



W_1 Tons	W_2 Tons		W_3 Tons		W_4 Tons			Total Wt. in Cwts.		
	Vc	Hc	Vc	Hc	10′ 0″	12′ 6″	15′ 0″	10′ 0″	12′ 6″	15′ 0″
2·345	2·510	1·004	1·673	0·669	0·335	0·502	0·669	21·79	23·35	24·69
Length of Rafter Slope on Largest Section = 27′ $5\frac{9}{16}$″								*16·21	*20·25	*21·26

Section: 10×5×30 lb. I or *9×4×21 lb. I

Table of B.M.'s and Reactions, for 10′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−6·450	+ 1·41	+1·759	+ 1·41	− 6·45	−0·645	−0·645	+1·172	+1·172
W_2	−1·313	+ 8·72	+0·572	− 5·35	−11·28	−0·131	−1·127	+1·579	+0·925
W_3	+7·517	+ 3·57	−0·381	− 5·81	+ 0·88	+0·751	+0·087	−0·617	−1·053
W_4	+1·862	+ 0·70	−0·460	− 0·70	− 0·93	+0·242	−0·093	−0·056	+0·056
Total	−6·450	+14·40	+1·871	−10·45	−18·66	−0·534	−1·865	+2·695	+2·153

Section: 12×5×30 lb. I or *10×4½×25 lb. I

Table of B.M.'s and Reactions, for 12′ 6″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−6·750	+ 1·52	+2·462	+ 1·52	− 6·75	−0·540	−0·540	+1·172	+1·172
W_2	−0·381	+ 9·61	+1·197	− 5·81	−12·83	−0·030	−1·027	+1·531	+0·973
W_3	+8·554	+ 3·87	−0·798	− 6·41	+ 0·25	+0·685	+0·020	−0·649	−1·021
W_4	+3·195	+ 1·22	−0·785	− 1·31	− 1·83	+0·356	−0·146	−0·100	+0·100
Total	−6·750	+16·22	+2·874	−12·01	−21·41	+0·471	−1·713	+2·603	+2·245

Section: 12×5×30 lb. I or *10×4½×25 lb. I

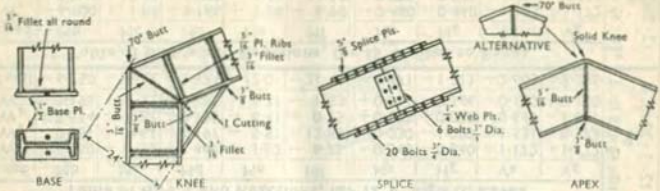
Table of B.M.'s and Reactions, for 15′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−6·900	+ 1·88	+3·166	+ 1·88	− 6·90	−0·460	−0·460	+1·172	+1·172
W_2	+0·744	+10·43	+1·973	− 6·12	−14·20	+0·050	−0·946	+1·480	+1·024
W_3	+9·466	+ 4·08	−1·315	− 6·95	− 0·50	+0·631	−0·033	−0·683	−0·987
W_4	+4·760	+ 1·80	−1·175	− 2·11	− 3·05	+0·466	−0·203	−0·156	+0·156
Total	+8·070	+18·19	+3·964	−13·30	−24·65	+0·687	−1·642	+2·496	+2·352

TWO-PIN RIGID FRAME

55′ 0″ Span

15′ 0″ Centres of Frames



W_1 Tons	W_2 Tons		W_3 Tons		W_4 Tons			Total Wt. in Cwts.		
	Vc	Hc	Vc	Hc	12′ 6″	15′ 0″	20′ 0″	12′ 6″	15′ 0″	20′ 0″
2·75	2·761	1·105	1·841	0·737	0·469	0·636	0·971	27·91	32·13	39·91
Length of Rafter Slope on Largest Section – 30′ 2$\frac{15}{16}$″								*22·35	*27·88	*32·51

Section: 12×5×32 lb. I or *10×4½×25 lb. I

Table of B.M.'s and Reactions, for 12′ 6″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−8·562	+ 1·97	+2·874	+ 1·97	− 8·56	−0·685	−0·685	+1·375	+1·375
W_2	−0·968	+11·13	+1·276	− 6·73	−14·66	−0·077	−1·173	+1·708	+1·046
W_3	+9·776	+ 4·49	−0·851	− 7·42	+ 0·65	+0·782	+0·051	−0·697	−1·139
W_4	+3·092	+ 1·16	−0·757	− 1·22	− 1·68	+0·335	−0·134	−0·087	+0·087
Total	−8·562	+18·75	+3·393	−13·40	−24·90	−0·685	−1·992	+2·996	+2·508

Section: 13×5×35 lb. I or *10×5×30 lb. I

Table of B.M.'s and Reactions, for 15′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	− 8·820	+ 2·12	+3·630	+ 2·12	− 8·82	−0·588	−0·588	+1·375	+1·375
W_2	+ 0·150	+12·02	+1·942	− 7·18	−16·29	+0·010	−1·086	+1·657	+1·097
W_3	+10·859	+ 4·78	−1·295	− 8·01	− 0·10	+0·724	−0·007	−0·731	−1·105
W_4	+ 4·692	+ 1·77	−1·147	− 1·99	− 2·84	+0·447	−0·189	−0·137	+0·137
Total	− 8·820	+20·69	+4·425	−15·06	−28·05	+0·593	−1·870	+2·895	+2·609

Section: 14×5½×40 lb. I or *12×5×32 lb. I

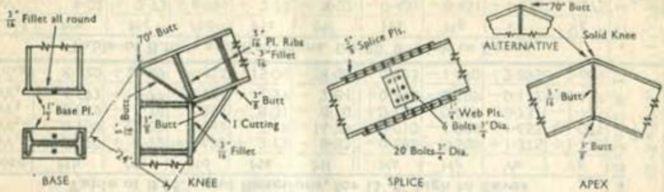
Table of B.M.'s and Reactions, for 20′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	− 9·02	+ 2·72	+4·991	+ 2·72	− 9·02	−0·451	−0·451	+1·375	+1·375
W_2	+ 2·71	+13·92	+3·222	− 8·02	−19·21	+0·136	−0·960	+1·557	+1·197
W_3	+ 12·80	+ 5·35	−2·148	− 9·28	− 1·81	+0·640	−0·091	−0·798	−1·038
W_4	+ 8·65	+ 3·30	−2·054	− 4·07	− 6·08	+0·667	−0·304	−0·268	+0·268
Total	+ 15·14	+25·29	+6·159	−18·65	−36·12	+0·992	−1·806	+2·664	+2·840

TWO-PIN RIGID FRAME

60′ 0″ Span

15′ 0″ Centres of Frames



W_1 Tons	W_2 Tons		W_3 Tons		W_4 Tons			Total Wt. in Cwts.		
	Vc	Hc	Vc	Hc	12′ 6″	15′ 0″	20′ 0″	12′ 6″	15′ 0″	20′ 0″
3·000	3·012	1·205	2·008	0·803	0·435	0·603	0·937	32·16	33·49	44·31
Length of Rafter Slope on Largest Section – 32′ 11 13/16″								*27·97	*29·41	*37·01

Section: 13×5×35 lb. I or *10×5×30 lb. I

Table of B.M.'s and Reactions, for 12′ 6″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	− 9·99	+ 2·16	+2·880	+ 2·16	− 9·99	−0·799	−0·799	+1·500	+1·500
W_2	− 1·64	+12·78	+1·021	− 7·77	−16·59	−0·132	−1·328	+1·884	+1·120
W_3	+11·06	+ 5·18	−0·681	− 8·52	+ 1·09	+0·885	+0·088	−0·747	−1·256
W_4	+ 2·97	+ 1·11	−0·739	− 1·13	− 1·53	+0·313	−0·122	−0·075	+0·075
Total	− 9·99	+21·23	+3·162	−15·26	−28·11	−0·799	−2·249	+3·309	+2·695

Section: 13×5×35 lb. I or *12×5×30 lb. I

Table of B.M.'s and Reactions, for 15′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−10·37	+ 2·34	+3·780	+ 2·34	−10·37	−0·691	−0·691	+1·500	+1·500
W_2	− 0·53	+13·86	+1·767	− 8·31	−18·46	−0·034	−1·230	+1·836	+1·168
W_3	+12·31	+ 5·54	−1·178	− 9·24	+ 0·35	+0·820	+0·023	−0·779	−1·224
W_4	+ 4·60	+ 1·73	−1·122	− 1·88	− 2·64	+0·427	−0·176	−0·121	+0·121
Total	−10·37	+23·47	+4·425	−17·09	−31·47	−0·691	−2·097	+3·215	+2·789

Section: 15×5×42 lb. I or *13×5×35 lb. I

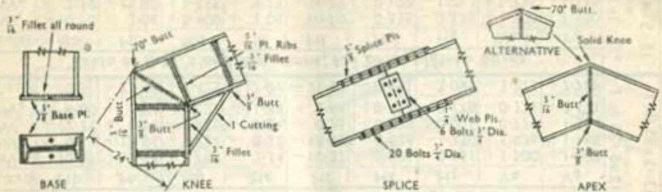
Table of B.M.'s and Reactions, for 20′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−10·70	+ 3·06	+5·400	+ 3·06	−10·70	−0·535	−0·535	+1·500	+1·500
W_2	+ 2·16	+15·80	+3·255	− 9·25	−21·75	+0·109	−1·087	+1·736	+1·268
W_3	+14·50	+ 6·17	−2·170	−10·53	− 1·44	+0·725	−0·073	−0·845	−1·157
W_4	+ 8·61	+ 3·29	−2·046	− 3·90	− 5·76	+0·649	−0·288	−0·239	+0·239
Total	+14·57	+28·32	+6·609	−20·62	−39·65	+0·948	−1·983	+2·997	+3·007

TWO-PIN RIGID FRAME

65′ 0″ Span

15′ 0″ Centres of Frames



W_1 Tons	W_2 Tons		W_3 Tons		W_4 Tons			Total Wt. in Cwts.		
	Vc	Hc	Vc	Hc	12′ 6″	15′ 0″	20′ 0″	12′ 6″	15′ 0″	20′ 0″
3·484	3·263	1·305	2·175	0·871	0·402	0·569	0·904	38·26	40·04	49·44
Length of Rafter Slope on Largest Section – 35′ 8⅛″								*29·80	*33·08	*39·21

Section
14×5½×40 lb. I
or
*12×5×30 lb. I

Table of B.M.'s and Reactions, for 12′ 6″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−12·34	+ 2·49	+3·170	+ 2·49	−12·34	−0·987	−0·987	+1·742	+1·742
W_2	− 2·42	+14·64	+0·816	− 8·91	−18·61	−0·195	−1·490	+2·062	+1·194
W_3	+12·41	+ 5·94	−0·544	− 9·76	+ 1·62	+0·993	+0·130	−0·796	−1·375
W_4	+ 2·83	+ 1·06	−0·718	− 1·05	− 1·39	+0·291	−0·111	−0·065	+0·065
Total	−12·34	+24·13	+3·268	−17·23	−32·34	−0·987	−2·588	+3·739	+3·001

Section
14×5½×40 lb. I
or
*12×5×32 lb. I

Table of B.M.'s and Reactions, for 15′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−12·86	+ 2·94	+4·303	+ 2·94	−12·86	−0·857	−0·857	+1·742	+1·742
W_2	− 1·28	+15·56	+1·729	− 9·46	−20·70	−0·084	−1·379	+2·014	+1·242
W_3	+13·80	+ 6·31	−1·153	−10·37	+ 0·85	+0·919	+0·056	−0·828	−1·343
W_4	+ 4·48	+ 1·70	−1·119	− 1·78	− 2·45	+0·406	−0·163	−0·106	+0·106
Total	−12·86	+26·51	+4·913	−18·67	−36·01	−0·857	−2·399	+3·650	+3·090

Section
15×6×45 lb. I
or
*13×5×35 lb. I

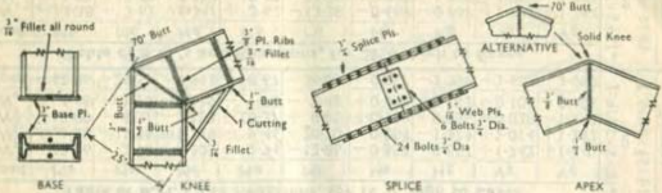
Table of B.M.'s and Reactions, for 20′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−13·38	+ 3·62	+6·341	+ 3·62	−13·38	−0·669	−0·669	+1·742	+1·742
W_2	+ 1·52	+17·78	+3·354	−10·52	−24·39	+0·075	−1·220	+1·914	+1·342
W_3	+16·26	+ 7·01	−2·236	−11·85	− 1·01	+0·813	−0·050	−0·895	−1·276
W_4	+ 8·51	+ 3·23	−2·088	− 3·79	− 5·50	+0·629	−0·275	−0·215	+0·215
Total	−13·38	+31·64	+7·607	−22·54	−44·28	+0·848	−2·214	+3·441	+3·299

TWO-PIN RIGID FRAME

70′ 0″ Span

15′ 0″ Centres of Frames



W_1 Tons	W_2 Tons		W_3 Tons		W_4 Tons			Total Wt. in Cwts.		
	Vc	Hc	Vc	Hc	15′ 0″	20′ 0″	25′ 0″	15′ 0″	20′ 0″	25′ 0″
3·752	3·514	1·405	2·343	0·937	0·536	0·870	1·205	44·73	56·90	67·90
Length of Rafter Slope on Largest Section – 38′ $6\frac{1}{16}$″								*42·88	*46·39	*52·61

Section

15×5×42 lb. I or *14×5½×40 lb. I

Table of B.M.'s and Reactions, for 15′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−14·69	+ 3·15	+4·465	+ 3·15	−14·69	−0·979	−0·979	+1·876	+1·876
W_2	− 2·06	+17·53	+1·575	−10·70	−22·98	−0·138	−1·532	+2·190	+1·316
W_3	+15·32	+ 7·13	−1·050	−11·69	+ 1·37	+1·021	+0·092	−0·877	−1·460
W_4	+ 4·35	+ 1·64	−1·089	− 1·68	− 2·27	+0·385	−0·151	−0·094	+0·094
Total	−14·69	+29·45	+4·951	−20·92	−39·94	−0·979	−2·662	+3·972	+3·286

Section

16×6×50 lb. I or *14×5½×40 lb. I

Table of B.M.'s and Reactions, for 20′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−15·38	+ 3·94	+6·566	+ 3·94	−15·38	−0·769	−0·769	+1·876	+1·876
W_2	+ 0·81	+19·92	+3·476	−11·79	−27·09	+0·041	−1·353	+2·090	+1·416
W_3	+18·06	+ 7·86	−2·317	−13·28	− 0·54	+0·902	−0·027	−0·944	−1·393
W_4	+ 8·43	+ 3·20	−2·015	− 3·61	− 5·20	+0·610	−0·260	−0·195	+0·195
Total	−15·38	+34·92	+8·027	−24·74	−48·21	+0·784	−2·409	+3·771	+3·487

Section

18×6×55 lb. I or *15×5×42 lb. I

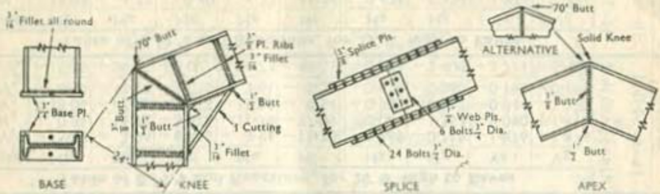
Table of B.M.'s and Reactions, for 25′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−15·65	+ 4·73	+ 8·40	+ 4·73	−15·65	−0·626	−0·626	+1·876	+1·876
W_2	+ 4·02	+22·29	+ 4·88	−12·97	−30·85	+0·161	−1·234	+1·991	+1·515
W_3	+20·57	+ 8·65	− 3·25	−14·86	− 2·68	+0·823	−0·107	−1·010	−1·327
W_4	+13·50	+ 5·14	− 3·22	− 6·31	− 9·40	+0·829	−0·376	−0·327	+0·327
Total	+22·44	+40·81	+10·06	−29·41	−58·58	+1·187	−2·343	+3·540	+3·718

TWO-PIN RIGID FRAME

75′ 0″ Span

15′ 0″ Centres of Frames



W_1 Tons	W_2 Tons		W_3 Tons		W_4 Tons			Total Wt. in Cwts.		
	Vc	Hc	Vc	Hc	15′ 0″	20′ 0″	25′ 0″	15′ 0″	20′ 0″	25′ 0″
4·267	3·765	1·506	2·510	1·004	0·502	0·837	1·172	49·98	59·77	70·21
Length of Rafter Slope on Largest Section – 41′ 2⅜″								*45·46	*48·10	*59·47

Section: 15×6×45 lb. I or *14×5½×40 lb. I

Table of B.M.'s and Reactions, for 15′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−17·60	+ 3·84	+4·801	+ 3·84	−17·60	−1·173	−1·173	+2·134	+2·134
W_2	− 2·97	+19·60	+1·196	−12·09	−25·39	−0·198	−1·693	+2·370	+1·386
W_3	+16·93	+ 8·06	−0·797	−13·07	+ 1·98	+1·129	+0·132	−0·924	−1·580
W_4	+ 4·19	+ 1·57	−1·020	− 1·55	− 2·09	+0·363	−0·139	−0·084	+0·084
Total	−17·60	+33·07	+4·977	−22·87	−45·08	−1·173	−3·005	+4·420	+3·604

Section: 16×6×50 lb. I or *14×5½×40 lb. I

Table of B.M.'s and Reactions, for 20′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−18·60	+ 4·48	+7·361	+ 4·48	−18·60	−0·930	−0·930	+2·134	+2·134
W_2	+ 0·01	+22·19	+3·321	−13·18	−29·88	+0·003	−1·492	+2·270	+1·486
W_3	+19·92	+ 8·79	−2·214	−14·80	− 0·01	+0·995	−0·002	−0·991	−1·513
W_4	+ 8·29	+ 3·11	−2·043	− 3·50	− 4·96	+0·589	−0·248	−0·177	+0·177
Total	−18·60	+38·57	+8·639	−27·00	−53·45	−0·930	−2·672	+4·227	+3·797

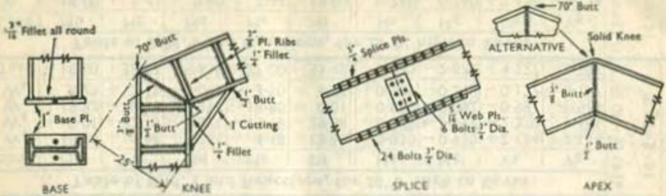
Section: 18×6×55 lb. I or *15×6×45 lb. I

Table of B.M.'s and Reactions, for 25′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−19·00	+ 5·44	+ 9·60	+ 5·44	−19·00	−0·760	−0·760	+2·134	+2·134
W_2	+ 3·39	+24·71	+ 5·11	−14·41	−33·97	+0·136	−1·358	+2·171	+1·585
W_3	+22·64	+ 9·61	− 3·41	−16·47	− 2·26	+0·905	−0·091	−1·057	−1·447
W_4	+13·44	+ 5·12	− 3·23	− 6·13	− 9·03	+0·811	−0·361	−0·299	+0·299
Total	+20·47	+44·88	+11·48	−31·57	−64·26	+1·092	−2·570	+4·006	+4·018

TWO-PIN RIGID FRAME

80′ 0″ Span | 15′ 0″ Centres of Frames



W_1 Tons	W_2 Tons		W_3 Tons		W_4 Tons			Total Wt. in Cwts.		
	Vc	Hc	Vc	Hc	15′ 0″	20′ 0″	25′ 0″	15′ 0″	20′ 0″	25′ 0″
4·552	4·016	1·607	2·677	1·071	0·469	0·803	1·138	58·20	68·47	85·69
Length of Rafter Slope on Largest Section – 43′ 11¾″								*52·43	*62·41	*68·54

Section: 16×6×50 lb. I or *15×6×45 lb. I

Table of B.M.'s and Reactions, for 15′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−19·70	+ 4·01	+4·734	+ 4·01	−19·70	−1·313	−1·313	+2·276	+2·276
W_2	− 3·86	+21·80	+0·866	−13·38	−27·77	−0·256	−1·850	+2·547	+1·460
W_3	+18·51	+ 8·92	−0·577	−14·53	+ 2·57	+1·233	+0·171	−0·973	−1·698
W_4	+ 4·04	+ 1·54	−0·977	− 1·44	− 1·91	+0·342	−0·127	−0·074	+0·074
Total	−19·70	+36·27	+4·623	−25·34	−49·38	−1·227	−3·290	+4·749	+3·810

Section: 18×6×55 lb. I or *16×6×50 lb. I

Table of B.M.'s and Reactions, for 20′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−20·98	+ 4·73	+7·647	+ 4·73	−20·98	−1·049	−1·049	+2·276	+2·276
W_2	− 0·96	+24·40	+3·145	−14·76	−32·84	−0·047	−1·641	+2·448	+1·559
W_3	+21·89	+ 9·84	−2·097	−16·27	+ 0·64	+1·094	+0·031	−1·039	−1·632
W_4	+ 8·17	+ 3·08	−1·984	− 3·33	− 4·68	+0·569	−0·234	−0·161	+0·161
Total	−20·98	+42·05	+8·808	−29·63	−58·50	−1·049	−2·924	+4·563	+3·996

Section: 20×6½×65 lb. I or *16×6×50 lb. I

Table of B.M.'s and Reactions, for 25′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W_1	−21·53	+ 5·83	+10·20	+ 5·83	−21·53	−0·861	−0·861	+2·276	+2·276
W_2	+ 2·63	+27·23	+ 5·23	−15·95	−37·22	+0·106	−1·488	+2·348	+1·659
W_3	+24·81	+10·63	− 3·49	−18·16	− 1·75	+0·992	−0·071	−1·106	−1·565
W_4	+13·35	+ 5·08	− 3·19	− 5·92	− 8·65	+0·792	−0·346	−0·275	+0·275
Total	−21·53	+48·77	+12·24	−34·20	−69·15	+1·029	−2·766	+4·349	+4·210

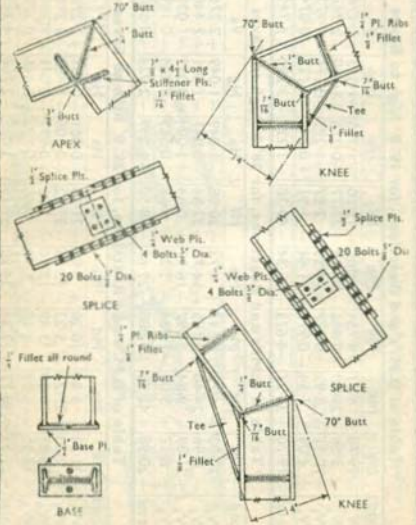
TWO-PIN RIGID FRAME

NORTH LIGHT TYPE

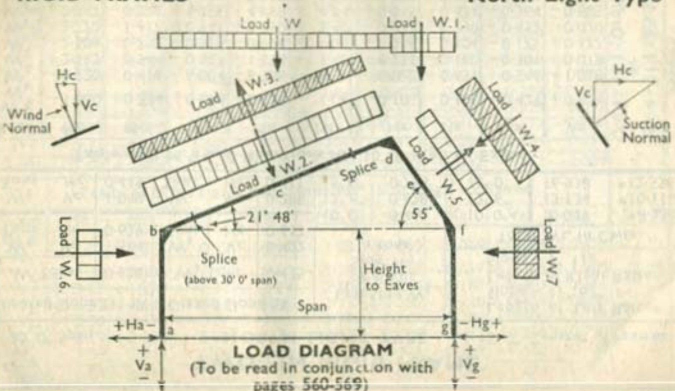
Typical Details

20′ 0″ and 25′ 0″ spans

See pages 560-563



RIGID FRAMES — North Light Type



LOAD DIAGRAM
(To be read in conjunction with pages 560-569)

TWO-PIN RIGID FRAME

20′ 0″ Span — NORTH LIGHT TYPE — 15′ 0″ Centres of Frames

Sheeted Slope 21° 48′			Glazed Slope 55°			Left Side W_6 or Right Side W_7 in tons		Length of Left Rafter Slope = 17′ $6\frac{3}{8}$″ Length of Right Rafter Slope = 7′ $11\frac{3}{8}$″ Total Wt. in Cwts.		
W Tons		0·628	W_1 Tons		0·425					
W_2 Tons	Vc Hc	1·574 0·629	W_4 Tons	Vc Hc	0·432 0·617					
W_3 Tons	Vc Hc	1·049 0·419	W_5 Tons	Vc Hc	0·288 0·411	10′ 0″ 12′ 6″ 15′ 0″	0·460 0·628 0·795	10′ 0″ 12′ 6″ 15′ 0″	10·009 13·134 16·938	*8·259 *10·116 *12·536

Table of B.M.'s and Reactions for 10′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg	Section
W & W_1	−1·060	+0·734	+0·074	−0·258	− 1·06	−0·106	−0·106	+0·429	+0·624	9 × 4 × 21 I (B.S.S.) or *7 × 4 × 16 I (Ord.)
W_2	−0·220	+0·416	−6·084	−6·296	− 6·51	−0·022	−0·651	+0·546	+1·028	
W_5	+2·513	+0·944	−0·627	−1·593	− 1·60	+0·251	−0·160	−0·301	+0·013	
W_6	+2·506	+0·792	−0·916	−0·978	− 1·04	+0·356	−0·104	−0·177	+0·177	
W_4	−3·770	−1·417	+0·941	+2·390	+ 2·40	−0·377	+0·240	+0·452	−0·020	
W_3	+0·147	−0·278	+4·056	+4·198	+ 4·34	+0·015	+0·434	−0·364	−0·685	
W_7	−2·530	−1·938	−1·339	−0·168	+ 1·02	−0·253	+0·207	+0·177	−0·177	
Total	−7·360	−2·899	−7·553	−9·125	−10·21	−0·736	−1·021	+1·058	+1·842	

Section

10 × 4½ × 25 I (B.S.S.)
or
*8 × 4 × 18 I (Ord.)

Table of B.M.'s and Reactions for 12′ 6″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W & W_1	− 1·06	+0·797	+0·203	− 0·19	− 1·06	−0·085	−0·085	+0·429	+0·624
W_2	+ 0·02	+0·127	−6·921	− 7·38	− 7·84	+0·002	−0·627	+0·468	+1·106
W_5	+ 3·10	+1·119	−0·862	− 1·93	− 2·04	+0·248	−0·164	−0·353	+0·065
W_6	+ 3·94	+1·146	−1·639	− 1·79	− 1·95	+0·472	−0·156	−0·294	+0·294
W_4	− 4·65	−1·679	+1·294	+ 2·90	+ 3·06	−0·372	+0·245	+0·529	−0·097
W_3	− 0·02	−0·085	+4·615	+ 4·92	+ 5·22	−0·001	+0·418	−0·312	−0·737
W_7	− 5·31	−4·344	−3·366	− 1·40	+ 0·58	−0·425	+0·203	+0·294	+0·294
Total	−11·04	−5·311	−9·219	−11·29	−12·89	−0·883	−1·032	+1·252	+2·089

Section

10 × 5 × 30 I (B.S.S.)
or
*9 × 4 × 21 I (Ord.)

Table of B.M.'s and Reactions for 15′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W & W_1	− 1·03	+0·875	+ 0·33	− 0·12	− 1·03	−0·069	−0·069	+0·429	+0·624
W_2	+ 0·18	−0·300	− 7·93	− 8·60	− 9·25	+0·012	−0·617	+0·389	+1·185
W_5	+ 3·67	+1·278	− 1·11	− 2·29	− 2·50	+0·245	−0·167	−0·404	+0·116
W_6	+ 5·63	+1·544	− 2·54	− 2·85	− 3·15	+0·585	−0·210	−0·439	+0·439
W_4	− 5·50	−1·918	+ 1·67	+ 3·43	+ 3·75	−0·367	+0·250	+0·606	−0·174
W_3	− 0·12	+0·200	+ 5·29	+ 5·73	+ 6·17	−0·008	+0·412	−0·259	−0·790
W_7	− 9·39	−7·916	− 6·44	− 3·53	− 0·61	−0·626	+0·169	+0·439	−0·439
Total	−16·04	−8·959	−11·25	−13·86	−15·93	−1·070	−1·063	+1·474	+2·364

TWO-PIN RIGID FRAME

NORTH LIGHT TYPE

25′ 0″ Span — 15′ 0″ Centres of Frames

Sheeted Slope 21° 48′			Glazed Slope 55°			Left Side W_6 or Right Side W_7 in tons		Length of Left Rafter Slope = 21′ 8$\frac{13}{16}$″; Length of Right Rafter Slope = 9′ 10¼″ — Total Wt. in Cwts.		
W Tons		0·785	W_1 Tons		0·531	10′ 0″	0·407	10′ 0″	13·321	*10·446
W_2 Tons	Vc Hc	1·967 0·787	W_4 Tons	Vc Hc	0·540 0·771	12′ 6″	0·575	12′ 6″	14·520	*12·973
W_3 Tons	Vc Hc	1·311 0·525	W_5 Tons	Vc Hc	0·360 0·514	15′ 0″	0·742	15′ 0″	18·670	*15·850

Table of B.M.'s and Reactions for 10′ 0″ high to Eaves

Section: 10 × 4½ × 25 I (B.S.S.) or *8 × 4 × 18 I (Ord.)

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W & W_1	− 1·64	+ 1·039	− 0·13	− 0·51	− 1·64	−0·164	−0·164	+0·536	+0·780
W_2	− 0·73	+ 0·849	− 8·72	− 8·66	− 8·60	−0·073	−0·860	+0·761	+1·206
W_5	+ 3·19	+ 1·261	− 0·67	− 2·06	− 1·95	+0·319	+0·293	−0·324	−0·036
W_6	+ 2·40	+ 0·812	− 0·78	− 0·81	− 0·84	+0·323	−0·084	−0·130	+0·130
W_4	− 4·78	− 1·891	+ 1·00	+ 3·10	+ 2·93	−0·478	−0·195	+0·487	+0·053
W_3	+ 0·49	− 0·566	+ 5·82	− 5·78	+ 5·73	+0·049	+0·573	−0·507	−0·804
W_7	− 2·13	− 1·692	− 1·26	− 0·07	+ 1·11	−0·213	+0·194	+0·130	−0·130
Total	− 8·55	+ 3·961	−10·30	−12·04	−13·03	−0·855	−1·108	+1·167	+2·116

Section

$10 \times 4\frac{1}{2} \times 25$ I (B.S.S.)
or
*$9 \times 4 \times 21$ I (Ord.)

Table of B.M.'s and Reactions for 12′ 6″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W & W_1	− 1·65	+1·154	+ 0·11	− 0·39	− 1·65	−0·132	−0·132	+0·536	+0·780
W_2	− 0·32	+0·663	− 9·49	− 9·83	−10·16	−0·026	−0·813	+0·683	+1·284
W_5	+ 3·93	+1·485	− 0·97	− 2·47	− 2·49	+0·314	−0·200	−0·376	+0·016
W_6	+ 3·90	+1·233	− 1·45	− 1·54	− 1·64	+0·444	−0·131	−0·222	+0·222
W_4	− 5·90	−2·227	+ 1·45	+ 3·71	+ 3·74	−0·472	+0·299	+0·565	−0·025
W_3	+ 0·22	−0·442	+ 6·33	+ 6·55	+ 6·77	+0·018	+0·542	−0·455	−0·856
W_7	− 4·82	−4·165	− 3·51	− 1·39	+ 0·72	−0·386	+0·189	+0·222	−0·222
Total	−12·37	−5·680	−11·80	−14·23	−15·94	−0·990	−1·276	+1·323	+2·302

Section

$10 \times 5 \times 30$ I (B.S.S.)
or
*$10 \times 4\frac{1}{2} \times 25$ I (Ord.)

Table of B.M.'s and Reactions for 15′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W & W_1	− 1·65	+1·240	+ 0·28	− 0·31	− 1·65	−0·110	−0·110	+0·536	+0·780
W_2	− 0·01	+0·317	−10·52	−11·17	−11·82	−0·001	−0·788	+0·604	+1·363
W_5	+ 4·65	+1·681	− 1·29	− 2·92	− 3·06	+0·310	−0·204	−0·428	+0·068
W_6	+ 5·67	+1·697	− 2·29	− 2·50	− 2·71	+0·561	−0·181	−0·336	+0·336
W_4	− 6·97	−2·522	+ 1·93	+ 4·38	+ 4·59	−0·465	+0·306	+0·642	−0·102
W_3	+ 0·01	−0·212	+ 7·01	+ 7·45	+ 7·88	+0·001	+0·526	−0·402	−0·909
W_7	− 8·94	−7·987	− 7·05	− 3·79	− 0·55	−0·596	+0·146	+0·336	−0·336
Total	−17·56	−9·481	−13·82	−16·90	−19·24	−1·171	−1·283	+1·514	+2·547

TWO-PIN RIGID FRAME

30′ 0″ Span — NORTH LIGHT TYPE — 15′ 0″ Centres of Frames

Sheeted Slope 21° 48′			Glazed Slope 55°			Left Side W_6 or Right Side W_7 in Tons		Length of Left Rafter Slope = 26′ 1″ Length of Right Rafter Slope = 11′ 9⅞″		
W Tons		0·942	W_1 Tons		0·637			Total Wt. in Cwts.		
W_2 Tons	Vc Hc	2·360 0·944	W_4 Tons	Vc Hc	0·648 0·925					
W_3 Tons	Vc Hc	1·573 0·630	W_5 Tons	Vc Hc	0·432 0·617	10′ 0″ 12′ 6″ 15′ 0″	0·355 0·523 0·690	10′ 0″ 12′ 6″ 15′ 0″	15·152 19·090 20·536	*13·100 *16·700 *17·938

Table of B.M.'s and Reactions for 10′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg	Section
W & W_1	−2·280	+1·432	− 0·36	− 0·80	− 2·28	−0·228	−0·228	+0·644	+0·935	10 × 4½ × 25 I (B.S.S.) or *9 × 4 × 21 I (Ord.)
W_2	−1·490	+1·231	−12·10	−11·51	−10·93	−0·149	−1·093	+0·976	+1·384	
W_5	+3·860	+1·575	− 0·70	− 2·58	− 2·31	+0·386	−0·231	−0·349	−0·083	
W_6	+2·252	+0·789	− 0·66	− 0·67	− 0·67	+0·288	−0·067	−0·097	+0·097	
W_4	−5·790	−2·363	+ 1·06	+ 3·88	+ 3·46	−0·579	+0·346	+0·524	+0·124	
W_3	+0·993	−0·821	+ 8·06	+ 7·67	+ 7·29	+0·099	+0·729	−0·651	−0·923	
W_7	−1·710	−1·375	− 1·03	+ 0·08	+ 1·21	−0·171	+0·184	+0·097	−0·097	
Total	−9·780	+5·027	−13·82	−15·56	−16·19	−0·978	−1·619	+1·523	+2·416	

Section

$10 \times 5 \times 30$ I (B.S.S.)
or
*$10 \times 4\frac{1}{2} \times 25$ I (Ord.)

Table of B.M.'s and Reactions for 12' 6" high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W & W_1	− 2·36	+1·533	− 0·08	− 0·70	− 2·36	−0·189	−0·189	+0·644	+0·935
W_2	− 0·92	+1·221	−12·67	−12·70	−12·72	−0·074	−1·018	+0·897	+1·463
W_5	+ 4·77	+1·858	− 1·05	− 3·07	− 2·94	+0·381	−0·236	−0·400	−0·032
W_6	+ 3·80	+1·272	− 1·27	− 1·32	− 1·37	+0·413	−0·110	−0·173	+0·173
W_4	− 7·15	−2·788	+ 1·57	+ 4·61	+ 4·41	−0·572	+0·353	+0·601	+0·047
W_3	+ 0·62	−0·814	+ 8·45	+ 8·46	+ 8·48	+0·050	+0·679	−0·598	−0·976
W_7	− 4·21	−3·765	− 3·33	− 1·17	+ 0·96	−0·337	+0·186	+0·173	−0·173
Total	−13·72	+5·884	−15·07	−17·79	−19·39	−1·098	−1·553	+1·418	+2·571

Section

$12 \times 5 \times 30$ I (B.S.S.)
or
*$10 \times 4\frac{1}{2} \times 25$ I (Ord.)

Table of B.M.'s and Reactions for 15' 0" high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W & W_1	− 2·38	+1·651	+ 0·18	− 0·58	− 2·38	−0·159	−0·159	+0·644	+0·935
W_2	− 0·46	+0·965	−13·66	−14·14	−14·62	−0·031	−0·975	+0·819	+1·541
W_5	+ 5·65	+2·120	− 1·41	− 3·58	− 3·60	+0·377	−0·240	−0·452	+0·020
W_6	+ 5·62	+1·772	− 2·08	− 2·22	− 2·35	+0·533	−0·157	−0·266	+0·266
W_4	− 8·47	−3·179	+ 2·11	+ 5·38	+ 5·40	−0·565	+0·360	+0·678	−0·030
W_3	+ 0·31	−0·644	+ 9·10	− 9·43	+ 9·75	+0·020	+0·650	−0·546	−1·028
W_7	− 8·19	−7·632	− 7·08	− 3·64	− 0·21	−0·546	+0·144	+0·266	−0·266
Total	−19·04	−9·804	−16·97	−20·52	−22·95	−1·270	−1·531	+1·588	+2·762

TWO-PIN RIGID FRAME

35′ 0″ Span — NORTH LIGHT TYPE — 15′ 0″ Centres of Frames

Sheeted Slope 21° 48′			Glazed Slope 55°			Left Side W_6 or Right Side W_7 in tons		Length of Left Rafter Slope = 30′ $4\frac{5}{16}$″ Length of Right Rafter Slope = 13′ $9\frac{3}{16}$″		
W Tons		1·190	W_1 Tons		0·769			Total Wt. in Cwts.		
W_2 Tons	Vc Hc	2·754 1·101	W_4 Tons	Vc Hc	0·756 1·079					
W_3 Tons	Vc Hc	1·836 0·734	W_5 Tons	Vc Hc	0·504 0·719	10′ 0″ 12′ 6″ 15′ 0″	0·302 0·470 0·637	10′ 0″ 12′ 6″ 15′ 0″	19·340 22·295 25·857	*16·982 *18·286 *22·616

Table of B.M.'s and Reactions for 10′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W & W_1	− 3·21	+2·035	− 0·87	− 1·30	− 3·21	−0·321	−0·321	+0·809	+1·150
W_2	− 2·53	+1·459	−16·39	−14·96	−13·54	−0·253	−1·354	+1·191	+1·563
W_5	+ 4·54	+1·919	− 0·70	− 3·14	− 2·65	+0·454	−0·265	−0·373	−0·131
W_6	+ 2·03	+0·737	− 0·55	− 0·54	− 0·53	+0·249	−0·053	−0·073	+0·073
W_4	− 6·81	−2·878	+ 1·04	+ 4·71	+ 3·98	−0·681	+0·398	+0·560	+0·196
W_3	+ 1·69	−0·975	+10·93	+ 9·97	+ 9·03	+0·169	+0·903	−0·794	−1·042
W_7	− 1·31	−1·028	− 0·74	+ 0·25	+ 1·25	−0·131	+0·171	+0·073	−0·073
Total	−11·33	+6·150	−18·51	−19·94	−19·93	−1·133	−1·993	+1·927	+2·786

Section: 12 × 5 × 30 I (B.S.S.) or *10 × $4\frac{1}{2}$ × 25 I (Ord.)

Section

$12 \times 5 \times 32$ I (B.S.S.)
or
*$10 \times 4\frac{1}{2} \times 25$ I (Ord.)

Table of B.M.'s and Reactions for 12′ 6″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W & W_1	− 3·35	+ 2·18	− 0·43	− 1·15	− 3·35	−0·268	−0·268	+0·809	+1·150
W_2	− 1·81	+ 1·70	−16·65	−16·11	−15·57	−0·145	−1·246	+1·113	+1·641
W_5	+ 5·61	+ 2·26	− 1·09	− 3·70	− 3·38	+0·449	−0·271	−0·424	−0·080
W_6	+ 3·62	+ 1·25	− 1·11	− 1·13	− 1·15	+0·378	−0·092	−0·136	+0·136
W_4	− 8·41	− 3·38	+ 1·63	+ 5·55	+ 5·07	−0·673	+0·406	+0·637	+0·119
W_3	+ 1·21	− 1·13	+11·10	+10·74	+10·38	+0·097	+0·831	−0·742	−1·094
W_7	− 3·56	− 3·26	− 2·95	− 0·88	+ 1·21	−0·285	+0·185	+0·136	−0·136
Total	−15·32	+ 7·39	−19·28	−22·09	−23·45	−1·226	−1·877	+1·582	+2·927

Section

$13 \times 5 \times 35$ I (B.S.S.)
or
*$12 \times 5 \times 30$ I (Ord.)

Table of B.M.'s and Reactions for 15′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W & W_1	− 3·43	+2·313	− 0·09	− 1·02	− 3·43	−0·229	−0·229	+0·809	+1·150
W_2	− 1·18	+1·608	−17·45	−17·57	−17·70	−0·079	−1·180	+1·034	+1·720
W_5	+ 6·66	+2·580	− 1·49	− 4·28	− 4·13	+0·444	−0·275	−0·474	−0·028
W_6	+ 5·49	+1·802	− 1·88	− 1·96	− 2·04	+0·501	−0·136	−0·215	+0·215
W_4	− 9·99	−3·870	+ 2·24	+ 6·42	+ 6·19	−0·666	+0·413	+0·714	+0·042
W_3	+ 0·79	−1·072	+11·63	+11·71	+11·80	+0·053	+0·787	−0·690	−1·147
W_7	− 7·30	−7·029	− 6·75	− 3·27	+ 0·23	−0·487	+0·150	+0·215	−0·215
Total	−20·72	−9·658	−20·91	−24·83	−27·30	−1·382	−1·820	+1·738	+3·085

TWO-PIN RIGID FRAME

40′ 0″ Span — NORTH LIGHT TYPE — 15′ 0″ Centres of Frames

Sheeted Slope 21°48′			Glazed Slope 55°		
W Tons		1·360	W_1 Tons		0·879
W_2 Tons	Vc Hc	3·147 1·259	W_4 Tons	Vc Hc	0·864 1·234
W_3 Tons	Vc Hc	2·098 0·839	W_5 Tons	Vc Hc	0·576 0·823

Left Side W_6 or Right Side W_7 in tons	
10′ 0″	0·250
12′ 6″	0·418
15′ 0″	0·585

Length of Left Rafter Slope – 34′ $7\frac{5}{8}$″
Length of Right Rafter Slope – 15′ $8\frac{7}{16}$″

Total Wt. in Cwts.		
10′ 0″	24·696	*21·464
12′ 6″	26·241	*23·018
15′ 0″	31·920	*24·482

Table of B.M.'s and Reactions for 10′ 0″ high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W & W_1	− 4·03	+2·607	− 1·41	− 1·76	− 4·03	−0·403	−0·403	+0·925	+1·314
W_2	− 3·74	+1·662	−21·52	−18·92	−16·33	−0·374	−1·633	+1·407	+1·740
W_5	+ 5·22	+2·284	− 0·66	− 3·75	− 3·01	+0·522	−0·300	−0·397	−0·179
W_6	+ 1·78	+0·675	− 0·44	− 0·42	− 0·41	+0·209	−0·041	−0·055	+0·055
W_4	− 7·83	−3·427	+ 1·00	+ 5·63	+ 4·51	−0·783	+0·451	+0·595	+0·269
W_3	+ 2·49	−1·108	+14·35	+12·62	+10·89	+0·250	+1·089	−0·938	−1·160
W_7	− 0·96	−0·701	− 0·45	+ 0·40	+ 1·23	−0·096	+0·154	+0·055	−0·055
Total	−12·82	+7·228	−24·03	−24·85	−23·78	−1·282	−2·377	+2·277	+3·109

Section
13 × 5 × 35 I (B.S.S.)
or
*12 × 5 × 30 I (Ord.)

Section

$13 \times 5 \times 35$ I (B.S.S.)
or
*$12 \times 5 \times 30$ I (Ord.)

Table of B.M.'s and Reactions for 12' 6" high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W & W_1	− 4·26	+2·762	− 0·87	− 1·61	− 4·26	−0·341	−0·341	+0·925	+1·314
W_2	− 2·92	+2·116	−21·41	−20·04	−18·66	−0·234	−1·493	+1·328	+1·819
W_5	+ 6·44	+2·663	− 1·13	− 4·41	− 3·84	+0·515	−0·308	−0·448	−0·128
W_6	+ 3·39	+1·213	− 0·97	− 0·97	− 0·96	+0·341	−0·077	−0·109	+0·109
W_4	− 9·66	−3·994	+ 1·70	+ 6·61	+ 5·76	−0·773	+0·461	+0·672	+0·192
W_3	+ 1·95	−1·410	+14·27	+13·36	+12·44	+0·156	+0·995	−0·886	−1·212
W_7	− 2·92	−2·685	− 2·45	− 0·50	+ 1·43	−0·234	+0·184	+0·109	−0·109
Total	−16·84	+8·754	−24·38	−27·03	−27·72	−1·348	−2·219	+2·144	+3·242

Section

$14 \times 5\frac{1}{2} \times 40$ I (B.S.S.)
or
*$12 \times 5 \times 30$ I(Ord.)

Table of B.M.'s and Reactions for 15' 0" high to Eaves

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
W & W_1	− 4·42	+ 2·89	− 0·46	− 1·48	− 4·42	−0·295	−0·295	+0·925	+1·314
W_2	− 2·11	+ 2·27	−21·90	−21·45	−21·00	−0·141	−1·400	+1·249	+1·898
W_5	+ 7·68	+ 3·07	− 1·56	− 5·02	− 4·66	+0·512	−0·311	−0·500	−0·076
W_6	+ 5·31	+ 1·83	− 1·67	− 1·71	− 1·75	+0·468	−0·117	−0·177	+0·177
W_4	−11·52	− 4·60	+ 2·34	+ 7·53	+ 6·99	−0·768	+0·466	+0·750	+0·114
W_3	+ 1·41	− 1·51	+14·60	+14·30	+14·00	+0·094	+0·934	−0·833	−1·265
W_7	− 6·39	− 6·29	− 6·19	− 2·75	+ 0·68	−0·426	+0·159	+0·177	−0·177
Total	−22·33	+10·06	−25·59	−29·66	−31·83	−1·489	−2·123	+1·997	+3·389

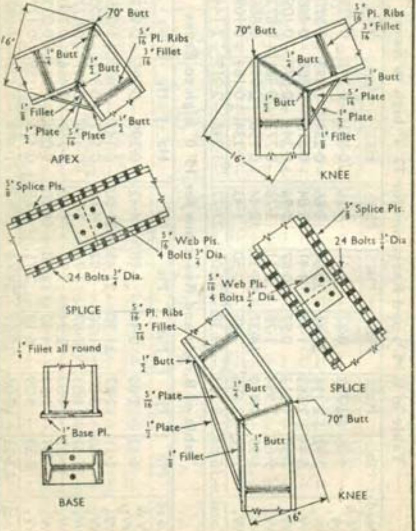
TWO-PIN RIGID FRAME

NORTH LIGHT TYPE

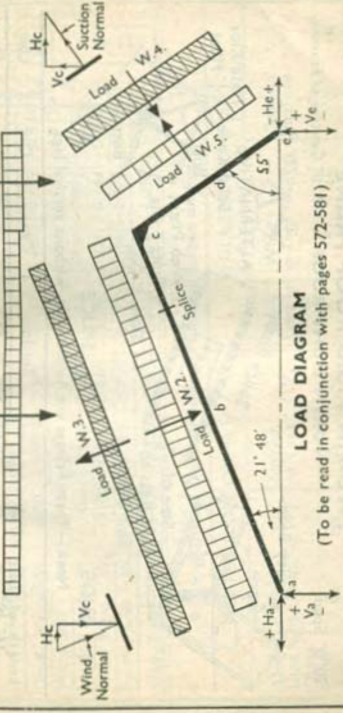
Typical Details

30′ 0″, 35′ 0″ and 40′ 0″ spans

See pages 564-569



RIGID FRAMES North Light Type, Rafters only

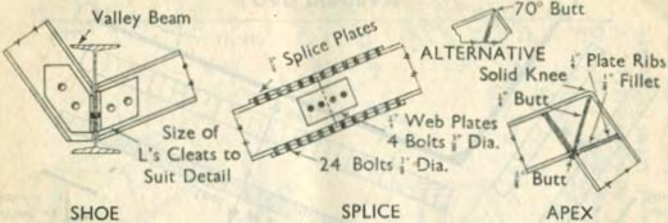


LOAD DIAGRAM

(To be read in conjunction with pages 572-581)

TWO-PIN RIGID ROOF FRAME

20′ 0″ Span — NORTH LIGHT ROOF — 15′ 0″ Centres of Frames



Note.—These Frames are Rafters only, without vertical legs.

	Section	Cwts.	Qrs.	Lbs.
Length of Left Rafter Slope —16′ 10″.	5 × 3 × 11 I	3	1	13
Length of Right Rafter Slope —7′ 7$\frac{9}{16}$″.	•5 × 2½ × 9 I	2	3	15

Section	
B.S.S. requirements	5 × 3 × 11 I
Ordinary requirements	*5 × $2\frac{1}{2}$ × 9 I

Table of Bending Moments and Reactions (Ft. Tons and Tons)

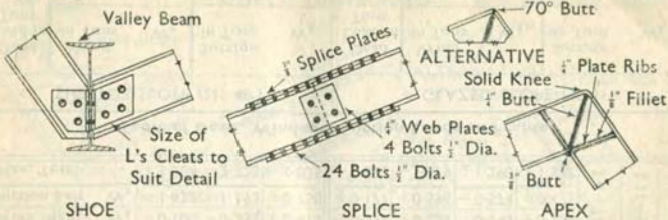
Loads	Mb	Mc	Md	Ha	He	Va	Ve
Dead: W and W_1	+0·803	−0·934	−0·291	−0·437	−0·437	+0·429	+0·624
Wind Left W_2	+2·435	−1·791	−1·137	−0·265	−0·894	+0·861	+0·713
Suction Right W_5	+0·066	+0·154	−0·407	+0·264	−0·148	−0·095	−1·193
Wind Right W_4	−0·100	−0·232	+0·610	−0·396	+0·221	+0·143	+0·289
Suction Left W_3	−1·623	+1·193	+0·758	+0·177	+0·596	−0·574	−0·475
Max. Total - -	+3·304	−2·725	−1·835	−0·833	−1·479	+1·290	+1·337

Table of Dead, Wind and Suction Loads on Frame

SHEETED SLOPE (21° 48′)					GLAZED SLOPE (55°)				
Dead Load in Tons W	Wind in Tons W_2		Suction in Tons W_3		Dead Load in Tons W_1	Wind in Tons W_4		Suction in Tons W_5	
	Vc	Hc	Vc	Hc		Vc	Hc	Vc	Hc
0·628	1·574	0·629	1·049	0·419	0·425	0·432	0·617	0·288	0·411

TWO-PIN RIGID ROOF FRAME

25' 0" Span — NORTH LIGHT ROOF — 15' 0" Centres of Frames



Note.—These Frames are Rafters only, without vertical legs.

	Section	Cwts.	Qrs.	Lbs.
Length of Left Rafter Slope $= 21' \, 0\frac{3}{8}''$.	$7 \times 3\frac{1}{2} \times 15$ I	5	0	19
Length of Right Rafter Slope $= 9' \, 6\frac{7}{16}''$.	*$5 \times 3 \times 11$ I	4	0	11

Table of Bending Moments and Reactions (Ft. Tons and Tons)

Loads	Mb	Mc	Md	Ha	He	Va	Ve
Dead: W and W_1	+1·258	−1·445	−0·298	−0·545	−0·545	+0·536	+0·780
Wind Left W_2	+3·815	−2·773	−1·766	−0·330	−1·117	+1·076	+0·891
Suction Right W_5	+0·102	+0·243	−0·568	+0·330	−0·184	−0·119	−0·241
Wind Right W_4	−0·153	−0·364	+0·852	−0·495	+0·276	+0·179	+0·361
Suction Left W_3	−2·543	+1·849	+1·177	+0·220	+0·745	−0·717	−0·594
Max. Total -	+5·175	−4·218	−2·632	−1·040	−1·846	+1·612	+1·671

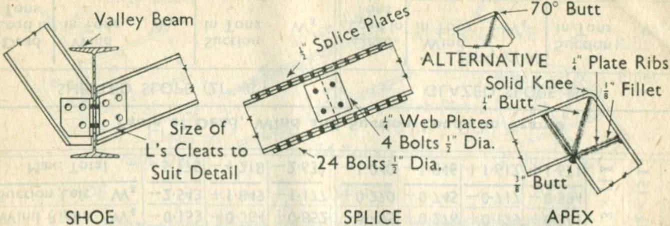
Section	
B.S.S. requirements	$7 \times 3\frac{1}{2} \times 15$ I
Ordinary requirements	*$5 \times 3 \times 11$ I

Table of Dead, Wind and Suction Loads on Frame

SHEETED SLOPE (21° 48′)					GLAZED SLOPE (55°)				
Dead Load in Tons W	Wind in Tons W_2		Suction in Tons W_3		Dead Load in Tons W_1	Wind in Tons W_4		Suction in Tons W_5	
	Vc	Hc	Vc	Hc		Vc	Hc	Vc	Hc
0·785	1·967	0·787	1·311	0·525	0·531	0·540	0·771	0·360	0·514

TWO-PIN RIGID ROOF FRAME

30′ 0″ Span — **NORTH LIGHT ROOF** — 15′ 0″ Centres of Frames



Note.—These Frames are Rafters only, without vertical legs.

	Section	Cwts.	Qrs.	Lbs.
Length of Left Rafter Slope $=25' \, 2\frac{7}{8}''$.	$8 \times 4 \times 18$ I	7	0	7
Length of Right Rafter Slope $=11' \, 5\frac{5}{16}''$.	$*7 \times 3\frac{1}{2} \times 15$ I	6	0	27

Section	
B.S.S. requirements	8 × 4 × 18 I
Ordinary requirements	*7 × $3\frac{1}{2}$ × 15 I

Table of Bending Moments and Reactions (Ft. Tons and Tons)

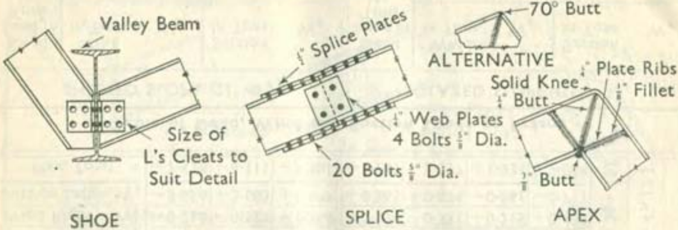
Loads	Mb	Mc	Md	Ha	He	Va	Ve
Dead: W and W_1	+1·808	−2·086	−0·441	−0·655	−0·655	+0·643	+0·935
Wind Left W_2	+5·483	−4·025	−2·549	−0·397	−1·341	+1·291	+1·069
Suction Right W_5	+0·147	+0·352	−0·911	+0·396	−0·221	−0·143	−0·289
Wind Right W_4	−0·220	−0·528	+1·366	−0·594	+0·331	+0·215	+0·433
Suction Left W_3	−3·656	+2·683	+1·699	+0·265	+0·894	−0·861	−0·713
Max. Total -	+7·438	−6·111	−3·901	−1·249	−2·217	+1·934	+2·004

Table of Dead, Wind and Suction Loads on Frame

SHEETED SLOPE (21° 48′)					GLAZED SLOPE (55°)				
Dead Load in Tons W	Wind in Tons W_2		Suction in Tons W_3		Dead Load in Tons W_1	Wind in Tons W_4		Suction in Tons W_5	
	Vc	Hc	Vc	Hc		Vc	Hc	Vc	Hc
0·941	2·36	0·944	1·573	0·629	0·637	0·648	0·925	0·432	0·617

TWO-PIN RIGID ROOF FRAME

35' 0" Span — NORTH LIGHT ROOF — 15' 0" Centres of Frames



Note.—These Frames are Rafters only, without vertical legs.

	Section	Cwts.	Qrs.	Lbs.
Length of Left Rafter Slope $= 29' \ 5\frac{7}{16}''$.	9 × 4 × 21 I	9	1	17
Length of Right Rafter Slope $= 13' \ 4\frac{5}{16}''$.	*7 × 4 × 16 I	7	1	24

Section	
B.S.S. requirements	9 × 4 × 21 I
Ordinary requirements	*7 × 4 × 16 I

Table of Bending Moments and Reactions (Ft. Tons and Tons)

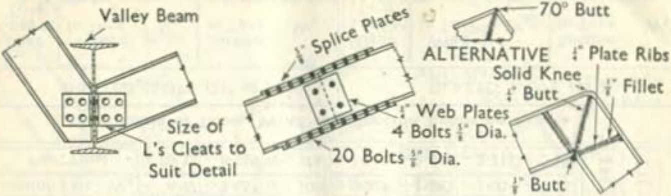
Loads	Mb	Mc	Md	Ha	He	Va	Ve
Dead: W and W_1	+ 2·67	−3·084	−0·676	−0·817	−0·817	+0·809	+1·150
Wind Left W_2	+ 7·46	−5·485	−3·461	−0·464	−1·565	+1·506	+1·248
Suction Right W_5	+ 0·20	+0·481	−1·239	+0·462	−0·258	−0·167	−0·337
Wind Right W_4	− 0·30	−0·722	+1·858	−0·693	+0·386	+0·251	+0·505
Suction Left W_3	− 4·97	+3·657	+2·308	+0·309	+1·043	−1·004	−0·832
Max. Total -	+10·33	−8·569	−5·376	−1·510	−2·640	+2·315	+2·398

Table of Dead, Wind and Suction Loads on Frame

SHEETED SLOPE (21° 48′)					GLAZED SLOPE (55°)				
Dead Load in Tons W	Wind in Tons W_2		Suction in Tons W_3		Dead Load in Tons W_1	Wind in Tons W_4		Suction in Tons W_5	
	Vc	Hc	Vc	Hc		Vc	Hc	Vc	Hc
1·190	2·754	1·101	1·836	0·734	0·769	0·756	1·079	0·504	0·719

TWO-PIN RIGID ROOF FRAME

40′ 0″ Span — NORTH LIGHT ROOF — 15′ 0″ Centres of Frames



SHOE — SPLICE — APEX

Note.—These Frames are Rafters only, without vertical legs.

	Section	Cwts.	Qrs.	Lbs.
Length of Left Rafter Slope —33′ $7\frac{15}{16}$″.	$10 \times 4\frac{1}{2} \times 25$ I	12	1	25
Length of Right Rafter Slope —15′ $3\frac{1}{8}$″.	*8 × 4 × 18 I	9	1	7

Section	
B.S.S. requirements	$10 \times 4\frac{1}{2} \times 25$ I
Ordinary requirements	*$8 \times 4 \times 18$ I

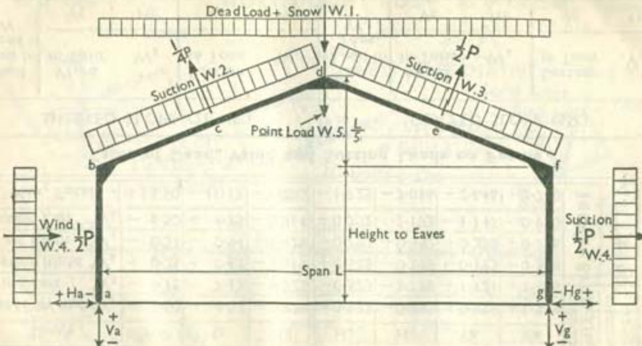
Table of Bending Moments and Reactions (Ft. Tons and Tons)

Loads	Mb	Mc	Md	Ha	He	Va	Ve
Dead: W and W_1	+ 3·50	− 4·01	−0·879	−0·933	−0·933	+0·925	+1·314
Wind Left W_2	+ 9·75	− 7·12	−4·522	−0·529	−1·788	+1·721	+1·426
Suction Right W_5	+ 0·25	+ 0·63	−1·619	+0·528	−0·295	−0·192	−0·384
Wind Right W_4	− 0·37	− 0·94	+2·429	−0·792	+0·442	+0·288	+0·576
Suction Left W_3	− 6·50	+ 4·75	+3·014	+0·352	+1·192	−1·147	−0·950
Max. Total -	+13·50	−11·13	−7·020	−1·725	−3·016	+2·646	+2·740

Table of Dead, Wind and Suction Loads on Frame

SHEETED SLOPE (21° 48′)					GLAZED SLOPE (55°)				
Dead Load in Tons W	Wind in Tons W_2		Suction in Tons W_3		Dead Load in Tons W_1	Wind in Tons W_4		Suction in Tons W_5	
	Vc	Hc	Vc	Hc		Vc	Hc	Vc	Hc
1·360	3·147	1·259	2·098	0·839	0·879	0·864	1·234	0·576	0·823

Two-Pin Rigid Frame. Rise of 1 in 2½ At varying Centres and of Constant Inertia.



Codes of Practice Committee Load Diagram. P = Unit Wind Pressure.

20′ 0″ SPAN

Centre to Centre of Section

Table of Coefficients for calculating **Bending Moments** in Ft.-Tons and **Reactions** in tons to conform to the loading arrangements of the Codes of Practice Committee. (See load diagram, above.)

Height	Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
10′ 0″ high To Eaves	W_1	−1·175	+0·460	+0·840	+0·460	−1·175	−0·118	−0·118	+0·500	+0·500
	W_2	−0·850	−2·042	−0·562	+1·143	+2·834	−0·085	+0·283	−0·472	−0·453
	W_3	+2·834	+1·143	−0·562	−2·042	−0·850	+0·283	−0·085	−0·453	−0·472
	W_4	+5·000	+2·500	0	−2·500	−5·000	+1·000	−1·000	−0·500	+0·500
	W_5	−1·796	+0·345	+2·486	+0·345	−1·796	−0·180	−0·180	+0·500	+0·500
12′ 6″ high To Eaves	W_1	−1·140	+0·560	+1·000	+0·560	−1·140	−0·091	−0·091	+0·500	+0·500
	W_2	−1·339	−2·351	−0·670	+1·302	+3·263	−0·106	+0·262	−0·428	−0·497
	W_3	+3·263	+1·302	−0·670	−2·351	−1·339	+0·262	−0·106	−0·497	−0·428
	W_4	+6·250	+3·125	0	−3·125	−6·250	+1·000	−1·000	−0·625	+0·625
	W_5	−1·745	+0·476	+2·697	+0·476	−1·745	−0·140	−0·140	+0·500	+0·500
15′ 0″ high To Eaves	W_1	−1·106	+0·640	+1·100	+0·640	−1·106	−0·074	−0·074	+0·500	+0·500
	W_2	−1·818	−2·626	−0·756	+1·474	+3·707	−0·122	+0·247	−0·381	−0·545
	W_3	+3·707	+1·474	−0·756	−2·626	−1·818	+0·247	−0·122	−0·545	−0·381
	W_4	+7·500	+3·750	0	−3·750	−7·500	+1·000	−1·000	−0·750	+0·750
	W_5	−1·679	+0·598	+2·874	+0·598	−1·679	−0·112	−0·112	+0·500	+0·500

N.B.—These Coefficients have been calculated for unit Suction, Wind, and Dead Loads of 1 TON per group. In order to calculate the actual Bending Moment and Reaction for each load group it is necessary to work out the actual load, W_1, W_2, W_3, etc., in tons and multiply these by the respective coefficients above.

$\frac{1}{5}$ Rise

TWO-PIN RIGID FRAMES

At Varying Centres and of Constant Inertia

Table of Coefficients for calculating **Bending Moments** in Ft.-Tons and **Reactions** in tons to conform to the loading arrangements of the Codes of Practice Committee. (See load diagram, page 582.)

25′ 0″ SPAN — Centre to Centre of Section

Height	Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
10′ 0″ high To Eaves	W_1	−1·490	+0·500	+0·900	+0·500	−1·490	−0·149	−0·149	+0·500	+0·500
	W_2	−0·591	−2·252	−0·544	+1·265	+3·083	−0·060	+0·308	−0·510	−0·414
	W_3	+3·083	+1·265	−0·544	−2·252	−0·591	+0·308	−0·060	−0·414	−0·510
	W_4	+5·000	+2·500	0	−2·500	−5·000	+1·000	−1·000	−0·400	+0·400
	W_5	−2·277	+0·279	+2·835	+0·279	−2·277	−0·228	−0·228	+0·500	+0·500
12′ 6″ high To Eaves	W_1	−1·475	+0·575	+1·050	+0·575	−1·475	−0·118	−0·118	+0·500	+0·500
	W_2	−1·047	−2·533	−0·677	+1·431	+3·546	−0·083	+0·284	−0·472	−0·451
	W_3	+3·546	+1·431	−0·677	−2·533	−1·047	+0·284	−0·083	−0·451	−0·472
	W_4	+6·250	+3·125	0	−3·125	−6·250	+1·000	−1·000	−0·500	+0·500
	W_5	−2·246	+0·430	+3·105	+0·430	−2·246	−0·180	−0·180	+0·500	+0·500
15′ 0″ high To Eaves	W_1	−1·440	+0·675	+1·200	+0·675	−1·440	−0·096	−0·096	+0·500	+0·500
	W_2	−1·527	−2·837	−0·776	+1·603	+3·982	−0·101	+0·266	−0·437	−0·487
	W_3	+3·982	+1·603	−0·776	−2·837	−1·527	+0·266	−0·101	−0·487	−0·437
	W_4	+7·500	+3·750	0	−3·750	−7·500	+1·000	−1·000	−0·600	+0·600
	W_5	−2·190	+0·570	+3·330	+0·570	−2·190	−0·146	−0·146	+0·500	+0·500

30′ 0″ SPAN

Centre to Centre of Section

Height	Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
10′ 0″ high To Eaves	W_1	−1·783	+0·510	+0·900	+0·510	−1·783	−0·178	−0·178	+0·500	+0·500
	W_2	−0·325	−2·422	−0·489	+1·439	+3·358	−0·032	+0·336	−0·535	−0·391
	W_3	+3·358	+1·439	−0·489	−2·422	−0·325	+0·336	−0·032	−0·391	−0·535
	W_4	+5·000	+2·500	0	−2·500	−5·000	+1·000	−1·000	−0·333	+0·333
	W_5	−2·725	+0·208	+3·140	+0·208	−2·725	−0·273	−0·273	+0·500	+0·500
12′ 6″ high To Eaves	W_1	−1·782	+0·600	+1·110	+0·600	−1·782	−0·143	−0·143	+0·500	+0·500
	W_2	−0·792	−2·749	−0·697	+1·562	+3·811	−0·064	+0·305	−0·503	−0·423
	W_3	+3·811	+1·562	−0·697	−2·749	−0·792	+0·305	−0·064	−0·423	−0·503
	W_4	+6·250	+3·125	0	−3·125	−6·250	+1·000	−1·000	−0·416	+0·416
	W_5	−2·728	+0·368	+3·463	+0·368	−2·728	−0·218	−0·218	+0·500	+0·500
15′ 0″ high To Eaves	W_1	−1·775	+0·690	+1·260	+0·690	−1·775	−0·118	−0·118	+0·500	+0·500
	W_2	−1·273	−3·068	−0·818	+1·713	+4·251	−0·084	+0·284	−0·474	−0·452
	W_3	+4·251	+1·713	−0·818	−3·068	−1·273	+0·284	−0·084	−0·452	−0·474
	W_4	+7·500	+3·750	0	−3·750	−7·500	+1·000	−1·000	−0·500	+0·500
	W_5	−2·694	+0·517	+3·728	+0·517	−2·694	−0·180	−0·180	+0·500	+0·500

N.B.—These Coefficients have been calculated for unit Suction, Wind, and Dead Loads of 1 TON per group. In order to calculate the actual Bending Moment and Reaction for each load group it is necessary to work out the actual load, W_1, W_2, W_3, etc., in tons and multiply these by the respective coefficients above.

$\frac{1}{5}$ Rise

TWO-PIN RIGID FRAMES

At Varying Centres and of Constant Inertia

Table of Coefficients for calculating **Bending Moments** in Ft.-Tons and **Reactions** in tons to conform to the loading arrangements of the Codes of Practice Committee.
(See load diagram, page 582.)

35′ 0″ SPAN
Centre to Centre of Section

Height	Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
10′ 0″ high To Eaves	W_1	−2·049	+0·522	+0·889	+0·522	−2·049	−0·205	−0·205	+0·500	+0·500
	W_2	−0·106	−2·630	−0·459	+1·560	+3·588	−0·011	+0·358	−0·552	−0·374
	W_3	+3·588	+1·560	−0·459	−2·630	−0·106	+0·358	−0·011	−0·374	−0·552
	W_4	+5·000	+2·500	0	−2·500	−5·000	+1·000	−1·000	−0·286	+0·286
	W_5	−3·145	+0·129	+3·404	+0·129	−3·145	−0·315	−0·315	+0·500	+0·500
12′ 6″ high To Eaves	W_1	−2·085	+0·619	+1·120	+0·619	−2·085	−0·167	−0·167	+0·500	+0·500
	W_2	−0·539	−2·951	−0·646	+1·702	+4·065	−0·043	+0·325	−0·527	−0·399
	W_3	+4·065	+1·702	−0·646	−2·951	−0·539	+0·325	−0·043	−0·399	−0·527
	W_4	+6·250	+3·125	0	−3·125	−6·250	+1·000	−1·000	−0·357	+0·357
	W_5	−3·186	+0·297	+3·780	+0·297	−3·186	−0·255	−0·255	+0·500	+0·500
15′ 0″ high To Eaves	W_1	−2·079	+0·721	+1·320	+0·721	−2·079	−0·139	−0·139	+0·500	+0·500
	W_2	−0·995	−3·254	−0·816	+1·858	+4·530	−0·067	+0·302	−0·500	−0·426
	W_3	+4·530	+1·858	−0·816	−3·254	−0·995	+0·302	−0·067	−0·426	−0·500
	W_4	+7·500	+3·750	0	−3·750	−7·500	+1·000	−1·000	−0·429	+0·429
	W_5	−3·179	+0·455	+4·088	+0·455	−3·179	−0·212	−0·212	+0·500	+0·500

40′ 0″ SPAN

Centre to Centre of Section

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
10′ 0″ high To Eaves									
W_1	−2·306	+0·520	+0·841	+0·520	−2·306	−0·231	−0·231	+0·500	+0·500
W_2	+0·123	−2·821	−0·402	+1·691	+3·788	+0·010	+0·379	−0·565	−0·361
W_3	+3·788	+1·691	−0·402	−2·821	+0·123	+0·379	+0·010	−0·361	−0·565
W_4	+5·000	+2·500	0	−2·500	−5·000	+1·000	−1·000	−0·250	+0·250
W_5	−3·536	+0·050	+3·635	+0·050	−3·536	−0·354	−0·354	+0·500	+0·500
12′ 6″ high To Eaves									
W_1	−2·364	+0·641	+1·121	+0·641	−2·364	−0·189	−0·189	+0·500	+0·500
W_2	−0·300	−3·147	−0·604	+1·851	+4·305	−0·024	+0·344	−0·543	−0·383
W_3	+4·305	+1·851	−0·604	−3·147	−0·300	+0·344	−0·024	−0·383	−0·543
W_4	+6·250	+3·125	0	−3·125	−6·250	+1·000	−1·000	−0·313	+0·313
W_5	−3·621	+0·220	+4·061	+0·220	−3·621	−0·290	−0·290	+0·500	+0·500
15′ 0″ high To Eaves									
W_1	−2·379	+0·760	+1·360	+0·760	−2·379	−0·159	−0·159	+0·500	+0·500
W_2	−0·742	−3·447	−0·806	+1·984	+4·783	−0·049	+0·319	−0·519	−0·407
W_3	+4·783	+1·984	−0·806	−3·447	−0·742	+0·319	−0·049	−0·407	−0·519
W_4	+7·500	+3·750	0	−3·750	−7·500	+1·000	−1·000	−0·375	+0·375
W_5	−3·644	+0·385	+4·413	+0·385	−3·644	−0·243	−0·243	+0·500	+0·500

N.B.—These Coefficients have been calculated for unit Suction, Wind, and Dead Loads of 1 TON per group. In order to calculate the actual Bending Moment and Reaction for each load group it is necessary to work out the actual load, W_1, W_2, W_3, etc., in tons and multiply these by the respective coefficients above.

$\frac{1}{5}$ Rise

TWO-PIN RIGID FRAMES

At Varying Centres and of Constant Inertia

Table of Coefficients for calculating **Bending Moments** in Ft.-Tons and **Reactions** in tons to conform to the loading arrangements of the Codes of Practice Committee.
(See load diagram, page 582.)

45′ 0″ SPAN
Centre to Centre of Section

Height	Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
10′ 0″ high To Eaves	W_1	−2·534	+0·540	+0·891	+0·540	−2·534	−0·253	−0·253	+0·500	+0·500
	W_2	+0·295	−3·038	−0·303	+1·839	+3·978	+0·030	+0·398	−0·576	−0·350
	W_3	+3·978	+1·839	−0·303	−3·038	+0·295	+0·398	+0·030	−0·350	−0·576
	W_4	+5·000	+2·500	0	−2·500	−5·000	+1·000	−1·000	−0·222	+0·222
	W_5	−3·900	−0·030	+3·840	−0·030	−3·900	−0·390	−0·390	+0·500	+0·500
12′ 6″ high To Eaves	W_1	−2·605	+0·675	+1·143	+0·675	−2·605	−0·208	−0·208	+5·000	+0·500
	W_2	−0·108	−3·373	−0·612	+1·940	+4·495	−0·009	+0·359	−0·555	−0·371
	W_3	+4·495	+1·940	−0·612	−3·373	−0·108	+0·359	−0·009	−0·371	−0·555
	W_4	+6·250	+3·125	0	−3·125	−6·250	+1·000	−1·000	−0·278	+0·278
	W_5	−4·033	+0·141	+4·314	+0·141	−4·033	−0·323	−0·323	+0·500	+0·500
15′ 0″ high To Eaves	W_1	−2·672	+0·765	+1·350	+0·765	−2·672	−0·178	−0·178	+0·500	+0·500
	W_2	−0·495	−3·645	−0·746	+2·144	+5·030	−0·033	+0·336	−0·535	−0·391
	W_3	+5·030	+2·144	−0·746	−3·645	−0·495	+0·336	−0·033	−0·391	−0·535
	W_4	+7·500	+3·750	0	−3·750	−7·500	+1·000	−1·000	−0·333	+0·333
	W_5	−4·088	+0·311	+4·710	+0·311	−4·088	−0·273	−0·273	+0·500	+0·500

50′ 0″ SPAN

Centre to Centre of Section

Height	Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
10′ 0″ high To Eaves	W_1	−2·750	+0·600	+0·750	+0·600	−2·750	−0·275	−0·275	+0·500	+0·500
	W_2	+0·485	−3·222	−0·212	+1·978	+4·168	+0·048	+0·417	−0·584	−0·342
	W_3	+4·168	+1·978	−0·212	−3·222	+0·485	+0·417	+0·048	−0·342	−0·584
	W_4	+5·000	+2·500	0	−2·500	−5·000	+1·000	−1·000	−0·200	+0·200
	W_5	−4·239	−0·109	+4·022	−0·109	−4·239	−0·424	−0·424	+0·500	+0·500
12′ 6″ high To Eaves	W_1	−2·878	+0·650	+1·050	+0·650	−2·878	−0·230	−0·230	+0·500	+0·500
	W_2	+0·141	−3·554	−0·443	+2·147	+4·744	+0·011	+0·380	−0·566	−0·360
	W_3	+4·744	+2·147	−0·443	−3·554	+0·141	+0·380	+0·011	−0·360	−0·566
	W_4	+6·250	+3·125	0	−3·125	−6·250	+1·000	−1·000	−0·250	+0·250
	W_5	−4·420	+0·062	+4·544	+0·062	−4·420	−0·354	−0·354	+0·500	+0·500
15′ 0″ high To Eaves	W_1	−2·942	+0·800	+1·350	+0·800	−2·942	−0·196	−0·196	+0·500	+0·500
	W_2	−0·275	−3·856	−0·729	+2·263	+5·249	−0·019	+0·350	−0·547	−0·379
	W_3	+5·249	+2·263	−0·729	−3·856	−0·275	+0·350	−0·019	−0·379	−0·547
	W_4	+7·500	+3·750	0	−3·750	−7·500	+1·000	−1·000	−0·300	+0·300
	W_5	−4·512	+0·234	+4·980	+0·234	−4·512	−0·301	−0·301	+0·500	+0·500

N.B.—These Coefficients have been calculated for unit Suction, Wind, and Dead Loads of 1 TON per group. In order to calculate the actual Bending Moment and Reaction for each load group it is necessary to work out the actual load, W_1, W_2, W_3, etc., in tons and multiply these by the respective coefficients above.

$\frac{1}{5}$ Rise

TWO-PIN RIGID FRAMES

At Varying Centres and of Constant Inertia

Table of Coefficients for calculating **Bending Moments** in Ft.-Tons and **Reactions** in tons to conform to the loading arrangements of the Codes of Practice Committee.
(See load diagram, page 582.)

55′ 0″ SPAN
Centre to Centre of Section

Height	Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
12′6″ high To Eaves	W_1	−3·113	+0·715	+1·045	+0·715	−3·113	−0·249	−0·249	+0·500	+0·500
	W_2	+0·325	−3·739	−0·429	+2·263	+4·929	+0·026	+0·394	−0·574	−0·352
	W_3	+4·929	+2·263	−0·429	−3·739	+0·325	+0·394	+0·026	−0·352	−0·574
	W_4	+6·250	+3·125	0	−3·125	−6·250	+1·000	−1·000	−0·227	+0·227
	W_5	−4·786	−0·017	+4·752	−0·017	−4·786	−0·383	−0·383	+0·500	+0·500
15′0″ high To Eaves	W_1	−3·207	+0·771	+1·320	+0·771	−3·207	−0·214	−0·214	+0·500	+0·500
	W_2	−0·050	−4·040	−0·653	+2·412	+5·474	−0·003	+0·365	−0·557	−0·369
	W_3	+5·474	+2·412	−0·653	−4·040	−0·050	+0·365	−0·003	−0·369	−0·557
	W_4	+7·500	+3·750	0	−3·750	−7·500	+1·000	−1·000	−0·273	+0·273
	W_5	−4·919	+0·153	+5·225	+0·153	−4·919	−0·328	−0·328	+0·500	+0·500
20′0″ high To Eaves	W_1	−3·280	+0·990	+1·815	+0·990	−3·280	−0·164	−0·164	+0·500	+0·500
	W_2	−0·911	−4·680	−1·083	+2·696	+6·455	−0·046	+0·323	−0·523	−0·402
	W_3	+6·455	+2·696	−1·083	−4·680	−0·911	+0·323	−0·046	−0·402	−0·523
	W_4	+10·00	+5·000	0	−5·000	−10·00	+1·000	−1·000	−0·364	+0·364
	W_5	−5·008	+0·490	+5·988	+0·490	−5·008	−0·250	−0·250	+0·500	+0·500

60′ 0″ SPAN

Centre to Centre of Section

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg	
W_1	−3·329	+0·720	+0·960	+0·720	−3·329	−0·266	−0·266	+0·500	+0·500	12′ 6″ high To Eaves
W_2	+0·506	−3·937	−0·315	+2·395	+5·110	+0·041	+0·409	−0·581	−0·345	
W_3	+5·110	+2·395	−0·315	−3·937	+0·506	+0·409	+0·041	−0·345	−0·581	
W_4	+6·250	+3·125	0	−3·125	−6·250	+1·000	−1·000	−0·208	+0·208	
W_5	−5·134	−0·098	+4·938	−0·098	−5·134	−0·411	−0·411	+0·500	+0·500	
W_1	−3·455	+0·780	+1·260	+0·780	−3·455	−0·230	−0·230	+0·500	+0·500	15′ 0″ high To Eaves
W_2	+0·163	−4·269	−0·544	+2·560	+5·688	+0·011	+0·379	−0·566	−0·360	
W_3	+5·688	+2·560	−0·544	−4·269	+0·163	+0·379	+0·011	−0·360	−0·566	
W_4	+7·500	+3·750	0	−3·750	−7·500	+1·000	−1·000	−0·250	+0·250	
W_5	−5·304	+0·074	+5·453	+0·074	−5·304	−0·354	−0·354	+0·500	+0·500	
W_1	−3·567	+1·020	+1·800	+1·020	−3·567	−0·178	−0·178	+0·500	+0·500	20′ 0″ high To Eaves
W_2	−0·666	−4·869	−1·003	+2·851	+6·701	−0·034	+0·335	−0·535	−0·391	
W_3	+6·701	+2·851	−1·003	−4·869	−0·666	+0·335	−0·034	−0·391	−0·535	
W_4	+10·00	+5·000	0	−5·000	−10·00	+1·000	−1·000	−0·333	+0·333	
W_5	−5·450	+0·415	+6·280	+0·415	−5·450	−0·273	−0·273	+0·500	+0·500	

N.B.—These Coefficients have been calculated for unit Suction, Wind, and Dead Loads of 1 TON per group. In order to calculate the actual Bending Moment and Reaction for each load group it is necessary to work out the actual load W_1, W_2, W_3, etc., in tons and multiply these by the respective coefficients above.

TWO-PIN RIGID FRAMES

$\frac{1}{5}$ Rise

At Varying Centres and of Constant Inertia

Table of Coefficients for calculating **Bending Moments** in Ft.-Tons and **Reactions** in tons to conform to the loading arrangements of the Codes of Practice Committee. (See load diagram, page 582.)

65′ 0″ SPAN
Centre to Centre of Section

Height	Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
12′6″ high To Eaves	W_1	−3·541	+0·715	+0·910	+0·715	−3·541	−0·283	−0·283	+0·500	+0·500
	W_2	+0·689	−4·163	−0·232	+2·534	+5·293	+0·056	+0·424	−0·587	−0·340
	W_3	+5·293	+2·534	−0·232	−4·163	+0·689	+0·424	+0·056	−0·340	−0·587
	W_4	+6·250	+3·125	0	−3·125	−6·250	+1·000	−1·000	−0·192	+0·192
	W_5	−5·460	−0·174	+5·112	−0·174	−5·460	−0·437	−0·437	+0·500	+0·500
15′0″ high To Eaves	W_1	−3·689	+0·845	+1·235	+0·845	−3·689	−0·246	−0·246	+0·500	+0·500
	W_2	+0·363	−4·425	−0·492	+2·690	+5·887	+0·024	+0·392	−0·573	−0·353
	W_3	+5·887	+2·690	−0·492	−4·425	+0·363	+0·392	+0·024	−0·353	−0·573
	W_4	+7·500	+3·750	0	−3·750	−7·500	+1·000	−1·000	−0·231	+0·231
	W_5	−5·673	−0·006	+5·660	−0·006	−5·673	−0·378	−0·378	+0·500	+0·500
20′0″ high To Eaves	W_1	−3·840	+1·040	+1·820	+1·040	−3·840	−0·192	−0·192	+0·500	+0·500
	W_2	−0·431	−5·055	−0·954	+2·991	+6·935	−0·021	+0·347	−0·544	−0·382
	W_3	+6·935	+2·991	−0·954	−5·055	−0·431	+0·347	−0·021	−0·382	−0·544
	W_4	+10·00	+5·000	0	−5·000	−10·00	+1·000	−1·000	−0·308	+0·308
	W_5	−5·878	+0·337	+6·551	+0·337	−5·878	−0·294	−0·294	+0·500	+0·500

70′ 0″ SPAN

Centre to Centre of Section

Height	Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
15′ 0″ high To Eaves	W_1	−3·914	+0·840	+1·190	+0·840	−3·914	−0·261	−0·261	+0·500	+0·500
	W_2	+0·544	−4·631	−0·416	+2·825	+6·070	+0·036	+0·405	−0·578	−0·348
	W_3	+6·070	+2·825	−0·416	−4·631	+0·544	+0·405	+0·036	−0·348	−0·578
	W_4	+7·500	+3·750	0	−3·750	−7·500	+1·000	−1·000	−0·214	+0·214
	W_5	−6·024	−0·085	+5·854	−0·085	−6·024	−0·402	−0·402	+0·500	+0·500
20′ 0″ high To Eaves	W_1	−4·099	+1·050	+1·748	+1·050	−4·099	−0·205	−0·205	+0·500	+0·500
	W_2	−0·213	−5·260	−0·918	+3·113	+7·154	−0·011	+0·357	−0·552	−0·374
	W_3	+7·154	+3·113	−0·918	−5·260	−0·213	+0·357	−0·011	−0·374	−0·552
	W_4	+10·00	+5·000	0	−5·000	−10·00	+1·000	−1·000	−0·286	+0·286
	W_5	−6·290	+0·259	+6·807	+0·259	−6·290	−0·315	−0·315	+0·500	+0·500
25′ 0″ high To Eaves	W_1	−4·171	+1·260	+2·240	+1·260	−4·171	−0·167	−0·167	+0·500	+0·500
	W_2	−1·062	−5·886	−1·289	+3·426	+8·147	−0·043	+0·326	−0·526	−0·400
	W_3	+8·147	+3·426	−1·289	−5·886	−1·062	+0·326	−0·043	−0·400	−0·526
	W_4	+12·50	+6·250	0	−6·250	−12·50	+1·000	−1·000	−0·357	+0·357
	W_5	−6·373	+0·593	+7·559	+0·593	−6·373	−0·255	−0·255	+0·500	+0·500

N.B.—These Coefficients have been calculated for unit Suction, Wind, and Dead Loads of 1 TON per group. In order to calculate the actual Bending Moment and Reaction for each load group it is necessary to work out the actual load W_1, W_2, W_3, etc., in tons and multiply these by the respective coefficients above.

$\frac{1}{5}$ Rise

TWO-PIN RIGID FRAMES

At Varying Centres and of Constant Inertia

Table of Coefficients for calculating **Bending Moments** in Ft.-Tons and **Reactions** in tons to conform to the loading arrangements of the Codes of Practice Committee. (See load diagram, page 582.)

75′ 0″ SPAN
Centre to Centre of Section

Height	Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg
15′ 0″ high To Eaves	W_1	−4·123	+0·900	+1·125	+0·900	−4·123	−0·275	−0·275	+0·500	+0·500
	W_2	+0·733	−4·831	−0·295	+2·980	+6·258	+0·049	+0·417	−0·584	−0·342
	W_3	+6·258	+2·980	−0·295	−4·831	+0·733	+0·417	+0·049	−0·342	−0·584
	W_4	+7·500	+3·750	0	−3·750	−7·500	+1·000	−1·000	−0·200	+0·200
	W_5	−6·360	−0·165	+6·030	−0·165	−6·360	−0·424	−0·424	+0·500	+0·500
20′ 0″ high To Eaves	W_1	−4·358	+1·050	+1·725	+1·050	−4·358	−0·218	−0·218	+0·500	+0·500
	W_2	−0·001	−5·470	−0·819	+3·248	+7·366	−0·001	+0·368	−0·560	−0·366
	W_3	+7·366	+3·248	−0·819	−5·470	−0·001	+0·368	−0·001	−0·366	−0·560
	W_4	+10·00	+5·000	0	−5·000	−10·00	+1·000	−1·000	−0·267	+0·267
	W_5	−6·692	+0·174	+7·039	+0·174	−6·692	−0·335	−0·335	+0·500	+0·500
25′ 0″ high To Eaves	W_1	−4·452	+1·275	+2·250	+1·275	−4·452	−0·178	−0·178	+0·500	+0·500
	W_2	−0·836	−6·091	−1·260	+3·553	+8·372	−0·034	+0·335	−0·535	−0·391
	W_3	+8·372	+3·553	−1·260	−6·091	−0·836	+0·335	−0·034	−0·391	−0·535
	W_4	+12·50	+6·250	0	−6·250	−12·50	+1·000	−1·000	−0·333	+0·333
	W_5	−6·813	+0·519	+7·850	+0·519	−6·813	−0·273	−0·273	+0·500	+0·500

80′ 0″ SPAN

Centre to Centre of Section

Load	Mb	Mc	Md	Me	Mf	Ha	Hg	Va	Vg	Height
W_1	−4·327	+0·880	+1·040	+0·880	−4·327	−0·289	−0·289	+0·500	+0·500	15′ 0″ high To Eaves
W_2	+0·892	−5·038	−0·200	+3·093	+6·417	+0·059	+0·428	−0·589	−0·337	
W_3	+6·417	+3·093	−0·200	−5·038	+0·892	+0·428	+0·059	−0·337	−0·589	
W_4	+7·500	+3·750	0	−3·750	−7·500	+1·000	−1·000	−0·188	+0·188	
W_5	−6·680	−0·242	+6·196	−0·242	−6·680	−0·445	−0·445	+0·500	+0·500	
W_1	−4·609	+1·040	+1·680	+1·040	−4·609	−0·231	−0·231	+0·500	+0·500	20′ 0″ high To Eaves
W_2	+0·222	−5·638	−0·727	+3·412	+7·589	+0·011	+0·379	−0·566	−0·360	
W_3	+7·589	+3·412	−0·727	−5·638	+0·222	+0·379	+0·011	−0·360	−0·566	
W_4	+10·00	+5·000	0	−5·000	−10·00	+1·000	−1·000	−0·250	+0·250	
W_5	−7·072	+0·099	+7·270	+0·099	−7·072	−0·354	−0·354	+0·500	+0·500	
W_1	−4·729	+1·280	+2·240	+1·280	−4·729	−0·189	−0·189	+0·500	+0·500	25′ 0″ high To Eaves
W_2	−0·608	−6·294	−1·208	+3·685	+8·601	−0·025	+0·344	−0·543	−0·383	
W_3	+8·601	+3·685	−1·208	−6·294	−0·608	+0·344	−0·025	−0·383	−0·543	
W_4	+12·50	+6·250	0	−6·250	−12·50	+1·000	−1·000	−0·313	+0·313	
W_5	−7·243	+0·440	+8·122	+0·440	−7·243	−0·290	−0·290	+0·500	+0·500	

N.B.—These Coefficients have been calculated for unit Suction, Wind, and Dead Loads of 1 TON per group. In order to calculate the actual Bending Moment and Reaction for each load group it is necessary to work out the actual load W_1, W_2, W_3, etc., in tons and multiply these by the respective coefficients above.

FORMULAS TO FIND HORIZONTAL THRUST *H* on TWO-PIN RIGID FRAME

On the preceding pages, a complete example has been worked out with full calculations of *H* based on the strain energy theory. Formulas, however, have been prepared which eliminate these laborious calculations and at the same time give sufficiently accurate results for horizontal thrust *H* for ordinary design purposes.

The following pages show these formulas for various conditions of loading and have been arranged, as far as can be ascertained at present, to conform to the loading requirements made up by the Codes of Practice Committee and which are about to be published as a further revision to the BSS. 449–1940.

When *H* has been calculated for each system of loads and added algebraically, the bending moment can be obtained as outlined on page 603 (omitting the Crane Loads if not required) and assuming that the leeward Eaves is the point of max. bending moment (which is usually the case).

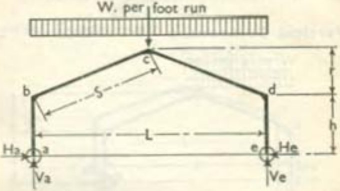
The Vertical reactions are also summated algebraically and, in calculating the Section of the Frame, the Stresses are obtained from both the axial load (vertical reaction Ve) and bending moment—the Combined Stress to be within the permissible limits of the Strut formula (all as pages 603 and 604).

On working out values of *H*, using these formulas and the Codes of Practice Committee loading, and comparing the result with the "*H*" values in our two-pin tables, it will be found that smaller values result. This is due to the different system of loading. We do not advise, however, that our Frame Scantlings be reduced, as in most cases they were fixed in relation to deflection limits and not from the maximum bending moment.

In all cases the formulas are based on the section being of uniform Inertia throughout.

VERTICAL ROOF LOADING

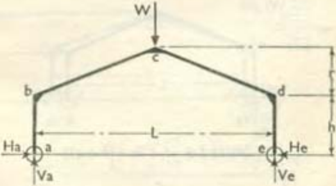
Dead Load and Snow Load
(Distributed)



$$-Ha = -He = \frac{WL^2}{32} \times \frac{(8h+5r)}{y} \qquad Va = Ve = \frac{WL}{2},$$

$$y = h^2\left(3+\frac{h}{s}\right) + r\,(3h+r).$$

Concentrated Load at Ridge



$$-Ha = -He = \frac{WL}{8} \times \frac{(3h+2r)}{y} \qquad Va = Ve = \frac{W}{2}.$$

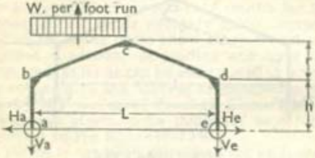
SUCTION ON ROOF

(Windward Slope)

Suction uniformly distributed and acting normal to Roof Slope

Resolved into Vertical and Horizontal Components

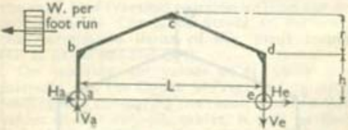
Vertical Component



$$Ha = He = \frac{WL^2}{64} \times \frac{(8h+5r)}{y}$$

$$-Va = \frac{3WL}{8} \qquad -Ve = \frac{WL}{8}$$

Horizontal Component



$$He = \frac{Wr}{16} \times \frac{[8h^2\left(3+\frac{h}{s}\right)+5r\ (4h+r)]}{y}$$

$$-Ha = Wr - He.$$

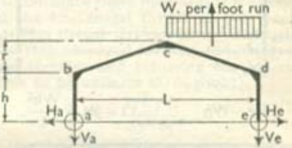
$$Va = -Ve = \frac{Wr(2h+r)}{2L}$$

SUCTION ON ROOF (Leeward Slope)

Suction uniformly distributed and acting normal to Roof Slope

Resolved into Vertical and Horizontal Components

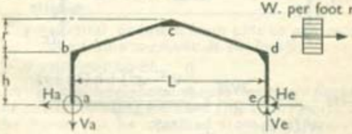
Vertical Component



$$Ha = He = \frac{WL^2}{64} \times \frac{(8h+5r)}{y}$$

$$-Va = \frac{WL}{8} \qquad -Ve = \frac{3WL}{8}.$$

Horizontal Component



$$Ha = \frac{Wr}{16} \times \frac{\left[8h^2\left(3+\frac{h}{s}\right)+5r(4h+r)\right]}{y}$$

$$-He = Wr - Ha.$$

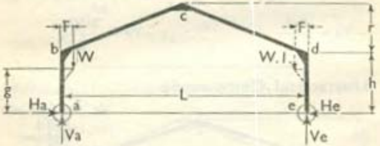
$$-Va = Ve = \frac{Wr\,(2h+r)}{2L}.$$

WIND ON SIDE OF FRAME

Load per 1′ 0″ run W on windward side equals that on Leeward Side

$$Ha = -He = Wh \qquad -Va = Ve = \frac{Wh^2}{L}.$$

CRANE LOADS



$$-Ha = -He = \frac{3WF}{4h} \times \frac{\frac{h}{s}(h^2-g^2)+h\,(2h+r)}{y}$$

$$Va = \frac{W(L-F)}{L} \qquad Ve = \frac{WF}{L}.$$

Note for Load W_1 acting on Vertical Leg $(D-E)$ of frame Horizontal Thrusts Ha and He can be proportioned from Load W.

and $Ve = \dfrac{W_1(L-F)}{L} \qquad Va = \dfrac{W_1F}{L}.$

FULLY WORKED OUT EXAMPLE OF
TWO-PIN RIGID FRAME

The two-pin frame, being without any ties or struts is termed "statically indeterminate" or redundant. The stability of this type of frame is dependent on the stiffness of its joints, and the whole frame is subjected to bending stresses. In order to calculate these moments it is necessary to find the horizontal thrust at the pins (or bases). This thrust (H) can be calculated from the following abridged formula (omitting temperature stresses and assuming the inertia of the frame to be uniform throughout).

$$H = \frac{\Sigma M_1 y ds}{\Sigma y^2 \, ds}.$$

H = the horizontal thrust at the Bases or Pins (in tons).

M_1 = the bending moment (in foot-tons) for vertical and horizontal loads on any portion of the frame, calculated on the basis of the frame being a simply supported beam. (Generally called the **free** bending moment).

ds = a portion or segment of the frame (in feet).

y = vertical ordinate from pins to centre of the frame segment ds (in feet).

Σ = summation of.

When the horizontal thrust has been found, the **actual** bending moment (M) at any point of the frame can be obtained from the formula $M = M_1 - Hy$.

On the following pages a complete example of a two-pin portal frame is worked out having the usual dead, wind, and suction loads, and in addition the vertical and surge loads from a 2-ton hand crane travelling on a gantry resting on brackets fixed to the flanges.

The first step (after a diagram has been drawn dividing the frame into segments (ds) and marking off the ordinates y) is to determine the free bending moments from each of the various load groups W_1, W_2, W_3, etc. The first pull-out sheet gives each step in detail, showing the workings for these moments. These values are then tabulated on the back of this sheet as well as the other various dimensions of the formula.

In the case of the W_2 load, we have divided this into 2 groups, one for the vertical component and the other for the horizontal component, and the algebraic sum of these two values is the combined moment. As far as the suction (W_3) is concerned, this being 10 lbs. against the 15 lb. wind, it is only necessary to take $\frac{2}{3}$ of the combined W_2 loads, reversing the sign in each case and noting always that the segments are also reversed.

It should also be noted that in order to calculate M_1 for each of the horizontal wind, suction, and crane loads it is necessary first of all, in each case, to assume a value H, equal to half the summation of the loads. When this has been done it is then necessary to calculate the actual horizontal thrust, using the original formula on page 601, and it is from this value of H that the actual bending moment ($M_1 - Hy$) is obtained.

Regarding the vertical reactions, these are obtained by the usual method of taking moments about the windward hinge and dividing the result by the span of the frame. For horizontal loads there will be a downward thrust at one pin and an upward thrust at the other, and for vertical loads (these being symmetrical) each pin shares half of the total vertical loads. The algebraic sum of both these values (from the horizontal and vertical loads) gives the vertical reaction (or shear) at each pin.

It will be observed that in this example a negative sign before the values of horizontal reactions H_a and H_u indicates that the reactions act inwards towards the centre of the frame, and likewise a positive sign indicates that the reactions act away from the frame.

The next step is to tabulate the whole of the actual bending moments (M) for each of the loads W_1, W_2, etc., and to add these up algebraically, with or without suction, in order to find the maximum B.M. for each segment of the frame. From this table (which is shown on the second pull-out sheet) the maximum bending moment on the frame as a whole is seen. This maximum B.M. has, however, been calculated at the centre of each segment and, in order to obtain the absolute maximum moment which usually occurs at point P (eaves), it is necessary to make a separate calculation which is as follows :— $(1{\cdot}869 \times 29{\cdot}25) + (1{\cdot}72 \times 1{\cdot}5) + (0{\cdot}09 \times 6{\cdot}25) = 51{\cdot}53$ foot tons.

The frame, as well as resisting bending moment stresses, must also resist direct loads, and the next step is to tabulate these from the various diagrams, in order to arrive at the maximum direct load on the segment carrying the maximum bending moment (in our case at point P). *See pages 604, 605, 606, 607.*

We can now calculate the section.

Absolute Maximum B.M. = 51·53 foot tons

Maximum Direct Load (on
this same segment) = 2·678 tons

Centres of Side Rails at this point = 6′ 3″. Height from Pin to Eaves = 29′ 3″. Assume a 16 × 6 × 50 lbs. R.S. Joist.

Effective height $lyy = 6{\cdot}25 \times 12 = 75$ ins.
Effective height $Lxx = 29{\cdot}25 \times 12 = 351$ ins.

$$\frac{l}{ryy} = \frac{75}{1{\cdot}24} = 60{\cdot}48 \quad \frac{L}{Rxx} = \frac{351}{6{\cdot}48} = 54{\cdot}17.$$

$F_1 = 5{\cdot}87$ tons/sq. in.

Bending stress $= \frac{M}{Zxx} = \frac{51{\cdot}53 \times 12}{77{\cdot}3} = 8{\cdot}0$ tons/sq. in.

Axial stress $= \frac{W}{A} = \frac{2{\cdot}678}{14{\cdot}71} = 0{\cdot}182$ „ „

Total combined stress $= 8{\cdot}182$ tons/sq. in.

Permissible stress F_2 $= 6{\cdot}57$ tons/sq. in.
$33\frac{1}{3}\%$ increase due to wind $= 2{\cdot}19$ „ „

Max. permissible $= 8{\cdot}76$ „ „

The section of $16 \times 6 \times 50$ lbs. R.S. Joist assumed is suitable.

From inspection it is evident that this section is also suitable for the inclined portion, as although the moment is the same, the direct load is smaller, and the Purlin spacing is smaller than 6′ 3″.

Calculation to find the DIRECT AXIAL forces in frame.

To arrive at the direct axial forces at any point on the frame it is necessary to resolve the horizontal and vertical thrusts. Consider the point *P* (at eaves) on the rafter slope.

Fig. I

Fig. II

Direct Axial Force due to Horizontal Thrust (see Fig. I).

$= H \cos \theta$

Horizontal Thrust at Pin $= 1{\cdot}869$ Tons.

FLEMING BROS. STRUCTURAL ENGINEERS AND IRON AND STEEL STOCKHOLDERS, 49 BATH STREET, GLASGOW, C.2.

Table of Calculations of Free Bending Moments (See other pull-out sheet for Dimensions)

TWO-PIN RIGID FRAME

44' 6" Span - 29' 3" High to Eaves - 15' 6" crs. of Frames

Dead Load W.1.

Sheets and Fixings	3·5 lbs./sq. ft.
Steel Angle Purlins	1·5 lbs./sq. ft.
Steel Frame	2·5 lbs./sq. ft.
Total	7·5 lbs./sq. ft.

Total Vertical Dead Load, (Horizontally Projected) $= \frac{45·833 \times 15·5 \times 7·5}{2240} = 2·4$ tons

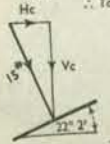
Per Panel $= \frac{2·4}{8} = 0·3$ tons

$V_A = V_U = \frac{2·4}{2} = 1·2$ tons

BM (FG) and (PN) = (1·05 × 2·67) = + 2·8 ft. tons
BM (GH) and (NM) = (1·05 × 8·11) − (0·3 × 2·78) = + 7·68 ft. tons
BM (HJ) and (ML) = (1·05 × 13·67) − (0·3 × 2·78) − (0·3 × 8·34) = + 11·02 ft. tons
BM (JK) and (LK) = (1·05 × 19·35) − (0·3 × 2·9) − (0·3 × 8·46) − (0·3 × 14·02) = + 12·7 ft. tons

Wind on Roof W.2. Wind Normal to roof Slope = 15 lbs. sq. ft.

∴ Total Load (Normal) $= \frac{25·0 \times 15·5 \times 15}{2240} = 2·6$ tons



Vert. comp. = cos 22° 2' × 15 = 13·91 lbs./sq. ft.
Horz. comp. = sin 22° 2' × 15 = 5·63 lbs./sq. ft.

Note.—Horz. and Vert. components are measured on over-all sloping surface.

∴ Vertical Load $= \frac{25·0 \times 15·5 \times 13·91}{2240} = 2·4$ tons

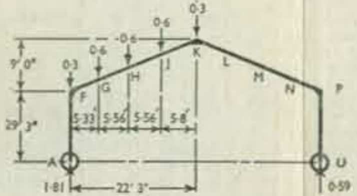
Per Panel $= \frac{2·4}{4} = 0·6$ tons

and Horz. Load $= \frac{25·0 \times 15·5 \times 5·63}{2240} = 0·98$ tons

Per Panel $= \frac{0·98}{4} =$ Approx. 0·25 tons

Wind on Roof (contd.) Vertical Component W.2.

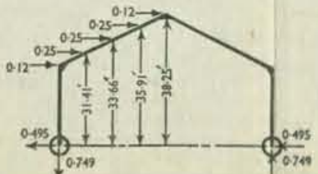
$V_A = \frac{2·4 \times 33·38}{44·5} = 1·81$ tons : $V_U = 2·4 - 1·81 = 0·59$ tons



BM (FG) = (1·51 × 2·67) = + 4·03 ft. tons
BM (GH) = (1·51 × 8·11) − (0·6 × 2·78) = + 10·58 ft. tons
BM (HJ) = (1·51 × 13·67) − (0·6 × 2·78) − (0·6 × 8·34) = + 13·97 ft. tons
BM (JK) = (1·51 × 19·35) − (0·6 × 2·9) − (0·6 × 8·46) − (0·6 × 14·02) = + 13·99 ft. tons

BM (KL) = (0·59 × 19·35) = + 11·42 ft. tons
BM (LM) = (0·59 × 13·67) = + 8·07 ft. tons
BM (MN) = (0·59 × 8·11) = + 4·78 ft. tons
BM (NP) = (0·59 × 2·67) = + 1·58 ft. tons

Wind on Roof W.2. Horizontal Component



Wind on Roof (contd.) Horizontal Component W.2.

Assume Horizontal Reactions Equal

i.e. $+H_A = -H_U = \frac{W}{2} = \frac{0·99}{2} = 0·495$ tons

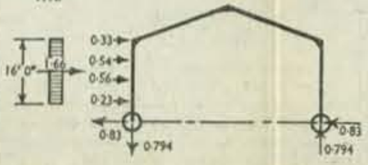
Vertical Reaction $-V_A = +V_U$

$= \frac{(0·12 \times 38·25) + (0·25 \times 35·91) + (0·25 \times 33·66) + (0·25 \times 31·41) + (0·12 \times 29·25)}{44·5}$

$= \frac{33·34}{44·5} = 0·749$ tons

BM (AB) = (0·495 × 3·0) = + 1·49 ft. tons
BM (BC) = (0·495 × 9·0) = + 4·46 ft. tons
BM (CD) = (0·495 × 15·38) = + 7·61 ft. tons
BM (DE) = (0·495 × 20·88) = + 10·34 ft. tons
BM (EF) = (0·495 × 26·13) = + 12·93 ft. tons
BM (FG) = (0·495 × 30·33) − (0·12 × 1·08) − (0·749 × 2·67) = + 12·88 ft. tons
BM (GH) = (0·495 × 32·54) − (0·12 × 3·29) − (0·25 × 1·13) − (0·749 × 8·11) = + 9·36 ft. tons
BM (HJ) = (0·495 × 34·79) − (0·12 × 5·54) − (0·25 × 3·38) − (0·25 × 1·13) − (0·749 × 13·67) = + 5·19 ft. tons
BM (JK) = (0·495 × 37·08) − (0·12 × 7·83) − (0·25 × 5·67) − (0·25 × 3·42) − (0·25 × 1·17) − (0·749 × 19·35) = + 0·36 ft. tons
BM (KL) = (0·749 × 19·35) − (0·495 × 37·08) = − 3·86 ft. tons
BM (LM) = (0·749 × 13·67) − (0·495 × 34·79) = − 6·98 ft. tons
BM (MN) = (0·749 × 8·11) − (0·495 × 32·54) = − 10·03 ft. tons
BM (NP) = (0·749 × 2·67) − (0·495 × 30·33) = − 13·01 ft. tons
BM (PQ) = − (0·495 × 26·13) = − 12·93 ft. tons
BM (QR) = − (0·495 × 20·88) = − 10·34 ft. tons
BM (RS) = − (0·495 × 15·38) = − 7·61 ft. tons
BM (ST) = − (0·495 × 9·0) = − 4·46 ft. tons
BM (TU) = − (0·495 × 3·0) = − 1·49 ft. tons

Wind on Side W.4.



Wind on Side W.4. (contd.)

Max. Wind Pressure 15 lbs./sq. ft. on upper ⅔ Vert. Surface
Height of Building to Apex = 28' 9" + 9' 6" = 38' 3"

38' 3" × ⅔ = 25' 6"

∴ Side exposed to Wind = 25' 6" − 9' 6" = 16' 0"

∴ Wind Load $= \frac{16·0 \times 15·5 \times 15·0}{2240} = 1·66$ tons

$-V_A = +V_U$

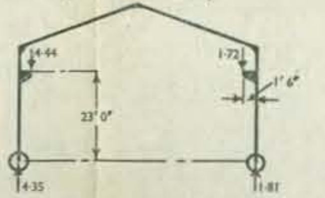
$= \frac{(0·33 \times 29·25) + (0·54 \times 23·0) + (0·56 \times 18·75) + (0·23 \times 12)}{44·5}$

$= \frac{35·33}{44·5} = 0·794$ tons

$+H_A = -H_U = \frac{1·66}{2} = 0·83$ tons

BM (AB) = (0·83 × 3·0) = + 2·49 ft. tons
BM (BC) = (0·83 × 9·0) = + 7·47 ft. tons
BM (CD) = (0·83 × 15·38) − (0·23 × 3·375) = + 11·99 ft. tons
BM (DE) = (0·83 × 20·88) − (0·23 × 8·875) − (0·56 × 2·125) = + 14·10 ft. tons
BM (EF) = (0·83 × 26·13) − (0·23 × 14·125) − (0·56 × 7·375) − (0·54 × 3·125) = + 12·62 ft. tons
BM (FG) = (0·83 × 30·33) − (0·23 × 18·33) − (0·56 × 11·58) − (0·54 × 7·33) − (0·33 × 1·08) − (0·794 × 2·67) = + 8·04 ft. tons
BM (GH) = (0·83 × 32·54) − (0·23 × 20·54) − (0·56 × 13·79) − (0·54 × 9·54) − (0·33 × 3·29) − (0·794 × 8·11) = + 1·89 ft. tons
BM (HJ) = (0·83 × 34·79) − (0·23 × 22·79) − (0·56 × 16·04) − (0·54 × 11·79) − (0·33 × 5·54) − (0·794 × 13·67) = − 4·73 ft. tons
BM (JK) = (0·83 × 37·08) − (0·23 × 25·08) − (0·56 × 18·33) − (0·54 × 14·08) − (0·33 × 7·83) − (0·794 × 19·35) = − 11·14 ft. tons
BM (KL) = (0·794 × 19·35) − (0·83 × 37·08) = − 15·41 ft. tons
BM (LM) = (0·794 × 13·67) − (0·83 × 34·79) = − 18·02 ft. tons
BM (MN) = (0·794 × 8·11) − (0·83 × 32·54) = − 20·57 ft. tons
BM (NP) = (0·794 × 2·67) − (0·83 × 30·33) = − 23·05 ft. tons
BM (PQ) = − (0·83 × 26·13) = − 21·69 ft. tons
BM (QR) = − (0·83 × 20·88) = − 17·33 ft. tons
BM (RS) = − (0·83 × 15·38) = − 12·77 ft. tons
BM (ST) = − (0·83 × 9·0) = − 7·47 ft. tons
BM (TU) = − (0·83 × 3·0) = − 2·49 ft. tons

Vertical Crane Loads W.5.
(From 2 Ton Hand Crane)



Crane Reaction (Left Hand Side) = 4·44 tons
Crane Reaction (Right Hand Side) = 1·72 tons
Assume Depth of Column at Crane Bracket = 16"
Crane Clearance = 10"

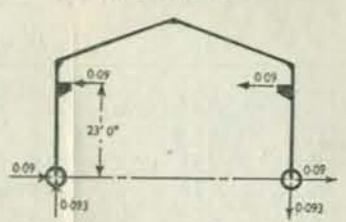
∴ Eccentricity $= \frac{16"}{2} + 10" = 18"$

$V_U = \frac{(4·44 \times 1·5) + (1·72 \times 43·0)}{44·5} = \frac{80·62}{44·5} = 1·81$ tons

$V_A = 6·16 - 1·81 = 4·35$ tons

BM (AB) = 0
BM (BC) = 0
BM (CD) = 0
BM (DE) = 0
BM (EF) = (4·44 × 1·5) = + 6·66 ft. tons
BM (FG) = (4·35 × 2·67) − (4·44 × 1·17) = + 6·42 ft. tons
BM (GH) = (4·35 × 8·11) − (4·44 × 6·61) = + 5·93 ft. tons
BM (HJ) = (4·35 × 13·67) − (4·44 × 12·17) = + 5·43 ft. tons
BM (JK) = (4·35 × 19·35) − (4·44 × 17·85) = + 4·92 ft. tons
BM (KL) = (4·35 × 25·15) − (4·44 × 23·65) = + 4·40 ft. tons
BM (LM) = (4·35 × 30·83) − (4·44 × 29·33) = + 3·89 ft. tons
BM (MN) = (4·35 × 36·39) − (4·44 × 34·89) = + 3·38 ft. tons
BM (NP) = (4·35 × 41·83) − (4·44 × 40·33) = + 2·90 ft. tons
BM (PQ) = (4·35 × 44·5) − (4·44 × 43·0) = + 2·66 ft. tons
BM (QR) = 0
BM (RS) = 0
BM (ST) = 0
BM (TU) = 0

Horizontal Crane Surge Loads W.6.
(From 2 Ton Hand Crane)



$-H_A = +H_U = \frac{2 \times 0·09}{2} = 0·09$ tons

$+V_A = -V_U = \frac{0·18 \times 23·0}{44·5} = 0·093$ tons

BM (AB) = − (0·09 × 3·0) = − 0·27 ft. tons
BM (BC) = − (0·09 × 9·0) = − 0·81 ft. tons
BM (CD) = − (0·09 × 15·38) = − 1·38 ft. tons
BM (DE) = − (0·09 × 20·88) = − 1·88 ft. tons
BM (EF) = (0·09 × 3·125) − (0·09 × 26·13) = − 2·07 ft. tons
BM (FG) = (0·09 × 7·33) − (0·09 × 30·33) + (0·093 × 2·67) = − 1·82 ft. tons
BM (GH) = (0·09 × 9·54) − (0·09 × 32·54) + (0·093 × 8·11) = − 1·32 ft. tons
BM (HJ) = (0·09 × 11·79) − (0·09 × 34·79) + (0·093 × 13·67) = − 0·80 ft. tons
BM (JK) = (0·09 × 14·08) − (0·09 × 37·08) + (0·093 × 19·35) = − 0·27 ft. tons
BM (KL) = (0·09 × 37·08) − (0·09 × 14·08) − (0·093 × 19·35) = + 0·27 ft. tons
BM (LM) = (0·09 × 34·79) − (0·09 × 11·79) − (0·093 × 13·67) = + 0·8 ft. tons
BM (MN) = (0·09 × 32·54) − (0·09 × 9·54) − (0·093 × 8·11) = + 1·32 ft. tons
BM (NP) = (0·09 × 30·33) − (0·09 × 7·33) − (0·093 × 2·67) = + 1·82 ft. tons
BM (PQ) = (0·09 × 26·13) − (0·09 × 3·125) = + 2·07 ft. tons
BM (QR) = (0·09 × 20·88) = + 1·88 ft. tons
BM (RS) = (0·09 × 15·38) = + 1·38 ft. tons
BM (ST) = (0·09 × 9·0) = + 0·81 ft. tons
BM (TU) = (0·09 × 3·0) = + 0·27 ft. tons

Note.—Crane Surge Loads can act in either horizontal direction

Point	y	y^2	ds	yds	y^2ds
AB	3·0	9·0	6·0	18·0	54·0
BC	9·0	81·0	6·0	54·0	486·0
CD	15·38	236·54	6·75	103·82	1596·65
DE	20·88	435·97	4·25	88·74	1852·87
EF	26·13	682·78	6·25	163·31	4267·38
FG	30·33	919·91	5·75	174·40	5289·48
GH	32·54	1058·85	6·0	195·24	6353·10
HJ	34·79	1210·34	6·0	208·74	7262·04
JK	37·08	1374·93	6·25	231·75	8593·31
KL	37·08	1374·93	6·25	231·75	8593·31
LM	34·79	1210·34	6·0	208·74	7262·04
MN	32·54	1058·85	6·0	195·24	6353·10
NP	30·33	919·91	5·75	174·40	5289·48
PQ	26·13	682·78	6·25	163·31	4267·38
QR	20·88	435·97	4/25	88·74	1852·87
RS	15·38	236·54	6·75	103·82	1596·65
ST	9·0	81·0	6·0	54·0	486·0
TU	3·0	9·0	6·0	18·0	54·0

$H = \frac{\Sigma M_1 yds}{\Sigma y^2 ds}$ $\sum +71509·66$

Note.—All Bending Moments tabulated are in Ft.-Tons; all vertical and horizontal reactions are in Tons.

W.1. DEAD LOAD

Point	M_1	M_1 yds	Hy (H = +·2)	M (M_1−Hy)
AB	0	0	+0·60	−0·60
BC	0	0	+1·80	−1·80
CD	0	0	+3·08	−3·08
DE	0	0	+4·18	−4·18
EF	0	0	+5·23	−5·23
FG	+ 2·80	+ 488·32	+6·07	−3·27
GH	+ 7·68	+1499·44	+6·51	+1·17
HJ	+11·02	+2300·31	+6·96	+4·06
JK	+12·70	+2943·23	+7·42	+5·28
KL	+12·70	+2943·23	+7·42	+5·28
LM	+11·02	+2300·31	+6·96	+4·06
MN	+ 7·68	+1499·44	+6·51	+1·17
NP	+ 2·80	+ 488·32	+6·07	−3·27
PQ	0	0	+5·23	−5·23
QR	0	0	+4·18	−4·18
RS	0	0	+3·08	−3·08
ST	0	0	+1·80	−1·80
TU	0	0	+0·60	−0·60

$\sum +14462·60$ $H = \frac{+14462·60}{+71509·66}$

∴ H = +0·202

V_a = +1·2	V_u = +1·2

H_a = 0 − (+0·202) = −0·202

H_u = 0 − (+0·202) = −0·202

W.2. WIND WINDWARD ROOF SLOPE

Point	Vertical Component				Horizontal Component				M Combined
	M_1	M_1 yds	Hy (H = +·2)	M (M_1−Hy)	M_1	M_1 yds	Hy (H = −·02)	M (M_1−Hy)	
AB	0	0	+0·60	−0·60	+ 1·49	+ 26·82	−0·06	+ 1·55	+ 0·95
BC	0	0	+1·80	−1·80	+ 4·46	+ 240·84	−0·18	+ 4·64	+ 2·84
CD	0	0	+3·08	−3·08	+ 7·61	+ 790·07	−0·31	+ 7·92	+ 4·84
DE	0	0	+4·18	−4·18	+10·34	+ 917·57	−0·42	+10·76	+ 6·58
EF	0	0	+5·23	−5·23	+12·93	+2111·60	−0·52	+13·45	+ 8·22
FG	+ 4·03	+ 702·83	+6·07	−2·04	+12·88	+2246·27	−0·61	+13·49	+11·45
GH	+10·58	+2065·64	+6·51	+4·07	+ 9·36	+1827·45	−0·65	+10·01	+14·08
HJ	+13·97	+2916·10	+6·96	+7·01	+ 5·19	+1083·36	−0·70	+ 5·89	+12·90
JK	+13·99	+3242·18	+7·42	+6·57	+ 0·36	+ 83·43	−0·74	+ 1·10	+ 7·67
KL	+11·42	+2646·59	+7·42	+4·0	− 3·86	− 894·56	−0·74	− 3·12	+ 0·88
LM	+ 8·07	+1684·53	+6·96	+1·11	− 6·98	−1457·01	−0·70	− 6·28	− 5·17
MN	+ 4·78	+ 933·25	+6·51	−1·73	−10·03	−1958·26	−0·65	− 9·38	−11·11
NP	+ 1·58	+ 275·55	+6·07	−4·49	−13·[illegible]	−2268·94	−0·61	−12·40	−16·89
PQ	0	0	+5·23	−5·23	−12·93	−2111·60	−0·52	−12·41	−17·64
QR	0	0	+4·18	−4·18	−10·34	− 917·57	−0·42	− 9·92	−14·10
RS	0	0	+3·08	−3·08	− 7·61	− 790·07	−0·31	− 7·30	−10·38
ST	0	0	+1·80	−1·80	− 4·46	− 240·84	−0·18	− 4·28	− 6·08
TU	0	0	+0·60	−0·60	− 1·49	− 26·82	−0·06	− 1·43	− 2·03

Vertical Component: $\sum +14466·67$ $H = \frac{+14466·67}{+71509·66}$

∴ H = +0·202

V_a = +1·81	V_u = +0·59

H_a = 0 − (+0·202) = −0·202

H_u = 0 − (+0·202) = −0·202

Horizontal Component: $\sum -1338·26$ $H = \frac{-1338·26}{+71509·66}$

∴ H = −0·019

V_a = −0·749	V_u = +0·749

H_a = +0·495 − (−0·019) = +0·514

H_u = −0·495 − (−0·019) = −0·476

W.3. SUCTION LEEWARD ROOF SLOPE

Point	⅔ of Combined M Values of opposite Panels on W.2. group	M With Signs reversed
AB	= ⅔ × − 2·03 = − 1·35	+ 1·35
BC	= ⅔ × − 6·08 = − 4·05	+ 4·05
CD	= ⅔ × −10·38 = − 6·92	+ 6·92
DE	= ⅔ × −14·10 = − 9·40	+ 9·40
EF	= ⅔ × −17·64 = −11·76	+11·76
FG	= ⅔ × −16·89 = −11·26	+11·26
GH	= ⅔ × −11·11 = − 7·41	+ 7·41
HJ	= ⅔ × − 5·17 = − 3·45	+ 3·45
JK	= ⅔ × + 0·88 = + 0·59	− 0·59
KL	= ⅔ × + 7·67 = + 5·11	− 5·11
LM	= ⅔ × +12·90 = + 8·60	− 8·60
MN	= ⅔ × +14·08 = + 9·39	− 9·39
NP	= ⅔ × +11·45 = + 7·63	− 7·63
PQ	= ⅔ × + 8·22 = + 5·48	− 5·48
QR	= ⅔ × + 6·58 = + 4·39	− 4·39
RS	= ⅔ × + 4·84 = + 3·23	− 3·23
ST	= ⅔ × + 2·84 = + 1·89	− 1·89
TU	= ⅔ × + 0·95 = + 0·63	− 0·63

H = ⅔ of combined H for W.2. with opposite sign

= (0·202 − 0·019) × ⅔

∴ H = −0·122

V_a = −0·892	V_u = −0·708

H_a = (−0·202 + (−0·476)) × ⅔ = +0·452

H_u = (−0·202 + 0·514) × ⅔ = −0·208

W.4. WIND ON SIDE

On upper ⅔ of Vertical Surface less height above eaves

Point	M_1	M_1 yds	Hy (H = −·27)	M (M_1−Hy)
AB	+ 2·49	+ 44·82	− 0·81	+ 3·30
BC	+ 7·47	+ 403·38	− 2·43	+ 9·90
CD	+11·99	+1244·80	− 4·15	+16·14
DE	+14·10	+1251·23	− 5·64	+19·74
EF	+12·62	+2060·97	− 7·06	+19·68
FG	+ 8·04	+1402·18	− 8·19	+16·23
GH	+ 1·89	+ 369·00	− 8·79	+10·68
HJ	− 4·73	− 987·34	− 9·39	+ 4·66
JK	−11·14	−2581·70	−10·01	− 1·13
KL	−15·41	−3571·27	−10·01	− 5·40
LM	−18·02	−3761·49	− 9·39	− 8·63
MN	−20·57	−4016·09	− 8·79	−11·78
NP	−23·05	−4019·92	− 8·19	−14·86
PQ	−21·69	−3542·19	− 7·06	−14·63
QR	−17·33	−1537·86	− 5·64	−11·69
RS	−12·77	−1325·78	− 4·15	− 8·62
ST	− 7·47	− 403·38	− 2·43	− 5·04
TU	− 2·49	− 44·82	− 0·81	− 1·68

$\sum -19015·46$ $H = \frac{-19015·46}{+71509·66}$

∴ H = −0·266

V_a = −0·794	V_u = +0·794

H_a = +0·83 − (−0·266) = +1·096

H_u = −0·83 − (−0·266) = −0·564

W.5. VERTICAL CRANE LOADS

(Max. Crane Reaction Left-Hand Side)
From 2 Ton Hand Crane

Point	M_1	M_1 yds	Hy (H = +·13)	M (M_1−Hy)
AB	0	0	+0·39	−0·39
BC	0	0	+1·17	−1·17
CD	0	0	+2·00	−2·00
DE	0	0	+2·71	−2·71
EF	+6·66	+1087·64	+3·40	+3·26
FG	+6·42	+1119·65	+3·94	+2·48
GH	+5·93	+1157·77	+4·23	+1·70
HJ	+5·43	+1133·46	+4·52	+0·91
JK	+4·92	+1140·21	+4·82	+0·10
KL	+4·40	+1019·70	+4·82	−0·42
LM	+3·89	+ 812·00	+4·52	−0·63
MN	+3·38	+ 659·91	+4·23	−0·85
NP	+2·90	+ 505·76	+3·94	−1·04
PQ	+2·66	+ 434·40	+3·40	−0·74
QR	0	0	+2·71	−2·71
RS	0	0	+2·00	−2·00
ST	0	0	+1·17	−1·17
TU	0	0	+0·39	−0·39

$\sum +9070·5$ $H = \frac{+9070·5}{+71509·66}$

∴ H = +0·127

V_a = +4·35	V_u = +1·81

H_a = 0 − (+0·127) = −0·127

H_u = 0 − (+0·127) = −0·127

W.6. HORZ. SURGE LOADS

(Acting Right to Left)
From 2 Ton Hand Crane

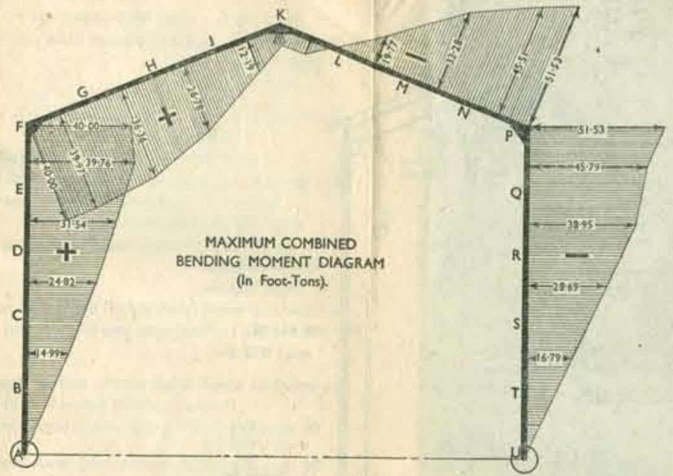
Point	M_1	M_1 yds	Hy (H = 0)	M (M_1−Hy)
AB	−0·27	− 4·86	0	−0·27
BC	−0·81	− 43·74	0	−0·81
CD	−1·38	−143·27	0	−1·38
DE	−1·88	−166·83	0	−1·88
EF	−2·07	−338·05	0	−2·07
FG	−1·82	−421·79	0	−1·82
GH	−1·32	−275·54	0	−1·52
HJ	−0·80	−156·19	0	−0·8
JK	−0·27	− 47·09	0	−0·27
KL	+0·27	+ 47·09	0	+0·27
LM	+0·80	+156·19	0	+0·80
MN	+1·32	+275·54	0	+1·32
NP	+1·82	+421·79	0	+1·82
PQ	+2·07	+338·05	0	+2·07
QR	+1·88	+166·83	0	+1·88
RS	+1·38	+143·27	0	+1·38
ST	+0·81	+ 43·74	0	+0·81
TU	+0·27	+ 4·86	0	+0·27

$\sum 0$ ∴ H = 0

V_a = +0·093	V_u = −0·093

H_a = −0·09 − 0 = −0·09

H_u = +0·09 − 0 = +0·09



MAXIMUM COMBINED
BENDING MOMENT DIAGRAM
(In Foot-Tons).

TABULATION OF BENDING MOMENTS AND REACTIONS (FT. TONS AND TONS). MAXIMUM CRANE REACTION ACTING LEFT-HAND SIDE.

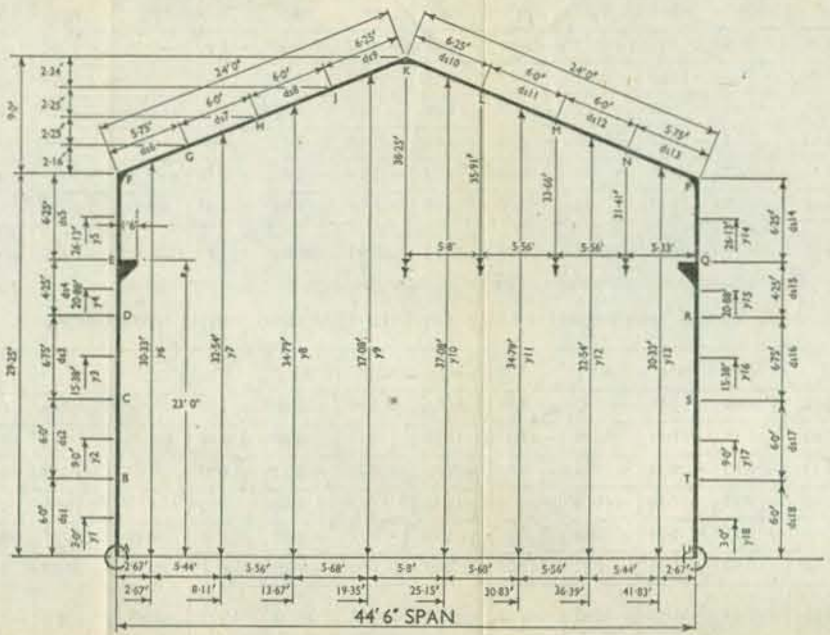
Loads	AB	BC	CD	DE	EF	F	FG	GH	HJ	JK	K	KL	LM	MN	NP	P	PQ	QR	RS	ST	TU	H_a	H_u	V_a	V_u
W.1.	−0·60	− 1·80	− 3·08	− 4·18	− 5·23	− 5·91	− 3·27	+ 1·17	+ 4·06	+ 5·28	+5·41	+5·28	+ 4·06	+ 1·17	− 3·27	− 5·91	− 5·23	− 4·18	− 3·08	− 1·80	−0·60	−0·202	−0·202	+1·200	+1·200
W.2.	+0·95	+ 2·84	+ 4·84	+ 6·58	+ 8·22	+ 9·13	+11·45	+14·08	+12·90	+ 7·67	+3·86	+0·88	− 5·17	−11·11	−16·89	−19·83	−17·64	−14·10	−10·38	− 6·08	−2·03	+0·312	−0·678	+1·061	+1·339
W.3.	+1·35	+ 4·05	+ 6·92	+ 9·40	+11·76	+12·92	+11·26	+ 7·41	+ 3·45	− 0·59	−2·57	−5·11	− 8·60	− 9·39	− 7·63	− 6·08	− 5·48	− 4·39	− 3·23	− 1·89	−0·63	+0·452	−0·208	−0·892	−0·708
W.4.	+3·30	+ 9·90	+16·14	+19·74	+19·68	+18·84	+16·23	+10·68	+ 4·66	− 1·13	−3·91	−5·40	− 8·63	−11·78	−14·86	−16·50	−14·63	−11·69	− 8·62	− 5·04	−1·68	+1·096	−0·564	−0·794	+0·794
W.5.	−0·39	− 1·17	− 2·00	− 2·71	+ 3·26	+ 2·95	+ 2·48	+ 1·70	+ 0·91	+ 0·10	−0·28	−0·42	− 0·63	− 0·85	− 1·04	− 1·14	− 0·74	− 2·71	− 2·00	− 1·17	−0·39	−0·127	−0·127	+4·350	+1·810
W.6.	±0·27	± 0·81	± 1·38	± 1·88	± 2·07	± 2·07	± 1·82	± 1·32	± 0·80	± 0·27	0	±0·27	± 0·80	± 1·32	± 1·82	± 2·07	± 2·07	± 1·88	± 1·38	± 0·81	±0·27	±0·090	±0·090	±0·093	±0·093
Max. Total	+5·00	+14·99	+24·82	+31·54	+39·76	+40·00	+39·97	+36·36	+26·78	+12·19	+5·41	+5·28	−19·77	−33·28	−45·51	−51·53	−45·79	−38·95	−28·69	−16·79	−5·60	+1·658	−1·869	+5·910	+5·236

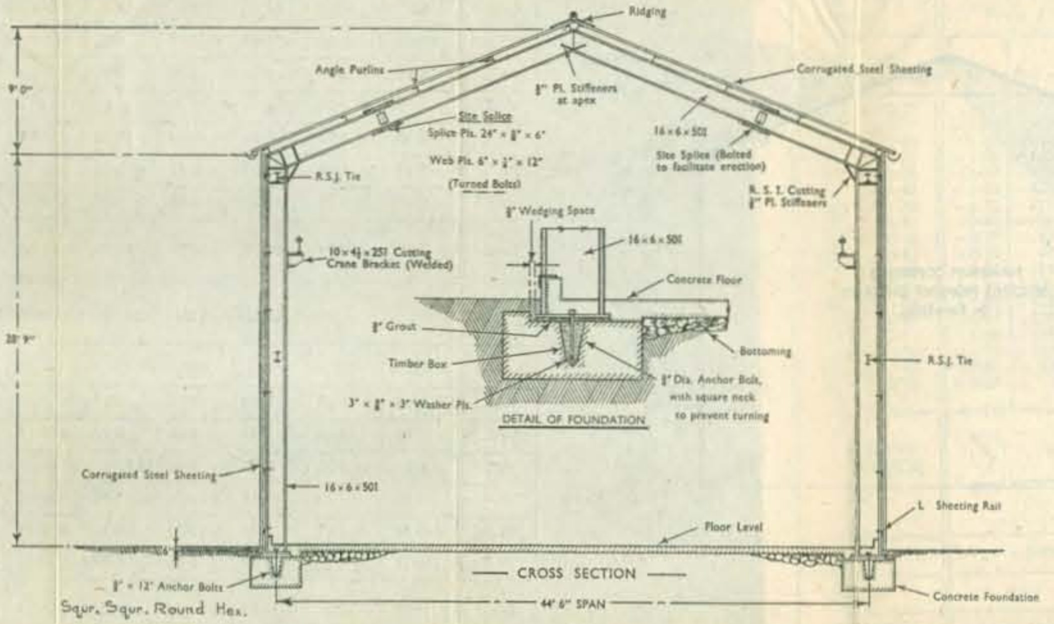
TABULATION OF BENDING MOMENTS AND REACTIONS (FT. TONS AND TONS). MAXIMUM CRANE REACTION ACTING RIGHT-HAND SIDE.

Loads	AB	BC	CD	DE	EF	F	FG	GH	HJ	JK'	K	KL	LM	MN	NP	P	PQ	QR	RS	ST	TU	H_a	H_u	V_a	V_u
W.1.	−0·60	− 1·80	− 3·08	− 4·18	− 5·23	− 5·91	− 3·27	+ 1·17	+ 4·06	+ 5·28	+5·41	+5·28	+ 4·06	+ 1·17	− 3·27	− 5·91	− 5·23	− 4·18	− 3·08	− 1·80	− 0·60	−0·202	−0·202	+1·200	+1·200
W.2.	+0·95	+ 2·84	+ 4·84	+ 6·58	+ 8·22	+ 9·13	+11·45	+14·08	+12·90	+ 7·67	+3·86	+0·88	− 5·17	−11·11	−16·89	−19·83	−17·64	−14·10	−10·38	− 6·08	−2·03	+0·312	−0·678	+1·061	+1·339
W.3.	+1·35	+ 4·05	+ 6·92	+ 9·40	+11·76	+12·92	+11·26	+ 7·41	+ 3·45	− 0·59	−2·57	−5·11	− 8·60	− 9·39	− 7·63	− 6·08	− 5·48	− 4·39	− 3·23	− 1·89	−0·63	+0·452	−0·208	−0·892	−0·708
W.4.	+3·30	+ 9·90	+16·14	+19·74	+19·68	+18·84	+16·23	+10·68	+ 4·66	− 1·13	−3·91	−5·40	− 8·63	−11·78	−14·86	−16·50	−14·63	−11·69	− 8·62	− 5·04	−1·68	+1·096	−0·564	−0·794	+0·794
W.5.	−0·39	− 1·17	− 2·00	− 2·71	− 0·74	− 1·14	− 1·04	− 0·85	− 0·63	− 0·42	−0·28	+0·10	+ 0·91	+ 1·70	+ 2·48	+ 2·95	+ 3·26	− 2·71	− 2·00	− 1·17	−0·39	−0·127	−0·127	+1·810	+4·350
W.6.	±0·27	± 0·81	± 1·38	± 1·88	± 2·07	± 2·07	± 1·82	± 1·32	± 0·80	± 0·27	0	±0·27	± 0·80	± 1·32	± 1·82	± 2·07	± 2·07	± 1·88	± 1·38	± 0·81	±0·27	±0·090	±0·090	±0·093	±0·093
Max. Total	+5·00	+14·99	+24·82	+31·54	+35·76	+35·91	+3[illegible]·45	+33·81	+25·24	+11·82	+5·41	+5·28	−18·34	−31·11	−42·65	−48·32	−42·98	−38·95	−28·69	−16·79	−5·60	+1·658	−1·869	+3·370	+7·776

FLEMING BROS. STRUCTURAL ENGINEERS, 49 BATH STREET, GLASGOW, C.2.

TWO-PIN RIGID FRAME — DIMENSIONS AND CROSS SECTION





But as point P is at eaves level all horizontal forces between this point and thrust H have to be deducted—in this case the crane thrust at Gantry Bracket and Horizontal Component of Eaves Suction Load.

$\therefore H = 1{\cdot}869 - 0{\cdot}09 - 0{\cdot}08 = 1{\cdot}699$ Tons

$\therefore$ Direct Axial Force (from H) $= 1{\cdot}699 \times \cos 22°2'$
$= 1{\cdot}699 \times 0{\cdot}9270$
$= 1{\cdot}574$ Tons

Direct Axial Force due to Vertical Thrust (see Fig. II) $= V \sin \theta$

Vertical Thrust at Pin $= 7{\cdot}068$ Tons

But similarly as point P is above Gantry level and beyond Vertical Component of Suction,

$\therefore V = 7{\cdot}068 - 4{\cdot}44 + 0{\cdot}05 = 2{\cdot}678$ Tons

$\therefore$ Direct Axial Force (from V) $= 2{\cdot}678 \times \sin 22°2'$
$= 2{\cdot}678 \times 0{\cdot}3751$
$= 1{\cdot}005$ Tons

Total Combined Direct Axial Force on the sloping portion of frame at point $P = 1{\cdot}574 + 1{\cdot}005$
$= 2{\cdot}579$ Tons

Check to find Direct Axial Force in frame at Eaves on Vertical Portion Point P

Direct Axial Force (from H) $= 1{\cdot}699 \times \cos 90°$
$= 1{\cdot}699 \times 0 = 0$

Direct Axial Force (from V) $= 2{\cdot}678 \times \sin 90°$
$= 2{\cdot}678 \times 1$
$= 2{\cdot}678$ Tons

$\therefore$ Total Combined Direct Axial Force on the Vertical Portion of frame at Point P
$= 2{\cdot}678$ Tons

As 2·678 Tons is the greater, frame has to be designed for same.

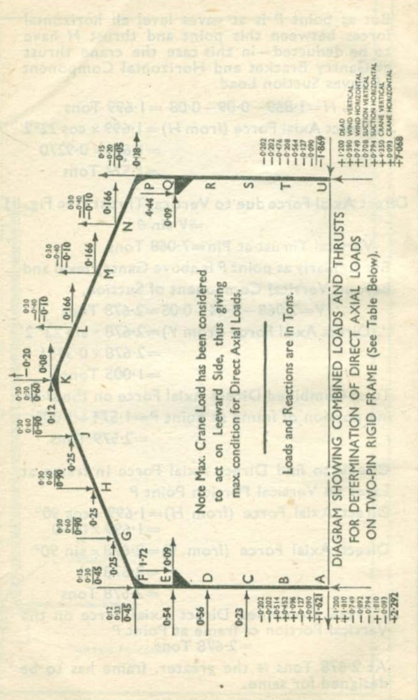


DIAGRAM SHOWING COMBINED LOADS AND THRUSTS FOR DETERMINATION OF DIRECT AXIAL LOADS ON TWO-PIN RIGID FRAME (See Table Below).

TABLE OF DIRECT LOADS IN TONS

Point	H	V	Angle θ	Cos θ	Sin θ	H Cos θ	V Sin θ	H cos θ + V Sin θ
A	1·621	2·292	90°	0	1	0	2·292	2·292
C	1·391	2·292	90°	0	1	0	2·292	2·292
E	0·201	0·572	90°	0	1	0	0·572	0·572
F	0·249	0·122	90° 22° 2′	0 0·9270	1 0·3751	0 0·231	0·122 0·046	0·122 0·277
GH	0·499	0·778	22° 2′	0·9270	0·3751	0·462	0·292	0·754
K	1·119	3·178	22° 2′	0·9270	0·3751	1·037	1·192	2·229
K	1·121	3·178	22° 2′	0·9270	0·3751	1·039	1·192	2·231
MN	1·533	2·778	22° 2′	0·9270	0·3751	1·421	1·042	2·463
P	1·699	2·678	22° 2′ 90°	0·9270 0	0·3751 1	1·574 0	1·005 2·678	2·574 2·678
Q	1·779	2·628	90°	0	1	0	2·628	2·628
S	1·869	7·068	90°	0	1	0	7·068	7·068
U	1·869	7·068	90°	0	1	0	7·068	7·068

ACKNOWLEDGEMENTS

The railway clearance line diagram outlined on page 457 indicates the requirements for goods and passenger lines *on a straight track* issued by the Ministry of Transport 1938 and can be purchased from H.M. Stationery Office. Each railway company has, however, its own individual requirements which sometimes differ from this standard, and it is imperative that the railway company be consulted before any definite design be drawn up. The L.M.S., for instance, now insist that the dimension of 4′ 7″ from rail to face of a new structure be a minimum of 5′ 1″ although they probably would allow the 4′ 7″ size to remain in the case of an existing structure. For certain goods yards also they might want a wider spacing than 6′ 0″ between the rails.

On pages 406 and 407 are shown superimposed loads for floors. These pages are extracted by permission of the Codes of Practice Committee. Copies of the Code, of which the crown copyright is reserved, can be obtained from the British Standards Institution, 28 Victoria Street, London, S.W. 1, England. Price 2/- post free.

ALPHABETICAL INDEX

A

D

E

P

R

S

ADDITIONAL to this ISSUE

Properties of Broad Flange Beams now being rolled.

Page **333** with revisions (letter O).

SUPPLEMENT

Owing to a number of requests, the following data is now incorporated.

Safe Loads on Welded Brackets additional to pages 506-513.

Stresses and Bolts relating to the Trusses outlined on pages 408-411 designed to B.S.S. and Codes of Practice Committee loading requirements.

Safe Loads on Bolted Brackets based on Concrete Beam Theory (additional to pages 314, 315).

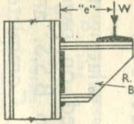
Safe Loads on Compound Struts of Double Channels.

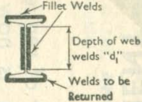
Safe Load on Timber Beams to L.C.C. Stresses.

ECCENTRIC LOADS ON WELDED BRACKETS

Welds not in same Plane as Load W

Safe Value of Welds taken as 5 Tons per sq. inch on Throat Thickness.





Flge. Fillet Welds	Web Fillet Welds	R.S. Joist Bracket	Safe Loads in Tons (W) for Eccentricity "e"												Depth of Web Welds "d_1"
Leg Lengths		Section	4″	5″	6″	7″	8″	9″	10″	11″	12″	15″	18″	21″	
$\frac{5}{16}$″	$\frac{3}{16}$″	8 × 4 × 18	8·72	7·35	6·32	5·52	4·88	4·39	3·98	3·63	3·34	2·69	2·25	1·94	6″
$\frac{3}{8}$″	$\frac{1}{4}$″	8 × 5 × 28	12·0	10·2	8·75	7·65	6·80	6·10	5·53	5·06	4·65	3·77	3·14	2·70	5″
$\frac{7}{16}$″	$\frac{1}{4}$″	8 × 6 × 35	15·8	13·5	11·6	10·6	9·18	8·26	7·49	6·85	6·32	5·11	4·28	3·68	5″
$\frac{5}{16}$″	$\frac{3}{16}$″	9 × 4 × 21	9·94	8·42	7·26	6·36	5·66	5·08	4·61	4·21	3·88	3·13	2·62	2·25	7″
$\frac{1}{2}$″	$\frac{1}{4}$″	9 × 7 × 50	21·7	18·8	16·5	14·7	13·1	11·9	10·8	9·91	9·15	7·44	6·24	5·37	5″
$\frac{3}{8}$″	$\frac{3}{16}$″	10 × 4½ × 25	13·0	11·3	9·82	8·67	7·74	6·97	6·35	5·81	5·37	4·35	3·64	3·14	7″
$\frac{7}{16}$″	$\frac{1}{4}$″	10 × 5 × 30	17·0	14·6	12·8	11·3	10·1	9·08	8·25	7·56	6·97	5·64	4·74	4·07	7″
$\frac{1}{2}$″	$\frac{1}{4}$″	10 × 6 × 40	21·8	18·9	16·6	14·7	13·2	11·9	10·9	9·98	9·20	7·46	6·27	5·39	7″
$\frac{5}{8}$″	$\frac{1}{4}$″	10 × 8 × 55	32·1	28·4	25·2	22·6	20·3	18·5	16·9	15·5	14·4	11·7	9·84	8·50	6″

Flge. Fillet Welds	Web Fillet Welds	R.S. Joist Bracket	Safe Loads in Tons (W) for Eccentricity "e"												Depth of Web Welds "d_1"
Leg Lengths	Leg Lengths	Section	6"	7"	8"	9"	10"	11"	12"	15"	18"	21"	24"	27"	
7/16"	1/4"	12 × 5 × 32	15·8	14·0	12·5	11·4	10·3	9·49	8·75	7·11	5·98	5·15	4·54	4·02	9"
1/2"	1/4"	12 × 6 × 44	20·1	17·9	16·1	14·7	13·4	12·3	11·4	9·28	7·81	6·74	5·92	5·27	9"
5/8"	5/16"	12 × 8 × 65	30·7	27·5	24·9	22·6	20·8	19·1	17·7	14·4	12·1	10·4	9·24	8·26	8"
1/2"	1/4"	13 × 5 × 35	19·0	17·0	15·3	13·9	12·7	11·7	10·8	8·80	7·41	6·38	5·61	5·00	10"
1/2"	1/4"	14 × 5½ × 40	22·2	19·9	18·0	16·3	15·0	13·8	12·8	10·4	8·80	7·58	6·67	5·94	11"
1/2"	1/4"	14 × 6 × 46	23·6	21·1	19·2	17·5	16·0	14·8	13·7	11·2	9·44	8·15	7·17	6·39	11"
5/8"	5/16"	14 × 8 × 70	35·4	32·1	29·2	26·7	24·5	22·6	21·0	17·4	14·6	12·5	11·1	9·90	10"
1/2"	1/4"	15 × 5 × 42	22·4	20·1	18·2	16·6	15·2	14·0	12·9	10·6	8·92	7·70	6·75	6·03	12"
1/2"	1/4"	15 × 6 × 45	25·4	22·9	20·8	18·9	17·4	16·1	15·0	12·2	10·3	8·89	7·83	6·98	12"
5/8"	1/4"	16 × 6 × 50	31·8	28·9	26·3	24·2	22·3	20·7	19·2	15·8	13·4	11·6	10·2	9·10	13"
5/8"	5/16"	16 × 6 × 62	32·6	29·5	26·9	24·5	22·6	20·8	19·3	15·9	13·4	11·6	10·1	9·08	12"
3/4"	5/16"	16 × 8 × 75	46·0	42·2	38·6	35·6	32·8	30·5	28·4	23·5	20·0	17·4	15·3	13·7	12"
5/8"	5/16"	18 × 6 × 55	38·6	35·1	32·1	29·4	27·1	25·1	23·4	19·3	16·3	14·1	12·4	11·1	15"
1"	5/16"	18 × 7 × 75	57·6	53·1	49·1	45·5	42·2	39·4	36·7	30·6	26·1	22·7	20·0	17·9	14"
3/4"	5/16"	20 × 6½ × 65	49·0	45·1	41·6	38·4	35·6	33·2	31·0	25·8	21·9	19·0	16·8	15·0	16"
1"	5/16"	20 × 7½ × 89	65·6	61·0	56·8	52·9	49·4	46·2	43·3	36·3	31·1	27·1	24·1	21·4	16"
3/4"	5/16"	22 × 7 × 75	56·3	52·2	48·4	45·0	41·9	39·1	36·6	30·6	26·1	22·8	20·1	17·4	18"
1"	5/16"	24 × 7½ × 95	76·2	71·7	67·2	63·1	59·2	55·7	52·5	44·4	38·3	33·5	29·8	26·0	20"

C **FLEMING BROS.**

RIDGE ROOF

The data under heading "B.S.S." gives the outlined on pages 408-411. That headed and numbers of Bolts for Trusses of the same ance with the loading requirements outlined by The 33⅓% increase in stress due to wind given Bars in both groups. The bold figures are for numbers are based on a permissible stress of per square inch for Bearing, assuming ⅝" dia- for the remainder. Weights shewn are for

	B.S.S.		CODES OF PRACTICE		
Member	Stress	No. of Bolts	Angle Section	Stress	No. of Bolts
	15′ 0″ Span		**2 cwts., 1¾** cwts.		
Rafter	**+0·98**	**2**	**3 × 2½ × ¼**	**+1·10**	**2**
Rafter	+0·98	2	2½ × 2½ × ¼	+1·10	2
Main Tie	−0·83	2	2 × 2 × ¼	−1·03	2
King Tie	0	1	2 × 2 × ¼	0	1
	20′ 0″ Span		**2¾ cwts.,** 2½ cwts.		
Rafter	**+2·04**	**2**	**3 × 2½ × ¼**	**+2·21**	**2**
Rafter	+2·04	2	2½ × 2 × ¼	+2·21	2
Main Tie	−2·08	2	2 × 2 × ¼	−2·05	2
Struts	+1·02	1	2 × 2 × ¼	+0·71	1
King Tie	−0·50	1	2 × 2 × ¼	−0·52	1
	25′ 0″ Span		**3½ cwts.,** 3⅛ cwts.		
Rafter	**+2·52**	**2**	**3½ × 2½ × ¼**	**+2·79**	**2**
Rafter	+2·52	2	2½ × 2½ × ¼	+2·79	2
Main Tie	−2·56	2	2 × 2 × ¼	−2·59	2
Struts	+0·89	1	2 × 2 × ¼	+0·62	1
Queen Tie	−1·18	1	2 × 2 × ¼	−0·86	1

\+ Denotes Compression in member
− Denotes Tension in member

TRUSSES ; Stresses and Scantlings

stresses and numbers of Bolts for the Trusses
" Codes of Practice " gives sections, stresses
span and arrangement but designed in accord-
the Codes of Practice Committee, Chapter 5.
by B.S.S. has been taken into account for all
Rafters subject to bending stresses. The Bolt
$4 + \frac{1}{3}$ tons per square inch for shear and $8 + \frac{1}{3}$ tons
meter for 2" and $2\frac{1}{4}$" Bars and $\frac{3}{4}$" diameter
Codes of Practice Trusses.

	B.S.S.		CODES OF PRACTICE		
Member	Stress	No. of Bolts	Angle Section	Stress	No. of Bolts
	30′ 0″ Span		$4\frac{3}{4}$ cwts., $4\frac{5}{8}$ cwts.		
Rafter	+3·57	2	$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	+3·75	3
Rafter	+3·57	2	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	+3·75	3
Main Tie	−3·65	3	$2 \times 2 \times \frac{1}{4}$	−3·45	3
Struts	+0·94	1	$2 \times 2 \times \frac{1}{4}$	+0·65	1
Queen Tie	−1·94	2	$2 \times 2 \times \frac{1}{4}$	−1·40	1
	35′ 0″ Span		$5\frac{3}{4}$ cwts., $4\frac{7}{8}$ cwts.		
Rafter	+4·10	2	$4 \times 3 \times \frac{1}{4}$	+4·38	3
Rafter	+4·10	3	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	+4·38	3
Main Tie	−4·20	3	$2 \times 2 \times \frac{1}{2}$	−4·03	3
Struts	+1·12	1	$2 \times 2 \times \frac{1}{4}$	+0·76	1
Queen Tie	−2·27	2	$2 \times 2 \times \frac{1}{4}$	−1·63	1
	40′ 0″ Span		$7\frac{1}{4}$ cwts., $6\frac{1}{2}$ cwts.		
Rafter	+5·07	3	$3\frac{1}{2} \times 3 \times \frac{1}{4}$	+5·20	3
Rafter	+5·07	3	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	+5·20	3
Side M/Tie	−5·18	3	$2 \times 2 \times \frac{1}{4}$	−4·83	3
Centre M/Tie	−2·17	2	$2 \times 2 \times \frac{1}{4}$	−2·75	2
Struts	+1·48	1	$2 \times 2 \times \frac{1}{4}$	+1·04	1
Ties	−3·02	2	$2 \times 2 \times \frac{1}{4}$	−2·10	2

E FLEMING BROS.

The data under heading "B.S.S." gives the outlined on pages 408-411. That headed and numbers of Bolts for Trusses of the same ance with the loading requirements outlined by The $33\frac{1}{3}$% increase in stress due to wind given Bars in both groups. The bold figures are for numbers are based on a permissible stress of per square inch for Bearing, assuming $\frac{5}{8}$" dia- for the remainder. Weights shewn are for

B.S.S.			CODES OF PRACTICE		
Member	Stress	No of Bolts	Angle Section	Stress	No. of Bolts
	45' 0" Span		$8\frac{1}{4}$ cwts., $7\frac{7}{8}$ cwts.		
Rafter	+5·71	3	$4 \times 3 \times \frac{1}{4}$	+5·85	3
Rafter	+5·71	3	$3 \times 2\frac{1}{2} \times \frac{1}{4}$	+5·85	3
Side M/Tie	−5·83	3	$2 \times 2 \times \frac{1}{4}$	−5·43	4
Centre M/Tie	−2·44	2	$2 \times 2 \times \frac{1}{4}$	−3·10	2
Struts	+1·67	1	$2 \times 2 \times \frac{1}{4}$	+1·17	1
Ties	−3·39	3	$2 \times 2 \times \frac{1}{4}$	−2·33	2
	50' 0" Span		$10\frac{1}{8}$ cwts., $8\frac{5}{8}$ cwts.		
Rafter	+6·34	3	$4 \times 3 \times \frac{5}{16}$	+6·50	3
Rafter	+6·34	4	$3 \times 3 \times \frac{1}{4}$	+6·50	4
Side M/Tie	−6·47	4	$2\frac{1}{2} \times 2 \times \frac{1}{4}$	−5·91	3
Centre M/Tie	−2·72	2	$2 \times 2 \times \frac{1}{4}$	−3·44	3
Main Strut	+1·85	1	$2\frac{1}{2} \times 2 \times \frac{1}{4}$	+1·30	1
Short Struts	+0·94	1	$2 \times 2 \times \frac{1}{4}$	+0·65	1
Queen Tie	−3·77	2	$2\frac{1}{2} \times 2 \times \frac{1}{4}$	−2·62	2
Ties	−1·25	1	$2 \times 2 \times \frac{1}{4}$	−0·89	1

\+ Denotes Compression in member
− Denotes Tension in member

stresses and numbers of Bolts for the Trusses
"Codes of Practice" gives sections, stresses
span and arrangement but designed in accord-
the Codes of Practice Committee, Chapter 5.
by B.S.S. has been taken into account for all
Rafters subject to bending stresses. The Bolt
$4+\frac{1}{3}$ tons per square inch for shear and $8+\frac{1}{3}$ tons
meter for 2" and $2\frac{1}{4}$" Bars and $\frac{3}{4}$" diameter
Codes of Practice Trusses.

	B.S.S.		CODES OF PRACTICE		
Member	Stress	No. of Bolts	Angle Section	Stress	No. of Bolts
	55' 0" Span		$11\frac{1}{2}$ cwts., 10 cwts.		
Rafter	+7·57	4	$4 \times 3 \times \frac{5}{16}$	+7·27	4
Rafter	+7·57	4	$3 \times 3 \times \frac{1}{4}$	+7·27	4
Side M/Tie	−7·71	4	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	−6·77	4
Centre M/Tie	−3·34	2	$2\frac{1}{2} \times 2 \times \frac{1}{4}$	−3·84	3
Main Struts	+1·44	1	$3 \times 2\frac{1}{2} \times \frac{1}{4}$	+0·93	1
Short Struts	+0·83	1	$2 \times 2 \times \frac{1}{4}$	+0·54	1
Queen Tie	−4·49	3	$2\frac{1}{2} \times 2 \times \frac{1}{4}$	−2·97	2
Ties	−1·13	1	$2 \times 2 \times \frac{1}{4}$	−0·73	1
	60' 0" Span		$13\frac{1}{2}$ cwts., 11 cwts.		
Rafter	+8·26	4	$2/3 \times 2 \times \frac{1}{4}$	+7·93	4
Rafter	+8·26	4	$3 \times 3 \times \frac{1}{4}$	+7·93	4
Side M/Tie	−8·40	4	$2\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	−7·39	4
Centre M/Tie	−3·65	2	$2 \times 2 \times \frac{1}{4}$	−4·19	3
Main Struts	+1·57	1	$3 \times 3 \times \frac{1}{4}$	+1·03	1
Short Struts	+0·91	1	$2 \times 2 \times \frac{1}{4}$	+0·59	1
Queen Tie	−4·90	3	$2\frac{1}{2} \times 2 \times \frac{1}{4}$	−3·24	2
Ties	−1·23	1	$2 \times 2 \times \frac{1}{4}$	−0·80	1

The data under heading "B.S.S." gives the outlined on pages 408-411. That headed and numbers of Bolts for Trusses of the same ance with the loading requirements outlined by The $33\frac{1}{3}$% increase in stress due to wind given Bars in both groups. The bold figures are for numbers are based on a permissible stress of per square inch for Bearing, assuming $\frac{5}{8}$" diafor the remainder. Weights shewn are for

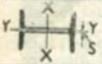
B.S.S.			CODES OF PRACTICE		
Member	Stress	No. of Bolts	Angle Section	Stress	No. of Bolts
	65' 0" Span		**$15\frac{3}{4}$ cwts.,** $14\frac{1}{4}$ cwts.		
Rafter	**+9·35**	**4**	**$2/3 \times 2 \times \frac{1}{4}$**	**+9·10**	**4**
Rafter	+9·35	4	$3\frac{1}{2} \times 3 \times \frac{5}{16}$	+9·10	5
Side M/Tie	−9·48	5	$3 \times 2\frac{1}{2} \times \frac{1}{4}$	−8·46	5
Centre M/Tie	−3·96	2	$2\frac{1}{2} \times 2 \times \frac{1}{4}$	−4·61	3
Main Strut	+2·46	2	$3 \times 3 \times \frac{1}{4}$	+1·70	2
Short Struts	+1·05	1	$2 \times 2 \times \frac{1}{4}$	+0·78	1
Queen Tie	−5·47	3	$2\frac{1}{2} \times 2 \times \frac{1}{4}$	−3·84	3
Ties	−2·20	2	$2 \times 2 \times \frac{1}{4}$	−1·60	1
	70' 0" Span		**19 cwts.,** $17\frac{1}{2}$ cwts.		
Rafter	**+10·1**	**5**	**$2/3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$**	**+9·80**	**4**
Rafter	+10·1	5	$2/2\frac{1}{2} \times 2 \times \frac{1}{4}$	+9·80	4
Side M/Tie	−10·2	5	$2/2\frac{1}{2} \times 2 \times \frac{1}{4}$	−9·11	4
Centre M/Tie	−4·27	3	$2/2 \times 2 \times \frac{1}{4}$	−4·97	3
Main Strut	+2·65	2	$3 \times 3 \times \frac{1}{4}$	+1·83	2
Short Struts	+1·13	1	$2 \times 2 \times \frac{1}{4}$	+0·84	1
Queen Tie	−5·89	3	$2\frac{1}{2} \times 2 \times \frac{1}{4}$	−4·14	3
Ties	−2·97	2	$2 \times 2 \times \frac{1}{4}$	−1·72	2

+ Denotes Compression in member
− Denotes Tension in member

stresses and numbers of Bolts for the Trusses "Codes of Practice" gives sections, stresses span and arrangement but designed in accord- the Codes of Practice Committee, Chapter 5. by B.S.S. has been taken into account for all Rafters subject to bending stresses. The Bolt $4+\frac{1}{3}$ tons per square inch for shear and $8+\frac{1}{3}$ tons meter for 2" and $2\frac{1}{4}$" Bars and $\frac{3}{4}$" diameter Codes of Practice Trusses.

B.S.S.			CODES OF PRACTICE		
Member	Stress	No. of Bolts	Angle Section	Stress	No. of Bolts
75′ 0″ Span			$20\frac{3}{4}$ cwts., 19 cwts.		
Rafter	+10·8	5	$2/3 \times 2\frac{1}{2} \times \frac{1}{4}$	+10·5	5
Rafter	+10·8	5	$2/2\frac{1}{2} \times 2 \times \frac{1}{4}$	+10·5	5
Side M/Tie	−10·9	5	$2/2\frac{1}{2} \times 2 \times \frac{1}{4}$	−9·76	4
Centre M/Tie	−4·58	3	$2/2 \times 2 \times \frac{1}{4}$	−5·31	3
Main Strut	+2·84	2	$3\frac{1}{2} \times 3 \times \frac{1}{4}$	+1·96	2
Short Struts	+1·21	1	$2 \times 2 \times \frac{1}{4}$	+0·90	1
Queen Tie	−6·31	4	$3 \times 2 \times \frac{1}{4}$	−4·43	3
Ties	−2·54	2	$2 \times 2 \times \frac{1}{4}$	−1·85	2
80′ 0″ Span			$22\frac{1}{4}$ cwts., $20\frac{1}{2}$ cwts.		
Rafter	+11·5	5	$2/3 \times 2\frac{1}{2} \times \frac{1}{4}$	+11·2	5
Rafter	+11·5	5	$2/2\frac{1}{2} \times 2 \times \frac{1}{4}$	+11·2	5
Side M/Tie	−11·6	5	$2/2\frac{1}{2} \times 2 \times \frac{1}{4}$	−10·4	5
Centre M/Tie	−4·74	3	$2/2 \times 2 \times \frac{1}{4}$	−5·67	3
Main Strut	+3·05	2	$2/2\frac{1}{2} \times 2 \times \frac{1}{4}$	+2·22	2
Short Struts	+1·33	1	$2 \times 2 \times \frac{1}{4}$	+0·96	1
Queen Tie	−6·94	4	$2/2\frac{1}{2} \times 2 \times \frac{1}{4}$	−4·80	3
Ties	−2·83	2	$2 \times 2 \times \frac{1}{4}$	−1·98	2

COMPOUND STRUTS — Two Safe Concentric Loads in Tons

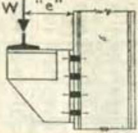
 Two Channels	Area in sq. ins.	Space S between webs in ins.	EFFECTIVE				
			5	6	7	8	9
10 × 3 × 19·28	11·34	$\frac{1}{2}$	72·6	68·7	64·1	58·8	53·0
10 × 2·6 × 15·06	8·86	$\frac{1}{2}$	54·8	50·8	46·3	41·1	36·2
9 × $3\frac{1}{2}$ × 22·27	13·10	$\frac{3}{8}$	87·8	84·5	80·7	76·5	71·6
9 × 3 × 17·46	10·28	$\frac{3}{8}$	65·9	62·3	58·1	53·2	48·0
8 × $3\frac{1}{2}$ × 20·21	11·88	$\frac{3}{8}$	79·9	77·1	73·7	70·1	66·0
8 × 3 × 15·96	9·38	$\frac{3}{8}$	60·7	57·7	54·1	50·0	45·5
8 × 2·26 × 11·22	6·60	$\frac{3}{8}$	38·7	34·8	30·4	26·3	22·3
7 × $3\frac{1}{2}$ × 18·28	10·76	$\frac{3}{8}$	72·6	70·1	67·4	64·2	60·6
7 × 3 × 14·22	8·36	$\frac{3}{8}$	54·6	52·0	48·8	45·4	41·6
7 × $2\frac{1}{8}$ × 9·75	5·72	$\frac{3}{8}$	33·1	29·4	25·6	21·8	18·6
6 × $3\frac{1}{2}$ × 16·48	9·70	$\frac{3}{8}$	65·6	63·5	61·1	58·4	55·4
6 × 3 × 12·41	7·30	$\frac{5}{16}$	47·5	45·2	42·5	39·5	36·0
6 × 2 × 10·75	6·30	$\frac{5}{16}$	34·4	29·8	25·2	21·0	17·5
6 × 1·92 × 8·75	5·14	$\frac{5}{16}$	28·3	24·6	20·7	17·4	14·5
5 × $2\frac{1}{2}$ × 10·22	6·01	$\frac{5}{16}$	37·8	35·2	32·4	29·2	25·8
4 × 2 × 7·09	4·17	$\frac{5}{16}$	24·0	21·2	18·4	15·7	13·2

Channels—battened at intervals and Dimensions and Properties

HEIGHTS IN FEET								Radii of Gyration	
10	12	14	16	18	20	22	24	Axis $y—y$	Axis $x—x$
47·3	36·9	29·2	23·3	18·8	15·6	13·0	11·1	1·29	3·82
31·6	24·0	18·6	14·5	11·8	9·76	8·06		1·14	3·87
66·3	55·5	45·4	37·3	30·5	25·6	21·8	18·5	1·57	3·56
42·8	33·4	26·4	21·1	17·1	14·2	11·9	10·1	1·29	3·49
61·3	51·8	42·8	35·2	29·1	24·3	20·5	17·6	1·61	3·19
40·8	32·5	25·8	20·6	16·8	13·9	11·6	9·86	1·34	3·16
19·1	14·2	10·7	8·4	6·79				0·99	3·10
56·6	48·2	39·9	33·2	27·3	23·1	19·5	16·6	1·65	2·82
37·6	30·2	23·8	19·3	15·6	13·0	11·0	9·28	1·38	2·80
15·7	11·5	8·80	6·91	5·55				0·96	2·67
52·0	44·5	37·2	30·8	25·6	21·5	18·3	15·7	1·69	2·44
32·6	26·0	20·6	16·6	13·5	11·2	9·42	8·03	1 37	2·41
14·7	10·7	8·14	6·30					0·87	2·23
12·3	8·89	6·73	5·30					0·88	2·32
22·6	17·4	13·5	10·8	8·70	7·10	5·95		1·19	1·99
11·3	8·26	6·30	4·93	3·96				0·95	1·56

ECCENTRIC LOADS ON BOLTED BRACKETS

TWO VERTICAL ROWS

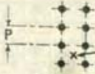


BASED ON CONCRETE BEAM THEORY.

Load W not on same plane as Bolt Group.

P = Vertical Centres of Bolts in ins.

c/c of Bolts can vary.



Centre of Rotation of Bolt Group.

See notes on pages 300 and 315

$\frac{7}{8}$" dia. Bolts P = 3" (4" Min. Flge. Breadth)

SAFE LOAD IN TONS (W) FOR ECCENTRICITY "e" (inches)

No. of Horz. Rows	6"	7"	8"	9"	10"	11"	12"	14"	16"	18"	20"	24"	28"	30"	36"
2	1·41	1·22	1·08	0·96	0·87	0·79	0·73	0·63	0·55	0·49	0·44	0·37	0·32	0·30	0·25
3	2·80	2·45	2·18	1·96	1·78	1·63	1·50	1·29	1·14	1·02	0·92	0·76	0·66	0·61	0·51
4	4·54	4·03	3·61	3·26	2·97	2·73	2·52	2·18	1·93	1·72	1·55	1·30	1·12	1·05	0·87
5	6·55	5·87	5·30	4·83	4·41	4·07	3·76	3·28	2·90	2·60	2·34	1·97	1·70	1·59	1·32
6	8·77	7·91	7·20	6·59	6·06	5·60	5·21	4·56	4·04	3·63	3·29	2·77	2·38	2·23	1·87
7	11·1	10·1	9·27	8·54	7·89	7·33	6·83	6·01	5·34	4·81	4·37	3·68	3·19	2·98	2·50
8	13·6	12·5	11·5	10·6	9·86	9·20	8·60	7·60	6·79	6·13	5·58	4·73	4·08	3·84	3·22
9	16·2	15·0	13·8	12·9	12·0	11·2	10·5	9·36	8·39	7·59	6·93	5·89	5·09	4·79	4·03
10	18·8	17·5	16·3	15·2	14·2	13·4	12·6	11·2	10·1	9·18	8·39	7·15	6·21	5·83	4·91

FLEMING BROS. L

$\frac{3}{4}''$ dia. Bolts $P = 3\frac{1}{8}''$ ($4\frac{1}{8}''$ Min. Flge. Breadth)

No. of Horz. Rows	SAFE LOAD IN TONS (W) FOR ECCENTRICITY "e" (inches) 6"	7"	8"	9"	10"	11"	12"	14"	16"	18"	20"	24"	28"	30"	36"
2	2·38	2·08	1·84	1·65	1·49	1·37	1·26	1·08	0·95	0·85	0·76	0·64	0·55	0·51	0·42
3	4·74	4·20	3·75	3·39	3·09	2·83	2·61	2·26	1·99	1·78	1·60	1·34	1·15	1·08	0·90
4	7·47	6·68	6·03	5·48	5·02	4·62	4·28	3·73	3·29	2·95	2·67	2·24	1·93	1·80	1·51
5	10·6	9·60	8·74	8·01	7·37	6·82	6·35	5·55	4·93	4·42	4·01	3·38	2·91	2·73	2·28
6	14·0	12·8	11·8	10·8	10·0	9·32	8·69	7·66	6·83	6·15	5·59	4·72	4·08	3·82	3·20
7	17·7	16·3	15·0	13·9	13·0	12·1	11·3	10·1	8·99	8·13	7·41	6·28	5·44	5·10	4·28
8	21·5	19·9	18·5	17·3	16·2	15·2	14·3	12·7	11·4	10·4	9·47	8·06	7·00	6·57	5·53
9	25·4	23·7	22·1	20·8	19·5	18·4	17·3	15·5	14·0	12·8	11·7	10·0	8·72	8·19	6·91
10	28·5	26·6	24·9	23·3	22·0	20·7	19·5	17·5	15·9	14·4	13·3	11·3	9·88	9·28	7·83

$\frac{7}{8}''$ dia. Bolts $P = 4''$ ($6''$ Min. Flge. Breadth)

No. of Horz. Rows	SAFE LOAD IN TONS (W) FOR ECCENTRICITY "e" (inches) 6"	7"	8"	9"	10"	11"	12"	14"	16"	18"	20"	24"	28"	30"	36"
2	3·63	3·19	2·83	2·54	2·31	2·11	1·95	1·68	1·48	1·32	1·19	0·99	0·85	0·80	0·66
3	7·28	6·49	5·84	5·30	4·84	4·46	4·12	3·58	3·16	2·83	2·56	2·14	1·84	1·72	1·44
4	11·3	10·2	9·26	8·46	7·78	7·18	6·69	5·85	5·18	4·65	4·22	3·55	3·06	2·86	2·39
5	15·9	14·5	13·3	12·3	11·4	10·6	9·87	8·69	7·74	6·97	6·34	5·35	4·62	4·33	3·63
6	20·9	19·3	17·8	16·5	15·4	14·4	13·5	12·0	10·7	9·71	8·86	7·51	6·51	6·10	5·13
7	26·0	24·2	22·5	21·1	19·7	18·5	17·5	15·6	14·0	12·8	11·7	9·95	8·65	8·12	6·84
8	31·4	29·4	27·6	25·9	24·4	23·0	21·8	19·6	17·7	16·2	14·8	12·7	11·1	10·4	8·79
9	36·7	34·5	32·5	30·7	29·0	27·5	26·1	23·6	21·5	19·7	18·1	15·6	13·7	12·8	10·9
10	42·4	40·1	38·0	36·0	34·2	32·5	30·9	28·1	25·7	23·7	21·9	18·9	16·6	15·7	13·3

RECTANGULAR TIMBER BEAMS (L.C.C. Bye-Law Stresses)

Safe Uniformly Distributed Loads in Pounds **per 1 inch of Beam Width.**

Maximum Bending Stress 800 lbs. per Sq. Inch, Horz. Shearing Stress 90 lbs. per Sq. Inch

Depth of Beam in Inches	SPANS IN FEET 5	6	7	8	9	10	12	14	16	18	20	22	24	26
3	160	111	79·6	62·7										
4	284	238	193	148	117	94·7								
5	445	371	317	278	228	185	128							
6	640	534	457	400	355	320	222	162	125					
7		725	622	545	484	435	363	259	198	156				
8		946	811	710	631	570	474	386	259	198	156			
9			1029	900	800	720	600	514	420	332	269			
10				1111	988	889	741	634	557	458	370	306		
11					1195	1075	896	768	672	597	492	407	342	
12					1422	1279	1067	914	800	710	640	528	444	378
14						1742	1452	1244	1090	968	872	793	705	601

Tabular Safe Loads include the Weight of the Beam and are applicable only when beams are sufficiently stiffened against Lateral Deflection

(See page xvi for notes)

Loads given are for non-graded Timber
For graded Timber multiply figures
(a) On left of zig-zag line by 1·25
(b) On right of zig-zag line by 1·33

For Section Modulus see page 343

FLEMING BROS. N

BROAD FLANGE BEAM SECTIONS
Dimensions and Properties

Size $d \times b$ ins.	Weight per foot in lbs.	Area in sq. ins.	Web Ratio Depth"/Web"	Thicknesses		Moment of Inertia		Moduli of Section		Radii of Gyration	
				Web	Flange	X—X	Y—Y	X—X	Y—Y	X—X	Y—Y
6 × 6	25	7·353	19·35	·31	·46	47·24	16·76	15·75	5·59	2·53	1·51
8 × 8	45	13·23	20·5	·39	·64	152·76	55·02	38·19	13·75	3·39	2·04
10 × 10	62·63	18·42	23·25	·43	·72	338·97	119·94	67·79	23·98	4·29	2·54
12 × 12	82·55	24·278	25·93	·47	·79	652	228	108·6	38·0	5·18	3·07
14 × 12	100	29·412	25·45	·55	·92	1048·44	265·25	149·78	44·21	5·97	3·00
16 × 12	110	32·352	29·1	·55	1·0	1504·6	288·3	188·1	48·05	6·82	2·98
18 × 12	122	35·882	30·5	·59	1·08	2094·5	311·4	232·7	51·9	7·64	2·946
20 × 12	135	39·706	31·746	·63	1·16	2808	334·65	280·8	55·78	8·41	2·903

O FLEMING BROS.

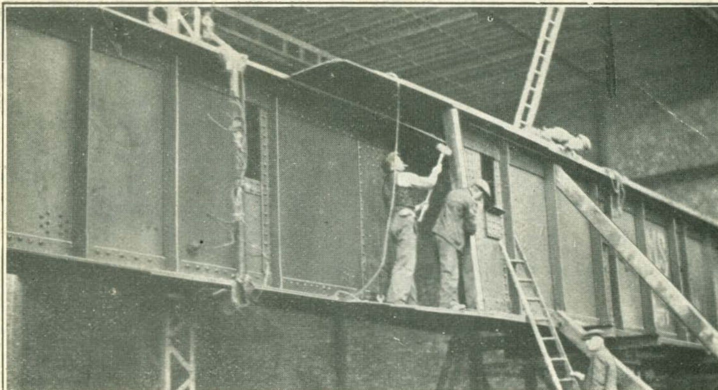
AREAS OF TWO FLANGE PLATES

Width	Thickness of each Plate									
	3/8"	1/2"	5/8"	3/4"	7/8"	1"	1¼"	1½"	1¾"	2"
10	7·5	10·0	12·5	15·0	17·5	20·0	25·0	30·0	35·0	40·0
12	9·0	12·0	15·0	18·0	21·0	24·0	30·0	36·0	42·0	48·0
14	10·5	14·0	17·5	21·0	24·5	28·0	35·0	42·0	49·0	56·0
16	12·0	16·0	20·0	24·0	28·0	32·0	40·0	48·0	56·0	64·0
18	13·5	18·0	22·5	27·0	31·5	36·0	45·0	54·0	63·0	72·0
20	15·0	20·0	25·0	30·0	35·0	40·0	50·0	60·0	70·0	80·0

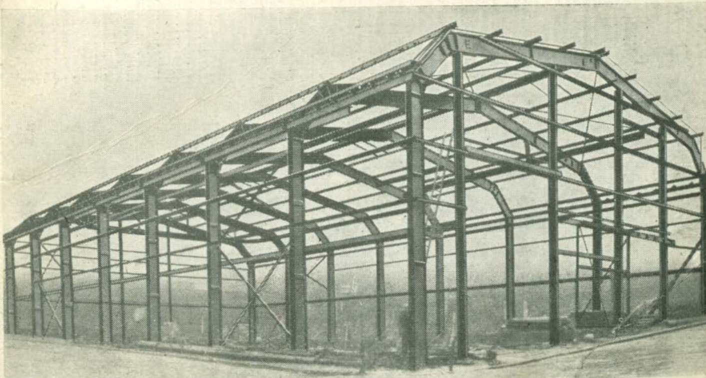
AREAS OF HOLES

Diameter of Hole	Number of Holes	Thickness t of hole in inches												
		5/16	3/8	1/2	5/8	3/4	7/8	1	1¼	1½	2	2¼	2½	2¾
13/16"	1	0·25	0·30	0·41	0·51	0·61	0·71	0·81	1·02	1·22	1·62	1·83	2·03	2·23
	2	0·50	0·60	0·82	1·02	1·22	1·42	1·62	2·04	2·44	3·24	3·66	4·06	4·46
15/16"	1	0·29	0·35	0·47	0·59	0·70	0·82	0·94	1·17	1·41	1·87	2·11	2·34	2·58
	2	0·58	0·70	0·94	1·18	1·40	1·64	1·88	2·34	2·82	3·74	4·22	4·68	5·16

MACLEHOSE, PRINTERS, GLASGOW



BALCONY GIRDER OVER 40 TONS, Near BIRMINGHAM
(In course of erection)



MODERN STEEL FRAME WELDED PORTAL STRUCTURE