Mr. Wilson said, he should like to be furnished with some further details of the testing of the New Victoria Bridge. It was stated that the load was 360 tons, but it did not appear how much of the bridge was covered lineally by that weight.

Mr. Fox said the load just covered one span.

Mr. Wilson had not offered any theoretical opinions, nor instituted any comparisons in his Paper, although the extraordinary instance of an entire bridge being built at different periods by two engineers upon totally different principles, when the circumstances to be dealt with were the same—the same loads to be carried—the same spans and rise of arches, and the same waterway—was, he thought, a matter for careful consideration.

Mr. C. Douglas Fox observed that the New Victoria Bridge afforded an example of a thoroughly continuous pier, above low-water mark, founded upon cylinders. In designing the bridge, it was necessary to provide against the difficulty, imposed by the terms of the Act of Parliament, of connecting the new work with the then existing Victoria Bridge; and that circumstance materially affected the whole character of the design. It was a matter of interest, that a length of 913 feet of horizontal girder was riveted up over each pier from end to end. This was completed at a temperature of about 60°, the operation being postponed till that temperature was obtained. The strains on this bridge were shown in Fig. 11, Plate 6. He might state that the ribs and horizontal girder were considered together as continuous girders of varying depth, or as two cantilevers projecting from each pier, and one from each abutment, with an intermediate horizontal girder in each span, the cantilevers being each 51 feet 6 inches, and the horizontal girder 75 feet in length. The question of the stability of the piers was in this case not taken into account. It would be seen that if one span were entirely loaded, and the next span unloaded, there would be an upward strain in the centre of the unloaded span. That strain, which was 30 tons, was consequent upon the horizontal thrust of 149 tons at the springing, and would put a tensile strain upon the horizontal girder and rib, which were blended at that point, of 4·3 tons per square inch of section. If the span at the other end of the unloaded span could also be loaded it would increase that strain to 8·6 tons: but it was hardly within the bounds of possibility that, for example, spans Nos. 2 and 4 should be fully loaded for the full width of the bridge, No. 3 being at the same time unloaded. He would mention again the results of the tests. When No. 2 span was loaded both No. 3 and No. 1 each rose up $\frac{1}{8}$ inch in the centre: and the effect of the stiff cross and vertical bracing, and of the way in which the cross girders were connected with the main ribs, was such that, when two of the ribs were loaded fully, the adjacent rib showed a deflection of $\frac{1}{8}$ inch. He made out, on the assumption that the variation of the temperature was 50° on each side of 60°, that the strain caused by expansion and contraction was equal to 4 tons on each inch: for iron, within the elastic limits, was compressed or extended ·000084 of its length by each ton of pressure or strain respectively upon a sectional inch; and by a variation of temperature of 50°, it was expanded or contracted ·000349 of its length, or rather more than the amount produced by 4 tons to the inch. A useful question for discussion, not raised in the description of these bridges, was the comparative merits of cast iron and wrought iron for an arched bridge of several spans, when the piers were not relied upon as perfectly stable. That would have no bearing upon the comparison of these two bridges, as they were both of wrought iron. But it was a matter which had been fully gone into by Sir Charles Fox himself, in which they had the assistance of Mr. Ordish, who superintended the preparation of the drawings for the iron-work details. They came to the conclusion that it was essential, where there were several spans of this kind, without perfectly stable piers, to adopt wrought iron to meet the tensile strain of which he had spoken. He would remark, with regard to the bridge over the South Western Railway, that when the centre span was loaded there would be a strain on each of the anchors of 68 tons, and the total weight on each column would then be a little over 400 tons.

The operation of rolling the girders of the bridge of 120-feet span into place was a matter of some difficulty, because the bridge crossed six lines of railway with a constant traffic, excepting during the four hours of the morning from half-past twelve to half-past four. The chief difficulty was to steady the girder sideways during the operation of rolling, the width of the bottom flange upon which it had to be supported being 2 feet and the depth of the girder 11 feet 6 inches. The first operation was the erection of the main girder on the abutment; it was trussed with temporary rods, and wooden distance-pieces were inserted between some of the ties, to meet the contingency of their having to act temporarily as struts; and several of the cross girders were attached to the tail of the main girder, and weighted with rolled beams. The girder was then rolled over about one-third of its length. As soon as the traffic was stopped a temporary railway of 4 feet $8\frac{1}{2}$ inches gauge was laid as expeditiously as possible, upon which a trolly, consisting of two wagons firmly connected side by side, each wagon having four wheels 2 feet diameter, was run under the girder, which was then firmly attached to it. Stout balks of wooden packing were fixed underneath the trolly and between the axles to receive the weight in case of the fracture of an axle. Small wooden

G 2

projections with wedges and chain guys were used to steady the trolly sideways; and struts were carried from the trolly up to the top of the girder, men being stationed there to keep them constantly tight by means of wedges. This operation was completed with ease in four hours, the motive power consisting of two crabs, one on each abutment, and crowbars applied to the trolly wheels. If, instead of the solid cast-iron rollers which were used on the abutment, it had been possible to substitute another trolly at that end, the work could have been performed in much less time. All the arrangements were carried out by the contractor, Mr. James Haywood, Jun., of Derby.

With reference to the Victoria Bridge, he would venture to throw out the following questions:—

1st. Might the piers of such a bridge as this be taken as really stable or not?

2nd. If the piers were perfectly stable, was not the bridge resolved into several bridges of single spans; and if so, would not cast iron have been the best material to use?

3rd. If the piers were unstable, and the platform and dead load light—as was the case in this bridge—must not tensile strain produced by upward motion in the unloaded span be provided for; and would not that be best done by wrought iron?

4th. Did not riveting up the horizontal girder from end to end increase the strength of the structure?

It would be noticed that he qualified the 3rd question by stating that the platform and dead load were light: the headway was limited, and therefore the platform was comparatively light. But there were instances of bridges over the River Thames on which there was a great thickness of ballast and heavy cast-iron plates; and it was found that, under such circumstances, cast-iron ribs had been advantageously used; as was also the case in road bridges where the metalling was of considerable weight, and where the dead weight was far greater in proportion to the live load than in the present instance. The reason he thought wrought iron was in the present case rightly used was that it had to meet the tensile strain; but in cases where there were heavy platforms, or a large amount of ballast, the strains were much modified and cast iron would be applicable.

Mr. Callcott Reilly observed, that this was probably an unique opportunity for deriving useful lessons for the future, from a comparison of what purported to be two entirely different systems of construction of iron arches, which had been adopted to attain precisely similar objects, and under circumstances exactly the same, so far as regarded the superstructure.

As these were the only Papers on the subject of iron arches in the entire series of twenty-seven volumes of the Minutes of Proceedings of the Institution, with one immaterial exception, it was to be regretted that the Authors had not given more information, bearing upon the principles which had guided them in the design of these two bridges.

Mr. Wilson had said, that there was a radical difference in the principles of the two designs, and he seemed to invite discussion on that question: yet, in his Paper, as well as in that by Mr. Fox, all mention of principle was carefully avoided. If the Authors desired to raise a discussion upon the principles of design of iron arches, they should have given a full explanation of their own principles, and a complete *résumé* of the calculations upon which these designs had been founded. It was commonly said that comparisons were odious; but it would be impossible, while comparing these two systems of construction, to avoid indicating an opinion that one was better than the other. Therefore, he hoped the Author whose designs were depreciated in the comparison, would attribute any objections that might be made to the true motive, that of an honest search after correct principle.

Coming to details, and taking Mr. Fowler's design first, he saw little room for criticism, in the absence of the calculations to which he had referred. Both designers were quite right, in his opinion, in preferring wrought iron to cast iron as a material for the arches of a bridge of several spans, which had to carry a relatively heavy moving load, and whose intermediate piers had but small stability to resist the unbalanced horizontal pressure which was applied to them when one arch was loaded, and one or more of the others unloaded, by the heavy moving load. The enlargement of span which resulted, sometimes produced tensile stresses upon the iron, as had been mentioned by Mr. Fox. It was that instability of pier which constituted the chief difficulty in the design of such arches as these, not specially because of mathematical intricacy in the analysis,—for that pervaded the whole subject,—but because experiments were wanting to establish general principles relative to the yielding of piers, under the action of unbalanced horizontal thrust. He would suggest, that engineers who had designed iron bridges of this kind, and who had the opportunity, should ascertain by actual measurement the enlargement of span due to the unbalanced thrust produced by heavy rolling loads, and should publish the results of those experiments, accompanied by a sufficient statement of the circumstances, so that in course of time a principle might possibly be evolved from a sufficient number of facts so obtained. In the absence of accurate data of the kind just referred to, Mr. Fowler had adopted a practical method, worthy of imitation, of equalising, to a certain extent, the varying stresses which resulted partly from enlargement of span under the influence of the heavy moving load, partly from the deflection, and partly from the changes of temperature. He alluded to the hinging of the arched ribs at the piers, which prevented the occurrence of any bending moment at the springing, by confining to one point the intersection of the line of resistance with the surface of the skewback; that point then coinciding with the like intersection of the neutral curve with the same surface under all conditions of load—a circumstance which facilitated the work of investigation and computation; and also to the use of cotters, which could be tightened according to the judgment of the engineer. He should be glad if an explanation were given of the principle which had guided the use of the cotters, and of the amount of cottering-up required under given circumstances. He presumed the principle to be this:—By tightening up the cotters, an initial compression was caused upon the lower edge of the arched rib, in addition to that which previously existed, and a corresponding diminution of the previously existing compression on the upper edge. The effect of this was to raise the crown of the arch above its former position. Those initial stresses would be properly considered in proportioning the sections of the ribs; and as they were of opposite kinds to those produced by the deflection, and by the enlargement of span caused by the unbalanced thrust due to the moving load, and also by change of temperature under certain conditions, the action of those disturbing forces would diminish the inequality of these initial stresses, and perhaps, going beyond that, would produce unequal stresses of the opposite kinds to the initial stresses. It was, therefore, obvious that the inequality of stress, produced by the action of those disturbing forces, would be much diminished by the judicious use of this cottering system. For his own part, he thought the arches might be hinged at the curve as well as at the springing, as proposed by some French engineers; this would have the effect of annulling altogether the stress produced by change of temperature.

Turning now to the other bridge; he gathered from Mr. Fox's remarks, that the whole series of arches, together with the horizontal girder connected with them, had been considered as one continuous beam. In opposition to that view he would suggest, that there were two essential characteristics of a beam, whether continuous or not, which were both absent in this structure, and that, therefore, it could not be properly treated as a continuous beam. He alluded, first, to the necessity which existed in every beam, or any combination of pieces which acted as a beam in supporting a vertical load, that the lines of action of all the supporting forces should be vertical, or in mathematical language, that the horizontal components of the supporting forces should be null. In this design the supporting forces at the three intermediate piers fulfilled that condition, but not so at the abutments. There the ribs rested upon oblique skewbacks, and there was obviously a horizontal component, at those two points of support, equal to the horizontal thrust at the crown, and that circumstance alone rendered the structure amenable to the laws which governed the strength of iron arches, as distinguished from beams.

The other essential characteristic of a beam, which was absent in this system, was that a vertical web was necessary, composed either of solid plate or triangular bracing, sufficient to convey the shearing forces, from the points of application of the loading forces, to the points of support at the piers. In this system there was no such web, there were only the radiating bars which occupied the spandrils, and which merely acted as posts in transmitting the load from the points of application on the roadway to the back of the arch; and, therefore, the shearing forces could only be transmitted to the supports through the arched rib, producing compression upon it.

Consequently, this system was really a system of arches, and not of continuous beams, and its principle was analogous to that of a suspension bridge, but inverted, stiffened by a horizontal girder, which acted much in the same way as was described in the discussion upon the Clifton Suspension Bridge, and upon Suspension Bridges of Great Span, last session[1]: and it might be treated by a similar method. In fact, that was the fundamental principle of all correct methods of calculation, applicable to metallic arches without braced spandrils.

Neither of the Authors had given any detailed weights, and, therefore, no conclusion could be arrived at as to the economy of material in these structures. The gross quantities of materials used in the Victoria Bridge had been given, but it was not stated how the weights were distributed. It would be useful to know how much iron was placed in the important members of each bridge, as well as the gross weight. It had, however, been incidentally mentioned that one of the bridges over the London and South Western Railway had two main girders of 120-feet span, and weighing 106 tons each, which were officially tested by a distributed moving load of 430 tons on the bridge. This pair of girders might, therefore, be assumed to have been designed to carry that moving load, and they weighed 212 tons. If that were so, he thought it an extravagant weight. He believed that bowstring girders might be made to carry the same moving load with considerably less than half that weight, and to satisfy every scientific and practical condition of success.

Mr. Gregory—Vice-President—said, perhaps the Authors would

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxvi., p. 265, *et seq.*

be kind enough to supplement their respective Papers by the calculations and detailed weights to which Mr. Callcott Reilly had alluded.[1]

Mr. G. H. PHIPPS said, he had always taken much interest in the subject of iron arches of large span, but especially so from the time of the rebuilding of Sunderland Bridge, on which he had been engaged under the late Mr. Robert Stephenson. He had made a design for a suspended tubular bridge on the site of the Victoria Bridge, which design he had alluded to in the discussion on the Clifton Bridge.[2] The piers were in line with those of the Chelsea Suspension Bridge, and the central span was 400 feet. This design was prepared at the time when the Victoria Station and Pimlico Railway was before Parliament, as he thought it might serve as an alternative, in case the waterway to be afforded by the Victoria Bridge was objected to.

Having had, at an early period, an opportunity of examining the particulars of the various strains on the Victoria Bridge, he had availed himself of them in his calculations.

There was no question that the area of the base of the foundations of these two bridges was amply sufficient for the dead weight to be carried, which amounted, in the Victoria Bridge, to rather less than 2 tons to the square foot, where a weight of about 8 tons per square foot was borne by the foundations of the Charing Cross Bridge.

Assuming, then, that it was decided to build an arched bridge in the present situation, and drawing no comparison at the moment between it and a girder bridge, he thought it was impossible to have designed better or more substantial piers and abutments than those of the Victoria Bridge. When piers were constructed for an arch bridge, it was not only necessary for them to have sufficient bearing surface upon the foundation, which would be all that was necessary for a girder bridge, but they must also possess sufficient lateral stability to meet the unbalanced thrust of neighbouring arches when unequally loaded. With a view to ascertain this, he had calculated the direction of the resultant for two cases. In the first, where one arch was taken as entirely unloaded, and the half of the adjoining arch most remote from it was loaded with 1 ton to the foot lineal on each line of way. The resultant cut the pier, at the level of the bottom of the river, at about 5 feet 6 inches from the centre line. In the other case, where one arch was supposed fully loaded, whilst those adjoining were unloaded, the resultant cut the pier, at the above level, at about 6 feet 6 inches from the centre line, being $2\frac{1}{2}$ feet from the outside of the masonry. He

[1] This information will be found in part in the Appendix to each Paper.—SEC. INST. C.E.

[2] *Vide* Minutes of Proceedings Inst. C.E., vol. xxvi., p. 302.

had calculated the stability at the level of the bottom of the river rather than lower down, because it increased so much at lower levels as to be clearly more than sufficient. The effect of the distance to which the resultant diverged from the axis of the pier was to throw an increased strain on the outer portion of the masonry. If the perpendicular pressure per square foot were taken at $2\frac{1}{2}$ tons, and if it were increased from the above cause to 8 tons, still, although that pressure was considerably within the limits which the material would bear, the divergence of the line of pressure from the axis of the pier should give rise to a slight curvature, an effect which, it was believed, had been observed in practice.

He would now refer to a few of the leading peculiarities of the superstructure of the Victoria Bridge. One of these was the circular abutting ends of the arched ribs; this was not a novelty, as it had been adopted in an aqueduct bridge across the River Calder; but still it might be desirable to inquire whether it possessed any advantages. The next point was the manner of filling in the spandrils. In most of the older bridges the space between the extrados of the arch and the roadway bearer was filled in with vertical pillars; whereas in the present instance they radiated to the centre of the arch. The third remark had reference to the horizontal beam over the top of the arch, and the peculiar expansion joint where it rested upon the pier. He thought the circular abutting joint at the springing of the arched ribs was not desirable for the following reasons:—When an arch was loaded irregularly, the curve of equilibrium no longer ran along the neutral axis of the material, but began at some point in the skewback, and diverging from the neutral axis to a degree commensurate with the irregularity of loading, often reached nearly to the extrados or intrados of the arch. Now, if the circular abutting joint were used, the above curve would be obliged to start from the abutment at the centre of the depth of the arch; whereas, if the arch started from a flat skewback, the curve of equilibrium might commence at some point either above or below the central line: it would not travel so widely away from the centre line, nor distress the material so much. There appeared to be a similarity of action in the case of an arch with flat ends, to pillars similarly formed, which, as all engineers knew, were much stronger than pillars with rounded ends. If the end of the arch, when flat, had a tendency to nip more at one point than another, the angle so pressing on the skewback had a tendency to correct the disturbance of the arch itself; and for these reasons he was inclined to prefer a flat skewback.

He did not think the radiating direction of the spandril-filling so good as the system with vertical pillars. In an arch in equilibrium, the load, at each equal division measured horizontally, came down upon the back of the arch vertically, and engaging itself with the forces already resident in the arch, produced a resultant strain passing along the axis; whereas, if the load were transferred to the arch through the medium of radiating pillars, the effect would be to throw more than its proper share of pressure upon the arch, in proportion to the inclination of the pillars towards the springing. It was true that in the design now under discussion, the inclination of the pillars was but small; still he thought, as a matter of principle, it was right to mention it.

It had been the fashion in recent bridges to make the horizontal girders above the arch very strong and deep, but he thought this member might safely be made lighter. In most of the older arched bridges the horizontal roadway bearers were made very light and shallow, and the supporting pillars vertical; and it was worthy of inquiry whether, if, in the present design, a portion of the metal were taken out of the girder, and transferred to the arch itself, greater stiffness would not be secured. No doubt these strong horizontal girders assisted to diminish the distortion of the arch, in a similar manner to that in which they corrected the disturbance of the curve in suspension bridges, but the degree in which they so acted was much greater in the suspension than in the arch bridge, in consequence of the far larger disturbance of the curve in a suspension bridge of the same span and versed sine with an arch bridge, with similar variation of loading. In a suspension bridge, unequally loaded, the chain arranged itself in the curve of equilibrium due to the load, while in the arch, although the curve of equilibrium underwent an equal change of figure, the arch itself underwent scarcely any alteration, at least, none in comparison to the chain; and this small movement constituting all the deflection which the girders above the arch would have to undergo, the strain against them would be proportionately small, and their corrective influence slight.

He would briefly refer to the expansion joint in the horizontal girders, which was intended, in the language of the Paper, to keep up "the continuity of strain." If by this it were meant that the effect of the expansion joint would preserve the same amount of strain, into which the horizontal girder would be thrown by contraction and expansion, he scarcely thought this result would be effected: for the India-rubber washers would have to bear an equal pressure to that of the metal of the girders, from an expansion due to half the variation between the highest and lowest temperatures, which half, if taken at 30°, would equal 2 tons on the square inch, or from 90 tons to 100 tons on each girder. The end of each girder would also, if allowed freely to expand with the above temperature, move through a space of about $\frac{1}{4}$ inch, but the girder would be subject to no strain; and some smaller amount of motion could only be counted upon. If this were taken at one half the above, or $\frac{1}{8}$ inch, the compression upon the iron would be only 1 ton on the square inch, or one half of what it would be were the girders attached immovably to the piers. His opinion that the piers probably underwent some slight deflection, in consequence of the departure of the line of pressure from the perpendicular, appeared to be borne out by the deflection of the arch under the load of 175 tons, or 1 ton per foot lineal of the arch. The deflection was stated to have been $\frac{7}{8}$ inch, but he considered this was double what it should have been, according to the data furnished; and he attributed the excess to the slight yielding of the piers. His conclusion, that the deflection was double what it should have been for an arch, was arrived at by computing the deflection of a girder 175 feet long and 17 feet 6 inches deep under a similar load, which he found to be about $\frac{7}{8}$ inch, and then assuming that the deflection of an arch should only be half as much. This followed from the fact that in the girder the deflection due to any element was as the compression or extension of that element taken into its distance from the point of support, and divided by the distance from the neutral axis, or half the depth of the girder. In the arch the calculation was the same, except that instead of dividing by half the depth, the whole depth had to be taken, giving, of course, one half the deflection only. It was, perhaps, worthy of inquiry whether a girder bridge might not have been constructed for less money, from the saving which might have been effected, under that system, in the piers and abutments and in their foundations.

With respect to the widening of the Victoria Bridge, the principal points for consideration appeared to be, the nature of the foundations, and the continuity of the horizontal girders above the arches, and the permanent bolting of them to the tops of the piers and abutments. Although cylinders of cast iron, sunk deep into the clay, formed an excellent foundation for supporting a vertical load, as in the case of a girder bridge, he thought they were not so well calculated to afford the requisite lateral stability for an arch bridge as solid piers. At or about the level of low water, the piers in both designs were of the same width, and at this level both were of the same class of material, but the stability of the piers of the widened portion was less than the others, in consequence of their resting only upon the base afforded by the four bearing cylinders, instead of bearing all along as in the original piers. This had the effect of reducing the area of the horizontal section at low-water level of 1,513 square feet by 396 feet, leaving 1,117 square feet as the total area; consequently the pressure per square foot of the brickwork was increased. There was also an evil connected with the apertures left between the cylinders, and between the last cylinder and the cutwater of the original bridge, as at high water there was an upward pressure of water equal to 176 tons on each pier, which of course lessened the stability. Had there been any saving in expense there might have been some reason for adopting the cylinders, but as the expense per superficial foot of the completed bridges appeared to have been identical, he should have preferred the solid pier. On the question of the continuity of the vertical girders, and securing them to the piers and abutments, there was an observation in Mr. Fox's Paper which almost seemed to imply, that the piers were wanting in lateral stability, and that the continuity of the horizontal girders was employed to aid the piers in this respect. Now, as an engineer, he would certainly object to erect an arch bridge upon piers of insufficient width of base and weight, to ensure the resultant of the strains falling far enough within the base to secure the necessary stability. If the piers were not so erected, he thought the action of the continuous girders scarcely competent to give them the requisite support: for whether the whole length of the longitudinal beam were either expanded or contracted from its length at a mean temperature, the whole of the strain would be taken upon the abutments, leaving the piers in a perfectly neutral condition, and at liberty to move through a small space before the longitudinal beams came into action. This alternate motion, in opposite directions, might loosen the iron standards to which the beams were secured, and disturb the brickwork, particularly at the abutments, where, with a variation of 30° each way from the average, a strain of about 2 tons on the square inch, or about 100 tons on each girder, would be concentrated.

Mr. R. P. Brereton thought the two structures under consideration were as similar as could be, although the observations of the Authors of both Papers would lead to the belief that the Engineers were influenced by different ideas when they designed these bridges; at all events, both bridges had answered the purposes they were intended to fulfil. The first had already been in use for ten years; and any misgivings that might have arisen, from the fact that the employment of light boiler-plate arches, without some means of stiffening, was comparatively novel, must have long ago been dispelled. The Engineer by whom the widening of the bridge had been carried out, had probably been induced to preserve a similarity in the appearance of the two structures. There was a slight difference in the construction of the piers: in the original bridge large coffer-dams were used, and there was a good base to each pier; in the other the base was slighter, and the piers were built without the employment of coffer-dams. The only other difference was that in the first bridge each horizontal girder was connected over the piers by an expansion joint, in which, by the use of India-rubber rings, under the nuts of the junction bolts, a certain amount of elasticity was obtained. The result of expansion and contraction, to the extent of about $\frac{2}{3}$ inch in length, due to a range of temperature of between 50° and 60°, which alone gave trouble in these matters, had produced a rising of the crown of $1\frac{1}{2}$ inch in each of the spans of the second bridge. As the whole top member was riveted together, it ought, theoretically, to have expanded considerably. But it turned out that the expansion recorded was due to the length only from the centres of the two end arches to the extreme ends of the girders; and the expansion, equal to $\frac{2}{3}$ inch to each intermediate arch, had disappeared; so that there would be no tension during high temperatures. He did not see the use of the upper member being very strong for the purpose of always resisting tension. There was no doubt that, when light iron bridges like this, weighing about a ton per foot run, or the same as the moving load, were irregularly loaded, they did not fall, and that something else was in operation than the mere arches. It could not be said that the piers did not tend to prevent such a result. They resisted a large amount of thrust; and there was no doubt the weight upon the pier-base of the first bridge was so insignificant that, if the whole of the load acted upon one half only, the pier would be stable. A light pier was enabled to bear the weight by the resultant forces being made to pass near its centre. One method of accomplishing this was by means of the flat girder arches acting as struts: these were very stiff, and the piers might be pushed hard against them, without producing a serious upward strain upon the girder not loaded with its full load. It was evident that in the second bridge there was no tension, in hot weather, upon the top ties of the middle arches. It was only to be got by such a contrivance as had been introduced in the first bridge, or by riveting up the girders at the hottest time of all. The numerous India-rubber connectors, $3\frac{1}{2}$ inches diameter and $1\frac{1}{2}$ inch thick, might be subjected to a constant strain. The range of length to be provided for in each of the spans was about $\frac{2}{3}$ inch, and there were four sets of India-rubber rings to be dealt with. If these expanded amongst themselves to the extent of $\frac{2}{3}$ inch, and a pressure was still maintained upon them, there would never be any compression upon the ties; while if the Engineer thought tension was necessary, that afforded an additional method of keeping a tolerably vertical resultant upon the piers. The same result might be effected by a strong, deep top member. This would lead to a considerable increase of weight being thrown upon the unloaded arch. Instead of cutting this member down he would have carried its full thickness over the top of the arch. It appeared the governing height of the structure was the headway required at the land openings. At the middle of the first arch, with the gradient of 1 in 60, there would be an increase of height of 3 feet. That would have enabled the whole roadway to have been raised 3 feet, and it might easily have been accomplished.

The introduction of light wrought-iron arches in large spans without diagonal trussing was first tried by Mr. Fowler in this bridge. He did not remember a previous instance beyond the Westminster Bridge arches, which were of small size; but in that case there were no heavy locomotive engines to be provided against. Wrought-iron arches of 100-feet span were introduced twenty years ago, and the span was gradually increased to 450 feet; but there were ties, and, therefore, a means of vertical and diagonal bracing; and by the use of these the arch was kept in proper shape. Mr. Fowler had departed from that system; in this instance there was a boldness of design which had been very successful.

Mr. BENJAMIN BAKER was not surprised that the innovations introduced in the widening of the Victoria Bridge had provoked criticism. In the first bridge the arched rib was of uniform section throughout, whilst in the widening it was reduced in section towards the ends, as a girder, the maximum transverse strength at the centre of the span being three times its maximum strength elsewhere. If the hypothesis on which that work was constructed were correct, he did not see how to avoid the conclusion, that the original bridge, in common with most other arched viaducts, was altogether wrong. Sufficient evidence had been advanced, in his opinion, to show that the departure from precedent in this instance was not attended with advantage.

Referring to the diagram showing the distribution of the load (Plate 6, Fig. 11), and the manner in which it was assumed to be carried, it was seen that the ironwork was supposed to carry the load partly as an arched rib simply, and partly as a continuous girder of varying depth, and the proportions in which it acted in the several capacities appeared to him to have been determined arbitrarily. Assuming, however, that such a distribution could be obtained in practice, he found, upon calculating the change of form which the ironwork would undergo in such a case, that the movement of the piers would be considerable, and the deflection much greater than was indicated—at least two or three times as great. He contended that it was physically impossible for the ironwork to act in the way shown on the diagram while the deflection was such as was stated. There could be no better authority than the structure itself, as to the mode in which the work was done. The ironwork showed that it was doing duty simply as an arch, and that it was not acting in the roundabout way indicated on the diagram. Ample evidence was afforded by the original bridge of the practical immobility of the piers; for if each pier had moved but $\frac{1}{8}$ inch, that movement would have been registered by an additional deflection at the crown of $\frac{5}{8}$ inch—an amount which could not possibly escape observation; and yet the additional strain thrown on the metal by such a movement of the piers would not exceed 7 cwt. per square inch.

His attention had been directed some months ago to the anomaly presented by the two bridges, and he had made a careful analysis of the strains to which each bridge was liable. He first ascertained that the piers were immovable by observing the deflection. That in the original bridge did not exceed $1\frac{1}{16}$ inch—the amount due to the compression of the arched rib, and consequently there was no balance to show movement of the piers. He next satisfied himself that, after the rounded shoe had taken its bearing, under a full load, no further movement need be anticipated. The radius of the end was too large, and the co-efficient of friction was too great, to admit of further movement. The rounded shoes and cotters had done important duty, in securing uniformity of strain under a full load; and beyond that, it was rather an advantage that they should not act. Having arrived at this point, the solution of the problem was comparatively simple, and the results reliable. The case was that of an arched rib fixed at the ends, and with stable piers. At the mean temperature the maximum strain, with a rolling load of $2\frac{1}{2}$ tons to the foot, uniformly distributed over both lines, would be 3 tons 14 cwt. per square inch. The maximum bending moment occurred at the unloaded haunch of the arch, when about $\frac{6}{10}$ths of the span were fully loaded. At the mean temperature, the resulting maximum strain would be 3 tons $16\frac{1}{2}$ cwt. per square inch. The maximum bending moment at the centre of the arch would occur when about one-half the span at the centre was fully loaded, the two haunches being light; and the maximum resulting strain at the crown of the arch would be 3 tons $16\frac{3}{4}$ cwt. per square inch, at the mean temperature. These strains would be increased by the expansion and contraction of the ironwork at extreme temperatures. It was considered that $\frac{1}{2}$ inch variation in length for each 100 feet was sufficient allowance for the range of temperature in this climate, and on that assumption the strains at extremes of temperature would be increased to 4 tons $6\frac{1}{2}$ cwt. per square inch at the haunch, and 4 tons $9\frac{1}{3}$ cwt. per square inch at the centre of the arched rib. It appeared then, that the maximum compression on any cross section of the arched rib averaged under $3\frac{3}{4}$ tons per square inch, and that the maximum stress on any part of the cross section was under $4\frac{1}{2}$ tons per square inch. If any question arose as to the strength of the original bridge, it could only be with reference to the factor of safety. It would be seen that the relative proportions of the several parts were correct. The horizontal girder, combined with the spandril-filling, was of the requisite strength to relieve the arched rib of the excess of strain at the haunch.

Returning to the widening of the bridge, as the spandril-fillings did not possess any bracing power, the detail being different to that in the original bridge, the load would be carried partly by the arched rib as an arch proper, and partly by the arched rib and the horizontal girder as continuous girders. By calculating the relative deflection of the two systems, it appeared that the arched rib carried $\frac{9}{10}$ths of the gross load as an arch, the remaining $\frac{1}{10}$th of the load being carried by the continuous girders. At a mean temperature, and with the same rolling load as before, uniformly distributed, the maximum strains would be 3 tons $15\frac{1}{2}$ cwt. per square inch at the crown of the arch, 5 tons 17 cwt. per square inch at the springing, and 1 ton 19 cwt. per square inch on the horizontal girder over the piers. Under the most unfavourable condition of loading, the maximum strain at the crown of the arch would be increased to 4 tons per square inch. The influence of changes of temperature in this instance was very important. The horizontal girder being in one piece, nearly 1,000 feet long, it followed that, if it expanded at all, it would do so towards the two land openings. In that case it would have to carry the tops of the piers and the crowns of the arches with it. If the tops of the piers were movable this might possibly happen, and then the increased strain due to the changes of temperature would attain a maximum at the end arches of the viaduct, and be equal to 3 tons 10 cwt. per square inch. As the piers were not movable, this expansion and contraction of the horizontal girder could not take place; and, that being so, the strains on it with the same range of temperature as before would be 1 ton 6 cwt. at the centres and 2 tons 14 cwt. over the piers. The final maximum strains on the several portions of the ironwork under the worst conditions would be as follows:—4 tons 11 cwt. per square inch at the crown of the arch, when the centre half of the span was loaded and the temperature high; 6 tons 14 cwt. per square inch at the springing of the arch, when fully loaded and the temperature low; and 6 tons 15 cwt. tension per square inch on the horizontal girder, under the same conditions. These amounts were much less uniform than those obtained for the original bridge, owing to the mass of metal which had been thrown into the centre of the arched ribs for the widening. Apart from the results due to a false hypothesis, it appeared that the constructional inferiority of the widening, as compared with the original bridge, was evident in at least two particulars. Thus a large proportion of metal had been thrown into a horizontal girder not free to expand or contract, and where, consequently, it was exposed to perfectly useless strains, amounting to 60 per cent. of the total stress which should have been incurred by the metal; whereas, if the same weight of iron had been placed in the arched rib, it would have been equally effective as far as resistance was concerned, and the strains from changes of temperature would have been neutralized to a great extent by the rise and fall of the arch. Again, a considerable percentage of the gross load was carried by a girder 4 feet 6 inches deep; whereas, had the arched rib been cottered up judiciously, as was done in the original bridge, that load would have been supported, with a far less expenditure of metal, by an arch of 17 feet 6 inches rise.

Mr. F. W. Sheilds agreed that these bridges ought to be considered in the light of a perfect arch throughout, rather than as combined arch and girder. With respect to the horizontal beam on the top, with radial bracing in the spandrils, he thought it would be a more advantageous principle of construction to lighten the top longitudinal beam, throwing the metal so saved into the arch, if a great amount of stability were required, and to unite the longitudinal beam and arch by a more perfect method than could be obtained by the radiated spandril bars. He thought this union would be best effected by a spandril formed of solid plate or latticework, which would distribute the pressure of the passing load, especially that from the driving-wheels of the locomotive, over a considerable length of the arch, rather than by transferring that pressure to single points on the arch by means of detached and oblique connections, such as the radiated spandril-bars.

The bearing of the arched rib upon the piers and abutments naturally drew attention to the contrast between the curved skewback in the original structure, and the ordinary skewback adopted in the adjoining bridge. Now, would such an arch move upon its bearings? It seemed to him that when once fixed in its place, it would never afterwards be strained to such an extent, either at the top or at the bottom, as to be raised off its bearing. In an ordinary skewback, it was evident that if the crown of the bridge was depressed, either by contraction in cold weather, or by a load on the top, there would be a tendency to an increase of pressure on the lower end of the bearing, and a diminution of pressure on the upper end; and the opposite effect would occur when the bridge was expanded from heat; and this would take place whether the skewback were curved or straight. But the arch would not, in practice, alter its position, either by rolling round the curved bearing, in the one case, or by partly rising off its seat in the other; and he, therefore, preferred the straight bearing as being the simpler construction. In the case of a bridge with many arches, he thought it would be better to make the springing over the head of the piers of iron rather than of masonry, so as to unite the arches perfectly together, and distribute the strain on any one loaded arch over the adjacent arch or arches. He thought, moreover, that there were additional advantages accruing from this arrangement. Besides obtaining a construction of homogeneous material, the fitting of the bearings or springings in the workshop could be done with a greater degree of accuracy, and the strain on any loaded arch would be thereby better distributed, not only on its own bearings, but also over the adjacent arches, as was shown by the latter rising at the crown in the manner mentioned in the Paper.

Mr. G. W. Hemans asked whether Mr. Sheilds suggested that the continuation between the bearings of the two arches should slide on the top, or be fixed to the masonry of the piers?

Mr. Sheilds: the bearing would not slide in practice, though when under heavy pressure it might make a slight pier bend over at the top, but it would form a vertical bearing on the top of the pier, and the thrust from one arch would be transferred more perfectly to the other. He held that the unloaded arch should aid in supporting the loaded arch, by means of a more perfect connection over the heads of the piers. There was no doubt that one of those slight piers alone would be insufficient to support the weight of a loaded arch if the adjoining arch were removed. He would therefore ensure, by perfect fitting and homogeneous construction, that when one arch was loaded, the metal of the adjoining arch or arches should be brought into strain to support it.

Mr. J. M. Heppel quite concurred with those who thought that there was little substantial difference in the mechanical condition of these structures. It was almost evident, from the deflections which occurred when placed under load, that there could have been no material difference in the way the strains were propagated through them. He thought there was a close analogy between the principle adopted in the original bridge and that of a stiffened suspension bridge. Mr. Wilson had said that the arch, by means of the peculiar springing, was placed under such conditions, that the strain through it was always uniform over its section: and it seemed to him that it assumed the condition of a chain, prevented from deflecting by some stiffening adjunct, such as a horizontal girder. In reference to this it had, however, been remarked, that the horizontal girder was in fact not acting very effectually, as was proved by the slight deflection that took place. In that opinion he concurred. It seemed, further, that whatever might have been predicted, *a priori*, the experiments showed from the trifling deflection of the horizontal girder, that it could not be doing much work simply as a girder. The stiffening of the arch, which seemed to be perfect, was, he thought, chiefly due to the connection of the horizontal member with the arch by the spandril-filling. This, to a certain extent, answered the purpose of the bracing ordinarily inserted between the top and bottom members of a beam.

He thought this bridge would have been improved, if a portion of the metal had been taken out of the horizontal member and added to the arch, and the connection between the two had been made more perfect, either by diagonals or by a continuous web. If the details of the deflections were examined, it would be found that the action resembled that of a stiffened suspension bridge, as was shown in the case of the Clifton Bridge.

The extent of the alterations of the spans was indicated by the rise of the unloaded arch, which was a scale that measured the motion in the springing of the arch. He thought that, in this case, the rise of the unloaded arch was about twice the horizontal motion at the springing, and that the piers at the springing moved backwards and forwards about $\frac{1}{16}$th of an inch, which proved that they possessed a great amount of stability. This did not depend upon the through tie; as before that could act effectually the piers must move through a considerably greater space. In fact, it seemed to him that Mr. Fox gave the piers less credit for stability than they deserved, and that, though the horizontal member might be capable of giving stability, it was, in the actual condition of things, not much drawn upon. Mr. Brereton had stated, that as long as the resultant was kept well within the base of the pier, no danger need be apprehended: but he thought so large a deviation as that which Mr. Phipps attributed to it would produce a serious disturbance of the distribution of the load on the foundations. He had occasion to investigate the question in the case of the proposed bridge over the Frith of Forth, where it was important to know what maximum amount of pressure would be brought to bear on any part of the foundations; when it appeared, that if the resultant were to fall so much as halfway between the centre and the edge, its amount at the latter point would become trebled. In some cases this might be serious; but he believed the London clay was a stubborn material, capable of supporting great pressure, and it was possible that a larger deviation from the centre might exist without any fatal disturbance.

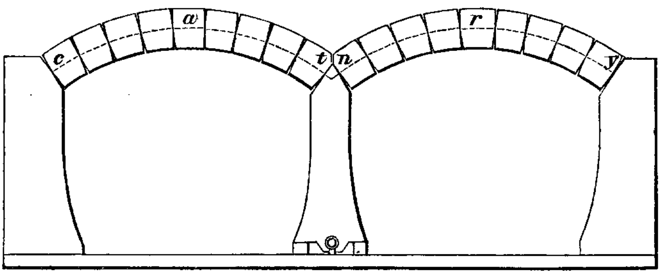
Mr. Brereton remarked that half the size of the present pier would have been sufficient to carry all the weight that came upon it.

Mr. E. A. Cowper hoped the Authors of the Papers would lay down the principles on which these bridges were constructed. It would be seen that, in the original bridge, the long girder, with India-rubber washers, was in balance, and could move either way, on a slight strain being applied longitudinally; therefore it was impossible it could do any real work in maintaining the bridge. He did not think this girder put the half-arch into the condition of a cantilever, having the rib of the arch in compression and the girder in tension; but that it slightly added to its strength by increasing the depth of the arch by means of the spandril-filling. In the second bridge, when the top girders were riveted together, they were in the same state of tension at each end, and therefore in balance; and if one arch was depressed, the next would rise, and there would be no perceptible alteration in the horizontal distance between the crowns of the arches, and no strain on the top girder. The piers would bend if there was more pressure against one side than the other. He had been on the top of a chimney which rocked an inch in a moderate wind; some chimneys would vibrate as much as 4 inches at the top; and he had known a large square chimney to move $\frac{1}{8}$ inch at 16 feet from the ground. With a foundation on clay at the bottom of a river, a high pier, with a moderate base, would certainly spring some distance, under a greater pressure on one side than on the other. If, however, it were now considered that the piers were infinitely strong and stiff, which was assuming an impossibility, it was clear that no horizontal tie could do any work, as its length would not alter. Nor with such piers could the half arches act as cantilevers, unless the arch were cut at the key-stone, and a piece were taken out to allow each half arch to hang from its pier as a cantilever, so that the horizontal tie could come into action when the load came on to the bridge. It was quite possible to imagine such a structure, though impossible to make it; indeed, if it had been possible, it would have been inadmissible; no engineer would dream of running a heavy load off the end of one long deflected cantilever on to or against the end of another long undeflected cantilever. It was a pity that, when the Institution was furnished with the general particulars of successful works, the theory of their construction was often left out, and the practical results only brought forward. In the present instance it would have been interesting to have heard by what principles the Engineers had been guided. He ventured to say that no bridge could be considered economical or safe, if the line of strain passed outside the rib or stones; and if so, it became of the greatest importance to ascertain the exact curve that the strain took, when the bridge was in any one condition of loading that would occur either under ordinary or extraordinary circumstances. This was precisely the same as in the Inverted Arch Bridge, which he had formerly described;[1] and the curves of strain that had to be provided for might be ascertained by the process he then mentioned. He maintained that the rib of the bridge should be made to contain all such curves within it: then, if it was constructed with flanges, and each flange was made capable of taking the whole strain, the rib would be safe, and perfectly stable if properly stayed side-ways and on a good foundation. By such a mode of calculation a perfectly stable rib could be constructed: it had been put in practice largely by himself in calculating the Richmond and Barnes Bridges, and numerous others; and nothing had occurred to shake his confidence in this mode of calculation, though he knew it was not commonly used. The only useful

[1] *Vide* Minutes of Proceedings Inst. C.E., vol. xxvi., p. 281.

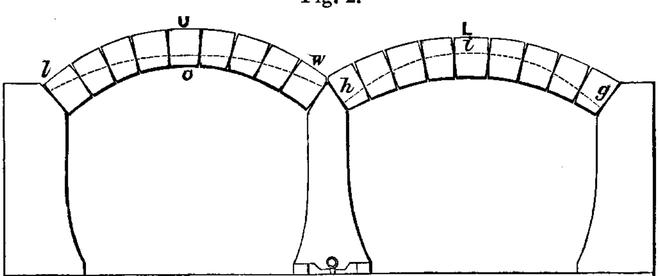
support that could be afforded by the spandrils was that, taking advantage of the metal being there to support the platform, it might be placed in diagonal lines, so as to assist, to some extent, in preventing the arched rib from bending. He believed that was the true way in which to calculate the metal in a bridge of this kind, and he thought if wrought iron must be used, it ought not to be considered as any better than cast iron, for it was not so able to resist compression, and this was the strain that the metal of an arched iron rib was called upon to bear. The rough wooden model (Fig. 1) would show the way in which one arch

Fig. 1.



supported the other when one arch was loaded and the other not. The centre pier was capable of motion on a pin at the base, but was supported by blocks of India-rubber, to represent the condition of a pier capable of some motion. The arches were catenary curves, and the arch-stones were slightly rounded on their beds, so that they could roll a little, to make the motion visible to the eye. The line of strain or line of contact of the rounded beds was a catenary curve, *c, a, t, n, r, y,* and it passed through the centres of the stones. But if the first arch was loaded, and the second arch left unloaded (Fig. 2), it would be found that the catenaries

Fig. 2.



were distorted to a considerable extent;—and that while the first arch was flattened the second was raised. The exact line of contact of these stones was shown to be on the line *l, o, w, h, i, g*, on the faces of the stones in the model. Now it was evident that the first, or loaded arch, L, was flattened by the load, and the outside edges of the stones thereby brought nearer together, so that the line of contact *h, i, g* was higher at the key-stone. It also appeared that the same flattening of the curve threw out the bottom edges of the stones against the skewbacks and lowered the line of contact at the springing, raising the curve of strain *h, i, g* higher than before. In the second, or unloaded arch, U, the reverse took place; so that the line of strain *l, o, w* was flattest in the arch which was highest, and the line of strain *h, i, g* was highest in the arch which was flattest. This would be so whether the arch were of stone or of iron; though in the case of a stone bridge the distortion would be less, owing to its greater weight. The unloaded arch supported the thrust of the loaded arch because the line of contact *l, o, w* was a flatter curve than the line of contact *h, i, g* of the loaded arch; therefore the horizontal thrust of the two was the same, and they were exactly in balance, and perfectly stable, the pier being free to move when strained; and if he took the blocks of India-rubber away from under the pier it would still stand. It was hardly necessary to observe, that in a stone or iron arch a very small motion or distortion would at once bring the curved line of strain to one side or the other of the rib; so that a minute yielding of the centre pier would enable the thrust of one arch to balance the other. The more clearly to show that the stringer, or horizontal girder along the top, could not do much duty, he would fasten upon the crowns of the arches in the model a stringer of paper with two pins, so that if the distance between them were altered at all it would tear away from these pins; but on repeatedly depressing first one arch and then the other, it would be seen that the paper was not in the least affected; therefore there could be no strain upon the horizontal tie. Again, if the paper tie were formed to represent the spandril-filling, as well as the tie, and were also fixed to the skewbacks between the arches, it would still move with the arches, and there would be only the smallest possible motion at the bottom of the spandril. In conclusion, he would draw attention to the admirable cast-iron bridge by Telford over the Birmingham Canal, called the "Summit Bridge;" to Southwark Bridge, constructed by Rennie; and others that were made when iron-founding was young in this country; and he must say that, whatever special causes there were for the Victoria Bridges being made of wrought iron, he maintained that cast iron was the right material for arched bridges of large or small span; and, now that iron-founding and fitting could be done in such perfection, they were undoubtedly the cheapest to make, the easiest to fit, and, owing to their greater weight, the firmest when up; besides which, they would last much longer.

Mr. William Bell remarked, through the Secretary, that if the piers and abutments were absolutely immovable, a simple arch rib would have been sufficient, because the increased thrust arising from loading one of the arches would have been resisted by the immovable springers, while the variations of temperature would simply have caused a variation in the rise of the arch-rib, and would have produced no internal strains, except those arising from the resistance to rising or falling of the top platform, which might have been made of small amount. If, by the increase or decrease of the forces acting on them, the piers and abutments were supposed to be subject to small displacements, either from their being considered as elastic beams of masonry, or from the compressibility of their foundations, the question became complex, and could not, he thought, be definitely answered, without an intimate knowledge of the amount of these small displacements. Referring, first, to Sir C. Fox's design, if the bridge were held to consist virtually of a series of cantilevers, the horizontal beam at the top of any semi-arch might be considered as a tie-beam, and thus must be sufficiently strong to sustain, by its tensile strength, the horizontal thrust of the loaded semi-arch with which it was connected. This semi-arch would then be subjected to the same internal strains, as if, instead of the tie-bar, the horizontal thrust had been balanced by the thrust of the opposite semi-arch. But with these tie-bars, the bridge would consist of piers and abutments, with cantilevers projecting from them, the under parts of these cantilevers having the shape of semi-arches. Each arch might thus theoretically be supposed to be severed at the crown, and if the piers and abutments had sufficient stability, without their being absolutely immovable, an expansion joint might be placed there, and no internal strains would be produced from the variations of temperature. This construction would have the advantage, that arrangement might be made for any pier or abutment to settle considerably without thereby endangering the safety of its neighbours; but it should be remembered, that to obtain this advantage, the top horizontal beam must be made thoroughly efficient as a tie-bar, in which case it would require a sectional area not much less than that of the arch-rib itself.

The special purpose, however, as he understood it, of the horizontal beam, and which was first carried out in Mr. Fowler's design, was that of making it resist the additional horizontal thrust produced by loading any of the arches, where, from the situation, it was obvious that a system of direct ties could not have been introduced at the springings of the arches.

The piers and abutments being supposed capable of movement, if the temperature were invariable, and the horizontal beam simply laid on the top, without being connected with the arches or the piers, the loading of one of the arches would produce a compression of the arch rib, with a diminution of its rise, and a horizontal thrust at the springing of the arch. This horizontal thrust would push the piers outwards, until it was balanced by their increased resistance to lateral movement, and the resistance of the arches to alteration of their form, which last would be modified by the resistance of the top platform to bending consequent on the alteration of rise of the arch.

Referring to Sir C. Fox's design, if the horizontal beam were firmly connected with each pier, the piers could not be pushed outwards without the beam being extended along the loaded span, and compressed along the unloaded spans, thus introducing forces of tension and compression, which would assist the forces of the piers and arches in neutralizing the thrust of the loaded arch. It should be borne in mind, that there must be a certain movement of the piers before these forces could be called into action. If now the temperature was supposed to vary, the horizontal beam could not change its length without causing displacement of the piers, the amount of this displacement being determined by the condition that the forces generated in the beam by alteration of temperature should be equal to the resistance of the piers and arches to further displacements, and thus the strain due to change of temperature might be much less than what would happen if the horizontal beam were supposed to be fixed at each end. There would also be a mitigation of the strain, arising from the fact of the rather greater extensibility and compressibility of riveted as compared with solid iron. Very little of the strain could be taken up by the beam bending vertically or laterally, as it would be found that it required considerable sinuosities to produce an appreciable alteration of length. If the horizontal beam were connected with the centres of the arches only, it would also be indirectly connected with the abutments, by means of the outside semi-arches, and the strains arising from changes of temperature would then be due to the difference between the motion of the abutments, including the giving of the end semi-arches, and the motion of the end of the beam supposed free, on a length of beam reaching between the crowns of the end arches. In this case the strains would possibly be greater than if the horizontal beams were connected with the piers.

Now suppose, as in Mr. Fowler's design, that the horizontal beam was severed over each pier, and the parts connected together by springs, capable of extension or compression; and suppose these springs were of such force as to double the compressibility or extensibility of the beam; that was, if a force of 1 ton were applied to the ends of the beam, the amount by which it would be compressed or extended would be double that which would take place if the beam were riveted up from end to end. Such a beam, if fixed at the ends, and subjected to changes of temperature, would only encounter the halves of the strains which would happen if the beam were in one piece, because the parts of the beam could now alter their lengths by compressing or extending the springs, until the forces of spring and beam were equal, which, since they were supposed to have equal extensions, would happen when the strain was half what it would have been if the beam had been in one piece. By the use of these springs, then, the strains arising from changes of temperature would only amount to one-half what they would be under other circumstances; but, on the other hand, the amount of motion necessary before a given amount of force could be got out of the beam, must be double what would be necessary if the beam were in one piece: thus with beams constructed with springs, the piers must be supposed to have a greater amount of motion before the force necessary to balance the thrust of the loaded arch could be obtained. But the necessary amount of force could also be got by doubling the sectional area of the beam. It was easy to see that by the use of springs of less force than in the above example, the strains arising from temperature might be reduced still further; while, on the other hand, the motion of the piers must be supposed to be increased, or the sectional area of the beam made greater, in order to get sufficient force to balance the thrust of the loaded arch, and *vice versâ* with springs of greater force.

The main element then in the solution of the question, was the amount of motion of the piers and abutments, without a knowledge of which it would be idle to make detail calculations; but if, treating the piers and abutments as elastic beams, there existed reliable data on the stiffness of built-up masonry, and the compressibility of foundations, there was sufficient knowledge of the properties of iron arches and girders to allow of a definite solution being arrived at.

Mr. E. W. Young was acquainted with some of the reasons which had guided the Engineer in the form of construction adopted in the widening of the Victoria Bridge.

It had been remarked that two engineers had adopted different principles of construction in a similar case; but this was not exactly the fact, and he wished to point out in what respects the two cases were different. The span, the rise of the arch, and the load to be carried were the same; but in the widened portion a junction with old work had to be effected, and a very wide bridge made, which somewhat altered the conditions. In the first place, there was greater rigidity and stability in each pier, as a whole, in the wider bridge, and further, as in the junction with the old work, great expense would have been incurred in making coffer-dams and constructing piers, similar to those of the old structure, the engineer advocated cylinder foundations. They were adopted, and piers built, which, though not so stable as the older piers for each yard of their length, were yet very firm from their great length. However, it was determined, if possible, to make the ribs better able to resist the effect of unequal loading. With a view to this, the ribs were deepened in the centre, and a large proportion of metal put into the flanges, so as to give a great amount of stiffness to the unloaded arch, to enable it the better to resist the thrust of the loaded arch. In the original bridge the depth of the rib was 3 feet 6 inches throughout, and there were two webs: in the widening there was only one web, but the rib was 4 feet 6 inches deep at the centre of the span, and there was much more metal in the rib at the centre than at the haunches. He had compared the stiffness of the two ribs; multiplying the area of the metal in the flange by the depth of the rib, 280 represented the stiffness of the rib in the widening; whereas by the same process, the relative stiffness of the original bridge would be found to be only 150. Another advantage of making the flanges of the rib of greater sectional area at the centre was that the curve of equilibrium might pass through either the top or bottom flange without causing a dangerous strain upon the metal. The area of the rib was 64 inches both in the top and the bottom flanges, so that the curve of equilibrium could pass through either flange without injury to the metal. The advantage derived from the stiffness of the rib was this:—that in the unloaded span it offered resistance to the horizontal motion of the pier caused by the loading of the adjoining span. The motion in the pier caused by the rising of the unloaded arch to the extent of $\frac{1}{8}$ inch, which was the rise produced by the test load, represented a horizontal motion of the pier equal to $\frac{1}{20}$ inch. It was expected that this bridge would act as an arch, on account of the great stability of the piers due to their width; but the bridge was, nevertheless, made strong enough to act as a continuous girder, supposing the extremely unlikely circumstance to occur of the bridge being loaded with locomotives over the whole width of one archway. It would not then fall down, even if the pier were to be pushed over; for then the ironwork would act as a continuous girder. Another difference in the two structures was with regard to the loading. The earlier bridge had a heavy platform, comparatively speaking, so as to keep the line of thrust within the piers; in fact, the dead load was $1\frac{1}{4}$ ton per foot run; whereas in the latter it was only three-quarters of a ton. This heavy dead load in the first bridge rendered a greater area of metal necessary to carry it than was required in the second. In fact, in the one case, he calculated the weight of metal in a span to be about 100 tons for each line of way, and, in the other, about 130 tons. That, of course, affected the question of cost. Then there was the heavier platform of the first bridge, which also added to its cost, and, as he considered the design of the pier in that bridge more expensive than the cylinder piers, he thought the last constructed bridge the more economical of the two. Under the alternations of heat and cold the horizontal girder was subjected sometimes to a compressive, at others to a tensile strain. This simply caused the rib to rise, but it did not prevent the bridge acting as a continuous girder, because, before the bridge could fall down, it would have to put a tensile strain on the bottom flange, or a compressive strain on the top flange exceeding the strength of the iron. It had been said that the ribs could not act as a continuous girder because of the spandril, there being no web; but he considered it as a cantilever having one straight top member and one curved bottom member, and, as the point of contrary flexure would not be in the spandrils, there was no necessity for a web.

An opinion had been expressed that it would have been better if the spandril-bars had been perpendicular: but there was one advantage in placing them in the position they occupied, inasmuch as the curve of equilibrium became the segment of a circle, because the pressure was perpendicular to the rib. As in a cylinder sunk in water, the curve of equilibrium of pressure passed through the centre of the metal of the cylinder: so in these bridges the curve of equilibrium was the segment of a circle, and not a parabola or a catenary. He wished to show that the horizontal girder resisted the thrust of the arch, because, supposing the horizontal girder to do its duty, before the rib could break the pier, it must push it over. In the model brought forward by Mr. Cowper, he thought if the cardboard had been fastened at the lower end, it would have come into play, the lower part of the cardboard being now free to move.

In this case there was a bending motion; and if a perpendicular line were drawn down the face of the pier, the moment one arch was depressed, that line would be bent, showing thereby that the pier would have to break. In the more recent bridge, the cast-iron standard was bolted to the bottom of the horizontal girder, and unless the pier were broken, it could not be pushed over; on that account, he considered the horizontal girder did duty in helping to resist the horizontal thrust of the rib.

Mr. Cowper asked if the pier was supposed to be stiff? If it had a strong vertical girder he admitted Mr. Young's premises.

Mr. Young: It was stiff to a certain extent, and it must be remembered that motion to the extent of the $\frac{1}{20}$th part of an inch at the springing, would produce a larger movement at the top of the pier. He did not say that it made an effectual resistance; but to some extent it helped in resisting the horizontal thrust of the rib. If he had now to design such a bridge as this, he should make the ribs of cast iron.

Mr. J. A. LONGRIDGE expressed a hope that the Authors of the Papers would, in their replies upon the discussion, state the specific reasons which led them to adopt wrought iron instead of cast iron for an arched bridge. He thought stone was the proper material to use except under circumstances where it could not be got, or only obtained at a great expense: but if iron were used, he considered cast iron the more suitable material, inasmuch as cast-iron resisted compression better than wrought iron, and all the material of the arch was in compression. Besides, cast iron was much cheaper, and for the same cost a greater mass of metal could be put into the bridge, thereby adding considerably to the stiffness, which was an important element in railway structures. He had no confidence in the combination of the girder and arch principle. He was of opinion that the horizontal girders did not strengthen the bridge, and that if the same amount of material had been put into the arch ribs it would have made a stronger bridge. He could not think that wrought iron had been adopted in these structures, and others now in progress, from any fear that the foundations might fail. If the foundations were bad, the girder was the proper form of bridge. He should like to be informed how the motion of $\frac{1}{20}$ inch in the abutments was measured; deducing it from the vertical rise of the arch was, he thought, fallacious; but that amount of motion could do no harm, even if it took place in a stone bridge. In conclusion, he expressed his decided opinion, that the adoption of wrought iron in these bridges was an entire mistake.

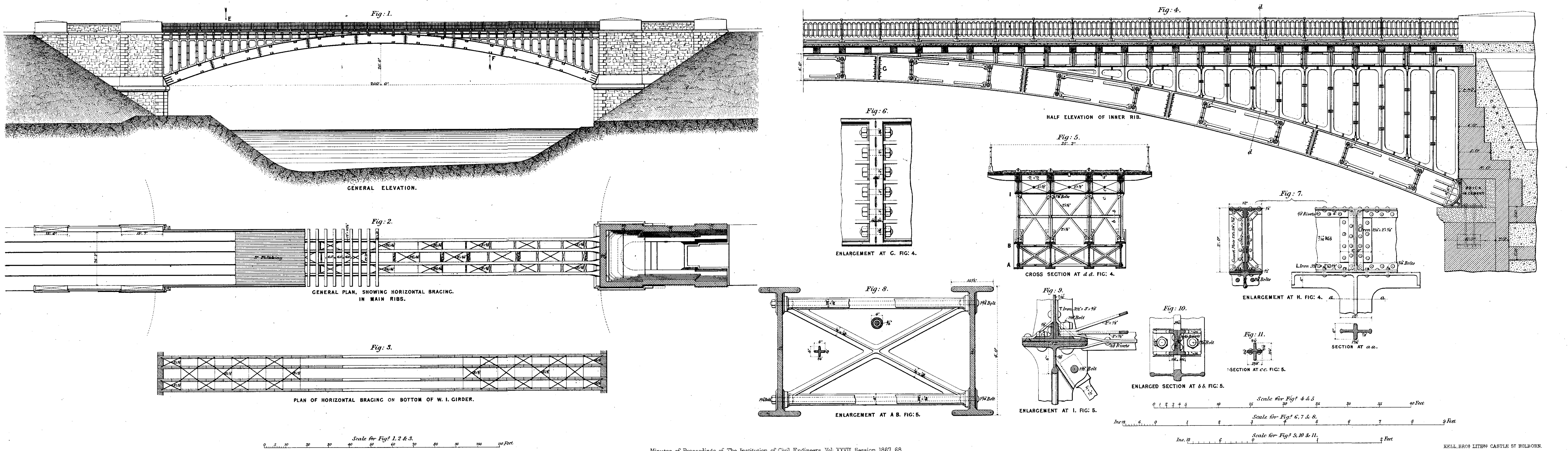
Mr. G. H. PHIPPS asked whether, in loading and computing the strains on these arches, every line of railway had been taken, as bearing its full train-load, or whether only one line at a time had been taken. He should have thought the computations would have been founded upon the full loading of all the lines.

Mr. YOUNG had assumed every line to be loaded with locomotives, and had endeavoured to show that, under that condition of loading, the bridge could not fall down, and he believed that, even when all the lines of rails were thus loaded, it would still act as an arch.

Mr. J. D. BALDRY said that a sentiment similar to that which animated Mr. Longridge had passed through his own mind; and it occurred to him that it might be interesting to place on record the particulars of a cast-iron bridge, of a larger span than the bridges under consideration; and by permission of Mr. John Fowler, Past-President, Inst. C.E., he would do so. There were two bridges of this description—one on the Severn Valley Railway, the other on the Coalbrookdale Railway, and both were erected over the river Severn. Plate 7 represented a bridge with an arch of 200 feet span, having a rise of 20 feet, and it was probably the cheapest bridge of the kind ever erected. It was built

SEVERN VALLEY BRIDGE.

FOR CARRYING THE COALBROOKDALE RAILWAY.



J. D. BALDRY, DELT

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subsequently to the Victoria Bridge. At first it was proposed to make the arches double, coupled together so as to form hollow voussoirs: but that plan was abandoned, partly because of the simplicity of construction of a single arch, and partly because a less amount of metal would allow the necessary depth, which should preserve the line of pressure within the arch, clear of the top and bottom flanges. Each arch was cast in nine parts, each piece weighing between 5 and 6 tons, and braced together as shown in Plate 7, Fig. 8, with two cast-iron frames having strong bolts, which were keyed up closely. The ends of these segments were planed and bolted together, the upper and lower bolts being of larger section than those in the middle. Fig. 6. The strain per square inch was calculated at $2\frac{1}{2}$ tons; this became afterwards somewhat increased by the addition of the full thickness of ballast. The casting into which the heel of the arch was placed was similar to the one used in the Victoria Bridge, Pimlico, except as to the steel keys. With regard to the use of this shoe there was no doubt that, during the course of erection, it was of great advantage, because it enabled the arch to take its proper bearing and adjustment, and facilitated the erection. In the Victoria Bridge he believed the steel keys or wedges were introduced to give the exact position; but in the present bridge there being no keys, the shoes were secured to the abutments by heavy bolts. After erection, the shoe ceased to be of use, so far as its circular form was concerned, because the rise and fall of the arch was not sufficient to give motion in the shoe; for instance, a variation of temperature of 100° would cause a movement in this bridge of 200 feet span, of about $2\frac{1}{2}$ inches at the crown. Now the rotation of the heel would be, supposing no friction, about $\frac{1}{16}$ inch; but the friction of the surface being considerable, this motion did not take place; indeed, a much larger amount could be absorbed by the elasticity of the metal; therefore, the only practical result from the rise and fall of the arch was the transference of something less than half a ton per inch, from the upper to the lower limb, and *vice versâ*. The next point to be noticed was the straight line which carried the load. In this case a wrought-iron girder was used. One reason for this being made of wrought iron was that if an exceptional load came on at E, Fig. 1, and the other side were unloaded and perhaps denuded of ballast, there would be an upward thrust at F, Fig. 1; but, by the close connection of this wrought-iron girder with the cast-iron arch, there was a great depth of girder to resist it, the wrought iron being in tension. This was not a case likely to happen, but he thought there was no harm in providing for it, as it was a weak point in these bridges. This girder was strengthened in the vertical web, at the points of contact with the columns, at H, Figs. 4 and 7, to resist the hammering effects of the passing loads.

With regard to the spandril columns which transmitted the load, some were rather long—about 17 feet; they were 4 feet apart, and strengthened at intervals by struts. These struts were continued through the bridge (Figs. 4 and 5), and being aided by vertical diagonal bracing, gave lateral strength to the bridge itself.

One of the essentials in constructing bridges of this description, was that the arch should be kept to its work by means of bracing and strutting, somewhat of this nature; which, more than by any other means, would prevent the vibration so much complained of, but which was inseparable from iron bridges. He had nothing to say in favour of this particular bridge, except that it had been carefully studied, and he thought every part was doing its proper duty: he might mention that the deflection from the passage of a train had recently been observed to be $\frac{3}{10}$ inch. He would endorse what had been said, as to the use of iron where masonry could be obtained; but it was for the engineer to consider, when the work was before him, whether he should adopt metal or stone, and whether cast iron or wrought iron. There were often excellent reasons for using wrought-iron arches, particularly where a bridge was of several spans, and the piers were narrow, as in a strong tide-way; but for single spans, he had no hesitation in saying, that the mass of metal that could be placed in one thickness gave a great advantage to cast iron, irrespective of the question of cost. Beyond certain spans, metal took the place of stone by necessity; but, for small spans, it appeared to him that, taking into consideration the vibration and the disturbing forces of the moving load on light bridges, stone was the proper material.

Captain Tyler said it had been his duty to inspect and test the bridge over the River Severn which had just been described, and he was happy to bear testimony to its excellence. It was, in his opinion, a great improvement upon the Victoria Bridge previously constructed over the River Thames. It combined stiffness with cheapness in a remarkable degree, and he had been, for that reason, much struck with it. He thought there was an advantage in the employment of cast iron, in the place of wrought iron, in bridges of this description. In the first place, wrought iron was less durable, and required more painting; in the second place, cast-iron bridges, even of large spans, could be constructed with greater cheapness; and in the third place, cast iron was the material best adapted to resist compression. He could not agree with the remark that it was not necessary to combine the qualities of the girder with the rib. A cast-iron or wrought-iron arch differed from one of stone, in that there was less depth of material, usually, between the extrados and the intrados, and in order to make the iron act properly, under unequal distributions of load, it was necessary to give it a greater or less degree of stiffness. To get the requisite amount of stiffness in the rib, a considerable addition must, by the introduction of flanges, be made to the amount of material that was required merely to resist the compression that came upon that rib as an arch. In fact, in that way, the qualities of a stiffening girder must be combined with those of an arch in the rib itself, to produce a sufficiently stiff rib. He agreed with the remarks which had been made, as to the inutility of unduly strengthening the horizontal member, for the purpose of assisting the arched ribs, or for employing that member further than was necessary for obtaining the requisite stiffness in the spandrils.

Mr. J. TAYLOR thought there was a tendency to depreciate the use of cast iron for bridges. No doubt the employment of wrought iron was advantageous where the span was large and the conditions of water-way stringent; but where a stream could be crossed by openings of moderate span, and where the conditions as to the water-way were not stringent, there was great advantage in the use of cast iron. He believed it could be used on an average at about one quarter less cost than wrought iron. He thought another important consideration was the comparative durability of the two materials. Cast-iron bridges were in existence that had been erected one hundred years; but there had not been the same lengthened experience with wrought-iron bridges; and it was known that the decay of wrought iron was very rapid, unless it was carefully watched and attended to. With regard to the two bridges under discussion, he thought that ribs of cast iron would have been better mechanically than ribs of wrought iron.

Mr. W. H. BARLOW was not prepared to recommend the use of cast iron, in structures of this description, unreservedly; the material seemed rather dependent on the magnitude of the bridge; and if it was a very large structure, he did not see how cast iron was to be used advantageously. It was pointed out with regard to the cast-iron railway bridge over the River Severn, that an arch of cast iron was obliged to have a certain amount of wrought iron in the top to give it stiffness; but if the arch was of wrought iron, there would be stiffness both in the arch itself, and in the member above the arch; and in his own practice he had found it beneficial to employ wrought iron in that manner. With regard to these particular bridges, he would remark that one of the advantages of the employment of wrought iron appeared to have been lost sight of, viz., each arch was made detached from its neighbour: stone was put in between them, and a complicated arrangement rendered necessary by the introduction of the stone. If the whole had been made continuous of wrought iron, there would have been a more complete combination of the parts, and greater stiffness over the haunches, as well as greater strength in the material. At the same time he considered the Victoria Bridge to be one of the most elegant structures that had been carried across the River Thames.

Mr. J. Fogerty, having had charge of the construction of the bridge over the River Severn at Coalbrookdale, stated that the cost of the cast iron was only £8 10*s*. per ton, and that of the wrought iron £14 per ton; and that the total cost of the bridge was a little over £11,000. In consequence of being obliged to make a separate pattern for every segment of the rib of the arch, he did not think a cast-iron bridge could be made with such expedition as a bridge of wrought iron, although he agreed that cast iron was the proper material for an arch; and probably that circumstance might have influenced Mr. Fowler in using wrought iron for the Victoria Bridge, which had to be finished within a limited time. The latter was erected in a little over twelve months, whereas the bridge over the River Severn was nearly two years in construction. It might have been completed in less time, but there was always a much greater amount of fitting and variety of parts in cast-iron structures. A slight difficulty was experienced in casting some of the segments of the arch of the Severn Bridge. The heel segment castings each weighed about 13 tons, including the heel or circular end, and the difficulty experienced was that the hydraulic pressure of the molten iron when rising upward in the large projecting end of the segment was so great as, in some instances, to blow up the flat portion of the casting. He recommended that such castings should be in detached pieces, and connected together with flanges. These castings were made at the Coalbrookdale Company's works; and he believed three or four of the first were bad castings, and one blew up altogether in consequence of the objectionable construction alluded to. He submitted there was no need for the ribs being circular at the springing ends, as there was no movement whatever at the springing of the arch. Two bridges of this description had been constructed over the River Severn and severely loaded, but in neither instance could the slightest movement be detected. A great deal of money was expended in fitting these semicircular castings to the springers, but he thought it would have been just as well for the springing to rise from a plane surface.[1]

Mr. J. H. Lloyd would be glad to hear whether this bridge was really to be considered as a girder bridge or an arch bridge? Whether there was, in fact, any effect from what was called the girder, or whether the whole effect was not due to the arch?

Mr. Vignoles was old enough to recollect two or three fluctuations in the opinions of engineers as to the relative merits of cast iron and of wrought iron for bridges. The bridge over the Severn reminded him of one of the best lessons he ever got from his grand-

[1] For details of this bridge, *vide* "Engineering," vol. i., p. 367.

father in engineering—now sixty years ago—in reference to a design which he could fancy was identical with that bridge. The subject was brought prominently forward on the occasion of the destruction of the cast-iron bridge at Chester, when the question of the introduction of wrought iron instead of cast iron was discussed. He was inclined to agree that, on the whole, the change from the iron arch to the girder was merely the transference of the same materials into another form. No doubt it was more economical, more expeditious, and more advisable to use wrought iron in bridges of large spans, but it was the reverse for bridges of ordinary span. In his opinion, there was nothing to account for the prejudice and antipathy against the adoption of cast iron for railway structures but the many accidents that had occurred fifteen or twenty years ago; and, in fact, cast iron for bridges under 200 feet span was as eligible as wrought iron, provided equal care was bestowed upon it.

Sir Charles Fox, in reply upon the discussion for his son, who had been unexpectedly summoned to Canada, stated that he not only desired, but made a request that the important question should be discussed, as to whether the arches of bridges of this kind should be constructed of cast iron or of wrought iron. His own opinion was that they should be of cast iron, but in this particular case they were governed by a variety of circumstances, one of which was that the then existing bridge, up to which they had to join, had wrought-iron ribs. He had been styled a "cast-iron man," because he advocated its use wherever it was possible; but it should be remembered, in the first place, that cast iron was cheaper, and, in the second place, that it was more durable than wrought iron. If he had to construct this bridge again, without any conditions connected with the joining to another bridge, he would make the ribs of cast iron, of such strength that the varying line of thrust should always be within the rib itself. He thought engineers had gone astray in the use of wrought iron in various structures, because he was sure that cast iron, if properly prepared and proved before it was put in place, was as safe as wrought iron. He had always taken care that every piece of metal on which safety depended should be proved on the spot immediately before it was put into place. In the case of the girders for the Exhibition Building of 1851, against which so much was said, he believed the reason why they did not fail was that every girder was put into a hydraulic press and proved the moment before it was required; and, as it was more important to record failures than successes, he would state that a number of girders broke in the proof, and were put on one side.

He saw no difficulty in making a bridge of this span of cast iron. There was an example in Southwark Bridge, erected years ago, and it must be admitted that it had stood its ground. The constructor of that bridge made no abstruse calculations as to whether the top girder or the spandrils were to add to its strength. He made the cast-iron ribs sufficiently strong, and there it stood a monument to Rennie, and one of the finest structures of that day.

Assuming the ribs to be of cast iron, he would make the top a continuous girder, only sufficiently strong to carry the superincumbent load. He admitted that, in this bridge, the top girder was stronger than was necessary, and he thought a considerable saving might have been effected in it, by putting corresponding strength in the arch, and making it of cast iron. He would, however, use wrought iron for the longitudinal girders for this reason:—He had ascertained, by a number of experiments, that the variations in the length of a wrought-iron girder, due to alternations of temperature in this climate, could be accommodated within the limit of elasticity of the girder itself; but that was not the case with cast iron. When the reservoir of the New River Company in the Hampstead Road was removed, many of the pipes leading to it, being of cast iron and flanged together, were found broken in two, in consequence of their inability to expand and contract. This proved that the limit of elasticity in cast iron was not sufficient to allow the metal to make the necessary accommodation, whereas wrought iron undoubtedly possessed that quality. Those accustomed to railway works knew that when rails were laid down in the middle of winter, it was a common practice for the platelayers to put a penny-piece between the two rails, to allow for the expansion of the metal in the summer. If a bar of wrought iron 15 feet long was compressed $\frac{1}{4}$ inch in the direction of its length, for the first time a small permanent set would take place, but it might subsequently be compressed a hundred times without that set being increased, provided the pressure was not greater than at first. Many years ago it was considered, in proving an iron structure, that if it took a permanent set it was damaged; but from a number of experiments he had come to the conclusion that all structures, the first time they were loaded, did take a permanent set. Some years ago he took a contract for the supply of chains for a suspension bridge; and in the specification it was stated that any link that took a permanent set should be rejected. He made a proving-press for the purpose, and found the first time any bar was stretched it took a permanent set. An engineer once said to him, "If your structures take a permanent set, I will reject them." His reply was, "Then you will reject them all, for they all do that." Therefore he had to ascertain what was the permanent set due to the first operation of proof. Another engineer said, "If any bar take a permanent set in my presence, I will reject it." He replied, "Very well: your proof is 10 tons per sectional inch: I will put it into the press and prove it to 11 tons, the day before you come, and take the stretch out of it; and with 10 tons you will have no permanent set." It was found on trial that a 12-feet bar put to that proof took a permanent set of $\frac{1}{40}$ inch; and it was accordingly agreed that a bar, not taking a greater permanent set than that, should be considered good. As a wrought-iron girder, when proved for the first time, would take a permanent set, it was desirable to ascertain exactly what that would amount to, and having ascertained that, the iron might be considered in a normal state; and provided a greater load were not put on, there would be no further alteration. Under these circumstances he would make the longitudinal girder of wrought iron, but only strong enough to carry the load.

He did not admit that these upright or diagonal spandrils were of much value. He thought the rib should be of cast iron, of sufficient strength in the top and bottom flange, that either flange might take the thrust of the arch under any altered position of the strain. He did not see any necessity for the horizontal girder having expansion joints over each pier, inasmuch as the total difference due to expansion and contraction was within the limits of elasticity of the material. Moreover, there was this fact with reference to the expansion joints of the original bridge, that from the autumn of the year 1865, to the spring of the year 1867, these joints had been carefully watched, and during that period, although there had been great alterations of temperature, they never moved a hair's breadth, and even the rust between one part and the other had not been broken. Therefore he did not see any advantage in going to the expense of these expansion joints when, in fact, there was no expansion.

As to the piers, he considered that, in structures of this kind, they ought to be regarded as vertical props, having no lateral stability. Engineers might be called upon to build a pier 100 feet high, and who could say that such a pier would not easily move? He remembered during the construction of the London and Birmingham Railway, it was proposed to erect a tall pier of brickwork, 6 feet square, and 20 feet high, to receive a transit instrument, for setting out the centre line of the Watford Tunnel. He believed it could be done equally well with an ordinary theodolite; and the result was that, before the transit instrument was fixed in its place, the tunnel was set out, the shafts sunk, and a heading driven from end to end. If, on looking through the transit instrument, when placed on the brick-pier, he gave the pier but a touch with the hand, so great was the motion produced, as to render any useful observation very difficult.

Then, as to the question of the cheapest way of making a pier, he believed it to be, as in this case, by cast-iron cylinders. If it had not been for the masonry in the existing bridge, with which a junction had to be effected, they would have brought the cylinders up to the springing. The Act required the foundations to be carried down to a certain depth below the bed of the River Thames, to allow the Conservators, if and when they thought fit, to dredge out the bed to a greater depth: he had, therefore, to sink below the foundations of the old bridge. If he had put in a coffer-dam for the excavation he might have dislodged the clay from under the old pier; therefore he put in cylinders to render that contingency impossible, and the result was, a pier which answered well and cost less money.

As to the variation of strains brought upon different parts of these two structures, the Government Inspector, on testing them, found the deflection nearly equal, and consequently the strains also must be nearly the same. In the two bridges, at Barnes and Richmond, to which reference had been made, and the ironwork of both of which he was permitted to construct, the arches were of cast iron. The Barnes Bridge was of three arches, each of 120 feet span, and the Richmond Bridge of 100 feet span. Both bridges had well answered their purpose; and, in fact, he had never heard a complaint of them.

It only remained for him to add, that the works at Battersea, which had cost £920,000, had been completed within $1\frac{1}{4}$ per cent. of the original estimates.

Mr. Wilson, in reply, said, in reference to the choice of wrought iron for the Victoria Bridge, that it was now so long since it was designed, he had almost forgotten the precise reasons which guided that choice; but he thought the chief inducements for adopting wrought iron, were economy in cost and facility and quickness of construction. A gantry was erected over the river, for the purpose of setting the masonry of the piers, and it was found that it would be sufficiently strong to erect wrought-iron arches: whereas to have erected cast-iron arches would have involved a large additional expenditure, as a much stronger centering would have been requisite. There were 80 inches of sectional area of wrought iron in the arched rib, and for the same cost only, 160 inches of cast iron could have been obtained, which would have been insufficient; although, had it been otherwise, he should have preferred the latter, as there could be no doubt that thin plates of wrought iron must be more subject to deterioration from vibration and corrosion, and an arch of cast iron of sufficient section must have a longer life. The two chief points of difference between the original bridge and the widening occurred in the foundations of the intermediate piers, and in the distribution of metal in the superstructure. With reference to the piers he had but little to say. He knew that certain difficulties occurred, which made it almost impossible to continue the original plan, although he would have preferred doing so; because a pier of solid masonry was more substantial, and certainly more reliable, than one built of two different materials. But he did not think the ability of the pier to resist the horizontal strain was in any way affected by the change; for the weakest part of the pier occurred above the low-water line, at a point considerably above the foundations, where the two were precisely similar in construction and sectional area. With respect to the superstructure, the differences were very great, and clearly showed that the two works had been calculated on totally different principles. In designing the original bridge, the calculations were all based upon the assumption that the piers were of sufficient stability to resist the maximum horizontal thrust that would come upon them; and, of course, having the piers to design with the other parts of the viaduct, the attention of the Engineer was specially directed to that object, and the greatest care was taken to ensure its being effected. He would not have alluded further to this, had it not been so specially referred to during the discussion; but as there seemed to be some doubt on the subject, he would make a few remarks, with a view to prove the statement he had made.

Assuming a load of sufficient amount to be applied to the pier, it would be found that fractures would occur at two points. The resistance to the upper fracture would be the tensile strength of the spandril-bars, holding down, by means of the horizontal girder, the upper portion of the pier to the arched rib; and the resistance to the lower fracture would be governed by the power of the masonry to resist the crushing at the arris.

In estimating the amount of resistance in a pier of this bridge, it was found that, if one arch was loaded and the other light, the unbalanced thrust was that due to the rolling load, and equal to 2,734 cwt. Of that amount, the upper portion of the pier would transmit 480 cwt. to the spandril-filling. In the same manner, the spandril-bars would transmit 280 cwt.; so that a total horizontal thrust of 760 cwt. would be transmitted, leaving 1,974 cwt. to be opposed by the lower portion of the pier.

Plotting the line of pressure, corresponding to this horizontal thrust, and the loads occurring at the different sections, the weakest point was found to be at **A**—**B** (Plate 8, fig. 2), where the centre of pressure was 4 feet from the axis of the pier, the weight above being 50 cwt. per square foot, and the thickness of the pier at that point being 15 feet. The moment of resistance of the section would be 450, whilst the bending moment would be 36,000 cwt., with a strain of 80 cwt., giving a total maximum compression on any part of the brickwork of 130 cwt. per square foot, or $2\frac{1}{2}$ times the mean pressure. Thus it would be seen the centre of the pressure, where the point of greatest weakness was, fell 4 feet from the axis of the pier, and that was at a point where the weight would give 50 cwt. per superficial foot. He knew of no reliable experiments, with regard to the elastic power of brick or stone work in resisting compression; but from the best data, it was calculated that the deflection of any one of those piers would not exceed $\frac{1}{8}$ inch, and that this could only result from a load of $1\frac{1}{4}$ ton per lineal foot on both lines of the railway, exactly fitting the span, without any part projecting over the arch. This could not occur in practice; therefore, the least possible maximum strain had been considered, and credit taken for all the advantages it gave. The arches, springing from fixed abutments, had been designed to act independent of one another; but, of course, credit was taken for the natural stiffness of the spandrils, and each arch was calculated to resist the greatest pressure resulting from one arch being loaded and the adjoining arch being light. The span of the arch was 175 feet; the rise 17 feet 6 inches, and the sectional area 80 inches. The calculated dead load was $12\frac{1}{2}$ cwt., and the rolling load also $12\frac{1}{2}$ cwt. per lineal foot: therefore the maximum thrust due both to the rolling load and to the fixed load was 2,734 cwt., and the compression per square inch at the centre, 68 cwt., while the strains at the springing, calculated in the ratio of that at the crown, would be 74 cwt.

Owing to the difficulty of ascertaining the relative degrees of stiffness in different parts of an arch of that character, it was almost impossible to determine the precise condition of the load on which the maximum bending moment would obtain; but it was assumed to take place when $\frac{6}{10}$ths of the arch were fully covered by the rolling load.

The effect of that load was indicated in Plate 8, fig. 1, which showed the line of thrust upon the arch. Assuming it to be unconnected with the horizontal girder and spandril-fillings, the greatest strain would be at the haunch, where the compression was 5 tons on the upper table, and 0·32 of a ton on the lower, the same being reversed on the opposite side. This showed that the line of pressure was kept within the arch; in fact, it was 0·2 of a foot within the flange. The result of this would be to reduce the strain of 5 tons, to 4 tons per square inch, by the additional strength imparted to the rib by the horizontal girder and spandril-filling. These strains would, of course, be further influenced by changes of temperature, tending to increase them. The observed range of expansion, in iron structures in this climate, being $\frac{1}{2}$ inch per 100 feet, the span of 175 feet would give an expansion of $\frac{7}{16}$ inch, producing a rise and fall at the crown, and an additional strain of $12\frac{1}{2}$ cwt. per inch, at the crown and springing. At a point in the haunch, where the maximum bending moment occurred, it would be 10 cwt., which, added to the strains already deduced from the most disadvantageous position of the rolling load, would give 90 cwt. as the greatest amount of compression that could occur on a rib of

Fig: 1.

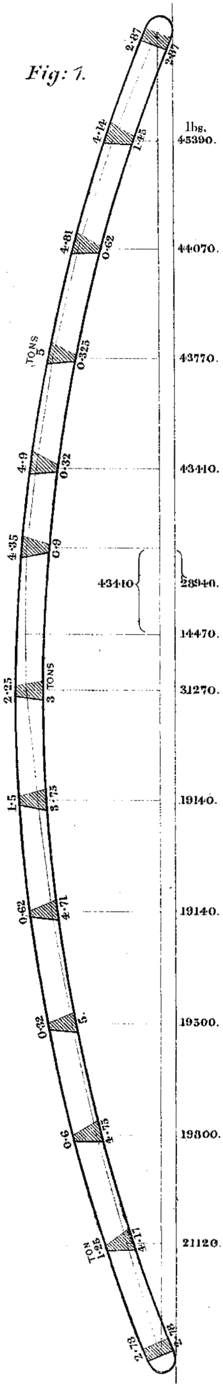


Fig: 2.

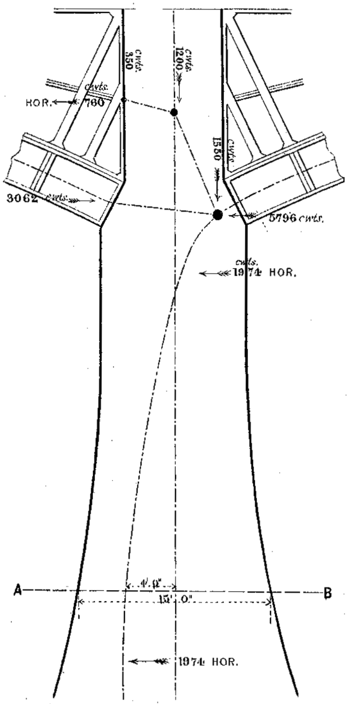
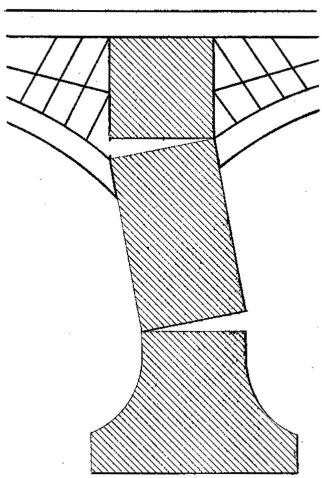


Fig: 3.



these dimensions, under the worst combination of circumstances—unfavourable distribution of load, and extreme temperature.

Allusion had been made to the expansion joint, and the segmental shoes to the ribs. The former was of little importance; it acted as a telescopic joint, giving stiffness and continuity, with freedom for expansion; and the chief object of the India-rubber washers was to get rid of any jarring motion that might take place. The latter afforded facilities in the erection of the bridge, by bringing the ribs to a bearing, and ensuring perfect equality of compression throughout the whole sectional area of the rib; and when once fixed in place and adjusted to its bearings, it would become a rusted journal of 3 feet 6 inches diameter, and unless the line of pressure fell without the flange, no movement could take place in the shoe; in other respects it came practically under the same conditions as a square end.

Some explanation was required respecting the relative cost of the two works. He had stated in his Paper, that the total cost of the original bridge was £84,000, which was equal to £2 13*s.* per superficial foot. As that appeared to be the precise cost, per superficial foot, of the widened work, he was startled with the similarity, as he thought there ought to have been a considerable difference; but it was accounted for, in some degree, by the circumstance that there was a large expenditure in the original bridge, in the face-work, and cut-water, and parapet railing, which was saved in the construction of the widening, which had only one face, and that was composed of the stone and iron-work from the original bridge, removed and re-set. Deducting £13,000 for this, it would reduce the cost of the original bridge, for the purpose of comparison, to £2 4*s.* per superficial foot, as against £2 13*s.*, the cost of the widening.

Sir CHARLES FOX begged to say one word with reference to the curved ends of the ribs: the only use of them was in fixing; when the skewbacks were in place, no further benefit was gained from them. It appeared to him that, if any motion were deemed desirable, it would be better to make the bearing faces of the skewback castings plane, and the ends of the girders very slightly curved, as in the case of the cast-iron bridge carrying the railway over Fairfield Street, in Manchester, where he had curved the ends of the ribs about $\frac{1}{10}$ inch, to allow a slight rolling motion, but in his opinion, the best way was to make both planes, and having bolted the skewbacks to the ribs, and fixed both in place, then to bed and back up the skewbacks with brickwork, or masonry set in cement, so as to secure them accurately in their proper places. That, in his opinion, was the cheapest and best mode of making sure that the ribs of an arch were accurately thrusting against the skewbacks.