WARP^2: Wind Assessment of Roofs to Pullout & Pullover for Priority Cultural Heritage Structures in the Philippines

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ABSTRACT: Cultural tourism is one of the priority sectors by which the Government of the Philippines aims to foster inclusive and sustainable socio-economic development, due to its potential for job creation. Filipino cultural heritage (CH) assets are particularly vulnerable to natural hazards (e.g., earthquake ground shaking, strong wind, and flooding) due to their age and type of construction. In particular, non-engineered CH roofs have been recognized as the most vulnerable component in the building envelope due to typhoon-induced wind uplift. Consequently, they may cause large amount of economic loss and disruption to CH assets. This paper introduces a simulation-based approach for non-engineered CH roof fragility derivation, i.e., to assess the probability of different levels of damage experienced by CH roofs over a range of wind hazard intensities. In this approach, two limit states are considered, corresponding to roof-panel pullout and pullover failure modes. An illustrative application of the proposed procedure is finally presented with reference to 17 priority CH buildings across the Philippines.

1. INTRODUCTION

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 in the country's sustainable development. In the recent *Germanwatch Climate Risk Index 2018*, the Philippines ranked 5th among the most affected countries by disasters (1997-2016), with about 85% of 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 earthquake and then super Typhoon Yolanda (international codename: Haiyan) caused several casualties, major damage, and a significant increase in poverty levels in the affected areas. Specifically, many cultural heritage (CH) assets (e.g., provincial capitols, heritage churches and other buildings of historical value) were seriously damaged, with some even totally destroyed, in 14 provinces in the Visayas.

In recognition of the country's vulnerability to natural disasters, 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 disaster risk reduction and management investments in priority sectors, including CH.

CH assets are 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.

This study proposes a simulation-based procedure named WARP^2 (*Wind Assessment of Roofs to Pullout & Pullover*) for the derivation of fragility functions for non-engineered CH roofs considering roof-panel pullout and pullover failure modes. In fact, post-event surveys in the Philippines and around the world reveals that most economic loss in high wind-hazard areas are related to the breach of the building envelope. The breach of a building envelope typically includes roof panel uplift, roof-to-wall connection failure, roof system damage, and rupture of window and door glasses due to excessive pressure or missile impact. With the roof heavily damaged or removed, walls may become unstable without sufficient lateral support and can collapse.

Similarly to the well-known HAZUS-MH Hurricane Model (e.g., Vickery et al. 2006a; b), the physical damage to a building subjected to winds is modeled here using an engineering-based demand and capacity approach, where once the wind-induced loads acting on a building are computed, the resulting physical damage is estimated in terms of failure of building envelope components.

An illustrative application of the proposed procedure is presented with reference to 17 priority CH buildings across the Philippines.

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), are built using wood frames and galvanized iron sheets. During strong 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-panel 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 due to shear while the fastener is still intact within the holding members.

In the proposed wind fragility assessment for roof panels, uncertainties in structural component characteristics (e.g., materials, geometries, models), i.e., those affecting roof-panel uplift capacity, are modeled probabilistically (details of statistical models for the variables used in this study are provided in Section 3.2). Given the preliminary nature of this study, the gravity and wind load effects (i.e., demand) are considered deterministic and modeled according to the National Structural Code of the Philippines (NSCP) 2015 (ASEP 2015). However, further developments of the proposed procedure include the full probabilistic modelling of wind loading (and any other load).

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.

In the Philippines, wind load provisions of the NSCP 2015 are fully consistent with the American Society of Civil Engineers (ASCE) Standard 7-10 (ASCE 2010) and are largely based on findings of wind measurements and wind tunnel tests conducted over past years. Both codes cover the following steps applicable to the determination of wind loads on main wind force-resisting systems (MWFRS) and components and cladding (C&C): definition of 1) risk category; 2) basic wind speeds; 3) enclosure type; 4) exposure category; and 5) topographic factors.

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

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

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

$$q_h = 0.613 K_h K_{zt} K_d V^2 \quad (2)$$

where, K_h is the exposure factor (NSCP 2015 Table 207E.3-1) accounting for terrain exposure condition; K_{zt} is the topography factor; K_d is the wind directionality factor (NSCP 2015 Table 207A.6-1) accounting, 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); V (in m/s) is the basic wind speed, i.e., the 3-sec gust speed at elevation of 10m 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 hazard 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 zone/area of the building envelope considered. 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 (i.e., positive internal pressures); or 2) if air sucked away from a space cannot be freely replaced (i.e., negative internal pressures). 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 critical combination of wind uplift pressure acting on a roof is relating 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 roof surface considered.

Moreover, the safety factor embedded in the ASCE 7-10 C&C pressure coefficients on roof surfaces was determined by experimentation to be 1.25. This number was obtained from an unpublished study comparing theoretical values and wind tunnel data, and through extensive discussions with experts in the field, about the codification of wind tunnel pressures and available damage statistics (Cope 2004). Assuming that the same level of safety is maintained in the design provisions for all building components, a factor of 0.8 is 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, this accounting for the modeling uncertainty.

2.2. Dead and total loads

The dead loads on roof panels, truss and roof-to-wall connection are based on the weights of roof panel material and the roof system, respectively. The dead load on the roof is available to counteract the effect of wind uplift, thus contributing to stabilize the roof and increase its resistance. The dead load is assumed to remain constant in time (added weight due to re-roofing, if any, is not considered here). The total pressure on roof is then calculated by adding the roof panel load (dead load) to the wind pressure. Finally, the uplift load per purlin (in N/m) is determined by multiplying the total pressure for the tributary width of purlin. A schematic drawing of typical (gable) roof arrangement for the considered case-study buildings is illustrated in Figure 1.

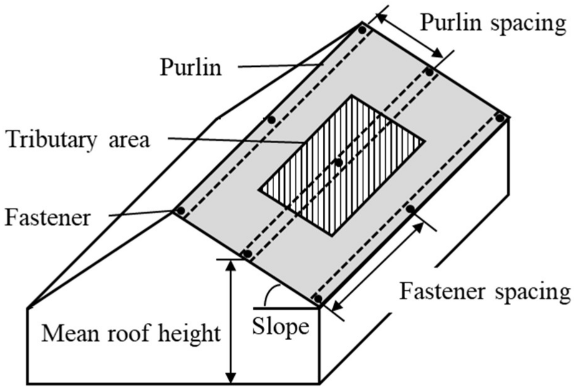

Figure 1: Schematic illustration of typical (gable) roof arrangement for considered buildings

2.3. Roof panel uplift resistance

The resistances to uplift forces 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 resistances, the forces on the fasteners are applied parallel to the length of fastener and perpendicular to the holding member. In the proposed approach, two types of fastener are considered, i.e., screw and nail. It is worth noting that all the values calculated based on the following equations in this sub-section are in N. To make the resistance terms be of dimensionally-consistent units with the demand terms (i.e., uplift load per purlin, in N/m), the determined resistances should be divided by the spacing of fasteners in the same purlin considered.

2.3.1. Uplift resistance for screws

Similar to the American Iron and Steel Institute (AISI) guidelines (e.g., AISI 2009), Section 555.4.4 of NSCP 2015 provides design methods for screw connections for cold-formed steel structural members. The provisions for pure shear and pure tension forces are based on Pekoz's work (1990).

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

$$P_{n,out} = 0.85 t_c d F_{u2} \tag{3}$$

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

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

$$P_{n,over} = 1.5 t d_w F_{u1} \tag{4}$$

where, t (in mm) is the thickness of the member 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 in contact with screw head or washer.

2.3.2. Uplift resistance for nails

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

$$P_w = K_w G^{5/2} d_s P \tag{5}$$

where, G is the specific gravity of the wood based on oven-dry weight; d_s (in mm) is the shank diameter of the nail; P (in mm) is the penetration of the nail in the member 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 1380 (in empirical unit) for SI unit consistency.

The determination of pullover resistance per nail is considered to use the same equation, i.e., Eq. (4), for screw. Alternatively, d_w is assumed as the diameter of the nail head in this case.

3. FRAGILITY 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:

$$g(R,Q) = R - Q = R - (W - D) \tag{6}$$

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) 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). The probability of failure (denoted as P_f) is equal to the probability that the undesired roof performance will occur, i.e., the probability that g is non-positive:

$$P_f = \Pr\left[g(R,Q) < 0\right] = \Pr\left[R - (W - D) < 0\right] \tag{7}$$

Fragility functions, defined as the probability of pullout/pullover failure given basic wind speeds (i.e., V), are also developed as a function of wind speed using the considered limit states and the methodology described above.

Given the quality of the available data, it is assumed that the uplift capacity of the roof is limited by the assembly's weakest link, i.e., a series system is considered. In particular, the considered limit states correspond to 1) pullout failure of the first fastener (i.e., screw or nail); and 2) pullover failure of the first panel. In fact, once failure of a single fastener occurs, the load is distributed to the surrounding fasteners causing failure to propagate thought the panel; similarly, there is a strong correlation between panel removal and subsequent damage to the building structure. The conceptual framework of the wind fragility assessment used here is shown in Figure 2. 1,000 plain Monte Carlo simulations are performed to estimate the probability of roof uplift failure as a function of V.

3.1. Assumptions in the proposed analysis

The following assumptions are made in the fragility analysis:

- In determining the wind pressure, turbulence effects on the building are not considered.
- Possible torsional effects on the building due to wind forces are negligible and, thus, not considered in the analysis.
- The considered area is categorized as Exposure B and each building lies on a flat ground surface, thus no topographic effect factors are considered, i.e., K_{zt} = 1 is assumed for all buildings.
- The acting wind forces are the same for the whole area of the roof panel, which means the same (largest) external pressure is considered for the entire panel.
- Fasteners and panels at the roof corners are investigated as they are subjected to the highest wind uplift forces (i.e., the local pressure coefficients are the highest of any point on the roof surfaces).

3.2. Data collection

The proposed approach relies on field surveying (through an *ad-hoc* form developed by the authors), experimental testing in the literature, and data from local suppliers to obtain the required input information for the proposed procedure. This includes all the variables affecting the uplift load on the roof as well as those affecting the pullover and pullout resistances against uplift (as discussed in

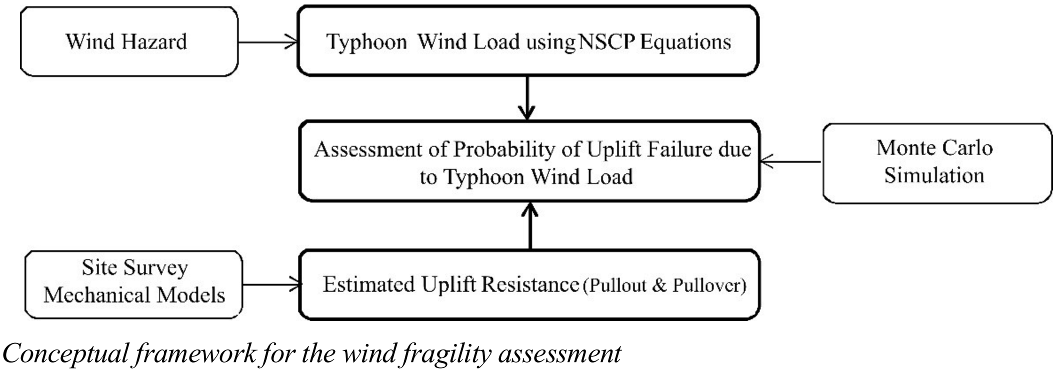

Figure 2: Conceptual framework for the wind fragility assessment

Section 2). The specific details regarding roof characteristics are used to build fragility functions for different, increasing, values of basic wind speed (V). Pullout and pullover probabilities in the case of wind speeds corresponding to those derived from the hazard analysis (as stipulated in the NSCP 2015) are also calculated.

The developed data collection form includes the following information:

- Type of roof;
- Roof dimensions (i.e., area);
- Mean roof height;
- Slope/Pitch of roof;
- Numbers of purlins;
- Material used for purlin (wood or steel);
- Type of fastener used (screw or nail);
- Number of fasteners per purlin bay;
- Fasteners characteristics (e.g., diameter and penetration);
- Type of material for roof panels.

Output of the data collection exercise for 17 case-study buildings located across the Philippines are summarized in ARS Progetti et al. (2016). It is worth noting that the reliability of the data collected on site is generally low, with several variables not recorded in the surveyed sheets. In these cases, the input data for each building to be used in the fragility assessment is based on engineering judgment, for example by considering, for each missing value of a given variable, the average of the recorded values for the same variable from the other buildings.

Table 1 summarizes the statistics (i.e., mean and coefficient of variation, 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 and/or data assumed based on engineering judgment, as discussed in Alvarez et al. (2013).

According to the occupancy/risk categories codified in NSCP 2015 Table 103-1, all the considered buildings can be classified as Category III (Special Occupancy Structures). Consequently, the basic wind speeds for the case-study locations as determined form the wind map provided in NSCP 2015 are: 250 kph (69.4 m/s) for Manila (MA), 260 kph (72.2 m/s) for Cebu (CE), and 270 kph (75 m/s) for Bohol (BO), respectively.

4. RESULTS AND DISCUSSION

As discussed in Section 3, plain Monte Carlo simulation is used for generating 1,000 realizations of every variable (i.e., oven-dried specific gravity of wood, nail/screw diameter/penetration, panel thickness, tensile strength of the panel) affecting the roof uplift resistance for both failure modes.

As an example, Figure 3 shows the generated 1,000 random samples of the roof uplift resistances (pullout and pullover) per purlin in the form of corresponding histograms for Dmiao Church (Bohol). On the other hand, the uplift load per purlin assuming a basic wind speed of 270 kph (75 m/s), i.e., the design wind speed for Bohol stipulated in the NSCP 2015, is also shown in Figure 3 as a vertical red dashed line.

Table 1: Statistics for roof uplift resistance variables

Variable	*Mean*	*Coefficient of variation*	*Statistical model*
Specific gravity of wood (G)	*Nominal (measured on-site) value or assumed*	*25%*	*Normal*
Fastener head diameter (d, in mm)	*Nominal (measured on-site) value or assumed*	*5%*	*Normal*
Nail shank diameter (d_s, in mm)	*20% of d*	*2.5%*	*Normal*
Nail penetration (P, in mm)	*Nominal (measured on-site) value or assumed*	*25%*	*Normal*
Panel thickness (t, in mm)	*0.79 (light weight metal sheet); 15 (clay tiles)*	*10%*	*Normal*
Ultimate tensile strength (F_u, in MPa)	*147 (light weight metal sheet); 1 (clay tiles)*	*35%*	*Normal*

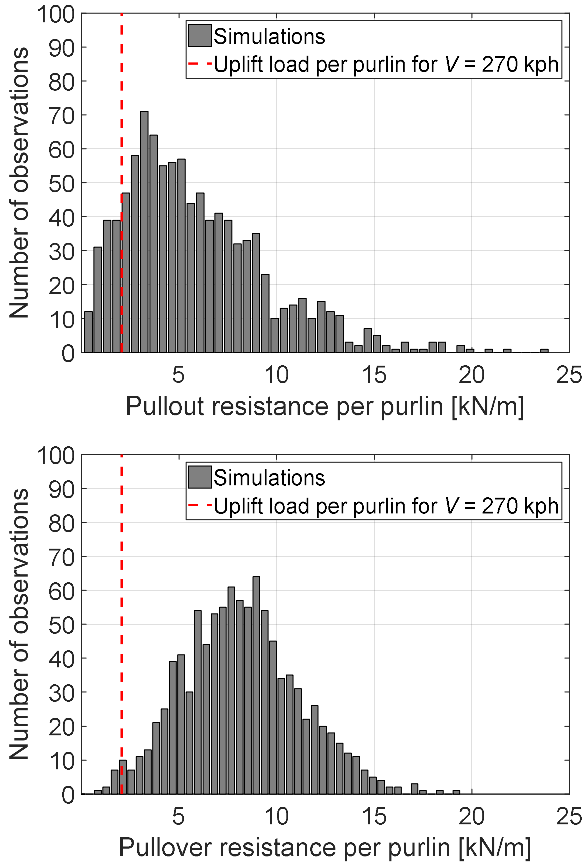

Figure 3: Simulation results of pullout and pullover resistance per purlin for Dmiao Church (Bohol)

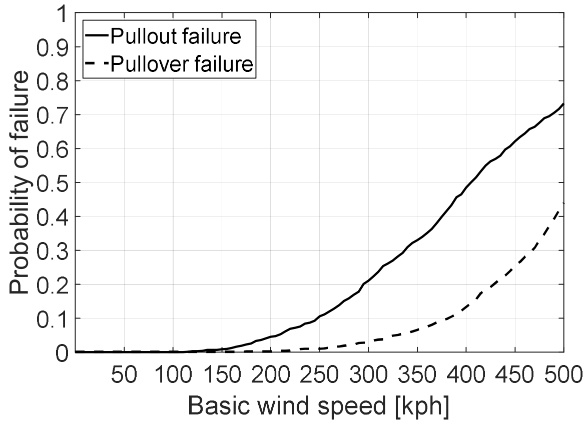

Figure 4: Fragility functions of pullout and pullover failure for Dmiao Church (Bohol)

An example of developed fragility functions for Dmiao Church is show in Figure 4. By entering the two fragility functions in Figure 4 with same basic wind speed (i.e., 270 kph), the probability of pullout failure of fasteners and the probability of pullover failure of panel for the specific example show same values as found by looking at the two histograms in Figure 3.

Table 2 below summarizes the results of the fragility assessment in terms of both probabilities of pullout failure and pullover failure for all case-study buildings.

Table 2: Results of fragility assessment for selected buildings

Building ID	*Probability of pullout failure*	*Probability of pullout failure*
1. *MA – Saint Agustin Convent*	*100%*	*99.5%*
2. *MA – Saint Agustin Tower*	*99.8%*	*73.1%*
3. *MA – Saint Agustin Church*	*100%*	*99.9%*
4. *BO – Maribojoc Punta Cruz Tower*	*94.5%*	*14.9%*
5. *BO – Panglao Tower*	*67.2%*	*6.6%*
6. *BO – Panglao Church*	*17.2%*	*3.4%*
7. *BO – Cortes Convent*	*99.5%*	*49.4%*
8. *BO – Corte Tower*	*98.4%*	*25.2%*
9. *BO – Cortes Church*	*39.8%*	*2.1%*
10. *BO – Loboc Mortuary Chapel*	*78.4%*	*3.3%*
11. *BO – Loboc Tower*	*77.1%*	*4.2%*
12. *BO – Loboc Church*	*100%*	*16.4%*
13. *BO – Alburquerque Convent*	*97.9%*	*1.7%*
14. *BO – Alburquerque Church*	*98.8%*	*10.9%*
15. *BO – Dmiao Church*	*13.5%*	*2.2%*
16. *BO – Dmiao Convent*	*97.7%*	*96.9%*
17. *CE -Dalaguete Church*	*16.1%*	*96.5%*

5. CONCLUSIONS

This paper proposes a numerical approach (named “WARP^2”) to assess pullout and pullover fragility of roof panel due to extreme wind loading (e.g., typhoon). The proposed procedure addresses the (typhoon) wind load determination, the surveying of various roofing systems for case-study buildings, and the analysis of the fragility of those roofing systems through comparison between uplift resistances and wind loads, using a probabilistic approach and structural reliability concepts. As an application, the wind fragility of 17 selected cultural heritage buildings located in

Manila, Bohol, and Cebu in the Philippines are assessed based on the data collection activities and the proposed approach.

Results of the analysis show that the probability of pullover failure for the considered buildings is generally low-to-medium (with few exceptions) while the probability of pullout failure is generally high-to-very high (with few exceptions). In particular, 3/17 buildings are characterized by fragility values less than 25% for their roofs, 4/17 buildings are characterized by fragility values between 25% and 50% for their roofs, 6/17 buildings are characterized by fragility values ranging from 50% to 75% for their roofs, and 4/17 buildings are characterized by fragility values larger than 75% for their roofs. However, the last four cases are also those featured by the lowest reliability of the input data used in the assessment and several assumptions have been made in the calculation. Results of the analysis are based on conservative assumptions and strongly depend on the poor quantity and quality of the available data. An improved data collection is highly recommended before recommending and/or considering any mitigation strategy for wind risk reduction. Also, recent studies suggest lower design wind speed for the considered location, thus the actual risk value are expected to be lower than the computed one. Thus, an improved wind hazard assessment for the consideration location is highly desirable.

6. ACKNOWLEDGEMENT

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