Wind Performance Assessment of Post-Disaster Housing in the Philippines

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Abstract

Although organizations build housing after typhoons and other disasters in resource-limited contexts that is intended to be safer than what existed previously, the performance of these houses in future typhoons—and the factors influencing performance—are unknown. This study develops a component-level, performance-based wind engineering assessment framework and evaluates the wind performance of twelve semi-engineered post-disaster housing designs, representing thousands of houses that were constructed in the Philippines after Typhoon Yolanda. We found that roof panel loss likely occurs first for most designs, at wind speeds equivalent to a category 2 hurricane/signal 3 typhoon. Roof shape determines whether this loss is caused by failure at the panel-fastener interface or purlin-to-truss connection. However, houses with wooden frames and woven bamboo walls may also experience racking failures at wind speeds equivalent to signal 2 or 3 typhoons, a situation exacerbated by strengthening the roof. Results also show that wind performance varied with roof shape, component spacing, panel thickness, eave length and connection between purlin and truss. Organizations can use these results to improve housing performance, taking specific care to increase wall capacity and ensure a continuous load path. This framework can be expanded to assess housing performance in other resource-limited contexts.

Keywords
Post-disaster housing, Housing performance, Wind assessment, Typhoons, Typhoon Yolanda

Introduction

Tropical cyclones (known as hurricanes in the Atlantic, and referred to here as typhoons) cause billions of dollars of damage and destroy thousands of houses each year, particularly in resource-limited communities (CRED and UNISDR 2018; Rentschler 2013). In these communities, most households do not have sufficient capital to reconstruct their house without assistance, so organizations provide housing reconstruction assistance, with the goal that the new houses can better withstand future hazards (Clinton 2006; Twigg 2017). If a house is provided, organizations will often use a single standardized design that is built many times in one or more communities (Da Silva and Batchelor 2010). At best, these designs are semi-engineered by architects and engineers, but in some cases, little to no engineering calculations are used, and the expected performance of these houses in future disasters is unknown (Harriss et al. 2020). Given their limited resources, organizations must make the decision whether to provide more households with a cheaper house or fewer households with a more expensive house (Schilderman 2010). Moreover, households who do not receive assistance in rebuilding their homes are likely to observe the houses that organizations built and attempt to implement similar designs (Turnbull et al. 2015). Organizations must therefore find low-cost, locally sourced, culturally acceptable, and hazard-resilient solutions for the houses they build and the houses that will imitate their designs (Kijewski-Correa et al. 2012). Because there is not a current framework for assessing post-disaster housing and little is known about either the performance of these post-disaster houses or the effect of design decisions on performance, there is a need for both an assessment framework and an evaluation of how constructed houses are expected to perform.

In this study, we develop a component-level performance-based wind engineering assessment framework to: 1) assess houses constructed in resource-limited communities by organizations after disasters, and 2) evaluate how various design changes can improve performance. Performance-based wind engineering assessments, which exist for differently-designed houses in other contexts but not post-disaster housing in resource-limited contexts, allow us to define a specific performance objective for a house in a future typhoon, and probabilistically assess the likelihood of meeting this objective, as well as the consequences of failing to do so (Barbato et al. 2013; Ellingwood 2015). An improved understanding of the typhoon performance of post-disaster housing constructed by nongovernmental organizations (NGOs) and governments in resource-limited communities is specifically needed to identify the vulnerabilities in common designs and to provide organizations with the information needed to improve the performance of these houses. This study responds to this need, with application to twelve semi-engineered post-disaster housing designs, representing thousands of houses that were constructed in the Philippines after Typhoon Yolanda.

Previous Assessments of Housing Wind Performance

Below we summarize prior observations of post-disaster housing safety, assessments of North American and Australian housing, and reconnaissance of wind damage to houses in resource-limited contexts.

Post-Disaster Housing

Commonly, organization-assisted post-disaster housing follows a ‘core’ shelter design, meaning the house is rectangular in shape, has few or no interior partitions, and has no more than three or four roof trusses. Typical materials used in these houses include wood or reinforced concrete (RC) frames; masonry, plywood, or woven walls; and corrugated galvanized iron (CGI) roof panels.

Despite the emphasis on building safer houses, there is little evidence to suggest whether post-disaster housing is safer than pre-disaster designs. For example, Lyons (2009) assessed NGOs’ and government agencies’ post-tsunami housing reconstruction in Sri Lanka and found that housing vulnerabilities were recreated in the reconstruction due to use of poor-quality materials and a lack of construction oversight. What we do know about the safety of post-disaster housing tends to come either from implementing organizations’ reports or case studies conducted shortly after housing reconstruction projects ended (Harriss et al. 2020; Schilderman 2014). These reports are limited in that they generally provide information only about design features used to enhance safety, but not the long-term performance of the houses (Peacock et al. 2007; Schilderman 2014). However, a few studies have examined the design features in post-disaster housing using visual audits. In the Philippines, after 2013’s Typhoon Yolanda, Opdyke et al. (2019) assessed housing safety in 19 post-disaster projects by observing whether a checklist of design improvements (*e.g.*, bracing and tie-downs) suggested by the Philippines Shelter Cluster (2014) were present in housing designs, finding that 11 of the designs had incorporated at least 5 of the '8 Key Messages'. Likewise, Stephenson et al. (2018) examined whether houses that were constructed by households using cash and materials from NGOs in three communities after Yolanda had three features that were expected to influence performance in typhoons (hip roofs, roof vents, and eaves no longer than 0.5 m). They found that a majority of houses had eaves longer than 0.5m and a gable roof, both of which can increase vulnerability to wind damage.

While these quick visual audits provide useful information about the potential vulnerabilities of post-disaster housing, they do not quantify the wind speed expected to cause damage or identify the specific failure mode. Nor do they show the value or detriment of certain design decisions in terms of performance. Detailed structural assessments can provide this information, but we could find no published studies that analytically assessed the wind performance of post-disaster housing. We found two studies involving experimental testing of post-disaster housing that included a series of tests on permanent houses designed by the Tongan Ministry of Works transitional shelters designed by the United States Agency of International Development (USAID). The tests of the Tongan house resulted in new design recommendations, including new truss tie downs (Boughton and Reardon 1984), and the test of the USAID shelter found that this shelter could not withstand wind speeds greater than 177 kph (110 mph), equivalent to a category 2 hurricane in the U.S. (Liu-Marques et al. 2012). There is, thus, an urgent need for structural assessments of post-disaster housing using component-based analyses, with demands and capacities quantified by prescriptive codes and values published in the literature and uncertainty propagated using a Monte Carlo approach.

Housing in North America and Australia

A significantly greater number of studies have examined the wind performance of timber housing in North America and Australia (*e.g.*, Ellingwood et al. 2004; Gavanski and Kopp 2017; Henderson et al. 2013a; b; Morrison et al. 2012; Unnikrishnan and Barbato 2017). However, the houses examined in these studies differ from the housing built in resource-limited communities in that they are larger, with more complex floor plans, different types of timber, superior framing and connections, and more redundant roof systems. In addition, North American houses typically have oriented strand board or plywood roof sheathing;

although, Australian houses commonly use metal roof cladding that is similar to the CGI often used in post-disaster housing.

Nevertheless, these previous studies highlight methodologies that can be used to assess the performance of post-disaster housing. For instance, Ellingwood et al. (2004) and Lee and Rosowsky (2005) proposed methods for assessing wind damage fragilities for different roof components. These fragilities define the likelihood of damage to these components as a function of wind speed. In addition, Li and Ellingwood (2006) illustrated how to incorporate uncertainty into a performance-based wind assessment. Studies of houses in Australia have further demonstrated component-based performance-based wind engineering approaches to assessing housing vulnerability (Henderson and Ginger 2007; Stewart et al. 2018). These assess the vulnerability of individual components (*e.g.*, walls or roofs) in a given design, and relate the structure's overall vulnerability to the vulnerabilities of its constitutive components (Goyal et al. 2012). For example, Henderson and Ginger (2007) proposed a series of failure mechanisms for components found in a typical Australian house with metal roof cladding and related these component failure mechanisms to global limit states of interest.

Previous Reconnaissance of Typhoon Damage to Houses

There are also studies that have documented typhoon-related housing damage in resource-limited communities. Common types of damage are loss of roof cladding (Prevatt et al. 2010; Shanmugasundaram et al. 2000), global roof system loss due to the failure of the connections between the roof trusses and walls (Mukhopadhyay and Dutta 2012), wall failures (Build Change 2014; Kijewski-Correa et al. 2017), and in extreme cases, overturning due to the lack of adequate foundations (Mukhopadhyay and Dutta 2012). Some of these failures have been documented in NGO-constructed houses (Kijewski-Correa et al. 2017). Each of these failure mechanisms can endanger occupant safety: for example, loss of roof cladding exposes occupants to the elements and loose cladding can become a wind-borne debris hazard; roof system loss and wall racking, which is the lateral collapse of walls and wall framing systems, can lead to collapse of the entire house (van de Lindt and Dao 2009).

Scope

Here, we develop and adapt a component-level performance-based wind engineering assessment framework to evaluate post-disaster housing in resource-limited contexts. We then assess wind performance of twelve core housing designs developed and constructed by NGOs and government agencies after Typhoon Yolanda in the Philippines to examine the probability of 1) roof cladding loss, 2) roof system failure, and 3) wall failure. These failure modes impair occupant safety, drive losses, and result in potential population displacement. To provide recommendations to organizations on how to improve housing resilience, we also use this framework to examine how varying the design of the roof components can improve performance.

Context

We assessed the performance of twelve housing designs (see Fig 1; Table 1) constructed after Supertyphoon Haiyan (locally referred to as Yolanda) in the Philippines, one of the most typhoon-prone countries in the world (Holden and Marshall 2018). Yolanda made landfall in the Philippines on November 8, 2013, killing over 6,000 people (NDRRMC 2014) and damaging or destroying more than 1.1 million homes (Shelter Cluster 2014). At its peak, Yolanda had wind gusts of nearly 380 kph (235 mph) and 1-min sustained winds of 315 kph (196 mph) (Mas et al. 2015). The islands of Leyte and Samar were particularly affected by Yolanda, which first made landfall in Guiuan, Eastern Samar, with a second landfall near Tacloban, Leyte's largest city (Mas et al. 2015). Reconnaissance following Yolanda revealed that most wooden houses were blown away or had severe roof damage, and the most common roof failure mechanism was the CGI tear out around the fastener (Build Change 2014; Chen et al. 2016; Mas et al. 2015).

Since Typhoon Yolanda, this region has experienced a number of typhoons, including: Typhoons Ruby (2014), Tisoy (2019), and Ursula (2019). Most recent was Typhoon Ursula (Dec. 2019), which followed a path similar to Yolanda. Maximum gusts experienced during Ursula in Guiuan were 195 kph (120 mph), about half of those experienced during Yolanda. Additional information about the paths of recent typhoons can be found in the Supplemental Information (SI). The authors conducted reconnaissance in Guiuan and Tacloban approximately one month after Typhoon Ursula to assess the performance of houses constructed after Yolanda. This reconnaissance revealed that the most common type of damage experienced during Ursula was loss of CGI panels. Nevertheless, as Fig 2 shows, some wooden houses in Guiuan—including houses constructed by NGOs after Typhoon Yolanda— collapsed due to wall racking.

The designs considered in this study (Table 1) consist mainly of one-story houses with a single room. A few designs are two-stories, and two are "loftable", meaning they were built as one-story houses with space to add an interior second floor. The frames are either RC or wood, with the exception of the "loftable" houses, which have load-bearing masonry or concrete walls. Framed systems use a variety of wall materials: plywood, masonry, concrete, or amakan, which is a woven bamboo material. Roof shapes are both gable and hip, and roof pitch ranges between 20 and 35 degrees. Nearly all of the studied houses with gable roofs resemble Fig 3: CGI panels supported on wooden purlins that were connected to either 3 or 4 wooden roof trusses (2 at the gable ends and 1 or 2 in the middle of the structure). Hip roofs have 2 or 3 main trusses and 2 or 3 hip trusses on each end. The two "loftable" houses have no trusses, and metal purlins that connect directly to the wall. Additional information and photos of the studied houses can be found in the SI.

Methods

Overview of Performance Assessment

We assessed the likelihood of failure under wind loading, following the framework reported in Fig 4. This assessment evaluates the performance of the twelve housing typologies in 3-sec wind gusts ranging from 90 kph (55 mph, signal 2/tropical storm) to 405 kph (250 mph, signal 5/category 5). Wind pressures are estimated for each velocity on potentially critical roof and wall components using ASCE 7-16 procedures for low-rise buildings (ASCE/SEI 2016). Failure was determined by checking component capacities against demands at a specified wind velocity of interest (step 5 in Fig 4):

$$\boldsymbol{R} < (\boldsymbol{W_U} - \boldsymbol{D}), \qquad (1)$$

where R = capacity of the given component, W_U = uplift wind force on component, and D = force from dead load acting on the component. We identified wall failure by checking whether the capacity of the wall or wall framing system was less than the lateral wind demand, W_L, at a specified wind velocity: $R < W_L$.

To account for uncertainty in both the wind loads and the component capacities, we used a Monte Carlo simulation to propagate uncertainty through the assessment.

In this study, we draw from van de Lindt and Dao's (2009) limit states to define three performance levels of interest for housing (in order of increasing severity): continued occupancy, life safety, and structural integrity. We relate these performance levels to quantitative measures of the selected component failures (see Table 2). Failure at the *continued occupancy* performance level implies that the structure does not provide protection from the elements; this state is compromised after the first roof cladding panel is lost. This occurs either due to failure at the CGI-fastener interface or failure of the connection between the purlins and the truss.

A house does not meet the *life safety* performance level, *i.e.,* it fails to protect occupant safety, if the roof system detaches due to failure of one of the roof-to-wall connections (van de Lindt and Dao 2009). Failure of the roof-to-wall connections not only compromises the primary living space, but potentially undermines the stability of the walls of the house. *Structural integrity* is compromised if the at least one of the walls has insufficient lateral capacity in the absence of the roof diaphragm (van de Lindt and Dao 2009), with specific emphasis herein on the racking of walls in wooden houses. To determine if these performance objectives are achieved, we assess the performance of all four of the components (panel-fastener interface, purlin-to-truss connection, roof-to-wall connection, and walls) that are related to these performance objectives (continued occupancy, life safety, and structural integrity) for each house. By creating a damage fragility for each component, the governing component failure and corresponding performance objectives (Table 2) can be identified. The following section will discuss the component-level failure mechanisms associated with each of these performance levels (third column of Table 2).

Wind Loading on Houses

Wind pressures were estimated using Equation 2 from ASCE 7 (ASCE/SEI 2016) (step 3 in Fig 4):

$$\boldsymbol{W} = \boldsymbol{q_h}[\boldsymbol{GC_p} - \boldsymbol{GC_{pi}}], \qquad (2)$$

where q_h = velocity pressure at the mean roof height, G = gust factor, C_p = external pressure coefficient, and C_{pi} = internal pressure coefficient. The velocity pressure (N/m^2) is determined by:

$$q_h = 0.613 K_z K_{zt} K_d K_e V^2, \tag{3}$$

where K_z = velocity pressure exposure coefficient, K_{zt} = topographic factor, K_d = directionality factor, K_e = ground elevation factor, and V = 3-s gust wind speed (m/s). K_z is based on the height of the structure and the exposure classification; we assumed all houses have an exposure B classification due to their location in built-up terrain consistent with suburban exposure. Because the housing designs we assessed were built in multiple locations and the specific topography around each house was unknown, we did not account for wind speed-up effects and assumed K_{zt} to be 1.0. K_d was taken to be 0.85 to account for the wind direction not likely aligning with the worst-case angle of attack. As all houses were located at sea level, K_e was taken as 1.0 for all designs.

Pressure Coefficients

Unfortunately, the available wind tunnel testing databases (*e.g.*, TPU 2007) did not have pressure coefficients for houses with eaves, so external pressure coefficients, C_p, (step 1a in Fig 4), were determined using Chapters 28 (Main Wind Force Resisting System – Envelope Procedure) and 30 (Components and Cladding) from ASCE 7-16 (ASCE/SEI 2016). These coefficients have been developed based on wind tunnel tests and expert judgment. For panels, fasteners and purlin-to-truss connections, the component and cladding coefficients were used, whereas the roof-to-wall connections and wall frames use the main wind force resisting system coefficients. We again used ASCE 7-16 (ASCE/SEI 2016) pressure coefficients for the regions of the roof where there was an eave because the recent wind tunnel test data (*e.g.*, Parackal et al. 2016) is for designs with more complex geometries than those included in this study.

The houses included in this study were not watertight (*i.e.,* there tended to be gaps between the top of the wall and the roof, and windows did not fully close) and the wall material for some houses was permeable (woven bamboo). Therefore, we assumed each house was partially enclosed with an internal pressure coefficient, C_{pi}, of 0.55 (step 1a in Fig 4). While this was likely a conservative estimate for the internal pressure of the intact structure, it is likely representative of the internal pressures the building would experience following envelope breach. For both the internal and external pressures, no redistribution of pressures is considered since our analysis focuses on first or governing component failures.

Dead Loads

The dead load (step 1b in Fig 4) on the structures is minimal, as these houses are lightweight. Included in the self-weight of the roof are the CGI panels, the wooden purlins, and the wooden roof trusses. Material weights are defined in Table S1 in the SI.

Component Demands

Based on these loads and the tributary areas of the components, the forces on each component were determined using a load-path analysis based on each component's tributary area. We specifically assumed that all components were simply supported, which is consistent with the design and construction of the houses. Based on our reconnaissance observations, the amakan and plywood walls were also assumed to retain their integrity, forming a diaphragm that transferred the wind pressures acting over the surface to the wall framing. Note that this analysis is intended to identify the first component failure, since load-sharing and pressure redistribution effects after the first failure are not considered when specifying component demands. While failures can propagate following the onset of failure in a given component, this study focused on establishing onset failures, given that a more sophisticated model/analysis could not be developed because of the limited information available about this type of construction and given the reduction in internal pressures caused by the structure changing from partially enclosed to partially open. Moreover, there is insufficient data to calculate new pressures and loads once a roof panel or other component was damaged.

Wind-Resisting Component and System Capacities

This section explains how we determined the expected capacities for each component and system (step 1c in Fig 4). The SI details specific assumptions for individual housing designs. Failure occurred when the demands in any given component exceeded its capacity according to Eqn. 2. After determining which components had failed for a given demand, we assessed whether a given performance level (Table 2) had been met.

CGI-Fastener Interface Capacities

We considered two failure mechanisms at the CGI-fastener interface: fastener pullout and CGI tear out around the fasteners. The studied houses had two types of fasteners: umbrella nails and roofing screws. The initial pullout capacities of these fasteners were taken from experimental tests of similar metal cladding attached to wooden purlins in Belize (Thurton et al. 2012). However, the withdrawal capacity of nails in wood is dependent on the specific gravity of the wood used, and pullout capacities of 1.3 and 1.4 kN (0.29 and 0.31 kips) for umbrella nails and roof screws, respectively, accounted for the greater specific gravity of Filipino coconut lumber (assuming medium hardness; Build Change 2015; Talatala et al. 2014) through the adjustment from ANSI/AWC (2015). The CGI tear-out capacity around the fasteners was calculated as:

$$\boldsymbol{R = c\, d^{\alpha} t^{\beta} f_u{}^{\chi}}, \qquad (4)$$

where d = head diameter of the fastener (mm), t = thickness of the CGI panel (mm), and f_u = ultimate strength of the CGI (MPa) (Mahendran and Tang 1999). C, α, β, and χ are constants based on the shape of the metal panels. Once the fastener pullout and panel tear-out capacities were calculated, the lesser value was used as the governing capacity at the panel-fastener interface. We assumed that not all fasteners would be properly placed during construction (not aligned with the centerline of the purlin), and those not properly placed would have a reduction in their capacity. We assumed that 3% of the fasteners would be improperly installed and that both the pullout and tear-out capacities would be reduced according to the triangular distribution from Stewart et al. (2018). Panel failure was then assumed to occur once ten percent or two of its fasteners fail, whichever is greater (Henderson et al. 2013b; Stewart et al. 2018)

Purlin, Truss, and Purlin-to-Truss Connection Capacities

Fig 5 illustrates the two relevant purlin-to-truss connections: hurricane straps and wooden cleats. In six of the housing designs considered, hurricane straps, were used to connect the purlin to the truss. We assumed these connections were similar to the H3 ties provided by Simpson Strong-Tie™, with a capacity of 2.2 kN (0.49 kips) (Simpson Strong-Tie 2019). Four designs used wooden cleats to connect the purlins to the truss; these connections often used only two nails (one nail into purlin and another into the truss). The likely failure mechanism of these connections was nail shear, so we calculated the shear capacity for nails in single shear (ANSI/AWC 2015). Although this shear failure can take on various forms, in almost all cases, the yielding was the governing shear failure mode. The remaining two designs did not have trusses nor purlin-to-truss connections.

Although less commonly observed, metal roof panels have been found to be pried from a house with the wooden purlins attached (Ginger et al. 2010; Parackal et al. 2018). To relate purlin-to-truss connection failure to panel failure, we assumed: 1) that all purlin-to-truss connections on a single purlin needed to fail for the purlin to fail, and 2) the purlin at the edge of a roof panel must fail for the panel to fail. These assumptions were based on the observed panel and purlin-to-truss connection failures in the Philippines following Typhoon Ursula, which showed the entire edge purlins failed before roof panel loss occurred. Future studies in other regions may consider different failure criteria (e.g., Parackal et al. 2018) informed by on contextual observations as these will vary with component detailing and regional practice.

We were also concerned with failure of the purlin members, but our analysis indicated that other components would fail first and thus such failure would not govern, even in cases with hurricane straps. For this reason, we do not further discuss the capacity of the purlin members. Likewise, trusses were not expected to govern failure given both the size of the wooden truss members and quality of the truss connections as many roof trusses were pre-fabricated off-site.

Roof-to-Wall Connection Capacities

There were six types of roof-to-wall connections in the studied houses: hurricane straps, wooden cleats, bolted wood, steel to concrete, toe-nailed, and wrapped rebar. The hurricane straps used to connect the roof to the wall (Fig 5b, similar to an H2.5 from Simpson Strong-Tie™) were larger and stronger than those used to connect the purlins to the truss; the assumed capacity of these connections was 5.8 kN (1.3 kips) (Ellingwood et al. 2004; Li and Ellingwood 2006). The capacity of the wooden cleats was determined as described above. We also determined the capacity of the bolted wood connections using shear capacity equations from ANSI/AWC (2015). As, the bolted connections used a single bolt to connect the truss to the wall on two sides, we used the shear capacity equations for bolts in double shear. For the designs with bolted connections, neither block shear nor wood splitting controlled. In two designs, steel channel purlins were connected to concrete ring beams at the top of load-bearing wall systems. These connections varied based on contractor and were either partially embedded in concrete or bolted to steel L-angles attached to the walls. We assumed the capacity of the bolted connections was 4.5 kN (1 kip) (Stewart et al. 2018) and that the partially embedded connections had the same capacity. The capacity of the toe-nailed connections was taken from previous literature with similar configuration and member sizes and assumed to be 2.9 kN (0.65 kips) (Cheng 2004; Khan 2012). In the Caputian-Amakan design, "flat bars" were placed at the edge of the roof panels and connected to the foundations in order to tie down the panels. These bars effectively pre-tension the roof system, which we modeled as an additional dead load, thereby, reducing the uplift forces experienced at the roof-to-wall connections. The Bangon and Caputian-Masonry roof-to-wall connections were rebar extending from the tops of the RC columns wrapped around the lower chord of the wooden truss, and, based on the third author's observations in Haiti, we assumed these connections would not fail (Kijewski-Correa et al. 2017).

Wall-Frame Capacities

Four types of wall-frame systems were included in this study: wood and RC frames and concrete and masonry load-bearing walls. In light-frame wood houses, wall failure from racking can occur under strong wind loads (Liu et al. 1990). In these houses, knee braces as in Fig 3c provide racking resistance (Erikson and Schmidt 2003), with additional lateral resistance provided by the plywood walls (Doudak and Smith 2009). For houses with amakan walls, we assumed that only the knee-brace frame provided lateral resistance.

To understand the capacity of the knee braces, we referred to tests by Erikson and Schmidt (2003) which revealed that the maximum force carried by the knee brace in an unsheathed wall system was 10.6 kN (2.4 kips); however, the knee braces in their systems had notched connections between the brace and beam/column, increasing the capacity compared to the bolted and toe-nailed braces found in the studied houses. Thus, we estimated the strength of the implemented knee-brace system using field data collected after Typhoon Ursula. In Candulo, approximately 50% of the houses experienced racking failures during Ursula (see Fig 2). A wind field map for Ursula is not available, but based on the available wind field data from Typhoon Yolanda (Kunze 2017), which followed a similar path of Ursula, and knowing the relative intensities of the two storms, we estimated that the maximum 3-sec wind gusts in this community during Ursula were 160 kph (100 mph) and that the resultant forces on the walls were 6.5 kN (1.5 kips). We used this value as the median capacity for the knee braces.

For plywood houses, we added to the capacity of the knee brace (6.5 kN, 1.5 kips) based on Salenikovich's (2000) tests on walls that are nailed at the base. Salenikovich found that the racking resistance of a 2.4 m (8 ft) and a 3.65 m (12 ft) wall were 3.5 kN/m (240 lb/ft) and 4.5 kN/m (308 lb/ft), respectively. On average, this adjustment increased the capacity of plywood wall frames by a factor of 2.5, which is consistent with other studies(Erikson and Schmidt 2003; Wolfe 1983). For houses with double sheathed walls, we increased the capacity by a factor of 1.9 based on results from Patton-Mallory et al. (1984), Lastly, we reduced the additional capacity provided by the sheathing to account for openings based on tests from Doudak and Smith (2009), who showed that the racking resistance of plywood walls with door and window openings were reduced by 55% and 50%, respectively.

Previous reconnaissance has documented wall failures due to uplift tensioning unreinforced masonry walls in hurricanes due to the absence of ring beams (Kijewski-Correa et al. 2017) or vertical reinforcement in walls (Suaris and Khan 1995). The houses included in this study contained both of these elements, so wall assessments were not included for RC (either frame or load bearing) and masonry structures as it was expected that the lateral capacity of these walls remains sufficient to resist wind loads even once the roof system dislodges.

Treatment of uncertainty

We used a Monte Carlo simulation to propagate the uncertainty in the wind loads and component capacities through the performance assessment (step 7 in Fig 4). The distribution parameters for the random variables are summarized in Table 3. Component capacities are assumed to be uncorrelated, *i.e.,* realizations of capacities for each component are independent of the capacities of other components (whether the same component type or not).

Sensitivity analysis

We also conducted a sensitivity analysis to assess the effect of design changes on a housing type's wind performance. Table 4 summarizes the variations we considered, which are each feasible to implement by organizations constructing post-disaster housing.

Results

Using the framework described above, we assessed the performance of twelve housing designs. In this section, we present the assessment of one housing design in detail, discussing first the expected failure sequence and its relation to the selected performance objectives, and then each failure mechanism in detail. We then compare the expected performance of the remaining houses. While the fragilities are presented by component, each component failure is related to a performance level (Table 2) as an indicator of housing system performance. We also discuss the findings of the impact of design modifications on the performance assessments. All wind speeds herein are defined as 3-second gusts.

Wind Performance of Candulo House Design

Fig 6 shows the estimated distributions of the wind speed instigating the first failure of each component in the Candulo house (Fig 1d) and their relationship to the performance levels detailed in Table 2. The first components expected to fail in this design are the walls, indicating the structural integrity performance objective is realized, at an estimated median wind speed of 160 kph (100 mph), *i.e.,* a signal 3 typhoon/category 2 hurricane. These wall frames are unbraced and have little sheathing stiffness (amakan walls), resulting in limited lateral resistance. The houses damaged by Typhoon Ursula in Fig 2 were located in Candulo, and community leaders estimated that 50% of these houses were damaged or destroyed during this storm, indicating that our results are consistent with the observed damage. To the best of our knowledge, there were no reported wall failures during Tropical Storm Urduja, and our analysis predicts a low likelihood of wall failure in wind speeds similar to those experienced in that storm.

The analysis also indicates that roof failure occurs at higher wind speeds than wall failure: a median wind speed of 220 kph (137 mph). Of the roof system components, the analysis predicts that failure initiates at the CGI-fastener interface. It is expected that the CGI panels will detach due to tear out around the heads of the umbrella nails used to fasten the panels to the purlins. The analysis also indicates that all analyzed components (walls, fasteners, purlin-to-truss and roof-to-wall connections) are predicted to fail in a future typhoon as strong as Yolanda. However, we note that the wind pressures do not account for redistribution after failure; as we witnessed in Candulo following Typhoon Ursula (see Fig 2), the roofs often remained intact after the racking failure. The other failure modes of roof-to-wall and purlin-to-truss connections are expected to have residual capacity beyond the load required to cause failure of the roof panels or walls. We therefore do not expect these failures to govern this design.

Wind Performance of Other Houses

Table 5 summarizes the expected failure mechanisms of the other housing designs, based on the median wind speed causing the onset of failure in four components: the first roof panel (1) considering capacity of the CGI-fastener interface and (2) considering purlin-to-truss connection, (3) roof-to-wall connections, and (4) wall-frame systems.

Fig 7 summarizes the onset failure wind speeds for the four components investigated in each house and their corresponding performance levels, and Fig 8 presents the results of the sensitivity analysis. The sensitivity analysis considers a gable roof and hip roof design with near-identical height, length, width, and number of purlins, based on the Bangon and Caputian-Amakan designs. The following sections discuss the observed governing failure mechanisms and the sensitivity analysis results for the purlin-to-truss connections, CGI-fastener interface, and roof-to-wall connections.

Governing Failure Mechanisms

We found three failure mechanisms governed the houses assessed in this study, linked to two performance levels: structural integrity and continued occupancy. Wall-frame system racking (structural integrity performance level) was the governing failure (3 of 12 designs), particularly for houses with amakan walls. The most common (9 of 12 designs) were governed by roof panel failure (continued occupancy performance level), with an almost even distribution of panel failures limited by the capacity of the CGI-fastener interface and purlin-to-truss connection. For gable roofs, panel failure is likely to initiate with failure of the purlin-to-truss connection; whereas, for hip roofs, panel failure is likely to occur because of failure at the

CGI-fastener interface. This difference in panel failure mechanism for hip and gable roofs is due to a combination of the increased capacity of the greater number of purlin-to-truss connections as well as reduced demand on the hip roofs (more favorable aerodynamic shape), which decrease the potential for purlin-to-truss connection failure.

This analysis is consistent with the communities' previous typhoon experience. The most common damage reported by households in these communities following Typhoon Ursula was roof cover loss, with approximately 40% of households stating that their roof panels had been damaged or blew off. Prevalence of roof damage was highest in the communities of Linao, San Pablo, and Sohoton. In these communities, the median wind speed at which roof panel failure occurs (Table 4) is less than the maximum wind speeds experienced during Typhoon Ursula. Very few respondents in Caputian (masonry houses) or Sagasumbut reported roof panel loss, and our analysis indicates that the median wind speed for panel failure in these houses is greater than the wind speeds of Ursula. These analysis results are also consistent with reports following Typhoon Yolanda that found, when houses were not completely destroyed, roof cover loss was the most common damage observed (Mas et al. 2015)

Wall-Frame Systems Failure (Structural Integrity Performance Level)

Fig 7a reports the median wind speeds (kph) of onset wall failures for houses with wooden wall-frame systems . For houses with amakan walls, wall failure was always the governing failure except for the Sohoton-Amakan design. Both the Candulo and Caputian-Amakan designs have strong roofs with hurricane straps and roof ties, respectively, and are expected to experience wall racking in our analysis. Indeed, we documented this failure in both communities following Typhoon Ursula, with greater prevalence in Candulo (see Fig 2) versus Caputian, which agrees with our analysis results. The predicted median wind speed of racking failure in Caputian was 190 kph (118), which is greater than the estimated wind gusts experienced during Ursula (164 kph (102 mph)). Therefore, the analysis is consistent with the observation that some, but well less than 50%, of the Caputian-Amakan houses failed due to racking in Ursula. Note that the footprint of the Sohoton-Amakan design is smaller than the other houses, reducing the wall loads, and thus panel failure at the purlin-to-truss connections governs in the analysis; we did not observe any racking failures of this design after Typhoon Ursula.

The analysis does not expect houses with plywood walls to be as vulnerable to wall racking. The only exception is the Sagasumbut-2Story design, which experiences racking at a median wind speed 13 kph (8 mph) less than that instigating the loss of the first roof panel due to fastener failure. Because the house is two stories tall and has only two walls in each orientation, the walls on the first floor must carry a larger load than either the 1-story designs or the Sagasumbut-Duplex design, which has interior partition walls.

Wall-frame racking that impairs structural integrity in these houses is expected at low wind speeds due to the limited capacity of the knee braces and flexibility of the amakan walls. As we witnessed after Typhoon Ursula, houses that experience this failure mechanism either collapsed or were uninhabitable due to residual drift. Amakan, or walls of a similar lightweight, woven material, are viewed by some post-disaster housing practitioners as a preferable alternative both because they are a permeable material that increases comfort in hot environments and because they expect any damage to be easily reparable. Practitioners expect that the amakan walls will "blow-out," reducing the drag coefficient and allowing the structural frame to remain intact, possibly with some minor racking that is easily correctable once the storm has passed. However, reconnaissance after Typhoon Ursula revealed that these walls do not "blow-out", but instead transfer sufficient load into the frames, resulting in story-mechanisms that are not easily reparable. We therefore recommend that organizations consider other wall materials with greater stiffness and/or provide appropriate lateral bracing and connections that can ensure load path continuity. This recommendation is particularly important where roof systems have been strengthened as in Candulo (hurricane straps) and Caputian (roof ties with "flat bars").

Purlin-to-Truss Connection Failures (Continued Occupancy Performance Level)

Two different types of purlin-to-truss connections were used in the studied houses, wooden cleats and hurricane straps, with differing performance shown in Fig 7b. While the uplift capacity of the hurricane straps is more than double that of the wooden cleats, the most influential factor in the performance of the purlin-to-truss connections is the roof shape. Both the cleat and strap connections on the hip roofs perform better than both connection types on gable roofs, a result of the hip roof's reduced wind pressures (up to 80% at the roof ridge and 25% on the roof edge).

Because the performance of the purlin-to-truss connection depends on both the connection type and roof shape, we conducted a sensitivity analysis to explore the effects of these two design features on panel failure rates, as shown in Fig 8a. For both hip and gable roofs, replacing wooden cleats with hurricane straps improves performance, increasing the median wind resistance by approximately 45%. Changing the roof shape from gable to hip and maintaining the purlin-to-truss connection also improves performance by 42% for both cleats and straps. So, while a gable roof with hurricane straps is expected to experience its first panel failure at nearly the same median wind speed as a hip roof with wooden cleats, a hip roof with straps will outperform both designs by almost a factor of two. However, the use of hip roofs, particularly with hurricane straps, is only advisable if the walls have adequate capacity to transfer the forces from this substantially stronger roof system to the foundation.

Failures at the CGI-Fastener Interface (Continued Occupancy Performance Level)

Fig 7c provides results for failure of the CGI panel loss due to failure at the CGI-fastener interface. The first panel failure occurs over a large range of wind speeds, depending on the housing design, from approximately 185 kph (115 mph) to 305 kph (190 mph). This suggests that failure at the CGI-fastener interface is unlikely in weaker storms, like Tropical Storm Urduja, but is expected in many of the housing types in storms like Typhoon Ursula. As shown in Fig 7c there is no trend between roof shape or roof elevation and these panel failure rates.

Fig 8c and d examine how different CGI gauges and fastener spacings affected fastener and panel failures. Increasing panel thickness from 28 to 26 gauge and 26 to 24 gauge increased the median wind speeds causing failure by 22% and 34%, respectively, for both hip and gable roofs. While panels on the hip roof performed slightly better than those on gable roofs (by 9%), panel thickness was more important because the panel performance was governed by CGI tear out around the fasteners. Decreasing the fastener spacing on the interior purlin lines from 300mm (12in) to 150mm (6in) (exterior purlin lines were assumed to already have a spacing of 150mm (6in)) improved the median wind speed causing failure by an average of 41%. Similar to panel thickness, the trend is not dependent on the roof shape.

We also assessed whether increasing the CGI thickness or fastener spacing is more beneficial to panel performance. Fig 8e shows that both have a considerable effect on panel performance: decreasing the interior nail spacing from 300mm (12in) to 150mm (6in) improves panel performance, in terms of median wind speed, by an average of 49% regardless of panel thickness, while increasing panel thickness and maintaining fastener spacing improves performance by 28%. Thus, we recommend that organizations providing post-disaster housing, whenever possible, use at least 26-gauge CGI (24-gauge is preferable) and at most 150mm (6in) spacings for both edge and interior fastener lines. The CGI panel thickness is especially important because this failure mode increases in prevalence as CGI corrodes with age, which is not considered here. While there are budgetary implications, notably for increasing panel thickness, greater fastener density has minor impacts to material costs and should be advocated at minimum. The associated increase in installation efforts could be offset through community volunteer labor.

Roof-to-Wall Connection Failure (Life Safety Performance Level)

Roof-to-wall connection failure was not anticipated to govern any of the considered designs, which agrees with our reconnaissance after Typhoon Ursula. The median wind speeds associated with failure of the roof-to-wall connections are shown in Fig 7d. The good performance of these connections is based primarily on the connection type rather than the house geometry. Bolted connections unsurprisingly, perform the best. Next, hurricane straps, and toe-nailed connections plus hurricane straps, perform similarly, with capacity 11-24% less than bolted connections. Toe-nailed and wooden cleat connections are expected to perform the worst. The expected performance of the wooden cleat connection appears to be better than that of a toe-nailed connection because, in this case, the house that included wooden cleats at the roof-to-wall connection also used “flat bars” to tie the roof to the foundations.

This suggests that bolted connections may be the best roof-to-wall connections option (Fig 8b), though this is very much dependent on the dimensional and material properties of the timber used and the geometry of the connection. In connections with multiple bolts, which was not the case in any of this study’s housing designs, wood fracturing or block shear can govern; thus, the connections must be appropriately designed to avoid these failure mechanisms. In light of the above, hurricane straps remain the most consistent means of assuring effective roof-to-wall load transfers. However, as the roof-to-wall connection is not the governing failure mode in any of the designs, and noted vulnerabilities remain in other elements of the load path that could even be exacerbated by improved roof-to-wall connections, organizational resources are likely better spent at first improving wall strength or roof panel performance.

Additional Considerations

We investigated two additional design decisions: purlin spacing and eave length. Both of these decisions affect the performance of multiple components, so we discuss them here.

Purlin spacing affects the performance of both the CGI-fastener interface and the purlin-to-truss connections. As shown in Fig 8f, decreasing the purlin spacing from 600mm (24 in) to 450mm (18 in) and from 450mm (18 in) to 300mm (12 in) on a hip roof increases the median wind speed instigating panel loss due to failure at the CGI-fastener interface by 30% and 12%, respectively. On a gable roof, these reductions in purlin spacing result in a 7% increase in the median wind speed at which panel loss due to purlin-to-truss connection failure occurs. Decreasing purlin spacing also improves the performance at the purlin-to-truss connections on a hip roof and the CGI-fastener interface on a gable roof, though these were not identified as governing failure mechanisms in the initial assessment. While decreasing purlin spacing is an option to improve roof performance in areas where access to thicker CGI is limited, it could be challenging to implement in regions with limited access to wood.

The additional uplift forces caused by extending the eaves results in poorer performance of the roof panels. Adding a 0.5m (1.6 ft) and 1.0m (3.3 ft) eave can decrease the median wind speed at which panel failure occurs by 10% and 25%, respectively. These forces also increase the loads on the roof-to-wall connections, but these connections were not the governing failure mechanism in the initial analysis, even for the longer eave case. Many households increase the eave length to provide protection from the sun and rain, so roofs should be designed to account for this increase in wind uplift demand.

System Effects

While we have provided recommendations for improving the performance of the roof panels and roof-to-wall connections, load path is jeopardized if organizations do not consider the wall and roof together. They must use care in understanding the capacity of wall systems when strengthening roofs (Kijewski-Correa et al. 2017). From a performance-based engineering perspective, it is preferable to have a weaker roof that will experience roof cover loss because this primarily impairs continued occupancy, than to have a strong roof that experiences no damage but instead results in higher demands on more vulnerable walls and impairs structural integrity. Roof cover loss is comparatively easier to repair, and, while flying CGI panels can cause injury and damage to neighboring structures, the safety of occupants sheltering in place and the speed of the recovery process are both improved through the prevention of severe damage or collapse at the system level as a result of wall failure. Therefore, organizations should only strengthen the roof, either at the CGI-fastener interface, purlin-to-truss connection, or roof-to-wall connection, if an accompanying analysis suggests the walls have sufficient capacity to resist the resulting increased load demands.

Although we did not expect the houses with RC frames or masonry walls to be damaged, some NGOs continue to implement load bearing unreinforced masonry walls without any confining elements or reinforcement. Reconnaissance following Hurricane Matthew in Haiti found substantial damage to unreinforced masonry walls due to uplift at the roof-to-wall connections, at times leading to complete wall failure and thus system-level collapse (Kijewski-Correa et al. 2017). Organizations that choose to build with masonry must ensure that appropriate confining elements and ideally wall reinforcement are included so that the wall system has adequate strength to complete the load transfer from the roof to the foundation.

Lastly, some households had added extensions to their house – a modification that could either increase or decrease a house's wind vulnerability depending on the addition's geometry and lateral resistance. Nevertheless, we calculated that in Candulo, the community that experienced the most wall racking during Typhoon Ursula, extensions would need to increase the current wall capacity by 180% in order for wall racking to not be the governing failure mode.

Limitations and Future Work

While this study advances the knowledge of the performance of non-engineered post-disaster housing in typhoons, there is uncertainty about the materials and design of these structures. These analyses are further limited by unavailability of data on the capacities of components and connections commonly found in houses constructed in resource-limited communities. Additional experimental tests of context-specific connections and assemblies would improve the accuracy of the models and findings. We particularly recommend testing of wooden cleat connections and wooden frames with plywood and amakan walls. As organic materials are often more accessible than commercial products like hurricane straps, it is important to explore strategies to improve the capacity of such locally available connection details and wall assemblies. In addition, our assumptions about variability in capacities (Table 3) are likely optimistic for a resource-limited context and future work many consider a larger coefficient of variation, or a higher rate of improperly installed fasteners; however, we do not expect these changes to greatly influence overall trends. Additionally, fatigue due to cyclic loading in typhoons has been documented (Boughton and Reardon 1984) and will affect the performance of the components included in this study, particularly hurricane straps and capacity of CGI cladding at the panel-fastener interface. However, given the limited data on the specific materials and connections used in the studied houses, we did not consider the effects of low-cycle fatigue in this study. We suggest that these effects be considered in future work.

This study is further limited by the availability of wind pressure distributions for structures with the geometries and eave lengths found in the houses included in this study. Wind-tunnel tests of homes with traditional geometries, particularly related to roof slope and eaves, two critical parameters for aerodynamic loading as well as ventilation and shading in tropical climates, would reduce the uncertainty in load demands. Moreover, wind pressures were not redistributed after failure of the first component, which changed both the surface area as well as the aerodynamic properties, limiting the interpretation of the failure sequence expected in these houses. Future investigations should consider more detailed finite element modeling of critical elements of the load path to capture load sharing along with the redistribution of pressures and load paths after the envelope has been breached. Lastly, we assessed the performance of newly constructed houses and did not account for deterioration of materials or use of lower-quality materials. In particular, our analysis did not account for corrosion of CGI or wood deterioration, both of which have been observed and may influence performance. Future work should examine the performance over the entire lifecycle to provide organizations with a more complete understanding of the investments that will lead to long-term resilience.

Conclusions

In this study, we provided a framework for component-level performance-based wind engineering assessment of post-disaster housing in resource-limited contexts and evaluated the wind performance of twelve housing types built by organizations in the Philippines after Typhoon Yolanda. While NGOs and government agencies build such houses after disasters with the goal of increasing post-disaster housing safety, their performance has not been assessed by an engineering analysis, limiting the ability to make recommendations to further improve safety. To address this need, we use component-level performance-based methods to quantify the median wind speeds causing the onset of common failure mechanisms related to three performance objectives and explore how minor changes in these designs might improve performance.

We found that the wall-framing system (and the relative capacity of the roof and wall systems)was the design feature that dictated the governing failure mode. Specifically, wood-frame houses with woven amakan walls, with one exception, were governed by wall racking (3 of 12 cases), failing to meet the structural integrity performance level and, thus, endangering occupants. These results agreed with reconnaissance following Typhoon Ursula. Walls were expected to fail at an average 3-sec gust of 180 kph (112 mph), which is equivalent to a low-strength category 3/signal 4 storm and is less than the maximum 3-sec wind gusts recently experienced at the studied locations during Typhoon Ursula. Houses with plywood walls are not expected to fail until an average wind speed of 230 kph (143 mph) due to the added capacity from the plywood sheathing. The remaining nine designs were governed by roof cover loss, failing at the continued occupancy performance level, either due to failure at the CGI-fastener interface or purlin-to-truss connection.

For houses with gable roofs, the first panel loss is expected to occur at an average 3-sec gust of 160 kph (100 mph), equivalent to a category 2/signal 3 storm, and for hip roofs at an average of 215 kph (135 mph), equivalent to a category 4/signal 4 storm. Panel loss on hip roofs was the result of fastener failure, while purlin-to-truss connection failure governed gable roofs. For failure at the CGI-fastener interface, decreasing nail spacing from 300mm (12in) to 150mm (6in) along all purlin lines, is the most beneficial modification, increasing the wind speed at which the failure occurs by 40%, or again, an entire signal rating. Additionally, increasing panel thickness improves performance by 21 to 35%. Replacing wooden cleats with hurricane straps at the purlin-to-truss connections can increase capacity by 45%, or an entire signal rating (*e.g.,* from a signal 2 to a signal 3 storm).

Roof-to-wall connections, related to the life safety performance level, were not expected to govern the failure of any of the studied houses.

From this analysis, we are able to compare across the housing designs, showing that houses with hip roofs, hurricane straps, shorter eaves, more-closely spaced fasteners and purlins, and plywood-sheathed walls perform better, and quantifying the relative improvement (in terms of wind speed) associated with these changes. Therefore, we recommend that practitioners consider a variety of design changes, including using hip roofs, thicker gauge CGI, decreased fastener spacing, and hurricane straps. The most influential improvements for vulnerable roofs were decreasing nail spacing and using hurricane straps. Most importantly, though, we recommend improving the resistance of the walls in wood frame housing to consider the entire load path when selecting the components to strengthen. We found that houses with weak walls and strong roofs were likely to experience racking and even collapse, affirming reconnaissance observations that this is the largest threat to occupant safety. It is preferable to have a weaker roof that will experience panel loss and relieve pressure on the walls than to have a roof that remains intact and propagates higher demands to the walls. In particular, we recommend using a stronger and stiffer wall material, such as plywood, instead of a porous, woven material like amakan. If materials like amakan are used for the walls, it is crucial that lateral strength of the system be enhanced through additional bracing.

This research expanded the study of housing wind performance from typical structures built in North America and Australia to include post-disaster housing in resource-limited communities. This study focused on performance of post-disaster housing built after a typhoon in the Philippines; thus, material and geometry assumptions were based on a specific context, though many features have commonalities worldwide. However, future work can build upon this framework, adjusting the assumptions and designs for different post-disaster events, and indeed housing built by households pre-disaster, in other resource-limited communities. The need for understanding post-disaster housing performance in order to identify those features that most improve housing performance on a limited budget is likely to grow in future years with an increase in disaster frequency and severity (UNISDR 2015). Thus, we anticipate that this framework can be adjusted accordingly for use in assessing standardized housing designs in new contexts.

Data Availability Statement

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request, including measurements of studied house designs and wind analysis code.

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Supplemental Materials

Table S1 and Figs. S1-S25 are available online in the ASCE Library (ascelibrary.org).

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Figure Captions

Fig 1. Examples of NGO or government housing designs constructed following Yolanda: a) Bangon, b) Sagasumbut-Duplex, c) San Pablo, d) Candulo, and e) Sohoton-Plywood

Fig 2. Examples of wall racking in Typhoon Ursula in houses constructed after Yolanda in Candulo, note the image on the right is a complete soft-story failure.

Fig 3. Schematic of typical roof and wall structures for the studied houses: a) section view of the panels, purlins, truss, and connections for any truss in a gable roof and common truss in a hip roof; b) elevation of a wooden wall frame with a knee brace.

Fig 4. Overview of the analysis process.

Fig 5. Examples of roof connections using a) wooden cleats and b) hurricane straps

Fig 6. Probabilities of failure onset in wall framing (structural integrity performance level), roof panels (continued occupancy performance level), and roof-to-wall connections (life safety performance level) for the Candulo house design as a function of wind speed (3-second gust). The vertical lines represent the maximum 3-sec gusts estimated for Candulo in three recent storms. [1kph = 0.62 mph].

Fig 7. Summary of median wind speeds (3-sec gusts) of onset failures in wall frames (structural integrity performance level), panels (continued occupancy performance level), and roof-to-wall connections (life safety performance level) for each housing type. Vertical lines indicate the maximum 3-sec wind gusts measured in three recent storms [1 kph = 0.62 mph].

Fig 8. Sensitivity analysis results for variations of a) purlin-to-truss connections, b) roof-to-wall connections, and CGI-fastener interface properties: c) panel thickness, d) fastener spacing, e) panel thickness and fastener spacing, and f) purlin spacing. GA = gauge and 150/150 = fastener spacings on edge/interior purlin lines. [1 kph = 0.62 mph].

Table 1. Design details of studied houses

Design Name	Location	# of Houses	# of Stories	Height (m)	Plan Dimensions (m x m)	Column Material	Wall Material	Roof Shape (# of trusses)[d]	Roof-to-Wall Connection	Purlin-to-Truss Connection	Purlin Spacing (mm)	Panel Fastener (Spacing)
Bangon	Leyte	150[a]	1	3.4	6.45 x 4	Reinforced concrete	Masonry/ plywood	Gable (4)	Wrapped rebar	Wooden cleats	575	Umbrella nails (150/300)
Candulo	Eastern Samar	105[a]	1	4.35	6 x 3	Coconut lumber	Amakan	Hip (3,6)	Hurricane straps	Hurricane straps	600	Umbrella nails (150/150)
Caputian-Amakan	Eastern Samar	119[b]	1	3.95	5.5 x 3.6	Coconut lumber	Amakan	Hip (2,4)	Wooden cleats	Hurricane straps	525	Umbrella nails (150/300)
Caputian-Masonry	Eastern Samar	119[b]	1	3.75	4 x 3	Reinforced concrete	Masonry	Hip (3,6)	Wooden cleats	Wooden cleats	625	Umbrella nails (150/300)
Linao	Leyte	1000[a]	Loftable[c]	5.2	6.5 x 4	N/A	Concrete	Gable	Bolted	N/A	650	J-bolts (300/300)
Sagasumbut -1Story	Leyte		1	3.15	4.5 x 3.65	Lumber	Plywood	Hip (3,6)	Toe-nailed & hurricane straps	Hurricane straps	650	Umbrella nails (150/300)
Sagasumbut -2Story	Leyte	484 total	2	6.5	3.65 x 2.45	Lumber	Plywood	Hip (3,6)	Toe-nailed & hurricane straps	Hurricane straps	850	Umbrella nails (150/300)
Sagasumbut -Duplex	Leyte		2	6.5	4.9 x 3.65	Lumber	Plywood	Hip (3,6)	Toe-nailed & hurricane straps	Hurricane straps	850	Umbrella nails (150/300)
San Pablo	Leyte	42	2	6.55	4 x 3.5	Coconut lumber	Plywood/ amakan	Gable (3)	Toe-nailed	Hurricane straps	400	Screws (150/150)
Sohoton-Amakan	Eastern Samar	63 total	1	3.45	4 x 4	Coconut lumber	Amakan	Gable (3)	Bolted	Wooden cleats	450	Umbrella nails (150/300)
Sohoton-Plywood	Eastern Samar		1	5.8	5 x 3.5	Coconut lumber	Amakan	Gable (3)	Bolted	Wooden cleats	450	Umbrella nails (150/300)
Tolosa	Leyte	558[a]	Loftable[c]	5.5	5.25 x 4	N/A	Masonry	Gable	Bolted	N/A	600	Screws (150/300)

[a] This same housing design was also used in other communities. This number includes those only in the studied community.
[b] Total number expected to be built. Some houses were still under construction or waiting to be built as of January 2020.
[c] House was built with 1-story with space to add interior second floor. [d] First number is the number of main trusses and second number is number of hip trusses.

Table 2. Performance levels and their corresponding failure modes and component-specific failure mechanisms

Performance Level[a]	Associated Failure Mode	Component-Specific Failure Mechanism
Continued Occupancy	Loss of 1st roof panel	Failure of the CGI-fastener interface CGI-fastener interface (Fastener pullout or CGI tear-out) for 10% of a panel's fasteners OR Failure of all the purlin-to-truss connections for a purlin at the edge of the roof
Life Safety	Roof system failure	Failure of one roof-to-wall connection
Structural Integrity	Wall failure	Wall racking of one wall

[a] As defined in van de Lindt and Dao (2009)

Table 3. Uncertainty parameters for wind load and resistance capacities

Source of Uncertainty	Distribution	COV	Source
Wind load on component	Normal	0.2	(Li and Ellingwood 2006)
Fastener pullout	Normal	0.25[b]	(Li and Ellingwood 2006; Stewart et al. 2018)
CGI tear-out	Normal	0.25[b]	(Stewart et al. 2018)
Hurricane straps	Normal	0.1	(Ellingwood et al. 2004; Li and Ellingwood 2006)
Wooden cleat connection	Normal	0.4	Assumed[a]
Toe-nailed connection	Normal	0.3	(Cheng 2004; Morrison and Kopp 2011)
Bolted connection	Normal	0.4	Assumed[a]
Racking resistance	Normal	0.4	Assumed[a]

[a] We assumed a COV of 0.4 for capacities that were calculated from equations or extrapolated from similar tests. As values were not drawn from experimental tests, we assumed a higher level of uncertainty than for values that have been validated through repeated testing.

[b] COV for fastener pullout/CGI tear-out also incorporates variance in capacity based on improperly installed fasteners.

Table 4. Design variations considered in sensitivity analysis

Component	Variations Considered			
CGI thickness	24-gauge[a]	26-gauge	28-gauge	
Fastener spacing	As-built	150mm/150mm (6in/6in)	150mm/300mm (6in/12in)	
Purlin-to-truss connection	Wooden cleats	Hurricane straps		
Roof-to-wall connection	Wooden cleats	Toe-nailed	Hurricane straps	Bolted
Purlin spacing	600 mm	450 mm	300 mm	
Eave length	0 m (no eave)	0.5 m	1 m	

Note: The first number in the fastener spacings indicates the spacing on the edge of the panel, and the second refers to the spacing along purlin lines on the interior of the panel.

[a] Gauge thickness increases as the number increases. 24-gauge is the thickest panel considered.

Table 5. Median wind speed (3-sec gust) causing onset of failure in different components for the studied houses [1 kph = 0.62 mph]. Bolded values indicate the median wind speed of governing failure mode.

	1st panel failure (CGI-Fastener) (kph)	**1st panel failure (Purlin-to-Truss) (kph)**	**Roof-to-wall failure (kph)**	**Wall failure (kph)**	**Expected failure mechanism based on first failure**
Bangon	187	**133**	N/A	N/A	Panel failure due to purlin-to-truss connection
Candulo	223	371	259	**161**	Wall racking
Caputian-Amakan	306	360	227	**190**	Wall racking
Caputian-Masonry	284	**252**	N/A	N/A	Panel failure due to purlin-to-truss connection
Linao	**148**	N/A	317	N/A	Panel failure due to fasteners
Sagasumbut-1Story	**234**	371	270	280	Panel failure due to fasteners
Sagasumbut-2Story	212	274	302	**199**	Wall racking
Sagasumbut-Duplex	**191**	353	266	252	Panel failure due to fasteners
San Pablo	288	**176**	180	191	Panel failure due to purlin-to-truss connections
Sohoton-Amakan	283	**180**	342	240	Panel failure due to purlin-to-truss connection
Sohoton-Plywood	283	**155**	342	203	Panel failure due to purlin-to-truss connection
Tolosa	**208**	N/A	333	N/A	Panel failure due to fasteners