

## Stress investigation and strengthening of the Royal Albert Bridge, Saltash

W. F. LEEMING & J. E. WHITBREAD

### Mr Leeming

The question of allowing increased loading on the bridge first arose in 1964-65, as the result of enquiries from commercial interests to Britain Railways for the transport of large quantities of china clay sand from Cornwall to south east England, using block trains of 100 ton capacity wagons with 25 ton axles.

49. There are at Paddington no original calculations for the bridge, although over the years many have been made for various purposes. These indicated that an increase in loading could be tolerated on all the main structural members as they existed, but more information was required with regard to possible fatigue affects.

### Mr Whitbread

Because of the interest in increased loadings, the Research Department of British Rail had to use a method of assessment which allowed for extrapolation from permissible loadings up to the proposed loadings on this complex structure, and models were therefore chosen.

51. Part of the results implied that secondary bending was occurring in the main hangers and that this could be reduced by installing new ties. Although the model test showed that these would never be subjected to compression, this seemed so unlikely that further tests were carried out on the bridge to examine the deformation of the trapezium bounded by the track girder, hangers and chains. These confirmed the model results and the new ties were specified.

52. The ties which were installed were bigger than was required so that their appearance would be compatible with the rest of the bridge. This may, in part, be the cause of the modifications being less effective than indicated by the model. With hindsight, it seems probable that the preload specified was insufficient to overcome the dead weight effects in the ties and that a higher preload could give increased control of the secondary bending in the hangers.

**Mr P. S. A. Berridge**, Bridge engineer, formerly with the North West Railway (India), the GWR and BR

I welcome the Paper because it draws attention to these unique self-contained suspension-bridge spans. In each, the chains have been prestressed at the time of assembly and they carry the weight of the tubular strut which was devised to resist the inwards drag at the ends of those chains (Fig. 16).

54. I would be interested to know how the Authors reproduced those conditions when making their Perspex model. With any suspension bridge of moderate length

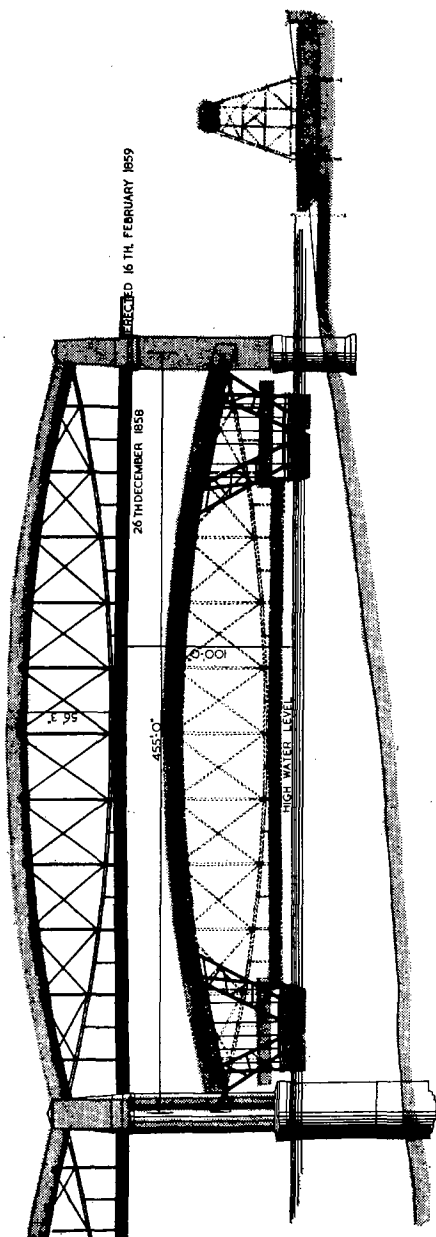


Fig. 16. Elevation of one span at Saltash



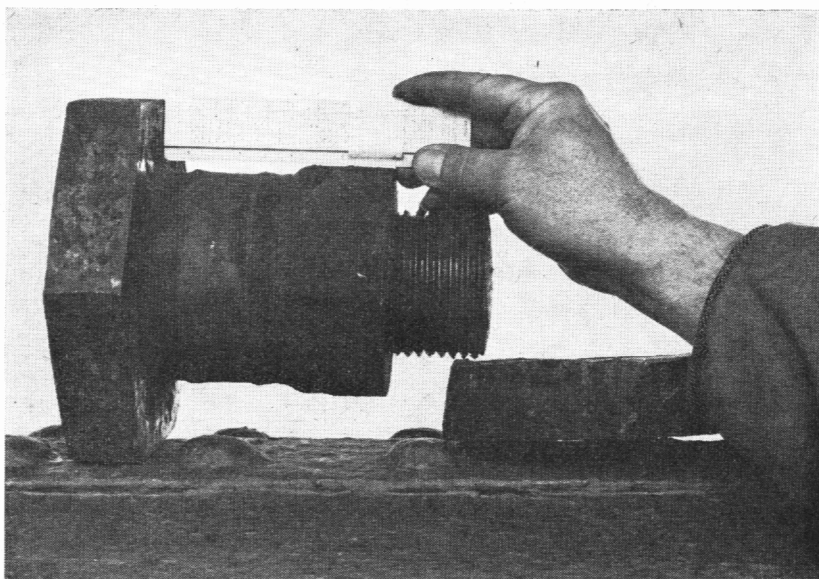


Fig. 18. Wear in one of the  $4\frac{1}{4}$  in. dia. pins securing a diagonal bracing to a standard at mid-span in 1960. Throughout, all pin-holes were initially drilled  $\frac{1}{8}$  in. larger than the diameter of the pins

like the spans at Saltash in which the ratio of live load to dead load is relatively high, it is difficult to maintain the catenaries in their correct shape during the passage of a train, as the Authors point out in § 21. I. K. Brunel got nearer to solving this problem than any other engineer. He evolved the idea whereby the overhead strut was 'sprung' against the suspension chains to keep them taut. At the Royal Albert Bridge where the upward curvature of the strut mirrors the downward sweep of the chains, he wedged eleven pairs of standards (his term for the verticals) between the tube and the chains (Fig. 17). With diagonal bracing he endeavoured to hold the standards vertical, and with the standards extended below the chains and fastened to the track girders which are continuous over the full 455 ft and are 'fixed' at the middle pier of the bridge, he hoped to hold the catenaries in shape.

55. The arrangement appears to have succeeded so long as the weight of trains did not exceed the design loading of about 1 ton per foot run of track. But the introduction of heavier locomotives in the 1920s brought trouble: the diagonal bracing proved to be much too flimsy (Fig. 18). In spite of tying all the standards together with new horizontal bracing fixed halfway up them, the constant slogging backwards and forwards between the chains and the track girders every time a train crossed the bridge caused the short hanger plates at mid-span to fail through corrosion fatigue. That was in the 1930s. Stainless steel bar hangers were fitted but, being too rigid, they were only temporarily successful. In 1960, the lower ends of the mid-span standards and the adjoining diagonal bracings had to be modified and new suspender links were fitted outside the chains (Figs 19 and 20). The latter restored articulation between the chains and the track girders.

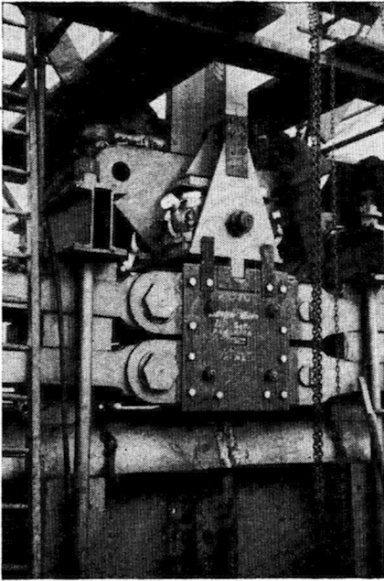


Fig. 19 (left). Fitting new suspender-links in 1960 ; a compressive force of 20 tons was 'jacked' back into each standard before the new bases were finally secured with high-strength friction-grip bolts

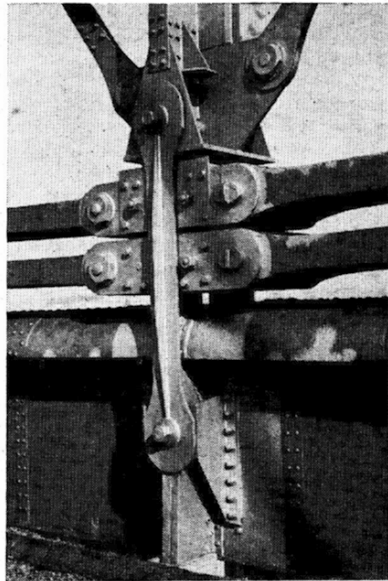


Fig. 20 (right). Suspender-link fitted in 1960

56. Turning to the 1971 strengthening of the longer hangers, described in the Paper, it seems to be a case of using a piece of string to tether an elephant. The catenaries will not be prevented from changing shape. The forces to which the new 'below-chain' diagonals are subjected must alternate between tension and compression every time a load crosses the bridge. None of the steelwork added during the lifetime of these historic spans has reduced the stress in the suspension chains.

57. There are about 2700 links in these chains, and of these, 1150 were made initially for Brunel's Clifton Bridge, work on which was stopped in 1843. Probably these are now the oldest specimens of wrought iron in use in any railway bridge in the world, and according to Brunel's contribution to the discussion on a paper by Howard<sup>1</sup> these early links were not made in one piece, as suggested by the Authors in § 28. Instead they were formed from rolled bars with enlarged parts for the pin-holes forged on at each end, and every link was proof-loaded in the presence of a trusted member of Brunel's staff. The balance of the links supplied in 1858 were rolled by Howard Ravenhill & Co., at the King and Queen Ironworks, Rotherhithe; each link being from a single 'faggot' of wrought iron rolled first of all transversely to form the broad 'eyes' at each end and subsequently rolled lengthwise to more than 20 ft.

58. In his paper Howard, speaking of the quality of the iron, said 'the best wrought iron begins to indicate a permanent stretch or set at about or a little over 10 tons per square inch.' His one-piece links were proof-loaded at 9 ton/sq. in. for an average working stress of 5 ton/sq. in. Although under the heaviest loading permitted in 1963 the maximum stress in the chains was only just greater than 5 ton/sq. in.,

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it was known that the stress intensity immediately adjacent to the pin-holes exceeded double that figure. I doubt the validity of the assumption (§ 16 (a)) that all members behaved elastically.

59. During the modification to the mid-span suspenders in 1960, test specimens were cut from the discarded flats of the diagonal bracing. These showed surprising results: one specimen cut *transversely* from a bar where it had been broadened to accommodate a pin-hole broke under a load of less than 10 ton/sq. in. and another 'pulled' parallel to the direction of rolling had an ultimate breaking stress of only 20.2 ton/sq. in. This is no discredit to the illustrious engineer who built the Royal Albert Bridge. I. K. Brunel was never guilty of wasting his client's money by building bridges which were unnecessarily strong for the trains they were designed to carry. Nevertheless, the loading proposed ((c) in Fig. 4), is more than twice that for which these spans were designed, and if this 25 ton axle loading is allowed, it seems probable that further quite rapid deterioration of these unusual spans must be expected.

**Mr R. G. Rebbeck**, BR RTC, Derby

I too am disturbed about the assumption of elasticity. This is the assumption made throughout. Even if load conditions remain constant throughout the life of a structure, it does not follow that if after a long time there is no appreciable fatigue damage, then the structure is completely elastic or in the high cycle regime. Fatigue damage may be caused entirely by large stress ranges which occur very infrequently due to a concurrence of unfavourable events; examples are dynamic effects, poor rail joints and wheel flats. In spite of the long life in terms of years, the bridge may be in the low cycle regime. In addition, loads have increased since the bridge was built and this makes the assumption of high cycle fatigue even less certain (§ 20 points out that there has been some fatigue damage).

61. I cannot believe in stresses going as high as 18 ton/sq. in. This must again be due to the assumption of elasticity. A realistic stress/strain curve for the material shows that such a stress cannot be achieved.

62. Although the assumptions made of the unmodified bridge may be unjustified and might also be unjustified if axle loads were to be increased to 25 tons, the stresses for the modified structure for 20 ton axle loads are so low that in this case only the bridge should be safe.

63. Fatigue is caused by reversed plasticity. It is therefore all important to calculate conditions using realistic material stress/strain properties; it is unjustifiable to extrapolate elastic analyses into this plastic region.

**Mr W. E. Gelson**, Sir Bruce White, Wolf Barry & Partners

My remarks relate to the arched tubes forming the top flanges of the river spans. From information which Mr Berridge has given me, the net cross-section of the tube near mid-span is about equal to that of the chains. The thickness is  $\frac{1}{4}$  in. I deduce that primary stresses in the tube are: dead load, 2.84; 1966 live load, 1.75; total, 4.89; total with the new proposed live load of 2.0 tonf, 5.18. The increase of total primary stress is 6%.

65. The critical buckling stress in a  $\frac{1}{2}$  in. plate with no imperfections, curved to 14 ft radius, taking the modulus of elasticity equal to 12 000 ton/sq. in., is 21.4 ton/sq. in. This figure is quite academic, however, as it considerably exceeds the yield point of the wrought iron.

66. The workmanship was doubtless to the highest standards obtainable in those days, when more initial distortions were built into structures than would be tolerated today. Photographs taken in 1933 (Figs 21 and 22) show distortions and distinct bulges in way of the internal diaphragms spaced at 40 ft intervals. A good view of the tubes from above can now be obtained from the towers of the adjacent road bridge. The position of the internal diaphragms can be deduced from the light and shade effects in bright sunlight (Fig. 21).

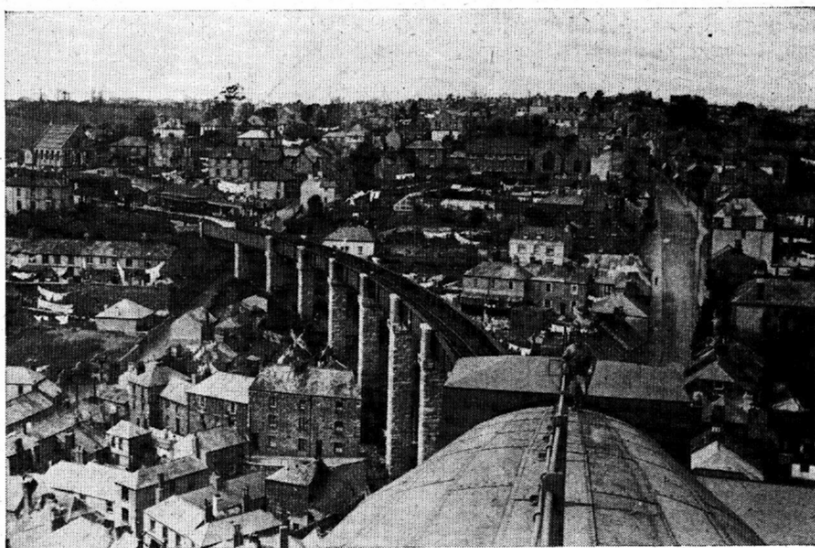


Fig. 21

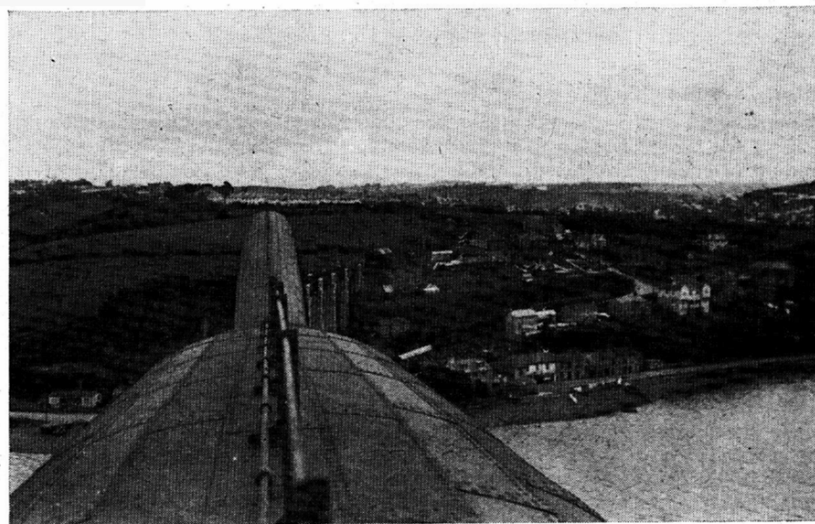


Fig. 22

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67. Mr Berridge found no loose rivets in the tubes when the Royal Albert Bridge was under his charge, and although I believe the tubes can stand the increased live load, it would be as well to keep an eye on the connexions.

**Sir Hubert Shirley-Smith**, Past President

I am interested in the Saltash Bridge as I had the pleasure of looking over it recently, and I have also seen drawings of the bridge, at Paddington. I am impressed with what Brunel achieved.

69. If the bridge is considered as a self-anchored suspension bridge, Brunel is the only British engineer who has built a suspension bridge that has successfully carried railway traffic, of ever-increasing weight, for nearly 120 years.

70. Engineers generally regard the four essential components of a bridge as safety, economy, durability and appearance. Looking at the Saltash Bridge from the safety aspect, I have never heard of anything that has seriously gone wrong with it. I know there are concerns for the suspended structure below the tubes and chains, but thanks to the magnificent work of the railway engineers in maintaining the bridge, it has been enabled to carry the heavy railway traffic of today without any serious results.

71. As to economy, the total cost of the bridge with the foundations and the deep centre pier which had to be sunk under compressed air, was only £225 000. Robert Stephenson's Britannia Bridge cost £601 865. The Britannia Bridge carried two standard gauge railway tracks, whereas the Saltash Bridge was designed for only one broad gauge 7 ft track. Owing to the much greater depth of the Saltash Bridge, the weight of metal is only about two-thirds that of the Britannia Bridge.

72. The third essential is durability. The bridge has already lasted over 110 years and may well continue for some years yet.

73. Finally there is appearance. Tastes change as the years go by, but I think it is a very fine functional job, and Brunel wasted no effort in pursuing aesthetics. A criticism of the design might be that there are places where it is difficult to provide proper maintenance, where water and wind can get in but there is insufficient space for painting and cleaning.

74. With the care which the Authors and other engineers on the railway can give the bridge, I hope it will still have many years of useful life ahead.

**Mr A. M. Sowden**, BRB Headquarters

The cost of the improvements has not been quoted, but it is quite apparent that a new lease of life has been given to this structure at a fraction of the cost of any other method of coping with heavier traffic. To this extent the work is typical of the ingenuity which is demanded of all the railway bridge engineers, and it shows what can be achieved by purposeful collaboration between the Research Division and the regional design engineers.

76. But is it typical of anything else? The Royal Albert Bridge is, of course, unique as a structure, so that one wonders if the investigation and strengthening can be regarded as anything more than a one-off exercise. Have any methods or techniques been learned which can be more widely applied? What lessons derive from this exercise? Is there any confirmation of the value of model analysis in determining what happens in structures?

77. I note that the improvement in the live load stress ranges recorded after strengthening was only about half that predicted by the model tests. Am I expecting too great a degree of accuracy from the model? How could it have been improved? In § 45 it is suggested that the discrepancy was due, at least in part, to 'shake-down' under the early loading cycles, and this has since been corrected. In what way was it corrected and with what results?

78. Was the improvement in the live load stress ranges itself improved? Could this have been aided by additional tension in the new members? In fact, how critical

was the force in each new diagonal? The first sentence of § 41 suggests it was purely nominal, to keep it from 'flapping' in the wind. But a subsequent section refers to strain gauges, adjustments after tensioning, shake-down, etc. affecting results, all of which suggest a requirement for at least some degree of precision.

79. The final sentence of the Paper refers to uncertainty 'because of the limits in the accuracy of the strain measuring method chosen'. Do the Authors mean in the model or in the actual bridge? Could improved accuracy have been achieved at reasonable cost, and could it have given better results overall?

80. Finally, what was the limiting stress range regarded as acceptable against fatigue failure in the wrought iron of which this bridge is constructed, and how was it determined?

**Mr J. W. Mann**, BR, Bristol

I should like to refer first of all to § 14 of the Paper where it is stated: 'In order that the model could be considered as an analogue of the span it was necessary that it should include any gross non-linearities in behaviour whether these were inherent in design, construction or the wear of components during their working lives.' Could I ask how the Authors attempted to reproduce stresses that were built in during the construction of the bridge? Mr Berridge has described the way the trusses were built up and supported along their length, and then chains hung from them. There are also certain indications that the bridge was built in a certain way. Did the Authors attempt in constructing the model to reproduce this sequence of construction as far as can be gathered from the drawings of the actual bridge?

82. Turning to § 19, the Authors state: 'Examination of the results implied that the fixing between tube and main hangers on the model was too stiff. However, as the bridge test had shown the most severe loading to be below the chains, it was decided to leave the model as built.' Also the connexions with other parts on the vertical hangers are very complicated, and I wonder whether any attempt was made to reproduce the various rotations that are possible at these connexions, in view of the small size of the model relative to the original. Surely it is difficult to reproduce this on such a small scale.

83. The fact that the improvement in the stress ranges was not what was expected may be because the structure is so highly redundant: that it was not possible to make full allowance for the extra relieving effect of all the bracing that has subsequently been added, and the way in which it was added, as well as the effect of subsequent repairs on the bridge which have changed the dead load stresses. Would this be a fair comment?

**Mr J. Simpson**, Sir William Halcrow & Partners

The effectiveness of the strengthening is judged in the conclusion of the Paper by the improvement in the live load stress ranges recorded in the bridge so that there should be no fatigue hazard under existing or future traffic. In particular it was felt necessary to reduce the stresses in the hangers below the chain. A 19th century researcher, Wöhler,<sup>2</sup> in tests carried out between 1859 and 1870 (that is, just after the erection of the Royal Albert Bridge) found that 10 million fluctuations from zero to 15.5 ton/sq. in. were required to break a wrought iron bar. Extrapolating this data, it would appear to take 250 million fluctuations from zero to 10 ton/sq. in. to cause failure of a wrought iron bar.

85. One has to trust the records in these tests, but it leads me to ask whether, contrary to previous contributors, the Authors feel that the reduction in stress they sought was a conservative measure, and in fact was not dictated by a high fatigue risk? Alternatively, perhaps the strengthening was necessary to take account of the fact that the stresses measured in the structure were static and not dynamic. Were in fact any dynamic stresses recorded for comparison with the static tests? I have in mind

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from my own experience the use of the Ferraday-Palmer type of recorder. The speed restriction on the bridge is 15 miles/h, and this seems to point to the fatigue risk being perhaps less than some speakers have suggested.

86. Welding repairs are not mentioned in the Paper as a possibility. Used with caution they have been successful in refurbishing many bridges, probably not as prestigious as the Royal Albert Bridge. I had in mind in this case the local strengthening of the hangers, since they were the members causing concern, possibly carried out in conjunction with stiffening the bracing above the chains. Would the Authors care to comment.

87. Finally, on a non-technical point, would the Authors say whether they have considered any similarity between the expertise they have brought out in this Paper and the era when British engineers were building railways abroad? It strikes me there is a possibility that nowadays this expertise could be applied to overseas railways, where bridges of similar age now require strengthening.

**Mr M. Barnbrook**, Bucks County Council

There is reference (§ 16 (b)) to the live forces being vertical. The Authors point out that one can have a train of some 890 tons in weight on this span, where the permanent speed restriction is 15 miles/h.

89. I am concerned that a train may approach at 30–40 miles/h, and the driver, realizing he is going too fast, will brake, transferring very high longitudinal forces to the bridge deck. Has the aspect of braking and traction been considered, particularly with the new bridge code (BS 116) in mind: this gives 70 tons braking and acceleration load for HA highway loading.

**The Chairman** (Professor A. L. L. Baker)

There has been no mention of the load factor. It strikes me that fatigue stresses are secondary. I would say that in the hangers and in the bulge in the top boom, under increasing load the stresses would not increase anything like in proportion to the load, and therefore the load factor is probably much higher than is indicated by the fatigue stresses occurring very close to, if not at, working load.

**Rev. M. Pearson**

In § 18 the Authors mention the need to ensure that the degree of fixity in the bridge members was reproduced in the corresponding members of the model. I should like to know how it is possible to assess the end fixity of bridge members, when even corrosion can in some instances cause a high degree of adherence and solidify the reaction between contiguous members of bridge parts.

92. Having obtained this information, how was it possible to reproduce this in the Perspex model?

**Mr Leeming and Mr Whitbread**

**Mr Berridge and Mr Rebbeck** both questioned the assumption that because a bridge is elastic it will always be elastic. This was not what it was intended to say in the Paper. If our remarks can be interpreted in this way this could be most dangerous. What it was intended to say was that because there has been no significant fatigue damage it is justifiable to use an elastic model; an elastic model will give adequate representation of the behaviour of the bridge. This is a very valid point and one which is well worth emphasizing to bridge engineers and to engineers in general.

94. On fatigue, **Mr Simpson** asked about the strength of the bridge in the light of the Wöhler test in the mid 19th century. Wrought iron of this age tends to be a variable material. Prior to 1966 British Rail Research Department had tested samples from bridges taken out of service, and it was found that a fatigue limit of about  $5\frac{1}{2}$  ton/sq. in. existed for this type of material. A recommendation was made at that

time that the maximum permissible live load stress range in wrought iron of that age should not be more than 5.2 ton/sq. in. When the Royal Albert Bridge was tested the stress range was higher and modifications were therefore suggested.

95. **Professor Baker** suggested that possibly by continuing the arch down to the track girder level in the bridge Brunel would have changed the fatigue characteristics and might have got a more efficient structure. It is probable that this would have produced a more flexible structure which in the context of railway bridges is not desirable. It is impossible to guess what was in Brunel's mind but it is probably significant that some of the flexibility of the track girder is supported by what is effectively two diagonals produced by chains rising about 20 ft above the track girder at the end of the span.

96. **Mr Berridge** asked about the behaviour and building of the model. The model was intended only to monitor the live load effects. There was no way in which the dead load stresses built into the structure could be obtained, and it seemed unnecessary to include them. There was no attempt to prestress the chains, but merely to ensure they were of the right shape and that when the model was put together its behaviour was similar to that measured on the bridge during tests.

97. The effect of notching on the fatigue tests is not known. Wrought iron of that age contains a significant number of comparatively large slag inclusions, and stress concentrations at the end of the inclusions would probably be comparable with those at the bottom of the notches. A reduction in cross-sectional area would seem the only possible allowance.

98. Reference was made to the accuracy of model techniques. There will always be some inaccuracies in a model, but if one is prepared to pay enough for an analysis method there can be very good agreement between model and prototype. As far as these tests were concerned there was a cost difference between mechanical strain gauges which have a sensitivity of about  $\frac{1}{4}$  ton/sq. in. in this material and electrical resistance strain gauges which have an accuracy perhaps ten times greater. A further point is that this bridge which has a 450 ft span would have required about 5 miles of cable effectively to strain gauge it, which would have been uneconomic. To obtain better accuracy would have meant producing detailed models of parts of the structure. This would have been both time consuming and expensive. As it was, the studies took nearly six years.

99. **Mr Gelson** refers to 'shake down'. There was a certain amount of distortion of the threads and bolts used for tensioning the diagonals. Increasing the load might have reduced this effect and the Paper implies that the force was very critical. However, the concern was that the force in the diagonal should not be too small.

100. Non-linearities were referred to by **Mr Mann**. These were not calculated, but observed and reproduced as closely as possible. In fact, the effects of non-linearities were almost non-existent on the scale examined. The aim was to look at the overall behaviour of the bridge and there was no significant non-linearity. If there had been, it would have meant modifying the end connexions and fixities to allow rotation to occur in the model to reproduce the behaviour of the bridge. About the most serious anomaly was a certain moment transmitted through pins at the top of the intermediate hangers which must have been caused by some rusting, but the effect was small.

101. On the effects of dynamic load referred to by **Mr Barnbrook**, while it cannot be said that British Rail drivers never exceed the speed limit, there is a tight curve and a viaduct at both ends of this bridge. There are few drivers who would want to exceed the speed limit in these circumstances. The driver himself is a built in speed gauge.

102. Lateral forces were not considered. Not enough is known about wind loading. Train vibrations will be small at 15 miles/h, and were not included in the analysis. Any trains braking or accelerating should be passenger trains, which are light. Freight trains tend to start off the end of the bridge and by the time they are passing

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over the main spans they are travelling at the sort of speed at which they will continue across the structure. This might not be true of a different bridge.

102. The point of the tests was to compare measurements between the bridge and the model. No calculations were made other than changing strains into stresses and comparing these with self-imposed fatigue limits so that load factors did not appear in the analysis.

103. **Mr Sowden** raised the question of cost. A sum of approximately £80 000 was spent on the job. This included not only the cost of the additional steelwork, but also the cleaning down and surface preparation for the application of a special protective system, paying particular attention to the chains. The latter were nearly all grit-blasted and cleaned, particularly between the links. As **Mr Berridge** quite correctly observes, nothing has ever been done to reduce the stresses in the chains, but the cleaning showed that the loss of metal in the individual links was remarkably small and that there has been no significant loss in the total area so there is no cause for concern on that score.

104. The point is also made that this test was for a unique structure: has it any 'spin off' knowledge or techniques for other work? It has assisted the Research Division, but the most important point is that knowledge of the behaviour of the structure is now in the possession of the Bridge Office at Paddington. Its behaviour has always been a subject for controversy, and judging from the discussion always will be.

105. **Mr Gelson** makes a valid point about the standard of workmanship and inbuilt distortions of the plates in the tubes, although the common thickness of these plates is  $\frac{3}{4}$  in. not  $\frac{1}{4}$  in. as quoted by him.

106. Whilst the new diagonals will extend the life of the bridge whatever happens in the future and certainly under existing loading, this is not governed by the diagonals or the hangers, but is solely dependent upon the condition of the pins connecting the chains at the top ends to the tube and of the links in that area, which is totally inaccessible for both maintenance and inspection. This point is touched on by **Sir Herbert Shirley-Smith**.

107. **Mr Simpson** asked whether or not local strengthening and stiffening using welding had been considered. Of course this possibility was considered but it would certainly not be by welding on bits and pieces. Although in special cases wrought iron is welded, it would certainly not be appropriate here.

## References

1. **HOWARD T.** Description of a method for rolling bars for suspension bridges. *Proc. Instn Civ. Engrs*, 1849, **8**, June, 273–281.
2. **WÖHLER.** Fatigue of materials. *American Civil Engineers Pocket Book*, 1911.