

INDIAN METAL BUILDING FORUM

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Connecting India's Metal Building Ecosystem

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RESOURCE CENTRE

Technical Knowledge Series

STEEL STRUCTURE FAILURE & LESSONS LEARNED

Steel Space-Frame Roof Collapse: Unbraced Compression Members under Snow Load

Hartford Civic Center Arena, Hartford, Connecticut, USA | January 1978

Learn from Failure
Improve the System
Prevent Recurrence

CASE STUDY NO:	IMBF-SF-005
COUNTRY:	United States of America
YEAR:	1978
STRUCTURE TYPE:	Long-span steel space-frame arena roof on four pylons
TECHNICAL CATEGORY:	Steel space frames / Compression member buckling / Node and bracing detail

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IMBF publishes this educational case study. IMBF was not the designer, builder or investigator of the structure.

A DOCUMENT CONTROL AND STATUS

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Technical review status

- ☒ Research Draft (compiled from referenced public sources; not yet reviewed)
- ☐ Technical Review
- ☐ IMBF Approved for Publication

This draft must not be marked approved until an authorised IMBF reviewer has checked it against the primary investigation report listed in the Source Register.

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The technical explanations and lessons presented are intended for general educational and professional-development purposes. They shall not be used as a substitute for: project-specific structural design; engineering calculations; site investigation; competent structural-engineering review; statutory approval; manufacturer instructions; applicable codes and standards; independent inspection; accredited testing or certification.

Readers must verify project-specific requirements with qualified professionals and the applicable statutory and code requirements. Where facts are uncertain or disputed, this document identifies them as such.

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How to read the references. Tags such as [S1] point to the Source Register on the last page. Text marked "IMBF interpretation" is engineering commentary by this series and is not a finding of the original investigators.

1 EXECUTIVE SUMMARY

Question	Answer
What happened?	The steel space-frame roof of a sports arena fell about 25 m (83 ft) to the arena floor under snow. [S1]
Where?	Hartford Civic Center Arena, Hartford, Connecticut, USA.
When?	About 4:15 am on 18 January 1978, as given in the detailed case study in [S1]. The summary on the same page says 17 January; the sources differ.
What structure?	A space frame about 91 m x 110 m (300 ft x 360 ft), two layers 6.4 m (21 ft) apart on a 9.1 m (30 ft) grid, carried on four pylons set 13.7 m (45 ft) inside the roof edges. [S1]
What failed?	Top-layer compression members buckled, the load moved to other members, and the whole frame folded. [S1]
Why?	The top chords were not braced at mid-length as the design assumed, their four-angle cross-shaped section was weak in buckling, and dead load was underestimated by about 20 %. Warnings from excessive deflection during construction were not acted on. [S1]
Consequence?	Total roof collapse. The arena was empty and no one was hurt; about five thousand people had been inside a few hours earlier. [S1]
Main lesson	Buckling strength depends on bracing that exists in the built structure, not in the analysis model. Measured deflection twice the predicted value is a warning that must stop the job.

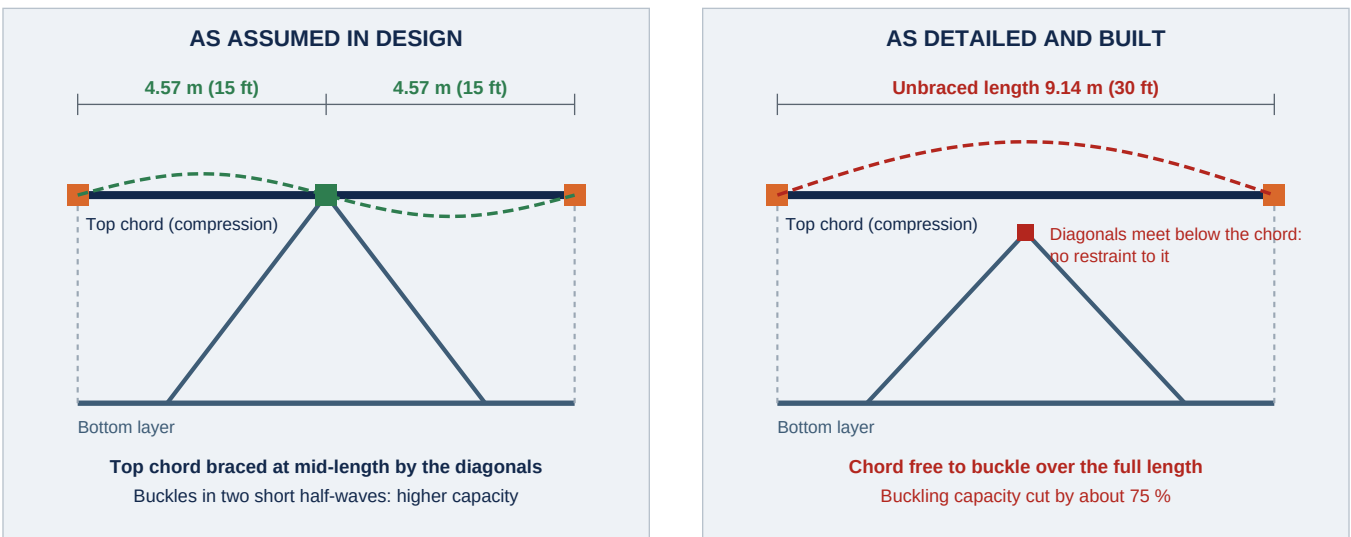


Figure 1. Original IMBF schematic of the top-chord bracing as assumed and as built. Conceptual, not to scale; buckled shapes exaggerated and shown in one plane for clarity.

2 PROJECT BACKGROUND

Only data found in the consulted source is listed. Missing data is stated as not established rather than estimated.

Item	Verified data	Source
Purpose	Indoor arena for sport and public events.	[S1]
Roof system	Steel space frame: two main layers of horizontal members on 9.1 m x 9.1 m (30 ft) grids, 6.4 m (21 ft) apart, joined by 9.1 m diagonals, with a middle layer of horizontal bracing members.	[S1]
Plan size	About 91 m x 110 m (300 ft x 360 ft).	[S1]
Supports	Four pylon legs, 13.7 m (45 ft) inside the roof edges, instead of boundary columns or walls.	[S1]
Member section	Built-up cross shape of four steel angles, with a smaller radius of gyration than a tube or I-section.	[S1]
Node arrangement	Top horizontal members met at a different point from the diagonals, not at one common node.	[S1]
Roof deck support	Roofing panels sat on short posts at the top-layer nodes, with varied heights for drainage, not on the top chords.	[S1]
Camber	None. Analysis predicted 330 mm (13 in) downward deflection at mid-span and 150 mm (6 in) upward at the corners.	[S1]
Erection method	Frame fully assembled on the ground and lifted into place by hydraulic jacks on the four pylons.	[S1]
Roof completed	16 January 1973.	[S1]
Stage at failure	In service for about five years.	[S1]
Conditions at failure	Largest snowstorm in the life of the building. Load on the roof on the day estimated at 3.2 to 3.5 kPa (66 to 73 psf) in total.	[S1]
Steel grades, bolt sizes, tonnage	Not established in the available source.	-

3 FAILURE TIMELINE



4 WHAT FAILED?

Element	Identification
Primary failed component	Top-layer compression members (top chords) of the space frame, worst at the exterior lines. [S1]
Secondary failed components	Remaining top members, then bottom-layer members that reversed into compression. [S1]
Connection involved	Top-layer node: diagonals attached below the horizontal members, so they could not brace them. [S1]
Load path involved	Roof panels -> posts -> top nodes -> space-frame members -> four pylons -> foundation.
Failure mode	Buckling of compression members (lateral buckling per the city's consultant; torsional buckling per a second study), then progressive collapse. [S1]

5 ENGINEERING FAILURE MECHANISM

Buckling and unbraced length. The load a slender compression member can carry falls with the square of its unbraced length. Doubling the length from 4.57 m to 9.14 m cuts the buckling capacity by about 75 %. The design assumed the shorter length; the built detail gave the longer one. [S1]

Weak section. Four angles arranged as a cross put the steel close to the centre. That gives a low radius of gyration and little resistance to twisting, so the member buckles early compared with a tube or I-section of the same area. [S1]

Eccentric nodes. Because horizontals and diagonals did not meet at one point, members also carried bending, and the diagonals gave no restraint to the chords. [S1]

Underestimated load. The frame weighed about 20 % more than assumed. On the day, total load was 3.2 to 3.5 kPa against a capacity that should have been at least 6.7 kPa (140 psf). [S1]

No redundancy. Once the worst members buckled, the frame could not shed their load safely and the failure spread across the roof. [S1]

Reported overload of top-layer compression members (city's consultant)	Overload
Exterior members, east and west faces	852 %
Exterior members, north and south faces	213 %
Interior members, east-west direction	72 %

Differing expert views, all from [S1]: the city's consultant found lateral buckling of top chords; Loomis and Loomis found torsional buckling near the pylons, with failure at 0.57 to 0.72 kPa of live load against 0.67 to 0.91 kPa of snow that night; a consultant for the architect's insurer blamed a weld at the scoreboard hanger, a view the other investigators did not share.

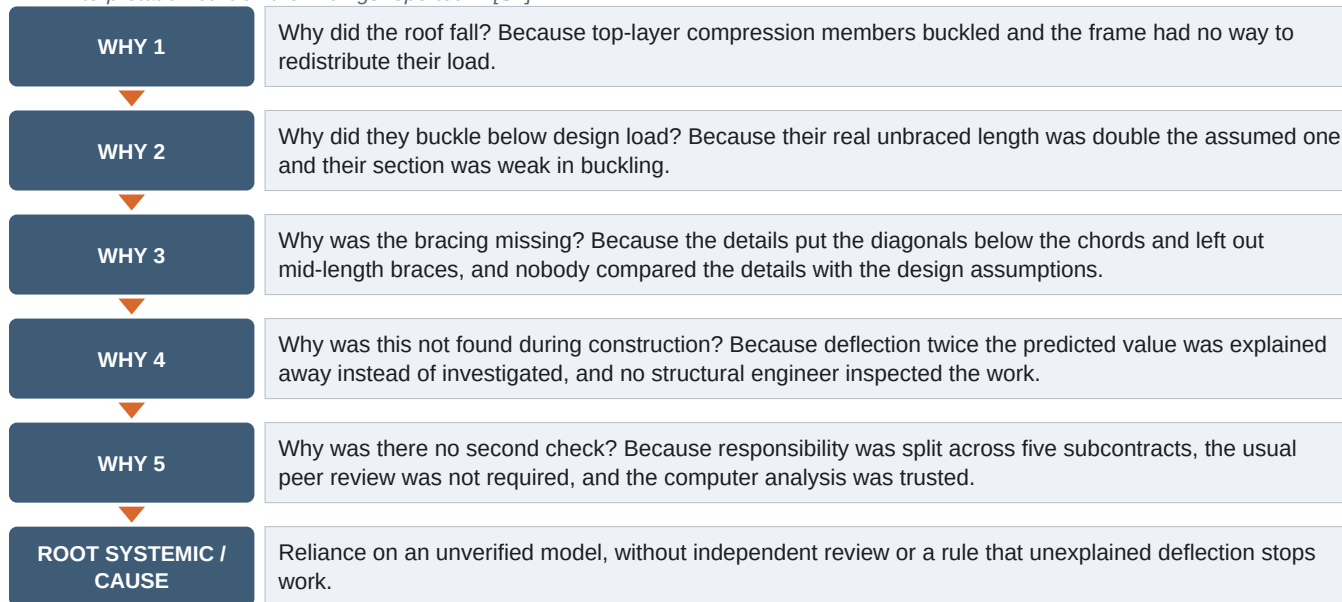
6 ROOT-CAUSE ANALYSIS

Area	Finding	Basis
Design	Compression members grossly overloaded; buckling mode not covered by the computer analysis; cross-shaped section; eccentric nodes; dead load underestimated; slenderness beyond code limits.	Reported [S1]
Material	Some steel did not meet specification (contributing, not sufficient alone).	Reported [S1]
Fabrication	Details omitted mid-length braces and attached diagonals below the chords; spacer plates too far apart; some diagonals misplaced.	Reported [S1]
Welding	One consultant blamed a scoreboard weld; other investigators disagreed.	Disputed [S1]
Bolting	Members with bolt holes beyond the code limit on net area are reported.	Reported [S1]
Erection	Excessive deflection seen on the ground and after lifting; reported and not acted on.	Reported [S1]
QA/QC	No structural engineer inspected the work; inspection agency findings were not resolved.	Reported [S1]
Operation	Snow load within what the roof should have carried.	Reported [S1]
Maintenance	A citizen's report of visible sag was answered with reassurance.	Reported [S1]
Management / communication	Five subcontracts with no single party responsible for the whole; no peer review for a novel long-span roof.	Reported [S1]

Level	Statement
Immediate cause	Buckling of overstressed top-layer compression members under snow load. [S1]
Contributing causes	Unbraced length twice the design value; weak cross-shaped section; eccentric nodes; 20 % extra dead load; material and fabrication deviations. [S1]
Root cause	An innovative design verified only by a computer model that did not include the governing buckling behaviour, with no independent review and no response to measured deflections that contradicted the model. (IMBF interpretation of [S1])

7 FIVE-WHY ANALYSIS

IMBF interpretation built on the findings reported in [S1].



8 CORRECTIVE ACTION

Immediate corrective action (as reported)

- The city appointed a three-member panel, which engaged a specialist firm to find the cause and plan demolition of the wreckage. [S1]
- How the roof was rebuilt: Not established in the available source.

Permanent corrective action (engineering remedy, IMBF interpretation)

- Use closed or I-shaped sections for compression members, with slenderness inside code limits.
- Make members meet at common nodes, and brace compression chords where the design says they are braced.
- Check member and system buckling explicitly, including torsional modes, and include real self-weight.
- Provide redundancy so that loss of a few members does not fold the frame.
- Compare measured deflection with prediction at each stage and resolve any large difference before proceeding.

9 PREVENTIVE ACTION (CAPA)

Problem	Root cause	Corrective action	Preventive action	Responsible party
Top chords buckled	Unbraced length double the design value	Add bracing at design points	Check that every assumed brace exists on shop drawings and on site	Engineer of record; detailer
Section weak in buckling	Cross-shaped four-angle member	Use tube or I-section	Section selection rule for compression members in space frames	Structural designer
Model missed the failure mode	Analysis did not include buckling	Second-order and buckling analysis	Independent peer review of novel long-span roofs	Owner; reviewing engineer
Deflection warnings ignored	No rule for acting on measurements	Investigate and re-analyse	Deflection survey hold points with stop-work limits	Erector; engineer of record
No one owned the whole	Fragmented contracts	Single point of structural responsibility	Engineer of record retained for site observation	Owner; project manager

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QA/QC FAILURE ANALYSIS: WHAT SHOULD QA/QC HAVE DETECTED?

QA/QC area	Check relevant to this type of failure
Material certificates	Steel grade and properties match specification. Some steel here did not. [S1]
Traceability	Members traced to heat numbers so that non-conforming steel can be located.
WPS / PQR, welder qualification, NDT	Applicable to hanger and node welds; a weld was blamed by one consultant only.
Bolt holes and bolting	Net section at bolt holes within code limits; hole sizes and positions as detailed.
Dimensional inspection	Spacer plate spacing in built-up members; position of every diagonal. Both deviated here. [S1]
Alignment	Members straight within tolerance. A construction photograph showed bowed top members. [S1]
Bracing	Every brace assumed in design present and connected to the member it braces.
Erection sequence	Deflection measured on the ground, after lift and after decking, and compared with prediction.
As-built verification	Actual weight checked against design dead load; as-built node geometry compared with design.
Coating, drainage	Not reported as factors in this case.

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IMBF OPTIMIZATION STRATEGY

Area	Optimisation
Design	List bracing assumptions on the drawings. Run buckling checks by a second method. Use realistic self-weight with allowance for connections.
Fabrication	Detailers confirm that each brace connects at the point the design assumes; node eccentricity limits stated.
QA/QC	Hold point for trial assembly and deflection survey of space frames and long-span trusses.
Erection	Stop-work limit when measured deflection exceeds prediction by an agreed margin.
Digital traceability	Survey results and non-conformance reports linked to members in the model.
Inspection	Engineer of record observes critical stages on site.
Maintenance	Periodic deflection survey of long-span roofs, and after heavy rain, snow or added roof loads.
Cost	Material savings from slender members are small against the cost of a lost roof.
Safety	Higher review level and robustness check for assembly buildings.
Productivity	Standard, proven node details avoid site fitting problems such as cutting fascia supports to suit a sagging frame.

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DIGITALIZATION

- Three-dimensional model used for both analysis and detailing, so that node geometry and bracing are the same in each.
- QR code on every space-frame member and node, linked to heat number and inspection record.
- Deflection surveys stored against the model with automatic comparison to predicted values.
- Photo-based inspection of nodes and member straightness before lifting.
- Digital as-built model handed to the owner for later monitoring.

DESIGN	Bracing assumptions and predicted deflections recorded in the model.
FABRICATION	Members and nodes marked; material traced; node geometry checked against the model.

INSPECTION	Dimensional, straightness and connection checks with photographs.
DISPATCH	Scan-out against assembly sequence.
ERECTION	Deflection surveyed at each stage and compared with prediction.
FINAL INSPECTION	As-built weight and geometry confirmed; open reports closed.
DIGITAL AS-BUILT / RECORD	Baseline for periodic deflection monitoring by the owner.

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INDIAN CONTEXT: WHAT SHOULD INDIA LEARN FROM THIS FAILURE?

- Space frames, long-span trusses and tubular roofs are widely used for stadia, airports, exhibition halls and industrial sheds. Their strength is governed by compression members and node details.
- In PEB frames the same principle applies to flange braces, purlin and girt restraint and bracing: if a brace assumed in design is omitted or connected to the wrong point, rafter and column capacity drops sharply.
- Software output is not a design check. A reviewer should test bracing assumptions, effective lengths and self-weight by hand or by an independent model.
- Roof loads added later, such as solar panels, services or ponded water, reduce a margin that may already be thin.
- Large public roofs should have independent proof-checking and site observation by the design engineer.

Reference	Subject	Relevance to this case
IS 800	General construction in steel: code of practice	Compression member design, effective length, built-up members, slenderness and net-section limits.
IS 875 (Part 1 and Part 2)	Dead and imposed loads	Realistic self-weight and roof loads.
IS 875 (Part 4)	Snow loads	Relevant for sites in snow regions.
IS 875 (Part 3) and IS 1893	Wind and earthquake loads	Other load cases for long-span roofs.
IS 2062	Hot-rolled structural steel	Material conformity.
Project specification and ITP	Contract documents	Trial assembly, deflection survey hold points and peer-review requirements.

Clause numbers are deliberately not cited. Standard titles and current editions must be confirmed against Bureau of Indian Standards publications at technical review.

14 IMBF PREVENTION CHECKLIST

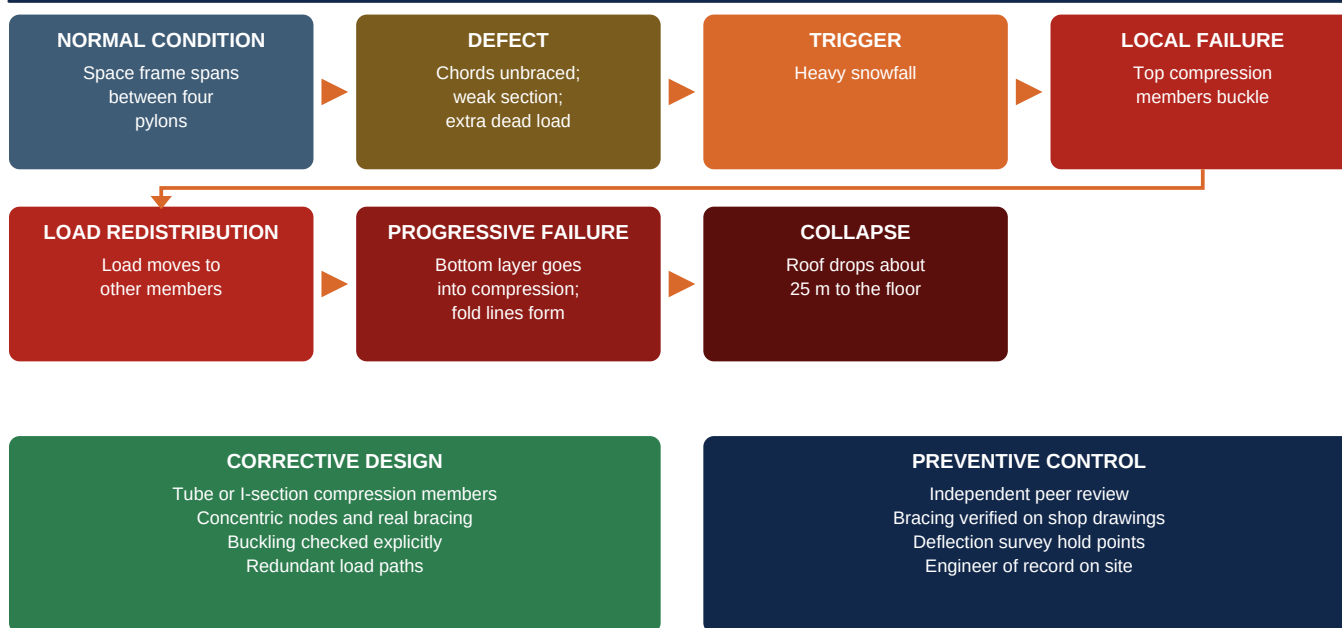
Inspection point	Required check	Status	Remarks
Design review	Buckling of compression members checked independently; self-weight realistic; peer review done	☐	
Connection review	Members meet at common nodes; braces connect to the members they restrain	☐	
Material traceability	Steel conforms to specification; heat numbers traced	☐	
Welding	Qualified WPS and welders for node and hanger welds	☐	
NDT	Critical welds tested to the specified extent	☐	
Bolting	Hole size, position and net section within limits	☐	
Dimensional inspection	Spacer plates, diagonals and node geometry as detailed; members straight	☐	
Bracing	Every design brace installed before load is applied	☐	
Erection sequence	Deflection surveyed at each stage and compared with prediction	☐	
Coating	Protection complete and nodes inspectable	☐	
Drainage	Roof drains clear; no ponding on the deflected shape	☐	
Final inspection	Unexplained deflection or bowing resolved before handover	☐	
As-built documentation	As-built survey and monitoring baseline handed to owner	☐	

15 PHOTOGRAPHS

No third-party photograph is reproduced in this document. Published photographs and report figures exist, but their copyright and publication permission were not established for this series, so they are not used. All figures here are original IMBF schematics.

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None reproduced	Not applicable	Not applicable	Not applicable

16 ORIGINAL IMBF FAILURE DIAGRAM



Original IMBF schematic. Conceptual, not to scale.

17 LESSONS LEARNED

5 engineering lessons

1. Buckling capacity depends on real unbraced length.
2. Choose compression sections for stiffness, not only area.
3. Bring members to a common node; avoid eccentricity.
4. Use actual self-weight, including connections.
5. Design in redundancy for long-span roofs.

5 management lessons

1. Keep one party responsible for the whole structure.
2. Require peer review for novel designs.
3. Retain the designer for site observation.
4. Treat public concerns and site reports as data.
5. Do not let software replace engineering judgement.

5 QA/QC lessons

1. Check that assumed braces exist in the details.
2. Verify steel against specification.
3. Measure deflection and compare with prediction.
4. Inspect built-up members for spacer spacing.
5. Close every inspection finding before the next stage.

5 preventive actions

1. Add a bracing-assumption schedule to drawings.
2. Set stop-work deflection limits.
3. Trial-assemble critical nodes.
4. Monitor long-span roofs in service.
5. Teach this case to detailers and design engineers.

18 IMBF KEY MESSAGE

"The Hartford roof told its builders it was in trouble: it sagged twice as much as predicted, on the ground and in the air. A brace that exists only in the calculation braces nothing. Verify the model against the built structure, and stop when they disagree."

19 SOURCE REGISTER

Ref	Source and organisation	Date	Used for	Consulted?
S1	Building Failure Cases: Hartford Civic Center, with the case study by R. Martin. University of North Carolina at Charlotte with the ASCE TCFE Education Committee. Cites Engineering News-Record (1978-1979), Levy and Salvadori (1992), Feld and Carper (1997), Kaminetzky (1991). https://engineering.charlotte.edu/research/areas/case-studies/asce-failure-cases-studies/building-failure-cases/	Accessed 4 Oct 2026	All technical data, chronology, investigation findings and differing expert views in this document	Yes, in full
S2	Failure Case Studies in Civil Engineering, 2nd ed., "Hartford Civic Center Coliseum (1978)", pp. 88-92. ASCE Technical Council on Forensic Engineering. ISBN 978-0-7844-1255-8.	2013	Confirms the case is documented by ASCE	Contents page only
S3	Engineering News-Record articles: 26 Jan 1978; 6 Apr 1978; 22 Jun 1978; 29 Jun 1978; 14 Jun 1979; 24 Jun 1979.	1978-1979	Cited by S1 for investigation findings	No
S4	Why Buildings Fall Down, M. Levy and M. Salvadori (W. W. Norton, 1992); Construction Failure, 2nd ed., J. Feld and K. Carper (Wiley, 1997).	1992; 1997	Cited by S1	No
S5	Report of the city-appointed investigation by Lev Zetlin Associates.	1978	Primary investigation report; not located online	No

Limits of this draft: it rests on one secondary source [S1]. The primary investigation report [S5] and the articles and books in [S3] and [S4] were not read. [S1] gives the date as 17 January in one place and 18 January in another, and describes the last event as basketball in one place and hockey in another. Imperial values were converted to SI by IMBF and rounded.

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