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National Wind Institute™

MAY 20, 2013 MOORE, OK, TORNADO INVESTIGATION REPORT

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PART I: PERFORMANCE EVALUATION OF STORM SHELTERS

Introduction

Prior to the May 20, 2013 tornado, Moore, Oklahoma, was also hit by two severe tornadoes on May 3, 1999 and May 8, 2003. All three tornado tracks can be seen in *Figure 1* where the May 3, 1999 path is indicated in red, the May 8, 2003 path is indicated in blue, and the May 20, 2013 path is indicated in green. There are slight overlaps of the May 3rd tornado path with both the May 8th and May 20th paths. *Figure 2* shows the population density of the city at the time of the May 20, 2013 tornado.

Above and below-ground shelters were inspected after the May 3rd, 1999 tornado and it was found that some met or exceeded the Federal Emergency Management Agency (FEMA) 3-20 *Taking Shelter from the Storm*, while others failed in compliance (Gardner *et al.* 2000). The most notable deviation from the guidelines appeared to be sub-standard door construction and inadequate locking mechanisms. Instances of door failure and damaged ventilation components were recorded. Storm shelter maintenance also proved to be major contributing factors to failures, including deterioration and rusted hinges and latches due to improper waterproofing and painting. By the May 8, 2003 tornado, the number of shelters in Moore had increased thanks to the FEMA shelter incentive grant program that offsets part of the cost of installing a shelter for homeowners. It must be noted that many newer shelters, including several above-ground shelters, were not in the tornado path (Kiesling and Tanner 2003).

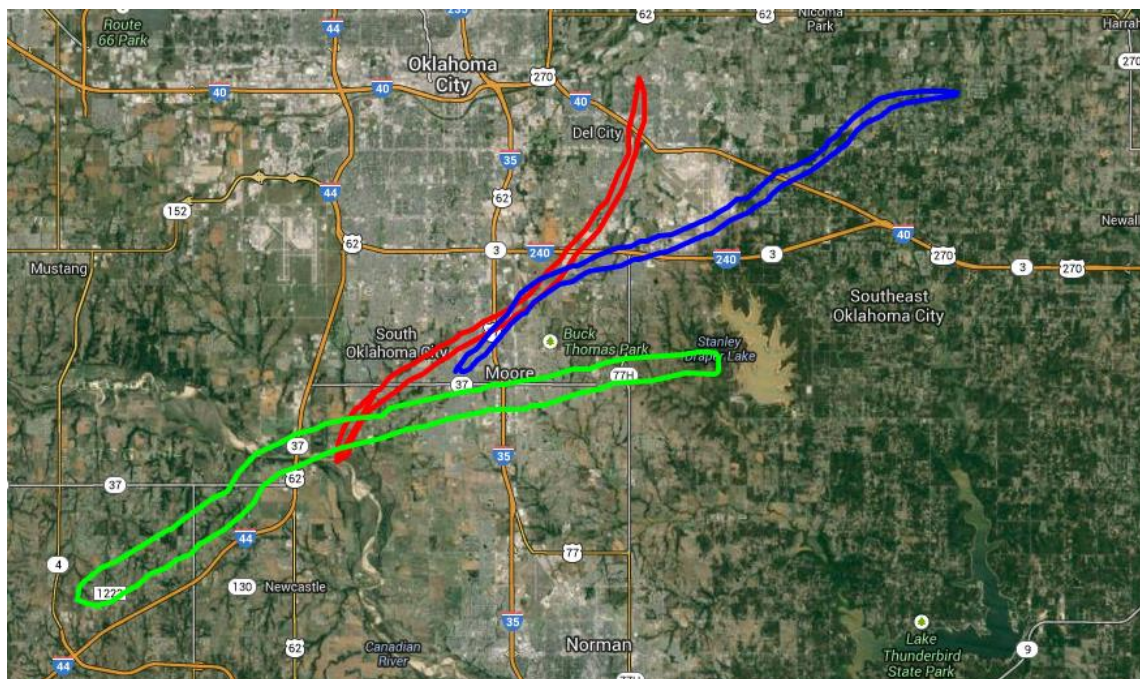


Figure 1.1. An overlay of three tornado paths through Moore, OK. Red is the May 3, 1999 path, blue is the May 8, 2003 path, and green is the May 20, 2013 path (Source: WDT, Inc.)

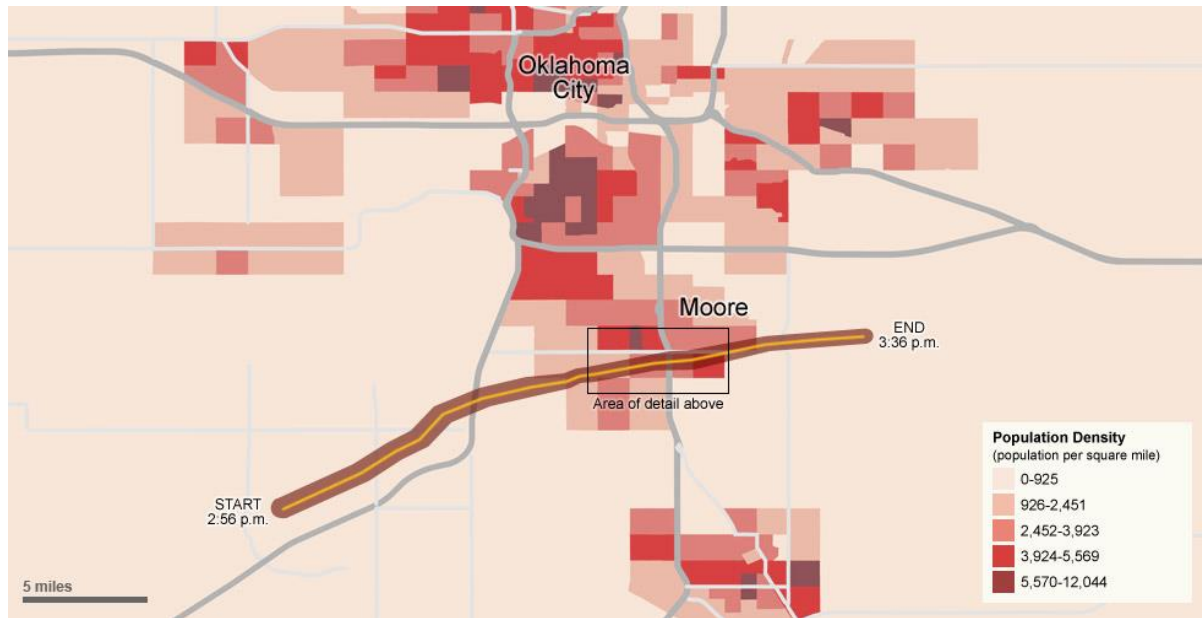


Figure 1.2. Population density in Moore and Oklahoma City, Oklahoma (Source: URS).

EF-Scale

The Enhanced Fujita Scale (EF-Scale) was developed at Texas Tech University to rate intensity of tornadoes before being adopted by the National Weather Service in February 2007 (WISE 2006). There are 28 Damage Indicators (DI), 23 of which are building types ranging from small barns (SBO), to automobile showroom (ASR), and strip mall (SM). For each DI, a set of Degree of Damage (DOD) is identified to describe the level of damage sustained. For One and Two-Family Residence (FR12), the Degrees of Damage are numbered between 1 (threshold of visible damage) to 10 (slab wept clean). In addition, three wind speeds are estimated for each DOD to account for inherent uncertainties in construction quality, design, and maintenance: expected wind speed (EXP), lower bound wind speed (LB), and upper bound wind speed (UB). For example, EXP, LB and UB wind speeds causing DOD5 of FR12 (i.e. entire house shift off foundation) are 121mph, 103mph, and 141mph respectively. Meanwhile, a range of wind speed is defined to rate a specific tornado intensity: EF0 (65-85mph), EF-1 (86-110mph), EF-2 (111-135mph), EF-3 (136-165mph), EF-4 (166-200mph) and EF-5 (>200mph). Wind speed estimation was provided by a panel of experts following the elicitation protocol established by the Senior Seismic Hazard Assessment Committee (SSHAC, 1997). Therefore, correlation of observed damage with wind speeds that caused the damage is important in understanding the uncertainties (variation) in the observed damage at a location and in calibrating/verifying the wind speeds in the EF-Scale.

An EF-rating was assigned to each shelter site and is shown on the maps in *Figures 3, 4 and 5* with a corresponding legend in *Tables 1 and 2*. Some of these ratings may have differed from the National Weather Service (NWS) ratings because they are based on the condition of each specific site, whereas the NWS assigns a rating based on the highest wind speed in the entire damage path.

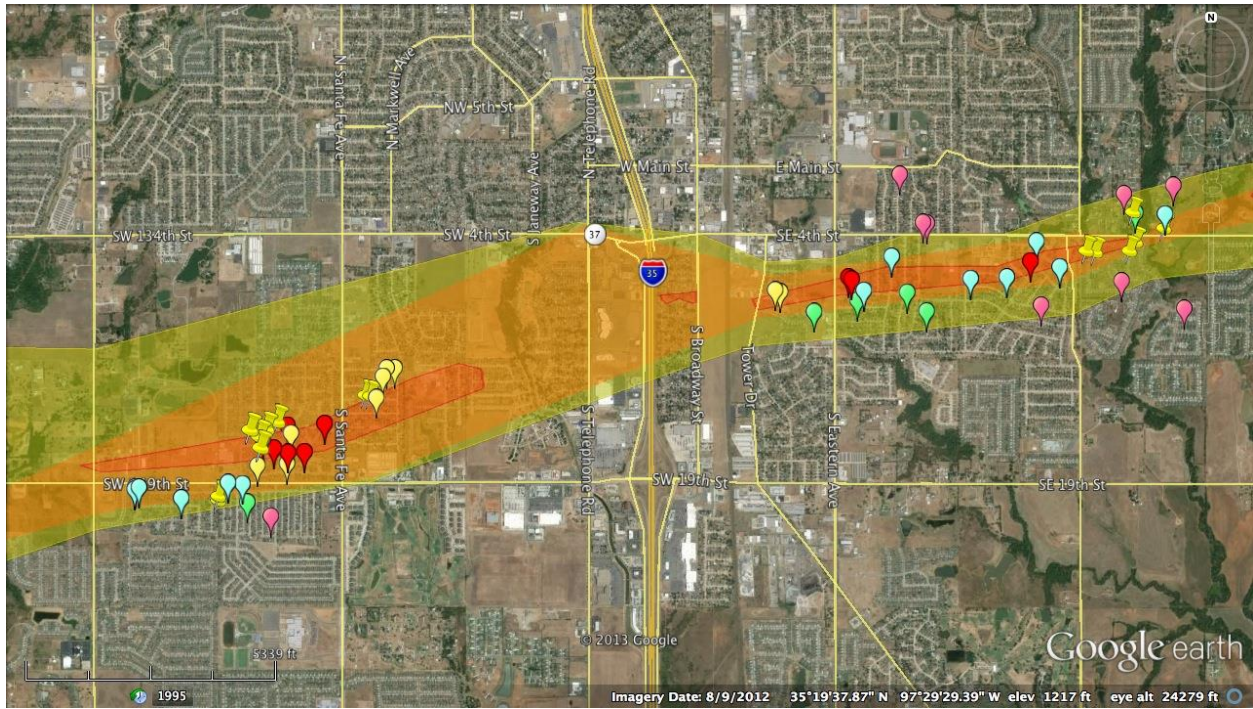


Figure 1.3. EF ratings from shelter visits Sunday, May 26, 2013.

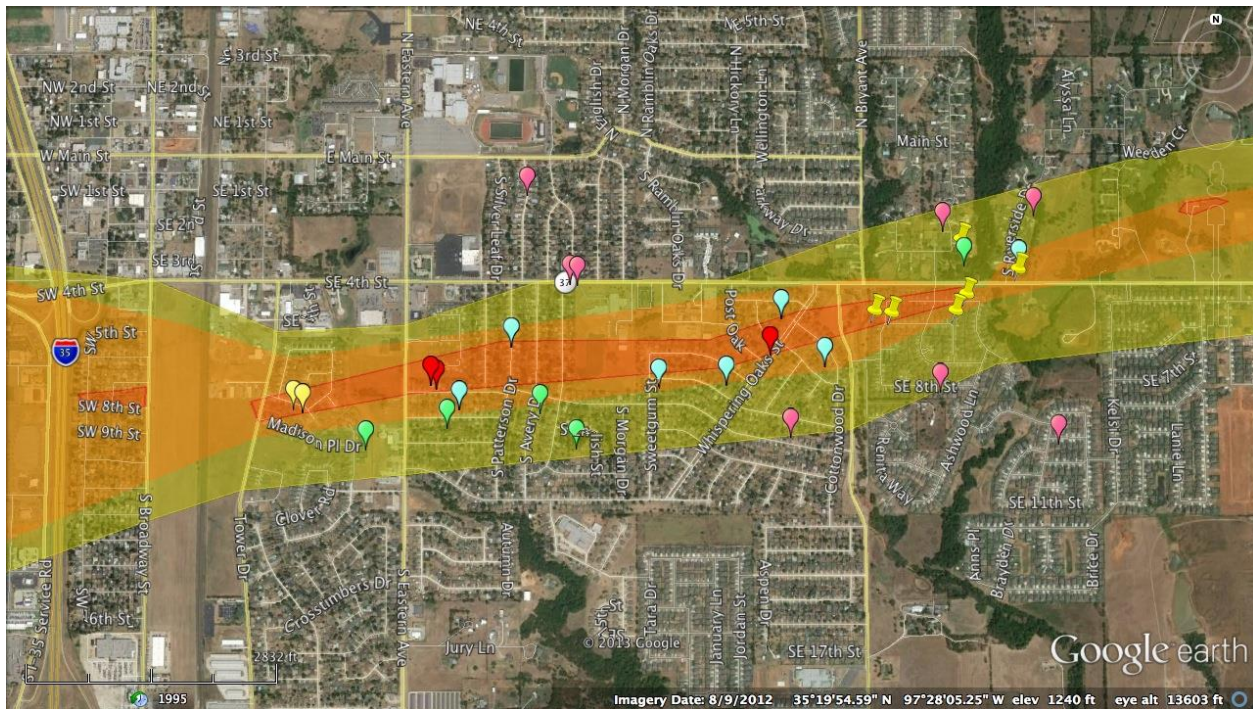


Figure 1.4. EF ratings from shelter visits Monday, May 27, 2013.

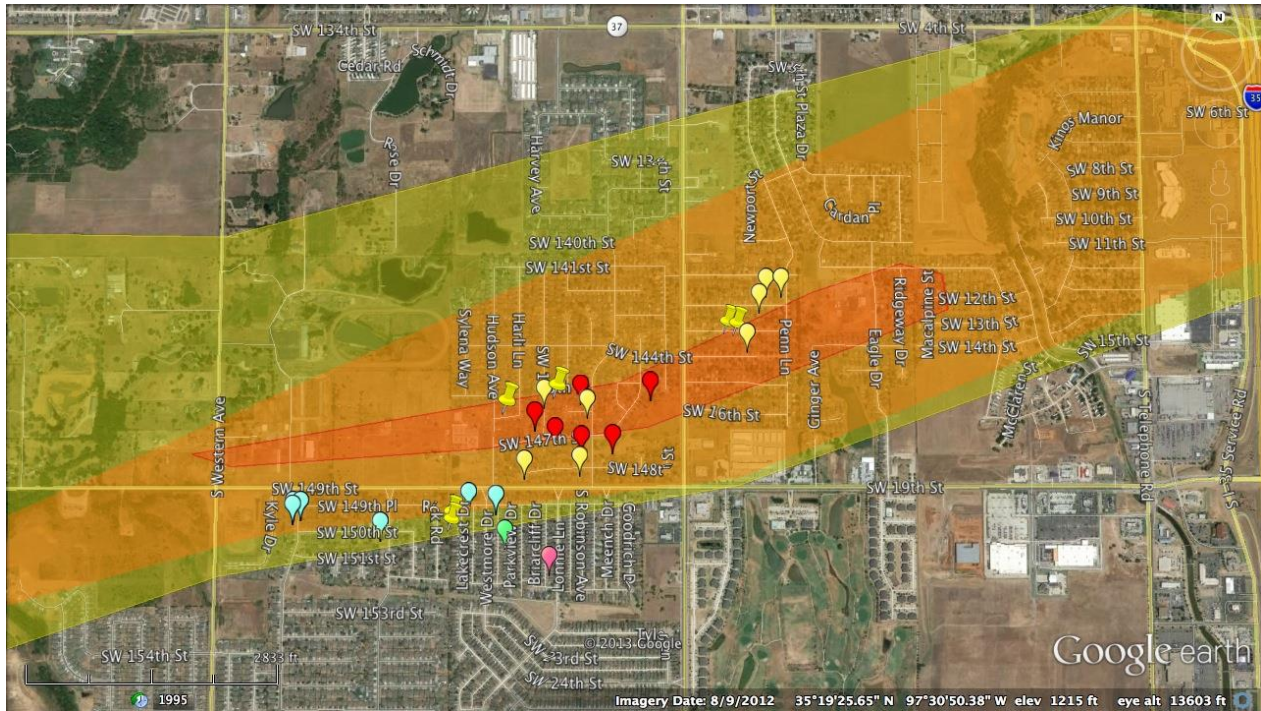


Figure 1.5. EF ratings from shelter visits Tuesday, May 28, 2013.

Table 1.1. EF rating given by Texas Tech team at specific location.

Symbol	On-Site Storm Strength Assessment
	EF-0
	EF-1
	EF-2
	EF-3
	EF-4
	Information missing

Table 1.2. EF rating given by the National Weather Service for the areas shaded in Figures 3, 4, and 5.

Color Fill	NWS Storm Rating
Yellow	EF-0
Orange	EF-2
Red	EF-4

Objective

The objective of the post-storm investigation was to assemble data on storm shelter performance within the path and close proximity of the tornado. This will establish the validity of the use of storm shelters, both above and below-ground, and to aid in the identification of issues with shelter designs, construction techniques, and materials that need to be addressed. The team mapped the track of the tornado and overlaid the locations of storm shelters from the FEMA database, the National Storm Shelter Association (NSSA) database, and the Oklahoma Sooner Safe Room Association (OUSSRA) database. All storm shelters that did not appear in or near the tornado path were deleted off the list of potential site visits, and several storm shelters that were not in any of the databases were found amongst the debris. Some storm shelters were tagged with incorrect GPS coordinates and/or addresses in the databases, and other difficulties in locating the documented storm shelters arose as a result of the destruction of street signs and landmarks.

The team assessed the storm shelters by first evaluating their performance based on observation of their current condition, and each was then further documented by size, manufacturer, and date of installation if available. Then they were cross-referenced with the databases to determine if the shelter was tested per guidelines from the International Code Council, National Storm Shelter Association, and Federal Emergency Management Agency (FEMA 2008, FEMA 2008, ICC and NSSA 2008). An EF rating for the specific site of the storm shelter was also assigned.

Shelter Databases and Survey Zones

Prior to team deployment, three databases containing the locations of storm shelters and safe rooms were acquired from FEMA, NSSA and the OUSSRA. The NSSA database included addresses for 236 shelters in Moore, OK, fewer than 80 of which were located within the tornado track. The FEMA database included addresses and GPS coordinates for 96 shelters and safe rooms, all of which were within the track. The OUSSRA database contained addresses for 98 shelters, and less than 40 were in the track. Based on shelter densities and geographic delimiters, Moore was divided into three Zones to facilitate logistics (*Figure 6*). Zone 1 contained shelters and safe rooms west of Briarwood Elementary, including areas where the 1999 and 2013 tornado paths crossed as seen in *Figure 1*. Zone 2 contained shelters and safe rooms east of Briarwood Elementary to I-35. Zone 3 contained all of the shelters and safe rooms east of I-35. No community shelters were listed in any of the databases.

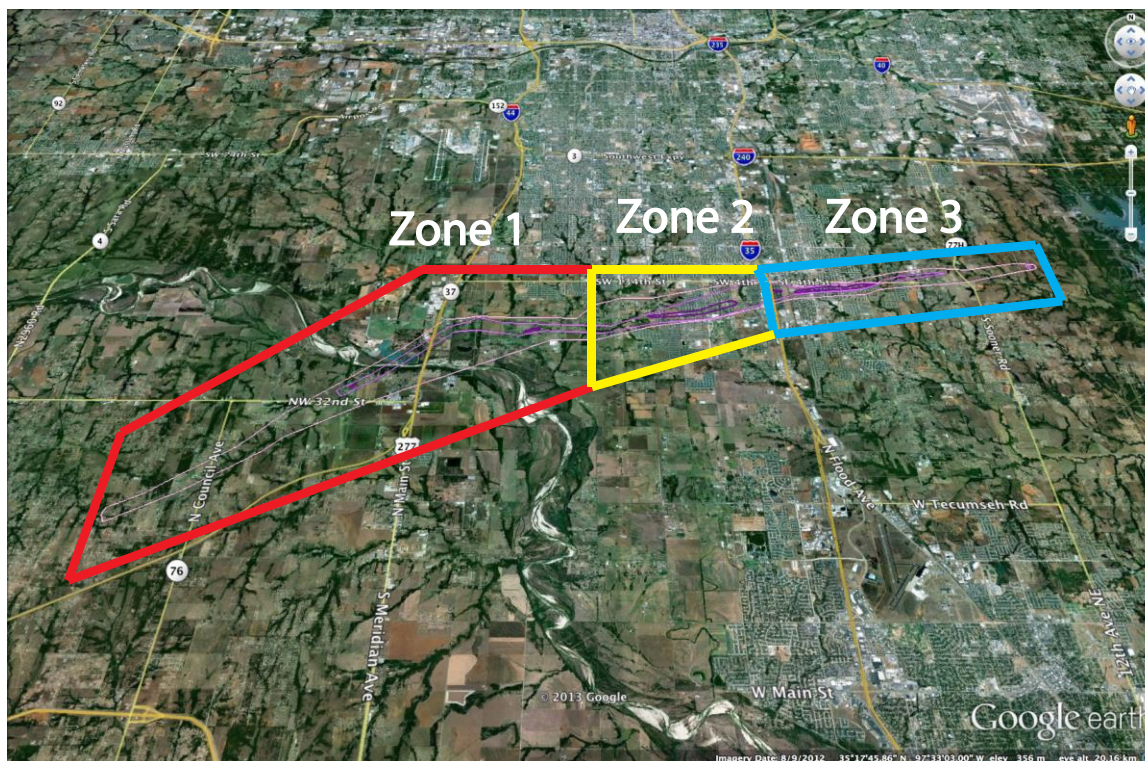


Figure 1.6. Geographic breakup of the tornado track into zones.

Observed Shelter Performance

Storm shelters and safe rooms have often been interchangeably used terms for rooms intended to protect occupants from the wrath of a tornado. For the purposes of this paper, the term “shelter” will be used for all. A table of the performance of all the shelters documented is provided in the Appendix A.

The shelter must have a door type that has been tested to withstand applicable debris impacts and pressure testing as listed in Chapter 8 of ICC-500, which is also the referenced requirement in FEMA P-320. The door must also include three locking points and three hinges per the Door Details and Signing Requirements drawing (Sheet 16 of 18) of FEMA P-320. The shelter is also required to have easy access to egress per Chapter 5 of ICC-500, which entails a minimum stair tread depth for residential shelters of eight inches and maximum riser height of 9 9/16 inches. Proper drainage is determined based on the shelter site and type of shelter, but in all cases prevents water from entering the shelter. Siting criteria can be found in Chapter 4 of ICC-500.

Exterior Below-ground Shelter

Flush-to-the-Ground

The team documented eight flush-to-the-ground shelters, all of which had hinged swinging doors. One shelter (No. 47) sustained a significant failure as the door was blown off due to sheer failure of rusty hinge pins (seen in *Figure 7*). The door of another shelter (No. 14) was stuck shut and the homeowner, an elderly man, opted to take shelter in an interior bathroom of his home with his pets. Additional deficiencies with shelters of this type were related to door thickness, locking systems, hinges, stairs, and drainage. Many of such shelters were not properly designed with beveled edges to allow water runoff to flow around the opening, and had been flooded with 6 inches to 48 inches of water. (Flooding was a common complaint among homeowners.) Many homeowners were under the impression that they were safer in below-ground shelters and were unaware of shelter installation and testing guidelines.



Figure 1.7. Door failure of shelter with only a single lock point (No. 47).

Clam Shell

Fifteen clam shell shelters were documented with no major failures observed. However, deficiencies were found consistently in doors and ventilators. Many doors had only one locking latch and an insufficient number and inadequate type of hinges. In *Figure 8*, a clam shell shelter (No. 3) was designed with only one lock, two hinges, and sustained vent damage at an EF-2 rated location. In *Figure 9*, a clam shell shelter with a tested door that included three locks and three hinges was documented at an EF-4 rated location. A flat-top clam shell shelter (No. 17) was

documented in an EF-2 rated location, as seen in *Figure 10*. Performance of these types of shelters was generally acceptable. Accessibility to the shelters can be an issue because these types of shelters are located outdoors and the distance people must travel to take shelter must be considered when deciding placement. One shelter was located about 100 feet from the house, which could pose a greater threat to potential occupants if they need shelter quickly. Standing water of several inches was also commonly observed.



Figure 1.8. Clam shell shelter with an insufficient locking system and damaged vent at an EF-2 rated location (No. 3).



Figure 1.9. Clam shell shelter with a tested door that has three locks and three hinges at an EF-4 rated location (No. 28).



Figure 1.10. Flat top clam shell shelter with three locks at an EF-2 rated location (No. 17).

In-Garage Floor

Twenty in-garage floor shelters with heavy sliding doors were documented. All appeared to have performed reasonably, with no major failures. *Figure 11* shows a shelter (No. 12) in an EF-3 rated location with a stair/ladder entry that would not allow for easy access, an inadequate locking system, and some flooding in the bottom. Other issues were similar to exterior below-ground shelters such as: door thickness, locking systems, roller bearings, stairs, and drainage. *Figures 12 and 13* are tested shelters (Nos. 9 and 61) that performed very well in an EF-4 and EF-0 rated locations respectively. Homeowners indicated they preferred the in-garage style shelter because it was cheaper, installation was quick, it did not take up any space, and it gave them confidence by being underground.



Figure 1.11. In-garage storm shelter with inadequate stairs, locking system, and flooding in the bottom at an EF-3 rated location (No. 12).



Figure 1.12. Tested shelter at a location assigned an EF-4 rating (No. 9). Car was present during the tornado.



Figure 1.13. Tested NSSA shelter at a location assigned with an EF-0 rating (No. 61).

Dome

Two dome style shelters were inspected. The shelter pictured in *Figure 14* (No. 43) was cast in place concrete with double plywood doors with lite gauge steel skin cladding, one lock, and two hinges per door. The shelter was observed at an EF-3 rated location. This shelter also had a few inches of standing water in the bottom. The other dome shelter the team documented had three heavy gauge locks with an 1/8 inch steel door. It was partially above-ground, located in an EF-4 rated location, and is shown in *Figure 15* (No. 40). Both appeared to have survived the storm with no major failures, although the construction of the door of the first dome shelter was inadequate.



Figure 1.14. Dome shelter with an inadequate door system that included a plywood and lite gauge sheet metal clad door, single locking point, and ventilator removed at an EF-3 rated location (No. 43).



Figure 1.15. *Dome shelter partially above-ground located in an EF-4 rated location (No. 40).*

Above-ground

Eight above-ground shelters were documented. Six had been tested, and it was noted that most of these safe room manufacturers proudly displayed their company name on their products. *Figure 16* is an untested above-ground storm shelter (No. 25) that was in the garage of the home in an EF-4 rated location, and survived without any failures, even though the rest of the home was destroyed. The other two safe rooms that were not tested were located on the same street of a fairly new well built neighborhood. These were builder installed walk-in closets made of concrete. The concrete portions of the shelters had been tested; however, the doors did not appear to be tested units. One of these shelters (No. 49) is shown in *Figure 17*. *Figure 18* shows a tested steel shelter (No. 46) that performed very well. Some above-ground safe room owners still questioned if they were safe because they were not below-ground.



Figure 1.16. An above-ground tested shelter that survived EF-4 wind speeds (No. 25).



Figure 1.17. An above-ground, concrete shelter that also served as a walk-in closet at an EF-2 rated location (No. 49). The concrete was tested, but the door was not.



Figure 1.18. Above-ground tested steel shelter that performed very well in an EF-3 rated location (No 46).

Conclusion and Recommendations

The TTU May 3, 1999 Moore, OK, storm investigation, along with the FEMA Mitigation Assessment Team, revealed the presence of numerous below-ground shelters. One above-ground shelter, shown in *Figure 19*, was observed in Dell City, OK, standing on the lawn of Beth Bartlett, the shelter owner. Following the admonitions of local media, many residents fled their homes, two of whom were killed seeking shelter under a local underpass (Miller *et al.* 1999). President Bill Clinton authorized the first FEMA Shelter Incentive Program to aid homeowners in the cost of installing a shelter. The TTU investigation of the May 8, 2003 tornado led to the finding of new below-ground and above-ground shelters constructed to the FEMA P-320 (1998) specifications.



Figure 1.19. Beth Bartlett and her mother standing inside her above-ground storm shelter where they took cover during the May 3, 1999 tornado.

A representative sample of 61 shelters of a possible 200 shelters included in three databases was observed and studied from the May 20, 2013 Moore tornado. All three types of shelters, exterior below-ground, in-garage floor, and above-ground, experienced various EF storm strengths, including EF-4, leading to no loss of life. Investigations revealed an increase in population of exterior underground shelters, plus the popularity of a new in-garage floor underground shelter design. Many of the shelters and shelter doors had been tested at the NWI Debris Impact Facility at TTU. Furthermore, numerous above-ground shelters were observed, both constructed on site and prefabricated steel units. The following is a list of recommendations following the findings from the post storm investigation:

- 1) A list of storm shelters and safe rooms with their locations shall to be aggregated and made available to local emergency responders. Immediately after a tornado is a time-critical period to search for survivors, and such a list will aid in the efficiency of finding survivors. Some municipalities have started such a list; however, not every storm shelter owner knows that it exists and the benefits of being added to it. The existence of the list needs to become public knowledge, although the contents of the list need to remain for emergency personnel use only.
- 2) The public must be properly educated on the myth that below-ground shelters are the only safe place in severe tornadoes. Properly designed and constructed above-ground shelters are just as safe as a properly designed below-ground shelter. When a storm shelter is not available, it is still highly ill-advised to attempt to outrun the tornado in an automobile.
- 3) Consideration for the difficulties of people of advanced age, or people with physical disabilities must be addressed when buying a storm shelter. Many issues can be resolved by choosing an above-ground shelter, including those challenges that manifest in situations of extreme stress

and make traversing stairs difficult. Below-ground shelters may prevent these people from accessing the shelter if they are physically incapable.

- 4) The public needs to be informed about the potential flooding of below-ground shelters which can begin to fill with water before the inhabitants can successfully operate the door to leave the shelter after the storm has passed. No deaths were reported from this predicament, but flooded shelters were a common problem in this investigation.
- 5) Proper maintenance and upkeep information needs to be clearly stated and given to shelter owners. Outdoor shelters must be maintained annually with rust removal and repainting.
- 6) Owners of shelters built before any standards or guidelines were in place need to update their shelters to include additional locking systems if there is only one lock.
- 7) A digital survey could benefit the data collection aspect of future investigations. This survey could be easily distributed to shelter owners with inquiries such as information about their shelter, federal programs taken advantage of to lower the out-of-pocket expense for the shelter owner, and overall use and utilization of the shelter.

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PART II: TORNADO WIND SPEED ESTIMATION

Introduction

The Enhanced Fujita Scale (EF-Scale) was developed at Texas Tech University to rate intensity of tornadoes and was adopted by the National Weather Service in February 2007 (Weather, 2014). There are 28 Damage Indicators (DI), 23 of which are building types ranging from small barns (SBO) to an automobile showroom (ASR) and strip malls (SM). For each DI, a set of Degree of Damage (DOD) is identified to describe the level of damage sustained. For One and Two-Family Residence (FR12), the Degrees of Damage are numbered between 1 (threshold of visible damage) to 10 (slab wept clean). In addition, three wind speeds are estimated for each DOD to account for inherent uncertainties in construction quality, design, and maintenance: expected wind speed (EXP), lower bound wind speed (LB), and upper bound wind speed (UB). For example, EXP, LB and UB wind speeds causing DOD5 of FR12 (i.e. entire house shifting off foundation) are 121mph, 103mph, and 141mph respectively. Meanwhile, a range of wind speed is defined to rate a specific tornado intensity: EF-0 (65-85mph), EF-1 (86-110mph), EF-2 (111-135mph), EF-3 (136-165mph), EF-4 (166-200mph) and EF-5 (>200mph). (However, wind speed estimation was provided by a panel of experts following the elicitation protocol established by the Senior Seismic Hazard Assessment Committee (SSHAC, 1997)). The correlation of observed damage with wind speeds that caused the damage is important in understanding the uncertainties (and variations) in the observed damage at a location and in calibrating/verifying the wind speeds in the EF-Scale.

Back-calculating wind speeds is a mathematically simple process where the force to cause an observed failure is equated to the applied force:

$$F = \frac{1}{2} \rho V^2 (GC_f) A \quad (1)$$

where F = limit state resistance for the observed failure; ρ = air density; (GC_f) = combined gust effect factor and force coefficient; V = wind speed to cause the failure, and A is the projected area of the member. The shape of the structure should be such that the force coefficients, GC_f , are well known, and with F , ρ , and GC_f known, the failure wind speed can be calculated.

Uncertainties exist in all the variables which make the exact failure wind speed calculated imprecise, and so, to make such uncertainties as small as possible, simple rigid structures made with materials that have well defined properties are then selected for analysis. Rigid structures with simple axisymmetric shapes (such as round poles where the pressure coefficients are well known and not a function of angle of attack) are preferred over complex shaped structures where the pressure coefficients are a function of many variables including angle of attack, height to length ratio, width to length ratio, slope of roof, etc.

Street sign structures are initially targeted for analysis since they are common in residential areas, geometrically simple, and have been observed to fail in bending. To use these signs for back-calculating wind speeds, the height of the sign, the size and shape of the sign, the geometry of the pole and the observed failure limit state need to be documented. Additionally, damage to surrounding structures needs to be documented to assign the EF-Scale to the structures.

Failure wind speeds are back-calculated using Equation (1) with the appropriate GC_f values. The back-calculated wind speeds are compared to wind speeds associated with the Degrees of Damage (DOD) for the appropriate Damage Indicators (DI) for the structures in close proximity to the structure used to compute the wind speed. The mean and range of failure wind speeds arising from the uncertainties in material properties, dimensions (and similar) in the back-calculated wind speeds will be compared to the mean and the range of wind speeds given in the EF-Scale. This comparison will provide insight into the accuracy of the wind speeds in the EF-Scale.

Structure Locations, Descriptions, Failure Modes, and Nominal Capacities

Simple signs and a brick mailbox are chosen as the structures to be used for this investigation. The natural frequency of the signs and the mailbox were in excess of 2 Hz and thus are considered rigid. Aerodynamic instability (such as flutter or galloping) was not considered herein.

Location of structures investigated

The damage path for the tornado is shown in *Figure 2.1* (NWS, 2013). Simple structures across the damage path that fell in the EF-0 through EF-4 regions were selected for analysis. The locations of these simple structures are shown in *Figure 2.2*.

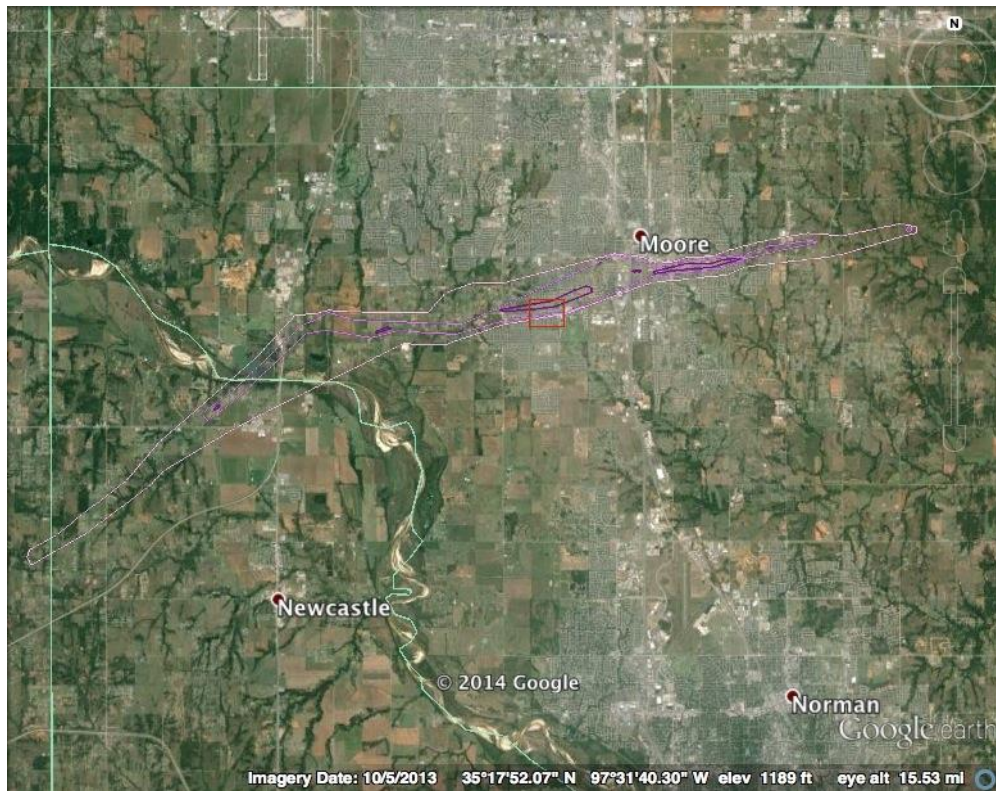


Figure 2.1 Damage path (NWS, 2013).



Figure 2.2 Close up of area for structures analyzed.

Structure Descriptions

A brief description of the six structures is given below. The sizes and material properties of the simple structures are summarized in *Table 2.1*.

Structure 1: Small sign

The dimensions of the sign, shown in *Figure 2.3*, are approximately 12 in wide by 17 in tall. This sign is supported with a perforated square steel tube post. Based on the small size of the sign, inspection of the pictures, and the Oklahoma Department of Transportation (ODOT) Specifications (ODOT, 2009), the post was assumed to be 1.5 in by 1.5 in square constructed from 12 gauge ASTM 1011 Grade 50 steel.



Figure 2.3 Structure 1, small sign.

Structure 2: Stop sign

The stop sign structure is shown in *Figure 2.4*. The dimensions of the stop sign dimensions are established from ODOT specifications. There are three different possible sizes, and this stop sign was assumed to be the smallest size, 30 in by 30 in. The stop sign also included street name signs on the post, which were assumed to be 30 in by 6 in. Based on the larger area of this sign, the post was assumed to be 2 in by 2 in square made from 14 gauge ASTM 1011 Grade 50 steel.



Figure 2.4 Structure 2, stop sign.

Structure 3: School crossing sign

The school crossing sign is shown in *Figure 2.3*. Its dimensions are established using the ODOT Specifications. Again, there are multiple sizes listed and the smallest size, 36 in by 36 in, is used. Additionally, there is a smaller sign below it containing a downward pointing arrow. The size of this sign is listed as 24 in by 12 in. This post is 2 in square tube of 14 gauge ASTM 1011 Grade 50 steel.



Figure 2.5 Structure 3, school crossing sign.

Structure 4: No through street sign

This sign, shown in *Figure 2.6*, could not be found in the ODOT Specifications. The dimensions of a “School Bus Stop Ahead” sign, which has the same shape and likely the same size, were used instead. The standard size smallest sign of this type is 30 in by 30 in. This sign was the only one that did not use perforated square steel post, and instead used a circular steel pipe. According to the Specifications, the steel pipe used for signs is a Schedule 40, 2 in ASTM Grade 50 steel pipe, with an outside diameter of 2.375 in and a thickness of 0.15 in.



Figure 2.6. Structure 4, No through street sign.

Structure 5: Street name sign

This sign (shown in *Figure 2.7*) like others, was not listed in the ODOT Specifications. However, it was assumed to be identical to the street name sign on the stop sign, 30 in by 6 in. Based on the small size of this sign, the 1.5 in, 12 gauge ASTM 1011 Grade 50 steel post size was assumed.



Figure 2.7 Structure 5, street name sign.

Structure 6: Brick mailbox

The brick mailbox was the structure with the least amount of readily available information in the selected databases, and almost every property of the mailbox was estimated from three pictures. First, the mailbox was assumed to have a CMU core with a one-brick-deep outer layer. There are many different types of bricks, and each type has a slightly different size and weight. From a measuring tape in one of the pictures, each brick was determined to have a cross section of approximately 2.75 in by 2.75 in. This corresponds with queen-sized bricks, which, according to multiple brick manufacturers, weigh approximately four lbs. By counting the two sides of the mailbox available in pictures, it was estimated that the mailbox consisted of 181 bricks, weighing 724 lbs. The core was determined to have approximately five CMU blocks weighing 230 lbs. The measuring tape, along with brick dimensions, allowed for reasonable estimate of the mailbox's height (46 in), width (25.5 in), and depth (23.5 in).



Figure 2.8 Structure 6, brick mailbox.

Failure Modes

The traffic signs experienced yielding due to either pure bending or a combination of both bending and torsion. The pictures were carefully studied to determine which of these cases applied to each sign. If the sign failure was approximately perpendicular to the sign face, a pure bending limit state was assumed, and if the sign fell at an angle relative to the face of the sign, then significant torsion was assumed to be present and a combined bending and torsion limit state was assumed.

For the signs with the perforated square tube posts, this failure (yielding from pure bending or combined bending and torsion) occurred at the joint between the post and the anchor tube. The bending failure of Structure 2 (stop sign post), is shown in the sign with the pipe post failed at the point where it entered the ground. The limit states used for each of the sign structures is given in *Table 2.1*.



Figure 2.9 Bending failure of post for structure 2, stop sign.

Nominal Capacities

Nominal capacities of the steel posts are established using the provisions of AISC (2010). For the signs that experienced pure flexural yielding, the yield moment was calculated as:

$$M_n = F_y Z_x \quad (\text{Eq. 2})$$

Where,

M_n = Yield moment (k-in)

F_y = Yield strength of the material (ksi)

Z_x = Plastic section modulus (in³)

The perforated square tubing does not have a constant plastic section modulus due to the non-constant cross section, and so stress concentrations at the holes can cause the cross section to yield sooner than expected. However, for this analysis, the plastic section modulus was taken at the cross section across the center of the holes - the weakest point in the total cross section. M_n was determined to be approximately 12 k-in for the 1.5 in square post, and 18 k-in for the 2 in square post. For the case of the circular pipe, M_n is 37.19 k-in.

This method of moment calculation was verified using finite element analysis, in which a shortened model of the 1.5 in post was analyzed. With a large displacement, nonlinear analysis found that a moment of approximately 12 k-in caused failure similar to the one observed.

Determining failure criteria for the structures in which torsion was considered was slightly more complicated. When torsion is considered, shear must also be considered as there is an interaction between the two that can cause failure. Therefore, the torsional and shear capacities were also determined for the square post cross-sections, using AISC equations G2-1 and H3-1, respectively (AISC, 2010). For the 1.5 in post, the torsional capacity, T_n , was 12.54 k-in, and the shear capacity, V_n , was 8.41 k. For the 2 in post, T_n was 17.25 k-in and V_n was 8.64 k. In the case of the mailbox, the resisting moment was calculated as:

$$M_r = W * \frac{B}{2} \quad (\text{Eq. 3})$$

3)

Where: M_r = resisting moment (lb-in)

W = total weight of mailbox (lb)

B = base dimension in the direction of tipping (in)

These capacities for each failure mode are summarized in *Table 2.2*.

Table 2.1 Structure description, dimensions, supports, and failure modes

Structure	Description	Dimensions	Support Structure	Support Material	Failure Mode
1	Small Sign	12" x 17"	1.5" x 1.5" 12 gage tube	ASTM 1011 Grade 50 Steel	Bending & Torsion
2	Stop Sign	30" x 30"	2" x 2" 14 gage tube	ASTM 1011 Grade 50 Steel	Bending
	Street Name Sign	30" x 6"			
3	School crossing sign	36" x 36"	2" x 2" 14 gage tube	ASTM 1011 Grade 50 Steel	Bending & Torsion
	Arrow Sign	24" x 12"			
4	No Thru Street sign	30" x 30"	2" diameter,	Schedule 40 ASTM Grade 50	Bending
5	Street Name Sign	30" x 6"	1.5" x 1.5" 12 gage tube	ASTM 1011 Grade 50 Steel	Bending
6	Brick Mailbox (W x T x H)	25.5" x 23.5" x 46"	NA	NA	Overturning

Table 2.2 Member capacities

Component	Capacity
1.5" Post	$M_n = 12 \text{ in-k}$
	$T_n = 12.5 \text{ in-k}$
	$V_n = 8.41 \text{ k}$
2" Post	$M_n = 18 \text{ in-k}$
	$T_n = 17.25 \text{ in-k}$
	$V_n = 8.64 \text{ k}$
Brick Mailbox (W x T x H)	$W_t = 954 \text{ lbf}$
	$B/2 = 12.75"$
	$M_r = 12.2 \text{ in-k}$

Wind Induced Forces

The failure wind speeds are computed using Equation 1 (presented above). Thus, the air density, gust effect factor, and force coefficients are required to calculate the failure wind speed. This section addresses the values used herein.

Air Density

Air density is computed using the Virtual Temperature method. To compute the air density using this method, the barometric pressure, temperature and relative humidity are required. Three Mesonet stations in the vicinity of the tornado are used to obtain a value for each of these parameters, and a mean value over the period of time that the tornado was on the ground is used for the calculation of the air density. These mean values are given in *Table 2.3*. The air density computed using these mean atmospheric values is 0.002 slug/ft³. When this value is used in computing the velocity pressure (in conjunction with [(3600 sec/hr) / (5280 ft/mi)]² to convert wind speed in ft/sec to mph) yields the coefficient 0.002197 for the V^2 term. This coefficient is 0.00256 when computed at standard temperature and pressure, and thus represents a 14% reduction in the coefficient.

Table 2.3 Values of barometric pressure, temperature and relative humidity used to compute air density

Parameter	
Barometric Pressure, in-Hg	27.06
Temperature, °F	80
Relative Humidity, %	79.2%
Air Density, slug/ft ³	0.002

Force Coefficients

The product of the gust effect factor, G , and the force coefficients, C_f , used in this investigation are obtained from ASCE (2010) and from Zuo *et al* (2014) when torsion is a significant part of the loading. The values used in the calculation of the failure wind speeds are given in

Table 2.4 Gust effect factors and force coefficients for each structure

Structure	Component	GC_f
1	Small Sign	1.53
	1.5" Post	1.28
2	Stop sign	1.25
	Street Name Sign	1.85
	2" Post	1.70
3	School Crossing Sign	0.45
	Arrow Sign	0.45
	2" Post	1.70
4	No Thru Street Sign	1.53
	2" Diameter Post	0.94
5	Street Name Sign	1.28
	1.5" Post	1.28
6	Brick Mail Box	1.32

Results

An estimate of the failure wind speed is obtained by equating the nominal resistance to the applied load effect and solving for V using Equation 1. The results of these calculations are given in *Table 2.5*. Discussion of the results is given below.

Table 2.5 Estimated failure wind speeds

Structure	Nominal Failure Wind Speed	Failure Wind Speed EF-Scale Rating	NWS EF-Scale Rating
1	136	EF-3	EF-2 (111-135)
2	115	EF-2	EF-2 (111-135)
3	120	EF-2	EF-2 (111-135)
4	124	EF-2	EF-2 (111-135)
5	123	EF-2	EF-2 (111-135)
6	149	EF-3	EF-4 (166-200)

The calculated failure wind velocities seem reasonable in relation to each other and to the surrounding damage. Torsion had a very small effect on the failure of the structures due to the high torsional capacities of the cross-sections and the very small eccentricities.

Structures

Structure 1 – Small Sign

The failure velocity for Structure 1 was the highest of all the signs due to the fact that it had the smallest area of all the signs. The calculated wind speed of 136 mph corresponds with low EF-3 damage. Structure 1 was located in what was likely an EF-2 area. There is some uncertainty due to the fact that there were few other structures surrounding Structure 1. However, Briarwood Elementary School, located approximately 100 yards away, experienced EF-5 damage.

Structure 2 – Stop Sign

Structure 2 was located near the outer edge of an EF-2 zone. The calculations suggest that the wind speed that caused this failure was at least 115 mph, and this structure experienced the lowest wind speeds at failure, mainly due to the large area of the sign and to the fact that the wind angle of attack that caused failure was perpendicular to the face of the sign.

Structure 3 – School Crossing Sign

Structure 3 was located approximately in the middle of the delineated EF-2 zone, and the surrounding damage suggests EF-2/EF-3 wind speeds. Although this sign had a larger area than Structure 2, the wind acted on the sign at a very sharp angle which means that a much higher wind speed was required to cause failure than when the wind acts perpendicular to the sign face. The wind speed at this location - slightly north of Structure 2 - was at least 120 mph.

Structure 4 – No Thru Street Sign

Structure 4 was located west of the first three structures, and approximately in the middle of the delineated EF-2 zone. Again, surrounding damage suggests EF-2/EF-3 wind speeds. This sign, like Structures 2 and 3, has a large area. However, the larger circular post requires a higher velocity to cause failure. The wind speed at this location was at least 124 mph.

Structure 5 – Street Name Sign

Structure 5 was located at the same intersection as Structure 4. This sign had a very small area, but the splaying of the second sign gave it more area. The wind velocity at the location of Structure 5 was at least 123 mph.

Structure 6 – Brick Mailbox

The mailbox, located on the border of EF-3/EF-4 damage, failed at the highest velocity of all the structures investigated. Wind speeds at this location were at least 149 mph. It should be noted that

soil suction was not considered in the analysis of this structure. In other words, the mailbox was assumed to be simply resting on the ground.

Sources of Possible Error

Due to the fact that only pictures were available and the actual structures could not be measured with 100% reliability, there are many sources for possible error including:

- Construction quality may differ from structure to structure. (This analysis rests on the assumption that all construction practices were followed as specified in the ODOT Specifications.)
- Most of the mailbox properties were assumed because of a lack of specific information. The number, type, and weight of bricks, the mailbox dimensions and even the contents of its core (CMUs) were estimates. This leaves substantial room for possible error; however, the calculated value is reasonable.
- The dimensions of the signposts were not measured, but were estimated from the pictures and the information contained in the ODOT Specifications. There is the possibility for a small error in these calculations.
- Nominal strengths are used for the calculations. Since strength is actually a random variable, using a deterministic value can introduce error into the results.
- Variation in the actual plastic section modulus of the signposts could not be calculated because the cross section is not continuous. However, the method used to compute the property was verified by a finite element analysis.
- The largest source of error was most likely due to the drag coefficients of the signs. Had the signs been rectangles with their faces oriented normally to the wind, the determination of drag coefficients would have been simple and accurate.

Conclusions

This analysis showed that the current official method of estimating tornado wind speeds, the EF-Scale, is accurate. Wind speeds of 149 mph caused failure of a mailbox on the border of an EF-4 zone, while wind speeds of 115-136 mph caused failures of signs in EF-2 and EF-3 zones. These computed failure wind speeds correlate well, leading to the conclusion that, in this case, the wind speed estimates associated with the EF-Scale residential damage are reasonable.

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PART III: PERFORMANCE EVALUATION OF BUILDING GLAZING

Introduction

Glazing in structures, both residential and commercial, commonly fail due to debris impact during tornados, damages that have exposed the interior of the structures to extreme wind pressure, debris and water which can then cause significant damage to the interior and precipitate complete structural failure. New products based on advances in the use of glazing in structures (such as triple insulating glass units, advanced laminated glass interlayers and structural use of glass) have recently entered the market. Consequentially, there exists little to no documentation of the performance of these new products exposed to tornadic winds.

As energy conservation becomes more important, triple-pane insulating glass units (TIGU) are used more often in residential and commercial structures. Generally, TIGUs are not designed to be debris impact resistant, although the presence of an additional glass lite may provide increased debris impact resistance over a double insulating glass unit. Additionally, stiffer laminated glass interlayers may be used to increase impact resistance and to maintain the enclosure even if the glass plies are broken. Laminated glass is used extensively in regions affected by hurricanes, but it is not currently required in regions affected by tornados.

However, the use of glass as a structural component has recently increased due to several high profile examples such as at the Apple Store in New York. Structural glazing consists of glass components that carry the loads of other building components in addition to environmental loads, such as glass floors, stairs, balustrades and fins. Currently, TIGUs and laminated glass are designed to resist wind and snow loads using ASTM E1300-12^{ae1} (ASTM, 2012) but this does not address debris impact. The closest standard specification and testing method for impact performance of exterior glazing is governed by ASTM E1986-12^a (ASTM, 2012) and ASTM E1886-05 (ASTM, 2005), respectively. However, both standards specifically mention hurricanes and not tornados, and so currently, a standard addressing structural use of glass does not exist. A deterministic approach using ASTM E1300-12^{ae1} (ASTM, 2012) and ASCE7-10 (ASCE, 2010) can be employed to estimate the wind speed required to initiate a failure in insulating glass and laminated glass.

During extreme wind events, the integrity of windows and door openings plays a critical role in reducing internal pressure and protecting building occupants and contents from debris and water. New glazing types (such as triple insulating glass units' high performance interlayers for laminated glass and structural glass) are gaining popularity, and so consequently, a few documented cases of their exposure to tornadic wind do exist. Therefore, we will particularly focus on these newer glazing types to document how they performed. Glazing from commercial and residential structures will be surveyed containing specimens of a target glazing type. Documentation will include: characterizing the surrounding terrain, assessing the damage to surrounding structures, recording the type of glazing construction, determining whether the glazing failed due to debris impact or wind pressure, and then establishing the relationship of damage to glazing over the entire

structure. Such a comparison of glazing failures across the building envelope and in the surrounding structures will facilitate determination of the failure mechanism for the glazing of interest.

General cases for determining glazing failure that have resulted from debris impact include: insulating glass units with at least one lite missing and at least one lite intact, broken lites remained mostly in place with a small area broken out of the lite, impact damage to the window frame, and debris still present in glazing. Using advanced analysis methods for window glass strength, a minimum wind speed to initiate failure will then be estimated for the glazing failures due to wind load and a maximum wind speed to initiate failure will be estimated for glazing that remained intact. The maximum and minimum bounds will be compiled into an envelope of wind speeds and overlaid spatially on the storm path. Both residential storm doors and commercial store fronts provide good indicators for the maximum wind speed as they are typically comprised of tempered glass which is more impact resistant than the annealed glass commonly used in residential structures. If one considers that a building can still remain habitable while its glazing is missing or damaged, it is clear that much of any available evidence will quickly be lost to cleanup and repairs.

Specimen Locations, Descriptions, Failure Modes, and Nominal Capacities

Location of specimens investigated

Figure 1 shows the locations of the structures surveyed containing the documented specimens with the EF-Scales delineated with purple lines. The darker purple lines indicate increasing EF-Scale values from EF-0 (lightest) to EF-4 (darkest). EF-Scale intensities typically do not vary uniformly between contour lines suggesting that interpolation may not be appropriate which means that specimens located between EF-Scale contour lines the range between the bounding EF-Scale contour lines is used. *Table A.1* lists 32 documented specimens with GPS location and the EF-Scale for each.

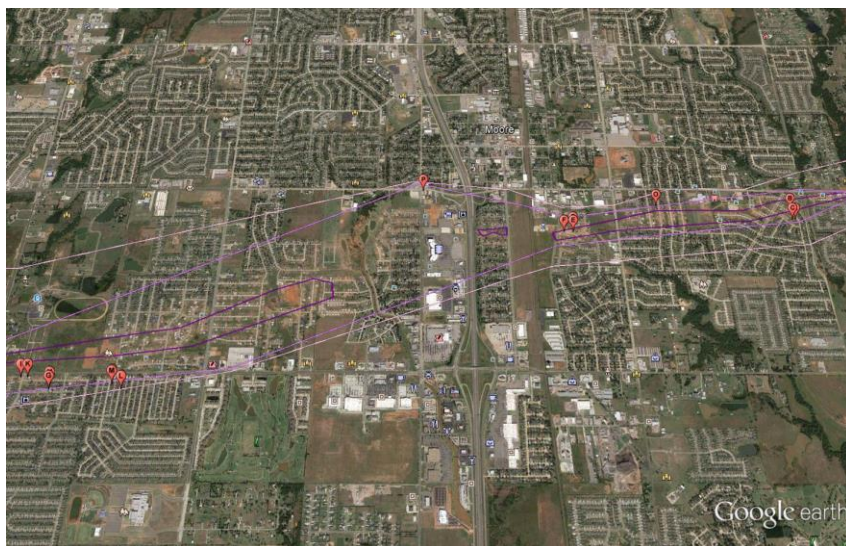


Figure 3.1. Specimen locations with EF-Scale contours.

Structure Descriptions

Table A.2 lists the type, rectangular dimensions and construction for each specimen. In the Type column, Res denotes the specimen is from a residential structure, Com denotes the specimen is from a commercial structure, (DH) denotes the specimen is a double-hung window, (P) denotes the specimen is a punched window, (D) denotes the specimen is a double door, while (S) denotes the specimen is a single door. In the Construction column, DIGU denotes a double insulating glass unit, SG denotes as single glazed lite, AN denotes annealed glass and FT denotes fully tempered glass. The glass lite thicknesses shown in Table A.2 are industry standard nominal thicknesses and are not the actual measured thicknesses of the specimens. All DIGUs are assumed to be comprised of annealed glass lites (unless otherwise noted) as heat-treated glass is not typically used in residential windows. Appendix B contains photographs of the specimens with each specimens identified with red labels.

Failure Modes

Several criteria were employed to assess the cause of fracture in the 15 fractured specimens. The assessment criteria included fracture pattern, damage to the structure in proximity to the glazing, presence of debris on the glazing, unfractured interior lite of a double insulating glass unit (DIGU), presence of an unfractured window(s) with similar geometry and construction adjacent to the specimen. The fracture pattern resulting from a high pressure failure significantly differs from a fracture pattern resulting from impact damage. Figure 2 shows the fracture of a 2.5 mm X 500 mm X 500 mm (3/32 in. X 20 in. X 20 in.) specimen loaded to failure in a lab test. The specimen shown in Figure 2 has similar thickness and rectangular dimensions to many of the documented specimens and fractured under a high uniform lateral pressure resulting in high stresses at the

instant of fracture. The fracture pattern consists of many small shards and appears to be completely broken across the entire area of the specimen.

In contrast, *Figure 3* shows a magnified view of Specimen L1 where the shards are significantly larger with large areas of unfractured glass indicating the fracture was not due to large stresses resulting from a high uniform lateral pressure, but rather due to impact damage caused by wind borne debris impacting the window. Using the assessment criteria, it was determined that all 15 fractured specimens fractured due to debris impact. *Table A.3* lists the criteria for each fractured specimen with a justification for determining the specimens fractured due to debris impact damage.

Specimen ASTM E1300 Load Resistances

The ASTM E1300 load resistance for each specimen was calculated according to the provisions in ASTM E1300-12^a (ASTM, 2012). Load resistance is defined in ASTM E1300-12^a (ASTM, 2012) as a uniform lateral load that a glass construction can sustain based on an 8/1000 probability of breakage and a 3sec load duration. However, as ASTM E1300-12^a (ASTM, 2012) has a simplified load sharing factor for apportioning the load between the two lites comprising a DIGU, an iterative method advance by Vallabhan and Chou (1985) was used to provide load share factors which closely match DIGU behavior subjected to high pressure loads. The load resistance for each specimen is listed in *Table A.4*. *Table A.4* also lists a reduced load resistance for DIGU specimens when the exterior lite was broken and the interior lite was undamaged.



Figure. 3.2. 2.5 mm X 500 mm X 500 mm (3/32 in. X 20 in. X 20 in.) specimen loaded to failure in test lab.



Figure 3.3. Specimen L1 fracture pattern.

Wind Induced Forces

In order to compare the specimen load resistance to corresponding EF-Scale wind speeds, wind speeds were calculated using the component and cladding provisions of ASCE7-10 (ASCE, 2010). The net pressure equation for component and cladding is given in Eq. 1 as follows

$$P = q_h \cdot (GC_p - GC_{pi}) \quad \text{Eq. 1}$$

where P denotes the net pressure, q_h the velocity pressure at mean roof height, given in Eq. 2. GC_p denotes the component and cladding external pressure coefficient and GC_{pi} denotes the internal pressure coefficient. Several velocity reduction factors including k_d , k_z and k_{zt} are assumed to be 1.0.

$$q_h = \frac{1}{2} \cdot \rho \cdot V^2 \quad \text{Eq. 2}$$

where ρ denotes the density of air and V denotes the wind speed velocity. Therefore, employing a deterministic approach, Eq. 1 and Eq. 2 are solved for the wind speed velocity denoted as V yielding Eq. 3.

$$V = \sqrt{\frac{2 \cdot P}{\rho \cdot (GC_p - GC_{pi})}} \quad \text{Eq. 3}$$

Where the air density, ρ was previously calculated in Section 3 and the net pressure is the load resistance of the specimen listed in *Table A.4*.

Pressure Coefficients

Since it was not possible to determine whether other components of the structure failed before or after the specimen was loaded, nominal wind speeds for both enclosed and partially enclosed were calculated. Additionally, the positive and negative zone 5 external pressure coefficients (1.0 and -1.4), respectively, were used, resulting in eight external and internal pressure coefficient combinations. However, the four combinations with like signed coefficients were not used as they always resulted with smaller values. The remaining combinations considered are listed in *Table 1*.

Table 3.1. *External and Internal Pressure Coefficient Combinations.*

Combination	GC_p	GC_{pi}	$\square GC_p$
A	-1.4	0.55	1.95
B	1.0	-0.55	1.55
C	-1.4	0.18	1.58
D	1.0	-0.18	1.18

Table A.5 lists the nominal winds speeds corresponding to the four pressure coefficient combination for each specimen. For DIGU specimens with a broken exterior lite, the nominal wind speeds were also calculated based on the single lite load resistance.

Results

Specimens

Table 2 summarizes the comparison of the nominal wind speed (NWS) calculated from the specimen load resistance compared to the corresponding EF-Scale wind speed ranges. To simplify the analysis, the specimens are sorted by whether all four pressure coefficient cases were: less than lower wind speed of the EF-Scale denoted in the table as column $NWS < EF_L$, exceeded the higher wind speed of the EF-Scale denoted in the table as column $EF_H < NWS$, or within the EF-Scale range denoted as column $EF_L < NWS < EF_H$. Additionally, the specimens are grouped by undamaged, all broken or exterior lite broken. The exterior lite broken group shows the specimens sorted by the NWS value based on the DIGU LR, and also based on the one lite remain LR. The most significant trend in the data is that no specimen with NWS less than EF_L fractured which goes to support the authors' opinion that all specimens fractured due to debris impact. Furthermore, the study found that four specimens fractured with NWS exceeding the EF_H wind speed which also supports the specimens were fractured due to debris impact.

Table 3.2. *External and Internal Pressure Coefficient Combinations.*

Damage Description	NWS < EF_L	EF_L < NWS < EF_H	EF_H < NWS
Undamaged	1	4	12
Ex. Lite Broken (DIGU LR)	-	1	5
Ex. Lite Broken (SG LR)	2	1	3
All Broken	-	5	4

Structural use of glass and triple insulating glass unit

The investigation did not produce any examples of structural use of glass or TIGUs despite investigating several structures still under or recently completed construction. The authors opine that the use of TIGUs would further reduce the complete failure of IGUs as a third glass lite must be fractured before the interior is exposed to the storm and debris.

Sources of Possible Error

Since only pictures were available and the actual specimens could not be measured with 100% reliability, there are many sources for possible error including:

- Variations in specimen load resistance due to estimating rectangular dimensions from photographs.
- Variations in specimen load resistance due to assuming typical residential construction of double insulating glass units.
- Variations in the nominal wind speed due to:
 - The order components failed in structures,
 - The lack of information of actual terrain exposure surrounding a specimen during a tornadic event,
 - The use of external pressure coefficients derived from boundary layer conditions rather than tornadic conditions.

Conclusions

Based on the criteria presented for determining the cause of specimen fracture coupled with the comparison of NWS values to corresponding EF-Scale wind speed ranges, all fractured specimens investigated were determined to be caused by debris impact. Of the 12 specimens with NWS exceeding the EF-Scale high wind speed, six were storm doors containing a fully tempered glass lite suggesting that impact resistant glazing could greatly reduce interior damage to structures due to wind, rain and debris ingress into the structure. While the impact resistant glazing will not prohibit structure failure in the most intense areas of a tornado, it can greatly reduce interior damage to structures on the periphery of the tornado.

References

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APPENDIX A

Specimen Details

Table A.1. Specimen Locations with EF-Scale.

Specimen	Latitude	Longitude	EF-Scale
A1	35°19'54"N	97°27'38"W	EF-2- EF-4
B1	35°19'57"N	97°27'39"W	EF-4
B2	35°19'57"N	97°27'39"W	EF-4
B3	35°19'57"N	97°27'39"W	EF-4
B4	35°19'57"N	97°27'39"W	EF-4
C1	35°19'54"N	97°28'31"W	EF-4
D1	35°19'51"N	97°28'51"W	EF-4
D2	35°19'51"N	97°28'51"W	EF-4
E1	35°19'50"N	97°28'51"W	EF-4
F1	35°19'50"N	97°28'54"W	EF-4
F2	35°19'50"N	97°28'54"W	EF-4
G1	35°19'7"N	97°31'28"W	EF- 2(line)
G2	35°19'7"N	97°31'28"W	EF- 2(line)
H1	35°19'8"N	97°31'28"W	EF-2- EF-4
I1	35°19'10"N	97°31'37"W	EF-2- EF-4
I2	35°19'10"N	97°31'37"W	EF-2- EF-4
J1	35°19'10"N	97°31'37"W	EF-2- EF-4
J2	35°19'10"N	97°31'37"W	EF-2- EF-4
J3	35°19'10"N	97°31'37"W	EF-2- EF-4
J4	35°19'10"N	97°31'37"W	EF-2- EF-4
K1	35°19'10"N	97°31'35"W	EF-2- EF-4
K2	35°19'10"N	97°31'35"W	EF-2- EF-4
L1	35°19'7"N	97°31'7"W	EF0
L2	35°19'7"N	97°31'7"W	EF0
L3	35°19'7"N	97°31'7"W	EF0
M1	35°19'8.3"N	97°31'10"W	EF0- EF-2
M2	35°19'8.3"N	97°31'10"W	EF0- EF-2

N1	35°19'8.68"N	97°31'10"W	EF0- EF-2
O1	35°19'58"N	97°28'23"W	EF- 2(line)
O2	35°19'58"N	97°28'23"W	EF- 2(line)
P1	35°20'2.84"N	97°29'39.43"W	EF-2
P2	35°20'2.84"N	97°29'39.43"W	EF-2

Table A.2. Specimen type and construction.

Specimen	Type	Height	Width	Construction	
		mm (in.)	mm (in.)		
A1	Res - Window (DH)	330 (13)	686 (27)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
B1	Res - Window (DH)	483 (19)	305 (12)		SG - AN 3mm (1/8 in.)
B2	Res - Window (DH)	406 (16)	406 (16)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
B3	Res - Sliding Door	1854 (73)	737 (29)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
B4	Res - Storm Door	1854 (73)	737 (29)		SG - FT 4mm (5/32 in.)
C1	Res - Window (DH)	406 (16)	305 (12)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
D1	Res - Window (DH)	610 (24)	610 (24)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
D2	Res - Window (DH)	610 (24)	610 (24)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
E1	Res - Window (DH)	914 (36)	610 (24)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
F1	Res - Storm Door	1854 (73)	737 (29)		SG - FT 4mm (5/32 in.)
F2	Res - Window (DH)	813 (32)	813 (32)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
G1	Res - Window (P)	1524 (60)	610 (24)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
G2	Res - Window (DH)	1524 (60)	813 (32)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
H1	Res - Storm Door	813 (32)	813 (32)		SG - FT 4mm (5/32 in.)
I1	Res - Storm Door	1854 (73)	737 (29)		SG - FT 4mm (5/32 in.)
I2	Res - Window (DH)	914 (36)	914 (36)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
J1	Res - Storm Door	1854 (73)	737 (29)		SG - FT 4mm (5/32 in.)

J2	Res - Window (P)	914 (36)	610 (24)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
J3	Res - Window (P)	914 (36)	457 (18)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
J4	Res - Window (P)	1016 (40)	914 (36)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
K1	Res - Door (D)	1930 (76)	864 (34)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
K2	Res - Window (DH)	406 (16)	610 (24)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
L1	Res - Window (P)	914 (36)	610 (24)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
L2	Res - Window (DH)	1016 (40)	762 (30)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
L3	Res - Door (S)	1778 (70)	711 (28)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
M1	Res - Storm Door	1854 (73)	737 (29)		SG - FT 4mm (5/32 in.)
M2	Res - Window (P)	1016 (40)	508 (20)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
N1	Res - Window (P)	1829 (72)	203 (8)		DIGU - 3mm 10mm 3mm (1/8 in. 3/8 in. 1/8 in.)
O1	Res - Window (DH)	305 (12)	203 (8)		SG - AN 2.5mm (3/32 in.)
O2	Res - Storm Door	1930 (76)	813 (32)		SG - FT 4mm (5/32 in.)
P1	Com - Window (P)	1676 (66)	1676 (66)		
P2	Com - Door (S)	1930 (76)	813 (32)		SG - LG 6mm (1/4 in.)

Table A.3. Specimen Failure Descriptions.

Specimen	Damage Description	Cause of Failure	Justification
A1	Exterior Lite Fractured	Debris Impact	Large Shards, Low Stress Fracture Pattern, Interior Lite Unfractured, Impact Damage to Eave Fascia, Holes in Screen
B1	Both Lites Fractured	Debris Impact	Large Shards, Low Stress Fracture Pattern, Impact Damage to Window Frame, Debris Stuck to remaining Shards
B4	Lite Fractured	Debris Impact	Impact Damage on Door frame, Debris Stuck to Door Frame
D1	Both Lites Fractured	Debris Impact	Large Shards, Low Stress Fracture Pattern, Debris Stuck to remaining Shards, Adjacent Windows Unfractured
G1	Exterior Lite Fractured	Debris Impact	Interior Lite Unfractured, Adjacent Windows Unfractured
H1	Lite Fractured	Debris Impact	Impact Damage on Door frame, Adjacent Windows Unfractured
I2	Both Lites Fractured	Debris Impact	Debris Stuck to Shards, Large Shards, Low Stress Fracture Pattern, Holes in Screen
J2	Both Lites Fractured	Debris Impact	Debris Stuck to Shards, Large Shards, Low Stress Fracture Pattern, Holes in Screen
J3	Both Lites Fractured	Debris Impact	Debris Stuck to Shards, Large Shards, Low Stress Fracture Pattern
J4	Both Lites Fractured	Debris Impact	Debris Stuck to Shards, Large Shards, Low Stress Fracture Pattern
L1	Exterior Lite Fractured	Debris Impact	Interior Lite Unfractured, Large Shards, Low Stress Fracture Pattern with Clear Impact Fracture Pattern, Debris Stuck to Remaining Shards
L2	Exterior Lite Fractured	Debris Impact	Interior Lite Unfractured, Large Shards, Low Stress Fracture Pattern with Clear Impact Fracture Pattern, Debris Stuck to Remaining Shards
M2	Exterior Lite Fractured	Debris Impact	Interior Lite Unfractured, Large Shards, Low Stress Fracture Pattern with Clear Impact Fracture Pattern, Debris Stuck to Remaining Shards
O1	Lite Fractured	Debris Impact	Large Shards, Low Stress Fracture Pattern, Adjacent Pane Unfractured, Debris Stuck to Shards, Holes in Screen
P2	Lite Fractured	Debris Impact	Low Stress Fracture Pattern with Clear Impact Fracture Pattern

NOTE: Unlisted Specimens were unfractured.

Table A.4. Specimen ASTM E1300 Load Resistances.

	Undamaged LR_{0.008}	Damaged LR_{0.008}
Specimen	kPa (psf)	kPa (psf)
A1	8.9 (185)	5.2 (108)
B1	8 (168)	
B2	8 (168)	
B3	2.2 (46)	
B4	11 (229)	
C1	14.6 (304)	
D1	7.6 (159)	4.2 (88)
D2	7.6 (159)	
E1	5.5 (114)	
F1	11 (229)	
F2	5.2 (108)	
G1	2.8 (59)	1.4 (30)
G2	3 (62)	
H1	12 (250)	
I1	11 (229)	
I2	4.3 (90)	
J1	11 (229)	
J2	5.5 (114)	
J3	5.6 (117)	
J4	3.9 (82)	
K1	2.2 (45)	
K2	8.9 (186)	
L1	5.5 (114)	2.9 (60)
L2	3 (62)	1.5 (32)
L3	2.3 (48.5)	
M1	11 (229)	
M2	4.8 (101)	2.6 (53.4)
N1	12.9 (270)	
O1	11.6 (242)	
O2	11 (229)	
P1	2.2 (45)	
P2	2.3 (47.4)	

Table A.5. Specimen Nominal Wind Speeds.

	Nominal Wind Speed (Undamaged)					Nominal Wind Speed (Damaged)			
	A	B	C	D		A	B	C	D
Specimen	m/s (mph)	m/s (mph)	m/s (mph)	m/s (mph)		m/s (mph)	m/s (mph)	m/s (mph)	m/s (mph)
A1	93.9 (210)	105.5 (236)	104.2 (233)	120.7 (270)		72 (161)	80.5 (180)	79.6 (178)	92.1 (206)
B1	89.4 (200)	100.6 (225)	99.2 (222)	114.9 (257)					
B2	89.4 (200)	100.6 (225)	99.2 (222)	114.9 (257)					
B3	46.9 (105)	52.3 (117)	51.9 (116)	60.3 (135)					
B4	104.6 (234)	117.1 (262)	116.2 (260)	134.1 (300)					
C1	120.2 (269)	135 (302)	133.7 (299)	154.7 (346)					
D1	87.2 (195)	97.4 (218)	96.6 (216)	111.8 (250)		64.8 (145)	72.9 (163)	72 (161)	83.1 (186)
D2	87.2 (195)	97.4 (218)	96.6 (216)	111.8 (250)					
E1	73.8 (165)	82.7 (185)	81.8 (183)	94.8 (212)					
F1	104.6 (234)	117.1 (262)	116.2 (260)	134.1 (300)					
F2	72 (161)	80.5 (180)	79.6 (178)	92.1 (206)					
G1	53.2 (119)	59.5 (133)	59 (132)	67.9 (152)		38 (85)	42.5 (95)	42 (94)	48.7 (109)
G2	54.5 (122)	60.8 (136)	60.3 (135)	69.7 (156)					
H1	109.1 (244)	122.5 (274)	121.1 (271)	140.4 (314)					
I1	104.6 (234)	117.1 (262)	116.2 (260)	134.1 (300)					
I2	65.7 (147)	73.3 (164)	72.9 (163)	84 (188)					
J1	104.6 (234)	117.1 (262)	116.2 (260)	134.1 (300)					
J2	73.8 (165)	82.7 (185)	81.8 (183)	94.8 (212)					
J3	74.6 (167)	83.6 (187)	83.1 (186)	96.1 (215)					
J4	62.6 (140)	70.2 (157)	69.3 (155)	80.5 (180)					
K1	110.9 (248)	124.3 (278)	123.4 (276)	142.6 (319)					
K2	94.3 (211)	105.5 (236)	104.6 (234)	121.1 (271)					
L1	73.8 (165)	82.7 (185)	81.8 (183)	94.8 (212)		53.6 (120)	59.9 (134)	59.5 (133)	68.8 (154)
L2	54.5 (122)	60.8 (136)	60.3 (135)	69.7 (156)		38.9 (87)	43.8 (98)	43.4 (97)	50.1 (112)
L3	48.3 (108)	54.1 (121)	53.2 (119)	61.7 (138)					
M1	104.6 (234)	117.1 (262)	116.2 (260)	134.1 (300)					

M2	69.3 (155)	77.8 (174)	76.9 (172)	89.4 (200)		50.5 (113)	56.8 (127)	55.9 (125)	64.8 (145)
N1	113.5 (254)	127.4 (285)	126.1 (282)	145.7 (326)					
O1	107.3 (240)	120.2 (269)	119.3 (267)	138.1 (309)					
O2	104.6 (234)	117.1 (262)	116.2 (260)	134.1 (300)					
P1	46.5 (104)	51.9 (116)	51.4 (115)	59.5 (133)					
P2	47.4 (106)	53.2 (119)	52.7 (118)	61.2 (137)					

APPENDIX B

Specimen Photographs



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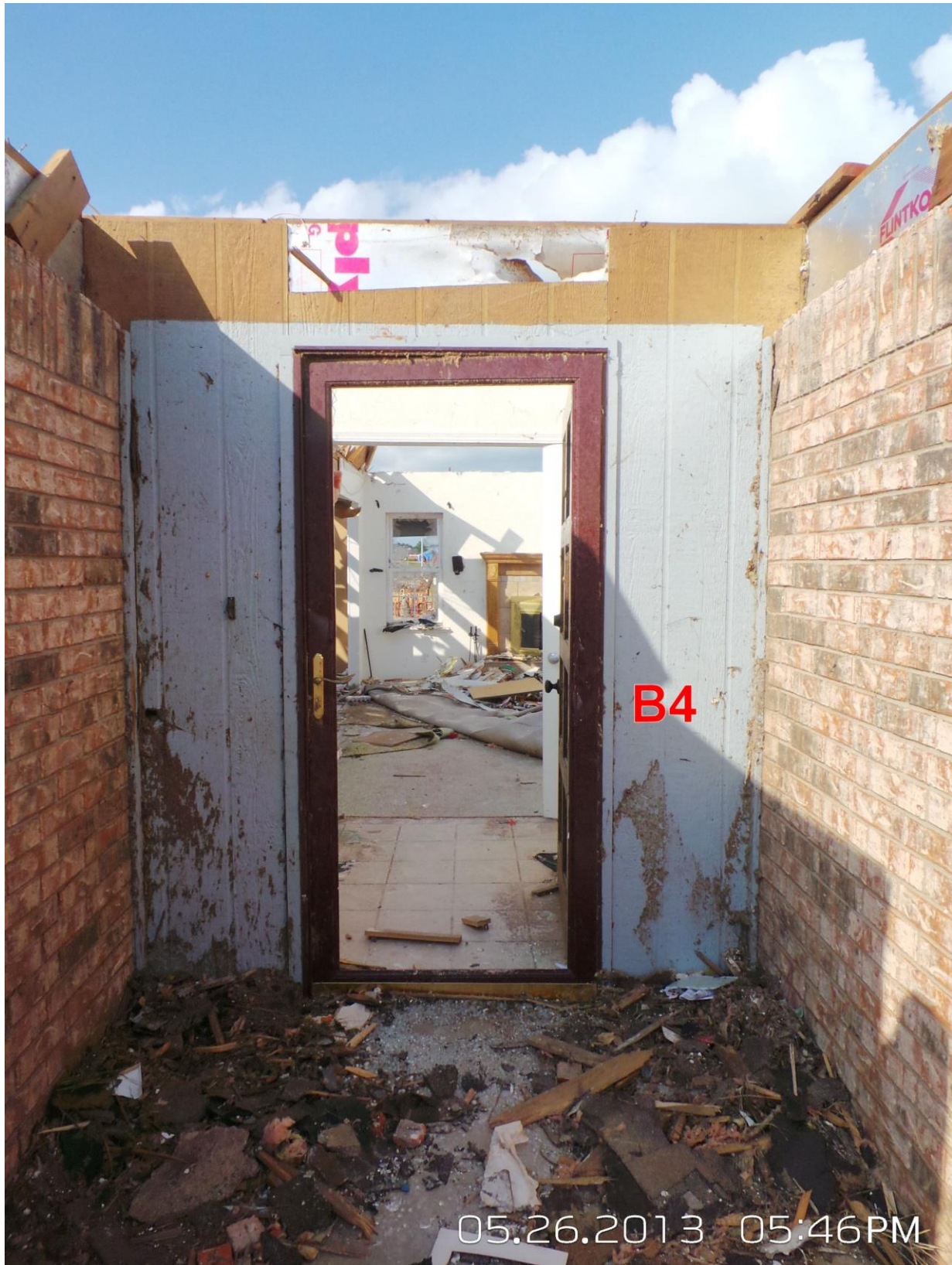


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Figure B.7. Specimen E1.



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