Structural Engineers Association of Kansas and Missouri Report

"Investigations and Recommendations based on the May 22, 2011 Joplin, Missouri Tornado"

May 2012

Structural Engineers Association of Kansas and Missouri
Joplin Tornado Committee Members

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Executive Summary:

The Joplin Tornado of May 2011 was one of the most destructive natural disasters ever to hit the state of Missouri. There were more than 160 deaths, 1,100 injuries and estimates of $3 billion in damages. The physical and psychological impact will not soon be forgotten. In consideration of the magnitude of devastation to the built environment, the Structural Engineers Association of Kansas &Missouri (SEAKM), a Member Organization of NCSEA, formed a committee to investigate the performance of structures affected by the tornado, whether directly or indirectly.

In general, buildings that are designed and built in accordance with building codes, such as those published by the International Building Code Council, are not required to resist tornado force wind effects. Tornado wind speeds vary and according to some estimates, the wind speeds can be in excess of two hundred per hour, as was the case with the Joplin tornado. The International Building Code (IBC) establish a base line design wind speed of ninety miles per hour as a minimum, based on the IBC 2006, and will vary depending on the area of the country the structure is to be built. In consideration of several articles that have been published recommending that tornado prone areas should be designed to hurricane force winds [1], such as those along the coastal regions, and some building systems are more prone to collapse during a tornadic event, the Committee has compiled this report of observations and recommendations.

The Committee's observations of wood structures indicates that the main area of is maintaining a strong load path through connections, from the roof diaphragm to the foundation system. Connections in wood with nailing procedures outlined in prescriptive guidelines need to be reviewed, for both the International Residential Code and the International Building Code. Overall, the wood structures observed were of an older generation of construction materials and methods and performed poorly. The few newer commercial buildings appear to have performed better, but still had significant damage. Typically in the past, wood structures can have an inherit redundancy within the framing system, with multiple interior walls intersecting and multiple connections to the outer structural frame; but today, with larger open spaces, this redundancy is reduced significantly and the prescriptive connections techniques in codes may no longer be appropriate.

Pre-engineered Metal Buildings are typically designed and constructed to provide column free spaces for the user. The one pre-engineered building that was investigated, although not directly in the path of the tornado, incurred damage to the envelope and standing seam metal roof. The damage or failed area, appear to be consistent with over pressures that were beyond the code design wind loads. Fortunately, the main structural frame remained stable and did not collapse. These building types are a staple in society, used for churches, manufacturing facilities and various other businesses, but can be vulnerable to tornado events, exceeding the anticipated design wind speed.

Structural steel and concrete framed buildings systems performed better than others, resisting the extreme wind loading by the tornado; although, not without damage. St. John's Hospital and their Medical Office Buildings (MOB) received damage, but the structural frames remained secure. The buildings envelope material was severely damaged, with some of the destruction caused by the ballasted roof systems used throughout the complex. Essential facilities, such as the hospital, need to consider a comprehensive tornado preparedness plan when considering the layout of the facility and the respective infrastructure. Emergency generators, electrical switch gear, mechanical systems and the building structures that support them need to consider the implications of impact of wind borne debris. The generator building was impacted by several cars which caused partial collapse of the buildings wall system and roof, rendering the rendering the facility inoperable and caused la.

Hard wall structures are a building type that is constructed to be very efficient in the use of materials, while providing the most building square footage for the minimum amount of cost. These buildings are typically noted as “big box stores,” where a couple of these types of buildings encountered a near direct hit by the tornado. The high wind speeds that occurred caused significant damage, including roof deck connection failure, leading to the failure of several structural framing members, and in some cases, almost total collapse of the hard wall system.

The roof deck diaphragms of buildings have a propensity to fail first when tornado winds impose high uplift pressures onto the building structure. This is most evident in hard wall buildings, as well as, some concrete and structural steel buildings. Roof deck diaphragms are an essential building component that typically does not incorporate a redundant load path, once it fails, other structural members will most likely fail, as seen in both hard wall structures investigated.

Summary of Recommendations:

The intent of these recommendations is to increase life safety for occupants and overall building integrity and robustness when impacted by tornado type winds. However, it should be understood that a structure built in accordance with them will not be “tornado proof.”

The Committee offers the following recommendations:

1) Implement and enforce a state wide building code in all 50 states, in particular Kansas and Missouri.
2) Determine, through further research, if the use of mechanical deck connections for steel metal deck thicknesses of 22 gage or less should be mandatory.
3) Verification by designer’s that roof deck fasteners consider simultaneous uplift tension and diaphragm shear reflecting the different wind and seismic factors of safety in accordance with the Steel Deck Institute Diaphragm Design Manual - Third Edition and AISI S100.
4) Require a job specific design of open web steel joist (OWSJ) connections to primary framing members and of joist girder connections.
5) Develop code requirements for a greater robustness, or redundancy for hard wall building type systems. These may be in the form of specifying a defined base moment design; a maximum length of continuous wall prior to a full height lateral load resisting member, wall or frame; or a system of continuous cross-ties.
6) Codes should require storm shelters, or an area of refuge in retail stores, manufacturing buildings and similar types of structures with a certain number of occupants, in particular for employees that may be inside the building during a tornado event. Design could be based on the principles of International Code Council ICC-500, *ICC/NSSA Standard for the Design and Construction of Storm Shelters*, and FEMA 361, *Design and Construction Guidance for Community Safe Rooms*.
7) Codes need to require storm shelters with the design based on ICC-500, or FEMA 361, for all elementary, middle and high schools, as well as other critical facilities, such as police and fire

stations, emergency preparedness centers of control and other post disaster structures including hospitals.

8) Codes need to review and consider requiring essential buildings to use impact resistant glazing systems and door units, similar to hurricane prone region requirements for areas prone to tornados.

9) Codes should prohibit the use of ballasted roofs in all construction.

10) Research the concept of implementing similar design considerations for wind load distribution to diaphragms, drag struts and chord attachments in high-risk tornado areas that are currently codified for seismic lateral force distribution.

11) Increase inspection requirements for big box structures in tornado prone regions, similar to those imposed in the Florida Building Code for hurricane prone regions.

12) Review and update prescriptive practices for residential construction to ensure a continuous load path through connections, from roof to foundation.

13) Place a renewed emphasis on code specified special inspections, with improvements to the requirements for special inspection of wood framed buildings, including residential.

14) Encourage installation of tornado shelters for those structures that are in existence, community or individual shelters. Shelters should be in conformance with the recognized standards mentioned above.

15) Further study the impacts to design and construction practices if Codes require the design of buildings for EF-1 tornado wind speeds; equivalent to 86 -110 mile per hour, three second wind gusts for tornado prone areas, which covers approximately eighty-five percent of the rated tornados that occur in the United States.

16) The continuation of the study of tornados, to further develop appropriate code design equations to be used in building structures. The current equations consider straight line winds, which are significantly different that those winds near the vortex of the tornado, where uplift forces are considerably higher and turbulent winds occur.

It is understood that tornados are an extreme loading event, with a low probability of occurrence, but it is also evident that society is impacted by these events and as professionals we need to lead in determining if and when enhancements to the Building Codes are required. We understand that most buildings do not need to be entirely designed to the maximum wind speeds of a tornado, but we need to be prudent in our building designs and consider these events to some extent. We have the training, experience, research and tools available to implement changes.

Introduction and Purpose Statement:

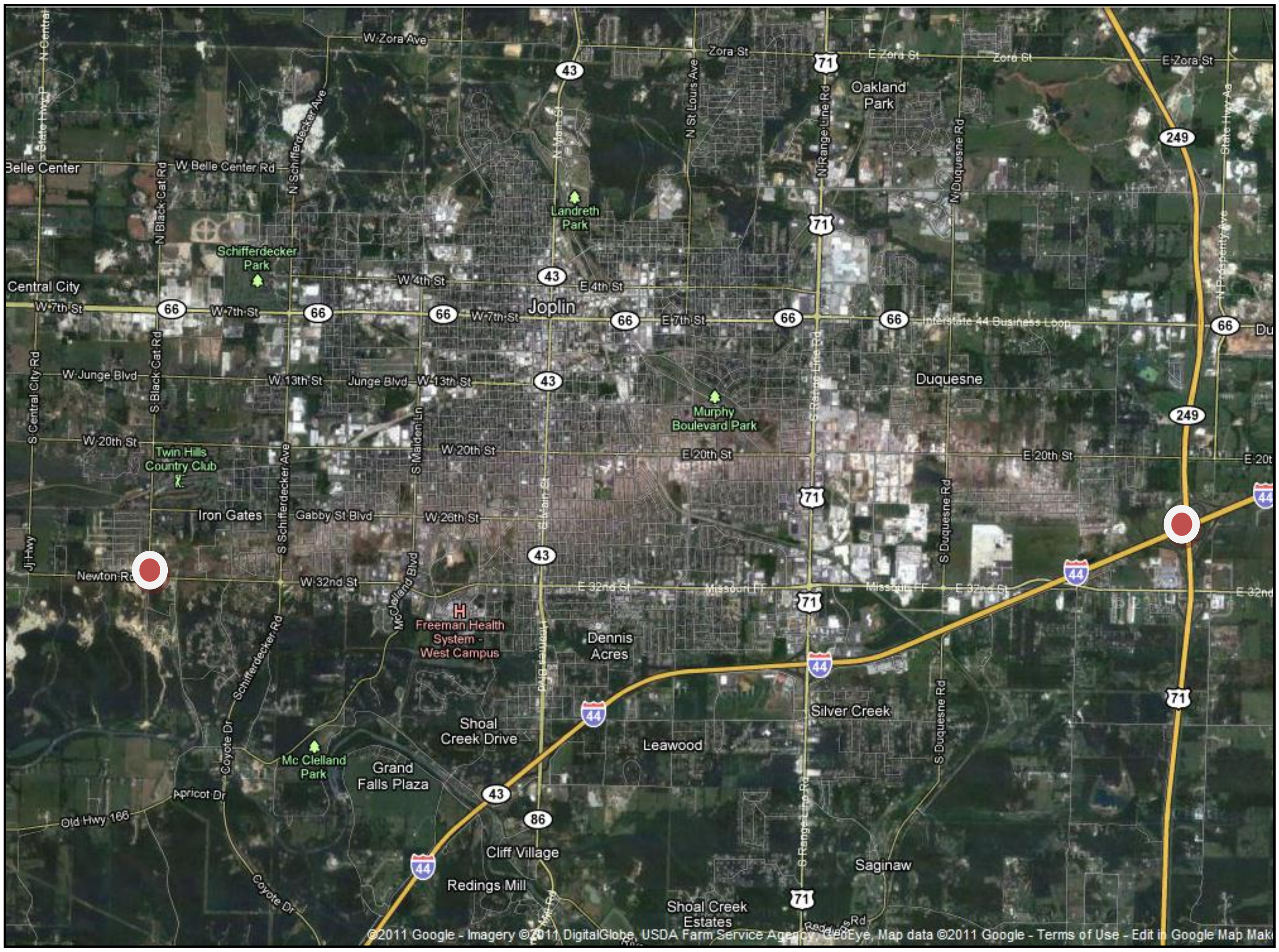
On Sunday, May 22, 2011 a series of storms in Oklahoma and Kansas converged into a category EF- 5 tornado that sliced through the central portion of the City of Joplin, Missouri. This tornado tracked from the west to east, meandering a little north then to the south, with a maximum width of approximately three-quarters of a mile, and an approximate length of 13.8 miles, of which 6 miles were within the City of Joplin [2]. Estimates from various sources indicate that there were over 8,000 structures damaged, or destroyed, 1,100 injuries, over 160 deaths, with estimated damages of nearing three billion dollars.

Figure 1: Mapping of the Joplin Tornado EF Rating

Courtesy of the United States Department of Commerce, National Weather Service, Central Region Headquarters, Kansas City, MO

The path of the tornado began near Wildwood Ranch, a residential development, near Newton Road and JJ Highway and continued to grow in intensity to a Category EF-5 Tornado near 26th Street and McClellan Drive. The area impacted contained single and multi-family residential housing, commercial buildings, light industrial and several critical infrastructure facilities, such as hospitals, fire stations and schools. It continued its path of destruction through the town of 49,000 residences, until it reached approximately US Highway 71 and Interstate 44 [2], **Figure 2**.

Figure 2: Google Map of Impacted Area of the Joplin Tornado 2011



This area of Joplin impacted by the tornado consisted of various building types, which may be categorized as follows; residential and commercial wood framed structures, residential and commercial concrete masonry wall unit structures, precast and tilt-up concrete commercial wall panel systems, pre-engineered metal buildings, and concrete and steel framed commercial buildings.

Since this disaster, several published articles were written questioning the adequacy of the current building code in the State of Missouri, the overall enforcement of the building code, and the use of certain structural systems within particular building types. The Structural Engineers of Kansas and Missouri are concerned with these statements and questions raised; therefore, in response, we formed a Committee in July of 2011 to study the Joplin Tornado with respect to the built environment and the questions posed in these articles. This report is the compilation of the SEAKM's Joplin Tornado Committee Members review of several published articles, interviews with other investigators of this disaster, including members from the American Society of Civil Engineers/ Structural Engineering Institute (ASCE/SEI) reconnaissance team, the National Institute of Standards and Technology (NIST) Joplin Report Team and personal on-site investigations and interviews with people within the City of Joplin, along with those that are involved with the rebuilding effort in the area.

Data Collection:

The Committee collected data from various sources, including discussions with the NIST team lead investigator, Dr. Marc Levitan, Miss Erin Ashley, Project Manager, FEMA MAT Team, and ASCE/SEI Damage Survey Lead Investigator Dr. David O. Prevatt, University of Florida, all of whom-investigated the damage caused by the tornado's destruction, which include Mr. Jon Shull, P.E., a SEAKM Joplin Committee Member, who lives in Webb City, Missouri, a town just north of Joplin, and responsible persons representing buildings that were severely damaged, including some of those buildings that are referenced in the various newspaper articles and our report.

Data was also collected through on-site observations of St. John's Hospital, remnants of various hard wall building structures and distant observations of other structures including schools, residential homes and infrastructure items, by four of the Committee Members.

Our report will at times rely on other reports, including, but not limited to the Tuscaloosa Tornado Report, "Damage Study and Future Direction for Structural Design Following the Tuscaloosa Tornado of 2011, available at www.davidoprevatt.com, the NIST Preliminary Joplin Report, the NOAA Joplin Tornado Report, Simpson Strong Tie investigations of the Tuscaloosa Tornado, along with various other sources noted within Appendix A at the end of this report.

Joplin Tornado Observations:

The SEAKM's Joplin Tornado Committee is providing observations and findings in this report for several building structural systems, including wood structures, pre-engineered metal buildings; steel/concrete framed buildings and hard wall structures (masonry wall, tilt-up concrete, or precast wall panel systems). These building structural systems are used throughout the United States and when constructed in accordance with a specified building code, perform in accordance with the design standard's requirements. Each of these structural systems will be reviewed independently, and observations, conclusions and recommendations will be provided.

Wood Structures:

Most wood construction buildings utilize a wood roof sheathing/deck supported by wood purlins, rafters, or trusses , designed for gravity dead, snow, live, wind and seismic. These roof members typically span to load-bearing wood stud walls. Floor construction also utilizes wood joists or trusses to span to the load-bearing walls. These walls collect the load and bear uniformly on the concrete foundation. The gravity load path can be described as: roof sheathing/deck-secondary framing-primary framing-walls-foundation.

The wind lateral force resisting system includes the same elements. Horizontal wind loads on the roof are distributed through the plywood horizontal diaphragm to the vertical shear walls. Uplift loads are carried through the gravity system. On the walls, the wind is resisted by the exterior cladding, which is also wood or gypsum sheathing that spans to the studs. The studs distribute the load to the floor or roof above and below, which consists of a plywood horizontal diaphragm on secondary framing members. The diaphragm spans to the vertical shear walls, which consist of either plywood or gypsum sheathing. These walls accumulate the lateral loads and carry them to the foundation. The lateral load path can be described as: roof deck- framing- shear walls- foundation.

Each of these structural elements is connected to its supporting member with nails, screws, staples, or bolts. If the connection is not properly designed to meet building code required loads, it will be the weak link and may result in the failure of the whole system.

The advantages of a wood structure are: low cost, readily available materials, renewable materials, ease of construction, and flexibility of on-site modifications. Wood systems can often be designed with inherit redundancy, with members that are closely spaced and there are typically multiple supporting walls. This allows adjacent members to take up additional load if one element is overstressed.

The disadvantages of a wood structure are: typically there is a lack of detail in construction documents, reliance on the knowledge of the construction worker, or builder on the requirements of proper fastening per building code requirements, possible use of under-qualified labor in the field and little or no inspection by Building Code Officials to verify that connections meet minimum design code requirements.

Wood buildings are classified as Residential or Non-residential. A residential building is specifically limited to a one- or two-family dwelling per the prescribed requirements of the International Residential Code. Residential construction is governed by the International Residential Code (IRC), published by the International Code Council. All other buildings, including multi-family residential, are governed by the International Building Code, also published by the International Code Council.

The IRC is more prescriptive in its requirements, including many tables for material sizes and connections. Most states, including Missouri, do not require that a licensed Professional Architect or licensed Professional Engineer prepare the plans for residential construction. Local jurisdictions, however, may have more stringent requirements for the involvement of licensed professionals.

Wood construction that is not residential must be engineered to meet the requirements in the IBC for loading applications, materials, and connections. A licensed Architect and licensed Engineer are usually required for the design and preparation of Construction Documents.

Inspection practices vary with the local jurisdiction. Inspectors are instructed to inspect the structural framing that is shown on the code-approved plans. This information is often quite minimal and typically does not include even the minimum nailing requirements, as this is specified in the IRC or IBC. Often the structural elements and/or connections are already covered up when the Inspector appears on site.

For non-residential buildings, the IBC recognizes that there are certain portions of the building that must be inspected in order to protect the life safety of the public. These inspections are usually not performed by city or county inspectors, but are performed by an agency that is not paid by the contractor. The contractor cannot get an occupancy permit until the Special Inspections (when required) have been satisfactorily completed.

The IBC does require Special Inspections for structural steel, concrete, and foundations in Chapter 17. However, it is not specific with regard to wood construction other than prefabricated elements, such as trusses, and high-load diaphragms. The building code relies on the Engineer of Record to specify what additional Special Inspections are required. Typical wood construction items that should require Special Inspections include: roof and wall nailing, truss or rafter tie-downs, shear wall nailing, chord construction, and hold-downs. The inclusion of these items would complete the quality control on the connected elements and the overall lateral load path.

Observations of Performance:

Most of the single family residential buildings in the center of the path of the tornado were blown away or quickly demolished. This committee could not determine the actual wind load that was applied to these structures. However, if they all had connections as described by the prescriptive nailing in the IRC, there is an assumption that each should have been able to withstand a minimum 90 mph wind load. Is this really true? A quick comparison of the minimum prescriptive connections compared with the minimum loads on a typical residential configuration appears to indicate that these nailing patterns would be adequate for only small structures with short spans.

The critical connection points and potential failure mechanisms are listed below.

1. Roof and wall sheathing removed by wind pressures. The prescribed nailing for the sheathing is based principally on the withdrawal of the nail. Losing the sheathing results in a loss of bracing for the framing members and compromises the capacity of the member.

2. Framing members were lifted off of the top plates. The prescribed truss or rafter tie down to the wall top plate is through toe nailing. The loss of the roof framing results in a loss of support for the walls; this may lead to a collapse.

3. Top plates lifted off of wall studs. The prescribed attachment is end nailing through the top plate into the end grain of the stud, which has limited capacity. The loss of the top plate results in a loss of support for the studs; this may lead to a collapse.

4. Studs lifted off of sill plates. The prescribed attachment is again end-nailing. Losing the connection at the base of the stud may have led to the collapse of the wall.

5. Sill plates lifted off of the foundation. Photographs indicate no anchorage in some cases or inadequate anchorage in others. The prescriptive attachment detail requires a one-half inch diameter anchor rods at six feet on center. Losing the anchorage at the base may have allowed a structure to be shifted from its foundation or the entire structure being lifted off its foundation leading to collapse.

6. Walls were racked, especially the walls adjacent to the garages. The amount of wall available for a shear wall is often inadequate to resist the resulting lateral load, which results in a failure of the boundary nailing and racking of the wall.

7. Many of the non-residential wood buildings performed better. Although we do not have access to the construction documents to verify each case, there is some evidence that the connections were more complete. Truss tie-down clips were noted, to attach the trusses to the top plate. In non-residential buildings, an engineer is required to calculate the uplift load on a truss in order to specify the correct tie-down anchor. A design which has this properly specified greatly enhances the capacity of the building to resist the loads.

Photo 1: Note the anchorage of the trusses to the sidewalls in this building: the top chord of the truss broke before the anchorage failed**.**

Photo 2: Destruction of the main roof system, second story, but the redundancy of the wood structure prevents total collapse.

Photo 3: Corner house destroyed, while adjacent homes remain intact.

Photo 4: A non-residential wood frame structure near the center of the tornado.

Photo 5: Non-residential building remains standing despite major damage.

Photo 6: Residential neighborhood in the center of the tornado**.**

Conclusions:

Residential:

1. The prescribed minimum code connections are not uniformly utilized and in most cased proved to be inadequate to resist wind loads imposed by the tornado. The failure of even one attachment can lead to the progressive failure of other elements.

2. Stronger positive connections through verification of nailing procedures or the installation of light gage metal connectors would increase the capacity of wood construction to meet high winds and low level tornados better.

3. Architects, Engineers, Building Officials and Builders need to follow and enforce Codes that are adopted and understand the consequences of their design, inspection and construction methods.

4. Code Committees need to review the requirements for connectivity of the diaphragms, shear walls and sill plates, to accommodate the larger spans of roofs and floors, along with overall size of homes that are being built today. Load paths must be followed and connections designed for their anticipated loads.

5. Storm shelters, or refuge areas need to be considered for any building type, with consideration of tornado activity. ICC 500 or FEMA 320 can be used for developing a storm shelter for residential construction.

Non-residential

1. Many of the non-residential wood buildings performed better. Although we do not have access to the construction documents to verify each case, there is some evidence that the connections may have been more robust than in residential buildings.

2. Truss tie-down clips used to attach the trusses to the top plate performed much better.

3. Engineered systems should provide calculations that include considerations for code uplift loads on a truss and specifies the correct tie-down anchor; the wood structure has the capacity to resist the minimum code loads.

4. Storm shelters or refuge areas need to be considered for any building type, with consideration of tornado activity. ICC 500 or FEMA 361 can be used for developing a storm shelter for a given occupancy, based on occupancy category.

The Committee refers the reader to FEMA 423 and ASCE's Joplin Tornado Report [3] for further information regarding wood structures.

Pre-engineered Metal Building Structures:

Pre-Engineered Metal Buildings (PEMB's) are steel structures that are designed and fabricated by a single manufacturing entity and sold through contractors to building owners or other contractors. The building manufacturer supplies the primary framing, secondary framing, horizontal and vertical bracing, open-web steel joists or purlins, roof and wall cladding, and all required fasteners and sealants. On occasion, the walls and/or roof of the PEMB may be of more conventional material. Should that be the case, those materials are not designed by the PEMB manufacturer. The PEMB manufacturer designs the superstructure only to the loads specified on their order form and provides reactions applied to the foundation system, the design of which is performed by others, using loads provided by the PEMB manufacturer.

A typical PEMB consists of rigid frames in the transverse directions, which carry both vertical and lateral loads. Stability in the lengthwise direction is generally provided by tension-only rod or cable bracing. The end frames in the building may be either rigid frames or simpler post-and-beam construction. Secondary framing usually consists of cold-formed, light-gage purlins and girts which span between the rigid frames. These sections are overlapped at the frames to provide continuity. The roof of the PEMB is usually roll-formed, pre-painted metal, either in a ribbed configuration secured with through fasteners or more commonly, a standing-seam metal roof (SSMR) secured with clips concealed in the joints between the panels. Wall cladding usually is a cold-formed, pre-painted steel panel, although many options are available.

Saint Paul's United Method Church

Observations were made at St. Paul's, a campus-like complex which consists of multiple PEMB's, some of which sustained substantial damage. The complex is located at 2423 West 26th Street, in an area that is at the periphery of the significant damage zone. The sanctuary was located just north of the centerline track of the tornado's path. Slightly to the east of the site the storm turned on a more northeasterly track. Due the location of the building complex, the damage appears to have been caused more by straight-line winds as opposed to the tornado vortex. Due to the directional change in the storm, high winds likely struck the buildings first from the northwest to southeast.

Observations of Performance:

1. The tall stud walls at the east and west ends of the sanctuary failed under direct wind load. The studs and top track were pulled clear of the supporting rigid frame and collapsed inward. There are some questions regarding the actual connections of the metal studs to the track, but actual drawings were not available to verify the required attachment and compare to the actual conditions.

2. Diagonal kicker braces from the wall studs to the purlins caused local buckling in the cold formed steel Z-section purlins. Some purlins were twisted and rolled over by the uplift forces.

3. The standing seam metal roof was blown upward and away from the eaves and east end wall of the structure. There is no evidence of clip failure; instead it appears that the applied batten strip at the panel edges failed, allowing the panel to pull away from the clips.

4. The primary frames appear to have remained intact and undamaged.

5. The building directly to the north of the large sanctuary area is oriented with the ridge running north-south. A lean-to structure is attached to the east and west sides of the main building. At the north wall, there is a gabled structure which forms a continuous roof line with the lean-to, effectively surrounding the higher main building. Winds from the east damaged the cladding of the structure mainly through impact of wind-borne debris. Winds from the north caused the roof of the high bay portion to peel back from the north end wall. These winds also heavily damaged the lower roof as well as the north walls of the higher and lower building. Heavy damage also occurred at the northeast corner of the building.

6. Purlins in the end bay of the high section of this building were rolled over due to high wind forces.

7. A two-story building is located to the east of the main complex, connected by a corridor building. This structure appeared to suffer damage to the roof and walls mainly through impact of wind-borne debris, which dented the metal roof and wall panels. Some side laps of the wall panels were opened up by the wind forces. It was noted that the majority of the windows in this building remained intact.

8. Immediately to the south of the main sanctuary is an independent steeple structure, which is framed with a braced tube structure. This was clad with composite panels on the walls and an SSMR that matched that of the sanctuary. The cladding of the steeple sustained heavy damage or was completely blown off. There appeared to be no damage to the primary structure of the steeple.

Photo 7: Saint Paul's United Method Church Complex constructed of multi-pre-engineered metal buildings.

Photo 8: Cold Formed Metal Stud Failure due to poor attachment to the underside of the purlin overhang.

Photo 9: Roof Damage to north slope of sanctuary.

Photo 10: Purlin damage at the east end of the sanctuary building, along with failure of the exterior wall framing.

Photo 11: Damage to northeast corner of building.

Photo 12: The free standing church steeple exhibiting
the loss of cladding, while the main structure system remains intact.

Conclusions:

1. The damage to the structures indicates that straight-line winds caused the majority of the damage which was consistent with code prescribed loading patterns.

2. The large wall at the east side of the sanctuary was not supplied by PEMB manufacturer and may or may not have been competently engineered. Connection failure at the top of this wall appears to have led to its failure. Even though the studs may have been properly sized, it appears that insufficient consideration was given to the upper connection details.

3. The "non-structural" standing-seam roof panels performed poorly with respect to the actual wind pressures of the tornado. The large flat expanse between the vertical ribs allow for large deformations leading to the panels being pried out of the retaining batten strip. The roof became unattached at that point. These roofs are typically made of 24 gage material and lack cross ribs which would stiffen the panel under uplift. This type of panel is more normally applied over solid substrate.

4. The combination of rigid frames and bracing formed a load-carrying system that is independent of the cladding, preventing total collapse of the structure under high wind forces.

Concrete and Steel Frame Buildings:

Concrete and steel frame buildings are engineered structures that are generally constructed with a column grid and may be single or multi-story. The vertical force resisting system supporting gravity dead, snow, live, wind and seismic loads, consists of a roof and/or floor deck(s) supported by a secondary framing system of roof purlins or joists which are supported by a primary framing system of beams and girders, then columns and finally the foundation. The load path can be described as: roof deck- secondary framing-primary framing-columns- foundation.

The lateral force resisting system consists of the following: envelope framing (cold formed steel studs, or aluminum frames for curtain wall systems), the lateral load is received by exterior cladding that distributes the load horizontally to the vertically oriented members. These members span vertically between the foundation, floors and roof diaphragms. The floor or roof diaphragm spans horizontally to vertical frames consisting of moment frames or braced frames. The moment frames consist of rigidly connected beams and columns then transfer the horizontal force to the foundation. Braced frames typically consist of beams and columns with diagonal bracing that transfer the force to the foundation.

Missouri law requires that these types of buildings to be engineered and designed by licensed Professional Engineers competent in structural building design and erected by licensed contractors. It is understood that for a structure to perform appropriately, it has to be constructed as designed.

For concrete frame buildings the floors and roof are typically reinforced slabs supported by reinforced concrete joists, beams and/or girders. The floor and roof framing is supported by columns that have vertical steel reinforcing bars confined by steel reinforcing ties or spirals. These columns maybe rectangular or circular shaped and are typically constructed using cast-in-place concrete methods. The lateral force resisting systems for these building systems are typically moment frames consisting of rigid connections between the columns and girders or beams. Reinforced concrete shear walls are also sometimes used to resist lateral forces.

Steel frame buildings consist of structural steel hot rolled or welded plate beams and columns. Roofs may be concrete slabs or steel ribbed sheets (metal deck) that are supported vertically by open web steel joists or rolled steel beams. The steel joists or beams transfer the vertical load to larger beams or girders and then to columns and eventually to the foundation. Steel framing may resist lateral loads by the use of moment frames or braced frames. Moment frames are similar to the concrete moment frames, in the manner that the joints are rigidly connected. Braced frames are typically constructed with diagonal bracing in various configurations, consisting of steel angles, channels, hollow steel section, or beams. Both systems take the lateral loads from the story above and transfer them to the respective foundation. Floors systems for these types of buildings are typically reinforced concrete that is cast into a composite or non-composite metal deck and are designed to interact with the steel beams and girders supporting the floor.

The roof and floor diaphragms are an important component of the lateral force resisting system. The diaphragm may be rigid, such as concrete, or somewhat flexible, such as steel ribbed sheet roof deck.

All of the connections of the frame buildings are important, from the welded, powder actuated fastening, or screwed roof deck to supporting members, to the bolted or welded connections of beams and columns and anchorage of the columns to the foundation.

In an engineered structure, each component, including the cladding and all connections are designed to meet code-required forces.

Observations of Performance:

1. The St. John's Medical Center consists of several multi-story buildings: some reinforced concrete moment frame buildings and some structural steel braced or moment frames. The actual hospital building is made up of two major structures. The original hospital building is a reinforced concrete frame building with waffle-slab floors. The original building was designed in 1965-1966 with construction beginning in 1967. As such, it would have been designed to either the 1961 BOCA National Building Code or the 1965 BOCA National Building Code. A major expansion of the hospital building proper occurred in 1983 with the addition of a steel braced frame building. Numerous other medical office buildings are also part of the hospital campus. These buildings would have been designed to various editions of the BOCA National Building Code or the International Building Code.

   The design wind speed for structures designed in the time frame of the hospital campus would have been 70 miles per hour as a "fastest-mile" wind. This velocity equates to an 84 miles per hour 3-second gust. The design wind speed in the current edition building code is based on a 90 miles per hour 3-second gust.

2. The roof of the main hospital building consists of lightweight insulating concrete applied over a cast-in-place roof slab on a concrete composite metal roof deck. A ballasted waterproofing membrane sits on top of the insulating concrete slab. The adjacent Medical Office Buildings (MOB) did not have the insulating concrete on the roof. Some of these buildings were provided with rigid insulation adhered directly to the metal deck and covered with the roofing membrane and ballast.

3. The exterior cladding for the structural steel framed buildings consists of EIFS on light gage metal stud attached to horizontal structural steel framing. As can be seen in **Photo 13**, the tornado damage includes broken glazing and impact damage to the EIFS. Once the winds penetrated the glazing, additional damage occurred to interior finishes and partitions as shown in **Photo 14**.

4. The membrane or ballasted membrane roofing material was significantly damaged with the winds pushing the ballast into furrows. The roofs of the main hospital buildings proper did not fail structurally, most likely due to the additional dead load of the light weight concrete over metal deck and the respective attachment methods that were used, but the roofs on the MOB's experienced significant damage where areas of the metal decking were completely missing.

5. The reinforced concrete frame hospital building was similarly damaged regarding glazing and roof membrane. The inset curtain wall/glazing system remained largely in place. The steel or concrete framed mechanical penthouses located on the roof of the buildings suffered the worst damage with completely or partially stripped wall and roof cladding as shown in **Photos 15 and 16**.

   In all cases the main engineered structural steel or concrete framing system of the hospital proper was intact and appeared undamaged.

   It was noted that the roof ballast system, containing two inch rock, became projectiles during the tornado and damaged much of the exterior facade and penetrated most of the respective glazing systems of the buildings around the complex. Piles of rock from the roof ballast was noted in several offices located on the west side of the buildings, along with conversations indicated that several halls were filled as well.

6. The mechanical and electrical buildings of the hospital were severely damaged by the tornado and the debris from the tornado impacting the walls. The investigating team arrived after the clean-up of the area, but did note that for the twelve inch concrete masonry unit (CMU) walls surrounding the generator room were removed because of significant damage. The wall appeared to be reinforced with one number four bar at forty-eight inches on center and fully grouted below grade. The mechanical room area constructed in a similar manner with CMU walls and a veneer was reinforced very sporadically along the length of the wall. The actual spacing of the reinforcement was not determined. Drawings of the area were not available to investigate further, reference **Photos 17 and 18.**

Photo 13: Concrete-frame building at left, steel frame building beyond.

Photo 14: St. John's Medical Hospital damage, note the ballasted roof, light weight concrete on roof and curtain wall glazing system destruction.

Photo 15: St. John's Medical Center, damage to penthouses and roof equipment. Note the missing metal roof deck on penthouse, looking NW.

Photo 16: St. John's Medical Center, penthouse at concrete frame building with mechanical penthouse constructed of structural steel and light weight concrete on metal roof deck.

Photo 17: St. John's Medical Center – mechanical and back-up generator buildings destroyed by the tornado. Area cleaned prior to site visit.

Photo 18: St. John's Medical Center – The mechanical and back-up generator masonry wall buildings appear to lack sufficient reinforcing bars. Note: the area had been cleaned up prior to our investigation.

7. As noted previously, St. John's Medical Center is a campus, containing not only the hospital building proper, but also contains outpatient facilities and medical office buildings. These structures were presumably designed with a normal importance factor, as opposed to the hospital buildings proper which would have been assigned with a higher importance factor.

   The framing systems for these other buildings are generally steel braced or moment frames. Wall cladding is typically a curtain wall system or insulated metal panels with concealed fasteners. Many of the roofs for these structures consist of a ballasted roof membrane over rigid insulation board supported by light-gage steel decking. Secondary framing supporting the decking is either steel wide-flange sections or open-web steel bar joists.

8. Most of the other structures on the St. John's campus experienced failures in both the wall and roof cladding, resulting in serious interior damage. The basic load-carrying structural system, with the exception of the roof diaphragm, remained intact and serviceable after the storm.

   The wall cladding performed poorly on these buildings, although the general construction of them was similar to that of the main hospital buildings. Whether their location to the south of the main buildings played a role in the severity of the damage is not known. Examples of wall cladding damage are shown in **Photos 19 and 20**.

9. Unlike the main hospital buildings, the roofs of the other buildings performed poorly, with most buildings losing some, but not all of their roof decking. In all of the cases observed, the decking had been attached to the supporting members by an arc spot welds (puddle welds). These connections appeared to have failed, allowing the decking to be removed by the high winds. These failures were generally in the deck metal adjacent to the welds. Examples of these failures are shown in **Photos 21 through 23**.

10. Various reports issued by FEMA, NIST and ASCE categorize the winds at St. John's Medical Center as EF-3, with indicated winds between 136 and 165 miles per hour. Based on the actual observed damage and the Degree of Damage Indicators published in "A Recommendation for an Enhanced Fujita Scale" by the Wind Science and Engineering Center at Texas Tech University, the damage is consistent with winds between 131 and 152 miles per hour (mph).

Photo 19: St. John's Medical Center - Steel Frame Building - damaged glazing and exterior cladding

Photo 20: St. John's Medical Center – Steel Frame MOB - damaged glazing and cladding

Photo 21: St. John's Medical Center MOB - Steel Frame Building with missing and damaged ballasted metal roof decking**.**

Photo **22:** St. John's Medical Center MOB - Steel Frame Building with missing and damaged ballasted metal roof decking.

Photo 23: St. John’s Medical Center –Failed metal roof deck to structure connection.

Photo 24: St. John's Medical Center MOB - Steel Frame Buildings with missing and damaged ballasted metal roof decking.

The Committee refers the reader to the NIST and SEI/ASCE reports regarding investigations of other concrete and steel framed buildings, including the Joplin High School.

Conclusions:

1. Maintaining the integrity of the building envelope is very important to sustaining reliability in the building's structural frame and providing life safety to the occupants. Curtain wall framing systems designed for impact resistance adds to the overall construction costs of the building, but may prove necessary for more essential buildings, such as hospitals, emergency response facilities and other facilities classified as Building Occupancy Category IV.

2. Steel structures that rely strictly on a metal deck to provide diaphragm action at the roof level should be reviewed by the governing code agencies and design entities to consider the development of a more robust design methodology for these connections to the structural steel. The failure of these connections appear to significantly contribute to the instability of the overall structure, leading to possible proportional progressive collapse, not only in steel frame buildings, but in other building types, such as hard wall structures.

3. Ballasted roof systems should be reviewed for use in tornado prone areas, in particular for essential building types. These ballasted roof systems can cause significant damage to not only the building's envelope, but injury to the occupants inside the building once the envelope is breached.

4. Concrete structures which consider the use of concrete roof and floor diaphragms, offer a very robust framing system for buildings. These types of buildings appear to have a significant redundancy in the sense of their use of moment frames in all directions. Designs considering the use of a total concrete system appear to be more capable of resisting unforeseen loading conditions.

5. Essential facilities, such as St. John's Hospital, experienced failures of their emergency back-up equipment, such as power generators, switch gear and mechanical systems. These systems are critical to the building's function as an essential facility, and as designers, we need to think more critically when placing them. At a minimum, the structures that house these functions, whether a separate building, or a penthouse, need to be designed to the same criteria, importance and occupancy category as the building that is relying on its systems to function during and after a tornado event.

6. Storm shelters or areas of refuge need to be considered for any building type, with consideration of tornado activity. ICC 500 or FEMA 361 can be used for developing a storm shelter for a given occupancy, based on occupancy category and may be a basis of design for areas of refuge.

Hard Wall Structures (Masonry, Tilt-up Concrete, and Precast Concrete Walls):

A hard wall building may be defined generally as a one-story building with walls composed of masonry or concrete and with a horizontal flexible roof diaphragm. The walls are most often composed of Concrete Masonry Units (CMU) masonry, concrete tilt-up or precast panels, but also may include reinforced brick or cast-in-place concrete. The flexible roof diaphragm is usually composed of light gage metal deck (without a concrete topping) or plywood sheathing. The metal deck or plywood sheathing is usually supported by open web steel joists and joist girders or fabricated trusses, but may be supported with structural steel or wood beams and girders. The primary interior framing system is typically designed to carry only vertical loads and contributes little to the overall lateral stability of the structure. The ends of the primary framing beams or joist girders are sometimes supported by columns, but are more often supported directly by the hard wall itself. Secondary roof framing members are also usually supported at the ends of the building by the hard wall system.

The walls themselves perform multiple functions. They are the main wall cladding element of the building and are part of the Main Wind Force Resisting System (MWFRS). The walls collect forces from the roof diaphragm and transfer them to the foundation system, enabling the building to resist wind and other lateral loads, such as earthquakes. The lateral load systems in these types of buildings require that both the roof and walls be able to function as intended, as there is little redundancy in the overall lateral system, due to their efficient use of materials.

These types of structures are widely used for buildings that require a durable, efficient use of materials and architecturally pleasing wall surface. Typically, this building type is used in “big-box” store buildings, but is also used for warehouses, combinations of flexible office and warehouse where flexibility use is optimized. Many cities across the nation have these types of buildings constructed for commercial enterprises; such as: Kohl’s, Target, Walmart, Home Depot, and Lowes.

During the Joplin tornado event many of these hard wall buildings lost their roof diaphragm. Failures of the roof diaphragm and framing connections to the wall were also observed. The hard walls were then subject to instability as they no longer had lateral support at the roof level, resulting in partial or complete collapse of the wall.

Photo 25: This is one example of a hard wall building system struck by the Joplin Tornado. It is noted that some walls remained standing due to a true cantilever condition of the tilt-up wall panels being support at their base and attached approximately four feet above, at a loading dock slab.

Observations of Performance:

1. The degree of damage depended in large part on the location of the building with respect to the storm track. Structures closer to the center of the track experienced more damage than those towards the edge of the track. Damage along the track was also highly variable.

2. Metal deck detached from the open web steel joist top chord. Arc spot welds 5/8" diameter at 6" on center detached over the full length of the joists (22 gauge deck). The deck was found blown upward and clear of the building footprint while the joists remained in the building.

3. Metal deck and joists uplifted and were blown clear of the building together. The bearing seat anchorage welds at the ends of the steel joists were torn free of the support at the wall. Anchor rods embedded into the top of walls connecting the joists failed as the joists uplifted.

4. There was no significant evidence that the hard walls became detached from the roof first, collapsed, and then allowed wind to enter the building and blow the roof away. Some of the collapsed hard walls were observed to have fallen inward into the building while others to the outside. Eyewitness accounts stated that the roof began to fail and then "disappeared" before the wall panels fell.

5. Steel embed plates with short studs pulled out of the top of the wall. It is uncertain if the failure was caused by wind or the resulting collapse and impact of the wall collapsing to the ground.

6. Bond beams at the top of CMU masonry walls separated from the top of the wall. The vertical rebar extending from the foundation and on up vertically through the wall only engaged 6 inches into the bond beam at roof and pulled free. It was observed that the roof structure was still connected to the bond beam but separated from the remainder of the wall, which had collapsed.

7. Some steel joist girders were observed to have a significant upward bow, as the joists and metal deck appeared to have pulled it upward prior to detaching.

8. Dynamic wind effects appeared prevalent. Examples included the apparent lack of dead load holding down the roofing and roof structure as well as light poles failing significantly above their base plates.

9. Architectural elements became a structural concern as their attachments failed. Examples included roof top HVAC units, entry facades/structures, canopies, awnings, downspouts and other items attached to hard wall buildings.

10. Welded connections of the open-web steel joist bearing seats to the joist girders failed, at one collapsed structure. These connections were typical 1"x 1/8" fillet welds, two per joist end bearing seat.

11. Although uplift appeared to be significant on the roof elements, no distress was observed in the following: joist girder to top of steel column connection, steel column base plates and uplift to footings.

12. No distress was observed in the shear walls due to in-plane shear forces or in plane overturning. No distress due to out-of plane bending in walls was observed (prior to collapse caused by instability of the wall).

Photo 26: Roof level bond beam detached from top of CMU wall.

Photo 27: Embed plate with short studs pulled free from top of CMU wall.

Photo 28: Metal deck detached from roof joists. Deck blown clear from building footprint and resting in parking lot.

Photo 29: Failure of metal deck connection to joist top chord.

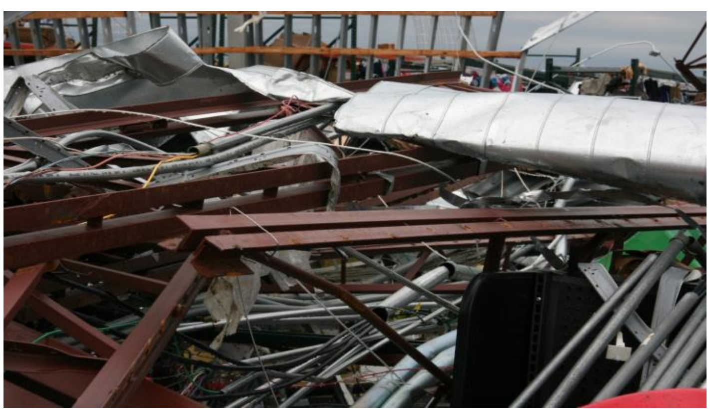


Photo 30: Arc spot welds on top of steel joist where metal deck pulled free. Failed weld at joist bearing with distorted clip angle. Joist remained inside the building footprint.

Photo 31: Both metal deck and steel joist blown clear from building footprint.

Photo 32: Upward bow of joist girder due to overstress loading of uplift pressures during tornado.

Photo 33: Upward bow of joist girder due overstress loading of uplift pressures during tornado.

Photo 34: Architectural elements and mechanical units dislodged during high winds.

Photo 35: Architectural elements and cold formed steel stud framing damage.

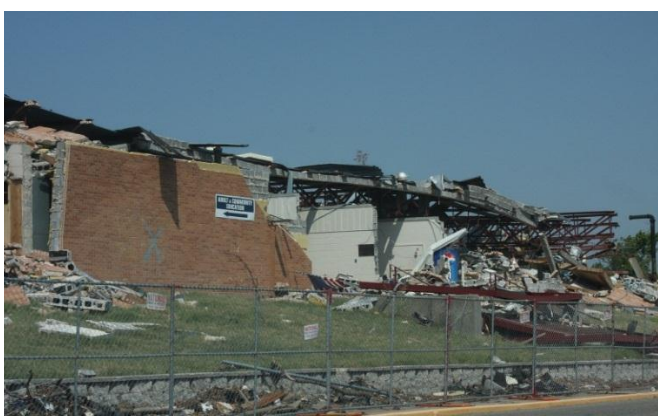


Photo 36: Hard wall collapse to the outside of building, with second level roof collapse.

Photo 37: 18 gage metal deck attached to joist, after demolition of building struck by the tornado.

Conclusions:

1. The load path resisting wind uplift appears to be a more significant concern than wind induced in-plane shear at shear walls, and out of plane bending in walls.

2. Metal deck to joist welds appeared inadequate for the uplift forces (22 gauge deck) that were encountered. A thicker gauge deck would help strengthen this limit state. It was reported that a structure with an 18 gage deck performed significantly better than neighboring 22 gauge roof deck structures.

3. The use of “typical” joist to joist girder welds appeared to be inadequate for the loads incurred during this event. Connections of joists to joist-girders or beams should be designed for all induced forces, including but not limited to the calculated uplift force, diaphragm chord and collector forces.

4. Roof to wall connections, and joist to wall connections appeared inadequate for uplift. Stronger connections, welds, stiffeners, and anchor bolts are encouraged. Use of long rebar welded to embed plates would likely perform better than short headed studs. A hook added to the top of vertical reinforcing bars in CMU walls would help prevent detachment of bond beam from top of the wall.

5. Engineers should use good judgment when calculating dead loads for use in resisting net uplift. Often the dead load is calculated as heavy, which is conservative for downward load combinations. However the lighter extreme should be considered when calculating net uplift. The actual dead load installed may be significantly less than initially considered. Furthermore dynamic wind effects could temporarily negate the downward contribution of dead load in resisting uplift. In fact, it may not be unreasonable to completely neglect dead load for a roof when calculating net uplift forces.

6. Building Codes should consider requirements for a more robust continuous cross ties across the building diaphragm, so as to preserve walls when the roof diaphragm fails. Wind force levels could be EF-0 or EF-1 and allowable stresses could be ultimate, factor of safety equal to 1.0 and allow for significant damage, but minimize the propensity for collapse of the hard wall system.

7. Attention to detail is appropriate for elements attached to the hard wall structures. If an architectural element dislodges and causes harm, the structural engineer may be accused of negligence.

8. Storm shelters or refuge areas need to be considered for any building type, with consideration of tornado activity. ICC 500 or FEMA 361 can be used for developing a storm shelter for a given occupancy, based on occupancy category and may be the basis of design for an area of refuge.

Tornado Effects on the Built Environment:

Tornados are one of nature’s most elusive adversaries to the built environment. Although with the use of modern technology, we are able to predict the potential of a tornado and possible path; we still struggle to actually record wind speeds and develop definite scientific design methodologies for the structural design of buildings. The design principles outlined in Federal Emergency Management Agency for storm shelters (FEMA 320 and FEMA 361) and the International Code Council ICC/NSSA Standard for the Design and Construction of Storm Shelters (ICC 500-2008) are based on wind effects and speeds that differ from actual tornado wind events; even the rating of tornado's (EF-X) are based on observed damage and comparing these observations to straight line wind damage of other types of wind events.

The current rating system used for tornados is based on the Enhanced Fujita (EF) Scale, which was developed by Texas Tech University (TTU) in cooperation with the National Weather Service (NWS) in 2004, http://www.spc.noaa.gov/efscale/ef-ttu.pdf. TTU held a forum with forty experts in wind engineering, meteorologists, architects and practicing engineers, with the concept of improving the previously used Fujita Scale, developed by Dr. T. Theodore Fujita of Chicago University in 1971, to better describe the observed damage, or "Degree of Damage" (DOD) after a tornado occurrence. Today, this EF system is the basis of all tornado wind event discussions. The Committee requests that the reader references the document link above for further information regarding the EF Scale rating system and the DOD determination. An in depth discussion of the document is beyond the scope of this report, but is relevant to the understanding of some of the discussions within.

Since the Joplin Disaster, some written articles have suggested that the adoption of building code amendments, or incorporation of requirements, such as those that are implemented in hurricane prone areas, may serve the public better than the current codes. **Table 1,** presented below, provides information regarding the current hurricane category system, the Fujita Scale and the Enhanced Fujita Scale for comparisons.

Table 1:

HURRICANE WIND SPEEDS		FUJITA SCALE			DERIVED EF SCALE		OPERATIONAL EF SCALE	
Category Classification	Peak 1-minute wind speed at 33 feet AGS (Sustained Winds) mph	F Number	Fastest 1/4-mile (mph)	3 Second Gust (mph)	EF Number	3 Second Gust (mph)	EF Number	3 Second Gust (mph)
		0	40-72	45-78	0	65-85	0	65-85
Category 1	74-95	1	73-112	79-117	1	86-109	1	86-110
Category 2	96-110	2	113-157	118-161	2	110-137	2	111-135
Category 3	111-130	3	158-207	162-209	3	138-167	3	136-165
Category 4	131-155	4	208-260	210-261	4	168-199	4	166-200
Category 5	155 +	5	261-318	262-317	5	200-234	5	Over
		ENHANCED FUJITA SCALE DEVELOPED BY TEXAS TECH UNIVERSITY IN 2004						

***** IMPORTANT NOTE ABOUT ENHANCED F-SCALE WINDS: The Enhanced F-scale still is a set of wind estimates (not measurements) based on damage. Its uses three-second gusts estimated at the point of damage based on a judgment of 8 levels of damage to the 28 indicators. These estimates vary with height and exposure.

Important: The 3 second gust is not the same wind as in standard surface observations. Standard measurements are taken by weather stations in open exposures, using a directly measured, "one minute mile" speed.

Information gathered from the NOAA website http://www.spc.noaa.gov/faq/tornado/ef-scale.htm

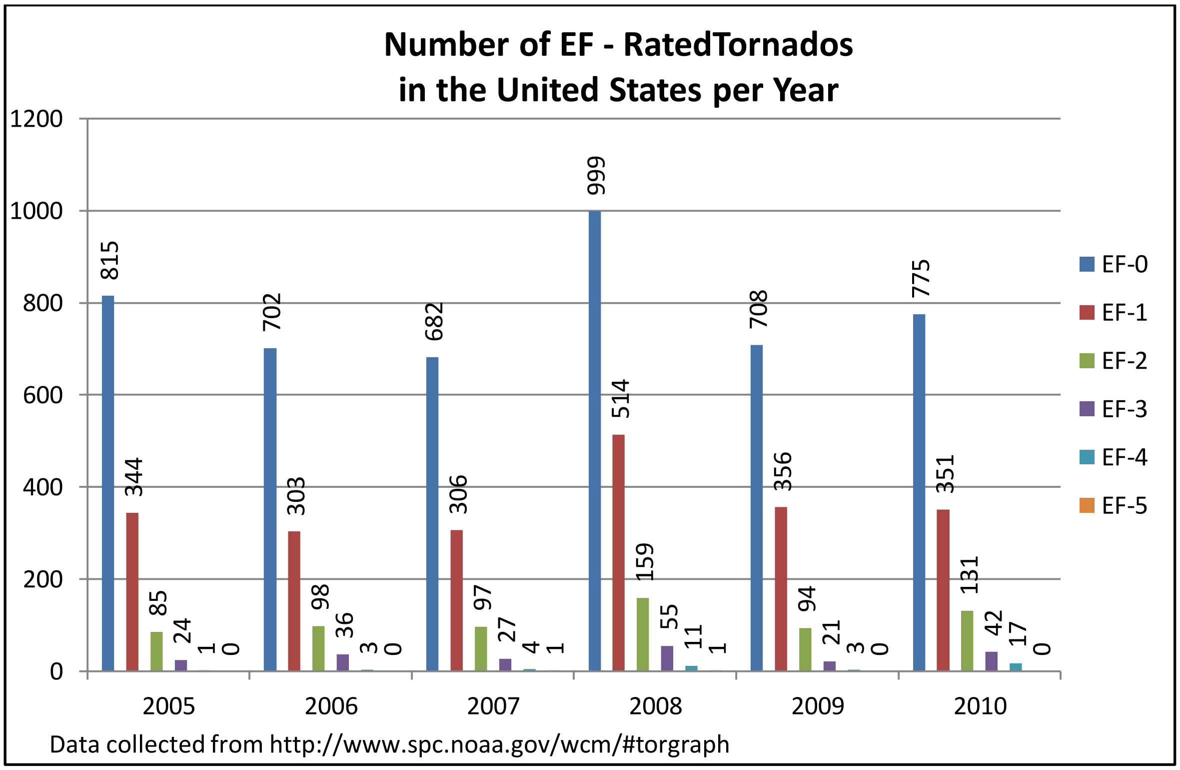
This chart indicates a fairly significant difference in wind speeds between hurricanes and the tornado EF scales, along with a different measurement system, peak one minute sustained wind and three second gust, respectively. It is noted that the physical conditions of the actual wind speeds and turbulences of a tornado event is very different from those of straight line winds. Although the

American Society of Civil Engineers document ASCE-7 provides correlations in their commentary regarding the two wind events, this report will provide information regarding tornados based on the current Enhanced Fujita (EF) system and the observed damage wind speeds.

We believe that the EF system, although not perfect, better represents the equations that practicing engineers use in their calculations because it is based on observations of damage that are correlated to a three-second gust speed at the standard measurement height of 33 feet (10 meters). It is now noted that the wind speeds indicated in the EF system do not necessarily correlate to an equivalent straight line wind speed due to the turbulent nature of the event, with high vertical uplifts, substantially higher suction, or negative pressures and radial acceleration winds around the vortex of the tornado.

There is an average of 1,550 tornados a year that have occurred in the United States in the past five years, with approximately 1,370 of these in the EF-0 and EF-1 Category. Simply stated, eighty-eight percent of the tornados are evaluated to have a 3-second gust wind speed rating of 110 mph or below, sixty-eight percent of these tornados are evaluated to have a maximum wind speed of 85 mph.

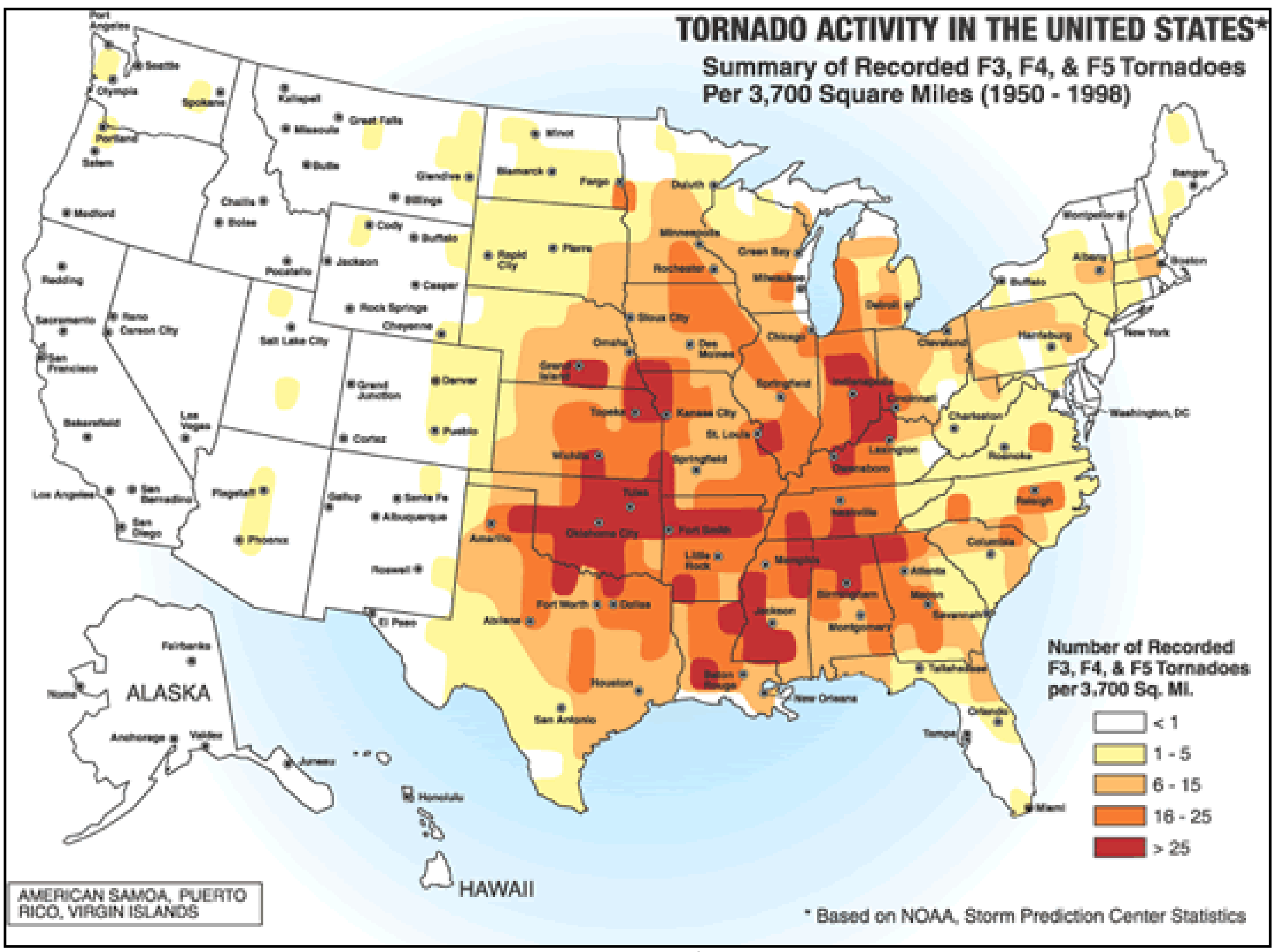
Figure 2: Tornado Events occurring in the United States 2005-2010



The geographical mapping of tornados for the year of 2010 indicates that tornados may occur anywhere in the United States, but there is a defined concentration of occurrences in the mid-section of the country, reference **Figure 4**, http://www.tornadohistoryproject.com/custom/818009/map. Historically, this assertion holds true as shown in **Figure 5**, where the darker colors indicate a higher intensity tornado occurrence. Please note that this map is based on the Fujita Scale system, but the relevance of the data is still valid, considering the concentration of higher EF rated tornados and the areas we are addressing in this report, the States of Kansas and Missouri.

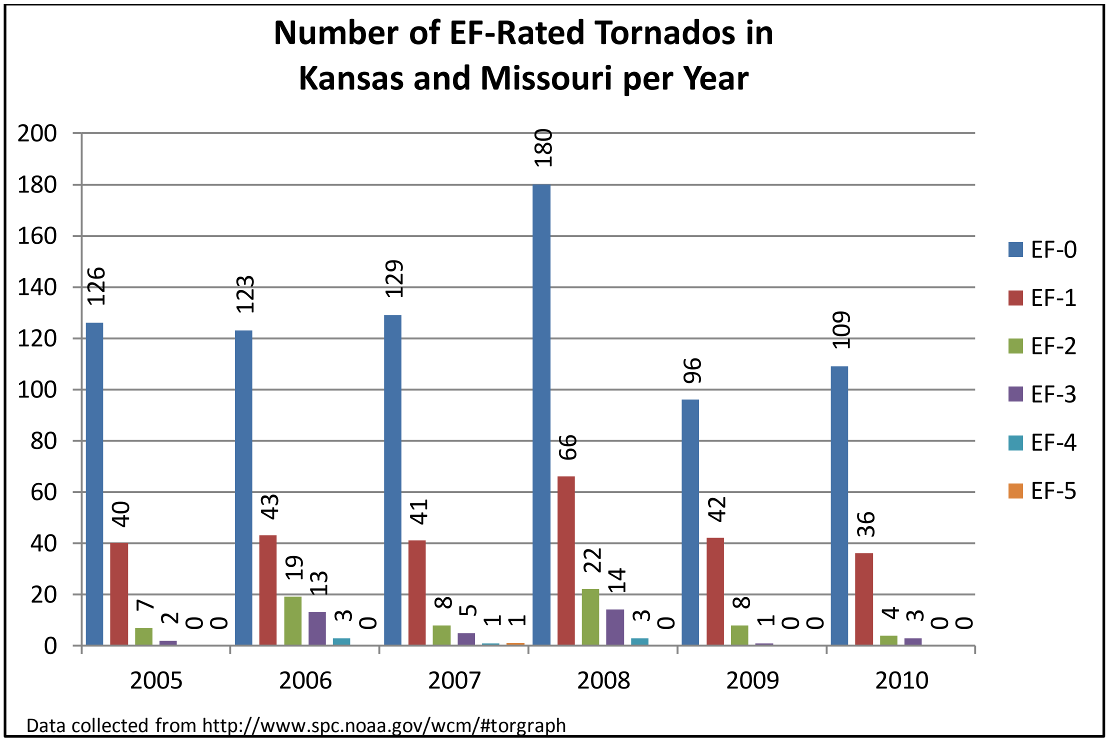
Figure 4: Mapping of the 2010 Tornados in the United States

Figure 5: Tornado Activity in the United States – Category F3, F4 and F5



Kansas and Missouri encounter approximately 190 tornados per year, with 172 of these occurring with an evaluated wind speed rating of 110 mph or below, or approximately ninety percent. Seventy-four percent of these tornados are evaluated to have a maximum wind speed of 85 mph, reference **Figure 6.**

Figure 6: Number of Tornados in Kansas and Missouri 2005-2010



The wind speeds assigned to a Category EF-0 tornado is rated with speed variation of 65-85 miles per hour, which is less than the prescribed minimum wind speed velocity of 90 mph per the International Building Code 2006. Also, this wind speed designation is based on a 3-second wind gust, at 33 feet above ground with an Exposure of “C”, similar to the basis used for the determination of the wind speed for the EF scale system; therefore, buildings constructed under the IBC 2006 are theoretically designed to resist wind speeds an EF-0 tornado, if properly designed and constructed in accordance to the code. One may pose a question, should the building codes consider a minimum of an EF-1 rated tornado with wind speeds of 86 to 110 miles per hour? The probability of a tornado occurring in this range is certainly higher than one rated at an EF-2, and this may help in mitigation of outlying damaged caused by higher rated tornados, but this design decision will increase the initial construction costs of the building.

The 2006 International Building Code references the American Society of Civil Engineers (ASCE) document ASCE7-05 for wind load calculations. The simplified approach will be discussed within this report for ease of explanation and illustration of the effects of wind velocity and the applied wind pressures to buildings. ASCE provides the following equation for the determination of velocity pressure in pounds per square foot.

$$qz = 0.002556 * Kz * Kzt * Kd * V^2 * I$$

We will assume the following; Exposure Category "C", Kz equals 1.0 which reflects an evaluation height of 30 feet,, Kzt equals 1.0, Kd equals 0.85, velocity "V" will vary based on the assumed maximum wind velocity on the given rated tornado and the importance factor (I) will be set to 1.0. **Table 4** presents the velocity pressure based on the tornado rating and the percent increase in velocity pressure for the basis of design.

Table 4: EF Rated Tornado Wind Speed Effects on Design Velocity Pressure

EF Rated Tornado	3 Second Gust (mph)	Design Velocity (mph)	Velocity Pressure qz (psf)	Percent Increase
0	65-85	90*	17.63	0
1	86-110	110	26.33	49%
2	111-135	135	39.66	125%
3	136-165	165	59.24	236%
4	166-200	200	87.04	394%
5	Over	250	136.00	672%

*Based on Minimum Code Wind Speed

The table above indicates an increase in the base design velocity pressure, approximately forty nine percent from the EF-0 to EF-1 and a significant increase of one hundred and twenty-five percent for an EF-2 tornado. The increase of design wind velocity to an EF-2 tornado will impact several considerations during the design of the structure along with impacting the construction costs. Of course, we realize that hurricane prone areas, such as those along the coast, do include measures to accommodate similar higher wind speeds, such as impact resistant glazing, special testing for envelope materials, special roofing materials and of course a more robust lateral force resisting systems in the building, but the probability of a hurricane impacting the structure in the indicated higher wind zones, is much greater than tornados of the same strength occurring in the States of Kansas and Missouri for a given area.

It should be noted that ASCE 7-10, adopted for use in IBC 2012, significantly changes many aspects of wind load determination by specifying a uniform Mean Recurrence Interval throughout the country and through the use of "ultimate" wind design wind speeds. Additionally, structures with occupancy categories other than II will use different design wind speeds rather than use an importance factor. Comparing actual factored design pressures shows minimal changes in the actual design pressures from previous codes, thus the conclusions above are still valid.

An alternative to increasing the design wind speed for an entire building is to provide storm shelters within the given building type; for example, schools may easily accommodate the requirements of the International Code Council ICC 500, ICC/NSSA Standard for the Design and Construction of Storm Shelters, by modifying planning, siting, corridors and restrooms to accommodate higher design wind speeds. The wind speed map provided in the document escalates the design wind pressures to those that approximate the wind speed rating of an EF-5 tornado for the States of Kansas and Missouri, reference **Figure 7**.

Figure 7: Shelter Design Wind Speeds for Tornadoes [8]

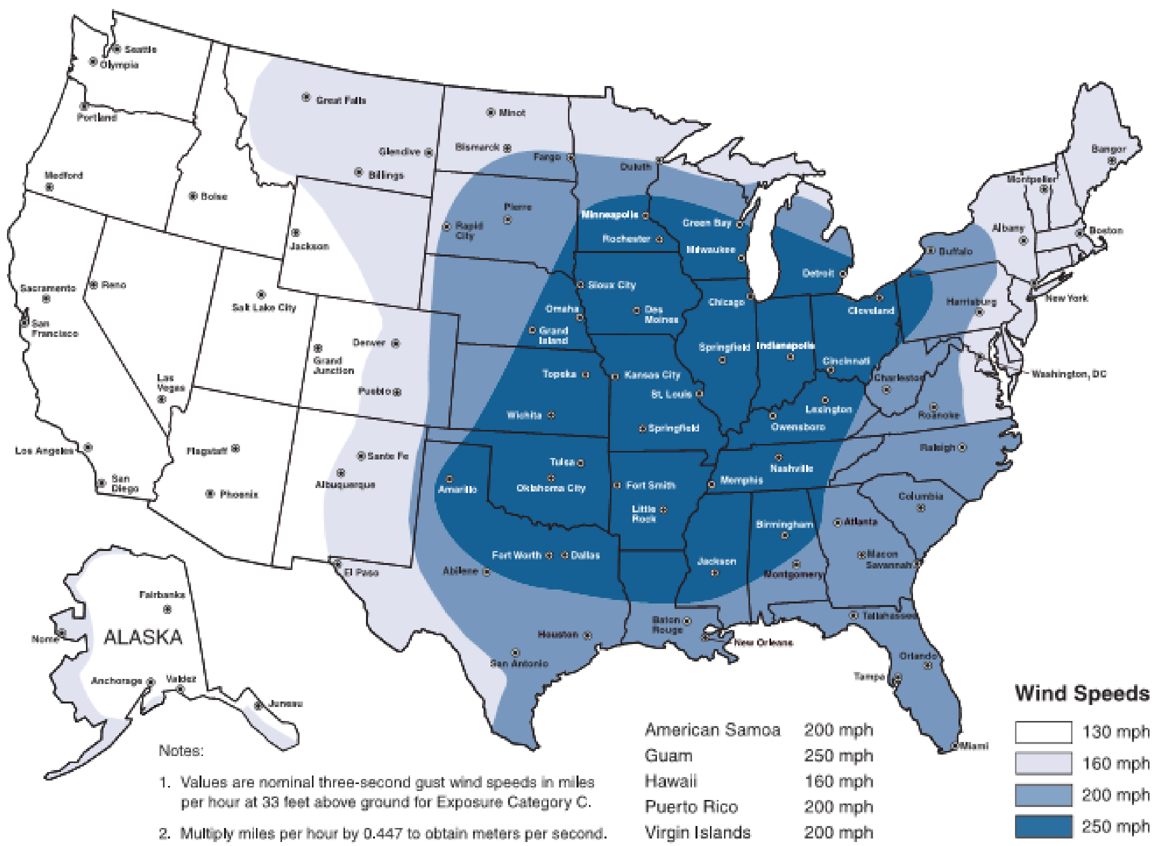


Figure 3-1. Tornado Safe Room Design Wind Speed Map (consistent with ICC-500 Tornado Hazard Map)

The ICC 500-2008 does allow a reduced factorization of this wind load, for example the wind load combinations for strength design may be replaced in the following manner: combination 3, replace 0.8 times calculated wind pressures to 0.5 times wind pressures and for load combinations 4 and 6, replace 1.6 times wind pressures to 1.0 times wind pressures. There are other considerations that must be implemented into to the design documents, such as requirements for internal over pressure, ventilation, restroom facilities, roof live loads, impact resistant glazing, defining occupant load, amongst various other details beyond the scope of this report. The ICC 500 provides further information regarding the design requirements for a given shelter and this report request that the reader obtains this document for further information. It is noted that the International Building Code of 2009 is referencing the ICC 500-2008 for storm shelter design criteria.

The Current State of Building Codes and Enforcement:

Professional Engineers are required to base their designs of buildings on an accepted building code, whether this acceptance is regulated by a local jurisdiction, or an assigned State Wide Building Code. Currently, these codes may vary from municipality to municipality and from state to state, which may, or may not include additional local jurisdiction amendments. Some rural areas may not have an accepted building code, leaving the Engineer of Record in cooperation with the building's owner, to determine the design requirements for the project at hand. Fortunately, the City of Joplin has adopted the International Building Code and International Residential Code of 2006 as their governing building codes. Joplin has enacted a series of updates to their Building Code program, dating back to 1961, as shown in **Table 5**. These codes listed in the table have evolved over the years and have incorporated various wind design criteria that has developed throughout the years based on scientific research, advancements in technology and acquiring additional knowledge from wind events such as the Joplin Tornado. This report will reference the International Building Code (IBC) and the International Residential Code (IRC) of 2006 for our discussions, conclusions and recommendations, unless noted otherwise.

Table 5: City of Joplin Adopted Codes [2]

Code Adopted	Adopted Date	Importance Factor Considerations*
2006 IBC/IRC	2008	Yes
2000 IBC/IRC	2003	Yes
1996 BOCA/NBC	1997	Yes
1990 BOCA/NBC	1990	Yes
1984 BOCA/NBC	1984	No
1978 BOCA/NBC	1980	No
1970 BOCA/NBC	1970	No
1965 BOCA/NBC	1966	No
1961 BOCA/NBC	1961	No

* Increase Factor of design loads based on Occupancy of Building

Today, both the States of Kansas and Missouri do not have an accepted State Wide Building Code, leaving the local municipalities to determine the applicable building code for their area and they may not readily enforce the requirements of special inspections for the construction, as outlined in codes. In the fall of 2011, Missouri Representative Tim Meadows of District 101, along with Missouri Representative Stacey Newman of District 073, sponsored and co-sponsored, respectively, House Bill HB 938, http://www.house.mo.gov/billsummary.aspx?bill=HB938&year=2011&code=R. This bill introduces legislation that would require municipalities to enact a building code equal to the State's accepted Building Code, within one revision cycle, or one that is more stringent. This bill is currently tabled, but may be brought up again in the next session. We encourage the States of Kansas and Missouri to adopt a State Wide Building Code and provide the funding necessary to enforce the legislation and support the local jurisdictions for monitoring the enforcement. It is important that a State Wide Building Code be enforced for several reasons.

First, when the State Wide Building Code (SWBC) is established, the citizens of the state will have consistent commercial and residential designs along with monitored constructed buildings throughout the state and not just in local metropolitan areas that have an accepted building code. Also, the enforcement of the SWBC allows the Professional Engineer to design in accordance to the latest Code information available. The Missouri bill provides funding for the requirements of the local municipalities to maintain records regarding the review of the construction documents and inspection during the construction of the buildings. The bill also allows the review of the construction, which can be performed by the local government, or third party inspectors.

Secondly, a federal bill, HR 2069; Safe Building Code Incentive Act of 2011, introduced by Representative Mario Diaz-Balart of Florida, Albiao Sires of New Jersey and Richard Hanna of New York, as an Amendment to the Robert T. Stafford Disaster Relief and Emergency Assistance Act, increases the responsibility of the States to enact a State Wide Building Code, while providing monetary incentives http://www.govtrack.us/congress/bill.xpd?bill=h112-2069. This bill advocates the States to enact a State Wide Building Code with considerations of enhancing the Federal Emergency Management Agency response funding increases. The Building Strong Coalition, a membership of various insurance companies, The American Institute of Architects, the International Code Council, Simpson-Strong Tie, along with various other groups, are supporting this bill, realizing that the acceptance and enforcement of building codes save lives and money [5].

Another reason for accepting State Wide Building Codes is that studies have shown that enforcing building codes help protect homes and buildings from “hurricanes, tornadoes, earthquakes, floods, fire and other natural disasters,” and for every dollar spent a reduction in losses of $4 can be achieved, in accordance to a FEMA sponsored study [6]. There are a few states that qualify for the incentive offered under the proposed HR 2069; some states need a few modifications, while others do not have any accepted State Wide Building Codes and will not meet the requirements for the monetary incentive of the federal bill, reference **Table 6**.

Table 6: State Wide Building Code Acceptance in Accordance to HR 2069 [7]

States Qualifying for Incentive Under the Proposed Stafford Act Enhancements	States That Could Qualify with Minor Legislative Modifications	States that have Adopted Statewide Codes, but Lack Enforcement	States that have not Accepted any Statewide Code
California	Connecticut	Arkansas	Alaska
District of Columbia	Delaware	Georgia	Alabama
Florida	Indiana	Kentucky	Arizona
Louisiana	Maryland	Ohio	Colorado
Michigan	Massachusetts	Tennessee	Idaho
Minnesota	North Carolina	West Virginia	Illinois
New Hampshire	Oregon		Iowa
New Jersey	Rhode Island		Kansas
New York	Wisconsin		Maine
New Mexico			Maryland
Pennsylvania			Mississippi
South Carolina			Missouri
Utah			Montana
Virginia			Nebraska
Washington			Nevada
			North Dakota
			Oklahoma
			South Dakota
			Texas
			Vermont
			Wyoming

Also, we notice in **Table 7**, that ten of the top fifteen states in the country that have the highest tornado losses, do not have State Wide Building Codes.

Table 7: Tornado Costs Ranking of States [6]

Rank	State	State Wide Building Code
# 1	**Texas**	**No**
# 2	Indiana	Yes
# 3	**Kansas**	**No**
# 4	Georgia	Yes
# 5	**Oklahoma**	**No**
# 6	Minnesota	Yes
# 7	Ohio	Yes
# 8	**Illinois**	**No**
# 9	**Missouri**	**No**
# 10	**Iowa**	**No**
# 11	**Nebraska**	**No**
# 12	Massachusetts	Yes
# 13	Pennsylvania	Yes
# 14	**Alabama**	**No**
# 15	Louisiana	Yes
Based on Tornado Costs for the Years : 1950-1994		

We certainly encourage all states to consider enforcing a State Wide Building Code, in an effort to minimize losses, both in regards to human causalities and in monetary considerations.

Recommendations for Further Research and Analysis:

The performance of structures during the Joplin tornado varied widely and depended on numerous factors. These factors included:

- Age of the structure.
- Location of the structure with regard to topography.
- Location of the structure with regard to the tornado path.
- Type of structure.
- Design traits (Was the structure a fully engineered structured, a pre-engineered or site-adapted structure or built to a prescriptive code?)
- Construction quality.

The intention of these recommendations is to increase the life safety of the occupants and reduce structural damage by improving the overall integrity and robustness of building when impacted by tornado type winds. It should be understood that these recommendations do not guarantee that a structure built in accordance with them will create a “tornado proof” building.

1) **State Wide Building Code acceptance in all 50 states, in particular Kansas and Missouri.**

We have the opportunity to enhance our built environment by passing this legislation, both federally and by local state governments. There is legislation already proposed in the State of Missouri, voters need to support it by contacting their local representatives. This legislation will enable local jurisdictions to enforce the State Wide Building Code and includes funding for this enforcement. Studies prove that building to a code provides a safer built environment for all. This legislation will not only provide for improved buildings, but better protection for the inhabitants at a lower cost, when considering insurance premiums and long term costs for society in these catastrophic events – tornados.

2) **Further research to determine if the use of mechanical deck connections for steel metal deck thicknesses of 22 gage or less should be mandatory.**

The roof diaphragm is essential to the overall integrity of a building's structural system. If the roof diaphragms remained substantially intact in the two "big box" structures reviewed for this report, the collapse of one and the partial collapse of the other may have been prevented. Inspections by several groups have revealed failures of the decking metal around supposedly sound spot arc (puddle) welds. It is not certain if the failure was due to poor welding, or failure of the light gage material due to the extreme wind uplift pressures. Today, steel deck manufacturers and their governing bodies do not recommend welding of side laps for 22 gage decks. It seems apparent that there is a need for further research for typical fastening of the deck to the supporting structure, whether by mechanical fastening with or without washers, or welding with washers. Self-drilling or self-tapping screws with heavy heads and integral washers or suitable powder actuated fasteners may provide a significant assurance in the design capacity required for the elements, in lieu of reliance on visual inspection of the spot arc welds.

3) **Verification by the designer that the roof deck fasteners consider simultaneous uplift tension and diaphragm shear and must reflect the different wind and seismic factors of safety in accordance with the Steel Deck Institute Diaphragm Design Manual- Third Edition**.

Steel Deck Manufacturer's data typically does not consider these simultaneously and sometimes provide notes regarding the different factors of safety for wind and seismic design values. Diaphragm capacity tables ignore the uplift component of roof deck loading, designers must understand this point and calculate the required capacity for the system. The lateral loading imposed on the roof diaphragm experiences both shear force due to lateral loads and an uplift force due to wind, or seismic over the entire roof. Additionally, inspection of damaged structures indicate that uplift in the field of the roof may have been much higher than traditional loading patterns currently indicates. This is due to the large atmospheric pressure drop in the vortex of the tornado. There is little or no available research into the wind patterns on a structure during a tornado, but designing the fastening of the diaphragm system for an amplified wind pressure load capacity in both shear and uplift seems appropriate.

4) **Designers must require a job specific design of open web steel joist (OWSJ) connections to primary framing members and of joist girder connections.**

In many instances, the joist to joist-girder and joist-girder connection to the building columns use a standard detail approach provided by the joist supplier. These details need to reflect the design practice of forcing any failure into the member itself rather than in the connection. Connection failures are often sudden and catastrophic, whereas member failures are more ductile and may not result in a catastrophic failure. designers, joist manufacturers, or the Engineer of Record, should design the connection based on the strength of the most critical component of the joist or joist-girder assembly such as top chord shear or end diagonal compressive capacity.

5) **Develop Code requirements for a more robust, or redundant system for hard wall building type systems. These considerations may be in the form of requiring a defined base moment design for these walls, a maximum length of continuous wall prior to a full height lateral resisting member, wall or frame, or a more robust system of continuous cross-ties.**

One of the buildings impacted by the Joplin tornado, experienced a near total collapse of the tilt-up wall panel system, all except those at the loading dock area where the base of the panel was below grade. These panels acted as a cantilever wall, being attached below grade at the foundation and at the slab-on-grade floor inside the building. Typical details, used in these types of buildings, place dowel bars from the tilt-up panel into the floor slab resulting in a fixed or partially fixed base condition. Alternatively, if a lateral bracing element, such as a perpendicular wall or steel brace, is placed to brace the wall system at some prescribed length, the potential of the wall system failing is greatly reduced.

Building Codes should also consider requirements for a more robust continuous cross ties across the building diaphragm, so as to preserve walls when the roof diaphragm fails. Wind force levels could be EF-0 or EF-1 and allowable stresses could be ultimate, factor of safety equal to 1.0 and allow for significant damage, but minimize the probability of collapse of the hard wall system.

6) **Codes should require a storm shelter, or at a minimum, an area of refuge in retail stores, manufacturing buildings and similar types of buildings with a certain level of quantitative occupancy level, in particular for employees that may be in the building during a possible tornado event.**

According to published accounts, lives were saved in the Wal-Mart, a hard wall type of building structure, because store employees and patrons were able to shelter in an employee break room. Although not specifically designed as a shelter, the inherit robustness and redundancy in the framing of the room provided sufficient protection for the building occupants that took refuge there. This area of refuge could be designed based on the ICC-500 document, which offers insight to sizing, ventilation requirements and a method of calculation of forces to design the structure. As stated in the ICC-500 document, the size may be specified based on the projected occupant load.

7) **Codes need to require storm shelters, design based on ICC-500, or FEMA 361, for all elementary, middle and high schools. Other critical facilities, such as police and fire stations, emergency preparedness centers of control and other post disaster structures including hospitals and emergency facilities must require these shelters also.**

   Society relies on the public school system to protect their children during their time being educated and we expect our critical facilities and infrastructure to with stand extreme loadings. The City of Joplin's Tornado provides sufficient evidence that schools need to consider alternative measures for offering security during these times of violent weather. It is very fortunate that at the time of the tornado, the schools were empty. It is unfortunate that some of the critical facilities were unusable after the event.

8) **Codes need to review and consider requiring essential buildings, Category IV Occupancy, to use impact resistant glazing systems and door units, similar to hurricane prone region requirements for areas prone to tornados.**

   The winds of the Joplin tornado caused significant damage to envelope materials of several buildings of the St. John's Hospital complex, which included the curtain wall framing system, cold formed metal stud framing and door systems and rendered the entire complex unusable. The hospital facilities may have been able to treat some of the injured, had these items not catastrophically failed. Critical structures should be using the same practices required in the "wind-borne debris" region of hurricanes to protect lives.

9) **Codes should prohibit the *use of ballasted roofs with rock or crushed stone in all construction for tornado prone areas.***

   During high wind tornado events, loose roof ballast (gravel) is proven to be ineffective at preventing roof blow-off. This is noted during both hurricanes and tornadoes. Roof ballast often becomes airborne debris which typically destroys glazing systems and other brittle exterior finishes. This debris can injure innocent people. As noted above, during the inspection of the roofs at St. John's Medical Center, the ballast of the roof was shifted into piles, was blown into the glass, broke the facade and landed in many of the rooms on the west side of the building, ineffectively holding the roof membrane in place, while potentially overloading other structural portions of the roof. Many hurricane prone regions of the country have enforced codes restricting the use of them, based on the same determinations as mentioned above.

10) **Research the concept of implementing similar design considerations for wind load distribution to diaphragms, drag struts and chord attachments in high-risk tornado areas that are currently codified for seismic lateral force distribution.**

   Enhanced design requirements for diaphragms, drag struts and chord development will lead to more robust connections of the diaphragm to the bearing walls and to other lateral-force-resisting system elements. This research should consider all aspects. It is noted that seismic design considerations, per code regarding diaphragm design, connections and chord development, are enhanced to due to uncertainty in actual loads lead to more robust connections.

11) **Code enhancement of increase inspection requirements for large box structures, similar to those imposed in the Florida Building Code.**

Hard wall box structures have little redundancy and ultimately depend on the connection strength of the roof diaphragm to its supporting members. These connections should be designated as critical connections and be inspected prior to permitting occupancy of the structure. Codes may be updated to require inspection requirements similar to that in the Florida Building Code in which structures over a certain size, requires a resident engineer to insure compliance with the design and code requirements (“Threshold Inspection”).

12) **Codes need to review and update prescriptive practices for residential construction to ensure a continuous load path through connections, from roof to foundation**.

Although most of the damaged residential buildings in Joplin were built under older or no building code, the noted destruction of some of the newer homes still causes questions regarding the means and methods of current prescriptive practices. Increasing the structural robustness of newly constructed houses seems a logical recommendation. Many houses are damaged or destroyed by straight-line winds on the periphery of the tornado track or by lower intensity tornadoes. Structural reinforcement and proper load paths would mitigate the amount of damage and potentially reduce casualties due to structural failure and airborne debris. While this strengthening would undoubtedly raise the cost of housing, insurers may reduce premiums on structures constructed in compliance with the enhanced standards. This reduction could reduce the ultimate insurance cost premium to the homeowner.

13) **Place a renewed emphasis on code specified Special Inspections, with improvements to the requirements for special inspection of wood framed buildings, including residential**.

As design and construction professionals, we are encouraged to review our past practices and determine ways to enhance our means and methods to better serve the public. In light of many disasters, there is a renewed effort in inspections and requirements of design and construction. The Hyatt collapse precipitated efforts to develop an inspection manual by the city of Kansas City, Missouri, reference article in Structure Magazine, July 2011. Several cities around the country have developed their own special inspection manuals that typically are more stringent than the accepted local building code, in particular for wood structures.

14) **Encourage construction of tornado shelters for those structures that are in existence, community or individual shelters. Shelters should be in conformance with the recognized standards mentioned herein.**

There are several available pre-manufactured storm shelters that meet guidelines set by organizations. These structures, for residential construction, typically follow FEMA 320 guidelines. The guidelines offer simple methods along with economical approaches to designing and constructing residential structures. This document along with others regarding tornado preparedness are available at www.fema.gov.

15) **Further study the impacts to design and construction practices if Codes require the design of buildings for EF-1 tornado wind speeds; equivalent to 86 -110 mile per hour, three second wind gusts for tornado prone areas, which covers approximately eighty-five percent of the rated tornados that occur in the United States**.

    It seems appropriate to consider the design of structures for a higher level of wind pressures, based on the current observed wind speeds through the Enhanced Fujita Scale Rating System. Although, we must realize that the wind speeds are inferred and not actual determined wind speeds. Again, the analysis and the system are not perfect, but it is the best analysis that we currently have to implement the wind speed and pressures for the design of structures today.

16) **The continuation of the study of tornados, to further develop appropriate code design equations to be used in building structures. The current equations consider straight line winds, which are significantly different that those winds near the vortex of the tornado, where uplift forces are considerably higher and turbulent winds occur**.

    As stated above, the basis of the design equations for wind pressures are straight line winds at thirty-three feet above ground surface, which is not the same type of winds that occur during a tornado.

We understand that most of these items will take time to discuss and come to a consensus amongst governing bodies on which recommendations to be implemented, but this is an opportunity for design professionals, the construction industry, along with legislative branches of government and the general public to learn from these devastating events and react.

We encourage the reader to continue their research by reading other Joplin reports that are, or will be soon issued by The National Institute for Standards and Technology, Federal Emergency Management Administration, the National Oceanic and Atmospheric Administration, along with reports regarding the Tuscaloosa, Alabama Tornado Reports, and Enhanced Fujita (EF) Scale, which was developed by Texas Tech University (TTU) in cooperation with the National Weather Service (NWS) in 2004, http://www.spc.noaa.gov/efscale/ef-ttu.pdf.

Disclaimer

This document is created from observations, research and discussions with other people reporting on the Joplin Tornado incident. Opinions and views expressed by the authors in this report are theirs and do not represent the view of any funding entity. All information in this report is gathered by the authors and believed to be factually correct, but readers are encouraged to research other documents noted within the report and form their own opinion of the information provided in this report.

Gratitude

The Committee thanks all of those that offered their assistance in developing this report and especially those that granted access to secured sites, drawings and eyewitness accounts of the disaster, without their assistance this document would not be possible.

Appendix of References

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