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DISCUSSION ON TECHNICAL NOTE 443

Strengthening of T moment of RHS joints

J. Szlendak and J. Bródka

Dr J. A. Packer, *Associate Professor, University of Toronto*

As the Authors are aware, various international researchers in addition to themselves have studied the behaviour of rectangular hollow section (RHS) welded T-connections loaded by in-plane bending moments. Recently, earlier German work on this topic was continued to investigate the influence of chord prestressing on the joint strength.[8] In these experiments, a decrease in strength with increasing chord compressive preload and decreasing width ratio β was found, resulting in the following empirical reduction factor for joint capacity[9]

$$f(n) = 1{\cdot}2 - (0{\cdot}5/\beta)n \tag{8}$$

This appears to be at variance with the Authors' observations that compressive chord forces have a negligible influence on the failure load of joints. Further experimentation on T-joints loaded by in-plane moments, with *high* levels of chord prestress, both with and without chord stiffening, seems warranted.

15. The most popular means of strengthening and stiffening T-joints is to weld a plate to the chord connecting flange; and research on the behaviour of this type of connection, loaded by in-plane bending moments, has already been reported by Korol *et al.*[10] Their theoretical modelling also employed yield-line theory, but in a more general form. Different failure mechanisms with corresponding solutions were postulated, depending on whether the yield-line pattern on the chord face was confined to the area covered by the stiffening plate or actually extended beyond it, as shown in Fig. 11, and the stiffening plate thickness was not restricted to 2 t_0. An amalgamation of the experimental data obtained by the Authors with that published by Korol *et al.*[10,11] would be prudent, along with a comparison of the respective yield-line solutions plus an evaluation of the succinct design recommendations, given by Korol *et al.*,[10] for plate-stiffened RHS T-joints subject to in-plane bending.

16. In their conclusions, the Authors allude to the International Institute of Welding (IIW) recommendations[12] for fillet weld sizes around RHS branch members, whereby the throat thickness is recommended to be greater than or equal to the branch member thickness for steels with yield stresses less than 350 N/mm^2, but 1·2 times the thickness of the branch member wall for steels with yield stress exceeding 350 N/mm^2. These provisions would then guarantee that the welds had a strength exceeding the base metal and would allow adequate deformation capacity for any type of loading. In practical structures, RHS wall thicknesses of up to 12·7 mm are not uncommon and at the 350 N/mm^2 yield stress level (the standard RHS grade in Canada, for example), huge multi-pass fillet welds would be required, particularly if the branch member were inclined. In such cases, end-

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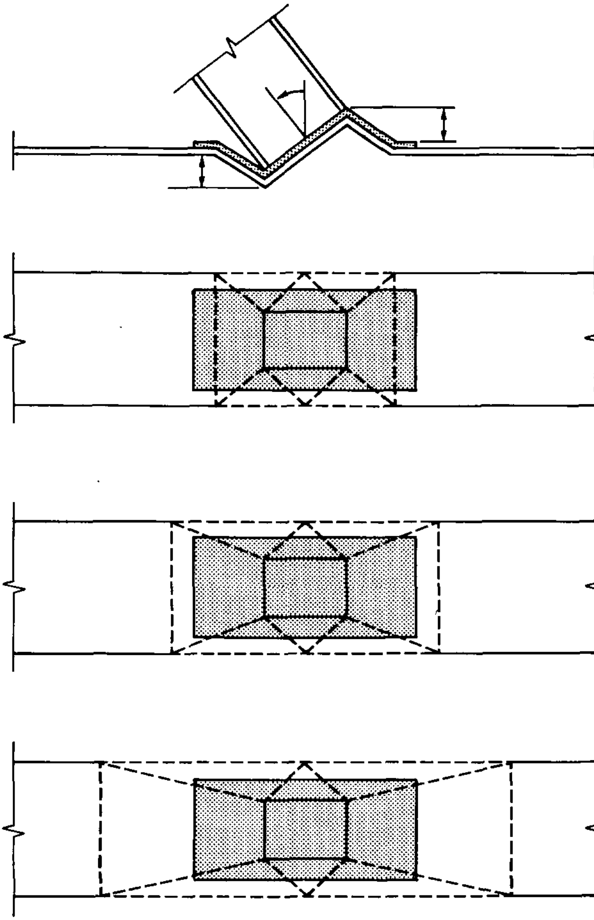

Fig. 11. Yield-line patterns considered by Korol et al.[10]

preparation of the branch members with subsequent butt welding is advisable. In RHS welded trusses, where the branch members are subjected to predominantly axial loading and frequently over-sized for aesthetic reasons, such fillet weld size recommendations are believed to be excessively conservative. Accordingly, CIDECT (Comité International pour le Développement et l'Etude de la Construction Tubulaire) research into the weldment design for RHS connections was commissioned at the University of Toronto and less restrictive design rules for the fillet weldments have now been proposed.[13]

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In Table 1, the compressive preload reduction factor $f(n)$[8,9] (equation (8)) is compared with our test results for the unreinforced in-plane bending T moment

Table 1

Test specimen no.	β	n	$\frac{M(n \neq 0)}{M(n = 0)}$	$f(n)$
U 1	0·46	0·41	0·80	0·75
U 2	0·50	0·41	0·80	0·79
U 4	0·49	0·45	0·80	0·74
U 5	0·48	0·45	0·70	0·73
U 7	0·72	0·24	0·93	1·03
U 9	0·74	0·24	1·00	1·04
U 11	0·73	0·42	0·94	0·91
U 13	0·73	0·42	0·89	0·91
U 14	0·71	0·23	0·93	1·04
U 16	0·72	0·24	0·93	1·03
U 18	0·97	0·40	0·89	0·99
U 20	0·99	0·40	0·80	1·00
U 21	0·97	0·21	0·83	1·09
U 22	0·97	0·21	0·77	1·09
U 23	0·97	0·24	0·93	1·08
U 24	0·98	0·24	0·90	1·08
U 26	0·96	0·41	0·85	0·99
U 29	0·49	0·47	0·50	0·72
U 31	0·49	0·46	0·60	0·73
U 32	0·50	0·50	0·77	0·70
U 33	0·50	0·50	0·69	0·70
U 35	0·74	0·25	1·00	1·03
U 36	0·73	0·25	1·00	1·03
U 38	0·75	0·49	0·65	0·87
U 40	0·73	0·48	0·65	0·87
U 42	0·74	0·24	0·95	1·03
U 43	0·74	0·24	1·00	1·03
U 44	1·00	0·48	1·00	0·96
U 46	1·00	0·48	1·00	0·96
U 47	1·01	0·25	0·98	1·08
U 49	1·01	0·24	0·97	1·08

joints.[3] This shows even more reduction in the strength of the joints than the formula (8) suggests. However, such a phenomenon could be observed especially for the ultimate strength of the joints. For the design strength, this influence is much smaller. Moreover, as is mentioned in the Technical Note, for the strengthened joints this influence is technically small, even for the ultimate load. Dr Packer's suggestion to test these joints further with the high levels of the chord prestress, i.e. $n = 0{\cdot}6/0{\cdot}8$ seems, therefore, to be warranted.

18. An analysis similar to the one reported by Korol *et al.*,[10,11] that is, with the yield-line pattern partially extended beyond the stiffening plate, has been shown by Szlendak in his thesis.[13] The different yield stress of the plate, as compared with the chord, and its thickness have been studied. In the conclusions, a length for the plate has been suggested which is considered to be sufficient. It is difficult to

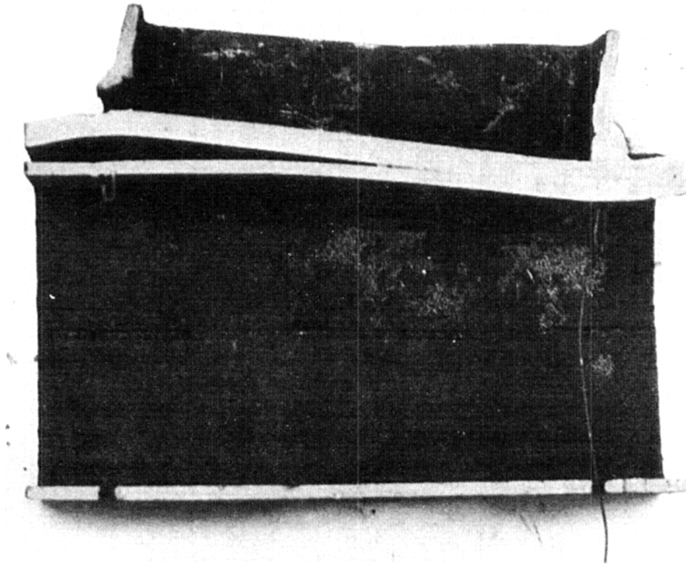

Fig. 12. Longitudinal cross-section of the joint strengthened with a chord flange plate

assume the yield-line model as shown in Fig. 11 to be a general case. The longitudinal cross-section of the yield-line pattern would suggest that in the tension side of the joint, the chord face wall is working together with the stiffening plate; however, as is shown in Fig. 12, they tend to be working together on the compression side only. Moreover, it would probably be difficult to estimate from such a model the yield strength of the joint for the almost equal or equal width ratio. This is possible when the model presented in the Technical Note is employed.

19. We agree with Dr Packer that for the large-sized RHS, especially with a considerable wall thickness, butt welding may be the better solution. Even then, however, the brittle fracture in the corners could be expected. For the plate stiffening, the designers should remember that the rotational capacity of such joints, especially for $\beta \approx 1$, is much less than for the others discussed above. This is a result of the much smaller flexibility of the plate, which should be thick enough to ensure the required strengthening. The new CIDECT research into the weldment design seems necessary.

20. We hope that Dr Packer's and our comments may contribute to a better understanding of the problem under discussion.

References

8. Mang F. *et al. The development of recommendations for the design of welded joints between steel structural hollow sections* (*T- and X-type joints*). University of Karlsruhe, December 1983, Report on ECSC Agreement 7210 SA/109.

9. De Koning C. H. M. and Wardenier J. *The static strength of welded joints between structural hollow sections or between structural hollow sections and H-sections. Part 2:*

Joints between rectangular hollow sections. Delft University of Technology, March 1985, Stevin report 6-84-19.

10. KOROL R. M., MITRI H. and MIRZA F. A. Plate reinforced square hollow section T-joints of unequal width. *Can. J. Civil Engng*, 1982, **9**, 143–148.
11. KOROL R. M., EL-ZANATY M. and BRADY F. J. Unequal width connections of square hollow sections in Vierendeel trusses. *Can. J. Civ. Engng*, 1977, **4**, 190–201.
12. INTERNATIONAL INSTITUTE OF WELDING, SUBCOMMISSION XV-E. *Design recommendations for hollow section joints—predominantly statically loaded*. IIW Annual Assembly, Oporto, 1981, IIW Document XV-491-81 (Revised).
13. PACKER J. A. and FRATER G. Performance of fillet weldments in hollow section connections. *Proc. Int. Conf. on Safety Criteria in Design of Tubular Structures*, Tokyo, July, 1986.
14. SZLENDAK J. *The ultimate load of a welded beam–column connections in rectangular hollow sections* (in Polish). PhD thesis, Technical University of Warsaw, 1982.