

Paper No. 5058<sup>1</sup>

“Some Developments in Railway-Carriage and Wagon Construction.”

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Mr.  
Hendriksen.

Mr. E. E. HENDRIKSEN observed that it was satisfactory to see welding so well used in railway rolling stock. The constantly varying stresses and strains set up by loadings and vibrations would give the work a thorough test. Many designers were concerned with the effect of time on the strength of a weld and of the neighbouring parent metal, and it was advantageous that in the work described the joints were open to continual inspection; the reports on their behaviour would be of value.

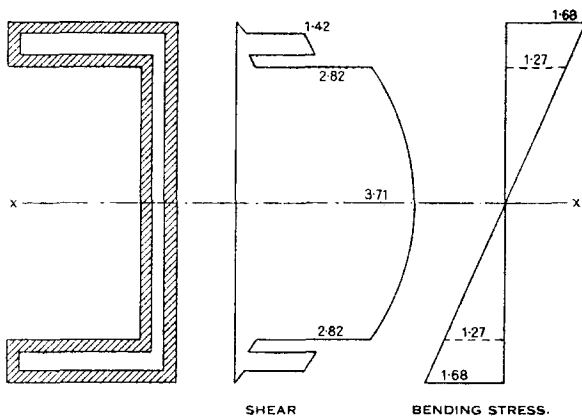
He wished to refer particularly to the welded connection of a steel section framing on to the plane face of a cross member. The calculations for the design were given on pp. 252 and 281. The bending moment at the end of the steel section was resisted by each part of the weld, and in proportion to its distance from the neutral axis. The use of the full section of the weld in finding the moment of inertia, and hence the stress  $f_2$  due to bending, was theoretically correct; it was to be noted that in the examples given the stress was increased by  $17\frac{1}{2}$  per cent. if the weld-metal adjacent to the flanges only were used in obtaining the stress from bending. That was larger than in most girder-designs, on account of the large effective thickness of the web of the weld-metal being equal to the flange thickness.

The question of shear stress in the weld-metal was of a more complex nature. In the first example, the Author had taken the shear over a depth of web equal to the clear distance between the flange welds, whilst in the example for test purposes (p. 281) the clear distance between the flanges of the steel section had been used. The shear stress  $f_1$  was given as 3.58 tons per square inch for the second method, whereas if the former method were applied it would be 3.92 tons per square inch, an increase of  $9\frac{1}{2}$  per cent.

<sup>1</sup> p. 231 (June).

It was well known that the actual shear stress in a member under Mr. flexure varied from zero at the extreme fibres to a maximum at the Hendriksen. neutral axis ; in a homogeneous rectangular member the distribution was parabolic, with a maximum of one-and-a-half times the average over the full area. In a large steel joist (with substantial flange areas), the shear distribution was fairly uniform over the clear depth of the web, and a very small portion was taken by the flange. In the examples under consideration, there were virtually two flange and web pieces placed adjacent, but with different clear distances. The shear stresses were somewhat complicated, and for clearness the test-example was shown in *Figs. 37* with the shear and bending

*Figs. 37.*



stresses. It would be seen that over the clear web depth, the shear varied from 2.82 to 3.71 tons per square inch. The Author's value of 3.58 was therefore only a little below the maximum. The weld-metal at any point was thus subject to equal horizontal and vertical shear stresses, and to a horizontal direct stress due to bending. The resulting condition was one of principal stresses of pure tension and compression, and a maximum shear stress. Those were obtained by the method indicated on p. 253, namely :

$$\text{Maximum principal stress} = \frac{f_2}{2} + \sqrt{\frac{f_2^2}{4} + f_1^2}$$

$$\text{Maximum shear stress} = \sqrt{\frac{f_2^2}{4} + f_1^2},$$

where  $f_1$  and  $f_2$  denoted the shear and bending stresses at the point under consideration.

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If that were applied to the figures given in *Figs. 37*, it would be found that the stress-conditions were:—

Extreme fibres:	principal stress of 1.68 tons per square inch,	shear stress nil.
Edge of web:	„ „ 3.53 „ „ „ „	2.89
Neutral axis:	„ „ 3.71 „ „ „ „	3.71

The stresses at the edge of the web were due to a combination of a shear stress of 2.82 tons per square inch with a bending stress of 1.27 ton per square inch.

It was evident that the governing factor in the strength of the connection was the shear stress of 3.71 tons per square inch at the neutral axis. Taking the ultimate shear strength of the weld metal at 18 tons per square inch, it was clear that the probable ultimate load would be at least  $11 \text{ tons} \times \frac{18}{3.71} = 53.4 \text{ tons}$ . As there would

no doubt be a certain redistribution of stress in the metal just before failure, a higher load could be expected, and that was indicated by the test. Further, since the shear stress alone was the governing factor, the two examples ought to be expected to give similar results. That was verified by the closeness of the test loads, 58.0 and 58.4 tons. That also indicated the uniformity in fabrication of the two welded connections and spoke highly for the operators. It would be instructive if observations at the time of testing could throw any light on the manner of failure. The considerations given above would be too laborious for ordinary design purposes, but they indicated the probable condition, and might lead to simple rules or criteria for regular use.

In the examples of the welding tests, it was interesting to notice that the average factors of safety with the adopted design stresses were

End fillet, stressed at 5 tons per square inch:	factor of 5.3
Side „ „ 4 „ „ „ „	4.5
Butt „ „ 5 „ „ „ „	5.7

It was obvious that a more uniform factor of safety was desirable, and hence the design stresses could well be readjusted.

In the test on the joint subject to bending and shear, it was of interest to note that if the shears and bending stresses given were combined in the manner previously indicated, the result would be principal stresses of 4.8 and 4.5 tons per square inch in the two cases, and a shear stress of approximately 3.7 tons per square inch in each case. An increase of the load to 12 tons would bring those values to approximately the design stresses, and using an average factor of safety of 4.9 (that was, between 4.5 and 5.3), the test load would be  $12 \times 4.9 = 59 \text{ tons}$ . That agreed with the actual test,

although it was based on the incorrect procedure of combining an Mr. average shear stress with the extreme fibre tensile stress. Hendriksen.

The AUTHOR, in reply, observed that he was very interested in The Author. Mr. Hendriksen's comments although he could not agree with all of them. It was certainly of great value to designers to know the effect of time on the strength of welds; reports to date on the welded work described in the Paper had not so far disclosed any weakness or failures of welds, although the vehicles concerned had been in constant service.

With reference to the remarks regarding the method of designing a welded joint subject to bending moment and shear, given on pp. 252 and 253, it had to be remembered that the method was stated to be only "approximate, owing to the several assumptions made" (p. 254). The method had the advantage of being quickly applied, and was intended for ordinary design-purposes, as it gave a safe upper limit. Admittedly, it was theoretically correct in the case of steel sections to take the full section into account in finding the moment of inertia and hence the stress due to bending, but it did not necessarily follow that the same procedure was to be adopted in an approximate method for determining the principal stresses for designing a weld similar to *Fig. 17* (p. 252). In that connection the following statement by Professor Morley in his book on the "Theory of Structures," p. 150, would be of interest:—

"*Principal stress in I sections.*—In I sections, whether rolled in one piece or built up of plates and angles, it has been shown . . . that the web area is of little importance in resisting the longitudinal direct stresses due to bending, or, in other words, it contributes little to the modulus of section, and . . . the flanges carry little of the shear stress."

In most cases it might be sufficiently accurate, when determining weld dimensions, to assume that the flange-welds took the bending stresses and the web-welds the shear stresses, even although the stress might be, as in the present case, increased by 17 per cent. on the safe side. It was a different matter when calculations underestimated stresses; Professor B. P. Haigh, however, had shown<sup>1</sup> that the yield loads for straight beams of channel sections, in certain cases ". . . were found to be only 43 to 80 per cent. of the calculated F value, according to the definition of the yield-point adopted," the usually accepted methods of calculation being used. It would be noticed that the maximum shear in *Fig. 37* was 3·71 tons per square inch, whereas if the shear were considered to be evenly

<sup>1</sup> "Constructional Tests on Mild-Steel Rolled Sections with Electrically Welded Joints." Trans. Inst. N.A., vol. lxxv (1933), p. 59.

The Author.

distributed over the clear distance between the inside flange-welds, the value obtained would be 3.89 tons per square inch (not 3.92, as stated by Mr. Hendriksen), which had the merit of being much more quickly determined and was also on the safe side. The distance between the welds of 6.16 inches used in the example on p. 281 was an oversight, and should have been 5.66 inches.

He agreed that *Figs. 37* was correct from theoretical considerations, as it was arrived at by the usual methods; the shear diagram had been obtained by calculating the horizontal shear values as done on p. 249 of the Paper, and the bending-stress diagram was easily drawn, with the value of 1.68 ton per square inch for the extreme fibre stress, as given on p. 281, example (ii). The method shown in *Figs. 37* should be used for any special cases, but it was, perhaps, rather laborious for ordinary drawing-office use. He rather questioned the statement that “. . . the shear stress alone was the governing factor . . .” in the failure of the welds tested, as he had tested many welds similar to that calculated on p. 281, but he had not seen one that showed first signs of failure on the neutral axis where the highest shear stress occurred. In all cases the members had commenced to fail at the top-flange welds, the failure gradually or suddenly proceeding downwards.

With regard to the welding tests shown on p. 280, Mr. Hendriksen had suggested that the factors of safety could well be re-adjusted, but the Author wished to point out that the tests shown on p. 280 were typical tests only, and had not been used for determining the factors of safety. Factors of safety for welds were not usually determined by the results of two isolated tests, but rather on the results of many tests, whilst other factors, such as fatigue and impact values, the position in which the electrode was to be used, and the properties of the parent metal, were also taken into account.