

# **Tropical Cyclone Larry**

**Damage to buildings in the Innisfail area**

**CTS Technical report: TR51**

**September 2006**



**CYCLONE  
TESTING  
STATION**



Australian Building Codes Board

## **Tropical Cyclone Larry Damage to buildings in the Innisfail area**

**Report: TR51**  
September, 2006

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JAMES COOK UNIVERSITY**

**TECHNICAL REPORT NO. 51**

**Tropical Cyclone Larry  
Damage to buildings in the Innisfail area**

**September 2006**

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## **Tropical Cyclone Larry – Damage to Buildings in Innisfail area Executive Summary**

Tropical Cyclone Larry made landfall on 20<sup>th</sup> March 2006 near the town of Innisfail and caused damage to a number of buildings in Innisfail and the surrounding district. The estimated wind speed at all places in the study area was less than the current design wind speed for the same sites.

This study estimates that the extent of the damage was as follows:

- Average of damage to building stock across the investigation area was 15% to 20% of its value.
- Average of damage to contemporary housing across the investigation area was less than 10% of its value.

More recent housing fared considerably better than older housing. This reflected a combination of the following:

- Current detailing represents a marked improvement on previous methods of construction detailing, especially roof tie down.
- The estimated wind speeds were less than the current design level.
- More recent housing is generally in better structural condition because it is newer and has not had to endure previous cyclone events.

Most commonly observed building failures included:

- Widespread failure of roller doors, often accompanied by loss of wall or roof panels.
- Loss of roof battens when fastened to rafters with one or two nails.
- Loss of rafters or trusses when anchored to top plates with skew nails only.
- Loss of struts, ridge members and connected rafters when struts were not tied down.
- More severe levels of damage for hill top sites.
- Failure of unreinforced masonry.
- Structural component failure of under designed cold formed steel sheds and garages.

Implications of this damage include:

- Performance of roller doors is not satisfactory, and in many cases, the resulting internal pressurisation of the structure has led to structural failures.
- Where structural anchorage had been installed in accordance with AS1684 – part 3 – Cyclonic areas, it performed well. However, in older buildings, the anchorage was often insufficient, and reconstruction should comply with current standards where possible.
- Construction details used in buildings on or near hill tops need to reflect the higher wind speed due to topographic acceleration of wind in those places. This is covered in both AS/NZS1170.2 and AS4055.



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## 1. Introduction

Tropical cyclone Larry crossed the North Queensland coast in the early morning of Monday 20<sup>th</sup> March 2006 and caused severe damage to infrastructure and crops in the region of Innisfail, shown in Figure 1.1. Wind damage was reported by the media well into the Atherton Tablelands and flooding was reported in the Innisfail area, the Tablelands and into the Gulf country.

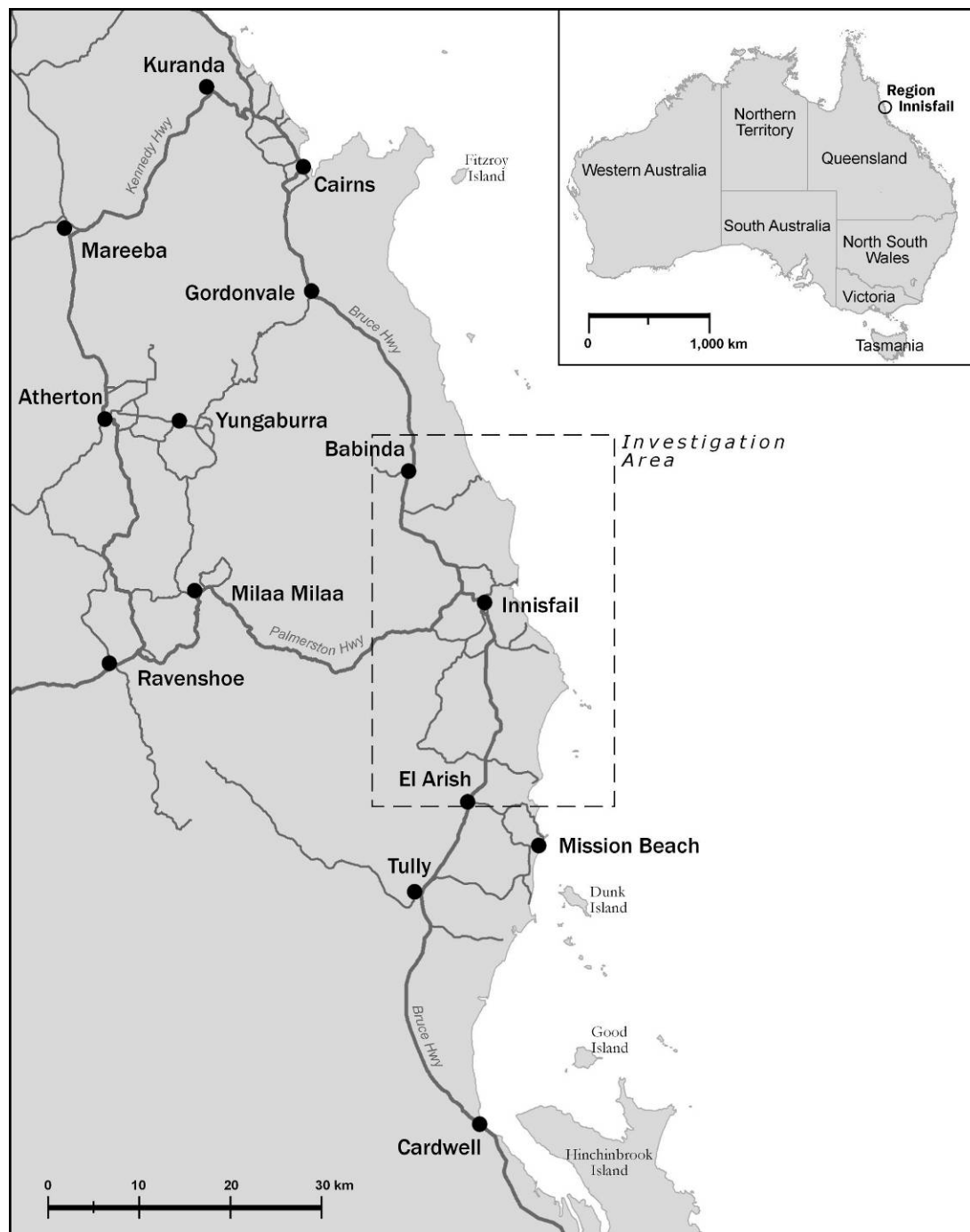


Figure 1.1: Locality of investigation area

The cyclone caused significant community disruption within the affected area. Lifelines (e.g. power, phones, roads) were severely disrupted. It took weeks to restore communications and power, with some properties not being able to be reconnected for months. Drinking water was not available for some days after the event. Fuel, food and other supplies were not generally available in the area in the weeks following the cyclone. It is anticipated that the rebuilding will take well over a year.

- This report examines only wind damage caused by Tropical Cyclone Larry and is restricted in its scope to damage within the Innisfail area (Figure 1.1).
- The report presents details of wind damage, an indication as to the extent of wind damage to different types of buildings, and ramifications of this damage for reconstruction.

This report focuses on the performance of buildings, which experienced strong winds in the areas in and around Innisfail; from Babinda to the north, Mirriwinni, Kurrimine Beach, Mourilyan and South Johnstone to the south.

## 1.1 Objective

The overall Cyclone Testing Station objective in conducting the investigation:  
*To investigate wind damage to buildings in the Innisfail area.*

More specifically the study sought to:

- Estimate the wind speed caused by the Cyclone throughout the study area.
- Determine whether the extent and type of damage could reasonably be expected from the estimated wind speeds.
- Determine whether buildings that had been built in accordance with BCA96 [1] performed adequately.
- Determine whether buildings that had experienced TC Winifred in 1986 performed adequately in this event. This includes buildings that had performed adequately under the loads in TC Winifred, and those that needed some repair after TC Winifred.
- Document types of construction that appeared to be more vulnerable to wind damage than others.
- Explain areas of more concentrated damage.
- Ascertain the adequacy of current codes and standards.
- Provide possible reasons for failures to damaged building components and where possible, provide recommendations for upgrading these details.

The study focused on housing, though some commercial, public buildings and sheds were also investigated.

## 1.2 Strategy

In order to achieve the objectives within the constraints of the rapid clean-up, the following strategy was adopted:

- A CTS investigation team was assembled comprising:
  - David Henderson, John Ginger and Cam Leitch from the CTS,
  - Geoff Boughton and Debbie Falck from TimberED Services,
  - Ray Loveridge from ABCB and
  - Dave Hayward from Timber Queensland.

- The first priority was to obtain information from detailed studies of buildings of interest before debris was cleared away. Buildings studied in detail included housing in the Innisfail Estate, East Innisfail, Belvedere and Kurrimine Beach, and some commercial buildings and industrial sheds. This study aimed at establishing the elements at which failure was initiated, and any factors that may have contributed to poor performance of buildings.
- Simple structures (mainly road signs) were investigated to estimate the wind field.
- Street-side assessments (Housing Surveys) were performed to set the damage investigations in a context of the extent of damage to each housing type. The CTS study team concentrated its assessments on Babinda, Mourilyan, Flyingfish Point, the Coconuts, Coquette Point, and parts of Innisfail Estate and East Innisfail. Street-side assessments of Innisfail, Kurrimine Beach and Cowley Beach were undertaken by an assessment team from Geoscience Australia, and the results were pooled to give a comprehensive view of damage throughout the study area.
- Street-side assessments of performance of light industrial sheds were performed in Innisfail and Wangan by CTS.

### 1.3 Early reports on damage in the study area

The Bureau of Meteorology published the following information on their web site in the day following the cyclone crossing the coast:

<b>Coastal Crossing Details</b>	
<b>Crossing time:</b>	The eye crossed the coast between 6:20am and 7:20am AEST Monday 20 March 2006
<b>Crossing location:</b>	Innisfail
<b>Category when crossing the coast:</b>	4/5 - subject to ground investigation
<b>Extreme values during cyclone event (estimated)</b>	
<small>Note that these values may be changed on the receipt of later information</small>	
<b>Maximum Category:</b>	4/5
<b>Maximum sustained wind speed:</b>	170/215 km/h
<b>Maximum wind gust:</b>	240/290 km/h
<b>Lowest central pressure:</b>	approx 940 hPa
<p>The worst building damage due to wind impacts was at Silkwood - 99% of houses damaged (southern eyewall passed over the area) - and Babinda - 80% of buildings damaged (northern eyewall passed over the area). 100% of crops were damaged in some areas. There was little impact from rain around the time of the crossing due to the cyclone moving so quickly.</p> <p>Larry was the first severe tropical cyclone to make landfall at a populated location on Queensland's east coast since Rona crossed near the Daintree River in February 1999. The most recent severe tropical cyclone to cross anywhere on the east coast of Queensland was Ingrid on 10 March 2005.</p> <p>Impact information provided by Queensland Department of Emergency Services and the Environmental Protection Agency.</p>	
<a href="http://www.bom.gov.au/announcements/sevwx/qld/qldtc20060317.shtml">http://www.bom.gov.au/announcements/sevwx/qld/qldtc20060317.shtml</a>	

In spite of the early indication of extensive damage in Silkwood, the specific objectives of the study related to more recent housing than that in Silkwood, so the



initial focus of the detailed investigations was at Innisfail Estate, Belvedere and Kurrimine Beach.

#### **1.4 Performance of Buildings during TC Winifred in 1986**

Category 3 Tropical Cyclone Winifred crossed the same area of the North Queensland coast in February 1986. Maximum wind gusts between 50 and 55 m/s at 10 m height in Terrain Category 2 (as per AS/NZS 1170.2 [2]) were experienced in the most severely affected areas.

Investigations of the damage to buildings by Reardon, Walker and Jancauskas [3] indicated that houses built to the provisions of the Queensland Home Building Code 1981 [4] generally performed well. Structural damage to newer homes could be attributed to the effects of topography on wind speeds. 'There is evidence that the full consequences for wind speed design of siting houses in very exposed positions on top of ridges, hills and bluffs is not fully recognised by designers.'[3].

The most common failure in older houses that predated the 1981 cyclone resistant requirements was loss of roof cladding, often with battens attached.

Reardon, Walker and Jancauskas [3] also highlighted the need to address the problems associated with

- Failure of guttering and awnings
- Failure of roller and tilting garage doors
- Water ingress through undamaged window and doors
- Corrosion of fasteners
- Poor performance of school buildings given that they are often used as emergency shelters

There are many similarities between the type and extent of damage to buildings that occurred during both TC Winifred and TC Larry.

## 2. Estimates of Wind Speed and Direction

In order to analyse the structural performance of the buildings, it is first necessary to estimate the wind field (i.e. wind speed and direction) in the study area. In particular, it is necessary to determine the relationship of the estimated wind speed to the current design wind speed.

The study area included settlements that were within the path of the eye, to the South of the eye and to the North of the eye. They included some beach-side settlements, and one at the foot of the Great Dividing Range. The peak gust wind speed and range of wind directions for each of these settlements will have been different even though they were caused by the same Tropical Cyclone.

A number of sources were used to estimate and verify peak wind speeds in the study area including:

- Advice from the Bureau of Meteorology.
- Advice from other investigators especially from Systems Engineering Australia, JDH Consulting, Geoscience Australia and Queensland Department of Public Works.

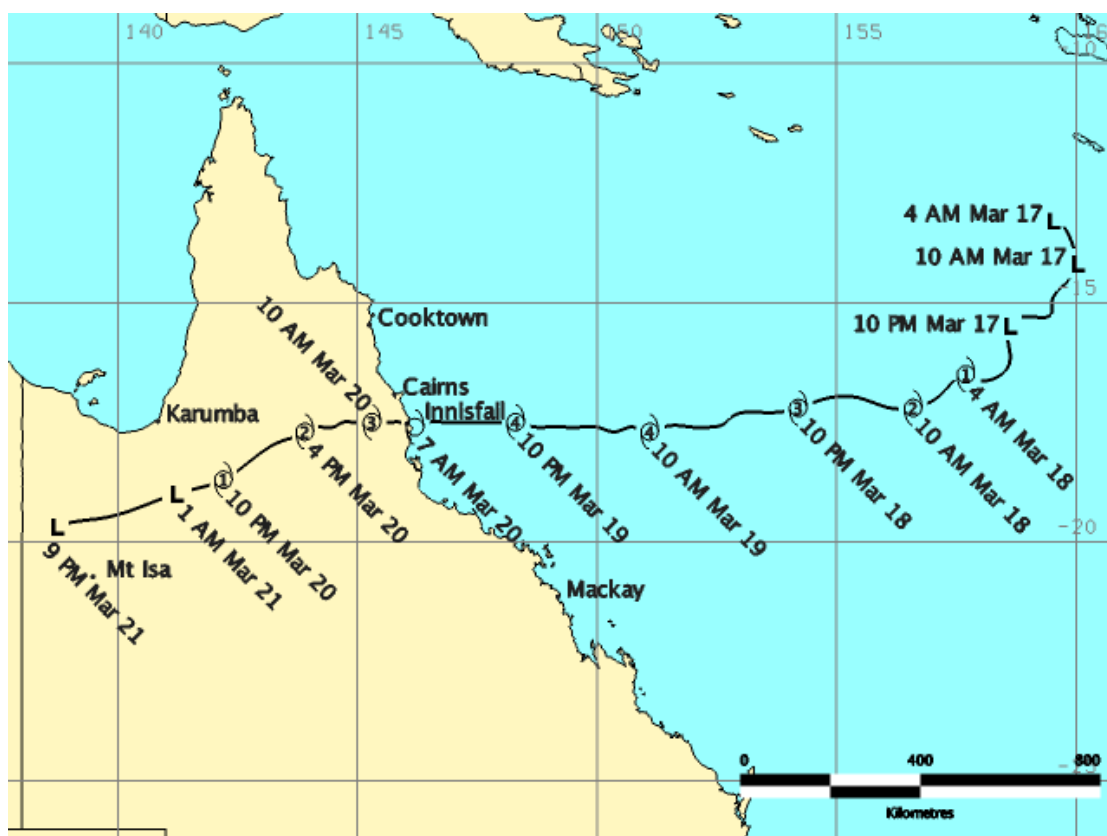
### 2.1 Characteristics of Tropical Cyclone Larry

Tropical Cyclone Larry formed in the Coral Sea about 1200 km east of Innisfail on 18 March 2006. The cyclone steadily intensified as it travelled in a westerly direction and possibly peaked as an extremely destructive Category 5 Cyclone before landfall. It crossed the Queensland coast about 10km ESE of Innisfail, at about 6.30 am on Monday 20 March 2006. The cyclone then travelled in a WNW track overland at a forward speed of about 30 kph with the eye passing directly over Innisfail. The eye of the cyclone had a diameter of about 25 km and measurements of central pressure and wind speed indicate that it was a Category 4 Cyclone at its landfall. Strong winds were experienced more than 50 km inland on the Atherton Tablelands. Figure 2.1 shows cyclone Larry's track obtained courtesy of the Bureau of Meteorology.

The Bureau of Meteorology cyclone categories are shown in Table 2.1.

**Table 2.1: Bureau of Meteorology Cyclone Categories**

Cyclone Category	Gust Wind Speed at 10 m height in flat open terrain			Central Pressure
	km/h	knots	m/s	
1	<125	<68	<35	990
2	125-170	68-92	35-47	970-985
3	170-225	92-122	47-63	950-965
4	225-280	122-155	63-78	930-945
5	>280	>151	>78	<925



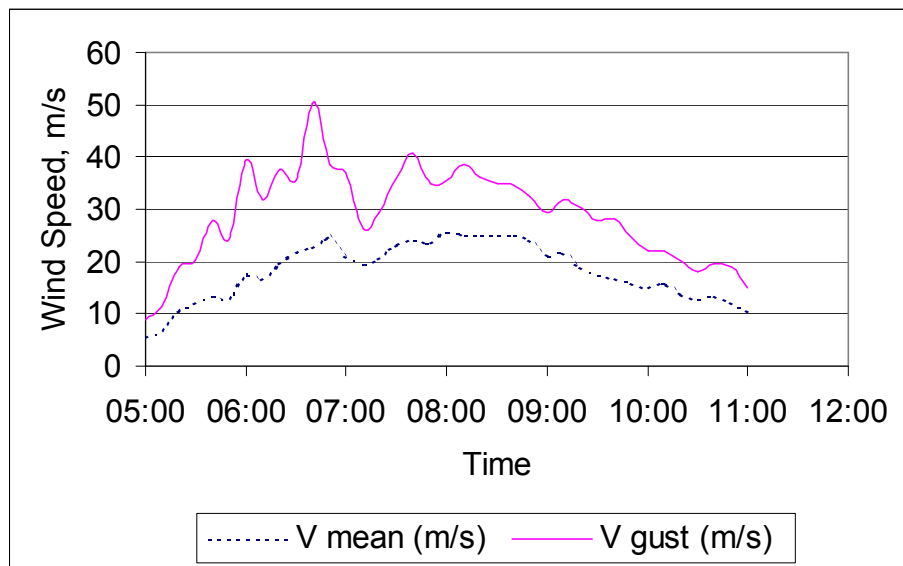
**Figure 2.1: Tropical Cyclone Larry – estimated track**  
(courtesy Bureau of Meteorology)

## 2.2 Wind speed estimations for the study area

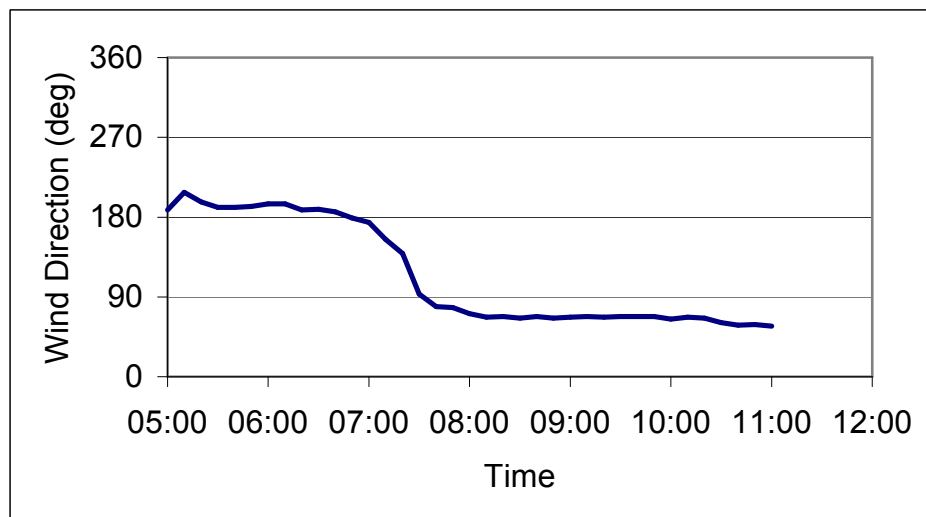
Wind speed, direction and barometric pressure measurements obtained from the South Johnstone AWS are shown in Figure 2.2. At this site, the approach wind direction changed from  $185^{\circ}$  (S) to  $80^{\circ}$  (E) with corresponding peak gust wind speeds of 51 m/s and 41 m/s respectively, and a minimum barometric pressure of 957 hPa. As the eye passed just to the North of the site, the pressure in the centre of the eye would have been lower than this value. The estimated extent of the eye is shown in Figure 2.3.

Based on evidence from damage to structures and vegetation, and discussions with residents, the strongest winds in Innisfail, were from the S to SSE and then from the W to NW following the passage of the eye, whilst the strong winds were SE to SSE in Kurrimine Beach and Mourilyan (in the areas to the South of Innisfail) and SSW to NNW in Babinda and Mirriwinni (areas to the North of Innisfail).

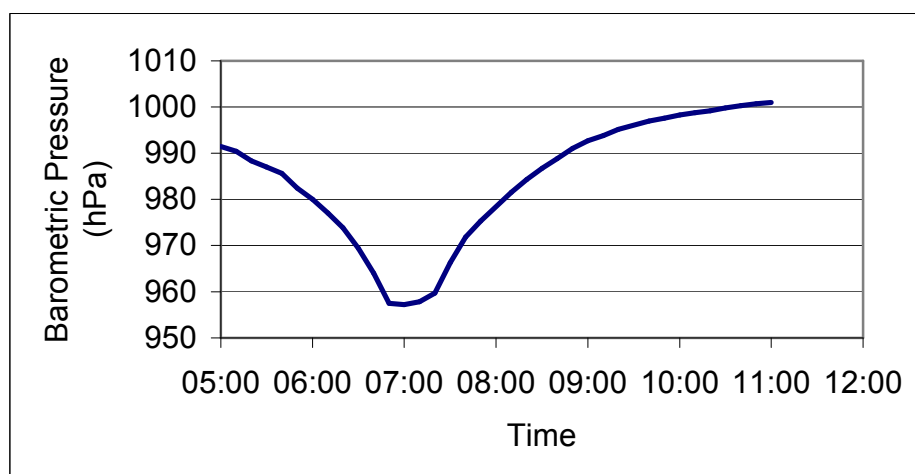
Maximum (10m reference height, in flat open terrain) gust wind speeds in these areas are estimated at between 50 and 65 m/s. Local wind speeds can be greatly modified by topographic features such as hills, escarpments and ravines etc. On a steep exposed hill, for example, the gust wind speed can be 50% higher than that on flat land. Damage to property, crops and infrastructure stretched from Cairns to the North, Milla Milla to the West and Cardwell to the South.



a) Wind speed

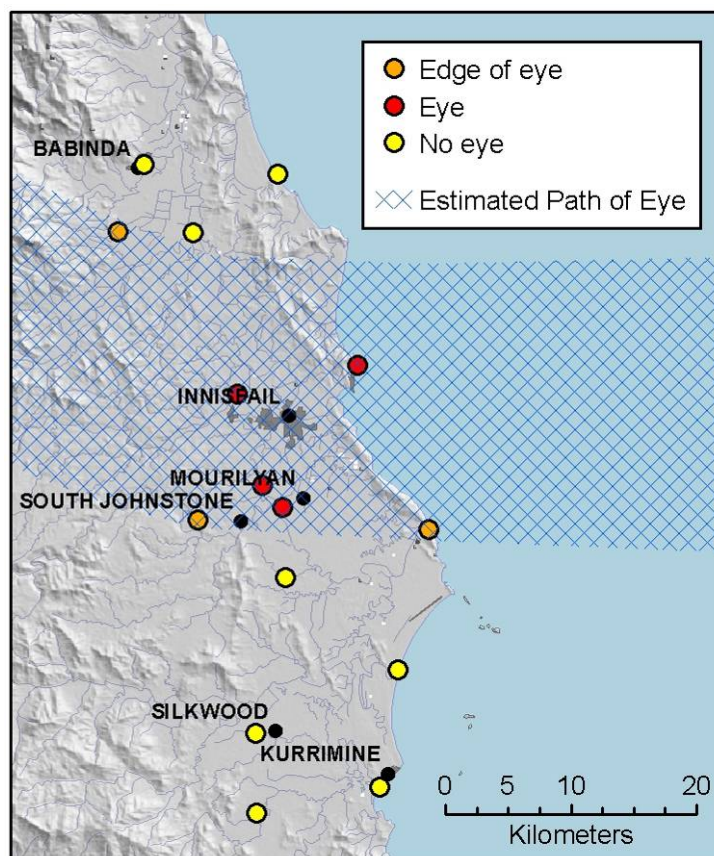


b) Wind direction



c) Atmospheric pressure

**Figure 2.2: South Johnstone AWS data**  
(courtesy Bureau of Meteorology)



**Figure 2.3: Estimated path of eye**

(Observations courtesy of Systems Engineering Australia and Willis Re Australia)

### 2.2.1 Analysis of simple structures

Within the scope of this study, some simple structures (i.e. road-signs) were used to estimate upper (U) and lower (L) bounds of peak gust wind speeds at different locations in the study area. These signs are generally flat plates that are attached to one or two cantilevered posts, and located in clear exposed approach terrain adjacent to the road. The wind loads acting on these plates can be determined and wind speeds deduced with confidence. Appendix A gives the theoretical basis for the analysis of these road-signs. Undamaged posts give an upper bound to the wind speed as the signs resisted the wind loads, while bent posts give a lower bound to wind speed as they failed during the event.

A large number of road-signs were examined during the study, and from these several were selected as providing the most reliable wind speed information. Figure 2.4 shows a bent-over road sign. Three point bending tests carried out on sample lengths of these posts give a plastic moment capacity of 4.1 kNm and a yield strength of about 420 MPa. The corresponding 10m height Terrain Category 2 (TC2) wind speed, to create the plastic hinge in the legs of that sign, is 43 m/s (L).

Figure 2.5 shows a map indicating locations of road signs selected for analysis (unambiguous terrain, simple topography, no shielding, no impact damage, etc), corresponding lower (L) or upper (U) wind speeds at 10m height in terrain category 2, in m/s, and the approach direction. This map indicates that the wind speeds are estimated within 50 m/s and 65 m/s in these regions.





**Figure 2.4: Lower bound wind sign**

### 2.2.2 Comparison of wind fields for Cyclones Larry and Winifred

The study area has been affected by a number of tropical cyclones in the past hundred years, but Tropical Cyclone Winifred (1986) is the most recent cyclone to have caused significant damage to the same area as Tropical Cyclone Larry. A number of buildings affected by TC Winifred were repaired and were in service during TC Larry. As the report draws some comparisons between the performance of these buildings, Table 2.2 compares the events and their wind effects.

**Table 2.2: Comparison of Cyclone Winifred and Cyclone Larry**

(Estimated) features	Cyclone Winifred	Cyclone Larry
Category after landfall	3	4
Forward speed	15 km/hr	30 km/hr
Eye diameter	50 km	25 km
Centre of eye	Cowley Beach	Flying Fish Pt
Direction of travel	Due west	WNW
Wind gust speed and direction at Innisfail	45 – 50 m/s south-west to north-west direction	55 – 65 m/s south to north west direction
Wind gust speed and direction at Kurrimine Beach	50 – 55 m/s southerly to northerly direction	55 – 65 m/s south east to north direction

Table 2.2 shows that Tropical Cyclone Larry was a little more severe, with wind speeds around 15% higher. As the wind load is proportional to the square of the wind speed, it is estimated that the wind loads applied by TC Larry on a regional basis were around 25% higher than those experienced in TC Winifred.

For a given site, the same may not be true as differences in wind direction may give shielding or topographic acceleration that may increase or decrease the wind speed at that site relative to the regional measurements. However, this report finds that the type of damage observed in TC Winifred is very similar to the type of damage observed in TC Larry.

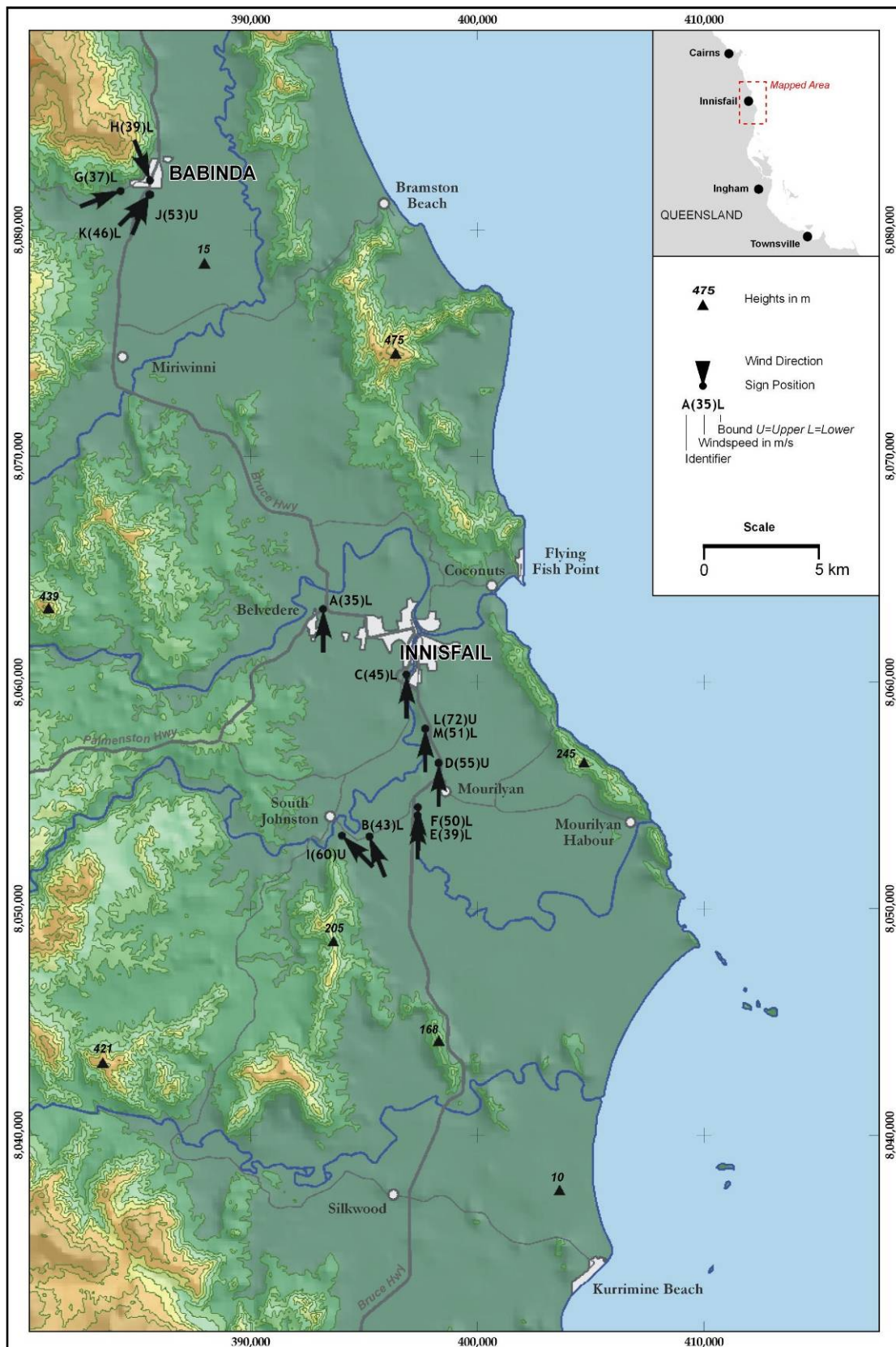
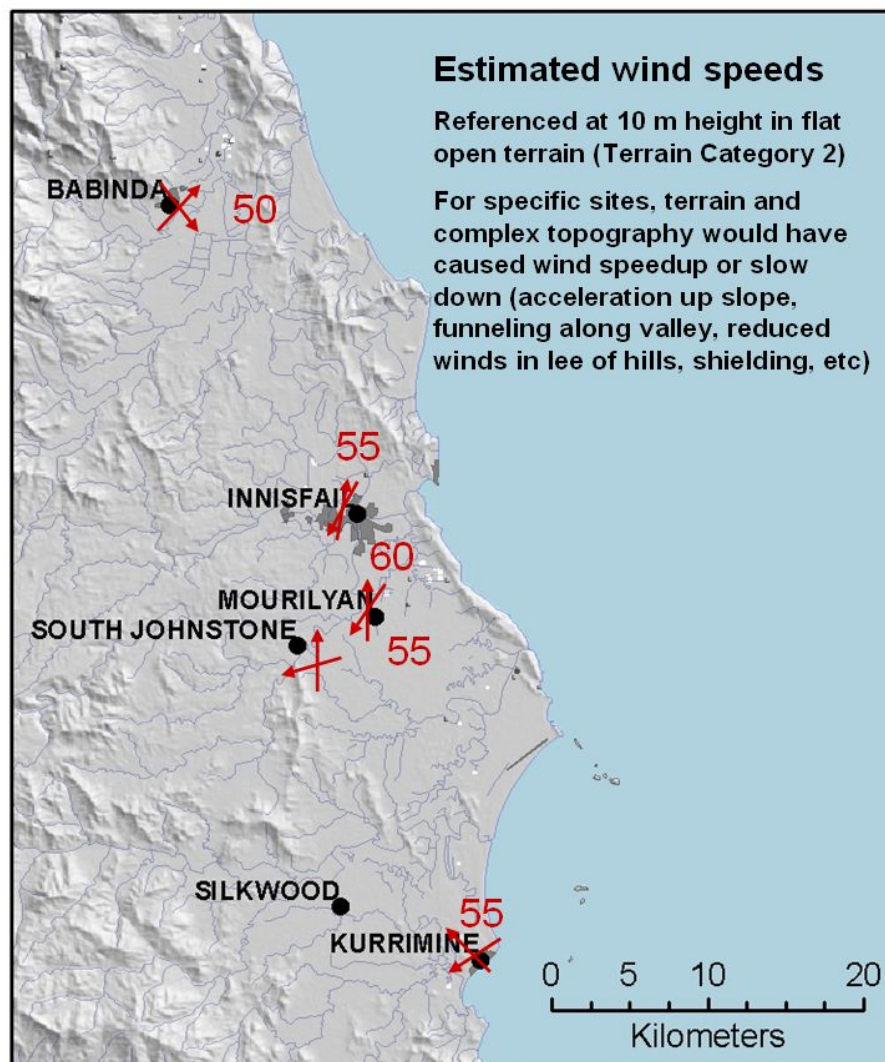


Figure 2.5: Upper and Lower wind speeds based on analysis of road-signs

The Bureau indicates that the steep mountain range adjacent to the Innisfail region has resulted in a complex wind field for Tropical Cyclone Larry. These features and its influence on the cyclone caused significant variability in local wind speeds.

For a cyclone considered to have a small eye diameter, the estimated peak wind speeds did not appear to lose intensity rapidly with distance away from the eye especially on the southern side of the track. The building damage survey (Section 5) indicates that the peak wind speeds impacting at Kurrimine were in the same order as the wind speeds near Innisfail. The broad brush wind speed estimates shown in Figure 2.6 are based on the sign analysis and the estimates of building damage across the region.



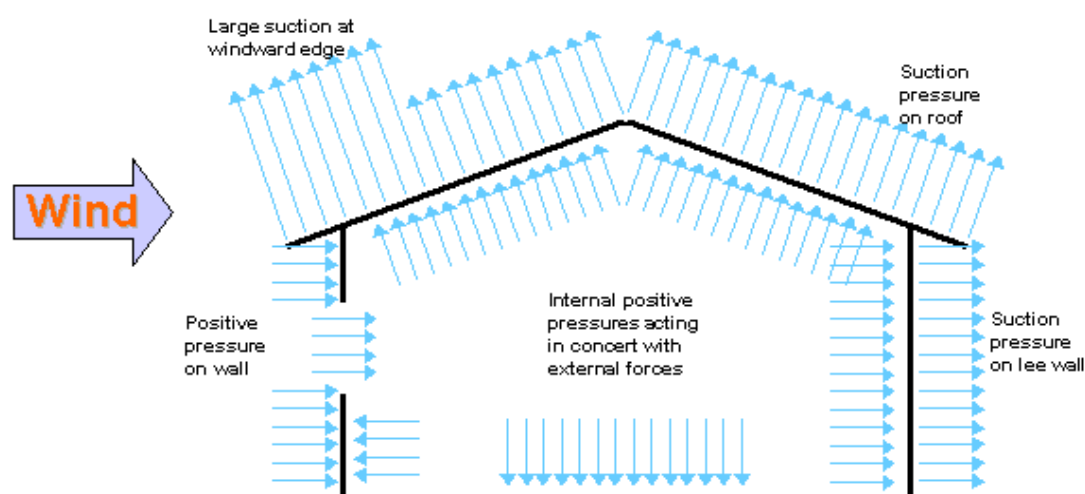
**Figure 2.6: Broad brush estimates of the peak wind speeds across region**

The maximum gust wind speeds (referenced to flat open country at a height of 10 m) in the study area were estimated at 50 m/s to 65 m/s. In the area covered by the eye shortly after landfall (including Flying Fish Pt, Innisfail, Mourilyan, Belvedere) the reference gust wind speed was estimated at 55 to 65 m/s. In the area south of the eye extending to South Johnstone, Silkwood and Kurrimine Beach, the same gust wind speed was estimated. In the area north of the eye from Miriwinni to Babinda, the peak gust wind speed was estimated at 50 to 55 m/s.



### 3. Wind Loading on Buildings

The roof of a house is subjected to external suction wind pressures. During a severe cyclone event a typical truss or rafter support needs to be able to resist forces pulling up on it equivalent to the weight of a small car. Figure 3.1 is a representation of the pressures acting on a house showing the high suction pressures at the leading edge of the roof. If there is a breach in the building envelope on a windward face such as from a broken window or failed door, the interior of the house is suddenly pressurised. These internal pressures act together with the external pressures greatly increasing the load on the cladding and structure.



**Figure 3.1: Wind forces with a dominant opening in windward wall**

#### 3.1 Wind Loading Design Considerations

The Australian Building Codes Board (ABCB) publishes the Building Code of Australia (BCA) [1] which stipulates design considerations for the majority of buildings in Australia. These requirements are met by compliance with a range of Standards relating to building construction (e.g. AS/NZS1170.2 [2]). Codes and standards have been used in the design and construction of engineered structures in Australia, for several decades.

Houses in Townsville and Darwin suffered significant damage during Cyclone Althea and Cyclone Tracy, respectively in the 1970s. This precipitated the development of the Home Building Code of Queensland [4] as an Appendix 4 to Standard Building by-laws, which was in widespread use by the mid 1980s. This required homes to be categorised by site design wind speed at eaves height, and it contained deemed to satisfy detailing for the different categories. Other related standards, such as wind loads for housing AS4055 [5], and residential timber framed construction for cyclonic regions AS1684.3 [6], are used in more recent housing design and construction.

Innisfail is located in Cyclone Region C as defined in AS/NZS1170.2 [2], where the ultimate limit state design wind speed (in flat approach Terrain Category 2) is 70 m/s. The design wind speed at the roof height of the building is factored to account for the

terrain, height and the topography. This factored design wind speed impacting on the building can be related to the pressures exerted on its elements through a series of coefficients defined in the wind loading standard, AS/NZS1170.2 [2].

AS4055 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 (i.e. non-cyclonic or cyclonic) and terrain, topography and shielding at the site. In cyclonic region C, classifications C1, C2, C3 and C4 represent increasing design wind speed. For instance a C1 classification represents a site with terrain category 3 exposure (suburban housing) with full or partial shielding on flat land. A C4 classification represents a site with terrain category 2 exposure with partial or no shielding at the top of a hill with a slope of 1:3 or more. The ultimate limit state wind speeds at roof height (8.5m or less) for C1, C2 C3 and C4 classifications are 50, 61, 74 and 86 m/s respectively, which includes the effect of wind speed-up over steep topography and terrain category (roughness). In addition, full internal pressurisation is implicit for ultimate strength limit state design of houses in cyclone regions. An under-classification of a site (i.e. C2 house in a C4 site) can result in very inadequate design details in the house. Furthermore, sites located on very steep hills (i.e. slope greater than 1:3) and open approach terrain (i.e. ocean) are not classified in AS4055, and hence houses on such sites must be designed according to the more detailed AS/NZS1170.2.

For timber framed housing, the housing construction methods specified in AS1684.3 are based on the design wind load data given in AS/NZS1170.2 and AS4055. For each classification C1 to C4, AS1684.3 gives design (uplift) wind load on roof battens and roof framing for some typical batten and frame spacings. In addition, AS1684.3 also gives 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 [7] and Domestic garage doors AS/NZS 4505 [8] use the design wind speeds and classifications given in AS/NZS1170.2 and AS4055 to specify design requirements for roller doors and windows respectively.



## 4. Building stock in study area

This section contains a short summary of the character of the buildings in the study area. It describes typical characteristics of buildings for each broad grouping.

### 4.1 Housing

Towns have 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. Houses also have varying degrees of exposure to wind forces, with those dwellings located in a suburban environment gaining shelter from surrounding structures as distinct from those exposed houses near the sea or in open terrain. Topographical features such as hills can concentrate or divert wind flow. Wind speeds impacting on a community will vary according to a tropical cyclone's intensity, size and distance from the community. Therefore an assessment of the wind resistance of housing requires knowledge of house types and their distribution throughout the community. Many houses were studied in the course of the investigation. They are classified by estimated age of original construction as summarised in Table 4.1. Many have undergone refurbishments at different times.

**Table 4.1: Age categories used for housing**

Age class	Features
Pre 1950 (Old Queenslander)	Early homes had high pitched often pyramid shaped roof. Later roof lines became more complex with multiple hips and gables, weatherboard external cladding
1950s to mid 1960s	Smooth internal linings, high ridge, irregular floor plan
Mid 1960s to early 1980s	Simpler rectangular floor plans. Extensive use of fibre cement external cladding on high set houses. Low to flat pitched metal roofs
Contemporary	Reinforced concrete blockwork for low set houses. Med to high pitch trussed roofs. Modern style trim, fascias, cladding, etc.

#### 4.1.1 pre 1950s (Queenslander)

Houses built in the early part of the century were considerably smaller than current ones. A common style was to have a central square core with verandahs on two or three sides. The entire house including verandah was supported clear of the ground on stumps.

The roof of the core is high pitched and often pyramid shaped with no ridge-line. Roof framing consists of rafters spanning from the top plates of the core walls to the apex or ridge. A king post typically provides the support from the apex down to ceiling joists or hanging beams. The roofs of the verandahs typically have a lower pitch.

In considering the total housing stock, there are few of these older houses left in original condition. It is often argued that the remaining ones must be the strongest as they have resisted a number of cyclones and, as they have a high market value, are generally kept in good condition.

From the 1930s to the 1950s, houses became larger, but the construction technique remained much the same. They were no longer square, or even rectangular, in plan which resulted in complex roof shapes with multiple hips and gables. Their construction included mortice and tenon wall frames with bearers bolted to stumps as

previously mentioned.

Some houses have cyclone rods, but mainly in the corners. It is not uncommon to have the eaves vented by using timber slats as opposed to eaves lining used in later construction.

Because of the portability of these houses, a number of them have been transported to new sites within the study area. Thus, houses which may appear to have been built in this era, may have been erected on the current site in the past few years. Also, many of the older houses have been extended and renovated. In some cases, changes were non-structural, such as replacement of roofing, inclusion of flat linings, and installation of new kitchens and bathrooms. In others, considerable structural upgrading was undertaken.



**Figure 4.1: An example of pre 1950s house (with metal wall cladding)**

#### **4.1.2 1950s to mid 1960s**

In these houses cyclone rods are typically present in perimeter walls at about 3 m spacing. Alternatively a specific number of rods were stipulated for a house. Sometimes the rods were extended to over-battens, but the holding nuts interfered with the roofing and hence were often embedded in the batten, weakening it severely.

These houses still had large often irregular floor plans. Their roof structure generally featured a high ridge-line, and weatherboards were often used as external wall cladding. Mostly these houses were high set, with sufficient room underneath for some habitable rooms.

Again, many of these houses had been renovated in the study area. Often windows were replaced and in some cases plasterboard was laid over fibro or hardboard internal linings. In some cases, renovation included some structural changes.



**Figure 4.2: An example of housing of the 1950s**

#### **4.1.3 mid 1960s to early 1980s**

In this period, there was significant pressure to reduce construction costs, and as a result, buildings became simpler, smaller and more streamlined. A typical house of this period was of rectangular shape, timber framed, elevated on stumps about 2.5 m high, with external walls clad with fibre cement or timber weatherboards and internal lining of either hardboard or plasterboard. The roofing was usually metal sheeting on a relatively low to flat pitch.

This period saw the introduction of single storey brick veneer construction again with the roofing typically metal sheeting on a relatively low to flat pitch.

Simple joints were used in the frame construction, and tie-down rods were installed regularly in these houses. The very low roof slope precluded the use of tiles, and roof slope was often achieved with graded purlins, which are deep purlins at the centre of the roof, grading down to battens at the edge.

Later in this period, Tropical Cyclone Althea (Townsville 1971) and Tropical Cyclone Tracy (Darwin 1974) had heightened awareness of the need to build wind resistance into the house structure.

Metal framing was being introduced at the very end of this period. Externally, a metal framed house looked the same as a timber framed house.



**Figure 4.3: An example of housing of the mid 1960s**

#### **4.1.4 Contemporary**

The Queensland Home Building Code (HBC) [4] was introduced in 1982. It was formulated because of the extensive damage to housing caused by Cyclone Tracy and to a lesser extent by Cyclone Althea, and the obvious need to provide adequate strength in housing. By 1984 it is reasonable to consider that houses in the cyclone region of Queensland were being fully designed and built to its requirements.

According to AS4055, houses in Cyclone Region C (which includes Innisfail) are designed to withstand a gust wind speed of 70 m/s (measured at 10 m height in open terrain), equivalent to a mid range category 4 cyclone. Furthermore, the application of high internal pressures resulting from a dominant opening is implicit in the design.

##### *Contemporary elevated*

Elevated housing lost some popularity in North Queensland at about this period, and it still had a more basic rectangular floor plan, but was more likely to have a steeper roof made with trusses, as shown in Figure 4.4.



**Figure 4.4: An example of contemporary elevated housing**



*Contemporary slab on ground*

The predominant regional building style in this period was single storey construction with a truss roof of low to high pitch with metal roof cladding. The external wall construction is typically reinforced masonry block or brick veneer construction. Throughout the study area, single leaf masonry block seemed more popular.

As opposed to the previous three house type groupings, there is a greater mix of the hip and gable roof shapes with a combination of both being common. This allows larger and more complex living areas, but these designs need large girder trusses with an associated increase in the truss hold down capacity.

In the study area, a number of these houses had tiled roofs as shown in Figure 4.5. In cyclonic regions each tile should be clipped independently to the battens.



**Figure 4.5: An example of contemporary low-set housing**

## **4.2 Public, commercial and industrial buildings**

Public, Commercial and Industrial buildings in Cyclone Region C are designed to wind speed specifications according to the wind load standard AS/NZS 1170.2 [2]. However, the design internal pressure used is dependent on the designer's assessment of the occurrence of a dominant opening. For instance, the failure of a roller door in a building designed as nominally sealed with low internal pressure is vulnerable to further damage as a result of high internal pressures caused by the dominant opening.

“Public Buildings” are those where people may congregate, such as schools, churches, libraries and shire halls. “Commercial Buildings” are those where a business undertakes a commercial trading activity with the public, such as shops, showrooms and offices.

However, it is important to determine the structural system used for these buildings, as this best categorizes its response to the applied wind loading. Note that the different classes of building can use the same structural system. For this report, the classification of the main structural systems used is; Steel Frame Buildings, Timber Framed Buildings, or Masonry Buildings.



**Figure 4.6: Timber frame commercial buildings**

#### **4.2.1 Steel framed buildings**

Contemporary industrial buildings are generally open plan, steel framed, metal clad buildings with spans ranging from 8 to 20 m, length 20 to 50 m and height 5 to 8 m typically used for light industrial workshops, storage, and farming applications. The roofs are generally gable ended and low pitch (less than  $10^\circ$ ). The frames are evenly spaced at intervals of about 6 m with roof and wall cladding attached to purlins and girts respectively, spanning across the frames. Cladding profile is either corrugated or rib-pan trapezoidal. Large roller doors or hinged doors are installed on the walls, and glass windows or louvres are installed in some buildings.

For this report, the contemporary industrial buildings are split into two groups, based broadly on the type of steel section used for the main framing members (e.g. column and rafters to the portal frames).

The first group, using the more traditional hot-rolled steel framing members are called “Hot Rolled Industrial Buildings”. The performance of these buildings will be similar to the Hot Rolled Steel Frame Public and Commercial buildings. The second group comprises the lighter (that is less mass of steel) sheds that use light gauge cold formed steel sections (typically “C” sections) for the main structural framing elements and are called “Cold Formed Industrial Sheds” in this report. Table 4.2 summarizes these two groups of industrial buildings.



**Table 4.2: Typical Details for the two defined Types of Industrial Buildings**

Item	Typical Details for Each Building Classification	
	Hot Rolled Buildings	Cold Formed Sheds
<b>Structural Framing</b>	Hot-rolled steel sections (UB, UC or RHS members)	Light-gauge cold formed sections (C or Z members)
<b>Frame Connections</b> (Column to rafter, Ridge joint)	Welded connections or bolted connections using thick steel plates (typically 10 mm thick)	Bolted connections, often using folded light gauge brackets (typically about 2 mm thick)
<b>Purlins and Girts</b>	Continuous lapped Z sections, fixed to the frames using bolts through the web of the section.	Three methods used: - Continuous Z sections - Simple span C sections - Top hat sections Purlins and girts often fixed to the frames through the purlin/girt flanges.
<b>Bracing</b>	Typically SHS or CHS struts with crossed tension bracing	Crossed tension bracing using light gauge straps. Often rely on end bay purlins acting as struts to support the top ends of wind columns (end wall mullions).

**Figure 4.7: Example of a cold formed steel frame industrial shed**

## 5. Patterns in damage

Damage surveys can show trends that can be used to suggest improvements in the resistance of buildings to future events. In this section, the results of street surveys are used to draw conclusions about the general features of buildings that proved more susceptible to wind damage during Tropical Cyclone Larry.

### 5.1 Estimations of extent of damage to housing

The street survey damage classification system was based on the one developed by Leicester and Reardon [9] for Darwin after cyclone Tracy, and has been used for all CTS damage investigations since then. It ranks the amount of visible structural damage with the categories ranging from negligible or non-structural damage such as loss of guttering or flashing to the extreme loss of all walls and roof structure. This level of survey is aimed to cover most houses in the affected areas and give an indication of the percentage and degree of damage that occurred.

The damage categorisation system relates only to structural damage visible from outside the buildings. The lack of dense gardens or fences typically allowed the observation of the two side-walls and front facing wall. It is likely that some lower level damage such as debris impact or even damaged roofing would have been missed. Therefore the survey results should be taken as being indicative rather than definitive of the damage trends. Definitions of the damage categories are given in Table 5.1.

As only one damage category is allotted to each house, the most severe one is reported. If a house has sustained impact damage (2) to a windward wall and also suffered loss of some roofing battens (3), only the damage category of 3 is presented in the data summary.

**Table 5.1: Housing Survey Damage Index**

Index	Description of damage	Damage Cost Index
1	No or Negligible damage (includes possible water ingress)	0.05
2	Missile/debris damage to cladding, gutters or windows	0.2
3	Loss of up to half of roof sheeting (including battens that were often still attached to the sheeting)	0.5
4	Loss of more than half of roof sheeting (including battens that were often still attached to the sheeting)	0.7
5	Loss of roof structure (includes loss of sheeting, battens, rafters, struts, and top plates)	0.9
6	Loss of up to half of walls	1.0
7	Loss of more than half of walls	1.0

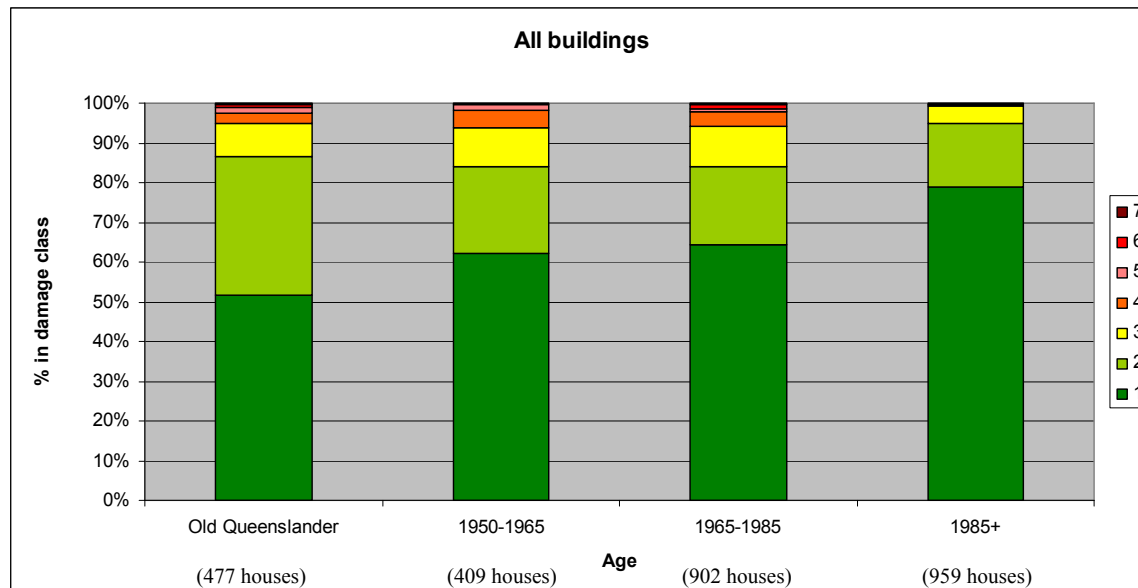
The Damage Cost Index shown in Table 5.1 is a rough estimate of the cost of repairing the observed damage expressed as a fraction of the value of the whole house. For example, it is estimated that to replace the entire roof structure and repair all of the consequent water damage to the rest of the house would cost around 90% of the value of the house. The values were obtained through discussion with members of the insurance industry and others involved in the costing of repairs after this event. For Damage Index 1 (DI-1) – Negligible damage – it was recognised that water ingress may contribute to the damage bill, even though no direct building damage had

resulted. A notional cost of 5% was included for even negligible damage to cover the cost of repairing water damage to linings, wiring, floor coverings etc.

This study was differentiated from the studies reported in Section 6.1 by the following:

- An attempt was made to classify the age and type of housing as described in Section 4.1. This would enable extent of damage to be evaluated separately for each type or age of house.
- Buildings that showed no sign of damage were included in this survey along with buildings that were obviously damaged.
- The survey data was collected by viewing the houses from the street. In some cases, damage could be seen by looking at the front of the building. In other cases, it could be inferred from the kerb-side collection piles.
- In some cases, the extent of damage to the roof was estimated by the amount of roof covered by tarpaulins, and the amount of debris around the property.
- It was often not possible to determine if there was any water damage to the inside of the house. (However, in some cases, it could be deduced from the material awaiting collection and in others, residents provided this information.)

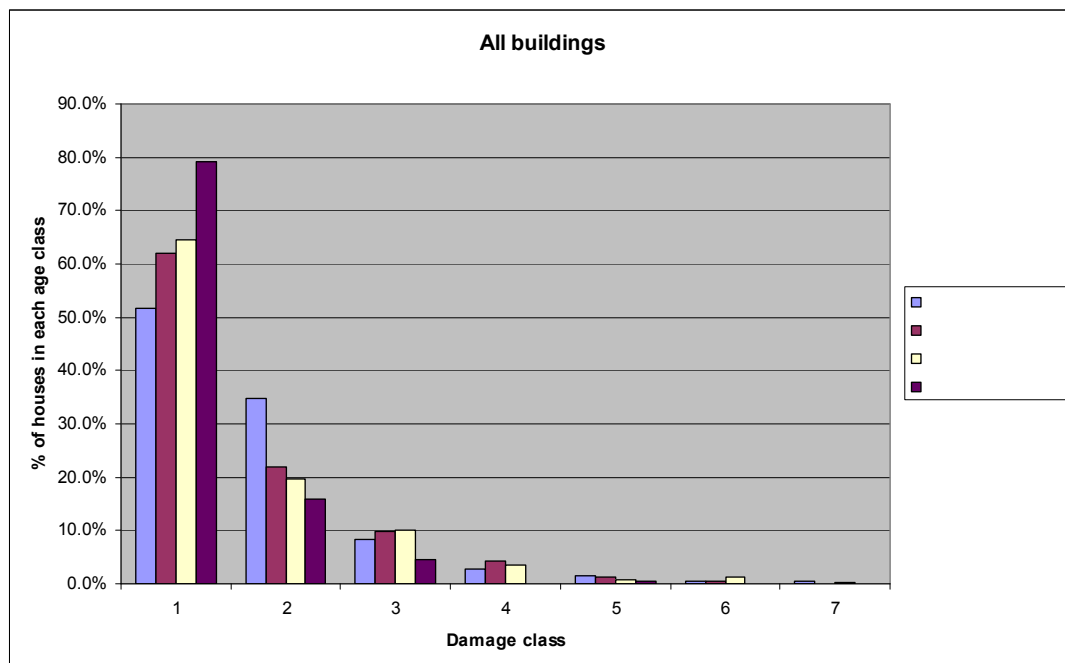
The results are presented as Percent of housing damaged vs Damage Index. Figure 5.1 presents this data for entire sampled housing stock (sample size 2747 houses), but subdivided amongst the four age classes of housing stock. It can be seen that overall, the more recent housing (1985+) had a much higher percentage of negligible damage (DI-1) than other age groups. This is reflected in the higher bar corresponding to damage index 1 for the 1985+ class compared with the other classes.



**Figure 5.1: Percentage of Houses vs Damage Index (All houses sampled)**

The CTS teams surveyed parts of Babinda, Kurramine Beach, East Innisfail, Innisfail Estate, Belvedere, Mourilyan, Flying Fish Point, the Coconuts, and Coquette Point. The street surveys were conducted in collaboration with Geoscience Australia survey teams using the same damage classification system. Approximately 50% of the street survey data analysed was collected by the CTS teams with the other 50% collected by GA teams.

Figure 5.2 shows similar data across the whole damage survey area. However, in examining each damage class, it can be seen that the recent housing performed best when compared with other age groups for each of the damage classes.



**Figure 5.2: Percentage of Houses by Damage Index (All houses sampled)**

The complete data contained information from areas with very different characteristics. For example:

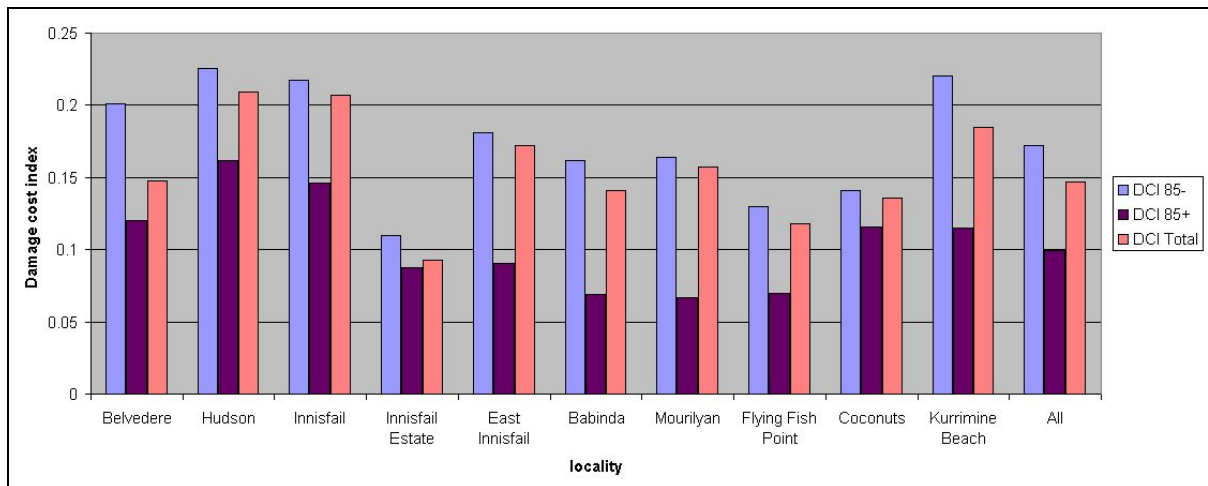
- East Innisfail has few houses in the 1985+ category, and it is characterised by variations in topography.
- Innisfail Estate has few Old Queenslanders, and mainly recent housing, but is sited on relatively flat topography.

If these two suburbs were the only ones viewed, then the conclusion could be drawn that modern housing performed better, but it may be equally valid to conclude that complex topography was the main influence in the performance of buildings. In order to separate any effects of location from the effects of building age, all of the localities could be considered separately. The findings from this analysis are presented in Figure 5.3 where the performance of each age group was reported as a single bar – a damage cost index. The damage cost index for any one house was derived from the damage index as shown in Table 5.1.

The final outcomes were not particularly sensitive to the actual values of Damage Cost Index chosen for each Damage Index. The damage to a particular age class in a given locality can be aggregated between all of the buildings and a weighted average found. This is the value shown in Figure 5.3.

Figure 5.3 shows that in comparing contemporary housing (DCI 85+) with housing that was built before the Home Building Code (DCI 85-), the contemporary housing has the lowest Damage Cost Index.



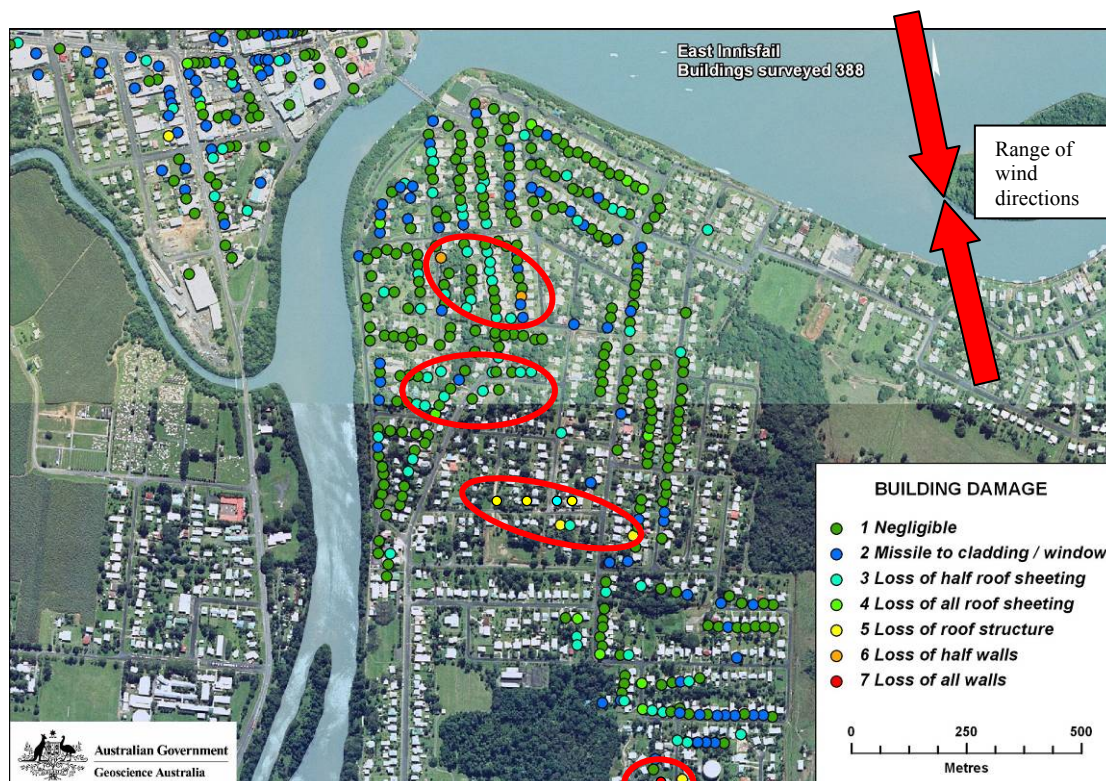


**Figure 5.3: Damage cost index by age and locality**

There were some areas that sustained more severe damage than surrounding areas. These areas were investigated separately to determine common features of the damage.

### 5.1.1 Houses on hilltops

Figure 5.4 shows the damage index of surveyed buildings in East Innisfail superimposed on aerial photos taken prior to the cyclone. It shows that the damage to houses in East Innisfail appears to be concentrated around four hill tops (highlighted with red ovals). Many of the damaged houses pre-dated the 1981 Building Code [4] changes, but there was a noticeable difference in performance of hill-top houses with nearby houses of similar age in flat topography. A few recently constructed hill top houses also sustained significant damage.

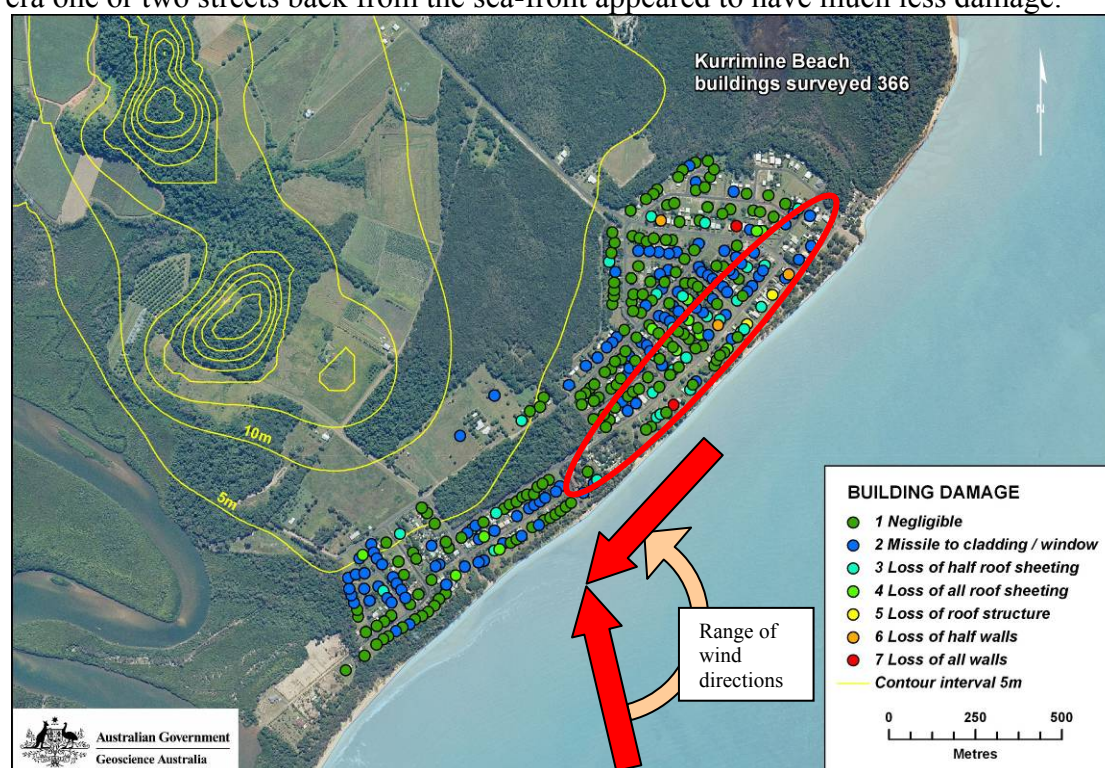


**Figure 5.4: Damage map for Innisfail East**  
(Map courtesy of Geoscience Australia)

There were also a number of hill top houses at Coquette Point and Flying Fish Point. Some of these houses sustained the highest damage in the area. In both cases, more recent hill-top houses sustained little structural damage except for that caused by airborne debris.

### 5.1.2 Kurrimine Beach – sea-front houses

Many of the older houses along the ocean-front at Kurrimine Beach performed poorly, and a significant percentage sustained severe damage (Damage Index 5-7 – shown in Figure 5.5). The debris from these homes contributed to the failure of houses in adjacent streets. The upwind terrain during the early part of the cyclone was ocean, then a few coconut trees. The trees offered no shielding, but some of the failures in the first row of beach-front houses were initiated by coconuts or palm fronds breaking windows, producing internal pressurization. Where inadequate batten to rafter or rafter to top plate connections failed, significant roof loss occurred. Houses from a similar era one or two streets back from the sea-front appeared to have much less damage.



**Figure 5.5: Damage on Map of Kurrimine Beach**  
(Map courtesy of Geoscience Australia)

No sea-front houses were damaged by storm surge. The maximum sea level was at least 0.3 metres lower than the beach-front houses. The damage was due only to wind effects.

### 5.1.3 Babinda, Mourilyan, South Johnstone and Silkwood

These four townships had a high percentage of older building stock.

Cyclone Larry caused structural damage to many houses and commercial buildings throughout the town of Babinda. While some buildings sustained very severe damage, there were no pockets in which many buildings were affected while similar buildings nearby were undamaged. Again, older houses generally performed poorly. Topography



in the Babinda area is fairly complex with high mountains rising directly to the west of the town and ridges from the mountains extending into the outer edges of the town.

The wind speeds at Mourilyan, South Johnstone and Silkwood were assessed as being similar to those at Innisfail, and the topography in Mourilyan was relatively simple. Topography was complicated by river valleys for both Silkwood and South Johnstone, but many of the buildings were on higher but flat land away from the rivers. Damage to the older building stock in these towns was significant. The damage was at a similar level to that of similar aged buildings elsewhere, but the higher concentration of older buildings gave the towns a significantly greater percentage damage statistic than locations with a greater mix of building ages e.g. Kurrimine Beach.

## 5.2 Estimations of extent of damage to sheds

Detailed surveys and street side surveys were conducted of industrial sheds. Findings from the detailed inspections are presented in Section 6.2 .



**Figure 5.6: Hot rolled steel framed shed**

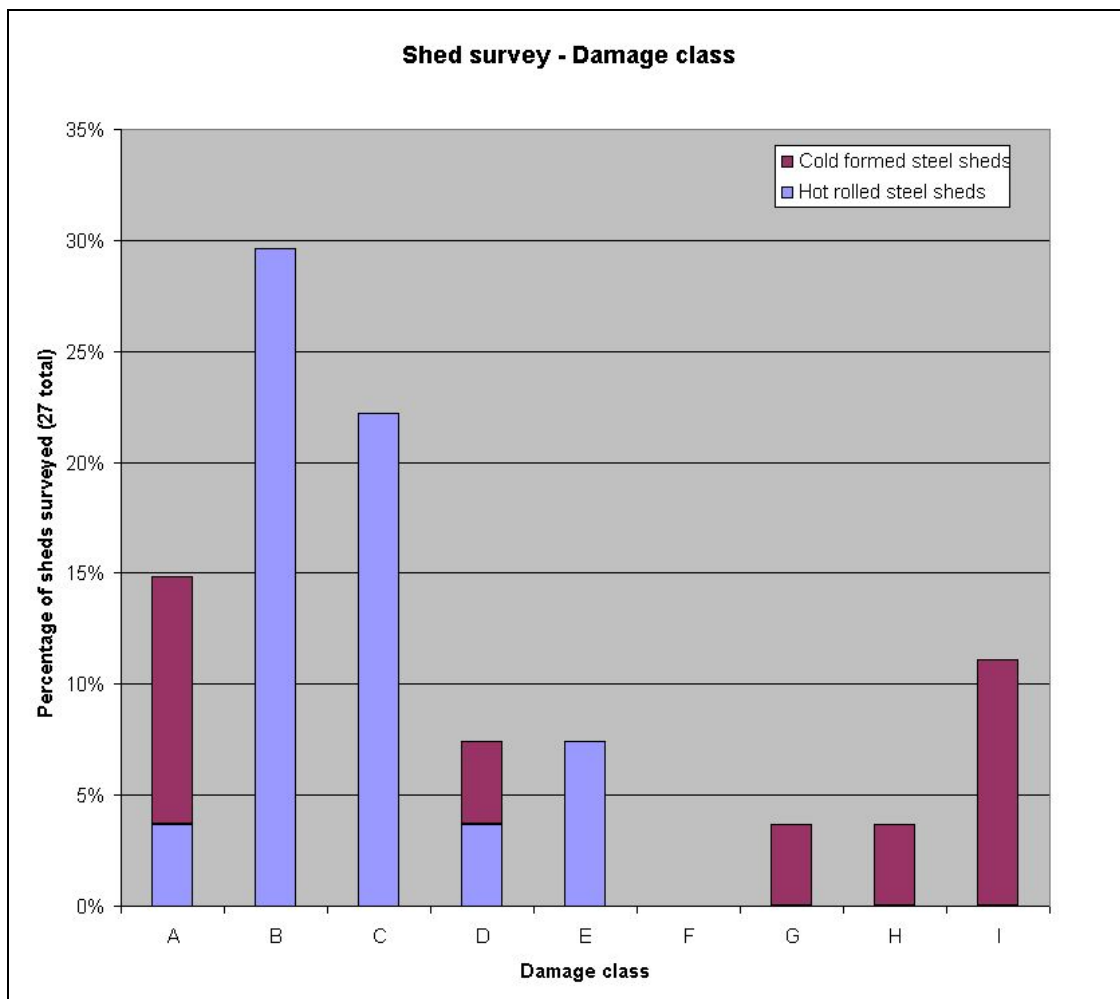
A street side assessment used the damage classes as given in Table 5.2. Although more than 40 sheds were surveyed, only 27 are presented in Figure 5.7, as they are from roughly the same Innisfail area. This is to minimise potential bias from varying terrain, wind speeds, etc. As only one damage category is allotted to each shed, the most severe one is reported.

Approximately 30% of these engineered structures suffered loss of cladding through to complete collapse. This is at wind loads less than the region's design wind speed where no structural failures should be expected. From the street survey the major structural failures (G, H, I) were all in the cold formed steel frame sheds. This is consistent with all the other shed inspection data, where major structural failures were observed in either cold formed sheds (Figure 5.8) or old timber trussed sheds (Figure 5.9).

Of the sheds that had roller doors, 60% had failed doors, often causing additional internal damage and in some cases leading to structural failures of the shed.

**Table 5.2: Shed Survey Damage Classes**

<b>Class</b>	<b>Description of damage</b>
A	Negligible
B	Loss of flashing/guttering
C	Missile damage to bldg envelope
D	Loss of wall cladding
E	Loss of up to half roof cladding
F	Loss of up to all roofing
G	Buckled Purlins
H	Damaged or failed frame
I	Complete Collapse

**Figure 5.7: Street side survey of industrial sheds**



**Figure 5.8: Collapse of cold formed steel industrial shed**



**Figure 5.9: Loss of timber purlins and new cladding from old industrial shed**

### **5.3 Performance of Roller Doors**

A large proportion of roller doors failed in both housing and commercial buildings. In some cases, the internal pressurisation of the building as a result of a roller door failure precipitated further damage to the rest of the building. Roller doors could therefore be identified as increasing the vulnerability in most types of buildings.

In general, roller doors to all classes of buildings performed poorly.

#### **5.3.1 Roller doors in Commercial and Industrial Buildings**

Most of the roller doors fitted to the larger commercial and industrial buildings had a similar pattern of damage, with the roller doors becoming disengaged from the tracks



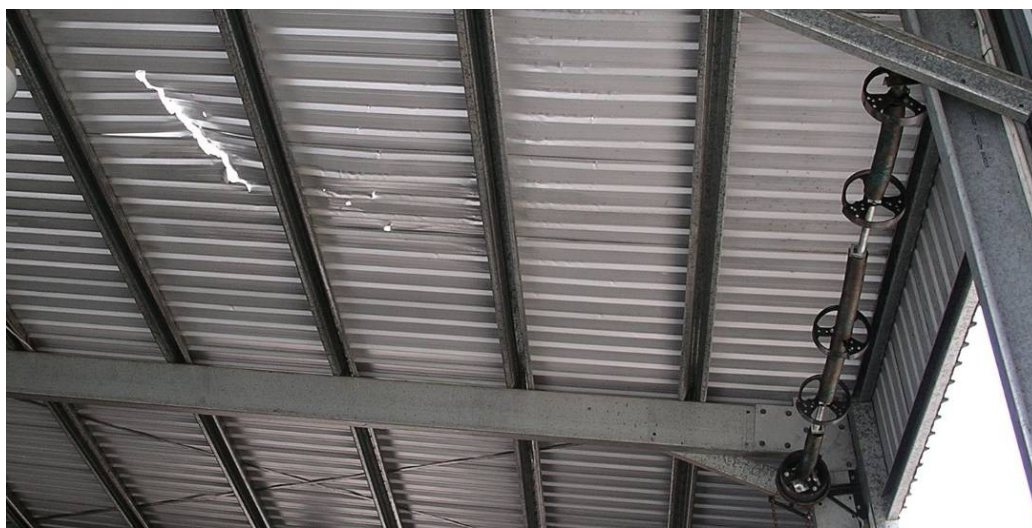
or the tracks disengaging from the walls. This failure pattern was observed for both of the common forms of roller door construction – linked slat type and pressed sheet type. In some cases failure of a roller door(s) on the windward wall was quickly followed by the failure of roller door(s) on leeward and side wall(s) as a result of internal pressurisation.

The ratio of the depth of the roller door slats or ribs to the span across the door opening is very small and so they are not very stiff when loaded with wind pressure (for either inwards or outwards loading). Consequently, the door midspan region undergoes large deflections when subjected to wind loads and this causes the ends of the slats or panel to be pulled inwards followed by the roller door failure when the sides become disengaged from the tracks. Figure 5.10 and Figure 5.11 show examples of roller door failure.

Roller doors can be designed with hooks fitted to each end of the slats (wind locks) so that when the slats are subjected to wind loading, they can use membrane action to support this pressure. However, the tensile forces generated in the slats by this membrane action can be quite large (especially if the deflection at the centre of the slat span is small) and so the tracks to each side of the slats need to be fixed very securely. Common failures associated with wind locks are shown in Figure 5.12.



**Figure 5.10: Torn sheet type roller door**



**Figure 5.11: Roller door curtain punctured roof cladding**



(a) Failed track bracket (b) Windlock torn from door (c) Wind lock in track

**Figure 5.12: Failures associated with wind locks**

### 5.3.2 In Housing

For housing, most of the roller doors observed had failed by the roller door becoming disengaged from the tracks. Failures of panel lift doors were also observed (Figure 5.13).



**Figure 5.13: Panel garage door failure**

On examining an apparently undamaged roller door, the tracks showed that they were starting to pull away from their fixings to the supporting masonry walls (Figure 5.14). This indicates that the membrane (tensile) force developed in the roller door was large enough to start pulling the track support fixings out of the wall.



(a) Twin doors

(b) Damage to track fixings

**Figure 5.14: Example of less obvious roller door failure in housing**



## 6. Performance of buildings

In this section, building performance is correlated to the age and type of the building. It draws on the street-surveys to indicate the extent of the damage, and on more detailed structural assessments that establish the reason for their failures.

### 6.1 Housing

It was very rarely that a reason for good or bad performance of housing could not be established.

- Good performance could be directly related to the use of appropriate details for the gust wind speeds. There were examples of where renovations had given an opportunity for structural aspects of older buildings to be upgraded.
- Poor performance could generally be attributed to the use of one or more inappropriate details for the site wind speeds. In some cases, deterioration due to lack of maintenance had rendered once good detailing, ineffective.

#### 6.1.1 pre 1950s

This housing had the largest range of performance which reflected the range of structural quality in the current building stock. Some had been structurally renovated, others had been given new linings and roofing, but the structure had suffered the effects of deterioration over time, without upgrading.

There were a number of examples of where failures could be attributed to deterioration of structural elements:

- In some cases, fasteners had corroded, rendering load paths ineffective, as shown in Figure 6.1.
- In some cases, timber had deteriorated so that connections were not effective as shown in Figure 6.2.



**Figure 6.1: Corrosion of Connections**

(Main photo shows floor to stumps anchorage. Inset shows a corroded roofing screw.)





**Figure 6.2: Deterioration of Timber**  
(Floor joist at external wall connection in refurbished house)

It was rare that the deterioration in timber caused the initial failure. In most cases, the simple lines of the roof structure on the “Old Queenslanders” made the roof structure reasonably water-tight, and deterioration of timber was at floor level. The deterioration caused wall and sometimes, floor failures to follow the initial roof structure loss.

Many failures could be attributed to the connections not having the capacity to resist the wind loads, even without deterioration. Common problems of this type included:

- Batten to rafter connection. (In a number of cases, the roofing had been replaced and stronger roofing connectors used compared with the spring head nails used during initial construction, however no attempt was made to increase the capacity of other elements in the roof structure.) Figure 6.3 illustrates this type of failure.
- Rafter to wall connection. Typical of this era, many roofs were anchored to the top of walls with skew nails. Many roofs were seen where the rafters were still attached to the walls with only skew nails even though the battens had separated from the rafters. However, the rafter to wall connection may have been ‘protected’ by prior failure of the batten to rafter connection. Had the roofing remained attached to the battens, the loading on the rafter to wall connection may have been in excess of its capacity. Figure 6.4, a typical pyramidal roof with “stick built” structure, illustrates this type of failure.

The most common type of failures on housing of this era was roof system failure, usually the batten-to-rafter connection as illustrated in Figure 6.3. It was primarily nailed connections that failed with mostly two nails per connection, but in a few older roofs, one 75 mm nail per joint. There were few failures that were associated with the originally installed roof. (Most roofing systems were screw fixed to the battens, and in many cases, the battens had been replaced at the same time, but fixed only with plain shank nails.)



**Figure 6.3: Failure of batten-to-rafter connection**  
(Refurbished roofing on an “Old Queenslander”)

During discussions with engineers, certifiers, and builders involved in the reconstruction following TC Larry, damage to roofs not apparent from ground level inspections has been reported as the rebuilding work continues. These “hidden” failures are where the nails have partially pulled out of the rafter leaving a separation between the battens and rafters in the pre-1985 homes. (A few cases of separation of top plate and studs were also noted.) The wind load has been sufficient to overload some of the nailed joints but the subsequent wind gusts (and short cyclone duration) have not been able to remove the roof. For the houses where a damage assessment has been conducted, broad estimates by the engineers, roofers, etc, of the number of houses suffering this “hidden” damage varies from 20 % to 70 %.

Batten to rafter connections are detailed in AS1684.3 [6] with simple details and design data in tables allowing the calculation of uplift force on a connection, and giving the capacity of a number of different connection details. Two deformed shank nails do not have sufficient capacity for edge zones in a metal deck roof, and plain shank nails are not even listed as options for use in the cyclone regions.



**Figure 6.4: Failure of rafter to wall connection**

Inspections were conducted on houses that had lost walls, as shown in Figure 6.5. When the roof had been lost, the collapse of walls occurred due to loss of support from the ceiling and roof structure combined in some cases with deterioration of the structure at floor level.



**Figure 6.5: Loss of walls following roof structure damage**

Many of the walls that had become detached from the house still had mortice and tenon framing. While some cases were found with window damage prior to roof loss, and wall failure after the roof loss, there were only one or two examples in which it was thought that the wall had failed prior to the roof loss due to deterioration of the structure.

There were only a very few cases in which these types of houses showed racking failures. In fact, in a few instances, large trees had fallen on “Old Queenslanders” and the walls had hardly deflected.

### **6.1.2 1950s to mid 1960s**

This type of housing had roof failures that were characterised by:

- Batten-to-rafter connection failures which allowed the roof to separate from the structure in large pieces. This was similar to the failures shown in Figure 6.3.
- Loss of the central part of the roof as shown in Figure 6.6. Generally these roofs were anchored with over-battens around the edge and relied on ridge beams and underpurlins for support of the rafters at the top and centre of each rafter run. So when these support points did not offer enough anchorage against the uplift forces, they failed, with sometimes leaving the outside connection of the rafters still attached to the wall, but the centre part of the roof missing.





**Figure 6.6: Loss of ridge beams**

The loss of roof beams, underpurlins and struts was seen in the “Old Queenslanders”, but the simpler roof lines of those structures meant that only a short section of ridge was missing. However in the housing of the 1950s and 1960s the longer roof lines meant that a substantial portion of roof was missing. Often such a large and heavy portion of roof did not travel far from the house it had detached from.

Failure at the batten-to-rafter connection remained the most common type of failure in this type of housing. Again, in most cases, the roofing was anchored with Type 17 screws indicating that at some stage the original springhead nails had been replaced. However, the batten-to-rafter connection had not been upgraded at the same time.

### **6.1.3 Mid 1960s to mid 1980s**

Figure 5.1 showed that housing of this era suffered the highest percentage of roof damage of all of the housing types.

The majority of housing in this category predated the revision of the Queensland Building By-Laws – Appendix 4 [4] which incorporated many of the lessons learned after Cyclone Tracy (Darwin 1974). However, they were only 25 to 35 years old and so not many had been subjected to renovation, while a substantial proportion of older houses had experienced refurbishments including some structural improvements.

The failure of batten-to-rafter connections as illustrated in Figure 6.3 was also prevalent in this housing, but this age also saw early hardwood nail plated trusses and graded purlin roofs, and some of these had showed failure at the roof to wall plate connection.

- The long span of trusses required more significant truss to wall plate anchorage than the previous rafter to top plate connections. Where insufficient uplift capacity was available in those connections, whole trusses complete with battens and roofing were lost.
- Graded-purlin roofs also suffered from the same problem as also shown in Figure 6.7 (b). Often in these cases, the anchorage at the edge of the roof was

barely sufficient, but at the centre of the roof, there was a significant mismatch between uplift load and connection capacity. These roofs often opened up from the centre and rotated about the external walls.



(a) trussed roofs



(b) graded purlin roofs

**Figure 6.7: Total roof loss**

For the roof shown in Figure 6.7 (b), the roof parted down the centre with the wind parallel to the ridge line and the two halves of the roof were found on opposite sides of the house.

Where refurbishment of the connections between battens and rafters, and between rafters and wall plates, had been undertaken, the performance was significantly improved.



**Figure 6.8: Improvements in batten-to-rafter connection**

Figure 6.8 shows a house in which batten-to-rafter connections were upgraded with straps fitted at the same time that the roofing was replaced. This roof section remained

intact, but extensive damage to the windward wall and roof edge combined with water ingress caused the ceiling collapse that enabled this photograph to be taken.

It was rare to find houses that had been fitted with batten-to-rafter straps that had lost battens during the cyclone. However, there were a number of cases of strapped batten-to-rafter connections in which the whole roof structure was lost due to failure of the connection at top plate level.

#### 6.1.4 Contemporary housing (post mid 80s)

Figure 5.1 and Figure 5.2 show that contemporary housing has the least structural damage of all of the age classes.

Roof damage to contemporary housing could be traced to either debris impact, or to inadequate fastening of tiles.

- There were examples of wind driven debris penetrating metal deck roofs, as shown in Figure 6.9 (a) and Figure 6.10.
- Some of the tile damage had been precipitated by debris impact on the roof.
- In other cases, the tiles including ridge capping had been poorly anchored.



(a) debris penetration through metal roof (b) tile damage after debris impact

**Figure 6.9: Debris damage to contemporary housing**



**Figure 6.10: Debris damage to Dutch gable and roof cladding leading to collapse of ceiling from water ingress**



*Elevated contemporary housing*

Two contemporary elevated houses experienced structural damage. The damage could be attributed to their hill top locations giving higher wind speeds than the structural detailing catered for. Other than these two cases, the only observed damage to contemporary elevated houses was debris impact or damage to fascia, guttering, soffits, etc. A number of other contemporary elevated houses had locations with higher wind speeds due to topography, but they had minimal damage.

*Contemporary slab on ground*

Most of the contemporary houses in the study area were slab on ground houses. The majority of these had hollow concrete block walls. Only one instance of a completed hollow concrete block house with wall damage was observed, and it had deficiencies in the filling of the cores. However hollow concrete block buildings under construction suffered some wall damage as shown in Figure 6.11. The cores had not yet been filled and highlight the necessity for the reinforced concrete cores.



**Figure 6.11: Damage to Concrete block work under construction**

## **6.2 Public, commercial and industrial buildings**

The performance of public and commercial buildings was mainly dependent on the structural systems and the age of the building. The older timber framed or unreinforced masonry buildings suffered more structural damage than the newer reinforced masonry or hot rolled steel framed buildings (Figure 6.12 and Figure 6.13).



**Figure 6.12: Damage to older commercial buildings Mourilyan**



**Figure 6.13: Damage to buildings in the Innisfail CBD**

In the Innisfail CBD, the damage would have been exacerbated in the older buildings due to the wind speed up over the hilly topography, as discussed in Section 3. The more recently constructed buildings had less structural damage compared with the older buildings.

As it is estimated that the wind speeds in TC Larry were about 90% or less than the design wind speed (only about 80% or less of the design wind load), a significant level of structural failures would not be expected to occur in engineered structures built in the last 30 years. However, windows, doors and roller doors on these buildings failed, causing water ingress and contents damage (Figure 6.14).



**Figure 6.14: Damaged sliding shop doors**

### **6.2.1 Performance of Cold Formed Steel Sheds**

As for all of the buildings observed, a significant number of these sheds suffered minor amounts of damage resulting from failure of guttering and other attachments or flying debris impact.

However, many of the modern cold formed industrial sheds appeared to be “too light”, that is the design capacity of the detail observed was inadequate for the wind action that was required to be resisted. As a result, many experienced significant damage even though the wind speeds were estimated to be less than the region’s design wind speed.

Many roller doors to the sheds also failed (with and without wind locks). The failure of a door usually resulted in a dominant opening, which generated high internal pressure. Many designs appear to have been based on the unsatisfactory assumption of low internal pressure, and so lightweight connections and components were employed, resulting in greater damage when subjected to high internal pressure.

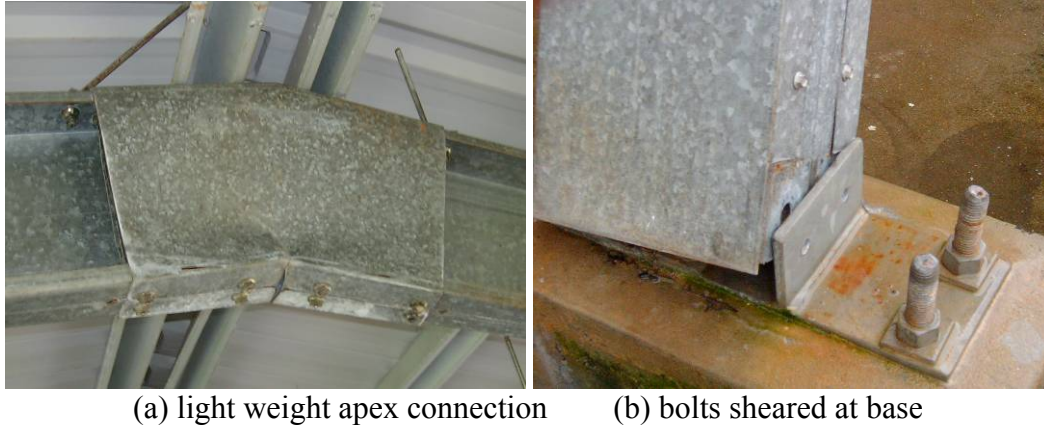
Extensive damage was also caused by the failure of corroded components. In some cases, the columns had corroded because they were directly cast into the concrete footing.

Other failures were caused by inadequate details such as undersized joints for the purlin/girt to frame connections or the joints in the structural frame. Typical poor details included:

- Under strength purlin to rafter joints (light gauge folded connection plates that are too light and create eccentric load paths).
- Poorly designed bracing (often no struts to support the top of the wind columns (mullions) and so the purlins in the end bay have to support the load as struts)



- Under designed frame moment connections (both at ridge and the knee) – light gauge connection plates often weaker than the sections to be joined, (eg a flat web plate with no flanges) as shown in Figure 6.15(a).
- Inadequate frame hold down connections to footings (e.g. bolts too small, cast in columns corroded, etc). An example is shown in Figure 6.15(b).



**Figure 6.15: Poor connection details**

### **6.3 General issues for buildings**

A number of other considerations were common to all types of building. They are discussed in this section under the following sub-headings.

- Mechanical services
- Corrosion
- Shadecloth
- Window leakage
- Parapets and Fascias
- Roof and Wall cladding

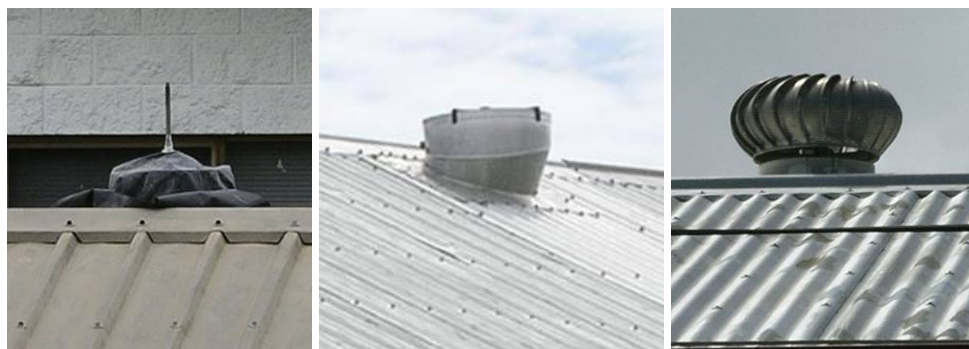
#### **6.3.1 Mechanical services**

There were many instances where air conditioning units and large ventilators had dislodged and become missiles that rolled or bounced along the roof causing impact damage during TC Larry. This type of damage was observed in the business district and was caused by the failure of mountings or connections of these mechanical units on the structure (Figure 6.16). In some cases this problem was exacerbated by corrosion resulting in a further reduction in strength. Some domestic ventilators also failed as shown in Figure 6.17. In each case, the loss of ventilators caused water ingress and contributed to debris damage.





**Figure 6.16: Damage to and from air conditioning units**



**Figure 6.17: Missing ventilators and a distorted ventilator due to fast rotation**

### 6.3.2 Corrosion

Corrosion has already been raised as a contributor to failure of some connections in houses, but corrosion was observed in all classes of building. Figure 6.1 shows some corroded connections in houses. Figure 6.18 shows some corrosion that was seen in damaged engineered buildings.

The large annual average rainfall of the affected area, together with its close proximity to the ocean can lead to accelerated corrosion. The chemicals used in agricultural spraying can also accelerate corrosion. Previous tropical cyclones also saturate most parts of buildings with water that carries an increased salt loads. (Wind-driven salt water is mixed with rain in the air layer over the sea and this air can be driven inland by the cyclone.)

All of this reinforces the need for correct material specification and for ongoing inspection and maintenance of the building stock.



**Figure 6.18: Corrosion in buildings**

### **6.3.3 Shadecloth**

Failures of both tension membrane shadecloth and draped shadecloth structures were observed in some carparks, nurseries, entertainment areas and farms. It appears that these structures are designed for wind speeds lower than design and that the sunlight porosity is mistakenly used for wind load porosity. Once the shade cloth is ripped, the risk of the cables failing at the supports is minimal.





**Figure 6.15 Damage to shade cloth structures**

#### **6.3.4 Window leakage**

Many people reported that windows leaked. This seems to be caused by a combination of:

- High differential pressures across the window that drive water through drainage channels from the high pressure region of the external wall to the lower pressure inside the building.
- High wind speeds cause wind flow up the windward face of buildings which drives water back under flashings that can normally cope with heavy rain without the high winds.
- Loss of fascias, gutters, soffits, vents, etc greatly increase paths for water ingress.

Water ingress can lead to damage to contents and fittings in buildings that otherwise had little damage. As community life styles and building contents becomes more vulnerable to water damage, this is an area that warrants a serious study to determine methods of improving performance of buildings against water ingress in tropical cyclones.

### 6.3.5 Parapet and fascia failure

The high pressures acting on elements such as parapets, soffits and fascias need to be considered. Failures of these elements as shown in Figure 6.19 increase water ingress, can lead to subsequent structural failures and add to the wind driven debris field.



**Figure 6.19: Parapet failure**

### 6.3.6 Cladding failure

Generally the performance of roof and wall cladding was adequate. Where failures were observed that were not attributable to wind driven debris impact, the cause was typically incorrect usage or installation, or corrosion. This highlights the need for continued vigilance in product testing and installation.

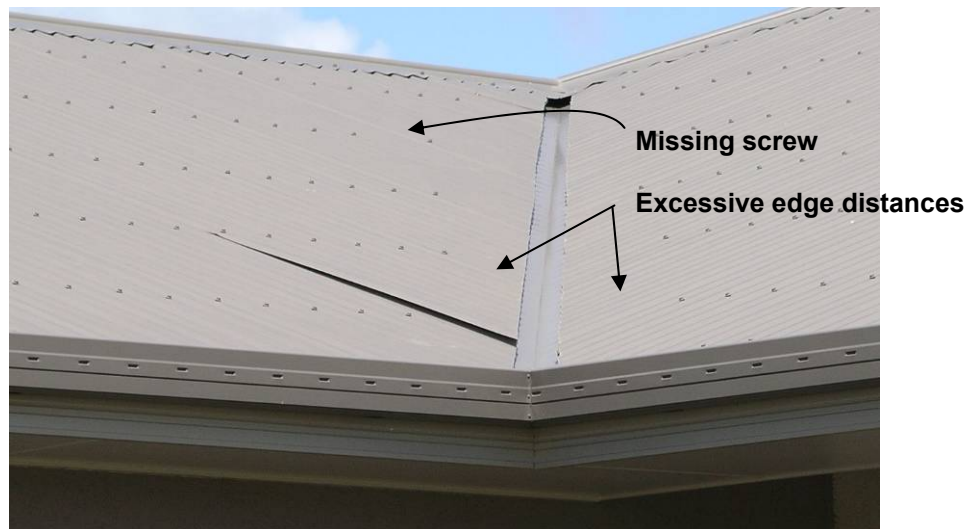
#### *Pierce fixed metal cladding*

For the observed failures of pierce fixed roof cladding, the failure was as a result of corroded fixings, incorrect fixings and/or spacings. Fatigue failure of metal cladding was observed in a few instances, but the fixing centres exceeded typical product test data (Figure 6.20).



**Figure 6.20: Inadequate fixing spacing and excessive side edge distance**





**Figure 6.21: Large edge distance and missing fixings**

Figure 6.21 shows a recently constructed house with a missing roof screw and excessive edge distance leading to an opening of the lap allowing water ingress. These types of errors in construction increase the vulnerability of the structure. If this house had been subjected to a longer duration event, there would have been loss of cladding.

#### *Secret fixed cladding*

Failure of secret fixed cladding, with examples shown in Figure 6.22, was observed in a few buildings, but the locations precluded any close inspection of the cladding or clips.



**Figure 6.22: Loss of secret fixed cladding**

#### *Tile roofs*

As noted in Section 6.1.4 the damage that was observed in tile roofs related to missing fixings, ridge and hip capping not fixed, and debris damage breaking tiles and dislodging adjacent tiles from the clips. Figure 6.23 shows damage at windward edge of a roof. Either the tiles were not correctly engaged with the tile clips or wind driven debris dislodged or broke the tiles in this area.

A number of people whose tiled roofs had performed well said that they had spent a little more money during construction to take the option of having every tile fastened. The minimum tile connection system should include fastening every tile, so it is worrying if some builders are presenting this, the only acceptable practice as an option.



**Figure 6.23: Detail of tile roof cladding failure**

## 7. Case studies

### 7.1 Houses on hilltops

As discussed in Section 5.1.1, there were a number of homes built on the tops of hills, ridges or in other elevated locations that were severely damaged, but may have performed adequately in flat locations.

The standard for wind loads for housing, AS4055 [8] has provided a means for assessing the effect of topography and relating that to a wind classification for housing since 1992. Prior to that, wind speeds for houses were assessed using earlier versions of the wind loading standard AS1170.2 [2], which also takes into account topography. In evaluating the topography of houses in this section, the wind classification is taken from the current edition of AS4055 [8].

#### 7.1.1 Construction after release of AS4055

##### *Case 1*

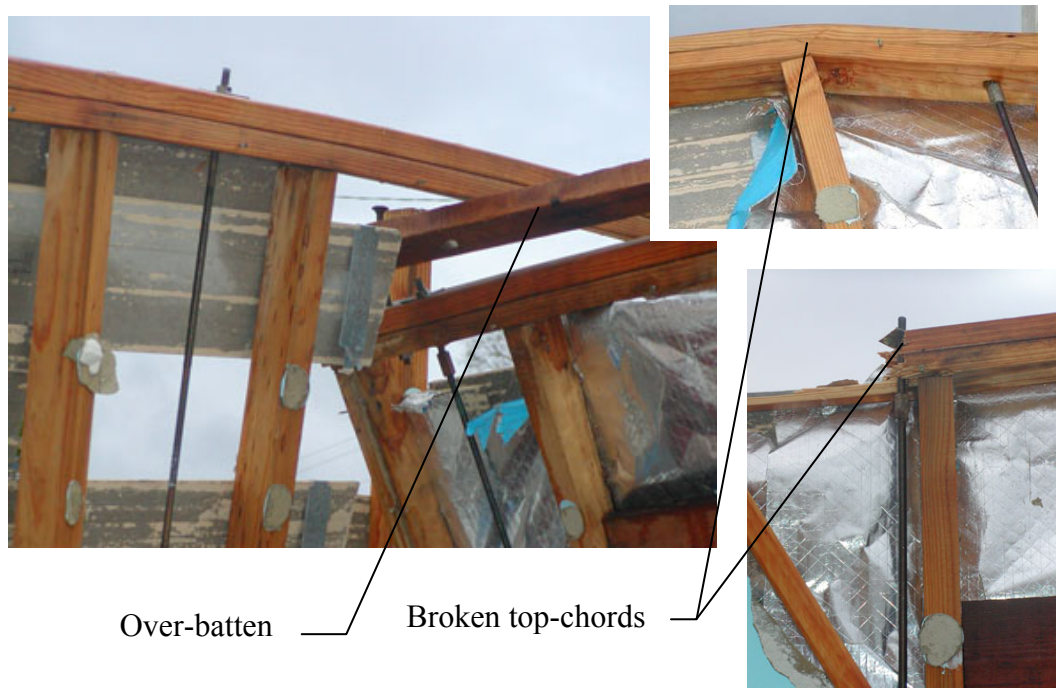
A relatively new home (around 10 years old) built on the crest of a hill sustained extensive damage (DI-7). The winds in the first part of the cyclone were from the south-east, approaching the house over open land (Terrain category 2.5), and up a steep hill.

The occupants of the house, and other witnesses confirmed that the roof detached around 45 minutes before the arrival of the eye of the cyclone. The estimated gust wind speed at that time was 45 m/s (referenced to 10 m height in flat terrain category 2). Estimations of the wind speed at roof height at the site at the time of the damage were 55 m/s after allowing for the topographic effects. This wind speed was between 10 and 20% greater than the peak wind speed at roof height of nearby suburban houses on flat land during the peak gust.

As indicated in Section 3.1, topographic factors can have a significant affect on wind speeds, and are included in estimated design wind speeds for building sites evaluated by either AS/NZS1170.2 [2] or AS4055 [8]. In terms of the site categorisation in AS4055 [8] the site should be classed as a C3 site.

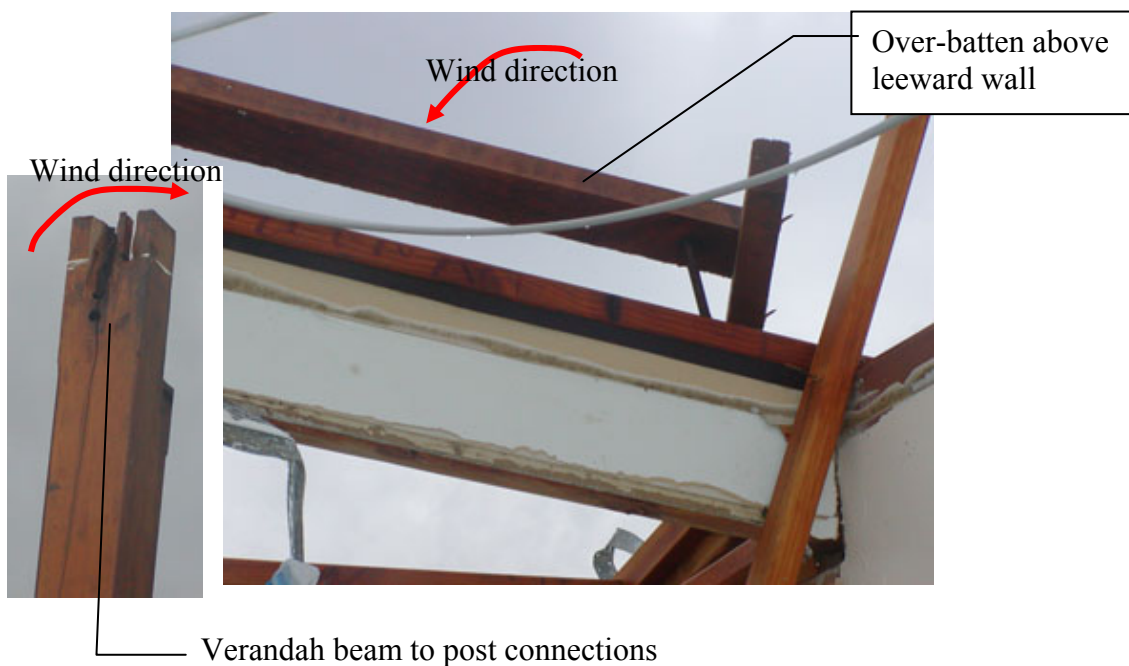
However, the detailing used in the house was at a general level appropriate for C1 or in some cases, C2 loading. The damaged house had screwed batten to truss connections that remained functional even as the roof structure left the house in one piece and travelled 250 metres before landing. The roof structure left the house complete with roofing, structural roof timber including trusses, ceiling joists and ceilings, flashings and guttering. The verandah beams were still attached to the leading edge of the roof, and some of the over-battens remained in the roof structure, but others were left in the house.

Overall, attention had been paid to anchorage of the structure in the house as shown by the extensive use of tie-down rods in the walls. Figure 7.1 shows that most of the wall tie-down rods remained effective, and though the walls were damaged once the restraint of the top edge was lost, most were still attached to the house.



**Figure 7.1: Wall anchorage intact**

The roof was attached with bolted over battens ( $75 \times 38$  hardwood) over most of the external walls, but at the verandah which was exposed to the windward side, the verandah beam was bolted to the top of hardwood posts with two M12 bolts in halved joints. These bolts had pulled out of the top of the post at the windward end of the verandah and out of the verandah beam at the other locations. In other places on the windward wall, the over battens had broken and allowed the trusses to lift.



**Figure 7.2: Roof anchorage**



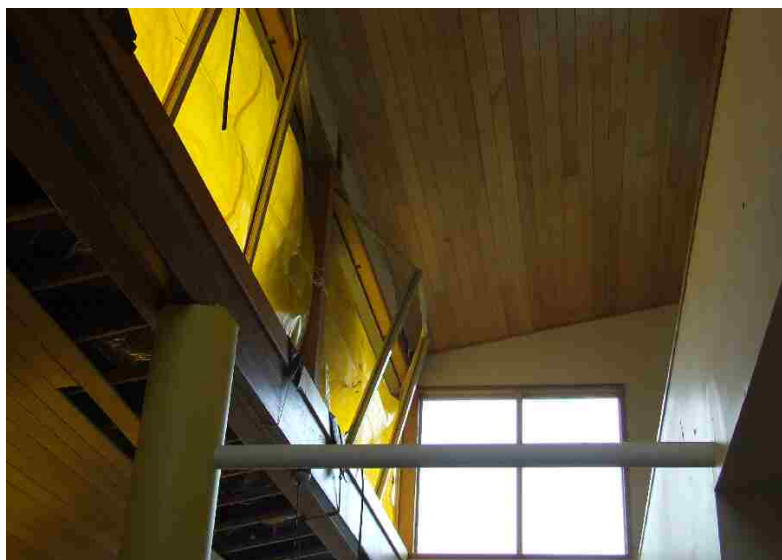
*Case 2*

A second case study of recent construction concerns a six year old timber framed contemporary house approximately 50m above sea level on a steep hill with a slope greater than  $45^{\circ}$ . The house had magnificent ocean views and suffered significant damage. For this high exposure special detailing is needed for many elements.

The very high upwind slope and open approach of the location does not allow the use of AS4055 [8] for calculating design wind loads, and using associated codes for constructing this house. Specialist engineers should be consulted for designing houses in such locations. The wind load standard AS/NZS 1170.2 [2] provides topographic multipliers in excess of 1.5 (i.e. the design wind speed at 10m is increased by more than 50%) for designing buildings on such steep topography.

Large glass windows on the face of the house overlooking the ocean were subjected to the strong winds (exacerbated by the topography) off the sea as TC Larry crossed the coast. Damage was initiated by the failure of the fasteners connecting the frame of the large windows on the upwind face causing the window units (frame and glass panes) to blow into the house (Figure 7.3 and Figure 7.4). The resulting high internal pressure caused the windows on the side-walls to blow outwards. Furthermore, one side wall had bowed outwards and a main rafter on the roof had lifted following the increase in internal pressure. Closer inspection also revealed a weak point in the wall frame attachment to the floor bearer, which had allowed the wall to bow outwards.

The failure of window frame connections in the study area was observed. In these cases, it appears that the frames of large windows are connected to the wall with fixings at widely spaced intervals. Therefore even if the glass panes and frame are able to resist the wind pressure the shear strength of frame connections are not sufficient. All detailing needs to perform at a level appropriate to the design wind speed.



**Figure 7.3: Failure of Windows & Frames at SE Wall at Mezzanine Level**



**Figure 7.4: Failure of Windows & Frames in windward Wall at Main Level**

### 7.1.2 Older hill-top construction

A number of houses in hill-top locations in the damaged area were constructed prior to the 1981 revision of the Queensland Building Code.

- *Case 3 – Low set hill top house DI-7* - Cyclone rods and over-battens were part of the original construction, and the battens and roof sheeting were replaced after Cyclone Winifred in 1986. Again, the detailing was adequate for C2, but was not suitable for the hilltop location. Improvements in batten to rafter fixing kept the roof structure together, but C3 detailing should have been used at the rafter to top plate connections.

The owner indicated that the windows were all intact before the roof lifted off.

However, even without internal pressure, the entire roof structure lifted off, causing the walls on the windward face of the house to collapse.

- *Case 4 – High set hill-top house DI-3* – inadequate batten to rafter connections caused loss of part of the roof, however, the whole roof structure had lifted off the walls approximately 12 mm. This hill-top location would also have required C3 detailing, and the batten to rafter connection fell short of the standard required for current C1 construction. Discussions with the resident indicated that the roof was damaged without internal pressurization.
- *Case 5 – Three houses in one hill-top street DI-4* – the sites on which these houses were built all had C2 classification and had battens fixed to rafters with one 75 mm nail. This detail would not be satisfactory for a C1 location, and the result was loss of all roof sheeting with battens still attached. See Section 7.2 .
- *Case 6 – Timber roof frame construction DI-5* – this site had a C2 classification, and in spite of a steep roof slope lost all of the roofing, battens and the central part of the roof structure. The rafters around the perimeter walls remained where the tie-down anchorage had adequate capacity, but the struts that supported the ridge beam had been skew nailed to internal walls and were the weak point in the anchorage. Thus the ridge beam and supporting struts had been lost.

Three other homes built prior to the 1980s on top of a steep ridge, with extensive views in two directions suffered roof damage. There was evidence that two had suffered full internal pressure, and one was almost totally destroyed (DI-6), while the other two sustained roof loss (DI-3 and DI-4). More recently constructed (contemporary) houses on the same ridge had little damage.

## 7.2 Roof structure

The single most common form of roof damage seen in the study was failure of the batten to rafter connections. In many cases, the houses affected were constructed prior to the 1981 revision of the Queensland Building Code [4], and the detailing used would have been at the discretion of a roof plumber. However, roof construction performed since 1981, should have incorporated adequate detailing of this connection.

### 7.2.1 Nailed batten to rafter connections

#### *Case 7*

A 30 year old 2 storey brick house with a low pitch gable roof sustained no damage in the first part of the cyclone. However, when the winds changed direction, the battens and roof sheeting lifted off. The failure could be attributed to inadequate batten to rafter connections (Figure 7.5). Two nails per batten/rafter connection were used instead of Type 17 screws or strapping. Nailed batten to rafter connections do not have adequate capacity to resist uplift forces generated by the winds generated by TC Larry. However, the same problem was noted in the report following TC Winifred [3].



**Figure 7.5: Batten to rafter connection failure**

*Case 8*

There are no shortage of similar cases to the one described in Case 7, however, a number of older houses with roofing that appeared to date well before cyclone Winifred had batten to rafter connection failures with one nail per joint. (Figure 7.6)



**Figure 7.6: Batten to rafter failure with one nail per joint**

### **7.2.2 Rafter to top plate connections**

*Case 9*

The failure of the roof structure and the collapse of many external and internal walls occurred due to inadequate connection of rafters to top plates in an extension. The rafters used in the extension were spliced to the existing rafters in the main part of the house. This increased the rafter span to approximately 4 metres, and only one pair of skew nails connected each rafter to the top plate. The original house was quite old, and it was hard to say when the renovation took place.

In this case, the failure of the roof was followed by failure of a number of the external and internal walls. It is shown in Figure 7.7(a).

*Case 10*

This case was also an older house and here all of the roof structure had lifted off in one piece. The roofing was again screwed to battens with large cyclone washers to spread the load. However, there was no sign of framing anchors or overbattens in tying the rafters to the top plate. Rafters were skew nailed to the top plate through the birdsmouth in the rafter.

Some rafters near the ridge was supported on struts and these were still attached to the rafters, but had become separated from the ceiling plane at the base. The house without its rafters is shown in Figure 7.7(b).





(a) Spliced rafters separated from walls

(b) Total roof loss

**Figure 7.7: Rafter loss with skew nailing to top plate**

### 7.2.3 Loss of ridge and struts

#### *Case 11*

Several houses exhibited similar damage to their roof structure; the rafters on the outside of the house were adequately connected to the top plates, but the connections between the ridge, struts and rafters failed, causing loss of roof structure in the middle of the house, as seen in Figure 7.8.

**Figure 7.8: Loss of ridge and struts**

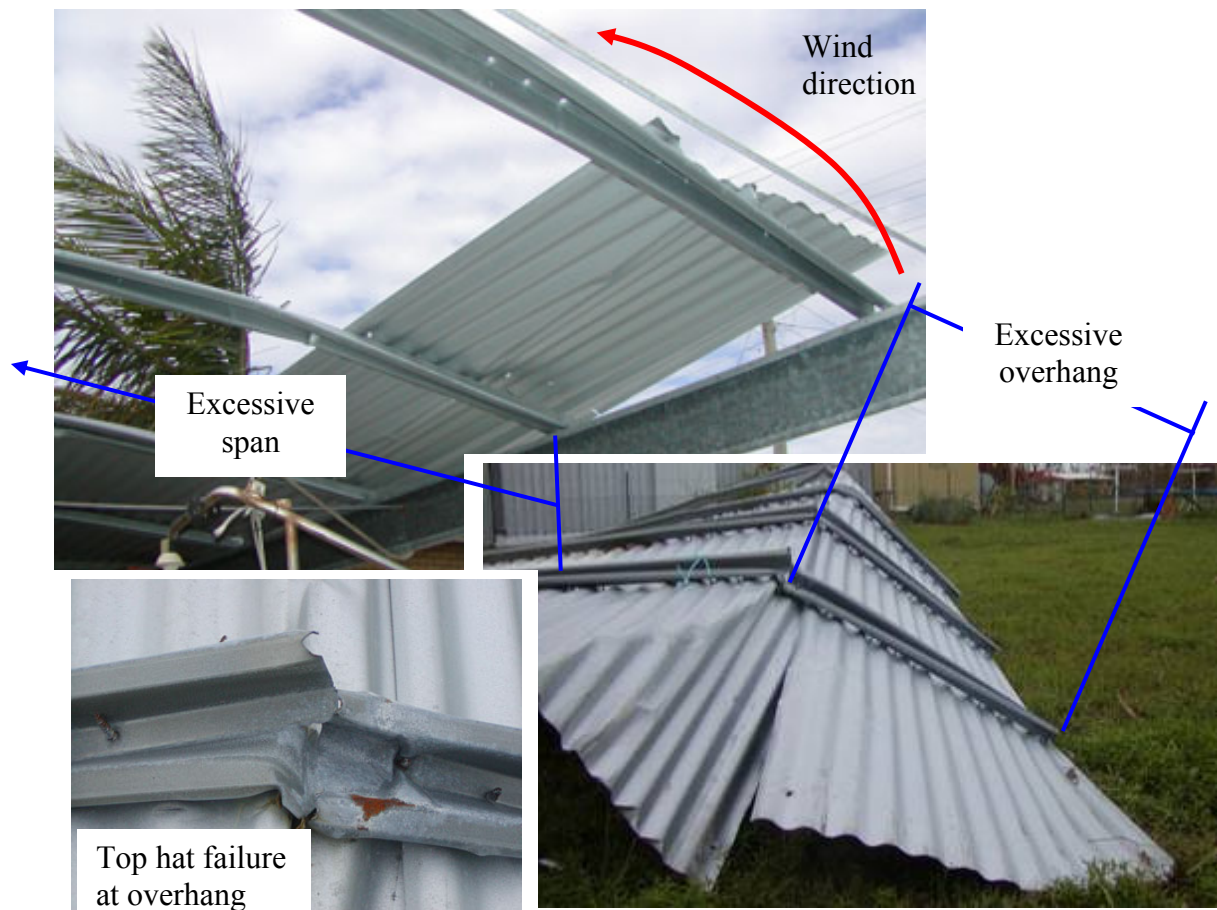
In the older houses, the roofs had been built using struts to support the central part of the roof. The struts carried gravity loads from the roof via internal walls, but under extreme uplift conditions, the skew nails at the base of the struts were not able to transmit tension from the strut into the walls. In many cases, the ridge and central part

of the roof was completely missing. However, where the rafters had been adequately tied to the external walls they remained, leaving the roof structure with nothing in the middle.

#### 7.2.4 Steel Frame roofs

##### *Case 12*

The roof and verandah of a house sustained severe damage after failure of the adjoining garage roof illustrated in Figure 7.9. The garage roof was constructed using steel C-section rafters with top hat purlins fixed with tek screws. The top hat overhangs (900 mm) and spans (2700 mm) were both well in excess of design standards. The garage roofing lifted when the top hats tore from the C section rafters, taking the adjoining verandah roof with it. The house roof battens and sheeting then peeled back to the ridge.



**Figure 7.9: Excessive spans in top hat battens**

##### *Case 13*

In this house, the beam to post detail failed due to tearing of the steel in the rafter around the bolt.

Spans for the C section rafters and purlins appeared to be within specification, and in spite of the twisted appearance of the roof as shown in Figure 7.10, most of the purlin to rafter connections held. However the C section fascia beam that carried the ends of

the rafters had torn through at the bolts that remained in the top of the post as shown in the circle at the top right of Figure 7.10.

While there was some corrosion evident in the roof, it did not appear to have played a role in the failure of the steel at the connection. However, the forces in the bolts would have been four or five times the forces in the purlin to rafter joints, and a number of the connections at the top of the posts had failed in a similar way.



**Figure 7.10: Failure of steel beam to post connection**

Figure 7.10 shows that the progress of the failure was halted once the external wall of the house was reached and the tributary area of connections between roof beams and wall plates was much smaller than that for the post connections.

### **7.2.5 Verandah beams and connections**

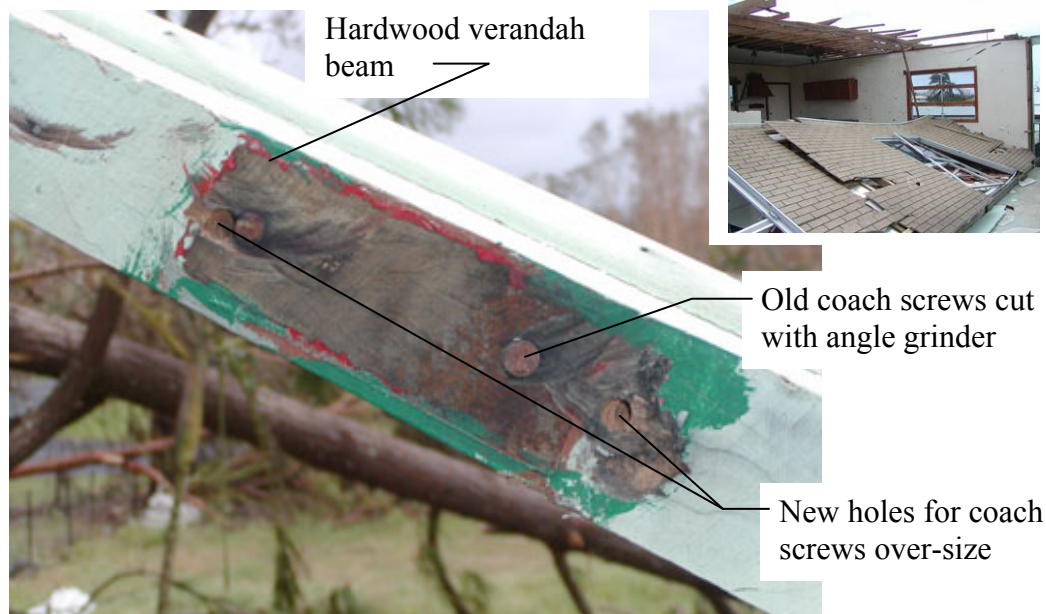
Verandah beams and connections tend to have higher tributary areas than connections between the roof structure and wall members elsewhere in houses. They frequently make use of a few high capacity fasteners (such as bolts or coach screws) to transmit the loads to posts. A number of the case studies (*Case 1* and *Case 13*) have already indicated that these fasteners proved a weak point in the house that lead to significant damage to the house.

#### *Case 14*

In this house, the entire roof had been removed and the windward wall had blown in (see inset in Figure 7.11). The front of the house incorporated a large verandah with heavy verandah beams screwed to the top of steel posts. A detail of the anchorage is shown in Figure 7.11. It appeared that the original coach screws which had rusted had



been cut off with an angle grinder and a new hole drilled to fit new coach screws. However, the hole appears to have been over-drilled, and the screw has withdrawn without causing much damage to the wood fibres. As the verandah roof was integrated into the house roof, once it had lifted, it took the rest of the house roof with it, including rafters, and struts.



**Figure 7.11: Repair of verandah beam to post connection**

### 7.2.6 Added structures – barbeque areas, awnings, garages

Many of these types of structures are not seen as a significant part of the main house. Some of them have been owner-built and may not have been subjected to the same supervision as the remainder of the structure. None the less, failure of the added structure can cause loss of part of the main building and will contribute to wind-bourne debris. *Case 12* was an “engineered carport” which had been added in the same roof line as the rest of the house, and the roofs were connected. Once the carport roof lifted, as it was under the main house roof and fastened to it, part of the main house roof was lifted as well.

#### *Case 15*

In this building, a barbeque area had been added to the house. Heavy timbers had been used, large diameter posts, and the owner believed that the structure was stronger than the roof structure of the house. It had been fixed to the house roof using heavy gauge steel brackets.



**Figure 7.12: Missing barbeque area roof**



However, the posts did not appear to have been bolted to the ground, but were simply encased in the brickwork that was part of the bar-be-que area. The early part of the cyclone had winds that placed the barbeque area on the windward side of the house and under high uplift generated by the roof, the posts pulled out of the brickwork and the roof folded back against the rest of the house. In being dragged backwards, the heavy steel brackets to the house roof tore off the roof over two rooms in the main house (Figure 7.12).

#### *Case 16*

This case featured an older garage built next to a house, but under the eaves of the main house. The garage was a total loss, but having lost its roof, it impacted on the house roof and lifted the eaves to cause damage to the main house roof structure, as shown in Figure 7.13.



**Figure 7.13: Garage damage and partial loss of house roof**

#### **7.2.7 Windows and doors**

Window panes broke in many buildings allowing full internal pressurisation. However, in a number of cases, the internal pressurisation was caused by other failures such as inadequate window frame fixings, poor door locks and wind driven debris puncturing the building cladding.



(a) window and door furniture    (b) large debris damaging whole walls

**Figure 7.14: Internal pressure from other than glass failure**

Figure 7.14(a) shows the failure of a French Door lock which allowed the door to swing open and admit windward wall pressures to the inside of the house.

Figure 7.14(b) shows an extreme case of debris impact. In this case, the impact of a whole roof structure removed a whole wall of the house and allowed full pressurisation of the remainder of the house. There were several such cases in the investigation area.

These cases of full internal pressurisation due to factors other than glass breakage mean that window debris protection screens or shutters would not have been effective in every case.

### 7.3 Previous events

In some cases, the factor that initiated failure can be traced to some previous damage. In a number of cases, repair work to the houses was undertaken, but not to a standard that would restore the full functionality of the structure. In other cases, damaged details may have been over-looked.

#### *Case 17*

A house lost its complete roof soon after passage of the eye. The building had weathered the wind for the first half of the event without noticeable damage, though there was some water ingress through windows.

However, with the first gusts from the new wind direction after the passage of the eye, the roof lifted from the garage first, and then as the garage and house shared the same roof, the roof lifted off almost the entire house. Careful examination of the details of the house found that the anchorage bolt for the garage door lintel beam had broken some time ago, and had a failure surface that was badly rusted. It is shown in Figure 7.15. The bolt was straight which indicated that the lintel had lifted off it nearly vertically which would have been the case if this had been the first part of the roof to lift. The garage lintel was identified in the roof debris nearly 100 meters away.



**Figure 7.15: Broken tie-down bolt at garage lintel**

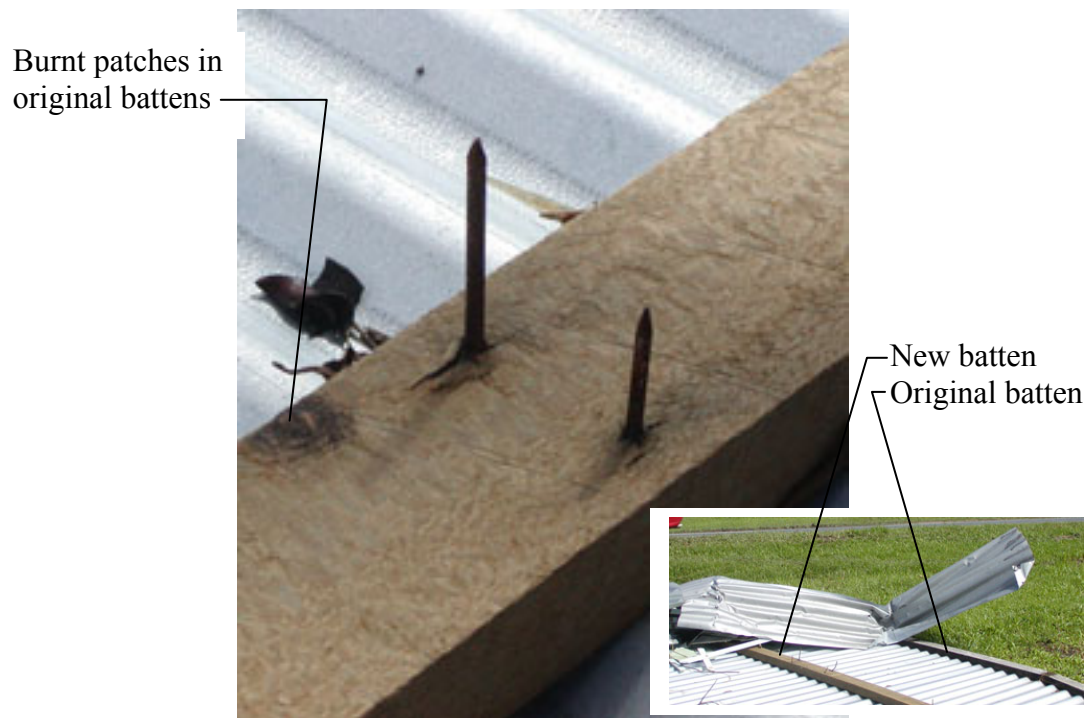
Discussions with the owner established that the house had been built just before the passage of cyclone Winifred and that some repair work had been completed after that event. It appears that the failure of this anchorage bolt was not observed at that time.

A number of other houses were found in which structural repairs had been completed after Cyclone Winifred, but further damage occurred to the repaired portions of the building in Cyclone Larry. This is a concern, as it indicates that the building was not repaired to a standard that would have complied with the building regulations in force at that time.

#### *Case 18*

This case concerns a house that had previously suffered a fire in the roof structure, and had afterwards been repaired. The roof structure contained a mixture of charred and new timber as shown in Figure 7.16.

The house had originally been built with nailed batten-to-rafter connections, and when repaired the new battens had also been nailed to the rafters with two 75 mm long nails. This had not proved adequate to resist the wind forces in TC Larry, and the roof and battens had lifted off the house. While it is difficult to say when the fire and subsequent repairs had occurred, Figure 7.16 shows that the roofing appears quite new and so the repairs are likely to have taken place since the building bylaws were amended in the early 1980s.



**Figure 7.16: Nailed batten to rafter connection in repaired house**

## **7.4 Roller Door Failures**

### *Case 19*

A large portal frame building had about twelve roller doors fitted and all of these except one failed. About half of the roller doors had an opening width of about 6 m, did not have wind locks and were of the pressed sheet type. The remaining six doors (all of which failed) had an opening width of about 7.5 m, were fitted with wind locks and were of the linked slat type (Figure 7.17).



**Figure 7.17: Two of the failed roller doors**

Figure 7.18 provides a detailed view of the wind locks fitted to one of these failed roller doors.





**Figure 7.18: Detailed View of Failed Wind Locks to Roller Door**  
(Wind locks fitted to every second slat)

#### *Case 20*

In this small shed, two roller doors failed. The doors were on the windward side at first and blew inwards. The slats bent sufficiently to also bend the spindle. The internal pressurisation of the building blew the leeward wall out.



**Figure 7.19: Failure of roller doors and back wall of shed**

## **7.5 Public, Commercial and Industrial Buildings**

#### *Case 21*

A class room block suffered severe damage with the loss of roof and walls as shown in Figure 7.20. The failure was at the rafter top plate connection with the resulting large sections of roof (cladding, battens and rafters) causing severe damage to adjacent blocks and accommodation (Figure 7.21). Due to the cleanup operations, close inspection of the wall top plate was not possible. It is postulated that the birds mouthed rafter connection to top plate was the weak link in the wind load path due to the upgrading of the roof cladding and batten connections. The windward wall collapsed inwards with the loss of support from the failed roof structure. It is interesting to note that an adjoining classroom block suffered similar damage during Cyclone Winifred [3].



**Figure 7.20: Loss of roof and wall structure**



**Figure 7.21: Impact damage from leaf of roof**

A double brick gable end and associated double brick wall failed on another classroom block (Figure 7.22). The gable end wall was in a lee side zone for the brunt of the winds and was subjected to combined external suctions and internal positive pressures. A crack in the double brick support pier of the windward wall was also observed suggesting the minimal tie down capacity available. Perhaps the uplift in the roof transferred to the brickwork, reduced the compression across the bed joints and hastened failure under bending actions. (Bending in unreinforced masonry can only be sustained in the presence of axial compression.)





(a) Crack in pier



(b) End wall collapse

**Figure 7.22: Gable end wall failure**

#### *Case 22*

The building shown in Figure 7.23 remained intact with little evidence of structural damage. Some flashing, guttering and roof vent failure resulted in water leakage into the rooms through the ceiling. This building was used as an operation centre for the recovery after Cyclone Larry.

Failure of flashing on a nearby single storey building on the same campus was investigated. It was observed that the majority of screws along the flashing had corroded to the point that the larger part of the shank was gone. Other screws from the main body of the roof were removed with no corrosion or fatigue or permanent

deformation observed (Figure 7.24). It appeared that the corrosion was limited to the edges of the cladding that were exposed underneath (i.e. unlined eaves).



**Figure 7.23: Loss of flashing at Dutch gable end**



a) screw from unlined edge of roof

b) screw from main roof section

**Figure 7.24: Screws from same roof**

### *Case 23*

This was a large assembly building that was constructed using hot rolled steel portal frames as the main structural support system. The building consisted of seven portal frames (each bay about 6 m long) that spanned about 18 m. with a knee height of about 6 m. A low level skillion roofed extension was built off one side of the portal frames and there were metal louvres fitted to the side wall above this low level skillion roof. Most of the external wall cladding was brick.

During the first part of the cyclone, the (Southerly) wind blew onto these louvres blowing- in almost all of the banks of louvres into the hall (Figure 7.25). The failures were at the louvre frame to building connection. This then caused the roller door to the end wall to fail.





**Figure 7.25: View of Failed Banks of Louvre Windows to Large Hall**

*Case 24*

A medium sized shed suffered end wall failure along with buckling of roof top hat battens and the ubiquitous failed roller doors. As there was no compression bracing observed in the roof structure this meant that the roof battens were to carry the combined loading from the end wall lateral loads as well as the large roof suction loads (Figure 7.26). The buckling of the battens led to failure and collapse of the end wall.



**Figure 7.26: Buckled top hat roof battens in end bay**

## 8. Design criteria

The majority of modern buildings performed well structurally in TC Larry. This would indicate that the provisions of the current code and standards are adequate in most areas, notwithstanding that the wind speeds in TC Larry were less than current design values.

The relevant design provisions are detailed in the BCA, *Building Code of Australia*, for the design and construction of buildings and other structures covered by building law. Referenced documents, such as AS/NZS1170.2, AS4055, AS1684.3 and AS4505 are called up within the BCA. These documents provide a detailed means of complying with the requirements of the BCA as a part of the deemed to satisfy (DTS) provisions of the BCA.

### 8.1.1 Wind loading standards

The buildings in the investigation area were covered by two wind loading standards.

*AS/NZS1170.2 - Structural design actions Part 2: Wind actions*

- The maximum gust wind speeds (referenced to flat open country at a height of 10 m) in the study area were estimated at 50 m/s to 65 m/s. This is less than the regional design wind velocity of 70 m/s for the same area specified in AS/NZS1170.2.
- In calculating the design wind speed for individual buildings, AS/NZS1170.2 uses topographic, terrain and shielding multipliers that will increase the design wind speed for specific buildings in exposed hilltop locations. Observations indicated that there was acceleration of the actual wind in these locations.
- Based on the survey data presented in this report, the winds experienced at all structures studied were less than the design wind speed calculated from AS/NZS1170.2. However, some cold formed steel sheds suffered significant levels of structural damage. It appears that misinterpretation or misapplication of criteria (e.g. local pressure factors, internal pressures) resulted in components and connections of insufficient strength to withstand the less than design wind loads. Design decisions such as not designing for a potential dominant opening results in a higher probability of failure when such openings occur (e.g. failure of a roller door resulting in large internal pressure).

*AS4055 - Wind loads for housing*

- This uses simplified models for site wind speeds, but they still address shielding, topography and ground roughness. The classification from the current version of this standard gave increased design wind speed for houses in exposed elevated positions. Damage to contemporary housing from wind pressure (as opposed to wind driven debris) appeared to be mainly from under-classification of the site (e.g. building a C2 house on a C4 site).

In AS/NZS1170.2 there is a requirement for buildings in regions C and D to be designed to resist internal pressures arising from dominant openings unless designed to resist debris impact from a 4 kg piece of timber projected at 15 m/s. There were a number of cases in which door or window fasteners failed under wind loads alone giving higher internal pressure, and other cases in which significant portions of roof (much heavier than 4 kg and travelling faster than 15 m/s) caused damage to buildings that led to internal pressurisation. Recent studies have shown that debris heavier than 4 kg can reach 15 m/s in less a matter of seconds over a short distance (<20 m).

Perhaps there is a need for the standards to be more explicit about requiring design for the ultimate limit state using full internal pressure regardless of protection afforded doors, windows and cladding.

### **8.1.2 Windows and glazing standard**

Heavy rain usually accompanies most cyclones and windstorms which generate large differential pressures across the building envelope. These pressure differentials could easily exceed 1kPa across windows and doors on the windward face of a building. According to the standard AS2047 [7] the water penetration resistance is set at between 150 to 450 Pa for windows and 150 to 200 Pa for adjustable louvre windows in C1 to C4. This appears to be a serviceability design requirement, which would not prevent water ingress into the building in extreme wind events (i.e. ultimate limit state). During the investigation, many examples of damage to contents resulting from water ingress were observed.

In some cases, failure of window (and louvre) frame connections occurred, implying that their design and construction do not comply with the standard AS2047. Where these failures were observed, the frames of the windows were connected to the wall using nails or screws spaced at large intervals. There is anecdotal evidence to suggest that builders assume that internal linings etc. provide additional support to the window frame. However, this does not appear to satisfy requirements in AS2047, as the strength of frame connections in these cases are not adequate, even if the glass panes and frame are able to resist the wind pressure. This should be of concern to the building industry, as the failure of a window on the windward face will create more water damage and will result in high internal pressures and potentially instigate more severe failures.

### **8.1.3 Roller doors**

Roller doors on all types of buildings performed poorly. It is clear that many of the roller doors, including some with wind locks, do not comply with requirements in wind loading standards and the domestic garage doors standard AS/NZS4505 [8], as failures occurred at loads significantly less than the design value.

Evidence suggests that in many cases, the roller doors disengaged from the vertical tracks even before the building experienced its highest wind speed during TC Larry. The building industry must ensure that roller doors are properly rated, or at the very least that buildings are designed for the appropriate internal pressures by assuming that the roller door will fail under moderately strong wind speeds, with the potential for creating a dominant opening.

## **8.2 Progress in building quality**

Even though the wind speed was less than the design wind speed for the investigation area, there is no doubt that the performance in TC Larry of the most recent buildings was overall better than the performance of the older buildings. This reflects the improvement in building regulation and general building practices for new construction.

In order to maintain progress in the ability of buildings to resist tropical cyclones such as TC Larry, it is important to ensure that all repairs, where practicable, from this event are to the current standards rather than the practice that may have been used in the building at the time of its construction.

## 9. Conclusions

Tropical cyclone Larry caused damage to buildings in an area centred on Innisfail in North Queensland. The overall level of damage appears to be around 15% to 20% of the value of the building stock in the region. This represents a significant cost of reconstruction, and means that recovery of the building stock in the region is expected to take more than a year.

By assessing damage to simple structures, buildings and vegetation, the peak gust wind speed referenced to open terrain and flat topography was estimated to have been in the range 55 m/s to 65 m/s for the town of Innisfail and adjacent areas (within the path of the eye of the cyclone) and in regions south of the eye extending to the town of Kurrimine Beach. It was estimated that the areas north of the eye experienced slightly lower peak gusts of 50 m/s to 55 m/s. These wind speeds were less than the regional design wind speed for the entire study area, so all buildings inspected were judged to have received peak gusts at the structure of lower speed than the design standards would have used as the site wind speed from the design event. However, in a few areas the estimated maximum gust wind speed was greater than 90% of the regional design wind speed, so the event was approaching the design event.

Tropical Cyclone Larry was a relatively fast moving event which meant that the duration of peak winds was relatively short. This meant that buildings experienced fewer wind cycles and the debris was transported fewer times than in slower moving events. As well, there was a shorter period in which rain was being driven into buildings. Had the cyclone been moving more slowly but with the same gust wind speeds, the debris damage, water penetration and cladding damage would have been worse.

Overall, newer buildings performed better than older building stock, with damage mainly to roller doors and attachments such as guttering, facias etc. This confirms that the current suite of loading, design and construction standards are effective without being overly conservative. However, even in newer buildings there was some damage that could have been avoided by strict application of the current suite of standards:

- In some cases, topographic effects had been ignored in the design and construction of newer buildings. These effects are modelled in both AS/NZS1170.2 and AS4055 the wind loading standards for general buildings and housing respectively.
- Batten to rafter connections under sheet roofs on some new construction (principally refurbishment of older buildings) still made use of one or two plain shank nails as the only anchorage. This is not in compliance with AS1684.3. There was no evidence of failure of batten to rafter connections that complied with AS1684.3.
- Some window and door fixings failed under wind load. In some cases, it was locks and catches, and in others it was the fixing of the frame to the structure. It is important that all components of windows and doors (including fixing to the building) be capable of resisting the design wind load.
- A significant proportion of roller doors in newer structures failed under wind loads. In many cases, wind locks had been fitted, but were ineffective due to poor anchorage of tracks to the structure or to flexibility of the doors and tracks.



- Cold formed steel framed industrial sheds did not perform as well as other types of newer buildings. Some standard connections and elements were too light to satisfactorily handle the wind loads.
- Water penetrated most buildings with resultant damage to contents, and where linings were of plasterboard, there was ceiling or wall lining damage.

Wind damage was more wide spread among buildings that were built prior to the release of the Queensland Home Building Code Appendix 4 (1981) [4]. In many cases, these buildings had been refurbished since the 1980s, but structural details remained the same. In some cases, these buildings had been repaired following TC Winifred (1986) but used the original construction details. Among the more common failure details noted for these buildings were the following:

- Batten to rafter connections under sheet roofs were commonly one or two plain shank nails. These do not comply with current provisions of AS1684.3.
- Roofs built using rafters, underpurlins and struts often had failures of the strut to wall connection which had not been designed to resist uplift. (A strut acts in compression for gravity loads and in tension for wind uplift.) AS1684.3 has requirements for tying down underpurlins and struts which were not seen on any of the damaged roofs.
- Water penetration also caused damage to contents of these buildings and where refurbished using plasterboard, to the linings themselves.
- Deterioration of fasteners, sheeting and metal frames due to rust, or to timber due to rot also compromised structural performance of these buildings.
- There were a few instances in which damage from previous events was not detected and hence remained in the structure for this event.

Most of these deficiencies can be remedied if reconstruction is performed to current building standards, and if subsequent refurbishment is required to ensure that at least the anchorage of roof elements down as far as the top of the walls meets the current standards.

The surveys showed very few buildings that had used debris screens on doors or windows. The inspections showed a range of debris damage including:

- Denting of cladding or roller doors from glancing blows from debris.
- Penetration of cladding under debris attack. In a few cases, the debris entered the buildings.
- Removal of gutters fascias and other trim due to debris impact.
- Breakage of glass due to debris impact.
- Demolition of part of the building due to substantial debris impact.

In some of these cases, the debris impact load was significantly higher than predicted in the test using a 4 kg mass projected at 15 m/s. It seems therefore appropriate for all buildings in the cyclone prone regions to be designed for the ultimate limit state using dominant openings.

Reconstruction should be accompanied by careful checking and supervision to ensure that all parts of the roof structure comply with the current building standards. These have demonstrated their effectiveness in this near-design level event.

## 10. Recommendations

### 10.1 Reconstruction Guidelines

Most of the structural failures encountered during the inspection could be explained in terms of:

- Inadequate detailing for the design wind speed at the site.
  - Education in correct use of AS4055 required
- Deterioration of structural elements (connections or members)
  - Awareness of importance for ongoing inspection and maintenance of buildings required
- Incomplete repair after previous events or during renovations
  - Education in use of AS1684.3 and HB132.2 required
- Impact of airborne debris
  - To reduce debris ensure all building attachments and fitments are designed and installed to the same rigour as the main structure

People involved in reconstruction following damage during TC Larry should be aware of the requirements and recommendations under the current building codes and standards. This includes regional rebuilding requirements detailed by the Queensland Building Services Authority and local councils. If builders/designers have come from outside of the cyclone prone area, they may require training to ensure that they become familiar with the current requirements for the region.

Also where only part of the roof has been damaged by wind (rather than debris), consideration should be given for the repair and retrofitting to be applied to the whole roof. This will ensure that in a future event should the wind approach be from a different direction, the other parts of the roof will have adequate capacity.

Soon after the event, guidelines were made available to the Queensland Department of Local Government, Planning, Sport and Recreation for distribution throughout the affected area. The Guidelines were provided by Timber Queensland, and are reproduced with permission as Appendix B to this report.

#### 10.1.1 Hill-top construction

AS4055 details information that can enable wind classification of sites. It is recommended that it be used to select appropriate detailing for reconstruction of damaged housing.

As a rough check, the wind exposure can be related to the view from the site. The following applies in North Queensland:

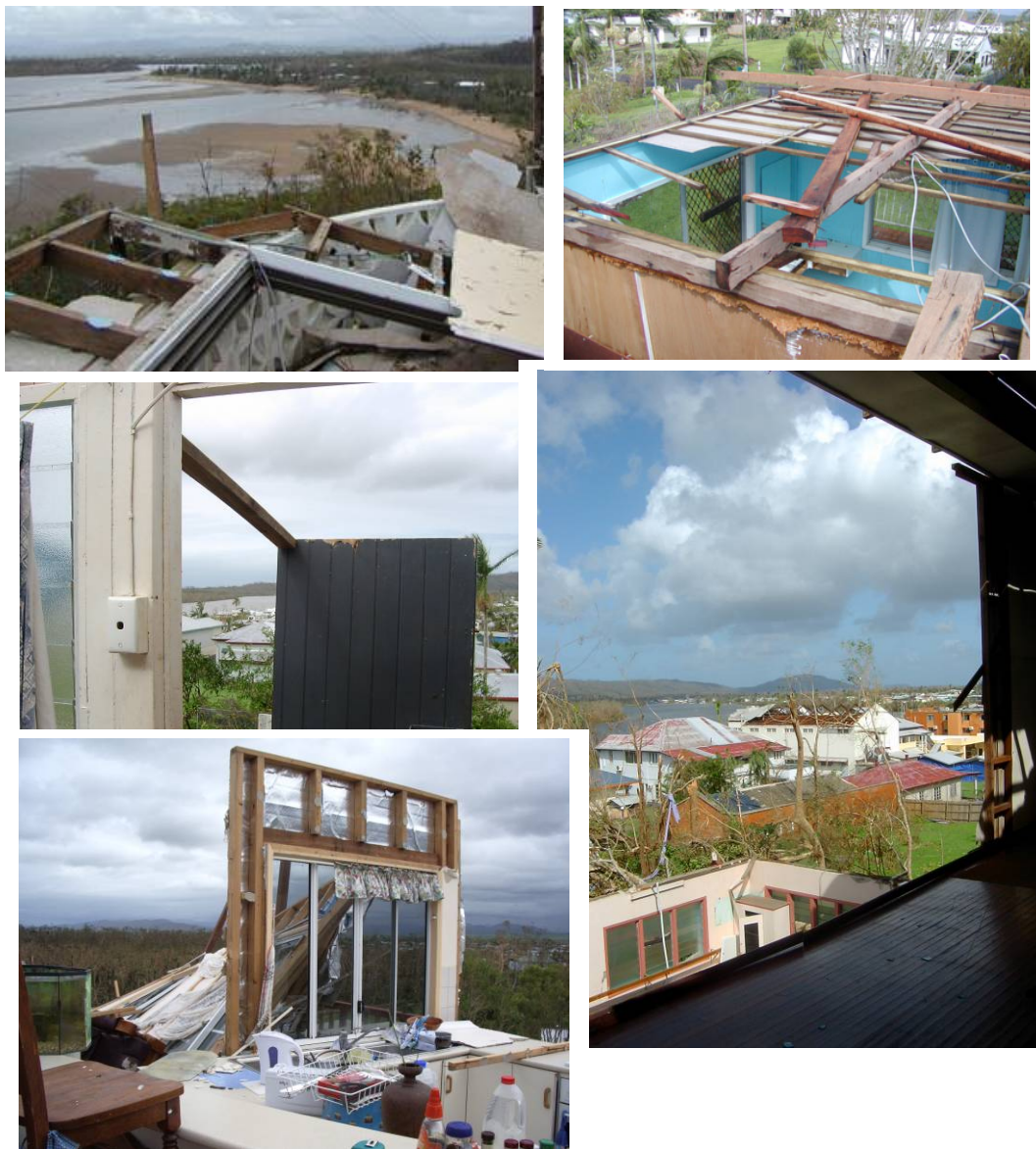
- *No view – C1.* With no view, it is likely that the topography is flat and the site is well shielded with neighbouring (same size or bigger) houses.
- *Some view – C2.* This is likely to be the case if there are a few shielding houses on gently rising ground, or if there are many surrounding houses and moderate slopes.
- *Good view – C3.* This is a view that adds significant value to the block and can only be achieved on moderate slopes with partial shielding or more steeply sloping land with many houses. The anchorage and bracing loads can be found for C3 classifications in AS4055.

- *Really great view – see an engineer.* Great views mean that the site is near the top of a steep slope. Any surrounding houses are not effective in shielding because of the slope, and a professional must be used to design all structural aspects of the house.

**This check should not be regarded as superseding the information in AS4055, but is given as a ready method of checking that the classification is appropriate.**

Also, in assessing the topographic and terrain effects in tropical cyclone areas, the sheltering effects of vegetation should be ignored. This is because it invariably is denuded by winds less than the design wind velocity.

Figure 10.1 illustrates a number of sites in which a great view was accompanied by significant topographic speed up of the wind and resulting structural damage.



**Figure 10.1: High correlation between great view and very high wind speeds**

### **10.1.2 Roof structure anchorage**

Roof structure should be checked on all buildings in which any repair work is to be carried out. In order to ensure sufficient structural capacity to sustain wind loads in future events, anchorages in the roof need to be brought up to the current requirements. Refer to Appendix B for specific details. As a minimum:

- Site should be classified for wind speed according to AS4055. (A rough check has been outlined in Section 10.1.1.)
- Rafter and strut anchorage to top plate should be brought up to capacity required in AS1684.3 by framing anchors or straps.
- Batten to rafter anchorage should be brought to an appropriate level by installation of batten straps or screws.
- Cladding should be fixed in accordance with manufacturers' specifications for the site wind classification.

Where the roofing is removed or partially removed, all of the details mentioned above are accessible and should be checked and upgraded if necessary.

## **10.2 Cold formed steel sheds**

Failure of structural components and members in engineered structures at wind speeds less than the regions design wind speed, is a cause for concern. Design parameters such as local pressure factors, full internal pressure modifiers, and combined load actions need to be properly considered and incorporated into the design. A greater awareness of these issues is required by designers and certifiers.

The shed's site design wind speed needs to be correctly determined for the actual site. This means that generic shed designs need to account for all permutations of terrain, shielding and topography. A dominant opening should be included in the design. Local pressure factors need to be applied to the correct purlin tributary area. Regular inspections and maintenance of the sheds connections and members should be conducted.

## **10.3 Added building components**

Additions such as carports, covered barbeque areas, garages, awnings and extensions have all demonstrated the potential to cause damage to other buildings should they become airborne in tropical cyclones. Their construction should be subjected to the same scrutiny and supervision as housing.

## **10.4 Windows**

In the investigation area, there was little evidence of the use of debris screens for protection of windows. A number of cases were observed in which such protection would not have prevented internal pressurisation.

- Door and window furniture or fixings failed, allowing the closed doors or windows to blow into the building.
- The size of the debris that impacted the building penetrated not only windows, but also doors and walls and so allowed internal pressurisation.

All buildings should be designed and detailed for full internal pressurisation.



Water ingress remains a problem in tropical cyclones. Window seals leak and drainage holes allow water to be blown from the outside of the house to the inside under the differential pressure across the windward wall. The water causes damage to linings and contents, so if the problems of water penetration can be fixed, then the cost of damage from the event can be minimised. It is recommended that detailed study be undertaken to determine cost effective methods of improving performance of buildings against water ingress during tropical cyclones.

### **10.5 Roller Doors**

Roller doors and fitments need to comply with requirements in wind loading standards and the domestic garage doors standard AS/NZS4505. A revision to AS/NZS1170.2 is proposed to clarify and make explicit what is already required in the standard. The amendment will require designers to ensure that roller doors their fitments and supports can withstand the wind pressure loading (including membrane action), otherwise, the building where these doors are to be fitted will need to be designed assuming that the roller door(s) has failed. This amendment is to highlight the issue and create a better understanding within the industry.

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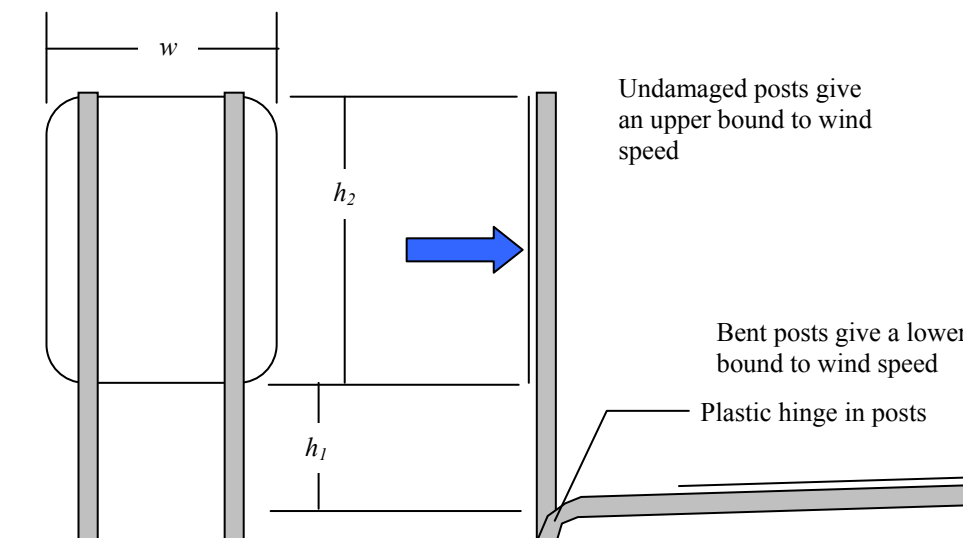
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## Appendix A – Determination of wind speeds from road-sign damage

Section 2.2.1 discussed the concept of using road-signs as devices for estimating wind speed by examining the performance of the posts.

The analysis of different road-signs was used to derive upper and lower bounds as shown in Figure A.1:

- Signs that had a plastic hinge in the posts indicated that the maximum bending moment had exceeded the plastic moment capacity. A sign in this condition could be used to estimate a lower bound on the wind speed providing the sign was free of evidence of impact damage, and the direction of fall was normal to the axis of the sign.
- The upwind terrain and topography was simple and unambiguous.
- The cross section and steel grade of the posts could be used to establish the plastic moment capacity.
- Undamaged posts give an upper bound to the wind speed while bent posts give a lower bound.
- The dimensions of the sign could be used to infer the load that would have been required to exceed the plastic moment capacity.
- The load could be used with the height of the sign and the upwind terrain and topography to deduce the wind speed that was exceeded to cause failure of the posts.



**Figure A.1 Road sign analysis – upper and lower bounds to wind speed**

### A.1 Relating wind speed to sign measurements

The peak net wind load ( $F_n$ ) across the sign can be given by Equation A.1.

$$F_n = \frac{1}{2} \rho \hat{V}_h^2 \cdot C_{F,n} \cdot A \quad (\text{A.1})$$

Here:  $C_{F,n}$  is the net drag force coefficient, equivalent to  $C_{fig}$  in AS/NZS1170.2. [2]



$A$  is the area of the plate (i.e. road-sign)

$\rho$  is the density of air =  $1.2 \text{ kg/m}^3$

$\hat{V}_h$  is the 3s gust velocity at the centroid (ie.  $l = h_1 + 0.5h_2$ ) of the sign

where the plastic hinge is at ground level.

The resulting maximum (i.e. base) bending moment  $M_{max}$  on the post(s) is given by Equation A.2, where the lever-arm  $l$  is the distance between the base and centroid.

$$M_{max} = F_n \cdot l = \left(\frac{1}{2} \rho \hat{V}_h^2 \cdot C_{F,n} \cdot A\right) \cdot l \quad (\text{A.2})$$

$l$  is the distance from the hinge in the posts to the centroid of the sign

The plastic moment capacity of the posts  $M_p$  is given by Equation A.3, where  $f_y$  is the yield strength of the material and  $s$  is the plastic section modulus.

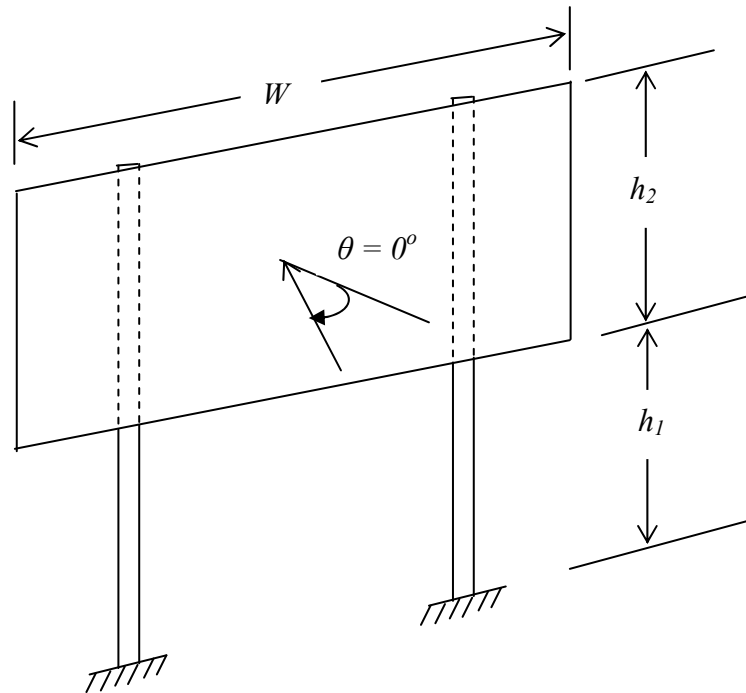
$$f_y = M_p / s ; M_p = f_y \cdot s \quad (\text{A.3})$$

A plastic hinge in the post(s) is created when the bending moment generated by the wind load exceeds the plastic moment capacity  $M_p$  of the post(s), as shown in Equation A.4. The failure wind speed at centroid height is then determined from Equation A.5.

$$M_{max} \geq M_p ; \left(\frac{1}{2} \rho \hat{V}_h^2 \cdot C_{F,n} \cdot A\right) \cdot l \geq f_y \cdot s \quad (\text{A.4})$$

$$\hat{V}_h^2 \geq f_y s / \left[\left(\frac{1}{2} \rho \cdot C_{F,n} \cdot A\right) \cdot l\right] ; \hat{V}_h \geq \sqrt{f_y s / \left[\left(\frac{1}{2} \rho \cdot C_{F,n} \cdot A\right) \cdot l\right]} \quad (\text{A.5})$$

This wind speed is then factored by accounting for the approach terrain and topography to obtain the post failure wind speed in terrain category 2 at 10m height,  $V_r$ .



**Figure A.2 Typical Road Sign**

Importantly, Hoerner [11] and Letchford and Holmes [12] indicate that for plates of these typical dimensions,  $C_{F,n}$  is almost constant for winds approaching within  $\theta = \pm 45^\circ$  from normal to the plate, as shown in Figure A.2. This means that these road signs can be used as robust indicator of wind speeds for winds approaching from two  $90^\circ$  sectors on opposite sides of the compass.

The calculated values of  $V_r$  are dependent on the dimensions of the sign and posts, the strength of the post material, and the values of  $C_{F,n}$  and Terrain Roughness ( $M_{z,cat}$ ). Posts from five failed signs were supplied to the CTS by the Main Roads Qld. Sample lengths of these posts were subjected to 3 point bending tests at the CTS to determine their plastic moment capacities  $M_p$ . Following an analysis of these parameters, failure wind speeds are estimated as  $V_r$ .

This process was used to determine upper and lower bounds to wind speed for a number of signs in the investigation area as detailed in Table A.1.

Figure A.3 shows a bent-over large signage area road sign and an upright smaller signage area road sign located across the Bruce Hwy just South of Innisfail. Three point bending tests carried out on sample lengths of these posts give plastic moment capacities of 4.2 and 4.1 kNm respectively. The corresponding 10m TC2 wind speeds required to create the plastic hinges are 51 (L) and 72 (U) m/s respectively, indicating that the maximum gust wind speed was between these two values in this area.



**Figure A.3: Upper (on left) and Lower (on right) bound wind sign (Bruce Hwy, south of Innisfail)**

**Table A.1 Signs used to Estimate Wind Speeds**

ID	Vr	Vr	U/L	Wind direction	Location	TC	Sign Area	No.	Leg properties		Measured by	Debris free?
	(kph)	(m/s)					(m <sup>2</sup> )		OD	Thickness		
							(mm)		(mm)			
A	124	35	L	S	Palmerston Hwy	2	4.1	2	76.1	2.6	CTS	y
B	154	43	L	SSE	Sth Johnstone	2	2.8	2	60.3	2.9	CTS	y
C	163	45	L	S	Bruce Hwy	2.5	2.0	2	60.3	3.5	MRD	y
D	198	55	U	S	Mourilyan	2	3.1	2	101.6	2.6	GA	y
E	139	39	L	S	Mourilyan	2	4.4	2	88.9	3.1 to 3.4	GA, MRD	y
F	180	50	L	S	Bruce Hwy	2	1.0	1	60.3	3.6	MRD	y
G	132	37	L	WSW	Boulders Rd	3	1.1	1	60.3	3.6	CTS	y
H	140	39	L	NNW	Babinda	3	1.9	2	42.4	3.2	CTS	y
I	215	60	U	SE	Sth Johnstone Res Stn	2	1.7	2	60.3	2.9	CTS	y
J	193	53	U	SSW	Sth Babinda	2	2.9	2	76	4.1	CTS	y
K	164	46	L	SW	Sth Babinda	2	1.1	1	60.3	3.6	CTS	y
L	258	72	U	S	Bruce Hwy, Sth Innisfail	2.5	0.5	1	60.3	3.6	CTS	y
M	184	51	L	S	Bruce Hwy, Sth Innisfail	2.5	1.1	1	60.3	3.6	CTS	y

Vr = estimated wind speed for 10 m height in open terrain

U/L = Upper or Lower Bound sign

TC = Terrain Category

OD = measured outside diameter of pipe

Thickness = Measured pipe wall thickness

Debris free = No indication that sign was bent due to impact from wind driven debris

## **Appendix B – Queensland Repair Guidelines**

The booklet, *Housing Tie down details for repair of cyclone damaged roofs*, was produced by Timber Queensland for the Queensland Department of Local Government, Planning, Sport and Recreation. It was distributed to repairers in the Innisfail region within four weeks of the cyclone impact. It is reproduced in this report with the permission of Timber Queensland.



# Cyclone-damaged roofs

## Queensland repair guidelines

### Tie-down details for repair of cyclone-damaged roofs

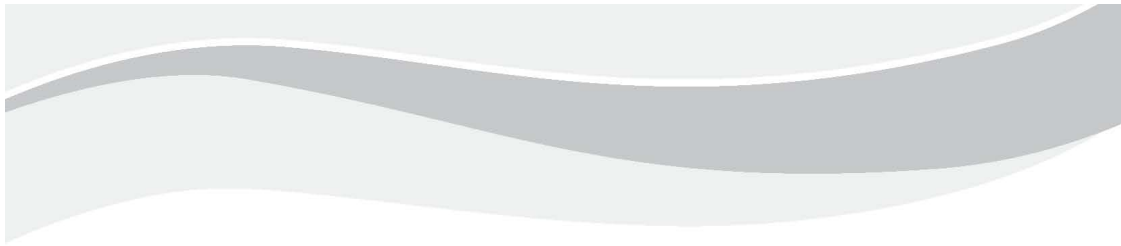


Tie-down details for repair of cyclone damaged roofs



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## Sheet roofs

### Wind Classification C2

#### (maximum design gust wind speed 61m/s = old 'W50')

The following simplified details are provided to assist builders and certifiers in the repair or reconstruction of roofs damaged as a result of a cyclone.

The details cover fixings ranging from roof battens to top plates. Specific details of roof sheeting fixings are given in accordance with the sheeting manufacturers specifications for Wind Classification C2.

In repairing or rebuilding roofs to the standards outlined in this guide, users should be aware that the whole building or structure will not necessarily have been brought up to a standard that complies with current building regulations.

Where more extensive damage has occurred (including to walls), reconstruction, repair or rebuilding should be carried out in accordance with current building regulations such as AS 1684 2006, Residential timber-framed construction, Part 3: Cyclonic Areas.

This guide aims to provide simplified tie-down details for a limited range of building geometries, types and member spacings. For more detailed tie-down design and information, reference should be made to AS 1684.3 – 2006.

## Procedure

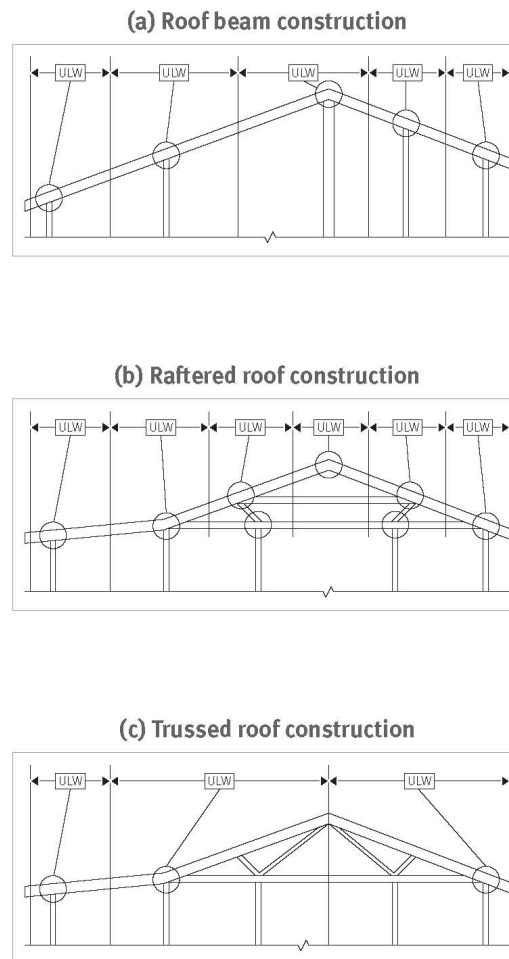
### Roof battens

1. From Table 1, determine the uplift force on the batten to rafter or truss connection.
2. From Detail Sheet 1, determine an appropriate connection with a strength equal to or greater than the uplift force determined from Table 1.

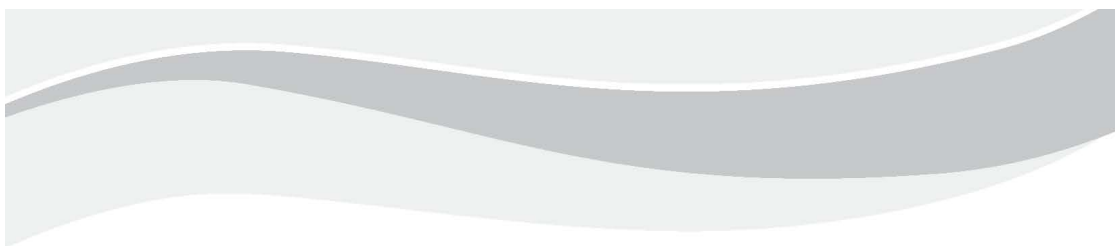
### Roof framing and trusses

1. From Figure 1, determine the Uplift Load Width, ULW, for the tie-down connection under consideration.
2. From Table 2, determine the uplift force on the connection under consideration.
3. From Detail Sheets 2 to 4, determine an appropriate connection with a strength equal to or greater than the uplift force determined from Table 2.

Tie-down details for repair of cyclone damaged roofs

**Figure 1 – Roof uplift load width 'ULW' for wind**



**Table 1 Uplift forces on roof battens**

Rafter or Truss Spacing (mm)	Batten Spacing (mm)	Uplift Force (kN)	
		Near Edges	General area
900	600	3.0	1.9
	900	4.5	2.8
1200	600	4.0	2.5
	900	5.9	3.8

**Note:** Near edges applies to the batten closest to the ridge, the batten closest to the end of the eaves overhang, and to the batten end connections at the gable or hip.

**Table 2 Uplift forces on roof framing**

Uplift Load Width 'ULW' (mm)	Rafter or Truss Spacing (mm)	Uplift Force (kN)
0 to 1500	900	4.4
	1200	5.8
1501 to 3000	900	8.8
	1200	12
3001 to 4500	900	13
	1200	18
4501 to 6000	900	18
	1200	23

**Note:** All timber to be hardwood, cypress or seasoned softwood.

### Uplift force on studs

Uplift force on studs (refer to Detail Sheet 4) at 450mm or 600mm centres will be 50 per cent of the value of the uplift force given for 900mm or 1200mm rafter or truss spacings.

## Other important issues

### Pre-drilling timber

Timber in older homes will be well seasoned and prone to splitting. Splitting will weaken the connection so timber should be pre-drilled to avoid this.

The pre-drilled hole should be no greater than 80 per cent of the diameter of the fastener being used.

### Split, decayed or insect damaged timber

If existing timber members are split, or damaged by rot or insects, the member will be weakened and should be replaced.

### Washer sizes

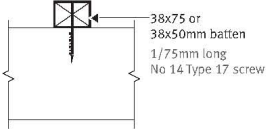
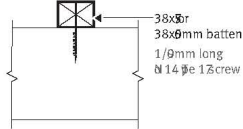
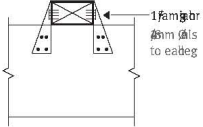
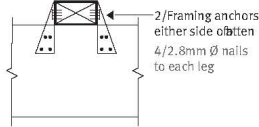
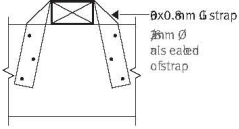
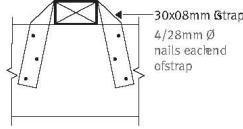
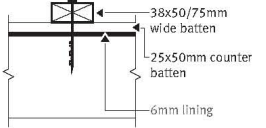
Bolt and coach screw connections will require washers. The following washer sizes are required:

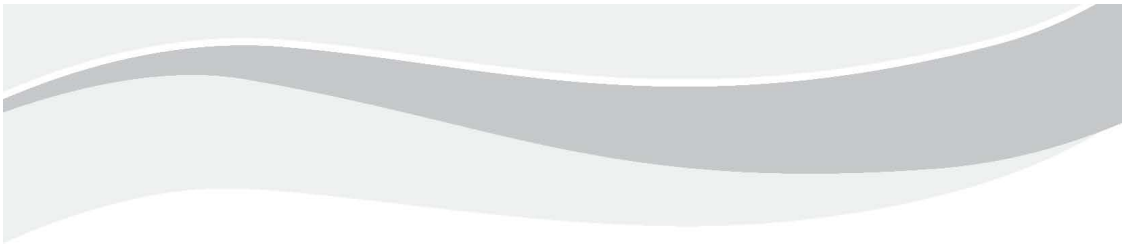
- Where an M10 bolt or coach screw is used – a 38mm X 38mm X 2mm washer is needed.
- Where an M12 bolt or coach screw is used – a 50mm X 50mm X 3mm washer is needed.
- Where an M16 bolt or coach screw is used – a 65mm X 65mm X 5mm washer is needed.

Tie-down details for repair of cyclone damaged roofs

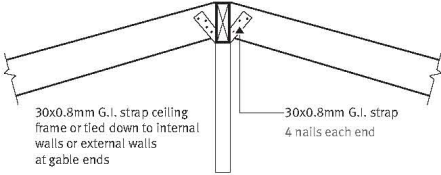
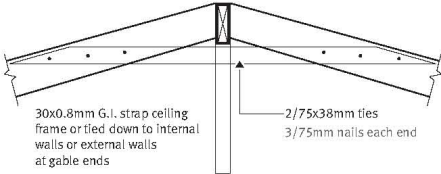
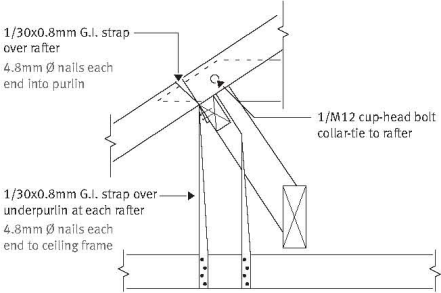
### Detail Sheet 1

#### Roof battens to rafters or trusses

Detail	Uplift capacity (kN)	Detail	Uplift capacity (kN)
<b>1</b>  <p>38x75 or 38x50mm batten 1/75mm long No 14 Type 17 screw</p>	4.5	<b>2</b>  <p>38x50mm batten 1/90mm long No 14 Type 17 screw</p>	6.0
<b>3</b>  <p>1/2mm batten 4/2.8mm nails to eabeg</p>	3.5	<b>4</b>  <p>2/Framing anchors either side of batten 4/2.8mm <math>\phi</math> nails to each leg</p>	5.9
<b>5</b>  <p>30x0.8mm G strap 4/2.8mm <math>\phi</math> nails extending of strap</p>	4.7	<b>6</b>  <p>30x0.8mm G strap 4/2.8mm <math>\phi</math> nails extending of strap</p>	5.9
<b>7</b>  <p>1/90mm long No 14 Type 17 screw 38x50/75mm wide batten 25x50mm counter batten 6mm lining</p>	2.6		



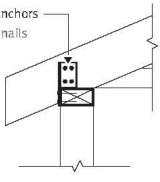
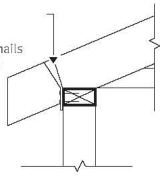
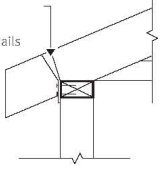
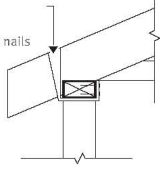
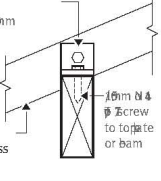
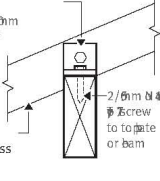
**Detail Sheet 2**  
**Rafters to rafters at ridge, rafter to underpurlins and to ceiling/wall frame**

Detail	Uplift capacity (kN)
<p><b>8</b></p>  <p>30x0.8mm G.I. strap ceiling frame or tied down to internal walls or external walls at gable ends</p> <p>30x0.8mm G.I. strap 4 nails each end</p>	<p><b>4.7</b></p>
<p><b>9</b></p>  <p>30x0.8mm G.I. strap ceiling frame or tied down to internal walls or external walls at gable ends</p> <p>2/75x38mm ties 3/75mm nails each end</p>	<p><b>4.7</b></p>
<p><b>10</b></p>  <p>1/30x0.8mm G.I. strap over rafter 4.8mm <math>\varnothing</math> nails each end into purlin</p> <p>1/M12 cup-head bolt collar-tie to rafter</p> <p>1/30x0.8mm G.I. strap over underpurlin at each rafter 4.8mm <math>\varnothing</math> nails each end to ceiling frame</p>	<p><b>5.9</b> (13 if underpurlin is tied down to wall studs or floor frame with a looped strap or M10 bolt)</p>

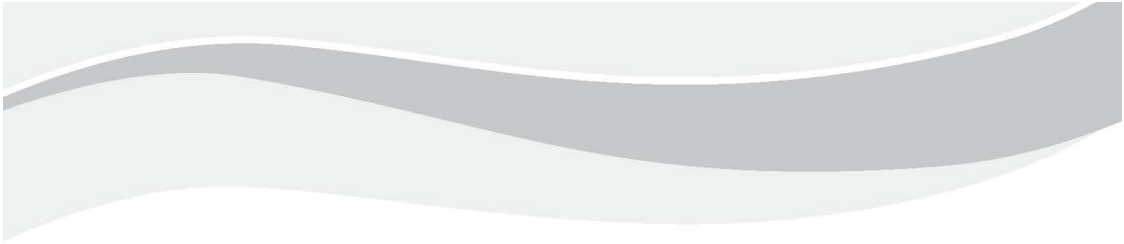
Tie-down details for repair of cyclone damaged roofs

### Detail Sheet 3

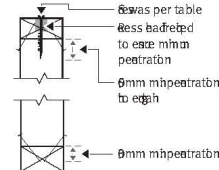
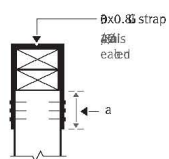
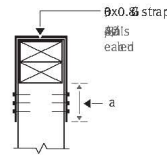
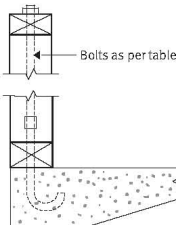
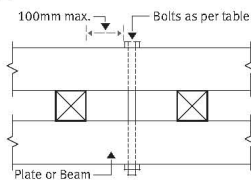
#### Rafters/trusses to top plate or wall frame

Detail	Uplift capacity (kN)	Detail	Uplift capacity (kN)
<b>11</b> 2/Framing anchors 4/2.8mm $\varnothing$ nails to each end 	5.9	<b>12</b> 16mm G strap 2/2.8mm $\varnothing$ nails to each end 	4.7
<b>13</b> 2/16mm G strap 2/2.8mm $\varnothing$ nails to each end 	8.4	<b>14</b> 16mm G looped strap 4/2.8mm $\varnothing$ nails each end 	13
<b>15</b> 10mm Single 1/16" or 2/4" 10mm 1/4" $\varnothing$ screw to rafter 10mm (min.) rafter thickness 	4.2	<b>16</b> 10mm Single 1/16" or 2/4" 10mm 1/4" $\varnothing$ screw to rafter 10mm (min.) rafter thickness 	5.1





**Detail Sheet 4**  
**Top plates to studs and general connections**

Detail	Uplift capacity (kN)	Detail	Uplift capacity (kN)
<div><b>17</b> </div>	<b>No 14 Type 17 screw or M10 coach screw</b> <b>3.6</b>	<div><b>18</b> </div>	<b>5.9</b>
<div><b>19</b> </div>	<b>8.4</b>	<div><b>20</b> </div>	<b>M10</b> <b>15</b> <b>M12</b> <b>20</b> <b>M16</b> <b>35</b>
<div><b>21</b> </div>	<b>M10 cup head</b> <b>10</b> <b>M12 cup head</b> <b>12</b> <b>M12 bolt/rod</b> <b>20</b>		



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