6 Hours from Catastrophe – The Hartford Civic Center Collapse

An Online Continuing Education Course for Engineers

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The Hartford Civic Center Collapse

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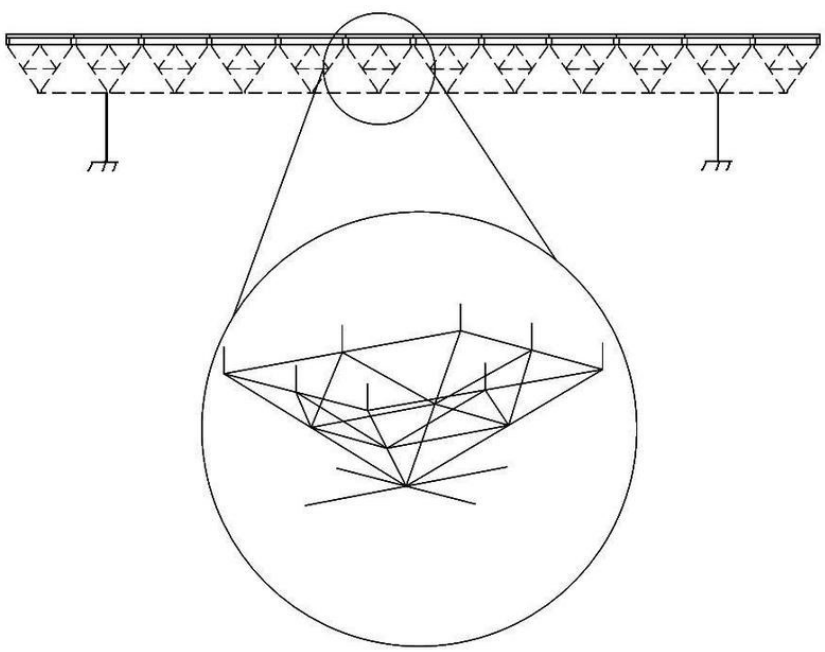
On January 18, 1978 at 4:19 AM, in what could have been one of the worst structural failure disasters in history, the roof of the three-year-old Hartford Civic Center Arena suddenly collapsed. Six hours earlier, the arena had been filled with 4,746 basketball fans watching a University of Connecticut versus University of Massachusetts basketball game. So sudden and complete was the failure that the 2 ½ acre, 2,900-ton roof fell the 83 feet to the arena floor and seating areas in only a few seconds from the time security guards in the retail area heard a loud crack. The mass of steel, lighting, and gypsum cement falling onto such a closely assembled crowd would likely have caused hundreds or thousands of fatalities had the incident occurred only a few hours earlier that evening. At the time of the collapse the building was unoccupied, and the failure caused no injuries.

Below, Figure 1: Aerial view of the Hartford Civic Center roof on January 18, 1978.

Construction

Construction of the Hartford Civic Center began in 1972. It was one of four major urban renewal projects that had begun in the 1960's including Constitution Plaza, Windsor Street and Bushnell Plaza. The renewal promised to bring new vitality to the city with restaurants, retail shops, a hotel and the new Civic Center to be the new home of the World Hockey Association team the New England Whalers. The roof of the Civic Center arena embodied this optimism. Designed with the aid of a new and complex computer program, the unique structure consisted of unusual pyramidal trusses and was supported by just four columns in order to provide an unobstructed view for every spectator. The space truss roof was assembled on the ground and lifted into place in 1973, another innovative and cost saving technique. Overall the cutting-edge design saved the city half a million dollars. Considered a major Engineering feat at the time, it was called the largest single span of roof ever lifted into place.

In 1970, Vincent Kling agreed to be the architect for the new Civic Center and shortly thereafter Kling hired Fraoli, Blum and Yesselman, Engineers (F B Y) to design the arena. In order to save money, F B Y proposed an innovative design for the 300-by-360-foot space frame roof over the arena. They convinced the city of Hartford that more money could be saved if the design were optimized to eliminate unnecessary redundancy by using a newly available two-dimensional structural analysis software package.



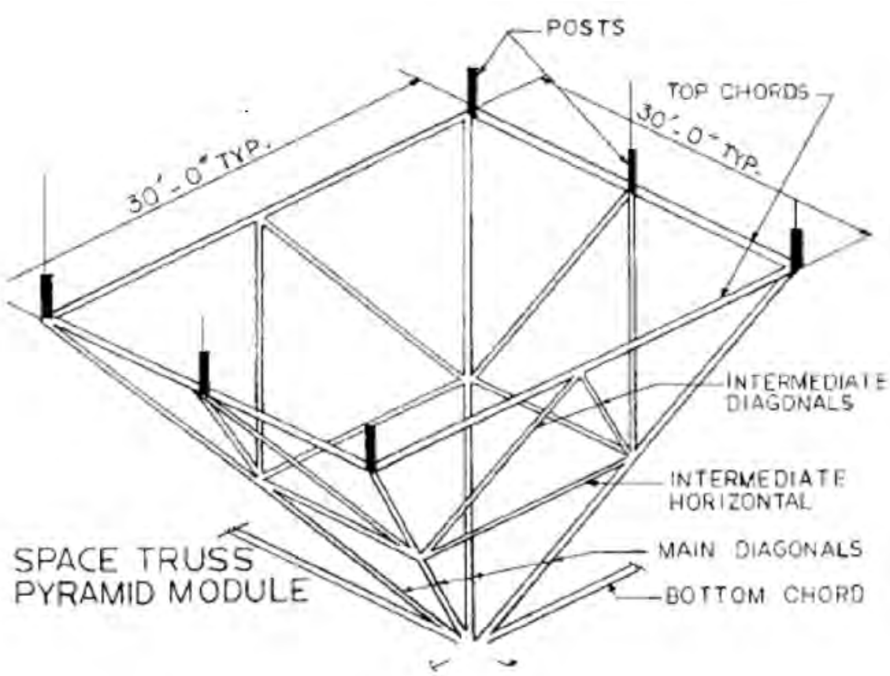
Above, Figure 2: General design of space frame truss system used in the Hartford Civic Center arena roof with pyramidal module enlarged. Two of the four pylon legs supporting the roof are indicated 45 feet from each end.

Investigations into the Cause of the Collapse

Immediately following the collapse, the city of Hartford appointed a three-member panel to manage the investigation. This panel hired Lev Zetlin Associates, Inc. (LZA) to ascertain the cause of the collapse and to propose a demolition procedure. LZA president, Dr. Charles Thornton produced an initial 120-page report (a) and a final 400-page report (b) of his findings and conclusions. The firm of Loomis and Loomis, Inc. (LLI) was also retained by Thornton to perform a second independent investigation. A structural consultant, Hannskarl Bandel, completed an independent investigation of the collapse for the Architect's insurance company. In the years to come, many other reports and articles would be written and published in such sources as the Engineering News-Record (ENR) and by the ASCE.

The roof proposed by F B Y consisted of two main layers arranged in 30-by-30-foot grids composed of horizontal steel bars 21 feet apart. 30-foot diagonal bars connected the nodes of the upper and lower layers and these in turn were braced by a middle layer of horizontal bars. The 30-foot bars in the top layer were also braced at their midpoint by intermediate diagonal bars which acted through the plane of the pyramid side.

Below, Figure 3: Typical pyramidal truss module layout as built in Civic Center space frame.



All horizontal and diagonal members of the truss modules were of a built up cruciform cross section made by assembling four “L” channels with spacers varying from .75” to .87” thick. This was done to allow a gap at the ends which could slide over a joining plate (purlin). The roof deck and its support structure were not fixed to the upper horizontal chords but instead rested on support posts seen in Figure 3 which varied in length to pitch the roof to provide positive drainage. “Raising the roof deck structure from the truss prevented this upper structure from providing additional buckling resistance to the top chords of the truss modules” (LZA). Upon examination of construction photographs and collapsed wreckage it was determined that some of the roof support posts were omitted on about half of the truss modules used in the building which further concentrated roof loads at the nodes (corners) of the other modules in these areas according to the LZA report.

The truss system was designed and tested by F B Y using their new structural analysis software to have an ultimate combined capacity of 140 pounds per square foot and LZA determined that the forces calculated by the software were correct, but the model analyzed by the computer had been greatly over simplified. The top chords of the truss modules would be in compression and the configuration of the four steel angles did not provide good resistance to buckling. The cruciform shaped built up section of the bars making up the truss has a much smaller radius of gyration than either an I-section or a tube section made from the same L channel material making the bars much more susceptible to buckling in compression. When the LZA investigation reviewed the computer-generated results of the truss testing it was determined that failure in buckling had not been taken into consideration.

In addition to the flawed choice of a cruciform cross section for the truss bars, the LZA investigation discovered that no horizontal mid-point braces were provided for the members in the top layer, see Figure 3. The exterior top chord members were only braced every 30-feet rather than the 15-foot intervals that had been modeled and specified by F B Y and the interior members were only partially and insufficiently braced at their midpoints. The two members attached to the midpoint of the top chord were both in the same plane as the long axis of the chord meaning that they could only provide bracing in one direction. The perpendicular direction was effectively un-braced for the full 30-foot length, see Figure 3.

This omission significantly reduced the load that the roof could safely carry by reducing the buckling strength of the top chords by 75%. In addition, certain perimeter top chord members with a roof support post located at midpoint were subjected to bending stress from the roof load applied through the post. Since the members were not designed for bending, this condition also led to a considerable overstress (LZA 1978 report).

Structural Engineers from F B Y designed and computer tested the Hartford Civic Center roof truss structure; however, they had under estimated its dead load weight by

20% even when taking into consideration the truss elements that were omitted by the builder. Bethlehem Steel Co. had produced the detail drawings and fabricated the nearly five thousand parts that would make up the roof truss. Inadequate communication and oversight between the Structural Engineers and Bethlehem had resulted in not only the omission of the horizontal midpoint bracing of the upper chords, but also in a misunderstanding regarding the manner in which the truss element connections were modeled and how they were performed in accordance with Bethlehem's detail drawings (Thornton – LZA, 1978).

Some of the effects resulting from the deviation from the as-modeled and computer tested design (a) to the as-detailed and built design (b) are shown in Figure 4 below. The diagonal members were attached some distance below the horizontal members, thus the flexibility of the connection reduced the effectiveness of the bracing by introducing a spring brace instead of the hard brace that had been assumed in the modeling. These modifications also introduced a bending moment into the upper chords which helped initiate lateral or torsion buckling (LZA 1978) (Loomis and Loomis 1979).

Below, Figure 4: Diagram from 1978 LZA investigation final report showing effects of the omission of midpoint horizontal bracing on the main top chords ability to resist buckling under the compressive forces applied to them. Also shown are the off-axis connections (b) substituted by the steel fabricator.