The Source
Of the Problem

Careful sleuthing after a roof collapse calls into question certain decisions with regard to bridging layout and design loads for uplift. But does the fault lie with the designers or with inconsistencies in the sources upon which those designers relied? **By Erik L. Nelson, Ph.D., P.E., M.ASCE, Deepak Ahuja, P.E., M.ASCE, Stewart M. Verhulst, P.E., M.ASCE, and Erin Criste, M.ASCE**

A roof collapse occurred at a commercial warehouse building during a storm event in the Dallas area in February 2001. The structure included open warehouse space and had interior demising walls. The foundation consisted of a conventionally reinforced concrete slab on grade with perimeter piers and interior footings. The size of the warehouse structure was 540,000 sq ft (50,166 m²). The interior construction and framing took the form of steel joists and girders at the roof framing and concrete tilt-up wall panels at the perimeter walls. The structure was built in 1996.

The roof was indicated as a mechanically attached, single-ply EPDM (ethylene propylene diene monomer) membrane over 1½ in. (38 mm) of isocyanurate foam. The roof deck was a 1½ in. (38 mm) deep painted metal deck with a thickness of 22 gauge. The typical roof joists were of the 26K9 (K-series) type with three rows of horizontal bridging for the top chord, four rows of horizontal bridging for the bottom chord, and one row of diagonal bridging (also called X bridging). The joists, approximately 50 ft (15.24 m) long and spaced 6 ft 3 in. (1.9 m) on center, spanned between column bays in a 50 by 50 ft (15.24 by 15.24 m) grid.

The available information regarding the structure and the storm event included storm data, design documents, and shop drawings for the framing. The net uplift load used for the design of the joists was listed on the design drawings as 10 psf (0.48 kPa).

Observations of the structure uncovered damaged members and inadequate connections. The following summarizes a few of the items observed:

Buckling at Joist Bottom Chord

Buckling Failure at End Web

Roof Technical Services, Inc., both

The top chord of an original joist was observed not to be straight; that is, it appeared to have been displaced out of the plane of the joist. Moreover, some lateral movement was evident at the bottom chord, occurring near the midspan of the joist between the bridging locations. The bridging for the steel roof joists consisted of both horizontal bridging (top and bottom) and X bridging. Generally, the typical connection at the X bridging was a bolted connection to an angle plate welded to the joist, and the typical connection for the horizontal bridging to the joists was specified as a fillet weld.

Measurements of the spacing between bridging locations along the bottom joist chords were taken. At the original joists, a typical bottom chord bridging spacing of 12 ft 8 in. to 12 ft 9 in. (3.86 to 3.89 m) was observed across the midspan of the joists. Typically, bottom chord bridging was located at the end panel points of the joists and was included as part of the four bottom chord bridging locations.

Based on observations, a typical failure mode for the joists was buckling at the bottom joist chords, near midspan of the joists. Buckling failures of the end web members of the joists also were observed, suggesting multiple or combined failure modes as a result of uplift (see photographs at left).

Several meteorological reports were available regarding the storm that affected the site. Based on the reports received, severe thunderstorms occurred with wind gusts of 75 to 80 mph (120.7 to 128.7 km/h). The maximum reported wind gust was indicated as 77 mph (123.9 km/h)—recorded at an airport approximately 3 mi (4.8 km) from the site.

One of the meteorological reports indicated that the storm was a thunderstorm with a rotating supercell (mesocyclone). It described a supercell thunderstorm as extremely violent and forceful and noted that such storms commonly have "intense microburst updrafts and associated downdrafts." Furthermore, the meteorological reports noted that supercell thunderstorms are the type that most frequently produce tornadoes and that the peak wind gusts in this case would probably have been even higher if a tornado had been produced by the storm.

The applicable building code for the design of the structure was the 1991 edition of the *Uniform Building Code*, or UBC (Whittier, California: International Conference of Building Officials). Based on figure 23-1 of that publication, the design wind speed for the site in question is 70 mph (112.6 km/h). This is based on a "fastest-mile" wind speed criterion, which is partially defined in the UBC as "the highest sustained average wind speed based on the time required for a mile-long sample of air to pass a fixed point." It should be noted that the fastest-mile wind speed criterion also appeared in the 1994 and 1997 editions of the UBC.

More recent standards and codes, including recent editions of ASCE 7 (*Minimum Design Loads for Buildings and Other Structures*) and the 2000 and 2003 editions of the *International Building Code* (Whittier, California: International Code Council), use similar parameters for determining wind pressures. However, in these standards and codes, other factors, such as site topography and wind gusts, are used more explicitly in determining wind pressures. Furthermore, these more recent standards use a peak gust wind speed rather than the fastest-mile wind speed.

The roof dead load for the warehouse structure was calculated to be only 5.32 psf (0.26 kPa), including the self-weight of the joist framing. Because of its geometry, the warehouse is considered an "open structure" by the UBC for determining wind pressures. Open structures generally have higher

Table 1 Wind Uplift Loading

Location	Gross uplift (psf)	Net uplift (psf)
Discontinuities	–24.46	–19.14
Field	–22.83	–17.51

Table 2 Incremental Wind Uplift Loading

Wind speed (mph)	q_S*	Gross uplift (psf)	Net uplift (psf)
70	12.6	–22.83	–17.51
80	16.4	–29.71	–24.39
90	20.8	–37.68	–32.36
100	25.6	–46.38	–41.06
110	31	–56.16	–50.84

*Wind stagnation pressure at the standard height of 33 ft (10.06 m) as defined in table 23-F of the 1991 *Uniform Building Code.*

uplift pressures because of wind than structures that are not "open." In this case, the design net uplift pressure at the field of the roof is approximately 60 percent higher for an open condition than for a condition that is not open.

The uplift loads for the subject warehouse roof were calculated in accordance with the 1991 UBC. Based on tributary area, the joists were regarded by that source as "elements and components" with respect to wind uplift loading. The gross and net uplift pressures are given in table 1 (the "field" of the roof being the main roof area and "discontinuities" the areas of the roof where architectural features result in increased uplift load, for example, near the eaves). The calculated net uplift exceeds the 10 psf (0.48 kPa) indicated on the construction drawings by more than 75 percent.

For purposes of comparison, the changes in the gross and net uplift wind pressures for incremental changes in the wind speed are given in table 2. The values indicated in the table are calculated for elements and components in the field of the roof for the structure, using the method described in the 1991 UBC.

As indicated in table 2, gross wind pressures increase as the square of the wind speed (using the "fastest mile" speed per the 1991 UBC). Therefore, wind speeds in excess of the design wind speed of 70 mph (112.6 km/h) could cause significant increases in the net uplift wind pressures on the roof and roof framing.

In structural engineering design, factors of safety are employed to account for unknown conditions, variability in materials, inherent design assumptions, and construction deficiencies and to provide for the safety of the public. Considering the allowable stress design (ASD) of steel structures, a factor of safety, F_S, of 1.67 is used for tension members and beams and an F_S value of $23/12$ = 1.92 is used for typical long compression members (those that perform as column members, et cetera).

Generally, a factor of safety is not a reserve capacity and it cannot be used as such during the design or construction of a structure. The factor of safety is a minimum design requirement as established by the applicable building code and applicable structural codes and standards.

While the Steel Joist Institute (SJI), of Myrtle Beach, South Carolina, does require an F_S value of 1.65 in the design, the actual value with regard to compression is the $23/12$ factor (F_S = 1.92, as noted above) applied to the Euler buckling formula. It should be noted that, in the original joist designs, a one-third increase was included for the allowable stresses arising from wind loads. This is in accordance with section A5.2 of the ninth edition of the *Manual of Steel Construction—Allowable Stress Design* (Chicago: American Institute of Steel Construction, 1989). Therefore, the actual F_S for the compression design of the joist members was about 1.44 ($23/12$ divided by $4/3$ for the wind stress increase).

As noted above, the proper net uplift design load for the joists at the roof of the warehouse was 17.51 psf (0.84 kPa), which is about 75 percent higher than the design load for which the joists were actually designed (10 psf, or 0.48 kPa). This 75 percent increase in uplift pressure would therefore exceed the F_S of 44 percent, leading to a probable failure for wind speeds approaching the design wind load.

It is further indicated in the structural drawings that the bridging shall conform to the SJI specifications, namely, the *Standard Specifications for Open Web Steel Joists, K-Series* (1989). The structural plan notes for the warehouse structure indicate that "steel joists shall be braced by horizontal and/or diagonal bridging as required by the Steel Joist Institute."

Furthermore, the applicable building code for the project—the 1991 UBC—includes the SJI specifications as a UBC standard. The SJI specifications indicate two types of bridging: horizontal bridging and diagonal bridging. The SJI specifications state that "horizontal bridging shall consist of two continuous horizontal steel members, one attached to the top chord and the other attached to the bottom chord."

Moreover, regarding the amount and spacing of bridging, the SJI specifications have this to say: "In no case shall the number of rows of bridging be less than shown in the bridging table. Spaces between rows shall be approximately uniform. See section 5.11 for bridging required for uplift forces."

Section 5.11 of the SJI specifications discusses uplift provisions for steel joists and is included here for reference:

> Where uplift forces due to wind are a design requirement, these forces must be indicated on the contract drawings in terms of net pounds per square foot. When these forces are specified, they must be considered in design of joists and/or bridging. A single line of bottom chord bridging must be provided near the first bottom chord panel points whenever uplift due to wind forces is a design consideration.

A footnote in that section refers the interested reader to the SJI's *Technical Digest No. 6—Design of Steel Joist Roofs to Resist Uplift Loads.*

For the 26K9 joists indicated in the joist shop drawings, the bridging table included in the SJI specifications would require four rows of bridging for spans of 46 to 59 ft (14 to 18 m). Therefore, the maximum spacing for bridging at the joist would be 11 ft 9 in. (3.58 m), a value obtained by dividing 59 ft (18 m) by five spaces. This approach for determining the spacing limitation for bottom chord bridging is also indicated in *Technical Digest No. 6—Design of Steel Joist Roofs to Resist Uplift Loads.* As noted above, this work is cited in section 5.11 of the SJI specifications. However the specifications do not say which version of that digest to follow. It is our opinion that the joists should have been designed according to the most current version of the digest. Based on our discussions with the SJI, at the time of the design and construction of the structure, the 1994 edition would have been the appropriate version.

The shop drawings for the steel joist roof framing indicate three rows of horizontal top chord bridging and four rows of horizontal bottom chord bridging (including bridging at each end panel point), in addition to the single row of X bridging. The X bridging serves as bridging for both the top and bottom joist chords and is indicated at one of the equally spaced top chord bridging locations closest to the midspan of the joists. Therefore, a total of four rows of top chord bridging and five rows of bottom chord bridging existed. As noted above, the roof joists were typically 50 ft (15.24 m) in length.

Based on the measured geometry of the joists at the site (including the location of the end panel points) and the fact that the X bridging was placed at one of the equally spaced top chord bridging locations, a total of six rows of bottom chord bridging would have been required to comply with the SJI maximum spacing limitation of 11 ft 9 in. (3.58 m). The bridging indicated in the shop drawings and the bridging layout observed at the site have typical spacings between points of bottom chord bridging that are in excess of the spacing limitations of the SJI specifications (12 ft 9 in. [3.89 m] and 12 ft 8 in. [3.86 m], versus 11 ft 9 in. [3.58 m]). If the proper net uplift pressure had been used, the spacing of the bottom chord bridging would not have exceeded the SJI limitations.

The bridging layout on the steel drawings and the layout observed at the site did not conform to section 5.4 of the SJI specifications because the bridging spacings were not approximately uniform. During site visits, the bridging spacing at a typical original joist was measured to be 12 ft 8 in. (3.86 m) at one side of the X bridging and 6 ft 8 in. (2.03 m) at the other side. Thus, the bridging spacings vary by up to 90 percent along a single joist.

As noted above, failures of joist end web members were observed. Under normal gravity load, these are tension members; however, load reversal occurs when net uplift loads control the design. Therefore, these members are in compression under net uplift conditions. For the particular joists at the subject warehouse, net uplift was the governing design condition for the joist end webs.

It should be noted that the roof joist calculations indicated a Kl/r ratio of 185.2 and a K factor of 0.8 for the end web members. (Here K is the effective length factor, l is the unbraced length of the member, and r is the radius of gyration. The quantity l/r is referred to as the slenderness ratio, and Kl/r is the effective slenderness ratio.) Based on these values, the l/r ratio for the end web members on the joists was 231.5. Section 4.3 of the SJI specifications defines the maximum allowable slenderness ratios (l/r) for use in K-series steel joists as follows:

Top chord interior panels	90
Top chord end panels	120
Compression members other than top chord	200
Tension members	240

In a case where a joist is to resist a net uplift, all diagonal members and all bottom chord and top chord members should be in compression in at least one of the load cases. In fact, the governing load case for the design of these end web members was the uplift condition, where they are in compression. Based on these criteria, the limiting l/r ratio for the end web member in compression, as indicated by the SJI specifications, would be 200. This is exceeded by the actual l/r of 231.5.

However, the SJI's *Technical Digest No. 6—Design of Steel Joist Roofs to Resist Uplift Loads* uses the tension member criteria of 240 for a limiting slenderness ratio of an end web member. Moreover, as indicated above, that digest also uses a K value of 0.8 for the calculation of allowable compressive stress in the member. But the Kl/r ratio is 185.2, which is less than the slenderness ratio of 200 indicated in the SJI specifications. This issue appears to be ambiguous and makes it difficult to reconcile the SJI specifications with the technical digest.

Table 3 Bottom Chord Net Uplift Capacities

Bridging spacing	Allowable capacity pressure: no stress increase (psf)	Allowable capacity pressure: one-third stress increase (psf)	Euler buckling capacity pressure (psf)
11 ft 9 in.*	10.91	14.55	20.92
12 ft 6 in.	9.64	12.86	18.48
12 ft 8 in.	9.39	12.51	17.99
12 ft 9 in.**	9.27	12.36	17.76
13 ft 0 in.	8.92	11.89	17.09
13 ft 2 in.	8.69	11.59	16.66
14 ft	7.69	10.25	14.73
14 ft 2 in.***	7.51	10.01	14.39
14 ft 6 in.	7.17	9.55	13.73

*Maximum allowable bottom chord bridging spacing per SJI.
**Maximum measured bottom chord bridging spacing at the site.
***Maximum allowable bottom chord bridging spacing per design.

The roof joists were analyzed for joist capacity considering different failure modes. As noted above, the particular failure modes observed at the joists included compression failures (buckling) of the bottom chords near midspan.

It appears that the bottom chords of the failed joists buckled laterally. (For the purposes of this discussion, it will be considered as buckling about the y-y axis.) The design calculations for the joists were available for review and they indicated an allowable bridging spacing, L_{yy}, of 14 ft 2 in. (4.32 m) for the stress in the bottom chord. This allowable bridging spacing was calculated using the one-third stress increase in the allowable bottom chord stress and the 10 psf (0.48 kPa) net uplift loading indicated on the design documents. This is an important reference point when considering the effect of the inadequate net uplift design load on the joist design.

The capacities of the joists, considering the failure mode at the bottom chord, are presented in table 3. The capacities are indicated in terms of the net uplift pressure (on the joists) for different bridging spacings. Table 3 includes the capacities based on the allowable load, the permitted one-third stress increase according to the American Institute of Steel Construction's *Manual of Steel Construction—Allowable Stress Design*, and the Euler buckling load (without the buckling safety factor). The bottom chord capacity was determined for a range of bridging spacings.

Table 3 indicates the increase in bottom chord capacity as the bridging spacing decreases. The net uplift pressure that should have been used for the roof design was 17.51 psf (0.84 kPa), which exceeds all of the allowable values listed in table 3. What is more, this proper net uplift value exceeds the capacity (no factor of safety) of the joists if they had a bridging spacing of 14 ft 2 in. (4.32 m), further indicating that the factor of safety for the joist design was eclipsed by the use of the improper design load. Finally, table 3 indicates that if the joists had been designed and constructed in conformity with the SJI specifications, the actual capacity of the bottom chord would have exceeded the proper design load. The actual capacity of the joists would have been 18 percent higher if the bridging layout had conformed to the SJI specifications (20.92 psf [1.00 kPa] for 11 ft 9 in. [3.58 m] spacing, versus 17.76 psf [0.85 kPa] for the 12 ft 9 in. [3.89 m] spacing observed).

This illustrates the effect of improper design loading and excessive joist bottom chord bridging spacing on the actual capacity of the joist for wind uplift. Any factor of safety in the joist design was eclipsed by the combination of the miscalculated design load and the failure to comply with SJI standards for the spacing of the bottom chord bridging. Of course, the use of a nonconservative design load may result in failure irrespective of the SJI standards. However, designing a bridging layout that complies with the SJI standard can only increase the capacity.

In the most recent standard specifications for K-series joists, which date to 2003 and became effective in 2005, the SJI has made changes, including clarification of top chord bridging and bottom chord bridging requirements. As part of that clarification, the SJI now requires that the number of rows of bottom chord bridging not be less than the number of rows of top chord bridging. The bottom chord bridging spacing will also have to be such that the bottom chord complies with the slenderness requirements of the SJI and any specified strength requirements. The language regarding bridging has been further clarified to distinguish between the bottom chord and top chord bridging, and the specifications now note that they may be spaced independently.

It is our opinion that the changes implemented by the SJI are helpful in clarifying the top and bottom chord bridging requirements. However, some ambiguity remains, including the determination of the governing slenderness ratio for a bottom chord member and an end web member if uplift controls the design. The use of l/r also remains in the SJI standard, which causes some confusion owing to the use of Kl/r in *Technical Digest No. 6—Design of Steel Joist Roofs to Resist Uplift Loads,* as noted above. The ambiguous "approximately uniform" spacing requirement has been removed from the 2003 SJI specifications for K-series joists.

The net uplift design load for the joists was inadequate, and the design load should have been about 75 percent higher. As noted in this assessment, the roof systems selected for the original construction were uniquely light. This should have been considered in the design process when addressing wind uplift. As noted, the factor of safety for the joist design was eclipsed by the use of the improper design load for uplift and by the failure to comply with the proper SJI bridging requirements for the bottom chord bridging.

The misuse of the SJI specifications regarding the bridging spacing apparently resulted from misunderstanding the SJI specifications for the joists. Ambiguity in the SJI specifications—for example, calling for "approximately uniform" spacing of the bridging—and a failure to explicitly state that the bottom chord bridging also is subject to maximum spacing requirements contributed to the misunderstanding.

Furthermore, there is ambiguity between the SJI specifications and that organization's *Technical Digest No. 6—Design of Steel Joist Roofs to Resist Uplift Loads* regarding the proper slenderness ratio for the end web members. This also requires clarification to prevent further misunderstanding. In the most recent standard specifications for K-series joists, SJI has made changes regarding bridging. The changes include clarifying the top chord bridging and bottom chord bridging requirements. It is our opinion that these changes will be helpful in properly determining the top and bottom chord bridging requirements, although some ambiguity remains.

In the case of the subject warehouse, the failure to comply with the SJI specifications lessened the capacity of the joists in uplift. These joists ultimately failed in a violent manner. An increase in the joist capacities for uplift could have prevented the failure or lessened its effect. The actual capacity of the joists would have been 18 percent higher if the bridging layout had conformed to the SJI specifications. This illustrates the role of factors of safety and minimum standards—for example, those promulgated by the SJI—in the arena of public safety. ■

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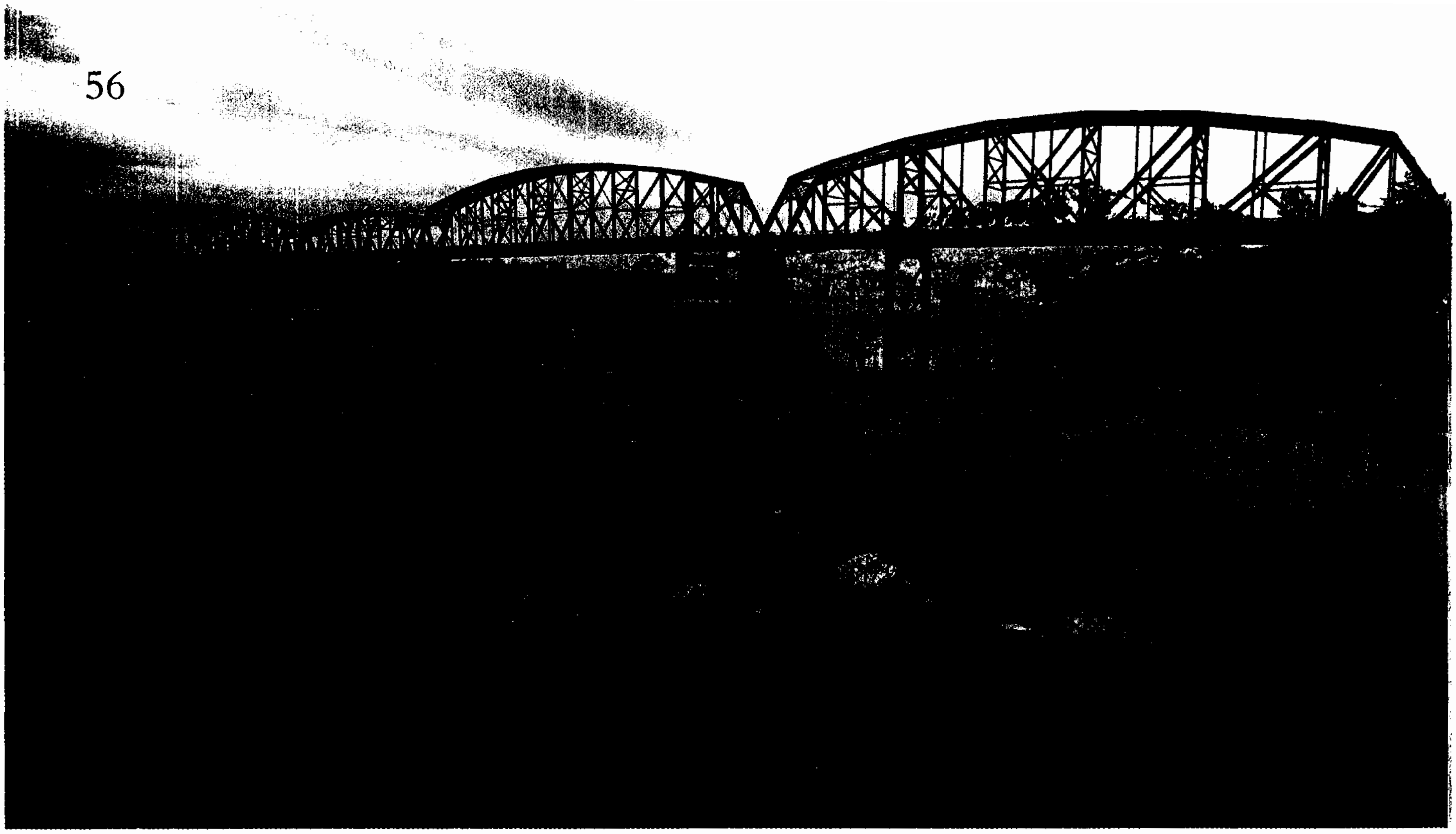

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ON THE COVER Designed by the French architect Jean Nouvel, the new home of the Guthrie Theater, in Minneapolis, incorporates many areas that can alter a visitor's sense of space, including the "yellow box" lobby, which cantilevers in two directions and is infused with a glow from lights below. Photograph by Roland Halbe Fotografie. (See page 40.)