

Effect of construction errors on the risks of snow-induced failure of roofs supported by open web steel joists

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ABSTRACT: The cause of the structural failures due to snow loads on lightweight steel roofs range from being overloaded due to drifted or extremely dense snow, to errors made in construction and design. Open web steel joist structures, which have a low dead load to snow load ratio, may be particularly vulnerable to snow-induced roof failure. This paper investigates the effects of construction errors on the reliability of these types of structures by describing common types of construction errors observed in properly designed OWSJ supported lightweight steel roofs, and quantifying the impact of construction errors such as weakened welds in the joists, joists being installed in incorrect positions, and joists being out-of-plumb on the overall structure's risk of ultimate or serviceability failure under extreme snow loads.

1 INTRODUCTION

The structural collapse of a roof due to snow loads can lead to costly repairs, injuries, and even death of building occupants. Bolduc (2011) showed that over 40% of the structural failures occurring in North America during 2010 were associated with roofs collapsed by snow. Metal or steel roof systems may be particularly vulnerable to snow-induced collapse. Geis *et al.* (2012) examined patterns in snow-induced building failures between 1979 and 2009, showing that the 1,029 incidents identified in the United States caused nineteen fatalities, 146 injuries, and up to \$200 million in building damages. Moreover, 53% of the incidents involved metal or steel constructed buildings. The study also reported that warehouses and factories were among the most affected building types. Numerous collapses of steel roofs have also been recorded in the northeastern United States during the winters of 1993-94 and 1995-96 (Peraza 2000), and during the January 1996 storm (DeGaetano *et al.* 1997).

Several studies have identified a common trend indicating the vulnerability of open web steel joist and truss supported roofs. Case studies show that a major contributor to snow-induced open web steel joist and truss supported roof failures are errors in the design, fabrication, or erection of the structure (Holicky & Sykora 2009). The general term "construction error" will be used in this paper to describe fabrication and construction errors; design errors are not directly considered here. The effects of these construction errors on open web steel joist and truss

supported roofs has caused collapses of structures such as schools (Tanzer 2011), as well as other heavily occupied buildings such as the Hartford Civic Center Coliseum (Martin & Delatte 2001), the Katowice Fair Building (Biegus & Rykaluk 2008), and department stores (Lavon & Stivaros 2005) under snow loads.

This paper investigates some of these construction errors and the behavior of an open web steel joist (OWSJ) supported roof structure subjected to snow loads when construction errors are present. Comparisons will be made between (1) properly designed and constructed OWSJ structures and (2) OWSJ structures with construction errors, in order to quantify the risk of these construction errors under snow loads present in Denver, Colorado.

2 OPEN WEB STEEL JOISTS

2.1 *Uses of OWSJs*

Open web steel joists are prefabricated structural elements that utilize the truss action of web and chord elements, such that the composite OWSJ reacts as a single flexural element to support a roof or floor system. OWSJs are prefabricated. All fabricators working with the Steel Joist Institute (SJI) must specify OWSJs that conform to SJI load tables. These tables state the allowable uniformly distributed load which can be safely resisted by a particular joist type with a defined span length. The SJI load tables are meant for the design of uniformly applied gravity loads,

but manufacturers can also provide joists designed to resist non-uniform loads. The SJI offers load tables for many different joist types, the three most common being: K-Series for medium to long spans and a uniform load per foot of less than 550 lbs/ft., LH-Series for longer spans and a uniform load per foot greater than 550 lbs/ft., and DLH-Series for deep, long span joists (Fisher *et al.* 2002, Yost *et al.* 2004). Joists are denoted by specific numbers which indicate the joist depth, joist series, and relative strength. For example, 18K10 refers to a K-series joist with a depth of 18 in. and a relative strength of 10 (implying the joist is stronger than joists with smaller relative strength values).

OWSJs are often preferred over conventional wide flange sections when designing and constructing long span roofs due to their economy, and the simplicity of the design process. OWSJs use less steel in their truss-like web system as compared to the solid steel wide flange section with the same load resistance characteristics. This high bending resistance to weight ratio makes OWSJs an efficient and desirable structural member in steel building construction and design (Fisher *et al.* 2002, Yost *et al.* 2004, Buckley *et al.* 2008). In addition, the load table-based design process can reduce design time and make the structure more cost efficient by facilitating selection of the joist that resists the required load in the most efficient manner (Fisher *et al.* 2002).

OWSJs' desirability is made apparent by the wide variety of steel buildings for which these structural members are popular. These structures include many buildings which can accommodate large human occupancies or expensive merchandise such as warehouses, industrial plants, offices, commercial shops and malls, schools and other academic facilities, civic and institutional structures, and recreational facilities (Fisher *et al.* 2002). The combination of the large capacity of these buildings with regards to both people and merchandise, along with their susceptibility to snow loads, makes understanding the behavior of these structures critical.

2.2 Components of OWSJs

There are four main components of OWSJs: top chord, bottom chord, web members, and end web members, as depicted in Fig. 1. The top chord is typically under compression when gravity loads are applied and consists of back-to-back angle sections, spaced so that web members can fit between them, and are continuous over the span length of the joist. The top chord is typically continuously laterally braced by the roof or floor decking that the joist supports. The bottom chord is fabricated from two back-to-back angle sections with spacing for web members (Buckley *et al.* 2008). The bottom chord is in tension when loaded and is only laterally braced

by angle sections acting as bridging between the joists at certain points. The location of bridging is determined through tables provided by the SJI (Fisher *et al.* 2002, Buckley *et al.* 2008).

Chord members intersect with the diagonally-oriented web members at "panel points". The web members and end web members may be in tension or compression depending on their orientation and position along the joist. For K-Series joists, the end web members are generally rods, while the interior web members can be fabricated as rods, single angles, or crimped single angles (Buckley *et al.* 2008). When the web members are angle sections, the webs are non-continuous, meaning separate web elements are used between each panel point. However, rod web members are typically composed of one long rod that is bent at every panel point (CANAM 2005). If the web members are angles, they must be crimped at the ends for connection with chord members. This crimping causes the centroid of the member to be eccentric to the line of loading, creating a moment in the section (Buckley *et al.* 2008).

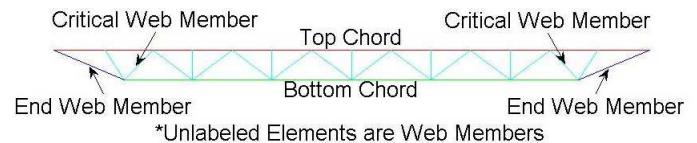


Figure 1: Arrangement of the Components Within an OWSJ

Two types of connections are commonly found in OWSJs. Welds are used to connect the web members and bridging to the chords while bolts are generally used to connect the top chords to the girders that typically support the joists (SJI 2005).

OWSJs are prefabricated by a number of different companies. Owing to the proprietary nature of this process, most of these companies keep design details confidential.

2.3 OWSJ Design for Denver, CO

The structure used in this study is a 90 ft. by 120 ft. single story building which has 3 bays of 40 ft. span wide flange section girders in one direction, as shown in Fig. 2. The roof is supported by 30-ft. K-series OWSJs in the orthogonal direction, which are spaced at 10 ft. intervals along the girders. The roof is 30 ft. high with HSS 6X6X3/8 columns. The structure is designed according to ASCE 7-10.

The following loads were considered to design the roof for Denver, CO: (1) a roof live load of 20 psf; (2) a roof dead load of 9 psf, based on data from Boise Cascade (2009) for a metal deck covered in composition roofing; (3) the self-weight of the joist from the SJI's load tables (2005); and (4) a ground snow load of 25 psf, from the City of Denver's Amendments to the Building Code (2011). Using ASCE 7-10 standards for determining roof loads

from ground snow load, the design roof snow load is 20 psf. These calculations are based on a structure that is partially exposed (exposure category B), risk category II, and thermal characteristics of a building not kept intentionally cold. The design was performed by uniformly applying the loads on the entire roof.

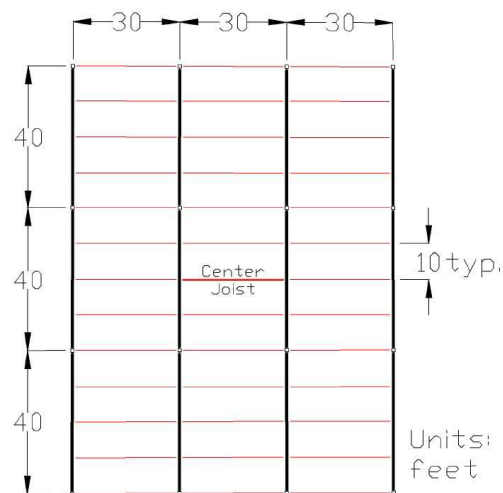


Figure 2: Plan View of the Structure showing the location of OWSJs (red) and Girders (black)

Using LRFD design and the SJI's load tables (2005), two joists of different depths were found to satisfy the design conditions:

- 18K10, which has a depth of 18 in., a self-weight of 11.7 lbs/ft., and a safe factored uniformly distributed load of 715 lbs/ft..
- 28K6, which has a depth of 26 in., a self-weight of 11.4 lbs/ft., and a safe factored uniformly distributed load of 715 lbs/ft..

The designed girder sections are W18X65. Two rows of L2X2X1/8 sections are used to satisfy SJI (2005) horizontal bridging requirements.

The sizes of chord and web elements and configuration of these elements for both of these joists was determined through a contact with a well-known OWSJ manufacturing company. In order to maintain confidentiality, neither the component sizes nor the name of the company is included in this paper.

3 CONSTRUCTION ERRORS

3.1 Possible Construction Errors

The authors collected and reviewed publications in conference proceedings and academic journals on roof failures, as well as a wide variety of roof failure case studies. A large number of the causes for failure of the roofs involved an error in the design, fabrication, or erection of the structure and its components. The identified construction errors can be generally categorized into six types: improper erection, mis-

placement of members, improper manufacturing of elements, welding errors, handling and construction damage, and deficient materials.

Improper erection of the structure is a major concern due to it resulting in a significant deviation from the appropriate design for the structure. These deviations could include the erection of a structure with incorrect spacing between joists, or installing the joists hanging slightly out-of-plumb (Green *et al.* 2007). Both of these errors create unanticipated stresses in the joist members that were not considered in the design. Other erection errors can include cutting or altering the joists in contradiction to recommendations by the SJI to leave joists unaltered (Green *et al.* 2007). In other cases, the error may be due to simply not following the construction and erection drawings, for example, leading to installing a member backwards (Geis 2012) or neglecting to install elements such as bridging in the structure (Biegus & Rykaluk 2008).

The misplacement of members occurs if joists are inadvertently swapped in the construction process. Structural failure may occur when a weaker member is substituted into a position where it is not capable of carrying the design loads. This initial failure can then lead to progressive collapse of the entire roof (Lavon & Stivaros 2005, Geis 2012). Misplacement of members can be avoided by reviewing construction drawings and comparing them with the erected structure and appropriately tagging joists (Ratay 2000). However, should the tags fall off or are placed on improper joists, many of the joists look similar to the naked eye. In fact, members used in different joists are generally very similar and may only vary in thickness by dimension on the order of 1/25th of an inch. The probability of a joist losing its tag or joists being placed in the wrong locations occurring is low, but it has occurred in the past (Geis 2012).

Improper manufacturing of elements involves the use of prefabricated or built-up sections which do not necessarily violate the construction or erection drawings, but which incurred errors during the fabrication process. In these types of errors, wrong types or sizes of components are used due to misunderstandings between designers and fabricators (Lavon & Stivaros 2005). These errors often leave the fabricator's facility unnoticed and are erected in the structure due to erectors assuming the fabricators' work is correct (Martin & Delatte 2001, Small & Swanson 2006).

The most common source of structural failure in properly designed structures concerns the connections between all elements (Ratay 2000). In particular, welds may be faulty and have irregularities or discontinuities such as insufficient weld length, porosity in the weld, cracks in the weld, lack of penetration, or lack of fusion with the base metal (Feld 1968, Ratay 2000, Martin & Delatte 2001, Velhulst

et al. 2009, Tanzer 2011). These errors are difficult to detect visually. This type of error can occur in any structural system, but the large number of welds involved in OWSJs, trusses, and space trusses along with the common use of these systems, has led to major failures in the past (Martin & Delatte 2001, Tanzer 2011).

Handling and construction damage may be encountered if OWSJs are handled or erected by methods not recommended by the SJI. Handling damage can result in the bending of components or in the crumpling of chord or web members where straps are attached in order to lift and move the joists. Construction damage varies from small dents due to a machine or element contacting another element, to components such as the chords being accidentally cut while drilling is taking place nearby. These errors are generally detectable by a visual inspection, as recommended by the SJI, before the joists are erected (Green 2007, Green *et al.* 2007).

The final construction error identified is the use of a deficient material in construction, which could have a variety of consequences. For OWSJs, the most relevant material deficiencies are insufficient yield and ultimate strength in steel (Tanzer 2011) and base steel that is inappropriate for welding and therefore contributing to weak welds (Feld 1968).

Many of these errors can be avoided by inspecting the structure prior to every step of the construction, in order to verify that the design is followed and that there is no visible damage to the elements and OWSJs. The SJI recommends, but does not require an inspector on site (Martin & Delatte 2001, Green 2007, Green *et al.* 2007, Holicky & Sykora 2009).

3.2 Construction Errors Considered in This Study

This study analyzes three of these types of construction errors. First, because the variation in as-built weld properties can be vast and this variation has such a large effect on the structural performance of a joist, weld deficiencies are analyzed. Second, the misplacement of a joist can have a significant impact on the structure, so analyses included a single joist in the structure that differed from the rest of the joists. The third and final construction error investigated concerns the improper erection of the structure, represented by a single joist that is slightly out-of-plumb.

4 NUMERICAL MODEL OF OWSJS

4.1 Properties of Nonlinear Models

Nonlinear models of the buildings shown in Fig. 2 were created in the software *OpenSees*. The developed model is capable of representing the major

failure mechanisms identified through literature review of case studies of building failures and other studies of OWSJs. Material nonlinearities were implemented in the model for the chords and web members of the joists through the use of nonlinear beam-column (fiber) elements. Steel material nonlinear properties for the grade 50 steel are similar to those proposed by Dodd & Restrepo-Posada (1995), which take into account steel properties beyond the yield point by defining a yield plateau region followed by a strain-hardening region. The Dodd-Restrepo model is based on experimental results and considers any strain past the strain at ultimate stress to be a failure of the material. In order to implement this material in *OpenSees*, the post-peak behavior was taken as a linear reduction from the ultimate stress to zero to represent fiber fracture. The geometry of the nonlinear elements is captured through the use of fiber sections that represent the shape of chord members and end web rods along their entire length. In the case of crimped angle web sections, meaning the legs of the angle are bent inwards in order to fit between the chord members, the model represents the crimped shape near the panel points and the uncrimped shape in the middle of the member. The columns, girders, and bridging are modeled as elastic beam-columns.

Other failure mechanisms considered in the model are buckling of the end and interior web members (Yost *et al.* 2004). The buckling of the web members is accounted for using equations from the AISC Steel Construction Manual (2005) to calculate the strains at which local torsional and flexural-torsional buckling occurs in angle and rod sections. These calculations assume a semi-rigid effective length factor of 0.70, which is supported by comparison of models and experimental studies. These buckling strains are specific to each web member and are used to modify the steel material used for the individual web member's fibers so that the material strength decreases after the buckling strain is reached following recommendations by Elnashai & Elghazouli (1993), in lieu of continuing on the path defined by the Dodd-Restrepo model. The bottom and top chords are assumed not to buckle because the former is in tension and the latter is braced by the presence of the roof system.

The failures of welded or bolted connections (Buckley *et al.* 2008) are accounted for through the use of zero-length elements, which are modeled at every connection between elements. Bolted connections are located where the chords of the joists meet the girders. The SJI requires that K-Series joist ends be connected by two ½" A307 bolts (SJI 2005). The shear and tensile properties for the connection details were determined and applied as force-displacement and moment-rotation curves in the appropriate directions in the zero length springs. Welds constructed using E70XX welding electrodes are as-

sumed to connect the grade 50 steel web members with the chords. The Lesik & Kennedy (1990) weld material model is employed to determine the force-displacement properties of the welds. The moment-rotation properties were based on the weld lengths required by the connection geometry and the elastic method as recommended by the AISC (2005). These models represent nonlinear behavior at the welded and bolted joints. Failure displacements and rotations are also calculated and implemented within the spring properties to represent weld fracture.

4.2 Implementation of Construction Errors

The modeled building was modified to accommodate the three construction errors described above. Table 1 lists the structures analyzed in this study. All three errors under investigation are applied to only the center joist in the structure (Fig. 2), while the other joists are modeled without construction errors.

Table 1: List of buildings under investigation designed for Denver, CO (25 psf of design ground snow load)

Bldg. No.	As-designed OWSJ	Error in center joist
1	18K10	No Error
2	18K10	Weld Length Reduced 65%
3	18K10	Misplaced Joist: 18K7
4	18K10	Out-of-Plumb 1"
5	18K10	Out-of-Plumb 2"
6	28K6	No Error
7	28K6	Weld Length Reduced 35%
8	28K6	Misplaced Joist: 28K12
9	28K6	Out-of-Plumb 1"
10	28K6	Out-of-Plumb 2"

The weak weld construction error is implemented by assigning the welds for the critical web members (the location of which is shown in Fig. 1) reduced weld lengths. The shorter welds reduce the rotation capacity and ultimate forces and moments that the connection can withstand. The weld reduction is different for the two structures to ensure that weld failure governs the building failure. The structure with 18K10 joists reduces the weld lengths in the critical web connections by 65%, while the weld lengths in the 28K6 structure are reduced by 35%.

The misplacement of joists is represented by the replacement of the center joist of the structure with another joist that looks similar to the properly designed joist. For the structure with 18K10 joists, the erroneous joist is an 18K7 joist. The 18K7 joist differs only 1/25 in. in chord thickness from the 18K10, but can only resist 502 lb/ft. (SJI 2005), which is 213 lb/ft. less than an 18K10 joist. The misplaced joist for the 28K6 structure is a 28K12 joist, which can resist 110 lb/ft. more than a properly designed 28K6 joist with 825 lb/ft. of resistance.

The out-of-plumb joists are executed in the model by defining the model geometry such that the top

and bottom chords are not aligned with each other. Out-of-plumbness at 1 in. and 2 in. is considered.

4.3 Pushdown Analysis

The models are analyzed using a pushdown load, under which the downward load on the roof is increased in a pre-defined pattern until one or more elements in the joist fails due to one of the mechanisms described above. The analysis is displacement controlled with the control node at the center of the structure in plan. The displacements and corresponding loads are recorded and plotted against each other in order to form the pushdown curve, which relates loads to displacements. During this analysis, the response of key nodes and elements is recorded to determine how the structure is failing. In this study, only uniform loads were considered in the pushdown analysis; however, drifted loads or other load patterns may be examined in future studies.

The models were validated by comparing results from an analytically derived pushdown curve to pushdown results from physical experiments on OWSJs performed by Yost *et al.* (2004). The models of three experimental specimens matched the pushdown curves of the experimental results well. The stiffness of the model joists were all within the range of stiffness determined through experimental results. The comparison of the ultimate load and displacement at ultimate load were also favorable with the modeled ultimate loads being only slightly higher than the experimental results and the displacements being within the range of experimental results. In addition, both the model and experimental joists failed in the same mechanism: buckling of the critical web member. Another point of comparison for the models is through comparison with the design loads for each joist defined in the SJI load tables. The modeled failure load for the joists used in this study exceeds the design loads provided in the SJI's load tables by over 250 plf for the 18K10 joist, as expected.

4.4 Pushdown Analysis Results

The results from the pushdown analyses of all ten building are presented in Figs. 3 & 4. On these figures, the roof load includes the load on the roof applied in excess of the dead load; the displacement represents the downward displacement of the control node (at center of the center joist). Due to the reported roof load being in excess of the dead load, the curve has a non-zero displacement under no load which corresponds to the displacement of the control node under dead loads only.

Fig. 3 provides the basis for comparison of *Bldgs. 1-5*, which all use the joist 18K10 as the base joist. The error-free building, *Bldg. 1*, fails at the point indicated on Fig. 3 due to buckling of the critical web

members in the majority of joists. *Bldg. 2* fails at a lower load than *Bldg. 1* because of the weakened weld connection between the critical web member and the bottom chord. *Bldg. 3* is unique because it is the only building out of the ten to exhibit significant ductility. Since the center joist of *Bldg. 3* is weaker than the surrounding joists, loads are redistributed from the center joist to others, until a load causing localized buckling at the mid-span of the top chord of the center joist is reached. The different joist is also the reason why *Bldg. 3* is less stiff than the other buildings. The buildings with out-of-plumb joists, *Bldgs. 4 & 5*, fail under different loads, but due to the same failure mechanism. The out-of-plumb joist creates torsion within the entire OWSJ, which adds a significant bending moment out of the plane of the joist to the web members. These members are designed to solely account for axial loads, causing them to buckle. This failure mechanism affects all the web members, but those designed originally for tension are affected the most due to their relatively low moments of inertia.

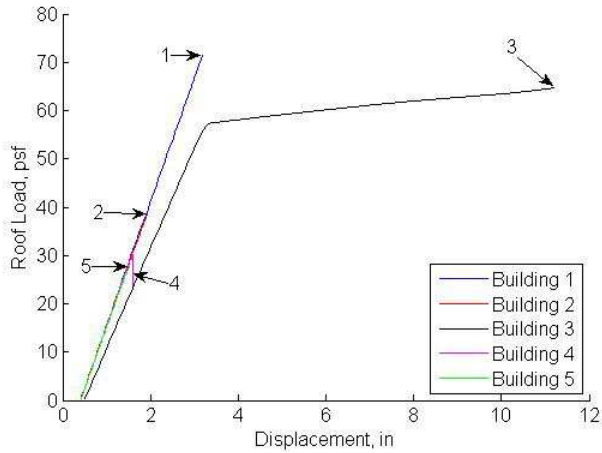


Figure 3: Pushdown results for buildings 1-5

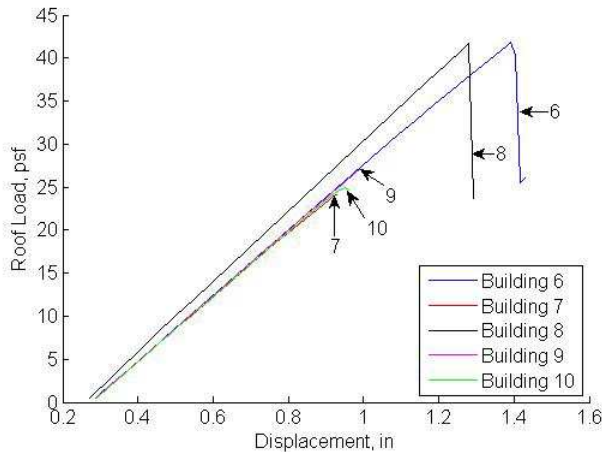


Figure 4: Pushdown results for buildings 6-10

Fig. 4 presents the pushdown curves for *Bldgs. 6-10*, all of which have a 28K6 joist as their base joist. The error-free building, *Bldg. 6*, fails due to the frac-

ture of the connection between the critical web members and the bottom chord. This failure mechanism is also encountered in *Bldg. 7*, but occurs at a lower load due to the weakened weld in that building. *Bldg. 8* demonstrates what happens when the center joist is stronger than the surrounding joists. Due to the joist misplacement, the stiffness of the building increases. However, failure still occurs at about the same load as the error-free building because the joists adjacent to the center joist have the original joist strength. As with *Bldgs. 4 & 5*, *Bldgs. 9 & 10* fail by web member buckling resulting from global torsion of the OWSJ.

5 RELIABILITY ANALYSIS

5.1 Methodology

Using the pushdown results, a reliability analysis can be performed to assess the risk of the roof failure under snow loads for Denver, CO. The model assumes the dead load and a roof live load of 14.3 psf (based on recommendations from Paz, 1994) is present on the roof at any given time. Different realizations of snow load are applied on top of the dead and live load. These snow loads represent the probability density function (PDF) for snow load in Denver and are applied to the model through Monte Carlo simulation. The PDF for Denver's ground snow load, shown in Fig. 5, was obtained through 121 years of recordings of snow depth and snow water equivalence at the Stapleton Airport in northeast Denver (provided by Cunningham, 2013). The history of ground snow load data at the site was fitted to a Log-Pearson III distribution with a mean annual maximum ground snow load of 5.77 psf with standard deviation of 4.54 psf. For simplicity, the ASCE 7-10 equations are used to translate ground to roof snow loads, taking the thermal, exposure, and importance factors equal to unity:

$$SL_{roof} = 0.7SL_{ground} \quad (1)$$

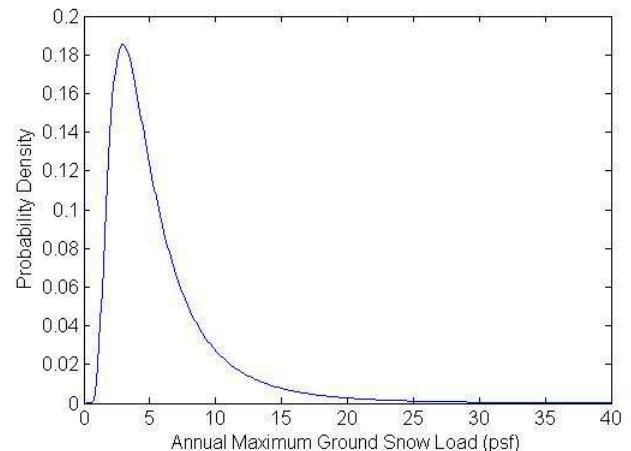


Figure 5: Probability density function for the ground snow load of Denver, Colorado

In the reliability analysis, 100,000 randomly selected annual maximum roof snow loads representing Fig. 5 and Eqn. (1) are considered. The limit states under investigation are: (1) total roof load surpassing the ultimate strength of the building, and (2) the downward deflection under total load exceeding 1/240 of the span length (1.5" in. this case). The pushdown curves are used to determine whether these limit states have been reached. These limit states were chosen because they represent failure of the structure which would be dangerous and costly (*i.e.* limit state 1), as well as serviceability failure of the structure which is not as dangerous, but could still be costly to building owners (*i.e.* limit state 2). The deflection limits in (2) are defined by ASCE 7-10 in the commentary (appendix C).

5.2 Reliability Results

The results of the reliability analysis for all ten buildings are presented in Table 2. For *Bldgs. 1-5*, which use an 18K10 base OWSJ, the safest building in terms of structural failure is *Bldg. 1* (*i.e.* the building with the lowest probability of limit state 1 occurrence), which is to be expected because it is free of construction errors. In Denver, CO this building will only fail once in about 11,111 years. For this discussion, suppose we judge an acceptable probability of structural failure by two conditions: (1) a safety index of 3.5 over 50 years, as recommended by the ASCE 7-10 commentary, which corresponds to an annual probability of failure of 5×10^{-6} (one failure in 200000 years), and (2) a safety index of 2.5 over 50 years which corresponds to an annual probability of failure of 1.25×10^{-4} (one failure in 8000 years). Results show that the error-free *Bldg. 1* is well within acceptable limits for a safety index of 2.5, but not within the limits for a safety index of 3.5. However, the serviceability limit will be violated about once every 60 years. Exceeding the serviceability limit state may cause visually apparent deformations, general architecture damage, or other damage to non-structural components (ASCE 2010).

Bldg. 3, which has an under-designed joist, also has a low probability of structural failure and has the closest results to those from *Bldg. 1* despite not satisfying the limits of either safety index (about 1 collapse in 5556 years). This relatively high level of reliability can be attributed to the center joist's ability to redistribute load while remaining intact. Interestingly, the under-designed joist has a more significant impact on serviceability than strength. *Bldg. 3* has a high probability of exceeding the serviceability limit state (about once every 9 years). This building has lower stiffness than the others, leading to higher displacements at any load relative to the other nine buildings under investigation.

Of the remaining construction errors, the 18K10 base OWSJ buildings is least affected by weakened welds. *Bldg. 2* produces a failure rate of about once in 344 years which remains relatively close to the results of *Bldg. 1*. On the other hand, *Bldgs. 4 & 5*, which have out-of-plumb joists, are the buildings with the worst reliability. For *Bldgs. 4 & 5* the probability of collapse corresponds to the buildings failing once every 82 and 48 years, respectively. The probability of exceeding the serviceability limit state is also slightly higher relative to *Bldgs. 1 & 2*, but still less than those from *Bldg. 3*. Despite this last point, the high probability of structural failure and the relatively high probabilities of serviceability failure indicate that *Bldgs. 4 & 5* are the most unreliable of the construction errors considered in the 18K10 joist buildings.

Table 2: Results of reliability analysis

Building No.	Probability of limit state occurring annually	
	Structural failure	$\Delta_{Tot} > L/240$
1	0.00009	0.01650
2	0.00291	0.01670
3	0.00018	0.10731
4	0.01213	0.01676
5	0.02066	0.02066
6	0.00177	0.00177
7	0.05007	0.05007
8	0.00176	0.00176
9	0.02427	0.02427
10	0.03893	0.03893

Results from *Bldgs. 6-10* lead to slightly different conclusions. The 28K6 base joist used in these buildings has a smaller bottom chord, leaving less space to weld the web members to the chord and leading to inherently weaker connections. This is observed immediately by recalling that *Bldg. 6*, the base building, fails by a weld failure. The probability of structural failure of the error-free building is outside of the acceptable limits for both safety index values considered, having a probability of structural failure of once in about 565 years. However, the deeper joists lead to higher stiffness of *Bldg. 6*, which produces lower probabilities of serviceability limits being exceeded as compared to *Bldg. 1*.

Comparing now the error-free building to those with construction errors, *Bldg. 7*, which has a weakened weld, proves to be the most unreliable building due to the susceptibility to weld failure of this structure. This building produces a probability of structural failure of about once in 20 years, which is dramatically less reliable than *Bldg. 6*. The misplaced joist building, *Bldg. 8*, produces roughly the same results as the error-free building due to their almost identical failure loads. The main difference is that *Bldg. 8* produces smaller displacements throughout the reliability analysis due to its larger stiffness. *Bldgs. 9 & 10*, much like *Bldgs. 4 & 5*, also have high probabilities of experiencing structural failure

which are much less than the results from *Bldg. 6* (once in 41 years, and once in 26 years respectively) and high probabilities of exceeding the serviceability limit. Although *Bldg. 7* is definitely the least reliable building of *Bldgs. 6-10*, the relatively poor performance of *Bldgs. 9 & 10* should not be disregarded.

6 CONCLUSIONS

This paper sheds light on the potential harm to structures whose roofs are supported by OWSJs with respect to both structural failure and serviceability conditions. The results presented in this paper demonstrate that having an error in the fabrication process such as a weak weld, or errors in the erection of a building such as misplacement of joists or erecting joists out-of-plumb can have negative effects on the reliability of the building. In particular, installing joists that are out-of-plumb in plane greatly decreases the reliability of the structure regardless of the base joist.

These results provide evidence to encourage fabricators and erectors to develop plans to avoid these construction errors. Although these analyses only considered small buildings under uniform snow load, the effects may be even more significant under drifted loads.

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