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Effect of tornadoes on residential masonry structures

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Abstract. In the early morning hours of February 23rd, 1998, seven large tornadoes ravaged central Florida. A total of 42 people were killed and millions of dollars of damage was done. A strip mall and other commercial structures sustained considerable damage and several residential areas were completely destroyed. Based on field observations, the paper examines the causes and sequence of structural failure for the masonry single family homes. Wind speeds are estimated based on the observed damage, and compared to the meteorological data. Finally, recommendations are given that could help to eliminate or reduce similar failures in the future. It was found that with simple, cost effective measures, most if not all of the damage could have been prevented.

Key words: tornadoes; masonry; damage; wind speed; mitigation; cost; Fujita scale.

1. Central Florida Tornadoes of February 22-23, 1998

During the late night and early morning hours of February 22-23, 1998, three large tornadic supercells traveled eastward across central Florida, spawning the most deadly tornado outbreak in Florida history. The resulting seven tornadoes killed 42 people and injured more than 260 others. More than 3,000 structures were damaged and over 700 were destroyed with damage estimates exceeding $100 million (NOAA/NWS 1998).

The southern supercell produced the longest tornado track of the outbreak, spanning 61 km. The National Weather Service rated the tornado as an F3 on the Fujita scale. Tornado damage was most severe in and around the city of Kissimmee, where 25 people were killed, more than 150 were injured, and over 1000 structures were either damaged or destroyed.

2. Types of structural failure

In the aftermath of the Kissimmee tornado, members of the Wind and Hurricane Impact Research Laboratory at Florida Tech surveyed the damaged structures. Most of the observations were made in the Lakeside Estates residential subdivision, in the commercial Shoppes of Kissimmee, and the Boggy Creek Marketplace/Buenaventura Lakes Shopping Center. The paper focuses on the residential structures made of masonry walls and timber roof systems. The effect of the tornadoes on the commercial structures are reported in another paper (Pinelli, O'Neill. Subramanian, and Leonard 1999).

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2.1. Envelope failures

Window failures were observed throughout the damage path of the tornado. 152 of the more than 300 damaged houses in the Lakeside Estates were observed to have had at least one significant window failure (Fig. 1). Flying debris was assumed to be the primary cause of these failures and none of the observed buildings had any protective devices (i.e., storm shutters) installed on their windows to help mitigate missile impact damage.

Door failures were another commonly observed type of envelope failure. *Garage doors* were by far the most prominent type of door failure (Fig. 2). Throughout the Lakeside Estates, a total of 115 garage doors failed due to both inward- and outward-acting wind pressures as well as air-borne missiles. Local inspections indicated several different failure mechanisms. In many cases, the garage door rollers separated from their tracks when the wind pressure caused large deflections of the garage door. In other cases, the rolling tracks themselves failed, either through torsional failure of the track or by failure of the connection between the track and the garage wall. A third commonly observed failure mechanism was due to the bending failure of the thin, flexible garage door panels, which span relatively long lengths and provide very little inherent bending resistance. The final observed failure mechanism was caused by missile impact on the light gauge metal garage doors. These failures were caused primarily by the impact of timber framing members from failed roof systems.

Interestingly, several cases were observed in which the garage door was the only envelope failure throughout the entire building. This confirmed that garage doors are a primary weak point in the building envelope and can fail at relatively low wind speeds.

Non-structural roof damage was widespread and included *shingle separation*. Although these non-structural damages were not directly detrimental to structural integrity, they created substantial amounts of air-borne debris which increased the risk to human life and also led to further structural and non-structural damages.

Fig. 1 Window failures in the Lakeside Estates

Fig. 2 Garage door failures in the Lakeside Estates

Another commonly observed type of envelope failure was the *separation of roof sheathing from the roof framing members*. Roof systems on the residential structures were comprised of structural plywood panels nailed to light-frame wood trusses. A total of 125 roof sheathing failures were observed in the Lakeside Estates subdivision. Positive internal pressures and large uplift wind forces acting on the roof caused these separations. These failures often occurred at the roof ridge, eaves and corners where localized wind effects were greatest. A number of possible causes led to the roof sheathing separation, including insufficient nailing, poor construction practices (i.e., missing the truss member when nailing through the plywood panel, etc.) or shoddy construction materials. In particular, inspections of damaged structures revealed that in most cases the fastening schedule did not meet the SBC (SBCC 1994) minimum fastening requirements.

2.2. Roof structure failures

Most of the tornado's structural roof damage was a result of either *insufficient lateral bracing* or *the failure of critical roof connections*. Wooden roof systems acted as horizontal diaphragms which transferred lateral loads to the masonry shearwalls. In addition to transferring shear loads, the

Fig. 3 Deflection of wooden roof trusses due to inadequate lateral bracing (Lakeside Estates)

plywood panels often provided the only lateral bracing to the wooden roof trusses. If the plywood sheathing separated from the roof system, the roof trusses deflected laterally and the system no longer provided any resistance to high wind pressures (Fig. 3).

The two most commonly observed roof connection failures occurred at the *roof-to-wall connections* and at the *gable end connections*. A large majority of the wood truss to wall connections were made with hurricane clips embedded into the masonry wall and nailed to the wooden roof trusses. In general, this type of connection proved adequate in resisting uplift wind forces as long as the roof system itself remained intact. If, however, the plywood panels separated from the roof and the trusses deflected laterally, the hurricane clip connections frequently failed through withdrawal of the nails connecting the clip to the truss. Failures of the wood trusses or the hurricane clips themselves were seldom observed.

Gable end connection failures were observed throughout the Lakeside Estates. Gable ends are particularly susceptible to high winds due to the severe localized effects that act on that part of the building. Despite this, common construction practices often provide little or no additional anchorage at the gable ends to help resist these increased wind loads.

2.3. Masonry wall failures

Masonry wall failures were observed in all damage areas and were generally the result of either insufficient anchorage or shear/bending failures of the wall itself. In short, all of the observed failures were due to a *lack of vertical and horizontal reinforcement* in the wall. In addition to the

Fig. 4 Insufficient wall to foundation anchorage in the Lakeside Estates

lack of reinforcement, none of the observed residential masonry structures had any of their cells grouted with concrete.

Several failures were observed in the Lakeside Estates. Some failures were a direct result of insufficient anchorage between adjacent walls as well as between the wall and the foundation (Fig. 4). Wind pressures acting normal to the wall caused these failures, and they generally occurred after the wall lost lateral stability from either the roof diaphragm or adjoining walls. Preserving the integrity of the structural system, including the roof, and simply designing with sufficient amounts of steel reinforcement could prevent these failures.

Masonry wall shear failures were also commonly observed throughout the damage area (Fig. 5). Shear failures were the result of in-plane wind pressures which the roof diaphragm transferred to the wall. The complete lack of horizontal reinforcement significantly reduced the shear capacity of the

Fig. 5 Shear failure of a masonry wall in the Lakeside Estates

Fig. 6 Bending failure of an unreinforced masonry wall in the Lakeside Estates. Notice that the masonry wall failure caused the tie beam to sag which subsequently led to the collapse of the roof

masonry walls and was the primary cause of the shear failures that were observed.

Bending failures were the final type of masonry wall failures that were commonly observed (Fig. 6). Wind loads acting normal to the wall caused these failures. Since no vertical reinforcement was provided, the walls were very brittle and failed at relatively low wind speeds.

3. Sequence of structural failures

From the observations, it was evident that most of the damaged wood-roofed buildings followed one of two failure sequences. The particular sequence in any specific structure depended on several factors including the orientation of wind flow with respect to the structure, construction practices/materials used and the relative strengths of the individual structural elements. The first of the two common failure sequences was as follows (see Fig. 7):

1. Window and door (especially garage door) failures resulted in positive internal pressures.
2. Large positive internal pressures combined with negative (suction) external wind pressures caused side walls to collapse due to bending failure. This collapse removed support to the roof and reduced the capacity of the tie beam to support the roof structure.
3. Reinforced concrete tie beam sagged or failed under the increased load and the roof collapsed.

Mitigation of the first sequence lies in strong garage doors and reinforced walls. The upper photograph in Fig. 2 is a particularly good illustration of this point. The houses on both sides of the home pictured there were destroyed. The house in question had also its garage door blown away like its neighbors. However, the owner had closets added all around the perimeter of the garage walls. This unexpected reinforcement of the walls gave them extra strength. They did not collapse and the home suffered only minor damage.

The second commonly observed failure sequence in wood-roofed structures was as follows (Fig. 8):

1. Window and door failures resulted in positive internal pressures.
2. Uplift wind pressures, combined with positive internal pressures, caused the plywood roof sheathing to separate and gable end connections to fail. These failures eliminated bracing to the roof trusses.
3. Roof trusses deflected laterally causing failure of the roof to wall connections. The trusses separated from the structure, removing roof support to the walls.
4. Loss of support from the roof system caused the windward masonry wall to act like a vertical cantilever, leading to large bending stresses at the wall to wall and wall to foundation connections. These stresses caused the connections to fail which lead to the inward collapse of the windward

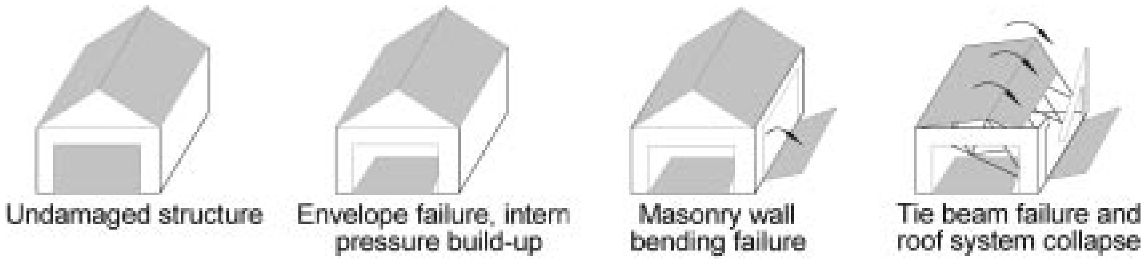

Fig. 7 First common failure sequence

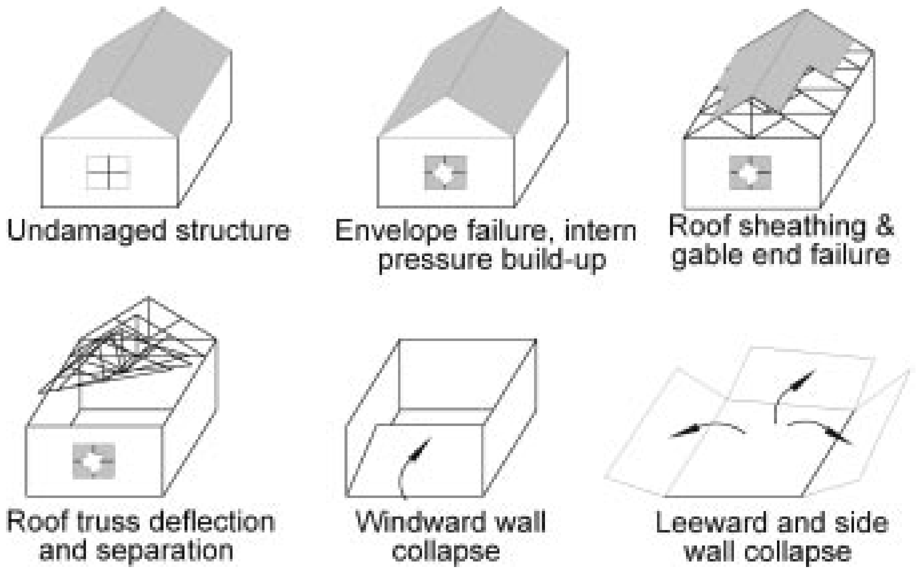

Fig. 8 Second common failure sequence

wall.

5. Loss of support from windward wall caused leeward and side walls to collapse outward due to negative (suction) wind pressures.

Regardless of the specific failure sequence, the following structural cause/effect relationships hold true for all masonry structures:

- Breaching of the buildings envelope significantly increases the net pressures acting on both the roof system and walls and is commonly the initiating cause of other structural failures.
- The roof system is both supported by and provides bracing and stiffness to the structural system as a whole; failure of the roof increases the stresses acting on the remaining structure, specifically on the walls braced by the failing roof.
- Masonry walls provide and receive bracing and stiffness from each other and the roof system; the failure of one wall both increases the stresses acting on adjacent walls and removes support to the roof system.

This indicates that an effective tornado-resistant design can only be achieved if the structural integrity remains intact throughout the entire building. When applying this idea to the load path concept, it is clear that when one element in the path fails, a new load path will result. Therefore, a tornado-resistant design should provide alternate, or redundant, load paths in areas where structural failures might be expected. If one element fails, members in the redundant path should have the capacity to accept the additional load and transfer it through the structure and into the ground.

4. Case studies

In order to quantify some of the impacts of the Kissimmee tornado, structural and economic analyses were performed on three specific buildings that were damaged in the Lakeside Estates. The first case illustrates an envelope failure with no other damages, the second case illustrates an envelope failure with roof structure damage, and the third case illustrates an envelope failure, roof structure damage and masonry wall collapse. The intent of these analyses was twofold: to determine the specific failure sequences and critical wind speeds and to calculate the cost of implementing mitigation measures.

4.1. Case study 1

Case Study 1 suffered envelope failures. In the entire Lakeside Estates housing area, about two-thirds of the affected structures suffered similar damages. The house in question was a 116 m^2, single-story residence with a single car garage that was incorporated into the main structure. The tornado struck the building from the rear and caused the failure of two glazed openings on the windward wall and the separation of roof sheathing on the windward and leeward eaves as well as along the leeward ridge. Fig. 9 illustrates the extent of the failures.

4.2. Case study 2

Case Study 2 was located in the subdivision that suffered the worst tornado damage in the entire Lakeside Estates area. It was a 153 m^2, single-story masonry residence with a two-car garage that extended out from the main structure and a roof system similar to that in Case Study 1. The tornado struck the house diagonally from the southwest and caused substantial envelope failures as well as roof structural failures. Every glazed opening (6 windows and a sliding glass door), approximately half of the roof sheathing, the garage door, the roof structure over the garage and the gable ends all failed. Additionally, the interior of the structure was completely destroyed. Fig. 10 shows two views of the building which illustrate the extent of the structural damage.

4.3. Case study 3

Case Study 3 was a 161 m^2 house located less than 150 m north-east of Case Study 2. It suffered

Fig. 9 Rear/side (weat/south) view of Case Study 1 showing window failure and roof sheathing separation at the eaves and along the ridge

Fig. 10 Views of the south side (top) and north side (bottom) of Case Study 2 showing garage door, roof sheathing and roof structure damages

massive damage consistent with a number of completely destroyed structures in the Flamingo Lakes subdivision. The tornado struck the building from the south-west, breaching the envelope and causing both roof structure and masonry wall failures. Additionally, the contents and interior of the structure were completely destroyed. Fig. 11 shows two views of Case Study 3 which illustrate the extent of structural damages.

5. Estimated wind speeds

After inspecting the construction details of the structure, and observing the corresponding damages, detailed engineering analysis resulted in the following failure sequences and critical wind

Fig. 11 Front view (top) and side view (bottom) of Case Study 3 showing envelope failures, roof structure failures and masonry wall collapse

speeds. In each case, the material properties were obtained from the corresponding manuals or codes. For example, the 8'' masonry blocks were assumed to have an $f'_m = 10300$ kPa (1500 psi), with type S mortar. The nails were assumed to be 8d common. The trusses were assumed to be Southern Pine. The structural calculations were based on the masonry code ACI 530-95, the National Design Specification for Wood Construction [NDS-1997], and the Standard Building Code [SBCCI-1994]. The corresponding wind pressure at failure was estimated based on tributary areas, and the corresponding 3 sec gust wind speed was computed based on the recommendations of ANSI/ASCE 7-95, Minimum Design Loads for Buildings and Other Structures [ASCE 7-95]. Exposure B and standard occupancy were assumed, and the procedures for building of all heights, and for component and cladding of building less than 18 m (60') in height were used. The detail of the calculations can be found in (O'Neill and Pinelli 1998).

With respect to the windows, residential window glazing is commonly 6 mm (1/4 inch) thick and it usually uses glass that is either annealed, heat strengthened or fully tempered. Unprotected glazing usually fails due to the impact of small air-borne missiles (i.e., roof gravel, shingles, etc.). Research by Minor [1994] has shown that all three common types of residential glazing can fail due to air-borne missile impacts at wind speeds of 35 m/s (75 mph) or less. The modes of glazing failures observed in all three case studies were consistent with air-borne missile impacts. Therefore, although the specific type of glazing used in the case studies is unknown, it is conservatively assumed that the glazed openings on the windward surface were the first elements to fail at wind speeds of 34 m/s (75 mph) or less.

5.1. Case study 1

The failure sequence is shown in Fig. 12.

1. Windward glazed openings failed through missile impacts at a wind speed of about 35 m/s (75 mph) or less;
2. Overhang roof panels on both the north and south eaves separated at a wind speed of about 40 m/s (90 mph)
3. Non-overhang panels along the west eave and panels on the south side of the ridge separated at a wind speed of about 45 m/s (100 mph). The upper limit wind speed was calculated to be between 45 m/s and 55 m/s.

Table 1 shows how the authors came up with a wind speed of 40 m/s for the overhang roof panels. It is representative of all the other wind speed calculations.

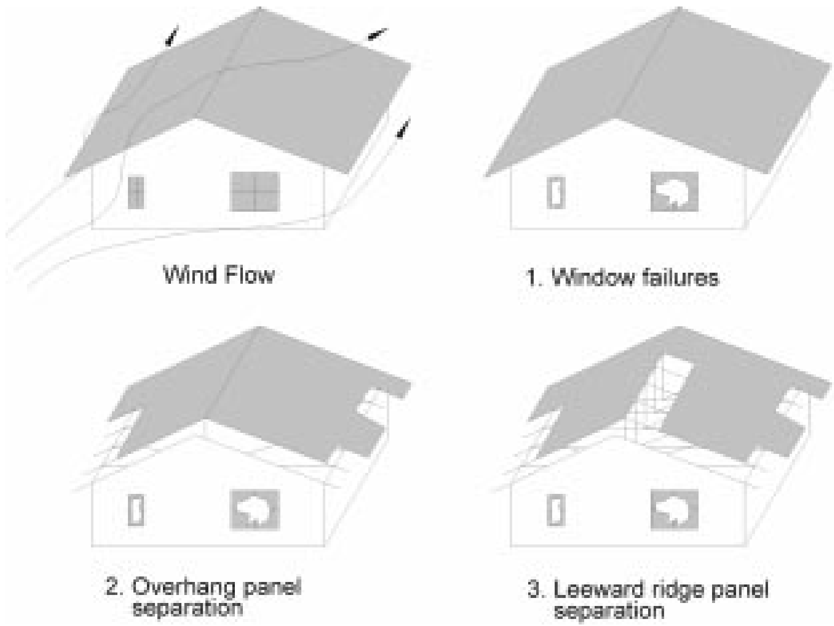

Fig. 12 Failure sequence, Case Study 1

Table 1 Sample wind speed calculations for roofs

Material	Codes	Equations leading to $V = 40$ m/s
Southern Pine Truss Members	**NDS-97**	W=1380 $G^{5/2}D$=35lb/in (6 kN/m)
Southern Pine roof sheathing	G=0.55; C_D=1.6;	$W'=W\ C_D C_M C_t C_{tn}$=56lb/in (9.8kN/m)
(3/4''-CDX)	$C_M=C_t=C_{tn}$=1	Withdrawal strength=98lb/nail (0.4kN)
2.5'' (65 mm) 8d common nails	**ASCE 7-95**	Resistance =1470lb/panel (6.5kN)
24'' (61mm) o/c nail spacing	K_z=0.85; K_{zt}=1; I=1.0	p_{max}=1470/32=46 psf (2.2 kPa)
(15 nails fasten each panel)	GC_p=-2.2; GC_{pi}=0.8	$p=q_h[(GC_p)-(GC_{pi})]$
nail penetration = 1.75'' (45 mm)	Tributary area = 32 ft^2 (3 m^2)	$q_h=0.00256K_zK_{zt}V^2I$

5.2. Case study 2

The failure sequence is shown in Fig. 13.

1. Glazed openings failed through missile impacts at a wind speed of about 35 m/s (75 mph) or less;
2. Garage door failed through withdrawal of the running track anchor bolts at a wind speed of about 35 m/s (80 mph);
3. Roof sheathing over the garage separated at a wind speed of about 40 m/s (90 mph), causing the roof trusses over the garage to deflect laterally;
4. Overhang roof sheathing along the southern, western and northern eaves separated at a wind speed of about 40 m/s (90 mph);
5. Separation of the southern eave sheathing caused the south gable end to fail due to inward acting wind pressures;
6. Roof sheathing on the northern half of the leeward roof surface separated at wind speeds between 40 m/s and 55 m/s (the upper limit wind speed was calculated to be approximately 55

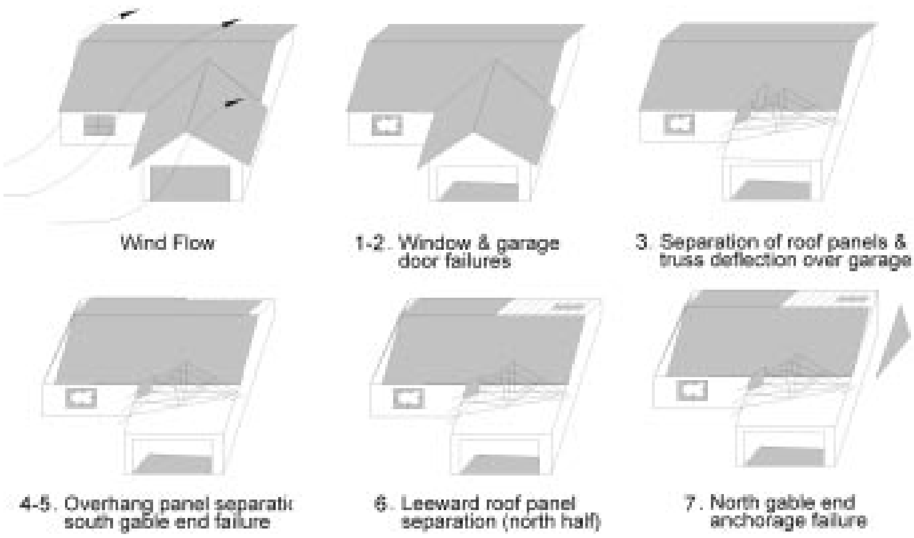

Fig. 13 Failure sequence, Case Study 2

m/s.);

7. North gable end failed due to suction wind pressures at a wind speed of about 45 m/s (100 mph);

5.3. Case study 3

The failure sequence is shown in Fig. 14.

1. Glazed openings failed through missile impacts at a wind speed of about 35 m/s or less;
2. Garage door failed inward at a wind speed of about 35 m/s;
3. Roof sheathing over the garage separated at a wind speed of about 41 m/s, causing the roof trusses over the garage to deflect laterally;
4. Overhang roof sheathing along roof eaves separated at a wind speed of about 40 m/s;
5. Roof sheathing along the ridge separated at a wind speed of about 45 m/s;
6. North masonry garage wall collapsed at a wind speed of about 50 m/s;
7. Roof sheathing in the interior of the roof surface separated at a wind speed of about 54 m/s.

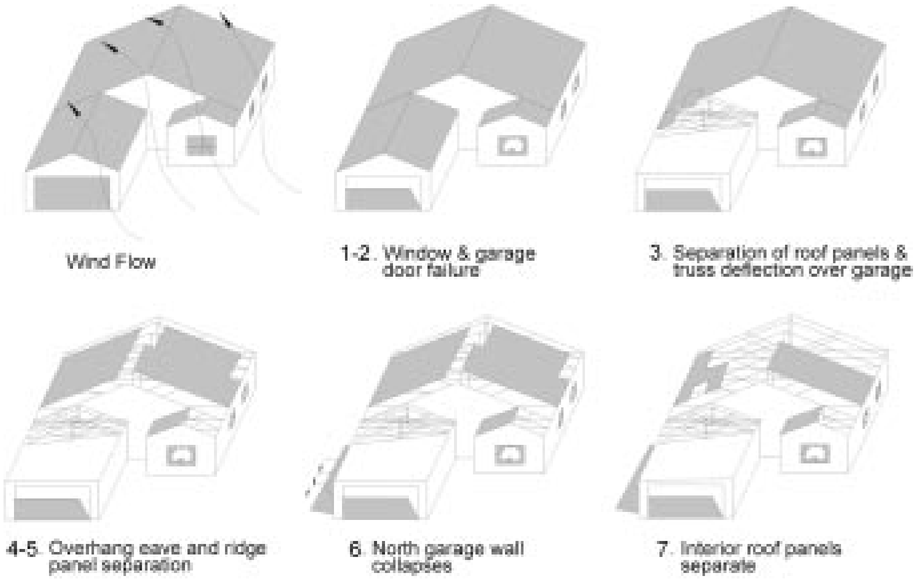

Fig. 14 Failure sequence, Case Study 3

Table 2 Sample wind speed calculations for masonry

Assumptions / Materials	Codes	Equations leading to $V = 50$ m/s
8" unreinforced, ungrouted masonry	**ACI 530:**	$f_{net} = f_b + f_{wall\ weight} = 29.1$ psi (200 kPa)
$f_m = 10300$ kPa (1500 psi)	$f_b = 25$ psi (170 kPa)	$M_{max} = f_{net} * S_x = 2525$ lb-in/ft (935 N-m/m)
type "S" mortar	**ASCE 7-95:**	$p_{max} = 8M_{max}/L^2 = 26.7$ psf (1.3 kPa)
full mortar bedding ($A_{net} = 41.5$ in^2 or 268 cm^2; Sx = 86.7in^3 or 1421 cm^3)	$K_z = 0.57$; $K_{zt} = 1.0$; $I = 1.0$	$p_{max} = q_H[(GC_p) - (GC_{pi})]$
wall height = 8 ft (2.44 m)	$G = 0.8$; $C_p = -0.7$; $GC_{pi} = 0.8$	$q_h = 0.00256 K_z K_{zt} V^2 I$
wall weight = 42 psf (205 kg/m^2)		

Table 2 illustrates how the authors calculated a failure wind speed of 50 m/s for the collapse of the north masonry garage wall in Case Study 3.

The observations therefore yielded an upper limit mean wind speed between 55 and 60 m/s. Obviously, these values are subject to error. The assumed material properties were mean values, and more importantly the code equations used to backtrack the wind speeds (see Tables 1 and 2) also yielded mean values. But even considering a possible 20% error, the calculated upper limit wind speeds are below the estimated maximum tornado wind speed of between 71-93 m/s (F3 tornado), issued by the National Weather Service (NOAA/NWS 1998). This discrepancy is not surprising, however, because the NWS estimate is based upon general and subjective observations of the most severe damage rather than detailed engineering analyses of particular structures. Other investigators have also shown that the calculated wind speeds based on engineering analysis are in general lower than the estimated wind speeds from the NWS (Phan & Simiu 1998, Mehta and Carter 1999). It is clear that the amount and severity of the damage is linked not only to the magnitude of the wind speed, but also to the quality of the constructions. The quality of the construction will in turn depend on the given design wind speed, on the quality of the workmanship, and the adherence to, and enforcement of minimum building code standards. Therefore, the usefulness of the Fujita scale could be greatly enhanced if it were re-assessed to take into account these variables.

6. Mitigation

6.1. Design wind speed

The American Nuclear Society tornado wind speed for the state of Florida is 67 m/s or 150 mph (based on a mean recurrence interval of 100,000 years, which is overly conservative for the design of most standard structures) (ASCE7 1995). According to studies by Twisdale (1978), there is less than a 3% chance of wind speeds exceeding 68 m/s, given a tornado occurrence in a high risk tornado region, like the NRC tornado region 1 in the US. More specifically, in Florida, only 1.5% of all tornadoes over the last 45 years have been ranked F3 or higher (max. wind speeds $\geq$ 71 m/s). Assuming that maximum wind speeds occur over 20% of the path area (a very conservative assumption), this indicates that in Florida only a very small percentage of the total area subjected to tornadoes suffers wind speeds above 68 m/s. In view of the above, and of the calculated wind velocities in the previous section, a wind speed of 68 m/s (150 mph) appears to be a more than reasonable upper bound for a design wind speed. In fact, an even lower design wind speed of 55 m/s (120 mph) would be both socially acceptable and more economical. For the sake of conservatism the upper value of 68 m/s was retained in this study.

6.2. Design recommendations

Despite the significant damages observed, especially in case studies 2 and 3, taking a few steps to strengthen the building during construction could have mitigated all of the major structural failures. The envelope would have been protected in winds of up to at least 67 m/s if storm shutters, certified for impact resistance, had been installed over all glazed openings, a 67 m/s -rated garage door had been used, the roof trusses had been blocked and the code minimum fastening schedule had been adhered to. Since the calculated upper limit wind speed was well below 68 m/s (150 mph), these steps would have been adequate to protect the envelope in all three case studies. Evidently, the shutters must be rolled down to be effective, and therefore the purchase of a severe weather alert radio should also be required.

Since the envelope would not have been breached, the internal pressure build-up would have been avoided and internal damages would have been avoided. Additionally, by keeping the roof sheathing intact, the gable ends would have been anchored sufficiently to resist higher wind speeds. This would have obviously been sufficient to mitigate the observed gable end failures.

Several other structural improvements could also be implemented. These improvements include using hurricane straps instead of clips (to increase the lateral or withdrawal resistance of the connection), anchoring the bottom chord of the gable ends and providing steel reinforcement in the exterior masonry walls.

6.3. Cost

A cost analysis of the proposed mitigation measures in each of the case studies shows that the additional cost of the improvements will be below US $5,000 (see Table 3).

That includes the cost of rolling shutters, a wind-rated garage door (compliant for example with the South Florida Building Code [SFBC 1998]), blocking the roof trusses, enforcing the minimum nailing code schedule for the roof panels (according to the Standard Building Code [SBCCI, 1994]), hurricane straps for the roof to wall connections, anchor straps for the gable ends, and masonry wall reinforcement. Most of the construction costs were calculated using the National Construction Estimator® software [1996] with the exception of the shutter costs and garage door costs which were obtained directly from local suppliers.

This cost should be compared to the cost of failure and reconstruction, including the cost of demolition, the cost of replacing the buildings contents and the cost of temporary shelter. It could be argued though that the risk for a given structure of being struck by a tornado is fairly low, and that therefore the extra cost would not be justified. However, the proposed mitigation measures serve the dual purpose of tornado and hurricane hazard mitigation. The likelihood for any given structure in Florida of being subjected to hurricane force winds is much higher. Therefore, the proposed mitigation measures could be enforced throughout the state of Florida for a very low cost/benefit ratio, if the benefits from hurricane mitigation are also included in the equation.

Table 3 Cost of improvements for case study 3

Recommended Improvement	Cost
Rolling shutters on all glazed openings, and severe weather alert radio	$2500
150 mph rated, SFBC approved doubled garage door	$1000
Blocking wooden roof trusses	$500
Using SBC minimum fastening schedule for roof panels (6'' o/c edge, 12'' o/c intermediate)	$100
Hurricane straps for roof/wall connection	$140
Anchor straps for gable end	$130
Vertical wall reinforcement and grouting	$430
Horizontal wall reinforcement	$100
Total cost of improvements =	$4900

7. Conclusions

The single most important step that can be taken to help mitigate tornado damages is to protect the structural envelope, which includes glazed openings, garage doors and roof sheathing. If the envelope remains intact, wind pressures throughout the building are effectively halved, interior damage is reduced and occupant safety is significantly increased. Strengthening the building envelope to withstand almost all Florida tornadoes, for a design wind speed of 68 m/s, can easily be achieved using standard construction methods like installing storm shutters and blocking the roof trusses, which increase the total cost of construction by only a few percentage points.

Structural and economic analyses on three masonry buildings, damaged by the Central Florida tornadoes, confirmed the effectiveness of these simple mitigation measures. It was shown that all of the observed structural damages could have been mitigated had recommended improvements been applied. They include: blocking of the roof trusses, gable end anchorage, minimum fastening schedule for the roof sheathing, improved roof to wall connections, vertical and horizontal wall reinforcement. The economic cost/benefit analyses showed that the recommended improvements would have been financially beneficial in all cases, regardless of the amount of damage induced. Additionally, the economic and structural benefits of the recommendations are even more pronounced in Florida since they would be equally effective in mitigating hurricane-induced damages.

The analyses also showed that the maximum wind speeds during the event were probably below the wind speed corresponding to the assigned classification of an F3 tornado. It is recommended that the tornado scale be reassessed to take into account engineering parameters like the local design wind speed and the current building code and practice to better correlate observed damage to tornado wind speed.

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(*Communicated by Ahsan Kareem*)