Case Study: Failure at an Abutting Lower Roof due to Snowdrift

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ABSTRACT

Roof failures can occur at abutting lower roof structures which are susceptible to snowdrift loads. Snowdrift accumulation at abutting lower roofs is dependent on several factors including snow density, quantity of driftable snow, geometry of the lower and upper roofs, and wind directionality. Presented herein is a case study involving the partial roof collapse of an abutting lower roof structure as a result of snowdrift accumulation on the lower roof framing. While the authors' forensic investigation of the partial roof collapse led to the discovery of design and construction deficiencies in the lower roof framing, the authors determined that the code-prescribed design snowdrift height was less than half of the actual observed/measured height of snowdrift on the lower roof. These findings have led the authors to questions regarding the safety and adequacy of the current code-prescribed design loads as they relate to snowdrift at lower roof structures with similar roof geometries and ground snow loads to those presented in this case study.

INTRODUCTION

On February 1, 2011, the abutting lower roof of a pre-engineered steel building (subject structure) in Oklahoma City, Oklahoma, partially collapsed as a result of snow drift accumulation. The original construction drawings were dated August, 2009. The building was approximately 60,000 square feet with a total of four (4) abutting lower roof structures attached to the perimeter of the facility to separately house mechanical and storage equipment used for production. **Figure 1** below shows a schematic roof plan of the structure. The roof framing for the structure consisted of steel bent frames and cold-formed Z-purlins.

The roof covering over the upper roof system consisted of a gable style roof with a pitch of 1:12. The abutting lower roof structures consisted of lean-to framing and monoslope roofing systems which sloped at a pitch of 1:12. The building design was symmetrical along the ridge of the upper roof structure as shown in **Figure 1**. Of the four abutting lower roof structures, two were located at internal corners of the upper, gable roof structure (**Figures 1 and 2**).

According to the National Oceanic and Atmospheric Administration's (NOAA) National Climatic Data Center (NCDC), the Will Rogers World Airport (KOKC), located in Oklahoma City, reported a snow depth of 12 inches as a result of the February 1, 2011 storm event (hereto referred to as "storm event" or "snow event"). In addition to snow accumulation, weather data from NOAA's National Weather Service (NWS) also indicates that the maximum wind speed recorded at KOKC predominantly came from the north at 41 miles per hour with maximum gust speeds up to 53 miles per hour.

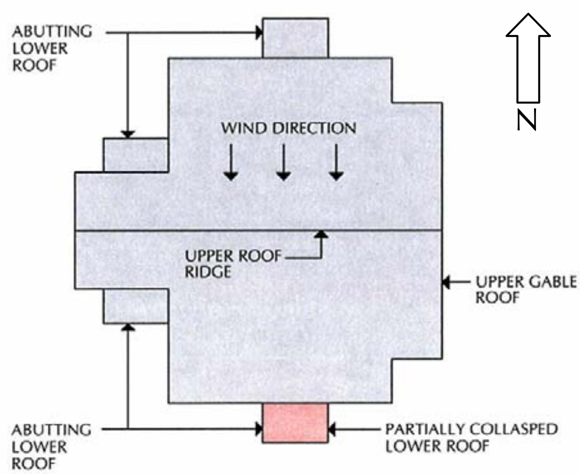

Figure 1: Schematic of subject structure

Only the southern, abutting lower roof structure experienced a partial roof collapse during the storm event (**Figure 1**). Photographs of the total snowdrift height at the southern lower roof structure were taken the morning after the storm event (**Figures 3 and 4**). As a result of the snowdrift, the southern lower roof deck was sagging throughout and had pulled away from the upper roof structure (**Figure 5**). The distance between the elevation of the lower and upper roof at this location was 12'.

The upper crest of the snowdrift measured approximately 10' from the projected surface of the lower roof (sagging in the lower roof deck would result in additional drift height, which was not included in the measured 10' drift height). The snowdrift accumulation at the southern lower roof resulted in several failed secondary framing members (Z-purlins), excessive deflection of the metal roof deck/framing, and damage to a portion of the metal roof deck (**Figure 6**). The authors observed that bolts were missing at several of the roof purlins at the purlin splice connections (**Figure 7**).

Figure 2: Southwest lower roof at internal corner of upper roof structure

Figure 3: Snowdrift geometry/height at southern lower roof

Figure 4: Snowdrift geometry/height at southern lower roof

Figure 5: Roof deck pulled away from upper roof structure

Figure 6: Failed Z-purlins

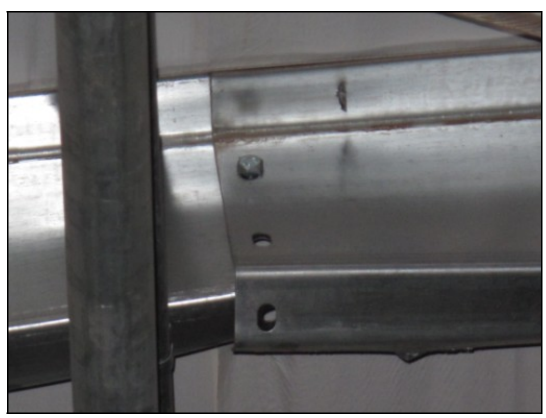

Figure 7: Missing bolt at splice

FORENSIC STRUCTURAL ANALYSIS

The Oklahoma City Department of Public Works (OCDPW) was contacted to identify the applicable building code for Oklahoma City at the time of the original design of the structure. According to the OCDPW, the 2003 International Building Code (IBC) was adopted by Oklahoma City on September 28, 2004 and was the adopted code during the original design of the structure. However, based on the original design plans, the 2006 IBC was utilized by the Engineer of Record in the original design.

After the partial roof collapse, the authors analyzed the as-designed roof framing for the southern lower roof structure per the adopted, applicable building code at the time of original design (2003 IBC). The intent of this analysis was to determine if these partially failed roof framing members were originally designed using minimum, code-prescribed loads. In order to evaluate the adequacy of structural elements as they relate to strength requirements, section property information for the light-gauge steel members was obtained from the manufacturer.

The authors' load calculations revealed that the purlin design at the partially collapsed lower roof structure was marginally inadequate for the required design snow loads per the 2003 IBC. Structural analysis revealed that the failed purlins were less than 10% overstressed for code-required vertical loads and the deflection of select purlins exceeded the allowable code limits for serviceability. The as-built conditions noted in the field indicated construction deficiencies, including missing bolts at Z-purlin to Z-purlin lap connections, which would further reduce the load carrying capacity of the as-built members.

Although design and construction deficiencies were identified at the failed framing members at the southern lower roof, further analysis of the failed structural members revealed that the code-predicted design snowdrift height was significantly lower than the observed height of snowdrift after the storm event. Such findings introduced questions about the adequacy of the current design code for the design of lower roof framing similar to that at the subject structure.

The current version of the IBC at the time of this writing is the 2012 IBC. The 2012 IBC references the 2010 American Society of Civil Engineers (ASCE 7-10) *Minimum Design Loads for Buildings and Other Structures* to determine load design criteria. Although this edition of the code was not utilized in the original design, the authors chose to use this edition of the code for this case study to compare current code-prescribed design loads to the loads that the structure actually experienced during the storm event.

VERTICAL LOAD ANALYSIS OF SOUTHERN LOWER ROOF

Using ASCE 7-10 in conjunction with the 2012 IBC, the design loads and loading conditions for the failed roof members at the southern lower roof structure were developed. Based on the authors' analysis, the controlling load combination for the failed roof members at the southern lower roof consisted of a combination of dead load, balanced snow load, snowdrift, and where applicable, partial loading.

Although dead load is included in the vertical load analysis, it is the focus of this case study to compare the measured height of snow that the failed roof members actually experienced to the design height of snow prescribed by current code. In addition to these controlling loads, each of the following scenarios were investigated as potential sources of vertical loads to the southern lower roof structure (unbalanced snow loads do not apply to monosloped lower roofs).

Roof Live Load

Roof live loads are provided to factor in non-occupancy related loads experienced by the roof framing during the life of the structure. They may include loads produced by moveable decorative appurtenances and/or maintenance workers, equipment, and materials (ASCE 7-10). An unreduced roof live load of 20 pounds per square foot (psf) was used as the minimum code-required roof live load for the southern lower roof structure.

Sliding Snow

Sliding snow occurs when accumulated snow at the upper roof breaks off and slides en-masse onto the lower roof structure resulting in additional loads on the lower level roof framing. The surcharge and distribution of the sliding snow loads is dependent on the slope, size and orientation of each roof system.

ASCE 7-10 states "sliding loads shall be superimposed on the balanced snow load and need not be used in combination with drift, unbalanced, partial, or rain-on-snow loads." Based on the analysis for this case study, snowdrift loads govern over sliding snow loads at the failed structural members.

Rain-on-Snow Surcharge Load

Rain can fall and accumulate on the lower roof of a structure while the roof is still covered with snow. In some cases, this additional rain load may not have been accounted for in the 50-year ground snow design load; therefore, the rain-on-snow provision adds an increased load to the balanced roof load case to account for the effects of rain (O'Rourke, 2010). For the parameters of this case study, the roof slope (in degrees) is greater than the code-prescribed maximum limit of W/50 (where W is the horizontal distance from eave to ridge, in feet); therefore, rain-on-snow loads need not be considered.

Minimum Snow Loads for Low-Sloped Roofs

In low ground snow load areas, the region's design ground snow load could come from a single storm event. Therefore, it is possible that a single storm could result in both the ground and roof having the same loads approaching the 50-year design snow load (O'Rourke, 2010). For these areas ASCE 7-10 requires consideration of a minimum load for monoslope roofs with slopes less than 15 degrees, like that of the southern lower roof structure in this case study. ASCE 7-10 states that this minimum roof snow load is a separate uniform case and "need not be used in determining or in combination with drift, sliding, unbalanced, or partial loads." For the southern lower roof structure, the minimum roof snow load does not govern in the snow load design.

Partial Loading

Partial loading should be considered for lower roofs with continuous structural members where a reduction in snow loading on one span may result in an increase in stress and/or deflection in an adjacent span (ASCE 7-10). ASCE 7-10 provides loading cases to consider when determining the effect of partial loading on continuous beam and other structural systems.

The roof framing at the southern lower roof consisted of continuous purlins extending across two spans. Therefore, the authors considered partial loading for each failed member where applicable by code.

GOVERNING SNOW LOADS

Balanced Snow Load

The balanced snow load is the basis for determining the snow load for all structures and is dependent on several factors, primarily the region-specific ground snow load. The balanced snow load is distributed uniformly on the lower roof and is assumed to act on the horizontal projection of the roof surface (ASCE 7-10). Based on ASCE 7-10 (Figure 7-1) the ground snow load for the location of the subject structure is 10 psf.

ASCE 7-10 provides equations to calculate the design flat roof snow load, p_f, and a roof slope factor to convert the flat roof snow load into a sloped roof snow load, p_s. The design height of the balanced, sloped roof snow load (h_b) is evaluated by dividing the sloped roof snow load by the snow density, or unit weight of the snowpack.

Drifts on Lower Roofs

Snowdrift accumulation at a lower roof occurs in the wind shadow, or aerodynamic shade, created by the elevation difference between the upper roof of a structure and the lower roof of an abutting structure. This elevation change is referred to as the roof step. During one or multiple snow fall events, snowdrift accumulation can result in large loads affecting lower roof framing.

The surcharge and distribution of snowdrift at lower roof structures changes depending on whether the abutting lower roof structure is located on the leeward or windward side of the upper roof. Leeward snowdrift naturally accumulates in a triangular geometry, which, based on an empirical database, ASCE 7-10 estimates as a distribution of one vertical to four horizontal (O'Rourke, 1985). The distribution pattern of windward snowdrift is more complex and is dependent on the height of the windward roof step (O'Rourke, 2010).

The windward and leeward drift heights for the southern lower roof structure were calculated separately, and the larger value was utilized to establish the design drift load for the roof framing (ASCE 7-10). The drift height, h_d, is related to the length of roof upwind of the drift, or the fetch, l_u. To determine the windward drift height, the length of the lower roof is used as l_u, whereas for leeward drift height the upwind fetch is defined as the length of the upper roof.

DETERMINING THE DESIGN DRIFT HEIGHT

According to *Drift Snow Loads on Multilevel Roofs*, published in the Journal of Structural Engineering (O'Rourke, 1985), approximately 350 case histories involving snowdrift were analyzed to produce an empirically-based relationship for the surcharge drift height. The resulting equation was then multiplied by a modification factor (less than 1) to allow the design engineer to use the 50-yr ground snow load as an input parameter in the ASCE 7 design code. Based on engineering judgment, the ASCE 7 Snow Task Committee selected a modification factor of 0.7. As such, the predicted design drift exceeded the observed drift for about two-thirds of the case histories (O'Rourke, 1985).

Although ASCE 7-10 provides provisions for snowdrift at lower roofs, it does not provide considerations or guidelines for calculating the leeward drift surcharge at a lower roof structure for varying styles and slopes of the abutting upper roof system (including the common gable or hip style roof).

In this case study, the leeward fetch distance for the lower roof drift is not necessarily limited to the distance between the eave and the ridge of the upper roof slope directly over the lower roof. Depending on the pitch of the upper roof, both roof slopes may contribute to the total drift surcharge at the lower roof and should be considered for the lower roof design load. Alternatively, it may be prudent to assume that the full eave-to-eave length of the upper roof would contribute to the leeward snowdrift surcharge.

The typical gable roof scenario is also presented in Chapter 13 of the *Guide to the Snow Load Provisions of ASCE 7-10* (O'Rourke, 2010). In this publication, O'Rourke suggests that the leeward upwind fetch for the total lower roof drift is generally calculated using the equation $l_u=l_b+0.75l_a$, where l_u is the total upper roof length, l_b is the length of the upper roof slope directly over the lower roof, and l_a is the length of the upper roof slope opposite the lower roof as shown in **Figure 8** below.

Although, this equation does not directly account for varying slopes of the upper roof gable, it includes a general contribution from both roof slopes and is an available, published resource for current design engineers as a supplement to the code. Therefore, this equation was used in the analysis of this case study to determine the appropriate leeward roof length of the southern lower roof.

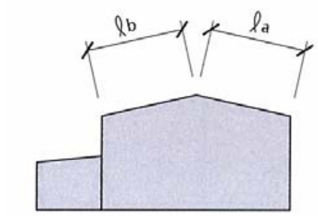

Figure 8: Gable style roof with abutting lower roof structure (profile view)

The eave-to-eave length of the upper roof above the southern lower roof (perpendicular to the roof ridge) measured 264 feet. The eave-to-ridge distance was 132 feet, therefore, using O'Rourke's equation, the leeward upper roof length for the southern lower roof was 231 feet.

Although not discussed in O'Rourke 2010, it should be noted that determination of the most conservative direction of possible wind contribution would involve analyzing wind directions other than those acting orthogonal to the building perimeter. Depending on the location of the lower roof structure and the geometry and pitch of the upper roof, a diagonal distance, representative of the most conservative upwind fetch distance affecting the lower roof, should be considered for the lower roof design. Non-orthogonal wind directions were not considered for this analysis so that the final comparison is based on published resources available for current design engineers.

SNOW DENSITY

Weather data from NOAA's National Operational Hydrologic Remote Sensing Center (NOHRSC) reported that the density of the fallen snowpack during the storm event was equal to 6.24 pcf (10% of the density of water). Stated differently, 10 inches of freshly fallen snow corresponds to 1 inch of water. NOHRSC's density measurements are derived from dividing the observed snow water equivalent by the observed snow depth. The snow water equivalent is calculated by taking a vertical core of the freshly fallen snowpack on the ground, weighing it, and converting that weight to a height using the density of water. Because NOHRSC's density data is limited to *ground snow accumulation,* these measurements do not properly reflect snowpack densities on roofs, which are subject to additional parameters including varying roof temperatures and snowpack from drift.

For the 350 case histories in the database used by O'Rourke regarding empirical snowdrift relationships, drift density data was available for 169 of these cases. Based on the available drift measurements, O'Rourke determined that the "commonly used rule-of-thumb" that 10 inches of freshly fallen snow corresponds to 1 inch of water is "unconservative for drifts". Rather, the study revealed that the mean density for snowdrift at lower roofs was 15.6 pcf (2.5 times higher than the "rule-of-thumb" value). O'Rourke concluded that for each case where the total drift loads were greater than or equal to 30 psf, the average density was 17.4 ± 4.9 pcf; therefore, in order to model design loads, a drift density of 17.4 pcf is recommended (O'Rourke, 1985).

ASCE 7-10 provides the equation, $\gamma=0.13p_g+14$ for snow density which represents the unit weight of snow as a function of the site-specific ground snow load. Based on this empirical relationship and a code-prescribed ground snow load of 10 psf (ASCE 7-10), the design snow density in the vicinity of the subject structure is 15.3 pcf.

Although O'Rourke recommends using a snow density of 17.4 pcf to model snowdrift design loads, the site-specific, current code-prescribed, design snow density for the subject structure is 15.3 pcf. Therefore, for the purpose of this paper, 15.3 pcf was utilized as the design snow density.

COMPARING THE DESIGN AND OBSERVED DRIFT HEIGHT

Using ASCE 7-10 (Figure 7-9), the leeward and windward design drift height for the southern lower roof are conservatively 4 feet and 1.125 feet, respectively (windward drift height is three-quarters of h_d as determined by Figure 7-9). Therefore, the controlling snowdrift is from the leeward direction.

The design height of the balanced, sloped roof snow load is determined by dividing the balanced, sloped roof snow load by the design density of the snowpack (ASCE 7-10). Using ASCE 7-10, the balanced, sloped roof snow load is 7 psf, after the exposure, thermal, importance, and slope factors have been applied. Therefore, the design height of the balanced, sloped roof snow load is conservatively 6 inches.

By combining the height of snow drift and height of balanced snow for the design case, a total depth of 4.5' represents a conservative, code-predicted total depth of snow load on the southern lower roof. In other words, if the structure was being designed according to ASCE 7-10, a maximum design height of 4.5 feet may be used to determine the loading of the lower roof framing members (note that the drift height decreases linearly as it moves from the leeward step).

However, the maximum observed height of snowpack from the storm event on the southern lower roof was measured as approximately 10 feet from the lower roof surface (possibly greater if roof deck deflection is taken into account), which is approximately 2.2 times higher than the predicted design snow height. Therefore, based on the site-specific design snow density (15.3 pcf as presented in the previous section), the total snow surcharge on the southern lower roof framing could have been up to approximately 2.2 times higher than the minimum, code-required design snow loads.

CONCLUSIONS

As-designed analysis of the failed roof members for the southern lower roof structure revealed that the purlin design was marginally inadequate for the required design snow loads per the 2003 IBC (adopted building code at time of original design). Although design and construction deficiencies were identified at the failed framing members, both of which largely contributed to the roof collapse, further analysis of the failed structural members revealed that the code-predicted design snowdrift height was significantly lower than the observed height of snowdrift after the storm event. This condition would likely result in unconservative design loads. Such findings introduced questions about the adequacy of the current design code for the design of lower roof framing similar to that at the subject structure.

Based on the 2012 IBC and ASCE 7-10, a maximum design height of 4.5 feet may be used to determine the loading of the roof framing at the southern lower roof structure. Therefore, based on the observed drift height of approximately 10 feet and the code-prescribed density of the snowpack (per ASCE 7-10), the total surcharge on the southern lower roof framing could have been up to approximately 2.2 times higher than the minimum, code-prescribed design snow loads.

Based on these findings, the authors propose that the current snow design code methodology be reviewed to determine its applicability to lower roof structures in areas with low ground snow load, similar to that described herein. Furthermore, design engineers should be aware that the current code-prescribed design loads are not conservative for lower roof structures with similar roof geometries and ground snow loads to those presented in this case study.

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