COLLAPSE OF A STEEL STRUCTURE AS A RESULT OF LOCAL BUCKLING

Heiko Merle, Jörg Lange

Institut for Steel Structures and Material Mechanics, Technische Universität Darmstadt, Germany
merle@stahlbau.tu-darmstadt.de, lange@stahlbau.tu-darmstadt.de

Keywords: stability, buckling, local joint collapse,

Abstract. The collapse of a steel structure required a detailed analysis of the reasons for the catastrophe. Several parts of the whole process of the design and construction had to be assessed. The results of this analysis are presented. Furthermore it will be shown how the design process, the autonomous checking procedure of the structural design, the workmanship, the construction on the erection side, the material characteristics as well as human failure affect the failure.

1 INTRODUCTION

Trusses with hinge joints experience only tension and compression forces. Compression forces in combination with hinge joints lead to a structural design using the well known Euler's cases especially the Euler case II. By designing the hinge joints the mounting of the joints has to be considered. This asks for a long connection area. Due to the compression, the joints have to be as compact as possible. These two oppositional requirements have to be considered in the design and the construction. The analysis of a collapse of a truss within the construction of a coal power plant showed that the failure of non-compact joints led to a catastrophe. The non-compact joints increased the buckling length severely beyond the Euler case II. The collapse of the joints led to a global collapse of the truss.

2 DESCRIPTION OF THE BEARING STRUCTURE AND ITS LOADS

Power plants have as main load-bearing structure a boiler supporting steelwork. This steelwork bears the steel boiler on its inner side. Several working platforms, power piping and the technical installations are attached to the supporting steelwork. During the erection the boiler supporting steelwork has to be built first. Subsequent the boiler itself as well as the platforms and installations will be fit in. Therefore several auxiliary steel structures are needed.

The building described in this paper contains auxiliary platforms up to the top of the main structure. These platforms were mounted in segments and held by the boiler supporting steelwork at several levels. One auxiliary structure was seated at level +78.00 m and rose up to the level of +145.00 m. At level 78.00 m the structure was held by a truss. The boiler supporting steelwork bore this truss. Figure 1 shows the truss with a height of 4740 mm and a length of 36500 mm. This girder was held out of its plane by several supports. Therefore buckling out of the plane can be neglected.

The girder's cross sections were H-Beams of different sizes. The tension forced bottom chord was a hot rolled H-Beam of 700 mm height and 300 mm width with a steel grade of S 355 (yield strength = 360 N/mm²). The upper chord was a welded H-Beam of 2100 mm height and 400 mm width with a steel grade of S 235 (yield strength = 240 N/mm²). This member was part of the final structure and therefore its design had to take care of additional requirements. Because of the large slenderness of the upper chord it

was stiffened with many braces. The trusses outer diagonals were hollow sections with a height and width of 350 mm and a thickness of 30 mm with a steel grade of S 355.
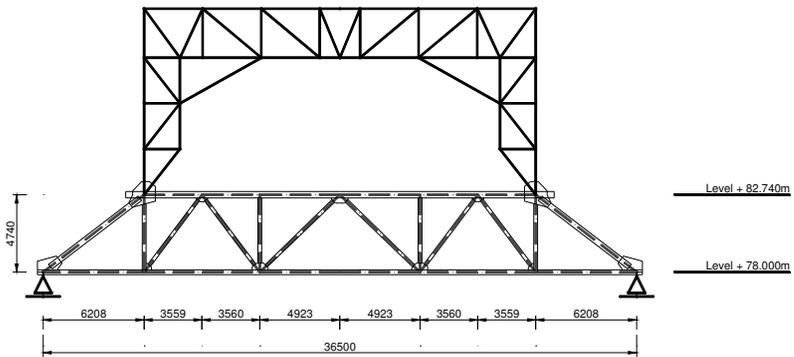
Figure 1: Truss at level +78.00m

A portal frame transferred the loads from the sections standing above into the truss. This structure may be reduced to the single girder with vertical loads from the portal frame. A static analysis of the upper auxiliary steel structure as a two-dimensional framework led to four main vertical loads for the girder. The loading points were the outer joints of the upper chord and the outer diagonal. These loads were design loads with specific partial safety factors. Figure 2 shows the loads and loading points.
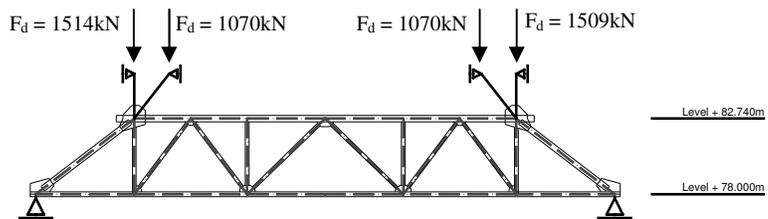
Figure 2: Truss and design loads

3 INTERNAL FORCES

The inner diagonals as well as the vertical struts were designed for small forces with max $N_d = 130$ kN. The bottom chord got a tension force of $N_d = 3300$ kN. The upper chord received a compression force of $N_d = 2500$ kN. Due to its large dimension buckling was eliminated. At last the outer diagonals were analyzed. They receive a compression load of $N_d = 4160$ kN. A comparison of the original structural design and a new and autonomous structural design after the collapse led to the same compression and tension forces. All members were strong enough to carry the applied loads.

4 DETAILED ANALYSIS OF THE OUTER DIAGONALS

The structural design of the outer diagonals is governed by buckling according to Euler's case II. Figure 3 shows that this Euler case relies on non-sway hinge joints at both ends. By using the national code the ideal buckling load as well as the ultimate load can be analyzed.
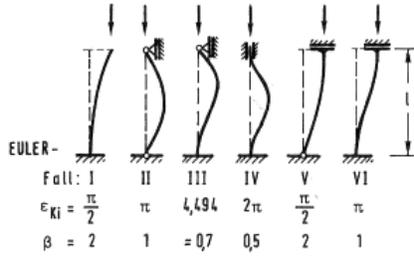
Figure 3: Euler's cases of buckling taken from [1]

$$N_{ki,z} = \frac{\pi^2}{s_k^2} \cdot EI = \frac{\pi^2}{(781cm)^2} \cdot 21000kN/cm^2 \cdot 66112cm^4 = 22442kN \tag{1}$$

By using the German code DIN 18800 [2] the effective slenderness can be calculated with the help of the buckling curve c.

$$\bar{\lambda} = \sqrt{\frac{N_{pl}}{N_{ki}}} = \sqrt{\frac{384cm^2 \cdot 36kN/cm^2}{22442kN}} = 0.785 \Rightarrow \kappa = 0.68 \tag{2}$$

The hollow sections ultimate load is defined by:

$$N_{ultimate} = \kappa \cdot N_{pl,d} = 0.68 \cdot 12557kN = 8539kN \geq N_d = 4160kN \tag{3}$$

The ultimate load is twice as big as the compression force of the outer diagonal. Therefore a collapse of the outer diagonal according to Euler's case II is not expected.
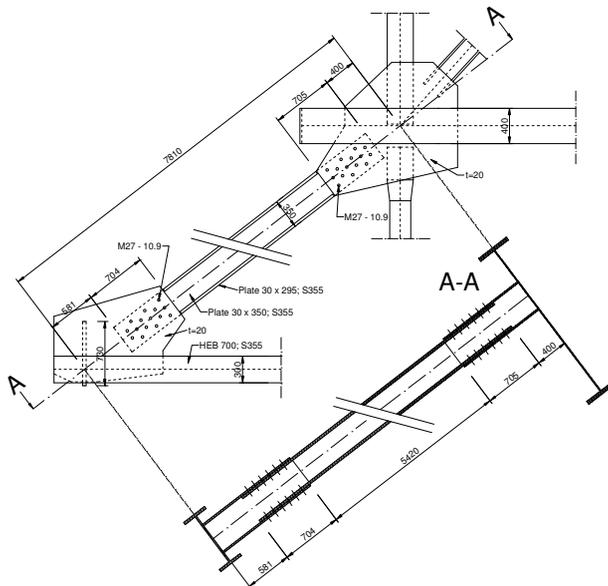
Figure 4: Construction of the outer diagonal and its joints

By analyzing the workshop drawings it can be assessed that the support conditions are not as assumed in Euler's case II. The top and bottom flanges of the hollow section were cut out at the end of the beam to enable the boltability. Figure 4 shows an overview of the outer diagonal and its joints. Special attention shall be given to the 704 resp. 705 mm of the compression strut that are without top and bottom flange.

The hollow section is connected with the joint plate by bolts. The joint plate is connected to the chord by a welding seam. Therefore the modeling of the outer diagonal to the Euler's case is incorrect. The slenderness of the joint plate and its length lead to the model of a segmented beam with different stiffnesses. Rotation springs as well as translation springs have to be used at the end of the segments. Whereas the Euler' case II has an elastic buckling mode the segmented beam has a combination of an elastic buckling and a rigid body motion (fig. 8). The stiffness of the springs as well as the relative stiffness of the equivalent end segments built by the joint plates are difficult to define. Therefore the whole truss was modeled as a three-dimensional finite element system.

5.1   Finite element method – elastic eigenvalue analysis

By using the finite element software SOFiSTiK the top as well as the bottom chord, the outer diagonals, and the joints of the girder were modeled with shell elements. The stiffeners of the upper chord were modeled too, to avoid local buckling effects. The inner diagonals and the vertical compression struts were modeled using beam elements. A very fine discretization of the model was used to get results as close to reality as possible. The model was supported as shown in figure 1. To avoid a horizontal displacement of the system a support in the point of symmetry was modeled.

The material characteristics were ideal elastic and ideal plastic functions with a yield strength of $f_y = 240$ N/mm² (S235) resp. $f_y = 360$ N/mm² (S355) and a modulus of elasticity of $E = 210000$ N/mm². The loads were as shown in figure 2. Figure 5 gives a view of the finite element model.
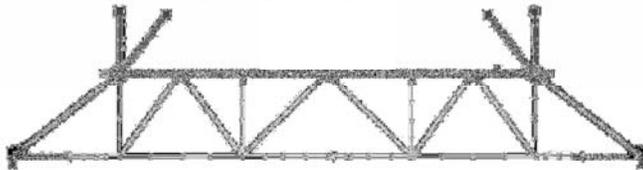
Figure 5: General finite element model

Figure 6 (left) shows the connection between the outer diagonal and the bottom chord whereas figure 6 (right) shows the connection between the diagonal and the upper chord.
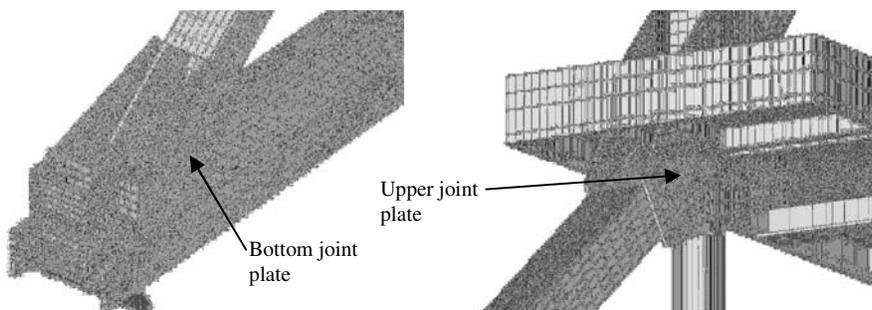
Figure 6: Connection between diagonal and chords in detail

By using the finite element model and the loads an elastic eigenvalue analysis was done. 10% of the design loads were applied to the system. Thirty eigenvalues with the associated buckling modes were

calculated. The eigenvalues can be divided into two major groups. Firstly there are many values with local buckling figures of the slender web of the upper chord. These values will not be pursued. The other group is made up of eigenvalues with buckling modes containing both outer diagonals.

The result of this analysis is shown in figure 7. Eigenvalues 1 to 3 are buckling modes of the outer diagonals. The first and second mode is a rigid body motion of the diagonal with a sideways buckling of the bottom joint plates. There is a symmetric rigid body motion of both diagonals in eigenvalue 1 whereas in eigenvalue 2 an asymmetric rigid body motion can be observed. The third eigenvalue describes a rigid body motion of the diagonal with a sideways buckling of the top joint plates. The first eigenvalues are close together between the load factor 13.1 and 14.1. By using 10% of the design load, the ideal buckling load can be calculated.

Eigenvalue 1:   Eigenvalue 2:   Eigenvalue 3:   Eigenvalue 27:
load factor 13.1   load factor 13.7   load factor 14.1   load factor 59.1
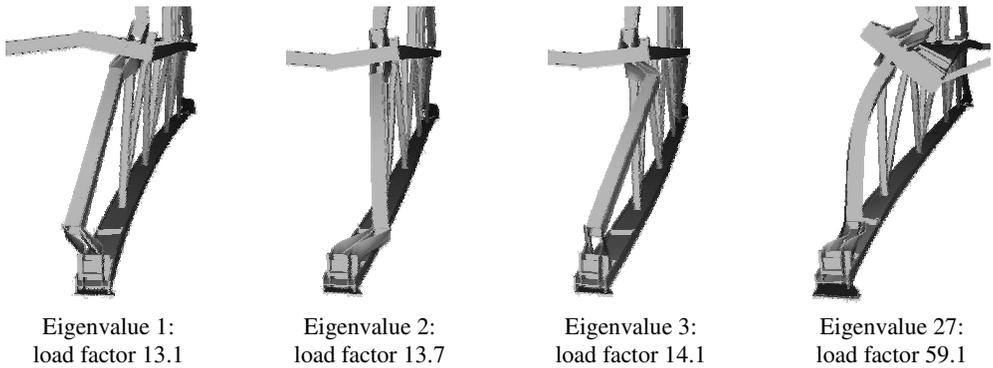
Figure 7: Results of the eigenvalue analysis: First, second and third eigenvalue with rigid body motion and 27th eigenvalue with buckling mode of Euler's case II

Eigenvalue 1: $N_{ki,1}$ = 5370kN   Eigenvalue 2: $N_{ki,2}$ = 5600kN   Eigenvalue 3: $N_{ki,3}$ = 5780kN

Figure 7 shows the 27th eigenvalue and the buckling mode analogous to Euler's case II. This buckling mode has an ideal buckling load of $N_{ki,Euler}$ = 24200 kN with the load factor of 59.1. Compared with formula (1) there is an explicit accordance between the two ideal buckling loads. This shows the usability of the finite element model. Altogether the first three eigenvalues show a rigid body motion. The failure mode is not the buckling of the outer diagonal but rather the buckling of the joint plates. The joint plates act like a spring at the end of the diagonal. Schmidt et al. [3] showed an equivalent system that is given in fig. 8.
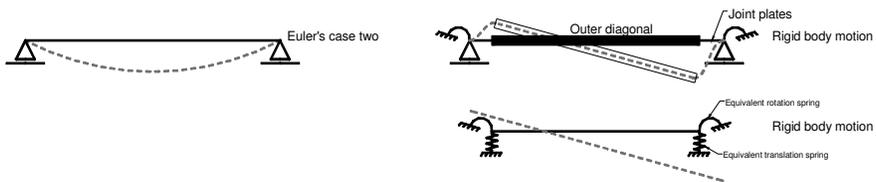
Figure 8: Equivalent spring system

5.2   Finite element method –plastic and nonlinear ultimate load analysis

To get the ultimate load of the finite element model a plastic and nonlinear load analysis was used. Therefore the ideal elastic and ideal plastic function of the material characteristics was used. To include the nonlinear secondary order theory the first eigenvalues buckling mode was fit in the model as its imperfection. To take a realistic value the imperfection was set to 1/500 of the length of the joint. Then an

ultimate load iteration was started. Within load steps the design load was raised to the ultimate load of the system.
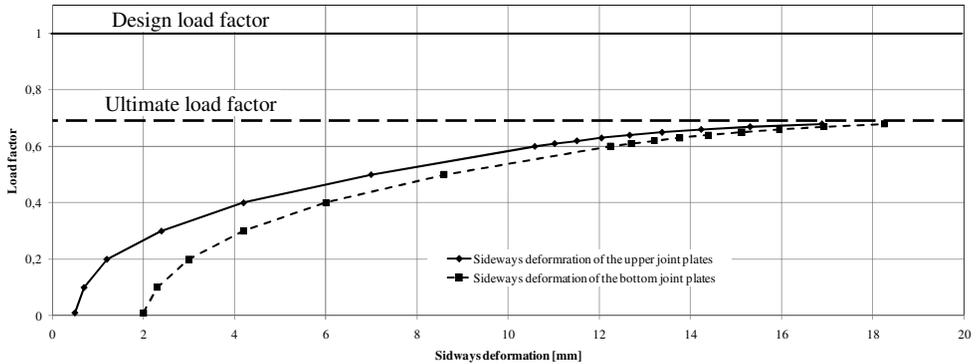
Figure 9: Sideways deformation of the joints in the plastic and nonlinear load analysis

Figure 9 shows the joint plates sideways deformation of the connection between the chords and the diagonals which are shown in figure 6. The plates get a clear nonlinear sideways deformation. At the load factor 0.69 the deformations will be infinite and the outer diagonals as well as the joint plates fail. At this load factor the system analysis is not convergent and the whole system collapse at 69% of the design loads. The deformation figure in the last convergent load step is affine to the first buckling mode of the first eigenvalue.

6 RECONSTUCTION OF THE COLLAPSE BY ANALYSING THE SCRAP

After the collapse the scrap was analyzed. Figure 10 shows in the upper left photography the upper part of the diagonal with its connection to the upper chord. The joint plates have a sideways deformation affine to the finite element solution. The upper right photography shows the diagonal with its joint plates to the bottom chord as well as the bottom chord. Again the sideways deformation of the joint plates is clearly visible. The bottom photography shows the bended bottom chord, the joint plates and the straight diagonal.

Figure 10: Debris after the collapse

As the finite element system showed, the connections between the outer diagonals and the chords fail first. In this moment the whole system is still stable. The inner forces of the truss without the diagonals create bending moments in the bottom chord. These bending moments create the extreme bending deformation shown in figure 10. Due to the large deformations the whole girder slips off the support and the system collapses.

7   CONCLUSION

In the process of design and construction of steelwork structures there is a straight progression. Firstly the structural analysis gives the loads and forces as well as profiles and defined materials. Subsequently the detail design with workshop drawings and detail drawings follows.

The truss was firstly designed as a framework. All structural elements and their material definitions were determined by using the theory of stability respectively the elastic second order theory. This structural analysis was done by the structural engineer. After this the design engineer determined the structural details. He has to be aware of problems like the boltability and the erection. The design engineer receives the forces and the stresses from the structural analysis. The load flow in the details had to be analyzed. The discrepancy between a compact design of the connection and montage aspects as well as the boltability had to solved. Therefore the structure had to fulfill structural restraints as well as the static requirements. After the detail design was finished the structure was fabricated.
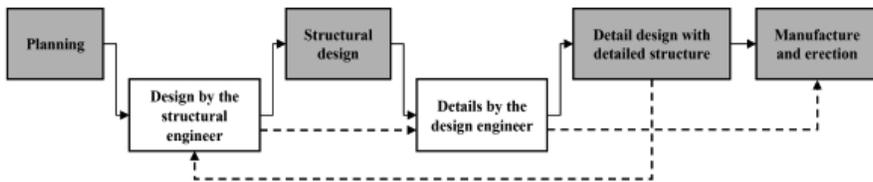
Figure 11: Progression of design and construction

There was no feedback loop between the engineers. Figure 11 shows that after the construction of the details the structural engineer should have checked the design. The outer diagonals in coherence with the new designed details had to be analyzed again. There the engineer had to notice that an important alteration of the support conditions of the outer diagonal happened. An updated structural analysis should have been done. This might have led to the recognition of the structures ultimate load and failure criterion. The design engineer would have had to redesign the details to fulfill the requirements of the structural design. This kind of feedback loop will take more time but maybe the collapse of the girder could have been avoided.

REFERENCES

[1]   Petersen C., "Stahlbau". Vieweg , Braunschweig , 1993

[2]   DIN 18800-2 "*Steel structures – Part 2: Stability – Buckling of bars and skeletal structures*", 2008/11

[3]   Schmidt H. et al., "Ein ungewöhnliches Stabilitätsproblem verursacht Schadensfall – An uncommon stability problem causes failure", Stahlbau **77** (12), 862-869, 2008

[4]   Unterweger, H.; Ofner, R, "Traglast von Verbandsstäben aus Hohlprofilen mit quasi-zentrischem Knotenblechanschluss – Load bearing capacity of bracing members with almost centric joints", Stahlbau **78** (6), 425-436, 2009