Learning from a Structural Failure

An inside look at the contributing factors to a South African structural collapse due to weld failure.

By Spencer Erling

At the South African Institute of Steel Construction (SAISC), we sometimes get to review technical reports on various authorities' opinions as to what caused the failure of a structure. We are not in the business of judging people or what happened, but rather we look at these situations to see if there are any lessons that should be passed on to the steel industry. That way, at least some good arises from an unfortunate situation.

What follows is my opinion as to what happened in a recent failure of a structure. Even if I am wrong as to the exact cause of the collapse, the lessons described are still applicable to future projects.

The major component that failed was a two-story high truss girder over 36 m (118′) long, designed to carry a composite steel and concrete floor about 2 m (6′-6″) above the bottom chord as well as a light-weight trussed roof at the higher level. The loads from the composite floor were transferred to the main girder through secondary girders and tertiary composite floor beams. The concrete was cast into permanent steel decking. See Figure 1.

The main girder was analyzed in a very simple stress diagram and the main axial forces determined. The forces shown in Figure 2 are my estimates and they are "tweaked" to highlight the lesson.

Top and bottom chord members and all web members were chosen to be square hollow sections. All the profiles were formed by "boxing" 200 mm × 200 mm angles (L8×8) of various thicknesses to form hollow squares. In the instances where the boxed sections would be overstressed, they were to be reinforced by welding channels to the outside or by welding flats to the inside of the profiles (before boxing).

What went wrong?

In the opinion of the writer, the weld connecting the first diagonal member to the top chord of the truss (node A in Figure 2) failed, causing collapse of the whole system. Fortunately, no deaths occurred.

It appears as though there were many contributing causes. To try to understand the lessons, let's look first from the design point of view and then follow up with the fabrication and construction point of view.

Design Problems

In analyzing the axial forces, eccentricities appear to have been ignored. If we look at the detail of node B (see Figure 3) and consider only axial forces, the theory would be that the forces acting through the centers of gravity of the members

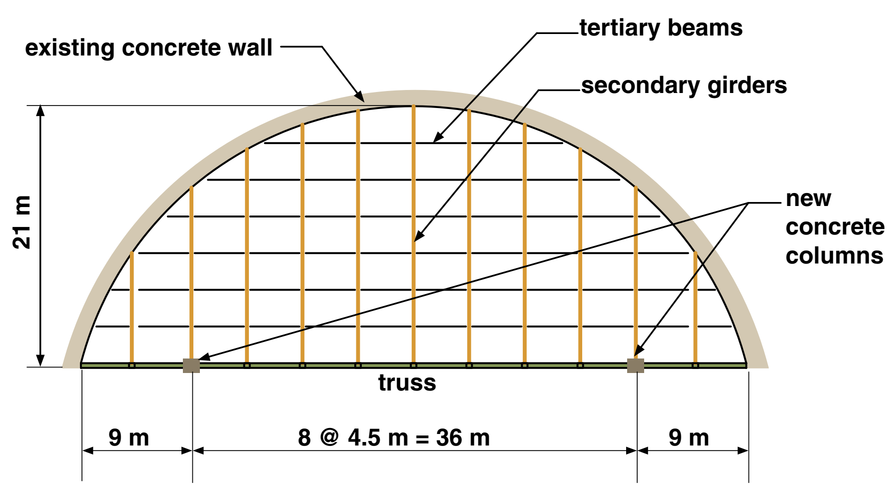

Figure 1. Plan view.

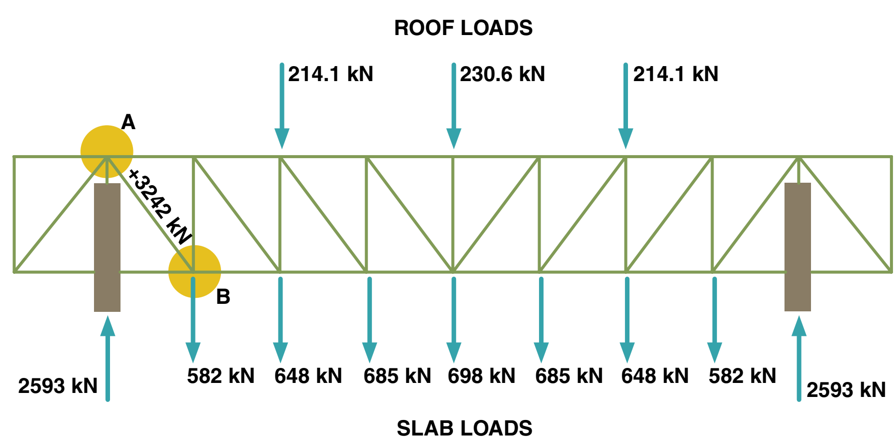

Figure 2. Elevation view of main truss, showing the approximated floor and roof loads. The truss is 6 m (20′) high.

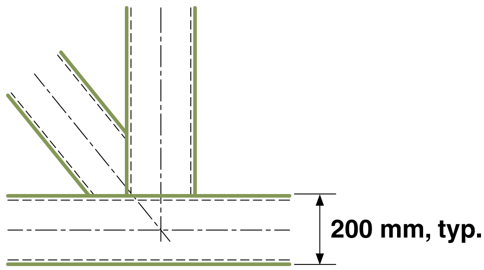

Figure 3. Strict interpretation of line diagram at node B.

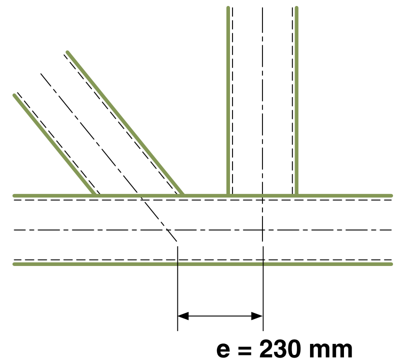

Figure 5. As-built geometry of node B. Note that other nodes had even worse eccentricities.

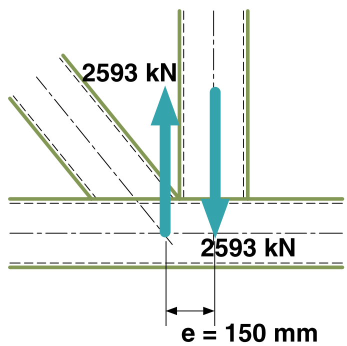

Figure 4. Contractor's interpretation of line diagram at node B.

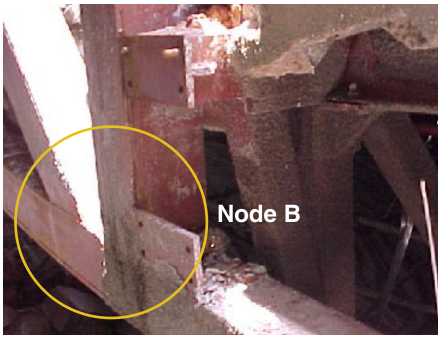

Figure 6. Photograph of node B after the collapse.

would all act through a common working point.

The engineers designed for the condition in Figure 3. However, the contractor's interpretation of the design drawings appears to have been similar to Figure 4.

Drawn simply to scale, using the contractor's interpretation on his detail drawings, an eccentricity of about 150 mm (6") occurs which results in a bending moment of about 390 kN-m (290 ft-kips) to be shared between the members framing into the node (obtained using a first order moment distribution analysis). Whatever the moment the first diagonal carries, if the member is already highly stressed in axial force alone, the addition of the moment puts the member in trouble: it would be severely overstressed. The situation was exacerbated by poor workmanship as shown in Figure 6, which added more eccentricity.

Similar eccentricity issues occurred at node A, but the determination of the eccentricities calls for some engineering judgment. This is a result of not being positive of just where the reaction occurs on the rather wide column, as shown in Figure 7. Most engineers seem to agree that it is likely to be at the quarter point of the concrete column—definitely not at the face of the column or in the middle of the column for this configuration.

Once again, the contractor's interpretation of the geometry was different from the strict axial force assumption of the engineers, as shown in Figure 8.

Whatever geometric approach is taken, the eccentricity and resulting bending moment needs to be considered in the design of the members.

In the case of node A, there is a simple solution that would have not only eliminated the eccentricity but also clearly ensured that the force path of reaction would be clear to the engineer, eliminating the need for a judgment call. This could be achieved by lowering the top of the column as suggested in Figure 9.

After detailed drawings were submitted for the engineers' approval, it was agreed that partial penetration welds would be used to connect the ends of the web members to the top and bottom chords (penetration of about 75% of the wall thickness of the hollow square members selected). What was not recognized by the contracting team is the fact that these partial penetration welds can never be executed in practice, not even under good workshop conditions.

Consultation at this stage with a welding expert would have stopped the job and forced a redesign of the truss girder, taking into account the transfer of the large design forces through the joints. The tubular solution would have been abandoned.

Construction Problems

For practical reasons such as crane access, lifting capacity in the shop, and transportation concerns, it was decided to weld the truss girder together on-site. This single decision had a very serious impact on the contractor's ability to fabricate—especially to weld the steelwork to acceptable standards—for two main reasons:

1. Half of the welds to internal members' ends now had to be done in the "overhead" position.

   By its very nature, gravity pulls the molten metal away from the molten pool being formed, making it almost impossible for the average welder to execute these welds competently. In addition, the welder has to deal with drops of molten metal falling on his helmet, shoulders, etc.

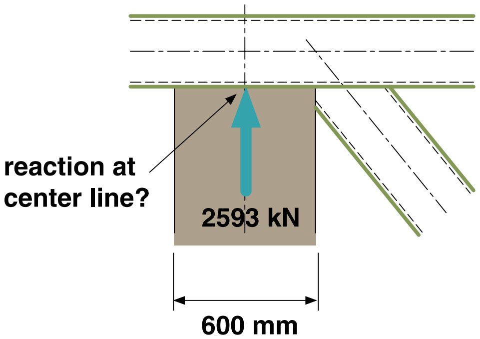

Figure 7. Strict interpretation of the line diagram at node A.

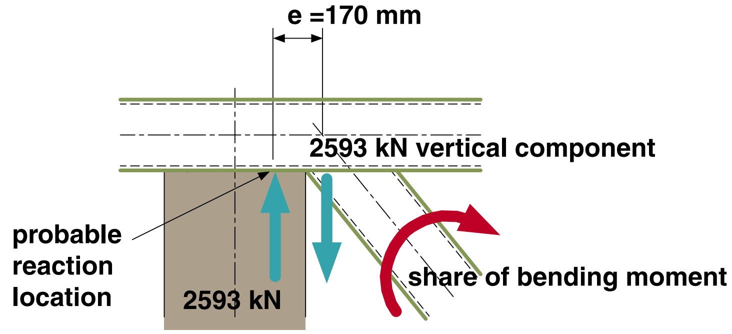

Figure 8. Contractor's interpretation of the line diagram at node A.

It is for this reason that the SAISC strongly recommends that overhead position welds should not be used for structural welds. The detail should have been changed to a design with an alternative welding position.

2. An important feature of fabricating structural steel under workshop conditions is the ability to ensure that the work is done to a suitable quality standard. By transferring this activity to a site without implementing adequate controls, quality standards will drop.

Lack of quality standards is, in fact, what happened, with many unacceptably poor workmanship errors contributing to the demise of the structure:

1. Poor cutting of web members to length, and poor end bevels
2. No preparation at the ends of the web members to ensure the partial penetration welds shown on the drawings could be achieved.
3. No one ever told the welder that partial penetration welds were required. Lacking weld preparations to guide him in the right direction, he proceeded to try to put down 6 mm (1/4”) fillet welds everywhere.
4. Very poor set up of joints. The best time to inspect penetration welds is when the joint has been set up.
5. Large gaps in the joints were reduced by introducing reinforcing bars of unknown quality into the joint to reduce the amount of welding required. This practice should never be condoned in any steel structure!
6. Eccentricities were exacerbated by poor workmanship, as we have seen.
7. No quality plans, weld procedures, proof of welder qualifications, inspection, non-destructive testing, or any other methods of ensuring good quality work were requested by the professional team nor offered by the contractor. These items are required in terms of the American Welding Society D1.1 welding code (referenced in South African design codes). These items exist to ensure that good quality welds can be made.
8. Nobody on the project team recognized that special qualifications are required for welding hollow square sections to each other. This was a major contributor to the collapse. The welder who actually did the work was a reasonably competent worker when it came to welding hot-rolled profiles, but he had not been trained in the additional implications of welding hollow shapes together.
9. Field-welded splices in the main chords were very poorly executed and in some cases not done at all.
10. No welding specialist—or for that matter, any structural steel inspector—was used to monitor the (poor) workmanship.

As you can see, the list of unacceptable fabrication and construction practices is long. Considering the confluence of the design and construction errors I have described, it should come as no surprise that the girder collapsed due to a weld failure. ★

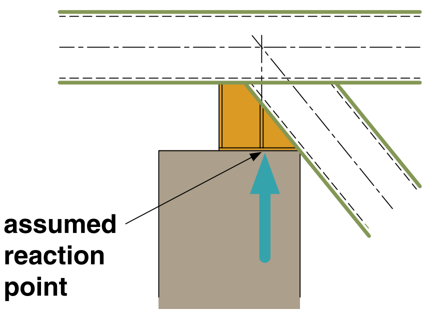

Figure 9. One possible solution that would eliminate eccentricity at node A.

Spencer Erling is Education Director for the South African Institute of Steel Construction (SAISC). This article is reprinted from the February 2005 issue of Steel Construction, *the official journal of SAISC.*