Roof Collapse: Forensic Uplift Failure Analysis

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ABSTRACT

Many factors affect the performance of structural roof framing, and if deficient components exist, the structural integrity is compromised. When a roof system is improperly designed, failure may result from under-design regarding net uplift pressures. Today's commonly used lightweight roofing products (EPDM, poly-isocyanurate) have made net uplift loads a more critical design load, and in some instances, the controlling case. In particular, a commercial warehouse building was under-designed for net uplift pressures, which in conjunction with unclear bridging spacing requirements per Steel Joist Institute (SJI) requirements, resulted in a roof collapse during a storm event. The net uplift design load for the steel joist roofing system should have been higher than what was specified on construction drawings. Additionally, a lack of clarity in the SJI requirement for joist bottom chord bridging resulted in excessive bridging spacing, which lessened the capacity of the roof framing considering uplift. Consideration of the lightweight roofing materials in the joist design and clarity in SJI uplift tables would have prevented the roof collapse.

INTRODUCTION

A roof collapse occurred at a commercial warehouse building during a storm event. The structure included an open warehouse space at the interior, with interior demising walls. The foundation consisted of a conventionally reinforced concrete slab-on-grade with perimeter piers and interior footings. The structure was indicated as being 540,000 square feet. The interior construction and framing consisted 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 membrane over 1 1/2" Isocyanurate Foam. The roof deck was a 1 1/2" deep, 22-gage wide rib painted metal deck. The typical roof joists were 26K9 (K-series) joists with three (3) rows of horizontal bridging for the top chord, four (4) rows of horizontal bridging for the bottom chord, and one (1) row of X-bridging. The joists were approximately 50' in length, spaced at 6'-3" on-center and spanned between column bays in a 50' by 50' 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.

OBSERVATIONS AT WAREHOUSE

Observations of the structure were made and items including damaged members and inadequate connections were noted. The following is a summary of a few of the items observed:

The top chord of an original joist was observed to not be straight (i.e. it appears to have been displaced out of the plane of the joist). Additionally, some lateral movement was evident at the bottom chord, occurring near mid-span of the joist between the bridging locations. The bridging for the steel roof joists consisted of both horizontal bridging (top and bottom) and cross bridging (also denoted as "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'-8" to 12'-9" was observed across the mid-span of the joists. Typically, the bottom chord bridging was located at the end panel points of the joists and was included as part of the four (4) bottom chord bridging locations.

Based on observations, a typical failure mode for the joists was buckling at the bottom joist chords, near mid-span of the joists. Buckling failures of the end web members of the joists were also observed, suggesting multiple or combined failure modes due to uplift. Refer to Figure 1 and Figure 2 below.

Figure 1. Buckling at joist bottom chord (Photo courtesy of Roof Technical Services, Inc.).

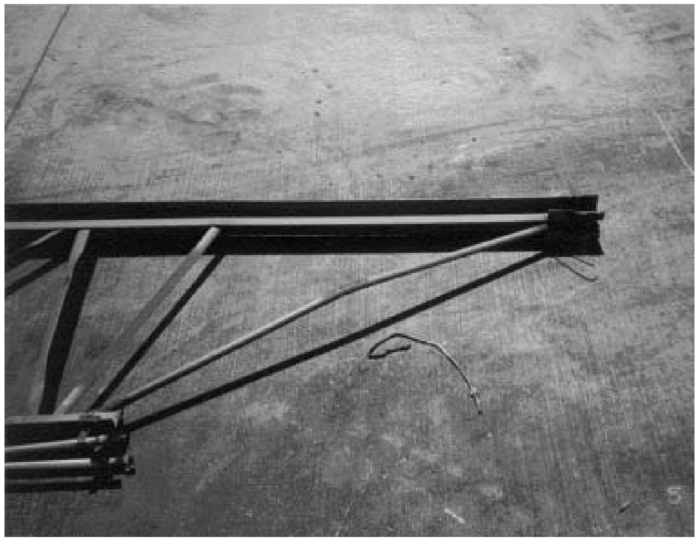

Figure 2. Buckling failure at end web (Photo courtesy of Roof Technical Services, Inc.).

METEOROLOGICAL REPORTS

Several meteorological reports regarding the storm occurring at the site were available. Based on the reports received, severe thunderstorms occurred with wind gusts of 75-80 mph. The maximum reported wind gust is indicated as 77 mph-recorded at an airport approximately 3 miles from the site.

One of the meteorological reports indicated that the storm at the site was a rotating supercell thunderstorm (mesocyclone). It was reported that a supercell thunderstorm is the most violent and forceful classified storm and that these types of storms commonly have "intense micro burst updrafts and associated downdrafts." Furthermore, the meteorological reports noted that supercell thunderstorms are the type which most frequently produce tornadoes and that the peak wind gusts would likely have been even higher if a tornado was produced by the storm.

DISCUSSION OF WIND LOADS AT THE SITE

The applicable Building Code for the design of the structure was the 1991 Uniform Building Code (UBC).

Based on Figure No. 23-1 in the 1991 UBC, the design wind speed for the site is 70 mph. This is based on a "fastest-mile" wind speed criteria, 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 criteria was also used by the 1994 and 1997 editions of the UBC.

More recent standards and codes, including recent ASCE 7 standards and the 2000 and 2003 editions of the International Building Code (IBC), 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 the determination of wind pressures. Also, 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, including the self-weight of the joist framing. Due to the geometry of the warehouse structure, it is considered as an "Open Structure" per the UBC for determination of wind pressures. Open structures generally have higher uplift pressures due to wind than structures which are not "open". In this case, the design net uplift pressure at the field of the roof is approximately 60% higher for an open condition compared to a not-open condition.

The uplift loads for the subject warehouse roof were calculated in accordance with the 1991 UBC. Based on the tributary area of the joists, they were considered as "elements and components" regarding wind uplift loading per the 1991 UBC. The gross and net uplift pressures are included in Table 1 below (the "field" of the roof refers to the main roof area and "discontinuities" refer to the areas of the roof where architectural features result in increased uplift load – such as near the eaves of the roof). The calculated net uplift exceeds the 10 psf indicated on the construction drawings by more than 75%.

Table 1. Wind Uplift Loading.

Location	Gross Uplift (psf)	Net Uplift (psf)
Discontinuities	-24.46	-19.14
Field	-22.83	-17.51

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

Table 2. Incremental Wind Uplift Loading.

Wind Speed (mph)	q_s	Gross	Net Uplift

		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

As indicated in Table 2, gross wind pressures increase as a 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 could cause significant increases in the net uplift wind pressures on the roof and roof framing.

FACTORS OF SAFETY

In structural engineering design, "Factors of Safety" (FS) are employed to account for unknown conditions, variability in materials, inherent design assumptions, construction deficiencies, and to provide for the safety of the public. Considering Allowable Stress Design (ASD) of steel structures, a FS=1.67 is used for tension members and beams and a FS=23/12=1.92 is used for typical long compression members (those which perform as column members, etc.).

Generally, a Factor of Safety is not a reserve capacity, and 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 SJI does require a 1.65 factor of safety in the design, the actual factor of safety with regard to compression is the 23/12 factor (FS=1.92, as noted above) applied to the Euler buckling formula. It should be noted that, for the joist designs performed, a 1/3 increase was included for the allowable stresses due to wind load per Section A5.2 of the AISC Specifications (AISC, 1989). Therefore, the actual factor of safety 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, which is about 75% higher than the design load for which the joists were actually designed (10 psf). This 75% increase in uplift pressure would therefore exceed the Factor of Safety of 44%, leading to a likely failure for wind speeds approaching the design wind load.

STEEL JOIST DESIGN AND BRIDGING REQUIREMENTS

It is further indicated that the bridging shall conform to the SJI specifications. 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."

Additionally, the applicable building code for the project (1991 UBC) includes the SJI Specifications as a UBC Standard.

The SJI Specifications indicate 2 types of bridging: horizontal bridging and diagonal bridging (also called X-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.*

Also, regarding the amount and spacing of bridging, the SJI Specifications state,

> *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:

> *5.11 UPLIFT*
>
> *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* ***<u>bottom chord</u>*** *bridging must be provided near the first bottom chord panel points whenever uplift due to wind forces is a design consideration.**
>
> **For further reference, refer to Steel Joist Institute Technical Digest #6, "Structural Design of Steel Joist Roofs to Resist Uplift Loads."*

Based on the bridging table included in the SJI Specifications; for the 26K9 joists indicated in the joist shop drawings, 4 rows of bridging are required for spans from 46' to 59'. Therefore, the maximum spacing for bridging at the joist would be 11'-9" (59'/5 spaces). This approach for determining the spacing limitation for bottom chord bridging is also indicated in SJI Technical Digest No. 6. As noted above, SJI Technical Digest No. 6 is specifically referred to for further reference by Section "5.11 Uplift" of the SJI Specifications. The SJI Specifications do not list which version of SJI Technical Digest No. 6 to follow. It is the authors' opinion that the joists should have been designed according to the most current version of the Technical Digest at that time. Based on our discussion with SJI, at the time of the design and construction of the structure, the most current SJI Technical Digest No. 6 was the 1994 version.

The steel joist shop drawings for the steel joist roof framing indicate three (3) rows of horizontal top chord bridging and four (4) rows of horizontal bottom chord bridging (including bridging at each end panel point), in addition to the one (1) 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 nearest the mid-span of the joists. Therefore, there were four (4) rows total of top chord bridging and five (5) rows total of bottom chord bridging. As noted above, the roof joists were typically 50'-0" in length.

Based on the measured geometry of the joists at the site (including the location of the end panel points) and based on the X-bridging being placed at one of the equally-spaced top chord bridging locations, a total of 6 rows of bottom chord bridging would be required for compliance with the SJI maximum spacing limitation of 11'-9". The bridging indicated in the shop drawings and the bridging layout observed at the site have typical spacings between points of bottom chord bridging which are in excess of the spacing limitations of the SJI Specifications (12'-9" and 12'-8" vs. 11'-9"). 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 with Section 5.4 of the SJI Standard Specifications because the bridging spacings are not approximately uniform. During site visits, the bridging spacing at a typical original joist was measured to be 12'-8" at one side of the X-bridging and 6'-8" on the other side. Thus, the bridging spacings vary by up to 90% along a single joist.

As noted above, failures of joist end web members were observed. Under normal gravity load, these members 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. 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 (defined as 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 when a joist is to resist a net uplift, all diagonal members, bottom chord and top chord members shall 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 this 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 Technical Digest No. 6, uses the tension member criteria of 240 for a limiting slenderness ratio of an end web member. Additionally, as indicated above, the SJI Technical Digest also uses an effective length factor of K = 0.8 for the calculation of allowable compressive stress in the member. As noted, 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 an ambiguity between the SJI Specifcations and SJI Technical Digest No. 6.

ANALYSIS

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

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 L_{yy} (allowable bridging spacing) of 14'-2" for the stress in the bottom chord. This allowable bridging spacing was calculated using the 1/3 stress increase in the allowable bottom chord stress and 10 psf net uplift loading as 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 capacity of the joists, considering the failure mode at the bottom chord, is presented in Table 3. The capacity is 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 1/3 stress increase per AISC, and the Euler buckling load (without the buckling safety factor). The bottom chord capacity was determined for a range of bridging spacings.

Table 3. Bottom Chord Net Uplift Capacities (psf).

Bridging Spacing	Allowable Capacity Pressure – No Stress Increase	Allowable Capacity Pressure – 1/3 Stress Increase	Euler Buckling Capacity Pressure
11'-9" [1]	10.91 psf	14.55 psf	20.92 psf

12-6"	9.64 psf	12.86 psf	18.48 psf
12'-8"	9.39 psf	12.51 psf	17.99 psf
12'-9" [2]	9.27 psf	12.36 psf	17.76 psf
13'-0"	8.92 psf	11.89 psf	17.09 psf
13'-2"	8.69 psf	11.59 psf	16.66 psf
14'	7.69 psf	10.25 psf	14.73 psf
14'-2" [3]	7.51 psf	10.01 psf	14.39 psf
14'-6"	7.17 psf	9.55 psf	13.73 psf

Notes: 1) Maximum allowable bottom chord bridging spacing per SJI.
2) Maximum measured bottom chord bridging spacing at the site.
3) Maximum allowable bottom chord bridging spacing per design.

Table 3 indicates the increase in bottom chord capacity as the bridging spacing decreases. The net uplift pressure which should have been used for the roof design was 17.51 psf, which exceeds all of the allowable values listed in Table 3. Also, this proper net uplift exceeds the capacity (no factor of safety) of the joists if they had a bridging spacing of 14'-2", 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 conformance to 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 increased 18% if the bridging layout had conformed with the SJI Specifications (20.92 psf for 11'-9" spacing vs. 17.76 psf for 12'-9" 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 mis-calculated design load and the failure to comply with SJI standards for the spacing of the bottom chord bridging. Of course, the use of a non-conservative design load may result in failure, irrespective of the SJI standards. However, designing a bridging layout which complies with the SJI standard can only increase the capacity.

CHANGES TO SJI SPECIFICATIONS

In the most recent standard specifications for K-series joists, dated 2003 and effective 2005, SJI has made changes, including clarification of top chord bridging and bottom chord bridging requirements. As part of this clarification, SJI 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 be required such that the bottom chord complies with the slenderness requirements of SJI and any specified strength requirements. The language regarding bridging has been further clarified to distinguish between the bottom chord and top chord bridging, noting that they may be spaced independently.

It is the opinion of the authors that these changes implemented by 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 due to the use of Kl/r in the Technical Digest as noted above. The ambiguous "approximately uniform" spacing requirement has been removed from the 2003 SJI specifications for K-series joists.

CONCLUSIONS

The net uplift design load for the joists was inadequate and the design load should have been about 75% higher. As noted in this report, the roof systems selected for the original construction were uniquely light. This should have been considered in the design process regarding 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 is the result of misunderstanding of the SJI Specifications for the joists. Ambiguity in the SJI Specifications, such as calling for "approximately uniform" spacing of the bridging and a failure to explicitly state that the bottom chord bridging is also subject to maximum spacing requirements has contributed to the misunderstanding.

Additionally, there is ambiguity between the SJI specifications and the SJI Technical Digest 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. Specific changes include clarification of top chord bridging and bottom chord bridging requirements. It is the opinion of the authors that these changes by SJI are helpful in clarifying the top and bottom chord bridging requirements, although some ambiguity remains.

In the case of the subject warehouse, the failure to comply with SJI 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 or lessened this failure. The actual capacity of the joists would have increased 18% if the bridging layout had conformed with the SJI specifications. This illustrates the role of Factors of Safety and minimum standards (such as those by SJI) in the arena of public safety.

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