



Final Report

Tornado Damage Investigation

Greensburg, Kansas

1699 DR-KS

October 24, 2007



FEMA

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This document was prepared by



URS Group, Inc.

200 Orchard Ridge Drive, Suite 101
Gaithersburg, MD 20878

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1.1 DESCRIPTION OF THE EVENT

On the evening of May 4, 2007, “supercell” thunderstorms formed across portions of the Midwestern United States, spawning tornadoes in several States. An intense supercell developed southwest of Greensburg, KS that evening, resulting in the formation of 12 tornadoes. One of these tornadoes formed in northwest Comanche County at approximately 9:00 pm and moved northeastward through Kiowa County. At approximately 9:45 pm, this tornado reached Greensburg, KS, a small community of approximately 1,400 people. The tornado traversed the town traveling from the town’s southern edge to its northwest border. The tornado had a reported damage path that was 1.7 miles across and the funnel cloud itself was estimated to be 1 mile across. The tornado destroyed or severely damaged many of the buildings in Greensburg and caused the deaths of 10 people. The death toll in Greensburg might have been higher were it not for a tornado warning issued by the National Weather Service 20 minutes before the tornado reached town, giving the residents of Greensburg time to take refuge.

1.2 PURPOSE OF REPORT

The purpose of this report is two-fold: to provide a preliminary “ground-truth” of the new Enhanced Fujita (EF) Scale wind speed classification system¹ and to document the damage caused by the tornado. The tornado that struck Greensburg was one of the first tornado occurrences since the EF Scale was developed and thus provided an opportunity to compare the wind speeds derived through use of the EF Scale (observed damage) with wind speed calculated through material failure analysis.

The EF Scale uses observed Degrees of Damage (DOD) to derive wind speed ranges, which are then used to classify tornadoes. The report prepared by Texas Tech University on the EF Scale includes tables showing DOD for 23 different building types and provides wind speed ranges for each of the DOD. The ranges list lower-bound, upper-bound, and

The Fujita Scale categorizes tornado severity based on observed damage. The six-step scale ranges from F0 (light damage) to F5 (incredible damage). Since February 2007, the National Weather Service has used the Enhanced Fujita Scale (EF Scale). This new scale ranges from EF0 to EF5. See <http://www.spc.noaa.gov/efscale/> for further information on the EF Scale.

Fujita Scale		EF Scale	
Fujita Scale	3-Second Gust Speed (mph)	EF Scale	3-Second Gust Speed (mph)
F0	45–78	EF0	65–85
F1	79–117	EF1	86–109
F2	118–161	EF2	110–137
F3	162–209	EF3	138–167
F4	210–261	EF4	168–199
F5	262–317	EF5	200–234

¹ See the Enhanced Fujita Scale paper developed but the Wind Engineering Research Center of Texas Tech University, in cooperation with the U.S. Department of Commerce and the National Institute of Standards and Technology (NIST), and submitted to the National Weather Service in October 2006. This rating scale was the basis for the EF Scale rating that was implemented by the NWS as of February 2007 for classifying tornadoes and replaces the previously used Fujita Scale.

expected wind speed (see Table 1). All of the wind speeds are presented as 3-second gusts² to match with the ASCE 7 *Minimum Design Loads for Buildings and Other Structures* engineering loads standard. To use the EF Scale, the degree of observed damage to buildings is compared to the DOD for the appropriate building type, thereby yielding a probable wind speed range.

The table below shows an example of the EF Scale tables. The range of wind speeds is a function of variations in the wind resistance of specific buildings (due to design, construction, and maintenance variations) and uncertainty in the wind speed necessary to cause a specific type of damage.

Degree of Damage (DOD)	Damage Description	Expected wind speed (Exp)	Lower-bound wind speed (LB)	Upper-bound wind speed (UP)
		Wind speed values in miles per hour (mph)		
1	Threshold of visible damage	65	53	80
2	Loss of roof covering material (<20%), gutter and/or awning; loss of vinyl or metal siding	79	63	97
3	Broken glass in doors and windows	96	79	114
4	Uplift of roof deck and loss of significant roof covering material (>20%); collapse of chimney; garage doors collapse inward or outward; failure of porch or carport	97	81	116
5	Entire house shifts off foundation	121	103	141
6	Large sections of roof structure removed; most walls remain standing	122	104	142
7	Top floor exterior walls collapsed	132	113	153
8	Most interior walls of top story collapsed	148	128	173
9	Most walls collapsed in bottom floor, except small interior rooms	152	127	178
10	Total destruction of entire building	170	142	198

Source: *A Recommendation for an Enhanced Fujita Scale* submitted to the National Weather Service by the Wind Science and Engineering Center at Texas Tech University on January 26, 2006 and revised in October 2006.

1.3 DESCRIPTION OF THE WORK

A site visit to Greenburg was conducted on May 10-11, 2007, with URS structural engineers and consultants who have shelter design experience, as well as hazard mitigation and storm damage investigation experience. A second site visit took place on May 21, 2007. The site investigations

² Unless otherwise noted, all wind speeds stated in this report are 3-second gust wind speeds at 33-feet above grade for Exposure C.

were coordinated with Jim Donley of FEMA Region VII and Chris Hudson of FEMA Headquarters Mitigation.

URS was tasked with assessing the damage in Greensburg and with classifying the tornado using the Enhanced Fujita tornado damage scale. The methods used in conducting this work are described in the section titled Study Methodology.

No attempt was made to compare the tornado wind speeds experienced in Greensburg with any building code design wind speeds. Most of the buildings in Greensburg were designed and constructed to older building codes, such as the Uniform Building Code (UBC), although a few newer buildings may have been designed and constructed in accordance with the International Building Code (IBC). Regardless of the code used (IBC or older code such as the UBC), there are currently no design guidelines in the IBC for catastrophic wind events of the magnitude experienced by Greensburg (although there are high wind provisions in the building code for hurricane-prone areas), so a building code comparison was judged to not provide any useful or concluding information.

The work also included developing three recovery advisories that the community, or any community that is impacted by a tornado, can use as reconstruction guidance. One tornado risk advisory was developed to provide the community with a better idea of how the risk of tornadoes in Greensburg might compare with other areas of the country and how that risk might affect decision-making about shelter locations, design, and construction. A second tornado risk advisory was written to provide storm shelter design criteria describing the design concepts important for tornado events. The third tornado risk advisory was written to provide residential sheltering guidance to residents who want to create a safe refuge inside their home. These advisories will soon be available on FEMA's website.

2.1 STUDY METHODOLOGY

In order to assess the damage in Greensburg, two site visits were conducted (May 10-11 and May 21). The observations made during the site visits were used for two purposes. First, the observed damage for 46 residential structures was used to rate the damage according to the EF Scale and thereby derive estimated wind speeds (see Section 3.1). The estimated wind speeds were then mapped (Section 3.3). In addition to these 46 residential structures, additional DOD assessments were made in the field; these assessments included damage to seven non-residential structures and trees in the tornado path.

Second, specific damage information was collected at four sites to perform structural analysis and evaluate the failure stresses in the materials. These buildings were selected where all or a portion of a building was damaged or destroyed, where the damage had not been disturbed, and where, in the Study Team's opinion, there was a possibility of back-calculating the pressures causing the failures to determine the approximate wind speed. Detailed information was collected for: the elementary school pre-engineered metal building (PEMB), the John Deere PEMB, the precast concrete tee roof failure at the hospital, and the glulam beam failures in the First United Methodist Church. After performing a materials analysis to calculate the wind speeds required to cause the observed failures (Section 3.2), the calculated wind speeds were mapped and compared to the wind speeds derived from the EF Scale assessment (Section 3.3).

2.2 FIELD OBSERVATIONS

1. The primary site observations were obtained from an initial study of overhead imagery and initial site inspections conducted on May 10-11, 2007:
 - a. The most severe swath of damage through the center of town was approximately five to six blocks wide. The damage on either side of this swath was less for about one to two blocks on each side, with even less damage observed for another one to two blocks on the extreme outer edges of the damage swath. See Figure 1 for an aerial view of Greensburg after the event and Figure 8 for a map of the damage swath.
 - b. Many of the residential buildings were observed to have basements and some of these buildings were shifted on their basement foundations.
 - c. The roofs of many of the buildings located near the edge of the storm's center and along the storm's path had roof coverings that either completely or partially survived the tornadic winds.
 - d. Most residential and commercial buildings in town were older than any of the model building codes. Exceptions were the John Deere Building built in the mid-1990s and the elementary school PEMB erected in 2002 (see Figure 8 for locations). The residential and commercial buildings observed during the site visit did not have any specific design features that would have been intended to protect building occupants from the effects of this tornado except for basements or below ground areas and any small interior room, such as a closet or bathroom, used as a best available refuge area. No "tornado shelters" designed to resist the wind and debris associated with a tornado were identified during this effort.



Figure 1: Greensburg, KS, after May 4, 2007 tornado event

2. Corey Schultz of PBA Architects (a subcontractor located in Wichita, KS) conducted a second site visit on May 21, 2007, specifically to evaluate roof covering damage. The data collected from this site visit is included in Appendix A.
 - a. Forty-six buildings were inspected (see Figure 8 for building locations).
 - b. Photos were taken to document the damage.

All 46 buildings were assessed and plotted using the EF Scale as described in Section 3.1. The four buildings subject to structural analysis are described in detail in Section 3.2 and the results of the calculated wind speed for these buildings are compared with the wind speed derived by applying the EF Scale. A map showing both sets of wind speeds is included in Section 3.3 as Figure 8. The tornado was rated an EF5 with an estimated wind speed greater than 200 miles per hour (mph).

3.1 APPLICATION OF THE EF RATING SCALE TO OBSERVED DAMAGE

Based on a review of the EF Scale and the damage levels associated with the 23 various building types, the Study Team plotted the Greensburg damage according to the EF Scale wind speeds, as close as possible, to delineate wind speeds and associated degree of damage. Information from the field visits, described in Section 2, was developed initially by driving around the town, taking photos of damage, and attempting to classify the damage as minor, moderate, major, or destroyed. The Study Team initially attempted to map this damage by plotting, in block-size elements, the type of damage that occurred by block; however, there were many places where there was a gradation of the primary damage level.

The second site visit (May 21, 2007) gave the Study Team another opportunity to classify the damage. All of the 46 buildings were classified by DOD so these buildings could be used to help determine wind speeds. The DOD and the associated EF Scale rating are shown in Table 2 below. The analysis and resulting damage classification by EF Scale is presented by building on a map of Greensburg (refer to Figure 8 in Section 3.3).

Table 2. Results of Greensburg building classification by DOD			
DOD	Number of houses	Expected Wind Speed (mph)	EF Scale Rating
2	15	79	EF0
3	7	96	EF1
4	13	97	EF1
6	7	122	EF2

Note: Four structures inspected did not have an associated photo, so identifying the DOD was not possible.

3.2 STRUCTURAL ANALYSIS RESULTS FOR FOUR BUILDINGS

For the four sites studied in detail, the Study Team used data on the ultimate strengths of the materials that failed to determine the wind pressures, and thus the wind speeds that caused the observed damage. Calculated wind speeds for building damage or failures are shown on Figure 8. Detailed calculations are included in Appendix B.

3.2.1 Elementary School

The school is a PEMB located on the southeast edge of Greensburg. This is the side of town that received the strongest tornado winds coming from the east. Figure 2 shows the school gymnasium blown over and leaning to the west, consistent with the wind coming from the east.

Materials Analysis

A key damage observation judged to be useful in determining the wind speed that caused the building collapse was the failure at a column base plate where all four steel bolts sheared off. The bolt failure was a twisting (shearing) tensile failure as illustrated in Figure 3 (example shown had three bolts sheared off).

The bolts were assumed to be A307 and are 5/8" in diameter. The ultimate tensile load required to snap the bolt was determined to be 25 kips³ (k) per square inch (ksi) or 7.67 k for a 5/8-inch diameter bolt. The required uplift would be 30.68 k for the column attached to this base plate. The tributary area for this column was 30 feet (height of wall) times 20 feet (column spacing).

The wind pressure (q) required for this force is $30.68\text{k}/600 \text{ square foot (sf)} = 51.1 \text{ pounds per square foot (psf)}$. The wind speed (velocity, V) required to develop 51.1 psf using the external pressure coefficients noted below could have ranged from 134 mph to 330 mph. The Study Team judged the combination of external and internal pressure coefficients that equal 1.0 fairly represent what the physical evidence suggested. The use of external and internal pressure coefficients that sum to 1.0 results in the velocity being approximately 153 mph (3-second gust) when the velocity pressure equation is solved for V (Velocity) and when $q = 51.1 \text{ psf}$.

Depending on how the building really failed, the roof coefficient (from ASCE 7) could be -0.9 and the wall coefficients could be +0.8 to -0.5. The internal pressure coefficients were assumed to be +/- 0.55 for a partially enclosed condition. A gust effect factor of 0.85 was used. There are many things that could have happened to shear four bolts in a column base plate. In one possible scenario, the building could start to collapse and then the weight of the building would push the building over, pulling the base plate from the concrete and shearing off the four bolts.



Figure 2: PEMB Elementary School



Figure 3: Bolt Failure

³ 1 kip = 1000 lbs

Comparison of Calculated Wind Speed with EF Scale Wind Speed

The calculated wind speed of 153 mph represents a DOD 8 for a metal building system (building type number 21) in the new EF Scale. The DOD 8 is described as total destruction of the building resulting from an expected wind speed of 155 mph. The Greensburg wind speed result for this elementary school PEMB provides one data point on the DOD 8 scale that corresponds with the field damage, as represented by the four failed column base plate bolts.

3.2.2 Church

The First United Methodist Church is located on the west side of Greensburg in an area that appears to have experienced mostly EF0-EF2 wind speeds (a range of 65-137 mph, see Figure 8) based on its ranking using the EF Scale (Section 3.1). The church roof was framed with 3 ¼-inch wide by 11½-inch deep laminated timber beams. These beams were bolted into steel saddles on each side of the church sanctuary. The saddles were connected to concrete girders shown in Figure 4.



Figure 4: Laminated Roof Beam Failure

Materials Analysis

The failure mode appears to be roof uplift. Uplift forces acting on the roof appear to have separated the laminated beams from the steel saddles as illustrated in Figure 5. As the laminated beam separated from the steel saddles, the original bolt holes became slots at this failure point. The bolt slots that are ripped in the beam are 4¼-inch long. There appeared to be some deterioration in the end of the beam that would likely reduce the shear strength parallel to the grain of the beam.



Figure 5: Glulam Beam from Church

The shear design value from the National Design Specification for Structural Glued Laminated Timber is 200-250 pounds per square inch (psi). The failure shear value might be approximately the same given the condition of the end of the beam.

The calculated area of each slot failure is 13.32 square inches (sq in), or 27.62 sq in for the two slots. It is likely the wind started lifting up on the roof and while the roof failure might have been complete and total within seconds, the failure could have occurred as a result of a series of events. As the roof lifted, the bolt slots are likely to have failed some, giving the bolts room to move and thus creating further stress on the wood, eventually allowing the wood beam to pull away from the bolts that secured the wood beam to the steel saddles. If the bolt slot was

lengthened to only 1 inch, instead of the full 4 ¼ inches, and this constituted failure, the force required to pull the wood beam up 1 inch is 1 inch x 3 ¼ inches x 200 psi = 650 pounds (lbs) x 2 bolts = 1,300 lbs.

The uplift force had to overcome the weight of the wood beam, which was approximately 227 lbs, so the total uplift force required was 1527 lbs.

The beams were spaced 4 feet on center and spanned 20 feet. There was a 3-foot overhang on each edge of the roof. The tributary area for the beam receiving the uplift pressure was approximately 64 sf. The uplift pressure was 1,527 lbs/64 sf = 23.2 psf. Assuming roof external pressure coefficients of -0.9 and a partially enclosed condition, the wind speed calculated from these conditions is 91 mph.

Comparison of Calculated Wind Speed with EF Scale Wind Speed

There is no building type in the EF Scale that represents a large, high ceiling timber structure. Using the EF Scale, the 91 mph wind speed lies within the range of wind speeds for an EF1 tornado. The church is located within an EF1 wind swath according to the analysis described in Section 3.1.

3.2.3 Hospital

The hospital is located north of Garfield Street and between Grove and Walnut Streets. The hospital roof was flat and framed with precast concrete double tees 40 inches wide and spanning 34 feet with 3-foot overhangs on each end (Figure 6). The precast concrete tee was calculated to weigh 9,979 lbs. There was an additional 15 psf of roof covering (roof felts and gravel ballast) on the roof at the time of the tornado strike. The precast concrete tee was not attached at all to the wall; it was only resting on the wall and held in place by gravity. The failure mode was the precast tee lifted off the hospital walls and was moved 80 feet from its original location on the roof. It is uncertain how the concrete tee reached this location. It could have tumbled across the roof or been picked up and thrown.



Figure 6: Hospital with Precast Concrete Twin Tees

Materials Analysis

The wind pressure required just to move the concrete roof tee (with its roof covering) was 90 psf. If the pressure coefficients are assumed to be those used for Components & Cladding and the edge pressures are increased for the flat roof system, a 147-mph wind speed would be required to create the suction forces large enough to lift the tee off the roof.

As a result of this HMTAP task order work, data was collected and presented as a case study in a new FEMA publication, FEMA 577- *Design Guide for Improving Hospital Safety in Earthquakes, Floods and High Winds: Providing Protection to People and Buildings* (2007).

Comparison of Calculated Wind Speed with EF Scale Wind Speed

The EF Scale building type most resembling this hospital is number 20, described as having precast concrete roofs. The calculated wind speed of 147 mph required to lift off the precast concrete twin roof tee corresponds with DOD 8, the DOD representing uplift of the slabs. A DOD 8 has an expected wind speed of 142 mph. These wind speeds represent a tornado rated an EF3.

There was a new PEMB at the hospital that was also damaged (described in a case study in FEMA 577). The DOD for the damage observed at that PEMB building indicated an expected wind speed of 155 mph. These results are consistent with the other wind speed determinations for the hospital.

3.2.4 John Deere Building

The John Deere Building is located on the northwest side of Greensburg near the western edge of the tornado. The building is a PEMB and was damaged by winds that came from the west. The primary damage was to the steel roof purlins. Most of the structural framing remained intact as illustrated in Figure 7. The failure mode was the purlins failed in buckling as the roof sheathing was torn off. This failure mode is a good determinant of the wind pressure as a DOD damage state is roof purlin buckling on PEMBs.

Materials Analysis

The purlins were 8-inch Z channels spaced 5 feet on center and 16 feet long. The Moment of Inertia (I) for the purlins was calculated to be 0.281 in⁴ and a critical buckling load of 2,181 lbs was determined from the buckling formula ($P_{cr} = \pi^2 EI/L^2$).

The wind pressure required to create a 2,181-lb buckling load is 27.8 psf. The wind speed required to develop this pressure is 122 mph.

Comparison of Calculated Wind Speed with EF Scale Wind Speed

From the EF Scale for metal building systems (building type number 21 on the scale) in the new EF Scale, DOD 5 is for buckling of roof purlins with an expected speed of 118 mph. The calculated speed of 122 mph agrees with the EF Scale DOD value. The 122 mph wind speed represents a tornado rated an EF2.



Figure 7: PEMB for the John Deere Building

3.3 MAPPED RESULTS

Figure 8 presents the wind speed map for the Greensburg, KS tornado. This map represents the combined result of the many observations involved in deriving wind speeds using the EF Scale and the wind speeds calculated from the materials analysis performed for four buildings. The width of the actual swaths of various wind speeds may have varied from those represented. The

data collected and the variability of a tornado make a more precise determination difficult and of little value.

This report validates the estimated wind speeds resulting from use of the new EF Scale DODs. This is the first effort the Study Team is aware of in which the DODs have been validated since the development of the new EF Scale.

The 46 buildings surveyed for damage and used to estimate the wind speeds based on DOD are shown on the map (Figure 8). Where possible, the specific DOD for the building is shown. The results of the calculated wind speeds for the four sites where detailed structural analysis was performed are also shown on Figure 8. The wind speeds calculated from structural analysis were consistent with the wind speed swaths derived from the DOD information.

During the site visits, damage to other infrastructure (i.e., power station, courthouse, high school, fire station, etc.) was observed and these facilities are identified on Figure 8. Calculations of estimated wind speeds that could have caused the damage were not performed for these buildings or vegetation.

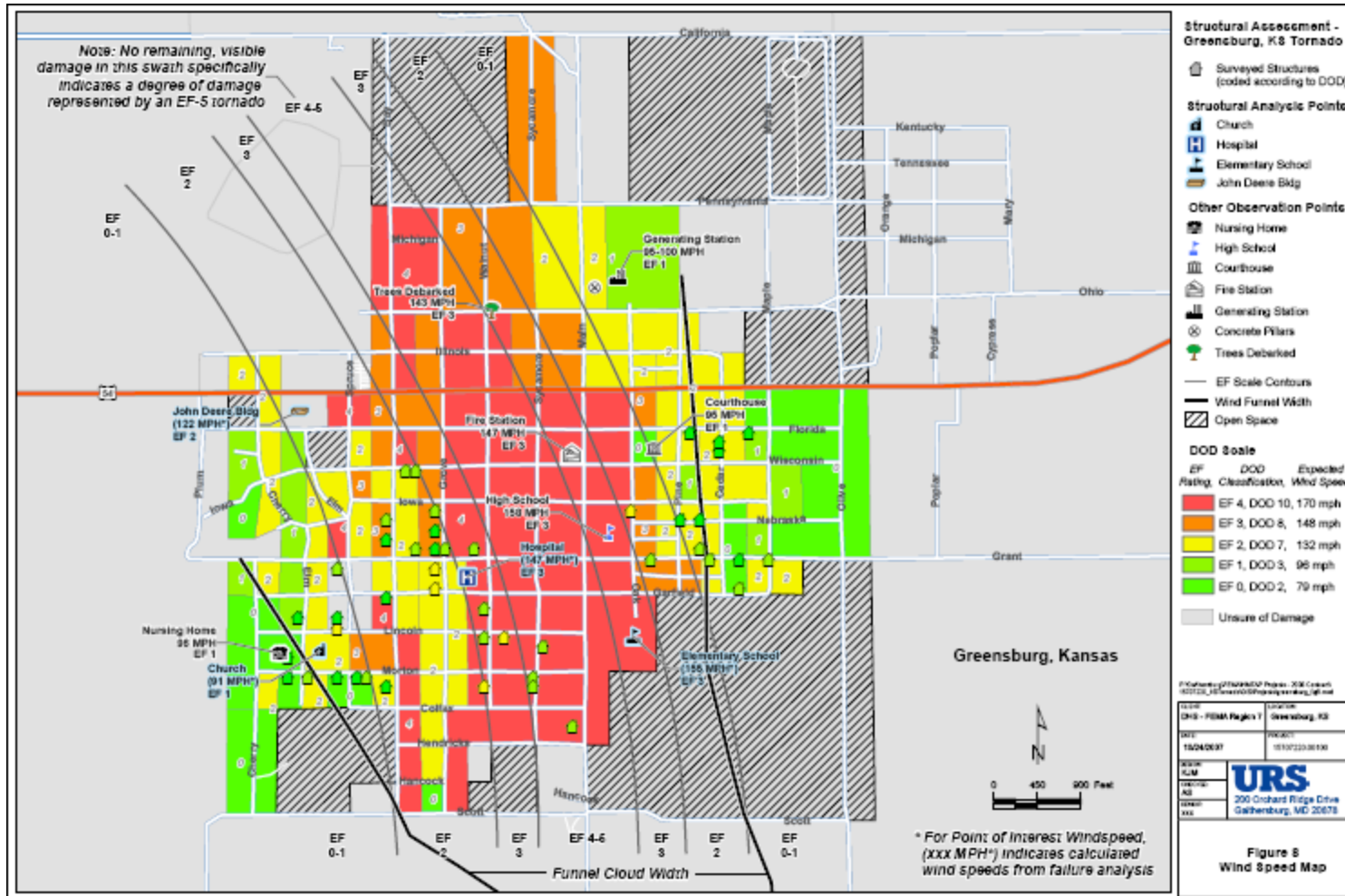


Figure 8: Wind Speed Map for Greensburg, KS Tornado

From the observed damage and the building-specific failures noted in the report, the EF Scale DOD determinations correspond well with the associated calculated wind speeds. The mapping of the approximate wind speeds can illustrate wide fluctuations for tornados because of the severity of and rapidly changing wind speeds within the storm. There have been few opportunities to gather this type of information about tornado damage since the revision of the EF Scale, and these few data points help build the body of information required to verify the EF study findings.

While the examples of failures from this event provide only a few specific data points for calibrating the new EF Scale, similar comparison studies should continue to build on these results. These studies would serve to validate the speeds associated with the DODs, or would serve to suggest that some of the DOD speeds need to be adjusted.

Specific recommendations for the EF Scale based on this study:

- Consideration should be given to adding two additional building types to the EF Scale. One would be old load-bearing masonry buildings (of the type built in the early part of the 20th century). There were many of these types of buildings in Greensburg and there are many throughout the Midwest. The other building type to add would be timber frame buildings similar to the First United Methodist Church sanctuary. Timber stresses are very different than other materials and are subject to wide variability in response to extreme loads due to the condition and age of the timber. DOD scales for both of these building types would help fill in gaps in the development of probable wind speeds after tornadoes.
- Testing of the methodology used in this study is encouraged. There are many ways building failures can occur and the failure modes selected and studied from this event for the four specific cases are only one of several ways the failures could have occurred. Different hypotheses can be developed for each failure case. These different hypotheses and failure modes should be debated and studied in order to further assess the DOD and EF Scale ratings.

Other findings arising from this study:

- This study supports the findings (discussed after the May 1999 tornado outbreak in Oklahoma and Kansas) that there are wind speed bands on the outer edges of even the deadliest tornadoes in which buildings and occupants can survive if those buildings are built to current high wind design building codes.
- This event confirmed that the tornado debris field is filled with many types of missiles (wood studs, steel stairs, pieces of steel tanks, bricks, tree limbs, automobiles, farm equipment, etc.). The current shelter test missile is a 15-lb, 2-foot x 4-foot wood stud. The Study Team recommends additional study on how debris fields in this suburban-like environment affect building performance since multiple missiles in the momentum range of the current test missile could strike a building, causing a breach in the envelope and thus potentially injuring or killing building occupants.
- Future studies may be able to use of DOD and failure analysis information to develop ways to improve specific building element performance in a tornadic event or to study other types of building materials or connections that might improve building performance. This type of

study might lead to better utilization of existing building space for ‘sheltering in place’ situations.

Wind designs developed for building code wind speeds in hurricane-prone areas (for most of the country this is a Category 3 hurricane on the Saffir-Simpson scale) would provide protection for nearly all tornadoes classified EF3 and lower. In tornado-prone areas, the high wind design needs to be accompanied by windborne debris protection because of the numerous high speed missiles generated by a tornado event. Adoption of these measures would provide protection for most tornado events in the country.

The primary protection from tornado events, however, should be a shelter—either in the residence or a nearby community shelter. Buildings built specifically to withstand the extreme winds of violent tornadoes and provide protection from wind-borne missiles will provide the most protection. Given that Greensburg, KS is located in ‘Tornado Alley’, and that the history of the area is replete with many examples of tornadoes, shelters built specifically for tornado events should be a significant part of rebuilding this community. There was a surprising lack of designated shelter space in this community, especially in light of the fact they are in a very active tornado region of the country.

AISC Manual of Steel Construction, Load & Resistance Factor Design, Second Edition, 1998

ASCE 7-05, *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers, 2005

FEMA 320, *Taking Shelter From a Storm: Building a Safe Room Inside Your House*, Federal Emergency Management Agency, 1998

FEMA 361, *Design and Construction Guidance for Community Shelters*, Federal emergency Management Agency, 2000

FEMA 577, *Design Guide for Improving Hospital Safety in Earthquakes, Floods and High Winds: Providing Protection to People and Buildings*, 2007

National Design Specification for Wood Construction, 1997

Adrienne Sheldon, P.E. URS
Thomas Smith, AIA TLSmith Consulting
Corey Schultz, AIA PBA Architects
Bill Coulbourne, P.E. URS

Report Review provided by: Scott Tezak, P.E. URS

DOD	Degree of Damage
E	Modulus of Elasticity (psi)
EF	Enhanced Fujita
I	Moment of Inertia (in ⁴)
IBC	International Building Code
k	kips; 1 kip = 1,000 pounds
ksi	kips per square inch
lbs	pounds
mph	miles per hour
PEMB	pre-engineered metal building
psf	pounds per square foot
psi	pounds per square inch
q	velocity wind pressure (psf)
sf	square foot
sq in	square inch
UBC	Uniform Building Code
V	Velocity

Appendix A

Data Collected from PBA Architects' May 21, 2007 Site Visit

Greensburg Kansas Tornado Post-Disaster
Roof Shape/Damage Assessment Field Data
21-May-07

Data Collected from PBA Architects May 21, 2007 Site Visit

RESIDENCES IN THE CENTER OF THE DAMAGE PATH

Structure #	Photo #	Location		Building Information									Miscellaneous Information				Notes
		Latitude	Longitude	Structure Type	DOD	Length	Width	Orientation of Long Dimension	Roof Shape	Eave Height	Ridge Height Adv. Eave (Est.)	Overhang Width	Exposure Class'n	Wind Direction	Type of Roof Damage	Power Pole/Tree Trunk Damage	
1	1	37°41'23N	097°19'29W	RES	6	42'	38'-0"	SQ	Gable	15'-0"	5'-5"	12"	B	S	Stht'g & Rafters	PP Leaning	2 story, Balloon framed
2	2,3	37°35'56N	099°17'38W	RES	4	35'	30'-0"	N/S	Gable	8'	5'-4"	12"	B	S	Stht'g missing	PP Down	1 story, all metal (Lustron)
3	4	37°35'53N	099°17'39W	RES	4	47'	26'	N/S	Hip	7'-2"	N/A	24"	B	SE	Stht'g missing SE Corner	PP Leaning	1 story
4	6	37°35'52N	099°17'39W	RES	4	60'	25'	N/S	Gable	6'-10"	1'-8"	12"	B	SE	Stht'g missing W Side	PP Leaning	1 story, low slope, portion of East shtth'g at garage
5	7	37°35'48N	099°17'35W	RES	4	39'	26'	E/W	Gable, L-Shape	8'-0"	7'-6"	12"	C	S	Stht'g missing NW Corner	Tree Trunk Broken	1 story, L-shape roof
6	8	37°35'52N	099°17'44W	RES	6	65'	37'	N/S	Gable, U-Shape	7'-0"	3'-6"	Varies 12"-36"	C	E	Stht'g & Struct missing East side	PP Down	1 story, low slope, dense hedge on S side damage from trailer, South dense tree hedge appeared to protect house from South wind
7	9,10,11	37°35'57N	099°17'44W	RES	4	64'	26'	E/W	Gable	7'-9"	5'-0"	8"	B	SW	Shingles missing, cave in @ S side	PP Leaning	1 story ranch, debris penetration of roof shtth'g
8	12	37°35'57N	099°17'42W	RES	6	37'	28'	E/W	Gable, L-Shape	7'	7'	12"	B	SW	NW Corner roof missing	PP Leaning	1 story ranch
9	13	37°36'00N	099°17'44W	RES	3	68'	28'	N/S	Hip	7'	N/A	24"	B	SW	N Hip Stht'g & Struct missing	Tree Leaning	1 story ranch, garage on North roof missing
10	14	37°36'02N	099°17'49W	RES	6	58'	24'	N/S	Gable, L-Shape	7'	5'	30"	B	SW	E/W Roof missing	FP Leaning	1 story ranch, E/W structure missing, garage roof missing
11	15	37°36'04N	099°17'49W	RES	6	40'	37'	N/S	Gable, L-Shape w/ shed roof	8'-4"	4'-8"	18"	B	SW	NE Roof missing	-	1 story ranch
12	18	37°36'06N	099°17'51W	RES	4	50'	26'	N/S	Gable @ N, hip	9'-2"	N/A	12"	B	SW	N gable roof missing	PP Broken	1 story ranch back half lower gable missing
13	21	37°36'06N	099°17'49W	RES	2	58'	26'	N/S	Hip	8'-0"	N/A	0"	B	SW	Shingles missing	PP Broken	1 story ranch, no overhang, has chimney over
14	23	37°36'06N	099°17'48W	RES	4	48'	26'	N/S	Gable	8'-4"	6'-2"	0"	B	SW	Shingles missing	PP Broken	1 story ranch
15	24	37°36'06N	099°17'45W	RES	4	50'	27'	E/W	Gable	8'-2"	5'-0"	0"	B	SW	Shingles missing	PP Leaning	1 story ranch
16	-	37°36'08N	099°17'49W	RES		32'	32'	N/S	Hip	8'-2"	N/A	0"	B	SW	Shingles missing, debris impact on South	PP Leaning	1 story ranch
17	25,26	37°36'10N	099°17'49W	RES	4	74'	44'	N/S	Irregular Hip	6'-10"	N/A	24"	B	SW	Shingles missing, Stht'g damage on W	PP Leaning	1956 1 story ranch, NE corner of roof missing
18	28	37°36'14N	099°17'51W	RES	3	45'	32'	N/S	Gable	8'	11'-4"	0"	B	SW	Shingles missing	PP Leaning	2 story
19	29	37°36'14N	099°17'52W	RES	3	56'	28'	N/S	Gable	9'	6'	24"	B	SW	Eave missing, shingles missing	PP Leaning	1 story, SW roof missing, N porch missing

Greensburg Kansas Tornado Post-Disaster
Roof Shape/Damage Assessment Field Data
21-May-07

RESIDENCES ON BAY STREET AND WEST

Structure #	Photo #	Location		Building Information									Miscellaneous Information				Notes
		Latitude	Longitude	Structure Type	DOD	Length	Width	Orientation of Long Dimension	Roof Shape	Eave Height	Ridge Height Adv. Eave (Est.)	Overhang Width	Exposure Class'n	Wind Direction	Type of Roof Damage	Power Pole/Tree Trunk Damage	
20	30	37°36'09N	099°17'54W	RES	3	52'	27'	E/W	Hip with protruding gables	9'	N/A	18"	B	SW	Impact damage @ peak	-	W protruding gables, 1 story, West shed roof missing
21	31	37°36'07N	099°17'54W	RES	2	35'	21'	E/W	Hip	7'-5"	N/A	12"	B	SW	South roof bowed, shingles missing	-	1 story ranch
22	32	37°36'01N	099°17'54W	RES	2	62'	25'	N/S	Gable	6'-8"	1'-8"	36"	B	SW	Shingles missing	-	1 story ranch, low slope
23	34,38	37°35'52N	099°17'54W	APT	2	89'	29'	N/S	Stepped Gable	7'-2"		2'-6' varies	C	SW	South roof struct missing, shingles missing	PP Leaning	1 story apt bldg, Elmore Heights Apartments
24	39	37°35'53N	099°17'56W	APT	4	"	"	E/W	Stepped Gable	"	"	"	C	SW	Heavy wall roof damage West end	-	Similar to 23
25	-	37°35'53N	099°17'57W	APT		"	"	"	"	"	"	"	C	SW	Heavy wall roof damage West end	-	Similar to 23
26	42	37°35'53N	099°17'59W	RES	2	60'	30'	N/S	Hip	8'	N/A	24"	C	SW	West hip Shth'g missing	-	1 story ranch, roof missing above garage door
27	43	37°35'53N	099°18'02W	RES	3	66'	26'	E/W	Gable	8'	4'	12"	C	SW	Shth'g missing @ SW corner	-	1 story double wide pre-manufactured house
28	-	37°35'53N	099°18'04W	APT		100'	24'-4"	N/S	Hip	7'-4"	8:12 slope	12"-20"	C	SW	Section of roof sheathing removed on South side	-	1 story apts across from nursing home
29	44,45	37°35'55N	099°18'04W	APT	2	200'	200'	E/W	Gable	8'-0"	8'-0"	24"	C	SW	S & W shingles missing	-	Nursing home - West end of canopy damaged
30	47	37°35'59N	099°18'03W	RES	2	70'	28'	N/S	Hip	7'-3"	5:12 slope	20"	B	SW	Covered	Trees broken	Roof was covered completely by tarp, there did not appear to be any structural damage except for garage roof
31	48	37°35'58N	099°17'59W	RES	6	83'	31'	N/S	Hip	7'-0"	unknown	20"	B	SW	Roof structure missing	PP Leaning	Compare to #32, side by side
32	49	37°35'59N	099°17'59W	RES	2	67'	32'	N/S	Hip	6'-9"	4:12 slope	26"	B	SW	Shingles missing	PP Leaning	Similar to adjacent #31, note lack of damage
33	50	37°36'04N	099°17'59W	RES	3	32'	26'	N/S	Gable	7'-9"	13'-4"	16"	B	SW	Shingles missing west side	PP Broken	2 story 12:12 pitch roof

Greensburg Kansas Tornado Post-Disaster
Roof Shape/Damage Assessment Field Data
21-May-07

RESIDENCES ON OAK STREET AND EAST

Structure #	Photo #	Location		Building Information									Miscellaneous Information				Notes
		Latitude	Longitude	Structure Type	DOD	Length	Width	Orientation of Long Dimension	Roof Shape	Eave Height	Ridge Height Adv. Eave (Est.)	Overhang Width	Exposure Class'n	Wind Direction	Type of Roof Damage	Power Pole/Tree Trunk Damage	
34	51	37°36'10N	099°17'29W	RES	6	32'	24'	N/S	L-shaped Hip	7'-3"	4:12 slope	20"	B	SE	E/W Wing missing, East shingles missing	PP Leaning	1 story ranch, East portion of house missing
35	52	37°36'05N	099°17'27W	RES	4	59'	26'	E/W	L-shaped Gable	6'-9"	5'-0"	12"	B	SE	Sth'g impact damage SW corner	PP Leaning	1 story ranch
36	54	37°36'06N	099°17'22W	RES	4	48'	28'	N/S	Gable	7'-7"	6'-0"	12"	B	SE	SW corner structure missing	PP Broken	1 story ranch, South porch missing
37	55	37°36'05N	099°17'21W	RES	4	39'	24'	N/S	U-shaped Gable	8'	7'-2"	12"	C	SE	SW corner structure missing	PP Broken	1 story ranch
38	56	37°36'02N	099°17'18W	RES	4	46'	33'	E/W	L-shaped Gable	7'-2"	2'-8"	8"	C	SE	Sheathing damage at SE corner	PP Broken	3 houses together - similar 1 story ranch-very little damage considering exposure classification
39	57	37°36'05N	099°17'15W	RES	3	45'	47'	E/W	Hip	15'-8"	8:12 slope	12"	B	SE	Sth'g damaged at ridge	PP Broken	2 story
40	58	37°36'05N	099°17'18W	RES	2	32'	33'	E/W	L-shaped Gable	7'-4"	5'-6"	8"	B	SE	Shingles damaged	PP Broken	1 story ranch
41	59	37°36'09N	099°17'22W	RES	2	59'	28'	N/S	Gable	9'-0"	6'	30"	B	SE	Sth'g missing west side	PP Broken	1 story bungalow
42	60	37°36'09N	099°17'24W	RES	2	49'	38'	E/W	Gable with dormers and shed on west	12'-6"	5'-4"	12"	B	SE	Tree limb debris impact on South side	PP Broken	2 story
43	61	37°36'16N	099°17'20W	RES	2	20'	25'	N/S	Gable with dormer	12'-6"	12'-4"	18"	B	SE	Shingles missing on E & W	Tree trunk broken	2 story
44	62	37°36'18N	099°17'17W	RES	2	37'	30'	E/W	Hip	9'-2"	12:12 slope	12"	B	SE	Impact damage	Tree trunk broken	1 story
45	63,64	37°36'17N	099°17'20W	RES	2	43'	28'	E/W	Dutch hip	9'-2"	10:12 slope	12"	B	SE	Shingles missing on N & S	Tree uprooted	1 story
46	65,66	37°36'18N	099°17'23W	RES	2	60'	33'	N/S	Gable with dormer	8'-4"	12'	36"	B	SE	Sth'g missing @ SW corner	Tree uprooted	2 story bungalow

Appendix B
Detailed Calculations of Damage Classification

ELEMENTARY SCHOOL PEMB BOLT FAILURE:

The tributary area of wind load applied to the column that was attached to the base plate and four bolts has a height of 30 feet and a column spacing of 20 feet or 600 sf.

The area of one 5/8-inch diameter bolt = 0.3068 in^2 . The F_v for one bolt in single shear = 3.07 k (k is kip or 1000 lbs). The allowable $F_v = 3.07 \text{ k}/0.3068 \text{ in}^2 = 10 \text{ ksi}$ (kips per sq in). (F_v = allowable shear)

If the ultimate $F_v = 0.4F_y$ and $F_v = 10 \text{ ksi}$, then $F_y = 10 \text{ ksi}/0.4 = 25 \text{ ksi}$.
For a A307 bolt, $F_t = 14 \text{ ksi} = 0.6F_y$ so $F_y = 14\text{ksi}/0.6 = 23 \text{ ksi}$; so there is good agreement about the yield strength of a A307 bolt at 25 ksi. (F_t = allowable tension)

Let's assume the failure is a brittle snapping/tension failure so each bolt fails at $(25 \text{ ksi})(0.3068 \text{ in}^2) = 7.67 \text{ k}$ and the column connection failed at $(7.67 \text{ k})(4 \text{ bolts}) = 30.68 \text{ k}$.

The total pressure required to fail the column on the 600 sf tributary area is $30.68 \text{ k}/600 \text{ sf} = 51.1 \text{ psf}$ (lbs per sf).

Back calculate the wind speed required to create this 51.1 psf pressure. The pressure is P in the wind pressure equation for the Main Wind Force Resisting System (MWFRS), which is $P = q(GC_p \pm GC_{pi})$. The internal pressure coefficient GC_{pi} is assumed to be 0.55 since the building as it is failing would seem to be partially enclosed.

As stated in the report, the external pressure coefficient GC_p could have ranged from -0.9 for the roof to +0.8 to -0.5 for the walls. If we assume mostly a push over failure so the wall pressures are more important than the roof pressure, then $P = q(0.85 * -0.5 - 0.55)$ and solving for q when $P = 51.1$, yields $q = - 52.4 \text{ psf}$.

Calculate the wind velocity using $q = 0.00256 * I * K_d * K_z * K_{zt} * V^2$. Let: $K_d = 0.85$, $K_z = 1.0$, $I = 1.0$, $K_{zt} = 1.0$ and solve for V , $V = 155 \text{ mph}$.

With the unknowns on the failure mode, let the term $(GC_p \pm GC_{pi}) = 1.0$ and solve for V , which yields **$V = 153 \text{ mph}$** . This method neglects the reduced capacity of the column/base plate connection since presumably, the roof and column are being pulled up by the wind.

CHURCH ARCHED BEAM FAILURE:

The wood laminated timber beam is 3-1/4 inches wide x 11-1/2 inches deep. The failure is parallel to the grain tension in the wood since the bolts "sliced" through the end of the beam from the bolt holes. The bolts appeared to remain attached to the steel saddles that were used to support the end of the beam.

Appendix B

Detailed Calculations of Damage Classification

From the National Design Specification for Wood, Table 5B axially loaded (tension) structural glued laminated timber has $F_t = 700\text{--}1800$ psi with most values in the range of 800–1000 psi. The allowable shear parallel to the grain $F_v = 200\text{--}250$ psi.

The area of failure considering that 1 inch of movement in the bolt hole constitutes failure is $(1\text{ inch})(3.25\text{ inches}) = 3.25\text{ in}^2$. Failure for the 2 bolts using shear failure of 200 psi = $(3.25\text{ in}^2)(2\text{ bolts})(200\text{ psi}) = 1300\text{ lbs}$.

The density of the wood beam is assumed to be 35 pcf (lb per cubic foot visually averaged from ASCE 7, Table C3-2) and the beam length was approximately 25 feet. The weight of the beam is therefore: $(11.5\text{ inches})(3.25\text{ inches})(26\text{ feet})(35\text{ pcf})/(144\text{ in}^2/\text{ft}^2) = 236\text{ lbs}$.

The beams were spaced 4 feet on center. The load on the beam saddle structure supporting the beam was 13 feet (1/2 the beam length from wall to ridge) + 3 feet (overhang) x 4 feet spacing = 64 sf. The uplift pressure to both tear the end of the beam from the bolts and overcome the weight is $(1300\text{ lbs} + 236\text{ lbs})/64\text{ sf} = 1536\text{ lbs}/64\text{ sf} = 24\text{ psf}$.

The uplift pressure would appear to be created by wind parallel to the roof ridge so if we use MWFRS coefficients for $G C_p$, $C_p = -0.9$ and $P = q(0.85 * -0.9 - 0.55) = q(-1.32)$. Solving for $q = 24\text{ psf}/-1.32 = -18\text{ psf}$.

Calculating the wind velocity from this velocity pressure using the same assumptions used in the elementary school example above: $18\text{ psf} = 0.00256 * I * K_d * K_z * K_{zt} * V^2$,
Solving for V = 91 mph.

Coefficients for Components and Cladding were reviewed but none seemed to provide results that made sense, in that the wind speed numbers were too low to create the type of damage that was observed. Also, since the failure was in a roof beam that was part of a post-beam type structure, the MWFRS concept seemed to better represent beam failure.

HOSPITAL:

The precast concrete double tees were 40 inches wide and spanned 34 feet with 3 feet of overhang on each side of the hospital. The cross sectional area was calculated to be 239.5 in^2 or 1.6632 ft^2 . The volume of the tee was $(1.6632\text{ ft}^2)(34\text{ feet} + 6\text{ feet long}) = 66.53\text{ ft}^3$. The weight of the tee assuming normal weight concrete (150 pcf) = $(66.53)(150) = 9979\text{ lbs}$.

There was a roof covering of built up asphalt with ballast on the concrete tee roof member which was estimated to weigh another 15 psf. The total unit weight was $9979\text{ lbs}/133.33\text{ sf} + 15\text{ psf} = 90\text{ psf}$.

It was judged that the roof tee better represented Components and Cladding than it did MWFRS, because the roof tee would normally expect to receive wind loading from only

one surface (i.e., the top of the roof). Because the overhangs are extensive (3 feet), the uplift on the tee was considered to be in 2 segments—the flat portion and the overhangs. The effective wind area for the overhangs is 10 sf; the effective wind area for the main roof area is 113 sf. The GCp for the overhangs is -2.8 and the GCp for the flat roof is -1.1.

The total uplift pressure of 90 psf is the sum of the uplift on the flat roof and the uplift on the overhangs. P1 is the overhang pressure and P2 is the flat roof pressure. A partially enclosed building is considered appropriate in this case because wind did get inside the building through broken windows and other building envelope penetrations.

$$P1 = q(-2.8-0.55) = -3.35q$$

$$P2 = q(-1.1-0.55) = -1.65q$$

$$90 \text{ psf} = [P1(20\text{sf}) + P2(113)]/133 \text{ sf} = [3.35q(20) + 1.65q(113)]/133$$

Solving for $q = 47 \text{ psf}$

Calculating the wind speed from the equation $q = 0.00256 * I * K_d * K_z * K_{zt} * V^2$

$$V = 147 \text{ mph}$$

JOHN DEERE BUILDING PEMB:

The failure mode studied in this building was buckling of the steel roof purlins, which are 'Z' shaped. The thickness was 1/16 inch or 0.0625 inch. The 'Z' shaped channel was 8 inches deep, 16 feet long, and had 3-inch wide flanges that formed the 'Z'. The moment of inertia about the axis that buckled (y axis) was calculated to be 0.281 in⁴ (determined by $bt^3/12 \times 2$ flanges; the thickness of the flange that constituted the y axis was so small to be negligible).

The load required to buckle the purlin is given by $P_{cr} = \pi^2 EI/L^2$ where E for steel = 29x106 psi. Using $I = .281$ and $L = 16 \text{ ft} \times 12 \text{ in/ft}$, $P_{cr} = 2181 \text{ lbs}$. The tributary area is the 16-foot length x 5-foot purlin spacing.

The distributed load required to result in a $P_{cr} = 2181 \text{ lbs}$ is $2181 \text{ lbs}/(16 \text{ feet} \times 5 \text{ feet}) = 27.3 \text{ psf}$.

Calculating the wind speed from the equation $q = 0.00256 * I * K_d * K_z * K_{zt} * V^2$ and $p = q(GCp - GCpi)$. Again the MWFRS is used to determine GCp. The gust factor G is 0.85; Cp = -0.7 is the coefficient for uplift on the roof when the slope is about 15°; GCpi is 0.55.

$p = q(-1.14)$ and $q = 27.3/1.14 = 24 \text{ psf}$. **Calculating wind speed from q yields V = 122 mph.**

If the Components and Cladding coefficients were more appropriate, in the field of the roof for an 80 sf effective wind area, GCp = -0.8, which would result in $p = q(-1.35)$ and $q = 20.2 \text{ psf}$. Wind speed required for this pressure $V = 96 \text{ mph}$.