Finite element analysis of low-rise non-engineered timber residential buildings in Dominica under hurricane loads

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1 INTRODUCTION

Hurricanes may remain the most dominant and destructive natural peril negatively impacting the economies of Caribbean states. Non-engineered private-sector homes are considered subject to the largest risk, with most of their structural data difficult to access for vulnerability assessment.

He et al. [5] note the utility of fragility/vulnerability curves accounting for the critical structural responses quantifying building performance. This requires a complete, validated finite element (FE) building model, preferably stochastic rather than deterministic. Non-engineered wooden buildings are expected to be one of the most fragile typologies to wind; however, previous studies which have built full numerical wind analysis models of low-rise timber buildings have been found to exclude elevated buildings and were mainly concerned with investigating the load paths and load sharing through the structure.

This paper presents an FE methodology developed for the analytical wind vulnerability assessment of low-rise, non-engineered elevated timber buildings with lightweight hip roofs. The objectives of this study include (1) visualising the structural response of an elevated building exposed to hurricane-level wind loads, (2) incorporating three-dimensional system effects, especially but not limited to the hip roof frame assembly, (3) accounting for as-built construction details for the building typology of interest as observed in the field. The ability of the structure to sustain loads beyond the first element failure is also investigated. It is noted by Pan et al. [7] that due to their substantial redundancies, timber structures can usually bear more loads past the initiation of damage at their most vulnerable point. Therefore, the limit states considered must go beyond the first element failure for damage prediction and mitigation.

The proposed FE methodology intends to fill a gap in the existing knowledge base regarding the construction materials, methods, and structural performance of typical non-engineered residential building typologies in the Caribbean and specifically Dominica, by facilitating the study of the initiation and propagation of their structural failure under hurricane loads. Three other common Dominican residential building typologies of varying wind fragility were also covered by this project, including timber and concrete masonry buildings with light hip/hip & valley roofs, concrete masonry buildings with light hip/hip & valley roofs, and concrete masonry buildings with heavy flat roofs. The present paper concentrates on the all timber light hip roof typology because, while is not the most common surveyed typology, it has been identified as the most vulnerable. Moreover, its construction is similar to the upper storey of timber and concrete masonry buildings with light hip/hip & valley roofs, which constitutes a larger class of buildings, across the whole Caribbean, and present similar level of vulnerability. Therefore, results of this analysis can be extrapolated to this second class.

2 METHOD

Given the non-engineered nature of the structures being investigated, the methodology proposed draws on diverse sources of information to identify existing construction practices and building typologies, to reliably underpin a tool for wind vulnerability assessment applicable to any region of interest.

2.1 Parameter Selection

To eventually model the building numerically, measurable quantities, including the sizing and spacing of structural and non-structural elements and connections, as well as their material properties, are required. Some of these parameters can also be found in fragility and vulnerability literature as critical in the definition of particular building typologies. Therefore, the first stage of this methodology is concerned with identifying the construction parameters required. Most of the construction parameters

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identified describe features of the roof construction, which is expected to be of concern due to the roof experiencing considerable uplift forces during hurricane events. The walls, which experience positive pressure or suction in different regions depending on the prevailing wind direction, also have their construction details captured. By comparison, the floor and foundation construction details are less apparent in building typology descriptions for fragility/vulnerability functions.

2.2 Damage Data Analysis

The second stage uses available empirical damage data, which could be beneficial in terms of reference and validation of the study. A desk study exercise is undertaken to locate construction, damage, and/or claim data to identify the typical residential building typologies across the region of interest. In the case of Dominica, a building damage assessment was led by the United Nations Development Program (UNDP) after Hurricane Maria (2017). If the geotagged locations of these buildings are coupled with the available wind speed estimates of a relevant hurricane event, an empirical fragility assessment can be carried out, indicating the relative performance of the typologies against wind hazard. A total of 262 houses were identified (on pillars or shallow foundations), which fell into the timber typology of interest and were most likely to have been solely impacted by wind hazard during the hurricane. Approximately 47% of these homes suffered extensive roof damage, whereas 35% were reported to have suffered extensive wall damage.

2.3 Field Survey

Primary data collection is at the core of this methodology and takes place in the third stage. To address the relative data scarcity on Caribbean residential construction, a detailed field survey was undertaken in Dominica by University College London (UCL). The construction data obtained in the second stage is compared with the required parameters identified in the first stage, to design and undertake a structural survey, which targets the outstanding information. Structural inspections were carried out for three timber frame buildings across the island, and their structural details were used to build a representative single-storey index building. The building is elevated on reinforced concrete pillars, with anchor bolts connecting the timber frame to the foundations. Considering these are non-engineered structures, the compliance of the as-built construction with existing codes/standards is also examined.

2.4 Structural Model

Finally, based on the information collected in the field, a prototype numerical model can be built for the typology of interest. In the likelihood that wind tunnel data is not available to compute the wind loading for the model (as in this study), the structural response of the building against hurricane loads is assessed with wind loading calculated according to ASCE 7-22 Main Wind Force Resisting System (MWFRS) Case 1, which accounts for elevated buildings. The loading is applied incrementally to allow for failure checks against the capacities of the structural and non-structural connections.

3 NUMERICAL ANALYSIS

A 3D frame model is built for the structure using commercial FE software SAP2000 [2], as shown in Figure 1. The joist-to-foundation connections and the joist-to-joist joints are modelled as pinned. As numerical models have yet to be developed in the literature for elevated buildings, the pinned connections for joists are assumed to be similar to the modelling of pinned wall stud-to-wall and sill plate connections by Martin et al. [6]. Timber wall studs are modelled, allowing bending about their major axis. The rafter-to-rafter and rafter-to-wall plate connections are modelled as pinned. As buildings belonging to this typology were encountered in the field both with and without bracing elements at the building corners, this is also incorporated as a sensitivity check. The bracing elements, when present, are modelled, allowing bending about their minor axis, as their placement differs by 90 degrees compared to the wall studs.

During Hurricane Maria (2017), the maximum 3-s gust wind speed over land was recorded as 165mph or 74m/s [4]. In the FE model, distributed wind forces are applied to the frame elements in an incrementally increasing manner from zero to 74m/s, expecting to cause systemic failure. The model is initially assumed to have zero damage. Two basic failure modes for the nailed members are explored: pull-out and slip failure of the nails. The nail capacities are calculated using the following equations in the U.S. National Design Specification [1] and Wood Handbook [3]. Wind loads are calculated and applied to the frame, assuming wind approaches perpendicular to the front elevation. This is done to analyse the windward roof-to-wall junction with a large overhang over the porch. Worst-case single

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values are taken for external pressure coefficients, whilst positive and negative internal pressure coefficients are considered. The analysis is ultimately stopped for a model once enough elements have failed and been removed at the same wind speed value, causing a drop in base shear (V_b) greater or equal to 10%.

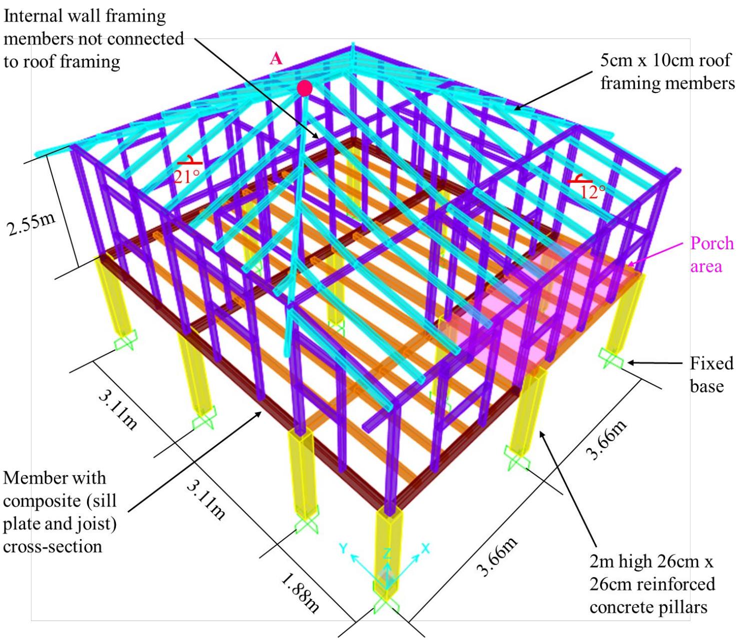

Figure 1: 3D frame model of elevated timber building with hip roof in SAP2000

The cladding, including timber floorboards, wall sheathing, roof sheathing and metal sheeting of the house are not included in the FE model; however, their connection capacities are assessed to determine whether they will fail at wind speeds lower or greater than the frame elements.

4 RESULTS

At each wind speed value, as loading is applied to the structure, horizontal and vertical displacements are extracted at Point A, the highest structural elevation, where the ridge board meets a rear hip rafter. Figure 2 shows that the presence of bracing, as expected, stiffens the structure.

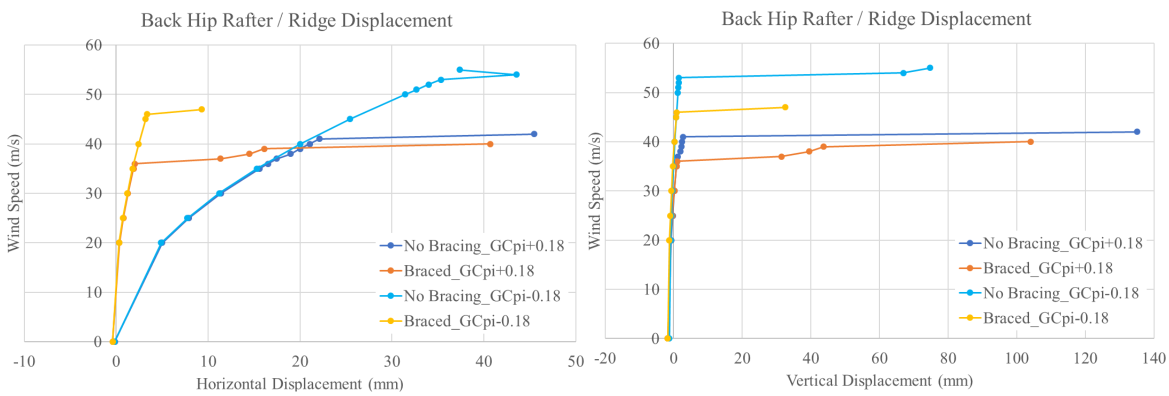

Figure 2: Wind speed vs. horizontal and vertical displacement curves extracted at the Back Hip Rafter / Ridge (Point A) with either positive or negative internal pressure coefficient, GC_{pi}

Positive internal pressures work in concert with the external pressures applied to the roof and floor elements and the wall frame elements at the back and side elevations. Therefore, element failures are expected to occur at lower wind speeds with positive internal pressures. Whether positive or negative internal pressures would develop will depend on the arrangement of the openings across the various elevations. In the field, openings are incorporated on all building elevations for this building typology. As such, no matter the wind direction considered, openings will always predominate in the suction regions, making the development of negative internal pressures the most likely.

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5 CONCLUSIONS

The following conclusions are formed based on research conducted in support of this project and, therefore, pertain only to the specific load applications previously described. It is noted that this differs from a typical building design, which must consider all wind attack angles from 0°-360°. The current study is focused on developing a numerical procedure to investigate the structural performance under given loading conditions.

1. Floor frame elements fail exclusively in slip, whilst wall studs failed exclusively in pull-out, indicating that they are affected significantly more by the uplift forces on the roof rather than the lateral wind loads. The roof frame elements exhibit both pull-out and slip failure.
2. The failure mechanism shown to recur for this building typology under positive internal pressures is the slip failure of the joists under the centre of the building. This study's findings show the importance of considering and modelling the floor and foundation construction details for elevated buildings, as the building envelope may be breached through these elements.
3. The failure mechanism shown to recur for this building typology under negative internal pressures is the rafter-to-ridge pull-out and rafter-to-wall plate slip failure of the front left-hand hip rafter at the connections, which leads to the failure of the attached main and jack rafters due to excessive displacements under wind loading. At this stage, the building envelope would be significantly breached at the roof and front elevation, highlighting further the crucial role of the roof frame connections in the building’s response to wind loading.
4. By assessing the UDLs applied to the frame elements and comparing these values to the capacities of the cladding and purlin connections, it is found that these capacities were not exceeded before the frame elements’ failures. The majority of the details of the structural elements adopted for the prototype model, as assumed from the field surveys, fall below the minimum requirements specified by Dominica’s Housing Standards [8].
5. The inclusion of bracing elements at building corners is shown to slightly reduce the wind speed at which the onset of failure occurs. This indicates that the addition of bracing alone as a possible retrofit measure is not beneficial to improving the building's response against wind loading without also improving the condition and/or connections of the structural frame elements.

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