Structural Failures
A Case Study

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Structure:
Hartford Coliseum

Location:
Hartford, CT

Size:
108,000 ft^2

Structural Engineer:
Fraioli-Blum-Yesselman

Architect:
Vincent Kling

Completion Date:
1974

Date of Failure:
January 18, 1978

Failure Investigated By:
Smith and Epstein
Lev Zetlin Associates, Inc.
Loomis and Loomis, Inc.

Causes of Failure:
Buckling of Truss Members
Joint Eccentricity

This roof was noted for being one of the first large-span roofs made possible by computer design and analysis, and was modeled as a space truss using a trusted program. The roof of this three-year-old structure collapsed at 4:15 AM on January 18, 1978 during a freezing rainstorm after a period of snow. Fortunately, there were no injuries sustained as a result of the collapse. The night before, there were over 5,000 people in the coliseum attending an event. Following several investigations of the collapse, it was determined that this was an instance primarily of inadequate structural design.

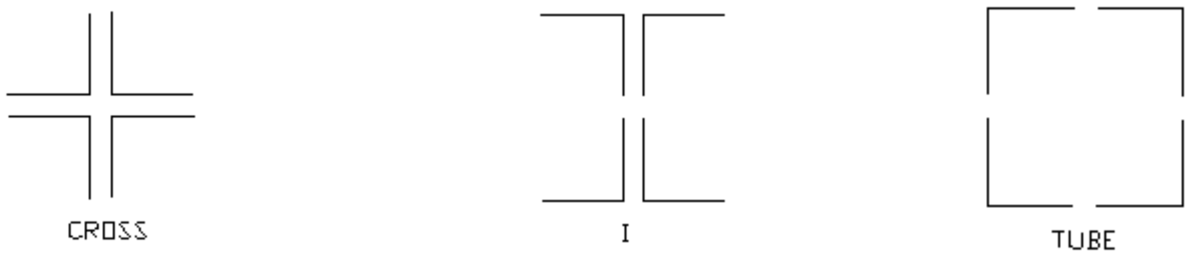
A triangular lattice steel space grid, supported on four reinforced concrete pylons giving spans of 270 feet and 210 feet, was used to support the roof. Smith and Epstein concluded that the interaction of top chord compression members and their bracing played an important role in the redistribution of load and the eventual collapse. They noted that certain compression members were braced against buckling only in one plane. As loads increased, these members buckled out of plane and redistributed the loads to other members. Over a period of time, more chords buckled and fewer and fewer members carried the load. This situation worsened until the remaining members were unable to withstand the added stress due to the loads present that night, and the final, sudden collapse took place.

<u>Background</u>

In order to reduce the cost of the roof, Fraioli-Blum-Yesselman proposed an innovative design for the 300 by 360 ft space frame roof over the arena. The proposed roof consisted of two main layers arranged in 30 by 30 ft grids composed of horizontal steel bars 21 ft apart. 30 ft diagonal bars connected the nodes of the upper and lower layers, and in turn, were braced by a middle layer of horizontal bars. The 30 ft bars in the top layer were also braced at their midpoint by intermediate diagonal bars.

The design of the coliseum roof differed from standard space frame roof designs in four ways:

1. The configuration of the four steel angles did not provide good resistance to buckling. The cross-shaped built up section which was used has a much smaller radius of gyration than either an I-section or a tube section.



2. The top horizontal bars intersected at a different point than the diagonal bars (rather than at the same point), making the roof especially susceptible to buckling.
3. The top layer of the roof did not support the roofing panels; the short posts on the nodes of the top layer did. Not only were these posts meant to eliminate bending stresses on the top layer bars, but their varied heights also allowed for positive drainage.
4. The space frame was not cambered. Computer analysis predicted a downward deflection of 13-in at the midpoint of the roof and an upward deflection of 6-in at the corners.

Lev Zetlin Associates (LZA) discovered that the roof began failing as soon as it was completed due to design deficiencies. A photograph taken during construction showed obvious bowing in two of the members in the top layer. The four major design errors above allowed the weight of the accumulated snow to collapse the roof (*ENR*, April 6, 1978). The load on the day of collapse was 66-73 psf, while the arena should have had a design capacity of at least 140 psf (*ENR*, June 22, 1978). These deficiencies caused the following undesirable results:

- The top layer's exterior compression members on the east and the west faces were overloaded by 852%.
- The top layer's exterior compression members on the north and the south faces were overloaded by 213%.
- The top layer's interior compression members in the east-west direction were overloaded by 72%.

The most overstressed members in the top layer buckled under the added weight of the snow, causing the other members to buckle. This changed the forces acting on the lower layer from tension to compression causing them to buckle also. Two major folds formed initiating the collapse (*ENR*, April 6, 1978).

The excessive deflections apparent during construction were brought to the engineer's attention multiple times. The engineer, confident in his design and the computer analysis which confirmed it, ignored these warnings and did not take the time to recheck its work. A conscientious engineer would pay close attention to unexpected deformations and investigate their causes. They often indicate structural deficiencies and should be investigated and corrected immediately. Unexpected deformations provide a clear signal that the structural behavior is different from that anticipated by the designer.

The joint of the truss members was modeled in the computer as having no eccentricity, an incorrect assumption. As a result of this inaccuracy, a bending moment was actually developed in the built structure, putting additional stresses in the member. A nonlinear collapse simulation was rerun using the correct model for the joint, and with loading conditions selected to approximate those of the night of failure. The result was that the simulated connection failed as it had under the real conditions.

Loomis and Loomis, Inc. (LLI) agreed with LZA that gross design errors were responsible for the progressive collapse of the roof, beginning the day that it was completed. They, however, believed that the torsional buckling of the compression members, rather than the lateral buckling of top chords, instigated the collapse. Using computer analysis, LLI found that the top truss members and the compression diagonals near the four support pylons were approaching their torsional buckling capacity the day before the collapse. An estimated 12 to 15 psf of live load would cause the roof to fail. The snow from the night before the collapse comprised a live load of 14 to 19 psf. Because torsional buckling is so uncommon, it is often an overlooked mode of failure (*ENR*, June 14, 1979).

Conclusion

The engineers for the Hartford Arena depended on computer analysis to assess the safety of their design. However, computer programs have a tendency to provide engineers with a false sense of security. The roof design was extremely susceptible to buckling which was a mode of failure not considered in that particular computer analysis and, therefore, left undiscovered (Shepherd and Frost, 1995). A more conventional roof design would have been much stronger. Instead of the cruciform shape of the diagonal members, a tube or I-member configuration would have been much more stable and less vulnerable to bending and twisting. Also, if the horizontal and diagonal members intersected at the same place it would have reduced the bending stresses in these members. Finally, the failure of a few members would not have triggered such a catastrophic collapse if the structure had been designed and built with more redundancy (Levy and Salvadori, 1992).

If the state of Connecticut had enacted a structural engineering practice act requiring engineers who design large and significant buildings such as the Hartford Coliseum to be licensed as a *Structural Engineer (SE)*, it is likely that the Coliseum engineers would have recognized the impact the configuration of the truss members had on the integrity of the structure. Selecting the weaker of the configurations without accounting for the resulting increase in buckling caused the failure to occur. Licensing of structural engineers in the state of Connecticut could have prevented this catastrophic failure.

Finally, the Hartford Department of Licenses and Inspection did not require the design of the project to be peer-reviewed, which it usually did for projects of this magnitude. If a second opinion had been required the design deficiencies responsible for the arena's collapse may have been discovered.