IRON, STEEL AND FIREPROOF CONSTRUCTION

PAUL N. HASLUCK

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WITH NUMEROUS ENGRAVINGS AND DIAGRAMS

EDITED BY

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PREFACE.

IRON, STEEL, AND FIREPROOF CONSTRUCTION contains, in a form convenient for everyday use, a comprehensive digest of information, contributed to the columns of BUILDING WORLD, one of the weekly journals it is my fortune to edit, and supplies concise information on the general principles and practice of the art on which it treats.

In preparing for publication in book form the mass of relevant matter contained in the volumes of BUILDING WORLD, much of it necessarily had to be re-arranged and re-written. From this cause the writings of several contributors are blended, but it may be said that the greater part of the book consists of articles written by Mr. S. G. N. Mann.

Readers who may desire additional information respecting special details of the matters dealt with in this book, or instruction on any building trade subjects, should address a question to the Editor of BUILDING WORLD, La Belle Sauvage, London, E.C., so that it may be answered in the columns of that journal.

P. N. HASLUCK.

La Belle Sauvage, London,
August, 1906.

CONTENTS.

LIST OF ILLUSTRATIONS.

IRON, STEEL, AND FIRE-PROOF CONSTRUCTION.

CHAPTER I.

INTRODUCTION: CAST-IRON STANCHIONS AND COLUMNS.

It is the purpose of this book to treat the subject of iron and steel construction and fireproof construction from the practical side rather than from a strictly theoretical standpoint; for, although a knowledge of theory is essential to the draughtsman before constructional work of any kind can be designed with confidence, still the theory is dealt with fully and clearly by many authors, and there is no excuse for anyone being deficient in that respect. In studying the subject it is advisable always to work out examples, and to draw any diagrams to scale.

The centres of all rivets, bolts, and main members are shown in red finished drawings; and the usual colours for the different metals are: Steel, purple lake; wrought-iron, prussian blue; cast-iron, indigo.

The necessity for protecting iron and steel constructions from fire hardly needs comment; but, even at the present day, it is not uncommon to see expensive patent fire-resisting floors in a building, while the main girders, columns, or stanchions, wholly or partly supporting them, are not protected at all, or, at the best, inadequately. Cast iron will remain sound, when supporting a load, to a higher temperature than either wrought-iron or steel. Cast-iron, under a working stress, begins to fail at about 1,300° F., and wrought-iron and steel at about 1,000°; and as the heat of an ordinary fire may be anything up to 2,500° F., the necessity for adequate protection is apparent; but long before failure occurred owing to the metal approaching the melting point, collapse or partial destruction would be caused by the expansion of the different members.

The expansion of wrought-iron and steel being about $\frac{1}{150000}$ part for each degree its temperature is raised, and that of cast-iron slightly less, it follows that by raising the temperature of a main girder, 25 ft. long, 500° F. only, it would expand 1 in.

Cast-iron is largely used at the present time for stanchions, owing to its having great compressive resistance, and owing to the facility with which it can be cast to suit all the requirements of structural ironwork

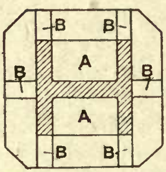

Fig. 1.—Section of H Stanchion.

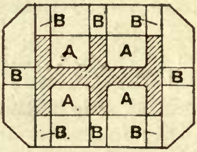

Fig. 2.—Section of H H Stanchion.

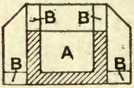

Fig. 3.—Section of Channel Stanchion.

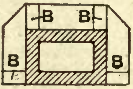

Fig. 4.—Section of Box Stanchion.

in buildings. But steel, being more reliable, is being increasingly used for the same purposes. The Americans are able to convert, with uniform results, tolerably large castings, such as automatic couplings for railway trucks and carriages, after leaving the mould, into what is practically wrought metal. As yet this is a somewhat tedious process, taking about a fortnight, but it is possible that there may be marked development in this direction. The chief points to be considered are the composition of the metal itself, and of the ore used for extracting the carbon. The temperature of the annealing furnace also requires careful regulating.

Figs. 1 to 7 show sections of different cast-iron stanchions and columns, each having its advantages under special circumstances, but the exigencies of the building will probably settle the point as to whether columns or stanchions are to be employed. In these illustrations A indicates stiffeners, B brackets, and C least width. Their relative strengths for equal weights (ends rounded) are approximately as follows: Hollow column, 100; H-shape, 75; channel shape, 50. The stanchion is much better adapted for building in brickwork, and is usually adopted when used in that position. A stanchion of H section may be considered as strong as a column of the same

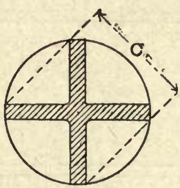

Fig. 5.—Section of Cross-shaped Stanchion.

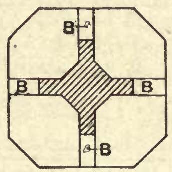

Fig. 6.—Section of a Square-ribbed Stanchion.

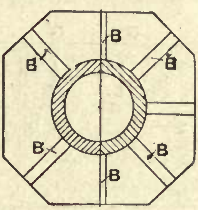

Fig. 7.—Section of Column with Alternative Arrangement of Base Brackets.

sectional area and ratio of length to least width if it has stiffeners about every 3 ft. of vertical height.

Stanchions are preferred to columns, because any defect in the casting can be more readily detected in the former than in the latter. A frequent defect in columns is that the metal forming the shaft is of unequal thickness, owing to the core springing when the metal is poured in; consequently, cores should be made large, in order to possess sufficient stiffness to resist bending.

For building work castings are made by pouring molten iron into sand in which an impression of the article required has been formed by means of a wooden pattern. The sand D is filled into iron frames, without tops or bottoms, called flasks (shown by Figs. 8 and 9). The wood pattern (Fig. 10) having been dusted with parting sand, so that the sand forming the mould does not adhere to it, an impression of exactly half of it is taken in the sand of the lower flask, the superfluous sand being then removed and the surface smoothed and dusted. The top flask is then placed in position and secured by means of bolts,

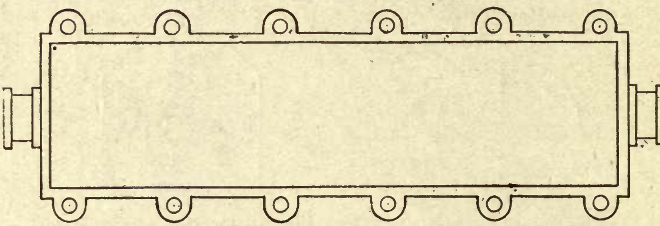

Fig. 8.—Plan of Moulding Flask.

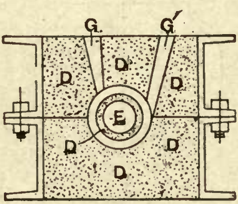

Fig. 9.—Section of Moulding Flask.

and then filled with sand rammed tightly round the pattern. The top flask is then raised, the pattern removed, and the halves again carefully put together without disarranging the sand, in which a perfect impression of the pattern remains. The core E consists of a perforated metal tube, bound round with straw bands, the whole covered with sand, and turned and smoothed to represent the hollow of the column. The projections F on the ends of the pattern (Fig. 10) are core prints. They make the impressions in the sand of the flasks to receive the prepared core, so that it is perfectly true and central. The damp sand forming the mould is painted with blacking made from charred oak to prevent the metal from being chilled when poured in. It also, under the action of the molten metal, evolves gases, and prevents too close a contact with the sand. One or more passages G are left for the metal to be poured in, and also for the escape of air and gases G′ (Fig. 9). The passages for pouring in the molten metal must be arranged so that the metal runs together from different parts at the same time; because if one portion gets partly cool before the adjacent metal reaches it, the iron will not form one mass, but will form what is called a "cold shut."

There is no satisfactory way of ascertaining whether a column is of even thickness except by drilling holes through the shaft, and this process must weaken the column, even if the holes are carefully plugged up again. The test by transverse stress, and comparing the deflec-

Fig. 10.—Wood Pattern for Moulding Cast-iron Column.

tion from several positions, is too expensive for ordinary work. It is very desirable, and is now generally specified, that columns should be cast in a vertical, or, at least, an inclined position, otherwise air bubbles and gases are liable to collect in the upper part of the mould and cause the top of the casting to be honeycombed and consequently weak.

When examining castings to ascertain their quality and soundness, several points must be attended to. If, when the edges are struck with a light hammer, a slight impression is made, the iron is probably of a suitable quality. If, on the other hand, fragments fly off, the iron is brittle. By tapping the surface all over with a hammer, the presence of bubbles and flaws hidden by vitrified sand from the mould, or purposely stopped with loam, can be easily detected; the ring will be dull, and not clear as will arise from sound metal. The exterior surface should be smooth and clear, and the edges sharp and perfect; an uneven or wavy surface indicates unequal shrinkage when cooling, and shows bad quality metal. A fractured surface should present a fine-grained texture, be of a bluish-grey colour, and possess a high metallic lustre. If by accident a projection happens to be broken off, it should not be "burnt on." If the damage is not sufficient to warrant the rejection of the casting, the part may be bolted on if this can be conveniently done. The process of burning on consists of making a mould of the part broken off to fit in position on the casting. The part of the casting where damaged is then raised to a high temperature, and the metal to make good the defect is run in and left to cool slowly.

The bases must be made large enough to distribute the load safely on to the stone base; the caps should,

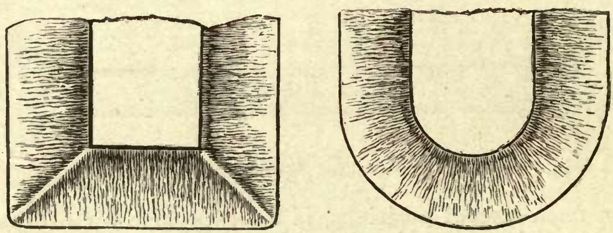

Fig. 11.—Crystallization in Square and Circular Castings.

on the contrary, be made as small as possible. A projection of more than 6 in. is seldom necessary; otherwise, an appreciable bending stress will be put on the column. The brackets supporting these projections should have a rake of at least 45°. Where columns rise tier upon tier, the caps and bases should be turned in a lathe in such a way that the surfaces are not only true but in parallel planes square to the axis of the column, in order that they may be exactly vertical when fixed. The holes for bolts connecting these surfaces should be drilled, and turned bolts used, so that there is no play in the hole; but this is seldom done in practice, the holes being formed to receive ordinary bolts which fit loosely into them.

A most important consideration with castings is that there should be no great or sudden change in the thickness of the metal; all re-entering angles should be rounded off; and, if necessary, metal must be wasted in order to obtain these ends, otherwise the unequal cooling and contraction will cause cracks at the angles. The explanation of this is that the crystals forming the iron arrange themselves at right-angles to the surfaces forming the angle (see Fig. 11), so that between the two sets of crystals there is a diagonal line of weakness.

Cast-iron contracts $\frac{1}{10}$ in. per foot in cooling, so that a pattern for a column 10 ft. long has to be made 10 ft. 1 in. The patternmakers' rules are made proportionately large, so that they can make the pattern direct from the drawing without the trouble of calculating the necessary allowance to be made on each measurement. Where surfaces have to be machined the parts should be clearly indicated both by colour and in writing on the drawing, so that the patternmaker can see at a glance what is required, and allow the additional thickness necessary.

Where appearance is a consideration, and hollow columns with their outer casings would look too large, solid cast-iron or steel columns may be substituted.

The comparative liability to oxidation of iron and steel in moist air is: Cast-iron, 100; wrought-iron, 129; steel, 133. Mild steel rusts faster than wrought-iron at first, and then slower. Cast-iron oxidizes in damp situations, but the rust does not scale off, as in the case of wrought-iron and steel, but eats into the metal to a depth of about $\frac{1}{16}$ in. and then stops for good. When a casting comes out of a mould, it has a protective coating of vitrified sand, but before it has time to rust one coat of paint should be applied, to be followed by another coat or two when the work is erected. The machined parts should be smeared with a mixture of white-lead and tallow.

CHAPTER II.

CALCULATIONS IN DESIGNING STANCHIONS AND COLUMNS.

THE first point to consider in designing a stanchion is the proportion its least width should bear to its height (see accompanying table). For cast-iron it is advisable

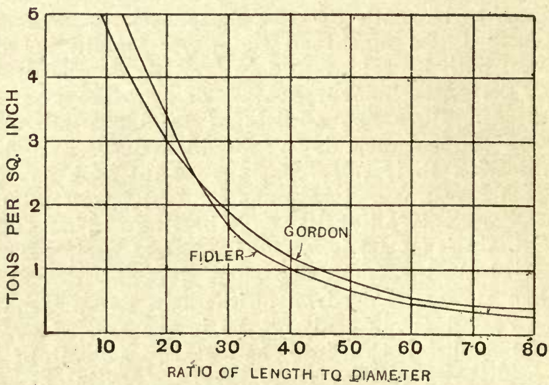

Fig. 12.—Curves comparing Formulæ for Breaking Weight of Columns.

not to exceed twenty times, and for wrought-iron and steel thirty times the least dimension. It must be remembered that in long columns—that is, columns which will fail by bending and not by direct crushing—metal near the neutral axis in whatever direction the column will tend to bend is worth little. The strength of a long column depends on the modulus of elasticity of the metal composing it, and on the square of the radius of gyration, which is equal to the moment of inertia about the axis in question divided by the area of the cross section (see table of areas of hollow columns on p. 18).

TABLE OF RATIOS OF LENGTH TO LEAST WIDTH.

Length in Feet.	*Diameter of Column or Least Side of Stanchion in Inches.* 6	7	8	9	10	11	12	13	14	15	16	18	20
6	12	10·3	9·0	8·0									
7	14	12·0	10·5	9·3									
8	16	13·7	12·0	10·8	9·6	8·7	8·0	7·4	6·9	6·4	6·0	5·3	4·8
9	18	15·4	13·5	12·0	10·8	9·8	9·0	8 3	7·7	7·2	6·7	6·0	5·4
10	20	17·1	15·0	13·3	12 0	10·9	10·0	9·2	8·6	8·0	7·5	6·7	6·0
11	22	18·8	16·5	14·6	13·2	12·0	11·0	10·1	9·4	8·8	8·2	7·3	6·6
12	24	20·6	18·0	16·0	14·4	13·1	12·0	11·1	10·3	9·6	9·0	8·0	7·2
13	26	22·3	19·5	17·3	15·6	14·2	13·0	12·0	11·1	10·4	9·7	8·7	7·8
14	28	24·0	21·0	18·6	16·8	15·3	14·0	12·9	12·0	11 2	10·5	9·3	8·4
15	30	25·7	22·5	20·0	18·0	16·4	15·0	13·8	12·9	12·0	11·3	10·0	9·0
16	32	27·4	24·0	21·3	19·2	17·4	16·0	14 8	13·7	12·8	12·0	10·7	9·6
17	34	29·1	25·5	22·6	20·4	18·5	17 0	15·7	14·6	13·6	12·7	11·3	10·2
18	36	30·9	27·0	24·0	21·6	19·6	18·0	16·6	15·4	14·4	13 5	12·0	10·8
19	38	32·6	28·5	25·3	22·8	20·7	19·0	17·5	16·3	15·2	14·2	12·7	11·4
20	40	34·3	30·0	26·6	24·0	21 8	20·0	18·5	17·1	16·0	15·0	13·3	12·0
21	42	36·0	31·5	28·1	25·2	22·9	21·0	19·4	18·0	16·8	15·7	14·0	12·6
22	44	37·7	33·0	29·3	26·4	24·0	22·0	20·3	18·8	17·6	16·5	14 7	13·2
23	46	39·4	34·5	30·6	27·6	25·1	23·0	21·2	19·7	18 4	17·2	15 3	13·8
24	48	41·1	36·0	32·0	28·8	26·2	24·0	22·2	20·6	19·2	18·0	16·0	14·4
25	50	42·9	37·5	33·3	30·0	27·3	25·0	23·1	21·4	20·0	18·7	16·7	15·0
26	52	44·7	39·0	34·6	31·2	28·4	26·0	24·0	22·3	20·8	19·5	17·3	15·6
27	54	46·3	40·5	36·0	32·4	29·4	27·0	24·9	23·1	21·6	20·2	18·0	16·2
28	56	48·0	42 0	37·3	33·6	30·5	28·0	25·8	24·0	22·4	21·0	18·7	16·8
29	58	49·7	43·5	38·7	34·8	31·6	29·0	26·8	24·8	23·2	21·7	19·3	17·4
30	60	51·4	45 0	40·0	36·0	32·7	30 0	27·7	25·7	24 0	22·5	20·0	18·0

TABLE OF AREAS OF HOLLOW COLUMNS.

Size of Column.	*Area.*	*Size of Column.*	*Area.*	*Size of Column.*	*Area.*
6 × 0¾	12 4	10 × 1¾	45·4	13 × 2	69·1
6 × 0⅞	14·1	11 × 1	31·4	14 × 1	40·8
6 × 1	15·7	11 × 1⅛	34·9	14 × 1¼	50·1
7 × 0¾	14·7	11 × 1¼	38·3	14 × 1½	58·9
7 × 0⅞	16·8	11 × 1½	44·8	14 × 1⅝	63·2
7 × 1	18·9	11 × 1⅝	47·9	14 × 1¾	67·4
7 × 1⅛	20·8	11 × 1¾	50·9	14 × 2	75·4
8 × 0¾	17·1	12 × 0⅞	30·6	14 × 2⅛	79·2
8 × 0⅞	19·6	12 × 1	34·6	15 × 1	44·0
8 × 1	22·0	12 × 1⅛	38·4	15 × 1¼	54·0
8 × 1⅛	24·3	12 × 1¼	42·2	15 × 1½	63·6
9 × 0¾	19·4	12 × 1⅜	45·9	15 × 1⅝	68·3
9 × 0⅞	22·3	12 × 1½	49·5	15 × 1¾	72 9
9 × 1	25·1	12 × 1⅝	53·0	15 × 2	81·7
9 × 1⅛	27·8	12 × 1¾	56·4	15 × 2⅛	85·9
9 × 1¼	30·4	12 × 1⅞	59·6	15 × 2¼	90·1
9 × 1½	35·3	12 × 2	62·8	16 × 1¼	57·8
10 × 0⅞	25·1	13 × 1	37·7	16 × 1½	68·3
10 × 1	28·3	13 × 1⅛	41·9	16 × 1⅝	73·3
10 × 1⅛	31·4	13 × 1¼	46·1	16 × 1¾	78·3
10 × 1¼	34 4	13 × 1½	54·2	16 × 1⅞	83·2
10 × 1½	40·1	13 × 1⅝	58·0	16 × 2	87·9
10 × 1⅝	42·7	13 × 1¾	61·9	16 × 2⅛	92 6
				16 × 2¼	97·2

A built-up stanchion should be composed of ordinary stock sections, otherwise inconvenience may arise through delay in delivery; and the sections should be chosen so as to give the least amount of labour in riveting together. If girders are to run into it, it should be of a shape to simplify the connections. It should also admit of being easily and solidly encased in fire-resisting material.

The formulæ for struts compiled by leading mathematicians show considerable variation. Fig. 12 is a diagram comparing Gordon's and Fidler's formulæ. It will be noticed that Fidler's formula gives higher results for short, and lower results for long, columns than Gordon's. Gordon's is the one chiefly relied upon in practice, and for cast-iron it is $B\ W = \dfrac{36}{1 + \dfrac{r^2}{400}}$ (see Fig. 25, p. 26).

B W = breaking weight in tons per sq. in.
36 = the ultimate crushing resistance of the iron per sq. in. in tons.
r = ration of length to least width.

This formula can be used for hollow or solid columns, and H stanchions if provided with stiffeners. The ends of the stanchions are considered to be fixed, as they can generally be supposed to be in cases connected with building work. If the ends can only be considered to be imperfectly fixed, the formula must be altered to

$$\text{B W} = \frac{36}{1 + \frac{r^2}{100}}.$$

For solid or hollow rectangular columns, ends fixed, $\text{B W} = \dfrac{36}{1 + \frac{r^2}{500}}$; ends imperfectly fixed $\text{B W} = \dfrac{36}{1 + \frac{r^2}{125}}$

Breaking Strength of Hollow or Solid Cast-Iron Columns per square inch.

Ratio of length to diam.	*Both ends fixed.*	*Ends imperfectly fixed.*
5	33·9	21·8
10	28·8	18
15	23	11
20	18	7·2
25	14	5
30	11	3·6
35	8·9	2·7
40	7·2	2·1
45	5·9	1·7
50	5·0	1·5

Breaking Strength of Hollow or Solid Rectangular Cast-Iron Columns per square inch.

Ratio of length to least width.	*Both ends fixed.*	*Ends imperfectly fixed.*
5	34·3	30
10	30	20
15	24·8	12·9
20	20	8·6
25	16	6·0
30	12·9	4·4
35	10·4	3·3
40	8·6	2·6
45	7·1	2·1
50	6·0	1·7

For example, let there be a hollow cast-iron column required 15 ft. long to carry a working load of 100 tons. Let its outside diameter be 10 in., then the ratio of its length to diameter (least width) will be $\frac{15 \times 12}{10} = 18$,

and $r^2 = 324$; therefore, $B\ W = \dfrac{36}{1 + \frac{324}{400}} = 20$ tons.

Supposing the column to carry the floor of a dwelling or office where there would be no violent vibrations such as result from machinery, a factor of safety of 6 will be

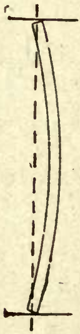

Fig. 13.—Column Rounded Both Ends.

Fig. 14.—Column having One End Fixed and the Other Rounded.

Fig. 15.—Column Fixed Both Ends.

sufficient, so $\frac{20}{6} = 3{\cdot}3$ tons per sq. in. will be the safe load, and the number of sq. in. of metal required will be $\frac{100}{3 \cdot 3} =$ 31. Now we know the area of the ring of metal and also its external diameter; if there is no table of areas of circles handy, the internal diameter can be obtained as follows:

$$31 = {\cdot}7854\ (10^2 - d^2)$$

$d = 7\frac{3}{4}$ in., so that the shaft would be $1\frac{1}{8}$ in. thick; the cap, base, and brackets would be made $1\frac{1}{4}$ in. thick. If it is rested on a base stone which would safely resist 20 tons per sq. ft., its area would be $\frac{100}{20} =$ 5 sq. ft., or 2 ft. 4 in. square.

Gordon's formula, which is one of the best known, is:

$$R = \frac{Ar}{1 + a\frac{l^2}{d^2}}$$

R = total safe load.

A = area of cross section.

r = safe resistance of metal per sq. in.

l = length in in.

d = least width in in.

a = a constant having the following values:

Cast-iron—both ends fixed

Circular (solid) $\frac{1}{400}$

" (hollow) $\frac{1}{800}$

Rectangular $\frac{3}{1600}$

Cross-shaped $\frac{3}{800}$

Wrought iron—both ends fixed

Rectangular, circular, or solid columns $\frac{1}{2500}$

Angle, tee, cross, square, H and ⊔ sections $\frac{1}{900}$

In the formula as given on p. 18 it will be noticed that a is taken at $\frac{1}{400}$ for both hollow and solid columns. If both ends are rounded, make a four times the value given above; and if fixed one end and rounded the other, two and a half times.

Roughly, the relative strengths of columns according to the method of fixing are represented by the following figures: Rounded both ends, 1 (Fig. 13); one end fixed, the other rounded, 2 (Fig. 14); both fixed, 3 (Fig. 15). The relative strengths of different metals in long columns are —cast-iron, 1; wrought-iron, $1\frac{3}{4}$; cast steel; 2·5.

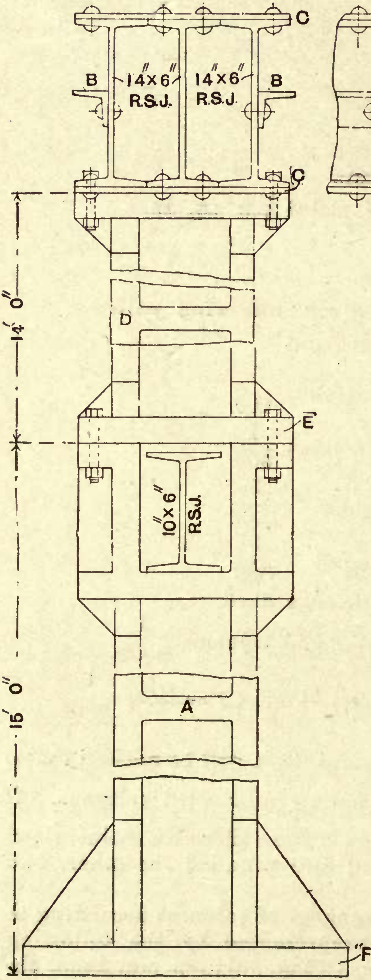

Fig. 16.—Side View of Massive Cast-iron Stanchion.

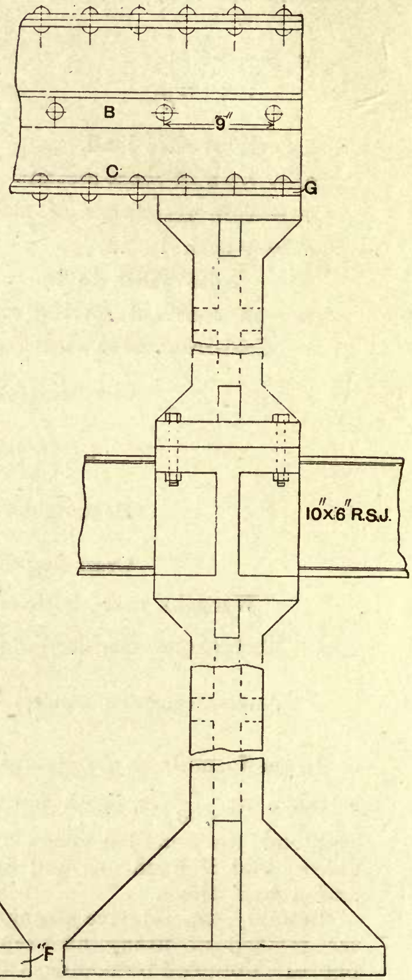

Fig. 17.—Edge View of Massive Cast-iron Stanchion.

Assume a cast-iron column 20 ft. long, 6 in. diameter, $\frac{1}{2}$ in. thick, and find the safe load with factor of safety 6.

$$R = \frac{8 \cdot 6 \times 6}{1 + \frac{1600}{800}} = 17 \cdot 2 \text{ tons.}$$

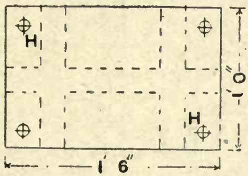

Fig. 18.—Plan of Upper Cap, Figs. 16 & 17.

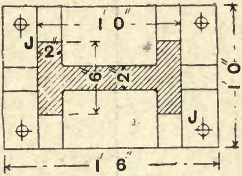

Fig. 19.—Plan of Upper Base, Figs. 16 & 17.

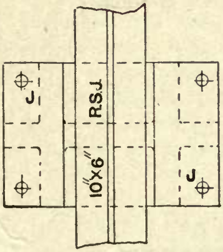

Fig. 20.—Rolled Steel Joist Resting on Lower Stanchion, Figs. 16 & 17.

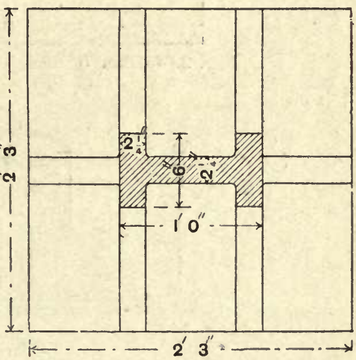

Fig. 21.—Plan of Lower Base, Figs. 16 & 17.

Figs. 16 to 21 give full details of a massive cast-iron stanchion. The letter references in Figs. 16 to 21 are thus explained: A, stiffeners, about 3-ft. 4-in. centres; B, 3-in. by 3-in. angle stringer to receive floor joists; C, $\frac{1}{2}$-in. plate; D, 2-in. metal; E, $2\frac{1}{4}$-in. metal; F, $2\frac{1}{2}$-in. metal; G, rivets countersunk; H, $\frac{7}{8}$-in. diameter holes; J, 1-in. diameter holes.

Figs. 22 and 23 show a plan and elevation of the base of a cast-iron column K, 15 ft. high, 14 in. diameter, $1\frac{3}{8}$ in. thick, which was designed to carry safely about 240 tons.

When it had no more than half this load on it, the brick pier in cement L, 5 ft. square, cracked vertically and

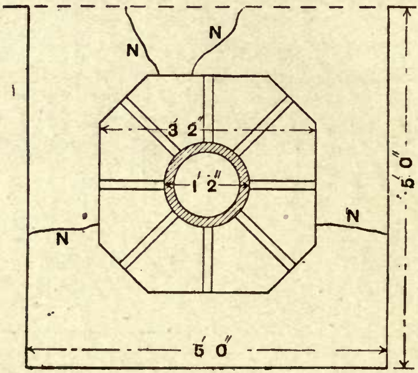

Fig. 22.—Plan of Base of Cast-iron Column.

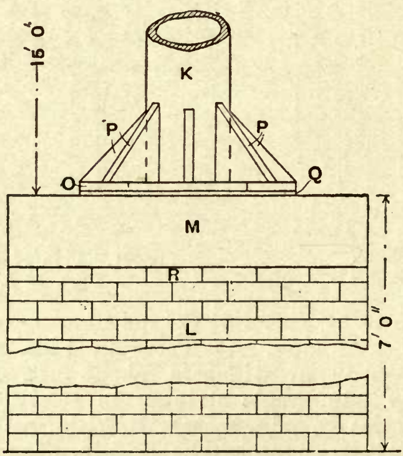

Fig. 23.—Elevation of Base of Cast-iron Column.

bulged badly, and was shored up just in time to prevent a collapse. The grey granite base stone M, 1 ft. thick, cracked into four pieces, as shown on plan at N. The column base O, $1\frac{3}{8}$ in. thick, with $1\frac{3}{8}$-in. brackets P resting on a $\frac{5}{8}$-in. iron-cement bed, remained perfectly sound. Fletton bricks with a frog, in cement, were used in the pier. It should have been built in blue wire cut bricks, which would not have a frog. Fletton bricks are made by the dry-clay process, the clay being ground and subjected to a pressure of about 200 tons on the brick in moulding. They were very close in texture, and had good surfaces and arrises, but appeared to lack toughness. Probably, however, they would safely withstand a load of 5 tons per foot super. if built in cement and properly bonded. Directly under the base stone was a 2-in. cut course of bricks R; consequently, the stone had an uneven bed, and this probably hastened the failure of the entire brick pier.

Now that iron and steel enter so largely into the construction of buildings, an Act should be passed governing this construction very fully; for the importance of any member corresponds with the work it has to do, and a failure is a far more serious matter now than it was in the old-time brick and timber erections. The New York, Boston, and Chicago building laws require cast-iron columns and stanchions to be computed by the formulæ graphically represented in the diagrams (Figs. 24 to 26).

It will be noted that New York is satisfied with a factor of safety of 5—that is, 16,000 lb. per sq. in. for an ultimate strength of 80,000 lb. per sq. in.; whereas Boston and Chicago require a factor of safety of 8—10,000 lb. per sq. in. The contrast shown by the diagrams is remarkable, for a column which would be allowed to carry 160 tons in New York would only be allowed 125 tons in Boston, and 115 tons in Chicago.

The New York building law stipulates a minimum thickness for cast-iron of $\frac{3}{4}$ in., and the height must not exceed twenty times the least dimension. Wrought-iron or steel columns or stanchions must be at least $\frac{1}{4}$ in. thick, and not exceed thirty times the least dimension in height. The usual practice in England is to make the thickness of metal in the shaft of a hollow cast-iron column one-tenth of the diameter; in no case should it be less than one-twelfth.

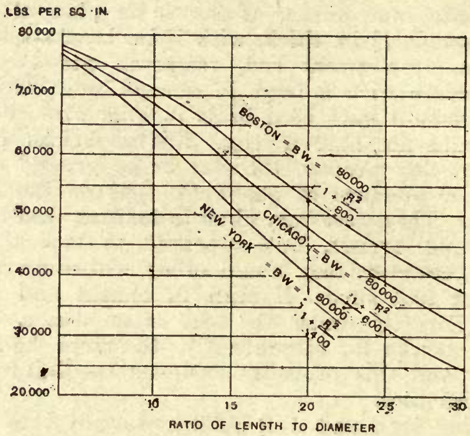

Fig. 24.—Curves showing Breaking Weight of Round Cast-iron Columns.

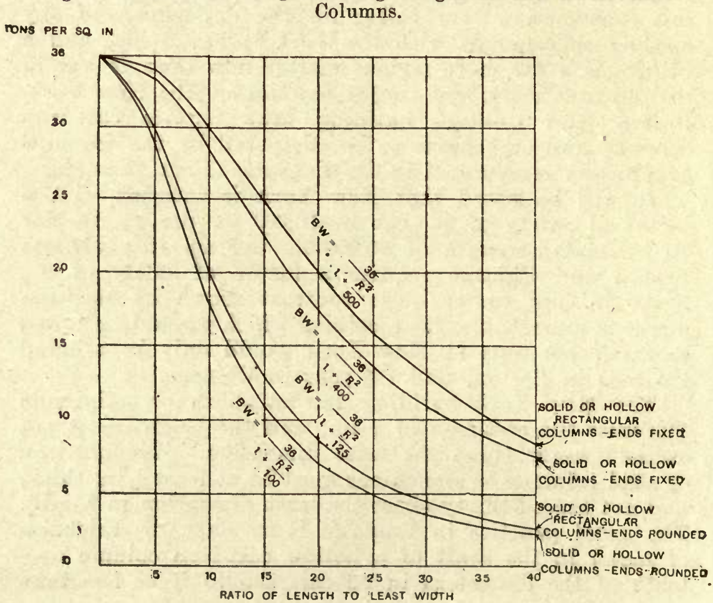

Fig. 25.—Curves showing Breaking Weight of Cast-iron Columns.

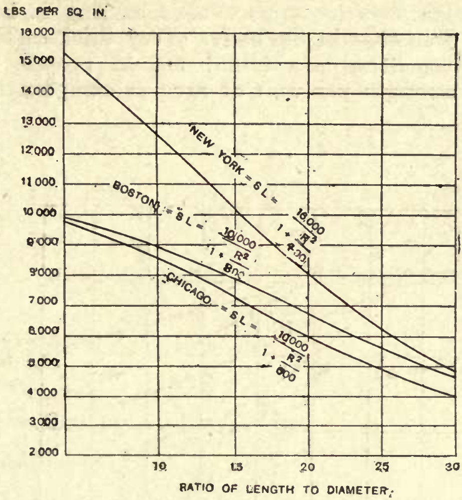

Fig. 26.—Curves showing Safe Load on Round Cast-iron Columns.

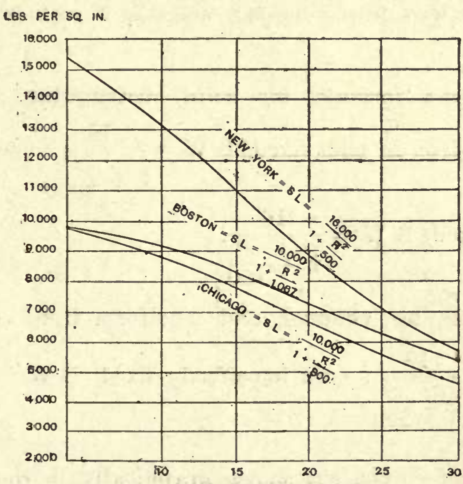

Fig. 27.—Curves showing Safe Load on Square Cast-iron Columns.

The metal on the surface of a casting is better and stronger than that in the body. Very thick castings from the same smelting are found not to possess the same ultimate strength per unit of area as comparatively thin ones.

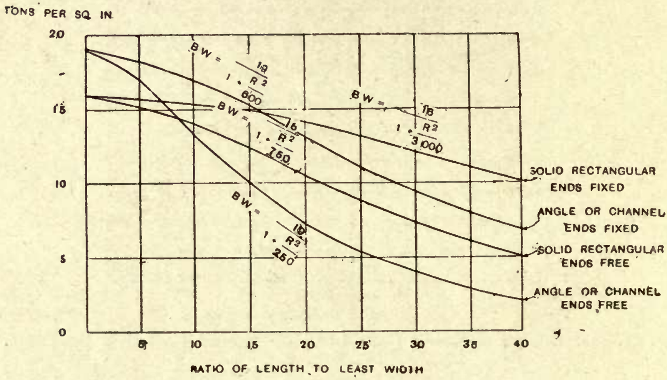

Fig. 28.—Curves showing Breaking Weight of Wrought-iron Columns.

Gordon's formula for solid rectangular pillars of wrought-iron is, ends fixed, $\text{B W} = \frac{16}{1 + \frac{r^2}{3000}}$; ends imperfectly fixed, $\text{B W} = \frac{16}{1 + \frac{r^2}{750}}$.

For angle, tee, channel, and cruciform iron, ends fixed, $\text{B W} = \frac{19}{1 + \frac{r^2}{900}}$; ends imperfectly fixed, $\text{B W} = \frac{19}{1 + \frac{r^2}{250}}$.

These formulæ are shown graphically in the diagram given above (Fig. 28).

Breaking Weight per square inch of Solid Rectangular Columns of Wrought Iron—Solid Round Columns 15 per cent. less.

Ratio of length to least width.	*Ends fixed.*	*Ends rounded.*
5	15·8	15·5
10	15·5	14·1
15	15·0	12·3
20	14·1	10·4
25	13·2	8·7
30	12·3	7·3
35	11·3	6·1
40	10·4	5·1
45	9·5	4·3
50	8·7	3·7

CHAPTER III.

STEEL STANCHIONS, BUILT AND SOLID.

ROLLED steel joists of suitable sections are also much used as stanchions, either singly or compounded with others, or with channel steels, Z bars and plates, as shown in Figs. 29 to 38. The caps and bases are formed with

Fig. 29.

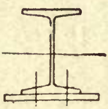

Fig. 30.

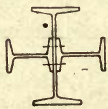

Fig. 31.

Fig. 32.

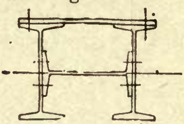

Fig. 33.

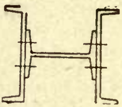

Fig. 34.

Fig. 29.—Rolled Steel Joists used as Stanchion. Fig. 30.—Rolled Steel Joist with Flange Plates. Fig. 31.—Four Rolled Steel Joists Combined. Fig. 32.—Three Rolled Steel Joists and Plates Combined. Fig. 33.—Three Rolled Steel Joists Combined. Fig. 34.—Two Channels Combined with Rolled Steel Joist.

plates secured to the shaft by angle and tee steels as shown in Figs. 39 to 43. In the cap and base the pitch of the rivet has to be varied to suit the connections, but in the

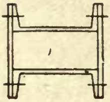

Fig. 35.

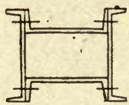

Fig. 36.

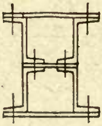

Fig. 37.

Fig. 38.

Fig. 35.—Two Channels Combined with Plates. Fig. 36.—Four Channels Combined. Fig. 37.—Four Z Bars Combined with Plates. Fig. 38.—Column in Four Sections Riveted Together.

shaft 4-in. to 6-in. pitch is usual. In these illustrations A indicates $\frac{3}{4}$-in. plate, with rivets countersunk on the under side, and B $\frac{1}{2}$-in. plate.

For built steel stanchions of British manufacture, such as those illustrated in Figs. 29 to 38, the following formula may be used :—

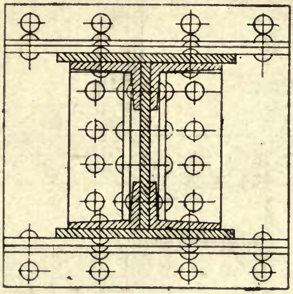

Fig. 39.—Plan of Base of a Built Steel Stanchion.

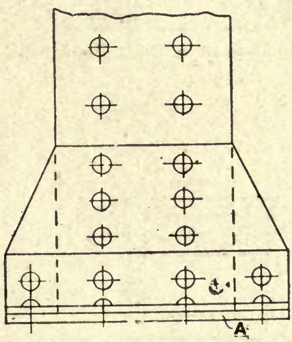

Fig. 40.—Elevation of Base of a Built Steel Stanchion.

Breaking weight per sq. in. with both ends fixed $= \dfrac{30}{1 + \dfrac{r^2}{900}}$; with ends free $= \dfrac{30}{1 + \dfrac{r^2}{225}}$. If foreign steel $= \dfrac{24}{1 + \dfrac{r^2}{900}}$; ends free $= \dfrac{24}{1 + \dfrac{r^2}{225}}$.

For dead loads a factor of safety of 4 is sufficient, and for live loads 6.

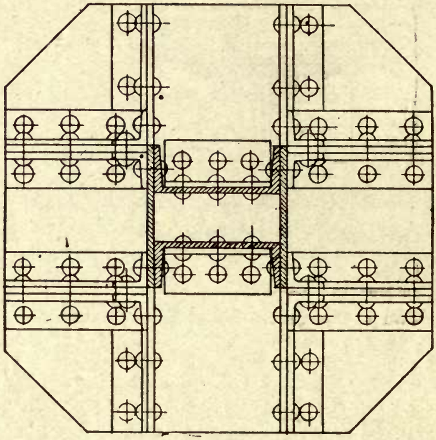

Fig. 41.—Plan of Base of a Built Steel Stanchion.

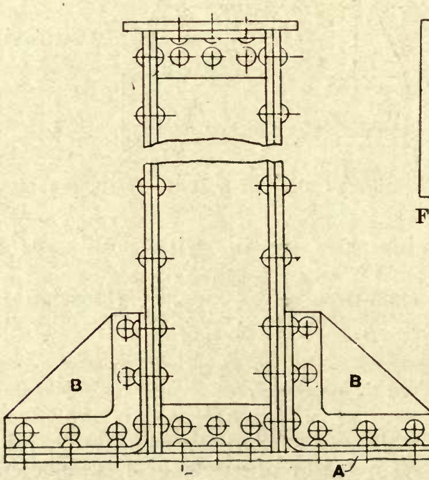

Fig. 42.—Elevation of a Built Steel Stanchion.

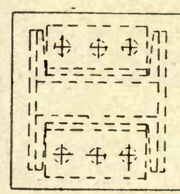

Fig. 43.—Plan of Cap of a Built Steel Stanchion, Fig. 42.

Solid rectangular mild steel columns, ends fixed,

$$B\,W = \frac{30}{1 + \frac{r^2}{2480}}\text{; if ends free} = \frac{30}{1 + \frac{r^2}{620}}.$$

Solid round mild steel columns, ends fixed, B W

$$= \frac{30}{1 + \frac{r^2}{1100}}\text{; ends free} = \frac{30}{1 + \frac{r^2}{350}}.$$

The above formulæ are shown graphically in the diagram given below (Fig. 44).

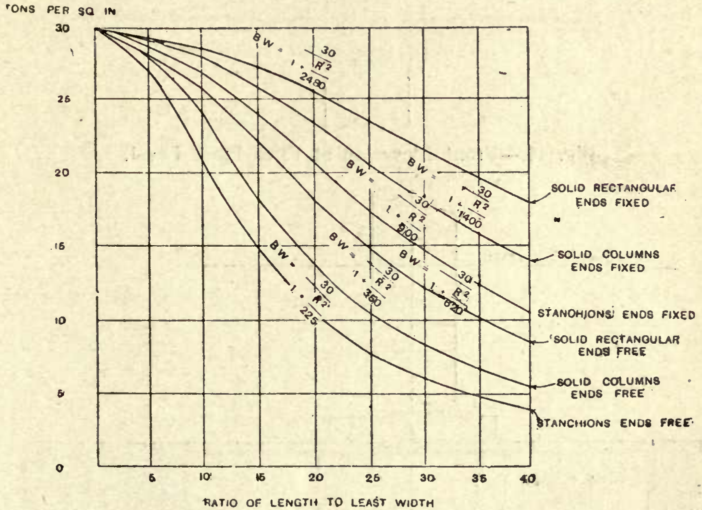

Fig. 44.—Curves showing Breaking Weight of Steel Columns and Stanchions.

Figs. 45 to 52 show sectional front and side elevations of solid steel columns supporting three floors, and supply details of the construction of the foundations and girder connections; Fig. 53 being a sectional plan at second floor level, and Fig. 54 a plan at base below ground floor. These figures are from working drawings kindly supplied by Messrs. Richard Moreland

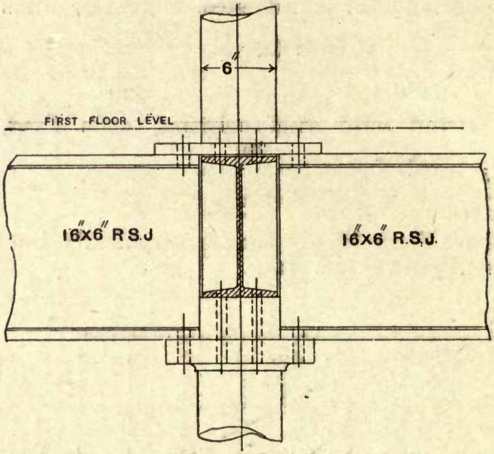

Fig. 46.—Front Elevation at First Floor Level.

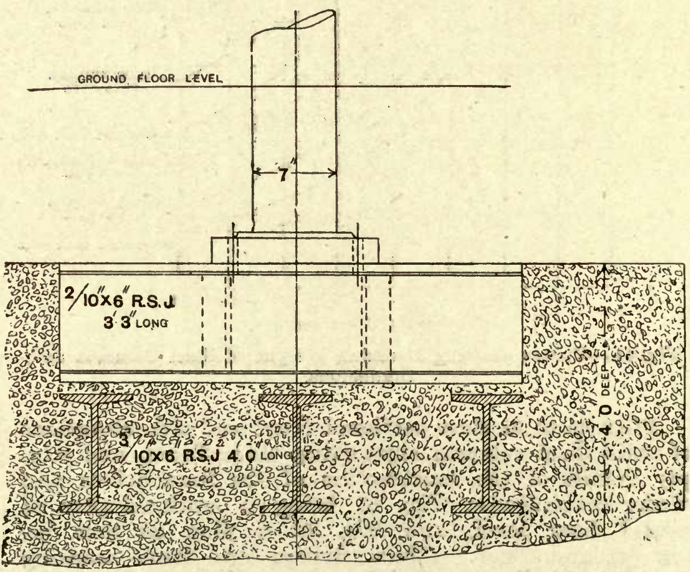

Fig. 45.—Base and Foundation of Column below Ground Floor,

and Son, Limited, engineers, 3, Old Street, E.C. It will be noted how neatly the girder connections are made; and the columns, being small, can be adequately protected without giving them a clumsy appearance. Indeed, such

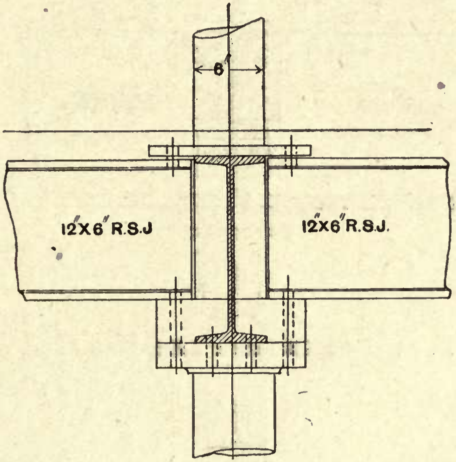

Fig. 47.—Side Elevation First Floor Level.

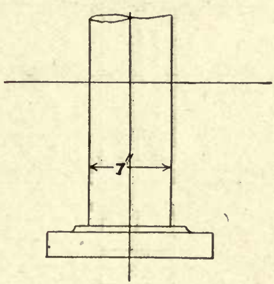

Fig. 48.—Base of Column below Ground Floor.

a section, exposing as it does the least possible surface in proportion to area, would undoubtedly remain sound if unprotected longer under fire than any other section. There is little doubt that there is a great future for these columns in London, and in other centres of population, where space is so valuable, and especially for shop fronts, where the least obstruction to space or light is resented.

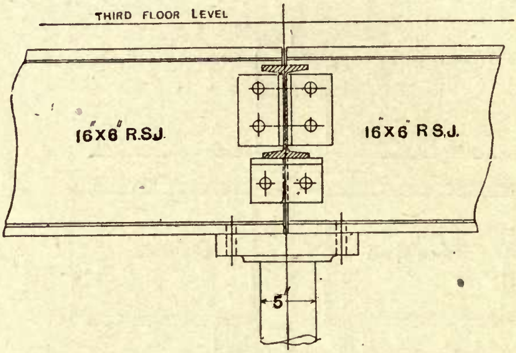

Fig. 49.—Front Elevation Third Floor Level.

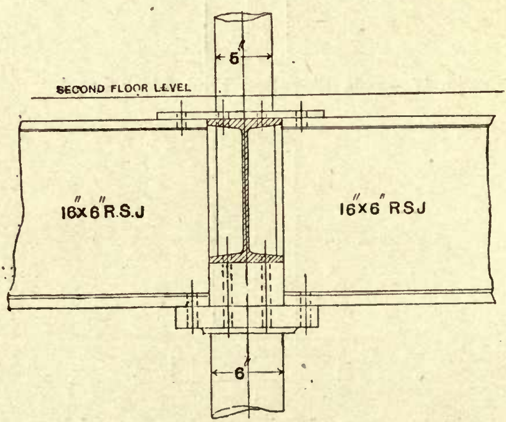

Fig. 50.—Front Elevation Second Floor Level.

The columns are made from ordinary mild rolled steel, and are very uniform in nature. The caps and bases are turned out of solid steel and shrunk on, this connection being equal in strength to a solid flange, care being taken to provide enough shrinking area. To obtain this, the thickness of the cap and base should be about half the

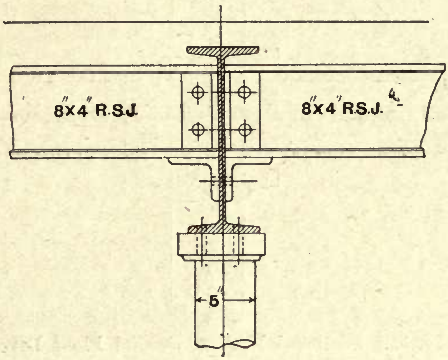

Fig. 51.—Side Elevation Third Floor Level.

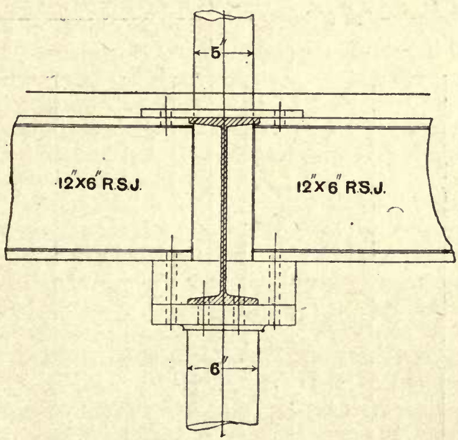

Fig. 52.—Side Elevation Second Floor Level.

diameter of the column. The bearing surfaces are all turned after shrinkage, and the result is a very neat and first-class engineering job. The firm last named keep

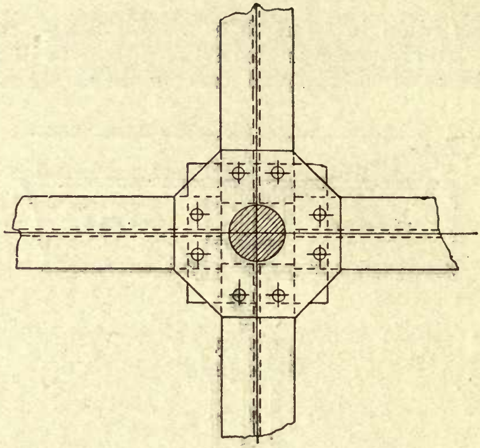

Fig. 53.—Sectional Plan at Second Floor Level.

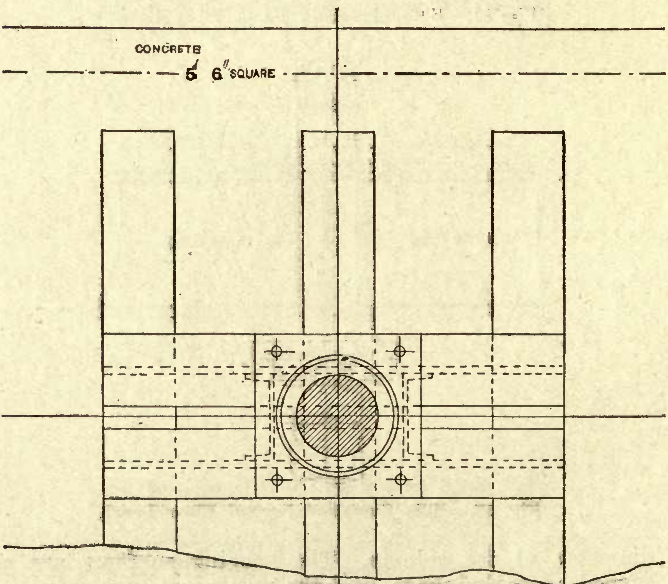

Fig. 54.—Plan of Base below Ground Floor.

a large stock of these columns, and, having recently put down a special modern plant, can supply them almost as cheap as cast-iron columns and quite as cheap as steel stanchions to carry the same weight.

A steel beam grillage (10-in. by 6-in. rolled steel joists) foundation is shown. The beams are embedded in good Portland cement concrete, and consequently distribute the load over a sufficiently large area of ground, without putting tensile strains on the concrete. In ordinary cases 3 tons per sq. ft. can be safely imposed on ground in the London district, but, of course, in bad localities and in proximity to the river this amount must be considerably reduced, and the area and strength of the grillage proportionately increased.

A table (taken from Messrs. Moreland's catalogue) of strength for solid steel columns is given. It will be seen from this table that a 9-in. solid steel column, 14 ft. long, will carry 210 tons safely. To support the same load under similar conditions a 15-in. diameter hollow cast-iron column, 1½-in. thick, would be required, or a 15-in. by 18-in. built steel stanchion, composed of two 14-in. by 6-in. rolled steel joists, and two 18-in. by ½-in. cover plates. To equal a 6-in. solid steel column, a 10-in. diameter hollow cast-iron column, 1⅛ in. thick, would be required, or a built steel stanchion composed of one 10-in. by 6-in. rolled steel joist, and two 12-in. by ½-in. plates. The above comparison illustrates very clearly the advantages of the solid steel column.

Notes, of which the following is the substance, are appended to the table: (1) The steel in the rolled bars is of superior quality, and so uniform in its nature that it entirely supersedes cast-iron, which as a metal for columns is so very unsatisfactory that the firm consider that the increased reliability that can be placed on the solid steel columns makes their use very desirable. (2) It may appear at first sight that hollow columns of steel should be used, but at the present time the steel makers are not prepared to supply these at anything like the cost of solid steel; and if cast steel were used it would cost a sum too high for practical purposes. A solid steel column is theoretically an expensive section to use, but the practical issue is in favour of it as regards expense.

Solid Steel Columns.

Diameter in Inches.	Weights per foot lb.	Table of Safe Loads in Tons for Heights in Feet. 8	10	12	14	16	18	20	22	24	26	28	30
3	25	13	9	7	6								
3½	35	20	15	11	8								
4	46	30	23	17	13								
4½	57	46	37	28	22								
5	70	64	53	42	33	26							
5½	85	85	69	57	45	36							
6	101	104	88	74	59	48	40						
6½	118	126	109	93	77	64	53						
7	136	150	135	116	100	85	69						
7½	156	182	162	143	122	105	84						
8	178	221	196	176	151	131	111	96					
8½	200	259	332	205	180	157	138	117					
9	224	299	273	235	210	184	165	140					
9½	250	341	309	273	244	216	195	167					
10	277	385	346	314	283	252	228	204	173	157			
10½	305	430	390	359	325	291	264	239	208	186			
11	334	476	437	409	371	333	304	276	247	219			
11½	365	529	488	453	417	375	348	313	282	254			
12	398	588	543	498	464	418	396	351	317	294	260	238	219

Weights do not include caps and bases.

(3) The columns being solid, the risk of failure in the event of fire is greatly lessened, and the small size is a great advantage; while there is but little difference in cost as compared with steel stanchions and cast-iron columns.

The caps and bases are made of solid metal, and are faced.

In the above table (p. 40) the ends are considered to be imperfectly fixed, and the loads given allow an ample margin for safety.

CHAPTER IV.

FOUNDATIONS FOR COLUMNS; LOADS ON COLUMNS.

A GENERAL idea of the safe crushing resistances of the chief materials used in forming the foundations of columns is afforded in the following list. With respect to base stones, it is taken for granted that they are true and solidly bedded, as it is impossible to make an allowance for bad or slovenly work. The figures represent tons per square foot: Granite, 25-50; limestone, 20; sandstone, 15; blue Staffordshire bricks (in cement), 15; brickwork, ordinary, in cement, 5; brickwork, ordinary, in mortar, 3; Portland cement concrete (5 to 1), 15; Portland cement concrete (10 to 1), 7. In relation to these figures, the base stone should never be less than 1 ft. thick; if rectangular, its thickness should be at least one-fifth its longest side; and if square, one-quarter its width. The weight coming down the stanchion must be conducted through the base stone, brickwork, and concrete, without putting a greater intensity of pressure on any of these materials than it can safely resist, and so as to put on the soil only so much as it can safely support. The maximum loads on foundations may be thus stated, the figures representing tons per square foot: Clay in thick beds, very dry, 4; moderately dry, 2; soft, 1; dry, coarse gravel, 6-8; compact dry sand, 4; solid dry natural earth, 4-6.

Figs. 55 and 56 show, respectively, plan and elevation of a foundation for a stanchion. The built steel stanchion A has a cast-iron base B, to which it is secured by bolts (see holes C, Fig. 56), in order to distribute the weight on to the base stone D. The height of the cast base is one-half its base width. The base stone has the proportion mentioned before. The Portland cement concrete E should be at least 18 in. thick, and it should not project beyond the brickwork more than half its thickness. The batter of the brickwork is about 1 to 2 of rise. There is a cement bed between the cast-iron base B and the stone base D.

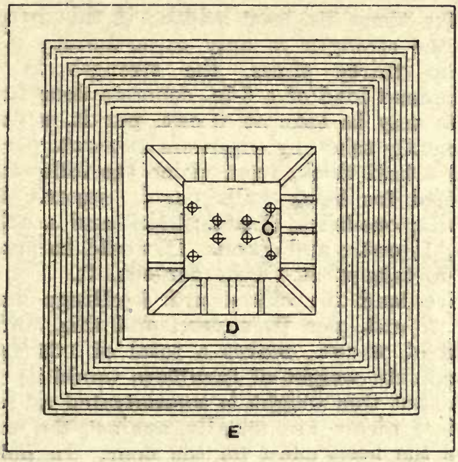

Fig. 55.—Foundation to Stanchion: Plan.

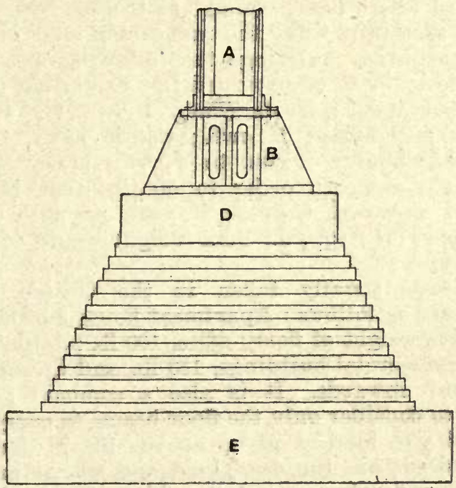

Fig. 56.—Foundation to Stanchion: Elevation.

In order to possess the full crushing strength of the material, no brick pier should have a height greater than twelve times its least width; if this proportion is doubled, the strength is only seven-tenths; if it is increased to thirty times, the strength is only one half. The dead load of a 6-in. concrete floor formed with steel joists may be take at ·6 cwt. per ft. super. Brickwork is usually taken by engineers at 1 cwt. per ft. super. brick and a half thick, from which the following weights are obtained for brick walls per ft. super.: Half-brick wall, $\frac{1}{3}$ cwt.; one-brick, $\frac{2}{3}$ cwt.; brick and a half, 1 cwt.; two-brick, $1\frac{1}{3}$ cwt.; and so on. It would be more correct, however, to take it at 1 cwt. per cub. ft.

The live load for offices and dwellings may be put down at ·75 cwt. per ft. super., and this, added to the dead load of ·6 cwt., makes a total of 1·35 cwt. In an average case the weight of furniture would be ·2 cwt. per ft. super., but this weight is concentrated at points, and the heaviest pieces are usually against the wall, where they have the least effect on the floor. In public buildings where a large number of people congregate, and warehouses where heavy goods are stored, care should be taken to ascertain what the maximum load is likely to be. For ordinary buildings the following may be taken as the safe loads to be provided for, excluding the weight of the floor itself: Roofs (flat), 1 cwt. per ft. super.; dwellings and offices, $1\frac{1}{2}$ cwt.; public halls, $1\frac{1}{2}$ cwt. to 2 cwt.; warehouses, 2 cwt. to 4 cwt.; heavy machinery, 3 cwt. to 5 cwt. In order to obtain some idea of the weight of a crowd, suppose it were possible to get six 12-stone men on 1 sq. yd., their weight would equal 1 cwt. per ft. super.

The loads usually taken in the United States of America are as follows: Apartment house and hotel, 70 lb. per ft. plus weight of floor; office, 100 lb.; assembly rooms, 120 lb.; commercial buildings, 150 lb. and upwards; roofs, 50 lb. and upwards. It is also a common practice in America to consider only the floor beams to carry the total dead and live load as given above; the girders are calculated to sustain the dead load and 80 per cent. of the live load, and columns and stanchions the dead load and only 70 per cent. of the live load. What justification there is for this proceeding is not quite clear.

CHAPTER V.

CALCULATING WEIGHTS OF STANCHIONS, GIRDERS, ETC.

AFTER designing a cast-iron or steel stanchion it is necessary, in order to estimate its cost, to find out its weight. Now a plate of wrought-iron 1 ft. square and 1 in. thick weighs 40 lb., mild steel is 2 per cent. heavier, and cast-iron of good grey quality 5 per cent. lighter. In taking off the weight of the following cast-iron stanchion it will be assumed that the metal weighs 40 lb. per ft. super 1 in. thick, as the rounding of the angles would add about 5 per cent. to the weight, and this would otherwise not be taken account of in the estimate.

The weight of a 1-in. sq. bar 1 ft. long being 3·3 lb., by multiplying the sectional area of anything by 3·333, or, what is the same thing, $\frac{10}{3}$, the weight per foot run is obtained. For example, imagine the weight of a 6-in. by 1-in. bar is required $\frac{6 \times 10}{3} = 20$ lb. per ft. run. Brackets and lugs must be cubed and taken at ·262 lb. per cub. in. The stanchion shown in plan in Fig. 57 and elevation in Fig. 58 is 12 ft. long, 9 in. by 9 in. in the shaft, with 15 in. cap and 24 in. base. A plan of the cap is shown in Fig. 59. The shaft F is 1 in. thick, and the cap G and the base H $1\frac{1}{4}$ in., and the brackets J and stiffeners K 1 in.

The area of the shaft is

$$2\,(9 \times 1) + 7 \times 1 = 25 \text{ sq. in.}$$

Therefore the weight per foot run is $\frac{25 \times 10}{3} = 83{\cdot}3$ lb., and the weight of the whole shaft $83{\cdot}3 \times 12 = 1{,}000$ lb. The base, being $1\frac{1}{4}$ in. thick, will weigh 50 lb. per ft. super., $2 \times 2 \times 50 = 200$ lb., and the cap $1{\cdot}25 \times 1{\cdot}25 \times$

50 = 78 lb. There are six stiffeners, three on each side—

$$6 \times 7 \times 4 \times 1 \times \cdot 262 = 44 \text{ lb.}$$

The projection of the cap being 3 in., the rate of the six brackets 45 degrees, the area of each will be $\cdot 5 \times 3 \times 3$, and total $3 \times 3 \times 3 \times 1 \times \cdot 262 = 7$ lb. Base brackets

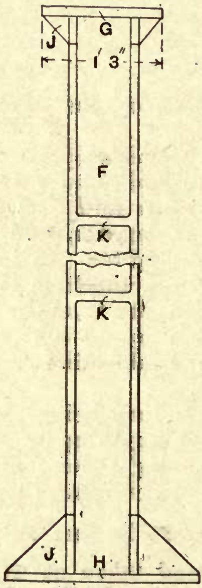

Fig. 58.—Cast-iron Stanchion : Elevation.

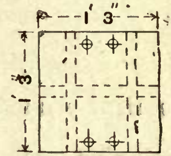

Fig. 59.—Cap for Cast-iron Stanchion : Plan.

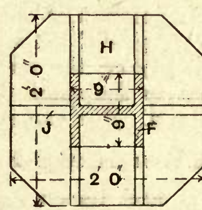

Fig. 57.—Base of Cast-iron Stanchion : Plan.

$3 \times 7 \cdot 5 \times 7 \cdot 5 \times 1 \times \cdot 262 = 44$ lb. Total weight of stanchion will be—

Shaft	1000 lb.
Base	200 ,,
Cap	78 ,,
Stiffeners	44 ,,
Base brackets	44 ,,
Cap brackets	7 ,,
	1373 lb. = 12 cwt. 1 qr.

The weight of a cast-iron column can be obtained in the same manner when the area of the shaft is known. Take for example a 10-in. column, 1-in. metal :—

Area $= \cdot 7854\ (d^2 - d_1^2)$ and if this is multiplied by $\frac{10}{3}$ the weight per foot run is at once obtained—

$$\cdot 7854\ (10^2 - 8^2)\ \tfrac{10}{3} = 94 \cdot 2 \text{ lb.}$$

If a column is tapered, the average diameter is taken when the approximate weight only is required; 3 ft. is added to the length of the shaft to allow for the cap and base if these are of average size.

The weights of the different sections comprised in built-up girders and stanchions can easily be obtained from the

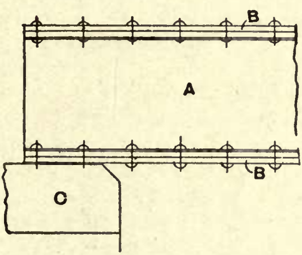

Fig. 60.—Part Elevation of Compound Girder.

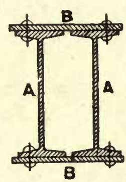

Fig. 61.—Section of Compound Girder.

manufacturers' tables, or the area can be obtained (the size and thicknesses being known), and this multiplied by $\frac{10}{3}$ will give the weight per foot run.

For example, take a compound girder (Figs. 60 and 61) 25 ft. long, composed of two 16-in. by 6-in. rolled steel joists A, with 14-in. by $\frac{1}{2}$-in. plate B at top and bottom, resting on chamfered stone template C. The average thickness of the flange is $\frac{27}{32}$ in., and of the web $\frac{9}{16}$ in., so that the sectional area of each joist is 18·23 sq. in., and of both 36·5.

$$\text{Weight of joists} = \frac{36 \cdot 5 \times 10}{3} = 122 \text{ lb. per foot run.}$$

$$\text{Weight of plates} = 2 \times \frac{7}{6} \times 1 \times 20 = 46 \cdot 6 \text{ per foot run,}$$

Total of girder—

Joists	122	× 25 =	3050	
Plates	46·6	× 25 =	1165	
			4215	
	7%		295	
			4510	= 2 tons 0 cwt. 1 qr.

Five per cent. must be added to all riveted work; there is also 2 per cent. to add for steel, making 7 per cent. altogether; $2\frac{1}{2}$ per cent. is also sometimes added to cover the rolling margin.

CHAPTER VI.

BOLTS, CONNNECTIONS, AND RIVETS.

THE length of a bolt is taken from the under side of the head to the point, and to obtain the weight add seven diameters to the length to allow for the head and nut.

Thus, a 1-in. bolt 12 in. long would weigh—

$$\cdot 7854 \times 1 \times 1{\cdot}6 \times 3{\cdot}3 = 4{\cdot}2 \text{ lb.}$$

The thickness of the bolt-head should be about three-quarters of the diameter, and of the nut one diameter, and the width across the angles of the head and nut two diameters.

When bolting up ironwork, care should be taken to obtain bolts of the exact lengths required, so that packing washers are unnecessary; but tapered washers are unavoidable where the nut rests against the sloping flange of a joist in order to obtain a true bearing. The nut should not be forced home, but left when it has got a good solid bearing; otherwise a serious strain will be put on the bolt without rendering the work more secure. The point should be downwards, so that should the head by any chance work loose the other part will remain in position.

The next thing to consider is the provision to be made for receiving and supporting the ends of girders on columns and stanchions that continue through more than one storey. Where a number of girders have to be supported by a cast-iron column or stanchion at the same level, what is known as a stilting is provided. This may form part of the lower column if practicable, or be a short separate casting bolted to the cap of the lower and base of the upper column. The only consideration to be given to it is that its sectional area must be equal to that of the column that rests on it, as it has to communicate its load to the column below.

As previously mentioned, it is of the utmost importance that where columns ascend through more than one storey they should be continuous, so that the load can be conducted to the foundations without a break. In no case should the base of one column rest on a girder or on girders supported on the cap of the column below, for directly the load comes on these girders they will deflect, and tend to tilt the column resting on them out of the vertical, and so set up bending stresses that the columns are not intended to resist. A similar objection arises when a column is supported by a girder at any distance between its supports.

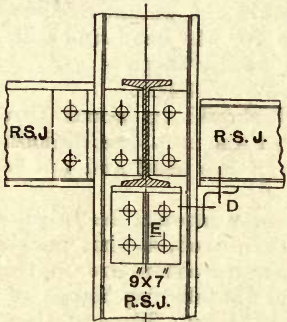

Fig. 63.—Elevation of Fig. 62.

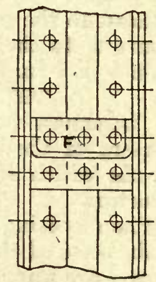

Fig. 65.—Elevation of Fig. 64.

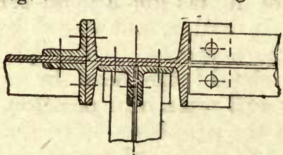

Fig. 62.—Plan showing Three Methods of Connecting Girders to Built-up Steel Stanchion.

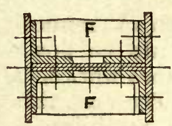

Fig. 64.—Plan showing Stiffener to Built-up Steel Stanchion.

The vibration arising from the deflection of the girder under a varying load will put a most injurious racking stress on all connections, tending to make them work loose, and, indeed, to strip the threads of connecting bolts if these are screwed up too tightly.

It is here only intended to consider the most simple cases of girder connections to columns and stanchions. Figs. 62 and 63 illustrate three ways in which girders may be connected to built-up steel stanchions or rolled steel joists used as such—namely, either by cleats D riveted to their webs and bolted or riveted (preferably the latter) to the shaft; or by simply resting on angle or T steel brackets E riveted to the shaft at the exact level required. In either case the girder obtains support by the shearing resistance of the rivets in the connection.

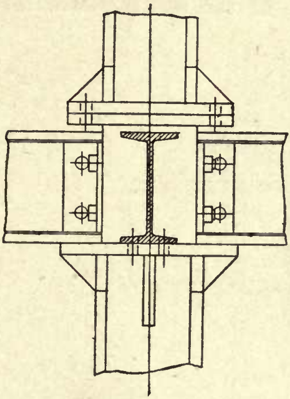

Fig. 67.—Elevation of Fig. 66.

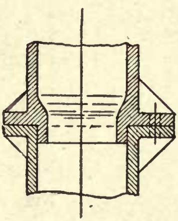

Fig. 68.—Spigot Joint to Stanchion.

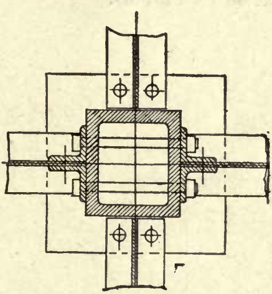

Fig. 66.—Plan of Square Stilting to Cast-iron Stanchion.

Figs. 64 and 65 show a T steel stiffener F for a built-up steel stanchion, the web being turned up and riveted to the flange.

Figs. 66 and 67 show a square stilting forming part of the lower stanchion. Two rolled steel joists are shown resting on the cap with their ends double cleated and secured by bolts passing through the stilting. If two girders obtained support on the two remaining sides, and were secured in the same way, the bolts passing through the stilting would have to be arranged to clear one another. An alternative method of securing the girders would be to make the body of the stilting circular, and

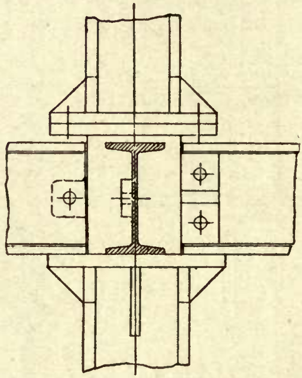

Fig. 70.—Elevation of Fig. 69.

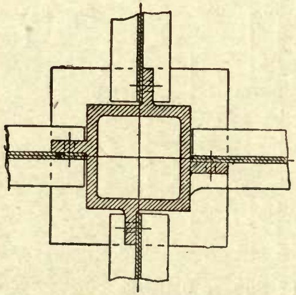

Fig. 69.—Plan of Square Stilting with Lugs to Cast-iron Stanchion.

then bolting the ends of opposite girders to bands passing round and gripping the body of the stilting. Another method, shown in the illustration, is to bolt the bottom flange direct to the cap; $\frac{3}{4}$-in. bolts would be used in such a case, and placed as near to the edge of the cap as possible—that is, within $1\frac{1}{2}$ in. diameters or $1\frac{1}{8}$ in. in this case, and with their points downwards. The base of the stanchion above is bolted to the stilting by four $\frac{3}{4}$-in. bolts, but a spigot joint (Fig. 68) may be used in such cases, either singly or assisted by bolts.

Figs. 69 and 70 show a plan and elevation of a square

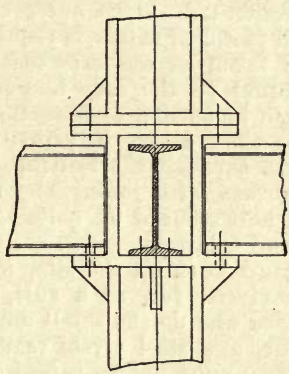

Fig. 72.—Elevation of Fig. 71.

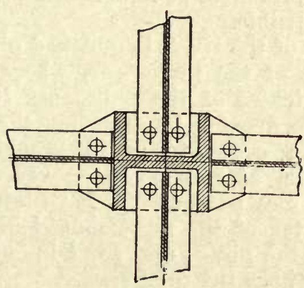

Fig. 71.—Plan of Pocket or Bracket Supports to Girders on Cast-iron Stanchion.

stilting with lugs cast on through which the bolts securing the ends of the girders pass. The holes in the lugs should be large enough to allow play for the bolt when the girder deflects, and to permit a solid bearing to be obtained on the cap; for, of course, the lugs are not intended to take any weight, but only to act as ties to prevent the girder creeping off its bearing.

In Figs. 71 and 72 the girders rest on brackets, and are secured to them by bolts. As the stress put on these projections may be either shearing or bending (probably the latter, for directly the load comes on the girder it will deflect), its extreme end will rise, and the whole load will be concentrated at the outer edge. In a case of this kind a high factor of safety is required; $1\frac{1}{2}$ tons per sq. in. should be taken as the safe shearing resistance, and this being equal to the safe tensile resistance the bracket will be safe in tension or shearing.

Figs. 73 and 74 show a case in which a girder is continuous and passes through the stilting, which has consequently to be formed with jaws. These may be in two separate castings and secured by bolts to the column or stanchion above and below.

When riveting up work on the site, it is advisable to use wrought-iron rivets, for, as a rule, it is men with but little experience who do the small amount of riveting there is to be done, and steel rivets require very careful manipulation. They must not be raised above a dull red heat, and the points must be hammered down and snapped as quickly as possible.

The diameter of the rivets in punched plates is governed by the thickness of the plates—or, where the thicknesses vary, by the thickest of them—because it is found that the hole must be greater in diameter than the thickness of the plate, in order to avoid breaking the punch. Drilled holes are, of course, not governed by this consideration; they may be smaller. For plates less than $\frac{1}{2}$ in. thick the diameter of the rivet should be equal to twice the thickness of the plate; and for $\frac{1}{2}$-in. plates and above, one and a half times the thickness.

The minimum pitch—that is, the distance from centre to centre of the rivet holes—is two diameters, and the minimum distance between the centre of the rivet and the edge of the plate one and a half diameters. The pitch of rivets in building work is usually 4 in. to 6 in., and in the tension flanges is sometimes even more than this; but although a 9-in. pitch is permissible for securing angle stringers supporting floor joists to the webs of girders, a greater pitch than 6 in. should not be permitted in the chief members. Where a girder is in an exposed position, the pitch of the rivets should not exceed twelve times the thickness of a plate, otherwise the joint will not be perfectly tight, and moisture may enter, set up rust, and eventually burst the plates asunder.

A rivet is slightly smaller than its nominal size, and the hole it fits into is made larger than its nominal size, the punch being $\frac{1}{32}$ in. greater in diameter. The die, too, is larger than the punch in order to allow clearance, so that the resulting hole is slightly conical in form.

When putting punched plates together for riveting up, the top of the plates must be put together, so that the edges coincide, otherwise there will be a sharp edge at

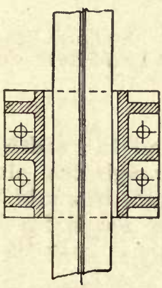

Fig. 73.—Plan of Stilting with Loose Jaws to Cast-iron Stanchion.

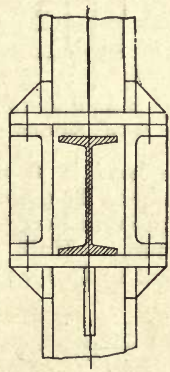

Fig. 74.—Elevation of Fig. 73.

the centre which will assist the shearing of the rivet. The conical form of the hole gives the rivet a better hold on the plates, and they have been known to keep the plates together when the heads were completely rusted away.

Steel rivets fill the holes better than iron ones, as they are worked at a lower temperature, and consequently contract less in cooling.

Machine riveting is much to be preferred to that done by hand, as the machine first forces the plates together, and then puts a pressure of as much as 50 tons on the rivet, so that the shank is squeezed up and fills any irregularities in the hole. The difference in the qualities of the two kinds of work is apparent if the heads are cut off; a hand-formed rivet can be punched out, but a machine-formed one would have to be drilled out.

Fig. 75 illustrates a snap, button or cup-headed rivet in a drilled hole before the point is formed. The depth

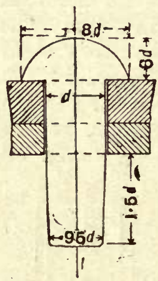

Fig. 75.—Snap Rivet in Drilled Hole : Section.

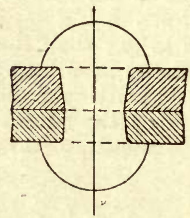

Fig. 76.—Finished Snap Rivet in a Punched Hole : Section.

of the head is ·6 of the diameter, and the width is 1·8 diameters. The point projecting through the plates is 1·5 diameters long; if for machine work, it would be 1·75 diameters. Half the length of the shank is slightly tapered, so that at the extreme end it is only ·95 of its

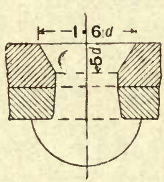

Fig. 77.—Countersunk Rivet in a Punched Hole : Section.

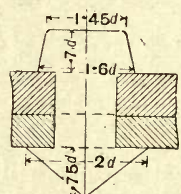

Fig. 78.—Pan or Hammer-headed Rivet in Drilled Hole : Section.

diameter at the root. The sharp outer edges of the plates are sometimes drilled off, as they are found to assist materially the shearing of the rivet.

Fig. 76 shows a snap-rivet in punched plates, and illustrates the way such plates should be put together for riveting up. Before the rivet is inserted, a drift punch is forced in the hole to do away with any unevenness.

Fig. 77 shows a countersunk point. The depth of the head is ·5, and its width 1·6 in relation to the diameter. The slope of the sides in the illustration is obtained by joining the extremes of the head with the centre of the rivet hole in the opposite plate.

A pan-headed rivet with a hammered conical head is shown in Fig. 78, with the proportions the different parts bear to one another figured on. The above-mentioned proportions are those which it is found necessary to give rivets in practice; but, theoretically, in the case of a rivet subject to a longitudinal strain, the tensional strength of the shank must be equal to the shearing resistance of the head. Considering a steel rivet, let d = diameter of rivet in inches, and h = height of head in a line with the shank. Now the area of the shank is $\cdot 7854\ d^2$, and the surface of the head to resist shear is $3\cdot 1416\ d \times h$, and taking the safe strength in tension at 6·5 tons and in shear at 5 tons per sq. in., we get—

$$\cdot 7854\ d^2 \times 6\cdot 5 = 3\cdot 1416 \times h \times 5,$$

and

$$h = \cdot 3\ d;$$

so that the height of the rivet head in a line with the shank must be at least ·3 of the diameter; in practice it is made at least $\frac{1}{2}\ d$.

The bearing surface of the rivet head has also to be considered, and this must equal the tensional resistance also. Let D = the diameter of the head, and d that of the rivet, and take the safe resistance to bearing at 8 tons.

$$\cdot 7854\ (D^2 - d^2) \times 8 = \cdot 7854\ d^2 \times 6\cdot 5$$

$$D = 1\cdot 34\ d;$$

but in practice the diameter of the head is made more than $1\frac{1}{2}\ d$, because, as explained before, the diameter of the hole may be larger than its nominal size.

The riveting gang consists of two riveters, the holder-up, and a boy to attend to the forge for heating the rivets. The tools they require are riveting hammers, tapered steel punches, a snap—a tool with hollow exactly the shape of the finished rivet point—and a sledge-hammer to strike the snap. The holder-up has either an ordinary hammer or a heavy iron, hollow in the end, to fit exactly the head of the rivet and keep it in position while the point is being hammered up. The rivets are heated by passing them through a perforated plate, by which means the points are heated while the heads remain comparatively cool. The riveters first drive a tapered steel punch through the hole where the rivet is to be inserted, to do away with any unevenness; and the holder-up, standing opposite, knocks it out again. The boy brings a red-hot rivet from the forge with a pair of tongs, and hands it to the holder-up, who places it in the hole, with the red-hot point towards the two riveters, and then presses his heavy hammer against it. The riveters at once hammer the plates directly round the hole together, and then proceed to hammer down the point to nearly the finished shape required. Before the point loses its colour, the snap is placed over it and struck with a heavy hammer, so that the point is formed perfectly true and even.

Bolts may have either a minus (Fig. 80) or a plus

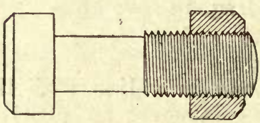
Fig. 79.—Plus Threaded Bolt.

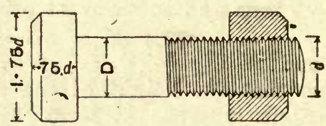

Fig. 80.—Minus Threaded Bolt.

thread (Fig. 79), but the use of the former in building work is limited. The minus thread cuts into the bolt and consequently reduces its effective diameter by twice the depth of the thread. Whitworth threads are in general use in this country, and the ratio the effective diameter bears to the nominal in the case of a minus thread is about (see Fig. 80) $d = \cdot 9$ D. Theoretically the resistance to stripping of the nut must equal the tensile resistance of the bolt where reduced in diameter by the thread. The proportions the head and nut should bear can be obtained by similar calculations to those given for rivets; but, owing to the metal being injured in cutting the thread, the depth of the nut is made equal to the diameter of the bolt, the depth of the head three-quarters of the diameter, and the width nearly two diameters. The thread on the bolt is called the male and that on the nut the female.

CHAPTER VII.

JOISTS AND GIRDERS.

ROLLED steel joists can be obtained from 3 in. to 20 in. deep, and from these most of the requirements of a building can be met. When a suitable section of joist cannot be found, a compound girder may be employed—

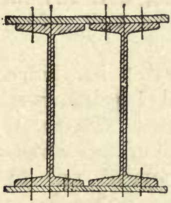

Fig. 81.

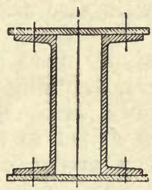

Fig. 82.

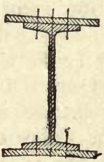

Fig. 86.

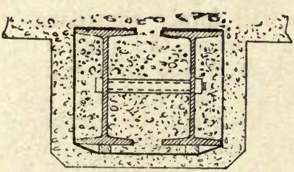

Fig. 83.

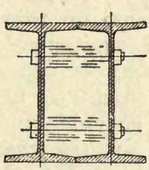

Fig. 84.

Fig. 85.

Fig. 81.—Compound Girder Composed of Two Rolled Steel Joists and Top and Bottom Plates: Section. Fig. 82.—Compound Girder Composed of Two Channels and Top and Bottom Plates: Section. Fig. 83.—Coupled Girders Spaced Apart and Encased in Concrete: Section. Fig. 84.—Coupled Girder with Cast-iron Distance Piece for Two Bolts: Section. Fig. 85.—Cast-iron Distance Piece for Two Bolts: Section. Fig. 86.—Rolled Steel Joist with Top and Bottom Plates: Section.

that is, a combination of joists or channels with plates, as shown in Figs. 81 and 82. Coupled girders may be used (Figs. 83 and 84) where the load is uniform and each joist receives its share of the load direct. The joists are bolted together about every 3 ft. with $\frac{3}{4}$-in. bolts passing through gaspipe, used as distance pieces. Cast-iron distance pieces through which the bolts pass, as shown in Figs. 84 and 85, are preferable in such cases. Fig. 86 shows a plate riveted to both flanges of an ordinary joist to strengthen it; the increase in strength with a plate on one flange only is trivial.

Full particulars and tables of strength of the different joists, etc., rolled are published by the different makers, so that, when the span and load to be supported are known, the most suitable section is readily determined. The minimum weights for the different sections are the most economical to use, because the rolls have to be spaced wider apart to obtain the heavier weights, and any metal added to the flanges is also added to the web, where it is not required.

The stock lengths of joists may vary 1 in. more or less from the nominal; cutting to dead lengths, with a variation of $\frac{1}{8}$ in. only, is charged extra. The taper of the flange is usually rolled to an angle of 98° with the web. The depth of a girder or joist ought not to be less than $\frac{1}{20}$ of the span, otherwise the deflection will be considerable. For girders, $\frac{1}{17}$ is a good proportion; any great increase on this ratio will imply considerable deflection, and consequent trouble when the plastering comes to be done. If sufficient depth is not allowed in a floor for the joists to have proper proportions, their deflection can be calculated, and they can be cambered to that extent, so that when loaded their soffits are level. The flange width must also be taken into account, because, if its ratio to the span is more than $\frac{1}{40}$, it will be liable to fail by buckling sideways. With joists in a solid floor this danger does not arise, as the concrete struts them.

Deflection is a most important consideration, and one often overlooked by architects, especially where brickwork or stonework is supported. Very slight deflection will cause cracks to appear, and give endless trouble to set right. All girders in important positions should have their deflection calculated under the load, and this should not exceed $\frac{1}{40}$ of an inch per foot of span. If they are cambered to this extent, their soffit will be level when they are fully loaded.

In the case of an important building where the architects insisted on several of the main girders being kept within the depth of the floor, for fear of spoiling the effect below, no stipulation was made to the engineers who were asked to compete as to permissible deflection, or what factor of safety was to be employed: with the result that, in order to keep the price down, a factor of safety of less than three was used, and before the building was completed deflections in many of the girders, and consequent cracks in the stonework, necessitated fixing intermediate stanchions, at great expense, in objectionable positions.

The next thing to consider is the connections of joists with one another. Fig. 87 shows a small joist joggled bottom flange and single cleated into another of deeper section. The joggle arises from the necessity of keeping

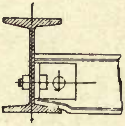

Fig. 87.—Joggle and Cleated Joint.

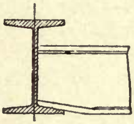

Fig. 88.—Common Joggle Joint.

the bottom flange of the joists flush—a frequent requirement in building work. If this joggle is forged into shape, and the flange remains connected to the web, this makes an excellent job; but that is seldom the case in practice. For cheapness' sake, it is more often done by cutting away a V-shaped piece of the web directly above the bottom flange, and hammering the flange up to meet it, as shown in Fig. 88. The disconnected bottom flange then only acts as a packing to prevent the web cutting into the flange of the other girder. A good job would be made by cutting away the bottom flange and riveting a small angle on each side of the web, or, better still, by double cleating the end and notching the bottom flange so that the load is conducted to the web of the main joist or girder by the bolts of the cleats. In this way the twisting stress on the bottom flange, tending to tear it away from the web, would be avoided. Of course, if there was an equal load on the bottom flange on each side of the web, there would be no twisting stress; the one would balance the other. When the load is communicated to the web of the main joist by the aid of cleats, it is apparent from Fig. 87 that a single cleated end is of no use, as the joist would swing round on one rivet; and even if the joists were deep enough for the cleat to have two bolts or rivets in it, there would still be a racking stress, and the only way to avoid this is by having a double cleated end, as in Fig. 89. Where a joggled or notched bearing is used, or even where one joist rests on the

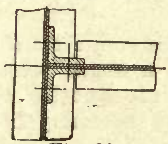

Fig. 89.

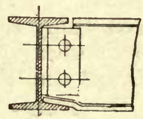

Fig. 90.

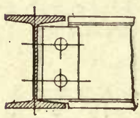

Fig. 91.

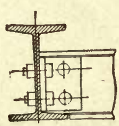

Fig. 92.

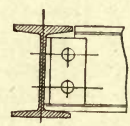

Fig. 93.

Fig. 89.—Notched and Double Cleated Joint Section. Fig. 90.—Notched, Joggled, and Cleated Joint. Fig. 91.—Double Notched and Cleated Joint. Fig. 92.—Notched Bottom and Cleated Joint. Fig. 93.—Notched Top and Cleated Joint.

bottom flange of another, it is necessary to cleat the joint to prevent any possibility of its creeping off its bearing. Joists with one end embedded in a wall and subject to a pull have their ends double cleated in order that the brickwork may get firm hold of them.

Fig. 90 shows a joist notched top flange, joggled bottom, and either single or double cleated. Fig. 91 shows a joist notched top and bottom flanges and cleated. It will be noticed that the web of one joist is cut to the taper of the bottom flange of the other. Fig. 92 shows a joist notched bottom flange and cleated; Fig. 93, notched top flange and cleated. Practically all the ways in which joists are connected together have now been enumerated, but it must be remembered that labours must be avoided as much as possible, as they increase the cost of work immensely.

The following table gives the approximate cost of labours on different sections of joists. It must be remembered, however, that the prices charged by the different contractors vary considerably, depending on the number of labours and the amount of machinery at command, but the following will be found to be fair average prices:

The cleats are taken to be full size for the section,

LABOURS ON ROLLED STEEL JOISTS.

Section of Joist.	*Cut.*				*Notch.*				*Joggles.*				*Cleat.* Including Bolts.			
	Square.		*Splay.*		*Square.*		*Splay.*		*Square.*		*Splay.*		*Single.*		*Double.*	
	s.	*d.*	*s.*	*d.*	*s.*	*d.*	*s.*	*d.*	*s.*	*d.*	*s.*	*d.*	*s.*	*d.*	*s.*	*d.*
$4 \times 1\frac{3}{4}$	0	6	0	9	0	6	0	9	1	0	1	3	1	0	1	9
4×3	0	9	1	0	1	0	1	3	1	3	1	9	1	0	1	9
$4\frac{3}{4} \times 1\frac{3}{4}$	0	9	1	0	0	9	1	0	1	3	1	9	1	0	1	9
5×3	1	0	1	3	1	0	1	3	1	0	1	3	1	0	1	9
$5 \times 4\frac{1}{2}$	1	6	1	9	1	3	1	6	1	9	2	6	1	3	2	6
6×5	1	6	2	3	1	6	2	3	2	6	3	6	2	0	2	9
$7 \times 3\frac{3}{4}$	1	6	2	3	1	0	1	9	1	9	2	6	2	0	2	9
8×4	1	6	2	3	1	6	2	3	2	6	3	9	2	0	3	0
10×5	2	3	3	6	2	3	3	3	3	0	4	6	2	6	3	3
12×5	2	6	3	9	2	6	4	0	4	0	6	0	3	3	4	3
14×6	3	6	5	3	3	6	5	3	5	0	7	6	4	0	5	6
16×6	4	6	6	6	4	6	6	3	6	6	9	6	4	0	5	6

and to have the holes for bolts punched in their returns; but holes in the girder or joist to which they are secured are not included. If these holes have to be drilled on the job, 6d. to 9d. per hole must be added to the price. Suppose it is desired to know the cost of square notched joggle and double cleated end of 8-in. by 4-in. rolled steel joist into same section. Then—notch, 1s. 6d.; joggle, 2s. 6d.; double cleated end with four loose bolts not more than $2\frac{1}{2}$ in. long, 3s.; drilling four holes on site and bolting up, 3s; total, 10s.

The requirements of the New York Building Law as

TABLE OF STRENGTHS OF FIRE-RESISTING FLOORS, giving safe load in cwts. per foot super., including weight of floor itself. Factor of safety 3, equal to 10·6 tons per sq. in. on metal ; joists 3 ft. centre to centre, with ends free.

Price per yard super.	Section of Joist.	Weight in lb. per foot.	Moment of Inertia.	Distributed load in tons one foot will carry f.s.3.	Span in Feet.																	
					8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
s. d.																						
6 6	4 × 1¾	5	3·6	12·9	1·3	1·0	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
7 0	4¾ × 1¾	6·5	6·5	19·4	2·0	1·6	1·3	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
8 6	5 × 3	11	13·65	38·8	4·0	3·2	2·6	2·1	1·8	1·5	—	—	—	—	—	—	—	—	—	—	—	—
8 9	6 × 3	13	20·75	49·2	—	4·0	3·3	2·7	2·3	1·9	1·7	1·5	—	—	—	—	—	—	—	—	—	—
10 3	7 × 3¾	16	36·0	73·1	—	—	—	4·0	3·4	2·9	2·5	2·2	2·0	1·7	1·5	—	—	—	—	—	—	—
11 3	8 × 4	19	56·5	100·4	—	—	—	—	—	4·0	3·4	3·0	2·6	2·3	2 0	1·9	1·7	1·5	—	—	—	—
11 9	9 × 3¾	20	75·0	118·54	—	—	—	—	—	—	4·0	3·5	3·1	2·7	2·4	2·2	2·0	1·8	1·6	1·4	—	—
14 6	10 × 5	29	141·7	201·48	—	—	—	—	—	—	—	—	—	—	4·1	3·7	3·3	3·0	2·7	2·5	2·3	2·1

regards joints are as follows: Where beams are framed into headers, the angles which are bolted to the tail beams shall have at least two bolts for all beams over 7 in. in depth, and three bolts for beams 12 in. deep and over, and these holes shall not be less than $\frac{3}{4}$ in. in diameter. Each one of these angles or knees, when bolted to the girders, shall have the same number of bolts as stated for the leg. The angle-iron in no case shall be less in thickness than the header or trimmer to which it is bolted, and the width of angle shall in no case be less than one-third the depth of the beam, except that no angle knee shall be less than $2\frac{1}{2}$ in. wide or need be more than 6 in. All wrought-iron or rolled steel beams 8 in. and under in depth shall have bearings equal to their depths if resting on a wall. Nine-inch to 12-in. beams shall have a bearing of 10 in., and all beams more than 12 in. in depth shall have bearings of not les thans 12 in. if resting on a wall. Where beams rest on iron supports, and are properly tied to them, no greater bearings shall be required than one-third the depth of the beams. The deflection of beams under a full load must not exceed $\frac{1}{30}$ in. per lineal foot span, and where arched construction is employed joists must be tied together at intervals of not more than eight times their depth.

CHAPTER VIII.

ASCERTAINING SAFE LOADS ON JOISTS AND GIRDERS.

THE relative strengths of beams loaded and supported in different ways are shown in Figs. 94 to 97, where W = concentrated load, w = load per unit of length, and l = span. From the figures and bending moments it will be seen that a beam will only support half the load con-

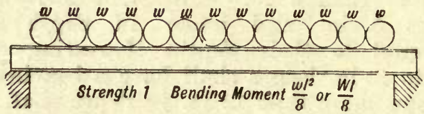

Fig. 94.—Beam with Load Distributed.

centrated at the centre (Fig. 95) when the bending moment is $\frac{Wl}{4}$, as it will if the load were equally distributed (Fig. 94) when the bending moment is $\frac{w\,l^2}{8}$ or $\frac{W\,l}{8}$, which shows that the load has only half the effect.

If the beam is a uniformly loaded cantilever (Fig. 96)

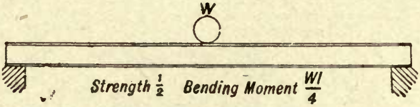

Fig. 95.—Beam with Load Concentrated.

it will only carry one-quarter of the load shown in Fig. 94, as the bending moment is $\frac{w\,l^2}{2}$ or $\frac{W\,l}{2}$, and if loaded with concentrated load at the unsupported end (Fig. 97) it will only carry one-eighth of that load (Fig. 94), as the bending moment is W l. Fixing one or both ends of a beam increases its stiffness; it will not deflect so much under the load as it would if the ends were only supported, and also its strength in most, but not all cases. For instance, considering a uniformly loaded beam as in Fig. 94, fixing both ends increases the strength 50 per cent., but if one end only is fixed there is no increase of strength. With a central load, as in Fig. 95, fixing both ends increases the strength 100 per cent., and fixing one end only increases the strength by 33 per cent.

The table of strengths for joists is obtained by equating the bending moment $\frac{w\ l^2}{8}$ or $\frac{W\ l}{8}$ with the moment of resistance of the joists $\frac{r\ I}{\frac{d}{2}}$; for instance, taking the

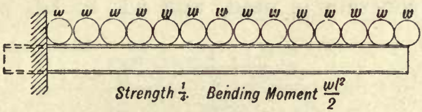

Fig. 96.—Cantilever with Load Distributed.

case of a 9-in. by 3¾-in. rolled steel joist at 14 ft. span, to find the safe load per ft. super. with joists 3 ft. centres. The total load in tons multiplied by the span in inches and divided by 8, is equal to the moment of inertia (explained later) of the section given in the table (75·02) page 64.

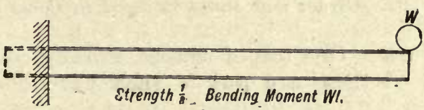

Fig. 97.—Cantilever with Load at End.

multiplied by the safe stress in tons per sq. in. of metal, divided by half the depth of joist in inches, or

$$\frac{W\ l}{8} = \frac{r\ I}{\frac{d}{2}} \quad \frac{W \times 14 \times 12}{8} = \frac{10{\cdot}6 \times 75}{4{\cdot}5} = 168 \text{ cwt.},$$

and as the joist supports 42 ft. super. of floor, the safe load is $\frac{168}{42} = 4$ cwt., including the weight of the floor itself, which would be about ¾ cwt. per ft. super. When the amount of safe distributed load 1 ft. will carry is known (column 5 in the table), by dividing this by the span the same result is obtained: 118·54 ÷ 14 = 168 cwt. = 4 cwt. per ft. super. The prices given are for floors at the ground level; 4d. to 6d. per yard must be added for hoisting or lowering to each floor level above or below, so that a 5-in. by 3-in. floor at second-floor level would cost 9s. 6d. per yd. super. Furnace breeze costs 1s. to 2s. per yard delivered in London, but coke breeze costs much more. For hoisting girders and joists not included in floor price, add 4s. per ton per floor. All labours, and any joists not included in the floor price, are taken extra. The floors are taken to be 7in. thick. For any extra thickness, add 4d. to 6d. per in. per yd. super. If ballast concrete is used, add 2d. to 3d. per in. per

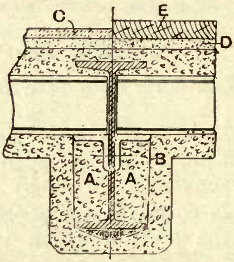

Fig. 98.—Girder and Joists Encased in Concrete.

yd. over breeze. One ton of cement usually consists of 11 bags. Two of these are put to each yard of breeze for lintel floors.

When pricing the encasing of girders or joists as in Fig. 98, take the concrete in each hollow between the flanges A at ½d. per inch in depth per foot run, and then the girth of the 2-in. outer casing, if supported by hoop irons, at ¾d. per inch girth per foot run. Wiring under side of joists to form a key for plastering, take at ½d. per inch per foot run. In this illustration B indicates hoop-iron, C ¾-in. mosaic finish, D 1 in. cement and sand floating, E 1½-in. wood-block finish.

It will be advisable here to describe for beginners how a parabola is drawn, as it will be necessary to use the parabola in order to explain future examples. If the bending stresses arising from an equally distributed load be worked out and plotted, it will be found that they decrease from the centre of the span towards the supports on the curve of a parabola.

Suppose A B (Fig. 99) is the clear span of a girder or joist carrying a uniformly distributed load of six tons, then the maximum bending moment will be at the centre of the span, and will equal $\frac{wl^2}{8}$ or $\frac{wl}{8}$, according to whether w is taken as the total load or the weight per foot run. Taking w as the total load, the bending moment is $\frac{wl}{8} = \frac{6 \times 20}{8} = 15$ foot-tons or 180 inch-tons. Now, from the point C, the centre of A B, draw C D at right angles to represent 180 inch-tons to any convenient scale. Extend

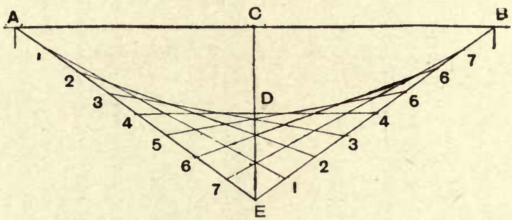

Fig. 99.—Method of Drawing a Parabola.

C D to E, making D E = D C, join E A and E B, and divide each into an equal number of parts by repeated bisections. Then join 1 and 1, 2 and 2, and so on. The lines will be tangents to the curve required, which can be easily drawn. The bending moment can then be obtained at any point in the beam by drawing a vertical to the curve and seeing what it scales.

It should be mentioned here that if C D is only about one-eighth of A B, the curve approaches an arc of a circle, passing through A, D, and B, and may be drawn as such; but the vertical scale with such a proportion is generally too small to be of any use.

It often happens that a girder has a concentrated load, and in order to compare with the manufacturer's tables of strength, which usually only consider equally distributed loads, it is necessary to find what uniformly distributed load would give the same maximum moment of stress. Let A B (Fig. 100) represent a beam with a concentrated load W at a distance y from support A. Then the maximum bending moment will be directly under the load W, and will equal the reaction $\frac{W(l - y)}{l}$ at A multiplied by the leverage $y = \frac{W(l - y)}{l} \times y$, and if we equate this with $\frac{w\,l}{8}$, the maximum bending moment from a uniformly distributed load, we get

$$\frac{w\,l}{8} = W\left(\frac{l - y}{l}\right) y$$

$$\text{and } w = 8\,W \times \left\{\frac{y}{l} - \left(\frac{y}{l}\right)^2\right\}$$

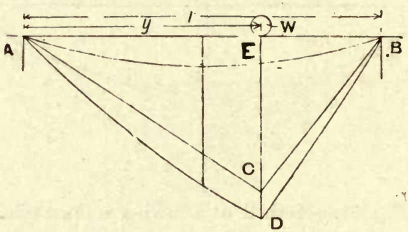

Fig. 100.—Diagram showing Bending Moments from Different Loads.

Taking in this example $l = 15$ ft., $y = 10$ ft., $W = 10$ tons, we get $w = 8 \times 10 \times \left\{\frac{10}{15} - \left(\frac{10}{15}\right)^2\right\} = 18$ tons, the equally distributed load, which would give the same maximum moment of stress; or 9 tons concentrated at the centre would have the same effect; the bending moment from a load concentrated at the centre being $\frac{w\,l}{4}$, or double the moment from a uniformly distributed load. For a beam carries double the load equally distributed that it will if concentrated at the centre.

If the beam, in addition to the concentrated load, had an equally distributed load of 3 tons, it would be more difficult to find the maximum bending moment. In such a case it is better to draw a diagram and set out carefully the moments obtained to a convenient scale (see Fig. 100).

The maximum moment from the concentrated load is of course directly under the load, and equals 33·3 foot-tons; and as this moment decreases gradually, vanishing at the supports (as can be seen by working out an example), if E C be drawn to scale to represent this moment, and the point C be joined to A and B, the bending moment at any point in the beam from the 10-ton load can be obtained by scaling the vertical.

The maximum bending moment at the centre from the

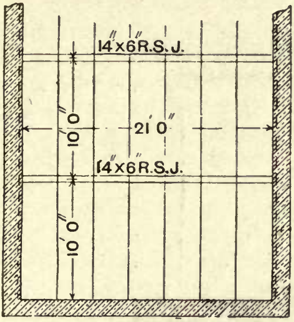

Fig. 101.—Floor Plan with Joists and Girders.

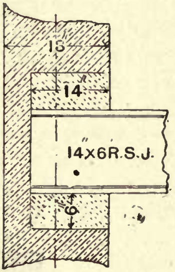

Fig. 102.—Fixing of Girder in Wall, Section.

equally distributed load of 3 tons = 5·6 foot-tons, and the curve of stress will be a parabola, as explained before; and if the stresses from these two loads be combined, we get the irregular figure A D B, from which we obtain the total stress in the beam at any point arising from both loads. It is at once evident from the figure that the maximum stress is under the concentrated load, and by scaling E D we obtain its value = 38·3 foot-tons. Knowing this, and equating it with the maximum moment for an equally distributed load, we get $\frac{w\,l}{8} = 38{\cdot}3$ and $w = \frac{38{\cdot}3 \times 8}{15} = 20{\cdot}4$ tons, the total uniformly distributed load, which would give the same maximum moment of stress.

Although, by taking special precaution, it may be possible to fix the ends of a joist carrying a small load, it is a mistake to imagine that a uniformly loaded girder has only to have its ends firmly pinned in the wall to be considered fixed, and consequently able to carry 50 per cent. more load. Fig. 101 is a plan of a floor supported at 10-ft. centres by 14-in. by 6-in. rolled steel joists. Fig. 102 shows one end of this girder with a 14-in. bearing in an 18-in. wall, and with a stone 6 in. thick at top and bottom to give a better hold of the wall. The clear span is 21 ft., and it is assumed that the floor joists are placed against the walls, so that the load on the side bays also

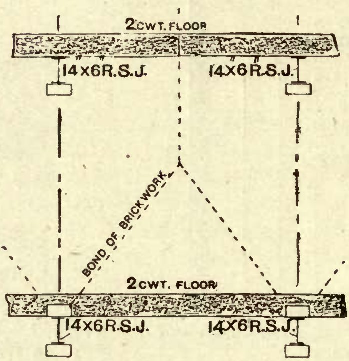

Fig. 103.—Elevation through Floors showing Portion of Wall Influenced by Fixing the End of Girder.

comes on the girder, and not directly on the wall. Let it be also supposed that there is a similar floor above the one under consideration, and also a $1\frac{1}{2}$ cwt. flat roof over (see Fig. 103). Imagining the end of girder fixed, the bending moment at the point of support will be $\frac{w\ l^2}{12}$, and if it is a 2-cwt. floor, the total load on the girder is 21 ft. by 10 ft. by 2 cwt. = 420 cwt. = 21 tons, or 1 ton per foot of span, and $\frac{w\,l^2}{12} = 36{\cdot}75$ foot-tons. To counteract this bending moment, there is the weight of the wall itself, the floor above, and the roof acting through the centre of gravity of the wall, that is, the centre in this case. Weight of wall 18 in. thick, 22 ft. high, and 10 ft. wide, $22 \times 10 \times 1\frac{1}{3}$ cwt. = 293 cwt.; parapet, $4 \times 10 \times \frac{2}{3}$ cwt. = 27 cwt.; floor, $10{\cdot}5 \times 10 \times 2$ cwt. = 210 cwt.; roof, $10{\cdot}5 \times 10 \times 1\frac{1}{2}$ cwt. = 158 cwt. = 688 cwt., or nearly $34\frac{1}{2}$ tons, acting with a leverage of 9 in. or ·75 of a foot; $34{\cdot}5 \times {\cdot}75 = 25{\cdot}87$ foot-tons to resist 36·75 foot-tons, the bending moment on the girder. Consequently the girder cannot be considered fixed, as it would move the wall first, apart from the consideration of the stonework or brickwork crushing.

It will be noticed in the above calculation that the wall is taken of the full width to the level of the bearing of the girder in question; in reality the most that can be expected is a V-shaped mass rising from the top of the girder and spreading out, following the bond in the work until it reaches the wall influenced by the adjoining girder (see Fig. 103). But no allowance has been made for the adhesion of the mortar, which might be anything from 20 lb. to 100 lb. per square inch, according to the quality of the mortar used, so that the extra quantity of wall taken may compensate for this. It may be easily ascertained whether it would be possible to fix the end by means of holding-down bolts to something immovable, such as a column cap. Take $\frac{3}{4}$-in. bolts, which may be considered to safely resist $2\frac{1}{2}$ tons each in tension; the average leverage these could obtain would be 6 in. Therefore, the number of bolts x required would be $x \times 2{\cdot}5 \times {\cdot}5 = 36{\cdot}75, x = 30$, a number it would be quite impossible to insert.

From the figures above given it will be apparent how necessary it is to discover under what conditions the safe loads in manufacturers' tables apply. Frequently in these tables the ends are considered to be fixed; but as this condition seldom obtains in practice, due allowance should be made in each case—for instance, if the load given is a uniformly distributed one, by deducting one-third to make it apply to free ends. A factor of safety of three is most common in makers' lists; but, for good work, in no case should a factor of less than four be employed. To avoid trouble from deflection, the most satisfactory method in building work is to specify that $\frac{1}{30}$ of an inch per foot of span is the maximum deflection under full working load that will be permitted.

Frequently, in the case of joists of foreign manufacture, there is no table of strength at hand; but when the depth, breadth, and weight per foot run are known, the approximate safe load (factor of safety three) can be obtained from the following formula:

$$W = \cdot 95\ (w - \cdot 3\ bd)\ \frac{d}{L}$$

where W = safe uniformly distributed load in tons.
w = weight of beam in lb. per foot.
b = breadth in inches.
d = depth in inches.
L = span in feet.

The above is for 24-ton to 26-ton steel, the usual strength of foreign manufacture. If it is required to be used for English 30-ton to 32-ton steel, it must be altered to the following:

$$W = 1 \cdot 2\ (w - \cdot 3\ bd)\ \frac{d}{L}.$$

Another useful approximate formula for English steel joists or plate girders is:

$$S\ L = \frac{7\ a\ d}{S}, \text{ and for foreign, } S\ L = \frac{5 \cdot 5\ a\ d}{S}.$$

S L = safe uniformly distributed load in tons.
a = area of one flange in inches.
d = depth in inches.
S = span in feet.

The above are only approximate, and must only be used to obtain some idea of the section required when there are no other data available.

The accurate mathematical formula for obtaining the moment of resistance of any beam is $M = \dfrac{r\ I}{y}$.

M = moment of resistance in inch tons.
r = limiting stress per sq. inch of metal.
y = half the depth.
I = moment of inertia, explained later.

By equating this formula with the bending moment, whatever it may be, the safe load on the beam can be obtained. Each side of the equation must, of course, be in the same terms—that is, inch-tons or foot-tons.

The moment of inertia of a rectangular beam is $\frac{1}{12}\ b\ d^3$

so that its value varies as the breadth and as the cube of the depth.

Fig. 104 shows how the vertical moment of inertia of a joist is obtained. The joist is first considered as a solid rectangular beam, and then the voids between the flanges are deducted.

$$I = \tfrac{1}{12}\,(b\,d^3 - b'\,d'^3).$$

Imagine Fig. 104 to be a 14-in. by 6-in. rolled steel joist with a web thickness of $\frac{7}{16}$ of an inch, and an average flange thickness of $\frac{5}{8}$ in., then $I = \frac{1}{12}\,(6 \times 14^3 - 5{\cdot}56 \times 12{\cdot}75^3) = 412$.

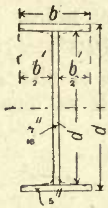

Fig. 104.

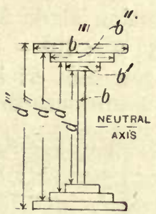

Fig. 105.

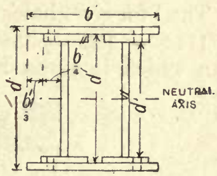

Fig. 106.

Fig. 104.—Moment of Inertia of a Single Joist. Fig. 105.—Moment of Inertia of a Girder or Stanchion. Fig. 106.—Moment of Inertia of a Compound Girder or Stanchion.

In Fig. 105 the flanges are divided into a number of rectangles, and I can be obtained as above described, or the same result may be obtained more simply as follows:

$$I = \tfrac{1}{12}\,\{b\,d^3 + b'\,(d'^3 - d^3) + b''\,(d''^3 - d'^3) + b'''\,(d'''^3 - d''^3)\}.$$

Working out the previous example by this method also we get $I = \frac{1}{12}\,\{{\cdot}44 \times 12{\cdot}75^3 + 6\,(14^3 - 12{\cdot}75^3)\} = 412$ as before.

The moment of inertia for this section is given in the manufacturers' tables as 422; the slight difference arises from the curved part of flange against web not having been included.

In the case of a compound girder (Fig. 106), proceeding by the first method mentioned, we get

$$I = \tfrac{1}{12}\,\{b\,d^3 - b'\,d'^3 - b''\,d''^3\}.$$

In this case the rivet holes should also be deducted for the bottom flange, which is in tension; there is no reason why they should be for the top flange, which is in compression, providing the rivets completely fill the holes.

If b''' is the combined width of the rivet holes, the deduction for both flanges would be

$$I = \tfrac{1}{12}\{b'''\,d^3 - b'''\,d''^3\}.$$

Take the case of a compound girder (Fig. 106) composed of two 14-in. by 16-in. rolled steel joists having average flange thickness of $\frac{27}{32}$ in. and web $\frac{1}{2}$ in., with 14-in. by $\frac{1}{2}$-in. plates top and bottom, with two $\frac{3}{4}$-in. rivets in each flange, then

$$I = \tfrac{1}{12}\{14 \times 15^3 - 2 \times 14^3 - 11 \times 12{\cdot}3^3\}.$$

$I = 1774$ with no deduction made for rivet holes.

The I for rivet holes $= \frac{1}{12}\{1{\cdot}5 \times 15^3 - 1{\cdot}5 \times 12{\cdot}3^3\} = 190.$

$1774 - 190 = 1584$, the moment of inertia of the section which is given in the manufacturers' table as 1599.

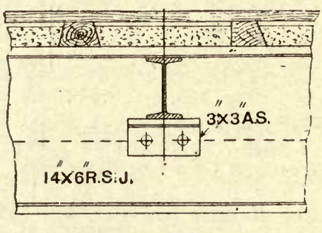

Fig. 107.—Support for Floor Joist on Girder and Boarded Floor Finish: Section.

Now proceed to work out the girders and joists as shown in Fig. 101. It will be noted that the concrete rests in a chase in the wall parallel to the floor joists, so that only half the load of the side bays comes on the girder. The area of the floor supported by the girder is consequently $10 \times 18 = 180$ ft. super. at 2 cwt. = 18 tons.

Looking through the maker's list, 14 in. by 6 in. at 46 lb. appears to be a likely section. Its depth will be $\frac{1}{18}$ of the span and its width $\frac{1}{43}$; this latter ratio is rather high, although seldom considered in practice, but as the joists will run into it as shown in Fig. 107 and in Fig. 98, it would be stiffened sideways.

By the approximate rules given before for obtaining the safe load uniformly distributed when only the weight per foot, the breadth, and the depth are known :—

$$W = 1{\cdot}2\,(w - {\cdot}3\,b\,d)\,\frac{d}{L} = 1{\cdot}2\,(46 - {\cdot}3 \times 6 \times 14)\,\frac{14}{21}$$
$$= 17{\cdot}56 \text{ tons,}$$

and $W = \dfrac{7\,a\,d}{s} = \dfrac{3{\cdot}75 \times 7 \times 14}{21} = 17{\cdot}5$ tons.

By the accurate method of equating the bending moment $\dfrac{w\,l}{8}$ with the moment of resistance $\dfrac{r\,I}{y}$

$$\frac{w\,l}{8} = \frac{2\,r\,I}{d} \qquad \frac{w \times 252}{8} = \frac{2 \times 10{\cdot}6 \times 422}{14}$$

from which $w = 20{\cdot}3$ tons safe load, and this agrees with the manufacturers' tables.

Having ascertained that the girder is strong enough, now see whether it is stiff enough; its deflection should not materially exceed $\frac{1}{30}$ in. per foot of span.

The formula for the deflection in inches of a girder having a uniformly distributed load is $D = \dfrac{5\,w\,l^3}{384\,E\,I}$,

and with a load w at the centre $D = \dfrac{w\,l^3}{48\,E\,I}$,

where w = load in tons;
l = length of span in inches;
E = modulus of elasticity—12,000 tons;
I = moment of inertia.

Working out the first formula we find the deflection is—

$D = \dfrac{5 \times 18 \times 252^3}{384 \times 12000 \times 422} = {\cdot}74$ of an inch, which is only a trifle more than $\frac{1}{30}$ in. per foot of span.

The floor joists must next be considered; each one has to carry 10 ft. by 3 ft. of floor at 2 cwt. = 3 tons.

By the table given before, it is found that the most suitable joist for the present case is a 5-in. by 3-in., at 11 lb., which gives 2·6 cwt. safe load per ft. super.

But proceeding to work out the case first by the approximate formula, which will be found to give better results for the smaller joists, we get—

$$w = 1{\cdot}2\,(11 - {\cdot}3 \times 3 \times 5)\,\tfrac{5}{10} = 3{\cdot}9 \text{ tons}$$

$$\text{and } w = \frac{7 \times 1{\cdot}12 \times 5}{10} = 3{\cdot}92 \text{ tons,}$$

and using the accurate formula—I being 13·6—

$$\frac{w\,l}{8} = \frac{2\,r\,\text{I}}{a} \qquad \frac{w \times 120}{8} = \frac{2 \times 10{\cdot}6 \times 13{\cdot}6}{5}$$

w = 3·9 tons, which is the same as by the approximate formula.

The joists would rest on 3-in. by 3-in. by $\frac{3}{8}$-in. angle, riveted to the web of the 14-in. by 6-in. The shearing stress is nearly 2 tons, so that one $\frac{3}{4}$-in. rivet would be sufficient; but two must be used, otherwise the cleat may swing round. If a terra-cotta lintel floor is used with $4\frac{3}{4}$-in. by $1\frac{3}{4}$-in. rolled steel joists, 2-ft. centres only, angle stringers, 3 in. by 3 in. by $\frac{3}{8}$ in., would be riveted to the web with $\frac{3}{4}$-in. rivets 9-in. pitch, as the exact position of the joists is uncertain, and nothing would be saved by putting on a bracket for each joist.

The compound girder shown in Fig. 82, composed of channel steels and plates, is a better section than that shown in Fig. 81, which is composed of rolled steel joists and plates, because the channel steels get a firm hold of the plates (which the joists do not), and consequently are able to transmit the stress better. From the way the girder (Fig. 81) has to be put together, it is impossible to get four rivets in each flange, although an additional rivet at top and bottom could be put where shown dotted in, but even then the connection would not be satisfactory, and owing to this defect it is advisable to make some deduction from the theoretical strength of the girder.

The projection of the plate or plates beyond the edges of the rolled steel joists should not exceed twice their thickness. Any number of plates can, of course, be added to the flanges, providing there is sufficient metal in the web to resist the shearing stress. In the case of a beam supported at both ends, the vertical shearing stress at any section is equal to the reaction at the support, less the weight between the support and the section in question. The shearing stress on a cantilever at any section is equal to the weight on the beam between the section in question and the outer end.

CHAPTER IX.

PRACTICE OF IRON AND STEEL CONSTRUCTION.

In cases where the wall is not thick enough for stone stairs to be pinned in and cantilever from it, or when their width is too great to rely on the cantilever fixing, a cranked joist is sometimes fixed to give a bearing to the unsupported ends. Of course, such a joist must be fixed before the steps are, otherwise the latter will probably not obtain a proper bearing on the joist. For a very light staircase or passage-way across a light well,

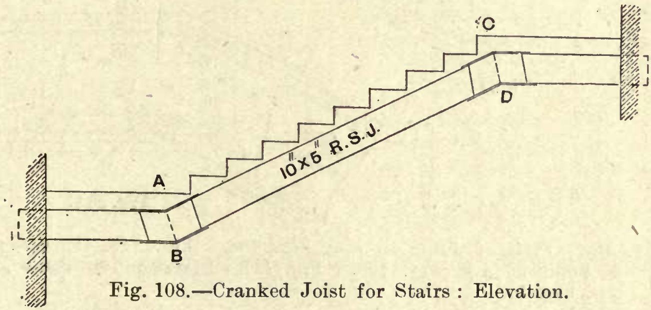

Fig. 108.—Cranked Joist for Stairs : Elevation.

cast-iron stringers may be employed, and would be less expensive, especially when a number of similar stringers are required, as they could all be cast from one pattern. But it would not be wise to employ cast-iron where it is likely to be subjected to concussions, such as would be caused by moving heavy packing-cases. The joints necessitated by cranking the joist to the rake of the stairs, as shown in elevation Fig. 108, and detail Figs. 109 to 111, would be expensive; a wood template would first be made for the smith to work to.

In proceeding to work out the example shown by Fig. 108, it is treated as a girder of 16 ft. span; owing to the bearings being horizontal there would be no tendency to slide. It will be supposed that the girders are required to carry a passage-way 6 ft. wide across a light well, and the load will be taken as 4 cwt. per foot super. horizontal. This load will be ample, and will include the weight of the stairs themselves and of any covering provided. The total load on the two girders will be 16 × 6 × 4 = 384 cwt. = 19·2 tons, that is, 9·6 tons equally distributed on each girder. An English girder 10 in. by 5 in. by 29 lb. will give this, with a factor of safety of 4, and its depth will be $\frac{1}{19}$ of the span. The bending moment

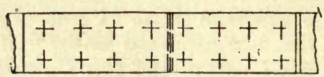

Fig. 109.

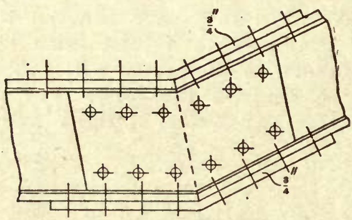

Fig. 110.

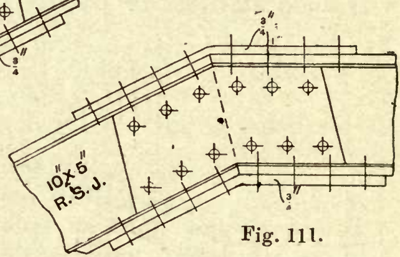

Fig. 111.

Fig. 109.—Plan of Joint at A B, Fig. 108. Fig. 110.—Elevation of Joint at A B, Fig. 108. Fig. 111.—Elevation of Joint at C D, Fig. 108.

at the joint 4 ft. from the support will be $= \frac{wx}{2}(x - l) =$ $\frac{\cdot 6 \times 4}{2}$ $(4 - 16) = - 14\cdot 4$ ft. tons, and as the depth of the girder is only 10 in., the force each flange will have to exert is $14\cdot 4 \times \frac{6}{5} = 17\cdot 3$ tons; and taking the safe stress in the metal at 6·5 tons per sq. in., the area required is $\frac{17\cdot 3}{6\cdot 5} = 2\cdot 66$ sq. in. Deducting two $\frac{3}{4}$-in. rivet holes, the net width of the plate will be $5 - 1\frac{1}{2} = 3\frac{1}{2}$ in., and the required thickness $\frac{3\cdot 5}{2\cdot 66} = \cdot 76$ in. thick, say $\frac{3}{4}$ in. The number of $\frac{3}{4}$-in. rivets required to transmit this stress to the flanges of the 10 in. by 5 in. next requires to be investigated. The rivets will be in single shear, and the safe shearing stress will be taken as 5 tons per square inch. The area of a $\frac{3}{4}$-in. rivet is ·44 sq. in., so that the number of rivets required in each flange on each side of the joint will be $\frac{17 \cdot 3}{\cdot 44 \times 5} = 8$; although plates on the web are not absolutely necessary, they would not be omitted in good work. Suppose there is a $\frac{3}{8}$-in. plate on each side of the web: then the extent to which they will assist the joint, considering them together as a rectangular beam, will be

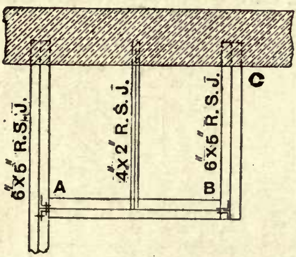

Fig. 112.—Landing for Stairs: Sectional Plan.

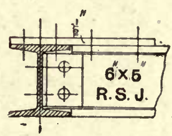

Fig. 113.

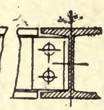

Fig. 114.

Fig. 113.—Detail of Joint A, Fig. 112. Fig. 114.—Detail of Joint at B, Fig. 112.

the moment of resistance of a beam 8 in. deep and $\frac{3}{4}$ in. broad; that is $\frac{rbd^2}{6} = \frac{6 \cdot 5 \times \cdot 75 \times 8^2}{6} = 52$ in. tons, or 4·3 ft. tons, r being safe stress per sq. in. of metal, b and d the breadth and depth respectively of the beam. In order to transmit this stress (the number of $\frac{3}{4}$-in. rivets required 5 in. apart), the distance between the rows would be $\frac{52}{5 \times \cdot 44 \times 5 \times 2} = 3$. From these particulars the joints A B and C D are detailed. It will be noted that the top joint C D is the least satisfactory, because when the girder deflects it has a tendency to draw the plates away from the flanges. In the lower joint the opposite is the case—the plates are pressed against the flanges. The flange need not be cut at B and C; it may have a **V**-shaped piece cut out, so that A and D can abut to receive the joint plate.

F

Fig. 112 represents a sectional plan through the webs of the joists forming a landing, as is frequently required in connection with staircases. If the joist B C could be firmly fixed in the wall, or be fixed by being cleated to a joist in the adjoining room, or—even better still—if one of the floor joists in the adjoining room could be continued through the wall and act as a cantilever, the unsatisfactory joint at A would not be required. If neither of these arrangements is possible, the only alternative is the joint at A, double cleated with a ½-in. plate on the top. This joint requires to be carefully made, otherwise the cantilever will not be level. For very light constructions, and where two bolts can be got through the

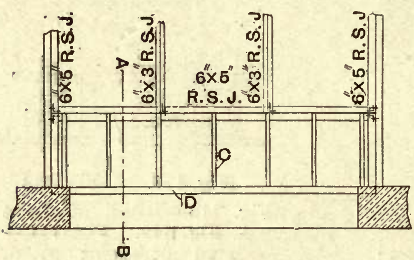

Fig. 115.—Framing for Bulkhead to Admit Light to Basement: Plan.

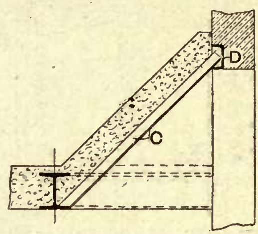

Fig. 116.—Section of Line A B, Fig. 115.

webs, the cleats might be relied on and the top plate omitted. Figs. 113 and 114 show details of the joints at A and B (Fig. 112) respectively.

Fig. 115 is a plan of a bulkhead formed to admit light to a basement, and Fig. 116 a section through it. The floor joists are trimmed to the necessary width, and 1½-in. × 1½-in. tee-steels C fixed 12 in. or 18 in. apart, and cranked at their ends to obtain support, as shown in the section, resting at the lower end on the trimmer and at the upper end in a steel channel D, to assist the concrete forming the slope. The spandrils are formed in concrete, but as they rise upright directly off the floor, they require no strengthening. The trimmer is notched, joggled, and cleated each end into the trim-joists, and the two floor joists running into trimmer are notched, joggled, and single cleated into it.

Fig. 117 shows a plan of a trimming for a skylight 10 ft. by 8 ft. Fig. 118 is a section through the trimmer, and shows how the concrete curb is formed to receive the wood sill of the skylight. The 8-in. by 4-in. trimmers are

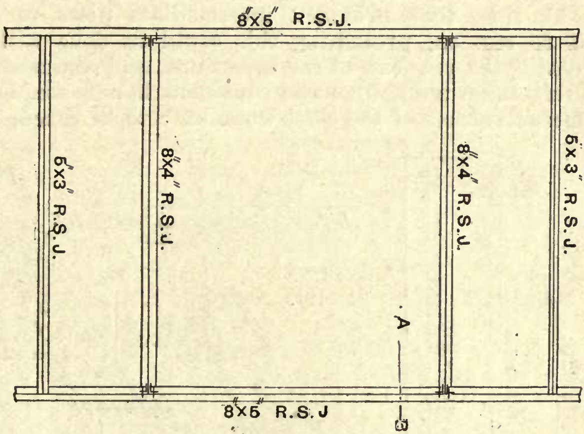

Fig. 117.—Plan of Trimming for Skylight.

notched, joggled, and cleated each end into the 8-in. by 5-in. joists, and the smaller intermediate 5-in. by 3-in. joists are cut to dead lengths, ½ in. short of the exact dis-

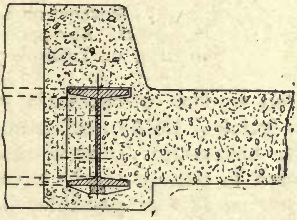

Fig. 118.—Section of Line A B, Fig. 117.

tance between the webs of the 8-in. by 5 in., and their bottom flange joggled; as it is impossible for the 8-in. by 5-in. joists to move apart, their ends need not be cleated. After the rough concrete surface has been floated with

1 in. cement and sand, the asphalt covering can be continued up the curb and under the wood sill, and a small channel formed on the inner side to take condensed water.

Architects would be well advised if, in designing a building, instead of carrying a large number of flues up side by side, they provided intermediate piers for the joists to rest on, presuming this could be done without increasing the quantity of the brickwork, and consequently avoiding large trimmings and considerable expense. Careful measurements of the steel work have to be taken, and

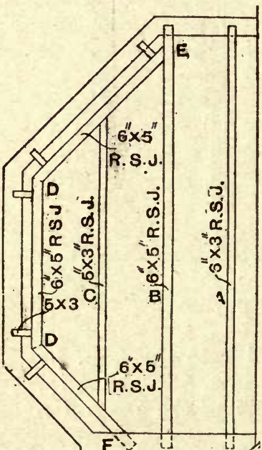

Fig. 119.—Floor in Turret: Plan.

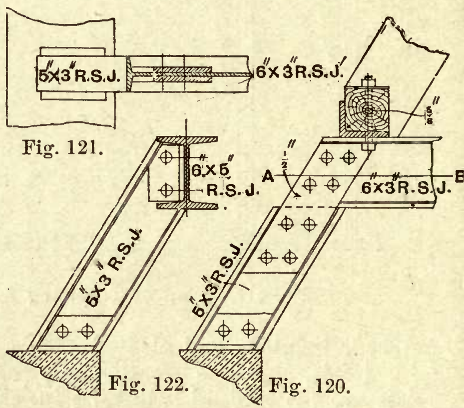

Fig. 121. Fig. 122. Fig. 120.

Fig. 120.—Detail of Cranked Joist, A, Fig. 119. Fig. 121.—Section on Line A B, Fig. 120. Fig. 122.—Detail Elevation of one of the Feet Supports for Trimming.

the work prepared in the contractor's yard and sent to the job ready for fixing. Delay and annoyance frequently occur because the necessary dimensions cannot be taken until the brickwork is nearly ready to receive the joists. All labour should, as far as possible, be put on in the yard, as the cost is enormously increased if the work has to be done on the site. The Building Act of 1894 states that 4 in. of brickwork must surround every smoke flue, so that no steel can come within 4 in. of any flue. The Act also requires a space of $\frac{1}{4}$ in. for every 10 ft. or part of 10 ft. in length of any bressummer to be left clear at each end, to allow for expansion. Another paragraph (see § 62 [2]) states: "Any storey constructed in the roof of any domestic building, the upper surface of the floor of which storey is at a height of above 60 ft. from the street level, shall be constructed of fire-resisting materials throughout."

Fig. 119 represents a half-plan of a floor in an octagonal turret; the walls, except at the back, being 18 in. below the top of the floor joists. Fig. 120 shows in elevation, and Fig. 121 shows in plan, the cranked joist A. The joint is made by riveting a ½-in. plate on each side of the web. Small angles are riveted on to give a good bearing surface on the walls. The other end is double-cleated, and firmly pinned in the wall. A 3-in. by 3-in. by ⅜-in. angle steel follows the rake of the sides in order to receive the wood plate from which the ribs forming the turret spring. A ⅝-in. bolt passes at intervals through the top flange of the joist, angle, and wood plate, and firmly secures it. Fig. 122 shows an elevation of one of the feet which support the 6-in. by 5-in. trimming; it is double-cleated to the 6-in. by 5-in., and has angles riveted on to give a good bearing on the wall. The 6-in. by 5-in. is jointed at D by having ½-in. plates riveted on each side of the web. At E it is single cleated into joist B. At F it is double cleated, and firmly pinned in the wall. Joist C is cut to dead length, and joggled on the bottom flange. The level floor, and also the slope, are filled in with concrete.

Fig. 123 shows an elevation of a knee girder, partly carrying a floor and partly the roof. The clear span is 16 ft., and the girders are spaced at convenient distances apart, so as to come in partitions, and thus hide the ugly gusset plate to the top joint, which would otherwise project below the ceiling. The girder is bolted to a 6-in. by 5-in. rolled steel joist in the floor, the other end of which is double-cleated and firmly pinned in the wall in order the better to counteract any tendency to spread. The bottom joint is shown in detail elevation and sectional plan in Figs. 126 and 127. The 8-in. by 4-in. rolled steel joist shown in the detail of the top joint (Fig. 128) runs between, and is cleated to the girders, and assists in keeping them vertical.

That the top joint (Fig. 128) is an important one is self-evident, and it would not be advisable to omit a gusset plate on the under side in such a position. The ½-in. gusset plate G is secured by a 2½-in. by 2½-in. by ⅜-in.

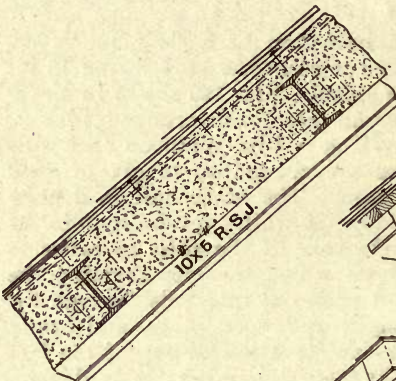

Fig. 124.

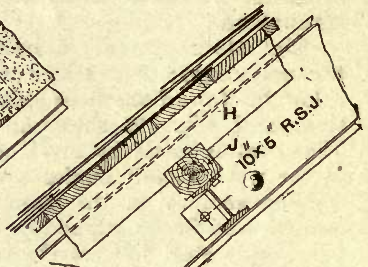

Fig. 125.

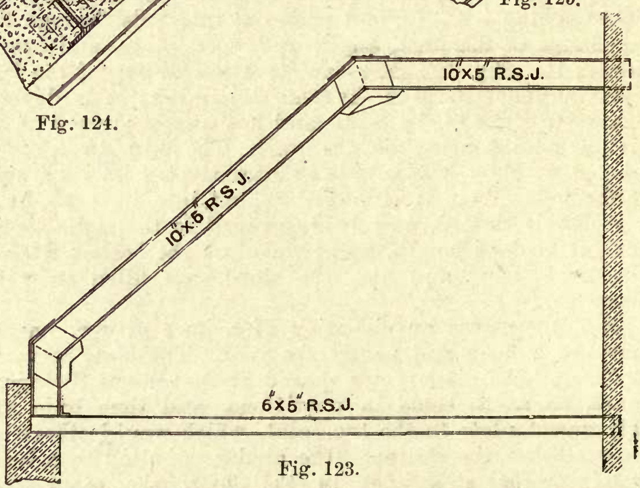

Fig. 123.

Fig. 123.—Knee Girder Carrying Floor and Roof: Elevation. Fig. 124.—Concrete and Slate Covering for Roof, Fig. 123. Fig. 125.—Wood Boarded and Slate Covering for Roof, Fig. 123.

angle on each side, as shown in section AB (Fig. 129). If the two top flanges had perfect contact theoretically no top plate would be required. In practice a cover plate is inserted, and also plates on the web.

In passing, what would be necessary for a fish-plate joint—that is, a plate on each side of the web, in Fig. 128—may be considered. The clear span, as already mentioned, is 16 ft., and the distance from the nearest support to the joint is 7 ft. Let the girders be 10 ft. apart, centre to centre, and let the load be 1 cwt. per ft. super. on plan.

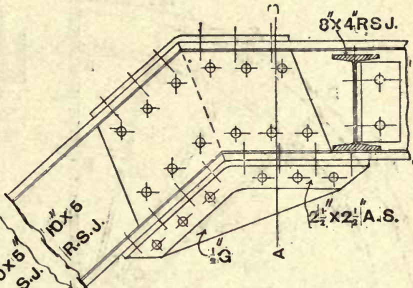

Fig. 128.

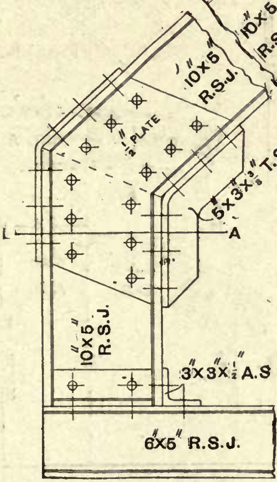

Fig. 126.

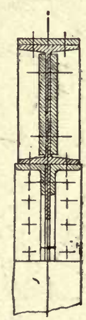

Fig. 129.

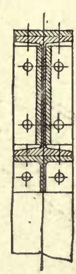

Fig. 127.

Fig 126.—Bottom Joint of Knee Girder, Fig. 123: Detail Elevation. Fig. 127.—Section on Line A B, Fig. 126. Fig. 128.—Top Joint of Knee Girder: Detail Elevation. Fig. 129.—Section on Line A B, Fig. 128.

Then the bending moment M arising from this load of ·5 of a ton per ft. run will be: $M = \frac{\cdot 5 \times 7}{2}(7-16) = 15\cdot 75$ ft. tons, or 188 in. tons.

Now, the moment of resistance $\overline{M}$ of a rectangular beam of depth d, breadth b, and s being the safe stress per sq. in.,

$$\overline{M} = \frac{s\,b\,d^2}{6}.$$

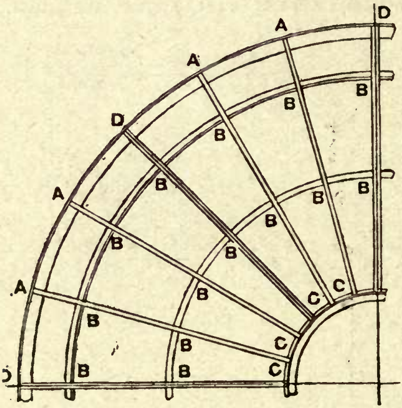

Fig. 130.

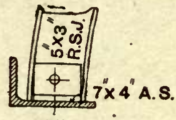

Fig. 132

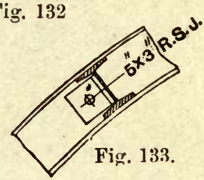

Fig. 133.

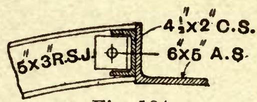

Fig. 134.

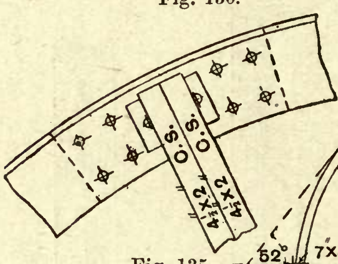

Fig. 135.

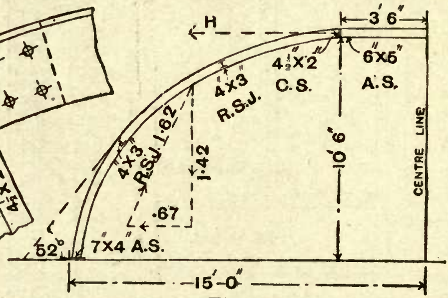

Fig. 131.

Fig. 130.—Elliptical Skeleton Dome: Plan. Fig. 131.—Elliptical Skeleton Dome: Section. Fig. 132.—Detail of Joint at A, Fig. 130. Fig. 133.—Detail of Joint at B, Fig. 130. Fig. 134.—Detail of Joint at C, Fig. 130. Fig. 135.—Detail of Joint at D, Fig. 130.

In the present case, in order to get the thickness of one plate, call b $2b$; s will be taken at 6·5 tons per sq. in., and d will be 8 in., the distance in the clear of the flanges of the 10-in. by 5-in. girder; so that, equating the bending moment and moment of resistance—

$$\frac{s\,b\,d^2}{6} = 188 \qquad \frac{6{\cdot}5 \times 2b \times 8^2}{6} = 188$$

$$\text{and } b = 1{\cdot}35 \text{ in.,}$$

so that the plate on each side of the web would have to be $1\frac{3}{8}$ in. thick.

What rivets are required to transmit this stress? The distance between the two rows of rivets will be $5\frac{1}{2}$ in., so that will be the leverage with which they will act; and $\frac{188}{5{\cdot}5} = 32{\cdot}4$ tons, the force to be exerted by the rivets in each row on each side of the joint by resistance to shearing.

The sectional area of one $\frac{3}{4}$-in. rivet is ·44 sq. in.; and as the rivets pass through the two fish-plates and the web of the 10-in. by 5-in., it is evident that the rivets must be sheared at two sections before the joint can fail, so that, doubling the rivet area, $\cdot 44 \times 2 = \cdot 88$ sq. in., and multiplying this by 5, the safe resistance to shearing per sq. in. of the metal, we get $\cdot 88 \times 5 = 4{\cdot}40$ tons as the resistance of each rivet; and dividing the total resistance required by the resistance of one rivet, we get the number of rivets required $\frac{32{\cdot}4}{4{\cdot}4} = 7\frac{1}{2}$; but as there cannot be half a rivet, in order to have sufficient in the joint, there must be 8 rivets at top and bottom on each side of the joint, and supposing they are placed at 3-in. pitch, each fish-plate would have to be 4 ft. long—2 ft. on each side of the joint.

The two sections Figs. 124 and 125 show alternative roof coverings. In Fig. 124 the rafters H obtain intermediate support on a wood plate J bolted to the top flange of a small rolled steel joist which is cleated to the web of the main girder. In Fig. 125 small rolled steel joists are placed 2 ft. centre to centre and cleated to the web of main girder, and then the slope is centred and filled in with breeze concrete, the surface of which is smoothed and the slates nailed to it direct.

Figs. 130 and 131 show a plan and section of the framing for an elliptical dome of 30 ft. external diameter and 11 ft. rise, with a 7-ft. diameter opening for light in the crown. A 7-in. by 4-in. angle is used for the bottom curb (see Fig. 132), and from it the ribs spring. The bottom curb is in tension, as it prevents the ribs from spreading under the load. There are two intermediate purlins (see Figs. 130 and 133). The first one is placed where the tangent to the curve makes an angle of 52° with the horizontal, as it is at this point that the horizontal thrust is at the maximum, consequently this is the point at which the dome would fail. The top curb (see Fig. 134) is in compression, and, owing to its having to provide a fixing for the light framing, an angle steel is used. Channel steels might equally well be employed for both top and bottom curbs. Joints are made in the curbs by means of cover plates, with sufficient rivets or bolts to transmit the stress.

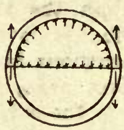

Fig. 136.—Method of Obtaining the Tension in a Hoop.

Every third rib, as shown in the illustration, is made of two 4½-in. by 2-in. channel steels, placed back to back and cleated at each end to the curbs (see Fig. 135). The intermediate ribs are of 5-in. by 3-in. rolled-steel joists, and each is cleated at ends to the curbs.

The ties or purlins are 4-in. by 3-in. rolled-steel joists, single cleated to the ribs. These, besides acting as a hoop round the dome, give support to the breeze concrete filling. As the top curb gives no support to this filling, a 4½-in. by 2-in. channel is fixed to do so. All the connections are shown riveted together; but, for convenience, and in order to facilitate erection, it may be advisable to use bolts in many of the joints. The centering for a concrete filling to a dome of this character requires much preparation and a considerable degree of ingenuity in putting together. It must also remain in position until the filling is thoroughly set—a matter of some weeks.

The dome shown in Figs. 130 and 131 may be considered theoretically without touching trigonometry. The load to be carried is very small, especially if the dome forms part of some internal ornamentation and is in reality only a shell. Take the load as 1 cwt. per ft. super. of horizontal surface. Although the pressure on the curbs is not continuous round the circumference, but concentrated at points, the curb may be considered in the same way as a pipe under pressure would be. Considering a half-section of a pipe (see Fig. 136) of which d is the diameter, r the radius, and p the pressure per sq. in. of whatever the pipe contains, the force tending to burst it is $d \times p$. To resist this force there is the metal on each side of the pipe, so that the tension T in the metal on one side only must be $T = \frac{d\,p}{2}$; or $r\,p$, where r is the radius, p in the case under consideration is obtained from the thrust of the twenty-four ribs, so that the pressure per ft. of circumference, if t is the thrust in each rib—

$$p = \frac{24\ t}{2\ r \times 3{\cdot}1416} = \frac{3{\cdot}82\ t}{r}$$

$$\text{and as } T = r\,p \quad \text{and} \quad p = \frac{3{\cdot}82\ t}{r}$$

$$T = r \times \frac{3{\cdot}82\ t}{r} = 3{\cdot}82\ t$$

The thrust on the top curb from each rib may be obtained by taking moments. The load supported by the ribs being triangular on plan, its centre of gravity will be at a point one-third in from the outer curb, or, in this case, 5 ft. The load coming on each rib is the area of the triangle multiplied by the weight per ft. super. :—

$$\frac{30^2 \times {\cdot}7854 \times 1}{24} = 29{\cdot}15 \text{ cwt.} = 1{\cdot}47 \text{ tons.}$$

By moments about the bottom curb, the horizontal thrust at the top H, multiplied by its leverage, 11 ft., must equal the load on the rib acting at its centre of gravity multiplied by its leverage, 5 ft. :—

$$H \times 11 = 1{\cdot}47 \times 5.$$
$$H = {\cdot}67 \text{ ton};$$

and by the equation $T = 3{\cdot}82\ t$. H and t being similar, $T = 3{\cdot}82 \times {\cdot}67 = 2{\cdot}56$ tons = the compression in the top curb.

The horizontal thrust on the top curb having been obtained, ·67 ton, we can obtain the thrust in the rib by drawing a triangle of forces, representing the horizontal thrust, ·67 ton, and the load on the rib, 1·47 tons.

So that the thrust in the rib, represented by the hypothenuse, will be $= \sqrt{\cdot 67^2 + 1\cdot 47^2} = 1\cdot 62$ tons. Owing to the method of fixing, the rib is not likely to bend sideways, but it is free to do so vertically. Its length is 18 ft., and a depth of 5 in. gives a ratio of 43; so that, using the formula given before, the safe load per sq. in. on the strut will be—(see p. 31).

$$\text{S. L.} = \frac{30 \times \frac{1}{4}}{1 + \frac{43}{900}^2} = 2\cdot 46 \text{ tons per sq. in.}$$

So that 1 sq. in. is more than sufficient; but it would not be advisable to use a less section than 5 in. by 3 in. at 11 lb., giving a sectional area of 3·25 sq. in.

The tension in the bottom curb will be the same as the compression in the top one; but, though this is so small, it is necessary to employ a comparatively large section of curb in order that the connections may be properly made.

In the case of a skeleton dome, with or without concrete filling, in an exposed position, diagonal braces should be inserted in each of the panels formed by the ribs and purlin rings. A more correct method would be to take the weight per ft. super of dome, then find the centre of gravity, and then draw the parallelogram of forces.

The following are the usual safe resistances in tons taken for steel, wrought-iron, and cast-iron (factor of safety 4):

	Tension.	*Compression.*	*Shearing.*	*Bearing.*
Steel	6·5	8	5	8
Wrought-iron ...	5	4	4	5
Cast-iron	1·5	8	2·4	10

It is sometimes necessary to use timber to shore up girders or to support tackle to raise them; and in order to give some idea of the sizes required, the following formulæ are supplied:

Safe uniformly distributed load in cwt. for a fir beam (factor of safety 5).

$$\text{S. D. L.} = \frac{b\,d^2}{s}$$

b = breadth in inches.
d = depth in inches.
s = span in feet.

For pitchpine, multiply the result by $1\frac{1}{2}$.

For a square fir strut with fixed ends, the breaking weight per sq. in. in tons

$$\text{B. W.} = \frac{2 \cdot 5}{1 + \frac{r^2}{250}} \qquad \text{ends rounded} = \frac{2 \cdot 5}{1 + \frac{r^2}{62}}$$

For pitchpine,

$$\text{B. W.} = \frac{3}{1 + \frac{r^2}{250}} \qquad \text{ends rounded} = \frac{3}{1 + \frac{r^2}{62}}$$

A factor of safety of 6 to 10 must be used. Taking the force required to crush a cube of any wood as unity, the force to break a timber strut with fixed ends is—

Length	12	times	least	thickness	$\frac{5}{6}$
„	24	„	„	„	$\frac{1}{2}$
„	36	„	„	„	$\frac{1}{8}$
„	48	„	„	„	$\frac{1}{6}$
„	60	„	„	„	$\frac{1}{12}$
„	72	„	„	„	$\frac{1}{24}$

The following are the ultimate crushing resistances for use in the above table : Oak, $3\frac{1}{2}$ tons per sq. in ; fir, $2\frac{1}{2}$ tons per sq. in. ; pitchpine, 3 tons per sq. in ; yellow pine, 2 tons per sq. in.

The following is a specification to accompany drawings and bill of quantities for first-class engineering work. For ordinary building work, the specification, if one were furnished at all, would not be nearly so strict :—

" On receipt of the drawings, the contractor is to examine them carefully before ordering the iron and steel, and is to call the attention of the engineer to any discrepancies between the drawings as scaled and the figured dimensions, or between various parts of the drawings, and the engineer's decision on these points shall be final. The whole of the iron and steel used must be of British manufacture. The steel to be made by the open-hearth process. The wrought-iron plates shall be free from blisters, scales, laminations, and all other defects, and shall be of such quality as to have an ultimate tensile strength of not less than 22 tons per square inch, with an elongation of not less than 8 per cent., measured in a length of 8 in., when tested in the direction of the fibre of the iron, and also to admit of bending cold without fracture as follows :—

Thickness in Inches.		To Bend to an Angle of
$\frac{5}{8}$ and $\frac{1}{2}$	...	35° with the grain, 15° across the grain.
$\frac{7}{16}$ and $\frac{3}{8}$	...	50° with the grain, 20° across the grain.
$\frac{5}{16}$ and $\frac{1}{4}$	...	70° with the grain, 30° across the grain.

"The steel plates shall be free from blisters, scales, laminations, and all other defects, and shall be of such quality that strips cut lengthwise shall have an ultimate tensile strength of not less than 26 tons, and not exceeding 30 tons per square inch of section, with an elongation of at least 20 per cent., measured in a length of 8 in.

"Strips of steel cut crosswise or lengthwise, $1\frac{1}{2}$ in. wide, heated uniformly to a low cherry-red, and cooled in water of 82° Fahrenheit, must stand bending double in a press to a curve of which the inner radius is one and a half times the thickness of the steel tested.

The ductility of any plate, beam, angle, etc., may be ascertained by the application of one or both of these tests to the shearings, or by bending them cold by the hammer. The angle and T-iron must be of the forms and sections shown on the drawings, and must be capable of standing the same tensile and bending tests as are specified for the plates.

"The angle T, H, and [steel to be of the same tensile strength, with the same percentage of elongation as the plates, and to stand such bar tests, both hot and cold, as may be sufficient, in the opinion of the engineer, to prove soundness of material and fitness for the service. The rivet iron must be of the very best and toughtest quality, and shall have an ultimate tensile strength of not less than 24 tons, and shall be capable of being bent through an angle of 120° when cold, without showing signs of fracture. The rivet steel must be of such quality that when tested in the direction of its length it shall have an elongation of 23 per cent. in 10 in., and an ultimate tensile strength of not less than 26 tons, and not exceeding 30 tons, per sq. in., and to bend double when cold without cracking or splitting. The plates to be of the exact thicknesses as figured on the drawings, and to be uniform throughout.

"The plates shall be carefully curved, or flatted, or bent to the required forms as shown in the drawings. The edges of all plates and joints shall be planed quite true to the requisite forms and dimensions, so that the edges of the plates may touch each other truly and over the whole surface, and so that the rivet-holes may be set correctly by measurement from the edges only; and great care must be taken in riveting on the butt plates that the perfect contact of the surfaces may be maintained. The rivet-holes in all wrought-iron plates, angles, and tee-bars are to be carefully marked off in their proper positions with a centre punch, and punched with a nipple punch. No drafting or rimering will be allowed, but the punching must be so accurate that when the work is put together a rivet $\frac{1}{32}$ in. less in diameter than the punched hole must pass easily through all the holes.

"All rivet-holes in steel plates, angle, T, H, and [bars are to be carefully marked off in their proper positions with a centre punch, and punched with a nipple punch $\frac{3}{16}$ in. smaller in diameter than the finished size, and afterwards carefully drilled out to the full diameter shown on the drawings. All burrs left by the drill are to be carefully removed, and all edges of holes in contact with either the head or point of a rivet must be slightly countersunk. At all joints, etc., all rivet-holes are to be punched $\frac{3}{16}$ in. smaller in diameter than the finished size, and afterwards carefully drilled out to the full diameter shown on the drawings when the various parts are in position in the work, so that a correct hole is obtained through the several thicknesses of plates, angles, etc., which may be required to be riveted together.

"Hydraulic riveting to be used as far as possible. The rivet to be red-hot throughout when inserted, and upset in its entire length, so as to completely fill the rivet-hole. All loose rivets, and rivets with cracked, badly formed, or deficient heads, shall be cut out and replaced by others; and rivets are to be cut by the contractor, and at his expense, whenever the engineer or his representative shall consider it necessary to ascertain that the work is properly executed. When the riveting is done by hand, it shall be done entirely by the hammer; the snap is only to be used at the last to give correct form to the rivet head, and must not cut into the iron or steel.

"Any of the iron or steel work cracked or split, at whatever stage of the work it may be discovered, will be condemned, and must be removed and replaced by other iron or steel of the specified quality.

"Where angles and plates require to be bent, for laps or other purposes, no abrupt or sudden bend or square shoulder likely to crack or injure the iron or steel will be allowed.

"The plates and all other iron and steel must be so placed in the work that the fibres shall run in the direction of the greatest strain; the butt plates especially require attention to this.

"Any bolts and nuts required shall be cut in a workmanlike manner with a clean and strong Whitworth thread, and shall be made of iron of an ultimate tensile strength of 24 tons per sq. in., and capable of being bent through an angle of 120° when cold without showing any sign of fracture. They must be heated, and dipped while hot into boiled oil. Any bolts passing through waterways are to be galvanised.

The whole of the work shall receive a coat of hot boiled linseed oil. Before oiling, the iron and steel shall be well scraped and cleaned from all rust and scales, but the contractor must, as far as possible, keep the work oiled before any rusting has commenced. All surfaces that have to be riveted in contact with each other shall be well oiled before being so riveted, and all surfaces, as well as those concealed from view, the exterior of the work, and all lap joints and angles, must be thoroughly oiled.

"The galvanised iron sheets to be No. 16 and 18 B.W.G., riveted at all laps with galvanised iron rivets and burrs, and secured with galvanised $\frac{3}{8}$-in. bolts to ironwork, and round-headed screws to woodwork. The sheets to be in the longest possible lengths; all laps and bolt- and screw-holes to be painted with red-lead ground in oil, to secure water-tight joints. The curved corrugated sheets to be bent to shape before being galvanised. The corrugated sheets to be made from soft, fibrous, and uniform metal, perfectly free from laminations, buckles, blisters, and all defects. The sheets to be carefully sheared to dimensions, thoroughly cleaned, and galvanised with the best zinc to the extent of ·15 lb. per foot super. (plain), including both sides. The galvanising is to be well executed, the sheets being drawn through the flux, and all to have two coats of turpentine, both sides, before leaving the manufactory. Samples are to be provided by the contractor on the requisition of the engineer, and must stand the Admiralty tests.

"The condensation gutters to be Vieille Montagne zinc, No. 16 gauge, $3\frac{1}{2}$-in. girt, with beaded edge and collars every 7 ft., with all necessary stopped ends and outlets, $\frac{3}{4}$-in. galvanised iron gas tubing, with tee-pieces and screw caps, to be provided as shown, to take water from condensation gutters into eaves gutters.

"The whole of the cast-iron shall be made from the best quality of tough grey pigs, and is to bear a tensile strain of 7 tons per sq. in. of sectional area before fracture, and $2\frac{1}{2}$ tons per sq. in. of sectional area before loss of elasticity. Two bars are to be cast in a dry mould from each melting, 3 ft. 6 in. long by 2 in. deep by 1 in. wide; and these, when placed on bearings 3 ft. apart, are to be loaded with a dead load at the centre, and to break with an average of 30 cwt., or a minimum of 28 cwt., and not to deflect less than $\frac{3}{10}$ in. with a load of 25 cwt. The castings are to be true and clean, and free from all defects, and unpainted till they have been examined; all bolt-holes to be cast of proper size and strength of metal. The cast-iron is to be painted one coat of red-lead ground in oil.

"The weights of all cast- and wrought-iron and wrought-steel work have been calculated from the dimensions shown on the drawings on the basis of 40 lb. per sq. ft. 1 in. thick for wrought-iron, 41 lb. per sq. ft. 1 in. thick for steel, and $37\frac{1}{2}$ lb. per sq. ft. 1 in. thick for cast-iron. An addition of 5 per cent. has been made in the case of wrought-iron and steel to cover rivet heads, and on no account will any additional allowance be made on the weights so calculated.

"All pieces of iron or steel work which weigh more than 2 tons are to have their weights plainly stencilled on them in white paint, and with figures not less than $1\frac{1}{2}$ in. high."

CHAPTER X.

PRINCIPLES OF FIREPROOF CONSTRUCTION.

Consideration of the manner in which ordinary dwelling-houses in most towns are constructed, and of the inadequate provision of good water pressure and other requisites for fire extinction, will in most cases lead to astonishment at the comparatively small number of great catastrophes by fire. It is a fact that the internal arrangement of most dwellings, with their floors, ceilings, doors, lathing, skirtings, architraves, and stairs all of wood, affords every facility for the spread of fire after an outbreak has once occurred. And yet similar constructive methods are adopted year after year, and in all probability will continue to be adopted unless legislative restrictions are enforced.

Legislation, so far as regards fireproof construction, cannot be said to be over-restrictive in Great Britain. The building bye-laws enforced by most sanitary authorities contain only a few requirements as to the position of timbers in walls, or in some cases the provision of party walls carried above the level of the roof; and the London Building Act, while containing some excellent regulations as to roofs, the provision of means of escape, and the position and construction of flues, still falls short of the severity of some of the Continental Fire Acts. For any special regard which is paid to the fireproof construction of large warehouses and other similar structures, the fire insurance offices deserve the credit, as they lay down various practical requirements which must be followed before the building can be insured at low rates.

Without resorting to any expensive methods of construction, there are many points in the planning and arrangement of buildings which may be considered with advantage. The popular notion of fireproof construction is confined generally to some form of concrete floor. As a matter of fact, the roof is quite as important a feature.

If a fire should start in the lower storeys of a building provided with the ordinary wooden floors and laths and plaster ceilings, it will, no doubt, make its way with ease upwards; but in very many cases the fire begins at the top of the building, either by reason of the flames from some adjoining building catching the timber roof-work, or, it may be, by some accidental cause in the upper part of the building. This danger of fire descending is particularly liable to arise in warehouses where the stores are inflammable liquids, such as oils, or certain chemicals which will liquefy under the action of heat. One of the most striking instances of this was the destruction of a large warehouse in Berlin a few years ago. The building, six storeys high, was of brick, with so-called fire-proof floors, composed of brick arches between iron girders which were carried by the walls and cast-iron columns. The doors of the separate rooms were of sheet iron. Five months after the building was completed, some temporary openings were made in the floor of the third storey, and, a fire having accidentally occurred, the burning materials fell through to the floor below, setting fire to the contents there, with the result that in five minutes from the time of the outbreak many of the floor arches had collapsed, and in an hour the whole structure was destroyed.

Isolation, not only of the building from the neighbouring buildings, but of the separate apartments one from another, is one of the chief requirements to be attended to. Fireproof construction, in the sense of erecting a building which will defy the fiercest fire, is an impossibility, and the efforts of the architect should be directed to the protection of the building from the attack of fire from outside and the narrowing down of the effects of an internal outbreak by confining the fire to as small a space as possible. With this object in view rooms used for the storage of goods should be kept small, for a large room necessarily means a large fire. Trap-doors from one floor to another should be avoided, and wells for stairs or for lighting should be detached as much as possible from the rooms by walls of fire-resisting materials.

Considered solely from the point of view of resistance to fire, the ideal building would be one built of strong brick walls, with roof and ceilings of groined brickwork, and with openings such as doorways and windows restricted as far as possible in size and number. Such a building is, however, placed outside the bounds of practicability by reason of its expense, the want of adequate light, and the valuable space occupied by the walls. In special circumstances some approach to this class of work may be possible. In the Bank of England, for example, Sir John Soane constructed nearly all the apartments so as to be fireproof, and without any carpentry whatever in the arches and domes, making use largely of hollow pots or cones of a coarse earthenware. These helped to lessen the weight of materials used without taking away from the strength of the structure.

Under the London Building Act of 1894, the following materials are classed as fire resisting: (1) Brickwork constructed of good bricks, well burnt, hard, and sound, properly bonded and solidly put together (*a*) with good mortar compounded of good lime and sharp clean sand, hard clean broken brick, broken flint, grit, or slag; or (*b*) with good cement; or (*c*) with cement mixed with sharp clean sand, hard clean broken brick, broken flint, grit, or slag; (2) granite and other stone suitable for building purposes by reason of its solidity and durability; (3) iron, steel, and copper; (4) oak and teak and other hard timber, when used for beams or posts, or in combination with iron, the timber and the iron, if any, being protected by plastering in cement or other incombustible or non-conducting external coating; in the case of doors—oak or teak or other hard timber not less than 2 in. thick; in the case of staircases—oak or teak or other hard timber with treads, strings, and risers, not less than 2 in. thick; (5) slate, tiles, brick, and terra-cotta, when used for coverings or corbels; (6) flagstones, when used for floors over arches, but not exposed on the underside, and not supported at the ends only; (7) concrete composed of broken brick, stone chippings or ballast, and lime, cement or calcined gypsum when used for filling in between the joists of floors.

Without commenting on the above list at present, it will be convenient to summarise here the characteristics of the ordinary building materials considered from the fire-resisting point of view.

No material has stood the test of fierce fires better than good brickwork. When all other materials have been either destroyed or distorted so as to be useless, the brick walls have been left standing, although the heat may have been intense enough to vitrify the face of the brickwork. The fire-resisting property of bricks depends chiefly upon the amount and relative proportions of the silica and alumina contained in the clay from which the bricks are made; the greater the proportion of alumina to silica, the greater is the infusibility of the clay, although it must be noted that the most refractory bricks are not the strongest for constructional purposes. One great advantage possessed by brickwork is that of comparative freedom from expansion under the effects of heat. It has been calculated that in a 10-ft. length of wall composed of fire-brick, when subjected to a heat of 2,000° F., sufficient to melt cast-iron, the linear expansion is a little more than $\frac{1}{2}$ in.

Terra-cotta is much to be preferred before stone, so far as fire-resisting properties are concerned, as a material for the construction of cornices, panels, and other ornamental enrichments. In the Cripplegate fire a few years ago it was noticeable that much of the terra-cotta work had withstood the flames successfully, and seemed none the worse, while of the stone façades almost every stone had split. It is probable that the chief use of terra-cotta in fireproof construction will, however, be not for ornamentation, but as a protective medium for ironwork. In American construction, in the tall steel-framed buildings of Chicago and New York, where safety from fire is of the utmost importance, it is used in this direction. The main columns carrying the weight of the whole structure are not cast-iron, but built-up steel stanchions, and to protect these from the effects of fire they are encased, first, in slabs of rough terra-cotta, known as "tiles," 3 in. in thickness, and outside this covering there are fixed thick blocks of ornamental terra-cotta which form the exposed face of the columns and the outside shell of the buildings. Another important use of this material is in the construction of fireproof floors. Hollow blocks, strengthened with internal ribs, are used to form skewbacks and arches between the iron joists. In a later chapter that will deal with the construction of fireproof floors, examples will be given illustrating this class of construction.

Although stone of any kind would seem on a first impression to form a good material for resisting fire, experience proves that it is far from being reliable under the action of heat. Granites, in particular, although included amongst the fire-resisting materials scheduled under the London Building Act, 1894, are of little value in this respect. The substance of the granite cracks and flies off in small pieces; indeed, a granite pillar 12 in. square has been reduced to sand by an actual fire in a building, while a wooden post standing next to the granite had only been burnt into for a depth of 1 in. Limestones are calcined by great heat. Sandstones are found to be the most refractory, but even these, when subjected to the heat of a fierce fire and afterwards to a sudden quenching with water, will split and tumble to pieces. In important parts of a structure, such as staircases, any kind of stone is, therefore, inadmissible; for the merely ornamental features of outside decoration it may find a use, but it must not be expected to withstand the combined effects of fire and water.

Of all building materials, Portland cement concrete is the one which, while having excellent qualities as a fire-resisting material, is at the same time comparatively cheap, and easily adapted to varying requirements. In fact, in the popular sense, fireproof construction identifies itself with "concrete floors," as this is the direction in which efforts have so far been made to obtain immunity from fire. Concrete in itself is well able to stand the effects of intense heat and subsequent quenching with water, and when, in addition to the concrete, there is provided an interlacing system of steel girders or rods, in the manner which will be more fully considered in future chapters, a floor is obtained which may be considered practically fireproof. For stairs, also, concrete, in its various modifications of granolithic, Victoria stone, and other special varieties, affords an excellent material.

Plasters of various kinds are little affected by heat. Those having gypsum as a base, such as plaster of Paris, have been used for many years in this connection To still further add to their efficacy, however, they require to be aided by the insertion of wire or metallic lathing. Portland cement and Keene's cement are to be recommended for forming skirtings or architraves in place of woodwork. A recent introduction is asbestic, a bye-product from the manufacture of asbestos from serpentine. When combined with a small proportion of lime, it is claimed that a material is obtained which is absolutely fireproof, and one which will not chip or crumble away. Nails can be driven into it and withdrawn with the same facility as in woodwork. One form of its use is in slabs $\frac{1}{4}$ in. thick, which can be used like wood or screwed or nailed into position. Petrifite, another recent invention, is the name given to a white cement composed chiefly of magnesite, a carbonate of magnesia and one of the constituents of magnesian limestone. It appears to possess many valuable properties, combining with almost all classes of materials, hard or soft, dirty or clean, to form a very strong substance, which can be applied to surfaces in the same manner as other plasters, or can be cast or moulded into blocks of any size and shape.

The woodwork used in ordinary building methods naturally constitutes the chief source of danger from fire. Wooden floors, one above another, with wooden stairs leading from one floor to the next, afford a ready means of spreading the fire when once an outbreak has occurred. When used in sufficient bulk, however, as was before pointed out, wood is capable of withstanding fires which will destroy heavy masonry. A thick mass of woodwork is extremely difficult to burn through, and for this reason a floor composed of planks bolted side by side in any of the methods which will be described in a later chapter, affords a better protection against fire than any construction of iron and concrete. It should be noted that the smooth surface of wrought woodwork does not take fire so readily as does a surface which is left rough from the saw.

In many details of ordinary building construction, the position of the woodwork is well controlled by the ordinary bye-laws in force. Thus, it is provided that door-frames and window-frames shall be set back at least 4 in. from the face of the building, with the object of preventing them falling outwards in case of fire and spreading the fire to other parts. It is forbidden to carry any woodwork across a party wall between two buildings, and the Model Bye-Laws provide that not only shall there be 9 in. of brick or stonework between the ends of beams on opposite sides of a party wall, but that there shall be no bond timbers, plates, or plugs built into a party wall. With respect to timber near the flues, it is ordered that no plugs shall be inserted within 6 in. of a flue. The London Building Act, 1894, provides that "timber or woodwork shall not be placed in any wall or chimney breast nearer than 12 in. to the inside of any flue or chimney opening nor in any chimney opening within 10 in. from the upper surface of the hearth." The spread of fire from one storey to another is frequently helped by wood flooring resting on iron girders which are unprotected on the under side from the action of fire, although the space between them may be filled in with some incombustible material such as concrete, or brick arches. The girders in such a case became hot enough to ignite the flooring above. Oak and teak offer considerably more resistance to fire than almost any other timber, and by many architects an oak staircase is considered to be, in case of fire, safer than a stone one.

There are many methods of preparing timber so as to increase its fire-resisting properties. They depend mostly on the impregnation of the fibres of the wood with certain chemical salts; for example, Sir William Burnett's system is to soak the timber for some days in a solution of 1 lb. of chloride of zinc to 4 gal. of water. Sir Frederick Abel recommends the surface to be painted with alternate coats of silicate of soda and limewash. Phosphate of ammonia, calcium sulphide, iron sulphate, silicate of iron and magnesia, and other salts have all been advocated, but none of these processes have had much practical success. Several preparations for applying as a paint have been received with favour, notably a preparation known as asbestos paint. Sir Frederick Bramwell, in 1884, attributed the saving of the wooden structures of the Inventions Exhibition from destruction by fire to the fact that they were coated with this preparation.

CHAPTER XI.

FIREPROOF COLUMNS AND STANCHIONS.

THE materials and methods adopted in constructing the columns of a building are of the utmost importance. Columns are subject to the effects of flame on all sides, and the isolated position which they occupy provides an ample supply of air by which the heat is increased. Already subjected, as they are, to the stress of the weights they sustain, it is obvious that, when heated by an intense fire, and afterwards suddenly cooled by water, they are very injuriously affected. It is sometimes wise, therefore, to dispense, as far as possible, with columns, by using beams of a greater depth than would otherwise be required, although this has the disadvantage of being a more costly mode of construction, and of curtailing the head room. The best material to use is undoubtedly good brickwork, as it is least subject to the effects of heat and cold; the amount of space it occupies will, however, militate against its use in many cases. As a rule, cast-iron is the material adopted.

As mentioned in the previous chapter, there are certain advantages connected with the use of wooden columns. This material, when sufficiently thick, will withstand fire for many hours, the damage it receives being confined to a reduction of sectional area. Its internal structure is unaltered, and it is not subject to any dangerous degree of expansion or contraction. The amount of damage done to a wooden column can also be readily seen, enabling firemen to work in a building amongst the flames with greater confidence than would be the case where metal columns were in use and liable to collapse without warning.

In America, columns of hard pine or oak are largely used in buildings of the factory and workshop class. They are usually of 9 in. diameter or 9 in. square. Care is taken that the timber is thoroughly seasoned, and to ensure this it is sometimes sawn down the centre, and the two halves bolted together with the central portion out-

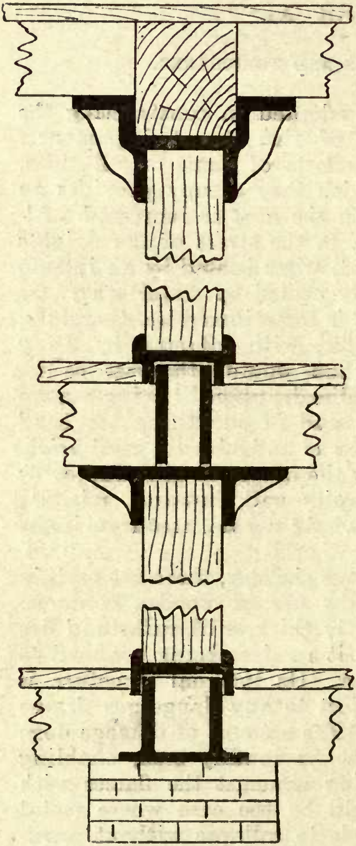

Fig. 137.—Timber Stanchion with Cast-iron Caps, etc.

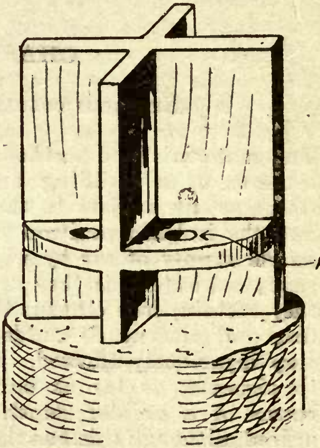

Fig. 138.—Section Cast-iron Stanchions.

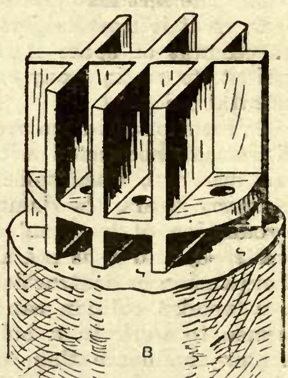

Fig. 139.—Ribbed Cast-iron Stanchion.

side. Another mode of drying the interior of the wood is to bore a $1\frac{1}{2}$-in. longitudinal hole through the centre of the column, with transverse holes of smaller diameter across the top and bottom.

In general practice, no oil or varnish is applied to the outside of any heavy timber for at least three years after it has been placed in the building, so as to avoid the occurrence of dry rot from the fermentation of the sap in the timber. Cast-iron caps and bases of simple design are also used, as shown in Fig. 137.

Columns are generally made of cast-iron, on account of its cheapness and the ease with which the castings can be prepared to suit various requirements. Wrought-iron and steel columns have, however, come into increased use during late years. As a fire-resisting material, cast-iron is very unsafe, as it easily collapses when suddenly cooled by water. In the warehouse at Berlin, referred to on p. 99, the floors were carried by cast-iron columns. After the fire, which only lasted one hour, it was found that out of one hundred columns, thirty-eight had been thrown completely out of position, while thirty-four others, although they remained standing, were so broken or damaged that they were rendered useless.

The city authorities of Hamburg some few years ago carried out a series of experiments on the fire-resisting properties of columns of cast- and wrought-iron. The cast-iron columns were 10 ft. 8 in. long, $10\frac{1}{2}$ in. diameter, and mostly of $\frac{1}{2}$-in. metal. Pressure was applied by a hydraulic press placed below the column, its cross-head being at the top of the column. A hinged oven, containing twelve large gas burners, an apparatus for measuring the heat, and a water jet, was then clamped about the column. After a heat of 1400° F., and a load of 3·2 tons per square inch had been applied, the average result produced deformation in thirty-five minutes. The deformation showed itself by a bulging all round in the middle of the heated part, especially where the metal was thin. With a smaller load, the heat sustained before deformation occurred was correspondingly higher. Jets of water had no effect until deformation heat had been reached.

In addition to columns of circular section, those of other sectional shapes are sometimes adopted, as in the cross-shaped column shown in Fig. 138, and the ribbed stanchion shown in Fig. 139. These have little, if anything, to recommend them in preference to the ordinary circular section, except that they are easy to work in with brickwork and joists. They have also the advantage of being easily protected from fire by encasing them in cement plastering B (Fig. 139), so as to form a circular column with no iron exposed. The stiffeners or horizontal flanges should have holes cast in them, as shown at A (Fig. 138), in order to form a key for the cement.

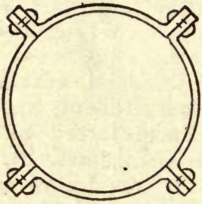

Fig. 140.—Round Column Built Hollow.

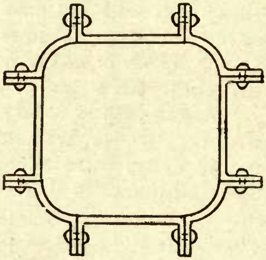

Fig. 141.—Square Column Built Hollow.

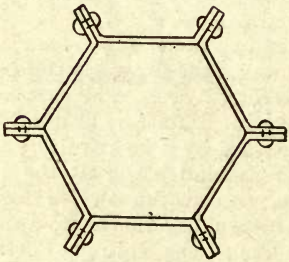

Fig. 142.—Hexagon Column Built Hollow.

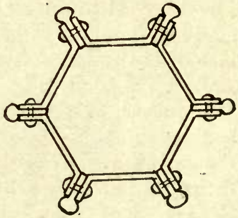

Fig. 143.—Hexagon Column Built Hollow.

Modern practice has largely adopted the use of columns or stanchions built up of rolled steel or wrought-iron bars riveted together. The forms which these built columns take are almost innumerable. Some of the most typical are those formed from ordinary angles, tees, channels, flats, and girders. Some special sections of iron, such as Z-iron, and Lindsay's angles and troughing, are also made. Some columns are made hollow, as shown at Figs. 140 to 143. It should be noted that a hollow column of wrought-iron or steel is liable to corrosion on the inside surface, where it is impossible to paint it after the column has been erected. This defect may be avoided by filling in the interior of the column with concrete. Pease's tubular construction, which will be found described in Chapter XIII., can also be used for forming

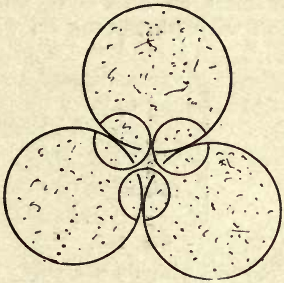

Fig. 144.—Pease's Triple Tubular Construction.

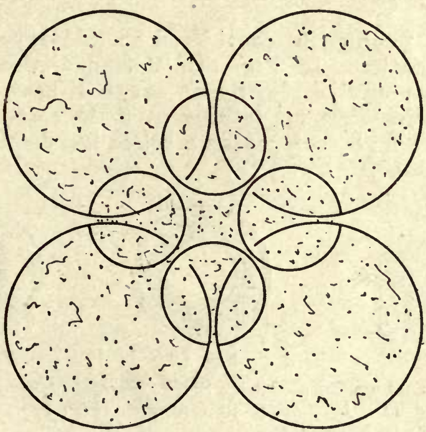

Fig. 145.—Pease's Four Tube Construction.

columns of an ornamental appearance by combining the tubes in the manner shown in Figs. 144 and 145. The interior is filled in with cement concrete, and serves to lock the tubes in place and stiffen the column.

The importance of adequately protecting an iron or steel member from fire depends on the work it has to do, and the consequent destruction which would result from its failure. The collapse of a stanchion would, in many instances, undoubtedly wreck the whole building; in such cases a clumsy appearance must be tolerated in order that ample protection may be afforded. Simply encasing a girder or stanchion with plastering on wire lathing (Figs.

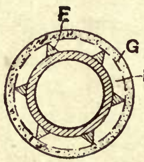

Fig. 146.

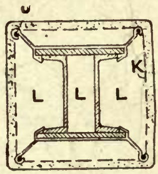

Fig. 147.

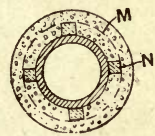

Fig. 148.

Fig. 146.—Column Protected with Plaster only: Section. Fig. 147.—Stanchion Protected with Plaster only: Section. Fig. 148.—Column Encased with Solid Concrete: Section.

146 and 147) cannot be considered an adequate protection in any situation. In Fig. 146, V-shaped pieces of sheet iron F are placed vertically at equal distances apart to block the wire or metal lathing G off the column, leaving an air space H; by this means a better key for the plastering is obtained.

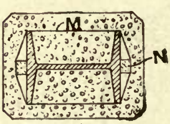

Fig. 149.

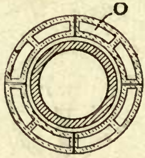

Fig. 150.

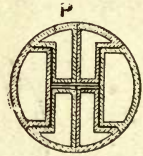

Fig. 151.

Fig. 149.—Stanchion Encased with Solid Concrete: Section. Fig. 150.—Hollow Tile Protection to Column. Fig. 151.—Hollow Tile Protection to Stanchion.

In Fig. 147 light brackets or clips are employed to grip the stanchion and secure vertical wires J in position at the corners, round which the lathing K is strained, L indicating an air space. Figs. 148 and 149 show a simple method of protecting a column and stanchion. The fine mesh galvanised wire netting M is blocked away from the surface with pieces of tile N, and the intervening space filled in with fine breeze concrete. In order to get the external surface of the concrete true and regular to receive the plastering, boxes or moulds about 4 ft. long are made and fixed round the column, into which the breeze concrete is run and allowed to set thoroughly before the next section in height is executed.

Porous tile casings, 2 in. to 4 in. thick, are admirable, providing the horizontal as well as the vertical joints are so formed that there is no possibility of the sections being displaced by the fire or the force of a water-jet. Any space behind the tiles should be filled in solid. This

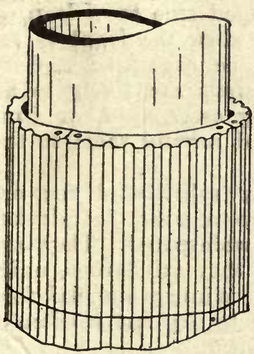

Fig. 152.—Tile Covering for Column.

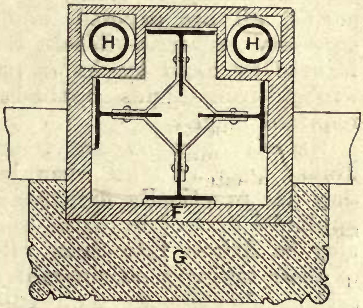

Fig. 153.—Fireproof Tiling and Terra-cotta.

will not only render the tiles more secure in position, but will prevent the fire entering the joints. Such casings must continue from floor to floor, and be independent of any combustible material for support.

Fig. 150 shows a tile casing O to a hollow column, and Fig. 151 a tile protection P to a stanchion in which the external diameter is increased as little as possible. There is no difficulty in designing a tile casing for any column or stanchion when the ultimate shape required is known. But as such things are not stocked to any extent, they usually have to be made to order. The tiles are made of various shapes, and many patents have been granted for different methods of combining them and securing them in place. As a general rule, they are fluted on the outside to form a key for a finishing coat of fine plaster or cement. Fig. 152 illustrates one type in which the tiles are secured to each other by iron pins or dowels, the whole being set in Portland cement, and plastered on the outside. A method of casing in a built stanchion with tiles, and facing it on the outside of the building with ornamental terra-cotta blocks, is shown in Fig. 153, in which F indicates the fireproofing, G the terra-cotta, and H the steam-pipes.

All ironwork should be free from corrosion when fixed, and be embedded in an air-tight and damp-proof casing. Pipes should not be carried down in proximity to columns or stanchions, but should be arranged side by side in a separate casing or chase, so that, when the front is removed, all are exposed, and can be examined and if necessary repaired. Where the casings are likely to be injured by hand trucks or packing cases, $\frac{1}{4}$-in. or $\frac{3}{8}$-in. wrought-iron shields should be fixed around them to the required height.

Asbestic plasters are frequently substituted for ordinary plasters, and when laid on a solid foundation they are practically fireproof and waterproof. They are applied $\frac{1}{2}$ in. to $\frac{3}{4}$ in. thick, and, as they are fibrous, nails may be driven into them without injury; but the fact of their taking a considerable time to dry out is often a serious objection to their employment.

CHAPTER XII.

FIREPROOF FLOORS.

Fireproof floors have been more elaborated and have received more attention from architects and engineers than any other branch of fireproof construction. This subject has been almost constantly studied since the days when Mr. David Hartley set fire to the lower storey of his experimental fireproof house in the presence of King George III. and the Lord Mayor of London, he and his friends demonstrating their faith in the safety of the structure by remaining in the upper room. This being so, only a few of the typical forms will be dealt with.

The iron girders supporting a floor should not be built into the walls at the ends, but should be left free to expand or contract without disturbing the brickwork. For this purpose they are best carried on brick corbels projecting from the walls. Stone corbels are not so good, as when exposed to fire they split, no doubt as the result of unequal heating. Concrete practically does not expand either under fire or in the process of setting when the floor is being made. In the latter case there may be a slight contraction with a return to the full dimension afterwards, but the contraction does not exceed $\frac{1}{32}$ in. in 30 ft. Iron joists, on the contrary, may expand under fierce heat to the extent of $1\frac{5}{8}$ in. in 10 ft. The expansion in a 10-ft. length of fire-brick flooring under a heat of 2,000° F. has been calculated to be about half an inch.

To determine the relative efficiency of different methods of protecting iron joists from fire, experiments were conducted a few years ago by Mr. Stanger with three similar sets of 5-in. by 3-in. rolled iron joists, each weighted with 30 cwt. of pig-iron distributed over 8 ft. of their length. One set was embedded in concrete made of Portland cement and sand in the proportions of 1 to 5, to a thickness of 15 in. by 7 in. The second set was protected with Doulton-Peto tiles, and covered with concrete (half Portland cement and half sand); and the third packed with silicate cotton and cased in a similar cement and in plaster-of-Paris to a depth of 1 in., the sectional area of the whole being 15 in. by 7 in. At 11.22 a.m. the three sets were placed over a furnace having a temperature of 2,000° F. The joists covered with concrete were the first to collapse, dropping down at 1.7 p.m.; those covered with Doulton-Peto tiles held up till 3.40 p.m.; but at 6.45 p.m. those covered with the cotton silicate were still unscathed,

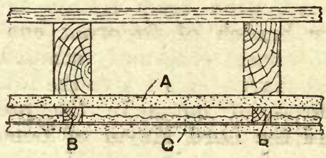

Fig. 154.—Asbestos Slabs under a Wooden Floor.

the deflection they had made not being ¼ in. The fire was then left to burn itself out, and, at the end of eleven hours' exposure the silicates were found to be practically unaffected.

When iron or steel joists are embedded in cement concrete, it is not necessary, nor is it advisable, to paint or oil them, as the cement adheres better to the bare iron than it would to the painted surface; and itself forms an efficient protection against rust. In this respect it is unlike lime concrete, which causes rusting to a marked

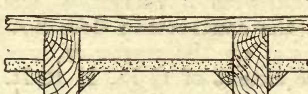

Fig. 155.—Asbestos Slabs Between Joists.

degree—so much so, in fact, that large blocks of stone fastened together by iron clamps set in lime mortar have been forced asunder.

One of the simplest and most economical methods of constructing a fire-resisting floor is to protect an ordinary wooden floor with slabs of asbestic plaster or of slag wool (silicate cotton), both of which can be obtained commercially in slabs, as cloth, or in the form of loose fibre or wool. The loose wool is useful for filling up the spaces between the joists as a pugging to deaden sound (as already described), as well as affording protection against fire. A convenient method of attaching the slabs is shown in Fig. 154. The slabs are formed by enclosing silicate cotton between sheets of galvanised wire netting, and are made of thicknesses varying from 1 in. to 3 in. They are secured to the under side of the joists, as shown at A by wooden fillets B B nailed underneath, the nails passing through the slabs. To these fillets are secured the laths, when a lath-and-plaster ceiling C is desired. Additional security can be obtained by placing other slabs between the joists, resting on triangular fillets as shown in Fig. 155. Owing to the comparative cheapness of these methods of construction, and the measure of security they

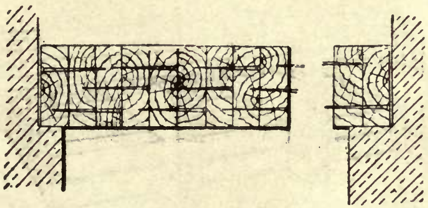

Fig. 156.—Solid Wood Floor.

afford, they are worthy of more general adoption in dwelling-houses and office buildings.

Woodwork, when used in solid masses, is an excellent material for fireproof construction. It is extremely difficult to destroy timber in bulk by fire, and in America, partly on this account, and also on account of the cheapness of timber, floors and walls are constructed of planks nailed together face to face. The walls of many of the large grain elevators and station buildings are constructed in this way. The system of forming floors by close timbering instead of the ordinary use of joists and flooring boards, was introduced into England by Messrs. Evans and Swain between 1870 and 1880. The joists, instead of being placed at some distance from each other, were laid close together, so that air could not penetrate between them, the planks being then spiked as shown in Fig. 156.

As an alternative method, the spikes could be driven in diagonally, and, if thought necessary, the under side of the planks could be protected with a plaster ceiling keyed into grooves formed in the planks. As a test of the capability of this system, a building was erected 14 ft. square inside of 14-in. brick walls, and measuring 7 ft. from the ground to the ceiling. The flooring was laid as described above, of deal battens 7 in. deep by 2½ in. thick, spiked together side by side. One-third of the under side was plastered, the joists being grooved for this purpose; one-third was plastered on nails partly driven into the planks, and the remaining third was left unprotected. The chamber underneath was packed almost full of timber, which was then lighted, and it was not until after five hours' continuous exposure to the flames that the un-

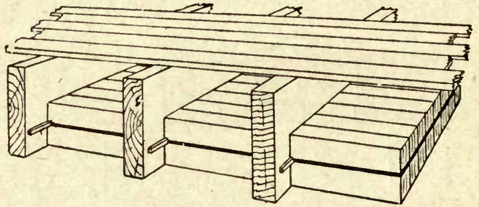

Fig. 157.—Hinton and Day's Wood-block Floor.

protected portion of the floor gave way. The system was afterwards adopted in large warehouses for the East and West India Docks, London, and in other buildings.

A modification of the system just described has been patented by Messrs. Hinton and Day, and is illustrated in Fig. 157. The joists are spaced apart in the ordinary way, but the spaces are filled in with solid blocks, having the grain placed vertically, tongued and grooved together in such a manner that the passage of air between them is prevented. The blocks are carried by fillets nailed to the sides of the joist. A test of this system of flooring was made at Westminster. Four walls of 9-in. brickwork were erected, and the under side of the floor to be tested was 9 ft. 6 in. from the ground. The lower part of the building was filled three parts full with inflammable material (no petroleum or grease, however), and a fierce fire maintained for more than two hours, after which it was extinguished, and the under side of the floor was found to be charred to a depth of $\frac{3}{4}$ in.

In American factory and workshop buildings a layer of mortar D is often introduced between two thicknesses of flooring as shown in Fig. 158. Here 8-in. by 4-in. wooden joists E support the flooring planks, which are 3 in. thick, on which a layer of mortar, $\frac{3}{4}$ in. thick, is spread. Floor-boards $1\frac{1}{2}$ in. thick, laid on the top of this, form the working surface of the floor. Sometimes the floor-boards are laid in two thicknesses, crossing each other diagonally, as shown in Fig. 159, in which F indicates the layer of mortar. The beams carrying the floors have air spaces round each end, and to avoid the danger of the wall being pulled down by a falling beam

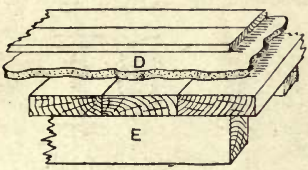

Fig. 158.—American System of Factory Wooden Floor.

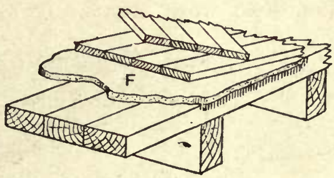

Fig. 159.—American Floor with Diagonal Double Boarding.

in case the latter should be burnt through, the upper end of the beam is cut away at both ends so that it can fall freely.

Gypsum Floors.—For nearly three hundred years floors have been made with gypsum, or hydrated sulphate of lime, which is the basis of plaster-of-Paris. In Nottinghamshire and Derbyshire, as well as in certain parts of France where the stone abounds, it is gently heated until the combined moisture is expelled. The resulting coarse powder is mixed into a paste and laid over rough boards, or between and over the joists, without the use of floor boards, forming a floor of 8 in. or 9 in. thickness, which sets quite hard in a few hours. As to the merits of this material, opinions differ; Mr. Hamor Lockwood, who has had a large experience in the construction of fireproof floors, says that it possesses the great disadvantage of not being able to stand either fire or water, and after being laid down for years will, on becoming soaked with water, expand to such an extent that the building in which it is used is endangered, and so, in case of fire, practically ensures the destruction of the building. On the other hand, in Rivington's "Notes on Building Construction" it is stated that "this substance [gypsum] does not lose its cohesive power even when it is roasted to a white heat and then drenched with cold water."

Brickwork Floors.—A common form of fire-resisting floor for factories and warehouse buildings is one made by dividing the floor space into bays, by means of cast- or wrought-iron beams, and then forming arches of brickwork between the beams, the spandrels of the arches being filled up with concrete to make a flat upper surface. At first cast-iron beams of an inverted ⊥-section were used, but these were eventually superseded by rolled iron or

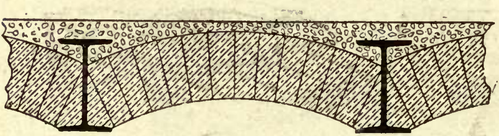

Fig. 160.

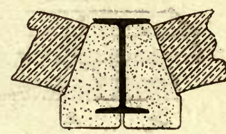

Fig. 161.

Fig. 160.—Brick Arch Floor. Fig. 161.—Blocks to Protect Girders from Fire.

steel beams of H section, as shown in Fig. 160. In this floor, it will be seen, the undersides of the girders are exposed to the flame, and a severe fire might cause the floor to collapse. A much better method is to enclose the girders in blocks of terra-cotta or fire-clay, in the manner shown in Fig. 161. The sides of the blocks are shaped to form the springing of the brick arch. In order that the weight of the brickwork may be lessened many forms of hollow bricks have been devised for the arches, but this type of floor as a whole is being generally displaced by more modern forms of greater simplicity. With a view to save headroom, and also to form a flat ground for the ceiling on the underside of the floor, many systems of hollow keyed blocks of fire-clay or terra-cotta have been introduced. Of these, Figs. 162 and 163 will serve as types.

Concrete Floors.—Amongst all the various forms of floors made with concrete, alone or in combination with iron, one of the most satisfactory and simple is that made of large slabs of concrete carried on steel joists, or on beams of concrete. The carrying capacity of such a floor naturally depends upon the quality of the concrete, but when it is found that loads of 14 cwt. per square foot can be carried on floors actually constructed for use, and not merely on experimental slabs, no fear need be felt as to the adequacy of such floors if properly constructed.

The cement used should be extremely fine, one practical maker specifying that not more than 10 per cent.

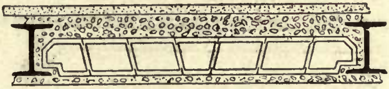

Fig. 162.—Floor made of Hollow Blocks.

shall be retained on a mesh of 14,400 per square inch. The size to which the aggregate should be broken will vary with the thickness of the floor required; for a 6-in. floor, 1-in. gauge will be sufficient, while for a 2-in. floor the aggregate should be of $\frac{3}{8}$-in. gauge. In choosing materials for the aggregrate, porous materials like brick, clinker, and coke will be found to require less cement than hard or smooth stones like granite or gravel, but the latter will, of course, resist weight better, and are more suitable for the upper surfaces of the floors.

Fig. 163.—Hollow-keyed Block Floor.

Experiments on the strength of concrete slabs have shown that with 1 part of cement to 6 of aggregate, a slab, 3 ft. 3 in. by 1 ft. 6 in. by 9 in. thick, with the supports 27 in. apart, breaks with $2\frac{1}{2}$ tons to 3 tons central load. With 1 part of cement to $4\frac{1}{2}$ parts of aggregate, a slab, 2 ft. 6 in. long by 1 ft. 6 in. broad and 6 in. thick, with the supports 18 in. apart, broke with $2\frac{1}{2}$ tons central load. With carefully prepared special materials, such as Stuart's granolithic, very much heavier loads can be sustained; thus, tests carried out on floors constructed of this material have shown that slabs of 5 ft. 6 in. span and 2 in. thick will safely carry a load of 5·35 cwt. per square foot of surface. Slabs of 6 ft. span and 3 in. thick carried $14\frac{1}{2}$ cwt. per square foot, while slabs of 7 ft. span and $2\frac{1}{2}$ in. thick carried $11\frac{1}{2}$ cwt. per square foot. Experimental slabs, 19 ft. by 13 ft. by 3 in. thick, carried a load of 63 tons without breaking.

Fig. 164 illustrates one bay of a floor constructed by Messrs. Stuart, and shows the flat slabs 3 in. thick designed to carry a load of $1\frac{1}{2}$ cwt. per square foot. In one direction, rolled steel joists, 10 in. by $4\frac{1}{2}$ in., are enclosed

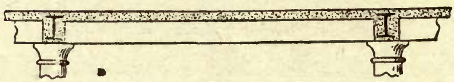

Fig. 164.—Granolithic Floor.

in granolithic to protect them from fire, while in the other direction beams of granolithic, measuring $9\frac{1}{4}$ in. by 6 in., are used.

Instead of using separate beams of concrete, as referred to in the last example, the beams may be made to form a part of the slab, as shown in Fig. 165. A slab of this description, made of granolithic 3 in. thick and measuring 17 ft. $5\frac{5}{8}$ in. by 13 ft. $\frac{1}{2}$ in., bearing on all four sides, carried a distributed load of 23 tons, which is equal to 2 cwt. per square foot. Extending this idea further, there has recently been constructed at the power station of

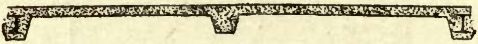

Fig. 165.—Granolithic Slab with Strengthening Rib.

the Edison Electric Illuminating Co. a concrete floor, 4 in. thick, in one undivided mass, strengthened by ribs or beams cast on the underside. At intervals of 15 ft., beams 18 in. deep by 9 in. wide are formed, running across the building and resting upon supporting piers of brickwork. In the other direction similar ribs are formed 3 ft. 6 in. apart; these being 14 in. deep and tapering from 6 in. to 4 in. in width. As a fire-resisting floor, this appears to be an excellent arrangement, while its cost is said to be less than if iron and concrete had been used in combination.

In cases where a wooden floor is desired over a concrete basis, it is not always easy to find a satisfactory method of combining the two materials. A common plan is to insert in the concrete, before it is set, wedge-shaped bearers such as are shown in Fig. 166, to which the floorboards are nailed. An objection to this method is the lack of ventilating space, and, as the bearers are inserted while there is still much moisture in the concrete, there is great danger of dry rot ensuing. This may be avoided by using wedge-shaped bricks of breeze concrete, let into

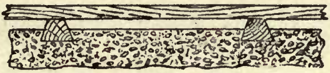

Fig. 166.—Concrete Floor, Boarded.

the concrete floor in the same way as the wooden bearers. The bricks are made 12 in. in length, 3 in. broad on the top surface, and $2\frac{3}{4}$ in. thick. Of course, they will not rot like wood; whilst the floor-boards can be nailed to them.

Concrete arches, supported on rolled steel joists embedded in concrete, form a good floor. This form of construction is shown in Fig. 167. The spans may vary from 6 ft. to 20 ft. or more, according to the quality of the concrete; the rise given to the arches is generally about 1 in. for every foot of span. Mr. Hamor Lockwood, who has constructed this class of flooring for many

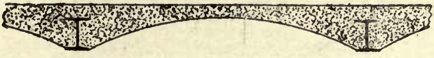

Fig. 167.—Concrete Arches.

years, says "it requires no skewbacks, there being no thrusting power as in the case of brick arching; and it can therefore be employed where bricks cannot." This statement is open to doubt, especially when we find a firm of the experience of Messrs. Stuart, the makers of granolithic, providing special arrangements to meet the thrust of such arches. As regards the strength of this form of flooring, some experiments were conducted by Mr. Legg, at the Hackney Town Hall, on a floor 16 ft. 6 in. by 13 ft., divided into three bays of 5 ft. 6 in. span. The rise of the arches was 5 in., and the upper surface of the floor was horizontal, the material being 4 in. thick at the crown and 9 in. at the haunches. The girders upon which the arches rested were rolled iron joists $8\frac{3}{4}$ in. by 4 in. A stack of bricks, 8 ft. 9 in. long, 4 ft. wide, and 6 ft. high, weighing $6\frac{3}{4}$ tons, or 4 cwt. to the super. foot, was sustained for several days without deflection in the arches or girders. With a better class of concrete, such as granolithic, much higher results can be obtained. Thus, an arch of 5-ft. 6-in. span, 2 in. thick at the crown, and $7\frac{1}{2}$ in.

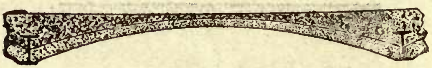

Fig. 168.—Concrete Arch with Inferior Quality of Concrete in Spandrels.

at the haunches, carried 6·32 cwt. per square foot. Another, of the same thickness but of $\frac{1}{2}$-in. wider span, carried 8·85 cwt. per square foot. One of 9-ft. span and only $1\frac{1}{2}$ in. thick at the crown carried 23·6 cwt. per square foot, and another, 21-ft. 6-in. span and 3 in. thick at the crown, carried 8·14 cwt. per square foot. An arch of 15 ft. span, with a rise of 18 in. and thickness at the crown of 3 in., broke with a distributed load of 44 cwt. per square foot; and as a factor of safety of 3 may be assumed for this class of floor, $14\frac{1}{2}$ cwt. may be taken as the safe distributed load per square foot for this arch. Where there

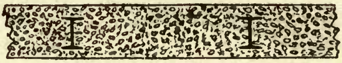

Fig. 169.—Concrete Flat Floor.

is a great difference in the thickness of the crown and the springing of the arches, the high-class material is used only in a comparatively thin arch, as shown in Fig. 168, the spandrels or haunches being filled in with a poorer quality of concrete.

A simple and good style of flooring is shown in Fig. 169, where steel joists are completely embedded in concrete, and the top surface is finished with a coating of material of finer quality. The sizes and distances apart of the joists will, of course, vary with the size of the floor and the amount of weight to be sustained, but as a rule they are placed 2 ft. or 2 ft. 6 in. apart. For a safe load of not more than 3 cwt. per square foot, and a span of 12 ft. to 15 ft., steel joists 3 in. by $1\frac{1}{4}$ in. may be used at 2-ft. 6-in. centres, the concrete floor being made 7 in. or 8 in. thick. For spans of 18 ft. to 20 ft. the floors may be 1 in. thicker, and the joists 4 in. by $1\frac{3}{4}$ in. at 2-ft. centres. It is a mistake to look upon the concrete between the joists as being only so much more additional dead weight for the joists to carry : as a matter of fact, the concrete plays an important part in stiffening the girders and so enabling them to carry greater loads without deflection. As Mr. Hobbs points out, " a bar of concrete 3 in. square will break with 3 cwt. when placed on bearings 3 ft. apart; and a $\frac{1}{2}$-in. square rod of iron, with a load of 28 lb., will bend so as to slip between the sup-

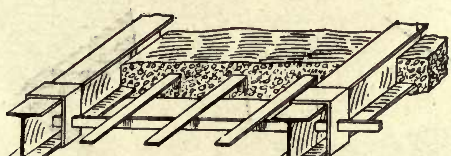

Fig. 170.—Concrete Floor with Iron Bars Embedded.

ports. Yet bed the iron in the concrete and it will carry 10 cwt." And again, " a concrete bar 8 in. by 6 in., with 6-ft. bearings, will break with 1,500 lb. suspended from the centre; put in five pieces of $\frac{3}{8}$-in. round steel, well below the neutral axis, and it will carry 9,500 lb." This principle of reinforcing the concrete by the addition of bars of iron or steel of small section has been applied in several ways. Sometimes the rods are simply laid on the bottom flanges of the girders between which they come; sometimes they are bent round the bottom flanges so as to be secured to the girders. Sometimes they are round in section, in other cases square or twisted. The amount of iron to be used seems to be from $\frac{1}{30}$ to $\frac{1}{40}$ the sectional area of the concrete. The best results are obtained when many bars of small sections are used, instead of only a few bars of larger section; thus, four bars of 1 in. by $\frac{1}{8}$ in. will give greater strength to the floor than one bar of 1 in. by $\frac{1}{2}$ in., as a greater surface of iron is presented for the adhesion of the concrete.

A modification of the above idea is shown in Fig. 170, where the girders are 3 ft. 3 in. apart, with flat bars spanning the spaces between carried on wrought-iron clips. On these flat bars other bars, $\frac{1}{2}$ in. square, are laid about 9 in. apart, and the whole is then buried in the concrete. On this system, with a span of 12 ft., the joists would be $4\frac{3}{4}$ in. deep, and the thickness of the floor about 8 in. For a span of 20 ft. the joists would be 7 in. deep, and the thickness of the floor about 10 in.

Expanded metal (see Chapter XIII.) is an excellent material for use in the manner described above. The lightness of the material and the mechanical bond obtained

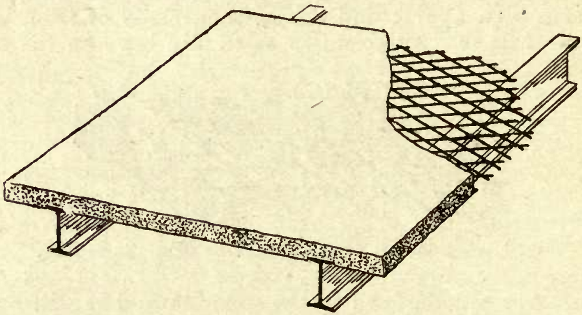

Fig. 171.—Concrete Floor with Expanded Metal Embedded.

in conjunction with the concrete render it valuable for this purpose, and the fact that it is now made in sheets of various sizes up to 16 ft. by 8 ft. allows it to be easily and rapidly laid in position. In 1896 Messrs. Fowler and Baker presented a report on a series of experiments conducted by them on the strength of concrete slabs compared with similar slabs strengthened by expanded metal embedded in them. Summarising these experiments, the report says that the use of expanded metal in the case of slabs of 3 ft. 6 in. span increased the strength of a flat concrete slab six to eight times, and, in the case of a 6-ft. 6-in. span, the strength was increased ten and eleven times. Fig. 171 illustrates the method of applying this material.

A well-known system is the one shown in Fig. 172, in which steel joists are placed 18 in. apart, and connected by wrought-iron strips passing alternately over and under the joists, the whole structure being then embedded in breeze concrete.

Fig. 172.—Lindsay's Concrete Floor.

A very strong floor for use in warehouses or factories, where heavy loads have to be carried, is formed by using Lindsay's troughing, in the manner shown in Fig. 173. The upper portion of the troughs is covered with concrete to form the floor, and, in order to protect the under

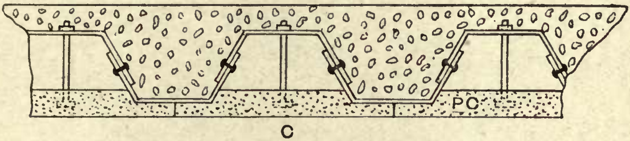

Fig. 173.—Lindsay's Trough Flooring.

side from fire, blocks of lighter porous concrete are suspended by bolts as shown in the illustration at P C. The space between the troughs and the ceiling blocks can be used to accommodate pipes for gas, water, or ventilation. Another large class of fire-resisting floors is based on the

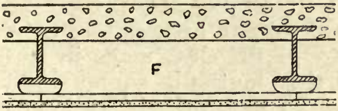

Fig. 174.—Fawcett's Floor: Cross Section.

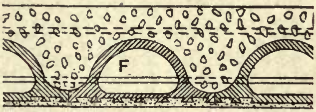

Fig. 175.—Fawcett's Floor: Longitudinal Section.

use of earthenware hollow blocks for bridging the space between the rolled joists.

A great many different systems have been put forward, but as the general principle is the same in all of them, it will be sufficient to notice the following: Fawcett's system, illustrated in Figs. 174 and 175, consists of iron or steel joists, spaced at 2-ft. or 3-ft. intervals, with the intermediate space filled in with earthenware lintels P of arched section, and covered with a layer of cement concrete. It will be seen that the ends of the earthenware lintels are notched to clip over the bottom flanges of the joists and so protect them from fire. The undersides of the lintels are recessed in grooves to form a key for a plaster ceiling.

Homan and Rodger's floor is very similar to the above, but the earthenware pipes are of triangular section, as shown in Fig. 176.

In the "Mulciber" system the space between the joists is spanned by fire-clay blocks, arched on the upper side,

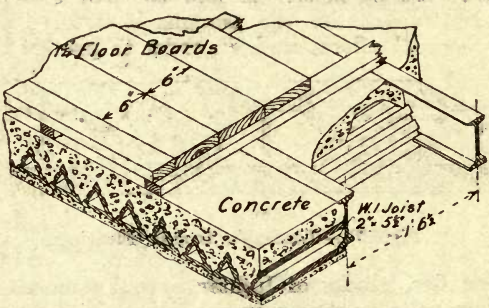

Fig. 176.—Homan and Rodger's Floor.

as shown in Fig. 177. Special blocks are used to cover the lower flanges of the joists.

In addition to the systems illustrated above, there are many which may be described as flat arches, formed of separate blocks of earthenware or fire-clay, made in such shapes that, in combination, they mutually support each other, in the same manner as an arch of brickwork. To lessen the weight of the floor each block is made hollow, with internal strengthening ribs. A typical example of this construction is shown by Fig. 163 (p. 119).

Pease's tubular construction will be found described in Chapter XIII. For the formation of floors the tubes may be used in the manner shown in Fig. 178, the interior of the tubes being filled in with coke breeze concrete, and the upper surfaces covered with the same material. With tubes formed of 20 B.W.G. iron, it is claimed that in a span of 10 ft. the deflection is ¼ in., with a load of 3 cwt. per square foot. Another modification of the system aims at making the floor not only fire-resisting, but fire-extinguishing. This it is proposed to do by keeping the tubes nearly full of water, at a level maintained by a tank and ball-tap. Short vertical pipes inside the tubes reach above the level of the water inside, and communicate with the open air on the under sides of the tubes. It is expected that the heat of a fire beneath would cause sufficient steam in the pipes to extinguish the fire by the escape of steam through the vertical pipe.

Only the very best cement, obtained from a reliable

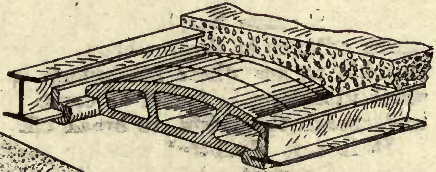

Fig. 177.—"Mulciber" Floor.

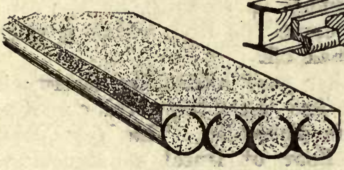

Fig. 178.—Pease's Tubular Floor.

maker, should be used for concrete floors, as, if badly burnt or insufficiently ground and air-slaked, it is liable to expand with irresistible force when settling, thrusting the walls in all directions. A wise precaution is to leave a margin all round the walls, to be filled in when the floor has thoroughly set. In one case known to the writer, the only reason that could be assigned for the concrete expanding was the unsuspected presence of a small percentage of lime in the furnace breeze used, which came from a railway yard; it was found that the enginemen threw lumps of chalk into the fireboxes of the locomotives to prevent the fuel they used clinkering badly.

Stone templates supporting heavy girders should have their front edges chamfered to prevent chipping, which would happen when excessive pressure was applied near the edge.

Terra-cotta lintel floors are convenient to use, as the lintels which span the space between the parallel joists and obtain support on their bottom flanges answer the purpose of centering, as well as providing an admirable surface to receive plastering; but although they do this, they are considerably more expensive than solid breeze concrete floors. The great drawback to lintel floors is that the steel work has to be planned to suit them; the joists must be spaced centre to centre exactly the length of the lintel, which is usually only 18 in. or 2 ft., and they cannot, unless fixed diagonally (which practice increases their span), be inserted when the joists are built in; and should one get broken when concreted over, it cannot satisfactorily be replaced unless the whole bay is cut out. The iron in the clay of which these lintels are sometimes made has been known to turn the plastering which adheres to them a brown tint. These floors are not suitable to use where machinery is working, as the constant vibration is found to cause the lips, which project underneath the joists, to break off.

It is very doubtful whether they afford the steelwork any better protection from fire than the solid concrete floors in which the joists are embedded. The New York Fire Department tested a floor of lintel construction, and, as a result, found that the tiles cracked and fell away badly under the application of water when at a high temperature, leaving the under side of the joists exposed. In hospitals, solid floors only will probably be employed in the future, as any hollow spaces would harbour germs if there happened to be any means of access. The lintels usually drop $1\frac{1}{2}$ in. below the soffit of the joist, so that its lip can pass clear under and protect its bottom flange, and admit a free circulation of air over the whole ceiling. Ventilators may be placed in the external walls to assist this circulation; but if anything went wrong with the under side of the floor, these ventilators would act as flues to a fire raging below, and supposing ventilators were not employed, the air, bottled up, overheated, would expand, and burst the floor open. The writer considers that so long as these lintels are made of a material which will not stand the application of water when heated, the solid concrete floor, with hard cinders and clinkers as a base, which projects 1½ in. to 2 in. below the bottom flange of the joists, is a better fire-resisting construction.

Hard terra-cottta is brittle, and much less able than the porous article to resist fire or the application of water when at a high temperature.

Porous terra-cotta is manufactured by mixing and thoroughly incorporating sawdust or chopped straw with the clay in the proportion of one of sawdust or straw to two of clay. In the burning, the combustible particles of sawdust or straw are destroyed, and cavities remain. A less porous article is made by mixing a small percentage of bituminous coal-dust with the clay. The coal particles must be of a rich quality, as no stain must be left after burning. Porous terra-cotta is a better non-conductor of heat than the hard quality, but it will not support such heavy loads; it is consequently well adapted for column or girder protection, but, owing to the extra labour involved in its manipulation before burning, it is more expensive.

Portland cement concrete with a stone base is a good non-conductor of heat. Its expansion is slight, and nearly corresponds to that of iron; but there is no doubt that it is seriously injured by the prolonged action of fire. This can be easily understood, because the water in combination—about 20 per cent. by weight—in the form of hydrates, is driven off, and the adhesive nature of tne cement totally destroyed. A fire has only to continue long enough to completely disintegrate the material. On the contrary, good, hard, coarse cinder of clinker concretes, preferably without any admixture of sand, resist the action of fire and quenching to a much greater degree. For although the surface in contact with the flames may be destroyed, and a water jet may wash it away to some extent, there is little doubt that the body will remain sound through any ordinary fire, although it may be afterwards necessary to reconstruct it wholly or in part.

There is only too much reason to fear that these cinder concretes have a corrosive influence on the iron or steel embedded in them, although to what extent the one or two coats of paint usually applied can be relied upon to counteract this is at present unknown. Good Portland cement mortar has been found to preserve iron from corrosion, so that if all ironwork were first given a coat of this mortar before the concrete is put in, there is little doubt that it would be perfectly protected from corrosion. Limestones used as a concrete base, when in direct contact with the metal, are found to be very injurious, and should not be employed.

Four to one is a common proportion for cinder concretes, and a top layer, $1\frac{1}{2}$ in. thick, of 2 of granite chippings to 1 of cement, makes a good floor finish. If it is desired to add sand to the cinders, sufficient must be added to fill the voids, and the proportion would usually be 1 of cement, $2\frac{1}{2}$ of sand, and 5 cinders. Waterproof paper on top of the centering boards prevents the escape of water and cement when placed in position, and causes

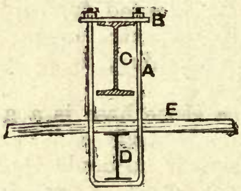

Fig. 179.—Section showing Stirrup Support for Centreing.

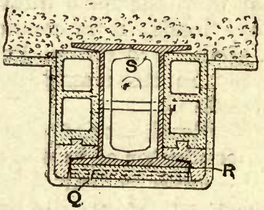

Fig. 180.—Hollow Tile Protection to Coupled Girder.

a good hard surface to form on the under side. Good cinder concrete weighs about 80 lb. per cub. ft., and rough cinder filling about 60 lb.

Fig. 179 shows how the centering is fixed to support a solid concrete floor until it has properly set. Iron stirrups A, made of $\frac{1}{2}$-in. or $\frac{3}{4}$-in. bar, with $2\frac{1}{2}$-in. by $\frac{1}{2}$-in. top plate B, are placed round the joists C to support small wood beams or iron joists D, which in turn carry the boards E supporting the concrete.

Figs. 83 and 98 (pp. 59 and 68) show the common method of protecting girders with fine breeze concrete. No. 20 hoop-iron is bound round the girder, at distances of 1 ft. or 18 in., and is blocked off flat surfaces by pieces of tile. Rough centering is then fixed, and fine rich concrete rammed well round. When set the centering is removed, the hoop-iron keeping the sides and soffit in position. In Fig. 83 the coupled girders are spaced apart 3 in., so that concrete can be filled in between them, and so assist to support the wide soffit.

If tiles are used as a girder protection, as shown in Fig. 180, they should be mechanically held in position, and not depend on the cement in the joints, because a fire will quickly destroy it and the tiles will fall away. The hollow soffit tile can be held in position by stout hoop-iron passing through it and bent round to grip the bottom flange. In this figure, Q indicates soffit tile; R hoop-iron clip; S cast-iron separator.

Figs. 98 and 107 (pp. 68 and 76) show different floor finishes. For wood-block, mosaic, or asphalt, a hard, smooth surface is required. This is usually obtained by

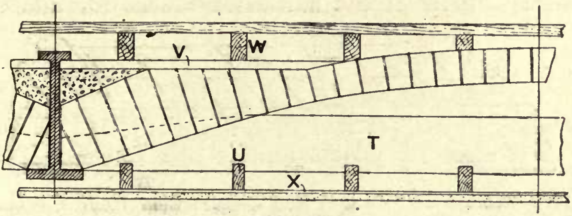

Fig. 181.—Section of Floor in Osborne House, Isle of Wight, erected about 1850.

floating the surface of the rough concrete with cement and sand 1 in. thick. For a boarded finish, nailing fillets, 3 in. by 2 in., are embedded in the concrete, 14 in. or 16 in. centre to centre (Fig. 107). By this means combustion would be retarded, owing to there being no air-space round the boards, and no space for dust and dirt to accumulate.

An old system of fireproof floor is shown in Fig. 181, ordinary bricks being used to form the arch. When it first came into vogue, cast-iron girders were in use, and later, wrought-iron joists substituted. Fig. 181 represents a section of a floor in Osborne House, Isle of Wight, which was built by the late Thomas Cubitt in 1850, about the time cast-iron girders and brick arches were first used. In any system of arch construction, tie-rods are absolutely necessary; for unless the abutments of the arch can resist the thrust, it will spread and collapse. If each bay of a floor were always uniformly loaded, the thrusts would neutralise one another, and tie-rods would only be required in the end bays against the walls; but such a condition hardly occurs in practice. The tie-rods in floors are usually $\frac{3}{4}$-in. in diameter, and spaced 4 ft. to 6 ft. apart, according to the loads to be carried. The correct position for the rods is in the centre of the skew-back or springing, but this necessitates the rods showing through the soffit, and to avoid this they are, at the risk of faulty construction, often placed sufficiently high to be buried in the arch altogether. In order to secure the ends of tie-rods in walls, 3-in. by $\frac{1}{4}$-in. continuous bond-iron is sometimes built in the walls at the required level all over the building. Formerly, concrete was only put in the spandrils of the half-brick arches for additional

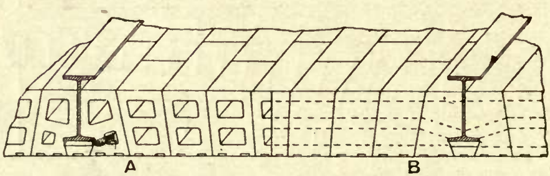

Fig. 182.—Section of Side and End Construction Hollow Terra-cotta Flat Arches.

strength, and the necessity of avoiding all air-space under the finished boarded floor—so as at least to retard combustion—either was not recognised or was avoided for fear of setting up dry rot in the timbers. Nowadays the concrete, if only very poor stuff, is levelled up above the crown of the arch, and the nailing fillets, with splayed sides, are embedded, and act as screeds for levelling the concrete surface. These fillets are usually placed transversely to the joists if the floor is of a double thickness; but if one thickness of boards only is laid, this cannot be done, owing to the desire to run the boards the long way of the room. These fillets are usually spaced 14 in. to 16 in. centres, and are $2\frac{1}{2}$ in. wide at the top, $3\frac{1}{2}$ in. at the bottom, and 2 in. thick. If the sides were not splayed they would work loose owing to shrinkage of the timber. The great failing of this type of construction (Fig. 181) is due to the unprotected state of the flanges of the beams. In order to obtain a level ceiling, binding joists were supported by the girders, and then ceiling joists to receive the lath and plaster ceiling were secured to them. Consequently, there was a large quantity of combustible material placed in proximity to the naked girder flanges and tie-rods.

Any type of fire-resisting construction should be judged not so much by its ability to carry heavy loads as with respect to the protection afforded the iron or steel members. A floor of light construction, which might be considered sufficient for dwelling-houses and hotels, where the quantity of inflammable material is not great, may be quite unsuited for a commercial building where large quantities of highly inflammable material are stored. The letter references in Fig. 181 are explained as follows: T binder, U ceiling joist, V sleeper, W floor joist, and X lathing.

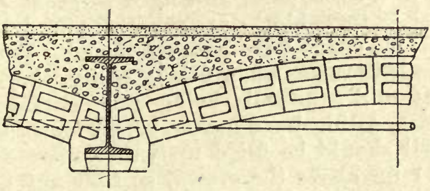

Fig. 183.—Section of Hollow Tile Segmental Arch.

Terra-cotta arches, both flat and segmental, are largely used in America, and they possess the great advantage of requiring only a small amount of water to be used in fixing them; the building consequently soon dries out. It is quite the contrary with solid concrete floors, or composite floors into the construction of which concrete largely enters. Fig. 182 shows both a side construction A and an end construction B flat arch. When the voids in the blocks run parallel to the joists, it is known as side construction; when at right angles to them, end construction. The end-construction arches are found to be nearly twice as strong as the side-construction arches, and will support as much as 15 cwt. to 20 cwt. per ft. super. before failure. The sides of the blocks are splayed parallel to the keystone or skew-back, and not radial, as would be theoretically correct. If the sides were radial, each block would be different, and the cost of manufacture would be prohibitive. The side-construction arches are more easily fixed than the end-construction arches, and take less mortar in the joints. To economise mortar, some floors have side-construction keystones and skewbacks, and end-construction fillers.

A safe rule for the depth of these arches is that the span in feet should not exceed two-thirds of the depth of the block in inches. Blocks are made up to 12 in. deep for side construction, with the maximum span of 7 ft. End-construction blocks are made up to 15 in. deep for 8-ft. spans. Six-inch, 7-in., and 8-in. blocks usually have one, 9-in., 10-in., 12-in. two, and 15-in. and 18-in. three or four interior webs.

For segmental arches, such as are shown in Fig. 183, side-construction blocks are employed. The blocks are made 4 in. to 8 in. square, and fixed to break joint. The maximum span these blocks are generally used for is 10 ft., but much wider spans have been employed. Porous blocks are preferred, because, when fitting badly, they are found to rub away and gradually bring more surface into contact, whereas solid, hard-burnt blocks chip and crack. The spandril should be filled up with concrete to a level of at least 1 in. above the crown, so that any concussions arising from falling bodies may be eased. The rise of the arch must be at least 1 in. per foot of span.

4 in. blocks are used for spans up to				8 ft.
6 in.	,,	,,	,,	16 ft.
8 in.	,,	,,	,,	20 ft.

The approximate thrust per lineal foot of brick arches can be found from the formula :

$$T = \frac{1{\cdot}5 \, W \, L^2}{R} \qquad \begin{aligned} W &= \text{load per sq. ft. in lb.} \\ R &= \text{rise in inches.} \\ L &= \text{span in feet.} \end{aligned}$$

For flat arches take one-half the depth of the arch as the rise.

CHAPTER XIII.

FIREPROOF PARTITIONS.

THE walls which serve to divide buildings from each other, and the interior walls and partitions serving to separate the rooms of a building, are usually of brickwork or wood. Wood partitions may be dismissed at once as a most unsafe form of construction with respect to danger from fire, although something may be done to minimise the danger by the use of special preservative paints or by other methods detailed below. Brickwork affords the greatest measure of safety, but in city buildings, where land is costly, recourse may be had to plaster partitions of various descriptions on a core of wire-mesh or expanded metal, by which means a considerable saving of space may be effected.

It is an essential feature in all party walls that no openings of any kind shall exist in them; that is, there shall be no doorways or window openings, or apertures of any kind, so that, if an outbreak of fire occurred, it would be confined to one building and not communicated to the next. In many towns it is insisted in the bye-laws that party walls shall be carried up through the roof covering, usually to a height of at least 15 in., measured at right angles to the roof covering, and also that they shall be corbelled out at the back and front of the building so as to project beyond the gutters and fascias. This is an excellent measure for the prevention of the spread of fire. Where the party walls are not carried up in this way, there is a danger of the flames being blown over the adjoining roofs, splitting the slates, and exposing the roof timbers to the action of the fire. Other local authorities are content with party walls that are carried up to the underside of the slates; while in other cases there is no restriction on the height, and consequently rows of cottages are built with the party walls extending only to the height of the uppermost ceilings, while the space between the ceilings and the roof timbers form a continuous tunnel along the whole row of buildings. This, it is scarcely necessary to point out, renders it almost inevitable that a serious fire in any one house will be followed by the destruction of the entire row. Bond timbers, plates, blocks, or plugs of wood should not be allowed in party walls, all beams and joists whose ends abut upon them being carried by brick corbels.

In addition to the carrying up of party walls in the manner described above, it is sometimes ordered that the external walls, when coming within 15 ft. of any other building, shall be carried up to the height of at least 12 in. above the adjoining roof or gutter, so as to form

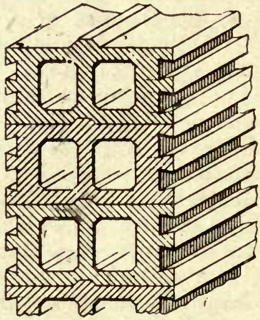

Fig. 184.—Lindsay's Cellular Bricks.

a parapet wall. In connection with the woodwork attached to interior walls, in the shape of skirtings and architraves, it should be pointed out that these form a means of carrying the flames from one part of the building to another, and in many cases they might with advantage be replaced by cement, especially in the case of skirtings.

In order to lessen the weight of partitions on upper floors, and so reduce the size of the girders required to carry them, bricks of porous terra-cotta are sometimes used. These are made from clay which has been mixed with granular, combustible substances, such as sawdust. The sawdust is consumed in the firing process, leaving the burnt clay thoroughly honeycombed with air cells and spaces. It is of such a consistency that nails and screws can be driven into it, and at the same time it weighs only about half as much as ordinary brickwork. Various forms of hollow or cellular bricks and terra-cotta blocks have also been devised with the view of lessening the weight of such partitions. Fig. 184 illustrates one such system, known as Lindsay's.

Internal walls formed with a wood framing may be made fire-resisting by the application of some good plastering, and this method used to be in general use in the houses of Paris. The method of construction, as described in Bree's "Glossary of Architecture," was as follows: "Upon the partition being framed, the spaces between the quarters were filled in with rough stone-rubble walling, and strong oak batten laths, from 2 in. to 3 in. wide, were nailed up to the quarters horizontally, at distances of 4 in. to 8 in. apart, holding up the stone and preventing it falling out. The next operation consisted in spreading a strong mortar, principally of plaster-of-Paris, over each face of the partition, completely covering the quartering and laths, and being pressed through from one side to the other. The rubble was completely embedded, and became one mass with the plaster, possessing the stiffness and strength of timber with the fireproof qualities of the plaster.

Wooden partitions can only be looked upon as in any sense fire-resisting when they are of such a massive construction and thickness as to put them almost out of the bounds of practicability, or when they are protected by some such preservative as asbestos paint or some of the processes mentioned in a previous chapter when discussing woodwork (Chapter X.). There are practical difficulties which prevent the general use of chemical solutions impregnated into the structure of the wood. For example, the finished woodwork would be damaged in the process, while if the unwrought wood is prepared in bulk it is rendered more difficult to work, and much of the cost of preparation is thrown away on wood which has afterwards to be cut to waste. Of the preservative paints, most are composed of asbestos, mica, borax, soluble phosphates, and tungstates, and similar materials, suspended in or mixed with a solution of silicate of potash or soda. In all such compounds, however, there is a great tendency to blister and flake off on exposure to the weather. Mixtures of silicates of lime and magnesia; of asbestos, chalk, and soluble silicates; of powdered glass, stone, lime, and silicate of soda; of gelatinous hydrate of alumina, with an alkaline silicate, have all been patented as useful for fireproofing and preserving wood. It was also claimed for petrifite (which does not as yet seem to have been manufactured on a commercial scale) that when it was applied to woodwork the latter was rendered incombustible.

A fireproof covering for partition walls, cornices, and ceilings can be obtained by the use of materials applied in the form of loose slabs. One of the most popular of these is asbestic, a bye-product from the manufacture of

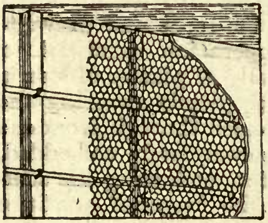

Fig. 185.—Johnson's Wire Lathing.

Fig. 186.—Jhilmil Lathing.

asbestos. It is used as a plaster, and, being fibrous and elastic, it does not chip nor crumble away, and nails can be driven into it freely. It is made up, amongst other forms, in slabs ¼ in. thick, and it is claimed for these that they can be worked as easily as wood. Salamander is another somewhat similar preparation, having asbestos as a constituent, but is mostly used in the form of embossed slabs of a highly decorative character. Petrifite was also used to form slabs coloured and grained to imitate marble and granite.

Plastering executed on the ordinary wooden laths is readily consumed by fire. Various metallic substitutes have therefore been devised to take the place of laths, and amongst the earliest of these must be counted the invention of Mr. L. Leconte in 1841 for using wire netting in this way, although it is said the invention was not a novelty at the time. The system known as Johnson's patent wire lathing is generally used at the present time. It is illustrated in Fig. 185, and consists of galvanised wire netting of $\frac{3}{4}$ in. or smaller meshes stretched over strips of varnished hoop-iron about $\frac{5}{8}$ in. wide. The hoop-iron strips are secured horizontally to upright wooden posts by staples, and are spaced from 6 in. to 9 in. apart. As will be seen from the illustration, the wire netting is spread on both sides of the wooden uprights. In 1889 a large fire occurred at a dye works at Manchester where this system of lathing had been applied, and the superintendent of the fire brigade reported that it stood the test of a fierce fire and of water satisfactorily, the damage being very slight.

Later applications of the idea of supplying a metallic framework for the support of the plaster are jhilmil and expanded metal. The first of these is shown in Fig. 186. It is formed by cutting a series of slits in a thin sheet of iron or steel, and forcing the strips left between the slits in alternate rows upwards and downwards. This affords a good key for the plaster.

An invention, which in general principle is very similar to the above, but one which possesses many important advantages over it, is the now well-known "expanded metal" invented by Mr. Golding, of Chicago. The peculiar feature of this article lies in its method of manufacture—plain sheets of rolled metal (steel for building purposes) are cut and expanded by machinery into meshes of a diamond shape, the cutting and expanding being done in the one process. The expansion effected in the manufacture is from two to twelve times the original width of the sheets, according to the size of the meshes and width of the strands formed. In practice the sheets used range from 24 B.W.G. to 4 inches plate, and the meshes are distinguished by the dimension across the shortway of the diamond, such as $\frac{3}{16}$ in., $\frac{1}{4}$ in., $\frac{3}{8}$ in., $\frac{3}{4}$ in., $1\frac{1}{2}$ in., 3 in., and 6 in. The strands of the several meshes are varied in section to give a variation in the weight or strength of the material, so that the various requirements of building construction may be met.

In Fig. 187 is shown the mode of applying expanded metal to the formation of a fireproof partition in a build-

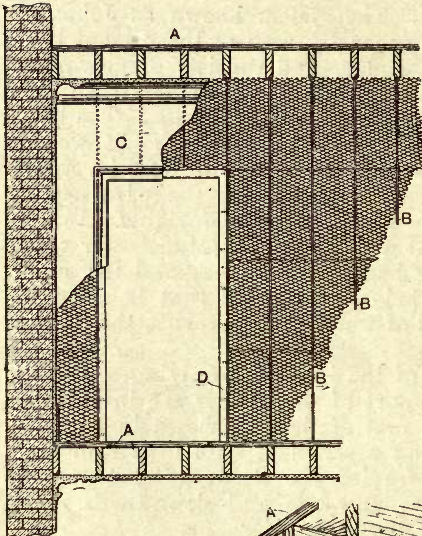

Fig. 187.—Expanded Metal Lathing Partition : Elevation.

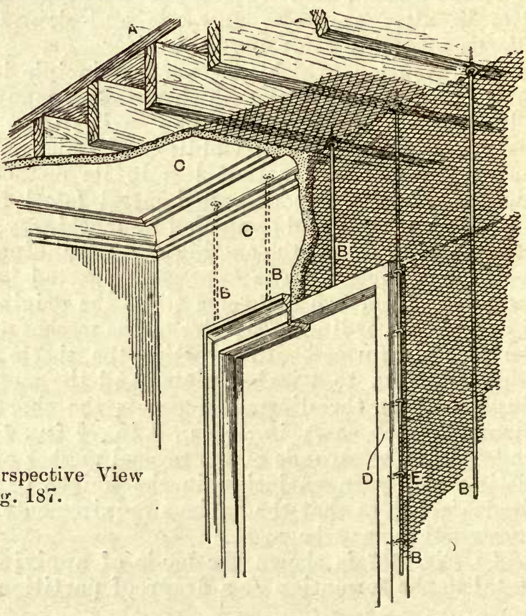

Fig. 188.—Perspective View of Fig. 187.

ing where the floors A are carried on wooden joists. Vertical tension rods B, of $\frac{3}{8}$ in. diameter, are stretched tightly between the joists of one floor and those of the floor above, being secured to them by staples and screw-eyes. A tension rod being fixed to each joist, they will, in most cases, be 12 in. or 15 in. apart. The sheets of expanded metal, about 2 ft. in height and 6 ft. or 8 ft. in length, are laced to the tension rods, and the plaster C is then applied to both

Fig. 189.—Sectional Plan of Partition and Door Jamb.

in front of them, and the plaster C is then applied to both sides, forming a partition wall 2 in. or $2\frac{1}{2}$ in. in thickness. A perspective view of the same partition is shown in

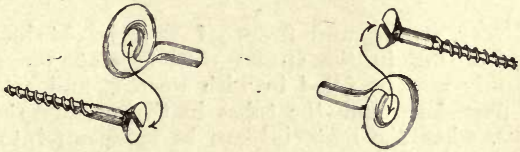

Fig. 190.—Details of Tension Rods and Fixings.

Fig. 188, and will serve to make clear the method of supporting the door frame D by staples E to the tension rods; also the method of applying the lathing to the ceiling.

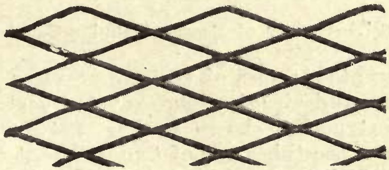

Fig. 191.—Enlarged View of Expanded Metal Lathing.

Fig. 189 is an enlarged sectional plan, showing the interlacing of the expanded metal between the tension rods; and Fig. 190 illustrates the eyes and screws used for fastening the tension rods at top and bottom. The peculiar structure of the expanded metal itself is shown in Fig. 191.

A curious and decidedly ingenious form of partition wall has been introduced under the name of Pease's Tubular Construction. Split tubes are interlocked into each other by sliding them lengthways, one tube through the slit of another, until they are assembled together in the fashion shown in Fig. 192, which is a plan of the ends of the tubes. By filling up the tubes with cement or fine concrete, they are fixed in position,

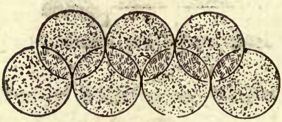

Fig. 192.

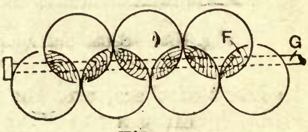

Fig. 193.

Figs. 192 and 193.—Varieties of Pease's Tubular Construction.

and a very strong and fireproof wall is obtained. Instead of filling in the spaces with cement, the smaller spaces only may be filled in with wood F, and a long tie bolt G passed through the tubes in the manner shown in Fig. 193, when the material can be made up into sheets of a convenient size for making portable buildings. It will be noticed in both the above cases that the surface of

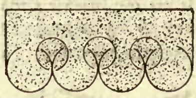

Fig. 194.—Variety of Pease's Tubular Construction.

the wall presents a series of vertical crevices, which would afford a very undesirable lodgment for dirt. To obviate this, the construction shown in Fig. 194 has been recommended, where the tubes are of two different sizes; and, in addition to filling the interior with cement, it is the practice to plaster over one or both of the faces. Of the strength and fire-resisting properties of this method of construction there can be no doubt.

CHAPTER XIV.

FIREPROOF STAIRS, ROOFS, AND CEILINGS.

Stairs.—In spite of the fact that in case of fire the stairs afford the only means of escape from the upper floors of most houses, this part of the building is usually so constructed that it is most easily destroyed by fire. When treads, risers, strings, and balusters are all of wood, and very often with the space underneath the stairs enclosed by matchboarding to form a cupboard where boxes and other inflammable articles are stored, the destruction of the whole is almost a certainty should the fire once obtain a hold of the lower part of the stairs. In buildings of the warehouse class the state of things is much better, but in dwelling-houses the usual construction is one which is far from desirable. As this fact has been pointed out, certainly for about sixty years, by various architects and engineers, there is little hope of improvement until legal restrictions are placed upon builders. A more fire-resisting construction would necessarily cost more, but even where cost is of little consideration, as in the case of large mansions, it is not unusual to find stairs of pitch pine, highly varnished, and, of course, very inflammable. Stone stairs are certainly safer than those of wood, but, as has been pointed out, stone is unreliable under the action of heat, and it is apt to split.

For use in dwellings there would appear to be nothing better than good thick oak or teak, for these woods, although liable to be destroyed in a protracted fire, will last long enough to afford time for the escape of the inmates.

Concrete, especially in the superior forms of granolithic and the allied mixtures, affords a very good material. Ordinary Portland cement concrete, with the aggregate crushed to $\frac{3}{8}$ in. or $\frac{1}{2}$ in. gauge, may be used, and it is advisable to insert one iron bar in each step. It is also well to have two or three bars of iron embedded in the concrete through the length of the stairs, that is, from the top to the bottom. The treads, if left smooth, will be found to be too slippery; they are therefore usually fluted with a roller, or sprinkled with some fine hard facing spar, which should be rolled into the concrete.

Some exceedingly fine stairs have been constructed with granolithic in large public buildings. In the technical schools, Birmingham, a building seven storeys high, a winding stair runs from the top to the bottom of the building, the steps being 8 ft. 6½ in. long, and what is known as self-supporting, or built into the wall at one end only. In the Manchester technical school also this material is used for the stairs and landings, the steps being in some cases 13 ft. long.

Roofs and Ceilings.—The ease with which an ordinary slated roof is attacked by fire from the outside is not sufficiently recognised by most people. While such roof coverings as thatch or tarred felt are obviously combustible, it may be thought that slates afford a fair amount of protection; but the fact is that slates, when attacked by the flames, split and fall out of position, exposing a mass of woodwork of small scantlings, which are easily burnt; from its position the woodwork of the roof is naturally very dry and combustible. In many districts it is customary to erect rows of houses with the party walls carried up only to the height of the ceilings of the top floors, so that the space between the ceilings and the slating forms one continuous chamber over all the houses; in such a case it is obvious that a fire which penetrates to the roof on one house will easily spread through all the rest. The remedy is found in a bye-law compelling all party walls to be carried up through the slating to a specified height above the roof, or, at all events, close up to the underside of the slates. If the roofs of some of our finest public buildings were examined, it would be found that they contain masses of woodwork, which are a great source of danger. The recent destruction of the Colston Hall at Bristol may be referred to in this connection; and it was not very many years ago that no less than 20 tons of light, inflammable deal planking was removed from the roof space of the House of Lords.

Before describing some of the modern systems of fire-resisting roofs, one curious attempt to solve the difficulty deserves notice. Early in the present century, Mr. Edward Cresy, best known as the author of the "Encyclopædia of Civil Engineering," designed and erected some labourers' cottages of the eccentric form shown in Fig. 195, no woodwork whatever being used in their construction except for the doors. The windows had iron frames, and the floors were of brick arches and cement; but their chief peculiarity was that the walls were carried up in the form of a catenarian arch, meeting at the top, and so forming a brickwork roof. The walls were 9 in. thick, and were covered all over with tiles set in cement. This experiment does not appear to have been repeated by any architect.

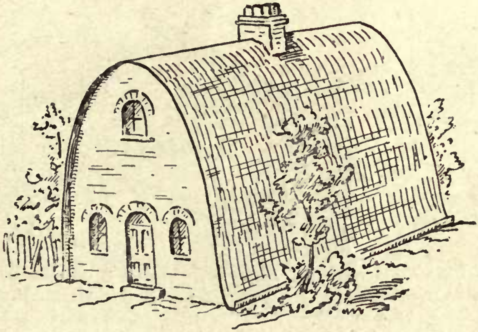

Fig. 195.—Fireproof Cottage with Brickwork Roof.

The modern fire-resisting roof is usually flat, formed either with wooden beams boarded and covered with lead, zinc, or some other protective covering, or it is made of concrete similarly to some of the floors described in Chapter XII., but of lighter construction, as it has not the same load to sustain. Flat roofs are useful as playgrounds for children, as drying grounds for laundry work, or, in the case of hospitals and infirmaries, as promenades for convalescent patients, and it is probable that they will be more generally adopted in the future than they are at present. There is some little difficulty in making them water-tight. As concrete alone cannot be depended upon to prevent the rain soaking through, it is usual to cover the roof entirely with asphalt—not the builder's so-called "asphalt" of pitch, tar, and cinders, but a mixture of, approximately, 10 per cent. of bitumen and 90 per cent. of carbonate of lime. It may be pointed out, by the way, that this has been legally decided to be incombustible. Fig. 196 shows how the asphalt is applied in two unbroken layers of ½ in. thick over a flat concrete roof. A skirting is formed at the parapet wall surrounding the roof, and the asphalt is carried up two or three courses of bricks and tucked into a joint of the brickwork. Water gutters must be formed at an inclination of not less than 1 in 40 to carry off the rain.

Vulcanite is used in the same way as the above. It has been recently stated that paper and vulcanite or wood

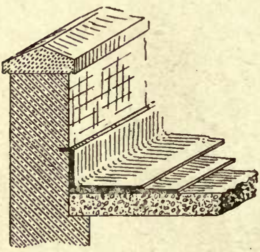

Fig. 196.—Flat Roof with Asphalt Covering.

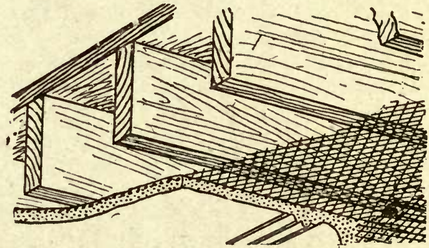

Fig. 197.—Ceiling of Expanded Metal under Wooden Joists.

cement roofs, which are generally employed on the Continent, provide an efficient weather- and fire-proof roof covering when properly laid and suitably covered with earth and pebbles.

In public buildings domed roofs have been constructed by interlacing light bars between arched ribs and then embedding the whole in concrete in the manner described in the chapter on fire-proof floors already alluded to. Wooden fillets are embedded in the upper surface of the concrete to receive the slating laths, while the soffit is plastered to form the domed ceiling.

When concrete floors are used, ceilings are dispensed with, the under surface of the floor being finished smooth, and plastered to serve as the ceiling. In floors constructed of cellular blocks of terra-cotta or fireclay, grooves are left to form a key for the plastering, as shown in the before-mentioned chapter on fireproof floors. Wooden floors may be protected on the underside by a ceiling formed of fireproof slabs nailed to, or suspended from, the underside of the joists. Tiles may also be used for this purpose, but are more difficult to fix securely than slabs of silicate cotton or some of the asbestos compounds, which may be obtained in convenient sizes and in artistic designs. The silicate cotton is usually in the form of slabs made by enclosing the material between sheets of wire netting, and it forms an almost indestructible protection. Asbestos may be obtained in various forms, either

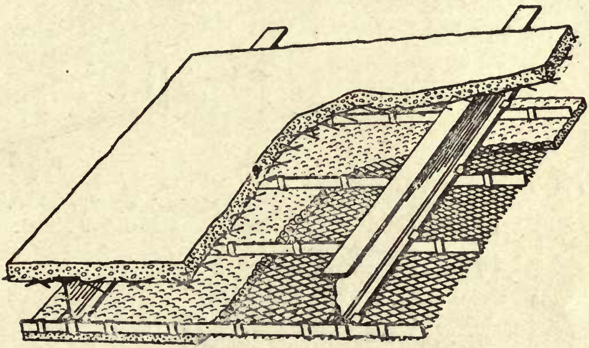

Fig. 198.—Ceiling of Expanded Metal under Steel Joists.

as plain sheets of millboard or in the more elaborate forms of Salamander and such like preparations, embossed with highly artistic patterns.

Expanded metal, which has been before referred to in connection with fireproof floors and partitions, can be readily utilised for forming ceilings. Applied to a wooden floor, the expanded metal is fastened to the underside of the joists by nails or staples, and is plastered over, the peculiar formation of the meshes affording an excellent key for the plaster. This construction is shown in Fig. 197. When used in conjunction with steel joists the construction is similar to that shown in Fig. 198. Flat strips of iron, known as ceiling bars, are suspended to the bottom flanges of the steel joists by steel clips of the shape shown in Fig. 199, where the end marked A is bent down after the clip has been slipped into its place. Hangers are used to support the ceiling bars, and clips or

Fig. 199.—Clip for Fixing Ceiling Bars under Steel Joists.

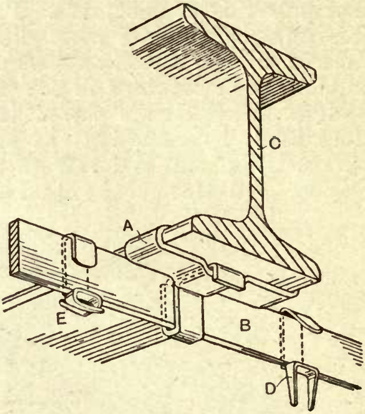

Fig. 200.—Method of Fixing Ceiling Bars for Expanded Metal.

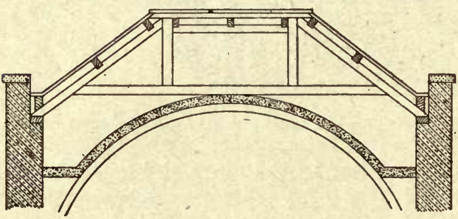

Fig. 201.—Domed Ceiling at the Foreign Office, London.

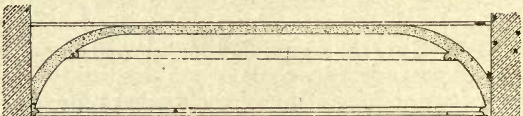

Fig. 202.—Domed Ceiling at His Majesty's Theatre, London.

wire to attach the expanded metal to the ceiling bars, and are shown in the enlarged detail at Fig. 200, where

B indicates the ceiling bar and C the floor joist, one lathing clip D being shown open, and the other one E as it appears when bent up to retain the meshing. Johnson's metallic lathing and Jhilmil are also similar materials to expanded metal (see page 138).

Figs. 201 and 202 show examples of large domed ceilings in public buildings, specially constructed to protect the woodwork of the roof. The first of these shows the ceiling over the Cabinet Council Room of the Foreign Office. It is of concrete, having gypsum as a base, and the dome is 36 ft. in diameter, with a thickness of 9 in. at the haunches. The second example, constructed in a similar way, is the ceiling over a dressing-room at His Majesty's Theatre, London. The dome is 30 ft. long, 20 ft. wide, and has a rise of 5 ft.

CHAPTER XV.

FIREPROOF CURTAINS, DOORS, AND WINDOWS.

Curtains.—Although the use of fireproof curtains is confined to theatres and is consequently of limited interest, a few notes on some of the various forms of these structures may be found useful. The curtain is used to fill up the opening of the proscenium so as to divide the auditorium from the stage, where the greater liability to fire exists.

The simplest kind of fireproof curtain is made of two thicknesses of sheet iron, or steel, attached to an iron framework, the space between the inner and outer sheets being either left empty or filled with some non-conducting substance, such as silicated cotton or asbestos fibre. The curtain is therefore a heavy structure, and has to be counterbalanced in order that it may be easily raised and lowered. Combined with water sprinklers to keep it cool, it is an efficient enough arrangement, but is objectionable on account of its extreme weight and the difficulty of handling it speedily. To overcome these defects as far as possible, Mr. Max Clarke introduced a light iron framework covered with silicate of cotton, which was attached to the framework by wire netting, and covered on the outer side with canvas painted as a drop scene.

Various forms of curtains have also been devised with asbestos cloth stretched on a framework of iron, one of the latest of these being at "Olympia," where four large screens made of Danville asbestic, each measuring 130 ft. by 90 ft., have been erected.

A curtain introduced by Captain Heath was made of canvas with a backing of spongy asbestos, and designed to be kept wound on a roller below the level of the stage. Beneath this roller was a long trough of water, containing another roller, under which the curtain passed when being unrolled. To the top of the curtain was attached a counterbalance weight which, being released on an alarm of fire, pulled the curtain upwards over the proscenium opening, the curtain being saturated with water when it arrived in position.

Another novel form of curtain was patented by Mr. E. W. Stead, and introduced at the Theatre Royal, Halifax. It is composed of two sheets of iron $\frac{1}{10}$ in. thick, and placed 3 in. apart. The curtain is divided horizontally into two parts, the top half being moved upwards and the lower half downwards. The upper half is the heavier, and is connected by steel ropes and pulleys to

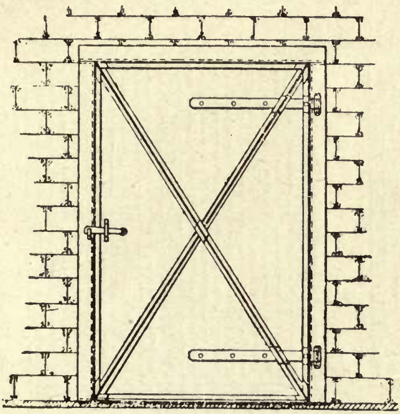

Fig. 203.

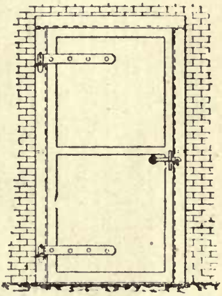

Fig. 204.

Figs. 203 and 204.—Sheet-iron Doors.

the lower half in such a way that its weight serves to keep the curtain closed. When it is desired to open the curtain, water is let into a tank attached to the lower half until it is heavy enough to descend, pulling up the upper half. In case of fire, the outlet valve of this tank can be opened, discharging the water, and closing the curtain in about thirty seconds. The proscenium opening measures 21 ft. 6 in. by 21 ft. 4 in., the top curtain weighing 38 cwt., and the bottom curtain and tank 35 cwt.

Doors.—The requirements of a fireproof door are: that it shall be capable of withstanding great heat, shall completely fill the opening in which it is fixed, and shall be closed easily, preferably by automatic means. Such doors are usually made of sheet iron riveted to a framework of angle iron and T-iron, or, in some cases, to thicker strips of flat iron, used as stiffeners. Fig. 203 is an illustration of a simple sheet-iron door with the edges strengthened by angle irons, riveted on, and T-iron diagonals. The door frame is of cast-iron for building into a wall. Fig. 204 shows a door stiffened by flat bands. This door would be more easily warped under the influence of heat than that shown in Fig. 203. Doors of either of these descriptions should be provided in duplicate—

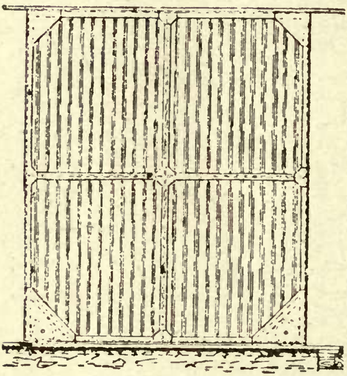

Fig. 205.

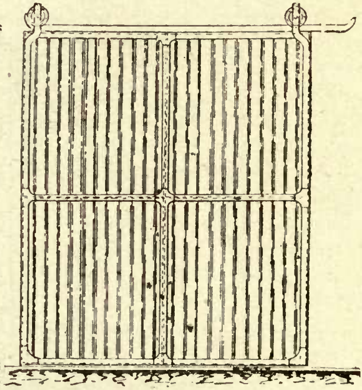

Fig. 206.

Figs. 205 and 206.—Corrugated Iron Doors.

that is, one fixed on each face of the wall, separated by a space of the thickness of the wall. In case of fire, one door may be heated to redness, and even considerably warped, without the other one suffering any damage. To decrease the weight, corrugated iron is sometimes employed, especially in large sliding doors such as those shown in Figs. 205 and 206, where the frame is of angle and T-iron and the doors slide on rollers—in the one case at the top, and in the other case at the bottom, of the door.

In all fireproof doors that are hung on hinges special care must be taken to insure their security by fastening them with strong bolts, both to the doors and to the walls or frames. The latches, being also subject to heavy wear, should be strong.

One of the most efficient kinds of fireproof door is made of wood encased in sheet steel. Two thicknesses of tongued and grooved boards, preferably of oak, cross each other diagonally, and are nailed together. They are then entirely covered with thin sheet steel, which completely protects the wood from contact with air; by which means, however great may be the heat, the wood cannot burn, and immunity from warping is attained.

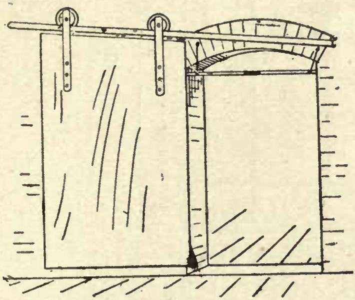

Fig. 207.—Sliding Door on Inclined Guides.

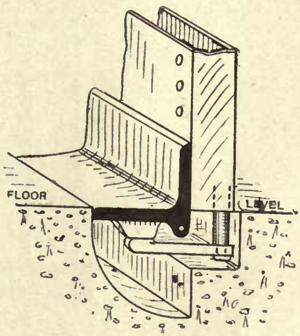

Fig. 208.—Arrangement for Automatically Covering Crevices at Bottom of Door.

Doors intended for strong-rooms are usually both fireproof and burglar-resisting, and are of the nature of safe doors. They are of double construction, hung in iron frames, the intervening spaces between being generally filled with some non-conducting substance.

In the case of sliding doors, provision should be made for closing up, as far as possible, the spaces left between the edges of the door and the wall. The top, and one edge of the door, are often made to fit into channel-iron grooves. Where a groove is also formed in the floor for the bottom of the door to run in, it is advisable to have a strip of wood to fit into it, and so protect it from being accidentally clogged with dirt. This bottom groove may be dispensed with by hanging the door on a sloping guide, as shown in Fig. 207. When the door is pushed back, it rises clear of the floor; when closed, it rests upon the floor, and forms a tight joint. With hinged doors, or doors hung on pintles, good joints round the two sides and the top can be easily insured, as the door is usually fixed in an iron frame. At the bottom, however, a certain amount of clearance has to be given to allow the door to open freely, and in order to close this crevice, the ingenious device shown at Fig. 208 is sometimes employed. It consists of an angular closing

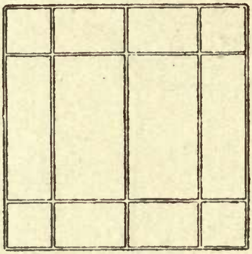

Fig. 209.—Cast-iron Window Sash.

piece hinged on pins at each end, and, when the door is open, lies with one face flush with the surface of the floor. The lower pintle of the door carries an arm, which is fixed at right angles to the face of the door in such a manner that when the door is closed tnis projecting arm pushes up the angular closing piece into the position shown in the illustration, and the space below the door is effectually guarded.

Arrangements for automatically closing doors are very desirable. One of the simplest and best systems applicable to sliding doors depends upon a fusible alloy melting at a temperature of 160° F. The door is hung upon an inclined guide, and is prevented from closing by means of a round stick about 1 in. in diameter, which reaches across from one edge of the door to the opposite side of the door frame, as shown in Fig. 207. This stick is divided diagonally at the centre, and a ferrule, made of two pieces of copper soldered together longitudinally with the fusible alloy, covers the joint in the stick. When this ferrule is exposed to a temperature of 160° F., it yields and splits open, causing the stick to separate, and the door to close by its own weight. To prevent the stick from falling in the way of the door, and to allow of the latter being shut at any time, the stick is connected to the top of the door frame by small chains at each end. Various other arrangements of wires or chains to hold back the doors, or with counter-weights to close the doors where

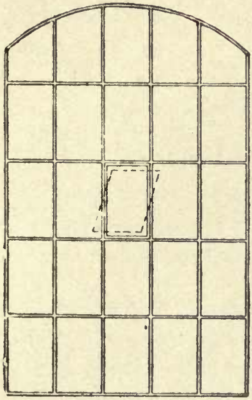

Fig. 210.

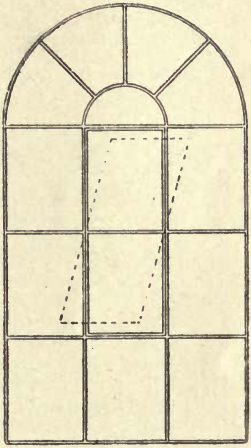

Fig. 211.

Figs. 210 and 211.—Cast-iron Window Sashes.

they are not hung on inclined guides, have been used, in which the melting of a specially provided portion of the wire or chain causes the doors to close automatically.

Windows.—Up to the present time little attention seems to have been given to the protection of window openings, which consequently offer a ready means of spreading a fire. Iron shutters of simple construction, and so arranged as to be easily closed, afford an efficient protection; revolving shutters, such as are used for shop fronts, are also good if kept in working order. Cast-iron window frames and sashes, such as those shown in Figs. 209 to 211, are in regular use, but, although fireproof, they do not prevent the breaking of the glass under intense heat, and the consequent passage of the flames. It should be noted that when the panes of these iron sashes are small, it is impossible to rescue anyone by way of the window, and cases have occurred where lives have been lost in this way. Glass is now made containing a wire mesh embedded in its substance, and is said to stand under great heat without falling to pieces.

INDEX.

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