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1. Introduction

Roofs during windstorms are subjected to intense fluctuating external pressures at windward roof edges. Loads on these areas can be greatly increased when combined with large positive internal pressures resulting from a breached windward wall, giving a large net uplift load. Thus the roof envelope and fixings generally experience the highest wind pressures of the building’s components, and are the components most susceptible to failure.

Cyclone Tracy caused catastrophic damage in Darwin in 1974. Walker (1975) reported over 90% of houses and 70% of other structures suffered significant loss of roof cladding as shown in Figure 1.1(a). The extensive loss of light gauge metal roof cladding was caused by low cycle fatigue of the cladding adjacent to its fasteners (Morgan and Beck 1977). With the cracking allowing the cladding to pull over one fastener, this led to an avalanche effect of overloading and failing the cladding at adjacent fasteners as shown in Figure 1.1(b). Following Cyclone Larry’s impact on Innisfail in 2006, fatigue failure of pierced fixed metal cladding was observed where the fixing centres exceeded typical product specifications (Henderson et al. 2006).

Figure 1.1: (a) Catastrophic failure of metal roof cladding, and (b) Cladding pulling over heads of fasteners (CTS collection)
A major component of insurance losses from tropical storms is the breach of the building envelope allowing the wind and rain into the building (Beaumont 2009). In the wake of a severe cyclone, repairs to houses and commercial properties can take several months to years to be completed (Henderson and Ginger 2008). Therefore, ensuring the elements of building envelopes are capable of resisting severe wind loads is crucial to the resilience of the community.

Low cycle fatigue was defined by Beck and Stevens (1979) as failure typically within 10000 load cycles. Following Cyclone Tracy, the Darwin Reconstruction Committee (DRC) needed a test to evaluate roofing to ensure a safer community. The DRC expediently stipulated a test method of 10000 cycles from zero to the permissible stress design load (suction) followed by a proof load of $1.8 \times$ design load, for evaluating roofing. This test regime was subsequently incorporated into the Darwin Area Building Manual (DABM). Morgan and Beck (1977) noted that the conservative DABM test regime was to be used as an interim acceptance criteria and commented that this should be replaced with a more realistic test, when additional data becomes available.

Further research led to the “Guidelines for the testing and evaluation of products for cyclone prone areas”, commonly referred to as TR440 (1978). The TR440 loading regime was subsequently rejected by the building regulators in Darwin on the basis that roofing systems, similar to those that failed during cyclone Tracy, were “satisfying” the test criteria. Thus, a building product evaluated for these two different test regimes, although meant to represent the same cyclonic loading process, can give different test outcomes. This lack of consistency has been mostly addressed with the introduction of a standard test method, the Low-High-Low (LHL) cyclic load test for metal roofing (BCA 2006). However, the LHL is built on several assumptions whose cumulative effect may not be completely understood.

With the advent of advanced pressure loading actuators (PLA), realistic fluctuating wind pressures can now be applied repeatedly to building elements. Coupled with time series pressure data from wind tunnel models, building cladding specimens can be subjected to generated cyclonic pressure traces to evaluate previous and existing test processes.
1.1 Objectives

*Why are we doing this?* In order to have efficient cladding designs and resilient roofs, we need to understand the damage mechanisms and estimate the onset of cracking of pierced fixed metal roof cladding when subjected to cyclonic winds.

*Do this by:* Understanding wind loads, material response, and variability associated with loading, materials and construction.

*Leads to:* Resilient structures and better use of materials

The objectives of this work involve several issues;

- Standardised tests apply the “same load” to all fasteners across a test specimen. Wind tunnel test data and full scale measurements show the wind pressure is highly turbulent and temporally and spatially varying across the building envelope. Information on the effective cladding tributary area for a typical fastener and the cladding’s extent of influence on adjacent fasteners is required in assessing the adequacy of the assumption of applying a uniform load across the test specimen;

- Investigation of peak cladding pressures and equivalent loading cycles for the corner and general roof areas for representative low rise buildings with and without wall openings needs to be assessed through the evaluation of the response of a typical cladding system to fluctuating wind pressures through the cladding’s deformation, cracking and loads at fixings;

- Comparison of the current low cycle fatigue cladding test criteria with the realistic inclusion of net pressures generated from dominant openings and different cyclone scenarios; and

- Determining an appropriate method for the prediction of levels of cladding damage for various building and cyclonic wind loading scenarios by analysing the pressure load cycles contributing to the cracking of cladding.

1.2 Overview

Chapter 1 presents the historical context of low cycle fatigue failure of roof cladding in Australia and lists the objectives. Chapter 2 contains a literature review examining fatigue failures of cladding and the previous research conducted on low cycle fatigue of roofing. The chapter shows
that linear elastic and more complex fatigue theories and models do not easily apply since the cladding design standards allow plastic deformation and cracking of the cladding in resisting the extreme design loads of cyclonic events. The chapter details the benefits and limitations of past and present Australian cladding test standards and typical methods for simulating cyclic loads for conducting the tests. The pressure loading actuator (PLA) is described along with its ability to supply realistic fluctuating wind pressures.

Chapter 3 describes the experimental methods and equipment used for applying static, cyclic and fluctuating wind pressures to cladding specimens as well as the equipment used to measure the displacement of the cladding, the movement of the screws and the load at the screws.

The response of corrugated pierced fixed cladding specimens subjected to static and cyclic air pressures is detailed and discussed in Chapter 4. Different loading scenarios with variations in cycle rate, cycle shape, load per cycle and material strength are presented. Comparisons are made between results obtained from previous line-load test methods with those from uniformly distributed air pressure tests conducted in this study.

Chapter 5 proposes a representative tributary area for a cladding fastener based on influence coefficients measured for fastener reactions and analysis of wind tunnel time series data from a high density pressure tap model. Comparisons of peak pressures and numbers of load cycles derived from the rainflow count method are made between different numbers of taps for area averaging and the influence coefficient method.

Chapter 6 details the application of fluctuating pressures representing wind loads, thereby enabling the assessment of cladding response to these dynamic pressures and a comparison to cyclic load tests. A damage index metric is proposed and assessed against several simulated cyclonic wind traces using the PLA. Historical cyclone traces are evaluated against the “design” cyclone and the current Australian cladding design fatigue test method, the L-H-L test.

Chapter 7 summarises the findings from the experimental testing and time series analysis of the response of pierced fixed corrugated metal roof cladding to a range of loading regimes. These loads vary from steadily increasing to constant amplitude cyclic to simulated cyclonic wind loads lasting hours. Conclusions and recommendations developed from this study are offered.
2. Literature review

2.1 Northern Australian metal roofing

2.1.1 Roof envelope

Typically in the northern regions of Australia, low rise buildings (houses, commercial premises, light-industry sheds, etc) have roofs covered with light gauge metal cladding.

For domestic construction, the metal cladding is screw fixed to battens spaced typically no more than 900 mm apart. The battens are traditionally made from hardwood but light gauge metal top hat shape battens are becoming common. The battens are screw fixed to trusses that are generally spaced at 900 or 1200 mm. The roof has a slope (pitch) typically in the range of 15 to 25 degrees. A multi-hip roof is shown in Figure 2.1; another is shown during construction in Figure 2.2.

![Figure 2.1: Multi-hip roof on recently constructed house](image1)

![Figure 2.2: Roof under construction](image2)
Light-industry and low rise commercial buildings, shown in Figure 2.3, typically have flat (3 to 10 degree) pitch roofs. The metal cladding is fixed to purlins that are spaced about 1200 to 1500 mm apart. The purlins span several metres between portal rafters.

![Figure 2.3: Industrial shed](image)

### 2.1.2 Light gauge metal roofing

The most common pierced fixed light gauge metal cladding profiles are either corrugated or rib-pan profiles shown in Figure 2.4 (a) and (b), respectively. The cladding is rolled from 0.42 mm or 0.48 mm base metal thickness (bmt) G550 coil, or 0.60 mm bmt G300 coil.

“G550” or “G300” refers to the specified characteristic yield strength of 550 MPa or 300 MPa, respectively. The coil material specifications are detailed in the Australian Standard AS1397 (Standards Australia 1993).

The high strength and thin gauge of the steel sheet is achieved from the cold reduction manufacturing process that rolls the steel from, typically, 2.5 mm down to 0.42 mm thickness. The steel coil is subjected to compressive forces through its section (between rollers) and tensile stresses along the coil to achieve this reduction in thickness. This process of cold reduction causes the grain structure of the steel to elongate in the rolling direction, giving the steel anisotropic properties across and along the rolling direction.
The directional difference in properties is partially mitigated through subsequent heat treatments. The G550 steel is stress relief annealed by heating the steel to below the re-crystallisation temperature followed by slowly cooling. In contrast, for the manufacture of the G300 coils, the cold reduced steel is heated beyond its re-crystallisation temperature where the grains return to a more random distribution. The corrosion protection coating (e.g., zinc-aluminium) is applied after the annealing process (BHP 1992).

2.1.3 G550 properties

Typically, the ultimate tensile strength of the G550 material significantly exceeds the specified strength of 550 MPa.

Rogers and Hancock (1997) conducted 370 tensile coupon tests to determine material properties of G550 and G300 flat coil. The coupons were cut from the longitudinal, transverse and diagonal directions, as shown in Figure 2.5. The specimens were all taken from the one coil for each strength grade material. From their tests on 0.42 mm bmt G550, the mean ultimate tensile
strengths for the longitudinal, transverse and diagonal directions were 681, 768 and 670 MPa, respectively. The yield and ultimate strengths of the coupons were the same, as plastic deformation did not take place. The mean Young’s modulus (E) for the 0.42 mm G550 was 219, 252 and 192 GPa, in the longitudinal, transverse and diagonal directions, respectively.

![Figure 2.5: Orientation of tensile test coupons (Rogers and Hancock 1997)](image)

Larger ultimate strengths of 720 and 780 MPa, for the longitudinal and transverse directions, respectively, were measured by Xu (1993). Results from a BHP study, that tested 15 different coils, were produced by Rogers and Hancock (1997), showing the variability in the longitudinal tensile strength ranged from 580 to 870 MPa. This variability of material properties in the base material does complicate experimental cladding test programs, as noted by Henderson et al. (2001), and Mahaarachchi and Mahendran (2004).

### 2.2 Fatigue failure

Fatigue failure occurs when a component fails following the application of fluctuating loads of lower magnitude than the component’s static load capacity (strength).

#### 2.2.1 Failure modes

Typically, the in-service life of designed structural components is within its material elastic stress range. That is, the component’s dimensions return to its unloaded state when the applied load is removed. However, if the stresses exceed their elastic capacity, permanent deformation occurs within the material. If loading continues (not necessarily at a higher load) the deformation increases which results in a reduction in cross section of the material and ultimately failure. This process is described as plastic deformation progressing to a ductile failure. Conversely materials,
such as glass, fail through crack propagation, showing no measurable plastic deformation. The process is described as a brittle failure. Typically, the failure modes of common engineering materials lie between these two processes (Smith 1991).

As discussed in Section 2.1.3, G550 is an anisotropic material that has properties closer to a brittle material, especially in its transverse (across roll) direction.

### 2.2.2 Load cycles

A single loading cycle acting on a structural component can be represented as shown in Figure 2.6, where the load varies between a maximum, $S_{\text{max}}$ and a minimum $S_{\text{min}}$, with a mean value $S_{\text{mean}} = (S_{\text{max}} + S_{\text{min}})/2$, a range $\Delta S = |S_{\text{max}} - S_{\text{min}}|$ and an amplitude $S_{\text{amp}} = \Delta S / 2$. Furthermore, a load ratio is defined as $R = S_{\text{min}} / S_{\text{max}}$, giving $R = -1$ for alternating loads and $R = 0$ for pulsating loads returning to an unloaded state during each cycle.

![Figure 2.6: Load cycles](image)

### 2.2.3 Miner’s rule

Miner’s rule is often used to predict the performance of metal components subjected to repeated loading. It was developed from research into the fatigue life of thick steel elements, such as machinery components subjected to a large number of low amplitude cycles. Miner’s rule states that the amount of accumulated damage at a given level of stress is proportional to the number of cycles, $n_i$, to the total number for failure, $N_i$, at that stress. That is, when failure takes place:

$$\frac{n_1}{N_1} + \frac{n_2}{N_2} + \frac{n_3}{N_3} + \ldots = \sum \frac{n_i}{N_i} = 1$$

Thus, fatigue failure may occur from a very large number of low level stress cycles or from a few cycles at a level near the ultimate static capacity.

Birnbaum and Saunders (1968) refer to the “infamous” Miner’s rule for dealing with fatigue failure with a deterministic solution and that the applicability of Miner’s rule is dependent on the
loading sequence of the cycles and may overestimate the true number of cycles to failure. They do also note that for many cases the rule gives reasonable results and is very simple to apply.

2.2.4 Fracture mechanics

Predominantly, fracture mechanics deals with the prediction of crack growth and the determination of the critical crack length under various loadings at which sudden catastrophic failure could occur. Common engineering components are manufactured in many different shapes and are connected with bolts, screws, or welds, etc. These changes in geometry, or holes for fixings, or flaws and stresses in welds can magnify (or concentrate) the nominal stresses from applied loads. These stress concentrations are often responsible for rapid crack growth.

The application of fracture mechanics to study the failure of components by analytical methods is complex (Hussain 1997; Smith 1991). In its simplest form the Linear Elastic Fracture Mechanics (LEFM) solution for unit applied stress surrounding a thin elliptical hole in an infinite, elastic, homogeneous and isotropic plate under uniform tensile load is given in Equation 2.1:

\[ \sigma = 1 + 2 \frac{\sqrt{a}}{\sqrt{r}} \]  

(2.1)

where \( r \) is radius and \( a \) is half crack length.

As the radius approaches zero the ellipse mimics a flat crack where the stress becomes theoretically infinite at the crack tip. This limit is never reached for real materials since some amount of plastic deformation will occur at the crack tip.

During load cycling the crack extends into this small plastic zone at the crack tip with each load cycle – crack growth by fatigue. The crack growth is taking place at loads below that required for failure under a static load. The rate of crack growth increases as the crack becomes larger and total failure occurs when the crack reaches a critical length.

The opening of the crack from the applied loads normal to the direction of the crack is referred to as Mode I (opening mode). Two other crack propagation modes are defined as Mode II (sliding mode) and Mode III (tearing mode). In LEFM, if two or three modes are acting simultaneously (mixed mode problem), the corresponding stresses and displacements from each mode may be simply added together (principle of superposition).

Techniques based on LEFM can be used to analyse the behaviour of stresses near a crack for various geometric and loading arrangements. For example, Tada et al. (2000) consider a crack
length $2a$, that is present through the thickness $t$ of the sheet of width $2b$ and subjected to a nominal stress $\sigma$. The stresses in the direction of the crack ($y$) and transverse direction to the crack ($x$), at a point $(r, \theta)$ where $r$ is the distance from the crack tip and $\theta$ is the angle from the transverse direction under Mode I cracking, are given by Equations 2.2 and 2.3:

$$\sigma_y = \frac{K}{\sqrt{2\pi r}} \cos \theta \left[ 1 + \sin \frac{\theta}{2} \sin \frac{3\theta}{2} \right]$$  \hspace{1cm} (2.2)

$$\sigma_x = \frac{K}{\sqrt{2\pi r}} \cos \theta \left[ 1 - \sin \frac{\theta}{2} \sin \frac{3\theta}{2} \right]$$  \hspace{1cm} (2.3)

where the stress intensity factor is given by $K = Q \sigma(\pi r)^{0.5}$ and $Q$ is a configuration correction factor to account for the effect of geometry and loading on $K$.

The stress intensity factors for typical materials and loading configurations are available in graphs and tables given in, for example, Rooke and Cartwright (1976). The rate of crack growth for many applications can be predicted using the theory of fracture mechanics with these typical experimentally derived coefficients.

The growth of a crack under constant amplitude cyclic loading is typically analysed by plotting a graph of half crack length, $a$, against the number of cycles, $N$. This relationship may also be expressed in terms of the range of the stress intensity factor ($\Delta K$) for the constant amplitude cycles. The rate at which a crack grows as the number of cycles is increased is plotted as log($da/dN$) vs log($\Delta K$), and consists of three regions, I, II and III, as shown in Figure 2.7.

- In region I, the crack does not start to grow until $\Delta K$ reaches a threshold value, regardless of how many cycles are applied. However, as soon as a crack begins to grow, the crack growth rate increases very rapidly as $\Delta K$ increases with crack length.
- Region II contains cracks of detectable size, which grow in such a way that the log-log graph is almost linear.
- In region III, the crack is large and plasticity begins to dominate as the crack grows to failure. As the stress intensity factor at the tip of the crack, $K$, increases with increased loading, it may reach the value of $K_c$, defined as the critical stress intensity factor, when the balance of elastic energy release from the loaded body exceeds the energy requirement for crack extension. At this point a running crack that is known as unstable fracture takes place. If unstable fracture of the structure is to be avoided, the stress intensity factor, $K$, at the tip of the crack should be kept at a value less than the characteristic $K_c$ of the material under investigation (Rogers and Hancock 2001). $K_c$ is typically determined through experiments.
In the analysis of fatigue crack growth, attention is typically focused on region II, where a linear relationship is given by
$$\log(\frac{da}{dN}) = m \log(\Delta K) + \log(c),$$
where $m$ and $c$ are constants. This leads to the Paris Law,
$$\frac{da}{dN} = c(\Delta K)^m,$$
which is used to predict the number of cycles required for a crack to grow from $a_i$ to $a_F$ in Equation 2.4.

$$N = \int_{a_i}^{a_F} \frac{1}{c(\Delta K)^m} \, da$$  \hfill (2.4)

**Figure 2.7: Logarithmic crack growth rate (Smith 1991)**

In thick sections (plane strain conditions), it is more difficult for plastic deformation to occur ahead of the crack tip. However for thin sections (plane stress), crack extension requires more energy in the form of plastic work.

A basic condition of the LEFM (including the Paris Law) is that applied stresses are in the materials elastic range, that is well below the yield of the material, and the crack tip plasticity should be small when compared to crack length. Another limitation of basic fracture mechanics analysis of fatigue crack growth assumes microstructural independence. Also LEFM does not model the growth rate of short fatigue cracks correctly (Hussain 1997). Broberg (1995) states that the difficulties associated with nonlinear fracture mechanics are related to the large scale of yielding.

The J-integral is the average measure of the elasto-plastic stress-strain field ahead of the crack. The approach is often used for the stress intensity analysis of materials, that exhibit plastic deformation near the crack tip, by examining the stress field in a contour outside this plastic zone (Rogers and Hancock 2001; Smith 1991). This method is unlikely to be useful in the analysis of crack growth for claddings as the crack is in a region of bulk plasticity (adjacent to head of screw fixing) many times larger than the crack length or material thickness.
Small scale methods, such as the crack tip opening angle (CTOA) or crack tip opening
displacement (CTOD), are used in determining the fracture toughness of a material that exhibits
plastic deformation. These methods are a measure of the pre-fracture deformation of the tip of a
sharp crack during the inelastic deformation range (Rogers and Hancock 2001; Smith 1991).

Li and Siegmund (2002) note the success of the crack tip based criteria in predicting crack growth
and direction during mixed mode fracture. However, they point out that the CTOA and CTOD
criteria are based on local deformation measures, do not take into account the micro scale failure
processes and, therefore, cannot accurately predict crack initiation. They add that the cohesive
zone model (CZM) is an appropriate method for the analysis of cracks subjected to the
combination of tensile loading due to membrane stresses and out of plane deformation. The tensile
loading and large scale out of plane deformation are both present in loaded cladding as discussed
in Section 2.4

2.2.5 Variable amplitude loading
A larger stress cycle in the midst of constant amplitude cycles increases the plastic zone ahead of
the crack tip (Smith 1991). As the crack grows through the plastic zone, its growth rate increases
until the tip reaches a point where the zone size corresponds to that for the current stress amplitude
at which time the crack grows at the constant amplitude rate. Smith (1991) notes that a large
positive cycle has more effect on the plastic zone than a large positive-negative cycle. Wind
pressures generate considerable variable loading, as will be discussed in Section 2.5.3

2.2.6 Fracture mechanics design strategies
Three common design strategies are: safe life, damage tolerant, and fail safe (Smith 1991). The
strategy that would appear to be closest to current cladding design for cyclonic regions is the
damage tolerant approach. It is based on the assumption that cracks exist within the structure and
will grow in the highly stressed areas during a fatigue load. According to this strategy, the design
needs to consider the largest crack that can go undetected, the critical crack length at which sudden
failure can occur, and the loading history that allows the crack to grow from the undetected length
to the critical length. For design purposes, serviceability criteria also need to be considered over
the time period of crack growth.

2.2.7 Short fatigue cracks
Short fatigue cracks are generally considered to be 0.005 to 0.5 mm in length. Hussain (1997)
defines the short crack as the crack length which shows high growth rates when compared to long
cracks. The higher growth rates may be observed when the cracks are of a comparable scale to that
of the local plasticity or when the cracks are of similar length to some microstructural parameter, such as grain size. The orientation of grains in relation to the direction of crack growth may increase, decrease or halt crack growth. It is therefore assumed that the strong anisotropic grain structure of G550 material will play a significant role in crack initiation and initial crack growth rate.

Non-linear predictive models, such as those developed by Navarro and Hussain (1997), incorporate effects such as crack closure and crack propagation through grains and grain boundaries. These models, just like models based on LEFM, require empirical coefficients derived from experimental results and are typically for thicker materials (i.e., plane strain).

2.3 Fracture toughness of G550 flat sheet

Rogers and Hancock (2001) conducted experiments on G550 tension test specimens to evaluate the Mode I fracture resistance in the elastic deformation range. They obtained $K_c$ values of 3767, 3182 and 3748 MNm$^{-3/2}$ for the longitudinal, transverse and diagonal directions, respectively, for 0.42 mm G550 material. The authors refer to the notch tensile test method described in Rooke and Cartwright (1976). They conclude, from their parametric study of perforated sections, that the probability of failure by unstable fracture increases with increasing specimen width, and for specimens that are loaded in a localised manner.

2.4 Fatigue performance of G550 cladding

The fatigue behaviour of the cladding is dependent on the load causing local plastic deformation (LPD), seen as dimpling under the screws around the fastener holes, shown in Figure 2.8. This is about 600 N per fastener for corrugated cladding fixed without cyclone washers. LPD strength strongly influences the fatigue life. The resistance to fatigue of corrugated cladding increases markedly if the cyclic load per fastener is kept below this LPD load (Beck and Stevens 1979; Mahendran 1990; Xu 1993). Beck and Stevens (1979) found that the crack orientations and sizes, observed from damage inspections conducted after Cyclone Tracy, resembled the repeated loading, cladding failure modes in the laboratory tests.
Xu (1995a) found that the cladding profile (cross section shape) does affect the process of fatigue damage accumulation. The fatigue resistance of corrugated cladding to low amplitude block loading is higher than that of rib-pan but, under high amplitude loading, the situation is reversed.

A variation in geometry (shape) of the same profile (differences between roll formers) affects both the LPD load and the fatigue resilience. The use of cyclone washers increases the LPD strength by increasing the bearing area under the fixing head, delaying cracking. The washers also delay the formation of cracks for the same load and the crack has further to propagate before failure.

Beck and Stevens (1979) showed that decreasing the range of the load cycles following crack initiation from a higher load range, resulted in slower crack growth and longer life than if the higher cycle load range was continued. The opposite occurred when loads were increased after the initiation of a crack, resulting in shorter life (Mahendran 1990). The latter finding agrees with the hypothesis of cracks extending through the plastic zone at the crack tip, as discussed in Section 2.2.5.

From extensive constant amplitude repeated load tests for the corrugated profile, a range of crack propagation modes were observed for cycling at different load levels (Mahendran 1990; Xu 1995b). A sample of the data collected by Mahendran (1989) is given in Figure 2.9. This data shows the decreasing number of cycles to failure for the increasing load per cycle. Furthermore, the differences in the number of cycles to failure, crack location and crack type for similar load ranges (lines 3, 4 and 5, or 8 and 9 in Figure 2.9) highlights the variable nature of fatigue load
testing of claddings, that is not just limited to the variation in material properties across the coil, but also includes tightness, alignment and position of the screw on the crest or rib.

![Figure 2.9: Constant amplitude block loading (Mahendran 1989)](image)

<table>
<thead>
<tr>
<th>Cyclic Load Range (N/f)</th>
<th>Number of Cycles to Crack Initiation N₀</th>
<th>Number of Cycles to Failure N_f</th>
<th>Type of Crack</th>
<th>Pull Through at</th>
</tr>
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<tbody>
<tr>
<td>(1) 0 - 300</td>
<td>n.o.</td>
<td>362,200</td>
<td>B</td>
<td>Edge Hole</td>
</tr>
<tr>
<td>(2) 0 - 325</td>
<td>110,000</td>
<td>290,140</td>
<td>B</td>
<td>Edge &amp; Central Holes</td>
</tr>
<tr>
<td>(3) 0 - 344</td>
<td>n.o.</td>
<td>274,150</td>
<td>B</td>
<td>Edge Hole</td>
</tr>
<tr>
<td>(4) 0 - 350</td>
<td>22,000</td>
<td>62,750</td>
<td>B</td>
<td>Central Hole</td>
</tr>
<tr>
<td>(5) 0 - 350</td>
<td>n.o.</td>
<td>320,450</td>
<td>B</td>
<td>Edge Hole</td>
</tr>
<tr>
<td>(6) 0 - 375</td>
<td>n.o.</td>
<td>102,200</td>
<td>B</td>
<td>Central Hole</td>
</tr>
<tr>
<td>(7) 0 - 387</td>
<td>n.o.</td>
<td>68,650</td>
<td>B</td>
<td>Edge Hole</td>
</tr>
<tr>
<td>(8) 0 - 400</td>
<td>n.o.</td>
<td>18,160</td>
<td>A-B</td>
<td>Central Hole</td>
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<tr>
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<td>n.o.</td>
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<tr>
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<td>15,600</td>
<td>B</td>
<td>Edge Hole</td>
</tr>
<tr>
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<td>Almost B</td>
<td>Central Hole</td>
</tr>
<tr>
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<td>4,420</td>
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<td>Central Hole</td>
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<td>340</td>
<td>A</td>
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</tr>
</tbody>
</table>

Note: n.o. refers to “not observed”

For the constant amplitude load tests, when the load per cycle was well below the LPD load, cracks propagated from the screw hole along the crest (Type B in Figure 2.10 and Segment 1 in Figure 2.11). Xu (1995b) notes this may be attributed to local transverse bending moments and the more brittle properties in the transverse direction. When the load per cycle approached the LPD load, cracks propagated in both the longitudinal and transverse directions (e.g., Type A-B in Figure 2.10 and approaching Segment 2 from Segment 1 in Figure 2.11). For load cycling through the LPD load, cracks initiated at the edges of the flattened crests where the cladding creases, with the cracks then progressing towards the screw hole and failure within 1000 cycles (e.g., Type A in Figure 2.10 and Segment 2 in Figure 2.11). The Type C failure is from a few cycles at high loads, similar to a straight static pull through failure. Similar studies were conducted on rib/pan profiled cladding (Xu 1992).
Figure 2.10: Crack patterns from constant amplitude cyclic loading (Xu 1995b)

Figure 2.11: S-N curve for corrugated cladding fastened without cyclone washers (Mahendran 1989)

The $S_{max}$-N curve, shown in Figure 2.11, produced by Mahendran (1990), was for corrugated cladding specimens spanning 650mm batten spacings, with $R = 0$, that is, a load cycle that returns to an unloaded state each cycle. The effect of varying $R$ on the fatigue performance is seen by superimposing $S_{max}$-N curves for a range of $R$ values from analysing the data presented by
Mahendran (1990). Figure 2.12 shows that increasing R (i.e., reducing $\Delta S$) for a given $S_{\text{max}}$, results in an increase in the number of cycles to failure. Section 2.5.4 shows that the majority of wind load cycles have $R > 0$, suggesting that the majority of cyclic tests with $R = 0$ may be conservative and not representative of wind load cycles during a cyclone.

Each mode of crack initiation and propagation indicate a different fatigue response depending on the load level. Miner’s rule relies on constant material properties and does not satisfactorily deal with this situation of changing profile shape, strength and stiffness (Beck and Stevens 1979; Mahendran 1993; Xu 1993). A modified Miner’s rule was suggested that used different empirical constants depending on the load level. However, this method did not adequately predict fatigue damage, for the cladding, from cycle histories especially when lower load cycles followed higher load level cycles.

![Figure 2.12: Cycles to failure (Henderson and Ginger 2005)](image)

2.5 Wind loading

2.5.1 Tropical Cyclones

A tropical cyclone is a large scale severe low pressure weather system where, in the Southern Hemisphere, winds circulate clockwise around the centre. Tropical cyclones can affect the Northern Australian region typically between November and April. Cyclones form over warm oceans in areas of relatively low surface pressures. Air moves towards the centre of the low
pressure system in a spiral due to the Coriolis effect (Pielke and Pielke 1997). The warm ocean through evaporation provides moisture to the air. As the moisture laden air spirals inwards it is forced upwards which serves to lower pressures further near the surface. With rotational winds of roughly 30 m/s the air no longer reaches the centre giving rise to the cyclone’s calm eye.

The destructive force of cyclones is usually expressed in terms of the strongest gusts likely to be experienced, which is related to the central pressure, speed of movement and internal structure of the storm system. The Bureau of Meteorology uses the five-category system shown in Table 2.1 for classifying tropical cyclone intensity in Australia.

Table 2.1: Australian tropical cyclone category scale

<table>
<thead>
<tr>
<th>Cyclone Category</th>
<th>Gust Wind Speed at 10 m height in flat open terrain</th>
<th>Central Pressure</th>
</tr>
</thead>
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<tr>
<td></td>
<td>km/h</td>
<td>knots</td>
</tr>
<tr>
<td>1</td>
<td>90-124</td>
<td>50-68</td>
</tr>
<tr>
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<td>122-155</td>
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<tr>
<td>5</td>
<td>&gt;280</td>
<td>&gt;151</td>
</tr>
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</table>

The main features of a severe tropical cyclone at the earth’s surface are the eye, the eye wall and the spiral rain bands. The eye is the area at the centre of the cyclone at which the surface atmospheric pressure is lowest and where the wind is slight and the sky is often clear. The cyclone’s intense winds are associated with the eye wall. For any given central pressure, the spatial size of individual tropical cyclones can vary enormously. Severe cyclones can have eye diameters from 15 to 50 km.

The vulnerability of society to extreme events such as cyclones is a function of (a) the event’s incidence (frequency, strength and location), and (b) society’s exposure (people, preparedness and infrastructure) (Pielke and Pielke 1997). Improving buildings’ resilience to these severe events reduces society’s vulnerability.

The impact wind speed on a structure is dependent on its position relative to the cyclone’s path, diameter and forward speed. The structure’s position relative to the path will also dictate the change in wind direction during the course of the event. For fatigue sensitive elements, the duration of the event may be more crucial than the peak wind.

In estimating or postulating a “design” wind load sequence for a structure, the probability of variations of cyclone parameters, such as duration, intensity and radius, for a population of
buildings is required. Jancauskas, et al. (1994) and Xu (1995a) developed wind load sequences for roof cladding systems with assumptions based on historical (past) cyclone parameters and occurrence. However, Trewin (2006) notes that due to improvements in satellite resolution, radar coverage, reporting, etc, a reanalysis of the Australian tropical cyclones data base has revealed a significant change to tropical cyclone mean occurrence and intensity, to that previously thought.

2.5.2 Wind loading as defined by the Building Code and Australian Standard

The Australian Building Codes Board (ABCB) sets the legislative framework for the performance of buildings, in the Building Code of Australia (BCA 2007), with the objectives of safeguarding people from injury arising from structural failures, loss of amenity and protecting property. The BCA’s (2007) structural performance requirements specify that a building or structure, to the degree necessary, must resist the wind actions to which it may reasonably be subjected and:

- Remain stable and not collapse,
- Prevent progressive collapse,
- Minimise local damage and loss of amenity, and
- Avoid causing damage to other properties.

In the Building Code of Australia (BCA 2007), the Australian Building Codes Board defines the societal risk for the ultimate limit state strength of a structure. The level of societal risk is evaluated depending on the location and type of structure as shown in Table 2.2. For example, a hospital has a higher level of importance (Level 3) that an isolated farm shed (Level 1). From Table 2.2, the design level for housing (Importance level 2 as noted in the Guide to the BCA 2007) is to be a minimum annual probability of exceedence of 1:500. The Wind loads for housing standard (Standards Australia 2006) derives its wind loads for housing based on housing being at Level 2 importance.

<table>
<thead>
<tr>
<th>Level</th>
<th>Building Type</th>
<th>Annual Probability of Exceedence</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Buildings or structures representing a low degree of hazard to life and other property in the case of failure.</td>
<td>1:200</td>
</tr>
<tr>
<td>2</td>
<td>Buildings or structures not included in Importance levels 1, 3 and 4.</td>
<td>1:500</td>
</tr>
<tr>
<td>3</td>
<td>Buildings or structures that are designed to contain a large number of people.</td>
<td>1:1000</td>
</tr>
<tr>
<td>4</td>
<td>Buildings or structures that are essential to post disaster recovery or associated with hazardous facilities.</td>
<td>1:2000</td>
</tr>
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</table>
Accordingly, a house is required to withstand its ultimate limit state design wind speeds thereby protecting its occupants. For cyclonic region C, shown in Figure 2.13, as defined in the wind loading standard AS/NZS 1170.2:2002 (Standards Australia 2002b), the regional 10 m height, 3 second gust wind speed ($V_g$) for a 500 year mean recurrence interval is 69 m/s, a mid-range Category 4 cyclone. This wind speed has a nominal probability of exceedance of about 10% in 50 years.

AS/NZS 1170.0:2002 provides designers with load combinations including wind actions to be applied on structural components and checked against their design strength. Failure occurs when the combined load exceeds the component’s strength. Structures designed according to AS/NZS 1170.0:2002 should have a negligible probability of failure (i.e., < 0.001 or as a percentage, < 0.1%) at ultimate limit state loads, that is, failures of structural elements would not be expected to occur at the ultimate limit state design load. Nevertheless, some component damage is expected at wind speeds close to the design loads.

![Figure 2.13: Wind Regions of Australia (Standards Australia 2002b)](image)

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2.5.3 Wind loads on low rise buildings

The wind field within a cyclone is a highly turbulent environment. The fluctuating winds, together with the building aerodynamics, generate spatially and temporally varying pressures on the building surfaces. The design of a typical low rise building uses the peak gust wind speed in determining the large positive and negative pressures on its surfaces. The characteristics of the loading from fluctuating winds on structures was detailed by Davenport (1961) and forms the foundation for analysis of wind loads on structures and components. The wind duration and temporally varying forces are important in assessing elements of the building envelope and frame, such as roofing, battens and connections that may suffer degradation from low cycle fatigue.

Both the magnitude and distribution of wind loads on a roof depend on many parameters, such as roof slope, configuration (hip, gable), terrain roughness and shielding, as well as wind velocity, direction and duration. The effects of these variables lead to different peak loads with varying number and distribution of load cycles (Xu 1995a).

Flow separation regions near corners of the building (edge discontinuities) are subjected to high suction pressures, with a reduction in peak suction pressure magnitude with increasing distance from the edge. Surry et al. (1999) note that for these corner regions the loads can be double those within only a few hundred millimetres and change within a fraction of a second. Kopp et al. (2010) show in Figure 2.14 the steep pressure gradient on the roof via measured pressure coefficients on a wind tunnel model of a generic low-rise, building with a nearly flat roof at the moment of the worst local peak pressure coefficient on the roof.

![Figure 2.14: Pressure coefficients for a low rise building (Kopp et al. 2010)](image)
However, wind loading standard’s design external pressures are simplifications of these real fluctuating pressures and are typically represented by uniform pressures within zones as shown in Figure 2.15.

![Figure 2.15: Representation of wind forces on a house from a design standard](image)

If there is a breach in the building envelope on a windward face, the interior of the house is pressurised as shown in Figure 2.16. Storm damage studies have shown large external pressures combined with large internal pressures acting in the same direction to be the main cause of roof and wall failures. The internal pressure is dependent on the external pressure distribution and the position and sizes of openings in the building envelope. A dominant opening on the windward wall can generate large positive internal pressures resulting in large net uplift pressures on the roof, especially near the windward edges.

![Figure 2.16: Design wind forces with a dominant opening in windward wall](image)
Ginger (2001) analysed the pressure data from the Texas Tech, full-scale test building shown in Figure 2.17 for two Areas, A and B, representing, respectively, the tributary areas of a cladding fastener and a batten-rafter connection. Figure 2.18 shows the variation of internal, external and net pressure, over time for Area, A. These are presented as pressure coefficients \( C_p \) referenced to the mean dynamic pressure at roof height and are defined as:

\[
C_p(t) = \frac{p(t)}{U^2} - \frac{1}{2} \rho U^2
\]

where, \( U \) is the mean reference wind speed, \( p \) is pressure, \( t \) is time and \( \rho \) is density of air.

From this data, Ginger and Henderson (2003) determined that the wind load standard AS/NZS1170.2, prescribed satisfactory peak design pressures for batten-truss connection tributary areas (Area B), but can greatly underestimate peak design pressures on cladding fasteners near the windward edges (Area A). Cochran and Cermak (1992) showed that the peak pressure coefficients at the edge of the corner of the roof from the full scale TTU data were almost double of that determined from wind tunnel studies using the typical 1/100 and 1/50 scale models. Cheung et al. (1997) determined however, that in employing a larger scale 1/10 model in an appropriately sized large wind tunnel, which allowed a closer match of Reynolds number, resulted in the model achieving peaks of 80% of the full scale values for the highest loaded corner tap.

Section 2.4 highlighted the magnitude of the peak loads as a major factor in determining the resilience of pierced fixed claddings. Surry et al. (1999) note that not only are the large magnitudes of the peaks not represented in current test procedures but also the implications of the high gradients in the spatial and temporal fluctuations of the pressures for different types of structural systems are not well understood.

2.5.4 Load cycles from wind

Wind loads on roof components comprise a broad range of frequencies (Ginger 2001). In Figure 2.18, it is not obvious how many load cycles are shown, let alone the cycle amplitude. Of the many counting methods suggested, the modified “rainflow” method is generally regarded as the most realistic for determining the range and magnitude of cycles required for predicting fatigue damage (Amzallag et al. 1994; Kumar 2000; Xu 1993). This counting method identifies complete hysteresis cycles of load. The loading characteristics of the cycles are presented in terms of means and ranges, from which the maximum, minimum and \( R \) values can be derived.
Table 2.3 gives the number of loading cycles contained within specified mean and range of Cps on Areas A and B, obtained by applying the rainflow count method to the net pressures collected over 15 mins (Henderson and Ginger 2005). The total number of net pressure cycles on Area A of 7811 is similar to the total number of net pressure cycles on Area B of 7730. The magnitude of the net pressure cycles for Area B are approximately 75% the cycles for Area A. Table 2.3 shows that Areas A and B experiences loads of similar characteristics, with about 95% of the cycles having a peak to peak range less than 10% of the peak net suction pressures.

The load ratios $R$ for the majority of the cycles are $R > 0$, although standard test methods for evaluating cladding and their fixings apply varying cyclic load blocks with $R = 0$. This implies that the standard test methods may be conservative in determining fatigue performance of building
envelope systems. However, the load cycles with $R > 0$ continue to contribute to the accumulation of fatigue damage.

Table 2.3: Distribution of net pressure cycles (Henderson and Ginger 2005)

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<th>0.84</th>
<th>1.67</th>
<th>2.51</th>
<th>3.34</th>
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<th>5.85</th>
<th>6.69</th>
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<th>4.45</th>
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One limitation of the rainflow method when applied to the analysis of cladding fatigue is the randomness of the magnitude of the load fluctuations with respect to time is lost. The occurrence of larger peak load cycles before, after, or during lower load cycles does have a significant effect on the fatigue performance of cladding (Boughton 1988; Mahendran 1990; Xu 1993). Cook (1990) notes it would be unsatisfactory to start with the low level cycles and work up to the large because, in nature, the load cycles will be mixed up randomly and fatigue damage initiated by early cycles of high load is distressed further by later cycles of low load.
2.5.5 Australian wind fatigue test criteria

Load cycle distributions were developed to assess the suitability of building elements subjected to prolonged fluctuating wind loads (i.e., to assess the likelihood of fatigue failure). For Australian cladding systems, fatigue failure of the cladding is taken to be when the cladding pulls over one fastener, because, once it is pulled over one fastener, the cladding can easily be pulled over adjacent fasteners as a result of overloading.

In most cases, the cyclic load distributions attempt to mimic the number and magnitude of the load fluctuations. Various wind induced fatigue load test criteria are represented in Figure 2.19. However, the randomness of the magnitude of the load fluctuations with respect to time is mostly lost.

For the tropical cyclone regions of Australia, the fatigue load testing procedures include just one ultimate limit state event, at the design wind speed for the structure. Xu (1995a) noted that, for temperate regions dominated by large scale weather systems, wind induced fatigue damage to metal roofs accumulates from storm to storm and from year to year. He suggested incorporating the fatigue damage from smaller intensity cyclones as the structure, based on probability of occurrence, would be subjected to a few of these events in its lifetime, thus adding to the load cycle history of the cladding, that is, accumulation of fatigue damage. Kumar and Stathopoulos (1998) similarly state that the roof cladding is exposed to a spectrum of wind speeds during its lifetime with the corresponding accumulation of fatigue damage.

Cyclone Tracy inflicted major damage to the infrastructure of Darwin in 1974. For the rebuilding of Darwin, the Darwin Reconstruction Committee stipulated a test method of 10000 cycles from zero to the permissible stress design load (suction) followed by a proof load of 1.8 x design load, for evaluating roofing. Morgan and Beck (1977) noted that the conservative DABM test regime was to be used as an interim acceptance criteria and commented that this should be replaced with a more realistic test, when additional data becomes available.

Research, reported at a workshop organised by the Experimental Building Station, by Melbourne (1977) and Morgan and Beck (1979), suggested loading regimes for roof systems based on pressure measurements from wind tunnel models and simulated tests on cladding. These recommendations were simplified to produce “Guidelines for the testing and evaluation of products for cyclone prone areas”, commonly referred to as TR440 (1978).
Figure 2.19: Cyclic load regimes (Henderson et al. 2001)
Various cycle counting techniques, such as “upcrossing” (Ho et al. 1995) and “rainflow” (Amzallag et al. 1994) methods have been used to determine the number of loading cycles during the passage of a cyclone. Positive pressures pushing the cladding onto its supports have generally been neglected as the net uplift pressures pulling the cladding against the fastener heads cause much higher stresses in the cladding (Mahendran 1990). Beck (1979) stated his regime was more realistic than TR440 as the load cycles about an offset mean were incorporated as opposed to the simplified zero to load approach of TR440. For corrugated cladding, TR440 was demonstrated to be un-conservative (Jancauskas et al. 1994), but conservative for trapezoidal cladding (Xu and Teng 1994).

The Random Block Loading (RBL) method suggested by Mahendran (1993) is based on combining external pressure coefficients measured on a 1/50 scale wind tunnel model with approach winds (i.e., speed and direction) determined from a Category 4 “design” cyclone using the rainflow method. As the wind speed varied throughout the analysis, it was assumed that the rainflow method cycle counts based on the constant speed wind tunnel study increase linearly with increasing wind speed. However, for the flat roofed building modeled by Baskaran and Chen (1998), the cycle counts increased quicker than the common linear assumption.

An extensive matrix of load cycles was formulated, where each cell of the matrix contains the number of load cycles relating to a percentage of the range and mean of the ultimate wind load. The rainflow method produced in the order of 75000 pressure cycles, however only about 8000 result in significant fatigue damage. Loading blocks are then randomly selected from the matrix over the duration of the cyclone. As noted in Section 2.5.3, typical wind tunnel model studies have not measured the large peak suction pressures recorded from full scale studies. Therefore, cycle counts and test pressures based on such wind model studies may not accurately represent the loads on a cladding fastener during the passage of a cyclone. Also, they did not incorporate internal pressures.

The RBL method was considered too complex for routine product evaluation and standardised testing and was simplified into a Low-High-Low (LHL) regime representing the passage of a cyclone with the increasing then decreasing wind speeds (Mahendran 1995).

Following a review of cyclic test regimes (Henderson et al. 2001) and an exploratory test programme, a modified LHL was incorporated into the Building Code of Australia (BCA 2006). In the formulation of the LHL assumptions such as cycle counts, load range, cyclone duration, wind direction change, building orientation and building geometry have been made (Henderson 2006).
2.6 Physical testing methods

A range of configurations and apparatus are used to carry out tests for evaluating metal cladding systems. They range from the individual fastener pull through test, double span systems loaded in the middle of the spans via a line load or double span systems subjected to a uniform load using airbags, to using air chambers that apply fluctuating air pressure to the cladding. Multispan roof cladding systems are mostly evaluated by testing double span arrangements. As high suction develops close to roof edges (eaves, ridges), the central batten represents the critical, that is, the most heavily loaded, support back from the eaves or ridge (Xu 1993).

Variability arising from installation of test setup (angle and tightness of screw) and of the material properties has yet to be satisfactorily quantified. The majority of the research test programs have involved midspan line load test rigs with the cladding screws at vertical, all uniformly tightened, and fixing one brand of cladding (same roll formed dimensions) to timber battens.

Rotation of loaded flanges of C and Z purlins can produce a different fatigue response to claddings fixed to the more rigid timber battens. The rotation at the head of screw can cause a load concentration towards one half of the fastener head or washer and can also result in fatigue/bending failure of the screw.

2.6.1 Single fastener pull through

The pull through test is perhaps the simplest method for examining the strength of the cladding adjacent to the fixing as it uses a cladding piece that only extends 100 to 200 mm from the fastener. Typically, the tests are conducted in a tensile testing machine with the small section of cladding restrained and the screw pulled through the cladding, simulating the cladding being sucked over the head of the screw. Maximum load or load versus displacement curves are obtained from the tests. With the small specimen size, setup time is minimised, so repeat tests can be performed with relative ease and speed.

The pull through test is akin to a withdrawal test that is used to determine the holding capacity of a screw into a piece of timber or steel. For a withdrawal test on a screw, the interaction and strength of the screw (including thread) and material of the substrate (e.g., batten) adjacent to the screw thread is the primary focus. However, for a cladding pull through test, the response, that is, deformation, of the cladding well away from the screw may play a role in influencing the interaction at the cladding and screw head.
Mahendran (1990) and Xu and Teng (1994) note that the failure of the roof sheeting is mostly dominated by the magnitude of the fastener reaction. This suggests that the pull through tests may be appropriate for evaluating the performance of the cladding. Mahaarachchi and Mahendran (2004) and Xu and Teng (1994) used finite element analysis (FEA) studies of rib/pan profiles and showed that the local plastic buckling under and around the fastener was influenced by the rib wall buckling under longitudinal membrane compressive stresses (i.e., loads and deformation over the cladding span). This casts doubt on the suitability of using simple screw pull through tests to determine the local failure strength of cladding, as the pull through test does not incorporate the cladding span, just a small section around the screw.

Nevertheless, Mahendran and Mahaarachchi (2004) conducted small scale tests investigating splitting behaviour of rib-pan cladding from over tightening of screws. From conducting tests using multi-span to small scale pull through tests, they showed that the failure loads (transverse splitting of rib under screw head) were approximately 1.5 kN per fastener and was approximately 5% greater than the static failure load from uniform loading. They noted that the small scale test specimens only needed to be 200 mm long (as longitudinal deformation only extended 100 mm either side of screw hole), but that the half pan either side of the rib needed to be restrained to simulate sheet width continuity.

For the pull through tests, a factor is required to convert the reaction at the screw to a pressure acting over the tributary area of the cladding. The factor is based on assumptions of cladding stiffness along the profile (i.e., run of cladding along the ribs or crests) and across the profile at laps etc. From airbox tests, under higher pressures, membrane action of the cladding as well as yielding and buckling of the cladding adjacent to the screw also changes the relationship between pressure acting on the cladding and the reaction at the screw.

### 2.6.2 Line load testing

Extensive research programs (Mahendran 1994; Xu 1997) have demonstrated that the interaction of the cladding and fixing is a crucial part of the cladding’s fatigue response to the applied loading. However, the testing programs were conducted using equipment, which was best practice at that time (15 to 25 years ago), that simulated the wind loads using a sinusoidal load patterns and applied the loads to the cladding as line loads opposed to air pressure.

The test programs used a then state of the art load control, double span test rig, shown in Figure 2.20, that simulated the pressure acting on the cladding by applying a line load across the profile in
each span. Simple beam theory was used to equate the line loaded double span with a uniformly loaded double span, on the basis that the main region of concern was the interaction of the cladding and fastener at the centre support. Therefore, by assuming the cladding acts as a stiff continuous double span beam, the central support reactions and moments for the two loading-span cases are equated. Thus, Mahendran (1993) and Xu (1995b) approximated a uniformly loaded double 900 mm span cladding to their test configurations of a double 650 mm test setup with a single load in each span. Simple beam theory relies on the assumptions of small deflections with plane sections remaining plane and no transmission of loads axially.

![Figure 2.20: Line load test rig (Mahendran 1993) for loading of a cladding sample](image)

Reaction at b: \[ R_b = 1.375 \times P \]

Moment at b: \[ M_b = -\frac{6}{32}P \times L_1 \]

Thus: \[ L_1 = 0.733 \times L_2 \]
The midspan load method is more reasonable than the pull through test method, as it attempts to generate the correct moment and reaction combination on the cladding, at the central support by using an equivalent span for the cladding test specimen. However, the line load can constrain the cladding profile.

Xu (1992) describes the effect on the failure load of up to 20% for changing the line load contact area in the ribs of trapezoidal cladding. He notes that a smaller influence would be expected for corrugated (arc-tangent) cladding due to its smooth profile shape. However, load applied via a line load cannot transmit load equally along its length to a distorted profile. It therefore concentrates load at the fixed crests, that deflect less, thereby reducing the load in the unfixed crests potentially reducing the lateral membrane loading at the cladding/screw connection.

In conducting line load tests with different cladding spans (equivalent to 900 and 1200 mm uniform loading), Xu (1995b) showed that for corrugated cladding a reduction in the ultimate static capacity of the screw/cladding was in the order of only 5%. However, for the rib/pan cladding, the reduction in the longer span was 15%, thus highlighting the deficiencies of using simple beam theory when non-linear membrane action is in play.

2.6.3 Air-bag

Although simulating uniform loading, the commonly used air bag loading method also has its drawbacks when testing profiled cladding. As the air bag does not make complete contact with the profiled (i.e., corrugated, ribbed) cladding surface, local effects are not accurately replicated (Parsons 1976). Seccombe et al. (1996) notes the air-bag may induce additional forces in the cladding at re-entrant corners as the air-bag ‘spans’ across the rib openings.

The majority of commercial testing during the 1980s up until the late 1990s for the development of load span tables was conducted using air-bag test rigs.

Cladding manufacturers generate product information (load span tables for multiple spans) by assuming the reaction at the fastener is the critical parameter and use simple elastic beam theory with the assumption of constant material cross section to extrapolate test results from double span to multiple spans.
2.6.4 Air-box (free air)

The Cyclone Testing Station (CTS) air-box is a 2 m wide 11 m long open topped channel. The test specimen forms the top (“lid”) of the air-box. Air pressure is applied to the inside face of the cladding. This loading simulates the combined outward pressure (suction) acting on the external face and an internal pressure pushing on the internal face of the roof. A computer controls the cycle rate typically set at 0.7 Hz.

Henderson et al. (2001) observed an occasional failure mode which was associated with cracking of the top sheet of cladding at the lap, as shown in Figure 2.21. In an air bag test rig, the air bag pushes against the cladding tightly closing the laps. However, this is not the case in the air-box. It also appears the transverse deflection of the cladding between the fixings along the battens is greater for the end pan/unscrewed crest adjacent to the lap, than for the pan at the centre of the sheet. This deflection leads to a rotation of the top lap sheet giving a possible uneven loading under the fastener.

Figure 2.21: Failure of cladding at lap during testing at CTS

2.6.5 Simulated wind load

As shown in Figure 2.18, the wind pressures acting on a building’s surface can be highly fluctuating with little resemblance to constant amplitude sinusoidal load cycles. Several systems have been developed or are under development to replicate the highly dynamic pressure characteristics of the wind loading and apply it to building elements.

2.6.5.1 BRERWULF

The BRE real time wind uniform load follower (BRERWULF) test chamber is approximately 5 m x 5 m which is mounted to the external face of cladding and capable of applying 8.5 kPa (positive
and negative). It can be used to reproduce a target pressure trace or conduct block loading tests (Cook et al. 1988). The frequency response of the rig varied with the size of the test chamber and characteristics of the test specimen. The cladding and air chamber needed to be sealed.

2.6.5.2 MSU

A novel loading system capable of applying spatially and temporally varying loads on standing seam metal roof cladding was developed at Mississippi State University (Sinno et al. 2003). The test rig, shown in Figure 2.22, uses large capacity electromagnets to apply an array of quickly varying uplift loads on top of a uniform positive pressure applied from an air-box underneath the cladding. As described by Surry et al. (2007), the development of the electromagnetic test rig was an extraordinary challenge which took 10 years to develop, with problems such as: developing strong enough but small enough magnets, writing control software to account for the feedback on one magnet from the varying fields of each neighbour, the large scale displacement of the cladding in the non-linear magnetic field, etc. The magnets are approximately 300 mm in diameter and additional metal, in the form of washers, is placed under the metal roof cladding, as well as on top of the plastic sheet used to seal the top of the air-box, to improve the capacity of the magnets. Ali and Senseny (2003) calculated a substantial reduction in the standing seams frequency response with the addition of the extra metal.

![Figure 2.22: MSU metal cladding test rig (Surry et al. 2007)](image)

2.6.5.3 PLA

Advanced real time pressure loading systems capable of high flow have been developed for the “Three Little Pigs” (3LP) full scale house testing facility at the University of Western Ontario (UWO) (Kopp et al. 2010). The pressure load actuators (PLA) permit the application of actual temporally varying wind pressures to a representative test section of the building envelope.
The 3LP research facility at UWO is capable of subjecting full-scale houses and other light-frame buildings to extreme wind events. A system of 100 pressure load actuators are able to be applied to the building’s surface, temporally and spatially varying pressures, that reproduce pressure time histories obtained from scale models in the wind tunnel.

The two main components for each PLA unit are a regenerative blower capable of delivering flows with high pressures at a large volume, and a highly reactive four port valve. The PLA unit was adapted from the BRERWULF system, except that the PLA’s design specifications required it to apply rapidly changing pressures to a surface that may have varying amounts of leakage. In this way the system can apply pressures to porous cladding, such as brick or siding, as well as adapt to changing leakage due to joints etc opening during loading. Previous systems, such as BRERWULF and the MSU chamber needed to be sealed.

The PLAs four port valve can be rapidly cycled between delivering positive pressure to the air box by connecting it to the exhaust port of the blower and delivering negative pressure to the air box by connecting it to the intake port on the blower. A shaped disc sits between the air-box side of the valve and the blower side of the valve. The varying shape of the holes in the disc allows the computer, via a series of lookup functions, to set the rate of flow (e.g., open the “tap” more to generate more flow). A high torque servo motor is used to rapidly position the disc.

The lookup function curves are relatively linear at pressures away from zero, thus allowing for fine control. However, there is a flattening of the response curve near zero pressures that means the valve finds it difficult to hold a constant pressure near zero.

Each PLA unit is attached to an air-box. The PLAs and air-boxes are supported by a large reaction frame that surrounds the test structure. The open end of the air-box is attached to the test specimen by a flexible skirt. The PLAs (and associated air-boxes) are distributed across the test house with higher concentrations in areas where the greater magnitude fluctuating pressures are required (i.e., at the windward corners). To achieve the realistic application of temporally and spatially varying wind pressures applied to the full scale structure, each PLA unit is supplied with an appropriately scaled pressure trace for its location. The complete system can therefore apply a uniform pressure from each PLA, or every PLA can supply a different, but repeatable, dynamic pressure trace representing the pressure acting on the buildings surface for a given wind direction.

The main drawback for this entire approach is the need for the air-box skirt to be attached to the surface of the building. For flexible claddings, such as long span metal deck roofing or plastic wall cladding, the added stiffness from the skirt would affect the cladding’s performance. For
nominally rigid linings, such as plywood, brick and composite panels, the added stiffness is minimal.

2.7 Numerical methods

2.7.1 Modelling of cladding

Finite element analysis (FEA) of Australian profiled pierced fixed cladding has been undertaken by several researchers, starting with Mahendran (1990) and Xu and Teng (1994). They conducted nonlinear FEA of cladding subjected to static loading which showed the analysis could be used to examine overall deformations and the stresses and strains in the vicinity of the fastener holes. Mahaarachchi and Mahendran (2004) note that, although these earlier studies show that nonlinear FEA can be used to conduct parametric studies of cladding (stresses, deformation), they were not able to predict the load for cladding pulling over the fastener (splitting of cladding) because the elements were assumed to have perfect elasto-plastic material properties with infinite ductility, due to the limitations of the various software packages at that time.

Mahaarachchi (2003) conducted detailed FEA, using ABAQUS (Hibbitt et al. 2000), of rib-pan crest fixed G550 cladding subjected to static uniform load. The various mesh models incorporated elements for the screw head’s flange and neoprene washer (Figure 2.23). The earlier FEA work did not include this representation of the flexible washer under the screw head.

The model constrains the screw head from any movement as the tested screws were stiff along their axis (0.01mm for 2 kN) (Mahaarachchi and Mahendran 2004). However, as noted in Section 2.6, from observations during air-box cladding tests at the CTS there is some lateral movement of the screw due to unequal membrane loading from cladding and rotation of the purlin flange. It is unknown what, if any, affect this has on cracking. Although, it is assumed to have minimal affect on static strength, it may influence crack initiation under cyclic load.

In advancing on previous studies, Mahaarachchi (2003) conducted model runs for both a half width sheet (two and a half ribs) and a single rib configuration as shown in Figure 2.23. Previous studies considered a single rib (half rib and half pan). The comparison showed that modelling the half width of cladding sheet, incorporating the unrestrained sheeting edge, as opposed to just one rib, was more accurate and compared favourably to his experimental results. As discussed in Section 2.6 cracking of cladding at the unrestrained rib (top sheet) adjacent to the fasteners has been observed during cyclic load tests in the CTS air-box.
Mahaarachchi and Mahendran (2004) modelled the initial geometric imperfections of the rolled cladding. They note that although there was no significant effect on the ultimate load in the nonlinear analysis, the model was able to converge much faster due to the post-buckling deformations proceeding correctly. The claddings residual stresses (from the roll forming process) were also incorporated into the model based on the work of Schafer and Pekoz (1998). However, the authors concluded that the effect on the ultimate load was minimal (0.08%). Nevertheless, with regard to cyclic loading, residual stresses may increase the risk of failure through an increased crack propagation rate (Smith 1991). Mahaarachchi and Mahendran (2004) did not model any variations in tightness of the screw fixing.

![FEA model of rib-pan cladding](image)

2.7.2 Modelling of cladding with load cycles

The failure load of pierced fixed cladding subjected to a cyclic load is less than its static failure load (i.e., strength). To enable parametric studies of all the variables associated with the resilience of metal cladding subjected to wind loads, a validated FEA of metal cladding subjected to cyclic loads is required.
ABAQUS (Hibbitt et al. 2000) fracture mechanics analysis techniques are based on the J-integral for crack initiation and CTOD for crack propagation. However, when considering the anisotropic out of plane plastic deformation of cladding the manual lists several limitations including: J-integral appropriate for linear material response but limited for nonlinear materials, stress intensity factors derived from LEFM, and crack propagation modelled only along a predetermined path. The package does incorporate anisotropic elements but does not allow them to be used in the cyclic load routines.

2.8 Summary

Following the devastation caused by Cyclone Tracy, extensive research into wind loading of metal roofing has been conducted in Australia by researchers such as Beck (1975), Mahendran (1989) and Xu (1995b). Their findings showed that wind loads on roof cladding and its response leading to low cycle fatigue cracking was significantly influenced by the peak load, load range and the sequence of loading. For the simulation of wind loads, the experimental investigations made use of the best cyclic load equipment at that time with the application of line loads to reduced span cladding specimens. Xu (1995a) noted at the time that the line load method constrained the profile and affected its performance.

Miner’s rule and fatigue theories, such as the Linear elastic fatigue model, were found to be unsuitable for prediction of fatigue failure due to the large deformation and yielding taking place in the cladding adjacent to its fixings. Cycle counts of varying means and ranges were derived from the fluctuating pressure using the rainflow analysis.

The current cyclic load test regime called the L-H-L was developed in the late 1980s and early 1990s and was based on the line load experiments and a five hour “design” cyclone with means and ranges from pressure cycles from one single storey house model. Full scale pressure measurements at TTU indicate wind loading standards may underestimate peak pressures at windward edges. These peak pressures act over small regions and short durations. Pierced fixed cladding response to the spatially and temporally fluctuating loads has yet to be determined.

With free air test apparatus, such as the PLA, now available, an assessment of the cladding response to fluctuating loads is possible. Also, the cyclic results determined from the line load test methods, on which the current Australian test criteria is based, can be examined using a uniformly distributed air pressure.
3. Experimental configurations and procedures

A range of physical tests were carried out on cladding specimens to investigate the performance of the 0.42 mm base metal thickness corrugated cladding when subjected to wind loads. The tests ranged from tensile tests on small test coupons to rapidly fluctuating pressures simulating severe cyclonic wind loads applied to a section of roofing. The static load tests were conducted to investigate the deformation and buckling characteristics of the cladding along with the load transfer to the cladding fasteners. The fatigue performance of the cladding and fixings were examined with the PLA during the cyclic load trials and compared to previous line load test data. Simulated cyclonic wind loads were used to compare the performance of the cladding system tested under existing test methods to its performance when subjected to the fluctuating pressures. This Chapter describes the types of specimens tested, apparatus used, the physical testing methods used in the application of loads, and the monitoring of specimen response to the loads applied.

3.1 Tensile tests

Tests were conducted to determine the material properties of cladding specimens used in this study. Properties determined were yield strength and ultimate tensile strength, elongation, and Young’s Modulus. Tensile tests were conducted as described in the Australian Standard AS 1391 (2005) “Metallic materials: Tensile testing at ambient temperature”. Testing of the samples was conducted at two locations, the Bluescope Steel quality control laboratory and at the James Cook University materials laboratory (Singh 2008).

3.2 Pressure loading and measuring response of cladding systems

Recently, commercial cyclic (fatigue) load tests have been conducted using an air-box. These tests show some variation in cladding response compared to airbag with the lateral movement of the screw head (bending of screw) and additional failure modes of the cladding in the air-box (Henderson et al. 2001). As discussed in Section 2.6.3, the airbag (and line load) test method may provide additional constraint on the cladding’s loaded (deformed) shape.

In order to analyse the response of roof cladding to realistic wind loads, the experimental setup must:

- Apply air pressure directly to the cladding surface (no line loads or airbags),
- Have an air pressure delivery and control system capable of static and cyclic loading,
- Use typical spans from standard buildings,
• Span the roofing over realistic supports,
• Have at least a full cladding sheet width, to allow lateral movement,
• Provide a very repeatable method for installation of test specimens (cladding and screws),
• Be able to measure the applied pressure and the load at a fastener, and
• Be designed to allow the visual monitoring of crack formation and growth.

A test system with these capabilities was developed following a series of configurations, trials and refinements.

Tests were conducted using a pressure applied to corrugated cladding specimens installed on to an air-chamber. Various loading regimes, such as uniform ramp, step, cyclic and simulated wind were used to study the reaction at the fastener and the condition of the cladding.

The test system consisted of:
• A pressure loading actuator (PLA) and computer control systems to supply and control the applied air pressure,
• An air-chamber for mounting of the test cladding sheets and containing the pressure,
• Measurement systems including pressure transducers, load cells, and displacement transducers and strain gauges for selected tests.
• A digital camera mounted above a test specimen.

3.2.1 Loading system (PLA at JCU)

An integral component of the test configuration is the blower and valve that supplies a controlled pressure to the air-chamber. Typically, the supply of pressure to a cladding sample has had a blower on the “inlet” side of a test chamber and a valve on the “outlet” side. The air-box of the Cyclone Testing Station is of this configuration.

The Pressure Loading Actuator (PLA), described in Section 2.6.5.3, has the air-chamber forming part of the loading circuit. The PLA is able to produce positive and negative pressures inside the test chamber.

Figure 3.1 shows the major elements of the PLA, excluding the computers and the control cabinet which housed the variable speed drive, servo controller, breakers, etc.
Various trials investigating maximum delivered pressure and controlled loading rates were conducted using the PLA to determine an appropriate (optimum) air-box size. Factors influencing the performance included the test chamber volume, leakage and the ability to install cladding test samples consistently. The trials showed that the PLA is capable of delivering +/- 25 kPa for a 900 × 900 × 100 mm chamber through to +/- 8 kPa for a 2000 × 1500 × 150 mm chamber. The larger chamber had a reduced performance (i.e., peak pressure and response) due to its increased volume and leakage at cladding laps and perimeter.

The smaller chamber, with a 900 mm long test sample that only had a central row of screws was able to produce higher pressures with excellent fidelity for an applied dynamic pressure trace. Even though such a setup would be convenient in allowing easy replacement of test samples, the boundary restraint between the profiled cladding ends and edges of the box provided too much restraint with the cladding receiving support through membrane action, thus adversely influencing the test results.

A dedicated computer system is used to control the tests incorporating the PLA. The PLA chases a “requested” (target) pressure within the test chamber via computer control. A one second portion of a requested pressure trace and the PLA achieved pressure trace is shown in Figure 3.2, highlighting the resolution of the pressure fluctuations to less than 0.1 of a second.
Figure 3.2: Plot of PLA achieved response against the requested target real time pressure trace

The performance of the system (or its ability to meet the target pressures) is dependent on the target trace (magnitude and rates of change), volume of chamber and leakage. If the PLA is unable to follow the target trace within user defined error limits the system shuts down.

3.2.1.1 Pressure traces

Several types of target pressure loading histories, such as wind load traces, sinusoidal, sawtooth wave ramp or step traces, were generated for the various test programs. The type of trace used depended on the type of test. The ramp and step traces were typically used for static load tests for measuring displacements and strains of the cladding for a given pressure. The cyclic load traces (e.g., sinusoidal) were used for the fatigue loading tests, whilst the wind traces, as described in Section 6.5, were used for investigating the cladding specimen’s response to cyclonic wind loading.

As noted in Section 3.2.1, the target trace may differ, within set bounds, from the actual “achieved” pressure trace. The achieved trace is the recorded time series of pressures that were applied to the test specimen. Any analysis of data (e.g., comparison of reactions versus applied pressure and S-N curves) was carried out using the achieved trace and not the target trace. The setup of the air-chamber, installation of the test specimen and setting the parameters of the control software to achieve a trace comparable to the target trace was from a partially “trial and error” approach by determining leakage rates for different pressures, load rates and configurations.
3.2.2 Air-chamber

The majority of tests were conducted in the air-chamber with a box size of 2000 mm long x 890 mm wide x 300 mm deep. The rig was able to accommodate up to a double 900 mm cladding span for a full width of corrugated cladding and produce a dynamic pressures in excess of +/- 12 kPa.

The air-chamber, shown in Figure 3.3, was fabricated from 3 mm steel plate and was designed to be sufficiently rigid under peak loads, but easy to fabricate and sufficiently lightweight to be moved around the lab space. Pairs of 75 mm x 12 mm plate located at the middle and both ends were tapped at 75 mm centres to allow multiple mounting locations for instrumentation and purlins or battens. The depth of the box was minimized in order to reduce the chamber’s air volume to enable the generation of the required pressure traces. The maximum purlin height that can be accommodated is 100 mm.

![Air-chamber in Configuration A with Z-purlin prior to installation of cladding](image)

Figure 3.3: Air-chamber in Configuration A with Z-purlin prior to installation of cladding

The air duct from the PLA was divided into two so that the air flows into either end of the air-chamber. A standard automotive air-filter was used at each entry to reduce the possibility of...
fouling by foreign matter, such as swarf from screws and being sucked into the valve. Pressure transducers were located in each bay of the chamber to monitor the pressure distribution and confirm that each span was subjected to the same pressure.

3.2.2.1 Cladding installation

The cladding forms the “lid” of the air-chamber. Therefore the PLA supplied pressure acts on the inside surface, that is, the cladding surface facing the air-chamber. The ability of the PLA to provide positive or negative pressures to the air-chamber presented an opportunity to develop a unique test system, that would replicate the standardized form of testing of cladding in an air-box and an alternative configuration that would allow an unobstructed view of the cladding deformation and cracking adjacent to the screw shank.

Two test configurations were designed for the air-chamber (test chamber). Configuration A, detailed in Figure 3.3, was designed such that the cladding (test specimens) was installed with its exterior surface facing upwards, that is a normal installation for a roof with screws driven through the roof into battens. Conversely, Configuration B had the exterior surface of the cladding facing into the air-chamber, thereby allowing a suction pressure to act on the cladding and allowing the ‘underside’ of the cladding (now facing up) to be observed, as shown in Figure 3.4.

![Figure 3.4: Air-chamber in Configuration B mode showing the load cells and screw stubs](image)

Cladding edge strip of top lap already installed into air-chamber

Mounts for screw stubs

JR3 load cell

Figure 3.4: Air-chamber in Configuration B mode showing the load cells and screw stubs
In either test setup a full cladding sheet was used. In addition to the full sheet width (860 mm), two edge strips were used to simulate the laps on either side of the sheet as sketched in Figure 3.5. Both strips were approximately 100 mm wide and were cut from the corresponding edges of another sheet. The edge strips were reused over multiple tests.

![Figure 3.5: Sketch of cladding with edge strips and screw numbering](image)

### 3.2.2.2 Configuration A (Positive pressure mode)

In Configuration A, shown in Figure 3.3, the cladding specimen is installed in a manner similar to that on a roof, that is, the roofing is screw fixed with a screw-gun (drill) into a batten or purlin. The test configuration is similar to that in a standard commercial air-box test such that the test pressure is applied as a positive pressure to the interior surface of the cladding, allowing a direct comparison with typical tests in commercial air-boxes.

For all Configuration A tests, the end supports were Z10010 purlins. The purlins were bolted to threaded 12 mm plate brackets with typical 12 mm purlin bolts. The brackets were bolted to the 75 mm threaded plates at the bottom of chamber. The middle support for the majority of the tests was 90 mm x 45 mm softwood purlin with a strength grade of MGP 12 or greater. Some of the tests used a Z10010 purlin as per the end supports. The timber purlins used 12 mm threaded rod with flanged nuts while the Z purlins used 12 mm purlin bolts for fixing to the purlin brackets.

To reduce the leakage of air along the long edges of the air-box, a thin strip of flexible plastic sheet was laid and taped along either side of the air-box prior to the installation of the cladding. The plastic extended approximately 200 mm into the air-chamber. The strip was not taped or connected to the cladding but basically held against the cladding due to the applied air pressure during the run. The plastic strip (or skirt) is a typical method used to reduce leakage in the large CTS air-box.

In order to improve PLA performance the internal air volume of the air-chamber was reduced by filling in voids behind the end purlins and along the long edges with polystyrene blocks. The blocks were approximately 20 mm below the level of the purlins.
When installing the cladding, the bottom lap edge strip was placed first followed by the full width cladding sheet then the top lap edge strip. The centre row of screws was installed using an electric drill mounted on a modified drill press. The drill press was used to minimise variability in alignment, angle and depth of the cladding screws. An epoxy resin foot moulded to a corrugated profile was attached to the base of the drill press to ensure the screws were installed perpendicularly to the cladding. A depth stop on the press plunge kept the screw penetration consistent. The moulded foot and press were constructed to allow, if required in further research into effects of variability of screw installation, adjustments to the mounting such that screws could be installed up to 8 mm off centre and 10 degrees off from perpendicular.

The direction of cladding lay allowed two tests for each purlin as the screw spacings are not symmetric about the central axis of the air-chamber, that is, the top sheet lap screw is approximately only 25 mm from edge of sheet, while the bottom sheet lap screw is 70 mm from its edge. Direction of lay is defined from which side the first cladding sheet (edge strip) is placed. After two tests the purlins/battens were replaced.

3.2.2.3 Configuration B (Suction mode)

For a typical roof on a building or in a standard test installation, like Configuration A, the screw head and seal conceal the cladding directly adjacent to the screw shank on its exterior face while the batten obscures any view near the screw location on the interior face. Configuration B was able to circumvent this limitation as it allowed observations of the cladding immediately adjacent to the screw shank, albeit on the cladding’s interior face.

For Configuration B, the cladding was installed with its exterior surface facing the inside of the air-chamber. The roofing was not screw fixed to any batten but held in place by a row of screw stubs, as shown in Figure 3.4. The middle row of screws was welded to rods in threaded brackets bolted to the 75 mm threaded plates running across the bottom of the air-box. The screws were the same 14 gauge diameter but with the thread and shank guard ground smooth so that the thread did not “catch on” the cladding as it was lowered over the welded screws. This is an acceptable assumption because the cladding on normal roofs is not engaged with the screw thread as the thread does not extend all the way along the shaft to the screw head. Each screw was carefully aligned onto the rod by having a tap point milled into the centre of the screw head with a mating stud machined in the end of the rod.

The rods in brackets were positioned using a plywood template that had the correct spacing as per the cladding crest spacing. The crests were not exactly at 76 mm centres leading to one of the
screw spacing’s being at 154 mm. This discrepancy is within allowable tolerances for the roll forming of corrugated cladding. The same template spacing was used to pre-drill the cladding prior to installation.

The installation of the cladding specimen in the air-chamber was carried out by initially placing the cladding in a template such that the centre row of screws could be drilled through the cladding at exact centres to align with the crests. The screws were drilled through the cladding and then removed. The cladding sheet, with its external face facing down, was then able to be placed over the row of inverted screws welded to brackets bolted to the centre mounting plates in the air-box, as shown in Figure 3.4. The first cladding sheet (edge strip) was placed prior to the full sheet being laid.

Rows of fixings at the end of the cladding were not used. The cladding was restrained at each end by being clamped between a corrugated profiled plywood “knife-edge support” and a corrugated profiled foam strip. The plywood end clamps were 100 mm in from each end giving a cladding span of 900 mm. During cyclic loading of the cladding, it was observed that the cladding tails (overhang) were able to move vertically upwards as the cladding midspan was being drawn inwards showing that the end restraints were correctly replicating pin ends and not a rigid fixed end restraint.

In a similar method to Configuration A, thin flexible plastic film was placed along both long sides of the air-box and taped to the box and cladding to reduce leakage. Clear plastic was chosen to allow continued observation of the cladding.

The advantages of Configuration B (i.e., inverted) installation were:

- “Screws” located in the same position on the crest for each specimen tested,
- Allowed the camera to be perpendicularly above the interior face of the cladding adjacent to the screw shaft (no batten in the way), and
- Monitoring of screw gun torque and screw tightness not needed.

The disadvantages of Configuration B installation were:

- The self weight (although only 0.04 kPa) acted in the same direction as the applied suction load,
- The “screw stubs” were rigid (no lateral movement) in comparison to a standard installation,
- Only a suction load could be applied, as there was no batten to push the cladding onto, and
• The JR3 three axis load cell face plate was subjected to a pressure imbalance (as detailed in Section 3.2.3.3)

3.2.3 Measurements during testing
Measurements of applied pressures or forces and resultant reactions, displacements and strains were made with a range of load cells, pressure transducers, displacement transducers and strain gauges connected at various times to data acquisition systems contained within standard desktop computers.

3.2.3.1 Data acquisition system (DAQ)
National Instruments (NI) analogue to digital signal converters were the primary electronic data recording system mounted in a PCI slot in a standard personal computer. The NI PCI-6220 cards are 16 single ended channels, 16 bit cards that were used in the non-referenced single ended mode (NRSE). One of the cards was in the master control computer for the PLA. The other card was in an independent desktop, referred to as DAQ-2.

There was no common clock signal or counter shared between the Master PLA computer and DAQ-2. When both DAQs (computers) were being used, the output files were synchronized by aligning the first of the pressure or load peaks from both sets of time series data from each computer.

Data sample and storage rate was typically 100 Hz. The storage rate was reduced to 50 Hz if the test run was expected to be over an hour. There was no pre-storage filtering of the data acquisition signals. All device power supplies were on the same ground. All the connections were shielded. NI voltage amplification modules were required to boost the output voltage from the S-type load cell and the strain gauges.

The NI software language, “Labview”, was used to develop the programs for the data collection.

3.2.3.2 Pressure
As discussed in Section 3.2.1, a pressure transducer was located in each bay of the air-chamber. One of the transducers, a Sensortech 35 kPa unit, was connected to the PLA control system as an integral part of the load control. The second transducer, either a Sensortech 35 kPa or, for some low pressure tests, a Sensortech 7 kPa model was connected to the DAQ.
Calibration of the transducers was first carried out using a manometer, and then calibrated against NATA certified pressure transducers and readouts.

The Sensortech 35 kPa units had a rated capacity of ±35 kPa, while the smaller transducer had a rating of ±7 kPa. The manufacturer’s specifications for the three transducers had a non-linearity and hysteresis of ±0.2 % full scale with a response time of 1.0 ms for responding to a pressure change of 10 to 90 % of full scale. The manufacturer’s possible “error” range of 0.4% for a full scale of 70 kPa gives a maximum possible error of 0.28 kPa. However, when calibrating against the NATA system, both 35 kPa transducers performed well within stated limitations, with a maximum recorded discrepancy less than 0.1 kPa at pressures below 1 kPa.

### 3.2.3.3 Reaction at a screw

The reaction at the cladding fastener is a critical parameter in assessing the performance of the cladding system under wind loading. The JR3 load cell was used to determine the dynamic reactions at a fastener. The air-chamber was designed to include the mounting of load cells and, specifically, a JR3, three axis load cell. For Configuration A, the mounts were above the air-chamber, while for Configuration B the mounts were bolted to the strengthened floor of the air-chamber.

The reaction at selected cladding screws was measured by either the three axis JR3 load cell or a standard shear beam S-type load cell. Screw stubs similar to those used in Configuration B were connected to each load cell. The load cell with screw stub was positioned at the cladding screw hole. For Configuration A, the screw stubs were cut to 15 mm so that they would not sit on the batten below the cladding. Figure 3.6 shows both load cell types with the screw stub prior to installation of the cladding for Configuration B.

The JR3 load cell is a proprietary cased, donut shaped, instrumented aluminium block that can measure loads in the X, Y and Z axes. The axis convention is shown in Figure 3.7. Foil strain gauges are used to sense the loads imposed on the load cell. A 6 x 6 calibration matrix is used to resolve the six output voltage channels into reactions in the X, Y and Z directions and the moments about these axes. The output of the unit is given by six analogue voltage signals.
The calibration matrix supplied by the manufacturer was checked by conducting separate calibrations (voltage vs applied load) of the three load axis and three moments, and then resolving the subsequent equations. Periodic spot checks of the JR3 were conducted by applying loads to the Z and usually Y directions, and whenever the air-chamber configuration was changed. Calibration
checks and spot checks indicated that there was some cross talk (or coupling) of approximately 1% and 3% of the Z load contributing to the load in the X and Y directions, respectively. This cross talk was evident when the Z load was relatively larger than the other loads. Gilbertson et al. (1999) noted that the JR3 load cells had a deviation of load in the three axes at lower load levels. The levels of cross talk for this test program were near the stated 1% full scale accuracy of the load cell.

The JR3 unit is very sensitive to installation of the connecting face plates. Care is needed to ensure a uniform torque is applied to all the face plate bolts and this reinforces the need to conduct spot checks of the load cell output whenever the unit is repositioned, or following several cyclic load tests.

For Configuration B, with the JR3 installed inside the air-box, its top surface face plate is subjected to the applied negative pressures. The pressure acting over the loadcell’s face plate acts in the opposite direction to the measured loads on the fastener and, therefore, needs to be added to obtain the force at the screw in the Z direction. A standard S-type load cell was installed at an adjacent screw location as a check on the Z direction load of the JR3. Also, the self weight of the cladding sample needs to be factored into the final load although, for the majority of the cases, it is small in comparison to the applied load.

3.2.3.4 Strain

Single element strain gauges with lead wire attached were used during some of the static load tests. The strain gauges had a gauge length of 2 mm and were glued according to manufacturer’s instructions using the supplied cyanoacrylic adhesive. The cladding Colorbond coating was not removed for the strain gauge tests. Each strain gauge was connected in a quarter bridge pattern to a NI signal amplifier. The strains were converted to stress at the extreme fibres (surface of the cladding) using a Young’s modulus of 210000 MPa, and a manufacturer’s specified gauge factor of 2.11.

3.2.3.5 Displacement

Displacement measurements during the uniform load tests were made using magnetic coil displacement transducers. The Temposonic transducers have a stated non-linearity of +/-0.02% over full scale. The transducers are connected to the DAQ and calibrated against NATA certified gauge blocks.
The transducers require a 24V DC power source and output 4 to 20 mA. Several power supplies and wiring configurations were trialled but signal noise at approximately 50 Hz could not be eliminated. Prior to using the deflection data, the time series was filtered to smooth the results. As the deflection data was only used during static and not cyclic or dynamic tests, the slight loss of time resolution was deemed acceptable.

3.2.3.6 Crack length and screw movement

For the first half of the test program, initiation of cracking of cladding near the fastener and subsequent crack growth was monitored by visual inspection. The crack growth was tracked with pencil marks on the cladding against a time stamp from the test clock running on the Master PLA computer. The tests were not paused for these marks to be made.

A Prosilica GigE digital black and white camera with C-mount lens was used to improve data quality. The camera communicated via an Ethernet cable with DAQ-2. The Labview code measuring the load and pressure transducers was modified to incorporate the Ethernet camera images, such that each captured frame included a timestamp incorporated into its filename. Length scales marked on the cladding or the diameter of the screw heads were used to scale the number of pixels for determining crack length and translational movement of screws during loading.

The camera was positioned directly above the head of a screw for Configuration A and conversely, above the screw shank (stub) for Configuration B. A camera traverse slide was fabricated to allow level and stable travel across the air-chamber’s width. The original intention was for the camera to be moved above each screw location during the test. However, due to a combination of the camera having a narrow depth of field from the wide aperture and being manually focused, the camera was setup over one screw for a whole test. The camera could then be setup over another screw in a subsequent test.

Two light sources were used to highlight the cracks against the cream coloured cladding surface. A bright light above the test was required for illumination and focus with a second light positioned at an oblique angle to cast some shadow for highlighting the cladding deformation and cracking. The lights were battery powered to minimise additional signal noise.
3.3  **Point load tests (influence line test method)**

Point load tests were conducted on roof cladding samples to investigate the uplift reactions at a cladding screw, at an internal support batten. The loads were generated by a series of discrete point loads pushing on various locations on the underside of the cladding specimen.

The cladding specimens were screw fixed to a custom built support frame consisting of four C150-12 cold formed purlins, spaced at 900 mm centres, and bolted through the web with standard cleat plates to an RHS frame, as shown in Figure 3.8. The first, third and fourth purlins spanned 2.4 m between cleats, whilst the second support (first internal support) was in two sections with a 160 mm gap. A STC load cell was placed in this gap and mounted to the RHS frame and was used to measure the uplift load at that cladding screw location, as shown in Figure 3.9(a). A specially machined bolt head with washer to accept an EPDM seal was fabricated for the load cell. The bolt with washer was placed through the pre-drilled hole in the cladding sheet and threaded into the load cell.

![Figure 3.8: Point load test setup](image)
A standard screw thread scissor car-jack was used to apply the point load. A STC load cell with an 80 mm diameter foam platen shaped to a corrugated profile was mounted on top of the jack, as shown in Figure 3.9(b).

![Figure 3.9: Load cell details for measurement of (a) screw reaction at the first internal support and (b) applied point load at mid-span of the first cladding span](image)

The point load was applied at several locations over the first two spans of the three span cladding specimen, as shown in Figure 3.10. Both Corrugated and Monoclad profiles were tested. Displacement transducers were used to measure cladding deflections with displacement transducers also positioned on the heads of the screws in order to remove any displacement of the purlins from the total movement of the transducers in the cladding spans. Linear interpolation was used to remove the movement of the purlins from the total measured deflection of the cladding, relative to ground. The cladding screws were installed using the modified drill press.

The point load tests were repeated for each load point location, shown in Figure 3.10, with the measurements recorded using a Tokyo Sokai TDS-602 data logger. Loads were applied such that plastic deformation was not observed in the cladding at the screw locations.
3.4 Summary

Experimental methods were used to determine material properties and system response of cladding specimens screw fixed to typical supports. The experimental methods ranged from the standard tensile tests of small coupons through to use of the advanced PLA for the simulation of cyclonic wind loads on representative cladding areas.
The ability of the PLA, to deliver both negative and positive pressures combined with the novel design of the air-chamber in allowing the cladding test specimens to be installed with the top surface facing up or facing down, allowed inspections of the cladding adjacent to the fasteners. A three axis load cell was used to analyse the relationship between the reactions at a screw and the changing damage state of the cladding.
4. Response of cladding to static and cyclic loading

The high tensile steel roof cladding is a strong material, with a base metal thickness of less than half a millimetre. When rolled with a profiled shape to add stiffness, this cladding, with a profile depth of less than 20 mm spanning over battens spaced 900 mm apart, can capably hold a sizeable person’s weight striding across a roof or withstand suction wind pressures trying to pull the cladding over the tiny heads of its fixings, albeit with a great deal of creasing and perhaps cracking. Wind-induced low cycle fatigue greatly reduces the static capacity of the pierced fixed cladding with severe consequences, as detailed, for example, in the report on Tropical Cyclone Tracy (Walker 1975).

Prior to the introduction of fatigue loading tests, cladding was evaluated by applying a static uniform proof load. The test was to ensure the cladding did not tear at the fixings or that the fixings did not pull out of their supports (e.g., battens or purlins). Static load for the testing of cladding refers to a steadily increasing load applied to the test specimen until either a proof load limit or failure is reached, as opposed to a load held constant for a long period of time to examine for effects such as, creep or sag, for, say, materials such as timber or concrete.

This Chapter presents the performance of a corrugated cladding system when subjected to static and cyclic uniform pressures. It delineates the formation and development of the two major fatigue induced crack types as well as exploring the lateral movement and loads applied to the screws from the response and deformation of the cladding to the applied pressures. The Chapter compares the results of the uniformly applied static and cyclic air pressure from this program with that of the earlier testing using the reduced span line load method and, in so doing, comments on its suitability in the development of Australian test standards.

4.1 Static uniform pressure loading

Static loading trials were conducted on 0.42 mm BMT G550 corrugated cladding installed into the air chamber using Configuration A setup (i.e., the cladding’s external face is on top). Static load trials were conducted with the applied loads starting in the elastic range and progressing through to permanent plastic deformation of the cladding.
4.1.1 Loading in elastic range

Deflections were measured at locations shown in Figure 4.1. Strain gauges were located adjacent to a screw (S3) fixing the cladding to the central purlin and at the crest of an adjacent unscrewed crest, as shown in Figure 4.2.

![Diagram showing location of screws and displacement transducers](image)

**Figure 4.1: Location of screws S1 to S6 and displacement transducers d1 to d5 for Configuration A**

![Diagram showing positions of strain gauges](image)

**Figure 4.2: Positions of strain gauges sg1 to sg6 located at screw S3 (strain gauges on top surface)**

Figure 4.3 shows the static load applied as a steadily ramped step trace with the pressure increasing to a peak of 2 kPa. The unscrewed crest at mid-span, d3, deflected more than the screwed crests. The response of all deflection gauges increased linearly with load. The deflection at d4 (unscrewed crest above central support) was similar in magnitude to the mid-span deflections of the screwed crests and 1 mm less than d3. With d3 rising 3 mm (~20% of profile) the cross sectional shape of the profile becomes distorted as the screwed crests are restrained by the screws.
Strain gauge sg6 was mounted longitudinally on the top surface in the centre of the unscrewed crest and measured compression with the application of air pressure, as shown in Figure 4.4. The compressive strains measured at the top of the crest above the support would be expected since the cladding is acting like a continuous beam spanning over the central support. That is, the bending moment induced stresses would be compressive at the extreme fibres of the top surface of the profile, at the location of contraflexure.
The two longitudinal strain gauges (sg1 and sg3) and the two transverse gauges (sg2 and sg4) reported tensile strains for the external surface of the cladding adjacent to the screw. The global compressive stresses from the continuous beam, measured by sg6 for the unscrewed crest, were overcome by the localised stresses in the thin cladding induced by the cladding being restrained by the screw.

### 4.1.2 Loading in plastic range

Subsequent trials repeated the stepped trace with peak pressures for each trial increasing to 3, 4 and 5 kPa. The stresses and deflections for the 4 kPa trial are shown in Figure 4.5 and Figure 4.6, respectively.

In Figure 4.5, when increasing the pressure step from 3 to 4 kPa, the stresses adjacent to the screw (sg1 to sg4) flip from tension to compression. The area of plastic deformation in the cladding under the head of the screw has extended beyond the strain gauges, that is, in an exaggerated sense; the strain gauges are on the “inside” face of a dish with the screw head sitting in the bottom of the dish. With the plastic deformation of the cladding, the strains at sg2 and sg4 have exceeded the gauges’ limits such that the derived values for stress are no longer realistic. However, these values still give an indication of the change in stress state and local deformation of the cladding.

![Figure 4.5: Change in stresses for pressures in steps up to 4 kPa](Note that the values for stress are not realistic beyond 110 seconds)
Figure 4.6: Displacement of cladding for stepped pressure trace up to 4 kPa

Figure 4.7 shows the “dished” shape deformation of the cladding surrounding the screw at the applied pressure of 4 kPa. Figure 4.7 also shows the vertical movement of the unscrewed crest relative to the screwed crest. The movement of the unscrewed crest at the central support was larger than that of the unscrewed crest at mid-span. The flattening of the screwed crests at the central support was “pushing” the unscrewed crest higher. The stresses at the top face of the unscrewed crest were still compressive.

Figure 4.7: Plastic deformation of cladding at top of the crest adjacent to the screw for applied pressure of 4 kPa

For the 5 kPa peak trial, at approximately 4.5 kPa, the fixed crests completely flattened with permanent creasing and an audible pop. The spikes in the deflection measurements, as shown in Figure 4.8, aligned with this occurrence. A consequence of the flattening (i.e., creasing across the crest with the valleys raising to be at a similar level to the screw head) was the loss of stiffness at those corrugations. Also, the unscrewed crests at the central support were pushed upwards such that they exceeded the corresponding mid-span deflections by approximately 7 mm.
The permanent creasing and crest movement is shown in Figure 4.9. Even with this increased cross-sectional shape, the compressive stresses at sg6 reduced with the flattening of the screwed crests. The cladding was mostly supporting the applied pressure through membrane action, and not as a continuous beam. The mid-span deflections of the screwed crests were all approximately 20 mm, which was 4 mm less than the deflection of the unscrewed crest at mid-span and 11 mm less than the unscrewed crest above the central support.

During the unloading phase, at approximately 2 kPa, the cladding at the fixings “popped” back due to the increasing compressive stresses from the cladding as it was returning to its unloaded state (i.e., the “stretched” mid-span and unscrewed crests were trying to come back to their unloaded state).

Figure 4.8: Large distortional changes in cladding for stepped pressure trace up to 5 kPa
If the wind pressure acted as a uniform (equal) load across the cladding surface and the cladding was a stiff continuous beam (i.e., plane sections remain plane) spanning over battens, the reaction at the screw would be given by $F_U = RC \times P \times A$: where $RC$ is the reaction coefficient, $P$ is the uniform pressure and $A$ is the tributary area of the fastener (i.e., cladding span $\times$ fastener spacing). $RC$ can be derived using the influence coefficient approach detailed in Section 5.1. For a double span beam (i.e., a continuous beam spanning over an internal support in the middle of its length) $RC$ equals 1.25.

A stepped static loading trace, Figure 4.10, shows the applied pressure, $Z$ reaction, and non-dimensional reaction coefficient ($RC$). The spike in the $Z$ reaction (at 48 seconds) coincided with the start of the fourth pressure step. The cladding adjacent to the fastener creased and buckled with an audible “pop”. The spike in the load was approximately a 10% overshoot. Figure 4.11, presents the $X$, $Y$, and $Z$ reactions measured by the JR3. The creasing of the screwed crests can also be detected as a spike in the $X$ direction and to a lesser extent in the $Y$ direction. The lateral load ($X$ reaction) on the “screw” is approximately 15% of the $Z$ reaction. During the fourth and fifth load steps the cladding continued to deform with the unscrewed crests deflecting more (above) than the screwed crests.

From the static load trials, the formation of the creasing (buckling) of the cladding adjacent to the screws typically occurred at loads between 600 to 650 N.
The reaction coefficient is related to the stiffness and deformation of the cladding. The reaction coefficient commenced at a nominal 1.2, reduced to 1.05 for the fourth pressure step and increased for the final pressure step. The reduction in stiffness of the profile across the central support...
reduces the reaction coefficient, that is, the reaction at the screws for the applied pressure is proportionally less than prior to the cladding deforming.

This closely resembles the results from line load testing reported by Xu (1992), shown in Figure 4.12, where the reaction coefficient drops to that of a hinged support but then increases through membrane action of the cladding with increasing load.

![Figure 4.12: Applied line load versus fastener reaction (Xu 1992)](image)

4.1.3 Summary of static loading

During a steadily increasing, uniformly applied pressure, light gauge steel pierced fixed cladding undergoes a transformation resulting in changes to its response characteristics. Initially, there is a linear response of the cladding, although the unfixed crest deflects more than the restrained screwed crest. Deformation (yielding) of the cladding under the screw head results in an increase in deflection for little increase in pressure. In comparing line load testing (Xu 1992) with the air-chamber, the plastic deformation occurred at a deflection of approximately 10 mm for the middle of the span at a pressure of approximately 4.0 kPa for both cases. The similarity in pressures is based on the line load test rig’s specified results, converted to give an equivalent uniform pressure as described in Section 2.6.2.
With the cladding buckled, the rate of deflection decreases, thereby indicating the stiffening of the cladding through membrane action. In examining the vertical deflections of the crests in the longitudinal direction (i.e., along the length of the cladding span), Xu (1992) also noted the non-linear behaviour with the increase in deflection for little increase in pressure during the initial plastic deformation of the cladding adjacent to the screws at the centre support fixings.

Both the air-chamber and line load test results compare favourably with respect to the cladding response in regard to deflections with increasing applied load and load for onset of plastic deformation. As Xu (1995b) noted that the static behaviour, that is, deformations, of the cladding will have a bearing on the cladding system’s fatigue performance.

4.2 Cyclic loading

Cyclic load trials, with variations in pressure range, peak, shape and frequency, were carried out to study the response of the cladding specimens. The response was measured in terms of numbers of cycles to failure for varying parameters, crack development and crack patterns, and relationship between applied pressure, cladding deformation and measured reaction at the fasteners. Appendix A contains details of the cyclic load trials with pressure, numbers of cycles and photographs of the final crack patterns.

4.2.1 Cycles to failure – Overall results

Figure 4.13 shows the number of cycles to failure versus the peak cyclic pressures \( P_{\text{max}} \) applied for a range of load ratios \( (R) \) as defined in Figure 2.6. Cyclic pressures were applied to cladding test specimens until failure with the cladding pulling over one of the screws. The specimens were tested in the air-chamber and loaded in either Configuration A or Configuration B modes. The applied cyclic pressures were typically sinusoidal at a frequency of 1.5 Hz. Some specimens were subjected to lower frequencies and different shaped loading traces, such as sawtooth or spiked, as is discussed in Section 4.2.2.

The cyclic load trials with a larger \( R \) sustained more load cycles than trials with a similar peak load but smaller \( R \), as shown in Figure 4.13. Section 2.4 details similar results from the line load test rig. The increased life, that is, the larger number of cycles to failure, for trials with larger \( R \) is due to the reduced flexure per cycle and the smaller change in stress state at the crack tip, as discussed in Section 2.2.4 and shown in Figure 2.12.
4.2.2 RMS of pressure for different shaped load cycles

The root mean square (RMS) of a pressure signal \( P_{\text{RMS}} \) can be determined using Equation 4.1 where \( P_i \) are uniformly spaced pressure measurements. The RMS can be used to compare the fluctuating pressure energy of different signals. Four traces, each with a \( P_{\text{max}} \) of -5.0 and an R of 0.2 are shown in Figure 4.14. The pressure RMS for the sinusoidal, spike, inverted spike and sawtooth traces was 3.3, 2.2, 4.3 and 3.3 kPa, respectively. The inverted spike trace with its longer duration of pressure near \( P_{\text{max}} \) has a larger RMS value than the sinusoidal trace.

\[
P_{\text{RMS}} = \left( \frac{P_1^2 + P_2^2 + \ldots + P_n^2}{n} \right)^{\frac{1}{2}}
\]

Figure 4.13: \( P \) versus \( N \) for cyclic loading with varying R values
Figure 4.14: Cyclic pressure traces showing the profile shapes used

Figure 4.15 shows the $P_{\text{RMS}}$ versus the number of cycles to failure while Figure 4.16 shows $P_{\text{max}}$ with number of cycles to failure. The arrows shown in Figure 4.15 indicate the separation in test results for $P_{\text{RMS}}$, where there is no separation for the results with similar $R$ when plotted using $P_{\text{max}}$, as shown in Figure 4.16. The inverted spike trace with its longer duration peak load had a similar number of cycles to failure when compared to the other traces with similar peaks and $R$. This helps demonstrate that the low cycle fatigue is dependent on the peak and amplitude as opposed to the duration of peaks. Therefore traces with similar $P_{\text{max}}$ and $R$ ratios have similar numbers of cycles to failure regardless of the shape of the four traces trialled within the variation of the test program as discussed in Section 4.4.
Figure 4.15: RMS of pressure traces versus numbers of cycles to failure

Figure 4.16: $P_{\text{max}}$ versus $N$ for different trace types with different $R$ ratios

(The blue circles denote the trials with arrows in Figure 4.15, and indicates how they align with trials of similar $R$, $P_{\text{max}}$ and $N$)
4.3 Crack patterns

Different crack patterns, for example: H and Star type, were observed during the test program. Figure 4.17(a) and (b) show examples of these types of crack patterns. The crack patterns have been divided into two categories: cracks initiating at crease points and cracks initiating at a screw hole and are detailed in Sections 4.3.2, and 4.3.3, respectively. Similar observations described in Section 2.4 were recorded by Mahendran (1990) and Xu (1992) using line load test loading systems.

![Figure 4.17: Example of (a) ‘H’ type and (b) ‘Star’ type crack patterns](image)

The $P_{\text{max}}-N$ data can be divided into regions showing the relationship between the load ratio, the maximum load per cycle and crack pattern. The crack morphology for the test specimens is shown in Figure 4.18.

![Figure 4.18: Number of cycles to crack initiation and to failure with crack type](image)
4.3.1 Crack initiation

The cycle count at which crack initiation ($n_{ci}$) was observed were recorded for the majority of trials. Crack initiation was defined when a crack greater than 1.0 mm was detected. However for air-chamber Configuration A tests, cracks originating under the screw head needed to be in the order of 3 mm long to be observed. Figure 4.18 shows the number of cycles for initiation as well as the number of cycles for failure for the test specimens. The values for initiation and for failure may not be for the same screw in the specimen being tested. They represent the threshold numbers of cycles for initiation and for failure of any one of the four “fully loaded” screws in each test specimen. Therefore, the plot of numbers of cycles for crack initiation gives an upper bound for number of load cycles for no observed cracking.

A crack is initiated earlier in the cycles for ‘H’ and ‘T’ type cracks, with the cracks forming at the crease points away from the screw head. The load per cycle is greater for these cracks than the star cracks that form under the screw head. In addition, significantly longer cracks are required for failure to occur from ‘H’ and ‘T’ type cracks, with corresponding quicker growth rates.

The nominal ratio of cycles to crack initiation to cycles to failure ($n_{ci}/N_i$) for the crease initiated cracks was 0.05. The cracks that initiated under the screw heads had an average ratio of 0.09 for the tests using Configuration B setup.

4.3.2 ‘H’ crack group

Crease points formed at the edges of the deformed crests for loads greater than the local plastic deformation (LPD) strength of the corrugated cladding. During cyclic loading, cracks initiated at these crease points. Crack growth continued at these points forming an ‘X’ shape with cracks growing longitudinally and transversely.

4.3.2.1 Formation and growth

Figure 4.19 (a) to (f) show:
(a) Formation of the crease points (at 33 cycles),
(b) Initial cracking at the crease points (at 50 cycles),
(c) Crack growth in the shape of an ‘X’ (at 140 cycles),
(d) Crack growth with one side towards screw and the other longitudinally (at 680 cycles),
(e) Crack growth from screw hole (at 1000 cycles), and
(f) Failure (adjacent screw at 1040).

The images were taken from a Configuration B test setup that is looking at the “underside” of the cladding at the screw shaft.
Figure 4.19: Development of H type crack

(a) crease  (b) crack initiation

(c) load and unload  (d) load and unload

(e) crack growth from screw hole  (f) failure
It can be observed from the load and unload pairs of images that the closing and opening of the different ‘arms’ of the ‘X’ shaped cracks coincide with the movement of the buckle line.

4.3.2.2 Movement at screw

Figure 4.20 shows the movement with respect to time of the screw shaft near the screw hole for the cracking presented in Figure 4.19. The images were taken in batches resulting in the blocks of data with periods of no data in between. For example, data blocks (images) were at 300, 450 and 650 seconds, with the test cycle rate at 1 Hz. Figure 4.21 shows the X and Y displacement of the screw about the origin.

The vertical lines in Figure 4.20 represent the movement occurring between the load and unload peaks of the load cycles with the magnitude increasing with numbers of cycles due to crack growth and cladding deformation. The large jump in magnitude of the movement near the completion of the test was due to the adjacent screw failing resulting in a larger eccentric tributary area being supported by the screw and the screw deflecting in the predominantly X direction, as shown in Figure 4.21.

![Figure 4.20: Movement of screw over time](image)
4.3.2.3 Reaction at screw for ‘H’ crack development

The ‘H’ cracks develop at the crease points formed in the buckling of the cladding crests adjacent to the screws. The buckling of the crests applied a lateral load to the screws. The X, Y and Z reactions at a screw at the start of a test are shown in Figure 4.22. The spike in reaction in the X direction was from the formation of the buckle and crease then the reverse flexure (accompanied by a “popping” noise) as the profile tried to regain its original shape during the unloading phase of the load cycle. Typically, the magnitude of the X reaction was in the order of 100 to 200 N for the first few cycles of buckling and crack formation. The reaction in the X direction then reduced within a few cycles to less than 10% of the Z reaction.
Figure 4.23 shows the variation of $Z$ reaction with pressure during the load and unload phases. The load and unload parts of the cycle are shown by the “gap” in the plots. Therefore the rate of change in reaction, during the unloading to loading parts of the cycle, is consistently different with the loading phase giving a higher rate of change than the unload phase, that is, the cladding is stiffer in the loading phase as it measured a higher reaction for same applied pressure.

![Figure 4.23: Z reaction (uplift) at screw compared to applied pressure](image)

4.3.3 Star crack type

Star crack initiation occurred in the cladding at the screw hole under the EPDM seal, which is a donut shaped neoprene seal that fits between the underside of the screw head and the cladding as a weather seal on the screw holes. Cracks formed at lower pressures than the pressures measured for LPD during the static loading trials.

4.3.3.1 Crack formation and growth

Tests were conducted on the cladding specimens in Configurations A and B. The crack development is shown in Figure 4.24 to Figure 4.29.

Since the star cracks initiated at the screw holes, the cracks were not observed, for Configuration A setup, until they had reached a length of approximately 4 mm (that is when its length was
sufficient to just become visible from beneath the screw head’s flange to the camera positioned directly above the screw head). Cracks, which would have a total length of approximately 3 mm to 3.5 mm in length, could be observed if viewed from an oblique angle.

Figure 4.29 shows the shape of the EPDM seal, squashed between the screw flange and cracked cladding for the last stages of the test. From the cyclic load tests, the seal was observed to form into a truncated cone shape. The truncated cone shape was a function of both the dishing of the cladding under the screw and the unconfined, outside circumferential edge compressing more than the inside edge that was against the screw shank. The now “conical” shaped seal promoted further splitting of the cladding along the developed cracks.

Figure 4.24: Deformation of crest of cladding under screw head in Configuration A

Figure 4.25: Cracks in cladding protruding from under screw head
Figure 4.26: Extension of cracks and deformation of cladding with cladding bearing on flanges of screw

Figure 4.27: Star crack failure (note rub marks from screw flanges)

Figure 4.28: Star crack and dishing under screw head in Configuration B (Photo at 1800 cycles)
4.3.3.2 Movement at screw

Screw movements were significantly reduced compared with the movement of the screw during the formation and growth of the crease type ‘H’ cracks. This is because the cyclic test pressures are lower and the cladding deformation for star cracking is significantly less than that for the crease cracking. With the development of cracks, the displacement at the screw head increases, as indicated by the movement in the x-y plane, shown in Figure 4.30. The x-y movement was dependant on the asymmetry of the crack growth and the contact of the cladding on the screw flange and shaft.

Figure 4.30: Displacement of screw for two time periods
4.3.3.3 Reaction at screw for ‘Star’ crack development

The reactions in the X and Y directions during star cracking were approximately half that of the ‘H’ crack for the first few hundred cycles, since there was no buckling of the crest. Figure 4.31 shows the X, Y and Z reactions over the duration of a trial. For the star cracks, the relationship between the reactions in the X and Y directions appeared to be dependent on the development of the cracks and also on the location of the cladding to the screw hole/shaft. The screw cuts a hole slightly larger than the screw shaft (shank). The X and Y reactions reduce from the initial values with crack formation but increase with non-uniform crack growth and eccentric loading on the screw. At the completion of the test, rub marks were typically observed on one quadrant of the shank with the opposite quadrant of the screw flange having pronounced rub marks. Figure 4.31 also shows a trial where an adjacent screw failed at approximately 1322 seconds.

![Figure 4.31: X, Y and Z reactions at screw over 10 sec periods at initial, mid and final stages of the trial](image)

Figure 4.32 shows the Reaction coefficient (RC), as defined in Section 4.1.2, for the peak pressures of each cycle varying over the duration of the trial. The RC value decreased from approximately 1.25 to 1.15, implying that the cladding was losing stiffness with its increasing crack growth.
In conducting static and cyclic tests using the line load method, Xu (1995) monitored the applied load versus the total middle support reaction. Figure 4.33 shows the reduction in normalised support reaction (load at screws) during load cycling. The results were for a cyclic (assumed $R=0$) prototype pressure of 3.2 kPa and 2.9 kPa for the corrugated and trapezoidal cladding profiles.

The Figure 4.33 shows an approximate 1% reduction at 1000 cycles, falling to a 15% reduction in the normalised support reaction by 5000 cycles. Xu notes that the reduction in the normalised support reaction is due to the loss of localised stiffness starting with crack initiation and worsening with crack propagation. In assuming the “normalised” support reaction had a Reaction coefficient of 1.25, this then leads to a RC of 1.15 at the end of the test on the corrugated cladding. No comments were made on the coefficient during the unload phases.
Figure 4.33: Total peak reaction force at central support (Xu 1995b)

Figure 4.34 shows the RC versus applied cyclic pressure for complete cycles, that is, not just at the peak but over the duration of the test. The reduction in the RC for the peak pressures can be observed. There is a far greater change in the RC for the unloaded phase over the duration of the test. It is associated with the flattening of the crest, loss of preload in the EPDM seal and crack growth. However, the large variation of the RC is at lower reactions hence having little influence as a proportion of peak reactions.

Figure 4.34: Reaction coefficient variation with pressure and time
Xu (1995b) notes that cyclic load tests based on load control at the screw reaction may not give the same results as those tests controlled by mid-span load.

In commercial practice the derivation of load span tables for cladding is based on the assumption of simple beam theory for extending double span test results to multi-span cases. The use of 1.25 for a reaction coefficient as opposed to approximately 1.1 leads to conservative outcomes for cladding span tables and screw pullout tables.

4.4 Variation in load cycles to failure

Two batches of four sinusoidal cyclic loading trials were conducted to compare the variations in numbers of cycles to failure with that of the line load test described by Mahendran (1989). Table 4.1 lists the applied sinusoidal peak pressures, $R$ and cycles to failure with crack type for the eight trials.

<table>
<thead>
<tr>
<th>Trial number</th>
<th>$P_{max}$ (kPa)</th>
<th>Config</th>
<th>$R$</th>
<th>Cycles $N_i$</th>
<th>Crack type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>C12</td>
<td>-4.8</td>
<td>A</td>
<td>0.4</td>
<td>1491</td>
<td>H</td>
<td>failed at screw S4</td>
</tr>
<tr>
<td>C13</td>
<td>-4.8</td>
<td>A</td>
<td>0.4</td>
<td>1845</td>
<td>H</td>
<td>failed at S4</td>
</tr>
<tr>
<td>C14</td>
<td>-4.8</td>
<td>A</td>
<td>0.4</td>
<td>2182</td>
<td>H</td>
<td>failed at S3</td>
</tr>
<tr>
<td>C15</td>
<td>-4.8</td>
<td>A</td>
<td>0.4</td>
<td>1890</td>
<td>H</td>
<td>failed at S4</td>
</tr>
<tr>
<td>C49</td>
<td>-3.5</td>
<td>A</td>
<td>0</td>
<td>1810</td>
<td>star</td>
<td>failed at S2 then S3</td>
</tr>
<tr>
<td>C50</td>
<td>-3.5</td>
<td>A</td>
<td>0</td>
<td>1980</td>
<td>star</td>
<td>failed at S2 then S3 and S4</td>
</tr>
<tr>
<td>C52</td>
<td>-3.5</td>
<td>A</td>
<td>0</td>
<td>1986</td>
<td>star</td>
<td>failed at S3 and S4</td>
</tr>
<tr>
<td>C54</td>
<td>-3.5</td>
<td>A</td>
<td>0</td>
<td>1478</td>
<td>star</td>
<td>failed at S4</td>
</tr>
</tbody>
</table>

The four ‘H’ crack type repeat trials had a spread of cycles to failure of nearly 700 cycles, or 40% of the mean of 1850 cycles. The spread of numbers of cycles to failure for star crack type was approximately 500 cycles, which was 30% of the mean.

The two batches represent two different crack types: the ‘H’ type crack where the crack started at crease points away from the screw and the star crack, where initiation was at the screw hole. The spread in cycle numbers to failure in both modes implies that the cutting action of the screw into the cladding was not a major contributor in the variation of cycles to failure.
Test results presented by Mahendran (1989) and discussed in Section 2.4 are given in Table 4.2. They also show a large spread of cycles to failure. The variation from these line load tests was also in the order of 40%. The load cycle range was below that of static local plastic deformation and less than the air-chamber trials and, as such, the numbers of cycles to failure were greater than the air-chamber trials.

Table 4.2: Variations in cycles to failure for line load tests (Mahendran 1989)

<table>
<thead>
<tr>
<th>Load range (N)</th>
<th>Cycles to failure ( N_i )</th>
<th>% difference from mean</th>
<th>Crack type</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-344</td>
<td>274150</td>
<td>25%</td>
<td>Star</td>
</tr>
<tr>
<td>0-350</td>
<td>62750</td>
<td>71%</td>
<td>Star</td>
</tr>
<tr>
<td>0-350</td>
<td>320450</td>
<td>46%</td>
<td>Star</td>
</tr>
<tr>
<td>0-400</td>
<td>18160</td>
<td>36%</td>
<td>Star</td>
</tr>
<tr>
<td>0-400</td>
<td>38710</td>
<td>36%</td>
<td>Star</td>
</tr>
</tbody>
</table>

With the availability of two load cells for measuring the reaction at screws, a measure of the variation of pre-load from the “installation” of the screws was undertaken for the second batch of repeat trials. The installation of the cladding was conducted in a similar manner to the other Configuration A tests, as noted in Section 3.2.2.1. The cladding screws were fixed into the purlins using the modified drill press. Screws S3 and S4 were removed in order to insert the load cell (JR3 and STC) screw stubs. The screw heads were installed such that the flanges were aligned by straight edge to adjacent screws and the EPDM seals were slightly compressed as per the manufacturer’s specifications.

The installation of the load cell screw stubs resulted in a range of pre-loads as shown in Table 4.3. The average value was approximately 65 N with a standard deviation of 46 N. The preload was, on average, about 10% of the local plastic deformation (LPD) load of the cladding. The variation in initial load on the screws for the tests appeared to have no discernable effect on the final failure. However, it would be anticipated that the variation in pre-load for cladding that is screwed onto a roof would be considerably more than that measured in the controlled laboratory test.

Table 4.3: Variation in cycles to failure for \( R=0 \) with initial load at load cell “screws”

<table>
<thead>
<tr>
<th>Trial</th>
<th>Max (kPa)</th>
<th>( R )</th>
<th>( N_i )</th>
<th>Preload at S3 (N)</th>
<th>Preload at S4 (N)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>49</td>
<td>-3.5</td>
<td>0</td>
<td>1810</td>
<td>na</td>
<td>44</td>
<td>failed at 2 then 3</td>
</tr>
<tr>
<td>50</td>
<td>-3.5</td>
<td>0</td>
<td>1980</td>
<td>14</td>
<td>18</td>
<td>failed at 2 then 3 and 4</td>
</tr>
<tr>
<td>52</td>
<td>-3.5</td>
<td>0</td>
<td>1986</td>
<td>136</td>
<td>72</td>
<td>failed at 3 and 4</td>
</tr>
<tr>
<td>54</td>
<td>-3.5</td>
<td>0</td>
<td>1478</td>
<td>110</td>
<td>47</td>
<td>failed at 4</td>
</tr>
</tbody>
</table>
Australian cladding design standards (Standards Australia 1992) give a material variability of 10% for standard G550 roofing for the determination of design loads from testing. The test results detailed here would suggest a far greater variability is actually present.

### 4.4.1 Failures of cladding at screws

Damage of the cladding adjacent to the screw holes S2 to S5 was classified based on crack extents. Table 4.4 details the percentages of damage classification based on a population of 38 trials. The table also includes a subset of 14 trials classed as star crack type failure. The crack extents were defined as:

(a) Failed: Cladding pulled over screw

(b) Extremely cracked: Any crack longer than 20 mm. For the star type crack pattern, the 20 mm was taken as two cracks aligning on either side of a screw hole plus the diameter of the screw hole.

(c) Cracked: Damage less than (a) or (b)

In examining the final patterns and damage extents, a trend of greater damage at screw S2 reducing to screw S5 was observed. The star crack failure subset had very similar percentages to the H type cracks, showing that the damage trend at the screws was not dependant on crack type. During cyclic loading, the unfastened crest between S1 and S2 deflects more than the unfastened crest between S5 and S6. With reference to Figure 3.5, screw S2 is adjacent to the top lap edge while S5 gains additional support as it is 35 mm further from its cladding edge and next to the underlap.

<table>
<thead>
<tr>
<th></th>
<th>S2</th>
<th>S3</th>
<th>S4</th>
<th>S5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failed or extremely cracked</td>
<td>92%</td>
<td>78%</td>
<td>70%</td>
<td>59%</td>
</tr>
<tr>
<td>Cracked</td>
<td>8%</td>
<td>22%</td>
<td>30%</td>
<td>41%</td>
</tr>
</tbody>
</table>

**Table 4.4: Aggregation of crack extents for cyclic loading**

<table>
<thead>
<tr>
<th></th>
<th>S2</th>
<th>S3</th>
<th>S4</th>
<th>S5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failed or extremely cracked</td>
<td>93%</td>
<td>71%</td>
<td>64%</td>
<td>64%</td>
</tr>
<tr>
<td>Cracked</td>
<td>7%</td>
<td>29%</td>
<td>36%</td>
<td>36%</td>
</tr>
</tbody>
</table>

### 4.4.2 Different coil strengths

Cladding manufactured from two G550 coils was used in this study. The majority of testing was conducted using Coil-A, with a specified longitudinal tensile strength of 780 MPa. Coil-B, with a lower strength of 744 MPa, was used for a limited round of comparative testing. The cladding
lengths rolled from both Coil-A and Coil-B were profiled on the same roll-form production line to obtain a consistent profile shape for all test specimens.

The longitudinal tensile strengths of the coils used in testing by Mahendran (1989) and Xu (1992) were 690 MPa and 720 MPa, respectively. From correspondence with Bluescope labs, they note that, with improvements in feedstock, the variation of material strengths has reduced and the mean strength has increased. Bluescope lab tests gave the mean longitudinal tensile strength value of G550 for 2008 as 755 MPa with a range between 660 and 865 MPa.

The $P_{\text{max}}$ versus the numbers of cycles to failure from the six cyclic tests using Coil-B, along with tests from Coil-A with similar cycle peaks and load ratios, are shown in Figure 4.35. The results fall within the variation of cycles to failure and lie along the trends of the corresponding $R$ values. Cracking and failure of the cladding is associated with plastic deformation and buckling. As buckling is dependent on the applied loading and stiffness, the ultimate tensile strength of the coil has less of an influence on cracking than its Young’s modulus and profile shape.

![Figure 4.35: Comparison of Coil-A and Coil-B](image)

4.4.3 Effect of contact area under screw head

Crack propagation is a function of the stress state in the vicinity of the crack tip. A donut shaped EPDM weather seal fits between the underside of the screw head and the cladding. Correct installation of the screw through the cladding and into the batten requires the seal to be slightly compressed. The seal can be severely distorted and bulging out from under the screw flange if the screw is installed incorrectly.
Three trials, each using similar load cycles, were conducted, with the EPDM seals omitted from one of the trials. The trials with the EPDM seals lasted approximately 2400 and 2800 cycles, while the trial without the washers resisted over 10000 load cycles. None of the screw locations of the screws without seals had cracks radiating towards or from the screw holes, as shown in Figure 4.36. In comparison, the trials with seals had cracks travelling towards the screw holes and, in some cases, also radiating from the screw holes, as shown in Figure 4.37.

![Images of screw heads with and without EPDM seal](image1)

(a) Top of sheet with rub marks from flange  
(b) Underside of same sheet  

**Figure 4.36: Cracking of cladding without EPDM seal under screw head**

![Images of screw heads with and without EPDM seal](image2)

(a) Top of sheet with rub marks from seal  
(b) Underside of same sheet  

**Figure 4.37: Cracking of cladding with an EPDM seal under screw head**

For the trial without the seals, the lipped edge of the screw head’s flange carried the load from the cladding to the screw, as shown by the dark circular rub mark in the cladding paint in Figure 4.36, reducing stresses at the screw hole. The seal having a smaller diameter than the screw head flange initially had a smaller contact area than the screw flange. The contact area increased for the seal with its compression and the deformation of cladding as can be seen by the rub marks on the
cladding in Figure 4.37. The uneven compression of the seal during loading and the continued cladding deformation also played a part in promoting crack growth as discussed in Section 4.3.3.1.

### 4.4.4 Screw fixed into Z-purlin

The majority of the cyclic load tests (Configuration A) used timber purlins as the central support. The Type 17 (timber cutting) cladding screws were embedded approximately 30 mm into the timber providing rigid end restraint (cantilever) for the screws.

Following the use of the camera to capture motion of the screw, a trial was run using a cold formed steel Z100-12 purlin. The applied pressure cycled from 2.3 to 5.1 kPa. The displacement of the screw head with movement per cycle from 0.5 to 1.0 mm can be seen in Figure 4.38. The initial 1 mm movement was in the Y direction which included the displacement of the top flange of the Z purlin as shown by the insert in Figure 4.38, as well as the rocking of the screw.

![Figure 4.38: Movement of screw head when fixing cladding to Z purlin](image)

An aside to the movement was also the slight but continuous rotation of the screw head, as shown in Figure 4.39, for the duration of the trial. The unscrewing (by one or two revolutions) of metal cutting screws into purlins has also been observed, by the author, during a few cyclic load cladding tests conducted in a commercial cladding test air-box facility. There was an anecdotal report of screws fixing a low rise metal clad hospital roof requiring re-tightening following Cyclone Vance, however the author was not able to confirm this as the work was completed prior to our inspection (Reardon et al. 1999). There were no signs of any rotation of any of the air-chamber trials that used Type 17 screws fixing into the timber purlins.
Trials with similar peak cycle pressures and $R$ values are given in Table 4.5. The cycles to failure were less than using the Z-purlin. Nevertheless, the variation in number of cycles to failure was within the spread of variations from other repeat tests as detailed in Section 4.4.

Table 4.5: Comparison of number of cycles to failure for Z-purlin

<table>
<thead>
<tr>
<th>Central purlin</th>
<th>Peak cycle pressure (kPa)</th>
<th>Ratio $R$</th>
<th>Number of cycles to failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber</td>
<td>-5.1</td>
<td>0.42</td>
<td>2867</td>
</tr>
<tr>
<td>Timber</td>
<td>-5.1</td>
<td>0.45</td>
<td>2864</td>
</tr>
<tr>
<td>Z-purlin</td>
<td>-5.1</td>
<td>0.45</td>
<td>3960</td>
</tr>
</tbody>
</table>

4.4.5 Cladding failing at an adjacent screw to load cell

The failure of cladding at a screw will result in the transfer of loads to adjacent screws resulting in a progressively increasing extent of failure. A few of the load trials conducted were able to continue cycling after the failure of the cladding at a screw. Typically, the cladding also then failed at an adjacent screw location within a few cycles. For a few of the trials, load at a screw was measured when an adjacent screw failed.

Figure 4.40 shows the sudden increase in load at the screw (JR3 positioned at S3) when the cladding pulled over at an adjacent screw (S2). The loss of applied pressure was due to the sudden volume change in the test rig from the failed crest moving suddenly upwards. The PLA control system was quick to compensate and continued with the sinusoidal pressure cycles.

The reaction coefficient for the loaded peaks prior to the failure at the adjacent screw was 1.12 averaged over the preceding cycles. This was based on the standard tributary area of 900 mm x 152 mm. In applying this reaction coefficient to the resultant peaks following the failure, a tributary width of 220 mm was calculated. This dimension compares favourably with the
assumption of half adjacent tributary area being applied to the screws on either side of the failure, e.g., $152 + 152/2 = 228$ mm.

Figure 4.41 shows the X, Y and Z reactions along with the calculated resultant at the screw. The X reaction increased to approximately 25% of the load in the Z direction. The large increase in load in the X reaction was due to the membrane forces in the cladding and the uneven tributary area now acting on the screw S3. For two other trials where this occurred the increase in the X reaction was 23% and 18% of the Z reaction after failure of the adjacent screw.
4.5 Comparison of air-chamber cyclic loading to line loading

Figure 4.42 compares the scaled peak cycle pressure versus number of cycles to failure for line load tests from Xu (1995b) with those from the air-chamber trials. Xu presented the $R=0$, double 650 mm span line load test results as a “prototype pressure”. Accordingly, the Figure 4.42 line load results have been scaled based on the ratio of the line load test span of 650 mm to the air-chamber test span of 900 mm. The line load results match the air-chamber $R=0$ results for the LPD region.

![Figure 4.42: Peak pressure with numbers of cycles to failure from the air-chamber trials and for Xu (1995b) line load test data](image)

The peak load per screw ($S_{\text{max}}$) versus numbers of cycles to failure is shown in Figure 4.43, for both the air-chamber trials and the line load data given by Mahendran (1989). The comparison between the two test methods for the various $R$ values has a consistent fit within the 30% variations of cycle numbers, as discussed in Section 4.4.1. The results are closer for small $R$ but this may be a function of fewer repeats in the larger numbers of cycle ranges. For the air-chamber tests, the vertical bars mark the load range measured by a load cell at a screw or use a reaction coefficient range of 1.15 to 1.25.
Figure 4.43: Peak load per screw versus number of cycles to failure for line load test rig (Mahendran 1989) and air-chamber (PLA) trials

4.6 Summary of cyclic loading

Double 900 mm span corrugated cladding specimens were installed into an air-chamber and subjected to cyclic pressures. The failure modes for the cladding were through the formation and growth of extensive cracks at or near the screw locations allowing the cladding to be sucked over the head of the screws. The crack patterns were classed into two categories: crease type cracking and star type cracking, with the crease type cracks typically forming at pressures greater than the local plastic deformation (LPD) limit for the cladding. With the air-chamber being able to test a full width cladding sheet, the trials showed more failures were associated with the screws adjacent to the top lap.

The cyclic trials demonstrated that the cladding subjected to pressure cycles with small amplitudes (large $R$) can resist more cycles than when subjected to similar peak pressure cycles but that have larger amplitudes (small $R$). The relationship between the applied pressure and fastener pullout load was shown to vary with increasing numbers of load cycles due to cladding deformation and damage. The lateral reactions on the screws were shown to be in the order of 30 to 50% of the vertical reaction at the screw for various scenarios, such as, cladding buckling, eccentric crack growth and adjacent screw failure. The lateral loads cause movement in the screw.
Within the variability of numbers of cycles to failure for similar peak loads, the air-chamber trials compared favourably with the previous line load test results presented by Mahendran (1989) and Xu (1992), and on which the current Australian cladding fatigue test criteria is based.
5. Wind loading tributary area for a cladding screw

5.1 Point load testing (Influence coefficient)

The uplift reaction at a screw is given by Equation 5.1: where $F_U(t)$ is the force at the screw at a given time, $p(x,y,t)$ is the pressure acting on the cladding at a distance $x$ and $y$ from the screw, $I(x,y)$ is the influence coefficient for the reaction at the screw for the pressure at $(x,y)$, and $dA$ is the area over which $p$ acts.

$$F_U(t) = \int p(x,y,t) I(x,y) \, dA \quad (5.1)$$

The structural influence coefficient is the value relating a structural load effect (e.g., reaction, bending moment and shear) for a unit load moving along the structure. In regards to the cladding, the structural influence coefficient of load at a screw is a function of the cladding’s properties (e.g., stiffness, strength, etc.) in relation to the screw location. Influence coefficients for fastener reactions were determined from a series of point load tests, as described in Section 3.3, on the triple span cladding specimen. The influence coefficient is defined here as the ratio of fastener reaction at screw S2 divided by the specific load that has been applied at various locations across a structure. The screw S2 was the screw adjacent to the overlap and located on the first internal support.

Table 5.1 provides the influence coefficients for the uplift reaction at S2, with the corresponding contour plot shown in Figure 5.1. The coefficients reveal that the majority of load was transferred along the screw fixed crests as per simple beam theory. However, there was typically 5% of load that was “felt”, when each point load was applied to the adjacent screw fixed crests, implying that adjacent fasteners also support some load. The vertical deflections of the cladding crests, given in Table 5.2, reinforce that the point loads transfer load to the adjacent screw fixed crests. Therefore, the majority of load applied to the cladding is transferred via the stiff cross sectional profile (i.e., along the crests) with a small percentage of the load carried to adjacent screws from the cladding acting as a plate.

Table 5.1: Influence coefficients for the uplift reaction at screw S2

<table>
<thead>
<tr>
<th>Screw position (mm)</th>
<th>0</th>
<th>225</th>
<th>450</th>
<th>675</th>
<th>900</th>
<th>1125</th>
<th>1350</th>
<th>1575</th>
<th>1800</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0*</td>
<td>0.02</td>
<td>0.04</td>
<td>0.04</td>
<td>0*</td>
<td>(S1)</td>
<td>0.04</td>
<td>0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>152</td>
<td>0*</td>
<td>0.27</td>
<td>0.55</td>
<td>0.76</td>
<td>1*</td>
<td>(S2)</td>
<td>0.73</td>
<td>0.47</td>
<td>0.21</td>
</tr>
<tr>
<td>304</td>
<td>0*</td>
<td>0.02</td>
<td>0.05</td>
<td>0.05</td>
<td>0*</td>
<td>(S3)</td>
<td>0.07</td>
<td>0.04</td>
<td>0.04</td>
</tr>
</tbody>
</table>

* Inferred values tabled as point loads were not able to be applied at screw locations.
Table 5.2: Vertical movement of cladding for point loads at locations along crest line S2

<table>
<thead>
<tr>
<th>Screw position (mm)</th>
<th>0</th>
<th>225</th>
<th>450</th>
<th>675</th>
<th>900</th>
<th>1125</th>
<th>1350</th>
<th>1575</th>
<th>1800</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>0.4</td>
<td>0.5</td>
<td>0.4</td>
<td>0</td>
<td>0.3</td>
<td>0.5</td>
<td>0.4</td>
<td>0</td>
</tr>
<tr>
<td>152</td>
<td>0</td>
<td>2.8</td>
<td>4.5</td>
<td>2.9</td>
<td>0</td>
<td>2.6</td>
<td>3.6</td>
<td>2.5</td>
<td>0</td>
</tr>
<tr>
<td>304</td>
<td>0</td>
<td>0.5</td>
<td>0.6</td>
<td>0.4</td>
<td>0</td>
<td>0.4</td>
<td>0.5</td>
<td>0.5</td>
<td>0</td>
</tr>
</tbody>
</table>

Figure 5.2 compares the experimental influence coefficient with those of idealised double span and triple span beams. The response from the cladding at the screw S2 for point loads along its crest was more linear than that derived from simple beam theory, for a triple span configuration. In combining the influence coefficients from the point loads on all three screwed crests (S1 to S3, inclusive) the combined curve approximates that of a continuous beam. The influence of the third span on the reaction at S2 can be seen in the difference in shape of the three screwed crest cladding curve (cladding - 3 crests) in the first and second 900 mm spans.

![Figure 5.1: Contour of Influence coefficients for uplift reaction at S2](image1)

![Figure 5.2: Influence coefficient along length of cladding crest line containing screw S2](image2)
5.2 Wind loads acting on a cladding screw tributary area

Full scale measurements and wind tunnel studies have shown that the wind pressures acting on a building surface are not a constant load applied over a large area, as discussed in Section 2.5.3. For the building envelope (e.g., the cladding), the large peaks are transient and act over a small area, relative to building geometry.

Surry et al. (2007) examined the effects of increasing tributary area on the averaged pressure in a corner region of a low rise, low roof pitch building. Figure 5.3 shows the decreasing peaks for an increasing area for a quartering wind. It can be observed that not only was the peak pressure reduced, but also the number of fluctuations, that is, the number of cycles, so that both the uplift and number of cycles depend on area.

Based on the reduction in peak pressures and comparison testing of a standing seam cladding system at Mississippi State University (MSU), as described in Section 2.6.5.2, Surry et al. (2007) suggested a reduction in the peak design pressure for the modelled standing seam roofing when tested using a uniform load. For the studied quartering wind direction and specific geometry, they suggested a conservatism of up to 50% between applying the peak design pressures using the North American E1592 test (a test that applied a uniform pressure) and that of a simulated wind load test in both model scale at the University of Western Ontario (UWO) and on a cladding specimen at MSU.

A standing seam cladding system is fixed with clips that are either nailed or screwed to the building’s structure (which could be a metal or plywood deck or purlins). Like pierced fixed metal cladding, failures of standing seam cladding systems are typically associated with its fixings, that is, its clips disengaging from the cladding or pulling out of the sub-structure. However, for the study by Surry et al. (2007), the typical clip tributary area described for the cladding system was 0.92 m², that is a 610 mm spacing between seams (ribs) and 1520 mm between rows of clips (e.g. purlins). The typical tributary area for the corrugated pierced fixed roofing is much smaller at 0.14 m² (152 mm × 900 mm). For the specific case detailed in Figure 5.3, the peak point pressure reduces by roughly 30% in moving from a single tap to a tributary area of 0.14 m². However, the tributary area for the corrugated system is a very elongated rectangle.
Specialised wind tunnel data, made available by UWO, of a low pitch low-rise building, with a very high density of pressure taps, was used to explore the effects of area averaging pressures over an elongated rectangle representative of the area of cladding being held down by a screw. The full scale dimensions of the building were 46 m × 30.5 m × 10 m high, with a roof slope of 0.25 on 12, with a drawing of the roof plan given in Figure B1, in Appendix B. The model was tested at UWO, with details of the model study described by Morrison and Kopp (2010).

The high density tap layout contains a corner region consisting of over 700 pressure taps spaced at 152 mm x 152 mm (equivalent full scale dimensions), as shown in Figure B2. This data was used to study the peaks and numbers of cycles at a screw using pressure data from selected taps in the region of influence for the screw. For the model’s geometry, it was assumed that fasteners were spaced at 152 mm and purlins were at 1220 mm which coincided with eight 152 mm (6") tap spacings, which enabled various combinations of taps for area averaging or applying influence coefficients.

Figure 5.4 shows a coloured surface plot of the variation of minimum peak pressures for the corner region of the building for winds from 0°, 30°, and 45°. The rectangle shown in the plots represented the tributary area of the cladding fastener for one of the highest loaded regions of the building.
nearly flat roof. The rectangle represents the second screwed crest in from the gable edge, with the centre of the rectangle sited on tap 409 which aligned with a screw location on the first purlin in from the edge purlin (no eaves on building). The first screwed crest would typically be covered by screw fixed flashing. The ‘V’ shaped line of large negative pressures radiating out from the corner for the 45° case highlights the region influenced by the conical vortices as described by Holmes (2007).

![Diagram showing contours of minimum peak pressure coefficients](image)

**Figure 5.4:** Corner region of building showing contours of minimum peak pressure coefficients for 0°, 30° and 45° wind angles
(Note the tributary area of screw, centred on tap 409, is shown as the black rectangle)

Table 5.3 gives the calculated minimum pressure coefficients over the time histories using the different area averaging and influence line methods. Comparisons were made between a single tap (409), two taps (407 and 411), and nine taps (405 to 413) for the area averaging over each time step in the wind tunnel data record for each of the four directions. Similarly, the influence line method used taps 401 to 501 with the double span simple beam theory coefficients and the linear interpolation coefficients as shown in Figure 5.2. Fifty one taps (101 to 201, 401 to 501 and 701 to 801) simulating a screwed crest on either side of 409, were used for the influence coefficients based on the experimental loading of the cladding sheet.
Table 5.3: Comparison of area averaging method with direction

<table>
<thead>
<tr>
<th>Direction</th>
<th>Screw Posit.</th>
<th>Number of taps</th>
<th>Method</th>
<th>Cp</th>
<th>Cp x A</th>
<th>% of one tap</th>
<th>Time stamp</th>
</tr>
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<tbody>
<tr>
<td>0</td>
<td>409</td>
<td>1</td>
<td>Point</td>
<td>-4.37</td>
<td>-0.81</td>
<td>100</td>
<td>19327</td>
</tr>
<tr>
<td>0</td>
<td>409</td>
<td>2</td>
<td>Area average</td>
<td>-4.21</td>
<td>-0.78</td>
<td>96</td>
<td>18842</td>
</tr>
<tr>
<td>0</td>
<td>409</td>
<td>9</td>
<td>Area average</td>
<td>-4.42</td>
<td>-0.82</td>
<td>101</td>
<td>3373</td>
</tr>
<tr>
<td>0</td>
<td>409</td>
<td>17</td>
<td>Influence double</td>
<td>-5.45</td>
<td>-1.01</td>
<td>125</td>
<td>3373</td>
</tr>
<tr>
<td>0</td>
<td>409</td>
<td>17</td>
<td>Influence linear</td>
<td>-4.31</td>
<td>-0.8</td>
<td>99</td>
<td>3373</td>
</tr>
<tr>
<td>0</td>
<td>409</td>
<td>51</td>
<td>Influence 3 rows</td>
<td>-4.69</td>
<td>-0.87</td>
<td>107</td>
<td>3373</td>
</tr>
<tr>
<td>30</td>
<td>409</td>
<td>1</td>
<td>Point (a)</td>
<td>-9.44</td>
<td>-1.75</td>
<td>100</td>
<td>12389</td>
</tr>
<tr>
<td>30</td>
<td>409</td>
<td>2</td>
<td>Area average (b)</td>
<td>-9.49</td>
<td>-1.76</td>
<td>101</td>
<td>12389</td>
</tr>
<tr>
<td>30</td>
<td>409</td>
<td>9</td>
<td>Area average (c)</td>
<td>-9.65</td>
<td>-1.79</td>
<td>102</td>
<td>12389</td>
</tr>
<tr>
<td>30</td>
<td>409</td>
<td>17</td>
<td>Influence double</td>
<td>-10.95</td>
<td>-2.03</td>
<td>116</td>
<td>12389</td>
</tr>
<tr>
<td>30</td>
<td>409</td>
<td>17</td>
<td>Influence linear</td>
<td>-8.90</td>
<td>-1.65</td>
<td>94</td>
<td>12389</td>
</tr>
<tr>
<td>30</td>
<td>409</td>
<td>51</td>
<td>Influence 3 rows</td>
<td>-9.81</td>
<td>-1.82</td>
<td>104</td>
<td>12389</td>
</tr>
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<td>409</td>
<td>1</td>
<td>Point</td>
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<td>-1.56</td>
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<td>Area average</td>
<td>-8.84</td>
<td>-1.64</td>
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<td>409</td>
<td>9</td>
<td>Area average</td>
<td>-8.74</td>
<td>-1.62</td>
<td>104</td>
<td>13405</td>
</tr>
<tr>
<td>45</td>
<td>409</td>
<td>17</td>
<td>Influence double</td>
<td>-9.76</td>
<td>-1.81</td>
<td>116</td>
<td>13689</td>
</tr>
<tr>
<td>45</td>
<td>409</td>
<td>17</td>
<td>Influence linear</td>
<td>-7.93</td>
<td>-1.47</td>
<td>94</td>
<td>13405</td>
</tr>
<tr>
<td>45</td>
<td>409</td>
<td>51</td>
<td>Influence 3 rows</td>
<td>-8.68</td>
<td>-1.61</td>
<td>103</td>
<td>13405</td>
</tr>
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<td>90</td>
<td>409</td>
<td>1</td>
<td>Point</td>
<td>-3.99</td>
<td>-0.74</td>
<td>100</td>
<td>653</td>
</tr>
<tr>
<td>90</td>
<td>409</td>
<td>2</td>
<td>Area average</td>
<td>-3.94</td>
<td>-0.73</td>
<td>99</td>
<td>653</td>
</tr>
<tr>
<td>90</td>
<td>409</td>
<td>9</td>
<td>Area average</td>
<td>-3.88</td>
<td>-0.72</td>
<td>97</td>
<td>653</td>
</tr>
<tr>
<td>90</td>
<td>409</td>
<td>17</td>
<td>Influence double</td>
<td>-4.64</td>
<td>-0.87</td>
<td>118</td>
<td>653</td>
</tr>
<tr>
<td>90</td>
<td>409</td>
<td>17</td>
<td>Influence linear</td>
<td>-3.77</td>
<td>-0.7</td>
<td>95</td>
<td>653</td>
</tr>
<tr>
<td>90</td>
<td>409</td>
<td>51</td>
<td>Influence 3 rows</td>
<td>-4.21</td>
<td>-0.78</td>
<td>105</td>
<td>653</td>
</tr>
</tbody>
</table>

The peak negative pressure coefficients for either the one, two or nine pressure taps were all very similar. The most obvious conclusion derived from Table 5.3 was that there is no significant variation from the point pressure measurement to an area averaged result, thus highlighting the effect of conical vortices on the elongated tributary area as opposed to the larger square area detailed by Surry et al. (2007). Figure 5.5 shows contours of external pressure coefficients for the 0° direction case for the 51 taps at three different times during the wind tunnel trace. The single tap, double tap and nine taps used are highlighted in the figure. Apart from 0°, the area averaged peak pressure coefficients for tap 409 for each wind direction were derived from a single time step, that is, a vortex (or suction bubble) dominated the tributary area, as shown in Figure 5.6.

For the four wind directions, the resultant “screw reaction” using the influence method based on the point load experimental results, was approximately 5% greater than the reaction derived from the area averaged methods. The large magnitude peaks, that acted on the gable edge taps (102 and 103), played little influence in the overall result as they were offset by the much smaller pressures on the opposite row (702 to 705) due to the high gradient of the pressure contours.
Figure 5.5: Snapshot of contours of pressure coefficients for 51 taps in the corner region for 0° wind direction, at three different time stamps

Figure 5.6: Snapshot of contours of pressure coefficients for wind angles of 30°, 45° and 90° for 51 taps in the corner region

It was interesting to note the regions of largest negative peaks were located at taps 102 and 401, which were right at the building corner edge. In assuming a screw located at tap 401 has a tributary
area extending to tap 404, the peak load for a unit pressure (i.e., $C_p \times \text{Area}$) for the eaves edge fixing (401) was 1.06 and 1.17 for the $30^\circ$ and $45^\circ$ cases. Even though the values were less than the values given in Table 5.3 for screw 409, the values were greater than the assumed 50%, and in some cases 40%, of 409 that would be typical for design of the cladding/screw at 401. Design standards assume a uniform pressure coefficient where, as the wind tunnel time series data demonstrated, there was a rapidly changing and steep pressure gradient.

Rainflow analyses of the time series data were undertaken to examine possible differences in numbers of loading cycles acting on screw 409. The results for the $30^\circ$ wind angle case is given in Appendix B. Table 5.4 is a subset of the data. Table 5.4(a) shows the numbers of cycles when using the times series for the single 409 tap. Parts (b) to (d) detail the differences in numbers of cycles for the two tap area averaging, nine tap area averaging and 51 tap influence coefficient data, respectively. Overall there was a slight reduction in the numbers of cycles, although these were predominantly in the first 10% range and therefore have minimal influence on potential fatigue damage, depending of course, on wind speed and cladding capacity.

<table>
<thead>
<tr>
<th>Range</th>
<th>0.00</th>
<th>1.00</th>
<th>2.00</th>
<th>3.00</th>
<th>4.00</th>
<th>5.00</th>
<th>6.00</th>
<th>7.00</th>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a) 1 tap – Numbers of cycles</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td>total</td>
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<td>826</td>
<td>254</td>
<td>108</td>
<td>41</td>
<td>25</td>
<td>13</td>
<td>5</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(b) Area average 2 taps – Numbers of cycles difference to 1 tap case (a)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>total</td>
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<td>-13</td>
<td>-1</td>
<td>-3</td>
<td>4</td>
<td>-2</td>
<td>3</td>
<td>2</td>
<td>1</td>
<td>0</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(c) Area average 9 taps – Numbers of cycles difference to 1 tap case (a)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>total</td>
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<td>-1</td>
<td>-6</td>
<td>4</td>
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<td>0</td>
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<td>1</td>
<td>0</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(d) Influence coefficient 51 taps – Numbers of cycles difference to 1 tap case</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>total</td>
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<td>5</td>
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<td>-1</td>
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<td>1</td>
<td>0</td>
</tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>95%</td>
<td>98%</td>
<td>98%</td>
<td>105%</td>
<td>100%</td>
<td>108%</td>
<td>92%</td>
<td>140%</td>
<td>200%</td>
<td>100%</td>
</tr>
</tbody>
</table>

Of more significance were the addition of just one or two cycles with a 40% to 60% mean and 70% to 90% range, meaning a cycle extending from near zero to nearly the peak, for the two tap, nine tap and 51 tap, cases. These cycles do have the potential of exacerbating fatigue damage, depending of course on wind speed and cladding capacity. For the 51 tap case, there was also an observed shift in the mean of the cycles, with the mean increasing for the same cycle ranges. This implies that the “troughs” of the cycles were moving away from “zero” load while the peaks were increasing.
For the other wind angles, the rainflow counts of cycles at 409 showed similar trends to those for the 30° case, with a slight reduction in overall cycles but a few additional large cycles. However, for the other directions the shift of cycles with larger means, shown in Table 5.4(d) was not as pronounced.

The little change in overall cycle numbers for an increasing number of taps showed that the small elongated tributary area of the cladding fastener, in the highly loaded corner region of a low pitch roof, was predominantly influenced by the formation of a conical vortex that enveloped the tributary area.

5.3 Summary

Point load testing to derive structural influence coefficients showed that the pierced fixed cladding tributary area is elongated along the crest or rib of the cladding. Only about 5% of reaction at a fastener is from the pressure applied in an adjacent screwed crest. Therefore, there is minimal influence on the load on a fastener from an adjacent screwed crest which informs the selection of a test specimen’s representative width.

From the analysis of high density pressure tap time series data for the elongated tributary area in the corner of a flat roof, the peak pressure and numbers of cycles derived from using a single tap or multiple taps within the area of influence, were similar (for the location and model geometry used). However, for cladding designs that may be approaching capacity, the addition of one or two of the larger magnitude cycles derived from the multiple tap cases would be detrimental. Also, the larger loads applied to the first cladding span would increase cladding deflection leading to increased deformation which may accelerate crack development depending on cladding design capacity.
6. Response to laboratory simulated cyclonic wind loads

6.1 Selection of representative fluctuating wind pressures

To obtain a series of wind loads acting over time on cladding and its fixings at the edge and other regions of a roof, information is required on the wind event, approach terrain and the building geometry.

The wind field (speed and direction) in a region impacted by a cyclone is not only dependent on the characteristics of the cyclone, for example the radius of eye, forward speed, and intensity, but also the approach terrain and topography. The winds impacting on a specific building are also dependent on the geometry of the building and its orientation to the wind field, that is, its location with respect to the passage of the cyclone.

The variation of wind speed and direction measured in cyclones indicate an increasing wind speed as the cyclone approaches, followed by a drop in the wind speed combined with a change in wind direction as it travels past the measurement point. Figure 6.1 illustrates the change in wind speed and direction for Cyclone Vance recorded by a Dines anemograph at Learmonth in Western Australia in 1999. Over the central five hour period the gust wind speeds increased from 36 m/s to 74 m/s then decreased to 31 m/s with a change in wind direction of approximately 120°. Figure 6.2 shows the position of Learmonth relative to the passage of the cyclone. It is not difficult to imagine a different anemograph if the cyclone had passed to the West of Learmonth; the wind speeds affected by the overland flow with a change in wind direction that would have been reversed. The duration of a cyclone (i.e., slow or fast translational speed) will also greatly affect the potential for wind induced fatigue damage (Ginger et al. 2007).

As mentioned in Section 2.5.5, Jancauskas et al. (1994) proposed a “design cyclone” for use in determining a standardised wind induced fatigue testing criteria that, in a slightly modified form, is referred to as the Low-High-Low test defined in the BCA (2007). The parameters chosen for the “design” cyclone had a maximum gust wind speed of 70 m/s, a radius to maximum winds of 25 km, a central pressure of 930 mb, and a forward velocity of 15 km/h. They noted that the parameters were based on those similar to Cyclone Winifred that crossed the coast near Innisfail, in Queensland, in 1986. Mahendran (1993) conducted parametric studies, varying the forward velocity and radius of maximum winds of the cyclone. He concluded that a cyclone of small forward velocity and large radius would cause a very severe fatigue loading. However, he added that it was unlikely that all the cyclone parameters would take the worst values, and the calculated
fatigue loading from Cyclone Tracy with its slow forward speed of 7 km/h and 11 km radius had almost the same fatigue loading as the design cyclone based on Cyclone Winifred’s parameters.
The parameters and cyclone model assumed a circular wind flow and calculated the wind speed impacting on a target building as the vectorial addition of the rotating winds striking the building and the component of velocity from the cyclones forward movement. From using the set “design” cyclone parameters and assuming the maximum winds impacting the target building at the worst angle, Jancauskas et al. (1994) presented the change in mean wind speed over time and the change in wind direction over time from the point of view of the target building. Figure 6.3 shows the change in wind speed at the target building and direction over a five hour duration as per Jancauskas et al. (1994), but modified for wind speeds at 10 m height in open terrain as opposed to 4 m roof height.

![Figure 6.3: Variation of mean wind speed and direction over time (Jancauskas et al., 1994)](image)

From their presented data of mean and peak wind speeds, Jancauskas et al. (1994) used a 3 second peak to mean gust factor of approximately 1.7. Recent measurements and analysis have shown that the gust factor can be in the order of 1.4 to 1.7 for coastal terrain depending on factors such as roughness and on-shore or off-shore winds (Harper et al. 2008). A lower gust factor will give a higher mean wind speed for the same gust wind speed. Schroeder et al. (2009) detailed that the average gust factors, in the eye wall region, varied from 1.4 to 1.6 for open to rough terrain, respectively.

The wind load acting on a structure, or part of it, can be analysed by combining time varying pressure coefficients, typically taken from wind tunnel studies, with the corresponding approach wind speed and direction, as described in Figure 6.3.
### 6.1.1 Building geometry

In order to derive a wind pressure trace to use in the PLA, several parameters such as building geometry, location of target cladding on building and magnitude of internal pressure, were required. Although, there are many different forms of building shapes, the predominant forms of buildings that use light gauge metal cladding are low rise residential buildings (houses) and low rise commercial buildings (e.g., large sheds).

Recent damage surveys following destructive cyclonic (and storm) events highlighted a higher than expected percentage of damage to “engineered” structures (sheds) (Henderson and Ginger 2008). It had been residential construction that typically showed greater levels of damage. However, the introduction of prescriptive design criteria and engineering details since the early 1980’s has greatly improved residential resilience. For example, a dominant windward wall opening is assumed in the wind loading design for housing (Standards -Australia 2006). This is not the case for engineered structures, such as commercial and industrial buildings, as the application of full internal pressure depends on the design considerations, such as, an assessment of the ability of the building’s envelope that includes windows and doors, to resist the applied wind pressures and wind driven debris.

One of the forms of housing, that was destroyed following Cyclone Tracy, was the rectangular, elevated, very low pitch roof house. Since the early 1980’s, roof truss fabrication plants entered the market place in cyclonic regions in Australia, with the result that a majority of housing since the mid 80’s has a roof slope of at least 10 to 15 deg. Since the early 90’s, typical roof slopes for housing (metal roofs) are about 20 degrees (Henderson and Harper 2003). In contrast, shed roofs typically have low roof slopes.

Industrial sheds typically have open interior floor plans that are subjected to the “same” internal pressure. The application of internal pressures on roof cladding in a house would be dependent on location of openings (man holes, vents) in ceiling and eaves linings in relation to openings in walls associated with the division of rooms and wall openings (e.g., Kopp et al. 2008).

Twenty six large sheds, 10 cold formed portal frame and 16 hot rolled portal frame, located in Townsville, which is in the cyclonic region Australia, were surveyed as part of a vulnerability study of these engineered structures (Leitch et al. 2007). The geometries of these buildings are given in Table 6.1.

For the surveyed sheds, there was no correlation between cladding span and the design pressures. That is, other factors such as purlin capacity, walk on roof capacities, and aesthetics may be
governing the designer’s choice, not the cladding wind load capacity. The study showed that the roof cladding design in six of the 26 sheds did not account for possible large internal pressure as per design standard requirements or used an incorrect wind load modifier such as for terrain or local pressure factor, as defined in Section 6.2.

Table 6.1: Data from survey of 26 industrial sheds from Townsville region

<table>
<thead>
<tr>
<th>Number</th>
<th>Length (m)</th>
<th>Breadth (m)</th>
<th>Eaves ht (m)</th>
<th>Roof slope</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>min</td>
<td>Max</td>
<td>min</td>
<td>Max</td>
</tr>
<tr>
<td>Cold Form</td>
<td>10</td>
<td>12.0</td>
<td>24</td>
<td>6.0</td>
</tr>
<tr>
<td>averages</td>
<td>15.5</td>
<td>10.0</td>
<td>3.7</td>
<td>10.6°</td>
</tr>
<tr>
<td>Hot Rolled</td>
<td>16</td>
<td>18.5</td>
<td>112</td>
<td>12.4</td>
</tr>
<tr>
<td>averages</td>
<td>54.2</td>
<td>25.4</td>
<td>6.8</td>
<td>5.8°</td>
</tr>
<tr>
<td>Total</td>
<td>26</td>
<td>12.0</td>
<td>112</td>
<td>6.0</td>
</tr>
<tr>
<td>averages</td>
<td>39.3</td>
<td>19.5</td>
<td>5.6</td>
<td>8.2°</td>
</tr>
</tbody>
</table>

The cladding spans ranged from 400 mm to 1500 mm. However, approx 60% of the sheds had a cladding end span to building height ratio of 0.14 to 0.22 as shown in Figure 6.4. Note that with the smaller ratio, more of the cladding span is contained within the area of higher pressures generated by the edge vortices.

![Histogram](image)

Figure 6.4: Histogram of cladding end span to building eaves height ratio

For this analysis, the building configuration used is shown in Figure 6.5. The building dimensions relate to a scaled wind tunnel model test contained within the United States’ National Institute of Standards and Technology (NIST) building data base, as described in Section 6.3 (Ho et al. 2005).
The model building had equivalent full-scale plan dimensions of 38.1 m × 24.4 m, a gable end roof of slope 1:12 (4.8°) and an eaves height of 4.9 m and was tested in the Boundary Layer Wind Tunnel II at UWO. The building is subsequently referred to in the text as Building 5-5 representing 5 m wall height with a 5° roof slope.

Areas A, B, C and D represent cladding fastener tributary areas of 0.152 m × 0.9 m on the roof and are shown in Figure 6.5, along with wall point pressure locations $W_L$ and $W_G$. Area A represented the corner region adjacent to the gable end while B and C were at the middle of the building length. For building models with a higher pitch roof, an area D was the mirror image of C on the other side of the ridge line.

![Figure 6.5: Diagram of Building 5-5 including location of pressure taps](image)

A single storey gable ended house with a 10° roof pitch was used for the wind tunnel study by Jancauskas et al. (1994) to determine the wind loads acting on the cladding at the gable end location. The 1:50 scale model, with equivalent full scale dimensions of 14 m long × 7 m wide, had 1200 mm eaves overhang on the long walls and 450 mm overhang on the gable ends (Capitano 1987). Three pressure taps were manifolded together to achieve the equivalent tributary area for a cladding fastener at the edge of the gable roof.
6.2 Wind loads from AS/NZS 1170.2

Wind loading codes and standards such as AS/NZS1170.2:2002 (Standards Australia, 2002) specify ultimate limit state design wind speeds based on return periods of 500 to 1000 yrs for design of typical buildings as discussed in Chapter 2. In Cyclonic Region C of Australia, this design gust wind speed \((V_{500})\) is 69.3 m/s at a 10m elevation in open country terrain (Terrain Category 2 according to AS/NZS1170.2:2002).

Design loads of cladding and fixings on low-rise buildings are typically calculated from pressures derived using nominal shape factors or pressure coefficients, provided in AS/NZS 1170.2:2002 (Standards Australia, 2002). The peak \((P_{\text{peak}})\) external pressures are calculated from Equation 6.1, where \(\rho\) is the density of air, and \(C_{\text{fig}}\) is the aerodynamic shape factor. For external and internal pressures, the aerodynamic shape factors are \(C_{\text{fig}} = C_{p,e}(K_a \times K_c \times K_l \times K_p)\) and \(C_{\text{fig}} = C_{p,i} \times K_e\), respectively. The relevant external and internal pressure coefficients, \(C_{p,e}\) and \(C_{p,i}\), are obtained from Section 5, and Appendix C in AS/NZS 1170.2:2002, and \(K_a\), \(K_c\), \(K_l\), and \(K_p\) are factors for area-averaging, load combination, local-pressure effects, and cladding permeability. The dynamic response factor \(C_{\text{dyn}}\) is taken equal to 1.0 for these types of (i.e., “static”) structures, and \(V_h\) is the 3sec peak design gust wind speed at mid-roof height, defined in Equation 6.2. \(V_R\) is defined as the regional wind speed based on the prescribed building importance level defined in the Building Code of Australia (BCA 2007) as described in Section 2.5.2. For a normal building in Cyclone Region C, \(V_R\) is 69.3 m/s. \(V_h\) is calculated using modifiers to account for wind direction \(M_d\), terrain and height \(M_{z,\text{cat}}\), shielding \(M_s\), and topography \(M_t\).

\[
P_{\text{peak}} = 0.5 \times \rho \times V_h^2 \times C_{\text{fig}} \times C_{\text{dyn}} \quad \text{(6.1)}
\]

\[
V_h = V_R \times M_d \times M_{z,\text{cat}} \times M_s \times M_t \quad \text{(6.2)}
\]

AS/NZS1170.2:2002 (Standards Australia, 2002) gives an external pressure coefficient \(C_{p,e}\) of -0.9 for edge regions of a low pitch roof, and internal pressure coefficients \(C_{p,i}\) of -0.2 and 0 for the nominally sealed and 0.7 for the dominant windward wall opening cases. The factors \(K_a\), \(K_c\), and \(K_p\) were all taken to equal 1.0 for this study. A local pressure factor \(K_l\) of 2.0 applies for the design of cladding and fixings, located in an area \(< a^2/4\) and within a distance \(a/2\) from the windward roof edge, and a \(K_l\) of 1.5 applies in an area \(< a^2\) and within a distance \(a\) from the windward roof edge. The dimension \('a'\) is the minimum of \(0.2 \times \text{building depth} (d), 0.2 \times \text{its breadth} (b)\) or its height \((h)\). A \(K_l\) of 3.0 has been proposed for a draft version of updated changes to the Australian wind loading standard. The draft details \(K_l\) of 3.0 located in an area \(< a^2/4\) only in the roof corners bounded by distance \(a\).
For Building 5-5, shown in Figure 6.5, with a regional ultimate limit states (ULS) design wind speed of 69.3 m/s situated in open terrain, design pressures given in Table 6.2, were determined using AS/NZS1170.2:2002.

<table>
<thead>
<tr>
<th></th>
<th>Cp,e</th>
<th>Kl</th>
<th>Design Pressure (external) (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area A</td>
<td>-0.9</td>
<td>2</td>
<td>-4.68</td>
</tr>
<tr>
<td>Area B</td>
<td>-0.9</td>
<td>2</td>
<td>-4.68</td>
</tr>
<tr>
<td>Area C</td>
<td>-0.3</td>
<td>1</td>
<td>-0.78</td>
</tr>
<tr>
<td>Area A</td>
<td>-0.9</td>
<td>3</td>
<td>-7.02 (draft standard)</td>
</tr>
<tr>
<td>Wall</td>
<td>+0.7</td>
<td>1.25</td>
<td>+2.28</td>
</tr>
</tbody>
</table>

### 6.3 Wind tunnel data

A range of generic low-rise building models were tested in the wind tunnel at the University of Western Ontario (UWO) in Canada, as part of a project initiated by the United States’ National Institute of Standards and Technology (NIST). The project provided a validated, quality controlled set of model scale measurements for use in analysing wind loads on buildings. For complete details of the test set-up and model configurations, refer to Ho et al. (2005). Detailed comparisons with previously published data and with current building codes or standards (including AS/NZS1170.2:2002 (Standards Australia, 2002)) can be found in St. Pierre et al. (2005), while analysis of internal pressures with regard to the NIST database can be found in Oh et al. (2007).

Pressure time series measurements from this NIST database have been used to derive pressure traces for roof cladding areas for the analysis of wind loading on cladding.

The building configuration, Building 5-5, shown in Figure 6.5, was tested at a length scale of 1/100 in a simulated open approach atmospheric boundary layer. The gust factor ($U_{3 sec}/U_{mean}$) at 10 m height in this terrain is assumed to be 1.5. The mean wind speed at the reference height of 10 m was 8.8 m/s so that $Re = 30,000$ based on the eave height. External point pressure measurement locations (taps), for example R1 and R2 are combined to give area averaged pressures representative of the load on the cladding fastener tributary area A. Two taps were also used for the area B with one tap located approximately 1 m from the ridge used for areas C and D. Taps $W_L$ and $W_G$ were used for the wall pressures.

Analysis of the time series data from a high density tapped wind tunnel model in conjunction with the point load influence testing detailed in Section 5 showed for pierced fixed cladding, the small
The building 5-5 time series pressure data for the taps were analysed for approach winds over a 180° range in 5° intervals. A 762 mm long, tuned, tubing system connects each tap to high speed pressure scanners, as described by Ho et al. (2005). Fluctuating pressures at these taps which were essentially measured simultaneously were sampled at 500 Hz for 100 seconds, for a single run. Measured mean and peak pressures are defined in terms of a pressure coefficient:

\[ C_p = \frac{p}{\frac{1}{2} \rho U^2} \]

where \( p \) is the pressure and \( \frac{1}{2} \rho U^2 \) is the mean dynamic pressure at mid-roof height. Pressure acting towards the surface is defined as positive.

Area averaged minimum peak and mean external pressure coefficients acting on areas A, B and C, and maximum peak for the wall, \( W_L \), for wind directions \( \theta = 0° \) to 180° are shown in Figure 6.6. The large suction pressures were generated on area A (taps adjacent to the gable end) by the formation of conical vortices for \( \theta = 30° \) to 60°. The peak suction pressures acting on Area B, from the wind normal to the long wall (~90°) were approximately 60% of the peaks acting on A from the quartering wind. Positive peak pressure coefficients greater than 2.0 were experienced on the wall area \( W_L \), for \( \theta = 45° \) to 110°.

Figure 6.6: Peak and mean \( C_p \) for Areas A, B and C and Wall \( W_L \) for Building 5-5
6.4 “Design” cyclonic wind trace

The “design” cyclone, as defined by Jancauskas et al. (1994) and described in Section 6.1, produced varying wind speed and direction over time. The largest peak pressure for a building configuration, for the “design” cyclone is achieved where the maximum mean wind speed coincides with the incident wind angle on the building that produces the largest $C_p$. Application of a gust factor of 1.5 in open terrain to the design cyclone proposed by Jancauskas et al. (1994) in Section 6.1 gives a maximum mean wind speed of 46 m/s at 10 m height in open terrain. Hence the mid roof height maximum mean wind speed would be 42 m/s.

A full-scale mean wind speed of 46 m/s, at the reference height of 10 m, gives a velocity scale of $U_r = 1/5.2$. Combined with the length scale $L_r = 1/100$, gives a time scale $T_r = 0.052$, essentially meaning that the 100 seconds in model scale is equivalent to 32 minutes, in full-scale, sampled at 24 Hz. However as the analysis is carried out for 15 minute blocks in full scale, the corresponding number of samples in model scale needs to be determined. In the case where the full-scale mean wind speed varies according to Figure 6.3, the relationship between the time span in the wind tunnel and full-scale and the relevant cycles in model and full-scale is given by Equation 6.3.

$$N_f = N_m \times (L_r / U_r) \times (T_f / T_m) \quad (6.3)$$

Here

$N_f$ – Number of cycles in full scale
$N_m$ – Number of cycles in model scale
$T_m$ – Time in model scale (i.e., 100 seconds)
$T_f$ – Time in full scale (i.e., 15 minutes)

With reference to Equation 6.3 an increase in the mean wind speed will also increase the number of cycles that will occur during the same time interval.

Two orientations of Building 5-5 (labelled as 25 and 75), with respect to the track of the “design” cyclone, are shown in Figure 6.7 (a) and (b). The building orientations shown in each figure are for 2.5 hours into the “cyclone”, that is, when the 42 m/s mean wind speed corresponds to the direction shown. Orientation 25 produced the highest external suction pressure coefficients (peak and mean) at area A, while orientation 75 was chosen as it produced positive mean wall pressure for the complete passage of the “design” cyclone. The variation of peak external suction on roof areas A and B, and peak external positive pressures on wall area, $W_L$, for orientations 25 and 75 are shown in Figure 6.7 (a) and (b), respectively. Figure 6.7 (a) shows the peak pressures for areas A and $W_L$ are -6.6 and 2.2 kPa for building orientation 25, while Figure 6.7 (b) shows the
peaks are -4.6 and 2.8 kPa for building orientation 75. The gable edge (area A) had the greater peak than that of area B for orientation 75.

Since the analysis is based on the wind tunnel time series data, it should be noted that the presented peaks are from the portion of data record used and are not the peaks derived from an extreme value analysis that would form the basis for a quasi-static structural design. From the -4.68 kPa design pressure given in Table 6.2, it can be seen that AS/NZS 1170.2:2002 (Standards
Australia, 2002) underestimates the design pressures on cladding fasteners for the gable corner region (area A). This is in keeping with findings by Ginger and Henderson (2003) who analysed the characteristics of the external, and the combined external and internal (i.e., net) pressures on the full-scale Texas Tech building, and showed that significantly higher pressures, than those specified in standards such as AS/NZS 1170.2:2002, could be experienced near the windward roof edge. This underestimation is being redressed with a proposed amendment to AS/NZS1170.2:2002 giving $K_I = 3$ for small tributary areas at the corners of roofs, increasing the design pressure to -7.02 kPa compared with the -6.6 kPa derived from the wind tunnel data.

Although the internal pressure will be small in the case of a building remaining nominally sealed, a dominant opening can significantly increase the internal pressure and also the net pressure for certain approach wind directions (Kopp et al. 2008; Oh et al. 2007). For Building 5-5 orientation 25, the net pressures across roof area A are calculated for a scenario assuming that a dominant opening will most likely be created on the long wall, near the position of the wall tap, $W_L$, at the peak of the cyclone. This is analogous to a windward window breaking or a door blowing inwards at approximately the 2.5 hour mark. For building orientation 75, it is assumed that the $W_L$ dominant opening is present for the total duration of the cyclone. The internal pressures are taken to be equal to the external pressure fluctuations at the opening. This is a reasonable assumption based on the analysis by Ginger et al. (2008) who showed that the internal pressure fluctuations will range between about 0.8 to 1.2 times the external pressure at a dominant opening for most combinations of opening size, building volume and approach wind speed.

The variation of the net pressures on roof area A for both building orientations for the change in wind direction with the change in wind speed is shown in Figure 6.8 and Figure 6.9. The peak net pressures for building orientations 25 and 75 are -7.1 and -5.2 kPa, respectively. Figure 6.8 clearly shows the increasing and then decreasing wind loads on area A, in a form similar to that specified by L-H-L test. Figure 6.9 has the increasing and decreasing pressures, but not as pronounced as building orientation 25, due to its lower external peak and the application of positive internal pressures for the whole duration inflating the net pressures in the earlier and later parts of the trace over that of orientation 25. Both figures highlight the fluctuating nature of the roof pressures having varying means, peaks, ranges and load ratios, $R$, defined in Section 2.2.
Figure 6.8: Pressures for varying wind angle and mean wind speed over time for Building 5-5, orientation 25 (dominant opening created at 135 minutes)

Figure 6.9: Net pressures for varying wind angle and mean wind speed over time for Building 5-5, orientation 75 (dominant opening at $W_L$ for entire duration)
6.5 Response of cladding to dynamic wind load

The PLA and air-chamber system was used to apply the dynamic pressure traces as a simulation of wind loading on a building’s cladding. The double span 900 mm cladding specimens were 0.42 mm BMT corrugated cladding installed in either Configuration A or B.

The derived pressure trace for the “design” cyclone for building 5-5 orientation 25, as shown in Figure 6.8, was applied to a cladding specimen (W5) representing roof area A in Figure 6.5.

In a purely emotive sense, there was great excitement in running any of the simulated wind loading trials. It may be an obvious statement, but the sight and sound of the cladding under load was quite different to that of the cyclic loading trials. With no repetitive cadence to the loading, the intermittent large peaks in the applied wind trace were all the more dramatic with the cladding creasing and popping, then suddenly flexing back to pulse randomly to the smaller applied pressure fluctuations for a few seconds or minutes until the next large peak that would increase the buckle (or crack).

Slight plastic deformation of the crests under the fixings was observed at approximately 80 minutes into the Building 5-5 orientation 25 trace. Creasing (i.e., permanent deformation) of the crests adjacent to the fixings, occurred approximately 107 minutes into the trace, at a pressure spike of 4.3 kPa. Accompanying the creasing was the sudden change in X reaction which was approximately 35% of the reaction in the Z direction, measured by the JR3, similar to the values measured during the cyclic load tests with the ‘H’ crack formation, as shown in Figure 4.22. Crack initiation was observed at the creases, six minutes later. Crack initiation is discussed further in Section 6.6.2.

Crack length increased, with cracks reaching lengths of about 20 mm, as shown in Figure 6.10. However, there was no observed crack growth during the last quarter (~70 minutes) of the trace, during which the pressure progressively dropped.

When subjected to the dynamic loading, the cladding responded similarly in terms of applied pressure and measured reaction at a screw for the increasing damage of the cladding as per the crack initiation and crack patterns of the cyclic load tests. Figure 6.11 shows the reaction coefficient for the applied pressures over three time intervals during the trace: (a) start, (b) onset of deformation and cracking, and (c) at the end with cracks up to 20 mm in length. The figures show
the change in stiffness and the response of the cladding system to the applied pressure with the amount of permanent deformation and cracking accrued by the cladding. The relationship of the reaction coefficient with respect to pressure for each time span was very similar to the cladding response during cyclic loading trials discussed in Section 4.3.

![Cracks initiated at crease points adjacent to screw](image)

**Figure 6.10:** Cracking of cladding at screw fixing for Building 5-5, orientation 25

![Coefficients over three time spans](image)

**Figure 6.11:** Reaction coefficient (RC) over three time spans of the design cyclone
The loading and response spectra of the selected three time spans are shown in Figure 6.12. No resonant frequencies were observed. The spectra of the applied pressure × idealised tributary area and that of the screw reaction are the same up until approximately 4 Hz for the start of the trace, 8 Hz for the middle and 5 Hz for the end component of the trace. As the mean increased, the fluctuating component of pressure also increased, as shown by the increasing spectral density. It is interesting to note that the larger the mean pressure the longer the frequency match between applied pressure and reaction. That is, interval (b) has the larger mean with the cladding subjected to membrane action increasing the cladding system’s stiffness and keeping the cladding in contact with the underside of the screw, longer than that of the other two spans as can also be observed in Figure 6.11(b).

![Figure 6.12: Spectrum of equivalent reaction (pressure × idealised area) and reaction at screw for three time spans of “design” cyclone trace](image)

The reactions in the X, Y and Z directions at a screw were measured using the JR3 load cell, as per the cyclic load trials. Figure 6.13 shows the reactions with a colour shading indicating increasing time. The “knee” in the Z reaction at approximately 300 N corresponds to the reverse flexure of the cladding crest at the crease lines similar to that observed during the ‘H’ crack cyclic loading trials, detailed in Section 4.3.2.
The cladding imparted a greater load along the Y axis (crest line) as opposed to the X axis (purlin line). The crease points formed on the same side of the purlin, that is, in the same cladding span, as shown in Figure 6.14, thereby placing an eccentric load dominant in the Y direction, on the underside of the screw head. Screw 2 (Figure 6.10) had crease points diagonally opposite, while Screw 5 (shown in Appendix A) had crease points form on the opposite side to Screw 3. The Y reaction reduced with time. For a Z reaction of 600 N, the Y reaction was approximately 36% of the Z reaction at the beginning of the interval while, at the end of the interval, the Y reaction was 25%. The Y reaction reduced due to the crease lines and cracks growing towards the screw. A progressive reduction in the X and Y reaction components was noted during the cyclic load tests, as described in Section 4.3.2.
6.5.1 **Comparison of “design” cyclone wind trace to L-H-L**

The cladding specimen subjected to the pressure trace for Building 5-5 orientation 25, “survived” the “design” cyclone with its peak of 7.1 kPa, even though there was considerable cracking and deformation. The cladding resisted the applied loads by not disengaging from its supporting structure, which is defined as a successful outcome according to Australian cladding test standards, described in Section 2.5. As noted in Section 6.6.1, with repeated dynamic wind load trials that accrue extensive cracking, there will be variability in “pass” and “fail” outcomes just as there was a large range in the numbers of cycles to failure for the sinusoidal cyclic load trials.

The maximum allowable ULS design pressure for 0.42 mm corrugated cladding over continuous 900 mm spans, from a leading manufacturer’s design tables, is 4.75 kPa that is based on the test method TR440. TR440 (1978) is a low cycle to high cycle test, and was the common test criteria for roofing prior to the recent introduction of L-H-L. In developing the original L-H-L test, Mahendran (1993) noted that the TR440 method was not as harsh in comparison due to the ‘high’ load being located at the end of the test as opposed to the middle.

Figure 6.15(a) shows a cladding specimen that was subjected to a L-H-L test in the CTS large air box, and was not able to resist a test load similar to that of a successful TR440 test at 4.8 kPa. The magnitude of load level and low R promoted excessive yielding of the cladding under the screw head producing cracks at the screw hole which led to the failure.
A second specimen was able to pass the L-H-L test with a 15% reduction of the load per cycle, with crack patterns, as seen in Figure 6.15(b). The early cycles did not cause excessive yielding under the screw, creasing occurred during the higher loaded cycles, away from the screw promoting crack growth at these stress concentrations which results in a longer crack path needed to cause failure. The crack pattern shown in Figure 6.15(b) for the L-H-L test was similar to that shown in Figure 6.10 for the simulated wind load trace. Cracks from the TR440 test were only slight and radiated from the screw holes. The simulated wind trace has a majority of cycles with load ratio $R > 0$, whereas, the L-H-L test has all cycles with $R = 0$, leading to a conservative outcome.

![Figure 6.15: Cracking patterns following L-H-L tests of cladding that (a) failed and (b) passed but had a 15% reduction in load per cycle](image)

### 6.5.2 Failure of cladding from simulated wind load

The wind load trace derived in Section 6.4 and applied to the cladding in Section 6.5 was increased to give a maximum mean wind speed of 50 m/s at 10 m height. With a gust factor of 1.5, this gives a design peak wind speed of 75 m/s. With reference to Equation 6.3 an increase in the mean wind speed will also increase the number of cycles that will occur for the same time interval.

Using this increased wind speed a trace for Building 5-5, orientation 25 was generated, still assuming that a dominant opening occurred in the long wall half way through the trace. The trace had a peak pressure of -10.6 kPa.

A cladding specimen (W6) was subjected to the increased pressure trace in the air-chamber. Slight flattening of the crests under the fixings was observed at approximately 45 minutes into the trace with the flattening increasing to a dishing of the top of the crests at approximately 60 minutes. At
approximately 75 minutes into the trace, as shown in Figure 6.16, at a pressure spike of -5.8 kPa, there was a sudden severe distortion of the cladding with the fixed crests flattening by the valleys moving up to the level of the screw heads, as described during the deflection load trials in Section 4.1.2. Buckling of cladding (creases) formed adjacent to the screw heads with cracks first observed seven minutes later. The X-Y movement of the screw head is shown in Figure 6.17. The formation of the creases can be seen by the jump at 75 minutes.

![Figure 6.16: Applied pressure trace showing failure at -6.6 kPa at 123 minutes](image)

![Figure 6.17: Movement at screw head showing jump at 75 minutes](image)
The cladding failed by pulling over the screws at approximately 123 minutes at a pressure of -6.6 kPa, as shown in Figure 6.18 (a) and (b). The ‘H’ and ‘T’ crack pattern shapes were similar to failures of cladding as documented by Beck (1975) and shown in Figure 6.19 (a) and (b).

The cladding had resisted a pressure of -8.3 kPa nine minutes prior to the failure at -6.6 kPa highlighting the nature of fatigue damage accumulation (crack growth) in that failure during wind loading is not necessarily associated with the largest peak but is also a function of load history (duration) just as it is in cyclic load trials.

![Figure 6.18: Roofing system “failed” by cladding cracking and pulling over head of screw (PLA test W6)](image1)

(a) ‘H’ crack pattern  
(b) ‘T’ crack pattern

**Figure 6.18:** Roofing system “failed” by cladding cracking and pulling over head of screw (PLA test W6)

![Figure 6.19: Crack patterns from corrugated cladding after Cyclone Tracy (Beck 1975)](image2)

(a) ‘H’ crack pattern  
(b) ‘T’ crack pattern

**Figure 6.19:** Crack patterns from corrugated cladding after Cyclone Tracy (Beck 1975)
6.6 Use of a Damage Index to compare traces

The response of the cladding (deformation and cracking) to the applied load levels and calculated reaction coefficients were similar during cyclic loading and simulated wind loading, as detailed in Chapters 4 and 6. Similar crack initiation and failure patterns were observed in the cyclic load trials, the simulated wind load trials and from damage investigations following cyclones. The power spectrum showed there was neither observed resonance of the cladding nor any significant loss of response from the load on the cladding to the fastener reaction for frequencies less than 5 Hz for cladding pre and post buckling.

Noting the similarities of the cladding and fastener response between cyclic loading and simulated wind loading, a simple method of comparing cladding performance under various wind loading traces was employed, incorporating data from the large amount of cyclic loading trials ($S_{max}$-$N$ data).

As described in Section 2.2.3, Miner’s rule is often used to predict the performance of metal components subjected to repeated loading. However, for corrugated cladding, the different modes of crack initiation and propagation indicate a different fatigue response depending on the load level. Miner’s rule relies on constant material properties and does not satisfactorily deal with this situation of changing profile shape, strength and stiffness (Beck and Stevens 1979; Mahendran 1993; Xu and Reardon 1993). As detailed in Section 2.4, a modified Miner’s rule was suggested that used different empirical constants depending on the load level. However, this method incorporated only $R=0$ cycles and did not predict fatigue damage for cladding from cycle histories, especially when lower load cycles followed higher load level cycles.

Holmes (2002) used a closed form solution to derive cumulative damage and fatigue life, for along wind turbulence-induced (high-cycle) fatigue damage of slender structures. In using a log-log S-N curve with $R=0$ for cladding, Xu (1995b) used the Goodman method to give an equivalent range but with a zero mean stress level to enable the summation of various cycles with different means and ranges. For complex loading histories with varying $R$ values, a simple modified version of Miner’s rule, described in Madayag (1969), is given in Equation 6.4, where $DI$ is the damage index, and $K$ is an empirically derived parameter for when failure occurs. Xu (1995b) suggested a value of 0.25 for $K$ based on some cyclic tests on corrugated cladding. He noted however that extensive random fatigue loading tests were required to satisfactorily establish the value of $K$.

$$ DI = \sum_i (n_i/N_i) < K $$

(6.4)
Wind loads on roof components have wide-band characteristics with a broad range of frequencies (Davenport 1961; Ginger 2001), making it difficult to determine what is a load cycle. As discussed in Section 2.5.4, the “rainflow method” as described by Amzallag et al. (1994) and by Xu (1993) has been used to determine an equivalent number of cycles, $n_i$, for a load level, $S_{\text{mean}}$, and range, $\Delta S$, for a given pressure trace.

From the rainflow analysis the cycles are presented in terms of means and ranges of the loads, from which the maximum, minimum and $R$ values can be derived. The DI, as per Equation 6.4, requires the ratio of $n_i / N_i$, where $n_i$ is calculated from a rainflow analysis and $N_i$ is extracted from the laboratory testing. A surface plot of the $S_{\text{max}}$-$N$ data with increasing $R$ values is shown in Figure 6.20. The surface plot is a representation of the experimental data presented in Figure 4.43. The surface is interpolated for $N_i$ for each cell in the rainflow matrix defined by its $S_{\text{max}}$ and $R$. The assumptions in using the surface plot to determine $N_i$ are discussed in Section 6.6.1. For pressure traces that crossed from negative to positive pressure, the positive pressure values were clipped to zero prior to determining the cycle’s mean and range using the rainflow method, as discussed in Section 2.5.5 as the positive pressures do not contribute to the fatigue loading of the cladding under the screw head.

![Figure 6.20: $S_{\text{max}}$-$N$ for corrugated cladding screw fixed at alternate crests](image-url)
The matrix of cycle counts of external and net pressures obtained from the rainflow method for roof area A, for building 5-5 orientations 25 and 75 are given in Table 6.3 and Table 6.4, respectively. Each cell from the table of rainflow cycle counts gives the number of cycles that have a similar range and mean. The Range and Mean, both in kPa, describe the range ($\Delta P$) and mean ($P_{mean}$) of the cycles. Approximately 68,400 and 72,700 cycles were obtained for the Net pressure cases for orientations 25 and 75 with 93% and 87% of these pressure cycles within the low 10% band of the peak pressure. Jancauskas et al. (1994) obtained a matrix containing 75,000 cycles, with approximately 90% of their cycles within the 10% band.

### Table 6.3: Numbers of cycles for external and net pressures (kPa) for building 5-5 orientation 25

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Table 6.4: Numbers of cycles for external and net pressures (kPa) for building 5-5 orientation 75

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The shaded cells within Table 6.3 and Table 6.4 represent the range and mean values where the cycles can have an approximate load ratio, \( R = 0 \), (i.e., when the mean is equal to half the range). The numbers of cycles applied in a L-H-L test, for a pressure of -4.8 kPa, derived in Section 3.2, are shown with brackets in Table 6.3(a). The L-H-L method contained more than 10,000 cycles in the middle range compared to a few hundred with the “design” cyclone for building orientation 25. With regard to the net pressure cycles, the AS/NZS1170.2:2002 (Standards Australia, 2002) calculated internal pressure (1.86 kPa) was added to give a design net pressure of -6.64 kPa. Although its peak value is similar to that derived from the net wind tunnel data, the L-H-L cycles greatly outnumbered the mid to high range wind tunnel load cycles as shown in Table 6.3(b).

The damage index (DI), as defined by Equation 6.4, was calculated using the \( R \) and \( S_{max} \) determined from the rainflow matrix means and ranges with the corresponding \( N_i \) values interpolated from the test data in Figure 6.20.
The rainflow matrix for building orientation 25 had a DI of 0.18 and 0.25 for the external and net cases while orientation 75 had a DI of 0.08 and 0.24. The two net cases had a similar DI even though the peak pressure for orientation 25 was nearly 2 kPa greater. However, orientation 75 contained a higher percentage of net cycles in the mid-pressure range increasing its DI to match that of orientation 25, as reflected in the broader shape of Figure 6.9 when compared with Figure 6.8. It can also be seen in comparing the rainflow matrices that orientation 75 has a slight shift towards cycles with lower $R$ than that of orientation 25. A high $R$ results in less accumulated damage than a lower $R$ for the same peak pressure.

6.6.1 Limitations and assumptions of the DI

The rainflow method aggregates load cycles based on the magnitude and mean but ignores the sequence of loads, as discussed in Chapter 2. Equation 6.4, which is an application of Miner’s rule with the rainflow cycle counts, also does not consider the load history when calculating the DI.

Therefore the Damage Index is employed as an indicator of damage only. In no way is it suggested that the DI be used as a substitute for physical testing, but only as a litmus test for comparing the damage potential of wind traces that have the similar form of increasing then decreasing wind loads.

The Damage Index from a simulated wind load test, labelled W5, of Building 5-5 orientation 25 was 0.24. Xu (1995b) suggested a DI of 0.25 as the criterion for failure. Damage indexes derived for other simulated wind load tests are given in Table 6.5.

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<td>-</td>
<td>Pass</td>
<td>0.25</td>
<td>Building 5-5 orientation 25 design cyclone</td>
</tr>
<tr>
<td>W4</td>
<td>7.2</td>
<td>7.2</td>
<td>Fail</td>
<td>0.28</td>
<td>Building 5-5 orientation 25 but no reduction in mean speed with direction change</td>
</tr>
<tr>
<td>W3</td>
<td>8.0</td>
<td>-</td>
<td>Pass</td>
<td>0.32</td>
<td>Building 6-5 orientation 50 net</td>
</tr>
<tr>
<td>W2</td>
<td>8.0</td>
<td>8.0</td>
<td>Fail</td>
<td>0.28</td>
<td>Building 6-5 orientation 50 net</td>
</tr>
<tr>
<td>W6</td>
<td>8.3</td>
<td>10.6</td>
<td>Fail</td>
<td>0.33</td>
<td>Building 5-5 orientation 25 design cyclone GF=1.55 50m/s</td>
</tr>
<tr>
<td>W7</td>
<td>5.6</td>
<td>5.6</td>
<td>-</td>
<td>0.03</td>
<td>Repeating 15 minute portion of trace for crack initiation</td>
</tr>
</tbody>
</table>
Test W2, had accumulated a DI of 0.28 at failure, which was 3 minutes past the -8 kPa peak at 116 minutes, and 45 minutes past the second highest peak of -7.4 kPa. The cladding had failed by pulling over screw S2 in a classic ‘H’ type crack. The cladding at the other screw locations, S3 to S5, was severely cracked but not to the same extent as S2, since the cracks that radiate from the screw hole to “complete” the horizontal bar of the ‘H’ were not observed. Photos of the cladding at the completion of the test can be seen in Appendix A.

Test W3 with a DI of 0.32, was a repeat of test W2, but successfully completed its trace. The crack patterns and extents were similar to the S3 to S5 screws of test W2. As per the variability of crack growth demonstrated by the cyclic load tests, it is to be expected that there will be a spread of DI values. For the purposes of this study, DI values larger than 0.25 will be considered as a highly likely failure criterion with pressure traces able to be compared providing the traces have the shape of increasing then decreasing and the trace contains at least several cycles of sufficient magnitude to initiate creasing for ‘H’ or ‘T’ crack development. For 900 mm span corrugated cladding 4 to 4.5 kPa would be sufficient to cause the crease cracks.

The $S_{\text{max}} - N_i$ data was used to derive the DI values as opposed to the $P_{\text{max}} - N_i$ data. This option was taken as the available data more than doubled with the inclusion of $S_{\text{max}} - N_i$ data from the previous line load studies by Mahendran (1989) and Xu (1995b). As demonstrated in Section 4.5 the overall failure modes and cycles to failure were similar within the test result variability to that of the tests loaded by the PLA with direct air pressure. The additional data points enabled the 3D surface to include a much broader range of R values, greatly improving the interpolation process.

Approximations of the reaction coefficients derived from testing were used to convert pressure to reactions at the fasteners. Typically, the RC was set at 1.20 prior to the first major peak that would induce creasing in the crest. The reaction coefficient would then be reduced with duration to a value of typically 1.10.

The other advantage of basing the DI on a load per fastener metric was that, by reducing the tributary area for pressure traces with peaks well beyond the capacity of a 900 mm cladding span, the loads per cycle could be reduced thereby reducing the DI. The modification of the tributary area could be used to give an estimate of an acceptable batten or screw spacing for use for comparing different building configurations and wind parameter scenarios.
6.6.2 Crack initiation estimate using DI

Section 4.3.1 describes the crack initiation for both the crease type cracks ‘H’ and ‘T’ and the star cracks. For crack initiation, an averaged DI value of 0.05 for the crease cracks and 0.09 for the star cracks was estimated, based on the cyclic load tests.

The simulated “design” cyclone wind loading test, W5, described in Section 6.5.1, had crack initiation observed at 107 minutes, six minutes after the first creases formed. From the cyclic loading ‘H’ crack trials, shown in Figure 4.18, crack initiation was typically observed within 30 cycles, that is, less than 20 to 30 seconds, for pressures above 4.0 kPa. For the wind loading trace, during those six minutes from first creasing to observed crack initiation, there were eleven pressure up-crossings exceeding 4.0 kPa, three exceeding 4.5 kPa, two exceeding 5.0 kPa and one just passing 5.5 kPa. The DI for the first 107 minutes of the W5 trace was calculated to be 0.037.

The DI values for crack initiation for tests W4, W6 and W7 were 0.057, 0.052 and 0.032, respectively. The cracks formed at crease points away from the screw, with test W4 taking up to 21 minutes for cracks to be first observed from the initial crease forming following a pressure peak of 4.4 kPa.

It is proposed that a DI of 0.05 could be used for estimating crack initiation for a given pressure trace provided the trace has a pressure peak of sufficient magnitude to form a crease away from the fastener, that is ‘H’ or ‘T’ type crack formation is assumed.

6.7 Variations in peak pressures and load cycle distributions

The “design” cyclone was based on the assumptions of the maximum wind speeds impacting a single storey house model at a quartering wind angle producing the peak suction pressures at the gable end. The original L-H-L cycle distributions were also derived on similar assumptions (Mahendran 1995).

6.7.1 Comparison of Open and Suburban terrain

The NIST wind tunnel time series database contained data on building models for both open and suburban exposures. The exposures were defined as having roughness length of 0.03 m and 0.3 m, respectively. A gust factor of 1.7 was applied to the time series data for the same building model, Building 5-5, but modelled in a suburban boundary layer profile.
Table 6.6 gives a summary of the peak external and net pressures derived from the time series data for Building 5-5 in both open and suburban terrain while Table 6.7 gives the pressures for Building 5-14° (14 roof pitch). Gust factors of 1.4 and 1.7 were used for open and suburban terrain, respectively. For example, the suburban trace used a mean wind speed profile which was derived for the “design” cyclone by modifying the 70 m/s design wind speed at 10 m high by a value of $M_{z,cat}$ of 0.89 from AS1170.2:2002, resulting in the design wind speed of 62 m/s. Applying the 1.7 gust factor gave a mean speed of 36 m/s at 10 m height. The mean pressures, peak external suctions and peak net suctions are plotted in Appendix C, for both terrains and for two roof slopes. The tables also provide the ratios of the wind tunnel derived pressures divided by the AS1170.2:2002 design pressures, including the $K_L$ of 3 from the proposed draft standard.

**Table 6.6: Building 5-5 external and net pressures for open and suburban terrain**

<table>
<thead>
<tr>
<th>Area</th>
<th>Pressure (kPa)</th>
<th>Ratio of Wind tunnel over Design pressure</th>
<th>Pressure (kPa)</th>
<th>Ratio of Wind tunnel over Design pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>A ext</td>
<td>-7.70</td>
<td>1.65 (1.10 if $K_L$ of 3)</td>
<td>-5.95</td>
<td>1.75 (1.16 if $K_L$ of 3)</td>
</tr>
<tr>
<td>A net</td>
<td>-8.61</td>
<td>1.35 (0.97 if $K_L$ of 3)</td>
<td>-6.49</td>
<td>1.38 (1.01 if $K_L$ of 3)</td>
</tr>
<tr>
<td>B ext</td>
<td>-5.34</td>
<td>1.14</td>
<td>-3.31</td>
<td>0.97</td>
</tr>
<tr>
<td>B net</td>
<td>-6.50</td>
<td>1.00</td>
<td>-4.43</td>
<td>0.94</td>
</tr>
<tr>
<td>C ext</td>
<td>-1.53</td>
<td>1.96</td>
<td>-1.14</td>
<td>1.99</td>
</tr>
<tr>
<td>C net</td>
<td>-3.96</td>
<td>1.52</td>
<td>-3.36</td>
<td>1.78</td>
</tr>
<tr>
<td>Wall</td>
<td>+3.35</td>
<td>1.47</td>
<td>+3.03</td>
<td>1.83</td>
</tr>
</tbody>
</table>

**Table 6.7: Building 5-14 external and net pressures for open and suburban terrain**

<table>
<thead>
<tr>
<th>Area</th>
<th>Pressure (kPa)</th>
<th>Ratio of Wind tunnel over Design pressure</th>
<th>Pressure (kPa)</th>
<th>Ratio of Wind tunnel over Design pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>A ext</td>
<td>-6.61</td>
<td>1.37</td>
<td>-4.48</td>
<td>1.27</td>
</tr>
<tr>
<td>A net</td>
<td>-7.99</td>
<td>1.19</td>
<td>-5.25</td>
<td>1.07</td>
</tr>
<tr>
<td>B ext</td>
<td>-2.81</td>
<td>1.05</td>
<td>-2.39</td>
<td>1.22</td>
</tr>
<tr>
<td>B net</td>
<td>-4.86</td>
<td>1.07</td>
<td>-3.99</td>
<td>1.20</td>
</tr>
<tr>
<td>C ext</td>
<td>-2.15</td>
<td>0.80</td>
<td>-1.54</td>
<td>0.79</td>
</tr>
<tr>
<td>C net</td>
<td>-4.44</td>
<td>0.97</td>
<td>-3.50</td>
<td>1.05</td>
</tr>
<tr>
<td>D ext</td>
<td>-3.69</td>
<td>1.38</td>
<td>-2.76</td>
<td>1.41</td>
</tr>
<tr>
<td>D net</td>
<td>-5.83</td>
<td>1.28</td>
<td>-4.52</td>
<td>1.36</td>
</tr>
<tr>
<td>Wall</td>
<td>+3.47</td>
<td>1.47</td>
<td>+2.65</td>
<td>1.55</td>
</tr>
</tbody>
</table>

The peak pressures from the open terrain were greater than those from the suburban terrain, as would be expected. However, AS1170.2:2002 typically underestimated the peak pressures for cladding. For the highly loaded gable end roof area A, the design pressures underestimated the wind tunnel results by more than 60%. The inclusion of a $K_L$ of 3 in the proposed draft wind...
loading standard reduces the underestimation to approximately 15%. The \( K_L \) of 3 is proposed for only the corner regions of low pitch roofs. For the 14° roof slope, AS1170.2:2002 underestimated the cladding pressures at the ridge.

The “design” cyclone pressure trace for Building 5-5, orientation 25, in suburban terrain with a dominant opening occurring on the long wall during the peak winds, is shown in Figure 6.21. It has a DI of 0.20. Pressures just exceeding -4 kPa occurred at approximately 75 minutes, with the peak suction of -6.5 kPa occurring at 108 minutes. The peak pressure coefficients derived from the suburban terrain model exceeded those of the open terrain, as detailed in Appendix C.

![Figure 6.21: Building 5-5, orientation 25, in suburban terrain with a dominant opening occurring on long wall at 135 minutes](image)

The shape of the pressure trace for the suburban terrain indicates large fluctuations. Table 6.8 shows these greater fluctuations with the rainflow analysis of the pressures revealing almost twice the number of \( R = 0 \) load cycles for the suburban trace when compared with the open terrain. Cycles with a load ratio \( R = 0 \) cause more damage than cycles with the same range but with an \( R > 0 \). The rainflow cycle count tables are given in Appendix C.
Table 6.8: Percentage of cycles with $R = 0$ within each 10% range column of the rainflow matrix

<table>
<thead>
<tr>
<th>Trace</th>
<th>Percentage of cycles with $R = 0$</th>
<th>Peak (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td>Open terrain (GF 1.4)</td>
<td>41</td>
<td>12</td>
</tr>
<tr>
<td>Suburban terrain (GF 1.7)</td>
<td>63</td>
<td>32</td>
</tr>
</tbody>
</table>

Based on the NIST database time series data, AS/NZS1170.2:2002, underestimates the cladding peak pressures. The underestimation of design pressures implies increased fatigue damage potential and an increased likelihood of the cladding fixings pulling out of their supports.

A possible solution to strengthen cladding systems on existing structures is the installation of screws at every corrugation in the highly loaded corner regions and near ridges at gable ends. The additional screws halve the spacing thereby reducing the tributary area. The DI value for the design cyclone with gust factor of 1.4 for cladding area A drops from above 1.0 to 0.05. The suggested solution does not involve removal of the roof to replace or add purlins or battens. However, an issue with reducing the screw spacing for hardwood battens is the potential for inducing a longitudinal split in the batten from the installation of the self drilling timber screws. The battens would have to be predrilled to reduce this possibility.

Another potential solution for the highly loaded roof areas is to replace the existing screws with screws fitted with cyclone washers, increasing the cladding capacity. However, the pullout capacity of the screw into the battens would need to be checked.

Any modifications to the cladding installation would of course require approval from the regulatory authorities that would most likely entail testing installation variations to the L-H-L. Advice would also need to be sought from the product manufacturers so that no warranty or service life agreements were breeched.

### 6.7.2 Roof slope and shape

The peak pressures for a design wind speed acting on cladding elements for the 5 m high building with gable roof slopes of 5° and 14° are given in Table 6.6 and Table 6.7, respectively. The roof
areas A, B, C and D are shown in Figure 6.5. The minimum, net and mean $C_p$s are plotted in Appendix C, for the two roof slopes.

The low pitch (5°) roof had the largest magnitude suctions which were at roof area A (gable end). It also had the largest peak suctions for roof area B and the smallest magnitude peak suctions for roof area C and D (i.e., middle of roof). The $C_p$s for the middle of the roof remained at a similar value for any direction. If the cladding has a balanced design based on a load per fastener for all roof areas, then the cladding in middle of roof is at a higher risk of failure as its “design wind load” can be exceeded from winds approaching from a greater range of directions than for fixings at a specific corner roof area.

For the medium pitch (14°) roof the largest peak pressures at the ridge (area D) were similar to the pressures at the roof edge in the middle of the building length (area B) which agreed with the results presented by Xu and Reardon (1998).

Xu and Reardon (1998) compared the pressure distributions from wind tunnel model studies of 15°, 20° and 30° hip roof pitch building geometries to similar size buildings with gable end roof geometries. They showed that the largest suction pressures for the 15° and 20° hip roofs were located behind the ridge line and of smaller magnitudes than the peak pressures on gable roofs. Whereas for the 30° roof, the largest suction pressure was at the roof corner and of similar magnitude, but different location, to a 30° gable roof.

Xu et al. (1996) showed that for a 20° hip roof, the accumulated load cycles were less than that of a 20° gable roof. They determined the location of highest fatigue damage was near the hip ridge intersection and was based on the worst wind direction for peaks. In determining accumulated load cycles and corresponding fatigue damage they only used the wind direction that gave the worst pressure for the selected roof location. They based this on the assumption that the worst damage occurred during the peak wind loads (Xu 1993). The method and assumption did not take into account any affect from pressures caused by dominant openings which are dependent on wind direction.

### 6.7.3 Placement of dominant opening

The generated pressure traces presented in Section 6.4, assumed a dominant opening occurred in the long wall. A repositioning of the dominant opening would result in a different pressure trace for the same roof location. The pressure trace shown in Figure 6.22 is for Building 5-5 orientation 25 with a dominant opening, represented by the wall pressure tap $W_G$, on the gable end. The
pressures were derived from using the “design” cyclone with a gust factor of 1.4. The opening now on the gable end is ‘open’ for the whole duration. The effect of having the gable end dominant opening ‘open’ for the whole trace is evident in the later part of the trace with the now negative internal pressures reducing the magnitude of the net roof suction pressures.

The -10.0 kPa net peak at 25° wind direction with the gable dominant opening exceeded the AS/NZS1170.2:2002 design pressure and was larger than the -8.6 kPa of the long wall dominant opening case. For the wind directions centred on the critical 25°, more of the peak wall pressures coincided with the peak roof suctions than the long wall dominant opening case. That is, there was a higher correlation of the windward wall (WG) peaks with area A pressure taps.

The rainfall cycle counts are given in Appendix C. The number of cycles and percentages of cycles in the means and ranges was similar overall to the other traces except there were a few more peak pressure cycles than when the opening was on the long wall. Therefore the case for the pressures generated with the gable dominant opening is more damaging than when the dominant opening was on the long wall.

Figure 6.22: Pressure trace in open terrain for Building 5-5 orientation 25 with a dominant opening in gable end wall for whole trace
6.7.4 Comparison of “design” cyclone to cyclone events

Tropical Cyclone Tracy impacted the city of Darwin on 25 December 1974. The cyclone had a small diameter but was a very intense, slow moving system that produced maximum wind gust velocities in the range of 65 to 70 m/s with the cyclone passing directly over Darwin. Appendix D details Cyclone Tracy’s path, estimated wind speeds for various suburbs and the SEA wind field model used to derive the mean wind speeds and change in wind direction for various locations across Darwin.

Estimates of mean wind speed (referenced to 10 m height in open terrain) and direction are plotted in Figure 6.23, for the locales of Alawa and Winnellie. Alawa and Winnellie were chosen based on wind speed, change in wind direction and available descriptions of building damage. The model wind trace for Alawa shows the eye wall skirting the locale as opposed to the eye passing over Winnellie. Therefore the wind speed trace for Alawa does not rapidly drop to and rise from near zero, but neither does it have a rapid change in wind direction such as that occurred at Winnellie.

![Figure 6.23: SEA modelled and “design” mean wind speed and Wind direction](image)

The differences between the Cyclone Tracy modelled traces compared to the “design” cyclone are the amount and rate of change in wind speed and direction. The greater angle change is because Cyclone Tracy had a smaller radius to maximum winds than the parameters chosen for the “design” cyclone. The larger mean wind speeds occur earlier and drop off quicker than the “design” trace. A less obvious difference is that for the first hour of the Cyclone Tracy traces, the wind direction had minimal change. This is over the same period when the wind speeds are rapidly increasing, thus sustaining the biggest wind speeds for longer, leading to the possibility that some building elements may be subjected to a higher level of accumulated damage than the “design” trace.
As per Section 6.4 the building orientation was aligned such that the maximum winds occurred in
direction with the peak $C_{pe}$ for the area A at the gable end, with a dominant opening in the long
wall, occurring 60 minutes into the trace. The calculated pressure trace for Alawa is shown in
Figure 6.24. Due to the nearly 180° wind direction change, the pressure trace drops off markedly
at approximately 140 minutes since cladding area A now becomes the downwind part of the roof
and therefore not subjected to the large suction pressures associated with the windward edge. The
trace for Winnellie is in Appendix D.

![Figure 6.24: Pressure trace for bo1 in Alawa](image.png)

The damage index (DI) values for the generated pressure traces for Building 5-5 orientation 25
with a dominant opening in the long wall was 0.29 for the Alawa trace and 0.14 for the Winnellie
trace. The rainflow counts are given in Appendix D.

The large peak pressures of the Alawa trace show that 900 mm span corrugated cladding when
fixed at alternate crests would have suffered severe cracking with possible failure. Damage
inspections of cladding and building systems were conducted by Beck (1975). Beck detailed
failure and loss of cladding to the gable end regions of low roof pitch housing in the Alawa
suburb. The cladding was fixed typically at every second crest at the ends of cladding lengths, but
used a wider screw spacing of every third or fourth crest for fixings to intermediate battens.
Increasing the spacing of the screws leads to a DI in excess of 1.0 for a Building 5-5 geometry.
Beck (1975) detailed cladding failures in the first few sheets at the gable ends of low roof pitch metal clad industrial sheds in the Winnellie suburb. The cladding was fixed with only the three screws at intermediate purlins as was the practice at that time. Beck noted relatively undamaged sheds near either side of a damaged shed. Halpern Glick Pty Ltd (1975) described the good performance of cladding on sheds with substantial ridge ventilators. They noted however, that the cladding was fixed at alternate crests at all purlins, and was 0.60 mm thick. The ridge ventilators would greatly reduce any large internal positive pressure arising from a windward dominant opening. The generated wind trace with building 5-5 for the “external pressure only” scenario for Winnellie resulted in the DI of 0.07. This suggests survival of the 0.42 mm thick G550 cladding in the highly loaded gable region of a building 5-5 geometry, but that crack initiation might have occurred.

In 1999, Cyclone Vance hit Exmouth in Western Australia, with wind speeds up to 70m/s, similar to those estimated from Cyclone Tracy. The maximum winds from Cyclone Vance did not pass over Exmouth, as its track was to the east of the town. Section 6.1 details the track and wind speed record. The 10 minute mean wind speeds at 10 m height, recorded by the automatic weather station (AWS) three cup anemometer, and direction are plotted in Figure 6.25. The peak gust wind speed measured by the AWS was 65 m/s. The form of the mean wind speed varying with time and the change in wind direction is similar to the “design” cyclone parameters proposed by Jancauskas et al. (1994) with both cyclones having a wide diameter as opposed to Cyclone Tracy.

![Figure 6.25: Mean wind speed and direction from AWS for Exmouth (Learmonth)](image)

The generated pressure trace based on the AWS wind speeds and direction for building 5-5 orientation 25 geometry with a dominant opening in the long wall, is shown in Appendix D. The peak pressure was -6.7 kPa, with a corresponding DI of 0.21. Detailed damage investigations of light gauge metal cladding observed minimal sign of fatigue failure of cladding (Reardon et al.)
The lack of fatigue damage was attributed in part to the fast forward motion of the cyclone (short duration) but more to the fact that Exmouth is located in cyclonic Region D, with a corresponding ultimate limit state design wind speed ($V_{R,500}$) of 85 m/s. Correspondingly, cyclone washers were used in the installation of cladding to increase the design capacity.

### 6.8 Comparison of pressure traces with L-H-L

The specimens subjected to L-H-L tests shown in Figure 6.15 (a) and (b) had a calculated DI of 0.30 at failure with a peak of 4.8 kPa and a DI of 0.13 at successful completion with a peak of 4.08 kPa, respectively. With this 15% reduction in the test load, the DI dropped dramatically, since the higher load cycle blocks were now just below local plastic deformation (the knee in the $S_{max}$-$N_i$ curve) giving a smaller $n_i/N_i$ value. The two load blocks of 4500 cycles with their $P_{max}$ of 0.45% of the test pressure, contribute significantly to the total DI. If the 4.8 kPa L-H-L test had successfully completed the test, its DI would have been 0.39 with the 9000 load cycles portion adding 0.17. The DI contribution for the design cyclone for building 5-5 orientation 25 for all the cells with a cycle range less than or equal to 45% of the peak is only 0.12, even with its 7.1 kPa peak.

The L-H-L as described in the Building Code of Australia (BCA 2007) has been modified from that originally proposed (Mahendran 1995). It now incorporates a test factor $K_t$. $K_t$ is a factor applied to the design pressure and is to account for variability of a product in its manufacture and assembly when only a few specimens are tested. The $K_t$ factor is taken from AS/NZS1170.0:2002 (Standards Australia 2002a) a 10% coefficient of variation of ultimate strengths of a parent population is commonly assumed by industry (Standards Australia 1992). $K_t$ ranges from 1.46 for one specimen tested to 1.21 for ten replications.

The L-H-L test is shown to be conservative if setting the L-H-L test pressure as per the “design” cyclone’s peak pressure. The application of the $K_t$ factor reduced the allowable design pressure thereby increasing the conservativeness of the L-H-L.

Figure 6.26 shows the number of cycles from the rainflow cycle counts as a percentage of the peak pressure for various generated cyclone traces and the L-H-L.
Figure 6.27 shows, for several traces, the number of cycles as a percentage of the number of cycles contained in the top 90% of the peak pressure, that is, from 10 to 100% of $P_{\text{max}}$. The 10% bands of the peak pressure represent the ten columns in the rainflow matrix. Each trace represents the numbers of cycles for various parameters such as different cladding areas (A, B and D), 5° and 14° roof slopes, different dominant opening locations, and different mean wind speed and wind angle parameters. All of the cyclone traces follow the same trend. However, the L-H-L has a greater percentage of cycles as a percentage of peak pressure when compared to these traces. To compound the issue, all the L-H-L cycles have a load ratio of $R=0$, which have lower numbers of cycles to failure than cycles with the same peak but an $R>0$. Figure 6.27 therefore shows that the L-H-L cycles over represent the numbers of cycles for a given load range, except for the 100% band where the generated traces may have two or three cycles approaching the peak pressure as opposed to the L-H-Ls one.

Figure 6.26: Number of cycles from rainflow analysis for generated traces
Figure 6.27: Comparison of cycle numbers from rainflow analysis with peak pressure for generated traces and L-H-L

Although the seven generated traces in Figure 6.27 used different combinations of building geometry, terrain, wind angle and speed, they were based on the “design” cyclone’s five hour period, which was the same time period used to formulate the L-H-L test (Jancauskas et al. 1994).

With the proposed AS/NZS1170.2:2002 increase in design pressures in the corner regions of a roof ($K_L=3$), the L-H-L will be conservative. Therefore a reassessment of the L-H-L test regime would be required. A revised L-H-L test regime is suggested by reducing the current L-H-L test loads by 30% that is, lowering the L-H-L curve in Figure 6.27 to that of median of the other curves. This revised load regime, shown in Figure 6.28, represents the loading patterns generated from cyclone traces. The postulated curve falls in the middle of the region inscribed by the cyclone traces. Two additional cycles at 100% of the peak load were also added, in line with the majority of the rainflow counts from the cyclone traces. The DI for this example, using a peak load of 7.1 kPa is 0.24. The DI for the PLA trial W5, with is peak of 7.1 kPa, was 0.25.
To determine a representative range of pressure cycles that is commensurate with a probability of occurrence for the design wind speeds, an evaluation of the risk (vulnerability assessment) would be required. That is, a Monte Carlo simulation or similar process would be required to determine a damage index for cladding areas on multiple building configurations with various dominant opening scenarios to be combined with cyclone wind field models using parameters from either historical best track data cyclone data bases or simulated cyclone track scenarios.
7. Conclusions and recommendations

The building envelope is subjected to large, fluctuating peak pressures during wind storms. Damage investigations continue to show failures of metal roof cladding as a result of wind loads exceeding its strength capacity. In cyclonic events the cladding can be susceptible to low cycle fatigue failures resulting in cladding being sucked from the buildings, diminishing the building’s integrity and adding dangerous elements to the wind driven debris field.

The thesis was conducted to analyse the performance of light gauge but high yield strength profiled metal cladding subjected to cyclonic wind loads. The experimental methods involved tensile coupon tests and static point load tests as well as static pressure, cyclic pressure and dynamic wind pressure tests on double 900 mm span 0.42 mm BMT G550 corrugated cladding specimens. The air pressure tests were carried out by using a Pressure Loading Actuator that was able to apply a positive and negative (suction) pressure via an air-chamber.

The primary findings of the thesis were:

- The extensive cyclic load tests conducted through the 1980s and early 90s using the profile confining, reduced span line load test rigs, forms the basis of current Australian building regulations for cladding in cyclonic areas. The thesis has demonstrated that the corrugated cladding response to the sinusoidal line load test data is similar to uniform cyclic pressures through the numbers of cycles to failure for similar load levels, and through the similar crack pattern formations. The uniform pressure (PLA) trials were also able to establish that:
  - The peak cycle load and amplitude govern crack initiation and growth not the cycle rate and cycle shape.
  - The numbers of cycles to failure for various load ranges matches the line load $S_{\text{max}}$-N curve within the variability range for these trials.
  - The air-chamber tests confirmed the occurrence of the star crack patterns for the pressures below local plastic deformation and the ‘H’ and ‘T’ crease type crack patterns for pressures causing buckling of the crests. However, a slight bias in increased crack length/cladding damage was observed for crests adjacent to laps. The line load tests could not incorporate a full width sheet.
  - The typical variation of the G550’s yield strength did not have a discernable influence on number of cycles to failure within the variability for repeat cyclic load tests.
The variability in the number of cycles to failure for repeat cyclic load tests was similar to the values for the line load tests.

The EPDM seal under the screw head promotes crack growth for cracks originating at the screw hole.

The relationship between the applied pressure and the reaction at a screw, the “reaction coefficient”, was not constant but varies depending on the deformation and cracking of the cladding, over the duration of the test as well as during each cycle. Therefore cladding design tables based on a simple beam theory coefficient do not reflect the real world process and can overestimate the reaction at the screw.

The lateral loads at the screw head could be a significant proportion (from 25% to 50%) of the vertical reaction at the screw during various scenarios, such as: the initial buckling of the cladding crest at the screw, unequal crack growth and with failure of an adjacent fastener.

There was potential for cladding screws fixing into Z purlins to unscrew during the load cycles.

- The application of fluctuating pressures representing wind loads, enabled the assessment of cladding response to these dynamic pressures and a comparison to cyclic load tests. Cladding crack patterns generated with the simulated wind trace from the PLA were similar in shape and length to crack patterns reported in damage surveys and the loads at which the claddings deformed and buckled, were of similar magnitude to those in the cyclic load tests. The application of the simulated wind loads also showed that:
  - The reaction coefficient varied with cladding damage as per the cyclic load tests.
  - The cladding specimens responded in a quasi-static manner. That is, there was no resonance in the cladding from the fluctuating pressures.
  - From monitoring the reaction at a screw, the screw was receiving the pressure fluctuations up to a frequency of approximately 5 Hz, demonstrating that the cladding transfers the wind fluctuations to the screws and, therefore, further justifying the application of cyclic load tests for cladding fixings and supports (roof battens).
  - A Damage Index metric based on $S_{\text{max}} \cdot N$ cyclic test data could be used as an indicator of extensive cladding damage and possible cladding failure for generated cyclonic traces.
  - The same DI could also be used to indicate crack initiation for the ‘H’ and ‘T’ crease type cracks.

- Wind tunnel test data and full scale measurements show the wind pressure is highly turbulent and temporally and spatially varying across the building envelope. Yet the line
load testing and typical product testing applies the “same load” to all fasteners in the test specimen. The thesis has shown by measuring fastener reactions to point loads applied at various points across the cladding, that there is minimal influence on the load in one screwed crest to an adjacent screwed crest, justifying the assumption of applying a uniform load across the test specimen. The analysis also showed that:

- Point load testing verified that the cladding fastener’s tributary area was elongated along the crest and based on screw spacing and batten spacing.
- The load sharing from one screwed crest to an adjacent screwed crest was less than 5%.
- The time series data from a high density tapped wind tunnel model showed for cladding near the gable edge, the small elongated cladding tributary area could be satisfactorily represented by a single pressure tap centred on the screw location, for both peak pressure and load cycles.
- The steep pressure gradient across the conical vortices, relative to the spacing of the cladding fasteners, results in a significant reduction in the peak pressures and, thus, the potential for fatigue damage reduces within the spacing of three to four screws from the gable end for a low pitch roof.

- The L-H-L fatigue loading test criteria was derived from extensive line load cyclic tests, a nominated cyclone trace and pressure measurements from the gable end of a single storey wind tunnel model that did not incorporate the internal pressure resulting from a dominant opening. This thesis has demonstrated that the L-H-L is a conservative test when compared to generated cyclone traces with the same peak pressures as it over represents the numbers of cycles at the median to high load cycles. The thesis also determined that:

  - Peak pressures for cladding loads determined from the two building configuration’s wind tunnel studies typically exceeded AS/NZS1170.2:2002. This is further exacerbated by wind tunnel studies potentially underestimating the peak pressures in the corner regions by 20 to 50% due to scaling issues associated with Reynolds number.
  - If AS/NZS1170.2:2002 is used to determine the design cladding pressures for a L-H-L test, the conservatism of the test is reduced.
  - The general roof area (away from corners, edges and ridges) is subjected to lower magnitude pressures relative to edges. In the central area of a low pitch roof, the pressure is sustained for winds approaching from most directions. Hence, forces close to design values are felt for a proportionally greater number of cycles than at the edges.
  - If a gable end opening located towards the corner, where the highest winds will come from, is created at the start of the cyclone, the pressures and load cycles
generated over the five hour duration give the worst case for design for the roof cladding at the gable end for the various configurations trialled.
- The proposed changes to AS/NZS1170.2:2002 with the inclusion of an increased local pressure coefficient $K_L=3$ for corner regions produces a closer match to the wind tunnel peak pressures determined in this study. However, the peak pressures for cladding fasteners near the ridge (~1 m) for the 15° roof pitch are still underestimated.
- With the inclusion of different dominant opening scenarios, aligning the maximum mean winds with the wind direction that gives the largest magnitude peak suction pressure coefficient may not give the worst fatigue loading pressure trace.

### 7.1 Recommendations for further research

#### Influence coefficients

The point load testing for determining influence coefficients was conducted on undamaged cladding. For the damage index calculations, a reduction in stiffness was assumed, based on the lower reaction coefficient, for cladding subjected to loads larger than its local plastic deformation (LPD). To better quantify the load distribution of pressure peaks across a cladding specimen that has suffered damage, it is suggested that additional point load testing, detailed in Chapter 5, be performed on cladding that has been preloaded to develop creasing and the formation of cracks. This could be performed by installing the cladding specimen in the air-chamber and applying cyclic pressures to initiate crack growth. The cladding samples would then be removed and installed into the point load test rig and tested to confirm the assumptions of the reduced reaction coefficient used in the Damage Index calculations.

#### Hip roofs

Load cycles for generated cyclone traces for various hip roof configurations are required to compare to the L-H-L cycle distribution. Wind tunnel time series data for pressure taps representative of cladding fasteners are required to investigate the damage index for cladding for different parts of the roof for different dominant opening scenarios for changing wind direction and speed.

#### Variability of numbers of cycles to failure

It is proposed that 10 or more cyclic load trials be conducted for each combination of means and ranges, to more reliably determine factors associated with the variability of numbers of cycles to
failure. For example, some of the trials would include the distortional effects of the screw’s seal while other trials would use a dummy seal machined from PVC such that it would not distort during the testing.

**Reduction in number of cycles towards end of test**

It has been noted there was minimal observed crack growth during the final 60 minutes for cladding specimens that completed the simulated cyclonic wind load trials (i.e., passed). Additional testing of cladding specimens could be undertaken to assess the possibility of reducing the number of cycles in the final load cycle block of the L-H-L. Care should be exercised, however, in making recommendations on cyclic load test criteria based on one profile or material. The fact that corrugated G550 cladding may not exhibit signs of damage accrual towards the end of the design cyclone does not mean that other profiles or materials would do the same.

### 7.2 Recommendations for building design

Based on the conclusions, six recommendations are proposed in the areas of improving building envelope resilience, economy and test protocols.

**Additional screws in highly loaded corners and gable ends**

The current Australian wind loading standard AS/NZS1170.2:2002 underestimates the peak pressures applicable for cladding design for the defined regional design wind speeds. It has been demonstrated from analysing wind tunnel data, generating cyclone traces and testing cladding to dynamic wind loads that, for the “design” cyclone parameters for either open or suburban terrain for the specified building configurations, the cladding in the highly loaded areas would be at a higher risk of failure if tested to the previous AS4040 test regime than the current L-H-L. This assumes that the cladding has been installed to the limits of the product manufacturers design tables (i.e., no excess capacity) and that the highest wind speed aligns with the worst wind direction for the geometry. It is recommended that additional screws be installed into the unfastened crests to reduce the tributary area or, conversely, the existing screws are replaced with fixings with cyclone washers. If installing additional self drilling screws into hardwood battens, care will be needed to ensure longitudinal splitting of the timber is minimised. Pre-drilling would be suggested.
**Cladding screw pullout from battens and purlins**

Batten and purlin manufacturers’ load span design tables should be assessed to ensure that the peak cladding pressures, that are underestimated in AS/NZS1170.2:2002, do not overload the pull out capacities of the cladding screws.

**Reduced number of screws in central area of roof**

In assuming the typical construction practice of keeping the same purlin or batten spacing and numbers along the length of the building (i.e., consistent batten run) and that design already incorporates pressures from a dominant opening occurring, an increase in screw spacing from every second crest to every third or fourth (as per non-cyclonic regions) may be possible due to the low design pressures in these roof areas. Before implementing this, manufacturer and regulatory approval would be needed along with evaluating the cladding with its reduced number of fasteners to current cyclic load criteria. The design and inspection process to ensure that the reduced numbers of fixings are only in the mid regions of the roof may make this impracticable.

**Laps in test specimens**

The research program has demonstrated typically greater crack lengths for crests near the top laps, as opposed to those away from the lap. Therefore, it is suggested that all commercial tests incorporate a lap in the test specimen.

**Lateral loads on cladding screws**

For the commercial testing of cladding to batten fixings, tensile cyclic pullout tests on a single screw should not be allowed unless it can be demonstrated that the lateral loads and movements imparted from the cladding to the screw head do not reduce the screw’s tensile holding capacity in the batten.

**Reducing the conservatism in the L-H-L**

With the proposed increase in design pressures in the corner regions of a roof, the L-H-L will be more conservative for edge regions. To determine a representative range of pressure cycles, an evaluation of the risk (vulnerability assessment) would be required. That is, a Monte Carlo simulation or similar process would be required to determine a damage index for cladding areas on multiple building configurations with various dominant opening scenarios and be combined with cyclone wind field models using parameters, for example, from simulated cyclonic events with a defined probability of occurrence developed for storm tide modelling.
7.3 Concluding statement

Wind induced fatigue of light gauge G550 metal is a complex process with the resilience of the cladding sensitive to variations in material geometry, peak loads, load history, and installation. The contributions of this thesis have demonstrated (a) the relevance of the experimental basis of the current test standard (L-H-L), (b) the improved resilience of the building envelope if designed to the L-H-L standard over the previous test criteria, (c) potential areas to be strengthened in current building stock, and (d) the need for increased design pressures with regard to current Australian wind loading standard on pierced fixed cladding systems.