Performance of housing in Brisbane following storms on 16 November 2008 *

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SUMMARY: *A significant level of convective storm activity occurred in southeast Queensland during the period 16 to 20 November 2008. The various storm systems caused widespread damage to housing in many parts of Brisbane, although the peak gust wind speeds were estimated to be less than the current design wind speed for the city. Surveys of damage indicated that housing built post-1980 performed better than older housing, when building regulations were less stringent. Detailed inspections of damage showed that failures could be attributed to inadequate construction details, either built poorly or, for contemporary housing, not built in accordance with current requirements. The paper summarises some anomalies, and recommends improvement in design standards, codes and procedures.*

1 INTRODUCTION

The Bureau of Meteorology (Bureau) recorded a strong level of convective storm activity in southeast Queensland during the period 16 to 20 November 2008, which caused significant damage to housing in many parts of Brisbane. The Cyclone Testing Station (CTS) conducted an investigation to assess the effects of wind-related damage to housing in The Gap and Redbank Plains from the 16 November storm, and in Paddington from the 19 November storm (Leitch et al, 2009). A general view of the study area is shown in figure 1.

This paper focuses on damage to housing in areas around The Gap and Redbank Plains caused by the storms on 16 November 2008, provides estimated wind speeds for the storm at The Gap, describes possible reasons for failures of structural components, and examines some anomalies in relation to the application of building codes and standards. The paper also summarises recommendations for reviewing current codes and standards as outcomes from the study

2 ESTIMATES OF WIND SPEED

To analyse the performance of housing, an essential first step is to obtain an estimate of the wind field (peak gust speed and direction) to which it was subjected. An overview of the approach used and estimated values adopted is presented in this paper. The report by Leitch et al (2009) provides a detailed account of the calculations and analysis.

The absence of instrumentation in the area and the localised nature of the storms, coupled with

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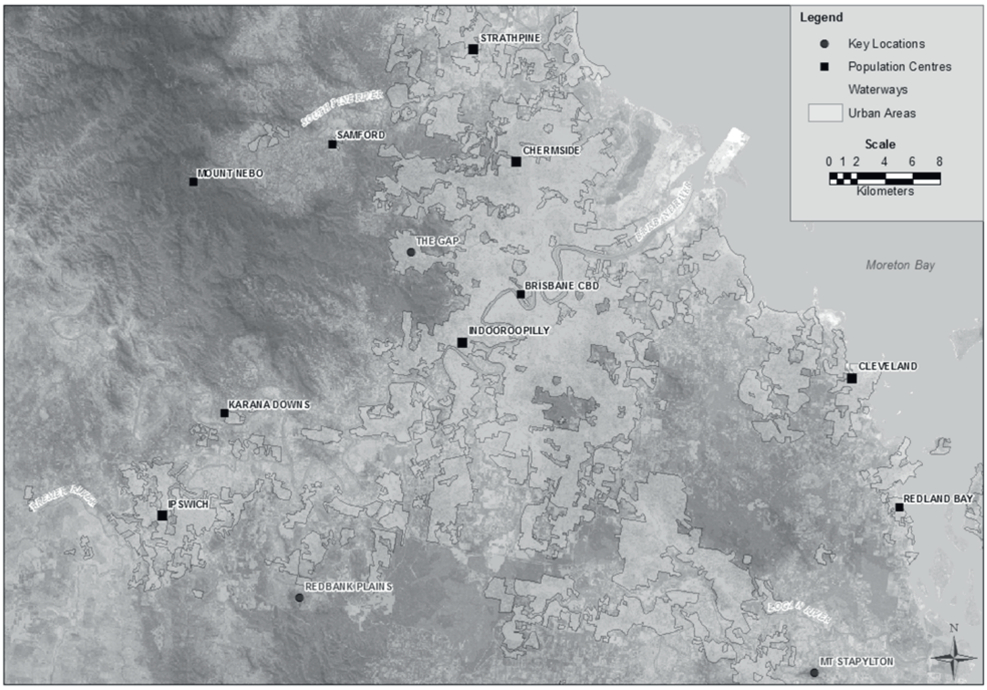


Figure 1: Locality of investigation area.

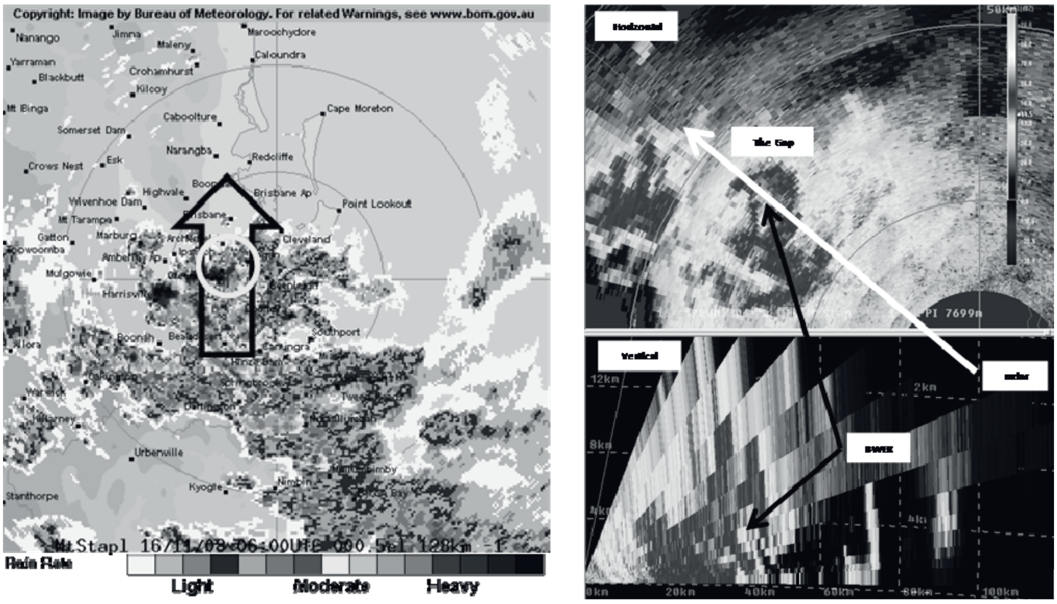


Figure 2: Bureau of Meteorology radar imagery of hydrometeor reflectivity on 16 November 2009 from Mt Stapylton radar. Left: Regional plan view at 0600 UTC centred on the radar showing 50 km range rings; circle indicates The Gap cell and arrow indicates approximate direction of motion. Right: Composite of detailed horizontal and vertical scans from the radar at 0624 UTC towards The Gap location (vertical scan has radar origin lower left).

the complex topography of The Gap, make the estimation of wind speeds difficult. Therefore, the wind speeds in the study area were estimated using a range of techniques that comprised advice from the Bureau, interpretation of radar images by Systems Engineering Australia Pty Ltd, calculations on a road sign showing signs of plastic failure and estimates based on the extent of damage to housing.

Figure 2 shows an overview of the very extensive regional severe storm activity at the time of the damaging winds that impacted The Gap and surrounding areas. The radar images are from the Bureau's Mt Stapylton site, about 45 km southeast of The Gap (refer figure 1), and indicate the density of hydrometeors (rain or hail). Of specific interest is the detail of the storm structure in the lower panels, whereby a so-called Bounded Weak Echo Region (BWER) is identifiable, which indicates an area of relatively low reflectivity surrounded by higher reflectivity. This feature is evidence of the high level of cell organisation; a likely mesocyclone circulation that is feeding the strong updrafts and downdrafts of a "supercell", with significant radar reflectivity as high as 12 km.

The small scale of the severe storm cells meant that the limited number of regional anemometer sites were not able to capture the maximum wind speeds, although some did record significant gusts during these events, as summarised in table 1.

The detailed assessment of high level wind speeds of The Gap storm used radar images (precipitation and Doppler) from the closest Bureau sites: the Mt Staplylton radar 45 km to the southeast was used to determine the basic storm track parameters, while the Redbank Plains radar (a research facility 20 km to the southwest) was ideally located to measure the along track speeds of the storm system. A series of images from this latter high-resolution radar were used to estimate the maximum Doppler mean horizontal wind speeds above the impact zone, as summarised in table 2. This table indicates that peak winds of the order of 50 m/s passed about 450 m above The Gap. Due to the expected highly coherent structure of downburst-related winds, these speeds might be regarded as being largely analogous to 3-s gust speeds. However, measurements from other events and experiments indicate that the maximum horizontal wind in such events could be expected to occur at about 100 m above ground, at least in relatively flat terrain, with a low-level frictional decay to the surface (see Vicroy, 1992).

The analysis of plastic bending of steel pipe supports of road signs is a useful tool to provide estimates of peak wind speeds and this approach was used successfully for the damage investigation following tropical cyclone Larry (see Ginger et al, 2007). For The Gap investigation, no obviously damaged signs were observed during the various initial inspections. One slightly bent sign support was located in The Gap on a subsequent inspection that specifically targeted such road signs. Calculations for this sign, using the range of common pipe wall thicknesses, give an estimated wind gust speed of between 40 and 50 m/s at the standard reference height of 10 m.

Based on inspections of the damage to housing in the study area, it was estimated that the peak gust wind speed at the standard 10 m reference height was also of the same order.

Using the range of techniques described above, it was estimated that the peak gust wind speed at 10 m height in Terrain Category 2 (flat open terrain) was of the order of 45±5 m/s (approx 160±20 kph). This compares with the regional design wind speed of 57 m/s.

3 OVERVIEW OF WIND LOADING ON BUILDINGS

The fluctuations in wind velocity that impinge on a building cause spatial and temporal variations to the pressures that are imposed on the external surface of the building envelope. Generally the windward wall is subjected to a positive (inwards acting) pressure, while the other surfaces (roof, leeward and side walls) have negative (outwards acting) pressures applied. Flow separation at building discontinuities (eg. leading edge of the roof) gives rise to much larger negative pressures on these local roof edge regions making them more vulnerable to failure. For a nominally sealed building, the internal pressure

Table 1: Maximum Bureau recorded wind gusts in the Brisbane region on 16 November 2008.

Station	Time	Direction		Peak 3-s gust wind speed		
	(EST)	deg		kts	km/h	m/s
Amberley AMO	16:21	116	ESE	45	83	23.1
Cape Moreton Lighthouse	23:33	150	SSE	35	65	18.1
University Of Queensland Gatton	16:08	157	SSE	38	70	19.4
Archerfield Airport	16:37	183	S	30	55	15.3
Gold Coast Seaway	19:35	165	SSE	32	59	16.4
Redcliffe	17:55	133	SE	32	59	16.4

Table 2: Maximum Doppler wind speeds from Redbank Plains radar on a radial to The Gap.

Time		Doppler max. "surface" mean speed (m/s)	Timing relative to The Gap
UTC	EST		
16/11/2008 6:30	16/11/2008 16:30	21	Before
16/11/2008 6:36	16/11/2008 16:36	53	After
16/11/2008 6:42	16/11/2008 16:42	42	After
16/11/2008 6:48	16/11/2008 16:48	49	After

is significantly smaller than the external pressures. However, if the windward wall of the building is breached, say by the failure of a door or a window broken by impact from wind-borne debris, then the resulting dominant opening allows the wind to pressurise the inside causing a large positive internal pressure to develop, as shown in figure 3. The combination of external negative pressures and internal positive pressure results in significantly larger net upwards load on the roof, and this is a common cause of its failure.

4 WIND LOADING DESIGN CONSIDERATIONS

The age of a house determines the wind loading design criteria that were applicable when it was built. Currently, the Building Code of Australia (BCA, 2008) published by the Australian Building Codes Board (ABCB) stipulates design criteria for housing in Australia. These requirements are met by compliance with a range of standards relating to building construction (eg. Standards Australia, 2002b).

In the 1970s, houses in Townsville and Darwin suffered significant damage during Cyclone Althea and Cyclone Tracy, respectively and this precipitated the development of the Home Building Code of Queensland (1975-1984) as Appendix 4 to Standard Building by-laws. These building regulations were significantly more stringent than earlier versions and were in widespread use by the mid 1980s. They require sites to be categorised by design wind speed at roof height, and contain "Deemed to Satisfy" detailing for houses in each of these categories. Related standards, such as AS4055 *Wind loads for housing* (Standards Australia, 2006b), and AS1684.2 *Residential timber framed construction – Part 2 Non-Cyclonic areas* (Standards Australia, 2006a), are applied to more recent housing design and construction.

Brisbane is located in Wind Region B as defined in AS/NZS1170.2 (Standards Australia, 2002b), where the 500-year return period ultimate limit state design wind speed at the standard 10 m reference height (in flat approach open terrain) is 57 m/s. The design wind speed at the roof height of a building has factors to account for the height, upwind shielding, terrain and topography. This factored design wind speed impacting on the building is related to the pressures exerted on its elements through a series of coefficients defined in the wind loading standard, AS/NZS1170.2 (Standards Australia, 2002b).

AS4055 (Standards Australia, 2006b) provides design wind speeds and wind loads (which are based on AS/NZS 1170.2) for the design of typical housing. A wind classification is stipulated depending on the wind region (ie. non-cyclonic or cyclonic) and terrain, topography and shielding at the site. Design data given in AS4055 provides an easy to use means of obtaining wind loads for typical houses, and for the selection and detailing of components. However, to simplify the design and to accommodate the design of a group of "similar" houses located in suburbs (with typical terrain, topography and shielding features), AS4055 categorises the house sites into a total of 10 wind classifications. In Region B, site classifications N2, N3 N4, N5 and N6 represent increasing design wind speed. However, this simplification means that AS4055 has some incompatibilities with AS/NZS1170.2, leading to significantly lower design loads, in some specific cases.

For timber-framed housing, the construction methods specified in AS1684.2 (Standards Australia, 2006a) are based on the design wind load data using either AS4055 or AS/NZS 1170.2. For each classification N1 to N6, AS1684.2 gives design (uplift) wind load on roof battens and roof framing for some typical batten and frame spacings. In addition, AS1684.2 also specifies uplift capacities for typical batten-truss/rafter connections, rafter-rafter connections and truss/rafter-top plate connections (nails, screws, framing anchors, straps, etc).

Standards on windows in buildings, AS2047 (Standards Australia, 1999) and domestic garage doors, AS/NZS4505 (Standards Australia, 1998) use the design wind speeds and classifications in AS/NZS1170.2 and AS4055 to specify requirements for windows and garage doors, respectively.

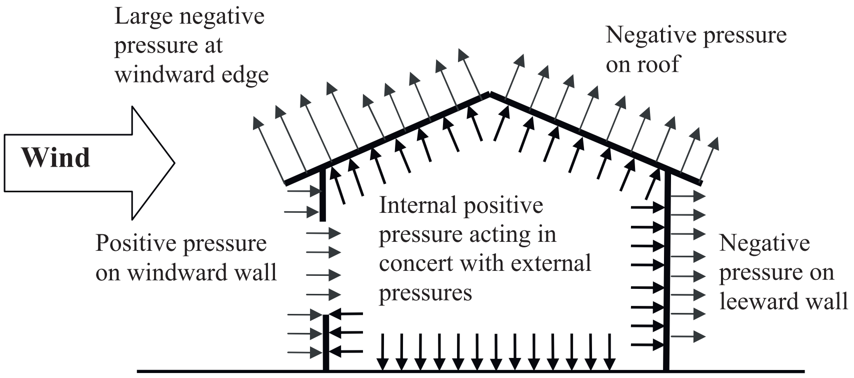


Figure 3: Schematic wind pressure distribution for building with a dominant opening on the windward wall.

5 HOUSING STOCK IN THE STUDY AREA

The Gap has a mixture of house types. Differences in size, shape, window size, cladding type, roof shape, age and methods of construction have an effect on the resilience of the house to resist wind forces.

One very important parameter is the building standards that were applicable at the time of the construction of the house. The building regulations were made significantly more stringent with the introduction of the Queensland Government's *Home Building Code – Appendix 4 to the Standard Building By-Laws 1975* (Queensland Goverment, 1975-1984). Damage to housing is assessed by categorising them into two age groups, pre-1980 and post-1980. However, it must also be noted that many of the older houses, in the pre-1980 group, may have been refurbished to various extents. Description of the house types and their typical characteristics are detailed in the report by Leitch et al (2009).

6 OVERVIEW OF HOUSING DAMAGE

Damage surveys can be used to provide an overview of any trends in construction or design issues that affect the resilience of housing to resist wind loads.

The street survey damage classification system was based on that developed by Boughton (2006). It ranks the amount of visible structural damage using a three digit Damage Index to grade the levels of damage for roof, damage to wall openings and wall damage. A Damage Number is assigned for a defined level of damage for each of the three parameters measured, as detailed in table 3. The Damage Numbers are assigned based on a brief visual assessment of the structural damage that can be seen from the street (ie. outside the house and viewing only the front area of the roof and walls and to a lesser extent, the two side walls). Therefore, the survey does not identify any internal damage (such as damage by water ingress) or partial "hidden damage" (such as batten to rafter nails being partially withdrawn).

A street survey of a sample of 97 houses from five streets in The Gap was undertaken, following the storm of 16 November 2008. This included 86 pre-1980 and 11 post-1980 houses. All houses in each street were surveyed to ensure that both damaged and undamaged houses were included. Roof damage caused by falling trees was classified as "Debris damage to roof" (R = 2) and if the tree also broke windows and walls, a Damage Index of 222 was assigned. Table 4 presents a summary of the relative

Table 4: Percentage damage for roof, openings and walls for all housing surveyed at The Gap.

Damage Number	Percentage damage for each Damage Number		
	Roof	**Openings**	**Walls**
0	70%	86%	91%
1	2%	1%	2%
2	7%	5%	3%
3	9%	0%	3%
4	2%	8%	0%
5	0%	0%	1%
6	0%	0%	0%
7	1%	0%	0%
8	3%	0%	0%
9	5%	0%	0%

Table 3: Housing survey damage measure using three digits.

Damage Number	Description of damage for		
	Roof (R)	**Openings (O)**	**Walls (W)**
0	None	None	None
1	Gutters downpipes	Debris not pierced	Debris not pierced
2	Debris damage to roof	Debris pierced	Debris pierced
3	Roof lifted < 10%	Windows/doors leaked	Carport/verandah damage
4	Lost roofing < 50%	Windward broken < 30%	One wall panel fallen
5	Lost battens < 50%	Frames lost < 30%	> 1 wall panels fallen
6	Lost battens > 50%	Windward broken 30%-70%	Racking damage, cladding attached
7	Lost battens > 50% and lifted rafters	Windward broken > 70%	Racking damage and lost cladding
8	Lost battens > 50% and damaged tie down	Windward broken > 70% and suction loss	Only small rooms intact
9	Lost roof structure > 50%, including ceiling	100% broken/missing	No walls remaining

Notes: 1. R3 = any combination of loss of roofing, battens, rafters but limited to less than 10% roof area; 2. Damage to carports and verandahs that is under the main roof is treated as roof damage.

quantity (percent of sample population) in each of the three Damage Numbers for all of the houses surveyed in The Gap.

The damage survey results are also presented as separate plots of percent in each Damage Number category (for each of the two age categories) for each of the three parameters used, roofs, openings and walls, in figures 4, 5 and 6, respectively. The data from figure 4 shows that there was notably more roof damage to the pre-1980 houses, compared to the

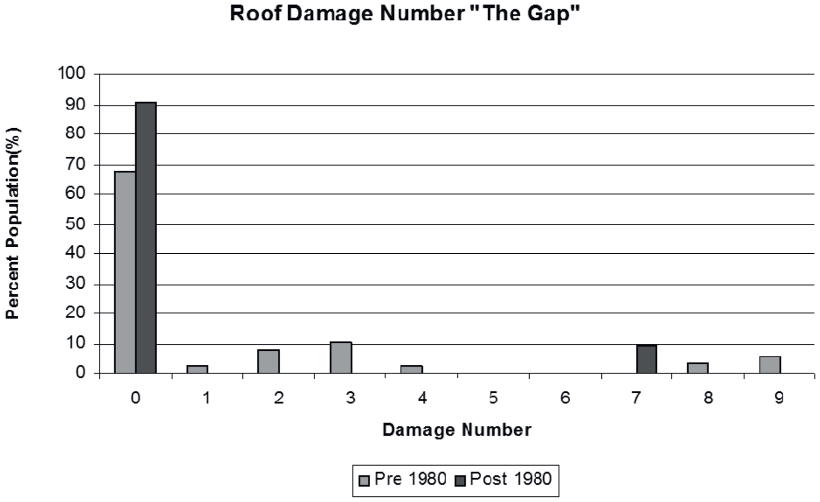


Figure 4: Percentage of houses versus roof Damage Number.

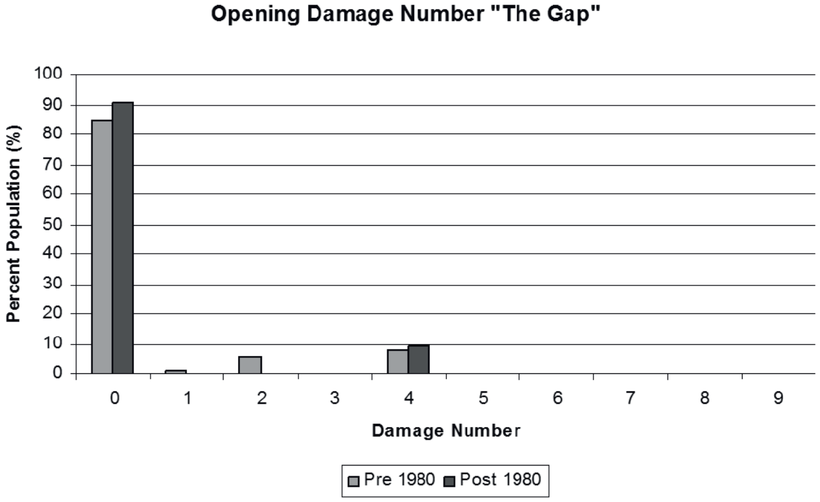


Figure 5: Percentage of houses versus openings Damage Number.

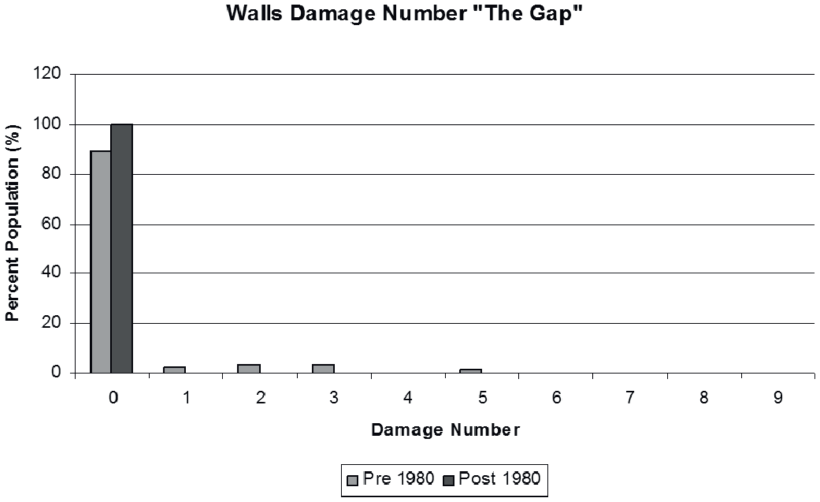


Figure 6: Percentage of houses versus wall Damage Number.

post-1980 houses, although the latter sample is small. Figure 5 shows that there were about 10% of houses with an openings Damage Number of 4, implying that these houses were very likely subjected to large internal pressure. Figure 6 shows that there was no wall damage to the post-1980 houses.

7 LEAKAGE THROUGH UNDAMAGED TILED ROOFS WITHOUT SARKING

Parts of Redbank Plains were also subjected to the earlier portion of the severe storm on 16 November 2008, which caused a significant water ingress problem through unsarked tiled roofs to some new housing. This water leaked onto the top of plasterboard ceilings, which became waterlogged and then collapsed. Figure 7 shows two views of typical examples of collapsed ceilings in two new houses. Figure 7(a) is a detailed view of a small area of collapsed plasterboard ceiling to the garage of a new house, while figure 7(b) shows a general view of a collapsed ceiling to another new house in the same suburb. Figure 8 is a general view of another new house after the collapse of the plasterboard ceiling and the exposed timber trusses that support the tiled roof can be seen.

Three ceiling collapse houses with undamaged tile roofs were inspected in Redbank Plains, with two of them having no sarking at all and the third having sarking installed to about the lower 1 m of roof (closest to the fascia), as shown in figure 9.

In summary, these unsarked or partially sarked tiled roofs were not weather tight, and so allowed the ingress of wind driven rain from this storm, which instigated the collapse of the plasterboard ceilings. Note that further investigation on how to prevent such ceiling collapses is warranted, as diaphragm action of the ceiling panels is often a critical link in the load path for houses to support lateral wind loading.

8 PERFORMANCE OF HOUSING

The performance of housing is summarised using four broad categories of tree damage, water ingress, roof failure, and window and door failure.

Figure 8: General view of typical ceiling collapse (unsarked tile roof).

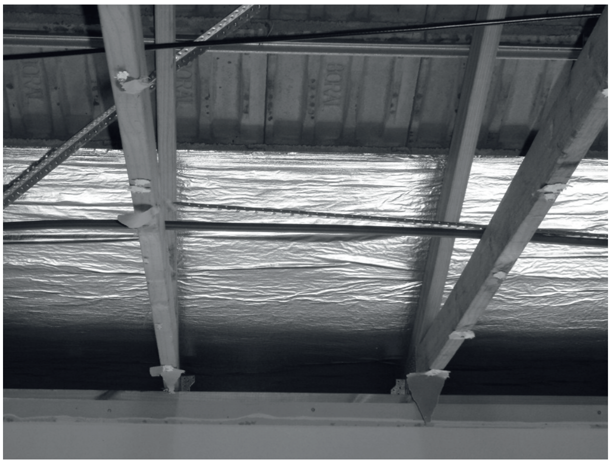


Figure 9: Tiled roof with sarking to lower 1 m of roof.

Figure 7: Two views of typical ceiling collapses due to water penetration – (a) detailed view of collapsed ceiling and (b) general view.

The Gap has a high density of trees, and many trees were both damaged and caused damage. At the smaller end of the size spectrum was shredded leaf litter that contributed to blockage of gutters in some cases. Many small shrubs or trees were damaged during the storms but did not contribute significantly to housing damage. However, large trees falling onto the roof did cause significant damage to the houses that they struck, as shown in figure 10, and this damage was recorded as Damage Number of 2 for the roof.

Water ingress is a major cause of insurance losses (to house contents and fittings) during these storms. For this investigation water ingress was categorised into three main areas, through undamaged windows and doors, through unsarked tiles, and overflow from blocked gutters. Many people in The Gap reported their windows had leaked during the height of the storm, likely caused by the high wind pressure on the outside surface forcing water through small gaps or even upwards through flashings. The second category was water driven through undamaged unsarked tiled roofs, as occurred in Redbank Plains and detailed in section 7. There was no evidence of this type of damage in The Gap, possibly because most of the house were pre-1980 with limited use of plasterboard ceilings.

Several residents reported that their houses suffered water ingress caused by hail (and perhaps leaf litter) that blocked gutters and downpipes, which then allowed the gutters to overflow back over the fascia boards and into the ceiling space. Figure 11 shows a damaged house in The Gap where the owner reported that the gutters overflowed after being filled with hailstones. This allowed water to penetrate into the ceiling space, saturating a panel of plasterboard that then collapsed, as can be seen in figure 11(a).

Table 4 shows that roof damage was the major failure from the storms, with about 20% of all houses surveyed having a roof Damage Number of 3 or worse. However, most of this damage occurred to the pre-1980 age class, as illustrated in figure 4. All of the detailed inspections into damaged roofs showed that the failures were caused by inadequate or under-strength details in the tie down load path. The three case studies in section 9 provide more details.

A detailed inspection was performed on one damaged tile roof where some ridge and barge tiles were dislodged during the storm, allowing water into the ceiling, which then led to the collapse of the plasterboard ceiling. Figure 12(a) shows the damage to the roof. Figure 12(b) shows that contrary to the requirements of AS2050 (Standards

Figure 10: Damage to housing caused by large falling trees in The Gap.

Figure 11: Overflow of gutter water into ceiling space – (a) view of hole in ceiling, after collapse of one plasterboard panel; and (b) view inside ceiling space showing water staining on rafter overhang.

Figure 12: Details of ridge tile roof failure and hip tiles without mechanical fastening – (a) view of failed ridge tiles under tarpaulin; and (b) view of ridge and hip tiles, which are still in place but some without mechanical fasteners.

Figure 13: Typical windows and doors failure – (a) hail damage and window pushed in; and (b) sliding door failed.

Australia, 2002a), not all of the ridge and hip tiles were mechanically fastened.

As shown in figure 5, window and door failure resulting in breakages/opening with an openings Damage Number of 4 was about 10% for both pre- and post-1980 houses. These houses with this level of openings damage would very likely have been subjected to a significant increase in positive internal pressure. However, even post-1980 houses are not required to be designed for full internal pressure. Therefore in a design wind speed event, all of these houses would have been subjected to larger wind forces than the current wind loading standard requirements and so would have an increased risk of failure.

Some doors and windows were damaged by hail or wind pressure. Figure 13(a) shows a bank of damaged hopper windows (under a tarpaulin) where a resident reported that one window had a small hole punched through it by a hailstone and the second was pushed inwards by the winds. Figure 13(b) shows where another house had a complete sliding door fail under the wind loading. Figure 14 shows broken/damaged windows from two different houses that was typical of the damage caused by wind-borne debris.

Other windows and doors failed due to inadequate fixings to their structural supports. One pre-1980 house had a sliding door that faced the strongest wind direction that was pushed inwards without the glass being broken. A post-1980 house (detailed in section 9) also had windows facing the strongest wind direction fail due to inadequate fixings to their supports. The storms caused failures to some garage doors (both panel lift doors and roller doors) and figure 15 shows two typical failures of roller doors from houses in The Gap.

During the investigations, several cases were recorded where residents reported that after windows or doors failed, the sudden increase in internal pressure caused a subsequent failure in all or part of the roof. For the first case, a windward window was broken and then between 10 and 20 tiles were dislodged from the roof. For the second case, a large glass sliding door on the windward wall

Figure 14: Typical wind-borne debris damage to windows.

Figure 15: Typical roller door failures.

of pre-1980 house failed and then a large section of roof structure failed. This failure is described in more detail in a case study in section 9.

9 CASE STUDIES

9.1 Inadequate tie-down: pre-1980 house

A pre-1980 house (originally built in 1966) lost the full width of the roof to the front balcony and rooms immediately behind. This house was located on flat terrain in The Gap, and had large glass doors and windows facing into the wind direction of the storm. Residents advised that during the storm one of the large glass windows to the front balcony failed, and then a large section of the roof (the whole roof width of about 12 m and a length of about 7 m) was lifted up from the house and deposited in the back yard in an upside-down position.

The roof structure used a system of timber battens and struts supported by 250 × 70 Oregon timber beams, spanning about 9 m across the house width, cantilevering about 1.6 m on both sides and spaced at about 2.8 m. These timber beams were held down with mortice and tenon joints to timber columns. The roof failed as a large unit when the timber beam to column mortice and tenon joints failed in uplift, caused by the sudden increase in internal pressure load when the windward windows failed.

Figure 16 shows a view of the front of the house, without the roof. The tenon joint to the top of one of the timber columns used to support the timber beams is circled, in the photograph. Figure 17 is a general view of the section of roof that had been broken away from the house and deposited upside-down in the backyard. A typical mortice to the beam is circled in this photograph. Figure 18 is a detailed view of a typical mortice to the roof beam, showing where the mortice and tenon joints failed. These joints from the timber column to main roof beams were too weak to support the large uplift loads that needed to be resisted, especially with the extra load caused by the sudden addition of large positive internal pressure.

9.2 Incorrect tie-down: post-1980 house

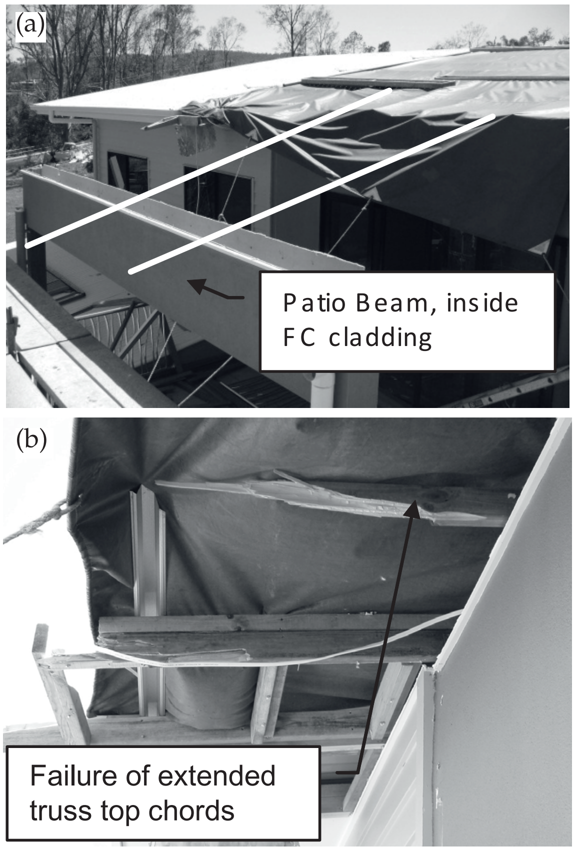
A new two-storey house lost all of the patio roof structure (cladding, battens and extended truss top chords) with roof failure extending into the area

Figure 16: General view of front of house without roof.

Figure 17: General view of upside-down roof.

Figure 18: Typical view of a failed mortice joint to roof beam.

above the main living area. Figure 19(a) shows a general view of the upper level of the house and the location of the missing patio roof that failed. Two white lines have been added to this photograph to indicate the typical locations of the former extended truss top chords. These main roof trusses had the top chord extended by about 4 m and were supported at their far ends by a single span structural timber beam, located towards the bottom and inside non-structural fibre cement (FC) cladding, as also shown in figure 19(a). Figure 19(b) is a view from underneath the patio roof and shows some typical failures of the extended truss top chords.



Patio Beam, inside FC cladding

Failure of extended truss top chords

Figure 19: Failed patio roof structure – (a) general view showing extent of the loss of the patio roof; and (b) view from underneath patio roof showing failed truss tails.

Failure of the patio roof was initiated by inadequate tie-down of the extended truss top chords to the patio beam/lintel. A closer examination of the members inside the non-structural FC cladding showed that there was a structural timber patio roof beam at the lower level, then short vertical studs (jack studs) supporting a single 70 × 35 MGP10 top plate towards the top. The truss top chords were fixed using looped metal straps to this top 70 × 35 MGP10 top plate, but this top plate was only nailed into the end grain of the jack studs sitting on top of the main structural patio beam. The patio roof was lifted off when the nailed connections from the top plate into the top of the jack stud members failed.

This tie-down configuration was not in accordance with any of the possible alternative details provided in AS1684.2, Table 9.20 (a) to (e) inclusive, which all require that: "The top plate shall be fixed or tied to the lintel within 100 mm of each rafter/truss, or the rafter/truss fixed directly to the lintel with a fixing of equivalent tie-down strength to that required for the rafter/truss."

Figure 20 is a detailed view inside the FC cladding and shows some of the nails remaining from the failed joints between the jack studs and the top plate. Figure 21 shows a large section of the failed patio roof located about 200 m away from the house.

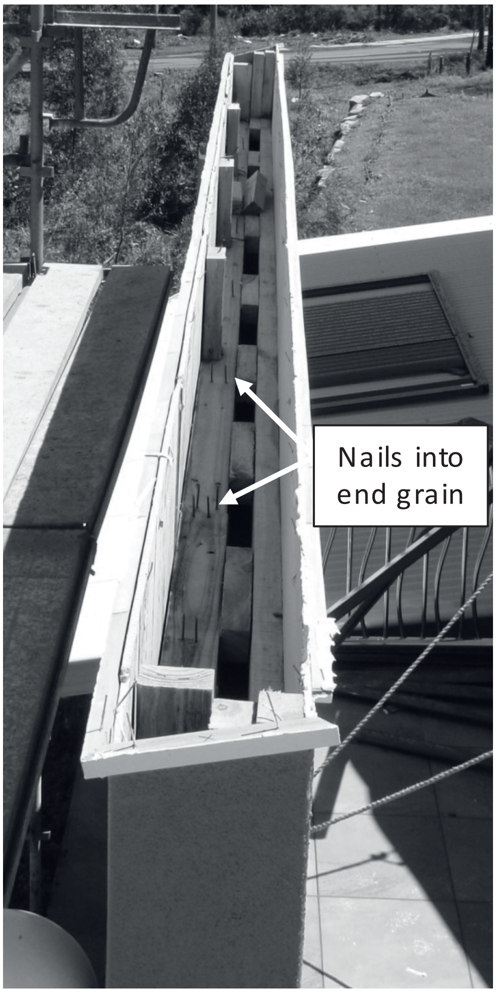


Figure 20: Detailed view inside FC cladding to patio beam showing failure of nailed joints to jack studs.

The photograph shows the metal straps from the extended truss top chords to the top plate, and also some of the jack studs over the beam/lintel and failed nailed joints. The patio system failed in a two-stage sequential manner. Firstly, the joints from the outside end of the extended truss top chords failed when the wind uplift loads exceeded the capacity of the nails into the end grain of the jack studs. The extended truss top chords were then acting as cantilevers and subsequently failed in bending (see figure 19(b)), due to the added tributary area and finally a large part of patio roof structure was lifted off the house and deposited about 200 m downwind from the house.

Also on this house, a nominally 1.0 m wide gable overhang failed (the overhang was torn off the roof) due to inadequate tie-down fixings of the out-riggers forming the overhang to the raking and standard trusses. The small approximately 300 mm back span of the out-riggers exacerbated the problem. Although AS1684 generally requires the back span to be twice the cantilever, smaller values (typically the standard truss spacing) can be used, provided the truss members and associated connections are designed accordingly.

The failures described here allowed significant water ingress to the house, causing loss of ceilings at several locations on both storeys.

9.3 Incorrect tie-down and window frame connection: post-1980 house

A new two-storey house, built near the top of a steep ridge in The Gap, sustained extensive damage. About

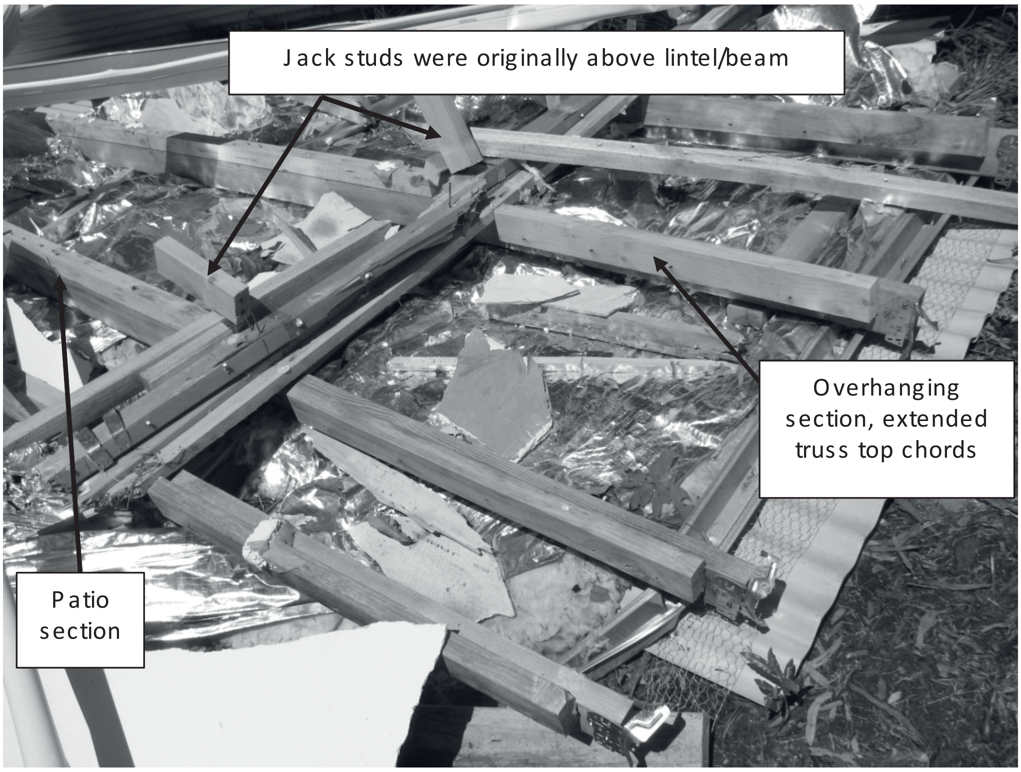


Figure 21: Failed patio roof structure about 200 m downwind from house.

Figure 22: Lintel hold-down, combination of metal straps and HD bolts at far end of ply bracing frame – (a) one end of top plate, strap failed and M12 bolt too far away; and (b) M16 bolt too far from lintel and not configured as per AS1684.

half of the roof structure and upper walls/windows to the house were lifted off the house. The house had a clear view over the edge of this ridge looking to the south, in the direction of the strongest winds and so was likely subjected to a large topographic wind speed up effect.

Two upper-storey rooms facing south had failure of the tie-down beside the lintels over large windows, subsequently causing the loss of the roof. It appears that the tie-down provided for these lintels used M16 rods in some cases and double metal straps for others. Figure 22(a) shows the remains of a top plate that was fitted above a lintel. This top plate still has a long double metal strap attached that has failed. An M12 bolt was also connected through the end of this top plate near the far end of a plywood bracing panel nominally 600 mm wide. Figure 22(b) shows an M16 rod beside another opening and it appears likely that the failure of the top plate was caused by excessive shear due to the M16 rod not being configured as required by AS1684 Table 9.20 (d) or (e). Note that it appears that the plywood bracing was being used as the tie-down beside the opening shown in Figure 22(a).

It also appears that opening on the rear wall was using the plywood bracing panel as the tie-down beside the opening, as shown in figure 23. This assumption is based on the absence of bolts or straps beside these openings. Using plywood as tie-down beside an opening is not in accordance with AS1684. Figure 24 shows one of the configurations required by AS1684 for M16 rods. As can be seen from this detail, shear in the top plate is virtually eliminated as opposed to the configuration shown in figure 22(b), which requires the top plate to transfer the uplift forces to the tie-down rod and is only applicable to tie-down capacities requiring rods up to M12.

Figure 23: Lintel to back window, held down by ply bracing panel.

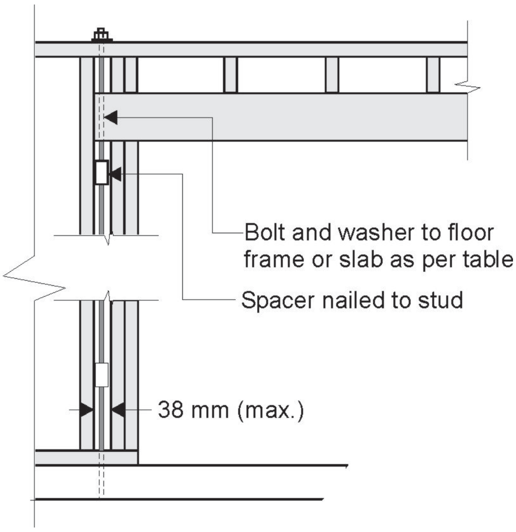


Figure 24: One method for lintel tie-down using M16 rods (as specified in AS1684.2).

This house also had all its windows facing the strongest wind direction fail or substantially dislodged from supporting jamb studs, as shown in figure 25. Note that the fixings from these window panels to their supports failed and that the glass itself was unbroken. Therefore these failures were caused by inadequate fixings from the window frames to the main structural window supporting members.

Figure 25: Failure of windward window panels – (a) complete unbroken window pushed in; and (b) window dislodged from jamb studs.

10 ANOMALIES IN DESIGN CRITERIA

The majority of contemporary housing suffered little or no structural damage from these storms, but this should be expected as the wind speeds from the storms were estimated to have been less than ultimate design values. However, the analysis of damage data from this investigation highlighted some anomalies in the design standards and codes.

10.1 Wind loading standards

The housing design requirements can be specified using separate wind loading standards, AS/NZS1170.2 or AS4055. AS/NZS1170.2 is the parent standard and can be used to determine design wind loads that are applicable for the majority of low to medium rise buildings, as well as many other structures. However, the simplified approach of AS4055 has some incompatibilities with AS/NZS1170.2, leading to lower design loads in some cases. The report by Leitch et al (2009) details the case of a house located close to the top of a hill and exposed to high winds from the direction leading up to the hill-top. For this case interpretation of AS4055 and its inherent simplifications will lead to unconservative design wind loads of almost 30% less than those calculated in accordance with AS/NZS 1170.2.

10.2 Windows: water penetration and connections to supports

A severe wind storm will generate large positive pressure differentials across windows and doors on the windward face of a building, these pressures could exceed 1 kPa for a moderate (less than design) wind event in Region B areas. However, AS2047, which specifies the selection and installation of windows in buildings, has water penetration test pressures set at between 150 to 450 Pa for windows for wind classes N2 to N6. This appears to be a serviceability design requirement only and so will not prevent water ingress for severe wind events such as these Brisbane storms.

As detailed in section 9, some cases of window damage caused by failure of the connections between the window frame and the supporting jamb studs were observed. For these cases, it appeared that these window or door frame connection fixings were inadequate. Clause 7.2 of AS2047 requires only that "Window assemblies shall be fixed into the building using recognized building practices", but more explicit guidance should be provided.

These two issues should be of special concern to the building and insurance industries as they increase the vulnerability of housing to severe wind events, and also to government, which often has to cover the final bill for catastrophic community events.

10.3 Garage doors

Many garage doors performed poorly in these storms, failing at loads estimated to be significantly less than those that would be caused by design wind speeds. Therefore the building industry should ensure that garage door systems are adequately wind rated or alternatively stipulate that the buildings should be designed for high internal pressure on the assumption that the door will fail in a severe wind event.

11 OUTCOMES FROM DAMAGE INVESTIGATION

Based on this damage investigation, the report by Leitch et al (2009) provided recommendations to review some design standards and codes as summarised here.

A review of some aspects of the following standards:

- Revise the factors used determine design wind speeds in AS4055 to be consistent with AS/NZS1170.2.
- Investigate a potential need for requiring all houses in non-cyclone areas to be designed for full internal pressure, unless the windows and doors are capable of resisting the applied wind pressures and an appropriate level of wind-borne debris impact loading. Note that proposed amendments to revise the current wind loading standard, AS/NZS1170.2, include a new explicit statement that closed doors (including roller doors) and windows shall be considered to be openings unless they are capable of resisting the applied wind pressures. This new statement, coupled with a campaign to educate designers, should increase awareness of this issue.
- Consider increasing the differential pressure limit for windows to remain watertight in AS2047.
- Provide explicit guidance on the fixing of windows and door frames to their supporting structure for the various wind classifications in AS2047.
- Ensure that design and installation specifications for domestic garage doors are adequate in AS/NZS4505.

A review of the following areas of the BCA:

- Review the application of the weatherproofing requirements to minimize the loss of amenity for occupants of housing (see Clause P2.2.2 in BCA (2008) Vol. Two, for example).
- Include requirements for roof lights to have resistance to both wind loading and impact from hailstones.
- Require tile roofs to all wind areas (ie. both cyclonic and non-cyclonic) have sarking installed over the full roof area. This may also require development of a practical sarking/lapping guide to ensure that water-proofing is actually likely to be effective. Note that in late 2009, the Queensland Government announced a proposal that from 1 May 2011 the BCA be amended to require all tiled roofs on new buildings to be sarked. A follow-up investigation is also warranted into whether materials used for ceiling panels, where ceiling diaphragm action is critical, should be water resistant, to ensure that they are available to support lateral wind loading.

12 CONCLUSIONS

The severe winds in and around The Gap caused by the storms on 16 November 2008 had an estimated peak gust wind speed of about 50 m/s at about a height of 450 m and near surface peak gust wind speed in the order of about 45±5 m/s at the standard reference height of 10 m in flat open terrain. This is significantly less than the regional design wind speed of 57 m/s and so all of the housing inspected for this investigation would likely have been subjected to peak gust wind speeds less than the current design values.

In general terms, post-1980 houses performed better that pre-1980 houses as would be expected due to the more stringent requirements introduced with the Queensland Government's *Appendix 4 to the Standard Building By Laws* (1975-1984) and subsequent TRADAC (1990a; 1990b; 1990c) timber framing manuals.

The main causes of damage were:

- damage caused by falling trees, which is independent of the age of the housing
- water ingress, sometimes through failed windows or doors, or sometimes through doors and windows that had not failed
- failure of windward windows or doors resulting in a sudden increase of internal pressure often leading to a consequential failure of the roof.

Some of the failures observed were possibly exacerbated due to anomalies in the design criteria used to specify the construction details, as follows:

- Inconsistencies between AS4055 and AS/NZS1170.2 when determining the effect of typical terrain, topographic and shielding features on the site design wind speed. One such case is where AS4055 specifies significantly less severe topographic effects than AS/NZS1170.2 for houses located near the top of steep ridges.
- Significant water ingress through undamaged tile roofs that were installed without sarking, as detailed in section 7.
- Wind-borne debris caused breakages to windows and doors resulting in a sudden increase in internal pressure. However, there is no requirement for housing in non-cyclonic regions to be designed to resist high internal pressure to cover this design loading case.

For all cases where detailed inspections of damage were made, the failures could be attributed to inadequate construction details, either built poorly or for contemporary housing, not built in accordance with current requirements. Some of these inadequate details included:

- Inadequate tie-down with weak connection details. A number of such cases included pre-1980 houses that had been recently re-roofed, but the newly installed connection details were not adequate.
- Failures in post-1980 houses due to tie-down connections not being in accordance with the relevant standards.
- Window or doors not adequately fixed to their supporting structural members (jamb studs).

- Garage door failures due to poor construction details or not having an adequate design capacity.

Based on this damage investigation, the report by Leitch et al (2009) recommended a review of some standards and codes to improve the resilience of housing to severe wind storms.

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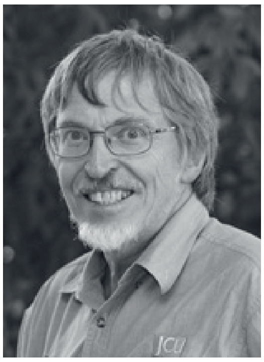

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