Oct 19th, 12:00 AM

Lessons From Recent Collapses of Metal Buildings

David B. Peraza

Fifteenth International Specialty Conference on Cold-Formed Steel Structures
St. Louis, Missouri U.S.A., October 19-20, 2000

Lessons from Recent Collapses of Metal Buildings

By David B. Peraza, P.E.
Senior Associate, LZA Technology, New York, NY

This paper presents case studies of three roof collapses of metal buildings, discusses the most signigicant features of each case, and summarizes lessons learned that can be used to reduce future failures.

Freezer Buildings

On March 5, 1994, two attached freezer buildings collapsed under a snow load. The buildings were located near Harrisburg, PA. The buildings were owned by a food distributor and contained racks nearly 30 feet high for frozen food storage. These racks actually prevented the roof from falling to the ground. The claimed loss, primarily for food spoilage, was on the order of $20 million dollars.

The buildings had been built in 1983 and in 1991, and their general arrangement is shown in Figure 1. For both buildings, the Owner had contracted with the same Design/Builder, who in turn subcontracted the design and fabrication to the same Metal Building Engineer/Manufacturer (Engineer).

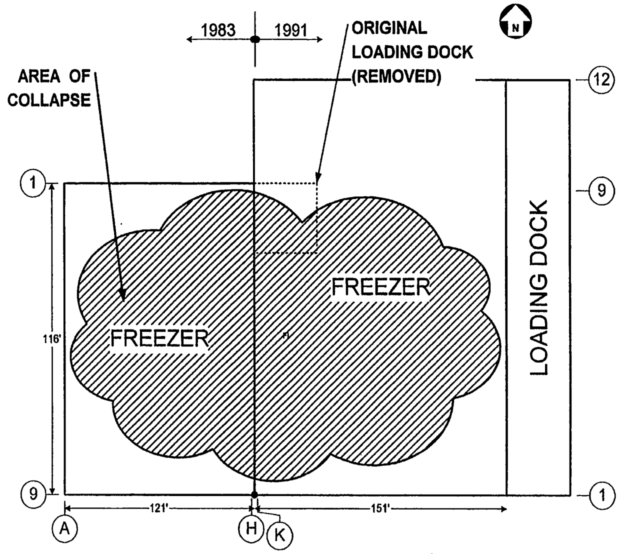


Figure 1: Roof Plan

At the time that the 1983 building was designed, it was anticipated that a freezer addition would be built at a later date along one side of the building. The vertical load from the future building was included in the design of the appropriate columns.

In 1991, the small loading dock was dismantled to make way for the freezer addition. Lean-to brackets were welded to each column where the addition would attach. These brackets, and the moments transferred through the brackets, became a focal point of the investigation.

LZA Technology was retained by the Builder shortly after the collapse. Subsequently, we were retained by attorneys representing the Builder's insurance carrier. The case eventually settled before going to trial.

Description of Structure

The 1983 Building measured approximately 121 feet by 155 feet in plan, and had a single slope roof. Its structure typically consisted of steel frames spaced at 19 feet and 20 feet, with one line of intermediate columns. The frames were composed of built-up plate girders, built-up columns, and intermediate round pipe columns. The secondary roof members consisted of 8-inch deep Z purlins spaced at 5 feet on center. Metal deck, 1 ½" deep, was screwed to the purlins and supported seven inches of insulation and a ballasted roof membrane.

The original loading dock was a "lean-to" structure. The three loading dock rafters attached to the freezer building columns via shim plates, as shown in Figure 3a.

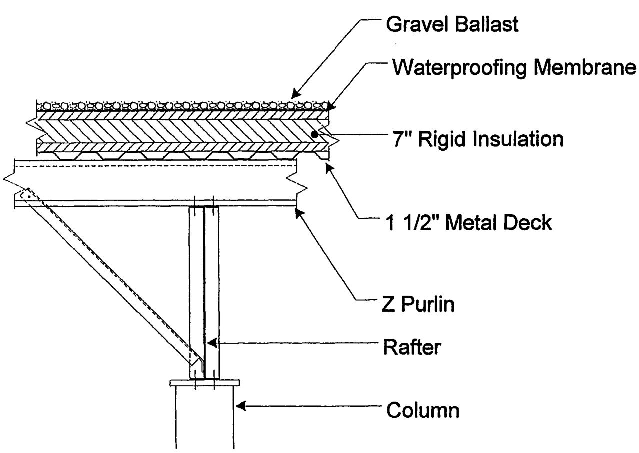
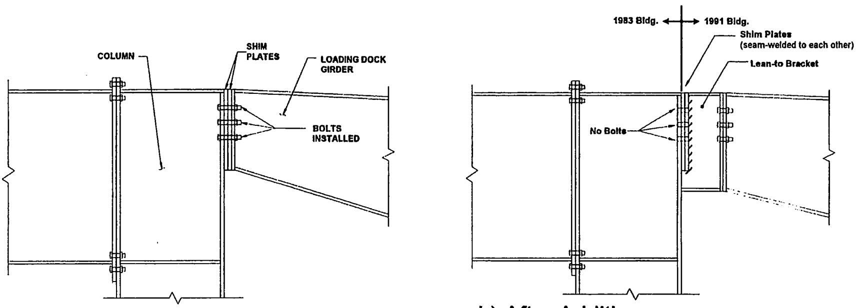


Figure 2: Typical Roof Construction

In 1991, the loading dock was dismantled to make way for the freezer addition. This involved, among other things, unbolting the three rafters from the columns. Prior to erection of the new building, lean-to brackets were welded to each of the eight columns along the high side of the old building. The addition consisted of two buildings: a freezer and a large, independent loading dock. The combined buildings are shown in Figure 1. The new rafters were attached to the old building's columns via welded lean-to brackets. The erection drawings did not contain any special instructions as to how to attach the



a) Original Construction   b) After Addition

Figure 3: Connection Details at Original Loading Dock

lean-to brackets, and they were erected as shown in as in Figure 3b. The construction of the 1991 freezer was similar to that of the 1983 building.

Snow Loads

Measurements of the snow weight on the roof made shortly after the collapse. These indicated an average weight of 45 psf. The measurements, which appear well documented, were made at two locations that appeared to be representative. Sample 1 contained 10 inches of snow over 4 inches of ice, and weighed 45 psf; sample 2 contained 13 ½ inches of snow over 2 inches of ice, and weighed 44 psf.

Data from the Harrisburg airport weather station indicates that the depth snow on the ground on the day of the collapse was 8 inches, and that earlier in the season it had been as high as 18 inches.

For the 1983 building, BOCA 1981 was in effect at the time of the design of this building, and the Engineer certified that the design met MBMA 1981. The roof snow design load calculates to 16.7 psf, based on 19 psf ground snow, the MBMA thermal factor of 1.1, and a 0.8 exposure factor.

For the 1991 building, BOCA 1990 was in effect. For unheated buildings, the Commentary to BOCA 1990 directs the user to the ASCE 7[1] standard. The roof design load calculates to 21 psf, based on 25 psf ground snow, a thermal factor of 1.2, and other appropriate formula factors.

Structural Analyses

Extensive structural analyses were made to assess the adequacy of the roof structure. The analyses focused on the adequacy of the purlins and the frames when subjected to the design loads. The design loads included the self-weight of the structure plus an allowance 45 psf for superimposed dead and live loads. Of this allowance, approximately 13 psf is used for the ballasted roofing system and suspended loads, leaving 32 psf for snow and for localized equipment loads.

For the 1983 building, the purlins near the end rafters were found to be overstressed 20% to 41% in combined shear and bending. These overstresses occur at the end of the purlin overlaps at the first interior rafter. Similarly, for the 1991 building, the purlins near the end rafters were overstressed as much as 38% in combined shear and bending.

For the 1991 building, widespread instances of web crippling of the purlins was found in the interior bays. Overstresses ranged from 20% to 30% for design loadings, as compared to AISI 1986/89. A check for web crippling could not be found in the design calculations

Regarding the rafters, serious overstresses were found near each lean-to bracket. At these locations, the 1983 rafter is overstressed 96% in negative bending, and the 1991 rafter is similarly overstressed 27% for design loads. The severe overstresses are due to the fact that the connections were detailed in a manner inconsistent with the design assumption. The design assumption was that shear connections would be used to attach the new rafters to the original building columns, and therefore the original columns would only have to be designed for the added vertical load. However, the connection was detailed as a moment connection. Figure 4 contrasts these two conditions and the effect on

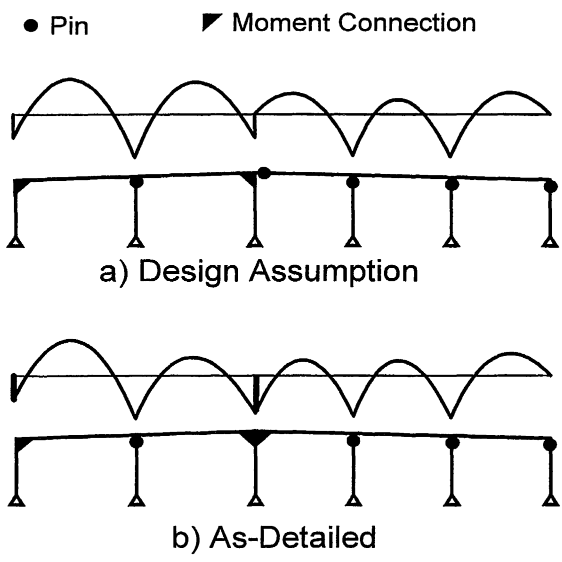


Figure 4 : Moment Diagrams for Frames

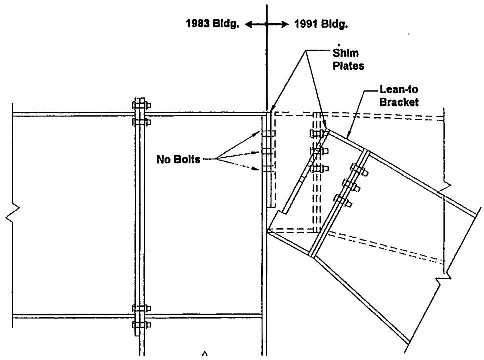


Figure 5: Failed connection.

the negative moment.

In addition, three of the lean-to brackets were inadequately fastened to the columns. The engineer overlooked that three of the columns, the ones to which the original loading dock rafters had attached, had shim plates. Originally, these shim plates were bolted to each other. These bolts were removed in order to dismantle the old loading dock, and were not replaced when the lean-to bracket was welded on, as can be seen in Figure 5. The only attachment between the shim plates was then the non-structural "seam welds" around their perimeter. These seam welds had to transfer the shear and the moment from the new rafters. Calculations showed these seam welds, which would never have been relied upon by a design engineer, actually did have substantial strength that prevented the collapse from occurring sooner.

Conclusions and Lessons Learned

Numerous design deficiencies related to the purlins were identified, some of which may have allowed the collapse to progress.

But the most serious deficiencies related to the rafters, more particularly the connections of the new rafters to the original columns. The detailing of these connections introduced moments that the original structure was not designed for, and neglected the fact that there were shim plates present at some columns that required special attention.

In spite of these deficiencies, it is likely that the structure would not have collapsed had it not been for the snow overload. This is due primarily to the fact that the loads used for the design were in excess of that required by code.

- Make sure that the detailing is consistent with the design assumptions. For example, connections that are assumed to transmit shear should be detailed as such, and connections assumed to transmit moment should be detailed as such.
- Provide clear and explicit erection instructions for special conditions, such as when connecting to an existing structure.
- Research regarding the effect of refrigerated roofs on snow load may be needed. In this case, the measured snow weight was more than twice the values required by code, and this magnitude of load is not explainable from weather records. This may be due to the freezer nature of the facility, which experiences minimal melt-off and

may have frozen drainage paths. Current codes address unheated roofs, but not roofs intentionally kept below freezing.

- Design engineers must be familiar with current codes, and must be thoroughly familiar with the limitations of design software. In this particular case, checks for purlin web crippling and combined shear/bending could not be found in the calculations.

Manufacturing/Warehouse Facility

This facility is located in Edison, New Jersey. It consists of two main buildings: a Manufacturing Building and a Warehouse Building. The two buildings are connected by a narrow "Breezeway" structure. The Manufacturing Building was built in the 1960s. In 1986, the owner contracted with a builder firm to provide a new addition, which consisted of the Warehouse Building and the connecting Breezeway. The roof height of

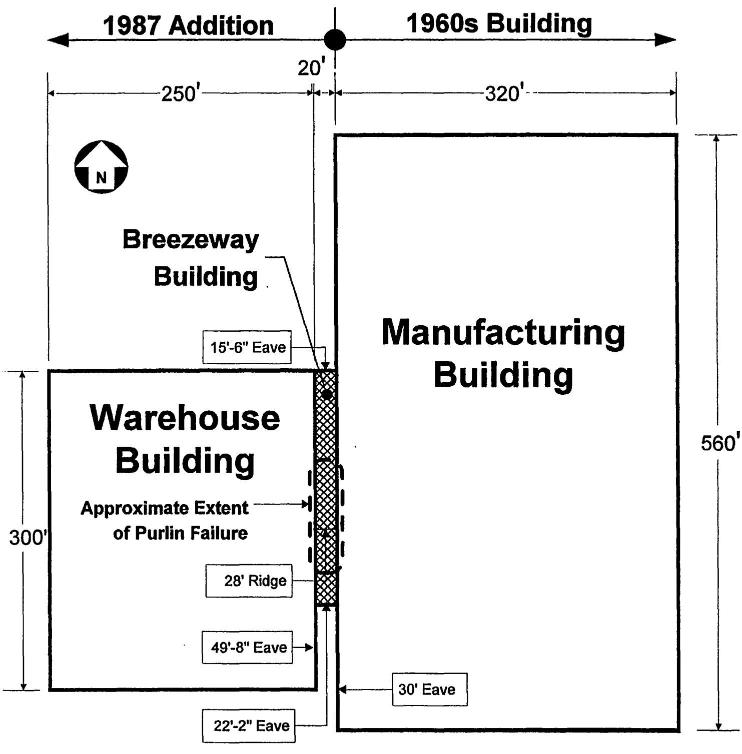


Figure 6: Roof Plan

the Breezeway me was substantially lower than that of the abutting buildings. Figure 6 shows the overall layout of the buildings.

Failure of a portion of the Breezeway roof occurred about a week after the January 1996 blizzard that blanketed the northeast United States. Sometime between the late evening hours of January 14 (Sunday), 1996, and the early morning hours of January 15, the roof purlins, particularly near the ridgeline of the roof, rolled and sagged excessively, although they did not actually collapse to the ground.

LZA Technology was retained, about a year after the failure, by attorneys representing the design/builder.

Description of Breezeway Structure

. The Breezeway measured approximately 20' by 220' in plan, and the roof sloped down from an off center ridge line at a rate of 1 inch per foot.

The roof sheathing is a 24-gauge standing seam roof system, with a low floating clip. This roof sheathing is supported on purlins, which span the 20-foot width of the Breezeway. The purlins are 12-gauge 8" deep cold-formed Z sections, spaced at intervals of 1'-9". The purlins are supported at their ends by 'rafter' beams, which are typically hot-rolled W12x16 sections. The purlins are bolted to upstanding 'fin plates' welded to the top flanges of the rafters.

Strap Bracing for Purlins

In the course of legal proceedings, two questions arose regarding the strap bracing: (1) Had it been installed, and, if so, what was its arrangement? These questions arose due to lack of clarity of the erection drawings.

Strap bracing is not shown pictorially on the erection drawings, but it is called for in a round-about manner. A note on the erection drawings reads, "Note: See sheet S43-X for PM-5 installation." Sheet S43-X is an 8 ½" by 11" standard detail sheet that shows the configuration of purlin bracing for a generic building, and "PM-5" turns out to be the strap material, per the shipping manifest.

An isometric view, reproduced as Figure 7, clearly shows plan bracing along the eaves, anchored the rafters. This arrangement is effective for anchoring the transverse strap that continues up the roof. But what happens to that transverse strap after a few purlins? Does it bend down to the bottom flange of the next purlin, creating the "criss-cross" shown in the elevation? The effectiveness of the bracing provided by such an arrangement is highly questionable, given the interrupted nature of the perpendicular straps and their "anchorage" to tension flanges. Or was it the fabricator's intent to have plan bracing wherever the criss-cross arrangement is indicated? If so, then the arrangement must differ significantly from the plan bracing at the eave, which extends over a pair of purlin bays. And to be effective, the plan bracing would have to be anchored to the rafters, which is not indicated. Or perhaps the intent was to have plan bracing along the ridgeline. And finally, what exactly is meant by “brace band”? The isometric indicates that the plan diagonals are the brace bands, but a note calls for brace bands at the 1/3 points for a 20 foot span, suggesting that the transverse straps are the brace bands. What this detail sheet means, only the fabricator knows for sure!

Ironically, the transverse bracing would be much easier to erect, and it would be much more effective, if it were simply run continuously from eave to peak, and then down to the other eave, thereby eliminating the criss-crosses.

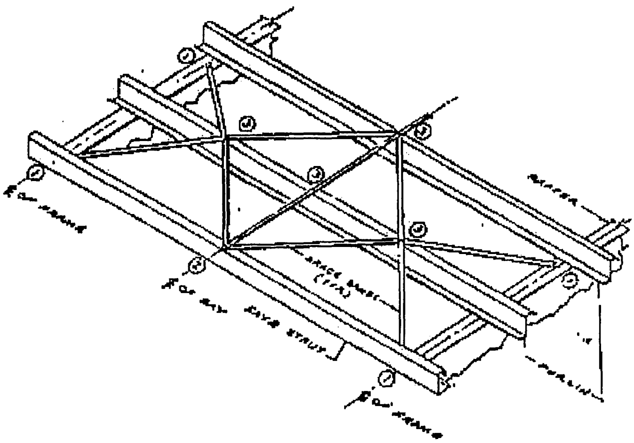


Figure 7: “Isometric View” reproduced from standard detail sheet

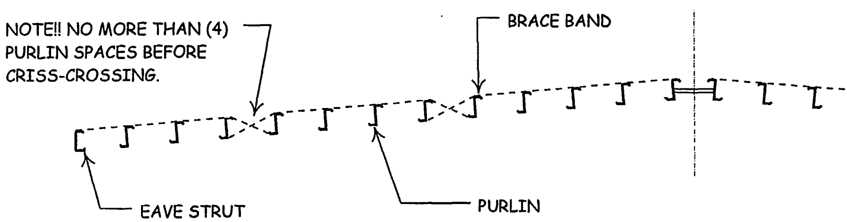


Figure 8: "Brace Band Installation " from standard detail sheet. Redrawn for clarity

Bracing Provided by Standing Seam Roof

Due to the numerous questions about the installation of the strap bracing, as discussed in the preceding section, it became important to estimate the degree of bracing provided by the standing seam roof.

The percentage was estimated using a recently published design guide[2], which is based primarily on research reported in 1990 by Professor Tom Murray at the Virginia Polytechnic Institute. Review of the test results on systems comparable to that used for the Breezeway indicates that the degree of bracing provided by the standing seam roof panels is on the order of 59%. In other words, even without any strap bracing, the purlins can carry approximately 59% of the load that they could carry if they were fully braced.

Snow Loads

The governing building code for the Breezeway design was The BOCA Basic/National Building Code/1984 (BOCA 1984). The ground snow load for this building is slightly less than 20 pounds per square foot for a 50-year mean recurrence storm interval. Applying the appropriate code coefficients results in the loads shown in

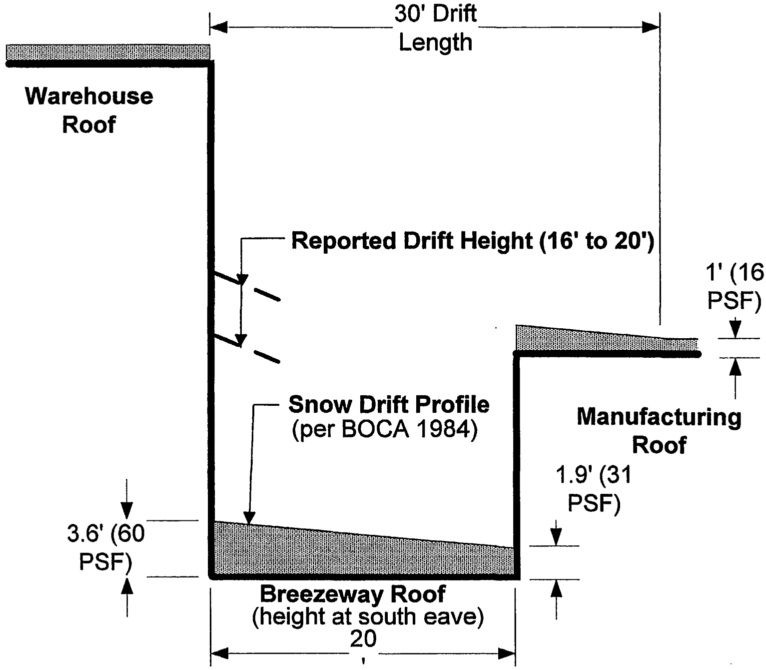


Figure 9: Roof and Snow-load Profiles

Figure 9. The snow heights are based on the BOCA 1996 snow density formula, since BOCA 1984 did not contain a snow density formula.

According to climatological data provided by NOAA for Newark airport, the January 7-8, 1996 storm deposited about 27 inches of snow over a 3 inch pre-existing snow cover. Low temperatures (15 to 19 degrees F) and high winds (30 to 40 mph) associated with the storm combined to create optimum conditions for the formation of large snow drifts. Then on January 12 an additional 1.3 inches of snow was added. The water equivalent of precipitation in January, per NOAA, totaled about 3 inches, which is equivalent to a load of about 15 psf. In addition, NOAA reports that 1 inch of snow that already existed on January 1. This is considered a lower bound on the precipitation, due to the fact that weather station observations tend to "under catch" actual precipitation, particularly in windy conditions. Prior to the storm, there was snow cover already present.

DeGaetano estimates that the actual snow load in the Newark area following the January 1996 storm was approximately 19 psf[2]. This value includes snow that existed prior to the storm and it also includes an adjustment factor to account for the "under catch" of the weather station measuring device. This value of 19 psf exceeds the BOCA uniform design snow load of 16 psf by about 20%, and is equivalent to the snow load that would statistically be expected to occur once every 100 years.

Anecdotal reports by employees indicated that there was 16' to 20' height of drifted snow on the Breezeway roof at the time of the failure.

Structural Analysis

We performed structural calculations to determine the adequacy of the roof purlins to safely support the code-required loads. The adequacy of the purlins is best expressed as a "stress ratio," which is the imposed moment divided by the safe allowable moment. A value less than 1.00 indicates it is understressed; a value more than 1.00 means it is overstressed.

To calculate the bending moment in the purlins, we used the code snow loads plus 9.7 psf for collateral loads plus purlin self-weight. For the assumed case of fully braced purlins, the purlins experience a stress ratio of only 0.48 in bending (54.5 k-inches applied/112.2 k-inches allowable) when subjected to code design loads. If purlin strap bracing had not been installed, then the purlins would have been braced solely by the roof panels. As discussed previously, this partial bracing would allow the purlins to carry approximately 59% of the load that they could carry if fully braced. Using the same design loads, we calculate a stress ratio of 0.82 for the purlins in bending (54.5 k-in./ 59% of 112.2 k-in.). This stress ratio indicates that even if bracing straps were absent, the purlins were capable of safely carrying the design loads.

However, there was no indication in the calculations that the engineer/fabricator relied on the standing seam roof to brace the purlins. All indications were that the purlins were intended to be braced by the straps.

Conclusions and Lessons Learned

In this case, it appears that the failure was caused by a drifted snow load that substantially exceeded the code-mandated values. This is based on the fact that the purlin design was adequate, plus the evidence that the snow loads present exceeded the code design values. Even if the strap bracing had not been installed, the purlins received sufficient incidental bracing from the standing seam roof to safely support the code-required loads. Evidence that the snow load exceeded the BOCA values is provided by meteorological data, published literature, and anecdotal observations by employees of the facility.

Although it does not appear that the strap bracing was the culprit in this case, the lack of clarity of the erection drawings with respect to this important element was conspicuous. It is quite possible that if proper bracing had been provided, that the purlins would have been able to support the imposed overloads without damage

Engineer/fabricators should carefully review the design assumptions that they make with respect to purlin bracing, and take steps to ensure that the bracing requirements are clearly communicated to the erector.

Warehouse

This building, which was used primarily for warehouse purposes, was located in Hamilton Township, New Jersey and was erected in early 1990. The building measured 110 by 180 feet. The eave height of this single slope building varied from 16' to 20'7", at a rate of 1/2" per foot. A masonry perimeter infill wall occurred on all sides, with a masonry parapet on three sides and a gutter on the low side. The parapet was about 8" high along the high side of the roof, and about 5'3" at the low side of the roof.

Frames were spaced at 29'-10" intervals. The frames had one line of intermediate columns, resulting in rafter spans of 55'.

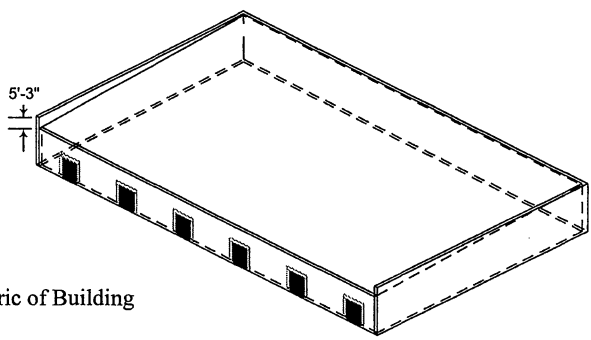


Figure 10: Isometric of Building

Roof purlins, which were spaced at 5'-3", consisted of 8" Z-shapes with 3 ½" flanges, made of 12 gage material in the end spans and 14 gage elsewhere. Purlin laps of nearly 5 feet were called for over the rafter frames. Purlins were attached to rafters via fin plates welded to the rafters. Roof decking was a 24 gage standing seam roof system fastened to the purlins using one-piece clips. Erection drawings called for a line of strap bracing at midspan of the purlins, for both top and bottom flanges.

A partial collapse of the roof occurred in the early morning hours of March 14, 1993 under snow conditions. LZA Technology was retained some years after the collapse by attorneys for the builder.

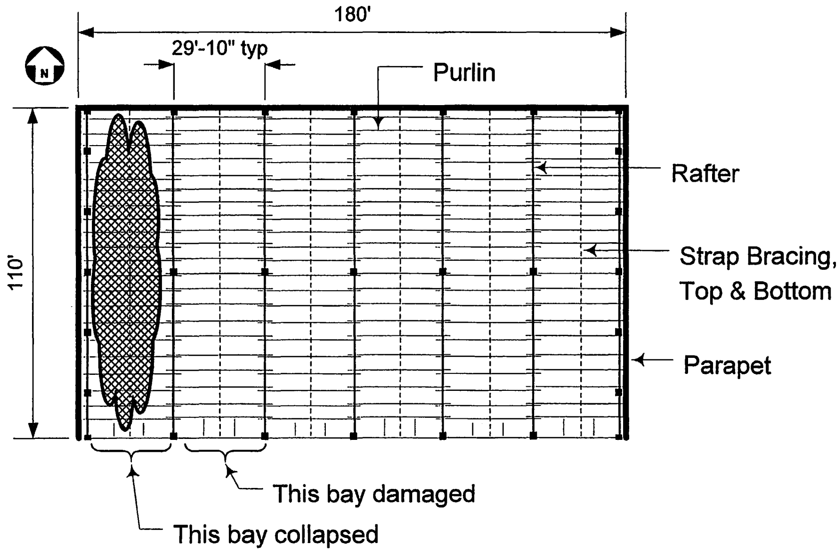


Figure 11: Roof Plan

Snow Loads at Time of Collapse

A meteorological study estimated that the total precipitation from the March 13-14, 1993 storm was between 12 and 18 inches, with a water equivalent of about 2 inches. This is equivalent to about 10 psf. During the storm, temperatures hovered around freezing, and precipitation included heavy wet snow, and rain, and freezing rain. Winds during the day (March 13) were from the East-Northeast and East-Southeast, with gusts up to 55 mph. There was no pre-existing snow load on the roof.

A representative from the engineer/manufacturer inspected the building several days after the collapse and concluded that the cause of the collapse was excessive build-up of ice and snow behind the parapets. No details of observed depths were provided.

The engineer for the foundation inspected the building the morning after the collapse. He stated that there were 1.5 to 2 inches of an ice/snow mixture on the roof adjacent to the collapse area.

Snow Drift Design

Another issue debated during the legal proceedings was the apparent lack of consideration for snow drifting along the parapets. We were able to show that the design had sufficient inherent conservatism that drift loads did not have to be explicitly considered.

BOCA 1987 governed the design of this building. Per BOCA, the ground snow load for the building was 25 psf. Using a snow exposure coefficient of 0.7, the uniform roof snow load was 17.5 psf. BOCA also required a drifted snow surcharge, as high as 58 psf, be considered included at the location of the highest parapet.

It was true that the calculations did not explicitly consider drifting snow. However, the design was based on a larger ground snow load (30 psf) than what was required by code (25 psf). The ground snow contours are closely spaced in the locale of the building, so it is likely that the highest snow load existing in the county, not the snow load at the location of the building, was used. This resulted in a uniform roof snow load of 21 psf, as opposed to the 17.5 psf required by code. The uniform snow load design therefore controlled over the live load case of 20 psf.

Our calculations indicated that the moments and shears used to design the purlins were slightly larger than the moments and shears developed by the code snow load, even considering drifting, and the code minimum live load. Therefore, even though the design did not expliclty consider snow drifting, the roof structure was indeed capable of safely carrying the snow drift loads.

Apparent Overstresses in Calculations

A red herring that received considerable discussion during the legal proceedings was the apparent overstress of the purlins, as stated by the engineer's own calculations.

The calculations appeared to indicate that certain purlins were overstressed 18% in bending, and that others are 0.8% overstressed. There are two reasons for this. Firstly, in the area of the apparent 18% overstress, the calculations were based on end-to-end lap lengths of 2'-2", as opposed to the lap of nearly 5 feet indicated on the drawings. The design engineer recognized that the overstress was false, apparently made a side calculation, and annotated the computer output, "OK by inspection with 4'-11½" total laps." Secondly, in the area of the slight overstress, the calculations were based on a span length over a foot longer than the actual span. Apparently the span used was from an earlier, discarded design scheme for the building.

We performed calculations that considered both of these factors, and were thus able to show that these overstresses did not in fact exist.

Bracing of Purlins

The most difficult issue in the case was the degree of bracing afforded to the purlins. The design engineer for the structure had assumed that the purlins were fully braced by the standing seam roof. The calculations did not include any test reports to substantiate this assumption. Engineers for the argued that the purlins were effectively unbraced for their entire span of nearly 30 feet.

This is an issue that has received considerable attention in recent years by the metal building industry. The first comprehensive test data on this subject was published in November of 1990 [3,4], about 9 months after the building was erected. Using the values in this literature for comparable roof systems, we estimated that the bracing afforded the purlins by standing seam roof was about 60%. In other words, the partially braced purlins could safely support a moment equal to 60% of the safe load for the fully braced condition. Our calculations indicated that this amount of bracing would result in a 28% overstress of the purlins when subjected to the code snow load. This degree of overstress is serious. However, climatological data indicated that the precipitation (10psf) was less than the code value (17.5 psf) by a significant margin. Ratioing the stress in the purlins accordingly, the purlins should have been capable of supporting the actual snow load without overstress.

Was it reasonable for the design engineer to assume that the purlins were fully braced by the standing seam roof? Probably not. Even though data on the bracing provided by comparable standing seam roof systems was not published until after the building was erected, it undoubtedly was known at the time that 100% bracing was an unrealistic expectation. In the absence of literature, it was incumbent on designers at this juncture in time to either use discrete bracing, such as straps, or to perform tests on their own specific systems.

Conclusions an Lessons Learned

The cause of this collapsed was not determined to a high level of certainty. Several allegations of improper design, such as the apparent overstress at purlin laps and the apparent disregard for drifting snow, were eliminated as potential causes. Likewise, it was shown that the degree of bracing afforded by the standing seam roof appeared sufficient to support the estimated precipitation, although the bracing was not as effective as the designer had assumed and though it would not safely support code loads.

It is even possible that other unidentified factors contributed to the collapse, such as deficiencies in erection, fabrication, or even abuse of the structure by tenants hanging loads from the roof that it was not intended to carry.

The most important lesson to be learned from this collapse is the importance of ensuring that the assumptions made by the designer regarding the degree of bracing provided to the purlins are based on solid engineering data. For it is likely that if a realistic degree of bracing had been assumed by the designer, that the roof would have survived the storm.

---

[1] ASCE 7-95, "Minimum Design Loads for Buildings and Other Structures," American Society of Civil Engineers, 1995.

[2] DeGaetano, Arthur T., Schmidlin, Thomas W., and Wilks, Daniel S., "Evaluation of East Coast Snow Loads Following January 1996 Storms," ASCE Journal of Performance of Constructed Facilities, May 1997.

[3] "Base Test Method for Gravity Loaded Standing Seam Roof Systems," Report CE/VPI-ST90/07 by The Charles E. Via, Jr. Department of Civil Engineering, Virginia Polytechnic Institute and State University, prepared for MBMA Project 502, December 1990.

[4] "Evaluation of the Base Test Method for Predicting the Flexural Strength of Standing Seam Roof Systems under Gravity Loading" Report CE/VPI-ST89/07, prepared for MBMA Project 403,dated November 1990.