*Discussion on Paper No. 6673**

The basis of the revised fatigue clause for B.S.153

by

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Mr O. A. Kerensky (Freeman, Fox and Partners) congratulated the Author on his Paper and especially on the work behind it. He also thanked the British Welding Research Association, and in particular Dr R. Weck, for giving the facilities and the means to carry it out in record time. Gt Britain was now in the forefront in that field and he was sure that the new B.S. clause would be referred to, all over the world.

56. He then told his audience of how it had all started. In 1937, the Institution had formed a powerful committee to prepare a report on the design of steel girder bridges, which should have supplemented the by then obsolete 1923 edition of B.S.153. The war interfered with the work of that committee and its report (the so called green book) was eventually published only in 1949. In the same year the British Standards Institution had set up a committee, under the chairmanship of Mr G. A. Gardner, to review the 1923 and 1937 editions of B.S.153, taking the Institution's report as basis.

57. Clause 19 of B.S.153 was soon reached. It stated that when reversal of stress occurred under the passage of live load, the total stress in a member should be determined by adding half the smaller stress to the greater, and that the riveted or bolted connexions should be designed for the sum of the two stresses. The corresponding clause (217) in the Institution's report was slightly more sophisticated and recommended that each stress be increased by 50% of the other, and the member designed to resist either combination. Design of connexions was left as in the British Standard. During discussion, in committee, the purpose, the origin, and the validity of the clauses were queried and no one could give a satisfactory answer. Allowances for 'fatigue' or for 'dynamic' effects were suspected, but the basis could not be explained.

58. Accordingly, a small panel was formed to investigate the whole problem of fatigue in bridges and Dr Weck of the B.W.R.A. and Dr Whitman of M.E.X.E.—internationally recognized experts in this field—were co-opted. From that date to the present day, the work on the fatigue clause had never really stopped. B.W.R.A., M.E.X.E., and the British Railways co-operated with the B.S.I. Committee, throughout. As the Author correctly said, the efforts of the original fatigue panel had resulted in a reasonably safe but rather limited clause (Clause 23) in the 1958 editions of B.S.153. The requirements were based mainly on foreign research work, and on American and German Specifications for welded railway bridges, which were then just published. They were too conservative in some respects, and unsafe in others.

59. However, during the Conference in Cambridge, in 1959, on Fatigue, it became apparent that the new clause could be improved in the light of new research results; more comprehensive rules were already in existence in Germany and the U.S.S.R., enabling these countries to design at higher stresses, while using inferior steels.

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* *Proc. Instn civ. Engrs*, vol. 24, April 1963, pp. 519–540.

60. An appeal was made to B.W.R.A. to undertake the work of collating the existing data and of supplementing it with the necessary further research to enable the B.S.I. Committee to produce a more comprehensive and more up-to-date clause dealing with fatigue effects in welded and riveted structures in mild and high tensile steels. B.W.R.A. were most willing to help and this was the work described by the Author, who was in charge of it. Further research was now in progress and, no doubt, in a few years' time still better rules would be possible, but the revised clause was a great and important advance on the original one and should lead to significant improvements in design and to greater safety. Returning to the Paper, Mr Kerensky emphasized some of the main features:

61. The first improvement was the increased range of types of welds and of number and kinds of stress cycles covered, which should give designers considerable scope for their ingenuity in devising better structures.

62. The second innovation was that actual stresses for mild steel and high tensile steel were now given. These stresses were intended to be such that a structure subjected to the permitted number of the corresponding critical stress cycles could fail. It was considered that a safety factor against fatigue failure was not necessary and that in fact, it was provided by the very allowance for fatigue effects. The designer or the client could always introduce additional safety by adopting pessimistic loadings or pessimistic numbers of each type of stress cycle or both. In fact, the Author had used pessimistic values for allowable stresses and therefore his figures were on the safe side.

63. Some engineers believed that the actual structures would behave better than the specimen continuously and rapidly tested to destruction, as was usually the case in the research laboratories. It was also thought that long rest periods should have beneficial effects. In fact quite a few engineers doubted the existence of fatigue in real structures, and questioned the validity of results based on small scale tests.

64. Although the research workers were confident that their experimental procedure gave reliable results, it was generally agreed that full scale tests and programme loading tests were very desirable, if not essential, particularly as the results of such tests could permit further economies in material and, what was even more important, would give more reliable means of determining the safe life of structures.

65. Full scale tests were now taking place at B.W.R.A. The story of how this became possible would be instructive: The work was being carried out by the machines lent by British Railways, mounted on and testing steel girders donated by the industry, the actual testing being partially paid for by the grant from the Research Council of this Institution. It was not difficult to imagine how much begging this involved by the members of B.S.I. and B.W.R.A. Committees. And to crown it all, the specially imported testing machine had several fatigue failures!

66. The third major change was the introduction of the conception of 'load spectrum'. That was of vital importance. All bridges were designed to carry maximum foreseeable loads. Almost all bridges regularly carry insignificant loads, occasionally fairly heavy ones and sometimes, if ever, the design ones. Under the old rules, the designer had to take a pessimistic number of maximum loadings and, more or less, ignore the effects of normal loadings. Yet, if repeated a sufficient number of times almost any loading affected the life of the structure, i.e. there was cumulative damage. In this, the effect of high stresses was much more significant than that of the low ones, but neither could be ignored with impunity. A rough rule for taking into account the effects of load variations was given. The Author appeared somewhat pessimistic about the production of design load spectra, but the position was not as bad as it appeared to be.

67. B.W.R.A. and B.S.I. Committees were unable to produce any load spectra, and it was left to the client and designer to derive one for each specific case. The British Railways had already produced a comprehensive document, giving type of loading and numbers of cycles of each to be used for different classes of bridges in the

British Isles. This, no doubt, would be further improved when the various tests now in hand, were completed, but the designer already had all the information he needed. The Bridge Committee of the Road Research Board (D.S.I.R.) was at present working on a spectrum for highway bridges. This again would, in the first instance, be based on somewhat philosophical approach and then modified, if necessary, as various tests and statistical computations became available.

68. Unfortunately, even if reasonably reliable spectrum were available, the method of applying it by a modification of the so called Miner's rule was not reliable and very expensive research was needed to establish a better method. This 'programme loading' testing required special equipment and a lot of time, but B.W.R.A. were hopeful of securing a special grant from the D.S.I.R. for this work.

69. He concluded by saying that when all these tasks were completed, new rules might be produced which should permit further economy of material combined with greater safety and perhaps simplified design procedure.

Mr P. S. A. Berridge (Assistant Engineer (Bridges) British Railways, Western Region) was grateful to the Author because the Paper spotlighted the need to take heed of endurance limits. This was so important today when the common shop fastener was the rigid weld instead of the yielding rivet. Fortunately the connector on site—the high strength bolt—gave a much greater grip over a greater area than was the case with the rivet. Consequently, joints made with high strength friction grip bolts were not so liable to fatigue failure.

71. After more than forty years of Indian and British railway bridging, he found difficulty in listing any real fatigue failures apart from loose rivets, small cracks in cleats, etc. But he instanced two examples, both of which were the result of bad design. The first, Fig. 8, was an 18-ft half through type railway bridge where the railbearers had been seriously weakened by notching at the connexions to the cross girders. The absence of flange covers between the railbearers had led to a high

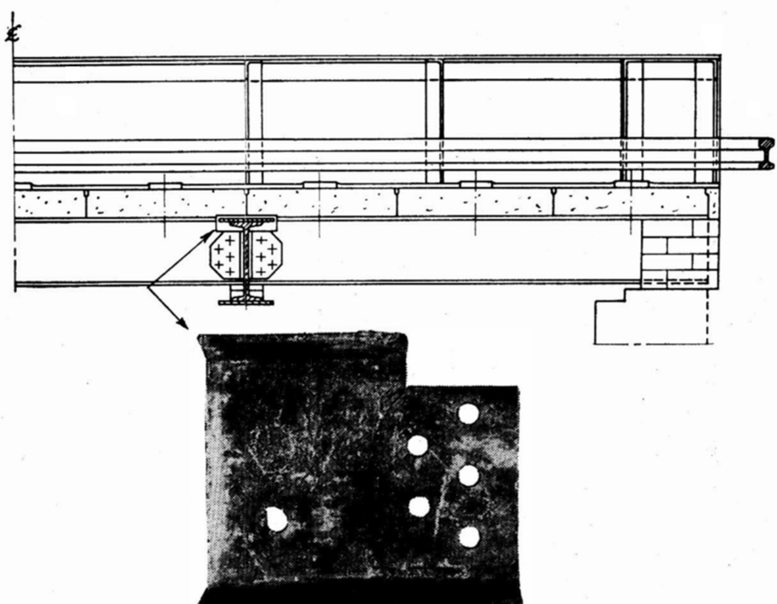


FIG. 8: FAILURE OF A RAILBEARER

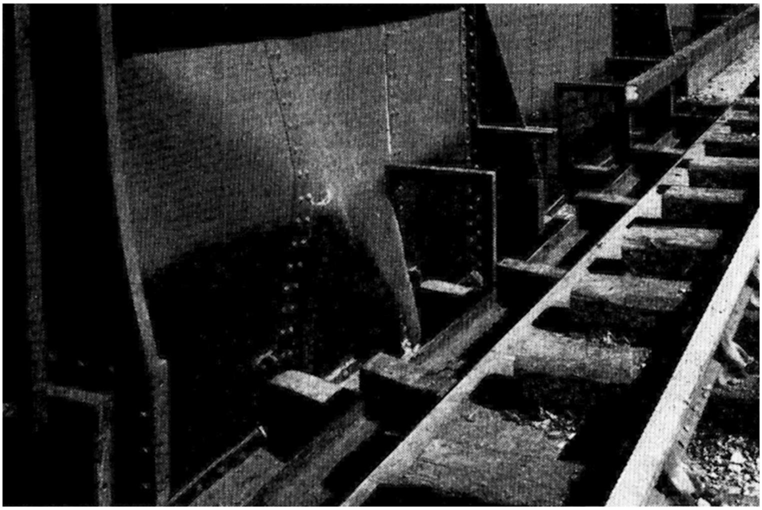


FIG. 9: FAILURE OF A WEB PLATE

concentration of stress at the notches. Applying the values given in Table 2 for Class G details in the revised fatigue clause for B.S.153, the permissible tensile stress at such a notch varied from 7·5 tons/sq. in. for 100 000 cycles to 5·0 tons/sq. in. for 600 000 cycles. Actually, the railbearers had broken after 18 years' service. As the spectrum of loading included 10 000 cycles of 20-ton axles in a year, it was remarkable that the railbearers had survived as long as they had, because, after making due allowance for load distribution, the tensile stress cycle under a 20-ton axle with 50% allowance for impact varied from 0·1 ton/sq. in. to 6·7 tons/sq. in.

72. His second example was the failure of the web plate in a 300-ft long wrought-iron plate girder continuous over three 100 ft openings at the Chepstow Bridge, Fig. 9. The failure had occurred in 1944, 92 years after the opening of the bridge. It had been estimated that 1½ million trains had crossed the span since its opening, and that the cycle of maximum live loading had not changed during the 21 years immediately preceding failure. The buckle had occurred at a point of maximum shear adjacent to a pier. There was no doubt that the web was grossly overstressed, the depth/thickness ratio of the web plate exceeded 288 while the yield value of the wrought iron varied from between 13 and 17 tons/sq. in. Failure had not been catastrophic, the web plate tearing at the riveting connecting it to the flanges and to the vertical cover straps. Had such a girder been welded instead of riveted, he doubted whether the failure would have been only partial.

73. Continuing, Mr Berridge asked if the Author could give any figures for the endurance limit values for wrought iron. He thought that he was asking for the impossible, because the texture of wrought iron was not so constant as in the case of mild steel. He also thought that it was very doubtful whether or not anybody might be able to give a reasonable forecast of when an existing girder, whether it be of wrought iron or steel, would reach its endurance limit.

74. Finally, he had drawn attention to the difficulty in the so-called repairing of continuous longitudinal fillet welds. Where such a fillet was found to be too small and had to be cut out over a short length and rewelded, it was almost impossible to eliminate entirely that visible change from the repair weld metal to the original fillet. He wondered if the requirements of Class B (ii) of the revised fatigue clause for

B.S.153 were not unnecessarily severe. He agreed that the continuous fillet weld made without a break or without a stop was the ideal. But in fabricating welded girderwork, it was probable that arcs would be struck by the striking of electrodes, either accidentally or on purpose, on the surface of the steel; cleats might be temporarily welded on to hold the parts together; and sometimes, holes might be drilled in the permanent steelwork for some temporary fixing during fabrication. No matter how neatly such devices were removed, holes plugged, and so on, he felt sure that those temporary welds would have had a lasting and possibly detrimental effect on the endurance life of a girder. It was to minimize the risk of such damage that he had incorporated a special clause in the specification for the supply of girderwork for the Western Region of British Railways. That clause stated that no permanent weld other than that shown on the drawings would be permitted; that all weld metal used temporarily during fabrication should be removed regardless of whether or not its retention would lower the class of constructional detail; and that where such weld metal was not submerged in the permanent welds, it should be ground smooth and flush with the surface of the parent metal.

Prof. N. S. Boulton (University of Sheffield) made a brief contribution, on the subject of cumulative damage, and said that in view of the small amount of information at present available regarding the fatigue strength of mild steel under random loading, it might be worthwhile to give some results obtained by A. A. Wood and S. K. Chaudhuri at Sheffield University for small rotating cantilever specimens. It might, however, be borne in mind that as they were small machined specimens which were annealed at 900°C, the results had only limited significance for the purpose of the Paper.

76. In the first series of tests, specimens of standard type were used with polished surfaces, and in the second series the specimens had a circumferential notch with semicircular root, with a somewhat less polished surface. The tests were performed with six load levels which were applied in a random order. For the first series of tests, the cumulative cycle ratio ranged from 0·93 to 1·59. For the second series, the range was from 0·75 to 0·83. The latter values for the notched specimens were on the unsafe side of unity, the value adopted in the Paper. As was stated by the Author,

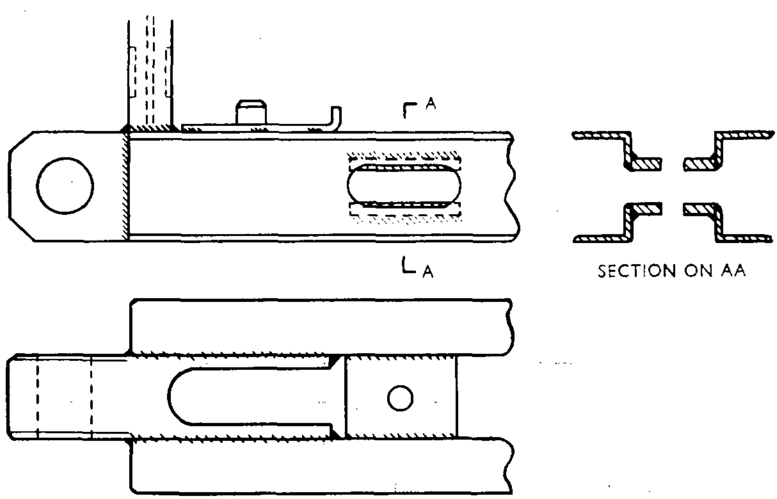


FIG. 10: DETAIL OF BAILEY BRIDGE PANEL

however, in § 46 of the Paper, the rules adopted might be regarded as tentative, and further research in this field was urgently required. The Author had also stated that the possibility of failure had not been entirely ruled out.

Dr J. F. Alder (Group Leader, Engineering and Chemical Group, Military Engineering Experimental Establishment) who expressed Dr Whitman's regret that other duties had precluded his attendance, said that the Author had referred in the Paper to another paper published by Dr Whitman and Dr Alder in 1960. As they had been associated with the revision of B.S.153, he felt that the meeting might be interested to hear a short summary of the work and results from their group at Christchurch.

78. Dr Alder remarked that most of those present were probably familiar with the Bailey bridge and its construction, consisting of panels each 10 ft long by about 5 ft high, with pins at each corner. The detail at one of the corners was shown in Fig. 10, from which it was seen that the requirements of manufacture and assembly in the field had left a most unpleasant group of welds.

79. It was also seen from the illustration that a slot was cut in the channels. That was in order to take the sway brace, and to hold the sway brace in position it was necessary to weld the two plates between the channels. This had given a Class F weld and a Class G weld, the latter being a fillet weld on the edge of the hole. This was at a point where already there was a stress concentration due to the presence of the hole. In addition, at the end carrying the male jaw block, there was also a load-carrying fillet weld down the back of the male jaw, gross change of section and, again, a Class G type of weld.

80. Normally, there was no concern with fatigue in the military use of bridges; they did not remain sufficiently long in use by military traffic. As was generally known, however, those bridges were usually left in position for further use by the civilian population. Consequently, it was necessary to know the behaviour of such structures under a spectrum of loads such as that referred to by the Author.

81. It was said by the Author that M.E.X.E. had been doing those fatigue tests, but, of course, they did not know what spectrum to put on. Consequently, for a

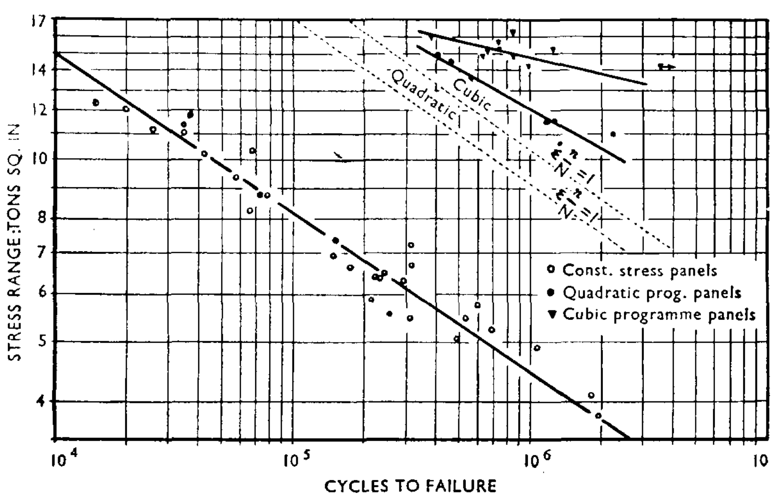


FIG. 11: PROGRAMMED AND CONSTRUCTION STRESS FATIGUE RESULTS FOR BAILEY BRIDGE PANELS

start, they had to think of a mathematical distribution; they had to make up the spectrum. A quadratic relationship and a cubic relationship between the percentage of load and the number of cycles had been chosen, and these two spectra were divided into six levels.

82. The results obtained were shown on Fig. 11. The main curve showed the basic results, using constant amplitude of loading. This was the normal type of fatigue data with which everyone was familiar. The tests were with pulsating load, going from just above zero load to the load applied. The tests referred to by Prof. Boulton were in rotating bending, and M.E.X.E. had found that considerably different results were achieved in the case of reversals as opposed to pulsating load.

83. The top right-hand corner of Fig. 11 showed the results of the programme tests, the lower curve being with the quadratic programme and the upper curve with the cubic programme. They were plotted as the maximum load in the cycle against the total number of applications in the cycle. A comparison could also be made in another way, by examining the cumulative damage ratio that was obtained from these results. Values of 2 and upwards were generally achieved. On the graph dotted lines showed the quadratic and the cubic curve, if $\sum(n/N)=1$. The results obtained were, therefore, found to lie on the safe side of the hypothetical value of the cumulative damage ratio.

84. The results obtained on the normal constant amplitude tests agreed very well with the G type of weld values given in the new tables. Thus, to follow what had been said earlier by Mr Kerensky, if the mean values were used, the safety factor was included with the cumulative damage ratio for that type of loading.

85. Another interesting point in connexion with the same graph was that by some mistake in manufacture, one or two panels had the G-type welds omitted in the sway brace hole, so that only an F-type detail was present. They had invariably given a longer life.

86. Since the tests were started, actual load spectra had also been investigated. It had been stated that the Road Research Laboratory might carry this out, but M.E.X.E. was already doing some pilot testing under civilian traffic conditions. The main difficulty had been that they had not as yet found a civilian bridge, under normal traffic conditions, which had a live load stress greater than 1 ton/sq. in. It was difficult to subdivide 1 ton/sq. in. into five or six levels.

87. Fortunately, however, there were some military bridges in civilian use, and Fig. 12 contained the results from a military bridge in use by the Bath Corporation. Comparison had been made with the two mathematical spectra used in the test series.

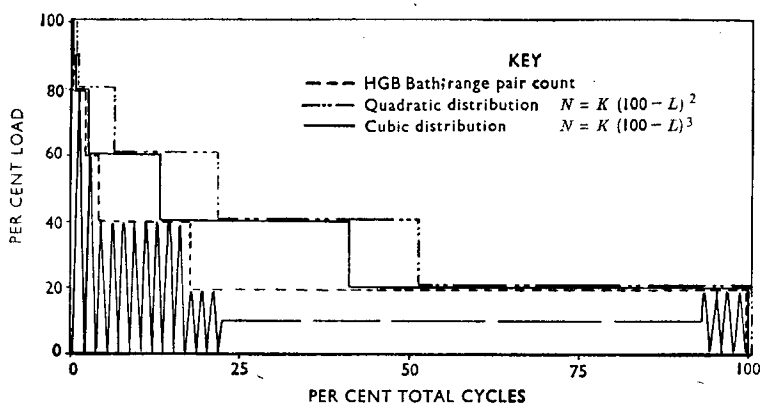


FIG. 12: MEASURED AND FATIGUE MACHINE HISTOGRAMS

The lower distribution shown was the actual spectrum from Bath, the upper was the quadratic, and the centre one the cubic distribution. In the case of the previous graph of the tests, the cubic programme was less severe than the quadratic. It could, therefore, be said from Fig. 12, which showed what had actually been measured at Bath, that actual practice would be less severe than the cubic programme.

88. For highway bridges, however, an extremely long time was required before a representative programme was likely to be obtained. The bulk of traffic rarely exceeded 25 tons even on trunk roads, but it was necessary to allow for the occasional passage of something of up to 100 tons. In these circumstances it took a lot longer than the British Railways experiments, which had load patterns which could more readily be forecast, before representative data were obtained.

89. Dr Alder concluded by showing a series of six slides which illustrated the development of cracks during one of the tests with the programme loadings.[9] This test had a maximum stress of 12 tons/sq. in., and the quadratic programme was applied. At 430 000 cycles, the first sign of a crack was visible to the naked eye. At 650 000 cycles the crack was easily seen, and another small one had started. By 800 000 cycles there were two definite cracks, of approximately equal length. By the time that 1 000 000 cycles were applied, cracks had also been produced in the channel on the other side; those cracks opened up considerably and reached the root of the channel. At 1 200 000 cycles, considerably enlarged cracks were easily visible, but the panel still took the load. The final failure occurred at 1 235 000 cycles.

90. These findings confirmed the general experience of how slowly the fatigue cracks had grown, the first signs became visible at one-third of the life; the cracks were easily visible at between one-half and two-thirds of the life. In some circumstances, however, cracks could become fast running, especially in thicker sections.

Mr R. P. Newman (Head, Members' Services Department, British Welding Research Association) said the fatigue clause of the 1958 Standard very often demanded assistance with interpretation. The fatigue-conscious would recognize that the revised clause removed many doubts and anomalies from that earlier clause; nevertheless the Author's Paper was likely to anticipate a number of queries from those who were less experienced in applying a fatigue criterion to the design of welded structures.

92. This consideration was important because, like the 1958 version, the revised clause undoubtedly found application in design fields other than that of bridge structures. It dealt with the problem in direct terms of permissible stress and, in effect, represented a fairly universal statement of that problem within the limits set by present knowledge, and by the Author's philosophical interpretation of such knowledge.

93. With regard to interpretation; one was asked to accept certain things as being reasonable, e.g. the general factor applied to higher strength groups and the specific factors applied to the low life end of the *S–N* curve. Mr Newman accepted these for the reasons given by the Author. He had drawn attention, however, to what in his view appeared to be a questionable decision on the classification of weld details.

94. His first impression from Fig. 2 was that the joints which had the best record of service behaviour, had been penalized more than joints with a somewhat notorious record of service behaviour. This appeared particularly noticeable for the transverse butt weld, Class E, for which the permissible stress at 2 000 000 cycles of repeated tension, e.g. was now nearly 1½ tons/sq. in. lower than the value given in the 1958 Standard.

95. In that connexion, the Author had indicated a dilemma in his reference to good and bad reinforcement shapes and, apparently, the question of definition between one and the other was too difficult a problem to place in the hands of works inspection. A penalty of 30% on permissible stress was accepted because one could not be explicit about surface configuration. The arrow for the relevant group in

Fig. 2 was located right at the bottom of the distribution of experimental results where, one suspected, it was anchored by results obtained from tests of particular types of butt joint that could be fully defined.

96. The Author, in his statement, had confirmed that the position was rather tantalizing, that any shop weld made with rutile electrodes in the downhand position was likely to be good enough for Class D without recourse to surface dressing. But surface dressing it had to be, if the higher stresses of this Class were used. Mr Newman was somewhat dubious about it having come across only relatively few service failures in full penetration transverse butt joints. He, in fact, recalled only two such failures. One occurred in an as-welded joint, and the other in a dressed weld of the kind required to satisfy Class D of the revised clause.

97. The new Standard had probably reached some kind of finality with regard to the behaviour of joints under constant stress amplitude. If this was so, one hoped nevertheless that interest in the research aspect of the overall problem would not diminish. The Author had shown in one of his graphs, if only incidentally, what appeared to be an attractive proposition of using simple methods to get much improved fatigue results, namely the use of treatments for inducing favourable residual stresses.

98. Mr Newman asked if the fatigue data obtained for such treatments were not as well established as some of the data used in the revision of the clause? If this were so, would the research effort that had been put into the development of these methods lead to some practical application, or would the advantage of higher fatigue strength lie unaccepted because the inspection problem would again be claimed to be too severe?

Mr J. C. Lucas (Research Department, British Railways, Derby) remarked that quite a lot had been said about the spectrum of loading and he wished to give a little information on how it had been done on British Railways. Part of Clause 24 stated that 'the number of cycles of each magnitude must be estimated by the engineer in the light of available data regarding the frequency of occurrence of each type of loading'.

100. His own job was to try to determine the frequency of occurrence of each of those types of loading. With main girders of more than 30 ft span, this was normally

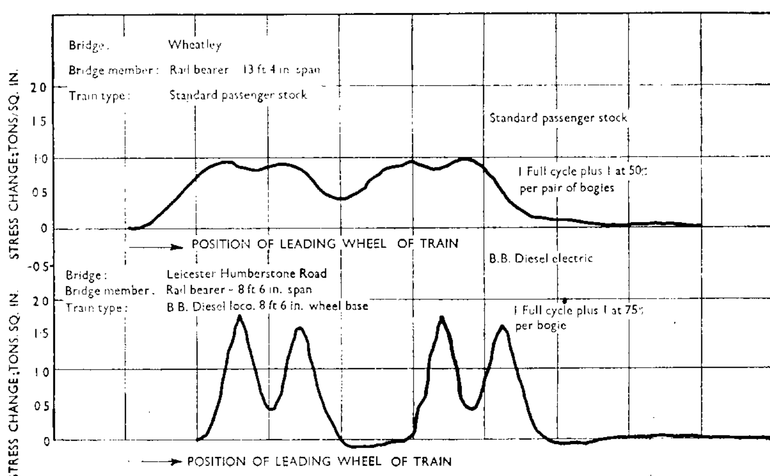


FIG. 13

straightforward, the assumption was that one train gave one cycle of loading. With floor systems, the task was not quite as easy. The possible effects of various types of trains were determined, both trains already in existence and those projected for the future. For all types of bridges and all types of projected trains that would have been virtually impossible, as a purely practical experiment, and if done theoretically it was not likely to be absolutely satisfactory.

101. Accordingly, three types of modern bridges were chosen with cross girder spacings of 8 ft 6 in., 7 ft, and 13 ft. 4 in., and from these, the basic strain diagrams were obtained, i.e. the variation of strain as the vehicle passed across the bridge. An attempt was made to obtain what might be called 'practical influence' lines. Strain gauges were placed on the centres of the various appropriate cross girders and rail

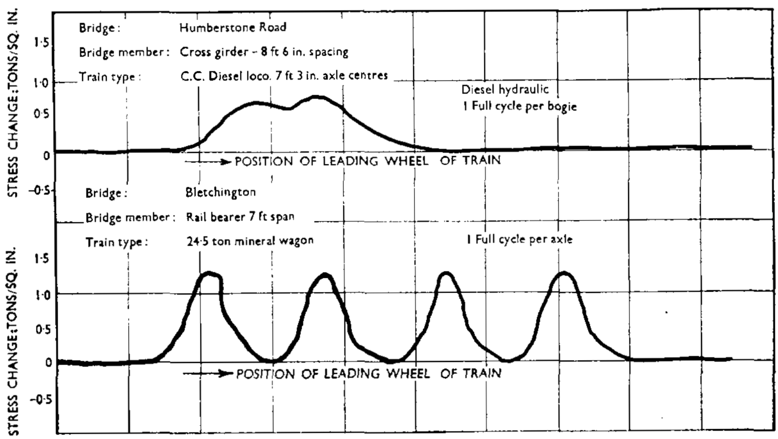


FIG. 14

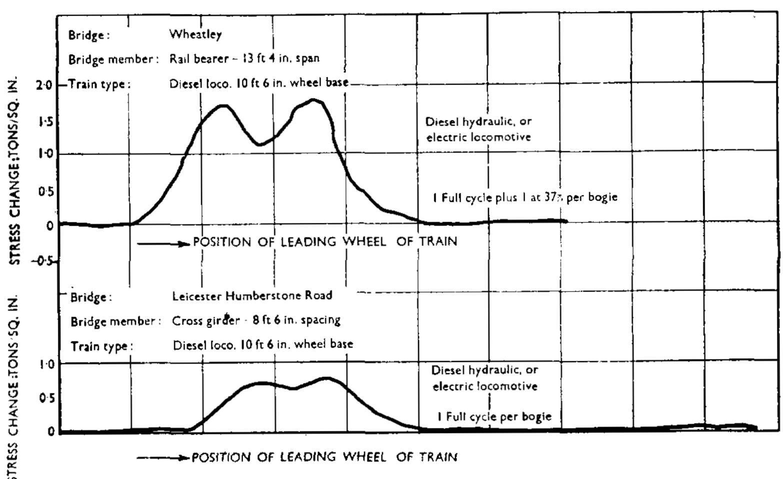


FIG. 15

bearers in the three bridges and a special train, which consisted of one long wheelbase tube wagon loaded to 18½ tons/axle, followed by an empty wagon and a locomotive, was propelled across the bridges. The output from the strain gauges was recorded continuously during the passage of the train. After the strain diagrams had been obtained, they were corrected so that they gave the influence of the leading loaded axle only. Hence, the diagram gave a practical influence line corresponding to an axle loading of 18½ tons. Then a computer programme which accepted those influence lines was devised. The computer was then fed with combinations of axle loads or axle spacings, and therefrom the basic strain diagrams were obtained.

102. Fig. 13 showed what was obtained from the diagram. The top line represented the 13 ft 4 in. span rail bearer with the standard passenger stock. The question of cycles was not quite as straightforward as it appeared. Although, basically, there was one complete cycle, there was half a cycle going down to 50% of the maximum, and not upwards.

103. This figure also showed the diagram for a British Railways diesel locomotive with a bogie wheelbase of 8 ft 6 in. crossing an 8 ft 6 in. span rail bearer. Fig. 14 illustrated that when the diesel locomotive crossed a cross girder of 8 ft 4 in. centres, 7-ft span, one cycle per bogie was produced, whereas for a 24½-ton mineral wagon, there was one complete cycle per axle for a rail bearer span of 7 ft 0 in.

104. Fig. 15 related to a longer rail bearer span of 13 ft 4 in. In this case, there was one full cycle of loading for the bogie, plus one at 37%.

105. The results shown in Fig. 16 were difficult of assessment, because the cycles of loading were not from the zero point but were from the maximum downwards for 24½ ton mineral wagons on 13 ft 4 in. span rail bearers. Investigation was carried out into the effect of these partial cycles which went from a maximum downwards, and it was seen that for practical purposes they might be treated as going from the zero up to the same value.

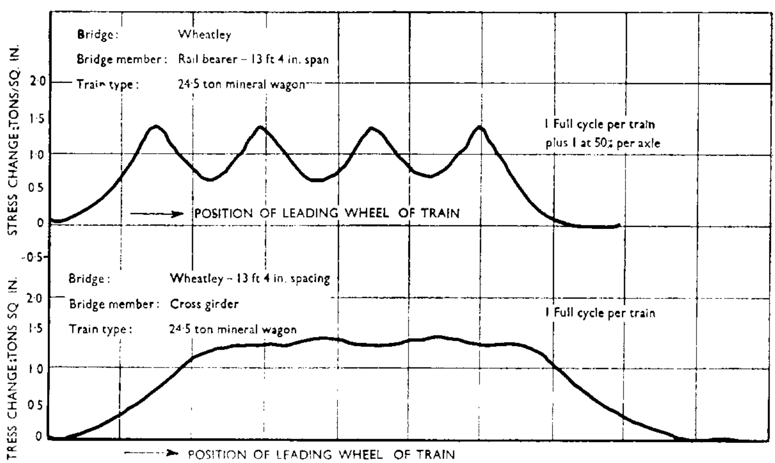
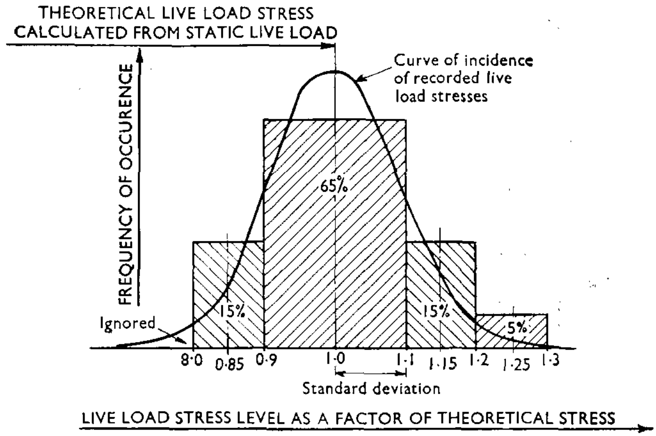


FIG. 16

106. In parallel with the above-mentioned work, a series of statistical investigations had been carried out on existing bridges. These tests had been carried out under normal traffic conditions and up to 10 000 trains which had passed over a total of ten bridges, had been recorded. It was found that for any one particular type of locomotive passing over any one bridge, the relationship between the standard deviation and the mean was roughly about 0·1. Therefore, he concluded, that with

this information, as well as all the information which had been obtained for these types of bridges, and for all the types of traffic predicted, it was possible to get what could be called the load spectrum for the British Railways.

Mr S. G. Oldfield (Bridge Section, British Railways, Western Region) said that his remarks were intended to show briefly how the stress spectra for British Railways and for London Transport were computed. Fig. 17 referred to the results of the tests carried out by Mr Lucas. This was a statistical analysis of the stress measurements and it indicated general frequencies of incidence of stress level as shown on Fig. 17. The curve at the top was taken as a histogram which represented the effects of all locomotives, rolling stock and bridge characteristics for main girders and floor members.



EXAMPLE

Loading combination	No. of cycles per passage of load	Theoretical stress level as a factor of live load design stress	Stress cycles per 100 passages Level	No.
Diesel	2	0·60	1·25 × 0·60 = 0·75	10
C-C loco.	(1 cycle		1·15 × 0·60 = 0·69	30
(20 ton axles)	per bogie)		1·00 × 0·60 = 0·60	130
			0·85 × 0·60 = 0·51	30

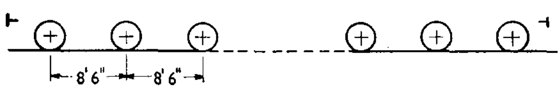


FIG. 17: FREQUENCY OF INCIDENCE OF VARIOUS STRESS LEVELS IN BRIDGE COMPONENTS WHEN SUBJECT TO TRAFFIC LOADING

108. It was seen that for every 100 passages of a certain load over the bridge, the stress recorded in the member for 65 out of the 100 came within the mean range, the centre of which was the theoretical live load stress calculated from the static live load. This diagram had formed the basis of the spectra computations.

109. There was a standard deviation of 0·1, and three times the standard deviation on the right-hand side showed that only 1 in 1000 stresses under any particular loading was likely to reach or exceed that maximum value. The theoretical live load design stress was calculated from the static weight of any loading combination and was taken

as a factor of the live load design stress which resulted from the application of the full design live load, and included impact, i.e. Type RB from B.S.153, to the bridge member.

110. The important point in that connexion was that the basis was the static weight of the loading combination, and it was compared with the effect of the full design load applied. In the example, in the lower part of Fig. 17, it was seen that for a particular loading combination on cross girders at 3 to 9 ft spacing with a C–C diesel locomotive with 20-ton axles at 8-ft 6-in. centres, it was found that each passage of the locomotive produced two complete cycles of stress. The static load of the locomotive bogie compared with the effect of the full design load, Type RB, gave a factor of 0·6. For 100 passages, the extreme right-hand side of the diagram showed the numbers of stress cycles as 10, 30, 130, and 30, which were based on percentages on the histogram. The magnitude of each of these numbers of stress cycles was given from the values above and below the mean as shown on the horizontal line.

111. Fig. 18 showed the main loading combinations that were considered. The total numbers of stress cycles were computed for a period of 120 years, which had been assessed as the average life of steel bridges of modern design before they required renewal due to corrosion and other forms of deterioration. It might be noted that a variation of 20 years either way would affect permissible stresses by only 0·1 to 0·2 tons/sq. in. This was because the class of detail and the level of the stress cycles—rather than the numbers—were preponderant in determining the permissible stress.

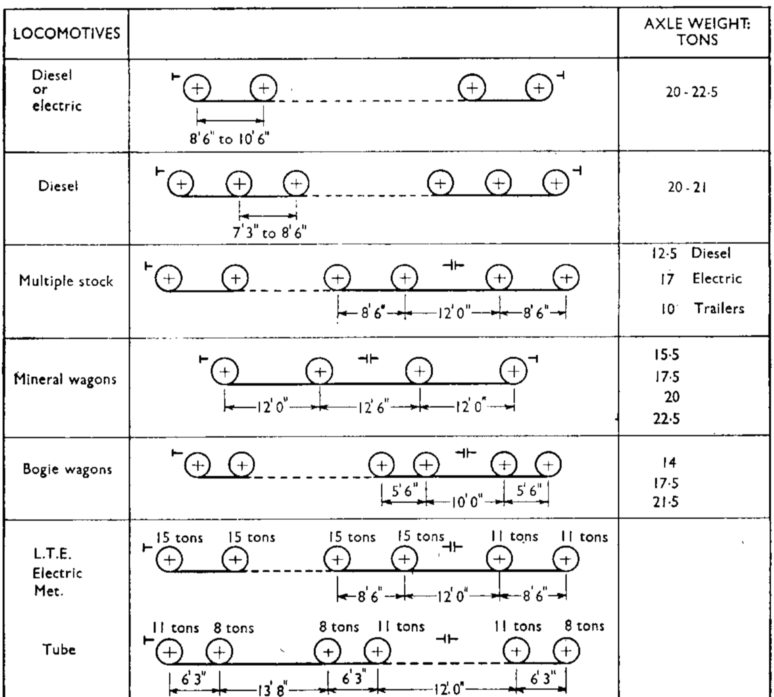


FIG. 18: LOADING COMBINATIONS

112. The necessary carefully-considered traffic forecast was based on a main line in a heavy industrial area, with 75% of the trains being freight and 25% passenger. In figures, this amounted to 108 freight trains and 36 passenger trains daily.

113. The various levels of stress cycles were gathered into five groups, as shown in Table 4. Investigations showed that Group 5 stresses, i.e. stress levels up to 30% of theoretical design stress, might be ignored, and this proved a more practical method of dealing with the lowest stress levels than that given in § (ii), A and B, Section C, of the revised Clause 24. For London Transport electric line bridges, however, where very large numbers of cycles occurred in the low-level Group 4, the B.S.153 paragraphs referred to should be applied.

TABLE 4: VARIOUS LEVELS OF STRESS CYCLES GATHERED INTO FIVE GROUPS

Group	Group stress level as a factor of live load design stress	Containing stresses at the following levels
1	1·0	0·9–1·0
2	0·8	0·7–0·89
3	0·6	0·5–0·69
4	0·4	0·3–0·49
5	0·2	0·0–0·29

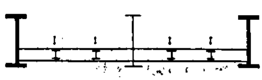
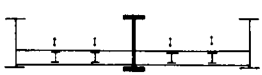
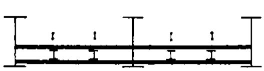
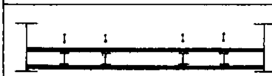
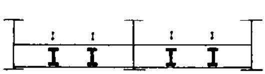
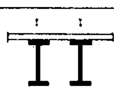
BRIDGE MEMBER		No. OF CYCLES : MILLIONS			
	Group No.	1	2	3	4
	Group mean stress level as factor of live load design stress	1·0	0·8	0·6	0·4
	Spans 15-30 ft	0·55	2·2	5·0	10·5
	Over 30 ft	0·1	0·25	4·75	1·6
	15-30 ft	0·1	0·5	1·0	14·0
	Over 30 ft	0·1	0·2	1·0	9·0
	Over 30 ft	0·2	0·25	1·5	5·5
	Spacings 3-9 ft	0·3	1·2	22·0	60·0
	Over 9-12 ft	0·4	1·7	13·5	35·0
	" 12-15 ft	0·55	2·2	5·0	10·5
	All spacings	0·1	0·25	1·25	10·5
	Spans up to 9 ft	1·9	7·5	220·0	100·0
	Over 9-12 ft	1·35	5·25	120·0	80·0
	" 12-15 ft	0·75	3·0	20·0	60·0
	15-30 ft	0·15	0·5	5·5	9·5
	Over 30 ft	0·1	0·25	4·0	2·0

FIG. 19: STRESS SPECTRA. BRITISH RAILWAYS BRIDGES

114. For the loading combinations that were considered, i.e. axles up to 22½ tons, no stress cycles were found in Group 1, but it was thought advisable to include in Group 1 a proportion, generally 25%, of the numbers of cycles in Group 2. In fact, since the spectra were made, bogie wagons with 25-ton or even 30-ton axles had been proposed on certain lines of British Railways. The point there was that the Type RB loading based on the B.S.153 for 0–8–0 locomotives coupled with two 25-ton axles at 6-ft centres and with high values for impact, were much in excess of any existing railway loading. Fig. 19 showed the British Railways spectra for principal types of bridge members. The maximum number of cycles (220 m.) occurred in Group 3 for rail bearers up to 9-ft span.

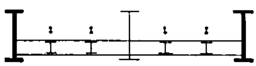
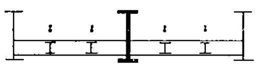
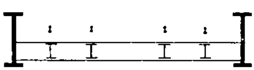
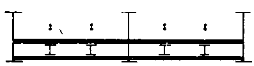
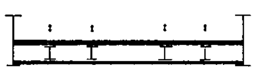
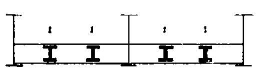
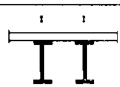
BRIDGE MEMBER (Carrying electric stock only)		No. OF CYCLES : MILLIONS			
		MET. & DIST.		TUBE	
	Group No.	3	4	3	4
	Group mean stress level as factor of live load design stress	0·6	0·4	0·6	0·4
	Spans 15 - 30 ft over 30 ft	 5·5 0·8	 104·0 15·0	 – –	 28·0 28·0
	15 - 30 ft over 30 ft	2·2 0·4	20·0 2·8	– –	6·0 6·0
	over 30 ft	0·2	15·0	–	3·0
	Spacings 3 - 9 ft Over 9 - 12 ft " 12 - 15 ft	 22·0 14·0 5·5	 87·0 96·0 104·0	 11·5 5·8 –	 220·0 124·0 28·0
	All spacings	1·1	32·0	–	3·0
	Spans up to 9 ft Over 9 - 12 ft " 12 - 15 ft	 19·0 56·0 93·0	 274·0 145·0 16·5	 23·0 23·0 23·0	 435·0 435·0 435·0
	15 - 30 ft over 30 ft	– –	110·0 16·0	– –	7·0 7·0

FIG. 20: STRESS SPECTRA. LONDON TRANSPORT BRIDGES

115. Fig. 20 showed the separate spectra which had been prepared for London Transport bridges for lines carrying only electric stock. The maximum number of cycles (435 m.) occurred in Group 4 on rail bearers. On London Transport bridges carrying electric stock only, no stress cycles occurred in Groups 1 and 2. They occurred only in Groups 3 and 4.

116. Mr Oldfield said that it should be noted that the group stress levels in the spectra had been directly related to the design loads, i.e. to Type RB loading in B.S.153, which in the case of British Railways was at 6 rev/sec and for London

Transport at either 4·5 or 3 rev/sec; if the design loadings were revised, then these stress spectra would also need complete revision.

Dr J. E. Spindel (Bridge Section, British Railways, London Midland Region), said that he would like to round off the Paper by discussing the effect of the revisions on the design of welded railway bridges. Fig. 21 showed a welded plate girder which was typical of main girders in steel railway bridges. The worst features in this design were seen to be the Class F welds at the ends of the stiffeners and flange plates. Dr Spindel pointed out, however, that these Class F welds were not necessarily at points of maximum stress. One way of designing girders properly, was to ensure that they were not. For quite short spans the minimum over maximum stress ratio varied from about 0·25 to 0·3 and permissible tensile stress at the Class F welds was about 8 tons/sq. in. There was, therefore, no need for any reduction of the normal working stresses for the girder as a whole.

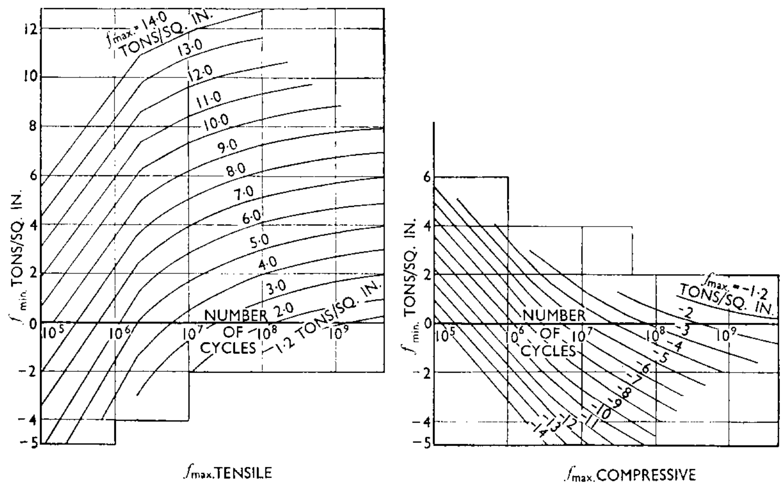


FIG. 21: CLASS F CONSTRUCTIONAL DETAILS

118. As the spans increased to about 50 ft, the stress ratio rose to about 0·4; the permissible stresses going up to about 8¼ tons/sq. in., if the girder was loaded by one track, or to a figure just under 9 tons/sq. in., if loaded by two tracks. In those circumstances there was no point in using high yield stress steel.

119. With floor members the situation was rather different. The number of stress cycles rose tremendously. Dr Spindel showed an illustration of a battle deck floor unit in which rail bearers were welded to the under side of the deck plates, and to the cross girders which were connected to main girders by high strength bolts. Class F welds were again in evidence at the joint between rail bearer and deck plate, the connexion of rail bearer to cross girder, and in the end plates of the bolted connexion. The permissible tensile stress at the Class F welds there was about 3·1 tons/sq. in., but it was disconcerting that the permissible stress in compression was only about 4·5 tons/sq. in. The stress figure could have been raised to about 7·7 tons/sq. in. by improving the welds to Class D. Indeed, if welds were omitted altogether, say in concreted decks, high yield stress steel could be used quite happily. The moral seemed to be to make the rail bearers continuous so as to reduce the number of stress cycles.

120. The cross girders shown in that floor were at 9 ft centres and carried two tracks where, even on Class F welds, stresses of up to about 7·6 tons/sq. in. could be used. The design of the connexions was a more difficult problem. It was a pity that Mr Berridge might be disappointed, but high strength bolts had failed in fatigue. A bolted connexion had been tested under repeated loading because of concern about the welds in the end plates. The part that broke was the high strength bolts. The stresses in end plates, welds and bolts were not easy to analyse. The only reasonable basis for design was fatigue tests on prototype connexions. These were planned and it was hoped that they would be done fairly soon.

121. In making the calculations described, difficulty was encountered in that the tables given in the revised clause of B.S.153 were not easily usable for design. Consequently, a great deal of plotting had been done. The final result obtained was shown in Fig. 21, for Class F constructional details. Minimum stress had been plotted on the vertical axis to natural scale which allowed negative and positive stress to be shown. Curves showed maximum stress to natural scale against numbers of cycles to logarithmic scale plotted on the horizontal axis. That meant that it was not necessary to work out the stress ratios when going through various group levels. Had the difference between maximum and minimum stress been plotted instead of maximum stress, it might have saved one more step in the calculation.

122. The first of the two questions that Dr Spindel put to the Author concerned the effect of combined stress. It was intersting, he continued, to note that elsewhere in B.S.153 combined stress effects were judged on the maximum shear stress as they were in some fatigue rules. In that case, he asked, why was it necessary to work to a principal stress and, did the test results that were available, really allow a clear distinction to be made between the two theories?

123. There was considerable scope for error in all calculations on fatigue life. Mr Kerensky had said that the safety factors were just about one, the margin being nil. The scatter of results had been presented and it was known there was scatter in workmanship, as Mr Berridge had pointed out. People might also know that they had probably made mistakes in guessing the load spectrum and that, in any case, if the type of traffic changed, the whole thing would be out.

124. The type of result presented by Dr Alder was encouraging, because it indicated a reasonable margin between the appearance of the first visible crack, and the time when the whole structure collapsed. That was something which should be emphasized from the point of view of people responsible for the maintenance of bridges. It might have been helpful if the Author had provided as much similar information as was available.

Mr P. J. Clark (Freeman, Fox and Partners) said that when the Author's experimental data first became available, he had been in the fortunate position of working for Mr Kerensky, on a bridge for British Railways. They had approached the B.W.R.A. for advice on applying the new data, and had then become the guinea-pigs on which the new clause was worked out. As a result, he was able to throw a little light on one or two points, which seemed important at that time, but which had proved to be not quite so critical in the final design.

126. As their design progressed, they had been in constant touch with the Author, and every letter received from him appeared to cut the life expectancy of their bridge by a further 50 years. They were busily occupied plotting results for 2 m. reversals, drawing curves, and extrapolating them to 200 m. reversals, and they hoped that they were on the right track. They had been asked to produce a bridge with a life expectancy of 100 years; and had finally acheived one of 200 years. This had seemed to offer a reasonable margin, in case the embryo code was altered before the bridge was built.

127. At the beginning of 1963, they had received the new load spectrum from British Railways and re-analysed their bridge accordingly. The result was alarming,

for it appeared that instead of an expected life of 200 years, they had obtained one of only 50 years. They therefore took the spectrum, removed a mineral wagon —a vehicle which was unlikely to travel regularly into Victoria Station—made a little adjustment to allow for a difference in cross-girder spacing, and found to their satisfaction that they were back to a figure of 300 years.

128. This showed that with so much scientific data available one had to be rather careful, to appreciate just what was significant. It was interesting to note that the difference in life between 300 years and 30 years on the bridge in question, represented considerably less than ½ ton/sq. in. on the working stress. It was a source of some consolation in designing for fatigue, when there was so much doubt about the validity of Miner's rule and the extrapolation of test results, that one could allow another ½ ton/sq. in., and sleep soundly at night!

129. Another feature which appeared from examination of spectrum for British Railways was that, given a certain cross girder section with an apparent design life of 25 years, one could either use a larger section to bring the life up to 100 years, or one could increase the spacing of the cross girders by, say, 3 in., which would move the cross girder into a different group where its design life was perhaps 100 years or more. This sort of thing would inevitably occur, he continued, as soon as a load spectrum was produced and working rules were given to designers. Such rules might have quite unexpected consequences in the design office.

130. He did not want to detract in any way from the Railway spectrum, which he considered to be a most valuable document, such results were an inevitable consequence of a set of simple rules. He would like to make a plea that when the British Railways Design Rules were issued, the designers should also have the complete spectra from which they were produced, since for non-standard types of structure only the full spectrum was of any value.

131. Mr Clark then took issue with Mr Berridge concerning the extra care required on the part of the engineer in specifying and supervising the work. He was particularly concerned with what happened to the steelwork after the engineer had applied Miner's rule to it, and had completed doing the design. As it passed through the shop, a fabrication cleat might be welded on to it, cut off, ground down, and painted over. There would be nothing to see, but there would then be a heat-affected zone, and an unpleasant stress concentration in the middle of an A-class tension flange.

132. From the Paper he understood that once a crack developed, all welds tended towards the same standard. He had himself seen a cracked weld on site, and presumably there were some cracked welds which were never seen. That worried him, because the code did not ask for all welds to be non-destructively examined, but only Class A welds. Did that mean, he inquired, that if there was a small crack or other imperfection in a Class D weld, it was not Class D but some lower class? That was very important. In the same vein, there was the question of building unsuspected stresses into structures. A designer might design to a specified stress condition. If, however, a bearing were set to the wrong level in a continuous structure, the stress in some part of that structure would be higher than anticipated. A similar stress increase could result from an incorrect sequence of welding. The whole stress pattern might then be lifted by, perhaps, 2 tons/sq. in., so that instead of a fatigue cycle from 4 to 6 tons/sq. in. the cycle would be from 6 to 8 tons/sq. in., and the theoretical life of the member had, perhaps, immediately gone down by ten times.

133. These, then, were a few of the considerations which Mr Clark felt should be carefully borne in mind, by both designers and contractors as they attempted to assess the importance of the fatigue problem.

Mr T. Haas (British Welding Research Association) said that the new Standard and the Author's contribution certainly represented a great advance on the old Standard. It was, however, necessary to point out that the presentation of the data in the form of permissible stresses did not make direct use of the variables which fully described the fatigue performance, namely, the mean *S–N* values and statistically defined scatter about them.

135. Instead, a fictitious *S–N* curve represented permissible values, implying variable safety factors, and when it came to applying those values in the context of a linear cumulative damage summation, $\sum(n/N)$ was arbitrarily equal to unity.

136. This meant that fatigue test results, past and future, from this country and abroad, must first be converted by somewhat tortuous methods before comparisons could be made with the British Standard. That this was not a minor trouble could be shown by the statement of Prof. Boulton, who was understood to use the original Miner's rule, in conjunction with actual test values, while in the Standard $\sum(n/N)=1$ was a 'private' cumulative damage rule and had no relation to Miner's rule.

137. In that context, the fatigue values were better expressed in the conventional manner, as, for instance the American AASHO tests had done, and then the scatter of the summation results expressed as powers of 10. In one of the AASHO tests, for instance, results were expressed as $\sum(n/N)=10^{\pm 0{\cdot}257}$ which meant that the scatter was between 0·55 and 1·81, with the logarithmic mean $10^{\circ}=1$. Here, interest centered around the lower scatter limit 0·55, which for the purposes of Standards could be extrapolated to practical design probabilities.

138. Therefore, when the British Standard was next revised, and if cumulative damage results from spectrum tests were available, the present form might be abandoned and a physically meaningful presentation adopted.

Mr C. B. Pennington (British Railways) asked two questions. In the amendment to B.S.153, Table 2, there was an arrangement for Class F where for $f_{min}/f_{max}=0$ and a life of 10^6 cycles, the allowable values of f_{max} were 2·1 tons/sq. in. when f_{max} was tensile and 2·8 tons/sq. in. when f_{max} was compressive. (Fig. 22).

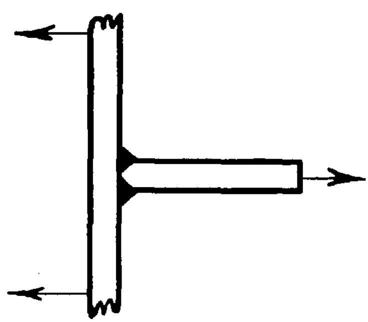


FIG. 22

140. Firstly, he wished to know whether or not the Author's figures were based directly on experimental results, and would the Author confirm that reversing the direction of the loading enabled one to use the figure of 2·8 tons/sq. in.

The Author, in reply, thanked Mr Berridge for showing pictures of fatigue cracks that had actually occurred in service. Too many people still believed that fatigue never happened, and certainly not to them. The Author wished that he had illustrated his Paper with practical examples, but problems brought to B.W.R.A.

by its members were guaranteed to be treated as confidential. It could only be said that numerous examples might have been shown, although admittedly, not in bridges. However, as stated in the Paper, fatigue cracks in bridges could easily become numerous in the future. There was much to be learnt from the mistakes in design, made in other fields. It would be of great benefit to everybody if there were less secrecy about service failures and congratulations were due to Mr Berridge for his example.

142. The failure shown in Fig. 9 was of particular interest, since it showed what was apparently fatigue buckling without fatigue cracking. It was believed to be the first one recorded in service. It had been found previously in the laboratory, but had never been adequately explained. It would be interesting to know if anyone else had encountered this phenomenon.

143. The detail leading to the failure shown in Fig. 8 was not covered by any of the clauses in the Standard. The experimental results closest to being relevant might be those for specimens with Charpy edge notches, for which the fatigue strength at 2×10^6 cycles was about $6\frac{1}{2}$ tons/sq. in.

144. With regard to the fatigue strength of wrought iron girders, it was believed that British Railways research department would, in the near future, be testing bridges which had been in service for a number of years, in order to determine their residual life. The results thus obtained should answer Mr Berridge's question.

145. With regard to Mr Berridge's advocacy of the use of high strength bolts, it would certainly seem reasonable to expect joints loaded with the bolts in shear (i.e., using friction grip bolts) to give good fatigue strengths. The Author was not aware of any fatigue test results that were available. However, if the joint was loaded so that tension was applied to the bolt, the evidence suggested, as Dr Spindel had pointed out, that fatigue strength was not as high as it was expected to be. In theory, prestressed bolts in tension took very little load and therefore should not break. But, in fact, British Railways research department had amassed ample evidence that they did.

146. As Prof. Boulton had pointed out, results for small polished and notched specimens were not strictly comparable to those for welded structures. However, any work in this field was of interest. Results for small polished specimens would help to build up the background for the work that had yet to be done on welded structures, where many variables would be involved. It was certainly not unduly surprising that values of $\sum n/N$ of less than 1 had been obtained with alternating loading. Directly comparable results for pulsating loading would be of interest.

147. It was very satisfying to note that Dr Alder, in his cumulative damage tests, had obtained better results for a Class F detail than for Glass G, thus confirming the constant amplitude results.

148. With Mr Newman's view, that butt welds had been penalized more than fillet welds, even though it had been the latter that had normally caused the trouble, there must be some sympathy. On the other hand, load carrying fillet welds had now been severely penalized and a considerable increase in design stress had been permitted for high class butt welds. Was it not likely that the fact that few failures had been recorded in butt welds was due to the existence of fillet welds in the same structures?

149. Mr Newman was quite correct in thinking that the bottom end of the scatter band of results for Class E transverse butt welds was anchored by, in particular, two types of weld which produce horrible reinforcement shapes or their equivalent, namely, butt welds made on a backing bar and butt welds made with deep penetration electrodes with no weld preparation. However, some types of automatic welding were quite capable of giving poor reinforcement shapes and trial welds should normally be made to define suitable welding conditions. In due course, some acceptable way might be found to define reinforcement shape, in such a way that unmachined welds

of good shape could be included in Class D. The following definition of such a shape was considered and might yet form the basis for a future Standard:

> 'Members fabricated with transverse shop butt welds made in the downhand position, having the weld reinforcement substantially symmetrical about the centre line of the joint, with a smooth contour, and with the profile running out smoothly into the material on each side of the joint, and with no under-cutting.'

150. It was certainly hoped that fatigue strength improvement methods would become more generally used, since the improvements that could be obtained were substantial and well defined. However, Mr Newman was right in thinking that the inspection problem was tending to stifle their use at present, although Commission XIII of I.I.W. were currently preparing specifications for the use of the various processes.

151. Dr Spindel's doubt as to whether fatigue failures occur in compression was, unfortunately, misplaced. For example, in the tests on large beams carried out by B.W.R.A. with support from the Civil Engineering Research Council, several failures in the compression flange had occurred, some of which had completely severed the flange. This could presumably be ascribed to the influence of residual stresses.

152. The reason for specifying, that points subjected to combined stresses should be designed on the basis of principal stress was simply that both the tests at Illinois and those at B.W.R.A. had shown that fatigue test results plotted on this basis gave good correlation with single stress results.

153. To deal adequately with the subject of crack propagation would require another Paper, although little of the information would refer to welded structures. Crack propagation measurements were, however, being made in the tests on large beams referred to above. The Author fully supported Dr Alder in his belief that, at least in thick sections, small fatigue cracks could set off brittle fractures. In his view this question urgently required study.

154. In reply to Mr Clark, the whole question of the influence of weld defects on the fatigue strength of butt welds was at present under study at B.W.R.A. However, the current work did not include the influence of cracks and every effort must be made to avoid them, at any rate in butt welds. To say that, once a fatigue crack had developed, all details tend to the same class was rather an over-simplification. The lower the class, the lower the design stress and consequently the slower the rate of crack propagation until, in the limit, a stress was reached below which a crack would not propagate. However, details in the lower classes have been put there because of their greater ability to initiate cracks.

155. The Author fully agreed with Mr Haas that the cumulative damage rule used in the Standard was not Miner's rule. It merely happened to be the same formula applied to a factored S–N curve. However, he did not agree that the Standard should quote the mean S–N values and the statistical scatter. The purpose of a Standard should be to present designers with the information they need in the simplest possible manner. Equally, in his view there was no difficulty about comparing test results with the design curves, particularly now that the various factors had been given.

156. In reply to Mr Pennington, practically no test results exist for this detail. Its allocation to any particular class was the result of guesswork. However, he might find some recently published work of interest.[10]

Reference

10. A. G. SENIOR *and* T. R. GURNEY. The design and service life of the upper part of welded crane girders. *Struct. Engr*, vol. 41, no. 10, October 1963, pp. 301–312.