Analysis of Manufactured Home Vulnerability in Extreme Winds

by

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Abstract

Fatality rates are higher in mobile and manufactured homes (MMH) than in permanent site-built homes during tornadoes. The physical processes driving the enhanced fatality rates have not been fully explored, but must be understood to drive effective mitigation measures. Over a two year period from 2019 to 2020, detailed forensic assessments were conducted of tornado damage to mobile and manufactured homes in Alabama. These post-tornado studies were analyzed to evaluate the relative performance of common MMH anchorage systems. Experimental tensile testing was conducted on tie-down straps collected from field samples. Finally, a numerical model was developed using the commercial software SAP2000 to perform a parametric assessment of the demand on common anchorage systems in MMH. Overall, the study finds that a primary driver of the high fatality rates is the typical failure sequence of these homes. MMH usually experience anchorage failures prior to any other structural failures, resulting in an increased risk of rolling or lofting. The brittle anchorage failures lead to complete destruction of the superstructure at low wind speeds, increasing the likelihood of fatalities occurring. This study finds that several factors contribute towards this non-ideal failure sequence, including defective tie-down straps, premature ground anchor pullout, inadequate code requirements, and the increasingly common use of alternative, pan-style anchorage systems. Identification of these factors guides future mitigation and education efforts, and gives targeted directions for future research.

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List of Abbreviations

MMH	Mobile/Manufactured Homes
HUD	U.S. Department of Housing and Urban Development
MHCSS	Manufactured Home Construction and Safety Standards
MHI	Manufactured Housing Institute
ASCE	American Society of Civil Engineers
CFR	Code of Federal Regulations
ASD	Allowable Stress Design
LRFD	Load and Resistance Factor Design
IBC	International Building Code
ASTM	American Society for Testing and Materials
IRC	International Residential Code
NFPA	National Fire Protection Association
WJE	Wiss, Janey, Elstner Associates, Inc.
FS	Factor of Safety
NWS	National Weather Service
EF	Enhanced Fujita
DI	Damage Indicator

DOD	Degree of Damage
DAT	Damage Assessment Toolkit
MOT	Modulus of Toughness
PDF	Probability Density Function
RMSE	Root Mean Squared Error
E	Modulus of Elasticity
F_y	Yield Strength
Sqft	square feet
Mph	miles per hour
Ft.	feet
Psf	pounds per square foot
In.	inch
Lb	pound
Ksi	kips per square inch

1. Introduction

Mobile and manufactured homes (MMH) are prefabricated wood-frame structures that can be transported on the steel frame upon which they are constructed. Manufactured homes are very practical and affordable, leading to their sustainable and even growing popularity. According to a Manufactured Housing Institute (MHI) report, manufactured homes made up 36.7% of all new homes sold in 1982. They remained very popular in the 1980s and 1990s, comprising at least 23% of all new homes bought every single year during that time span [1]. One reason for their success was that in 1985, the cost of a manufactured home ($20.19 per square foot (sqft)) was less than half of the cost of a site-built home ($45.18 per sqft) [2]. A report from 2001 estimated that manufactured homes made up over 25% of all dwellings in the United States [3].

However, since 2005, manufactured homes have only comprised between 10-15% of new homes purchased. This is not due to a change in cost, as the average cost of a manufactured home in 2017 ($50.42 per sqft) was still less than half the cost of a site-built home ($111.05 per sqft) [4]. One reason behind the slight decrease in popularity may be the commonly perceived notion that manufactured homes are not safe in disasters. Many well documented hurricanes and tornadoes have led to the destruction of MMH, and questions about their resiliency have repeatedly arisen after these events. From 1985 to 2017, 54% of all housing-related tornado fatalities occurred in MMH, despite comprising only 6% of the total U.S. housing stock [5]. Additionally, a study by Ashley (2007) separated the fatality statistics into 5-year intervals from 1986-2005, and the results showed that the percentage of fatalities occurring in MMH grew from 37.2% (1986-90) to 56.7% (2001-05) [6]. These studies support the idea that occupants sheltering in MMH during an extreme wind event are at a higher risk of fatality.

The objective of this study is to determine the cause of that enhanced fatality risk, particularly focusing on MMH exposed to tornado hazards. In order to do so, the design requirements that govern manufactured homes must first be understood. Also, an understanding of the load path for resisting lateral wind loads is essential, as one component failure in this path could lead to the destruction of the entire structure. One critical part of the load path is the anchorage system, as it is highlighted as a common failure pattern in the results of post-tornado damage assessments performed by T.P. Marshall et al. [7]–[9]. Therefore, the different types and requirements for them will be discussed in detail. Second, past hurricane studies will be reviewed to provide a broad view of performance statistics and common failure mechanisms. These failure mechanism statistics can help highlight deficiencies in the load path, as well as patterns of failure. Also, many experimental tests have been performed on ground anchors, which are a common component in the anchorage system load path. Third, post-tornado damage assessments conducted by the author and others reveal patterns in MMH failure mechanisms, similar to post-hurricane damage assessments, and the key findings from those studies will be presented and discussed. Fourth, anchorage tie-down strap samples collected from those assessments were tested, and the results will be presented and compared to the ground anchor tests performed in the past. Finally, the results from a numerical modeling of a manufactured home will be presented to check the adequacy of the requirements governing their design. Each of these steps works to present a holistic assessment of the issue of manufactured home vulnerability, and to answer the question of why these homes have an enhanced fatality risk.

2. Structural Design Background

Although the terms “mobile” and “manufactured” are used interchangeably when referencing these homes, they have technical differences. Mobile homes are prefabricated, transportable houses that were built and installed prior to 1976, while manufactured homes are those constructed and installed during and after 1976. Throughout this study, “MMH” will be used when referring to both mobile and manufactured homes, while the term “manufactured home” will only refer to those constructed during or after 1976. The change in nomenclature coincides with the National Manufactured Housing Construction and Safety Standards Act of 1974, which was passed by the U.S. Department of Housing and Urban Development (HUD). The structural design of manufactured homes is governed by HUD, and the standards and requirements for designing and implementing manufactured homes can be found in the Manufactured Home Construction and Safety Standards (MHCSS). The HUD act passed in 1974 required all manufactured homes installed after June 15, 1976, to meet the MHCSS requirements. This was the first step in improving the structural design of manufactured housing. In those original requirements, wind loading was determined from location, split into two wind zones. The *Hurricane Wind Zone*, near the coast, used a basic “fastest mile” design wind speed of 90 miles per hour (mph) at 30 feet (ft) above ground in open terrain. The *Standard Wind Zone* used a “fastest mile” wind speed of 70 mph with the same parameters [10]. However, the destruction of manufactured homes due to Hurricane Andrew in 1992, as well as other disasters, prompted HUD to update the MHCSS in 1994. Before the update, manufactured homes designed using the MHCSS were *10 times* more likely to fail in a 10-year exposure period than if they were designed using ASCE 7-88 in hurricane-prone areas [11], [12]. The MHCSS update in 1994 included re-zoning *Wind Zones 1 and 2* and adding a *Wind Zone 3*. *Wind Zones 2 and 3*, which were the closest to the coast, were updated to closely resemble the wind design provisions that were required by ASCE 7-93 [13], [14]. However, the original MHCSS provisions for the *Standard Wind Zone* were applied directly to the new *Wind Zone 1* without any updates. This version of the MHCSS is still in effect and the provisions for *Wind Zone 1* will be further discussed, as the majority of manufactured homes impacted by tornadoes fall within this zone.

2.1. Wind Loading Design Requirements

The wind loading provisions for manufactured homes can be found in 24 CFR Part 3280 of the MHCSS [15]. This section provides requirements covering everything from interior layout to the transportation of the home. The lateral design load requirements are in Subpart D, *Body and Frame Construction Requirements*, which provides requirements to ensure the strength and durability of the structure. Included in this section are the specific design parameters such as dead, live, and wind loads. However, it states that the "roof live load or snow load shall not be considered as acting simultaneously with the wind load", and that "floor live loads shall not be considered as resisting the overturning moment due to wind." Therefore, the wind loads typically control the design in Alabama and the surrounding states, as it requires greater resistance and thus prioritized over the live loads and snow loads.

For homes located in *Wind Zone 1*, the design wind load is prescribed as a simultaneous combination of a 15 pounds per square foot (psf) net horizontal load and a 9 psf uplift load (plan view). The horizontal pressure is only applied to the side wall and not the vertical roof projection, so long as the roof angle is less than 20 degrees. These pressures are calculated from the 70 mph fastest mile wind speed (V_{fm}) using Allowable Stress Design (ASD). Presently, structures are typically designed using ASCE 7-16, which utilizes an ultimate 3-second gust wind speed (V_{ult}) with Load and Resistance Factor Design (LRFD). Equation 16-34 in the 2006 International Building Code (IBC) can be used to convert between the two types of wind speeds, as shown below [16]. First, the fastest mile wind speed must be converted to a 3-second gust (V_{3s}) (Eq. 2-1), and then it must be converted from ASD to the LRFD ultimate wind speed (V_{ult}) (Eq. 2-2).

$$V_{3s\,ASD} = 1.05 * V_{fm\,ASD} + 10.5 \qquad \textit{Eq. 2-1}$$

$$V_{ult\,LRFD} = V_{3s\,ASD} * \sqrt{1.5} \qquad \textit{Eq. 2-2}$$

The value of 1.5 represents a factor of safety (FS) that the MHCSS requires to be applied to loads on the anchorage system. Therefore, the anchorage must simultaneously resist both a net horizontal pressure of **22.5 psf** and a vertical uplift of **13.5 psf.** This differs from modern ASD to LRFD conversion in ASCE 7-16 [17], in which separate load combinations are used to account for the difference between the two methods. For LRFD, the 3-second wind speed pressure is multiplied by 1.0 to calculate the ultimate load, but for ASD the 3-second wind speed pressure is multiplied by 0.6 to reduce it to an allowable load. However, for manufactured home design, the 70 mph fastest mile wind speed is already given as an allowable load design wind speed. Therefore, the resulting net lateral pressure of 15 psf is an allowable design load. The FS of 1.5 is then used to increase the pressures to an ultimate load design. Using the equations listed above, the 70 mph fastest mile design wind speed converts to an ultimate 3-second design wind speed of 103 mph. It is important to note that the loads for homes in *Wind Zones 2 and 3* are not required to be increased by this factor of safety, as it is assumed that the factors of safety are incorporated into the choice of design wind speed.

Subpart D also lists requirements for specific anchorage components. There are different types of anchorage systems, but almost all of them use some configuration of flat metal straps and ground anchors. These metal straps must be successfully tested in accordance with ASTM D3953–91, the *Standard Specification for Strapping, Flat Steel and Seals* [18]. Additionally, they must be a minimum of 1.25 inches (in.) wide and 0.035 in. thick, and they must have a weather resistant coating equivalent to a zinc coating of 0.30 ounces per square foot. The spacing requirements for these straps are found in a different location, but Subpart D does state that a strap and ground anchor must be installed no more than 2 ft from each corner of the home. Lastly, every strap-to-ground anchor connection must have the capacity to withstand a 3,150 pounds (lbs) working load, with an ultimate resistance to an additional 50% load. Functionally, each anchorage component must ultimately resist 4,725 lbs of tension without failure. Other than obvious failure mechanisms such as ground anchor pullout or strap breakage, the MHCSS does not specify failure criteria for the anchorage system. However, the International Residential Code (IRC) specifies ground anchor failure as a vertical displacement of 2 in. or a horizontal displacement of 4 in. [19]. Although this does not apply to all states, some have adopted a similar requirement. The Alabama Manufactured Housing Commission Administrative Code defines serviceability failure as 2 in. vertical displacement or 3 in. horizontal displacement [20]. However, the IRC failure criteria was used in several of the ground anchor testing studies discussed in Section 3.2.

2.2. Anchorage System Requirements

The installation procedures of a manufactured home are governed by 24 CFR Part 3285 of the MHCSS [21]. Subparts C and D provide important requirements regarding the installment of the foundation, while Subpart E provides anchorage system installation requirements. Foundations of manufactured homes typically consist of concrete masonry block piers placed on square or continuous footings, though the piers can also be pressure-treated wood. These piers support the I-beam rails spanning the length of the home, and the spacing is determined by the structure load and the bearing capacity of the soil. Typical pier spacing is 4-8 ft, measured center-to-center, down the length of the frame. The I-beams are designed to transfer all loads on the structure into the foundation, so they must be sized to sufficiently do so. There are no specific size requirements in the MHCSS, but the design tables for pier heights and spacing assume an I-beam depth of 10 in. The vertical loads from the home are transferred through the I-beams into the piers, while the lateral loads are transferred through the I-beams into the anchorage system. The anchorage system typically attaches to these I-beams, sometimes incorporating the piers into the design as well. The lateral loads are ultimately transferred into the ground using steel strapping or compression struts, depending on the type of anchorage system used. A more detailed analysis of a typical manufactured home load path will be discussed in Section 2.3. The MHCSS focuses primarily on a tie-down strap and ground anchor combination, and the requirements will be listed in Section 2.4.1.

2.3. Typical Manufactured Home Load Path

A manufactured home designed in accordance with the MHCSS requirements outlined in Sections 2.1 and 2.2 will have the major structural components labeled in Figure 2-1, except that a single-wide MH will not have the marriage line shown. Double-wide manufactured homes are transported in two separate sections and merged together at the permanent location of the home.

The marriage lines represents where the two halves are connected, usually through the floor structure, roof structure, and end walls. A single-wide home would not include a marriage line, or any of the components related to it. Instead, the home would be approximately half of the width shown and would only require two rows of piers supporting the two I-beam rails. Also, the diagram displays a strap and ground anchor anchorage system, as described in the MHCSS. However, approved alternative anchorage systems may transfer the loads differently than shown below. These alternate systems are described in Section 2.4.

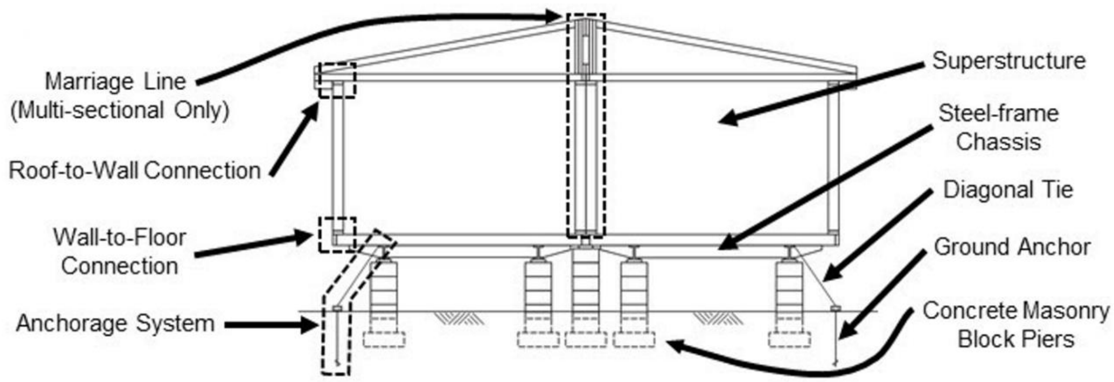

Figure 2-1. Typical structural components of a manufactured home (modified based on Figure 2-6 in FEMA (2009) [22]*)*

The load path typically begins with the side wall and roof structures. Wind loads are collected by the cladding on these components and transferred into the roof trusses and stud walls. In the case of roof loads, the roof trusses then transfer the loads into the walls through the roof-to-wall connection. The stud walls then transfer both the roof loads and the side wall loads into the floor diaphragm through the wall-to-floor connection. Next, the floor diaphragm transfers all loads, including the gravity dead load of the structure itself, into the steel-frame chassis (I-beam rails) spanning the length of the structure. Finally, the chassis transfers the loads into the foundation and anchorage systems. The concrete masonry block piers function to resist the gravity loads, while the anchorage systems resists the lateral loads and any uplift loads that may occur. The critical failure points in the load path are the roof-to-wall connection, the wall-to-floor connection, and the anchorage system. Additionally, in double-wide homes, separation of the marriage line is a potential failure mechanism. These failure mechanisms will be discussed in further detail in Section 4.1.

2.4. Types of Anchorage Systems

While the metal tie-down strap and ground anchor system that is highlighted in the MHCSS requirements have traditionally been the most popular anchorage system, other alternative anchorage systems have been developed and approved. These include anchorage of the home to a permanent concrete slab, spread footings, reinforced masonry stem walls, and a variety of systems commonly referred to as “pan systems”. Pan systems consist of metal tubular braces attached from the I-beam rails beneath the home to a metal pan or concrete footing anchored in the ground. These systems take advantage of the fact that typical manufactured homes have sufficient dead load to prevent any net design wind uplift force and focus on resisting lateral forces only. The traditional tie-down and ground anchor system is the most common system currently found on installed homes, but the pan system is gaining popularity due to its affordability and ease of installation. There are different variations of each of these systems, and the most common of those variations will be discussed below.

2.4.1. Tie-down Straps and Ground Anchors

The tie-down strap and ground anchor system consists of metal straps connecting from the I-beams beneath the structure to helical steel anchors secured in the ground. These span the length of the structure on both sides, and the spacing requirements for these straps are listed in Part 3285 of the MHCSS. The requirements for maximum strap spacing differ depending on the *Wind Zone*, but the provisions for homes within *Wind Zone 1* are shown in Table 2-1. MHCSS minimum strap spacing requirements for Wind Zone 1. The strap spacing requirements are dependent upon the width of the home, the max pier height, and the I-beam spacing. The spacing values are calculated using the previously listed wind pressures, the minimum anchorage working strength requirements, a roof slope less than 20 degrees, and a side wall height of 7.5 ft.

The most common strap configuration is a "diagonal strap", which is defined as being greater than 15 degrees from the fully vertical position, and these are required regardless of the *Wind Zone*. Straps that are span vertically to attach to the outer edge of the structure are called "vertical straps", and are required on homes located in *Wind Zones 2 and 3*. These two different configurations are highlighted in Figure 2-2 and Figure 2-3, which are captured during field assessments. It can also be seen in Figure 2-3 that the diagonal strap spans to the opposite I-beam rather than the closest one, which is sometimes chosen when a vertical strap is present.

Table 2-1. MHCSS minimum strap spacing requirements for Wind Zone 1

Nominal Floor Width, Single/Double-wide	Max Height from Ground to Strap Attachment	I-beam Spacing 82.5 in	I-beam Spacing 99.5 in
12/24 ft	25 in	**14 ft 2 in**	**N/A**
	33 in	**11 ft 9 in**	**N/A**
	46 in	**9 ft 1 in**	**N/A**
	67 in	**N/A**	**N/A**
14/28 ft	25 in	**18 ft 2 in**	**15 ft 11in**
	33 in	**16 ft 1 in**	**13 ft 6 in**
	46 in	**13 ft 3 in**	**10 ft 8 in**
	67 in	**10 ft 0 in**	**N/A**
16/32 ft	25 in	**N/A**	**19 ft 5 in**
	33 in	**19 ft 0 in**	**17 ft 5 in**
	46 in	**16 ft 5 in**	**14 ft 7 in**
	67 in	**13 ft 1 in**	**11 ft 3 in**

Figure 2-2. Typical diagonal tie-down straps, attached to the near I-beam.

Figure 2-3. Vertical strap configuration (left) and a helical ground anchor (right)

The heads of the ground anchors can also be seen in the two figures above. The ground anchors are typically 30 in. to 60 in. long, with helical disks around the bottom of the shaft. These are typically drilled into the ground until only the head shows, angled slightly away from the direction that the strap spans. The length of the anchor and diameter of the helical disks is dependent upon the soil classification. The metal strap, after being attached to the frame of the home, is then coiled inside the head of the anchor and tightened until the desired tension is met. In addition to the ground anchors, a stabilizer plate is often required at each anchor. These steel plates, usually 12 in. to 17 in. wide, are driven into the ground adjacent to the ground anchors, in the direction that the strap tensile force is coming from. Their purpose is to add to the bearing surface area the soil is pushing against to resist lateral movement of the anchor.

2.4.2. Alternative Anchorage Systems

In Alabama, the most common form of alternative anchorage systems is the pan system. There are different variations of this type of system, but our post-tornado assessments indicate that the most common type of pan system is what is commonly referred to as an Oliver pan system. More specifically, it is the 1100 “V” Series Pan System [23], created by Oliver Technologies, Inc. The basic configuration can be seen in Figure 2-4 below. These systems have gained popularity recently because of their cost effectiveness and ease of implementation.

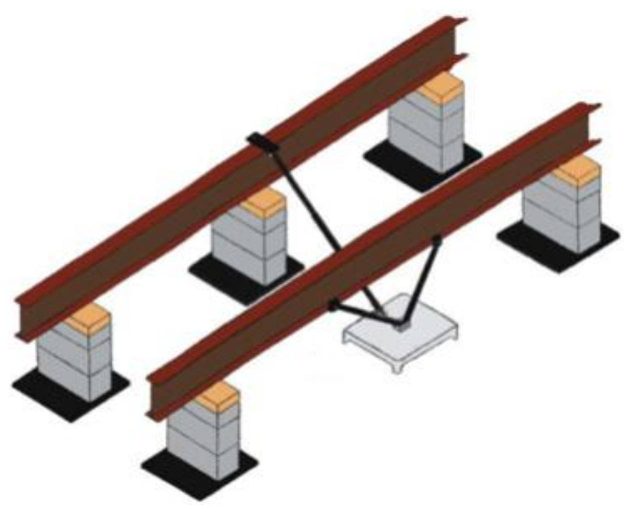

Figure 2-4. 1100 "V" Series Pan System [24]

Essentially, a metal pan with elongated, sharp corners is firmly pressed into the ground, directly under one of the main rails where a pier would usually be. Metal tubular braces are then attached to the top of the pan and connected to the main I-beam rails. The most important brace spans from the pan to the top of the opposite rail, and its purpose is to provide lateral resistance for the home. Two other braces extend up from the pan to the I-beam rail above, angled away from each other to form a “V”, hence the name. The “V” shaped braces help transfer the dead weight of the home down into the pan, pressing it further into the soil. This is important because the friction between the pan and the soil ultimately creates the lateral force resistance for the home. If the home was lifted several inches, the pan would no longer be pressed into the soil and the lateral brace would have nothing to transfer the load into. This issue is common with most pan systems and can only be remedied by utilizing something other than the dead weight of the home as a way of attaching the pan to the ground. The “V” shaped braces also provide longitudinal lateral resistance for the home. There are typically two pan systems per home, with one located near each end. They are placed beneath opposite I-beam rails, so the longer lateral arms extend in opposite directions of each other. If the roof slope is less than 20 degrees, only two pan systems are required for both single-wide and double-wide homes in *Wind Zone 1*. Additional pan systems may be required if the roof slope is greater than 20 degrees, but that scenario is very uncommon with manufactured homes. It is important to note that single-wide homes with pan systems also typically have four traditional straps and ground anchors located at each corner of the home to provide additional lateral and uplift force resistance. A picture of an actual Oliver pan system is shown in Figure 2-5 and was taken during a field assessment.

A variation of the typical pan system is the Xi2 System [25]. The Xi2 System was created by Tie-Down Engineering, a manufactured home anchorage development company. The Xi2 System is very similar to the Oliver pan system, with the only major difference being that the metal pan is placed beneath a pier rather than replacing one. It also does not require the “V” struts for longitudinal resistance, although one or two is sometimes utilized. Figure 2-6 below shows a basic diagram of the system. Similar to the pan system, homes typically have two Xi2 systems placed near the end of each home. The required number of Xi2 systems may increase if the roof slope is greater than 20 degrees, and it also increases to 3 systems if the length of the home is greater than 76 ft and located within *Wind Zone 1*.

Figure 2-5. Oliver Pan system in use.

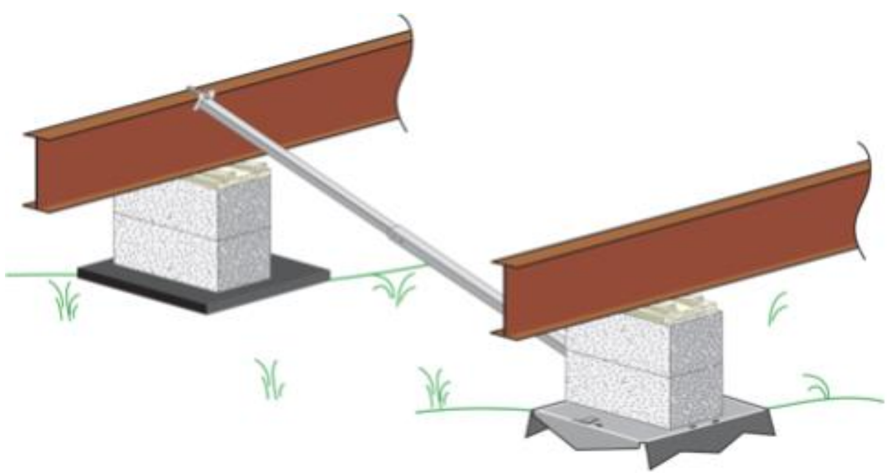

Figure 2-6. Xi2 system diagram [25]

Another variation of the pan system is the Longitudinal and Lateral Bracing System (LLBS) [26], created by Minute Man Anchors, Inc. It closely resembles the Oliver pan system and the Xi2 system with a few differences. The main difference is that the lateral and longitudinal braces are attached to a concrete footing rather than a metal pan. Similar to the Xi2 system, the footing is located beneath a pier, but the footing is large enough that the braces can be bolted to it beside the pier. As with the other pan systems, this system requires one brace within 10 ft of each end of the home.

The Vector Dynamics Foundation System was created by Tie-Down Engineering, the same company that makes the Xi2 System, and is approved for use with single-wide and double-wide homes. The vector system uses the same metal straps that have been previously described but utilizes two attachment points to increase their resistance to uplift. A diagram of the system configuration can be seen in Figure 2-7. Overall, the anchorage system uses a combination of the vector dynamics attachment and the traditional strap and ground anchor configuration, alternating each system down the length of the home. As for the vector system itself, special metal pans are placed underneath adjacent piers and the ground anchors are attached to these pans on the outside edge of each pier. A brace is then placed spanning between the two piers, attached to the metal pans and the ground. The brace must have adequate compressive strength to withstand the forces transferred from the straps. Once the brace is in place, the metal tie-down straps are connected from the head of the anchor, over the top of the I-beam frame, to the opposite end of the brace. The system is designed to resist both lateral forces and uplift forces. A number of requirements must be met to install this system, and they can be found in the Tie-Down Engineering Vector Dynamics Foundation System User Manual [27].

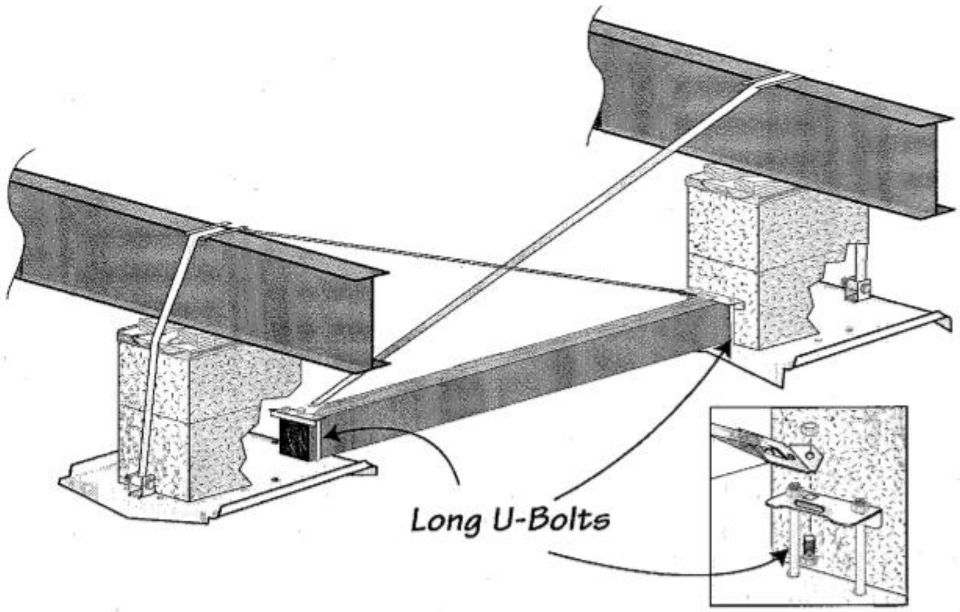

Figure 2-7. Vector Dynamics system diagram

3. Past Studies Assessing MMH Performance

Field studies assessing the structural performance of MMH have been conducted after many severe hurricanes, most notably Hurricane Andrew in 1992. Post-hurricane damage provides valuable information regarding the structural performance of MMH, as hundreds of homes in the affected area experience similar wind loading pressures. Therefore, structural components can be compared over large sample sizes of homes, oftentimes highlighting load path deficiencies.

While large-scale assessments identify general load path issues, experimental testing enables the evaluation of specific structural components under known loads. Presently, the main manufactured home component subjected to experimental testing has been ground anchors, as there is a large degree of uncertainty in the load capacity of them. Section 3.2 highlights the results from these studies to provide a comparison between capacity expectations and reality.

3.1. Post-Hurricane Damage Assessments

In 1992, Hurricane Andrew struck the southern coast of Florida as a Category 4 hurricane, causing immense wind damage and flooding. The hurricane then reentered the Gulf of Mexico heading west until turning north to directly hit the Louisiana coast as a Category 4. Heavy wind damage was dealt to all types of structures, but especially to MMH. In Dade County, Florida, over 5,000 MMH were destroyed [28]. Many damage assessments were performed in both Florida and Louisiana with an emphasis on determining the factors contributing to such a high degree of MMH destruction [29]–[34]. A greater emphasis was placed on this damage because 97% of MMH located in Dade County were destroyed, while only 11% of site-built homes in the same area were destroyed [31].

One major issue that emerged was the inadequacy of the anchorage systems of the MMH in both states. At the time, Florida and Louisiana did not strictly enforce the MHCSS regulations regarding the installation of MMH, so anchorage systems were often neglected in order to keep costs low. Many homes in Louisiana either did not have an anchorage system or had too few straps and ground anchors [31], [33]. Similarly, many of the MMH assessed in Florida had anchorage systems but did not meet the minimum requirements for them. The major failure mechanism of anchorage systems in Florida was ground anchor pullout [30], [31], which will be discussed in further detail in Section 3.2.

The inadequacy of the anchorage systems resulted in greater damage to the homes. In a field study performed by Levitan et. al. [33] in Louisiana, a total of 217 damaged MMH were assessed. A primary damage mechanism was assigned to each home, selected from the following three categories: anchorage, roof/wall, and tree/missile. Of the 217 homes assessed, 55% of them were damaged due to anchorage failure while only 34% were damaged due to roof/wall failure. Additionally, when the data is constrained to homes experiencing major damage, the damage due to anchorage failure rose to 57%, while the damage due to roof/wall failure rose to 39%. These results are very similar to that of a similar study performed after Hurricane Elena in 1985. Using the same parameters, major damage to MMH in Hurricane Elena was caused by anchorage failure in 44% of cases, while roof/wall failure was to blame in only 29% of cases [33], [35]. The values from Hurricane Elena are slightly lower due to an increase in damage caused by projectiles, but the ratios of anchorage failure to roof/wall failure are very similar in the two events. Clearly, anchorage failure was a major issue in both hurricanes, leading to the destruction of homes.

In 1996, Hurricane Fran struck the North Carolina coastline as a Category 3 hurricane, causing damage from a combination of the high wind speeds and storm surge. Although the loading type was different, many of the same MMH anchorage issues emerged from field studies. Approximately 50% of MMH experienced anchorage failure in the form of strap failure in tension, ground anchor pullout, and even the collapse of the piers due to localized scour [36], [37]. The FEMA report listed several causes for the failure of these systems, including strap corrosion, the use of unreinforced masonry block piers in a flood risk area, and the use of inadequately sized ground anchors.

More recently, a detailed study of the damage caused by Hurricanes Katrina and Rita showed apparent improvements in manufactured home anchorage systems [38]. Both hurricanes struck Louisiana in 2005, causing immense damage from both wind and flooding. A total of 251 MMH were assessed in detail, and only 10.0% of those homes were classified as having anchorage failure. Additionally, the homes were separated into three categories based on age of the home: Pre-1974, 1974-1994, and Post-1994. These groupings were chosen based on the major HUD updates to rules and regulations for manufactured homes. The Pre-1974 homes experienced anchorage failure in 23.3% of cases, the 1974-1994 homes' anchorage failed in 15.6% of cases, and the Post-1994 homes only experienced anchorage failure in 0.8% of cases. Although the failure percentages are lower than those from other studies, indicating that the damage may not have been as bad in these events overall, the relative trends within the two events still hold true. Each grouping had at least 60 units assessed, so the sample sizes were large enough to reliably indicate this trend. All the assessed homes were located in *Wind Zone 2 or 3,* so the updates to the MHCSS in 1994 had an increased effect on them. The results clearly indicate this effect, as less than 1% of homes constructed after 1994 experienced anchorage failure.

3.2. Ground Anchorage Experimental Testing

Several studies have been performed to test the capacity of the ground anchors commonly used in MMH anchorage systems. These ground anchors function to resist both uplift and lateral forces transferred from the structure through the tie-down straps, and it is important that they match or exceed the capacity of the straps. As stated in Section 2.1, the tie-down strap and ground anchor system is required by HUD to resist a 4,725 lb tensile load without failing. Failure, according to the Alabama Manufactured Home Standards, occurs when the anchor head displaces 2 in. vertically or 3 in. horizontally. Past studies have shown that oftentimes the ground anchors do not meet requirements [11], [13], [39]–[45].

The issues surrounding pullout capacities of ground anchors are not new, as evidenced by studies performed by Yokel et al. in 1979 and 1981 [39], [40]. These thorough reports tested several different scenarios, including different ground anchor lengths, disk sizes, disk shapes, and pullout angles. Informed by the author's field studies described in subsequent sections, the majority of ground anchors installed have helical disks of varying sizes and lengths, so the performance of helical anchors will be focused on from all past studies. In the 1979 report, of the 9 vertical load tests performed on multiple anchor types, only 5 of them met the minimum requirements of the NFPA 501A-1975, which matched the current minimum requirements. Similarly, in the 1981 report, 6 tests were performed on 6 in. single helix anchors in both sand and silt at varying load angles, and none of them met the minimum requirements. The report concludes simply, stating that "Present anchoring techniques as used in the field do not provide the necessary support for the diagonal ties." It also highlights that because the tests revealed a high variability in ground anchor stiffness, it "will cause certain straps to be overloaded and fail, before other straps are loaded to capacity."

Further studies performed by Yokel et al. and Marshall [11] indicate even more concerning results and were compiled in a study by Pearson et al. in 1996 [41]. Table 3-1 below summarizes the results from Pearson et al., showing the capacity results from 4 ft long anchors with a 6 in. helical disk. Of the five scenarios tested by vertical loading, only one group met the minimum displacement requirements. The average load at the ultimate pullout capacity exceeded the requirement, but resulted in an average vertical displacement of 7 in.

Table 3-1. Pullout capacities of ground anchors due to vertical loading (adapted from Table 3 in Pearson et al. 1996) [33]

Soil Type	Number of Tests	Load at 2" Vertical Displacement (lbs)	Ultimate Load (lbs)	Maximum Vertical Displacement (in.)
Silt[a]	11	4,295	5,173	7.17
Silt[b]	5	2,320	3,640	11.04
Sand[a]	6	4,488	5,063	3.82
Sand[b]	3	5,100	5,953	4.35
Clay[a]	3	3,067	3,433	5.83
Weighted Mean Values		**3,938**	**4,773**	**7.00**
Note: [a]Moist, [b]Wet				

Additionally, other tests were performed by loading the anchors at various angles, as well as installing the anchors at slight angles against the load. The results are shown in Table 3-2, summarized from the results presented in Pearson et al. (1996). Additionally, Figure 3-1 demonstrates the measurement location for each angle, taken directly from Pearson (1996). The resulting loads at the failure criteria displacement are extremely low, with only two of the seven cases reaching even half of the required load. Furthermore, the displacements recorded at the ultimate pullout capacity are quite large, reinforcing the concern of Yokel et al. (1981) about large displacements of anchor heads causing the failure of tie-down straps.

Table 3-2. Pullout capacities of ground anchors due to horizontal loading (adapted from Table 4 in Pearson et al. 1996) [33]

Soil Type	Angles θ_1/θ_2 (degrees)	Load at 4" Horizontal Displacement (lbs)	Ultimate Load (lbs)	Maximum Displacement (in)
Silt	90/60	2,967	7,565	10.07
Silt	90/45	1,350	7,933	13.85
Sand	90/40	2,583	6,187	9.08
Clay	90/40	767	3,267	18.60
Silt	135/60	433	3,387	25.15
Silt	135/45	413	4,243	33.80
Silt	135/15	433	4,775	54.00
Mean Values		**1,278**	**5,337**	**23.51**
Note: Each case based on 3 tests. All tests conducted under moist soil conditions.				

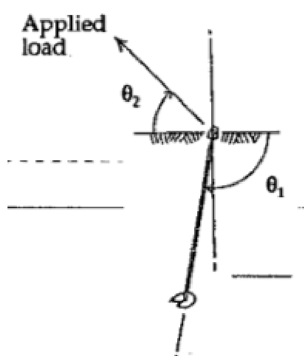

Figure 3-1. Diagram of angles referenced by Table 3-2 (Excerpt from Figure 11 in Pearson et al. 1996) [41]

Additional tests performed by the Wiss, Janney, Elstner (WJE) Test Program studied the impacts of adding stabilizer plates to the ground anchor system. These results were presented by

Longinow et al. and summarized by Pearson et al. [41]–[43]. While stabilizer plates do add resistance capacity, the overall impact is minimal. Although in the testing they increased the maximum capacity by approximately 700 lbs, the anchors still did not come close to meeting the load-displacement requirements, even with the loads listed corresponding to a 4 in. horizontal displacement rather than 3 in. Additionally, it appears that the added maximum capacity came at a consequence of even larger displacements. Further testing was performed by WJE to assess the effects of varying the anchor and load angles on the overall capacity, and the results reinforce the very limited effect stabilizer plates have on load capacity and displacements. Similar to the results from Marshall (1994) [11], none of the configurations tested met the requirements for horizontal deflection.

In a more recent study performed by Crandell et al. for a 2008 HUD report [44], which was published in an abbreviated form in 2011, 66 ground anchors were tested at 3 different sites with varying soil conditions. The anchors were also pulled at 3 different angles (30, 45, and 90 degrees) to test both the vertical and horizontal failure criteria. At each site, the soil was thoroughly tested and classified, and the ground anchors were chosen to meet the requirements for each particular soil condition. The results showed that in all cases the anchors failed well below the required capacity. In the vertical pull tests, the ground anchors failed at an average force of about 2,500 lbs. The horizontal 30 degree pull tests, performed on the same anchors with stabilizer plates, met the minimum horizontal displacement criteria at loads between 1,500 lbs and 2,500 lbs on average. Even with larger-than-required ground anchors and stabilizer plates, the average load at failure was approximately 2,700 lbs for the 30 degree pull tests. Similarly, the average failure load for anchors pulled at 45 degrees was approximately 3,500 lbs. These results add to the overwhelming conclusion that ground anchors do not consistently provide the required capacity.

Upon reviewing the many ground anchor studies performed in the past, Mays suggests that a realistic mean for the ultimate load capacity for ground anchors is 2,363 lbs, rather than the 4,725 lb capacity currently required [45]. He further explains that the failure issue may be due to the low factor of safety associated with ground anchors. While studies suggest that upward loading in soil should require a FS of 3.0 – 6.0, the current HUD code prescribes a FS of only 1.5 [45], [46]. However, using the suggested realistic mean for anchor capacity, the actual FS of these ground anchors is less than 1. This indicates a legitimate issue with ground anchor design, and examples of anchor pullout are discussed further in Section 4.1.

4. Post-Tornado Damage Assessments

Field assessments following damaging tornado events are critical tools for capturing perishable data on the in-situ performance of MMH during extreme wind events. Oftentimes, especially after tornadoes, the debris is cleaned up quickly and the potential data goes with it. Therefore, field studies must be performed quickly and efficiently in the aftermath of a tornado or hurricane. The studies described below were conducted in the immediate aftermath of tornadoes by a number of different teams. Section 4.1 presents the findings from detailed analysis of MMH load paths, conducted in the aftermath of tornadoes occurring within Alabama. These results highlight common load path failures seen in MMH, especially anchorage systems. Also, reoccurring issues inherent to each type of anchorage system are discussed in detail. Section 4.2 draws data from the National Weather Service (NWS) tornado database to compare the results found in Section 4.1 to a significantly larger, less detailed data sample. Section 4.3 presents data from Sections 4.1 and 4.2, as well as other studies, to explore the enhanced risk of MMH in tornadoes. Finally, Section 4.4 discusses the key findings from all of the post-tornado assessments.

4.1. Qualitative Analysis of MMH Load Paths

This section describes data collected over a 2 year period from 2019 to 2020 by researchers at Auburn University and other collaborators. The purpose of each assessment was to document all structures that were impacted by the event and record as much available pertinent information for each as possible to understand why a given structure performed as it did. Similar to the past hurricane studies, data gathered from damaged structures helps to highlight common failure mechanisms. Additionally, these assessments may highlight any structures that performed better than expected, and hopefully identify the reasons for their success. Overall, field assessments of tornado damage seek to understand the interaction between tornado wind loads and structural load paths, identifying both the successful components and the areas that need improvements.

4.1.1. Assessment Methods

The post-tornado damage assessments conducted utilized several different techniques, including door-to-door forensic assessments, drone imagery on the scale of a single home to an entire neighborhood, drive-by assessments, and aerial imagery of the entire tornado damage path. Additionally, in the case of MMH with traditional anchorage systems, tie-down strap samples were collected to be tested to their full capacity. These results are shown in Section 5. Each assessment technique offers unique value, and they can all be combined to get a more complete understanding of a tornado and the damage it caused. Door-to-door forensic assessments offer the most detailed analysis of structures and are usually conducted in areas experiencing the worst damage. Specific failure mechanisms can be revealed from this analysis, and the strap samples were all collected during these assessments. Drive-by assessments are useful for quickly obtaining a general understanding of the damage, determining the path of the tornado, and determining the width and intensity of the tornado by assessing cross-sectional cuts through the damage path. Drone imagery can be useful for analyzing large buildings such as warehouses, entire neighborhoods, and MMH parks. The three-dimensional point clouds from drones enable near door-to-door level remote assessments after the fact. Finally, aerial imagery is typically captured in the immediate aftermath of the tornado, and it can assist in locating the target area for other assessments, mapping out the damage path and width, and even enable the mapping of tree-fall patterns to model the tornado intensity [47].

The information for each structure assessed was compiled in the Fulcrum smartphone application, which is a mobile data collection platform developed by Spatial Networks, Inc. The app attaches building data and photos to the location of the structure, enabling the convenient and efficient storage of the data. The gathered data included external building attributes, structural attributes, and damage notes. As previously stated, one of the purposes of these assessments was to identify any patterns that emerged in load path failures, component failures, or unexpected structural successes, so it was important to record all of the available structural information about each home. Additionally, other resources such as public county records databases were used to gather information such as the age of each structure.

The Enhanced Fujita (EF) Scale was used to classify the damage to each structure [48]. The EF Scale uses descriptive damage states of specific building types to estimate tornado wind speeds. The building type are classified as Damage Indicators (DIs), such as single-wide (DI 3) and double-wide (DI 4) manufactured homes. For each DI, there is a generally progressive list of damage descriptions known as degrees of damage, or DOD. Each DOD classification is associated with an expected wind speed, as well as an upper bound and lower bound wind speed, required to cause the observed damage. The resulting estimated wind speed for the given structure is determined by the quality of construction relative to the average quality. An above-average constructed building will result in an expected wind speed near the upper bound, while a poorly constructed building with the exact same damage may indicate a wind speed closer to the lower bound. For a given tornado, several structures and other damage indicators (such as trees) are used to determine an estimated wind field and resulting EF Rating. The EF Rating simply bins the overall expected wind speeds into 6 categories, EF0 – EF5. Table 4-1, Table 4-2, and Table 4-3 below show the DOD descriptions for single-family residences (site-built homes) (DI 2), single-wide manufactured homes (DI 3), and double-wide manufactured homes (DI 4), respectively.

Table 4-1. DOD's for Single-Family Residences (DI 2)

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

Table 4-2. DOD's for Single-Wide Manufactured Homes (DI 3)

DOD	Damage Description	EXP	LB	UB
1	Threshold of visible damage	61	51	76
2	Loss of shingles or partial uplift of one-piece metal roof covering	74	61	92
3	Unit slides off of block piers but remains upright	87	72	103
4	Complete uplift of roof; most walls remain standing	89	73	112
5	Unit rolls on its side or upside down; remains essentially intact	98	84	114
6	Destruction of roof and walls leaving floor and undercarriage in place	105	87	123
7	Unit rolls or vaults; roof and walls separate from floor and undercarriage	109	96	128
8	Undercarriage separates from unit; rolls, tumbles, and is badly bent	118	101	136
9	Complete destruction of the unit; debris blown away	127	110	148

Table 4-3. DOD's for Double-Wide Manufactured Homes (DI 4)

DOD	Damage Description	EXP	LB	UB
1	Threshold of visible damage	61	51	76
2	Loss of shingles or other roof covering (<20%)	76	62	88
3	Damaged porches or carports	78	67	96
4	Broken windows	83	68	95
5	Uplift of roof deck and loss of significant roof covering material (>20%)	88	75	108
6	Complete uplift of roof; most walls remain standing	93	77	110
7	Unit slides off CMU block piers	94	78	109
8	Removal of entire roof structure leaving most walls standing	97	80	117
9	Destruction of roof and walls leaving floor and undercarriage in place	113	93	131
10	Unit rolls, displaces, or vaults	114	82	130
11	Undercarriage separates from floor, rolls and tumbles, badly bent	127	109	145
12	Complete destruction of the unit; debris blows away	134	119	154

All of the above assessment techniques were utilized to compile useful structural damage information from several tornadoes onto Fulcrum to be processed and analyzed. In each tornado studied, the goal was to assess every affected structure along the damage path, including those with no visible damage. The results are presented and discussed in Section 4.3.

4.1.2. Observations from Each Local Tornado Assessment

A total of six tornadoes were studied to compile the data in this section, and a summary of each event can be seen in Table 4-4. Also, a figure showing the locations of each event can be seen in Figure 4-1. The statistics given in the table and the tornado paths shown in the figure were sourced from reports provided by the National Weather Service (NWS) local forecast offices. Each tornado was assessed following the procedures outlined in Section 4.1.1 above, with minor variations dependent upon the size of the event. The listed tornadoes were chosen for damage assessments due to the presence of damaged MMH. The following sections detail each assessment, outlining the methods and techniques utilized. Additionally, some general observations gathered from each event are briefly discussed.

Table 4-4. Summary of tornadoes assessed

Location	**Date**	**Path Length**	**Path Width**	**EF Rating**	**Injuries**	**Fatalities**
Lee County, AL	3-Mar-19	26.64 mi	1600 yds	EF-4	90	23
Carrollton, AL	11-Jan-20	6.33 mi	-	EF-2	-	3
Town Creek, AL	16-Dec-19	18.58 mi	370 yds	EF-2	3	2
Spring Hill, AL	6-Feb-20	1.29 mi	400 yds	EF-1	1	1
Troy, AL	14-Apr-19	2.81 mi	300 yds	EF-1	0	0
Kingville, AL	23-Feb-19	9.22 mi	700 yds	EF-1	0	0

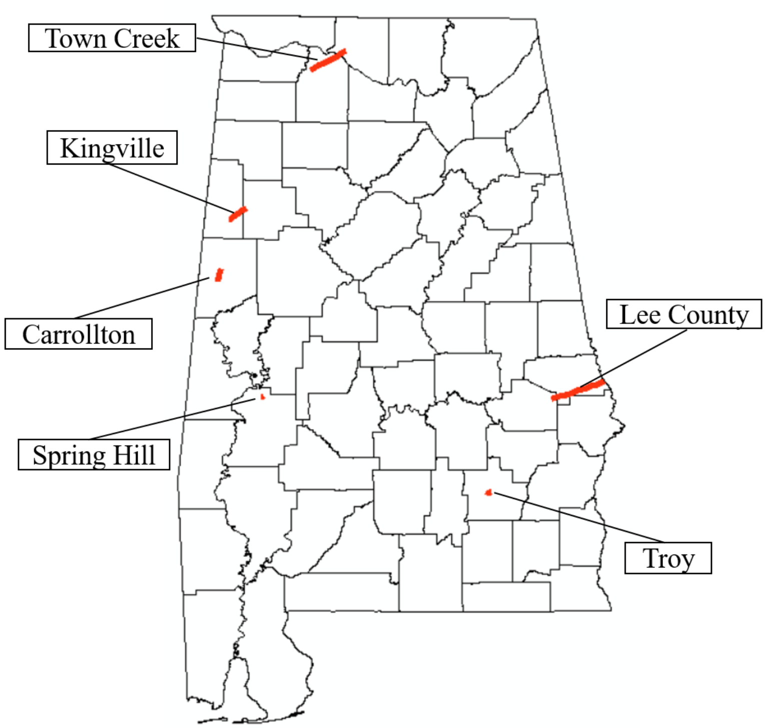

Figure 4-1. Map of tornadoes assessed

Lee County, AL

The largest tornado studied to date was an EF-4 tornado that hit Lee County, AL, on March 3, 2019. The tornado path extended into Georgia as well, but the majority of the damage it caused was in Alabama so that was the focus of the assessment. It significantly impacted the community, killing 23 people and injuring 90 others. Due to the significance of the event, every building throughout the damage path was assessed either remotely through the aerial imagery, through drive-by assessments, or through door-to-door assessments. However, this study will focus only on the results from MMH. Multiple trips were taken in the aftermath of the tornado to study the damage, and a total of 95 MMH were assessed through a combination of door-to-door and drive-by assessments. Additionally, a detailed forensic analysis was conducted on each MMH that experienced structural damage. These MMH make up the majority of the database for this section. The damage was initially mapped using aerial imagery, which was used to locate major areas of interest. These areas were then visited by teams of researchers to conduct door-to-door assessments. Overall, a number of different failure mechanisms were seen in MMH due to this tornado and are described below. While an MMH with any one of these failure mechanisms would be considered destroyed, the failures exist along a spectrum with respect to life safety.

A failure mechanism seen in one particular structure was separation at the marriage line. Figure 4-2 shows two perspectives of the same structure failing in this way. Overall, this particular failure is less of a life safety risk, as a large portion of the structure remained together. This failure is due to inadequate connections between the two halves, and was infrequently observed.

Figure 4-2. Failure at the marriage line

The least severe type of structural failure in MMH is the failure of the roof-to-wall connection, as it leaves the structure remaining upright and relatively safe. A picture of this type of failure can be seen in Figure 4-3. This type of failure occurs when the dead weight of the home and the anchorage systems provide adequate resistance of the uplift forces, but the connection between the roof trusses and the top plate of the wall cannot withstand the uplift forces. As seen in the left side of Figure 4-3, a consequence of losing the roof diaphragm is that the walls lose their top attachment point, and act as a cantilever, making them more prone to failure. Overall, this type of failure is typically not life-threatening to occupants and is preferred over the following failure mechanisms because the loss of the load-collecting elements in the roof also reduce the uplift loads acting on the building.

Figure 4-3. Failure of the roof-to-wall connection

The next critical point in the MMH load path is the wall-to-floor connection. This could occur if the roof-to-wall connection fails first, leaving the walls vulnerable, or it could occur simply due to an inadequate connection. Figure 4-4 shows a single home from different sides, illustrating the consequences of this type of failure. In a tornado, the stud walls experience a combined loading of lateral pressures and uplift. If the windward wall fails at the floor connection, the home will most likely be destroyed, placing the occupant in significant danger of being struck by debris.

The final and most critical component in the lateral load path is the anchorage system. As discussed in the post-hurricane MMH studies, anchorage failures are common under high wind loads, and the same issues were revealed in this tornado. There are different degrees of anchorage failure as defined by the EF Scale, indicating different wind speeds. The first and least destructive type of anchorage failure occurs when the MMH simply slides off of the piers, as seen in Figure 4-5. In this case, the anchorage system does not rupture, but deflects beyond what the piers can withstand. This may be due to ground anchor deflections as discussed in Section 3.2, or yielding of the straps. However, this type of failure is not life-threatening to the occupants, as the MMH remains intact.

Figure 4-4. Failure of the wall-to-floor connection

Figure 4-5. Sliding off the home off the foundation

The other two failure mechanisms of anchorage systems are much more catastrophic to the structure, and consequently to the occupants. The first is the rolling of the MMH, as seen in Figure 4-6. In this case, the anchorage fails, resulting in the lateral loads rolling the home in the windward direction. Sometimes the structure essentially remains impact as seen in the picture, but the debris within the home may put the occupant in danger. The worst type of failure for any MMH, however, is the lofting of the home. When winds are strong enough and/or of sufficient duration, the structure is lifted up and thrown, or lofted, in the windward direction. In most cases the home is completely destroyed upon impact, resulting in a low chance of survival for any occupants. The result of lofting can be seen in Figure 4-7, as evidenced by the chassis being wrapped around a tree several yards off of the ground.

Figure 4-6. Rolling of a MMH

Figure 4-7. Lofting of a MMH

Assessments following the March 3rd 2019 Lee County tornado found that many of the homes with traditional tie-down strap and ground anchor systems that experienced failure had pre-existing issues, such as corrosion or inadequate strap spacing. Some anchorage straps were corroded completely through, essentially leaving the home with no lateral support. Corroded straps from two separate homes can be seen in Figure 4-8. Another issue with these systems was ground anchor pullout, as described in Section 3.2. Again, failed anchors from two separate homes can be seen in Figure 4-9. The failure of another type of anchorage system, the pan system, often resulted in the rolling of the home. Failure of these systems can be seen in Figure 4-10.

Figure 4-8. Strap failure due to corrosion

Figure 4-9. Anchor pullout failure

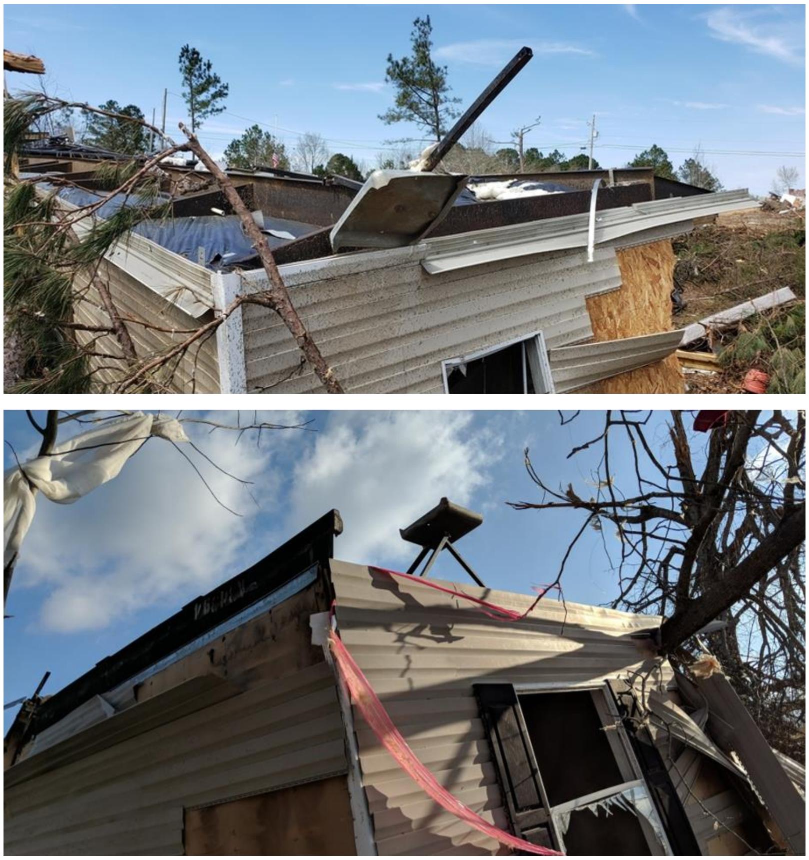

Figure 4-10. Failure of pan systems

Carrollton, AL

The tornado that struck homes along Settlement Rd. near Carrollton, AL, was rated an EF-2, and unfortunately it killed 3 people. A total of eight MMH were impacted, resulting in varying degrees of damage. One structure in which fatalities occurred was equipped with a traditional strap system, but it experienced a unique failure mechanism. The tie-down straps actually slipped out of the heads of the ground anchors, leaving the anchors still firmly planted in the ground. The heads of the ground anchors from which the straps slipped can be seen in Figure 4-11.

Figure 4-11. Evidence of straps pulling out of anchor heads

The other fatality occurred in a recently installed manufactured home that was placed on a concrete slab and had a variation of a pan anchorage system. The anchorage system appeared to have lateral load resisting struts spanning from the concrete slab underneath one I-beam up to connect to the other side I-beam, as well as longitudinal “V” struts spanning parallel to the I-beams. Figure 2-4 shows this basic layout, but with the struts connecting to a pan instead of a concrete slab. The structure was completely swept from the slab, leaving only two of the longitudinal struts still anchored into the concrete. The failure of the longitudinal struts appeared to have occurred in the connection to the I-beam rails as seen in Figure 4-12 (left), while the lateral strut was missing and appeared to have failed at the bolted connection to the slab as seen in Figure 4-12 (right).

Figure 4-12. Failure of pan system struts at the I-beam (left) and the foundation attachment (right)

Other anchorage issues leading to failure included corrosion of tie-down straps, large deflections of ground anchors, and complete pullout of ground anchors. Figure 4-13 shows evidence of all three of these issues found on different homes, and these were are all repeated from the Lee County damage observations. There were four MMH destroyed by the Carrollton tornado, and the failure of every single one of them was initiated by the anchorage system.

Figure 4-13. Corrosion failure (left), anchor deflection (middle), and anchor pullout (right)

Town Creek, AL

The tornado that struck homes near Town Creek, AL, was also rated an EF-2, but the only major impacts were to three MMH. The results were similar to those previously shown, in that all three homes were destroyed due to a failure of the anchorage system. The two fatalities occurred in a home that used a traditional strap and ground anchor system, which primarily failed due to anchor pullout. Of the 11 straps accounted for, 7 of them pulled the anchor out of the ground and 4 of them failed in tension. Another home that failed was a modular home that appeared to have no anchorage system. The third home that failed was a new manufactured home, less than a year since being installed, that had a pan anchorage system. The occupants were hospitalized with head injuries but were not killed. The frame of the home was resting several hundred yards from its original location, with one of the pans still attached. The other pan was near the destroyed frame, but must have torn loose during the movement of the frame. The still-attached pan, the detached pan, and the apparent connection failure point can be seen in Figure 4-14.

Figure 4-14. Failure mechanisms of a pan system

Spring Hill, AL

The EF-1 tornado that struck two MMH near Spring Hill, AL, was small but unfortunately resulted in a fatality. These two homes were both single-wide homes, installed parallel to each other approximately 40 ft apart. One of the homes had a Xi2 pan system and the other had a traditional strap system. The tornado struck the first home, which had a pan system, and lofted it towards the home behind it, which had a traditional strap system. The pan system home impacted the strap system home, causing the ground anchors on the windward side to pull out of the ground. Both homes were then lofted further and unfortunately destroyed. It is impossible to say whether the home with the strap system would have resisted the wind loads otherwise, but the impact from the other structure likely led to premature failure to some extent. Pictures of the failed pan system can be seen in Figure 4-15. Interestingly, one pan remained anchored in the ground and failed at the frame connection, while the other was pulled out of the ground upon failure. As for the strap system, several straps failed at the connection to the I-beam, which can be seen along with anchor pullout in Figure 4-16.

Figure 4-15. Failure mechanisms of a Xi2 pan system

Figure 4-16. Anchor pullout (left) and strap failure at the I-beam (right)

Troy, AL

This EF-1 tornado struck a MMH park, causing significant damage to several MMH. Fortunately, there were no reported injuries or fatalities. A total of 30 MMH were assessed, and the discrepancy in the damage ratings of individual homes that were located very close together was intriguing. Multiple homes were flipped, rolled, or destroyed, while homes directly next to them were essentially undamaged. This may have been partly due to the narrow path of the tornado, but another factor may have been the anchorage inconsistencies. Several of the assessed homes had anchorage systems that degraded over time due to corrosion and were never replaced. Figure 4-17 shows two strap failures resulting from corrosion on a single home, which was rolled upside down. The ground anchor on the right showed no signs of movement in the ground, indicating that the strap may have been completely corroded through.

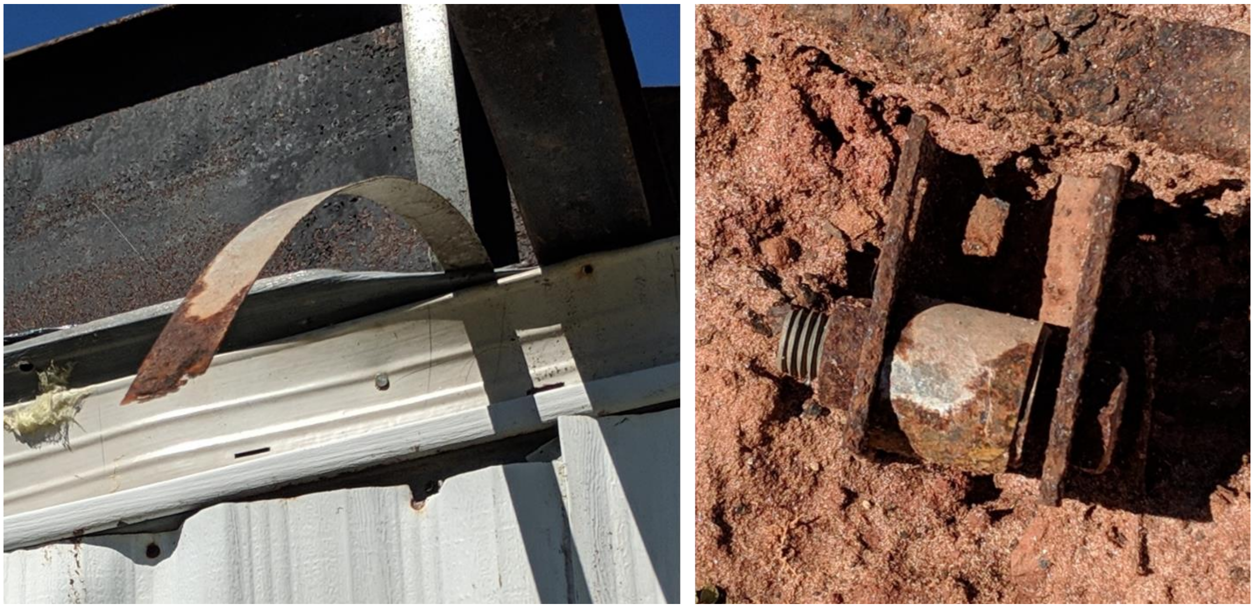

Figure 4-17. Corrosion induced strap failure

Another anchorage issue documented in Troy was the lofting of a pan system onto another home, similar to the case in the Spring Hill tornado. Additionally, the four traditional straps on the corners of the pan system home all pulled out of the ground. Pictures of the lofted home can be seen in Figure 4-18. The other home impacted also had a pan anchorage system, which appeared to have been shifted from its initial resting place. Figure 4-19 shows a picture of the impacted home's anchorage systems, with the red circle indicating the lack of a connection to the I-beam rail. This may have been a pre-existing condition, or it may have failed due to the impact from the lofted home. Finally, another lofted home with a pan system is shown in Figure 4-20. The picture also shows the devastation caused by lofting and impacting the ground. For reference, notice the home in the background with very little damage. Anchorage failure can lead to catastrophic failure of the structure as a whole, even in lower wind speed events.

Figure 4-18. Lofting of a pan system

Figure 4-19. Connection failure of an impacted pan system

Figure 4-20. Lofting of a pan system leading to destruction

Another concerning trend from this event was the failure of several anchorage systems resulting in the structure simply being rotated or slid several feet. While this failure mechanism is certainly preferable to the rolling or lofting of homes, it indicates that while wind speeds here were not strong enough to fail the structure, the anchorage system is the weakest link in the load path. Homes slightly outside the center of the tornado path experienced much lower wind speeds, but Figure 4-21 shows two separate homes that were rotated off of their foundations due to anchorage failure. Both homes were equipped with pan anchorage systems and most likely failed due to the inability to resist much uplift force. Additionally, the corner straps of the home shown on the right failed due to anchor pullout. T.P. Marshall (1993) and McDonald (1971) suggested that the twisting or rotating of a home occurs due to inconsistent resistances across the length of the home [8], [49], which could feasibly occur in pan systems and inconsistently installed strap systems. Twisting may also occur in small tornadoes where the diameter of the vortex is of a similar scale to the length of the home, resulting in an uneven loading across the length of the home.

Figure 4-21. MMH lifted and rotated off of foundations

Kingville, AL

Although the Kingville tornado path was over 9 miles long, it only struck a single MMH. The single-wide home was lofted approximately 40 ft from its original location, resulting in other connection failures as well. It had a traditional strap and ground anchor system, with ground anchor pullout being the primary failure mechanism, shown in Figure 4-22. A unique failure was seen in one of the pulled-out ground anchors, as the head appeared to have detached from the shaft. However, it most likely occurred after being pulled out, and can be seen in the right side of Figure 4-22. The strap system may have been more complex than usual, as several straps were attached to a single anchor head, which is shown in Figure 4-23. However, due to the destruction of the structure, the details of the system were unable to be documented.

Figure 4-22. Anchor pullout (left) and detachment of the anchor head (right)

Figure 4-23. Failed strap system with multiple apparent connections

4.1.3. Statistics and Results

Although several characteristics of each home were recorded during the field assessments, the focus of the following analysis is the anchorage information gathered. The anchorage failures were focused on due to the overwhelming evidence pointing to it as the primary failure mechanism in destroyed homes, as shown in Section 4.1.2 above. A total of 140 MMH were documented through the field assessments, but unfortunately not all information was available for each one. In homes that were not severely damaged, oftentimes the anchorage system was hidden behind the vinyl skirting or stem walls. Additionally, the drive-by assessments conducted after the Lee County tornado did not capture detailed anchorage information, although those were typically homes with little to no damage and located well outside the tornado center. However, every single structure assessed was assigned a DI and DOD. Of the 140 homes assessed, the anchorage system was identified in 65 of them, and the frequency of each major anchorage type is shown in Figure 4-24 (left). The right side of Figure 4-24 shows the distribution of damage for each anchorage type. The y-axis, "DOD Ratio", represents the ratio of the degree of damage assigned to the home divided by the maximum degree of damage possible for that particular home type. This was done to normalize the degree of damage, as single-wide and double-wide homes are rated on different scales. It is very important to keep in mind that these results do not account for the actual loading on the home, as some homes documented were much further from the tornado path and thus would be expected to have less damage.

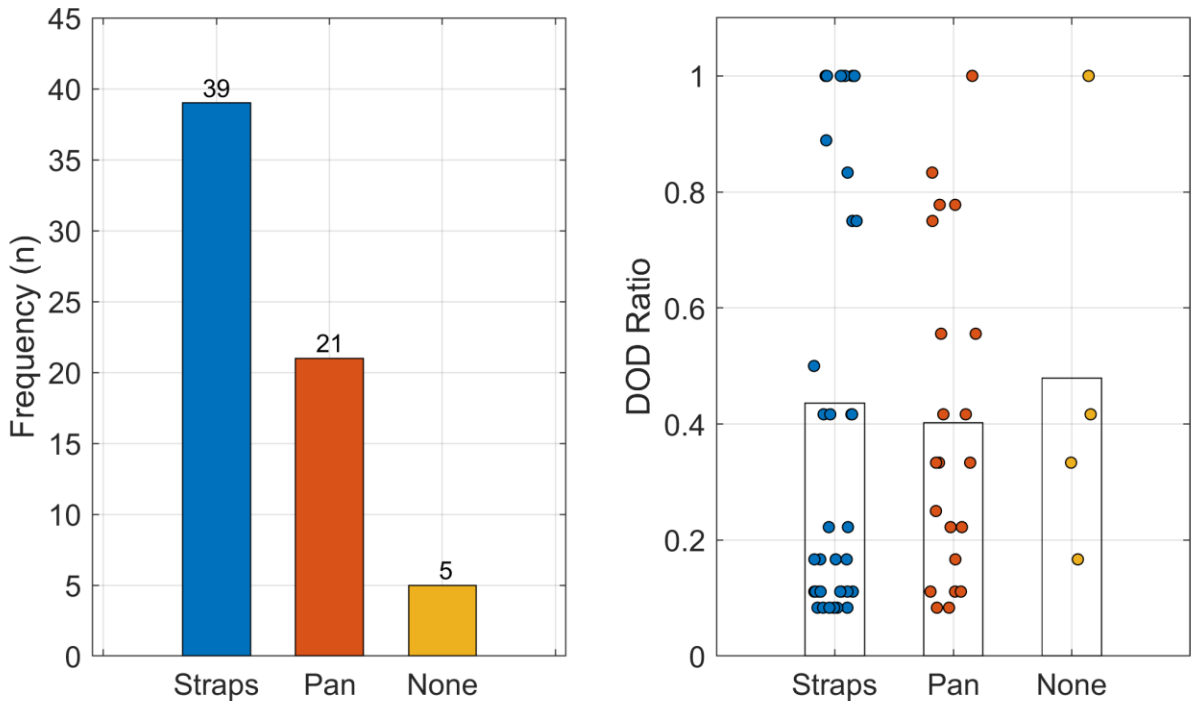

Figure 4-24. Anchorage type statistics (left) and damage distribution (right)

To reinforce the field observation that anchorage failure was the primary failure mechanism leading to destruction, each DOD was grouped into one of three categories: cladding damage, roof/wall structure damage, and anchorage failure. The specific grouping of the DOD's can be seen in Table 4-5. The cladding group included any damage that was not structural, such as roof or wall cover, broken windows, or even the loss of sheathing. The roof/wall structure group, as the name implies, included any DOD’s related to the failure of the structure above the floor frame. Lastly, the anchorage failure group included any damage description that implied the anchorage failed, such as the rolling or lofting of the home and the complete destruction with the debris blown away. The results can be seen in Figure 4-25. The most concerning issue appears to be the failure of the anchorage system, as it is more likely to occur than typical structural damage despite the fact that it should require higher wind speeds. It is also interesting to note that the same trend emerges in both single-wide and double-wide MMH, although the pattern is more severe in single-wide homes. This fact points to there being an overarching issue with anchorage system performance, not just with a specific home type or size.

Table 4-5. DOD's grouped into each failure mechanism category

	DOD’s Included	
Failure Mechanism	**Single-wide**	**Double-wide**
Cladding	1-3	1-5
Roof/Wall Structure	4, 6	6, 8-9
Anchorage System	5, 7-9	7, 10-12

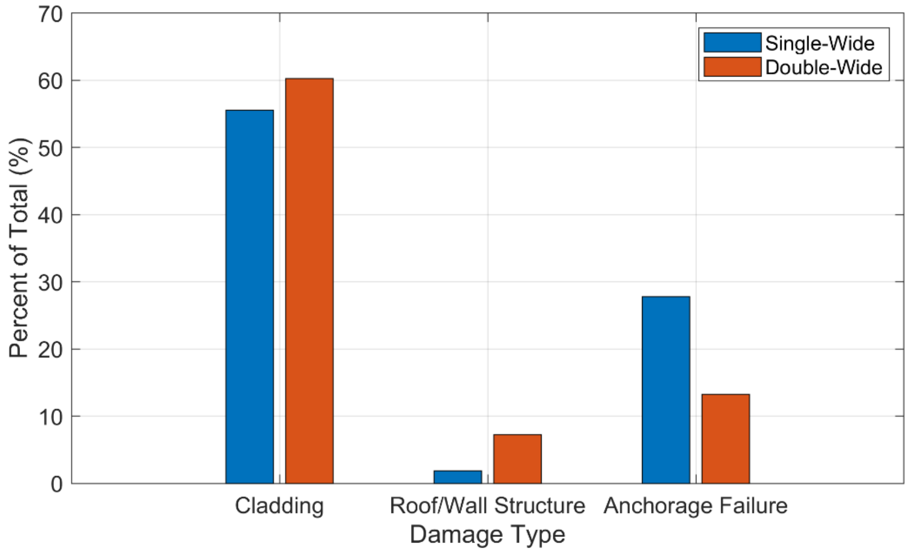

Figure 4-25. Failure mechanisms for MMH

In contrast, a MMH is typically destroyed from the bottom-up, with the anchorage failing first, and then everything else is destroyed as the structure rolls and/or is lofted. As seen in the field assessment pictures in Section 4.1.2 above, the failure of the anchorage system usually results in the destruction of the home. However, a top-down failure such as a failed roof-to-wall connection typically leaves the structure still standing, protecting the occupants within, and also reduces the wind loads acting on the structure.

Traditional tie-down strap and ground anchor systems are the most common type of anchorage system, making up 60% of the field data sample, and the HUD code outlines clear requirements for implementing this system. However, as seen repeatedly in the field assessment observations, these systems commonly fail under tornado loads. This indicates that there is a problem with either the code requirements or the actual installation of the anchorage system. As discussed in Section 2.2, several factors affect the HUD spacing requirements for the straps and ground anchors, including the width of the home and the maximum height off of the ground. Of the 39 MMH structures that had strap systems, 22 of them contained all of the data necessary to calculate the maximum allowed strap spacing, as well as the actual number of straps present. It was assumed that the I-beam rail spacing was 99.5 in., unless the code did not allow it for the given pier height, in which case the 82.5 in. I-beam spacing was used. This assumption was based on the author's field experience. These results are shown in Figure 4-26. The red line indicates the code requirements, with any data point above the line representing a home that meets those requirements. The color corresponds to the DOD ratio of the home, representing a normalized damage value. Of the 22 homes, 16 of them met code requirements (73%), and they mostly performed well. However, the sample is likely biased as it was more difficult to collect all the data necessary for this assessment for homes that were completely destroyed.

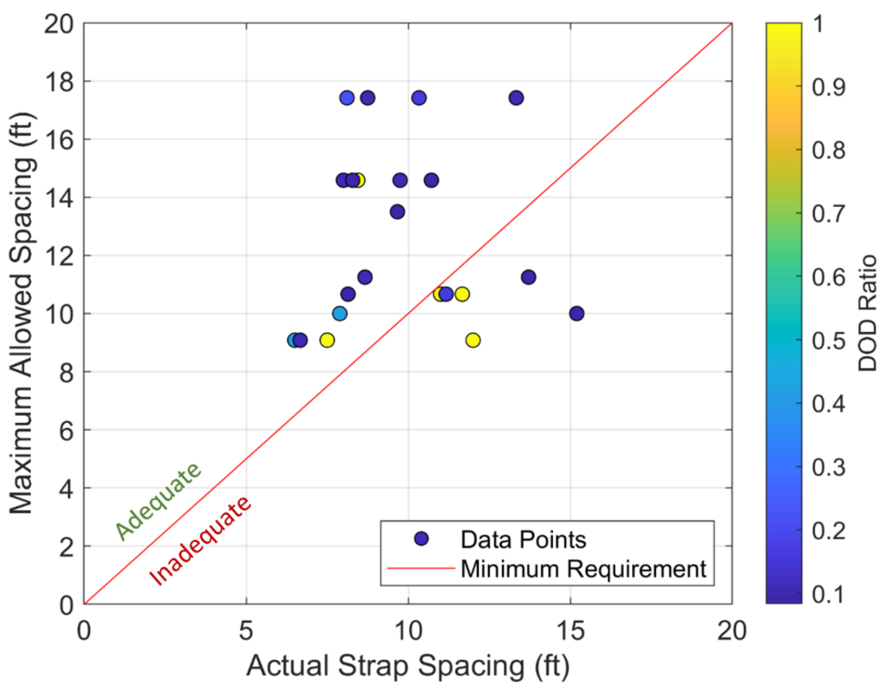

Figure 4-26. Comparison of required strap spacing to actual spacing

A more detailed analysis of the strap data can be seen in Figure 4-27, with the results separated by the homes that met the code requirements and those that did not. The data in the "Inadequate Straps" column includes the homes with too few straps calculated in Figure 4-26, as well as homes with no apparent anchorage system at all. Also, the damage results for pan anchorage systems are included for comparison. The color represents the EF rating assigned to the tornado that was responsible for causing the damage to each data point. Although the EF rating and DOD are highly correlated, the EF rating is determined for the tornado overall. Therefore, it gives a more complete picture of the tornado as a whole and aids in identifying anchorage systems that performed better or worse than expected. However, the distance from the home to the center of the tornado path is not accounted for, so the EF rating should only be used to put the overall event into context for each home.

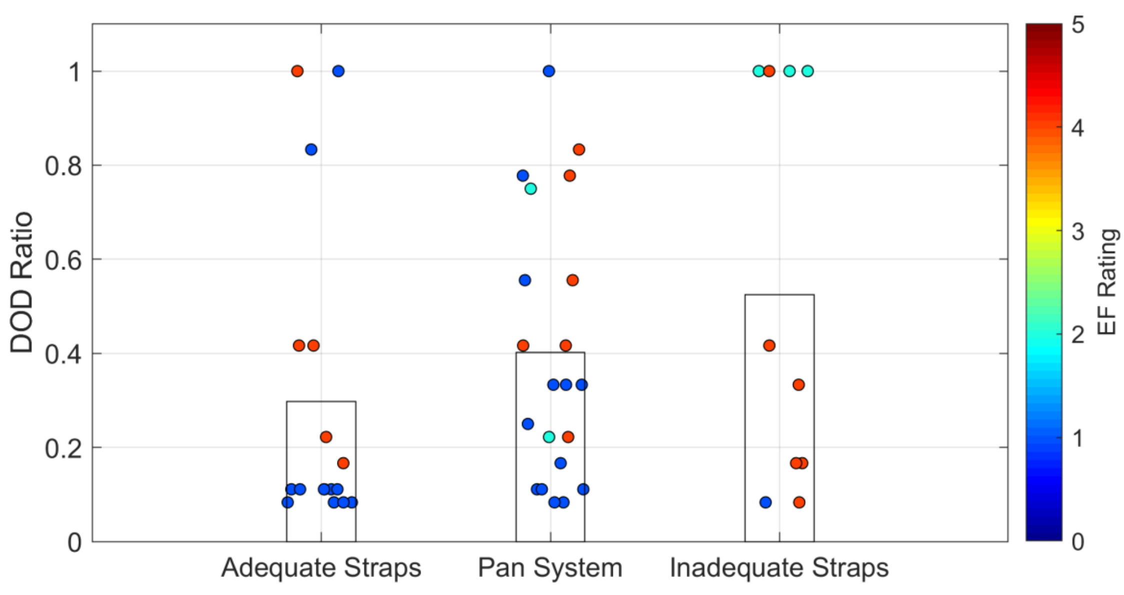

Figure 4-27. Comparison of damage ratios for strap and pan systems

Although Figure 4-24 above shows that strap systems and pan systems performed similarly overall, the more detailed analysis shown in Figure 4-27 indicates otherwise. On average, pan systems experience approximately a 10% greater DOD ratio than that of adequate strap systems. Additionally, the DOD ratio increase from adequate strapping to inadequate strapping is greater than 20%.

4.2. Quantitative Analysis of the NWS Tornado Database

After most tornadoes that occur around the country, the NWS sends a team to record the damage and estimate an overall EF rating. The purpose of each NWS deployment is to capture the characteristics of each tornado, such as path length, path width, maximum intensity, and any cycling intensity patterns if present. They are looking for damage indicators that can provide this information, and redundancy is not efficient. Therefore, they do not assess every structure in a given area, instead seeking out ones that provide the most helpful indicators of the upper and lower bounds of the wind intensity. This provides a few limitations to the dataset, as the assessed structures were most likely those experiencing the worst damage in a given area. This may lead to an overestimation of the average damage across all structures in a given event. However, the data sample is still useful for identifying failure patterns in the load paths of structures, and it can be used to quantitatively reinforce the trends discovered in the field assessments described in Section 4.1.

Since 2011, most NWS forecast offices publish this data through the Damage Assessment Toolkit, or DAT [50]. The DAT is a GIS-based web map that allows users to view and extract tornado paths, polygons and individual damage locations. Individual damage location data was compiled between 2011 and 2019, and filtered to include three types of structures: single-family residences (site-built homes), single-wide MMH, and double-wide MMH. The useful information recorded for each data point included the DI, DOD, location, EF Rating for the event, an estimated wind speed (based on the observed damage and surrounding context), the number of injuries, and the number of deaths. The filtered data totaled 16,667 homes, with just over 80% of them being site-built homes.

The DOD provides a specific damage rating for each home type and can provide insight into the overall failure mechanism of the structure. Figure 4-28 shows the DOD statistics for each home type. It is important to note that site-built homes have a maximum DOD of 10, while single-wide MMH have a maximum DOD of 9 and double-wide MMH have a maximum DOD of 12. An emerging trend for site-built homes is that the percentage of homes decreases as the degree of damage increases. This trend is logical, as higher wind speeds should be required to cause greater damage, and higher wind speeds simply occur less often. It also implies that a site-built home is typically destroyed from the top-down, with the roof failing first, then the exterior walls, and finally the interior walls and everything else. However, neither of the MMH types follow this trend. While the most common degree of damage for single-wide homes is minor roof cover damage, the second most probable damage state is the complete destruction of the unit (DOD 9). Even more concerning is the fact that the complete destruction of a double-wide MMH (DOD 12) is the most probable outcome in a tornado, with it occurring nearly 25% of the time. These trends highlight the most important difference in the failure patterns between site-built homes and MMH.

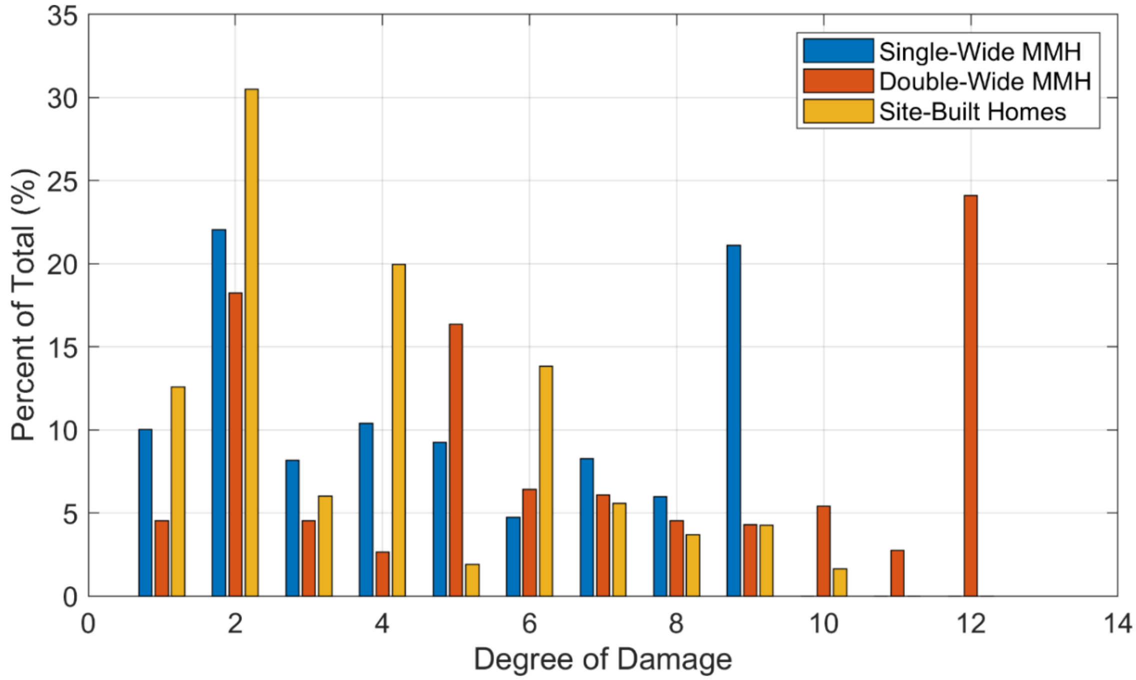

Figure 4-28. DOD distribution of each type of structure

The DOD distribution results are simplified in Figure 4-29, and it further illustrates the different failure mechanisms. The figure was created using the same parameters used to create Figure 4-25, with the addition of site-built homes. For the site-built homes (DI 2), DODs 1-4 were classified as "Cladding" damage, DODs 6-9 were classified as "Roof/Wall Structural Failure", and DODs 5 and 10 were classified as "Anchorage Failure". Both MMH type anchorage systems show a similarity to the field assessment data results in that the distribution of damage is concentrated at each extreme end of the damage spectrum. However, for site-built homes, the likelihood of anchorage failure is much lower than that of roof/wall failure. This reinforces the idea that MMH have a tendency to fail from the bottom-up while site-built structures typically fail top-down.

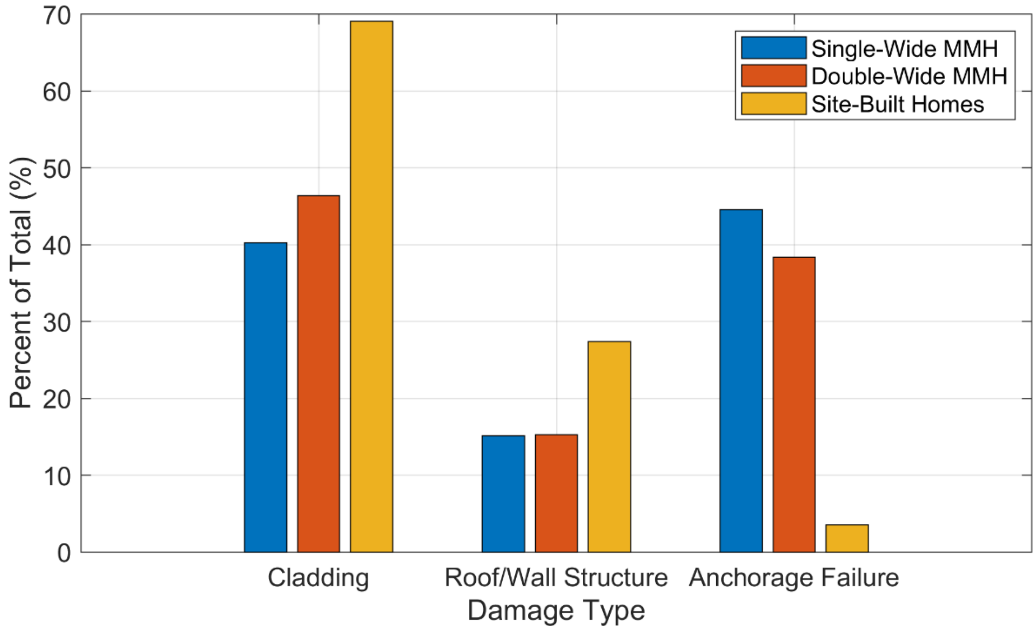

Figure 4-29. Failure mechanisms for MMH

The last analysis performed on the NWS data utilized the estimated wind speeds for each data point to create fragility curves in relation to anchorage performance. A fragility curve represents the probability that a specified damage state will be met or exceeded given a certain intensity measure. In this case, the damage state was binomial, with one representing anchorage failure and zero representing no anchorage failure. The intensity measure used was wind speed, divided into 10 mph increments ranging from 55-155 mph. The mathematical definition, taken from Porter (2017) [51], of this description can be seen as follows:

$$F_d(x) = P(D \geq d \mid X = x) = \Phi^{-1}\left(\frac{ln(x)-\mu}{\sigma}\right) \qquad \textit{Eq. 4-1}$$

where $P(A|B)$ is the probability that A is true given that B is true, D is the damage state for each individual record, d is the damage state limit that defines failure, X is the wind speed for each record, x is the particular wind speed group that is being evaluated, Φ^{-1} is the inverse normal CDF, and μ and σ are the expected value and standard deviation of the distribution.

In Figure 4-30, the data is plotted for both single-wide and double-wide MMH for comparison. The figure shows that double-wide anchorage systems typically can resist higher wind speeds than those of single-wide homes. For example, on average, a single-wide anchorage system will fail 50% of the time at an ultimate wind speed of about 97 mph, while a typical double-wide anchorage system will fail less than 5% of the time at the same wind speed. These results imply that single-wide and double-wide homes perform differently under similar wind speeds, challenging the fact that the MHCSS requires the same design for both types. Another important takeaway is that at 103 mph, a single-wide manufactured home anchorage system will fail approximately 70% of the time. This is alarming because HUD requires that these homes be designed to resist the pressures resulting from an ultimate wind speed of 103 mph. This issue is not quite as prevalent for double-wide homes, although they have the same design wind speed and still fail about 15% of the time.

While these findings are concerning, it must be remembered that the wind speeds shown in Figure 4-30 are only estimates based on damage, and can therefore be biased by perceptions of the wind speeds required to damage single-wide homes vs double-wide homes held by the NWS surveyors.

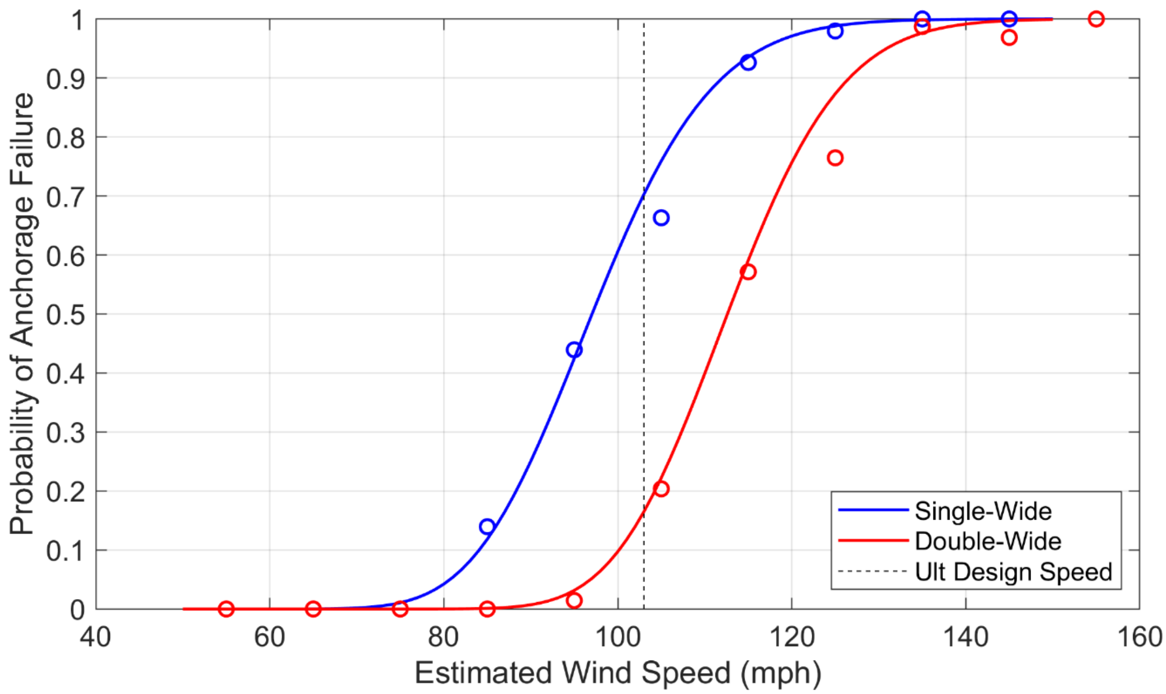

Figure 4-30. Fragility curves for each MMH type using data from the NWS DAT

4.3. Increased Fatality Risk of MMH in Tornadoes

The injury and death statistics from the NWS data are important information to analyze, as their sociological affects can be devastating. Table 4-6 shows the breakdown of this data for all tornados documented in the DAT since 2008. The differences in the probability of an injury or fatality occurring in a MMH as compared to a site-built home are significant. Even more concerning is the trend as the tornado strengthens, as shown in Figure 4-31. It shows the likelihood of a fatality occurring in each of the three building types for each EF rating. As expected, the likelihood is very low for an EF-0 and even for an EF-1, but the chance of a fatality occurring in either type of manufactured home greatly increases for an EF-2 or greater tornado. In comparison, the fatality risk in a site-built home remains relatively low even as the tornado strength increases.

It is important to keep in mind that this data does not include all of the homes affected by the tornado or even all of the fatalities if data for a particular tornado is not uploaded to the DAT by the NWS office, but the diverging results between the structure types do illustrate trends in performance. These results reinforce the idea that MMH present an enhanced risk of injury or fatality for any occupants during a tornado.

The main reason for this increased fatality risk lies in the pattern of failure. As previously discussed, site-built homes fail top-down, while manufactured homes typically fail bottom-up. The main issue with a home failing bottom-up is that the structure, which functions to protect the occupant from debris and the force of the winds, is destroyed very quickly. The occupant is then in danger either from the destruction or exposed to the tornado winds and debris. In the case of a site-built home, even if the structure fails, the interior walls will most likely remain standing to provide protection from debris. Also, the progressive failure of the structure gives the tornado time to pass by before the occupant is fully exposed.

Table 4-6. Injury and fatality statistics from the NWS DAT

Home Type	**Likelihood of Injury**	**Likelihood of Death**	**Number of Homes with an Injury**	**Number of Homes with a Death**	**Total Number of Homes**
Single-Wide	4.96%	1.78%	117	42	2357
Double-Wide	3.76%	1.88%	34	17	904
Site-Built	0.97%	0.47%	130	64	13406

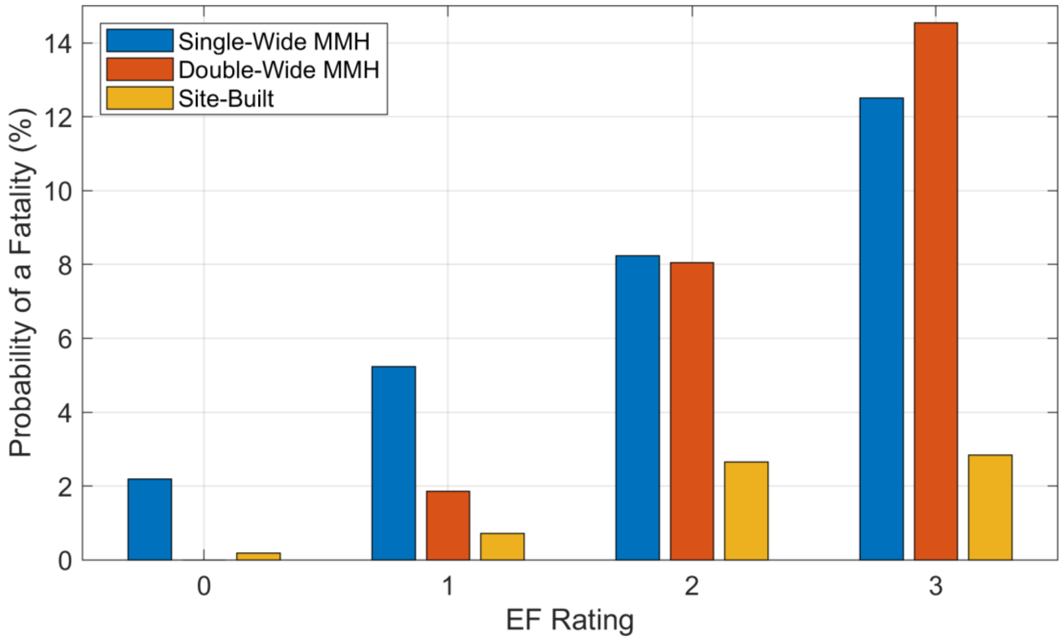

Figure 4-31. Probability of a fatality occurring in each structure type at increasing EF ratings

4.4. Key Observations and Findings

Overall, three main observations can be highlighted from the combined qualitative and quantitative studies. The first observation is that the large sample of MMH from the DAT followed the same damage pattern highlighted in field study observations discussed in Section 4.1, indicating that MMH are more likely to fail due to anchorage failure than roof/wall structure collapse. This points to a bottom-up failure sequence, increasing the risk of fatality for anyone sheltering inside. While this data is not new, these results from recent tornado events indicate that improvements have not been made in regard to this issue for *Wind Zone 1* homes.

Second, there seems to be an issue with strap anchorage systems being inadequately installed and/or maintained. Of the applicable homes, only 73% met the strap spacing requirements of HUD, and several more had significant corrosion on the straps. This issue is only amplified by the results showing the discrepancy in performance between the adequately installed systems and the inadequate ones. Strap systems that do not meet HUD requirements experience, on average, over 20% more damage than those installed properly. Many of the homes with inadequate systems may have originally been installed correctly, but over time corrosion destroyed the integrity of the straps, sometimes fully disconnecting the ground anchor from the frame. Corrosion was a reoccurring issue seen in the field, present in over 41% of homes with strap anchorage systems. Usually if corrosion is found on one strap, it is most likely an issue on several straps due to the similar environmental conditions. However, even if only one strap is affected by corrosion, the integrity of the anchorage system as a whole is at risk. The results of corrosion are explored further in Section 5.

The last observation from the field assessment data compares the performance of strap and pan anchorage systems. As seen in Figure 4-27, properly installed strap systems typically outperform pan systems during tornado wind loading. Although several factors contribute towards these results, the main factor is most likely the capacity of each system to resist uplift. Strap systems are designed to resist uplift independent of the home's weight, while basic pan systems have little to no uplift resistance. Furthermore, pan systems actually rely on the downward force from the home to maintain a high static friction between the pan and the soil. This frictional force works to resist lateral loads applied to the home, such as wind. Consequently, if a large enough uplift force decreases the static friction between the pan and soil, the lateral resistance of the home is also decreased. Tornadoes are able to produce peak wind speeds well above design, exerting enough uplift to compromise the lateral resistance of the pan system. The combined lateral and uplift forces can then loft or roll these homes, even in weaker tornadoes. Additionally, strap systems (if not compromised due to corrosion) typically have some inherent plastic capacity as the straps deform, dissipating more energy. However, pan systems would behave as a very brittle system, with little to no plastic deformation once uplift forces decrease the lateral resistance. More research and field data is needed, but the discussed failure patterns do appear in the data thus far.

5. Tie-Down Strap Testing

With the importance of the performance of MMH anchorage systems highlighted in the previous section, there is a need to ascertain the specific cause of the disproportionate failures observed in anchorage systems. Previous testing has already highlighted the inadequate capacity of the ground anchors, but the in-situ performance of the tie-down straps has not been studied. Actual capacities are necessary to modeling the system performance of these structures.

Oftentimes in the wreckage of a MMH destroyed by a tornado, portions of the anchorage system can be found. Certain anchorage systems, such as pan systems, are difficult to evaluate or test in their damaged state, but others involving metal tie-down straps can be useful. While some straps in the debris are mangled and useless, many quality sections can be found either still attached in the ground or to the structure frame. The field assessment team collected many of these straps from the debris of destroyed manufactured homes with the purpose of performing tensile tests on them. A total of 147 strap sections from 12 different locations were collected and tested, and a breakdown of the data can be seen in Table 5-1. The procedure for testing each specimen is described in Section 5.1, while the results are presented in Section 5.2.

Table 5-1. Summary of strap samples collected

Group ID	Event	Estimated Age[1]	Samples Collected	Straps Fractured	Anchor Pullouts	Total Specimens
1	Lee County	1991	9	7	2	21
2	Lee County	1995	3	3	0	9
3	Lee County	2003	3	3	0	11
4	Lee County	2000	6	2	4	18
5	Lee County	1997	2	0	2	5
6	Lee County	N/A	1	0	1	2
7	Town Creek	1983	8	4	4	21

8	Town Creek	2019	4	1	3	13
9	Carrollton	1995	6	3	3	10
10	Carrollton	2014	4	3	1	12
11	New Straps	2019	1	0	0	10
12	New Straps	2020	1	0	0	15
[1]Provided by county property assessment offices, representing the year the home was constructed in some cases and other in other cases representing the year it was purchased.						

5.1. Testing Procedure

The metal straps were tested in accordance with ASTM D3953-15, the *Standard Specification for Strapping, Flat Steel and Seals*. The tensile tests were performed using an Instron Tensile Testing machine in the Center for Polymers and Advanced Composites laboratory. There were a total of 46 full sections of straps collected, plus 2 new rolls of strapping purchased from separate companies for control purposes. The full sections of straps were then cut into 14 in. sections to meet the testing requirements, resulting in a total of 147 test sections. The 14 in. length allowed for 3 in. on each end to be clamped, a 6 in. gage length centered between the clamps, and a 1 in. spacing between the edge of the each clamp and the gage length section. Before testing, each test section was labeled and measured, and notes were made about any corrosion issues or other defects present. Each strap was 1.25 in. wide, which is standard, but the thicknesses varied. Each strap's thickness was measured at multiple locations along the length using a caliper accurate to the 5/10,000th of an inch, and the individual measurements were averaged together to calculate the thickness. The minimum thickness allowed by HUD is 0.035 in., but tie-down straps are typically manufactured in 0.002 in. intervals, so the test sections were typically 0.036, 0.038, or 0.040 in. thick.

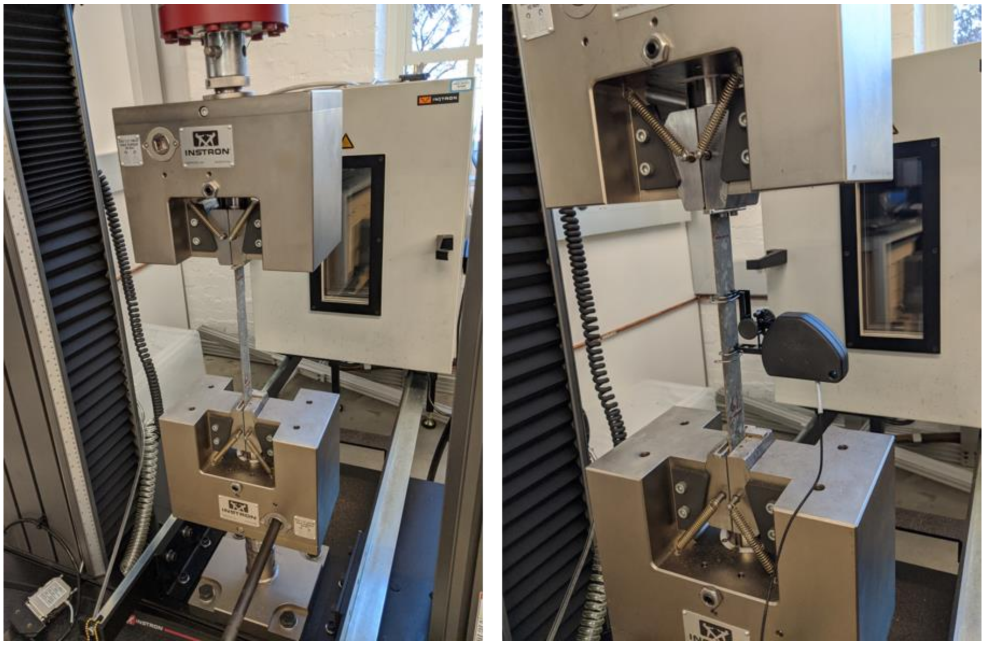

Figure 5-1. Strap testing setup prior to loading

Once all the test sections were documented and labeled thoroughly, each was tested in tension until failure. The testing process first required zeroing the force value and then placing the strap into the testing machine clamps and tightening them. Next, the extensometer was attached to center of the strap gage section. It provided strain information to the machine and indicated when the yield point had been reached and can be seen in Figure 5-1 (right). Before the testing began, the displacement and strain values were zeroed. The tensile force was then applied at a displacement rate of 2.5 in. per minute, per ASTM D3953-15. Once the extensometer indicated the yield point had been reached, the test was briefly paused and the extensometer was removed. This short pause caused the small dip in the test data seen in Figure 5-3, but did not affect the overall results. The testing then resumed until the strap failed, which was defined as the complete fracturing of the cross-section. Figure 5-2 shows a test section before and after failure in the testing machine. The machine recorded the force and displacement of the clamps over time and indicated the breaking point.

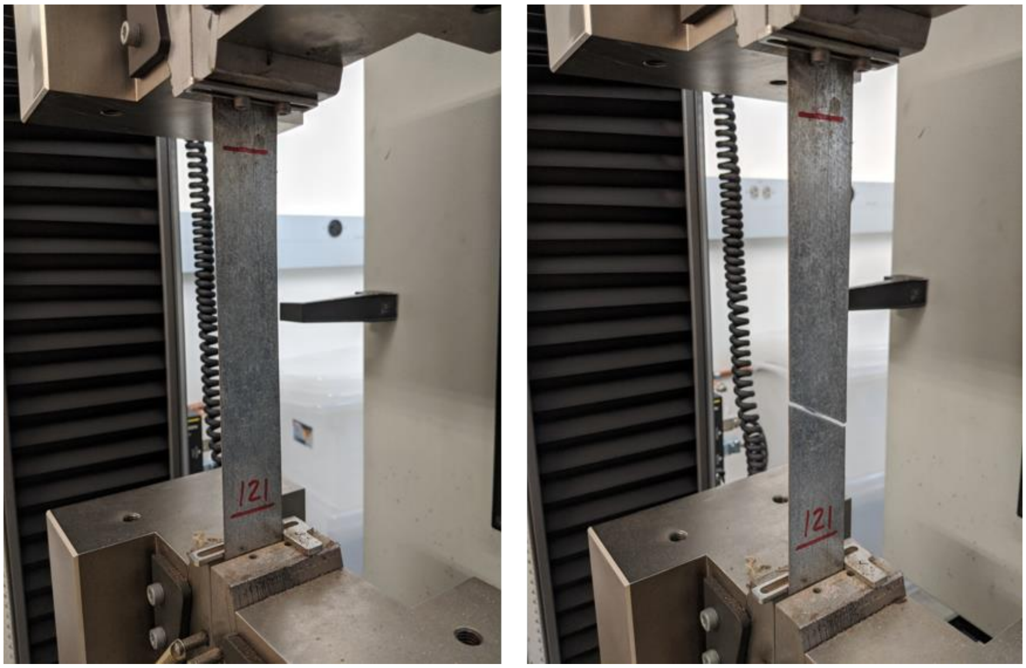

Figure 5-2. Before and after pictures of a strap tested to failure

5.2. Testing Results

The test results were grouped by the individual homes they were collected from to keep specimens of the same age and environment together. The results reinforced these groupings, as straps from the same location performed very similarly in terms of the ultimate failure capacity. In total, there were 12 groupings, including 10 collected from tornado-stricken homes and 2 consisting of new straps. The Instron testing machine reported the results in terms of the tensile force applied and the displacement of the clamps. A typical report from one group of 11 straps can be seen in Figure 5-3.

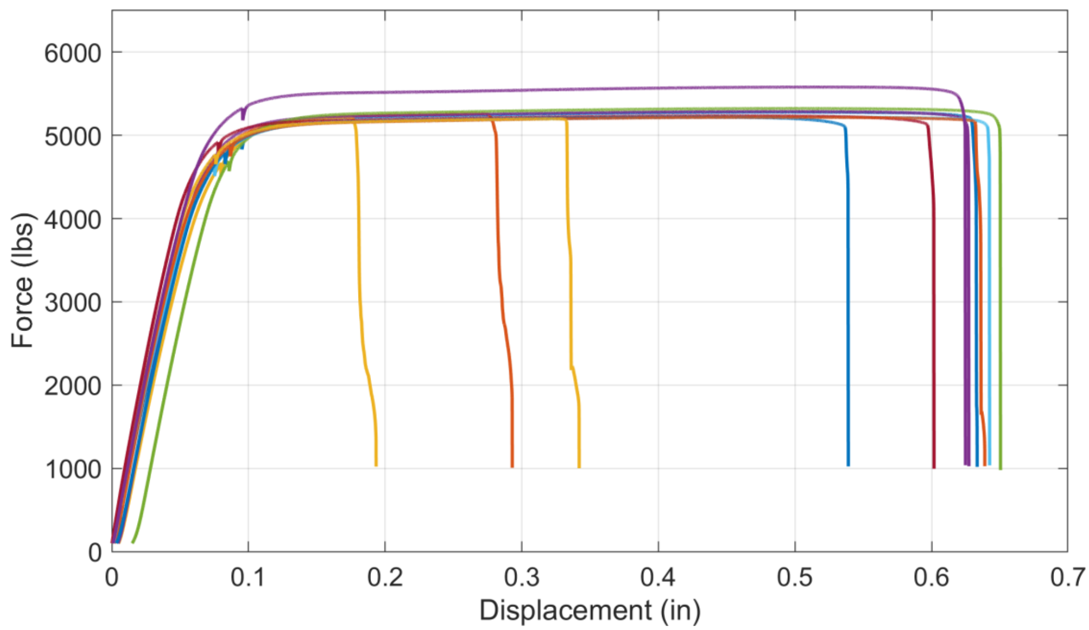

Figure 5-3. Force vs. displacement for a single strap grouping

This data was collected for each of the 12 groupings and analyzed. The most straightforward variables extracted for each strap were the thickness and the maximum tensile force, or the breaking force. These two variables are compared in Figure 5-5, along with the expected values from ASTM D3953-15. The different colors simply represent the groups of straps from separate homes, while the "x" markers represent straps that had defects. These defects included corrosion and significant kinks or bends in each strap that could potentially affect the structural integrity of the strap, and examples of them can be seen in Figure 5-4. The ASTM expected breaking strength values are listed in a table corresponding to specific strap thicknesses, so interpolation was used to expand it to all applicable thickness values. However, the HUD requirements simply state that each strap must withstand 4,725 lbs of force, regardless of thickness. Therefore, although some data points fall below the ASTM expected line, the ones exceeding 4,725 lbs still meet HUD requirements.

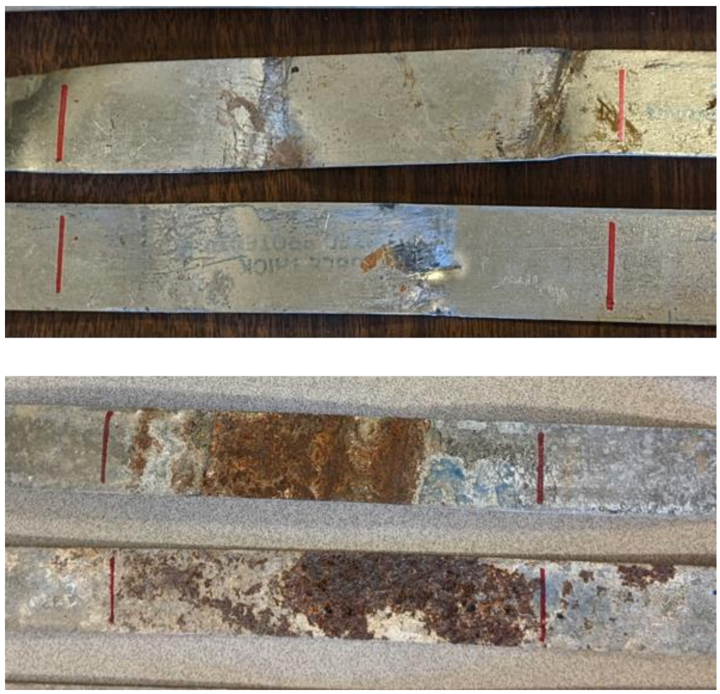

Figure 5-4. Examples of defects such as sharp kinks (top) or corrosion (bottom)

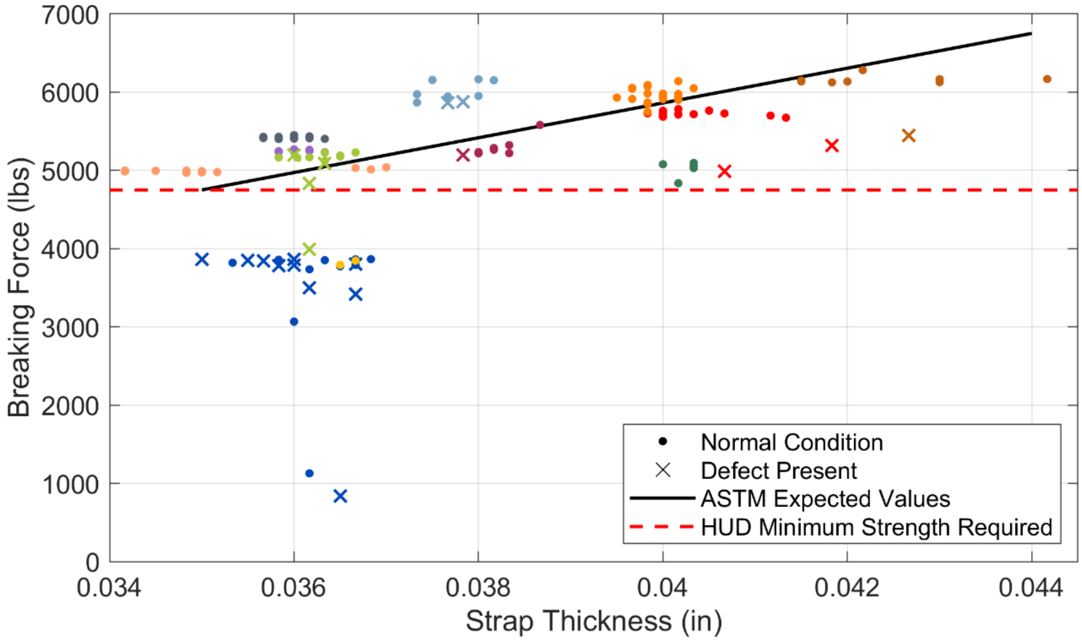

Figure 5-5. Comparison of expected strap breaking force against actual breaking force

Next, using the breaking force and the cross-sectional area, the maximum stress for each strap was calculated. Also, the force-displacement plots were converted to stress-strain curves and the modulus of toughness was determined for each. The modulus of toughness (MOT) is represented by the total area under the stress-strain curve and was calculated using the midpoint rectangle numerical integration method with a Δx value of 0.0001. The resulting MOT value is important because it factors in both strength and energy dissipation. Plasticity is an important characteristic of tie-down straps, as straps with a higher plastic capacity are able to dissipate a greater amount of energy prior to fracturing. In addition to those four factors, the age of each strap grouping was estimated using the year built for the home found in the county records database for the corresponding home. These average result of each of these factors can be seen in Table 5-2.

Table 5-2. Mean test results for each strap grouping, including new straps

Group	Age of Structure	Thickness (in)	Breaking Force (lbs)	Stress (ksi)	MOT (ksi)	Number of Samples
1	Unknown	0.0366	**3820**	83.5	4.14	2
2	1983	0.0399	5964	119.4	5.91	21
3	1991	0.0361	**3483**	77.2	3.04	21
4	1995	0.0424	6081	114.7	8.12	9
5	1995	0.0377	5985	127.0	7.41	10
6	1997	0.0402	5018	99.8	7.07	5
7	2000	0.0404	5657	112.1	6.55	18
8	2003	0.0382	5276	110.6	8.56	11
9	2014	0.0353	4996	113.5	11.39	12
10	2019	0.0363	5056	111.5	10.64	13
New 1	2019	0.0360	5245	116.6	14.36	10
New 2	2020	0.0360	5415	120.5	14.13	15

The frequency distribution for each factor is shown in Figure 5-6, combining data from all homes together. The highest frequency of thickness is concentrated just above the minimum HUD requirement of 0.035 in., which is expected. Also, aside from the two strap groupings that tested well below the 4,725 lb requirement, the majority of straps met or surpassed the required breaking strength. The cause of poor breaking strength for the weak strap groups is unknown, as every single strap failed below 4,000 lbs, regardless of defects. The lack of a more significant sample size prohibits determining if the weak groups are actually outliers, or if they simply appear to be so because of incomplete data. Regardless, the results confirm the possibility of homes anchored with straps of inadequate strength.

The maximum stress is dependent upon the breaking strength, so it also shows a small peak to the left of the main concentration of values. However, the modulus of toughness shows a very wide distribution of values, ranging from slightly greater than 0 to approximately 15 kips per square inch (ksi). This illustrates a high degree of variability, indicating that it may change with time or other factors.

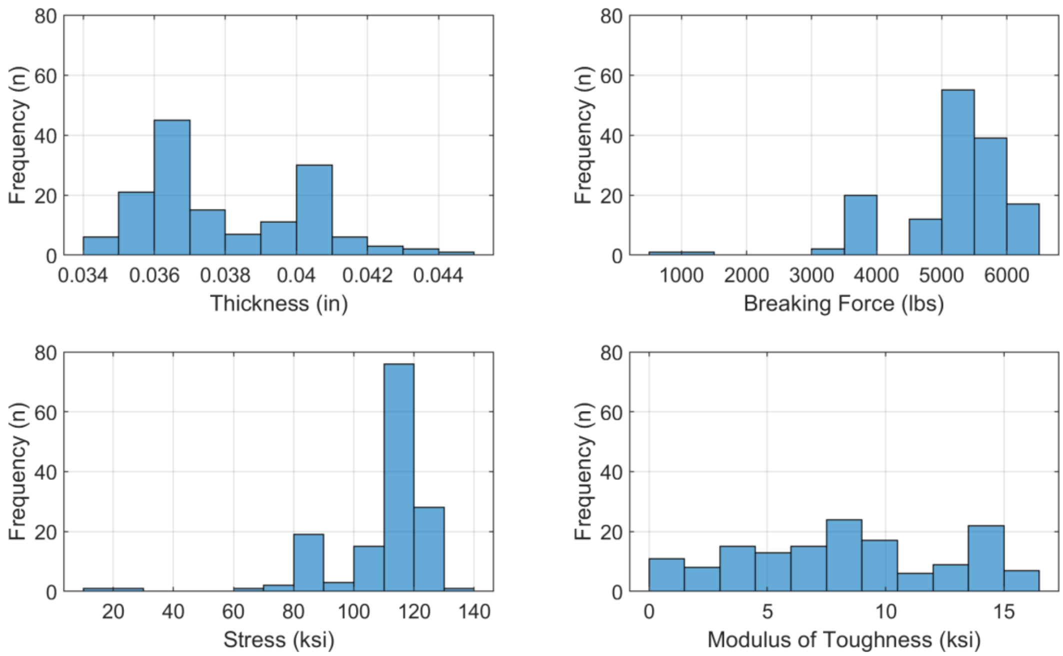

Figure 5-6. Histograms of all tested data divided into thickness, breaking force, stress, and MOT

These same four main variables were also plotted against the age and are shown in Figure 5-7. The hypothesis was that the strength of each strap would be negatively correlated with age due to corrosion and wear, but the results indicate otherwise. The stress, which represents a breaking strength normalized by thickness, remains very consistent over time, aside from the weaker grouping previously discussed. However, the modulus of toughness does indeed appear to decrease over time, which indicates that tie-down straps may lose a portion of their plasticity over time. Combining these two observations implies that straps typically become very brittle over time due to wear and/or corrosion, but they do not lose their ultimate strength capacity.

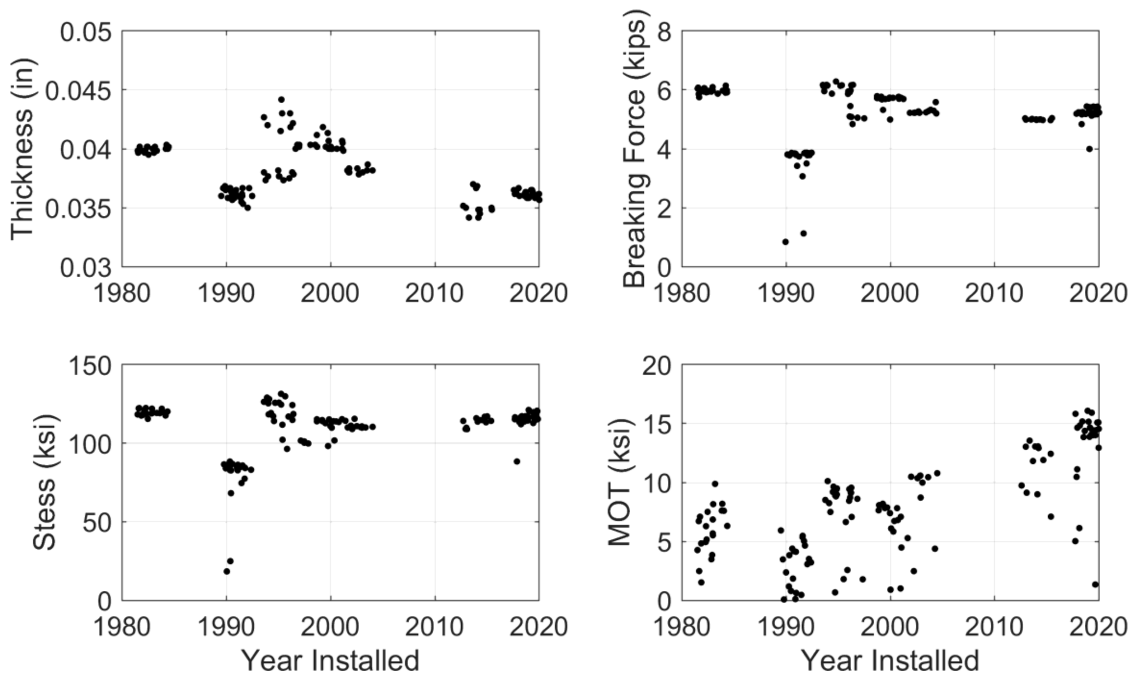

Figure 5-7. Thickness, breaking force, stress, and MOT compared against estimated age

A correlation test was performed on the data to determine the significance of the relationship between MOT and age. Using linear regression, an R-value of 0.72 was calculated. Next, using a T-distribution test, the T-statistic value was calculated to be 12.39. This, along with the degrees of freedom for the sample, was used to calculate the P-value, which was < 0.05. This insinuates there is a significant correlation between MOT and age, and the best-fit line is shown in Figure 5-8 (left). Additionally, the equation for the line is listed in the top left of the plot, showing a slope value of 0.241 with a 95% confidence interval ranging from 0.203 to 0.279. Essentially, the modulus of toughness decreases from its original value at a rate of approximately 0.241 ksi every year that it is in use. This reinforces the idea that anchorage straps should be replaced as they age due to their decrease in plastic capacity.

One may assume that the decrease in plasticity is directly tied to corrosion over time, but Figure 5-8 (right) seems to dispute this. The plot simply shows the likelihood of corrosion existing on any given strap of a specific age and is calculated by dividing the number of straps with corrosion by the total number of straps in each group. While the likelihood of corrosion does increase over time, older strap groupings with no corrosion still do exhibit lower MOT values. This implies that the overall decrease in MOT over time is consistent regardless of the presence of corrosion.

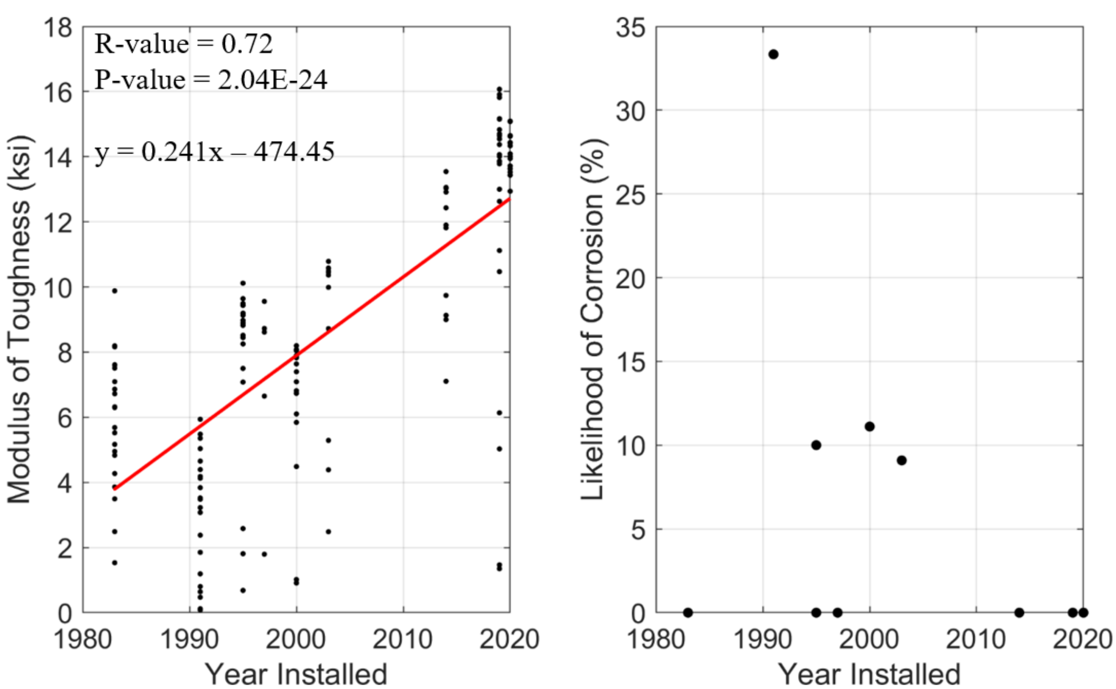

Figure 5-8. Statistical test of correlation between MOT and age (left), and the influence of strap corrosion on that correlation (right)

Although heavy corrosion is not the only factor causing this decrease, it certainly exacerbates the issue. As evident from Figure 5-8 (right), four of the strap groupings exhibited significant corrosion, while two other groupings had straps with other defects present. The direct effects of these defects are shown in Table 5-3. Surprisingly, the ultimate breaking force is not greatly affected by the defects, although 3 of the 6 groupings experienced a 7.5% or greater loss of strength. However, the MOT is extremely sensitive to the presence of defects. Of the 6 strap groupings, 4 of them lost over 64% of their MOT capacity, with 2 of those over 85%. The effects of corrosion can also be seen in the fracture mechanism of the straps, shown in Figure 5-9. The top two straps in the picture had no corrosion and fractured at an approximate 45 degree angle after elongating, similar to the majority of the straps. However, the two straps with corrosion fractured jaggedly across the width, indicating a non-uniform cross-section. This was a common issue with all of the heavily corroded straps.

Table 5-3. Effects of defects on breaking force and MOT

Group ID	**Mean Breaking Force with Defects (lbs)**	**Mean Breaking Force w/out Defects (lbs)**	**Percent Decrease**	**Mean MOT With Defects (ksi)**	**Mean MOT w/out Defects (ksi)**	**Percent Decrease**
1	3455.3	3508.6	1.52%	2.815	3.246	13.27%
2	5446.5	6160.3	11.59%	0.684	9.047	92.44%
3	5198.1	5284.3	1.63%	5.289	8.882	40.45%
4	5153.5	5720.4	9.91%	0.964	7.244	86.69%
8	4777.5	5180.3	7.78%	4.742	13.267	64.26%
9	5867.6	6014.7	2.44%	2.196	8.708	74.78%

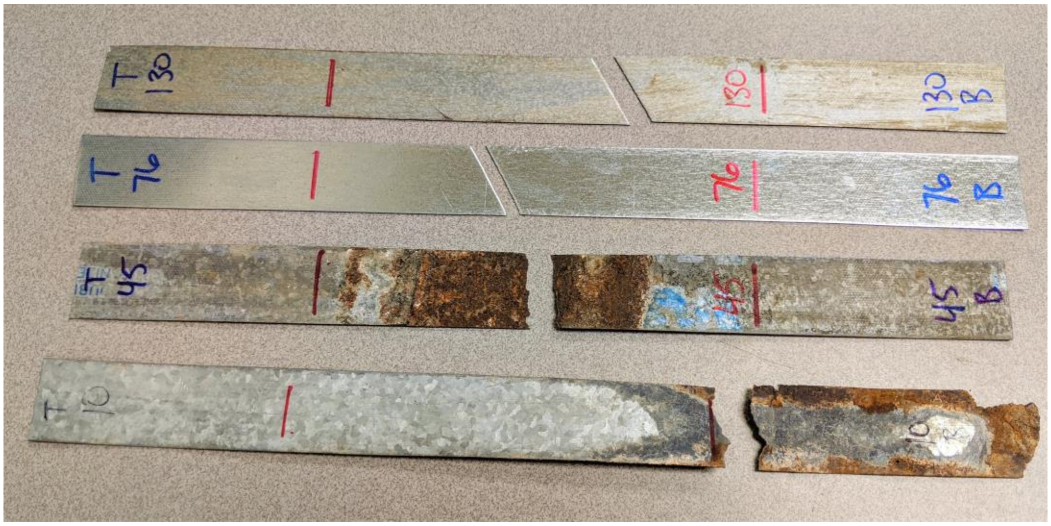

Figure 5-9. Comparison of strap failure between no-defects and corrosion

A strap's loss of plasticity over time is a significant issue, as the modulus of toughness represents the total amount of energy that can be dissipated prior to fracture. As seen in Figure 5-9, a strap continues to elongate once it reaches its ultimate strength in order to continue dissipating energy. Although manufactured home anchorage systems are designed to equally distribute the loading among the straps, the actual results are never perfect. Under design-level loads, a structure may distribute a higher force into one strap than the rest, causing that strap to exceed its capacity. A strap with sufficient plasticity would begin to deflect, causing the load to be partially redistributed into other straps. This at least enables the structure to resist the loading for a longer amount of time and may ultimately prevent the strap from completely fracturing. However, if the strap was very brittle, it would fracture shortly after reaching its ultimate capacity. The load originally resisted by the fractured strap would be redistributed to other straps, overloading and fracturing them as well. Even if the structure failed in both scenarios, the added time required to cause failure in the plastic-strap case could possibly prevent injury or fatality for the residents.

5.3. Fitting a Distribution to the Results

The results for stress and MOT are independent of thickness, so a distribution can be fitted to the data that will capture the performance of all straps regardless of size. However, given the small sample size of each strap grouping, each was expanded using Monte Carlo simulation methods. First, a kernel density function was used to model the actual results for stress and MOT within each strap grouping. Kernel density estimation can be used to calculate the probability density function (PDF) of a given random variable in a non-parametric fashion using the equation shown below [52],

$$f(x) = \frac{1}{nh} \sum_{i=1}^{n} K\left(\frac{x - X_i}{h}\right) \qquad \textit{Eq. 5-1}$$

where h represents the smoothing parameter, or bandwidth, and n is the number of samples. The equation used to calculate the optimal bandwidth is as follows [52]:

$$h = 1.06\, \sigma\, n^{-1/5} \qquad \textit{Eq. 5-2}$$

where σ is the standard deviation of the sample. Next, a Monte Carlo simulation was performed to increase the sample size to 1,000 points for each strap grouping by sampling from the kernel PDF. Figure 5-10 shows the before and after results of this expansion for a single strap grouping and property. One important note is that the new straps were not included in this analysis, as they were never exposed to environmental conditions or pre-loading of any kind. Also, the strap group with only two data points in it was excluded as well due to the lack of sufficient data to fit even a kernel density function. Overall, 9 strap groups were expanded to 1,000 data points each, totaling 9,000 data points to fit a distribution to. The primary reason for expanding the data points to 1,000 was to ensure each group contributed equally to the overall results, given that varying numbers of samples were taken from each home.

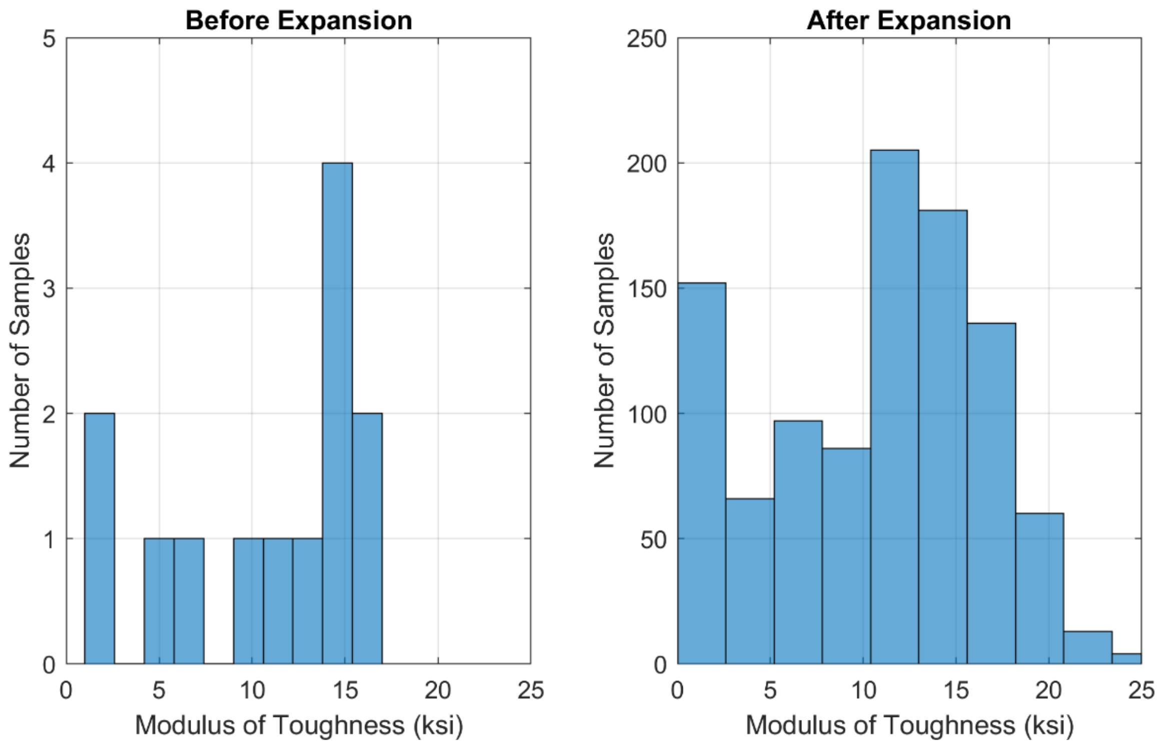

Figure 5-10. Results from Monte Carlo expansion for one strap grouping

After expanding the sample size, the Pearson Chi-squared Goodness of Fit statistical test was used to test the fit of several distribution types [53]. The types of distributions were then ranked based on the chi-squared statistic results. The chi-squared statistic is calculated using Equation 5-3 below, taken from Pearson (1900):

$$\chi^2 = \sum_{i=1}^{n} \frac{(O_i - E_i)^2}{E_i} \qquad \textit{Eq. 5-3}$$

where O_i is the observed value, E_i is the expected value from the given distribution, and n is the total number of observations. It is important to note that both the normal distribution and Gumbel distribution were not able to be tested for the MOT data, as there is a clear lower limit of 0. In this case, the distribution that best fit both the stress and MOT data was a Weibull distribution, as seen in Table 5-4, and is associated with the lowest chi-squared value for each property. This follows logic as the Weibull distribution was originally proposed with the purpose of modeling material breaking strengths [54].

Table 5-4. Results from analysis of best-fit

	Chi-Squared Statistic	
Distribution	**Stress**	**MOT**
Weibull	1153	1507
Normal	2580	N/A
Gumbel	2576	N/A
Gamma	4279	2587
Lognormal	8203	5145
Inverse Gaussian	8071	10518

Weibull distributions use two shape parameters to better fit the data. These parameters were determined by testing a wide range of values for each, calculating the root mean squared error (RMSE) for each combination, and choosing the values that resulted in the best fit. Table 5-5 lists the shape parameters that were used in creating the distributions for each data set. That is followed by Figure 5-11, which shows normalized histograms of the expanded data with the Weibull probability density functions plotted over them.

Table 5-5. Weibull shape parameters used in distribution fit for each property

Strap Result Type	Shape Parameter A	Shape Parameter B
Stress	115.7	17.30
Modulus of Toughness	9.5	2.93

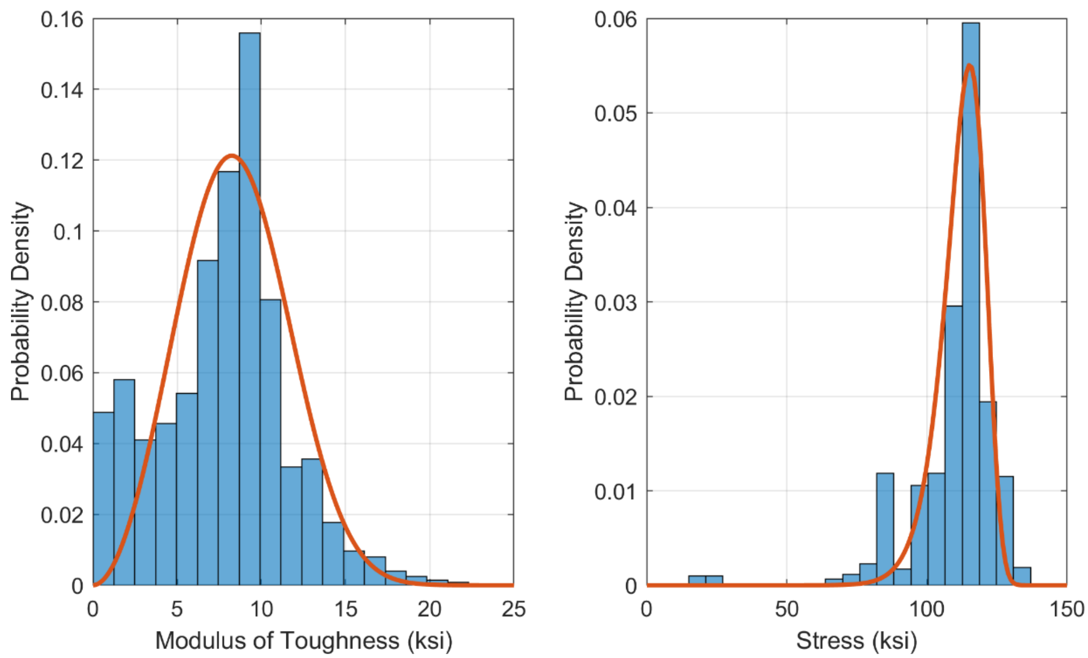

Figure 5-11. Weibull best-fit distribution for MOT (left) and stress (right)

These distributions can be used to calculate risk, such as probabilities of anchorage failure at given wind speeds. For example, assuming that a strap thickness of 0.036 in. is used and design level wind speeds exert 4,725 lbs of force into the straps, the total stress in the strap would be 105 ksi. Using the stress distribution curve, the probability of the stress exceeding the maximum stress capacity of the strap is approximately 17%.

6. Structural Analysis of a Manufactured Home

The design requirements provided by the MHCSS were used to model a manufactured home in SAP2000, a structural analysis software developed by Computers and Structures, Inc. [55]. The purpose was to present a framework for incorporating the anchorage requirements from Table 2-1 into a numerical model for further parametric analysis. The design and test parameters, as well as the results, are described in the sections below.

6.1. Modeling Parameters

Several different models were tested, but they all were created using the same material properties and element dimensions. First, three types of materials were created and used: concrete, steel, and wood. For the concrete and steel, the default SAP values for modulus of elasticity (E), Poisson's ratio, shear modulus, and yield strength (F_y) were used. The steel was specified as A992 steel with a yield strength of 50 ksi. However, SAP did not have a default wood material, so it had to be created. The wood properties used were taken or verified from the *Wood Handbook* [56], a finite element analysis study by Quayyum [57], and the AWC's *National Design Specifications for Wood Construction* [58]. The model also closely resembled tests performed by Nelson et al. [59], in which No. 3 grade Spruce pine-fir lumber was used for the stud walls and rafters. Wood behaves as an orthotropic material, so three values were given for the modulus of elasticity and Poisson's ratio. The property values assigned to each type of material can be seen in Table 6-1.

Table 6-1. Material properties for the SAP model

Material Type	**Modulus of Elasticity [E1, E2, E3]***	**Poisson's Ratio [U12, U13, U23]***
Steel	29,000 ksi	0.3
Concrete	3600 ksi	0.2
Wood (frame)	[1,050, 1,050, 1,050] ksi	[0.33, 0.34, 0.34]
Wood (sheathing)	[740, 232, 232] ksi	[0.8, 0.8, 0.8]
* Orthotropic materials are listed with three values, one for each loading configuration.		

After the material properties were defined, the frame and area elements were created and assigned dimensions and properties. For the frame chassis I-beams, M10x9 beams were used and ran along the top of the piers for the length of the home, again referencing the model constructed by Nelson et al. [59]. The piers were modeled as 12x12 in. concrete columns, placed directly under the I-beams and spaced according to the MHCSS requirements. The pier size was not important, as the compression capacity was increased substantially to prevent vertical deflections and friction forces were not modeled directly. The bottom of the piers were modeled as roller supports, which only allowed for vertical resistance, representing the unreinforced masonry piers that are used in practice. Additionally, the tension limit of each pier was set to zero to model the actual behavior, also reflective of the unreinforced masonry piers used in practice. The floor joists were modeled as 2x8 in. nominal wood members, spanning the width of the structure and spaced at 12 in. on center. Also, a 2 in. thick shell area element was added as the floor sheathing. The wall studs and roof rafters were both modeled as 2x4 in. nominal studs, spaced at 12 in. on center. Although the roof truss members are typically smaller than 2x4 in., the roof system was simplified overall and functioned only to transfer the uplift loads into the walls. The exterior side of the wall studs and roof rafters was covered with a 0.5 in. shell area element, representing wall and roof sheathing. The area element was modeled as a shell to allow for in-plane and out-of-plane stiffness, so the orthotropic material properties were taken into effect. The last element was the anchorage straps, which were modeled as cables and could only resist tension. The top ends attached to the I-beam chassis, while the bottom ends were modeled as a pin supports at the same level as the pier bottoms. The location of the ground attachment point for each strap change based on the strap angle specified and the spacing of the I-beams. An important note is that the straps were only modeled on the windward side of the structure, as leeward straps would not be engaged in tension until after failure, specifically, complete sliding or lofting of the home, or unless uplift forces were enough to overcome the weight of the home. In all test cases, the weight of the structure was greater than the uplift forces on the roof, so no leeward-side straps were needed. One of the models tested can be seen in Figure 6-1, with the frame elements extruded to show the relative sizes. The area elements (red) actually cover the entire exterior but are only partially shown to enable a view of the frame elements (blue).

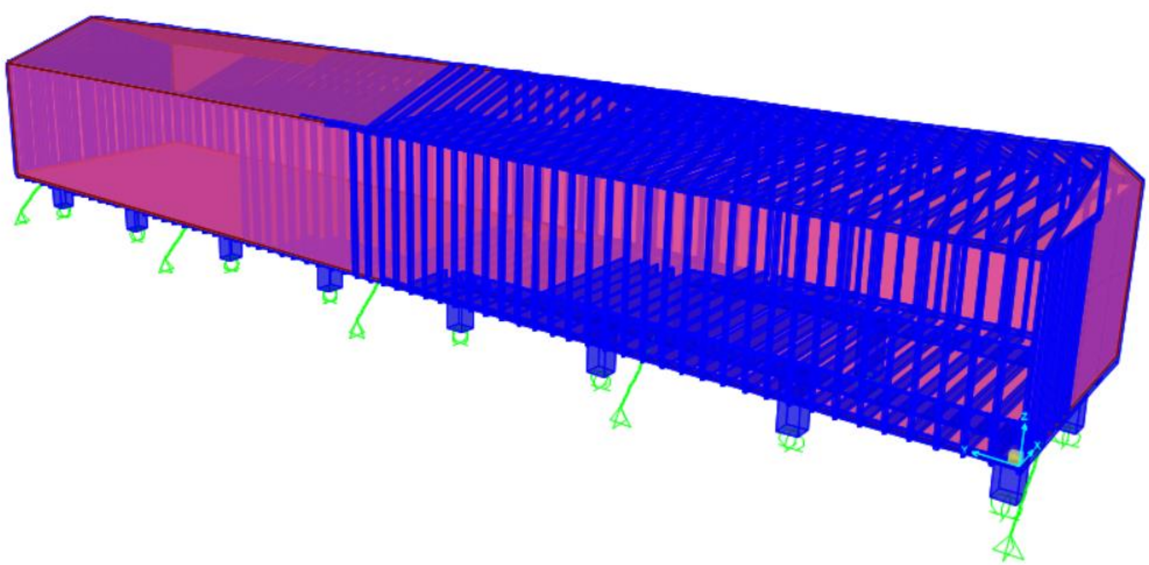

Figure *6-1. Typical layout of a manufactured home* model*ed in SAP2000*

It is important to note that all of the floor components were constrained together as a rigid diaphragm, meaning each floor joint deflected together in the horizontal plane. This behavior results in an even distribution of the loads across the straps, which is most likely conservative in regards to the max strap tension experienced. Realistically, a greater portion of the loads would most likely be transferred into the end shear walls, resulting in higher tensile forces in the end-wall straps. The rigid floor diaphragm simplification represents a best-case scenario, where each strap along the windward wall is loaded equally. Therefore, the actual forces in the end-wall straps during design-level loading may be even greater than those presented in this paper.

The loads applied matched the design wind loads from the MHCSS, with a 15 psf net lateral pressure and a 9 psf uplift pressure. However, since the point of the analysis was to test the anchorage resistance, the 1.5 factor of safety was applied to the loads to increase them to a 22.5 psf net lateral pressure and a 13.5 psf uplift pressure. The wind loads were applied as uniform pressures on the sheathing area elements. All of the self-weight loads of the structural members were set to 0, but a gravity load of 40 psf was applied to the entire floor area of the structure. The weight of manufactured homes varies, with older homes weighing as little as 25 psf [13], but newer homes are suggested to weigh between 35-50 psf [60]. However, as long as the weight of the home is greater than the uplift pressure of 13.5 psf, the overall results are not affected. Essentially, the piers resist all of the downward force, so when the home outweighs the uplift force, the anchorage straps are only resisting the lateral force. The purpose of the analysis was to assess the risk of anchorage failure at design-level loading, so the loads remained unchanged through all tests.

6.2. SAP2000 Analysis Results

The first set of tests performed sought to check the basic requirements of the MHCSS anchorage design guide, listed in Table 2-1. The table lists maximum strap spacing values for different scenarios, varying by pier height, width, and I-beam spacing. In the modeling, it was assumed that 5 anchorage straps would be used. Therefore, the length was determined by multiplying the maximum allowed spacing for each case by 4, with an added 2 ft on the external side of the 2 end straps. The design prescriptions specify the location for the straps to attach to the ground as a maximum of 4 in. inset from the edge of the wall, so it was assumed that they attached directly underneath the edge of the structure. Using the calculated length for 5 straps and the MHCSS assumed eave height of 7.5 ft, a model was tested for each of the scenarios given in the table. The results of these tests are listed in Table 6-2, which is set up similarly to the design guide table. However, instead of listing the anchorage spacing requirements, the maximum strap tensile force is listed in pounds and bolded. As discussed in Section 2.1, the minimum working force to be resisted by a single strap-ground anchor system is 3,150 lbs, and the results match that value closely. Therefore, the design prescriptions are successful in designing an anchorage system that matches the anchorage requirements.

Table 6-2. Resulting strap forces given code-compliant strap spacing

Nominal Floor Width, Single/Double-wide	Max Height from Ground to Strap Attachment	I-beam Spacing 82.5 in	I-beam Spacing 99.5 in
12/24 ft	25 in	**2703.5**	**N/A**
	33 in	**2555.2**	**N/A**
	46 in	**2488.9**	**N/A**
	67 in	**N/A**	**N/A**
14/28 ft	25 in	**3023.2**	**3236.5**
	33 in	**2979.5**	**3269.4**
	46 in	**2861.6**	**3261.0**

	67 in	**2759.3**	**N/A**
16/32 ft	25 in	**N/A**	**1726.3**
	33 in	**3165.7**	**3330.1**
	46 in	**3101.7**	**3347.7**
	67 in	**3089.9**	**3415.4**

However, these values from the MHCSS requirements are based on specific assumptions about the eave heights and strap angles. The code assumes an eave height of 7.5 ft, but modern manufactured homes may at times be taller. These homes are designed to be transported already constructed, so the maximum height is limited by interstate overpass clearance, which is approximately 14 ft. The bottom of the homes are typically transported at a height of 1.5-2 ft off of the ground, and many actually have collapsible roof structures to allow for taller homes [61]. Therefore, eave height values were tested up to 11 ft, a large but possible height. Also, the MHCSS requirements allow for a maximum strap angle of 60 degrees, so additional strap angles were tested. Strap angles calculated from the code requirements are between 30-50 degrees, with an average of 41 degrees. However, using the strap data shown in Figure 4-26, the actual strap angles seen in the field were typically between 35-65 degrees, with an average angle of 48 degrees. To test these additional variables, a manufactured home with a length of 68 ft and width of 14 ft was chosen. A pier height of 33 in. was used, with the I-beams spaced at 99.5 in. and 5 straps along the windward wall. These dimensions matched a previously tested case, which had a maximum strap tensile force of 3,269.4 lbs. However, as seen in Figure 6-2, varying the strap angle and increasing the eave height significantly affected the maximum tensile force experienced. The dotted line in the figure represents a value of 4,725 lbs, which is the ultimate required capacity of the straps. A typical strap angle of 50 degrees, along with an eave height of 10 ft, increases the tensile forces in the strap to a value greater than their prescribed capacity. According to the MHCSS, this specific case would require the anchorage system to be designed by a registered engineer or architect, and rightfully so. Overall, Figure 6-2 illustrates the significance of both the eave height and strap angle in regards to the tensile forces in the straps, and these factors should not be overlooked in the design and installation of the anchorage system.

Additional refinement of the model presented above is needed. The model and framework presented above can be used however to further explore the impacts of corroded or missing anchors. It can also be used to explore the number of anchors required to provide the necessary resistance for a higher design wind speed level. Ultimately, the refined model is needed to evaluate the number of anchors required to prevent the premature failure of the anchorage system by ensuring it is as strong as or ideally stronger than the superstructure in order to facilitate safer failure sequences.

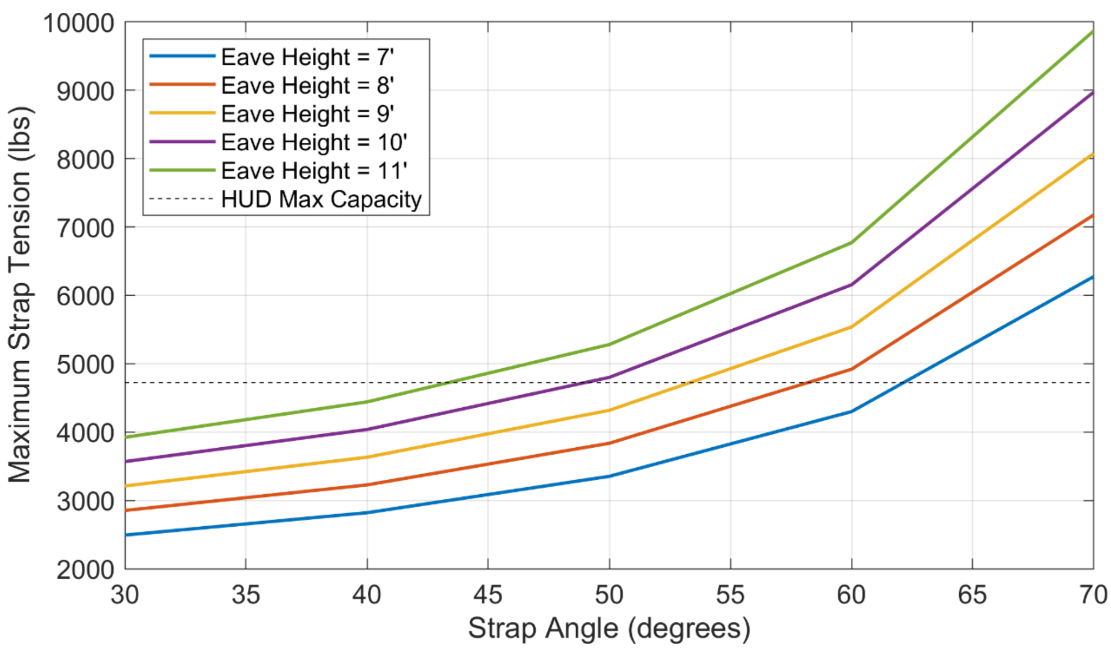

Figure 6-2. Effects of varying eave height and strap angle on max strap tension

7. Conclusions and Future Research

The objective of this study was to determine the cause of the enhanced fatality risk inherent to MMH. The results indicate that the primary reason for this is due to their typical pattern of failure. MMH anchorage systems typically fail prior to other important structural components such as the walls or roof, and that leads to the destruction of the entire structure. Site-built homes are generally observed to fail top-down, while manufactured homes typically fail bottom-up. The main issue with a home failing bottom-up is that the structure, which functions to protect the occupant from debris and the force of the winds, is destroyed very quickly, and the occupant is either in danger from the destruction or exposed to the tornado winds and debris. This brittle failure sequence results in the sudden, premature destruction of the home, which increases the risk of fatality. There are several factors resulting in the enhanced vulnerability of these anchorage systems, including issues with the code requirements, individual component capacities, and those inherent to specific anchorage system types. These contributing factors are listed and described below.

1. The MHCSS anchorage requirements provide an adequate design for strap systems with the same conditions assumed in the code. However, certain aspects of the code may allow for enhanced vulnerability of the structure and could potentially be improved upon.
   a. The design requirements for strap spacing treat single-wide and double-wide MMH identically, which implies that they perform identically. However, the post-tornado damage assessment results revealed a consistent pattern of single-wide anchorage systems performing worse than those of double-wide homes, despite having the same requirements for strap systems. This is most likely due to the increased weight and inherent uplift resistance of double-wide homes as compared to single-wide homes. Therefore, the strap

spacing requirements for single-wide homes may need to be modified to account for the discrepancy in performance.

b. Although the hurricane and tornado damage assessments both revealed an issue with anchorage system failure, a more detailed look at the data shows a difference in performance over time between the two types of events. The hurricane study performed by Hebert et al. implies that anchorage design during hurricane wind loading has improved with each MHCSS update [38]. The most significant improvement occurred after the MHCSS update in 1994, following the numerous assessments performed on the damage caused by Hurricane Andrew and studies of the inadequacies of the MHCSS lateral load design standards, especially when compared to design standards such as ASCE 7-88 [10], [11], [13], [62]. This is encouraging, but the data from tornado studies indicates that the same strides have not been made in regards to resisting tornado wind loads. This is somewhat predictable, as tornado-induced loads are not as well understood compared to hurricane-induced wind loads, which mostly behave as straight-line winds. However, the main reason for the discrepancy between hurricane and tornado performance improvements may simply be due to the typical location of occurrence. Hurricanes normally affect MMH along the coast, which fall within *Wind Zones 2 and 3*. The 1994 MHCSS update improved the design criteria for homes located in those hurricane-prone zones. Tornadoes typically affect homes within *Wind Zone 1*, which was essentially unchanged from the original MHCSS requirements. The *Wind Zones* may need to be updated to account for tornado-prone regions, requiring homes in those regions to be designed with higher wind resistance capacities.

c. Homes in *Wind Zone 1* have a tendency for the anchorage to fail prior to the superstructure, despite having an additional factor of safety of 1.5 applied only to the anchorage. One reason may be that the superstructure is constructed in a very controlled environment, while the anchorage system is installed on-site upon placement of the home. This may allow for a higher degree of variability in anchorage installation, leading to inconsistency in performance. A higher F.S. may be required to account for this variability.

2. Tie-down strap and ground anchor systems seem to have a number of different factors that can potentially lead to failure, including ground anchor deflection and pullout, strap rupture, and issues such as corrosion.
   a. Past studies of ground anchor capacities reveal a serious issue with both serviceability and ultimate load capacity. Even in cases where the ultimate capacity is greater than the required value of 4,725 lbs, the resulting displacements, especially horizontal, are extremely large. These weak capacities can result in complete pullout of the ground anchors, most likely leading to complete failure of the anchorage system.
   b. The strap tensile tests resulted in adequate load capacities for most cases. The force required to rupture the straps appeared to remain unchanged with time, but the plasticity of the straps was negatively affected by time. Defects such as corrosion, which varies based on age and environment, also decreased the plastic capacity of the affected straps. These factors may cause a strap system to become very brittle over time, decreasing the total amount of energy that can be dissipated during extreme loading.
   c. Overall, the post-tornado data revealed that strap systems are slightly more likely to fail due to strap rupture than ground anchor pullout, despite the poor results from the ground

anchor capacity studies. One reason for this may be the large deflections of the anchor heads prior to pullout. If a given strap-ground anchor combination deflects a large distance under at-capacity loading, the load will be partially redistributed into other straps that have not yet deflected. The now-overloaded straps may rupture, especially if corrosion and/or age has made them brittle, causing a reduction in the overall anchorage capacity and will most likely lead to failure.

3. Pan anchorage systems typically have different failure mechanisms than strap systems and experience a greater damage ratio than adequate strap systems on average.
   a. The primary reason for this is the lack of uplift resistance, which oftentimes results in pan system homes being rolled or lofted. The design relies upon the weight of the home to produce a large frictional force between the pan and the ground, so uplift forces may significantly decrease the lateral load capacity. Additionally, they typically have only one pan on each end of the home, resulting in very little redundancy in the capacity and load path.
   b. A common issue observed in field assessments was the failure of the connection between the I-beam flange and the metal strut, as well as the connection of the metal strut to the ground support. Both of these connections are critical, as their failure disconnects the load path.
   c. All of these factors increase the vulnerability of the system, increasing the likelihood of complete failure even in weaker tornadoes.

Overall, the enhanced fatality risk in MMH is primarily tied to the performance of the anchorage system. Improving the design capacity and reliability of this critical point in the load path should be the first step in attempting to reduce the overall vulnerability. To do so, several steps still need to be taken. A more detailed analysis of the relationship between ground anchor head displacement and strap rupture may be required to better define the problem. Also, continued post-tornado and post-hurricane damage assessments are important for collecting new data and continuing to grow the dataset. In the process, more tie-down strap samples should be collected and tested to expand the reliability of those results. Additionally, the damage assessments need to begin documenting basic soil properties and making more detailed notes about ground anchor pullout and large deflections, as that appears to be a critical factor. Finally, a more detailed numerical analysis of a manufactured home strap anchorage system should be performed to better understand the distribution of loads into the straps, as well as the redistribution of those loads under certain scenarios. All of these steps will work to better define the specific issues with anchorage failure, and lead to viable solutions.

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