Considerations in the Design of Large-Size Welded Tubular Truss Joints

J. G. BOUWKAMP

THE RATIONAL DESIGN of welded truss joints built from tubes has been complicated by the radial flexibility of the chord tube wall. For certain types of joints this flexibility might be efficient, under static axial loads, since it provides, under increasing loads, a load-distributing action between branch members and chord tube. However, for most joints the flexibility is a source of severe stress concentrations. In connections subjected to dynamic loads, as for example off-shore well drilling structures, stress concentrations can be most hazardous.

Although even an approximate analysis of tubular joints of different configuration is extremely difficult, several designs are presently used and aimed at reducing the influence of the wall flexibility. This objective can be achieved by using thicker-walled tubes, or by inter-welding incoming branch members. In the latter instance the force acting normal to the chord tube can be reduced considerably. A second group of joints incorporates gusset plates or stiffening rings, which are intended to either contain the load transfer primarily within the gusset plate or to provide an element—the stiffening ring—to stiffen the chord wall and to distribute the forces over a larger part of the chord member wall.

Any structural danger in the latter two types of joints might no longer be related to the chord member flexibility, but a new potential hazard is introduced by the excessive rigidity of both the gusset plates and the ring stiffeners. This will undoubtedly cause a considerable restraint in the walls of the joining members and subsequently result in severe stress concentrations. It seems that a solution for the structural design of welded tubular joints under severe load conditions can be obtained only through a rational evaluation and comparison of the several different sources of potential failure. It is the objective of this paper to review those sources and to discuss possible structural arrangements to improve the overall stress distribution.

J. G. Bouwkamp is Associate Professor of Civil Engineering, University of California, Berkeley, Calif.

DIRECT MEMBER CONNECTIONS

These types of joints are designed either with or without an intersection between the incoming branch members, as shown in Fig. 1.

Although the non-interwelded joint has an advantage from the point of view of fabrication and welding as compared to the interwelded joint, it is structurally an inferior connection.

Considering the column wall between the two incoming web members, it is obvious that for thin-walled members $(t/D < 0.025)$ excessive deformations of this wall can be expected when the joint is subjected to large static loads. For alternating loads the critical area of the column wall is no longer located between the two web members, but rather along the root of the welds between the horizontal branch member and the column tube (line **a** in Fig. 1). The continuously reversing loads cause the circular cross section of the column to deform into an ellipse of varying shape. The successive compressive and tensile forces in the horizontal member shorten and elongate the diameter of the column section in the direction of the load (Fig. 2). This results in local
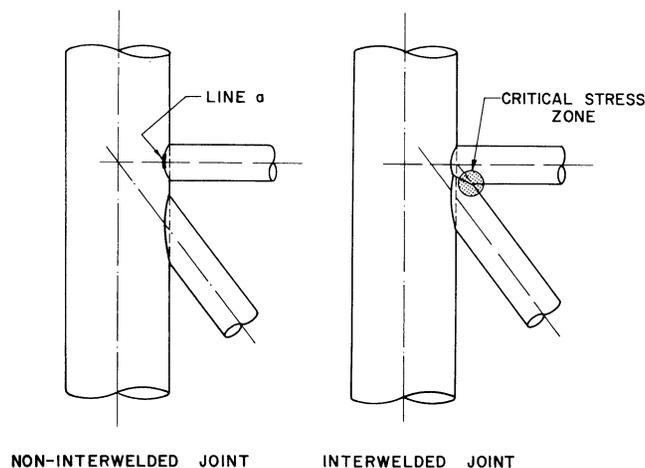

NON-INTERWELDED JOINT   INTERWELDED JOINT

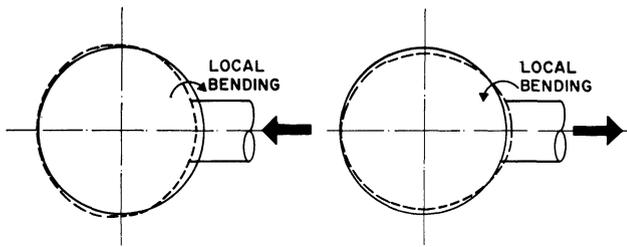*Fig. 1. Direct member joint*

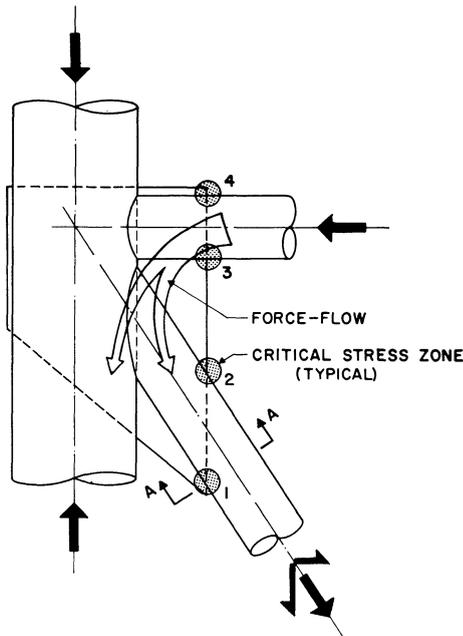*Fig. 2. Tube deformation under load reversal*

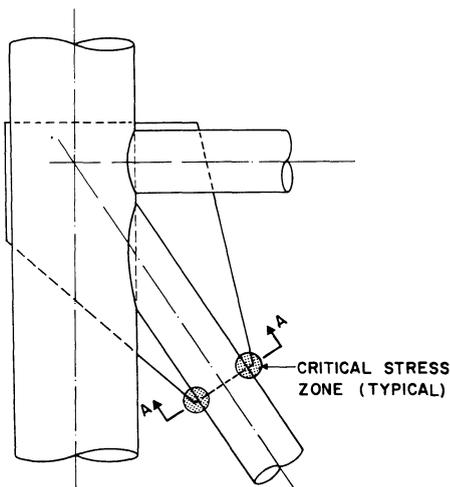*Fig. 3. Gusset joint Type I*

Fig. 4. Gusset joint Type II

bending of the column wall in the critical area. Depending on the load intensity and the flexibility of this wall, a slowly progressing or a sudden failure can be expected. For thinner walls one can expect that failure in the column wall will start locally after a limited number of severe load cycles, and will progress slowly by tearing of the wall.

For thicker walled members the increased radial stiffness will reduce the column wall deformations and subsequently the local bending stresses. This will improve the joint and result in an extended life for this type of connection. However, one can expect that due to the larger overall stiffness and better stress distribution failure will be more sudden and complete. A sudden failure of the wall of the horizontal web member along the entire weld between column and branch tube is also quite possible.

Another way to reduce the danger of flexural failure of the wall of the column tube is to interweld the web members. In this case a limited amount of welding between the web members (30 to 40 percent of the total weld line of the horizontal member) can be most efficient. The danger zone in these joints is located in the branch-tube walls near the common weld between the two web members (see Fig. 1). Failure of the wall of the diagonal tube in that area can be expected, both under static loads and under conditions of load reversal. One way to improve the structural efficiency of the joint is to increase the length of interwelding. The load in the diagonal member will then be distributed more evenly over the welds connecting the diagonal member to both the column and the horizontal member. A similar improvement can undoubtedly be achieved by increasing the wall thickness of the column member. The increased radial stiffness of this member will reduce the deformations of the column along the weld between the diagonal and the column member. Therefore a larger portion of the total load will be taken by this weld. Consequently the load to be transferred through the weld between two web members will be reduced.

It is not yet possible to make a prediction about the expected life of welded joints with directly connected members. Only general tendencies as effected by structural changes in member dimensions and the overall geometry of the joint can be noted. For example, increasing the column wall thickness over the length of the joint will increase the number of load cycles prior to failure. Also, for joints with thinner column walls a similar effect can be achieved by interwelding the two web members. The entire behavior depends primarily on the stiffness distribution. Only by balancing the stiffnesses of the different stress-raising elements can one design a fully effective joint. Approximate methods of analysis to investigate certain structural influences in these joints are under development.

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GUSSET PLATE CONNECTIONS

The gusset plate is often selected, particularly for large diameter column members, in order to prevent the concentrated action of the branch members on the column section. In such joints (see Fig. 3) the force in the horizontal web member is directly balanced by the horizontal component of the diagonal web-member force. The vertical component, however, is transferred by the welds connecting the gusset plate to the column wall. When structurally permissible, the gusset plate will pass through the entire column section and penetrate both sides as shown in Fig. 3.

Although this structural arrangement undoubtedly improves the critical state of stress in the wall of the column member, it develops critically stressed areas in the web members. These areas are located in the wall of the tubes near the edge of the gusset plate (see Fig. 3). The response of these joints under static and dynamic loads is different. Under static loads Area **1**, as shown in Fig. 3, is most critical. The higher diagonal force, distributed over the wall of the member, suddenly encounters the large stiffness of the gusset plate. Although the initially high stress concentrations in the tube wall will cause local yielding, ultimately under increased loads the stress distribution will become more uniform. As a result, a high ultimate static load will still be reached.

Under dynamic loads the response is quite different and depends on the restraint of the radial deformation of the wall of the branch members as influenced by the geometry of the gusset plate. The stresses in a Type I joint, Fig. 3, will still be high in the area marked **1**. However, local longitudinal and transverse bending in the tube wall will be of minor significance, since the complete tube section can deform radially without much restraint. This is shown in Fig. 5a. The compressive member force will cause an expansion of the column section, while the tensile force will cause the section to contract. With a gusset plate present on only one side of the section the tube can thus deform relatively unrestrained. Therefore the local bending in the tube wall at the beginning of the gusset plate (Area **1** in Fig. 3) will be relatively low. However, a quite severe state of stress will develop when the gusset plate cuts through the diagonal member as indicated in Fig. 4. In joint Type II the gusset plate completely restrains the two opposite sides of the tube wall. This will undoubtedly result in severe stress concentrations in the tube wall near the points of penetration. Figure 5 shows schematically the restricted deformations of the circular section. The cross-sectional restraint introduces longitudinal bending stresses which are magnified by the presence of the axial member forces. Considering the local stress development in the diagonal member of both joints, it becomes obvious that for axial loads Type II is structurally less desirable than Type I.
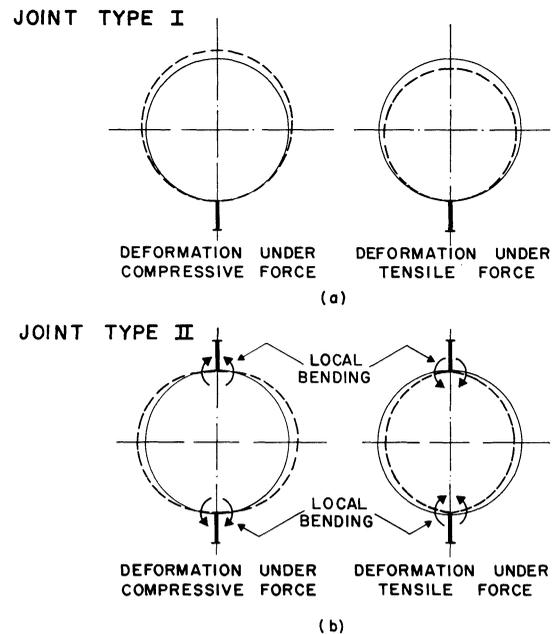

Fig. 5. Deformation of Section A-A of joints Type I and II

Both types, however, are similar in their response as far as the stress distribution in the wall of the horizontal web member is concerned. Under alternating loads the repeated reversal of the bending stresses in the tube wall at the points of penetration (Areas **3** and **4**, Fig. 3) is most severe. Area **3** is even more critical than Area **4** since the force flow (see Fig. 3) in this joint is concentrated in the gusset plate between the two web members. One can therefore expect that failure under repeated load reversals will occur in Area **3** of the tube wall, next to the gusset plate.

To reduce the critical local bending stresses and to improve both the static and dynamic load response of the joint, it is advisable to reduce the stiffness and subsequent local restraint of the gusset plate. This can be achieved by tapering those parts of the gusset plate which are immediately effecting the deformations of the tube wall. Figures 6 and 7 show examples of Type I and II joints with profiled gusset plates.* It should be noted that in this case Type II is probably better structurally than Type I, since the force in the diagonal member is distributed over two identical gusset-plate sections causing a more even load transfer.

In case the gusset plate cannot penetrate the entire column section (see Fig. 8) because of reasons other than structural, an additional cause of stress concentration has to be considered. Along the weld line between the one-sided plate and the column tube, not only a vertical shear

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For convenience, these plates are shown with sharp reentrant corners. In practice, these corners would be shaped to a radius as required in AISC Specification Sect. 1.23.2.
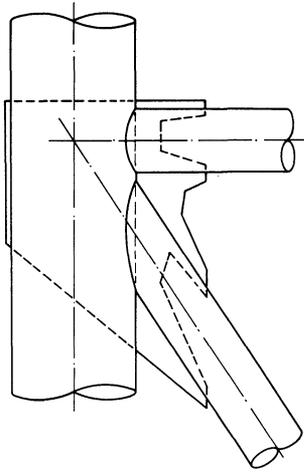
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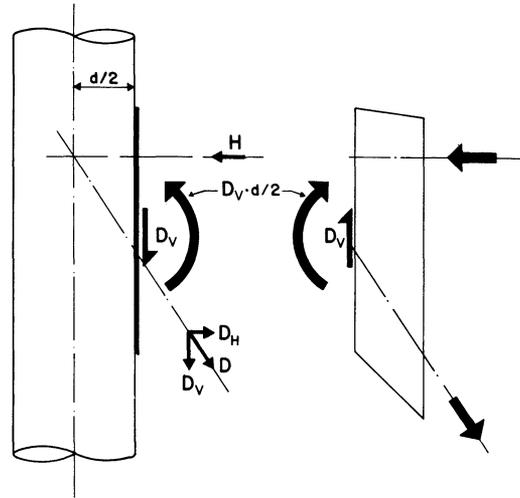*Fig. 6. Gusset joint Type I with tapered gusset plate*

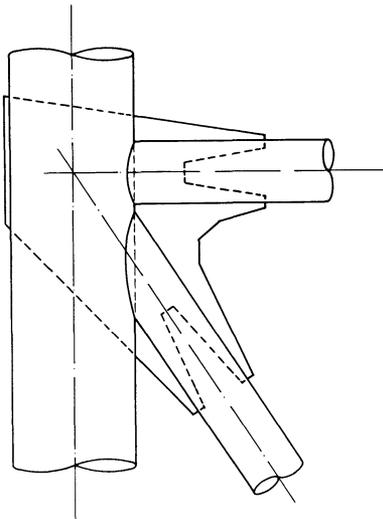*Fig. 9. Force diagram, one-sided gusset plate*

Fig. 7. Gusset joint Type II with tapered gusset plate
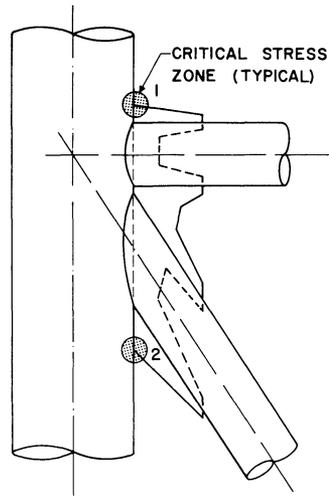

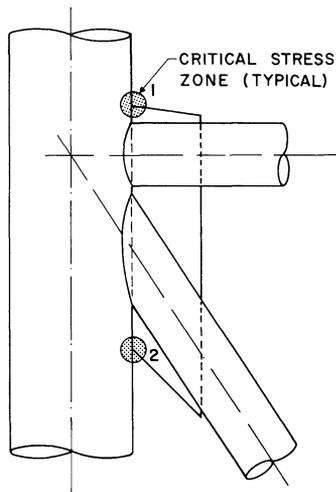*Fig. 10. One-sided tapered gusset plate*

Fig. 8. One-sided gusset plate

force, the vertical component of the force in the diagonal member, but also a moment $D_v \times d/2$ has to be transferred (see Fig. 9). This moment causes a rotation of the gusset plate against the bending stiffness of the web members and the radial stiffness of the column member wall. The resulting rotation of the plate introduces local bending moments in the tube wall in Areas **1** and **2** as shown in Fig. 8. The longitudinal moments in these areas are critical, particularly under the magnifying influence of the overall compressive force of the column member. The stress concentrations in the different areas of the web member walls are the same as discussed for joints with gusset plates penetrating the column section completely. To reduce the stress concentrations in those areas the gusset plate can again be tapered as shown in Fig. 10.

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Stress concentrations occur not only in the member walls but also in the gusset plate. However, the importance of the stresses in the plate is secondary to that of the stresses in the tube walls. The stresses in the gusset plate are due only to a concentrated load transfer and are therefore less critical than the superimposed membrane and bending stresses in the tube walls.

In the previous discussion the load transfer is assumed to take place entirely through the welds between tubes and gusset plate. This is not completely correct, since the welds directly connecting the branch members to the column wall are also transferring part of the load. However, in general one can expect that this load is only a small fraction of the total load to be transferred. Therefore the design of gusset plate joints should consider the detailed local behavior of the tubular members, particularly in areas where the gusset plate penetrates the tube walls.

In joints with one-sided gusset plates the interior of the column tube is often filled, partly or completely, with concrete. It is difficult to evaluate the response of such a joint. Should the concrete be expansive it probably will reinforce the steel column wall most effectively. However, any crushing of the concrete or separation between steel wall and concrete core will terminate the effectiveness of such a concrete fill. The joint response will then be basically similar to that of an "unreinforced" column section.

A possible solution to prevent the high stress concentrations in Areas 1 and 2, as shown in Fig. 8, is the application of a transverse stiffening plate or ring (see Fig. 11). In this case the radially increased stiffness will prevent any substantial rotation of the gusset plate and will subsequently prevent the development of dangerous stress concentrations.

RING-STIFFENED JOINTS

The purpose of the ring-stiffened joint (see Fig. 12) is basically to radially stiffen the flexible wall of the column member. These stiffeners indeed reinforce the column tube wall most effectively in the case of static loads. However, for dynamic loads a very serious problem arises in respect to the column tube wall between the two branch members. The tube wall in that area is subject to repeatedly changing critical longitudinal bending moments. These moments result from the shear deformation in the column wall between the two sets of ring stiffeners which are acting as rigid diaphragms. The tube wall between these diaphragms is subject to shear forces resulting from the horizontal forces acting laterally in the column wall. These forces deform the tube and cause critical bending stresses in Area 1. Therefore this area is most sensitive for failure under alternating loads. A gusset plate in that part of the joint might be effective in reducing the relative movement of the two rigid ele-
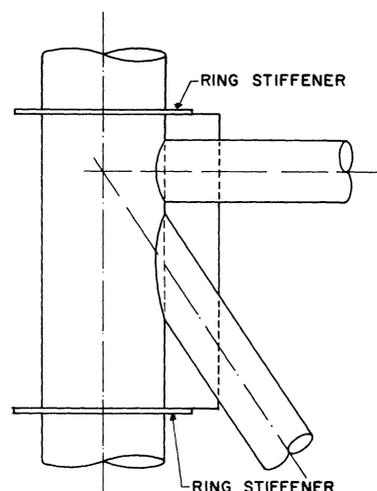

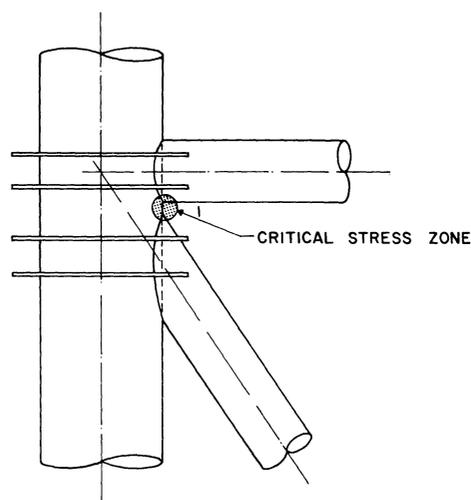*Fig. 11. One-sided gusset plate with stiffening end rings*

Fig. 12. Ring-stiffened joint

ments, but might on the other hand cause the development of stress concentration in other regions. The only effective way to reinforce such a joint is by increasing the wall thickness of the column member. However, if the increased wall thickness provides a sufficiently large radial stiffness to prevent serious local transverse bending stresses the joint might perform even better without ring stiffeners.

The use of a concrete fill in these joints can indeed be effective. To what extent these fills will be reliable for long periods of time is yet unknown.

An alternative, and probably structurally more effective design, incorporates ring stiffeners which coincide with the axes of the web members as shown in
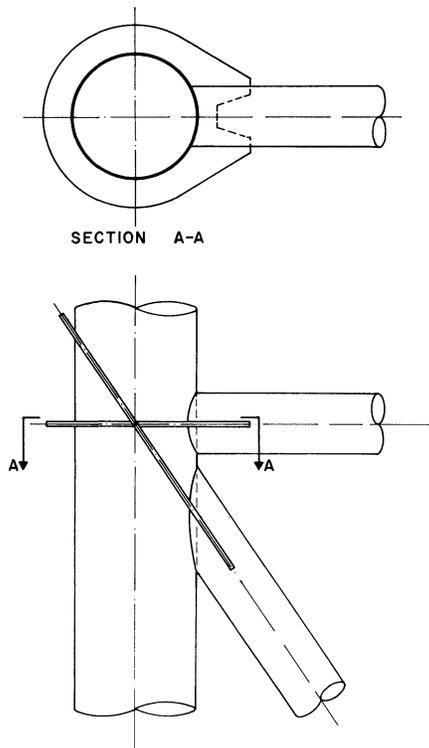
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SECTION A-A

Fig. 13. Ring-stiffened joint

Fig. 13. In these joints it might well be possible to omit that portion of the ring beyond the point of intersection of the two incoming stiffening rings at the center of the joint.

In general, to control the development of stress concentrations in the web-member walls near the location where the rings penetrate the tubes, a tapered extension of the ring is recommended as shown in Fig. 13.

CONCLUSIONS

Directly welded tubular joints, as well as joints with in-plane gusset plates and transverse ring stiffeners, show very distinct areas in which, due to severe stress concentrations in the walls of the tubular sections, a premature failure under repeated loads might occur. These critical areas are either located in the wall of the column member or in the walls of the branch members.

In the column they are a result of the radial flexibility of the column tube wall. The critical wall areas in the branch members are a direct result of the restraint of the tube wall imposed by the rigid gusset plates. The plates prevent these tubes from deforming freely radially, and introduce areas of local bending in the wall immediately adjacent to the locations where the gusset plates penetrate the tube wall.

To prevent the development of the critical areas in a column member, the thickness of the column tube should be increased. Another way to improve the stress distribution is to interweld the web members. In that case a considerable portion of the loads in the web members will be transferred through the weld between the branch members, rather than via the wall of the column tube.

For gusset plate type joints the development of stress concentrations can be prevented by tapering the gusset plates where they penetrate the column wall.

Gusset plates which do not penetrate through the complete column section cause the development of other areas of critical stress. These areas are located in the column wall near the ends of the gusset plate. These stresses can only be reduced by providing additional radial stiffness to the column wall. This added stiffness can be obtained by increasing the thickness of the column tube wall or by placing a ring stiffening element at either end of the one-sided gusset plate.

In general one can expect that concrete, used to partially or completely fill the column member, contributes to the local stiffness of the column wall. When expanded concrete has been used to pretension the column wall radially, one can expect that the stress distribution will probably be changed most effectively. However, should crushing of the concrete and a separation between the steel wall and concrete core develop, the beneficial effect on the joint strength will reduce rapidly.

Ring stiffeners provide only an added stiffness to the column wall directly in line with the incoming web members. The column wall between these members is still critical as in the case of a non-interwelded joint of profiled circular sections (direct member connection). Only a heavier column wall or possibly a concrete fill will have a favorable effect on the structural response of such a joint.

In general it can be concluded that for tubular joints the cause of most failures under load reversals and static loads are due to an inadequate distribution of the stiffnesses of both the tubular members and the stiffening plate elements.

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