Tornado Fragility and Risk Assessment of an Archetype Masonry School Building

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Abstract

Tornadoes are a low-occurrence high-consequence hazard and not only threaten the life safety of building occupants but have recently resulted in billions of dollars in direct and indirect damages for single events. Design codes do not consider tornado loads for building and other structures (with the exception of nuclear facilities) because the occurrence rate has historically been considered too low. The advent of performance-based seismic design has revolutionized the engineering thought process and as a result building owners can consider performance objectives that enable a building to perform better in an extreme hazard than observed with current design code. In this paper, the performance of a masonry school building subjected to tornado wind loads is investigated using a fragility methodology. The tornado fragility assessment methodology is described along with proposed damage states for consideration in loss estimation. An array of masonry material types are considered based on the Masonry Standards Joint Committee (MSJC) code to enable applicability of the fragilities developed herein for a range of designs throughout the United States.

Keywords: Tornado; Risk Assessment; Fragility; Masonry; School building; Hazard Curve.

1. Introduction

1.1. Tornado Risk Assessment

Tornadoes are a low-occurrence high-consequence hazard and have resulted in billions of dollars in direct and indirect damages for single events and continue to threaten life safety in many regions of the United States [1]. While a number of studies have been conducted focusing on

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tornado dynamics (e.g., [2]), tornado wind pressure distributions (e.g., [3, 4]), design of structures for tornadoes (e.g., [5, 6]), and damage prediction for buildings in tornadoes (e.g., [7]); there still remains a substantial dearth of tornado-risk-assessment in the general open literature. Moreover, there is an extensive body of literature on seismic fragility analysis [8] and vulnerability assessments in the case of strong straight-line winds (i.e. Hurricanes) [9, 10]. However, there are only several tornado risk assessment studies, which have focused on risk for nuclear power plants [11] and other critical infrastructure but not typical buildings within communities. Thus, in order to perform a community risk analysis for tornado hazard, a significant body of research is needed to fill this dearth.

The low probability of occurrence of tornadoes has prevented consideration of tornado loads in modern building codes, but this is changing as a result of deadly and damaging tornadoes over the last five years (Tuscaloosa, 2011; Joplin, 2011; Moore, 2013). Although a number of studies had been done on tornado-induced loads on buildings [12, 13], the data deficiency motivated researchers to investigate tornado-induced forces on buildings [14, 15]. In this regard, tornado-like vortices have been simulated and investigated in laboratories, including the VorTECH simulator at Texas Tech University [16], the Iowa State University (ISU) simulator [14], and the Wind Engineering, Energy and Environment (WindEEE) Dome at Western University [17]. At ISU, Haan et al. [14] studied tornado-induced wind loads for a laboratory-simulated tornado and compared them with the provisions of building codes. Their study revealed that tornadoes can generate load coefficients greater than those prescribed by ASCE 7-05 [18] for straight-line wind over open terrain. Based on the discussion by Kopp and Morrison [19] on the study done by Haan et al. [14], it is clear that further studies are needed to investigate spatial correlation of tornado loads on the building envelope, translation speed of the vortex, duration effects, and changes to the internal pressure during a tornado, which were the main disagreements among the researchers. Kikitsu et al. [20] strengthened the formula proposed by Simiu et al. [21], for calculation of tornado-induced pressure, to consider the effect of both opening and leakage on internal pressure. In fact, this proposed approach provided a better model for tornado loading; however, additional studies would provide statistics for the parameters related to leakage and opening in different types of buildings as well as other coefficients.

Based on the work completed by van de Lindt et al. [6], tornado-induced loads can be calculated by applying tornado coefficients to pressure coefficients using the common approach in ASCE 7-10 [18]. Tornado coefficients used in this study are based on the study done by Haan et al. [14]. Note that Haan et al. [14] used ASCE 7-05, but using ASCE 7-10 does not alter the values since they are the same for both editions. Using this procedure, van de Lindt et al. [6] proposed a dual-objective design philosophy to reduce damage for tornadoes in the EF0 to EF2 (Enhanced Fujita scale [22]) range, and focus on life safety in tornadoes of higher intensity, i.e. EF3 to EF5. Moreover, this method has been used in tornado risk analysis of wood-frame buildings [23, 24] and performance assessment of earthen masonry dwelling buildings [25].

1.2. Schools in Tornado-Prone Regions

Pressures caused by high wind speeds cause damage to buildings and other components making up community infrastructure. While all buildings are important, certain buildings capture the attention of communities because of their purpose. Some of these buildings are facilities such as emergency facilities, schools which host communities' children each day, and hospitals. According to the wind hazard map in ASCE 7-10 [18], the majority of the United States, including Tornado Alley and Dixie Alley, has a basic wind speed of 54 m/s (120 mph) for school buildings (risk category III and IV buildings). This wind speed is for straight-line winds from a Derecho or hurricane, but in theory possess the same wind speed of an EF2 tornado, although strong tornado wind speeds have never been explicitly measured with enough quality for use in analysis. Tornado wind speeds for significant tornadoes have been measured only through mobile Doppler radar but not at ground level. Approximately 97 percent of all recorded tornadoes are rated as EF2 or below [26]. Tornadic winds produce an intensified mean flow and enhanced turbulence at ground level [27], and include a large static pressure drop and a vertical wind velocity component, which together may result in a higher level of damage, especially roof damage, than their straight-line wind counterpart corresponding to the same horizontal velocity. However, this latter point is still a focus of on-going research around the world and lacks consensus.

Therefore, at issue is that damage caused by an EF2 tornado is not consistent to that of a 54 m/s (120 mph) straight-line wind underscoring the fact that no design codes or guidance address tornado-induced loads except those developed by the U.S. Nuclear Regulatory Commission (e.g.

[11]) for reactor design. In fact, the wind pressures that tornadoes exert on a building were shown in one study to be more than twice [14] the minimum pressures derived from building codes, although as mentioned there has been disagreement amongst researchers to date on the approach used during the scale experiments [19]. In general, most buildings including schools will experience significant damage if they are in the path of a strong tornado because they have not been designed for this type of loading.

1.3. Observed Damage to Schools from Tornadoes

Over the years tornadoes have destroyed or severely damaged a significant number of schools, resulting in injuries and fatalities. One well-known case was Xenia senior high school in Xenia, Ohio in the United States (Figure 1) which was hit by an F5 tornado in 1974 [26]. The tornado passed directly over the school. The enclosure walls failed on the west and south sides; roofs collapsed over the three long spans—the auditorium, the boys’ gym, and the girls’ gym; and the lightweight roof was torn off by the extreme winds. St. Augustine elementary school in Kalamazoo Michigan was hit by an F3 tornado in 1980 [26]. The magnitude of the damage to the school was severe enough that demolition was eventually required. The loadbearing west wall collapsed inward, and the east wall fell outward. The roof fell into the building when the walls collapsed. Slender unreinforced masonry walls and the long-span roof structure were determined to be hazardous elements of this type of construction during post-event inspection. Kelly elementary school in Moore Oklahoma was hit by an F4 tornado in 1999 [26] to such an extent that the remaining structure was demolished and the school was reconstructed. Roof-to-wall connections were sufficient for gravity loads, but could not bear the high uplift loads caused by the wind. Unreinforced masonry walls failed when the roof system lifted off and was removed by tornado winds. Perhaps one of the most tragic school events occurred in the Enterprise, Alabama tornado in 2007 (EF4 tornado), when a school with unreinforced masonry walls and hollow-core concrete roof planks collapsed and eight student fatalities were reported [28]. On May 22, 2011, an EF5 tornado struck Joplin, Missouri. In the path of the tornado, Joplin High School, Franklin Technical Center, and Joplin East Middle School were extensively damaged while some other schools sustained moderate to intense damages. Although Joplin High School was built in 1968 and Joplin East Middle School was built in 2009, their performance from the tornadic winds was very similar [29, 30]. On May 20, 2013, Moore, Oklahoma, was impacted by an intense EF5

tornado. The tornado struck Briarwood Elementary School, Plaza Towers Elementary School, and Highland East Junior High School. Briarwood and Plaza Towers Elementary Schools sustained enough damage to be considered a total loss, while for Highland East Junior High School the primary damage during the tornado was to the gymnasium, and only minimal damage occurred to the classroom building which was further away from tornado center. Moreover, seven fatalities occurred in the Plaza Towers Elementary School when a hallway being used as a place of refuge collapsed [31].

The study presented in this paper discusses performance of the building envelope, nonstructural, and structural building systems of an archetype unreinforced masonry school building using a tornado fragility analysis methodology and characterizes the resulting fragilities for components into damage state fragility curves for the school building based on proposed building damage states. The results can serve two major purposes: (1) they can be utilized to inform guidelines for better design practices related to new and existing schools in tornado-prone regions, e.g. using steel reinforcement with full grouting in all cells instead of unreinforced masonry construction, and (2) the fragility curves developed herein can be used to be representative of masonry school buildings in community risk and resilience assessment and improvement models.

Figure 1. Xenia Senior High School, Xenia, Ohio: (a) before tornado, (b) after tornado
(Excerpted from FEMA [26], photo credit: WERC, TEXAS TECH UNIVERSITY)

2. Tornado Fragility Methodology

2.1. Fragility Modeling

A fragility can be defined as a conditional probability of exceeding a specific amount of damage as a function of an intensity measure [32]:

$$Fr(x) = P[D(x) \geq R | IM = x] \quad (1)$$

where IM = random variable intensity measure describing the intensity of the demand on the system (e.g., 3-s gust wind speed; earthquake spectral acceleration at the fundamental period of the building, peak ground acceleration, etc.), $D(x)$ = demand on the system at intensity measure of x , and R = resistance of component or system.

In this study, a probabilistic analysis of the system was performed using Monte Carlo Simulation (MCS) to develop tornado fragility curves. This approach generates samples of random demand and resistance variables from their specified distributions, or from statistical parameters of their components, and evaluates whether wind-induced failure would occur. This procedure is repeated for a range of wind speeds to complete the fragility curve and dependencies are included as will be expanded on later.

For convenience and often a good statistical fit, a lognormal distribution is used to describe a fragility (i.e., λ_R and ξ_R) to model structural systems or their components [33]:

$$Fr(x) = \Phi\left(\frac{\ln(x) - \lambda_R}{\xi_R}\right) \quad (2)$$

in which $\Phi[\cdot]$ = standard normal cumulative distribution function, x = intensity measure (i.e., 3-s gust wind speed for tornado fragility curves), λ_R = logarithmic median of resistance R (in units that are dimensionally consistent with demand), and ξ_R = logarithmic standard deviation of resistance R .

2.2. Tornado Load Modeling

The procedure discussed in ASCE 7-10 [18] for calculation of wind forces resulting from straight-line winds have been extensively studied, and the equation for calculating velocity pressure is:

$$q_z = 0.613 K_z K_{zt} K_d V^2 \quad (\text{N/m}^2; V \text{ in m/s}) \quad (3)$$

$$q_z = 0.00256 K_z K_{zt} K_d V^2 \quad (\text{lb/ft}^2; V \text{ in mph})$$

where, K_z = velocity pressure exposure coefficient, K_{zt} = topographic factor, K_d = wind directionality factor, V = basic wind speed, q_z = velocity pressure calculated at height z , and q_h = velocity pressure calculated at mean roof height h .

Wind pressure for the main wind force resisting system (MWFRS) is then calculated as:

$$p = qGC_p - q_i(GC_{pi}) \quad (\text{lb/ft}^2) (\text{N/m}^2) \quad (4)$$

where, q and q_i are velocity pressure calculated at height z or h , G = gust-effect factor, C_p = external pressure coefficient, and GC_{pi} = internal pressure coefficient.

For components and claddings (C&C) wind pressure is determined as:

$$p = q_h \left[(GC_p) - (GC_{pi}) \right] \quad (\text{lb/ft}^2) \text{ (N/m}^2\text{)} \quad (5)$$

where, q_h = velocity pressure evaluated at mean roof height h , (GC_p) = external pressure coefficients, and (GC_{pi}) = internal pressure coefficient.

The winds from a tornado extend 9,000 to 15,000 meters (30,000 to 50,000 feet) above the ground and are a turbulent phenomenon. In general, there are many key differences between tornadoes and straight-line wind. In this study, the ASCE 7-10 [18] methodology for straight-line wind loads with certain modifications was applied to obtain the tornado-induced forces.

The first modification is with regard to the directionality factor, K_d . Justification for using the directionality factor is mentioned in the ASCE 7-10 commentary [18] as the following: “This factor accounts for two effects: (1) The reduced probability of maximum winds coming from any given direction and (2) the reduced probability of the maximum pressure coefficient occurring for any given wind direction.” Indeed, these effects consider circumstances in which it is probable that the strongest winds come from a non-critical direction. All of these considerations deal with straight-line winds but the maximum wind speed can occur from any direction in tornadoes, which may exert a maximum pressure coefficient on buildings from any direction [30, 34]. Thus, the directionality factor is conservatively neglected in the tornado-induced load calculation and is set equal to 1.0 in determination of tornadic loads in the present study.

The second consideration for wind loading is that different exposures in ASCE 7-10 [18] result in a different coefficient for straight-line winds. Several studies have been completed regarding the effects of ground surface roughness in tornadoes [35-40]. Sabareesh et al. [39] showed that with introduction of roughness the internal pressures decrease in magnitude irrespective of swirl ratio, external pressures decrease in magnitude compared to internal pressures, and the net local roof wind force increases in magnitude. However, in this study, regardless of the ground surface roughness and the fetch length of the surrounding terrain, the analysis is based on Exposure C.

Moreover, there is an ongoing debate related to modification of pressure coefficients. Several studies have shown that tornadoes result in greater pressure on buildings than straight-line winds for the same reference wind speed [12-15, 38, 41]. However, there is no consensus in the intensity of pressure that tornadoes can apply on building surfaces since tornado loading still possesses significant uncertainty primarily related to researchers understanding of the phenomena. Therefore, additional parameters are defined in this study to modify pressure coefficients. The modification of the pressure coefficient is divided into two parameters to separate internal and external pressure coefficients which can be different in ASCE 7. The first parameter is defined herein to modify the external pressure coefficients, T_e , a *tornado external pressure adjustment* which is similar to past studies; and the second coefficient is T_i , a *tornado internal pressure adjustment*, which needs further investigation and is set to 0.0 or 1.0 in this study.

The modifications described above are applied to the method outlined in the wind provisions of ASCE 7-10 [18] to assess wind loads from tornadoes in this study. Moreover, local topographic effect was neglected, i.e. K_{zt} is assumed to be 1.0, and all the analyses either for the main wind force resisting system (MWFRS) or for components and claddings (C&C) were determined based on velocity pressure calculated at mean roof height h , i.e. q_h . Since the near-surface wind flow in tornadoes is turbulent and has not been adequately studied, this assumption is felt to be rational. Thus, the tornado-induced wind load is calculated by following equations:

$$q_h = 0.613 K_h K_{zt} V^2 \quad (\text{N/m}^2); V \text{ in m/s} \quad (6)$$

$$q_h = 0.00256 K_h K_{zt} V^2 \quad (\text{lb/ft}^2); V \text{ in mph}$$

where, K_h = velocity pressure exposure coefficient at mean roof height h , K_{zt} = topographic factor, V = 3-sec gust wind speed, and q_h = velocity pressure calculated at mean roof height h .

$$p = q_h \left[T_e (GC_p) - T_i (GC_{pi}) \right] \quad (\text{lb/ft}^2) (\text{N/m}^2) \quad (7)$$

where, q_h = velocity pressure evaluated at mean roof height h , G = gust-effect factor, C_p = external pressure coefficient, (GC_{pi}) = internal pressure coefficient, T_e = tornado external pressure adjustment, and T_i = tornado internal pressure adjustment.

In order to calculate tornado-induced wind loads by using Equation 7, two alternative approaches were considered in this study—namely approach A and B, which are as follows:

Approach A: Using ASCE 7-10 pressure coefficients along with tornado pressure adjustments derived based on the study done by Haan et al. [14].

Haan et al. [14] studied tornado-induced wind loads in a laboratory-simulated tornado on a 1:100 scale one-story gable roof building model with a 91 mm by 91 mm (3.6 in. by 3.6 in.) plan, an eave height of 36 mm (1.4 in), and roof angle of 35°. They compared the measured pressures with the ASCE-7 standard, and found that external pressure coefficients appeared to be greater than those prescribed by ASCE 7-05 [18] for straight-line winds over open terrain. Based on their study, the uplift pressure coefficients for the MWFRS and for C&C exceeded those from the ASCE 7-05 [18] wind provisions by a factor of 1.8–3.2 and 1.4–2.4, respectively. They assumed a fully sealed building in their study, which negated the effect of internal pressure change on mitigating the static pressure drop caused by the tornado vortex. However, there exist openings and some level of leakage in typical buildings which could significantly decrease the effect of static pressure drop [41]. Therefore, using these coefficients as T_e in Equation 7 requires that T_i be set equal to zero since the internal pressure has already been considered inherently in coefficients identified by Haan et al. [14]. It is worth mentioning that although openings reduce the effect of the static pressure drop, at the same time they may cause notable internal pressure by letting wind flows into building [16, 20]. A similar observation was made for lateral pressure coefficients, and revealed that the pressure coefficients ranged from 1.0 to 1.5 times larger than those coefficients in ASCE 7-05 [18] for MWFRS and 1.2 to 1.5 times larger for C&C. Using the values obtained for adjusting the lateral pressure coefficients for C&C (i.e. T_e ranges between 1.2 and 1.5) in Equation 7, T_i should be set equal to zero for the same reason explained above for the MWFRS and C&C. However, lateral pressure coefficients for MWFRS were calculated by integrating the pressures over the model surface, which canceled out the static pressures effect (internal pressure and pressure drop) only in this case [14]. Therefore, using this T_e value for lateral pressure coefficients for MWFRS in Equation 7 should be done by setting T_i equal to 1.0.

Approach B: Using ASCE 7-16 pressure coefficients along with both T_e and T_i equal to 1.0.

The ASCE 7-16 coefficients for roof pressures are notably larger than the ASCE 7-10 based on recent wind tunnel test results [42], and are believed to be derived from the best pressure measurements generated to date. Moreover, as discussed earlier, internal pressure is neglected in approach A (except for estimating lateral pressures on MWFRS). However, in some cases, including internal pressure and using tornado pressure adjustments equal to 1.0 in Equation 7 may result in a higher load than when using approach A. In this regard, the alternative approach B is defined by setting the tornado pressure adjustments (i.e. T_e and T_i) equal to 1.0 and the ASCE 7-16 pressure coefficients [42] were used representing the best knowledge in pressure coefficients currently available for straight-line winds.

The differences between the approaches A and B for calculating tornado-induced load by using Equation 7 are highlighted in Table 1.

Table 1. Difference between Approaches A and B for Calculating Tornado-induced Load

Parameters Description		Approach A		Approach B
Tornado Pressure Adjustment	Uplift Pressure	MWFRS	T _e 1.8 - 3.2	1.0
			T _i 0.0	
		C&C	T _e 1.4 - 2.4	
			T _i 0.0	
	Lateral Pressure	MWFRS	T _e 1.0 - 1.5	
			T _i 1.0	
		C&C	T _e 1.2 - 2.0	
			T _i 0.0	
External Pressure Coefficients, GC _p			ASCE 7-10	ASCE 7-16

Statistics for wind load parameters are summarized in Table 2 and Table 3. Since the school building studied here has a flat roof, the nominal values for GC_p for roof cover are based on the values in ASCE 7 for gable roofs with $\theta \leq 7^\circ$. As mentioned before, the topographic factor, K_{zt} , is deterministic and set equal to 1.0 in this study. The ASCE 7-16 parameters [42] used in this study are same as those of the ASCE 7-10 [18] except the external pressure coefficients for roof C&C which are tabulated in Table 4.

268 Table 2. Wind Load Statistics

Parameters	Descriptions	Nominal	Mean	COV	Distribution	References
K_z	0 - 4.57 m (0 - 15 ft)	0.85	0.82	0.14	Normal	[23, 43]
	6.10 m (20 ft)	0.90	0.84			
	7.62 m (25 ft)	0.94	0.88			
	9.14 m (30 ft)	0.98	0.94			
	12.19 m (40 ft)	1.04	1.00			
G	-	-	0.82	0.10	Normal	[44]
T_e	Uplift Pressure	MWFRS	1.8-3.2	-	Uniform	[14, 23]
		C&C	1.4-2.4			
	Lateral Pressure	MWFRS	1.0-1.5			
		C&C	1.2-2.0			

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270 Table 3. Wind Load Statistics- Pressure Coefficients

Parameters	Descriptions	Nominal	Mean	COV	Distribution	References
GC_{pi}	Enclosed Buildings	0.18	0.15	0.33	Normal	[23, 43]
	Partially Enclosed Buildings	0.55	0.46			
C_p	Wall	0.80	0.69	0.15	Normal	[44]
	Roof	-0.90	-0.81			
GC_p	Parapet	1.50	1.43	0.18	Normal	[44]
	Door	-0.86	-0.81	0.12		
	Window	-0.81	-0.77	0.12		
	Roof Cover- Zone 1	-1.00	-0.95	0.12		
	Roof Cover- Zone 2	-1.80	-1.71	0.12		
	Roof Cover- Zone 3	-2.80	-2.66	0.12		

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272 Table 4. External pressure coefficients for roof C&C in ASCE 7-16

Parameters	Descriptions	Nominal	Mean	COV	Distribution	References
GC_p	Roof Cover- Zone 1'	-0.90	-0.86	0.12	Normal	COV and Mean:
	Roof Cover- Zone 1	-1.70	-1.62	0.12		[44]
	Roof Cover- Zone 2	-2.30	-2.19	0.12		Nominal:

2.3. Resistance Modeling

Recall that many of the school buildings described earlier that failed in tornadoes were constructed using masonry walls, which is a common building type for schools around the U.S. For edge-supported (by roof, floor, and adjacent walls, etc.) unreinforced masonry walls under uniform lateral pressure loading, the major failure mechanism is associated with cracking due to flexural tensile stress [9]. Therefore, tensile strength is the most important failure metric in unreinforced masonry design, which is a function of several factors including tension direction, such as tension parallel to or perpendicular to the bed joint; type of units, i.e. solid units or hollow units; the unit grouting condition, i.e. ungrouted, partially grouted, or fully grouted; and the mortar type, such as Portland-Cement Lime mortar (PCL) or Masonry Cement (MC) mortar, which can be type M, S, or N mortar. The out-of-plane modulus of rupture values in the strength provisions of the Masonry Standards Joint Committee (MSJC) code [45] are approximately 1.9 times the allowable stress design values. Since no source data is available for partially grouted masonry, its values are determined on the basis of linear interpolation between fully grouted hollow units and ungrouted hollow units based on amount (percentage) of grouting [45]. In this study, 25 percent of grouting was assumed as the definition of partially grouted masonry which would be consistent with grouting of cells at 1.53 to 1.83 meter (5 to 6 feet) on-center for reinforcement. Two types of concrete masonry units (12" CMU for gyms and auditoriums, and 8" CMU for the rest of the building) and five types of masonry material are considered in this study.

A summary of the flexural tension values is given in Table 5; all the values are related to tension normal to bed joints in hollow units. The table gives the sample size, mean, and coefficient of variation of each masonry material type. Furthermore, the current allowable flexural tension stress and modulus of rupture in the MSJC code [45], and the ratio of the mean flexural tension stress to allowable stress is highlighted in the aforementioned table. All flexural tension values given in Table 5 are based on net cross-sectional properties [46], and normal distributions were assigned for all of these materials.

Table 5. Statistics of Masonry Flexural Tension- Normal to bed joints, Hollow units [46].

Type	Masonry and Mortar Types	Sample Size	Mean	COV	Modulus of Rupture in MSJC Code	Allowable Stress in MSJC Code	Ratio of Mean to Allowable Stress
M1	Fully Grouted, M/S, PCL	6	2.0 MPa (289.6 psi)	0.11	1.12 MPa (163 psi)	0.59 MPa (86 psi)	3.37
M2	Partially Grouted, M/S, PCL	-*	1.47 MPa (213.0 psi)	0.31	0.61 MPa (88 psi)	0.32 MPa (46 psi)	4.63
M3	Ungouted, M/S, PCL	80	1.29 MPa (186.5 psi)	0.48	0.43 MPa (63 psi)	0.23 MPa (33 psi)	5.65
M4	Ungouted, N, PCL	61	0.69 MPa (100.5 psi)	0.45	0.33 MPa (48 psi)	0.17 MPa (25 psi)	4.02
M5	Ungouted, N, MC	17	0.36 MPa (52.7 psi)	0.45	0.16 MPa (23 psi)	0.08 MPa (12 psi)	4.39

* The values for partially grouted masonry are determined on the basis of linear interpolation between fully grouted hollow units and ungouted hollow units.

Precast concrete single Tee beams and precast concrete hollow-core beams were used for the roof system in this study which is consistent with the roof system in the Xenia senior high school failure in 1974. In fact, prestressed concrete is a technique for overcoming concrete's weakness in tension so that it can be used in beams with a longer span. Conventionally, precast concrete beams are designed for the gravity loads, and not for the uplift loads. Hence, although prestressing removes a number of design limitations and has several advantages under normal loading conditions, is a disadvantage when subjected to wind uplift loads and works to undermine the resistance once the dead load is offset by uplift forces, resulting in negative moment failure. This type of failure occurred in several wind events (e.g., Typhoon Paka, Guam 1997; a tornado in Missouri, May 2003) [28].

The statistics for negative moment resistance and material properties of precast concrete beams are provided in Table 6. The statistics for negative moment resistance of beams are calculated based on material properties and conventional procedures in precast concrete structural analysis. The section properties and strands pattern of each beam were determined using a PCI (1971) design manual [47]. The old Xenia senior high school, which was destroyed in the 1974 tornado, had been built in 1957, but the archetype herein was designed to be representative of a 1970's school.

Table 6. Summary of Statistics for Negative Moment Resistance and Material Properties of Roof Beams

Parameters	Description	Mean	COV	Distribution	References
f_c	(Concrete 28-day cylinder strengths)	45.85 MPa (6.65 ksi)	0.130	Normal	[48]
f_y	(Yield strength of reinforcement bars)	475.74 MPa (69 ksi)	0.073		[49]
E_{ps}	(Modulus of Elasticity of Grade 270 Strands)	195928 MPa (28417 ksi)	0.034		[50]
f_{pu}	(Ultimate Strength of Grade 270 Strands)	1992.6 MPa (289 ksi)	0.087		[50]
Negative Moment Resistance	4HC8+2 With Strand Code 58-S	19.66 kN.m (174 kip.in)	0.086	Normal	Based on Calculation and PCI (1971) [47]
	8ST36+2 With Strand Code 128-D1	221.56 kN.m (1961 kip.in)	0.076		
	10ST48+2 With Strand Code 148-D1	275.23 kN.m (2436 kip.in)	0.080		
	10ST48+2 With Strand Code 188-D1	265.06 kN.m (2346 kip.in)	0.095		

Moreover, the connections between the precast concrete decks and wall are not usually designed for uplift loads. Therefore, because the deck dead load itself is often inadequate to resist the tornado uplift, bolt break out can be another roof failure mode in this case. Bolt break out statistics are given in Table 7.

Table 7. Roof-to-Wall Connections Statistics

Description	Mean	COV	Distribution	References
Bolt #5; 8" CMU; Fully Grouted	49.73 kN (11.18 kip)	0.10	Normal	[51]
Bolt #5; 8" CMU; Partially Grouted	32.65 kN (7.34 kip)	0.12		
Bolt #5; 12" CMU; Fully Grouted	51.38 kN (11.55 kip)	0.15		
Bolt #5; 12" CMU; Partially Grouted	40.52 kN (9.11 kip)	0.10		

Roof covering damage (including rooftop equipment) is the most common type of wind damage. Built-up roof (BUR) covers and single-ply membrane (SPM) covers can be used as a roof cover

system. BUR covers are comprised of multiple plies of roofing felts adhered to each other and to the insulation substrate with a full mop of hot asphalt, coal tar or cold adhesive [9]. Failure of the perimeter flashing is usually the first wind-induced failure for BUR covers, which then expedites peeling for the roof cover membrane at the newly exposed edge; the peeling failure can continue and result in extensive roof cover damage. Bubbling is another roof cover failure, where the roof cover is separated from the substrate by high wind uplift pressures. A bubbled part of the roof membrane may become breached by impact from flying debris or by tearing at the perimeter of the bubble, or it may expand to an area where the roof cover is torn and therefore that bubbled section can also tear. Moreover, since BUR covers are often surfaced with gravel or slag, the loose aggregate of the built-up roof can blow off and cause damage to windows in nearby houses. Window and door damage is among the most common wind damage. The importance of building envelope damage cannot be understated because of water intrusion into buildings at envelope breaches. The resistance statistics for the BUR roof cover, windows, and doors are shown in Table 8.

Table 8. Resistance Statistics for Roof Cover, Window, and Door

Component		Mean	COV	Distribution	References
BUR Roof Cover	Flashing Resistance	328.36 N/m (22.5 plf)	0.30		
	Peeling Resistance	2.39 kPa (50 psf)	0.15		
	Bubbling Resistance	7.18 kPa (150 psf)	0.15	Normal	[9]
Windows Resistance		1.92 kPa (40 psf)	0.20		
Doors Resistance		2.39 kPa (50 psf)	0.20		

Dead loads have a beneficial effect in wind-induced fragility analysis since they resist uplift loading and must therefore be included in the analysis. Dead load statistics for precast concrete roofs and concrete masonry units are provided in Table 9.

Table 9. Dead Load Statistics

Element Type	Mean	COV	Distribution	Sources
Hollow-Core Beam (4'HC8'')	4.73 kN/m (324 plf)	0.1	Normal	COV and Distribution Type: [32]; Mean Values: [47]
Single-Tee Beam (8'ST36'')	11.59 kN/m (794 plf)			
Single-Tee Beam (10'ST48'')	15.54 kN/m (1065 plf)			
CMU 8'	2.30 kPa (48 psf)	0.1	Normal	COV and Distribution Type: [32]
CMU 12'	3.54 kPa (74 psf)			

2.4. Damage States

In this study, damage states for school buildings are defined based on the approach used by Vann and McDonald [52]. The damage states used for school buildings are defined in Table 10 and are governed by the performance of the building envelope, roof structure, and walls. Damage is classified into five states, varying between 0 and 4 for consistency with HAZUS-MH-MR4 [9]:

Damage State 0 (No Damage): Little or no visible damage from the outside; No broken windows or doors; Minimal loss of roof cover, with no or very limited water penetration.

Damage State 1 (Minor Damage): Maximum of one or two broken windows or doors; Moderate roof cover loss that can be covered with a tarp to prevent additional water entering the building¹; Marks or dents on walls requiring painting or patching for repair.

Damage State 2 (Moderate Damage): Still capable of being occupied; Major roof cover damage; Moderate window breakage; Some resulting damage to interior of the building from water.

Damage State 3 (Severe Damage): Not able to be occupied, but repairable; Major roof cover loss; Major loss of windows and doors; Extensive damage to interior from water; Non-load-bearing wall failure.

¹ It should be noted that unlike hurricanes, not all tornadoes are accompanied by rainfall. One example is the 2011 EF4 Tuscaloosa tornado that had very little rainfall associated with it until later in the day from a second supercell that spawned tornadoes in different locations.

Damage State 4 (Destruction): Not able to be occupied and not repairable; Roof system failure; Load-bearing wall failure.

If any of the shaded damage indicators in a given row of Table 10 occur, the school building is considered to be in that damage state. For example, for a school to be considered to have sustained damage state 4 (Destruction), the building must have sustained either roof structural failure or load-bearing wall failure. It is worth mentioning that in damage state 4 the first four damage indicators will typically occur, which means damage state 4 depends only on roof structural failure and load-bearing wall failure.

Table 10. Damage States for School Building

Damage State	Roof Cover Failure	Window/Door Failures	Parapet Failure	Non-load-bearing Wall Failure	Roof Structural Failure	Load-bearing Wall Failure
0	≤ 2%	No	No	No	No	No
1	> 2% and ≤ 15%	1 or 2	No	No	No	No
2	> 15% and ≤ 50%	> 1 or 2 and ≤ 25%	No	No	No	No
3	> 50%	> 25%	Yes	Yes	No	No
4	Typically > 50%	Typically > 25%	Typically Yes	Typically Yes	Yes	Yes

* Each damage state is defined as occurrence of any of the shaded damage indicators in a given row.

3. Tornado Hazard Model

Five types of masonry construction (with properties summarized in Table 5) were investigated in this study in order to provide fragility functions applicable to different locations throughout the United States. Therefore, five cities across the US that each have a different tornado hazard, were selected. Norman, Oklahoma as a part of Tornado Alley was selected to be representative of a region of high tornado activity. Xenia, Ohio and Omaha, Nebraska were selected as moderately tornado-prone regions. Tampa, Florida and Phoenix, Arizona were selected as regions with relatively weak and very low risk of tornado, respectively. The tornado hazard curves for these locations are shown in Figure 2 in the form of complementary cumulative distribution functions (CCDF). In order to develop each hazard curve, the mean annual probability of occurrence of

each EF scale was obtained from the tornado hazard maps created by Standohar-Alfano and van de Lindt [53]. Table 11 provides the mean annual occurrence probability of each EF category for these five U.S. cities.

Table 11. Mean Annual Probability of Occurrence of each EF Category

EF Category	EF0	EF1	EF2	EF3	EF4	EF5
Speed Range- m/s (mph)	29≤v<38 (65≤v<86)	38≤v<50 (86≤v<111)	50≤v<61 (111≤v<136)	61≤v<74 (136≤v<166)	74≤v≤90 (166≤v≤200)	90<v (200<v)
Norman, OK	3.69E-04	2.18E-04	7.00E-05	1.20E-05	1.05E-06	1.43E-08
Xenia, OH	8.05E-05	3.70E-05	8.93E-06	1.21E-06	9.38E-08	1.65E-09
Omaha, NE	1.20E-04	4.95E-05	8.55E-06	9.04E-07	1.28E-08	1.26E-10
Tampa, FL	4.52E-05	8.43E-06	2.81E-07	2.28E-09	1.83E-14*	1.56E-19*
Phoenix, AZ	3.02E-06	7.24E-07	4.87E-09	1.83E-11	1.09E-15*	7.50E-22*

* Mathematically computed; but may be an unrealistic extrapolation.

Based on the speed range of each EF category mentioned in Table 11, the probability of exceeding the lower limit of wind speed related to each EF scale can be evaluated by summation of the occurrence probability of each EF scale above that upper EF rate. For example, since an EF4 tornado is defined as a tornado with wind speeds between 74 m/s (166 mph) and 90 m/s (200 mph), and an EF5 tornado has speeds exceeding 90 m/s (200 mph); then, the probability that a tornado exceeding 74 m/s (166 mph) would be, for example, 1.07E-06 for Norman (i.e. 1.43E-08 plus 1.05E-06). Therefore, the mean annual probability of exceeding the lower limit of wind speed related to each EF scale was calculated and is presented in Table 12.

Table 12. Mean Annual Probability of Exceeding the lower limit of wind speed related to each EF Category

EF Category	EF0 and above	EF1 and above	EF2 and above	EF3 and above	EF4 and above	EF5
Speed Range- m/s (mph)	29≤v<38 (65≤v<86)	38≤v<50 (86≤v<111)	50≤v<61 (111≤v<136)	61≤v<74 (136≤v<166)	74≤v≤90 (166≤v≤200)	90<v (200<v)
Norman, OK	6.69E-04	3.01E-04	8.31E-05	1.31E-05	1.07E-06	1.43E-08
Xenia, OH	1.28E-04	4.72E-05	1.02E-05	1.30E-06	9.55E-08	1.65E-09
Omaha, NE	1.79E-04	5.90E-05	9.46E-06	9.17E-07	1.29E-08	1.26E-10
Tampa, FL	5.39E-05	8.72E-06	2.84E-07	2.28E-09	1.83E-14	1.56E-19
Phoenix, AZ	3.75E-06	7.29E-07	4.89E-09	1.83E-11	1.09E-15	7.50E-22

403

404 According to the data provided in Table 12, a second-order exponential function is a good fit to
 405 the data in Ln-Ln space (i.e. Equation 8) in order to complete the tornado hazard curve which is
 406 then expressed as:

$$407 \quad \ln(H_V(v)) = \alpha e^{\beta \ln(v)} + \gamma e^{\lambda \ln(v)} \quad (8)$$

408 In other words, the following equation is proposed here for the complementary cumulative
 409 distribution function (CCDF) of tornado hazard curves only for wind speeds in excess of 22 m/s
 410 (50 mph), which is satisfactory for tornado risk assessment since fragility functions have a value
 411 of zero for wind speeds less than 22 m/s (50 mph):

$$412 \quad H_V(v) = \exp(\alpha(v)^\beta + \gamma(v)^\lambda) \quad v \geq 22 \text{ m/s (50 mph)} \quad (9)$$

413 where, $H_V(v)$ is mean annual probability of exceeding 3-sec gust wind speed in tornadoes, V is 3-
 414 second gust wind speed, α , β , γ , and λ are constants evaluated by fitting the above equation over
 415 the hazard curve data.

416

Figure 2. Tornado Hazard Curves for Selected Locations—Dots show the values in Table 12

4. Tornado Risk Assessment

The probability of exceeding each damage state (DS) is then obtained by convolving the tornado fragility with the tornado hazard curve, which can be expressed as:

$$P_f = \int_{x=0}^{x=\infty} Fr(x) \cdot h_{IM}(x) dx = \int_{x=0}^{x=\infty} fr(x) \cdot H_{IM}(x) dx \quad (10)$$

where, $Fr(x)$ is the fragility function in the form of a cumulative distribution function (CDF) which was described in Equation 1, $fr(x)$ is the derivative of $Fr(x)$ or the fragility function in the form of a probability density function (PDF), $H_{IM}(x)$ and $h_{IM}(x)$ represent the CCDF and PDF of the hazard, respectively, and IM represents the intensity measure (e.g., 3-s gust wind speed). The CCDF of tornado hazard was defined in Equation 9, and the hazard PDF can then be calculated by:

$$h_{IM}(x) = \frac{d}{dx} [1 - H_{IM}(x)] \quad (11)$$

P_f is probability of exceeding the DS in one year provided that the hazard curve is based on the annual probability of exceeding an IM . Therefore, in order to get the n -year probability of exceeding the DS, P_f^n , one can write:

$$P_f^n = 1 - (1 - P_f)^n \quad (12)$$

The Poisson distribution gives the same probability as the binomial distribution in Equation 12 for obtaining the n -year probability of exceeding the DS; however, it should be noted that if the Poisson distribution is used then the “annual rate” should be used in lieu of the “annual probability” for the hazard curve.

5. Illustrative Example

5.1. School Building Archetype

Xenia Senior High School located in Xenia, Ohio, was investigated as an unreinforced masonry high school in this study in order to serve as a typical archetype for community risk and resilience analysis. As mentioned earlier this high school was hit and destroyed by an F5 tornado

in 1974 [26]. The construction types varied among the main parts of the school—original building, and three additions (A, B, and C) [26]. The structural system for the original building and addition B was a lightweight steel frame with open-web steel joists. Addition A had loadbearing masonry walls with hollow-core precast concrete roof planks, and addition C had precast concrete frames with concrete double-tee roof beams. Moreover, the girls’ gym included loadbearing masonry walls with precast concrete tee beams while the auditorium and the boys’ gym included loadbearing masonry walls with steel trusses. The school floor plan is shown in Figure 3. However, the building studied in this paper is assumed to be a one story building with a structural system consisting of unreinforced masonry walls constructed of 0.2-meter (8-inch) concrete masonry units and 0.1-meter (4-inch) face bricks. The roof consists of hollow-core precast concrete roof planks (Figure 4). Moreover, this high school has three unreinforced masonry long-span rooms, i.e. two gymnasiums (girls’ gym and boys’ gym) and one auditorium constructed of 0.3-meter (12-inch) concrete masonry units and 0.1-meter (4-inch) face bricks, which have roofs with precast concrete tee beams (Figure 4). Details of roofs and walls are summarized in Table 13, and the beam cross-sections used in the roof system of the main building and the girls’ gym are shown in Figure 4.

Figure 3. The High School Archetype Floor Plan (Excerpted from FEMA [26])

Figure 4. Cross-sections Used in the Roof System of the Main Building and the Girls' Gym

Table 13. High School Building Construction Details

Building Part	Dimensions	Walls	Precast Concrete Roof System*		
			Beam Type	Name in PCI [47]	Strand Pattern
Main	Walls Height = 4.3m (14ft) Parapets Height = 0.9m (3ft)	8' CMU	Hollow-Core	4HC8+2	58-S
Girls' Gym	39m × 24.5m × 8m (128ft × 80ft × 26ft)	12' CMU	Single Tee	8ST36+2	128-D1
Boys' Gym	43m × 30.5m × 9m (140ft × 100ft × 30ft)	12' CMU	Single Tee	10ST48+2	188-D1
Auditorium	44m × 27.5m × 9m (144ft × 90ft × 30ft)	12' CMU	Single Tee	10ST48+2	148-D1

* Roof details can be found in PCI [47].

6. Results and Discussion

6.1. Door and Window Fragility Analysis

Monte Carlo Simulation (MCS) was used to develop the school building fragility for each damage state, and the details of the procedure are presented in flowchart form in Figure 5. Based on the three limit states presented in Table 10 for doors and windows (the third column), the fragility curves of these three damage indicators were developed for both the enclosed and partially enclosed building condition. Based on the discussion in section 2.2, the enclosed or partially enclosed condition of building has no effect in the wind loads calculation in Approach A (except for calculating the lateral pressures on MWFRS), but they affect wind loads related to Approach B. A total of 120 doors and windows were considered in the analysis, specifically 20

doors and 100 windows. Fragility curves are presented in Figure 6 for limit states 1-3 as a function of 3-sec gust wind speed. Since more than 25 percent failure of doors and windows are assumed to occur in damage state 4, they have no effect on the fragility curve for this damage state.

* It is noted that for a large building, loads may differ from one side of the building at a point in time. However, since the peak load from a 3-sec gust wind speed is used, i.e. a static analysis, the spatial effect across the building is neglected in this study.

Figure 5. Flowchart of Developing Fragility Curves

Figure 6. "Doors and Windows" Fragility Curves

6.2. Roof Cover Fragility Analysis

Observations of roof cover damage in hurricanes has indicated that because of building aerodynamics the highest uplift pressures occur at roof corners, the second highest at the roof perimeter, and the remainder of the roof has even lower pressures [28, 54]. Investigation following tornadoes appears to be somewhat consistent [6]. Therefore, based on limited knowledge of tornado pressures compared to hurricanes at this stage there is no logical reason to use different pressure zones beyond those which are defined by ASCE 7-10 for roof wind load calculations. Figure 7 shows the failure probability of roof perimeter flashing and the probability of roof cover peeling or bubbling for the different zones indicated in this Figure. According to Figure 7, for Approach A fragilities, the roof cover has a 16% probability of peeling or bubbling for only a 45 m/s (100 mph) 3-sec gust wind speed in zone 1, and it has a significantly higher peeling or bubbling probability in zone 2 and 3, namely 85% and 100%, respectively. On the other hand, using Approach B will result in no failure in zone 1', and 5%, 35%, and 85% probability of failure respectively in zones 1 to 3 for roof peeling and bubbling.

Figure 7. Built-Up Roof (BUR) Covers Probability of Failure

Roof cover fragility curves for the limit states defined in Table 10 (the second column) are presented in Figure 8. In order to calculate the roof cover probability of failure, the roof area is divided into square areas. Monte Carlo Simulation was implemented in the same process as displayed in Figure 5, and the failure condition of each square areas was found by comparison of

randomly generated values of resistance and load using roof cover resistance and wind loads statistics. It is worth mentioning that, although areas are considered to be statistically independent with respect to roof cover resistance, they are completely dependent through their exposure to wind loads. As indicated in the flowchart in Figure 5, in order to find the binary failure condition for the areas, all randomly generated resistances were compared to a single random demand (i.e., equal velocity pressure, q_h , for all the roof area in each iteration, but different wind pressure, p , for different zones) in each MCS sample (the first process in Monte Carlo Simulation part of flowchart shown in Figure 5). Similar to the doors and windows, roof cover has no effect on the fragility curve for damage state 4 since more than 50 percent failure of the roof cover is assumed to be included in DS 4 for roof cover.

Figure 8. Built-Up Roof Covers Fragility Curves

Although the external pressure coefficients for roof C&C were increased for zones 1 to 3 in ASCE 7-16, the fragilities for roof cover based on ASCE 7-16 are not necessarily more fragile than those based on ASCE 7-10 since a new zone (i.e. zone 1') was defined in ASCE 7-16 which has lower external pressure coefficient than the three zones in ASCE 7-10 [18]. Buildings with the smaller horizontal dimension less than 2.4h do not include zone 1', therefore their fragilities are shifted left when ASCE 7-16 coefficients are used. However, in a building which major part of the roof is zone 1', although limit state 1 and 2 may have a higher failure probability based on

ASCE 7-16 coefficients, they might have a lower probability for limit state 3 since zone 1' has a significant contribution to this limit state (i.e. more than 50% of the roof cover fails).

6.3. Roof Structural Fragility Analysis

During intense tornadoes, roofs can be lifted off and can re-settle often collapsing as the result of significant wind uplift pressure. The weight of building components (e.g., roof precast concrete beams in the school building in the present study) helps resist uplift loads. Consequently, these components usually survive extreme winds if they are also well tied to all other building components. However, concrete roof panels that are not properly connected or reinforced have failed during severe winds. Conventionally, precast concrete beams are designed for gravity loads (i.e. positive moment), but not for the uplift loads (i.e. negative moment). Hence, the combined effects of large wind uplift forces and the beams own pre-stressing forces can result in lifting of the panels and thus negative bending failure.

Two modes for roof panel failure were considered in this study and include the roof-to-wall connection failure and the bending failure of the roof deck under negative moment caused by tornado wind uplift pressures. The roof structural failure probability, P_{rsf} , was evaluated based on the union of these two failure probabilities defined as:

$$P_{rsf} = P_{cf} \cup P_{bf} \quad (13)$$

where, P_{cf} is probability of roof-to-wall connections failure, and P_{bf} is probability of bending failure of the roof beam. These two events are not statistically independent since they are subjected to the same wind uplift pressure, which was considered in the analysis as discussed earlier.

MCS were utilized to evaluate survivability of a beam in bending and connections (or to calculate P_{cf} and P_{bf}), and subsequently the survivability of the roof (or the probability of roof failure). Structural analyses were conducted under the assumption of a simply supported beam (i.e., simply supported single tee beams for the boys and girls gymnasiums as well as auditorium, and simply supported hollow-core roof panels for the rest of the building) in order to find demand forces. Figure 9 shows P_{cf} , P_{bf} , and P_{rsf} as a function of 3-sec gust wind speed for the boys' gymnasium roof. As indicated in Figure 9, the fragility curves for the union of bending

failure and connection failure lies on the fragilities of the bending failure, which reveals the fact that these two events are stochastically dependent.

Figure 9. Structural Roof Fragility Curves for the Boys' Gymnasium

The main point here is that all the single beams constructed in a roof are somewhat tied together such that they cannot be assumed to be statistically independent. Thus, the influence of all the roof beams is inherently applied as a simply supported beam assumption with bending failure at midspan. In other words, the curves in Figure 9 were based on a single tee beam, but these curves are felt to be representative of the failure probability of the roof. In this study, four separate roof portions were assumed, including the boys' gym roof, girls' gym roof, auditorium roof, and roof for the rest of the building. The total probability of roof structural failure for the school was calculated by the union of roof structural failure for these four parts which are stochastically dependent as discussed throughout. Figure 10 shows the roof structural fragilities for the school building.

Figure 10. Roof Structural Fragility Curve for School Building

6.4. Fragility Analysis of Masonry Walls

As mentioned before, five types of masonry construction, labeled URM1 to URM5, (with properties summarized in Table 5) are considered in this study in order to provide applicability across the range of masonry construction seen in the United States. In order to analyze masonry walls subjected to tornado wind loads, a 0.3 m (1 ft.) strip of wall was assumed as a simply supported beam typical to what is done conventionally in design to wind load for masonry walls. In this study, each non-load-bearing wall and load-bearing wall has four representative models corresponding to each part of the school (i.e. boys' gym, girls' gym, auditorium, and the rest of the building). The school building fragilities for each non-load-bearing wall and load-bearing wall were calculated by the union of the fragilities for these parts.

Figure 11. Unreinforced Masonry Load-Bearing Wall Fragility Curves for the Boys' Gymnasium (5 Different Material Assumptions)

6.5. School Building Fragility Analysis

To this point in the analysis, the failure probability of each damage indicator in Table 10 has been calculated for both an enclosed building and partially enclosed building condition. The individual component failure can then be used to develop the school building fragility curves as defined in Table 10 by MCS and based on the assumption that each damage state is defined as the occurrence of any of the shaded damage indicators in a given row.

Internal pressure in buildings during a tornado is affected by two phenomenon, wind flow into the building and static pressure drop in the tornado vortex. Although the latter case was not explicitly considered in this study, the effect of wind flow on internal pressure was included in the calculation of wind-induced forces by considering different values for the internal pressure coefficient when the building is either enclosed or partially enclosed. As discussed in section 2.2., this is effective only in load calculation using Approach B as well as forces on walls in Approach A. If doors and windows on all sides of the building remain undamaged and closed during a tornado, wind into the building will be minor. Once there is a breach in the building envelope from the failure of a door or a window, or penetration of the roof or walls, the wind pressure in the building increases. In summary, in the current study, the process to generate school building fragility curves in each of the damage states works under the assumption that any failure of doors or windows results in a partially enclosed building condition.

Finally, in Figure 12 to Figure 14, five fragility curve groups for five building types termed URM1 to URM5 based on their masonry material types are presented. Since damage states 1 and 2 are not dependent on the masonry material, which is the only difference for the buildings, all the school buildings have the same fragility curves for damage states 1 and 2. On the other hand, damage states 3 and 4 have the non-load-bearing wall and load-bearing wall as one of their damage indicators; therefore, fragility curves related to these damage states are dependent on the masonry construction, and any changes in masonry material statistics result in a change in their fragility curves. Therefore, one can see in Figure 12 to Figure 14 that the fragility curves for URM3 to URM5 cross each other since the masonry construction does not perform as well as the roof cover. In seismic fragility analysis, this will typically not occur if only one engineering demand parameter is used (e.g. inter-story drift ratio).

Figure 12. School Building Fragility Curves (Approach A) for Different Masonry Material Assumptions

Figure 13. School Building Fragility Curves (Approach B) for Different Masonry Material Assumptions

Figure 14. School Building Type URM5 Fragility Curves

The fragilities presented in this paper can be included in the building stock as a representative archetype school building for community risk and resilience modeling. While there is no way to explicitly confirm the accuracy beyond the physics and mechanics basis of the model and process, some level of validation based on damage observation following tornadoes is possible. The school in Xenia, Ohio was subjected to winds in excess of 90 m/s (200 mph) and its roof was destroyed. Inspection of Figure 10 shows that there is between a 40% and 85% chance of severe damage to the roof system analyzed in this study which was modeled after that school, indicating a likely causal relationship.

It should be noted that eventual comparison to Eurocode is warranted but not included herein because the Haan et al. [14] tornado amplification coefficients used for analysis are only applicable to ASCE 7. Additionally, approximately 75% of tornadoes worldwide occur in the U.S. east of the Rocky Mountains. Therefore, in the interest of brevity this comparison is not included herein.

It is well known that even moderately intense and certainly intense tornadoes form debris clouds at the base resulting in objects becoming missiles. Experience confirms that heavy and large objects such as cars and buses can become debris at high wind speeds and cause significant

damage. On the other hand, light objects can also penetrate the building envelope, resulting in a change in internal pressure. Although windborne debris clearly plays a significant role in tornado fragility assessment of structures, its effects are beyond the scope of the current study and will be investigated in future studies.

6.6. Risk Assessment of School Buildings

The probability of exceeding each damage state is evaluated herein by convolving a tornado hazard curve [53] and the masonry school building fragilities for each damage state. Figure 15 illustrates risk assessment of building type URM3 located in Norman, Oklahoma by convolution integral. The shaded area represents the probability of exceeding damage state 4 in one year which is equal to 0.0126% (i.e. 1.26E-04). Since the hazard curves were developed based on annual probabilities, the P_f would be the exceedance probability per year. Then, 50-year probabilities of exceeding damage states are calculated using Equation 12. Therefore, school building type URM3 located in Norman will exceed damage state 4 with a probability of 0.63% in 50 years which is approximately equal to an 8000-year return period.

Figure 15. Risk Assessment by Convolution of Fragility Curve and Hazard Curve

Damage state exceedance probabilities in 50 years were calculated and tabulated in Table 14 for all combinations of building type and all selected cities, although in theory, some combinations would not occur for modern building codes. As previously mentioned, DS1 and DS2 do not change for the five building types since based on their definition they are not based on masonry materials.

Table 14. School Buildings 50-year Probability of Exceeding each Damage State (in %) for Selected Cities (Approach A Values)

City	Damage State	Building Type				
		URM1	URM2	URM3	URM4	URM5
Norman, OK	DS1	1.46	1.46	1.46	1.46	1.46
	DS2	0.78	0.78	0.78	0.78	0.78
	DS3	0.38	0.55	0.92	1.34	1.94
	DS4	0.07	0.34	0.63	0.96	1.30
Xenia, OH	DS1	0.30	0.30	0.30	0.30	0.30
	DS2	0.13	0.13	0.13	0.13	0.13
	DS3	0.05	0.09	0.18	0.29	0.46
	DS4	0.01	0.05	0.11	0.18	0.26
Omaha, NE	DS1	0.46	0.46	0.46	0.46	0.46
	DS2	0.16	0.16	0.16	0.16	0.16
	DS3	0.05	0.10	0.27	0.45	0.76
	DS4	0.00	0.05	0.14	0.25	0.38
Tampa, FL	DS1	0.20	0.20	0.20	0.20	0.20
	DS2	0.03	0.03	0.03	0.03	0.03
	DS3	3.6E-03	0.02	0.15	0.27	0.56
	DS4	6.0E-05	8.5E-03	0.04	0.08	0.16
Phoenix, AZ	DS1	9.9E-03	9.9E-03	9.9E-03	9.9E-03	9.9E-03
	DS2	1.4E-03	1.4E-03	1.4E-03	1.4E-03	1.4E-03
	DS3	1.5E-04	1.2E-03	7.7E-03	1.4E-02	2.9E-02
	DS4	2.0E-06	4.0E-04	2.2E-03	4.1E-03	7.9E-03

7. Summary and Closure

In this paper, the performance of an archetype masonry school building was investigated using a tornado fragility methodology. The tornado fragility methodology was explained by introducing a tornado-induced wind load calculation procedure similar to that established by ASCE 7-10 for

straight-line wind loads calculation with several modifications. Since tornado loading still possesses significant uncertainty in its formulation as well as researchers understanding of the phenomena, two alternative approaches, i.e. Approach A and Approach B, are used in this study to calculate tornado-induced wind forces on building surfaces. Approach A uses ASCE 7-10 pressure coefficients along with tornado pressure adjustments derived based on the study conducted by Haan et al., while an alternative Approach B is defined in which the tornado pressure adjustments (i.e. T_e and T_i) are set equal to 1.0 and the ASCE 7-16 pressure coefficients were used representing the best knowledge in pressure coefficients currently available for straight-line winds.

Four damage states were defined for school buildings based on performance of building components; including doors and windows, roof cover, roof structural system, load-bearing wall, and non-load-bearing wall. Then, fragilities for each damage states were developed by checking the performance of damage indicators (i.e. the breach of the predefined performance limit states) which are representative of damage states. Moreover, five variants of masonry construction in five different locations across the U.S. that each have a different tornado hazard were investigated in this study in order to provide fragility functions applicable to different locations throughout the United States.

The convolution analysis presented herein provides some insight into why tornadoes have not yet been considered in modern building codes. Inspection of Table 14 shows that the probability of exceeding damage state 4 in 50 years is quite small. However, as the last decade across Tornado Alley and Dixie Alley in the U.S. have shown, it may be necessary to take the consequences into account for these high-consequence low-probability (HCLP) events.

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