Resilient Coastal Communities under Wind and Flood Hazards:

Understanding the Trade-offs in Residential Building Designs

Apr. 30, 2019

Wei Zhang, Ph.D., P.E.
Christine Kirchhoff, Ph.D., P.E.
Di Wu, M.S., E.I.T.
Jeffery Weston, M.S.
Xuan Li, M.S., Ph.D. Candidate
Xiaolong Ma, M.S., Ph.D. Candidate

Connecticut Institute for Resilience and Climate Adaptation

Sponsored by a grant from the Connecticut Institute for Resilience and Climate Adaptation (CIRCA). CIRCA is a partnership between the University of Connecticut and the State of Connecticut Department of Energy and Environmental Protection. More information can be found at: www.circa.uconn.edu

Resilient Coastal Communities under Wind and Flood Hazards: Understanding the Trade-offs in Residential Building Designs

Prepared by:

Wei Zhang, Ph.D., P.E.
Christine Kirchhoff, Ph.D., P.E.
Di Wu, M.S., E.I.T.
Jeffery Weston, M.S.
Xuan Li, M.S., Ph.D. Candidate
Xiaolong Ma, M.S., Ph.D. Candidate

University of Connecticut
School of Engineering
Department of Civil and Environmental Engineering

ACKNOWLEGEMENTS

Funding for this work was partially provided by Connecticut Sea Grant, University of Connecticut through Award No. NA14OAR4170086, Project Number R/CH-1.

The project is also sponsored by a grant from the Connecticut Institute for Resilience and Climate Adaptation. The Connecticut Institute for Resilience and Climate Adaptation (CIRCA) is a partnership between the University of Connecticut and the State of Connecticut Department of Energy and Environmental Protection. CIRCA's mission is to increase the resilience and sustainability of vulnerable communities along Connecticut's coast and inland waterways to the growing impacts of climate change on the natural, built, and human environment.

More information about CIRCA can be found at circa.uconn.edu.

The support is greatly appreciated. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect the views of the sponsors.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1 Introduction

1.1 Background and significance

Sixty percent of the population in Connecticut lives along the coast and altogether Connecticut has more than $542 billion in assets vulnerable to the ravages of coastal storms. Past storms that brought heavy precipitation, high winds, and storm surge damaged coastal properties and put lives at risk. For example, Sandy and Irene caused over $360 million damages in Connecticut including destroying over 4,000 single family homes in Fairfield and New Haven Counties alone (Figure 1-1) (FEMA 2014). Though past storms have caused enormous impacts, climate change induced alterations in weather patterns will increase the risks communities face as storms become more frequent and more intense and sea level keeps rising (Emanuel 2013). Moreover, ongoing coastal urban development and densification only intensifies the problem as more lives and property are put in harm's way.

Figure 1-1 Flooding in Fairfield Beach after Hurricane Sandy

Figure 1-2 Elevated home on Fairfield Beach Road showing the overall response of neighborhood after Hurricane Sandy

While wind hazard is not traditionally considered as great of a risk in Connecticut coastal regions as the risk from flooding, Connecticut falls within established hurricane zones and is exposed to hurricane-force winds generated in hurricanes, nor'easters, isolated thunderstorms and winter storms. For example, the maximum wind gust during hurricane Sandy reached 85 miles per hour (mph) (NOAA 2012) while wind gusts of 78mph were recorded in the recent winter storm (Jan 27, 2015) in south New England. Finally, while rare, Connecticut has experienced wind speeds greater than 100mph. The 1938 New England Hurricane produced wind speeds of 156mph, killed 682 people, and caused property losses estimated at $4.7 billion in 2005 dollars (BG 2005). Unfortunately, recent research from experts at MIT and the National Hurricane Center suggests much of the United States, including New England, is overdue for another major hurricane (AP 2015). Locating at the northeast coast of United States, Connecticut is prone to be affected by hurricanes and will be subjected to enormous financial losses if a hurricane hit Connecticut coastline directly. Meanwhile, in 2015 Corelogic estimated that 98,000 homes with a total reconstruction value of over $33 billion are at risk of damage from hurricanes and the damage losses will keep increasing every year without suitable actions carried out to reduce the hazard risk (Hartford Business 2015).

In response to extensive flood damage resulting from Hurricane Irene, Sandy and Nor'Easter Alfred, the State of Connecticut as well as Connecticut towns and residents have made large investments in improving coastal resilience, especially for flooding. These investments spanned preparedness, recovery and mitigation including rebuilding and elevating residences and protecting coastlines and critical infrastructure. For example, as of September 2014, 48 single family residential (SFR) buildings were elevated in Fairfield Beach (Figure 1-2) with additional elevations in the planning stages. In addition to these physical changes, several coastal communities and the State of Connecticut also made changes to building codes and zoning regulations. For example, Darien, Greenwich, Stamford, and New Haven require an additional safety margin for vulnerable structures, requiring an additional foot of freeboard above FEMA's base flood elevation (BFE). And at the state, Connecticut's Flood Management statutes now go beyond FEMA's elevation requirements. Housing projects in a floodplain using Community Development Block Grant (CDBG) funds must be elevated to the 500-yr flood elevation, far higher than the elevation that FEMA requires which is based on the 100-year flood elevation.

Because coastal communities experienced so much damage from flooding in the past (CTNHMP 2014), the biggest adaptive response to date has been to elevate SFR above new higher flood levels (sea Figure 1-3). While elevating homes minimizes flood risk, these newly elevated homes may now be at greater risk from exposure to damaging winds. Complicating wind hazard exposures in Connecticut and other New England regions are SFR building designs which tend to be multiple stories high (e.g., typical colonial) with steeply sloping roofs. These typical design elements exacerbate the potential risk from wind damage because building height and roof slope increase wind loads. While wind retrofit design is required for newly built Connecticut buildings, existing homes do not require wind retrofit design unless the home is substantially renovated. This means that while existing SFR homes in floodplains must be elevated, no wind retrofit design elements are required to be installed during the elevation process unless the home is being substantially renovated. Without wind retrofit designs, windows will not have storm shutters to handle the hurricane force winds, and roofs will not have strengthening measures to secure them in place. Adding to the risk is the potential that existing homes may be substantially weaker now than they were when they were first built. This weakness manifests through the cumulative damages existing homes experience over years of exposure to coastal storms. The question coastal communities' must consider is, do SFR elevation requirements without consideration of additional wind load exposure make their community more (or less) resilient?

Figure 1-3 Elevated buildings in Fairfield after Sandy

Potential trade-offs between wind and flood risk are not a focus of attention of coastal communities nor hazard scholars. To better understand the role of wind loads in current building design processes, we spoke with planners and building inspectors in Connecticut coastal communities. Since building codes emphasize avoiding structural failures to protect public safety, building inspectors mainly look for sufficiency of structural components transferring loads to the ground rather than looking for sufficiency in design to withstand wind (roof tie down, etc.). According to one building inspector, no one pays attention to wind currently, only flooding. Continuing, the building inspector said there is no path in the residential building code to require wind retrofit design in existing SFR homes that are going to be elevated. This suggests there may be barriers to changing practices to address multi-hazard risk resulting from both existing perceptions of flood and wind risks and current decision paths for flood mitigation and permitting. And, for hazard scholars, while some have examined multi-hazard exposures (Fronstin and Holtmann 1994; McCullough and Kareem 2009; Phan et al. 2007), multi-hazard analysis of wind and flood are usually carried out separately (Pan et al. 2014; Sparks et al. 1994; Tomiczek et al. 2014). This suggests a gap in the methodologies used to assess building damage which to date have not considered the joint effects of multi-hazard exposure.

Potential trade-off between wind and flood risk necessitates a resilience assessment of elevated and non-elevated coastal residential buildings. In order to assess individual building damages under multi-hazard risks and evaluate the vulnerability of the entire coastal community along Connecticut's Shoreline, numerical simulations based upon finite element analysis need to be carried out to evaluate structural performance of the typical elevated and non-elevated building models under the combined effects of wind and flooding loading. While some hazard scholars have examined multi-hazard exposures (Fronstin and Holtmann 1994; McCullough and Kareem 2009; Phan et al. 2007), multi-hazard analysis of wind and flood are usually carried out separately (Pan et al. 2014; Sparks et al. 1994; Tomiczek et al. 2014). Their research findings suggest a gap in the methodologies used to assess building damages which to date have not considered the joint effects of multi-hazard exposure.

1.2 Purpose and outline

The objective of this research is to address the gaps that exist in current design processes and multi-hazard assessment for coastal residential buildings. To achieve this objective, we first advance and apply multi-hazard assessment methodology that considers the combined impacts from wind and flood hazards (Ellingwood et al. 2007; Heaney et al. 2000; Rashed 2006). Then, we explore how town planners and building inspectors, and residential builders in our partner communities mitigate wind and flood risk including in zoning and building codes, and in the design, build, and inspection process. We aim to help our community partners understand the tradeoffs between flood and wind risks in their communities through collaboratively producing vulnerability and resiliency maps and co-developing more resilient design parameters for new and retrofit SFRs to advance coastal community resilience in Milford and Fairfield. We also review the literature to understand drivers of SFR elevation, if trade-offs in wind and flood (or other trade-offs) are considered, and how individual risk perception motivates protective behaviors includes preparing education and outreach materials that expand and enrich the current program employed by our partner communities on flood hazard to include multi-hazard mitigation. The education and outreach materials and methodology are designed to be transferable to other coastal communities in Connecticut and will be shared through multiple venues (e.g., Sea Grant Climate Adaptation Academy, annual Design and Trades Conferences, Wrack Lines, etc.). To summarize, the following four tasks are assigned to fulfill the purpose.

- Task1: To quantify the potential natural hazard for Milford and Fairfield coastal communities including flood parameters determination and GIS-based community flood level map generation (Chapter 2).
- Task 2: To statistically group SFR buildings based on key design parameters from GIS datasets on SFR buildings provided by the two partner communities. Representative building models from the two coastal communities are extracted and modeled using the finite element analysis approach (Chapter 3).
- Task 3: To define the failure criteria for SFR buildings and generate vulnerability curves for those representative building structures and a series of GIS-based community resilience maps for different multi-hazard scenarios (Chapter 4).
- Task 4: To review the literature to understand drivers of SFR elevation, if trade-offs in wind and flood (or other trade-offs) are considered, and how individual risk perception motivates protective behaviors (Chapter 5).

The outline of this report is organized as follows. First, the flood parameters are determined, and the flood map generation procedures are defined to generate the flood maps of different return periods for the Fairfield and Milford coastal communities (Task 1). The detailed information of representative real-world SFR buildings in the two partner communities are then provided based on the GIS datasets and are used as structural models in the following analysis. To help communities understand and evaluate the trade-offs under multi-hazard scenarios, a physics-based damage assessment methodology for coastal communities using the finite element analysis approach under wind and flood loads was built and applied to those representative residential buildings in our two partner communities. Potential risks to the coastal wind and flood hazard are analyzed based on the building type, building material, building height, roof type, and roof materials typical in our partner communities and historical wind and flood data (Task 2). To identify the structural performance under wind and wave hazards, several failure criteria for different structural components are defined based on the literature. Then, vulnerability curves for representative residential building structures are produced as well as the GIS-based resilience maps that help translate research outputs to a more usable format to inform decision making. In addition, to increase community resilience, recommendations on whether or not elevate the low-rise residential buildings are proposed (Task 3). Finally, we review the literature to understand existing drivers of SFR elevation as well as the methods and benefits of the flood proofing retrofits and if trade-offs in wind and flood (or other trade-offs) are considered. We also review literature to understand individual risk perceptions and how that may inform SFR elevation decision making (Task 4).

CHAPTER 2 Flood Hazard for the Community

2.1 Background

The past decades have witnessed an upward global trend in natural disaster occurrence throughout the world. Among the most recurring and devastating natural hazards, hydrological and meteorological disasters are the main contributors to this pattern, which result in enormous losses of human lives and property (Skakun et al. 2014). In particular, the U.S. is amongst the top ten countries which have experienced a large number of catastrophic events over the last decades especially for hurricane and flood hazards (Guha-Sapir et al. 2014).

Sixty percent of the state's population and more than $542 billion in infrastructure assets are located along Connecticut's coast. Coastal communities are vulnerable to natural hazards including hurricanes and severe storms that bring strong winds and rain, storm surge, and flooding. Recent storms including Hurricane Irene (2011) and Hurricane Sandy (2012) were both costly and deadly. Hurricane Irene, ranked as the seventh costliest hurricane in the U.S., caused catastrophic damages to the coastal communities and shorelines of Long-Island Sound, including to the coastal communities of Fairfield and Milford, CT. Hurricane/Post-Tropical Cyclone Sandy resulted in over 200 deaths; making Sandy the deadliest hurricane to hit the U.S. East Coast since Hurricane Agnes in 1972, as well as the deadliest hurricane to hit the U.S. mainland since Hurricane Katrina in 2005 (NOAA 2012). Damages estimated from Sandy exceed $50 billion in the U.S., making Sandy the second-costliest hurricanes in the U.S., with more than 6,000 properties along the shoreline of Connecticut damaged.

Similar to other Connecticut coastal communities, the town of Fairfield and city of Milford have experienced significant damages from recent extreme storms. For example, during Superstorm Sandy, Fairfield experienced the largest number of damaged homes in Fairfield County, with at least 900 residential building structures affected. The area between Fairfield Beach and Shoal Point, as shown in Figure 2-1, sustained the most significant damages from wind and flooding. Damage to the residential building structures on Fairfield Beach road are shown in Figure 2-2.

Figure 2-1 Flooded coastal community in the Fairfield Beach area during Hurricane Sandy

Figure 2-2 Damaged buildings on Fairfield Beach Road during Hurricane Sandy

Growing evidence shows that climate change may bring more frequent and severe natural hazard events, accompanied by sea-level rise and extreme flooding (Elsner et al. 2008). The Intergovernmental Panel on Climate Change (IPCC) reported that sea surface temperature rise may increase the frequency and intensity of hurricanes (IPPC 2007). And, with population density in flood-prone coastal communities expected to grow by 25% by 2050 (Aerts et al. 2014) along with steadily increasing coastal property values, social and economic losses in the coastal community from natural hazards are likely to be much larger in the future (Stewart et al. 2003).

Destructive consequences of recent flooding events have reignited the coastal resilience discussion. Due to the vulnerability of Fairfield and Milford to hurricane-induced flood hazards and the importance of supporting the local economy, it is necessary to perform resiliency analysis for the Fairfield and Milford coastal communities subjected to the extreme multi-hazard events.

2.2 Description of the study area

The Town of Fairfield encompasses 30.2 square miles and is located in Fairfield County, Connecticut (Figure 2-3), approximately 50 miles northeast of New York City.

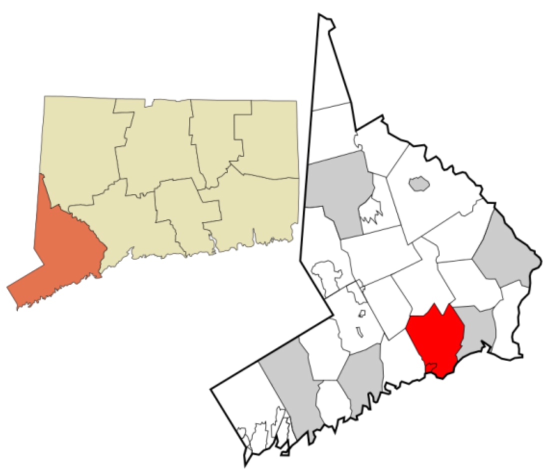


Figure 2-3 Town of Fairfield (red), in Fairfield County(orange), Connecticut

According to the Coastal Area Management boundary developed by the Connecticut Department and Environmental Protection, the Town of Fairfield is divided into eleven areas, which were recorded in Coastal Boundary Map of Fairfield Town dated in 1983 (Figure 2-4). Among these eleven areas, the Shore Area is a waterfront area, which is situated in south Fairfield on the coast of the Long Island Sound. A large portion of the Shore Area containing most of Fairfield Beach Road and Pine Creek Avenue is significantly different from the remaining areas in Fairfield and has an unusual density of development warranting its regulatory scheme as a Beach District (Figure 2-5). Historically, the Shore Area was a densely clustered area of seasonally occupied, summer beach cottages. However, over years, cottages were converted and expanded to the year-round occupied community, i.e., Fairfield coastal community, facing more challenges to infrastructure maintenance, population density, public safety, and natural hazard concerns.

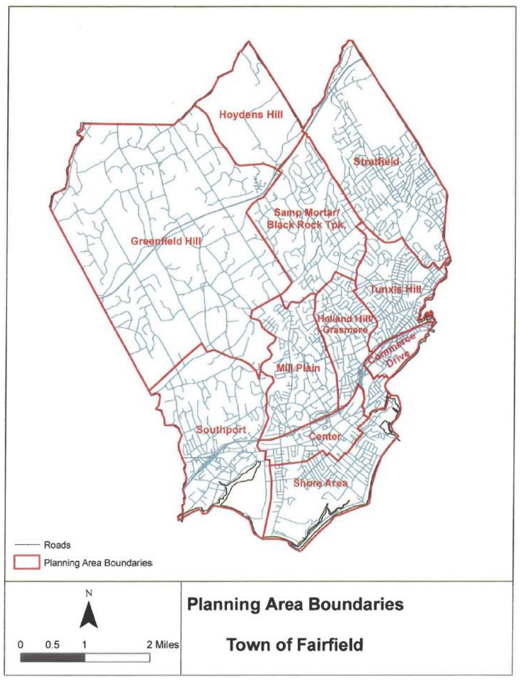


Figure 2-4 Coastal boundary map of Fairfield town (Source: Fairfield Town Plan of Conservation and Development, 2016)

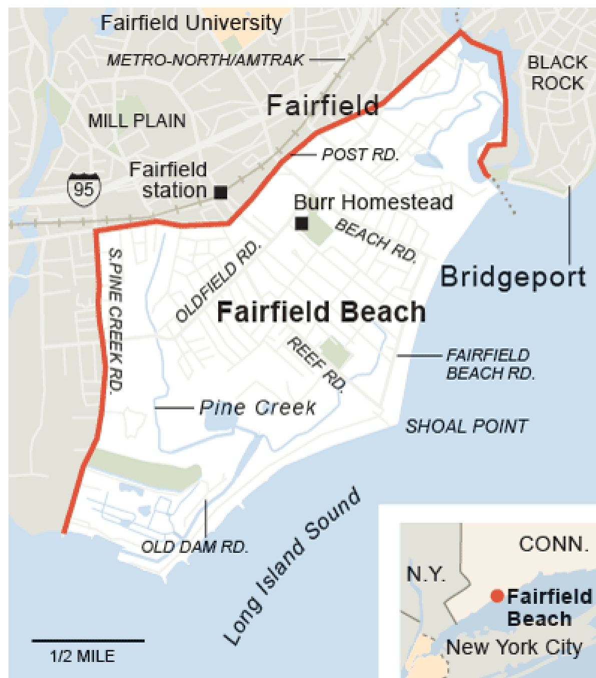


Figure 2-5 Fairfield Beach, Connecticut (Source: The New York Times, 2013)

Similar to the town of Fairfield, the City of Milford is also a coastal community in Connecticut located on the coast of the Long Island Sound. Milford encompasses 26.13 square miles (colored red in Figures 2-6 and 2-7) in New Haven County (colored pink in Figure 2-7), Connecticut. The City of Milford is between Bridgeport and New Haven (Figure 2-7) and serves as an important part of the New York-Newark Bridgeport, NY-NJ-CT Combined Statistical Area. Based on the census in 2016, the population of Milford was estimated to be 52,536. Similar to the town of Fairfield, Milford is also subjected to an increasing damage potential due to natural hazards from wind, floods, hurricanes, storm surge, and their combinations.

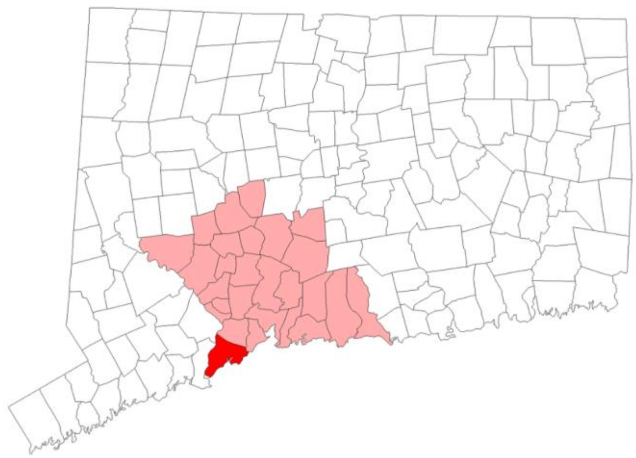


Figure 2-6Maping showing City of Milford (red), in New Haven County (pink), Connecticut

Figure 2-7Map showing Milford (red) and neighboring towns

2.3 Coastal flood hazards

Connecticut coastal regions, including Fairfield and Milford coastal communities, face a high level of flood risks. Both Fairfield and Milford coastal communities fall within flood zones and are exposed to coastal flooding caused by hurricanes or coastal storms.

Digital Flood Insurance Rate Maps (DFIRMs) and Flood Insurance Study (FIS) reports, which are published by the Federal Emergency Management Agency (FEMA) in support of the National Flood Insurance Program (NFIP), were used to obtain the flood hazard map for Fairfield (Figure 2-8) and Milford (Figure 2-9) coastal community. Coastal flood risks shown in **Error! Reference source not found.** and **Error! Reference source not found.** depict the magnitude and severity of flood hazards at different locations in Fairfield and Milford, respectively. There are three main categories to describe the flood hazard levels including: 1) the areas with 1-percent annual chance flood event (VE Zone, AE Zone); 2) the areas with 0.2-percent annual chance flood event (X Zone); and 3) the areas of minimal flood risk (X Zone).

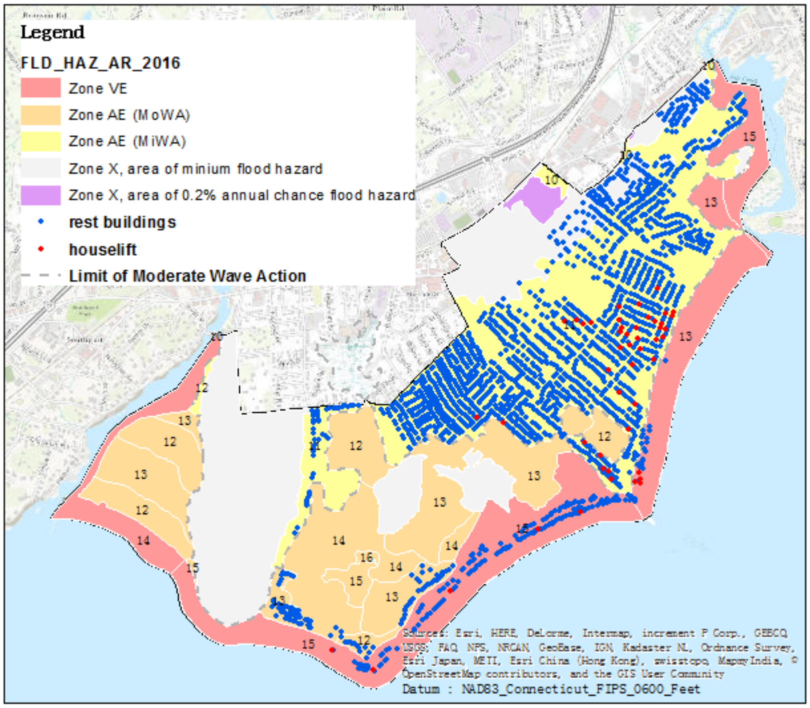


Figure 2-8Flood hazard map for Fairfield coastal community showing flood hazard zones

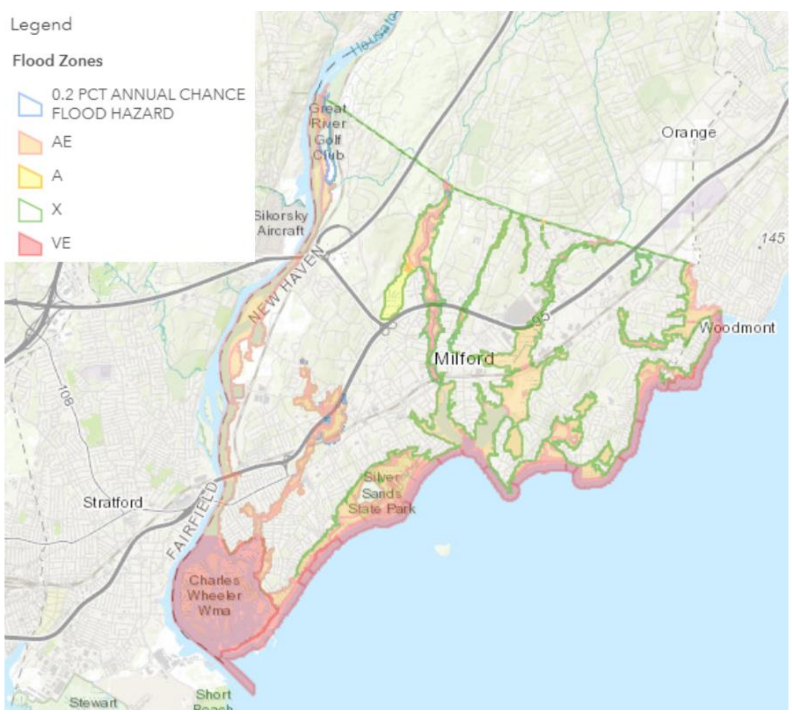


Figure 2-9Flood hazard map for Milford coastal community showing flood hazard zones

2.4 Flood conditions

Considering the locations of the Fairfield and Milford coastal communities as well as the past severe flooding events in these two communities, it is of great importance to understand flood risk and anticipated flood conditions during different what-if hazard scenarios.

2.4.1 Term definition for flooding

In this section, several terms will be defined for flooding as followings:

- *Stillwater elevation (SWEL)* is the elevation of the water surface which is caused by storm surge only.

- *Wave run-up* is the water rush up in a sloped shoreline or a structure intercepting the stillwater level.

- *Wave setup* is the rise in water surface as breaking waves approach the coastline. It is based on the characteristics of breaking wave and the slope of the profile and is typically obtained by adding 1.5 to 2.5 ft to SWEL during a base flood event (Figure 2-10).

- *Total water level (TWL)*, also known as mean water level or mean water elevation, serves as the propagating wave surface and is equal to the SWEL plus wave setup. The 1% annual chance of total water level is dependent upon offshore bathymetry, astronomical tides, wind setup, pressure setup, and wave setup (FEMA 2011).

- *Wave height* is the vertical distance between the wave trough and wave crest of a wind-driven wave propagating over the water surface. The wave height at a particular site could be affected by four factors including water depth, fetch length, wind speed, and duration.

- *Base flood elevation (BFE)* shown on SFHAs is selected to be the maximum of wave crest elevation or the wave run-up elevation as the wave propagates inland during the base flood event. It is noteworthy that BFEs shown on the FIRM panel are rounded to the whole foot, which might be less accurate for flood load calculations.

- *Wave run-up elevation* is the elevation reached by the wave run-up referenced to NAVD or other vertical datums.

- *Stillwater depth* is equal to the stillwater elevation (including wave setup) minus the lowest eroded ground elevation.

- *Lowest eroded ground elevation (GS)* is the lowest ground surface profile at the base of the structure accounting for long-term erosion or erosion during the base flood event ignoring the effects of localized scour.

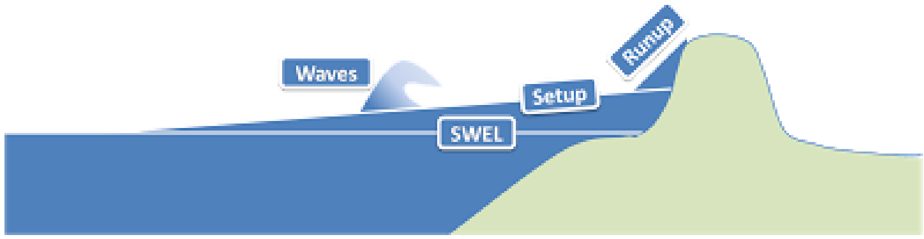


Figure 2-10 Storm surge, stillwater elevation, and added effects of wave setup and wave run-up

2.4.2 Relationship between flood elevation parameters

Flood parameters mentioned above are necessary for flood map generations and flood damage evaluations. Figure 2-11, which is from the Coastal Construction Manual (CCM), further illustrates the relationships between these parameters.

The relationships between SWEL, GS, associated stillwater depth, BFE and wave heights are shown in Figure 2-11. Based on the Coastal Construction Manual, the maximum wave height (H_b) in the shallow water is usually 78% of the stillwater depth (d_s). For instance, the wave height in a case of 1% annual chance flood is determined by the following equation:

$$H_b = 0.78 d_{100} \tag{2-1}$$

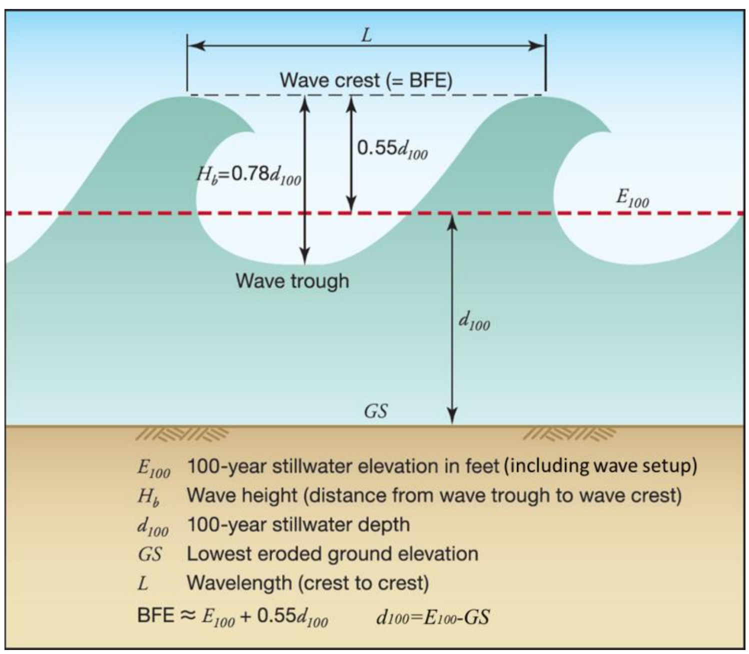


Figure 2-11 Schematic diagram showing 100-year SWEL, stillwater depth, and BFE

In the shallow water, the waveform is distorted and varies at different locations due to the varying terrain. This poses a challenge to generating flood maps. To better generate the flood maps, the wave crest is selected to be above the SWEL with a distance of 70% of the wave height, which means the maximum wave crest elevation is $0.55d_s$ above the stillwater level. For instance, this value will be $0.55d_{100}$ during a 100-year base flood event. As discussed earlier, the 100-year stillwater depth (d_{100}) is the height difference between the 100-year stillwater elevation (E_{100}) including wave setup and the ground elevation (GS), expressed as:

$$d_{100} = E_{100} - GS \tag{2-2}$$

For a coastal shoreline with gentle ground slopes and few buildings and vegetation, the base flood elevation (BFE) can be determined using the following equation:

$$BFE \approx E_{100} + 0.55d_{100} \tag{2-3}$$

Combining Equation 2-2 and 2-3, the base flood elevation (BFE) can also be calculated by:

$$BFE \approx GS + 1.55d_{100} \approx 1.55E_{100} - 0.55GS \tag{2-4}$$

2.5 Flood level in Fairfield and Milford coastal community

With the flood parameters for Fairfield and Milford coastal communities, it is straightforward to calculate the flooding loads as well as the structural performances in these two communities. However, additional efforts are still required to deal with the complexity of the land topography and the map scale limitations within the communities.

2.5.1 Acquisition of flood parameters

Three steps need to be carried out to determine the flood characteristics that affect the flood loads and the flood levels. Firstly, the preliminary values for E_{100} and BFE will be chosen. Secondly, the stillwater depth, wave height, and lowest eroded ground elevation for 100-year return period will be determined. Finally, the stillwater depth, wave height, and maximum wave crest elevation for other given return periods will be calculated based on the 100-year return period results. The flowchart of estimating 100-year flood parameters is shown in Figure 2-12.

2.5.1.1 Preliminary values for E_{100} and BFE

As shown in Figure 2-12, there are three sources of E_{100}: (a) 100-year SWEL from FIS report (including wave setup from DRIRM), (b) 100-year TWL from FIS report, and (c) the average value of the above two data sets. E_{100} can be obtained from any one of these three values. In this research, the average values are used as the preliminary E_{100} value for the following calculations.

As discussed earlier, the values of BFE shown in the flood hazard zones on DFIRM are elevations rounded to the nearest foot. Therefore, if an SFHA is mapped as the AE zone (EL 14), the actual BFE ranges from 13.5 ft to 14.4 ft. In the process of determining flood parameters, these rounded whole-foot elevations shown on DFIRM are adopted as the preliminary values of BFE. After analyzing E_{100} and BFE along each transect, elevations are interpolated to the coastal flood zones. Revisions of E_{100} and BFE might be carried out to satisfy the conditions required in the following steps of food parameter determination.

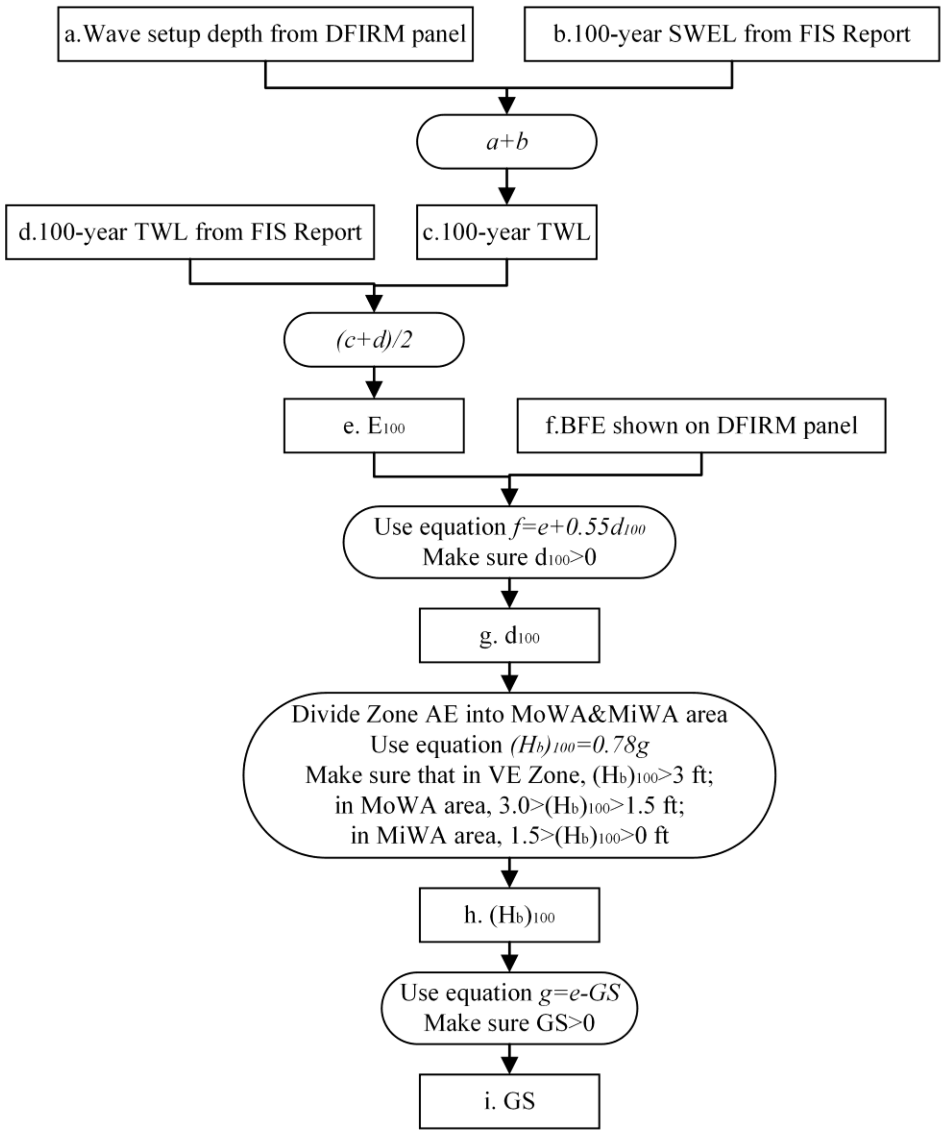


Figure 2-12 Flowchart for estimating 100-year flood parameters

2.5.1.2 Stillwater depth, wave height, and ground elevation for the 100-year return period

This step is to determine (a) stillwater depth, (b) wave height, and (c) lowest eroded ground elevation during the 1% annual chance flood event. It is worth noting that several restrictions must be satisfied during this step:

1) All the values of flood parameters must be no less than zero.
2) Calculations of coastal flood elevations should follow equations 2-1 to 2-4.

3) Wave heights should satisfy certain limitations in each coastal flood zone. Specifically, wave heights in VE Zone are no less than 3 ft, wave heights in MoWA area are between 1.5 ft and 3 ft, and wave heights in MiWA area should be no bigger than 1.5 ft.

In order to satisfy the requirements listed above, iterations and slight adjustments of preliminary flood elevation parameters should be conducted as follows:

1) Redefine E_{100} value for specific transect, i.e., replace values obtained from (c) by (a) or (b), whichever is smaller.
2) Redefine BFE for a specific flood hazard zone, i.e., pick a larger value within the allowable BFE range as discussed earlier.

After several iterations, the modified flood parameters are within the range defined in SFHAs. The DFIRM panel with more precise BFEs is shown in Figure 2-13. In summary, typical 100-year flood characteristics affecting flood level determination are obtained using the flowchart depicted in Figure 2-12. Some of these parameters will be further used to calculate the stillwater depth, wave height, and maximum wave crest elevation for other return periods.

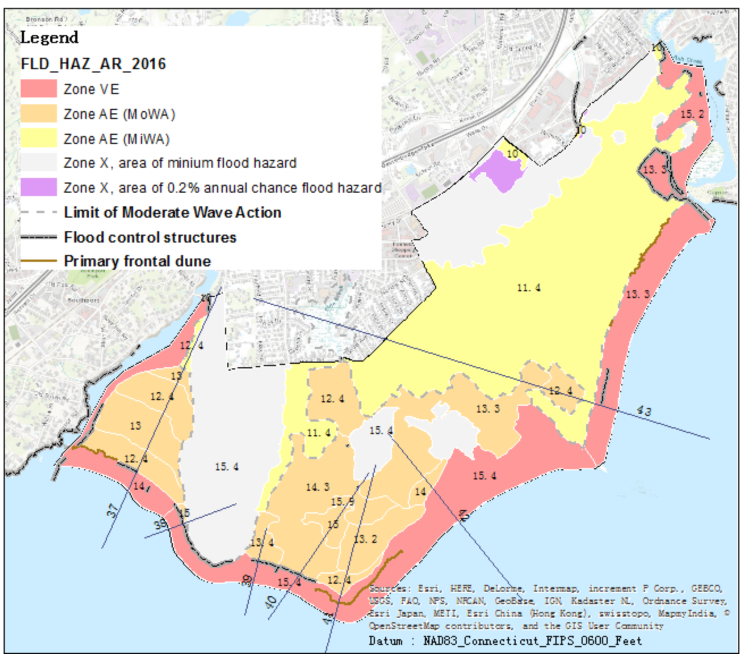


Figure 2-13 SFHAs with modified BFEs in Fairfield coastal community

2.5.1.3 Stillwater depth, wave height, and maximum wave crest elevation for 10-, 50-, and 500-year return periods

After obtaining the lowest eroded ground elevation from the previous calculations, flood parameters for other return periods can be obtained using a similar method. The flowchart of determining flood characteristics for a 10-year return period is shown in Figure 2-14 as an example. The flood characteristics of 50-year and 500-year flood hazard events will follow the same evaluation procedures. All the values should be positive and therefore are checked to assure that all prerequisites are met. Almost all of the parameters meet the requirements in this step since the two primary parameters (BFE and 100-year TWL) that they based on are obtained from the iterative procedures proposed in the last subsection with sufficient accuracy.

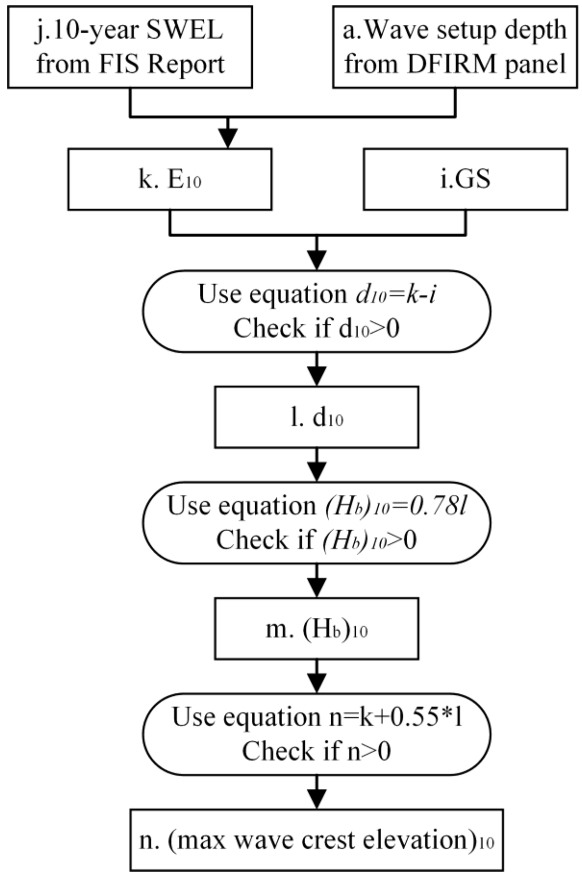
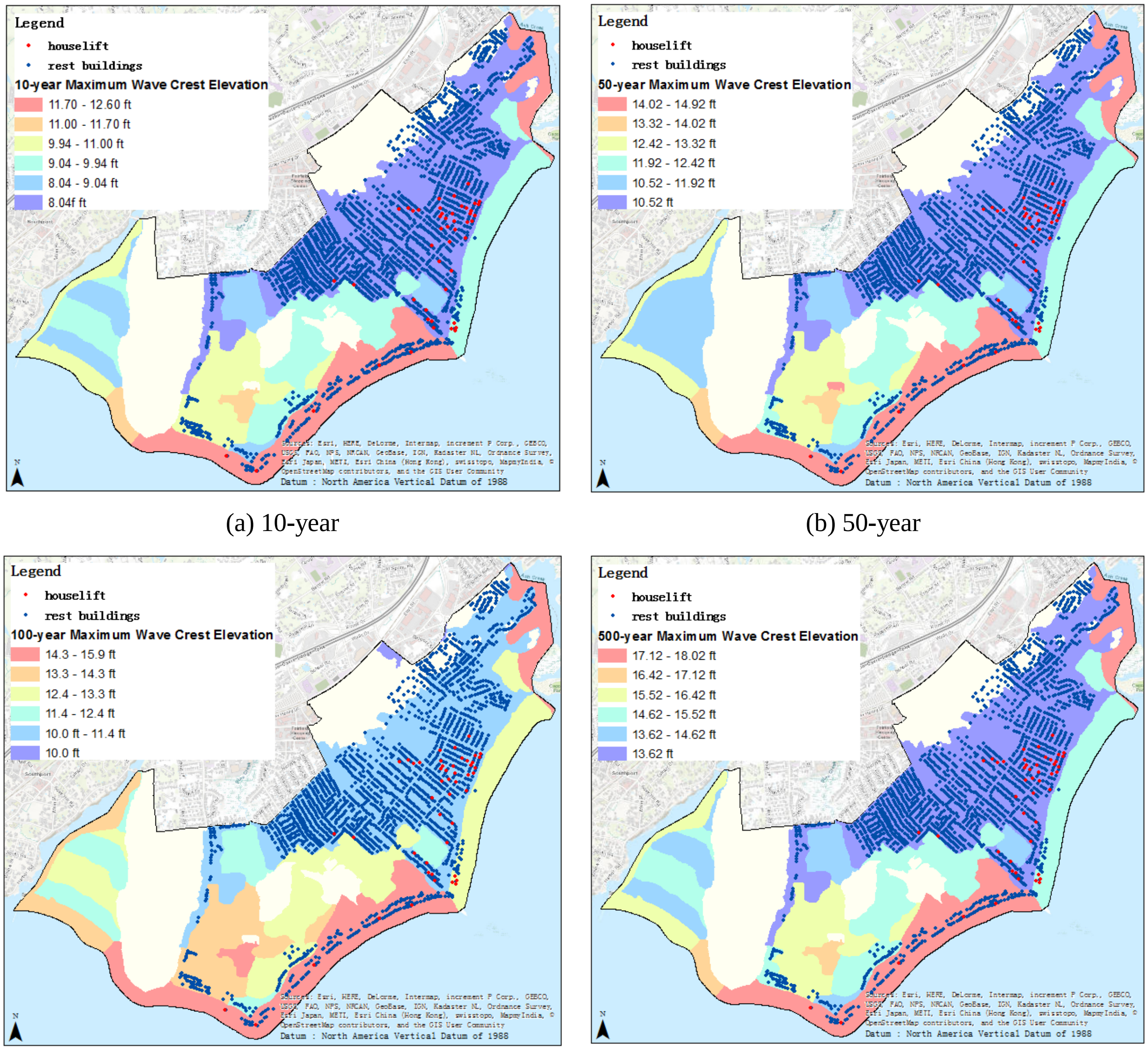


Figure 2-14 Flowchart for estimating 10-year flood parameters

Flood parameters obtained in this step can be applied to generate the community-focused GIS database containing expected damages in the following sections. For a demonstration purpose, the calculated maximum wave crest elevations with different return periods in Fairfield coastal community are plotted in Figure 2-15.



(a) 10-year (b) 50-year

(c) 100-year (d) 500-year

Figure 2-15 Maps of calculated maximum wave crest elevation corresponding to different flood recurrence periods in Fairfield coastal community

Note that some flood zones, such as X Zones, which have minimum flood hazard or 0.2% annual chance flood hazard, show no BFEs or maximum wave crest elevations. For the VE and AE Zones where BFE data can be obtained from FIS report and DFIRM panel, the calculations are conducted to estimate flood characteristics using the flowcharts shown in Figure 2-12 and Figure 2-14 according to the given return period.

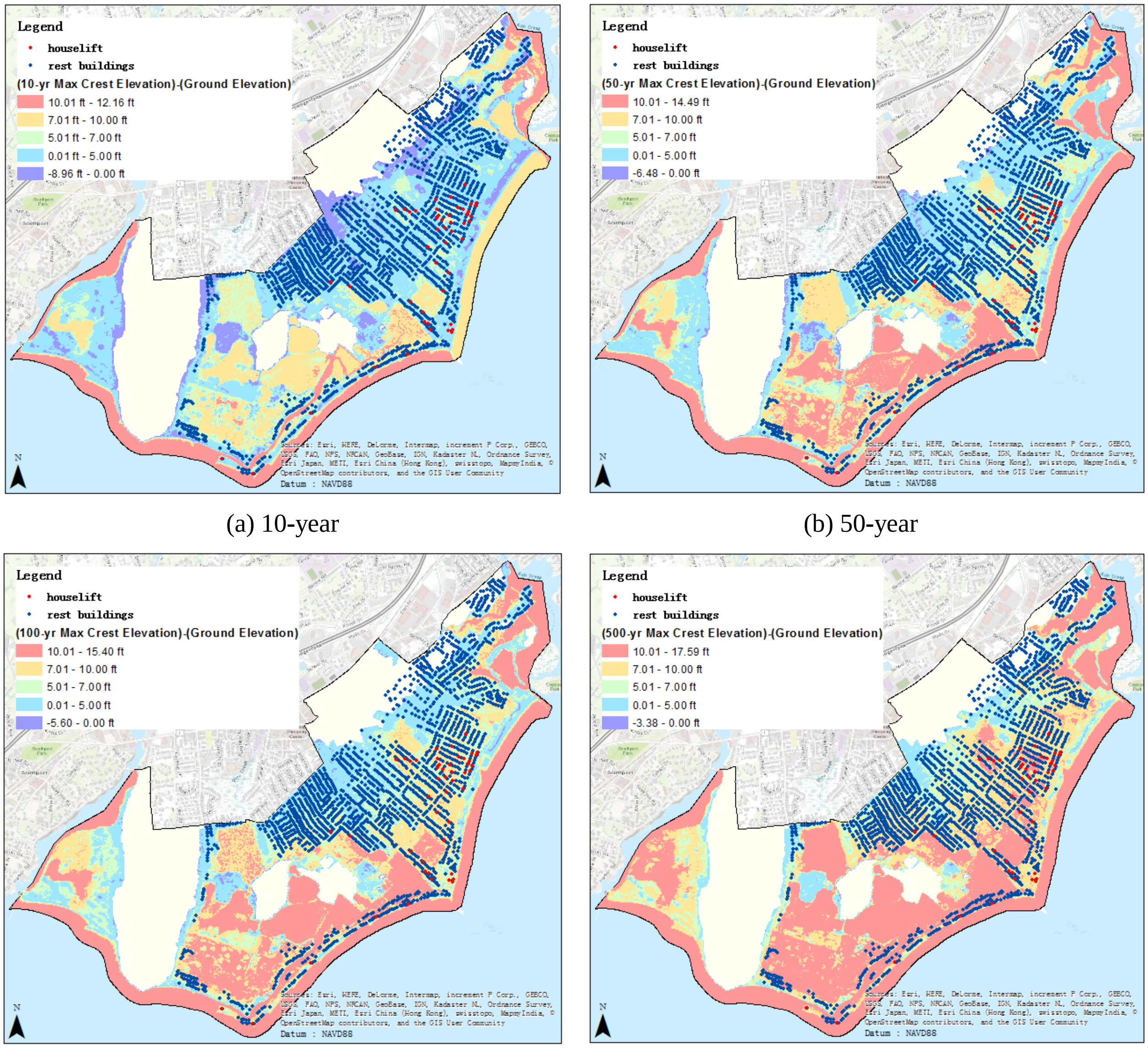
2.5.2 GIS-based community flood level map

In this section, flood parameters obtained earlier are used to build a community-focused GIS database and generate community flood level maps for different recurrence intervals. Along each coastal transect, flood elevations are calculated based on the data extracted from FIS and DFIRM panel attribute tables, the change of the ground elevation and engineering judgment. Between coastal transects, flood elevations are interpolated into flood zones using various sources including the local topography, and the land-use data to evaluate the severity of flooding events within the community. Flood levels for the residential building structures are examined using the GIS maps in this section. Furthermore, the total number of flooded homes is discussed to show how many residential buildings are affected by a given return period of the flooding water level.

2.5.2.1 Maximum wave crest elevation to existing ground elevation

This section utilizes the maximum wave crest elevation maps and the existing ground elevation data to generate a new series of maps taking the effect of maximum wave crest on the coastal communities into account. The existing ground elevation is acquired from a digital elevation model (DEM) dataset obtained from the Connecticut Department of Energy & Environmental Protection (DEEP). The ground elevation data and flood elevation data are expressed in the NAVD 88 vertical datum with the unit of US Survey feet, which represents the number of feet above mean sea level in that datum. In this research, all of the elevation data are referred to the NAVD 88 datum unless otherwise stated.

After both the maximum wave crest elevation maps and the topographic maps for the Fairfield and Milford coastal communities are obtained, the ground elevation (topographic elevation) at one particular location is subtracted from the maximum wave crest elevation at that location to calculate the flood depth based on the maximum wave crest. This flood depth will be the actual maximum flooding height for a residential building structure at the corresponding location. Figure 2-16 shows the maximum flood level that the residential homes encounter during 10-year, 50-year, 100-year, and 500-year return periods for Fairfield. As shown in Figure 2-16, the maximum flood level has an increasing trend with the increase of return periods in the community. Meanwhile, the flood level data, referred to the difference between maximum wave crest elevation and ground elevation, positively correlates with the flood recurrence period. In addition, the flood level decreases towards the inland direction, indicating that the flood level is higher in VE Zone than that in AE Zone. However, there are still some exceptions for some elevated buildings. This is possibly due to the relatively limited elevated building data in the VE Zones. The estimated flood level is so important that the residential homeowners might consider elevating their residences with ample foundations to resist the potential flood hazard. After discussions with town engineers and other building engineers, nine feet is typically used as the elevation height to avoid potential damages from flooding.



(a) 10-year

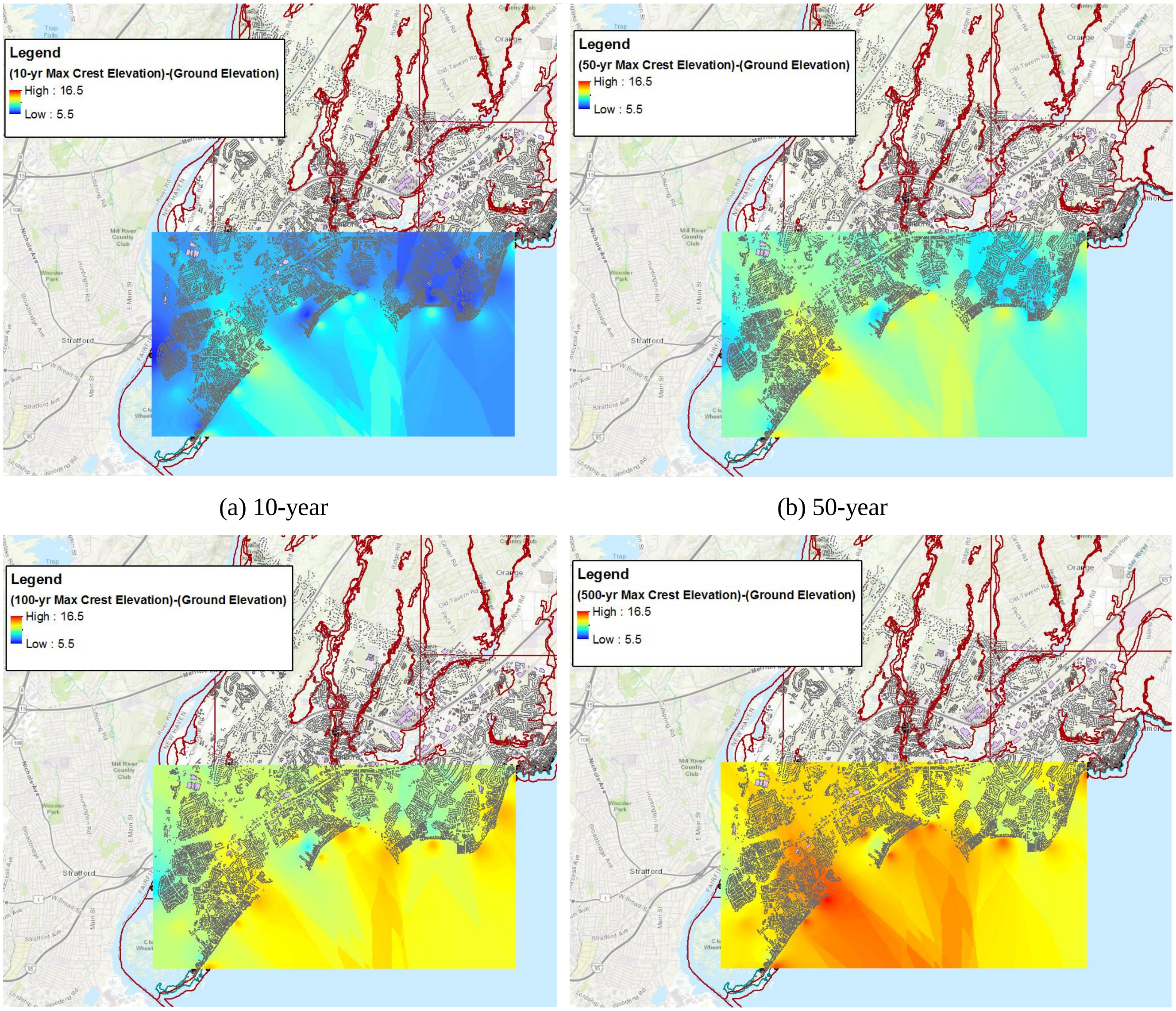
(b) 50-year

(c) 100-year

(d) 500-year

Figure 2-16 Fairfield coastal community flood hazard maps based on maximum wave crest elevations for different flood recurrence periods

Figure 2-17 shows the flood hazard maps based on maximum wave crest elevations for the Milford coastal community subjected to flooding events with 10-year, 50-year, 100-year, and 500-year return periods. Same conclusions could be drawn for the Milford coastal community that the maximum flood level increases as the return period of the flooding event increases. Meanwhile, the potential damage due to maximum wave crest elevation is reduced towards the inland direction. To avoid severe flood hazards, residential buildings located in the regions represented by the red color should be elevated to a certain height based on the generated flood hazard maps shown in Figure 2-17. Overall, the GIS-based flood level maps demonstrate the maximum flood level in the Fairfield and Milford coastal communities during various flooding events. The total flood levels for the study regions are discussed in the next section.



(a) 10-year (b) 50-year

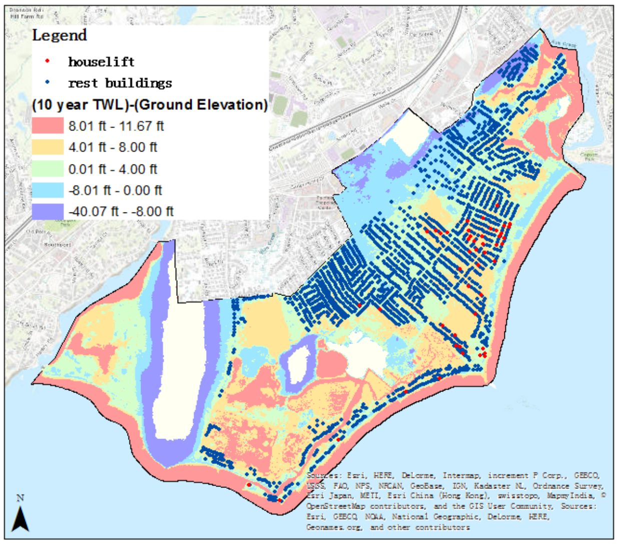
(c) 100-year (d) 500-year

Figure 2-17 Milford coastal community flood hazard maps based on maximum wave crest elevations for different flood recurrence periods

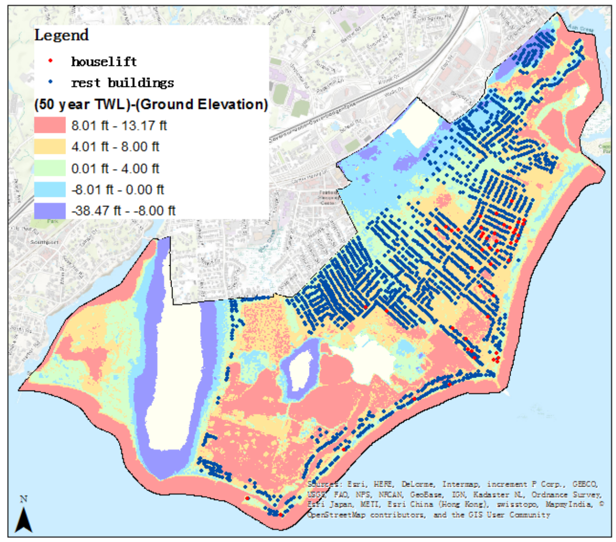
2.5.2.2 Flooding height

The flooding height is obtained by subtracting the existing ground surface level from the total water level (TWL). The TWL data are then combined with the existing ground elevation map to illustrate the actual total flood elevation in Fairfield and Milford corresponding to 10-year, 50-

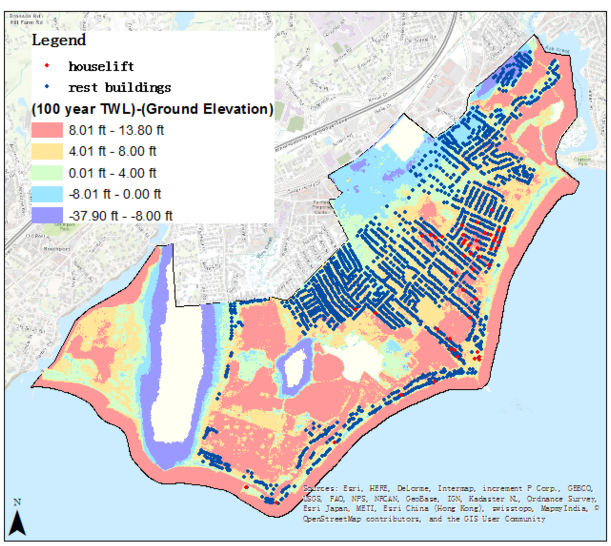
year, 100-year, and 500-year flood return periods. Figure 2-18 shows the flood hazard maps based on the total water level under various return periods for the Fairfield coastal community. As shown in Figure 2-18, the total flood level positively correlates with flood recurrence periods and decreases towards the inland direction. Residential building structures are less likely to suffer from severe flood hazards during a 10-year flood event, compared with those during a 100-year flood event. Figure 2-18 suggests that the total flood level is below 4 ft during a 10-year flood event, while it is above 4 ft in a 100-year flood event bringing more potential damages to the structures. Meanwhile, comparisons of Figure 2-16 and Figure 2-18 also show that the total flood level is lower than the maximum flood level due to not considering the effects of wave height.



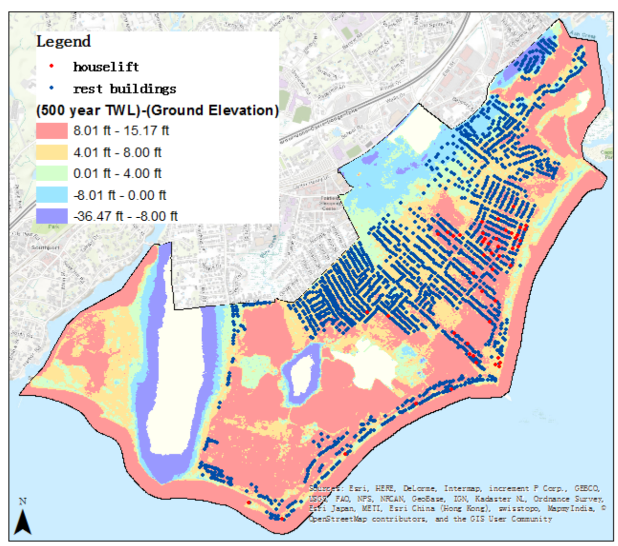
(a) 10-year



(b) 50-year



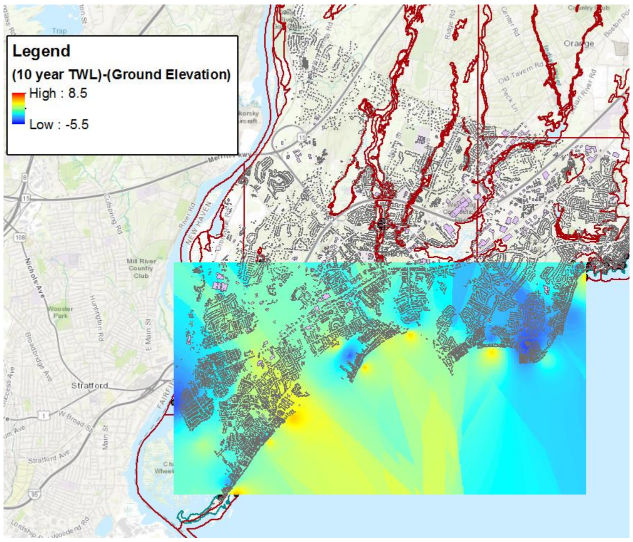
(c) 100-year



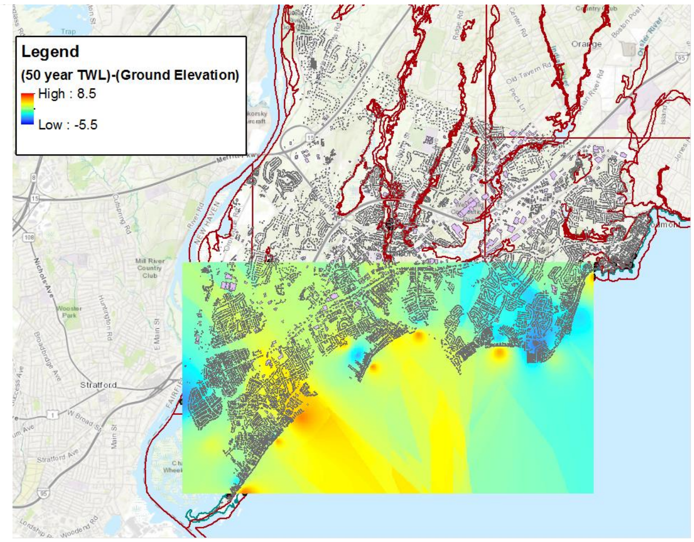
(d) 500-year

Figure 2-18 Fairfield coastal community flood hazard maps based on total water levels for different flood recurrence periods

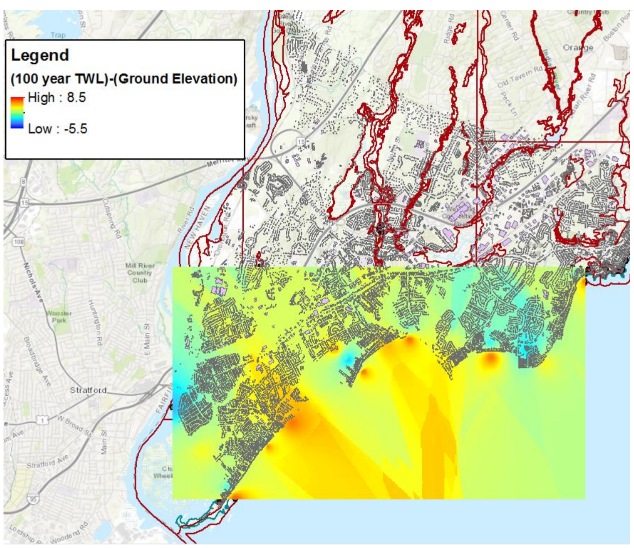
The flood hazard maps based on total water levels for the Milford coastal community subjected to flooding events with 10-year, 50-year, 100-year, and 500-year return periods are shown in Figure 2-19. Similar to the Fairfield coastal community, the total flood level in the Milford coastal community positively correlates with the flood recurrence periods and decreases towards the inland direction.



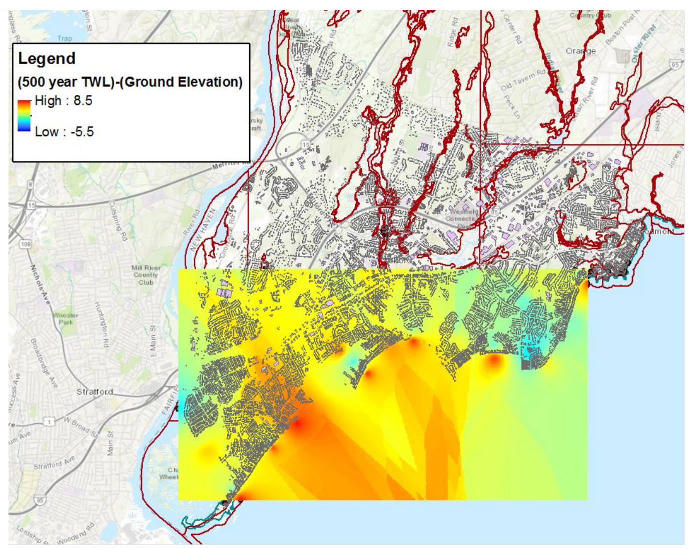
(a) 10-year



(b) 50-year



(c) 100-year

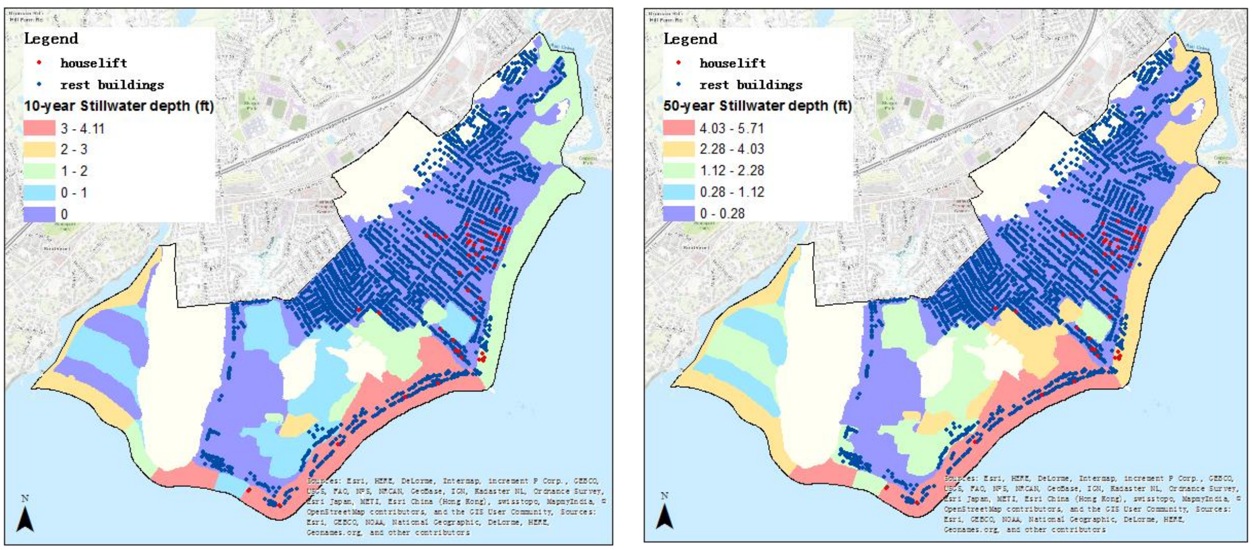


(d) 500-year

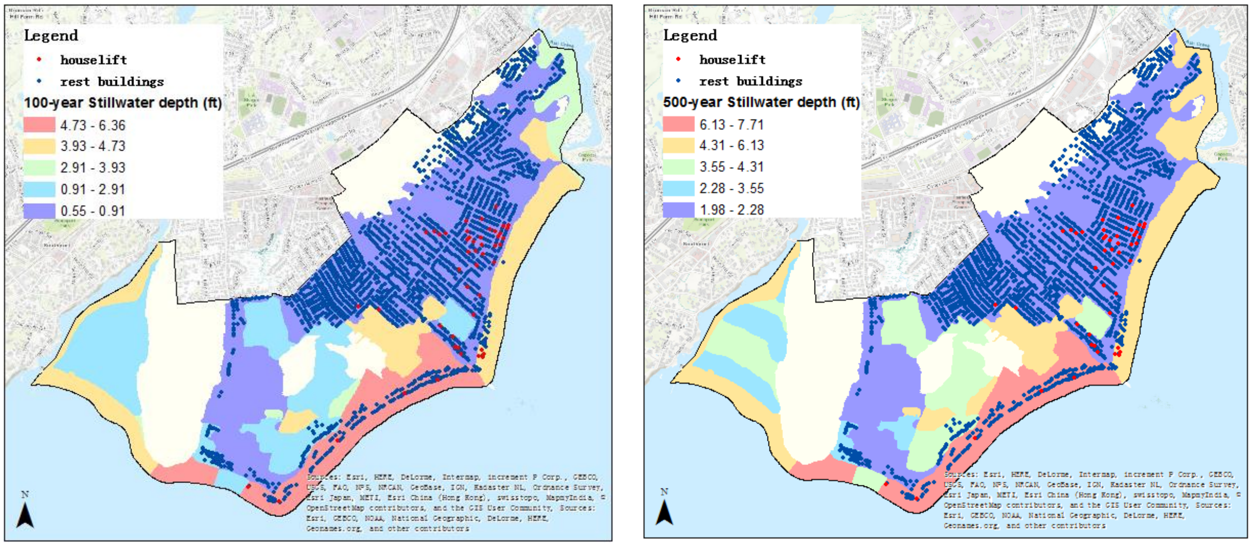
Figure 2-19 Milford coastal community flood hazard maps based on total water levels for different flood recurrence periods

2.5.2.3 Stillwater flood depth

Stillwater depth (d_s) is the height difference between the stillwater elevation (including wave setup) and the lowest eroded ground surface elevation (GS) adjacent to the building. As defined in the Coastal Construction Manual, GS is not the lowest existing pre-event ground elevation. Instead, it represents the amount of long-term erosion during a base flood event excluding the effects of local scour around the foundation of the building. Stillwater depth is the most important coastal flood parameter in the flood load calculations. Almost all the other flood parameters and flood load calculations depend directly or indirectly on the Stillwater depth (d_s). Figure 2-20 shows the stillwater flood depth in the Fairfield coastal community during different flood scenarios. The values of d_s are calculated using the flowcharts shown in Figure 2-12 and Figure 2-14 and are further interpolated into each flood zone. As shown in Figure 2-20, stillwater flood depth is separated into five different ranges using the natural breaks (Jenks) method. Similar to Figure 2-16 and Figure 2-18, the stillwater depth shown in Figure 2-20 has an increasing trend with the increase of return periods in the Fairfield coastal community.



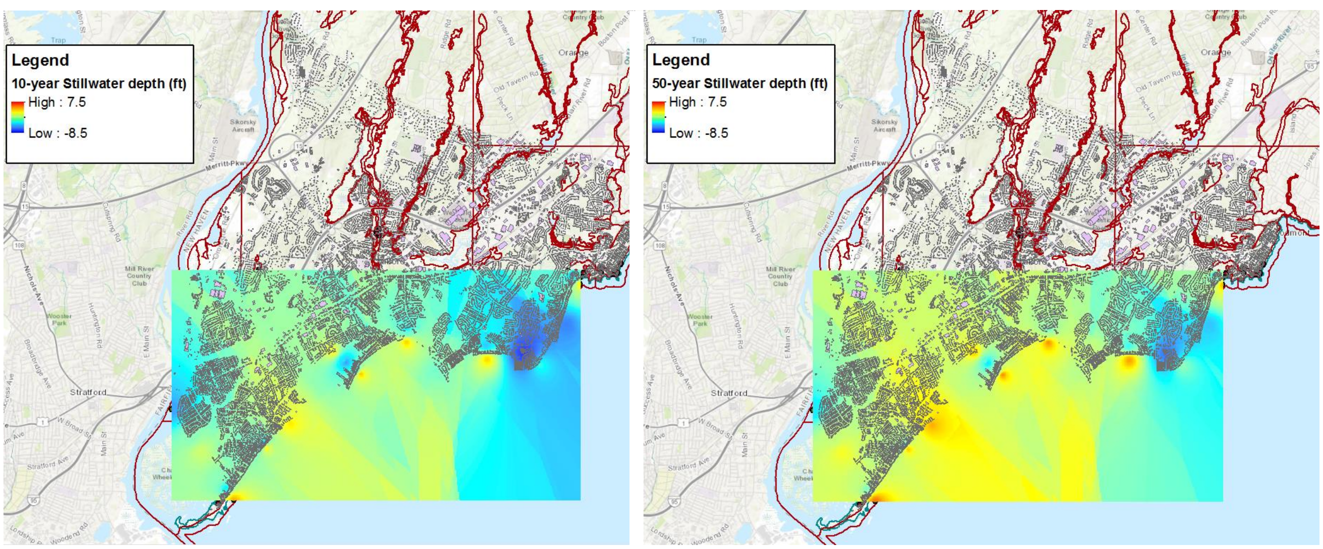
(a) 10-year (b) 50-year



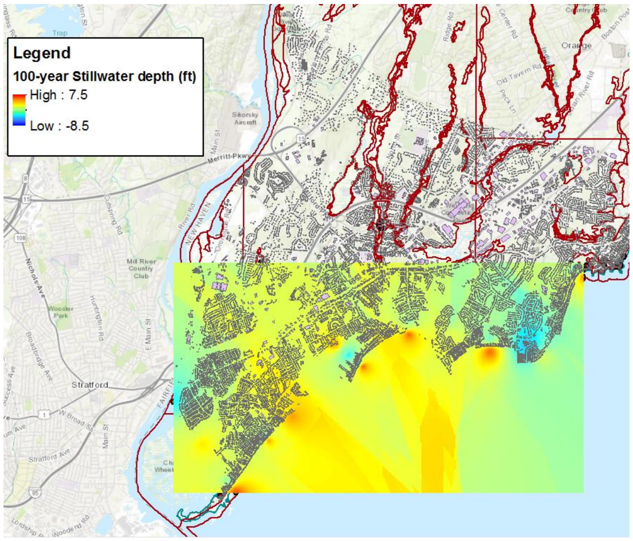
(c) 100-year (d) 500-year

Figure 2-20 Fairfield flood hazard maps based on stillwater flood depths for different flood recurrence periods

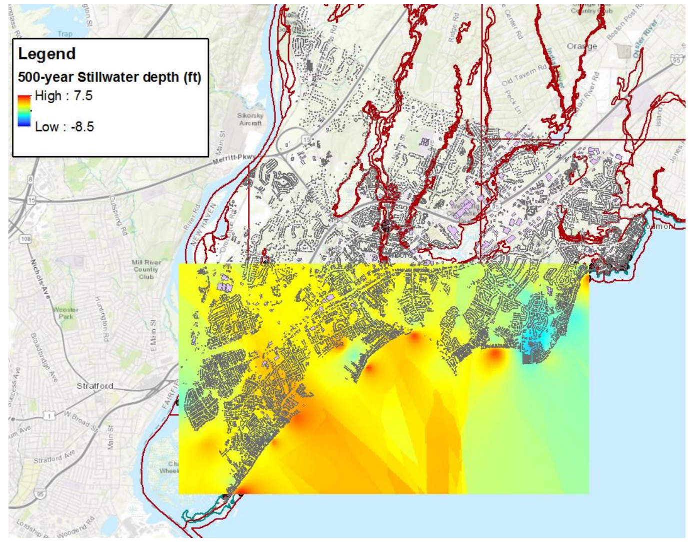
Figure 2-21 shows the stillwater flood depth in the Milford coastal community during different flood scenarios. Again, the same conclusions could be drawn that the stillwater depth has an increasing trend with the increase of flood return periods in the Milford coastal community.



(a) 10-year (b) 50-year



(c) 100-year



(d) 500-year

Figure 2-21 Milford coastal community flood hazard maps based on stillwater flood depths for different flood recurrence periods

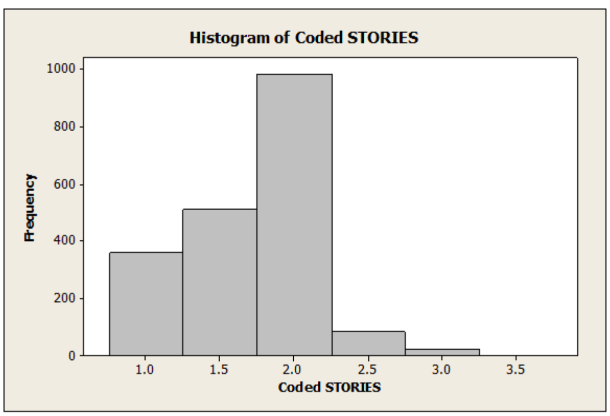
In summary, a series of GIS-based flood hazard resilience maps are generated in this chapter using high-quality flood elevation data and high-resolution DEM information for the Fairfield and Milford coastal communities. These flood level maps could show actual flood elevation height for each residential building structure in the coastal community. With these flood levels information, the vulnerability of the entire coastal community due to the potential flood hazard risk at any given return periods could be assessed. Furthermore, these data could also help provide suggestions for the elevation heights of residential buildings to avoid flooding on the main building structures. Flood parameters obtained from this chapter will be directly used in the flood load calculations (e.g., hydrostatic load, hydrodynamic load, etc.) in the following chapter.

CHAPTER 3 Single Family Buildings Modeling

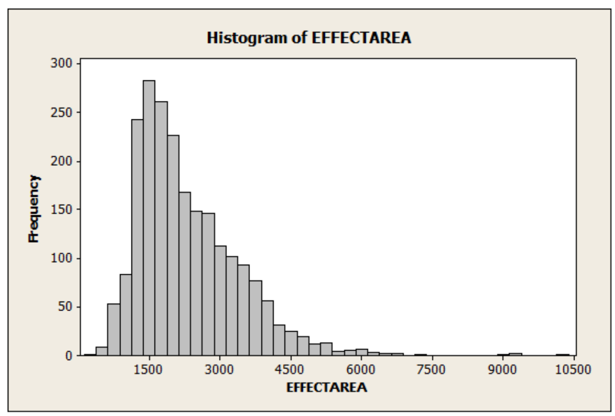
After identifying the flood parameters, single-family buildings modeling and analysis, as well as flood and wind loads modeling, are introduced in this chapter. Firstly, the totals of 2021 coastal residential structures are statistically grouped by key design parameters, including the year built, the number of stories, and the type of roof and the footing size of the buildings. Based on the statistical analysis, three representative building models with and without elevation system are selected to approximately represent all the single-family buildings in the Fairfield and Milford coastal communities. The structural details in the numerical modeling approach for both no-elevated and elevated buildings are also described in this chapter.

3.1 Statistical analysis and building models selection

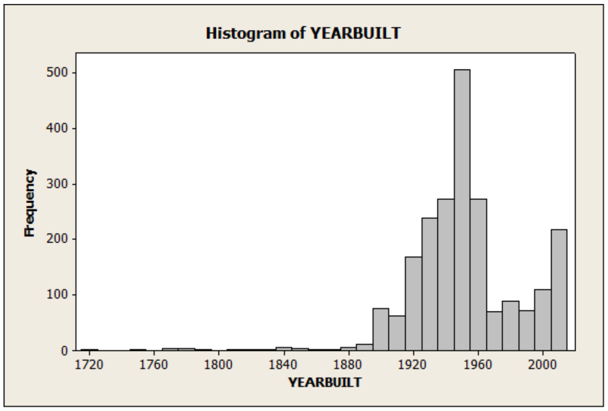
Based on the GIS dataset for the residential building inventory in the coastal community provided by the town of Fairfield, the architectural and construction parameters for 2021 residential buildings are extracted from the dataset to perform a statistical analysis. To reduce the modeling and simulation cost, several parameters, such as the number of stories, the footing size of the residential buildings, and the year built are used to group these residential building structures. The statistical information of residential buildings in the GIS dataset provided by the city of Milford are similar to those provided by the town of Fairfield. Therefore, only the statistical analysis results for residential builds in Fairfield are shown in Figure 3-1.



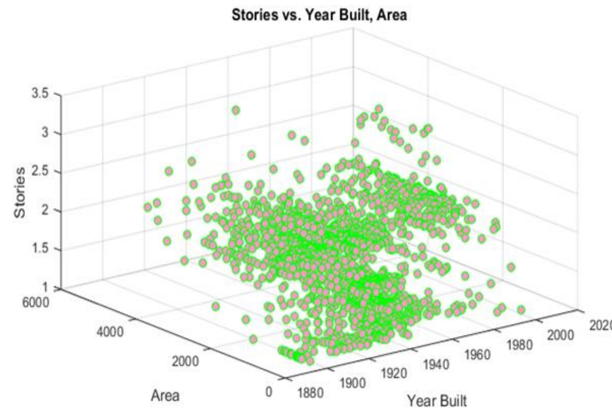
(a) Number of stories



(b) Footing size of the building



(c) Year of built



(d) Statistics result

Figure 3-1 Statistical information on residential buildings dataset

With a further discussion with town engineers, three representative building structures that could represent the typical designs and construction methods in the Fairfield and Milford communities are used in this research. Detailed design parameters for the three building structures are summarized in Table 0-1. Meanwhile, a typical construction scheme for residential building structures is shown in Figure 3-2.

Table 0-1 Details of three representative residential building structures

Model	1	2	3
Year built	1930's	1960's	1990's
Floorplan	24'x48' rect.	24'x48' rect.	24'x48' rect.
Stories	1.5	1	2
Roof pitch	9:12	5:12	5:12
Wall framing	2x4 stud, 16" o.c.	2x4 stud, 16" o.c.	2x4 stud, 16" o.c.
Wall sheathing	3/4"x10" boards	1/2" plywood	1/2" plywood
Roof framing	2x6 rafter, 16" o.c., collar ties	2x4 truss, 24" o.c.	2x10 rafter, 16" o.c.
Roof sheathing	5/8" plywood (considered to be updated due to age)	1/2" plywood	5/8" plywood
Roof to wall connection	2-16d toe nails, rafter to plate	2-16d toe nails, rafter to plate	Hurricane ties, use typical holding power
Interior walls	Single down center	Single down center	Single down center
Material	SYP	SYP	SYP

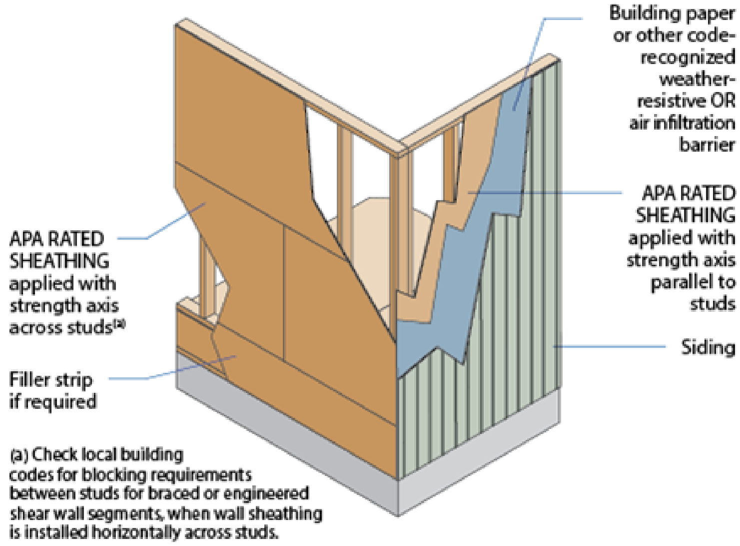
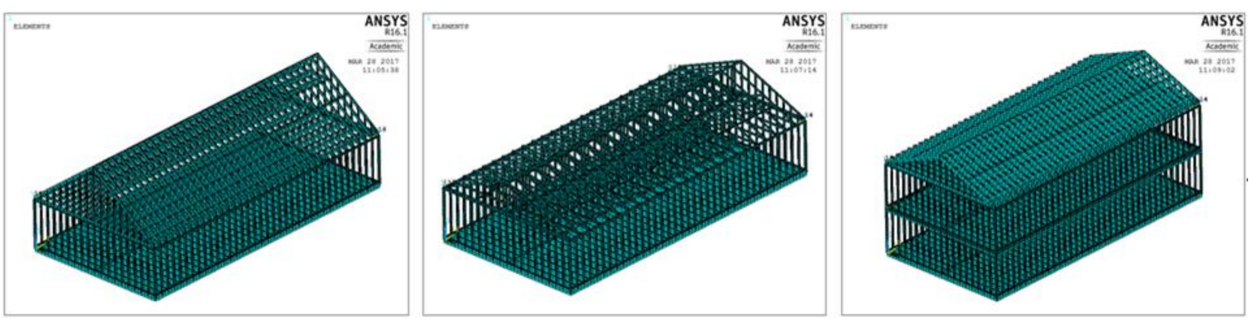
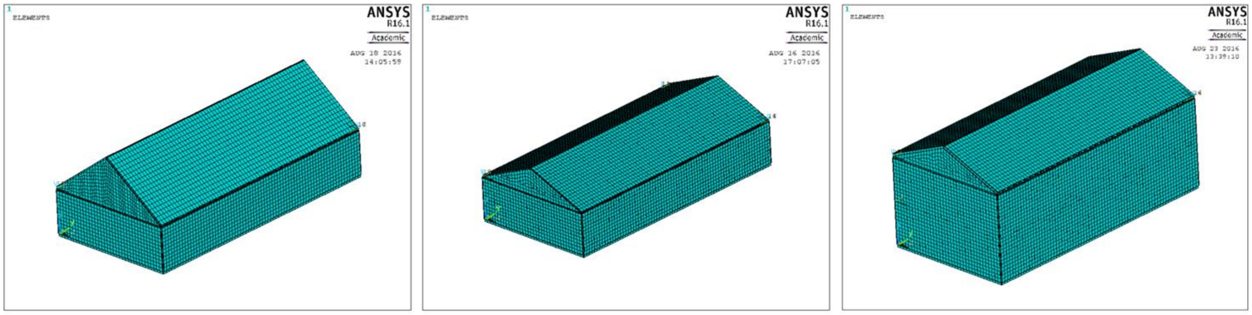


Figure 3-2 Typical construction configuration for residential building structures

3.2 Residential building modeling

As discussed in the last subsection, three types of home designs are adopted in this research to represent the residential building structures in the Fairfield and Milford coastal communities, namely, the models that present the building built around 1930, 1960, and after 1990. The building framing structures and detailed modeling procedures follow the International Building Code (IBC 2011). The finite element models for the building frames and sheathing/roof are shown in Figure 3-3.

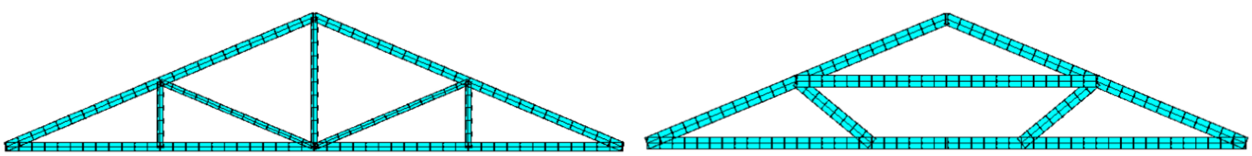




Model 1 (1930's) | Model 2 (1960's) | Model 3 (1990's)

Figure 3-3 1930's, 1960's, 1990's framing patterns and sheathing/roof in finite element models

Framing techniques and material dimensions vary by the age of the structure. In order to capture custom construction practices in the local community, in model 1 (the 1930's), smaller lumber is used for stick-framed roof than what is commonly used in modern practices. Based on the localized information provided by Fairfield and Milford communities, this style of home has a longer history of using boards for wall sheathing rather than the plywood used in modern construction. Since the original sheathing of the roof has been replaced in most of these old buildings, model 1 uses plywood as the material of the roof sheathing. To better represent a typical residential building in the 1930s, a 1.5-story building structure with a steeper roof slope is used in model 1. Model 2 (the 1960's) seeks to present a more mass-produced building during that period from 1950-1970. Prefabricated engineered trusses were used to frame the roof rather than using the stick roof framing method with rafter ties or purlins on every other as shown in Figure 3-4. Model 3 (1990's) represents a common 2-story configuration built after 1980 with heavier lumber roof framing and thicker roof sheathing to increase mean roof height and resist higher wind loads.



(a) Howe truss roof framing | (b) Stick-framed roof

Figure 3-4 Schematic diagram of roof framing system

3.3 Elevation system

Elevation systems for the coastal residential buildings were encouraged in recent years to reduce flood damages. Typical elevation system could use either concrete, steel or wood. After communicating with town engineers, local construction firms, and local builders, as well as referring to the pile layout on Coulbourne's paper (Coulbourne 2013), concrete piles and steel girders will be used in this research as shown in Figure 3-5.

(a) Concrete piles (b) Steel girders

Figure 3-5 Elevation systems in Fairfield coastal community

The elevation model in this research is made up of 16” diameter pre-stressed reinforced concrete piles, and a support frame of W10x12 steel beams rigidly connected to piles with concrete expansion anchors. Rim joist and floor joists rest on steel girders. The elevation height of home is 9’ in accordance with common practices in the local community. The elevation system applied in all three residential building structural models is illustrated in Figure 3-6. Moreover, the corresponding elevated building models are depicted in Figure 3-7.

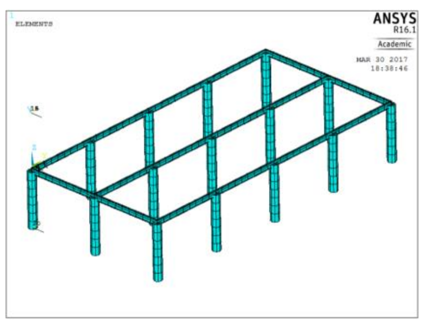
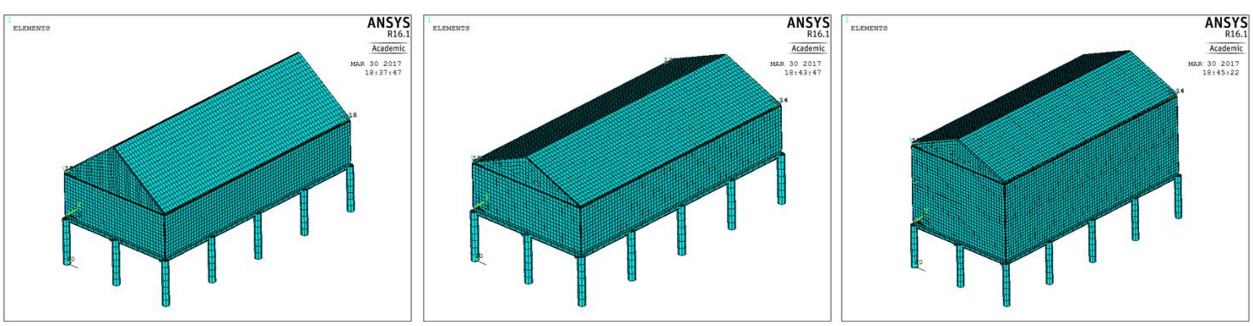


Figure 3-6 Structural model of the elevation system



Model 1_elevated (1930’s) Model 2_elevated (1960’s) Model 3_elevated (1990’s)

Figure 3-7 Structural models of elevated residential buildings

3.4 Material properties

The mechanical properties of wood vary significantly in grain orientations and moisture conditions. In this research, an average isotropic material model is adopted with wood property in the weakest grain direction to consider the worst case scenario. Lumber properties are referred to the Forest Products Laboratories Report: FPL-GTR-190 (Kretschmann 2010), and properties for plywood sheathing are taken from the American Plywood Association (APA 1997). For the elevation systems, the concrete column is made up of 41.4MPa compressive strength mixture (ACI Committee 318 2005; McCormac 1986), while W10x12 beam girders are made with A992 steel (American Institute of Steel Construction 2005). The details of material properties can be found in the table below:

Table 0-2 Material properties (Weston and Zhang 2016)

Component	Young's Modulus Ex/Ey/Ez (MPa)	Shear Modulus Gxy/Gxz/Gyz (psi)	Poisson's Ratio
Framing Lumber	8.3e3	-	0.4
Plywood Sheathing	2e3/2e3/13.1e3	1e3/1e3/1e3	0.08/0.08/0.08
Concrete Columns	23.2e3	-	0.3
W10X12 Beams	200e3	-	0.3

CHAPTER 4 Vulnerability Assessment

4.1 Failure criteria

4.1.1 Failure criteria for sheathing

In this research, the damage is defined by sheathing loss. Sheathing is evaluated for three failure modes: stress, nail withdrawal and nail pull through. First of all, the Von Mises stress in sheathing may exceed an allowable limit and cause sheathing failure due to strength. Secondly, the relative displacement between sheathing and framing may exceed an allowable value which will result in water intrusion. Thirdly, the nail head may pull through and give rise to sheathing failure. Among these three modes, the first and third are coupled. Details of the three failure criteria include in this analysis are presented in Table 0-1 below:

Table 0-1 Sheathing failure criteria (Weston and Zhang 2016)

Failure Criterion	Allowable limit	Comment	Reference
Von Mises stress in sheathing	1.131 MPa	Wet condition and 1/6th safety factor	APA (APA 1997)
Nail withdrawal	L/120	Relative displacement, L is spacing of framing	IBC (IBC 2011)
Nail pull through	316, 409N axial force	Wet condition, 1.3cm and 1.59cm respectively.	(Herzog and Yeh 1999)

4.1.2 Damage index

After performing the static analysis for all building models subjected to different combinations of wind and wave loads, the failed shell elements as defined in the previous subsection could be detected automatically using an ANSYS macro file. To predict the sheathing vulnerability, the damage index, which is defined as the ratio of the failed shell elements to the overall shell elements on the specified sheathing, is selected as the parameter to evaluate the performance of different building models under wind and wave loads and to obtain the vulnerability curves.

In this research, the damage index is evaluated separately on the roof and walls due to the difference of framing and sheathing systems in these two parts. Meanwhile, this separated treatment could also benefit in accounting for the sensitivity of the behaviors of these two parts to wind and wave loads independently. In addition, the damage index for the walls is also evaluated separately for the seaward wall (front wall), and non-seaward walls (left, right, and back walls) since only the seaward wall is subjected to both wind and wave loads. In contrast, the other three walls only have wind loads acting on them. Presently, window and door failure are not considered.

4.2 Vulnerability curves

The results of vulnerability curves are presented and discussed according to each one of the building models in both elevated and non-elevated configurations. For all following plots of vulnerability curves, the abscissa (d_s) represents the stillwater depth, the ordinate represents the damage index, and different curves represent different wind speeds. The performances of three house models under combined wind and flood hazards are very similar. For the sake of brevity, only detailed discussions for model 1(1930's model) are provided in the following subsections.

4.2.1 Non-elevated model

Based on the definition of damage index, vulnerability curves for model 1 with traditional foundations regarding damage rate for the seaward wall, non-seaward walls, all walls, and the roof are presented in Figure 4-1 to Figure 4-4. As shown in Figure 4-1 the sheathing damage rate of the seaward wall is almost the same when the stillwater depth is below 1.31 ft for all the cases with wind speed ranging from 70 mph to 130 mph. Even though a small local peak could be observed on the vulnerability curves under the wind speed of 140 mph and 150 mph, the largest variation of the damage rate is only about 0.15. This suggests that wind loading is the dominant factor that could affect sheathing failure on the seaward wall when the stillwater depth is less than 1.31 ft. As the stillwater depth increases above 1.31 ft, wave loading gradually becomes the dominant factor resulting in sheathing failure rather than wind loading. As shown in Figure 4-1, the damage index increases rapidly as the stillwater depth increases over 1.31 ft. Meanwhile, the largest difference of the damage index among all wind speeds varies from 0.21 at 1.64 ft stillwater depth to 0.94 at 3.61 ft stillwater depth. This suggests that the wind loading is not an important factor that could affect sheathing failure on the seaward wall compared with wave loading when the stillwater depth is large enough.

Figure 4-2 shows the damage index for the other three walls not directly impinged by the incoming wave. The results indicate that the sheathing failure ratio depends on the wind speed only when the stillwater depth is below 2.3 ft since these three walls are only subject to wind loads. However, when the stillwater depth is above 2.3 ft, the impact of wave loads acting on the seaward wall becomes significant and could be transferred to the entire building system through the building frame. Therefore, the sheathing damage rate for the other three walls could be affected by the stillwater depth when it is large enough, and the loads can be transferred to propagate the damages to the entire building structural system. Different from the damage index for the seaward wall, the damage index for the other three walls has an increasing trend from either wind speed increase or stillwater depth increase.

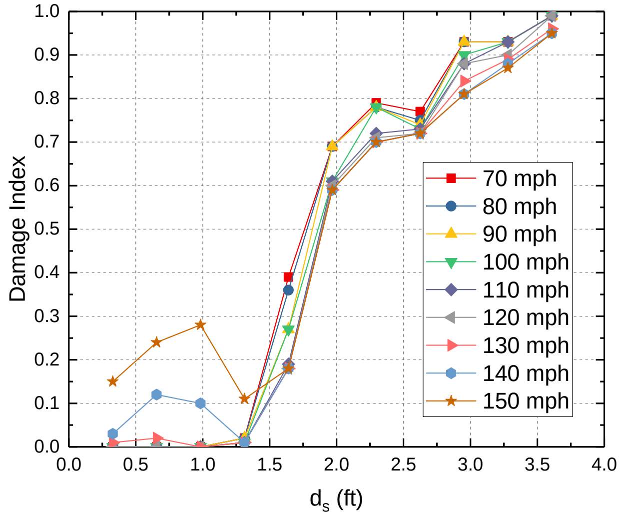


Figure 4-1 Seaward wall damage index for non-elevated model 1

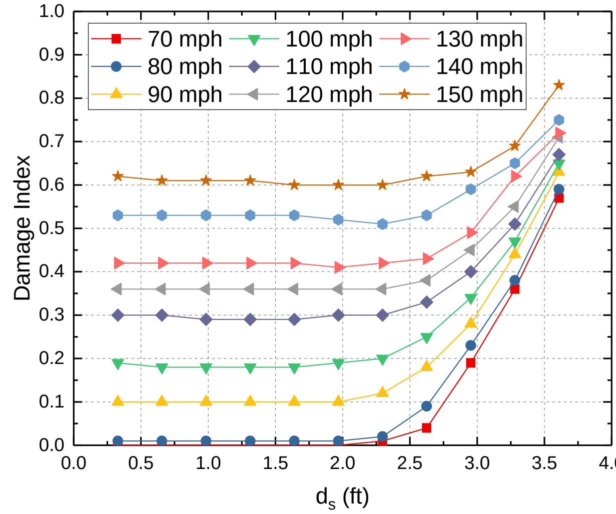


Figure 4-2 Non-seaward walls damage index for non-elevated model 1

The results of the damage index for all four walls are shown in Figure 4-3. A similar trend can be observed for all the non-seaward walls except that the effect of wave loads on sheathing damage is triggered at a lower stillwater depth (1.31 ft). Meanwhile, the results also suggest that larger wind speed and stillwater depth will lead to a larger sheathing damage rate if seaward and non-seaward walls of the building under wind and wave loads are considered together. Figure 4-4 shows the damage index for the roof sheathing. The trend is similar to the damage index for the sheathing of non-seaward walls that are only subject to wind loads. As shown in Figure 4-4, the wave loads have an almost negligible effect on sheathing damage until the stillwater depth increases up to 2.75 ft that the wave loads on the seaward wall could play an important role in the response of the entire structure. Furthermore, with a large stillwater depth, the effect of wave loads on roof sheathing damage is more pronounced for the cases with lower wind speed.

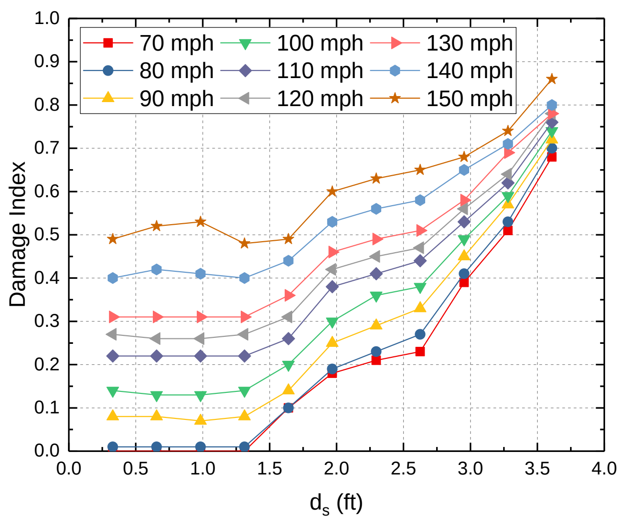


Figure 4-3 All walls damage index for non-elevated model 1

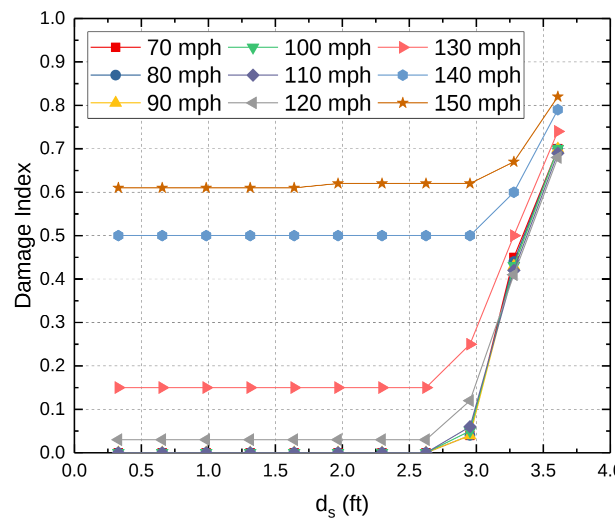


Figure 4-4 Roof damage index for non-elevated model 1

4.2.2 Elevated model

Vulnerability curves for the building model 1 with elevated foundations are presented in Figure 4-5 to Figure 4-8 for the seaward wall, non-seaward walls, all walls, and the roof. In the elevated building model, the slamming wave loads are applied only on the seaward wall when the wave crest is above the floor joists, that is when the stillwater depth increases over 6.56 ft. As discussed earlier, the elevation system modeled in the present study is very robust with large stiffness and perfectly fixed bottom connection to the ground. The effect of wave loads acting on the foundation piles on the overall structural behavior as well as sheathing damages could be neglected. Therefore, the damage index is only affected by the wind speed and is invariant to the stillwater depth when the wave crest is below the floor joists as shown in Figure 4-5. A transition from wind dominant sheathing failure to wave dominant sheathing failure could also be observed on the seaward wall of the elevated model 1 building when the seaward wall is subjected to wave slamming force. As shown in Figure 4-5, the damage index first decreases due to the compensation from wave pressure in the opposite direction and then increases rapidly as wave slamming force becomes the dominant factor in sheathing damages. Furthermore, the effect of wind speed on the seaward wall sheathing damage is limited and higher wind speed will result in a lower sheathing damage rate after wave slamming force becomes the dominant factor.

The sheathing damage rate for the other three walls subjected to wind and flood load is shown in Figure 4-6. The results indicate that the wave loads acting on foundation piles could not affect the sheathing damage of these three walls. Although the wave loads not directly act on these three walls, the wave slamming force on the seaward wall could be transmitted to other structural components through the framing systems and increase the sheathing damage rate of non-seaward walls. As shown in Figure 4-6, the damage index for the non-seaward walls is not invariant anymore and grows rapidly after the wave crest becomes higher than the floor joists.

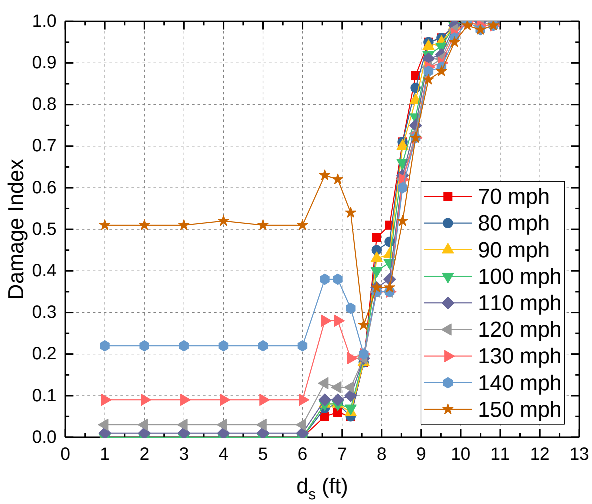


Figure 4-5 Seaward wall damage index for elevated model 1

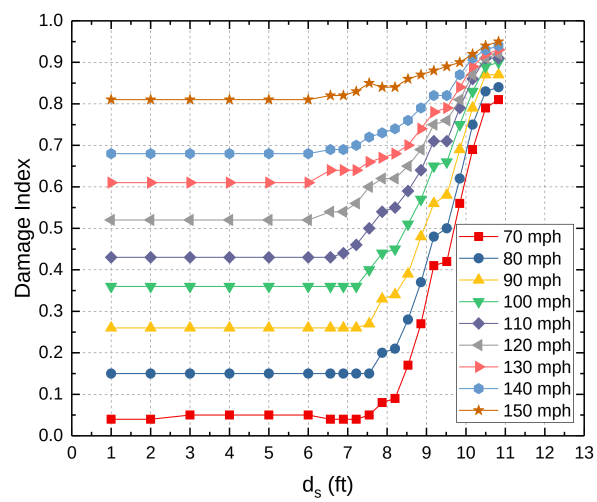


Figure 4-6 Non-seaward walls damage index for elevated model 1

The results of the damage index for all walls are similar to those for the non-seaward walls as shown in Figure 4-7. Taking all walls into account, a higher wind speed could lead to a higher sheathing damage rate, and a larger stillwater depth could also increase the sheathing damage rate after the wave crest is above the floor joists.

The results in Figure 4-8 indicate that the damage index of the roof mainly depends on the wind speed at first since only the wind loads directly act on it. After the stillwater depth increased to be over 9.51 ft, the damage index has a sudden increase suggesting that the wave slamming force on the seaward wall could have a prominent effect on the behavior of the entire structural system when the stillwater depth is large enough.

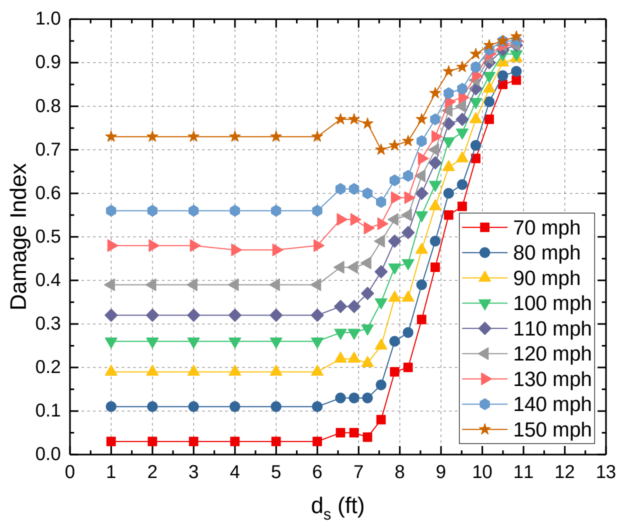


Figure 4-7 All walls damage index for elevated model 1

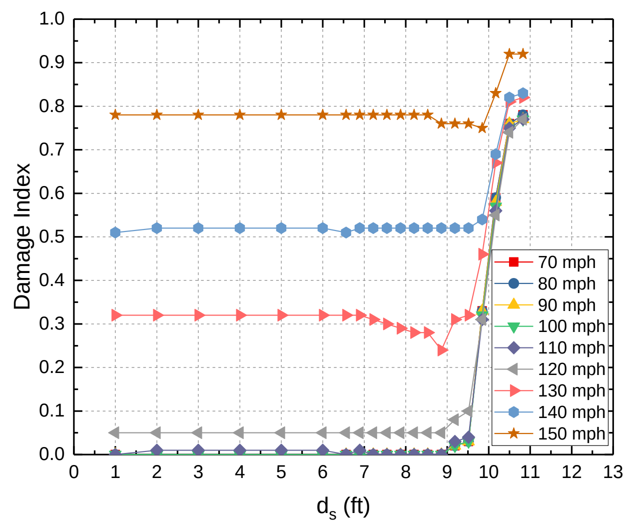


Figure 4-8 Roof damage index for elevated model 1

4.2.3 Comparison between non-elevated and elevated model

For the seaward wall, the model with elevated foundations has a higher sheathing damage index under small stillwater depth. Considering the natural wind profile above the ground, wind speed increases as the height increases. Therefore, a higher pressure on wall sheathings for the elevated building is expected. When the stillwater depth exceeds 2 ft, the seaward wall damage index for the elevated building is lower than that for the non-elevated building, especially under large stillwater depth. With a relatively large stillwater depth, wave loads become dominant affecting the sheathing failure for the non-elevated building. However, the wave loads might only act on the foundation piles for the elevated building, which has a negligible effect on the sheathing failure of the seaward wall as discussed earlier.

For the non-seaward walls, no wave loads are acting on them. The elevated model always has a higher sheathing damage rate due to larger wind pressure at a high elevation until the stillwater is high enough (above 3 ft) to affect the entire structural system of the non-elevated model. A similar conclusion could also be reached for the damage index of all walls and the roof. The elevated model outperforms the non-elevated model after the stillwater depth exceeds 2.5 ft regarding all walls' failure rate and 3.3 ft regarding the roof failure rate.

4.3 GIS-based multi-hazard resilience maps

In this section, a series of multi-hazard resilience maps for the coastal community are generated to show expected damages on residential buildings based on vulnerability analysis in the last chapters. To evaluate the expected damages in the entire community, the damage indices of walls and roof of representative residential buildings are obtained by interpolation based on the generated vulnerability curves described in Section 4.2. Typical what-if multi-hazard scenarios, including (a) low wind speed combined with high flooding water level (500-year flood+70 mph wind), (b) minimum multi-hazard (10-year flood+70 mph wind), (c) extreme multi-hazard (500-year flood+150 mph wind), (d) high wind speed combined with low flooding water level (10-year flood+150 mph wind) and (e) design wind speed combined with base flood (100-year flood+120 mph wind), are discussed to assess the vulnerability of the entire community.

In this research, a local community of 2021 residential buildings (1973 non-elevated and 48 elevated) is evaluated. Each building is referred to the three building models as discussed earlier with respect to their year of built and the number of stories. Considering the statistical analysis of building data discussed in Section 3.1.1, damage index of Model 1, 2 or 3 is assigned to each building based on its number of stories. For the buildings whose number of stories are not provided in the residential homes inventory, the damage ratio of the specific representative building model will be assigned to them according to their year of built. For all subsequent resilience maps, the damage index of each building will fall into the following vulnerability categories with particular color: 0-0.2 (blue), 0.2-0.4 (light blue), 0.4-0.6 (green), 0.6-0.8 (orange) and 0.8-1 (red). In order to distinguish baseline configuration and elevated configuration, the circle symbol with black outline represents the non-elevated residential building, while the square symbol with a grey outline represents the elevated residential building. It is noteworthy that the damage index is evaluated separately for the roof and walls due to the difference of the material as well as the monetary cost of these two parts.

4.3.1 Resilience maps for walls damages

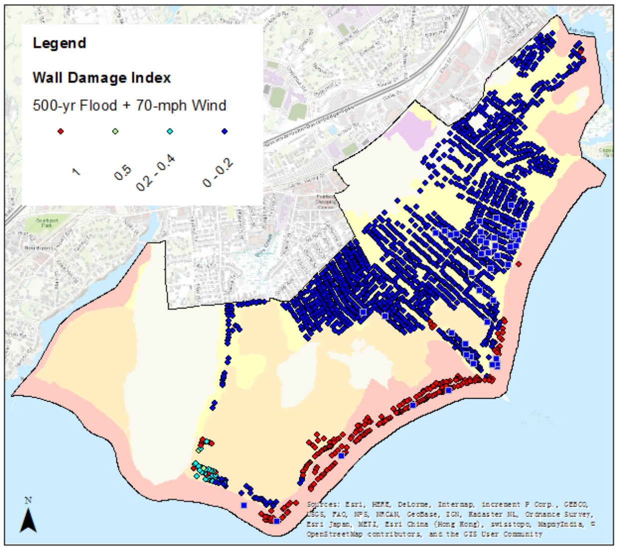
Figure 4-9 provides a map of all the assessed residential buildings to reflect damage levels under different what-if multi-hazard scenarios. Meanwhile, the walls damage indexes for the non-elevated residential buildings under different what-if multi-hazard scenarios are summarized in Table 4-2 to quantitatively demonstrate multi-hazard effects. Figure 4-9(b) illustrates the expected damages under the minimum multi-hazard in Fairfield coastal community, where almost all the residential buildings have the lowest levels of damage. Higher levels of damage could be observed in the VE Zone along the shoreline of Long Island Sound as the flood return period and the corresponding flood level increase, which could be reflected by a large number of red dots shown in Figure 4-9(a). Similar conclusions can also be drawn that 35.2% and 18% of non-elevated buildings have a damage index above 0.6 for higher flood level and lower flood level, respectively, under the same wind speed as shown in Figure 4-9(c) and Figure 4-9(d). In addition, it is noteworthy that elevated residential buildings in the VE Zone have minimum damage compared to the non-elevated buildings which have significantly large damage ratio as shown in Figure 4-9(a).

Residential building structures in the MiWA area under the extreme multi-hazard as shown in Figure 4-9(c) are more vulnerable to the multi-hazard damage. The mean damage index for 86.4% of non-elevated residential buildings is 0.05 under the low wind speed condition as shown in Figure 4-9(a). However, the mean damage index of these non-elevated residential buildings increases to 0.38 when a high wind event occurs as shown in Figure 4-9(c). It should be noted that elevated residential buildings, compared with the non-elevated residential buildings, have larger damage ratios when the wind speed is 150 mph due to their high elevations. However, elevated residential buildings in the VE Zone still have a smaller damage ratio compared with non-elevated buildings in the same area under the extreme multi-hazard event. This phenomenon indicates that building elevation is an effective measure to protect residential buildings located in the VE Zone which are frequently subjected to more wave actions. In contrast, in the MiWA zone, elevated residential buildings might have larger damage ratios than non-elevated residential buildings, especially under extreme wind event (i.e. 150 mph).

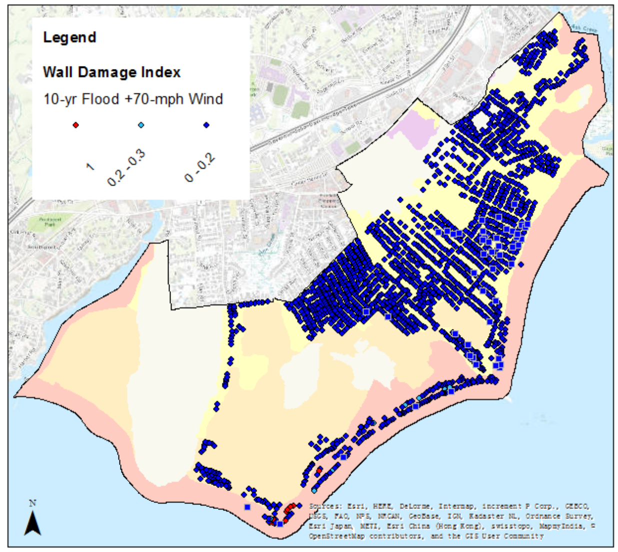
Table 0-2 Summary of wall damage index for non-elevated buildings under different multi-hazard scenarios

Number (percentage) of non-elevated buildings with different walls damage conditions							
Multi-hazard Scenario	Flood return period	Wind Speed (mph)	Damage index of walls				
			0-0.2	0.2-0.4	0.4-0.6	0.6-0.8	0.8-1
a	500-year	70	1704 (86.4%)	28 (1.4%)	3 (0.2%)	0	238 (12.1%)
b	10-year	70	1954 (99%)	9 (0.5%)	0	0	10 (0.5%)
c	500-year	150	0	924 (46.8%)	356 (18%)	455 (23.1%)	238 (12.1%)
d	10-year	150	0	1109 (56.2%)	508 (25.7%)	346 (17.5%)	10 (0.5%)
e	100-year	120	1308 (66.3%)	466 (23.6%)	5 (0.3%)	0	194 (9.8%)

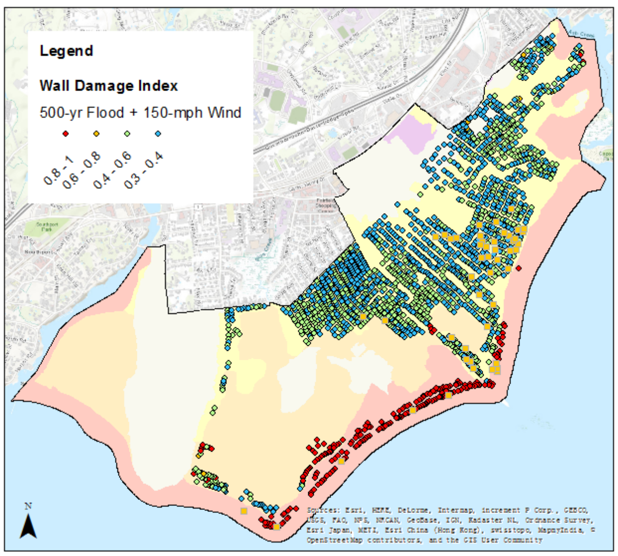
Figure 4-9(e) shows the typical multi-hazard combination of design wind and base flood. As shown in Table 0-2 and Figure 4-9(e), 66.3% of non-elevated buildings are slightly damaged in this scenario, and 23.6% of non-elevated buildings have a damage index of 0.26. The majorities of non-elevated residential buildings assessed in the VE Zone are completely destroyed, while elevated residential buildings have very good performance in the same location.



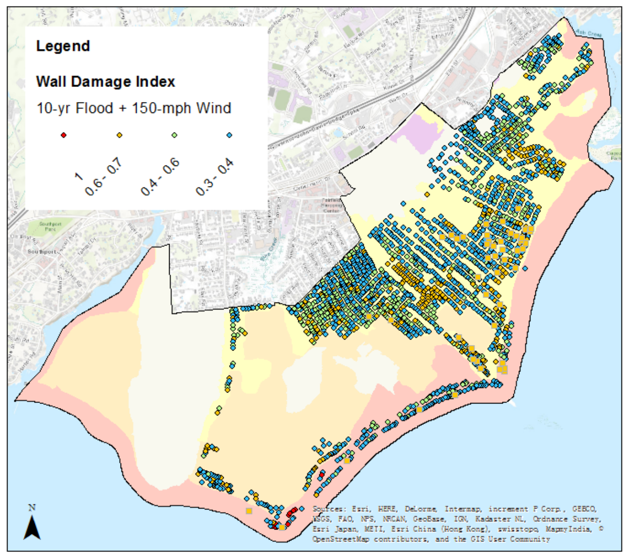
(a) 500-year flood+70 mph wind



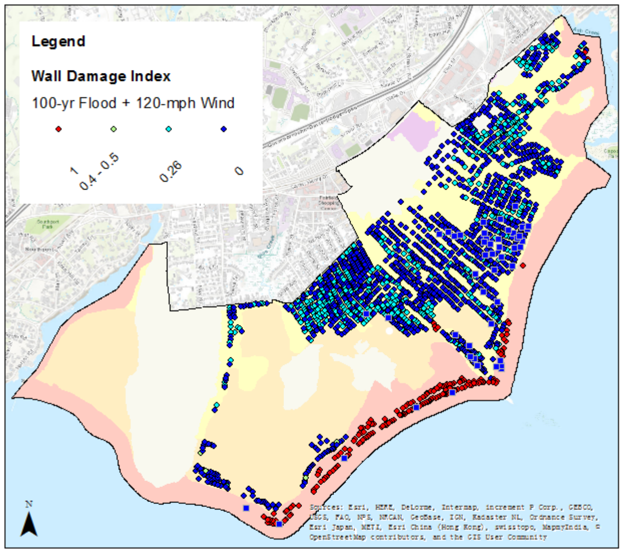
(b) 10-year flood+70 mph wind



(c) 500-year flood+150 mph wind



(d) 10-year flood+150 mph wind



(e) 100-year flood+120 mph wind

Figure 4-9 Assessed damages on building walls in Fairfield coastal community based on damage index for different multi-hazard scenarios

4.3.2 Resilience maps for roof damages

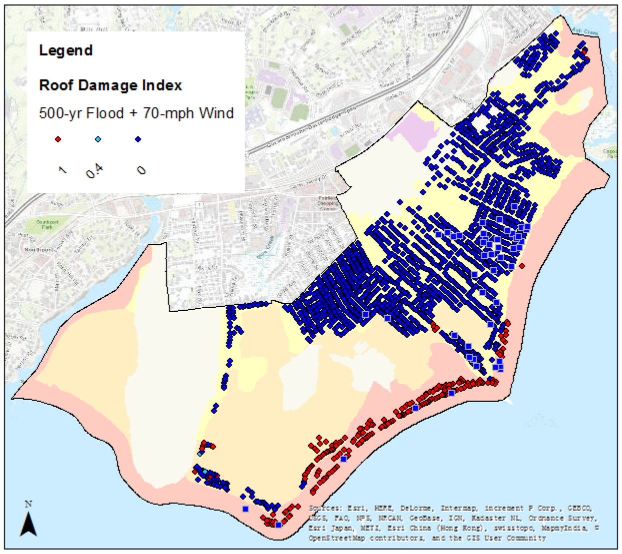
Figure 4-10 shows the expected roof damages under various combinations of wind and flood hazard. Meanwhile, the roof damage indexes for the non-elevated residential buildings under different what-if multi-hazard scenarios are summarized in Table 4-3. Almost all of the residential buildings have good performance under the minimum multi-hazard scenario. However, for the 500-year flood event as shown in Figure 4-10(a), 12.1% of the roof of non-elevated residential buildings are severely damaged or completely destroyed, especially for the buildings located at the beach front. In contrast, the elevated buildings seem not to be affected by this severe flood risk at the same location. However, it is noteworthy that neither the soil-structure interactions nor the erosion on the piles with associated failure modes are considered in this research. These simplifications could probably overestimate the capability of the elevated residential buildings in resisting the flood loads and hence overpredict their performance in a severe flood event.

Compared with the building roof damages under the multi-hazard scenario with a low wind speed shown in Figure 4-10(b), a significant increase of roof damages could be observed when the community is subjected to high wind speed as shown in Figure 4-10(d). Under this high wind speed (150 mph), roofs of the elevated buildings are completely destroyed since they are located at a higher elevation and hence are more sensitive to wind loads. Note that even non-elevated residential buildings in VE Zone have better structure performance in terms of roof damages than the elevated buildings under the multi-hazard scenario of lower flooding water level combined with higher wind speed.

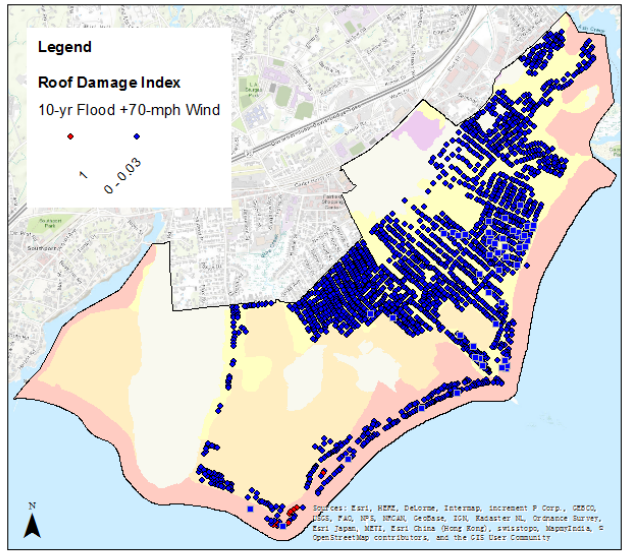
Table 0-3 Summary of roof damage index for non-elevated buildings under different multi-hazard scenarios

Number (percentage) of non-elevated buildings with different roof damage conditions							
Multi-hazard Scenario	Flood return period	Wind Speed (mph)	Damage index of roof				
			0-0.2	0.2-0.4	0.4-0.6	0.6-0.8	0.8-1
a	500-year	70	1732 (87.8%)	0	2 (0.1%)	0	239 (12.1%)
b	10-year	70	1963 (99.5%)	0	0	0	10 (0.5%)
c	500-year	150	0	0	951 (48.2%)	469 (23.8%)	553 (28%)
d	10-year	150	0	0	1109 (56.2%)	481 (24.4%)	383 (19.4%)
e	100-year	120	1442 (73.1%)	0	0	0	531 (26.9%)

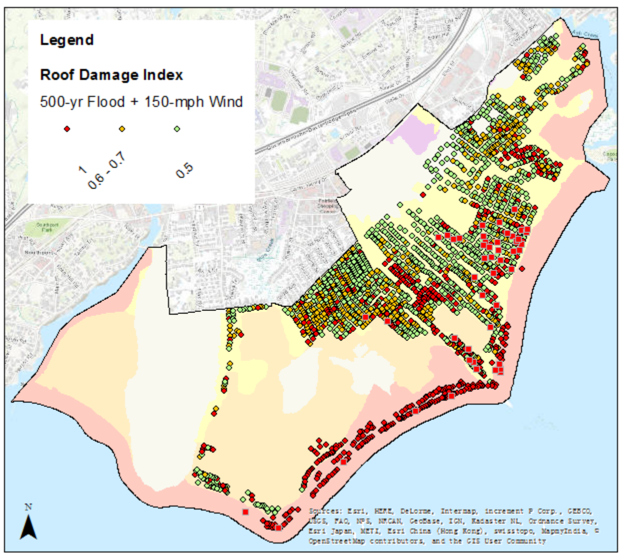
The typical multi-hazard scenario of design wind and the base flood is introduced in Figure 4-10(e). According to Table 0-3, 73.1% of non-elevated residential buildings are safe in this scenario, while 26.9% of non-elevated buildings have roof damage indexes above 0.8. This large damage ratio is mainly due to the roof failures of model 2, which is used to represent 393 housing units in the coastal community, under the wind speed of 120 mph at which the roofs are vulnerable to the wind loads and get severely damaged. Several reasons could contribute to this observed phenomenon. Firstly, the HOWE truss roofing system in model 2 is designed to efficiently carry weight rather than uplift pressure. However, the uplift force is proved to be the most damaging factor for all three building models under strong winds. In addition, the HOWE truss roofing system has a smaller size of lumbers and hence make the sheathing failure happen under low wind speeds. Furthermore, it can also be observed from Figure 4-10(e) that elevated residential buildings have a reduced capacity to survive at a high wind of 120 mph in the MiWA area in the Fairfield coastal community.



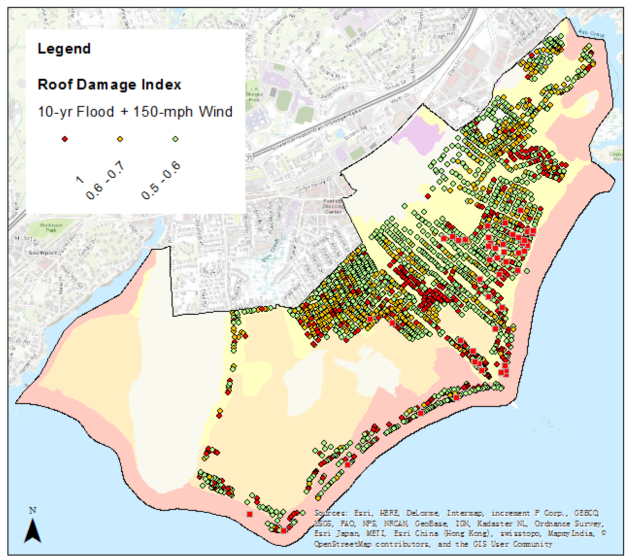
(a) 500-year flood+70 mph wind



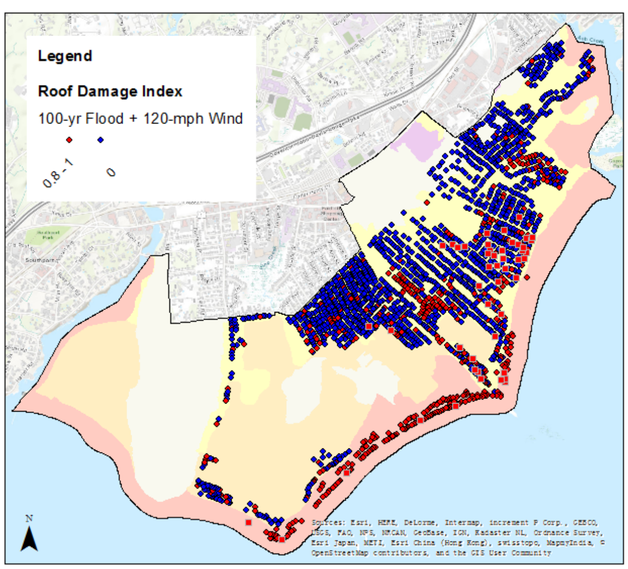
(b) 10-year flood+70 mph wind



(c) 500-year flood+150 mph wind



(d) 10-year flood+150 mph wind



(e) 100-year flood+120 mph wind

Figure 4-10 Assessed damages on building roof in Fairfield coastal community based on damage index for different multi-hazard scenarios

CHAPTER 5 Social Part

5.1 Literature Review

A literature review was conducted to understand the state of the science of social science research on homeowner SRF elevation decisions, the influence of building code on SRF hazard mitigation, and risk perceptions and individual decision making. An extensive literature review was conducted using the search terms "elevation", "superelevation", "retrofit", "residential", "house" or "housing", "property", "flooding", and "building code" using Google, Google Scholar and Scopus search engines. The literature review found very little research has been conducted on homeowners SRF elevation decisions or on the influence of building codes on SRF hazard mitigation while more substantive work has been done on risk perceptions for hazard mitigation. We review results of the literature review next.

5.1.1 Review of Research on Homeowner SRF Elevation Decision Making

There are two actions homeowners can take to reduce the financial consequences of low probability adverse events: investing in loss reduction measures such as home elevation (or cheaper options like purchasing and installing storm shutters) and purchasing insurance. Most individuals are reluctant to invest in protective measures, even if they recognize the likelihood of a disaster. The two reasons why people do not invest are: 1) people ignore the chance of future damage even when they are provided information about risk and 2) people think the benefits of mitigation given they accrue over the lifetime of the house do not justify the large upfront costs (Kunreuther et al. 2013; Kunreuther et al. 2011). A 1974 survey of more than 1,000 California homeowners in earthquake-prone areas revealed that only 12 percent of the respondents had adopted any protective measures (Kunreuther et al. 1978). Fifteen years later, there was little change, despite the increased public awareness of the earthquake hazard. In a 1989 survey of 3,500 homeowners in four California counties at risk from earthquakes, only 5 to 9 percent of the respondents in these areas reported adopting any loss reduction measures (Palm et al. 1990). Other studies have found a similar reluctance by residents in flood-prone areas to invest in mitigation measures (Burby 1991; Laska 1991).

Work et al. (1999) surveyed 30 homeowners in 1995 who elevated their SRF after Hurricane Emily struck the Outer Banks of North Carolina in 1993. The interviews were designed to determine the methods, motivations, and benefits of floodproofing retrofits. Interviews revealed that elevated homes were originally built from 1919 to 1985. Work et al. (1999) found that most homeowners determined the height to elevate their home based on 1) having sufficient space for parking beneath the house or 2) contractor or advice from others. Others chose the elevation height based on previous flood elevation, landscape or aesthetic concerns, and intuition. No homeowners indicated an awareness of FIRM documents or base flood elevations for the area. All but one homeowner had experienced flooding prior to elevating their house including from Hurricane Emily and from earlier storms (e.g., Hurricane Gloria in 1985, and the storm of the century in 1993) and all but one homeowner named "floodwaters" as their motivation for retrofitting the house. Half of the interviewees had filed at least two insurance claims after prior flood events prior to elevating their home. The majority of interviewees reported the primary benefit from SRF elevation was "peace of mind" associated with reduced flooding risk (Work et al. 1999, pg. 92). Only two interviewees mentioned being motivated by reduced insurance premiums.

Concluding Remarks

This research improves significantly the understanding of the resiliency of coastal community under wind and flood hazards and the potential trade-offs between hazard mitigation designs. An SFR multi-hazard assessment methodology is developed in this research and is tested through applying the assessment in two Connecticut coastal communities: Fairfield and Milford. Different multi-hazard scenarios are run and coastal community multi-hazard resiliency maps are generated. Iteration of mapping products increase their usefulness for our two partner coastal communities. The generated resilience maps are used to inform discussions about potential changes in planning strategies, building ordinances (including potential development of model building code language that encourages the use of effective mitigation methods to strengthen weak connections with minimum costs), public or building education efforts, or other hazard mitigation efforts. This research will lay the foundation for enhancing the resilience of Connecticut coastal communities against multiple hazards, such as wind and flooding, through appropriate multi-hazard assessment, build awareness of risks of damage to SFR from multi-hazards, and aid our two partner Connecticut coastal communities in designing strategies for innovative hazard mitigation and retrofit efforts. Beneficiaries of this research include coastal municipalities and residents in having not only more resilient homes and communities, but the knowledge of what to think about and request from building contractors in retrofitting/building coastal homes. The building industry will also have the information necessary to address multi-hazards in the design and construction process. Finally, other coastal communities can adapt the methodologies and educational materials proposed in this research to advance their multi-hazard resilience.

REFERENCES

ACI Committee 318. (2005). *Building Code Requirements for Structural Concrete and Commentary*. ACI Committee.

Aerts, J. C., Botzen, W. W., Emanuel, K., Lin, N., de Moel, H., and Michel-Kerjan, E. O. (2014). "Evaluating Flood Resilience Strategies for Coastal Megacities." *Science (New York ...*, 344(6183), 473-475.

American Institute of Steel Construction. (2005). *Steel Construction Manual*. *Steel Construction Manual*, American Institute of Steel Construction, Chigago, IL.

AP. (2015). "The flip side of years of no hurricanes: good luck runs out." *Available at: http://www.nytimes.com/aponline/2015/05/31/science/ap-us-sci-hurricane-luck.html*.

APA. (1997). *PDS- Plywood Design Specification*. *Methods*, Tacoma, WA.

BG. (2005). "The Great Hurricane of 1938." *the Boston Globe, July 19, 2005. http://www.boston.com/news/globe/magazine/galleries/2005/0724/hurricane1938/*.

Burby, R. "Hurricane Katrina and the Paradoxes of Government Disaster Policy: Bringing about Wise Governmental Decisions for Hazardous Areas," Annals of the American Academy of Political and Social Science 604, 2006, pp.171-91; S.B. Laska, Floodproof Retrofitting: Homeowner Self-Protective Behavior, Boulder, CO: Institute of Behavioral Science, University of Colorado, 1991.

Coulbourne, W. (2013). "Foundation Design in Coastal Flood Zones." *Advances in Hurrican Engineering-Learning from Our Past*, American Society of Civil Engineers, 92–107.

Ellingwood, B. R., Smilowitz, R., Dusenberry, D. O., Duthinh, D., Lew, H. S., and Carino, N. J. (2007). "Best practices for reducing the potential for progressive collapse in buildings." *U.S. National Institute of Standards and Technology (NIST).*, 216.

Elsner, J. B., Kossin, J. P., and Jagger, T. H. (2008). "The increasing intensity of the strongest tropical cyclones." *Nature*, 455(7209), 92–95.

Emanuel, K. A. (2013). "Downscaling CMIP5 climate models shows increased tropical cyclone activity over the 21st century." *Proceedings of the National Academy of Sciences*, 110(30), 12219–12224.

FEMA. (2011). "Coastal Construction Manual." *Fema P-55*, II(August), 400.

FEMA. (2014). "FEMA MOTF Hurricane Sandy Impact Analysis." *http://www.arcgis.com/home/item.html?id=307dd522499d4a44a33d7296a5da5ea0*.

Fronstin, P., and Holtmann, A. G. (1994). "The determinants of residential property damage caused by Hurricane Andrew." *Southern Economic Journal*, 61(2), 387–397.

Guha-Sapir, D., Hoyois, P., and Below, R. (2014). "Annual Disaster Statistical Review 2013: The Numbers and Trends." *Brussels: Centre for Research on the Epidemiology of Disasters (CRED)*.

Hartford Business. (2015). “CT homes’ hurricane damage exposure tops $33B.” *Retrieved from http://www.hartfordbusiness.com/article/20150604/NEWS01/150609957*.

Heaney, J. P., Peterka, J., and Twight, L. T. (2000). “Research needs for engineering aspects of natural disasters.” *Journal of Infrastructure Systems*, 6(1), 4–14.

Herzog, B., and Yeh, B. (1999). “Nail Withdrawal and Pull-Through Strength of Structural-Use Panels Nail Withdrawal Strength of Plywood and OSB.” *APA – The Engineered Wood Association*.

IBC. (2011). “2012 International Building Code.” Washington DC.

IPPC. (2007). “Climate Change 2007: The Physical Science Basis.” Agenda 6, 07 (2007): 333.

Kretschmann, D. E. (2010). *Chapter 5 - Mechanical Properties of Wood*. *Wood Handbook - Wood as an engineering material*, Madison, WI.

Kunreuther, H., R. Ginsberg, L. Miller, P. Sagi, P. Slovic, B. Borkan, and N. Katz, Disaster Insurance Protection: Public Policy Lessons, New York: John Wiley and Sons, 1978.

Kunreuther, H., R. J. Meyer, and E. Michel-Kerjan, "”Overcoming decision biases to reduce losses from natural catasrophes", in Behavioral Foundations of Policy, E. Shafir, ed., Princeton, NJ: Princeton University Press, 2013.

Kunreuther, H., E. Michel-Kerjan, and M. Pauly. 2013. Making America More resilient toward natural disasters: A call for action. Environment 55(4): 15-23

Laska, S.B. “Floodproof Retrofitting: Homeowner Self-Protective Behavior” (Boulder: Institute of Behavioral Science, University of Colorado, 1991)

Li, Q., Wang, C., and Zhang, H. (2016). “A probabilistic framework for hurricane damage assessment considering non-stationarity and correlation in hurricane actions.” *Structural Safety*, 59, 108–117.

McCormac, J. (1986). *Design of Reinforced Concrete*. Harper & Row, New York.

McCullough, M., and Kareem, A. (2009). “Anatomy of Damage to Coastal Construction: A Multi- Hazard Perspective.” *Structures Congress 2009*, April 2009, 1-10.

NOAA. (2012). “Hurricane Sandy.” *http://www.weather.gov/okx/HurricaneSandy*.

R. Palm, M. Hodgson, R. D. Blanchard, and D. Lyons, Earthquake Insurance in California: Environmental Policy and Individual Decision Making, Boulder, CO: Westview Press, 1990.

Pan, F., Cai, C. S., Zhang, W., and Kong, B. (2014). “Refined finite element modelling and damage predictions of low-rise building envelopes under hurricane loads.” *Wind and Structures, an International Journal*, 18(6), 669-691.

Phan, L. T., Simiu, E., McInerney, M. A., Taylor, A. A., Glahn, B., and Powell, M. D. (2007). “Methodology for development of design criteria for joint hurricane wind speed and storm surge events: proof of concept.”

Rashed, T. (2006). "Geospatial Technologies, Vulnerability Assessment, and Sustainable Hazards Mitigation in Cites." *In GIS for Sustainable Development: Bringing Geographic Information Science into Practice towards Sustainability, Editor Campagna, M.*

Skakun, S., Kussul, N., Shelestov, A., and Kussul, O. (2014). "Flood Hazard and Flood Risk Assessment Using a Time Series of Satellite Images : A Case Study in Namibia." 34(8).

Sparks, P. R., Schiff, S. D., and Reinhold, T. A. (1994). "Wind damage to envelopes of houses and consequent insurance losses." *Journal of Wind Engineering and Industrial Aerodynamics*, 53(1–2), 145–155.

Stewart, M. G., Rosowsky, D. V, and Huang, Z. (2003). "Hurricane risks and economic viability of strengthened construction." *Natural Hazards Review*, 4(1), 12–19.

Tomiczek, T., Kennedy, A., and Rogers, S. (2014). "Collapse Limit State Fragilities of Wood-Framed Residences from Storm Surge and Waves during Hurricane Ike." *Journal of Waterway, Port, Coastal, and Ocean Engineering*, 140(1), 43–55.

Weston, J., and Zhang, W. (2016). "Finite Element Modeling of Nailed Connections in Low-rise Residential Home Structures." Structures Congress, Denver, CO.

Work, PA, SM Rogers Jr., and R. Osborne, Flood retrofit of coastal residential structures: Outer Banks, North Carolina. Journal of Water Resources Planning and Management 125(2): 88-93, 1999.