INQUEST INTO THE DEATHS OF

JOHANNA PAULINA MARIA HEYNEN
AND MARILYN JEANNE MCDOUGALL

Finding of Inquest - Cause of Death

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Inquest conducted by Mr Wayne Chivell, State Coroner, South Australia

CORONERS ACT, 1975 AS AMENDED

SOUTH *AUSTRALIA*

FINDING OF INQUEST

An Inquest taken on behalf of our Sovereign Lady the Queen at Adelaide in the State of South Australia, on the 6th, 7th, 8th, 9th, 10th, 13th, 14th, 15th, 16th, 27th, 28th, 29th and 30th days of September 2004, the 5th, 14th and 15th days of October 2004 and the 1st day of June 2005, before Wayne Cromwell Chivell, a Coroner for the said State, concerning the deaths of Johanna Paulina Maria Heynen and Marilyn Jeanne McDougall.

I, the said Coroner, find that Johanna Paulina Maria Heynen aged 70 years, late of 17 Hawaii Court, West Lakes, South Australia, died at Riverside Golf Club, 26 Lochside Drive, West Lakes, South Australia on the 2nd day of April 2002 as a result of crush asphyxia.

I, the said Coroner, find that Marilyn Jeanne McDougall aged 52 years, late of 84 Avro Avenue, Hendon, South Australia died at Riverside Golf Club, 26 Lochside Drive, West Lakes, South Australia on the 2nd day of April 2002 as a result of crush asphyxia.

INQUEST INTO THE DEATHS OF
JOHANNA HEYNEN & MARILYN MCDOUGALL
FINDING OF THE STATE CORONER

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EXECUTIVE SUMMARY

1. On Tuesday 2 April 2002, a section of the roof over the Dining Room at the Riverside Golf Club collapsed. There were 60 to 80 women in the room at the time.

2. As a result of the collapse, Johanna Paulina Maria Heynen and Marilyn Jeanne McDougall sustained fatal injuries. Several emergency services workers placed themselves at considerable risk to enter the building and determine whether there were any survivors. Several other women sustained substantial injuries, and many others were understandably shocked and distressed.

3. The portion of the building where the collapse occurred was an addition built in 1995 by Brian MacKenzie, the proprietor with his wife Valerie of the business Brian MacKenzie and Associates[1]. The addition was designed by architect Barry Matthews. Mr Matthews prepared working drawings, but he was not asked to, and nor did he prepare, detailed specifications for the roof of the structure.

4. The addition was granted development approval by the City of Hindmarsh and Woodville (now the City of Charles Sturt) on 3 July 1995.

5. The development approval was granted subject to conditions. These involved the submission of computations for the roof trusses, and the obtaining of a Certificate of Occupancy prior to occupation of the building. Neither of these conditions were complied with.

6. MacKenzie ordered prefabricated timber roof trusses from Wingfield Timber Supplies. The trusses were fabricated by Trussfab using computer software developed by Pryda Australia Pty Ltd and licensed to Trussfab. Both Wingfield Timber Supplies and Trussfab were part of MSP Group Pty Ltd, owned by Mark Pickard. In fabricating the trusses, Trussfab used timber dried, planed and graded by Tasmanian Board Mills Limited, a mill owned by Allen Taylor and Company Limited, a wholly owned subsidiary of Boral Limited.

7. The additions were completed in September 1995.

[1] Transcript, page 70

8. The collapse was investigated by the Department of Administrative and Information Services - Workplace Services. They retained Mr John Goldfinch, Consulting Engineer, to examine the cause of the collapse. It was apparent that a double girder truss, which carried the ends of the trusses in the existing roof as well as part of the weight of the addition, had failed at its western end, in the western top chord member, and at two nailplate joints in the bottom chord. Sections of the failed double girder truss were sent to Melbourne for examination by the CSIRO.

9. Mr Goldfinch reported:

- The manner of the failure at the western heel of the double girder truss was 'unique';
- Testing by the CSIRO did not provide a clear picture of the complex mechanics of the heel failure;
- The timber used in the trusses, F17 visually stress-graded hardwood, contained small areas of brittleness, excessive slope-of-grain, and tight gum veins which rendered it more liable to failure;
- There were clear deficiencies in the way that tile battens were fixed to the double girder truss;
- The insufficient lateral restraint of the top chords of the double girder truss as a result of the poorly installed tile battens exacerbated the western heel failure of the double girder truss.

10. Evidence produced by Boral Timber disputes that excessive slope-of-grain, brittleness or the presence of tight gum veins were present in the timber of the failed double girder truss, or that these factors played any part in the failure of the trusses.

11. Testing by Pryda Australia Pty Ltd using full scale replicas of the double girder truss (rather than replicas of sections of it used by the CSIRO) established that when the truss was adequately braced, heel failure did not occur at all. At much higher loads, (more than double the service loads), the truss failed at the nailplates, not the heel.

12. The Pryda testing established that the western heel failure in the Riverside Golf Club double girder truss was more likely to have been due to the failure of the top chord of the truss, due to lateral buckling made possible by insufficient lateral bracing.

13. Mr Goldfinch accepted the soundness of this conclusion, although he maintained the possibility that the heel failed first.

14. Mr Goldfinch performed further calculations which established that the double girder truss was overstressed. He argued that the Pryda software, used in the design of the double girder truss, should have allowed a reduction factor (K_2) of 0.7, because this was a commercial-scale building, rather than 1.0 which is appropriate for housing construction. I agree with Mr Goldfinch, and reject Pryda's argument that this was not a commercial-scale structure. The Pryda software did not allow the operator to adopt a K_2 factor of 0.7, and it should have done.

15. I therefore accept Mr Goldfinch's opinion that the double girder truss was overstressed. However, the Pryda testing established, and I accept, that notwithstanding the overstress, if the double girder truss had been adequately braced, it would not have failed on 2 April 2002.

16. Accordingly, I find that lack of adequate lateral bracing of the double girder truss was the principal cause of the collapse of the double girder truss on 2 April 2002. It has not been established that other factors, such as overstressed members and alleged defects in the timber, were contributing factors.

17. AS 2050-1995 (Installation of Roof Tiles) refers only to rafters, and not to trusses. Despite that, the building industry seems to have regarded the Standard as applicable to truss roofs. That Standard requires that tile battens maintain the structural integrity of the roof, and that there be no more than one batten join in every three battens on one rafter. Those conditions were not complied with in this case.

18. There were many more than three splice joins on individual trusses, and many of the battens were split, no doubt because they had been nailed too close to the end. The roof tiler, Mr Fenech, installed the tile battens. He was not supervised by his principal contractor, Monier PGH Holdings Pty Ltd, when the battens were installed.

19. AS 1720.1-1988 Timber Structures, Part 1: Design Methods requires that the nail used in a batten join should be hammered in at a point not less than 20 times the diameter of the nail from the end of the batten. Since the trusses in this case were only 35mm thick, compliance with that standard was impossible when constructing a join or splice over a truss. A 28mm nail would need to have been hammered in 56mm back from the end.

20. Even if the batten had been installed perfectly, and clearly they were not, Mr Goldfinch said that they would only have been barely adequate for the task.

21. There is an urgent need for designers and manufacturers of truss roofs to recognise that, particularly in the case of heavily loaded girder trusses, lateral bracing is a critical issue. Lateral bracing should be specifically designed for the roof according to appropriate engineering standards, and installation of such bracing should be adequately supervised and certified by properly qualified technicians.

22. The City of Hindmarsh and Woodville, now the City of Charles Sturt, placed conditions on development approval for the building which included a condition that the computations for the roof trusses be submitted. This should have afforded them the opportunity to check the adequacy of the design of the truss roof before it was installed.

23. The condition was of doubtful legal validity, but in any event it was not complied with. The City did nothing to enforce compliance with the condition. The Building Surveyor said that even if he did receive the computations, he would not have been able to check the computer program which designed the trusses so his checking would have been of little value.

24. The obligations on the builder to certify his building, and on the owner to seek a Certificate of Occupancy at the completion of the works, were not complied with either.

25. This should have given the City of Hindmarsh and Woodville the opportunity to check whether the conditions had been complied with, although the Building Surveyor said that they may not have done so.

26. In summary, neither the builder nor the architect, engineer, software designer, truss manufacturer, roof contractor, roof tiler or Local Government authority took any responsibility for the overall integrity of the roof structure.

27. Any roof structure, but particularly one as large as this one, should be designed according to proper engineering standards. Full roof framing and bracing plans should have been drawn up and appropriate structural calculations carried out. This data should have been presented to and evaluated by the City of Hindmarsh and Woodville before development approval was granted, or at some mandated stage of the building process the Building Surveyor should have exercised his statutory authority to check and ensure the adequacy of the design. If Local Government authorities do not have sufficient resources to be able to fulfil this role, then steps should be taken to ensure that these resources are made available.

1. Introduction

1.1. Tuesday 2 April 2002 was 'Ladies Day' at the Riverside Golf Club. The club is situated at 26 Lochside Drive, West Lakes. At about 12:30pm that day, most of the 60-80 ladies who teed off in the morning would have been in the Dining Room for lunch.

1.2. The Dining Room is situated on the north-eastern corner of the building and looks out over the golf course.

1.3. Between 12:30pm and 12:45pm, a section of the roof of the Dining Room collapsed, as a result of which Mrs McDougall and Mrs Heynen received fatal injuries, and eight other ladies received non-fatal injuries.

1.4. The Dining Room had been renovated and extended in 1995. A large portion (about 12 metres) of the northern wall was demolished, and the room was extended northwards by about 10 metres in a half-octagonal shape (see the diagram at paragraph 5.10). The roof section of this extension was involved in the collapse.

1.5. The focus of the inquest was to determine the reasons why the collapse occurred.

2. Events of 2 April 2002

2.1. Mrs Heynen went to the Riverside Golf Club to meet a friend, have lunch, and play golf in the afternoon[2].

2.2. Mrs McDougall was a regular player at the club and had told her daughter the night before that she would be playing there that day[3].

2.3. A number of the ladies who were in the Dining Room gave eye-witness accounts of what occurred. For example, Margaret Harrison said:

> 'We were all still seated just chatting, suddenly with no pre warning there was a quite deafening noise a very loud cracking noise. I looked around and I was aware that things were falling down and I was struck on the head by a piece of the ceiling, which landed on me. I was hit mainly on the head and the left shoulder. When I was hit on the head I had a lot of pain, I put my hand on my head at the centre rear and I could feel blood to my head.

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[2] Exhibit C1a, p1
[3] Exhibit C4a, p1

> I felt very faint; I had been knocked out of my chair by the falling ceiling and was on the floor. I could hear someone yelling, 'Get out as quick as you can'. I was worried about more coming down, I got up and walked out and someone took me to the ladies cloakroom and washed my face and head with face washers. After I had my face washed I was taken into the lobby by Ina Fowliss who took me outside onto the verandah.' [4]

2.4. Similar accounts were given by Elaine Fabian[5], Ruth Simmonds[6], Fay Snelling[7], Carmel Evans[8], Rylice Tierney[9], Anne Heyden[10] and Marilyn Wildy[11].

2.5. Sandra Price, the Bar Attendant, said that she turned on the air-conditioning at about 12:15pm. She said that she heard several ladies in the dining room expressing alarm, and then a 'loud sharp crack' just before the roof fell in. Ms Price said that she had never heard any unusual noises coming from that area before[12].

2.6. The first emergency services worker to attend the scene was Senior Constable Gerard Powell of South Australia Police. His statement is Exhibit C24a. He said that as he arrived he saw the injured ladies outside the building. He was told by a witness that she thought there were still two people trapped inside. At considerable risk to himself, Senior Constable Powell entered the building and noted the collapsed roof in the dining room. He saw one of the deceased ladies sitting on a chair, trapped by a large beam. He called out to her but there was no response. Several fire officers entered the building and discovered a second lady, also seated, trapped under a large beam. The officers could not find a pulse in either victim. They decided that there was no urgent need to extract the bodies until the area had been made safe for the rescuers.

2.7. After the ambulance and police personnel attended, and the injured women were treated and taken to hospital, State Emergency Services personnel began painstakingly removing the debris. They were aware that the bodies of the two

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4 Exhibit C7a, p2
5 Exhibit C8a
6 Exhibit C9a
7 Exhibit C10a
8 Exhibit C11a
9 Exhibit C12a
10 Exhibit C13a
11 Exhibit C14a
12 Exhibit C15a, p2

deceased women were still in the Dining Room. At one point, the debris dropped a further 30 centimetres or so while the workers were attempting to recover them[13].

2.8. Mr John Goldfinch, a Consulting Engineer, was called to the scene, and he assisted with advice on placing protective supports so that work could continue.

2.9. The body of Mrs Heynen was retrieved first at about 7:20pm. She was still seated at a chair, with her upper body pressed forward by the debris. Mrs McDougall's body was recovered at 8:25pm. She had been lying on the floor, almost immediately under where the roof truss failed, and was covered in debris[14].

3. Cause of death

3.1. A post-mortem examination of the body of Mrs Heynen was performed by Dr R A James, Chief Forensic Pathologist, on 3 April 2002 at the Royal Adelaide Hospital[15].

3.2. Dr James found that Mrs Heynen had suffered three fractured ribs on the left side. He noted marks on the right side of the neck extending up under the chin, across the back from below the left scapula, across the midline to above the right scapula to the shoulder. There was another mark across the lower back. He described these marks as the 'compression sites'[16]. He also noted florid haemorrhages beneath the skin of the upper chest extending up to the mid neck. The lungs showed florid subpleural haemorrhages over the visceral pleura of both lungs, and focal haemorrhages on the scalp.

3.3. Dr James described the above findings as 'obvious features of crush asphyxia'. I accept his conclusion, and find that the cause of Mrs Heynen's death was crush asphyxia.

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[13] Exhibit C25a, p2

[14] See Exhibit C20a, p8 and photographs taken by Sergeant Sheldon, Exhibit C29c - nb. Sergeant Sheldon's statement, Exhibit C29a, is incorrect in that it misidentifies the two bodies, see pp3-4, as does the statement of Ms Awwad, Exhibit C19a, p13

[15] See Exhibit CH2a

[16] Exhibit CH2a, p4

3.4. A post-mortem examination of the body of Mrs McDougall was performed by Professor R W Byard, Forensic Pathologist, on 3 April 2002 at the Royal Adelaide Hospital[17].

3.5. Professor Byard found marked congestion and petechial haemorrhages on the face, conjunctivae, upper chest, both arms, and right thigh. There were a large number of 'parchmented' areas of skin with 'contact pallor' on the posterior of the right shoulder, right back, right thigh and left knee. Mrs McDougall had suffered fractures to five of the right and seven of the left ribs in the postero-lateral area. There was a 40mm bruise on the anterior (front) of the head.

3.6. Professor Byard commented:

> 'Death was due to crush asphyxia with parchmenting and abrasions on the back of the chest associated with bilateral rib fractures and extensive petechial haemorrhages of the upper body. Although no injuries to the brain or skull were demonstrated, the presence of bruising of the anterior aspect of the head may indicate that the deceased was not conscious at the time of the crush injury. No other underlying organic diseases or injuries were present which could have caused or contributed to death.' [18]

I accept Professor Byard's conclusions, and find that the cause of Mrs McDougall's death was crush asphyxia.

3.7. Toxicological analysis revealed that neither Mrs Heynen nor Mrs McDougall had alcohol or other drugs in their blood[19].

4. <u>Background - Brief chronology of events</u>

Date	Description of Event
1951	First occupation of site by Riverside Golf Club. The clubrooms initially consisted of an old tramcar, and then a steel-framed fibro structure was built.
1971	New bar and lounge area built.
1974	Dining room and offices added.
1975	Further modifications. By this time, the building was a large, rectangular structure, running east/west and facing north out onto the golf course.

[17] See Exhibit C5a
[18] Exhibit C5a, p1
[19] See Exhibits C3a and C6a

Date	Description of Event
1993	Old tiled roof replaced with flat grey concrete tiles. At the time it was noted that a number of the trusses were bowed, so the original Oregon trusses were paired up with new Pinus Radiata trusses. The new tiles were supplied by Monier PGH Holdings Pty Ltd.
Early 1995	Riverside Golf Club discusses extension to existing dining room. Member Keith Hall (licensed general builder) produces concept plan[20]. Committee contacts two builders – one is DiMella Constructions. DiMella instructs architect Barry Matthews to prepare preliminary drawings for purpose of quotation. Hall points out to Riverside Golf Club that it is desirable to have all quotations on the same drawings. With DiMella's consent Matthews prepares working drawing to be used in tendering process[21]. Three quotations sought. One withdraws. Hall approaches Brian MacKenzie, licensed general builder, to submit quotation. MacKenzie had previously carried out painting and general maintenance at Riverside Golf Club.
16 May 95	Application for development approval sent to City of Hindmarsh and Woodville (now City of Charles Sturt). Hall named as applicant[22]. Application considered by John Mazzarolo, Building Surveyor.
30 May 95	Monier PGH Holding Ltd provides estimate to MacKenzie for supply and fix of roof tiles[23].
13 June 95	Contract signed between Riverside Golf Club and MacKenzie[24]. Hall named in contract as Supervising Representative of Riverside Golf Club.
28 June 95	MacKenzie obtains estimate from MSP Group Pty Ltd (owned and operated by Mark Pickard) trading as Wingfield Timber Supplies for supply of timber work including roof trusses[25]. Wingfield refers request to Trussfab, another business operated by MSP Group Pty Ltd. Peter Graham, Manager of Trussfab, later attends site and inspects. Roof trusses then designed at Trussfab using computer software developed by Pryda Australia Pty Ltd and licensed to Trussfab. Trussfab then fabricates trusses and supplies to Wingfield.

[20] Exhibit C32c
[21] Exhibit C30a
[22] Signed application Exhibit C33a, p19
[23] Quote Exhibit C31b, p52
[24] Exhibit C31a
[25] Exhibit C31b, p45

Date	Description of Event
3 July 95	Corporation approves development by letters addressed to Hall and Riverside Golf Club[26]. Approval given subject to conditions, most important of which were: • No occupancy of the building to take place until a Certificate of Occupancy issued by the Corporation (issued on the basis of statement by the builder that work had been carried out in accordance with the approval); • Computations for roof trusses be submitted to Corporation for approval before that stage of the work is reached; Common ground at inquest that neither of these conditions complied with.
12 July 95	Work commences – foundations commenced.
9 August 95	Wingfield delivers roof trusses to site[27].
10 August 95	Roof trusses placed in position by MacKenzie, Bruno Stocco (roof carpenter) and assistant. Stocco installs steel 'speed bracing'[28].
18-21 August 95	Monier engage Ian Fenech, roof tiler, to install sarking, tile battens, and tiles. Fenech works with assistant for the next three days.
27 September 95 (approx)	MacKenzie completes work.

5. <u>The investigation</u>

5.1. As I have already mentioned, Mr John Goldfinch was called to the scene of the collapse, and he arrived at about 4:05pm on 2 April 2002. He advised Emergency Services personnel about temporarily bracing the collapsed roof so that the bodies of the deceased could be removed safely.

5.2. Mr Goldfinch was then retained by the Department of Administrative and Information Services - Workplace Services to investigate the cause of the roof collapse and prepare a report of his findings. The report is Exhibit C39.

5.3. Mr Goldfinch is a Civil Engineer and a Fellow of the Institute of Engineers Australia. He has National Professional Engineers registration for structural and geotechnical engineering. He has practised in the structural and geotechnical sub disciplines of civil engineering for more than 30 years. He has developed an interest in the

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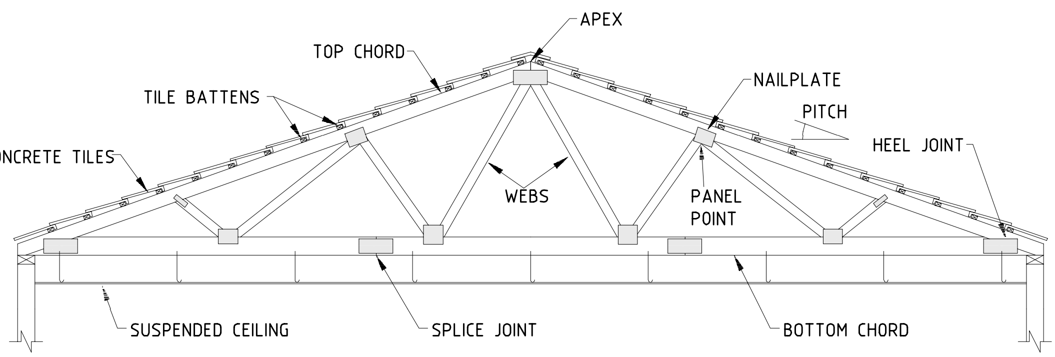
[26] Exhibit C31b, p54
[27] Invoice, Exhibit C31b, p9
[28] See Exhibit C32b

structural engineering aspects of timber truss roofs in the last few years and has conducted several investigations into the failures of such structures.

5.4. When Mr Goldfinch gave oral evidence, he acknowledged that he had gained further experience in these matters through contact with Mr Graham Cooper, the Manager, Technical Development, of Pryda Australia Pty Ltd (as it was in 1995, but which is now ITW Australia Pty Ltd trading as Pryda Australia). I will refer to Mr Cooper's evidence later in these findings. Mr Goldfinch also acknowledged the assistance of Mr John Taddich of MiTek, another company involved in the industry, and other people with whom he has had discussions. It follows from this evidence that although Mr Goldfinch has a wealth of technical knowledge as a structural engineer and wide experience in building matters, he deferred to some extent to the superior experience of Mr Cooper in the very specialised area of the structural engineering aspects of timber roof trusses.

5.5. In order to understand Mr Goldfinch's findings, and opinions, it is necessary to illustrate some of the features of the roof construction at Riverside Golf Club diagrammatically. I therefore reproduce below several illustrations from Mr Goldfinch's compendious report[29].

5.6. Firstly, the various components which make up a timber truss are illustrated below[30]:

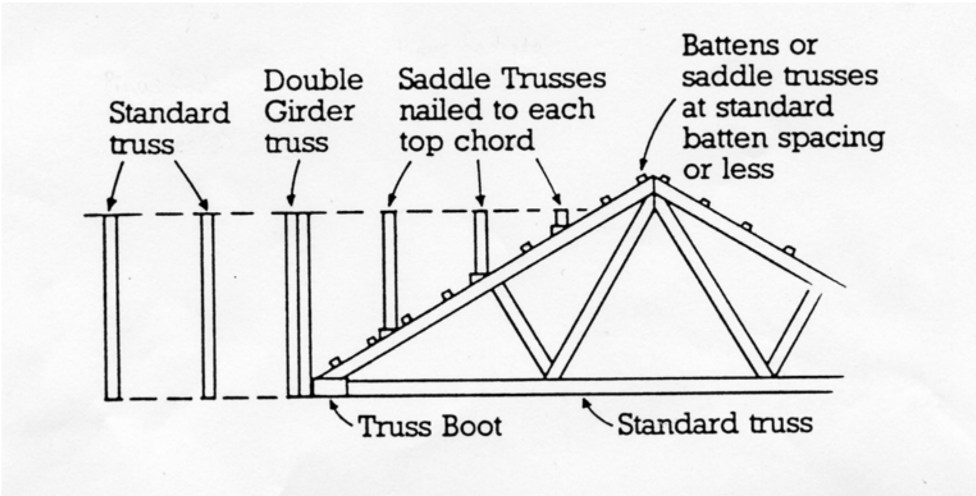


[29] Exhibit C39
[30] Exhibit C39, p30, Figure 4

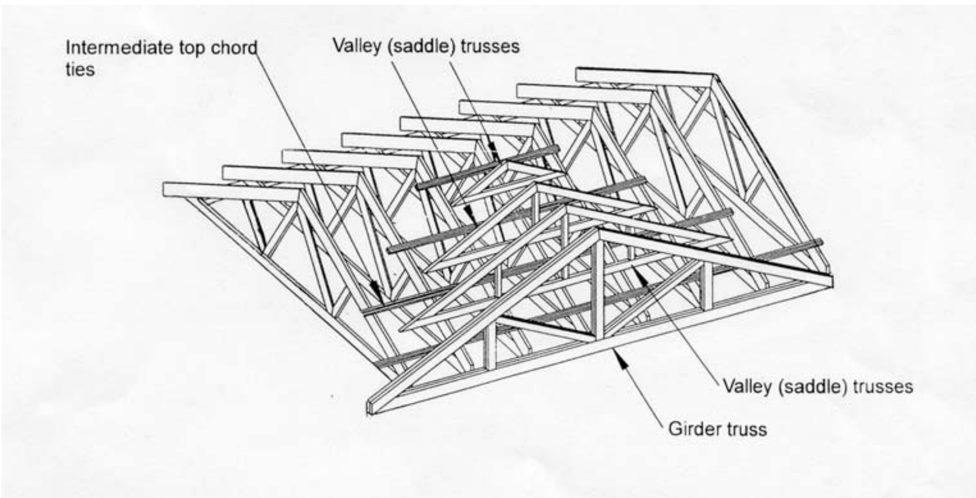
5.7. It is also necessary to set out some of the terminology used:

- *Truss* - a framework of interconnected individual timber members joined together to form a slender triangulated structure which acts like a beam capable of supporting in-plane loading between two or more support points;
- *Top chord* - an inclined timber member that establishes the upper edge of a truss;
- *Bottom Chord* - a horizontal timber member that establishes the lower edge of a truss;
- *Web* - an internal member connecting the top and bottom chords by nailplates in a triangular pattern to provide strength to the structure;
- *Heel Joint* - the joint at the remote ends of the span of the truss where the top and bottom chord members meet and are joined together by nailplates;
- *Panel Joint* - the point of intersection where one or more webs of a truss meet at, and are joined by a nailplate to the top or bottom chords;
- *Splice Joint* - is the point where two members are butt-jointed together by a nailplate to form a continuous member;
- *Nailplates* - galvanised steel plate connectors manufactured from light gauge steel with teeth formed within the parent metal. Nailplates are pressed into opposite faces at the connection points of timber members to form the gusseted or spliced joints at member ends of a timber truss;
- *Girder Truss* - a truss that supports other trusses usually at right angles;
- *Double Girder Truss* - two girder trusses fixed together to provide added strength;
- *Valley or Saddle Truss* - a series of triangular trusses of progressively diminishing proportions that are supported upon other trusses. They are used to form the gable shape and valleys of a roof between two different roof segments, both of which extend at right angles to one another;
- *Battens* - normally hardwood timber members of small cross-section fixed parallel to one another to the top chords of trusses, or to the rafters of a roof system. They are used to support the cladding material such as concrete roof tiles as in this case. When correctly fixed, battens also act as lateral restraints to the truss top chords.

5.8. The construction added a half-octagonal shaped space which protruded in a northerly direction from the existing building. Set out below is a diagram which illustrates the way in which the trusses for an extension are added to the trusses of an existing roof[31]:



5.9. Such a structure produces, in building parlance, a 'Scotch valley', as illustrated below[32]:



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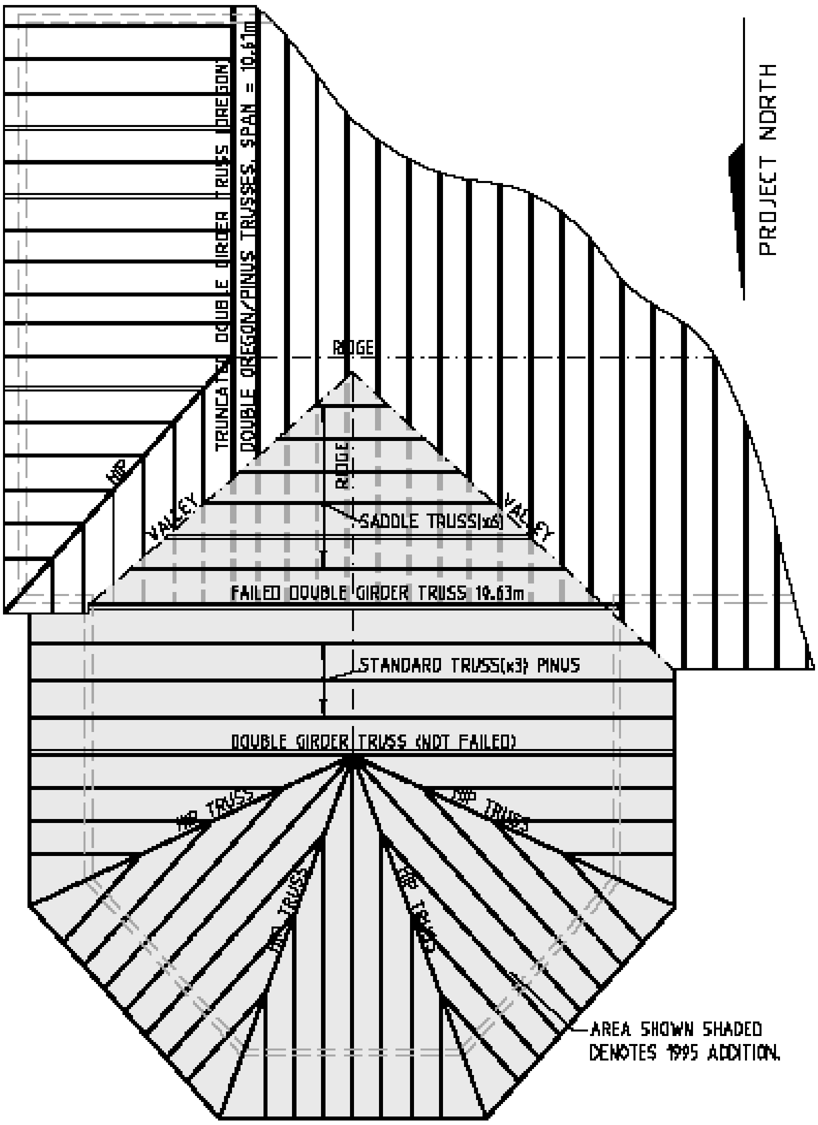
[31] Exhibit C39, p29, Figure 3
[32] Exhibit C39, p28, Figure 2

5.10. A plan of the timber roof trusses for the Riverside Golf Club construction is reproduced below[33]:



5.11. <u>Initial impressions from the scene</u>

Mr Goldfinch said that on inspection of the collapsed roof on 2 April 2002, it was clearly apparent that a double girder truss had failed. The double girder truss in question was installed to replace a section of the northern external wall which had been demolished to provide an opening into the new extension[34]. He said:

> 'The failed Double Girder truss provided support to the northern end of an existing set of parallel double timber nailplate trusses that belonged to the 1993 strengthening of the original 1975 construction. It was constructed as two separate individual hardwood truss plys made up of Tasmanian Oak timber in top and bottom chords of stress grade F17.

[33] Exhibit C39, p24, Figure 1

[34] See the diagram at paragraph 5.10

The two plys were nail laminated together to form the Double Girder truss. Such trusses are commonplace in building construction where a change in roof direction is necessary.

The Double Girder truss was one of two such trusses used in the 1995 extension. The other Double Girder truss and trusses connecting to it at the northern end of the extension have remained in place and did not fail.

The failed Double Girder truss was observed to have suffered a profound fracture at its western support end and also along its top chord member in the first panel down from the apex. Two nailplate joints connecting the Pinus web members to the bottom chord member west of the apex had also failed.' [35]

5.12. A photograph with an aerial view of the collapsed roof, looking generally west, is reproduced below. The western support end, where the double girder truss failed, is in the vicinity of where the workers are standing[36].

5.13. With the assistance of Mr Mark Pickard, the owner of Trussfab, Workplace Services Inspectors Ms Jane Awwad and Mr Graham Stephens, engineer Mr Ralph Belperio and others, Mr Goldfinch supervised the reconstruction of the roof components. All of the timber pieces of the failed roof section, including the tile battens, were painstakingly collected, collated, and transported to a building owned by Mr Pickard

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[35] Exhibit C39, p7
[36] Exhibit C39, p34

where they were reassembled. This process demonstrated a number of important issues in analysing the mode of failure of the double girder truss:

- The failed top chord panel had a lateral bow towards the south between the apex and the eastern heel joint;
- The failed top chord panel to the west of the apex had, before fracture, developed a corresponding lateral bow in a northerly direction;
- There were clear deficiencies in the way the tile battens were fixed to the trusses. Mr Goldfinch said:

> '… it was clearly apparent that both the timber tile battens themselves and their anchorage to the top chords of trusses using 50 mm long x 2.8 mm diameter plain steel nails was particularly poor. This appeared to be the case in spite of the obvious trauma of collapse. Not only were many of the fixing nails significantly bent, but most tile battens had split ends and either nails, or witness marks where nails had been driven, indicating that they (the nails) had been positioned too close to the ends of individual tile battens …' [37]

5.14. The construction of timber structures is subject to a considerable number of variables. Mr Goldfinch commented in his report:

> 'In reality, timber is an heterogeneous and anisotropic material, subject to differences within the same species, some degree of biological variability and a wide array of natural irregularities and defects, such as knots and gum veins. Even within the same piece of timber there can be stronger and weaker parts depending upon factors such as the brittleness of the wood, slope-of-grain angle and the positioning of defects such as knots and gum veins at critical locations (e.g. at nailplate joints).
>
> Nevertheless, experience has shown that for the most part the creation of analogue models for design, and assumptions made with respect to the uniformity of properties of a variable material such as timber, most often produces safe designs because loads and the strength capacity of materials are both factored for safety in the design process.'[38]

5.15. To a very great extent, an analysis of the tragedy at the Riverside Golf Club on 2 April 2002 is concerned with the question of whether there had been sufficient ‘factoring for safety’ in the design of the failed double girder truss.

5.16. Mr Goldfinch said that the double girder truss failed at the following points:

> '(a) At the western heel end.
> (b) In the western top chord in the first panel down from the apex.

[37] Exhibit C39, p46
[38] Exhibit C39, p26

(c) At the bottom chord timber butt joint splice just to the west of the apex.

(d) In the series of four parallel nailplates at the first panel point in from the western heel end along the bottom chord.

(e) In the four parallel nailplates located at the second panel point in from the western end heel along the bottom chord.' [39]

5.17. It was what he described as the 'uniqueness of the heel failure', however, which caused Mr Goldfinch to discuss the issue with Dr R H Leicester from the CSIRO, which led to the construction of a number of trusses for examination and testing.

6. CSIRO testing

6.1. Several copies of the failed western end heel joint of the double girder truss were fabricated by Trussfab and transported to Melbourne where they were examined and tested by Dr Leicester and his assistants at the CSIRO.

6.2. These were not entire trusses, but were 3.7 metre-long sections measured from the heel joint. They were tested by loading until failure of the heel joint was induced. The other components of the truss were not examined as part of the test.

6.3. The first of these tests involved the placement of a small saw cut through the bottom chord near the support end of the truss, to simulate the weakness created by the presence of 'quite prolific gum veins' noted in the bottom chord of the failed double girder truss at that point.

6.4. Under a load of 54.3 kilonewtons (kN), which is 1.55 times the calculated service on the double girder truss load (35 kN), complete failure of the heel joint occurred. Mr Goldfinch described the features of this failure as 'amazingly similar' to that seen in the failed double girder truss[40].

6.5. The next tests involved placing loads on the heel joint without the weakness induced by the saw cut. These tests produced failure at multiples of the service load of 1.50 (52.5kN), 1.99 (69.8kN) and 2.47 (86.3kN) respectively. The CSIRO report[41] commented that the first of these subsequent failures, at a load (52.5kN) even smaller than the one weakened by the saw cut, could have been caused by the presence of

[39] Exhibit C39, p35

[40] Transcript, page 1245

[41] Exhibit C39, Appendix 5

brittle wood in the test joint[42]. On the basis of that suspicion, that test result should be excluded.

6.6. The CSIRO engineers did not get a clear picture of the complex mechanics involved in the failure from these tests. They commented:

> 'What is clear, is that simply determining the tensile and compressive actions in the top and bottom chords of a truss of this type is not sufficient to predict the behaviour of the heel joint, or indeed to design it to resist the more complex stresses which the wood at the heel is subjected to.' [43]

6.7. <u>Timber quality</u>

The CSIRO engineers examined the quality of the timber in the failed double girder truss and made the following observations:

- AS2082 (Timber - Visually Stress Graded for Structural Purposes) allows F17 timber to have a slope-of-grain of 1:8 (or 7°). In some small local areas near the fracture of the top chord in the failed double girder truss, the CSIRO noted a slope-of-grain of 12°. Mr Cooper and Mr Goldfinch revisited the reconstructed roof structure during the inquest and they noted that the slope-of-grain was even greater than that. Mr Cooper said:

  > 'Mr Goldfinch and I measured it this morning and measured it at 15 degrees; that's visible to the naked eye. It's required to be checked within the visual grading rules that are in existence in Australia at the present time and, indeed, at that time, and the brief comment is that at that particular slope the strength of timber dramatically falls off and, strictly speaking, that piece of timber is not in any structural grade at all and was even worse or lower strength than we assumed it would have at 12 degrees.'[44]

- AS2082 also prohibits brittle 'heartwood' being used in such members. A small zone of brittle wood was also noted in the top chord in the vicinity of the fracture site;

- AS2082 permits unlimited tight gum veins and a limited number of loose gum veins in structural timber. The CSIRO noted the presence of tight gum veins in the bottom chord of the double girder truss in the vicinity of the western heel. They commented that this would have weakened the timber in that area, to the extent discussed below.

[42] See CSIRO Report, Exhibit C39, Appendix 5, p17

[43] Exhibit C39, Appendix 5, p16

[44] Transcript, page 895

The issue of timber quality will be discussed in more detail in the section entitled 'The Boral Position'.

6.8. Multiple truss effect

The CSIRO report also raised the issue of the so-called multiple truss effect. It referred to a paper written in 1994 which examined the postulated loss of strength efficiency due to the uneven distribution of loads between two trusses in such a system[45]. Mr Cooper disputed this, saying that the heavy steel brackets and bolts, used in Pryda systems to mount the supported trusses to the double girder truss, would have prevented rotation, and would have negated this effect[46].

6.9. Results

The CSIRO attributed values to each of these factors and then compared the relative strengths of the timber in the failed double girder truss and the timber in the test trusses. The values they arrived at were:

- 10-15% - weaker wood in the failed truss due to brittleness;
- 13% - weaker from multiple truss effect;
- 10% - weaker from gum veins;
- 20% - weaker from ageing of the timber due to the so-called mechano-sorptive effect whereby stress redistribution occurs within timber as it ages.

On the basis of these figures, the CSIRO engineers calculated that the test trusses were stronger by a factor of 1.15 x 1.13 x 1.1 x 1.2 = 1.7. Since one of the test trusses failed at the heel joint at only 1.5 times the assumed service load, they concluded that failure of the Riverside Golf Club double girder truss heel joint was a 'plausible' explanation for the collapse(p45).

6.10. However, these figures should be treated with caution:

- If the multiple truss effect (1.13) is ignored on the basis of Mr Cooper's evidence, the factor is reduced to, on my calculations, 1.5 x 1.1 x 1.2 = 1.5;
- The failure at 1.5 times the assumed service load was attributed to brittle timber. If that result is excluded, the next test disclosed failure at 1.99 times the assumed

[45] Enjily and Whale, 1994
[46] Transcript, page 643

service load. If the test timber was only 1.5 times stronger, but did not fail until almost twice the assumed service load was applied, failure of the heel joint is a less plausible explanation for the collapse;

- The observations of weakness due to brittle wood and gum veins were disputed by Boral's experts, as I will presently discuss. If those factors (1.15 and 1.13) are excluded, the explanation is even less plausible.

6.11. The second hypothesis examined by the CSIRO was that lateral buckling of the failed double girder truss, as noted by Mr Goldfinch and others at the site, was either a causative or at least a contributory factor to the failure.

6.12. Unfortunately, because they did not have full trusses to examine, the CSIRO engineers were restricted to a theoretical analysis and a 'simple laboratory experiment' (p46) which did not attempt to test the strength of the complete failed double girder truss (p37).

6.13. The engineers concluded that lateral movement of the top chord alone was 'unlikely' to have caused the fracture of the heel of the double girder truss. They heavily qualified this conclusion however, saying:

> 'Further testing of full-length trusses under in-service loading is required to definitively determine if lateral buckling may have caused or contributed to the heel failure on the Riverside truss' [47]

They added:

> 'It should be noted that very limited time and funding was available to undertake this investigation. Before specific technical recommendations can be given, a deeper understanding is required of the behaviour and characteristics of the type of truss examined. Many of these trusses are installed in buildings around the world, and catastrophic failures such as in the Riverside Golf Club are rare (and perhaps unique). Further experimental and analytical work is recommended to gain the understanding required to determine the extent of any problem which may exist in heel joint configuration used in such trusses.' [48]

6.14. <u>Mr Goldfinch's initial position</u>

Mr Goldfinch accepted Dr Leicester's conclusions. In particular, he rejected the suggestion that the top chord panel failed first. He argued that if that had happened, the strength of the triangular structure would have been lost. All that would have

[47] Exhibit C39, Appendix 5, p46

[48] Exhibit C39, Appendix 5, p47

remained was the bottom chord as a 'simple beam of grossly insufficient strength and stiffness properties to carry the load …'. He argued that this would have caused the bottom chord to collapse in the centre, thereby dragging both the western and eastern heel ends from their supports and into the interior of the building. Instead, the truss failed at the western end only. He concluded:

> 'It is more probable that there was initial fracture of timber at the western heel, followed by failure of the top chord and then in-plane bending failure of the bottom chord only when the Double Girder truss fell down from its western end impacting with both tables and chairs below and the concrete section of flooring which was part of the 1995 extension.' [49]

6.15. As to the nailplate failures in the bottom chord, Mr Goldfinch argued that these occurred when the double girder truss fell to the floor, and were thus an effect of the collapse, and not a cause of it[50].

6.16. Mr Goldfinch was prepared to acknowledge, however, that insufficient lateral restraint, as a result of the poorly-fixed tile battens, had allowed lateral buckling of the top chord of the double girder truss. This would have applied a torsional or twisting force at the western heel. However, like the CSIRO, he concluded:

> 'I do not believe, however, that the twist applied at the western heel or the tension forces at web joints would have been of sufficient magnitude to induce failure by this process alone. Rather, it is my opinion that these actions had an exacerbating influence on what was an initial western heel timber fracture.' [51]

6.17. <u>Other factors which did not contribute to the collapse</u>

Mr Goldfinch dealt specifically with these matters in his report. He acquitted the following factors from blame for the failure:

- Air conditioning componentry - the main air conditioning units for the area in question were supported independently of the failed double girder truss. The lightweight cylindrical air ducts, inlet and outlet vents, and a supplementary air conditioning unit weighing 60kgs or so, were installed immediately north of the failed double girder truss. Mr Goldfinch said that they were insufficiently heavy to cause or contribute to the collapse ;

[49] Exhibit C39, p74
[50] Exhibit C39, p74
[51] Exhibit C39, p75

- Wood rot and timber parasites - there was no evidence of rot or decay in the timber or attack by borers or termites;
- Soil movement – soil at the site was naturally occurring dune sand which would have been stable and is the best possible class of soil on which to found a structure. Mr Goldfinch was satisfied that neither soil movement or any resulting instability of the steel support structure contributed to the collapse[52].

7. The Boral Position

7.1. After completion of the oral evidence, and while writing these findings, it became apparent that the alleged manufacturer and/or supplier of the timber used by Trussfab in the manufacture of the double girder truss should be given the opportunity to be heard.

7.2. On the basis of the oral evidence of Mr Pickard that Wingfield Timber purchased the timber from Boral Timber, I wrote a letter dated 31 January 2005 to the Manager of that business giving notice of these proceedings. Since that time, I have received from Finlaysons, the lawyers representing Allen Taylor and Company Limited, the following material:

- Affidavit of Murray David Floyd dated 15 April 2005 which I have marked Exhibit C45.
- Affidavit of Andrew Stephen Williams dated 15 April 2005 which I have marked Exhibit C46.
- Affidavit of Geoffrey Neville Boughton dated 2 May 2005 which I have marked Exhibit C47.
- Affidavit of Charles Herbert dated 3 May 2005 which I have marked Exhibit C48.
- Written submissions.

7.3. Each of those affidavits had exhibits to them, which I have also read and considered without objection from the other parties already given leave to appear at the inquest. Finlaysons advised that if other parties had no objection to this course, they were

[52] Exhibit C39, pp39-41

content to rely on the contents of those documents, and not seek a further oral hearing. No such objection was received by the due date, 29 April 2005.

7.4. On the basis of that material, the following facts emerge:

- The timber used by Trussfab was dried, planed and graded at Tasmanian Board Mills Limited (now Boral Mills Ltd (in liquidation)). In 1995, Tasmanian Board Mills was owned by Allen Taylor and Company Limited, a wholly owned subsidiary of Boral Limited. Boral Timber is a business name owned by Boral Limited. I will therefore refer to this group of entities by the collective description 'Boral'.

- It is not known whether the timber was supplied to Wingfield Timber Pty Ltd direct, or through an intermediary. That matters little for my purposes.

7.5. There were three issues concerning timber quality raised in the CSIRO report and examined by Messrs Goldfinch and Cooper, which were addressed by the Boral experts.

7.6. Slope-of-grain

AS2082 allows a slope-of-grain of 1:8 in visually stress graded timber. A slope-of-grain of 12° was noted by the CSIRO in the top chord of the failed double girder truss near where it fractured. Visual inspection of the trusses by Messrs Goldfinch and Cooper during the inquest revealed a slope-of-grain of 15º. Mr Cooper commented that this slope-of-grain invalidates its F17 grading, as I mentioned earlier (paragraph 6.7).

7.7. Mr Charles Herbert, the Chief Timber Inspector, Forests NSW, prepared a report for Boral's solicitors in which he stated:

> 'I did not see any slope of grain which exceeded the limits for slope of grain for structural grade 3 AS 2082-1979. In one place, however, on the top edge of the top chord of the northern truss adjacent to where a sample area had been taken by others for examination and adjacent to where the top chord had failed, there was localised sloping grain with a slope of about 1 in 3. This sloping grain was localised to a depth of about 20 to 25 mm. The remaining 100 to 110 mm or so of the 'depth' of the top chord had considerably less slope of grain and was within the limits of 1 in 8.'[53]

[53] Exhibit C48, p2

7.8. Mr Herbert explained that because this sloping of grain was 'localised' to a depth of only 20 to 25 mm, this did not affect the overall slope-of-grain, which he said was within the limit of 1:8 set by AS 2082-1979[54].

7.9. Professor Geoffrey Boughton, a Timber Engineering Consultant and a former Senior Lecturer in Structural Engineering at Curtin University of Technology also prepared a report for Boral's solicitors. He pointed out that CSIRO's testing disclosed that even those sections of wood with the higher slope-of-grain still exceeded the strength required by the standard (50Mpa)[55].

7.10. Brittle wood

There was also a small zone of 'brittle' wood noted by the CSIRO in the top chord of the double girder truss in the vicinity of the fracture site (see paragraph 6.10 above).

7.11. Mr Herbert pointed out that there is a distinction between 'heartwood', which is permitted by AS 2082-1979, and 'pith' or 'heart' which is not. He said:

> 'I did not see any brittle heart in any of the chords. None of the chords appeared to have been cut from within about 200 mm of the growth centre (or heart) of the log.'[56]

7.12. Professor Boughton agreed:

> 'It is my opinion that none of the hardwood timber used in the truss could have been regarded as 'brittle heart' material, and the strength of the timber that was described as 'brittle' indicated that the material had performance that exceeded the performance requirements of F17 timber.'[57]

7.13. Gum veins

The CSIRO report referred to the presence of tight gum veins in the bottom chord of the double girder truss in the vicinity of the western heel (paragraph 6.7 above)

7.14. Mr Herbert said:

> 'There were gum veins in the top chord of the southern truss at the western end of the heel joint. The timber in the end of the top chord had failed and was in several pieces – although the fibres of the wood had separated in the vicinity of the gum veins and in places had separated from one side of part of the gum vein, the gum veins themselves

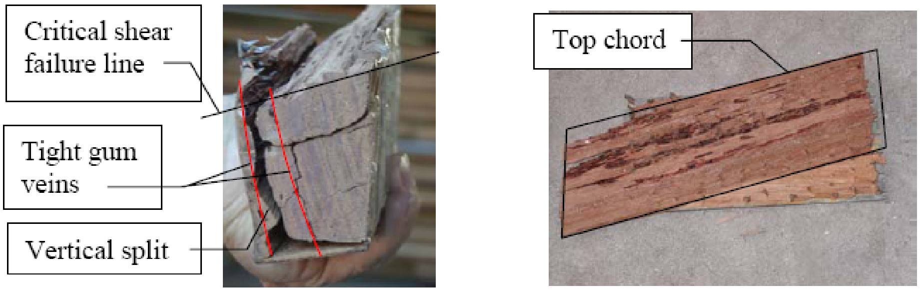
[54] Exhibit C48
[55] Exhibit C47, p16
[56] Exhibit C48, p4
[57] Exhibit C47, p17

had not failed. Had a gum vein failed, I would have expected to see a separation of the gum vein. Such a separation was not evident.'[58]

7.15. Professor Boughton agreed. He made the following points:

'As shown in Figure 7(a), the line of fracture is adjacent to, not through the gum vein. More than two thirds of the failure surface is through wood adjacent to the gum vein as seen by the lighter colour in Figure 7 (b).

The plane of the shear stresses causing failure in the heel area are perpendicular to the direction of the gum veins as indicated by the critical shear failure line in Figure 7(a).



(a) end view of top chord (b) side view of split with gum vein

Figure 7 – tight gum vein in southern truss top chord

There is no evidence of the wood fibres being dragged downwards. This would be the case if the failure was caused by shear stresses (The fibre distress can clearly be seen at the other shear failures, eg on the vertical shear failure in the heel shown in Figure 6.) This indicates that the fracture was not caused by large shear stresses typical of much of the wood in the heel zone. It is quite typical of wood subjected to a suddenly applied shock load typical of being dropped onto a hard surface.

The failure shown in Figure 7 is parallel to the longitudinal axis of the top chord. Had the vertical split near the gum veins occurred first, then wood on both sides of the break would still have been continuous with the top chord and therefore attracted load. It also would have been still connected by the nail plates and therefore transferred load to the bottom chord. It would have been in contact with the bottom chord in bearing and transferred load to the reaction. The structural system would not have been compromised at all and therefore the single vertical split in the top chord could not have precipitated the collapse.'[59]

7.16. Analysis of failure

Professor Boughton's analysis of the failure of the double girder truss was as follows:

- Initially, buckling developed in the top chords, which grew over time. A classic 'S' shaped curve developed because the top chords were carrying very high

[58] Exhibit C48

[59] Exhibit C47, pp15-16

compression loads. Effective lateral restraint was required from the roofing battens, but this has been 'questioned' (see later);

- Secondly, a split developed in the top chords because of shear stresses due to a combination of:
  - The distance of the heel plate from the support;
  - Transfer of compression stress from the upper part of the chord to the heel nail plate;
  - Shear stresses caused by torsion due to buckling;
  - The lower than normal pitch angle of the roof truss.
- Thirdly, the top chords suffered a flexural failure under combined compression and bending forces. The northern truss failed first, then the southern one, perhaps only microseconds later. This failure led to total transfer of the load to the bottom chords;
- Fourthly, most of the resultant shear forces produced a failure of the bottom chord at the inboard edge of the comfort nailplate where it was scarf cut to only 25 mm deep at that point;
- Finally, a shock loading of the top chord after failure of the bottom chord then produced a total shear failure of the western heel joint.

7.17. Professor Boughton summarised the failure mechanism as follows:

> 'The buckled top chords would have been silently increasing in deformation over a number of years prior to the collapse due to the mechanism of creep buckling indicated by both Cooper [C34 section 9] and Goldfinch [C39 section 11.7.10]. This was allowed by inadequate restraint by the battens as documented by Cooper [C34 section 8].
>
> The split in the top chords may have occurred at any time of the day or night in the few years prior to the failure, and developed as a result of the buckling in the top chord and high shears due to the distance between the heel plate and the support.
>
> At the time of collapse, the western top chords broke in the third panel due to minor axis deformation caused by buckling in compression. This would have been a very audible crack and would have started significant deflection of the girder truss.
>
> Before the deflection was significant enough to cause a flexural (bending) failure in the bottom chord, the shear failures due to overloading at the western heel would have

occurred. This would have produced a very loud bang and the collapse of the roof would have followed immediately.' [60]

7.18. Professor Boughton's findings conflicted with those of the CSIRO. In particular, he disagreed that excessive slope-of-grain, brittle wood or gum veins, played a part in the failure. Without these factors, the calculations I set out in paragraph 6.9, which are intended to establish the extent of the resultant weakness in the timber in the failed double girder truss (1.15 x 1.13 x 1.1 x 1.2 = 1.7) should be amended to 1.15 x 1.13 = 1.3. This is lower than the 1.5 service load ratio achieved in the weakest test specimen. He therefore discounted the possibility that the western heel joint failed first. He said:

> 'In [C39 sections 1.3.4, and 11.6.21] Goldfinch was critical of the low angle of the roof trusses and the full scarf cut in the bottom chord. I agree that both of these factors contributed to the heel joint having a lower strength than could be anticipated using other configurations. This report shows that these problems were further compounded by the low bite of the heel nail plate. However, the CSIRO tests [C39 app5] showed that the heel had adequate strength to carry the applied loads with a well restrained truss top chord.
>
> It is my opinion that the failure of the heel was as a result of other failures in the truss. Once the top chord had broken, the truss action was compromised, and even had the heel delivered enough strength to carry the overload, the bottom chord would have failed.'[61]

8. <u>The Pryda position</u>

8.1. Mr Graham Cooper is a Structural Engineer and is the Manager, Technical Development of Pryda Australia Pty Ltd, a manufacturer of steel nailplates and other componentry for timber roof trusses. The company also sells software on licence which is used by truss manufacturers such as Trussfab to design timber roof trusses, and manufacture them using highly automated equipment. The failed double girder truss was manufactured by Trussfab by this process. Mr Cooper told me that about 140 truss fabricators use their software[62].

8.2. Mr Cooper said that he had been involved in the manufacture of roof trusses since the 1970s, and had been involved in developing the software for that process since the

[60] Exhibit C47, pp9-10
[61] Exhibit C47, p14
[62] Transcript, page 547

1980s. He said that the software used by Trussfab in the manufacture of the double girder truss in question here was developed in 1989.

8.3. Mr Cooper was overseas at the time of the collapse. He arranged for four members of his staff to attend and photograph the scene. He inspected the reassembled trusses personally on 29 April 2002[63].

8.4. After the collapse, Mr Cooper obtained the job file from Trussfab on a floppy disk. In order to verify the original truss design, the 1995 software was installed on a computer, enabling the information on the disk to be retrieved. By this process, the data used in the design of the original trusses was retrieved and could be analysed. The drawings and associated output data were appended to Mr Cooper's report[64] as Appendix B.

8.5. A further set of designs was produced from the retrieved input data, but using current software called Pryda Roof V3.0.2.3. This software provided design criteria which was appropriate to both 'residential' and 'commercial' building types. This is significant because there is a particular issue concerning the so-called 'K_2' factor, which dictates the adequacy of safety margins in such design. The factor is different for different building types and higher safety margins are required for buildings classified as 'commercial'. In 1995, the Pryda software did not address this issue, as I will discuss later. At this stage, it is sufficient to point out that Mr Cooper said that designs using current software, and applying the K_2 factor for 'commercial' buildings, the design criteria produced by the software were 'still satisfactory, but closer to the maximum capacity'[65].

8.6. As to the quality of manufacture of the Riverside trusses, Mr Cooper commented:

> 'The truss design from the Pryda Roof software (from both the 1995 and 2002 versions) indicated that some smaller timbers could have been used in the chords and webs, and that some smaller nailplates could also have been used. I conclude that the use of larger members was in accordance with the design philosophy of the truss designer at that time, Mr Peter Graham, who is known to be a very competent designer.
>
> My inspection of the trusses showed that the truss seemed to be well made, with timber which was marked to the required grade or better, and joints which were tight. One

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63 Exhibit C34a, p1

64 Exhibit C34a

65 Exhibit C34a, Appendix C (see also Transcript, page 1291)

standard truss, T3, had a pair of nailplates missing from an end of the short compression web, but this was not a factor in the failure.' [66]

8.7. Truss installation

Mr Cooper referred to the Pryda Roof Truss Erection Manual. This is a publication which all of the nailplate suppliers produced, and which explained the correct method of installing roof trusses. Since that time an Australian Standard (AS4440-1997) has been developed which covers this issue but, in 1995, such manuals were the only source of guidance in the area.

8.8. Mr Cooper said that there were a number of aspects of the installation of the roof battens at the Riverside Golf Club which did not comply with the specifications in the manual. He said:

'a) No more than one third of the battens may be spliced at any one truss

b) No two splices may be adjacent to each other

c) The nail to be used in battens is 3.75dia x 75mm

Five or six battens were spliced adjacent to each other on the girder truss and the truss T1 beside it. Many battens were in short lengths, and I saw about 10 pieces that were 600 long and a similar number that were 1200mm long. The battens were fixed with 2.8mm dia x 50mm long nails. This type of batten installation contravenes the guidelines for installation that were current at the time.

It is likely that this is the root cause of the failure as too many nails are close to the end of the battens and with the amount of end splitting that occurred would have been unable to resist lateral buckling of the top chord of the girder truss. Nails need to be at least 20 nail diameters from the end of the timber (56mm), and many nails were observed to be 10-25mm in from the cut end. Almost all of them had a split running from the nail to the end of the batten, and this is often observed with most hardwood timbers.

The top chord of the girder must be restrained by the battens to prevent out-of-plane buckling. The battens themselves also need to be stabilised, and this is commonly provided by diagonal steel cross-bracing in the roof plane, but the battens may also be stabilised by being fixed to a rigid support in their own right.' [67]

8.9. The difficulty with the Pryda manual is that it was usually delivered with the trusses, with the expectation that it would be read by the truss installer, the roof carpenter. The person who installed the tile battens, namely the roof tiler, would not have necessarily seen the manual. In this case, neither the builder, Mr MacKenzie, or the

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[66] Exhibit C34a

[67] Exhibit C34a, p4

roof carpenter, Mr Stocco, saw the manual. Mr MacKenzie denied that it was delivered with the trusses[68]. Mr Stocco could not clearly state whether a manual was delivered or not[69]. The roof tiler, Mr Fenech, said that he had never seen the manual[70].

8.10. Roof plane bracing

This is also referred to as cross bracing, or speed bracing. Mr Cooper commented:

> 'Two pair of steel tension cross bracing were installed with one brace at approximately 65 degrees to the ridge line, and the other at approximately 50 degrees, which is well in excess of the maximum of 30 degrees specified in the Pryda Installation Guide of that time, and even the 30-45 degrees that exists today. This steep angle reduces that capacity of the brace to transfer truss buckling loads to the supports, but a brief analysis shows that they could still work even at this angle.' [71]

This topic was not pursued during the investigation, and I do not consider it likely that it was a significant factor in the collapse.

8.11. Pryda testing

Unlike the truss sections built for the CSIRO, Pryda constructed six full-scale trusses to the same design as those used in the failed double girder truss at the Riverside Golf Club. Loads were applied to these trusses in situations where the top chords were fully braced to prevent sideways movement to simulate the restraint which battens are intended to provide. Other trusses were allowed to displace sideways as the load was applied to simulate ineffective lateral bracing.

8.12. Mr Cooper summarised the test results in his further report as follows::

> **'Fully braced trusses**
>
> - These two trusses failed at various web and splice nailplate joints. Pryda was unable to achieve heel failures in these trusses, and there was no sign of distress in the heel region after failure.
> - The damage observed in a number of places along these failed trusses was quite different to the RGC experience with nailplates fracturing at various locations, and is close to the failure modes expected analytically. After this experience Pryda discontinued this approach, and changed to unbraced trusses.

68 Transcript, page 99
69 Transcript, page 843
70 Transcript, page 754
71 Exhibit C34a, p5

Unbraced trusses

- The failures at different locations along the truss were quite similar to the RGC photos in several areas.
  - Top chord fracture in the panel mid-way between the heel and apex joints.
  - Heel nailplate peeling sideways.
  - The deflected shape of the top chord was identical with a classic S-shape curve.
  - Bottom chord web nailplate failure by tearing in tension on one side and buckling on the other side.
  - Timber in heel shattered quite badly in Test 4 and to lesser degree in Test 6, but not at all in the other tests.' [72]

8.13. Mr Cooper set out the results of the testing as follows:

Truss	Top chord bracing	Load at failure	Mode of failure
No.1	Fully braced	6900kg	Fracture of splice & web nailplates
No.2	Fully braced	7230kg	Fracture of splice & web nailplates
No.3	Battens nailed at web nodes	4650kg	Sideways buckling of top chord
No.4	One batten nailed mid-panel	4760+kg	Sideways buckling of top chord
No.5	Battens nailed at heel and apex	3220kg	Sideways buckling of top chord
No.6	Battens nailed at apex only	3100kg	Sideways buckling of top chord

'The permanent load on the truss at RGC at the time of the collapse was 5500 kg for the double truss, or 2750 kg for each ply. The test results above are for a single ply. While the number of trusses tested is quite small, the results are indicative as the on-site conditions were very closely replicated with the correct distribution of loads on both the top chord and on the bottom chord.' [73]

8.14. Like the CSIRO testing, it can be assumed that the timber used in the Pryda testing was stronger than in the failed double girder truss, and yet the failure load in test number 6 was at only 3100kg, or 1.13 times the assumed service load (2750kg per truss) with no lateral support. With full lateral support, failure did not occur until 6900kg (2.5) or 7230kg (2.6). In both of these latter cases, the mode of failure was also different. The failure was at the nailplates rather than in the top chord.

[72] Exhibit C34c, p1
[73] Exhibit C34c, Appendix A

8.15. Mr Cooper said that the testing, which he regarded as more indicative than that carried out at CSIRO, supported the theory outlined in his initial report. He said:

> 'Given an inability of the batten nails to transfer the lateral restraint forces into the battens, then the top chord of the girder truss would have started to buckle towards the octagonal end on the half of the truss that failed …
>
> A simple S-curve was introduced into the top chord of the girder truss, so that the top chord on the other half of the truss followed this curvature by buckling in the opposite direction … This sideways buckling would have pushed the battens towards the hexagonal hip end, and would have also introduced buckling in the three standard trusses …
>
> The process of creep would have seen this action steadily increase over time, putting enormous stresses into the girder truss top chord, as its capacity to resist compressive forces would dramatically reduce. While the top chords were suffering these actions, the truss must have been under considerable distress. I estimate that just prior to collapse, the top chord may have buckled out of line by as much as 200 or 300 or even 400mm. This is based on observing the remaining part of the top chord from the apex to where it is broken.
>
> The trusses being carried by the girder truss were all well fixed by heavy-duty truss boots to the girder bottom chord with substantial bolting. These trusses provide an extremely rigid connection and would have held the bottom chord of the girder truss in place with no rotation possible.
>
> As the top chord buckled sideways, with the batten nailing unable to adequately prevent this from happening, the truss would nevertheless try to resist this by the webs cantilevering from the bottom chord, and also by the heel joint trying to provide torsional restraint …
>
> The final action was that the top chord of the girder truss at the heel had buckled so far, and the bottom chord was so rigidly held, that the top chord in fact twisted upwards and sideways away from the heel joint, and failed due to torsion which applied additional stresses which were essentially perpendicular to the grain of the timber - its weakest direction …
>
> …
>
> …
>
> Once the top chord(s) had come to a point where the timber capacity was exceeded in this complex failure mode, and shattered, the whole weight of the girder truss reaction would be thrown onto the small portion of bottom chord that remained (25mm deep), and this was then overwhelmed in vertical shear, despite this not being a normal failure mode with timber.' [74]

8.16. In summary then, Mr Cooper did not agree with Mr Goldfinch that the western heel of the double girder truss failed first. His opinion was that the heel failure was the final event after the top chord fractured due to lateral buckling, which was caused by

[74] Exhibit C34a, pp6-7

inadequate lateral restraint, which should have been provided by properly installed tile battens.

8.17. Mr Cooper's theory as to the sequence of failure was very dramatically and clearly demonstrated by the computer animation he produced[75].

9. Consensus position

9.1. At my request, a conference was convened between Mr Goldfinch and Mr Cooper at which the extent of their disagreement could be discussed. The inquest was adjourned and other evidence heard while the two men conferred privately and compared their calculations and modes of approach.

9.2. When the inquest resumed on 15 September 2004, a considerable degree of consensus had been achieved, which reflects great credit on both Mr Goldfinch and Mr Cooper.

9.3. One issue resolved was the degree of 'point loading' onto the bottom chord of the failed girder truss applied by an incoming truncated girder truss, which was part of the original roof. Mr Goldfinch had originally calculated the load at 12.5kN whereas Mr Cooper had calculated it at 9kN. It was agreed that the point loading was 11.5kN. Mr Cooper said that this was a relatively minor adjustment and brought the total load on the failed girder truss as calculated by Mr Goldfinch down from 55kN to 54kN[76].

9.4. It was further agreed that the Manual of the Truss Plate Institute of Canada of 1996[77] provided an appropriate truss model design to analyse the trusses involved here. On that basis, it was agreed that Mr Goldfinch would prepare new engineering calculations using that model[78].

9.5. As to the stresses which would have been applicable to the truss members in this case, it was agreed as follows:

- A design check using a 'limit state' technique of the type introduced in AS1720.1-1997 would be undertaken, thereby applying current day standards[79];

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75 Exhibit C34d
76 Transcript, page 887
77 Exhibit C39, Appendix 2
78 Transcript, page 888
79 Transcript, page 890

- Revised calculations indicated slight overstress in top and bottom chords near the western heel position of 11% and 12% respectively. There was a 15% overstress in the bottom chord at the position of the incoming truncated girder truss. At the top chord notch position there was a 9% overstress. These values were derived for the design life of the structure (50+ years). As timber strength varies over time, there would have been no overstress in truss members at the time of failure[80].

  Mr Cooper described these overstresses as 'slight'. He said structures were designed to carry loads of around 1.5 to 2 times in excess of service loadings. He explained:

  > 'By way of explanation to the over-stress question, structural failures normally require loads of one and a half to two times in excess of the service loads.'[81]

- All nailplates and web members were found to be satisfactory and within stress limits as were all remaining parts of the top and bottom chords[82].

9.6. Mr Goldfinch confirmed that although the word used in the second dot point above was 'overstress', the stresses were still within the margin of safety demanded by the relevant timber engineering Code. I was told that basic engineering principles usually require a safety margin of 200% of the design loads to be placed on the member, so overstress of 111%, 112%, and 115% were not considered significant. Mr Goldfinch's evidence was:

> 'Q … even though there was slight over-stress observed and you've reached agreement about the degree of over-stress, in your opinion that did not cause the failure of the Riverside truss.
>
> A. No, it did not.'[83]

9.7. Mode of failure

The two engineers reached a consensus position which substantially accepted Mr Cooper's thesis:

> 'J Goldfinch accepts that G Cooper's thesis is sound, as summarised in Cooper's report and animation submitted to this court on Friday, 10 September.' [84]

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[80] Transcript, page 891
[81] Transcript, page 892
[82] Transcript, page 892
[83] Transcript, page 899
[84] Transcript, page 892

9.8. However, Mr Goldfinch did not abandon his thesis altogether:

> 'J Goldfinch remains of the opinion that it is also possible that there may have been a failure of the western heel joint with the failure incorporating lateral buckling of the type described in detail by Mr Cooper and as indicated in the load testing carried out by Pryda.' [85]

9.9. Significantly, both men accepted that the lack of lateral restraint in the top chord of the failed double girder truss was fundamental to the failure of that truss[86].

10. <u>Further calculations - Mr Goldfinch's revised position</u>

10.1. On the basis of that agreement, Mr Goldfinch re-did the calculations using the Canadian model in TPIC-1996. He then discovered that his earlier calculations had been based upon incorrect assumptions, so that the 'overstresses' referred to in his earlier evidence were incorrect[87]. His new calculations were marked Exhibit C39a.

10.2. On the basis of those calculations, Mr Goldfinch's revised opinion was that the design of the failed double girder truss at Riverside was not adequate[88].

10.3. <u>The K_2 factor</u>

Mr Goldfinch said that, in 1995, the relevant design standard was AS1720.1-1988, to which I have already referred. This aspect of the Standard came into effect in 1993. The Standard states:

> '2.5.8 Material and application factor. For all timber members, the basic working stresses given in Tables 2.3 and 2.4 shall be multiplied by the material and application factor K_2, shown in table 2.11.' [89]

[85] Transcript, page 893
[86] Transcript, page 894
[87] Transcript, page 1248
[88] Transcript, page 1252
[89] Exhibit C34h

Table 2.11 is as follows:

TABLE 2.11

MATERIAL AND APPLICATION FACTOR

Consequence of failure classification*	Material and application factor (K_2)	
	Basis for assignment of structural properties	
	From in-grade verification	All other methods
Normal	1	1
High	0.9	0.7

10.4. The asterisk in table 2.11 refers the designer to the following explanation:

> 'Normal consequence of failure can be interpreted as that associated with housing construction, secondary framing in commercial or industrial scale structures and primary elements in farm buildings.
>
> High consequence of failure can be interpreted as that associated with primary structural elements in commercial or industrial scale structures, bridges and similar.'

10.5. The Pryda software used in 1995 by Mr Graham at Trussfab permitted only one option, a K_2 factor of 1.0[90].

10.6. Mr Goldfinch's original calculations were based upon the K_2 factor of 1.0. On that basis, he said that the design of the Riverside double girder truss was adequate[91].

10.7. However, his revised calculations were based on a K_2 factor of 0.7. This is the appropriate factor for 'primary structural elements in commercial or industrial scale structures' as described in table 2.11 above. On that basis, Mr Goldfinch said that he regarded the design of the Riverside double girder truss as inadequate[92].

10.8. I suspect that the Pryda software had simply not picked up the 1993 amendment to AS1720.1, and so it did not provide for the use of alternate K_2 factors for different building types.

10.9. Mr Clelland SC, counsel for Pryda, argued that the K_2 factor issue is a 'distraction'. He pointed out that if the same truss had been installed in a house, a K_2 factor of 1.0 would have been used[93]. I think that Mr Clelland's analysis is overly simplistic. In my opinion, table 2.11 in AS1720.1 emphasises the scale of the structure, rather than

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[90] See the annexure to Mr Cooper's report, Exhibit C34a, Appendix C

[91] Transcript, page 1254

[92] Transcript, page 1254

[93] Submissions, p3

its type. A double girder truss spanning 10.63 metres, or nearly 35 feet, is, in my opinion, a commercial scale structure. In my opinion, it is irrelevant whether it is part of a house, a golf clubroom or a factory - the issues are the same. If the Pryda engineers formed a contrary view, in my opinion they were in error.

10.10. I note that Professor Boughton shares this view[94].

10.11. Mr Goldfinch conceded that if the initiating cause of the double girder truss failure was lateral buckling, then he was unable to say that the K_2 factor would have made a difference[95].

10.12. On that basis, while I reject Mr Clelland's argument that the K_2 factor is a distraction, it may be, in view of the findings I will make, that it has not been a decisive factor in the causation of this tragedy.

10.13. Mr Goldfinch said that if Trussfab had used a K_2 factor of 0.7, and had used the correct loads as agreed between him and Mr Cooper, then there were overstresses of 51% adjacent to the western heel joint[96], whereas it was only 6% overstressed using a K_2 factor of 1.0. There was a 36% overstress in the top chord at the notch using K_2=0.7, whereas there was no overstress where K_2=1.0. This was the point where the shear failure occurred. At the point where the truncated girder truss from the old roof structure met the double girder truss, 2.9 metres in from the heel, there was a 54% overstress with K_2=0.7, but only 7% if K_2=1.0[97].

10.14. Mr Goldfinch said that if a K_2 factor of 0.7 had been used then in his opinion a stronger truss would have been designed, perhaps a triple girder truss, which may have withstood the forces at work[98]. He said that even using the original output data from the 1995 Pryda software, if a K_2 factor of 0.7 had been applied, these significant overstresses should have been apparent[99]. To this extent, he disagreed with Mr Cooper's comment in Exhibit C34a, Appendix C that even applying a 'commercial'

[94] Exhibit C47, paragraph 2.3.1
[95] Transcript, page 1333
[96] Transcript, page 1274
[97] Transcript, page 1275
[98] Transcript, page 1280
[99] Transcript, page 1289

building type, with a K_2 factor of 0.7, the design was ‘still satisfactory but closer to the maximum capacity’[100].

10.15. If it eventuates that a double girder truss built using K_2=0.7 would be no more liable to failure than the Riverside Golf Club double girder truss, then that would be more a matter of luck than good management on Pryda’s part, and reflects no credit on them. It was incumbent on Pryda to have modified its software as soon as the alternate K_2 factor came into force in the amendment to AS1720.1-1988 in 1993.

10.16. Mr Goldfinch encapsulated his revised opinions in Exhibit C39d, as follows:

- 'While I regard it as possible that the top chord fracture and failure of the two bottom chord web nailplates occurred as the double girder truss impacted with tables and chairs and the concrete floor actual load testing by Pryda on a full sized truss of the same configuration indicates that the top chord fracture and bottom chord nailplate failure most probably occurred prior to any impact with other objects.

  I accept that it is possible that fracturing of the top chord may have initiated the collapse.' [101]

- 'After discussion with Engineer, Mr Graham Cooper and revisiting the report of the CSIRO I am satisfied that stress distribution in the timber section *‘.. extending beyond the main heel nailplate at the western support end’* has such a complicated distribution of stresses in at least bending and shear that it is not possible to predict overstress in this region of the heel.' [102]

  'I am satisfied that prediction of heel member overstressing beyond the main heel nailplate is not possible and may only be able to be achieved by full scale tests on heels as suggested in the CSIRO report.' [103]

  However, he said that two areas of overstress in both top and bottom chord members adjacent to the western heel joint, but in-span (east) of the main heel nailplate, are established[104].

- 'I do not now completely *discount fracturing of the top chord panel as being an initiator or primary cause of the double girder collapse* … on the basis that this was able to be shown as an initiator of truss failure by full scale load testing performed by Pryda.

  Nevertheless, I maintain it is equally possible and plausible that either simultaneous failure of the western end heel initiating in the southern truss ply may have caused

[100] Transcript, page 1291
[101] Exhibit C39d, p1
[102] Exhibit C39d, p1
[103] Exhibit C39d, p2
[104] See diagram in C39, p10, as amended

fracturing of the top chord. I remain of the opinion that fracturing of the southern ply at the western end heel could have been the initiation of failure resulting in immediate uncontrolled lateral buckling and fracturing of the top chord.' [105]

- On either scenario, the failures could have been almost instantaneous, so that it may be only of academic interest as to which failed first (see also T1295);
- If the 2.8mm nail recommended in AS1720.1-1988 had been used, it would have been significantly overstressed[106]. His revised calculations confirm that a 2.8mm nail would be 27% overstressed. However, a 3.8mm nail, as recommended by Pryda, would not have been overstressed at all;
- 'Using the working stress method of analysis for the effectiveness of 2.8mm diameter batten nails I have found that at age 6 years 8 months (the time of failure) this sized nail would be 27% overstressed in single shear.

  I have also found that it would only provide an effective lateral restraint system in terms of its stiffness capacity if the batten nail joint at each truss top chord was 100% efficient. Any less efficiency than this would mean ineffective lateral restraint. Given the observed condition of split battens and nails too close to the ends of batten butt end joints nail efficiency in this case is deemed by me and Mr Cooper to be well less than 100%.' [107]
- If a K_2 factor of 0.7 rather than 1.0 had been used, it would have revealed significant overstresses (51% in top chord, 26% in another part of the top chord, 36% at the notch position, 51% in the bottom chord near the heel and 54% in the bottom chord where it met the truncated girder truss) in the Riverside Golf Club double girder truss. Using $K_2 = 1.0$, a significant overstress was indicated[108].

11. Further information

11.1. One result of the delays in finalising the inquest, because of the need to notify Boral, has been that there has been even further opportunity for re-consideration of these issues.

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105 Exhibit C39d, p2
106 Exhibit C39, p62
107 Exhibit C39d, p6
108 Exhibit C39d, p7

11.2. Thus, on 25 May 2005 I received from Mrs Sheppard, counsel assisting me, a copy of a facsimile message from Mr Goldfinch in the following terms:

> 'It has recently come to my attention that a 'carbon copy' failure at heel joint positions (and not lateral buckling failure of the top chords) of 4 double ply hardwood girder trusses has occurred at a commercial property in Elizabeth.
>
> The similarities with Riverside are:
>
> - Two ply trusses made of Tasmanian Oak F17 timber sourced from the same timber mill in Tasmania
> - Top chords 140mm & bottom chords 190mm
> - Top chords notched and cut down to 120mm seated on top of fully scarf cut bottom chords at heel joints
> - Heel joint failure or incipient failure pattern is identical to CSIRO testing and Goldfinch failure thesis
> - Heel joint failure has occurred around 6+ years after installation.
>
> These 'new' trusses actually span further than at Riverside. The tile battens are intact in this case and it appears the girder trusses have not fallen down because of the fortuitous placement of internal walls providing surrogate support. These trusses are now propped at the failed heel positions.'

11.3. This further information gives rise to a dilemma. Do I delay the finalisation of this inquest further, until the full circumstances of this 'carbon copy' truss failure is investigated and compared with the Riverside Golf Club collapse, which would take many months, or do I proceed on the evidence I have now?

11.4. Clearly, Mr Goldfinch is implying in his message that the absence of lateral buckling failure of the top chords is support for his original position, rather than the position which he adopted during the inquest.

11.5. The fact remains that there was clear lateral buckling failure of the top chords of the Riverside double girder truss, and Pryda have demonstrated that if the truss had been adequately braced this would not have occurred.

11.6. I have therefore decided to proceed on the evidence presently before me, because I anticipate that little will be gained by further investigation. Clearly, this is an area which calls for systematic and well-planned research, and the analysis of anecdotal cases is unlikely to be conclusive in the short-term.

12. Conclusion as to the mode of failure

12.1. On the basis of the above evidence, taken as a whole, I conclude, on the balance of probabilities, that Mr Cooper's explanation, supported to the extent that it has been by Mr Goldfinch and Professor Boughton, for the failure of the double girder truss at Riverside Golf Club on 2 April 2002 is the correct one.

12.2. Although Mr Goldfinch's alternative theory, that the heel shattered first, is a 'possibility', as he says, my finding is based upon my view of which is the more probable of the two theories. I do not reject the alternative theory out of hand. However, supported as it is by Pryda's full-scale testing, I am satisfied on the balance of probabilities that Mr Cooper's theory is correct, and so I adopt it for the purpose of these findings.

12.3. On that basis, I find that:

- Due to inadequate lateral bracing of the double girder truss, which was heavily loaded from the bottom chord rather than, as is more usually the case, on the top chords, the top chord began buckling into an 'S' shape;
- This bucking steadily increased over time (the process of 'creep') thereby progressively increasing the compressive forces on the top chords;
- Because the bottom chord of the double girder truss was fixed by steel truss bolts where the other trusses met the double girder truss at right angles, it was prevented from rotating in sympathy with the distortion of the top chords;
- The top chord buckled so far that it twisted upwards from the heel joint and failed due to torsion (twisting) both in the area of the top chord where the brittle wood and high slope-of-grain were found, and also in the vicinity of the heel joint which was weakened by hidden gum veins;
- The whole weight borne by the truss would then have been forced onto the small portion of bottom chord remaining which then failed in shear.

12.4. For the above reasons, while I accept Mr Goldfinch's evidence that the double girder truss was overstressed in several important areas, due mainly to the fact that a K_2 factor of 1.0 was used rather than 0.7, I am not satisfied that this was causative of the collapse. The Pryda testing established that, if the double girder truss had been adequately laterally restrained, it would not have failed on 2 April 2002. Whether it would have failed later, as mechano-sorption progressed as the timber aged, is a matter for speculation.

12.5. It follows that the lack of adequate bracing of the double girder truss was the principal cause for the collapse of the double girder truss on 2 April 2002. Even though there was overstress in the design, this has not been proven to have been causative of the collapse. Other factors, such as defects in the timber have not been clearly proved, and no conclusion can be drawn as to whether, and to what extent, they might have exacerbated the situation.

13. Issues arising at inquest

13.1. Installation of tile battens

Having accepted that the principal factor in the collapse of the double girder truss at the Riverside Golf Club was the lack of lateral restraint in the top chords of the double girder truss, it is necessary to examine the circumstances in which this came about.

13.2. It was common ground, among all the witnesses who gave evidence on the subject, that tile battens have traditionally provided the principal means of lateral restraint in a truss roof.

13.3. Battens, which are comprised of lengths of hardwood with a cross-section 20 x 40mm (¾" x 1½"), are nailed at right angles to the trusses at a spacing of 330mm or so. The roof tiles are lipped so that they sit on, and can be fastened to, the battens.

13.4. AS1720.1-1988 requires that, where the nail hole has not been pre-bored, the nail should be hammered in at a distance which is not less than 20 times the diameter of the nail from the end of the batten.

13.5. Compliance with this standard would require that nails should be hammered at the following minimum end distances:

Diameter of nail	Minimum end distance	Comments
2.8mm	56mm	Size of nail provided by Monier to Mr Fenech and used by him
3.15mm	63mm	Used by Pickard's employees in nail guns
3.75mm	75mm	Required in Pryda Manual, Exhibit C34, Appendix E, 'Roofing Battens'

13.6. Australian Standard 2050-1995 (Installation of Roof Tiles), came into effect in January 1995, the year this extension was built. This seems to have been regarded as the Standard applicable to Mr Fenech's operation. The Standard has the following features which are relevant for present purposes:

- It applies only to roofs with a pitch of 15º or greater (so the Riverside Golf Club roof was not covered by the Standard). Where the roof pitch is less, the manufacturer's advice should be sought;
- Paragraph 3.2.5 specifies:

> '**Batten joins**. All batten joins shall be such that the structural integrity of the batten is maintained. All batten joins shall be staggered so that each grouping of three battens does not contain any more than one join on the one rafter, with the exception of steel battens which may be lapped over the same rafter or joined between rafters using a mechanical joiner designed to comply with AS1170.1 and AS3623.' [109]

- Appendix D, which sets out tables for acceptable fastenings, including nails, only applies to battens being fixed to <u>rafters</u>, and not to <u>trusses</u>;
- Nails measuring 2.8mm x 65mm are specified for <u>rafter</u> spacing up to 600mm (as here). There is no specification for truss roofs. For rafter spacing greater than 600mm, nails of 3.15mm x 75mm are specified;
- At no point does the Standard refer to the role played by tile battens in providing lateral restraint for trusses. Indeed, Appendix B refers to 'permanent and temporary braces' being installed by other trades prior to tiling.

13.7. Clearly, it is not possible to comply with both AS1720.1-1988 and AS2050-1995 in relation to a truss roof (even if the Standards did apply to truss roofs). The obligation to join battens over the centre of a truss, and the obligation to nail at least 56mm back from the end of the batten, are in direct conflict. Trusses are customarily only 35mm wide. These Standards were obviously developed with rafter-roofs in mind.

13.8. As Mr Goldfinch observed:

> 'So, pray tell, how could a tile batten fixer make a splice, if he had to, over the top of a 35 mm piece of timber when he has to be 56 mm away from the end. So we might think, well, okay, we'll adopt the alternative given in the timber engineering code, AS1720, and we'll pre-drill the holes. That gives the leniency of saying that we would need to be 10 nail diameters away from the end of the piece of timber, namely in this case, 28 mm. He

[109] Exhibit C33h, p10

> would still fall off the midpoint of a 34 or 35 mm batten, being seventeen and a half millimetres.' [110]

13.9. In my opinion, the only sensible conclusion which can be drawn from this is that AS2050-1995 does not apply to truss roofs. If that is so, then there is no applicable standard for truss roofs, which is a very unsatisfactory state of affairs.

13.10. Even if the tile battens were fixed with the 2.8mm diameter nails supplied by Monier perfectly, Mr Goldfinch's calculations reveal that there is little 'factoring for safety' in this method of lateral restraint. He said:

> 'That tells me that our 2.8 mm diameter nail as used to fix the nail battens; if it was 100 per cent effective, would just get by, but nothing can go wrong. There must be no splits in the timber, there must be the correct nail spacing away from the end of the piece of timber. There must be all battens to the correct size, the truss must be true and straight when it is first installed. The bracing, the speed brace must be perfect; it must be the perfect design world for that to work.' [111]

13.11. The evidence, however, clearly reveals that the tile battens were not fixed perfectly, far from it.

13.12. Mr Ian Fenech was the roof tiler who installed the tile battens and tiles at the Riverside Golf Club in 1995. He had 32 year's experience in roof tiling, although he had no formal training[112]. He was an independent contractor to Monier PGH who had in turn contracted with Mr MacKenzie, the builder. Mr Fenech said he was simply given the address of the job by Monier. He provided his own tools and equipment. Monier supplied the battens, tiles, sarking, nails, cleats and whatever else was required for the job.

13.13. There is no evidence before me that Monier exercised any supervision over Mr Fenech. Mr Fenech was unable to recall whether their supervisor attended at the job, although he said that the supervisor 'sometimes' got up on the roof to inspect his work[113].

13.14. Mr Fenech said he had never seen the Pryda Roof Truss Erection Manual in all his years of working as a roof tiler[114].

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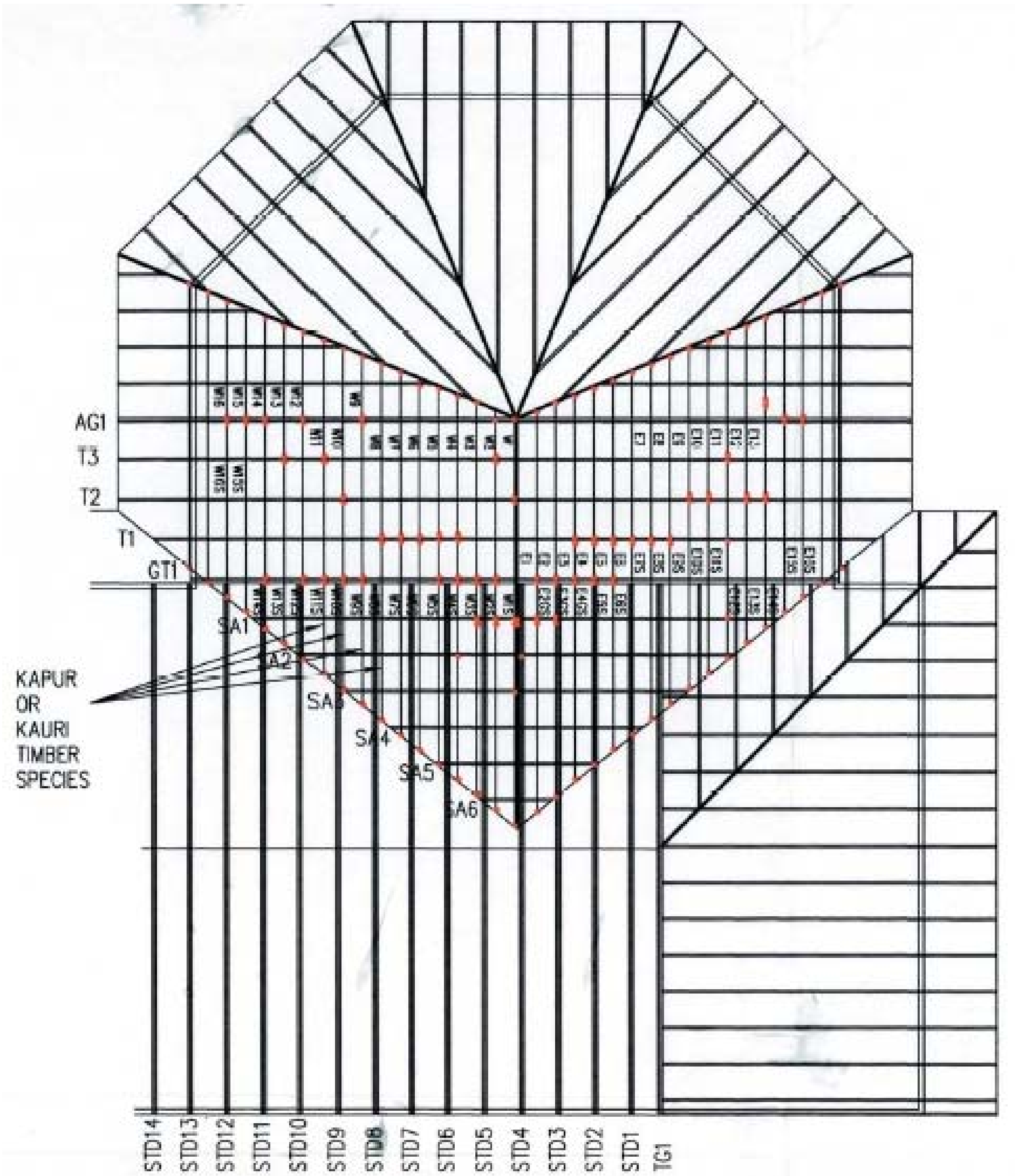
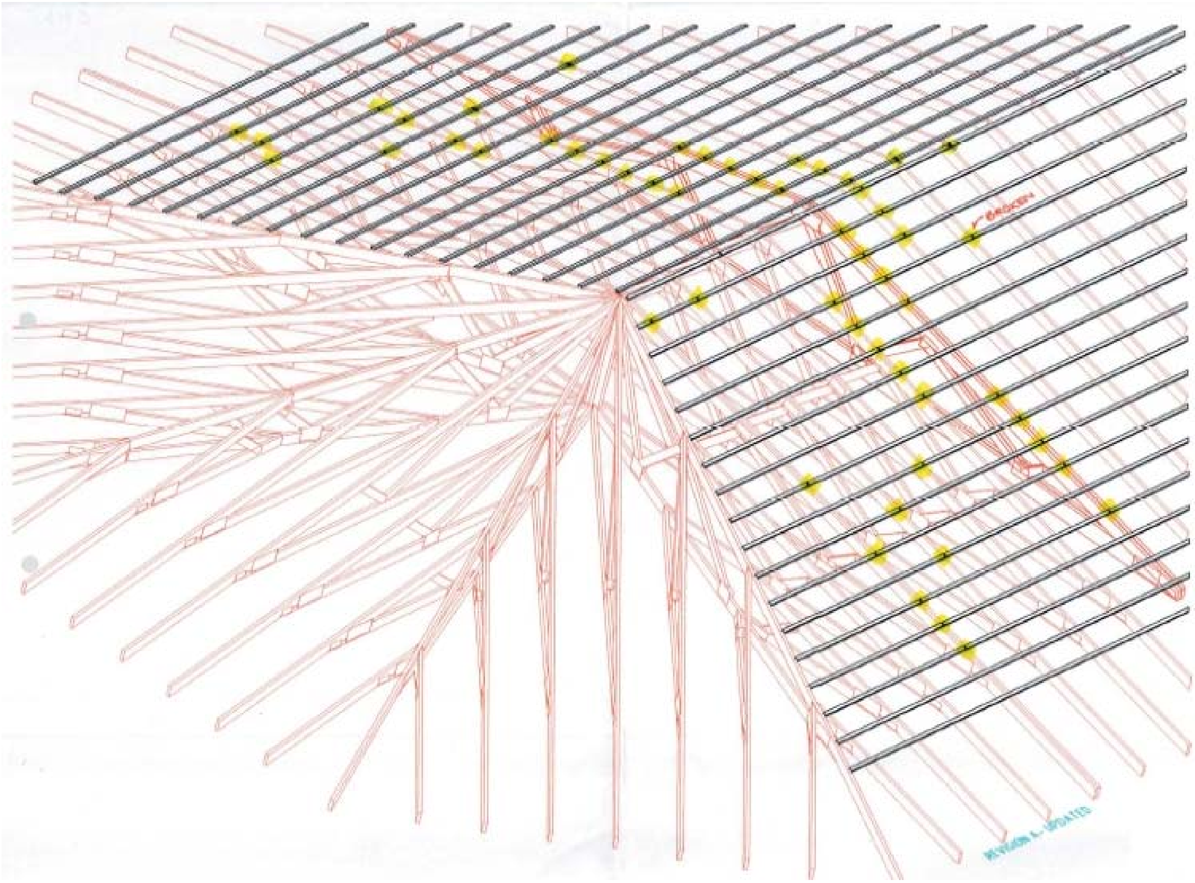
[110] Transcript, page 901
[111] Transcript, page 903
[112] Transcript, page 728
[113] Transcript, page 751
[114] Transcript, page 754

13.15. The evidence of Mr Goldfinch and of Mr Cooper satisfies me that there were a number of respects in which the attachment of roof battens to the double girder truss in this case was unsatisfactory. The evidence of Mr Mark Pickard, the Proprietor of Trussfab, who assisted Mr Goldfinch in the reconstruction of the structure after the collapse, vividly illustrates the defects in the attachment of roof battens. In Exhibit C41a and Exhibit C41b, diagrams produced by Mr Pickard, it can be plainly seen how many splices (joins) there were[115] (indicated by red dots).



[115] Exhibit C41a appears above and Exhibit C41b appears on the next page



13.16. Mr Pickard described the number of splices on the double girder truss as 'very inappropriate … terrible … disgusting'[116].

13.17. Mr Fenech had some difficulty explaining the number of split batten ends, and the number of splice joints which are clearly evident in the photographs Exhibits C36a and C41c. He argued that:

- The splits only occurred as a result of the roof collapse when the falling trusses pulled the nails through the ends[117];
- The ends visible in the photographs were not ends at all but broken battens caused by the collapse[118].

13.18. The evidence of Mr Goldfinch and Mr Cooper referred to above, and that of Mr Pickard at T1176 clearly contradict Mr Fenech's evidence, and I reject his evidence wherever it conflicts with theirs.

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[116] Transcript, page 1164

[117] Transcript, pages T749, T800

[118] Transcript, page 810

13.19. On the basis of that evidence, I find that the way in which Mr Fenech installed the tile battens at the Riverside Golf Club in 1995 was very unsatisfactory. Many of the tile battens had split ends where they were nailed, rendering them ineffective, and there were far too many splices on the double girder truss thereby providing insufficient lateral restraint.

13.20. Mr Fenech told me that any defects in the way in which he fixed the battens to the trusses in this case were 'standard practice' and unavoidable having regard to the difficulties I have outlined above. He said:

> 'Yes well they said they weren't nailed properly but, you know, that's why it's standard practice. It's just - it's the way it's done. If that wasn't nailed properly there's a lot of roofs in Adelaide are going to fall down.'[119]

13.21. It is not possible to verify the accuracy of these comments without dismantling a number of roofs at random in Adelaide and checking the nailing of the tile battens.

13.22. Mr Pickard led me to believe that he would not allow that standard of work from his tiling gangs, and that he instructs his supervisors to get up on the roof and check the tilers' work assiduously[120]. It is impossible to know how many other employers take this approach.

13.23. As I have already observed, AS2050-1995 is even ambiguous about whose job it is to install the tile battens, if they are to be described as bracing. The advisory section entitled 'Appendix B - Information and work not normally provided by the roof tiler', Section (c) 'work by other trades' states:

> '(i) Installing all rafters and trusses and all permanent and temporary braces.' [121]

13.24. At least in this section of the Standard, trusses are referred to. It would appear that the Standard has kept up with the advent of trusses at least to this extent. If tile battens are the principal source of lateral restraint or bracing in a truss roof, it was argued that this is not the responsibility of the roof tiler. Others argued that this advice is merely directed at the installation of speed bracing, which is a responsibility of the roof carpenter, in this case Mr Stocco. I am sure that Mr Stocco would have

[119] Transcript, page 756
[120] Transcript, page 1186
[121] Exhibit C33h, p13

been surprised if it had been put to him that it was his responsibility to install the tile battens.

13.25. As Mr Goldfinch said, the current state of Australian Standards and general industry understanding in this area leaves much to be desired. The ambiguities and contradictions make them difficult if not impossible to follow. The responsibilities of each trade, disparate as they are, are not clear. Mr Pickard acknowledged this, and said that he intended to discuss with Mr Goldfinch how these issues can be addressed now that he realises how critical they are[122].

13.26. The CSIRO report stated:

> 'In a related study (Leicester 1976) it was shown that for trusses loaded on the top chord, the friction between the loading battens and the top chord was adequate to provide a rigid restraint for trusses. It may be that for this reason it has not in general been necessary to place much attention to top chord fixings. However, in the case of girder trusses, which are predominantly loaded on the bottom chord, there is no such friction restraint on the top chord, and consequently extreme care must be taken in the case of critical trusses to ensure that lateral restraints have both the strength and stiffness required for the top chord to perform satisfactorily.' [123]

Mr Goldfinch agreed, saying that 'all of us .. have been living in a fools paradise' on this issue[124].

13.27. Lateral bracing is a particularly critical issue when, as here, a heavily loaded double girder truss is involved. The issue of lateral restraint of the top chords, particularly where it is the bottom chords of the truss which are loaded, is critical. In my opinion, it is completely unsatisfactory for the designers of such roofs to leave such a critical issue to the vagaries of disparate tradesmen, in the hope that they will comply with such ambiguous and contradictory standards, and in the knowledge that if those standards are not complied with to the letter, the truss may fail. This is so even if a catastrophic failure, such as the one in question here, is unprecedented.

13.28. In my opinion, it is extraordinary that this had not been recognised as an engineering issue before the events of 2 April 2002. Companies such as Pryda have been involved in the manufacturing of roof trusses for 30 years or so, and millions of these trusses

[122] Transcript, page 1457
[123] Exhibit C39, Appendix 5, p36
[124] Transcript, page 1263

have been fabricated. Have they always assumed that the trusses will be perfectly created and perfectly braced by the roof tiler? Were they not aware that AS2050-1995 did not appear to apply to truss roofs, or that it was in total conflict with AS1720.1-1988 if it did? This is not the cautious approach and 'factoring for safety' which the community is entitled to expect from engineers and from the building industry generally.

13.29. This criticism applies with equal force to Monier. They are a very large company with extensive engineering resources. It was also incumbent on them to ascertain the extent of their responsibility when contracting to install tile roofs, and to adequately supervise their subcontractors to ensure that these responsibilities are discharged.

13.30. It is clear from the evidence in this inquest that the building industry is highly compartmentalised, and that each specialised area is becoming more specialised as time goes on. This may have resulted in less communication between the respective 'compartments' rather than more. It is to be hoped that this has not led the truss designers and manufacturers to think that it is not their problem if the trusses are not installed correctly. In my opinion, it is incumbent upon companies such as Pryda and Trussfab to design the roofing system, as a whole, in a way which takes account of how it is to be installed, and how it is to be braced. They should factor in a sufficient margin of safety to take account of potential errors in these processes. I suspect that the only way this can be done is by designing a bracing system that does not rely on tile battens.

13.31. Roof pitch

It was common ground that the existing pitch of the roof at the Riverside Golf Club was 14.7º. The plans drawn by Mr Matthews called for the extension to be constructed so that the roof pitch matched the existing roof[125]. The precise pitch was not specified on the plans.

13.32. In fact, when the trusses were constructed by Trussfab, the pitch was 13.5º, which is significantly lower.

[125] Exhibit C30a

13.33. The evidence suggests that this pitch was arrived at by Mr Graham when he measured up for the construction of the trusses. He explained that the pitch of the extension roof was designed so that it was slightly lower than that of the existing building, despite the intention of the architect, because it comprised a 'Scotch valley'. If the peak of the Scotch valley was at the same level as the existing roof, he argued that it would be more difficult to waterproof[126]. Mr Pickard confirmed that this was standard practice[127].

13.34. Neither Mr MacKenzie, the builder, nor Mr Hall, the representative of the owner, knew anything about this. It was not until the roof had been installed that they became aware that the peak of the extension was at a different height, by which time it was probably too late to do anything about it[128].

13.35. The low pitch of the roof gave rise to a disagreement between Mr Cooper and Mr Goldfinch as to the appropriateness of the design of the heel joint in the double girder truss. Mr Graham, the Manager of Trussfab who was probably responsible for designing the truss, said that the Pryda computer program provides a number of options for heel cuts. The most commonly used design is a 'scarf' cut which was most suitable in this instance because:

> 'We had a height issue with the heel to make sure that the new truss didn't actually stick out past the line of the existing roof. So we had a heel height issue where we had to get the new truss to match in with the existing truss.'[129]

13.36. Mr Goldfinch said that there were a number of other options available, and diagrams for two of those options as well as a demonstration of a scarf cut are reproduced over the page[130].

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[126] Transcript, page 1037
[127] Transcript, page 1144
[128] Refer Transcript, pages 147, 231 and 232
[129] Transcript, page 1035
[130] Exhibit C39, Appendix 3

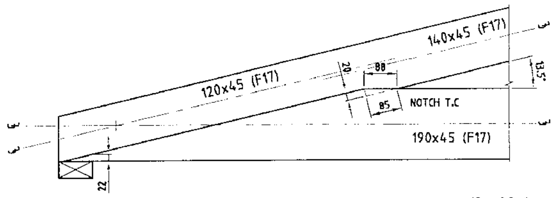

FIGURE 5: SKETCH OF FAILED DOUBLE GIRDER TRUSS (Scarf Cut)

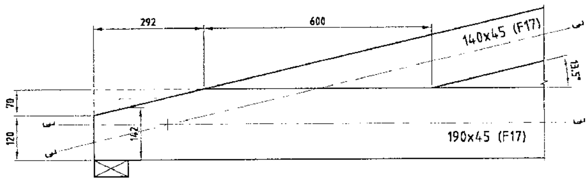

FIGURE 6: SKETCH OF ALTERNATIVE DOUBLE GIRDER TRUSS CONFIGURATION (FULLER BOTTOM CHORD)

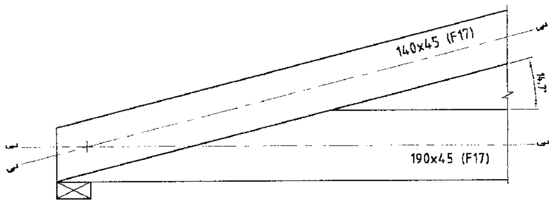

FIGURE 7: SKETCH OF HIGHER PITCHED TRUSS AT CORRECT ANGLE OF 14.7°

13.37. It can be seen that in relation to the scarf cut demonstrated in Figure 5 above, there is only a thin sliver of timber from the bottom chord which becomes progressively narrower as the point of the heel joint is approached.

13.38. Both Mr Cooper[131] and Mr Pickard[132] confirmed Mr Graham's evidence that a scarf cut was appropriate in these circumstances.

13.39. Mr Goldfinch explained that at the point where the heel truss fractured, the timber from the bottom chord was only 22mm deep (marked), and once it fractured it transferred all of the stress into the top chord thereby setting up a 'complicated distribution of shear, bending and axial stresses which ultimately applied a tension stress perpendicular to the grain of the top chord member and once this had manifested, total failure occurred'[133].

13.40. Mr Goldfinch pointed out that the Truss Plate Institute of Canada Design Manual (he referred to this since there is no relevant Australian Standard on the subject) dictates

[131] Transcript, page 670
[132] Transcript, page 1149
[133] Exhibit C39, p83

that there should be a minimum timber depth of 100mm over the support where a scarf cut bottom chord is used. The 22mm in this case was below this minimum requirement. He was unsure whether this TPIC standard existed in 1995[134].

13.41. Mr Goldfinch said that he was unable to demonstrate that this was a cause of the failure of the double girder truss heel. He said:

> 'Only by future testing will discovery be made as to the better performance the author would anticipate from this different configuration (ie scarf cutting the top chord rather than the bottom chord which would have given 142mm above the support rather than 22mm).' [135]

13.42. Mr Cooper agreed that more research is needed on this topic[136].

13.43. <u>Independent verification of roof design – role of Local Government</u>
As I have mentioned in the chronology in paragraph 4.3, the City of Hindmarsh and Woodville, now the City of Charles Sturt, required, as a condition of the development approval granted on 3 July 1995, that 'computations for roof trusses be submitted to Corporation for approval before that stage of the work is reached'[137]. This condition was not complied with.

13.44. The only possible reason for the imposition of that condition was that the Corporation required an opportunity to check the computations in order to independently verify their adequacy. The Building Surveyor, Mr John Mazzarolo, is a structural engineer and qualified to consider such issues.

13.45. There was no request made to Trussfab to supply the computations. Mr MacKenzie said he was aware of the requirement but left it to Mr Hall[138]. Mr Hall assumed that Mr MacKenzie had done it[139]. Mr Mazzarolo confirmed that the computations were not supplied, and said that the condition was rarely complied with. He said:

> 'We never really got computations anyway.'[140]

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[134] Exhibit C39, p70
[135] Exhibit C39, p83
[136] Exhibit C39i, Recommendation 3.2
[137] Exhibit C31b, p54
[138] Transcript, pages T96, T104
[139] Transcript, page 336
[140] Transcript, page 387

13.46. Mr Graham, the Manager of Trussfab, told me that if he had been requested to do so, the computations would have been supplied[141]. Mr Pickard, the owner of the Company, confirmed this[142].

13.47. There seems to be some imprecision involved in the use of the word 'computations'. Mr Mazzorolo said that if he had merely received the 'input data' used by Mr Graham (length of span, pitch, type of roof, type of ceiling, etc), all he would have been able to do was verify that these figures were accurate from the working drawings[143].

13.48. Mr Mazzarolo said that it was accepted 'industry-wide' that these computer-designed trusses were adequate for the task[144]. Unless the building surveyor knew the assumptions that were made by the designers of the computer program, then he or she could do little to check the design itself[145]. It would seem that one example of such an assumption is whether the appropriate K_2 factor had been incorporated in the program or not.

13.49. If the computations had been supplied, Mr Mazzarolo said that he would probably have sent them to consultants to check them because he was too busy to do so himself[146]. Presumably, the consultants would have had the same limitations on their ability to check the program as he did.

13.50. The information that would have been supplied was that reproduced by Pryda using the 1995 software[147]. Mr Cooper confirmed this[148].

13.51. Before Mr Mazzarolo could exercise any degree of independent judgment about the adequacy of the roof design, he would have required a full roofing layout plan. Mr Goldfinch made this point in his article 'Timber Nailplate Roof Trusses Revisited'[149].

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[141] Transcript, page 1054
[142] Transcript, page 1153
[143] Transcript, page 436
[144] Transcript, page 417
[145] Transcript, page 397
[146] Transcript, pages 428, 429
[147] See Exhibit C34, Appendix B
[148] Transcript, page 559
[149] Exhibit C33g, p10

Mr Mazzarolo said that it had never been his practice to require the provision of such a plan[150].

13.52. Mr Mazzarolo told me that Corporations had emphasised the inspection of footings over the years. This was no doubt inspired by the litigation which has taken place in this area since the High Court decision in The Council of the Shire of Sutherland V Heyman (1985) 157 CLR 424. Local Government Corporations had not previously appreciated that a failure of a truss roof was likely[151].

13.53. Following the enactment of the Development Act 1993, the traditional obligation on Local Government to inspect building work as it progressed was de-emphasised. The concept of mandatory inspections was abolished, and self-regulation by builders was given greater emphasis. The obligation on a builder to certify that the building had been constructed in accordance with the approved plan was created. Such certification was necessary so that the owner could obtain a Certificate of Occupancy for the building. No such certification was provided here by the builder, and no Certificate of Occupancy was issued by the Corporation.

13.54. Mr Donald Freeman, the Manager, Building Standards and Policy, at Planning SA, said that it was up to each Corporation to 'self-manage' their risks, and that they still had an obligation to enforce building standards in their particular area. He said:

> 'While the Act and the Regulations did not originally specify a particular level of mandatory inspections, there was a clear intention that the local council, as part of its overall responsibility for orderly development in their area, would undertake compliance and enforcement activity. Hence councils have the ability to appoint authorised officers under Section 18 to investigate breaches of the Act and those officers have substantial powers to inspect and collect information under Section 19. A relevant authority (ie council) is also able under Section 84 to issue directions (verbal if necessary by an authorised officer) to correct breaches of the Act. Each council must determine the level to which it will undertake this work and they are afforded a protection from liability under Section 99 of the Act for any act or omission made in good faith after the development has been approved. The expectations on Councils to self-manage their risks in relation to compliance and enforcement activity (ie, inspections) was communicated and discussed both prior to and after introduction of the Act. This included the degree to which Councils could rely on Section 99 by having a well thought out policy on the levels and circumstances for inspections. Councils also have the option under Section 59 of specifying certain mandatory stages during the work when the

150 Transcript, page 448

151 Transcript, pages 467 to 469)

> Council must be notified. As previously mentioned, this gives Councils the opportunity, when notified, to direct that work stop pending an inspection by an Authorised Officer. There is an obligation on the Council that this inspection must be undertaken within 24 hours.' [152]

13.55. Instead of requiring a mandatory notification at the stage of construction when the roof was about to be installed, as it did with the footings, the City of Hindmarsh and Woodville granted a 'conditional approval' instead. Mr Freeman told me that his legal advice was that conditional approvals were a nullity[153].

13.56. Whatever the legal position, such conditions are of no value if they are not enforced by the Corporation. At no stage did Mr Mazzarolo or his staff check whether the conditions attached to the approval had been complied with. If the Corporation had imposed a mandatory notification, they probably would not have followed that up either. They did not even check whether a Certificate of Occupancy had been applied for or obtained. Even if they had, it would not necessarily have alerted Corporation that the computation had not been supplied[154]. Mr Mazzarolo said that since 1993, that had been the case with most buildings[155].

13.57. This is most unsatisfactory. If what Mr Mazzarolo has told me applies generally across the State, then Local Government is playing no useful independent role in overseeing building standards in South Australia. In my opinion, it is not to the point to say that even if the computations had been supplied, the outcome would not have been different[156]. In a building of this scale, which is providing facilities for the use of large numbers of people, somebody must take responsibility of the structural integrity of the roof design as a whole. I agree with Mr Goldfinch's observations as to the expanded role which Corporations should take in checking the viability and integrity of roof structures when he said in his report:

> 'A system must be put in place where truss fabricators supply at least the following information possibly by electronic transfer directly to Councils' Building and Development officers.
>
> - A full roof framing plan showing all or typical truss member lateral restraints and fixing details.
> - A truss diagram showing the magnitude and position of all truss loads with any point loads properly dimensioned regarding their distances from supports.

152 Exhibit C42, p5
153 Transcript, page 1476
154 Transcript, page 437
155 Transcript, page 474
156 Transcript, page 435

- An overall roof bracing plan showing how brace forces are to be transferred into the supporting sub-structure such as walls, beams, and columns and with verification of the overall stability of the structure satisfied by structural calculations provided by the truss designer, if suitably qualified, or by an engineer that is suitably qualified to provide such advice.
- A full and comprehensive set of structural calculations showing individual member designs as well as nailplate analysis of all different joint types.' [157]

13.58. I note Mr Freeman's evidence that the Development Act is under review. He said that the introduction of expiation notices to provide revenue should encourage Local Government to be more rigorous in enforcement of compliance with the Act. This remains to be seen.

13.59. Conclusions - roof design

The only conclusion that is available on the totality of this evidence is that there was no one authority which took responsibility for ensuring that the overall design of this roof structure was adequate. The software designer and truss manufacturer played no role in considering whether the lateral bracing of the truss was adequate, other than to provide the Pryda Roof Truss Erection Manual which provides some guidance to the truss installer, but which is not addressed to tilers. The builder did not supervise the work of the tiler who installed the tile battens, and nor did the principal roof contractor, or the tile supplier. The tiler himself was obviously only concerned with his area, namely the installation of the tiles. Understandably, he might consider that the integrity of the roof structure as a whole would have been the concern of somebody else. The Corporation simply had no information about any of these issues, and did not take any steps to find out.

13.60. Of course, having been supplied with that information, it is incumbent upon Corporations to put appropriate mechanisms in place in order to verify that these plans and specification are appropriate, and that the structure has been constructed in accordance with those standards as part of its ongoing risk management process. Corporations should require the builder to certify that the construction was in accordance with the specifications submitted, before issuing a Certificate of Occupancy. They should also ensure that a Certificate of Occupancy has been issued before occupation, and enforce the law if it has not.

[157] Exhibit C39, p130

14. Developments since 2002

14.1. Mr Graham, the former Manager of Trussfab, is now a private consultant who designs roofs for the building industry. He told me that, in his opinion, the degree to which tradesmen in the building industry are in demand, and modern building methods, have resulted in standards of workmanship getting worse, not better. He said:

> 'I see that there has been a trend with a lot of builders, a trend for them to head more towards administration roles than a technical builders role nowadays and consequently they rely on suppliers and various trades to meet certain criteria. And there is a lot more of a paperwork chase but not necessarily the attention to detail on site.'[158]

Mr Goldfinch expressed similar concerns. He said:

> 'Q. … irrespective of design issues, from your inspections in the field now, should we be concerned that there are a number of roofs out there that are unsafe and might collapse like this Riverside roof collapsed.
>
> A. Yes.'[159]

14.2. Mr Graham said that since the experience of the Riverside Golf Club collapse, he no longer places any reliance on tile battens to provide lateral bracing of a heavily loaded double girder truss, and now designs an independent bracing system[160]. He said that architects are now more frequently specifying that a roof should be certified by a structural engineer (such as himself) before the building is regarded as complete[161]. Such a certification should occur before the Certificate of Occupancy is issued. I agree that this would also provide an additional safety mechanism, so that any defects in either the design or assembly of a truss roof might be detected.

14.3. I was informed by Mr Walsh QC, counsel for Gerling Insurance, the insurers of Trussfab, that Mr Pickard no longer uses visually stress-graded timber for the construction of roof trusses, and has not done so for 3½ to 4 years. He told me that they now use either laminated veneer lumber (LVL) or machine graded pine. He told me that this timber can be provided in whatever grading is required. LVL is an engineered timber product made from parallel fibre plywood. He said:

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158 Transcript, page 1107
159 Transcript, page 1123
160 Transcript, pages 1111 to 1113
161 Transcript, page 1124

> 'It is therefore consistent in texture, it does not contain gum veins, brittleness, slope-of-grain problems - in other words it does not have any factor affecting its integrity as against natural timber.'[162]

Mr Walsh told me that LVL is used for larger structures such as chord members in girder trusses, and the machine-graded pine is used for less critical members.

14.4. <u>The Timber Nailplate Truss Review Committee report</u>

Following the State Conference of the Australian Institute of Building Surveyors in 2003, the above Committee was established to consider these issues. It comprised Mr Goldfinch, Mr Mazzarolo, Mr Claus Williger from the City of Port Adelaide and Enfield, Mr Peter Harmer of Katnich Dodd, private certifiers, and Mr Michael O'Callaghan of the Timber Development Association (SA Chapter).

14.5. On 15 June 2004 the Committee made the following recommendations:

> '1. Timber nailplate manufacturers should immediately work towards a user friendly computer software design program that can be readily checked by appropriately qualified engineers and certifiers. All nomenclature should be clearly described and explained along with clear diagrams and sketches, joint designs and setting out the number of nailplate "teeth" required per timber element per joint. At the moment these matters are not entirely clear in the computer software presentations being submitted for approvals.
>
> Mr Tadich has advised the Committee on behalf of the MiTek Company that this matter is currently being addressed. The Committee assumes that the other two nailplate manufacturers would be likely to follow suit.
>
> 2. On the topic of site installation issues the Committee recommends the use of a simple check list as recommended in Timber Development Association of SA Inc. publication "A Guide to: Bracing, Connection and Installation Details for Timber Nailplate Roof Trusses as Required by AS 4440-1997" (cost $15.00). A type example of an installation check list made up using this Guide was prepared by Committee member, Mr Michael O'Callaghan …
>
> 3. It is recommended that a check list of the type made up by Mr O'Callaghan on behalf of the Committee be made part of either Appendix A or B of AS 4440. The committee further recommends the checklist as an excellent means of record keeping for Builders when fulfilling their obligation to provide Written Statements for the granting of Certificates of Occupancy in the case of commercial and industrial structures of Class 2 to 9 inclusive and for domestic constructions of Class 1 (Regulation 83AB).
>
> 4. The Committee recommends that all truss manufacturers who have not already done so, should have a quality check list which, inter alia, should not only label

[162] Transcript, page 1616

trusses in accordance with the roof layout, but also identify the fabrication plant (e.g. T2A South Coast Trusses).

5. The Committee recommends that computer software truss design programs used by the nailplate manufacturers and in truss fabrication plants should be checked and certified by an independent expert party such as the CSIRO Division of Building & Engineering, Highett, Victoria. The committee comments that it would expedite the approval process if truss designs (not just the programs) were independently certified, this being the current approach adopted by the steel framing industry.

   In Sth. Aust. this would allow reference to Section 104 or Regulation 88 of the Development Act and Regulations by AIBS members and Private Certifiers to approve a building method and design "...accredited by a prescribed person or body ….." (refer Section 104(3) of the Development Act 1993). Part 1.2 of the Building Code of Australia Vol. 2 (Housing) would also be satisfied.

6. The Committee recommends that it is imperative that an Australian Standard pertaining to the design of timber nailplate roof trusses must now be completed and published as a matter of considerable urgency. Both the USA and Canada, at least, have fully comprehensive design codes specifically for timber nailplate trusses.

7. Significant clarification and simplification of AS1720.1-1997 Clause 4.2.3.2 in conjunction with Appendices C and E at C3 and E7.1 and E7.3.1 respectively, in relation to the analysis of batten nails as a means of effective lateral restraint to truss top chords must be undertaken by Standards Committee TM/1 as a matter of considerable priority. The current assumption made by designers of lateral restraint to top chord truss members by tile batten nails may be unconservative and dangerous especially in the case of primary structural support elements such as girder trusses carrying secondary trusses. These sections of AS1720.1-1997 are currently far from user friendly to the extent they are not being referred to by designers or certifiers.

8. The committee recommends that nailplate manufacturers, if they have not already done so, should commit to instituting testing programmes to find out more about,

   (a) The long term behaviour of truss heel joints for different timber species and for both standard and girder trusses under typical design loads with different exposure conditions.

   (b) The efficacy of 10mm plasterboard direct fixed to truss bottom chords at 600mm transverse centres to act as an effective means of lateral restraint for bottom chords.' [163]

14.6. These recommendations were the subject of comment throughout the inquest.

14.7. As part of their consultation process, Messrs Cooper and Goldfinch further considered the recommendations made and incorporated some of them in a series of joint

[163] Exhibit C33b

recommendations to me which are now Exhibit C39i. Their recommendations are as follows:

'1.0 Timber Quality

1.1 As the process of visual stress grading of timber is reliant upon human judgement it is always possible that some defects outside of the grading rules may be present in a length of timber assessed in this manner. Sometimes these defects may not be visible as can occur in the case of tight gum veins.

Hardwoods are commonly assessed by the method of visual stress grading by timber suppliers. For this reason the most appropriate means of taking account of defects in visually stress graded hardwoods may be to reduce the characteristic capacity of these hardwoods by lowering the capacity reduction factor "Ø" for primary structural elements as defined in Clause 1.8.2.20 of AS 1720.1-1997.

Any such change to the Timber Engineering Code AS 1720.1-1997 could be put into effect relatively quickly in the next amendment to that Standard.

1.2 We have given our consideration to effecting changes to Australian Standards AS 2082 and AS 4446 (even though this precludes roof trusses) to prohibit the use of timber with tight gum veins located at, or adjacent to, joints in timber engineered structures. These Codes currently permit an unlimited number of tight gum veins in nailplate jointed fabrications.

The major difficulty in dealing with tight gum veins however, is that they may not in all cases be readily visible and may therefore go undetected.

Because tight gum veins constitute a defect that may remain hidden within a timber member we regard the introduction of lower capacity reduction factors (Ø) as already recommended in Item 1.1 above to be a possible means of dealing with tight gum veins.

2.0 Tile Batten Fixing

2.1 We recommend that the existing rules set out in Australian Standards AS 2050, AS 1684 and AS 4440 should be brought into line with respect to nail lengths and diameters. These Standards already deal with the fact that it is only permissible to splice one in every three battens over any one truss top chord member.

2.2 It is recommended that truss designs assume that batten nailing at splice joints will be ineffective and will not be able to comply with the Timber Engineering Code requirements for nail end distances as set out in AS 1720.1-1997. For "engineered" timber structures designers should therefore proceed on the basis of calculating the lateral bracing force to be taken out by the batten nails assuming one in every three battens to be redundant (i.e. ineffective in transferring lateral buckling loads).

In cases where the truss designer requires greater than this level of batten nail efficiency and the standard nailing system is inadequate, then the truss designer would be responsible for providing alternative lateral restraint bracing to either supplement or take the place of the batten nail restraints.

It is recommended that this design requirement be incorporated in the proposed new Australian Standard AS 1684.5 for timber nailplate truss design (refer 3.1 below).

3.0 Design and Approval

3.1 We recommend that Standards Australia should do whatever is necessary to immediately finalise and publish an Australian Standard specifically for the design of timber nailplate roof trusses. Design recommendations for truss heel joints and their fabrication as set out in the Canadian Design Manual (TPIC-1996) or in some other equivalent international code of practice should be incorporated for girder truss heel designs.

It is understood that such a Standard in draft form is to be submitted to Standards Australia by the end of this year (2004).

3.2 Further testing of low pitch truss heel joints using different timber cutting configurations could perhaps be arranged on a bi-partisan approach by all three Australian Nailplate manufacturers in order to verify and establish design rules for girder truss heel joints.

3.3 It is also recommended that some long duration heel joint tests be done under the influence of service load only to determine whether or not mechano-sorptive creep effects are significant for nailplated joints that may be subjected to changes in humidity and temperature.

3.4 Satisfaction of requirements under the Development Act and Regulations 1993 in South Australia may be made simpler with the launch of the proposed AS 1684.5 design code for design of timber nailplate roof trusses. It is understood that the proposed new Code will contain unambiguous descriptions of all of the timber engineering design factors that are otherwise left open to interpretation in existing Standards AS 1170 and AS 1720.1-1997.

3.5 Concerning the matter of computer design software verification, Goldfinch and Cooper have different views. By mutual consent, resolution of this matter will be effected by Goldfinch and Cooper meeting with Planning SA representatives to determine a future strategy.

3.6 All timber nailplate truss roof designs should incorporate a clearly identifiable statement regarding the use of tile battens as an integral part of the structural roof bracing system in cases where they are designed to fulfil this role. Any alternative bracing system must similarly be clearly identified in the truss design and on the roof framing plan. It is recommended that these matters be dealt with in the proposed new design Standard.

3.7 Publication of a Checker's Manual prepared on a tri-partisan approach by the three Australian nailplate manufacturers would be of assistance to Council Building Surveyors and Private Certifiers.

It is envisaged that this Manual could feature type examples of a typical manual truss design check and an example of a nailplate joint design at say a heel and bottom chord splice and include a lateral restraint analysis for strength and stiffness of tile batten nails to illustrate in a user friendly manner the application of appendices C and E at C3 and E7.1 and E7.3.1 in conjunction with Clause 4.2.3.2 of AS 1720.1-1997.

This would assist Council Building Surveyors and Private Certifiers to check designs.

In relation to the desirability of a Checker's Manual it is understood that it could take the form of a Commentary to the proposed new truss design Standard AS 1684.5. The Commentary could cover bracing design rules and assumptions, giving examples of how these can be applied.

4.0 Fabrication

4.1 It is recommended that all timber nailplate truss roofs should incorporate the following as a minimum requirement.

- A roof framing plan clearly showing all permanent bracing of any kind, including lateral restraint bracing to truss top and/or bottom chord.
- A reference to AS 4440-2002 regarding rules for installation of timber nailplate truss roof systems.

4.2 All truss manufacturers should have a quality checklist in place to ensure trusses at the time of manufacture comply with all design and manufacturing quality control requirements including accurate plate placement and to ensure a roof framing plan is delivered to site in a protective covering that is clearly visible and securely attached in a prominent position at the time of delivery of trusses.

4.3 A minimum of say three trusses in any one roof assembly should be clearly marked with the truss fabricator's company name in a manner that will remain permanent (e.g. paper stickers for this purpose are unacceptable, but an ink stencil would be satisfactory).

4.4 All trusses should be labelled with their design designated nomenclature (e.g. T1A, TG1, ….. etc) for ease of identification on site and to prevent misplacement when referred to the roof framing plan. Any trusses requiring special conditions should be clearly identified e.g. special bracing requirements, irregular support conditions, etc.

5.0 Construction

5.1 The national builders' affiliate bodies, such as the Master Builders Association (MBA) and the Housing Industry Association (HIA) should implement, if they have not already done so, and maintain, appropriate training schemes for both roof tilers and first fix roof carpenters to ensure an up to date knowledge base is maintained within these trades with respect to trussed roof assembly.

5.2 Licensing and licence renewal of trades could perhaps be linked to proof of attendance at training seminars held at salient times, such as when new Standards are introduced or existing Standards are altered in any significant manner. Such a scheme could be implemented by both the MBA and HIA organisations as these bodies already have excellent training schemes in place however, at this point in time attendance is not compulsory and can be quite poor on occasions.

5.3 Completed timber nailplate truss roof structures should be inspected by a licensed works supervisor (in South Australia) or licensed builder. For this purpose we strongly recommend the reference text titled, "A Guide to: Bracing, Connection and

> Installation Details for Timber Nailplate Roof Trusses as Required by AS 4440-1997" (cost around $20). This is a publication of the Timber Development Association of South Australia Inc.
>
> This is a highly recommended reference text which contains, inter alia, clear and concise large scale diagrams of appropriate connection details along with a blank ready-made checklist for supervisors to fill in. Also included is a type example of how the checklist would be completed in typical manner by a suitably qualified inspector.' [164]

14.8. I note that Professor Boughton disagreed with the recommendations in paragraphs 1.1 and 1.2 of Exhibit C39i above, because he did not agree that gum veins played a part in the collapse. He said:

> 'It would be inappropriate to react to factors that were not critical to the collapse.'[165]

14.9. Professor Boughton also disagreed with Mr Goldfinch's recommendation that tight gum veins be prohibited at, in adjacent to, joints in timber engineered products, for the same reason.

14.10. In view of the level of disagreement among experts, I am not in a position to resolve these disputes on the information available before me. It is a matter best left to be determined by Standards Australia. They can take into account all relevant information, and assess it using their particular expertise and experience.

15. <u>Recommendations</u>

15.1. Taking into account the evidence before me, and in particular the views of the expert witnesses who have given evidence, I make a number of recommendations pursuant to Section 25(2) of the Coroners Act 1975.

15.2. <u>Computer software</u>

The Timber Nailplate Truss Review Committee recommended that all timber nailplate truss design computer programs should be 'user-friendly' and verifiable[166]. They also recommended that such programs should be certified by an independent body such as

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[164] Exhibit C39i

[165] Exhibit C47, paragraph 4.1.1

[166] Recommendation 1

the CSIRO. Mr Cooper disagreed, and it was resolved to discuss the matter further with Planning SA[167].

15.3. In my opinion, this is a decision for building certifiers and for Government. Certifiers have a duty to insist that they be able to check structural designs to ensure they are adequate. Designers and manufacturers must either fully disclose the necessary aspects of their software, or have it certified by an independent body so that a certifier may accept that certification pursuant to Section 104(3) of the Development Act. I do not see this as a matter for negotiation. It is for the certifier to set the criteria upon which approval will be granted, and they are accountable for the adequacy of those criteria. The owners of the software have a choice. Either they meet the certifiers criteria, or their customers do not get development approval.

15.4. Accordingly, I recommend that Planning SA, Local Government and private certifiers establish a system whereby a truss roof design may be evaluated appropriately. Such a system should involve either access to sufficient material in the computer design program to properly assess the design, or the acceptance of a properly certified program pursuant to Section 104(3) of the Development Act.

15.5. A new Australian Standard

The Timber Nailplate Truss Review Committee described the development of a new Australian Standard for the design of timber nailplate truss roofs as 'imperative', and 'a matter of considerable urgency'[168].

15.6. Messrs Goldfinch and Cooper agreed. They advised that a draft of a proposed new AS1684.5 was to be submitted to Standards Australia by the end of 2004[169]. I have no information as to the current status of the new Standard.

15.7. I recommend:

15.7.1. The proposed new AS1684.5 be finalised and published as soon as possible.

15.7.2. The current uncertainties and inconsistencies in AS1720.1, AS2050, AS1170 and AS4440 as they apply to truss roofs must be rectified.

[167] Recommendation 3.5
[168] Recommendation 6
[169] Recommendation 3.1

15.7.3. The new Standard should deal with the design of truss heel joints in a similar way to the Canadian TPIC-1996 design manual.

15.7.4. The Standard should require the development of a full roof plan designed on sound engineering principles. The design should incorporate adequate lateral restraint of trusses, particularly heavily loaded double girder trusses, with all appropriate structural calculations endorsed. The plan should also refer to AS4440 in relation to truss installation.

15.7.5. The design for lateral restraint should assume that tile battens are ineffective since even if they are perfectly installed, they do not provide an adequate margin for safety in engineering terms. In truss roofs, the battens cannot be installed perfectly because the truss thickness is insufficient to provide adequate nailing distances from batten ends in splice joints. Alternative bracing strategies must be developed.

15.7.6. The new Standard should incorporate, perhaps as an appendix, a 'Checkers Manual' to provide builders and certifiers with a guide to assist in assessing the adequacy of roof truss design and installation. This will also assist builders when they are required to certify the work.

15.7.7. The new Standard should provide a system whereby the roof plan is communicated to all who need to see it, including the builder and installer on site. In particular, a copy of the plan should be securely attached to the trusses in a protective cover and delivered undamaged to the site with the trusses.

15.7.8. All trusses and other components of the roof should be labelled with the manufacturers name and any other required information, and with a reference to that component in the roofing plan.

15.7.9. The new Standard should make it clear who carries the responsibility for the proper installation of a truss roof in accordance with the roofing plan, and for the supervision of such work, and the certification that the roof has been appropriately installed.

15.7.10. The new Standard should specify what types of timber are permissible for use in the manufacture of roof trusses. If it is not possible to ensure structurally sound timber by visual stress-grading (eg because gum veins or brittleness may not be visually identifiable), then visually stress-graded timber should not be permitted, particularly for primary structure elements.

15.7.11. Consideration should be given to reducing the capacity reduction factor 'Ø' (the successor to the K_2 factor) provided by AS1720.1-1997 when visually stress-graded timber is used in any structural element.

15.8. Role of Local Government

The role of Local Government in the granting of Development Act approval needs to be addressed. Since the passing of the Development Act in 1993, Local Government Building Surveyors seem to have taken a more passive role in the assessment of applications. I recommend that the Minister for Local Government conduct an assessment to ascertain the extent to which Local Government is not enforcing conditions imposed on grants of development approval, not enforcing the laws in relation to Certificates of Occupancy, not conducting an independent appraisal of the structural engineering aspects of proposed buildings, not requiring vital information about the structural integrity of the roof in a proposed building, assuming without verification the adequacy of commercially-available software programs for the design of structural components in a building, not carrying out random, or indeed any, inspection of building works, and not requiring an independent verification that the roof has been constructed in accordance with the plan.

15.9. If such omissions are widespread, then I recommend that the Minister for Local Government should consider how they might be addressed either in relation to reviewing the Local Government Act, reviewing the financial resources of Local Government to enhance their capacity to act, or in other ways.

15.10. Research

The Timber Nailplate Truss Review Committee recommended that further research into the design of timber nailplate truss heels is required[170]. Messrs Goldfinch and Cooper agreed[171], as did the CSIRO[172]. I also agree.

15.11. I recommend that further research be performed, perhaps by the three major timber nailplate manufacturers in cooperation with the CSIRO, into the design of heel joints

[170] Recommendation 8
[171] Recommendation 3.2
[172] Exhibit C39, Appendix 5, p47

in girder trusses, taking into account different timber species, and different climatic conditions, over an extended period.

15.12. Training

Messrs Goldfinch and Cooper recommended that builders' affiliate bodies such as the Master Builders Association and the Housing Industry Association conduct training schemes for roof tilers and first fix roof carpenters to ensure that their knowledge base is up to date, particularly when Standards are replaced or altered[173].

15.13. They also recommended that attendance at such training should be a condition of renewal of their building licences.

15.14. I agree, and recommend that the feasibility of those suggestions be explored, both by the South Australian Government who, through the Builders Licensing Board, issues the licences, and by the Master Builders Association and the Housing Industry Association respectively.

15.15. Generally

I recommend that all participants in the industry concerned with the design, manufacture and erection of truss roofs be reminded that they all should carry responsibility for the integrity of the roof as a whole. In particular, in accordance with ordinary engineering principles, designers and manufacturers of roof trusses should incorporate into their designs a system for the lateral restraint of roof trusses, particularly heavily-laden girder trusses, which has a reasonable margin of safety. The system should allow for and guard against defective or inadequate installation rendering the roof construction potentially unsafe.

Key Words: Roof Collapse; Building Industry; Local Government; Development Act; Structural Engineering; Roof Trusses

In witness whereof the said Coroner has hereunto set and subscribed his hand and

Seal the 1st day of June, 2005.

Coroner

Inquest Number 35/2004 (0847/2002) & 36/2004 (0848/2002)

173 Recommendation 5.1