



TORNADO OUTBREAK *of* 2011

IN ALABAMA, GEORGIA, MISSISSIPPI,
TENNESSEE, AND MISSOURI

4 Observations on Residential Building Performance

The MAT observed residential building performance across the area impacted by the April 25–28, 2011 tornado outbreak and the May 22, 2011 Joplin, MO, tornado.

According to data assembled by NOAA's NWS SPC, of all reported tornadoes in the United States between 1950 and 2006, nearly 95 percent have been rated as the equivalent of EF2 or less (up to 135 mph for 3-second gust) (FEMA 2008a). While the April 25–28, 2011 tornado outbreak and May 22, 2011 Joplin tornado were extraordinary in scale and number of lives lost, the majority of residential damages observed, described, and documented by the MAT was determined to have resulted from wind speeds estimated to be 135 mph or less based on the EF scale damage ranking indicators. Winds of this magnitude generate substantial forces that can result in significant damage, but could be mitigated through enhanced wind-resistant construction procedures.

While past MATs have focused primarily on building performance, this MAT was also tasked with gathering damage information needed to determine tornado ratings using the EF scale when possible (refer to Appendix E for more detail). Not all observed one- and two-family residences

were rated by the MAT. In some cases, ratings were not assessed due to limited accessibility that prevented thorough observations. The NSF-funded *Damage Study and Future Direction for Structural Design Following the Tuscaloosa Tornado of 2011* includes EF scale contour maps developed from extensive post-event DOD data collection and subsequent EF ratings (Prevatt et al. 2011b).

The EF scale assigns each damage indicator (DI) with specific damage description categories called degrees of damage (DOD) to estimate tornado wind speeds. Refer to Section 2.2 and Appendix E for additional information.

Photographs in this chapter that were taken from sites that were rated include the assigned DOD and EF rating. It is important to note, however, that engineering judgment was exercised when assigning the wind speeds that range between a specified lower and upper bound. In some cases, the observed DODs were considered to be inflated by poor construction practices or failure to adhere to the model building codes. Accordingly, wind speeds selected in such cases fall into the lower bound prescribed by the EF scale and may result in a lower EF rating by the MAT. Furthermore, images of a particular DOD may not always be the highest DOD observed at a particular site. In some cases, a photograph of a lower DOD is included in this report to better illustrate a specific failure mode. Figure captions will indicate when an EF rating provided for an image is inconsistent with the illustrated DOD.

This chapter is divided into two parts. The first describes observed damage to one- and two-family residences organized by the type of damage defined in eight DOD categories (2 through 9). The second describes damage to two multi-family residential complexes presented as detailed case studies.

4.1 One- and Two-Family Residences

The main purpose of presenting one- and two-family residential damage observations in the order of EF scale and DODs is to illustrate the order of progressive failures and the need to maintain continuous load path connections to mitigate high-wind damage. More specifically, DOD observations advance our understanding of the relationship between wind speeds and damages, and how certain damages may be greatly reduced or avoided altogether through enhanced design practices. The following section briefly describes the EF scale-prescribed damage for residential buildings and progressive damage observed by the MAT, and is followed by detailed descriptions of observed damage of residential buildings grouped by the following eight DODs.

- +Loss of roof covering and siding (DOD 2)
- +Glazing damage (DOD 3)
- +Uplift of roof decks (DOD 4)
- +Gable end walls: vulnerability related to uplift of roof deck (DOD 4)
- +Garage doors collapse inward (DOD 4)
- +House shifts off foundations (DOD 5)

■+Roof structure removed (DOD 6)

■+Collapse of framed walls (DOD 7–9)

Trigger mechanisms or vulnerable features that appeared to initiate the observed failure mode are described when applicable. Likewise, observed damage that is not explicitly listed as a DOD is included with the category most closely related to that failure mode.

4.1.1 EF Rating Evaluation of Residential Buildings

The MAT’s investigation of residential buildings and subsequent wind-speed determinations use the prescribed EF scale for “One- and Two-Family Residences between 1,000 and 5,000 square feet with typical wood framed construction” as outlined in *A Recommendation for an Enhanced Fujita Scale* (TTU 2006). One- and two-family residential structures are designated as DI 2 in the EF scale system and are accompanied by a specific list of DODs with which wind speeds can be estimated through observed damage. Based on a progression of damage from minimal visible damage to complete destruction, observed DODs specific to one- and two-family residences are shown in Table 4-1. A second DOD table for DI 5, which illustrates the progression of multi-family residential damages, is provided in Section 4.2.1.

Table 4-1: Degrees of Damage for One- and Two-Family Residences

DOD	Damage Description	Lower- and Upper-Bound Wind Speed Range (3-second gust in mph)	Expected Wind Speed (3-second gust in mph)
1	Threshold of visible damage	53–80	65
2	Loss of roof covering material (<20%), gutters, and/or awning; loss of vinyl or metal siding	63–97	79
3	Broken glass in doors and windows	79–114	96
4	Uplift of roof deck and loss of significant roof covering material (>20%); collapse of chimney; garage doors collapse inward; failure of porch or carport	81–116	97
5	Entire house shifts off foundation	103–141	121
6	Large sections of roof structure removed; most walls remain standing	104–142	122
7	Exterior walls collapsed	113–153	132
8	Most walls collapsed except small interior rooms	127–178	152
9	All walls collapsed	142–198	170
10	Destruction of engineered and/or well-constructed residence; slab swept clean	165–220	200

SOURCE: TTU 2006

Definitions:

DOD = degree of damage mph = miles per hour

4.1.2 Description of Progressive Damage for One- and Two-Family Residential Buildings

The first group of damages addressed—loss of roof covering and exterior siding, or DOD 2—typically precedes other phases. While nonstructural, damage to these elements can allow water intrusion which may weaken other systems and damage building contents. Damaged roof and wall covering elements may also become wind-borne debris that can cause building damage (Figure 4-1), injuries, and death.

Figure 4-1:
Wind-borne asphalt shingle penetrated the gypsum board on both sides of this interior wall at Chastain Manor Apartment Complex (Tuscaloosa, AL)



The second group of damages addressed—glazing damage and garage doors collapse inward, or DODs 3 and 4—often accelerate the disintegration of the structure through wind pressurization of the interior. Whether the result of wind-borne debris shattering glazing or wind pressure causing garage doors to collapse, a breach in the building envelope subjects it to increased pressurization and allows the intrusion of wind-driven rain. Other common building envelope vulnerabilities include, but are not limited to, soffits, doors, and gable end walls.

The third group of damages addressed—uplift to roof decks, included with DOD 4—begins with the uplift of roof decking and may coincide with, but frequently follows, breaching of the attic level building envelope. The loss of roof decking weakens the roof structure's ability to resist in-plane shear forces, and often results in the failure of the roof structure (DOD 6). Further contributing to the loss of the roof structure are failed roof-to-wall connections. When the roof structure is removed, lateral support (bracing) for the walls is lost. Collapse of exterior and interior walls constitute the later stages of overall structural failure (DOD 7–9), which typically progresses from the top down due to the loss of lateral support after the roof structure fails. This near-final phase of destruction is facilitated by the breakdown of connections between floors and walls, or by under-braced exterior walls that cannot resist in-plane shear forces.

4.1.3 Loss of Roof Covering and Exterior Siding (DOD 2)

The MAT observed widespread loss of roof covering and siding; this was evident on both lightly damaged residential buildings and those with more advanced stages of wind-induced damage. Nearly all observed roof coverings were asphalt shingles (Figure 4-2). Figures 4-3 and 4-4 illustrate exterior walls with sections of vinyl siding peeled off by high winds.



Figure 4-2:
Example of DOD 2 (loss
of asphalt shingles)
(Tuscaloosa, AL; photograph
courtesy of Tuscaloosa
County EMA)
[MAT EF Rating = 0]



Figure 4-3:
Example of DOD 2 (loss of
siding) (Tuscaloosa, AL;
photograph courtesy of
Tuscaloosa County EMA)
[MAT EF Rating = 0]

Figure 4-4:
Example of DOD 2 (loss of siding) through DOD 6 (large sections of roof structure removed) (Joplin, MO)
[MAT EF Rating = 2]

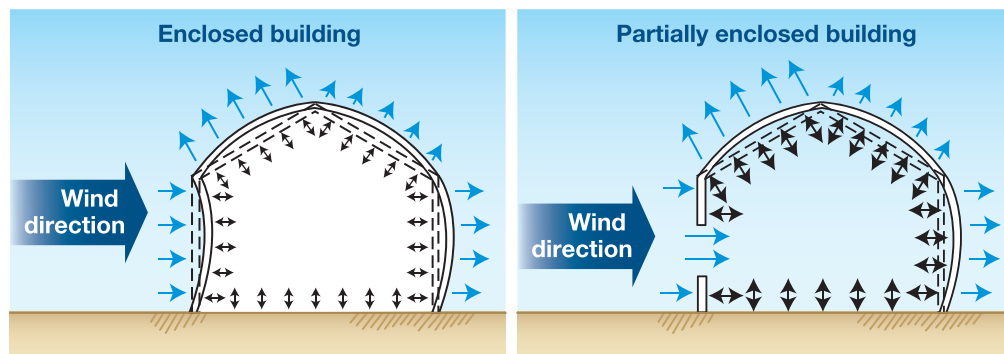


4.1.4 Glazing Damage (DOD 3)

The MAT frequently observed damage to window and door glazing in residential buildings. Most glazing types are extremely vulnerable to the wind-borne debris prevalent in tornadoes. Once the glazing is compromised, the building envelope is breached. This leads to increased pressurization of the interior, which increases stresses in structural components and connections between components that can, in some cases, initiate a chain reaction of structural failures in the building. Figure 4-5 illustrates the increased forces from pressurization on a partially enclosed building with a breached building envelope as compared to the enclosed building.

Figure 4-5:
Effect of wind on an enclosed building and a partially enclosed building

SOURCE: FEMA P-55, 2011



The MAT observed some buildings that benefitted from the installation of insulated glazing units (i.e., double-paned windows), where the outer pane was sacrificed but the inner pane remained intact, as illustrated in Figure 4-6. Energy code changes that require increased efficiency are leading to more double- and triple-paned glazing units in new residential construction. However, most windows of this type were not designed to provide extra protection from wind-borne debris and were breached on impact, as shown in Figure 4-7.



Figure 4-6:
Double-glazed window with outer pane sacrificed (remaining fragments are circled in red), leaving the inner glazing intact (Mercy Village, Joplin, MO); refer also to Section 4.2.3 for a case study of Mercy Village

Figure 4-7:
Example of DOD 3 showing
window with shattered
double-glazing (Harvest, AL)
[MAT EF Rating = 2]¹



4.1.5 Garage Doors Collapse Inward (DOD 4)

The MAT observed many failed overhead garage doors. Garage doors, particularly older double garage doors, are especially vulnerable to the effects of wind pressure. Older garage doors were not manufactured and rated to resist high winds. Wind pressure rated garage doors are now available, and may be code compliant while not meeting the wind pressure demands of some tornadoes.

Positive wind pressure against the doors can lead to inward deflection as shown in Figure 4-8. Garage doors can also fail under negative wind loads. In Figure 4-8 the wider double door (16 feet or 18 feet wide) incurs a greater resultant force under the same wind pressure than the adjacent single door (8 feet or 9 feet wide) because of its larger area. Therefore, the threshold for failure of the larger, similarly constructed double garage door is lower than that of the smaller single door. In addition to the actual garage door failing, the lifting and track hardware is vulnerable to failure under wind pressures too.

Residential buildings whose garage doors collapse often exhibit progressive collapse in and above the garage that exceeds the damage elsewhere because of increased pressurization when the garage door fails. Figures 4-9 and 4-10 illustrate this effect; note the extensive damage above the garage in both homes—the ceiling and roof assembly are completely blown off—compared to the opposite side of the buildings, where some of the ceiling and roof remain intact.

¹ More extensive building damage not apparent in this image resulted in a higher site DOD and EF scale rating.



Figure 4-8:
Example of DOD 4 showing a wide garage door collapsed inward, while narrow garage door to left is intact. Note also the wind-borne missile in roof above (Joplin, MO). [MAT EF Rating = 2]²



Figure 4-9:
Example of damage including loss of large sections of roof (DOD 6) apparently initiated from garage door failure (DOD 4) (Joplin, MO) [MAT EF Rating = 2]



Figure 4-10:
Example of how garage door failure (DOD 4) initiated progressive failure, including loss of the garage roof (DOD 6) (Joplin, MO) [MAT EF Rating = 2]

² More extensive building damage not apparent in this image resulted in a higher site DOD and EF scale rating.

4.1.6 Uplift of Roof Decks (DOD 4)

Many roof decks were observed to have separated from rafters or roof trusses. Most often, roof decking was in the form of 4-foot x 8-foot x ½-inch (nominal) OSB sheathing panels. In some older construction, nominal 1-inch x 8-inch planks were observed to comprise the roof deck.

When isolated areas of the roof were observed to be missing decks, as shown in Figure 4-11, the missing portions were often at corners, along roof overhangs, and along hips and/or ridges (Zones 2 and 3 as shown in Figure 4-12), where uplift pressures are greatest. Figure 4-13 shows a home where the roof decking separated above the eaves in an area where vinyl soffit material has been blown away, leading to increased pressures on adjacent roof decking. Nails between the decking and rafter or truss failed to resist uplift forces and allowed the decking to be pulled away from the structural framing. Roof decking above wide overhangs is particularly vulnerable to wind damage, as illustrated in Figure 4-14.

Figure 4-11:
Example of DOD 4 showing roof decking blown off along eaves and hip (Zone 2 in Figure 4-12), where uplift pressures are greater than in the field of the roof (Joplin, MO) [MAT EF Rating = 2]³



³ More extensive building damage not apparent in this image resulted in a higher site DOD and EF scale rating.

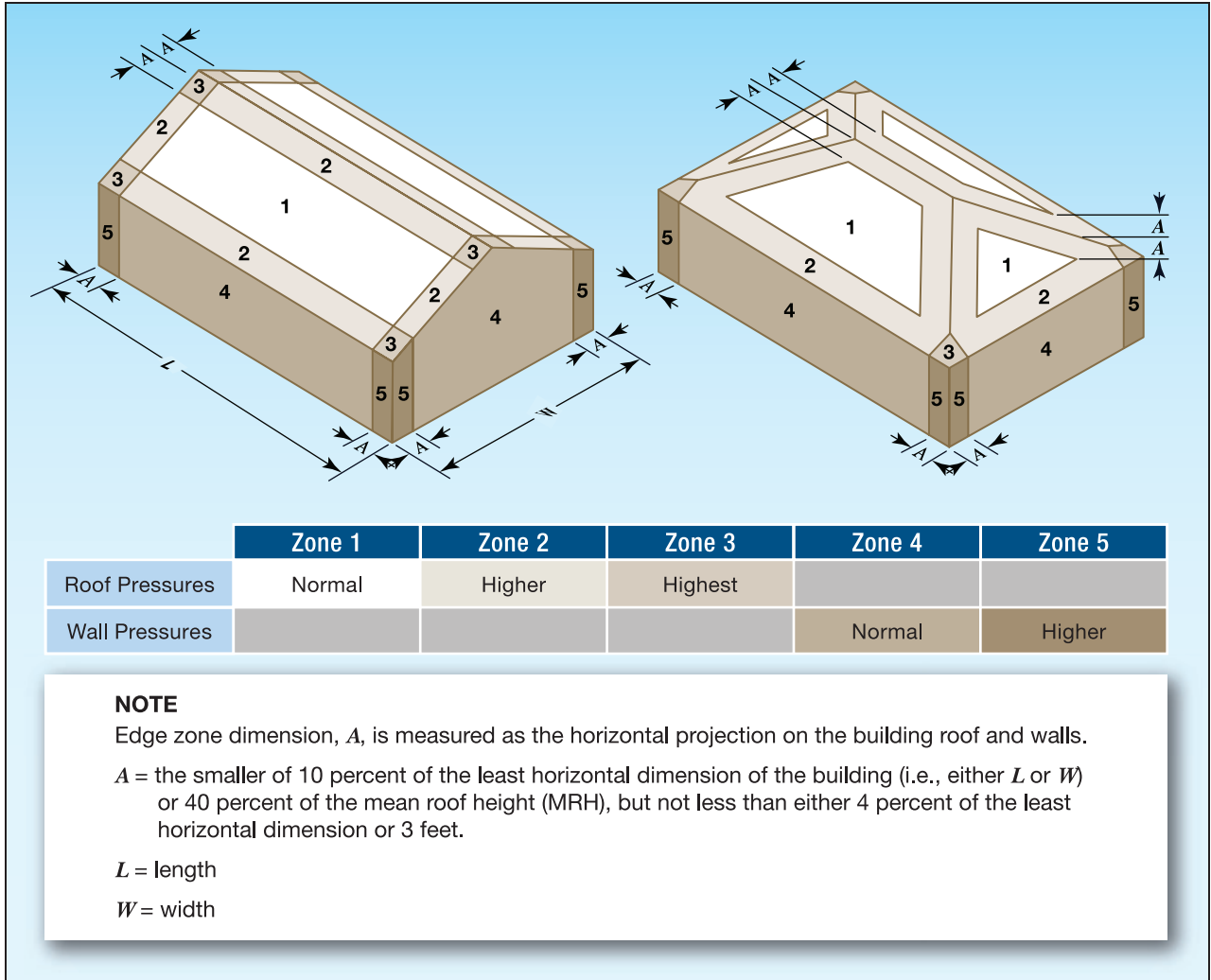


Figure 4-12: Component and cladding wind pressures

SOURCE: FIGURE 8-20 OF FEMA P-55 (2011A)

Figure 4-13:
Example of DOD 4 showing where roof decking lifted and blew off above damaged soffit in roof Zone 2 (see Figure 4-12) (Birmingham, AL) [MAT EF Rating = 2]⁴



Figure 4-14:
Example of DOD 4 showing roof decking removed at wide eaves on hip roof (Phil Campbell, AL) [MAT EF Rating = 1]



Poor construction methods can also decrease resistance to wind uplift. Figure 4-15 shows a residence where at least one of the roof deck nails along the panel edges failed to penetrate the rafter below. The expected performance is subsequently rendered weaker than intended by the building code or design professional. Improper spacing of fasteners and the use of improperly sized fasteners result in the same effect.

⁴ More extensive building damage not apparent in this image resulted in a higher site DOD and EF scale rating.



Figure 4-15:
Red circled roof deck nail not
connected to rafter (edge
shown with red dashed line)
(Harvest, AL)

4.1.7 Gable End Walls: Vulnerability Related to Uplift of Roof Deck (DOD 4)

The MAT observed gable end wall failure on many residential buildings with roof deck loss. While not specifically included in the DODs for One- and Two-Family Residences (Table 4-1), this failure mode compromises the building envelope. Once the attic level envelope is breached, increased pressurization can initiate or accelerate roof deck separation. Gable end walls that lack adequate bracing are susceptible to failure from wind pressures. In Figure 4-16, the roof area adjacent to the failed gable end wall lost more roof decking than the rest of the roof.

Figure 4-16:
Example of DOD 4 showing
damage to gable end of
simple gable roof
(Harvest, AL)
[MAT EF Rating = 1]



4.1.8 Entire House Shifts Off Foundation (DOD 5)

The MAT observed few instances of entire houses shifting off of their foundations. Framing-to-foundation connection failure was most often observed to follow wall collapse (DOD 7 through 9) and accordingly, those observations are included in Section 4.1.11. The older house depicted in Figure 4-17 was observed to have shifted off the raised pier and beam foundation. This house had no continuous exterior foundation walls, which provide more area for bottom plate anchorage than isolated piers. Further contributing to the observed failure of the bottom plate-to-foundation anchorage was the lack of bracing or connectivity between the top of the exterior piers.

Figure 4-17:
Example of DOD 5 showing
residential building shifted
off masonry piers
(Cullman, AL)
[MAT EF Rating = 2]



4.1.9 Roof Structure Removed (DOD 6)

When roof decking resists uplift pressure and transfers those forces through fasteners to rafters or trusses, that load must be transferred from the rafter or truss to the framed wall below and from there to the foundation in a continuous load path. Failure to transfer uplift through roof-to-wall connectors results in the loss of the roof structure, as shown in Figure 4-18.

The “birds-mouth” notched rafters spanning from hip to wall in Figure 4-19 separated from the plate and outlookers, and shifted toward the building corner. They were framed onto a single plate across the top of the joists below, a configuration that, while common, is not prescribed in the 2009 IRC, and requires special attention in the application of roof-to-wall connectors and roof-to-ceiling tie-backs. The rafter shown in Figure 4-20 was found nearby and observed to have two small toe nails withdrawn from the plate. Even with proper nailing, the rafter shown in Figure 4-20 would likely have become the next weak link at this location by failing under stress because of improper or non-code-compliant notching. Cutting and notching limitations for sawn lumber rafters are found in Section R802.1.7 of the 2009 and 2012 IRC.



Figure 4-18:
Example of DOD 6 showing failed roof-to-wall connections that resulted in loss of roof structure (Harvest, AL)
[MAT EF Rating = 2]



Figure 4-19:
Example of roof-to-wall connection failure facilitated by non-code-compliant construction: notched rafters, known as “bird’s mouth,” (red circle) displaced when roof-to-wall connections failed; red arrow indicates rafter bearing plate, yellow arrow indicates outlooker (Harvest, AL)
[MAT EF Rating = 1]

Figure 4-20:
Over-notched rafter found
on ground with two toe nails
withdrawn from plate (red
circle) (Harvest, AL)
[MAT EF Rating = 1]



Metal connectors designed to transfer uplift forces from the rafter or truss to the wall below greatly enhance connectivity and were observed to outperform toe nail-only connections. In order to transfer uplift and lateral loads consistent with their maximum design capacities, metal connectors must be installed per the manufacturer's instructions.

Applications of metal connectors not in accordance with manufacturers' installation instructions, including insufficient nailing and using the wrong nail size, can lead to the connection not performing to design capacity. For example, a 6d box nail as shown in Figure 4-21 has a withdrawal capacity of 96 pounds when face-nailed into a Southern Yellow Pine #2 double 2x4 top plate, as compared with 217 pounds for a 10d common nail.

Spacing of roof-to-wall connectors is also critical to the performance of roof-to-wall connections. The house shown in Figure 4-22 had a roof-to-wall connector on the indicated roof truss, but not on the adjacent one. While it used to be typical for designers to specify rafter-to-wall connectors at every other or every third rafter to meet the design requirements of basic design wind speeds, the greater loads exerted during tornadoes can render this minimal design-level connector schedule ineffective, even with correctly installed hardware.



Figure 4-21:
Example of DOD 6 showing trusses were connected to walls with small hurricane ties. Red circle indicates area of inset photograph. Inset shows gauge indicating undersized 6d box nail remaining in roof-to-wall connector; the yellow circle indicates the appropriate nail size of 8d. (Phil Campbell, AL). [MAT EF Rating = 2]



Figure 4-22:
Example of DOD 6 showing insufficient connection of single roof-to-wall connector on remaining chord of roof truss at left (red circle) and none on truss at right (Tuscaloosa, AL) [MAT EF Rating = 2]

4.1.10 Collapse of Framed Walls (DOD 6–9)

When roof and ceiling or roof truss-to-wall connections fail and leave the top of the framed wall unsupported, walls become especially vulnerable to collapse. Therefore, the roof/ceiling or floor connection to the top and bottom of the framed wall is critical to maintain stability and prevent wall collapse. Figure 4-23 shows a home where the roof system was blown off, removing the lateral support for the top of the wall and allowing it to be blown in.

Figure 4-23:
Example of DOD 6 showing failure of roof framing that resulted in loss of lateral support for the top of this wall (red arrow) (Phil Campbell, AL)
[MAT EF Rating = 2]



The floor system above the garage shown in Figure 4-24 separated from the framed exterior walls, allowing both the floor and walls to collapse. As shown in the inset to Figure 4-24, the double top plate of the garage entry wall was pulled away either with the fallen floor system or collapsed portion of the garage entry wall. The deeper floor system above the garage, which was installed to span the garage without intermediate support, appeared to be framed onto a lower top plate that interrupted the continuity of the top plate and weakened the connection to the adjacent walls.

Walls with inadequate bracing were observed to be especially vulnerable to collapse under in-plane shear forces. Garage entry walls, like that shown in Figure 4-25, often have a small percentage of full-height sheathed lengths with respect to the overall wall length and often collapse before other exterior walls collapse. Any exterior wall that lacks code-compliant (2009 and 2012 IBC R602.10) lengths of full height solid sheathed (or alternatively braced) sections is susceptible to failure from in-plane shear.

Continuous load paths can be improved by extending the continuous wood panel wall sheathing across the floor system and bottom plate and/or by using metal straps to connect the wall to floor and floor to sill. At the second floor band, extend wall sheathing from upper and lower walls to meet at the band midpoint. Proprietary wall hold-down hardware (described in Appendix G, Section G3.3) is another effective attachment option.



Figure 4-24: Example of DOD 6 showing pressurization of this garage from failure of the garage door/wall removed the support for the second story floor system (red circle). Inset shows where top plate of garage entry wall separated with floor or wall (red circles). Note former location of entry wall top plate—separated and missing from top of studs—is lower than the interior wall top plate due to deeper floor system (Harvest, AL). [MAT EF Rating = 2]



Figure 4-25: Example of DOD 6 where most walls remained standing, but under-braced garage entry wall failed (Joplin, MO) [MAT EF Rating = 2]

Under-braced framed wall collapse was not exclusive to walls with overhead garage doors, however. The MAT also observed openings in framed walls that had large windows or entry doors. Figure 4-26 shows what appears to have been a sunroom. In this instance, wall collapse may have been further enabled by a weak connection between the wall, raised floor system, and bottom plate. Often the bottom plate of the wall is merely nailed to the raised floor system.

Figure 4-26:
Example of DOD 6 showing under-braced framed sunroom wall failure. Note long window bottom on right (red arrow) and the failure of nailed wall-to-floor connection on left wall (yellow arrow) (Harvest, AL). [MAT EF Rating = 2]



4.1.11 Wall Framing-to-Foundation Connection Failure: Damage Related to Collapse of Framed Walls (DOD 7–9)

As noted in Section 4.1.8, the MAT observed that failures of framing-to-foundation connections often followed wall collapse. Examples of failed connections included bottom plates of framed walls attached directly to stem walls and slabs. Failure of foundation anchorage was observed along the exterior stem walls of garage slabs in newer houses where walls were framed atop CMU, as shown in Figures 4-27 and 4-28. The homes shown in Figures 4-27 and 4-28 are located in the same community. Figure 4-27 shows the top of the garage stem wall with the wall bottom plate missing. In Figure 4-28, the bottom plate of the wall remained connected to the top row of CMUs, but the top row of CMU separated from the foundation wall below because there was no reinforcement or other tension connection within the CMU wall. Furthermore, the MAT observed the absence of grout in the cells of the damaged CMU walls in both of these homes, including locations where anchor bolts should have been installed.



Figure 4-27:
Example of wall framing-to-foundation connection failure. Wall and bottom plate separated from foundation where anchorage of collapsed framed wall failed because anchors lacked embedment in grout. Note CMU wall with no reinforcement or solid grout (Harvest, AL).
[MAT EF Rating = 2]



Figure 4-28:
Example of wall framing-to-foundation connection failure. Bottom plate remains connected to top row of CMU, but CMU wall failed due to lack of reinforcement for continuous load path (Harvest, AL).
[MAT EF Rating = 2]

Other bottom plate-to-slab foundation connection failures were observed where bottom plates were attached to the concrete slab foundation using only concrete nails (often called cut nails), as shown in Figures 4-29 and 4-30. The illustrated wall-to-slab failure is typical in that either the plate was separated from the slab by lifting around the nails (nails remained embedded), like in Figure 4-29, or the nails pulled out of the concrete with the plate leaving behind small cones of missing concrete, as shown in Figure 4-30. This damage was rated EF3 due to missing walls, but the damage may have occurred in part due to poor connections rather than solely to high winds.

Proper installation of **foundation anchorage** is critical for a foundation to perform as intended under design conditions.

Non-code-compliant foundation attachments were observed to fail prematurely when exposed to high winds.

Although the residence in Figures 4-29 and 4-30 appears to be older construction, the MAT observed recently constructed dwellings were also observed to have driven nails used to attach bottom plates to masonry or poured concrete foundations instead of IRC-required anchor bolts. Figure 4-31 shows a newly constructed residential building in Tuscaloosa, AL (completed December 2010) where the bottom plates in some areas had been secured with only concrete cut nails. Concrete nails provide significantly less resistance to uplift and lateral forces than similarly spaced ½-inch-diameter anchor bolts with 7 inches of minimum embedment.

Figure 4-29:
Wall-to-foundation connection failure where concrete nails remained in stained concrete slab and bottom plate (missing) pulled over the heads of the nails (red circles) (Hackleburg, AL) [MAT EF Rating = 3]





Figure 4-30:
Wall-to-foundation failure where bottom plate and concrete nails were pulled out by high winds. Shallowly embedded concrete nails pulled small cones of concrete up with bottom plate (red circles) (Hackleburg, AL).
[MAT EF Rating = 3]



Figure 4-31:
Wall-to-foundation connection failure. Note slab failure along right edge where the bottom plate separated and the nail was removed (red arrow). A remaining cut nail is indicated by the red circle. (Tuscaloosa, AL).
[MAT EF Rating = 4]

4.2 Multi-Family Residences

The MAT visited only a few multi-family residences in the post-event investigations. The MAT observations at Chastain Manor in Tuscaloosa, AL, and at Mercy Village Apartments in Joplin, MO, are presented below as case studies in building performance of multi-family residential buildings. Some of the buildings at Chastain Manor were in the direct path of a powerful tornado—rated by the NWS as an EF4 in this vicinity—and suffered significant damage. While a direct hit from an EF4 tornado is rare, the observations from Chastain Manor illustrate the value of on-site safe rooms and storm shelters and comprehensive emergency operations planning, particularly for residential dwellings in tornado-prone regions. Conversely, Mercy Village did not take a direct hit and incurred fewer damages by comparison. Furthermore, the MAT observed that damage at Mercy Village seemed to be less severe than surrounding buildings and consequently reviewed the construction drawings after the site visit. The following sections discuss the MAT’s findings.

4.2.1 EF Rating Evaluation of Multi-Family Residential Buildings

Table 4-2 shows the DODs for multi-family residences (DI 5) and their respective wind speeds.

Table 4-2: Degrees of Damage for Multi-Family Residences

DOD	Damage Description	Lower- and Upper-Bound Wind Speed Range (3-second gust in mph)	Expected Wind Speed (3-second gust in mph)
1	Threshold of visible damage	63–95	76
2	Loss of roof covering (<20%)	82–121	99
3	Loss of roof decking; significant loss of roof covering (>20%)	107–146	124
4	Uplift or collapse of roof structure leaving most walls standing	120–158	138
5	Most top story walls collapsed	138–184	158
6	Almost total destruction of top two stories	155–205	180

SOURCE: TTU 2006

Definitions:

DOD = degree of damage mph = miles per hour

4.2.2 Chastain Manor Apartments (Tuscaloosa, AL)

Location of Facility in Tornado Path: The MAT observed Chastain Manor, a senior living community in northeastern Tuscaloosa, AL. Figure 4-32 shows an aerial view of Chastain Manor after the tornado. The NWS rated the center of the tornado circulation in the vicinity of the Chastain Manor buildings as an EF4. According to the property developer,⁵ approximately 22 of the 25 leased units were occupied when the tornado struck; there were two reported fatalities. The apartment community had only opened in December 2010, and fewer than half of the available units were leased at the time of the tornado strike.

⁵ TBG Residential, 3825 Paces Walk, SE, Suite 100, Atlanta, GA 30339

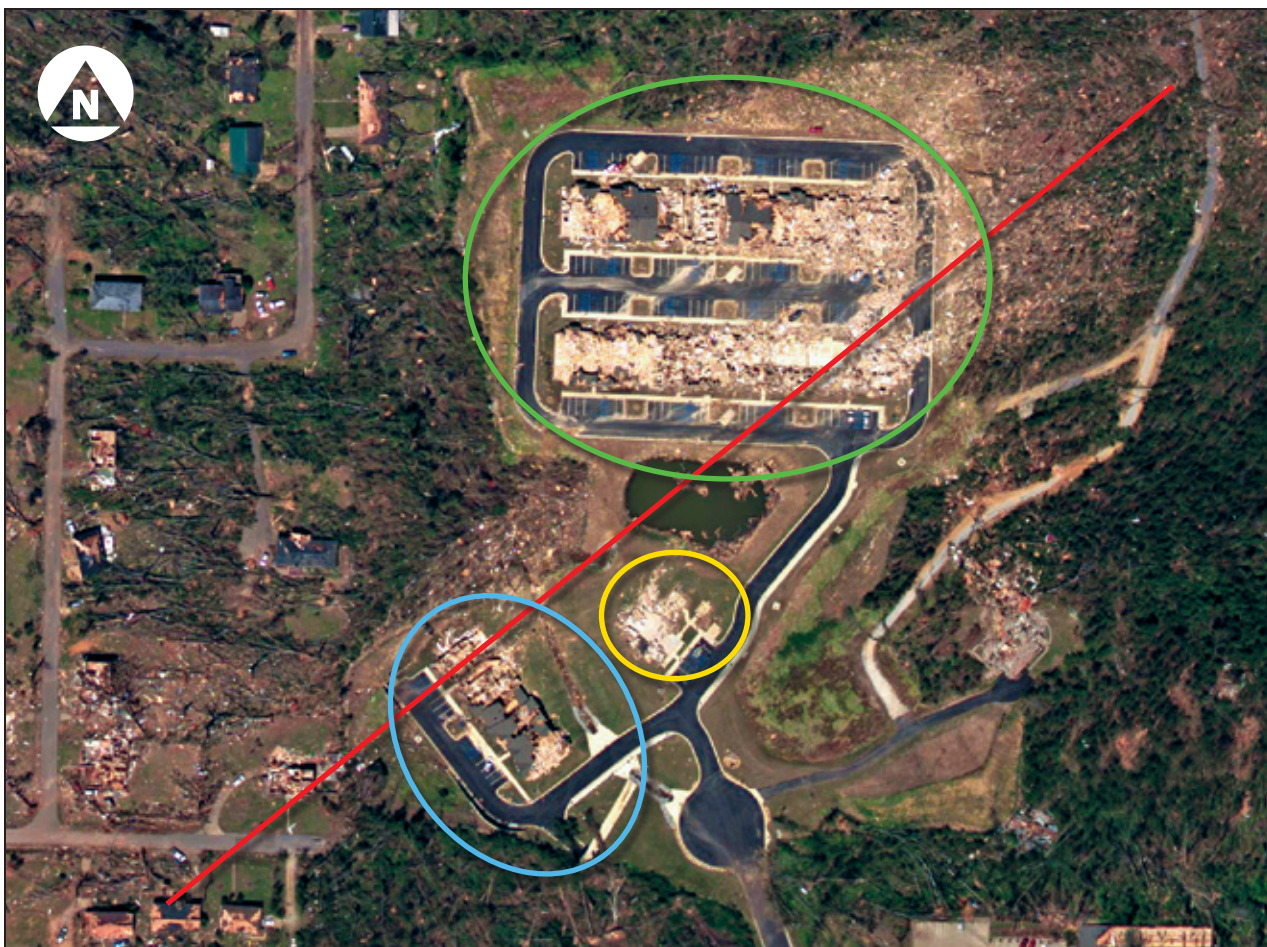


Figure 4-32: Aerial view of Chastain Manor Apartments in relationship to the approximate centerline of the tornado damage swath (red line).⁶ The green circle indicates the two-story apartment buildings, the yellow circle shows the location of the leasing office, and the blue circle shows the location of a one-story apartment building (Tuscaloosa, AL). SOURCE: ALL AERIAL PHOTOGRAPHS ARE FROM NOAA IMAGERY ([HTTP://NGS.WOC.NOAA.GOV/STORMS](http://ngs.woc.noaa.gov/storms)) UNLESS OTHERWISE NOTED

Facility Description: Chastain Manor is a 56-unit senior apartment home community that opened in December 2010. Unlike Mercy Village, the MAT did not have access to construction drawings for Chastain Manor. The complex is divided into two sets of dwellings, including a set of one-story units and a set of two-story units. A small one-story leasing office foundation was situated between the two sets of dwellings.

The single row of connected one-story units was on the property's higher ground, with shared, open entranceways between units. Basic construction consisted of pre-engineered wood roof trusses that spanned from front to back. Main roof trusses were supported by girder trusses at each end in some areas and by exterior bearing walls in others. All roof trusses were attached with hurricane framing connectors where supported by framed walls. Single-story walls—mostly non-load-bearing because

⁶ 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.

of the girder trusses described above—were framed with 2x4 studs at 16 inches on center atop slab-on-grade foundations. Exterior walls were sheathed with 7/16-inch wood structural panels. The units had porch columns that the MAT observed lacked any positive connection to the slab below.

The two rows of two-story apartment buildings on the lower-lying terrain had shared, open entranceways and stairs between the units. Basic construction was similar to the one-story units with respect to roof framing and framed walls, but the foundations differed somewhat. Although the two-story building foundations were slab-on-grade (similar to the one-story building), some units were separated by masonry retaining walls necessitated by grade changes, so that slabs separated by the retaining wall were at different elevations. Exterior porches of the two-story units were constructed with suspended concrete slab floors in the upper units supported by steel beams on 4¼-inch (outside) diameter standard steel pipe columns. Each observed column (upper and lower) was originally attached to the concrete slab with four ½-inch-diameter expansion bolts through ½-inch-thick steel base plates. The embedment depth, while modest at approximately 2 inches, was more substantial than the one-story unit's porch columns.

General Wind Damage: The wind damage observed by the MAT varied significantly across Chastain Manor despite the similarity in the layout of the units (within each set of buildings), materials used, and construction method.

In the one-story units, observed damage ranged from uplift of roof decking (DOD 3) for a few connected units at one end of the building to uplift or collapse of roof structure with most walls standing (DOD 4) for the remainder of the building (Figure 4-33). Where they remained intact, the unanchored porch columns in the one-story units were rotated and/or out of plumb (Figure 4-34).

As indicated by the green circle in Figure 4-32, the units at the northeast end of the two-story apartment buildings were bisected by the center of the tornado and sustained the greatest damage. For the selected DI 5, observed damage to the two-story apartment buildings varied from uplift of roof decking (DOD 3) as shown on several units in Figure 4-35 to complete destruction (DOD 6) as shown by the slab swept clean (Figure 4-36). Despite the enhanced column connection in the two-story units (shown in Figure 4-35), some of the porches were destroyed by the tornado, and two columns were found embedded in the adjacent hillside (Figure 4-37).

Figure 4-33:
One-story Chastain Manor Apartments suffered damage varying from roof decking uplift to collapse of roof structure





Figure 4-34:
One-story Chastain Manor
Apartment unanchored porch
column that rotated at top
and bottom of column (red
circles)



Figure 4-35:
Example of DODs 3 and 4
showing two-story Chastain
Manor Apartments with
varying roof damage. Note
upper and lower steel pipe
porch columns (red circles).

Figure 4-36:
Example of DOD 6 showing two-story Chastain Manor Apartments completely destroyed by the tornado with slabs swept clean



Figure 4-37:
Two-story Chastain Manor Apartment steel porch column (shown in-place in Figure 4-34) was blown away and embedded in neighboring hillside



Bottom plates in the leasing office (Figure 4-38) and two-story apartment buildings (Figure 4-36) were removed from the thickened slab foundation in multiple locations. In some areas, the bottom plates were stripped away from the foundation, leaving the anchor bolts embedded in the foundation with washers still attached (Figure 4-39). The washers used between the anchor nut and plate were 1 inch in diameter. While the washer size for the bottom plate anchor is not specified in Section 2308.6 of the 2012 IBC, high-wind areas along the coast are required to use 3-inch-square washers, which significantly increase resistance to plate uplift.



Figure 4-38:
Most of the bottom plates
were blown off the Chastain
Manor leasing office



Figure 4-39:
Two-story Chastain Manor
Apartment slab with bottom
plate pulled over remaining
anchor bolt and 1-inch
diameter washer; red circles
indicate where bolt and plate
were connected prior to
damage

MAT EF Rating: Using DI 5 (Apartments, Townhouses, and Condos), the MAT selected DOD 6 for the two-story units at Chastain Manor, which were the most damaged at the site. Applying the expected wind speed range for DOD 6 (155–205 mph), the MAT derived the tornado intensity as EF4 (166–199 mph) based on the observed building damage for the two-story units. Hence, the estimated wind speed experienced by the building greatly exceeded the basic design wind speed of 90 mph.

4.2.3 Mercy Village Apartments (Joplin, MO)

Location of Facility in Tornado Path: The MAT visited the Mercy Village Apartments located in Joplin, MO. Figure 4-40 shows an aerial view of the apartments prior to the tornado, and Figure 4-41 shows an aerial view of the apartments after the tornado. The Mercy Village Apartment building was approximately 1,100 feet from the center of the tornado damage swath, rated by the NWS as EF5 in this location. According to Mercy Housing, Inc. management,⁷ approximately 60 out of the total 70 apartment residents were at home when the tornado passed by. There were no reported fatalities.

Facility Description: Mercy Village Apartments, a 66-unit retirement community on the campus of St. John's Regional Medical Center, is a three-story, wood-frame apartment building. The construction drawings were produced in 2003, so the apartment building is approximately 7 or 8 years old. The drawings indicate the building was engineered to the requirements of the 2000 IBC.

Figure 4-40:
Aerial photograph showing
Mercy Village (red circle)
prior to tornado

SOURCE: © GOOGLE EARTH



⁷ Jennifer Erixon, Mercy Housing Inc., 1999 Broadway, Suite 1000, Denver, CO 80202.



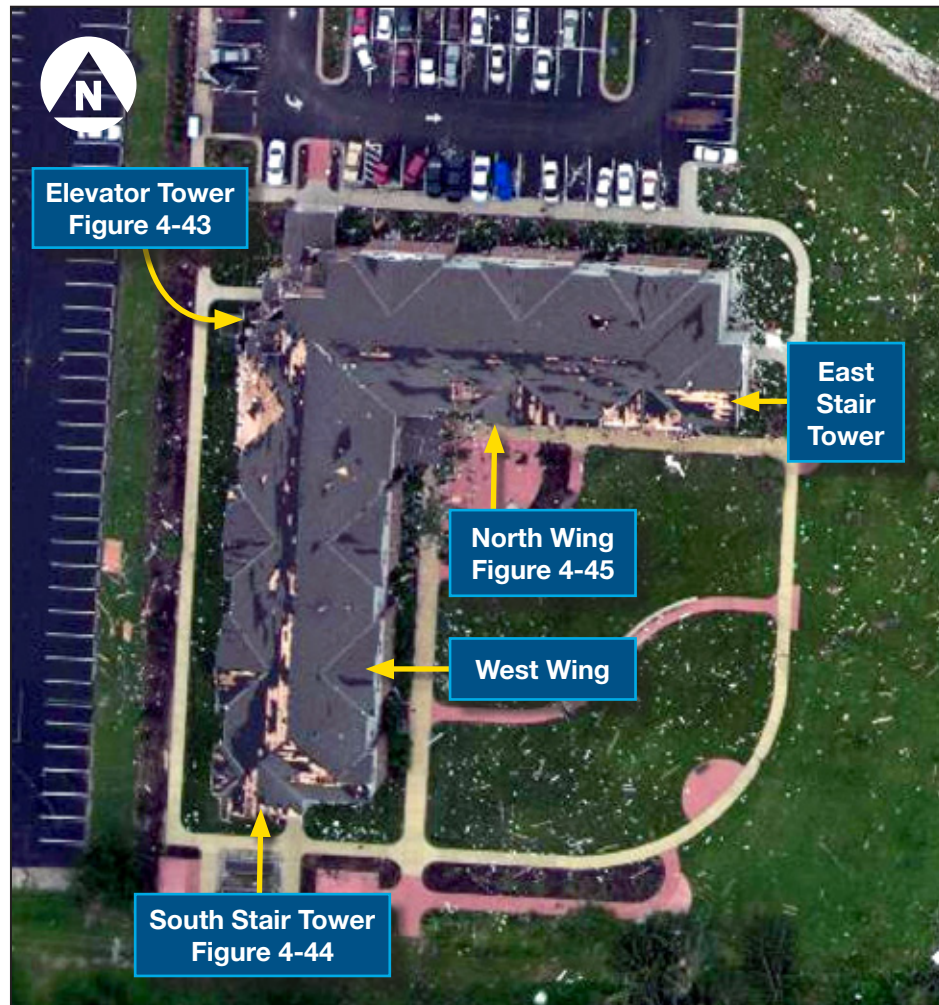
Figure 4-41:
Aerial photograph of Mercy Village (red circle) after the tornado; red line indicates the approximate centerline of the tornado damage swath (Joplin, MO)

The layout of the building is “L” shaped with a central core on the northwest corner and wings extending in perpendicular directions. One wing is oriented north-south and the other east-west. The exterior of the building is a combination of brick veneer and fiber cement board siding. Basic construction consists of pre-engineered wood roof trusses at 24 inches on center that are attached to framed walls at each end with metal hurricane framing (roof-to-wall) connectors. The bottom chords of gable end trusses were observed to be braced at 48 inches on center. Braces extend 9 feet back from end walls and are attached to blocking between the trusses. As configured, the described bracing was designed to resist potential inward or outward deflection of the gable end truss bottom chords and served to protect the integrity of the roof envelope.

Elevated floors between units are framed with 18-inch-deep pre-engineered wood floor trusses spaced at 16 inches on center and spanning from exterior walls to the center corridors. Floor framing between corridors is 2x10 floor joists at 16 inches on center. Exterior walls were observed to be framed with 2x4 studs at 16 inches on center and sheathed with 7/16-inch wood structural panels. Interior walls are similarly framed, but sheathed with 5/8-inch thick gypsum board. Designated interior shear walls are reinforced and attached to adjoining structural elements with proprietary hold-down hardware and metal straps. One side of the transverse shear walls has a resilient channel against the stud for sound attenuation, which negated approximately half of the design-intended shear wall capacity. Exterior walls were observed to be attached to the foundation with anchor bolts.

General Wind Damage: The MAT observed wind damage to Mercy Village Apartments that included damage to siding and brick veneer, structural damage to stair towers and dormers, glazing damage, roof covering damage, and roof decking damage. Since most of the exterior envelope remained functional, the damage to the interior spaces was limited to areas where the exterior envelope was compromised. Units where glazing was destroyed suffered damage. The damage to the corridors was primarily from water infiltration. Figure 4-42 shows the locations of the damaged areas at the Mercy Village Apartments.

Figure 4-42:
Aerial photograph of Mercy Village after tornado showing building areas identified in “General Wind Damage” (Joplin, MO)



Sections of brick veneer fell off the elevator tower and caused damage to the single-story roof sections around the main entrance, maintenance room, and bathrooms, as shown in Figure 4-43. Aside from the elevator tower, damage to the brick veneer was minimal. The observed brick veneer damage was likely caused by debris impact and did not appear to indicate building movement relative to the foundation (or any significant movement of the building below the third floor). The brick veneer was attached with adjustable wall ties at 16 inches on center vertically and 32 inches on center horizontally as specified.

A section of wall between the third floor and roof was blown away at both stair towers (located at the each end of the building). Additionally, the gable end walls at both tower ends were either missing or suffered extensive damage, as shown in the south stair tower in Figure 4-44. A short section of wall that extended from the end of the stair tower back to the building was still present at each end, but was unsecured and no longer vertically plumb. The stair stringers and landings separated slightly from the perimeter wall above the second floor, and the end wall bowed outward at both stair towers. Similarly, the gable face of the easternmost dormer on the south wall of the east wing was missing. Much of the dormer roof decking was also blown off.



Figure 4-43:
West wall and elevator tower where brick fell off and damaged one-story roof (red circle)



Figure 4-44:
Damage to the gable end wall at south stair tower end. Note third floor wall is missing and second floor wall is bowed outward (red arrow).

The gable end wall of the east stair tower (not shown in Figure 4-44) was damaged, but remained intact, due in part to the truss bracing described in the previous section (Facilities Description). The truss braces on the east stair tower appeared to have held the building together long enough to prevent more extensive roof damage in that area.

Nearly all of the glazing on the north wing was damaged by wind-borne debris or wind pressure, as shown in Figure 4-45, and about half the windows on the west wing had damaged glazing. In many of the rooms where glazing was damaged on the third floor, especially on the north wing, the roof appeared to have lifted slightly off the walls and been set back down. The MAT did not observe any horizontal displacement. The MAT inferred the lifting movement from cracks observed between the interior walls and the ceiling.

Sections of roof decking were missing in various locations at the building, but the vast majority of the building's roof decking remained intact. There were some locations where 2x4 nail plates (a.k.a. "sleepers") were installed between the top chord of the trusses and the sheathing to address truss misalignment that occurred during construction. In these locations the nail plates had pulled away from the top chord of the trusses, as shown in Figure 4-46 and the inset, because of decreased nail penetration in the truss chord. Although the roof decking remained tight against the nail plates, there was inadequate nail capacity between the nail plates and truss top chords to resist the required tributary area uplift. Where the decking was directly attached to the trusses, it remained tight and secure.

MAT EF Rating: Using DI 5 (Apartments, Townhouses, and Condos), the MAT selected DOD 3 for this facility. Using the expected wind speed range for DOD 3 (107–146 mph), the MAT derived the tornado rating as EF2 (110–137 mph) based on damage to the building. Hence, the estimated wind speed experienced by the building was above the basic wind speed of 90 mph that the building was

Figure 4-45:
Damaged glazing on the interior courtyard of north wing building; majority of glazing was damaged





Figure 4-46: Nails through roof sheathing and nail plates have separated from roof trusses (red circles) because of wind pressures. Inset shows close-up of 2-inch nail withdrawal where nail plates separated from the roof truss.

designed to withstand. The NWS rated the core of the track in the vicinity of Mercy Village as an EF5, which is above the MAT EF2 rating for this building. Mercy Village was approximately 1,100 feet away from the centerline of the tornado; accordingly, wind speed decay would result in a lower speed at the facility.

4.3 Summary of Conclusions and Recommendations

Table 4-3 provides a summary of the conclusions and recommendations for Chapter 4, *Observations on Residential Building Performance*, and provides section references for supporting observations. Additional commentary on the conclusions and recommendations is presented in Chapters 10 and 11.

Table 4-3: Summary of Conclusions and Recommendations for Residential Building Performance

Observation	Conclusion (numbered according to Chapter 11)	Recommendation (numbered according to Chapter 12)
<p>Examples of recent non-compliant (IRC) construction:</p> <ul style="list-style-type: none"> • Over-notched rafters lacking connection to floor diaphragm: Figures 4-19, 4-20 (Section 4.1.9) • Discontinuous top plate: Figure 4.24 (Section 4.1.10) • UngROUTED CMU below missing bottom plate anchors: Figure 4-27 (Section 4.1.11) • Bottom plate attachment with cut nails: Figure 4.31 (Section 4.1.11) 	<p>Conclusion #1 Failure to adopt a current version of code or having no uniform code leaves residential buildings vulnerable to wind damage. At the time of publication of this report, current codes are the 2012 or 2009 IRC.</p>	<p>Recommendation #1 Adopt and enforce current model building codes. At the time of publication of this report, current codes are the 2012 or 2009 IRC.</p>
<p>Examples of non-compliant (IRC) bottom plate attachment:</p> <ul style="list-style-type: none"> • Figure 4-31 shows a newly constructed residential building in Tuscaloosa, AL (completed December 2010) where the bottom plates in some areas had been secured with only concrete cut nails. (Section 4.1.11) • Additional examples of IRC exceptions to bottom plate attachment in Figures 10-1 and 10-2 	<p>Conclusion #2 Failure to adhere to the structural provisions of the model building code as written can result in buildings that are vulnerable to structural damage.</p>	<p>Recommendation #2 Increase emphasis on code compliance.</p> <p>Recommendation #3 Maintain and rigorously enforce the adopted model building code since amendments or lax enforcement practices may weaken the continuous load path of the building.</p>
<p>Examples subcategorized by specific failure modes in following rows</p>	<p>Conclusion #9 Voluntary implementation of better design and construction practices could mitigate damage.</p> <p>Improved design and construction and implementation of details and techniques that are already required in coastal high-wind regions will significantly reduce property damage caused by tornadoes rated EF2 or less (i.e., estimated wind speeds of 135 mph or less) and to buildings located at the periphery of more severe events.</p>	<p>Recommendation #15 Implement voluntary best practices to mitigate damage to one- and two-family residential buildings.</p> <p>Prescriptive guidance is provided in Appendix G to enhance performance of components, cladding, and critical load path connections observed to have failed during the spring 2011 tornado events. The prescriptive guidance is intended to improve building performance as described in the following rows.</p>

Table 4-3: Summary of Conclusions and Recommendations for Residential Building Performance (continued)

Observation	Conclusion (numbered according to Chapter 11)	Recommendation (numbered according to Chapter 12)
<p>Examples of roof or wall covering that became wind-borne debris endangering surrounding buildings and their occupants as shown in Figure 4-1 (Section 4.1.2)</p> <p>Examples of damage that wall or roof covering that likely led to water intrusion in Figures 4.2 and 4.3 (Section 4.1.3)</p> <p>Examples of unprotected glazing and wide garage doors (16 or 18 feet wide): Figure 4-6, 4-7, 4-8 (Section 4.1.4 and 4.1.5)</p> <p>Examples of increased damages resulting from breaching of the building envelope: Figures 4-9 and 4-10 (Section 4.1.5)</p> <p>Examples of damages that appeared to be triggered by increased pressurization resulting from damaged soffits and gable end walls: Figures 4-13 and 4-16 (Section 4.1.6)</p> <p>Example of poorly fastened roof deck to roof structure that appeared to play a role in the loss of roof decking: Figure 4-15 (Section 4.1.6)</p> <p>Examples of failed roof to wall connections are shown in Figures 4-18 through 4-22 (Section 4.1.9)</p>	<ul style="list-style-type: none"> • Loss of Roof and Wall Covering: Roof and wall covering blown away by high winds and uplift forces became wind-borne debris that endangered surrounding buildings and their occupants. Buildings that suffered roof covering loss were often further damaged by water intrusion. • Component Damage: Component damage, whether shattered glazing or collapsed garage doors, often led to other structural and non-structural damage because of increased pressurization and water intrusion that followed breaching of the building envelope. Unprotected glazing and wide garage doors (16 or 18 feet wide) were particularly vulnerable as was expected from previous MAT assessments. • Uplift of Roof Decking: Loss of roof decking often appeared to be triggered by increased pressurization resulting from damaged soffits, window failures, and gable end walls. Poor fastening of roof decking to the roof structure also appeared to play a role in the loss of roof decking. • Loss of Roof Structure: The weak link most often identified as responsible for loss of roof structure was the roof-to-wall connection. 	<ul style="list-style-type: none"> • Improve roof and wall coverings per Section G.3.1. • Increase awareness of glazing damage and strengthen garage doors per Section G.3.1. • Strengthen roof decking (sheathing) attachment per Section G.3.2. • Strengthen roof-to-wall connections per Section G.3.2.

Table 4-3: Summary of Conclusions and Recommendations for Residential Building Performance (concluded)

Observation	Conclusion (numbered according to Chapter 11)	Recommendation (numbered according to Chapter 12)
<p>Examples of wall collapse observed to result from failed attachment of floor and ceiling systems to walls: Figures 4-23 and 4-24 (Section 4.1.10)</p> <p>Examples of wall collapse observed to result from inadequate bracing of framed walls: Figures 4-25 and 4-26 (Section 4.1.10)</p> <p>Examples of failure of wall connection to foundation: Figures 4-27 through 4-31 (Section 4.1.11)</p>	<ul style="list-style-type: none"> • Wall Collapse: Wall collapse was observed to result from failed attachment of floor and ceiling systems to walls and inadequate bracing of framed walls. • Failure of Wall Bottom Plate Attachment: Foundations typically performed adequately, but in some instances the connection of walls to the foundations system failed because of inadequate connection of the bottom plate. 	<ul style="list-style-type: none"> • Improve wall performance through sheathing attachment, hold-down installation and better top plate splicing per Section G.3.3. • Improve wall-to-floor connections and bottom plate attachment per Section G.3.3.
<p>Example of the exceptional case where DOD 5 preceded DOD 6: Figure 4-17 shows example of house shifting off of foundation prior to loss of roof structure and wall collapse (Section 4.1.9).</p>	<p>Conclusion #43 Order of DOD choices for DI 2 (One- and Two-Family Residences) in the EF rating scale does not follow observed damage patterns.</p> <p>As noted in Chapter 4, most residences rated by the MAT followed the order of DODs prescribed by the EF scale closely, with the exception of DOD 5 (Entire House Shifts off Foundation). It was very unusual for DOD 5 to precede DOD 6. In the one documented case (Figure 4-17), the observed residence was older construction.</p>	<p>Recommendation #45 Modify EF scale DI 2 (One- and Two-Family Residences).</p> <p>Based on the MAT’s observations for DI 2 (One- and Two-Family Residences), DOD 5 (“entire house shifts off foundation”) was rarely witnessed, unlike DODs 4 and 6, and should be eliminated from the list of DODs.</p>