Causes of Structural Failures with Steel Structures

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Abstract

This paper is based on the experience from investigating over 400 structural collapses, incidents and serious structural damage cases with steel structures which have occurred over the past four centuries. The cause of the failures is most often a gross human error rather than a combination of "normal" variations in parameters affecting the load-carrying capacity, as considered in normal design procedures and structural reliability analyses. Human errors in execution are more prevalent as cause for the failures than errors in the design process, and the construction phase appears particularly prone to human errors. For normal steel structures with quasi-static (non-fatigue) loading, various structural instability phenomena have been observed to be the main collapse mode. An important observation is that welds are not as critical a cause of structural steel failures for statically loaded steel structures as implicitly understood in current regulations and rules for design and execution criteria.

Keywords: Steel structures; static loading; fatigue; structural failures; collapses; incidents; structural instability; gross human error; execution; construction phase; welds; regulations.

1 Introduction

This paper is based on the experience from investigating over 400 failures, incidents and structural damage cases with steel structures in buildings, bridges, chimneys and other civil engineering structures. These accidents have occurred in Sweden and elsewhere over the past four centuries.

The experience from being a consultant and investigator of accidents with steel structures will be summarized with a few examples being discussed more in detail. Because of the sensitive nature of many such accidents the reference for most cases will be made here in such a way that identification should not be possible.

The purpose of the discussion is to identify what seems to be the major problems with regard to structural safety of steel structures, in order to reduce the risk for similar events to happen again. It should be emphasized that the background for the paper is not a research program but rather the result of the writer being commissioned for investigating a large number of structural collapses, incidents and structural damages which have occurred in Sweden and elsewhere. It could be argued whether the observations discussed in this paper are representative and whether the conclusions drawn in the paper are of a general nature. However, considering the large number of cases, constituting a fair portion of the accidents which have occurred with steel structures in the region, it is believed that the findings should have some general significance.

2 Causes of structural failures

Most of the failures with steel structures investigated are with (quasi-)static (non-fatigue) loading. This is a reflection of the fact that most civil engineering structures are indeed acting under service conditions where variations in loading is not great enough to cause fatigue problems.

The discussion of causes of structural failures with steel structures will focus here on three different conditions: failures for (quasi-)statically loaded steel structures in the erection/construction phase, such failures occurring in the service phase and finally fatigue damage.

2.1 Static loading, construction phase

Of the failures occurring for normal steel structures with static loading, a large proportion occur during the construction phase. The most common cause is incorrect erection procedures which do not consider the various instability problems during all the stages of erection and installation.

2.1.1 Floor deck in commercial building

A well-publicized collapse during construction of a three-story commercial center building occurred in Stockholm in 2008, see Figures 1 through 4 [1]. This case is of interest here because it is a lucid example of gross human error. This failure is however not representative for most cases investigated, in that the error was in the structural design, not the execution and workmanship.

Figure 1. Commercial center building under construction, view half an hour before structural collapse

Figure 2. Same view as in Figure1, after collapse

Figures 1 and 2 show a view of the floor deck before and after the failure. The structural member causing the failure is the welded 1400 mm high I-girder visible in the center of Figure 2, introduced in the structural frame in the planning of the building because the column to the right of the I-girder had to be relocated from the normal system grid, the I-girder providing intermediate supports for deck girders in the perpendicular direction.

Figure 3. Welded 1 400 mm high I-girder which initiated the structural collapse shown in Figure 2

The I-girder was designed using a computer program. When transferring the dimensions obtained from the program to the drawings, the throat thickness of the fillet welds was confused for the web thickness. Thus the web thickness as specified on the drawings of the 1 400 mm high I-girder was 7 mm, instead of at least 12 mm required from a correct design. However, a further gross human error was introduced in that no web stiffeners were provided over the right-end column support.

Figure 4. Local buckling of the web of the I-girder over the column support, where web stiffeners are lacking

Figure 3 shows the deformed I-girder after the collapse, the fallen perpendicular deck girders and prefab deck elements fallen to the ground.

The typical local buckling behavior of the overloaded web of the I-girder over the column support where web stiffeners are lacking is visible in Figure 4.

When simulating the collapse behavior in the investigation after the failure all details including the action of the web of the critical I-girder were accurately predicted.

Although the consequences of this collapse were indeed very serious, with one person killed and at least two persons injured, a cynical person might argue that it was good luck that *two* gross human errors were introduced in this case. Had only one of them been at hand it might be that the collapse had occurred on the occasion when the commercial center building was opening with 50 000 persons occupying the complete building.

2.1.2 Roof trusses in sports arena

The collapse during construction of the steel structure shown in Figures 5 and 6 is representative of a class of failures where human error has been made in failing to provide conditions for full stability of the structure during the complete construction phase. The building in this case is a track and field sports arena with roof trusses spanning 50 m.

The subcontractor for the steel structure had erected the steel structure including temporary bracing elements of the upper chord of the roof trusses as required to provide lateral support for the trusses in the construction phase until final lateral support is provided by the roof deck.

However, the next subcontractor, engaged for the erection of the roof structure consisting of laminated wood purlins and a plate roof, had problems due to the temporary bracing elements hindering movements of the crane used to lift the roof material. Negligent of the important purpose of the temporary bracing elements, the subcontractor loosened them one by one, until the two trusses now without intermediate lateral supports collapsed due to lateral-torsional buckling. The action to remove temporary bracing without reflection should be considered a gross

Figure 5. Roof trusses collapsed during construction when roof structure is being erected

Figure 6. Joint in collapsed roof trusses from Figure 5, with some fractured welds

human error. Such behavior could be eliminated by proper instructions and supervision.

In some joints of the trusses the welds were fractured after the collapse, see Figure 6. It is in cases like this easy to jump to the conclusion that defective welds are the cause of the collapse. In fact, in the case of Figures 5 and 6 an experienced weld inspector had made up an initial report that defective welds in the joints of the trusses were the probable cause of the collapse. However, it is easy to verify that these 50 m trusses when no side support is provided will fail by lateral-torsional buckling when subjected to their own weight only. The fractured welds are a consequence of the trusses being subjected in the fall to unforseen weak axis bending, possibly also by dynamic effects when hitting the ground. The normal collapse mechanism in this class of collapses is instability in some form, such as column buckling, lateral-torsional buckling or local buckling.

2.2 Static loading, service phase

2.2.1 Conveyor belt gallery

The most serious of the collapse cases investigated by the writer in terms of the consequences is that of a conveyor belt gallery, Figure 7. In this case two persons were killed and great economical losses were made due to a long interruption in the production. Part of the steel structure fell down some 30 m to the ground, with instability effects to some members and fractures in others, Figure 8.

The main reason for the collapse was corrosion damage internally in a bolted joint carrying a steel cross-beam in which the conveyor belt gallery was hanging at its upper end in a process building, see Figure 9. Only a small portion of the cross-sectional area of the bolts were remaining at the time of the collapse. This collapse could be attributed mainly to the gross human error in neglecting to implement a rational inspection program for regular checks of the status of this structure in a very corrosive environment. The inappropriate design with the cross-beam hanging from supports instead of resting on the supports may also be considered a human error.

Figure 7.   Collapsed conveyor belt gallery

Figure 8.   Remains from collapsed conveyor belt gallery seen from a 30 m level in process building

Figure 9.   Detail of cross beam carrying the conveyor belt gallery in the process building, with bolts loaded in tension and damaged by corrosion

2.2.2 Floor deck in symphony hall

Another example of a failure of a steel structure during service concerns a floor deck in the hall of a building designed for performances and rehearsals of a symphony orchestra, Figure 10. At the time of the collapse the premises were used for a rock concert with 700 youths jumping on the floor in pace with the rock music, instead of the limited number of 250 persons sitting listening to a symphony as assumed in the structural design. When a 4 x 2 m floor section collapsed due to load action on the floor being 2 to 3 times greater than foreseen for the intended service, about 50 youths fell to the floor below, of which 29 had to be treated at the hospital for broken limbs and other injuries.

The special floor deck in the hall consists of wooden slabs resting on steel trusses, which in turn may be placed at various heights on steel columns below. This is to make possible various arrangements on the floor, such as preparing an orchestra pit. In the collapse the overloaded trusses fractured in some welds and some members in the trusses collapsed due to instability, Figure 11. Again, the main cause of the collapse is a gross human error, in this case by accepting the premises to be used for activities causing much heavier loads than specified for the design. Too small welds in the trusses may also have contributed to the failure.

Figure 10. Collapsed floor deck in concert hall

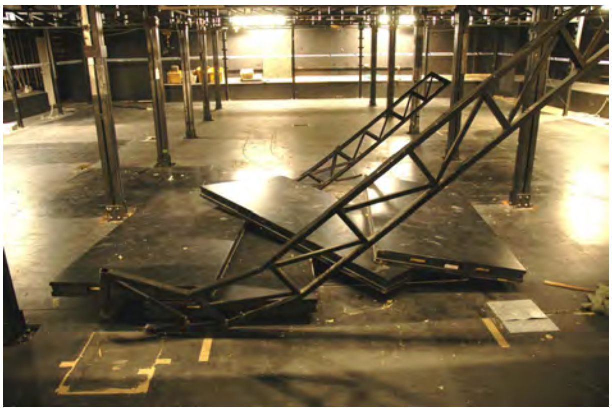

Figure 11. Collapsed steel truss beams in the floor deck of Figure 10

2.2.3 Roof over walkway

A further failure during service which is of particular interest with respect to the normal behavior of welds is shown in Figure 12. A number of cantilevered IPE beams carry a roof over a walkway along the facade of a building. The beams are attached by welds to steel plates anchored in the concrete wall of the building.

The walls of the building were prepared with insulation and bricks, and openings were left for attaching the cantilever beams as indicated in Figure 12. A welder was then asked to perform the weld attachment of the cantilever beams inside these openings. The welder made a gross human error in that he did not refuse to carry out the welding because of too bad accessability, especially for the important weld to the upper flange of the beams subjected to tensile forces from vertical loads on the roof. Instead he tried to

Figure 12. Detail of fractured welded attachment of cantilever beams for a roof over walkway

make the best possible workmanship under the present circumstances. This resulted in a weld at the upper flange of the beams to be completely filled with slag inclusions.

The roof over the walkway performed without problems until the following winter. Then piles of snow from the higher roof of the building fell down on the lower roof over the walkway causing the complete roof with several cantilever beams to collapse. It could be argued whether the structural designer in this case had also made a gross human error since the actual snow load was much greater than he had anticipated. The roof located at a lower level than the roof of the building was designed for normal snow load increased by 20 percent ("snow pocket"). While this assumption may be much on the unsafe side for the actual conditions with today's knowledge and rules for loads in snow pockets, this was according to common practice at the time.

Of particular interest in the behavior of the breakdown shown in Figure 12 is that the fracture started as expected in the weld filled with slag inclusions at the upper flanges with tensile forces. However, when the crack propagated into the web of the beams the fracture chose to enter the base material in the beams away from the weld. This was the case even though the weld along the web was far from perfect (it has not even been de-slagged as may be observed in Figure 12).

2.3 Fatigue damage

Fatigue damage may occur in steel structures, but is not the most common cause for failures in civil engineering construction. This is related to the fact that most civil engineering structures are not heavily affected by varying loads causing fatigue, except those loaded by moving vehicles such as trains, trucks and cranes, and dynamic wind loading. Of the total number of structural failures, incidents and structural damage cases investigated, roughly 10 percent relate to fatigue damage. In many of these cases the causes are gross human errors, for instance, the steel structure has been designed without considering fatigue action at all, or the structures are not properly detailed for the actual fatigue conditions.

From a safety standpoint it is relevant that any fatigue cracks developing in a steel structure may often be observed and repaired before the crack length will become critical and cause a complete collapse or breakdown of the structure.

Many of the fatigue cases investigated are with tall steel chimneys, where load action from vortex shedding may provide the variation in stress (stress range) necessary to cause a fatigue crack to initiate and propagate. Today design methods exist to model the fatigue action from oscillations due to vortex shedding and a detailed fatigue analysis considering the quality of the weld toes, but this has not always been the case.

2.3.1 Steel chimney A

Figure 13 shows an example of a 90 m high steel chimney where fatigue cracks were observed after 12 years of service. The cracks were made visible by magnetic-particle testing, Figure 14. Figures 15 and 16 show an example of a detected crack at the edge between weld passes at the top of the vertical stiffeners, and after grinding down to about half the thickness of the shell plate, respectively. The incident of fatigue cracks in this chimney was to a great extent caused by a gross human error behind the regulations at that time defining load action related to vortex shedding.

Figure 13. 90 steel chimney where fatigue cracks due to vortex shedding were detected at the base

Figure 14. Magnetic-particle testing of welds at the base of the chimney in Figure 13. The lower part of the chimney made of weather-resistant steel has been blast-cleaned before testing

Figure 15. Fatigue crack detected by magnetic-particle testing at the toe between two weld passes around a vertical stiffener at the base of the chimney in Figures 13 and 14

Figure 16. Fatigue crack indicated by magnetic-particle testing after grinding the detail in Figure 15 down to half the thickness of the shell plate

A number of fatigue cracks in the shell and in anchor bolts could be detected in this chimney. As corrective actions the base of the chimney was repaired by adding an outside skirt with new anchor bolts to unload the cracked parts, and in addition the chimney was fitted with helical strakes (Scruton strakes) at the top of the chimney. This chimney has now been in service for many years without further fatigue problems.

2.3.2 Steel chimney B

A detail from the base of another 90 m high steel chimney where several fatigue cracks were observed a few months after installation of the chimney is shown in Figure 17 [2]. Most of the cracks were located in the weld toe at the top of the vertical stiffeners, and in some places the cracks were visible even to the naked eye, Figure 18. The position of the cracks at the *lower* weld toe as evident in Figure 18 is due to the unsuitable design of the stiffeners, causing an even greater stress concentration effect on this side of the weld as compared to the upper weld toe to the shell.

In this case the cause for the undue fatigue damage after a few months of service was a combination of two gross human errors. A mass damper at the top of the chimney was incorrectly installed, which did not reduce large-amplitude oscillations at vortex shedding as expected, and critical details were badly designed, causing unnecessarily grave stress concentrations at the weld toes.

Figure 17. Base of 90 m high chimney with malfunctioning mass damper at the top

Figure 18. Example of fatigue crack at a stiffener at base of chimney shown in Figure 17, in this case visible in the cracked paint

The structure was rectified by weld repair of the cracks, correcting the malfunctioning mass damper and finally also providing a supplementary damping device [3].

2.3.3 Steel frame supporting hydraulic press

A final example of fatigue damage occurred in a steel structure is shown in Figure 19. A frame

Figure 19. Fatigue cracks in two parallel hot-rolled beams HEB1000, initiated at the weld toe of the fillet weld to the cover plate at the lower flange and then propagating up to half the height of the web of the beams

consisting of four rolled beams HEB 1000 carries a concrete floor with an 800-ton hydraulic press.

The beams are connected two and two via cover plates welded to the bottom flange of the beams. Cracks have initiated from the weld toe at the cover plate and propagated into the lower flange of the two beams in Figure 19 and then further into the web, in one of the beams up to half the web height.

Apparently the initial design of this steel structure had not considered any fatigue effects at all. A check analysis of the structure for the actual fatigue loading conditions revealed that the safe fatigue life of the structure had indeed been surpassed. The only surprising thing about this incident is that the steel frame had not collapsed completely. The fact that the structure was still in place with near critical crack lengths is probably due to arch action in the slab.

3 Conclusions

In this paper a few examples of failures with different types of steel structures have been reviewed.

A general conclusion is that the cause of failures most often is a gross human error, in a few cases a combination of two gross human errors. Thus, gross human error is the main cause for collapses of steel structures, rather than a combination of unfortunate variations in parameters affecting the actions and response of the structures, as may be considered in the probabilistic and semi-probabilistic design of structures [4, 5].

Human errors in execution of steel structures are more prevalent as cause for the failures than errors in the design process. Human errors appear particularly serious in causing collapses from structural instability during the construction phase.

For normal steel structures with (quasi-)static (non-fatigue) loading, various structural instability phenomena have been observed to be the main collapse mode.

The general experience with failures due to fatigue in modern welded steel structures is that fatigue cracks almost invariably initiate at the toes of a weld, that is, at the transition between weld and base material (Figure 18), or at any edges between individual weld passes (Figure 15). Thus, the important criteria for such structures are the quality of the weld toes. Visual inspection, if necessary supplemented by magnetic-particle testing, is the most efficient way to ensure adequate fatigue properties in the production of such structures. This is true also for inspecting existing steel structures with fatigue loading.

Based on experience from failures, of which only a fraction and then mostly with major structures are presented in the literature, several changes would be required in regulations and rules for the design and execution of steel structures. One important matter regards the evaluation of welds. The current emphasis on internal discontinuities in welds should be pared with a more strict evaluation of external discontinuities, in particular, for welded structures with fatigue loading.

Since most failures in steel structures are caused by gross human errors, in particular, in the construction phase, it seems imperative to extend in regulations and rules the requirements for competence of the individual performing checking of design and inspecting execution of steel structures. In the current European standard for execution of steel structures, EN 1090-2 [6], there is an exemplary requirement for 100 percent visual inspection of welds. However, there are no detailed and relevant competence requirements for the individual performing such tasks. Also, since other deficiencies than weld discontinuities are of great concern with respect to the safety of steel structures, then it seems important to widen the competence of the inspectors to cover not only the welds of the structure but the complete technology of steel structures. An attempt in this direction has been made by the Swedish industry for steel construction with a program for training and competence evaluation of supervisors and inspectors engaged in steel structures [7].

It is important that lessons be learned from structural failures in a more coordinated and guided way than hitherto, in order to make effective use of manpower and other resources in the design and execution of structures, and to improve the safety of structures by avoiding similar events to happen in the future. In this respect much can be learned from the aviation industry, "black-box thinking" [8]. Instead of trying to hide the information on structural failures we should consider every collapse or incident a learning opportunity for improving the design and execution of structures.

4 References

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