

Mr. ZERAH COLBURN said, he had refrained from entering more into detail, to avoid making the Paper tedious, but he should be happy to reply to any questions. No doubt, in looking at the diagrams, it would be considered that American engineers had practised great economy in designing these bridges. The fact was, that if bridges could not be constructed of iron at a moderate cost, the Railway Companies in the States would not adopt them, but would continue to use timber bridges, which could be built at from £5 to £7 per lineal foot. It was on the score of cost, alone, he believed that American engineers had adopted cast iron for all parts in compression; so many square inches of section could be put into the tubular form for one-third the cost in cast iron that would be incurred with wrought-iron plates. In the case of the Green River Bridge, the top chords and the vertical posts were of cast-iron pipes. Care was requisite in casting the pipes. Mr. Fink tested the iron for all the pipes, and holes were drilled in the side that lay uppermost in the sand, to ascertain that the cores had not floated, and thus that the metal was of uniform thickness. The fact of cast iron being used in the top chords was a sufficient explanation, why bridges were not made continuous over two, or more, spans in America. In nearly all these bridges almost the whole of the iron was made to do work in carrying the load. They no doubt differed from what English engineers were accustomed to, and he feared the diagrams would present an extraordinary appearance to English eyes; but they were faithful representations.

Mr. Colburn then called attention to the drawing of a bridge with truss rods to every upright post. The span was 125 feet, and the depth was 23 feet from the centre of the top chord to the centre of the bottom chord; yet all the iron posts, the truss rods, the diagonal tension rods, and the leaning end columns were not together equal to a quarter of an inch in thickness of iron, if spread out as a plate, over the whole side, and half of that thickness was cast iron; still no portion of the iron was subjected to a greater working strain than 4 tons to the square inch. Under ordinary circumstances a working strain of 3 tons to the square inch was the utmost that existed. He contrasted with that design some plate girder bridges, erected three years ago, on the Boston and Worcester Railway. They were 87 feet span and 7 feet 6 inches deep. The plates were 6 feet 3 inches wide and 7 feet 6 inches high. At every vertical joint there was a pair of butt straps, 8 inches wide, double riveted on each side of the web, and over these a pair of angle irons, 3 inches by 6 inches. Midway between the vertical joints, there were two angle irons, 6 inches by 3 inches, the longer sides being turned over at right angles, to form a knee by which they were riveted to the top and bottom chords. Mr. Philbrick, the engineer, adopted that form of stiffening the

webs of girder bridges, because, as he said, he had found that in England and in France, the T irons used for stiffening had begun to split in some cases. He had only called attention to the butt straps and angle irons used by Mr. Philbrick, because these, spread out as plates over the side of the web, were alone equal to one-quarter of an inch in thickness, while in the Murphy-Whipple Bridge the whole of the members forming the web amounted to less than a quarter of an inch.

He might add, that none of the bridges shown in the drawings had broken down, and he had never heard that they had exhibited any signs of weakness. They were considered in America, and he believed them to be, very strong, and good bridges. If they had any peculiar merit, it might be in the fact that they were not overloaded with ballast. The trusses were deeper, and, therefore, there was not so much iron put into the sides as might be expected to keep the strains down.

In Bollman's Bridge the test load was $1\frac{1}{2}$ ton per lineal foot, for a single line. In America the assumed load was arrived at, by supposing that there was on the bridge a train of loaded goods' waggons, with two engines and a snow plough weighing about 15 tons; and, in addition, when the bridge had a floor, that this was covered with snow, equal to a weight of 30 lbs. per square foot of floor; and that a side wind, equal to 30 lbs. per square foot was blowing. It was further assumed that, as every wheel had a break, all the breaks were put on at once, which would throw the strain upon the top chords of the bridge. That was the assumed load of the bridge of 200 feet span, and it was adopted by Mr. Murphy, Mr. Whipple, and Mr. Bollman.

In answer to inquiries from several members, Mr. Colburn said that the highest breaking strain of the cast iron alluded to in the Paper was $20\frac{1}{2}$ tons per square inch; but it was certainly never subjected to that strain in a bridge. It was boiled cast iron, that was melted and kept in fusion for several hours, and partially decarbonized. Previous to the present war in America, every cast gun was made of iron melted two, three, or four times.

Mr. F. J. BRAMWELL regretted to hear that the arrangements for the reading of other communications were such, that it was desirable to limit the present discussion; as he feared there would be great difficulty in investigating the principles of construction of the Trusses described in the Paper, unless some considerable time were devoted to that purpose.

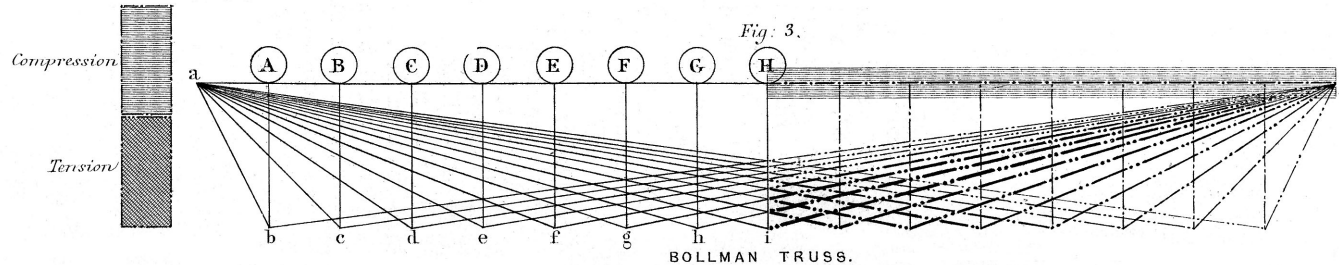
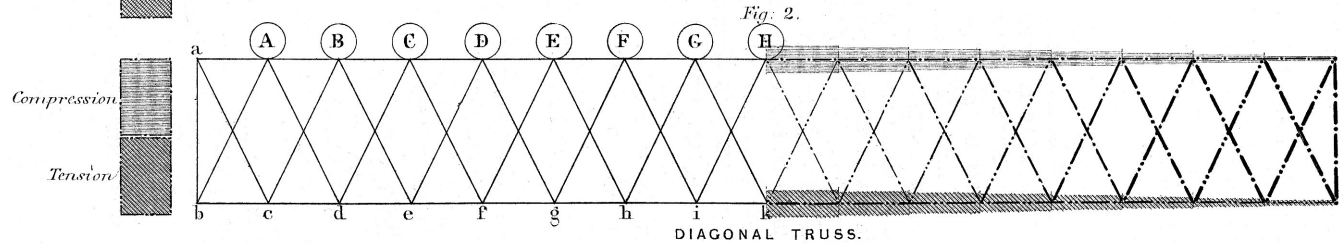
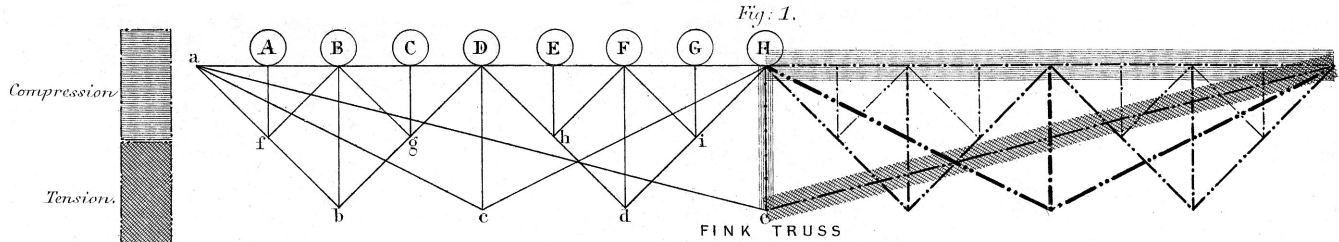
Before entering into the question of the Trusses, however, he wished to give a few facts bearing on the strength of cast iron. They were corroborative, to a great extent, of the statement on this subject made by the Author. That statement

seemed to excite surprise, but Mr. Bramwell was quite prepared for such a statement, having had occasion, some time back, to investigate the strength of a particular quality of cast iron, which gave results nearly agreeing with those mentioned in the Paper. He exhibited two samples of Acadian cold blast iron, the produce of Nova Scotia, which had been broken, in his presence, by the proving machine at Woolwich Arsenal. Being No. 1 iron, a mixture of scrap was required before strength could be obtained. But as the experiments were made to test the quality of this individual iron, it would have rendered those experiments nugatory, had scrap from any other source been mixed with the iron. The only way of proceeding, that occurred to him, was to melt some of the iron into pigs, to make scrap for the second melting, reserving trial bars of the first melting. The order of the experiments was as follows:—The iron was put into an air furnace, and as soon as it was fused, eight sample bars were cast. These bore an average tensile strain of $7\frac{1}{2}$ tons per square inch of section. The iron was kept in fusion, and two hours after, eight other bars were cast. These did not break until a strain of 8 tons 6 cwt. was attained, being nearly 1 ton more per square inch than the strain which broke the first lot of bars. After a further interval of one hour and three-quarters, eight other bars were cast, which broke with a strain of $10\frac{8}{10}$ tons; so that by keeping the metal in fusion for three hours and three-quarters, the tensile strength was increased from 7 tons 10 cwt. to $10\frac{8}{10}$ tons, or about 50 per cent. The iron was then poured out, and pigs were cast from it. On the next occasion, the furnace was charged with half of the fresh No. 1 iron, and half of the pig iron from the prolonged fusion. The result was, that the bars cast immediately upon melting required for their fracture a tensile strain of 11 tons; while those cast after two hours' continued fusion only bore 9 tons 4 cwt. As this result did not agree with those obtained from the bars cast before and from those cast after this particular lot, Mr. Bramwell had no doubt, there was some error in the marking to identify these bars. The bars that were cast after two hours' further fusion gave an average tensile strength of $18\frac{1}{2}$ tons. The strongest, of which he exhibited a sample, did not break until the tensile strain per square inch rose as high as 19 tons 12 cwt. As an evidence of the unusual strength of this iron, he might state that the whole of the weights belonging to the proving machine were not quite sufficient to enable this strain to be applied, so that extra weights had to be obtained. This sample of cast iron was undoubtedly the strongest that had ever come under his notice. The specific gravity of the metal increased from 7·09 for the No. 1

pig, to 7·29 for the sample which bore the strain of 19 tons 12 cwt.

The Author, in alluding to the method of melting iron in America, stated that its toughness was increased. Mr. Bramwell hardly knew in what sense the term toughness was used. If it meant merely tenacity, or capability of bearing tensile strain, no doubt that was increased; but if it meant tensile strain, coupled with movement through a considerable space before rupture took place, so that the 'work done' to produce fracture would be large, then he questioned whether the melting spoken of did, after a certain point, add to such toughness; although no doubt it added to the power to bear a great load, if very gradually and quietly applied. He regretted that there were no means of ascertaining, by the machine at Woolwich, the facts involved in this question of toughness. The samples were however tested for their power to resist compression. This was found to be, practically, about three times and a half greater than that to support tensile strain; a lower ratio no doubt than was ordinarily obtained. It would perhaps be considered surprising, that the iron he had spoken of was rejected as being bad, because it contained silicon. It must be borne in mind, that this silicon could not be detected in the behaviour of the iron, but required the careful analysis of the chemist to discover it. Nevertheless, the silicon being there, the iron was as much condemned as if the silicon had exercised its supposed baneful influence, so as to render the iron bad under mechanical test. This seemed to him to be yielding too much to chemical authority, which, after all, must base its opinion, as to the injurious, or non-injurious, character of any foreign substance in a metal, upon the results derived from mechanical test.

With regard to the principles involved in the construction of the Trusses of American Iron Bridges, he referred to three diagrams (Plate 21) he had caused to be prepared, showing, respectively, the Fink truss (Fig. 1), an ordinary diagonal truss which was so well known in this country (Fig. 2), and the Bollman, or Harper's Ferry Bridge, truss (Fig. 3). In these diagrams, one-half of each figure was drawn merely in lines, running along the centres of the various members of the truss; while in the other half of each figure the members were indicated either by dotted lines simply, or by dotted lines and shading on each side, so as to show, by the width of the shading or of the dotted lines, the relative amounts of iron required in each member; and by the different kinds of dotted lines and the different kinds of shading, to distinguish between the parts in compression and those in tension; discarding, for the sake of simplicity, all consideration of rivet holes, joints, and such



matters, and assuming the whole of the iron to be brought to an uniform thickness, and to be equally valuable for compression and for extension. Thus if the shading along one member of the truss were twice as wide as along another, then it indicated that there must be double the sectional area of metal in the first of these two members, that there was required to be in the second of such members. In this way the diagrams would at once instruct the eye, as to the consumption of metal in each part of each truss, while the difference of the shading would show, which part was in compression and which in tension. Further, at the end of each figure a parallelogram had been drawn, which indicated the relative amount of each kind of metal required in the three different trusses. Assuming that there would be no special circumstances of convenience, or inconvenience, attending the manufacture of any particular truss, there could be no question, that was the best truss which consumed the least metal in its construction. And if the decision, as to the merits of the Fink truss (Fig. 1), and of the Bollman truss (Fig. 3), were to be arrived at by the consideration of this test, Mr. Bramwell believed he was correct in saying, that they were both of them inferior to the ordinary diagonal truss (Fig. 2), and in the following ratio :—

Diagonal truss required	262	parts of metal.
Fink	396	„
Bollman	370	„

The distribution of the metal would be as follows :—

DIAGONAL TRUSS :	Compression . . .	136	
	Tension . . .	126	
		—	262
FINK TRUSS :	Compression . . .	200	
	Tension . . .	196	
		—	396
BOLLMAN TRUSS :	Compression . . .	185	
	Tension . . .	185	
		—	370

These results were arrived at by a calculation which he believed to have been based on correct principles, and to have been accurately worked out. He would, however, endeavour to explain the causes of the extra consumption of iron in the Fink and the Bollman trusses.

First, as to the Fink truss :—The diagram was made on the assumption, that the depth of this truss, and of course of [1862-63. N.S.] 20

the other two trusses also, was one-eighth of the span, and that the truss was loaded with fifteen units of load, one over each of the vertical struts. The struts on the left-hand side were lettered from A to H, the latter letter being over the central strut. The effect of these units of load would be, that unit A would, by its vertical A *f*, be carried one-half by the tie rod *f a*, and one-half by *f B*; and that the unit B would be carried on its vertical B *b*, which would also have to carry the half of unit A, brought on by the tie-rod *f B*, and also one-half of the unit C, brought on, in a similar manner, by the tie-rod *g B*. In this way the vertical B *b* would get a load of two units, one-half of which, or one unit, would have to be upheld through the tie-rod *b a*, but as the upper part, *f a*, of this tie was already strained by the weight due to one-half of the unit A, and as the angle of the tie was 45° , the tie *b a* would bring a compression of exactly a unit and one half on the top member of the girder. It was clear that a downward load of one unit and a half must also be brought, by the tie *b g D*, on to the point D, and that a similar load of a unit and a half must be brought on to the same point by the tie *d h D*, making together a load of three units. This added to the unit D itself, would cause a load of four units to have to be carried by the vertical D *c*. Of these four units two would be carried by the tie *c a*, and the other two by the tie *c H*. The tie *c a* would bring the compression on to the top member of the girder due to these two units. If the tie *c a* had been at the same angle as the tie *b f a*, this compression would only have been two units; but as the tie *c a* was at an angle only half as favourable as the tie *b f a*, the compression brought on to the top member of the truss by the tie *c a* would be double that of the vertical load carried by it, or a compression of four. The last source from which compression would come upon the top member of the truss would be from the strain of the tie *e a*. This tie, it would be seen, had to bear one-half the duty of upholding the central strut H *e*. This strut H *e* would have to support, first, two units of load brought on it by the tie *c H*; secondly, one unit of load brought on by the tie *d i H*, owing to that tie upholding one-half of the load of two on F *d*, and, thirdly, one-half of a unit of load brought on by the upper part *i H* of the same tie *d i H*, which upper part would have to support the half of the unit of load on G *i*. These three sources of strain would amount together to three and a half units. There would of course be an equal load brought on by the corresponding ties to the right hand of the strut H *e*, which strut would thus be loaded with seven units, to which must be added the unit on H itself, making eight as the total load on the strut H *e*. Of this eight, one-half, or four, would be upheld by the tie *e a*, and by this tie would exert a compressive force on the top member of

the girder. It would be seen, that the angle at which this tie ea lay was twice as unfavourable as that of the tie last considered, and four times as unfavourable as that of the tie bfa , the angle of which was 45° . Owing to this unfavourable angle of the tie ea , the load supported by it would exert a compression of four times its own amount, or sixteen. The sum of the compressions, therefore, on the top member of the girder would be,

For the tie	fa	$\frac{1}{2}$	unit $\times$ by the effect of the angle	$1 = \frac{1}{2}$
"	"	ba	1	" " " " $1 = 1$
"	"	ca	2	" " " " $2 = 4$
"	"	ea	4	" " " " $4 = 16$
				$21\frac{1}{2}$

It would be seen, that this compression extended from end to end of the top member of the truss, so that the top member would require to have as great a sectional area at the extremities as in the middle. Assuming that one part of iron was required to resist one unit of load, then the sectional area would be $21\frac{1}{2}$, and the length of the truss being taken as 8, the amount of iron consumed in the compression bar would be equal to $21\frac{1}{2} \times 8$, or 172 parts of iron.

Before considering the amount of iron in the other members of the Fink truss, Mr. Bramwell would compare the compression bar, or top member, of this truss, with that of an ordinary diagonal truss (Fig. 2). In the latter case it was well understood, that as fifteen units of load, distributed as shown, were equal to a central load of eight, and as the girder was eight times as long as it was deep, the compression in the centre would of course be sixteen units of load. This then would represent the maximum sectional area, which would exist only at the very middle of the compression member, as it would become less and less towards the ends of the girder; the diminution being in such a ratio, if the number of points of attachment of the ties and struts were infinite, as would give a parabolic curve, the area of which would of course be two-thirds of the parallelogram containing it. Therefore $16 \times \frac{2}{3} = 10\frac{2}{3}$, would represent the average depth of the compression member, which multiplied by the length 8, would give $85\frac{1}{3}$ as the amount of iron in that member, or rather less than one-half of the iron in the similar member of the Fink truss. In the diagonal truss, with the number of ties and struts as drawn, this theoretic amount was diminished only to the extent of $1\frac{1}{3}$; that was to say, the whole amount of iron was 84, arrived at thus:—

2 0 2

				Pressure.
Between the points <i>a A</i> the compression would be $3\frac{1}{2}$ units, which, multiplied by the effect of the angle				= $13\frac{3}{4}$
"	<i>A B</i>	"	$10\frac{1}{2}$	$51\frac{1}{4}$
"	<i>B C</i>	"	$16\frac{1}{2}$	$84\frac{1}{2}$
"	<i>C D</i>	"	$21\frac{1}{2}$	$103\frac{3}{4}$
"	<i>D E</i>	"	$25\frac{1}{2}$	$123\frac{3}{4}$
"	<i>E F</i>	"	$28\frac{1}{2}$	$141\frac{1}{4}$
"	<i>F G</i>	"	$30\frac{1}{2}$	$151\frac{1}{4}$
"	<i>G H</i>	"	$31\frac{1}{2}$	$153\frac{3}{4}$
Giving for the sum of the compressions				<u>84</u>

This divided by the number of spaces 8, made $10\frac{1}{2}$ as the average depth.

It was, he thought, not improbable that a cursory view of the drawing of the Fink truss might lead to the idea, that it was one that must be economical; inasmuch as it appeared to have a compression member only, and to be without any corresponding formal tension member, the diagonal ties being made to serve the purpose of a regular tension member. But even if this were so, and if the ties were made at no greater consumption of metal than the ties and struts of a diagonal girder, which he would prove was not the case, even then the Fink truss would have no advantage; inasmuch as its one compression member contained slightly more iron than the compression member and tension member together of the diagonal truss. He had stated that the compression member of this latter girder contained 84 parts. The tension member contained rather more, thus,—

				Tension.
Between the points <i>b c</i> the tension would be 4 units, which, multiplied by the effect of the angle				= 2
"	<i>c d</i>	"	11	$5\frac{1}{2}$
"	<i>d e</i>	"	17	$8\frac{1}{2}$
"	<i>e f</i>	"	22	11
"	<i>f g</i>	"	26	13
"	<i>g h</i>	"	29	$14\frac{1}{2}$
"	<i>h i</i>	"	31	$15\frac{1}{2}$
"	<i>i k</i>	"	32	16
Giving for the sum of the tensions				<u>86</u>

This, divided by their number 8, made $10\frac{3}{4}$ as the average depth. The amount of metal 86 in that member, added to that in the compression member, gave 170 parts for the metal in those two members, as compared with 172 parts in the compression member alone of the Fink truss.

He would now revert to the consideration of the Fink truss:—

It would be remembered that the main tie bar *e a* had to transmit four units of load; but these became multiplied in their power of strain on the tie, by the number of times the length of that tie exceeded the depth of the girder. This depth being 1, and the

half length of the girder being 4, the length of the tie would of course be equal to $\sqrt{1^2 + 4^2} = \sqrt{17}$. The strain, therefore, would be the load $\times \sqrt{17}$. But as the length of the tie was also $\sqrt{17}$, it followed that the amount of metal must be $\sqrt{17} \times \sqrt{17} \times$ the load of 4; in other words, the amount of metal increased as the square of the length of any sloping tie. In this case the amount was 2 ties each $17 \times 4 =$

Parts of
Metal.
136

Similarly, it could be shown, that the ties *ea*, *cH*, and their counterparts on the other side of the truss, would require $\sqrt{5} \times \sqrt{5} \times 2$ load $\times 4$ of them =

40

That *ba*, *bD*, *dD*, *dH*, and their counterparts on the other side of the truss, would require $\sqrt{2} \times \sqrt{2} \times 1$ load $\times 8$ of them =

16

and that the upper parts of these ties, *fa*, *gD*, *hD*, *iH*, and their counterparts on the other side of the truss, with the corresponding short ties, *fB*, *gB*, *hF*, *iF*, and their counterparts on the other side of the truss, would require $\sqrt{\frac{1}{2}} \times \sqrt{\frac{1}{2}} \times \frac{1}{2}$ load $\times 16$ of them =

4

Making the total of iron in tension, as already stated,

196

Further, the compression member, it had been proved, would contain

Parts of
Metal.
172

The strut *He* would contain 1×8 units =

8

The strut *De*, and the corresponding one on the other side of the truss, would contain 1×4 units $\times 2$ of them =

8

The struts *Bb*, *FD*, and their corresponding ones on the other side of the truss, would contain 1×2 units $\times 4$ of them =

8

The struts *Af*, *Cg*, *EH*, *Gi*, and their corresponding ones on the other side of the truss, would contain $\frac{1}{2}$ long $\times 1$ unit $\times 8$ of them =

4

Making the total of iron in compression, as already stated,

200

In order to complete the comparison between the Fink truss and the diagonal truss, it would be necessary to ascertain the amount of metal in the struts and ties of this latter truss. The lengths of the various struts and ties would be uniformly $\sqrt{1\frac{1}{4}}$, the multiple of strain would therefore be $\sqrt{1\frac{1}{4}}$, and the amount of iron in each of them $1\frac{1}{4} \times$ the units of load :--

On the strut $A b$ there would be 4 units.

"	$B c$	"	$3\frac{1}{2}$	"
"	$C d$	"	3	"
"	$D e$	"	$2\frac{1}{2}$	"
"	$E f$	"	2	"
"	$F g$	"	$1\frac{1}{2}$	"
"	$G h$	"	1	"
"	$H i$	"	$\frac{1}{2}$	"

Making a total of 18 "

This sum, multiplied by the $1\frac{1}{4}$, would give 22 $\frac{1}{2}$
 To which must be added the end upright $a b$ = 3 $\frac{1}{2}$

Making 26

Or, for the two sides of the truss 52

Adding the amount of metal in the compression member
 before given 84

The total of iron in compression would then be, as already
 stated 136

Next, as regarded the parts in tension :—

On the tie $a c$ there would be $3\frac{1}{2}$ units

"	$A d$	"	3	"
"	$B e$	"	$2\frac{1}{2}$	"
"	$C f$	"	2	"
"	$D g$	"	$1\frac{1}{2}$	"
"	$E h$	"	1	"
"	$F i$	"	$\frac{1}{2}$	"
"	$G k$	"	0	"

Making a total of 14 "

This, multiplied by $1\frac{1}{4}$, would give 17 $\frac{1}{2}$

Making for the two sides of the truss 35

Adding the amount of metal in the tension member before
 given 86

The total of metal in tension was thus found to be 121

But to this should be added the ties $G k$, which, although not
 called into action, when an uniform load was on the girder,
 would have to bear a strain if the load were a passing one.
 They might then be required to support a single unit of load.
 Under similar circumstances two ties on each side might
 require to be strengthened, so as to bear a further half unit
 each ; then 4 ties 1 unit $\times$ the $1\frac{1}{4}$ = 5

Giving as the total of iron in tension, as already stated, 126

He would not occupy any further time, by entering into the details of the calculation of the quantity of metal in the Bollman Truss; but would simply refer to the figures already given, by which it would be seen, that while it was more economical than the Fink Truss, in the proportion of 37 to 39·6, it was less economical than the ordinary diagonal truss, in the proportion of 37 to 26·2.

The cause of the waste of metal in the Fink and in the Bollman trusses appeared to him, on a little consideration, to be sufficiently obvious. The strains which related to the centre part only of a truss, and which might be got rid of in a comparatively short distance near that centre, were, in the Fink and in the Bollman trusses, carried to the very ends of the compression member, so that the whole length of that member sustained a pressure that need only be borne near the centre. Further to carry out this bad arrangement, the tie-rods were of necessity placed at most unfavourable angles, by which not only was an unnecessary amount of metal consumed, but the already useless strain on the compression member was aggravated.

It was unnecessary to investigate the construction of the Murphy-Whipple truss, as the drawing showed that truss to be free from the radical error, of carrying the whole of the compression to the very ends of the top member. It was, however, clear, that the use of struts in a vertical position was not so economic, as the use of struts in a diagonal position, inasmuch as the struts when placed vertically did not assist in the progression (if such a term might be allowed) from end to end of the truss. The ties, also, were not disposed at the most economic angle; but with those exceptions, he had little doubt the Murphy-Whipple truss would, on investigation, prove to be one of good construction, and equal to such as were in ordinary use in England.

He regretted having to pronounce so unfavourable an opinion on these trusses; but he was glad to be enabled to do so, without the fear of his criticism being displeasing to the Author, who, it was understood, had merely submitted these particular trusses, as matters of interest in the history of railway works in America, without expressing approval of their construction, and for the very purpose of having their merits, or demerits fully discussed.

Mr. FLEEMING JENKIN said, there was an apparent discrepancy in the general practice of Engineers in the use of cast iron under compression. From the Paper it would appear, that the cast-iron compression bars of all the girder bridges were strained up to a maximum of from $4\frac{1}{4}$ to $4\frac{1}{2}$ tons to the square inch, whereas in the two cast-iron arches mentioned by the Author, the strain per square inch was limited to 2 tons and 1·6 ton respectively. Nevertheless, in both cases the strains were due to loads of a similar nature; and at first sight there was no reason why the cast iron

should not be capable of bearing an equal strain in both classes of bridge. The practice in England agreed with that in America. The strain allowed on cast-iron arches was from $1\frac{1}{2}$ ton to 2 tons as a maximum; whereas in columns and other constructions, strains of 4 tons were admitted without hesitation. He thought it was a point of some interest to examine, how it happened that under similar conditions the same material should be so differently treated. He believed the fact to be, that the calculated strains upon cast-iron arches were exceedingly fallacious; no experiments had been made upon the strains in metal arches, whether of cast iron or of wrought iron, whereas excellent experiments had been made upon girders and columns. The strains upon every part of a girder bridge were well known, and the metal could be accurately apportioned to the work it had to perform; whereas he was not aware of any experiment having been made on arches. The calculations which had been based upon the theory devised for stone bridges were, in his opinion, altogether fallacious, and he thought the subject deserved much more attention than it had hitherto received. The common formula, by which a cast-iron arch was calculated, supposed the existence of an uniform load. This formula gave the compression on the crown of the arch, as equal to the compression which would result at the centre of the top bar of a girder, of a depth equal to the rise of the arch, and loaded in the same manner. This calculation would be correct, provided the arch were formed of two rigid beams, such as the rafters of a house, hinged at the centre, and loaded uniformly, with reference to a horizontal line; but an arch was not hinged, nor perfectly flexible at the centre, nor were the two sides perfectly rigid; therefore the formula could only apply approximately. Still as the arch was really comparatively flexible at the crown and rigid at the spandrils, he believed the formula would, on experiment, be found sufficiently exact. On the other hand, he had never met with any satisfactory calculations of the strain thrown upon a metal arch bridge by a passing load. Professor Rankine mentioned, that a stiffening girder, similar to that required in a suspension bridge, would be required for a cast-iron arch; but, so far as he knew, no person had given the actual strains produced by a passing load, and it was in fact difficult to calculate them. The rigidity of the cast-iron arch played the part which the load over the spandrils took in a common stone arch, bringing the line of pressure from the crown down to the springings. That rigidity, or resistance to bending, was a difficult thing to calculate, especially with such forms as were now common, in which the rigidity was partly due to the main rib, partly to a top girder, partly to the intermediate filling of the spandrils, and partly to a sort of compound beam built of these three parts, which were usually very imper-

fectly connected. The large unknown strain, produced by the flexure due to a passing load, was, he believed, the practical reason why Engineers increased the section of the cast-iron ribs of arches, to such an extent as to reduce the maximum strain, when calculated from the uniform load alone, as low as $1\frac{1}{2}$ ton or 2 tons; whereas they were able to allow 4 tons on the girder or column, because it was known from experiment, that the actual maximum strain upon these forms would not exceed the calculated strain. With the present form of arch it was almost impossible to calculate the strains. He had endeavoured to design a form of arch which, being a simple piece of framework, or trussing, would permit the calculation of the strains on every member, due to every arrangement of the load. Each member was strained in the direction of its length only. That form was shown in the model of an arch he had placed upon the table. He would not now enter upon a mathematical investigation of the subject, as he hoped soon to have an opportunity of practically testing the merits of that form, and he would then bring the results before the Institution. He thought he should be able to show, that a cast-iron arch could be made, in which the material was subjected to the full strain of 4 tons per square inch, and that such bridges could be built at less cost than the common plate, or parallel girder bridge.

Mr. E. A. COWPER could not allow the observations, with reference to the strains upon cast-iron arches to pass unchallenged. The Washington aqueduct might, perhaps, be taken as a good example of a column placed in the form of an arch. He argued that columns were often compressed with a greater pressure per square inch than arched ribs, if the whole section was taken as the area. A vertical column, without side strain or pressure, was in a very favourable condition for bearing weight; and indeed it was astonishing what columns would sustain. It had been stated, that if the column were in the form of an arch, there was great difficulty in calculating the compression per square inch. To this he dissented, believing that the strains upon a cast-iron arch could be readily estimated. It was necessary to ascertain, first, where the line of strain would come, under the various circumstances, of a passing load, &c., and having found how near the edge, or flange of the rib the strain would pass in the worst case, it was easy to calculate the section that it was necessary to give to the rib, so that the iron might be put to a fair strain per square inch. It was an expeditious, and if carefully performed, an accurate method, of finding the true line of strain of the Distorted Catenary Curve, to hang up a flexible chain to represent the weight and form of the rib, and then to load it, first with the various weights of the bridge itself, if the spandrels had any heavy filling-in, and then with the passing load in various positions. The true curves,

under the various circumstances, would thus at once be obtained, and could be traced on a drawing-board, placed behind the chain. This plan he had practised for many years, no doubt in common with others, and had used it in settling the sections of the ribs of the Barnes, Richmond, and other bridges. Of course, in a bridge in which the strength was intended to be given entirely by compression, it was no more admissible to make the rib so shallow, as to allow the line of strain of any one of the distorted catenaries to pass outside, than it would be, to make a column for bearing compression so crooked, that the vertical line of pressure came outside the column. He need hardly say, that the true line of compression in a bridge would never follow the fancy curves that were often given to arches, such as semicircles and ellipses, but would always be in a catenary curve, more or less distorted, according to the various loads upon it and their position. He had, many years since, designed a suspension bridge, on precisely the same principle as that of an arch inverted, including all the lines of strain, or tension, within the chain, and providing sufficient depth in the section of the chain to take the tension where it came. The chain was, in fact, a wrought-iron plate, with top and bottom flanges, like an ordinary deep wrought-iron girder. Such a bridge would be perfectly stable, as it could not move out of shape, nor deflect from the elasticity of the material so much as an ordinary girder. When, however, it was intended that an arched rib should not depend entirely on compression, but should also rely on the tension of its material, from being made of wrought iron well bolted, or riveted together, or of cast iron, in pieces provided with substantial flanges, and proportionate bolts and nuts, then it might be looked upon, to some extent, as having the stiffness of a girder, and might be made so narrow, that some parts of the distorted catenaries would pass outside it, in which case it was easy to calculate how much compression there would be on one side and how much tension on the other side, as it was a simple case of leverage between the line of strain and the inside and outside flanges of the rib. He had stated that, under certain circumstances, an arched rib might to some extent be looked upon as a girder, and he had spoken thus guardedly, because an arched rib, thoroughly stiffened by bracing, or spandrels extending over the crown of the arch, and having a considerable depth, would so resist the action of rising and falling with the expansion and contraction due to variations of temperature, as to cause fracture in some part or other. For an arched rib pre-supposed fixed abutments to take the thrust, so that when the metal expanded by heat, the arch must rise, or break by crushing. This was not a fanciful objection, as it occurred in every-day practice. For instance, Southwark Bridge was erected in cool

weather, so that in the full heat of the sun, the centre arch rose in the middle $1\frac{3}{4}$ inch, and the various castings forming the spandrils, being deep and well fitted together, without any allowance for expansion, began to break consecutively. The cause of fracture was owing to the stone abutments being fitted tightly against the ends of the spandrils. It had been found necessary to jump holes in the stone, immediately behind the ends of the spandrils, so as to relieve them, and to allow the rib to rise as it expanded. The effects of expansion and contraction in Southwark Bridge were well shown by the hand-railing drawing out of the stone piers in winter, and being thrust into them in summer, the play over the abutments amounting to $\frac{3}{4}$ of an inch. The stone paving was also always out of order over the piers, and in some cases the heavy stones had flushed at their top edges. He thought many more experiments ought to be made, on the end pressure that columns of various sections would bear. The few experiments of that kind which he had tried agreed very well with those made by Mr. Eaton Hodgkinson; but proper experiments with columns on a large scale were expensive, chiefly on account of the heavy pressures required to crush them. There was also danger in measuring the deflections when the crushing weight was nearly approached, as a hollow column, for instance, would be separated into many pieces, with great violence, when it was crushed or broken. He had found the want of reliable experiments of this class, when designing large shear legs of 120 feet in height, to lift 40 tons, and had been obliged to rely on his judgment alone as to strength: the result, however, had been satisfactory.

Mr. PHIPPS said, the Paper was for the most part limited to several varieties of trussed girders. On these he would not at present offer any observations, but would confine his remarks to one, or two iron arched constructions, also referred to in the Paper, as he had paid particular attention to the subject of cast-iron arches. Mr. Jenkin had truly remarked, that whenever these arched constructions were loaded irregularly, it became difficult to calculate the strain to which the material then became subject. For instance, iron arches, particularly on railways, might be loaded upon one-half of the span only. In such a case the curve of equilibrium would often shift so much from the middle of the arched rib, as almost to touch the extrados and the intrados of the arch on opposite sides of the centre. Having obtained, however, the position of the curve of equilibrium, it then became a question, how to estimate the effect of the pressure in its detrimental action on the outer fibres of the rib. This was a point upon which, he agreed, that no practical, or reliable, information was to be obtained from books, and, in consequence, he had invented a method which he had found both simple and accurate in

its application. To illustrate the degree to which the shifting of the line of pressure might affect the strain upon an iron arch, when placed under compression, he stated that in a prismatic bar of iron, in shape like a 3-inch plank, the pressure on the outer fibres on one side would be doubled, and on the other side be reduced to nothing, by the removal of the line of pressure only the one-sixth part of the whole width away from the centre of gravity. Now in this, as in every other case, whenever a piece of metal, or other elastic material, was compressed by a force on any other line than that of its centre of gravity, the action upon the outer fibres might be obtained, by conceiving the material, first of all to be compressed squarely throughout, as if the strain were applied over the line of centre of gravity, and then, finding the strain due to the angular motion, by the same process that would be used for obtaining the strain on that portion of a cantilever contiguous to where it was attached to a wall; using the total compressive force applied as the weight, and the distance of that pressure from the line of centre of gravity as the leverage. Then the strain on the outer fibre thus obtained, added to the former square-on pressure, would give the whole detrimental action upon the outer fibre of the material. Thus, as he had previously explained in reference to the Charing Cross Bridge,¹ when a pressure of 475 tons was applied, square on, to a piece of iron of 161.25 square inches sectional area, all the fibres would be strained uniformly up to 2.94 tons on the square inch; but by removing the line of pressure 3.6 inches away from the centre of gravity, the pressure on the outer fibres would be increased up to 6.86 tons on the square inch. Mr. Phipps added that, in cases where the arched rib proper was well connected with the roadway bearer above, by an efficient system of diagonal bracing, the centre of gravity of the whole section of the arched rib and roadway bearer together must be taken, as that around which the previously-named angular motion must be computed.

Mr. F. W. SHEILDS could not agree, that the object of the bracing in a girder bridge was merely to keep the top and bottom members asunder, and to enable them to do their work. In his opinion the chief object of bracing in all parallel girders, was to carry the vertical weight of the bridge, with the extraneous load upon it, to the points of support at the piers; and that duty was just as important as, and quite distinct from, the work to be performed by the top and bottom flanges. The bracing ought to be specially designed for that duty, and in that view he thought the American bridges brought under their notice, even those of the simplest form, showed a want of scientific knowledge

¹ *Vide ante*, p. 535.

in their construction. Thus, if a girder bridge be supposed to consist of twelve bays, and the driving-wheels of a locomotive loaded with 12 tons rested over the third bay, that load would be transmitted to each pier, by the bracing, in the inverse proportion of the distance of the loaded point from the piers, being nine tons to the nearer, and three tons to the further pier. One of the diagrams exhibited was just in such a case, and he observed that at the third bay, there was no diagonal tie from the lower flange which could carry the load to the further pier. Consequently the girder would have to depend merely upon the rigidity of the top flange, considered as an independent beam of perhaps a few inches deep, for the transmission of that part of the load which reached the further pier, and which should be sustained by the whole girder of several feet deep. Diagonals of great inclination were not advisable, independent of all increase of strain arising from their inclination, as they were free to deflect on a curve struck from one of their ends as a centre and the other end as a radius, and that curve would coincide in practice, for a considerable distance, with a vertical line, so that there was little or no resistance to deflection. There were some bridges in the Regent's Park upon that principle, which fully bore out his foregone conclusions, for he found that by jumping upon those bridges, the whole could be set in motion by his own weight alone.

Mr. ZERAH COLBURN remarked, with reference to the comparison which had been made between the various trusses, that it was well understood in the States, as applying to the Fink and Bollman Bridges. The Murphy-Whipple Bridge (Plate 20, Figs. 3 and 4) was believed to be the best class known in the States; and if there was not much difference between that and many bridges in this country, it was no doubt owing to the fact, that when the principles were fixed, there could not be much room for difference between one good bridge and another.

Mr. HAWKSHAW, President, expressed his regret that, owing to the late period of the Session, and the necessity for reading two other Papers before its close, the discussion upon a subject of so much general interest as American Iron Bridges should have been necessarily restricted. The Institution was under great obligations to Mr. Colburn for the able and elaborate manner in which the communication had been made; and he hoped that the Author would, on other occasions, contribute to, and take part in, the proceedings.