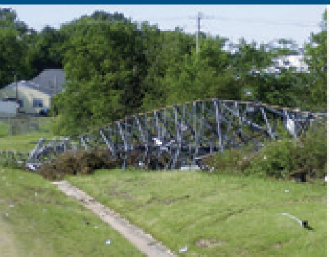
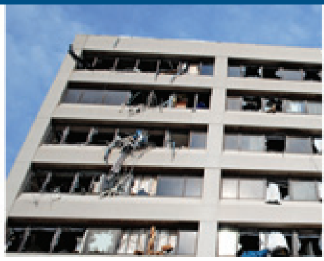

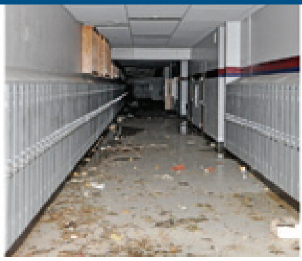
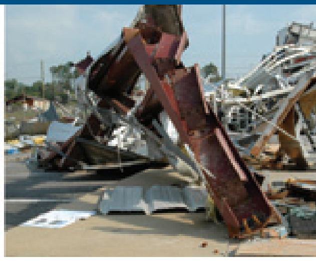

TORNADO OUTBREAK *of* 2011

IN ALABAMA, GEORGIA, MISSISSIPPI, TENNESSEE, AND MISSOURI

5 Observations on Commercial and Industrial Building Performance

The MAT visited numerous tornado-damaged commercial and industrial buildings to assess building performance and reasons for failures.

This chapter describes the results of the MAT's observation of commercial and industrial buildings damaged during the April 25–28, 2011 tornadoes in the mid-south of the United States and the May 22, 2011 tornado that struck Joplin, MO. It provides a general description of the damage observed across the impacted area and provides seven case studies with detailed damage descriptions. This chapter evaluates commercial and industrial building structural designs and the effects of various design decisions and construction techniques on a building's resistance to tornado damage. The MAT's observations focused on the Main Wind Force Resisting System (MWFRS) of the observed building, with special attention on continuous load paths and structural connections. Although failures may have propagated from secondary building elements or the building envelope, it was the failure of the MWFRS or portions of the critical load paths that resulted in loss of significant sections of building or partial to full building collapses, in several instances causing loss of life.

Summary of Primary Failure Modes Observed by the MAT

The major structural failures observed by the MAT were caused directly by extreme wind loads that exceeded the design strength of the building structural systems. Many of the failures observed by the MAT were likely a combination of the MWFRS being overloaded by secondary building elements or by insufficient load path connections of the MWFRS. The term failure is used in this chapter to mean a structural material or building structural system that was loaded beyond its resistance capacity. In this context, failure does not imply a design failure occurred; it means that the building or component was challenged by a force larger than it was capable of resisting.

The larger commercial buildings observed by the MAT were designed to function as enclosed buildings. Portions of the building shells were designed to act as both the envelope and the MWFRS that transfers loads into the foundation in lieu of internal bracing. Therefore, when damage to the roof and walls occurred, damage to the MWFRS also occurred. When the building envelope of this type of building is breached, the resulting pressurization effectively changes the enclosed building into a partially enclosed building (refer to Section 3.1 for additional information). Once the building is effectively a partially enclosed building, the key structural components experience significantly higher wind loading than they were designed to resist. The MAT observed buildings that were damaged at wind speeds lower than the design wind speed because of increased pressurization.

Role of Existing Building Code

It is important to note that current building code wind speeds do not represent the influence from tornados. ASCE 7-10 does not provide requirements for minimum design loads specific to all tornadic events, but does address tornadoes in the Commentary. Section C26.5.4 *Limitation* (p. 513) as follows:

> "It is recognized that tornadic wind speeds have a significantly lower probability of occurrence than the basic wind speeds. In addition, it is found that in approximately one-half of the recorded tornadoes, gust speeds are less than the gust speeds associated with basic wind speeds."

Thus while the forces from tornadoes of lesser intensities, such as those rated EF0 and EF1, fall within the design parameters of wind speeds represented in the current ASCE 7 standard, the forces from very strong tornadoes (EF3, EF4, and EF5) are well above the forces currently required for building design (refer to Section 2.2 and Appendix E for more information on the EF rating scale). Many of the damaged commercial and industrial buildings observed by the MAT were large structures. The buildings appeared to have been designed in accordance with the governing codes in effect at the time they were built. Therefore, it is most likely that the dramatic building failures observed by the MAT were not the result of poor design or construction, but rather the result of forces being applied to these buildings that were above the expected design parameters.

Organization of Chapter

The observed failures of commercial and industrial buildings (summarized in Sections 5.1, 5.2, 5.3, and 5.4) were more closely associated with the construction type of the building rather than the use of the building. Therefore, this chapter is organized by building construction type rather than by building use. The location of each building described in this MAT report is shown in Figures 5-1 through 5-3 with each building location shown in relationship to the centerline of the tornado damage swath.

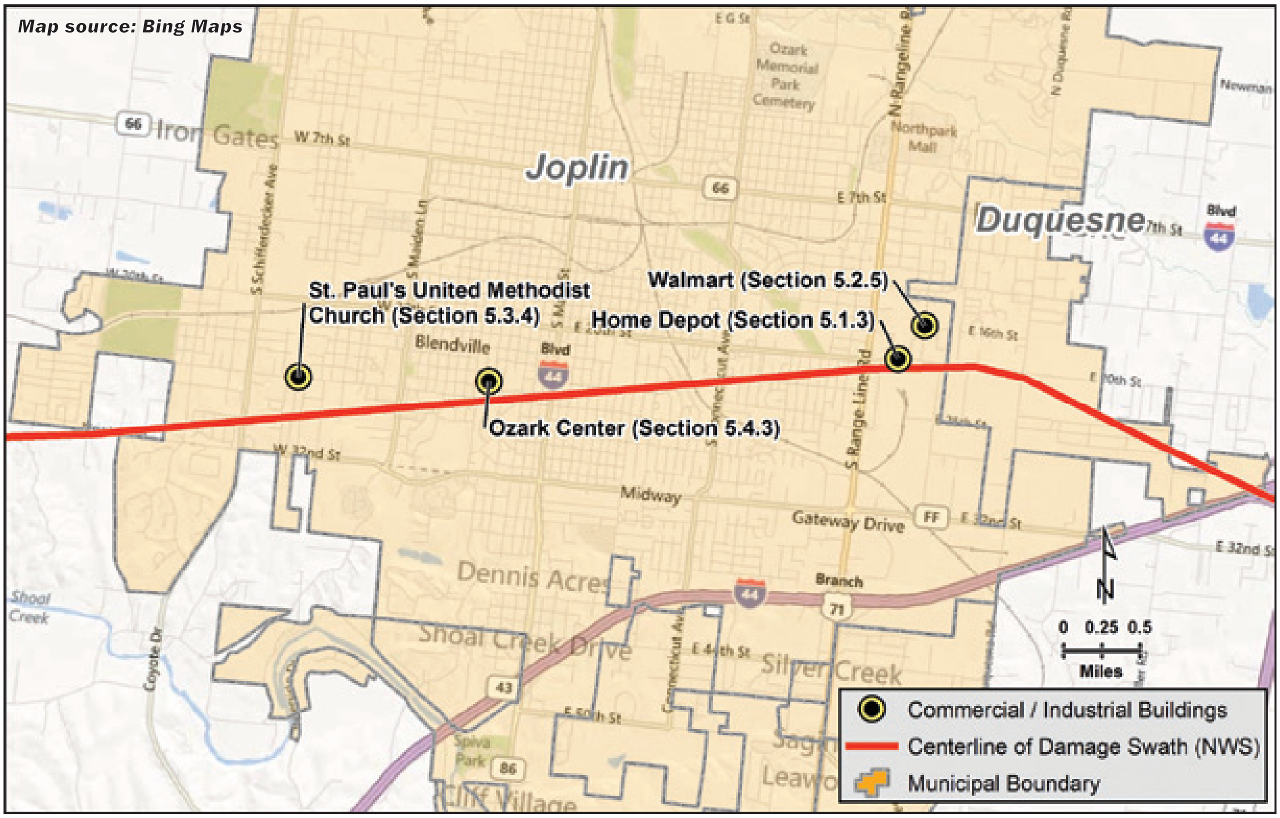

Figure 5-1: Location of Joplin, MO, buildings described in Chapter 5

SOURCE FOR TORNADO TRACK: HTTP://WWW.CRH.NOAA.GOV/SGF/?N=EVENT_2011MAY22_SUMMARY

This chapter summarizes five building types and the typical failures observed by the MAT specific to each building type. Where significant time was spent evaluating a particular site or issue, additional information is provided for that location as a case study. The types of structural failure conditions observed by the MAT were common across various locations due to common commercial construction methods and the consistency of materials manufacturing.

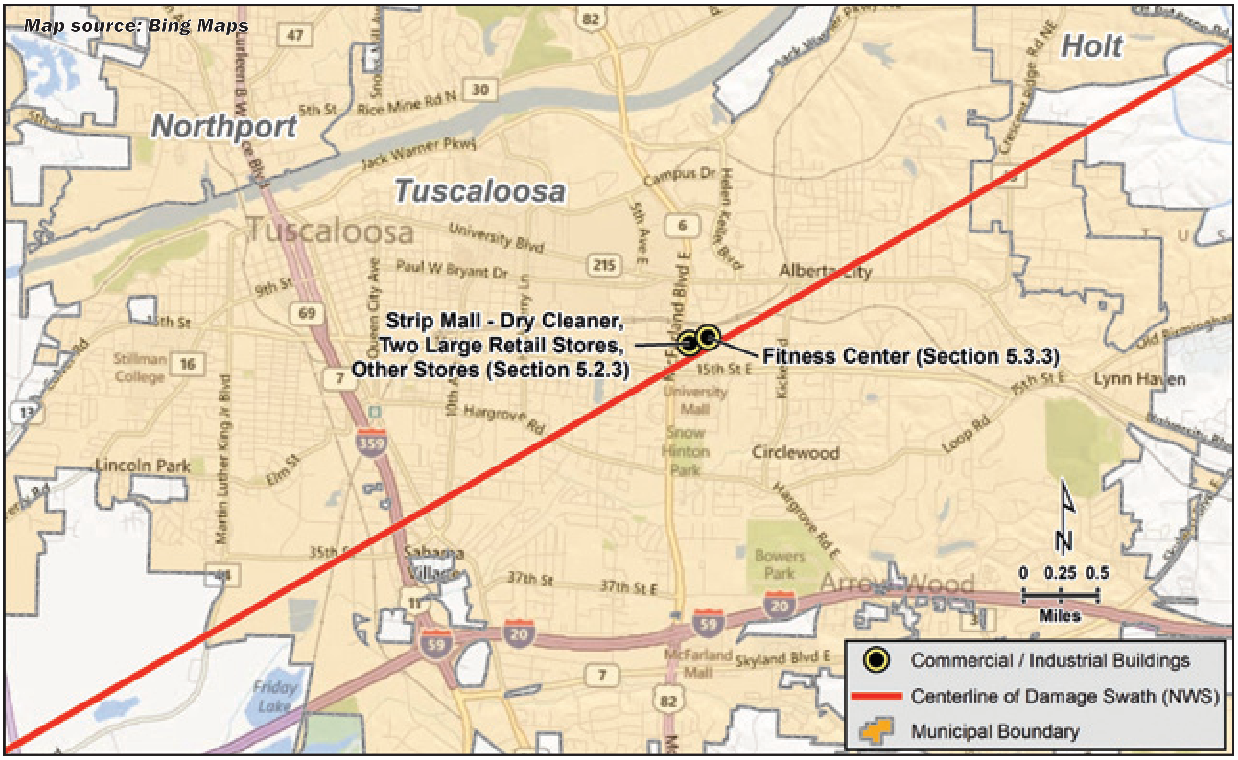

Figure 5-2: Location of Tuscaloosa, AL, buildings described in Chapter 5

SOURCE FOR TORNADO TRACK: HTTP://WWW.SRH.NOAA.GOV/SRH/SSD/MAPPING/

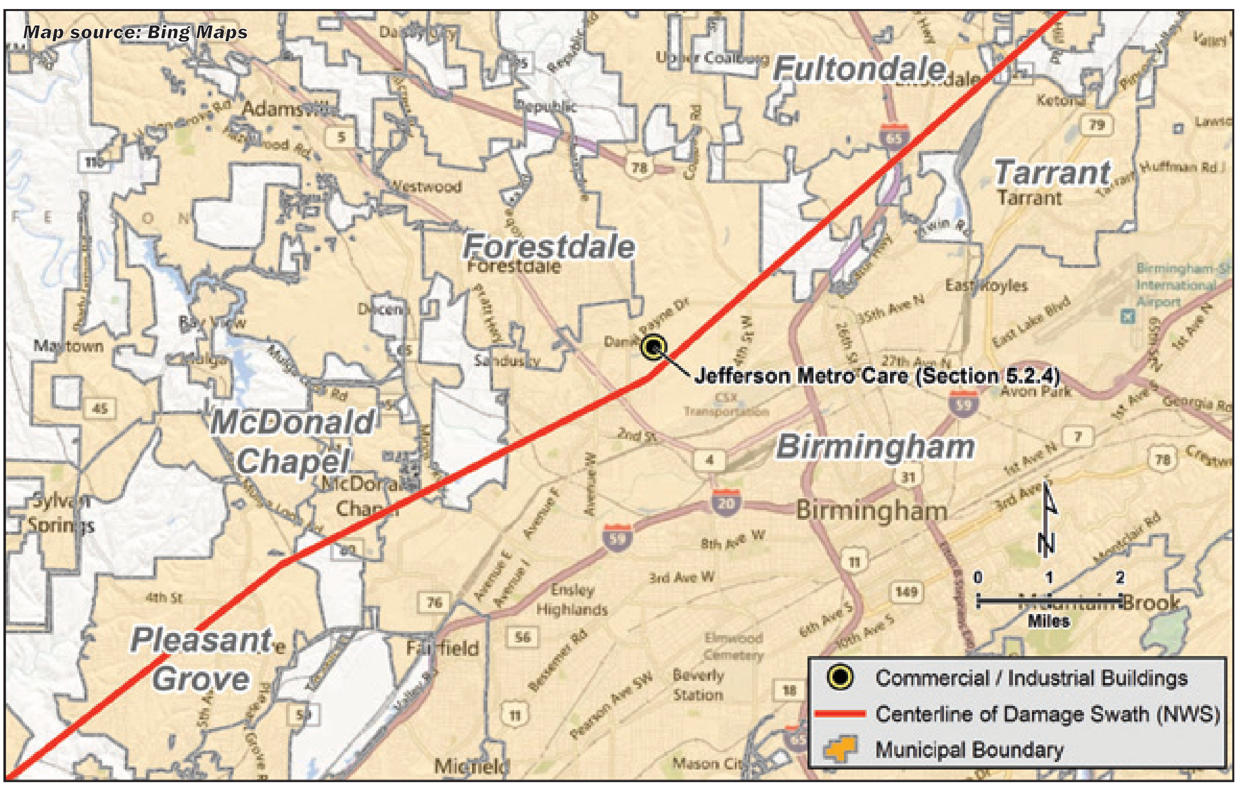

Figure 5-3: Location of Jefferson Metro Care medical office in Birmingham, AL, described in Section 5.2.4

SOURCE FOR TORNADO TRACK: HTTP://WWW.SRH.NOAA.GOV/BMX/?N=EVENT_04272011TUSCBIRM

5.1 Tilt-Up Precast Concrete Walls with Steel Joist Roof System

The MAT observed several damaged buildings constructed using tilt-up precast walls with a steel joist roof system. This type of building construction is described in Section 5.1.1 and its typical failure modes in Section 5.1.2. One of these, a Home Depot, was assessed in detail and is presented as a case study in Section 5.1.3.

Following the tornado in Joplin, MO, the Tilt-Up Concrete Association formed a task force to investigate claims made by an article that criticized the failure and failure modes of tilt-up concrete wall construction during the tornado. Their report, *Analysis of Damage from Historic Tornado in Joplin, Missouri, U.S.A. on May 22, 2011, a Report to the Technical Committee of the Tilt-Up Concrete Association by the Natural Disaster Task Force,* was published in January 2012.

5.1.1 Description of Construction Method and Load Path

The construction erection procedures of precast concrete and tilt-up concrete panel construction are similar in process. These construction practices were developed to eliminate the use of difficult, expensive, and time-consuming vertical forming of wall elements. In these casting methods, concrete wall panels are made by placing concrete in forms that are laid flat on a casting bed. The panel is then either brought to the project site or picked up from its onsite casting bed and "tilted" into place. During construction, the panels are braced until the connections and load transferring systems are in place.

The tilt-up method reduces the scaffolding work associated with masonry work or poured concrete lifts associated with cast-in-place methods. This construction is typically used for long span roof systems and high ceilings, and is therefore commonly used in large commercial super-centers (supermarkets, household goods, and building material supply stores), as well as warehouses, industrial buildings, agricultural facilities and other high-ceiling single-story applications.

Tilt-up concrete panels are typically relatively thin, usually 7 to 12 inches thick. The individual panels may be multi-story, and some designs have reached heights of 50 feet and higher. Wall panels are typically supported on concrete foundations and may be connected to the floor slab with a cast-in-place perimeter strip between the wall and the slab. Although in many applications panels do not need anchorage due to their heavy weight, the code requires a minimum of two ties per panel and connections that rely solely on friction from gravity may not be used. Interior column and frame systems are commonly used for intermediate support of multiple stories or roof systems. Roof systems in these types of structures may rest on a corbel formed into the wall panel or, more commonly, may be attached with embedded weld plates and brackets at the top of the tilt-up concrete panels.

The load paths of these buildings are straightforward because of the small number of elements involved, which makes the relatively few connections and components in the building very important. The elements of these types of diaphragm structures are connected in a system that allows the various loads to be transferred from element to element down to the foundations:

- Uniform vertical loads are carried by the roof deck to the joists. Vertical point loads are taken directly to the joists. Horizontal loads are distributed to the roof deck and gathered at shear walls.

- The joists then transfer the loads through the joist seats to the joist girders at panel points. In some cases, the joists transfer the loads through their seats to beams or walls.
- The joist girders then transfer loads to either columns or walls. The walls sit on foundations and convey the accumulated forces directly via contact and anchorage.

The connections between these building elements are therefore critical due to the loads flowing through them. Horizontal and vertical uplift loads on the roof deck are typically transferred via puddle welds to the joists that support the deck, or to collector elements at the walls. Welds are also used to transfer forces from joists to joist girders, joists to walls, joist girders to columns, joist girders to walls, and to connect columns to base plates.

5.1.2 Typical Failure Modes Observed by the MAT

Structural failure and catastrophic collapse of this building type was observed in several locations. One example of tilt-up construction, a Home Depot, was assessed in detail and is presented as a case study (Section 5.1.3). Although some failures may have been the result of overload on the long span roof systems, the more common condition observed was the failure of the roof deck-to-joist connections and the roof-to-wall panel connections. These connection failures in the MWFRS diaphragm and at the top of the wall allowed the large sections of wall panels to collapse.

Due to the open nature of most buildings using this construction method, the collapse of the very heavy full-height floor-to-roof wall panels did not produce interior pockets of space where occupants could take cover and survive during catastrophic structural failures. Significant damage to building interiors and resulting injury to occupants occurred when these buildings' non-redundant main structural support systems were overloaded. When one panel failed, the loads shared by the adjacent wall panels increased markedly, resulting in the propagation of failure to more of the wall panels, sometimes leading to complete collapse of the exterior walls and roof.

> A typical 9-inch-thick x 25-foot-wide x 30-foot-high panel weighs about 84,000 pounds.

5.1.3 Home Depot (Joplin, MO)

The 108,000-square-foot Home Depot, located in Joplin, MO, is a typical example of a tilt-up concrete building destroyed by a very intense tornado. According to a local Home Depot representative, there were seven fatalities. Twenty-eight people in the store survived.

Location of Facility in Tornado Path: The MAT inspected the Home Depot in Joplin, MO (location shown in Figure 5-1), which was destroyed during the tornado. Figure 5-4 shows the building after the tornado and its location in relationship to the centerline of the tornado damage swath. The NWS rated the center of the tornado circulation in the vicinity of the building as an EF4 to EF5.

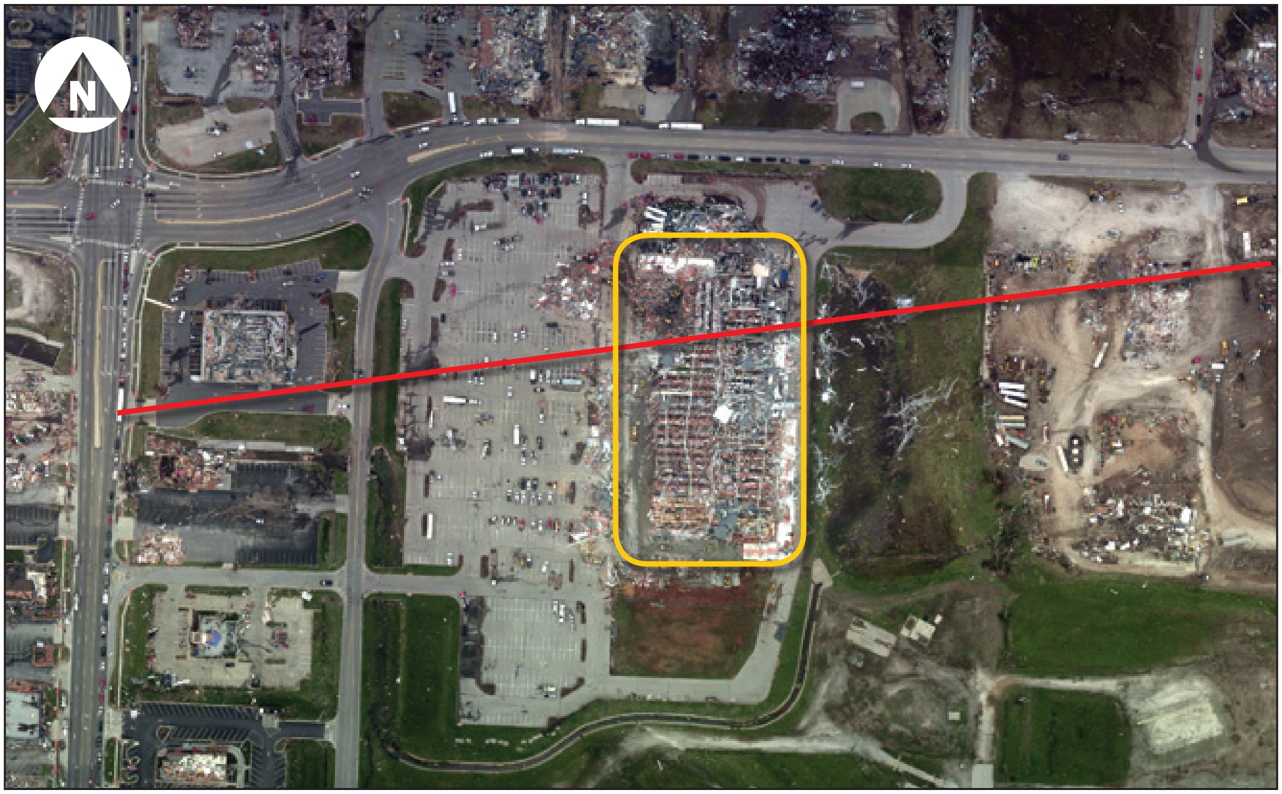

Figure 5-4: Aerial view of Home Depot (yellow circle) in Joplin, MO, in relationship to the approximate centerline of the May 22, 2011 tornado damage swath (red line)[1] (Joplin, MO)

SOURCE: ALL AERIAL PHOTOGRAPHS ARE FROM NOAA IMAGERY (HTTP://NGS.WOC.NOAA.GOV/STORMS) UNLESS OTHERWISE NOTED

Facility Description: The Home Depot had a footprint of 240 feet by 450 feet. The structural system for this large building used the following structural and roof covering elements:

- Membrane roofing
- Insulation board
- Metal roof deck
- Open web steel joists
- Open web steel joist girders
- Square tube columns supporting joist girders
- Precast concrete exterior walls
- Shallow foundations

1 The red line in this and all similar figures represents the center of the damage swath. The track location is approximated by the MAT based on post-event aerial photographs. The actual centerline of the vortex is offset from the centerline of the damage.

General Wind Damage: After the tornado, some of the precast concrete walls were still standing in the northeast corner of the building and partially along the southwest side. The remainder of the wall panels had collapsed. Some of the wall panels had collapsed inwards, while others had collapsed outwards.

Roof System

The connection failures and loss of lateral support of the structural elements led to the total collapse of the roof structure, which in turn led to the collapse of the walls. Large portions of the roof membrane, insulation board, and metal deck diaphragm were lifted from the building and moved outside of the building footprint to the open field east of the Home Depot (Figure 5-5). The roof on the front (east) bay remained attached to the joists inside the collapsed foot print.

As previously noted, the roof deck is typically connected to the joists by puddle welds. An example of a failed puddle weld used to connect the metal deck to the top chord of the steel joists is shown in Figure 5-6. The MAT noted this type of connection on each of the metal deck structures observed at the Home Depot. The roof metal deck acted as a lateral diaphragm and was the primary load-carrying system for lateral loads in the building. Once the roof deck connections failed the steel open web bar-joists and joist girders lost their lateral support and became unstable.

One of the advantages of the steel joist system is that it is a more cost effective system than a traditional steel system of wide flange beams and girders. The joists are lightweight and they can be widely spaced and be used on long spans. The system is primarily designed to carry downward vertical loads and can carry horizontal loads that are parallel to the length of the joist.

A disadvantage of the system is that the combination of the elements used in constructing the joists and joist girder and its length create members that have little horizontal capacity when loaded laterally in an un-braced condition, as was the case described above when the roof deck connection failed. This weakness is also evident in the joist girders, which became un-braced when the connection between the joists and joist girders failed. Figures 5-7 and 5-8 demonstrate the lack of rigidity in an un-braced joist system.

Figure 5-5:
Field east of Home Depot with roof debris (Joplin, MO)

Figure 5-6:
Failed puddle welds that connected the metal roof deck to the top chord of the joist (red arrows) (Joplin, MO)

Figure 5-7:
Joist girder and column failure. The column-to-joist girder is shown by the red arrow (Joplin, MO).

Another common practice in the industry is not welding the bottom chord to the stabilizer plate on joist girders at the column support. The bottom chord was not connected at the Home Depot and the MAT noted several instances of separation at this location (Figure 5-8). In some buildings, the bottom chord is welded if the system is designed as a moment frame system, but if analysis determines that the structure is adequate for the design loads without welding the bottom chord, it is appropriate and encouraged to not make this connection rigid. Alternatively, welding the bottom chord at this location can help keep the bottom chord from buckling provided the connection is capable of resisting the tension induced at this point by the uplift of the joist. However, complications can occur if the construction sequence gets out of order and if snow loads are expected. The bottom plate must be welded after all the dead loads are in place to prevent damage to the lower chord as it deflects and rotates to carry the dead load. If the bottom chord is to be welded to the stabilizer plate, this must be indicated on the plans and considered in the girder design as a special loading condition. Yet another option is to use a loose fit bolt in a slotted hole in lieu of a welded connection at the stabilizer plate.

When single-story, large footprint and multi-story commercial buildings fail during tornadoes, large amounts of debris may be generated at the building sites (see Figures 5-7, 5-12, 5-16, 5-21, and 5-34). To address the structural concerns related to this, FEMA 361 and the ICC 500 provide design criteria to account for debris on the roofs of safe rooms and storm shelters and also state that falling and collapse hazards need to be considered with designing, siting, and constructing these protective areas. FEMA 361 and this publication also provide guidance on operational considerations that state equipment and communication systems should be maintained within safe rooms and tornado refuge areas to assist with the rescue and extraction of individuals from such areas when a building collapse occurs.

Figure 5-8:
Separation at bottom chord to stabilizer plate (red arrow) (Joplin, MO)

Interior Columns

The interior columns at the Home Depot were hollow structural steel (HSS) tube sections located on a grid approximately 40 feet x 50 feet, a common industry practice for lightweight steel frames. The columns were attached to the foundations with a 4-bolt base plate that was welded to the column and a 4-bolt connection at the top to the joist girders.

The column elements performed well while the column tops were pulled to the east by the roof translation. Once the roof deck and steel joist connections broke, the lateral and uplift loads on the columns were reduced and the translation stopped.

The MAT observed a column that buckled against the racking system (Figure 5-9). Some columns failed when the hooked anchor bolts for the column pulled out of the concrete. Other columns experienced failure at the base plate due to shear and tension (Figure 5-10). As the column rotated, the force on the compression side of the base plate sheared the bolts and the tension side pulled the hooked anchor bolts free. The code allows the use of hooked anchor bolts when columns are subject to compression only. When anchor bolts are subject to tension a more positive anchorage is created by using headed anchor bolts in lieu of hooked anchor bolts.

Figure 5-9:
Buckled column (Joplin, MO)

Figure 5-10:
Bolt failure at interior column resulting from shear and tension. The hooked anchor bolts pulled out of the slab (red arrow) (Joplin, MO).

Exterior Walls

The exterior walls of the Home Depot building were precast tilt-up concrete panels. In this type of construction, the connection at the base of the wall is a steel plate or angle welded to an embedded steel plate in the footing and the wall. The roof connections consist of an embedded steel plate in the wall connected to the roof members. Where the steel joists are perpendicular to the wall, there are pockets or a ledger angle where the wall panels support the joists. Figure 5-11 shows an example of a failed joist support pocket at the Home Depot building.

Figure 5-11:
Joist support pocket at top of a precast wall. The red arrow points to weld marks from the connection to the framing member seat on the embedded plate. The yellow arrow points to the light blue insulation layer between the concrete shells (Joplin, MO).

Figure 5-11 also shows that the panels of the Home Depot building were insulated, which means there were two layers of concrete with a layer of insulation in the center. The two layers of concrete are usually connected by ties and concrete ribs at the perimeter and sometimes in the center of the panel. This detail was not exposed, however, and the MAT was unable to observe the connection.

The design dictates whether there are other connections along the vertical joints between individual wall panels. The MAT did not observe any panel-to-panel connections in the Home Depot building.

For wall panels parallel to the steel joists, the connection to the roof diaphragm is provided by a deck support angle attached to the wall panel with bolts or weld plates. The roof deck is then attached to this angle. There are more substantial connections from the joist girders to the wall panels than from joists to wall panels. There is also an additional connection where the joist bridging attaches to the wall. The bridging is provided by the steel joist supplier. This connection is often very small and lightly designed as the required bridging member sizes are also small. This bridging is one of the methods used to keep the bottom chords from buckling as it reduces the un-braced length of the chord.

When the connections between the panels and the roof system fail or the roof system becomes unstable due to loss of the diaphragm, the panels became tall cantilevered walls. Exterior tilt-up walls are not typically designed to withstand this condition, and certainly not when subjected to large lateral forces created by high winds. With high wind pressures, they can become unstable and collapse. It is worth noting that they do not fail in bending, which is typically the worst design loading condition and occurs during the initial construction lifting operation. Instead, they collapse by failing to resist rotation about the bottom of the panel when subjected to lateral loading.

The MAT observed several types of failures of the roof-to-wall connections at the Home Depot building including failures of the joist girder-to-wall connections, failure of the joist-to-joist girder connection (Figure 5-12), failure at joist seats (Figure 5-13), and failures of the weld plates (Figures 5-14 to 5-16). These kinds of failures were part of a chain of failures that led to the collapse of most of the walls.

In the area shown in Figure 5-12 the product racking system maintained some integrity as the building structural elements failed; this area could possibly have been used as a location to take refuge as an option of last resort. However, the level of protection would have been poor, as protection from both wind-borne debris and store contents would have been minimal.

Figure 5-12:
Failure of joist-to-joist girder connection shown by broken welds (yellow arrow); red arrow shows location where bridging angle is touching insulation bundles
(Joplin, MO)

Figure 5-13:
The joist seats came free of their bearing locations when both the seat-to-joist weld (yellow arrow) and the seat-to-embed plate weld (red arrow) broke (Joplin, MO)

Figure 5-14:
Example of weld plate and joist failure. The joist seat was torn from the joist (red arrow) and the anchor studs from embed plate were torn out of the concrete (yellow arrow) (Joplin, MO).

Figure 5-15:
The panel at the weld plate failed (red arrow) (Joplin, MO)

Figure 5-16:
Example of a weld plate failure. Note the attached joist (red arrow) and the joist pocket at the top of the wall panel (yellow arrow) (Joplin, MO)

Foundations

The foundations for the Home Depot were not damaged by the tornado event. The MAT did not note any movement of the interior foundations. The anchor bolt failures (described in *Exterior Walls* above) occurred before there was any movement of the foundation.

MAT EF Rating: Using DI 12 (Large Isolated Retail Building), the MAT selected DOD 7 ("complete destruction of all or a large section of the building") for this building. Using the expected wind speed for DOD 7, the MAT derived the tornado ranking as EF4 (165–170 mph winds). Therefore, the estimated wind speed experienced by the building was well in excess of the 90 mph code design requirements for this location. The MAT EF4 rating for the Home Depot is the same as the NWS rating of EF4 for the center of the tornado circulation at this location.

Functional Loss: The Home Depot in Joplin, MO, is a complete loss.

5.2 Load Bearing Masonry with Steel Joist Roof System

The MAT observed numerous buildings constructed using load bearing masonry walls with steel joists as the roof system. This type of building construction is described in Section 5.2.1 and its typical failures modes in Section 5.2.2. Detailed case studies for the buildings are presented in Section 5.2.3 through Section 5.2.5.

5.2.1 Description of Construction Methods and Load Path

Older masonry construction: Masonry construction varies depending on the type and size of the concrete blocks and whether the masonry system is reinforced. Older construction is often unreinforced or inadequately reinforced and is more likely to collapse in what are current design wind speeds. Owners and operators of older buildings constructed prior to the implementation of current building codes can either retrofit the masonry with reinforcement to allow for better performance or should be aware that occupants in these buildings will need to seek more substantial buildings during high-wind events. Refer to Chapter 9 for information on refuge areas and safe rooms/storm shelters.

Bond beams in multiple story construction: Reinforced and unreinforced masonry walls can be used in multiple-story construction. Intermediate stories and roof systems can be attached to a grouted bond beam or corbel constructed into the masonry wall. Steel joists or trusses may span between these walls to create floors. Roof trusses are attached at the top of the wall using either a top plate or they may rest on top of a bond beam. The bond beam is intended to serve two purposes: lateral load transfer along the length of the wall or vertical load transfer from the roof system.

Wall-to-footing/wall-to-roof connections: In order to provide load path continuity at the connection between the masonry wall and footings, some physical connection must be made between the reinforcing steel in the footing and in the wall. Reinforcing steel is used for this connection since the tensile strength of masonry and grout materials is extremely low and it can only be relied on for compression. Reinforcing steel used to make the wall-to-footing connection must be of a sufficient size and length (development length) to transfer the loads. Similarly the wall-to-roof connection needs to be able to provide a complete load path into the wall reinforcement from the roof elements.

5.2.2 Typical Failure Modes Observed by the MAT

Older masonry construction: Inevitably, many of the older buildings the MAT observed collapsed during the tornado outbreak. These failures were observed not only in the direct path of the tornado, but also on the tornado periphery where wind speeds were lower and somewhat closer to design level wind speeds.

Bond beams in multiple story construction: The MAT checked the top sections of toppled walls for the presence of bond beams. Failures of the bond beams were noted by the MAT in buildings located along periphery areas of the tornado damage swath, suggesting either wall or roof loads larger than the wall system was designed to transfer or a concrete strength that was insufficient.

Wall-to-footing/wall-to-roof connections: Failures observed by the MAT occurred in two primary locations: the roof-to-wall connection or at the footing-to-wall connection. The MAT found reinforcing steel in walls and footings to be spaced too infrequently or it was absent altogether in some cases. Where present, development lengths of failed sections of wall were measured and found to be inadequate.

5.2.3 Strip Mall – Dry Cleaner, Two Large Retail Stores, and Other Stores (Tuscaloosa, AL)

Location of Facility in Tornado Path: The MAT observed a small strip mall in Tuscaloosa, AL (location shown in Figures 5-2 and 5-17), which was destroyed during the April 27, 2011 tornado. This strip mall contained a dry cleaner, two large retail stores, a fitness center and other businesses. The dry cleaner was located on the far west end of the strip mall, retail store “A” was located at the northeast end of the mall, and retail store “B” was adjacent to and south of the first store. The fitness center is described in detail in Section 5.3.3.

The dry cleaner was very near the centerline of the tornado damage swath; the NWS rated the center of the tornado circulation in this location as an EF4. The MAT made detailed observations of the dry cleaner, described below.

Facility Description: The dry cleaner building had a footprint of roughly 150 feet by 160 feet. The structural and roof covering systems for this building used the following elements:

- Membrane roofing
- Insulation board

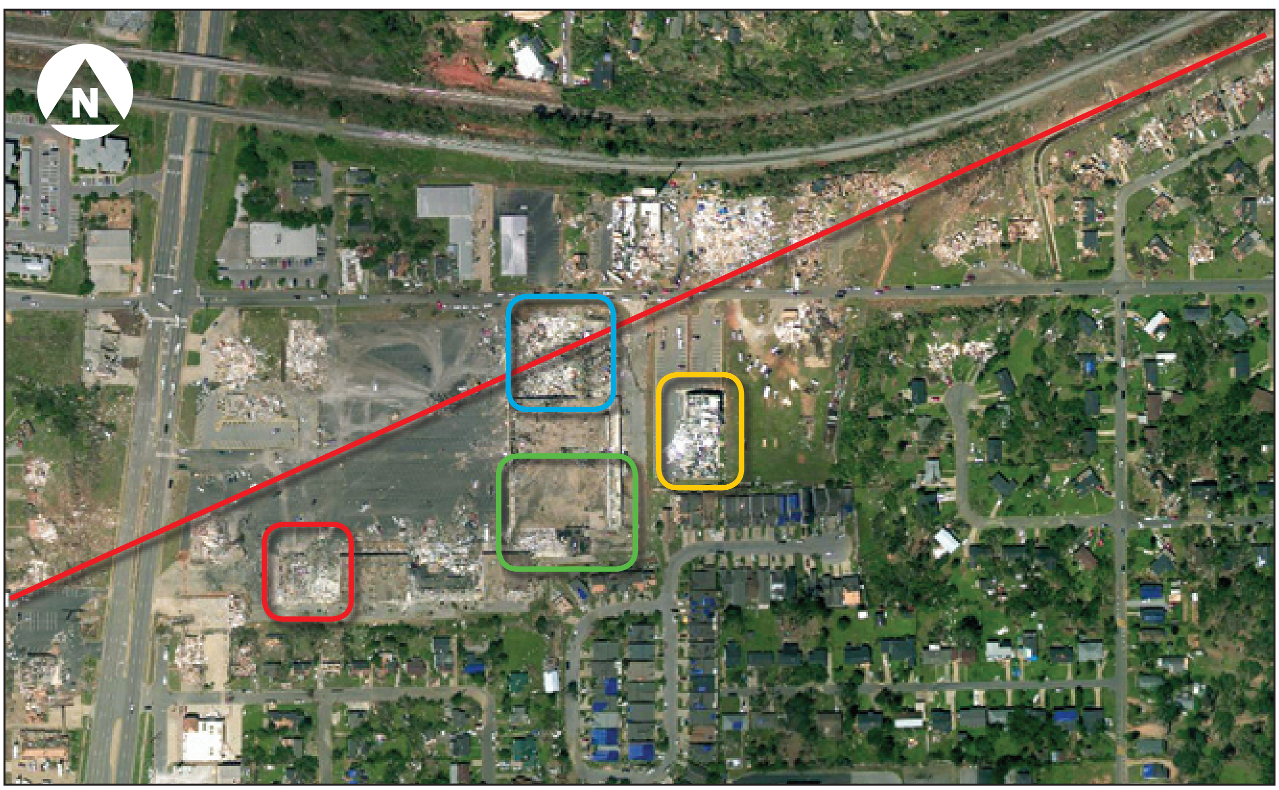

Figure 5-17: Aerial view showing the locations of the dry cleaner building (red box), fitness center (yellow box), retail store “A” (blue box) and retail store “B” (green box) in relationship to the approximate centerline of the April 27, 2011 tornado damage swath (red line) and (Tuscaloosa, AL)

- Metal roof deck
- Open web steel joists
- Unreinforced CMU exterior walls
- Shallow Foundation

General Wind Damage: After the tornado, only one exterior wall and two interior walls of the dry cleaner were left standing. Most of the CMU walls collapsed inward on the building and the roof was either torn away or collapsed in on the building.

Exterior Unreinforced CMU Walls

The exterior walls on the dry cleaner building were constructed of unreinforced CMU. The walls had some horizontal joint wire reinforcing but no vertical reinforcing or grouted cells. The walls on the front and rear of the building failed, collapsing inward on the building. The connections at the roof failed causing the walls to behave as a tall cantilever wall, which caused the bending stresses to exceed the material stress capacity. Figure 5-18 shows collapsed unreinforced masonry walls. The wall shared with the adjoining building was left standing, as were a few of the smaller interior walls. These walls were supported by roofing from two sides. Since the roof was left mostly intact on the other side of the wall, the wall had some lateral support and remained standing.

The connection at the base of the wall typically consists of reinforcing steel that is embedded into the foundation and then extended up into the CMU cells. The cells are then grouted, locking the reinforcing in place and allowing it to transfer both lateral and uplift load. The walls of the dry cleaning building did not have any visible steel connection between the base of the CMU wall and the foundation. The CMU walls relied on the block mortar joints and self-weight to support the wall. Figure 5-19 shows the lack of reinforcing in the entire wall and also at the connection between the base of the CMU wall to the foundation. The failure sequence is captured in Figures 5-20 and 5-21.

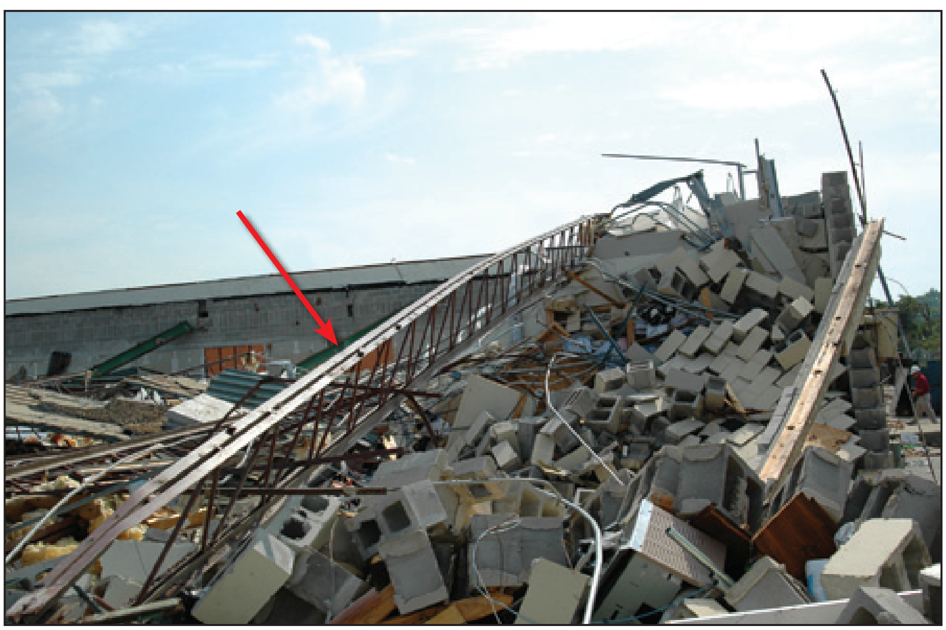

Figure 5-18:
Steel joist (red arrow) in midst of collapsed unreinforced masonry wall at the dry cleaner store (Tuscaloosa, AL)

Figure 5-19:
Solid steel hot rolled sections (red arrow) left in beam pockets of CMU building section. These supported the steel joists shown in Figure 5-18. Also note lack of reinforcement in the wall and wall-to-foundation connection (Tuscaloosa, AL).

Figure 5-20:
Sequence of failure for CMU wall at the dry cleaner building: wall buckling and initial separation (red arrow) was followed by complete separation of wall from bond beam (yellow arrow) and then by collapse of wall (green arrow) (Tuscaloosa, AL)

Figure 5-21:
Global wall instability failure. The CMU blocks are lying loosely on the ground and many have rotated (yellow arrow) due to the complete separation of all the blocks. Note lack of reinforcement in the wall and especially between wall and footing (red arrow). Inset shows a close-up view of the separation of the blocks (Tuscaloosa, AL).

Exterior Unreinforced CMU Walls: Roof System

The roof connection consisted of an embedded steel plate attached to the bond beam connected to the roof members. Where the steel joists were perpendicular to the wall there were typically pockets or a ledger angle where the joists were supported (Figure 5-22). This connection tied into a bond beam running along the front and back walls of the dry cleaner building. When the wall failed and collapsed (Figure 5-23), the bond beam also failed and collapsed, bringing the bar joist roof system down with it.

MAT EF Rating: Using DI 10 (Strip Mall), the MAT selected DOD 8 (“collapse of exterior walls; closely spaced interior walls remain standing”) for this building. Using the expected wind speed for DOD 8, the MAT derived the tornado ranking as EF3 (140–150 mph winds). Therefore, the estimated wind speed experienced at the building was well in excess of the 90 mph building code design requirements for this location. The MAT EF3 rating for the dry cleaner is lower than the NWS rating of EF4 for the center of the tornado circulation near this location; however, the building was not located directly in the core of the track.

Functional Loss: The dry cleaner building is a complete loss as the exterior walls and roof were destroyed. The two large retail stores are also complete losses.

Figure 5-22:
View of steel joist pockets in CMU wall where joists pulled out (red arrows); the exterior CMU wall fell inwards, shown in the foreground (Tuscaloosa, AL)

Figure 5-23:
Steel joists with joist seat and bond beam on top of collapsed wall (red arrow). Wall fragmentation shows lack of reinforcement in the wall (Tuscaloosa, AL).

5.2.4 Jefferson Metro Care (Birmingham, AL)

Location of Facility in Tornado Path: The MAT visited the Jefferson Metro Care facility in Birmingham, AL (location shown in Figure 5-24), which was destroyed during the April 27, 2011 tornado. The Jefferson Metro Care facility was located just north of the centerline of the tornado damage swath. The NWS rated the center of the tornado circulation in the vicinity of this facility as EF2.

Figure 5-24: Aerial view showing the Jefferson Metro Care Facility (red box) in relationship to the approximate centerline of the April 27, 2011 tornado damage swath (red line) (Birmingham, AL)

Facility Description: The Jefferson Metro Care building had a footprint of roughly 130 feet by 75 feet. The structural and roof covering system for this building used the following elements:

- Built-up roofing
- Insulation board
- Metal roof deck
- Open web steel joists
- Steel beam girders
- CMU with brick veneer exterior walls
- Shallow foundation

General Wind Damage: Most of the exterior walls of the facility withstood the tornado, but a large portion of the northwest roof was damaged when the building envelope was breached at the front windows. Inflow winds resulted in high uplift forces on the roof. The windows along the front exterior walls were blown in by windward pressure.

Exterior CMU Walls with Brick Veneer

The exterior walls of the Jefferson Metro Care building were constructed of CMU with a brick veneer. The MAT was unable to inspect the reinforcement in the majority of the walls that did not fail.

The roof connection to the exterior walls consisted of an embedded steel plate attached to bond beams. The roof joists were then welded to the steel embed plates. This connection tied into a bond beam running along the front and back walls of the building. The bond beam was supported by, but not connected to, a steel beam over the front windows. The bond beam along the front of the building broke away from the steel beam when the roof system was torn away, as shown in Figure 5-25. The interior joist seats tore away from their support on interior steel beams when they folded over the roof toward the rear of the building.

Figure 5-25: Failed bond beam-to-structural steel connection over front windows (red arrow) (Birmingham, AL)

Roof System

The roof consisted of an open web steel bar joist system with a metal roof deck and membrane roofing. Most of the roof failures were a result of failure of the welds for the metal deck diaphragm, bar joists, and main structural beams. Figures 5-26 and 5-27 show the failed roof deck connections. Once struck by high winds, the roof decking was pulled off the bar joists as it was pulled over toward the rear of the building.

Another failure identified was the lack of continuity between the roof structure and the walls. Figure 5-28 shows where a bond beam cell has been stuffed with paper to limit the flow of the grout indicating a serious quality control issue during construction. Figure 5-29 shows a similar condition where the CMU cell is still attached to the steel joist but detached from the wall. Figure 5-30 shows a CMU bond cell that was torn from both the wall and the steel joist.

Figure 5-26:
Roof joist lifted off front (red arrow) and folded over rear half of building (yellow arrow) (Birmingham, AL)

Figure 5-27:
Failed roof deck with no connections between the roof deck and the joists (red arrows) (Birmingham, AL)

Figure 5-28:
This CMU cell was found on the ground adjacent to the structure (shown upside down). The bond beam cell is sealed with paper to keep grout from flowing into lower cells (red arrow) and thus there was no connection to lower elements. The embed plate can be seen attached (yellow arrow) (Birmingham, AL).

Figure 5-29:
Bar joist with embed plate and bond beam cell still attached (red arrow) (Birmingham, AL)

Figure 5-30:
Embed plate with bond beam cell (red arrow) on roof (Birmingham, AL)

MAT EF Rating: Using DI 9 (Small Professional Building), the MAT selected DOD 7 ("uplift or collapse of entire roof structure") for this building. Using the expected wind speed for DOD 7, the MAT derived the tornado rating as a high EF1 (100–105 mph winds). Therefore, the estimated wind speed experienced by the building was in excess of the 90 mph building code design requirements for this location.

The nearest damage survey point assessed by NWS had a rating of EF2. The Jefferson Metro Care building falls within the swath projected from NWS for an EF1 rating, which matches the EF rating derived by the MAT.

Functional Loss: The main floor experienced moderate damage from wind-borne debris and water damage after the roof system was torn away from the front part of the building. This damage rendered the building uninhabitable. The tenant, Jefferson Metro Care, relocated to another nearby facility to resume their practice. The building will need significant repairs to the roof and interior before it can be fully functional.

5.2.5 Walmart (Joplin, MO)

Location of Facility in Tornado Path: The MAT inspected the Walmart in Joplin, MO, which was severely damaged during the May, 2011 tornado. Figure 5-1 shows the location of the Walmart with respect to the tornado damage swath in Joplin. Figure 5-31 shows the building after the tornado and the tornado damage swath in the vicinity of the building. The Walmart was located just north of the centerline of the tornado damage swath. The NWS rated the center of the tornado circulation in the vicinity of the Walmart as EF5. According to a local Walmart representative, there were three deaths among the 200 occupants who were inside the facility during the tornado.

Figure 5-31: Aerial view of a Walmart in Joplin, MO (red box) in relationship to the approximate centerline of the May 22, 2011 tornado damage swath (red line) (Joplin, MO)

Facility Description: The Walmart building footprint was approximately 180,000 square feet, 300 feet x 600 feet. The structural and roof covering system consisted of the following elements:

- Membrane roofing
- Insulation board
- Metal deck roof
- Open web steel joists
- Open web steel joist girders
- Square tube columns supporting joist girders
- Exterior reinforced CMU walls
- Shallow foundations

General Wind Damage: The north portion of the Walmart building remained standing (Figure 5-32) with the majority of its roof structure in place. The south portion of the building lost its roof structure and some of the exterior walls collapsed. At the time the MAT visited, site cleanup of the interior space had been in progress for several days and most of the store contents had been removed.

Figure 5-32: The relatively undamaged west elevation of Walmart after the May 22, 2011 tornado (Joplin, MO)

Roof System

There are two damage levels that occurred within this structure.

North half of building: Within the west side of the north half of the building, the structure remained relatively undamaged, though water infiltration occurred in two places. The roof membrane was compromised, which allowed water infiltration. The exterior envelope of the structure was also compromised, at the north entry on the west side, which allowed water into the interior space via the doors.

The east side of the building within the north half was compromised. The east wall and roof were destroyed beginning at approximately the loading docks on the east side (Figures 5-33 and 5-34). The failures resulted in significant water infiltration. Figure 5-35 is looking north inside the space; note the water level inside the Walmart bag in the lower right corner of the photograph.

Figure 5-33:
Interior of the north half of the Walmart, looking east. Fallen roof structure shown in right side of the picture (red arrow) (Joplin, MO).

Figure 5-34:
Destroyed east side of north half of Walmart (note loading dock facing south) (Joplin, MO)

Figure 5-35:
Interior of the north half of building showing water infiltration collected in bags (red arrow) (Joplin, MO)

<u>South half of building:</u> The south half of the building was hardest hit as it was closest to the tornado track. The roof system, including structural members, failed and compromised the integrity of the load carrying systems.

Puddle welds were used to connect the steel roof deck to the top chords of the steel joists. The MAT observed many instances where this connection failed. Figure 5-36 shows the deck supporting the steel joist since the joist girder is no longer there. Figure 5-37 is taken from the outside of the roof portion of the building looking north; the insulation board is still in place on much of the roof, but the roof membrane is missing.

The typical connection of steel joists to joist girders is provided by welds from the joist seat to the girder top chord. Figure 5-38 shows the failure of these welds in this roof assembly. Another industry practice is not welding the bottom chord to the stabilizer plate on joist girders at the column support. This allows for slight flexural movement and rotation at the supports of the girders as they get loaded and unloaded. If the system is designed as a moment frame system this is often welded. At the Walmart, the bottom chord was not welded. Without the bottom chord being welded all of the torsional resistance of the joist must occur at the top chord angle seat connection. The MAT observed several instances of separation such as shown in Figure 5-39.

Figure 5-36:
Interior view from the south half of the building looking toward the east. This shows the boundary of roof damage to the south portion of the building. Note deck supported by steel joist (Joplin, MO).

Figure 5-37:
View of the south half of the building looking north showing roof damage. Note the missing roof membrane (Joplin, MO).

Interior Columns

The interior columns were steel HSS (tube) sections. The MAT observed several instances where the columns were leaning at a severe angle, but were still attached to the foundation, indicating good anchorages that survived large deformations. Figure 5-40 shows a column that is bent completely over, but is still attached to the foundation. Figure 5-41 shows the roof structure that remained standing in the south portion of the building.

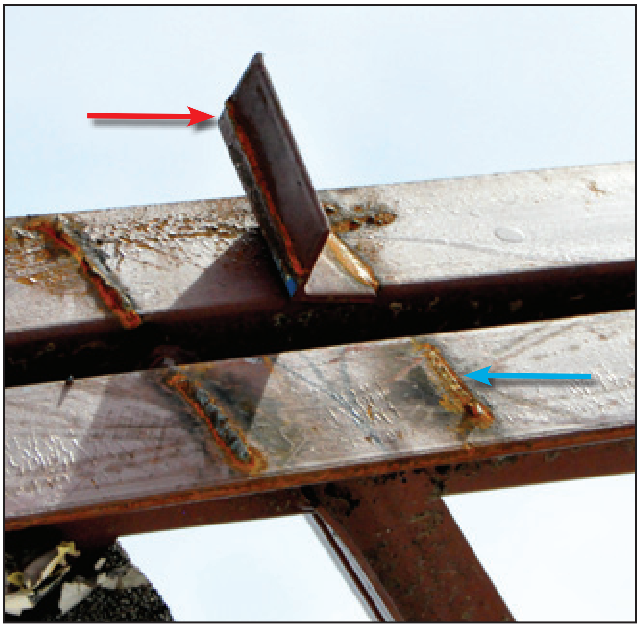

Figure 5-38: Typical connection of two steel joists to joist girder. While the joist seat from one joist remains (red arrow), the weld failed at the other joist seat connection (blue arrow) (Joplin, MO).

Figure 5-39: Joist girder rotated at the column; the bottom chord was not attached to the stabilizer plate (shown by red arrow). The joists were attached with welds to the joist girder top chord. This weld connection failed in the location shown (yellow arrow) (Joplin, MO).

Figure 5-40:
Collapsed column with hooked anchor bolts remains attached to the foundation at the base (Joplin, MO)

Figure 5-41:
Roof structure remaining in south half of Walmart. The red arrow shows the location of a column embedded in the wall (Joplin, MO).

Exterior Walls

The exterior walls of the Walmart were reinforced CMU. In the northwest portion of the building the walls performed adequately (Figure 5-41, upper right side). The walls on the south half of the building and northeast half of the building collapsed. The connections at the roof failed and caused the walls to behave as a tall cantilever wall which caused the bending stresses in the wall and the shear and moment stresses at the base of the wall to exceed the material stress capacity.

The connection at the base of the wall typically consists of reinforcing steel embedded into the foundation and then extended into the CMU cells. The cells are then grouted, locking the reinforcing in place and allowing it to transfer both uplift and lateral loads. Figure 5-42 shows reinforcing cast into the foundations.

Figure 5-42:
Reinforcing steel in Walmart foundation (Joplin, MO)

The roof-to-wall connection for a CMU wall system is similar to the connections for a precast tilt-up wall system. The connections consist of an embedded steel plate connected to the roof members. Where the steel joists are perpendicular to the wall there are pockets or a ledger angle where the joists are connected. The walls that are parallel to the joists are connected to the roof diaphragm with a deck support angle attached to the panel with bolts or weld plates and the deck is attached to the angle. There are more substantial connections at the joist girders. An additional connection also occurs where the joist bridging attaches to the wall, which is provided by the steel joist supplier. This connection is often neglected as the typical bridging member sizes are small or are poorly connected to the joists and girders reducing the effectiveness of the roof in resisting load reversals and uplift.

The roof connections at Walmart were of this typical design. The joists were connected to the walls by welding to embedded steel plates grouted into the CMU. The joist girders were supported on columns embedded in the walls (Figure 5-41 red arrow). The Walmart roof joists were connected to the joist girders with welds at the joist seat to top chord connections (Figure 5-39).

In some areas of the store, the roof and walls stayed intact enough that refuge could be found. The MAT observed a relatively undamaged space located in the southern end of the Walmart (Figures 5-43 through 5-45). Although the performance may have been circumstantial, this smaller space could have been a candidate for an area of refuge and designed/constructed accordingly.

Figure 5-43:
Partial collapsed wall in southern half of store (note deck support angle at top of wall); area of limited damage shown by red arrow (Joplin, MO)

Figure 5-44:
Area of relatively limited damage. Photograph was taken looking north (Joplin, MO).

Figure 5-45:
Another view of the area shown in Figure 5-44. Photograph was taken looking east (Joplin, MO).

MAT EF Rating: Using DI 12 (Large Isolated Retail Building), the MAT selected DOD 6 ("inward or outward collapse of exterior walls") for this building. Using the expected wind speed for DOD 6, the MAT derived the tornado rating as EF4 (165–175 mph winds). Therefore, the estimated wind speed experienced by the building was well in excess of the 90 mph building code design requirements for this location. The MAT EF4 rating for the Walmart is lower than the NWS rating of EF5 for the center of the tornado circulation near the building.

Functional Loss: The Walmart in Joplin is a complete loss.

5.3 Light Steel Frame Buildings

The MAT observed damaged buildings that were constructed using light steel frames. This type of building construction is described in Section 5.3.1 and its typical failure modes in Section 5.3.2. Two buildings of this construction type were assessed by the MAT and are described in detail in Sections 5.3.3 and 5.3.4. A lack of wind resistance was observed in the roof purlins and the frame-to-foundation connection in this light steel frame construction.

5.3.1 Description of Construction Method and Load Path

Light steel frame construction is common for commercial buildings. These buildings are typically only one or two stories. They range from steel stud framing systems, which are constructed in a manner similar to wood framed buildings, to pre-engineered steel rigid frame truss buildings (i.e., pre-engineered metal building [PEMB]) that are fabricated offsite and erected on foundation slabs and covered with light gauge steel panels.

Steel stud framing systems: Steel stud framing systems are commonly used for either light steel framed buildings or infill walls for other building systems. These walls are typically braced by using steel straps or angles attached to the outside of wall systems. The interior of the walls are usually gypsum wallboard and the exterior is covered with brick veneer, an exterior insulation and finishing systems (EIFS), or textured paneling systems. Steel framing also allows for large openings for glazing or doors, making it common for commercial store fronts. Roof systems are either wood or steel truss systems and depend on larger steel sections to carry loads down the framing system and into the foundation.

Pre-engineered metal buildings: PEMBs consist of a series of pre-engineered trusses, which are a set of columns and roof beams fabricated into a continuous steel frame section or "bent". These sections are bolted to a foundation or slab by anchor bolts. The walls and roof are framed with a system of channels or z-shaped purlins (for roofs) and girts (for walls) before being covered with light gauge steel panels. Due to the extent of prefabrication available, these buildings can be quickly constructed for a relatively low cost. The frames resist lateral loading along the column and beam lines, but as these loads are applied, significant loads are transferred to the foundations of the building.

5.3.2 Typical Failure Modes Observed by the MAT

Light steel frame buildings have been developed to make this construction type economical to build. These structures often experience significant structural damage in high-wind events because there is no redundancy in their design and they are best suited where only normal downward vertical loads are the primary design loads. Failures observed by the MAT typically occurred either in the base plate/anchor bolt system or the anchor bolt pulling out of the foundation

High winds often damage the exterior finish or glazing of light steel frame buildings. Most of the exterior finish or glazing failures observed by the MAT in light steel frame buildings were the result of unprotected glazing or insufficient attachment of exterior cladding or veneers to the structural frame. Once the glazing is breached, the building interior is exposed to wind pressures, which subject the lightly built roof system to increased uplift loads.

5.3.3 Fitness Center (Tuscaloosa, AL)

Location of Facility in Tornado Path: The MAT inspected a fitness center in Tuscaloosa, AL, which was destroyed during the tornado. The location of this building is shown in Figure 5-2. Figure 5-46 shows the building after the tornado and the tornado damage swath in the vicinity of the building. This building was just east of the buildings discussed in Section 5.2.3. The fitness center was located on the southern periphery of the centerline of the tornado damage swath. The NWS rated the center of the tornado circulation in the vicinity of this building as an EF4.

Facility Description: The footprint of the building that sustained the most damage was roughly 90 feet by 130 feet. The structural and roof covering system consisted of the following elements:

- Metal roofing and siding
- Metal roof purlins
- Insulation
- Secondary metal framing
- Steel clear-span moment frame system
- Shallow foundations

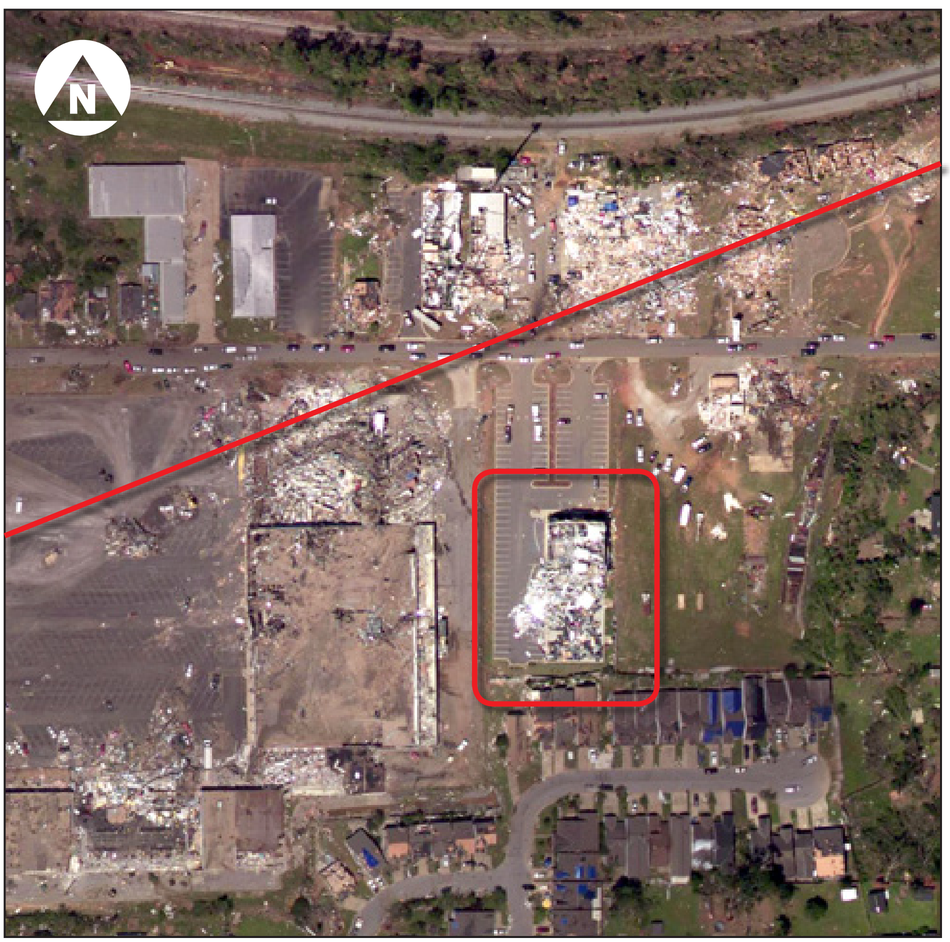

Figure 5-46:
Aerial view of the fitness center (red box) in relationship to the approximate centerline of the April 27, 2011 tornado damage swath (red line) (Tuscaloosa, AL)

General Wind Damage: The southern part of the building was completely destroyed, while some of the northern part of the building was left standing (Figure 5-47). The failures observed were due to a breach of the building envelope from inflow winds that then resulted in excessive wind pressures being exerted on the MWFRS.

Figure 5-47:
Front (north side) of fitness center building (Tuscaloosa, AL)

The MWFRS of the north end of this building performed well relative to the buildings in the immediate surroundings and exhibited ductility through much of the failure, providing cavities in which people could survive. The main column frame anchorages to the foundation performed well in the context of extreme overload (Figure 5-48). The column tore free from the base plate at the weld leaving the base plate and anchor rods in place. The steel anchor rods and base plates were stressed to the point of full yield—characterized by exaggerated deformation—which led to a failure of the welds to the columns (Figure 5-49).

MAT EF Rating: Using DI 21 (Metal Building Systems), the MAT selected DOD 7 ("progressive collapse of rigid frames") for this building. Using the expected wind speed for DOD 7, the MAT determined the tornado rating as EF3 (140–145 mph winds). Therefore, the estimated wind speed experienced by the building was well in excess of the 90 mph building code design requirements for this location.

The MAT EF3 rating for the fitness center is somewhat lower than the NWS rating of EF4 for the portion of the tornado track near the building. The nearest NWS survey point was a small retail building approximately 1,000 feet west of the fitness center. The fitness center was not in the center of the tornado track and accordingly, wind speeds away from the center would result in a lower speed at the building.

Functional Loss: Most of the fitness center building in Tuscaloosa was destroyed.

Figure 5-48:
The anchor bolts of this base plate connection performed well while the weld along the base of the steel column failed (red arrow) (Tuscaloosa, AL)

Figure 5-49:
Ductile end column at southwest corner of building (red arrow). The anchor bolts remained attached to both the foundation and the column (Tuscaloosa, AL).

5.3.4 St. Paul's United Methodist Church (Joplin, MO)

Location of Facility in Tornado Path: The MAT inspected St. Paul's United Methodist Church in Joplin, MO, which was heavily damaged during the tornado. The church was located on the periphery of the tornado track; the NWS rated the center of the tornado circulation in the vicinity of the church as EF2. Figure 5-1 shows the location of the building relative to the tornado damage swath. Figure 5-50 shows a close-up aerial view of the building and its proximity to the tornado damage swath.

Figure 5-50: Aerial view of St. Paul's United Methodist Church (red box) (Joplin, MO) in relationship to the approximate centerline of the May 22, 2011 tornado damage swath (red line) (Joplin, MO)

Facility Description: The footprint of the building that sustained the most damage was roughly 11,700 square feet with dimensions of 90 feet by 130 feet. The structural and roof covering system consisted of the following elements:

- Metal roof decking
- Metal roof purlins
- Insulation
- Secondary metal framing
- Steel clear-span moment frame system
- Shallow foundations

General Wind Damage: The southern wing of the St. Paul's United Methodist Church complex was heavily damaged to the point of being substantially destroyed. The MWFRS used for the building exhibited good performance and was left standing as well as several interior walls (Figure 5-51). However, the roof, siding, and end walls were completely removed. The damage to these building envelope elements was due to the breaching of the building envelope from tornado winds, which resulted in a failure of these secondary elements relieving the internal wind pressure from the MWFRS (Figure 5-52). The primary main column frames and their anchorage to the foundation performed very well (Figure 5-53).

Figure 5-51:
Intact PEMB main frames (red arrow) (Joplin, MO)

Figure 5-52:
Roof system purlins intact with metal roof clip released (red arrows) (Joplin, MO)

Figure 5-53:
Secondary framing (light gage infill walls) failed (yellow arrow) while the main frames survived (red arrow) (Joplin, MO)

MAT EF Rating: Using DI 21 (Metal Building Systems), the MAT selected DOD 3 ("metal roof or wall panels pulled from the building") for this building. Using the expected wind speed for DOD 3, the MAT derived the tornado ranking as EF1 (100–105 mph winds). Therefore, the estimated wind speed experienced by the building was in excess of the 90 mph building code design requirements for this location. The MAT EF1 rating for the church is lower than the NWS rating of EF2 for the center of the tornado circulation near the building.

Functional Loss: The southern wing of the St. Paul's United Methodist Church complex will need to be completely rebuilt. Although large portions of the MWFRS remained intact, the secondary elements suffered severe damage. This exposed the interior to major wind damage that will require full reconstruction.

5.4 Reinforced Concrete Frame with CMU Infill Walls

The MAT inspected one building constructed using a concrete frame with CMU infill walls. This type of building construction is described in Section 5.4.1 and its typical failure modes in Section 5.4.2. The MAT findings for the building are described in Section 5.4.3. The building was located outside of the periphery of the tornado damage swath; the NWS rated the center of the tornado circulation in the vicinity of this building as EF4 to EF5. The damage may have been due to the building being taller than any of the surroundings and therefore more exposed to the high winds.

5.4.1 Description of Construction Method and Load Path

Reinforced concrete frame buildings are commonly used in multi-story commercial and industrial buildings. The building's primary structural elements are cast-in-place concrete, which creates a large heavy structural frame. The structural elements are the floor system, the beams or joists for the floors, the columns, and the foundations. This construction typically results in substantial redundancy in the structural systems.

5.4.2 Typical Failure Modes Observed by the MAT

The failures observed by the MAT in reinforced concrete frame buildings were limited to the secondary elements and the building envelope. The MWFRS of the buildings remained undamaged by the tornado winds.

5.4.3 Ozark Center for Autism (Joplin, MO)

Location of Facility in Tornado Path: The MAT inspected the Ozark Center for Autism in Joplin, MO, which was damaged during the tornado. The building is located just outside the periphery of the tornado damage swath; the NWS rated the center of the tornado circulation in the vicinity of this facility as EF4 to EF5 (Figure 5-1). Figure 5-54 shows a close-up aerial view of the building after the tornado and its relationship to the tornado damage swath.

Facility Description: The building footprint of the Ozark Center for Autism is approximately 450 feet x 250 feet. The structural and roof covering systems include:

- Standing seam metal roof
- Ballasted roof covering (original system)
- Poured-in-place concrete roof and floor slabs
- CMU elevator and stair shafts
- Poured-in-place concrete columns
- CMU infill walls
- Exterior furring and metal wall panels over the CMU
- Steel roof trusses (east extension)

General Wind Damage: The structural core of the Ozark Center for Autism was not significantly damaged; the structural systems on this building performed very well. The building envelope, however, was heavily damaged. The primary damage occurred to the roofing materials and glazing (Figure 5-55). The metal architectural panel siding on the building failed, as would be expected in this type of event.

After the tornado, the damage to the building consisted of:

- Loss of exterior skin
- Loss of roof

Figure 5-54: Aerial view of the Ozark Center for Autism (red circle) in relationship to the approximate centerline of the May 22, 2011 tornado damage swath (red line) (Joplin, MO)

- Loss of exterior glazing
- Water damage to the building interior
- Loss of exterior building walls at the two-story extension

Building Construction

The Ozark Center is a three-story main building that has a two-story extension on the east side (Figure 5-56). Figure 5-57 shows the typical interior layout of the main building with a perimeter beam and column system. There are two rows of center columns in the two-story extension. The slab is thickened between the rows of center of columns at each level. The remainder of the building is cast-in-place concrete.

Figure 5-55:
East elevation of the Ozark Center for Autism showing damage to glazing and siding (Joplin, MO).

Figure 5-56: East elevation from northeast corner of building. The structural core of the taller building performed well, as did the CMU infill. The wing in the nearside of the figure is a two-story extension. Wood wall-framing debris can be seen in the foreground (Joplin, MO).

Figure 5-57:
Two-story extension on east side of building shown in Figure 5-56; view shows typical interior layout (Joplin, MO)

Roof System

The roof system on the main building consists of a poured-in-place concrete roof deck that was subsequently covered over by adding steel purlins attached to the roof deck at approximately 5 feet on center. The original concrete roof deck was undamaged. The MAT observed clips in place that would accept new roof material, most likely a metal roof deck system. The connection of the roof material to the purlins had failed and the roof material was not observed at the site (Figure 5-58).

Figure 5-58:
Roof of the third-story main building. The roof overbuild purlins are shown with green arrows while the roof clips that unlatched are shown by red arrow (Joplin, MO).

The roof framing system at the two-story extension is constructed with engineered steel girders and steel joists that span between them. A metal roof deck is connected to the joists (Figure 5-59) and a ballasted roof system is placed over that. The same layered roof construction was used on the three-story main building.

A portion of the deck in the northeast corner of the two-story extension failed when the puddle weld connections failed, but the core structure remained in place (Figure 5-60).

Figure 5-59:
Roof section at two-story extension showing how the metal roof deck diaphragm is connected to the joists (Joplin, MO)

Figure 5-60:
View of roof of the two-story extension observed from the third floor. Note the failed decking at the corner (red arrow) (Joplin, MO)

Floor System

The floor system is a reinforced poured-in-place concrete slab that spans between the perimeter beams and interior columns. It is approximately 6½ inches thick with a dropped section between the center columns. The MAT did not observe any damage to the floor system.

Exterior Walls

The exterior walls of the main building consisted of a 4-foot-high CMU wall that was framed between the concrete columns. The CMU walls did not show signs of distress. The glazing that spanned from the top of the CMU walls to the underside of the concrete beam above was destroyed.

The exterior walls of the two-story extension were wood-framed walls with studs spaced at approximately 16 inches on center; these walls were destroyed. Portions of the wood can be seen in the foreground of Figure 5-56.

Building Beams and Columns

The building layout is on column lines that are 21 feet x 17 feet. The two center columns are approximately 6 to 8 feet apart. The MAT did not observe any damage to the concrete beam-and-column structural frame system.

MAT EF Rating: Using DI 17 (Low-Rise Building), the MAT selected DOD 5 ("uplift of lightweight roof structure") for this Ozark Center. Using the expected wind speed for DOD 5, the MAT derived the tornado ranking as EF3 (150-mph winds). Therefore, the estimated wind speed experienced by the building was well in excess of the 90 mph building code design requirements for this location.

The MAT EF3 rating for this building is substantially higher than the NWS rating of EF0 for this area. The NWS rated the center of the tornado circulation for this tornado as an EF5, but the Ozark center was outside the swath derived by the NWS. It is clear, however, the building incurred damage from tornado wind speeds. It is possible that the height of the building contributed to the damage, as it is considerably higher than the surrounding structures.

Functional Loss: The Ozark Center will need repairs to non-structural elements, as the main structure performed well and remained intact.

5.5 Summary of Conclusions and Recommendations

Table 5-1 summarizes the conclusions and recommendations for Chapter 5, *Observations on Commercial and Industrial Building Performance*, and provides references for supporting observations. Additional commentary on the conclusions and recommendations is presented in Chapters 10 and 11.

Table 5-1: Summary of Conclusions and Recommendations for Commercial and Industrial Building Performance

Observations	Conclusions (numbered according to Chapter 10)	Recommendations (numbered according to Chapter 11)
Specific failure states and building survivability that could be addressed in the codes are seen in: • Home Depot (Section 5.1.3) • Fitness Center (Section 5.3.3)	**Conclusion #3** **Wind provisions of the current codes and standards are insufficient to manage building performance in overload events.**	**Recommendation #4** **Include failure states and survivability in building codes and standards.**
Large-footprint commercial structures with long-span roofs that would have possibly benefited from being Risk Category III under ASCE 7-10: • Home Depot (Section 5.1.3) • Walmart (Section 5.2.5)	**Conclusion #3** **Wind provisions of the current codes and standards are insufficient to manage building performance in overload events.**	**Recommendation #5** **Change risk category for large-footprint commercial structures with long-span roofs to Risk Category III under ASCE 7-10.**[2]
Tornado hazard was not adequately addressed in the codes and standards used for construction: • Home Depot (Section 5.1.3) • Strip Mall (Section 5.2.3) • Jefferson Metro Care (Section 5.2.4) • Walmart (Section 5.2.5) • Fitness Center (Section 5.3.3)	**Conclusion #3** **Wind provisions of the current codes and standards are insufficient to manage building performance in overload events.**	**Recommendation #6** **Improve design approach in ASCE 7 and IBC to address risk consistently across hazards.**
Buildings that experienced wind loads that exceeded design wind loads: • Home Depot (Section 5.1.3) • Walmart (Section 5.2.5) • Fitness Center (Section 5.3.3)	**Conclusion #3** **Wind provisions of the current codes and standards are insufficient to manage building performance in overload events.**	**Recommendation #7** **ASCE 7 should improve the commentary on code limitations.**
Building codes and standards do not have clear risk tolerances defined, leading to misinformed decisions when seeking shelter from a tornado: • Home Depot (Section 5.1.3) • Walmart (Section 5.2.5)	**Conclusion #3** **Wind provisions of the current codes and standards are insufficient to manage building performance in overload events.**	**Recommendation #8** **Clarify risk tolerance in ASCE 7 and IBC.**

2 A Risk Category is assigned to buildings based on the risk to human life, health, and welfare associated with potential damage or failure of the building (per ASCE 7-10). The assigned Risk Category, I through IV, dictates the mean return interval for a design event that should be used when calculating the building's resistance to the events. In ASCE 7-05, Risk Categories were called "Occupancy Categories."

Table 5-1: Summary of Conclusions and Recommendations for Commercial and Industrial Building Performance (continued)

Observations	Conclusions (numbered according to Chapter 10)	Recommendations (numbered according to Chapter 11)
Buildings that could have potentially benefited from redundancy of the MWFRS, ductility of connections, resilience, alternate load paths, design for load reversal, robust perimeter element design, continuity of boundary elements, good connectivity, and inclusion of discrete MWFRS components: • Home Depot (Section 5.1.3) • Strip Mall (Section 5.2.3) • Jefferson Metro Care (Section 5.2.4) • Walmart (Section 5.2.5) • Fitness Center (Section 5.3.3) • St. Paul's United Methodist Church (Section 5.3.4) • Ozark Center for Autism (Section 5.4.3)	**Conclusion #3** **Wind provisions of the current codes and standards are insufficient to manage building performance in overload events.**	**Recommendation #9** **Include best practices for wind design in IBC.**
Buildings that did not have a best available refuge area identified, a FEMA 361 or ICC 500-compliant safe room or storm shelter: • Home Depot (Section 5.1.3) • Strip Mall (Section 5.2.3) • Jefferson Metro Care (Section 5.2.4) • Walmart (Section 5.2.5) • Fitness Center (Section 5.3.3)	**Conclusion #3** **Wind provisions of the current codes and standards are insufficient to manage building performance in overload events.**	**Recommendation #16** **Install a storm shelter or safe room or identify best available refuge areas in large-footprint buildings.**
Lack of adequate signage provided to building users and occupants regarding building's design capacity: • Home Depot (Section 5.1.3) • Walmart (Section 5.2.5)	**Conclusion #10** **There was inadequate signage in commercial buildings.** There is a lack of adequate signage in large commercial buildings to give building users and occupants a better understanding of a building's design capacity.	**Recommendation #17** **For all public buildings, install signage in a conspicuous place at building entrances.**
According to management personnel interviewed by the MAT at a Lowes in Tuscaloosa, AL, flip charts helped the response of the store operators during the high stress and confusion of the tornados event by providing emergency protocols. Flip charts could have been potentially helpful for: • Home Depot (Section 5.1.3) • Strip Mall (Section 5.2.3) • Jefferson Metro Care (Section 5.2.4) • Walmart (Section 5.2.5) • Fitness Center (Section 5.3.3)	**Conclusion #11** **Emergency operations flip charts can aid in decision making.**	**Recommendation #18** **Place decision-making check lists or flip charts in prominent locations.**

Table 5-1: Summary of Conclusions and Recommendations for Commercial and Industrial Building Performance (concluded)

Observations	Conclusions (numbered according to Chapter 10)	Recommendations (numbered according to Chapter 11)
Buildings which used unreinforced masonry as primary support: • Strip Mall (Section 5.2.3)	**Conclusion #12** **URM performed poorly as primary support.**	**Recommendation #19** **Do not use URM in primary or critical support areas of a building.**
The MAT noted that the connections between primary structural members on many buildings were the initial point of failure of the structural systems: • Home Depot (Section 5.1.3) • Walmart (Section 5.2.5) • Ozark Center for Autism (Section 5.4.3) • Jefferson Metro Care (Section 5.2.4)	**Conclusion #13** **Connections between primary structural members were often the initial point of failure.**	**Recommendation #20** **Use screws in deck-to-joist connections instead of puddle welds.**
Buildings that could have potentially benefited from enhancements to building connections beyond code requirements: • Home Depot (Section 5.1.3) • Walmart (Section 5.2.5) • Ozark Center for Autism (Section 5.4.3) • Jefferson Metro Care (Section 5.2.4)	**Conclusion #13** **Connections between primary structural members were often the initial point of failure.**	**Recommendation #21** **Include enhancements to building connections beyond the code requirements.**
Large-footprint commercial structures with long span roofs which progressively collapsed: • Home Depot (Section 5.1.3) • Walmart (Section 5.2.5)	**Conclusion #14** **Lack of redundant stability systems or non-discrete structural systems contributed to progressive collapse.** This type of failure occurred in large-footprint commercial structures with long-span roofs occurred when small local failures progressed to larger areas of failure.	**Recommendations #22, #23, and #24** **(#22) Incorporate redundancy in the MWFRS.** **(#23) Incorporate more redundancy in the design of large-footprint buildings.** **(#24) Use discrete structural systems in large, long-span buildings.**