# RESEARCH AND DEVELOPMENT TRENDS IN STRUCTURAL APPLICATIONS OF TIMBER FOR EXPANSION OF THE NON-RESIDENTIAL MARKET FOR FOREST PRODUCTS IN AUSTRALIA

KEITH CREWS

1990 GOTTSTEIN FELLOWSHIP REPORT

# JOSEPH WILLIAM GOTTSTEIN MEMORIAL TRUST FUND

The Joseph William Gottstein Memorial Trust Fund was established in 1971 as a national educational Trust for the benefit of Australia's forest products industries. The purpose of the fund is *"to create opportunities for selected persons to acquire knowledge which will promote the interests of Australian industries which use forest products for the production of sawn timber, plywood, composite wood, pulp and paper and similar derived products."*

Bill Gottstein was an outstanding forest products research scientist working with the Division of Forest Products of the Commonwealth Scientific Industrial Research Organization (CSIRO) when tragically he was killed in 1971 photographing a tree-felling operation in New Guinea. He was held in such high esteem by the industry that he had assisted for many years that substantial financial support to establish an Educational Trust Fund to perpetuate his name was promptly forthcoming.

The Trust's major forms of activity are,

1. Fellowships - each year applications are invited from eligible candidates to submit a study programme in an area considered to be of benefit to the Australian forestry and forest industries. Study tours undertaken by Fellows have usually been to overseas countries but several have been within Australia. Fellows are obliged to submit reports on completion of their programme. These are then distributed to industry if appropriate.

2. Seminars - the information gained by Fellows is often best disseminated by seminars as well as through the written reports.

3. Wood Science Courses - at approximately two yearly intervals the Trust organises a week-long intensive course in wood science for executives and consultants in the Australian forest industries.

4. Study Tours - industry group study tours are arranged periodically and have been well supported.

Further information may be obtained by writing to,

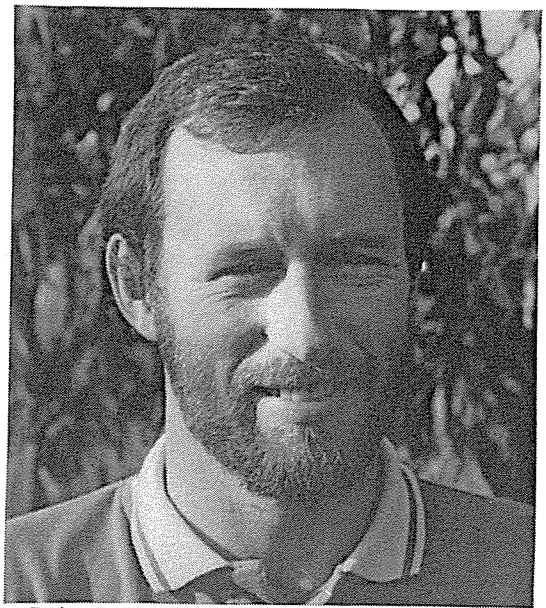
The Secretary,
J.W. Gottstein Memorial Trust Fund,
Private Bag 10,
Clayton, Victoria,3168 Australia

# GOTTSTEIN REPORT

A Study Tour Investigating Research and Development Trends in Structural Applications of Timber for expansion of the Non- Residential market for Forest Products in Australia

July, 1992

Keith Crews
Gottstein Fellow, 1990



Keith Crews is a Sydney based Consulting Structural Engineer, specialising in the design of timber structures. At the time of applying for a Gottstein Fellowship, he was living and practicing at Taree, on the mid north coast of NSW.

He is also now the co-ordinator for education and research for Timber Engineering at the University of Technology, Sydney, where he is involved in post-graduate research into behaviour and design of large spanning timber structures, including connection design.

Presently, his main Consulting work centres on development of rehabilitation and design procedeures for timber bridges, including stress laminated timber bridge decks. In this work he consults to the timber industry and is engaged as Principal Consultant to the Roads and Traffic Authority, NSW for the Timber Bridge Initiative Research and Development program.

The Gottstein Fellowship has enormous impetus for both the research and consulting work Keith is currently undertaking, as well as providing the opportunity to establish an invaluable 'network' of researchers and collegues in overseas Universities and Timber Research establishments.

The implementation of Keith's findings has already made a significant contribution to the acceptance of timber bridges, both for rehabilitation and for new construction, in NSW. The knowledge gained from the Study Tour and presented in this report will also be used as an integral part of future educational strategies in the Civil and Structural Engineering degree courses at the University of Technology, Sydney and hopefully will assist the development of Timber Design education in other Universities throughout Australia.

## EXECUTIVE SUMMARY:

It is anticipated that the information presented in this report will form the basis of future research projects and the development of design procedures and design aids for Architects and Engineers, both at undergraduate and graduate levels.

Recommendations for its use and application are presented in Section 1 of the report and are summarised below:

(i) This study highlights the importance of *technology transfer* as a means of applying the "state of the art" timber technology in Australia and avoiding unnecessary duplication of research, wherever possible.

(ii) Linked to (i), is the need for an information infrastructure to facilitate communication of research findings

(iii) It is imperative that a forum for the determination of research priorities be established, involving the industry, NAFI and promotional organisations and relevant research establishments, such as CSIRO and Universities.

(iv) The establishment of recognised Centres for Excellence in timber design, involving education, research and specialist consulting, and reflecting the needs of industry is imperative. Furthermore, "networking" of all such centres involved in timber research is essential.

### Implementation of Study Findings:

Whilst some of the findings of this report may not find immediate application and are concerned with longer term benefits to the timber industry, a significant proportion of the findings from this Gottstein Fellowship have already had implementation.

The main initiative in this regard is the construction of Australia's first hardwood stress-laminated bridge during the beginning of December 1991, at Yarramundi Lagoon, near Richmond, NSW. Similarly, implementation of epoxy injected dowel connection technology has also been successfully undertaken in January, 1992 in a Sydney church auditorium. The importance of this immediate application must not be understated nor missed by the industry.

The Gottstein Fellowship has allowed the author to both report on overseas developments and also implement significant and greatly beneficial research findings within a 12 month period of undertaking the study tour.

# CONTENTS:

# SECTION 1:

# GOTTSTEIN REPORT

*Background, Introduction, Scope and Itinerary*

**Background and Introduction:**

It has been widely recognised, particularly over the last few years, that timber is an under-rated and under-utilised material in the non-residential building market. From the author's perspective as a Structural Engineer, it is "the forgotten material".

The result of years of neglect in tertiary educational institutions is that most Architects and Engineers in Australia have little, if any training in the design of timber for structural purposes. Fortunately, this situation is slowly being rectified in a number of, but not all, engineering and architecture courses.

The timber industry too, has a major responsibility in overcoming the long standing neglect of timber for structural purposes. Not only must it give whole hearted endorsement of educational and research initiatives, but it must accept responsibility for the development and implementation of quality assurance programs and procedures for establishing reliable and consistent material properties, such as in-grade testing, in order to satisfy the demands for a reliable, performance guaranteed product, if it is to gain a significant market share in non-residential construction.

The amount of research required to address some of the technical issues facing designers who wish to specify timber as an engineering solution, is quite formidable. In fact, unless we draw extensively on the results of overseas research and design experience, we may never succeed in this endeavour. It was within this context that the author sought a Gottstein Fellowship, to study overseas R & D projects specific to non-residential engineered structures and to identify how *technology transfer* can be used to implement relevant research into an Australian context.

**Purpose of the Gottstein Fellowship:**

An explicit purpose of the Joseph William Gottstein Memorial Trust, is to create opportunities for Fellowship recipients to acquire knowledge, which will promote the interests of Australian Forest products and increase their acceptance and use. A Gottstein Fellowship is a strategic, medium to long term investment in the future of the timber industry in Australia, which not only is a great privilege and opportunity for the recipient to develop his or her professional skills, but also equips the industry to meet the challenges of a changing market place and society.

**Scope of Study Tour and Findings:**

The expressed purpose of the author's Fellowship was to facilitate "*Technology Transfer*" of both research and design procedures for engineered timber structures, particularly in the non-residential building market. The main focus of the author's application submission to the Trust, was to investigate and report on the research, design and performance of connections and jointing systems for non-residential timber structures. This is presented in Section 2 of this report.

However, because the opportunity afforded by the Fellowship allowed the author to visit some of the leading timber research institutions in the world, this original brief was extended to include a number of other research areas relevant to the use of timber in engineered structures, such as: stress laminated bridge deck technology, volume and creep effects in glulam beams, MSTF (Multi Storey Timber Framed) construction, floor vibration and characterisation of material properties for design. These are in turn presented in Sections 3 to 6 of this report.

A summary overview of the study findings and recommendations for each of the sections in this report are detailed and presented, along with appropriate acknowledgments, in Section 7.

**Summary of Recommendations:**

Whilst details of recommendations are presented in Section 7, it is appropriate to highlight general principles which undergird the findings of this study.

(i) This study highlights the importance of *technology transfer* as a means of applying the "state of the art" timber technology in Australia and avoiding unnecessary duplication of research, which is not only impractical, but can often be both time consuming and expensive. A key recommendation of this report is the identification and establishment of mechanisms or infrastructures to facilitate technology transfer and collaborative research programs with appropriate overseas researchers, wherever possible.

(ii) Linked to (i), is the need for an information infrastructure, to not only communicate information and create awareness of relevant overseas research and developments, but also to be able to provide easy access to the details of such findings.

(iii) It is also imperative that a forum for the determination of research priorities be established, involving the industry, NAFI and promotional organisations and relevant research establishments, such as CSIRO and Universities.

(iv) The establishment of recognised Centres for Excellence in timber design, involving education, research and specialist consulting, and reflecting the needs of industry is imperative. The imminent establishment of the Timber Engineering Research Centre in Sydney has the potential to contribute significantly to this role, and as such, should be supported by the industry. Furthermore, networking of all such centres involved in timber research throughout Australia should be a priority for both the Centres themselves and for the Timber Industry.

**Implementation of Study Findings:**

A commonly expressed concern about study grants and scholarships, is that a lot of money is spent to produce a large report, which then gathers dust on shelves and results in very few tangible benefits to the industry which sponsored the study.

Whilst some of the findings of this report may not find immediate application and are concerned with longer term benefits to the timber industry, a significant proportion of the findings from this Gottstein Fellowship have already had significant implementation.

The main initiative in this regard is the Timber Bridge Initiative in NSW. This represents one of the largest research and development programs ever undertaken for timber engineering in this country and apart from the research and testing already completed has resulted in Australia's first hardwood stress-laminated bridge being constructed during the beginning of December 1991, at Yarramundi Lagoon, near Richmond, NSW.

Similarly, implementation of epoxy injected dowel connection technology has also been successfully undertaken in January, 1992 in Sydney church auditorium. The importance of this must not be understated nor missed by the industry. The Gottstein Fellowship has allowed the author to implement significant and greatly beneficial research findings within a 12 month period of undertaking the study tour.

Both the projects mentioned above have resulted from technology transfer from the work of overseas research work, coupled with some validation testing and research to implement the technologies in Australia. Such projects highlight the value to the industry of research and development work which is "needs driven", and as such, clearly illustrate that strategic R & D coupled with technology transfer, is deserving of the industry's whole hearted support and encouragement. It is the author's belief that the Gottstein Fellowship is one very important mechanism which provides the opportunity for this to occur.

**Study Tour Itinerary:**

The author's study tour was undertaken in two stages during the last quarter of 1990. The first stage encompassed visiting specific centres in North America, the UK and Europe, whilst the second stage involved a shorter duration, spent in New Zealand. The details of this are given below. It should be noted that some of the research material presented in this report also references other centres visited (particularly in Europe) during an AFIJ study tour, undertaken in September 1988.

**STAGE 1:**

| **Date:** | **Location / Sponsor:** |
|---|---|
| 12/09/90 | Travel: Sydney - USA |
| 13-15/09/90 | Uni of Wisconsin , Madison - Prof Mike Olivia<br>Forest Products Laboratory - Russ Moody |
| 16/09/90 | Travel: Madison, Wisconsin - London |
| 17-22/09/90 | Brighton Polytechnic - Prof Barry Hilson |
| 23/09/90 | Travel: London - Zürich |
| 24-25/09/90 | ETH Zürich - Prof Ernst Gehri |
| 26/09/90 | Travel: Zürich - London - Toronto - Fredrickton |
| 27-28/09/90 | Uni of New Brunswick, Fredrickton - Dr Ian Smith |
| 29/09/90 | Travel: Fredrickton - Vancouver |
| 30/09/90<br>to 13/10/90 | Uni of British Columbia - Prof Ricardo Foschi<br>Prof Borg Madsen |
| 13/10/90 | Travel: Vancouver - Sydney |

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**STAGE 2:**

| **Date:** | **Location / Sponsor:** |
|---|---|
| 25/11/90 | Travel: Sydney - Christchurch (NZ) |
| 26/11/90 to 01/12/90 | Uni of Canterbury - Dr Andy Buchanan |
| 03/12/90 | Hunter Timbers, Nelson - Alan Hunter |
| 04-05/12/90 | Travel to Rotorua |
| 06-08/12/90 | FRI Rotorua - Dr Bryan Walford<br>Mr Hank Bier |
| 09/12/90 | Travel to Auckland |
| 10-11/12/90 | Uni of Auckland - Dr Tony Bryant<br>Dr Richard Hunt |
| 12/12/90 | Travel: Auckland - Sydney |

# SECTION 2:

# CONNECTIONS & JOINTS FOR STRUCTURAL TIMBER

*An Overview of Research & Design Techniques*

## JOINTS AND CONNECTIONS:

### Introduction:

At a meeting during September, 1990 (at the Forest Products Laboratory in Madison, Wisconsin), with Dr. Erv Schaffer, (Assistant Director - Wood Products Research), three major areas of timber engineering / design requiring urgent research were highlighted:

1. Durability of timber (including protection systems, design detailing & quality control)
2. Fire performance and fire resistance for structural timber elements
3. Design and detailing of connections

A major thrust of the author's Gottstein Fellowship for 1990 was to study the design and development of connections, (particularly moment resisting connections) in Europe, North America and New Zealand, with a view to facilitating "technology transfer" of the current research and design work being undertaken in these respective regions and relating these to Australian practices.

This paper presents an overview of the research developments and design techniques for timber connections observed by the author during a study tour during the last quarter of 1990, which was made possible as a Fellow of the Gottstein Memorial Trust.

### Importance of Connections:

Joints and connections are usually required to transfer loads within a structure from one member to the other and then through the structure to the foundations. In many situations connections are used to develop adequate stiffness against rotations and movements which exist between the structural elements.

The need to understand connection behaviour is of the utmost importance for all structural materials and in particular, timber. Without an understanding of connection behaviour and an appreciation of the unique, intrinsic qualities of timber as a material, it is almost impossible for a designer to successfully achieve creative expression and form in structural timber as an architectural and engineering medium.

**Historical Background:**

Traditionally, connections have consisted of a multitude of handcut details which have been developed by carpenters through centuries of experience of working with wood. Throughout Europe in particular, these hand cut details have been carefully adapted to the intricate characteristics of wood and reflect an amazing understanding and appreciation of fundamental wood technology and material behaviour.

These traditional timber connections include: notched joints which allow skewed connection members, lap joints which allow structural connection of timbers within the same plane, and mortise and tenon joints which serve to secure the adjacent faces of two timber members, for example, to secure compression members against structural movement.

However, there are a number of significant drawbacks with traditional connections for use in modern, large spanning structures. Particular limiting characteristics include:

- the weakening of the timbers due to notch effects
- the high degree of workmanship and labour required
- and the difficulty of assembly
- the force transfer is usually by bearing or wood/wood contact

As such, these intricate hand-cut details are used less frequently to-day, than in the past.

In most traditional forms of timber joints, the performance of the connections normally relies upon bearing, with occasional tie-members being pegged or dowelled into place (usually with wooden dowels). Obviously these were not constructed without considerable knowledge of wood technology, as the wood species and moisture play an important part in the dowel performance.

**Modern Connection Techniques:**

With the industrial revolution and the facilities for production of iron and steel, it became possible to develop plated joints and use bolts and nails for joining timber members together. These types of mechanical fasteners and connectors can be grouped into:

1. Nails, spikes and staples
2. Wood screws
3. Bolts, lag screws, split ring connectors and shear plates
4. Light metal and framing connections.

In addition to these mechanical joints, recent research (in Europe in particular) has also focussed upon the development of glued connections and epoxy injected connections which utilise steel dowels and rods set in epoxy inside the timber, in a similar way to a steel reinforcing rod set in concrete. These glued connections also include macro-finger jointed epoxy connections, which have been developed in a effort to "weld" timber elements together and effect efficient load transfer between them.

**Characteristics of Connections:**

The structural adequacy of mechanical joints subjected to axial load and/or bending moments is usually assessed with respect to their strength characteristics. Factors which influence strength and deformation characteristics of timber joints include:[1]

a. specific gravities of timber members
b. direction of grain in timber members
c. orientation of growth rings and grade effects in timber members
d. moisture content of timber members prior to and after loading
e. connector materials and geometry
f. joint member geometry
g. arrangement of joint members with multiple shear planes
h. number and position of the connectors
i. the method of joint fabrication
j. the previous loading history (cummulative load effects)
k. the rate at which load is applied and
l. the regime of loading (eg: repetitive, long or short duration)

**Fundamentals of Load Transfer:**

In general, joints may be divided into two main types, depending upon the principle by which they transfer load from one member to the other. These types are characterised by either:

1. transmission of the forces through wood/wood contact area or
2. the transmission of the forces via mechanical connection

It is relatively simple for compression forces to be taken into account by contact transfer between members. However, tension forces present considerably more difficulty because once the timber is cut, the structural continuity of the structural element is permanently disrupted.

Research into "macro" finger joints using glued connections and epoxied dowel type connections with steel/epoxy transfer between timber elements, has demonstrated that a kind of "welding" can occur between structural timber elements, but it is not possible to reconstitute the original structural continuity by filling with an additional material, as is the case when welding steel. Generally, the connection can only be obtained by deviation of forces around the discontinuous fibres, which usually results in significant additional localised stresses and eccentricities appearing in the region of the connection.

The glued macro finger joints are really the only type of connection which act as the surface of the fibre of the wood material. All other jointing systems involve some form of penetration which results in localised destruction of the timber fibres and discontinuity of structure at the point at which the connection is fastened. This weakening of the cross section of the timber member is unavoidable and local stress concentration around these connectors may reach extremely high values.

In order to define the optimum type of connection or joint detail, the exact characteristics of both the timber material and the joint and its mechanical performance must be known. Amongst the most important criteria to be considered are:[2]

1. the maximum load per connector that can be transmitted
2. the load deformation behaviour within the connection (particularly of the nails and/or dowels which transfer load from the timber member through the connecting element, usually a steel plate)
3. the stiffness of the joint (this is particular related to the precision of fabrication of the connection detail)
4. the area of contact needed for transmission of the loads
5. weakening of the cross-section of the timber member
6. efficiency of the joint
7. long-term behaviour (under both static dead load and dynamic live load)

**Determination of Connector Performance:**

With all connector types, research into the performance of timber joints has taken one more of the following forms:

1. determination by direct means of the strength or deformation characteristics of joint types, usually by prototype testing.

2. establishment of empirical relationships by extrapolation of test data

3. establishment of modification factors from direct test data for influences such as duration of loading and environmental conditions

4. analytical models such as finite element models (FEM), which enable the prediction for strength and/or displacement of the complete joint assembly by means of mathematical representation. Input data for

these analytical models tends to be based upon the both geometric and experimentally derived material properties of the joint components, as well as the loading "events" experienced by the connection.

**Previous Research:**

Research between 1910 and 1940 placed considerable emphasis on the determination of strength values for the more common combinations of connection type, timber species, environmental conditions and types of loading. This often resulted in the derivation of modification factors to account for influences such as variations in moisture content, changes in environmental conditions, duration of loading, connection spacings, members thicknesses and clear edge distances.

In many instances, an empirical formula was derived, relating strength at the proportional limit or ultimate load to some form of predicting parameter, usually the density of the timber, which then enabled the extrapolation of test results to timber species outside of the specific test program. This sort of information has generally been codified and to a certain extent is still reflected in many design codes in use to-day.[3]

**Current Trends in Research:**

Broadly speaking connection design can be categorised into six general groups consisting of:

1. Bolted connections (including screws and large dowels)
2. Small dowel connections
3. Nail connections
4. Glued and epoxy injected connections
5. Nail plate connections
6. Split rings and shear plates

As a part of his Gottstein Fellowship for 1990, the author investigated current research into the first four categories.

**Bolted Connections - Europe:**

Bolted timber connections have widespread use throughout the world in a variety of forms, but it is in Europe in particular that high load capacity mechanical connections have been developed to a very high level of design sophistication. The use of bolts grouped in a circular bolting pattern has become fairly widely accepted throughout Europe as a means of connection design for moment resisting connections, particularly between columns and rafters in large free-spanning timber structures.

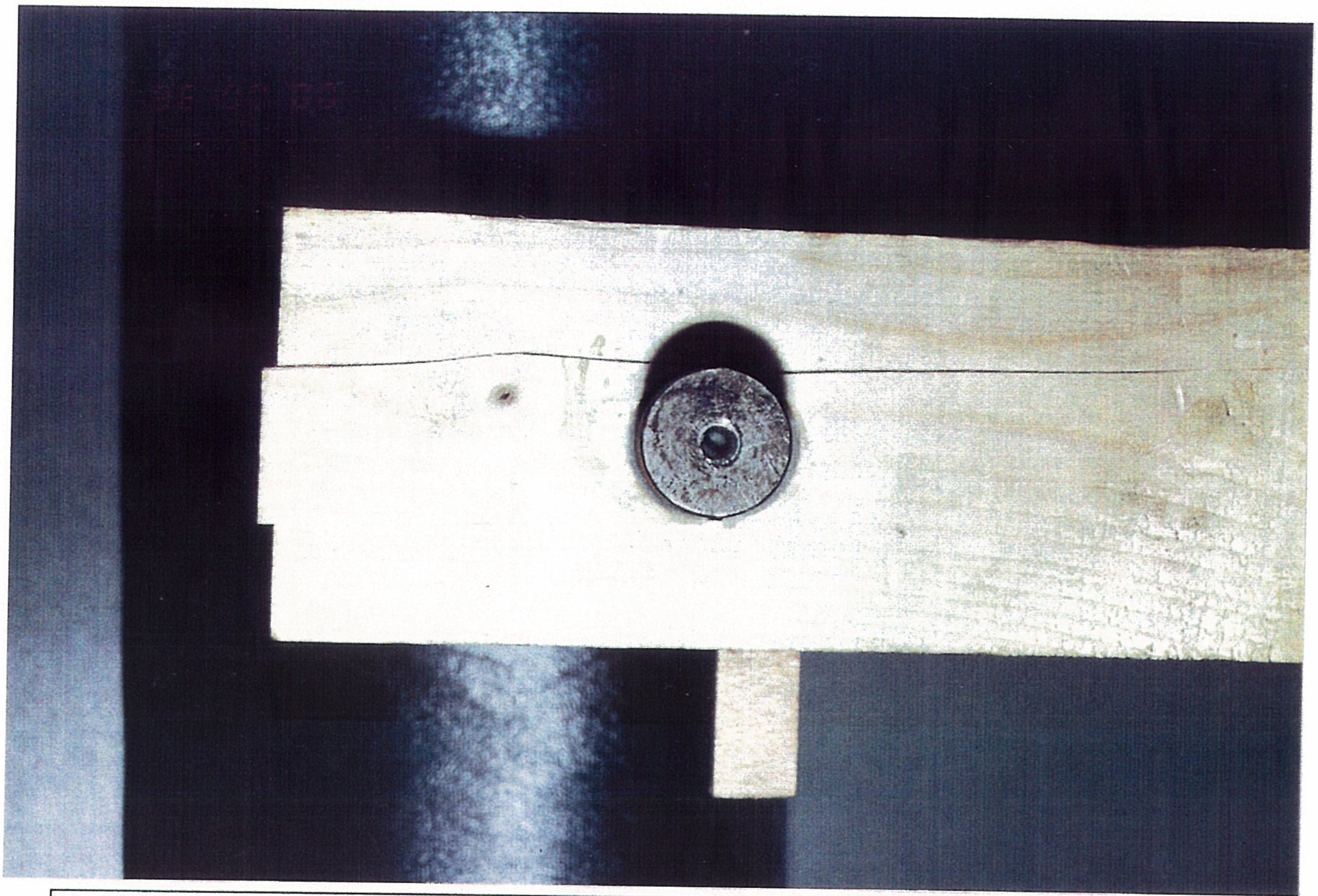
Research work by Dr. Peter Rodd in the early 1970's investigated connection behaviour with a view to developing empirical methods of joint strength calculation, which could be used for design purposes or for verification of other theoretical methods.

Early joint tests confirmed that timber in compression beneath a circular connector displays a marked degree of elasticity, providing that longitudinal splitting of the timber does not occur. Initial results from these tests indicated that the timber beneath the connector had not undergone vertical compression of the timber fibres over the full projected width of the connector (as had been previously thought).

Instead, there existed in the timber beneath the connector, two well defined parallel vertical lines, between which the timber had been compressed in a vertical direction and outside of which, the timber had been deflected around the perimeter of the connection. The deflection of the timber around the perimeter had also caused compression of the adjacent timber in a horizontal direction. A major fact of governing the ratio of vertically crushed to horizontally deflected timber was found to be the co-efficient of sliding friction between the timber and the circumference surface of the connector. The greater this co-efficient, the wider the crush width of the timber and the stronger the joint; the lower the co-efficient the lesser the width of vertically crushed timber and the weaker the joint. [4]

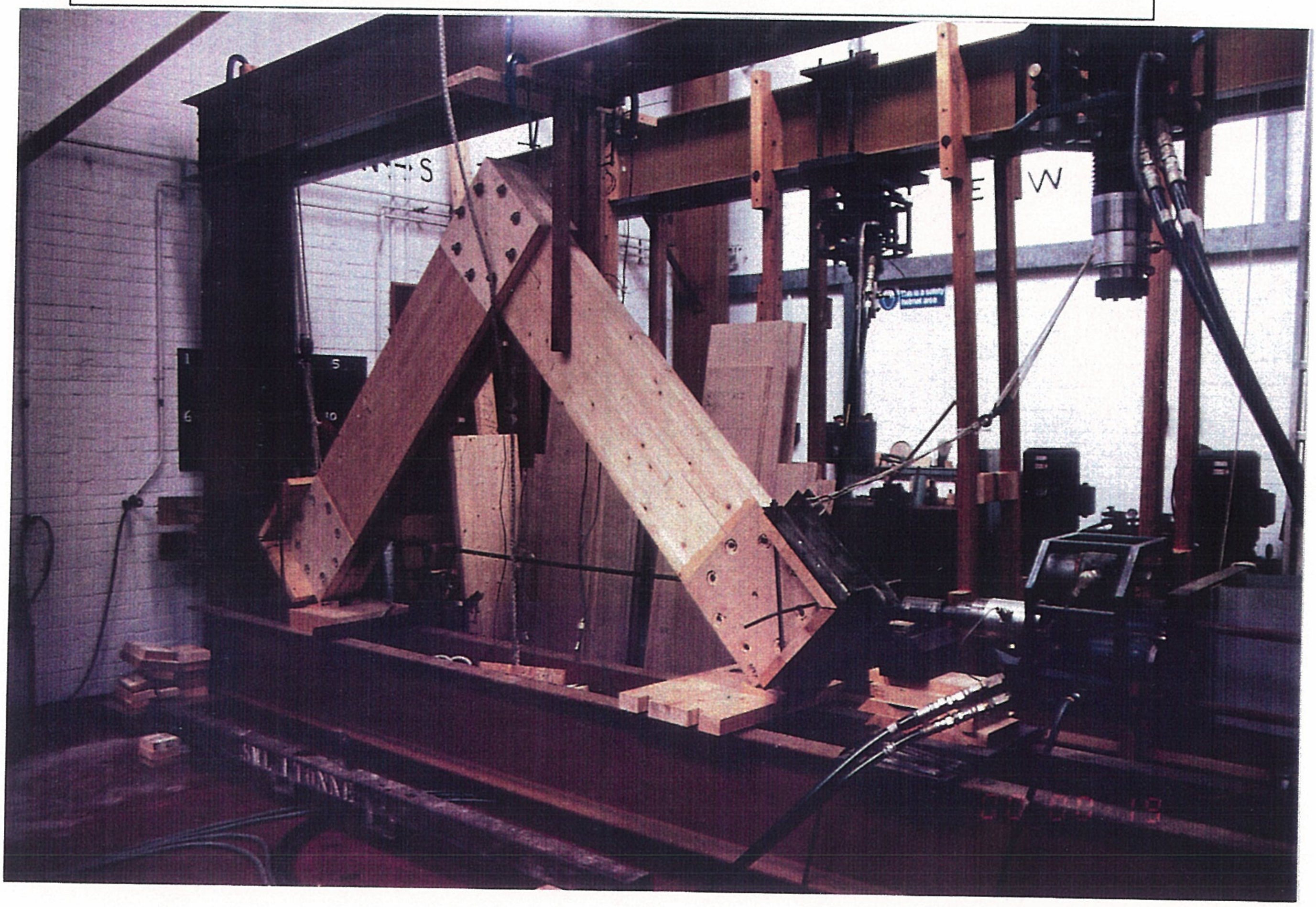
This early work by Dr Rodd and similar work on split ring connections by Prof. Barry Hilson has formed the basis of ongoing work that is currently being undertaken by the Structural Research Unit (STRU) at Brighton Polytechnic. The underlying aim of this work has been to develop an efficient system of constructing moment resisting connections for glue-laminated timber portal frames, which can be readily fabricated on site.

Observations of earlier work, demonstrated an increase of strength for joints made with circular dowel type fasteners which have a higher frictional co-efficient between their circumference surface and the timber on which they bear.



Photograph 2.1 - Resin injected knurled dowel used for embedment tests - Brighton Polytechnic

Photograph 2.2 - Full scale testing of circular bolted moment resisting connections - Brighton Polytechnic



**Resin Injected Dowels:**

This "friction" principle has been extended, by bonding the surface of the fasteners to the timber with epoxy resin, in order to produce the bond equivalent of an infinite co-efficient of friction. It was also discovered, that an additional advantage of using resin injected dowels is that this technique also overcomes the problem of initial loss of joint stiffness (due to tolerance in the dowel hole) when using high friction dowels created with a knurled surface on the circumference of the dowel. (refer **photograph 2.1**) Thus, the advantages of resin bonding are in fact two fold:-

1. the resin provides a suitable bond between the fastener and the timber effectively giving a infinite co-efficient of friction &

2. resin bonding increases the joint stiffness by elimination of take-up movements.

This latter is particularly important in joints that contain a multiple number of fasteners of the type that fit into pre-cut holes or grooves, where the question of differential load take-up has to be addressed if such joints are to be used in determinate structures and those which are sensitive to deflection. By filling gaps between the fastener and timber with resin, all fasteners call be assumed to "take-up" load simultaneously, which leads to stiffer joints and does not necessitate the significant load/slip ductility from the fasteners as would be required with subsequential load take-up for the same ultimate strength.

A three year project commencing December 1987 has been undertaken by Rodd, Hilson and Spriggs at Brighton Polytechnic, to investigate moment resisting connections between glulam members made with resin injected bolts. At the time of the author's visit to Brighton Polytechnic, a considerable amount of testing work had already been completed and subsequent work on the derivation of theoretical means (analytical model) which could be used to determine the strength and the moment rotation behaviour for these particular types of connections, was well advanced.

**Analytical Models:**

Much of this theoretical work is based upon existing and well proven mathematical models for predicting performance of normal laterally loaded mechanically fastened joints (e.g. such as for nails and bolts), and then modifying these models where necessary, to account for the injected resin frictional bond and subsequent modified slip behaviour. The analytical models for prediction of joint behaviour are based upon rivet/bolt group theory, as is used in steel connections.

Modifications of this theory are necessary to account for the anisotropic material characteristics of the glulam. These properties have been evaluated from a number of prototype "full size" joint tests which have been undertaken in the laboratory (refer **photograph 2.2**). Initial data from this project was presented at Auckland, New Zealand in 1989 at the Second Pacific Engineering Conference.[5]

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**Research Conclusions:**

At the time of the author's visit to Brighton Polytechnic in September 1990 the testing program was drawing to an end. Discussions with Dr. Peter Rodd highlighted the following conclusions:

1/ Despite the superior embedment performance of the resin injected bolts (and/or dowels), both in terms of stiffness and strength characteristics, the governing factor in the ultimate strength of the connection is tension perpendicular to grain loading.

2/ Both the experimental tests and analytical models have confirmed that as the load is applied to the connection with increasing magnitude the centre of rotation for the connection shifts from the centre of the bolt group towards either the external or re-entry corner, depending upon whether the joint has been closed or opened respectively.

3/ The conclusions to-date are that the ultimate capacity of a circular bolt group for a moment resisting connection, is limited by perpendicular to grain (tension) forces. As such, the use of additional bolts may make the joints stiffer, but would not change the ultimate failure mode nor the ultimate strength capacity of the joint, which is not based upon the steel but rather is based upon the cleaving/splitting failure mechanism due to perpendicular to grain stress effects.

4/ The tests verify that the use of large diameter bolts ensures non-yield or rigid bolt behaviour and so the failure mechanism for the connection tends to be a brittle failure, rather than a ductile failure, due to the fact that timber splits as a result of perpendicular grain loadings.

5/ The shift in the centre of rotation away from the centroid of the bolt group results in significant differences in the loads per bolt for each bolt in the connection group. The higher forces tend to occur in bolts closest to the corners of the connection. Additional experimental work on this type of connection is currently being undertaken by Technical University of Delft, Holland and the University of Karlsruhe, Germany, where investigations into the geometric layout of bolt patterns is being undertaken in order to try and create situations where uniform bolt loads occur at "service loads" in the structure.

6/ Observations at Brighton Polytechnic to-date, indicate that the failure load (due to perpendicular to grain splitting) is a function of the ultimate load of the connection, not the stress beneath the bolts.

7/ Recent experimental work has been undertaken utilising plywood reinforcement which has been glue-bonded to the sides of the glulam laminates of the connection. The initial results indicate that the 3mm thick plywood reinforcement limits splitting and increases ductility of the connection, (within the range of expected service loads) but does not significantly increase the ultimate strength at failure (max increase of about 20%), nor does it change the non-ductile failure mode. The purpose of the plywood reinforcement is to overcome the "size effects" of the bolts, such that the effects are localised only and no additional shear planes are induced in the timber laminates beneath the circular connection (which is also the rationale for using smaller diameter fasteners, such as nails).

8/ It is believed that the effect of reducing shear planes beneath the circular connection would be further enhanced by limiting the size of the dowels to no more than 5-6mm in diameter.

It appears to-date, that the 3mm thick plywood plates allow a slight increase in stiffness, up-to 20% increase in allowable ultimate load and definitely increase the ductility of the connection. Ongoing work is being undertaken by Robert Spriggs, (at Brighton Polytechnic) under the direction Prof Hilson and Dr Rodd for the development of a predictive analytical model for these types of circular bolt groups used as moment resisting connections.

**Other Connection Research:**

As well as the work being currently undertaken on resin injected bolts, considerable research has been undertaken by Brighton Polytechnic over the last number of years which has been linked to the research assessment and development of design procedures in regard to Eurocode 5. Considerable work was undertaken during the mid 1980's, investigating the characteristic properties of nailed and bolted joints under short term lateral loads. This research encompassed embedment testing with nails and bolts which were loaded laterally and involved the use analytical models for predicting joint behaviour, linked with testing where necessary, to establish the characteristics of joint/member material properties required for the analytical models.

The underlying purpose of this research was the development of appropriate design techniques for incorporation into a probability based limits state design code format. There has been considerable research over the last four decades aimed at providing suitable analytical models for predicting short term strength and/or stiffness properties under lateral loading.

Work by Johansen in the 1940's assumed that both the fastener and timber in a connection are ideal "rigid-plastic" materials. This formed the basis for most of the load-deformation models that have been used over the past 40 years. A more refined model has been developed by Dr Ian Smith, which assumes that the fastener is an ideal "elasto plastic" material and makes no approximation of the nonlinear load-embedment characteristics for the joint members (refer **fig:2.1**). Experimental work has validated Dr Smith's research to the extent that the analytical model is able predict short term initial stiffness of joints to plus or minus 20% and ultimate loads of the connections to plus or minus 10%.[7]

The research program has evaluated the characteristic properties of nailed and bolted connections under short term lateral loads in five stages which are presented as five related papers in the journal of the Institute of Wood Science published in 1989 and 1990. The five stages of the research program were:

1/ Research, Philosophy and Test Program;

2/ Embedment Test Apparatus for Wood and Wood-based Sheet Materials;

3/ Analysis and Interpretation of Embedment Test Data in Terms of Density Related Trends;

4/ The Influence of Testing Mode and Fastener Diameter upon embedment Test Data;

5/ An Appraisal of the Current Design Data in B.S. 5268 - Part 2 1984, the British Standard for "Structural Use of Timber".

**Relative Ductility - Nailed and Bolted Connections:**

Conclusions of the research program indicate that for short term test observations, bolted connections are more prone to brittle failure than nail connections. As such, higher load factors relating working stress to ultimate strength might therefore be reasonably applied to bolted joints. In nailed joints, higher load factors are generally used to reduce the ultimate load capacity, to a level which will normally give a load/slip relationship that is sufficiently low to ensure serviceable connections at working load levels. Design capacities are normally sufficiently low enough to preclude safety concerns. Bolts are often used in connections where slip is less critical and design loads can be assigned a higher proportion of the ultimate capacity than is appropriate for nailed connections.

The work presented in this research program, used in conjunction with yield theory, gives a logical basis for calculation of the strength of nailed and bolted joints in timber. Conclusions by Hilson, Whale and Smith recommend that rather than using tabulated or simplified data in codes of practice, a set of basic material property equations and yield equations should be used in design procedures (this is a departure from present practice in BS5268 part 2). This technique would then enable designers to calculate the appropriate ultimate load for any connection arrangement and would identify the likely mode of failure to be expected. The technique could also be readily modified to be incorporated into a limit state format,[8] provided a methodology which allows for joint stiffness (serviceability) is also included in the design procedure.

**Eurocode 5:**

The European Economic Community (EEC) has agreed that all internal barriers to the free movement of goods and services shall be removed before the end of 1992. To ease this process in the building industry a system of common European Building Codes is being established, known as the "Eurocodes". The ultimate aims for the Eurocodes to become the only design codes of practice throughout all the EEC countries; however, initially their use will be optional and they will co-exist with existing national codes.

Timber structures will be covered by Eurocode No:5 "The Common Unified Rules for Timber Structures". Eurocode 5 adopts a partial co-efficient limit states design philosophy and the design of nailed and bolted joints in this code is based upon ultimate load theory developed by Johansen.

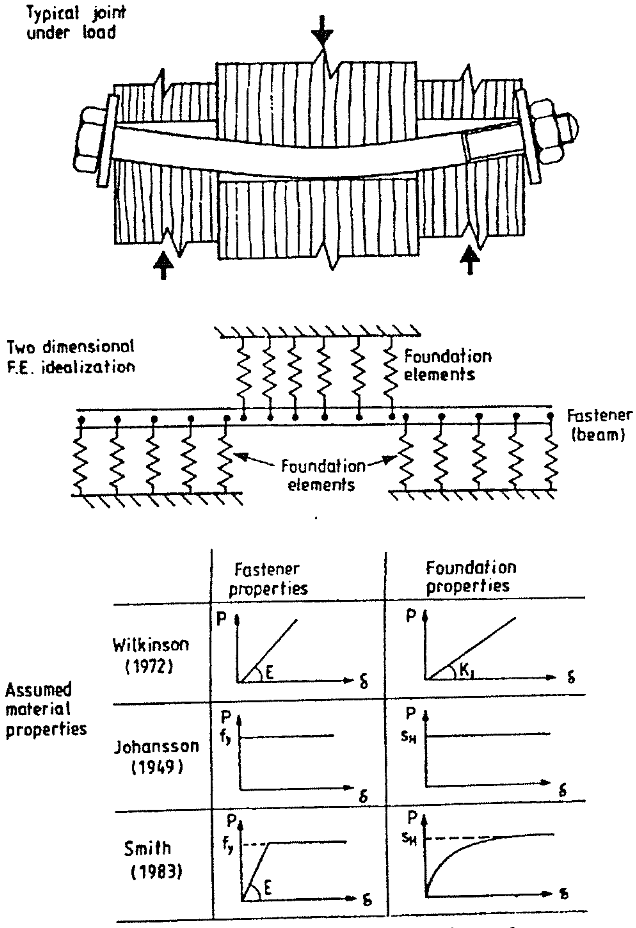
**Johansen's Theory:**

Assuming both the fastener and the timber are ideal rigid-plastic materials, Johansen was able to derive equations for the ultimate strength of joints with dowel type fasteners. These equations predicted failure loads due to either bearing failure of the timber members or the simultaneous development of the bearing failure in the timbers members and plastic hinge formation in the fastener. The precise mode of failure is dictated by both the joint geometry and the member properties. This theory is illustrated in **figure 2.1**. One important property is found in the embedment test which may be regarded as a symmetrical 3 piece joint test using steel side members and a wood, or wood based sheet material centre member, in which a bearing failure is forced under lateral mode.

Embedment stress is defined as the ultimate bearing stress and is calculated by dividing the ultimate load by the projected area of the dowel in the timber. It can been seen from Johansen's equations, that the input required in order to predict the ultimate load capacity of the joint comprises:

- some geometric parameters
- plastic moment capacity of the fastener
- the embedment strength of the timber.

In order to obtain the necessary data base of material properties, a comprehensive testing program has been undertaken, the full details of which have been described by Smith & Hilson.[9]




Analytical models for joints with dowel type fasteners.

**fig 2.1 - Analytical Techniques for representing ductile behaviour of steel dowels and / or bolts**

Comprehensive tests encompassing many timber species and fastener diameters have been carried out Brighton Polytechnic and the South Bank Polytechnic, in order to obtain the embedment strength data for timber and sheet material. From an analysis of all this data, the relationships detailed in **figures 2.1 & 2.2** were established for embedment strengths in the research work undertaken by Hilson, Whale & Smith[10]

### Validation Testing:

Validation testing of the Johansen equations has been undertaken in the above referenced testing program, the results of which were published in the Journal of the Institute of Wood Science - 1987.[11] This work, and similar work in the United States by Aune and Patton-Mallory[12] concludes that the Yield theory provides reliably accurate predictions to within plus or minus 10% of the ultimate strengths for the joints tested.

Hilson & Whale have concluded that Johansen's theory may be used for nailed and bolted connections and therefore, provides a consistent approach to determination of the ultimate load capacities for these types of connections[13].

The draft Eurocode 5 uses Johansen's theory as the basis for the calculation of the ultimate strength of joints made from nails, staples, screws, bolts and dowels, but it uses simplified forms of the equations which appear to involve some approximations. A detailed assessment of the clauses in Eurocode 5 was undertaken by Brighton Polytechnic and published by Hilson & Whale in 1988.[14]

Basically, the conclusions to this research are as follows:

**Nailed Joints:**

In nailed joints, if the constraints imposed by Eurocode 5 are accepted, designs to Eurocode 5 will have similar numbers of nails with fewer nails than in designs to BS5268. There is however, an interplay between 3 effects:

1/ tabulated nail strengths in BS5268 are higher than those obtained from Johansen's theory

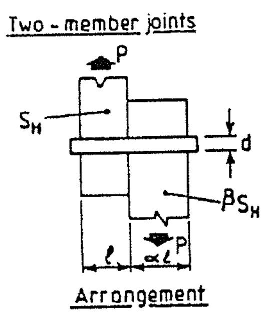
2/ the Eurocode 5 approach ensures mode 3 failures whereas the majority of the configurations specified in BS5268 would produce mode 2 failures (refer to the arrangement for 2 member joints in **fig 2.2**).

3/ The partial safety factor approach adopted in Eurocode 5 ensures a lower overall load factor than opted for BS5268. For bolted joints Eurocode 5 designs would normally required fewer bolts than designs to BS5268 because of the high load factors implicit in most of the BS5268 values.

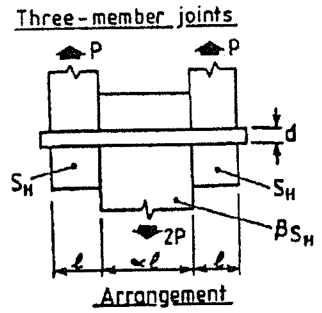
**The Eurocode design philosophy for connections:**

In general terms, the assessment and review[15] of the research underlying the Eurocode design procedures for load carrying capacity in joints (which was undertaken at Brighton Polytechnic), revealed that all the equations have been based upon a basic set of simplified yield theory equations. In response to this approach used in the draft edition of Eurocode 5 and for the reasons given below, Hilson and Whale have recommended that <u>all</u> Johansen's equations should be presented for the design of bolted, dowelled and nailed connections. These are detailed in **figure 2.2** for both 2 member and 3 member connections.

During the evolution of the current Eurocode 5 design equations[16], simplifications have been made (most of which are quite justifiable in themselves), by removing the need to cater for joints with unusual proportions, and concentrating upon the mainstream of design solutions or what might be generally accepted as "standardised" connection geometric configurations. This has in turn necessitated the introduction of a variety of subsidiary clauses which stipulate minimum member thicknesses and fastener penetrations, in order that joint proportions remain within the bounds of applicability of the simplified equations. Rules permitting linear interpolations of joint strengths have been then re-introduced to allow the construction of joints with these peripheral dimensions, albeit under conservative design approximations.




$$
R_y = \min \begin{cases}
S_H d \ell & \text{(a)} \\
S_H d \ell \alpha \beta \quad \text{(only for two-member joints)} & \text{(b)} \\
S_H d \ell \alpha \beta / 2 \quad \text{(only for three-member joints)} & \text{(c)} \\
S_H d \ell \left[ \dfrac{\sqrt{2\beta(1+\beta) + 4\beta(2+\beta) d^3 f_y / (16 S_H d \ell^2)} - \beta}{2+\beta} \right] & \text{(d)} \\
S_H d \ell \alpha \left[ \dfrac{\sqrt{2\beta^2(1+\beta) + 4\beta(1+2\beta) d^3 f_y / (16 S_H d \alpha^2 \ell^2)} - \beta}{2\beta+1} \right] & \text{(e)} \\
\quad \text{(only for two-member joints)} & \\
S_H d \ell \left[ \dfrac{\sqrt{\beta + 2\beta^2(1+\alpha+\alpha^2) + \beta^3 \alpha^2} - \beta(1+\alpha)}{1+\beta} \right] & \text{(f)} \\
\quad \text{(only for two member joints)} & \\
4 d^2 \sqrt{\dfrac{10 \beta S_H}{1+\beta}} \sqrt{\dfrac{f_y}{240}} & \text{(g)}
\end{cases}
$$



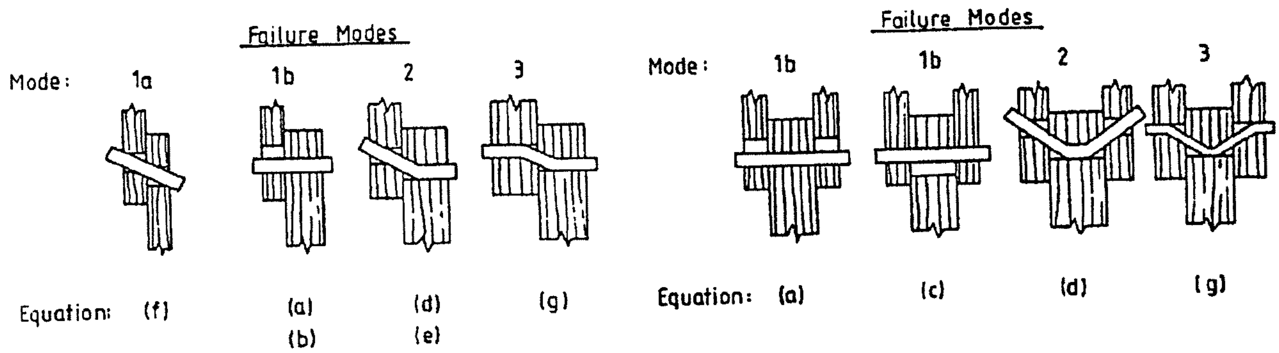




**figure 2.2[17] Johansen's yield theory equations and associated failure modes**

**where:**

$\delta$, $l$ and d are joint geometry factors &

$\beta$, $S_h$ and $f_y$ are joint member properties

The advent of board-materials in wood joint construction has necessitated yet more specialised clauses, so that these unusually slender materials could be incorporated within the narrow generalised design procedure which have been defined. Despite this, the existing proposals in Eurocode 5 preclude most board materials from being used in joints, with the only possibility for remedying this situation relying upon large degrees of conservatism being introduced. In Prof Hilson's opinion, the resulting design equations are often limited in application, cumbersome in operation, and not amenable to the introduction of new materials.

**Reasons for using the Full Set of Yield Theory Equations:**

Prof Hilson and Dr Whale, recommend that a significant revision to the Eurocode 5 design rules should now be made by reverting to the use of the original full set of yield theory equations for all types of fastener. This would remove the mystique which surrounds many of the clauses and permit the Engineer maximum flexibility in being able to calculate strengths for joints with any proportions or material combinations. As a consequence it would also furnish Engineers with a means of assessing the sensitivity of joint strength to variations in the design parameters.

Apart from the existing rules for fastener spacings and distances, axially loaded fasteners and multiple fasteners, the code rules would, therefore, consist of five design formulae for use with nails, staples, bolts, dowels and screws, with separate equations expressing the way in which the input data to these formulae vary for the different fastener types and/or joint member materials. No stipulations on minimum member thicknesses need be given, and only minimum permissible penetration depths need be specified.

**Summary Conclusions on Eurocode 5 joints clauses:** [18]

1/ The omission of design rules for split-rings, shear plates and toothed-plates is a regrettable consequence of the way in which established predictive formulae are used in the Eurocode as the basis for design. Rather than the use of approval systems such as Agrément, as a means of establishing this design information, the existing information in BS5268 : Part 2 : 1984 should be adjusted to characteristic values for limit states format and documented in a publication which has the status of an Approved Document. The Agrément system of approval is considered to be more appropriate for punched metal plate fasteners.

2/ The design approach adopted in EC5 for different fasteners has limited the range of possibilities open to Engineers and, in simplifying the design equations used in certain instances, has introduced a number of clauses which would otherwise be unnecessary. It is recommended that the existing clauses for the lateral load carrying capacity be replaced by a single set of five equations applicable to all fasteners. Material property equations would then be the only additional requirement for each of the fastener and joint member types.

**Research Deficiencies in Eurocode 5:**

The summary by Hilson & Whale also highlighted a number of variations, where the theory presented by Eurocode appears to be deficient or incorrect. These included:

1/ The slip moduli equations were considered to be 2 to 3 times too small. This degree of conservatism could be reduced to an acceptable level by replacing the characteristic Modulus of Elasticity by the mean Modulus of Elasticity.

2/ The requirements for round nails to have characteristic strength of at least 600MPa, should be replaced by a requirement that the characteristic yield strength of the nail exceeds **50*(16-d) MPa**, (where d is the diameter of the nail shank)

3/ The equations in the code make no allowance for the influence of nail heads on joint strength.

4/ The stipulation of predrilling is required from wood with a characteristic density of 350 kg/m$^3$ or more, is unduly conservative. The requirement for predrilling could be raised to characteristic density of 500 kg/m$^3$ or more, without invalidating the design equations for load bearing capacity.

5/ Load carrying capacities for board materials to wood joints may need to be reduced in certain cases in order to allow broader range of board material thicknesses to be used (and thus ensure that designs are conservative).

6/ The accepted values for withdrawal capacity of nails may need to be modified if the withdrawal equation is to provide lower bound predictions for all of the experimental joints tested.

7/ Other areas of concern in the Eurocode provisions relate to clarifying a number of clauses which define edge distances to fasteners and fastener permutations involving nails and combinations of screws and bolts.

8/ Similar concerns were expressed about EC5 assumptions in regard to the strengths of steel plates in steel to wood connections and it is recommended by Hilson and Whale, that these clauses should be replaced by guidelines giving a minimum permissible plate thickness either in absolute terms, or in terms of the fastener type.[19]

In addition to highlighting deficiencies within the code, Hilson and Whale's report also validated certain clauses for which research data could not be found but seemed reasonable, as well as endorsing clauses which had been confirmed by research evidence.

As can be seen from the recommendations detailed above, the work completed at Brighton Polytechnic has focussed upon providing a rational procedure for the design of mechanical connections in timber, using the full set of yield theory equations, with supplementary data on edge distances, spacing and material properties, in order to provide the Engineer / designer with the greatest amount of flexibility possible. Such an approach is to be welcomed, as it not only takes the mystic out of connection design, but it also readily lends itself to development of computer design aids, using spreadsheets which utilise the design algorithms and take the tedium out of repetitive hand calculations.

**Other European Developments:**

As mentioned previously, additional research has been undertaken into the design of moment resisting connections using bolts and dowels, at both the Technical University of Delft, Holland, and the University of Karlsrühe, Germany. Much of this research work has been driven by the need to meet the limit state requirements for Eurocode 5 and a number of the research projects have been funded specifically with Eurocode 5 development in mind.

The main emphasis of this research has been on achieving more efficient bolting patterns and the trend of this work, particularly in Karlsrühe, is moving away from circular bolting patterns and experimenting with "donut" shaped bolting patterns, which attempt to ensure uniform distribution of load on all bolts in the bolt group, at the service loads for the structure. Similar work to that undertaken by Dr. Rodd at Brighton Polytechnic has also been completed at the Technical University of Delft by Leijten, looking at the effects of resin injection dowels and bolts for circular moment resisting connections.[20]

Over the last few years, research in some parts of Europe (particularly in Switzerland), has focussed on smaller diameter mechanical fastenings such as high-strength nails and small diameter dowels. An overview of some of the systems currently in use has been previously documented by Syme[21] (Gottstein Fellow 1987).

In order to investigate these types of connections and fasteners in greater detail, the author spent several days at the ETH Zürich with Prof Ernst Gehri, looking at developments in connection design, as well as bridge applications for structural timber (which are reviewed in Section 3 of this report). Much of the work undertaken by Gehri parallels similar work undertaken by Natterer and Winter at the EPFL in Lausanne (which the author visited during an AFIJ Study Tour in September, 1988).

Gehri's research emphasis over the last few years has focussed on the development of highly efficient "high-load capacity" connections, utilising steel fin plates which are fastened to the timber members using small diameter steel dowels. An example of this is the "knötensystem", which utilises a steel sphere connecting timber members, each of which has an internal plate and dowel connection.

This connection type, which has been described by Syme[22], allows for the development of complex connections such as are utilised in space frames and was specifically developed for this type of application. Essentially, the system allows modular design of space frame structures, where the connection details are frequently repeated and necessitate highly predictable and reliable performance characteristics.

**Design Rationale for Dowels:**

The theory underlying the use of dowels is similar to the objective of the resin injected bolts and dowels used by Brighton Polytechnic, except that the dowels in this case are generally of a much smaller diameter than the bolts. The dowels normally consist of cylindrical steel bars (generally without a nut or thread), which are driven into pre-drilled holes having a diameter of 0.2 to 0.5 mm smaller than that of the dowel.

Such dowels are considered by the Swiss to be of greater importance in connections than bolts and appear to offer superior connection performance, because:

1/ they are not subject to subsequent slippage due to clearance of holes which occurs with bolts.

2/ they are also not as sensitive to the effects of shrinkage.

3/ In addition to these advantages, high connection load capacities can be achieved using dowels, because higher forces can be transferred through larger numbers of fasteners with smaller dowel diameters, than a small number of larger diameter fasteners. This in turn avoids the high stress concentrations which often occur with large diameter bolts, by redistribution of the applied forces over a greater number of smaller interfaces between the steel and timber.

In Switzerland and parts of Germany, this is the preferred type of connection for glue laminated wood and it is often used in conjunction with steel insert plates (refer to **photograph 2.3**). Normal dowel diameters are of the order of 6 to 8mm, though experimental work by Prof Gehri has concentrated of late on smaller dowel sizes. It should be noted that although dowels in wood provide sufficient resistance to lateral movement due to their tight fit, at major connections it is sometimes necessary to add clamping bolts or additional load carrying dowels with a head and nut (the shear capacity of which are not taken into consideration in the design) to ensure structural integrity of the connection.

Recent work by Gehri has focussed upon the development of high capacity end plate connections utilising dowels in truss structures (**figure 2.3**), with particular application to roof and bridge structures. The theory for design of these connections is based upon Johansen's theory, with the connection being designed for ductile failure. i.e. yield to the steel dowels and the ultimate strength being governed by the capacity of the timber. Considerable full scale testing of these high capacity joints was undertaken in the early to mid 1980's and has been extensively reported.[23 24 25]

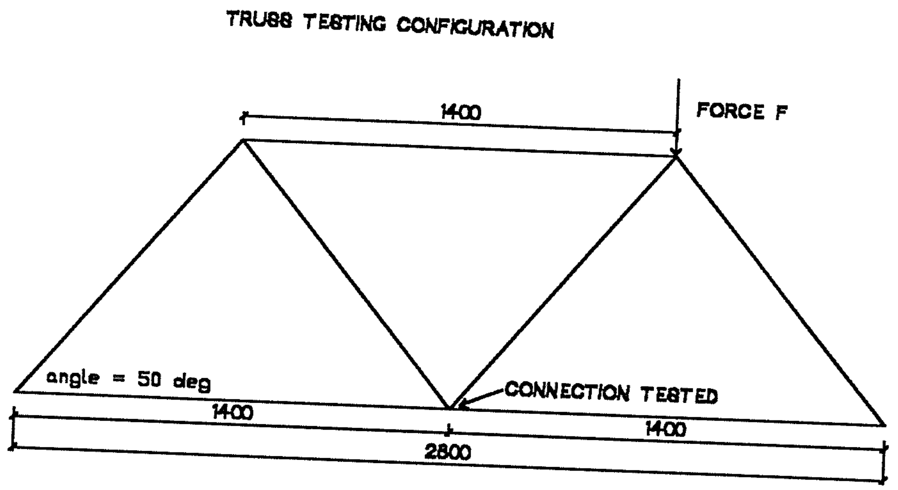
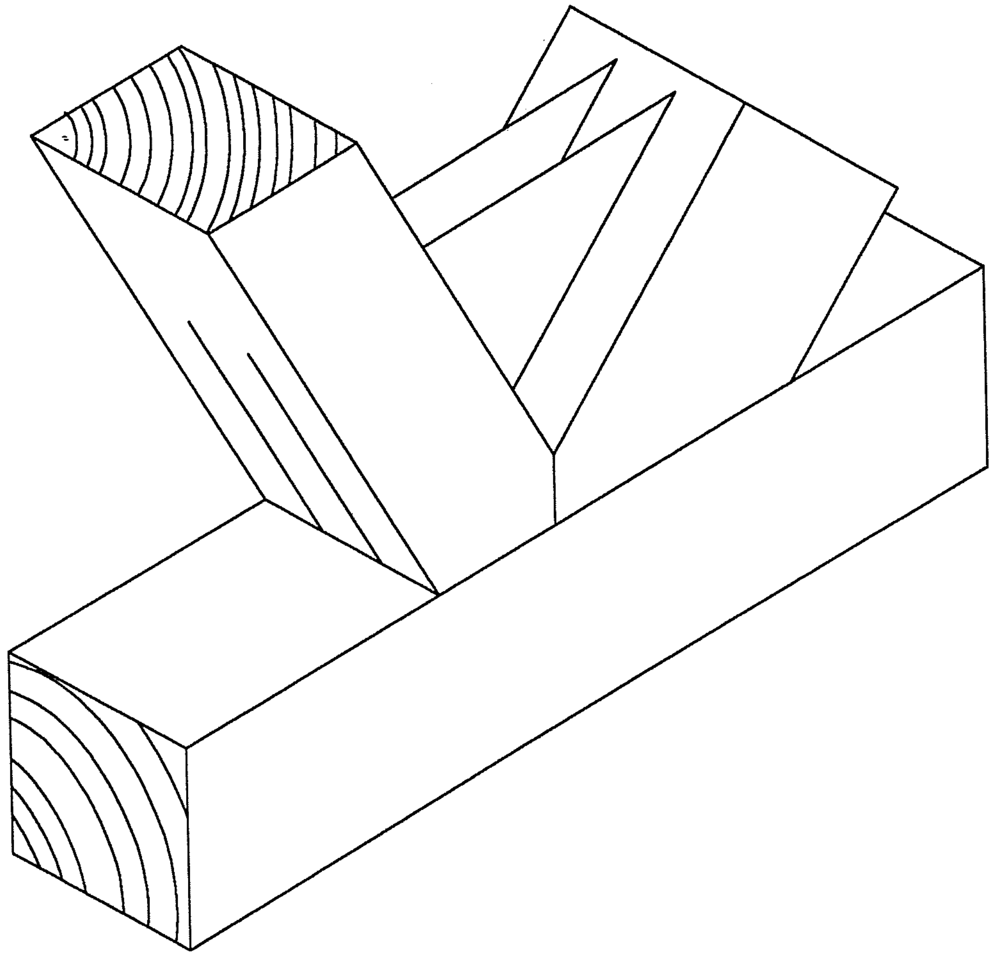



**figure 2.3 - Truss arrangement for Connection Testing at ETH Zürich (Gehri)**

A schematic summary of this research work is presented in **figure 4**, highlighting the relative the efficiency of different fin plate/shear interface arrangements, details for which are presented in **figures 5-7**. In all the testing data the trusses in which the connection type is tested were loaded to destruction in order to determine the ultimate load capacity and behaviour of the connection. The results also indicate not only the ultimate strengths of the connection, but testing confirmed that a careful selection of fin plane interfaces and dowel size will result in a high strength ductile connection which has a "gentle" failure mode rather than a brittle and "catastrophic" timber failure mode.



**schematic of connection tested by Gehri at ETH Zürich**

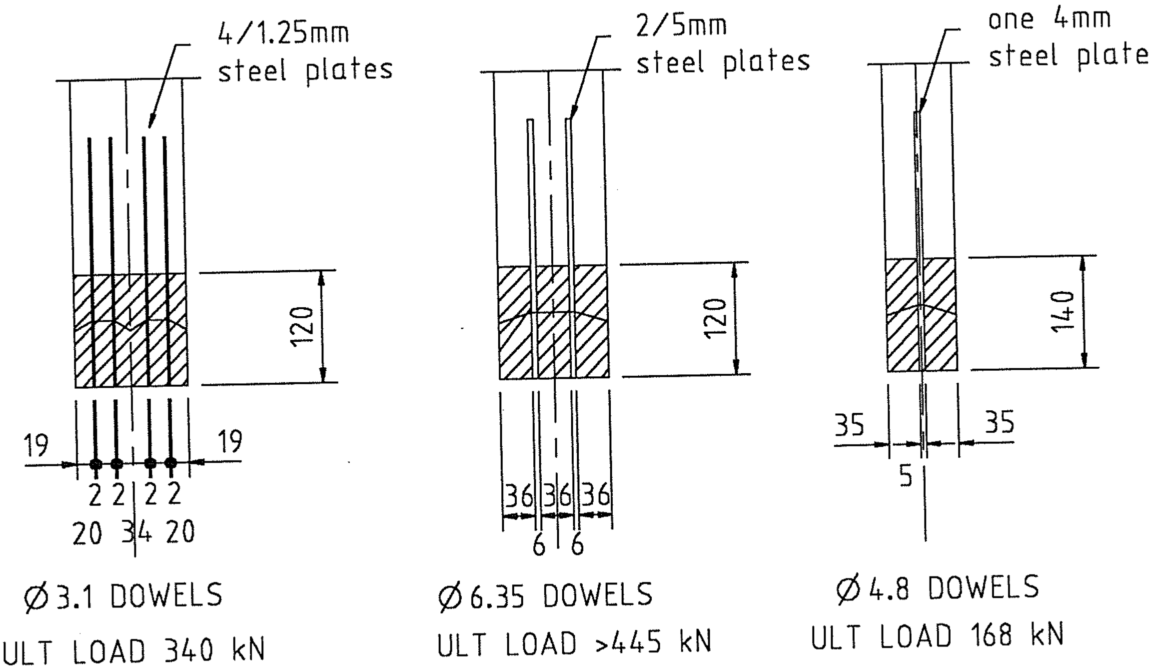
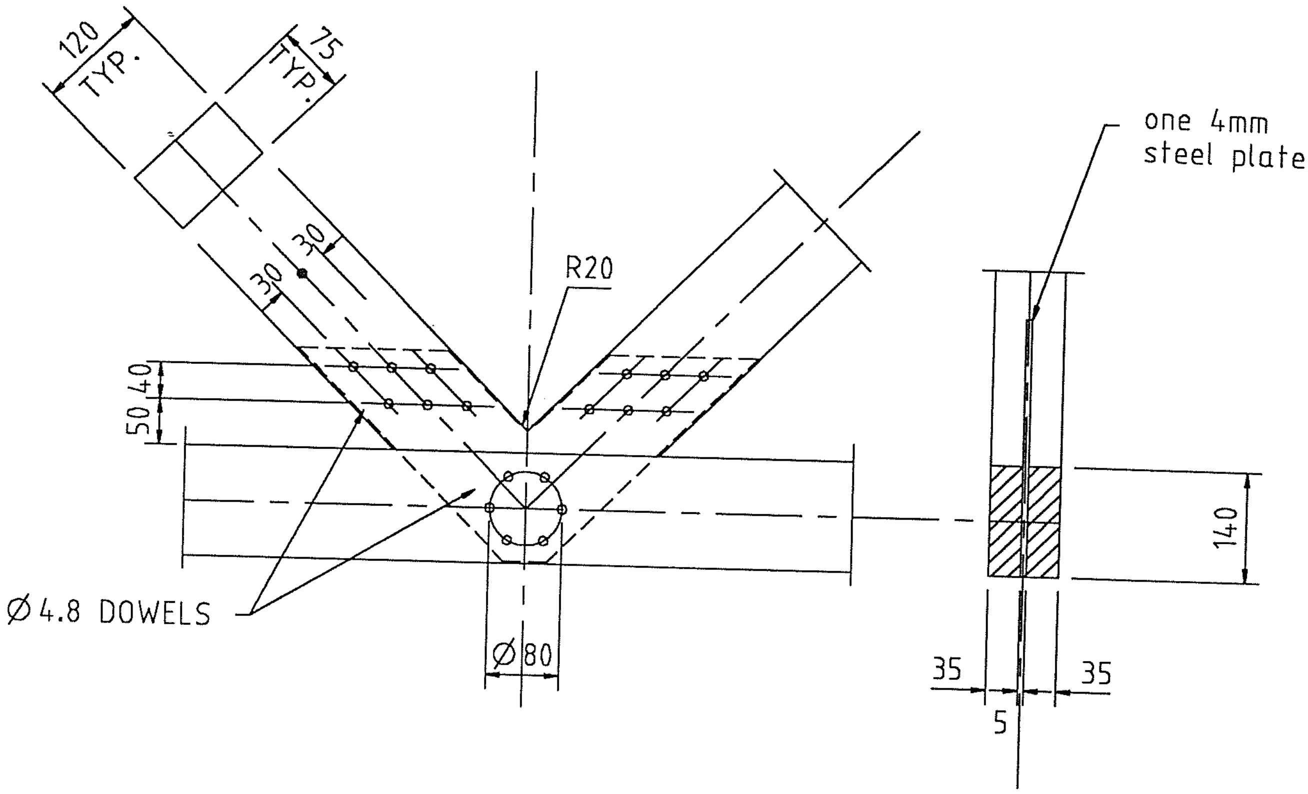



**figure 2.4 - summary of connection / fin plate geometry for truss tests (Gehri):**



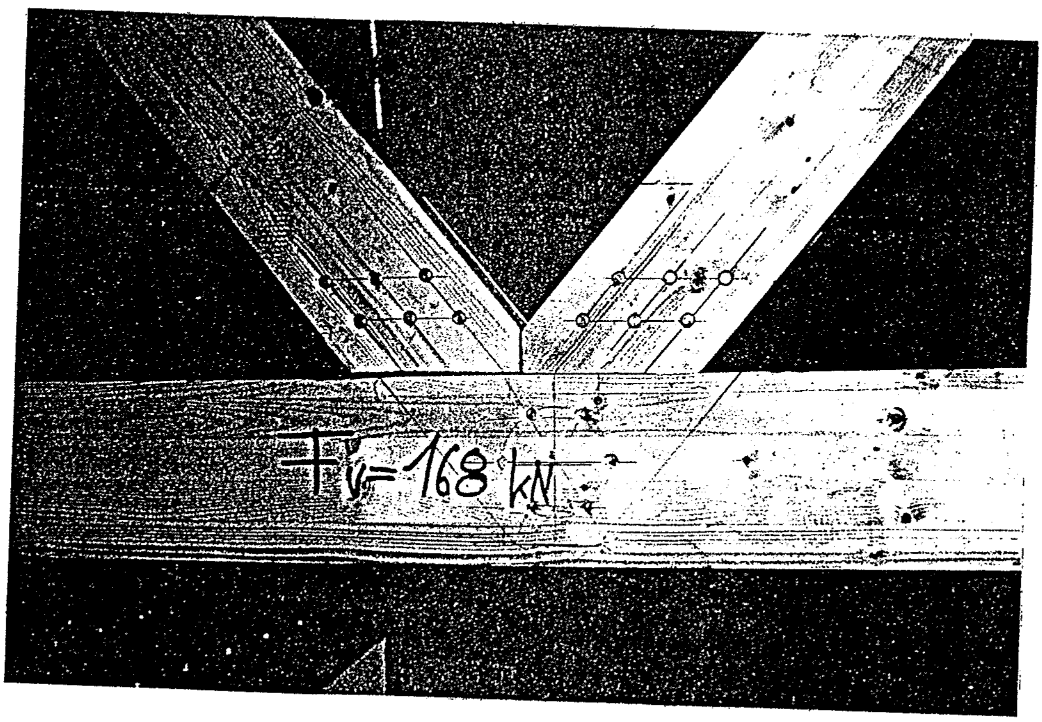
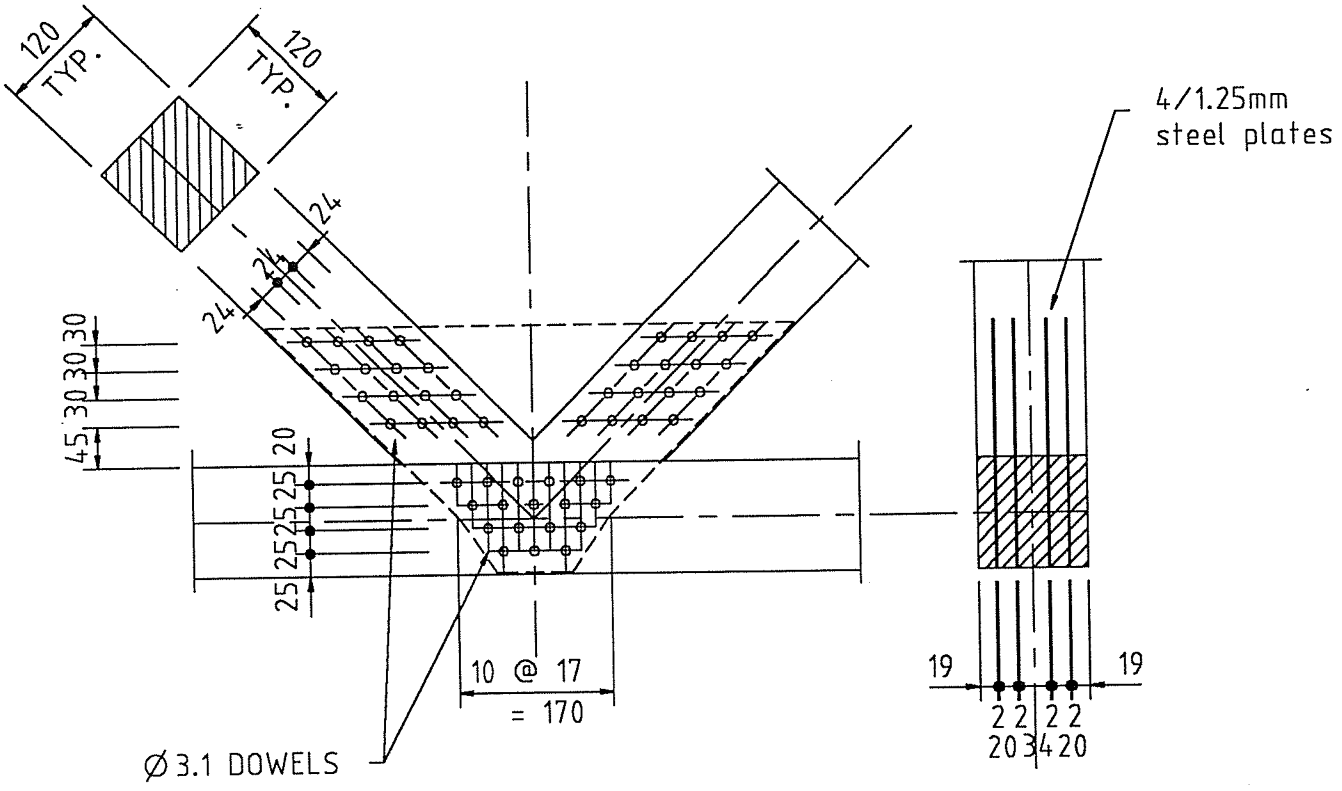




figure 2.5 - single 4mm plate - Ult Load 168 kN



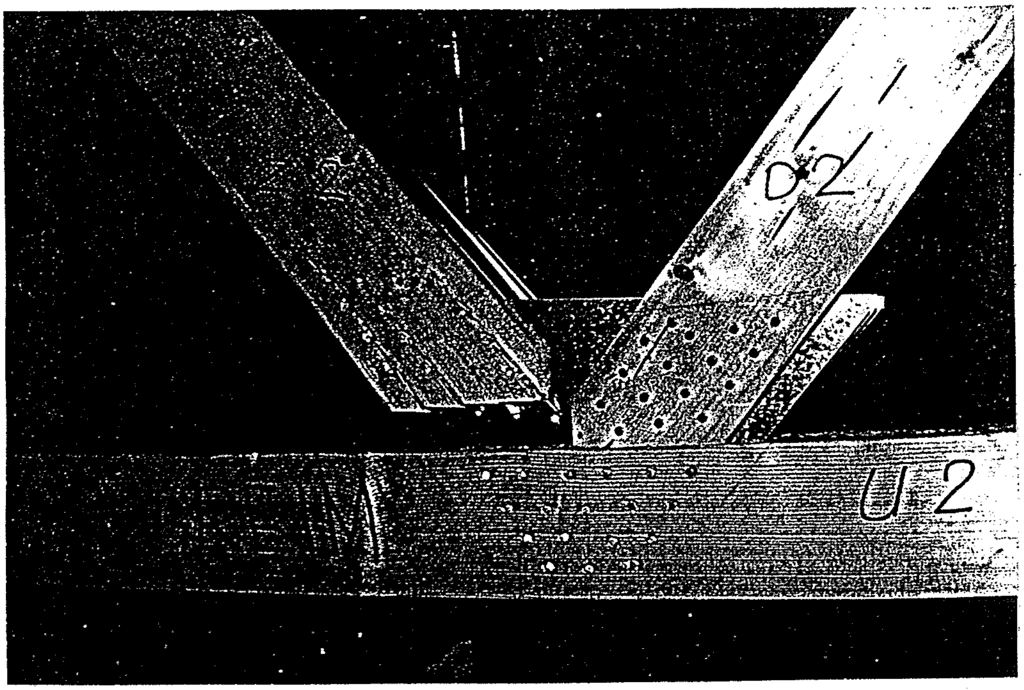
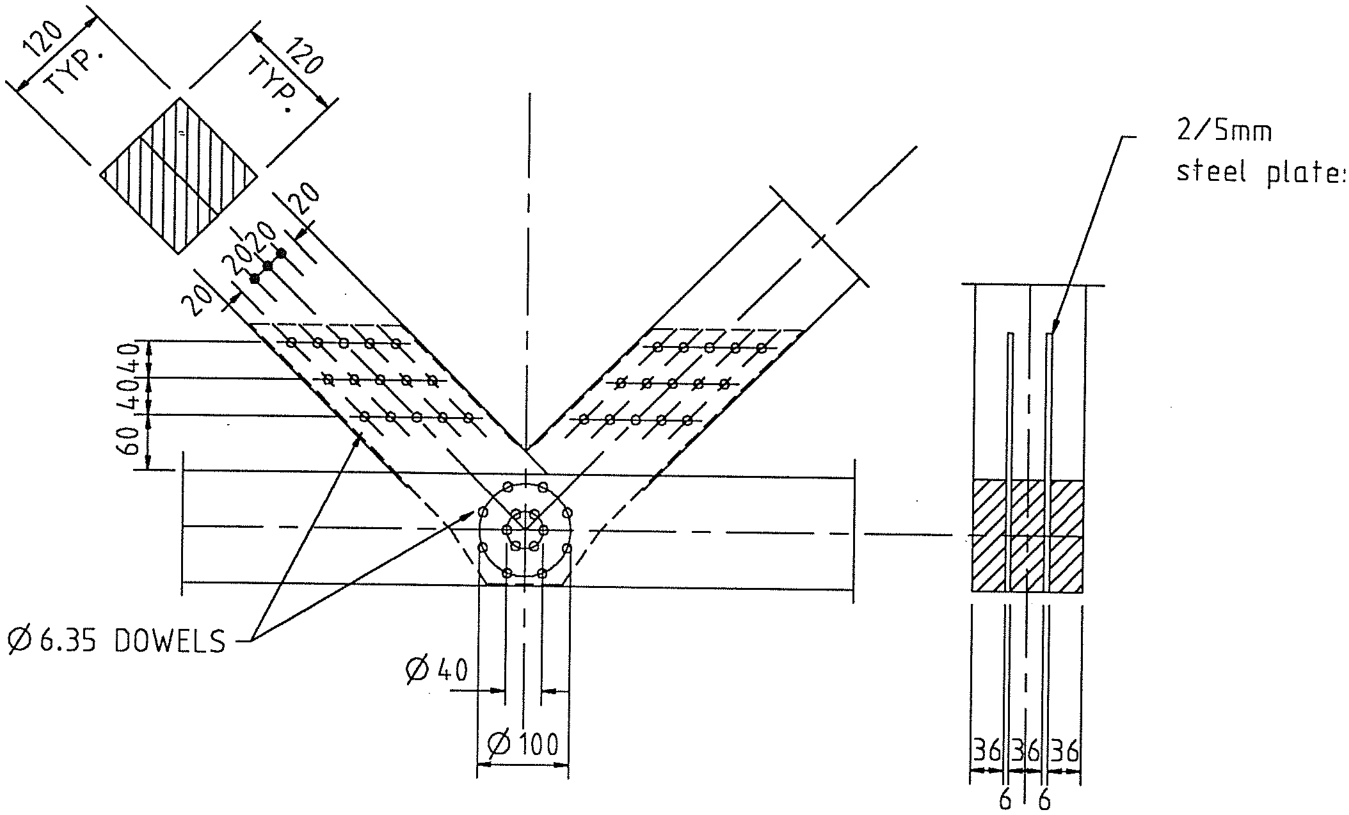




**figure 2.6 - four 1.25mm plates - Ult Load 340 kN**



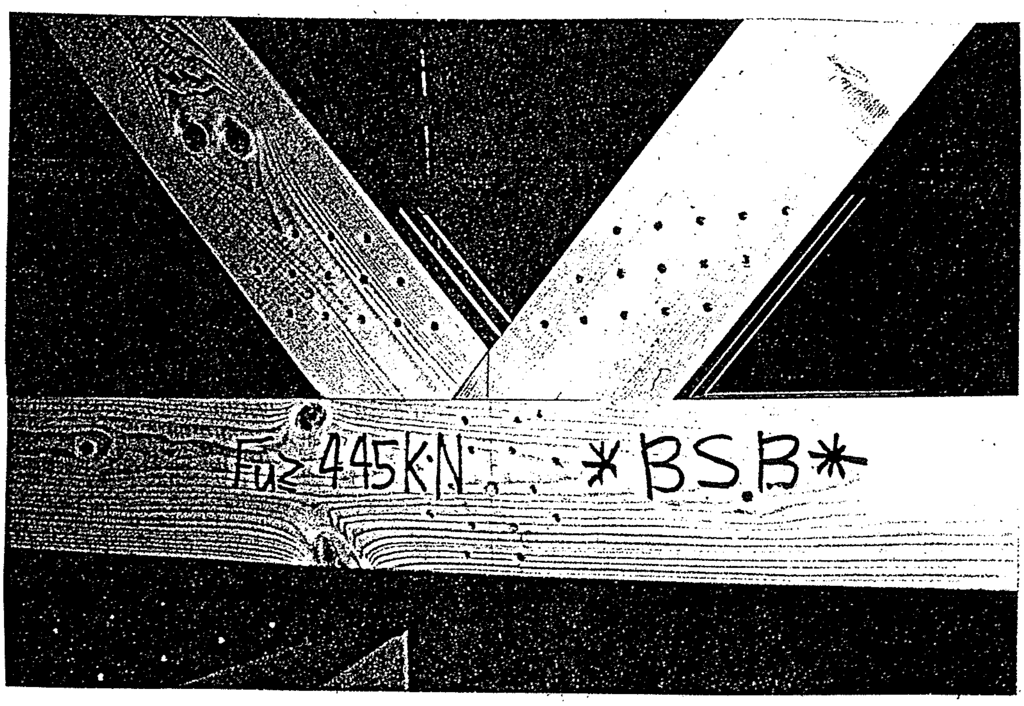




**figure 2.7 - two 5mm plates - Ult Load >445 kN**

**Moment Resisting Connections:**

In addition to the work documented above, which has been essentially undertaken with regard to large magnitude tensile and compressive loads in timber members, a substantial amount of full-size testing has also been undertaken at ETH Zürich, for moment resisting connections using both dowelled fin-plates and glued macro finger joint connections, both of which are permitted under the provisions of Eurocode 5.

The theory utilised by Prof Gehri (like that used by researchers throughout Europe) for the dowelled moment resisting connections, is based upon Johansen's theory with load distribution according to a modified rivet theory. The connections tested were designed for use as portal knee joints for Glulam members and performed in a predictable, ductile manner, although the ultimate failure mode of the connection is governed by the perpendicular to grain tensile forces. However, it is possible to design these connections such that the steel dowels will yield prior to tension perpendicular to grain failures occur, thus ensuring predictable ductile behaviour. (refer to **photograph 2.4**)

The use of the macro finger jointed haunch has had limited application but with reasonable success, in a number of parts of Europe (refer to **fig 2.8** and **photograph 2.5**). Whilst this connection is technically feasible and is permitted under the provisions of Eurocode 5 (although the long term behaviour of the glue / timber interface is still being investigated), it does however, necessitate very strict quality control measures and as such fabrication must occur in a controlled workshop environment. As such, the author does not consider this type of glued moment resisting connection to be feasible for application in Australia at the present time.

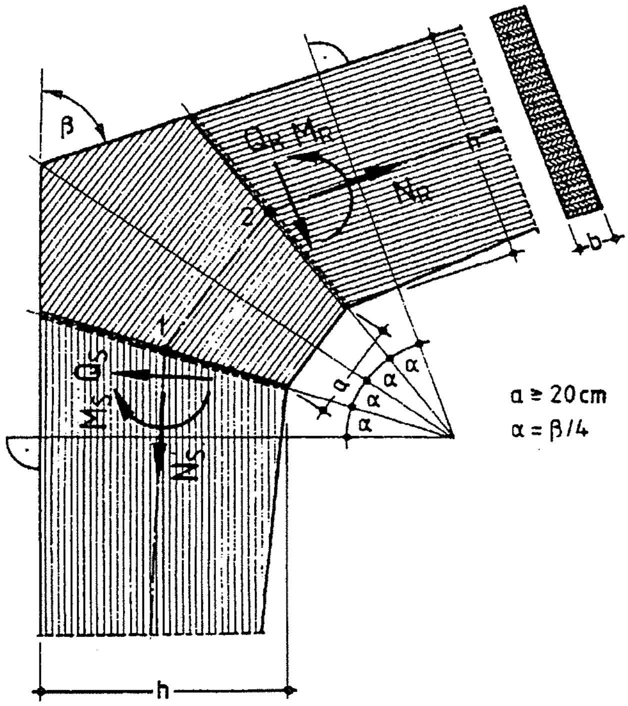
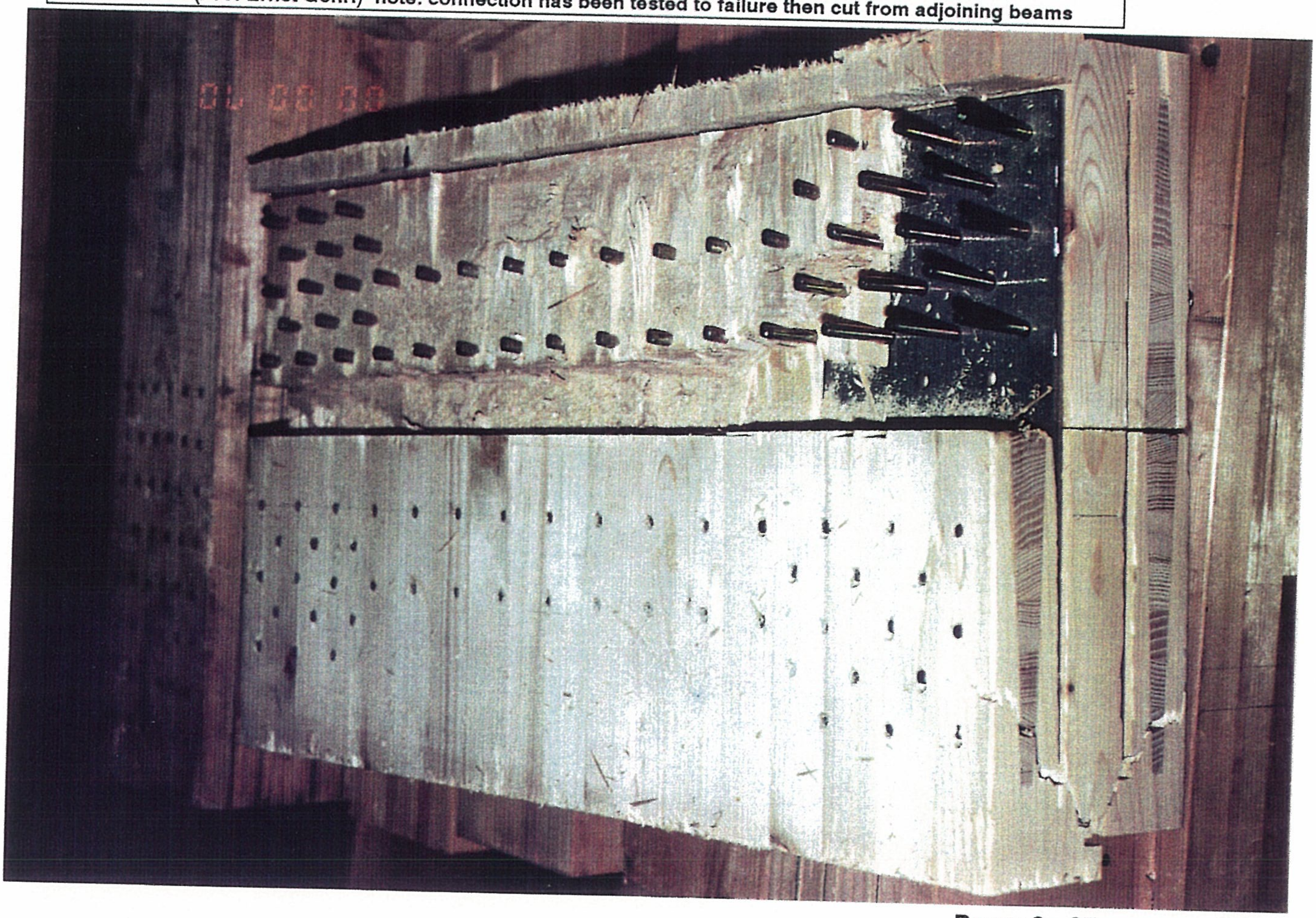



fig 2.8 - Macro Finger Joint detail

The development of this connection into a fin plate (mechanical) connection, using small diameter dowels has been investigated in some detail by Prof Gehri of ETH in Zürich. The aim of this research has been to produce a connection of similar efficiency (about 50-60%) to the glued finger joint connection but with the versatility that it could be more readily assembled outside of workshop conditions and more easily transported, since the glued finger joint haunch is difficult to transport due to its bulky size.

Photograph 2.3 - Example of complex Fin-plate dowelled connection. Church at Gucy - (near Lausanne, Switz)

Photograph 2.4 - Full scale test sample of a dowelled moment resisting connection
ETH - Zürich (Prof Ernst Gehri) note: connection has been tested to failure then cut from adjoining beams



Prof Gehri's work to date on load duration effects of dowelled connections under high load, indicates that long term loading strength effects may not be as critical as previously thought, but further work needs to verify this premise. Ongoing research is currently taking place, investigating the long term strength of these types of connections (refer to **photograph 2.6**) Initial results indicate that long term strength is probably about 70% of the short duration loading strength.

The research also indicates that the failure mechanism for dowelled moment resisting connections is still governed by perpendicular to grain effects in the outer laminates, as a result of tensile stresses being induced by shear effects from the dowel-timber interface. As would be expected, design procedures must control these effects to predictable levels and the connection is designed for ductile behaviour, by optimising the fin plate / dowel geometry to ensure steel yield prior to timber failure.

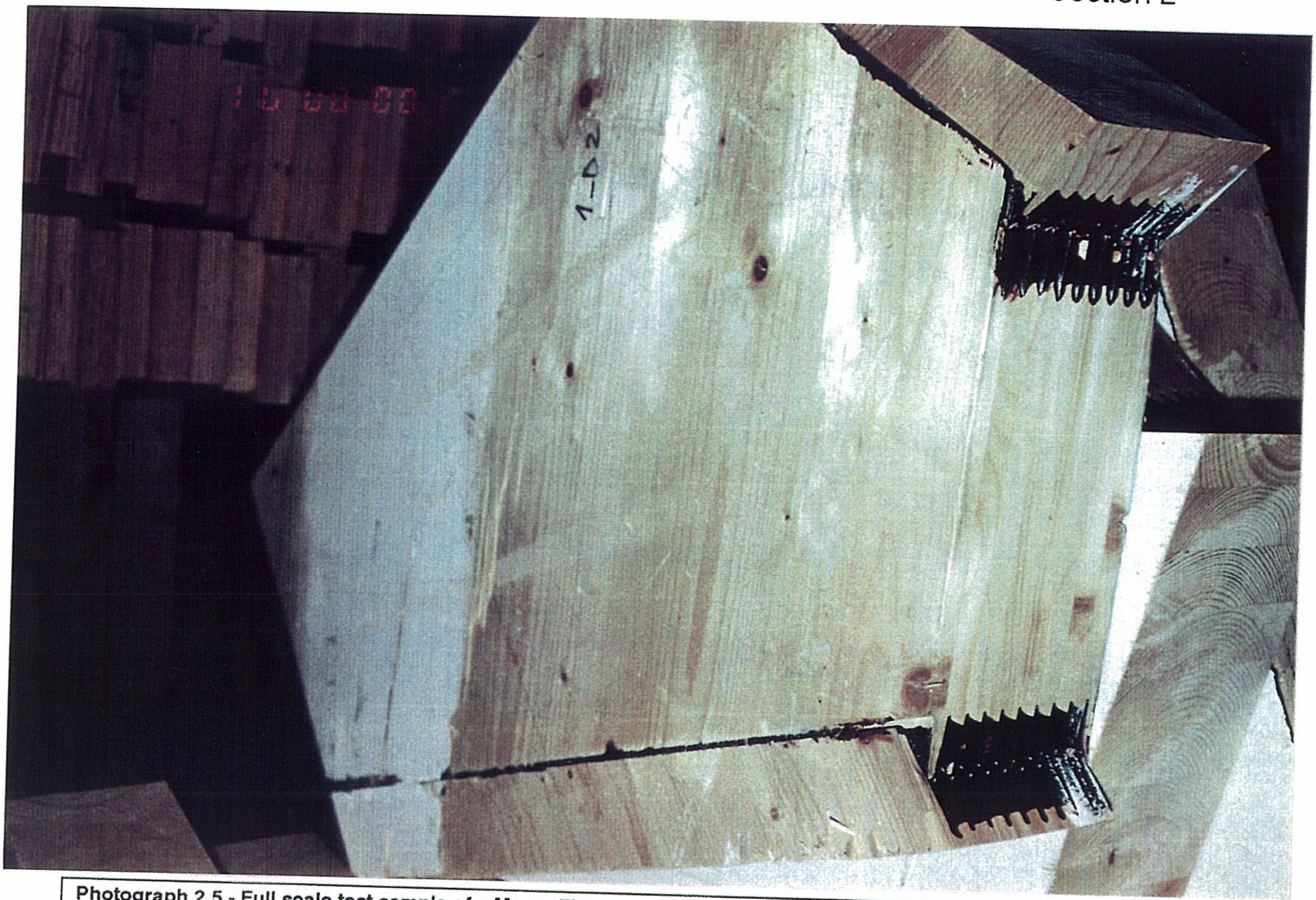
**Epoxy Injected Steel Rods:**

Significant work is also being undertaken at ETH in investigating the use of inserted steel rods (usually injected with epoxy) which are used in areas of localised high shear or high compressive forces on timber members, particularly in the vicinity of the connections. These "stitch rods" tend to act as shear reinforcement in a similar way to that of stirrups used in concrete design. This technique is also used in situations where localised highly concentrated loads occur in timber members, allowing for re-distribution of the bearing stresses away from the outer laminates and facilitating load transfer to sections within the depth of the beam.

Applications of this technology are being specifically developed for use with prestressed post tensioned timber "T" beams using Glulam beam web members with plywood flanges, which are glued and mechanically fastened to achieve shear transfer and composite "T" action between the beam and the deck. These beams are utilising epoxy inserted "stitch rods" to assist in overcoming the high bearing stresses and are commonly being used to span up to 20 metres for pedestrian footbridges and low traffic load applications. The technique is being further developed for vehicular load applications and appears to have considerable future potential.

**Other Significant Connection Research (Europe):**

The other major area of research being currently undertaken in Europe focuses on the design of glued bolts in Glulam for use in shear and moment resisting connections, as both spliced and knee joints. The majority of this work is being undertaken by Riberholt at the Technical University of Denmark and an introductory overview of this work has been previously presented by Syme.[26]




Photograph 2.5 - Full scale test sample of a Macro Finger Joint glued moment resisting connection
ETH - Zürich (Prof Ernst Gehri) note: connection has been tested to failure then cut from adjoining beams

Photograph 2.6 - Load duration (creep) testing of high axial loaded connections

More recent developments of this technology have also occurred in New Zealand at the University of Canterbury, Christchurch and these are presented in greater detail, later in this report.

**Mechanical Connections - North America:**

Current US design practices for determining the strength of a single bolt connection are based mainly upon research conducted by Trayer in 1932. The strength of a multiple bolt connection is calculated by summing the single bolt strength values, after multiplying by a modifying factor which depends on how the load is assumed to be distributed to each bolt. These modification factors are derived from research completed in the 1960's by Kramer[27] and Lantos[28]. Design strength requirements, for both single and multiple bolt connections, are specified by having sufficient spacing and an edge distances as recommended by Trayer. However, recent brittle failures of bolted connections have raised considerable doubts about the basic theory and understanding of behaviour of multiple bolted connections using the current US design practices.

Recent work undertaken by Soltis and Wilkinson of the Forest Products Laboratory at Madison, Wisconsin has focussed on a major re-evaluation of the existing theory for the design of bolted connections and the introduction of European yield theory as defined by Johansen in 1949, to form an alternative method of design of these connections. This alternative method is based upon ductility considerations and the use of smaller bolt diameters, neither of which have been traditionally used in the United States.

**Brittle Failure Modes:**

Traditionally, bolted design in the United States of America has tended to allow the use of single or small groups of large diameter bolts (with resultant very high localised stresses) instead of the approach more generally adapted throughout Europe, which tends to use larger numbers of smaller diameter bolts, which effectively redistribute load induced stresses in the timber over a greater area.

This trend towards using large diameter bolts has resulted in some catastrophic brittle failures in timber, as the connections lack ductility and tend to rely on a limited number of very highly loaded fasteners, which do not yield prior to wood failure. Thus, the design is governed by the load capacities of the timber members, which often fail with little or no warning prior to brittle fracture, particularly if the failure mode is perpendicular to grain tension.

Most of the research to date has been done with parallel to grain loading, for which Yield theory agrees more closely than for the currently available test results from perpendicular to grain loading, for which fewer test data exists.

Dr Soltis also believes that the current research and design literature (in the US) on load / slip behaviour and on the distribution of properties for multiple bolt groups is inadequate, if bolt data is to be used for limit states design or multiple bolt connections. Soltis' research indicates that in the past, *significant conclusions appear to have been drawn from results based on a very small sample size and a narrow range of connection properties*, such as the ratio of end distance to bolt diameter.

It is clear that the existing design practices which base the strength of multiple bolt connections as the sum of single bolt values multiplied by a modifying factor, are somewhat limited in their validity. The modifying factors used are based on an elastic theory of load distribution which is only valid to the proportional limit. The theory is well verified by a number of experimental studies for two bolts in a row where the modifying factor is unity. However, considerably less experimental verification exists for two to four bolts in a row and for more than four bolts in a row, data to substantiate the existing theory is very limited.

The conclusions drawn by Soltis,[29] indicate that the yield theory presents a valid means for assessing the current design practices in the United States of America.

These preliminary conclusions may be summarised as follows:

1/ that neither theory nor experimental data exist in the United States of America to determine load distribution to more than one row of bolts.

2/ No data or theory exists to recommend staggering or symmetrical bolt patterns for multiple rows of bolts.

3/ Similarly the effects of moisture content and shrinkage, particularly with regard to larger bolts is not known.

4/ The effects of other factors such as fabrication, tolerances, duration of load and preservative and or fire treatments has also not been researched.

Soltis has confirmed that the yield theory as proposed by Johansen, predicts joint strength reasonably well. Trayer's empirical curve also fits this theory reasonably well <u>*for single bolts*</u>. However, (unlike Trayer's research) Yield theory also includes wood properties (bolt bearing and compressive strengths) and bolt properties, (yield stress) to give a more rational approach to design and permits adaptions to variable material properties. The yield theory also provides insight into the joint parameters such as the L/D ratio and the effect of changing bolt yield stress.[30]

Discussions between the author and Dr Soltis at the Forest Products Laboratory in Madison, Wisconsin, indicated that future research in the United States of America will focus upon development of analytical models which are in line with yield theory and in effect, will parallel some of the work undertaken by Smith, Hilson and Whale[31], at Brighton Polytechnic in the United Kingdom.

The emphasis that Dr Soltis has proposed, is to move from an empirical method to a much more rational design procedure, which has been verified by experimental testing of full sized connections, particularly for multiple bolt groups and moment resisting connections, (for which no significant research has been undertaken to date). The emphasis will also focus upon developing design procedures which will ensure connections which are ductile in behaviour rather than brittle.

A significant philosophical trend and change in the traditional approach to this type of research in the United States of America appears to be the intention to look at research that has been undertaken outside of United States of America. In the past this has often not happened and Dr Soltis has had some difficulty in obtaining acceptance of Johansen's theory (or European yield theory) by some of his contemporaries in the U.S. However, his validation work at the Forest Products Laboratory has added weight to his argument and is now being generally accepted by researchers in the United States.

**Research on Bolted Connection Design in New Zealand:**

Some recent work on the performance of bolted timber joints has also been undertaken by the Building Research Association of New Zealand (BRANZ - which the author visited in 1989). The aim of this research was to undertake a series of tests on bolts loaded in single shear which incorporated departures in end and edge distances, bolt hole size and washer size from requirements of the New Zealand standard NZS3603, (the 1981 code of practice for timber design). An additional aim of this research was to quantify the effect of such departures, on joint performance. The testing samples were undertaken with joints of 12 mm diameter galvanised mild steel bolts and constructed using Pinus Radiata. Test were undertaken in tension, both parallel and perpendicular to grain and at both the green and dry stage of the conditioning of the timber. The results and conclusions are summarised as below:

1/ An increase in bolt hole diameter did not appear to affect joint performance.

2/ A weak correlation between joint strength and timber density was only found with joints assembled dry and tested dry.

3/ Timber thickness had only a significant affect on strength for joints assembled dry and tested dry perpendicular to the grain.

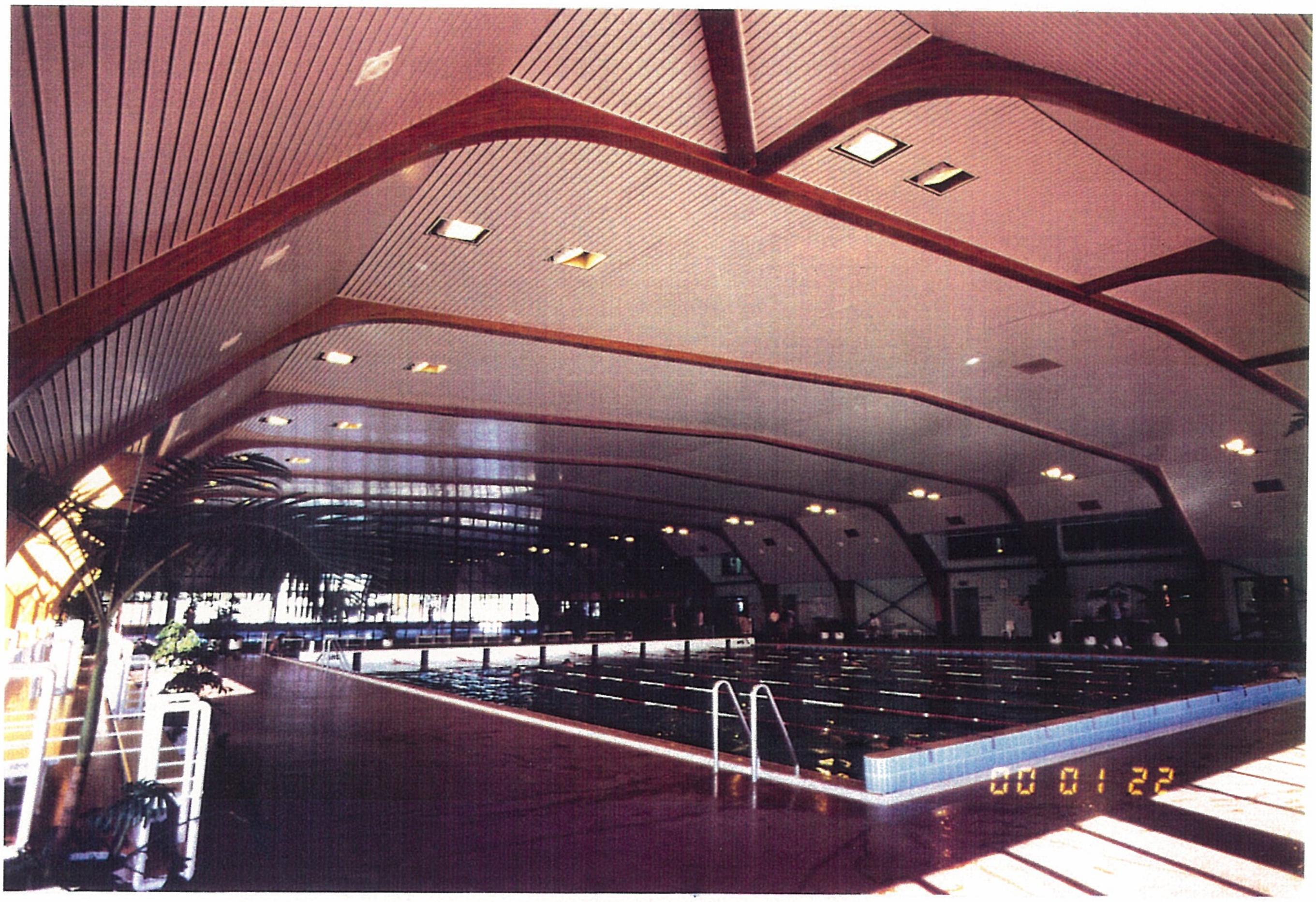
4/ Code design loads appear to be conservative with the exception of joints tested perpendicular to the grain and having reduced end distances. The design loads calculated from the test results were all in excess of the code loads.

5/ Of all joints tested perpendicular to the grain, 20% failed due to splitting of the member away from the joint. This would indicate a possible strength upper limit regardless of the bolt size.

6/ Reducing the edge and or end distances significantly reduced joint strength. Intermediate end distances were tested for joints loaded parallel to the grain and these gave intermediate results.

7/ The effect of coach bolts or small washers on ultimate loads and the load deflection characteristics was found to be significant.

As a result of this study, further research has been recommended into the effect of end distance and the performance on joints loaded perpendicular to the grain as well as into the effect of bolt diameter, particularly where it applies to joints loaded perpendicular to the grain, since a large percentage of failure modes in New Zealand suggested independence of this mode from the bolt diameter for Radiata Pine.[32]




**Photograph 2.7 - Aquatic Centre Rotorua**

**An excellent example of curved glulam beam elements combined with simple concealed bolted connections in a corrosive environment, to create a design solution which is structurally efficient and aesthetically pleasing**

**Nails and Glulam Rivet Connections:**

Over the past 30 years significant research has been undertaken throughout various parts of the world investigating the stiffness and load capacities of nails and nail connections. Considerable work has been undertaken in Australia in this area by the CSIRO, notably by Mack,[33] who in the 1960's undertook investigations into the strength and stiffness of nail joints under short duration loading.

This work continued through the 1970's, with specific emphasis on the grouping of timber species (Australian timbers in particular) for the design of timber joints, linking timber density groups to particular joint capacities and the withdrawal resistance of plain steel nails.[34] [35] Much of this work has formed the basis of the current provisions of AS1720, the Australian Design Code for Timber Structures.

**Load-Slip Behaviour of Nails:**

Similar research work has also been undertaken throughout North America and Europe. Work in the mid 1970's by Dr Ricardo Foschi, investigated the load-slip characteristics of nails. The load-slip characteristic of a nail is defined usually as a nonlinear relationship between a force applied to the head of a nail (which has already been driven into the wood) and the head displacement, caused by the force or forces assumed to have been applied in a normal direction to the nail shank.

Prior to Foschi's work most analytical studies were usually based upon theories of beams on linear elastic foundations, with the reaction proportional to the deformation which had occurred due to the imposed loads.

However, work in the early 1960's by Noren[36] highlighted the fact that the actual nail behaviour is quite different. The load transfer interaction between the nail and the timber was observed to be elasto-plastic, where the wood fails in bearing along the nail shank, with development of crushing of the wood fibres. As well as this, yielding of the nail and bending occurs in some cross-sections and after sustained load, the nail is usually permanently deformed. Foschi's work[37] in the early 1970's dealt with the elasto-plastic problem of nail deformation and through the development of a theoretical design procedure, allows estimation of the ultimate loads upon a nail.

Whilst the theory developed was of a general nature, a particular nail was studied in detail. This nail (invented by Prof Borg Madsen at the University of British Columbia, Vancouver) is usually known as "the Glulam rivet" and is used in heavy glue-laminated construction; being driven into the timber member through a pre-drilled steel plate, as shown in **figure 2.9**.

Foschi's research showed that the linear approximations based on the theory of beams and elastic foundations is not an accurate method for predicting the ultimate load capacities of nails. However, these approximations are valid for providing an estimate of initial stiffness, especially in the case where nails have their heads rigidly clamped to the plate connector.

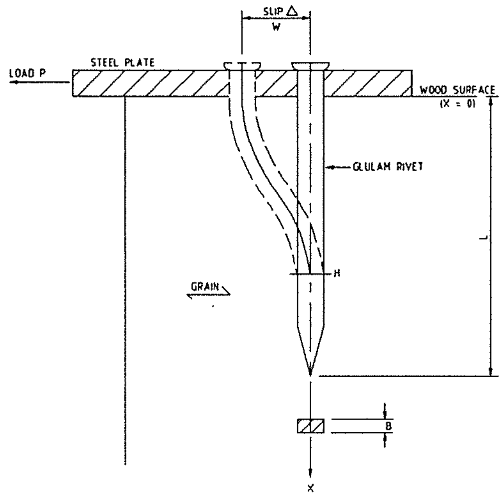



**figure 2.9 Glulam Rivet**

Prof Foschi's work has led to the development of analytical models which are useful in predicting the yielding mechanism of the nail due to bending, and the nonlinear bearing behaviour of the wood material due to the effects of load. This model is then useful for determining both ultimate and allowable loads for various types of nail and small bolt connections.

Further work by Foschi and Longworth[38] investigated group behaviour of Glulam rivets and nails and the effects of loads distribution and nailing patterns. This research work developed a stress analysis method for Glulam rivet nail connections for loads both parallel and perpendicular to the grain. Various failure modes were studied and as a result, ultimate loads were derived for these various failure modes and a rational method of analysis for timber connection design was developed. The conclusions of this work were:

1) Nail spacings control the mode of failure of the connection, with a wider spacing producing nail yielding modes.

2) Closed nail spacings may produce wood failures, usually by shear around the groups of nails. This type of failure is quite sudden and can occur at loads much lower than the nail capacity. Such brittle failures are undesirable.

3) For the same nail spacing a longer end distance increases the ultimate load based upon shear failure.

4) It is possible to estimate failure loads for either mode of yielding of the nails and bending or wood failure around the group nails and to design the connection so that wood failures will not occur before nail yielding, thus optimising nail utilisation.

Throughout the 1980's until the present date, Prof Foschi has continued to develop analytical techniques for the design of nailed connections which have been tested and correlated with experimental data and have been found to be very effective models in accurately predicting the performance and properties of a nailed or bolted timber connection. Much of this work has formed the basis of the Canadian Design Codes and Foschi's world renowned expertise in this area of analytical modelling and the development of reliability based design procedures had meant that the work undertaken at the University of British Columbia has major significance for design of mechanical timber connections throughout many parts of the world.

**Eccentric Loading Effects:**

Additional work in the late 1980's by Foschi and Karacabeyli[39] investigated the behaviour of connections under eccentric loading, again with particular application to Glulam rivet connection. Three essential aspects of eccentric loading have been addressed:

1) what is the failure load when controlled by rivet yielding and wood bearing?

2) what is the failure load when controlled by wood failure around the rivet cluster?

3) how do various connection parameters influence the ultimate load carrying capacity of the connection?

The conclusions (to date) of this investigative work highlight the fact that the load carrying capacity for wood failure can not easily be calculated for these type of connections. Foschi's work describes a method of analysis (which can be undertaken using desk-top computers) for the development of design tables that take into account the eccentric effects.

**Perpendicular to Grain Effects:**

In general terms, the research work highlights the need for connection detailing to minimise the possibility of high tension perpendicular to grain stresses, which tend to be the usual failure mode for this type of connection. Interestingly, this perpendicular to grain failure mode characterises the failure mechanism for virtually all connections which involve either moment resisting capabilities or eccentric loading effects.

Ongoing research work is continuing at both University of British Columbia and Forintek, with the aim of further developing predictive models and design aids for the use in the design of high load capacity connections using nails and

Glulam rivets. It is important to note that whilst much of the work undertaken has focussed on the use of the Glulam rivet connection, the theory involved has general applicability to most nail and bolt/dowel connection types. Recent work by Karacabeyli and Fraser[40] has focussed on specific application of these connection design techniques not only to Glulam members, but also to solid timber with an emphasis on the performance characteristics of specific timber species.

The author was privileged to spend two weeks in October 1990 at the University of British Columbia investigating these design techniques under the supervision of Professor Ricardo Foschi and Professor Borg Madsen, and as a result of this experience, is of the opinion that the techniques developed at University of British Columbia and Forintek have direct applicability to the use of mechanical connections for Glulam and LVL timber engineering applications within Australia.

**Fracture Mechanics of Nailed Connections:**

The author also had the opportunity to spend several days at the University of New Brunswick with Dr Ian Smith. Dr Smith has previously undertaken extensive research into nailed and bolted connection design,[41] [42] (as referenced elsewhere in this report) and in discussions with the author, some considerations concerning the design and performance of mechanical connections were also reviewed.

Dr Smith expressed the opinion that a fundamental basis for design of mechanical connections which utilise "driven" fasteners (eg: nails), is to use a blunt fastener such a Glulam rivet which breaks the fibre, rather than a sharp fastener, which tends to split the fibre. The rationale behind this is based on investigative work undertaken by Dr Smith, which suggests that splitting of the fibres (which can occur with small nails), can in fact create micro-shear planes within the wood structure, thus inducing an inherent weakness in the timber member which in turn reduces the strength and / or factor of safety in a connection.

Also, the way in which a connection fastener is driven appears to be of considerable importance. It is much more preferable to drive a connection through external plates in such a way as to ensure that the head of the connection remains parallel to the outside plate and is not free to rotate. Doctoral work undertaken by Hunt[43] at Auckland University in 1984, concluded that head restraint and the angle of loading with respect to grain, have a significant effect on the strength and stiffness of nailed joints. Investigative work by Dr Smith at the University of New Brunswick is in agreement with these research conclusions (presented by Hunt and Bryant),[44] regarding rotation of the head of a mechanical fastener. Similar conclusions have been reached by

Foschi, in his investigative work into the performance of Glulam rivet connections.

Dr Smith's research into performance of mechanical connections has focussed of late on the intrinsic properties of timber as a material. Design with timber is not just a problem of geometric properties (eg: such as edge distance clearances, bolt/nail pattern and spacing) but is also inherently linked to the wood science/organic properties of the material which involve rheology, growth rates, fibre structure, and an understanding of timber as an anisotropic, non idealised material. One of the key areas to understanding failure mechanisms within timber particularly with regard to mechanical connections is the need to develop a sound understanding of fracture mechanics and the relationship of this to the wood science properties of timber.

Another key area of research which Dr Smith highlighted during the author's visit to the University of New Brunswick, was the need for an understanding of the dynamic responses of mechanical connections and the effects of cyclic loading, load history, and fatigue, particularly for smaller connections such as nails. A modal synthesis method of analysing these variables and developing predictive tools is being developed by Dr Smith in order to gain a greater understanding of dynamic analysis and load / history responses of structures.

**Connection Categories:**

Generally speaking, Dr Smith considers that connection types can be categorised into two essential groups. There is the "one-off" larger type of connection, which necessitates full-scale testing and could be defined as a "quality controlled" connection. Dr Smith feels that the best approach to the design and use of such connections is that generally adopted by Swiss designers, notably Professor Julius Natterer and Professor Ernst Gehri, who test **full-scale** prototype connections under service loads in order to develop predictive models which will quantify joint carrying capacity.

The second group involves the use of multiple connections; smaller fastenings which are more widely used rather than the one-off type connection. Such connections need to have their performance criteria determined by sample testing and then characteristic values of that sample can be used to define characteristic strength and serviceability responses for those particular types of connections.

Interestingly, it has been Dr Smith's experience to date that custom built buildings still predominate for the glulam industry in North America and Europe, rather than standardised mass produced buildings, so in many situations the first category of joints and the approach of testing prototype connections is likely to predominate for some time. The overriding design consideration for the design and performance of connections, is that failure must occur in the connection, not in the members and that this failure should be a ductile failure of the steel elements, rather than a brittle failure of the timber members.[45]

**Australian and New Zealand Research:**

The use of nailed steel and plywood gusset joints as moment resisting connections is now widely accepted for knee and apex joints in portal frames made with glue laminated or LVL timber members. Much of the early research into this type of connection was undertaken by Walford and others at the Forest Research institute at Rotorua in New Zealand during the 1970's. Walford's application was similar to the work undertaken by Madsen and Foschi using the glulam rivet.

However, the application in New Zealand was unique, in that the joint was designed for moment resistance, whereas only direct loads were transmitted in the first examples of the glulam rivet connection. An Auckland engineer, Mr Tony Gibson began using the concept commercially in timber portal frame buildings (using steel plate gussets) and Mark Batchelor further developed the technique whilst undertaking Masters studies at Auckland University. Batchelor used plywood gusset plates in order to avoid the cost of predrilling the steel and to enable the use of pneumatic nail guns.

The results[46] [47]of Bachelor's work were presented at the first Pacific Timber Engineering Conference at Auckland University in 1984. It was from this conference that the concept of plywood gusset plates was translated across the Tasman and applied in the Australian building industry, (notably by Mr Kevin Lyngcoln - Plywood Association of Australia and Consulting Engineers Peter Law, Bruce Hutchings and Peter Yttrup) being actively encouraged and promoted by the various State Timber Promotion organisations.

**Methods of Analysis and Design:**

Design procedures for this type of connection have continued to be developed in New Zealand, notably by Dr Brian Walford at the Forest Research Institute (FRI) in Rotorua[48] and by Dr Tony Bryant and Dr Richard Hunt at Auckland University.[49] [50] [51]

Most of the design procedures used in Australia and New Zealand have tended to assume a linear load slip relationship between the nail and the glulam members. This has been based upon either:

a) thin wall tube analogy or
b) shared rivet group analogy.

In thin wall tube analogy, the nailed area is likened to the cross section of a tube subject to torsional loading. Shear stress within the nailed area is assumed to be constant. In shared rivet group analogy, the load on the nails is assumed to be proportional to the moment being resisted by the connection and the distance of that point from the centre of rotation, (which is often assumed to be a constant point at the centroid of the nail group).

Both these design procedures assume simplifications about load distribution and stress concentrations within the moment resisting connection. More detailed and rigorous methods of analysis and design that take into account nonlinear rivet group theory have been assessed by Walford [52] and compared with the simplified linear models. Walford's research concludes that nail loads at present used and specified by New Zealand Standard NZS3603 are probably too conservative for gusset joint design.

**Photograph 2.8**

**Nailed plywood knee connection for a 18m span glulam portal framed factory (Hunter Laminates - Nelson)**

Several reasons are given in support of this conclusion:

1) The large number of nails used in these joints means that the variability is reduced.

2) In cases where steel side plates are used fixity of the nail head results in stiffer joints. Increased head rigidity results in significantly less (greater than 50%) observed slip for a given load, as well as an increase in the maximum capacity of the connection.

3) The large number of nails generates pressure between the gusset and member resulting in a significant moment capacity due to friction. It is noted that it is possible for this pressure to dissipate in time as wood shrinks and swells with seasonal moisture changes and as such if it is not recommended that this behaviour be relied upon in the design of gusseted joints. It is also difficult to quantify the frictional component as a part of the design procedure, with any degree of confidence. [53]

**Nonlinear Behaviour:**

However, recent work undertaken by Bryant and Hunt at Auckland University suggests that some of the simplified assumptions presented in Walford's assessment, may not in fact be as conservative as first thought. Full-scale testing of nailed gusset plate moment resisting connections (at Auckland University), has shown that in certain situations, embarrassingly low nominal stresses occurred in the outer laminates at the point of failure. Hunt and Bryants' work concludes that these nominally low stresses have occurred because:

1) Stresses in the joint region can be significantly higher than those predicted by the usual simplified (linear engineering calculations)

2) Nails or nail holes trigger failure in clear grain glulam laminates at average values of stress that are sometimes one third of the standard modulus of rupture test values. (These results give confirmation of the fracture mechanics approach proposed by Dr Ian Smith).

3) Nail patterns have a significant effect on stresses in the joint region. Patterns that concentrate moment resistance in outer laminates cause higher stress than patterns that distribute resistance over many laminates. As such stress concentration effects and the effects of nails on wood strength should be included in a rational design method. [54]

These conclusions (drawn by Hunt and Bryant) are in line with the concerns expressed by Dr Smith at the University of New Brunswick and Prof Foschi at the University of British Columbia in Canada. The problem of fatigue has probably not been a major issue to date, because of the fact that whilst the nails do yield at relatively low loads they seem to have a high plastic load carrying capacity and because there are so many nails in these types of connections, there is considerable scope for load sharing to occur.

The most comprehensive research work that the author has been able to find on these types of nailed connections, is that which has been undertaken by Hunt (and Bryant) at Auckland University as a part of Hunt's Doctoral work.[55] Hunt's research confirms that a nonlinear analytical approach is much more acceptable, as it can highlight stress concentrations and takes into account more accurately, the mechanical properties of both the nails and the timber.

Discussions between the author and Doctors Bryant and Hunt at Auckland University in December 1990, confirmed that ongoing research into this area needs to continue and much tighter controls need to be put into place, for not only the design of these connections, but particularly for specification and use of nails. The strengths of nails are not generally stipulated by relevant design or manufacturing standards which control material quality, and this situation needs to be rectified by some sort of performance specification to which any nails must conform.

**Failure Criteria:**

Current research work by Hunt and Bryant focuses on the failure criteria for timber in joint regions. The author's discussions with Dr Bryant and Dr Hunt, reviewing the moment resisting joint experiments at Auckland University, indicated that significant numbers of joints failed as a result of brittle timber fracture in the outer timber laminates at loads that are below those that engineers might expect from rules in the New Zealand Design Codes.

The results of this research are based on laboratory testing, undertaken in conjunction with finite element analyses that take into account the orthotropic nature of wood and realistic load/slip relationships for each nail (based on work by Foschi) and deformations of the joint plates. The results to date, show that some joint designs give timber stresses that are significantly higher than those given by usual engineering calculations and that tension tests of the outer laminations in particular, show that brittle perpendicular to the grain tensile failures can occur at stresses close to permissible design levels. The results conclude that nails can trigger tension failures at average stresses which are at about the same as the allowable design "failure" stresses which occur in the timber of moment resisting connections.

Subsequent to this research, Hunt and Bryant have published the following recommendations with regard to design of moment resisting connections using nails:

a) Ensure that the nail pattern is such that nails do not cause increases in nominal stresses, in the higher stressed outer laminates of a glulam member.

b) Spread the nails over a reasonably large proportion of the joint areas so that the structural effects can be shared over many laminates and that the dangerous consequences of too many nails in a weak laminate are avoided.

c) Do not put any nails in the outer laminates. This will mean that the outer laminates are not weakened by the presence of nails and also helps to reduce high stresses in the outer laminates.[56]

**Long Term and Dynamic Behaviour of Nailed Portal Joints:**

Two other areas of concern and necessitating further research were noted by the author in discussions with researchers in New Zealand during November and December 1990.

Firstly, discussions with Dr Peter Moss at the University of Canterbury, Christchurch, highlighted the need for a more thorough understanding of the dynamic responses of portal frames which have moment resisting connections built up from nailed plywood gussets. The concern expressed is that whilst the nails are probably only being loaded to about 40% of their ultimate load capacity, nevertheless, the service loads often produce plastic deformation of at least some of the nails (usually in the corners of the nail pattern), whereby the flexibility of the joint is considerably increased due to load / slip deformations.

Neither the effects of fracture propagation within the timber fibres (due to splitting of the fibres) nor the effects of stress reversals on the nails is clearly understood. As a result, it is a distinct possibility that over a period of time when stress reversals have occurred due to either wind or earthquake loading, the cyclic nature of these stress reversals may in fact be reducing the long term capacity of the nails as well as inducing stress concentrations in corners of the moment resisting connections. This may become more of an issue as the change to limit state formats occurs where a limit state may be the "load history" taking into account the cyclic loading of the structure.

It must be noted however, that the elasto-plastic response of these types of connections does appear (at present) to give a satisfactory factor of safety, but pressures to increase the allowable nail loads and a lack of understanding of long term dynamic responses may erode this factor of safety to a point where structural failures could occur.

The second area where research may need to be addressed was highlighted in discussions with Dr Bryan Walford and Mr Hank Bier at the Forest Research Institute in Rotorua. As mentioned previously, much of the initial investigative research into this type of connection was undertaken by FRI. The author's discussions at FRI highlighted the desirability and need for a research program encompassing ongoing monitoring of buildings which are constructed using nailed gusset plate moment resisting connections, to assess load histories and to monitor responses over time, for these types of portal framed structures.

Such an exercise would be invaluable in refining design procedures and looking at the load histories of these types of structures. It is the author's belief that the ongoing research into these types of connections which has been undertaken in New Zealand at the present time, could have significant implications for design procedures which will lead to refinements and a better understanding of these types of connections, both in New Zealand and in Australia.

**Recent research on nail connections in Australia:**

Over the past twenty years, a considerable amount of research has been undertaken into the performance of nailed connections using Australian timber species, in particular, hardwoods. Much of this work has been undertaken by the CSIRO, in particular by Mack[57 58 59] and more recently by Lhuede. Recent research by Lhuede[60] has investigated the performance of nails in plywood / solid wood connections and concluded that the interaction of the nailed plywood connection with solid timber, accords well with the predictions of the Johansen yield model when the measured embedment stresses in the plywood and solid wood are used in the relevant equations.

The nail capacity of joints constructed with plywood sides and solid timber centre pieces, was found by Lhuede to be up to 17% higher when compared to solid wood / wood connections (for 12.5mm thick plywood and 2.8mm diameter nails. i.e. T/D=4.5). However, at lower values of T/D, for example on T/D=1 with 4.5mm nails, the load capacity of the plywood sided joints was 70% of the solid wood connection. This work suggests that current code provisions may be conservative where wood splitting does not occur.

A significant transfer and application of European technology to Australian timbers has been recently undertaken at the University of Technology, Sydney, by Christophe Sigrist. Much of Sigrist's work has focussed on the relationship between shear interfaces, nailing patterns, edge and end distances and joint capacities, for high capacity tension connections using nails.

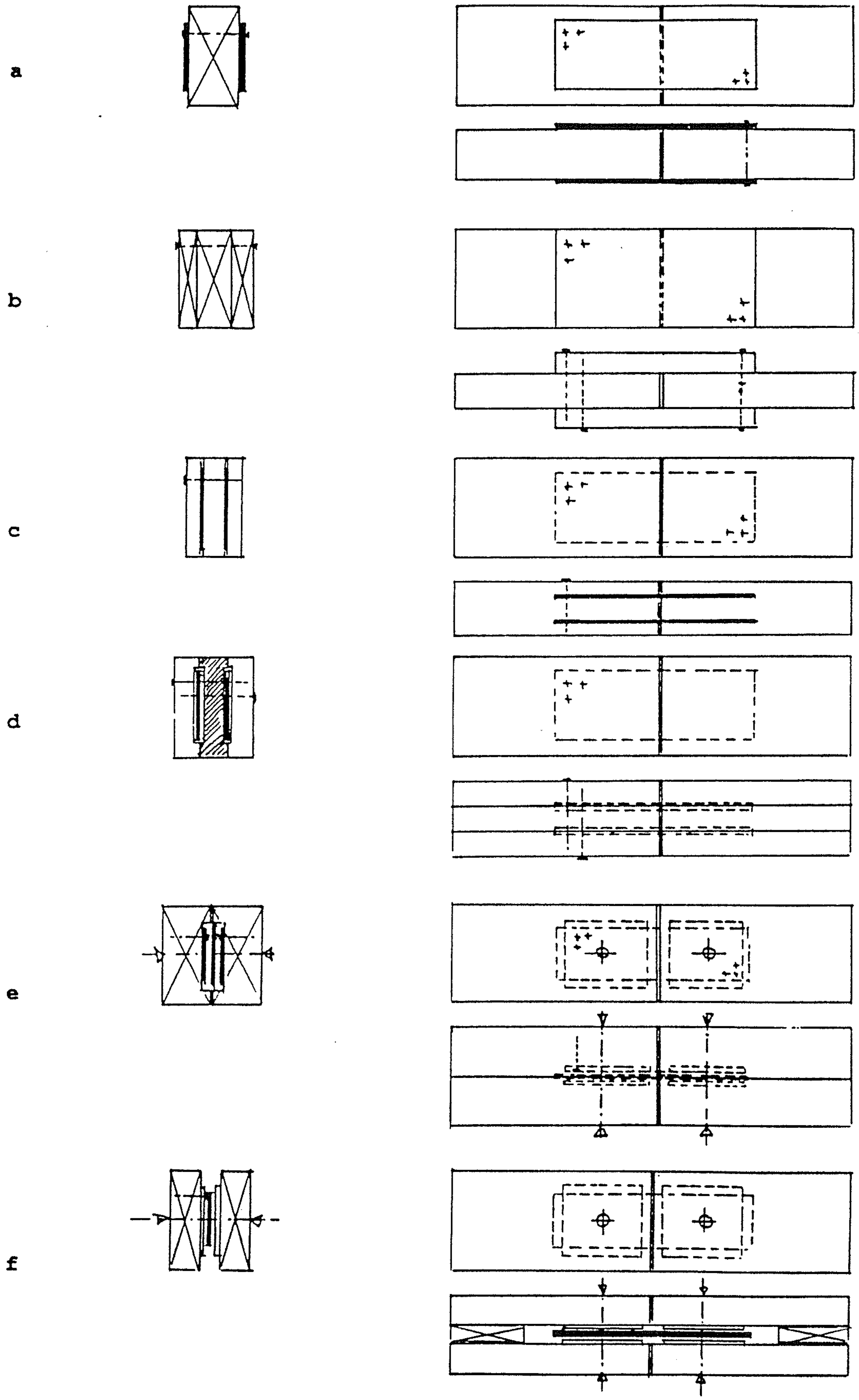



**figure 2.10 - Possible tension splice layouts for high capacity nailed joints (Sigrist)[61]**

**Figure 2.10** shows a number of possible layouts for tension joints using single/multiple members and nails working in single/double shear. Work by Natterer and Sigrist in Lausanne[62] and by Gehri and Fontana in Zürich[63] has shown that connections using nails in multiple shear, exhibit much more desirable load-deformation behaviour than similar connections which load the nails in single shear.

Slenderness conditions for nails are similar to other types of rod connections and the optimum member width / diameter ratio can be defined as follows:

**T/D** = 6-10 for internal members and 4-8 for outer members

where:-

"**T**" is the member thickness & "**D**" the diameter of the connector.

Application of multiple shear joints in Switzerland has been further developed over recent times to use small dowels and / or nails particularly in connections of the type shown in **figure 2.10** - cases "**c**" and "**d**". Nailed connections which utilise short nails with special shanks in order to fix steel side plates onto the timber member have now become firmly established in Switzerland as a viable structural application for a timber connection. The distinct advantage of this type of connection is that it has led to a very high degree of pre-fabrication, allowing much of the work to be undertaken under ideal "workshop" conditions, which in turn, ensures a precise and well finished building member, reducing on-site fabrication costs. Critics of this approach should note that this is typical of existing practice for most steel fabricators.

Practical nail diameters vary from about 3.5mm to 6mm in diameter with corresponding lengths from about 40mm up to 100mm. In many situations the use of steel plates in thicknesses from 3 to 8mm has been found to produce connections which are both economical in terms of cost and statically desirable as an engineering solution. The nail holes are normally pre-drilled using the prefabricate nail plates as a template and by use of this type of connection in conjunction with reinforcing plates (which is quite a common practice), it is possible to design connections with a load carrying capacity of up to 500kN. The reinforcing plate is a safeguard against local instabilities or buckling of the plate, due to the very high load transfers which occur.

Christophe's work indicates that current design codes tend to underestimate the load deformation performance of nailed timber connections. As a consequence of this many design aids currently available, tend to lead to unfavourable geometric layouts for fasteners and result in joints which are considerably oversized. The combination of these factors can constrain the designer's ability to find an economic and feasible design solution.

**Code Comparisons:**

A comparison between three different design codes (**fig 2.11**) gives an appreciation of the large differences which exists between the codes of different countries, in this case, between Australia, Germany and Switzerland. Most codes present capacity of a nailed connection in terms of a basic working load which is then modified by several factors which depend upon:

1) Load duration.
2) The number of shear faces
3) Reduced timber dimensions and nail penetration
4) Use of side plates, eg: steel, plywood and fibre boards
5) Number of nails placed in line
6) Pre-drilling of the timber.

| **Design Code:** | AS1720.1- 1988<br>Australia | DIN E1052 - 1986<br>Germany | SIA 164 - 1981<br>Switzerland | |
|---|---|---|---|---|
| between nails - along grain | 20D | 10D/12D* | 10D/12D* | A |
| - across grain | 10D | 5D | 5D | |
| distance from loaded: | | | | |
| - end | 20D | 15D | 15D | |
| - edge | 5D | 7D/10D* | 6D | |
| distance from unloaded: | | | | |
| - end | n/a | 7D/10D* | 7D/9D* | |
| - edge | n/a | 5D | 5D | |
| between nails - along grain | 10D | 5D | 7D | B |
| - across grain | 3D | 5D | 4D | |
| distance from loaded: | | | | |
| - end | 10D | 10D | 10D | |
| - edge | 5D | 5D | 6D | |
| distance from unloaded: | | | | |
| - end | n/a | 5D | 7D | |
| - edge | n/a | 3D | 4D | |

**notes: * - designates nail diameters D>4.0mm**
**"A" - nail holes not predrilled**
**"B" - nail holes predrilled**

**figure 2.11 Code Comparisons for edge and end distance requirements**[64]

European codes usually require the verification of the tension stress in a timber member, considering only the nett cross section. No additional reduction due to notch effect (which is especially important if large diameter nails are used) is normally required.

A comparison of the Australian code with the German and Swiss codes permits the following conclusions:

1) Large differences exist between the different codes

2) A conservative approach to defining the permissible load per nail, tends to downgrade the capacity of the nail as a modern connector.

3) New forms of application and special detail solutions are currently being incorporated in the revised timber codes in Europe (Eurocode 5). For many of these applications, the allowable design loads are likely to be considerably increased, in recognition of recent research in this area.

4) Comparing the load introduced by one nail per unit area between Australia and Germany, the results are as follows:

   i) Australia 1.07 N/mm$^2$ and 1.90 N/mm$^2$
   ii) Germany 1.82 N/mm$^2$ and 2.73 N/mm$^2$
   for softwood and hardwood respectively.

**Differences in Codes:**

These figures highlight differences between the Australian and German codes of 70% for softwood and 44% for hardwoods, when considering the effects of lateral loads (in each case the Australian code is more conservative). Sigrist's work has not only focussed upon nail geometry but also upon material characteristics and behaviour under load. In most design procedures defects are not allowed within the nail group but extensive research has also shown that defects located away from the connection may still drastically influence the load capacity of the joint.

**Research Conclusions:**

Sigrist concludes that classification of timber species into single joint groups without relation to the actual stress grade of the timber (as determined from ingrade testing) is normally a very conservative approach to the design of nailed connections. He concludes that if timber was graded in respect to connections, rather than species groups, far higher permissible values for the use of nailed connections could be derived. Differentiation between the timber grades for the design of both cross-sectional area and the connection (nail types, plates, shear interfaces and geometry), would certainly lead to safer and more appropriate lateral load values for design of nailed connections, including the use of high capacity nailed connections.

## Glued and Epoxy Injected Connections:

In his Gottstein report on "A Study Tour of Timber Engineering in Europe", 1987, Syme[65] highlighted the use of glued bolt connections, which at the time were being currently used in parts of Europe.

Two techniques for this type of connection have been used over the last number of years.

The older technique uses slightly undersized holes, some 1 to 2mm smaller than the thread diameter, which are half filled with glue. The threaded rod is then screwed into the hole leaving only a 10mm clearance at the bottom of the hole. This technique necessitates a groove along the length of the thread to allow the glue to mix around the embedded portion of the dowel, as well as permitting relief of hydrostatic pressure from the epoxy on the timber. The disadvantages of this system are that splitting can easily occur within the timber member and quality control is difficult to monitor, as it is almost impossible to determine whether or not the glue has been evenly distributed across the interface between the dowel thread and the timber.

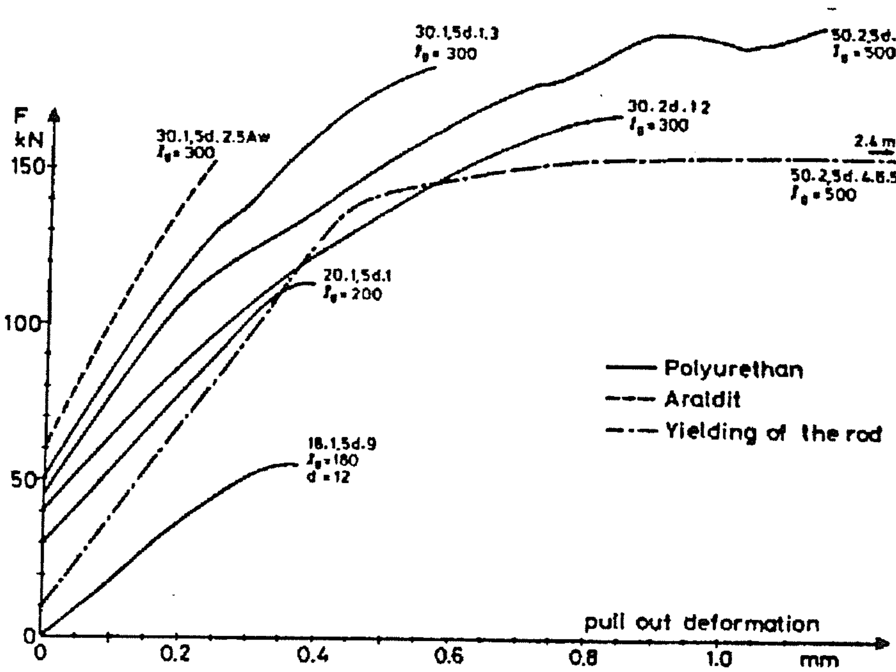



**figure 2.12 - Force / deformation curves for M20 bolts (Riberholt)[66]**

The second and more recent technique highlighted by Syme, is that which has been developed by Riberholt at the Technical University of Denmark. In this technique the rods are placed in oversized holes, about 1mm - 2mm larger than the thread diameter and an epoxy glue is injected through a hole in the timber at the bottom of the embedment hole for the dowel rod. This system permits much greater quality control, as it is possible to determine whether or not an adequate glue bond is likely to have occurred between the dowel and the timber interface, since the glue is extruded at the top of the connection once an adequate quantity of glue has been injected, to ensure continuous bonding between the timber and the dowel. **Figure 2.12** indicates typical force / deformation curves for these types of connection.

Epoxy injected connections have been used successfully in Denmark for over ten years and have resulted in moment resisting connections which have a capacity of approximately 75% of that of a glue laminated beam. General design provisions for these types of connections are being prepared for incorporation into Eurocode 5, where the limit states for glued connections focus on glue line strength, dowel strength, and glueline rigidity as well as quality control to ensure that minimum predictable performance criteria are satisfied for each of the above limit states. Ongoing work is being undertaken by Riberholt,[67] results of which are currently being prepared for incorporation into the CIB design code for timber structures.

The response of the epoxies to load / duration and mechano-sorptive effects is of considerable importance. Whilst ongoing research in this area continues, it should be noted that work to date by Riberholt indicates that ***provided adequate control of moisture content in the timber members which form the connection is maintained***, long term performance of the epoxied connection is satisfactory (refer **fig 2.13**)

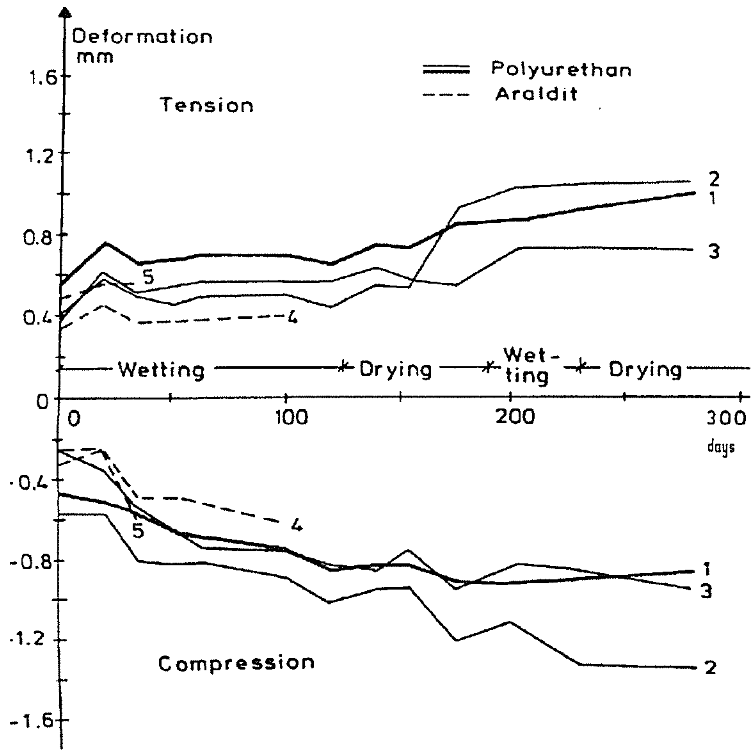



**figure 2.13 - deformation of epoxy dowelled connection with long term load and variations to moisture content**

## Other European Applications of Epoxied Dowel Technology:

In an interview with Professor Borg Madsen at the University of British Columbia in October 1990, Professor Madsen advised the author that during a recent trip to the USSR he had observed fairly widespread use of similar epoxy-dowel connections being used in moment resisting applications. An interesting modification of this technology by the Russians, has been to skew the connections at an angle to the laminates, in order to redistribute stresses through a number of laminates, rather than concentrating stresses on glue lines. This technique appears to have been successful, but Professor Madsen advised that very little published data is presently available.

As mentioned previously in this report, some similar work in use of epoxy injected dowels has been undertaken by Professor Gehri at the ETH Zürich. However, the work that the author was able to assess in regard to epoxy dowel connections tended to focus much more upon the stiffening of beam members and local reinforcement of members under highly concentrated loads, rather than upon the use of this type of technology in connections.

## Australian Applications:

A recent application of reinforced glue laminated timber has been undertaken in Queensland, Australia by Gardner[68] and a patented system developed by Kauri Timbers Pty Ltd.

A significant difference between this Australian work and that undertaken by Professor Gehri, is that Gehri's emphasis is currently upon the use of pre-stressed, post-tensioned rods and cables in PVC ducting inside the laminates, couped with epoxy "stitch rods" to cope with high shear stresses, bearing stresses etc., whereas Gardener's work focuses upon beam elements which have steel reinforcing rods set in epoxy and built into the outer top and bottom laminates of the beam, to form a "composite" beam section (**figure 2.14**).

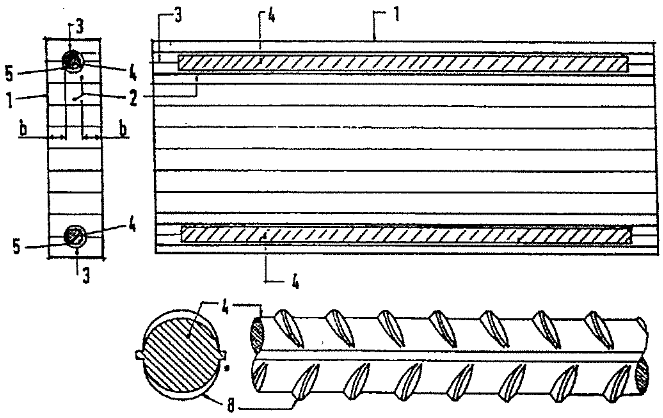


**figure 2.14**

**Details of Kauri timbers reinforcement system**

Basically, this system is constructed by placing large diameter, high strength deformed steel bars in oversize longitudinal grooves within the outer laminations of both the top and bottom of the timber member, before gluing the bars into place using high strength epoxy resins. Not only is this system used for composite beams, but it has also been adapted by Gardner for use in moment resisting connections (see **fig 2.15**)

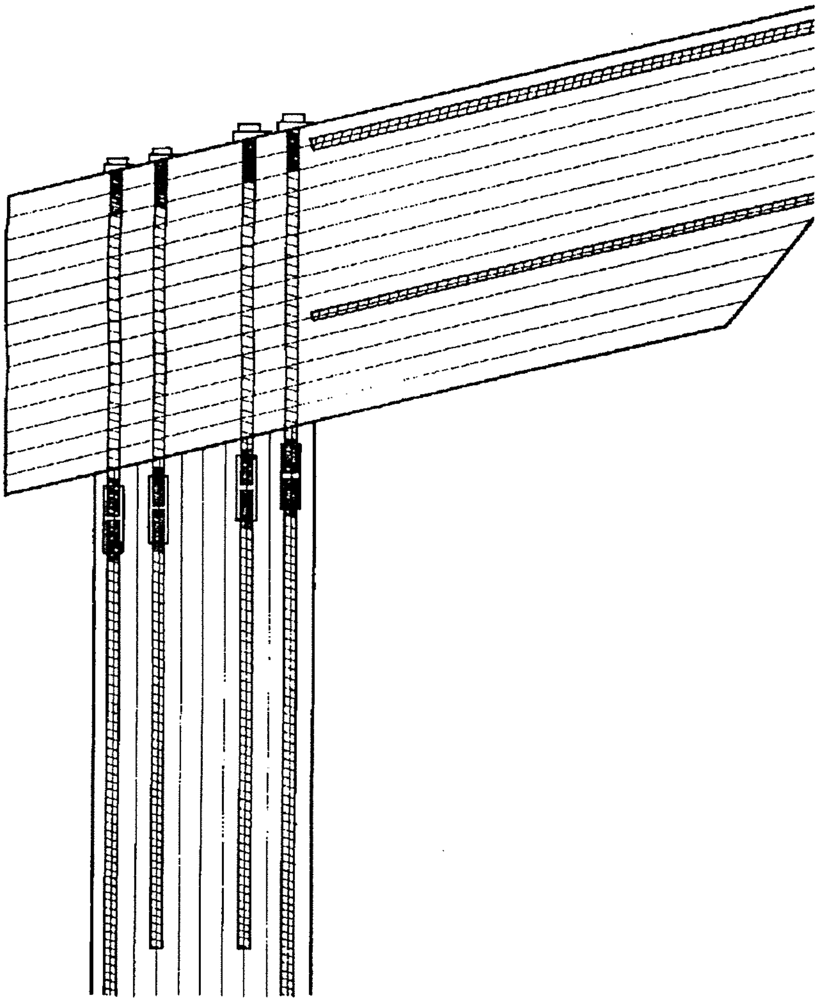


**figure 2.15**

**typical portal knee (haunch) detail as developed by Gardner**

## Research in New Zealand:

Research testing into a similar type of connection has been undertaken at the University of Canterbury, Christchurch, New Zealand by Dr Andy Buchanan (in consultation with Professor Borg Madsen, on sabbatical from UBC). This work indicates that the failure mechanism for this type of connection is a combined tension parallel to grain and tension perpendicular to grain failure, at the end of the tension dowels.[69] This is shown schematically in **figure 2.16.**

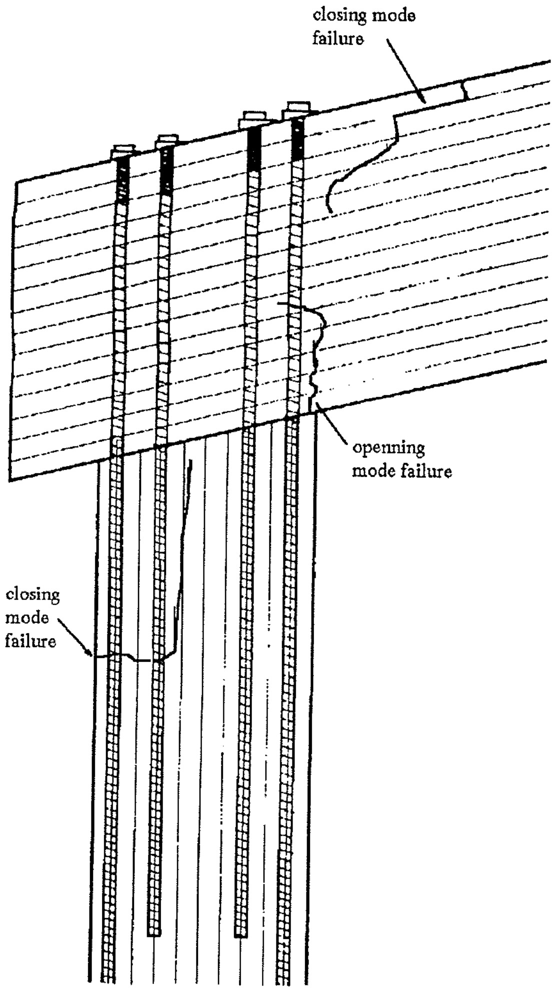
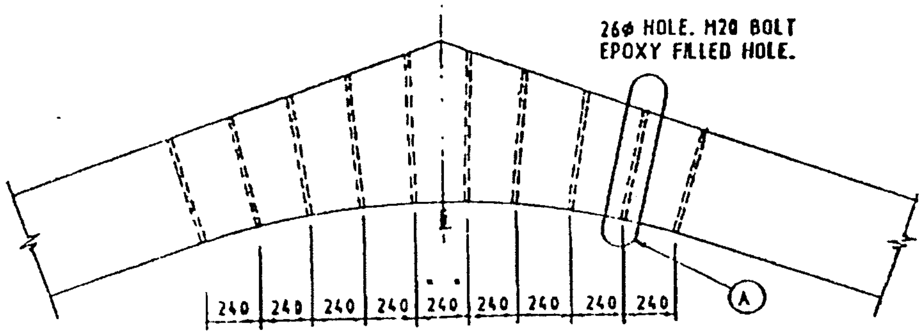



**figure 2.16**

**schematic of failure modes for haunch detail as tested by Buchanan at University of Canterbury, NZ**

A more direct application of Riberholt's technology using epoxy injected dowels has been recently undertaken at the University of Canterbury, Christchurch, New Zealand by Kevin Townsend and Dr Andrew Buchanan. The project sought to consider the use of the Danish technology under New Zealand conditions. Up to the time of this work little use of steel dowels bonded by epoxy in the glulam construction industry had occurred in New Zealand. Recent uses included swimming pool designs, by Buchanan and Fletcher in 1989, (see **photographs 2.9 & 2.10**) and a space frame roof, both of which have been documented by the New Zealand journal, "Timber Construction" in 1989 and 1986 respectively.




ELEVATION AT APEX

At the time that this research was undertaken during 1989 and 1990, there was no proven or significant published research data on the use of steel dowels bonded by epoxy in moment resisting and beam spliced connections in either North America, Australia or and New Zealand, although some examples have been used in the past for repairing glue-line failures (notably by Law and Yttrup,[70] [see **fig 2.17**] and by Gopu[71]).

DETAIL A:

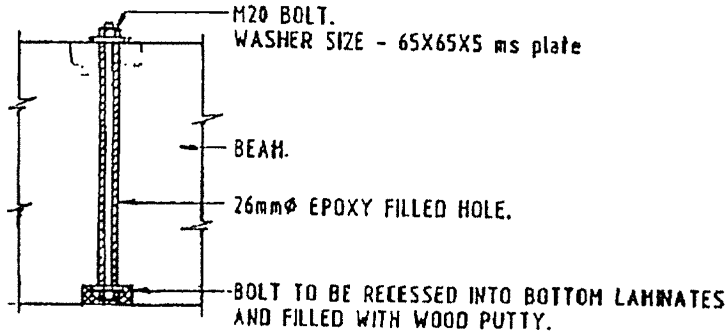



**fig 2.17 repairs to glulam glue line failures**

## Applications of Riberholt's Research in New Zealand :

Steel dowel bonding by epoxy into glue laminated timber has many advantages over presently accepted connection methods. The glulam industry is limited by transportable beam length and the use of steel dowel bonding by epoxy, enables export of larger prefabricated beam sections which can be assembled at the destination site using splice connections (preferably at points of inflection on the timber members).

Currently, most timber portal frame connections utilise bulky plywood and / or metal plates. These connections, whilst structurally satisfactory are unsightly in architecturally designed buildings where a high degree of aesthetic finish is required.


Photograph 2.9 - Concealed Epoxy injected threaded rods in main portal arches for indoor pool enclosure at Jelly Park Pool, Christchurch NZ

Photograph 2.10 - Entry to Jelly Park Pool - moment resisting connections using epoxy injected rods

Discussions with glulam manufacturers and fabricators who erect the nailed gusset plate connections on site, also indicate that installation of the nail fasteners is labour intensive and is perhaps less cost efficient than is widely believed. The following list details the advantages of steel dowel bonding by epoxy:

1) Steel components are protected from corrosion.

2) Hidden fastenings meet aesthetic requirements.

3) No fire protection is required on the joint as the structural connection is protected by the Glulam (though further investigation into fire performance of the epoxy needs to be undertaken)

4) Shorter member lengths are possible in meeting transportation limitations.

5) Two-part epoxy is easily mixed and provides a bond stronger than the natural wood.[72]

Townsend's work sought to reference the theoretical and research basis undertaken by Riberholt in Denmark and apply that to the New Zealand context, in which the main structural timber used is Radiata Pine. A further initiative in this testing program undertaken by Townsend was the use of deformed dowels (deformed reinforcement rods - see **fig 2.18**), whereas previous work by Riberholt had used threaded dowels and that by Rodd (at Brighton Polytechnic), had utilised knurled dowels (in shear rather than tension).

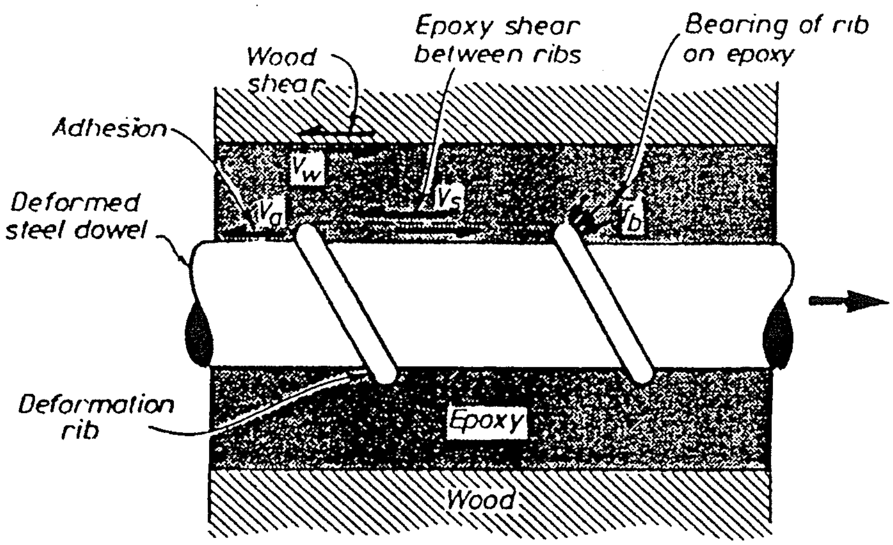



**figure 2.18 - details of the shear transfer mechanism (idealised) in epoxy injected rods (from Townsend)[73]**

**Applicability and Benefits of Epoxied Bolts in Connections:**

The conclusions of Townsend's work indicate that steel dowels epoxy bonded into Radiata Pine glulam timber have excellent potential for making strong and ductile connections. Advantages of the connection system are:

a) high strength
b) aesthetic appearance
c) corrosion resistance
d) fire resistance
e) relative ease of assembly.

For tension parallel to the grain tests, the results conclude that provided adequate embedment of the rods occurs, the joint failure is by steel yield without pullout or brittle failure of the timber. A design formula has been developed for strengths for rods with shorter embedment lengths but it is recommended that for design purposes, this formula should be modified by a suitable factor of safety, to instil confidence that pull-out will not occur. This formula[74] is given below:

$$F = 9.2*d*l_g*(r_d)^2*(r_e)^{0.5}$$ for deformed rod **eqn: 2.1**

or

$$F = 16.5*d*l_g*(r_d)^2*(r_e)^{0.5}$$ for threaded rod **eqn: 2.2**

*where:*

$F$ = pull out force **(N)**
$d$ = bar diameter **(mm)**
$l_g$ = embedment length **(mm)**
$r_d$ = ratio of hole diameter to bar diameter (>1)
$r_e$ = ratio of edge distance (from bar centre-line) to bar diameter (>2.5)

(based on the use of Araldite K-2005 epoxy; for use with K-80 these force values can be increased by 10%)

The design procedure based on **equations 2.1 & 2.2** and developed by the author in consultation with Dr Buchanan incorporates factors of safety for both short term $\mathbf{K_{1(st)}}$ and long term $\mathbf{K_{1(lt)}}$ load durations, such that:

$$\mathbf{F_{(allow)} = F / (K_{1(st)} * K_{1(lt)})}$$ **eqn 2.3**

These factors effectively increase the minimum allowable embedment length to ensure that pull-out failures will not occur. In order to further safeguard against timber or epoxy bond failure and ensure ductile behaviour of the connection, the **$F_{(allow)}$** value calculated in **equation 2.3** is used as the force at which yield would occur in the steel dowels used in the connection. The actual design force in the dowels is limited to the allowable stress for the particular grade of steel and imposed load event (eg: axial or shear).

Despite the conservative nature of this design procedure, these connections are still quite cost effective, as well as being both aesthetically pleasing and structurally efficient. The design procedures will be developed and refined as further research is completed at both the University of Canterbury, Christchurch (by Dr Buchanan) and the University of Technology Sydney (by the author).

When pull-out failures did occur in Townend's research, the bar itself pulled out leaving the epoxy bonded to the wood (refer to **photograph 2.11**). As this occurred the wood and the epoxy split apart, allowing the deformed bar to withdraw. Townsend's work also reports that threaded steel rods appear to have twice the pull-out strength of similar deformed reinforcing bars with the same embedment length. This is a particularly pertinent observation when comparing the work undertaken by Riberholt with that undertaken by Townsend and Buchanan.

The effects of moisture content of the timber were also investigated (in part) but no thorough conclusions have been reached on this to date; initial results indicate strength losses of between 10% and 30% for the epoxies if "wet" wood is used.

For tension perpendicular to grain tests it was concluded that provided that bars are carried through to the far side of the member being connected, epoxy reinforcing bars can develop the yield strength for the same embedment lengths as those parallel to the grain.

For shear tests, when bars epoxied into the end grain are subjected to shear force perpendicular to the grain, predictable ductile behaviour does occur. After testing full-size glulam beams with full spliced connections, Townsend concluded that provided care is taken with the epoxying process, full-strength splices can be made in straight glulam beams using deformed reinforcing bars as dowels epoxied into the end grain.

On the basis of the above mentioned results, Townsend concludes that sufficient research has been carried to allow this technology to be put to immediate use for the design of spliced connections.

However, there are some unanswered questions which should be investigated as a part of future research:

1) The structural performance of the epoxy under wet or slightly moist conditions has not been investigated exhaustively and more work needs to be undertaken in this area

2) All of the tests of this program were of short duration. Some long duration tests should be initiated to answer any concerns about long-term behaviour. Some of these long-term tests should include exterior weather conditions. This is particular relevance in understanding the long term performance of the epoxies under load, about which very little is presently known

3) Other epoxy formulations (other than the Araldite K80 and K2005) should be tested.

4) There are many possible innovative applications for these connections, if it is possible they should be tested under realistic conditions before being used widely. [75]

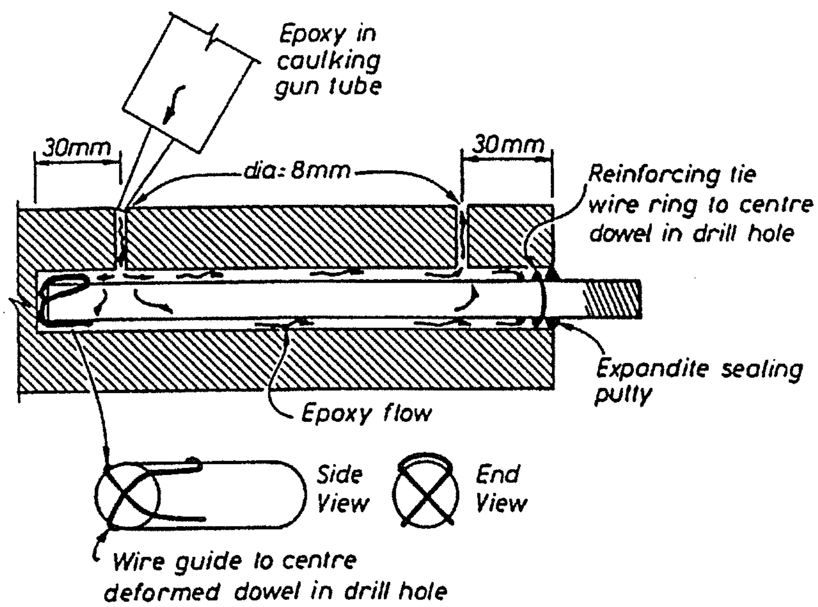
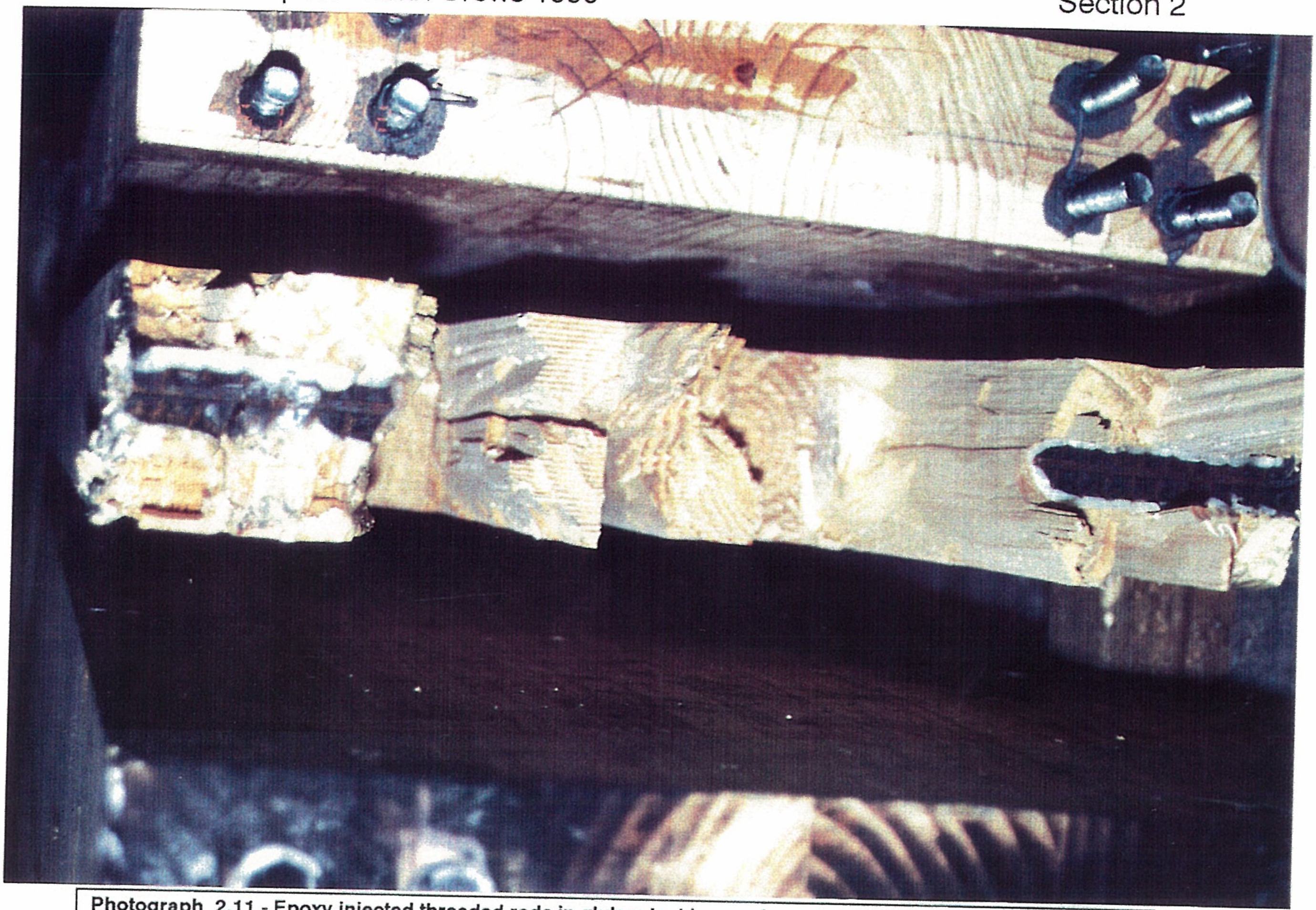



**figure 2.19 - typical details of the holes required for Epoxy Injected rods (from Townsend)[76]**



Photograph 2.11 - Epoxy injected threaded rods in glulam test beams for moment resisting connections

Photograph 2.12 - Steel portal haunch using epoxy injected rods (note: connection is patented by Hunter Laminates) Dr Buchanan shown in photo supervising testing.

**Application to Moment Resisting Portal Connections:**

Subsequent to this research work by Townsend, a further research program has been undertaken by Buchanan and Townsend[77] at the University of Canterbury, Christchurch. This research program was sponsored by Hunter Timbers Ltd of Richmond (Nelson) in New Zealand, to investigate the performance and consideration of alternative connection types for portal frame knee joints with epoxy steel dowels.

The program investigated the performance of three types of portal frame knee joints. These included "square" joints with the rafter running over the top of the column (similar to that promoted by Gardner) and illustrated in **figure 2.15**, "mitred" joints with both the rafter and column meeting at a diagonal steel plate and "steel knee" joints with a pre-fabricated structural steel knee haunch bracket (see **photograph 2.12**) The test results are summarised as follows:

a) All of the joints were loaded to failure. Failure in every case was a wood failure or yielding of the steel dowels or the steel bracket. No pull-out of epoxy bars occurred, (based on embedment lengths derived using **eqn: 2.1**)

b) With the exception of the splitting failures, wood stresses on the nett sections were generally satisfactory for No. 1 framing grade Radiata Pine glue laminated timber.

c) All of the "square" joints suffered wood failures, usually perpendicular to grain failures (see **fig 2.16**). All occurred at a finger joint or at a hole drilled through the rafter which acted as a stress concentrator. Stresses in the gross section of the timber were very low and performance was improved when the gap between the rafter and column was filled with epoxy.

d) The mitred joints were very strong in the closing mode with no wood failures, however in the opening mode, tension perpendicular to grain stresses caused premature splitting failure in two of the three specimens tested.

e) The "steel knee" exhibited excellent behaviour, ductile yielding of the rods and the steel bracket produced excellent hysteresis load cycle loops, with ductile "failure" occuring at stress levels about 2.8 to 3.2 times the allowable design stress for the glulam. It should be noted that this type of connection has been patented for use by Hunter Timbers Ltd in NZ and provides excellent potential for future use in both the industrial/commercial market and with some modifications for use in buildings where a high architectural finish is required.

**Research Conclusions:**

Recommendations of the moment resisting connection program undertaken by Buchanan[78] conclude that:

1) portal frame knee joints can be designed and constructed using details such as those proposed in the test report

2) finger joints should not be permitted in the outside laminations within 1000mm of the knee, unless they have been proof tested

3) steel rods may be either deformed reinforcing bars or threaded steel rods. (noting that the threaded rods have embedment strengths about twice those of the deformed rods). The threads and nuts must be tight fitting and two nuts on each thread are recommended.

4) epoxy gluing of steel rods should be done under factory conditions. Embedment lengths should be similar to those recommended by Townsend,[79] and should incorporate suitable safety factors (as proposed in **eqn 2.3** of this report).

5) design stresses in the glue laminated timber should be calculated on the nett section remaining after holes are drilled. (Design procedures utilising this technology are currently being developed by the author and by Dr Andrew Buchanan).

   This recommendation penalises the "square" joints (promoted by Gardner) where holes are drilled across the grain. It should be noted that the embedded reinforcement parallel to the rafter laminates in Gardner's connection (**fig 2.15**) may provide some shear resistance which would not otherwise occur. However, the results of the University of Canterbury tests still indicate that the square connection is susceptible to perpendicular to grain tension failures and as such, the failure mode observed was always non ductile.

**Future Research:**

It is the author's opinion that the work currently being undertaken by Dr Buchanan shows enormous potential as a construction technique which is cost effective and which can be fairly readily handled by fabricators, both for on-site and factory fabrication of glulam timber elements. It is obvious that further research needs to be undertaken in this area and as a result of this, the author will be commencing a program of load duration testing for these types of connectors and development of the design techniques which will characterise their moment resisting performance.

It is envisaged that this work will be undertaken at the University of Technology, Sydney in conjunction with the University of Canterbury, NZ. This research program commenced during the second half of 1991 as a part of the author's post-graduate research at UTS.

The author is also currently working on designs for two church structures which will be built during early 1992 in Sydney and Taree respectively, utilising the epoxy dowel technology. Monitoring of these design examples and development of appropriate design procedures for epoxy injected rods will be undertaken by the author, in conjunction with Dr Andrew Buchanan at the University of Canterbury.

## CONCLUSIONS:

Understanding of timber connections, their design and behaviour, is arguably one of the most important design issue facing timber Engineers and Architects in Australia (and most parts of the world) today. The research programs overviewed in this report and seen by the author, represent many millions of dollars of timber industry and government funding, which must be seen as an *essential investment* in the future of timber as an engineering construction material.

The fact that the researchers mentioned in this report have been so willing to share their vast experience and professional expertise is a great encouragement to continue this imperative work (both in Australia and overseas) to increase the use of and confidence in, our greatest renewable building resource - timber.

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# SECTION 3:

# NEW BRIDGE APPLICATIONS IN STRUCTURAL TIMBER

*An Introduction to Stressed Deck Technology*

## NEW BRIDGE APPLICATIONS IN STRUCTURAL TIMBER:

### Introduction:

Historically, timber bridges have played a major role in the development of transport routes throughout most countries in the world, not the least of which has occurred in Australia. Traditional construction techniques have used timber piles, topped with headstocks supporting large diameter girders, upon which is placed a deck consisting of transverse planks and sometimes a longitudinal running surface. Typical spans for this type of construction is about 9m (or 30ft) and many such bridges are now in existence, particularly in the Eastern states of Australia, which have a service life in excess of 50 years.

Despite the fact that maintenance on many existing timber bridges has often been at best under funded and at worst, ignored, experience both here and overseas has shown that in the majority of cases, the major cause of maintenance expenditure is degradation of the deck and wearing surface and those structural elements which are unprotected from weathering. Often, the decking material and perhaps the girders will be in need of replacement, whilst the sub-structure (which is usually protected from weathering by the deck) is in relatively good condition. Traditional maintenance of these structures has involved "adhoc" replacement of decking material and tightening of bolts. More recently, some innovative solutions such as "Bridgewood" decking have been used with some success, and RTA experience in NSW has shown that this product is a viable alternative to traditional redecking for bridges on low trafficked, secondary roads.

Maintenance practices over the last 30 years, both in Australia and North America, have usually resulted in the replacement of short span timber bridges by concrete structures, which were intended to be "maintenance free". However, a number of factors have affected this policy and led to a resurgence in timber as a viable structural medium for bridge applications:

- concrete reconstruction has a significant cost
- the maintenance performance of concrete has often been disappointing and is fairly expensive
- funding for maintenance programs has become more stringent and as such, dollars for maintenance have to be "stretched further"
- the fact that often only the decking itself needs to be replaced has, coupled with cost factors, forced Bridge Authorities to look for alternative methods of maintenance construction, which utilised those parts of the bridge structure which had not undergone significant deterioration

It was in response to an economic / maintenance environment shaped by these factors, that the concept of prestressing timber laminates together to form a deck, was created.

## A Brief History of Stress Laminated Timber:

The concept of prestressing (post tensioning) laminated wood bridge decks was conceived in Ontario, Canada in 1976 as a method of upgrading existing deteriorating nail laminated wood decks. The success of this technique as a method of rehabilitation prompted considerable research and development by the Ontario Ministry of Transportation in conjunction with Queens University, aimed at introducing stressed deck technology into new construction for bridges, on both main highways and secondary roads. This initial Canadian work led to the formation of a comprehensive set of design specifications, which have been subsequently adapted into the Ontario Highway Bridge Design Code, being included in the 1983 edition.[1]

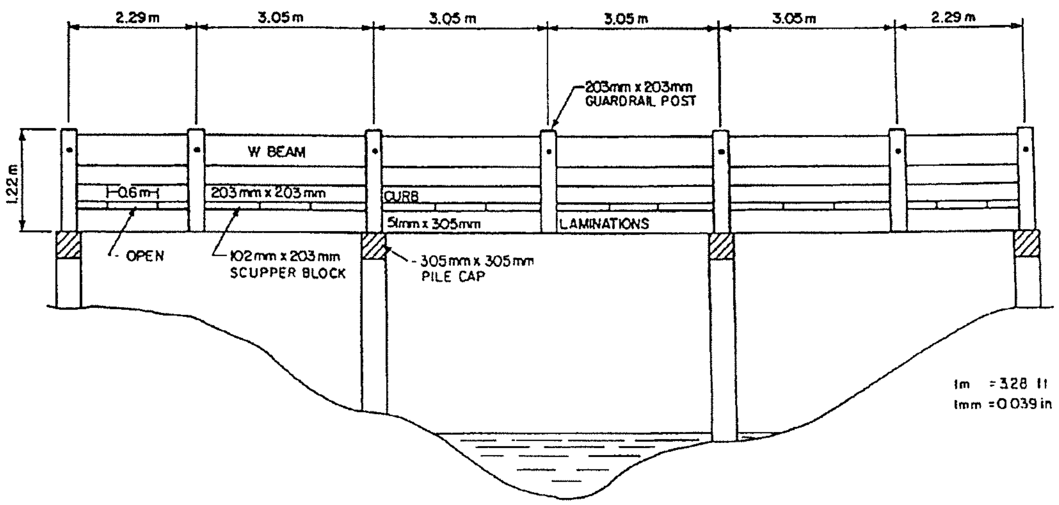
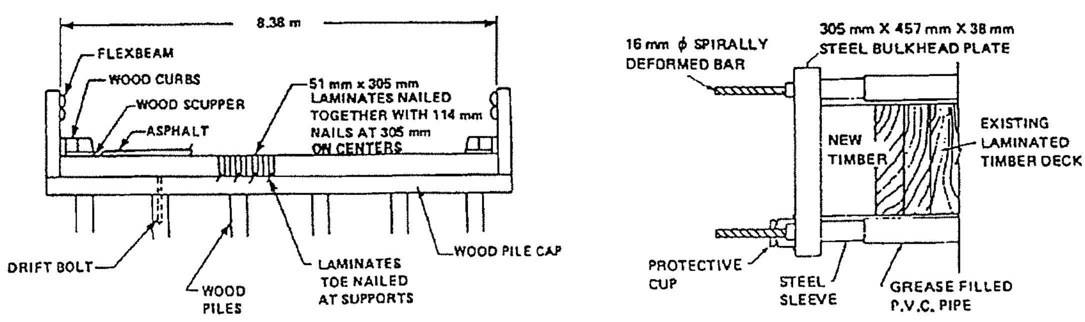



**fig 3.1 - Herbert Creek Bridge - Elevation**




Cross-Section

Prestressing Detail

**fig 3.2 - Herbert Creek Rehabilitation Details**[2]

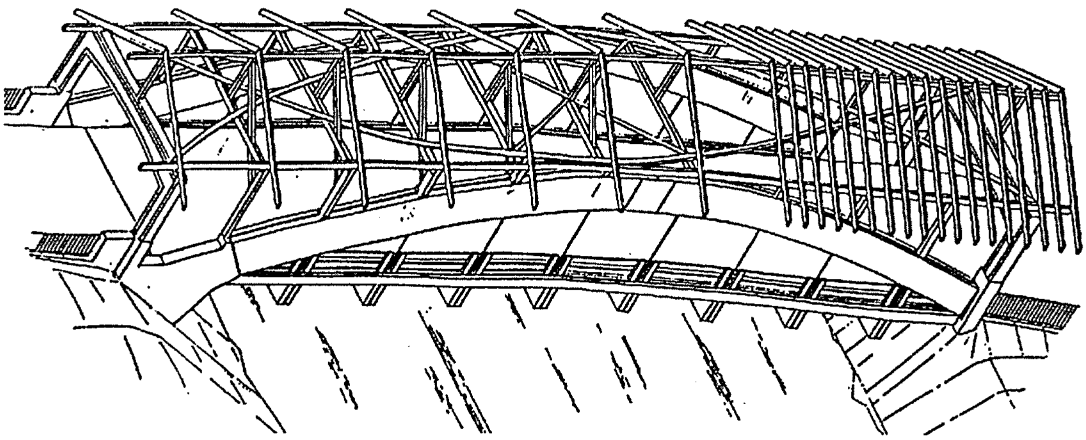


**fig 3.3 - Glulam Arch Bridge with Stress Laminated deck (Switzerland) - 43.4m span**

Shortly after this (during the early 1980's), considerable research into these prestressing techniques was also undertaken in Switzerland, principally by Prof Ernst Gehri, at the ETH Hönggerburg at Zürich. Essentially, this research was done as a follow-on to the initial research done in Canada by Taylor (and others). As well as looking at the mechanical properties of the stressed timber decks, the Swiss research has also had a strong emphasis on design detailing for durability, in order to ensure long-term serviceable performance of the timber components.[3]

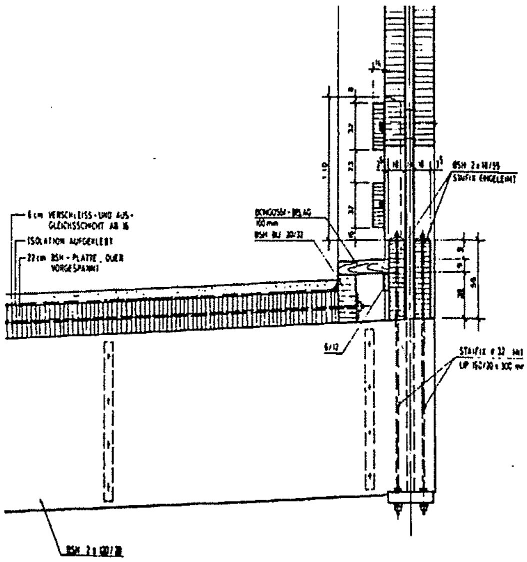


**fig 3.4**
**Typical Cross Section**

The concept of stressed laminated bridges in the United States of America is relatively new and few of these bridges had been built in the U.S. prior to 1988, although (as mentioned above), bridges of this type had successfully been built in Ontario, Canada since 1976. However, since 1988 stress laminating has received a great deal of attention in the United States, and numerous bridges have been built in the ensuing two years.

Investigations into prestressed bridge decks have also been undertaken in New Zealand at the University of Canterbury, Christchurch by Dr John Dean.[4] These investigations were commissioned in 1987 by the structures research sub-committee of the National Roads Board of New Zealand (now called Transit New Zealand), to explore the long term creep of timber bridge decking laminates prestressed together using Radiata Pine.

At the time of writing, numerous research and development programs into stressed deck technology are being undertaken in North America, Switzerland and more recently, Australia. The current focus is on the introduction of treated softwoods and hardwoods into a variety of prestressed systems including: simple plates, continuous span decks, propped cantilever decks, Vierendeel trusses, parallel chord trussed plates and "T" beams and box sections utilising either LVL or Glulam webs.

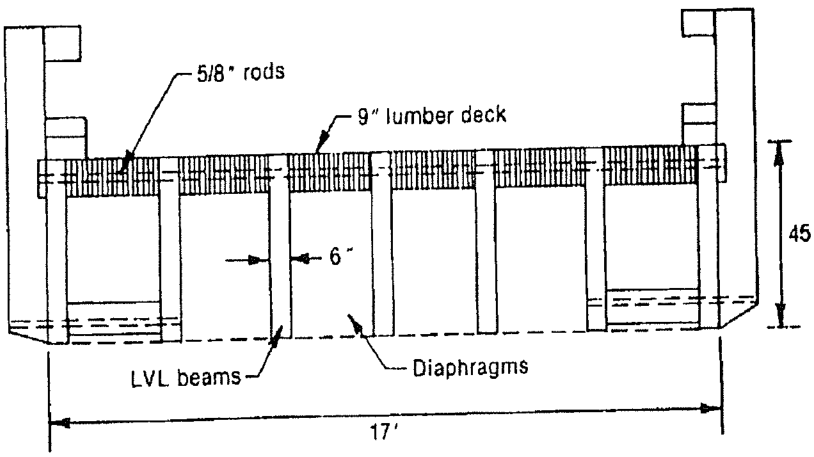



**fig 3.5 - Cross Section through "T" beam Timber Bridge**

## Timber Bridge Initiative:

The development of design procedures and evaluation of performance criteria of stress laminated bridge systems built in the United States is essentially being undertaken by the Forest Products Laboratory in Madison, Wisconsin. The FPL has implemented a nation-wide bridge monitoring program as a part of a government initiative known as the *Timber Bridge Initiative*, which has been sponsored by the United States Department of Agriculture, Forest Service. Funding for the Timber Bridge Initiative was endorsed by the United States Congress and began in 1989.

The initiative has been developed as a response to the problem of maintaining an existing infrastructure of timber bridges. Highway officials at all levels of Government within the United States of America are facing significant problems with regard to the on-going maintenance of bridges throughout the country. Investigative work has indicated that nearly one-half of bridges in the United States of America are considered to be deficient in some way.[5] More than 80% of the nation's bridges are located in rural areas and many rural officials have been confronted by a two-fold problem:

1) large bridge maintenance needs and
2) small budgets to meet those needs.

The USDA - Forest Service has recognised that a significant opportunity exists in the United States of America to improve rural transportation networks and coupled with the desire at Congress level, to revitalise rural economies by utilisation of locally manufactured products, wood for bridge construction became a very desirable option. Many of the some 575000 highway bridges in the United States are on double lane rural roads which are ideally suited for timber construction. Technological advances in the treatment preservation and design of timber products make the timber bridge an economical, safe and attractive alternative to steel and concrete.

From Congress' point of view, the new technologies for constructing timber bridges have presented considerable opportunities to improve worsening rural bridge conditions and to support economic development through increased employment, especially in rural areas. It has also not been overlooked that wood is a renewable natural resource.

The primary aims and directions of the *Timber Bridge Initiative* are:

1) to diversify local economies by improving rural transportation networks
2) expanding the range of markets for wood products; and
3) creating service opportunities for timber construction.

These aims are being achieved through four distinct, yet interrelated goals:[5]

1) Demonstration timber bridges, in order to stimulate awareness of viable efficient alternatives to traditional bridge construction techniques and materials.

2) Research to optimise the balance between existing and developing technology in the use of wood as a construction material.

3) Technology and information management to help transfer appropriate technology on the use of wood in bridge construction through direct technical advice and information distribution.

4) Rural revitalisation to stimulate the growth and development of rural economies through a balanced use of renewable natural resources.

Whilst the size of the bridge problem in absolute terms is much larger in the United States than in Australia, in relative terms the two countries are facing similar problems; initial research commissioned by Western Wood Products Association[6], indicates that about 10000 timber bridges are in service in the east coast states of Australia and at least 50% of these require major remedial work to be undertaken within the next 5 to 10 years. It is the author's belief that the path undertaken by USDA Forest Service and the endorsement and funding of the Timber Bridge Initiative by Congress has important ramifications for the Australian context and establishes principles which are directly applicable to Australia.

**Technology Transfer:**

The first (as far as the author is able to ascertain) reference to prestressed bridge deck technology in an Australian publication, was made by Mr Ron Beckett and Dr John Ivering[7] in July 1986 at a timber design seminar held at the then Institute of Technology, Sydney. Reference to this technology was also made by Mr Peter Juniper, following his return from a Gottstein Study tour of North America in 1986.[8]

However, the initiative for <u>application</u> of this technology within an Australian context did not begin in earnest until 1989. It is the author's opinion that much of the credit for this initiative must to given to Mr John Keith, who at the time was employed as Senior Structural Engineer for the Timber Development Association in NSW, being employed by both the TDA and the NSW Timber Advisory Council (TAC). Acknowledgment must also be made to Messrs Carter and Black, of the Roads and Traffic Authority of NSW (RTA), who had the vision and perception to relate this technology to Bridge Maintenance procedures in NSW and the commitment to implement a major research and development program, in order for the technology to be established with credibility in Australia.

Mr Keith's first contact with this type of technology occurred during his Master of Engineering studies at the Ecôle Polytechnique Federale de Lausanne in Switzerland. It was during his time in Switzerland that Mr Keith became aware of stressed laminated timber deck bridge construction through Professor Richard Gutkowski of Colorado State University who at that time was chairman of the ASCE committee on bridges in the United States of America. Mr Keith immediately perceived that the stressed laminated wood bridge deck construction was very suitable for application to our own timber bridge systems, particularly in the replacement of timber bridge "rattlers", i.e. the traditional spiked plank method of timber bridge deck construction that has been used in thousands of locations throughout the east coast of Australia.

As a result Mr Keith, actively canvassed support from TAC and RTA in NSW for a research program that would see the introduction for this technology into Australia. In 1989 the NSW Timber Advisory Council (TAC) approved an application from Mr Keith for a research program into stress laminated wood bridge deck construction.

In the same period the RTA of NSW, through Messrs Don Carter and Don Black also recognised the potential for their numerous timber bridges in NSW and instigated the approval of funds for a similar research and testing program.

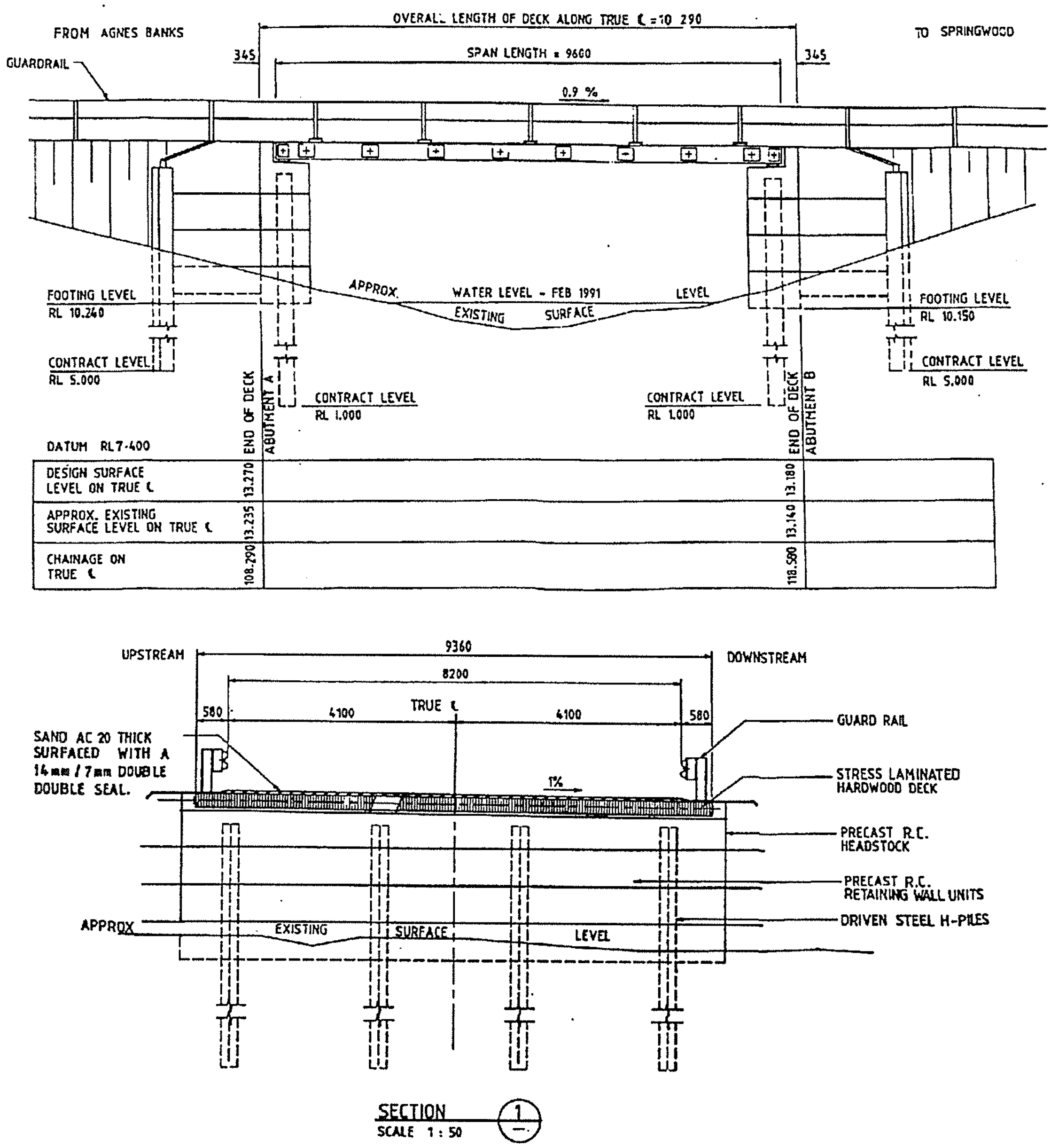



**fig 3.6**

**Yarramundi Bridge (near Richmond NSW)**
**Site of Australia's first Hardwood Stress Laminated Bridge**
**Design by: Roads & Traffic Authority NSW and the author**

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Recognising the benefit for each group's expertise to tackle a common task, the RTA and TAC combined their efforts, funds and resources for a joint venture program which began in 1990. This joint venture appointed the author as the Principal Structural Consultant for the development of the stressed timber bridge program in Australia. The author is currently working jointly with the Joint Venture group, in developing a timber design procedure for stressed timber deck bridges, as well as developing and supervising a research and development program for the testing of timber laminates and the construction of prototype bridges, in conjunction with the University of Technology, Sydney.

The application of stressed deck technology in Australia is drawing extensively on overseas experience (particularly from North America) and much can be learnt not only from research results but also from the infra-structure established to assist in co-ordinating research and facilitate acceptance, growth and appropriate application of the technology.

Photograph 3.1:

The photograph at left illustrates a small stress-laminated timber foot-bridge built as a student project by the University of Wisconsin on the outskirts of Madison, Wisconsin

**Goals of The USDA-FS Timber Bridge Initiative:**

The four goals of the USDA Timber Bridge Initiative can be elaborated as follows:

1) *Demonstration Timber Bridges*. Construction of bridges from locally available timber resources is a key to this goal. Demonstration bridges have proved to be very effective in creating awareness of viable alternatives to concrete and steel construction whilst stimulating local industry, with the timber bridges generally costing about 30% less than concrete and steel.

This is primarily due to:

a) lower materials and construction cost,
b) lower maintenance cycles,
c) lower lifecycle costs.

Accordingly, the use of wood as a construction material makes sound economic sense. From Congress's point of view, this provides an incentive to market previously under utilised wood resources, thereby diversifying local economies. The environmental benefits of this improved use of wood have also not been missed by the United States Congress. In 1989 80 bridges in thirty States were approved for construction, using a combination of Federal and Private funds. As at the 30th April 1990, 18 of these had been built. A further 67 demonstration bridges have been approved in 1990 for future construction. They consist of stressed laminated and glue laminated members, softwood, hardwood and combinations of these.

2) *Research*. This goal is designed to develop an optimal balance between existing structural systems and methods and the development of new technology. Major activities focus on:

a) improving design standards and construction procedures
b) defining strength and properties of selected wood species,
c) grading procedures for hardwood structural timber and
d) performance information and monitoring of timber bridge construction.

The focus of the research effort occurs at the Forest Products Laboratory located in Madison-Wisconsin, (where the author was privileged to visit, during September 1990) which has an annual budget of about $16 million . Much of the work is co-operative in nature, including collaborative activities with Universities, such as the

University of Wisconsin and the University of West Virginia, as well as with industry groups. The technology transfer act of 1986 provides for such co-operation and for the sharing of results. An important component of this research goal will be the on-going monitoring of the performance of selected demonstration bridges on national forest roads. This has already lead to the acceptance of the design principles by AASHTO.

3) *Technology Transfer and Information Management.* A Timber Bridge Information Resource Centre (TBIRC) has been established in Morgantown, West Virginia to help implement the Timber Bridge Initiative and specifically focus on the technology transfer goal. The author was unable to visit this centre due to time constraints. but was assisted with information and a functional overview of TBIRC by staff of the Forest Products Laboratory in Madison. In addition to overall program management and co-ordination, primary activities at the centre include technical assistance, co-ordination of conferences, workshops and seminars, information distribution and co-ordination with field advisers.

4) *Rural Revitalisation.* From a government point of view, this is the cornerstone of the initiative, since it is seen as providing a way towards economic diversity and community stability as a part of the "Rural Development through Forestry Strategy". The timber bridge initiative is seen to provide a tangible, efficient example of how local economies can be expanded and revitalised. Typical activities include: broadening the market base, creation of local jobs and long-term employment prospects and creation of additional service industries.

Funding for the program to date has been $3.3 million per fiscal year for both the 1989-1990 fiscal years. It is envisaged that this will increase to $4.3 million for the 1991 fiscal year, $4.7 million for the 1992 fiscal year, after which funds will gradually be reduced over the ensuing two years to a point where the initiative is self-sustaining.[9]

**Important Principles:**

A significant number of key principles which can be applied to Australia, emerge from the United States experience with the Timber Bridge Initiative.

1) the need for adequate funding to undertake research and development. In order to gain acceptance by national highway and road Authorities such as AASHTO, (or in Australia, the RTA and AUSTROADS), it is necessary to undergird the entire design

procedure for any new material or construction technique with a thorough research base.

2) The timber industry in the United States is similar to Australia, in that it has a history of being fairly disjointed in many of the ways it functions. As such, one of the initial tasks of the National Forest Products Association (NFPA) was to establish a timber bridge task force, in order to co-ordinate national efforts in the timber bridge arena. This task force will in turn help the AASHTO bridge technical committee to write the relevant specifications, so that the results of the research work being undertaken are quickly and efficiently translated into design procedures, which can be readily accepted and widely used by consultants and government instrumentalities.

3) As well as this, the committee has addressed the importance of marketing their product in such a way that it is seen to compete economically, to be an environmentally acceptable solution and to be a strategic initiative in addressing problems of depressed rural economies. The marriage of solid research with marketing and extensive economic considerations, has proven to be very efficient in promoting the concept of stressed timber deck bridges in the United States.

Figure 3.7:

**Typical Research data for plate stiffness tests, comparing theoretical (analytical) results with those measured during testing.**

**These test results highlight the variable relationship between plate stiffness and prestress levels for prestressed bridge decks under static loads. The correlation between theoretical and experimental data has validated the use of Orthotropic plate theory, provided suitable calibration of the analytical model is undertaken.**

**This testing program has been undertaken as a collaborative effort between Forest Products laboratory and the University of Wisconsin at Madison, USA**

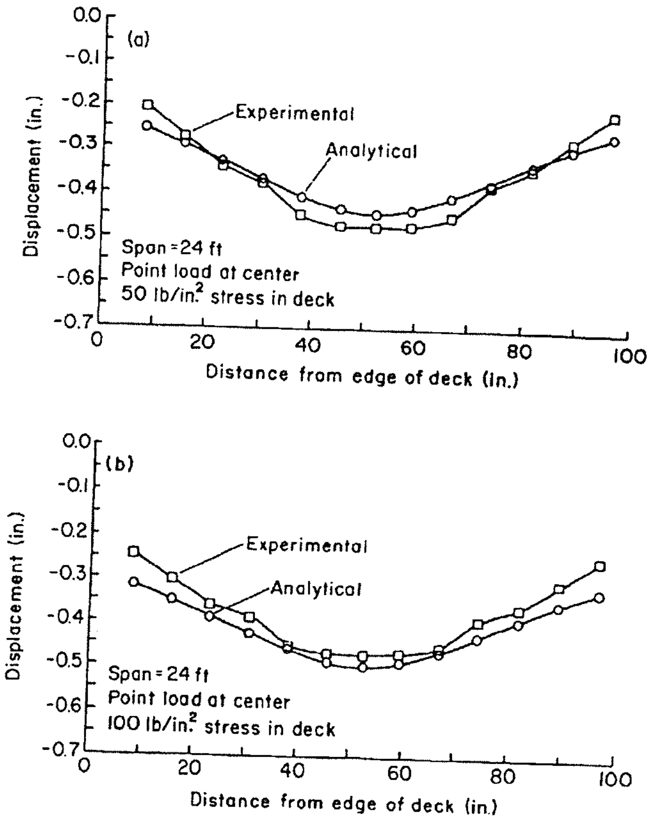



**fig 3.7[10]**

**Advantages of Timber Bridges:**

The advantages of timber bridges are seen to be as follows:

1) *Good performance characteristics.* The performance characteristics of timber, if handled correctly, make it a durable and reliable material to use in bridge construction. This characteristic translates into reduced maintenance costs and prolonged bridge life, provided that the design and construction details are undertaken on a solid research base, with an emphasis on design detailing for durability, as well as serviceability and load considerations.

   Timber has excellent sound and thermal insulation qualities, which serve to dampen traffic noises and reduce the problem of bridge freezing, which is particularly prevalent in North America. Modern timber bridges are also uniquely resilient to impact loads and are surprisingly fire resistant. An especially attractive feature of wood bridges in snow and ice regions, is its high resistance to de-icing chemicals. Such chemicals cause significant and surprisingly rapid deterioration of both steel and concrete bridges and component parts.

   Whilst this problem of corrosion is not nearly as apparent in Australia, the principles are important, since many of our small spanning bridge structures are located in situations where routine maintenance can be a problem. Rapid deterioration, either due to corrosion of steel elements or degradation of concrete surfaces, can result in very significant refurbishment costs with steel and concrete bridges. Recent experience with some coastal bridges in NSW has indicated that degradation of the concrete and subsequent onset of corrosion, may be much more of a problem than is currently recognised.

2) *Ease of Construction.* Bridges constructed of timber are relatively simple to design and standard plans are available in the USA. Prefabricated components are available in "packages" that make timber bridges easy to install. Lifting can be accomplished with smaller equipment (than steel or concrete equivalent structures), since timber bridges are generally lighter in weight than most other bridge materials. Timber's inherently high strength to weight ratio is very significant in this area.

   These construction characteristics make timber bridges easy to handle, modify and upgrade. Local work crews can install bridges using equipment and tools owned by local government organisations, which results in additional savings and labour costs. Erection of timber bridges proceeds very rapidly, particularly when timber substructures are used and the decks are at least partially prefabricated, since forming or curing is not required as in concrete systems. Timber bridge systems can often be erected by relatively unskilled labour, compared to that required for steel or concrete finishing and the educational requirements for training bridge crews in the art of timber construction, can be easily undertaken by engineers or foremen who have been appropriately briefed in the design and detailing issues.

3) *Favourable Economics.* Timber bridges are cost effective and economic. The relative costs for bridge materials vary very widely, and tend to site specific, but construction characteristics of timber coupled with the performance of modern timber bridges, make the modern timber bridge a cost competitive alternative. Relatively little maintenance is needed to ensure a good life expectancy, provided design detailing has been adequate and quality control of material supply has been carefully monitored.

   Maintenance is generally limited to repairing damage, controlling moisture and replacing the wearing surface, noting that in the USA no painting is required. Further savings can realised by re-using existing components such as abutments for beams when making improvements. Since timber is a renewable resource, its harvest, sale, manufacture, treatment and fabrication translate into jobs and money for rural areas. In addition the construction of timber bridges utilises people, equipment and other materials that are often available locally. Training for local personnel is seen to be a part of the total timber bridge initiative, coupled with the training and provision of adequate design aids and details for engineering designers.

4) *Pleasing Aesthetic Qualities.* Modern timber bridges can be quite simple or architecturally complex. The warmth and beauty inherent in timber, provides a very pleasing aesthetic quality. Although timber bridges are especially suited for wooded areas, they are versatile and adaptable to most situations.

5) *Cost Effectiveness of Timber Structures.* A number of studies have been undertaken by government and academic institutions to look at the cost advantages of using timber for bridge construction. Studies undertaken in the State of Wisconsin during the second half of 1986 by the University of Wisconsin, (Madison) indicate that the average bridge cost (for steel and/or concrete structures, including deck and substructure) was US$513.5 per $m^2$ ($47.70 per $ft^2$), whilst the average cost of a timber bridge was US$463 per $m^2$ ($43.00 per $ft^2$). Timber has been found to be even more cost competitive for bridge rehabilitation, since a new deck can often be placed on existing abutments, whilst also providing increased load capacity due to the light weight of the timber deck itself[11]. Similar cost savings (of about 20-25% for new construction and up to 50% for rehabilitation of existing bridges) are expected in Australia.

## Technical Issues:

### Preservation - Design Detailing and Timber Treatment:

Traditionally, timber has been perceived an inferior material for exposed bridges because of fears of deterioration due to decay or insect attack. However, given the traditional neglect of timber structures and the fact that many of them are in excess of 50 years old (as is the case in NSW) it is not surprising that surface deterioration has occurred, often as a result of flood damage, and/or termite attack. Whilst termite attack does not appear to be as significant problem in North America as in Australia, it is nevertheless worthy of note that many of these older timber structures which have little or no maintenance expenditure upon them, have performed far more satisfactorily throughout their design life than comparable concrete or steel structures would have, if the maintenance of these materials had been ignored to the same extent as those for timber.

Pressure treated timber construction has proven longevity, and well built timber bridges have a life span of 50 years and more. European and particularly Swiss experience has shown that timber bridges when adequately detailed and particularly when protected from weather and moisture degradation, can in fact perform satisfactorily for hundreds of years.

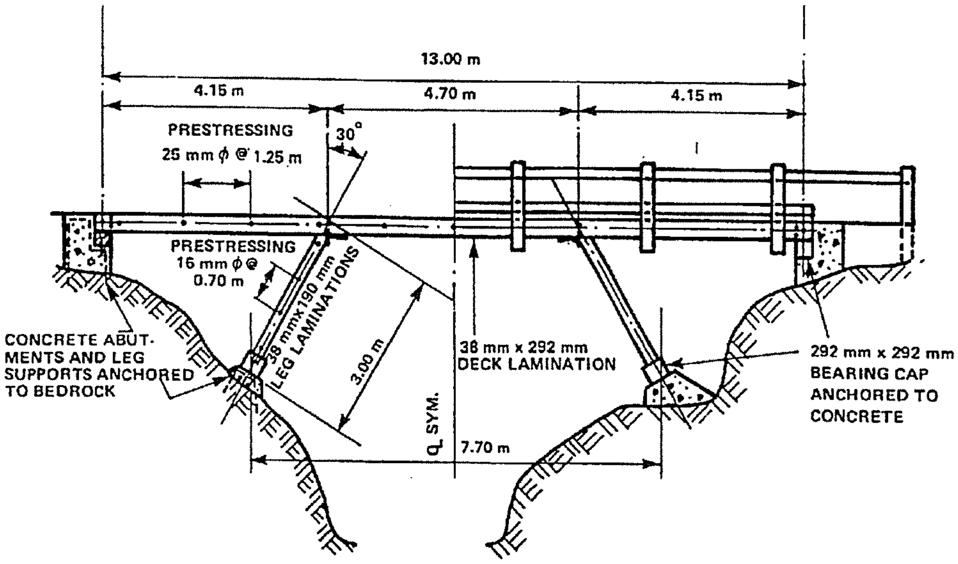



**fig 3.8 Fox Lake Bridge - Canada**

**An example of a propped plate bridge which utilises prestressed timber elements for both the decking and the props to achieve a structurally efficient solution for spans greater than 9-10m. This bridge was proof tested with a 250% overload and still had sufficient stiffness to conform with serviceability deflection limits.[12] This type of construction is ideally suited to plantation softwood species such as Radiata Pine**

**Overcoming Engineering Preconceptions:**

In the USA and in Australia some engineers have been hesitant to use timber in recent years, partly as a result of unsatisfactory experiences with old timber deck bridges, which were connected either by nailed fastenings that loosened with time or with spikes in the running boards or bolts that also loosened with time. The loosened decks often resulted in cracking of overlay surfaces, and repeated surface maintenance, which in turn led to moisture traps occurring between the overlay and the timber material. The inevitable result of this was accelerated deterioration of the decking and at times, of the substructure.

In North America these problems were addressed some years ago with the introduction of transverse decks made of glue laminated wood supported on glue laminated girders. However, the use of a system of separate deck and support girders creates a bridge with a relatively large structural depth. This increased depth may require a raised roadway to maintain sufficient hydraulic clearance above many streams, which would also normally result in increased bridge costs. This type of design has often proven to have cost disadvantages when compared with concrete structures, which could span the same distance with a relatively thin concrete plate or deck.

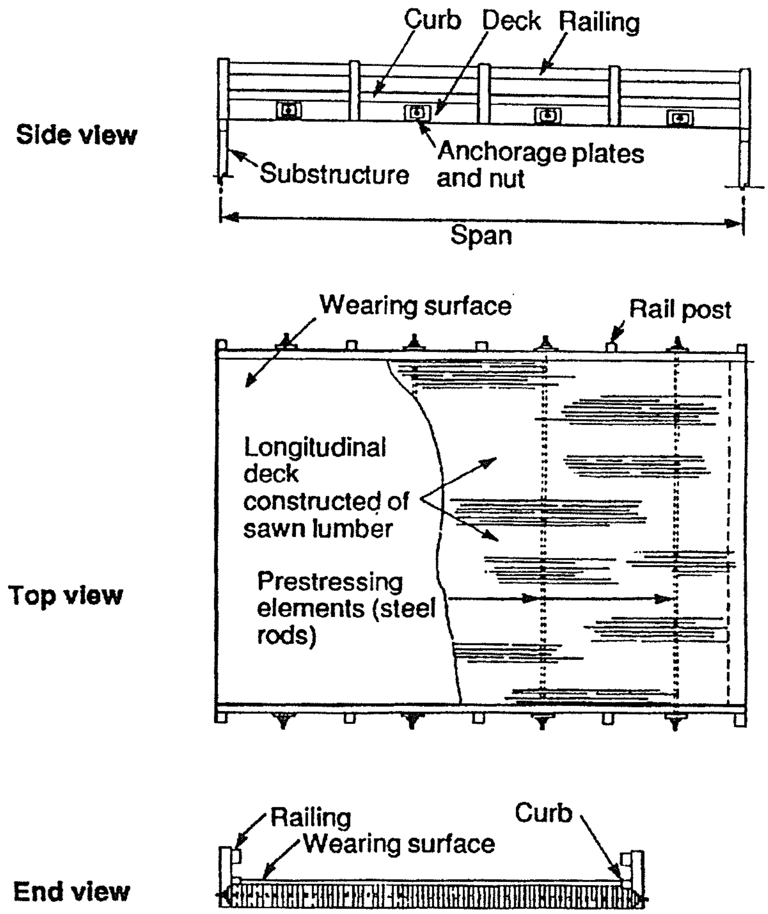



fig 3.9 Schematic Plans of Stress Laminated Timber Decks[9]

**Vertical Laminating:**

A longitudinal deck laminated with heavy drive spikes (as is used often in Australia) often provided a more economical alternative because of its thinner section. However, this type of longitudinal timber deck bridge also faces a limited range of economical efficiency in span, because of the scarcity of useable long, large section, solid sawn timber. In addition to this, maintenance costs can be high if the bridge deck is to properly maintained and recent litigations as a result of accidents "caused" by running boards, has further increased the costs associated with maintaining such bridges.

**Stress Laminating - Application of Prestress Forces:**

Longitudinal timber decks are formed by laminating vertical pieces of timber together until a solid deck of the desired width is achieved. The system serves a dual purpose, in providing a deck which is the both the wearing surface and is also the structural system for transferring the imposed loads and forces to the supports. A *stressed deck* is a special form of longitudinal deck, where laminating is achieved by compressing the individual members together through an applied prestress in the transverse direction (usually post-tensioned). The prestress compression allows lateral transfer of vertical shear between laminae through friction. The deck will also resist transverse bending induced by loads, if the bending tensile stress is less than initial compressive stress. The initial compressive stresses commonly used in practice are of the order of 700 kPa (in the timber).

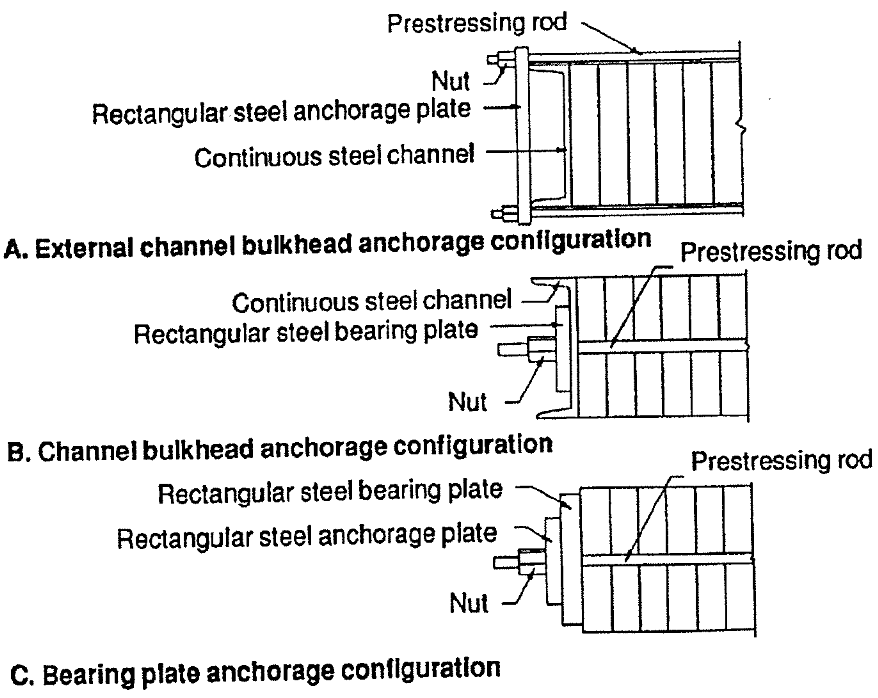
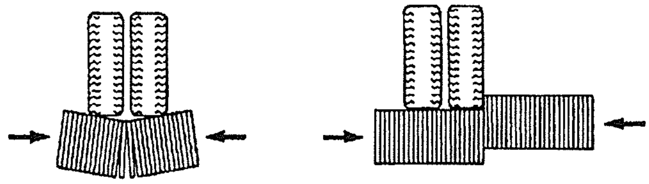



**fig 3.10 - Typical Stress Rod Anchorage Details**

**Plate Action:**

Once this precompression force is satisfactorily applied, the stressed deck will normally then act as an orthotropic plate, efficiently resisting truck-wheel loads, since the load can be distributed laterally from the wheel across some finite width of the deck and then transferred longitudinally to the supports. The critical design factor or element in using stressed timber deck systems, is to achieve and maintain adequate compressive prestress force across the laminates so that "plate action" is maintained.



Transverse bending produces a tendency for opening between the laminations on the deck underside.

Transverse shear produces a tendency for laminations to slip vertically.

**fig 3.11**

**Prestress forces cause the laminates to behave as an "Orthotropic plate", allowing them to then resist both transverse bending and transverse shear**

**Commercial Timber Lengths:**

Stressed timber decks may be built with short laminae that do not reach across the entire span. The forces in a discontinuous lamina at a butt joint are transferred into the transverse adjacent lamina through friction. The adjacent lamina carries the forces past the butt joint and then shifts them back into the second discontinuous lamina. Obviously the spacing of butt and adjacent laminae is critical and must be strictly controlled to achieve this type of splicing effect and to avoid a significant loss of strength and longitudinal stiffness in the deck. It should be noted that stiffness rather than strength is usually the governing factor in design of stressed timber decks.

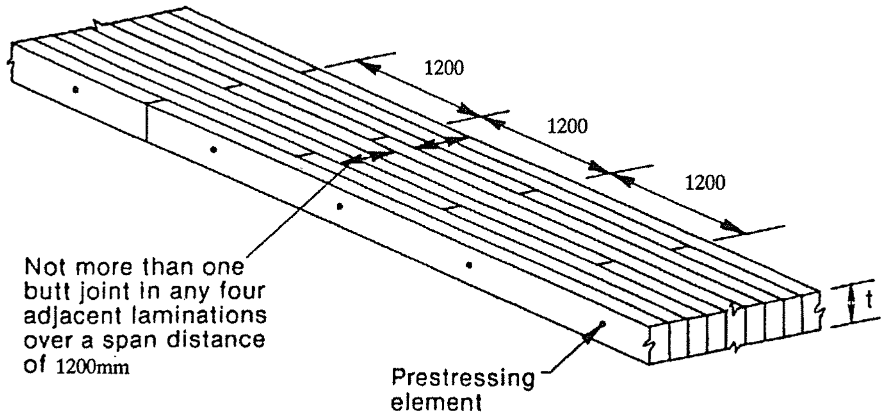



**fig 3.12 Butt Jointing Patterns in North America**

In Switzerland this problem is addressed by finger jointing of the timber laminae in order to form continuous lamina members of the desired length.

These are then pre-cut as in "single elements", prior to drilling of the holes to take the prestressing forces. This technology is quite cost effective in Switzerland where finger jointing plants are readily available, but in North America and Australia, where the cost of finger jointing could be significant, such an approach may not be valid. The effect of the butt joints in a prestressed timber deck appears to be a loss of approximately 15-20% of the longitudinal stiffness of the deck.

The perceived advantages of the prestressed laminated longitudinal deck are that it provides an economical solution to the need for efficient bridges since it uses: relatively thin sections (comparable to the thicknesses required for concrete plates), short individual laminae, has low maintenance needs and is capable of being easily erected by semi-skilled labour.

**Key Factors for Design and Construction:**

The success of this type of bridge depends on:

1) Quality Control of the laminates, particularly with regard to stiffness, strength and moisture content

2) Adequate preparation (preservative treatment) of the laminates (usually using some form of effective envelope treatment), both to address problems of insect attack and moisture degradation.

3) Maintenance of the prestressed compressive force to a level adequate to maintain the plate stiffness required to resist the inservice loads.

4) Ongoing Monitoring of test bridges and inservice bridges, coupled with a strategic maintenance plan to ensure longevity of service life. The following criteria must be closely monitored:

   - moisture content
   - stressing rod force
   - vertical creep
   - deflection (and dynamic) behaviour under service loads
   - degradation and deterioration of timber members

**Bridge Monitoring and Evaluation in the United States:**

In order to evaluate the performance of stress laminated bridge systems built within the USA, the Forest Products Laboratory at Madison implemented a nationwide bridge monitoring program in late 1988. The purposes of the program are to monitor and evaluate bridge performance in order to develop, confirm or improve methods of design, fabrication and construction. This monitoring program allows representative information on the performance of different bridge system and materials in various geographical and environmental conditions to be obtained and collated. Data on existing stress laminated bridges has been collected over a minimum of two years and includes the following:

1) *Moisture content:*

The moisture content is measured at between 4 and 6 locations in the deck using an electrical resistance type moisture meter. The importance of moisture content centres on the fact that it affects not only creep performance of the prestressing but also stiffness if the plate as a whole.

2) *Stressing Rod Force:*

The residual compressive force in each of the stressing rods is monitored, usually using load cells incorporating some form of strain resistance gauge. Loss of prestress influences plate stiffness and if the prestress force falls below a critical minimum (usually 40% of the initial prestress force of 700 kPa) the orthotropic plate behaviour of the deck starts to deteriorate and the structural integrity of the system is jeopardised.

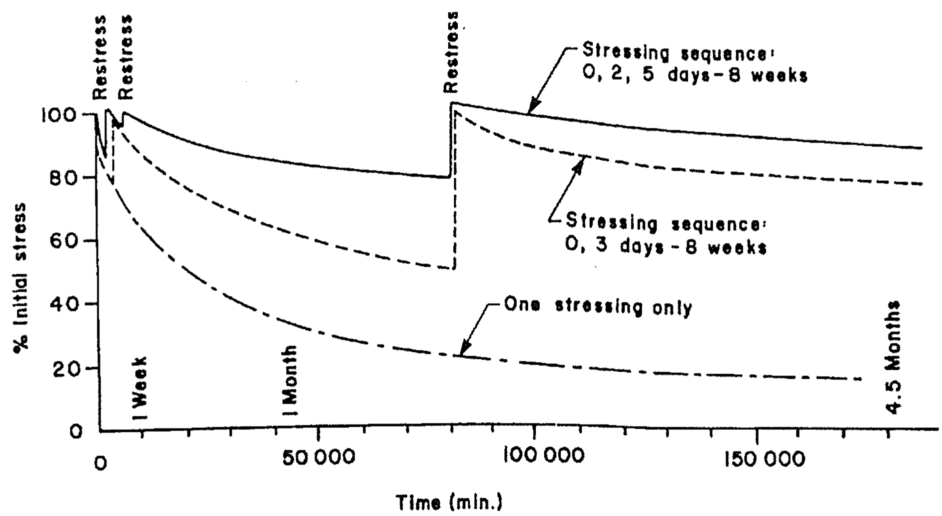



**fig 3.13**
**Effects of creep & restressing cycles for Douglas Fir (Taylor & Csagoly)[13]**

3) *Vertical Creep:*

Readings of the long term vertical creep and deflection are taken at the bridge's centre span and lower edges of the bridge deck . Such vertical displacements are usually monitored to ensure that serviceability limits for the bridge structure, due to self weight and imposed dead load, are not exceeded.

4) *Load Test Behaviour:*

The bridge behaviour under vehicular loadings is determined by measuring the relative displacements of the bridge deck from an unloaded to loaded live load condition. In some situations, the effects of dynamic impact loads need to be assessed, though little work has been done on this to date. This has not been a major problem where deflection limits (for live load) do not exceed span/500. On the basis of initial research by FPL, it would appear that this is mainly due to the fact that the timber bridge plate has significant inherent damping qualities, which allow considerably greater deflections over those which normally be accepted for concrete or steel structures (eg: span/800).

5) *Evaluation of Deterioration and Degradation:*

The overall bridge condition including anchorage systems, wearing surfaces and integrity of the moisture envelope seal, needs to be assessed on a regular basis.

**Discussion on Implications and Effects of Moisture:**

A significant initial observation of the 30 bridges which have been monitored over the last two year period, is the effect of moisture content on bridge performance. The moisture contents of timber at installation and after a period of in-service life, are considerations for the design of all exposed wood structures. Changes in moisture content can affect strength, stiffness and dimensional stability.

Changes in strength and stiffness are recognised in design of timber bridges, as is normal for most timber structures, by applying "wet use" reductions factors, where applicable (eg: for unseasoned timber). The primary concern in stress laminated systems is the dimensional stability of the wood under changes of moisture content. Below the fibre saturation point which is approximately 30% for Douglas Fir, wood swells (expands) as moisture is gained and shrinks (contracts) as moisture is lost. For stress laminated bridges these dimensional changes will directly affect the prestress level as well as the width of the deck (particularly if tangential shrinkage occurs) and if significant, these changes may also result in cracking of the wearing surface and rupture of the membrane.

The level of prestress in the rods is affected by moisture changes, decreasing when moisture is lost and increasing when moisture is gained, mainly due to shrinkage / swell effects. In order to minimise sudden changes in prestress levels, avoid cracking and checking and minimise stiffness losses in the laminates, it is desirable to ensure that any moisture changes within the bridge deck laminates occur gradually over an extended period of time, rather than as short duration fluctuations.

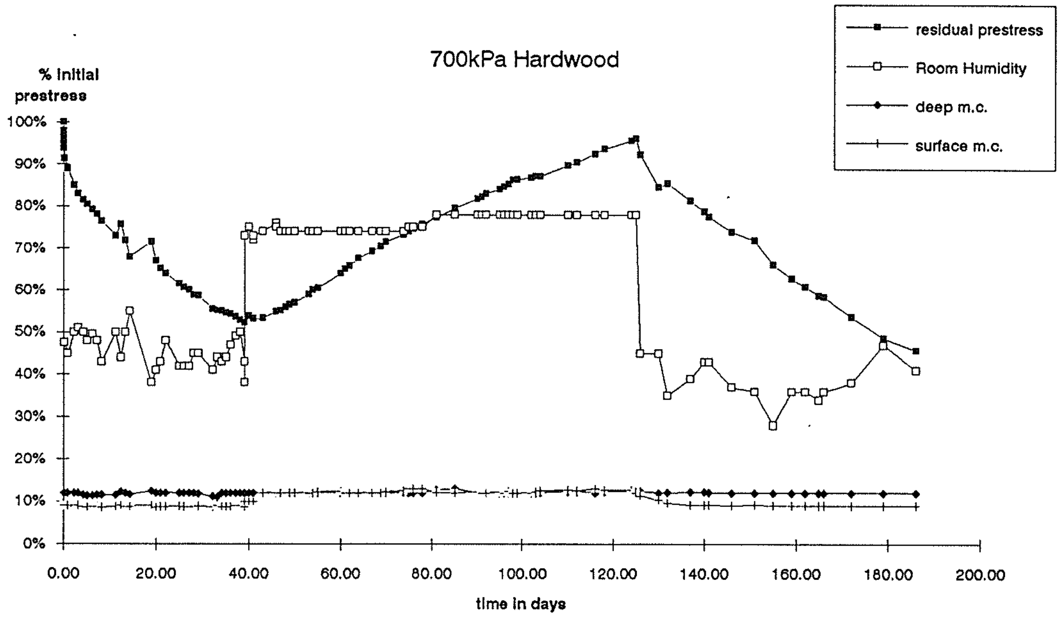



**fig 3.14 - Creep / Moisture change effects in KD F27 Silvertop Stringy-bark[14]**

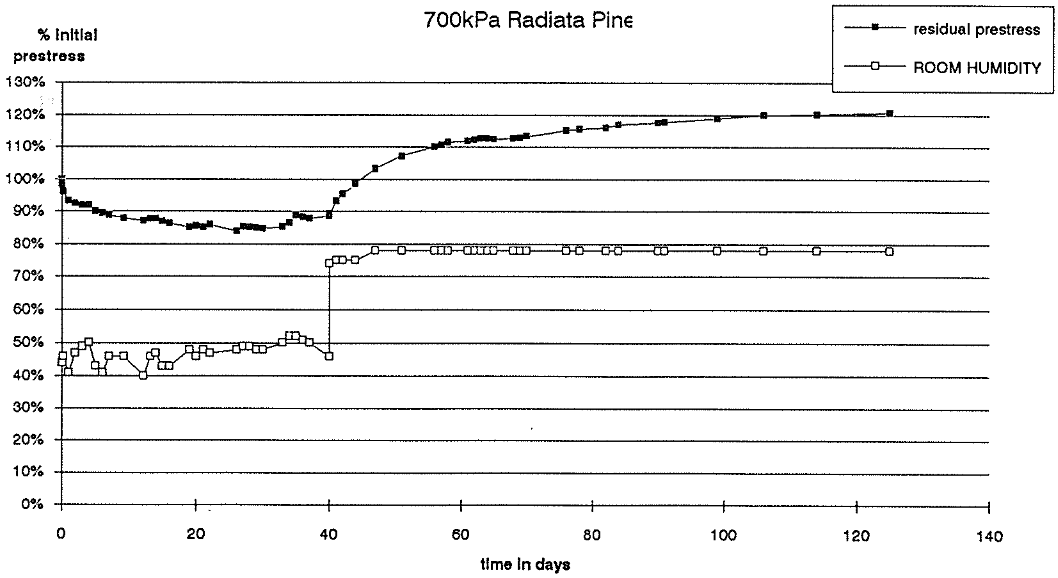



**fig 3.15 - Creep / Moisture change effects in KD F5 Radiata Pine[14]**

## Problems with High Moisture Contents:

Overseas research and field monitoring to date has indicated that of the above "key factors" influencing service life of stressed timber bridges, moisture content and design detailing for durability are the single most important issues.

Generally, US experience has shown that moisture content changes in stress laminated decks can be attributed to:

a) differences in wood moisture content at the time of construction and equilibrium wood moisture content at the site

b) short term changes as a result of superficial surface wetting

c) seasonal changes in equilibrium moisture content [15]

Most bridges constructed in the US to date, have used timber at a relatively high moisture content level with typical values at the time of construction ranging between 24% and 28%. In three examples that the FPL have monitored, the initial moisture contents of the timber from which the bridge deck was built, were in excess of 30%. Field measurements have shown that the global moisture content changes towards an equilibrium level are relatively slow in a stressed laminated deck. Thus the observed effects are minimal during the first several months after bridge construction. However, the effects become more pronounced as the decks eventually lose moisture.

**Loss of Stiffness:**

These moisture losses eventually lead to deck shrinkage, with a substantial loss of prestress, often resulting in a significant decrease in the stiffness of the deck. In one particular situation (in Wisconsin) the loss of stiffness had been such that the deflection under live load exceeded span/150. In this type of high deflection environment, the dynamic effects of loading, particularly from high axle loads, become very significant and bridges in this situation are bordering on becoming unserviceable.

Based on the research to date, FPL currently recommends a maximum a wood moisture content of 19% for stress laminated deck construction (using Douglas Fir), in order to minimise stress loss as the wood reaches equilibrium moisture content (EMC) in its particular environment. In Australia this figure will be 12 to 15% in conformity with local kiln drying practices.*

In addition to this, envelope treatments are used to minimise the moisture changes, usually creosote (in the US) or a similar oil borne / water repellent treatment (eg: PROCCA) and the use of such moisture repellant treatments will be specified in Australian appliactions. Research into the effectiveness of such treatments will be undertaken at UTS during the second half of 1992.

*Research work to date on the Yarramundi Lagoon prototype bridge in NSW has indicated that even if significant expansion occurs (eg: due to emersion by flooding), the net increase in prestress force is minor (less than 15%) and does not result in overstressing or bearing failure at the anchorage plates.

**Preliminary Conclusions:**

Dr Mike Ritter of FPL and Prof Mike Oliver of the University of Wisconsin, report that after two years of monitoring, stress laminated bridges throughout the USA are generally performing very well. However, some serviceability related problems have been experienced, mainly in regard to moisture content. These problems have been most common in the first few bridges built and have been generally caused by high levels of moisture content in the laminates at the time of construction, wood crushing at the stressing rod anchorages and vertical creep. These problems have been satisfactorily addressed in bridges which have been constructed subsequently, and have led to the development of preventative design procedures, which are now in use by Design Engineers and specifiers (eg: AASHTO design manual).[16]

**Design Detailing:**

In Switzerland where detailing timber for moisture protection has evolved over several hundred years, Prof Gehri from ETH in Zürich recommends that as well as using some form of envelope treatment to the laminates, a bituminous membrane should be placed over the entire deck structure. Penetrations through the top layer of the deck, where moisture could be trapped, are to be avoided wherever possible. Research undertaken both in Switzerland and in Brighton Polytechnic in Great Britain, indicates that the loss of stiffness associated with timber laminates is not only a function of the change of moisture content, but also a function of the rate of change of moisture content, and so a membrane which greatly reduces the rate of change of moisture content is considered to be highly advantageous.

Since stiffness is the single most important structural characteristic for prestressed bridge decks, control of moisture content and prevention of moisture ingress are major considerations being addressed by current research (particularly in Australia) in terms of design detailing, material specification and quality control of the timber laminates and deck systems.

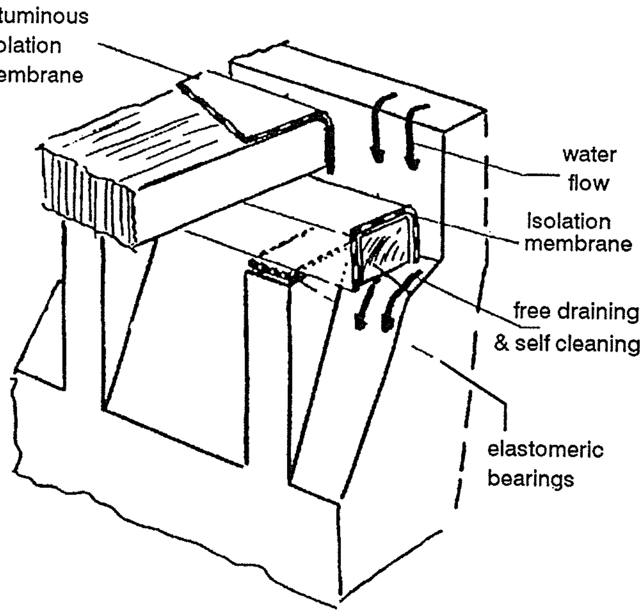



fig 3.16
**Principles for detailing abutments**

## Current Research and Development in Australia:

As was mentioned earlier in this paper, a major research and development program for prestressed timber decks has been initiated at the University of Technology, Sydney. This program focuses on a "technology transfer" of the current "State of the Art" research and design development for prestressed timber bridge decks which is being undertaken overseas, into an Australian context. It should be noted that the Gottstein Fellowship awarded to the author, has enabled this technology transfer to occur, bringing with it significant short and long term benefits to the Australian timber industry, both for maintenance of existing timber bridges and development of new markets for structural timber.

The purpose of this testing program is to develop an "engineered system" for both new and replacement bridge decks to the AUSTROADS design standards, for use by both state and local government authorities. In order for such new technology to gain acceptance by Government instrumentalities and Consulting Engineers, a thorough research and testing program is necessary to characterise the structural performance and undergird the basis of design procedures for prestressed timber deck systems.[17]

The scope (in broad terms) for the Consultant (the author), involves:

1) specification and supervision of the testing procedures This involves determining the specific testing procedures from which the material properties and structural performance are characterised and understood

2) analysis of test results and preparation of analytical predictive models which will be used to formulate rational design procedures

3) preparation of design, construction and maintenance procedural manuals (in conjunction with the RTA and UTS) for use by the relevant roads and highways authorities as well as local government throughout Australia.

## Current Testing Program:

The testing program will include the construction of partial decks spanning up to 7.2m within the laboratory which will be loaded so that deflection patterns and plate behaviour can be carefully monitored and then inputted into a predictive analytical model using orthotropic plate theory.

The current testing program is divided into 6 main stages,[18] the first 5 of which will be undertaken at the UTS Timber Structures Laboratory in Sydney:

*Stage 1 - Physical Properties:*

- Grading; involving visual stress and in some instances machine stress grading
- determination of modulus of elasticity MOE (from 4 point loading, NDT methods and correlations with machine stress grading)

- shear properties (interlamina slip)
- creep & moisture effects (prestress effects and losses)

It should be noted that stiffness rather than strength is usually the limiting factor for design of prestressed timber decks

*Stage 2 - Partial Decks:*

- construction of large (half sized) decks in the laboratory for quantifying performance characteristics and orthotropic plate properties and correlating/testing analytical models. It should be noted that whilst Stages 1 & 3 focus on determination of material properties, Stages 2, 4 & 5 have been designed to quantify structural behaviour of the plate systems

*Stage 3 - Plate Tests:* (refer **photograph 3.2**)

- determination of the various orthotropic plate parameters (transverse stiffness $E_T$ and shear stiffness $G_{LT}$) that are required to design a prestressed, post tensioned laminated bridge deck.
- determination of shear properties of the orthotropic plate

*Stage 4 - Dynamic Responses:*

- dynamic response effects
- natural frequency (vibration) and damping effects
- comparison of plate stiffnesses for both static and dynamic loads

*Stage 5 - Fatigue and Cyclic Load Responses:*

- quantification of load - history for a prestressed timber deck
- effects of accelerated cyclic loading and fatigue on mechanical performance and material properties, under laboratory conditions

*Stage 6 - Field Monitoring of Prototype Bridges:*

- this will initially involve monitoring of 2 prototype bridges, constructed near Richmond and Windsor, about 80km west of Sydney, during November, 1991 and April 1992.

- key monitoring factors include:
    - prestress levels
    - static deflections
    - dynamic deflections
    - dynamic / damping responses
    - monitoring of moisture contents, and relating changes in moisture content to changes in prestress levels and plate / deck stiffness
    - visual inspections to quantify any deterioration or degradation of the laminates, wearing surface, protective coatings / treatments and the water protection membranes

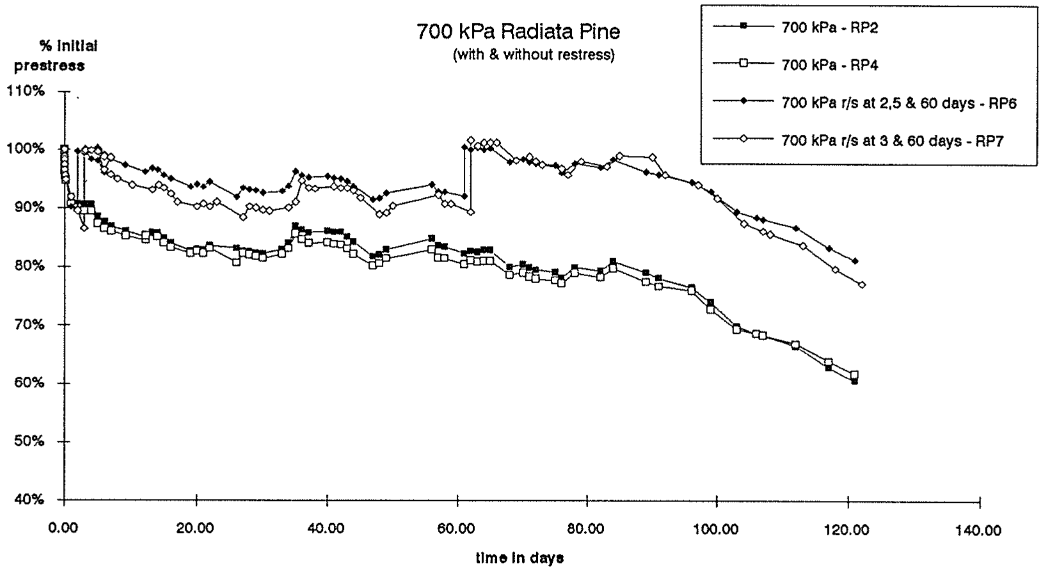



**fig 3.17 Restressing Cycles for F5 Radiata Pine[14]**

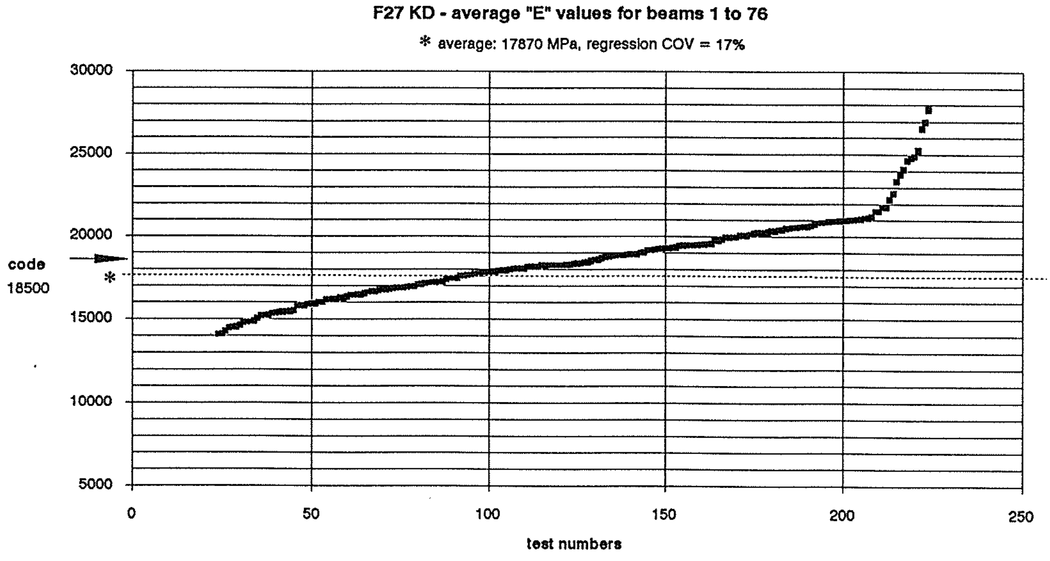



**fig 3.18 - Distribution of MOE values derived from 4 point bending tests for F27 Hardwood[19]**

## Market Potential:

The test program encompasses the use of kiln-dried Australian hardwood, imported Douglas Fir and locally grown Pinus Radiata species. The aim of the research is to present a detailed research base and design procedure for the widespread use of this form of construction and technology, initially for the Roads and Traffic Authority of NSW and subsequently for adoption by AUSTROADS, on a national basis.

Investigations by the author indicate that there are approximately 10000 timber bridges in the eastern states of Australia, about half of which are in NSW. Approximately half of these will require major maintenance or replacement over the next 10 years.

Traditionally, both the RTA and Local Government have adopted a policy of replacement of deteriorating timber bridges by concrete or concrete & steel alternatives. However, due to major cutbacks in funding for such works and increasing pressures on maintenance budgets (as has been the case in North America), this traditional approach is no longer acceptable. Furthermore, in many instances only the deck and perhaps some of the bridge girders are in need of replacement, since the remainder of the sub-structure beneath the timber bridge has been protected from moisture ingress and deterioration by the deck.

The appropriateness of the stressed deck system is that it has enormous potential to stretch maintenance dollars further, either by refurbishment of existing bridges or by replacement with a solution which can be up to 25% more cost effective, both in terms of initial construction and ongoing maintenance costs. It is the belief of the author that development of this Timber Bridge Initiative in Australia has enormous potential for the timber industry, both in economic terms (the NSW market alone is estimated to be worth about $130 million over the next 5-10 years) and in facilitating the development of a new market niche for engineered timber products.

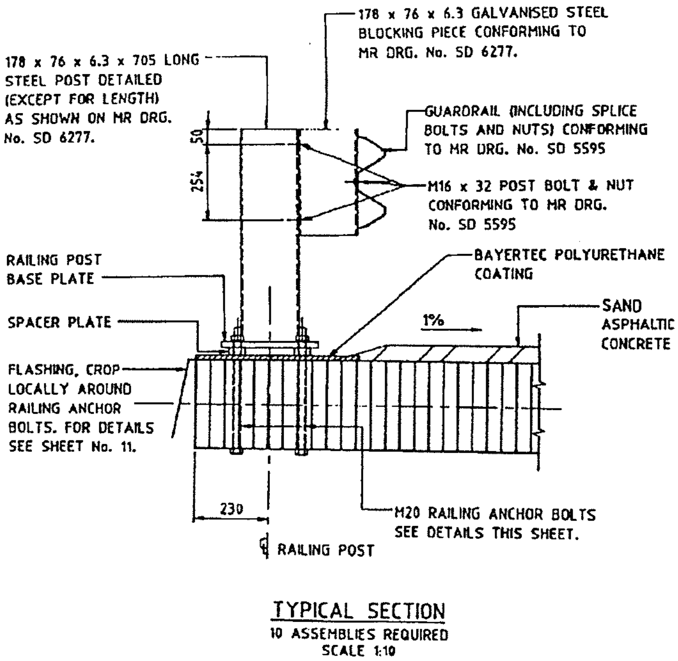
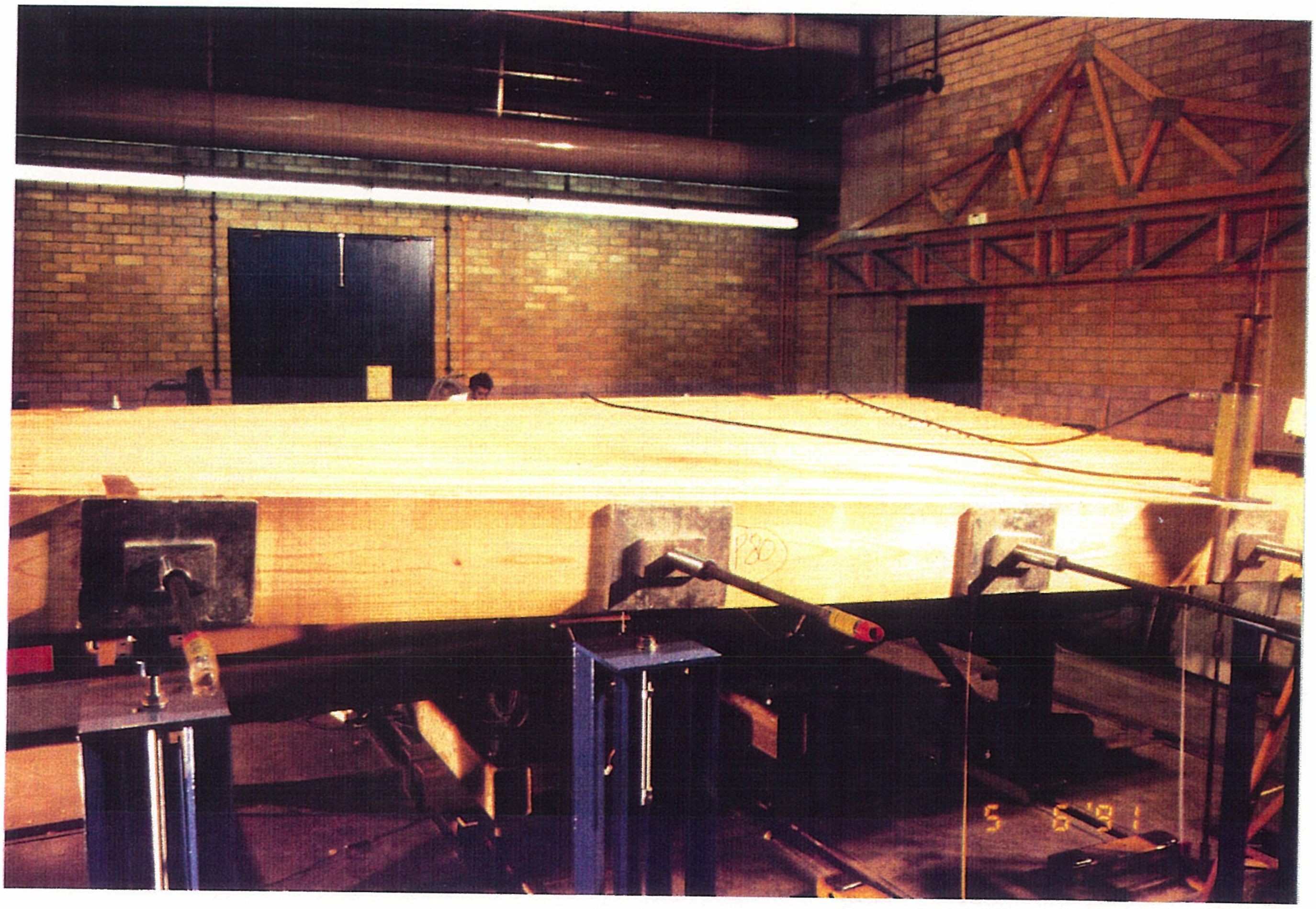



**fig 3.19 - Edge detail: RTA prototype bridges**

## The Role of Radiata Pine:

As mentioned previously, investigations into prestressed bridge decks have also been undertaken in New Zealand at the University of Canterbury, Christchurch by Dr John Dean.[4] To date, this research indicates that the initial elastic loading strains in the Pinus Radiata laminates are significantly higher than those obtained from similar tests on Creosote impregnated Douglas Fir laminates, undertaken in Canada. Creep strains have also been found to be higher than those measured on the Canadian laminates, even when expressed as multiples of the initial elastic strains and stress relaxation losses due to creep are also larger than those observed in the Canadian tests.

It would appear from the results to date, that full exposure to normal climatic and humidity changes caused considerably higher losses in the laminates, than when they were partially protected by weather covers. This is to be expected and as such, issues of surface protection and sealing to minimise changes in moisture content, as well as protection from insect / fungal attack, are being addressed in the UTS tests.



**Photograph 3.2 - Twisting Plate tests on radiata pine at UTS**
**(plate size 3.6m * 3.6m, used to determine shear modulus of orthotropic plates)**

The conclusions of the research undertaken by Dr Dean indicate that the Pinus Radiata laminates may be suitable for prestressed deck bridges provided they are protected against moisture content changes and that there is provision for monitoring and restressing for up to two years after construction and continued routine monitoring and restressing after that period. This conclusion is in accord with North American research, which places a high emphasis on the need for re-stressing at specific times (as a part of ongoing maintenance), after the initial stressing cycles, to ensure that at no time does the residual level of prestress fall below 40% of the value of initial prestress force induced at the time of construction of the deck.

The current Australian testing has focussed on determination of material properties and basic structural characteristics of single span plates. Typically, single span culvert type bridges in the eastern states span about 9 metres (or 30 ft) and it is this criterion that has been focussed upon in the current testing. Unfortunately, this span configuration is disadvantageous to Radiata Pine, in comparison with other species; eg: F27 KD hardwood 290mm deep will span 9 - 9.5m, whereas 290mm deep F5 radiata appears to be limited to about 6-7m maximum.

However, Radiata pine has a number of significant advantages which when coupled with more sophisticated structural solutions (eg: "built-up" sections, to span further) will make it an extremely competitive solution with other timber species, let alone concrete or steel alternatives.

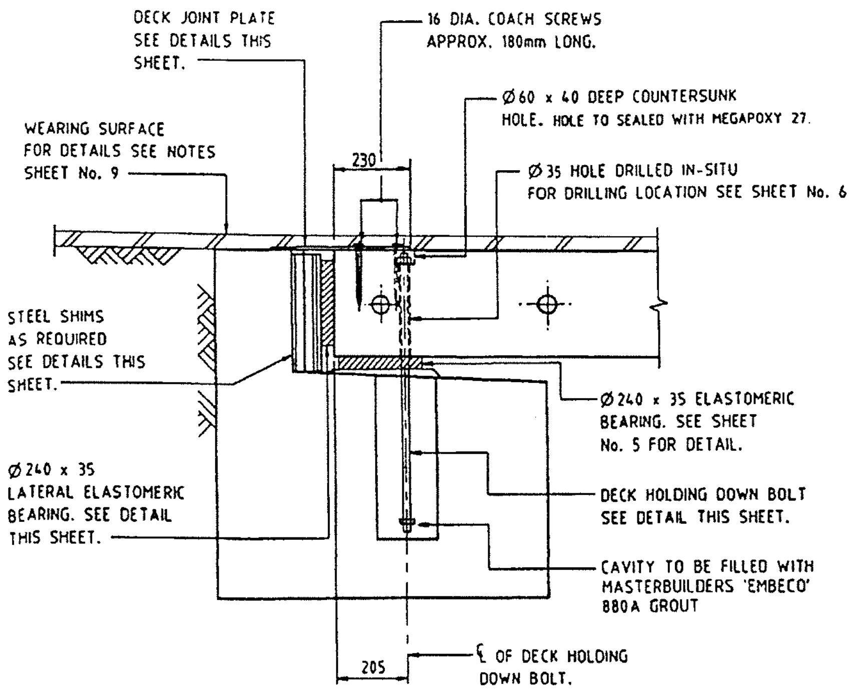



**fig 3.20 - Abutment Detail for Prototype Stressed Deck Bridges**

The apparent advantages of radiata pine for application in the timber bridge market are:

a) (increasing) resource availability of a plantation species
b) quality assurance of mechanical properties, especially stiffness
c) reliable supply, quality control and IGT
d) sapwood readily treated for preservation and durability
e) high strength/weight ratio
f) potential use of "value added" products eg: glulam and LVL
g) cost effectiveness of 150-200mm (nominal) sections in F8 & F11 grades
h) potential of utilising/machining smaller sections in "built up" systems

It should also be noted that some hardwoods possess the qualities mentioned in (b) and (c) and will offer similar benefits to those defined in (g) and (h).

**Future Directions:**

The Forest Products Laboratory in conjunction with University of Wisconsin and the University of West Virginia are currently exploring the use of built up sections using parallel cord trusses, "T" sections and "box girder" sections to increase longitudinal stiffness. As a result of time spent at FPL and the University of Wisconsin during 1990, the author is in close contact with this research and are keen to apply it to Australia, using locally grown timber. A testing program for "built-up" sections utilising Radiata Pine is scheduled to be defined before the end of 1991, with testing to commence at UTS early in 1992 as an extension to the existing testing and research program.

At the University of Wisconsin, Madison, Prof Mike Oliver in co-operation with FPL has been testing many configurations of stressing rods in the use of parallel cord trusses. A demonstration project of a stressed "T" section, using laminated veneer lumber, (called the LVL bridge) was constructed in Idaho as apart of the Timber Bridge Conference in 1989. The bridge has a clear span of 47 feet (15m) with a width of 20 feet (6.1m) and a skew of 41$^o$ and similar prototype work is being undertaken by West Virginia using "T" sections and box sections.

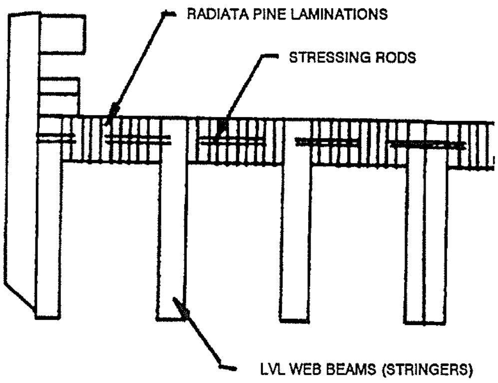



**figure 3.21 Schematic Section of "T" beam Bridge using prestressed Radiata Pine laminates in conjunction with LVL stringers or webs. The expected span of such a bridge is 15-18m for AUSTROADS T44 loads**

**Potential for Radiata Pine:**

The RTA has already indicated an interest in pursuing research and development of "built-up" sections as the next logical extension to the current testing program. The emphasis of this research will focus on the development of more "efficient" structural systems and their behaviour, with an emphasis on "value added" products, such as LVL and Glulam. Potential spans of 15-20m (or greater) are possible utilising these techniques, and the likely use of F8 and F11 timber of smaller (and more readily available) section sizes (eg: nominal 150-200mm) is seen to be advantageous.

As mentioned above, the next logical step (beyond the current research and development program) for the development of timber bridges in Australia is to quantify the performance characteristics of structural systems other than simply supported decks.

The purpose of this is to facilitate the use of both scantling size timber members and processed products such as LVL and Glulam in structural systems, (such as trussed plates, "T" beams, box sections and propped cantilever decks), which tend to utilise the timber more efficiently in terms of both strength/stiffness criteria and cost effectiveness of the timber sections.[20] Potentially, Radiata Pine has more to benefit from this research than any of the other commercial species, for the reasons previously mentioned.

The emphasis of this next testing stage will focus on the structural performance of timber elements, rather than material characteristics, which have essentially been defined in the 1st and 3rd stages of the current R & D program (although some testing of shear and torsional properties may be necessary for LVL). For radiata pine the material properties of the structural systems will be based upon the current work at UTS and the In Grade Testing data produced by CSIRO and NSW Forestry Commission (Wood Technology).

The emphasis of this next testing stage will focus on the structural performance of timber elements, rather than material characteristics, which have essentially been defined in the 1st and 3rd stages of the current R & D program (although some testing of shear and torsional properties may be necessary for LVL). For radiata pine the material properties of the structural systems will be based upon the current work at UTS and the In Grade Testing data produced by CSIRO and NSW Forestry Commission (Wood Technology).

**Methodology:**

There are considerable advantages in continuing and extending this R & D program, mainly because the establishment costs have already been met by the current program and the "built-up" sections research can readily access and extend the work currently being undertaken at UTS. In effect, the timing is right from both a research and marketing perspective, as well as ensuring the most effective use of limited funds, by complementing and extending a research program which is already running successfully.

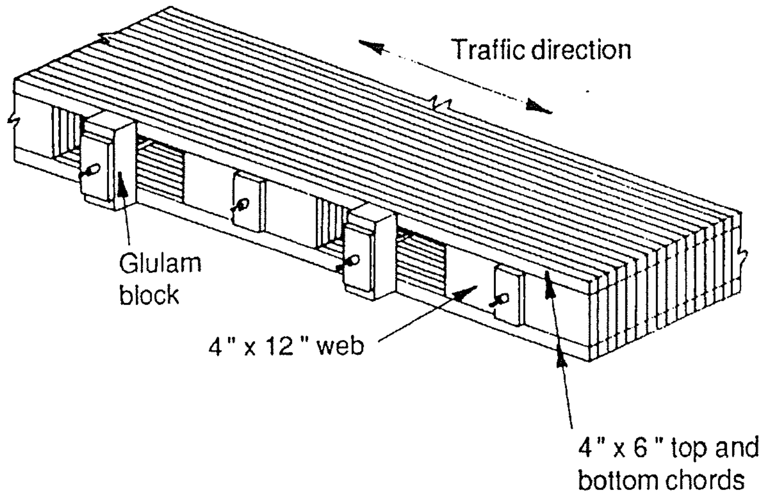



**fig 3.22 - Parallel Plate "Truss" deck, using prestressed timber laminates[9]**

The proposed methodology for "built-up" sections is to:

- develop possible design options eg: "T" beams, box sections parallel chord plates and propped deck systems
- construct and load test these options at UTS
- analyse data and correlate with analytical design models
- present data analysis results
- develop and detail design procedures, including relevant analytical models

**Conclusions:**

It is the opinion of the author that the current research project represents a significant application of the principles derived from the US Timber Bridge Initiative. The program has already demonstrated its ability to:

- embrace a sound research basis which is needs driven, practical and immediate in its application

- pool funding resources from both government and industry for a common project

- recognise market potential and identify the economic advantages of an "engineered" approach to timber bridge maintenance and construction

- operate and be managed by a team of personnel with complementary expertise in the areas of design, management, marketing, materials understanding and timber engineering research

If the Timber industry and Government authorities continue to work within the framework already established, then the success of timber applications in bridge structures is guaranteed. The current testing program at the University of Technology Sydney is an embodiment of these principles, which is already proving itself to be highly successful. Hopefully this will prove to be a "model" for future "needs driven" research projects, where both government and the timber industry can develop new technology and appropriate markets for timber engineered products.

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**ACKNOWLEDGMENTS**:

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- Prof Steve Bakoss, Head - School of Civil Engineering, UTS
- Mr Don Black, Manager - Engineering Services, RTA - NSW
- Mr Stephen Bolden, Development Engineer, Radiata Pine - NAFI
- Mr Don Carter, Design Group Leader - Bridge Branch, RTA - NSW
- Dr John Ivering, Senior Lecturer - School of Civil Eng, UTS
- Mr John Keith, Consulting Engineer, John Keith & Associates
- Mr Milo Taragel, Engineer Manager - Structures Laboratory, UTS
- Mr Terry Tolhurst, Manager, Timber Advisory Council of NSW

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# SECTION 4:

# GLULAM - RESEARCH & DEVELOPMENT

*An Overview of Recent Research Trends*

## GLULAM - RECENT RESEARCH AND DEVELOPMENTS:

### Overview of Research Trends Observed:

During the author's Gottstein study tour during September/October 1990, three significant research initiatives into the performance and understanding of glue laminated timber were observed in both North America and the United Kingdom. These were (in general terms):

1/ Size effects and the development of design stresses for glulam (FPL in Madison, Wisconsin - USA).

2/ Market Trends & Potential - An Evaluation & Assessment (Brighton Polytechnic - UK)

3/ Creep and moisture (mechano-sorptive) effects (Brighton Polytechnic - UK).

The importance of glue laminated timber as a reliable engineering material has been noted as a key motivating factor for the research projects observed by the author throughout North America, Europe and New Zealand. The trends observed have implications for both design and marketing applications of glulam in Australia.

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### Size & Volume Effects - North America:

Considerable research has been undertaken of late into the effects of increasing section size on strength properties for timber members. Much of this research has been undertaken in North America, notably by Professor Borg Madsen and more recently, significant work which has focussed on the design properties of glulam members has also been undertaken by Mr Russ Moody and members of his research staff at the Forest Products Laboratory (FPL) Madison, Wisconsin.

Moody's work has involved a comprehensive analysis of North American glulam beam data, which suggests that bending strength decreases with increasing volume of the beams. This relationship has been known for some time, however the magnitude of these volume effects has been reviewed as a result of Moody's work.[1]

Up to date, glulam beams have been designed (in the United States and elsewhere) using a formula that reduces design stresses for beams greater than 12 inches or 300mm in depth (which is similar to the $\mathbf{k_{11}}$ size factor used in the current (1988) edition of the Australian Timber Structures Code **AS1720.1**)

This work was based on previous work by the FPL, (by Bohannan[2]) that applied Weibull's weakest link theory to tests of clear Douglas Fir specimens. The design formula resulting from this research reduces the design capacity of deep beams with similar loadings and span to depth ratios. This formula is given below:

$$C = (12 / D)^{1/n}$$ **eqn: 4.1**[3]

where:

| | |
|---|---|
| **"C"** | indicates the size factor |
| **"D"** | indicates the beam depth in inches; and |
| **"n"** | indicates an exponent of 9 for present glulam use. (if n =9, then the exponent in **eqn 4.1** equals 0.111) |

Bohannan's work led to the derivation of the adjustment factors for different methods of loading and span to depth ratios. Whist these span to depth ratio adjustments recognise that length affects bending strength, this formula (which has served as a basis for glulam beam design for many years) does not include any adjustment in design stresses for varying beam width.

By way of comparison, AS1720.1 defines the $k_{11}$ size factor as:

$$k_{11} = (300 / d)^{0.167}$$ **eqn: 4.2**[4]

where:

| | |
|---|---|
| **"$\mathbf{k_{11}}$"** | indicates the size factor for flexural & tension |
| **" d"** | indicates the depth in mm |

A comparison of the respective factors for different beam depths is shown in **figure 4.1**.

Research in Canada on both shear strength[5] and tension perpendicular to grain strength[6] suggests that the *volume of material* under stress determines the ultimate strength of the glue laminated material. Additional work (on clear wood samples) recognised the need to include both a length effect[7] and width effects[8] in developing prediction models for strength of large beams.

An overview of this earlier research work was undertaken by Moody[9] at FPL and published in 1988. This overview noted that tests on timber at research establishments throughout the world had observed a decrease in strength with an increase in volume. Much of the data suggested that the width of the structural timber used to manufacture glulam beams directly influences its tensile strength. As well as this, there was substantial evidence indicating that the volume effects observed in clear wood test were also applicable to glulam beams.

| **Depth of Beam:** | | **Code Factors:** | |
|---|---|---|---|
| inches | mm | C | k11 |
| 12 | 305 | 1.000 | 0.997 |
| 14 | 356 | 0.983 | 0.972 |
| 16 | 406 | 0.969 | 0.951 |
| 18 | 457 | 0.956 | 0.932 |
| 24 | 610 | 0.926 | 0.888 |
| 30 | 762 | 0.903 | 0.856 |
| 36 | 914 | 0.885 | 0.830 |
| 40 | 1016 | 0.875 | 0.816 |
| 48 | 1219 | 0.857 | 0.791 |
| 60 | 1524 | 0.836 | 0.762 |

**figure 4.1 - Comparison of ASTM and AS1720.1 size effect factors**

## Developing a Volume Effect Model for Glulam Timber:

The objective of Moody's initial work was to determine if a modified version of the size effect factor is applicable to the bending strength of glulam beam. The approach taken was to apply a form of the size effect model and by statistical analysis, determine the exponents that would best explain variations in the observed data.

Two sets of data were investigated, encompassing 687 visually graded Douglas Fir beams and 188 visually graded Southern Pinewood beams. Statistical analysis of the data was then undertaken, which led to verification of the initial assumption that a size effect equation would indeed be valid. Moody's work at this point in the research program noted that few large beams were present in the data sets and in fact the size of beams most affected by any change in design practices to account for beam volume, <u>were not present</u>. In practice, this meant that the basis for applying a size effect factor to very large glulam beams (with commensurate large spans) was extrapolation of data based on small sized beams (mostly less than 500mm deep and 120mm wide) which were tested on spans generally less than 7.5 metres. Moody therefore concluded[10] that the present design formula (**eqn: 4.1**) did not accurately reflect the size factor that should be used for reducing the allowable design strength of beams subject to bending.

**Prediction Models for Large Size Glulam Beams:**

This observation of data extrapolation was very significant, as it has led to ongoing work which is continuing at present, into the size effects of very large glulam beams. The extensive re-analysis of previous "small sample" clear wood stress data (**figure 4.2**) has led to Moody proposing a size effect factor which is based upon the ratio of volume of a standard beam with the volume for the design beam for a given loading configuration:

$$C = (V_0 / V)^{1/10}$$ **eqn: 4.3**[11]

where:

"**C**" indicates the size factor
"**$V_0$**" indicates the standard beam volume; and
"**V**" indicates the design beam volume

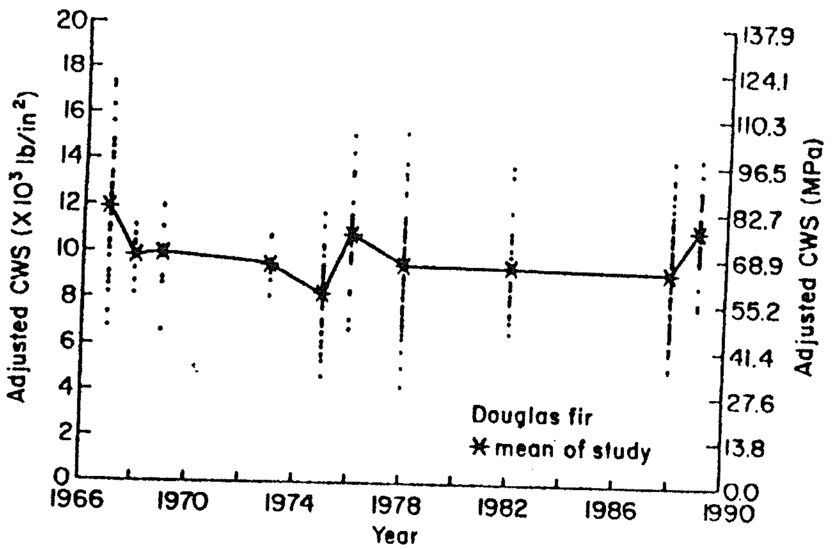


**figure 4.2**[12] **- Re-analysis of Clear Wood Stress Data for visually graded Douglas Fir**

Recent work at FPL[13] has served to relate volume effects to reliability based design procedures, which have been proposed for the USA timber industry. These new design procedures will require information on the statistical distributions of strength and stiffness properties for all materials and components, therefore the development of reliable prediction models for glulam beam performance is essential.

As it is not feasible to develop such data from laboratory testing alone for all grades, sizes and combinations in materials which can be used for glulam timber, it is necessary that reliable simulation models be developed and utilised to predict glulam beam performance. A number of significant models, notably those developed by Foschi and Barrett[14], Bender[15], Ehlbeck[16] and Govindaragoo[17] have been evaluated and developed as part of Moody's current research program.

The basic approach of the modelling is to simulate the material properties of the constituent laminating timber members, taking into account: knots, material properties of the laminates, finger joints, glue lines etc. in order to develop a mathematical representation for the performance of the beam member. The models are used to predict the ultimate moment carrying capacity of a beam; the validity and accuracy of each model depending upon accurate characterisation (by laboratory testing) of the material properties and constituent timber and end or finger joints.

Based on the analysis of research work completed to date, the proposed change to the size effect factor equation (**eqn: 4.3**) compared to the previous equation (**eqn: 4.1**), is shown graphically in **figure 4.3**.

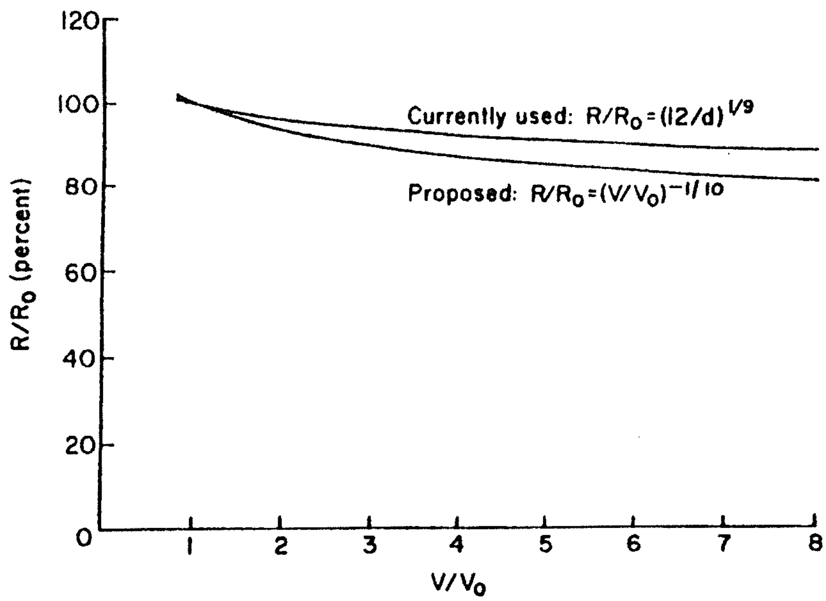


**figure 4.3[18] - A comparison of the currently used size effect and the proposed volume effect for glulam beams in bending assuming:
(1) Constant span depth ratio, (2) constant width for existing size effect,
(3) constant width/depth ratio & (4) similar methods of loading**

## Testing of Large Size Glulam Beams:

The initial research findings published by Moody indicated that at present, none of the models have been completely verified, but ongoing evaluation continues as a part of the process of developing reliability based design procedures for timber structures.

At the time of the author's visit to the FPL in September 1990, significant testing of very large sized glue laminated beams was in the process of being undertaken, including testing to failure of very large sections, which as mentioned above, had not been previously tested. This testing program (sponsored by the American Institute of Timber Construction - AITC), involved testing mean bending strength and 5% tolerance limits for two groups totalling 45 Douglas Fir glulam beams, 15 of which are believed to be the largest glulam beams ever tested, with dimensions of 222mm (wide) 1220mm (deep) and spanning 19.5 metres.

Final details of this research work and the development of analytical models into a system of design prediction, (particularly for a reliability based design procedure) have still to be finalised. However, the initial results indicate that failure to adequately consider volume effects in very large glulam timber members is an unconservative assumption which results in a significant overestimation of the allowable strength capacity of large glulam beams.

The results of this full-size testing have confirmed the premise that for very large glulam beams, significantly greater reductions in strength than were previously thought, are required to take account of volume effects. In providing significant additional data on size effects, these tests have also validated the earlier work undertaken by Moody at FPL, confirming that the volume effect relationship is applicable to large sized glulam beams. A simplified equation (based on a refinement of **eqn: 4.3**, which was presented earlier) has been recommended by Moody for incorporation in ASTM D3737, and is presented below:

$$C = (V_0 / V)^{0.102}$$ **eqn: 4.4**[19]

where:

"**C**" indicates the size factor
"**$V_0$**" indicates the standard beam volume;
with a width "b" = 130 mm
depth "d" = 305 mm
& span "l" = 6400 mm
"**V**" indicates the design beam volume

Moody and Falk also comment that whilst the verification has only been applied to the available Douglas Fir beams which were tested, the equation is geometric and not species dependent. This was confirmed by previous testing of Southern Pine beams, which indicated that variation of the volume effect between softwood species is insignificant. The same effects on hardwoods (with significantly greater densities) has not yet been investigated, but given that most large glulam beams in North America are made from softwoods, the assumption is valid.

Obviously this work has important ramifications for the glulam industry, particularly with the tendency (at least in North America) to use glue laminated timber in very large clear span applications. Such large beams may in fact fail to utilise glue-laminated material in the most efficient way, and a preferred structural solution may involve using smaller sized sections in a different resolution of the structural form, which is constructed from smaller glulam members.

A comparison of the new volume effect factors with the $k_{11}$ factor from AS1720.1, is given below in **figure 4.4**,

| **depth of beam:** | **width:** | **span:** | **volume:** | **Code Factors:** | | |
|---|---|---|---|---|---|---|
| mm | mm | mm | mm^3 | C | C for d/b=2.5 | $k_{11}$ |
| 315 | 130 | 6620 | 2.71E+08 | 0.993 | 0.996 | 0.992 |
| 360 | 130 | 7560 | 3.54E+08 | 0.967 | 0.957 | 0.970 |
| 405 | 130 | 8510 | 4.48E+08 | 0.944 | 0.923 | 0.951 |
| 450 | 150 | 9450 | 6.38E+08 | 0.910 | 0.893 | 0.935 |
| 630 | 150 | 13230 | 1.25E+09 | 0.850 | 0.806 | 0.883 |
| 765 | 190 | 16070 | 2.34E+09 | 0.797 | 0.760 | 0.855 |
| 900 | 190 | 18900 | 3.23E+09 | 0.771 | 0.723 | 0.832 |
| 1035 | 220 | 21740 | 4.95E+09 | 0.739 | 0.692 | 0.813 |
| 1215 | 220 | 25520 | 6.82E+09 | 0.715 | 0.659 | 0.792 |
| 1530 | 220 | 32130 | 1.08E+10 | 0.682 | 0.614 | 0.762 |

**figure 4.4 - Comparison of revised volume effect factors with $k_{11}$ in AS1720.1**
**The depths have been rounded to a nominal laminate width of 45mm**
**Column 5 indicates the volume effect factor for the width given in column 2, whilst the values in column 6 are calculated for a fixed d/b ratio of 2.5**

On the basis of the data presented in **figure 4.4** there may be some case for revising the current expression in AS1720.1 for the derivation of $k_{11}$, although in fairness to present Australian design practices, very large glulam beams of the type most sensitive to the volume effect are not commonly used (to date) in Australia.

It should also be noted that in most cases (particularly in Australia where most applications do not have to consider snow loads) deflection governs the design - not strength, although if precambering is used to counter dead load deflections, strength may become a governing factor. However, the North American experience has highlighted the fact that the designer must still check for the reduced strength capacity of large spanning, deep section glulam beams.

**Glulam Usage in the United Kingdom:**

Over the past five years Professor Barry Hilson at Brighton Polytechnic has undertaken a significant review of the glulam industry in the UK, both within itself and in comparison to the industry in Europe. This study has encompassed an overview of the industry, its problems and its potential, and forms a significant basis for the research that is currently being undertaken at Brighton Polytechnic, as well as presenting a comprehensive set of principles which can be utilised by the glulam industry in other parts of the world.

Throughout Europe and North America today, much of the timber being harvested is coming from second and third generation forests. Due to factors such as the high percentage of juvenile wood, multiple knots and associated cross grain in this type of material, the quality of available timber for use in modern construction is on the decline.

This is one of the most pressing problems facing people involved with timber engineering today and really undergirds the need for a forest products industry which can provide reconstituted solid wood products. Such "manufactured timber products disperse the inherent defects as effectively as possible and result in a product in which the inherent variability is reduced, thus producing a higher characteristic design value. This of course not only relates to glulam but also to a host of other reconstituted products, such as our laminated veneer lumber (LVL), structural wafer board, oriented strand board (OSB) and veneer and particle board timber panels, as well as plywood.

During 1986, an extensive evaluation by Pellicane, Hilson and Smith,[20] focussed on a critical appraisal on the UK glulam industry. In this evaluation the researchers have developed and presented a *philosophical overview* of timber construction, in comparing timber usage in Europe, particularly in the Scandinavian countries, with that in the United Kingdom. This has significance not only in Britain but also is relevant for Australia, in that the attitudes toward timber in the United Kingdom have some parallel (at least philosophically) to the same sorts of architectural attitudes which are prevalent in Australia. By way of example, in Finland where one in four jobs is either directly or indirectly related to forest industries, it is not surprising that timber construction is very popular. However, in the UK and to a certain extent Australia, the opposite view tends to be taken.

Historically, Britain's wealth and growth has been primarily based in iron and coal products. These national resources were instrumental in making it the birthplace of the industrial revolution in the 19th century. Since the production of steel and products made of steel has been of great importance to the UK's economy, it is thus understandable that people would frequently think of steel as the first choice of building material, rather than even thinking to consider timber as an alternative.

A similar trend is also observed in the use of masonry products, such as brickwork and concrete. Masonry products have historically had architectural significance for Great Britain, as masonry buildings were perceived to be "prestigious" and "solid" and gave the appearance of longevity. This attitude towards masonry products and steel has been paralleled in some ways in Australia.

Whilst Australia has a strong tradition of timber usage during its development as a nation (since European settlement), the British influence upon our architectural perceptions has led to a general preference for masonry construction. From the earliest days of European settlement, the Georgian influence created a public perception that masonry buildings were both "prestigious and permanent", whist timber was not durable and used in "temporary" or "cheap" buildings.

Similarly, our development as a nation in the twentieth century based on iron ore and steel production, has been reflected in the post-war domestic building market (which had a strong emphasis on brick-veneer and full brick construction) and the architectural trends (which focussed mainly on concrete and steel) seen through the commercial and industrial sector expansion of the 1970's, 1980's and into the 1990's. In the United Kingdom, as in Australia, it appears that timber is not often thought of in terms of a material to be used as *a rational solution to an design and / or engineering problem.*

**Glulam Usage:**

A comparison of glulam usage in selected countries by volume and population is given in **figure 4.5**. The appraisal undertaken by Pellicane, Hilson and Smith[21] not only identified historical factors which have influenced perceptions regarding timber, but also highlighted economic considerations, quality assurance, material selection and nondestructive material evaluation as major considerations which affect the use of glulam throughout the United Kingdom. Interestingly, it was observed that material and quality control issues are often more significant to specifiers and designers, than purely economic considerations in influencing a decision to use glulam in a structural application.

The study highlighted the fact that there is keen competition amongst European countries in order to meet *market demand* for glulam structures, which of recent times has been particularly high in the Middle East and in parts of Africa. These market forces are also driven and influenced by an overall decline in domestic building in Europe and the UK, which is symptomatic of the economic problems faced by many countries in the European community. In this very competitive situation of trying to meet international markets, UK glulam producers claim to be at a disadvantage for several reasons.

Firstly, the Scandinavian glulam producing facilities are modern, highly automated and capable of high volume production. An economy of scale enables them to produce quality structural members at a lower unit cost than in Britain.

Secondly, British glulam producers feel that the price they pay for the raw material, most of which comes from Scandinavian sources, is greater than the price paid internally in the supply countries, even when transportation costs are deducted. This is believed to be due to the partial government subsidy and support of the forest products industry in Scandinavia.

| Country: | Annual Volume ($m^3$) | Population (millions) | per capita consumption ($m^3 * 10^{-6}$) |
|---|---|---|---|
| Finland | 40200 | 5 | 8040 |
| Denmark | 40000 | 5 | 8000 |
| Norway | 19400 | 4 | 4850 |
| West Germany | 260000 | 55 | 4730 |
| Sweden | 29000 | 8 | 3625 |
| USA | 500000 | 250 | 2000 |
| France | 80000 | 55 | 1450 |
| Holland | 20000 | 14 | 1430 |
| UK | 10000 | 55 | 180 |

**figure 4.5: Glulam Use in selected countries by volume and population (from Pellicane, Hilson & Smith[22])**

Note: The column for per capita consumption is slightly misleading here, as it includes timber products which are exported to other European countries. As such it should probably read "per capita production"

## Manufacturing and Automation:

In the UK, glulam producing facilities are old, labour intensive and lacking in modern quality control procedures. This contrasts markedly with the facilities available in Sweden, Holland, Denmark, West Germany, France and other European countries, where modern computer equipped and highly automated plants produce quality products in a climate controlled environment.

It appears that whilst the glulam producers in the United Kingdom appreciate the value of automated production, modern quality control procedures and rational material grading, the cost of this modernisation is extremely expensive, and the present response of the UK producers is that the current demand levels (domestic) for Glulam do not warrant this magnitude of expenditure.

Interestingly, a similar situation to that currently in the UK existed in France several years ago. At that time, there were approximately 60 producers of glulam, none of the facilities were automated and no company possessed an overwhelmingly large share of the glulam market. However, within the past decade, one particular company made significant capital investments in modern production grading and quality control equipment and at last count that company had captured approximately 20% of the French glulam market. (The company is AMB, Aux Métiers Du Bâtiment which the author visited in a study tour of Europe in September 1988). From the author's personal observations, it appears that the decision to automate and institute effective quality control measures has increased designer and specifier confidence in glulam products, thus increasing the total market share for glulam as an engineering material in large spanning structures.

The problem of quality assurance is linked fairly closely to that of automated production. The quality of glulam manufactured by the European production facilities on the Continent, is of a very high standard and both manufacture and quality control procedures tend to be highly automated. The product produced in Great Britain appears to be of high quality, but is very labour intensive and so the relative cost effectiveness of automated versus manual quality assurance procedures is a major issue. Greater variability and inefficiency which are often (though not always) inherent features of non-automated production are also factors which need to be addressed. (see footnote)

**Computer Aided Production:**

There are many advantages to computer aided production. One particular benefit involves the automatic recording of all data pertinent to every piece of timber laminate which goes into a glulam member. This is useful if a consumer complaint should be filed as a result of an inspection by representatives of quality assurance agencies. Also data generated through both non-destructive and destructive tests of material samples during production (i.e. MSR and proof loading), can be quickly obtained and stored on computer very efficiently throughout extensive production periods. Not only does this record production data, but in the process of doing so, it creates a "data base" of material properties which is invaluable for design and research purposes.

This is very important for timber, because being a non-homogeneous material which has biological origins, it has many variables which effect its quality, and its physical and mechanical properties. Other popular construction materials such as steel and concrete are made in a prescriptive matter to ensure product uniformity.

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Note: It is important to note reasons as to why an automated plant has the potential to produce a better quality product. The author is indebted to Mr Kevin Lyngcoln - Executive Officer of the Plywood Association of Australia for highlighting the fact that automated plants generally require higher management skills and a better understanding of the importance of process quality control. Thus it is not high-tech machines that achieve better quality control, rather it is the attitudes, process systems, technical and management skills of professional personnel which achieve quality control.

Both steel and concrete undergo extensive routine sampling during manufacture and these samples are routinely tested to guarantee satisfactory performance. (This does not mean that steel and concrete are any less variable than timber, but the sampling can provide a means of ensuring that the material properties of the final product fall within acceptable limits of variability). Therefore, structural products made from timber are often at a disadvantage (as far as the engineering specifiers and designers are concerned), as they often have virtually no fabricational or onsite quality control.

This lack of quality assurance in turn leads to greater variability in the quality of the product and coupled with the inherent variability of timber, results in what are unacceptably high (to designers) ranges of the characteristic strength and stiffness values for a structural material. This situation, if it continues, will keep structural timber at a disadvantage relative to other materials.

**Selective Grading of Laminate Material:**

It is a well known fact in the glulam industry, that material placed near the outer edges of beams which are loaded to undergo flexure (or bending), is more critical in determining the beam behaviour, both in terms of stiffness and strength, than the timber placed in the centre of the web. For example, in Holland where there are two grades of timber from which Glulam is made, it is mandatory that at least 30% of the laminations be from the higher grade and not more than 70% from the lower grades. However (ironically), no consideration is made as to where in the member the high quality material is to be located.

When asked why producers both in the UK and Holland did not use some rational procedure to identify the higher quality material for use in the outer laminations of bending members, the results were unanimous. No economic benefit is realised (as far as the fabricator is concerned) by the selected assembly of laminations in glulam. The implies that the additional costs incurred to segregate materials into various classes, (usually by non-destructively evaluated parameters), sorting the material, then assembling the beams so that the stronger timber material is located in the outer laminates, cannot be recovered (at present) by a higher selling price in the market place. Under the existing situation, there in no recognised "added value" for the increased structural performance in beams made from selectively laminated material.

This is a ridiculous situation, since one of the advantages of any composite product including Glulam, is that it can be an "engineered product", with a rational and reduced distribution of variance in material properties. This in turn satisfies the need of structural engineers and designers for a material which has "consistent" (within accepted ranges of variability) characteristic material properties and can also optimise the load carrying capability of the product, leading to a more efficient utilisation of the material.

The research work by Pellicane, Hilson and Smith indicates that any grading system must provide an incentive for the fabricator (usually financial) in order to encourage material efficiency in optimisation of laminate grades. This issue of material grading and selection must be viewed within the broader context of characterisation of properties of engineering materials. All materials (including steel and concrete) exhibit a certain degree of inherent variability in their strength and stiffness properties, usually related to the manufacturing process. As mentioned previously, the variation in timber material properties is further complicated, due to a whole host of both biological and manufacturing / processing factors. It must be noted that even timber materials which have been "selected" using non-destructive material evaluation techniques, exhibit a certain degree of inherent variability in strength and stiffness behaviour.

**Relevance to Australia and New Zealand:**

This issue is of particular relevance for the radiata pine industries in Australia and New Zealand where the issue of segregation of laminates, proof load testing and non destructive stiffness testing is becoming a key issue for fabricators, designers and code writers. This is particularly apparent in New Zealand, where the author had discussions with code writers at both the Universities of Auckland and Christchurch, the Forest Research Institute at Rotorua and with Mr Alan Hunter of Hunter Timbers, Nelson, who is one of the leading Glulam manufacturers in New Zealand and who exports extensively into the Australian market.

Recent research[23] at the University of Auckland by Drs Hunt and Bryant, has indicated that laminate failures are possible at working stress levels in the outer laminates of glulam portal frame (nailed gusset / knee) connections, which have been tested to "normal" in-service load levels under laboratory conditions. As a result of this work, negotiations have commenced and are continuing between industry representatives, designers, code writers and researchers, in order to development an appropriate system for non-destructive testing of both laminates and / or finished glulam beams, as well as some form of "proof testing" of the timber used in the outer laminates, to ensure that minimum strength levels can be guaranteed for laminates in the most highly stressed extreme fibres of glulam beams (particularly when subject to bending and at moment resisting connections).

However, it must be stressed that this is only one of a number of requirements which when included in a "process based" quality control system for the laminated beam industry, will lead to a timber engineering product which has predictable and reliable structural performance.

The concept of a multiple grading system for outer or highly stressed laminates appears to have met less resistance in New Zealand (at least in principle) than in Europe and the UK. This is probably due to the fact that the code writers (in NZ) are encouraging the use of this type of grading system, recognising the superior performance of a selectively laminated beam through code clauses and are seeking to specify how the grading system will work in practice. Obviously for this to be effective, the specification system must be defined in consultation with industry, particularly the glulam manufacturers.

The final result of this process, will be a graded product which has less variability, higher structural strength and an increased market value to the producer. The main problem to be addressed in the short term is the phasing in of the system as far as the producers are concerned, since it necessitates high capital expenditure for the non-destructive testing equipment in a construction market which is presently depressed. However, the work done to date is encouraging and it is the author's belief that once implemented, it will assist the industry to produce a glulam product which has increased reliability, higher characteristic strengths and lower variations in material quality.*

**Non-Destructive Evaluation Techniques:**

Regardless of the process used, quantification of materials for pertinent strength and stiffness properties is essential if one is to operate on a rational structural designing approach. Precise strength properties can only be actually determined by loading a member to *rupture*, or some other mode of catastrophic failure, which renders the specimen unsuitable for use. Thus, techniques which are *non destructive* in nature represent a much more realistic approach to material evaluation, provided that the properties measured by non-destructive means can in fact be correlated with reasonable accuracy to the rupture strengths, required to determine an ultimate limit state for strength.

Currently most stress grading is done by visual means throughout Europe and North America. It has been shown that visual growth assessment can be (and often is) an inaccurate predictor of the actual strength of the given member. However, since design codes in the UK allow no significant economic advantages to be had from machine stress graded material, fabricators cannot justify the purchase of these machines.

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(*) Note:
Once again it must be stressed that product quality is not "tested in, but built in". Without doubt, proof testing of laminates (and in certain instances, beams) is an important aspect of a quality assurance process and along with other non-destructive means of assessing design properties, provides an objective means of assuring the minimum strength (and/or stiffness) of timber laminates. However, a quality control process would also need to address other areas, such as the quality of the glue-line in terms of its strength, stiffness, durability and reliability and this can only be accomplished by tight control at each stage of the manufacturing process. The author is again indebted to Mr Kevin Lyngcoln for his insights into these areas.

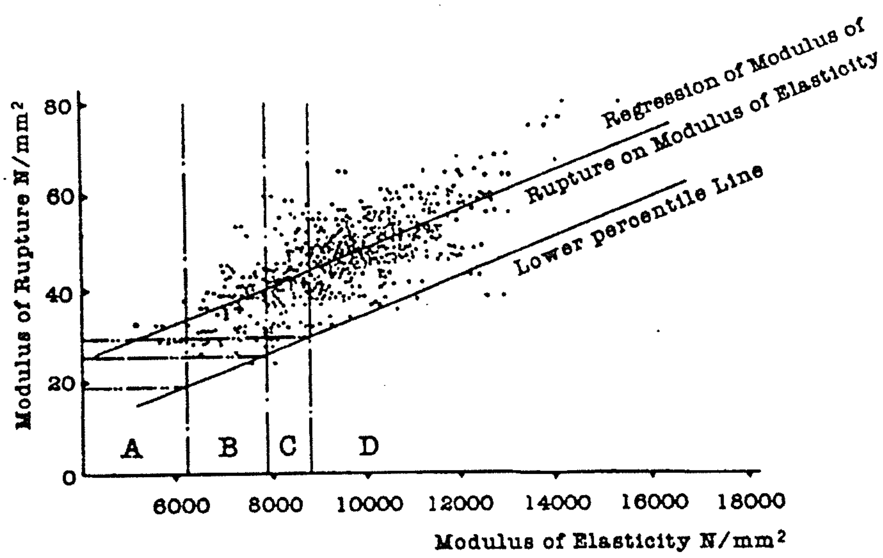


**fig 4.6[24] - Regression between MOR and MOE as the basis of Machine Stress Grading**

This problem has been addressed particularly in Canada and more recently in Australia with in-grade testing programs which are discussed elsewhere in this report. The future of rational material evaluation must move from visual techniques which are subjective in nature, to techniques which allow for material evaluation based on understanding of the relationship between strength and certain non-destructively evaluated parameters, such as stiffness.

An example of this is shown in **figure 4.6**, where a statistical regression of Modulus of Rupture and Modulus of Elasticity is used to segregate material into four strength gradings, with characteristic strength and stiffness values.

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**Recommendations:**

The study highlighted a number of significant areas where efforts could be made to improve longer term prospects for the glulam industry in the UK. These were defined in terms of:

1) *Education:* Making people more aware of the construction material and its mechanical and physical properties. Highlighting the advantages which can be derived from its use in construction.

2) *Promotion:* In relation to timber products, promotion and education are not independent and it can be rationally argued that the lack of demand for timber products must be related to the lack of education about timber. Conversely the low level of demand for timber products has not produced any motivation to educate students and professionals about the material, therefore a vicious circle exists

which must be broken. Education and promotion undoubtedly must be linked together to break this cycle.

3) *Research:* The problem of developing efficient material utilisation with maximised economic return is a continuously changing function of current material resource and technological state of the art. Therefore, several research areas have been identified in which it is felt progress can be made towards more rational and efficient material utilisation. These are:

   (i) Non-destructive material evaluation
   (ii) Adhesion
   (iii) Connections
   (iv) Product development and diversification: (eg: joists, composite and panel elements)

The study concluded the structural glulam members are an underutilised structural resource in the UK compared with other countries. The reasons for this appear to be:

1) related to a basic philosophical preference for other materials, which is based upon historical traditions and perspectives
2) the lack of public and professional awareness of the advantages of timber construction, and
3) a variety of domestic and international economic factors.

During the short term no strategy to increase glulam utilisation appears to be reasonable, however, over the medium and longer term, the study indicated that significant increases in utilisation can be achieved as a result of efforts in several directions.

The education of both practising professionals and students in the areas of civil engineering, architecture and construction as to the advantages and issues concerning timber construction is of prime importance. Promotional efforts to the public and professional people would be invaluable, and ample funds should be allocated to strategic promotional activities, with careful attention paid to targeting the most important audience. Research in many areas of timber engineering has been lacking in the UK, areas in which it is felt that research investment can produce the best financial return to the industry are nondestructive evaluation of material properties, timber connections, adhesion, and product development, as mentioned above.[25]

**Further Research into Education and Promotion:**

Further work by Prof Hilson, in conjunction with a TRADA sponsored study group which undertook a tour of European glulam manufacturing facilities in 1986, has led to a development strategy for promotion and design education within the United Kingdom.[26] This development strategy highlighted the need for a number of key initiatives:

1) *A Manufacturers Association:* It was felt that a UK version of the West German Manufacturers Association would seem to be desirable provided the number of potential members and support is large enough.

2) *Quality Assurance:* With current government emphasis on quality assurance, it was suggested that a suitable scheme should be introduced, offering certification and performance guarantees of glulam products.

3) *Courses in Further and Higher Education:* In view of the very small amount of timber engineering taught in universities and colleges (less than 1% of student contact time), a short term aim should be to increase this amount of time in existing courses. Longer term aims should include the attraction of industrial sponsorship for posts in colleges such as Research Fellowships and Chairs in Timber Engineering.

4) *Promotional Initiatives:* Possibilities for these could be channelled through avenues such as the Glulam Manufacturers Association and TRADA. On the manufacturers side, high quality brochures illustrating exciting applications of glulam need to be strongly pursued.

A number of promotional strands could be included:

- editorial support
- advertising material
- national competitions for architects, engineers and students
- education/continued professional development
- slide sets and other materials for teachers or educators
- design aids & span tables (or computer programs) for engineers
- a range of costed standard designs (eg: industrial buildings)
- a regular journal depicting illustrated case studies.

5) *A link with British Timber:* It was noted that it is possible that the upsurge in supply of whitewood from Britain's own forests could be harnessed into glulam production. It is a well established fact that

> British sawmillers must target the UK construction industry if they are to sell their increasing production profitably. It is the interests of the glulam industry to work in collaboration with the wider timber industry, rather than see itself in competition with it.

The development strategy proposed by the TRADA study, has been influential in setting both educational and research priorities at Brighton Polytechnic as well as facilitating the establishment and direction of the Glued Laminated Timber Association (GLTA) in the United Kingdom. During time spent with Professor Hilson of Brighton Polytechnic the author was able to see some of the promotional material developed by the GLTA and other industry groups for use by professional architects and engineers. This included high quality colour brochures which presented case studies and examples and how glulam timber can be utilised, as well as extensive design handbooks for engineers covering the technical and detailing aspects of design.[27]

The applicability of this to Australia and New Zealand must not be overlooked. A number of similar initiatives have already been commenced in terms of education, research and promotion, but there still remains a great deal to be done. Much can be learnt from monitoring strategies such as those being implemented in the United Kingdom, which can be of enormous benefit to the timber (and particularly to the Glulam) industry in Australia.

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**Creep and Moisture Effects in Glulam:**

Much of Professor Hilson's recent work has been directed at meeting some of the research and educational needs which were identified in the evaluation and appraisal studies of the glulam industry in the UK (detailed above).

It is apparent from the author's observations of research work at Brighton Polytechnic, that a number of projects being currently undertaken have been identified and initiated from the priorities established by these earlier studies undertaken by Prof Hilson during the mid 1980's.

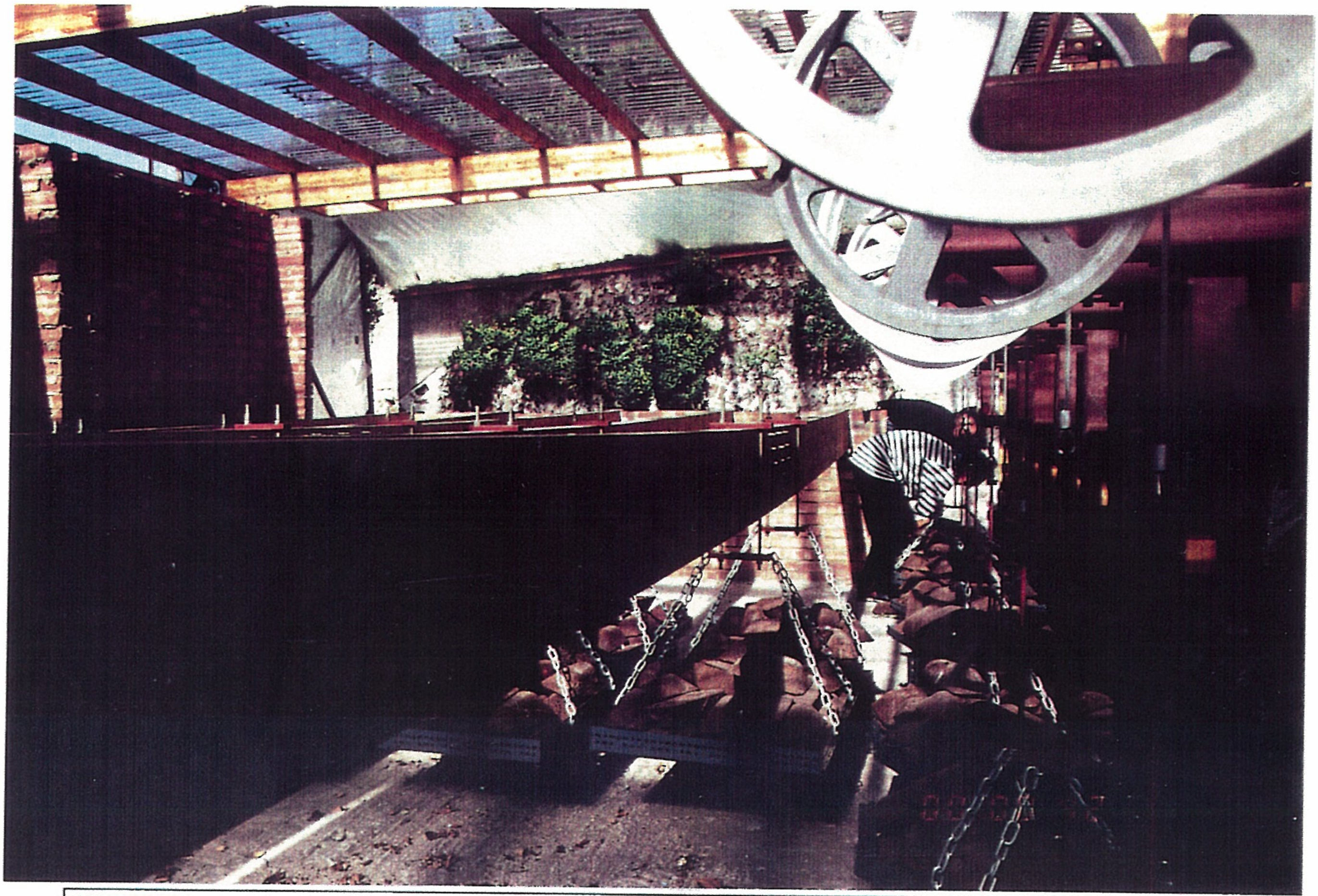
Amongst the most significant of these are research into dowelled and resin injected moment resisting connections for glulam portal framed buildings, (details of which are presented in Section 2 of this report) and research into the creep / moisture relationship (mechano-sorptive effects) and long term load behaviour of glue laminated beams. The testing facilities established for this later project are both extensive and impressive. The glulam creep tests consist of two main projects which involve:

1) continuous monitoring of both moisture content and environmental conditions for glulam beams under constant bending (set in vertical creep rigs) with different surface treatments. The tests are undertaken entirely within a controlled environment (humidity and temperature) which is continuously monitored by computer. Moisture contents are taken at a number of locations and depths in each beam under test, using moisture resistance probes which are linked to the data acquisition system.

2) periodic monitoring of beams under constant bending moment, mounted both horizontally and vertically (in creep rigs), and exposed to "normal" climatic variations (see **photo 4.1**)

Initial tests noted that the both the short term load / deflection behaviour and the long term moisture contents for solid wood and glue laminated beams (which were "equivalent" in their strength and stiffness) had negligible differences. Generally the stability of the glulam beam is superior and of course the variance of mechanical properties is not as great, as that for the solid wood beams.

The current research into long term behaviour of glulam beams, has revealed that the use of an epoxy sealant as an envelope treatment to the surface of the beam, ensures that the time taken for moisture changes within the beams occurs over a much greater period of time than for an "equivalent" untreated beam. Whilst this is not unexpected, the effect of this slowing down of the rate of moisture change as a beam adjusts to a new EMC (equilibrium moisture content) has proven to be quite significant, in that the *rate of change* of moisture content within a given beam has been observed to have a marked influence on the stiffness of the beam.



Photograph 4.1 - Long term creep tests in glulam beams - Brighton Polytechnic

Photograph 4.2 - An example of glulam construction - 56m clear span MDF factory - Nelson NZ
Timber supplied by Hunter Timbers, Nelson

The results to date have indicated that whilst the treated and untreated (surface sealant treatment) beams will eventually reach the same EMC, they do so *at different rates*. The sealed beams, which change more slowly and over a longer period of time, have been found to have *markedly higher "effective" stiffnesses* (ie: less mechano-sorptive deformation) than those observed in the untreated beams, which have deflections up to twice those of the treated material when both types reach the EMC for the given environmental conditions. The effect of cycling humidity and temperature further exaggerates this difference in deflection behaviour.

**Conclusions:**

The initial conclusions (based on the work completed to date) indicate that a definite relationship exists between the magnitude of creep deflections* under constant load and the rate of change of moisture content within the timber. The greater the rate of change of moisture content, the greater the loss of effective stiffness (or the greater the deflections). As the nature of these tests is long term, it is hoped that the monitoring program will continue for some time to come in order to establish a significant data base of long term mechano-sorptive behaviour, so that the observations can be suitably characterised and quantified for inclusion in design procedures.

The results of this research project could potentially be incorporated in the Australian design code as modifcations to the $j_2$ value in AS1720.1. One possible suggestion is to permit a reduced $j_2$ factor of say, 1.5 (as is permitted in NZS 3603 for glulam) for glulam which is effectively sealed at a "seasoned" moisture content, so that absolute changes and rate of change in moisture content are retarded so as to minimise mechano-sorptive deformation effects.

Suffice to say, the provision of an epoxy sealant coat as an envelope treatment for a beam, will slow down the rate of change of moisture content, thus ensuring superior long term stability and greater stiffness of the glulam member under constant load. Given the fact that detailing for moisture and weathering is already a major factor considered by most competent timber designers, the structural advantages of increasing the effective long term stiffness of a glulam element by using an "envelope" sealant, are not only desirable but are achievable at minimal cost to a building project.

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(*) Note:
This "creep" is essentially long term defection under sustained load, made up of pure creep and mechano-sorptive deformation. The rate of change of moisture affects the mechano-sorptive deformation, as detailed in the text.

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# SECTION 5:

# BUILDING APPLICATIONS FOR TIMBER STRUCTURES

*An Overview of Current Research of:*

*Seismic Design for MSTF in New Zealand (Section 5.1)*
*Floor Vibration and Damping (Canada) (Section 5.2)*
*Fire Resistant Construction (USA & NZ) (Section 5.3)*

## Section 5.1 - Multi-Storey Timber Framed Buildings and Seismic Design:

### Introduction:

The prospect of utilising multi-storey timber framed construction in Australia for 3 or 4 storey medium density residential buildings, is currently a major thrust of promotional and development work by NAFI and by the Western Wood Products Association (Australian Office). The market potential of this form of construction, which has been traditionally dominated by masonry products (in particular the "3 storey full-brick walk-ups"), is enormous. The use of this form of construction is widespread throughout North America (see **photograph 5.1**) and the relevance and applicability of transferring this technology to Australia has been well documented by Juniper.[1]

Similarly, in New Zealand light timber framed construction has been mainly confined to single storey domestic construction, although the technique is permitted for taller residential (and in some instances commercial) buildings. Apart from the normal fire resistance considerations which relate to the use of this type of construction, New Zealand designers must also pay special attention to lateral load resistance, particularly due to earthquake events.

Over the past few years considerable work has been undertaken at the University of Canterbury, Christchurch - New Zealand, investigating the strength and performance characteristics of multi-storey timber framed walls. The main testing program currently in effect at the time of the author's visit (in November 1990) was doctoral work by Mr Bruce Dean investigating the earthquake resistance of multi-storey walls.

This research work has involved the fabrication of a 9m, 3 storey timber shear wall with full height edge cords, nail laminated from 150*50 framing materials and conventional framing used elsewhere. Sheathing types used include 7.5mm plywood, 9mm plywood and 12.5mm plywood as well as recent tests using gibraltar boards structural bracing. The 9m high by 3.6m wide wall, is one of the largest structural components to be tested to date in New Zealand and is tested lying on its side (see **photograph 5.2** and **figure 5.3**). A large steel reaction frame is attached to the wall and hydraulic jacks are used to model the effect of earthquake forces and displacements.

### Structural Considerations for Seismic Design:

Details of this research project and its relationship to the size and design of wood structures have been recently presented by Buchanan, Dean and Deam[2]. The objective of earthquake resistant design is to ensure that buildings should suffer no damage when subjected to minor earthquakes, and only minor or non-structural or repairable damage under moderate earthquakes. When subjected to a major earthquake, significant damage may occur but there should be no collapse or breakdown of the essential structural integrity of the skeletal frame of the building.

Earthquake ground motions cause buildings to displace horizontally and induce horizontal inertial forces. The magnitude of these forces depends upon:

(a) the ground acceleration,
(b) the stiffness of the structure (and degree of damping) and
(c) the ductility of the structural system.

In order to resist earthquake forces, buildings must have the ability to undergo horizontal displacements without significant loss of strength. Stiff, brittle structures tend to perform poorly in earthquakes because sudden failures can occur at relatively small displacements. Structures made from ductile materials and connections behave well in earthquakes because they can be subjected to large displacements without generating excessive forces. This type of structural response is illustrated in **figure 1** and is often referred to as non linear or elasto-plastic behaviour, as distinct from the "normal" linear elastic behaviour which forms the basis of most traditional engineering design procedures.

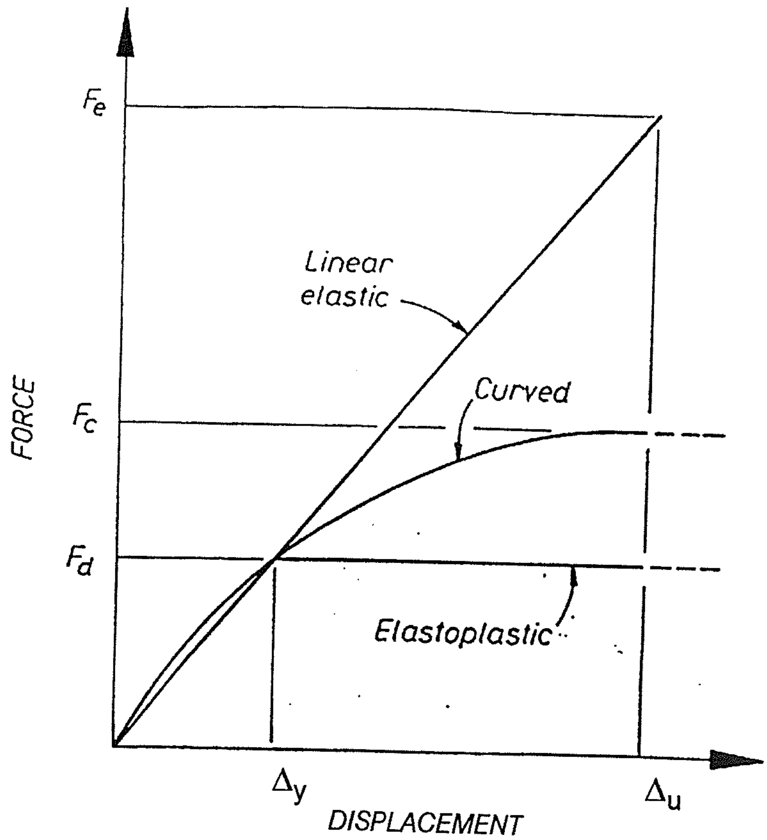
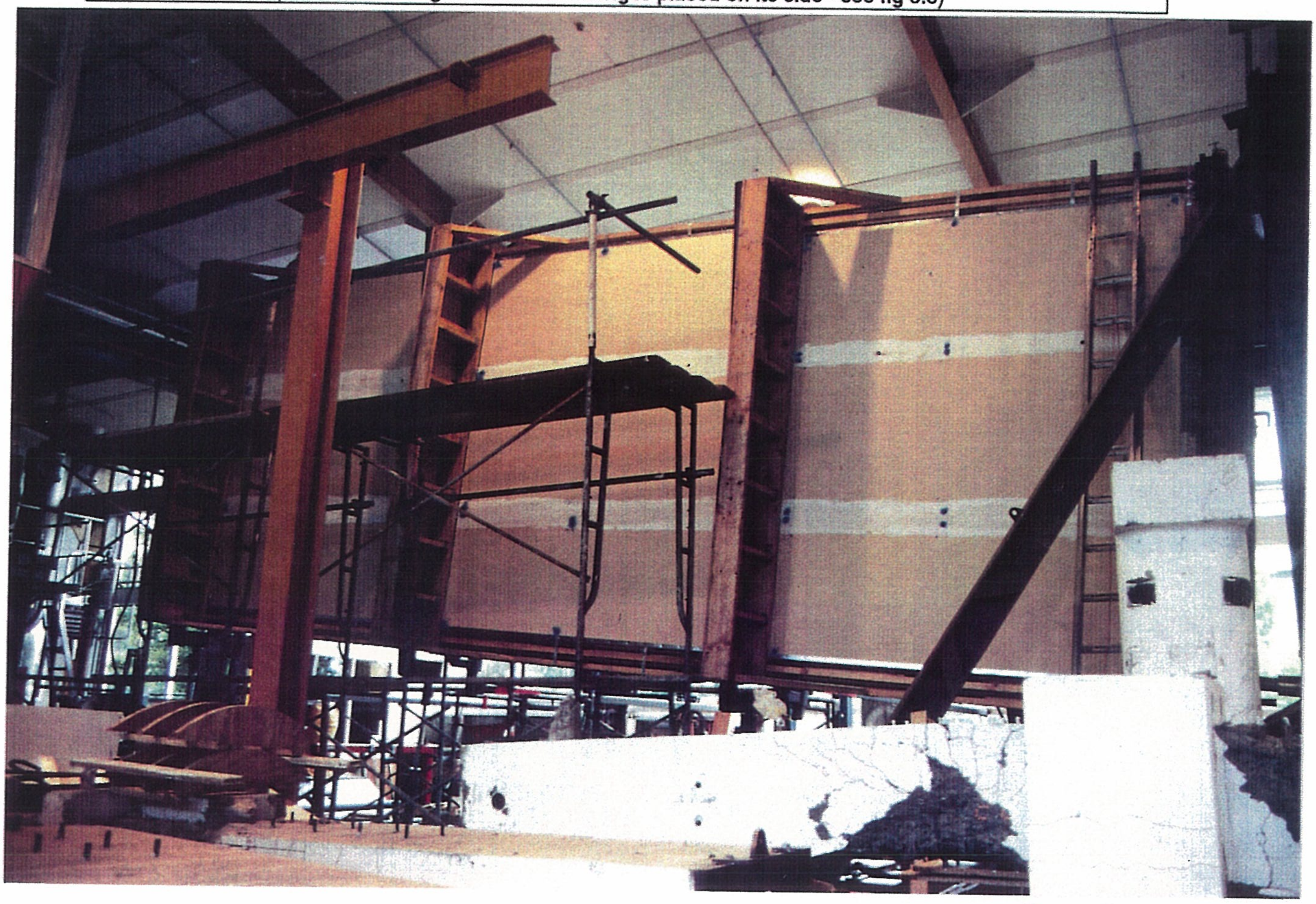



**Figure 5.1 - Idealised Load vs Displacement Curves**

**Photograph 5.1 - 3 storey MSTF residential construction near Madison Wisconsin, USA**

**Photograph 5.2 - Seismic test rig for a 3 storey timber frame at University of Canterbury, NZ (note: due to height limitations the rig is placed on its side - see fig 5.3)**



Due to these inherent characteristics, timber buildings have a good reputation for earthquake resistance. This resistance to seismic load events has been evidenced in the behaviour of domestic dwellings and other low rise structures, which incorporate "soft" connections (which are capable of undergoing plastic deformations) and redundant lateral load resisting systems. Historical records in New Zealand and overseas countries show that when failures have occurred in timber buildings, they have often been associated with poor design or detailing such as: poor foundation connections, no sub-floor bracing or torsional response resulting from unsymmetrical layout of lateral load resisting elements.

Recent studies by Buchanan and Dean[3] at the University of Canterbury has shown that for most typical earthquake records, the lateral displacement of a building will be approximately the same whether the structure behaves in a linear or ductile manner, and regardless of the shape of the load displacement curve or hysteresis loops. If a structure is to undergo large displacements, then all parts of the structure must be strong enough to avoid failure when the ductile parts of the structure achieve their strength capacity. Design of the members and connections to permit these large displacements to occur is known as *capacity design.*

**Capacity Design:**

The *capacity design* procedure must take into account the intrinsic material and mechanical properties of the wood, the connections and bracing systems used within the frame. The fact that wood is essentially ductile in compression can be used to advantage in seismic design. However, the brittle behaviour in wood tension and hence bending requires that these failure modes be suppressed by careful design detailing, to eliminate brittle tension and perpendicular to grain splitting failures. These structural behavioural modes are illustrated in **figure 5.2**:

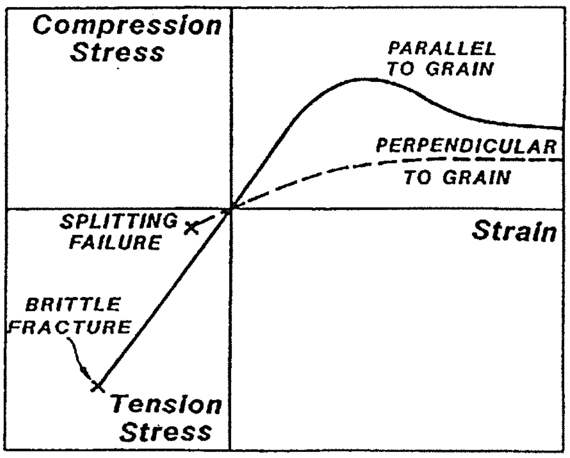



**figure 5.2 - *STRESS-STRAIN RELATIONSHIPS***

## Connection Behaviour:

Nails loaded in shear generally produce ductile connections capable of resisting many reversals of cyclic loading. Under such loading, nailed connections develop slackness as the nails bend (or nail heads rotate) and cause local crushing of adjacent wood fibres. However, large displacements can usually occur without failure, thus producing desirable behaviour. Small diameter bolts tend to behave like nails. For bolts about 12mm or less in diameter, the bolt shaft can bend without producing timber fracture and so produces ductile behaviour and load sharing between any number of bolts. However, larger diameter bolts often lead to brittle timber failures because of the rigidity of the bolt, particularly when the load component is perpendicular to the grain. (These relative characteristics of small diameter and large diameter bolts and the importance of ductility in connections are presented in detail in Section 2 of this report).

Tooth plates exhibit undesirable non-ductile behaviour modes when loaded seismically. The usual failure modes are either toothnail withdrawal or "postage stamp" failure plate, both of which occur suddenly with a complete loss of load. On the other hand nail plates can be used to transfer very large forces with nails which are well distributed over the surface of the timber member. Nail plates have been shown by Deam[4] to be capable of excellent ductile behaviour, provided that the nails themselves are the weakest link in the connection. However, the plates are prone to ductile buckling and compression failure in certain circumstances, and this must be considered in designing under seismic load.

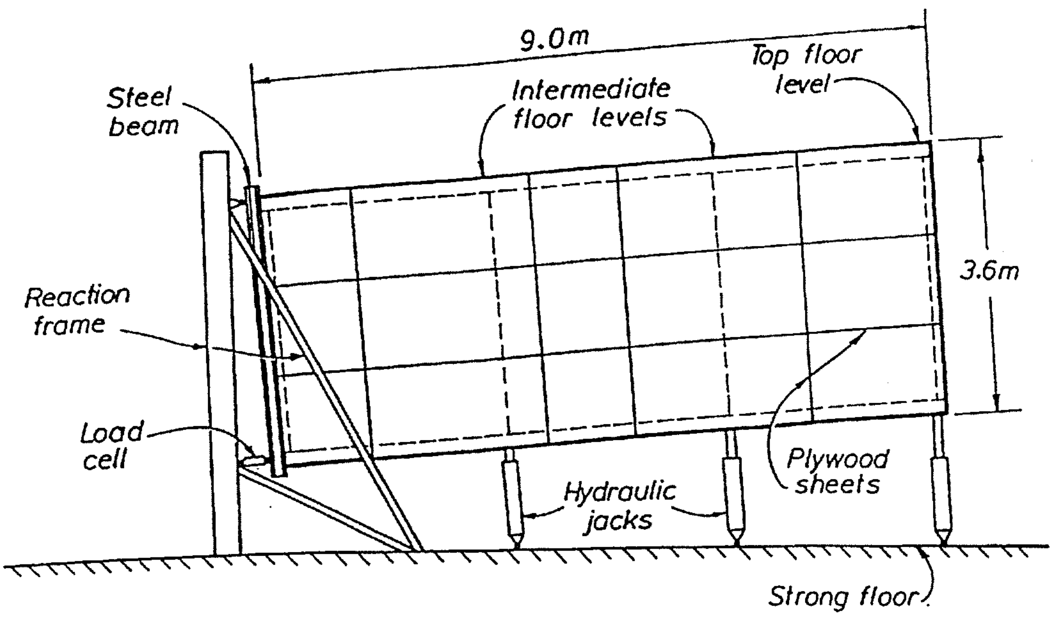



**figure 5.3 - Test rig arrangement at the University of Canterbury**

**Design Methodology:**

The proposed design method which has been formulated as a result of Deam's research is as follows, for the *capacity design* of a ductile wood structure.

1) Establish a structural system.

2) Calculate the mass.

3) Calculate seismic forces according to building code rules by the following procedure:

a) Obtain the appropriate design spectrum for the site for a linear elastic structure.
b) Estimate the stiffness of the structure. Allowance should made for both increased flexibility due to connections and the stiffening effect of non-structural walls. It is noted that these may tend to cancel each other out.
c) Calculate the period of vibration (using the Rayleigh formula or similar method) assuming a linear elastic response.
d) Use the spectrum to give the design forces for elastic response.
e) Establish the ductility available in the structural system.
f) Calculate the design forces. This is undertaken with the following formula:

$$F_d = F_e / u \qquad \textbf{equation 5.1}$$

Where:

$F_e$ is the design forces for elastic response
u is the ductility of the structural system; and
$F_d$ is the design force.

4) Proportion the weakest members or connections to resist the design forces.

5) For ductile structures estimate the increase in forces that will occur when large or moderate displacements are imposed. Proportion stronger members and connections to ensure that these increased forces can be resisted when the weaker ductile components have reached their load capacity. This process is referred to as "capacity design".

6) Check displacements of the structure against those permitted by the code or otherwise considered reasonable. Displacement should be estimated both for safety during unlikely major earthquakes and for serviceability during more frequent small and moderate earthquakes. *(design procedure quoted from Buchanan, Dean and Deam[5])*

**Research Conclusions:**

The conclusions[6] of this research project to date are that:

1) Well designed timber structures have excellent resistance to earthquake forces.

2) Ductile structures may be designed for much lower lateral forces than structures which remain elastic.

3) For timber structures to have predictable ductile behaviour, the connections must be designed to allow displacements to occur without significant loss of strength. This often requires that other parts of the structure be overdesigned to resist lateral modes of the capacity of the ductile components.

It should be noted that some of the work undertaken by Bruce Deam, investigating the seismic ductility properties of timber frame shear walls, has led to extensive reverse cycle testing of joints incorporating different nails and sheathing of different types and thicknesses. This in turn has led to recommendations on preferred nail joint details for ductile structures. The results of this project indicate that the use of thicker wall sheathing (i.e 10-12mm) is preferred, as it ensures much better ductile behaviour in the fastenings, possibly due to the restraining effect on the head of the nails which in turn ensures much better load slip characteristics between the fastener and the timber.

**Ongoing Research:**

At the time of the author's visit to the University of Canterbury during November 1990, the research project previously mentioned was continuing, investigating the relative performances of varying thicknesses of shear wall bracing and the bracing effects of plasterboard (eg: Gyprock) wall linings.

An application of this work and further research into the feasibility of MSTF for commercial buildings was also being undertaken by post graduate fellow, Geoff Thomas, under the supervision of Drs Buchanan and Dean. Thomas' work[7] focused on the feasibility of Multi-storey Light Timber Framed Buildings and used the construction (in masonry materials) of a recently completed YMCA hostel in Christchurch as a case study for potential application of MSTF.

This research project was undertaken by the University in conjunction with quantity surveyors and was completed in early 1991. On the basis of his research and the work of the quantity surveyors, Thomas concludes that multi-storey light timber framed buildings are feasible both from technical and practical aspects, resulting in significant time and cost savings for the client. Interestingly, the major barriers to the use of timber as a construction material in commercial building were found to be subjective, rather than technical issues. Resistance generally stemmed from conservatism and unrealistic perceptions of timber as a non-structural material by people involved with the construction and property industries and within regulatory bodies.

**Conclusions and Applicability to Australia:**

The research work into ductile connection behaviour and the load slip characteristics of nailed connections subject to lateral loading, has significant implications for the use of moment resisting connections (utilising plywood sheeting with glue laminated members for portal frames) in Australia and should be addressed in any future research into cyclic loading and fatigue for these types of structures.

The research work undertaken by the University of Canterbury into establishing rational design procedures for seismic loads is of particular relevance to Australia as code committees now address the problem of redefining earthquake design procedures for Australian conditions following the major disaster at Newcastle at the end of 1989. Considerable development of design procedures and analytical modelling of earthquake events has been undertaken at the University of Canterbury and the author was greatly impressed by the expertise which has been established in this area. The relevance of "technology transfer" across the Tasman cannot be understated.

## Section 5.2 - Vibration & Dynamic Responses of Timber Floors:

### Introduction:

A considerable amount of research into the dynamic responses of timber structures, particularly floor systems, has recently been undertaken in Canada, principally at the University of New Brunswick by Dr Y H Chui and Dr Ian Smith. Complementary work on floor vibration response has also been undertaken by Professor Ricardo Foschi at the University of British Columbia in Vancouver.

Recent trends to more material efficient design and construction of timber floors has resulted in lighter and more flexible structures. Consequently, vibrational serviceability of timber floors has been an area of interest to designers, researchers and codewriters.

At the time of the author's visit to the University of New Brunswick (UNB), work by Chui and Smith and doctoral work by L J Hu, focussed on vibrational analysis of wooden floors, and development of a rational method of evaluating the dynamic response of these floor systems.

### Modelling the Dynamics of Timber Floor Systems:

As noted above, the trend toward lighter and more flexible structures has often resulted in a high proportion of floors having unacceptable vibrational performance. Although essentially two-dimensional structures, timber floors have traditionally been designed based upon the static behaviour of a single joist under design loading. This is evidenced in the code practices of many countries including Australia, which produce allowable span tables for different grades and sizes of timber members, usually based upon a static live load of 1.5kPa.*

However, this simplified approach ignores the contribution of the flooring to the overall stiffness and strength of the floor system. Some codes, such as those in the United Kingdom, define static load limits in order to try and limit disturbing floor vibrations; however, this has generally not proven to be a satisfactory solution as excessive vibration can still occur even in floor which have been designed to a static deflection limit. Work by Chui and Smith[8] suggests that more appropriate criteria which relate to the actual vibrational responses such as natural frequency and acceleration levels should be considered when designing floor systems.

As the evaluation of vibrational responses is more complicated to perform than that of static behaviour, the adoption of these type of criteria requires a "designer useable" method of calculation.

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(*) Note:
Australian practice has been to include load sharing for a live load of 1.5kPa distributed and 1.8kN concentrated. A further load case of 0.5kPa (0.33LL) "permanent LL" with DL is also considered

Work by Foschi[9] at the University of British Columbia, Vancouver has resulted in the development of finite element models to simulate the dynamic response of floor systems and which attempt to quantify the subjective responses of people walking or standing upon the floor.

Quantifying subjective responses is inherently difficult. Depending upon the configuration of the floor (span, joist stiffness, joist spacing, etc) one person may find the vibrations caused by footfalls from another source as objectionable, whereas another person may not undergo discomfort as a result of the same vibrational source (refer to **figure 5.4**).

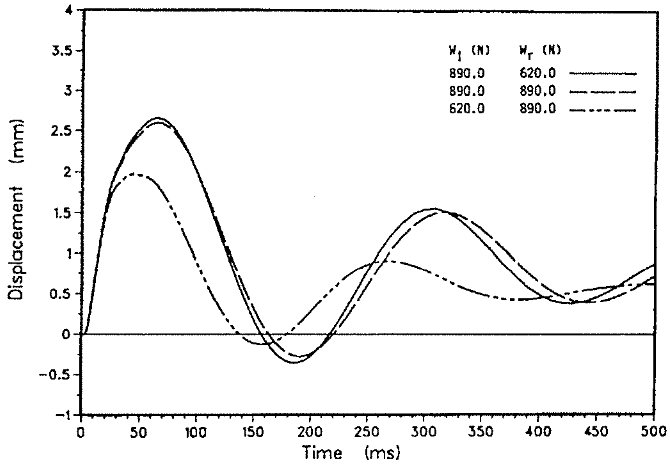



**figure 5.4 - Relative Effects of impactor's and receiver's weights**[10]

## Quantifying subjective "Comfort" levels:

Foschi investigated experimental studies of human responses to transient vertical vibrations, which quantified the results as a function of the vibrational frequency, peak displacement and damping. He then used these results by inputting numerical values which were assigned to various "quantified" subjective ratings, from imperceptible through distinctly perceptible to severe vibrational distress. Foschi's model involves a multiple regressional analysis of the results which relates three factors:

- (i) the subjective loading,
- (ii) the frequency and peak displacement or amplitude and
- (iii) the damping ratio.

On the basis of this work, reliability based design criteria have been proposed which agree with similar criteria currently in use in Scandinavian countries. This research also highlights the fact that the current design practice in North America is not satisfactory for controlling floor vibration. Significantly, it has been concluded that more rigid deflection standards should probably be implemented since floor spans for residential constructions will most likely be controlled by the serviceability limit state of excessive vibrations rather than bending strength.[11]

## Alternative Dynamic Models:

Chui's recent work[12] has compared Foschi's results with another dynamic response model, known as the Rayleigh-Ritz method. This predicts the natural frequencies and vibrational response due to defined excitation and has been preferred by Chui to other numerical techniques, such as finite element, since the models based on FE methods often require fairly large computer facilities, whereas the Rayleigh-Ritz approach can be incorporated into computers with a relatively small core memory, such as PC's.

The accuracy of the computer model has been tested and evaluated by comparing the prediction responses with the actual behaviour of structural floor systems. In order to validate the particular model, six floors were built and tested in a laboratory at the Department of Forest Engineering, University of New Brunswick. Hammer and human foot-fall impacts were applied to each floor and the response was measured using an accelerometer, whilst the impact force / time history was recorded by a force transducer. The test data were then analysed by means of modal synthesis technique.

The analysis of test data produced the natural frequencies and the associated damping ratios and mode shapes of the test floors and these measured results were then compared with the predicted natural frequencies and vibrational responses using the analytical model. The comparison of these results indicated that agreement between the predicted and measured natural frequencies is very good, (refer **figure 5.5**) particularly for the first 2 or 3 natural frequencies of the timber floor, although it was recognised that for the higher natural frequencies, the percentage difference increased. A comparison with acceleration and damping effects was less favourable, with the average difference in the order of about twenty five percent, however, the degree of accuracy is deemed to be acceptable for design purposes.

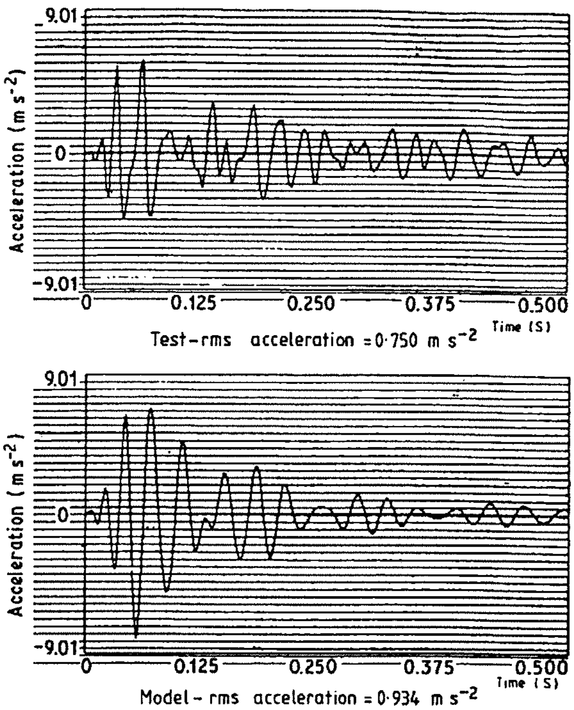



**figure 5.5 - Typical comparison between test data and predicted time histories for timber floors (from Chui,[13] University of New Brunswick)**

**Research Conclusions and Recommendations:**

Chui concludes that the mathematical model developed has been shown to produce accurate predictions of natural frequencies of timber floors and reasonable predictions of time-history of floor response to human loading. Other conclusions regarding the sensitivity of the floor to changes in parameters have also been drawn:

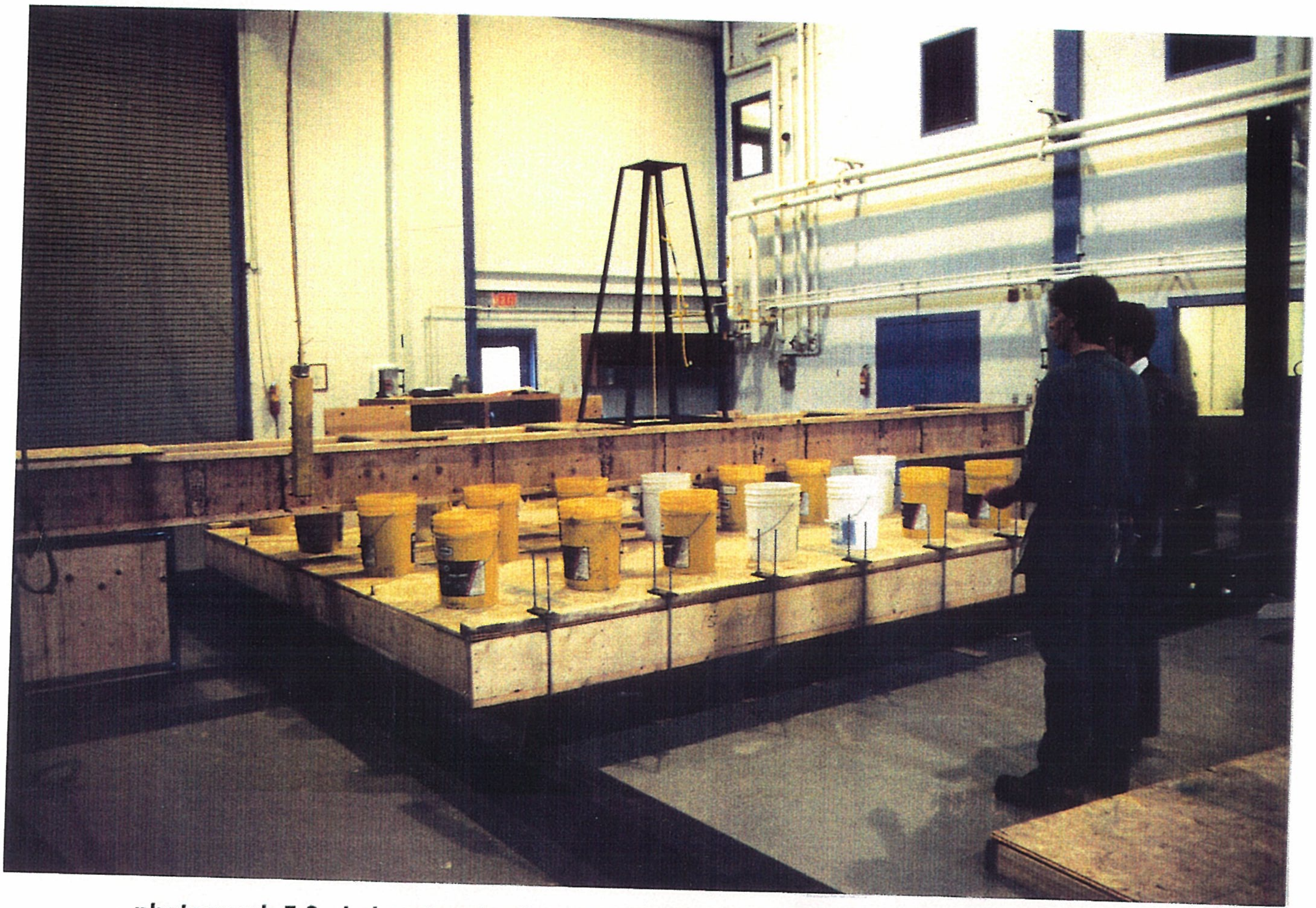
1) To optimise floor performance joists should be designed to span in the shorter direction of a floor area.
2) Floor behaviour is not sensitive to joint stiffness within the practical range. Hence the use of stiffer joists may not lead to a noticeable improvement in vibrational response.
3) Timber floors are orthotropic, plate-like structures. It is desirable however to increase the degree of isotropy either by installing between joists strutting between the main joists or the use of a flooring system which has a high bending stiffness in the across joists direction.[14]

A significant conclusion of this work was highlighted in discussions between the author and Dr Chui. This was the second conclusion detailed above. Despite the view traditionally held by most engineers, increasing the size and stiffness of the floor joists *does not necessarily mean that the vibrational response of the floor will necessarily be improved*. Indeed, both Dr Smith and Dr Chui expressed the view that the effect of *damping is much more significant than stiffness*, and that it is highly desirability to induce isotropic rather than orthotropic behaviour in the floor system.

**Current Research:**

Doctoral work currently (at the time of writing) being undertaken by L J Hu, includes ongoing large scale testing of floor systems in the laboratory (see **photograph 5.3**) and development of a modal synthesis model, which overcomes some of the limitations of the Rayleigh-Ritz method previously proposed by Chui. The limitation of the Rayleigh-Ritz method which has been highlighted by Hu's work,[15] comes from the difficulty in defining suitable mode shape functions to which the accuracy of this particular method is strongly related. The modal synthesis model adopted by Hu is used to predict the frequencies of timber floors based on Hurty's free interface method.

This modal synthesis method is preferred to the Rayleigh-Ritz method because mode shape functions are applicable to the entire floor system are not defined at the outset. Instead, they are synthesised from mode shapes that are selected from those of the separate elements of a system i.e. the sheathing and joists. In addition, the advantages over other approaches are the potential for modelling of the complicated interconnection between sheathing and joists in an explicit fashion and computational efficiency compared with the finite element approach. Hu's work to date has shown that this modal synthesis method yields acceptable results for stiffened plates such as exist in timber joist floor systems. It also enables the effect of damping considerations to be accurately modelled and offers great potential as a computerised design procedure with considerable advantages over other techniques which have been developed to date.



**photograph 5.3 - Laboratory testing of dynamics of timber floors at UNB (by Hu)**

## Additional Research into determination of Mechanical Properties:

Additional work by Chui and Smith, in the area of dynamic response of timber products, has investigated vibration testing as a means of the determining the dynamic modulus of elasticity of wood. This research has shown that depending on the mode of testing, different moduli of elasticity (eg: for each principal axis) can be evaluated. The advantages of using the vibrational approach to determine MOE over the static bending alternative are:

1) The results are not sensitive to geometric imperfections such as twist and bow of the test specimen.

2) The time taken to conduct the vibration test is shorter than the alternative static test. Usually the time required to complete a vibration test is 10 - 15 seconds, whereas a static bending test would take 2 - 3 minutes to complete.

It must be noted that Modulus of Elasticity determined from vibrational tests is different from the statically derived parameter; however the difference in values is generally very small with $E_d$ (dynamic) being slightly higher than $E_s$ (static).

The results of this testing program have highlighted the difficulties in conducting vibration tests and discuss a number of the factors which can seriously impair the accuracy of the test results if not dealt with properly. In addition to highlighting these problems, recommendations have been made concerning uniformity of testing procedures and equipment, etc to ensure that this application of nondestructive testing for determination of material properties (MOE), can be used as a reliable and cost effective quality control procedure.

**Conclusions and Relevance to Australia:**

It is the author's belief that the work being currently undertaken on floor vibration and the development of suitable, accurate analytical models for predicting vibrational responses, is of considerable significance to the domestic timber industry in Australia, as well as in North America. The implications of the research completed to the present time are of considerable importance in developing far more "efficient" means of using timber in floor systems for both residential and commercial applications. The important factors to be considered include design details such as:

(1) increasing damping effects due to the incorporation of adequate connections between wall frames and floor structures,
(2) inclusion of sub-floor damping elements and
(3) modifications to existing procedures used to define floor systems which are essentially orthotropic, rather than isotropic

Thus, basic modification of existing design and detailing practices could result in considerable improvements to the "comfort levels" experienced by residents of dwellings with timber floors, without significantly increasing the cost of the flooring system or increasing the amount of timber required to achieve such systems.

As mentioned above, the traditional approach of increasing the stiffness and size of the joists as a procedure to overcome vibrational problems, has been proven to be not particularly valid and as such, often results in a fairly inefficient use of the timber resource. It is the author's belief that the published literature of research that has been undertaken to date at UNB already has significant application to the design of floors in an Australian context. The ongoing research at the University of New Brunswick should be monitored closely, with a view to adaptation and inclusion in Australian design procedures.

It should be noted that significant work* in the area of floor design has been undertaken within Australia since the early 1980's, most notably by McDowall at the Capricornia Institute of Advanced Education (now CUQ), under the auspice and support of the Plywood Association of Australia. This work, whilst not funded to the extent of some of the overseas research presented here, nevertheless provides a sound research basis for floor dynamics which is of a world class standard.

Recent initiatives by NAFI in respect to Timber Framing will continue to develop this work, with the development of a simplified design procedure for floor systems, which will take into account composite action, load sharing, load distribution and orthotropic effects. Future projects will focus on dynamic responses and damping effects, hopefully developing the work undertaken by McDowall through ongoing interaction with some of the overseas research work presented in this report.

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(*) Note:
Since this work is outside the scope of the author's original brief in reporting on overseas developments, it has not been specifically referenced in this report

## Section 5.3 - Fire Resistant Timber Construction:

### Introduction:

Whilst visiting the Forest Products Laboratory in Madison, Wisconsin in September 1990, the author was able to spend some time in obtaining an overview of research into fire endurance and fire protection, which is currently being undertaken at FPL. The major thrust of the work at present is the development of analytical (usually finite element) models which can be used as predictive and/or research tools for a modelling of fire spread, fire performance of timber members and timber structure. In the United States, fire resistance ratings of wood members and assemblies have traditionally been obtained by testing the assembly in a furnace, in accordance with ASTM Standard E-119. (This type of fire testing is also used for the determination of fire resistance properties of other materials).

### Design Procedures:

The ratings listed in the American design procedures are limited to the actual assembly tested and normally do not permit modifications such as adding insulation, changing member size, changing or adding interior finish or increasing the spacing between members. In recent years two fire endurance design procedures for timber which allow greater flexibility, have gained acceptance in USA and Canadian building codes. In addition to these two initiatives, other procedures and models have been proposed or are being developed.

When attention is given to all details, the fire endurance of a wood member or assembly usually depends upon three items:

1) Performance of its protective membrane (if any)
2) The extent and rate of charring of the structural timber element.
3) The load carrying capacity of the remaining uncharred portions of the structural timber elements (usually referred to as the residual section).

### Research into Fire Performance of Structural Timber Members and Assemblies:

Fire endurance research activities and facilities at FPL are concerned with the ability of a timber member to withstand the degradation effects of fire whilst acting as a fire barrier and supporting a load. It has been observed that fire endurance is generally concerned with the post-flashover portion of the fire. The importance of fire endurance in fire safety is reflected in building code requirements with fire resistance ratings (e.g. 1hour, 2hours, etc.) for building elements.

The FPL fire endurance research program is oriented largely towards supporting:

1) the increased uses of timber in commercial and multi-storey residential buildings where fire resistant assemblies are required by building codes, and

2) the use of new constructional wood components.

Research and development experience has shown that new timber engineering technologies can in fact complete with current levels of fire safety and this is a prime motivation for research. The research facilities and equipment at FPL is quite impressive and includes a tension / furnace apparatus, an ASTM large vertical furnace and a small vertical fire resistance furnace. Additionally an ASTM smoke release apparatus is also available for charring studies.

Whilst current efforts are concentrating on models of parallel cord truss systems and wood joists floor systems, previous research at FPL has focused on the development of fire endurance models and the accumulation of fire performance data. These previous research programs have resulted in analytical procedures for walls, unprotected joists floor, glue laminated timber and unprotected roof trusses. The fire endurance models currently in use were developed largely in co-operation with universities.

The Virginia Polytechnic Institute and State University and the FPL have developed a reliability based model for fire endurance of glue laminated beams, a model for unprotected wood joist floors and a simple model for unprotected wood truss floors which was limited to evaluating the residual strength of individual wood components. In a co-operative effort with the FPL, the University of California-Berkley developed a fire endurance model for wood frame walls. Although this wall model predicts the buckling failure of a single stud, it is primarily a transfer model for predicting the temperature distribution and thermal degradation within the wall assembly.[16]

**Development of Analytical Models for Fire Endurance:**

Current research activities include testing of wood, stud walls, evaluation of the National Forest Products Association (NFPA) system one floor model and testing of wood in an ASTM E-906 furnace and development of a fire endurance model for trusses. Previous work by the FPL with a single joist model was so indicated the need to include load sharing and composite action and the ASTM E-906 furnace has been used to determine the charring rate of wood under various defined heat fluxes for incorporation in the model, which has been developed concurrently with the University of Wisconsin, in Madison.

The type of analytical models required for prediction of fire resistance of timber members are extremely complex in nature, due to the large number of variables that are required to be taken into account in the analytical procedures. A key variable in understanding the fire performance of a particular timber member is known as the charring of the wood.

**Wood Charring:**

Timber undergoes thermal degradation or pyrolysis when exposed to fire. The pyrolysis and combustion of timber have been studies extensively and is understood as a transforming reaction, whereby the fire converts the wood to char and ash and pyrolysis results in a subsequent reduction in the timber density. The pyrolysis gas undergoes flaming combustion as it leaves the charred wood surface, whilst glowing combustion and mechanical disintegration of the char eventually erode or ablate the outer char layer.

The charring rate generally refers to the linear rate in which wood is converted to char, under standard fire exposure the charring rate tend to be fairly constant after a higher initial rate. Establishing the charring rate is critical to evaluating fire resistance because char has virtually no load bearing capacity. Fortunately for designers, there is a fairly distinct demarcation between charred and uncharred wood, (see **figure 5.6**) with the base of the char layer being defined by wood reaching a temperature of approx. 290°C. In order to determine the charring rate FPL has developed both empirical models based on experimental data and theoretical models based on the chemical and physical properties of the timber.

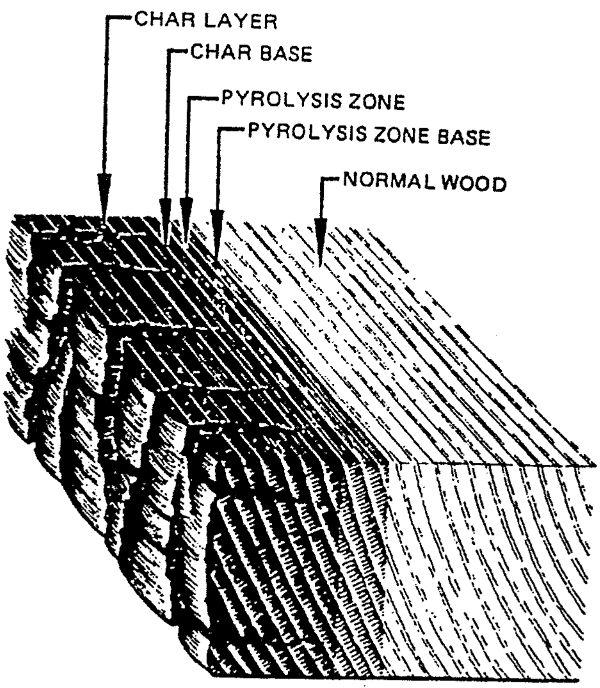


**figure 5.6 - Degradation zones in charred timber (from White[17])**

**Empirical Models:**

Expressions for charring rate in the standard ASTM test (E-119) are the result of many experimental studies. The empirical model that is most commonly used, generally assumes a constant transverse to grain char rate of 0.6mm per minute for all timber types when subjected to the standard fire exposure. However, there are differences both between and amongst species associated with the density, chemical composition and permeability of the wood material, as well as the fact that the moisture content of the wood affects the charring rate and these are not taken into account in this ASTM test.[18]

Research work by Schaffer[19] at FPL has reported that the transverse to grain charring rates are a function of density and moisture content for White Oak, Douglas Fir and Southern Pine and as a result of his work, regression equations were developed for the different species. In practice, the linear charring model is adequate, provided the model takes account of the moisture content and density variables. It should also be noted that the charring rate parallel to the grain is approximately twice that of that transverse to the grain. Similarly as a beam or column chars, the corners become rounded and this rounding effect is generally considered to have a radius equivalent to the char depth on the sides.

Some research for these empirical models has been undertaken on the effect on fire retardant treatment and adhesives. It has been noted that glulam members which have been bonded with phenolic or resorcinol adhesives, have a charring rate consistent with that of solid wood.

Fire retardant treatments are designed to reduce flame spread. The fire retardant's effect on the charring rate may be to only slightly increase the time until ignition of the wood, although it has been found that some fire retardants reduce flammability by lowering the temperature in which the charring occurs. This however, may increase the charring rate. Research to date has confirmed that in practice, few fire retardants have been found to actually improve charring resistance.

**Theoretical Models:**

Because of the considerable cost associated with undertaking full-size testing to determine the fire resistance variables, considerable work has been undertaken into the development of theoretical models for wood charring. Theoretical models allow calculation of the charring rate for geometry other than a semi-infinite slab and for non standard fire exposures. Unfortunately no completely satisfactory model has yet been developed.

Significant problems have been encountered in trying to effectively model the complex interactions which occur in the burning of timber. Some of the influences that effect the theoretical analysis and create problems are the structural effects and internal heat transfer, kinetics of the pyrolysis reactions, heat of reaction of the pyrolysis reactions and variations of thermal properties of during pyrolysis.

The most advanced models to-date have attempted to address these issues by the formulation of a mathematical (numerical) model for the complex chemical and physical processes which occur in the heat pyrolysis reaction. However, any model can only be validated by the acquisition of reliable data (for use in the model) and a significant amount of research (with associated costs) has to continue in order to "fine tune" the models to an acceptable limit.

Many models for wood charring are based on the standard conservation of energy equation. As a refinement of this approach, numerically derived heat transfer methodologies are currently being developed to include the more complex variables which interact within the pyrolysis reaction. These new models take into account convection effects, differential charring rates, parallel and perpendicular to grain effects and more recently, inservice moisture content, moisture desorption and surface recession.

It has been observed that there may not only be a moisture desorption but also an increase in moisture content behind the char front caused by moisture movement away from the char surface. A major problem with the use of these more sophisticated models is the lack of adequate data to be used as input. Most theoretical models for wood charring not only define the charring rate, but also provide results for the temperature gradient. This temperature gradient is important in evaluating the load carrying capacity of the wood remaining in the uncharred section.

**Load Carrying Capacity:**

During the charring of wood caused by fire, the temperature gradient is fairly steep in the timber section remaining uncharred and it has been observed that some loss of strength undoubtedly results from the remaining timber or residual section being exposed to these elevated temperatures. Recent work by Schaffer[20] combines parallel to grain strength and stiffness relationships, with temperature and moisture content and the gradients of temperature and moisture content within a fire exposed element to obtain graphs of: relative modulus of elasticity, compressive strength and tensile strength as a function of distance below the charred layer. Theoretical models based on this work can be used to determine the temperature gradient within the wood material which remains uncharred.

Recently developed techniques[21] have also resulted in the development on a reliability based model to predict the strength of glue laminated members under normal temperature conditions and then extend this model to include fire endurance analysis. The model for the glue laminated beams uses a transformed section analysis to determine stresses within the laminates. This is then used as input for the fire endurance model by increasing the char depth at each time increment, until the calculated stresses within the laminates exceed the corresponding tensile strength values. The critical moment permitted by lateral torsional buckling has also been calculated and the lowest value of moment leading to either a predicted rupture or to buckling governs the time to failure.

The complexity of these models cannot be overstated and ongoing research focuses on the need to better understand the relationship of strength and stiffnesses of timber as functions of temperature and moisture content, coupled with the inherent rheological characteristics of the timber including density, moisture content, grain orientation and effects of defects.

**Fire Endurance Research in New Zealand:**

Considerable research into a fire performance of glulam members, particularly those with nailed connections, has been undertaken in recent years in New Zealand, both the University of Canterbury, and by the Building Research Association of New Zealand (BRANZ). During the author's visit to the University of Canterbury in November-December 1990, some discussions were held in regard to work which had been previously completed by post-graduate students into investigating the fire performance nailed connections between glue laminated timber members using steel or plywood gusset face covered with gypsum plaster.

**Fire Performance of Nailed Connections:**

Post-graduate research by Chinniah[22] has investigated the load slip behaviour of nails in shear, when gusset plates for nailed moment resisting portal haunch connections, were heated to simulated fire exposure. This research work involved measurement of temperature along the nails and quantifying load slip relationships, in order to predict the moment rotation characteristics of large scale joints including full scale test specimens, which were actually tested for a typical portal framed building.

Whist the basic fire resistance and residual strength characteristics of heavy glulam and timber sections have been fairly well established, little is known about the fire performance of metal or metal/timber connectors between the glue laminated members. Connections are a vital part of any structural system and premature failure due to fire or heat effects may subsequently hamper evacuation of occupants and subsequent fire fighting.

Overseas research indicates that the load capacity of fire exposed connections is reduced considerably during a fire, steel connections absorb heat and cause charring of adjacent timber, whilst timber or plywood gussets possess inadequate inherent fire resistance. The fasteners which form the connection conduct heat into the timber, initiating softening of the local fibres, local charring and loss of anchorage. Lack of design information regarding the behaviour of these connections in a fire situation has been considerable concern for the local construction industry in New Zealand, as has been the case in other countries, such as Australia.

Chinniah's research also investigated fire protection of connections using a number of different types of protection, with the main focus being on the use of fire resistant plaster boards and the application of intumescent coatings.

**Conclusions:**

The conclusions of Chinniah's work[23] are as follows:

1) Under stimulated fire conditions the greater slip of nailed steel gusset plate connections (as compared to timber gusset plate connections), increased for loads and also with higher gusset temperature for a given load.

2) As a result of the findings of (1), gusset slip increased as the thickness of the simulated fire protection board decreased. Highest slips were measured for the simulated intumescent paint protection. An approximate linear expression (based upon Foschi's research[24] for nail slip under normal service conditions) was developed from the research to relate slip to the gusset temperature at 1hr of simulated fire exposure.

3) For the plywood gussets only one thickness of protection was simulated. Gusset slip increased for greater loads. Measured slips for plywood gusset were generally larger than for steel gussets with the same protection.

4) The ultimate load capacity of nails decreases with simulated fire exposure. The ultimate load capacity of nails was approximately 10 times the basic load under cold (ie: normal service) conditions and reduces by 40% after 1hour of simulated fire exposure.

5) The predictive model developed by Chinniah has been assessed in full scale testing at BRANZ and it was found that the model predictions were close to the measured deflections for most of the fire test periods. However, at the end of the test periods, the deflections became considerably greater than those predicted from the model.

6) The results of this study were also used to predict the theoretical behaviour of a typical glulam portal frame building exposed to fire. For both steel and plywood gusset one layer of 19mm gibraltar board provided sufficient protection such that failure of the frame after 1hour of fire exposure is calculated to occur in the glulam members, whilst the joints remain well below ultimate strength. Frame deformations due to joint rotations have been found to be much less than those due to the charring of the members.

Chinniah has highlighted the need for future research in the following areas:

1) Load deformation characteristics of nails in connections protected by thicknesses of gibraltar fireboard less than 19mm thick and by other types of board, should be investigated

2) Most of the conclusions arrived at in the study undertaken by Chinniah were the results of single tests only. When one considers the inherent variability that exists in the material like timber, it is desirable that a larger sample group of testing is undertaken to improve confidence in the results.

3) The time / temperature results provided by the BRANZ study for Chinniah's project were obtained by exposing only one side an unloaded, protected joint to fire and measuring the appropriate temperatures. Subsequent BRANZ tests indicate that the fire severity may be greater if the joint undergoes full exposure. This is an aspect that needs considerable more research.

4) The variation of embedment strength of timber and bending strength of the nails with increasing temperature, could also be studied and from this, an ultimate strength model could be developed to predict the load carrying capacity of the nails under fire conditions.[25] (This type of study is imperative if fire protection is to be included as a limit state in a reliability based design procedure).

**Application and Relevance for Australia:**

Fire performance of timber structures is a major obstacle to the expansion of markets for timber in Australia. The work undertaken by FPL at Madison and researchers in New Zealand, (both at BRANZ and the University of Canterbury), has direct applicability in providing research data which could be incorporated into Australian testing procedures. It is imperative that such procedures should be developed (as a part of the current NAFI projects in this area) in order to address the problem of gaining acceptance of fire resistance design and construction for incorporation within Australian Building Codes.

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# SECTION 6:

# DEVELOPMENTS IN MATERIAL CHARACTERISATION & DESIGN PROCEDURES

*An Overview of Recent Research and Developments*

**Introduction:**

Specific design procedures and research programs which undergird these procedures, have been described in the relevant sections of this report which deal with particular aspects of design eg: such as connection and joint design. However, in addition to these specific procedures, the author (during his Gottstein Fellowship in 1990) was able gather information concerning general design procedures and the characterisation of material properties for structural design in timber.

Specific mention must be made of the fact that much of this information contained in this Section, was gathered at the University of British Columbia in Vancouver (UBC). Work undertaken over the last ten years at this institution by Professor Borg Madsen and Professor Richardo Foschi, has led to significant advances in the understanding of the intrinsic behaviour and material characteristics of timber as an engineering material. This Section gives an overview into some of the work which has been undertaken at UBC and elsewhere.

**Ingrade Testing:**

The concept of ingrade testing (IGT) has been widely credited to be the brainchild of Professor Borg Madsen, which he first proposed at a meeting of IUFRO, at the working group of timber engineering in London in 1969. Despite initial resistance both from his contemporaries and from industry sources, Professor Madsen managed to persuade the Canadian timber industry to participate in a large scale ingrade testing project.

The results of this research program sent shock waves throughout the industry. Douglas Fir was shown to have been seriously over-rated as a structural timber under the Canadian National grading system, whilst some of the other species including Pine, were shown to have been under-rated. Also, the grading rules which have been based (like most grading rules of the time) on tests on small clear specimens were shown by Madsen to give little differentiation between grades.[1]

The testing of small clear specimens has certainly played a very important role in the development of timber engineering in the past, especially in countries like Australia which utilise a large variety of species for structural purposes. Generally, such testing is quick, easy and does not require highly specialised equipment. However, by the mid 1970's the international timber engineering community was testing more and more full size timber elements and was beginning to realise that application of simple stress ratios to the results of tests of small clear specimens, was dangerously over-rating tensile strength in softwood species (particularly Douglas Fir) at the low end of the grade, whilst often under-rating the compressive strength of "full size" timber members (ie:

the small clears / stress ratio system was shown to be inaccurate for full size members in actual structures).

This disparity between results derived from small clear and full size specimens was mainly attributed to the effects of increasing knot size, which is more serious in tension than in compression. Furthermore, it was demonstrated that the effects of moisture content and long duration loading, which had been previously determined using small clear specimens, were in fact quite different for commercial grades in sizes. Madsen postulated that the only reliable way to establish *true design strength* for grades of timber, (ie: as used in service) was to test pieces from these grades and then derive their 5th percentile* strengths, applying suitable factors for safety and long duration loading.[2]

Distribution of strength within a "population" of pieces of timber is not very predictable and the simple application of Gaussian statistics can lead to gross inaccuracies - which are usually unnecessarily conservative. The ingrade testing approach postulated by Madsen and statisticians who were working under his leadership, considered the application of "non parametric statistics" to be more relevant. The idea behind this is to test enough pieces to be able to rank the results in order of strength and plot these in order to establish where the 5th percentile lies.[3] Obviously, to do this with any degree of reliability and in order to obtain an accurate representation of variability of the timber material, it is necessary to test large numbers of pieces.

**Proof Loading:**

As it is only the 5th percentile strength that is needed, all that is in fact necessary is to "proof load" the timber to a level that will break, say 10% of the pieces of the population and plot these results to give a good picture of the lower "tail" of the distribution from which the 5th percentile values can be read. The undamaged pieces in the sample are then available for sale in the normal way. In order to obtain an estimate of the mean strength for reliability based "limit state" design, it is necessary to test only a small, random sample of the pieces to destruction. The mean stiffness (Young's modulus "E") which is necessary for establishing "serviceability limits" in structural design, can be obtained easily as the stiffness of every piece is derived from its deflection under the "proof" load.[4]

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(*) Note:

It could on fact be any percentile which provides reliability commensurate with the risk / cost model for the structure

**Applications of IGT:**

Much of Madsen's work has been used to form the basis for reliability design procedures, which have been incorporated into the Canadian design codes. Recently, this approach to material property characterisation has been applied in a number of overseas situations, notably in South Africa by Bryant[5] and more recently, in Australia (for locally grown Radiata Pine and other species eg: Cypress Pine) by Leicester and Grant.[6]

The application of ingrade testing has considerable benefits for limit state design formats, where it is necessary to specify engineering design properties in terms of "characteristic" values. Unlike the currently used working stresses (in Australia), these characteristic values can be checked by direct measurement on a sample of structural sized timber. Conversely, if the specified characteristics have never been measured directly by the testing of structural sized timber then there is the potential for adverse litigation in the event of the structural failure. This potential for litigation is viewed with considerable concern throughout North America and has been a major incentive for research into quantifying characteristic values, so that design procedures can indeed be reliable.

Techniques currently used in Australia* and overseas for the grading of timber, such as visual and machine stress grading, are actual only sorting processes which provide an *indirect means* of assigning strength properties to a population of graded timber. In grade testing, on the other hand, provides a means of verification of the properties of a graded population of full size sticks of timber, which have already been sorted into specific structural stress grades. These can be subjected to ingrade testing to verify the strength assigned as a result of the sorting process.

Strength and other properties are measured directly from the stress grade testing of full size timber. By the application of statistical sampling techniques, stress levels corresponding to a lower bound exclusion limit can be estimated to verify the assigned stress grades.[7] It is this the sort of approach which has now been implemented throughout the timber industry in Canada, as a result of Madsen's work. In-grade testing leads to the development of reliable characteristic properties of timber, (for a particular species and grade) which in term can be incorporated into reliability based design procedures, for limit state design utilising timber in engineered structures.[8]

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**(*) Note:** It should be noted that the current systems used in Australia and reflected in both the Timber Framing and Timber Structures Codes, were basically derived for housing construction and that they have basically served well in this regard. It is not the author's intention to create a panic situation in respect of the existing grading system, but to simply highlight the potential benefits of applying Madsen's work to Australian design practices and perhaps recognise some of the limitations inherent in the "idealising" a non-homogeneous material. This is particularly so of engineering applications where the risk and cost of failure can be very high.

**Relating IGT Research to Size and Volume Effects:**

Following on from the extensive work undertaken on ingrade testing in Canada, Prof. Madsen has focused of late on size and length effects (generally now referred to as volume effects) within timber members and how these influence the characteristic strengths of the members under service conditions.

Tension tests conducted in Canada as a part of the ingrade testing program showed that a depth effect exists for timber* under tension. Similar tests also have shown that unconservative errors in excess of 20% can occur if length effects are omitted in the design process, even if a depth effect is included. Subsequent ingrade testing in which different gauge lengths were used for different beam sizes has shown that in addition to the established depth effect, the length effect must also be incorporated into the design procedures. Madsen has then developed a design method which involves both depth and length effects and enables the characteristics strengths for tension to be established.[9]

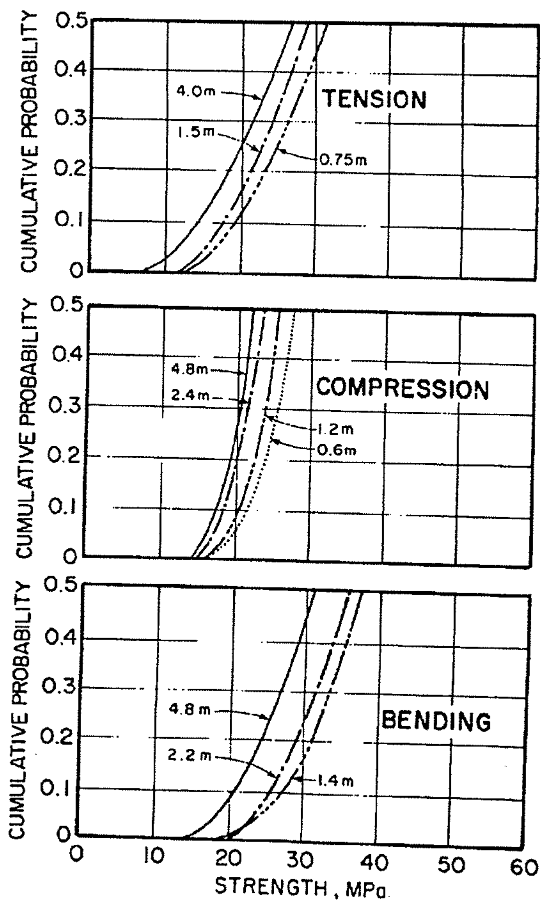


**figure 6.1 - fitted distributions for tensile, compressive and bending strength, taking consideration of length effects[10]**

(*) Note: Madsen's work has been essentially based on softwood timber, predominantly Douglas Fir. There is no conclusive evidence relating these effects to hardwoods, although recent work at UTS by the author on determining MOE and MOR values for wide kiln dried hardwood, suggests that a volume effect is applicable. This will be further investigated in future testing.

Data on compression tests from the ingrade testing programs has not been able to determine (to-date) whether the effects which are influencing the characteristic strengths are due to either depth of length. The data have been interpreted as a length effect and characteristic compression strengths are to be determined in conjunction with a column formula which takes into account buckling parameters.[11] The data on MOE (Young's modulus) from the ingrade testing program, does not appear to exhibit either length or depth effects. These "length effects" for tension, compression and bending are illustrated in **figure 6.1**.

**Introduction to Characteristic Strength and Length / Depth Effects:**

One of the major problems that Engineers have in designing with timber, is a tendency to regard timber as a uniform, homogeneous material, similar to steel. Whilst we are accustomed to the idea that strength varies between structural members made from the same material, the idea that strength varies <u>along the same structural member</u> is foreign to us. The assumption that variability and strength is <u>between</u> members rather than <u>within</u> members, makes it easy for us to establish design criteria in designing for a limiting state, which the member will undergo in service e.g. for a maximum bending moment.

However, the research work undertaken by Madsen[12] over the past 10 years and the data collected, has indicated that the assumption of uniform strength within a member leads to large discrepancies when applied to material such as timber, where the natural growth characteristics usually create localised weak spots along the length of the member. For example in a bending member where the strength varies along the length of the member failure is not determined alone by the stress caused by the maximum bending moment. Madsen's work has shown that failure will occur when the local moment causes a stress in excess of the strength of the local weakness, in other words, the failure is directly related to the probability of a weak spot being subjected to too high a stress anywhere along the moment curve, not just where the maximum moment occurs, as predicted by simple statics. This is illustrated diagrammatically, in **figure 6.2**:

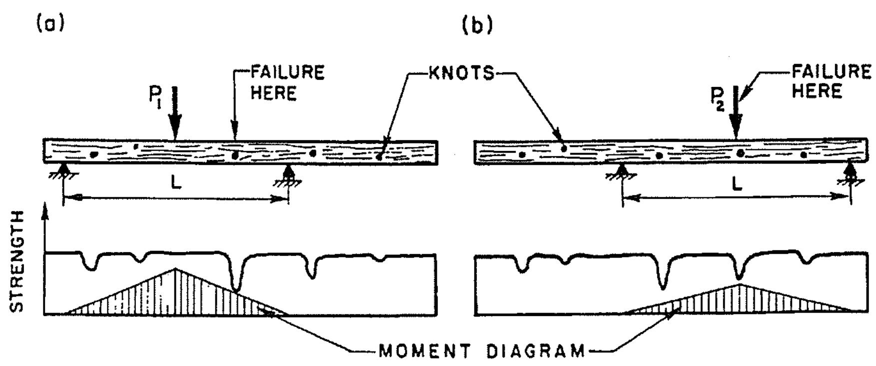


**figure 6.2 - effect of length increases the probability of failure (due to excessive stress) at or near a defect[13]**

It is these observations of material behaviour that have led Madsen to developing a more rational approach to design methods for timber. Madsen's research has shown conclusively that these effects are not trivial; in fact, in many cases these size effects are greater than the adjustment factors applied for other causes such as moisture content and duration of load. Unfortunately the weak spots occur randomly, so the longer a member is, the greater is the probability that a weak spot will be located in a highly stressed zone. As well as this, the load configuration that will be applied to the member will influence its apparent bending strength.

For example, a load configuration which consists of a concentrated load located at the centre or midspan, creates maximum moment over a relatively short length, whilst the same total load, consisting of two concentrated loads at the third points of the span, creates a high bending moment over the middle third of the span. The probability of having a weak spot subjected to a high stress is much greater for the latter load configuration, since the bending moment is uniformly high over a much greater proportion of the beam (see **figure 6.3**). This observation is an example of the *length effect* and it highlights the fact that the strength of a structural member is very dependant upon both its length and the loading condition to which it is subjected.

Therefore, it becomes necessary to define the characteristic strength of the material in terms of standardised arrangements for specific span and loading conditions, which are then adjusted (i.e. the characteristic values determined for this standardised condition) to suit the actual design conditions of span and loading configuration. Madsen's premise is that simply stating a characteristic strength is not sufficiently informative, since the research indicates that the characteristic strength is a function of both load and geometry of the member.

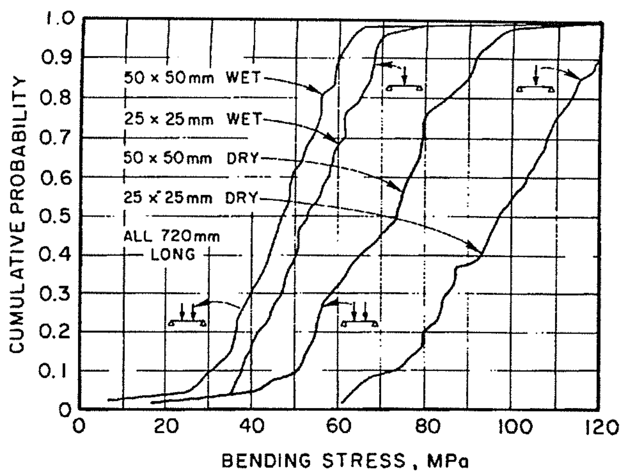


**figure 6.3 - cumulative distribution of raw data: tests on small section defect free Douglas Fir[14]**

This is extremely important for limit state design, since reliability based design procedures only make sense (and are only valid) if the design models used, properly reflect "true" or "realistic" material behaviour.

Work by Madsen and Foschi has shown that reliability based design procedures which reflect this material behaviour are not necessarily more complicated than ones presently used and do in fact result in more consistent reliabilities than previous design methods could attain.[15]

Research work recently published[16 17] has quantified the size volume effects for various types of commercially available softwood timber. Research into defect free Douglas Fir concluded:

1) that size effects were observed and quantified in members subject to bending
2) wet and dry materials behaved differently with respect to size effects, with the size effects being smaller for wetter material
3) a very small length effect was found in dry material, whilst a length effect could not be found for wet material.
4) All the data indicated that a *volume effect* is a more appropriate representation of the size effect than the product of length and depth, as has been claimed in previous research by others.[18]

It is recognised that in most design situations, timber engineers would not be working with defect free material, such as was tested in the Douglas Fir research program. However, the importance of this testing highlights the fact that size effects are applicable to all timber and whilst the length effects may by minimised due to the use of theoretically defect free material, the size and volume effects must nevertheless be considered as a part of the design procedure.

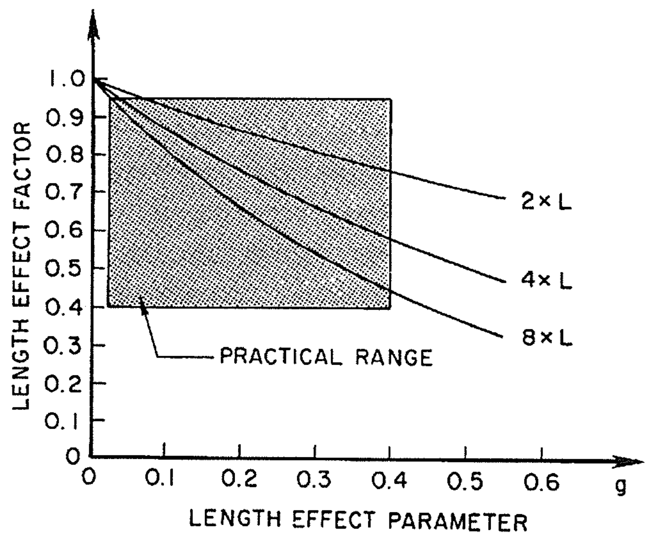


**figure 6.4 - Graph for determining the length effect factor, with the length effect parameter being derived from the slope of the strength distribution at the 5th percentile (Madsen)[19]**

Previous work by Madsen and Buchanan[20] has quantified the length effect by use of a weakest length theory, which provides a first approximation to solving the problem of defect effects, both at a macroscopic and microscopic level within the timber element. Further research work[21] into length effects of commercially available timbers (which would normally have included defects), recommends that size effects for design purposes should be expressed in terms of the size effect parameter, which is a function of the relationship between strengths and size. It is also recommended that strength for design purposes should be expressed relative to a specified (standard) length and it has been suggested that a length of 3m would be most suitable for commercial timber. the range and variability of these effects is illustrated in **figure 6.4.**

**Reliability Based Design Procedures:**

The reliability of a structural system is the probability that it will perform as intended in a given situation. The design of the system is normally controlled by several intervening variables, some representing the mechanical & geometry properties of the structure and others characterising the load demands.[22] Normally these variables are random and can only be described in statistical terms, thus the general objective of reliability based design is to account for the uncertainty of the intervening variables using principles of probability theory and then, to dimension the structure allowing for a tolerable risk of non-performance (which can be defined as the probability of failure).

This may be expressed algebraically as:

$$G = C - D \qquad \textbf{equation: 6.1}^{23}$$

where: $G$ is the performance function
$C$ is the capacity to withstand demand and
$D$ is the demand on the system.

Thus non performance or failure of the system will be indicated by all of those combinations of the variables resulting in: $G < 0$ i.e. where demand exceeds capacity. Conversely all those combinations resulting in $G > 0$ correspond to the performance as intended (survival). The situation where $G = 0$ is then defined as a *limit state* between failure and all those variable combinations that satisfy the equation $G = 0$ are said to belong to this limit state.

In a structural reliability context, limit states are commonly classified as either ultimate (strength and/or stability) limit states or serviceability limit states. An ultimate (strength) limit state corresponds to the limit of load carrying capacity of all or part of the structure. Some common examples are exceeding the material strength of the member, fracture, fatigue, instability and overturning. Serviceability limit states correspond to disruption in the functional use of the structure; typical examples being excessive cracking, deflections and vibrations.[24]

The ingrade testing and subsequent research work undertaken by Madsen and Foschi at UBC have sought to characterise these variables and to define the intervening variables that effect the capacity of the structural system, which is normally constructed from some form of timber beam or column element with connections. New sections of the 1989 Canadian Code for Timber Engineering have now been developed using principles of reliability based design. The methodology undergirding the code clauses, including the statistical treatment of data and calibrated procedures, is presented in Foschi, Folz and Yao.[25]

**Why Limit States?**

The rationale behind the decision to move from working stress design to a limit state format in Engineering Codes, is the fact that reliability based design is a much better approach to engineering analysis using probability theories of non-performance or failure to take account of the inherent variability and uncertainties of materials properties and loads. Research by Foschi[26] has shown that the application of working stress design procedures does not lead to uniform safety, neither does it provide the means for taking into consideration that the actual statistical characteristics of the material being used are particularly relevant for timber engineered structures, given the inherent variability of timber.* Reliability based design, on the other hand, provides a framework for a more rational procedure and more definable consistency from a safety point of view.

From a designers point of view, many engineers find that the statistical bases of reliability based design procedures are somewhat overwhelming. However, Foschi has shown that although familiarity with probability theory and statistical variations is desirable for engineers, it is not a prerequisite for being able to apply reliability based procedures.[27]

The aim of the code writer must be to present the statistical and probability information in such a way that it is useable by designers, who have little or no understanding of the theory behind them. The normal approach utilised for this is to use design equations incorporating suitable load factors, which multiply the design values for dead and live load effects and also incorporate a performance factor for the characteristic strength of the material.

**Practical Implications of Reliability-Based design for Timber Structures:**

This format of limit state design has been generally documented throughout the world. It is interesting to note that whilst reliability based design is the correct way to improve the engineering application of the timber products, (particularly in the larger structures where these products are competing with steel or concrete) reliability-based design in Canada has had very little change upon the way that ordinary residential dwellings are constructed.

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**(*) <u>Note</u>:**
It should also be noted that many "limits states" codes in Europe and North America have simply been changed by "soft conversion" from basic working stress design to limit states design. Such codes generally do not incorporate the statistical / probabilistic modelling advocated by Foschi's Reliability Based Design procedures, and inherently contain the same limitations in regard to uniformity of safety as the basic working stress codes.

Some changes have occurred for longer spanning floor joists or roof members, but essentially the impact of reliability-based procedures on the design of domestic construction is minimal. It will however, highlight problems such as vibrational responses of floors where the work undertaken by Smith & Chui, University of New Brunswick, (refer to Section 5 of this report) complemented by the work undertaken by Foschi at UBC, can be applied to ensure greater comfort levels within residential buildings.

Foschi notes that one of the main advantages of reliability design (which is not particularly addressed by codes) is its application to structural optimisation and the ability to evaluate and implement quality control strategies, since a reliability based design method offers the framework by which to realistically compare one design against another. Foschi believes, (and the author is in agreement with this) that reliability-based design should not be seen just as a new procedure to determine structural dimensions, but as an industry tool to advance the development of more reliable wood products for engineered uses.[28]

Significantly, it should be noted that it takes time for the engineering profession (as well as other design professions) to accept change to procedures which have a long standing and essentially a good track record. As a general rule, Structural Engineers tend to be naturally conservative and are traditionally trained to think in a deterministic manner, which unfortunately is an idealised and somewhat unrealistic representation of the world around us. Reliability design, which is based upon extensive testing for determination of characteristic material values, (such as has been undertaken by Madsen at UBC) provides a more rational design method, particularly for design of timber structures. This is because the Reliability-based design procedures attempt to deal with timber as a non-idealised non-homogeneous material - which in fact it is! The end result should be better design and more efficient utilisation of the timber resource.

It is the author's belief that reliability based design procedures, coupled with ingrade testing and a thorough understanding of the intrinsic characteristic strengths and material properties of timber, will provide engineers conversant with these approaches with the tools needed to design efficient, cost effective structures, which utilise timber (and thus promote it) to the best possible advantage.

**Computer-Aided Design Procedures:**

An important factor in gaining the acceptance of reliability based design procedures (which are often more computationally intensive than traditional, simplified design procedures) is the wide spread use and acceptance of personal computers throughout the design professions, not only in Australia but throughout the world. The development and use of structural analysis and design software has become very widespread. In all the institutions which the author visited as part of his Gottstein Fellowship, the use of computers for research and development of design procedures was seen as being absolutely essential, not only as a research tool, but also for having the ability to provide engineers with extremely powerful design procedures, which potentially can implement "state of the art" technology in the design office.

Whilst this is highly desirable, a note of caution needs to be mentioned with the use of computer software. Computer software is not a substitute for the use and application of sound engineering judgement, rather such programs are a tool which when carefully used by an experienced Engineer, can allow the Engineer to perform calculations and interpretations of data, which would otherwise be almost impossible. However, it must be stressed that no computer program is a substitute for sound engineering judgement.

Nevertheless, the importance of the computer as a tool in the design office cannot be overstated. This has been recognised by the steel industry in Australia with the recent introduction of the new limit state Steel Structures Code AS4100. Integral with the development and introduction of this code has been the release of software, which was developed concurrently with, and actually used to test some of the code clauses published in AS4100.* This software was then released with the code through the two largest Engineering software companies in Australia. As a result of this decision to release software that utilises the higher tiers of the limit state code, designers have a major incentive to computerise their design procedures in order to be able to produce the most cost effective and technologically sound designs.

## Computer Software & Design Aids:

An interesting application of computer rated design techniques has been developed at the Forest Research Institute in New Zealand by Structural Research Engineer, Hank Bier. Bier's work[29] has resulted in the publishing of a suite of spreadsheet programs, which allow designers to be able to design timber elements in accordance with the New Zealand design codes. These programs will run on most of the major commercially available spreadsheet programs without modification.

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**(*) Note:**
It should be noted that the complexity of AS4100 has been a source of contention for some practicing engineers in certain sectors of the industry, who have expressed considerable reluctance to use the code. This is unfortunate, as the code writers themselves sought to avoid this problem by using a tiered approach to the code, with the higher tiers (which allow more sophisticated design at the price of being much more computationally intensive) being "computerised" to encourage their use. Perhaps a similar tiered approach would be of benefit in future revisions and developments of AS1720

The use of a spreadsheet allows the designer to see (and hopefully understand) the rationale underlying the program in a way that is not possible with compiled programs, as well as permitting customisation of the programs to suit specific requirements. Whilst use of the programs is fairly simple the fact that they utilise a spreadsheet format tends to make to programs user friendly. Some of the algorithms contained within these programs are quite sophisticated and allow designers to do fairly complex calculations simply and effectively.

It is the author's belief that enormous potential exists with these types of programs and when based on limit state / reliability design theory, they will enable designers to effectively handle the numerically intensive calculations required to accurately model or analyse a particular structure in timber.

**Publications:**

The importance of appropriate "user-friendly" design aids for both Architects and Engineers is recognised in all of the centres visited by the author as a part of this Gottstein Fellowship. However, the development and availability of these varies widely. The most comprehensive design and detailing resources were generally found in Europe. Promotional organisations such as Lignum and Cedotec (Switzerland) and Arbeitsgemeinschaft [EGH] (Germany) have produced excellent publications presenting case studies, design examples and detailing. Similarly, documented research programs from ETH Zürich (Prof Gehri) and EPFL Lausanne (Prof Natterer), provide significant design information.

Unfortunately, apart from a few notable exceptions (eg: the "Timber Design and Construction Sourcebook"[30]) most of these design aids have not been translated into English. Some excellent promotional material has been developed for Glulam in the United Kingdom, and the "Technical Handbook" produced by Moelven,[31] provides Structural Engineers with invaluable design information and worked examples for design of glulam structures. The need for development of further design aids and documentation in English has been identified as a high priority by Prof Hilson at Brighton Polytechnic in the UK, who is currently preparing a major book for educating designers in the timber structure - particularly glulam.

Apart from a number of text books, there appeared to be a paucity (by comparison with Europe) of design and detailing aids in North America. The best information on specifics of timber design generally appear to be published by some of the timber promotional organisations, by the Forest Products Laboratory - Madison (eg: the Timber Bridges Manual[32]) and by Universities such as UBC and UNB in Canada and the associated Forest Research organisations. The publications, whilst technically excellent, seem somewhat fragmented as far being readily available and useable in a design office environment. Details of a number of "design-related" publications and technical literature have been presented by Juniper, in his Gottstein Report.[33]

**Commercial Software:**

As mentioned above, computers are widely used in all the research centres visited by the author. However, apart from the spreadsheet programs developed by Hank Bier at FRI, all of the programs observed during the study tour were intended for research and development, verification of laboratory modelling and / or code based procedures, rather than for use in a design office.

Apart from the use of commercial finite element programs and some CAD detailing software seen at EPFL, most of the software has been developed "in-house" for very specific applications. Some of these programs are capable of very sophisticated analyses, particularly software developed at UBC by Prof Foschi, and provide researchers with excellent predictive tools for modelling the inherent material / structural complexities of timber. Perhaps the fact that accurate modelling and design of timber is more complex than for other materials (eg: steel), acts as a restriction to the development of commercial software for Architects and Consulting / Design Engineers.

In discussions during the study tour concerning computerised design aids, the author demonstrated some commercial software which has been developed by Integrated Technical Software in Melbourne, Australia and the response to this was excellent. Whilst this software is a 3D frame analysis package without a timber design post-processor for design, the responses indicate that this software is as good, if not better than anything commercially available overseas.

Some other Australian software has been developed by engineers involved in timber design, but these programs have limited application in their present form. Dr Alex Heaney from the University of NSW has developed some excellent spreadsheet programs for teaching purposes and similar programs have been developed by the author for both teaching and design purposes. Given overseas experience, there is some incentive to develop these basic tools into integrated design modules for commercial applications.

**Conclusions:**

There exists an enormous amount of overseas research into characterisation of material properties and behaviour of timber structures, and the overview presented here is far from exhaustive. However, it is the author's belief that many of the principles of this research can be successfully applied to design procedures in Australia. Certainly the development of limit state design procedures, if linked with characterisation of material properties (eg: in-grade testing) and consideration of issues such as length and size (or volume effects), will provide engineers with a much better "feel" for designing with structural timber.*

Coupled with this, must be the development of computerised design aids and resource material with "worked design" examples and details of timber engineered structures, particularly of connections. Some of the European material could be fairly readily adapted and if linked with spreadsheet software, similar to that produced by FRI, could reduce some of the "number crunching" required to successfully design timber structures.

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**(*) Note:**
It should be noted that the work of converting AS1720.1 to LSF is currently (July 1992) being undertaken by Dr Bob Leicester at CSIRO and Mr John Carson at NAFI.

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# SECTION 7:

# CONCLUSIONS, RECOMMENDATIONS & ACKNOWLEDGMENTS

## Conclusions:

The Gottstein Fellowship has given the author the opportunity to visit some of the leading timber research institutions in the world, and from this experience, to present an overview of the current "state of the art" in a number of areas of research relevant to the design of timber structures for the non-residential market. Obviously, not every research establishment in the world has been visited, but rather a carefully targeted representation of those whose work is directly applicable and transferable to Australia. The aim of this Fellowship has been to facilitate *technology transfer* of overseas experience into an Australian context, through presentation of research findings, current developments and directions in the centres which the author was privileged to visit.

## Summary and Recommendations:

A summary of the material presented in this Gottstein Report is given below, with comments and recommendations for further research, development and application in Australia:

*Section 2 - Connections & Joints:*

An overview of current research into bolted, dowelled and nailed mechanical connections has been presented. The common theme for much of this research focuses on the recognised need to quantify design procedures that will:

- (i) produce 'ductile' behaviour in connections,
- (ii) avoid high stress concentrations and
- (iii) minimise perpendicular to grain tension stresses (and subsequent brittle failure mechanisms).

The relevance and development of analytical techniques for modelling the behaviour of connections has been discussed, and it is recommended that these techniques should be developed further and validated for application with Australia timber species by laboratory testing.

Overseas and recent Australian experience with glued and epoxy injected dowelled and threaded rod connections is also presented. It is the author's belief that these type of connections have enormous potential for use in Australia, although caution must be exercised until further testing concerning both the load behaviour of these connections and the long term performance of the epoxies is better understood. As such, it is recommended that collaborative research projects with overseas Universities should be established to facilitate the acceptance of this technology in Australia.

*Section 3 - Stressed Laminated Timber Bridge Decks:*

Whilst this technology was previously mentioned by Peter Juniper in his Gottstein Report (1986), there had been no serious application of it in an Australian context until very recently. Since the beginning of 1990, research into applying this technology to Australian hardwood and softwood timbers has assumed enormous significance, particularly in New South Wales. During these last 2 years, a major Research and Development program was been established by the Roads and Traffic Authority of NSW, in conjunction with the University of Technology, Sydney, the Forestry Commission of NSW and relevant organisations representing the timber industry.

The Gottstein Fellowship has made an invaluable and immediate contribution to this project, through the involvement of the author as Principal Consultant. It is envisaged (and recommended) that this R & D program will continue over the next couple of years, involving:

- (i) development of more sophisticated applications of stress lamination techniques for bridge decks and structural members
- (ii) documentation of research
- (iii) development of evaluation, design, construction and maintenance procedures and
- (iv) education of designers, decision makers and implementors of this technology.

It is not only recommended that funding for this research continue to guarantee its successful implementation, but that industry support (by way of awareness, funding and quality assurance programs) for this work increase as more bridges are constructed using this technology. It is also imperative that funding be made available to provide on-going monitoring and evaluation of these bridges, to ensure that mistakes do not occur and that design and maintenance procedures are refined as necessary. To this end it is recommended that a data base / inventory of all bridges built or rehabilitated using this technology be established as a part of a quality assurance program for timber bridges. The principles defined in the Timber Bridge Initiative in the United States are directly applicable to the Australian market and should be used as a model for establishing the technology in Australia.

It is the author's belief that if this technology is implemented in a careful, strategic manner with a thorough, scientific research basis and an emphasis on quality assurance from the timber industry, then this technology in itself will have a major influence in establishing (or perhaps more accurately re-establishing) structural timber as a viable and reliable material in the eyes of Structural Engineers and Asset Managers in both the private sector and all levels of government where asset management is a consideration.

*Section 4 - Glulam:*

A review of 3 major research initiatives into the design and use of glulam has been presented. The work of Russell Moody at the Forest Products Laboratory into volume effects has direct relevance for glulam design everywhere and it is recommended that this work be considered for incorporation into AS1720.1. Similarly, recent research by Prof Borg Madsen at the University of British Columbia (presented in *Section 6*) considering length effects should also be assessed and adapted for inclusion into AS1720.1

The evaluation and assessment of the Glulam industry in the United Kingdom provides valuable insights into some of the issues facing the industry in Australia and as such it is recommended that this study be examined to identify guidelines for the local industry which will increase productivity, quality assurance and competitiveness with non-timber structural alternatives.

It is recommended that the extensive research into load-duration and mechano-sorptive effects for glulam, undertaken at Brighton Polytechnic by Prof Barry Hilson, be monitored closely and complementary work (perhaps as pilot or correlation studies) be undertaken to translate this work to Australian timber species. The implications of this work on detailing for durability and creep should not be overlooked, particularly the apparent effective increase in long term stiffness for beams under flexure, which are sealed to slow down the rate of change of moisture content.

*Section 5 - Building Applications:*

An overview of three distinct but inter-related research areas is presented, detailing current research on:

- (i) raking resistance (due to seismic load events) of Multi-Storey Light Timber Framed buildings in New Zealand
- (ii) Floor vibration and damping effects
- (iii) recent research developments in fire resistant construction in both the USA and New Zealand

The recent New Zealand research on MSTF demonstrates the technical viability of this form of construction, both in terms of structural efficiency and cost effectiveness. A number of projects recently completed at the University of Canterbury should be assessed (as a high priority) by those representing industry groups who are attempting to implement MSTF construction for medium density residential buildings in Australia. A huge amount of time and duplicate research could be avoided if complementary, collaborative research agreements were established across the Tasman.

Work by Dr Ian Smith at the University of New Brunswick into the dynamic response and occupant 'comfort levels' for timber floor systems, has direct applicability to Australia. This work notes the inadequacy of simply increasing the stiffness of floor joists to change vibrational behaviour, and highlights the importance of damping effects and the need to create isotropic rather than orthotropic behaviour in floor systems, as key issues for utilising timber far more efficiently than is presently done in timber floor systems.

The Forest Products Laboratory has undertaken very significant R & D work into fire performance and the development of analytical or predictive models for fire resistant construction. The complexity and expense of this work is noted, and the US experience should be adapted to Australia and research validated for inclusion in Australian Building Codes. Similarly, the work into fire performance of glulam members and nailed portal connections undertaken at the University of Canterbury and by the Building Research Association of New Zealand, appears to be readily transferable to Australia and this should be done as a matter of priority.

*Section 6 - Material Characterisation and Design Procedures:*

The author's two week study period at the University of British Columbia provided exposure and access to the enormous amount of research undertaken by Professors Borg Madsen and Ricardo Foschi into the characterisation of material properties, ingrade testing techniques, analytical modelling of timber and reliability based design procedures. The overview of this presented in this report highlights the practical importance of this work and it is recommended that further detailed study of this work be undertaken with the aim of translating the principles of the Canadian research into an Australian context both for establishing research priorities and in development of the limit states format (LSF) for AS1720.1.*

An overview of current design aids also reveals that whilst some excellent material is available, (particularly in Europe) the timber industry needs to make the development of such aids - particularly computerised design programs such as the spreadsheet programs developed by Hank Bier at the FRI in Rotorua, a high priority.

This is particularly important in the development of limit states codes, which are often more 'calculation intensive' than current working stress codes (this will probably not be an issue for a 'soft conversion' of the existing AS1720.1 to LSF, but it will be important for future editions). The development of computerised procedures which access code clauses is generally perceived as an important issue for gaining acceptance of the structural engineers who use the code, keeping in mind the proliferation of design aids and software available for both concrete and steel.

It is recommended that Timber Promotional and Technical Advisory groups be given funding to develop suitable design aids (in consultation with design consultants and recognised software houses) to meet this need, both as published documents and in software form.

(*) Note: This work of conversion to LSF is currently being undertaken by Dr Bob Leicester at CSIRO and Mr John Carson at NAFI)

**General Recommendations:**

In presenting both the above specific and the general recommendations following, the author acknowledges that his perspective on research needs for the timber industry in Australia as a whole, is limited. Nevertheless, the experienced gained from the Gottstein Fellowship has greatly influenced and broadened his understanding of these needs in his role as a designer, researcher and educator.

As a basic principle, it is recommended that 'technology transfer' mechanisms and collaborative research projects be established wherever possible for any relevant research areas, so that as 'the clever country' we can avoid unnecessary duplication of research (and consequent waste of scarce funds). The importance of drawing on and learning from the experience of overseas researchers, so that we do not 're-invent the wheel', cannot be overstated.

Whilst organisations such as IUFRO and various international conferences exist, a more coherently defined infrastructure for creating awareness of overseas research and for accessing details of such, should be established, perhaps in the form of a data base which is readily accessible to interested persons. NAFI could have a key role in establishing and maintaining this, in conjunction with CSIRO and relevant Australian universities eg: UTS - Sydney, QUT - Brisbane, Monash University (Caulfield) - Melbourne, UCQ - Rockhampton.

A mechanism or forum for identifying research needs and establishing priorities for programs should be established at a National level, involving industry representatives, technical staff from NAFI and timber promotional organisations and relevant government and university research establishments. Certainly within NSW, the imminent establishment of the Timber Education and Research Centre in 1992 has the potential to be extremely beneficial to the future of the timber industry, and this venture will potentially be in an excellent position not only to establish priorities, but also to implement technology transfer through research, education and specialist consulting programs.

This Centre is being established as a joint venture between the University of Technology, Sydney, the Forestry Commission of NSW and the Building Research Centre at the University of NSW, with strong representation from industry, designers and specifiers on its board. It is hoped that the establishment of this centre will overcome some of the fragmentation of research efforts which presently exists, through co-ordination of such efforts not only within the Centre, but through consultation with other organisations, such as NAFI and CSIRO. Networking is essential!

The potential for our timber resource as the only truly renewable and sustainable building material, is enormous - we cannot take this for granted! Investment in research, education and promotion, which draws on overseas experience through *technology transfer*, is a wise and strategic investment in the future of the timber industry in Australia.

## Acknowledgments:

This document marks the end of a 2 year commitment; beginning with the awarding of a Gottstein Fellowship, spending nearly 8 weeks overseas in North America, Europe and New Zealand, then collating the experience and undertaking the arduous process of documenting my findings.

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