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Vulnerability model of an Australian high-set house subjected to cyclonic wind loading

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Abstract. This paper assesses the damage to high-set rectangular-plan houses with low-pitch gable roofs (built in the 1960 and 70s in the northern parts of Australia) to wind speeds experienced in tropical cyclones. The study estimates the likely failure mode and percentage of failure for a representative proportion of houses with increasing wind speed. Structural reliability concepts are used to determine the levels of damage. The wind load and the component connection strengths are treated as random variables with log-normal distributions. These variables are derived from experiments, structural analysis, damage investigations and experience. This study also incorporates progressive failures and considers the inter-dependency between the structural components in the house, when estimating the types and percentages of the overall failures in the population of these houses. The progressively increasing percentage of houses being subjected to high internal pressures resulting from damage to the envelope is considered. Results from this study also compare favourably with levels of damage and related modes of failure for high-set houses observed in post-cyclone damage surveys.

Keywords: high-set house; vulnerability; wind load; wind damage; tropical cyclone; probabilistic model; component strength; component failure.

1. Introduction

Townships typically comprise a wide range of house types, with differences in size, shape, window size, cladding, roof shape, age and methods of construction. The resilience of a house to wind loading is dependent on these features and the strength of their components and connections. Types of components and their strengths vary between the house types and even within each type, because of variations in materials and construction practices. Houses also have varying degrees of exposure to wind, depending on the climate, approach terrain and topography at the site. Furthermore, the level of expected wind damage is dependent on the wind speed and nature of the windstorm. An assessment of the resistance to wind damage of houses requires knowledge of wind load, building type and material and structural form. Probability concepts can be used to analyze these variables, and estimate the risk of component failures during a cyclone.

Basic structural analysis techniques cannot be employed to calculate load effects (i.e. bending

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moments, shear forces etc.) for the frames of these houses, because of the load-sharing between components and their complex connections. An understanding of the inter-dependency between components can only be gained by testing prototype houses such as those conducted by the Cyclone Testing Station (CTS). Findings from such full-scale house tests and post cyclone damage surveys have shown that the predominant modes of wind-induced failures are associated with the load capacity (i.e. strength) of the connections being exceeded (Reardon 1996). The CTS has compiled a housing component-strength database (which includes data from inspections of the existing housing stock) for assessing the likely failure load and failure mode of houses located in cyclonic regions of Australia. These failures focus on the chain of connections from the roof cladding fixings via the wall tie-down to the base of the structure or the ground.

Pham, *et al.* (1983), Holmes (1985), Melchers (1985), Leicester, *et al.* (1985) and Pham (1985) introduced reliability concepts when developing limit states based structural design standards in Australia. They used probabilistic models of dead, live and wind loads and strength of the structural components to derive load combination formulae and define the level of safety. Statistical parameters were used to account for the uncertainty and variability associated with loads and component strength. Recent studies by Unanwa, *et al.* (2000), Ellingwood, *et al.* (2004), Pinelli, *et al.* (2005) and Li and Ellingwood (2006) have assessed the vulnerability of residential construction in the US, to wind loading using similar methods.

This paper analyses the vulnerability of high-set, timber framed houses of rectangular plan, with fibre cement sheet exterior wall cladding, and metal roof cladding on a low to flat pitch gable end roof, using probabilistic methods. Furthermore, this study also incorporates progressive failure with

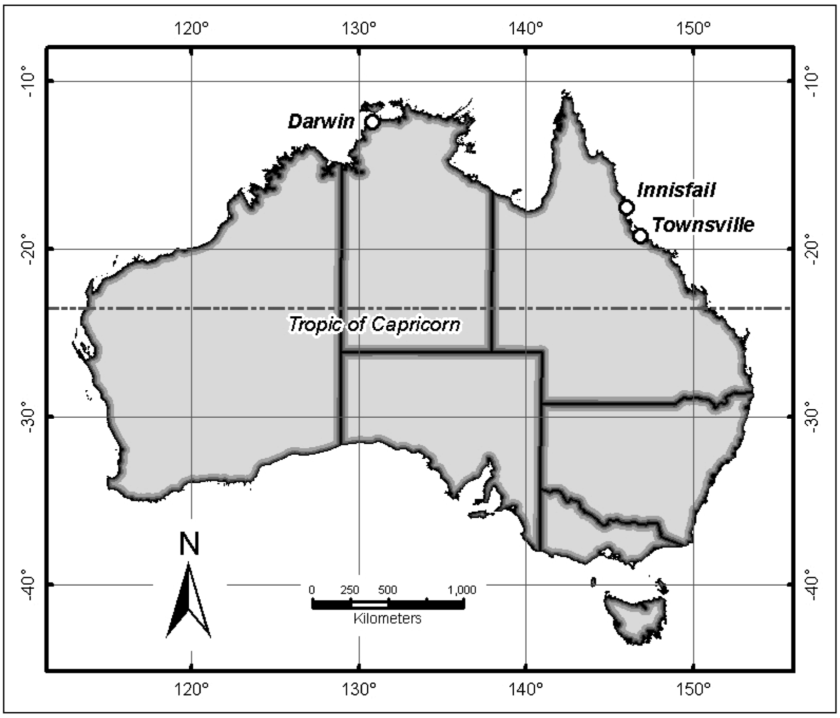

Fig. 1 Locations of Darwin, Townsville and Innisfail in Northern Australia

increasing wind speed in estimating levels of damage to a population of these houses. This house type was a common form of construction from the early 1960s to the mid 1970s in the northern parts of Australia, that sustained cyclone damage in both Townsville (Cyclone 'Althea' 1971) and Darwin (Cyclone 'Tracy' 1974) shown in Fig. 1. Post-disaster survey data of damage sustained by these houses in cyclones 'Althea' and 'Tracy' is used (JCU, 1972 and Walker 1975) for verifying damage estimates presented in this paper.

2. Design loads, component strength and probability of failure

Criteria adopted in structural design standards used in Australia, are related to a specified limit state, such as the ultimate limit state of component or structural failure. The basic framework for probability based, limit state design is provided by reliability theory (Ellingwood, *et al.* 1982). In this approach, the loads and resistances are taken as random variables and the required statistical information is assumed to be available. AS/NZS 1170.0 (Standards Australia 2002) provides calibrated combinations of factored, permanent (dead), imposed (live) and wind actions (loads) to be applied on structural components and checked against their factored resistances.

This paper analyses wind action effects resulting from external and internal pressures on components of high-set houses. Ultimate limit state wind loads are significantly larger than the other (i.e. dead) loads considered. Data on loads and component strengths are required in order to calculate their risk of failure or its vulnerability. The information required is the probability distributions of load and strength variables, and estimates of their mean and standard deviation or coefficient of variation (COV). The mean and COV of component strength variables should be representative of those actually found in these houses. Generally, for these applications, there is some data obtained from controlled experiments but little actual in-situ data is available, and hence the probability distributions and statistical parameters must be estimated from physical reasoning and experience.

Wind loads for the design of cladding and fixings on such buildings can be calculated from pressures derived from nominal shape factors or pressure coefficients, provided in AS/NZS 1170.2 (Standards Australia 2002). The design pressures are calculated from Eq. (1a), where ρ is the density of air, V_h is the 3s-peak design gust wind speed at mid-roof height and C_{fig} is the aerodynamic shape factor. Quasi-steady, external pressure coefficients $C_{p,e}$ and internal pressure coefficients $C_{p,i}$ combined with factors for area-averaging K_a, surface-combinations K_c and local-pressure effects, K_l are used to determine C_{fig} values for external and internal pressures as shown in Eqs. (1b) and (1c). External and internal design pressures acting over the tributary area, A are combined to get the nominal, net design wind load, W_N on the component.

$$p_{design} = 0.5\rho V_h^2 C_{fig} \tag{1a}$$

$$\text{External } C_{fig} = C_{p,e}.(K_a \times K_c \times K_l \times K_p) \tag{1b}$$

$$\text{Internal } C_{fig} = C_{p,i} \cdot K_c \tag{1c}$$

The regional 3s-peak gust wind speed at 10 m elevation in terrain category 2 (open country), V_R for a specified return-period (R yrs), detailed in AS/NZS 1170.2 (Standards Australia 2002) was developed from statistically analysing long-term wind data measured at meteorological stations located in various regions. V_R is modified by wind direction, terrain/height, shielding and topography multipliers M_d, $M_{z,cat}$, M_s and M_t respectively in Eq. (2), to calculate V_h.

$$V_h = V_R M_d (M_{z,cat} M_s M_t) \tag{2}$$

A measure of the resilience of a component to wind loading is estimated by the probability of failure of the component as a result of its strength being exceeded by the wind load. This is calculated by comparing the wind load effect W with corresponding resistance R (in the same units as W). The wind load effect W and resistance R are represented by random variables with probability density functions $f_W(W)$ and $f_R(R)$, with known means μ_W and μ_R and COVs. The expected values are given by the means, and the variability represented by the spread of the distributions, or the COVs. Failure occurs when the wind load effect exceeds the resistance of the component. Hence, the aim of design is to ensure that $R > W$ throughout the life of the component. Therefore, the reliability can be measured in terms of the probability, $P\,[R > W]$. A typical dead load value (which is very small compared to the wind load) is combined with the wind load in assessing these failures, in this study.

Assuming that W and R are statistically independent, the probability of failure is given by Eq. (3).

$$p_f = \int_{-\infty}^{\infty} F_R(W) f_W(W)\, dW \tag{3}$$

where, $F_R(R)$ is the cumulative probability distribution, such that $F_R(W) = \int_{-\infty}^{W} f_R(R)\, dR$.

Therefore, the measure of vulnerability is a function of the means, the degree of dispersions of W and R, and the shapes of their probability density functions. The probability of failure can be evaluated exactly when W and R are assumed to have Gaussian (normal) or log-normal probability distributions.

3. Details of high-set house

External features of the type of high-set house constructed during the 1960s and into the mid 1970s were collected and recorded in the survey of the housing stock in Townsville, as part of the Tropical Cyclone Coastal Impacts Program (Henderson and Harper 2003). Detailed descriptions, including sizes, types and spacings of structural frame members and connections, were also surveyed in a sample of these houses and presented in that report. Descriptions of these houses built in Darwin have been taken from the Cyclone 'Tracy' damage investigation report by Walker (1975), with reference to selected external feature survey and building plans of typical housing of that era. Examples of this type of house found in the cities of Townsville and Darwin are shown in Figs. 2 and 3, respectively.

A typical house of this period is of rectangular plan, timber framed, elevated on piers about 2 m high, with external walls clad with fibre cement or timber weatherboards and internal lining of either hardboard or plasterboard. The generic framing layout used in these types of houses is shown in Fig. 4. Generally, dimensions are in the order of 12 to 14 m long and about 8 m wide. The rafters, which are skew nailed at the ridge are spaced apart at about 900 mm intervals and nailed to the wall top-plate. The roofing is metal sheeting on a relatively low or flat pitch roof. The metal cladding is screw fixed, at spacings of up to 300 mm, to timber battens spaced at nominally 900 mm apart. The majority of the inspected houses have a pitched timber roof frame typically consisting of 100×50 mm rafters, 75×50 mm collar ties on every second rafter pair, and 100×50 mm ceiling joists adjacent to the rafters. In these houses cyclone rods are present in perimeter walls

Fig. 2 High-set house – Townsville

Fig. 3 High-set house - Darwin

at about 3 m spacing. Alternatively a specific number of rods were stipulated for a house. Often the rods extended to over-battens, but the holding nuts interfered with the roofing and are often embedded in the batten, weakening it severely. The wall framing consists of 100×50 mm studs between top and bottom plates spaced at 450mm intervals with noggings at nominal mid height between the studs.

There are differences in geometry, materials and construction methods, of the houses between the cities and also the houses within each city. The differences in overall size, construction material, spacing of members, fixings, etc. have been incorporated into the study through the coefficient of variation for the resilience of the connection or the construction type. These houses were designed and constructed according to practices and methods used prior to the introduction of standards for domestic construction that have been employed in cyclonic areas since the early 1980s (Queensland Home Building Code 1981).

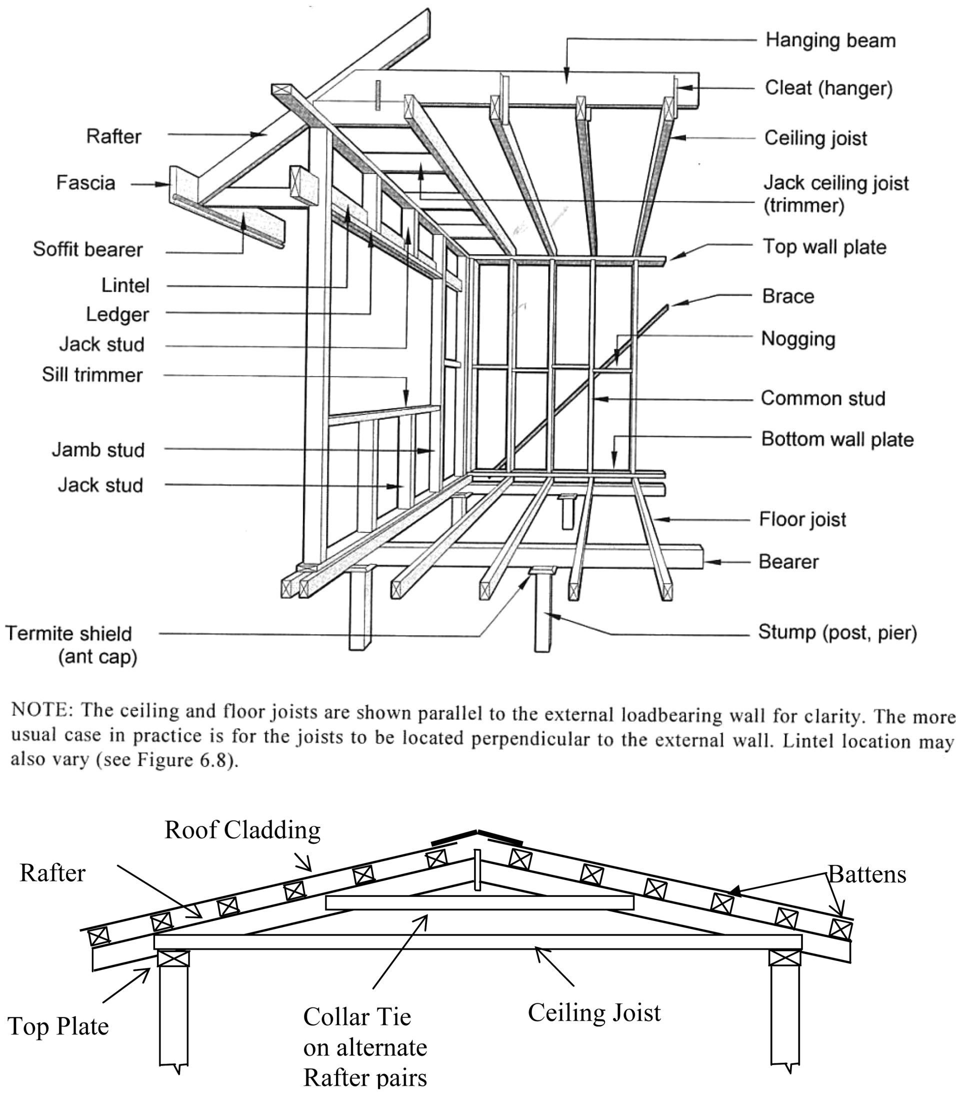

Fig. 4 Schematic diagram of timber framing elements in high-set house (AS1684.3 1999)

4. Wind loads– probabilistic model

Wind loads acting on the house components are given by the probabilistic model in Eq. (4), where V is the maximum 3s gust velocity at 10m height in terrain category 2 in 50 yrs (life of structure) and the parameter B includes all the other components of the wind load (Holmes 1985, Pham 1985).

$$W = B\,V^2 \tag{4}$$

The product of the variables shown in Eq. (5), gives parameter B.

$$B = \lambda.\ A.\ (C.\ E^2.\ D^2.\ G.\ \rho/2) \tag{5}$$

The variables within the bracket can be related to the nominal values given in AS/NZS 1170.2 (Standards Australia 2002), where:

- C is the quasi-steady pressure coefficient,
- E is a velocity height multiplier that accounts for the exposure and height,
- D is a factor for wind directionality effects,
- G is a factor that accounts for gusting effects and is related to K_a and K_l,
- ρ is the density of air,
- A is the tributary area, and
- λ is a factor to account for modelling inaccuracies and uncertainties in analysis methods.

The nominal values of these parameters are combined to give B_N, which is used to deduce the nominal design wind load W_N from Eq. (6), where V_N is the nominal design wind speed. For ultimate limit state design, this nominal design wind speed is the characteristic gust wind speed with an estimated probability of exceedence of 5% to 10% in a period of 50 yrs giving a mean return period of 500 to 1000yrs. In standards that employ a permissible stress design approach, the design wind speed has a mean return period of about 50 yrs, with a corresponding safety factor (typically 1.67) applied to the nominal load.

$$W_N = B_N\, V_N^2 \qquad (6)$$

where, $B_N = \lambda_N \,.\, A_N \,.\, (C_N \,.\, E_N^2 .\, D_N^2 . G_N \,.\, \rho_N/2)$

Combining Eqs. (4), (5) and (6) gives:

$$[W/W_N] = [B/B_N]\,[V/V_N]^2 = ([\lambda/\lambda_N]\,[A/A_N]\,[C/C_N]\,[E/E_N]^2\,[D/D_N]^2\,[G/G_N]\,[\rho/\rho_N])\,[V/V_N]^2 \qquad (7)$$

Each of the variables contained in B are assumed to have a log-normal probability distribution with estimated mean and coefficient of variation (COV), deduced from surveys and other studies (Pham, *et al*. 1983, Holmes 1985). Ellingwood and Tekie (1999) also used similar values in their analysis. Statistical data of these variables is used to obtain the mean and COV of the random variable B, which also has a log-normal probability distribution. In these estimations, values in AS/NZS 1170.2 (Standards Australia 2002) are generally considered conservative, on average. However, pressure coefficients, related factors and multipliers prescribed in standards are mainly derived from wind tunnel model studies. Such model studies have shortcomings resulting from incorrect Reynolds Number (Re), insufficient details, and incorrect turbulence scaling. These deficiencies can in some cases underestimate pressures especially on small tributary areas near windward roof edges, as evident from full-scale measurements described by Ginger (2000). Table 1 gives estimated mean and coefficients of variation (COV) of the normalized parameters contained in $[B/B_N]$. The estimation of these values is a difficult procedure requiring extensive data, especially at full-scale. Scarcity of such data requires these values to be estimated. Applying these estimated values gives the variable $[B/B_N]$ which has a log-normal distribution with mean values that range from of 0.56 on wall components supporting large tributary areas to 0.66 on roof components supporting small tributary areas, and COVs of 0.36 and 0.38 depending on the fixing considered.

Wind speed data from many sites have been analysed, and Holmes (1985) and Pham, *et al.* (1983) have applied the Fisher Tipett Type I (Gumbel) extreme value distribution to the 50 year life, peak gust wind speeds in Australia. Galambos, *et al.* (1982) have applied a similar Fisher Tippett Type I extreme value distribution in their analysis of wind loading in the US. In this study the wind load

Table 1 Mean and COVs of normalized parameters of B

Parameter	Mean	COV
λ/λ_N	1.00	0.05
A/A_N	1.00	Fixing specific (either 0.10 or 0.15)
C/C_N (roof)	0.95	0.15
C/C_N (wall)	0.85	0.15
E/E_N	0.95	0.10
D/D_N	0.90	0.10
G/G_N (Large Tributary Area)	0.90	0.10
G/G_N (Small Tributary Area)	0.95	0.10
ρ/ρ_N	1.00	0.02

acting on the components of the building are calculated for a fixed wind velocity, V, at the reference height of 10m in terrain category 2 (open terrain) and assessed against the component's strength, to estimate the probability of failure. This process is carried out for a range of wind speeds.

5. Strength of components and connections – probabilistic model

Post windstorm damage surveys (Walker 1975, Reardon, *et al.* 1999) and full-scale house testing at the CTS (Reardon 1996), have shown that the predominant modes of failures in these types of houses are associated with the wind load exceeding the capacity of the joints between the components as opposed to the bending or shear capacity of members. This study therefore focuses on the chain of connections from the roof cladding fixings down to the ground.

This analysis estimates the proportion of these high-set houses experiencing each of the possible modes of failure with increasing wind speed. The strength capacities of the connections are estimated from laboratory tests and full-scale test data (e.g. Boughton and Reardon 1984) which takes account of load sharing between components, fatigue strength and inter-dependency between components.

The variability of the strength of components and connections in these houses are associated with the differences in design, materials, construction practices and workmanship. This variability that exists even in houses that are designed to the same specifications, and other uncertainties are represented using probabilistic models. The strength of each component is given by a log-normal distribution with a specified mean and coefficient of variation, similar to the method applied by Holmes (2001).

The following likely failures with increasing wind speed are considered in this study:

- Roof cladding splitting and pulling over head of fastener
- Cladding fastener pulling out of batten
- Batten joint failing at Rafter
- Rafter joint failing at ridge
- Rafter joint failing at top plate
- Wall cladding pulling over fastener
- Wall cladding fastener pulling out of stud

• Wall racking failure
• Subfloor bracing failure

The strength of each of these components is described in terms of a mean and COV as defined in the Cyclone Testing Station Report TS614 to Geoscience Australia (Henderson and Ginger 2005). Estimates of some of the mean and coefficients of variation (COV) for connection capacities, normalized with respect to the nominal strengths, (R/R_N), are given in Table 2.

Table 2 Mean and COVs of normalized resistance (R/R_N)

Connection	Mean	COV
Cladding pulling over fastener	1.55	0.25
Fastener pulling out of batten	1.30	0.15
Batten nailed to rafter	1.69	0.30
Rafter to top plate	1.41	0.20

The vulnerability of each of the house components is assessed by estimating the percentage of houses sustaining the specified component failure, with increasing wind speed. This however does not describe the overall damage to the house type, because progressive failure with increasing wind speed depends on the load-sharing and structural inter-dependency and strengths of the components. For example, the failure of one of the connections in a house can prevent the occurrence of a dependent connection failing (i.e. batten joint failure at the rafter will negate the loss of cladding from the batten), or alternatively accelerate another failure mode (i.e. failure of rafter to top plate connections will lead to the collapse of the walls due to lack of lateral resistance, or loss of wall cladding will increase racking failure). Therefore rules governing the failure mode assumptions are required for estimating the performance of this type of house. To facilitate this, the connections considered are grouped into sub-structure classes with associated modes of failure, as detailed in Table 3.

Table 3 Failure modes

Sub-structure class		Failure mode
A	Roof envelope	Cladding pulling over fastener or Cladding fastener pulling out of batten or Batten joint failing at Rafter
B	Roof structure	Rafter joint failing at ridge or Rafter joint failing at top plate which also includes graded purlin construction.
C	Wall structure	Wall racking failure from bracing component failure or wall collapse following loss of support from failure of roof structure
D	Subfloor bracing support of piers	Subfloor bracing failure which does not consider footing failure or overturning.

A breach of the building envelope, which includes doors and windows, can lead to a significant increase in internal pressure and result in increased loads on components of the house. Depending on the geometry of the house and the structural element under consideration, the increase in load can range from 40% to over 100%. Failure of the building envelope can be caused by a broken window from wind driven debris, failure of door lock, or loss of some cladding. The flying debris damage potential in a windstorm is dependent upon the available upwind debris, its impact velocity and the resistance provided by the building envelope.

The failure of elements such as roofing, fascias, gutters, battens etc. which disengage from the house, add to the wind borne debris field, increasing the potential for damage in surrounding houses. This leads to a 'snowball' effect, as other houses downwind are subjected to larger internal pressure resulting in further damage and more debris being generated. With this increase in load associated with higher internal pressures, there is a greater probability of failure of the building elements. Therefore, a critical part for the prediction of damage levels is estimating the proportion of houses subjected to large internal pressure during a cyclone. In this study, the percentage of houses being subjected to large internal pressures as a result of a dominant opening in the envelope is taken to increase from 0% to a maximum of 90% of the house population for approach (10m height open terrain) gust wind speed of 40 m/s to 80 m/s. These estimations are based on damage surveys, the knowledge that some houses have vented eaves, and the assumption that others would have openings on both the windward and leeward walls.

6. Results

The failure of each of the sub-structure classes A, B, C and D with increasing wind speed, are

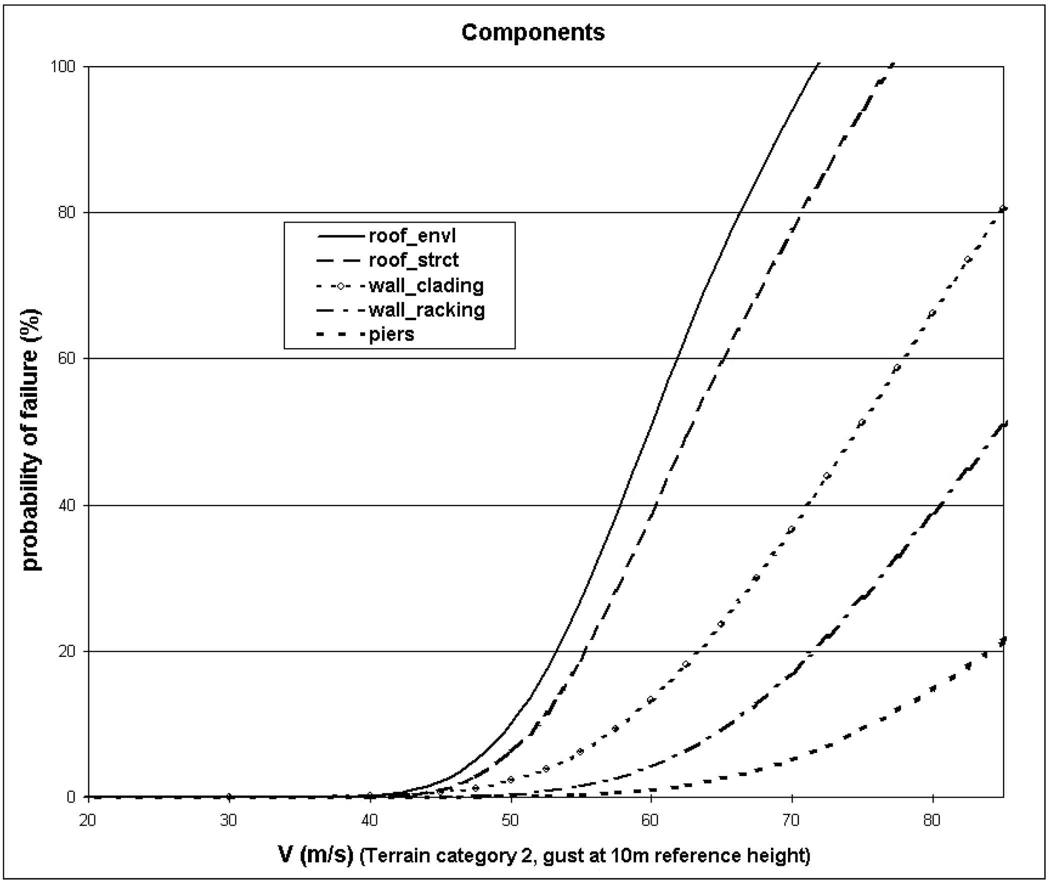

Fig. 5 Estimated probability of failure of components in the modeled houses

considered independently, in the first part of this study. Probabilities of failure with increasing approach (10m height terrain category 2) gust wind speeds, for the four structural element classes (roof envelope, roof structure, wall racking and piers) and wall cladding (which leads to wall racking failure) are shown in Fig. 5. The analysis assumes that the population of houses is located in flat, suburban approach terrain category 2.5 (defined as being between terrain category 2 and 3 as per AS/NZS 1170.2:2002). The increasing proportion of houses subjected to large internal pressure is incorporated into this analysis. This analysis is carried out to show the relative strength of each of the components of the house. Fig. 5 shows that component failures are expected at a wind speed threshold of about 42 m/s, and that the roof envelope is the most vulnerable component of the house.

Fig. 6 shows the percentage estimates of the high-set houses damaged with increasing (10 m height terrain category 2) gust wind speed, for houses located in flat approach suburban terrain (i.e. terrain category 2.5 derived per AS/NZS1170.2:2002). The levels of damage range from relatively minor (i.e. roof cladding failure) to major (collapse of walls or pier failure). In this part of the study, the inter-dependency of failure modes has been considered using the failure paths shown in the tree diagram of Fig. 7. The probability of failure of each component in the sub-structure classes are calculated at discrete wind speeds, *V*, followed by the application of rules related to the failure paths

Fig. 6 Estimated percentage houses damaged with increasing wind speed

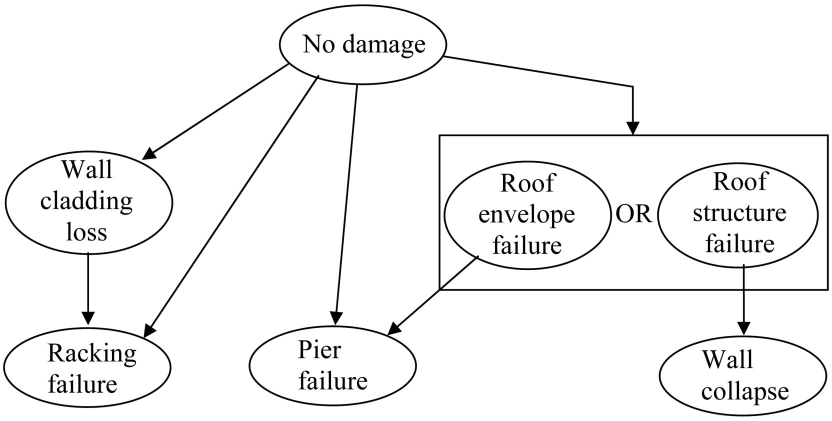

Fig. 7 Tree diagram of failure modes and propagation paths

identified in Fig. 7. Incremental failures in each of the modes are proportioned after consideration of inter-dependency such that the combined failure percentage at each wind speed increment equals the overall failure percentage of the houses. For example, for each wind speed increase, the change in percentage failures for the roof envelope and roof structure is computed, as shown in Fig. 5. The aggregate of both these failures are estimated and the increase in percentage of failure is proportioned to these modes in the same ratio as their change over the wind speed step. That is, the component (or mode) that has the higher probability of failure over the wind speed increment is given the proportionally larger percentage of failure.

The failure of wall cladding can occur from flying debris impact or the cladding disengaging from their fixings as result of the net wind loads exceeding the fixing strength. These wall cladding failures are accounted for separately in this study. The loss of wall cladding due to wind suction to houses not suffering loss of roofing or roof structure is estimated by the 'Damage including wall cladding' curve in Fig. 6. The additional damage indicated by this curve in comparison with the 'excluding wall clad envelope' curve is an estimate of the houses that have only sustained wall cladding damage.

7. Verification

The results presented in this study are verified by comparing them to the nature of component damage and the levels of damage of these types of houses observed in cyclones. However, as severe cyclones are rare events, there is limited field data available for verification purposes. Post-cyclone damage investigation reports on Cyclone 'Althea' which hit Townsville in 1971 (JCU 1972) and Cyclone 'Tracy' which caused catastrophic damage to housing in Darwin in 1974 (Walker 1975) provide some details of the type and amounts of damage sustained by this high-set house type. More recently, the post cyclone survey reports on Cyclone 'Winifred' (Reardon, *et al*. 1986) and Cyclone 'Larry' (Henderson, *et al*. 2006) which hit Innisfail in 1986 and 2006 respectively, provide field data on high-set houses that have been partially upgraded.

These damage investigation reports showed that the predominant failure of a house structure is associated with the inability of the connections to transfer the applied wind load to the next element in the tie-down chain. Failures at the connections were observed in both the older and recent (i.e.

upgraded) construction, with dislodgement of battens from rafters and rafters from top plates just two of many such examples.

Cyclone 'Althea' crossed the coast approximately 40 km north of Townsville, in December 1971. The cyclone had a large radius to maximum winds in the order of 40 km. A maximum wind speed of 54.5 m/s was measured at the Garbutt aerodrome at the standard reference height of 10 m. Damage to housing in the exposed coastal suburb of Pallerenda, was estimated by JCU (1972) at 68% (8% demolished, 22% not habitable, 38% damaged but habitable). Henderson and Harper (2004) noted that a majority of these houses would have been built in the mid to late 60's, with the assumption that construction of a reasonable proportion was similar to the house modeled in this study. The collapse of walls in a quarter of the houses that were inspected in detail was attributed to the loss of wall top plate support due to the failure of the roof structure (JCU 1972). Walker (1975) details many examples of this failure mode in Cyclone 'Tracy'. Table 4 presents the damage inferred from the report on Cyclone Althea and the estimates of damage from this study. The model was run assuming the population of houses was in terrain category 2. Estimates of damage and type of damage predicted from this study compare reasonably well with the damage in the report on Cyclone 'Althea'. However, the estimate of wall racking failures is slightly higher and the estimate of roof element damage is slightly lower than the damage survey data.

Cyclone 'Tracy' made a direct hit on Darwin in December 1974. Due to the anemometer failing during the event, the peak gust wind speeds (at 10m height in terrain category 2) were estimated in the order of 65 to 70 m/s (Walker 1975). The cyclone was very intense with a small radius to maximum winds of only 7 km. It had a slow forward movement, subjecting the buildings to high peak wind speeds over a long period of time. Walker (1975) reported extreme damage to 70% of the domestic housing (53% destroyed, 16% roof and walls damaged) with two thirds of the remaining houses suffering some roof cladding damage. In the northern suburbs, damage to the elevated houses was in the order of 95%. There was extensive loss of light gauge metal roof cladding. The cause, detailed by Morgan and Beck (1977), was low-cycle fatigue of the cladding adjacent to its fasteners. That is, cracking of the cladding allowed the cladding to pull over one fastener, leading to the progressive effect of overloading and failing of the cladding at adjacent fasteners. Table 5 presents the parameters and damage estimates determined from surveys by Leicester and Reardon (1976) of 1500 high-set houses in Darwin following Cyclone 'Tracy' and

Table 4 Damage in coastal suburb for Cyclone Althea and Present study

	Report on Cyclone Althea	Present Study
V	55 m/s	55 m/s
Total damage	68%	55% (excl wind damaged wall cladding) 58% (incl loss of wall cladding)
Wall collapse following loss of roof structure	17%	10 %
Roofing	51% is remainder and assumed to	20 %
Battens	be associated with roof cladding	11 %
Roof structure	and roof structure.	10 %
Subfloor collapse	-	0 %
Racking	0 %	4 %

compares them with the estimates of damage from this study.

Overall estimates of damage predicted by this study compare favorably with the damage survey data in Table 5. Although the model closely matches the results for structural wall failures, it over-estimates damage to the roof envelope (cladding and battens). This over-estimation could be as a result that of some of the 1500 surveyed houses having hip roofs. Hip roofs are subjected to lower uplift pressures than gable roofs and therefore have a higher resilience at the same wind speed for the similar type of pitched frame construction. The hip roofs also have increased lateral stability than similarly constructed gable roofs.

Table 6 compares damage threshold velocities for elevated houses developed by Leicester and

Table 5 Damage comparison between Cyclone 'Tracy' damage investigation and Present study

		Report on Cyclone 'Tracy' (Leicester and Reardon 1976)		Present study
	V	70 m/s		70 m/s
	Terrain category	2.5		2.5
	Total damage	96%		97% excl wall clad damage 100% incl wall clad damage
	Worst feature of damage			
1	Negligible	4%		0%
2	Missile damage	3%		-
3	Loss of half roof sheeting	13 %		26% loss of cladding
4	Loss of all roof sheeting	14 %	(total 27 %)	16% loss of battens (total 42%)
5	Loss of roof structure	11%		6%
6	Loss of half walls	30%		30%
7	Loss of all walls	22%		16%
8	Loss of half floor	1%		-
9	Loss of all floor	1%		-
10	Collapse of floor piers	1%		2%

Table 6 Damage threshold velocities

Worst feature of damage		Damage threshold velocity for elevated houses (Leicester and Reardon 1976)	Present study damage threshold velocity
2	Missile damage	38 m/s	-
3	Loss of half roof sheeting	40 m/s	45 m/s
4	Loss of all roof sheeting	46 m/s	45 m/s
5	Loss of roof structure	51 m/s	45 m/s
6	Loss of half walls	53 m/s	50 m/s
7	Loss of all walls	62 m/s	55 m/s
8	Loss of half floor	> 62 m/s	-
9	Loss of all floor	> 62 m/s	-
10	Collapse of floor piers	> 62 m/s	65 m/s

Reardon (1976) with the damage threshold velocities derived from this study. Table 6 shows that these threshold velocities which have been factored from roof height in a suburban environment to a 10 m height in open terrain, compare satisfactorily with the damage threshold velocities calculated in this study.

Fig. 8 shows an aerial view of a small portion of the damage to housing in Darwin following Cyclone 'Tracy'. The photo shows that there is a spread of damage in the high-set houses ranging from loss of roof envelope through to destruction of complete structure.

Fig. 8 Damage to housing following Cyclone 'Tracy' (Walker 1975)

Cyclones 'Winifred' and 'Larry' crossed the coast just south of Innisfail in 1986 and 2006 respectively. The maximum gust wind speeds (at 10m height in terrain category 2) were in the range of 50 to 65 m/s. Post-cyclone damage investigations after these events were presented in the CTS reports by Reardon, *et al.* (1986) and Henderson, *et al.* (2006). Walk-by surveys after Cyclone 'Larry' of more than 1000 pre-1985 houses indicated that about 15% to 20% were subjected to damage ranging from minor (i.e. loss of roof sheeting, guttering etc.) to major (i.e. destruction of house). It is interesting to note that these reports detail a shift in failure mode away from the loss of roof cladding to a predominant loss of cladding with battens still attached. The shift is due to the partial upgrading of the structure with the strengthening of the roof cladding connection but not the subsequent joints (i.e. batten to rafter connection) along the roof hold-down path. The analysis of the high-set house model with roofing screws at contemporary specifications (i.e. strengthening the roof cladding connection) changes the predominant failure mode from the cladding fixings to the batten to rafter connection and the total percentage of houses damaged drops from 39% to 26%, modeled with an approach gust wind speed (V) of 55 m/s in terrain category 2.5.

8. Conclusions

Damage to high-set rectangular plan houses with low pitch gable roofs (built in the 1960s and early 70s in the northern parts of Australia) from wind speeds experienced in cyclones is estimated in this paper. The study estimates the likely failure mode and percentage of failure for a representative proportion of houses with increasing wind speed. Probabilistic methods are employed to specify the distribution of wind loads and also the component resistances. Findings from full-scale house testing, and component joint tests, have been incorporated in this study.

The study focuses on the chain of connections from the roof cladding fixings to the sub-floor bracing. Failure of sub-structure components identified as roof envelope, roof structure, wall structure and sub-floor bracing are analyzed by considering failure modes of loss of roof cladding, loss of roof structure, wall failure and sub floor racking failure. The progressively increasing percentage of houses being subjected to high internal pressures resulting from damage to the envelope is considered. The wind load and the component connection strengths are treated as random variables with log-normal distributions. These variables are derived from experimental studies, structural analysis, damage investigations and experience. This study also incorporates progressive failures and considers the inter-dependency between the structural components in the house when estimating the types and percentages of the overall failures in the population of these houses.

This study has demonstrated the use of probabilistic methods for estimating percentages of high-set houses suffering damage in cyclonic winds, and related modes of failure. It should be noted that the high-set house is based on the construction practices and design methods used prior to the introduction in the early 1980s of revised building standards for domestic construction in cyclonic areas. More recent construction showed significantly improved performance, as described by Henderson, *et al.* (2006).

With the favourable comparison to historical damage survey results, the probabilistic method is being applied to newer forms of house construction with complex geometry to estimate vulnerability of current construction housing to cyclonic winds. Another application of the approach used in this study is for evaluating the effectiveness of remedial and strengthening upgrades to the houses (such as additional tie down or shutters) by comparing estimated damage levels pre and post upgrade.

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