DISCUSSION.

Mr. J. I. Haycroft said that as the author concluded his paper, which evidently had been compiled with care, by inviting members to give further information bearing on this branch of engineering, he would supplement the particulars given, as regarded fire-proof floors, by bringing under our notice a system known on the Continent of Europe as the "Monier system." He quite agreed with the author that the system of brick arches and concrete between rolled beams, with the tie rods exposed, was faulty in the extreme. There was, however, no necessity to have the tie-rods, or indeed any portion of the iron work, exposed. The trough system of flooring should be styled as "uninflammable" rather than fire-proof. It was, however, a very strong and stiff form of flooring, and, as such, was extensively used in the decking of bridges. Among other systems of fire-proof flooring, not mentioned by the author, might be cited Whichcord's, in which the rolled beams were encased in fire-clay blocks, the backs of these forming skew-backs, on which brick arches were turned. This system was fire-proof, but was expensive owing to the dead weight carried.

Fox and Barrett's system consisted of placing the rolled joists close together, from 1 ft. 6 in. to 2 ft. apart. On the bottom flange of the joists sawn timber battens were placed about half an inch apart, concrete being filled in between the joists which protruded between the battens, and thus formed a key for the ceiling plaster.

Homan's patent was an improvement on Fox and Barrett's system, the rolled joists being placed further apart, from 3 to 4 feet, the timber battens being replaced by T iron fillets, 9 in, apart; this necessitated temporary staging to support the concrete between the fillets until sufficiently set to be plastered. A floor constructed on this system was very strong and rigid.

Allen's system consisted of concrete, strengthened by iron bars; the bars were about 3 in. by 1 in., placed on edge, across the building, about 2 ft. apart, and built into the walls on either side; across these bars were placed half-inch iron rods, also 2 ft. apart, thus forming a network with meshes 2 ft. square A temporary staging was placed slightly below this network, and concrete filled in to from 4 to 6 in. in depth. Allen's system had not been extensively used, and only occurred in the form of floors.

The Monier system, as seen by Fig. 6, Plate XXXV., was somewhat similar to Allen's, insomuch as it consisted of a network of iron rods encased in cement mortar, not concrete.

Perhaps it would be as well before describing the many uses to which this system was applied, to draw attention to Fig. 5, Plate XXXV. This represented an experimental arch, constructed of the materials shown in Fig, 6, Plate XXXV., viz: Longitudinal iron rods $\frac{5}{16}$ in. diameter, the transverse rods being a shade less, viz., $\frac{1}{4}$ in. diameter; the transverse rods rested on the longitudinals, and were merely kept from moving by a single strand of wire; the network so formed had meshes of about 4 in. square.

The cement mortar consisted of ordinary cement and sand in the proportion of 1 to 3. The arch in Fig. 5, Plate XXXV., was 26 feet span, with rise of 2 ft. 8 in., and was constructed 2 in. thick at the crown, and 5 in. at the springing, the skewbacks being rolled beams, built into solid concrete abutments.

The result of the uniformly distributed loading, on half the span, was shown in the following table, from which it would be seen that fracture took place under a loading of 7,800 lbs. (3½ tons), with a deflection at B of 1$\frac{3}{16}$ in.; the weight of the arch itself, unloaded, averaging 29 lb. per square foot.

Load in lbs. uniformly distributed.	Deflection.					Remarks.
	A	B	C	D	E	
	Inch.	Inch.	Inch.	Inch.	Inch.	
5550	$-\frac{5}{32}$	$-\frac{3}{16}$	0	$+\frac{1}{8}$	$+\frac{9}{64}$	
6660	$-\frac{7}{16}$	$-\frac{1}{2}$	$-\frac{5}{64}$	$+\frac{9}{16}$	$+\frac{5}{16}$	Fine cracks showing at most exposed parts
7800	$-\frac{29}{32}$	$-\frac{19}{16}$	$-\frac{7}{16}$	$+\frac{11}{16}$	$+\frac{15}{16}$	Fractured with this load.

Weight of arch unloaded, 29 lbs. per square foot.

Fig. 1, Plate XXXV., showed this system applied to form fire-proof flooring in the monumental buildings of the new Art Gallery, Copenhagen, where the arch was only 5 inches thick throughout, on a 25 ft. span; the tie rods were perfectly protected from fire; the materials over the arch might be terra cotta, lumber, or other uninflammable material, and was used merely as filling to support the flooring, which could be of tiles or other suitable material.

Fig. 2, Plate XXXV., was the Monier construction, applied to form a roof of 42 feet span, at the Hellerup Glassworks, Copenhagen The thickness at the crown was 4 in., the haunches and over the side walls being widened out to 12 in.

Fig. 3, Plate XXXV. showed the Monier construction used as floors in the laundry at St. John's Hospital, Copenhagen, the spans being 16 ft. 8 in. with a rise of 12 in., the material being 4 in. in thickness.

Fig. 4, Plate XXXV, showed another phase of the system where the rolled beams were 5 ft. apart, and the construction was horizontal.

At the Bremen Exhibition, of 1890, a bridge of 130 ft. span, for pedestrian trafic, was completed inside of six weeks, the arch itself being completed inside of thirty-six hours. This system was very general in its application, and it had been extensively used for storage reservoirs for water and gas, also for the entire construction of houses, and for large sewer pipes. Flags or plates of various sizes, made on the Monier system, were also extensively used for footpaths, and for walls of buildings, the Diorama at Leipzig absorbing 80,000 of them. The Monier construction had also been used in the construction of breakwaters, the iron being protected from oxidation. rendered it very suitable for such a purpose.

Numerous experiments had been made as to the fire resisting properties of materials constructed on the Monier system, with complete success; after being submitted to a temperature of 1,960 Farh., a Monier pipe was found uninjured, as regarded strength or shape. One item contributing to the strength of structures, on this system, was the fact that the co-efficient of expansion for cement and iron were practically identical, thus precluding the possibility of fracture, which was always experienced in the case of a combination of brickwork and iron. As the author's paper was confined to building construction, he (the speaker) would not dwell on the applicability of this system to bridge construction, beyond expressing an opinion that it was but a matter of time when the Monier system would completely do away with the use of timber bridges, the latter being so costly in repairs, whilst the Monier system was practically everlasting.

Mr. A. M. Howarth considered the subject of fire-proof construction was steadily growing in importance. The need of fire-proof buildings in the business quarters of our great cities had been well demonstrated, and their superiority had become so generally recognised that at present but few structures of any size and importance were designed which were not more or less of this type. This change had been facilitated to no small extent by a number of signal improvements made of late in this type of building construction, ensuring not only a much higher degree of security, but considerable reduction in cost compared with methods formerly practised.

Steel columns and beams clothed with fire-proof materials were gradually and effectively replacing the older designs of cast iron and timber. He did not propose to criticise the use of cast iron for either columns or bearers, but would proceed to describe what he considered to be some of the best forms in which we might use steel in building construction.

As the author of the paper had not referred to the use of steel in foundation work, he would like to show that this was a matter of great importance. Of course in Sydney usually it was not a very difficult matter to secure a good natural foundation owing to the prevalence of good rock, or hard clays and shales at shallow depths. But these excellent conditions did not always exist, and in designing the foundations of walls and piers of buildings to rest upon a yielding stratum, proper provision must be made for the uniform distribution of the weight. In case the walls were of different heights, thicknesses, and live loadings, the width of the foundation must be proportioned according to the resultant varying total loads, so that the bearing unit of ground area would be equal, and a uniform settlement of the building thereby ensured.

The use of timber beams embedded in concrete as a means of obtaining wider bearing surfaces was to be condemned, unless the wood was in a position to remain constantly moist. Where this was not the case the timber, being liable to dry rot, would therefore decay, and thereby cause destruction and uneven settlements in the foundations. Old iron and steel rails had been used in lieu of timber; as they offered, however, little resistance to deflection, if allowed to project beyond the masonry to any considerable distance the concrete bedding was liable to crack, and therefore impair the solidity of the foundation. Steel I beams, as extensively used in America, were found to be superior to rails in every respect. A greater depth could be adopted, and deflection thereby reduced to a minimum, and a sufficient saving thereby effected to more than compensate for their additional cost per lineal foot of wall foundation.

The column which appeared to offer advantages superior to any other iron or steel column was that one known as the Z bar and plate pattern, Fig. 1, Plate XXXVI. Its claims for superiority were based mainly on the following qualities:—

1st.—Economy of material. The bars were furnished at a reduced cost compared with channels, I beams or patented special sections.

2nd.—For ordinary loads, economy of construction was secured in using two lines of rivets where four or six lines were required in any other type. Heavy loading, of course, would require six lines of rivets if outside plates were used; but, under most circumstances, the additional strength could be secured by merely thickening the bars and web plates, and thereby using two lines of rivets as before.

3rd.—High ultimate resistance to compression. Careful tests made upon full size specimens were detailed in a report to the American Society of Civil Engineers, April, 1888, by C. L. Strobel. The results for lengths ranging from 64 to 88 radii showed an average ultimate resistance per square inch of 35,650 lbs. These results were quite as favourable as those obtained for closed hexagonal, octagonal, or circular columns.

4th.—Great adaptability for effecting connection of columns to floor beams and girders. This quality was of great importance in keeping down the cost of manufacture and erection.

If it was intended to circulate a stream of water with in the column during a fire, the diaphragm plate would allow this to be done in the manner described by the author.

The old method of constructing fire-proof floors was by means of brick arches, whose rise would usually average about 1/20 of the span. Moderately large spans, when heavily loaded, produced horizontal thrust of large value. The amount of thrust was easily obtained by the formula—

$$T = \frac{1{\cdot}5\ W\ L^2}{R}$$

W = load per foot; R = rise in inches; and L^2 = square of span in feet.

Corrugated iron or "traegerwellbech," when used for fire proofing, was generally exposed below to form the ceiling, and it was thus open to the objection that the moisture in the atmosphere might condense upon the cool surface of the iron, and drop in sufficient quantities to injure the goods stored beneath it.

The modern types of fire-proof floor construction which had grown most rapidly in favour were those described by the author, and which left little more to desire unless it was in the direction of economy in cost of material. Two other types not mentioned were shown in Figs. 4, 5 and 6, Plate XXXVI.

With reference to the statement as to the relative values of iron bark and iron compared to their respective weights, &c., he said "that iron or wood, whether used as columns or girders, are pretty nearly equal in weight for equal strength." He (the speaker) might not have understood the sentence in the manner that the author desired, but he would explain his own idea of the relative strengths of steel and timber members of equal weight.

For example, given a column of ironbark 20 ft. x 12 in. x 12 in., and a steel Z bar column 20 ft. x 9 in. x 9 in. x $\frac{3}{8}$ in., we should find that the weights were alike, viz., 75 lbs. per lineal foot. To use either of these columns in a building where frequent and large alterations of loading were constantly taking place, it was self-evident that the load should in no case cause such an elastic shortening of the columns as would induce a gradual disintegration of the floors, walls and roof. The modulii of elasticity of steel and ironbark were as 11 to 1, or 35,000,000 lbs. and 3,2000;000 lbs. respectively, The crippling strength of the steel column was 356 tons, and $\frac{2}{7}$ of this was a safe working load of 102 tons. The working unit per square inch of material $= \frac{102}{20}$ or 5·1 tons. The load required to produce the same amount of elastic shortening in the ironbark column was obtained thus: $5·1 \div 11 = 464$ tons per square inch, which, multiplied by the area of 12 in. x 12 in. = 66·816 tons. The strength, therefore, of the timber columns was only 65 per cent. of one made of steel when deflections were equal.

The author also compared ironbark and steel as used in girders, and quoted examples of each safely bearing 26 tons on a span of 16 feet. The steel beam weighed 960 lbs. and would be about 16 in. x 6 in. = 60 lbs. per foot. When fully loaded its deflection would be $\frac{5 W L^3}{384 E.I}$: W = weight 26 tons : L^3 = cube of length in inches, E = Mod. of Elas. = 12,000 tons : I = moment of inertia = 700. The solution of this equation gave a deflection of ·4 inches. To produce a similar deflection in the ironbark beam under similar loading, the same equation would be used as for the steel girder, excepting that the "moment of inertia would require to be increased to 5,313, and the modulus of elasticity to 12,000 ÷ 11 or 1,100. The solution of this equation would show that the beam would have to be 15¾ in. wide, and not 13 in., as specified. This increased size would cause the beam to weigh 2,040 lbs., or more than twice the weight of the steel one.

Mr. Dauncey (a visitor) stated that he did not favour the use of long single pieces of steel in buildings. Could anyone tell him why it was that a piece of steel collapsed suddenly? Some remarkable instances of steel breaking without apparent reason had come under his notice. He had seen a steel armour-plate, 5 in. thick and 6 ft. square, which ripped from side to side like paper, yet it had never been touched since it was manufactured. He had also seen railway axles go the same way. The fact was there was a chemical change going on in a piece of steel which the best steel makers could not explain. It might be due to various reasons, but whatever it was he thought before long, by using these long girders in buildings, and particularly with the strains put upon them, that would be likely to result in accidents. He considered iron the most reliable for use in building construction.

Mr. W. D. Cruickshank said in reference to the remarks made concerning the uncertainty of steel, that he could not agree with the speakers, so far as steel used in the construction of engines and boilers was concerned. His experience had given him the utmost confidence in this material.

Mr. G. Ashcroft did not think the chemical change alone would account for the uncertainty of steel, being of opinion that there must be some molecular change. The uncertainty of steel was one great argument against its use. He believed the recent accidents to the Baldwin engines had occurred at a time when molecular change was going on.

Mr. R. Pollock, in speaking of the factor of safety used by the author for columns, stated that Mr. Staten Smith, an American authority, in calculating the strengths of the Phœnix type of column, used a factor of safety of 4, but which was varied with various proportions of the lengths to the diameters.

The formula was—

$$4 + \frac{\Upsilon}{20} = \text{factor of safety.}$$
$$\Upsilon = \text{ratio of length to diameter.}$$

If this were applied to the example given by the author of the column, 12 ft. 6 in. long, 9 in. diameter, the factor of safety would be 4·6; thus—

$$4 + \frac{12}{20} = 4{\cdot}6, \text{ not 3 as stated}$$

The author's suggestion of circulating water through the columns and girders of buildings in the event of fire taking place was ingenious, but one of the principal objections existed in the fact that it would only be required very occasionally, and therefore, when wanted, would most probably be found unworkable.

He could not agree with the remarks made concerning the unreliability of steel, as such improvements had been made in the methods of manufacture of late years, and the large amount of data available as to its behaviour under various conditions which he considered proved that every confidence might be placed in it.

Mr. J. W. Ashcroft, in reply, stated that a factor of safety of 3 was that generally used in Sydney, but that he preferred to use 5.