Wind-uplift fragility analysis of roof sheathing for cultural heritage assets in the Philippines

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Abstract

Recent catastrophic events in the Philippines (e.g., the 2013 Typhoon Haiyan) have highlighted that ageing cultural heritage (CH) assets are especially vulnerable to typhoon-induced extreme wind. Non-engineered CH roofs have been recognized as the most vulnerable component in the building envelope due to wind uplift, often resulting in large economic losses and disruption to those assets. Effective prioritization of high-vulnerable structures is a key to effective risk mitigation and resilience-increasing strategies.

This paper introduces a simulation-based approach for non-engineered CH roof fragility analysis (i.e., to assess the likelihood of different levels of roof damage over a range of wind hazard intensities) and risk assessment. In the proposed approach, two common failure modes are considered, corresponding to (first) roof-fastener pullout and roof-panel pullover. The overall aim is to identify the highest-risk candidate assets and prioritize more detailed data collection campaigns and structural assessment procedures (e.g., properly accounting for load redistribution and fastener failure progression), and ultimately to plan further repair/strengthening measures.

An illustrative application of the proposed procedure is presented with reference to eight priority CH assets in Bohol, the Philippines. The analysis results indicate that, especially for roof panels supported by wood-type purlins, pullout fragility/risk can be significantly high, while the pullover one is generally lower.

Keywords: Wind fragility/risk assessment; roof pullout/pullover failure; wind uplift demand/capacity; cultural heritage assets

1. Introduction and Motivation

The Philippines is among the top global disaster hotspots, being exposed to a wide range of natural and man-made hazards. This represents a limiting factor to the country's sustainable development. In the recent *Germanwatch Global Climate Risk Index 2020* [1], the Philippines ranked 4th among the most affected countries by disasters (in the period 1999–2018), with a significant percentage of gross domestic product (GDP) in areas at risk. Located in the Pacific Ring of Fire, it is highly exposed to earthquakes, volcanic eruptions, and other geological hazards, as well as to multiple typhoons and monsoon rains. The year 2013 was a devastating year for the country: a significant M7.2 earthquake in Bohol (on October, 15) and then super Typhoon Haiyan (in November) caused several casualties, major damage, and a significant increase in poverty levels in the affected areas across the islands of Cebu, Bohol and the Visayas region. Specifically, the Bohol earthquake affected 3.2 million people, caused 227 casualties, displaced over 348,000 individuals, and damaged over 78,000 houses [2]. Haiyan, a Category 5-equivalent typhoon with wind speed over 300 km per hour, affected a total of 16.1 million people; nine of the country's 17 administrative regions were affected by the typhoon, covering 12,139 barangays (villages) in 44 provinces, 591 municipalities, and 57 cities. The typhoon caused 6,300 reported fatalities and 1,062 people missing [3]. Many cultural heritage (CH) assets (e.g., provincial capitols, heritage churches and other buildings of historical value) were seriously damaged, with some even irretrievably lost, in 14 provinces in the Visayas. More recently, the 2019 Pacific typhoon season was the costliest ever recorded. It was an above-average year with a grand total of 29 named storms, 17 typhoons, and four super typhoons. The season's activity started right out of the gate when Pabuk, the first named storm

of the year, attained tropical storm status on January 1 to become the earliest-forming western Pacific tropical storm on record. The activity continued to the final days of the year when Phanfone, the final named storm of the season, dissipated on December 29. An average of 20 typhoons are experienced each year in the Philippine Area of Responsibility (PAR), six of which typically make landfall with another four passing by closely enough to cause loss. The PAR is an area in the Northwestern Pacific where the Philippine Atmospheric, Geophysical and Astronomical Services Administration (PAGASA) monitors weather occurrences; tropical cyclones that enter or develop in the PAR are given Philippine-specific names.

In recognition of the country's high exposure and vulnerability to natural hazards, the enactment of the Philippine Disaster Risk Reduction and Management (DRRM) Act in 2010 (Republic Act 10121) enabled substantial progress in shifting the emphasis from emergency response to preparedness, mitigation and prevention. Significant resources have been provided for ex-ante investments and new areas of engagement have been considered in the policy dialogue. However, challenges remain in enabling implementation of DRRM investments in priority sectors, including CH.

CH assets are exposed to and vulnerable to natural hazards as well as climate change, and a host of other factors [4,5]. At the same time, CH is increasingly recognized as a driver of resilience for communities and wider society. CH contributes to (1) inclusive economic development, by attracting investment and promoting green and local jobs related to a wide range of sustainable activities in areas such as tourism (a priority sector in the Philippines), conservation, construction, and art in general; and (2) inclusive social development, by enhancing the feeling of place and belonging, mutual respect and sense of collective purpose, thus contributing to the social cohesion of a community and reducing inequalities. In particular, the tourism sector is one of the priority sectors by which the Government of the Philippines (GoP) aims to foster inclusive and sustainable socio-economic development, due to its potential

for job creation. Under the 2011-2016 National Tourism Development Plan (NTDP), top cluster destinations were identified for which key tourism programs, such as cultural tourism, were targeted for development. Recognizing the potential for tourism in various CH sites across the country, the NTDP articulated the urgent need to safeguard vulnerable heritage sites. From a structural engineering perspective, CH assets are largely non-engineered structures, i.e., they have been built without following prescriptions related to any specific building code¹. More in general, while the building codes in the Philippines have evolved over the years and the current unified national code, i.e., the National Structural Code of the Philippines (i.e., NSCP 2015 [6]) compiled by the Association of Structural Engineers of the Philippines (ASEP), has provisions for both seismic and wind designs, the extent to which wind provisions are adopted and enforced nationwide is uncertain, particularly in the case of ageing CH assets. Moreover, several CH roofs in the Philippines, especially in low income areas, have been built using wood frames and lightweight metal sheets (LWMSs), e.g., Fig. 1. Such timber-framed constructions are among the most vulnerable structures to wind-hazard areas (e.g., [7]) and, due to their lightweight roofs, these components are extremely susceptible to uplift pressure/load caused by strong wind (e.g., [8,9], among many others).

Fig. 1. Example of Filipino CH-asset roofs featuring wood frames and lightweight metal sheets (LWMSs)

¹ It is worth noting that some renovations/repairs post-2013 in the Philippines may have been carried out in accordance with recent building codes and wind standard provisions.

101

102 In fact, post-event surveys in the Philippines and around the world reveal that most economic
103 loss in high wind-hazard areas are related to the breach of the building envelope (e.g., [10]),
104 resulting in considerable monetary losses due to direct damage and downtime after wind events,
105 as observed in several studies (e.g., [11]). The breach of a building envelope typically includes
106 roof panel uplift, roof-to-wall connection failure, roof system damage, and rupture of window
107 and door glasses due to excessive pressure or missile impact. In particular, failure of roof
108 panel/sheathing may lead to water penetration and severe interior damage to a property and its
109 contents. Such a damage in roof structural systems can cause significant increase of wind
110 pressure on interior surfaces and loss of sufficient lateral support to walls, resulting in
111 progressive failure of other portions of the building and even failure of the structural system as
112 a whole in some instances (e.g., [10]). Research carried out in the United States (US, e.g.,
113 [12,13]) further indicates that following the first removal of a roof panel by wind uplift, the
114 magnitude of subsequent losses can amount to significant percentages of the total insurance
115 claims. Moreover, with the roof heavily damaged or removed, walls may become unstable
116 without enough lateral support and collapse. Therefore, the accurate and effective prediction
117 of roof damage caused by strong wind plays a key role in the identification of highly-vulnerable
118 assets and the development of optimal retrofitting strategies for those assets (e.g. [14]).

119 Within this context, this study introduces a straightforward simulation-based approach named
120 WARP² (*Wind Assessment of Roofs to Pullout & Pullover*) for the derivation of fragility
121 relationships and typhoon risk assessment of non-engineered CH-asset roofs. The proposed
122 approach considers both roof-fastener pullout and roof-panel pullover as dominant failure
123 modes. These failure modes are related to the breach of the asset roof, which is closely related
124 to the performance objective of minimizing the overall building damage, as discussed above.
125 In the proposed approach, the uplift capacity of the roof is limited by the assembly's weakest

link and the considered failure modes correspond to (1) pullout failure of the first roof fastener (i.e., screw or nail); and (2) pullover failure of the first roof panel. In practice, once the failure of a single fastener occurs, the load is redistributed to the surrounding fasteners causing failure to propagate through the panel/roof (e.g., [9]); similarly, there is a strong correlation between the first panel removal and subsequent damage to the building structure and resulting losses (e.g., [15]). However, the simplifying assumptions made here are deemed appropriate for a rapid yet effective risk prioritization of various assets within a large building portfolio, for instance to allocate resources to more detailed data collection campaigns and structural assessment procedures (e.g., properly accounting for load redistribution and fastener failure progression, considering other structural components/connections), and ultimately repair/strengthening planning and conceptual design (e.g., [16]).

Similarly to the well-known HAZUS-MH Hurricane Model (e.g., [17,18]), the physical damage to a building subjected to strong wind is modeled here using an engineering-based demand and capacity approach, where once the wind-induced loads (i.e., demand) acting on a building/building component are computed, the resulting physical damage is estimated in terms of failure of the first roof fastener or first roof panel through *ad-hoc* capacity models. It should be emphasized that similar engineering-based approaches have been proposed in the literature, e.g., [7] and [12], among several others. However, the resistance statistics employed in most of the existing studies are generally based on experimental testing for similar connections in engineered roofs characterized by well-defined nail types and patterns. This is not the case for the considered CH assets. Therefore, the capacity models used in this study are derived explicitly using various mechanical models (including code-based models) and based on local data collected from field surveys. This unique set of data can enable the practical development of *ad-hoc* capacity models for non-engineering CH-asset roofs.

An illustrative application of the proposed procedure is presented with reference to eight priority CH assets located in Bohol, the Philippines. The methodology proposed in this study covers the determination of wind uplift loads/demands and resistances/capacities, the surveying of various roofing system for the case-study CH assets, and the fragility analysis of those roofing systems through comparison between uplift resistances and wind loads, using a fully probabilistic approach and structural reliability concepts. Once the wind fragility curves are derived through WARP², they are then combined with statistical information and probabilistic models of annual extreme wind speeds for Bohol to estimate the wind risk profiles of the considered assets.

2. Demand and Capacity Definition

Many roofs in the Philippines, especially in low-income areas, informal settlements, and in the case of non-engineered structures (such as CH assets), were largely built using wood frames and LWMSs (e.g., galvanized iron sheets). During extreme wind speeds such as typhoons, these roofs are highly vulnerable to wind uplift. Two basic failure modes of roof panels are typically observed and defined as (1) *pullout failure*, when a roof fastener (e.g., screw or nail) is pulled out from the holding member due to wind-induced uplift loading; and (2) *pullover failure*, when a roof panel fails/fractures while the connecting fastener is still intact within the holding members. These two failure modes are schematically illustrated in Fig. 2. It is worth noting that (1) the holding-up effect of surrounding LWMSs due to the overlap is not considered in the pullover failure model; (2) fatigue-induced connection failure is not considered in the analysis. A significant reduction of the ‘static’ uplift capacity (described in Section 2.3 below) due to highly fluctuating winds was reported in the literature, based on experimental testing (e.g., [19]). To the authors’ knowledge, no similar studies are available in literature for CH roofs using LWMSs. More in general, the typhoon-risk prioritization of CH-

asset roofs – as proposed in this study – should not be significantly affected if fatigue effects are neglected. In fact, most of the considered CH assets share similar roof-panel/purlin materials and fastener types. While neglecting fatigue effects may results in an underestimation of the absolute values of the fragility/risk estimates for the considered assets, considering fatigue effects would lead to a uniform reduction of the roof uplift capacity for all the considered assets, not affecting the relative ‘ranking’ and relative comparisons performed in the study.

Fig. 2. Schematic illustration of the considered roof uplift failure modes: (a) pullout failure; and (b) pullover failure

In the proposed wind fragility and risk assessment framework, the full probabilistic modelling of gravity (i.e., dead) and wind-load effects (i.e., wind uplift demand) are included. Uncertainties in structural component characteristics (e.g., materials, geometries, models), i.e., those affecting the roof-panel uplift capacity, are also modeled probabilistically. Details of statistical characteristics for these random variables (RVs) used are provided in the following sections.

2.1. Wind Load Definition

Wind loading on a building depends on the flow pattern around the building, which, in turn, depends on the building geometry, dimensions, surroundings, and wind-flow characteristics.

For instance, relatively large fluctuating suction pressures are typically generated in flow separation regions close to the leading edges of a roof. Variations in wind flow due to different types of discontinuities together with velocity fluctuations generate a complex spatially and temporally varying pressure field on the building surface. However, a code-based approach, accounting for those aspects in a simplified fashion, can be practically employed to determine the wind loads acting on the considered roofs.

In the Philippines, wind load provisions of the NSCP 2015 [6] are fully consistent with the American Society of Civil Engineers (ASCE) Standard 7-10 [20] and are largely based on findings of wind measurements and wind tunnel tests conducted over past years. Both codes specify several procedures to the determination of wind loads on main wind force-resisting systems (MWFRS) and components and cladding (C&C) elements. These basic parameters are (1) risk category (or occupancy category); (2) basic wind speed; (3) wind directionality factor; (4) exposure category; (5) topographic factors; (6) gust effect factor; (7) enclosure type; and (8) internal and external pressure coefficients. Typically, wind load is evaluated differently for C&C and MWFRS. In the proposed approach, roof panels, fasteners and purlins are all modeled as C&C elements.

According to both ASCE 7-10 and the NSCP 2015, the wind pressure (W , in N/m^2) acting on C&C elements is determined by:

$$W = q_h [GC_p - GC_{pi}] \quad (1)$$

where, q_h is the velocity pressure evaluated at the mean roof height of h ; G is the gust factor; GC_p is the external pressure coefficient; and GC_{pi} is the internal pressure coefficient. In both codes, the velocity pressure q_h (in N/m^2) is evaluated as:

$$q_h = 0.613K_zK_{zt}K_dV^2 \quad (2)$$

where, K_z is the velocity pressure exposure factor (NSCP 2015 Table 207E.3-1); K_{zt} is the topography factor; K_d is the wind directionality factor (NSCP 2015 Table 207A.6-1); V (in m/s) is the basic wind speed, i.e., the 3-second gust speed at elevation of 10 m on an open terrain (i.e., Exposure C in both ASCE 7-10 and NSCP 2015). In addition, both wind load provisions classify buildings into five occupancy/risk categories, depending upon the consequences to human life in the event of failure, and upon whether the building is designated as an essential facility.

In Eq. (1), the term GC_p (i.e., external pressure coefficient) for C&C depends on the considered zone/area of the building envelope. Particularly, the most severe wind pressures on a roof occur in the regions of flow separation at the ridge, eave and corners. Values for GC_p are provided in the codes for different types of roofs and roof angles, and as a function of an effective wind area. For the determination of GC_p , the basis is the NSCP 2015 Figures 207E.4-2A to 207E.4-7 for several types of roofs. In the same equation, the term GC_{pi} (i.e., internal pressure coefficient) depends on the building enclosure classification (i.e., open, partially enclosed or enclosed building). Specifically, internal pressures develop (1) if air blown into a space cannot freely leave that space, in which case the internal pressures are negative; or (2) if air sucked away from a space cannot be freely replaced, in which case the internal pressures are negative. Values of GC_{pi} are given in NSCP 2015 Table 207A.11-1. By aerodynamic convention, positive and negative pressures are directed, respectively, toward and away from the surface on which they act. The net pressure is the vector sum of the external and internal pressures acting on a surface. Therefore, according to Eq. (1), the most adverse combination of wind uplift pressure acting on a roof is that related to a negative value of GC_p coupled with a positive value of GC_{pi} . The obtained negative value of W indicates that the estimated (uplift) wind pressure is directed away from the considered roof surface.

In addition, the safety factor embedded in the ASCE 7-10 C&C pressure coefficients on roof surfaces was determined to be 1.25. This value was obtained from an unpublished study comparing theoretical values and wind-tunnel test data, and through extensive discussions with experts in the field on the codification of wind tunnel pressures and available damage statistics [21]. Assuming that the same level of safety is maintained in the design provisions for all building components, a factor of 0.8 is then added to the calculation of surfaces pressures represented in Eq. (1). In this manner, the reduction factor of 0.8 is used to ‘remove’ the safety factor embedded in the code provisions for load calculations. Table 1 summarizes the statistical properties of wind load coefficients adopted in this study for Eqs. (1) and (2). They are collected from some relevant studies in the field, including Amini and van de Lindt [8], Ellingwood and Tekie [22], and Lee and Rosowsky [7]. The parameters given in Table 1 are the nominal value of each RV obtained from the code (i.e., NSCP 2015), the bias (i.e., the ratio between the mean of a RV and its nominal value), and the coefficient of variation (CoV) defined as the ratio of the standard deviation to the mean of each RV.

Table 1. Summary of statistics for dead load and wind load parameters

Variable	Nominal	Bias	CoV	Distribution	Source
Dead load (D) of roof panel ^a	200 N/m ²	1.05	10%	Normal	Ellingwood et al. [23]
K_z (exposure D)	Building specific, see Table 2	0.96	14%	Normal	Ellingwood and Tekie [22] & Lee and Rosowsky [7]
K_d	0.85	1.05	14.6%	Normal	Ellingwood and Tekie [22]
GC_p (C&C)	Building specific, see Table 2	0.95	12%	Normal	Amini and van de Lindt [8] & Ellingwood and Tekie [22]
GC_{pi} (enclosed)	0.18	0.83	33%	Normal	Ellingwood and Tekie [22] & Lee and Rosowsky [7]
GC_{pi} (partially enclosed)	0.55	0.83	33%	Normal	Lee and Rosowsky [7]
K_{zt}	Deterministic (1.0)				-

2.2. Dead and Total Loads

Dead loads on roof panels, truss and roof-to-wall connections are based on the weights of the roof-panel materials and the roof system, respectively. The dead load on the various roof

components counteracts the effect of wind uplift, resulting in a stabilizing effect for the roof and contributing to increase its resistance to wind uplift. Specifically, the considered dead load (indicated as D) in this study is the self-weight of the roof sheathing panel; it is assumed constant in time, i.e., added weight due to re-roofing, if any, is not considered here. According to Ellingwood et al. [23], such a dead load can be modeled probabilistically by a normal distribution with assumed values of bias and CoV of 1.05 and 0.10, respectively (see Table 1). In particular, since the material of roof panels for the considered CH assets is LWMS, the nominal value of the dead load is taken as 200 N/m^2 . This value was obtained from local surveys/measurements carried out by the authors in the Philippines.

The total pressure on the roof is then calculated by adding the dead load D (acting in a direction opposite to the wind uplift) to the wind pressure (W). Finally, the uplift load per purlin (in N/m) is determined by multiplying the total pressure for the tributary width (i.e., spacing) of purlin.

A schematic drawing of typical (gable) roof arrangement for the considered case-study buildings is illustrated in Fig. 3.

Fig. 3. Schematic illustration of a typical (gable) roof arrangement for the considered buildings and definition of the relevant geometric variables characterizing the roofs

2.3. Roof Panel Uplift Capacity

The resistances to uplift wind loads are essentially dependent on the type of fasteners installed in the roofing system and their connections with the roof panel and purlins. For the determination of pullout and pullover capacities, the forces on the fasteners are applied parallel to the length of the fastener and perpendicular to the holding member. In the proposed approach, two types of fastener are considered, i.e., screws and nails. It is worth noting that all the quantities calculated based on the equations presented in this sub-section are in N. To make the capacity terms of dimensionally-consistent units with the demand terms (i.e., uplift load per purlin, in N/m), the determined capacity values are divided by the spacing of fasteners (see Fig. 3) in the same considered purlin.

2.3.1. Uplift Capacity for Screws

Similar to the American Iron and Steel Institute (AISI) guidelines (e.g., [24]), Section 555.4.4 of NSCP 2015 provides design/assessment methods for screw connections for cold-formed steel structural members. The provisions for pullout and pullover resistances of screw are based on Pekoz's work [25].

Specifically, in the case of pullout failure, the nominal pullout resistance per screw ($P_{n,out}$, in N) is calculated as

$$P_{n,out} = 0.85t_c d F_{u2} \quad (3)$$

where, t_c (in mm) is the lesser of the depth of penetration (t_n) and thickness of the member (i.e., purlin) not in contact with screw head (t_2); d (in mm) is the nominal screw diameter; and F_{u2} (in MPa) is the ultimate tensile strength of the member (i.e., purlin) not in contact with screw head or washer.

In the case of pullover failure, the nominal pullover resistance per screw ($P_{n,over}$, in N) is determined by

$$P_{n,over} = 1.5t_1d_wF_{u1} \quad (4)$$

where, t_1 (in mm) is the thickness of the member (i.e., roof sheet) in contact with screw head; d_w (in mm) is the larger of the diameter of the washer and the screw head; and F_{u1} (in MPa) is ultimate tensile strength of the member (i.e., roof sheet) in contact with screw head or washer.

2.3.2. Uplift Capacity for Nails

In the case of wood-type purlins (the majority in the case-study buildings, as discussed below), nails are generally used as fasteners. For this case, the National Design Specification (NDS) for Wood Construction 2017 [26] provides an empirical equation for the design nail withdrawal (pullout) capacity for single smooth shank nails used as wood-to-wood and metal-to-wood connections (P_w , in N); it is expressed as

$$P_w = K_w G^{5/2} d_n t_n \quad (5)$$

where, G is the specific gravity of the wood based on oven-dry weight; d_n (in mm) is the shank diameter of the nail; t_n (in mm) is the penetration of the nail in the member (i.e., wood purlin) holding the nail point; and K_w is an empirical constant having a value of 9.515, which is converted from the original value of 1,380 (in empirical unit) for SI unit consistency.

The determination of the pullover resistance for nails is based on the same equation used for screws, i.e., Eq. (4). In this case, d_w is assumed as the diameter of the nail head while the other parameters (i.e., t and F_{u1}) have the same definition as above.

3. Fragility Derivation and Risk Assessment

In the proposed approach, the desired performance objective is specified as the breach of the building roof due to pullout or pullover failure (as introduced in Section 2). In this case, a general limit state function (g) for roof performance can be described as:

$$g = R - Q = R - (W - D) \quad (6)$$

where, R represents the roof uplift resistance (capacity) for pullout/pullover failure modes (Section 2.3), and Q represents the total load effect (demand) due to wind-induced uplift force (W , Section 2.1, for a given basic wind speed V) and the dead load (D , Section 2.2). As mentioned in Section 2, all the terms in Eq. (6) are expressed in the unit of N/m (i.e., resistance/load per purlin). It is also assumed that the uplift resistance of the roof is limited by the assembly's weakest link, i.e., a series system is considered.

Following Eq. (6), the probability of roof uplift failure (denoted as P_f) is then defined as the probability that the undesired roof performance will occur, i.e., the probability that g is negative:

$$P_f = \Pr[g < 0] = \Pr[R - (W - D) < 0] \quad (7)$$

In the proposed approach, uncertainties in the uplift resistance R and total load on the roof Q are modeled probabilistically, as also discussed above. In particular, Table 1 shows all the RVs for the determination of Q . All the parameters included in Eqs. (3)-(5) are also considered as RVs to estimate R , the statistics of them mainly depend on the on-site data collection for the case-study buildings; details are provided in the following section.

In this study, plain Monte Carlo sampling (e.g., [27]) is used to estimate P_f of Eq. (7), i.e., the probability of roof uplift failure. The estimated probability of failure ($\overline{P_f}$) can be computed as:

$$\overline{P_f} = \frac{n_f}{N} \quad (8)$$

where, n_f is the number of samples for which the roof uplift failure occurs (i.e., $g < 0$), and N is the total number of simulations. In order for the estimate of $\overline{P_f}$ to be sufficiently reliable (i.e., $\overline{P_f} \rightarrow P_f$), a sensitivity analysis (not reported here for brevity) was performed to identify the number of samples required to generate ‘stable’ $\overline{P_f}$ values. Results of the analysis indicated that 1,000,000 simulations allow one to estimate a probability of roof uplift failure of 10^{-4} (i.e., 0.01%) with a CoV of 10% or less. Therefore, this sample size is deemed appropriate for the derivation of fragility relationships as proposed in this study. The conceptual framework of WARP² is shown in Fig. 4.

Fig. 4. Conceptual framework for the proposed wind fragility assessment, WARP² (NSCP is the National Structural Code of the Philippines)

Fragility relationships for the two considered failure modes are derived using the considered limit states and the methodology described above (i.e., Eqs. (6)-(8)). In particular, fragility is defined as the probability of pullout/pullover failure given increasing basic wind speed values ($V = x$) of interest; this is denoted as $P[\text{Pullout failure} | V = x]$ and $P[\text{Pullover failure} | V = x]$, respectively.

Finally, the annual roof uplift risk, denoted as $P[\text{Roof uplift failure}]$ where ‘roof uplift failure’ refers to either pullout or pullover failure, can be determined, once both the fragility results ($P[\text{Roof uplift failure} | V = x]$, i.e., $P[\text{Pullout failure} | V = x]$ or $P[\text{Pullover failure} | V = x]$) and the wind hazard in terms of probability density function (PDF) of annual (maximum) basic wind speeds (denoted as $f_V(x)$) are known. The latter may be derived from annual extreme wind speed data obtained from local weather stations of interest, using various fitting models (e.g., [28]). The annual roof uplift risk can be conceptually computed by integrating a given fragility relationship (for a given asset of interest) with the PDF of V at the site of interest. Adapted to this specific study, such integral can be converted into a discrete summation for numerical calculation purposes:

$$P[\text{Roof uplift failure}] = \sum_{\text{all } x_i} P[\text{Roof uplift failure} | V = x_i] P[V = x_i] \quad (9)$$

where, $P[V = x_i]$ is the annual probability that a specific basic wind speed ($V = x_i$) will occur, and it can be generally computed from cumulative distribution function (CDF) of annual basic wind speed V . Note that V in this study, as defined in Eq. (2), should be the 3-second gust speed at elevation of 10 m on an open terrain.

4. Illustrative Application: Priority CH Assets in Bohol, the Philippines

This section illustrates an application of the proposed assessment framework to eight priority CH assets in Bohol, the Philippines. These structures are presented in Table 2 with building IDs. Fig. 5 shows the geographic location of each considered asset, using the same building IDs. The selection of this area was based on a preliminary desk assessment that considered typology, concentration and exposure of the existing heritage. In particular, Bohol includes a wide range of historic buildings that are representative of the country’s CH building stock.

Table 2. Data used as input in the fragility/risk assessment for the eight selected CH assets in Bohol, the Philippines

Building ID	Roof					Purlin			Fastener				Wind load	
	Roof type	Roof area [m ²]	Mean roof height [m]	Roof slope [°]	Roof panel material ^a	Purlin material	Wood purlin density ^b [kN/m ³]	Purlin spacing [m]	Fastener type	Fastener spacing [m]	Fastener head diameter [mm]	Fastener penetration [mm]	Exposure factor, K_z	External pressure coefficient (GC_p)
1. Panglao Church	Gable	362.1	11.56	24	LWMS	Steel ^c	-	1.5*	Screw	0.15	13	70*	1.21	-2.6
2. Cortes Convent	Monoslope	476.7	6.75	6	LWMS	Wood	6.1	2.5	Nail	0.30*	13*	70*	1.10	-2.6
3. Cortes Church	Gable	485.3	10.00	29	LWMS	Wood	7.2	0.9	Nail	0.24	10	76	1.18	-1.2
4. Loboc Church	Gable	638.5	14.85	30	LWMS	Wood	7.2	1.5	Nail	0.30*	10	16	1.27	-1.2
5. Albuquerque Convent	Gable	567.9	10.69	13	LWMS	Wood	7.2	0.8	Nail	0.30	20	20	1.19	-2.6
6. Albuquerque Church	Gable	226.6	11.45	29	LWMS	Wood	6.1	0.6	Nail	0.50	7.5	50	1.21	-1.2
7. Dmiao Church	Gable	106.8	13.31	30	LWMS	Wood	7.2	0.8	Nail	0.32	15	130	1.24	-1.2
8. Dmiao Convent	Gable	1119.1	11.65	15	LWMS	Wood	7.2	3.5	Nail	0.30	15	120	1.21	-2.51

^a All the roof panels in this study are lightweight metal sheets (LWMSs). The self-weight is given in Table 1, the thickness and tensile strength are given in Table 3.

^b The specific gravity of the wood purlin (G in Eq. (5)) can be calculated directly from this parameter (divided by the gravitational acceleration).

^c C-purlin (LC – 150×50×18×3) is used. The thickness and tensile strength are given in Table 3.

* These values/parameters are assumed based on engineering judgement.

Fig. 5. Surveyed CH assets in Bohol, Philippines; map data by © 2020 Google.

4.1. Assumptions in the Proposed Analysis

As discussed in Section 2.1, several parameters should be determined for the computation of wind loading and, subsequently, of the (total) uplift demand on the considered roofs. The considerations/assumptions made for the analysis of the considered priority CH assets in Bohol are discussed here:

- According to the occupancy/risk categories codified in NSCP 2015 Table 103-1, all the considered assets are classified as Category III (Special Occupancy Structures), based on their function as heritage/religious facilities.
- As per NSCP 2015, the basic design wind speed (V^* , for Occupancy Category III) corresponds to a 15% probability of exceedance in 50 years (i.e., a 300-year mean recurrence interval). According to the wind map provided in NSCP 2015, V^* -value for the case-study location (Bohol) is determined in the range of 260 and 280 kph (72.2 – 77.8 m/s); a median value of 270 kph (75 m/s) is selected in this study for illustrative purposes.

- The wind directionality factor K_d is assumed as 0.85 for C&C elements [29], to account, in an approximate way, for the reduced probability of maximum wind coming from any direction and the reduced probability of maximum pressure coefficient occurring for any wind direction (i.e., non-coincidence of building orientation and unfavorable wind direction).
- The considered CH sites are categorized as Exposure D (to account for the surrounding ground surface roughness). This assumption is based on the fact that the considered CH assets are located along the tropical cyclone (typhoon) coastlines (see Fig. 5); therefore, the exposure factor K_z can be determined from Table 207E.3-1 in NSCP 2015, to account for the terrain exposure condition (shown in Table 2). In addition, it is also assumed that each CH asset lies on a flat ground surface, thus no topographic effect factors are considered, i.e., $K_{zt} = 1$ is assumed for all buildings. The statistical properties of these three factors (K_z , K_{zt} and K_d) in Eq. (2) are summarized in Table 1 above.
- In terms of the enclosure classification, it was initially assumed that all the considered structures were enclosed; a common assumption for buildings. However, several surveyed CH assets feature large windows and entrances and may be considered as partially enclosed. To investigate this aspect, the fragility relationships for the considered CH assets are derived (and compared) for different enclosure types. Accordingly, the code-specific (nominal) values of the internal pressure coefficient (GC_{pi}) for enclosed and partially enclosed structures and their statistics are summarized in Table 1.
- To determine the external pressure coefficient (GC_p), the roof type/geometry data collected on site (shown in Table 2) are adopted. In order to characterize the most

unfavorable loading case (i.e., fasteners and panels of the roof are subjected to the highest wind uplift forces, usually at the roof corners), it is assumed that the acting wind forces are the same for the whole area of the roof panel, which means that the same (largest) external pressure is considered for the entire panel for each building. Following this consideration, the GC_p values (based on the NSCP 2015 Figures 207E.4-2A to 207E.4-7) of the eight CH assets are tabulated in Table 2,

- The turbulence effects on the building are not considered in the determination of the wind pressure. The possible torsional effects on the building due to wind forces are also negligible and, thus, not considered in the proposed analysis.

4.2. Data Collection

The proposed approach relies on field surveying (through an *ad-hoc* form developed by the authors, similar to the one proposed in [30]), experimental testing in the literature, and data from local suppliers to obtain the required input information/data to characterize the (total) uplift load on the roof components as well as the pullover and pullout resistances against uplift. The proposed data collection form includes four main categories: (1) building/roof-related parameters; (2) purlin-related parameters; (3) fastener-related parameters; and (4) wind loading-related parameters. In particular, the structural-specific values required for (4) (for the calculation of Eqs. (1)-(2)) are determined as per NSCP 2015, given the information collected in category (1), as discussed above. Output of the data collection exercise for the eight case-study CH assets located in Bohol are summarized in Table 2.

It is worth noting that the reliability of the data collected on site is generally low, with some variables not recorded in the surveyed sheets due to difficulties in accessing roofs, extensive damage for some assets (due to the 2013 events), or to the fact that only non-destructive investigations were allowed for these valuable CH assets. In these cases, the input data for each

building to be used in the fragility assessment is based on engineering judgment (indicated by
 asterisks in Table 2), for example by considering, for each missing value of a given variable,
 the (weighted) average of the recorded values for the same variable from the other (similar)
 buildings. The use of new technologies for data collection, such as photogrammetry, drones,
 thermal and omnidirectional cameras, could facilitate and significantly improve the data-
 collection process during the fieldwork. For instance, drones have been extensively used for
 façade and roof inspections of CH assets in the Philippines (e.g., [30]).

As show in Table 2, all the considered CH buildings feature LWMS as roof-panel material; the
 use of such lightweight roofs (with self-weight of 200 N/m^2 , see Table 1) makes the considered
 roof panels extremely susceptible to wind hazards. Post-event surveys for some of the
 considered assets also highlighted that some roofs consisted of portion of ‘sandwich’-type roof
 panels, not always visible/identifiable through building-interior inspections and hence not
 explicitly modelled in the analysis presented here. This is a conservative assumption as the
 presence of sandwich-type roof panels does not affect the pullout capacity (which depends on
 fastener properties and wood/steel purlin properties) and may only slightly affect/increase the
 pullover capacity (which largely depends on the panel thickness and tensile strength of the roof
 panel). In addition, most of the assessed buildings (seven out of eight cases) feature wood-type
 purlins and nails were used as metal-to-wood connections. Therefore, Eqs. (4) and (5) can be
 used to determine the roof pullover and pullout resistances, respectively. From the available
 surveying data, it appears that only one CH asset in Bohol (i.e., Panglao Church, building ID
 = 1) uses steel-type purlin (i.e., C-purlin of LC – $150 \times 50 \times 18 \times 3$, based on the on-site survey)
 with screw fasteners. This case represents an illustrative application of Eq. (3) to estimate the
 probability of roof pullout failure.

Table 3 summarizes the statistics (i.e., mean and CoV) as well as the statistical model for each
 variable in Eqs. (3)-(5) used in the simulation procedure. These statistics are a combination of

on-site data, local experimental testing data, code-specific data and data assumed based on engineering judgment, as further discussed in Alvarez et al. [32]. It should be noted that d_w (effective pullover diameter) in Eq. (4) is replaced by d (screw diameter) in this study, since no washer of fastener was found in the on-site survey.

Table 3. Summary of statistics for roof uplift resistance variables

Category	Variable	Mean	CoV	Distribution
Fastener	Fastener head diameter (d , in mm)	Nominal (measured on-site) value or assumed	5%	Normal
	Fastener shank diameter (d_n , in mm)	20% of fastener head diameter (d)	2.5%	Normal
	Nail penetration (t_n , in mm)	Nominal (measured on-site) value or assumed	25%	Normal
Roof panel (LWMS)	Roof panel thickness (t_1 , in mm)	0.79	10%	Normal
	Tensile strength of roof panel (F_{u1} , in MPa)	147	35%	Normal
Purlin	Specific gravity of wood purlin (G)	Nominal (measured on-site) value or assumed	25%	Normal
	Steel C-Purlin thickness (t_2 , in mm)	3 (LC – 150×50×18×3)	2.5%	Normal
	Tensile strength of Steel C-Purlin (F_{u2} , in MPa)	215 ^a	10%	Lognormal

^a According to the Philippine National Standards (PNS) 127, Class 1 steels, the specified yield stress, F_y is taken as 170 MPa [31]. The specified tensile strength is 15% higher than F_y according to the local experimental tests. A bias value of 1.1 is also assumed to determine the mean tensile strength.

4.3. Results and Discussion

4.3.1. Fragility Results for Case-Study CH Assets

In this sub-section, fragility relationships for the considered CH assets are derived through the proposed WARP² approach. Fragility relationships up to a basic wind speed value of 120 m/s (with an increment of 1 m/s) are calculated for roof pullout and pullover failures.

Two enclosure types (scenarios) for these buildings are assumed in the analysis: (1) partially enclosed; and (2) enclosed. Plain Monte Carlo simulation is used for generating 1,000,000 realizations of every RV affecting the demand (Q) characterization and the roof uplift resistances (R) for both failure modes.

Figs. 6(a) and 6(b) show the developed fragility relationships for roof pullout and pullover failures for the Loboc Church (building ID = 4), considering the building as partially enclosed and enclosed, respectively. No parametric model (e.g., lognormal) is fit to the empirical fragility values. The smoothness of the derived fragility relationships is mainly attributed to (1) the use of 1,000,000 simulations to compute each fragility value, making the estimates relatively stable; (2) the use of 121 specific basic wind values to construct each fragility relationship, making the resulting curves appearing as continuous.

Fig. 6. Fragility relationships for pullout (solid lines) and pullover (dashed lines) failures for the Loboc Church (building ID = 4), considering the asset as (a) partially enclosed; and (b) enclosed; the vertical red dashed lines indicate the design basic wind speed level for Bohol ($V^* = 75$ m/s)

As discussed in Section 4.1, the design wind speed (V^*) for Bohol is equal to 75 m/s. By entering the two fragility relationships (for each enclosure scenario) with this basic wind speed value (i.e., vertical red dashed lines in Fig. 6), the conditional probability of pullout failure of the first roof fastener and the probability of pullover failure of the first roof panel can be computed. Table 4 summarizes the results of such fragility assessment in terms of both probabilities of pullout failure and pullover failure for the eight case-study CH assets, assuming the two enclosure types. As indicated in Fig. 6 and Table 4, the fragility estimates assuming partially enclosed structures are higher than those considering the same structures as enclosed,

for both pullout and pullover failures. This is expected because of the much higher internal pressures (refer to the different nominal values of GC_{pi} in Table 1) when large openings appear in the structures. Hence, in the following fragility results discussions, the fragility relationships derived for the partially enclosed scenario are chosen as the ‘baseline’ conservative scenarios.

Table 4. Results of fragility assessment for the considered CH assets subjected to the design basic wind speed ($V^* = 75$ m/s)

Building ID ^a	Probability of roof failure (partially enclosed structures)		Probability of roof failure (enclosed structure)	
	Pullout failure	Pullover failure	Pullout failure	Pullover failure
1	0.004%	32.8%	0.004%	25.1%
2	100.0%	98.7%	100.0%	97.3%
3	68.3%	11.0%	56.1%	6.0%
4	100.0%	64.9%	100.0%	45.1%
5	99.8%	12.5%	99.6%	9.2%
6	99.8%	50.1%	99.5%	31.6%
7	30.9%	6.8%	22.0%	3.8%
8	99.7%	99.8%	99.4%	99.5%

^a Building IDs are same to Table 2.

To better interpret the roof uplift failure probabilities summarized in Table 4, and given that the considered assets show similar geometric features (e.g., in terms of total roof areas), a simple (but meaningful) risk scale is also proposed for comparative purposes. According to this scale, all the probability estimates (given the design wind speed V^* stipulated in the NSCP 2015) are arbitrarily categorized in four risk groups or levels, namely low (green), medium (yellow), high (orange), and very high (red), respectively. These four risk groups are defined by three threshold values for the various probability estimates, i.e., 25%, 50%, and 75%. Figs. 7(a) and 7(b) illustrate the application of the proposed scale to the results of fragility assessment of the eight selected CH assets considered as partially enclosed.

Fig. 7. Application of the proposed risk scale to the eight selected CH assets (partially enclosed) in Bohol, the Philippines: conditional probabilities of (a) pullout failure; and (b) pullover failure, at the design basic wind speed level ($V^* = 75$ m/s)

Visual inspection of Fig. 7 reveals that the probability of pullover failure (given $V^* = 75$ m/s) for the considered CH assets is generally low-to-medium (i.e., 5/8 cases with probability of failure no more than 50%), while two cases exhibit very-high ($> 98\%$) probability of pullover failure. In contrast, the probability of pullout failure is generally high-to-very high (with five cases showing almost 100% probability of failure). However, the probability of pullout failure (given the design wind speed of 75 m/s) for the Panglao Church (building ID = 1) is almost zero; this may be attributed to the use of steel-type purlin and screws. The better mechanical properties of steel purlin (i.e., in terms of the ultimate tensile strength) and the use of screws can contribute to the significantly higher pullout resistance for this type of fasteners against wind uplift. According to Table 4 and Fig. 7, the roofs of the selected CH assets in Bohol are more susceptible to pullout failure, rather than pullover failure, with the exception of the Panglao Church, where the steel purlins are used.

It is worth noting that the definition of the threshold values defined above is mainly a subjective choice to develop a prioritization scheme, based for example on retrofitting strategy and resource availability [30,33]. A more advanced prioritization scheme, for instance based on an explicit loss calculation, may also be implemented. This would require the derivation of

system-level (i.e., building-level) fragility relationships to be combined with damage-to-loss functions and repair/replacement costs, converting damage estimates to loss estimates. As discussed in [30], this is a challenging task because of the difficulty in putting a value on the non-market nature of many CH assets, for instance in terms of (intangible) cultural significance of the asset. This depends on a wide range of criteria including social, cultural and economic attributes together with physical/architectural ones (e.g., [34]).

4.3.2. Sensitivity Analysis for Fragility Relationships

Various factors related to the data collection process may affect the demand and/or capacity estimation in the proposed WARP² approach, and consequently, the results of the proposed fragility analyses.

The data summarized in Table 2 and Table 3 was collected from rapid visual surveying, recent experimental testing, and data provided by local suppliers, as described above. Given that (1) most of the case-study CH assets were built (approximately) several decades ago, without following any modern building code; (2) destructive inspection/testing was avoided during the on-site survey due to the importance/value of these CH assets; (3) available experimental tests were mainly conducted for alternative and new-made specimens rather than those used for CH assets; and (4) the information collected from the local suppliers were based on higher-quality materials fabricated recently; some of the structure-specific parameters and their statistical properties adopted in this study (Table 3) may be considered as representative of an average, moderate construction quality. However, their validity in reflecting the relatively poor construction quality of existing CH assets is questionable.

A possible scenario representing relatively poorer quality characteristics for the surveyed assets is also considered, as follows: (1) the mean values in Table 3 are reduced of 5% to represent a lower quality of materials/roof members during their fabrication/constructions; and (2) the CoV

values in Table 3 are increased of 25% to reflect a higher variability in connection resistances (e.g., [9]). Figs. 8(a) and 8(b) depict the fragility relationships representing moderate/poor connection qualities, for roof pullout and pullover failures of the Cortes Church (building ID = 3), respectively. A 11% decrease in median and a 21% increase in dispersion are computed in Fig. 8(a) for the roof pullout fragility estimation. Similarly, a decrease of 7.5% in median fragility and an increase of 20% in dispersion are found for the roof pullover fragility estimation (Fig. 8(b)). These findings are representative of the other considered case-study buildings, and as expected, they indicate that more roof damage and higher uncertainty in the fragility prediction can be simulated due to the relatively poorer construction quality assumed here. These results highlight that the fragility estimates are highly sensitive to the input information for the determination of the roof uplift capacity; more accurate data and resources for its collection are essentially needed to improve the fragility assessment, especially for historical buildings such as those CH assets considered in this study.

Fig. 8. Fragility relationships for (a) pullout failure; and (b) pullover failure, considering different construction qualities for the Cortes Church (building ID = 3; partially enclosed)

4.3.3. Prediction of Annual Roof Uplift Risk

According to Eq. (9), the prediction of the annual risk of pullout and pullover failure depends on both the fragility curves for roof uplift failure (as discussed in the Section 4.3.1 above) and

the annual frequency of the basic wind speed value of interest (i.e., $P[V = x_i]$). To obtain the latter, annual extreme wind data taken from Tagbilaran City Station, Bohol (station ID = 644) of PAGASA are used. The wind data (has been converted to 3-second gust) was measured by anemometers at a height of 10 meters above the ground and covers the period from 1961 to 2000. The generalized extreme value (GEV) model is used to fit the obtained wind data. The GEV model is generally considered a suitable statistical representation for extreme wind speeds; the point process or the peaks-over-threshold method can also be used but they may introduce a bias in the determination of ‘peak’ values [35]. The PDF, i.e., $f_V(x)$, for the GEV distribution with location parameter μ , scale parameter σ (> 0), and shape parameter ξ is expressed as:

$$f_V(x) = \left(\frac{1}{\sigma}\right) \exp\left(-\left(1 + \xi \frac{(x - \mu)}{\sigma}\right)^{-1/\xi}\right) \left(1 + \xi \frac{(x - \mu)}{\sigma}\right)^{-1-1/\xi} \quad (10)$$

Using the maximum likelihood estimation method, these three parameters for Bohol are estimated as $\mu = 16.57$, $\sigma = 5.64$, and $\xi = 0.0386$, respectively. Further details of the estimation method and process can be found in Garciano et al. [35] and Garciano and Koike [36]. It should be noted that the GEV model with its parameters given here is consistent with the wind hazard analysis derived for the NSCP 2010 [37]. The V -value for Bohol, considering a 300-year return period, estimated from this model is about 53 m/s, which matches well the basic wind speed codified in the NSCP 2010 (i.e., 200 kph / 55.6 m/s), but is far below the V -value recommended by the NSCP 2015 ($V^* = 75$ m/s, used in fragility assessment above). Because the extreme wind speed data used to develop the wind hazard map in the NSCP 2015 is unpublished, it was decided to adopt the GEV model discussed here, for illustrative purpose, to compute and compare the annual roof uplift risk among the case-study CH assets. Results in terms of annual risk of first-roof-fastener pullout and first-roof-panel pullover failures are reported in Table 5 for all the eight priority CH assets considered in this study.

Values in parentheses indicate the mean return period of roof uplift failure in years (i.e., the reciprocal of the corresponding values of the annual uplift failure risk).

Table 5. Annual risk of pullout and pullover failures for the considered CH assets (partially enclosed)

Building ID ^a	Pullout failure ^b	Pullover failure ^b
1	0.003% (30,148)	0.41% (242)
2	23.98% (4)	2.07% (48)
3	1.15% (87)	0.28% (355)
4	24.77% (4)	0.55% (182)
5	12.98% (8)	0.32% (316)
6	10.09% (10)	0.44% (226)
7	0.35% (289)	0.27% (373)
8	11.39% (9)	3.45% (29)

^a Building IDs are same to Table 2.

^b Values in parentheses indicate return period in years.

Fig. 9 shows 1,000,000 random samples of the uplift demands (Q) and the pullout/pullover resistance (R) per purlin in the form of the corresponding histograms for the Dmiao Convent (building ID = 8). For the sake of comparison, the histograms have been normalized to show the probability of occurrence of each bin value with a uniform bin width. In this Monte Carlo simulation, the basic wind speed is also modeled as an RV with the statistical distribution described by the GEV model of Eq. (10) above. The overlapping area identified in Fig. 9 is proportional to the annual probability of pullout/pullover failure and hence to the annual roof uplift risk, as defined above.

The values of the annual risk of roof uplift failure achieved through the simulation-based approach are almost identical to those in Table 5 computed through numerical integration, and only negligible differences are found due to the uncertainty within the Monte Carlo sampling process.

Fig. 9. Simulation results for (a) pullout capacity; and (b) pullover capacity versus uplift demand (considering the GEV model to represent basic wind speed V) for the Dmiao Convent (building ID = 8; partially enclosed).

According to Table 5, the annual risk of pullout failure for the majority of the considered CH assets (7/8, i.e., all the CH assets having wood purlins) is significantly high, indicating unacceptably low mean return periods of roof uplift failure. These return periods, ranging from four to 289 years, with six of them lower than 100 years, are well below the mean recurrence interval of the design basic wind speed (i.e., 300 years). This inspection illustrates that the CH roof panels may be at extremely high risk of pullout failure when they are subjected to wind speed even well-below the considered design values in the modern standards/building codes; therefore, some retrofitting strategy is urgently required. The return periods of pullover failure for these CH assets are generally larger than 180 years (with two cases owing return periods even larger than 300 years). Additionally, a very low annual risk (associated with a significantly high return period) of pullout failure is computed for the Panglao Church (building ID = 1). This is again, due to the use of steel as purlin material. These findings are consistent to those summarized in fragility assessment above, even though different wind hazard levels are considered. It is also interesting pointing out that considerably higher annual risk of pullout failure for the Dmiao Convent (building ID = 8) than the annual risk of pullover failure is observed in Table 5, although the conditional probabilities of these two failure modes at $V^* = 75$

m/s are almost identical. This is due to the fact that the annual risk calculation integrates the probabilities of failure at all the basic wind speed levels of interest, each weighted by its probability of occurrence, rather than a single failure probability given the design wind speed (as shown in Table 4).

5. Concluding Remarks

This paper introduced a simulation-based approach, named WARP², to assess pullout and pullover fragility and risk of roof panels subjected to extreme wind uplift loading. The proposed procedure addresses the statistical determination of wind-induced loads and uplift capacities with respect to the pullout of roof fasteners and pullover of roof panels. Specifically, the proposed procedure relies on the field surveying of various roofing systems for case-study CH assets, and the estimation of the fragility and risk of those roofing systems through comparison between uplift capacities and wind-induced demands, using a probabilistic approach and structural reliability concepts. For illustrative purposes, the wind fragility and associated risk of eight selected priority CH assets located in Bohol, the Philippines, have been estimated through the proposed WARP² approach.

The application results indicate that: (1) the conditional probability of pullout failure (given $V^* = 75\text{m/s}$, i.e., the design wind speed for Bohol) for the considered CH assets is very high, while the conditional probability of pullover failure is generally lower; and, similarly, (2) the pullout risk for the selected CH assets is extremely high (with two exceptions), while the roof pullover risk is generally acceptable (with two exceptions), as indicated by the estimated return period of uplift failure. In particular, in the case of roof panels supported by wood-type purlins, the roof performance is controlled by the fastener pullout. In contrast, for the only CH roof consisting of steel-type purlins, the probability of pullout failure is almost zero as significantly high pullout resistance is provided by steel purlins and screws featuring higher tensile strength.

A sensitivity analysis was also performed for two essential factors - i.e., enclosure classification and construction quality - that may affect the fragility/risk outputs, confirming that the quality of the collected data is a critical ingredient of the proposed assessment framework.

The proposed approach can be effectively used to identify the highest-risk assets and prioritize more detailed data collection campaigns and structural assessment procedures (e.g., properly accounting for load redistribution and fastener failure progression, considering other structural components/connections), and ultimately to plan further repair/strengthening measures. Ongoing and future developments will aim to improve the quantification of the wind vulnerability through the definition of more advanced numerical models which also consider degradation effects and climate change impact.

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